MONONGALIA GENERAL HOSPITAL

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

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS REPORT

THESIS

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

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HIROKI OTA
STRUCTURAL

DR. A. M. MEMARI

monongalia general hospital

morgantown, west virginia

//building statistics

//location: 1200 J.D. Anderson Drive, Morgantown, WV
//function: Hospital
//size: 405,994 SF
//height: 59 feet
//construction dates: June 2006 – May 2009
//delivery method: Design-Build with GMP contract
//cost: $67,484,560

//project team

//owner: Monongalia General Hospital
//architect: Freeman White, Inc.
//civil engineer: Alpha Associates, Inc.
//construction manager: Turner Construction Company
//geotech/environmental consultant: Potesta Engineers
//m.e.p.: Freeman White, Inc.
//structural engineer: Atlantic Engineering Services





//architecture

//masonry and curtain wall façade with variable elevation
//houses multiple hospital functions such as the emergency department, imaging department, icu, and private patient units
//flat roof - combination of ballasted and adhered roof systems

//electrical/lighting system

//480/277V 3 phase, 4 wire system
//208/120V 3 phase, 4 wire system
//two 1500KW diesel engine generators
//electronic type ballasts, 95% power factor
//time switches provided for all exterior lighting

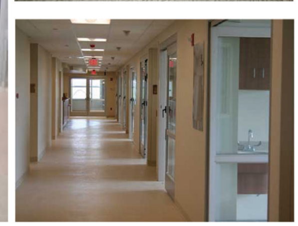
//structural system

//concrete structure with masonry veneer
//5” slab on grade
//8” two-way Slab system on all floors supported by concrete edge beams and columns
//shallow spread footing foundations

//mechanical system

//seven rooftop VAV-AHU with capacities ranging from 37000 to 11500 cfm
//water-cooled chiller, cooling tower, and steam boiler



//hiroki ota//structural option

http://www.engr.psu.edu/ae/thesis/portfolios/2009/hqo5001/

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Executive Summary

Purpose

According to Eve Hinman, “hardening structures against weapon effects has been, until recently, of concern almost exclusively of the military. However, with the increase of terrorist activities directed against civilian targets, there is a growing interest in applying these principles to the design of non-military structures.” The Monongalia General Hospital has decided to research and integrate higher levels of structural safety against blast and progressive collapse due to accidents or terrorist attacks to ensure that catastrophic events due to blast and collapse can be prevented to the greatest extent possible. This thesis will study different blast, collapse, and design scenarios and compare the results to choose the most effective approach to mitigating such events.

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 three portions of the building.

Blast and Progressive Collapse Analysis

Various design methods and approaches have been conducted to test the adequacy of the Monongalia General Hospital’s structural system against blast and progressive collapse scenarios. Through the analyses, two viable design methods were compared in terms of structural integrity, cost, and its effects to the schedule proposed by the construction manager. Through further investigation of the two design methods, the Monongalia General Hospital’s existing conditions with slight changes to the reinforcement in the concrete has proved to be adequate to resist a progressive collapse scenario induced by a blast event.

Alongside the structural analysis and design, other elements such as the curtain walls have also been analyzed against blast loads and the necessary design changes have been investigated. The investigation yielded two alternatives which are both more than adequate to resist the blast loading found by following the procedure on ASTM E1300-04. The alternatives were also analyzed in terms of conductive properties as well as the cost to implement the design on the Hospital.

Acknowledgements

I would like to thank the Pennsylvania State University Architectural Engineering Department, its faculty and staff; and the Turner Construction for providing me with a multitude of invaluable knowledge, experiences, professional advice, and support throughout the past four years and during the writing of this thesis:

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I would also like to send out a special thank you to my family, fellow Architectural Engineering classmates, and friends for their support and friendship.

Monongalia General Hospital

1200 J.D. Anderson Drive
Morgantown, WV

Blast and Progressive Collapse Analysis Report

Introduction

The Blast and Progressive Collapse Analysis Report (Thesis Report) analyzes the Monongalia General Hospital against blast and progressive collapse scenarios. This report primarily emphasizes in the progressive collapse analysis due to the complex nature of blast loading. This thesis report analyzed two locations seen to be a critical location in an event of a blast and redesigns the floor system to accommodate for such an occurrence. These redesigned floor systems will be compared in terms of cost and schedule effectiveness as well as its structural integrity. The façade of the Hospital will also be analyzed against blast loading and new designs will be compared in terms of heat transfer qualities as well as their cost.

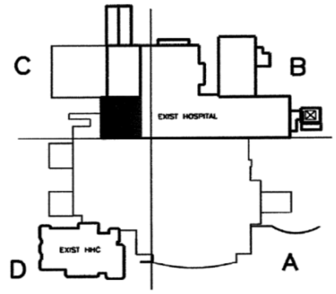
The Monongalia General Hospital

The Monongalia General Hospital is located on 1200 J.D. Anderson Drive, West Virginia. 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 $68,000,000 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)

Architectural Discussion

The Monongalia General Hospital's plan can be divided into four different quads, A, B, C, and D (Figure 1).

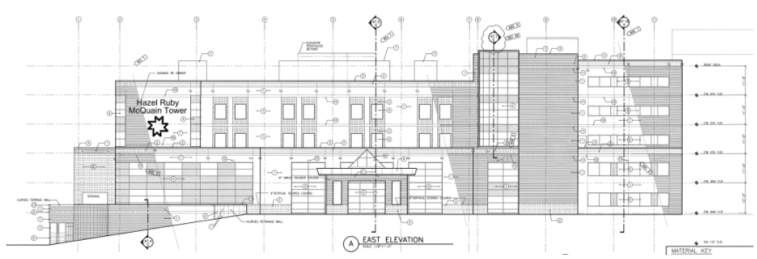
Figure 1: General Hospital Plan



Source: Drawing A1.0; Freeman White

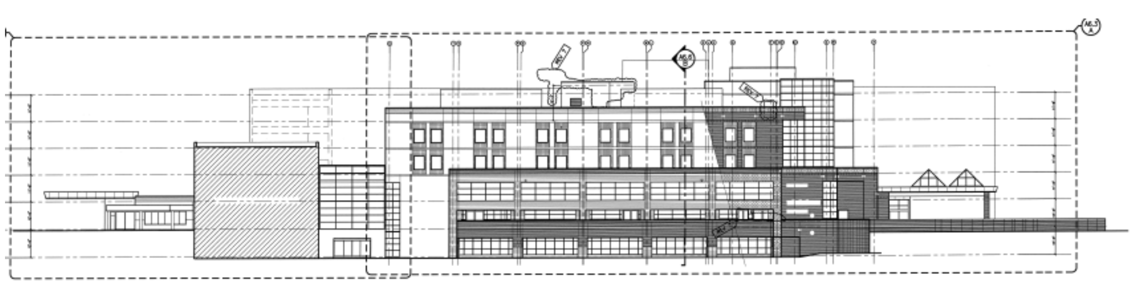
The first floor of the Monongalia General Hospital occupies 92,086 square feet and houses a boiler/chiller room, electrical rooms, doctors' offices, labs, nurse stations, storage spaces, and a dining space equipped with a food services kitchen. The second floor follows a similar layout but provides more 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. See Figure 2 and 3 for building cross section:

Figure 2: East Elevation



Source: Drawing A6.2; Freeman White

Figure 3: South Elevation



Source: Drawing A6.1; Freeman White

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 elevation (Photograph 1 and 2).

Photograph 1: Monongalia General Hospital from South-East, During Construction

Source: Turner Construction

Photograph 2: Monongalia General Hospital from South-East, Exteriors Completed

Source: Turner Construction

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 Systems Discussion

The Monongalia General Hospital's main structural system is a combination of concrete shear walls and moment frame with a total of one hundred 24 inch x 24 inch columns with varying heights supporting the building. Shear walls are located around openings on plan such as elevator shafts and stair ways to provide resistance against lateral loads. The floor system in the Hospital is a flat slab supported by columns and exterior beams.

The columns provide some resistance against lateral loads and rests atop concrete spread footings placed no more than ten feet below grade. 5 inch slab on grades are also provided as a foundation system on the ground floor. Keeping in mind that the Hospital has varying elevations, the columns all vary in height, providing support from the ground to the roof of the building. These columns are usually 27 feet apart and follow a square grid pattern. From the second floor up to the roof, the floor system is primarily an eight inch flat slab system supported by the aforementioned columns and by edge beams usually 24 inches x 18 inches in size. Interior beams are almost nonexistent in the structural system except by areas where there are openings for stairwells and elevator shafts and other select locations. Composite floor systems utilizing W12 shapes, although very minimal; can be found in Quads A and C providing support for heavier loads such as a power generator and as a canopy above the lobby entrance.

Shear walls are located in three major locations in the building (see Figure 4) and are responsible for resisting lateral loads. These walls, much like the columns vary in height depending on their location in the Hospital due to its varying elevations. The shear walls are located where there are openings on plan and used as elevator shafts and stairwells.

Each structural member type and system is discussed in further detail in the Structural System section of this report.

Lighting and Electrical Discussion

The hospital is powered by two systems: a 480/277V 3 phase, 4 wire system and a 208/120V 3 phase, 4 wire system. All of the mechanical systems are linked to the 480/277V as well as the medical equipment used throughout the Hospital. Lighting fixtures are served by the latter, and utilizes electronic type ballasts all with a 95% power factor. Time switches are provided for all exterior lighting in the Hospital grounds. Two 1500KW diesel engine generators housed in Quad C and will provide emergency power in times of need.

Mechanical Systems Discussion

Seven rooftop VAV-AHU’s with capacities ranging from 11,500 to 37,000 cubic feet per minute work as the mechanical system of the Hospital. Water-cooled chiller, cooling tower, and steam boiler are also located on the rooftop. Hot and cold water are provided to all toilets, operation and examination rooms, and the kitchen. Heating is provided in all rooms and hallways via an electrical duct heater. Due to its importance in the hospital, the mechanical systems are connected to the emergency power generator located in Quad C.

Construction Method Discussion

The Monongalia General Hospital was built under a design-build contract with a GMP set at $68,000,000 by the Turner Construction Company. The ground was broken with 70% of the documents and the sub-contracts are based on these documents in lieu of 100%.

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. See Appendix E for building design loads

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 inch thick normal weight concrete and the slabs used in floors above are 8 inch two-way flat plate slabs that utilizes 5000 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 feet 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 6). 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.

Problem Statement

For the interest and purpose of the thesis; attacks to hospitals can cause catastrophic events: lives can be lost and significant monetary damages are inevitable. To ensure that such events do not occur, the Monongalia General Hospital has decided to research and integrate higher levels of structural safety against blast and progressive collapse due to accidents or attacks. According to Eve Hinman, "hardening structures against weapon effects has been, until recently, of concern almost exclusively of the military. However, with the increase of terrorist activities directed against civilian targets, there is a growing interest in applying these principles to the design of non-military structures". The location of the Hospital is in close proximity to the West Virginia University and being the largest healthcare facility in the area; its existence can very well be a great target for terrorist attacks.

Proposed Solution

Structural Depth

In order for the structure to withhold against a progressive collapse scenario caused by a blast, the structural system must be capable of temporarily carrying loads over spans longer than they were initially designed for. However, blast characteristics must be studied and multiple blast scenarios must be simulated in order for structural analysis to begin. The following are some notable documents that were studied to gain knowledge of blast and collapse characteristics:

Carino, Nicholas J., and H. S. Lew. "Summary of NIST/GSA Workshop on Application of Seismic Rehabilitation Technologies to Mitigate Blast-Induced Progressive Collapse." Oakland, California. Feb. 2009. NIST. Feb. 2009 <http://www.fire.nist.gov/bfrlpubs/build01/PDF/b01055.pdf>.

Hinman, Eve. "Approach for Designing Civilian Structures Against Terrorist Attack." Concrete and Blast Effects. Ed. William Bounds. American Concrete Institute: Farmington Hills, MI, 1998. 1-17.

National Research Council. Protecting Buildings From Bomb Damage: Transfer of Blast Effects Mitigation Technologies from Military to Civilian Applications. Washington D.C.: National Academy Press, 1995.

Ngo, T., P. Mendis, A. Gupta, and J. Ramsay. "Blast Loading and Blast Effects on Structures." Electronic Journal of Structural Engineering International (2007): 76-91.

Smilowitz, Robert. "Means for Risk Reduction and Analytical Approaches." NIST-SEI Joint Workshop. Building and Fire Research Laboratory. Feb. 2007. NIST. Feb. 2009 <www.bfrl.nist.gov/861/861pubs/collapse/workshop/3.Smilowitz_2MU.pdf>.

United States of America. Department of Defense. UFC 4-023-03 Design of Buildings to Resist Progressive Collapse. By Whole Building Design Guide.

The design to resist progressive collapse will be based primarily for situations where a column is removed and the floor system must withstand a larger span. Two scenarios will be iterated, one in the entrance lobby area to simulate a collapse in the interior of the building, and another in the exterior if a corner column was to be removed.

Schedule and Cost Breadth

In the event of a redesign as such issues will inevitably arise, the cost and the schedule must be paid close attention to. Stronger structural members could require a change of material or an increase the member size which will directly affect the schedule and the cost of the project. As of September 2008, the project volume was estimated to be over $67,000,000 and still rising. In order to reflect the existing monetary issue, yet still making the hospital resistant to blast and progressive collapse, much attention needs to be paid, and a possible alternative must be planned out. Also, possible schedule changes must be accounted for in the event of a redesign, factors such as use of different materials and/or construction methods can be major influences.

Architectural Breadth

The façade of the Hospital uses brick veneer and curtain walls around the perimeter, in the event of a blast, the brick veneer and curtain walls could be of great danger and cause unnecessary injuries and deaths. In order to prevent such events from occurring, the brick veneer and curtain walls will be analyzed for its effectiveness against such loading conditions. More emphasis will be placed on the curtain wall system of the Hospital due to its location around the lobby area and the catastrophic results it could yield in an event of a blast. In the event of a redesign, the different alternatives will be compared in terms of heat transfer properties and the cost.

Blast Analysis

Introduction

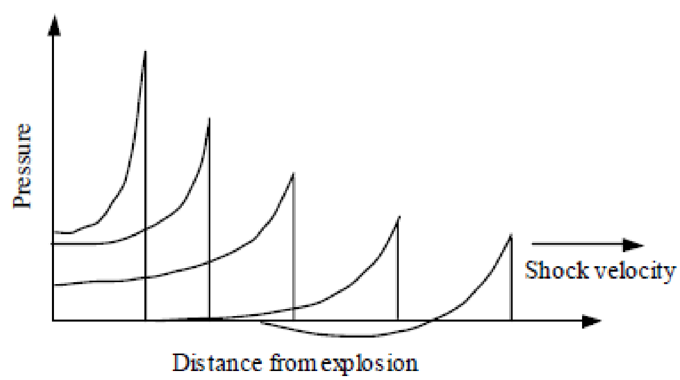
To being the study for this thesis, first and foremost, blast theory had to be addressed to familiarize oneself with the characteristics and the nature of the blast as well as how the structure responds to such event. The analysis and design of blast loading on buildings is based on a complex interaction addressing the likelihood of the blast in addition to how the structure will behave during the event. “These activities are often referred to as probabilistic analyses and parametric studies.” (Happold).

“Blast loading can be characterized in a simple equation relating the charge weight and the standoff distance.” (Ngo, et al.) Blast loads are directly proportional to the stress wave propagation resulting in a dynamic loading situation on the structure. Blast loads on buildings can be categorized into two major classes, open air blasts which occur outside the building and confined explosions which happen inside the building. The loading can be written in a simple equation as follows:

$$P = \frac{W}{R^3}$$

In which P is the incident pressure on the building, R is the standoff distance or the radial distance between the origin of the blast and its target, and W is the weight of the explosive, in terms of equivalent TNT mass. Figure 4 illustrates the relationship between the incident pressure P and the standoff distance R:

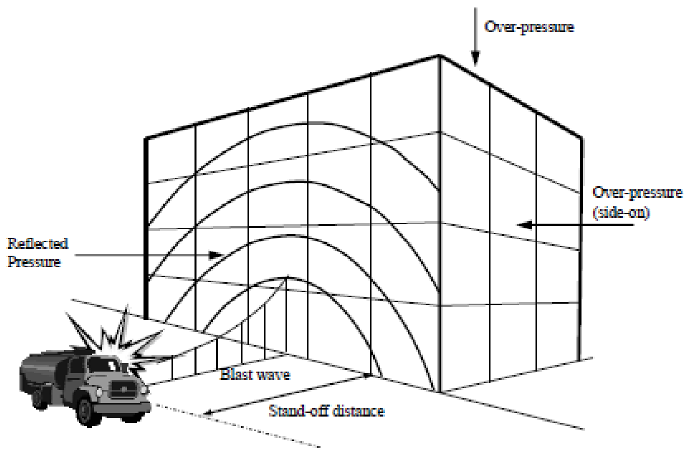
Figure 4: Blast Wave Propagation



Source: Ngo, Mendis, Gupta and Ramsay

As mentioned earlier, blasts can be classified, in terms of the location of the blast; into two major classes, open-air and confined. Open-air blasts are more defined and its characteristics are easier to analyze. Charges situated extremely close to a target structure impose a highly impulsive, high intensity pressure load over a localized region of the structure; charges situated further away produce a lower-intensity, longer-duration uniform pressure distribution over the entire structure. Figure 5 illustrates the types of pressures caused by blast loading:

Figure 5: Open-Air Blast Loading



Source: Ngo, Mendis, Gupta and Ramsay

Eventually, the entire structure is engulfed in the shock wave, with reflection and diffraction effects creating focusing and shadow zones in a complex pattern around the structure.

Analyzing confined blast is a much more complicated process than that of an open-air blast. Internal blasts produce complex loading profiles as a result of blast overpressure and then a re-reflection of the pressure due to the confinement created by the structure. "Depending on the degree of confinement of the structure, the confined effects of the resulting pressures may cause different degrees of damage to the structure." (Ngo, et. al) The complexity of this type of explosion requires the use of semi-empirical Computational Fluid Dynamics (CFD) and Solid Mechanics (CSM) modeling programs to better predict the blast loading effect on the structure (National Research Council).

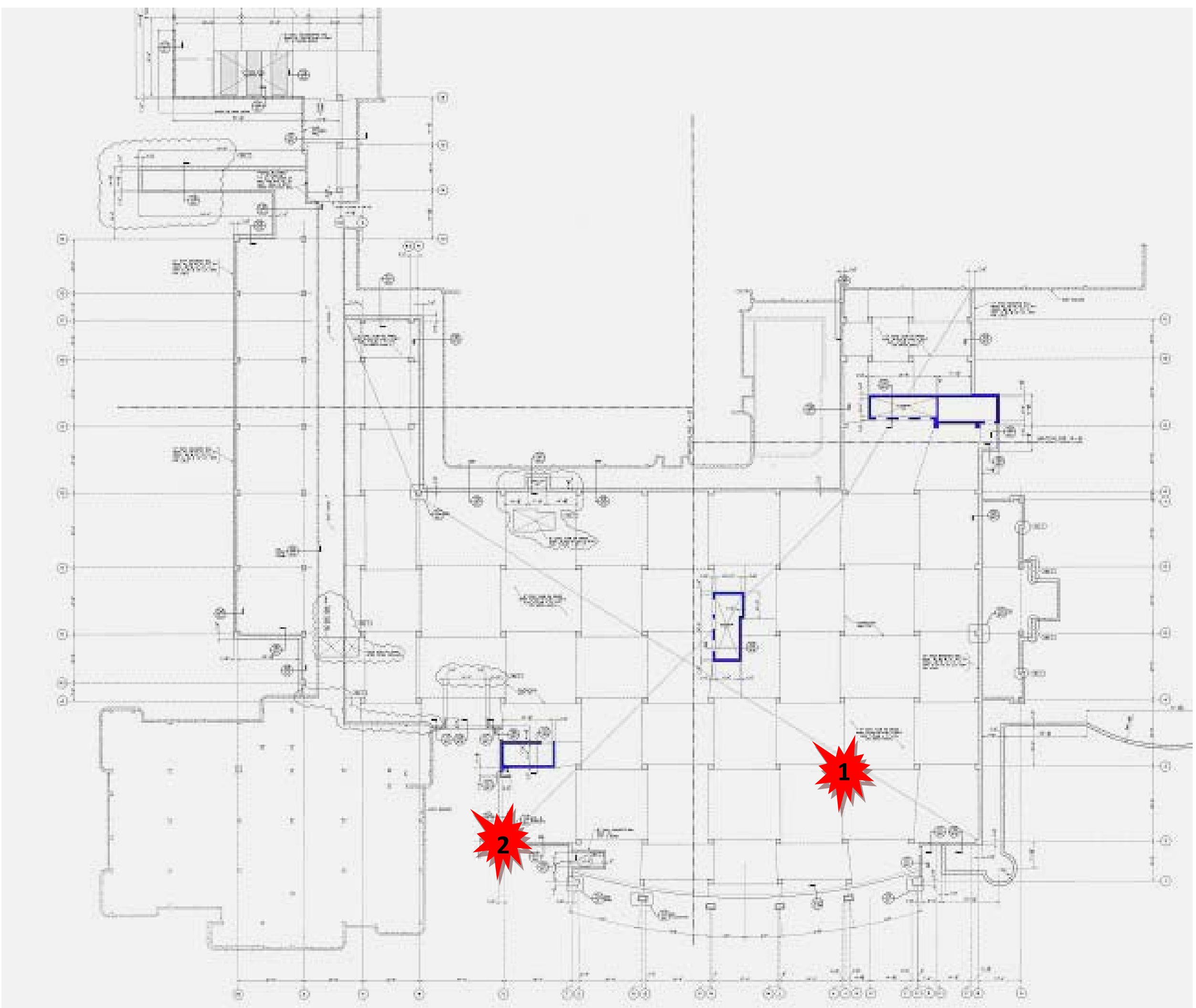
A solid understanding of blast and its effects are a crucial part of blast resistant design of a structure. For the interest of this thesis, both types of scenarios will be considered however, since blast loading is a complex load to analyze (in terms of predicting the location as well as the amount of charge used), it will be assumed that the blast has damaged or eliminated a part of the structure and a progressive collapse analysis will be conducted at those locations.

Scenarios

For the interest of time, an extensive study of different threat scenarios cannot be conducted however; two locations deemed to be critical will be analyzed. The first location of interest (marked 1 in Figure 6) will be located by the east lobby area. The lobby is a public area and placing of charges is a relatively easy task, also, there are five floors resting above the lobby which could be disastrous if a column was to be eliminated. In such an event, the slab on the second floor must be capable of resisting a 54 foot span which can cause a collapse mechanism to occur. For this analysis, a column will be removed from the first floor and the slab will be analyzed for its load carrying capacities and redesigned if inadequate. Due to the unpredictable nature of the location of a blast, the location will be assumed and in the event of a redesign, the design will be used for all remaining interior floors.

The second location of interest (marked 2 in Figure 6) is the corner column on the southwestern portion of the building. This too, will be an easy target since the perimeter of the Hospital is surrounded by parking lots and the potential for a car bomb, or even a truck colliding into the column is feasible. In this scenario, once again will assume the blast has already occurred and the corner column will be removed and the edge beam and slab will be analyzed for its capacity to carry the loads in a 30 foot cantilever situation. Figure 6 shows the locations of the area of interest:

Figure 6: Locations of Study



Source: Drawing S2.0; Freeman White

Progressive Collapse Analysis

Introduction

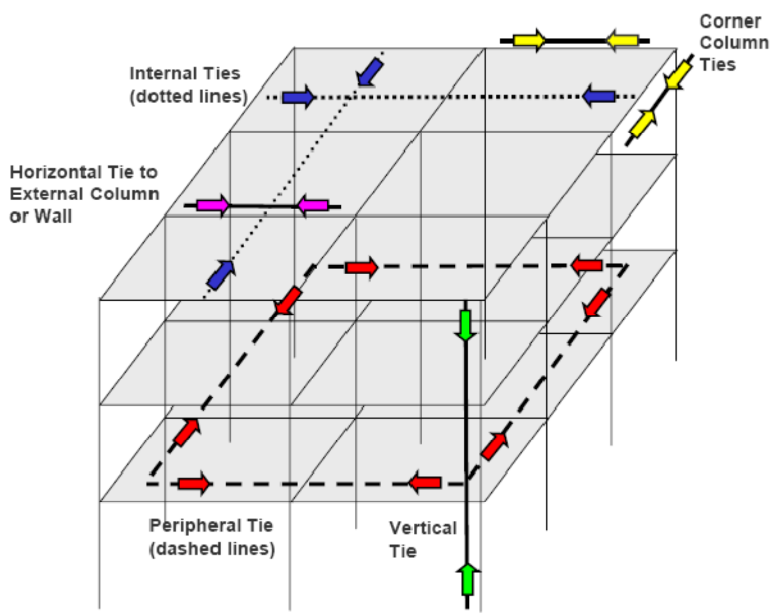
As stated in the Blast Analysis section, the blast is assumed to have occurred, causing a localized failure in the structural system (locations shown in Figure 6). Much like blast analysis, progressive collapse characteristics needed to be studied. Progressive collapse, as defined in ASCE Standard 7-05 is "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it."

Progressive collapse is caused by a combination of two things: "an abnormal loading which triggers a localized damage to a member or members, and a structure that lacks adequate continuity, ductility, and redundancy to prevent the localized damage from spreading." (UFC 4-023-03) Two design approaches are used in this thesis to analyze and design the structural members, as recommended by the ASCE Standard 7-05 and the UFC 4-032-03, the Direct Design Method and the Indirect Design Method. "Breen, Ellingwood and Leyendecker have made a distinction between direct and indirect design. Indirect design incorporates implicit consideration of resistance to progressive collapse through the provisions of minimum levels of strength, continuity, and ductility. Direct design incorporates explicit consideration of resistance to progressive collapse through two methods." (Mehrdad and Serkan)

The Direct Design Method can be broken down into two different approaches, the Alternate Path Method and the Specific Local Resistance Method. The Alternate Path Method requires the structure to be capable of bridging over a missing structural element, and the Specific Local Resistance Method which requires the structure or parts of the structure to provide sufficient strength against a threat. Keeping in mind the assumption made for this thesis, the Alternate Path Method will be utilized, since the assumption allows for local failure to occur. One must also note that using the Specific Local Resistance Method is "a highly detailed, analytical design approach" (Smilowitz) and also very conservative—resulting in larger members. Due to its level of detail and for the interest of time, the Alternate Path Method was chosen as the analysis method for this thesis.

With the Indirect Design Method, resistance to progressive collapse is considered implicitly "through the provision of minimum levels of strength, continuity and ductility". (ASCE Standard 7-05) This method will be used to determine the tie forces joining the different structural members and if the reinforcements are adequate to resist a collapse situation. Figure 7 illustrates the different areas that are required to be checked:

Figure 7: Tie Forces



Source: UFC 4-023-03

This method can be combined with the use of the Alternate Path Method described earlier; it satisfies the assumption made for this thesis—considering the structural members' response independent from the blast loads which caused the removal of the members.

Scenarios

The scenarios for progressive collapse will follow along with those of blast, as shown in Figure 6.

Approach

In order to utilize the design methods described earlier, a load combination must be defined. As specified by the UFC 4-023-03 and the National Institutes for Standards and Technology, the following load case will be applied for the analysis:

$$(0.9 \text{ or } 1.2)\ D + 0.5\ L + (0.2\ W \text{ or } 0.2\ S)$$

According to the NIST, the load combination is derived from a combination of assumptions: the load factors are less than unity due to probability and damage. The probability of a full design snow or wind load at the time of the blast is very high; in fact, the chance of exceeding this annual probability is about 0.05. The damage to the structure and its contents following an explosion allows for the live load factor reduction. A similar concept could be utilized on the dead load of the structure; however, due to uncertainty in the actual dead load and suspicion in the dependability of a damaged structure, the load factor could be taken as 1.0 or higher, especially as the number of unaffected stories increases. (NIST) With this in mind, the load combination was tailored to better suit the thesis as such:

$$1.2\ D + 0.5\ L + 0.2\ W$$

The dead load is conservatively assigned with a load factor of 1.2 and the wind load was taken as it was shown in Tech 1 that it was the critical lateral force on the Hospital. The snow load was neglected as it was not a critical vertical load on the building. SAP 2000, ETABS, and PCA Slab were used to assist in the analysis.

Structural Analysis and Redesign

Following the guidance provided by the UFC 4-023-03, ASCE Standard 7-05, and the ACI 318-08, the two locations as illustrated in Figure 6 will be analyzed and redesigned to perform safely in the event of a blast. The results of the redesign will be reflected in the cost and schedule of the Monongalia General Hospital project and compared to the original and its effectiveness will be discussed in the Cost and Schedule Analysis section of this report.

Indirect Method

The Indirect Method is used to determine the tie reinforcement required to resist an abnormal loading situation. UFC 4-023-03 classifies design requirements into four categories based on the level of protection (LOP) required: Very Low (VL), Low (L), Medium (M), and High (H). For construction requiring VLLOP to LLOP, only the indirect approach need be considered; and For MLOP and HLOP buildings, however, tie forces must be considered *as well as* an alternate path analysis. To be conservative, the Hospital will be assumed to be a MLOP building, requiring both the Indirect and Direct Design Method for analysis. Detailing is a crucial part of the Indirect Method as the reinforcement must be ensured to act continuously to resist the catenary action mechanism causing the collapse. The different types of forces to be calculated for this analysis are shown in Figure 7. The method was applied at a typical frame within the building since the majority of the building; especially those that are deemed to be critical follow a typical plan layout. The following equations are classified by the different location of the ties. These equations are taken from the Department of Defense and the UFC 4-023-03:

- Basic Strength and Peripheral Ties

$$Ft \leq \begin{cases} 4.5 + 0.9no \\ \\ 13.5 \end{cases}$$

Where: no = Number of Stories

- Internal Ties

$$Ru \geq \begin{cases} \dfrac{D+L}{156.6} \; \dfrac{lr}{16.4} \; \dfrac{Ft}{3.3} \\ \\ \dfrac{Ft}{3.3} \end{cases}$$

Where: D = Dead load
L = Live load
Lr = Distance between the supports
Ft = Basic strength

- Horizontal Tie to Columns

$$Ru \geq \begin{cases} 0.03[4(D+L)]At \\ Ru' \leq \begin{cases} 2.0\ Ft \\ \dfrac{ls}{8.2} \end{cases} \end{cases}$$

Where: At = Tributary Area
ls = Floor to floor height

- Vertical Column Ties

$$Ru = At\ (D+L)$$

- Corner Column Ties

$$Ru \geq \begin{cases} 0.03[4(D+L)]Av \\ Ru' \leq \begin{cases} 2.0\ Ft \\ \dfrac{ls}{8.2} \end{cases} \end{cases}$$

Where: Av = Vertical tributary area

- Tie Forces

$$Ru = \Omega o\ \Phi\ \text{fy As}$$

According to the Department of Defense and the UFC 4-032-03, the reinforcement must provide a design strength greater than the required tie strength multiplied by a strength reduction factor, Φ, and an over-strength factor, Ω_o. The strength reduction factor is prescribed in ACI 318-08 as 0.75 for steel reinforcement under tension. The over-strength factor is prescribed as 1.25 by the UFC 4-032-03 for both concrete and reinforcement. The reason for the over-strength factor is due to the fact that structural sections designed are usually larger than required, the design capacity is reduced by its appropriate factor, and the material strength itself is a conservative statistical average such that most elements are higher than the design strength. All of these reasons allow for an over-strength factor for this application. Table 1 summarizes the results of the tie forces and the reinforcement required:

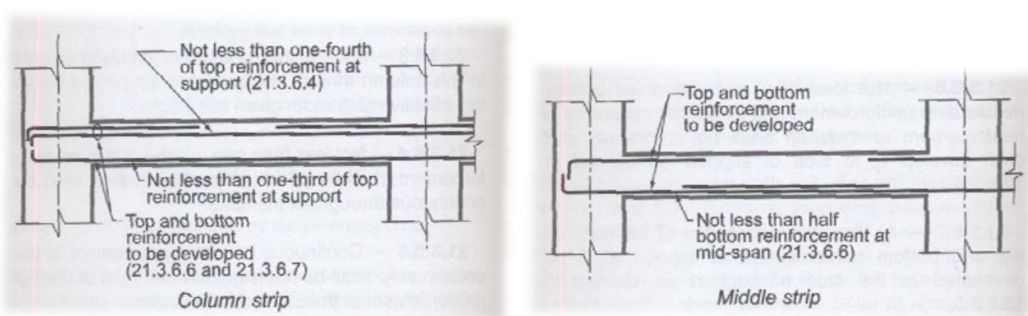
Table 1: Tie Forces and Reinforcement Areas

Tie	*Tie Force (kips)*	$As_{REQ'D}$ *(in^2)*	$As_{PROV'D}$ *(in^2)*
Peripheral	9.9	0.176	0.93
Internal (E-W)	6.02 /ft_{width}	0.107 /ft_{width}	1.607 /ft_{width}
Internal (N-S)	5.31 /ft_{width}	0.0945 /ft_{width}	0.408 /ft_{width}
Horizontal	14.8	0.263 /ft_{width}	0.33 /ft_{width}
Vertical	123.3	2.19	6
Corner Column	121.3	2.16	6

The calculations can be found in Appendix F

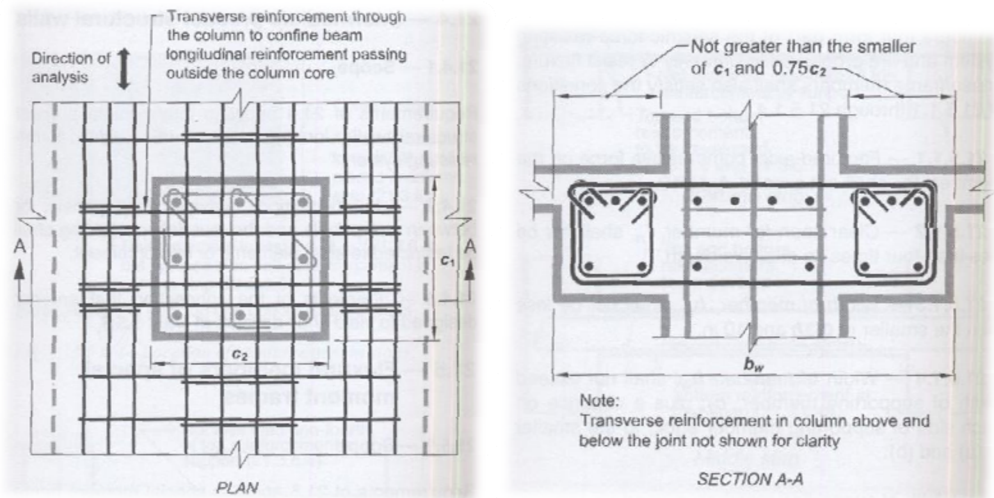
The reinforcement provided in the existing construction has proved to be adequate through the calculations; however the details must be changed to properly reflect the reinforcement requirements. Corley and Hayes et al. have examined the effects of alternative seismic design and strengthening, and have pointed out positive impacts of seismic resistance on progressive collapse resistance. Many other papers have pointed out the physical characteristics of blast and seismic loading that trigger the collapse, and the relationship between the two types of loading under a common detailing scheme based on seismic provisions. “A key finding of was that strengthening the perimeter elements using current seismic detailing techniques improved the survivability of the building.” (Hayes, et. al) New details to reflect this requirement have been illustrated in Figure 8 and Figure 9:

Figure 8: Column and Middle Strip Detailing



Source: ACI 318-08

Figure 9: Transverse Reinforcement Around Columns



Source: ACI 318-08

Other detailing requirements as per UFC 4-023-03 call out for the continuity of the reinforcements by the use of mechanical splices. Splices for the column reinforcement must be located at third points of the story height using Type 2 Mechanical Splices, and must be continuous along the entire height of the building. The reinforcement for beams and slab must also be continuous through the length of a bay, using Type 2 Mechanical Splices as well. Ties for corner columns and connections between horizontal and vertical members must utilize a 135 degree seismic hook. The cost by added detailing will increase and this will be discussed in the Cost and Schedule Analysis section of this report.

Direct Method

Two locations have been chosen for analysis for a collapse situation as mentioned earlier (see Figure 6) and the assumptions made for this report have made the Alternative Path Method the viable approach to the analysis and redesign of the structure. These locations were chosen due to its accessibility and the magnitude of damage the collapse could induce on the building in the event of a blast.

The first location of interest (marked 1 in Figure 6) will be located by the east lobby area. The lobby is a public area and placing of charges is a relatively easy task, also, there are five floors resting above the lobby which could be disastrous if a column was to be eliminated and a collapse triggered. In such an event, the slab on the second floor must be capable of resisting a 54 foot span which can cause a collapse mechanism to occur. The second location of interest (marked 2 in Figure 6) is the corner column on the southwestern portion of the building. This too, will be an easy target since the perimeter of the Hospital is surrounded by parking lots and the potential for a car bomb, or even a truck colliding into the column is feasible.

The existing two way slab was analyzed in the aforementioned two locations, simulating a column removal. The first location will simulate a 54 foot span due to the removal of a column from the lobby area. Initially, the existing two way slab was analyzed for its load carrying capacity under the given condition, ceterus paribus; and the design was concluded to be inefficient due to the amount of reinforcement required to safely span a 54 foot span, PCA Slab was used to model and analyze the 54 foot span condition and hand calculations were done to verify the program's output. Table 2 is a summary of the analysis conducted at Location 1:

Table 2: Two Way Slab Analysis—Required Reinforcement, Location 1

	Frame A		*Frame B*	
	M^+	M^-	M^+	M^-
Column Strip	(19) #5	(33) #5	(18) #5	(33) #5
Middle Strip	(25) #5	(57) #5	(10) #5	(25) #5

The calculations can be found in Appendix F.

A No. 5 reinforcing bar was used for the analysis to stay consistent with the Hospital's existing design. Upon completing the analysis, it becomes obvious that in the column strip regions for both negative and positive moments, the steel reinforcement required is extremely high. Note that for the analysis, existing conditions were kept as constant such as the slab thickness at eight inches and the compressive strength of the concrete was kept at 5000 psi. These values could have been altered to possibly keep the structural integrity of the existing design. These constants could have been altered for the analysis however, it would have called for a 17 inch thick slab, significantly increasing the project cost as well as affecting the architectural integrity of the structure by having shorter floor to floor heights.

An alternative floor system will be introduced to this design as to not hinder the project cost and the architectural integrity of the Hospital. A post-tensioned slab was designed due to its strength characteristics, noting especially that the post-tensioned slab works in a two-way action; increasing the robustness of the structure. The design of the new slab called for a two inch increase in the slab thickness and uses 39- half inch diameter, seven wire strand tendons. The calculations can be found in Appendix F.

The second location will simulate a corner column collapse, introducing a 30'-4" cantilever situation. Table 3 summarizes the results of the analysis done by hand calculations and verified by PCA Slab:

Table 3: Two Way Slab Analysis—Required Reinforcement, Location 2

	Frame C			*Frame D*		
	M^-_{EXT}	M^+_{INT}	M^-_{INT}	M^-_{EXT}	M^+_{INT}	M^-_{INT}
Column Strip	(32) #5	(50) #5	(61) #5	(31) #5	(31) #5	(20) #5
Middle Strip	(10) #5	(13) #5	(27) #5	(23) #5	(27) #5	(16) #5

The calculations can be found in Appendix F.

A No. 5 reinforcing bar was used for the analysis to stay consistent with the Hospital's existing design. The ineffectiveness (the use of excessive steel) of the design in Location 2 becomes apparent by looking at the required reinforcement needed to provide enough structural integrity during the event of a collapse. Similar to Location 1, a post-tensioned slab will be utilized to maintain structural integrity. These calculations can be found in Appendix F.

Conclusion of Structural Analysis and Redesign

At this point, two different types of design methods have been utilized to come up with a design to resist a progressive collapse situation. No solid conclusions can be made as to which design method is more effective, however some alternatives can be dropped due to its excessive use of steel. The following section, Cost and Schedule Analysis will analyze the different designs in terms of cost, and its influence on the schedule.

Cost and Schedule Analysis

The structural analyses and re-designs have both proved to be successful in terms of providing the right structural capacity to withstand a collapse scenario however; the changes must be reflected on the overall cost and schedule of the project. For this analysis, only the changes are assumed to have any impact on the cost and schedule of the project, and all other factors are assumed to be constant. A cost analysis will be conducted first to filter the different design methods in terms of cost, and a schedule analysis will be conducted with the top two choices for design in terms of structural integrity, and the cost for construction.

Cost Analysis: Existing Conditions vs. Indirect Design Method Redesign

As analyzed in the Structural Analysis and Redesign section through the Indirect Design Method, the existing concrete reinforcements proved to be adequate to resist the tie forces but concluded that additional detailing must be necessary. The primary resource for the estimated costs was taken from R. S. Means 2008. The steel reinforcement estimates were taken off by using a square foot approximation method by estimating the amount of reinforcement in one square foot and then multiplying by the total area of the slab. The concrete quantities were obtained directly from the construction documents. Table 4 summarizes the findings:

Table 4: Existing Conditions vs. Indirect Design Method Redesign

Existing Conditions: Elevated Slab					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	1230 tons	990.00	475.00		$1,801,950.00
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,869,668.08
Redesigned Conditions: Elevated Slab (Indirect Design)					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	1260 tons	990.00	475.00		$1,845,900
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,928,268.08
				Difference: Redesign - Existing	$43,950.00

The primary changes in cost are due to the 30 ton increase in reinforcing steel. The increase was due to a different detailing scheme for the connections between the different concrete members as well as the need for longer continuity of the reinforcements to resist the catenary action mechanism. With respect to the overall cost of the building, a $43,950.00 increase is a minute change; the design done by the Indirect Design Method seems to be a viable option.

Cost Analysis: Existing Conditions vs. Direct Design Method Redesign

As part of the Direct Design Method, the existing two-way slab was analyzed and redesigned to resist the collapse mechanisms at Locations 1 and 2 (See Figure 6). These two locations' designs will be assumed typical and reinforcement steel take-offs will be reflected accordingly. Table 5 summarizes the results with an existing 8 inch slab, and Table 6 summarizes the results with a 17 inch slab as recommended by ACI 318-08, followed by Table 7 summarizing the results with a PT-slab:

Table 5: Existing Conditions vs. Direct Design Method-Two-Way Slab

Existing Conditions: Elevated Slab					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	1230 tons	990.00	475.00		$1,801,950.00
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,869,668.08
Existing Conditions: Elevated Slab (8" Thick) After Redesign (Indirect Design)					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	5500 tons	990.00	475.00		$8,057,500.00
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$9,125,218.08
				Difference: Redesign - Existing	$6,255,550.00

From the Structural Analysis and Redesign section, it was quite obvious the excessive amounts of steel required for a two-way slab system made the construction inefficient. Table 5 summarizes the hike in cost from the existing design to a new design by the Direct Design

Method with all concrete and concrete accessory parameters kept constant. The new two-way slab will add over six million dollars to the project cost, a very inefficient design.

Table 6: Existing Conditions vs. Direct Design Method-Two-Way Slab (17" Thick)

Existing Conditions: Elevated Slab					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	1230 tons	990.00	475.00		$1,801,950.00
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,869,668.08
Existing Conditions: Elevated Slab (17" Thick) After Redesign (Indirect Design)					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	7469.49 yd^3	111.00			$829,113.39
Placement	7469.49 yd^3		13.55	4.94	$138,110.87
Reinforcing Steel	2000 tons	990.00	475.00		$2,930,000.00
Formwork	70149.2 ft^2	1.55	3.43		$349,343.02
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$4,381,720.68
				Difference: Redesign - Existing	$1.512.052.60

With a 17 inch thick slab, the amount of steel reinforcement compared to that seen in Table 5 has been significantly decreased however costs related to the concrete and the placing of; have increased; although no detailed design calculations have been done, a scaling factor has been calculated to roughly adjust the quantity values. The differences in cost must also be noted, and compared to that of Table 5, the 17 inch thick slab will raise costs by about 1.5 million dollars, as opposed to over six million dollars with an 8 inch thick slab. In terms of cost and structural integrity, the 17 inch thick slab is efficient however; this will decrease the floor to floor height of the building. This change in floor heights could affect a multitude of other systems in the building such as the mechanical and plumbing systems, electrical wiring, among many others that run within the ceiling of each floor covered by acoustic panels. The effects of these are beyond the scope of this thesis and will not be discussed in detail.

Table 7 will summarize the differences between the existing two-way slab and a PT-slab designed by the Direct Design Method. Like the redesign done for the two-way slab by the same method, the PT-slab will be reflected into the existing design and assumed typical, and take offs will be conducted accordingly.

Table 7: Existing Conditions vs. Direct Design Method-PT Slab

Existing Conditions: Elevated Slab					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	5290.89 yd^3	111.00			$587,285.46
Placement	5290.89 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	1230 tons	990.00	475.00		$1,801,950.00
Formwork	49689 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,869,668.08
Redesigned Conditions: PT Slab (Direct Design)					
	Quantity	*Unit Cost*	*Labor Cost*	*Equipment Cost*	*Total Cost*
5000 psi Concrete	6613.62 yd^3	111.00			$587,285.46
Placement	6613.62 yd^3		13.55	4.94	$97,828.00
Reinforcing Steel	500 tons	990.00	475.00		$293,000.00
Prestressing Steel	600 tons	1800.00	475.00		$1,365,000.00
Formwork	62111.3 ft^2	1.55	3.43		$247,451.22
Slab Finishing	198755 ft^2		0.68		$135,153.40
				Total	$2,958,864.00
				Difference: Redesign - Existing	$89,195.95

Through the direct design, the PT slab design has yielded a total cost of over three million dollars, more than a five hundred thousand dollar increase from the existing construction. From the data in Tables 4 to 7, the designs to note are the Indirect Design of the existing elevated slab and the Direct Design-PT-Slab. Table 8 is a quick comparison between the Indirect Design Method and the Direct Design Method's PT-slab design:

Table 8: Indirect Design vs. Direct Design-PT-Slab

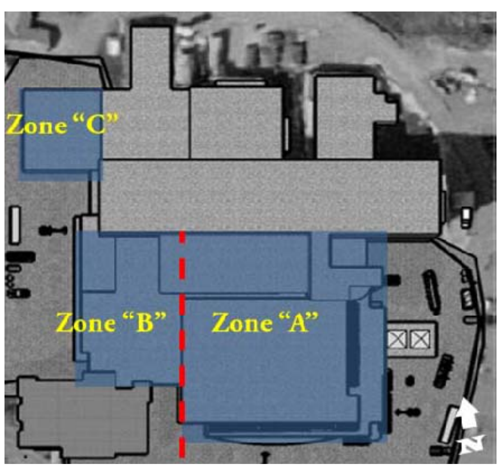
<table>
<tr><th>Design Method</th><th>Major Cost Contributors</th><th>Total Cost of Construction</th><th>Difference to Existing Construction</th><th>Impact to Overall Project Cost</th></tr>
<tr><td>Indirect Design Method</td><td>Reinforcing Steel $1,801,950.00</td><td>$2,869,668.08</td><td>$43,950.00</td><td>+0.077%</td></tr>
<tr><td rowspan="2">Direct Design Method-PT-Slab</td><td>Reinforcing Steel $293,000.00</td><td rowspan="2">$2,958,864.00</td><td rowspan="2">$89,195.95</td><td rowspan="2">+0.156%</td></tr>
<tr><td>Prestressing Steel $1,365,000.00</td></tr>
<tr><td colspan="2">Difference</td><td>$89,195.92</td><td>$45,245.95</td><td></td></tr>
</table>

The increases in cost presented by the two different design methods are rather minute in terms of overall project cost, although the Indirect Design Method is the more economic choice, at a 0.077 percent increase in overall project cost as opposed to a 0.156% increase if the Direct Design Method is used to design the PT-slab.

Schedule Analysis

Much like the Cost Analysis subsection, only the changes are assumed to have any impact on the schedule of the project, and all other factors are assumed to be constant however; any discrepancies encountered will be considered and the assumption will be adjusted accordingly. The schedule separates the main structure of the Hospital renovation into two, Area A and Area B, in which each area is constructed one after another in an end-to-start fashion. Figure 10 shows the different areas of the construction:

Figure 10: Zones of Construction



Source: Brutico

From the Cost Analysis subsections, the Indirect Design Method and the Direct Design Method – PT-Slab designs have been chosen as the top two design methods in terms of their cost impact to the overall project volume. A schedule analysis will be conducted to determine which design method is the more beneficial choice for the Hospital. For the interest of time, a detailed schedule analysis was not conducted and the phasing plan provided by Turner Construction (see Appendix B) was used as the overall schedule of the project and adjusted accordingly. The following Tables 9 to 10 summarize the results of the schedule analysis:

Table 9: Area A Construction Statistics

	Work Days	
Original Schedule	137	
Indirect	138	-
Direct-PT	-	165
Difference	-1	-28

Table 10: Area B Construction Statistics

	Work Days	
Original Schedule	89	
Indirect	87	-
Direct-PT	-	107
Difference	+2	-18

Upon the Schedule Analysis, it becomes apparent that the PT-Slab designed by the Direct Design Method is an inefficient design. For both areas, the schedule will be pushed behind by a total of 46 days as opposed to the schedule advancing a day if the Indirect Design Method was used to design the building for progressive collapse. This is due to the fact that in the Indirect Design Method, the construction methods are not different but uses longer spans of reinforcing steel. On the other hand, a PT-slab takes significantly more time to construct since the tendons can only be tensioned once the concrete has cured, also during the jacking process; safety is a major concern, which limits the amount of work done in the area during the process.

Conclusion of Cost and Schedule Analysis

The designs from the Structural Analysis and Redesign section have been analyzed in the Cost and Schedule Analysis section for its economic effects as well as its effects on the schedule. Although the cost and schedule estimates were merely rough calculations, it becomes apparent that the Direct Design Method is not an effective choice of design for the Monongalia General Hospital. The designs in terms of cost had very little impact on the overall project volume but when compared in terms of construction time, the PT-slab took significantly longer than collapse mitigating-detailed concrete. Table 11 summarizes the findings of the Cost and Schedule Analysis section for the Indirect Design Method and the Direct Design Method-PT-slab:

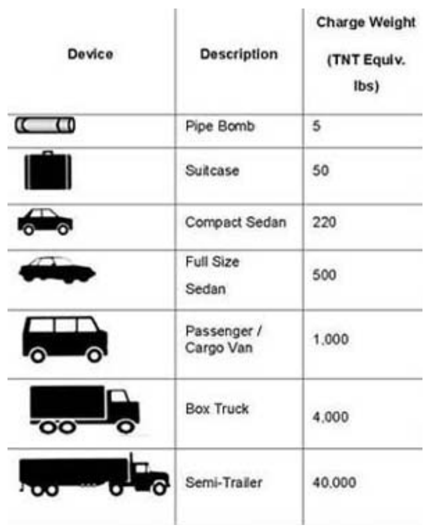
Table 11: Comparing Design Methods

Method	*Indirect Design Method*	*Direct Design Method-PT*
Total Cost Increase (% / $)	+0.077% / $43,950.00	+0.156% / $89,195.95
Schedule Gain(+)/Loss(-)	+1 Day	-46 Days

Architectural Analysis

The façade of the Hospital primarily utilizes brick veneer and curtain walls around the perimeter, in the event of a blast, the brick veneer and curtain walls could be of great danger and cause unnecessary injuries and deaths. In order to prevent such events from occurring, the brick veneer and curtain walls will be analyzed for its effectiveness against such loading conditions. As mentioned earlier, the location and the magnitude of the blast cannot be predicted, however several assumptions have been made to make the analysis possible. A standoff distance of 75 feet and a charge size equivalent to 100 pounds of TNT, a size comparable to two suitcases, as shown in Figure 11:

Figure 11: Blast Charge Description



Device	Description	Charge Weight (TNT Equiv. lbs)
	Pipe Bomb	5
	Suitcase	50
	Compact Sedan	220
	Full Size Sedan	500
	Passenger / Cargo Van	1,000
	Box Truck	4,000
	Semi-Trailer	40,000

Source: U.S. Department of Transportation

A charge of 100 pounds was chosen, a value assumed for its conservativeness compared to that of Norville and Conrath's studies, stating that "most intentional blasts in the United States are relatively small … generally on the order of 10 lb or less."

The designs will also go through a simple heat transfer analysis, analyzing the conductive heat flow through the wall system. Emphasis will be placed more on the curtain wall construction in which the designs will be compared in terms of thermal characteristics as well as cost. The thermal characteristics are based on values taken from the Heat, Air, and Moisture Toolbox and the results were inserted into the following equation for conductive heat flow:

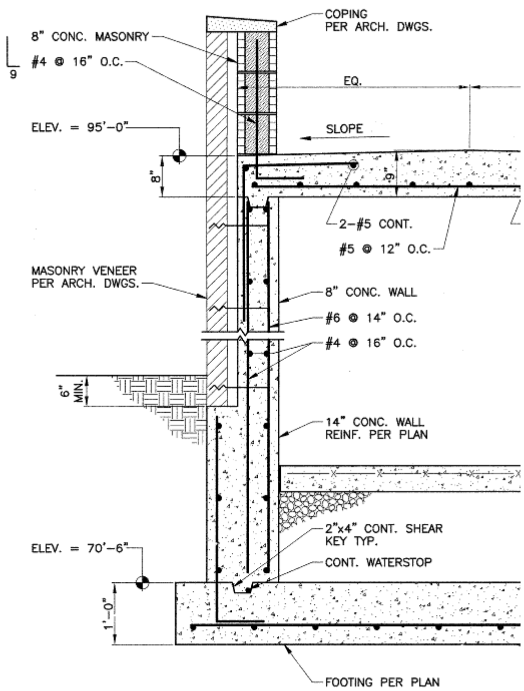
$$Q_c = A * U * \Delta T$$

Where: A = Surface area
U = Coefficient of heat transfer
Δ T = Temperature difference

Brick Veneer

The brick veneer takes up about 60% of the building envelope, and its chances of encountering a blast situation is highly likely. A typical unreinforced brick veneer section was taken and analyzed against blast loading taken from ASTM F 2248-03.

Figure 12: Typical Wall Section



Source: Drawing S5.1; Freeman White

Through the analysis, it was found that the veneer itself is incapable of carrying the blast load. Some design alternatives could include the use of reinforcements (the veneers are unreinforced) however, it could raise the project cost significantly. Also, to keep in mind, it is backed up with an 8 inch concrete wall designed to resist seismic loads. Figure 12 shows a typical wall section using brick veneer. The brick veneer was also analyzed for its thermal transfer qualities using the Heat, Air, and Moisture Toolbox, a computer program designed to determine heat flows through different wall systems. From the analysis run on the program, the R value of the construction was found; Table 12 summarizes the conductivity characteristics of the wall construction:

Table 12: Conductive Properties of the Brick Veneer Wall

Material	*R (hr*ft^2*°F/BTU)*	*U (BTU/hr*ft2*°F)*
Air Film	0.17	5.88
Brick	0.64	1.56
Cavity	0.98	1.02
Rigid Insulation	7.9	0.13
Concrete Wall	0.87	1.15
Gypsum Board	0.46	2.17
Air Film	0.64	1.56
Total	11.66	0.09

From Table 12, the conductive heat flow can be calculated and the following values were obtained:

Summer: 1.62 BTU/hr per square foot
Winter: 4.68 BTU/hr per square foot

Curtain Wall

Curtain walls do not make up the majority of the building façade however its critical placement is adjacent to the area around Location 1 (see Figure 6), and in the event of a blast its effects could be devastating. As mentioned earlier, the assumptions of standoff distance and charge weight was used to analyze the curtain wall system following the analysis procedure of ASTM F 2248-03. Calculations can be seen in Appendix B. Table 13 summarizes the design results:

Table 13: Curtain Wall Designs

Glass Type	*Load Resistance*	*Maximum Charge Capacity*
1/4" THK, Heat Strengthened, 1 Lite, Existing	98 PSF	100 lb_{TNT}
1/4" THK, Heat Strengthened, 2 Lite	195 PSF	300 lb_{TNT}
1/4" THK Fully Tempered, 1 Lite	217 PSF	400 lb_{TNT}
Demand	**98 PSF**	**100 lb_{TNT}**

The existing 1 lite, heat strengthened glass just meets the demand, and therefore concluded that it is not a viable design against the assumed blast. Two other designs have been iterated, the first alternative is to use a 2 lite action with the same glass used in the existing construction, this provides significantly higher load resistance and is capable of withstanding a 300 pound TNT equivalent charge. The second alternative is to use a 1 lite, fully tempered glass, which provides a load resistance of 217 PSF, an equivalent to 400 pound TNT equivalent charge. Table 14 to 16 summarizes the conductive heat transfer characteristics of the three different types of curtain wall construction, the calculations can be found in Appendix B:

Table 14: Conductive Properties of Heat Strengthened, 1 Lite Curtain Wall

	Summer	*Winter*
∑R ($hr*ft^2*°F/BTU$)	0.97	0.89
*U (BTU/hr*ft2*°F)*	1.03	1.13
Q (BTU/hr) per panel	275	873

Each panel is 5.5’ x 2.7’

Table 15: Conductive Properties of Heat Strengthened, 2 Lite Curtain Wall

	Summer	*Winter*
∑R ($hr*ft^2*°F/BTU$)	1.8	2.06
*U (BTU/hr*ft2*°F)*	0.56	0.49
Q (BTU/hr) per panel	149	378

Each panel is 5.5’ x 2.7’

Table 16: Conductive Properties of Fully Tempered, 1 Lite Curtain Wall

	Summer	*Winter*
∑R ($hr*ft^2*°F/BTU$)	4.43	4.35
*U (BTU/hr*ft2*°F)*	0.23	0.23
Q (BTU/hr) per panel	61	177

Each panel is 5.5’ x 2.7’

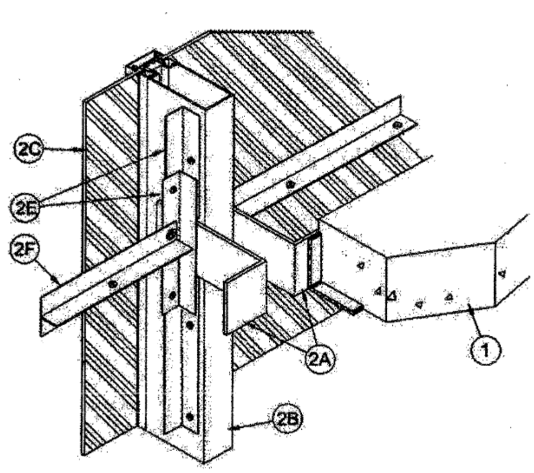
It becomes apparent that the conductivity of fully tempered, 1 lite curtain wall construction has the best thermal qualities among the different design alternatives. Following the thermal characteristics analysis, R.S. Means 2008 will be referenced to price the different glass types. Table 17summarizes the different costs associated with the construction:

Table 17: Costs of Different Curtain Wall Construction

Glass Type	*Cost per square foot*
1/4" THK, Heat Strengthened, 1 Lite, Existing	\$5.30
1/4" THK, Heat Strengthened, 2 Lite	\$10.60
1/4" THK Fully Tempered, 1 Lite	\$16.95

The fully tempered, 1 lite construction has the best thermal properties however, it is the most expensive at \$16.95 per square foot. The 2 lite construction of heat strengthened glass has thermal properties better than that of the existing construction and provides more than adequate capacity against the assumed 100 pound TNT equivalent charge. Although the 2 lite construction of heat strengthened glass seems like a viable option, the entire construction detail of the curtain walls around the building must be changed, (see Figure 11 for existing curtain wall detail) to accommodate two layers of glass with a half inch cavity in between them. With this matter in mind, the best option will be to use a 1 lite construction using fully tempered glass.

Figure 11: Typical Curtain Wall Detail



Source: Drawing G2.2; Freeman White

Conclusion of Architectural Analysis

A tough decision must be made after observing the results of this section. First the brick veneer wall was analyzed against the blast load and found to be inadequate to resist the blast loads, this problem is easily solved by providing adequate reinforcements, however this will significantly increase the cost of the project and was not investigated any further. The curtain walls on the other hand have been analyzed in detail in terms of blast resistance, thermal characteristics, and cost. A tough decision must be made: to choose an expensive but safe curtain wall construction with the use of fully tempered glass, or to keep the existing curtain wall which is only adequate to resist the 100 pound TNT equivalent. Conservatively, the former was picked for the safety of the building as well as its high thermal characteristics.

Conclusion

In recent years the research and development of blast resistant design and progressive collapse mitigation design has increased in number triggered by catastrophic events such as, in recent memory the September 11 World Trade Center incident. Hypothetically, for the interest of this thesis; the Monongalia General Hospital has decided to improve their structural system by incorporating blast and collapse mitigation design. Upon studying the characteristics of blast loading and progressive collapse, two locations were selected as the basis of study for these events (see Figure 6). Due to the complex nature of blast loading, due to its unpredictability, a significant assumption was made; the analysis took place after the blast had occurred and a column removed due to the blast.

The analysis and redesign of the structural system incorporated two different methods of design, the Indirect Design Method and the Direct Design Method. The Indirect Design Method provided the amount of required steel to prevent the catenary action mechanism which triggered the collapse and called out for a new detailing, and the Direct Design Method's Alternative Path approach simulated a column removal in which the floor slabs were required to span longer and mitigate a collapse situation. The latter was used in the aforementioned two locations in the Hospital and the redesign yielded a post-tensioned slab, 10 inches thick. The designs from the two design methods were then compared in terms of effects to the cost and schedule of the project.

Cost and schedule changes are a major part of the project, and in order for the new design to be incorporated, it must have positive effects on both the overall cost and the schedule of the project. The assumption made during this analysis was that the only factors affecting the cost were the reinforcements and post-tensioning, with all else being constant. The different designs from the two design methods were analyzed first, in terms of cost. Both designs from either method had very little effect to the overall cost of the project. When compared in terms of schedule changes, the effectiveness of one design method over the other was apparent. The Indirect Design Method almost had no effect to the schedule, however the post-tensioned slab designed using the Direct Design Method delayed the project by about 2 months. The effect of this delay to the project would have increased the overall project cost as well as delaying the entirety of the construction since the schedule followed an end-to-start flow during the construction of the structural system. The more effective design was apparent by the end of the cost and schedule analysis; the Indirect Design Method was picked to be the design approach to mitigate progressive collapse.

The architecture of the Monongalia General Hospital was studied against blast effects as well. For the architectural analysis, the blast was assumed to be an open-air blast located 75 feet away from the building with a 100 pound TNT equivalent charge, this assumption is conservatively based on Norville and Conrath's "Blast Resistant Glazing Design" which stated that charges used within the United States were in the order of 10 pounds. For the analysis, the curtain wall and the brick veneer, two major elements of the Hospital's architecture; were analyzed against blast loading. The brick veneer proved to be incapable of resisting the blast, however with proper reinforcing, this could be a possibility. This was not further investigated since the addition of reinforcement would have significantly increased the overall project volume. The analysis of curtain walls have been done with more emphasis due to its location: around the lobby area. The existing curtain wall and two alternatives were investigated for its adequacy to resist the blast, its thermal characteristics, and the cost. The existing curtain wall was just enough to resist the demand load but conservatively, a higher strength fully tempered glass was picked to replace the existing, with no changes to the framing of the curtain wall system. This design however will increase the cost of the project since fully tempered glass is more expensive.

Through the investigation conducted for this thesis, the Monongalia General Hospital's existing conditions with slight changes to the reinforcement in the concrete has proved to be adequate to resist a progressive collapse scenario induced by a blast event. Other elements such as the curtain walls have also been analyzed against blast loads and the necessary design changes have been investigated although the increase in cost is, unfortunately inevitable. However in the long run, the new curtain walls have provided better thermal properties to one of the most energy guzzling areas of the building, allowing the Hospital to conserve significant amounts of energy used for heating and air conditioning. One must note that even with a design providing enough structural strength to mitigate collapse and resist events of blast, the structure is not fully blast resistant and necessary precautions must be made to prevent such events from occurring. Preventive measures are key to preventing acts of terrorism and these measures must be taken by the Hospital to assure the safety of their patients, doctors, and the community.

MONONGALIA GENERAL HOSPITAL

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

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

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX B

FIGURES

Figure B 1: Hospital Divided in Four Quads

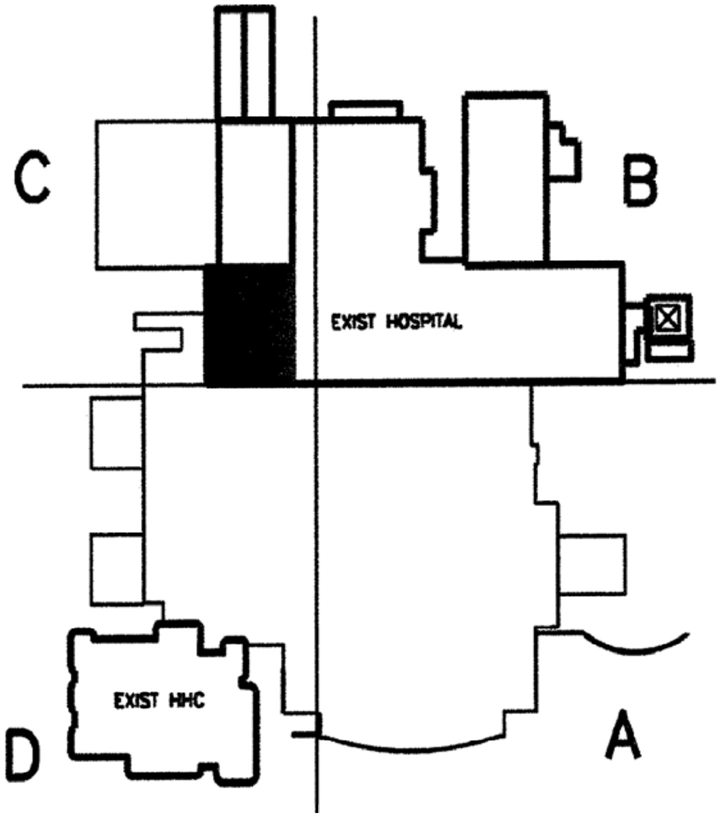
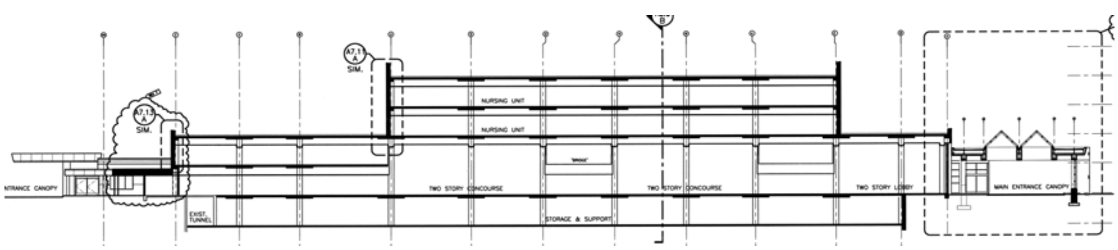
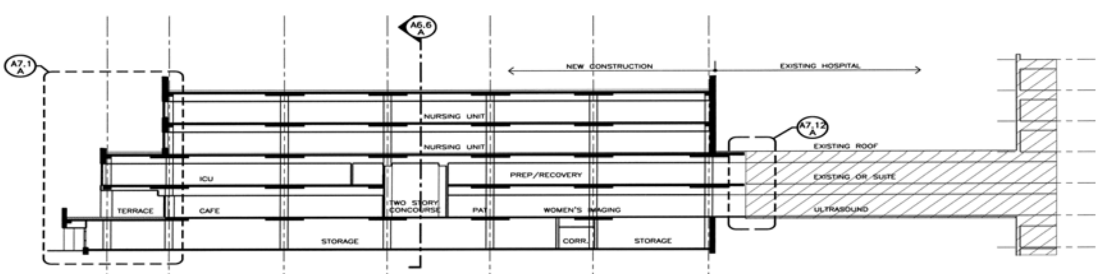


Figure B 2: Cross Section of the Monongalia General Hospital



West Section



South Section

Figure B 3: East Elevation of the Monongalia General Hospital

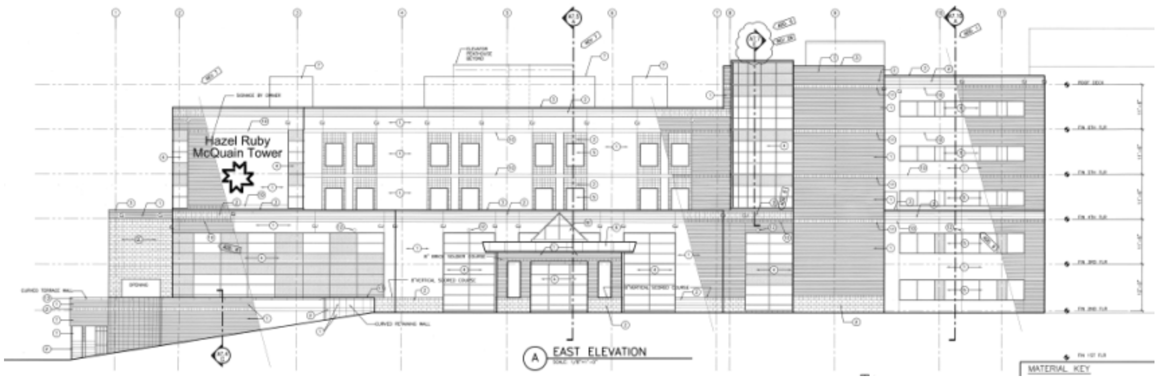


Figure B 4: South Elevation of the Monongalia General Hospital

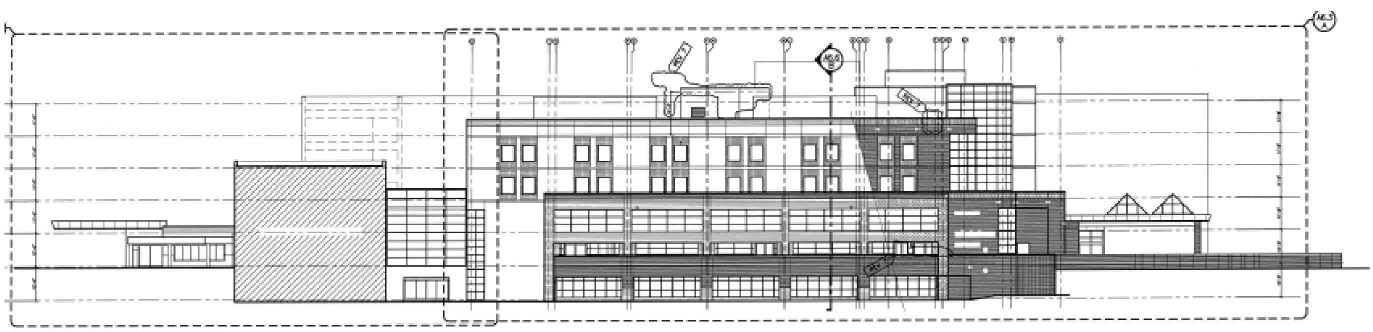


Figure B 5: Location of Shear Walls (Colored in blue) and Blast (Colored in Red)

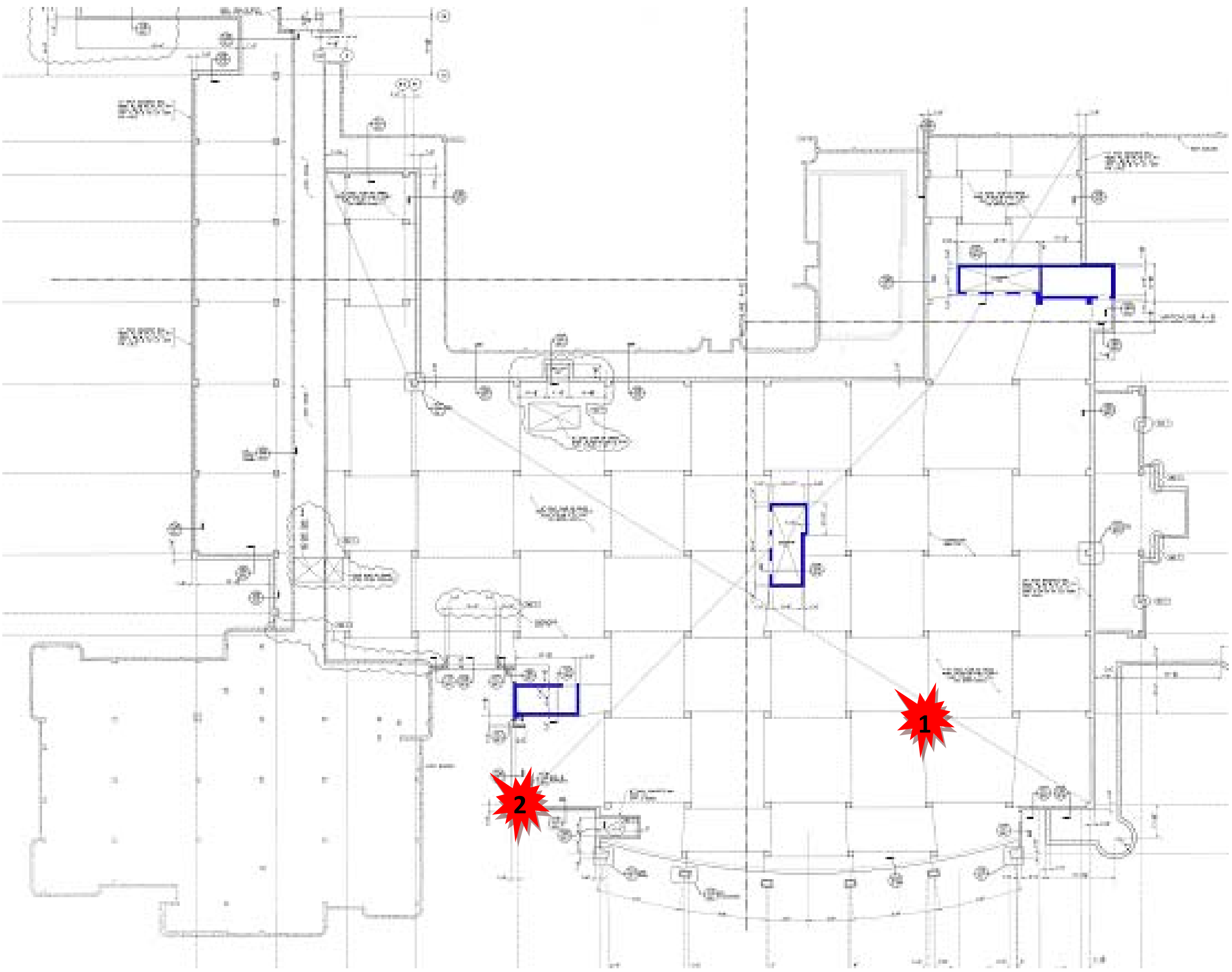


Figure B 6: ETABS Model of the Monongalia General Hospital

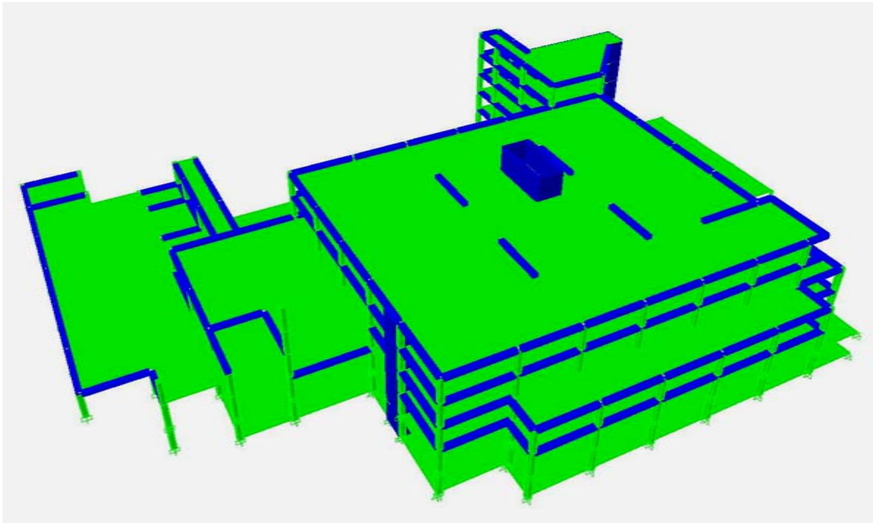


Figure B 7: ETABS Model of the Monongalia General Hospital

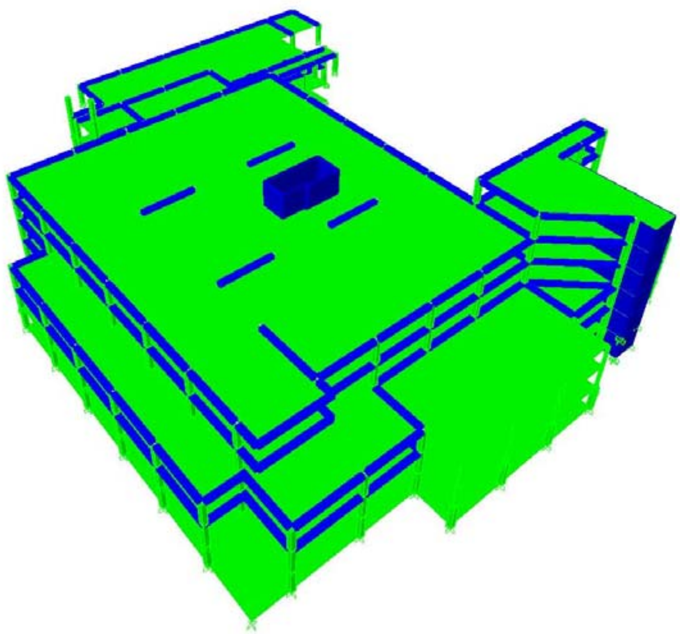


Figure B 8: SAP 2000 Model of the Monongalia General Hospital

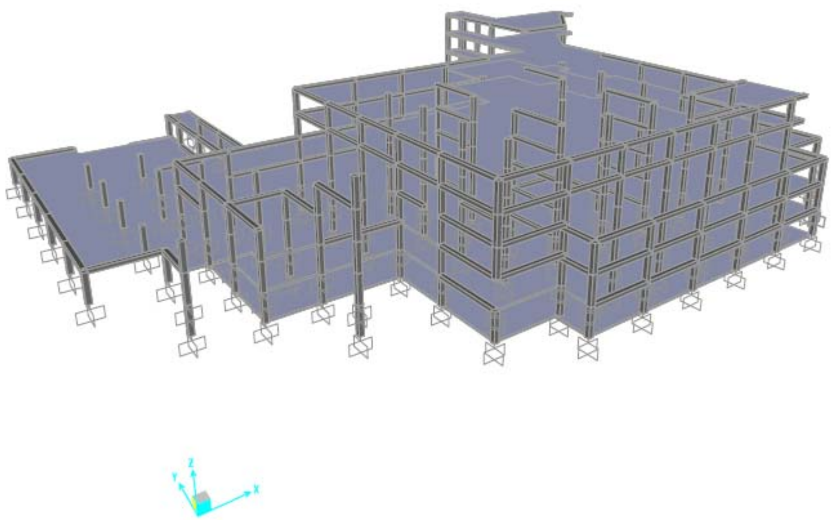


Figure B 9: SAP 2000 Model of the Monongalia General Hospital

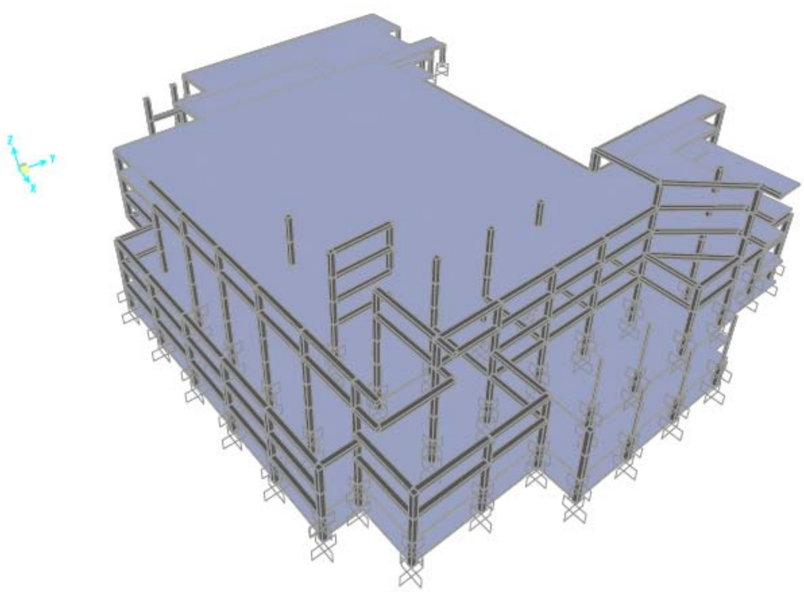


Figure B 10: Division of Areas by Construction Phases

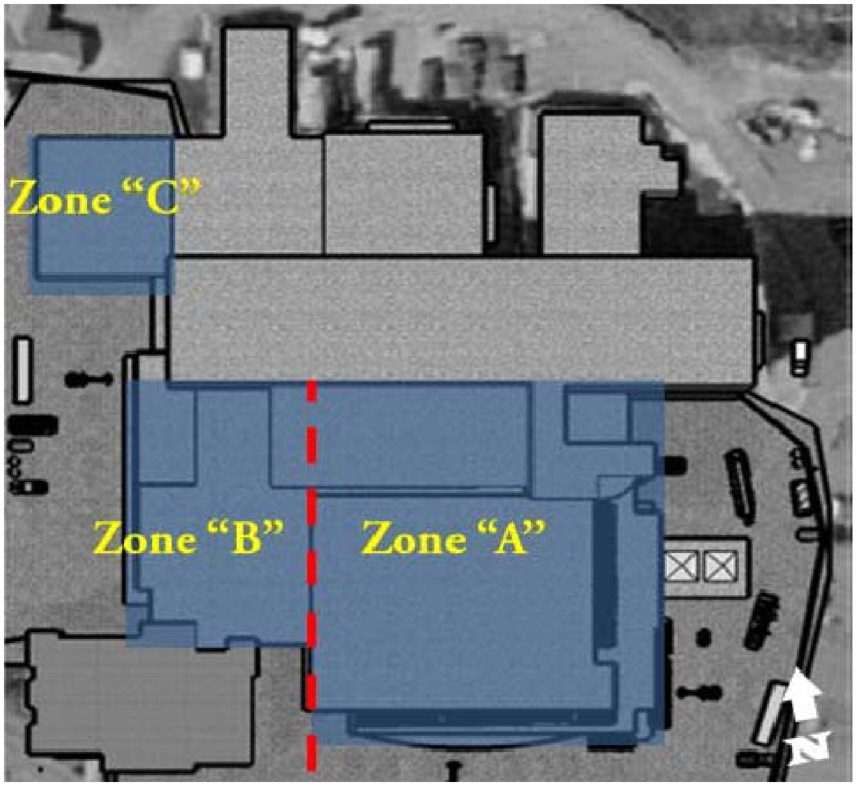
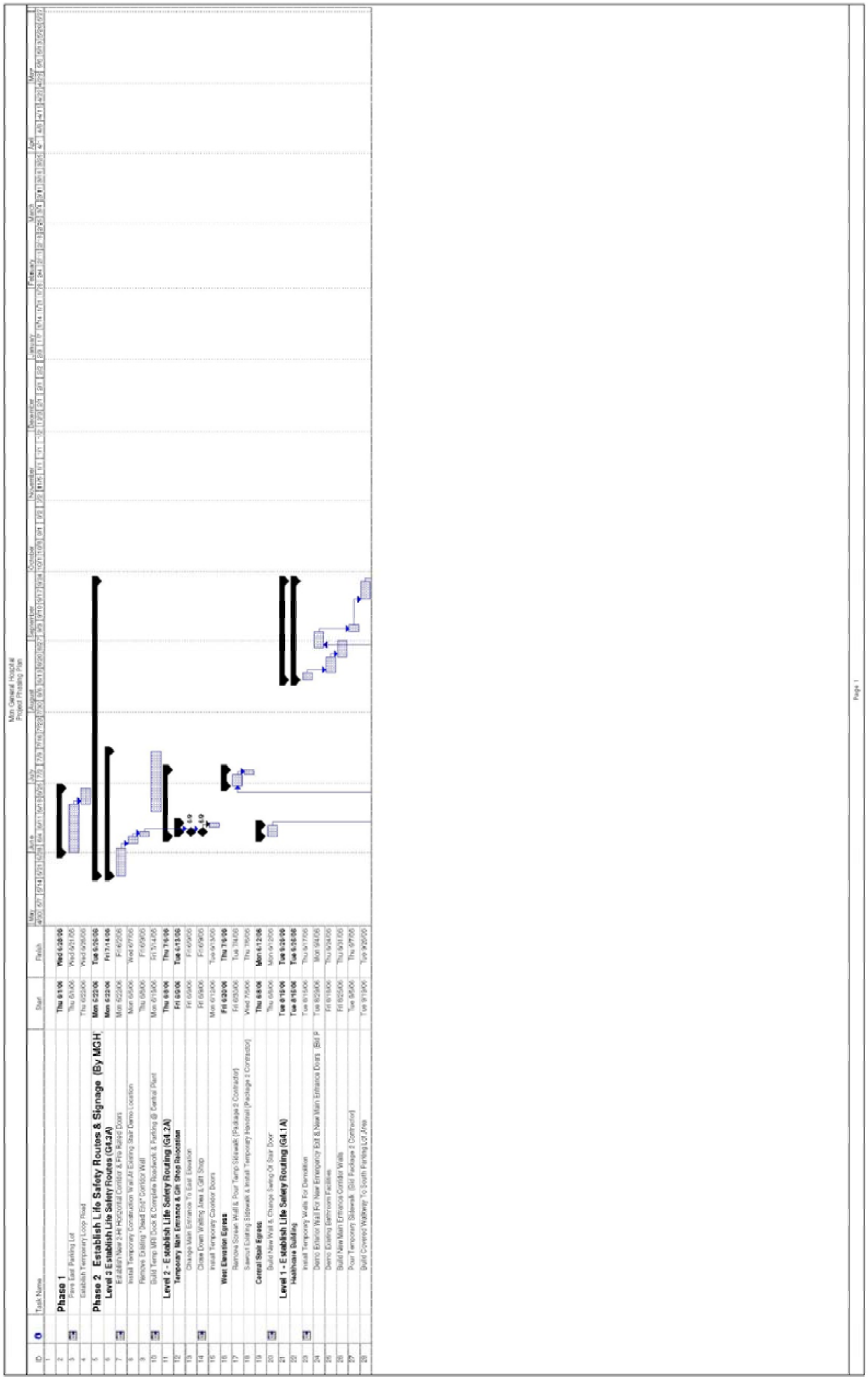
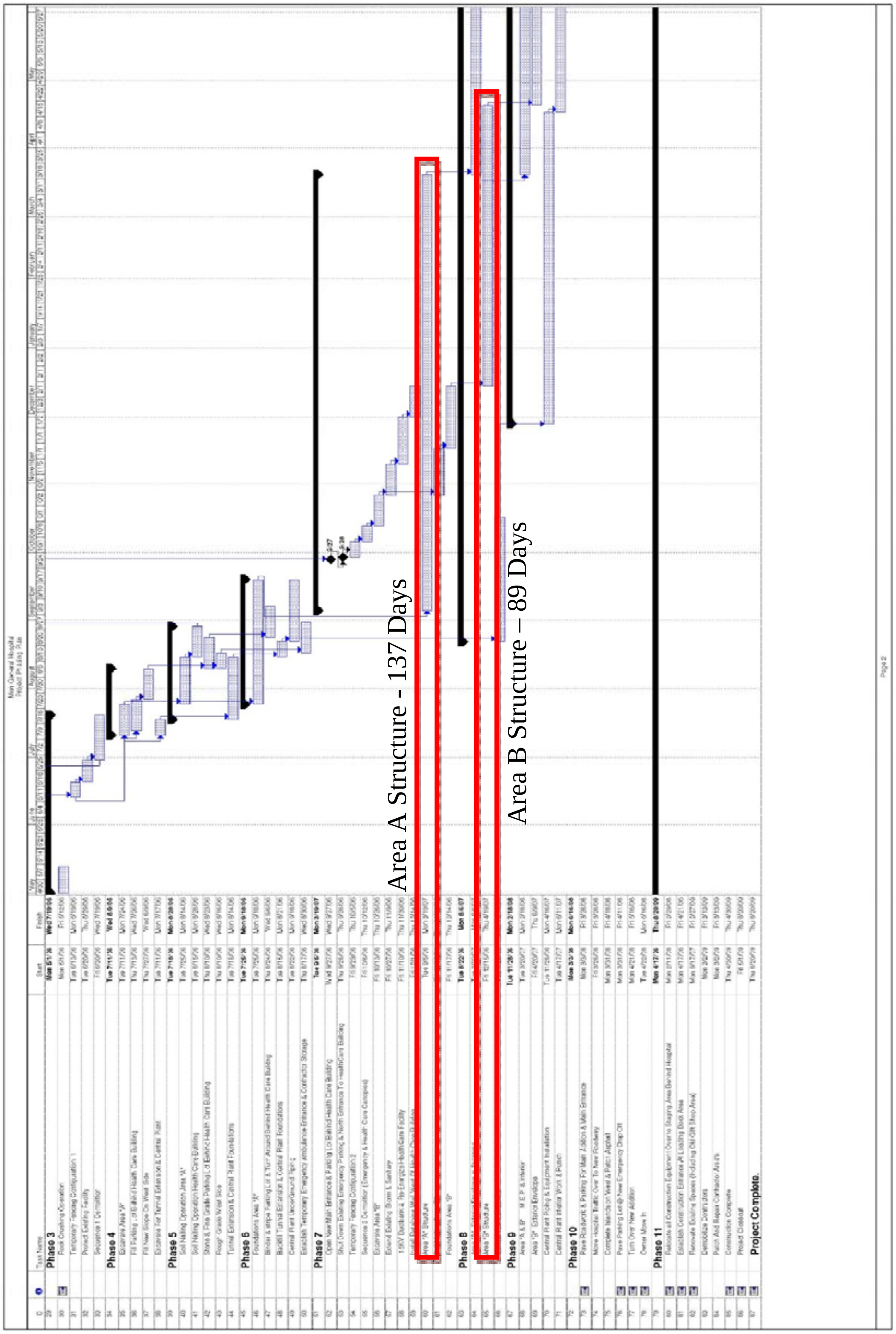


Figure B 11: Monongalia General Hospital Phasing Plan

See the following pages


Phase 1
Phase 2 Establish Life Safety Routes & Signage (By MGH)
Level 3 Establish Life Safety Routes (G4.3A)
Level 2 - Establish Life Safety Routing (G4.2A)
Level 1 - Establish Life Safety Routing (G4.1A)

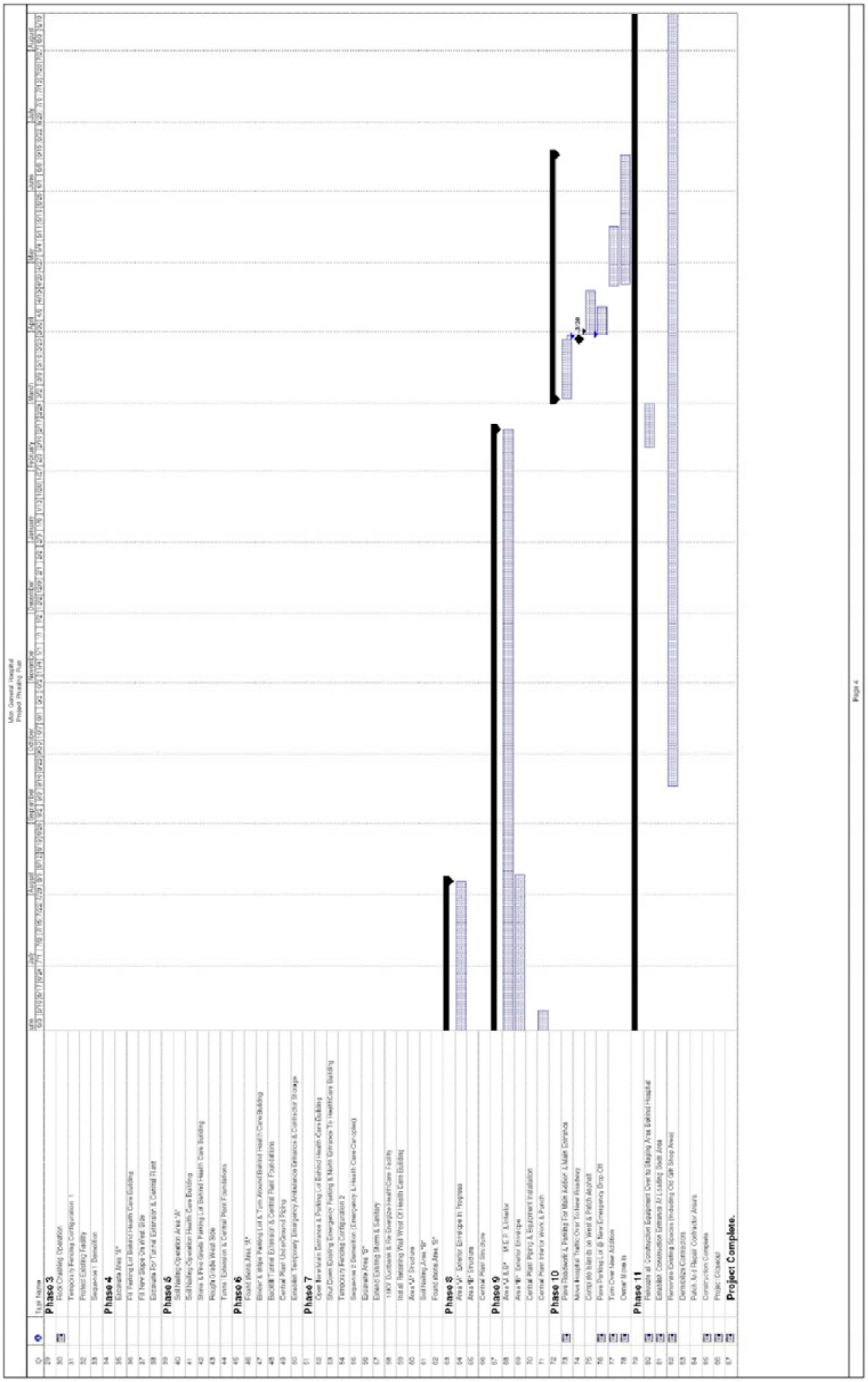

Area A Structure - 137 Days
Area B Structure – 89 Days

Mon General Hospital
Project Phasing Plan

ID	Task Name
1	
2	**Phase 1**
3	Pave East Parking Lot
4	Establish Temporary Loop Road
5	**Phase 2 Establish Life Safety Routes & Signage (By MGH)**
6	**Level 3 Establish Life Safety Routes (G4.3A)**
7	Establish New 2-Hr Horizontal Corridor & Fire Rated Doors
8	Install Temporary Construction Wall At Existing Stair Demo Location
9	Remove Existing "Dead End" Corridor Wall
10	Build Temp MRI Dock & Complete Roadwork & Parking @ Central Plant
11	**Level 2 - Establish Life Safety Routing (G4.2A)**
12	**Temporary Main Entrance & Gift Shop Relocation**
13	Change Main Entrance To East Elevation
14	Close Down Waiting Area & Gift Shop
15	Install Temporary Corridor Doors
16	**West Elevation Egress**
17	Remove Screen Wall & Pour Temp Sidewalk (Package 2 Contractor)
18	Sawcut Existing Sidewalk & Install Temporary Handrail (Package 2 Contractor)
19	**Central Stair Egress**
20	Build New Wall & Change Swing Of Stair Door
21	**Level 1 - Establish Life Safety Routing (G4.1A)**
22	**Healthcare Building**
23	Install Temporary Walls For Demolition
24	Demo Exterior Wall For New Emergency Exit & New Main Entrance Doors (Bid P
25	Demo Existing Bathroom Facilities
26	Build New Main Entrance Corridor Walls
27	Pour Temporary Sidewalk (Bid Package 2 Contractor)
28	Build Covered Walkway To South Parking Lot Area

June | July | August | September | October | November | December | January | February | March | April | May | June | July | August

Page 5


Phase 3
Phase 4
Phase 5
Phase 6
Phase 7
Phase 8
Phase 9
Phase 10
Phase 11
Project Complete.

Mon General Hospital
Project Phasing Plan

ID	Task Name
1	
2	**Phase 1**
3	Pave East Parking Lot
4	Establish Temporary Loop Road
5	**Phase 2 Establish Life Safety Routes & Signage (By MGH)**
6	**Level 3 Establish Life Safety Routes (G4.3A)**
7	Establish New 2-Hr Horizontal Corridor & Fire Rated Doors
8	Install Temporary Construction Wall At Existing Stair Demo Location
9	Remove Existing "Dead End" Corridor Wall
10	Build Temp MRI Deck & Complete Roadwork & Parking @ Central Plant
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16	**West Elevation Egress**
17	Remove Screen Wall & Pour Temp Sidewalk (Package 2 Contractor)
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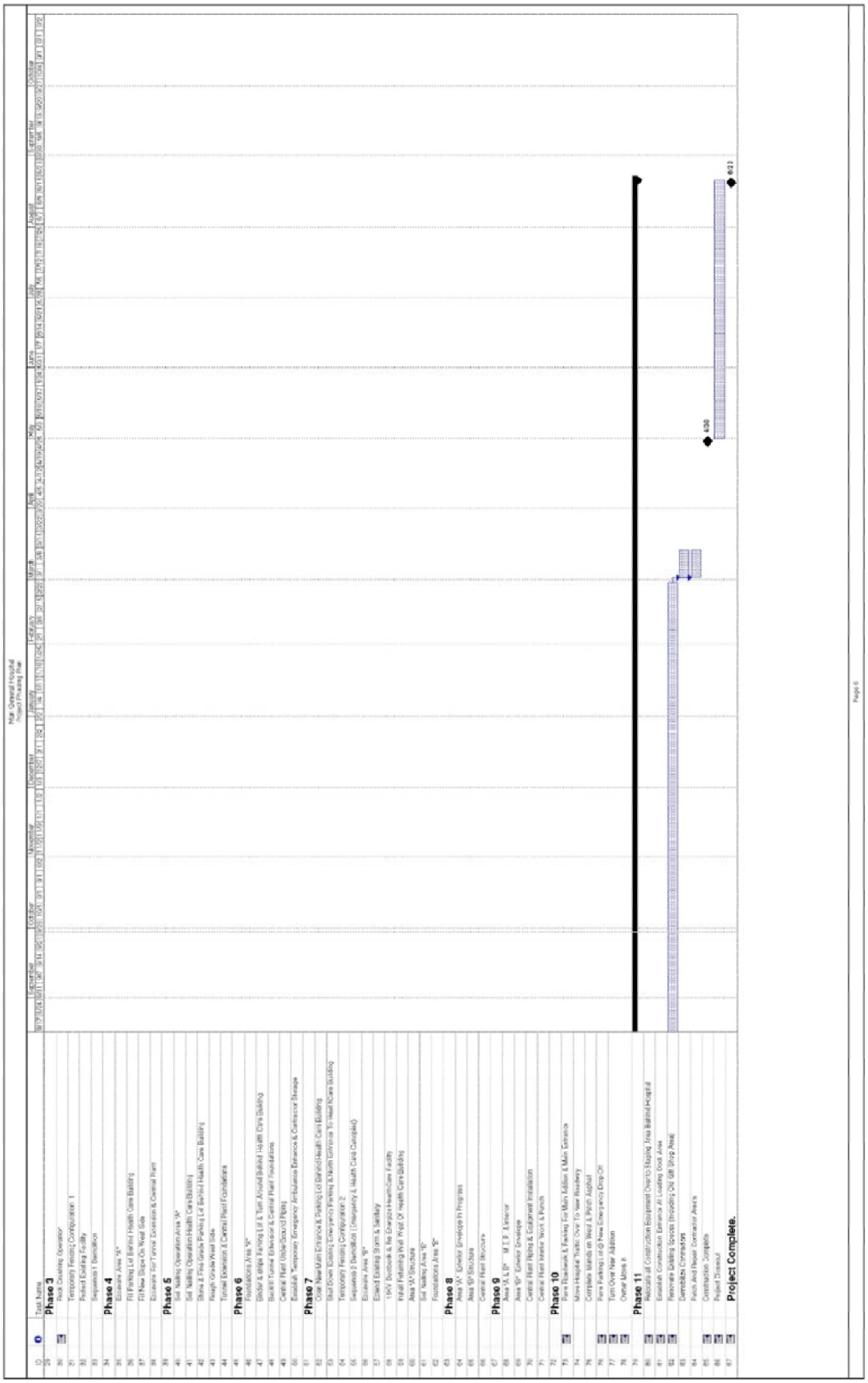
25	Demo Existing Bathroom Facilities
26	Build New Main Entrance Corridor Walls
27	Pour Temporary Sidewalk (Bid Package 2 Contractor)
28	Build Covered Walkway To South Parking Lot Area

Page 6



MONONGALIA GENERAL HOSPITAL

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX C

PHOTOGRAPHS

Photograph C 1: View from South-East

Photograph C 2: Aerial Photo of the Monongalia General Hospital

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

MONONGALIA GENERAL HOSPITAL

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX D

CODES AND STANDARDS

Building Design 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.

Miscellaneous Codes and Standards

American Concrete Institute Committee 318, Building Code Requirements for Structural Concrete, 2008

American Society of Civil Engineers Standard 7, Minimum Design Loads for Building and Other Structures, 2005

American Society of Testing Materials, Standard Practice for Determining Load Resistance of Glass in Buildings, E 1300-04, 2006

Department of Defense, UFC 4-023-03, Design of Buildings to Resist Progressive Collapse

MONONGALIA GENERAL HOSPITAL

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX E

BUILDING DESIGN LOADS

Gravity Loads

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

See Figure E 1 and E 2 for wind loading diagram:

Figure E 1: Wind Loading – North to South

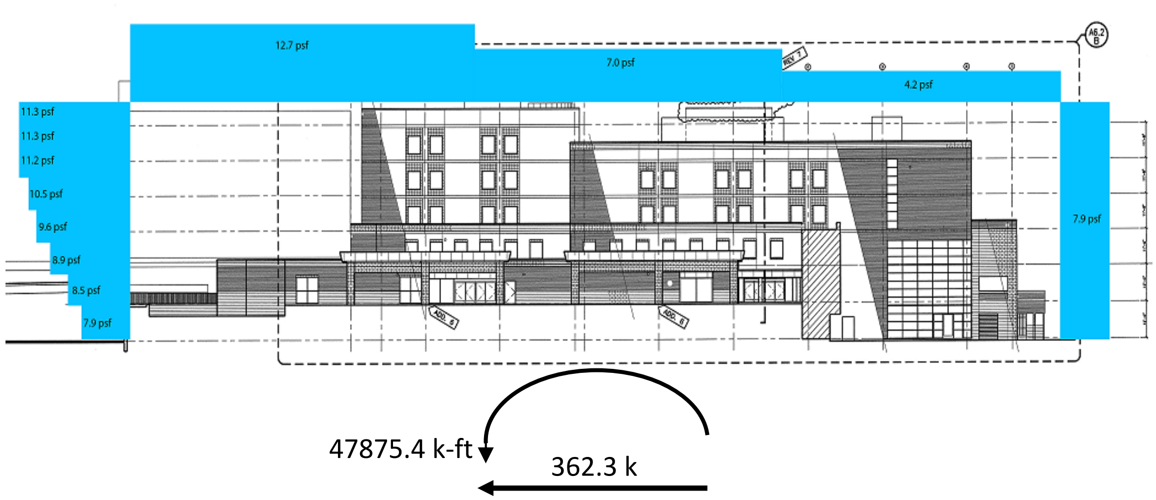


Figure E 2: Wind Loading – East to West

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

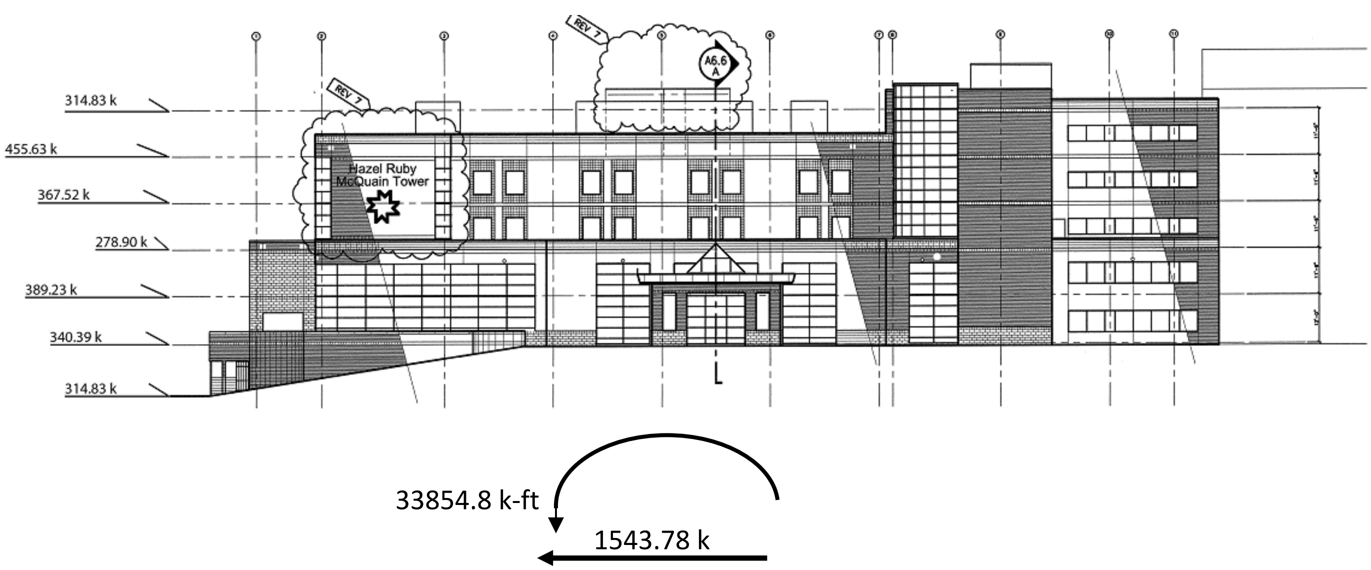
- Occupancy Category	IV
- Importance Factor	1.5
- Seismic Category	A
- Site Class	C
- Spectral Acceleration, Short Period	0.133
- Spectral Acceleration, 1 Second	0.052
- Site Coefficient, F_a	1.2
- Site Coefficient, F_v	1.7
- R-Factor	5.0

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**

See Figure E 3 for seismic loading diagram:

Figure E 3: Seismic Loading



MONONGALIA GENERAL HOSPITAL

BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX F

CALCULATIONS

INDIRECT METHOD - TIE FORCES

- PERIPHERAL TIES

$$F_t \leq \begin{cases} 4.5 + 0.9 n_o & ; n_o = 6 \text{ STORIES} \\ 13.5 k/ft \end{cases}$$

$$F_t \leq \begin{cases} 4.5 + 0.9(6 \text{ STORIES}) = 9.9 k/ft & \text{* CRITICAL} \\ 13.5 k/ft \end{cases}$$

$$F_t = \Omega \phi f_y A_s \quad ; \Omega = 1.25 \quad \phi = 0.75$$

$$\Rightarrow A_s = \frac{F_i}{\Omega \phi f_y} = \frac{9.9 k/ft}{(0.75)(1.25)(60 ksi)} = 0.176 in^2$$

$$A_{s,PROVIDED} = (3)\#5 = 0.93 in^2 > A_s = 0.176 in^2 \quad \text{(GOOD)}$$

- INTERNAL TIES (EAST - WEST)

$$R_u \geq \begin{cases} \frac{D+L}{156.6} \cdot \frac{L_r}{16.4} \cdot \frac{F_t}{3.3} \\ \frac{F_t}{3.3} \end{cases}$$

$$\Rightarrow D = 20 PSF + 90 PSF = 110 PSF$$
$$L = 40 PSF$$
$$L_r = 34.33'$$

$$R_u \geq \begin{cases} \frac{110 PSF + 40 PSF}{156.6} \cdot \frac{34.33'}{16.4} \cdot \frac{9.9 k/ft}{3.3} = 6.02 k/ft & \text{* CRITICAL} \\ \frac{9.9 k/ft}{3.3} = 3 k/ft \end{cases}$$

$$\Rightarrow A_s = \frac{6.02 k/ft}{(0.75)(1.25)(60 ksi)} = 0.107 in^2$$

$$A_{s,PROVIDED} = 1.067 in^2/ft > A_s = 0.107 in^2 \quad \text{(GOOD)}$$

- INTERNAL TIES (NORTH - SOUTH)

$$R_u \geq \begin{cases} \frac{110 PSF + 40 PSF}{156.6} \cdot \frac{30.33'}{16.4} \cdot \frac{9.9 k/ft}{3.3} = 5.314 k/ft & \text{* CRITICAL} \\ 3 k/ft \end{cases}$$

→ CONT'D.

INDIRECT METHOD – TIE FORCES

- INTERNAL TIES (NORTH-SOUTH) CONT'D.

$$\Rightarrow A_s = \frac{5.314\,k/ft}{(0.75)(1.25)(60\,ksi)} = 0.0945\,in^2/ft$$

$$A_{s\,PROVIDED} = 0.408\,in^2/ft > A_s = 0.0945\,in^2/ft \quad (GOOD)$$

- HORIZONTAL TIE TO COLUMNS

$$R_u \geq \begin{cases} 0.03[4(D+L)]A_T = 0.03[4(110psf+40psf)](28.67')^2 = 14.8k \ \text{*CRITICAL} \\ R_u' \leq \begin{cases} 2.0T_t = 2.0(9.9k/ft) = 19.8k \\ \frac{l_s}{8.2}T_t = \frac{11.5'}{8.2}(9.9k/ft) = 13.9k \end{cases} \end{cases}$$

$$\Rightarrow A_s = \frac{14.8k}{(0.75)(1.25)(60ksi)} = 0.263\,in^2$$

$$A_{s\,PROVIDED} = 0.33\,in^2 > A_s = 0.263\,in^2 \quad (GOOD)$$

- VERTICAL COLUMN TIES

$$R_v = A_T(D+L) = (28.67')(14.33')(110psf+40psf) = 123.3k$$

$$\Rightarrow A_s = \frac{123.3k}{(0.75)(1.25)(60ksi)} = 2.19\,in^2$$

$$A_{s\,PROVIDED} = 6\,in^2 > A_s = 2.19\,in^2 \quad (GOOD)$$

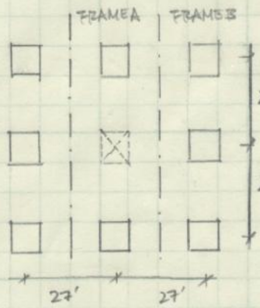
- CORNER COLUMN TIES

$$R_u \geq \begin{cases} 0.03[4(D+L)_{VERT}]A_{T\,VERT} = 0.03[4(500psf)](28.67')(70.5') = 121.3k \ \text{*CRITICAL} \\ R_u' \leq \begin{cases} 2.0T_t = 19.8k \\ \frac{l_s}{8.2}T_t = 13.9k \end{cases} \end{cases}$$

$$\Rightarrow A_s = \frac{121.3k}{(0.75)(1.25)(60ksi)} = 2.16\,in^2$$

$$A_{s\,PROVIDED} = 6\,in^2 > A_s = 2.16\,in^2 \quad (GOOD)$$

SLAB DESIGN: 2 WAY (INT)



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

NORMAL WEIGHT CONCRETE

24 x 24 COLUMNS (TYPICAL)

INTERIOR SLAB.

- SLAB THICKNESS CHECK.

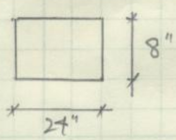
$$t_{MIN} = \frac{l_n}{30}$$

$$\Rightarrow l_n = 27' - {}^{24''}/_{12''/ft} = 25'$$

$$t_{MIN} = \frac{(25')(12''/ft)}{30} = 10'' > t_{EXIST} = 8''$$

$$\Rightarrow \text{ACI } 318\text{-}08 \ \S\ 9.5.3.2(b): t_{MIN} = 4'' < 8'' \ \therefore \text{ USE } t = 8''$$

- TORSIONAL CONSTANT



$$C = \sum \left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right)$$

$$= \left[1 - 0.63\left(\frac{8''}{24''}\right)\right]\left[\frac{(8'')^3(24'')}{3}\right]$$

$$\therefore C = 3235.84 \text{ in}^3$$

- PARAMETERS

- FRAME A.

$$I_s^{A_4} = \frac{(25')(12''/ft)(8'')^3}{12} = 12800 \text{ in}^4$$

$$\beta_t = \frac{C}{2 I_s^{A_4}} = \frac{3235.84 \text{ in}^3}{2(12800 \text{ in}^4)} = 0.126.$$

→ CONT'D.

SLAB DESIGN: 2 WAY (CONT)

- PARAMETERS CONT'D

- FRAME A

$\alpha = 0$ (NO BEAMS)

$\frac{l_2}{l_1} = \frac{27'}{2(27')} = 0.5$

$\alpha \frac{l_2}{l_1} = 0.$

- FRAME B

$I_s^{B6} = 17800 \, in^4$

$\beta_t = 0.126$

$\alpha = 0.$

$\frac{l_2}{l_1} = 1.0$

$\alpha \frac{l_2}{l_1} = 0.$

- TOTAL FACTORED MOMENTS AND REDISTRIBUTIONS.

- FRAME A.

$M_o = \frac{w_u l_2 l_1^2 (1 - 2c/3l_1)^2}{8}$

$= \frac{(152 \, PSF)(27')(54')^2 (1 - 2(24")/3(54')(12"/ft))^2}{8}$

$\therefore M_o = 1423 \, k \cdot ft$

$\Rightarrow M^+ = 0.35 M_o = 498 \, k \cdot ft$

$M^- = 0.65 M_o = 925 \, k \cdot ft$

- FRAME B

$M_o = \frac{(152 \, PSF)(27')(27')^2 (1 - 2(24")/3(27')(12"/ft))^2}{8}$

$\therefore M_o = 338 \, k \cdot ft$

→ CONT'D.

SLAB DESIGN : 2 WAY (INT)

- TOTAL FACTORED MOMENTS AND REDISTRIBUTIONS. CONT'D.
  - FRAME B
    - $\Rightarrow M^{+} = 0.35 M_o = 118 k \cdot ft$
    - $M^{-} = 0.65 M_o = 220 k \cdot ft.$
- DISTRIBUTION OF MOMENTS
  - FRAME A.
    - INTERIOR NEGATIVE MOMENTS

$\alpha\, l_2/l_1 = 0$	$l_2/l_1 = 0.5$
	75%.

925 k·ft → 75% TO CS (693.8 k·ft)

925 k·ft → 25% TO MS (231.3 k·ft)

    - POSITIVE MOMENTS

$\alpha\, l_2/l_1 = 0$	$l_2/l_1 = 0.5$
	60%.

498 k·ft → 60% TO CS (299 k·ft)

498 k·ft → 40% TO MS (199 k·ft)

  - FRAME B
    - INTERIOR NEGATIVE MOMENTS

$\alpha\, l_2/l_1 = 0$	$l_2/l_1 = 1.0$
	75%

220 k·ft → 75% TO CS (165 k·ft)

220 k·ft → 25% TO MS (55 k·ft)

→ CONT'D.

SLAB DESIGN: 2 WAY (INT)

- DISTRIBUTION OF MOMENTS CONT'D
  - FRAME B
    - POSITIVE MOMENTS

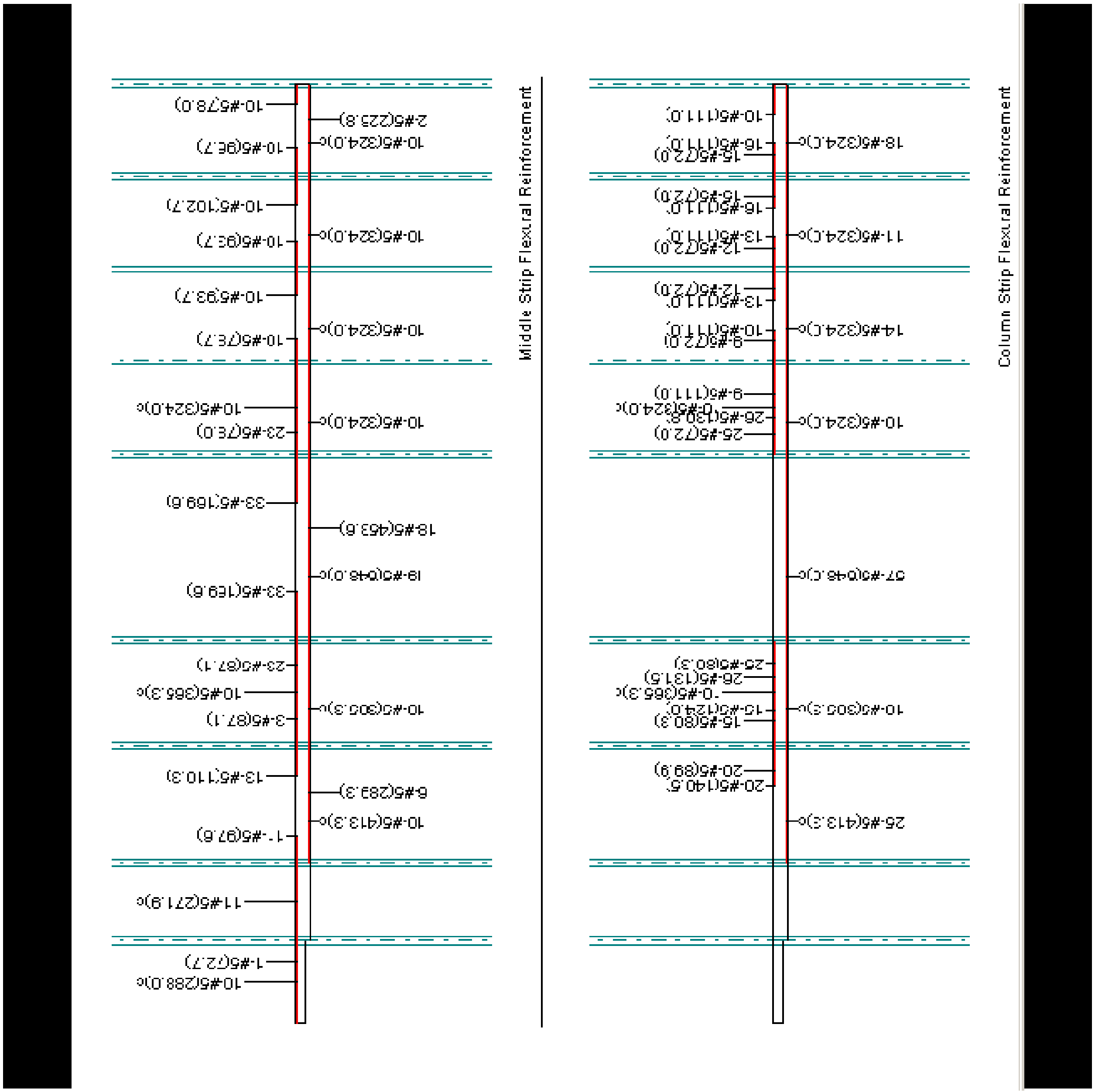
	$l_2/l_1 = 1.0$
$\alpha\, l_2/l_1 = 0$	60%

118 k·ft → 60% TO CS (71 k·ft)

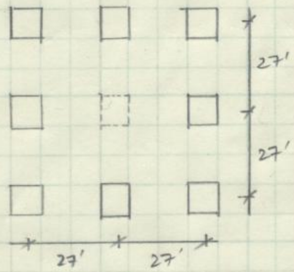
118 k·ft → 40% TO MS (47 k·ft)

- DESIGN
  - SEE FOLLOWING SPREADSHEET.

FRAME A	**CS**				**FRAME B**	**CS**		
Item	*Description*	*Interior Span*			*Item*	*Description*	*Interior Span*	
		M^-	M^+				M^-	M^+
1	M_n	693.75	299		1	M_n	165	71
2	b_{CS}	162	162		2	b_{CS}	81	81
3	d_{eff}	6.31	6.31		3	d_{eff}	6.31	6.31
4	$M_u = M_n/\varphi$	991.0714	427.1429		4	$M_u = M_n/\varphi$	235.7143	101.4286
5	$M_n(12/b)$	51.38889	22.14815		5	$M_n(12/b)$	24.44444	10.51852
6	$R = M_u/bd^2$	1843.794	794.6587		6	$R = M_u/bd^2$	877.0481	377.3965
7	ρ	0.02	0.0146		7	ρ	0.0163	0.0066
8	$A_{steel} = \rho bd$	20.4444	14.92441		8	$A_{steel} = \rho bd$	8.331093	3.373326
9	$A_{s,min} = 0.002bt$	2.592	2.592		9	$A_{s,min} = 0.002bt$	1.296	1.296
10	$N = A_s/A$	33	19		10	$N = A_s/A$	33	18
11	$N_{min} = w_{strip}/2t$	10	10		11	$N_{min} = w_{strip}/2t$	5	5
FRAME A	**MS**				**FRAME B**	**MS**		
Item	*Description*	*Interior Span*			*Item*	*Description*	*Interior Span*	
		M^-	M^+				M^-	M^+
1	M_n	231	199		1	M_n	55	47
2	b_{CS}	81	81		2	b_{CS}	40.5	40.5
3	d_{eff}	6.93	6.93		3	d_{eff}	6.93	6.93
4	$M_u = M_n/\varphi$	330	284.2857		4	$M_u = M_n/\varphi$	78.57143	67.14286
5	$M_n(12/b)$	34.22222	29.48148		5	$M_n(12/b)$	16.2963	13.92593
6	$R = M_u/bd^2$	1017.99	876.9701		6	$R = M_u/bd^2$	484.7574	414.2472
7	ρ	0.0197	0.0162		7	ρ	0.0085	0.0073
8	$A_{steel} = \rho bd$	11.0582	9.093546		8	$A_{steel} = \rho bd$	2.385653	2.048855
9	$A_{s,min} = 0.002bt$	1.296	1.296		9	$A_{s,min} = 0.002bt$	0.648	0.648
10	$N = A_s/A$	57	25		10	$N = A_s/A$	25	10
11	$N_{min} = w_{strip}/2t$	5	5		11	$N_{min} = w_{strip}/2t$	3	3


Middle Strip Flexural Reinforcement
Column Strip Flexural Reinforcement

SLAB DESIGN : PT SLAB (INT)



$f'_c = 5000psi$ $f'_{ci} = 3000psi$
$f_y = 60000psi$ $1/2"\phi$ 7 WIRE STRANDS $f_{pu} = 270ksi$

24 × 24 COLUMN (TYPICAL)

- POST-TENSIONING

ESTIMATED PRE-STRESS LOSSES = 15ksi

$f_{SE} = 0.7 f_{pu} - \text{LOSSES}$

$= 0.7(270ksi) - 15ksi$

$\therefore f_{SE} = 174ksi$

$P_{EFF} = A_p \times f_{se}$

$= (0.153in^2)(174ksi)$

$\therefore P_{EFF} = 26.6k/\text{TENDON}$.

- PRIMARY SLAB THICKNESS

ASSUME 10"

- SECTION PROPERTIES

$A_c = b \times h$

$= (54')(12"/ft)(10")$

$\therefore A_c = 6480in^2$

$S = \dfrac{bh^2}{6}$

$= \dfrac{(6480in^2)(10")}{6}$

$\therefore S = 10800in^3$

→ CONT'D.

SLAB DESIGN: PT SLAB (INT)

- DESIGN PARAMETERS

ASSUMING CLASS U

- JACKING

$f'_{ci} = 3000 \text{ psi}$

COMPRESSION $= 0.6f'_{ci} = 0.6(3000 \text{ psi}) = 1800 \text{ psi}$

TENSION $= 3\sqrt{f'_{ci}} = 3\sqrt{3000 \text{psi}} = 164 \text{ psi}$

- SERVICE

$f'_c = 5000 \text{psi}$

COMPRESSION $= 0.45f'_c = 0.45(5000 \text{psi}) = 2250 \text{psi}$

TENSION $= 6\sqrt{f'_c} = 6\sqrt{5000 \text{psi}} = 424 \text{ psi}$

- TARGET LOAD BALANCE

$0.75 w_{DL} = 0.75(110 \text{PSF}) = 82.5 \text{psf}$

- TENDON PROFILE

$a_{INT} = 9''$

$a_{END} = 5.25''$

- PRESTRESS FORCE REQUIRED

$w_B = 0.75 w_{DL,S} = \dfrac{(0.75)(145 \text{PCF})(10'')(54')}{12''/\text{ft}} = 4.9 \text{ k/ft}$

$P = \dfrac{w_B l^2}{8 a_{END}}$

$= \dfrac{(4.9 \text{k/ft})(54')^2}{(8)(5.25''/12''/\text{ft})}$

$\therefore P = 1020 \text{k}$

→ CONT'D.

SLAB DESIGN: PT SLAB (INT)

- PRECOMPRESSION ALLOWANCE

  - NUMBER OF TENDONS

    $\frac{1020k}{26.6k/\text{TENDON}} = 38.3$ TENDONS => USE 39 TENDONS.

  - ACTUAL FORCE FOR BONDED TENDONS

    $P_{ACT} = (39\ \text{TENDONS})(26.6k/\text{TENDON}) = 1037k$

  - BALANCED LOAD

    $W_{BAL} = \frac{1037k}{1020k}(4.9k/ft)$

    $\therefore W_{BAL} = 5k/ft.$

  - ACTUAL PRECOMPRESSION STRESS

    $\frac{P_{ACT}}{A_c} = \frac{1037k}{6480in^2} = 160\ psi > 125\ psi$ (GOOD)

    $< 300\ psi$ (GOOD)

- INTERIOR SPAN FORCE

  $P = \frac{(4.9k/ft)(54')^2}{8(5.25''/12''/ft)}$

  $\therefore P = 4082.4k.$

  $W_B = \frac{(4082.4k)8(5.25'')}{(54')^2(12''/ft)}$

  $\therefore W_B = 4.89k/ft.$

  $\frac{W_B}{W_{DL}} = \frac{4.89k/ft}{6.5k/ft} = 74\% < 100\%$ (ACCEPTABLE)

  $\therefore P_{EFF} = 4082.4k.$

→CONT'D.

SLAB DESIGN: PT SLAB (INT)

- SLAB STRESSES. (ASSUME SIMPLY SUPPORTED ; 54').
  - DEAD LOAD

$$M = \frac{(3k/ft)(54')^2}{8}$$

$\therefore M = 1094\ k\cdot ft.$

  - LIVE LOAD

$$M = \frac{(2.16k/ft)(54')^2}{8}$$

$\therefore M = 787\ k\cdot ft$

  - BALANCED LOAD

$$M = \frac{(2.58k/ft)(54')^2}{8}$$

$\therefore M = -940\ k\cdot ft.$

- STRESSES AT TRANSFER

$$f_{TOP} = \left(\frac{-1094k\cdot ft + 940k\cdot ft}{10800\ in^3}\right)(12''/ft)(1000lb/k) - 160psi = -331psi < 1800psi\ (GOOD)$$

$$f_{BOT} = \left(\frac{1094k\cdot ft - 940k\cdot ft}{10800\ in^3}\right)(12''/ft)(1000lb/k) - 160psi = 11psi < 164psi\ (GOOD)$$

- STRESSES AT SERVICE.

$$f_{TOP} = \left(\frac{-1094k\cdot ft - 787k\cdot ft + 940k\cdot ft}{10800\ in^3}\right)(12''/ft)(1000lb/k) - 160\ psi = -1205\ psi < 2250\ psi\ (GOOD)$$

$$f_{BOT} = \left(\frac{1094k\cdot ft + 787k\cdot ft - 940k\cdot ft}{10800\ in^3}\right)(12''/ft)(1000lb/k) - 160psi = 430\ psi > 424\ psi$$

(WITHIN 5% GOOD)

→ CONT'D.

SLAB DESIGN: PT SLAB (INT)

- ULTIMATE STRENGTH DESIGN

- INTERIOR SPAN

$f_t = 430psi > 2\sqrt{f'c} = 2\sqrt{5000psi} = 141 psi$ (REINFORCEMENT REQUIRED)

- MINIMUM POSITIVE MOMENT REINFORCEMENT REQUIRED.

$y = \left(\frac{430psi}{430psi + 1205psi}\right)(10")$

$\therefore y = 2.6"$

$N_c = \left[\frac{(1094k \cdot ft + 787 k \cdot ft)(12"/ft)}{10800 in^3}\right](\frac{1}{2})(2.6")(54')(12"/ft)$

$\therefore N_c = 1761 k.$

$As_{MIN} = \frac{1761k}{(0.5)(60ksi)}$

$\therefore As_{MIN} = 58 in^2$

$\Rightarrow As_{MIN} = \frac{58 in^2}{54'}$

$\therefore As_{MIN} = 1.09 in^2/ft.$

$\Rightarrow$ USE #10 @ 12" O.C.

- CHECK MINIMUM REINFORCEMENT.

$A_{ps} = (0.153 in^2)(39 \text{ TENDONS}) = 5.97 in^2$

$f_{ps} = 174000 psi + 10000 + \frac{(5000psi)(54')(12"/ft)(10")}{300(5.97in^2)}$

$\therefore f_{ps} = 202 ksi$

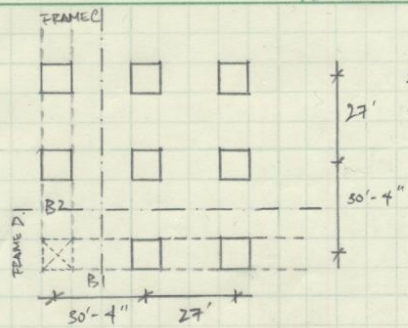
$a = \frac{(58in^2)(60ksi) + (5.97in^2)(202ksi)}{0.85(5ksi)(54')(12"/ft)}$

$\therefore a = 1.7"$

$\phi M_n = [(58in^2)(60ksi) + (5.97in^2)(202ksi)](10" - \frac{1.7"}{2})(0.9)$

$\therefore \phi M_n = 3215 k \cdot ft > 1881 k \cdot ft$ (GOOD)

BEAM DESIGN/SLAB DESIGN.



$f'_c = 5000 psi$ EDGE BEAMS: 24" × 18" (TYPICAL)
$f_y = 60000 psi$ COLUMNS: 24" × 24" (TYPICAL)

- FROM SAP 2000:

$M_{B1} = 555 k \cdot ft$

$M_{B2} = 315 k \cdot ft.$

=> THESE MOMENTS WILL BE USED TO DESIGN THE SLAB AND THE BEAM. THE GREATER MOMENT WILL CONTROL AND USED FOR ANALYSIS.

- CHECK BEAM B1

$$\rho_{MAX\phi} = 0.85\beta_1 \frac{f'_c}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + 0.005}$$

$$= 0.85(0.85) \frac{5000 psi}{60000 psi} \cdot \frac{0.003}{0.003 + 0.005}$$

$$\therefore \rho_{MAX\phi} = 0.0226.$$

$$\Rightarrow A_{s1} = 0.226(24")(14") = 7.59 in^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$= \frac{(7.59 in^2)(60 ksi)}{0.85(5 ksi)(24")}$$

$$\therefore a = 4.46"$$

$$c = \frac{a}{\beta_1}$$

$$= \frac{4.46"}{0.85}$$

$$\therefore c = 5.25"$$

→ CONT'D.

BEAM DESIGN / SLAB DESIGN

- CHECK BEAM B1 CONT'D.

$M_{N1} = A_s f_y \left(d - \frac{a}{2}\right)$

$= (7.59 in^2)(60 ksi)\left(14" - \frac{4.46"}{2}\right)$

$\therefore M_{N1} = 446 k \cdot ft.$

$M_{N2} = \frac{M_u}{\phi} - M_{N1}$

$= \frac{555 k \cdot ft}{0.9} - 446 k \cdot ft$

$\therefore M_{N2} = 170 k \cdot ft$

$A_{s2} = \frac{M_{N2}}{f_y (d - d')}$

$= \frac{(170 k \cdot ft)(12"/ft)}{(60 ksi)(14" - 4")}$

$\therefore A_{s2} = 3.4 in^2$

$A_s = A_{s1} + A_{s2}$

$= 7.59 in^2 + 3.4 in^2$

$\therefore A_s = 10.99 in^2$

$f'_s = \varepsilon_u \frac{c - d'}{c} E_s.$

$= 0.003 \left(\frac{5.25" - 4"}{5.25"}\right) 29000 ksi$

$\therefore f'_s = 20.7 ksi.$

$A'_s = A_{s2} \frac{f_y}{f'_s}$

$= (3.4 in^2)\left(\frac{60 ksi}{20.7 ksi}\right)$

$\therefore A'_s = 9.89 in^2$

$\therefore A_s = (9)\#10 \quad (11.43 in^2)$
$A'_s = (10)\#9 \quad (10 in^2)$

→ CONT'D.

BEAM DESIGN / SLAB DESIGN

- CHECK YIELDING AND DESIGN MOMENT CAPACITY.

$\rho_{cy} = 0.85(0.85)\left(\frac{5\,ksi}{60\,ksi}\right)\left(\frac{4''}{14''}\right)\left(\frac{0.003}{0.001}\right) = 5.16\%$

$\rho = \frac{11.43\,in^2}{(24'')(18'')} = 2.65\% < \rho_{cy} = 5.16\%$ => DOES NOT YIELD : CASE II

=> ASSUME $f_s > f_y$; $c = ?$

$0.85 f'_c b \beta_1 c + A'_s f'_s = A_s f_y$

$0.85(5\,ksi)(0.85)c^2 + ((10\,in^2)(0.003)(29000\,ksi))c - ((10\,in^2)(0.003)(29000\,ksi))(4'') = (11.43\,in^2)(60\,ksi)\,c$

$\therefore c = 14.7''$

$a = \beta_1 c$

$= (0.85)(14.7'')$

$\therefore a = 12.5''$

$\varepsilon'_s = \frac{0.003}{14.7''}(14.7'' - 4'') = 0.0021 < \varepsilon_y$ (GOOD)

$\varepsilon_s = \frac{0.003}{14.7''}(14.7'' - 14'') = 0.0014$

$M_n = (10\,in^2)(0.0021)(29000\,ksi)(14'' - 4'') + 0.85(5\,ksi)(24'')(12.5'')\left(14'' - \frac{12.5''}{2}\right)$

$\therefore M_n = 1533\,k\cdot ft$

$\phi M_n = (0.9)(1533\,k\cdot ft)$

$\therefore \phi M_n = 1380\,k\cdot ft > 555\,k\cdot ft$ (GOOD)

BEAM DESIGN/SLAB DESIGN

- FROM SAP 2000:

$M_{o_C} = 1436$ k·ft

$M_{o_D} = 2530$ k·ft

=> THESE MOMENTS WILL BE USED TO DESIGN THE 2 WAY SLAB.

- DISTRIBUTION OF MOMENTS

- FRAME C

$M_o = 1436$ k·ft

$M^-_{EXT} = 0.3M_o = 430.1$ k·ft

$M^+_{INT} = 0.5M_o = 718$ k·ft

$M^-_{INT} = 0.7M_o = 1005$ k·ft.

- FRAME D

$M_o = 2530$ k·ft

$M^-_{EXT} = 0.3M_o = 759$ k·ft

$M^+_{INT} = 0.5M_o = 1265$ k·ft

$M^-_{INT} = 0.7M_o = 1771$ k·ft

- FRAME PARAMETERS (VALUES TAKEN FROM TECH 2)

$I_s^{\beta_t} = 7762.25 \text{ in}^4$

$\beta_t = 1.4$

$\alpha = 1.77$

$\frac{l_2}{l_1} = 1.0$

$\alpha \frac{l_2}{l_1} = 1.77$

→ CONT'D.

BEAM DESIGN/SLAB DESIGN

- DISTRIBUTION OF MOMENTS

- EXTERIOR NEGATIVE MOMENTS

		l_2/l_1	1.0
$\alpha\, l_2/l_1 > 1.0$		$\beta = 0$	100
		$\beta = 1.4$	x
		$\beta = 2.5$	75

$\Rightarrow$ INTERPOLATING: $x = 86\%$

- FRAME C

430.1 k·ft $\rightarrow$ 86% TO CS (370.5 k·ft)

$\rightarrow$ 14% TO MS (60 k·ft)

- FRAME D

759 k·ft $\rightarrow$ 86% TO CS (653 k·ft)

$\rightarrow$ 14% TO MS (106 k·ft)

- INTERIOR POSITIVE MOMENTS.

	$l_2/l_1 = 1.0$
$\alpha\, l_2/l_1 > 1.0$	75%

- FRAME C

718 k·ft $\rightarrow$ 75% TO CS (538.5 k·ft)

$\rightarrow$ 25% TO MS (179.5 k·ft)

- FRAME D

1265 k·ft $\rightarrow$ 75% TO CS (948.8 k·ft)

$\rightarrow$ 25% TO MS (316 k·ft)

$\rightarrow$ CONT'D.

BEAM DESIGN/SLAB DESIGN

- DISTRIBUTION OF MOMENTS CONT'D.
  - INTERIOR NEGATIVE MOMENTS

	$l_2/l_1 = 1.0$
$\alpha\, l_2/l_1 > 1.0$	75%

  - FRAME C

    1005 k·ft → 75% TO CS (753.8 k·ft)

    1005 k·ft → 25% TO MS (251.3 k·ft)

  - FRAME D

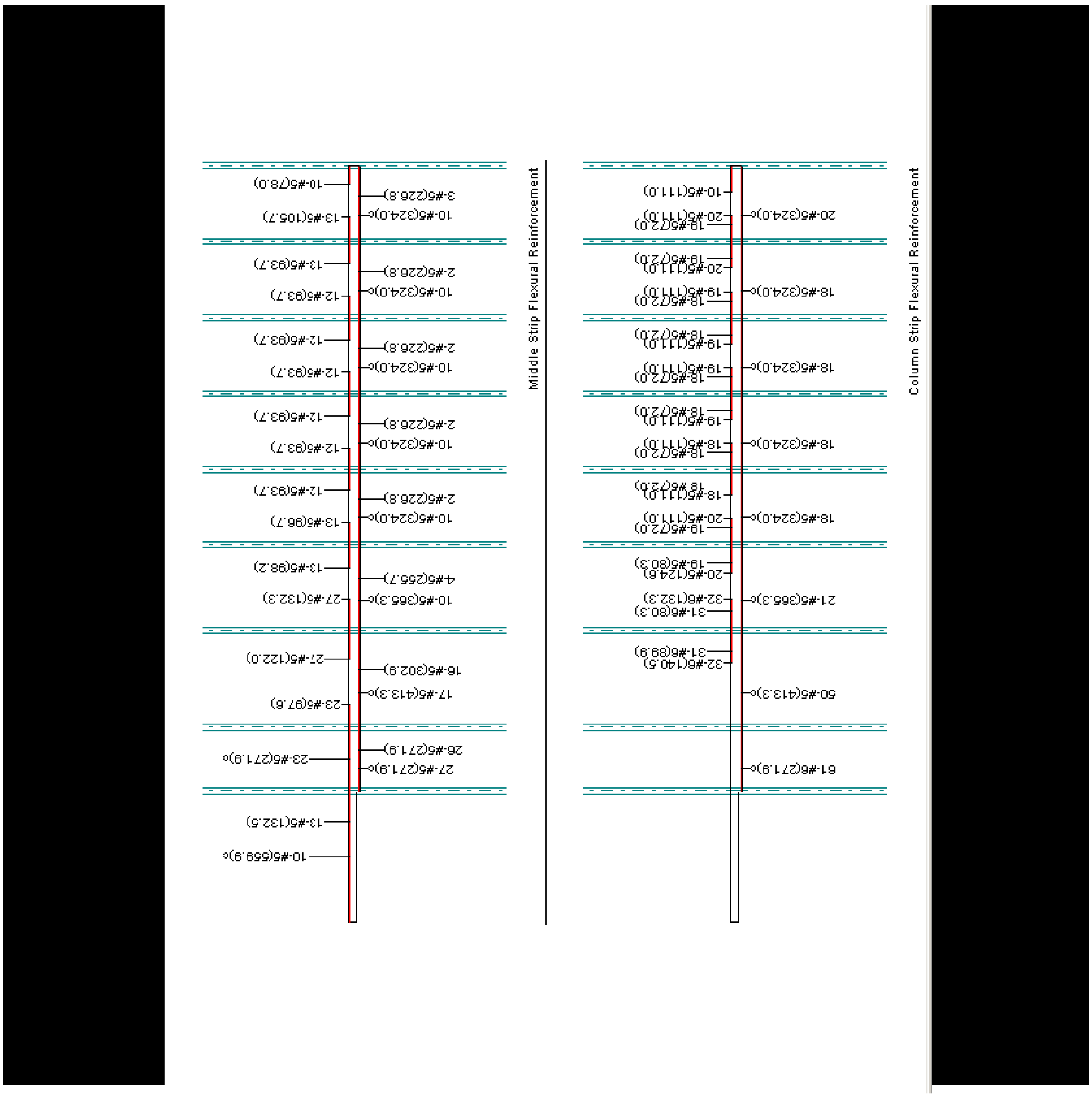
    1771 k·ft → 75% TO CS (1328 k·ft)

    1771 k·ft → 25% TO MS (442.8 k·ft)

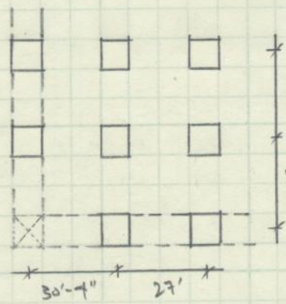
- DESIGN
  - SEE FOLLOWING SPREADSHEET.

FRAME C	**CS**			
Item	*Description*	*Exterior Span*		
		M^{-}_{EXT}	M^{+}_{INT}	M^{-}_{INT}
1	M_n	370.5	538.5	753.75
2	b_{CS}	91	91	91
3	d_{eff}	6.31	6.31	6.31
4	$M_u = M_n/\varphi$	529.285714	769.28571	1076.7857
5	$M_n(12/b)$	48.8571429	71.010989	99.395604
6	$R = M_u/bd^2$	1752.95718	2547.8204	3566.2388
7	ρ	0.02	0.02	0.02
8	$A_{steel} = \rho bd$	11.4842	11.4842	11.4842
9	$A_{s,min} = 0.002bt$	1.456	1.456	1.456
10	$N = A_s/A$	32	50	61
11	$N_{min} = w_{strip}/2t$	6	6	6
FRAME C	**MS**			
Item	*Description*	*Exterior Span*		
		M^{-}_{EXT}	M^{+}_{INT}	M^{-}_{INT}
1	M_n	370.5	538.5	753.75
2	b_{CS}	45.5	45.5	45.5
3	d_{eff}	6.93	6.93	6.93
4	$M_u = M_n/\varphi$	529.285714	769.28571	1076.7857
5	$M_n(12/b)$	97.7142857	142.02198	198.79121
6	$R = M_u/bd^2$	2906.65544	4224.653	5913.3375
7	ρ	0.02	0.02	0.02
8	$A_{steel} = \rho bd$	6.3063	6.3063	6.3063
9	$A_{s,min} = 0.002bt$	0.728	0.728	0.728
10	$N = A_s/A$	10	13	27
11	$N_{min} = w_{strip}/2t$	3	3	3

FRAME D	**CS**			
Item	Description	Exterior Span		
		M^-_{EXT}	M^+_{INT}	M^-_{INT}
1	M_n	653	948	1328
2	b_{CS}	91	91	91
3	d_{eff}	6.31	6.31	6.31
4	$M_u = M_n/\varphi$	932.85714	1354.2857	1897.1429
5	$M_n(12/b)$	86.10989	125.01099	175.12088
6	$R = M_u/bd^2$	3089.5575	4485.2993	6283.2041
7	ρ	0.02	0.02	0.02
8	$A_{steel} = \rho bd$	11.4842	11.4842	11.4842
9	$A_{s,min} = 0.002bt$	1.456	1.456	1.456
10	$N = A_s/A$	31	31	20
11	$N_{min} = w_{strip}/2t$	6	6	6
FRAME D	**MS**			
Item	*Description*	*Exterior Span*		
		M^-_{EXT}	M^+_{INT}	M^-_{INT}
1	M_n	106	316.25	442.75
2	b_{CS}	45.5	45.5	45.5
3	d_{eff}	6.93	6.93	6.93
4	$M_u = M_n/\varphi$	151.42857	451.78571	632.5
5	$M_n(12/b)$	27.956044	83.406593	116.76923
6	$R = M_u/bd^2$	831.59373	2481.052	3473.4729
7	ρ	0.02	0.02	0.02
8	$A_{steel} = \rho bd$	6.3063	6.3063	6.3063
9	$A_{s,min} = 0.002bt$	0.728	0.728	0.728
10	$N = A_s/A$	23	27	16
11	$N_{min} = w_{strip}/2t$	3	3	3


Middle Strip Flexural Reinforcement
Column Strip Flexural Reinforcement

SLAB DESIGN: PT SLAB (EXT)



f'_c = 5000 psi f'_{ci} = 3000 psi
f_y = 60000 psi 1/2 ϕ 7 WIRE STRANDS
f_{pu} = 270 ksi

24" x 24" COLUMN (TYPICAL)

- POST-TENSIONING

  ESTIMATED PRE-STRESS LOSSES = 15 ksi

  f_{SE} = 174 ksi

  P_{EFF} = 26.6 k/TENDON.

- THICKNESS

  ASSUME 10"

- SECTION PROPERTIES

  A = 3458 in^2

  S = 5763 in^3

- DESIGN PARAMETERS

  SEE SLAB DESIGN: PT (INT)

- TARGET LOAD BALANCE

  $0.75 w_{DL}$ = 82.5 psf.

- TENDON PROFILE

  a_{INT} = 9"

  a_{END} = 5.25"

- PRESTRESS FORCE REQUIRED.

$$w_B = 0.75 w_{DLS} = \frac{(0.75)(145\text{ PCF})(10")(30.33')}{12"/ft} = 2.75\text{ k/ft}$$

→ CONT'D.

SLAB DESIGN: PT SLAB (EXT)

- PRESTRESS FORCE REQUIRED CONT'D.

$$P = \frac{w_B l^2}{8 a_{END}} = \frac{(2.75k/ft)(30.33')^2}{8(5.25''/(12''/ft))} = 722.6k$$

- PRECOMPRESSION ALLOWANCE

- NUMBER OF TENDONS

$$\frac{722.6k}{26.6k/TENDON} = 27 \text{ TENDONS} \Rightarrow \text{CONSERVATIVELY USE 30 TENDONS}$$

- ACTUAL FORCE FOR BONDED TENDONS

$$P_{ACT} = (30 \text{ TENDONS})(26.6k/\text{TENDONS}) = 798k.$$

- BALANCED LOAD

$$w_{BAL} = \frac{798k}{722.6k}(2.75k/ft)$$

- ACTUAL PRECOMPRESSION STRESS.

$$\frac{P_{ACT}}{A} = \frac{798k}{3458 in^2} = 230 psi > 125 psi \text{ (GOOD)}$$

$$< 300 psi \text{ (GOOD)}$$

- SPAN FORCE

$$P = \frac{(2.75k/ft)(30.33')^2}{8(5.25''/(12''/ft))} = 722k$$

$$w_B = \frac{(722k)(8)(5.25'')}{(30.33')^2(12''/ft)} = 2.74 k/ft$$

$$\frac{w_B}{w_{DL}} = \frac{2.74 k/ft}{3.67k/ft} = 74\%.$$

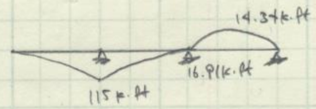
$$\therefore P_{EXT} = 722.6k.$$

→ CONT'D.

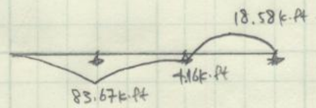
SLAB DESIGN : PT SLAB (EXT)

- SLAB STRESSES

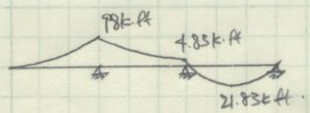
  - DEAD LOAD



  - LIVE LOAD.



  - BALANCED LOAD



- STRESSES AT TRANSFER (CRITICAL MOMENTS CHECKED)

$$f_{TOP} = \left(\frac{-115k\cdot ft + 98k\cdot ft}{5763 in^3}\right)(12"/ft)(1000 lb/k) - 230 psi = -265 psi < 1800 psi \text{ (GOOD)}$$

$$f_{BOT} = \left(\frac{115k\cdot ft - 98k\cdot ft}{5763 in^3}\right)(12"/ft)(1000 lb/k) - 230 psi = -194.6 psi < 1800 psi \text{ (GOOD)}$$

- STRESSES AT SERVICE (CRITICAL MOMENTS CHECKED)

$$f_{TOP} = \left(\frac{-115k\cdot ft - 83.67k\cdot ft + 98k\cdot ft}{5763 in^3}\right)(12"/ft)(1000 lb/k) - 230 psi = -439 psi < 2250 psi \text{ (GOOD)}$$

$$f_{BOT} = \left(\frac{115k\cdot ft + 83.67k\cdot ft - 98k\cdot ft}{5763 in^3}\right)(12"/ft)(1000 lb/k) - 230 psi = -20.38 psi < 2250 psi \text{ (GOOD)}$$

- ULTIMATE STRENGTH DESIGN

$f_t = -450 psi < 2\sqrt{f'c} = 141$ psi (NO REINFORCEMENT REQUIRED)

CURTAIN WALL DESIGN

- GLASS DETAILS (ONE PANEL)

  1/4" THICK, HEAT STRENTHENED (GTF = 1.8); 1 LITE (LSF = 1.0)

  w = 5.5' (1676.4 mm)
  h = 2.7' (822.3 mm)
  } => NFL = 54.3 psf (2.6 kPa) (ASTM E1300-04)

- DETERMINE 3-s EQUIVALENT DESIGN LOAD. AT 75' AND 100 lb TNT

  STANDOFF DISTANCE = 75' (23.5 m) (ASSUMED)

  EQUIVALENT CHARGE = 100 lb TNT (45.3 kg TNT) (ASSUMED)

  3-s EQUIVALENT DESIGN LOAD = 98 psf (4.5 kPa) (ASTM F2248-03 FIG. 3)

- DETERMINING LOAD RESISTANCE AT 75' AND 100 lb TNT

  LR = NFL × ADJUSTMENT FACTORS

  = NFL × GTF × LSF

  = (54.3 psf)(1.8)(1.0)

∴ LR = 98 psf = 3-s EQUIVALENT DESIGN LOAD (OK)

=> THIS WILL CAUSE THE GLASS TO SHATTER, ALTERNATIVE GLASS TYPES MUST BE INSPECTED

=> ALTERNATIVES:

1) USE 2 LITE; GLASS WILL LOAD SHARE (LSF = 2.0)

   LR = 195 psf

2) USE FULLY TEMPERED GLASS (GTF = 4.0); 1 LITE

   LR = 217.2 psf

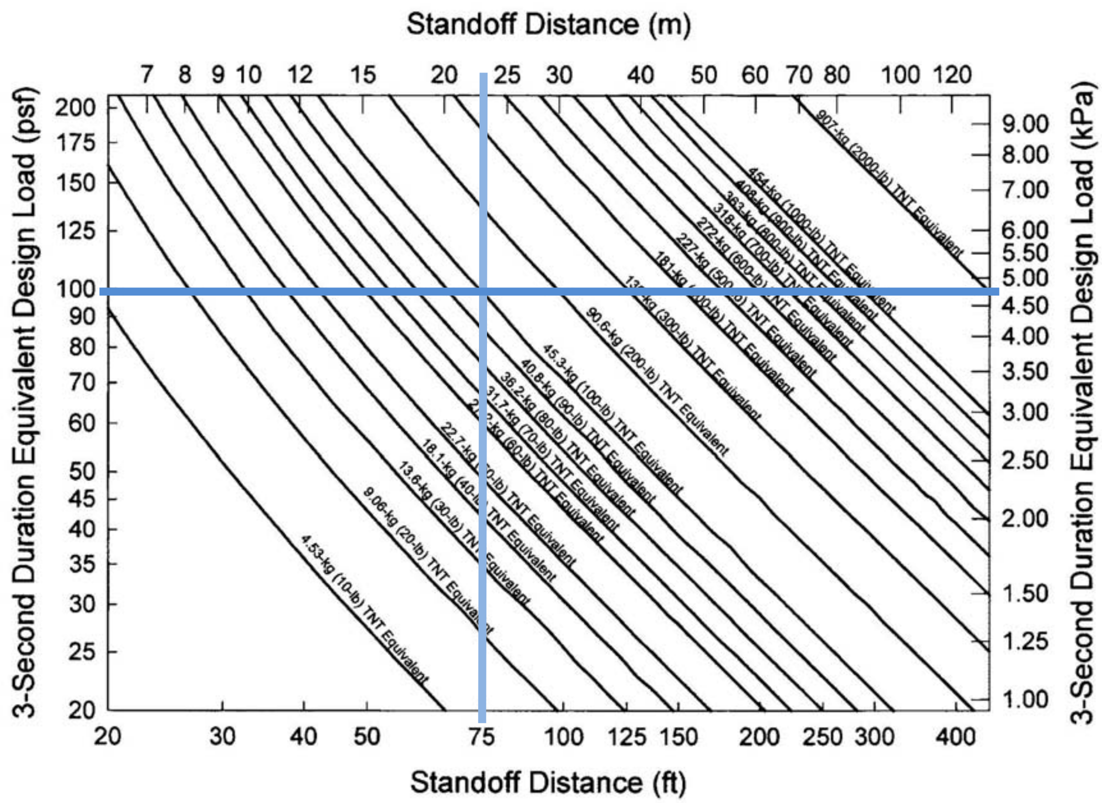
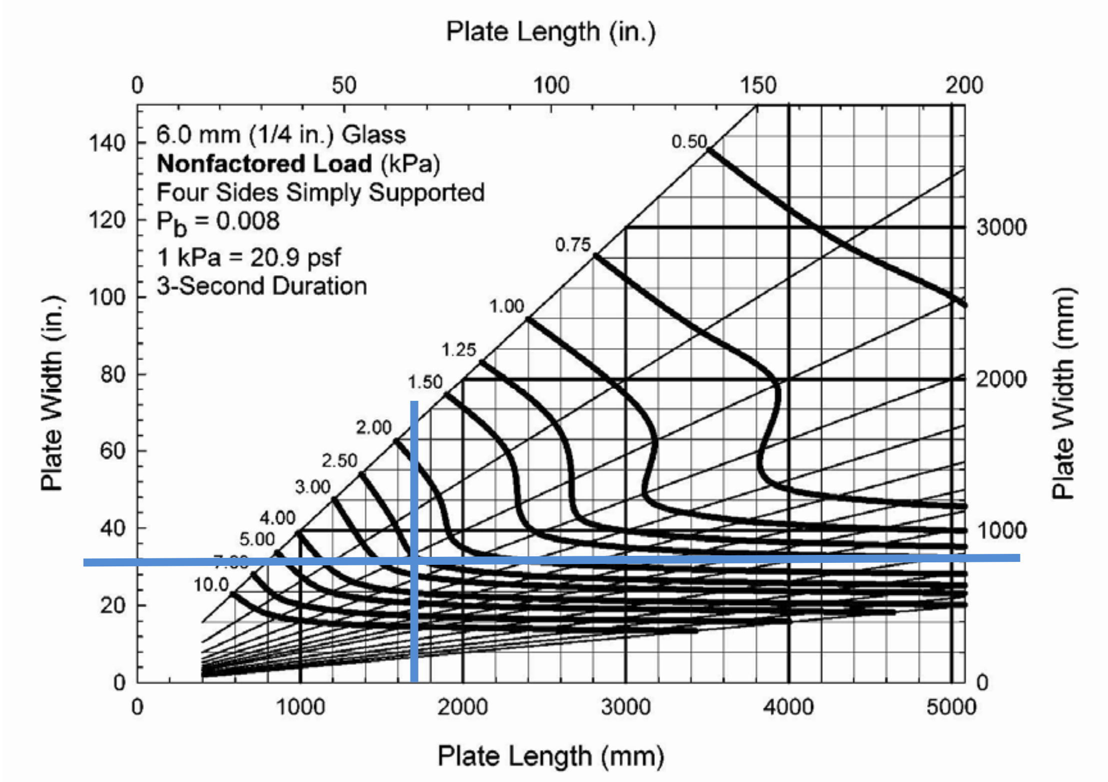


Fig. 3. Chart that relates standoff distance and charge size to equivalent 3-s duration equivalent design loading from ASTM F 2248 (Reprinted with permission from ASTM F 2248-03, copyright ASTM International, 100 Barr Harbor Dr., West Conshohocken, PA 19428.)



Conductive Properties of Heat Strengthened, 1 Lite Curtain Wall

Layer	*Conductivity, k (W/m*K)*	*Thickness (m)*	*Thickness (in)*	*Conductance, C (W/m²*K)*		*Resistance, R (m²*K/W)*	
				Summer	*Winter*	*Summer*	*Winter*
Exterior Air Film	N/A	N/A		23.00	34.00	0.0435	0.0294
Glass Lite 1	0.96	0.0064	0.25	151.18		0.0066	
Interior Air Film	N/A	N/A		8.30		0.1205	
					$\sum R_{SI}$	0.17	0.16
					$\sum R$ (hr*ft²*°F/BTU)	0.97	0.89
					U (BTU/hr*ft2*°F)	1.03	1.13

Conductive Properties of Heat Strengthened, 2 Lite Curtain Wall

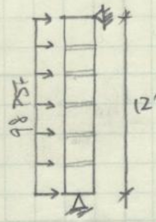
Layer	*Conductivity, k (W/m*K)*	*Thickness (m)*	*Thickness (in)*	*Conductance, C (W/m²*K)*		*Resistance, R (m²*K/W)*	
				Summer	*Winter*	*Summer*	*Winter*
Exterior Air Film	N/A	N/A		23.00	34.00	0.0435	0.0294
Glass Lite 1	0.96	0.0064	0.25	151.18		0.0066	
Air Space	N/A	0.0127	0.5	7.14	5.00	0.1401	0.2000
Glass Lite 2	0.96	0.0064	0.25	151.18		0.0066	
Interior Air Film	N/A	N/A		8.30		0.1205	
					$\sum R_{SI}$	0.32	0.36
					$\sum R$ (hr*ft²*°F/BTU)	1.80	2.06
					U (BTU/hr*ft2*°F)	0.56	0.49

Conductive Properties of Fully Tempered, 1 Lite Curtain Wall

Layer	*Conductivity, k (W/m*K)*	*Thickness (m)*	*Thickness (in)*	*Conductance, C (W/m^2*K)*		*Resistance, R (m^2*K/W)*	
				Summer	*Winter*	*Summer*	*Winter*
Exterior Air Film	N/A	N/A		23.00	34.00	0.0435	0.0294
Glass Lite 1	0.96	0.0064	0.25	151.18		0.6164	
Interior Air Film	N/A	N/A		8.30		0.1205	
					$\sum R_{SI}$	0.78	0.77
					$\sum R$ ($hr*ft^2*°F/BTU$)	4.43	4.35

U (BTU/hr*ft2*°F)	0.23	0.23

BRICK VENEER ANALYSIS



$A = 43.6\,in^2/ft_{WALL}$
$I = 47.8\,in^4/ft_{WALL}$

$f'_m = 1500\,psi$

- MID-HEIGHT STRESSES, AXIAL LOAD

$P_{MH} = (27\,PSF)(6') = 162\,lb/ft_{WALL}$

$f_a = \frac{P}{A} = \frac{162\,lb/ft_{WALL}}{43.6\,in^2/ft_{WALL}}$

$\therefore f_a = 3.72\,psi$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{47.8\,in^4/ft_{WALL}}{43.6\,in^2/ft_{WALL}}}$

$\therefore r = 1.05''$

$\frac{h}{r} = \frac{(12')(12''/ft)}{1.05''} = 137 > 99$

$\Rightarrow F_a = 0.25 f'_m \left(\frac{70}{h/r}\right)^2$

$= 0.25(1500\,psi)\left(\frac{70}{137}\right)^2$

$\therefore F_a = 101\,psi$

- MID-HEIGHT STRESSES, BLAST LOAD

$M_{MH} = \frac{(98\,PSF)(12')^2(12''/ft)}{8} = 21168\,lb\cdot in$

$f_b = \frac{Mc}{I} = \frac{(21168\,lb\cdot in)(7.63''/2)}{47.8\,in^4}$

$\therefore f_b = 1689.4\,psi$

$F_b = \frac{1}{3} f'_m$

$\therefore F_b = 500\,psi$

→ CONT'D.

BRICK VENEER ANALYSIS.

- CHECK UNITY

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$$

$$\frac{3.72\,psi}{101\,psi} + \frac{1689.4\,psi}{500\,psi} = 3.41 >> 1.0 \quad \text{(NO GOOD)}$$

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BLAST AND PROGRESSIVE COLLAPSE ANALYSIS

APPENDIX G

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