University of North Carolina's Imaging Research Building

Final Report

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Structural Option

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University of North Carolina
Imaging Research Building

125 Mason Farm Road, Chapel Hill, NC

Building Statistics

Size: 325,000 SF
Cost: $280 Million
Building Height: 8 above grade + 2 subgrade = 10
Architect: Perkins + Will
Structural/Civil: Mulkey Engineers & Consultants
MEP: Newcomb and Boyd
CM: Choate Construction

Architecture

The UNC Imaging Research Building will be a state of the art imaging and cancer research facility located at UNC Chapel Hill. It will have an L-shaped floor plan that will include facilities for a 7 Tesla Magnet, a 1.5Ghz NMR, a Cyclotron, MRI machines, PET/CT Scanners and other imaging equipment on its two sub-grade levels. It will also include university offices and a number of other different functioning research labs. The façade will be a mixture of glazed aluminum curtain wall and precast panels.

Structure

The UNC Imaging Research Building will have a concrete superstructure with mass walls below grade in order to shield radiation from there imagaing machines. The foundation will consist of a combination of mat footings, wall and shearwall footings resting mostly on bedrock.

MEP

The cooling sytem will consist of with custom air handling units and precision room air conditiong units utilizing campus chilled water. Campus chilled water is used in plate and frame heat exchangers to privede chilled water to cooling coils in AHU's and chilled water to precision room air conditioning units. The heating system will use to district heating water to provide hot water to heating coils in air handling units and heating water to terminal unit heating coils. The equipment used will be three heating water pumps with high efficiency motors.

Daniel Hesington - Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2010/drh5015/index.html

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Executive Summary

The following report investigates and discusses the effects of redesigning the above grade gravity and lateral systems of the UNC Imaging Research Building from concrete to steel while maintaining key architectural concepts. Using RAM Structural System, the floor system was reduced from 30" to 24 ¼", opening up 5 ¾" of vertical trade space. This is because girders were limited to 18" in depth. Columns were also kept to a minimal 14" in depth, compared to the typical 24"x24" columns in the existing structure. Also by replacing the existing shear walls and replacing SCBF' as the main lateral force resisting system above grade, the number of lateral frames was reduced while still meeting both strength and drift requirements. With all of the gravity and lateral designs, hand calculations were completed to confirm the results that were determined with RAM.

An overall cost analysis and schedule comparison for the two framing systems was also completed. An initial square foot cost estimate was done followed by a detailed estimate of both options. To make an "apples-to-apples" comparison, only the beams and girders, columns, and lateral frames were evaluated. The cost of the existing concrete system was estimated to be approximately 4.83 million, while the cost for the redesigned steel framing was estimated to be 3.68 million. As far as erection time is concerned, the steel system had the advantage taking only 225 days versus 315 days for concrete, but the use of more crews (other than the suggested amount by R.S. Means) would increase this schedule, increasing the cost as well.

Using the Depart of Defense's Unified Facilities Code, the glass façade on the south face of IRB was designed for blast loading to effectively protect the occupants of the building. It was determined that 5/16" heat strengthened, laminate panels between mullions will effectively withstand an equivalent TNT charge of 220 pounds at a standoff distance of 50 feet. This is the equivalent of a roadside attack by a small compact vehicle. A redesign of this magnitude would certainly incur a cost increase compared to the existing façade, but in today's heightened risk of terroristic attacks, it is a consideration that might be of value.

Overall, it was determined that the steel structure would be a viable alternative to the existing concrete design. While certainly not a complete evaluation of the two systems, the research and analysis done in this report are substantial enough to make this assertion.

Introduction

The Imaging Research Building, also known as IRB, is located on the University of North Carolina's Chapel Hill campus on Mason Farm road. It has an "L" shaped floor plan containing a re-entrant corner, with the long face dimensions of 282'-4" by 247'-3". It has an overall height of 180'-0" from Basement 2 (second floor sub grade) to the roof, with setbacks at the mechanical mezzanine levels. The building's usage will be a combination of research space, laboratories, and office space for UNC.

Figure 1.1 - View of IRB from Northwest

Architectural Design Concepts

The Imaging Research Building at UNC Chapel Hill was designed by the architecture firm Perkins + Will. Its primary usage is the driving force behind many of the structural decisions for the project. Once it is open, it will contain the most advanced imaging equipment in any one spot in the world. First, the two sub grade floors house several heavy pieces of imaging research equipment that have large Gaussian fields. Because of this, foundations, walls, and slabs were made thicker than usual, which will result in the use of mass concrete pouring techniques when constructed. For example, the foundation where a 1.5GHZ NMR machine will sit required a 6' thick mat footing.

Above grade you will find typical bays sizes of 21'-4" by 21'-4", and 21'-4" by 31'-4" driven by the laboratory space requirements on every floor. A bridge also connects the new imaging research facility to the existing Lineberger Cancer Center on the second floor. At the eighth floor, a large area houses all of the mechanical equipment with a partial mezzanine at the floor above, which services all of the imaging and laboratory equipment below. These architectural and usage restraints have a generous effect on the structural system.

Structural System

Foundation

The geotechnical engineering study was performed by Tai and Associates on November 12, 2008. The study indicates that the subsurface materials on the site consist of pavement and topsoil, fill, residual soil, weathered rock, and rock and boulders. Based on this composition, Tai and Associates determined a net allowable bearing pressure of 6000 pounds per square foot for Mulkey to use in their foundation calculations.

The result is a mixture of spread footings under the columns, and a combination of spread and mat footings under the large imaging research equipment and shear walls. The walls below grade range from 18” to 36” in thickness¸ and in one location a 36” wall spans both sub grade floors to the first floor unbraced. An example of a typical mat footing can be seen in **Figure 1.1**. As with the other mat footings, this one is combined and sits under two pieces of large imaging equipment. It is 6’-0” thick and also supports a shear wall that steps 6’ in elevation. Another area of note in the foundation design is a 6’-0” thick concrete footing which will support a cyclotron, another heavy piece of imaging equipment.

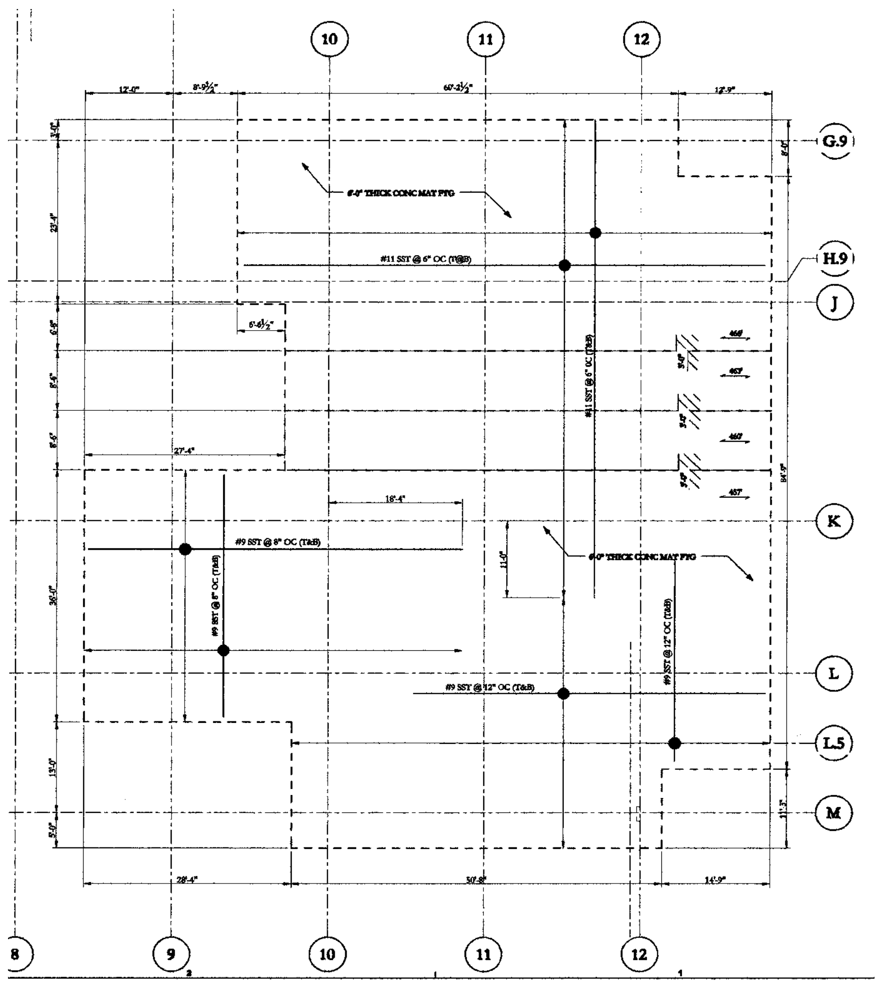

Figure 1.1 – Mat Foundation under Imaging Equipment

Superstructure

The first floor and the floors above to the eighth floor is a 6" one-way cast-in-place slab (NWC) with a compressive strength (f'c) of 5 ksi. The beams on these levels are mostly 18"x20" T-Beams, which change directions at the re-entrant corner where the building changes directions. The girder dimensions vary, but are typically 28"x30".

Most of the columns in the Imaging Research Building are 20"x20" square columns with #3 ties above the first floor, and 24"x24" below grade, with all them having a compressive strength of 7 ksi. The typical frame consists of four bays with three of them being approximately twenty feet in width and the other being thirty feet in width to accommodate the laboratories that occupy these spaces on almost every floor of the building.

For more detail on the superstructure, a section of the third floor framing is provided in **Figure 1.2** for reference.

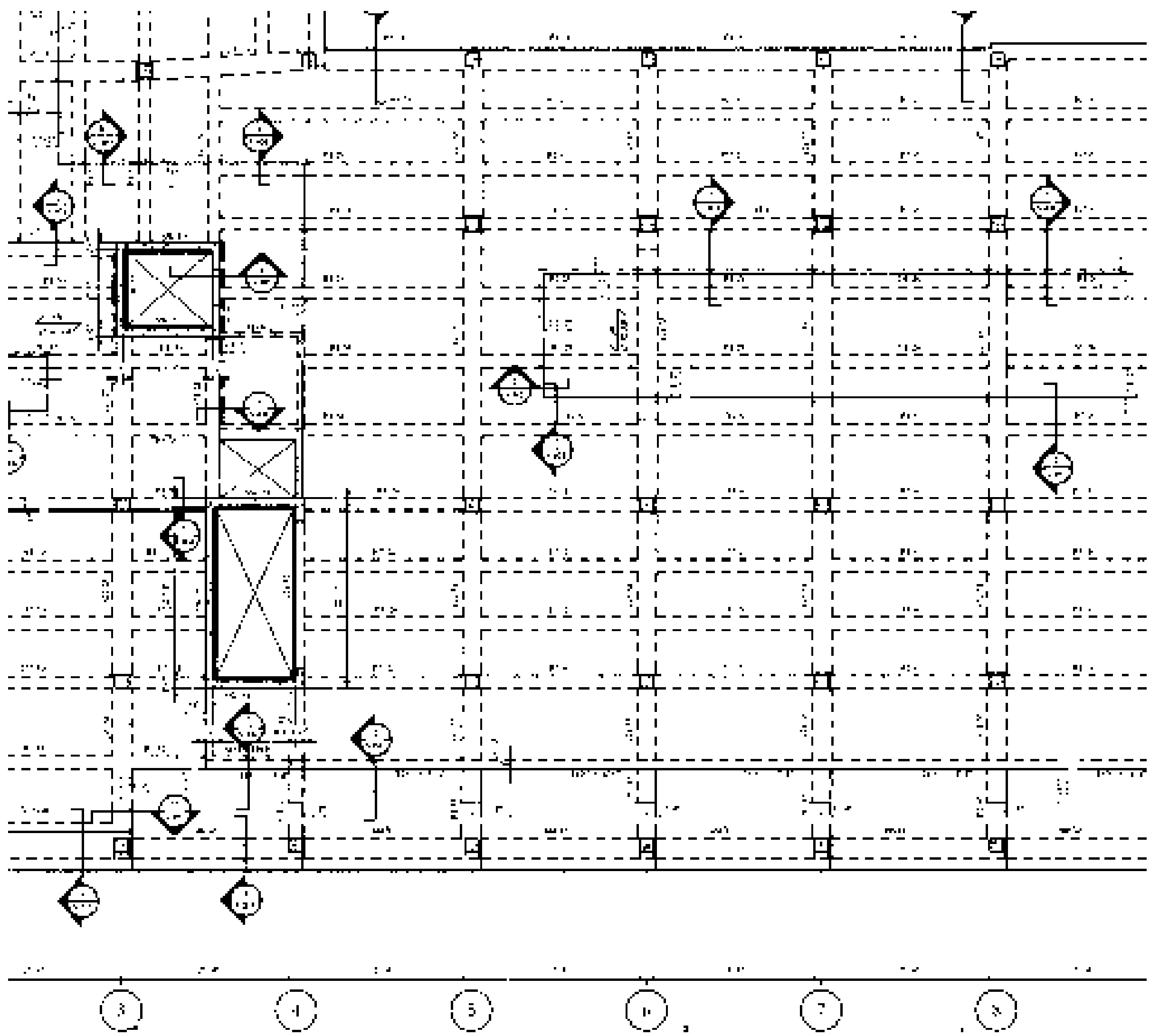

Figure 1.2 - Section of Third Floor Framing

Lateral System

Ordinary reinforced concrete shear walls are used as the main lateral force resisting system in the UNC Imaging Research Building. The largest shear walls wrap around the main elevator and stairwell cores while the other ones encase mechanical closets. Most of the shear walls run from the foundation to the mechanical mezzanine with only half of them continuing to the roof level. There are thirty-three shear walls either 12" or 16" thick. **Figure 1.3** shows the location of the existing shear walls and **Figure 1.4** depicts the shear walls around the main stair and elevator core

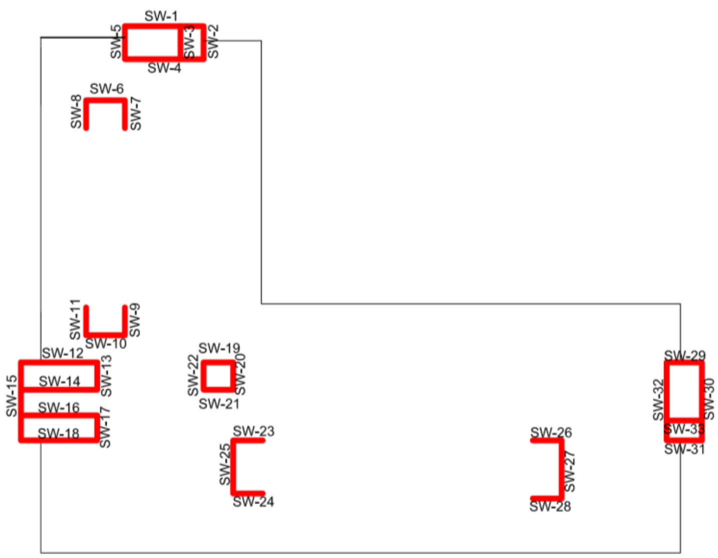

Figure 1.3 – Location of Existing Shear walls

(Note: Not to Scale)

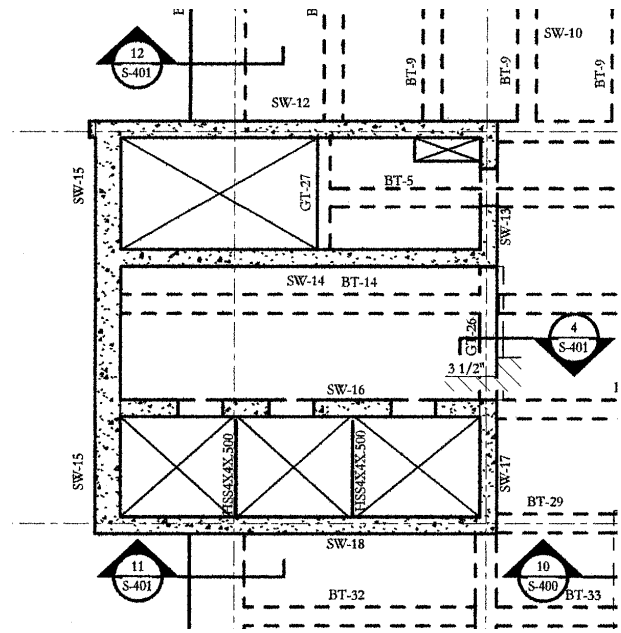

Figure 1.4 – Shear walls around Elevator Core

Problem Summary

Problem Statement

Currently, IRB is designed as a complete concrete structure. The main reason for this is because of the existence of the highly magnetic imaging equipment on the two sub grade floors of the building. There is also equipment on the first floor as well, but after that there is no other magnetic equipment that would determine a need for a concrete column, beam and floor system.

There are several reasons though why concrete was chosen as the remainder of the building's superstructure. As far as the lateral system is concerned, shear walls are regarded as the cheapest method for resisting lateral loads. There is also no problem connecting the lateral system into the rest of the framing. Not only that, but the one-way cast-in-place slab is a simple floor system to design and construct. Therefore, it is relatively inexpensive both in design and construction. Also, it works for heavier live loads as in the Imaging Research Building because there is very little deflection when used in combination with beams. But more importantly, penetrations in the slab cause few structural problems because there is not a lot of large rebar or tendons running through it and it is easy to reinforce around them after they have been created. This is very important on a project like the IRB where there are a number of mechanical systems and equipment lines for the imaging laboratory equipment penetrating through the floors.

However, the concrete superstructure is very bulky and heavy. The 20"x20" columns reduce the usable floor space and the 30" deep girders for the floor system take up a lot of critical room that mechanical and other trades could use. Also, the cast-in-place beam and slab system requires a lot of formwork that will be time consuming and costly. This results in a longer construction schedule which will delay the opening of the building.

After reviewing this information, the goal is to reduce the overall weight of the building, increase usable floor space, and increase vertical trade space, while not incurring much of a cost increase, if any at all. It has already been determined in Technical Report 2 that the composite steel floor system in combination with steel framing would be the most likely candidate for replacing the existing floor system and framing to meet these goals.

There are some problems that will need to be addressed in the proposed solution. The lateral system will have to be changed, unless a solution can be generated to tie the new steel framing to the shear walls. Also, the issue with the highly sensitive imaging equipment will also have to be addressed.

Proposed Solution

Floor System

To meet the goals outlined in the problem statement, the superstructure of the building will be changed from concrete to steel **only** above grade. Hence, the new structure of the building will be a concrete base for the two basement levels, with steel above. The new floor system will preliminarily be composite steel and composite deck. From the study done in technical report two, the implication of a composite steel framing system should decrease the overall depth of the floor system, allowing more space to be freed for other trades as seen in **Figure 2**.

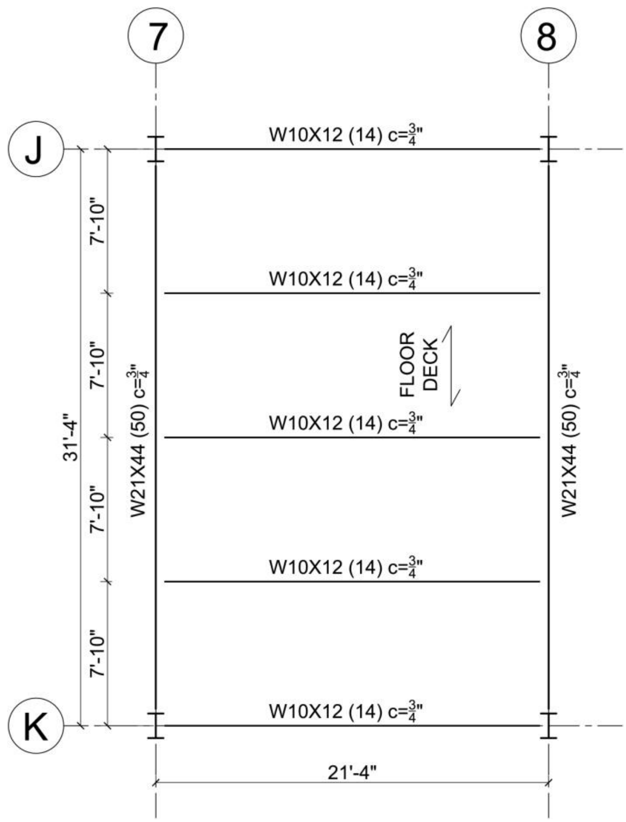

Figure 2 - Typical Composite Floor Framing

(Note: Preliminary Design from Tech Report 2)

While columns weren't addressed in technical report two, the steel columns should be smaller than the existing 20" by 20" concrete columns. In turn, more usable floor space will become available unless further study indicates that the need for increased fire protection negates the smaller depths.

Lateral System

For the lateral system, it will also be changed to either brace frames or moment frames unless enough evidence suggests a cost effective shear wall connection can be employed. Since cost drives most projects, if it is determined that a new lateral system is economical, it will be designed and summarized. The location of the new lateral system will be where the existing shear walls are located from the first floor to the roof.

Foundation System

Finally, an analysis will be done to determine the impact of the steel structure on the foundation. Since it was preliminarily determined in technical report two that steel framing will reduce the overall weight of the structure, the foundations should be redesigned to be shallower, and therefore less expensive. The goal will be to eliminate the mat slabs as much as possible and redesign the foundation as spread and continuous footings.

Solution Method

The design of the steel framing will be based on the 13th edition of the AISC steel manual. Analysis for gravity and lateral loads will be done with a model created in RAM Structural System based on LRFD. Input for the model will consist of loads as determined from ASCE 7-05 and trial sizes of the members. Live load reduction will be considered and load combinations from ASCE 7-05 will be set up and run to determine the required sizes of the members for the steel framing. Time permitting, the new members will be spot checked by hand.

After the gravity framing as been determined, research will be conducted to determine the type of connections available and the cost of the connections for steel framing into shear walls. The cost of braced frames and moment connections will also be surveyed. The method that is most cost effective will be chosen and designed in either RAM or ETABS for a new lateral system, or by hand for the steel to concrete connection.

Finally, with the new overall building weight, the new impact on the foundations will be analyzed with hand calculated spot checks. RAM foundation will be used to redesign the foundations if it is warranted.

Design Goals

The goal of this depth study was to determine the feasibility of changing the structural system of the Imaging Research Building from a one-way cast-in-place slab system with ordinary reinforced shear walls to a composite steel system with steel braced frames. Other goals that were present during the redesign of IRB are as follows:

- To maintain the current column layout as much as possible in order to maintain the open floor plan as required by the usage of the building and to limit the impact on the architecture of the building.
- To design the new composite floor system efficiently so that the total depth of the system is less than the original to free up vertical trade space.
- To use RAM Structural System to design the gravity and lateral members, and confirm these sizes with hand calculations
- To eliminate the need for mat slabs for portions of the foundation due to the significant weight of the existing structure and replace them with more economical spread footings.
- To present a design that has a shorter construction schedule with less material and construction costs than the existing design for IRB
- To design a blast resistant façade with connections to the new steel framing.
- To follow all codes and standards during the redesign.

Structural Depth

Introduction

The Imaging Research Building was originally designed as a heavy, one-way cast-in-place concrete beam and slab system to meet the demand of the heavy live loads, shielding of imaging equipment and the inevitable mechanical openings that would be required. Steel was chosen for the redesign due to the lower weight, shorter erection time, high tensile strength, and because concrete was the focus of the previous three technical reports. Out of the possible steel framing systems, a composite steel system was chosen (see **Figure 3**) because of its ability to maintain the current spans of the building while decreasing the total floor depth. Also in conjunction with the material change to steel, the lateral system was changed to braced frames, as this choice does not interfere with the architecture of the building, and it is the next most economical option next to the existing shear walls. The conclusions from this study will be used to compare the redesign to the existing structure later in the report, and determine whether not a steel system would have been a feasible option for IRB's design teams.

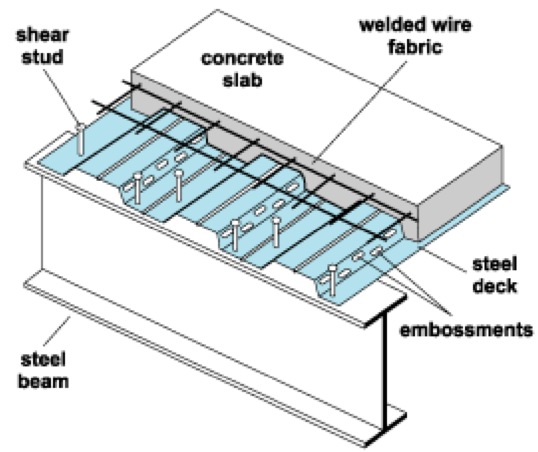

Figure 3 - Composite Floor System with Metal Deck

Codes and Design Standards

As with the previous technical reports, the building code used for the final report was the 2006 International Building Code (IBC), and loads were determined using the American Society of Civil Engineers (ASCE) 7-05. The steel framing was designed referencing the American Institute of Steel Construction (AISC) Manual for Steel Construction, 13th Edition. Additionally, the composite steel deck was selected using the Vulcraft Steel Roof and Floor Deck Catalog based on the Steel Deck Institute's (SDI) standards. The following factored load combinations from Chapter 2 of ASCE 7-05 were considered during the redesign:

(Note: D_i, F, F_a, H, R, T, & W_i are assumed to be zero)

1.4D
1.2D + 1.6L +0.5(Lr or S)
1.2D + 1.6(Lr or S) + (L or 0.8W)
1.2D + 1.6W + L + 0.5(Lr or S)
1.2D + 1.0E + L + 0.2S
0.9D + 1.6W
0.9D + 1.0E

Materials

Structural Steel

W-Shapes: ASTM A992
Shear Studs: ASTM A490
Base Plate: ASTM A572

Concrete (Below Grade) (28 day compressive strength)

Elevated Slabs on Metal Deck: 3500 psi
Elevated Slabs and Beams: 5000 psi
Columns, Shear Walls: 7000 psi
Basement Walls, Site Walls: 7000 psi
Slab on Grade, Footings, Grade Beams: 4000 psi

Reinforcement

Welded Wire Fabric: ASTM A185
Reinforcing Bars: ASTM A615, Grade 60

Design Procedure

The first step considered in the design of the new substructure was the layout of the column grid and framing. Because of the strict requirements for usable floor area of the required laboratory spaces on the typical floors, and the location of the individual pieces of imaging equipment on the lower floors, it was determined that it was not necessary to change the bay sizes or column grid. Next, based on the determined floor loads and the typical spans between beams, a composite deck was selected. After this, the computer modeling software RAM Structural System was utilized to model the existing conditions below grade, and the new steel superstructure above grade. Once the beam sizes were generated with the appropriate number of shear studs, hand calculations were done to check the validity of the designs. These

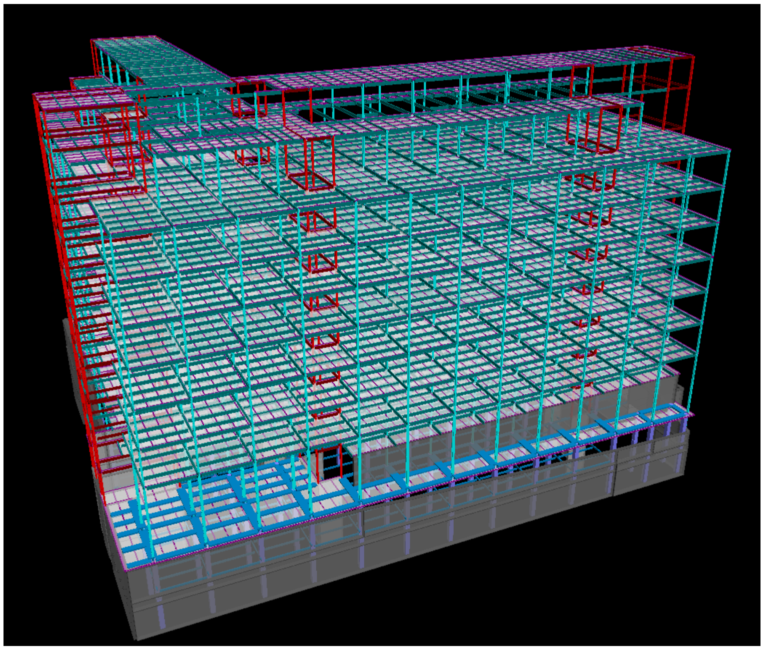

Figure 4.1 - RAM Model

calculations can be found in **Appendix D**. Columns were also sized using RAM and checked by hand, which can be found **Appendix E**.

Once the beams and columns were designed, the lateral system was then developed. Since there were no conflicts with architectural requirements, the previous lateral system being shear walls, braced frames were chosen as the new lateral system with shear walls continuing below grade. Because eliminating the shear walls below grade was not an option, the location of the braced frames simply picked up where the shear walls stopped at the first floor. As far as the design method is concerned, again RAM was used to determine the sizes, and the validity of these sizes was checked by hand. These calculations can be found in **Appendix F**. Serviceability requirements were also checked to make sure they were not exceeded.

Finally, preliminary calculations were done using RAM Structural System to investigate the effects of the structure on the foundation. However, the result of this analysis determined that it was not necessary for a complete redesign. The reasons why are included in the foundations section of the report.

Design Loads

Gravity Loads

As stated in Technical Report one, the determination of gravity loads for the existing structure by Mulkey Engineers and Consultants was done using the 2009 North Carolina State Building Code (2006 International Building Code with Revisions), which adopts ASCE 7-05 for its minimum design loads for buildings. The final report also uses ASCE 7-05 as the main reference in accordance with the requirements of AE Senior Thesis. The only addition to Table 1 from previous technical reports is the addition of the new composite slab and deck.

Table 1 -Gravity Loads

Description	Mulkey	ASCE 7-05
DEAD (DL)		
Reinforced Normal Weight Concrete	150 pcf	150 pcf
Slab + Deck	65 psf	65 psf
LIVE (LL)		
Roof	30 psf	20 psf
Offices	50 psf	50 psf
Public Areas, Lobbies	100 psf	100 psf
Laboratories	100 psf	60 psf
Corridors, 2nd & Above	100 psf	100 psf
Corridors Ground	100 psf	100 psf
Stairs	100 psf	100 psf
Catwalk	40 psf	40 psf
Storage	125 psf	125 psf
Heavy File Storage	200 psf	250 psf
Mechanical Rooms	150 psf	150 psf
Level B1	150 psf	N/A
SNOW (S)		
Snow	16.5 psf	16.5 psf
SUPERIMPOSED (SDL)		
Finishes, MEP, Partions	25 psf	25 psf
Bathroom Terrazo	40 psf	N/A
Lobby Terrazo	60 psf	N/A
Mechanical Courtyard	300 psf	N/A
3T MRI Room	250 psf	N/A
7T Sheilding	75 psf	N/A
Hot Cells	350 psf	N/A
Water Tank	350 psf	N/A

Lateral Loads

Wind loads were also previously determined in Technical Report 1 using ASCE 7-05 Section 6.5, which describes Method 2 – Analytical Procedure. The variables used and the calculations for this analysis are located in **Appendix B**. Seismic loads were also previously calculated in Technical Report 1 using chapters 11 and 12 of ASCE 7-05 for the existing concrete structure. Because of the change in the framing from concrete to steel though, and the use of lightweight concrete for the new floor slabs, the seismic loads had to be recalculated using the new material weight takeoffs. The calculations for the new seismic loads can be found in **Appendix C**. However, because of the decrease in the weight of the building the wind is now the controlling load case in both the north/south and east/west directions as seen in **Figure 4.2** and **Figure 4.3** below.

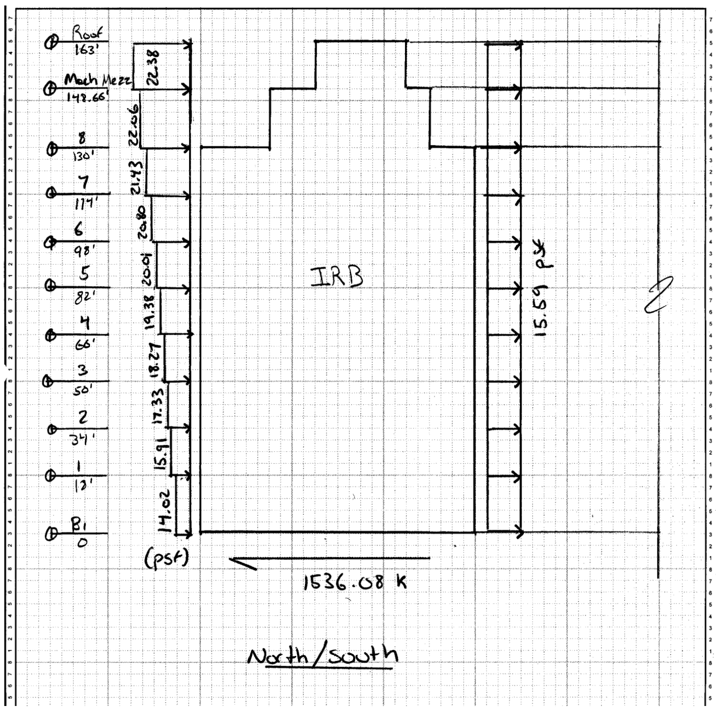

Figure 4.2 - North/South Wind Loads

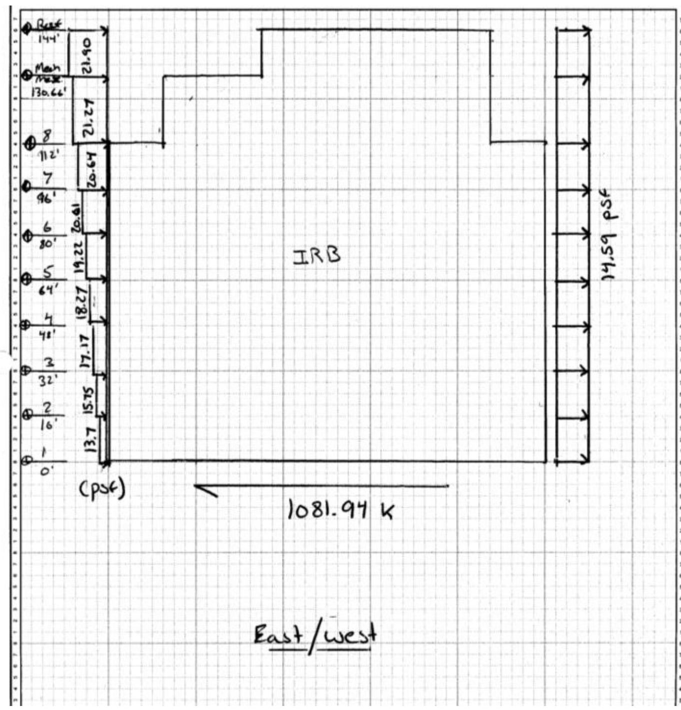

Figure 4.3 - East/West Wind Loads

Design Process

Gravity Framing

Composite Beam, Girder and Deck Design

The composite deck was selected using the Vulcraft Deck Product catalog which references the Steel Deck Institute's standards. Three factors were considered during the selection process: fire rating of the floor system, superimposed live load, and the max unshored span of the deck. First, it was determined by code that a restrained assembly fire rating of 2 hours is required of the floor system. Since the deck will be protected on each floor though by either an acoustical tile, gypsum board or spray tile, it was determined that a 1.5", 2", or 3" fluted deck could be used. Next, using chapter 4 of ASCE 7-05, it was determined that for the above grade floors (1-7), a live load of 100 psf be applied for the laboratory and corridor spaces, but it can reach as much as 200 psf in the heavy file storage areas. Because of these loads, and a max clear span of 9'-0" between beams, it was determined that the best solution would be a 2", 20 gage deck with 4 ¼" lightweight concrete. Vulcraft's 2VLI20 deck type was used for the design. The max unshored clear span for a 3 span condition was then checked to make sure the deck would not fail during construction. The pages used for the selection from the Vulcraft catalog can be found in **Appendix A**.

Using RAM Structural System, the composite beams were sized with the required number of shear studs using the Load and Resistance Factor Design (LRFD) method from the AISC 13th edition steel construction Manual. The controlling load combination of 1.2D + 1.6L was used to design the members and deflection limits were set based on the criteria below:

Live Load Deflection: $\Delta_{LL} = L/360$

Total Load Deflection: $\Delta_{TL} = L/240$

Pre-Composite Deflection: $\Delta_{TL} = L/360$

After the first optimization of the beam sizes, all of the members were W18's or less, except for 7 girders that were W24x68's which supported the largest bays in the middle of the floor plan. This was unacceptable since the goal was to reduce the overall floor depth from the original concrete design.

Since adding another row of columns to pick up the load was not an option due to the fact that they would interfere with crucial laboratory space, two options were considered, camber and increasing the plastic section modulus of the girders. Research was conducted to see which of the two methods would be more cost effective. Presentation slides from Dr. Louis Geschwindner estimated the cost of cambering a single member to be $30-$75 while the cost of increasing the weight was approximately $0.40 per pound. Initially, W18x86's where chosen so that the maximum floor depth would be 24 ¼", 5 ¾" thinner than the existing concrete design. Upon further analysis though, the design failed deflection limits. Not wanting to increase the floor depth another 3" to W21's for half of the floor plan because of 7 girders, the use of camber was also introduced. With a camber of ¾" though, the minimum size that could be used was a W18x97. Assuming that it costs $75 per beam to camber, plus another $14 dollars for the weight increase over the original optimized W24 sections, it will cost roughly an extra $620 dollars to use the W18x97's. This is not significant when compared to the total cost of the building.

After the beams were finalized in RAM, spot hand calculations were done to confirm these sizes. As mentioned earlier these supporting calculations can be found in **Appendix D**. The floor plans with the rest of the beam and column sizes can be found in **Appendix G**.

Below, **Figure 5.1** shows a typical floor plan with the composite beams and girders. The size of the member is listed first, followed by the required number of shear studs in parentheses, and finally the camber if there is any.

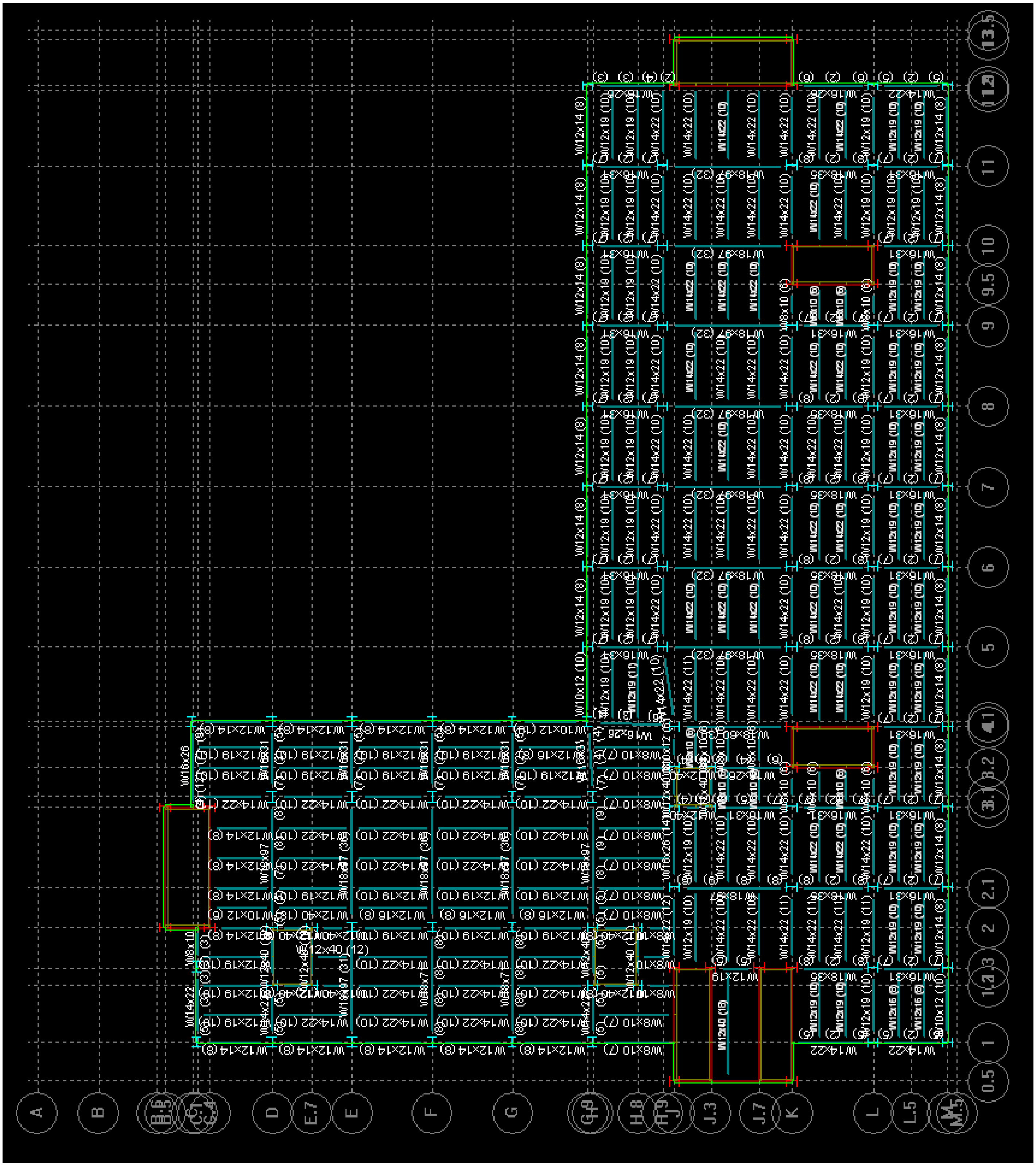

Figure 5.1- Typical Floor Plan

Column Design

The load path for the columns starts with the gravity loads in the building being carried by the slab and deck, and then the beams transfer the load to the girders, which in turn transfer the load into the columns and down to the building's foundation. Again, the columns were designed using RAM, with live load reduction according to ASCE 7-05 Section 4.8 and 4.9. The goal was to minimize the architectural impact and have the depths of the column be as small as possible without a substantial loss of efficiency. Also, column splices were designed for construction purposes. The result is a column splice at every second floor starting at the first floor. Repetition of sizes was also used again to cut down on the number of different sections required for fabrication. Repetition was also used to reduce confusion during erection in the field.

The AISC Steel Manual was used to spot check several of the column designs by hand. These calculations can be found in **Appendix E**. The RAM model with a visual representation of the code check can be seen in **Figure 5.2**, below.

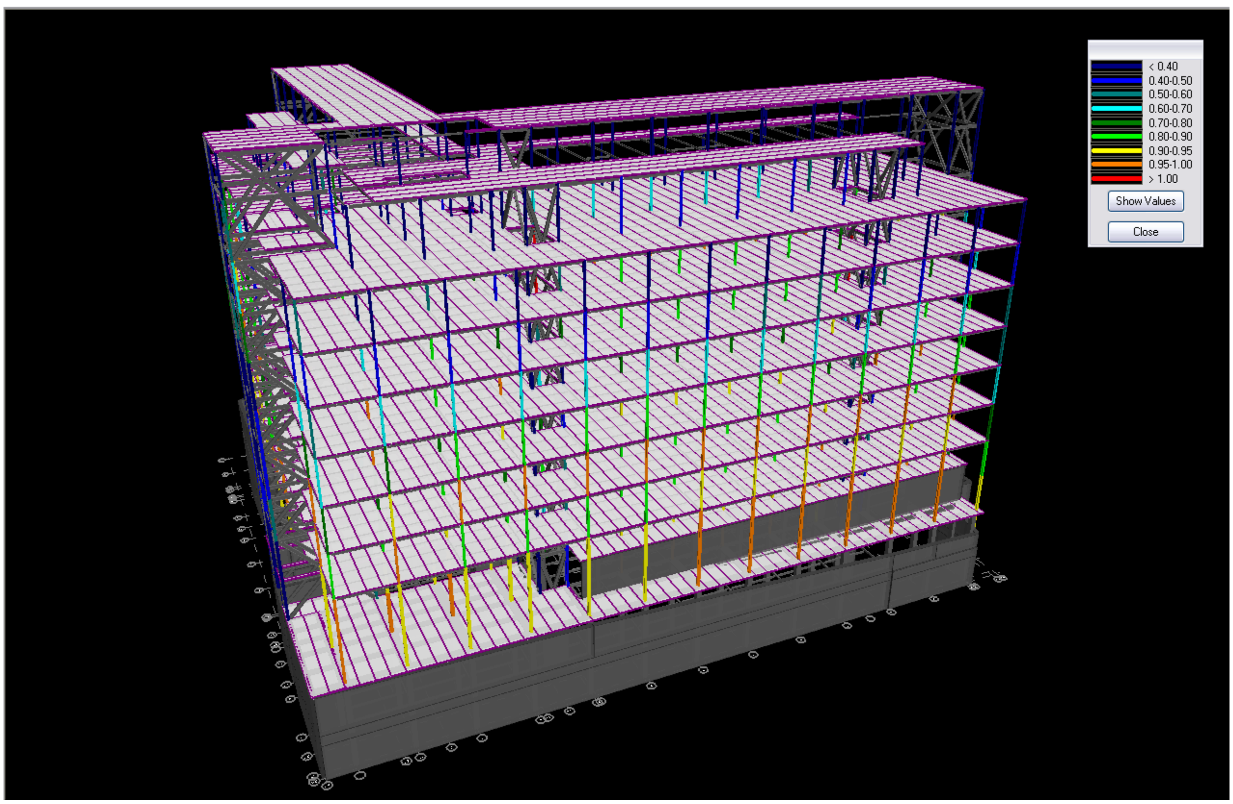

Figure 5.2 - Column Layout with Code Check

Lateral Framing

Introduction and System Choice

Braced frames, moment frames, and shear walls were all considered as the lateral system for the steel redesign. As stated earlier, shear walls were the original lateral force resisting system in the concrete structure. While connections from the steel framing to the existing shear walls was briefly investigated, their usage was eliminated since they had already been analyzed in techincal report 3. Moment frames were also considered, and a preliminary trial was run in RAM, but there was sizeable drift when the moment frames were placed in the same location as the shear walls. The location was important because of arcitectural restrictions of placing the lateral frames anywhere around the perimeter of the building. Furthermore, conversations with design professionals indicated that the moment frames were typically the most expensive system due to laber and didn't provide as much resistance as the others.

Therefore, braced frames were chosen as the lateral system for the steel redesign. Again, the main goal was to keep the braced frames in the same location as the shear walls. With an entire glass façade, and with the goal of minimizing the architectural impact of the redesign, placing the braced frames around the perimeter wasn't an option. Also, since the shear walls would pick up again below grade, it made the most sense to try and keep the lateral systems as consistent as possible. Unfortunately, the way the shear walls are laid out is not typically the same way braced frames would be placed. The shear walls were convenianetly placed around elevator and stairwall cores, and mechanical closets, thefore resulting in many clusters or groupings.

Modeling Assumptions and Considerations

Again, RAM Structural System was used to model the MLFRS. The paramaters for both wind loading and seismic loading were calculated by hand for the input. The following is a list of modeling assumptions and requirements for the RAM Frame model.

- A rigid diagphram was model at every floor with the lateral load being assigned to the diaphragm
- As mentioned earlier, load combinations were generated and used in accordance to all relevant codes.
- Lateral forces were applied to the center of mass
- Braces were assumed to be pinned at each end
- P-Delta effects were taken into account with the model according to ASCE 7-05

Initial Design

After running a preliminary analysis in RAM with the braced frames substituted for the shearwalls, it was clear that the frames around the mechanical closets could be eliminated. The final configuration can be seen below in **Figure 5.3** and **Figure 5.4**.

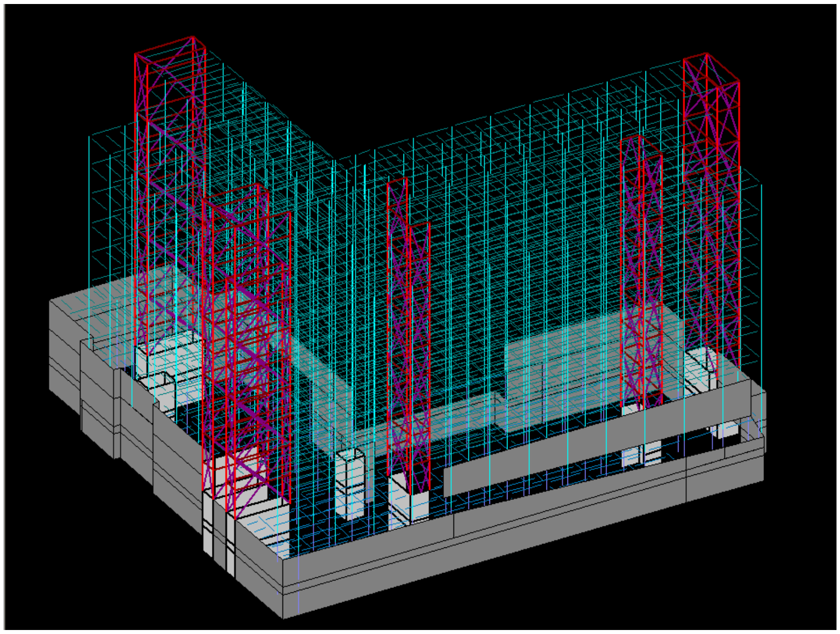

Figure 5.3 – 3D Model with Braced Frames

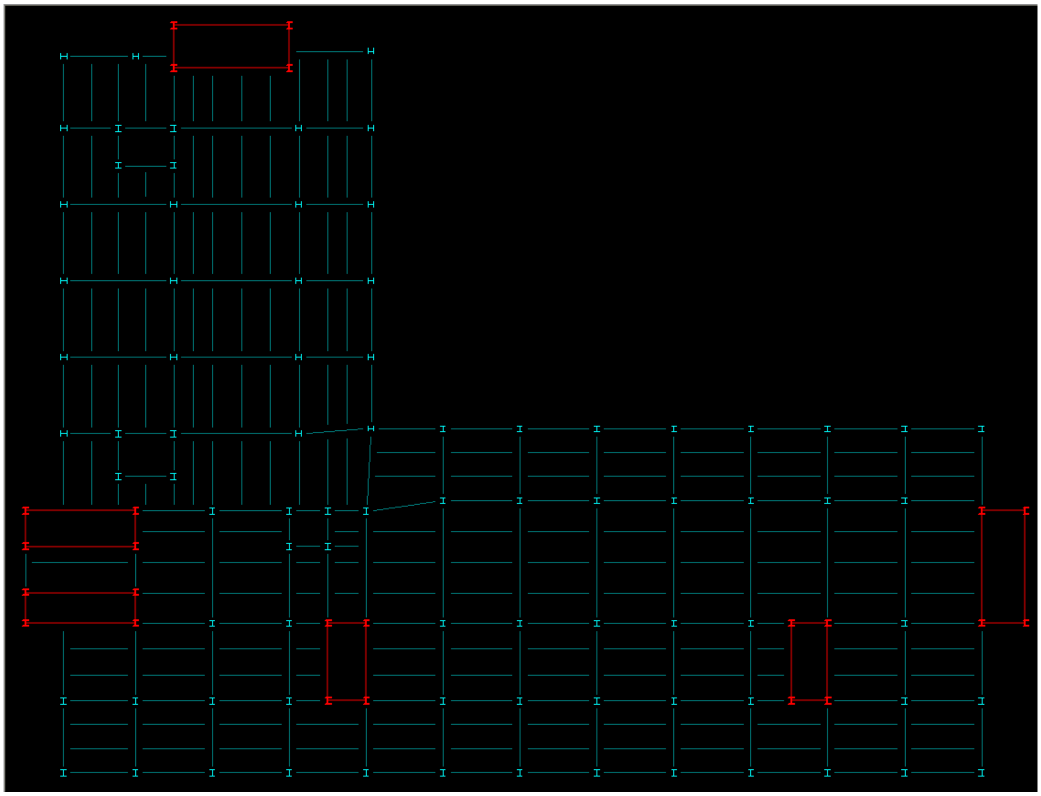

Figure 5.4 - Plan View of Lateral Frames in Red

An initial attempt was made to configure the braced frames around existing doorways, but because of the variation of door locations and the amount of time permitting to design each individual brace this attempt was compromised. Instead, research was conducted including discussion with design professionals to determine the most efficient frame pattern and connection. Special Concentric Brace Frames (SCBF's) were chosen over Buckling Restrained Braced Frames (BRBF's). The reasons being, that BRBF's are still relatively new and not as common as SCBF's. SCBF's also have multiple bracing configurations to choose from and multiple ways to design the seismic connections. Also, BRBF's tend to cost more and their complexity in modeling makes it very hard to manage drift control. SCBF's were also chosen over Ordinary Concentric Braced Frames (OCBF's) due to the better ductility of the system. Though more expensive, the SCBF's provided more resistance to drift, and therefore made the most sense when having to follow a very specific lateral frame layout plan.

Finally, the style of the SCBF's had to be chosen. Since IRB is not in a high seismic zone, the conventional chevron (V braces) could have been used in this case. Again, the bracing system chosen had to maximize strength and drift control with the given frame locations and a preliminary trial in RAM determined the V braces to be inadequate. Therefore the 2 story "X" or modified "X" was investigated. The "X" configuration dissipates the energy along the height of the frame during an earthquake, and the braces buckle simultaneously at all floors. It is also one of the most efficient designs in strength and drift control. Therefore, this was the configuration chosen for lateral system.

Final Design

The goal when assigning shapes in RAM was to be as consistent as possible and to again utilize repetition. Initially, the goal was to break down each frame elevation into three sections and have only three sets of beam, column and brace sizes, but this proved to be unfeasible because of the variation of loads on the frames. Also, since one of the main goals throughout the design of the steel structure has been to minimize architectural impact, the maximum column sizes used in the frame design were W18's. Although this is 2" deeper than the largest shear walls used in the original design, the gravity columns in the steel redesign are much smaller than the concrete ones and a lot of space has been gained there.

The braces, however, took on several iterations before satisfactorily meeting strength and drift requirements. At first, a combination of W16's and W14's were used for consistency in shapes and repetition. After several attempts though, the use of I-sections proved not viable for the braces. Therefore, the decision was made to use hollow structural steel (HSS)

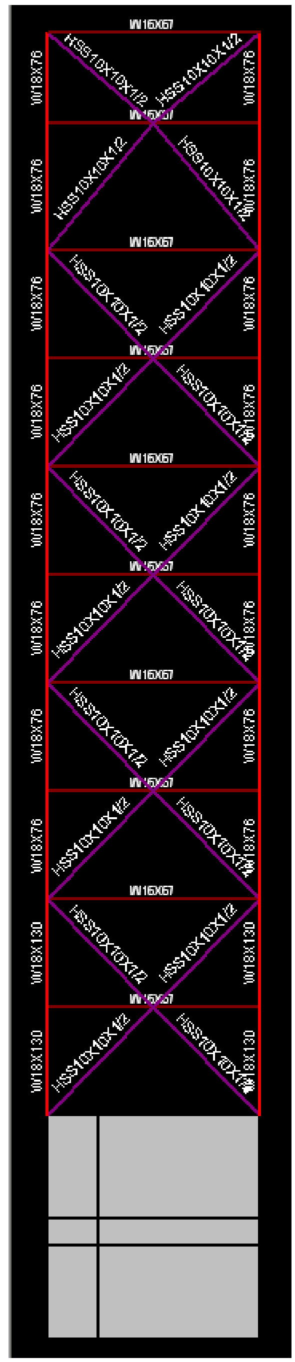

Figure 5.5 - Typical Modified "X" Braced Frame

shapes. The HSS shapes proved more efficient in strength, but the drift was still controlling the design, and was over the recommended limit. After several more attempts at increasing sizes, finally it was determined that two braced frames on the west face of the building could be combined to one larger one, which dropped the drift well within the accepted limit. An elevation of a typical lateral frame can be seen in **Figure 5.5**. A hand calculation spot check confirming the brace designs along with the rest of the later frame elevations can be found in **Appendix F**.

Serviceability

As stated previously, drift was the controlling factor for the lateral design. After the lateral analysis in RAM though, seismic was no longer the controlling load case in the x-direction. Instead, with the new building weight, wind was now the controlling load case in both directions. The seismic drift recommended limits still need to be checked to verify that serviceability is met in the event of an earthquake. The allowable seismic story drifts for IRB are determined by Table.12-1 in ASCE 7-05 based on Occupancy Category III. The two criteria considered for lateral drift and displacement are:

Wind: h/400

Seismic: $0.020h_{sx}$

RAM Frame was used to determine the drifts from both the wind and seismic loads. The drifts determined from the wind analysis were used as calculated in the evaluation while seismic drifts were amplified according to Section 12.8 in ASCE 7-05 using the following equation:

$$\delta x = \frac{C_d \; x \; \delta_{xe}}{I}$$

A summary of the story drift and the overall drift for both wind and seismic loads in the East-West and the North-South directions can be found in **Table 2**, below.

Table 2 - Story and Overall Drifts for Steel Redesign

Floor	Height Above Ground-z (ft)	Story Height (ft)	Wind North/South Drift (in)		Wind East/West Drift (in)		Wind Allowable Drift (in)		Seismic North/South Drift (in)		Seismic East/West Drift (in)		Seismic Allowable Drift (in)	
			Story	Total	Story	Total	Story	Total	Story	Total	Story	Total	Story	Total
Roof	162.00	14.33	0.38	2.70	0.27	1.93	0.43	4.86	0.20	1.41	0.22	1.49	3.44	38.88
Mech Mez.	148.66	16.66	0.33	2.43	0.24	1.74	0.50	4.46	0.18	1.31	0.20	1.37	4.00	35.68
8	130.00	16.00	0.33	2.06	0.24	1.47	0.48	3.90	0.18	1.08	0.19	1.15	3.84	31.20
7	114.00	16.00	0.33	1.73	0.24	1.23	0.48	3.42	0.18	0.89	0.19	0.96	3.84	27.36
6	98.00	16.00	0.31	1.39	0.22	0.99	0.48	2.94	0.16	0.71	0.17	0.76	3.84	23.52
5	82.00	16.00	0.29	1.06	0.21	0.76	0.48	2.46	0.15	0.53	0.17	0.57	3.84	19.68
4	66.00	16.00	0.24	0.75	0.16	0.54	0.48	1.98	0.12	0.37	0.12	0.40	3.84	15.84
3	50.00	16.00	0.22	0.46	0.17	0.33	0.48	1.50	0.10	0.22	0.12	0.23	3.84	12.00
2	34.00	16.00	0.22	0.22	0.17	0.17	0.48	1.02	0.10	0.10	0.12	0.12	3.84	8.16

Foundations

After running preliminary designs in RAM, it was the determined that the reduction in weight in the structure was not significant enough to do a complete redesign of the foundations. While, the existing spread footings could be reduced in size, the mat foundations supporting the lateral frames and imaging equipment would remain approximately the same. As far as overturning is concerned the previous analysis done in technical report 3 supports the assertion that this is not of concern. In order for overturning to occur the entire mat foundations which connect the majority of the building would have to rotate. If more time permitted, an advance mesh analysis in RAM concept would be suggested to confirm these assertions.

Structural Depth Summary

The main goal of introducing a steel structural system while maintaining architectural concepts was achieved with the redesign. The floor system was reduced from 30" to 24 ¼", opening up 5 ¾" of vertical trade space. Columns were also kept to a minimal 14", as compared to the typical 24"x24" columns in the existing structure. While this does not appear to be significant, the amount of space gained can be utilized by the architects. As far as the lateral system was concerned, we were able to reduce the number of lateral frames while still achieving both strength and drift requirements. SCBF's were chosen as the main lateral force resisting system, and it was also determined that wind will control the serviceability guidelines in both the north-south and the east-west directions. With all of the gravity and lateral designs, hand calculations were completed to confirm the results that were determined with RAM.

Breadth Topics

Construction Management Breadth

One of the main reasons the Imaging Research Building was switched from concrete framing to steel framing above grade was to determine if the use of steel could reduce overall construction cost and schedule time for the building. An analysis of the two systems was conducted to make this determination. As a result, the existing structure cost and schedule will be compared to the proposed steel structure, and a decision on the feasibility of the proposal will be presented.

Site

As mentioned previously, the Imaging Research Building is located on the University of North Carolina's Chapel Hill campus. The major access points for delivery of materials are off of route 15/501 and state road 86. As shown in **Figure 6.1**, the site is tight, with the adjacent Lineberger Cancer Center tight to the north side of the building. Because of the small site, staging will also be difficult for the construction team, with only space on the west side of the site. Finally, construction noise and vibration will need to be considered again because of the Lineberger Cancer Center in the immediate vicinity.

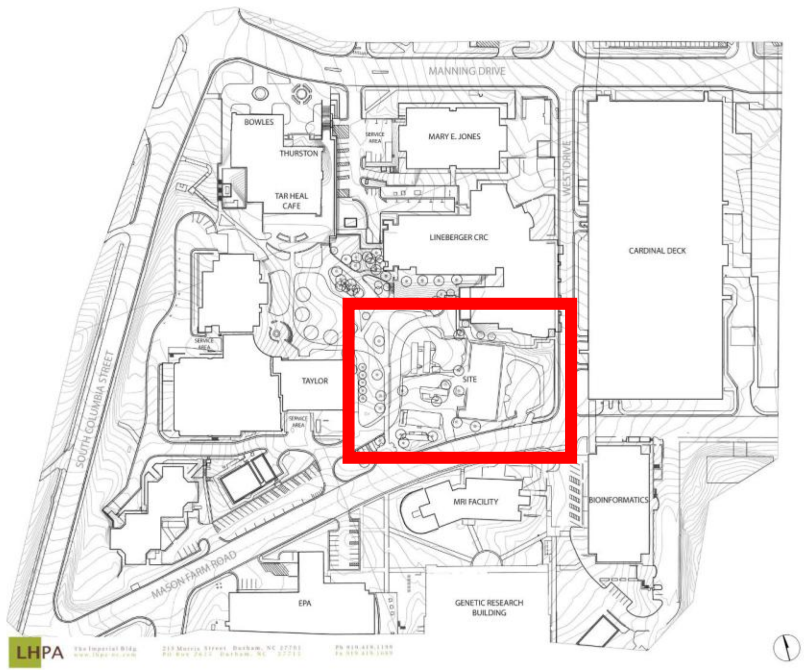

Figure 5.1 - IRB Construction Site

Construction Methods

The goal for the construction of the steel framing is to be as fast and efficient as possible. One of the benefits of steel over concrete is that, by the nature of the material, erection time will already be lessened due to the ease of fabrication. Another technique to speed the erection time is the use of repetition in member sizes. This was planned for in advance during the design of the gravity and lateral systems, and therefore the field coordination time and the chances of mistakes have been greatly reduced. Another factor to consider was if the structure would be erected by sections or floor-to-floor construction. After some research into construction methods in the central North Carolina area, constructing each floor in its entirety before proceeding was selected as the construction method of choice.

Costs

A detailed cost analysis was performed on both the existing concrete structure and the new steel design. As an approximation, 2009 R.S. Means Construction Cost Data online catalog was use to make an initial square foot cost estimate. In order to produce this initial estimate, the parameters of building area, building type, location, city cost index, and building material had to be set. Some assumptions had to be made in the form of a simple building model with basic components, but the program was then able to calculate costs for both the substructure and the superstructure. After analyzing each report, the total cost estimates were determined not to have enough deviation or significance for inclusion in this report. However, the different material costs for floor construction was a presentable comparison. The floor and roof construction costs for each material are presented in **Table 4.1**.

Table 4.1 - Square Foot Cost Estimate Comparison

Building Material	*Floor Construction Cost*	*Roof Construction Cost*	*Total Building Cost*
Concrete	$4,360,500.00	$295,500.00	$113,650,500.00
Steel	$3,850,000.00	$176,500.00	$98,750,500.00

While the initial square foot cost estimate was a good first attempt, a more detailed estimate was warranted. This involved a more in-depth takeoff for the respective systems. The goal was to produce an "apples-to-apples" comparison of the two systems. To achieve this, for both the concrete and the steel designs, only the beams, girders, columns and lateral systems above grade were priced.

The existing concrete system was the first to be analyzed. A takeoff was done of a typical floor to use as a base figure, and the remaining floors were estimated by square footage. R.S. Means was used to obtain prices for all of the concrete building components for both the columns and beams, including placement, formwork, concrete, and reinforcement.

As far as the steel redesign is concerned, again the main structural members were included in the pricing. The W shapes for the beams, columns and girders, and the HSS shapes for the lateral braces were all taken into account. A takeoff from the RAM model created for the gravity and lateral system designs was used to determine the quantity and length of the shapes.

After the unit amount for each building component for both systems was determined, R.S. Means was used to develop material, labor and equipment costs. A summary of these costs for both the concrete and steel systems can be found in **Table 4-2**. The more detailed tables of both the concrete and the steel estimates can be found in **Appendix H.**

Table 4.2 - Structural Material, Labor, and Equipment Totals		
Steel		
Summary	***Cost Per Square Foot($/SF)***	***Total Cost($)***
Material Total	$40.26	$3,351,091.08
Labor Total	$2.09	$174,227.83
Equipment Total	$1.87	$155,824.62
Total	$44.22	$3,681,143.53
Concrete		
Summary	***Cost Per Square Foot($/SF)***	***Total Cost($)***
Material Total	$24.77	$2,062,368.33
Labor Total	$32.26	$2,685,458.12
Equipment Total	$0.95	$79,250.54
Total	$57.99	$4,827,076.98

Scheduling

A schedule for each structural system was developed using the time acquired based on crew labor and unit –amounts. For the construction of the existing concrete framing of IRB, the building was divided into 4 zones. These zones were created based on the limit of the area of any single slab pour. This is shown in **Figure 7.2,** below.

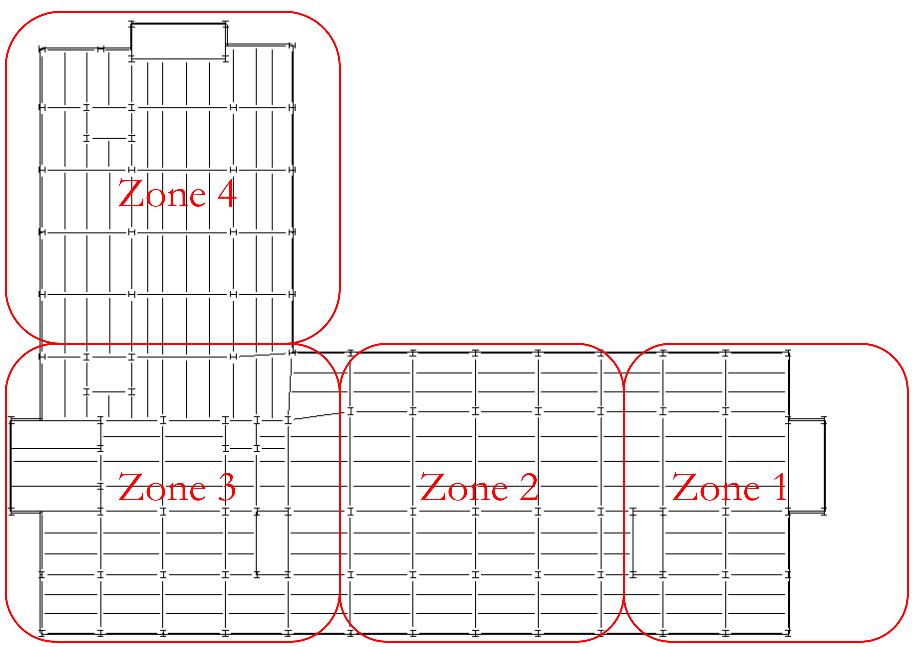

Figure 7.2 - Concrete Framing Pour Zones

Zones were also required for the steel framing. Instead of 4 zones required for the concrete structure, the steel structure only needed 2 zones. This is because the metal deck used for the slabs in the steel framing is stronger than the plywood forms assembled on-site for the concrete. Again, the zones required for the steel construction can be found in **Figure 7.3,** below.

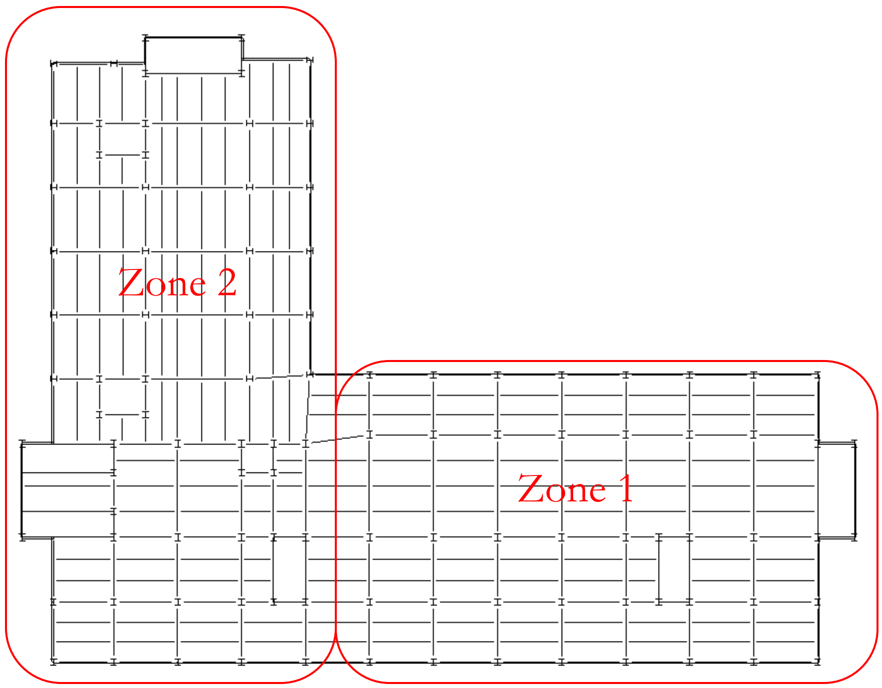

Figure 7.3 - Steel Framing Pour Zones

As mentioned previously, the construction method used for the both the concrete and steel structure is floor-by-floor construction. As a result, all of the members and slabs had to be formed, poured, and cured, before the slabs were formed, poured, and cured. Since the above grade faming was the only thing being changed, it was decided that a full schedule was not needed. Instead, since the only parts of the process being analyzed was actual construction time for the framing, and not lead time, the overall estimated construction duration for each system is summarized below.

Construction Management Summary

The detailed estimated of both framing options provided and accurate basis for comparing the two. The cost of the existing concrete system was estimated to be approximately 4.83 million, while the cost for the steel framing was estimated to be 3.68 million. As far as erection time is concerned, the steel system had the advantage taking only 225 days versus 315 days for concrete, but the use of more crews (other than the suggested amount by R.S. Means) would increase this schedule, increasing the cost as well.

Enclosure Breadth: Blast Glazing

Introduction

In today's society, terroristic attacks have become ever more prevalent. While the structure itself is very important to withstand such explosions, other building components such as the façade need to be taken into account. As of late, glazing has been at the forefront of research into blast protection, and it's only expected to grow in the future. According to a December, 2008 article in glass magazine, "The U.S. government will be investing great amounts of capital into protective glazing systems during the next 10 to 15 years to make the changes necessary to their existing buildings and for all new construction (Jeske, Glass Magazine). " Therefore, it is not a stretch to think that a building such as IRB could become a target for potential terrorists or even accidental explosions as well. Therefore IRB's curtain wall system will be redesigned to resist a potential blast load.

There are two major codes governing blast design, GSA/Interagency Security Committee Security Design Criteria and the U.S. Department of Defense Unified Facilities Code UFC 4-010-01, Minimum Antiterrorism Standards for Buildings. The ISC provides a graphic representation of how the effects of glass during an explosion equate to an equivalent hazard level. The numbers in **Figure 7.1** correlate to the performance condition in **Figure 7.2**.The DoD's criteria has a different set of requirements than the ISC as seen in **Figure 7.3**.

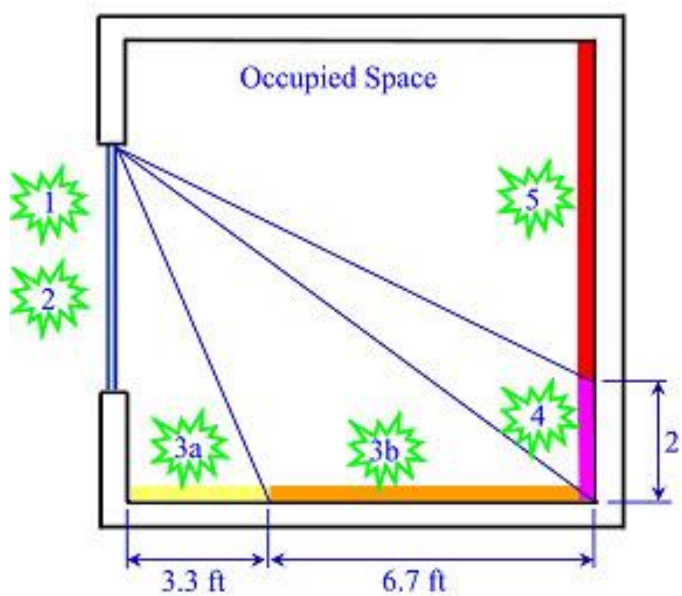

Figure 7.1 - Location of Glass during Explosion

Performance Condition	Protection Level	Hazard Level	Description of Glazing Response
1	Safe	None	Glazing does not break. No visible damage to glazing or frame.
2	Very High	None	Glazing cracks but is retained by the frame. Dusting or very small fragments near sill or on floor acceptable.
3a	High	Very Low	Glazing cracks. Fragments enter space and land on floor no further than 3.3 ft. from the window.
3b	High	Low	Glazing cracks. Fragments enter space and land on floor no further than 10 ft. from the window.
4	Medium	Medium	Glazing cracks. Fragments enter space, land on floor and impact a vertical witness panel at a distance of no more than 10 ft. from the window at a height no greater than 2 ft. above the floor.
5	Low	High	Glazing cracks and window system fails catastrophically. Fragments enter space impacting a vertical witness panel at a distance of no more than 10 ft. from the window at a height greater than 2 ft above the floor.

Figure 7.2 - Glazing Response According to ISC

Protection Level	Hazard Level	Description of Glazing Response
High	None	Glazing does not break. Doors will be reusable.
Medium	Minimal	Glazing will fracture, remain in the frame and results in a minimal hazard consisting of glass dust and slivers. Doors will stay in frames, but will not be reusable.
Low	Very Low	Glazing will fracture, potentially come out of the frame, but at a reduced velocity, does not present a significant injury hazard. Doors may fail, but they will rebound out of their frames, presenting minimal hazards.
Very Low	Low	Glazing will fracture, potentially come out of the frame, and is likely to be propelled into the building, with the potential to cause serious injuries. Doors may be propelled into rooms, presenting serious hazards.
Below Anti-Terrorism Standards	High	Doors and windows will fail catastrophically and result in lethal hazards.

Figure 7.3 - Glazing Response According to DoD

Design

The DoD code references two ASTM specifications that will be used for this redesign, ASTM F 2248-03 and ASTM E 1300-04. In order to develop a load that could be used for the design, ASTM F 2248-03 provided a method of conversion from a TNT charge to a 3-second design pressure. ASTM E 1300-04 was then used to design a glass unit that has a load resistance greater than the blast load.

The first step in determining the equivalent three second blast design pressure was to determine the standoff distance and the charge size in TNT pounds. Since a security plan wasn't available, the standoff distance was determined using existing civil drawings. Since Mason Farm road approaches IRB at an angle the distance from the curtain wall varies, but the average standoff distance was determined to be approximately 50 feet.

As far as charge size is concerned, a guide developed by the Unite State Department of Transportation (USDOT) was utilized to determine that the scenario of attack. An assumption was made that a charge in a small compact sedan would be most likely. This has an equivalent TNT charge weight of 220 pounds. Using ASTM F 224-03 it was determined that the 3-second equivalent design pressure was approximately 250 psf or 11.96 kPa (see **Appendix I** for charts).

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

Figure 7.4 - Equivalent Charge Guide

The next step was to determine the effective area to be designed for, and the glass type to be used. Since the largest opening will yield the highest forces, the largest square area between the mullions was determined from the architectural drawings, 5 ½' by 2'. As far as the glazing, heat strengthened glass, annealed glass, and fully tempered glass were all possible option. While more expensive, heat glass was chosen since it is not only stronger than the annealed glass, but it is also more attractive then the fully tempered.

Load Resistance is determined by the following equation. The factors of 2 and 1.8 are based on the fact that the glass has two equivalent lites and that is heat strengthened, respectively.

$$LR = 2 \times 1.8 \times NFL$$

Assuming that all four edges of the glass are supported by mullions, **Figure 7.5** from ASTM E 1300-04 was used to determine the non-factored load (NFL).

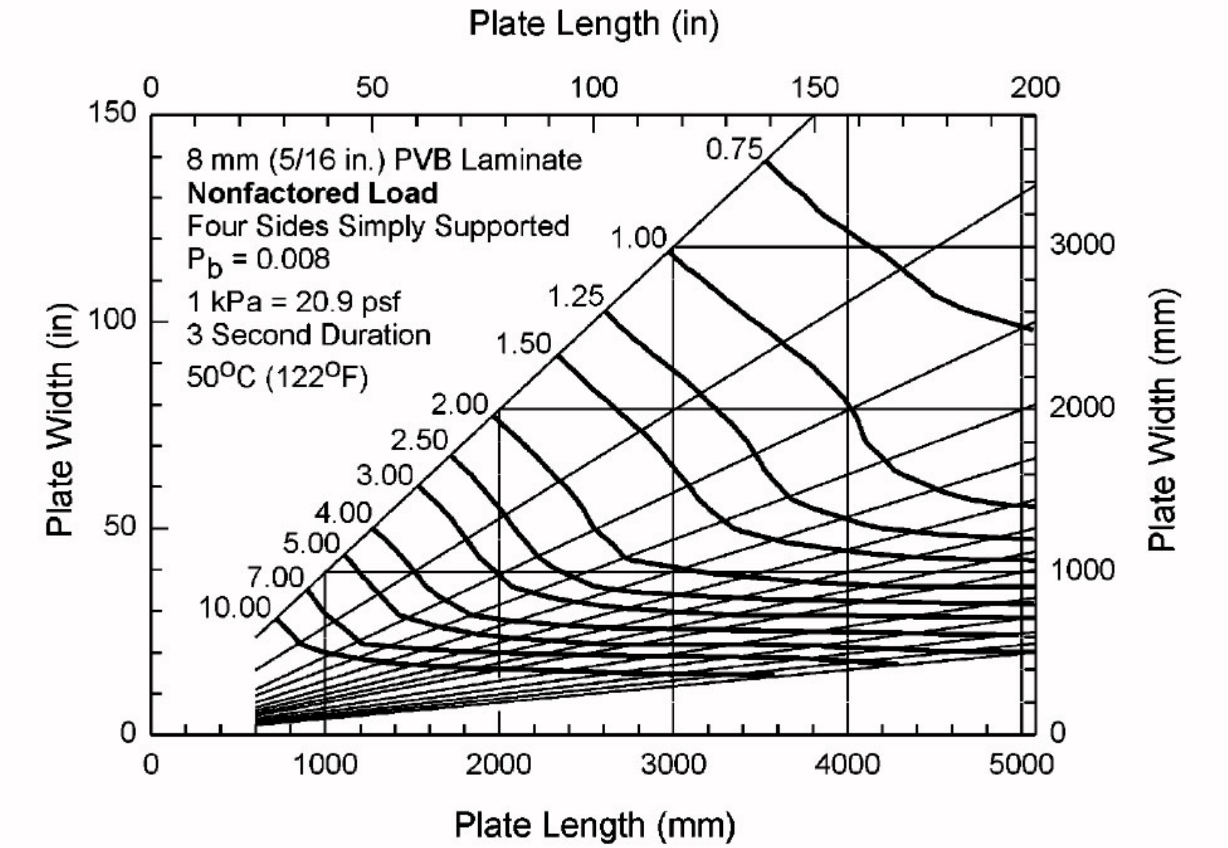

Figure 7.5 - Non-factored Load Chart

After several iterations, it was determined that the most efficient design would be a 5/16" heat strengthened, laminated insulated glass. According to the equation above this design provides a load resistance of 18 kPa for the area of the glazing on IRB's façade, compared to an equivalent load of 11.96 kPa – determined from and equivalent 220 pound charge at a standoff distance of 50 feet. As far as the mullions, frames, and connections are concerned, mullions and frames are to be designed to the specified blast load with a deflection limit of L/160 while connections need to be able to withstand two times the capacity of the glass.

Enclosure Breadth: Blast Design Summary

Using the Depart of Defense's Unified Facilities Code, the glass façade on the south face of IRB can be designed for blast loading to effectively protect the occupants of the building. 5/16" heat strengthened, laminate panels between mullions will effectively withstand the equivalent TNT charge of 220 pounds at a standoff distance of 50 feet. While certainly an increase in cost than the existing façade, in today's heightened risk of terroristic attacks, it is a consideration that might be of value.

Conclusions and Final Remarks

This thesis study was conducted to investigate the feasibility of switching from a concrete structure with 6" one-way cast-in-place slabs to a steel composite framing structure. The main goal was to maintain the key architectural concepts while introducing the new system. Both the gravity and lateral systems were redesigned, along with a cost and schedule analysis, and a redesigned blast resistant façade.

RAM Structural System was used to reduce the floor system from 30" to 24 ¼", opening up 5 ¾" of vertical trade space. This is a result of choosing a 2" composite deck with 4 ¼" lightweight concrete, and girders limited to 18" in depth. Columns were also kept to a minimal 14" in depth, compared to the typical 24"x24" columns in the existing structure. Also, as far as the lateral system is concerned, the shear walls were replaced with SCBF' as the main lateral force resisting system above grade. Doing this enabled the number of lateral frames to be reduced while still meeting both strength and drift requirements. With all of the gravity and lateral designs, hand calculations were completed to confirm the results that were determined with RAM.

An overall cost analysis and schedule comparison for the two framing systems was also completed. An initial square foot cost estimate was done followed by a detailed estimate of both options. To make an "apples-to-apples" comparison, only the beams and girders, columns, and lateral frames were evaluated. The cost of the existing concrete system was estimated to be approximately 4.83 million, while the cost for the redesigned steel framing was estimated to be 3.68 million. As far as erection time is concerned, the steel system had advantage taking only 225 days versus 315 days for concrete, but the use of more crews (other than the suggested amount by R.S. Means) would increase this schedule, increasing the cost as well.

The glass façade on the south face of IRB was designed for blast loading to effectively protect the occupants of the building. It was determined that 5/16" heat strengthened, laminate panels between mullions will effectively withstand an equivalent TNT charge of 220 pounds at a standoff distance of 50 feet. This is the equivalent of a roadside attack by a small compact vehicle. A redesign of this magnitude would certainly be an increase compared to the existing façade, but in today's heightened risk of terroristic attacks, it is a consideration that might be of value.

Overall, it was determined that the steel structure would be a viable alternative to the existing concrete design. Based on this evaluation, with the shorter construction time, and reduction in costs, the steel composite framing should have certainly have been an option while the design team was making their preliminary designs. The drawbacks, as noted in the proposal, are the heavier live loads and the slab penetrations that are inevitable. These constraints play to the favor of the existing concrete structure.

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    - Christopher Nesbit

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