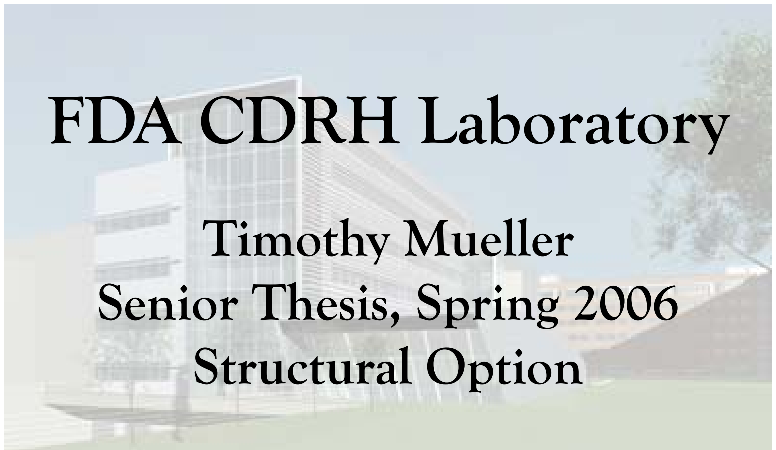


FDA CDRH Laboratory

Timothy Mueller
Senior Thesis, Spring 2006
Structural Option

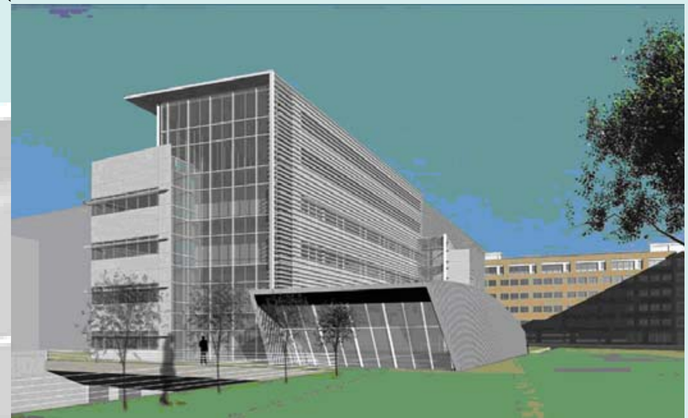
Ali M. Memari, PhD., P.E., Advisor

The Pennsylvania State University Architectural Engineering Department

FDA-CDRH Laboratory
Silver Spring, Maryland

Project Team

- **Architect:** Kling in Assoc. w/ RTKL
- **Structural Engineer:** Kling
- **MEP:** RTKL
- **Interiors:** Kling
- **Site:** Greenhorne & O'Mara
- **Owner:** GSA
- **GC:** Tompkins Construction
- **CM:** Heerey-Tishman



Architecture

- **Height:** Central core w/ 5th floor penthouse, four story main structure, one floor below grade
- **Façade:** Many decorative aluminum and sheet metal panels, large panels of glazing & horizontal sunshields enclose the main edifice
- **High Bay Laboratory:** Located on West side, includes a decorative curved metal roof

Project Overview

- **Purpose:** Center For Devices & Radiological Health Laboratory & Office Building on the new US Food And Drug Administration's White Oak Consolidation Campus.
- **Cost:** $63 Million
- **Delivery Method:** Design-Bid-Build
- **Size:** 139,805 Sq Ft
- **Major Building Code:** IBC 2000
- **Start Date:** March 22, 2005
- **Finish Date:** November 1, 2006

Structural

- **Foundation:** Step footing with spread footing below columns
- **Superstructure:** One-way cast-in-place concrete w/ monolithic poured columns & beams
- **Floors:** One-way concrete pan-joist system w/ overall depth of 520mm
- **Roof:** Typical W-shape steel w/ concrete deck
- **Unique protection:** Progressive collapse beams

Lighting/Electrical

- **Main Power:** 300/3990KVA Converted to 480/270KVA
- **Building Service:** 208/120V, 480Y/270V, & assorted specialty values such as 380Y/220V
- **Lab and office Lighting:** linear fluorescent troffers & linear under-cabinet fluorescent
- **Hall Lighting:** incandescent & compact fluorescent downlights
- **Main Laboratory Lighting:** incandescent and fluorescent High Bay luminaires

Mechanical

- **Cooling:** Four main Air Handling Units (23596CMF/Unit) in penthouse, each equipped with enthalpy wheel
- **Heating:** Two 300KW electric boilers (457Kg/Hr Steam/Unit) in penthouse

Construction Management

- **Building placement:** Coincides with campus Service tunnel & bridge
- **Crane:** Pick up Located near site entrance & accessible for entire site
- **Excavation:** Sloped pit to reduce potential for collapse
- **Leed rating:** Silver

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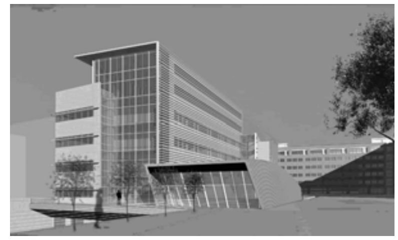

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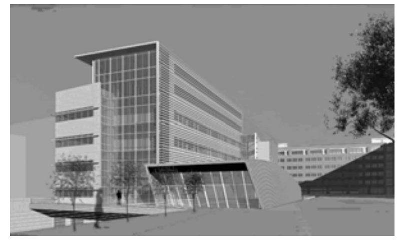

FDA CDRH Laboratory
Silver Spring, Maryland

Executive Summary

The FDA CDRH Laboratory, located in Silver Spring, Maryland, is on the Food and Drug Administration's White Oak Consolidation Campus. The current building is a cast-in-place concrete structure which is very resilient to vibrations, as well as lateral loads due to its very large concrete members and monolithic joint design. I have proposed to design the building using a steel structural system. Due to the current use of steel in the roof systems and penthouse area of the laboratory, the decision to construct the entire building of steel and remove a great deal of a trade from the site allowed for greater ease in coordination and removes problems caused with site congestion. This idea proved to be an even bigger benefit after researching the possibilities of steel construction.

Two steel layouts were looked at, one running the beams in the north-south direction, known as design a, and one with the beams running in the east-west direction, known as design b. After looking at vibration controls and project costs, it became clear that design b allowed for the most cost-effective system. The steel design also proved to be a cost savings over the steel design, and also provided possible time savings. All other concerns the were addressed in the current design, and would need additional consideration in the steel design, such as an additional lateral resistive system, fire protection, and blast control were all found to not cause a great enough price increase to be of concern in the overall consideration of the best building system.

Finally, the architecture of the building was also considered. It was concluded that a more traditional façade not only provided more continuity to the consolidation side of the FDA, but also provided for additional blast protection, without a great deal of increase in the overall cost. The increase in cost that did occur was small enough that the savings from changing the structural system could easily compensate.

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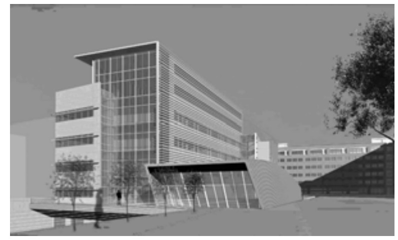
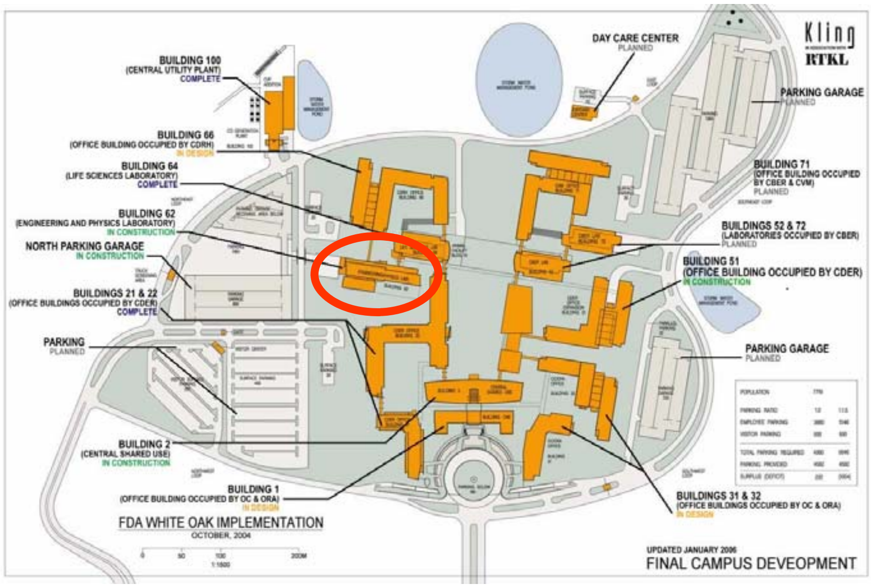


FDA CDRH Laboratory
Silver Spring, Maryland

Project Background

The FDA CDRH Laboratory is an office and laboratory space located on the Food and Drug Administration's White Oak Consolidation Campus. The campus, which was originally the naval ordinance testing grounds, is now being leased by the FDA to consolidate its many offices throughout the capital region. The site, when completed, will include one existing building that is to be completely renovated, several new laboratory and office buildings, parking garages, and a central utility plant, which powers the entire site.

The White Oak Campus is owned by the U.S. General Services Administration (GSA), the government's premier acquisition agency. The site was designed by Greenhorn and O'Mara. The CDRH Laboratory, seen circled in red in the image to the left, is to be used as the Center for Devices and Radiological Health Laboratory and Office Building.



Construction on the laboratory began on March 22, 2005, and is scheduled for completion on November 1, 2006 at a cost of $63 million. The architect for this laboratory is Kling in Association with RTKL, with Kling acting as the structural engineer, and RTKL as the MEP engineer, designed the CDRH Laboratory using the IBC 2000 as their primary code. The general contractor, Tomkins Construction, will be delivering the building using a design-bid-build, process with Heerey-Tishman, as the construction manager.

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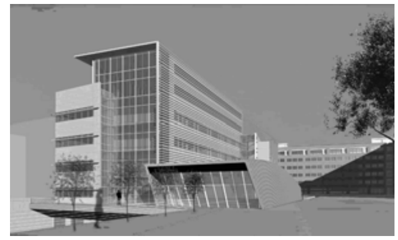
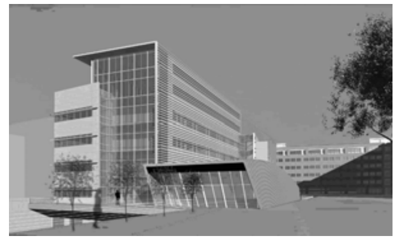


FDA CDRH Laboratory
Silver Spring, Maryland

General Architecture

The CDRH Laboratory building is a 139,805 square foot space made up of a five-story central core, with the top story reserved for the penthouse suite, and a four-story section on the east side. The main entrance to the building is centered on the east side, while the delivery and ground level entrance is located on the north side. The below grade ground floor through the fourth floor are combined use spaces; a perimeter of individual offices and the core of the building are used for laboratory spaces. A large high bay laboratory, displayed in the image below, is the signature for the CDRH Laboratory. This laboratory is to be used for larger equipment, including an anechoic chamber, which is capped with a decorative curved roof on the west side of the building.

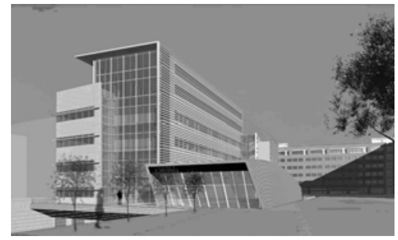
There are three roof systems found at the CDRH laboratory. The first is a planted roof that is at grade over a section of the ground floor. The second level is a sheet metal roof assembly that is over the specialized high bay laboratory space. The third roof system is over the fourth floor and penthouse, and is termed an inverted roof assembly. It is made of metal deck, concrete deck, waterproofing membrane, protection sheet, rigid insulation, filter fabric, and stone ballast. The exterior of the laboratory is mostly made of metal panels that incorporate a metal exterior shell and insulation.



There are also horizontal metal pieces that act as both sun shields over ribbon windows on the north and east walls, and full façade elements that run horizontally the entire height of the building on the west elevation. The north wall also has a cast stone ground floor façade and, as with the south elevation, an aluminum window curtain wall, constructed completely of glazing with and aluminum mullions, dominates the above grade levels.

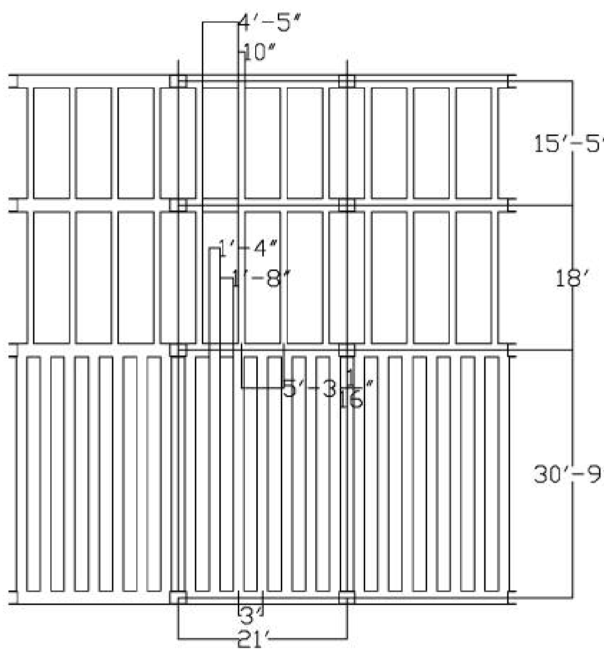
Existing Structural System

The structure of the CDRH Laboratory, with the exception of the roofing systems, is typically made of cast-in-place concrete with a one way slab system. The foundation system is made of a stepped footing lining the entire perimeter of the building with a maximum step depth of 3' (900mm). Spread footings with a typical dimension of 10' (3050mm) X 10' (3050mm) X 3' (920mm) are located below each of the columns and are placed within the main building grid line. The main grid line is made of 15 bays at an average spacing of 21' (6,400mm) spanning in the north-south direction, and three bays, increasing from east to west with 15'-5" (4700mm), 18' (5500mm) and 30'-9" (9375mm) spans. There is a 287KPa

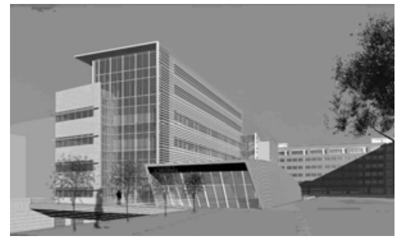


bearing minimum required for all footings. The ground floor varies between a 6" (150mm) and 8" (200mm) deep slab-on-grade with 150X150/MW19XMW19 W.W.F. on 8" (200mm) porous fill on a compacted sub-grade.

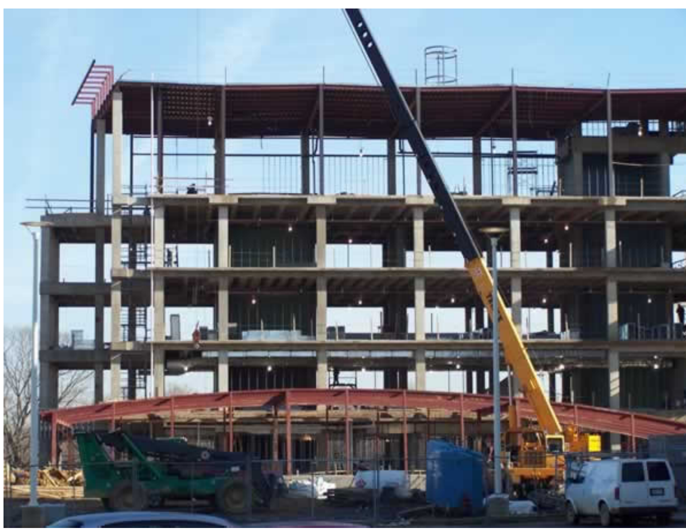
The typical floor system throughout the building is made of 4.5" thick one way slabs, spanning in the north to south direction. There are two typical joist layouts, both of which are pan-joist systems due to the monolithic pour of the slab and joist. The first typical bay is made of 10" wide by 16" deep joists, spaced 5'-3" on center. These joists span either 18' or 15'–5" and are designed with the same requirements as the beams due to their large size and spacing. They are reinforced with #3 top reinforcement and #6 bottom reinforcement. The shear forces are resisted with #3 rebar. The second typical bay is also a pan-joist system with the joist dimension of 16"X16". They are spaced 3' on center and span a distance of 30'-9". They too must be designed like a beam due to their large size and spacing. The top and shear reinforcement is #3 rebar, with the bottom being #8 reinforcement. These bays feed into a system of beams, also poured monolithically. The image to the right is a representation of the typical layout of the one-way cast-in-place concrete bays found in the CDRH laboratory.



The typical beam is 19.7" wide by 20.5" deep and spans 21'. The reinforcement at the

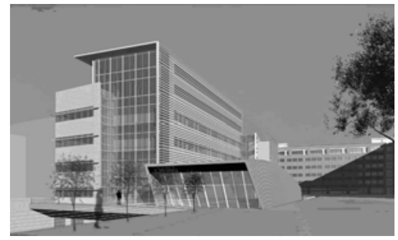


midspan is comprised of 3 - #9 rebar with endspan reinforcement of 6 - #9 rebar. The shear forces are resisted with #3 rebar at 6" and then R rebar at 9". All elements used in the pan-joist system, as well as the beams are made of 4000psi concrete. The beams then feed into the typical 24" (600mm) X 18" (450mm) columns, which are made of 5000psi concrete and 6-#8 rebar. This is a fixed connection causing for resistance against moments, which make up the entire lateral resistive system. The total weight of the current system is quite large, at 163.83K per controlling (30'-9" span) bay, with a total depth of 20.5".



The current system also has steel construction in the penthouse area with the typical steel column being either W14X122 or W10X73. The steel beams in the penthouse roof, as well as the high-bay laboratory are typically a W14X32, W12X21 with secondary beams typically being either a W10X17 or a W8X15.

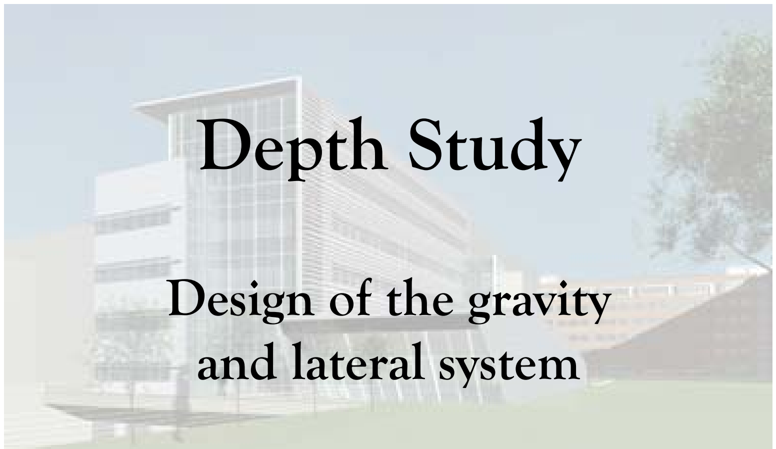
In the greater Washington, D.C. metro area one will find a great deal of concrete construction. This is due to the height restriction found in the District itself, and the ability to increase the number of floors because of a thinner structural sandwich than typical steel



construction. Although the Silver Spring area is not under this same height restriction, the location does play a part in the local skilled labor and customary design in the area, utilizing the high demand of concrete.

The high density of concrete is advantageous to control vibration, which is a major concern in a laboratory situation. There is also no need for fireproofing and the use of normal weight concrete helps to protect against blasts. Other forms of blast protection are progressive collapse beams which are 20" (500mm) X 30" (760mm), used at the north end of the building. This is the location of the loading dock (the location most susceptible to an attack). The beams were made to be large enough to support the load of the building for a short period of time if the center columns were removed.

For more information on the current Laboratory structural system please refer to Technical Report 1.



Depth Study

Design of the gravity and lateral system

as well as considerations of the following:

Vibration Controls
Blast Controls
Height
Weight and Foundation
Fireproofing

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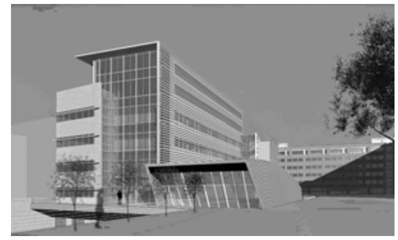

FDA CDRH Laboratory
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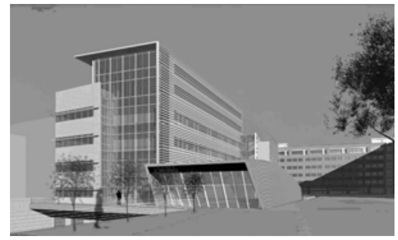
Introduction

Due to the vibration stability caused by concrete construction, the main section of my building was designed with cast-in-place concrete. The construction of this building using concrete was due to many considerations; it allows for an extensive amount of vibration control and is a common construction product in the greater D.C. metropolitan area. There is also no need for an additional lateral resistive system beyond the "moment frame", due to the monolithic nature of cast-in-place concrete and the low lateral forces caused by the long and stout shape of my building, allowing for an open floor plan. All these reasons solidified the choice to use concrete in the construction the CDRH Laboratory. The importance of an open floor plan in a building of a government laboratory comes from the government's desire to easily change a floor's layout every few years, with as little of the superstructure imposing on the layout of the interior walls as possible. When there are no cross braces or shear walls needed, the only structure that requires attention when changing the floor layout are the columns.

I have chosen to complete my thesis by designing the structural system of the CRDH Laboratory using steel design rather than concrete. This is due to the possible cost savings found with steel construction, as well as the great amount of time savings that is possible with this project. This project also lends itself well to steel design due to the use of steel currently for the penthouse construction and roof systems. By using steel there is a reduction of two major trades currently being used on site, down to just the steel trade with a few concrete laborers for pouring the foundation and slabs. There were a few concerns that will be addressed in my thesis design, including the need to have a design that produces an acceptable amount of vibration control, as well as blast resistance. A lateral system that provides as few expensive moment connections as possible, while not causing interruption in the open layout with cross bracing, is another detail that will be addressed in my thesis. Upon completion of my design of the



structural system, a building that is comparable to the current design must be produced. Therefore, changing the current floor plan, column layout, and exterior openings can only be done if it does not change the final building that is to be presented to the owner. The penthouse and laboratory spaces will not be altered from their current design because they are already designed using steel. They will be constructed in the same manner as the current design to avoid changing the design that is being presented as the final building to the owner.

Loading

Loading for the bays to be studied in this report will be found using the typical laboratory sections of the building.

Dead load values derived from ASCE 7-02, Section 3		
USF2X deck and Concrete:		48psf
Concrete:	150pcf	
Decking (22 or 20 gage):	2psf	
Superimposed:		25psf
Ceiling: Acoustical Fiber board	1psf	
Floor: VCT	1psf	
Mechanical/Electrical:	10psf	
Partitions:	13psf	
Snow load values derived from ASCE 7-02, Section 7		
Snow load (Washington D.C.):		30psf
C_e:	0.9	
C_t:	1.0	
I:	1.0	
P_g:	25	
P_f:	15.75psf (<30psf)	
All live load values come from ASCE 7-02, Section 4		
Live Load:		125psf
Light Manufacturing (Most Laboratory Spaces): 125psf		
Light Storage (Supplementary Laboratory Spaces): 125psf		
Live loads are not reducible (>100psf)		

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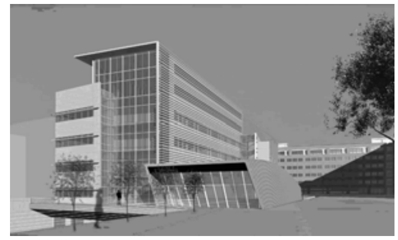

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Wind load values derived from ASCE 7-02, Section 6

Wind Load (N/S):	5.82K (floor 1)
	11.64K (floor 2)
	12.13K (floor 3)
	13.20K (floor 4)
	18.12 K (penthouse)
	11.63K (roof)
Wind Load (E/W):	7.73K (floor 1)
	15.46K (floor 2)
	15.92K (floor 3)
	15.92K (floor 4)
	16.96K (penthouse)
	14.53K (roof)

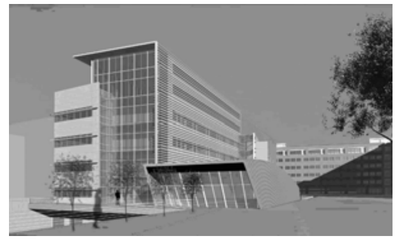
Occupancy type:	II
Importance factor:	I
Exposure factor:	B
Enclosure classification:	Enclosed
Internal pressure coefficient:	0.18
Topographic coefficient:	1.00
Basic wind speed:	90mph

Seismic load values derived from ASCE 7-02, Section 9

Seismic Load (equal in both directions):	295.72K (base shear)
	19.04K (floor 1)
	38.08K (floor 2)
	57.12K (floor 3)
	98.00K (floor 4)
	83.48K (penthouse)

Response modification factor:	R
Occupancy Factor:	I
Seismic use group:	I
Seismic design category	B
Site Class Definition:	C

For detailed information on how to determine wind and seismic forces please refer to Technical Report 3.



Throughout my thesis I looked at the following loading combinations provided by ASCE 7-02:

1.4D
1.2D + 1.6L + 05S
1.2D + 1.6S + (0.5L or 0.8W)
1.2D + 1.6W + 0.5L + 0.5S
1.2D + 1.0E + 0.5L + 0.2S
0.9D + (1.6W or 1.0E)

The controlling condition in both N/S and E/W direction is

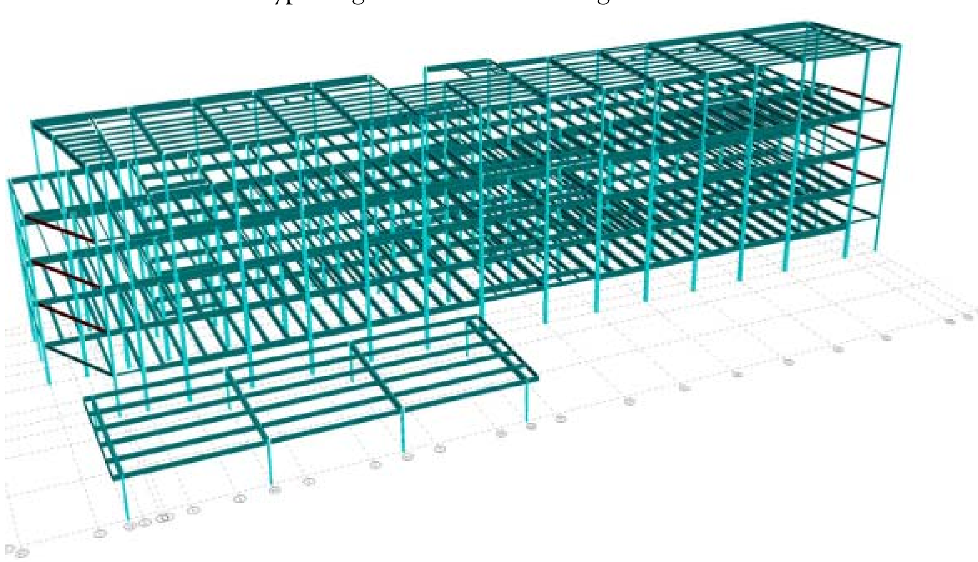
1.2D +1.0E + 0.5L + 0.2S

for all floors except the first floor which was controlled in both directions by

1.2D + 1.6W + 0.5L + 0.5S

Gravity Analysis

The gravity analysis was done using RAM Structural System, 2003, and considers of vibration controls as well as the typical gravitational loading conditions.



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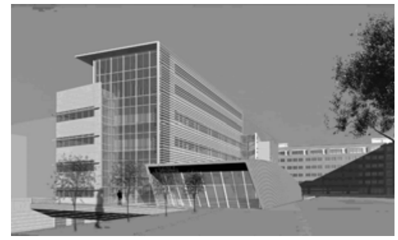

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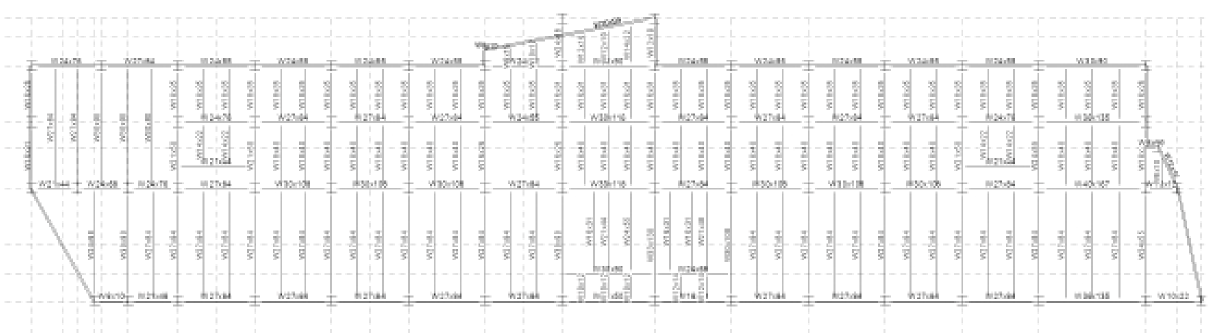
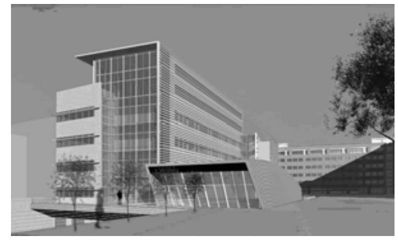
A steel non-composite system with form deck system was used due to the additional vibration control found in larger slabs, and larger members needed to support the larger slabs. Without using shear studs to transfer loads between members, the members also do not transfer vibrations, allowing for additional controls. For the decking, I used the 2001 United Steel Decking manual to find that for the loading condition that I have chosen, I would need a 5" slab over a UF2X form deck made of 22 gage steel. This system will use 44-W2.9XW2.9 welded wire fabric.

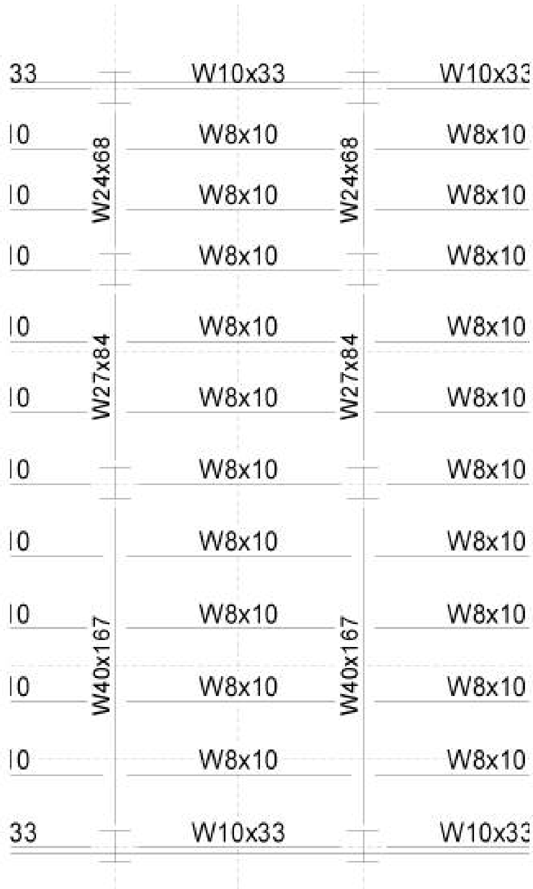
Design A

To begin the design of the CDRH Laboratory, two layouts of a gravitational system were considered. One layout was an exact replica of the current concrete layout. In this layout, all spanning directions of concrete members were replicated in the steel design, with changes in spacing to allow for ideal member sizing. The results of this design can be seen on this typical layout.



The average beam member sizes came out to be very small due to the small spans with the typical member being a W8X10. However, with longer spans and a great deal of loading from the joist members the girdgers are very large, with the typical member being a W40X167. The typical beam and girder layout and sizes can be seen on the next page.



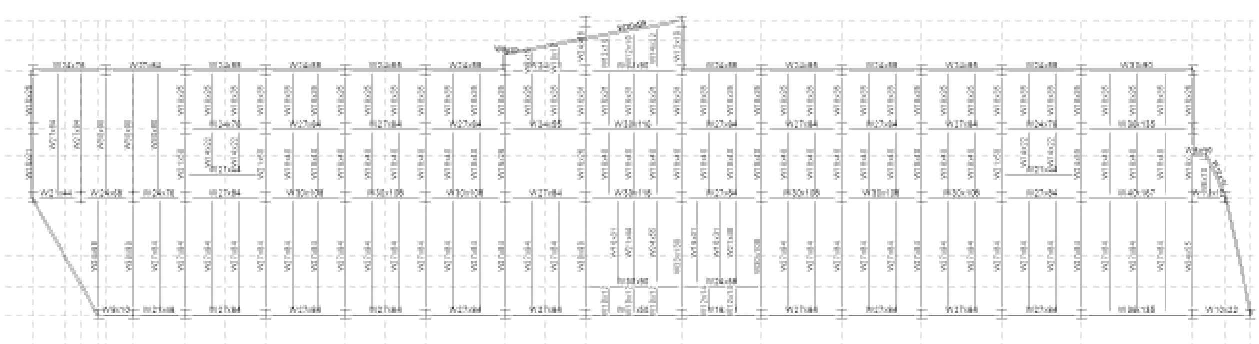


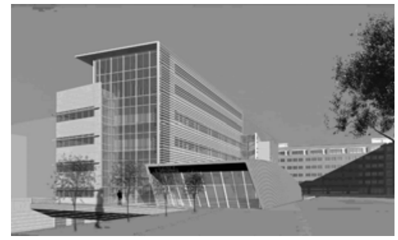
This system, beyond not having a very economical layout, using many uncommon member sizes for the design, also did not provide a very good vibration control system.

The system has a vibration velocity of 6,214 μ in/sec, at the worst case condition of the 30'-9" span, when a person is walking slowly. This velocity only meets the first level of vibration criteria for sensitive equipment (vibration velocity of 8,000 μ in/sec or less), which is used for computer systems, operating rooms, surgery, and bench microscopes at up to 100x magnification. With a vibration velocity of 141,086 μ in/sec, this system will not meet any vibration criteria for sensitive equipment when a person is running.

Design B

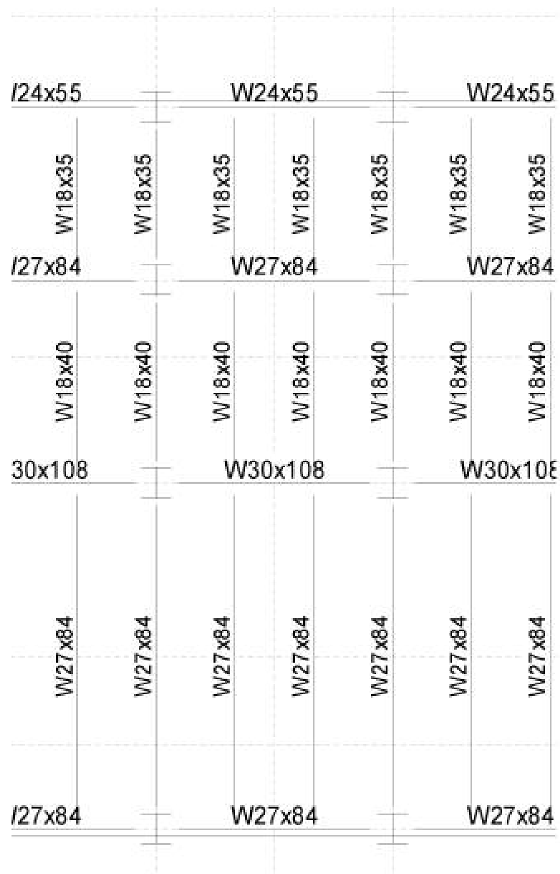
The second layout that I utilized was to span the steel members in the opposite direction as before, and allow for a slightly larger spacing. This can be seen in the layout plan below.



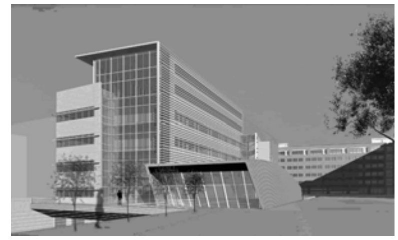


This worked very well with the overall layout of the building because of the typical span of 21' is easily broken into a 3 – 7' section. Due to the slightly longer spans, 20 gage steel would need to be used for the UF2X decking. Allowing for slightly longer spans of the joists, the overall member sizes of beams and girders became more consistent, with the typical beams being W27X84, W16X26, and W18X35, in the 30'-9", 18'-0", and 15"-5" spans respectively.

This system has a more typical layout for a building with these types of spanning and loading criteria, as well as much better vibration control. The typical beam and girder placement and sizes can be seen to the right.



This system has a vibration velocity of 255 μ in/sec, at the worst span condition of the 30'-9", when a person is walking slowly. This velocity meets the fifth level of vibration criteria for sensitive equipment (vibration velocity of 500 μ in/sec or less), which is used for electron microscopes at up to 30,000x magnification, microtomes, magnetic response imagers, and microelectronics manufacturing equipment class C. With systems in the non-critical 15'-5" span, all 7 vibration criteria levels are met with a slow walking person. With a vibration velocity of 5,794 μ in/sec, this system will even meet the first level of vibration criteria for sensitive equipment (vibration velocity of 8,000 μ in/sec or less), when a person is running in the worse span condition.

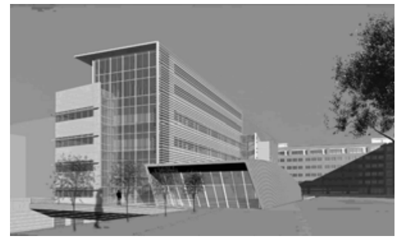


Lateral Analysis

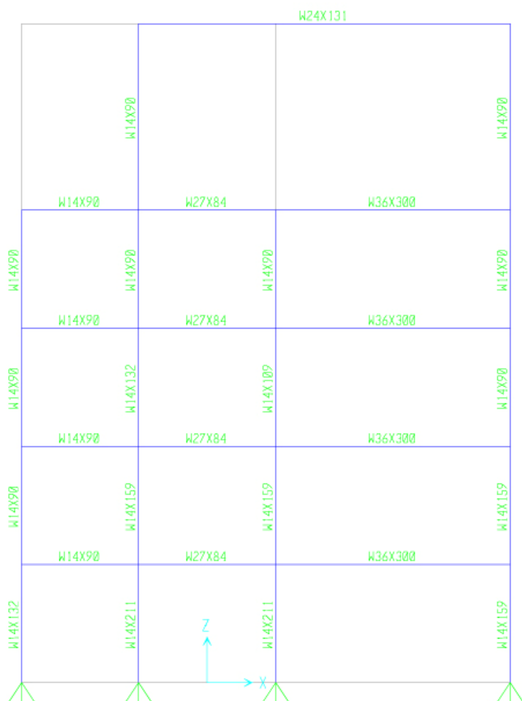
The lateral analysis was designed using SAP2000. The layout of the lateral resistive system was designed to consider the "open floor plan" desired in government spaces such as the CDRH Laboratory. This meant that only moment connections could be used in the interior sections of the building. However, on the exterior sides spanning in the north-south direction of the building, cross-bracing could be used. The ribbon windows did require either braces to be visible in the window, or small braces that could be hidden above the windows. When using two resistive frames, the exterior frames would be used to resist torsional effects caused by unsymmetrical bracing. The columns were placed in the strong direction facing the east and west walls. The wind forces are greater in the north-south direction because of the larger size of the east and west walls for the wind to act against. This fact determined the directionality of the columns, as well as the use of smaller moment frames to resist the lateral loading in this direction. The controlling load combination was a seismic loading for most floors; however, the wind and seismic loading cases were similar for all floors. This is very different from the current system due to the great decrease of building mass causing a large decrease in seismic forces. Because of this minimal difference in controlling loading, the deflection criteria used was h/400. However, if a less conservative method is desired, the less stringent seismic criteria of 0.02h deflection per floor could be used. All lateral resistive systems found in my building were also designed well within the seismic overall criteria of not deflecting enough to cause permanent damage to the buildings systems, or the widely accepted value of h/180.

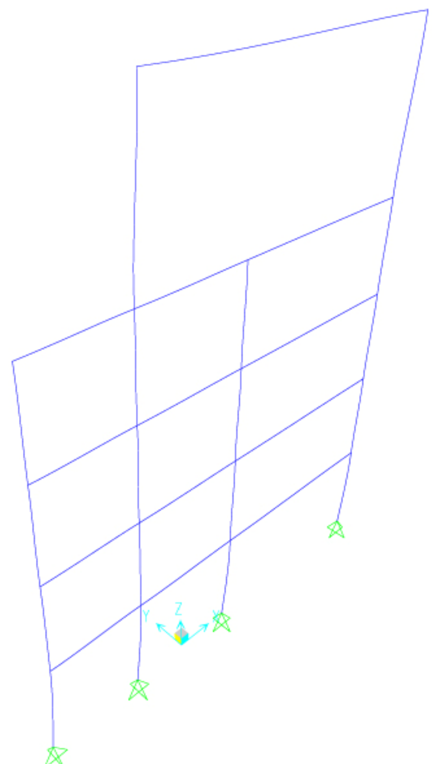
Design A

For the lateral system of the first design, the average column was a W14. Although not as successful in the gravitational system, the larger girders found in the first designed gravity frame allowed for fewer frames to be involved in the lateral resistive system in the east-west

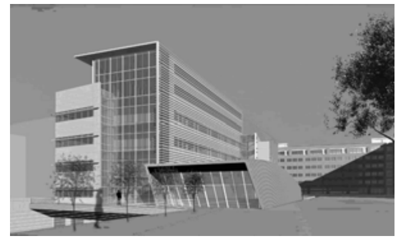


direction. Only three frames needed to have moment connections to be able to control the overall deflection within the criteria of h/400. On the next page are images of the exact member sizes used, as well as the deflected shape.





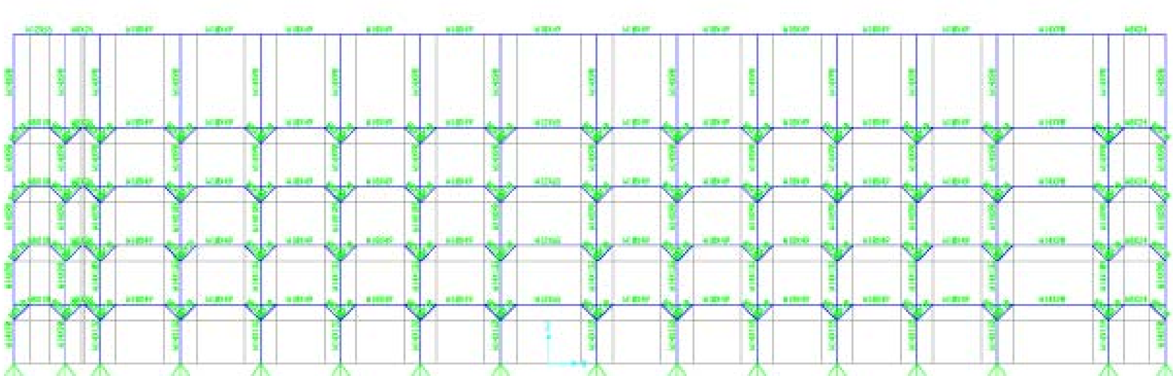
With two moment frames, the total building deflection did pass the h/400 criteria. However, multiple floors did not pass the criteria. To allow for all floors to pass the criteria, an additional frame would need to be installed. With two moment frames, only 52 moment connections would need to be used to resist the controlling lateral loading for the overall building. However, with the addition of the third frame, there would be 78 total moment connections. Bracing could not be utilized in the exterior walls in this direction due to the all-glass curtain wall found on both the north and south ends of the building.

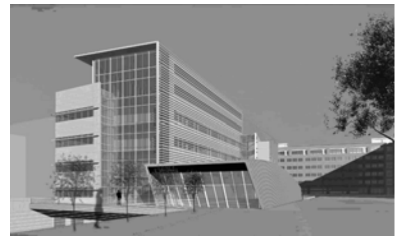


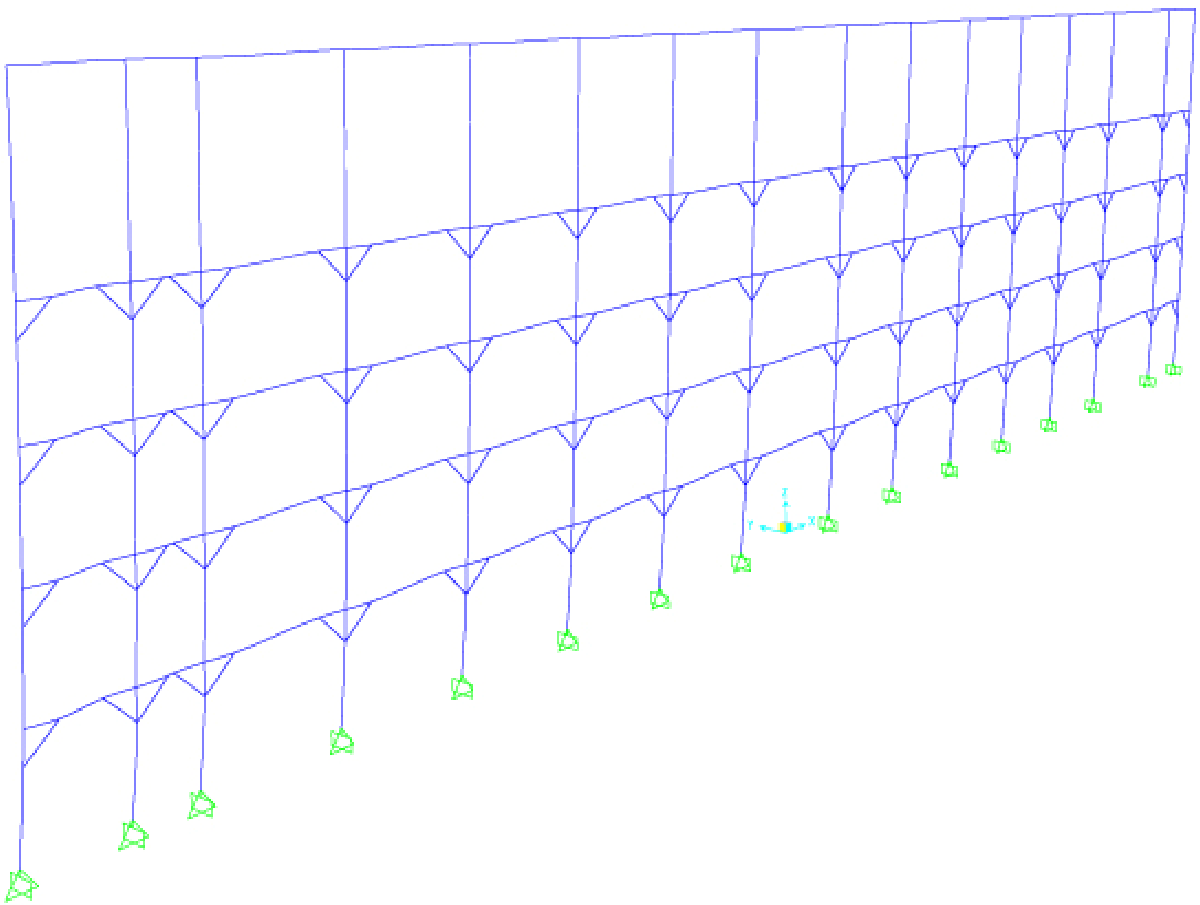
The deflections produced from having three moment frames can be seen below.

Moment Frame Second Redesign		
Story	Allowable Drift (in.)	Story Drift (in.)
1	0.4830	0.4688
2	0.4830	0.2619
3	0.4830	0.2230
4	0.4830	0.1829
Penthouse	0.7599	0.3941
Full Building	3.1749	1.5307

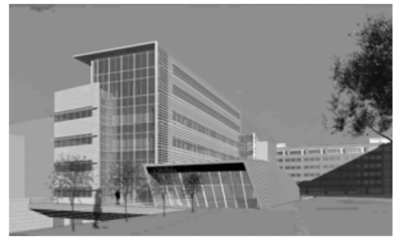
The resistive system in the north-south spanning frames proved to need only corner bracing in just the two exterior frames, even though the beams were smaller than those in the second gravity design. The 6'-0" W12X19 braces were small enough to be able to fit above the ribbon windows found running along the east and west sides of the building. All members, even with the smaller beams used in this design, passes the h/400 criteria if two braced lateral systems were used. The layouts below show that a total of 120 braces and 60 moment connections will be needed for this design, with the same member sizes that were used in the gravity design. The deflected shape can be seen on the next page.







Using this lateral resistive system, there would be no need for moment connections on the lower floors. However, 30 moment connections would be necessary in the penthouse. These moment connections could be eliminated, and replaced with braces similar to those found in the lower levels of the frame. However, due to my conditional statement that the penthouse would not be changed from its original design, this floor was kept as a moment frame. This is where the 60 moment connections for this direction of the building came from.

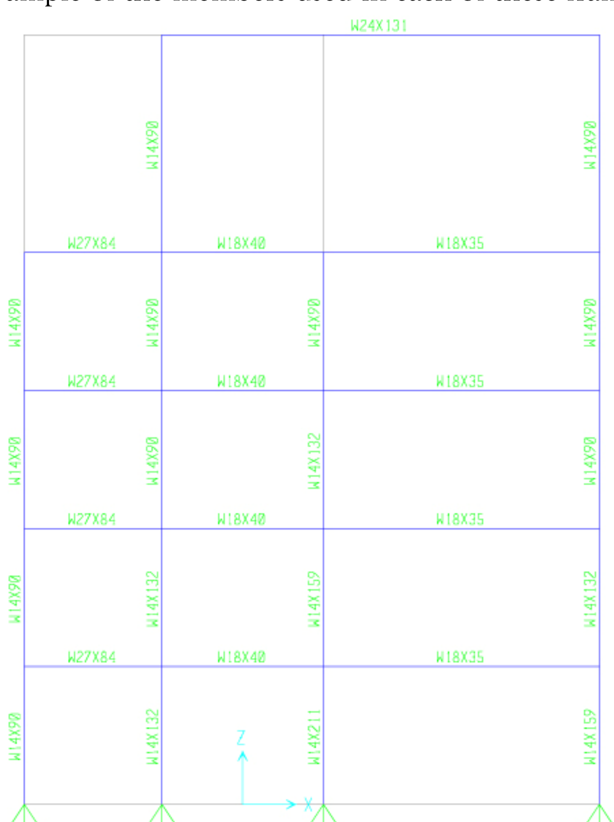


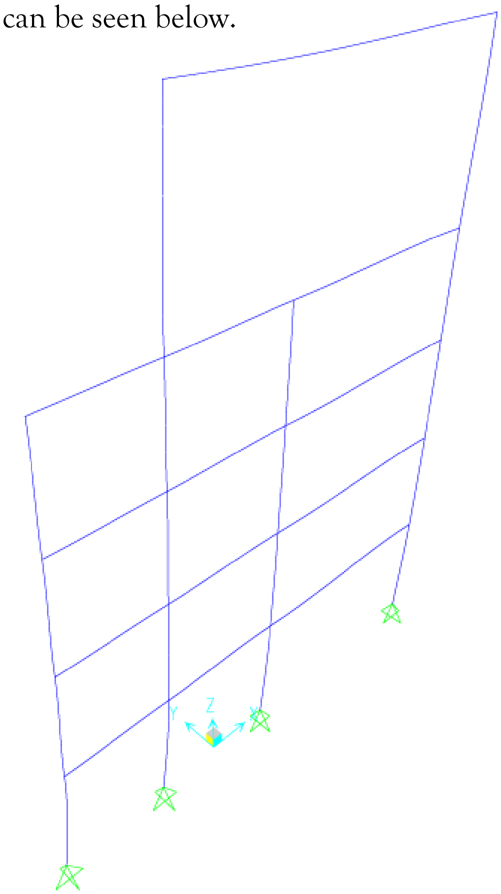
This layout will produce the following deflections.

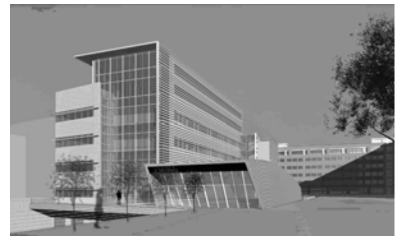
Braced Frame First Redesign		
Story	Allowable Drift (in.)	Story Drift (in.)
1	0.4830	0.3367
2	0.4830	0.2280
3	0.4830	0.1947
4	0.4830	0.1748
Penthouse	0.7599	0.2740
Full Building	3.1749	1.1416

Design B

The second design, due to its smaller girders, will need three additional moment frames in the east-west spanning lateral resistive system, making a total of six moment frames. An example of the members used in each of these frames can be seen below.



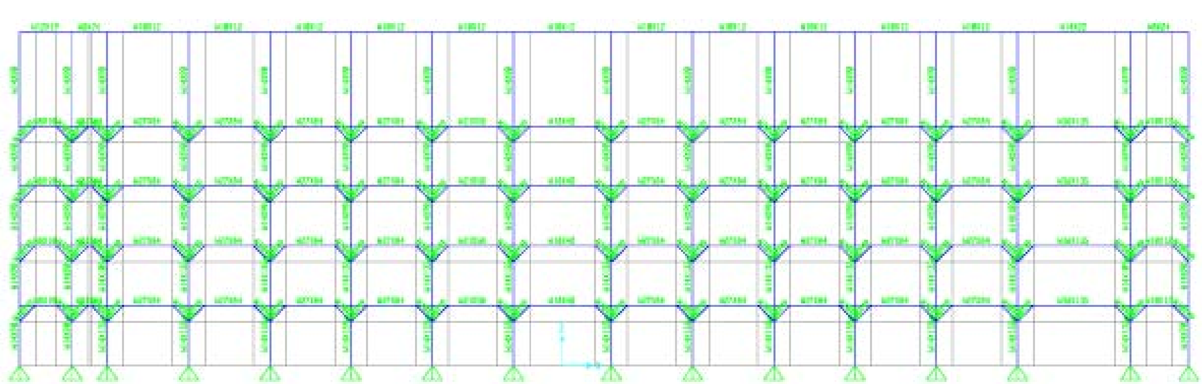


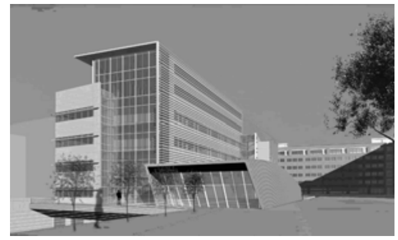


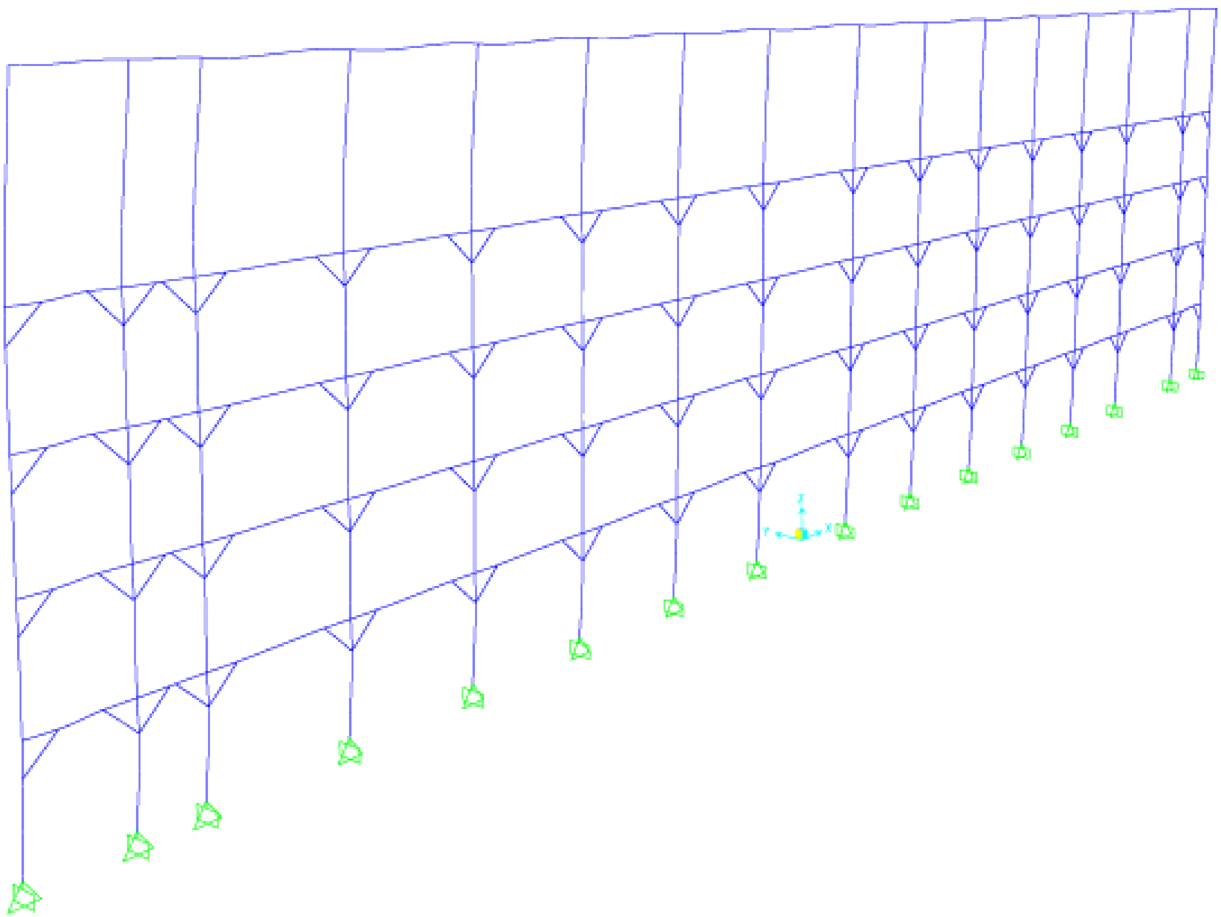
All deflections found in these frames passed the h/400 deflection criteria. Having six resistive frames provides the need for 156 moment connections in the east-west direction. These six bays will produce the following deflections.

Moment Frame Second Redesign		
Story	Allowable Drift (in.)	Story Drift (in.)
1	0.4830	0.4688
2	0.4830	0.2619
3	0.4830	0.2230
4	0.4830	0.1829
Penthouse	0.7599	0.3941
Full Building	3.1749	1.5307

The lateral resistive system in the north-south spanning direction will still need both exterior frames to have 6' knee braces in the corners of the bays. However, due to the larger beams found in this design, W10X49, a common shape in this design, can be used for the bracing members. The layouts below show that a total of 120 braces and 60 moment connections will be needed for the lateral system in this building based on the gravity layout of the second design. The deflected shape can be seen on the following page.

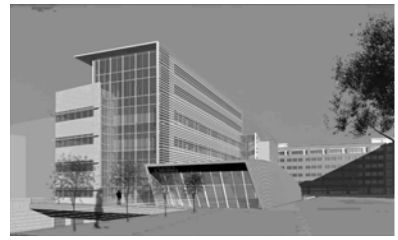






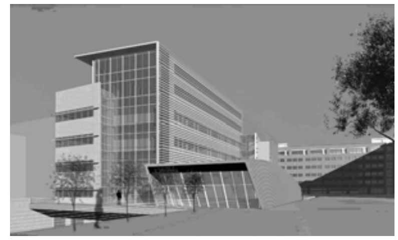
Again, 60 moment connection will be needed at the penthouse level. Below is displayed the displacement of this system as compared to the allowed values of the h/400 criteria.

Braced Frame Second Redesign		
Story	Allowable Drift (in.)	Story Drift (in.)
1	0.4830	0.3740
2	0.4830	0.1973
3	0.4830	0.1845
4	0.4830	0.2108
Penthouse	0.7599	0.7421
Full Building	3.1749	1.7087



Blast Control

Another concern when changing from a more solid concrete structural system to a steel system is that of blast resistance. There are many conditions working in favor of the FDA building against a blast even without additional controls. The location of this laboratory, the center of the limited access White Oak campus, is the first line of defense against a blast. There is also only one road access point on the north end of the building, proving it difficult to introduce a large bomb to the building. Also, the use of parking garages on the campus instead of interior, below grade garages, allows for another major threat to be eliminated, due to the fact that interior columns are much more susceptible to total collapse caused by a blast. The redundancy in the design is also an additional help. However, the long, thin design of the building is more susceptible than that of a square building. The use of normal weight concrete is another positive to blast resistance. However, adding an extra layer of welded wire mesh in the upper portion of the deck will allow reinforcement against the uplift that is caused in many blasts. Another change that can help provide additional protection is to provide moment connections at all joints. Square columns, HSS shapes, rather than W-shape columns are another way to protect against a blast due to their additional resistance to torsional loading. However, to produce a system that is similar to current system, a beam similar to that of the progressive collapse beam had to be designed. The progressive collapse beam is to support the load of the building in case of an explosion, to reduce the threat of progressive collapse and complete destruction of the building. To do this, a beam that can support the load of two bay spans without any deflection criteria will provide the same resistance as the p.c. beams if the center column is lost in a blast. It was found that typically, a W40 shape would be necessary for these beams to support the load of two bays, with a range in sizes from W40X230 to W40X431. The overall cost of a blast resistant system as compared to a no-resistive building of this size would increase structural the costs by 5%.



Height

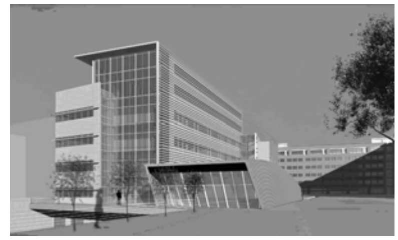
By designing the FDA CDRH Laboratory out of steel rather than concrete, the overall height of the building did increase slightly, due to the increased depth of the floor sandwich. The total height of each floor increased by 8.25", and the penthouse increased by 1', having the total building increasing in height by 3'-9". However, this is not a concern on this site, due to the lack of height restrictions, and only caused a slight increase in wind loads.

Weight/Foundation

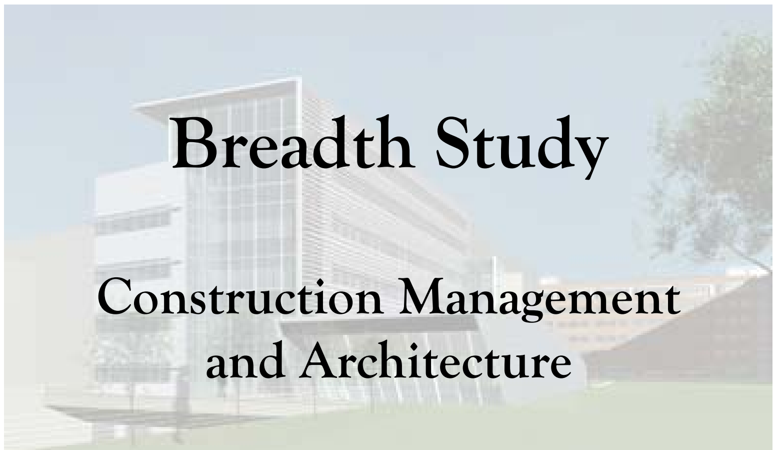
The loading of the building, unlike the height, decreased by using steel rather than concrete. Because of the extremely large concrete joists used in the current system, a great deal of weight was added to CDRH Laboratory. My steel design has a total mass of ¼ that of the original design, weighing in at just under 6 million kips. This decrease in weight caused a need to check the overturning moment, as well as the torsional shear forces. The overturning moment came out to 11,419 ft-kips, while the resistance to that is almost 27 times greater, equaling 307,160 ft-kips. The torsional shear forces were also found to be extremely small and able to be neglected. A benefit that came from the large decrease in weight is a decrease in the size of the building foundation. With a decrease in the overall weight of the building, the overall area of the foundations could be reduced to be $^1/_3$ of the original area, due to the unchanged live loading. This means that the average spread footing could be reduced from a 10X10X3 to 6X6X3.

Fireproofing

Due to the use of steel and the deep decking used, additional fire protection will be needed; there is only 3" of normal weight concrete at the most shallow section of the slab. It is required to have 3/8" of a compatible cementious fireproofing on all decking, with 1" on all beams and girders, and 1-3/8" on the columns, based on the UL certification for a building with construction similar to the CDRH Laboratory.



It is apparent from the use of fewer, more equally sized members and the additional vibration control, that the use of the second design would be the better choice, even with the additional moment connections that would be required in its moment system. However, when looking further into the pricing of each system using cost comparisons, one will see that the second design is the most economical system, even when compared to the current concrete construction.



Breadth Study

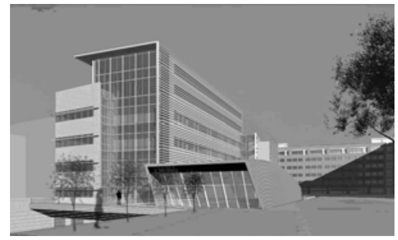
Construction Management and Architecture

CM cost and schedule analysis with considerations of the following:

Current System
Designed System A
Designed System B

Architectural façade design with consideration of the following:

Traditional Brick
Precast Brick
E.I.F.S.



CM Breadth

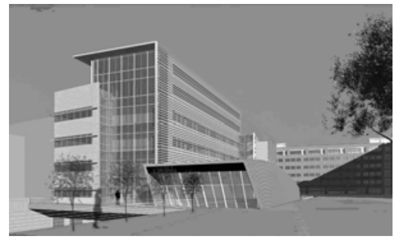
For my first breadth topic, I chose to look at the overall cost of both systems to find which is most economical. I made a take-off of all materials used in both the current system and the proposed systems. I then used R.S. Means, 2006 to find the overall cost.

Cost of Current System

For the original system I looked at the price for each bay based on the cost of the foundation system and slab-on-grade, including all material costs for concrete and rebar, labor of forming, pouring and finishing, and cost of equipment used. I also priced the cast-in-place concrete for all floors except the penthouse. These prices also included the cost for materials, labor, and equipment. The final price section was for the steel found in the penthouse and the cost for the concrete and decking for the roof systems. These again were priced with the same three categories of material, labor, and equipment. I did not look at any cost beyond the structural system, because these are the only changes that I am proposing for my thesis. As I have made it a criteria to provide an equivalent building as currently being provided, there should be no additional costs. The resulting price of the current structural system came to $4,492,275.00. The overall price of the construction was multiplied by 0.975 to reduce the overall cost to make it equivalent to the cost in the greater Washington, D.C. area.

Cost of Design A

For the cost of the first design, I looked at the cost of a reduced foundation size, due to the lower weight of the building, pricing all the same material, labor, and equipment as in the current system. I also priced all of the steel members, decking, concrete used in slabs, and reinforcement needed. These again were priced based on material costs, the labor for installation, and equipment required to complete the job. After including the D.C. price reduction, the



overall cost of this building came to $3,799,940.00. However, this system is not equivalent to the system that was provided using steel construction, due to the low vibration criteria, as well as no blast protection.

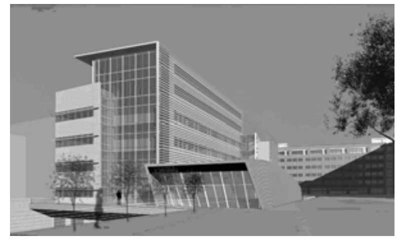
Cost of Design B

The second design, however, is more equivalent to the current system, with its increased vibration control. The overall price, looking all the same elements at the first design, came to a total of $3,392,223.00. Although this building allows for a better system, with overall larger members, the price did decrease from design a. The reasons for this decrease are, in part, due to the use of less material overall. There are far fewer large members than the smaller members used in the first design, and no extremely large members, like the ones found in the first design. This price decrease is also caused by the need for fewer connections, due to longer spans and a lower number of beams. However, because of a lack of blast resistance, this design still is not equivalent to the concrete design. Multiplying by the estimated price increase of making a building of this size and nature blast resistant causes the total price of the building to go up to $3,561,834.15. When looking at all four price increases

System	Price	Savings
Current (Concrete)	$4,492,275.00	$0.00
Redesign A (Steel N-S)	$3,799,940.00	$692,335.00
Redesign B (Steel E-W)	$3,392,223.00	$1,100,052.00
Redesign B w/ Blast Resistance	$3,561,834.15	$930,440.85

one can see that by designing the structural system using steel rather than concrete, there is a possible cost savings of 20.7%, even with an equivalent system of blast control and high vibration control.

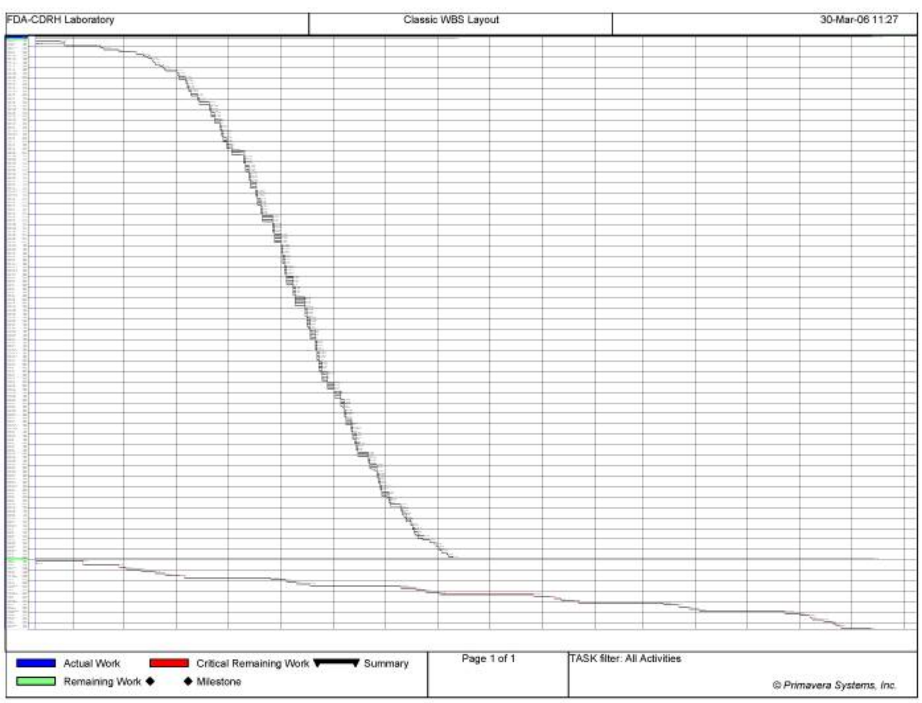
Timothy Mueller
Structural Option
Ali Memari, Advisor

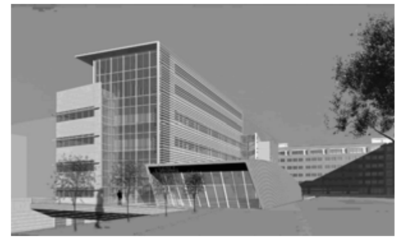


FDA CDRH Laboratory
Silver Spring, Maryland

Schedule Analysis

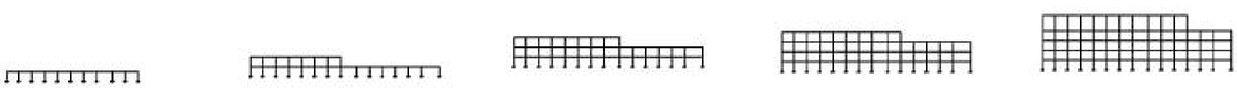
Another benefit of the use of steel is the onsite schedule time decrease. While more lead time would be needed for the ordering and manufacturing of the steel elements, that is not needed for concrete. This job is well suited to a project requiring a large amount of lead time, due to its environment of being in a later phase of a very large campus project. The amount of time that is truly saved is variable, due to the possible use of additional crews. However, using RS Means to find time estimates for the construction of both the current system and designed system, and entering this data into Primavera, 2006, one can see the possibility of a great deal of time savings possible in steel construction over concrete. The schedule below shows the greatly increased slope, due to the ability of overlapping jobs during steel construction (top line), as compared to the low slope, due to the low amount of overlap of jobs possible in concrete construction (bottom line).



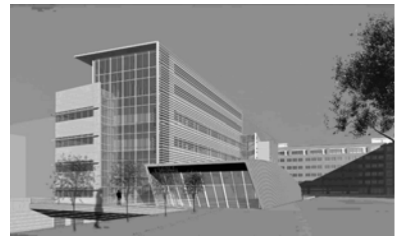


Another way to show that the greatest savings thought scheduling is through steel construction is by showing how the common process is to phase the construction in vertical sections. In this common steel construction method, sections can be completed, then moved. When the next phase is started on the base, the first phases second level can be started as can be seen in the diagram below.

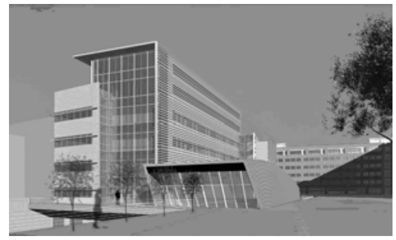
Concrete, on the other hand, is normally constructed in phases based on floors, giving very little possibility for overlap of systems. This construction process is displayed below.



The steel process is even better suited to the construction of the CDRH Laboratory due to its long and narrow footprint. This allows the long direction to be broken into 5 phase sections, which are equal to the five floors found in the laboratory. This way, a majority of the construction is being completed during optimum construction, when multiple phases are being constructed at the same time. If the splicing of the columns were to be changed from every floor, to having columns span several floors, such as having one splice just about the 3rd floor, there would be a reduction in connections needed. Making two beams, the first being 52' and the second being 38', is a viable layout to allow for less connection to be made, and more support available for the height roof columns. The only downfall with this construction is that the steel erection could not have as great of an overlap, since all floors between splices would need to be completed before continuing to the next level. This may provide cost savings in having less column connections, however, it would cause for time loss, without a great deal of structural benefit.



As stated in the structural depth, the second steel design provided the greatest cost savings. The savings of the second design, as compared to the first, although using larger members, provided for less members overall, and less connections. Both steel versions also provided a cost savings over the extremely expensive concrete structure. This is, in part, provided by the large cost of forming all of the cast-in-place members that make up the columns, girders, beams, and decking of the current system. This savings was also provided by having a vibration controlled system that was sufficient for the project, without having to provide massive concrete members. After looking at the possible construction schedule as well as the process, the CDRH Laboratory was well equipped to be constructed from steel, providing another process in favor of the designed systems.

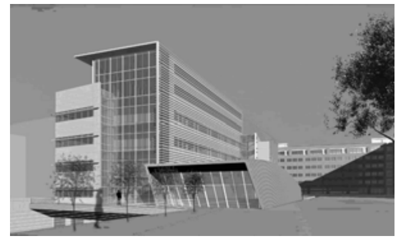


Architectural Breadth

As stated before, the FDA White Oak Campus is made up of one original building, along with a great deal of new construction. The original building has an elegant, solid feel to it, with a stately entrance and a primary construction of a red-orange brick. The new buildings on site were built with similar attributes, including an organized, traditional window pattern, and the same exterior façade of red-orange brick. These buildings have a more updated look, using some aluminum sheathing around upper columns and a glass entrance rather than a stone one. However, the CDRH Laboratory does not have the same architectural dialoged with its historical predecessor. It uses a very modern looking steel façade with ribbon windows and

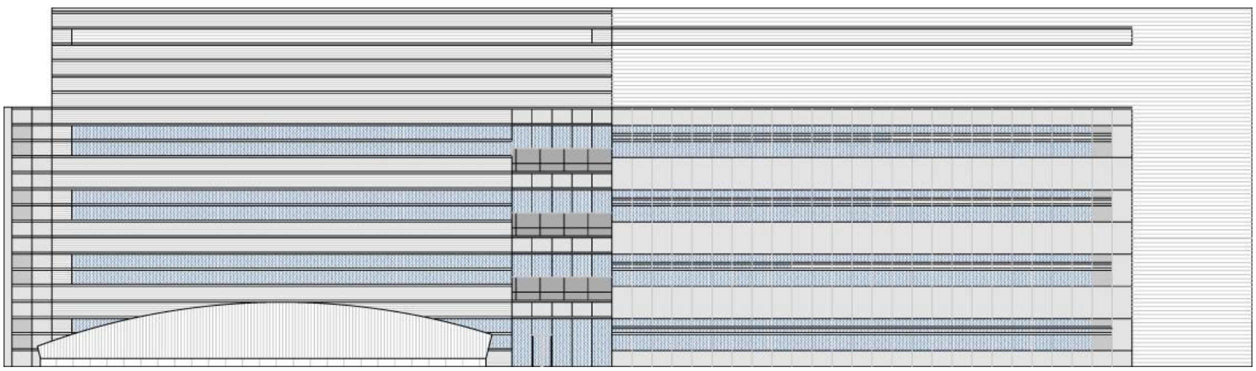
extreme horizontality with its sunshields. This takes away from a uniformed campus feel, which is possible for a location that is being designed and built at the same time. I would like to provide a more centralized image for the FDA campus, which would better pertain to their original intent of bringing their dispersed offices to one central location.

The image above shows the extreme contrast found between the almost space aged looking laboratory, with the traditional brick façade of one of the new office buildings located on site.



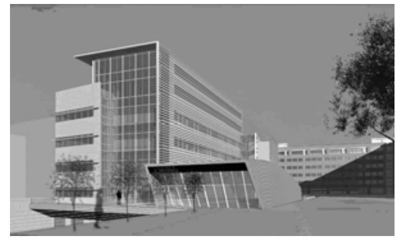
Façade Design

I propose to change the current façade from its horizontal, steel look as seen in this elevation



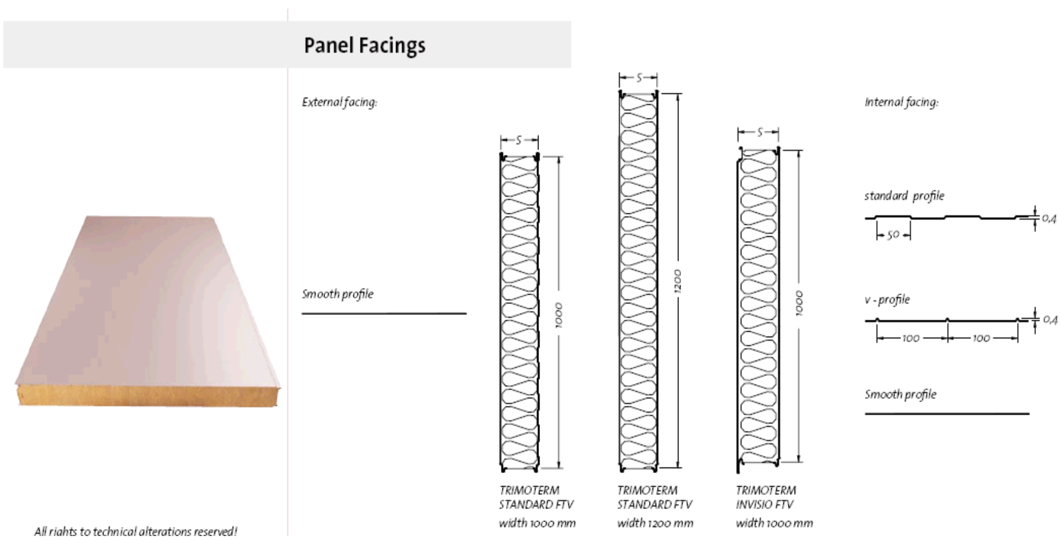
to one with a much more traditional look, that is similar to the office buildings found on site.

The new elevation takes many cues from the surrounding buildings. The upper level aluminum columns break up the extensive amount of blank brickwork that is needed around the 25' penthouse. The ribbon windows are broken up to reflect the traditional window pattern found in an office building. The cast stone balconies work with this traditional façade just as well as with the modern façade, and provide a look of significance for the entrance.



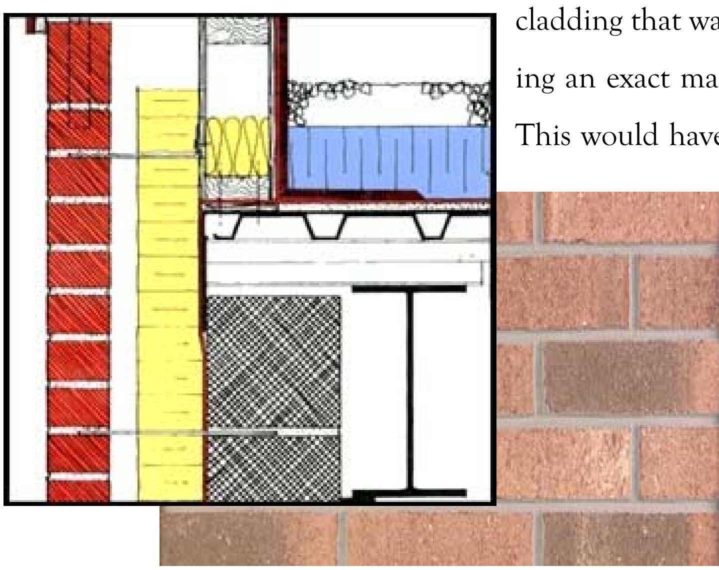
Current Façade Insulated Steel Panels

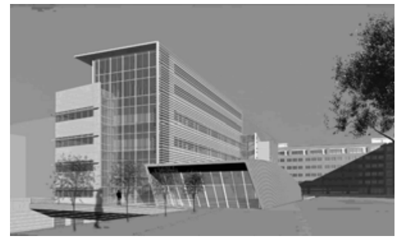
The traditional looking material that would replace the steel panels seen in the current building could be made from many different materials. I will propose three possible materials.



Traditional Brick

The first façade design that I will look at is a traditional brick façade. This is the same cladding that was used on the office buildings, providing an exact match to that of the rest of the campus. This would have a traditional construction process as seen to the far left. The color also seen to the left is similar to the color currently being used on the office buildings on the White Oak Campus. The use of the same color brick would allow for continued continuity throughout the site.

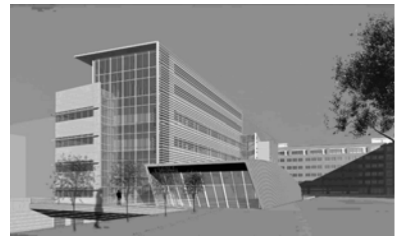


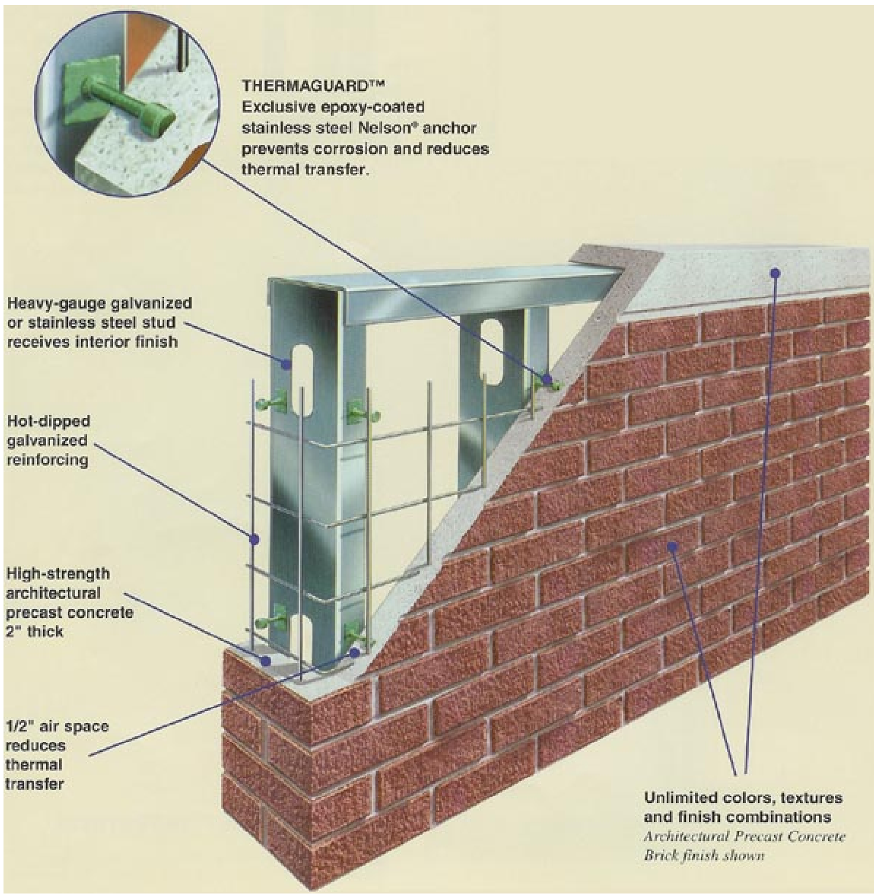


However, there are some downfalls to this façade. Unlike the original steel cladding, this façade does not provide insulation or a simple placement of interior wall finishes, will require additional trades to be on site, and will take a great deal more time to lay all the brick, as compared to just placing the prefabricated, pre-insulated, composite steel panels. Brick construction is also a great deal heavier than the current system, weighing 48psf as compared to the 11psf of the current system. This will also cause an increase in member sizes, not only because of the weight increase, but also because of the increased deflection criteria from l/360 to l/500 needed for the supporting beams. This will cause an increase in the typical beams to change from the current W27X84, W21X50, W18X40, to W30X90, W24X76, and W24X56 respectively. The average cost of this system also increases from the current façade system from $1,086,093.35 to $1,425,856.92. This, along with the structural cost changes, changes the overall cost by $509,516.02. This cost increase is not only because of the expense of material and labor, but the additional use of the brick cladding between the windows, rather than the ribbon window that is currently used.

Precast Brick

The next possible cladding application is a pre-cast brick façade, also know as a slenderwall façade. This façade will provide an almost identical look to that of the traditional brick already found on the campus. However, this cladding has the added bonus of having the ease of construction comparable to that of the current system. It, like the current system, could be fabricated off site, and can easily be put in place. It also has the potential for immediate placement for interior wall finishes, as found with the current system. This system is slightly heavier than the current system, weighing in at 28psf, which would require a larger crane than the current system.

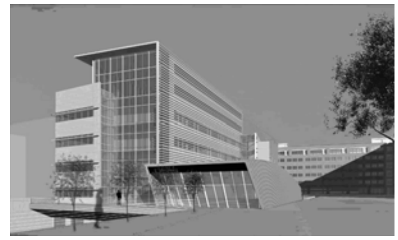




The structural member size increase in the typical beams for this system, as compared to the current system, would produce typical members of W30X90, W24X76, and W21X48. The average cost of this system also raises the façade cost from $1,058,941.01 to $1,418,678.90. This, along with the structural cost changes, causes an overall cost change of $488,900.10.

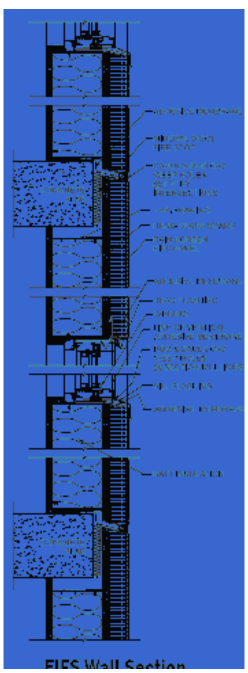
E.I.F.S. (exterior insulation and finish systems)

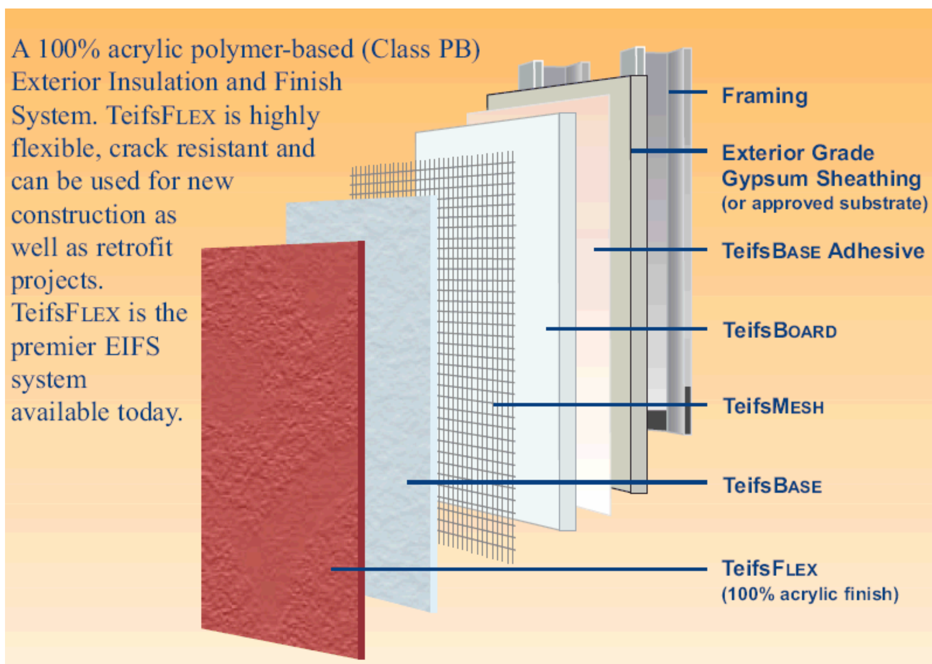
The finals façade that I looked at was an E.I.F.S. (exterior insulation and finish systems) façade. This façade provides an equivalent system to that found in the current structure. It is also prefabricated with the potential for immediate placement of interior wall finishes, like the



pre-cast panels; however, this system also weighs the same as the current system, 11psf. There would be a slight increase in the structural members, due to the increased area of cladding vs. glazing of the traditional layout as compared to the modern one. The new typical member sizes are W27X84, W24X55, and W21X44.

This system does have a reputation for being sub-par, especially with water penetration. This is normally caused by improper installation, especially in residential construction. However, advances in design, which can be seen below, along with proper installation, allows this cladding system to provide equal protection from the elements.





There is one major drawback for this cladding. Its look is not as prestigious as any of the other façades. The price for this façade is less than the current façade, at only $1,028,164.27. The total cost decrease, even with the increase in the price of the structural system, and the additional material needed to make a traditional looking window rather than the ribbon window, is $46,306.63

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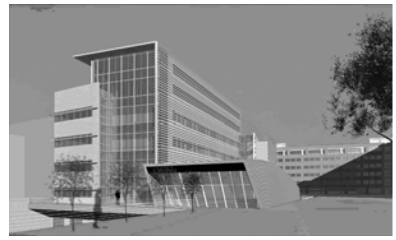

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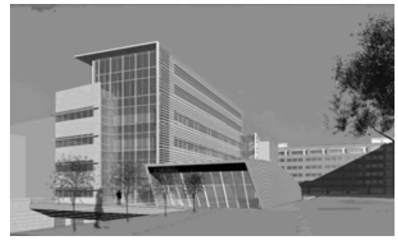
Façade	Cost	Difference	Stuctural Cost	Difference	Building Total	Difference Sum
Steel (Current)	$1,086,093.35		$3,561,834.15		$4,647,927.50	
Brick	$1,425,856.92	$339,763.57	$3,731,586.60	$169,752.45	$5,157,443.52	$509,516.02
Precast	$1,418,678.90	$332,585.55	$3,718,148.70	$156,314.55	$5,136,827.60	$488,900.10
E.I.F.S.	$1,028,164.27	-$57,929.08	$3,573,456.60	$11,622.45	$4,601,620.87	-$46,306.63

As shown above, there are many possibilities for producing a more traditional exterior façade for the CDRH Laboratory. Although some do allow for a great deal of cost savings, the end result on a project such as the CDRH Laboratory is very important. The E.I.F.S. panels provide equal protection for the building as the steel façade, as long as they are installed properly. A drawback of the panels is the "inferior" image that is associated with E.I.F.S., causing these panels to not be a viable solution to a client who is looking for a signature laboratory. The precast panels do not provide a great deal of cost savings as compared to the traditional brick layout, but do provide a great deal of time savings, and result in an almost exact replica of the surrounding buildings façade. The type of materials is an additional consideration. Due to the LEED Silver Rating that is currently being pursued by the CDRH Laboratory, the insulation values of each system as well as the use of renewable and reusable materials are very important. All of the systems can provide comparable insulation values. However, the current system and the E.I.F.S. systems have this already included in their exterior shell, while the precast and brick system must have 6" stud back-up walls to provide room for proper insulation values. Another consideration is the blast control capabilities of each system. The exterior shell of a building is its first line of defense against attack, and the precast and brick systems provide the sturdiest systems in resisting a blast.



Summary & Conclusions

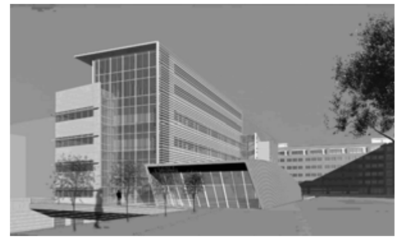
After looking at many types of construction scenarios, my final recommendation for the design of the CDRH Laboratory would be to utilize steel construction with beams spanning in the east-west direction (design b), with an exterior façade made of precast brick. This recommendation comes from many components. First, the cost savings of a steel building as compared to the concrete structure that was designed for this laboratory. Using the same façade with the concrete system, as compared to an equivalent steel structure, including the cost of the façade, results in a 16.7% savings.

Current Buiilding Total (Concrete+Steel Façade)	$5,578,368.35
Proposed Building Total (Steel + Steel Façade)	$4,647,927.50
Total Savings	$930,440.85

There is still a 7.9% cost savings found when comparing a steel structure with a more expensive façade and equivalent blast and vibration control.

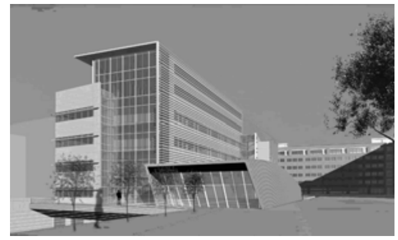
Current Buiilding Total (Concrete+Steel Façade)	$5,578,368.35
Proposed Building Total (Steel + Precast Façade)	$5,136,827.60
Total Savings	$441,540.75

The drawback to the steel as compared to the concrete is the vibration control that could be lost when going from a building with very deep concrete beams to one with conventional steel construction. However, by utilizing a non-composite system in my steel design, additional concrete was used on the slab, and the vibrations were less likely to travel though the building. When looking at the cost of this building, the savings of not using shear studs almost negated the additional cost of the concrete slab. To be confident of the vibration control provided by the steel design, calculations were performed and proved that although the system may not be as resistant as the concrete structure, it was more than adequate for the function of the building. The necessary addition of fire protection to the steel design also did not cause a great price increase, as compared to the concrete construction in which additional fire protection was not



needed. Another addition to the steel building that was not needed in the concrete building was an additional lateral resistive system. This, again, was not a great price increase.

Two large braced frames help a great deal in savings as compared to moment connections, and many of the moment connections found in the building were also needed in the penthouse construction in the concrete building. The moment frames that were needed were only provided in one direction and on less than half of the small frames. Although there were fewer frames required in the first design, the lack of vibration control, as well as the additional cost for many more members completely negates any savings in the lateral system. Additional savings could be found in the decreased amount of materials needed in the foundation design. There was a slight increase in height, causing for the need of additional façade material. This, however, is not a concern for the overall aesthetic of the building, due to the extremely large floor heights already provided in the current design, and the lack of height limitations in the area. Finally, time savings can be easily provided in a steel building, from the use of available lead time, to the ease of bay progression construction as compared to floor progression construction. Blast control of equal comparability to that found in the concrete building can easily be provided in a steel system, at a price that still allows for a great cost savings as compared to that of the current building. Additional blast protection can be found in the solidarity of the precast panels, which I have proposed for the façade of the building. The precast panels, although slightly more expensive than the current system, provide a more unified look to the FDA White Oak Campus, and provide increased safety against attack by having a more solid façade material along with less glazing (the weakest point in a façade against attack). By providing the FDA with the changes that I have recommended in my thesis, as compared to the current design, not only do they save a great deal of money—almost 1% of the $63 million total cost of construction—but they will receive a building more resistant to attack, without giving up required vibration control.



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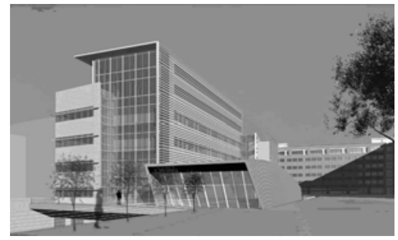

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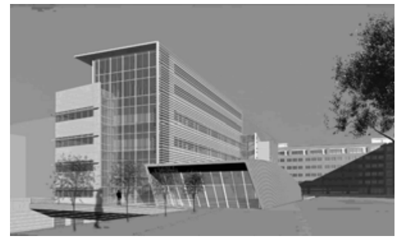

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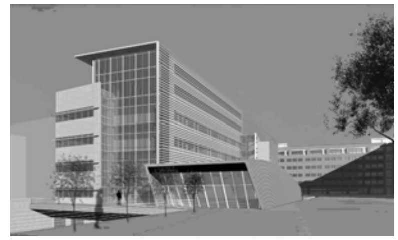

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Calculation Appendix

Detailed calculation, data, and images, involving the following are available upon request:

Load Determinations
Deflection Criteria
RAM input
SAP input
Vibration Calculations
Building Material Cost Estimate
Scheduling Input
Exterior Façade Display