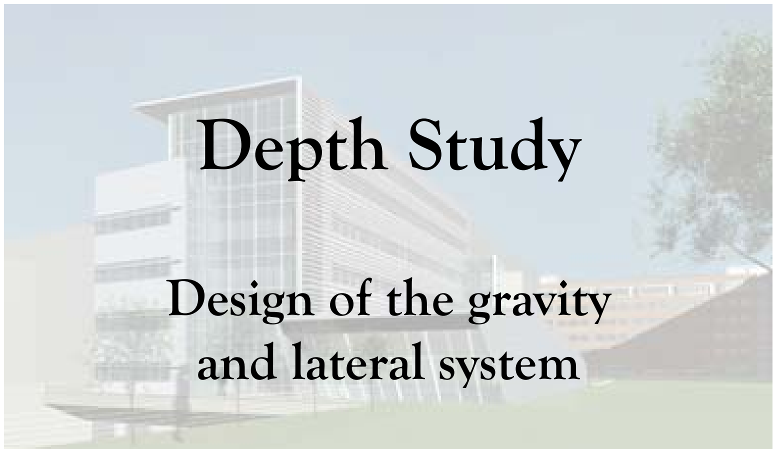

Depth Study

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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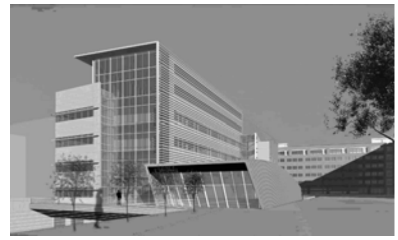
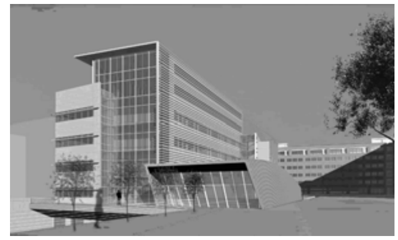


FDA CDRH Laboratory
Silver Spring, Maryland

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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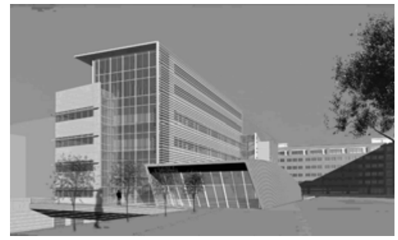

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Wind load values derived from ASCE 7-02, Section 6

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.

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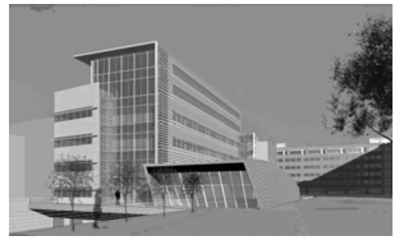

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Throughout my thesis I looked at the following loading combinations provided by ASCE 7-02:

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

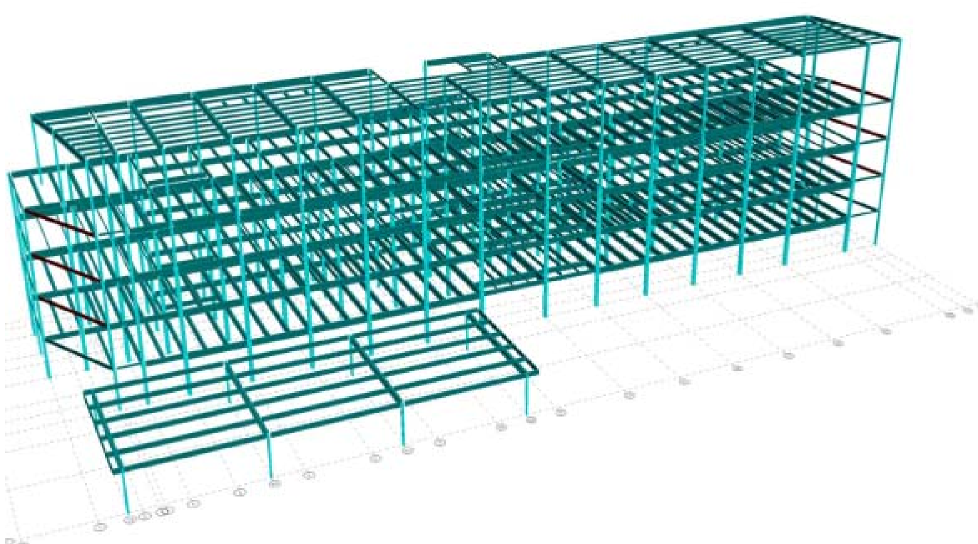
1.2D +1.0E + 0.5L + 0.2S

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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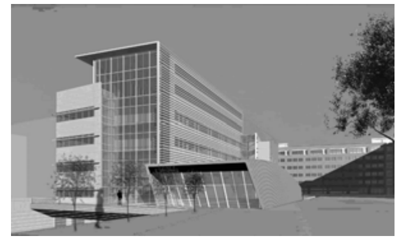

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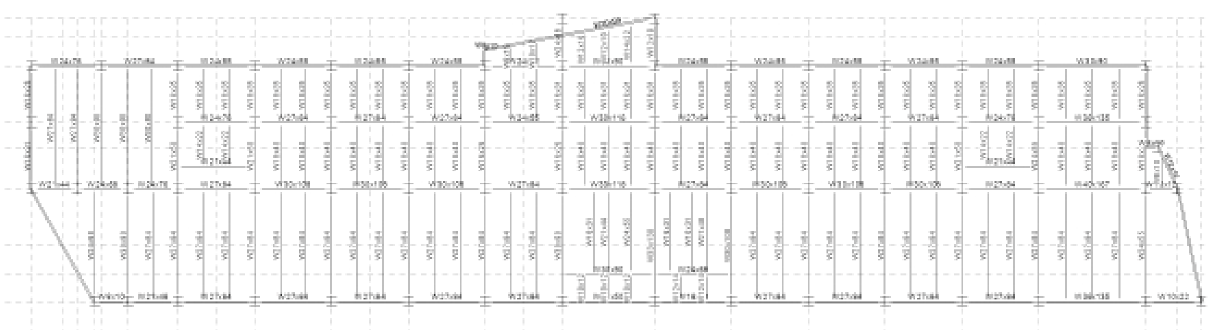
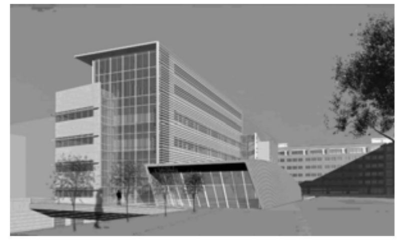
A steel non-composite system with form deck system was used due to the additional vibration control found in larger slabs, and larger members needed to support the larger slabs. Without using shear studs to transfer loads between members, the members also do not transfer vibrations, allowing for additional 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.

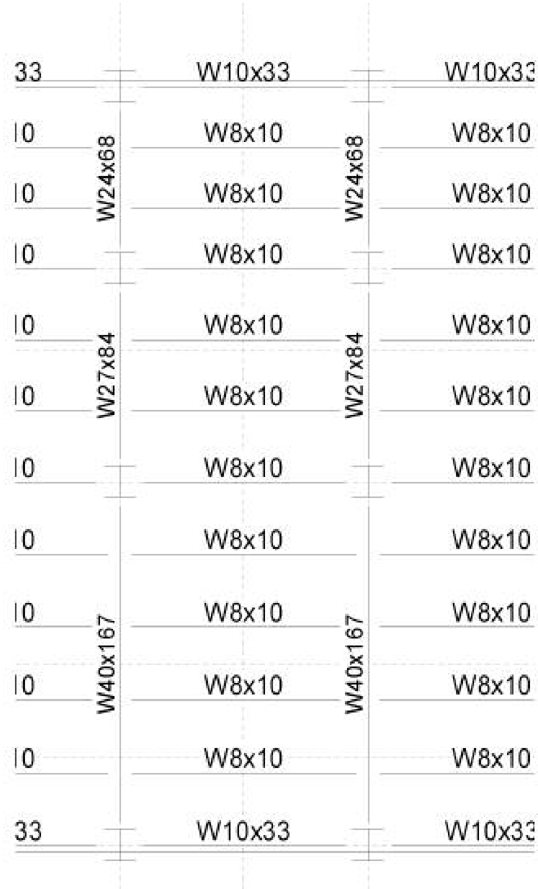
Design A

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.



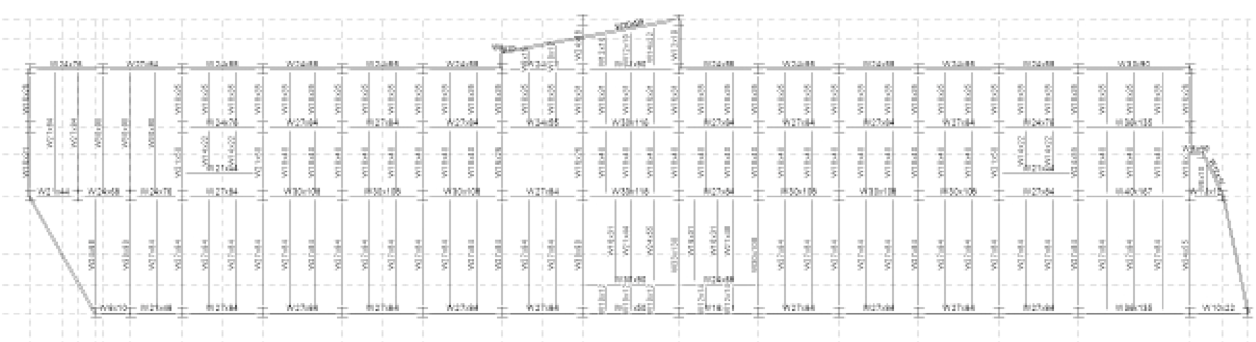


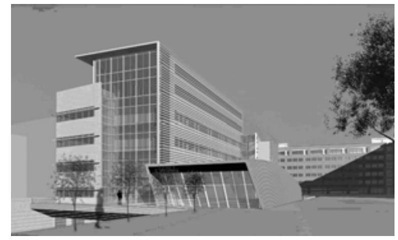
This system, beyond not having a very economical layout, using many uncommon member sizes for the design, also did not provide a very good vibration control system.

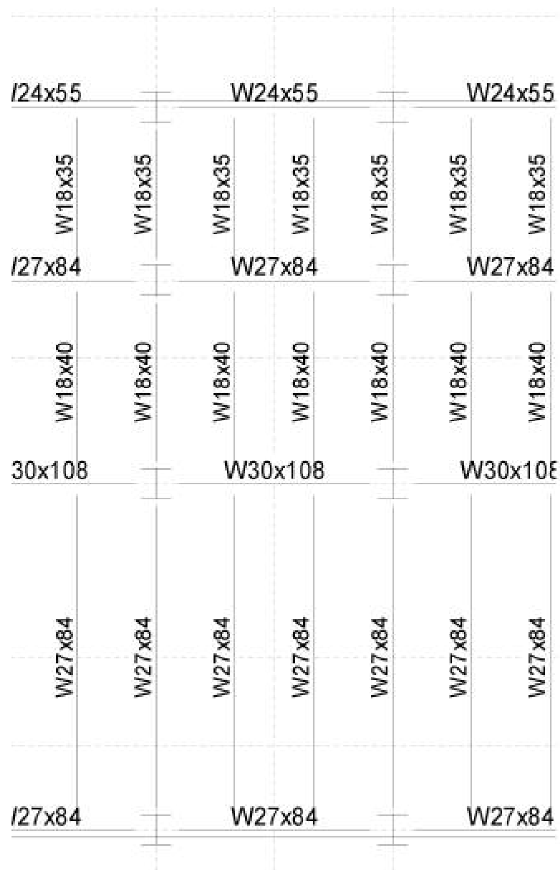
The system has a vibration velocity of 6,214 μ in/sec, at the worst case condition of the 30'-9" span, when a person is walking slowly. This velocity only meets the first level of vibration criteria for sensitive equipment (vibration velocity of 8,000 μ in/sec or less), which is used for computer systems, 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.



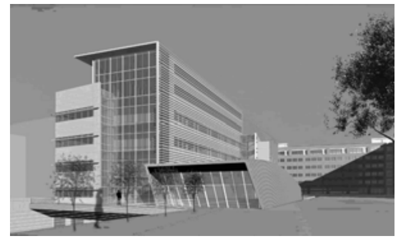




This worked very well with the overall layout of the building because of the typical span of 21' is easily broken into a 3 – 7' section. Due to the slightly longer spans, 20 gage steel would need to be used for the UF2X decking. Allowing for slightly longer spans of the joists, the overall member sizes 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.

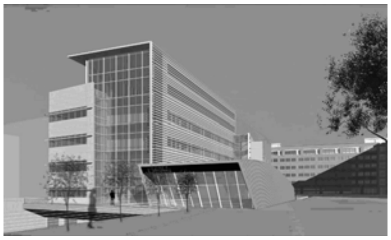


Lateral Analysis

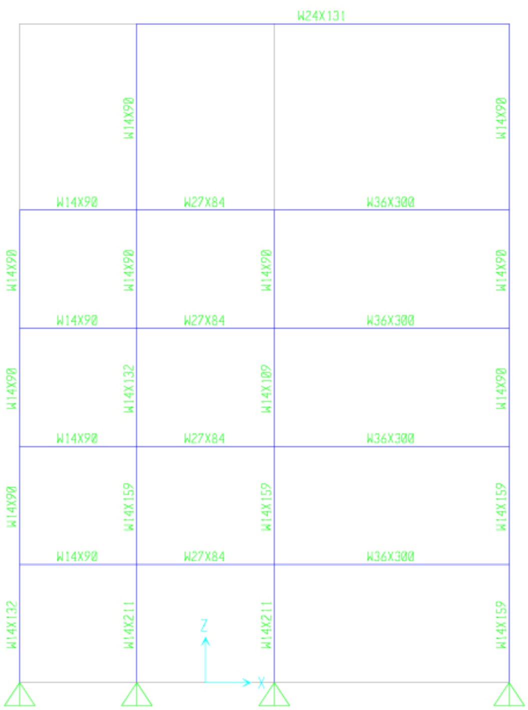
The lateral analysis was designed using SAP2000. The layout of the lateral resistive system was designed to consider the "open floor plan" desired in government spaces such as the CDRH Laboratory. This meant that only moment connections could be used in the interior sections of the building. However, 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.

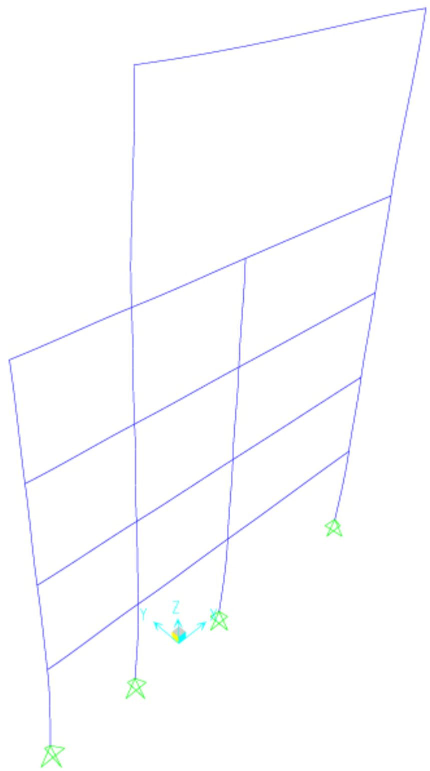
Design A

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

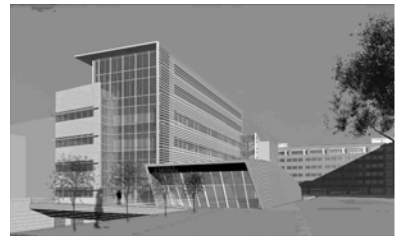


direction. Only three frames needed to have moment connections to be able to control the overall deflection within the criteria of h/400. On the next page are images of the exact member sizes used, as well as the deflected shape.





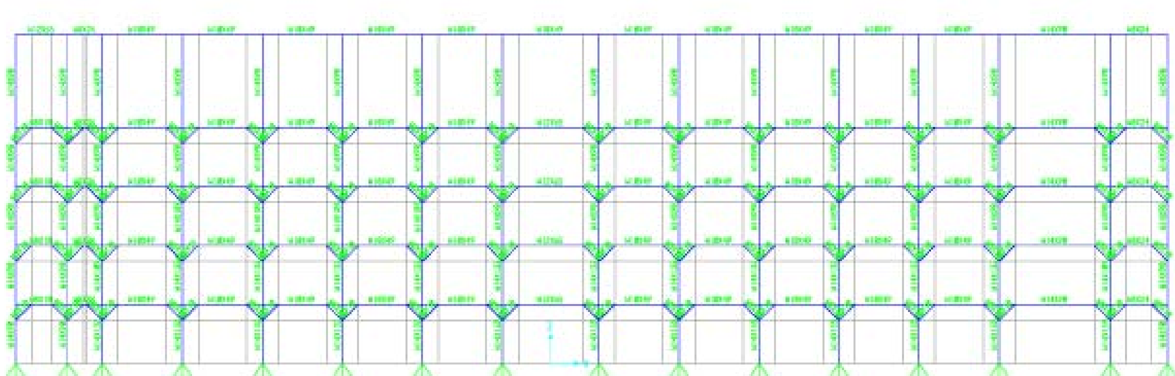
With two moment frames, the total building deflection did pass the h/400 criteria. However, multiple floors did not pass the criteria. To allow for all floors to pass the criteria, an additional frame would need to be installed. With two moment frames, only 52 moment connections would need to be used 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.

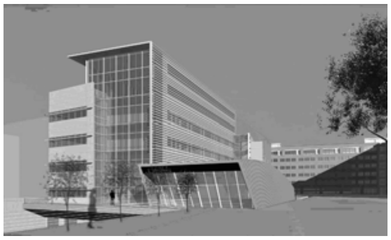


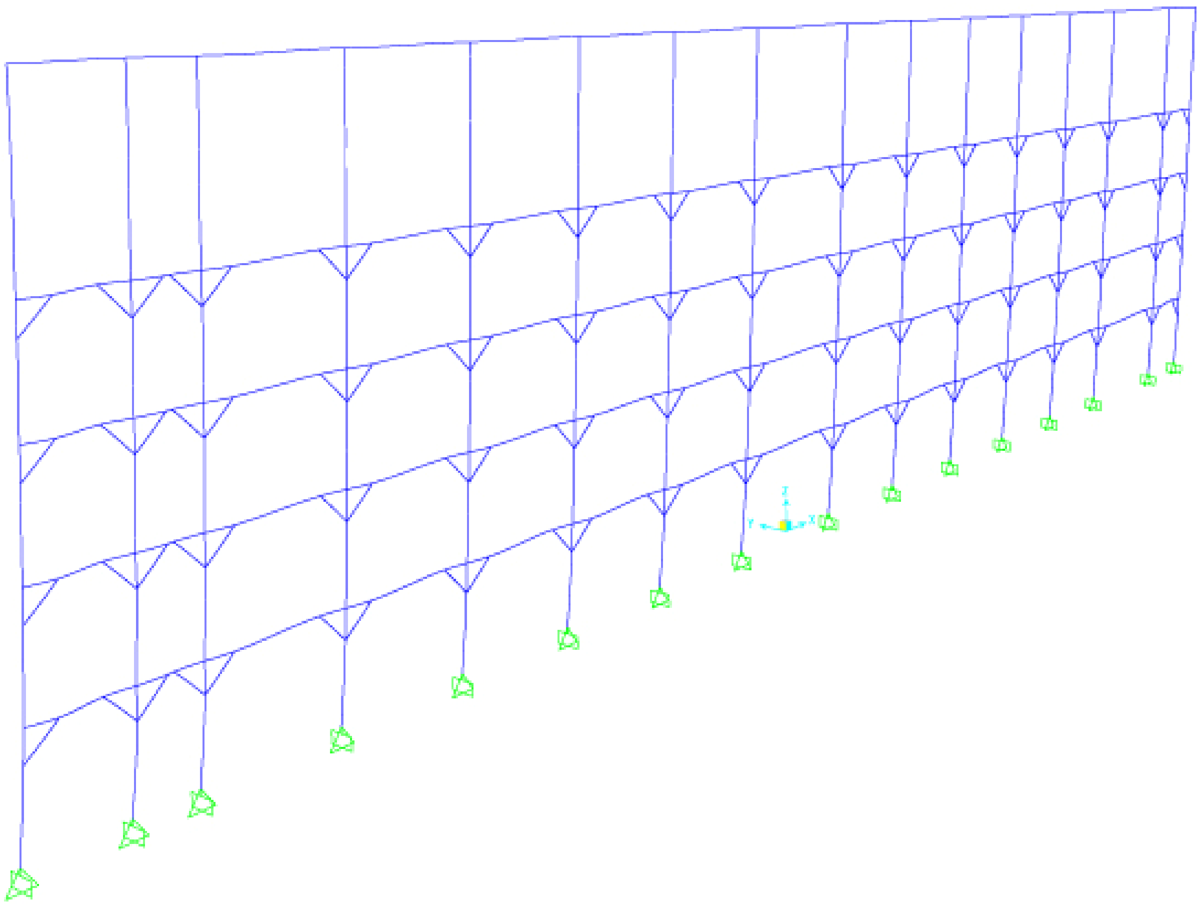
The deflections produced from having three moment frames can be seen below.

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

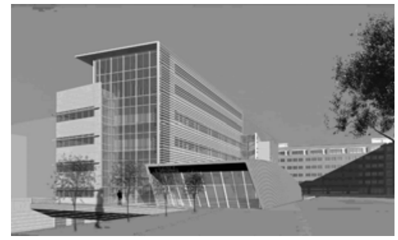
The resistive system in the north-south spanning frames proved to need only corner bracing in just the two exterior frames, even though the beams were smaller than those in the second gravity design. The 6'-0" W12X19 braces were small enough to be able to fit above the ribbon windows found running along 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.

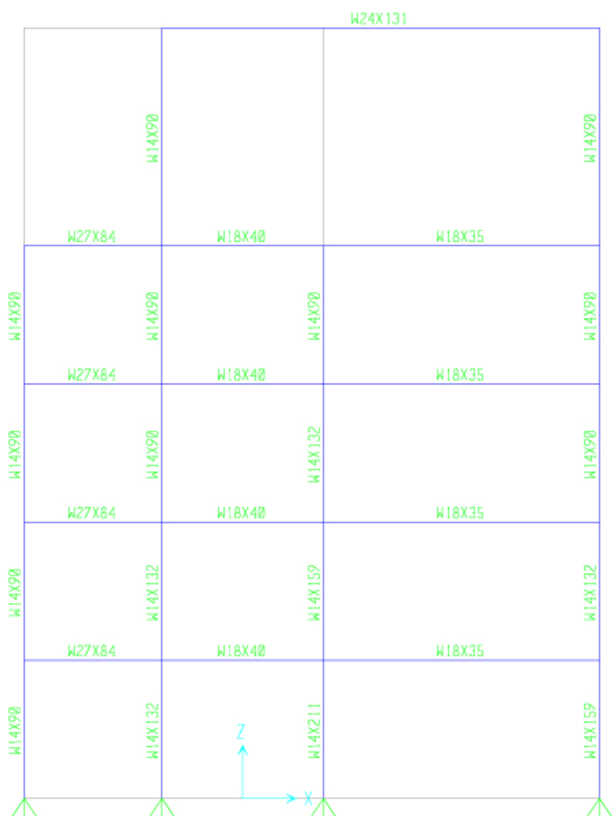


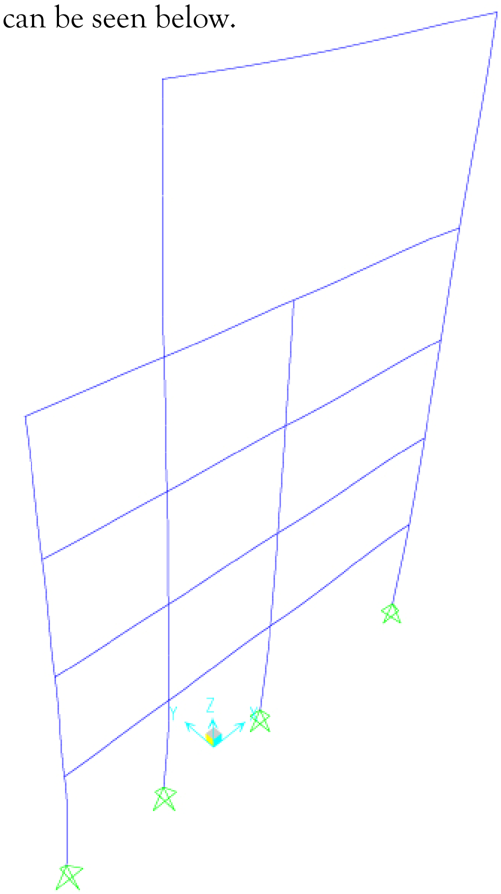
This layout will produce the following deflections.

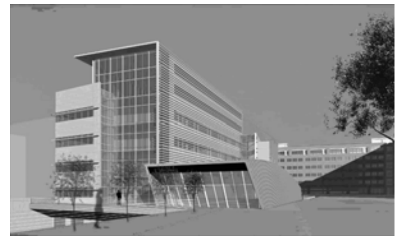
Braced Frame First Redesign		
Story	Allowable Drift (in.)	Story Drift (in.)
1	0.4830	0.3367
2	0.4830	0.2280
3	0.4830	0.1947
4	0.4830	0.1748
Penthouse	0.7599	0.2740
Full Building	3.1749	1.1416

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.



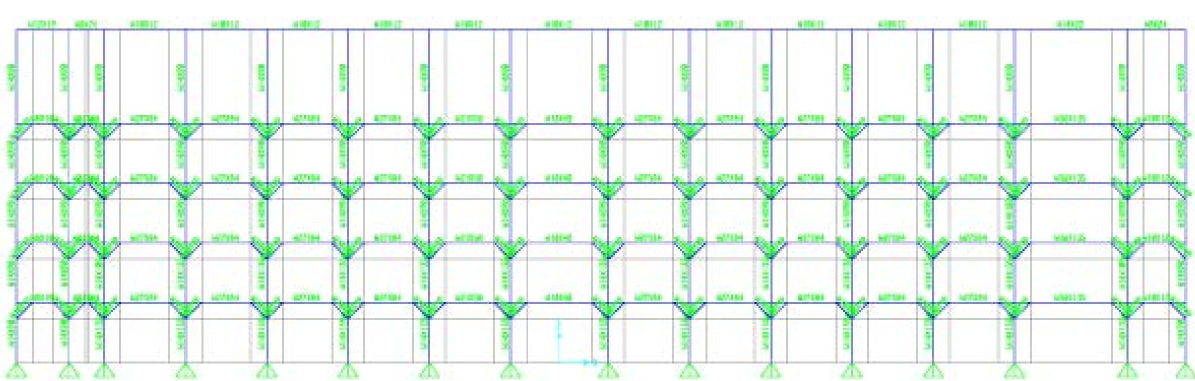


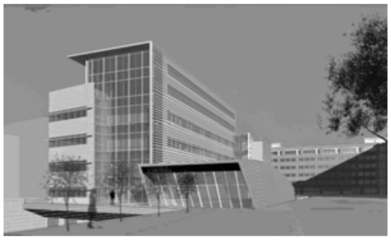


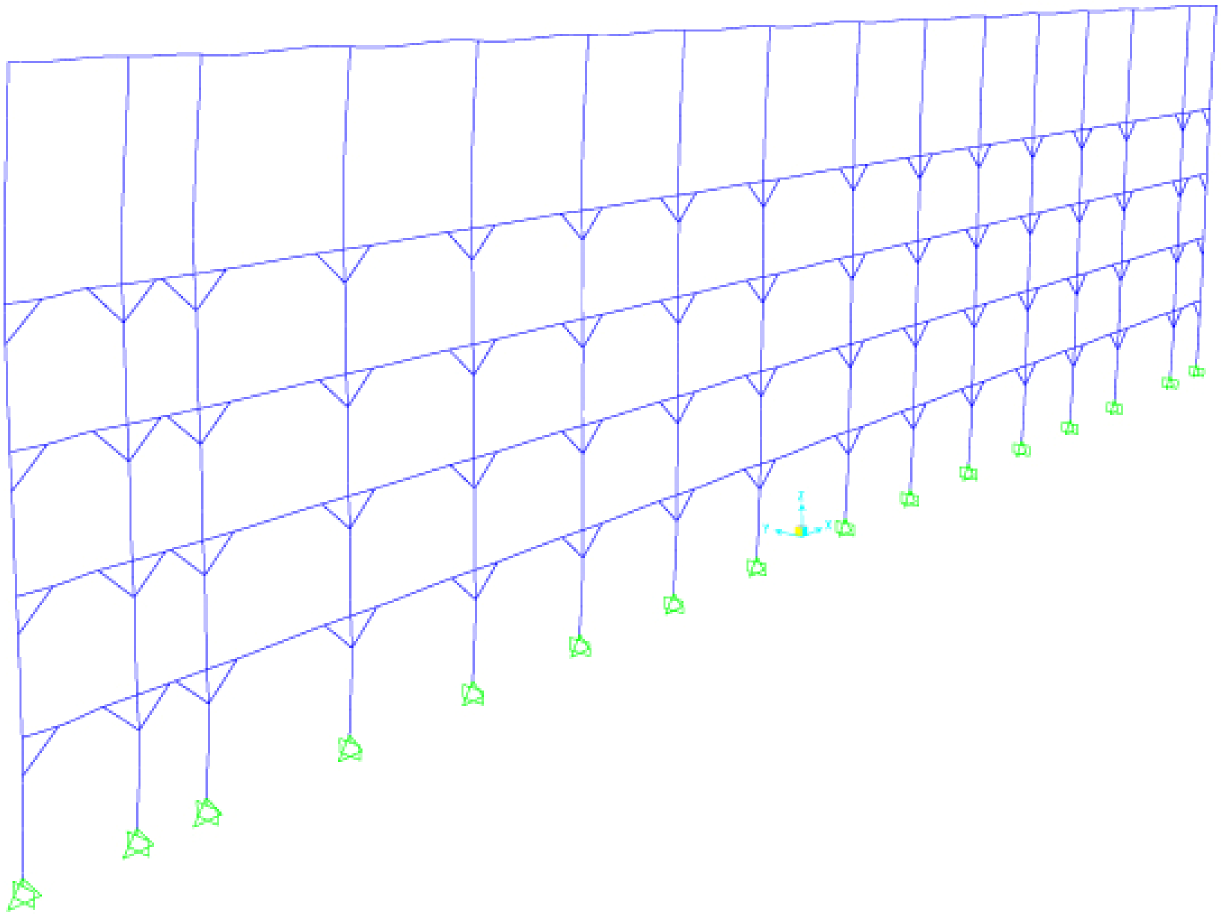
All deflections found in these frames passed the h/400 deflection criteria. Having six resistive frames provides the need for 156 moment connections in the east-west direction. These six bays will produce the following deflections.

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.

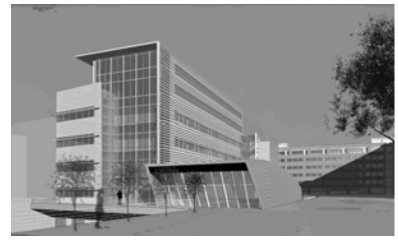






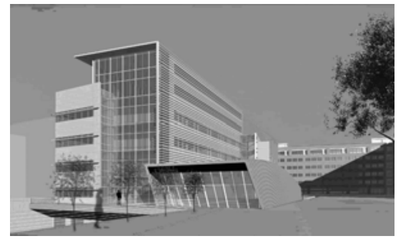
Again, 60 moment connection will be needed at the penthouse level. Below is displayed the displacement of this system as compared to the allowed values of the h/400 criteria.

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

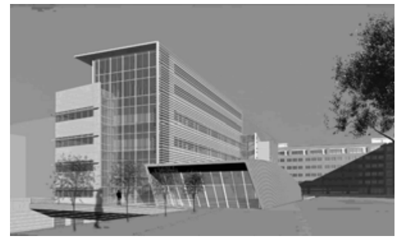
By designing the FDA CDRH Laboratory out of steel rather than concrete, the overall height of the building did increase slightly, due to the increased depth of the floor sandwich. The total height of each floor increased by 8.25", and the penthouse increased by 1', having the total building increasing 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.