# Life Sciences Building

The Pennsylvania State University, University Park Campus
University Park, Pennsylvania

# Final Thesis Report

Lateral Force Resisting System Redesign / Diaphragm Check
Architecture Study
Construction Cost / Schedule Study


**Kirk Michael Stauffer – Structural Option**

AE482W – Architectural Engineering Senior Thesis
Dr. Andres Lepage, Faculty Advisor


4 / 9 / 2008

# LIFE SCIENCES BUILDING

**KIRK M. STAUFFER** STRUCTURAL OPTION

http://www.engr.psu.edu/ae/thesis/portfolios/2008/kms491/

## UNIVERSITY PARK, PENNSYLVANIA

## PROJECT TEAM

OWNER - The Pennsylvania State University

ARCHITECTS - Payette Associates
Bower Lewis Thrower Associates

STRUCTURAL ENGINEER - Gannett Fleming

M/E/P ENGINEER - Bard, Rao, + Athanas

CONSTRUCTION MANAGER - Skansas

## BUILDING STATISTICS

154,000 GSF 6 Levels [97' tall]

CONSTRUCTION DATES - 7/2002 - 9/2004

DELIVERY METHOD - CM at risk

CONTRACT AMOUNT - $37,790,085

## ARCHITECTURE

-Creates "Gateway to the Sciences" at end of pedestrian mall by utilizing a bridge connection to the adjacent Chemistry Building.
-Design relates to campus through material use and various punched fenestrations.
-Modern building technologies implemented as indication of progressive campus.
-Houses general classrooms, research labs, offices, and greenhouses.

## ELECTRICAL / LIGHTING SYSTEM

-15 kV incoming service distributed by 4000 A main switchboard.
-Electric distributed is both 480Y/277 V and 208Y/120 V.
-Indoor diesel 750 kW 480Y/277 V emergency generator.
-Mostly flourescent light fixtures, special fixtures used for labs, darkrooms, greenhouses.
-Natural lighting / sun shades on south curtain wall.

## STRUCTURAL SYSTEM

FOUNDATION - Steel piles with reinforced concrete caps and reinforced concrete spread footings
FLOOR SYSTEM - Concrete slab on composite steel deck supported by composite steel beams and girders.
LATERAL FORCE RESISTING SYSTEM - Steel moment and braced frames throughout building in two orthogonal directions.
COLUMNS - Steel columns individually or as part of lateral force resisting system.

## MECHANICAL SYSTEM

-281,000 CFM outside air supplied by eight air handling units.
-150 tons of cooling by two rooftop air cooled chiller units.
-Heating steam from central steam plant.
-Individual temperature control by 36 variable and constant volume boxes.
-High efficiency filtration on laboratory air intake and exhaust.

THE PENNSYLVANIA STATE UNIVERSITY

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## Executive Summary|

The Life Sciences Building is a mixed classroom, laboratory, and office building at The Pennsylvania State University – University Park Campus, University Park, Pennsylvania. The building is 'L' shaped, 7 floors (97') tall, and 154,000 GSF. The building has concrete floors with a steel frame using composite floor deck, composite beams and composite girders.

Previous assignments have indicated that the existing composite steel gravity framing system is the best of all alternatives to handle varying spans and loads, irregular column placement, and the need to integrate lateral systems into the structure. However, previous assignments also have revealed that the lateral system could be redesigned to be more efficient. In addition, the building was classified as Seismic Design Category "A" which gave little experience using the seismic loading provisions of ASCE 7-05.

This study relocated the building to a location that results in a SDC of "D" to gain experience doing seismic design in a high seismic region. The lateral system was then redesigned with the goal of becoming more efficient than the previous lateral system. It was recognized that moving the building from SDC "A" to SDC "D" would skew the comparison of the efficiency of the lateral force resisting systems. While the analysis and redesign of the lateral system was taking place concurrent studies would be undertaken to examine the effects of the redesign on the building architecture and the construction cost and schedule. A separate structural study had to verify that the gravity framing system (which was to be left unchanged from its existing low seismic design) could handle acting as a structural diaphragm in a high seismic region.

The Life Sciences Building was able to be successfully relocated to the campus of the University of Washington in Seattle, Washington. This achieved the desired result of a Seismic Design Category of "D". The lateral force resisting system redesign would also have to be considered a success. A highly irregular building that was designed for lowest possible seismic environment was able to be redesigned structurally and adapted to be functional in a high seismic location. No modifications needed to be made to the diaphragm (gravity framing) and the building met all code requirements after redesign.

However, the effects of the "move" and lateral force resisting system redesign didn't only affect the structure. Several minor changes in the building architecture and construction cost and schedule had to be made to accommodate the new lateral force resisting system. However, the goal of designing a new structure that could resist seismic forces while changing as little of the existing building architecture as possible was achieved. Very few minor modifications were made to the building as a result of the lateral force resisting system redesign – the original aesthetic aims and vison of the architect were preserved. The cost and schedule study showed that although the building cost would increase (as expected) the increase was a relatively small percentage of the total project cost. Also increases in the construction time due to the seismic detailing requirements of the structure were found to be able to be negated by bringing the steel fabricator on board early in the project. The schedule impacts were also minimized by greatly reducing the number of lateral force resisting frames and by a net reduction in the number of steel members.

## Overall Building Description|

The Life Sciences Building at The Pennsylvania State University, University Park Campus, University Park, Pennsylvania is a six story steel frame structure that is roughly shaped like an "L". The longer leg of the "L" runs in an east – west direction across the northern edge of the site. The shorter leg of the "L" runs north – south along the west central portion of the site. There is also an attached mechanical vault structure at the end of the long leg of the "L" and a two level above grade connection that ties into the knuckle of the "L".

The building can be conveniently broken down into three sections. The first section – referred to herein as the "long leg of the 'L'" – is the part of the building running east – west along the northern edge of the site occurring to the east of column line C. The long leg of the 'L' contains the bulk of the labs, offices and all of the classrooms. The second section – referred to herein as "the knuckle" – is the part of the building that runs east – west along the northern edge of the site and occurs to the west of column line C. "The knuckle" is also the part of the building where the above grade connection to the Chemistry Building ties into the Life Sciences Building. The "knuckle," which is structurally separated from the building, was not considered in any part of this project. The Life Sciences Building was assumed to stand alone without any above grade connection. The third and final section – referred to herein as the "short leg of the 'L'" – is the part of the building that runs north – south along the west central portion of the site and ties into the knuckle at its northern end. The "short leg of the L" contains lab space and a large auditorium on the first floor level.

Other notable features of the Life Sciences Building include the two story above grade connection to the adjacent Chemistry Building which occurs on the third and fourth floors. A one level mechanical vault was constructed with the building at the lowest floor level and is located at the tip of the long leg of the "L" (far east side of building). This mechanical vault is constructed entirely of reinforced cast in place concrete and the roof of the vault is used as a loading dock / truck parking area for the Life Sciences Building. A greenhouse structure is part of the short leg of the "L". The greenhouse is located on the fourth floor which is also the rooftop of the short leg of the "L" (southernmost portion of building). All of these unique design features, except the greenhouse, were assumed not to exist during the study of the Life Sciences Building to help simplify an already complex building.

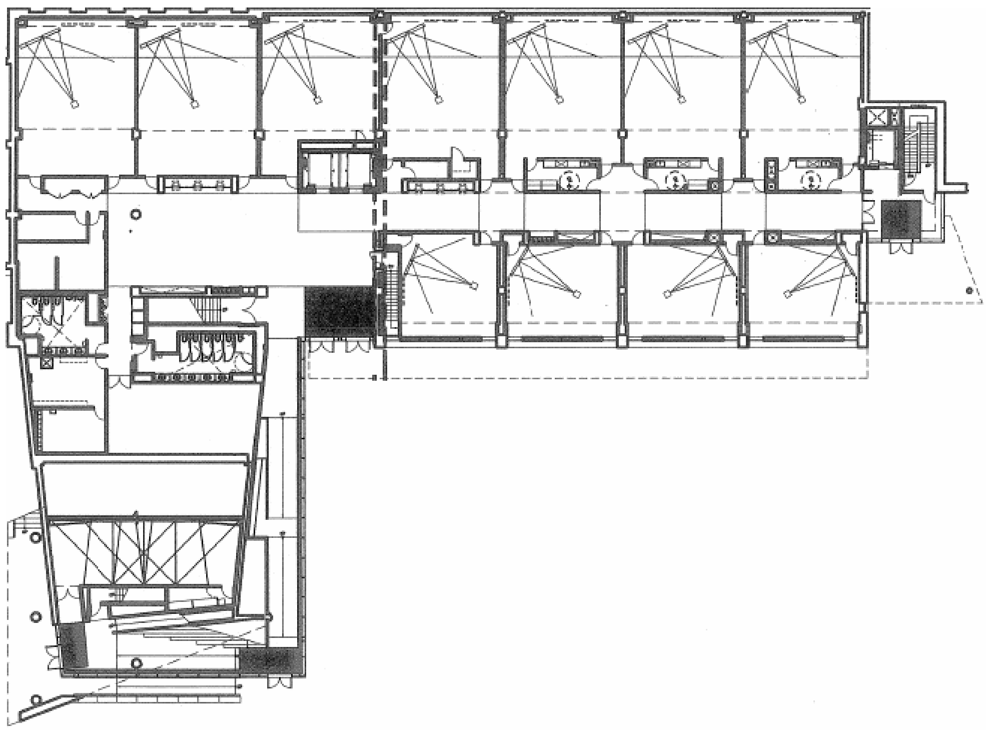
## Overall Building Description (continued)|

Floors of the Life Sciences Building are referred to in this and all other reports by using the following convention:

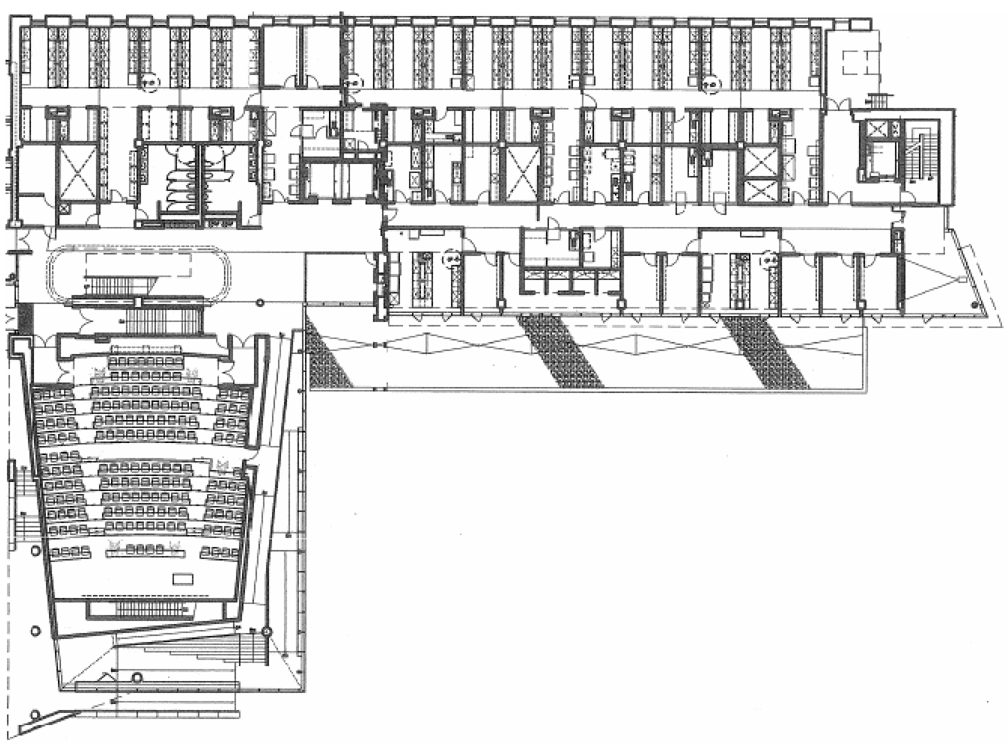
| | | | |
|---|---|---|---|
| B | Basement | 1150'-0" | |
| V | Vault | 1156'-6" | ** |
| G | Ground Floor | 1166'-8" | |
| 1 | First Floor | 1180'-8" | |
| 2 | Second Floor | 1194'-8" | |
| 3 | Third Floor | 1208'-8" | |
| 4 | Fourth Floor | 1222'-8" | |
| P | Penthouse | 1236'-8" | |
| R | Roof | 1263'-0" | |

** Mechanical vault area which is located adjacent to main structure with a roof used as a loading dock area. (Mechanical Vault was not considered in this report.)

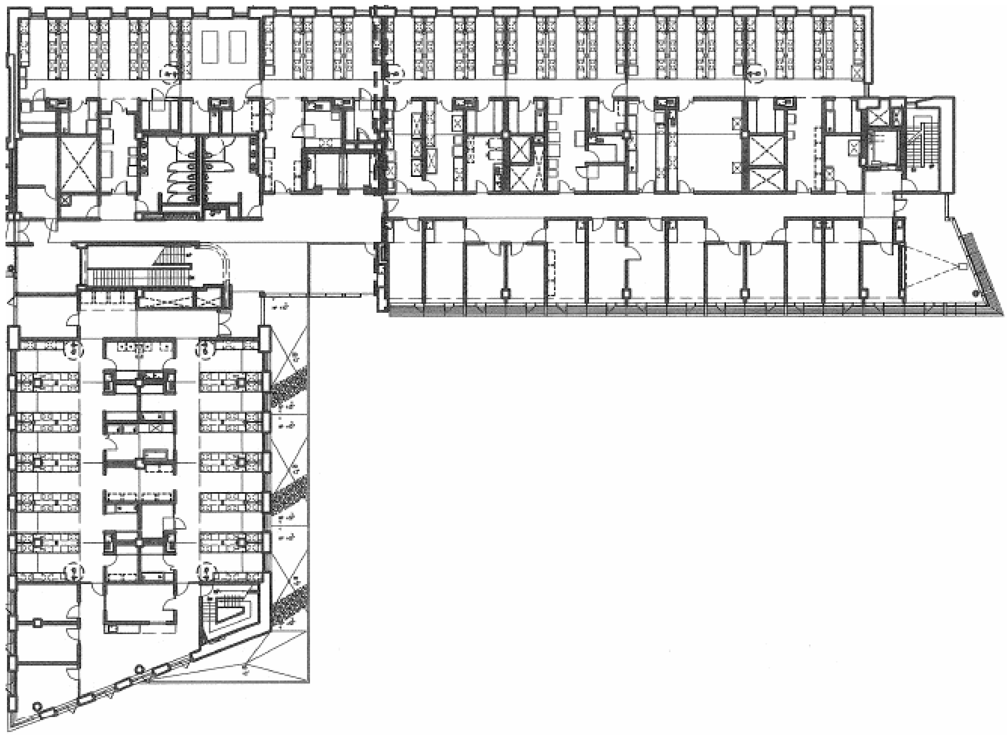
## Architectural Floor Plans – Ground Floor|



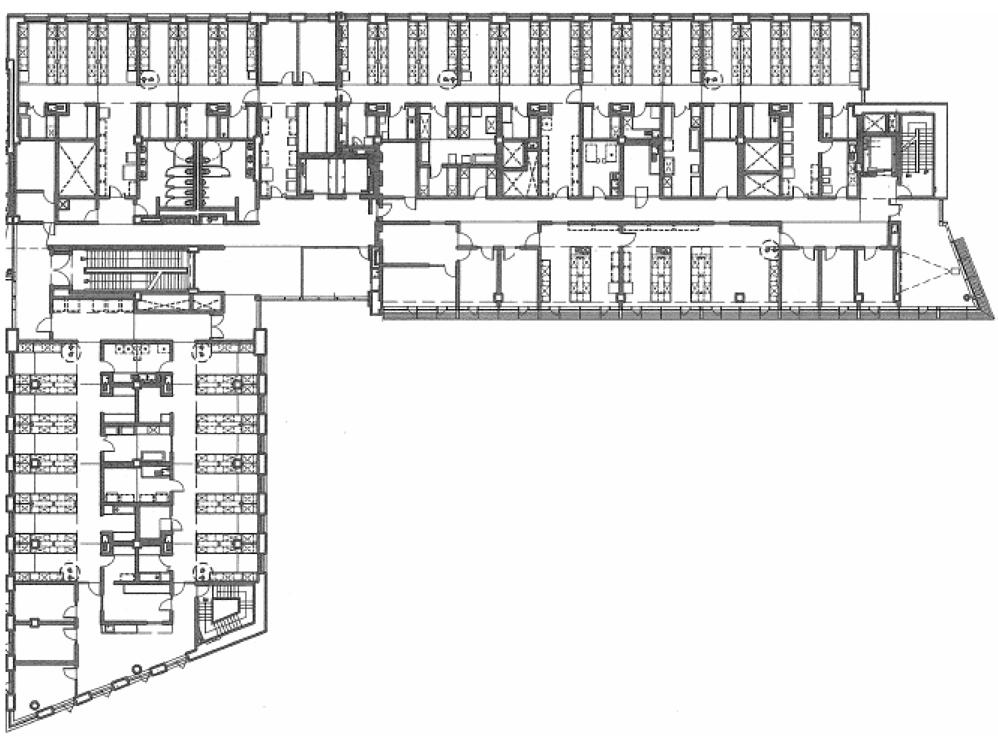
## Architectural Floor Plans – First Floor|



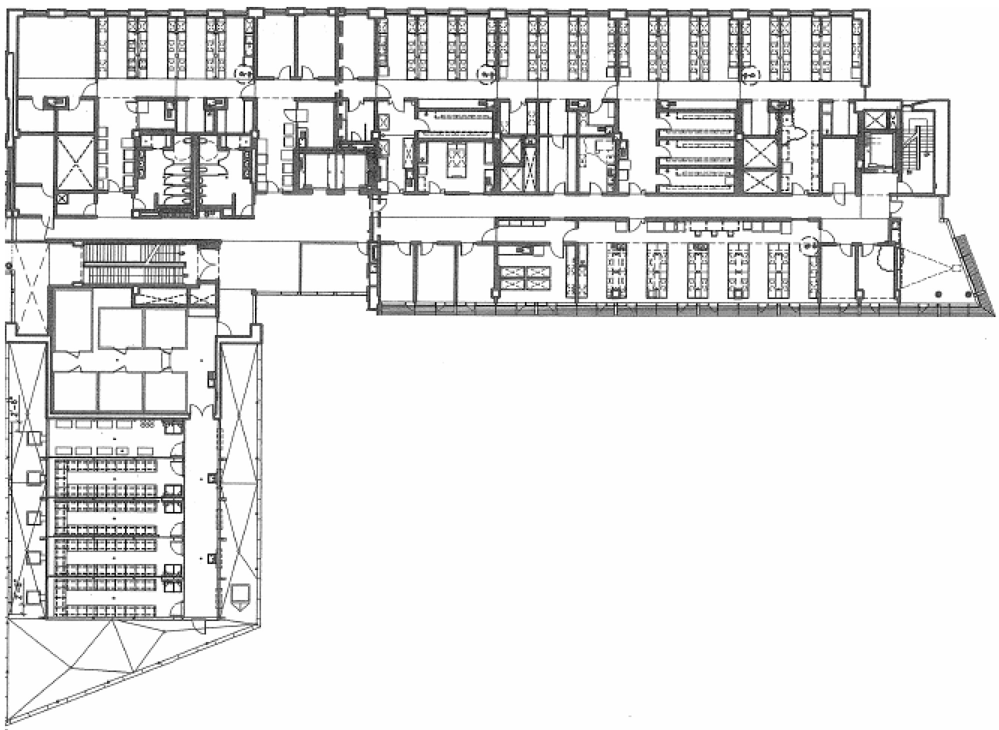
**Architectural Floor Plans – Second Floor|**



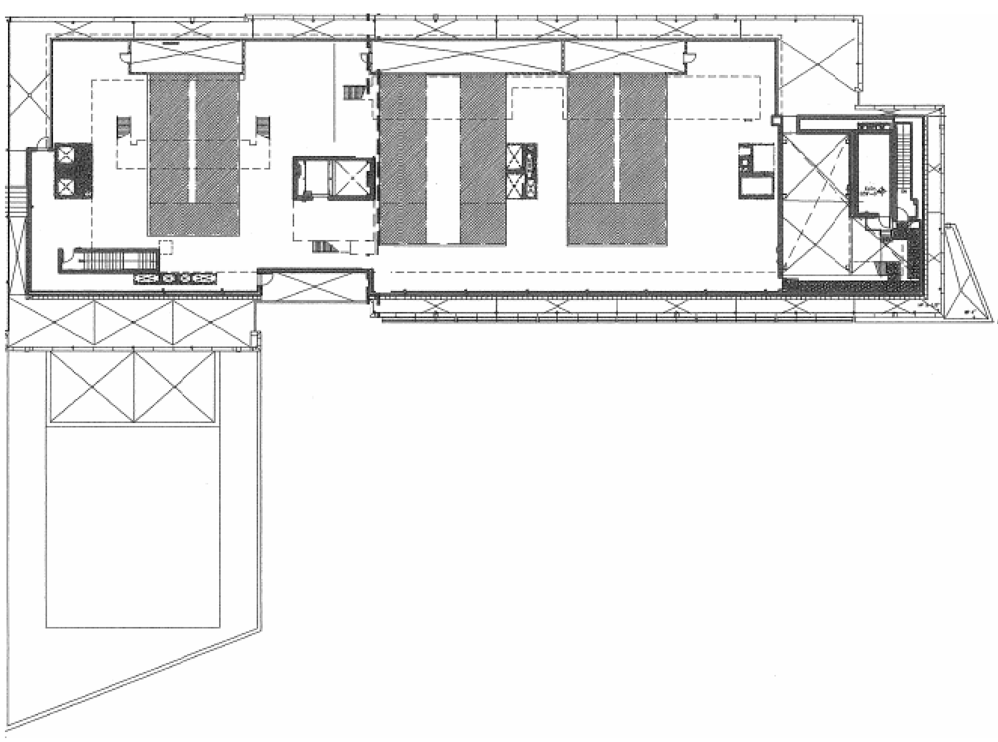
**Architectural Floor Plans – Third Floor|**



## Architectural Floor Plans – Fourth Floor|



## Architectural Floor Plans – Penthouse|



## Existing Structural System Summary|

### Foundation|

The original design of the Life Sciences Building used a combination of several foundation types to adapt to several different base slab elevations and varying subsurface conditions.

The vault area of the building was built on a continuous reinforced concrete mat foundation. Columns and walls of the vault bear on thickened portions of the mat foundation. The mat foundation has a thickness of 2'-0" and is reinforced with #6 and #7 bars at 12" on center. The bearing capacity of the soil underneath the mat foundation at the State College, Pennsylvania site was determined to be 2 ksf for exterior walls and 2.5 ksf for columns.

The foundation of the long leg of the "L" consists primarily of reinforced concrete spread footings. The maximum allowed bearing pressure on the soil underneath the spread footings (for the State College, Pennsylvania site) was given as 6 ksf. Underneath walls the foundation ranges from 1'-6" to 2'-3" thick and from 5'-6" to 10'-2" wide. To support columns the spread footings range from 1'-7" to 4'-0" thick and from 5'-6" to 17'-4" wide.

To support the rest of the building, including the knuckle and short leg of the "L", footings are supported on driven steel H – piles. The soil bearing capacity was considered to be 6 ksi on the gross section area of the steel H – pile for the State College, PA site. The skin friction value for H – piles is currently unknown. The piles in use are HP10x57 and HP12x74 sections with allowable working loads of 100 k and 130 k respectively. Piles are driven in groups to an average depth of 25' and capped. Piles are driven vertically in the center of pile caps and battered outward on the perimeter of pile caps on a 1:6 (H:V) batter. The piles are arranged in groups of 2,3,4,5,6,8,11, and 16. The pile caps are reinforced concrete and range in thickness from 3'-0" to 5'-0" deep. Grade beams span between pile caps to support the exterior walls.

### Floor Framing|

The typical basement slab on grade (occurring underneath the long leg of the "L") is 6" of 4000 psi concrete on 6" of PennDOT 2A aggregate reinforced with WWF6x6 – W4xW4. The typical ground level slab on grade (occurring underneath the short leg of the "L") is 5" of 4000 psi concrete reinforced with WWF6x6 – W2.9x2.9. The typical floor deck is composite 18 gage, 2" thick fluted with 4-1/2" of concrete cover for a total thickness of 6-1/2". The concrete is normal weight, 4000 psi with one layer of WWF4x4 – W5.5xW5.5. All beams and girders are composite steel wide flange sections using 5" long, ¾" diameter shear studs welded directly to the beam. The shear studs have a shear transfer capacity of 13.3 k/stud.

## Existing Structural System Summary (continued)|

Floor Framing (continued)|

Beginning with the ground floor level of the long leg of the "L" the floor framing system takes on a typical layout. This framing system is typical and occurs on the ground through fourth floors. The typical floor deck is composite 18 gage, 2" thick fluted with 4-1/2" of concrete cover for a total thickness of 6-1/2". The concrete is normal weight, 4000 psi with one layer of WWF4x4 – W5.5xW5.5. Infill beams for the ground through fourth floors are typically composite W16x26 (spaced 8'-0" o.c.) and composite W16x31 (spaced 8'-8" o.c.) which are precambered and span 31'-0". The girders supporting the W16x26 infill beams are composite W24x68 and span 31'-0". The girders supporting the W16x31 infill beams are composite W30x99 and span 41'-0". .

The framing of the short leg of the "L" is typical on the second through fourth floors, but becomes quite complex on the ground floor to accommodate an auditorium with a sloped floor. The floor framing system for the second through fourth floors of the short leg consists of the typical composite floor system bearing on composite W14x22 infill beams. The W14x22 infill beams are spaced at 8'-8" o.c. and span 20'-8". They are supported by W21x57 composite girders which span 26'-0". Each girder supports two infill beams at third points.

The mechanical penthouse level occurs at the top of the long leg of the "L". The penthouse houses air handlers, cooling towers, and various other pieces of mechanical and electrical equipment. The penthouse was designed for comparatively heavy live and dead loads so the beams and girders are much larger than the typical floor framing for the long leg of the "L". Although the beams and girders do have a similar layout and spacing when compared to the other floors. The penthouse floor structure begins with the typical composite floor deck and slab that can be found throughout the rest of the building. This slab bears into various W18 infill beams ranging from composite W18x40 to W18x97 (used to frame around openings in the slab). The most typical infill beams are W18x46 and W18x50 but larger sizes are also common where slab openings exist or support structures for the mechanical equipment bear down on the infill beam. The typical span of the beams and girders is 31'. The girders are most typically composite steel W33x141 and W33x201.

## Existing Structural System Summary (continued)|

### Roof Framing|

The typical roof deck is 20 gage, 1-1/2" deep, wide rib steel roof decking. The roof consists of low roofs that are framed as part of the mechanical penthouse floor system. From the low roof, set back in from the building perimeter, a sharply angled roof / wall system extends upward to form the enclosure of the mechanical penthouse. On the top of the space created by the angled roof / walls there is another flat roof to completely enclose the mechanical penthouse. As stated previously the low roof is framed as part of the mechanical penthouse floor system. The sharply angled roof is framed by noncomposite W18x60 girders running at an angle that is more vertical than horizontal. These girders run from the low roof to the top of the mechanical penthouse enclosure and act as beams / columns by forming the walls and supporting the higher flat roof. The girders are spaced at 31'-0". W12x26 infill beams then span horizontally in between the W18x60 girders. The infill beams span the entire 31'-0" space between the girders and are spaced with three equal spaces measured from the low flat roof to the top of the high flat roof. Finally, the top of the mechanical penthouse covered by the high flat roof is framed by W16x40, W16x31, and W16x26 beams in various configurations that allow large openings for the vents that ventilate the laboratories. The flat roofs are both covered with the typical roof deck. The sloped roof / walls are covered with plywood and light gauge steel framing.

### Lateral System|

The existing, wind load controlled, lateral force resisting system (and system of columns) is made up of a combination of braced and moment resisting frames. Due to the complex geometry of the footprint of the building; numerous lateral force resisting systems were needed to be located throughout the structure.

The building is shaped roughly like an "L" with the long side of the "L" running east to west. In the existing lateral force resisting system, a steel moment resisting frame runs along each of the long exterior walls of the building in the east – west direction. Additionally in the east – west direction are three combined moment / braced frames located internally in the short leg of the "L". One moment frame runs east –west on the end of the short leg of the "L" and is rotated from the east – west axis by about twenty degrees. The total number of frames providing lateral support to the building in the east – west direction is six.

For the existing lateral force resisting system in the north – south direction, three braced frames located across the interior of the long leg of the "L" provide lateral support. Also, on the far east end of the long leg of the "L" a braced frame provides north – south lateral support. In the short leg of the "L" one moment frame runs along each exterior wall. Additionally, in the north – south direction, a braced frame located at the outside corner where the long and short legs of the "L" meet provides additional lateral support. The total number of frames providing lateral support to the building in the north – south direction is seven.

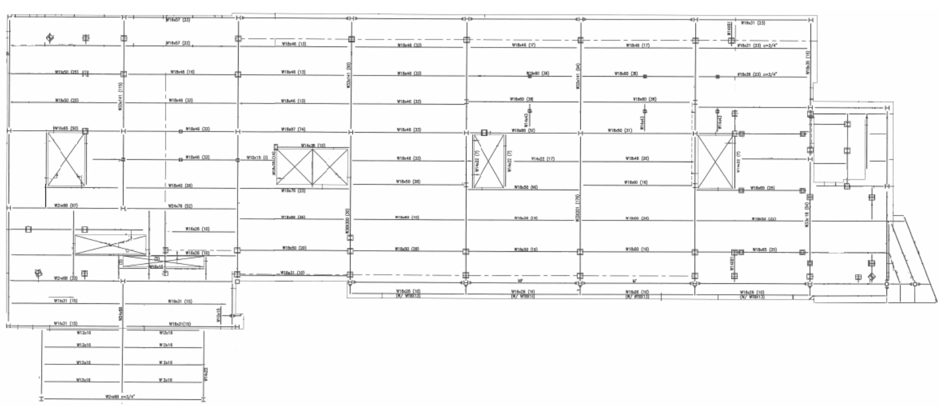
## Existing Structural System Summary (continued)|

### Columns|

The existing system of columns and lateral force resisting system was designed so that very few columns weren't a part of a moment frame or braced frame. Most column loading depends on many more factors than just the accumulation of gravity loads. The columns range in size from W10 up to W14. The weights generally vary from 33 lbs/ft to 311 lbs/ft. Estimated column gravity loads vary from 60 k to 1100 k, with the vast majority of column compression loads in the range of 200 k to 800 k.

## Structural Floor Plans – Floors 2, 3, 4 (typical)|

## Structural Floor Plans – Penthouse (typical)|



## Thesis Proposal – Introduction / Goals|

The major goal of this thesis exercise was to provide experience analyzing and designing building lateral force resisting systems. A specific emphasis was placed on determining, analyzing, and designing lateral forces due to seismic effects. The goal specific to the Life Sciences Building was to determine whether or not a building with a lateral force resisting system designed for the lowest seismic loading possible could be redesigned for a high seismic location without major changes to the building architecture and cost of the structure.

The existing lateral system (wind load controlled design) for the Life Sciences Building was analyzed in depth during Technical Assignment III and was found to be unnecessarily complicated and very inefficient. Ideas for improving the lateral system included the elimination of and redesign of lateral force resisting frames. The gravity framing system was studied in depth in Technical Assignment II and found to be the best option for the building. It was shown that the existing composite steel gravity framing system was the best suited of all alternatives to handle the varying spans, varying loads, irregular column placement, and the need to integrate lateral systems into the structure.

Considering the results of the Technical Assignments the most logical choice for a thesis topic was to redesign the lateral system. Also, the three Technical Assignments that were completed in the fall failed to provide any experience with seismic design because the Life Sciences Building was determined to be in Seismic Design Category "A". The final thesis proposal evolved into finding a way to gain understanding and experience in the interrelated topics of lateral force resisting systems and seismic analysis and design.

The formal proposal became analyzing the building lateral system in a location that results in a SDC of "D" and using that seismic loading to investigate, analyze, and redesign the complex lateral system in more detail than was done in Technical Assignment III. The lateral system was then redesigned in an attempt to resist lateral forces more efficiently while two breadth studies were concurrently undertaken to examine the effects of the redesign on the building's existing architecture and previous construction costs.

## Structural Depth – Problem Statement|

The Life Sciences Building was initially located on the campus of The Pennsylvania State University at University Park, Pennsylvania. Because of the buildings location and soil conditions the building was determined to be in Seismic Design Category “A”. Most practicing engineers would consider this a blessing. However, a student paying tens of thousands of dollars every year to learn how to design buildings to withstand earthquake forces does not. As a result the choice was made to (theoretically) move the Life Sciences Building to the Seattle, Washington campus of the University of Washington. This placed the building in Seismic Design Category “D” and allowed for a better investigation of the building lateral force resisting system as well as experience with seismic analysis, design, and detailing.

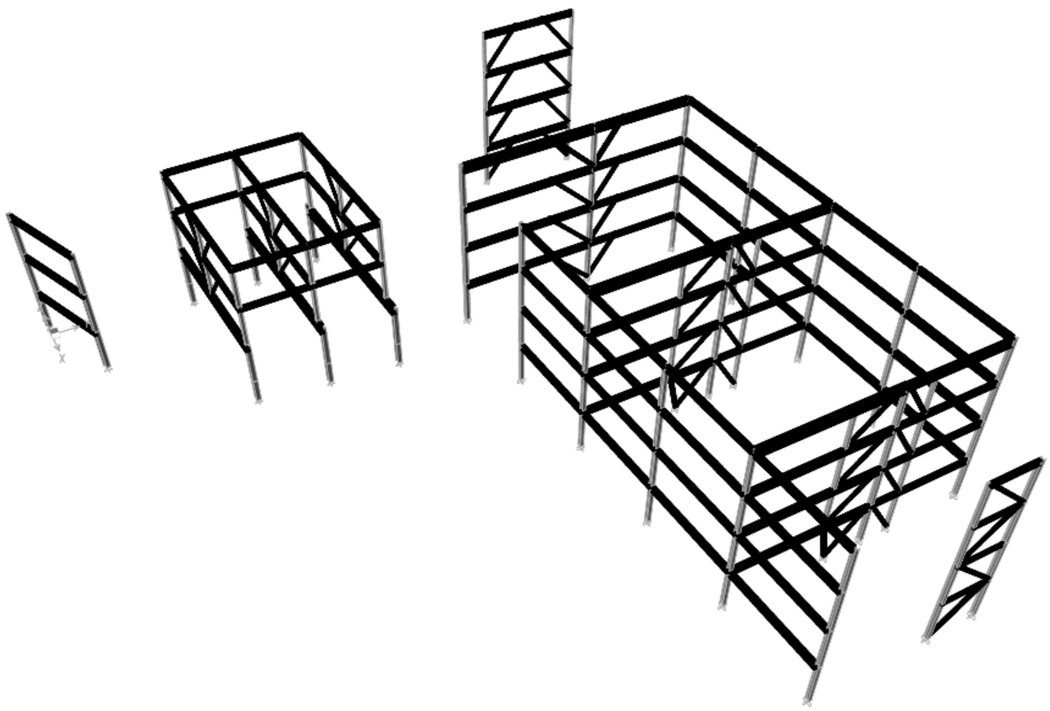
Analysis in Technical Assignment II confirmed that the existing concrete on composite steel deck and composite steel beams and girders was the best gravity framing system available to suit the varied span lengths and irregular framing plan of the Life Sciences Building. Because of this it was decided to leave the gravity framing of the building as unchanged as possible. However, a check needed to be made to ensure that the gravity system, specifically the concrete slab on composite metal deck, can act as an effective seismic diaphragm and transfer lateral loads into the seismic force resisting system.

In contrast to the gravity framing system, the lateral force resisting system of the Life Sciences Building was very – almost needlessly – complicated and inefficient. As stated previously, lateral forces are resisted through a combination of moment resisting frames, concentrically braced frames, eccentrically braced frames, and hybrid frames consisting of two or more types of lateral frames. Technical Assignment III showed that there is much room for improvement in the design of the lateral force resisting system. Analysis and understanding of the lateral force resisting system in Technical Assignment III was limited due to the complicated nature of the system. Analysis through ETABS did show that several of the many frames in the lateral force resisting system are taking very low portions of the total lateral load – some are taking almost none of the lateral load.

The following pages will provide some background into the existing lateral force resisting system. They will cover the complete lateral force resisting system analysis and redesign for the Life Sciences Building after being placed in Seismic Design Category “D”. A check of the building diaphragms also follows to ensure that the gravity framing system doesn’t need to be modified.

## Structural Depth – Existing Lateral System Description|

### Existing Lateral System Diagram (from ETABS)|



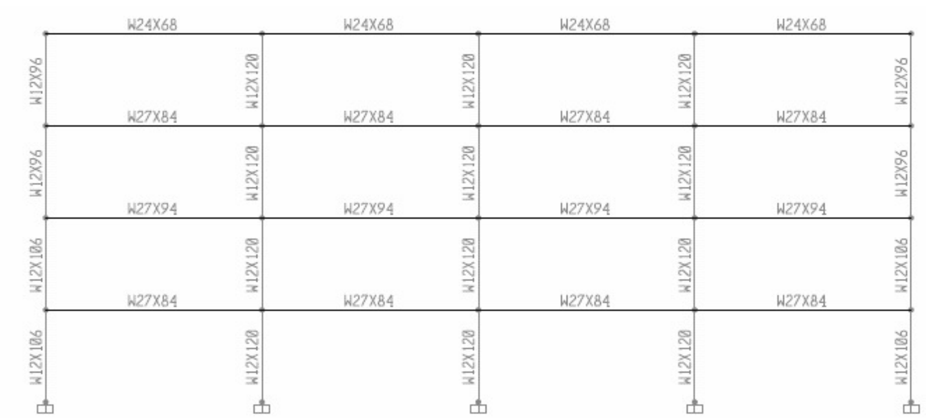
### Existing Lateral System Description|

The building lateral system consists of moment resisting frames, concentrically braced frames, eccentrically braced frames, and frames that are hybrid combinations of moment and braced frames. In the east – west direction there are three moment frames, and three hybrid frames that are combinations of moment and eccentrically braced frames. In the north – south direction there are three concentrically braced frames, two eccentrically braced frames, and two hybrid moment / concentrically braced frames. The system is further complicated by the fact that although most of the frames are on two orthogonal axes – there are three lateral resisting frames that are rotated at various angles from the orthogonal axes due to architectural constraints. Nearly all of the lateral force resisting frames are tied into frames in the orthogonal direction with moment connections. The lateral frame illustrations shown in this section have their bases at the first floor level but many extend down through the first floor diaphragm to baseplates at the ground and basement floor levels.

## Structural Depth – Existing Lateral System Description (continued)|

East – West Existing Lateral System Description|

The lateral system in the east – west direction as stated above consists of three moment frames and three hybrid frames that combine both moment resisting and concentrically braced elements. Two of the moment frames, Moment Frame 1 and Moment Frame 4 occur along the exterior wall of the long leg of the “L”. Moment Frame 1 and Moment Frame 4 are illustrated below:



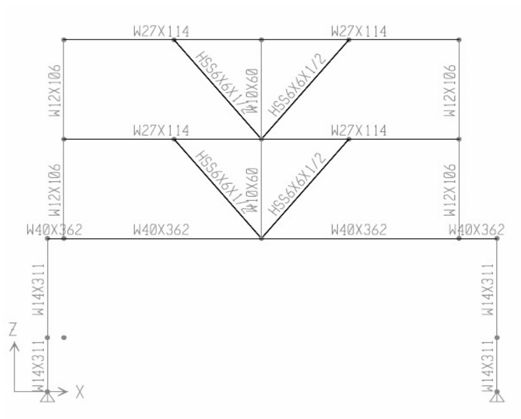

**Moment Frame 1**


**Moment Frame 4**

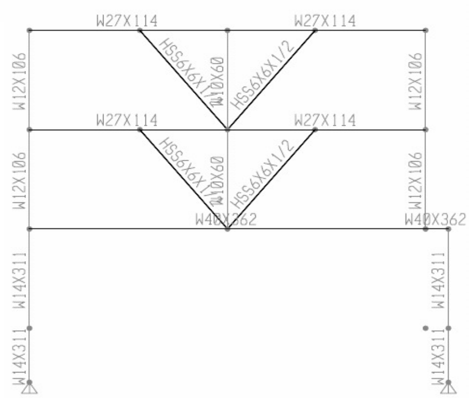
## Structural Depth – Existing Lateral System Description (continued)|

East – West Existing Lateral System Description (continued)|

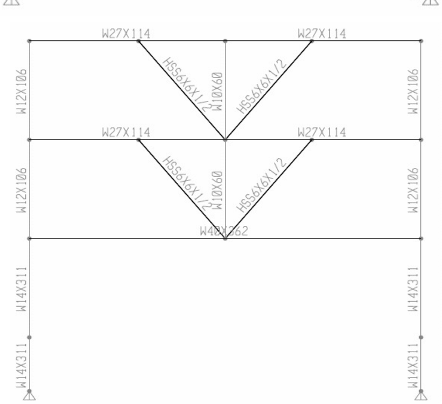
The only other moment frame running in the east – west direction is Moment Frame 9. It runs angled at twenty degrees from the east – west axis at the very end of the short leg of the "L". It is illustrated at the far left of the three dimensional view of the lateral force resisting system shown at the beginning of this section. The three hybrid moment / eccentrically braced frames that take east – west lateral loading are all very similar with slight changes in each frame to adapt to the architectural restrictions of the building. Views of all three hybrid frames are shown below – note the large clear span moment frame on the lowest level. The clear span is needed to allow for an auditorium to be located on the first floor of the building, convenient to the major entrances and exits. These three hybrid frames occur in the short leg of the "L".




Hybrid Frame 5.3




Hybrid Frame 6

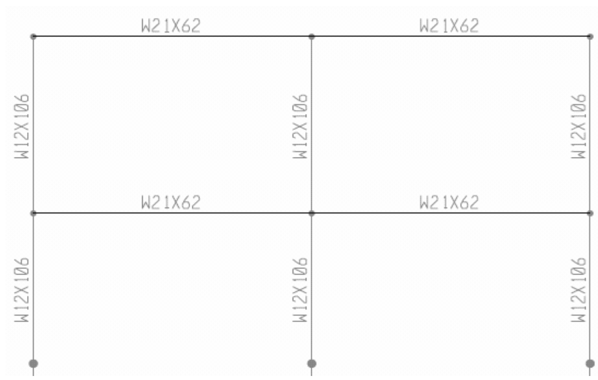



Hybrid Frame 7

## Structural Depth – Existing Lateral System Description (continued)|

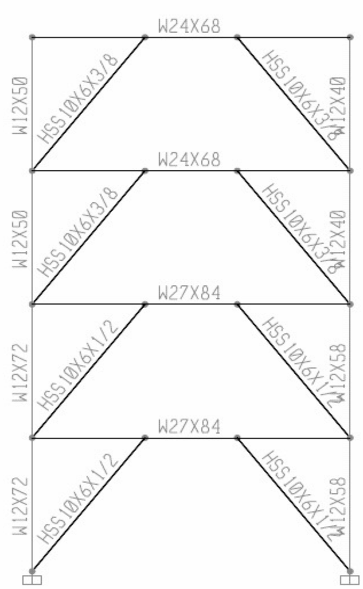
North – South Existing Lateral System Description|

In the north – south direction lateral forces are resisted by a total of seven frames. Two of these frames – Hybrid Frame C.2 and Hybrid Frame D.8 – are hybrid frames combining concentrically braced elements with moment resisting elements. These two hybrid frames occur on the outside walls of the short leg of the "L". The upper two stories of each three story frame is illustrated below (bracing in the lowest level of the frame is not visible due to the limitations of ETABS).




**Hybrid Frame C.2**
**Hybrid Frame D.8 (identical)**
(first floor braced level not shown)

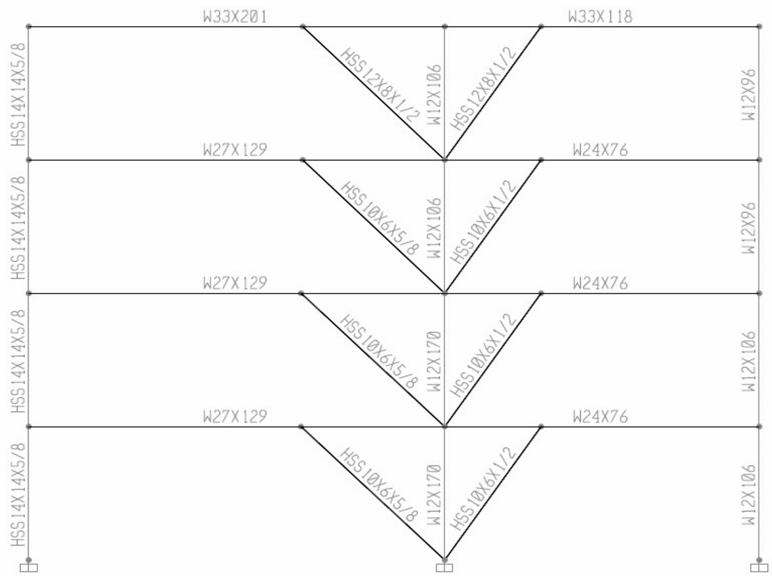
Two eccentrically braced frames are utilized to resist lateral forces on the building in the north – south direction. They are located at the "knuckle" where the long leg and short leg of the "L" meet. The first eccentrically braced frame – Braced Frame C – is one bay wide and is located at the outside corner of the "L". The second eccentrically braced frame – Braced Frame E – is located internally where the long and short legs meet and is roughly in line with the exterior wall of the short leg of the "L". The eccentrically braced moment frames that occur in the "knuckle" of the "L" are shown below.




**Braced Frame C**

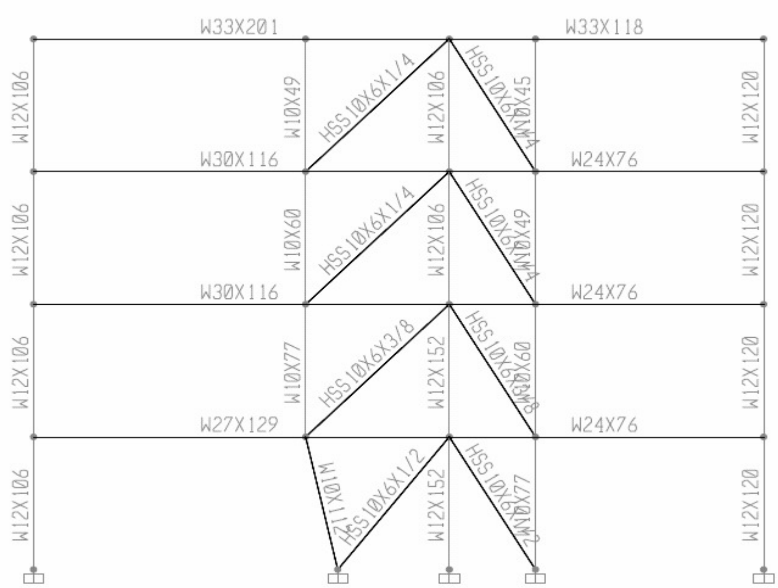
## Structural Depth – Existing Lateral System Description (continued)

### North – South Existing Lateral System Description (continued)




**Braced Frame E**

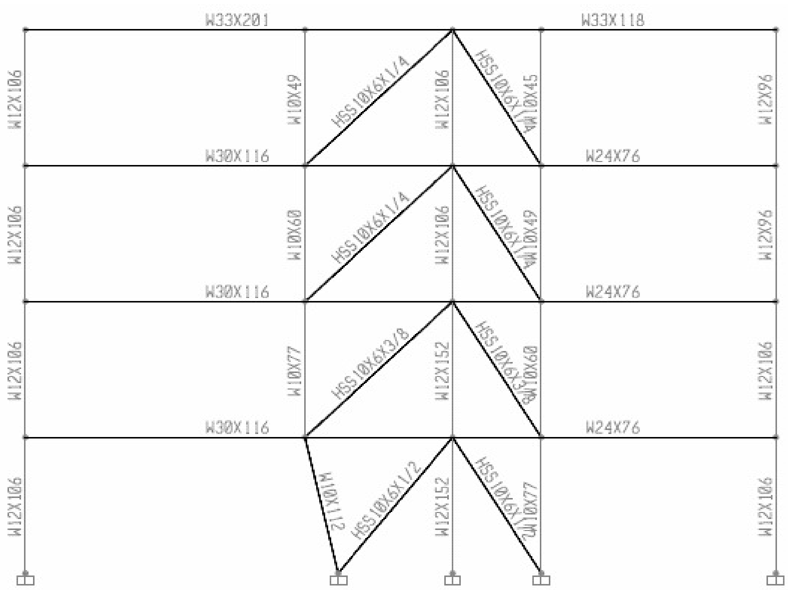
Finally, three concentrically braced frames provide lateral force resistance within the long leg of the "L". Two north – south braced frames – Braced Frame G and Braced Frame J – span the entire orthogonal distance between east – west Moment Frame 1 and Moment Frame 4. The last lateral force resisting frame in the north – south direction – Braced Frame K – is located at the far east end of the building at the end of long leg of the "L". These three frames are illustrated below.



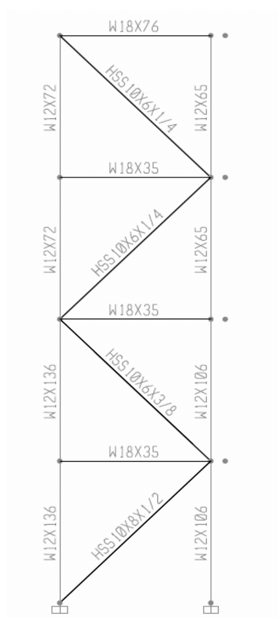

**Braced Frame G**

## Structural Depth – Existing Lateral System Description (continued)|

### North – South Existing Lateral System Description (continued)|




**Braced Frame J**




**Braced Frame K**

## Structural Depth – Existing Lateral System Analysis Results|

Existing East – West Lateral Force Resisting System|

Because the existing lateral force resisting system of the Life Sciences Building was very complicated and composed of many different frames; ETABS was used to determine the portion of the total lateral force that was distributed to each frame when the building was subjected to an east – west wind loading condition. Wind load was the controlling load case for the initial design of the Life Sciences Building in University Park, PA. A break down of the load and percentage of the total east – west base shear taken by each frame follows:

| From Hand Analysis: | $V_{tot,e\text{-}w}$ = 217.11 | |
|---|---|---|
| | Force (k) | Percentage |
| **Moment Frame 1** | 56.21 | 25.9 |
| **Moment Frame 4** | 58.68 | 27.0 |
| **Hybrid Frame 5.3** | 9.24 | 4.3 |
| **Hybrid Frame 6** | 8.97 | 4.1 |
| **Hybrid Frame 7** | 8.84 | 4.1 |
| **Moment Frame 9** | 12.71 | 5.9 |
| **TOTAL E-W FRAMES** | **154.65** | **71.2** |
| | | |
| **N-S Frames** | **62.46** | **28.8** |
| **Interior Columns 2.8** | 33.66 | |

As expected, Moment Frame 1 and Moment Frame 4 each took on a relatively equal portion of the base shear. Due to their similar stiffness Moment Frame 1 and Moment Frame 4 should each resist a relatively similar portion of the lateral loading. It is also interesting to note from the ETABS analysis that the interior columns of the long leg of the "L" resist a substantial portion of the east – west lateral load.

The three hybrid moment frames in the east – west lateral system also are interesting when analyzed further with ETABS. The lower moment frame portion only takes around 9k of shear at the base level for each frame. However, the braced frame that is on top of the moment frame has shear forces ranging from 41k to 45k. This is probably due to the reduced stiffness of the long span moment frames as compared to the braced frames above. The rigid diaphragms at the second floor level transfer the shear from the braced frames into other lateral force resisting frames elsewhere in the building.

## Structural Depth – Existing Lateral System Analysis Results (continued)|

Existing North – South Lateral Force Resisting System|

Once again the complicated nature of the lateral force resisting system and the sheer number of uniquely designed frames involved necessitated the use of ETABS to determine the portion of the total lateral force that was distributed to each frame. This time the building was subjected to a north – south wind loading condition. A break down of the load and percentage of the total north – south base shear taken by each frame is as follows.

| From Hand Analysis: | $V_{tot,n\text{-}s}$ = 356.11 | |
|---|---|---|
| | **Force (k)** | **Percentage** |
| **Braced Frame C** | 81.76 | 23.0 |
| **Hybrid Frame C.2** | 0 | 0.0 |
| **Hybrid Frame D.8** | 0 | 0.0 |
| **Braced Frame E** | 17.91 | 5.0 |
| **Braced Frame G** | 113.14 | 31.8 |
| **Braced Frame J** | 88.85 | 25.0 |
| **Braced Frame K** | 50.48 | 14.2 |
| **TOTAL E-W FRAMES** | **352.14** | **98.9** |
| | | |
| **E-W Frames** | **3.97** | **1.1** |

A limitation of my model becomes evident when looking at the results for the two hybrid frames. I was unable to model the north – south bracing in this direction which resulted in each frame taking no north – south wind base shear forces.

Another result that stands out is the comparatively low percentage of the base shear taken by the opposite direction's lateral force resisting system for north – south wind loads. In the previous analysis for east – west wind loads the north – south system resisted almost one third of the east – west lateral forces. However, when the north – south wind loads are applied, the east – west lateral system only resists about 1% of the forces that were applied in the orthogonal direction. It could be due to the increased relative stiffness in the north – south direction when compared to the east – west direction because the north – south direction uses braced frames rather than moment frames.

The analysis also shows that generally, concentrically braced frames are more effective than eccentrically braced frames at resisting lateral loading. Additionally, the deflections in the north – south direction were considerably less than the deflections in the east – west direction – even without modeling two braces in the north – south hybrid frames. This shows that moment resisting frames tend to deflect the most, followed by eccentrically braced frames. Concentrically braced frames seem to be the optimal choice when stiffness and deflections are an issue and will be the preferred alternative in redesign.

**Structural Depth – Existing Lateral System Building Code|**

The Life Sciences Building was designed in the late 1990s and the building was completed and occupied in September 2004. When the Life Sciences Building was originally designed it used the most current building codes at the time. However many radical changes have taken place regarding building codes between the original design of the Life Sciences Building and now.

Building Code / Loading|
- Building Officials and Code Administrators
  - BOCA 1996
- Pennsylvania Department of Labor and Industry
  - PA L&I Title 34 1996
- American Society of Civil Engineers
  - ASCE 7

Reinforced Concrete|
- American Concrete Institute
  - ACI 318 – 95

Structural Steel|
- American Institute of Steel Construction
  - AISC – Codes and Specifications (most current at the time of design)

Cold Formed Steel Decking|
- Steel Deck Institute
  - SDI – Steel Deck Design Manual (most current at the time of design)

**Structural Depth – Current Redesign Building Codes|**

In the reanalysis and redesign of the Life Sciences Building the most current building codes at this time will be used. The following codes will be used extensively in the reanalysis and design of the Life Sciences Building:

Building Code / Loading|
- International Code Council
  - IBC 2006
- American Society of Civil Engineers
  - ASCE 7 – 05

Reinforced Concrete|
- American Concrete Institute
  - ACI 318 – 08

Structural Steel|
- American Institute of Steel Construction
  - AISC – 13th Edition Steel Manual
  - AISC – Seismic Design Manual (October 2006 Printing)

Cold Formed Steel Decking|
- Steel Deck Institute
  - SDI – Diaphragm Design Manual, 3rd Edition

## Structural Depth – Current Redesign Material Strength|

The following material strengths were used in the redesign of the Life Sciences Building. The material strengths reflect what is commonly available in the Seattle, Washington area. The steel material strengths meet the requirements of the AISC Specification for Structural Steel Buildings and also meet the additional requirements on structural steel material specifications set forth in the AISC Seismic Provisions for Structural Steel Buildings.

### Reinforced Concrete|

Compressive Strength
- $f'_c = 4000$ psi

Reinforcement Bars (ASTM A615 Grade 60)
- $f_y = 60000$ psi

Welded Wire Fabric (ASTM A185)
- $f_y = 70000$ psi

### Structural Steel|

Beams, Columns, Other Framing Members = ASTM A992
- $F_y = 50$ ksi  $F_u = 65$ ksi

Plates, Bars, Angles = ASTM A572 Gr. 50
- $F_y = 50$ ksi  $F_u = 65$ ksi

Hollow Structural Sections = ASTM A500 Gr. B
- $F_y = 46$ ksi  $F_u = 58$ ksi

All bolts will be ¾" ASTM A325N (threads included)
- $V_n = 15.9$ k / bolt

Shear Studs will be ¾" diameter 5" long
- $V_n = 13.3$ k / stud

### Steel Deck|

Roof Deck
- $F_y = 33$ ksi

Composite Floor Deck
- $F_y = 33$ ksi  $F_u = 45$ ksi

## Structural Depth – Current Redesign Initial Considerations|

The shape of the building is an "L", which results in one of the most difficult conditions for seismic design – the reentrant or "inside" corner. Two significant problems are created by the "L" shape of the building. Research into the problems was performed using The Seismic Design Handbook (Naeim, 2001).

The first problem is that the rigidity between both legs of the "L" is bound to vary with a number of factors. This variation in rigidity between the two legs of the "L" results in stress concentrations at the "notch" of the reentrant corner. The variation in rigidity between the two legs is further exacerbated by the fact that each leg is a different height, with the transition occurring at the "notch." This difference in height results in a vertical discontinuity of a setback in elevation.

The second problem is torsion. Torsion will occur in the building due to its "L" shaped form. This is because the location of the center of mass and center of rigidity will vary with every earthquake condition and will rarely coincide. The location of the center of mass not coinciding with the center of gravity will lead to a net rotation of the building.

The stress concentration at the notch is related to the torsional effects. Factors directly influencing stress concentration and torsion include:

- The height and length of the legs of the "L"
- The building mass.
- The structural system.

Two solutions to the problem of reentrant corners exist. One involves separating the "L" structurally into two rectangular buildings and using some type of joint assembly to combine the two functionally and architecturally. Complications arise using this method; including, allowing for the relative motion of each building, making each building able to stand on its own, and ensuring the fire, smoke and weather proofing of the joint.

The second solution involves designing the structure to tie the building together at locations of stress concentration. It also requires using strengthened and stiffened elements at strategic locations to reduce torsion on the building. Specific methods of tying the building together and minimizing locations of stress concentrations include, adding collector beams to transfer forces across the intersection, adding structural walls instead of collectors, and stiffening the lateral support system at each end of the "L" shape. Another alternative to cut down on stress concentrations at the "notch" is to splay the corner or build it out.

**Structural Depth – Current Redesign Initial Considerations (continued)|**

Additional considerations involving the seismic design include placing the seismic force resisting elements as far from the center of the building as possible to maximize their moment arm. In addition to placing the seismic force resisting elements as from the center as possible, it is also important to make sure they are balanced to prevent torsion.

The greatest concentration of forces in the diaphragms will occur where the long leg of the "L" meets the short leg of the "L". The "notch" area of the diaphragm should be solid and free from openings (stairwells, elevator shafts). This becomes a significant problem in this building because there is a large open stairwell located right at the line of intersection between the two legs of the "L". To overcome openings in the diaphragm for elevators and stairwells at the "notch" collectors or drag struts may need to be employed.

The auditorium and its requirement for long span moment frames with a high ceiling on the first floor of the short leg of the "L" may create a soft first story. This will have to be carefully considered in the design of the seismic force resisting system.

## Structural Depth – ASCE 7-05 Section 3.1: Dead Load|

Dead loads for the seismic force resisting system redesign were taken as the self weights of the building materials. The partition load allowance was added to classroom, lab and office areas but was taken as part of the live load for this analysis, except when it was considered for the effective seismic weight of the structure. Additional superimposed dead loads will be added to the classroom, lab and office areas for finishes, as well as added to the structures that are directly above mechanical and electrical rooms. The values used for these superimposed dead loads follow:

| Classrooms, Labs, Offices | |
|---|---|
| Collateral Dead Load | 10 PSF |
| Partition Load (given w/ Live Load) | 20 PSF |

| Electrical / Mechanical Rooms | |
|---|---|
| Collateral Dead Load (on structure above) | 30 PSF |

## Structural Depth – ASCE 7-05 Section 4.1 – 4.11: Live Load|

Live loads used were the loads prescribed for the original design. Live loads were compared with recommended values from IBC 2006 and ASCE 7 – 05 for reanalysis. Several loads specified by the user that were higher than recommended values from IBC 2006 and ASCE 7 – 05 were left unchanged from the original design as a conservative assumption. The following lists the live load assumptions that were used in the original design – which are also the live loads that were used to perform all calculations:

**Assembly Areas**

| | |
|---|---|
| **Fixed Seats** | **60 PSF** |
| **Lobbies / Moveable Seats*** | **100 PSF** |

**Corridors**

| | |
|---|---|
| **All Levels*** | **100 PSF** |

**Classrooms, Labs, Offices**

| | |
|---|---|
| **Reducible Live Load** | **80 PSF** |
| **Partition Load**** | **20 PSF** |

**Electrical / Mechanical Rooms**

| | |
|---|---|
| **User Defined Load*** | **200 PSF** |

**Stairs / Landings**

| | |
|---|---|
| **Horizontal Surface Load*** | **100 PSF** |

**Storage Areas**

| | |
|---|---|
| **Light Storage*** | **125 PSF** |

* Indicates that load is non-reducible because it is a heavy live load according to IBC 2006 and ASCE 7 – 05 (S.4.8.2).
** Indicates that load is non-reducible because it is a partition load which will constantly be applied to the structure.

**Structural Depth – ASCE 7-05 Section 6.1 – 6.7: Wind Load|**

The building will be designed using the Analytical Procedure of ASCE 7-05 Section 6.5 and its sub-sections.

**Structural Depth – ASCE 7-05 Section 6.5.1: Wind Load|**

The Analytical Procedure is permitted to be used because the building meets the following conditions:

**Enclosed (S.6.2 & S.6.5.9)**
**Regular Shaped (S.6.2)**
**Rigid Structure (S.6.2)**

**Approximate Fundamental Period (S.12.8.2.1)**

$T_a$ **= .838 s f = 1.193 Hz**

$T_a$ **= .618 s f = 1.618 Hz**

**Structural Depth – ASCE 7-05 Section 6.5.4: Wind Load|**

The Basic Wind Speed for Seattle, Washington was determined using Figure 6-1. The Basic Wind Speed was found to be:

**V = 85 MPH**

The Wind Directionality Factor was found from Table 6-4. The Wind Directionality Factor was found to be:

$K_d$ **= .85**

**Structural Depth – ASCE 7-05 Section 6.5.5: Wind Load|**

The Importance Factor was found using Table 6-1. The Importance Factor was dependent on the building's Occupancy Category of III. The Importance Factor was determined to be:

**Occupancy Category III**
**I = 1.15**

## Structural Depth – ASCE 7-05 Section 6.5.6: Wind Load|

The exposure in every direction around the building was classified according to Section 6.5.6.2. The classification in all direction was Surface Roughness B – which is consistent with the urban campus of the University of Washington. The exposure was then developed in Section 6.5.6.3 using the Surface Roughness and the direction and distance where it prevailed. The Exposure of the building was determined to be:

**Exposure B**

Using Table 6-3 and its footnotes the Velocity Pressure Coefficients were found. The formula to calculate the velocity pressure coefficients was found to be:

$$K_z = 2.01\,(z / z_g)^{(2/\alpha)}$$

**z must be > 15**

## Structural Depth – ASCE 7-05 Section 6.5.7: Wind Load|

The Topographic Factor need not be considered for this building and its location.

## Structural Depth – ASCE 7-05 Section 6.5.8: Wind Load|

The Gust Effect Factor is permitted to be conservatively taken as .85 for rigid structures. This provision was used to determine the Gust Effect Factor as:

$$G = .85$$

## Structural Depth – ASCE 7-05 Section 6.5.9: Wind Load|

The Enclosure Classification was established using Section 6.5.9 and Section 6.2. The building was determined to be classified as:

**Enclosure Classification: Enclosed**

## Structural Depth – ASCE 7-05 Section 6.5.11: Wind Load|

The Internal Pressure Coefficient was determined using Section 6.5.11.1. Using the Enclosed Enclosure Classification the Internal Pressure Coefficient was found to be:

$$GC_{pi} = \pm .18$$

External Pressure Coefficients are determined using Section 6.5.11.2. The External Pressure Coefficients were determined using Figure 6-6 and the geometry of the building for both of the prevailing wind directions (North – South, East – West). These External Pressure Coefficients are incorporated into the spreadsheet that was used in the calculation of wind pressures.

## Structural Depth – ASCE 7-05 Section 6.5.10: Wind Load|

Velocity Pressure is determined in accordance with Section 6.5.10. Velocity Pressure is calculated in the spreadsheet on the following page for every floor height using the formula:

$$q_z = .00256\ (K_z)\ (K_{zt})\ (K_d)\ (V^2)\ (I)$$

## Structural Depth – ASCE 7-05 Section 6.5.12: Wind Load|

The Main Wind Force Resisting System (MWFRS) for rigid buildings is designed in accordance with the provisions of Section 6.5.12 in combinations with the Velocity Pressures determined in Section 6.5.10. The Design Wind Pressures for MWFRS in buildings of all heights will be determined by the following equation and the overall result of this equation can be seen in the table below:

$$p = q\,(G)\,(C_p)$$

**East - West Wind Loading**

| height range | h (max) | qz | G | Cp | Windward Wall Pressure (PSF) |
|---|---|---|---|---|---|
| 83.5 '- 97' | 97 | 17.712 | 0.85 | 0.80 | 12.04 |
| 63' - 83.5' | 83.5 | 16.970 | 0.85 | 0.80 | 11.54 |
| 49' - 63' | 63 | 15.657 | 0.85 | 0.80 | 10.65 |
| 35' - 49' | 49 | 14.573 | 0.85 | 0.80 | 9.91 |
| 21' - 35' | 35 | 13.237 | 0.85 | 0.80 | 9.00 |
| 7' - 21' | 21 | 11.439 | 0.85 | 0.80 | 7.78 |
| 0' - 7' | 7 | 10.391 | 0.85 | 0.80 | 7.07 |

| h | qz | G | Cp | Leeward Wall Pressure (PSF) |
|---|---|---|---|---|
| - | 17.712 | 0.85 | -0.428 | -6.44 |

**North - South Wind Loading**

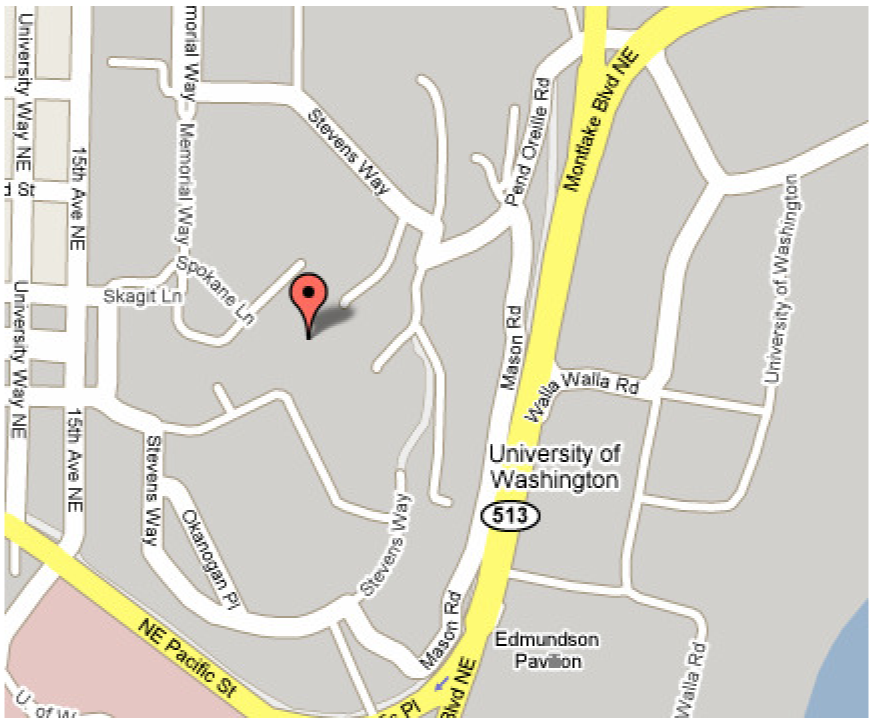
| height range | h (max) | qz | G | Cp | Windward Wall Pressure (PSF) |
|---|---|---|---|---|---|
| 83.5 '- 97' | 97 | 17.712 | 0.85 | 0.80 | 12.04 |
| 63' - 83.5' | 83.5 | 16.970 | 0.85 | 0.80 | 11.54 |
| 49' - 63' | 63 | 15.657 | 0.85 | 0.80 | 10.65 |
| 35' - 49' | 49 | 14.573 | 0.85 | 0.80 | 9.91 |
| 21' - 35' | 35 | 13.237 | 0.85 | 0.80 | 9.00 |
| 7' - 21' | 21 | 11.439 | 0.85 | 0.80 | 7.78 |
| 0' - 7' | 7 | 10.391 | 0.85 | 0.80 | 7.07 |

| h | qz | G | Cp | Leeward Wall Pressure (PSF) |
|---|---|---|---|---|
| - | 17.712 | 0.85 | -0.5 | -7.53 |

**Wind Loading Does Not Control.**

## Structural Depth – ASCE 7-05 11.4.1: Seismic Design Criteria|

Using software provided by the United States Geological Survey website the ZIP code for the University of Washington's Seattle campus (ZIP: 98195) was input to determine the Mapped MCE Acceleration Parameters for my building. The results were then refined by using the more accurate latitude and longitude (Latitude: 47.656167, Longitude: -122.3071) of the center of campus to recalculate the Mapped MCE Acceleration Parameters. Pertinent information from this process follows:




**Proposed Location Of Building – Latitude: 47.656167, Longitude: 122.3071**

$$S_S = 1.298$$
$$S_1 = .442$$

## Structural Depth – ASCE 7-05 11.4.2: Seismic Design Criteria|

Using a report issued by the Washington State Department of Natural Resources; the NEHRP (National Earthquake Hazards Reduction Program) Site Class for the Seattle, Washington campus of the University of Washington was determined. An excerpt of the map with the proposed site of the building shown by a star and Site Class of the proposed site are indicated below:


**Site Class "C"**

## Structural Depth – ASCE 7-05 11.4.3: Seismic Design Criteria|

The Site Coefficients used to adjust the Mapped MCE Acceleration Parameters were determined from ASCE 7-05 Table 11.4-1 and Table 11.4-2. The Mapped MCE Acceleration Parameters determined from the USGS software and Site Coefficients from ASCE 7-05 were multiplied together to determine the MCE Spectral Response Acceleration Parameters. These calculations are detailed below:

$$S_{MS} = F_a(S_S) = 1.0 * 1.298 = 1.298$$
$$S_{M1} = F_v(S_1) = 1.358 * .442 = .600$$

## Structural Depth – ASCE 7-05 11.4.4: Seismic Design Criteria|

The values for the MCE Spectral Response Acceleration Parameters were each multiplied by two – thirds to determine the values of the Design Spectral Acceleration Parameters. The calculations are shown below:

$$S_{DS} = (2/3)(S_{MS}) = (2/3) * 1.298 = .865$$
$$S_{D1} = (2/3)(S_{M1}) = (2/3) * .600 = .400$$

## Structural Depth – ASCE 7-05 11.5.1: Seismic Design Criteria|

The occupancy category assigned to the building was Occupancy Category III due to the fact that the building was a college building with a capacity of more than 500. The building could also be classified as Occupancy Category III because it has an auditorium where more than 300 people congregate at one time.

The importance factor was found using Table 11.5-1 and the building's classification in Occupancy Category III. A summary is below:

**Occupancy Category III**
**I = 1.25**

## Structural Depth – ASCE 7-05 11.6: Seismic Design Criteria|

The Seismic Design Category of the building was determined using Table 11.6-1 and Table 11.6-2 which assigns a seismic design category based on $S_{DS}$ and $S_{D1}$. Both tables put the building well within the requirements for Seismic Design Category D.

**Seismic Design Category D**

## Structural Depth – ASCE 7-05 11.8: Seismic Design Criteria|

It is recognized by this report that the additional requirements of Section 11.8 would need to be satisfied. However, in this academic exercise geotechnical investigation reports are not available for the site because in reality the building is not being constructed at the location being considered for this report.

## Structural Depth – ASCE 7-05 12.2.1: Seismic Design – Design Basis|

The seismic force resisting system was selected to accommodate various architectural considerations and requirements. The seismic force resisting system also had to satisfy the height requirements for Seismic Design Category D set forth in Table 12.2-1.

In the North – South direction large cavities in the plan between walls made the two most viable alternatives steel braced frames or concrete shear walls. Because of the limited height of the building, irregular shape of the prospective shear wall outlines, and the rest of the building being constructed of steel – reinforced concrete shear walls were ruled out as a possibility. Special Steel Concentrically Braced Frames were ultimately chosen to resist seismic forces in the North – South direction.

The East – West direction options were limited to moment frames. This was influenced partly due to the layout of the plans of the building which provided very little space at the interior of the building for any type of shear wall or braced frame. However, there were two other factors behind the decision to use moment frames. The first was the fenestration on each side of the long leg of the "L" – one entire side being an aluminum and glass curtainwall system and the other side having punched windows at a small but regular interval. The second factor was the need to provide a large clear span over several bays of framing on the first floor for an auditorium. In the East – West direction the seismic force resisting system was chosen to be Special Steel Moment Frames.

**North – South Seismic Force Resisting System:**
**Special Steel Concentrically Braced Frames**
**$R = 6$ $\Omega_O = 2$ $C_d = 5$**

**East – West Seismic Force Resisting System:**
**Special Steel Moment Frames**
**$R = 8$ $\Omega_O = 3$ $C_d = 5.5$**

## Structural Depth – ASCE 7-05 12.2.2: Seismic Design – Combinations (Different Directions)|

The use of Special Steel Concentrically Braced Frames in the North – South direction and the use of Special Steel Moment Frames in the East – West direction are permitted by Section 12.2.2 because the two systems are orthogonal.

## Structural Depth – ASCE 7-05 12.2.3: Seismic Design – Combinations (Same Direction)|

Each orthogonal direction has its own seismic force resisting system so the requirements for Section 12.2.3, "Combinations of Framing Systems in the Same Direction," can be ignored.

## Structural Depth – ASCE 7-05 12.2.4: Seismic Design – Combination Detailing|

One column is shared in the seismic force resisting system for each orthogonal direction. Because of this structural arrangement the requirements of Section 12.2.4 needed to be met. The members must be detailed for the highest R value required by the type of seismic force resisting system. In the case of this building the column needed to be detailed to meet the requirements of a Special Steel Moment Frame. These requirements were met and the column was designed using the worst case load combination that results from applying 100% of the seismic force in one direction and 30% of the seismic force in the other direction.

## Structural Depth – ASCE 7-05 12.2.5: Seismic Design – System Specific Requirements|

The requirements of Section 12.2.5.5 only required that Special Steel Moment Frames be detailed and continuous directly down to the foundation (seismic base). This requirement was met by bringing concrete shear walls (part of the foundation) up to the seismic base at the first floor level. No other sub sections of Section 12.2.5 are applicable to the building.

## Structural Depth – ASCE 7-05 12.3.1: Seismic Design – Diaphragm Flexibility|

The diaphragm was modeled as rigid to simplify analysis. This is consistent with the SDI – Diaphragm Design Manual, 3rd Edition which makes note of the very rigid properties of concrete filled steel decks.

## Structural Depth – ASCE 7-05 12.3.2: Seismic Design – Irregular and Regular Classification|

A substantial amount of effort in the early stages of the lateral force resisting system redesign went into trying to eliminate all of the irregularities of the structure so that Equivalent Lateral Force analysis could be used to determine seismic forces. These efforts were mostly wasted as it became almost impossible to eliminate Horizontal Structural Irregularities of Type 1A (Torsional Irregularity) and Type 1B (Extreme Torsional Irregularity). This is due to the "L" shape of the building. The only other irregularities present in the structure were; Horizontal Structural Irregularity Type 2 (Reentrant Corner Irregularity) and Vertical Structural Irregularity Type 2 (Weight (Mass) Irregularity). The Reentrant Corner Irregularity was due to the "L" shape of the building. The Weight Irregularity was due to the very highly loaded mechanical penthouse on the top floor of the building.

## Structural Depth – ASCE 7-05 12.3.3: Seismic Design – Additional Irregularity Requirements|

The limitations and additional requirements for systems with structural irregularities for structures in Seismic Design Category "D" prohibited buildings from having Vertical Structural Irregularity Type 5B (Extreme Weak Story Irregularity). Additional requirements were set up for diaphragm and collector element forces determined when the building was designed using the Equivalent Lateral Force Method. Because the Life Sciences Building was ultimately designed using a Modal Response Spectrum Analysis this provision didn't apply.

## Structural Depth – ASCE 7-05 12.3.4: Seismic Design – Redundancy|

The redesign of the lateral force resisting system was set up so that both directions had a redundancy factor of 1. This was done to avoid using amplified loads to design certain lateral force resisting system members. The requirements of Table 12.3-3 were met by the lateral force resisting system in each orthogonal direction.

$$\rho = 1.0$$

## Structural Depth – ASCE 7-05 12.4: Seismic Design – Seismic Load Effects & Combinations|

The applicable seismic load effects and combinations were used in the final calculations of every member of the lateral force resisting system. The applicable seismic load combinations for most elements were:

$$1.38D + 1.0E + [.5 \text{ or } 1.0]L + .2S$$

$$.72D + 1.0E$$

The applicable amplified seismic load combinations for the Special Concentrically Braced Frames in the north – south direction were:

$$1.38D + 2.0E + [.5 \text{ or } 1.0]L + .2S$$

$$.72D + 2.0E$$

The applicable amplified seismic load combinations for the Special Moment Frames in the north – south direction were:

$$1.38D + 3.0E + [.5 \text{ or } 1.0]L + .2S$$

$$.72D + 3.0E$$

## Structural Depth – ASCE 7-05 12.5: Seismic Design – Direction of Loading|

All of the subsections Section 12.5.1 through Section 12.5.4 applied to the loading of the analysis model of the Life Sciences Building. Section 12.5.4 directed that the analysis should meet the requirements of Section 12.5.3. Because the lateral force resisting system had a column that was shared between the systems in each orthogonal direction the procedures of Section 12.5.3a were used. This required that 100% of the lateral forces in one orthogonal direction be combined with 30% of the lateral forces in the other orthogonal direction. The worst case of the two was used to design the column. All other elements of the lateral force resisting system were able to be designed by applying 100% of the lateral force in each direction one at a time and using the worst case – consistent with the requirements of Section 12.5.2.

## Structural Depth – ASCE 7-05 12.6: Seismic Design – Analysis Procedure Selection|

Table 12.6-1 was used to determine the structural analysis method that was used to design the building lateral force resisting system. Initially to calculate rough sizes of the members by hand – using truss analysis and the portal method – the Equivalent Lateral Force Method was used. However, the Equivalent Lateral Force Method was not permitted to be used to calculate the seismic loading for the actual design of the structure. This was due to the Life Sciences Building having a Weight (Mass) Vertical Structural Irregularity because of the heavily loaded penthouse level on the top floor. This automatically forced the final design of the building lateral force resisting system to be done using the Modal Response Spectrum Analysis Method of Section 12.9.

## Structural Depth – ASCE 7-05 12.7.1: Seismic Design – Foundation Modeling|

The connections of the frames to the foundation were modeled as fixed. This is a conservative assumption and accurately reflects the actual connections as they were designed. All of the base connections consist of the member extending down into the foundation concrete (past the seismic base) for a substantial distance to develop shear and tension capacity in the concrete. A concrete grade beam was designed at the top level of the foundation to distribute the massive amounts of shear and tension that are developed as a result of seismic force reactions at the frame supports. This real life design results in a connection that can be idealized as fixed in the structural analysis model. The seismic base was taken at grade level. The concrete foundation flexibility and all levels below the assumed seismic base of the first floor level were not modeled. This is justified because the foundation up to the seismic base level – built of long and thick reinforced concrete walls – can be assumed to have a much greater stiffness than the steel frames it supports.

**Structural Depth – ASCE 7-05 12.7.2: Seismic Design – Effective Seismic Weight|**

The effective seismic weight was calculated for every level. It included 25% of the storage live load, the actual partition weight of 20 psf, the total operating weight of all permanent equipment, the superimposed dead loads for finishes, the superimposed dead loads for ceiling structures above mechanical rooms, and the live load for the mechanical penthouse.

**Structural Depth – ASCE 7-05 12.7.3: Seismic Design – Structural Modeling|**

An extensive and accurate model was created in ETABS using the member sizes determined using Equivalent Lateral Force Method forces and hand distribution methods (portal method, truss analysis). The model contained the lateral system with its members and connections modeled realistically, effective seismic weight (transformed to mass) distributed over rigid diaphragms, and the response spectrum parameters for the building site in Seattle, Washington. The automatic calculation procedures of ETABS were used several times to verify and check results. However hand calculations were preferred to ETABS throughout the design process.

The ETABS model was able to determine member forces and member and overall structure displacements. It also considered P – delta effects and panel zone deformations. Due to the building structure's irregularities the model was a 3D representation. ETABS exceeded the minimum requirement of two orthogonal degrees of freedom as translation across the plan and one degree of freedom as rotation about the axis of the plane defined by the plan.

Additional information about the ETABS model will be described as it relates to the following sections of the report.

**Structural Depth – ASCE 7-05 12.8: Seismic Design – Equivalent Lateral Force Procedure|**

Because the Equivalent Lateral Force Method (referred to herein as ELF) was only used at the beginning of design to determine the approximate base shear and story forces it will be covered very briefly. Only information from the sections of the ELF method that is relevant to the Modal Response Spectrum Analysis Method is included. The Modal Response Spectrum Analysis sections of the report will contain information that is more pertinent to the final redesign of the lateral force resisting system.

**Structural Depth – ASCE 7-05 12.8.1: Seismic Design – Seismic Base Shear|**

The seismic base shear determined for the structure in each direction is listed below. Note that the base shear in the Special Concentrically Braced Frame direction (north south) is much higher than the base shear in the Special Moment Frame direction (east west). This is due to the different R factor and period for each system. These seismic base shears were used to determine the preliminary forces to allow the rough design of members by hand before building a model in ETABS. The seismic base shears were then later used to scale the Modal Response Spectrum Analysis model in ETABS.

$$\mathbf{V_{ns} = 1538.51\ k}$$
$$\mathbf{V_{ew} = 660.81\ k}$$

**Structural Depth – ASCE 7-05 12.8.1.1: Seismic Design – Seismic Response Coefficient|**

Following the calculations in Section 12.8.1.1 and using the period found in Section 12.8.2 the seismic response coefficient was found for the structure in each of the orthogonal directions. The minimum seismic response coefficient was found to control. The values for the seismic response coefficient used in each direction are listed below:

$$\mathbf{C_{s,ns} = .180}$$
$$\mathbf{C_{s,ew} = .135}$$

**Structural Depth – ASCE 7-05 12.8.2: Seismic Design – Period Determination|**

The period was determined using the formula in Section 12.8.2.1 and the values from Table 12.8-2. The period was then multiplied by the coefficients in Table 12.8-1. The period for each direction is given below, along with the real period in each direction that was calculated using the Modal Response Spectrum Analysis of ETABS:

$$\mathbf{T_{a,ns} = .770\ s} \qquad \mathbf{T_{ETABS,ns} = .882\ s}$$
$$\mathbf{T_{a,ew} = 1.344\ s} \qquad \mathbf{T_{ETABS,ew} = 2.051\ s}$$

## Structural Depth – ASCE 7-05 12.8.3: Seismic Design – Story Forces|

It is important to note that these story forces were not used for the design of anything in the building. The only number from the Equivalent Lateral Force Analysis Method that was used was the base shear in each direction. They were used to scale the force ETABS model (not the displacement). These story forces are only provided as a reference for the story forces that the preliminary redesign based on ELF and hand calculations used. These story forces and their corresponding base shear were checked using the automatic load calculation feature of ETABS and found to be acceptable.

***NORTH SOUTH: SPECIAL CONCENTRICALLY BRACED FRAMES***

| | | | | | |
|---|---|---|---|---|---|
| | | **k=** | 1.0245 | | |
| | | | 608769.6 | | |
| **Level** | **Height Above Base** | **Story Weight** | **Cvx** | **Fx (story)** | **Mot** |
| ***Roof*** | 83 | 476 | 0.07 | 125.88 | 10447.96 |
| ***Penthouse*** | 56 | 4903 | 0.50 | 866.43 | 48519.87 |
| ***4th*** | 42 | 2632 | 0.20 | 346.38 | 14548.06 |
| ***3rd*** | 28 | 3102 | 0.15 | 269.47 | 7545.09 |
| ***2nd*** | 14 | 3102 | 0.08 | 132.46 | 1854.51 |
| | | | | 1740.62 | 82915 |

***EAST WEST: SPECIAL MOMENT FRAMES***

| | | | | | |
|---|---|---|---|---|---|
| | | **k=** | 1.230165 | | |
| | | | 1330748 | | |
| **Level** | **Height Above Base** | **Story Weight** | **Cvx** | **Fx (story)** | **Mot** |
| ***Roof*** | 83 | 476 | 0.08 | 60.90 | 5054.37 |
| ***Penthouse*** | 56 | 4903 | 0.52 | 386.56 | 21647.56 |
| ***4th*** | 42 | 2632 | 0.20 | 145.66 | 6117.85 |
| ***3rd*** | 28 | 3102 | 0.14 | 104.25 | 2919.05 |
| ***2nd*** | 14 | 3102 | 0.06 | 44.44 | 622.15 |
| | | | | 741.81 | 36361 |

## Structural Depth – ASCE 7-05 12.8.4: Seismic Design – Horizontal Forces / Torsion|

The horizontal distribution of forces to the various frames was done by the ETABS model performing a Modal Response Spectrum Analysis. The ETABS analysis considers inherent torsion, described in Section 12.8.4.1, that is a result of rigid diaphragms combined with a center of mass and center of rigidity that do not coincide. Accidental torsion, Section 12.8.4.2, is considered using a 5% eccentricity entered into ETABS. Per Modal Response Spectrum Analysis Section 12.9.5, "amplification of torsion per Section 12.8.4.3 is not required where accidental torsional effects are included in the dynamic analysis model."

## Structural Depth – ASCE 7-05 12.8.5: Seismic Design – Overturning|

Overturning was determined using the ELF forces in the previous story shear table. ELF forces are conservative because they are greater than the forces determined modally. The overturning moments are significant, especially for the Special Concentrically Braced Frames. The foundation design (not covered in this report) must address the overturning moments of the Special Concentrically Braced Frames.

## Structural Depth – ASCE 7-05 12.8.6: Seismic Design – Story Drift|

The story drift was determined by scaling the Modal Response Spectrum Analysis model. The displacements were found on the model by scaling the response spectrums in each orthogonal direction – creating two displacement response spectrum cases (also two force response spectrum cases). A scale factor was determined using the real period of the structure using ETABS, a scale factor (Cd / I), and the base shears determined using the real period (not approximate period).

Additional story drift considerations, including the values for allowable story drift are determined using Section 12.12 and its subsections. The maximum story drift for the Life Sciences Building will be computed as the "largest difference of deflections… of the story under consideration," per Section 12.12.1. The allowable story drift will be divided by the redundancy factor to meet the requirements of Section 12.12.1.1. The Life Sciences Building is in compliance with Sections 12.12.2 – Section 12.12.4.

The maximum story drifts for seismic forces acting in the north – south direction are as follows:

$$\Delta_4 = 1.6098"$$
$$\Delta_3 = 1.5091"$$
$$\Delta_2 = 1.5352"$$
$$\Delta_1 = .9598"$$

The maximum story drifts for seismic forces acting in the east – west direction are as follows (amplified by 6% for RBS moment connections):

$$\Delta_4 = 1.8814"$$
$$\Delta_3 = 1.9942"$$
$$\Delta_2 = 1.9707"$$
$$\Delta_1 = 1.4832"$$

The allowable story drift calculated with ASCE 7-05 using Table 12.12-1 is:

$$\Delta_a = .015(hsx) / \rho = 2.52"$$

## Structural Depth – ASCE 7-05 12.8.7: Seismic Design – P – Delta Effects|

P – Delta effects needed to be considered for the redesign of the lateral force resisting system for the Life Sciences Building. P – Delta effects were considered as part of the ETABS Modal Response Spectrum Analysis and are included as part of the results.

## Structural Depth – ASCE 7-05 12.9: Seismic Design – Modal Response Spectrum Analysis|

A Modal Response Spectrum Analysis was required for the Life Sciences Building. This was due to the building having several structural irregularities that could not be eliminated. The following sections will provide a greater insight into the development of the ETABS Modal Response Spectrum Analysis model.

## Structural Depth – ASCE 7-05 12.9.1: Seismic Design – Number of Modes|

A total of 9 natural modes of the structure were analyzed using the ETABS model. The combined modal mass participation was 99.99% in the east – west direction and 99.93% in the north – south direction. This was well above the target of 90% in each direction set forth in the code.

**N – S Participation = 99.93%**
**E – W Participation = 99.99%**
**Target Participation = 90%**

## Structural Depth – ASCE 7-05 12.9.2: Seismic Design – Modal Response Parameters|

The value for each force related parameter of interest was scaled as previously described in the Equivalent Lateral Force portion of this report. An Equivalent Lateral Force base shear was determined for each of the orthogonal directions of the structure using the approximate period. Then the Modal Response Spectrum Analysis model was modified to have two different response spectrum cases – one for the force in each orthogonal direction. The response spectrum cases were then scaled to make the Modal Response Spectrum Analysis base shear match the Equivalent Lateral Force Analysis base shear found using the approximate period.

The value for each displacement related parameter of interest was determined by scaling the Modal Response Spectrum Analysis model for two more response spectrum cases. The displacements were found on the model by scaling the response spectrums in each orthogonal direction – creating two displacement response spectrum cases (in addition to the two force response spectrum cases). A scale factor for each spectrum case was determined using the real period of the structure that was given by ETABS, a scale factor (Cd / I) depending on the system chosen, and the base shears that were determined using the real period (not approximate period).

## Structural Depth – ASCE 7-05 12.9.3: Seismic Design – Combined Response Parameters|

The combined response parameters were chosen from the options in ETABS to satisfy the requirements of ASCE 7-05. The ETABS modal combination uses Complete Quadratic Combination (CQC) and the ETABS direct combination uses Square Root of the Sum of Squares (SRSS) in accordance with ASCE 4. The Complete Quadratic Combination method considers coupling between closely spaced modes caused by modal damping. The Square Root of the Sum of Squares method does not take into account any modal coupling.

## Structural Depth – ASCE 7-05 12.9.4: Seismic Design – Scaling Design Values|

The majority of this Section was discussed under Section 12.9.2 with that discussion of scaling. The only thing to note is that the (.85)(V/Vt) force reduction was applicable in both orthogonal directions and applied to the ETABS model through scaling the two force spectrum cases.

## Structural Depth – ASCE 7-05 12.9.5: Seismic Design – Horizontal Shear Distribution|

The ETABS model automatically distributed the shear horizontally to the lateral force resisting frames in accordance with Section 12.8.4. Amplification of torsion was not required for the Modal Response Spectrum Analysis because the dynamic analysis model (ETABS model) considered accidental torsion effects.

## Structural Depth – ASCE 7-05 12.9.6: Seismic Design – P-Delta Effects|

P – Delta effects needed to be considered for the redesign of the lateral force resisting system for the Life Sciences Building. P – Delta effects were considered as part of the ETABS Modal Response Spectrum Analysis and are included as part of the results.

## Structural Depth – ASCE 7-05 12.9.7: Seismic Design – Soil Structure Interaction Reduction|

The soil structure interaction reduction was not considered as part of the ETABS Modal Response Spectrum Analysis model.

## Structural Depth – ASCE 7-05 Section 12.10: Diaphragm Design|

### Diaphragm Forces|

It was important to verify that the existing gravity framing system and concrete slab on composite metal deck would be able to transfer lateral (seismic) loads into the Special Concentrically Braced Frames and Special Moment Frames. The diaphragm was initially designed to transmit only wind loads in a low seismic region, and substantial changes were made to the lateral system resulting in a fewer number and greater spacing of frames. The diaphragm strength was verified using the Steel Deck Institute Diaphragm Design Manual (3rd Edition).

The diaphragm design forces were calculated with and verified to not be taken as less than the minimums as calculated using ASCE 7-05 Section 12.10.1.1. The minimum forces determined using ASCE 7-05 controlled in all but one case. At the penthouse level in the concentrically braced frame direction the modal response spectrum analysis determined that the diaphragm force is higher than the minimums found using the formula in ASCE 7-05. In every case the highest diaphragm force was used. Also in accordance with ASCE 7-05 the diaphragm forces were multiplied by the redundancy factor of 1.0. The shear force transmitted through the diaphragm to each seismic force resisting frame along its length at every floor was calculated and listed in the tables below:

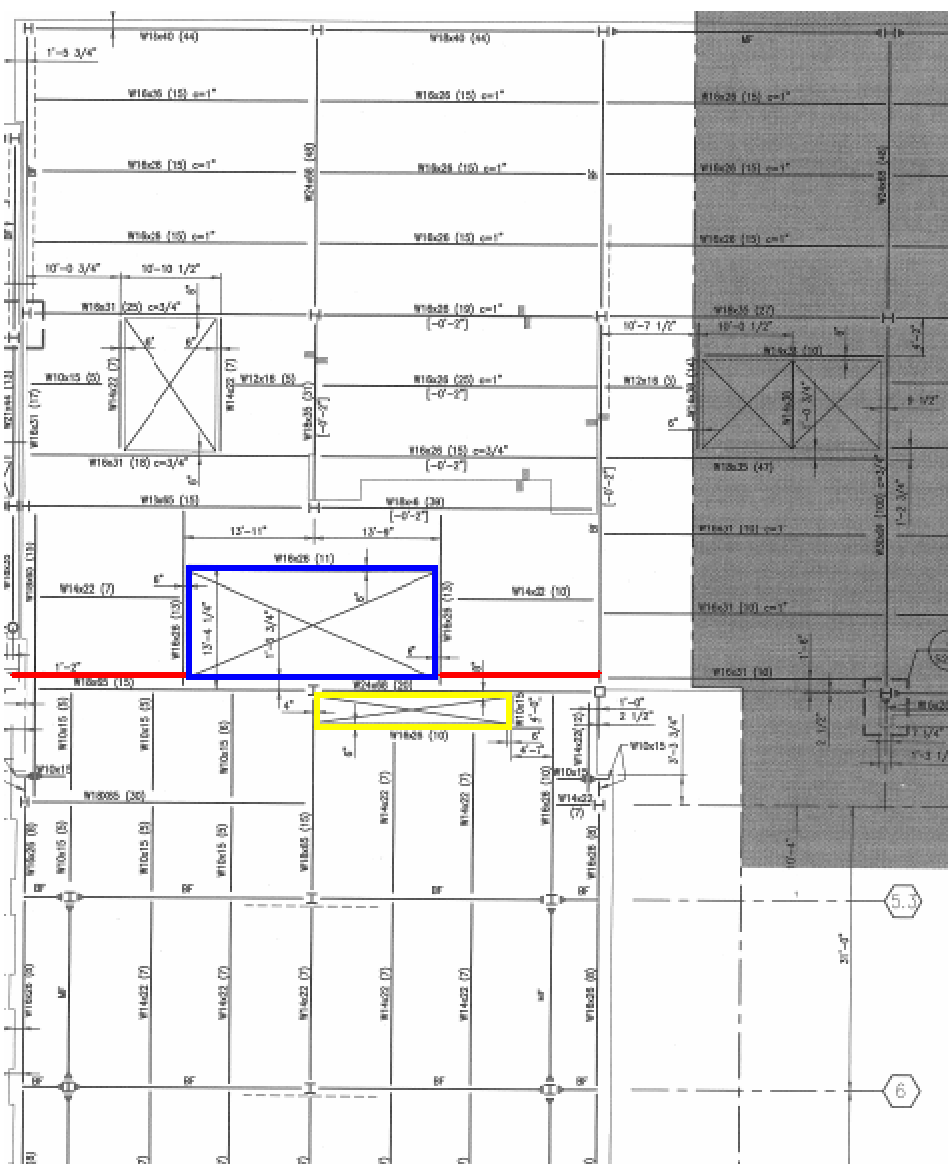
| ***PENTHOUSE DIAPHRAGM*** | | | | |
|---|---|---|---|---|
| ***E-W Frames*** | ***%*** | ***Shear*** | ***Length*** | ***Unit Shear*** |
| SMF - 1 | 51.5 | 599.1 | 186.0 | **3220.7 plf** |
| SMF - 4 | 48.5 | 564.2 | 183.5 | **3074.4 plf** |
| ***N-S Frames*** | | | | |
| SCBF - C | 36.2 | 448.4 | 22.7 | **19782.4 plf** |
| SCBF - G | 29.8 | 369.1 | 45.3 | **8142.3 plf** |
| SCBF - K.6 | 34.0 | 421.1 | 22.7 | **18580.1 plf** |

| ***TYPICAL FLOOR DIAPHRAGM*** | | | | |
|---|---|---|---|---|
| ***E-W Frames*** | ***%*** | ***Shear*** | ***Length*** | ***Unit Shear*** |
| SMF - 1 | 38.9 | 260.9 | 186.0 | **1402.9 plf** |
| SMF - 4 | 35.3 | 236.8 | 183.5 | **1290.4 plf** |
| SMF - 8 | 25.8 | 173.1 | 57.0 | **3036.3 plf** |
| ***N-S Frames*** | | | | |
| SCBF - C | 22.4 | 150.3 | 22.7 | **6619.4 plf** |
| SCBF - E | 23.7 | 159.0 | 22.7 | **7003.6 plf** |
| SCBF - G | 22.8 | 152.9 | 45.3 | **3376.3 plf** |
| SCBF - K.6 | 31.1 | 208.6 | 22.7 | **9190.4 plf** |

## Structural Depth – ASCE 7-05 Section 12.10: Diaphragm Design (continued)|

Diaphragm Forces (continued)|

The diaphragm forces that occur around the most critical section of the diaphragm were analyzed next. This only occurs at the second through fourth floors and the following framing plan is typical for all floors involved. The critical section occurs along the column line where the long and short legs of the 'L' intersect. Stress concentrations at this location are expected to be the highest. The following is a diagram showing the critical section of the diaphragm (critical section shown in red, diaphragm opening for stair shown in blue, removed relocated diaphragm opening shown in yellow):



**Structural Depth – ASCE 7-05 Section 12.10: Diaphragm Design (continued)|**

Diaphragm Forces (continued)|

The maximum shear along the critical section was found to be (greatest for any floor) 31k. When the maximum shear is divided by the length of the critical section (length of intersection minus opening = 31') the shear at the critical section is found to be an extremely manageable 900 lbs/ft. However during the critical section analysis it was discovered that for the shear to be resisted by Special Moment Frame 4 a collector needs to be designed for the entire length of column line 4.

Diaphragm Strength|

The diaphragm strength was calculated using the SDI – Diaphragm Design Manual, 3rd Edition. The strength of the diaphragm was calculated using the formulas given in Section 2 of the manual and the tables given in Appendix V of the manual. Because the diaphragm used in the Life Sciences Building was similar to the diaphragms listed in the tables – except for the strength and thickness of concrete – the tables were able to be used to aid in the strength evaluation. The values given in the table were adjusted to compensate for the thicker slab and stronger concrete.

The table in Appendix V on page AV-97 was used because it most closely matched the existing concrete on composite deck assembly. The existing diaphragm system for the Life Sciences Building consisted of:

**2" Composite Metal Deck, 18 gage (t = .0474")**
**4.5" Normal Weight Concrete Topping, f'c = 4000 psi**
**WWF4x4 – W5.5 x W5.5**
**Span = 10' (maximum)**

The table that was used to evaluate the strength of the existing diaphragm was based on:

**2" Composite Metal Deck, 18 gage (t = .0474")**
**2.5" Normal Weight Concrete Topping, f'c = 3000 psi**
**WWF6x6 – W1.4xW1.4**
**Span = 10' (maximum)**

## Structural Depth – ASCE 7-05 Section 12.10: Diaphragm Design (continued)|

Diaphragm Strength|

The values in the table were adjusted to compensate for the thicker and stronger concrete using simple formulas in Chapter 2. The increase in concrete strength and thickness gave a net increase in the unit shear strength of the diaphragm of 5300 lbs/foot. The final design of the deck became:

**2” Composite Metal Deck, 18 gage (t = .0474”)**
**4.5” Normal Weight Concrete Topping, f’c = 4000 psi**
**WWF4x4 – W5.5 x W5.5**
**Span = 10’ (maximum)**
**8 Side Lap Welds per Span, 5/8” Puddle or 1-1/2” Fillet**
**1 Structure Weld per 1’ of Bearing, 5/8” Puddle**

$$\Phi V_{max,diaphragm} = 6115 \text{ lbs/ft}$$
$$\Phi_{filled\ diaphragm} = .5$$

Necessary Changes|

In order to transfer the forces from the diaphragm in the short leg of the “L” into Special Moment Frame 4 a collector element (designed of steel according to the AISC Seismic Provisions) needs to run the entire length of column line 4. This collector can just be a redesign of the existing beams to make sure that they are capable of taking the force from the diaphragm and transmitting it into Special Moment Frame 4. All other moment frames have enough length to keep the shear stresses in the diaphragm within their limits.

Collectors must also be designed for every Special Concentrically Braced Frame at every level. Examining the diaphragm force table at the beginning of this diaphragm section it can be noted that the shears at the Special Concentrically Braced Frame at all of the levels are well over the maximum shear that the diaphragms can handle. This is probably due to the high forces in the Special Concentrically Braced Frame system and their relatively short interface with the diaphragm. In order to make sure that the forces are able to be transferred from the diaphragm into the Special Concentrically Braced Frames collectors must be designed along column lines C, E, G, and K.6. These collectors should be designed using the AISC Seismic Provisions and use overstrength load combinations.

## Structural Depth – ASCE 7-05 Section 12.10: Diaphragm Design (continued)

Necessary Changes (continued)
Finally, structural steel members of the seismic force resisting system and collectors should have enough shear studs per foot to develop the shear strength of the diaphragm. This should not be a problem because the building was designed using composite steel beams and girders. However, the seismic force resisting system was not designed using composite steel because the concrete cannot be depended on in tension. Because the building uses shear studs with a capacity of 13.3k and the diaphragm has a capacity of 11.215k the maximum spacing between shear studs should be 14" on collectors and other members of the seismic force resisting system. The spacing on Special Moment Frames is already 12" between the protected zones to avoid additional bracing. Care should also be taken to avoid welding shear studs in the protected zones of Special Moment Frames and Special Concentrically Braced Frames.

The new diaphragm shear values at the diaphragm – seismic force resisting system intersection are shown in the table below. These values are determined assuming collectors for the Special Concentrically Braced Frames C, G, K.6 extend the entire width of the long leg of the "L" (between Special Moment Frames 1 and 4). The values also assume that the collector for Special Concentrically Braced Frame E runs to Special Moment Frame 4. The only location that still exceeds the diaphragm shear strength is Special Concentrically Braced Frame C; only at the penthouse level. This is considered acceptable because the forces from the modal analysis were used to design the penthouse level. These forces were considerably higher than the forces that were obtained using the diaphragm force formula in Section 12.10 of ASCE 7-05. Therefore the design still meets ASCE 7-05 code requirements.

| ***PENTHOUSE DIAPHRAGM*** | | | | |
|---|---|---|---|---|
| ***E-W Frames*** | ***%*** | ***Shear*** | ***Length*** | ***Unit Shear*** |
| SMF - 1 | 51.5 | 599.1 | 186.0 | **3220.7 plf** |
| SMF - 4 | 48.5 | 564.2 | 245.5 | **2298.0 plf** |
| ***N-S Frames*** | | | | |
| SCBF - C | 36.2 | 448.4 | 72.0 | **6227.6 plf** |
| SCBF - G | 29.8 | 369.1 | 144.0 | **2563.3 plf** |
| SCBF - K.6 | 34.0 | 421.1 | 72.0 | **5849.1 plf** |

| ***TYPICAL FLOOR DIAPHRAGM*** | | | | |
|---|---|---|---|---|
| ***E-W Frames*** | ***%*** | ***Shear*** | ***Length*** | ***Unit Shear*** |
| SMF - 1 | 38.9 | 260.9 | 186.0 | **1402.9 plf** |
| SMF - 4 | 35.3 | 236.8 | 245.5 | **964.5 plf** |
| SMF - 8 | 25.8 | 173.1 | 57.0 | **3036.3 plf** |
| ***N-S Frames*** | | | | |
| SCBF - C | 22.4 | 150.3 | 72.0 | **2087.0 plf** |
| SCBF - E | 23.7 | 159.0 | 84.7 | **1877.8 plf** |
| SCBF - G | 22.8 | 152.9 | 144.0 | **1062.1 plf** |
| SCBF - K.6 | 31.1 | 208.6 | 72.0 | **2897.5 plf** |

## Structural Depth – Final Lateral System Design Process|

All of the prior thesis technical assignments provided very limited experience with the seismic design provisions of ASCE 7-05. The Life Sciences Building's original design location of State College, Pennsylvania placed it in Seismic Design Category "A". A building being in Seismic Design Category "A" means that ASCE 7-05 Chapter 12 doesn't even need to be consulted to calculate lateral loads. The story shears due to seismic loading were equal to the seismic weight of each floor multiplied by .01.

The first step in design was to find a location that would allow for a Seismic Design Category of "D". The site was preferably on a college campus and some way to determine the site class was also needed. The campus of the University of Washington in Seattle, Washington was chosen.

Due to the inefficiency of the existing lateral system, the design of the seismic force resisting system started by examining the architectural floor plans. The layout and design of the existing system was not considered in the layout and design of the redesigned system. Special Concentrically Braced Frames were considered as the preferred seismic force resisting system due to their reputation as the most economical choice. Specific locations on the architectural plans where concentrically braced frames could be used were highlighted and considered. The most feasible locations were then chosen for further consideration.

Special Concentrically Braced Frames were able to be used in the north – south direction. Structural changes to accommodate Special Concentrically Braced Frames included changing some light gage metal stud framed cavity walls to concrete shear walls. These concrete shear walls were needed to bring the seismic base up to the base level of the steel seismic force resisting systems. Special Concentrically Braced Frames were able to be used successfully because they fit in the walls that were along staircases and mechanical shafts. They were also able to be used in exterior walls between punched window openings.

In the east – west direction suitable locations to use Special Concentrically Braced Frames did not exist. The preferred alternative for the seismic force resisting system in the east – west direction became Special Moment Frames. Special Moment Frames are the least efficient use of structural steel. But, architectural flexibility was needed in the east – west direction and no other seismic force resisting system would work. Most of the Special Moment Frames in the structural redesign used existing column lines and beam lines. By using existing column lines and beam lines the need to redesign the gravity framing system was eliminated. There was the addition of one column that didn't exist in the previous structural system. Only one other change was made to the existing structural steel layout, a Round HSS column was changed to a W – shape so that the column could become part of a Special Moment Frame. The addition of concrete shear walls to raise the seismic base to a uniform level was also required throughout the ground level.

## Structural Depth – Final Lateral System Design Process (continued)|

The lateral system design began with four iterations of hand calculations using story shears determined from the Equivalent Lateral Force Method (ignoring all irregularities). The story shears and base shear were used to perform truss analysis on the braced frames and portal method analysis on the moment frames so that rough member sizes could be obtained. Each time the design was placed into ETABS so that the story drifts and irregularities could be evaluated. After several changes to members in the ETABS model to control drifts and attempt to limit irregularities a final lateral force resisting system was settled on.

The final design of the lateral force resisting system was ready to be subjected to the rigorous standards for structural steel set forth in the AISC Seismic Provision. Seismic forces from the Modal Response Spectrum Analysis in ETABS were combined with hand calculated gravity loads using seismic load combinations for every member of the lateral force resisting system. The worst case loading for every member was chosen from all of the similar frames. The forces on the worst case member governed the design of all of the members for the similar frames. This was done in order to have as many elements repeat as possible and also use the smallest amount of different structural steel shapes possible – as constructability and economy issues. These forces were then checked using the AISC Seismic Provisions for Special Concentrically Braced Frames and Special Moment Frames.

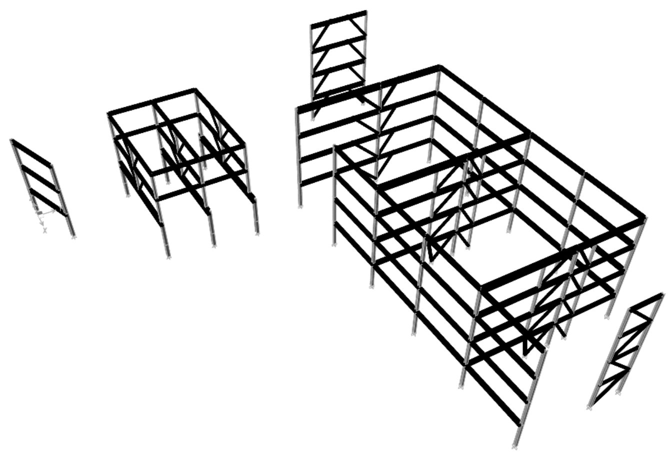
*A brief summary of the results of the AISC Seismic Provision check follows; however, reviewing the hundreds of pages of calculations and sketches that are provided in the appendix is encouraged. The entire lateral force resisting system from frames to collectors to diaphragms – even including all of the connections – was evaluated and detailed. It is nearly impossible to sum up everything that the calculations in the appendix contain in a concise and organized manner using a word processor format. Great care was taken to make sure that the calculations are legible, organized, and easy to follow. They should be able to explain themselves as a stand alone document without a written formal report.*

## Structural Depth – Final Lateral System Design Summary|

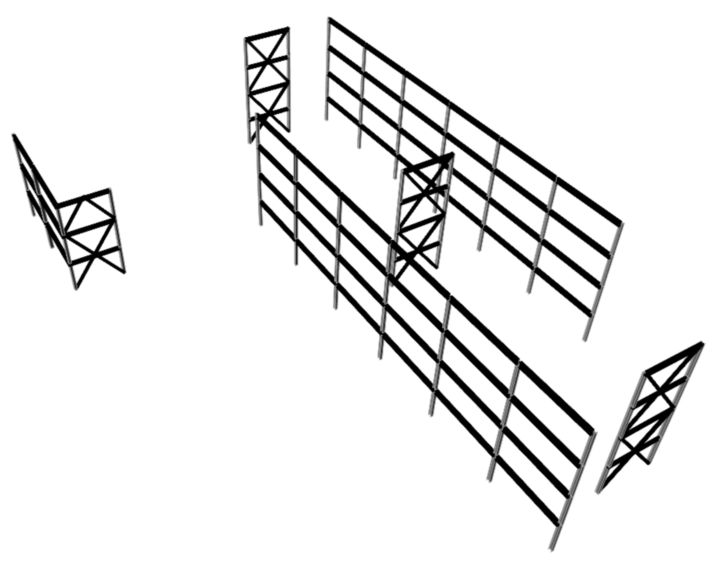
The redesigned lateral system resulted in a total of four Special Concentrically Braced Frames and three Special Moment Frames. Three of the four Special Concentrically Braced Frames are identical (same dimensions, members, connections). The third Special Concentrically Braced Frame has the same dimensions but a shorter height and smaller members due to its location in the shorter part of the building. Two of the three Special Moment Frames are similar, they have the same members and connections, however the end bay of one of the frames is shorter than the typical 31' span.

The details of the final design of the lateral force resisting system will be broken down over the next several pages.

Existing Lateral System (Wind) – State College, PA|



Redesigned Lateral System (Seismic) – Seattle, WA|



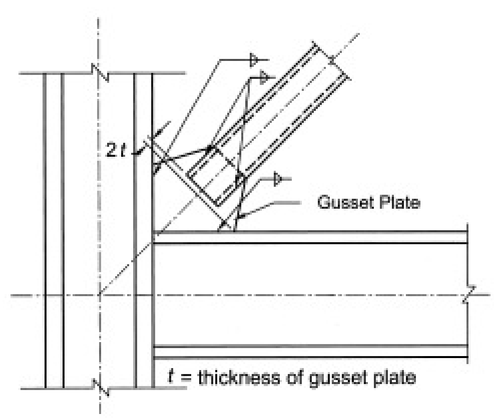
## Structural Depth – Final Lateral System Design Summary (continued)|

### Special Concentrically Braced Frames – C, E, G, K.6|

Special Concentrically Braced Frames C, E, G, and K.6 were designed using the AISC Seismic Provisions. Every member used in the design of the frame had to meet seismic local buckling requirements before design could begin. The frames were first checked by verifying that the strength of the braces, columns, and beams was sufficient to resist the controlling load cases (sometimes amplified seismic load with overstrength was used).

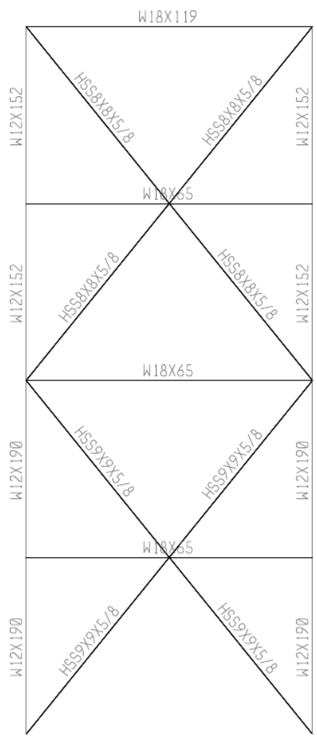
Next the connections were designed to develop the full strength of the bracing members. The HSS bracing was attached to the connection gusset plate by cutting a slot down two sides of the brace, sliding it over the gusset plate, then welding it into place with a fillet weld on each edge (4 total fillet welds). The HSS bracing members needed to be reinforced at their connections with the gusset plates. This was achieved by welding a reinforcing plate using a longitudinal fillet weld along the entire length of the plate to the face of each unnotched side of the HSS at the connection location. The connection of beams to the columns was designed as part of the gusset plate – brace system. Column splices are needed to be located at least four feet above the beam – column connection and must be made with partial joint penetration welds that develop half of the flexural capacity of the smaller column.

The dimensions of the connection of the bracing members at the gusset plates were set up so the end of the brace and its Whitmore section were offset between 2x-3x the thickness of the gusset plate to allow for plastic hinging in the gusset plate. Due to the Whitmore section and the 2x-3x thickness offset the length of the brace is shortened substantially. The new shorter length of the bracing member is used to determine the maximum compression force that will be applied to the gusset plate. The new maximum tension and compression forces that can be applied by the brace to the gusset plate are then used to check the limit states of the gusset plate and the beams and columns the gusset plate is welded to.



**Structural Depth – Final Lateral System Design Summary (continued)|**

Special Concentrically Braced Frames – C, E, G, K.6|
The detailed dimensions of welds and connections for frames C, G, and K.6 are shown below in tabular form. (Please see the calculation appendix for more detailed descriptions and diagrams):




**SCBF – C, G, K.6**

| | |
|---|---|
| **Floor To Floor Height** | **14'-0"** |
| **Length** | **22'-8"** |
| **Members** | **(as shown)** |
| **Seismic Local Buckling** | **OK** |
| **Gusset Plates** | |
| **Plate Thickness** | **t = 1.5"** |

| **HSS8x8x5/8 to Gusset Plate** | | **HSS9x9x5/8 to Gusset Plate** | |
|---|---|---|---|
| **Weld Length (4 Welds)** | **L = 19"** | **Weld Length (4 Welds)** | **L = 22"** |
| **Full Length Fillet Weld Thickness** | **D = 5/8"** | **Full Length Fillet Weld Thickness** | **D = 5/8"** |
| **HSS8x8x5/8 Reinforcement Plates** | | **HSS9x9x5/8 Reinforcement Plates** | |
| **Number at Each End** | **QTY = 2** | **Number at Each End** | **QTY = 2** |
| **Plate Thickness** | **t = 7/8"** | **Plate Thickness** | **t = 7/8"** |
| **Plate Width** | **w = 4.5"** | **Plate Width** | **w = 5.5"** |
| **Plate Length** | **l = 19"** | **Plate Length** | **l = 22"** |
| **Weld Length (4 Welds)** | **L = 19"** | **Weld Length (4 Welds)** | **L = 22"** |
| **Full Length Fillet Weld Thickness** | **D = 1/4"** | **Full Length Fillet Weld Thickness** | **D = 5/16"** |

## Structural Depth – Final Lateral System Design Summary (continued)|

Special Concentrically Braced Frames – C, E, G, K.6|

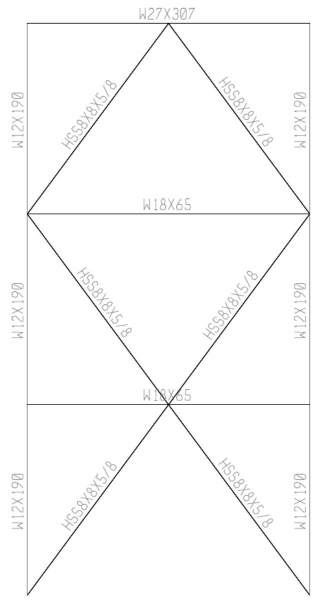
The detailed dimensions of welds and connections for frames C, G, and K.6 are continued below in tabular form. (Please see the calculation appendix for more detailed descriptions and diagrams):

| | |
|---|---|
| **X W18x65 to Gusset Plate (HSS8x8x5/8)** | |
| **Weld Length (2 Welds)** | **L = 62.75"** |
| **Full Length Fillet Weld Thickness** | **D = 7/8"** |
| | |
| **X W18x65 to Gusset Plate (HSS9x9x5/8)** | |
| **Weld Length (2 Welds)** | **L = 62.75"** |
| **Full Length Fillet Weld Thickness** | **D = 1"** |
| | |
| **K W12x152 to W18x119 to Gusset** | |
| **Weld Length on Column (2 Welds)** | **L = 23.875"** |
| **Full Length Fillet Weld Thickness on Column** | **D = 5/8"** |
| **Weld Length on Beam (2 Welds)** | **L = 29.5"** |
| **Full Length Fillet Weld Thickness on Beam** | **D = 5/8"** |
| | |
| **K W12x190 to W18x65 to W12x190 to Gusset** | |
| **Weld Length on Column (2 Welds)** | **L = 24", 27.5"** |
| **Full Length Fillet Weld Thickness on Column** | **D = 1"** |
| **Weld Length on Beam (2 Welds)** | **L = 29.5", 34"** |
| **Full Length Fillet Weld Thickness on Beam** | **D = 3/4"** |
| | |
| **K W12x152 to Baseplate to Gusset** | |
| **Weld Length on Column (2 Welds)** | **L = 43.5"** |
| **Full Length Fillet Weld Thickness on Column** | **D = 1/2"** |
| **Weld Length on Base (2 Welds)** | **L = 20.5"** |
| **Full Length Fillet Weld Thickness on Base** | **D = 1-1/4"** |

## Structural Depth – Final Lateral System Design Summary (continued)|

Special Concentrically Braced Frames – C, E, G, K.6|

The detailed dimensions of welds and connections for frame E are shown below in tabular form. (Please see the calculation appendix for more detailed descriptions and diagrams):




**SCBF – E**

| | |
|---|---|
| **Floor To Floor Height** | **14'-0"** |
| **Length** | **22'-8"** |
| **Members** | **(as shown)** |
| **Seismic Local Buckling** | **OK** |
| **Gusset Plates** | |
| **Plate Thickness** | **t = 1.5"** |

**HSS8x8x5/8 to Gusset Plate**

| | |
|---|---|
| **Weld Length (4 Welds)** | **L = 19"** |
| **Full Length Fillet Weld Thickness** | **D = 5/8"** |

**HSS8x8x5/8 Reinforcement Plates**

| | |
|---|---|
| **Number at Each End** | **QTY = 2** |
| **Plate Thickness** | **t = 7/8"** |
| **Plate Width** | **w = 4.5"** |
| **Plate Length** | **l = 19"** |
| **Weld Length (4 Welds)** | **L = 19"** |
| **Full Length Fillet Weld Thickness** | **D = 1/4"** |

**K W12x190 to W18x65 to W12x190 to Gusset**

| | |
|---|---|
| **Weld Length on Column (2 Welds)** | **L = 24"** |
| **Full Length Fillet Weld Thickness on Column** | **D = 1"** |
| **Weld Length on Beam (2 Welds)** | **L = 29.5"** |
| **Full Length Fillet Weld Thickness on Beam** | **D = 3/4"** |

**K W12x190 to Baseplate to Gusset**

| | |
|---|---|
| **Weld Length on Column (2 Welds)** | **L = 43.5"** |
| **Full Length Fillet Weld Thickness on Column** | **D = 1/2"** |
| **Weld Length on Base (2 Welds)** | **L = 20.5"** |
| **Full Length Fillet Weld Thickness on Base** | **D = 1"** |

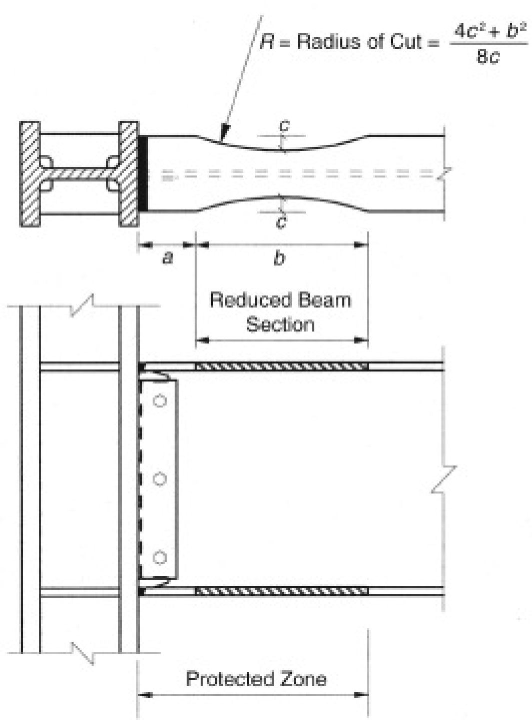
**Structural Depth – Final Lateral System Design Summary (continued)|**

Special Moment Frames – 1, 4, 8|

Special Moment Frames 1, 4, and 8 were designed using the AISC Seismic Provisions and AISC 358-05 for prequalified Special Moment Frame connections. Every member used in the design of the frame had to meet seismic local buckling requirements before design could begin. The frames were first checked by verifying that the strength of the braces, columns, and beams was sufficient to resist the controlling load cases (sometimes amplified seismic load with overstrength was used). The worst case for all of the similar members was used to design the rest of the members in the rest of the similar frames.

First the prequalified reduced beam section (RBS) connections were designed for all of the beams. A diagram showing a typical RBS connection is shown below. Next the flexural and shear strength of the beam was checked and bracing was designed if needed. Then the columns were checked using the column beam moment ratio guidelines in the AISC Seismic Provisions. After the columns were verified to meet the column beam moment ratio guidelines they were checked for strength. Last, continuity plates and doubler plates were designed if needed or the column size was increased (only while it was still economical) to eliminate the need for continuity plates and doubler plates. Column splices should be designed using demand critical complete joint penetration welds that develop the flexural capacity of the smaller column spliced and are needed to be located at least four feet above the beam – column connection.

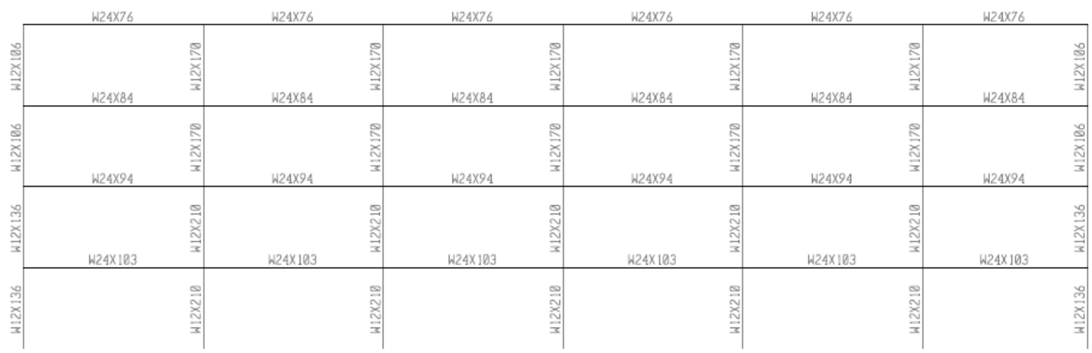
It was noted previously, but will be mentioned again, RBS connections increase the drift of the Special Moment Frames by a percentage that can be calculated using formulas in the AISC Seismic Provisions and AISC 358-05. The RBS connections designed for the Life Sciences Building increased the drift by about 6%.



## Structural Depth – Final Lateral System Design Summary (continued)|

Special Moment Frames – 1, 4, 8|

Pertinent information about the detailing of Special Moment Frame 1 and 4 is shown below. Also an elevation of Special Moment Frame 1 is shown below, Special Moment Frame 4 has identical construction (members, RBS, connections, continuity plates, doubler plates) but one bay of beams on the end is shorter:




All of the beams in Special Moment Frame 1 and 4 have the same details for connections and bracing, the only difference is that they are different W-shape sections. The typical reduced beam section connection used in SMF-1 and SMF-4 has the following values and the spacing of bracing is also given. The bracing assumes that shear studs will be spaced between the protected zones of the beams no greater than 12" apart. (See the appendix for more detailed information):

| **RBS Dimensions** | **Bracing Spacing** |
|---|---|
| **a = 5"** | **QTY: 4** |
| **b = 18"** | **3.375' from CL** |
| **c = 1.35"** | **10.125' from CL** |

The requirements for continutity plates and doubler plates for each of the column shapes is listed below:

| | **Doubler Plate** | **Continuity Plate** |
|---|---|---|
| **W12x210** | **(2) .75" thick** | **None** |
| **W12x170** | **(2) .75" thick** | **None** |
| **W12x136** | **(1) .5" thick** | **5" x .625"** |
| **W12x106** | **(1) .5" thick** | **5" x .75"** |

All welds in Special Moment Frame 1 and Special Moment Frame 4 will be complete joint penetration groove welds which are considered to be demand critical. The only welds not considered demand critical on the Special Moment Frames are welds for doubler and continuity plates.

## Structural Depth – Final Lateral System Design Summary (continued)|

Special Moment Frames – 1, 4, 8|

Pertinent information about the detailing of Special Moment Frame 8 is shown below. Also an elevation of Special Moment Frame 8 is shown below:


All of the beams in Special Moment Frame 8 have the same details for connections and bracing, the only difference is that they are different W-shape sections. The typical reduced beam section connection used in SMF-8 has the following values and the spacing of bracing is also given. The bracing assumes shear studs spaced at 12" between the protected zones. (See the appendix for more detailed information):

| **RBS Dimensions** | **Bracing Spacing** |
|---|---|
| **a = 7"** | **QTY: 3** |
| **b = 18"** | **at CL** |
| **c = 2"** | **10' from CL** |

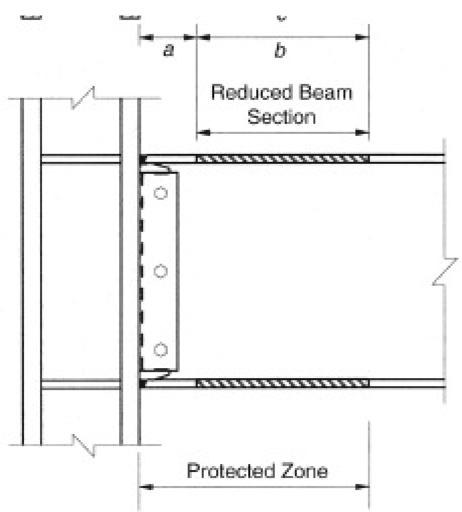
The requirements for continuity plates and doubler plates for each of the column shapes is listed below:

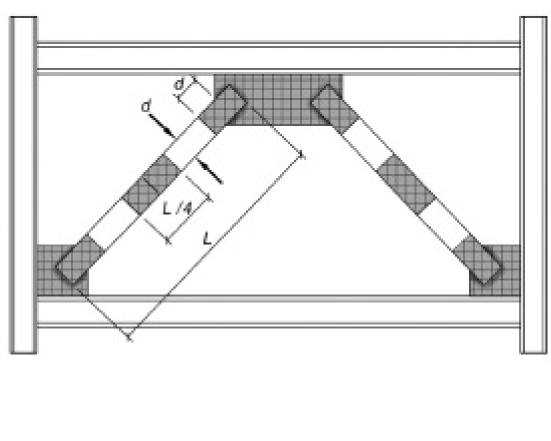
| | | **Doubler Plate** | **Continuity Plate** |
|---|---|---|---|
| **4th Floor** | **W14x311** | **(2) 1.75" thick** | **None** |
| **4th Floor** | **W12x190** | **(1) 1.5" thick** | **5.75" x 1.5"** |
| **2nd / 3rd Floor** | **W14x311** | **(2) 2.75" thick** | **None** |
| **2nd / 3rd Floor** | **W12x190** | **(2) 1.25" thick** | **5.75" x 1.5"** |

All welds in Special Moment Frame 8 will be complete joint penetration groove welds which are considered to be demand critical. The only welds not considered demand critical on the Special Moment Frames are welds for doubler and continuity plates.

**Structural Depth – Final Lateral System Design Summary (continued)|**

Protected Zones|
Are defined by AISC as areas of members in which limitations apply to fabrication and attachments. They are basically areas of the seismic force resisting system where nothing can be welded, no penetrations or cuts can be made, no shear studs can be attached, and no bolted, screwed, or shot in attachments can be made. Illustrations of the protected zones for Special Moment Frames and Special Concentrically Braced Frames are made below:



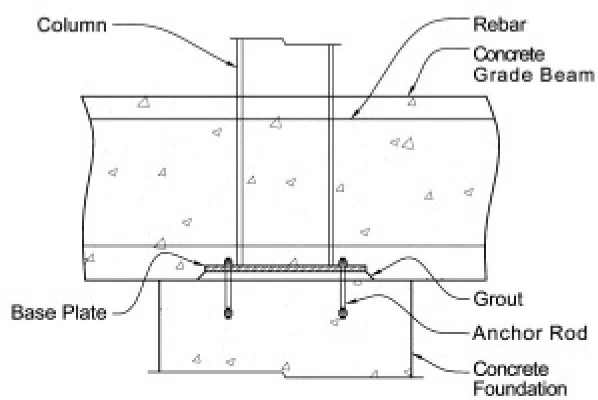
## Structural Depth – Final Lateral System Design Summary (continued)|

Analysis and Design Limits|

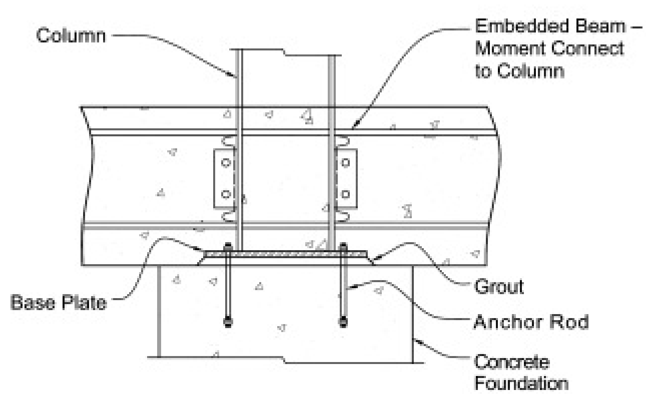
Due to the limited amount of time to complete this exercise, several aspects of the of the change from a wind controlled lateral force resisting system to a seismic controlled lateral force resisting system couldn't be completed because my focus was on the steel system.

The connections of the Special Moment Frames to the foundation and shear walls should be developed as shown in the details below. The details were chosen because embedding the steel column deep into the concrete will help distribute the concentrated shear forces from the moment frames into the concrete better. This is also an easy and cost effective way to make sure that the bases of the moment frame columns are actually fixed (consistent with the modeling assumptions). The design of these bases would be done consulting the AISC 13th Edition Steel Manual, ACI 318-08, and more specifically AISC Design Guide 1 and Appendix D of ACI 318-08.

The maximum factored shear force due seismic loading for MF-1 is about (44k / column), for MF-4 about (43k / column), and for MF-8 about (80k / column). The maximum factored moment due seismic loading for MF-1 is about (371k-ft / column), for MF-4 about (355k-ft / column), and for MF-8 about (672k-ft / column).




*(a)*



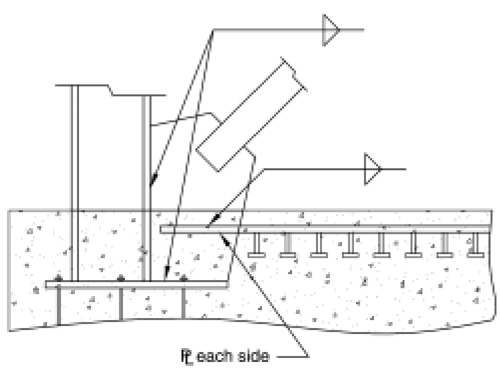

*(b)*

## Structural Depth – Final Lateral System Design Summary (continued)|

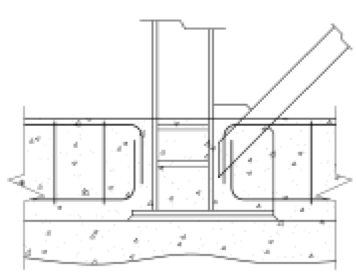
Analysis and Design Limits|

The base connection of the Concentrically Braced Frames should be developed as shown in the details below to help the concrete handle the large shear and tensile forces that these frames develop. It will be important to put in enough steel reinforcement (boundary elements) below and around the steel column bases to help distribute the shear and tension throughout the concrete. This connection detail would be designed using the AISC 13th Edition Steel Manual, ACI 318-08, and more specifically AISC Design Guide 1 and Appendix D of ACI 318-08. It would also require the redesign of the foundation at all of the Special Concentrically Braced Frames (a cost only partially reflected in the breadth study).

The largest factored tension force developed by a Special Concentrically Braced Frame at its baseplate is about 822k (even counting the effect of .72DL acting downward on the column). The maximum factored shear force resisted by the baseplate of a Special Concentrically Braced Frame is 230k (entire truss shear is 460k).




(a)



(b)

## Architecture Breadth – Introduction / Goals|

Although the resistance of seismic forces is primarily the concern of the structural engineer; the magnitude of these seismic forces and the systems that can be used to resist them are influenced greatly by choices made by the architect regarding the design of the building. The building architecture is required by code to accommodate whatever structural system is needed to resist seismic forces. However, the building structure should make every attempt possible to accommodate the functional and aesthetic aims of the architect's vision to whatever extent is considered economical. The goal of the structural redesign of the Life Sciences Building was to design a new structure that could resist seismic forces and place it into the architectural shell of the Life Sciences Building that was previously designed while changing as little of the building architecture as possible.

Three categories have been defined (Naeim, 2001) that classify the architectural design decisions that influence the building's seismic performance. The first category, defined as the "building configuration" includes the building's size, shape, and proportions – the geometrical properties of the building's form. The second category is "structurally restrictive detailed architectural design" and includes all architectural details that may not be compatible with the required seismic details – an example being wall or column detail or frame layout that cannot possibly meet known engineering standards. The third and final category of architectural design decisions which influence the building's seismic performance is "hazardous nonstructural components." This category includes nonstructural components of the building which have been detailed by the architect but may not be designed to resist seismic forces. These nonstructural components, when subjected to the forces and displacements of an earthquake, could become life safety hazards.

Factors involving the first category, building configuration, for the Life Sciences Building were determined prior to this study and needed to be left unchanged for this study. A complete architectural and structural redesign of the building would be impossible to complete in time. This presented additional complications because the Life Sciences Building was initially designed, proportioned, and detailed to be built for the lowest seismic design category (SDC "A") which is used in State College, Pennsylvania. A result of the relocation of the Life Sciences Building to Seattle, Washington was a change in seismic design category to SDC "D". This left the building poorly configured and proportioned in regard to resisting the effects of an earthquake.

**Architecture Breadth – Introduction / Goals (continued)|**

This presented several complications for structural redesign that could only be resolved through changes to the building in the second category – structurally restrictive detailed architectural design. These changes to the building detailing resulted in a few minor changes to the plan and several modifications to the architectural details from the previous low seismic design which will be outlined in the following breadth study. A great majority of the substantial changes to the structure were able to be made while still preserving the original function and aesthetics of the building which was one of the goals of the structural redesign.

Changes involving the third category, specifically the seismic design and detailing of non structural components would need to be considered if the Life Sciences Building was actually constructed in Seattle, Washington. Due to limited time and the desire to focus on other aspects of seismic design the design of non structural components will not be considered as part of this breadth study.

## Architecture Breadth – Design Process|

The goal of the structural redesign with regard to the building architecture, as stated previously, was to design a new structure that could resist seismic forces while changing as little of the existing building architecture as possible. The existing lateral system was determined to be very inefficient in Technical Assignment III – many of the frames resisted a negligible amount of lateral loading. The existing lateral system also contained too many irregularities to be considered as a seismic force resisting system.

Due to the inefficiency of the existing lateral system, the design of the seismic force resisting system began using only the architectural floor plans. No consideration was given to the layout and design of the previous system. Special Concentrically Braced Frames were considered as the preferred seismic force resisting system due to their reputation as the most economical choice. Specific locations on the architectural plans where braced frames could be used were highlighted. The best locations of the many highlighted were studied further to develop a preliminary lateral force resisting system layout.

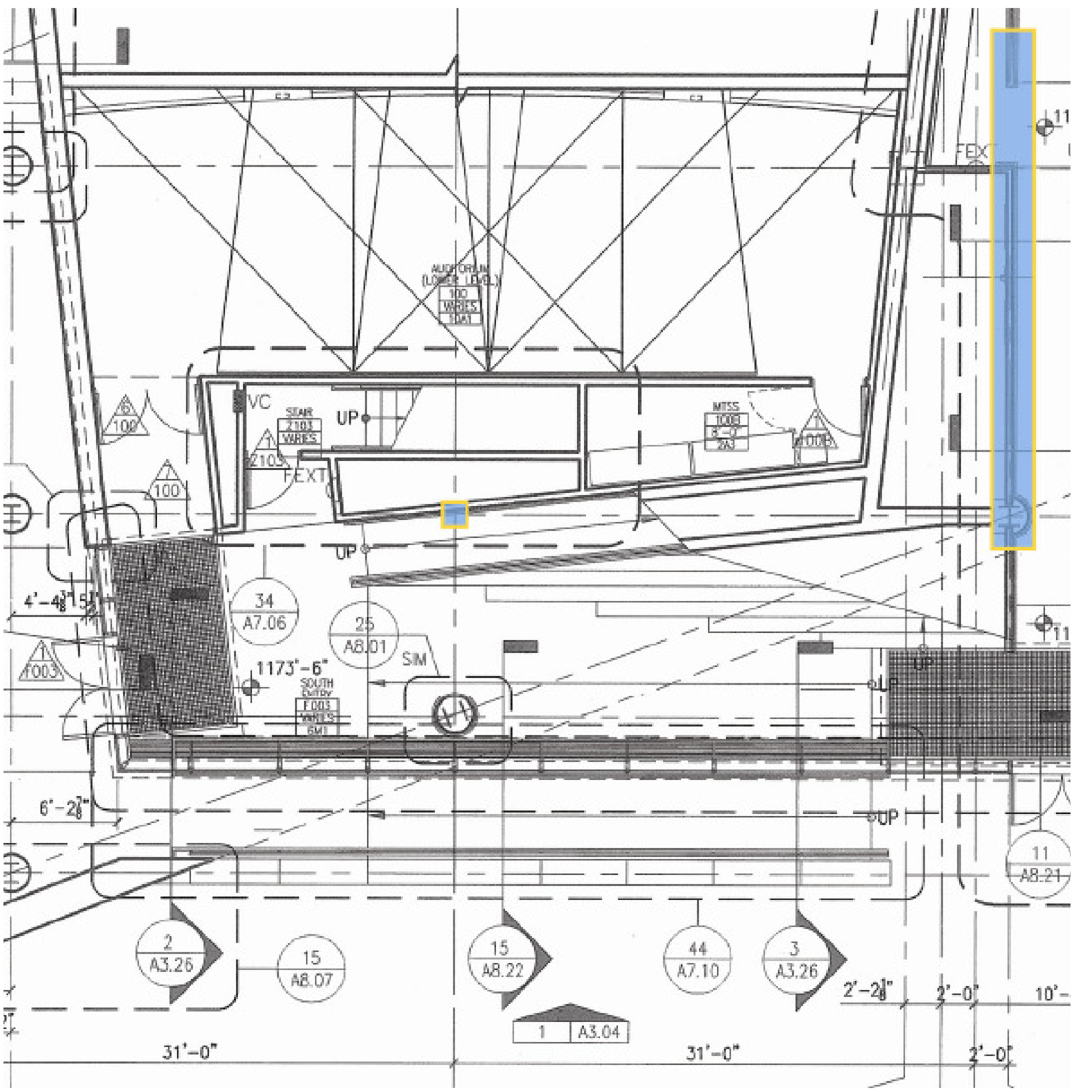
Special Concentrically Braced Frames were able to be used without major changes to the architectural design in the north – south direction. The only major change that needed to occur was the elimination of two punched windows on the façade and the shifting of a staircase by about ten feet. Minor changes included the extension and addition of some walls to hide the Special Concentrically Braced Frames. Other minor changes weren't noticeable architecturally, but included changing some light gage metal stud framed cavity walls to concrete shear walls that were used to bring the seismic base up to the base of the steel seismic force resisting systems. Special Concentrically Braced Frames were able to be used successfully because they fit in the walls that enclosed staircases and mechanical shafts. They were also able to be used in exterior walls between punched window openings.

However, in the east – west direction suitable locations for braced frames were not able to be found. Thus, the preferred alternative for the seismic force resisting system in the east – west direction became Special Moment Frames. Special Moment Frames are one of the least efficient and least economical choices to be used in a lateral force resisting system. The benefits, specifically the architectural flexibility, of Special Moment Frames far outweighed the drawbacks. Special Moment frames were able to be used with very few architectural impacts – most of the special moment frames in the structural redesign used existing column lines and beam lines. There was the addition of just one column; which occurred in a cavity wall. The most significant change architecturally was replacing an exposed and painted Round Hollow Steel Section with a W-Shape. The W-Shape was then covered with an architectural aluminum column cover similar to the column covers used on W-Shapes elsewhere in the building. Other minor changes again included changing light gage metal stud framed cavity walls into concrete shear walls to make the seismic base all at the same level.

## Architecture Breadth – Representation of Proposed Changes|

To accommodate a Special Moment Frame along column line 8 in the east – west direction a column had to be added at the intersection of column line D and column line 8. This additional column will have minimal impact on the building because it will be placed within a wall on each story. The area affected by adding this column is shown below as a blue square outlined in yellow. Existing column locations on column line 8 were able used in the Special Moment Frame without architectural changes.

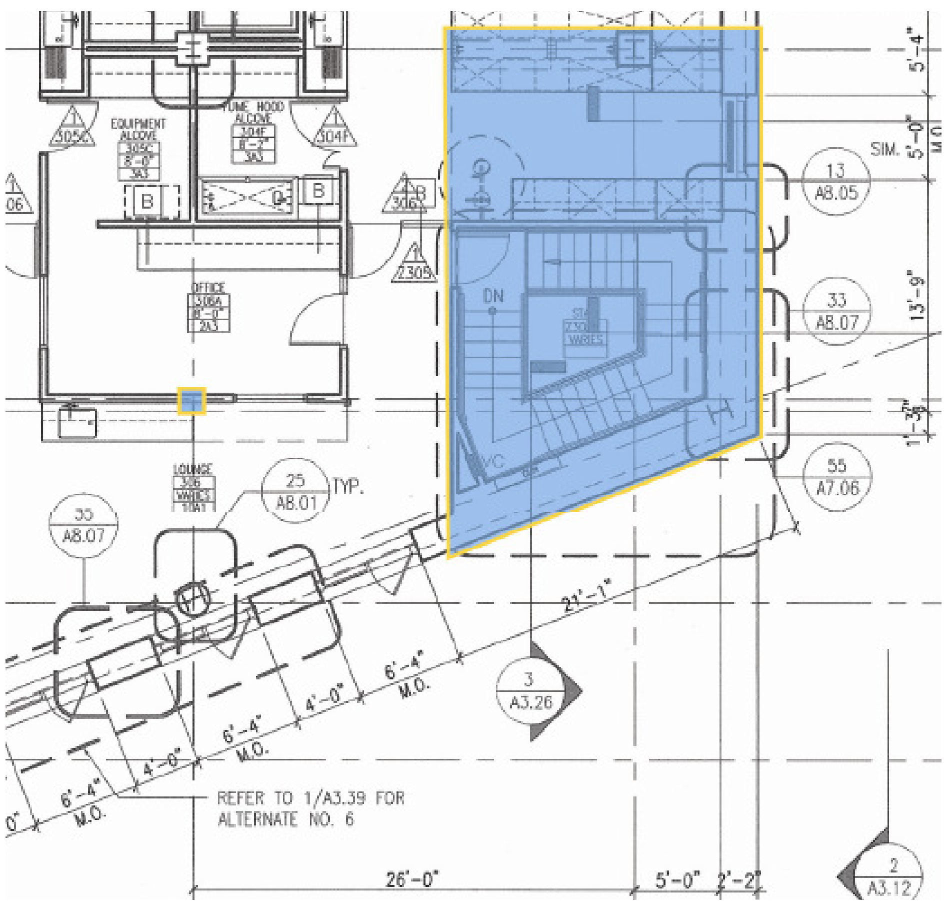
To create a uniform height for the seismic base a concrete shear wall needed to be extended from the foundation to the first floor level. This concrete shear wall is used to support a Special Concentrically Braced Frame along column line E. The impact of the concrete shear wall is shown below as a long blue rectangle outlined in yellow.



## Architecture Breadth – Representation of Proposed Changes (continued)|

The upper two floors of the short leg of the "L" have similar (typical) floor plans and are located immediately above the floor that is shown on the previous diagram. These two floors are also affected by the addition of a column for the Special Moment Frame along column line 8 and the addition of a Special Concentrically Braced frame along column line E. The effects on the plan caused by the addition of a column at the intersection of column line D and column line 8 are again shown by a blue square outlined in yellow. Again, this column occurs within walls at every floor so its impact on the building architecture is minimal.

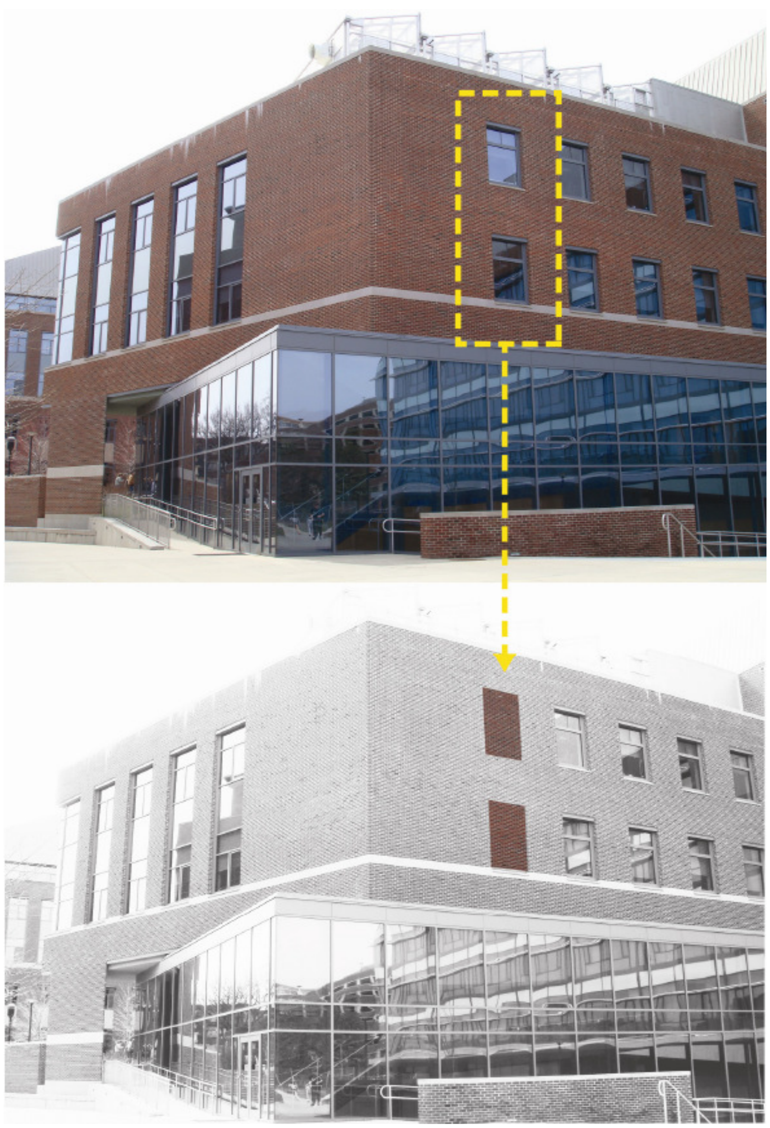
The most significant change to the building architecture anywhere in the building occurs at the location of the stairwell on the upper two floors shown below. To allow for the beams of the moment frame along column line 8 to pass through; the stairwell must be relocated completely to the north of column line 8. This still allows the stairwell from the top two levels to connect to the stairwell on the lower level (shown in the previous diagram directly north of and parallel to column line 8). However, this relocation of the stairwell results in the loss of one bay of laboratory space, shown below as the laboratory bay that is shaded blue along with the stairwell. Also, one window on each floor will need to be removed (window is also shown shaded blue) to allow for a Special Concentrically Braced Frame to be located in the exterior wall – a revised elevation will follow.



## Architecture Breadth – Representation of Proposed Changes (continued)|

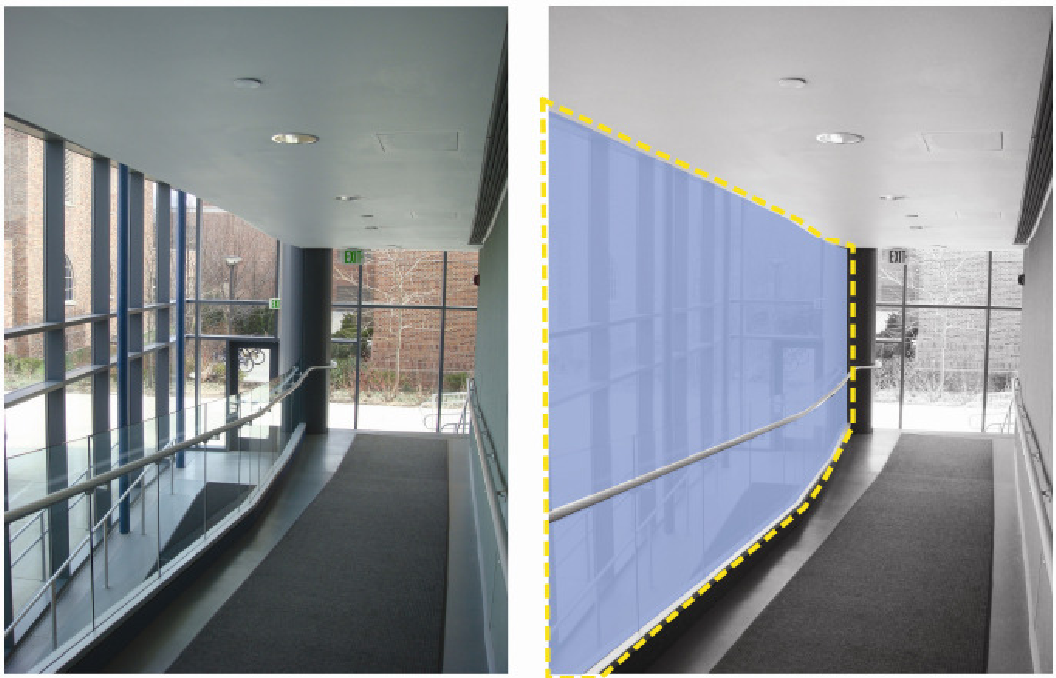
The Special Concentrically Braced Frame along column line E that has been discussed needed to be added in the exterior wall to the east of the staircase that was shown in the previous diagram. This braced frame will extend about twenty four feet (measured along the outside dimensions) from column line 8 northward. As a result of adding this concentrically braced frame the concrete shear wall at the lowest level was added to bring the seismic base up to the first floor level. The final change which needed to be made, a change which influences the appearance of the building the most, was the elimination of two windows on the east façade of the building.

The diagram below shows the before and after photographs of the east façade. The windows were placed at a rather arbitrary distance from the corner of the building. Because the distance from the corner was not related to anything else the elimination of two windows seems to only affect the architecture of the building at this specific location.



## Architecture Breadth – Representation of Proposed Changes (continued)|

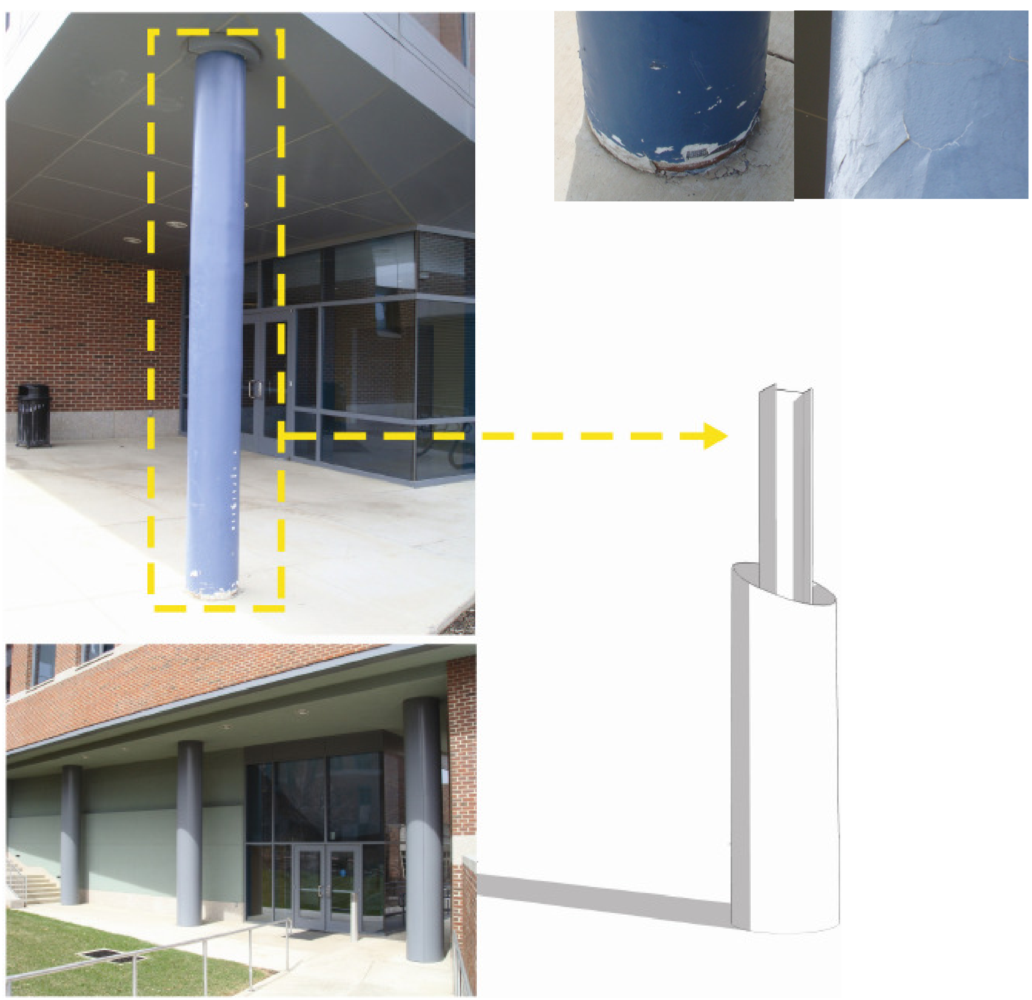
Below is a final rendering that shows the effect that the concrete shear wall supporting the Special Concentrically Braced Frame along column line E will have on the lobby space.



## Architecture Breadth – Representation of Proposed Changes (continued)

The last few architectural modifications are extremely minor when compared to the modifications that have already been discussed. The only architectural modification that hasn't been covered yet that changes the appearance of the building is the change of an exposed Round HSS to a steel W-Shape with an aluminum column cover to match other column covers used throughout the building. The description of several other changes to the building that don't influence aesthetics will follow.

In the previous design of the structure a Round HSS was left exposed at the building exterior and then covered with an intumescent coating and painted. Because Round HSS are very hard to integrate into Special Moment Resisting Frames, the decision was made to change the Round HSS to a W-Shape and cover it using the same architectural aluminum column cover that is used throughout the building to cover columns elsewhere. This change can be considered to improve the appearance of the structure because it gives all exposed columns a uniform appearance and takes care of the problem that occurs due to intumescent coatings weathering poorly. A diagram describing the change of the column is shown below, along with a close up of the existing column damage.

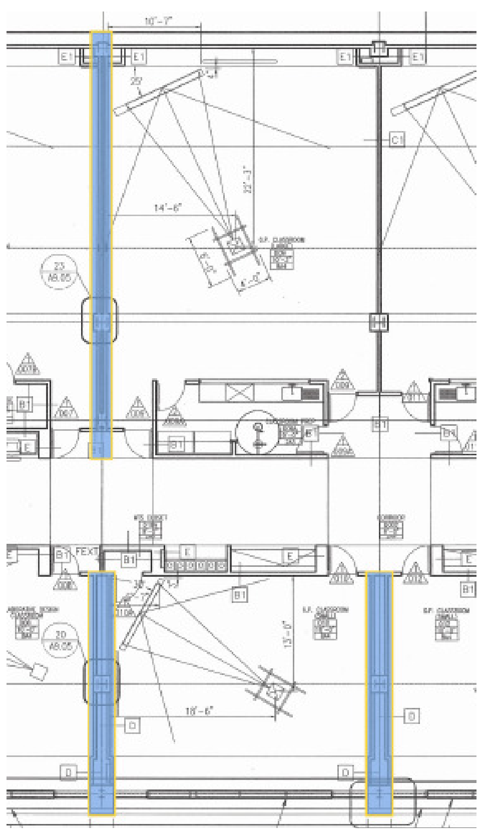


**Architecture Breadth – Representation of Proposed Changes (continued)|**

The final architectural modification is so minor that it could be considered only a structural modification, but I chose to include it because it still affects some aspects of the building architecture. The change has no affect on the overall appearance or function of the building. It only involves changing several light gage metal stud cavity walls into solid concrete shear walls that are used to bring the seismic base up to the first floor level. Accordingly, the change only affects the building on at the basement and ground floor levels.

To give the east – west Special Moment Frame along column line 4 a sufficiently rigid seismic base, the cavity walls separating classrooms on the south side of the building were changed to become concrete shear walls. The change to concrete shear walls had no affect on the building architecture, but could affect the building should a renovation attempt to reconfigure the classrooms along the south wall on the ground floor. The change is highlighted on the diagram below as a blue shaded rectangle with a yellow outline. This change is typical for all cavity walls along the south side of the building.

The final architectural modification was the change of a light gage metal stud wall to a reinforced concrete shear wall. This requires widening the light gage metal stud wall and extending it out into the classroom entrance alcove in the hallway. The change only occurs at one place, along column line G, and is done so that the Special Concentrically Braced Frame along column line G has a suitable seismic base. The change is again illustrated on the diagram below, note that the wall now interrupts the classroom entrance alcove.



## Construction Cost and Schedule Breadth – Introduction / Goals|

The goal of the construction cost and schedule breadth study was not to find a firm number for total cost difference in dollars between the existing lateral system and the redesigned lateral system. There are too many factors that change between the two lateral systems design for any side by side comparison to be valid. This would be similar to comparing *"apples to pears."* The same can be said about the schedule. To give a change in schedule by using a number of weeks, days, or man hours would be a waste of effort due to the vast differences between the two lateral force resisting systems. The number of variables that change between the two lateral force resisting systems makes an "apples to apples" comparison almost impossible for any aspect of the construction process.

The overall goal for the construction cost and schedule breadth study was to gain a better understanding of the additional costs and additional time involved with the fabrication and erection of a steel structure in a high seismic region. This was accomplished using rough estimations of the additional amounts of material needed to make the lateral force resisting system capable of resisting high – seismic loading. Rough values from reputable sources for the costs of the raw materials, fabrication, and field work were used to put a monetary value on each design. No adjustments were made for time or location so that a rough comparison could be made between the cost of each lateral force resisting system.

A brief overview of the variables influencing the cost and schedule that are different between the two lateral force resisting systems will follow. These specific variables complicate the ability to compare the two buildings side by side. The Life Sciences Building was originally designed about a decade ago as part of a multi year, two building construction project. The original design was based on BOCA 1996 and the current redesign was based on IBC 2006 and ASCE 7-05. The original building was bid in 2002, the current redesign costs are based on the most current data available. Differences in trade experience and local labor practice exist between State College, Pennsylvania and Seattle, Washington. The connections in the original design were designed and fabricated using as what must be assumed to be R<3. The connections in the redesign must be designed to meet the requirements for Special Moment Frames and Special Concentrically Braced Frames. The steel ASTM designations that were considered standard for the structural shapes of the original design have changed because of time and the compliance with the AISC Seismic Provisions. The amount of welding (especially field welding) in the structure has gone up – most of the welds are now demand critical complete joint penetration welds – due to the requirements of the AISC Seismic Provision.

There is no effective way to estimate and compensate for all of the following differences between the structures that will allow for a firm dollar value price difference to be placed on the two seismic force resisting systems. However, a rough estimate that allows for a "quick and dirty" side by side comparison of the two lateral force resisting systems follows.

## Construction Cost and Schedule Breadth – Process|

The total weight of steel (separated into HSS and W-Shapes) for the new lateral force resisting system was determined. Only the elements of the newly designed lateral force resisting system were considered because the gravity system of the building remained relatively unchanged. These weights were then multiplied by the raw material cost per pound given by Charlie Carter of AISC. The raw material cost was ($0.44 / lb) for W-Shapes and ($0.49 / lb) for HSS. To find the total cost of the structure the raw material cost was divided by its percentage (.27) of the total cost. This procedure uses information about the percentages of total cost – which includes raw materials, fabrication, erection, and overhead – that were given in an article from the March 2008 issue of Modern Steel Construction. The final cost for the steel framing of the redesigned lateral system was then determined to be:

**New Steel LFRS: $764,496**

A quick check of this value was done using the total weight of steel (not separated by shape) and current values for the cost of structural steel given by the State of Washington Department of Transportation on their website. These values of the cost of structural steel per pound were current and the base value ($1.70 / lb) for the year of 2007 was chosen. The value should also provide an accurate reflection of the cost of construction in Seattle, Washington. This value confirmed the value of the AISC method to be accurate as the WSDOT method gave a value of $790,988

To calculate the cost of adding several new concrete shear walls to bring the seismic base up to the first floor level the State of Washington Department of Transportation values were consulted again. The value for structural concrete was chosen (making sure not to choose the value for concrete pavement). The value of structural concrete was given as ($567.75 / cy) for the year of 2007. The value again provides an accurate reflection of the cost of structural concrete construction in Seattle, Washington. The total cost of the concrete shear walls added as part of the redesigned lateral force resisting system was:

**New Shear Walls: $161,241**

This value for concrete shear walls was compared using the price per cubic yard of structural concrete for the Seattle, Washington area (f'c = 9000 psi) from Paul Parfitt's thesis report from a year ago. Using a value of ($650 / cy) the cost of the shear walls was determined to be $184,600. This value seems valid because concrete with strength of 9000 psi would be considered high strength and cost more than the average value. The WSDOT value for structural concrete shear walls was used because the foundation and shear walls were unable to be designed for this report. Therefore the strength of concrete was unknown and using the overall average for the State of Washington seemed the most logical choice.

## Construction Cost and Schedule Breadth – Process (continued)|

Finally, the cost of the existing lateral force resisting system will be calculated to find a rough relative difference in the cost of the existing lateral system and the cost of the new lateral force resisting system. This will provide insight into how much more seismic force resisting systems cost than wind controlled lateral systems.

When the cost of the existing lateral force resisting system was calculated, much care was taken to be sure to get an accurate idea of the existing lateral force resisting system cost only and not count gravity members. Counting gravity members in the estimate of the lateral force resisting system would make the estimate of the cost of the existing lateral force resisting system artificially high. The cost estimate of the existing lateral force resisting system would then be compared to the cost estimate of the redesigned lateral force resisting system.

The estimate of the existing lateral force resisting system for frames that no longer resisted lateral forces in the redesigned system included finding members that were used to resist lateral and gravity loads; then replacing them with a member size that resisted only gravity loads. Many bracing members could be eliminated completely in the redesign and their cost was counted toward the existing system total. Several existing braced frames and existing moment connections were used to span the first floor auditorium in the short leg of the "L". However, these frames are necessary no matter what lateral system is chosen and their cost was not part of the existing system estimate.

The estimate for the existing lateral force resisting system was calculated using the AISC method that was used previously for the new system. This will allow for the most consistent comparison possible of values between the existing and redesigned lateral force resisting systems. The AISC method gave a value of:

**Existing Steel LFRS = $612,441**

The AISC cost was found to be consistent with the WSDOT value of $628,204.

Mention should be made that it was felt to be justified to use the same value for the percentage of raw material costs for both systems. Although it is recognized that seismic detailing, fabrication, and erection costs should and will be higher than those costs for wind controlled or low seismic applications; it was felt that the 27% raw material percentage was justified for both lateral force resisting systems. This is because the original system was composed of many unique lateral force resisting frames as compared to the fewer and repeated frames that comprise the redesigned system. Also, the new system's frames have far fewer individual members and connections (which are repeated more often) so each frame will require a similar if not lesser amount of detailing, fabrication, and erection time. These two factors are what lead to the choice of making the detailing, fabrication, and erection costs the same for both frames.

## Construction Cost and Schedule Breadth – Process (continued)

Firm numbers for the schedule couldn't be determined because it is impossible to estimate how quickly a crew can make connections and changes to connections were the major change to the building design. Also factors such as the weather – it's always snowy in State College and always rainy in Seattle – may limit the number of days work can be done. This is especially true due to the redesigned frames large number of demand critical welds that are made in the field and subjected to a rigorous quality control program.

In addition, schedule wasn't such a critical aspect for this project as compared to others because it is a university building and not something that needs to open and generate revenue. So there were no incentives in reducing the schedule to save time, the contractor would set their own schedule to maximize their profit.

The change to a high seismic lateral force resisting system would without a doubt extend the schedule. All of the welds in the lateral system have now become demand critical complete joint penetration groove welds, most of which need to be performed in the field. As stated earlier the weather would influence the schedule, as well as the availability of qualified welders. Weld inspections in the field would also add time to the schedule. The fabrication time would surely increase as larger, thicker, and heavier gusset plates would become a part of the Special Concentrically Braced Frames and reduced beam sections, doubler plates, and continuity plates would become a part of the Special Moment Frames.

The only ways to reduce the schedule increase as a result of the change to a high seismic lateral force resisting system would involve fabrication. It would be important to bring the structural steel fabricator into the project as early as possible so that they could provide input for seismic detailing. By bringing the fabricator on board early it would also allow them to begin procurement of the raw materials and then start fabrication of the steel members as soon as possible. Finally having the fabricator provide input at an early stage of design may help find ways to cut back on the number of field welds which would reduce the erection time and cost.

It is estimated that the change of the lateral system from a wind controlled lateral force resisting system to a high seismic lateral force resisting system would increase the fabrication time anywhere from a month to two weeks. However this could be offset by the reduction in the number of members that was achieved by redesigning the lateral force resisting system. It could also be offset by bringing in the steel fabricator earlier (per prior discussion). It is also estimated that the erection time would be increased by 4 days using R.S. Means (E-6 + E-9 crew working together). This also could be offset potentially by the reduction in the number of lateral frames and the reduction in the number of members composing those frames as well as by the benefits of bringing the fabricator on earlier.

## Construction Cost and Schedule Breadth – Final Numbers|

It was found that The Life Sciences Building could economically be relocated from State College, Pennsylvania to Seattle, Washington. The real challenge in the change between wind controlled lateral systems and seismic controlled lateral systems is not feasibility or cost but the myth that seismic force resisting systems are far more expensive. However, the results of this study could be misguided due to the low height of the Life Sciences Building. The results also could be affected because the existing lateral system was very inefficient – reducing the strength and stiffness of the current redesigned system to resist the controlling wind loads of State College, Pennsylvania may yield a substantial savings over the original existing system. The final comparison between the costs of the lateral force resisting systems is:

**Existing (Wind) LFRS = $612,441**
**Redesign (Seismic) LFRS = $925,737**

**Redesign LFRS = + $313,296**

## Thesis Conclusion

The primary purpose of this thesis exercise was to gain a better understanding of lateral force resisting systems and to also become familiar with and gain experience using the seismic design provisions of ASCE 7-05. The secondary purposes (what determines if the thesis was a success or not) were to confirm that the existing lateral force resisting system of the Life Sciences Building was inefficient and also to get an idea of the relative cost differences between building lateral force resisting systems that are controlled by wind and seismic loading. Additionally this thesis exercise provided experience using the AISC Seismic Design Provisions, AISC 358 – Prequalified Seismic Moment Connections, and the SDI – Diaphragm Design Manual.

The secondary purposes of confirming that the existing lateral force resisting system was inefficient and getting an idea of the relative cost differences between wind controlled and high seismic lateral systems are objectives which can either be achieved or not. Both of these objectives have been achieved through this thesis exercise.

This thesis exercise yielded the results that were expected when it was first proposed. The existing lateral force system of the Life Sciences Building was determined and confirmed to be inefficient. This can be seen through the relatively large overall increase in lateral forces as a result of the change from Seismic Design Category “A” to Seismic Design Category “D” which lead to a relatively small increase of project costs due to the redesign for these higher forces. The relatively small difference in cost between the inefficiently designed wind controlled lateral system and the redesigned seismically controlled lateral system is a sign that the existing lateral force system could be designed more economically. It is also a good indicator that the cost of a lateral system in a high seismic region is indeed more than a wind controlled lateral system, but not by as much as most people would think.

The structural redesign of the building was a success. The redesigned lateral force system overcame many obstacles that would not have existed had the building been designed with seismic considerations in mind from the beginning. What makes it more impressive is that the redesigned lateral force resisting system was able to overcome those obstacles without any significant changes to the building function or aesthetics. The diaphragms were checked and found to be acceptable for the new high seismic loads placed upon them. Also, the number of lateral frames was reduced from thirteen to seven – a reduction of almost fifty percent. This reduction in the number of frames corresponded to a reduction in the number of individual steel members and connections. Finally, the final cost change from a wind controlled lateral force resisting system to a (high) seismic controlled lateral force resisting system was less than one percent of the total building cost.

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