Koshland Integrated Natural Science Center

Haverford College
Haverford, PA

The Pennsylvania State University
Department of Architectural Engineering

PENNSTATE

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Structural

Senior Thesis Study
Spring '06

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Koshland Integrated Natural Science Center

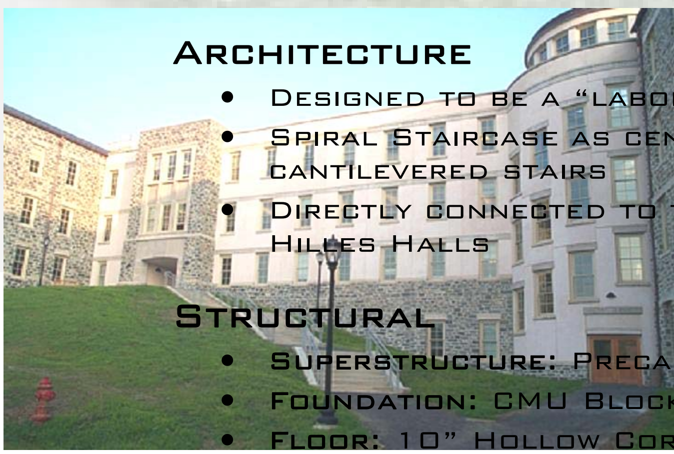
Haverford College, Haverford, PA

Project Overview

- Total Square Footage : 185,423
- 4-story Educational Building
- Spaces include Laboratories, Classrooms, & Offices
- Total Project Cost: $42.6 Million

Project Team

- **Owner:** Haverford College (Physical Plant)
- **Architect:** Ayers/Saint/Gross (ASG) Architects & Planners
- **Engineer:** CUH2A
- **General Contractor/CM:** Skanska USA Building, Inc.
- **Laboratory Planner:** Earl Walls Associates



Architecture

- Designed to be a "laboratory of the 21st century"
- Spiral Staircase as central core of the building with cantilevered stairs
- Directly connected to the existing Sharpless and Hilles Halls

Structural

- **Superstructure:** Precast Concrete Framing
- **Foundation:** CMU Block Walls/Retaining Walls
- **Floor:** 10" Hollow Core Precast Planks w/2" topping
- **Envelope:** Stone & Brick Façade/White Precast Concrete Panels
- **Roof:** Steel Framing w/Metal Deck

Mechanical

- Energy wheels create "space-neutral" air
- Fan-Coil Units maintain temperature controlled rooms
- 110 fume hoods operate at 900 cfm exhaust in labs
- Water stored in two 240 ton chillers

Lighting

- Variations of direct and indirect lighting are used in all communal spaces
- Photocells and Relay-based lighting In lobby

HAVERFORD

Executive Summary

Executive Summary

The Koshland Integrated Natural Science Center, located in Haverford, Pennsylvania, is four story laboratory building and is a new edition to the Haverford College campus. The building is comprised of laboratory, classroom, and office spaces as well as numerous communal areas. The KINSC is directly connected to the two existing structures, Sharpless and Hilles Halls, but is very distinctive in its architecture and engineering. The existing structural system is primarily precast concrete, including the floor system, the framing, and the lateral system.

This report provides an in-depth study on the comparison between the existing precast concrete system and a proposed structural system of steel framing with composite concrete slab on metal deck as the proposed floor system. The proposed lateral system consists of steel braced frames. The design of the structural system was performed with the use of the RAM Structural System program. The lateral system was designed with the aid of STAAD Pro. The purpose of this study is to examine any possible benefits that could come from using the proposed system over the existing system.

Also included in this report are two breadth studies, in the areas of Construction Management and Mechanical emphasis. The C.M. breadth directly correlates to the structural depth study in that it compares the existing and proposed systems in a cost analysis and project schedule. The Mechanical breadth study involves the investigation of the thermal resistance between floors of both the existing and proposed systems due to the strict temperature controls necessary for the laboratory areas in the building.

Ultimately, the purpose of this report is to decisively indicate which structural system proves to be more efficient. The results are based solely on the investigations performed within this thesis study. The conclusions of this thesis study project that the proposed steel system is more efficient than the existing system in terms of cost and schedule. In no way was the purpose of this report to undermine the decisions made by the engineers of the existing design.

Design Professionals

Owner:

Haverford College (Physical Plant)

Architect:

Ayers/Saint/Gross (ASG) Architects & Planners

Engineer:

CUH2A

General Contractor/CM:

Skanska USA Building, Inc.

Laboratory Planner:

Earl Walls Associates

Acknowledgements

Acknowledgements

Throughout this past year, there were many groups and individuals who played important roles as far as the completion of this thesis study. I would like to take this opportunity to express my gratitude to everyone who helped to make my thesis study a success. First and foremost, I would like to especially thank John Diaz and Ron Tola at Haverford College for allowing me to use the Koshland Integrated Natural Science Center as the subject for my thesis. It proved to be a very interesting and educational project to study. Also, I would like to thank Sam Rozycki and CUH2A for donating the drawing sets for the KINSC, offering any relevant information pertaining to the project, and also for answering the numerous questions I had asked throughout the entire study. Furthermore, I would like to extend a tremendous thank you Valerie Gillespie for helping me to get started altogether. She definitely helped get the ball rolling. I would also like to thank Dr. Parfitt and Dr. Hanagan for directly answering an onslaught of questions over the past two semesters, and the entire Penn State AE department faculty who offered suggestions, patience, and understanding throughout the duration of my thesis study.

On a more informal note, I would like to also thank a number of my friends and family members who helped make this possible. Sincere thank you to all of my AE friends who offered help throughout this study and helped maintain my sanity this year. I would like to also thank my mom, dad, and brother for their patience, understanding, and support throughout this past year. And last but not least, I would like to thank Abby Roth for her relentless support and encouragement that lasted the duration of my thesis study.

Again, thank you all…

Introduction & Building Background

Intro & Building Background

The Koshland Integrated Natural Science Center, designed by Ayers/Saint/Gross out of Maryland, is a recent addition to the Haverford College Campus. The four story laboratory offers a home to a number of the science and math departments at Haverford. The KINSC was designed with the intent to be considered a "state-of-the-art" laboratory for the 21st century. The building definitely deserves this title with its cleverly innovative design.

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Structurally, the science center was designed as a precast concrete building primarily. The floor system is a 10" hollow core plank system that spans to supports of precast concrete beams. The beams are then supported by precast concrete columns. Similarly, the lateral system of the KINSC is entirely precast shear walls. The entire building acts as three separate sub-structures by wing, as each wing is separated by 2" expansion joints. Therefore the East Wing, West Wing, and the Link all act independently in terms of loading. The East and West Wings are quite similar in their build up. However, the Link is primarily built of a CMU bearing wall system. Hollow core plank is still used as the floor system, as well as shear walls acting as the lateral system. In addition, the KINSC was designed and constructed so that it is directly connected to two previously existing buildings, the Sharpless and Hilles Halls. All three of these buildings are similar in exterior architecture. Figure 1 below illustrates a simple layout of the KINSC (the shaded building).

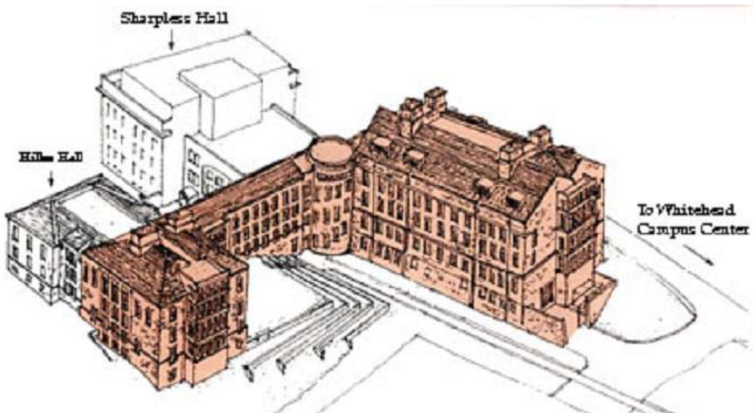


Figure 1: KINSC & attached Sharpless and Hilles Halls

With indifference to the majority of the KINSC, the roofs of the East and West Wings were designed as steel roofs. They were designed and constructed as steel bent frames with their own braced frame systems. The bent frames allow the fourth floor to be utilized as a mechanical space in the East Wing and a Library mezzanine in the West Wing. In addition, the structure below grade, including the foundation, the ground floor, and the first floor framing, is strictly precast concrete. This includes footings, retaining walls, and precast piers. The ground floor is slab on grade.

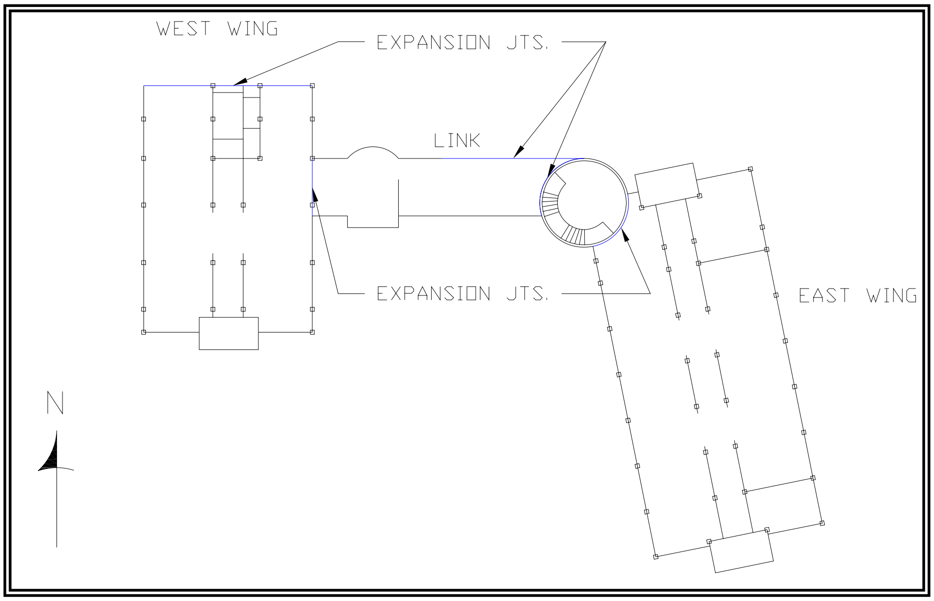
As stated previously, the East and West Wings are similar in their layout. A typical exterior bay in the East and West Wings is 31'-6" X 21'-0" and a typical interior bay of 13'-8" X 21'-0". As for the Link, essentially there are no interior columns or beams. The precast hollow core planks span from one exterior CMU bearing wall to the other in the N-S direction. There are typically three different sizes of columns used for structural framing throughout the building. Columns sized as 16"x16" and 20"x20" were used fluently throughout the typical floor. There are columns sized at 18"x36" used in certain specified areas such as locations where the loading is increased significantly. Precast concrete beams span between the exterior precast columns creating the perimeter of the East and West Wings. These beams are typically sized as 24"x12". Then there are precast beams that span between the interior precast columns generally in the N-S direction for the East and West Wings. The typical sizes of the interior beams are 24"x12" or 20"x16" depending on location. As for the flooring, 10" hollow core planking with a 2" topping slab generally spans in the E-W direction for the East and West Wings. In the Link, the same hollow core plank with topping spans in the N-S direction from exterior bearing wall to exterior bearing wall. The exterior bearing walls ranged in thickness from 8" to 14" depending on the story level. The lateral system of the KINSC is strictly a shear wall system. The East and West Wings each account for two 8" precast shear walls spanning

Typical elevation during the construction of the KINSC

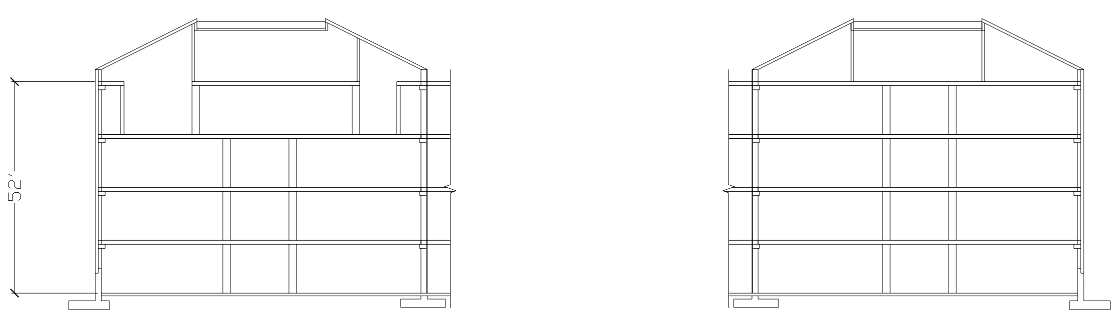
31'-6" in the E-W direction. There is also an 8" CMU block shear wall spanning 16'-6" in the N-S direction located in the Link. For a better understanding of a typical floor layout of the KINSC, refer to Figure 2 below.

Figure 2: Typical Floor Plans of the East Wing, West Wing, and Link of the KINSC

A: Typical floor layout showing expansion joints between the building wings

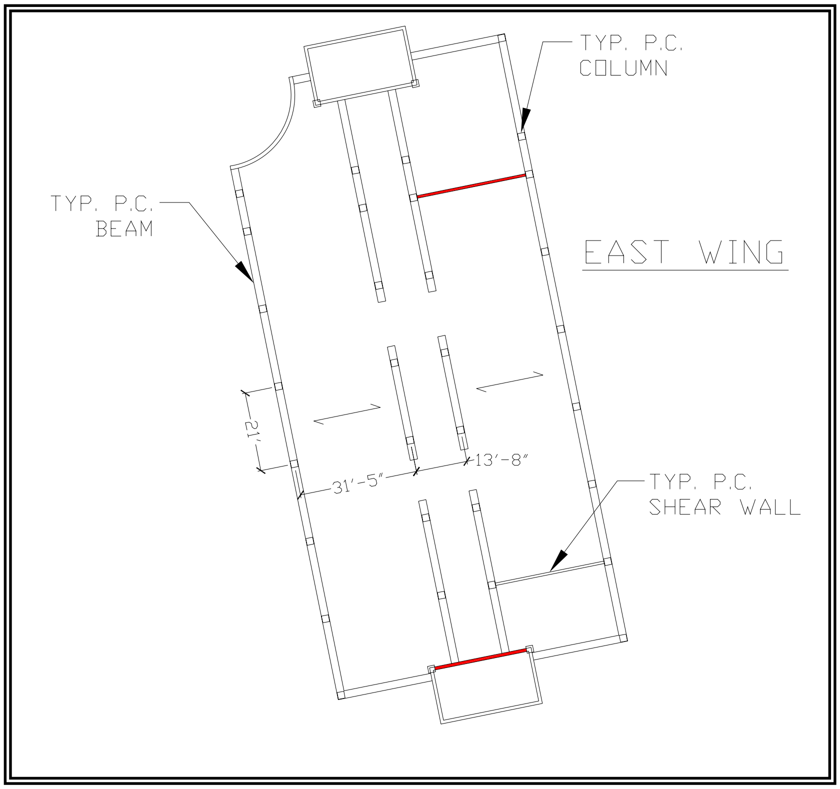


B: Typical Section through the East & West Wings showing building height and bent roof frames

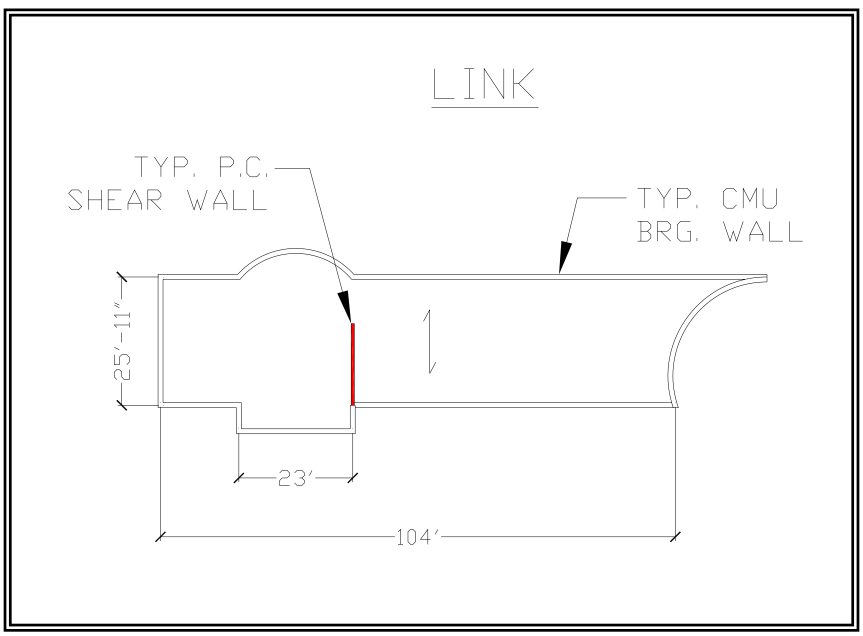


West Wing **East Wing**

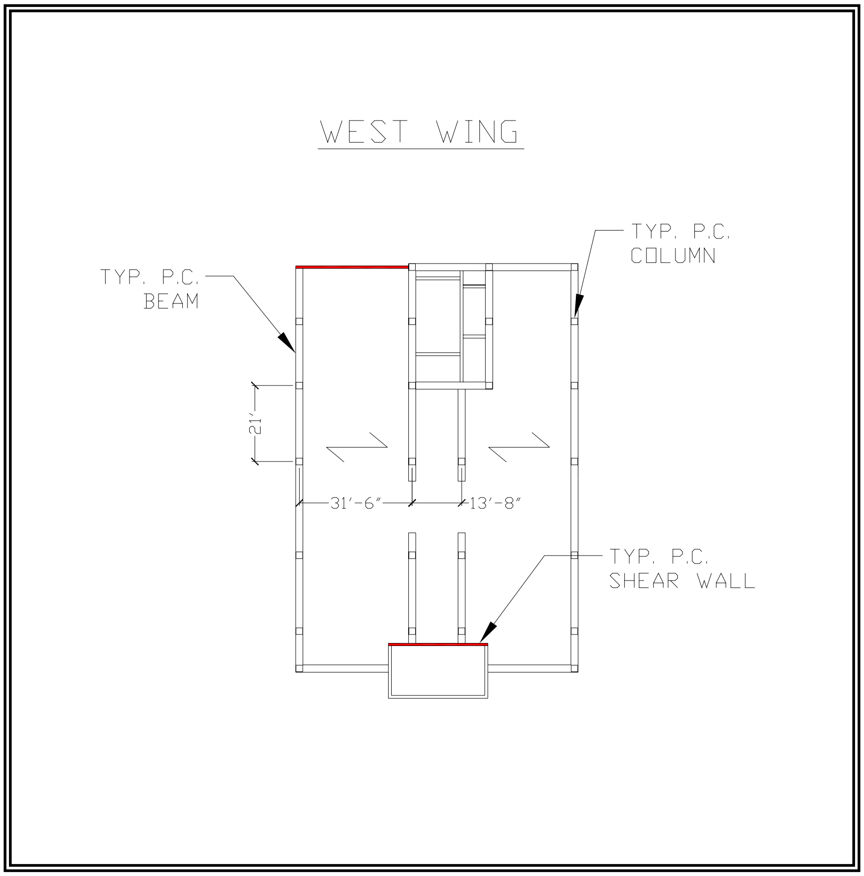
C: Typical Floor Plan – East Wing



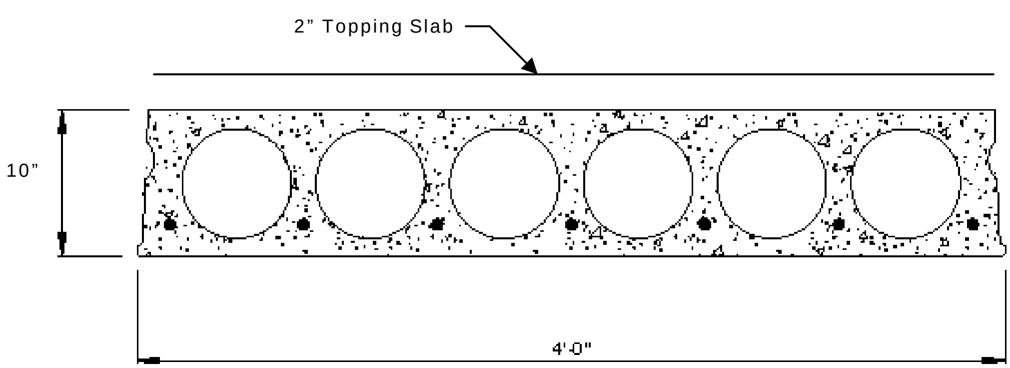
D: Typical Floor Plan - Link



E: Typical Floor Plan – West Wing



F: Typical 10" Hollow Core Plank with 2" Topping Slab



Thesis Proposal

Proposal

Problem Statement:

From previous investigation, it became apparent that the existing design of the Koshland Integrated Natural Science Center is an incredibly efficient design on several levels. The design expertise of the professionals who were involved in the KINSC is quite prevalent and was expected to be so. As results of earlier research, the design of the framing members, floor system, and lateral system all exceeded design requirements as per BOCA 93 and ASCE 07. Also, the design of the KINSC meets all code requirements concerning physical restrictions on the building by a considerable amount. Therefore, when considering an alternative design to this building, the final decision did not easily come about. However, I would like to further investigate some other options. I would like to consider redesigning the structural system of the existing KINSC at an attempt to find an equally effective or more efficient system.

To determine whether a different system is more efficient, it will be compared to the existing system in a number of categories. These categories will include, but are not limited to being the most cost effective, ease of constructability, most efficient construction schedule, and material availability. This proposal will research an alternative system that could possibly prove to be a more viable solution than the existing system in any of these categories.

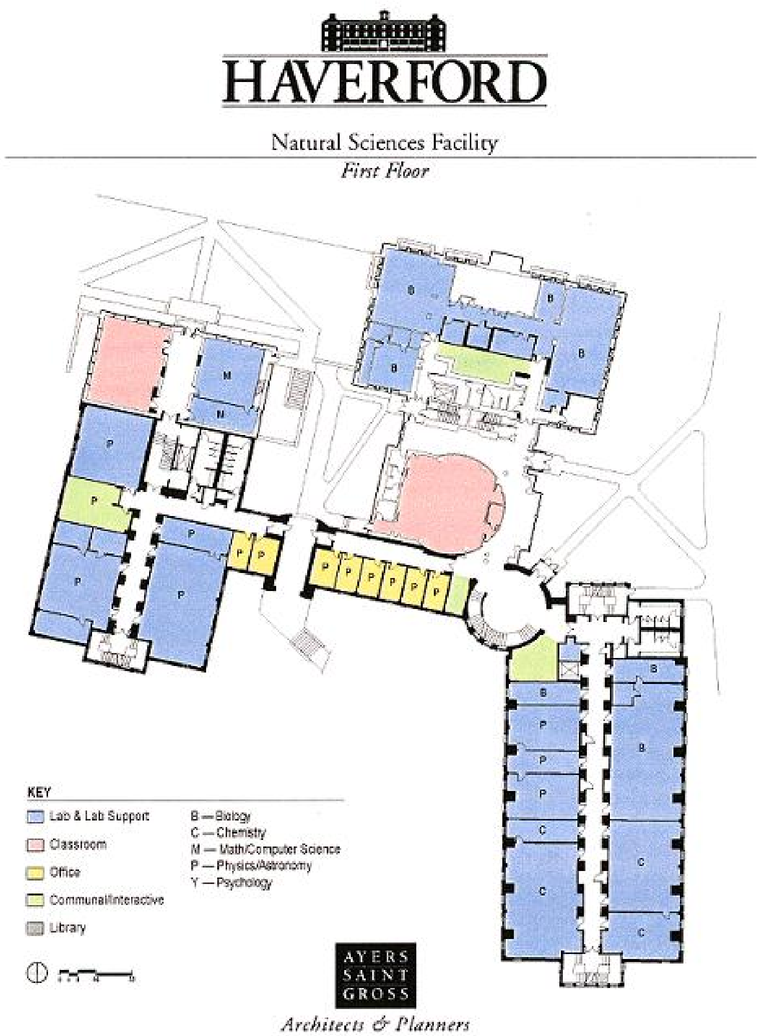
Solution Process:

As a viable solution to an alternate structural system for the KINSC, the first modification to be considered is altering the framing of the typical floors to an entire steel frame. A transition from precast concrete to steel framing seems to be a logical comparison for this structure. This will consequently affect aspects such as CM issues like schedule and cost, perhaps the mechanical system, and possibly architecture. In addition, since the controlling lateral load case is seismic, changing the building framing to steel may reduce those loads due to a lighter overall building weight. A second adjustment will be to change the floor system from hollow core planks to a composite floor system while maintaining a similar floor depth and similar spans. As confirmed by an earlier study, a full composite floor system is a reasonable option for this building.

Furthermore, I would like to change the lateral force resisting system to a braced frame system. The purpose of making these alterations to the structure is simply to investigate the overall affects they have could have on the project, in anticipation of positive results.

All relative structural elements of the building will have to be considered throughout this alternate design. This includes the foundation, floor systems, beams, columns, fire protection, interior and exterior walls, and the roof. Obviously, since the redesign incorporates a different primary material for the building, in this case, steel, the sizes of basically every member will be altered. This, in return could give cause for the layout of the structural members to change as well. The floor spans and location of the floor framing members will remain unchanged where it is possible. When dealing with the lateral system, braced frames will take the place of shear walls to distribute the lateral loads that act on the structure. Location of these steel braced frames will be carefully decided to possibly offer a more efficient lateral load resisting system. Some structural aspects will remain the same. For instance, the expansion joints found throughout the building will remain a part of the redesign to maintain the independent behavior of the three sections of the building. The precast piers in the basement, retaining walls, and footings will all remain the same. Since the proposed structure will be composed of steel, the overall building weight will decrease. Therefore the existing size of footings, piers, and walls below grade will be sufficient to support the new loads. Also, the roof of the East and West Wings will remain the same as the existing system. This is the case because the existing roof systems are currently steel bent frames which will coincide nicely with the proposed steel framing system of the remainder of the building. Lastly, the central stairwell will not be altered in design. Due to an innovative structural design, the stairs cantilever out of the exterior wall of the stair case as they spiral upward. For the purpose of maintaining the cantilevered stairs, no change in structural design will be implemented. The design results of this alternative system will be thoroughly compared to the design of the existing system with hopes of proving to be a more viable solution.

The use of RAM Structural System will most likely be the primary means of computer design for this study of an alternative system. A 3D model of the KINSC will be created in RAM and all steel structural members will be sized according to the calculated gravity loads that will be applied to the structure. As for the design of the lateral system, a different program besides RAM may be utilized. A reliable option for the design of the lateral system is STAAD Pro. The use of STAAD would allow for the braced frames to be designed and analyzed individually. All aspects of the steel design will be based on the Manual of Steel Construction, Load and Resistance Factor Design, 3rd Edition. Also, all concrete design will be in accordance with the ACI-02 code, and all lateral loads will be based on the ASCE7-02 regulations. The IBC 2003 will be followed strictly throughout the design. Based on the results from the RAM and STAAD designs, the most efficient structural system can be determined.



Layout of entire complex – KINSC, Sharpless, and Hilles Halls

Structural Depth Study

Structural Depth Study

Design Criteria:

The following information pertains to building codes used throughout this depth study and the overall design considerations for the proposed structural system:

Code Basis:	2003 International Building Code (IBC) ASCE 7 – 02 AISC LRFD, 3rd Edition
Design Considerations:	Structural System Cost Construction Schedule

Gravity Loads:

The following gravity loads were used consistently throughout the design of the proposed steel structural system. Many of the loads were maintained from the design of the existing precast concrete system.

Table 1: Gravity Loads for Proposed Design

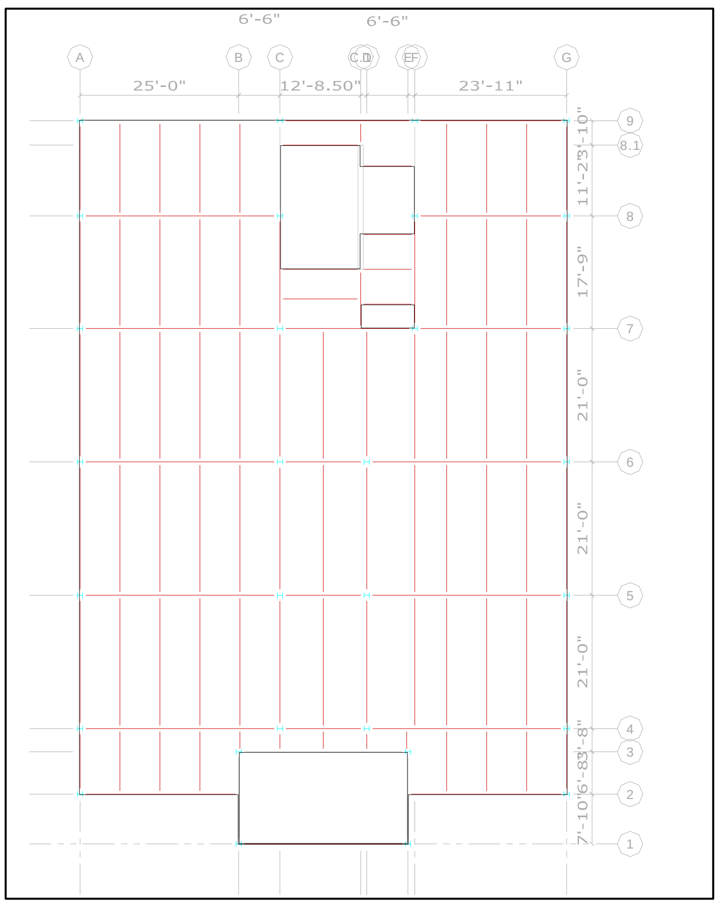
Live Loads		
Location	**Load Description**	**Load (psf)**
Roof	Ground Snow Load	30
Floor	Typical	100
	Libraries	300
	Lobbies/Corridors/Entrances	100
	Stairs	100
	Mechanical Room	125
	Storage	125

Dead Loads		
Location	**Load Description**	**Load (psf)**
Roof	Ceiling	5
	MEP	10
	Roofing & Insulation	8
	Deck & Sheathing	5
	Slate Roofing	10
	Total	**38**
Floor	Ceiling	5
	MEP	10
	4" Composite Slab on Deck	48
	Partitions - 6" lite wt. CMU	30
	Total	**93**

Typical Floor Layout:

One main objective kept in sight throughout this structural study, was the intent to have the layout of the floor plan remain unchanged. This was viewed as a somewhat crucial objective due to the fact that the laboratories required large spaces with relatively open floor plans. Ergo, the typical bay size of 31'-6" by 21'-0" as used in the existing system, was utilized for the proposed steel system as well. The redesign of the KINSC also maintained the floor-to-floor height of 13'. In the East and West Wings I chose to have the girders spanning in the E-W direction as opposed to the N-S direction, in which the existing design entails. This decision was made to allow for a more typical spacing of the beams across the entire floor. The dimensions of the bays in the N-S direction vary more often than the dimensions in the E-W direction. Therefore, from a constructability standpoint, in terms of cost and schedule, the repetition in beam sizes and spacing can be viewed as beneficial. A typical floor layout of the redesigned KINSC can be viewed below in Figure 3.

<u>Figure 3A</u>: Typical layout of steel framing – West Wing



<u>Figure 3B</u>: Typical layout of steel framing – East Wing

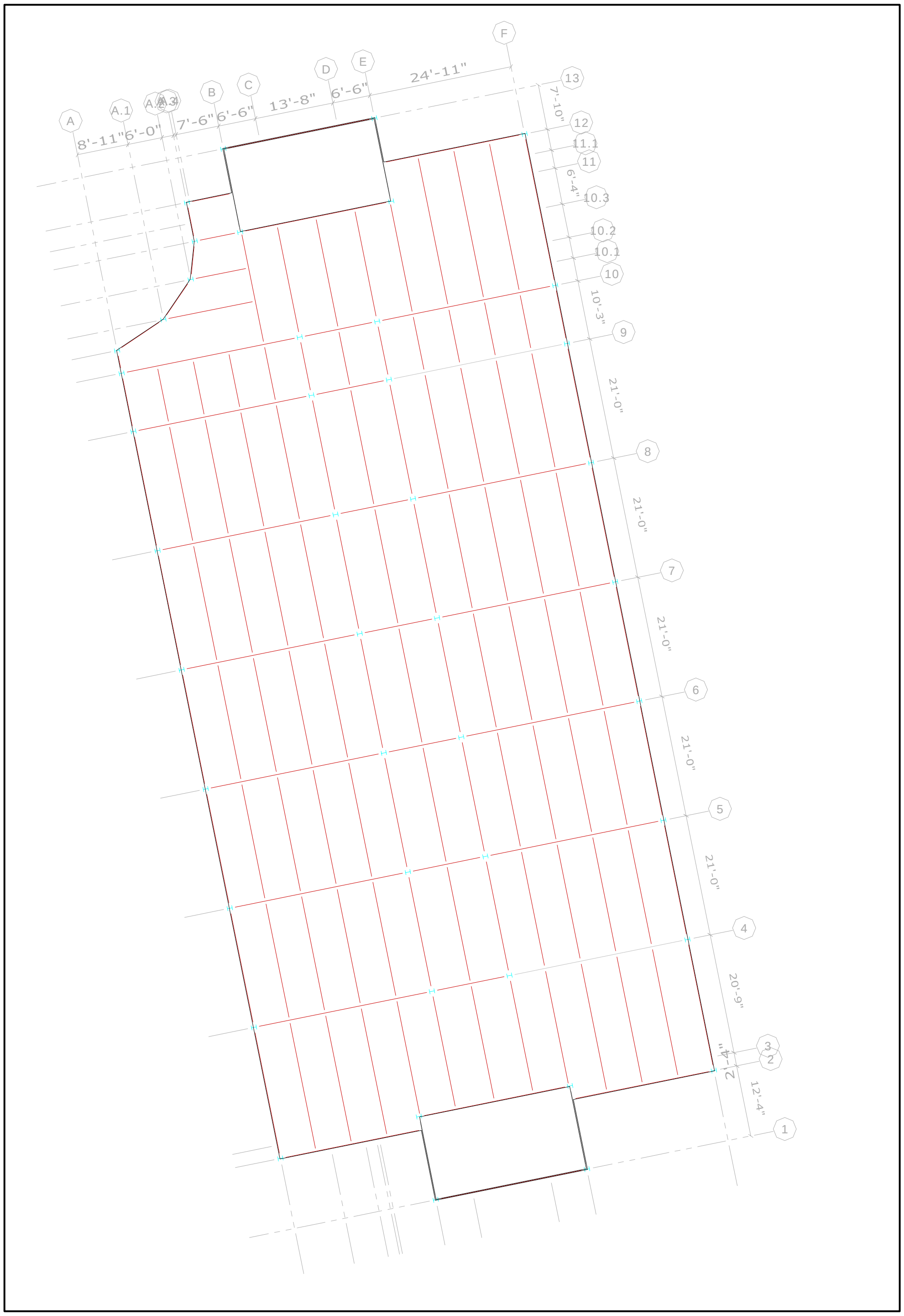
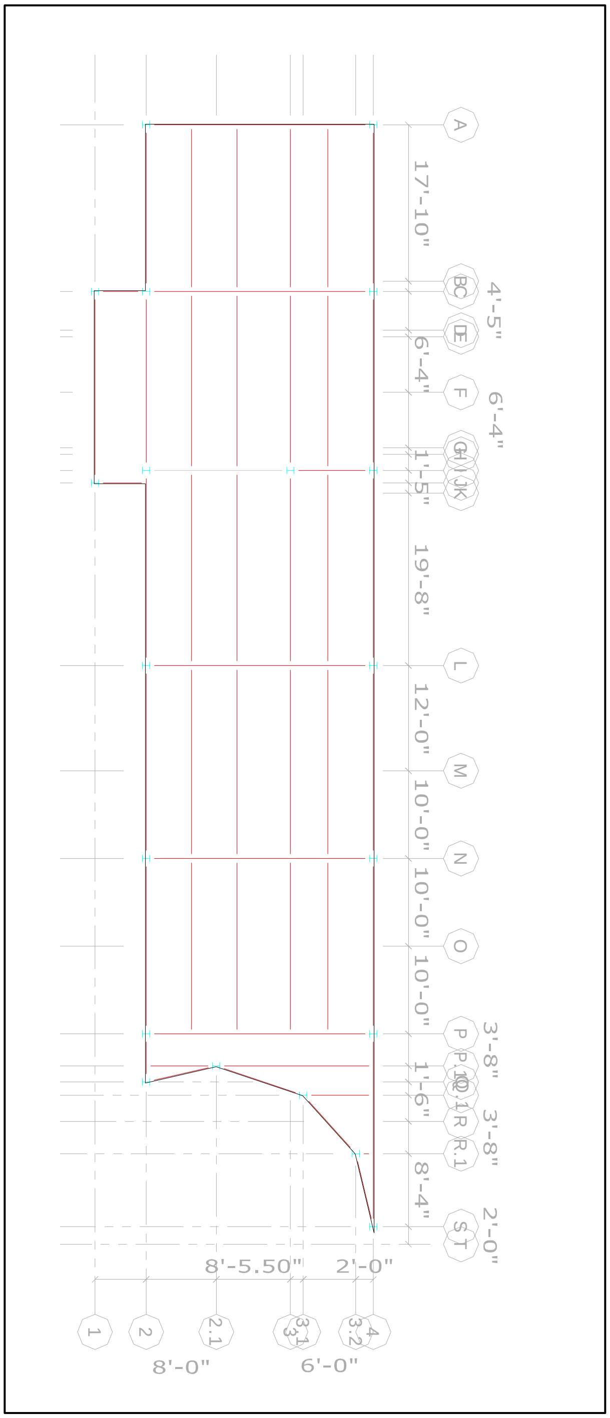


Figure 3C: Typical layout of steel framing – Link



Composite Slab Design:

For the design of the composite concrete slab, the use of the Wheeling deck catalog was implemented. Using the calculated loads for the structure, and the typical span between beams of approximately 6.5’, a 1.5 SB normal weight composite deck was selected. Furthermore, a 4” composite concrete slab was assumed for the design of the floor system. The reinforcing for the slab on deck was chosen to be 6X6 – W1.4XW1.4 welded wire fabric. Shear studs of ¾” diameter with a length of 4” were selected to ensure the composite action. The selected deck, slab, and stud sizes were inputted into the RAM structural program for the design of the steel structure. Refer to Appendix A for deck information and load calculations.

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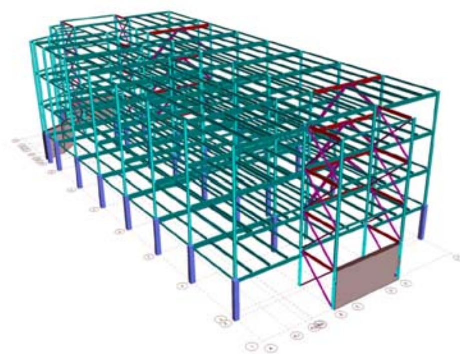
Structural Steel System – Gravity Loading:



To begin the design of the steel framing due to gravity loads, a model of the KINSC was created in RAM. Due to the multiple expansion joints found throughout the building, separating the KINSC by wing, three individual models were made in RAM. Each one of these models represents a wing. The following steps were repeated for each of the three models created, using the loads and building information corresponding to the proper wing. The first step in creating the model was setting up the building grid. Once the grid was laid out, the columns were placed. It was important to ensure the columns were arranged with the correct orientation to allow for weak or strong axis bending. Following the layout of the columns, the steel beams and girders were placed. All beams, girders, and columns were W-shapes. Once the framing members were all laid out, the deck, slab, and shear studs were selected and applied over the floor framing. The next step of the modeling process was the defining and placing of the corresponding floor loads. After the application of the loads was completed, this process was repeated for each of the four stories.

The next step in the design process, following the modeling of the structure, was the steel beam design. Initially, a design code for the steel design was selected. For this depth study, the LRFD 3rd Edition was chosen. Next, RAM performed the design of all steel beams based on the design code specified and the information from the model. The designs of all beams were then obtained and recorded. The RAM output design values for the beams were very reasonable.

The final step in the gravity design process was the column design. RAM performed a gravity design of the steel columns based on the axial loads acting on the columns. The columns were designed and sized based on the limitations found within the steel design code specified. A number of the columns sizes were slightly increased to account for uniformity or the possibility of increased loading. Several RAM design outputs, including plans and designs, can be reviewed in Appendix B.

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Lateral Loads:

For the design of the lateral force resisting system in the KINSC, forces due to wind and seismic activity were calculated using the standards and methods in accordance with ASCE 7-02. When analyzing the existing precast concrete system in an earlier study, it was found that seismic forces acting on the building controlled over wind loads. With the transition from the existing precast to the proposed steel system, the overall weight of the structure would be decreasing. Therefore, it was not guaranteed that the proposed steel design of the KINSC would be controlled by seismic forces. This being the case, the lateral loads acting on the building, both wind and seismic, were recalculated, again based on ASCE 7-02. The results concluded that seismic forces would again control of wind loads acting on the building. The complete calculations for both lateral load cases can be reviewed in Appendix C. For a summary of the seismic information pertaining to the proposed design of the KINSC and the controlling seismic forces, see Tables 2 and 3 below.

Table 2: Seismic Information for redesign of KINSC

Seismic Information	East & West Wings	Link
Building Location	Haverford, PA	Haverford, PA
# of stories	4	3
inner story ht.	13	13
Bldg. height	53	39
Seismic Use Group	II	II
Importance Factor	1.25	1.25
Site Classification	B	B
0.2s Acceleration	0.35	0.35
1.0s Acceleration	0.08	0.08
Site Class Factor:		
Fa	1.00	1.00
Fv	1.00	1.00
Adjusted Accelerations		
S_{ms}	0.35	0.35
S_{m1}	0.077	0.077
Spectral Response Accelerations		
S_{DS}	0.233	0.233
S_{D1}	0.051	0.051
Seismic Design Category	**B**	**B**

Table 3: Calculated seismic forces acting on the KINSC

Seismic Analysis

East Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	1297	53	55,579	0.263	73	-	3,860
4	2389	39	76,562	0.362	100	73	3,913
3	2389	26	52,162	0.247	68	173	1,777
2	2389	13	27,068	0.128	35	242	461
1						277	
	Σ = 8464		Σ = 211370	Σ = 1.000	Σ = 277		Σ = 10011

Link

Vertical Distribution of Seismic Forces

$k_{E\text{-}W} = 1 + (T_{E\text{-}W}\ 0.5)/(2.5 - 0.5) =$ 0.906

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	561	39	15,516	0.333	37	-	1,432
3	1057	26	20,227	0.435	48	37	1,245
2	1057	13	10,794	0.232	26	85	332
1						110	
	Σ = 2674		Σ = 46537	Σ = 1.000	Σ = 110		Σ = 3009

West Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	892	53	38,203	0.262	50	-	2,655
4	1650	39	52,873	0.363	69	50	2,703
3	1650	26	36,023	0.247	47	119	1,228
2	1650	13	18,693	0.128	25	167	319
1						191	
	Σ = 5841		Σ = 145792	Σ = 1.000	Σ = 191		Σ = 6904

Structural Steel System – Lateral Loading:

For this structural depth study, the existing lateral system of precast shear walls was altered to a system of steel concentrically braced frames. The design of the proposed braced frames system began with the layout of the braced frames on the floor plan of the KINSC. To maintain torsional resistance throughout the building, the location of the frames coincides with the location of the existing precast shear walls. However, a number of additional frames were added to ensure that there will be adequate support for the controlling lateral loads in both directions. Keep in mind that the KINSC acts as three structures independent of each other in lateral loading due to the use of 2" expansion joints located between each wing of the building. Once the layout of the braced frames was decided, models of the individual frames were created in the design program STAAD Pro.

Initially, the beam and column sizes as outputted by RAM due to the gravity loads were used in the braced frame design as a starting trial size. Once the frames were created in STAAD, the controlling seismic loads, as seen in Table 3, were applied to the corresponding structures. Lateral forces in the braced frames due to torsion created on the building were also calculated for each frame. The complete set of torsional forces due to seismic loading can be reviewed in Appendix C. The calculated loads were distributed to each frame by the frame stiffness. In most cases, frames in the same direction were designed to have equivalent stiffness, therefore distributing the lateral load evenly among those frames. Following the application of the loads, the frames were analyzed. With the help of STAAD, the lateral drift of each frame was obtained. All drift values were designed to comply with a deflection limit of L/600, with L being equal to the total height of the braced frame. For all braced frames, this drift limit of L/600 was approximately 1.04". This standard corresponds with the ASCE 7-02 design code. When considering this deflection limit, a number of the member sizes were increased in the model to ensure the drift did not exceed L/600. Furthermore, a design check was carried out for any braced frame columns that would be affected by biaxial loading. In STAAD Pro, the columns that would see biaxial loading were modeled in both the strong and weak axis and checked for axial loads. The design of all biaxial columns passed these checks. With all of the braced frames meeting the standard drift limit when considering direct and torsional lateral forces, the study of the proposed steel braced frame system was complete. Several STAAD output tables, verifying the story drift of the building, deflection limits, and frame designs, can be reviewed in Appendix D following this report.

Steel Connections Design:

As another portion of this structural depth study, two typical steel connections were designed by long-hand calculations. The first connection designed was a beam-to-girder shear connection and the second was a girder-to-column shear connection. Both designs were carried out in accordance with the LRFD 3rd Edition Steel Manual.

The beam-to-girder connection that was selected for design was a single angle bolted shear connection. A typical beam size of W10X12 and girder size of W21X44 were used for the design calculations. The connection was ultimately designed for a number of limit states. Conservative loads and assumptions were used for the design of the steel connections. The following limit states were checked for the bolts, the angle, and the members:

- Angle Shear Yield
- Angle Shear Rupture
- Angle Block Shear Rupture
- Angle Flexural Yield
- Angle Flexural Rupture
- Beam Web Block Shear
- Coped Beam Flexure
- Bearing / Tear Out
- Bolt Shear

Using the limit states listed above as design criteria, a typical beam-to-girder shear connection consisting of an L3½” X 3½” X ½” with 2 bolts and a length of 6” was selected. A325N bolts with a ¾” diameter were used for this design. The controlling limit states were beam tear out and bolt shear.

For the second connection design, a typical girder of W21X44 and column of W10X33 were utilized. The design of the typical girder-to-column connection also resulted in a single angle bolted connection. This type of connection was selected to ensure that the bolting of the connection to the column remained somewhat simple. Also, when considering this connection, the size of the angle had to carefully be selected to ensure that the connection could fit within the given dimensions of the column. Virtually, the same limit states checked in the first connection were checked throughout the design of this connection, with the exception of Coped Beam Flexure. The final design of this girder-to-column connection resulted in an L3" X 3" X ½" with 6 bolts and had an overall length of 18". Again, the bolts used for this connection were A325N bolts with a diameter of ¾". Bolt shear proved to be the controlling limit state for this typical girder-to-column connection. Explicit calculations for both connections can be reviewed in Appendix E following this report.

C.M. Breadth Study

Construction Management Breadth Study

Problem Statement:

Directly correlating with the purpose for the structural depth study, the first breadth study of the KINSC was a Construction Management study. The initial reason for investigating an alternative structural system was to directly compare the existing and proposed systems in categories such as project cost and project schedule in search of potential benefits of the proposed system. Therefore, in this breadth study, a detailed cost comparison and project schedule were carried out.

Solution Process:

First, a cost estimate for the existing and proposed structural systems was put together. In each cost estimate, the components of only the structural system were investigated. Structural aspects such as the steel roof and the precast foundation elements were not included in either cost estimate because they remained unchanged in design for both the existing and proposed systems. Thus, the prices of those structural elements would have impacted the cost equally in both estimates. To prepare the cost estimates for both systems, R.S. Means 2006 Catalogs were used as well as the Construction Management computer program, CostWorks. Once cost estimates for both systems were completed, they were compared as a total cost number and as a cost per square foot of building number.

The second investigation to be carried out for this breadth study was the project schedule comparison between the existing and proposed structural systems. Similar to the cost comparison, this investigation made use of the R.S. Means Catalogs for 2006 in terms of Crew numbers and daily output. From the start, it was assumed that the construction for the KINSC was completed in phases starting with the East Wing, then the Link, finishing with the West Wing. For each wing, the floors were erected logically in ascending order to the completion of the fourth floor. This was typical for the existing precast system and the proposed steel system. The daily output values of the structural members for each floor were inputted into the Microsoft Office Project program. The program output displays the total duration of the construction for the structural systems only. The project schedules for the existing system and the proposed system were then directly compared.

Existing System:

When the cost estimate for the existing system was prepared, only the structural elements were included, such as the following: precast concrete columns, beams, hollow core plank flooring, concrete topping, CMU bearing walls, and precast shear walls. Each of these structural elements were totaled in terms of square footage, linear feet, or quantity, and then multiplied by the R.S. Means unit costs to produce a total cost for each structural element. The total cost for the entire existing precast system was then calculated. As a result, the total estimated cost for the existing precast concrete structural system was found to be roughly $1.59M. With an approximate area of 92,000 square feet, this yields a cost of nearly $17.31/square foot. Table 4 below displays the total cost of the existing precast system as an absolute cost and as a cost/ft. value.

With the use of the R.S. Means Catalogs as well as CostWorks, the daily output values were obtainable for each structural element of the existing precast system. As previously stated, it was assumed for simplicity that the KINSC was constructed by Wing, starting with the East Wing, then the Link, and then followed by the West Wing. All wings could have been constructed simultaneously to condense the project duration; however that would have required more crews which would increase the labor cost. Since the schedules are comparative between the two systems, an engineering decision was made to maintain lesser crews to save cost. To allow the schedule comparison to remain accurate and relative, the same decision was made for the proposed steel system. The daily output was calculated for all structural elements and laid out by floor. Furthermore, a logical construction process was planned out and inputted into the Microsoft Office

Project program. Once the construction of all floors for the three wings was completely scheduled, the total project duration resulted as being 21 weeks.

Proposed System:

The same procedure was conducted for the cost estimate of the proposed system. So, similar to the existing structural system, the cost estimate for the proposed steel system was prepared taking the following structural elements into consideration only: steel beams, columns, braced frames, composite concrete slab, metal deck, shear studs, connections, and fireproofing. Again, to gain the overall cost for each of these structural elements, they were totaled in terms of square footage, linear feet, or quantity, then multiplied by the R.S. Means unit costs. The total cost of the entire proposed steel system was then calculated to be approximately $1.36M. Given an approximate area of 92,000 square feet, a cost of $14.85/square foot was calculated. Refer to Table 4 below to view the absolute total cost and the total cost/ft. value. The table also provides the percent of total cost saved by using the proposed steel structural system.

The methods used to construct the project schedule of the existing precast system were repeated for the project schedule of the proposed system. The daily output values were obtained for all structural elements with the use of R.S. Means and Costworks. The construction process was planned out by ascending floors and inputted into the Microsoft Office Project program. The resulting total project duration for the proposed steel structural system proved to be 27 weeks. Refer to Appendix F for the estimated project schedules for both structural systems as well as the breakdown for both cost estimates, precast and structural steel.

Table 4: Summary of Building Cost and Percent Savings

Building Cost Breakdown		
Building System	**System Cost**	**Cost/sq. ft.**
Steel System	1361978.90	14.85
Precast System	1587370.96	17.31
Total Savings:	**225392.06**	
% Savings:	**14**	

Conclusions:

The results from this Construction Management breadth study are slightly inconclusive. From the cost comparison done between the existing precast structural system and the proposed steel structural system shows that there is a 14% saving in total structural cost if the proposed steel system is implemented. However, the construction schedule verifies that the proposed steel system is predicted to take an extended six weeks past the finish date of the existing precast system. Since neither system outweighs the other with certainty, an engineering decision was made declaring that the proposed steel system that will save 14% of the total cost is the more efficient system in terms of cost and schedule.

Mechanical Breadth Study

Mechanical Breadth Study

Problem Statement:

For the second breadth study of the KINSC, a mechanical investigation was chosen. Being a "state-of-the-art" laboratory facility, an incredibly innovative mechanical system had been designed for this building. Due to this overwhelmingly efficient mechanical system, the options for improving the mechanical system were quite limited. However, it was noted that many of the laboratories are required to be temperature controlled, due to the type of testing or experimental work that will be taking place in the labs. Considering that a number of the labs lie directly above or below areas such as mechanical rooms, classrooms, or libraries, this presents an issue. With the requirement for temperature control, it is mandatory that the thermal transfer between all perimeter barriers of the labs meet certain requirements. As the existing floor system of precast hollow core plank was altered to the proposed system of composite concrete slab on metal deck, it was necessary to ensure that the total thermal resistance of the new floor system meets the Standard 90 minimum requirement as per the ASHRAE Handbook of 2001. As for the exterior walls and roof, they have not been altered from the existing design, which currently meet the Standard 90 requirements.

Solution Process:

To ensure that the thermal resistance of the proposed floor system was sufficient, the resistance values, or R values, were researched and recorded for each component of the flooring and ceiling system. Then the total R value was calculated for the typical floor section found separating the laboratories by story. This total R value was then compared to the Standard 90 minimum total R value as given by the ASHRAE Handbook from 2001. The Standard 90 minimum value for mass floors of non-residential buildings located in the specified location zone was used for this study. Reference Appendix G for all tables and values used from ASHRAE Standard 90.

Existing System:

As called out in the existing system, the finish for the laboratory floors is strictly the 2" topping slab found on the 10" hollow core plank. In addition the laboratory ceilings were left to show the exposed structure. The total thermal resistance from this precast plank floor system was approximated to be 6.79 Km/W. Using the location zone, 4-A, for Haverford, Pennsylvania, taken from Table B-1 of the ASHRAE Standard 90.1, this total thermal resistance had to satisfy the Standard 90 minimum value of 6.3 Km/W, which was easily accomplished. The Standard 90 minimum R-value was taken from Table 5.5-4 of the 2004 ASHRAE Standard, Energy Standard for Buildings Except Low-Rise Residential Buildings.

Proposed System:

Initially, the proposed design was going to coincide with the existing conditions regarding the finishes. The lab floor finish would consist of the top of the 4" composite concrete slab. Also, the laboratory ceilings would be left to expose the bare steel framing and metal deck. With this as the proposed finishing for the labs, the only layers of material that would be contributing to the thermal resistance through the floors were the concrete slab and the fireproofing. The total R-value from these two layers, as per the 2001 ASHRAE Handbook was calculated to be 3.63 Km/W, which does not meet the Standard 90 required minimum resistance of 6.30 Km/W. Therefore, changes leading to an increased total R-value were needed. After some investigation, a linoleum tile for the lab floors and an acoustical ceiling tile, which also provided a reasonable air space, were selected and then added to the typical floor section. With these additions to the floor and ceiling systems, the total R-value between floors increased to 7.54 Km/W. This total R-value does satisfy the Standard 90 minimum thermal resistance of 6.3 Km/W. Table 5 below displays the resulting R-values from the existing and proposed floor systems when compared to the Standard 90 requirement.

Table 5: Recorded R-values for all components contained in the Existing and Proposed floor systems.

Floor System	Floor Type	Component	Relevant thickness	R value (K*m/W)	Standard 90 required R-value (K*m/W)	Status
Existing	Mass Floor	10" Precast Hollow Core Plank w/ 2" Topping Slab	10" + 2"	**6.79**	6.3	**Acceptable**
Proposed	Mass Floor	4" Concrete Lab + 1.5 " Metal Deck	4.75"	0.38	6.3	**Acceptable**
		Tile, Linoleum		0.05		
		Fireproofing		3.25		
		Acoustical Ceiling Tile		2.86		
		1/2"-4" Air Space		1		
		Total Sum		**7.54**		

*References: 2001 ASHRAE Handbook, Spancrete manufacturer's website

Conclusion:

This Mechanical breadth study was intended to ensure that the thermal resistance between the floors meets ASHRAE Standard 90 required minimums due to the temperature control requirements for the laboratories. With the use of the 2001 ASHRAE Handbook and the Spancrete manufacturer’s website, the thermal resistance values, or R values, were obtained for a typical floor section of the existing precast system as well as the proposed steel system. Initially, the R value for the floor of the precast system met the Standard 90 minimum requirement, but the R value for the floor of the steel system did not. After some investigation, a typical floor tile and acoustical ceiling tile were selected and added to the typical floor section. This also provided an air space within the section. Once these additions were made, the R value for the proposed steel system surpassed the required minimum value set by ASHRAE Standard 90. This study ensures that the thermal resistance through the floors of the KINSC shall not violate the temperature control requirements for the labs.

Findings & Conclusions

Findings & Conclusions

This report holds the conclusive results of the year long thesis study performed on the KINSC. The purpose of the thesis study was ultimately to research an alternate structural system that could prove to be more efficient than the existing precast concrete system in terms of a cost comparison and construction schedule. The alternate structural system that was proposed was a steel framed system with composite slab on deck as the flooring.

Throughout this thesis study, several investigations were carried out. The first depth study involved a redesign process of the structural system of the KINSC. A steel framing system with composite slab was designed for the building with the help of some design/analysis programs. RAM Structural System was used to design the steel building for the gravity loads acting on the building in addition to the use of STAAD Pro, which was utilized for the design of the braced frame lateral system. All structural designs were performed in accordance with the LRFD 3rd Edition and the ASCE 7-02.

The second investigation performed was a construction management breadth study. Within this study, a cost comparison between the existing precast structural system and the proposed steel structural system was performed. In addition, a comparison of the construction schedule for the two systems was also performed. The design tools used for these investigations consisted of the R.S. Means 2006 catalogs, the CostWorks estimating program, as well as the Microsoft Office Project program, used to layout the project duration. The findings from the cost comparison proved that the proposed steel system is the more economical system, as it saves nearly 14% of the total cost of the structural system. The existing precast system resulted in a cost of $17.31/square foot, while the proposed steel system ran a cost of $14.85/square foot. However, the results of the construction schedule comparison proved that the existing system can be completely erected nearly 6 weeks prior to that of the proposed steel system.

The third and final investigation carried out for this thesis study was a mechanical breadth study. Given the requirement for different temperature controlled laboratories in the KINSC, I felt it was necessary to ensure that the thermal resistance between floors met the ASHRAE Standard 90 minimum requirements. Therefore, thermal resistance values, or R-values, were calculated for each of the structural systems. These values were then compared to the Standard 90 minimum value for non-residential structures with mass floor systems. It was found that the existing precast floor section did meet the Standard 90 requirements. However, initially, the assumed floor section for the proposed steel structural system did not maintain an overall R-value that passed the Standard 90 requirement. Therefore, additions such as new floor tile, acoustical ceiling tile, and an air space were included in the typical floor section. The resulting overall R-value finally surpassed the Standard 90 required minimum.

From this thesis study, an overall conclusion can be made as to which system proves to be the most efficient structural system. These conclusions are only based on the objectives of a cost comparison and construction schedule comparison between the two structural systems. From an engineer's standpoint, I found that the proposed steel framed system with composite slab on deck proved to be the more efficient structural system. I feel that the 14% total structural cost outweighs the 6 week extension in construction schedule.

References

References

- ACI 318 – 02, Building Code Requirements for Structural Concrete and Commentary
- AISC, Manual of Steel Construction, Load and Resistance Factor Design (LRFD), 3rd Edition
- ASCE 7 – 02, Minimum Design Loads for Buildings and Other Structures
- ASHRAE Handbook (2001), Fundamentals
- ASHRAE Standard 90.1 – 2004, Energy Standard for Buildings Except Low-Rise Residential Buildings
- CostWorks 2005
- R.S. Means (2006), Building Construction Cost Data
- Strescon Manufacturer's Website, www.strescon.com
- Spancrete Manufacturer's Website, www.spancrete.com
- Wheeling Deck Product Catalog

Appendix A

Steel Deck Details & Calculations

Composite Steel Deck Load Calculations:

DL = 45 psf
LL = 100 psf
TL = 1.2(45)+1.6(100) = 214 psf

Total slab depth = 4" + 1.5" = 5.5"

Slab Reinforcing: 4" Slab w/ 6x6 – W1.4xW1.4 WWF

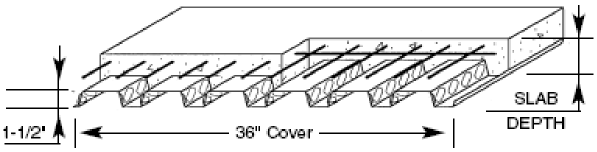
Deck Span = 7'-0

Deck Selected from Wheeling Deck Catalog:

1.5SB Normal Wt., 20 gage, Triple Span

$W_{allowable}$ = 400 psf > W_{actual} = 214 psf; **Acceptable**

1.5 SB Normal Weight



Section Properties (per ft. of width)

Gage	t in	Wd psf	Sp in³	Sn in³	Ip in⁴	In in⁴	As in²	Fy ksi
22	0.0295	1.7	0.172	0.180	0.146	0.182	0.478	50
20	0.0358	2.0	0.218	0.229	0.190	0.221	0.581	50
18	0.0474	2.7	0.301	0.311	0.284	0.294	0.769	40
16	0.0600	3.4	0.388	0.394	0.374	0.373	0.973	40

145 pcf Normal Weight Concrete

Total Slab Depth D / Wt. Conc. / Area Conc.	Gage	Maximum Unshored Clear Spans: Single Span	Double Span	Triple Span	Composite Properties: Iavg in⁴/ft	Sc in³/ft	Superimposed Live Loads - psf: No Studs — Span - Feet and Inches: 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
4"	22	5'-10"	7'-9"	7'-11"	3.573	0.887	400	34[illegible]	292	251	217	189	166	146	129	114	101	90
36.3 psf	20	6'-9"	9'-0"	9'-2"	3.854	1.052	400	400	352	303	262	229	201	178	158	140	125	111
20.6 in²	18	7'-2"	9'-5"	9'-8"	4.333	1.345	400	400	360	310	269	235	206	182	161	142	128	115
	16	8'-4"	10'-6"	10'-11"	4.782	1.638	400	400	360	310	269	235	206	182	161	142	128	115
4-1/2"	22	5'-6"	7'-5"	7'-6"	5.107	1.087	400	400	360	309	268	233	205	180	160	142	126	113
42.4 psf	20	6'-4"	8'-7"	8'-8"	5.496	1.291	400	400	400	373	324	283	249	220	195	174	156	140
24.8 in²	18	6'-9"	8'-11"	9'-3"	6.160	1.653	400	400	400	383	332	290	255	226	200	179	160	143
	16	7'-10"	10'-0"	10'-4"	6.789	2.018	400	400	400	383	332	290	255	226	200	179	160	143
5"	22	5'-3"	7'-1"	7'-2"	7.022	1.293	400	400	400	370	320	279	245	216	191	170	152	136
48.4 psf	20	6'-1"	8'-2"	8'-4"	7.544	1.538	400	400	400	400	388	339	298	264	235	209	187	168
29.3 in²	18	6'-5"	8'-6"	8'-9"	8.431	1.972	400	400	400	400	398	348	307	271	241	215	193	173
	16	7'-6"	9'-6"	9'-10"	9.280	2.415	400	400	400	400	398	348	307	271	241	215	193	173
5-1/2"	22	5'-0"	6'-9"	6'-10"	9.360	1.503	400	400	400	400	374	326	287	253	224	199	178	159
54.4 psf	20	5'-10"	7'-10"	7'-11"	10.036	1.791	400	400	400	400	400	397	349	309	275	245	220	197
34.1 in²	18	6'-2"	8'-2"	8'-5"	[illegible]	[illegible]	400	400	400	400	400	400	360	318	283	253	227	204
	16	7'-2"	9'-2"	9'-5"	12.298	2.824	400	400	400	400	400	400	360	318	283	253	227	204
6"	22	4'-10"	6'-6"	6'-7"	12.157	1.717	400	400	400	400	400	374	329	290	258	229	205	183
60.5 psf	20	5'-7"	7'-6"	7'-8"	13.012	2.048	400	400	400	400	400	400	400	355	316	282	253	227
39.4 in²	18	5'-11"	7'-10"	8'-1"	14.468	2.636	400	400	400	400	400	400	400	366	326	291	261	235
	16	6'-10"	8'-9"	9'-1"	15.883	3.242	400	400	400	400	400	400	400	366	326	291	261	235

D, Wc, Ac	Gage	Single Span	Double Span	Triple Span	Stud Factors: 2' o.c.	3' o.c.	Superimposed Live Loads - psf: Studs @ 1'-0" O.C. — Span - Feet and Inches: 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
4"	22	5'-10"	7'-9"	7'-11"	0.91	0.84	400	400	400	371	305	255	215	182	156	135	118	103
36.3 psf	20	6'-9"	9'-1"	9'-2"	0.88	0.82	400	400	400	400	329	275	231	197	169	146	127	111
20.6 in²	18	7'-2"	9'-5"	9'-5"	0.86	0.80	400	400	400	400	370	309	260	221	190	164	142	125
	16	8'-4"	10'-6"	10'-11"	0.83	0.78	400	400	400	400	400	341	287	244	209	181	157	138
4-1/2"	22	5'-6"	7'-5"	7'-6"	0.92	0.85	400	400	400	400	387	339	299	261	224	193	168	147
42.4 psf	20	6'-4"	8'-7"	8'-8"	0.89	0.83	400	400	400	400	400	392	330	281	241	208	181	158
24.8 in²	18	6'-9"	8'-11"	9'-3"	0.87	0.81	400	400	400	400	400	400	370	314	270	233	203	177
	16	7'-10"	10'-0"	10'-4"	0.84	0.79	400	400	400	400	400	400	400	347	297	257	223	195
5"	22	5'-3"	7'-1"	7'-2"	0.93	0.86	400	400	400	400	400	393	347	307	274	245	220	198
48.4 psf	20	6'-1"	8'-2"	8'-4"	0.89	0.84	400	400	400	400	400	400	400	371	330	285	248	217
29.3 in²	18	6'-5"	8'-6"	8'-9"	0.88	0.82	400	400	400	400	400	400	400	392	350	314	277	243
	16	7'-6"	9'-6"	9'-10"	0.85	0.81	400	400	400	400	400	400	400	400	400	351	305	267
5-1/2"	22	5'-0"	6'-9"	6'-10"	0.93	0.87	400	400	400	400	400	400	395	350	312	279	250	225
54.4 psf	20	5'-10"	7'-10"	7'-11"	0.90	0.85	400	400	400	400	400	400	400	400	378	339	305	276
34.1 in²	18	6'-2"	8'-2"	8'-5"	0.88	0.83	400	400	400	400	400	400	400	400	400	359	323	292
	16	7'-2"	9'-2"	9'-5"	0.85	0.82	400	400	400	400	400	400	400	400	400	400	400	354
6"	22	4'-10"	6'-6"	6'-7"	0.94	0.88	400	400	400	400	400	400	400	392	349	313	281	253
60.5 psf	20	5'-7"	7'-6"	7'-8"	0.91	0.86	400	400	400	400	400	400	400	400	400	381	343	310
39.4 in²	18	5'-11"	7'-10"	8'-1"	0.89	0.84	400	400	400	400	400	400	400	400	400	400	363	328
	16	6'-10"	8'-9"	9'-1"	0.86	0.83	400	400	400	400	400	400	400	400	400	400	400	400

1) Refer to the Design Notes, Note 7, for information on live load limits for fire-rated construction. See Page CD-3.
2) If stud spacing exceeds 1'-0" o.c., reduce live load by applicable stud factor listed above for actual stud spacing.
3) If welded wire fabric is not used, the live loads should be reduced by 10%.

Appendix B

RAM Output Designs & Values

RAM Takeoffs of Beams & Columns:

East Wing Beam Takeoff:

Gravity Beam Design Takeoff

RAM Steel v10.0
DataBase: east wing no roof
Building Code: IBC

Page 3/3
03/22/06 10:22:40
Steel Code: AISC LRFD

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	148	2196.85	22127
W10X12	262	5469.17	65881
W12X14	49	938.42	13284
W12X16	9	240.00	3846
W14X22	12	309.50	6835
W16X26	10	276.92	7237
W16X31	1	31.42	976
W18X35	3	94.25	3303
W18X40	4	125.67	5046
W21X44	36	1131.00	50031
	-----		-------------
	534		**178566**

Total Number of Studs = **9199**

East Wing Column Takeoff:

Gravity Column Design TakeOff

RAM Steel v10.0
DataBase: east wing no roof
Building Code: IBC

04/02/06 16:38:00
Steel Code: AISC LRFD

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W10X33	106	1378.0	45530
W10X39	25	325.0	12718
W10X45	12	156.0	7060
W10X49	1	13.0	637
	144		65945

Link Beam Takeoff:

Gravity Beam Design Takeoff

RAM Steel v10.0
DataBase: link
Building Code: IBC

04/01/06 03:14:51
Steel Code: ASD 9th Ed.

STEEL BEAM DESIGN TAKEOFF:

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	122	1894.79	19085
W10X12	40	874.50	10534
W16X26	7	181.42	4741
W18X40	8	187.75	7539
W21X44	5	129.58	5732
	-----		-------------
	182		**47631**

Total Number of Studs = **2322**

Link Column Takeoff:

Gravity Column Design TakeOff

RAM Steel v10.0
DataBase: link
Building Code: IBC

04/02/06 16:43:48
Steel Code: AISC LRFD

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W10X33	65	845.0	27920
W12X53	1	13.0	690
	66		28610

West Wing Beam Takeoff:

Gravity Beam Design Takeoff

RAM Steel v10.0 | Page 2/3
DataBase: west wing no roof | 03/22/06 11:00:03
Building Code: IBC | Steel Code: ASD 9th Ed.

Steel Grade: 50

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	256	4037.48	40666
W8X13	1	26.67	348
W10X12	29	628.17	7567
W12X14	11	217.34	3077
W12X19	6	111.83	2120
W14X22	33	696.17	15374
W16X26	11	262.59	6862
W16X31	1	20.17	627
W18X35	7	220.50	7728
W18X40	14	418.33	16797
W21X50	2	63.00	3151
W24X55	2	63.00	3494
W24X62	2	63.00	3923
W24X76	3	84.25	6422
	-----		-------------
	378		**118157**

Total Number of Studs = **5564**

West Wing Beam Takeoff:

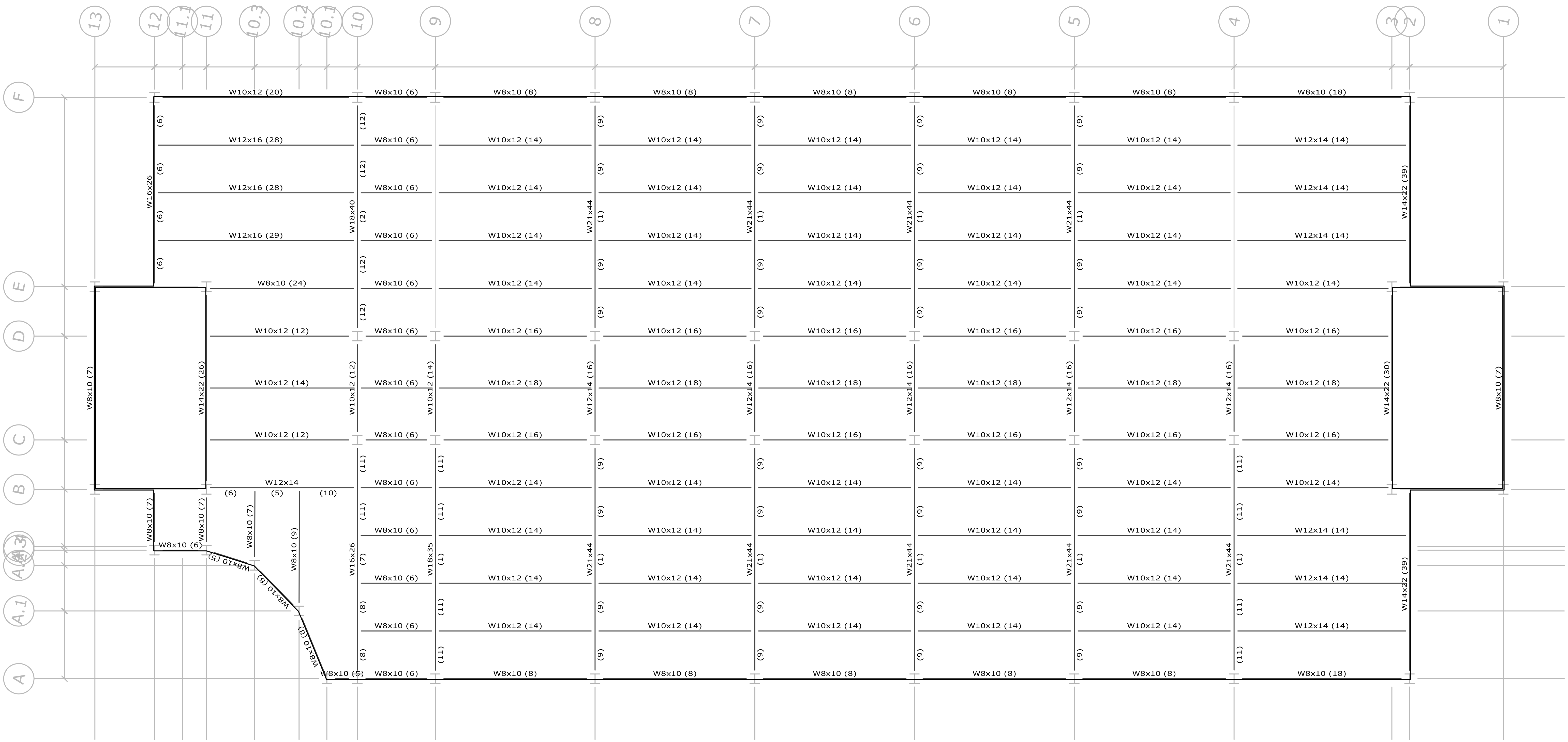
Gravity Column Design TakeOff

RAM Steel v10.0
DataBase: west wing no roof | 04/02/06 16:45:45
Building Code: IBC | Steel Code: AISC LRFD

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W10X33	88	1144.0	37799
W10X39	5	65.0	2544
W10X45	7	91.0	4118
	100		44461



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W8x10 (8)
W8x10 (18)
W12x16 (28)
W12x16 (29)
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W12x14
W16x26
W18x40
W21x44
W14x22 (39)
W8x10 (7)
W14x22 (26)
W10x12 (12)
W10x12 (14)
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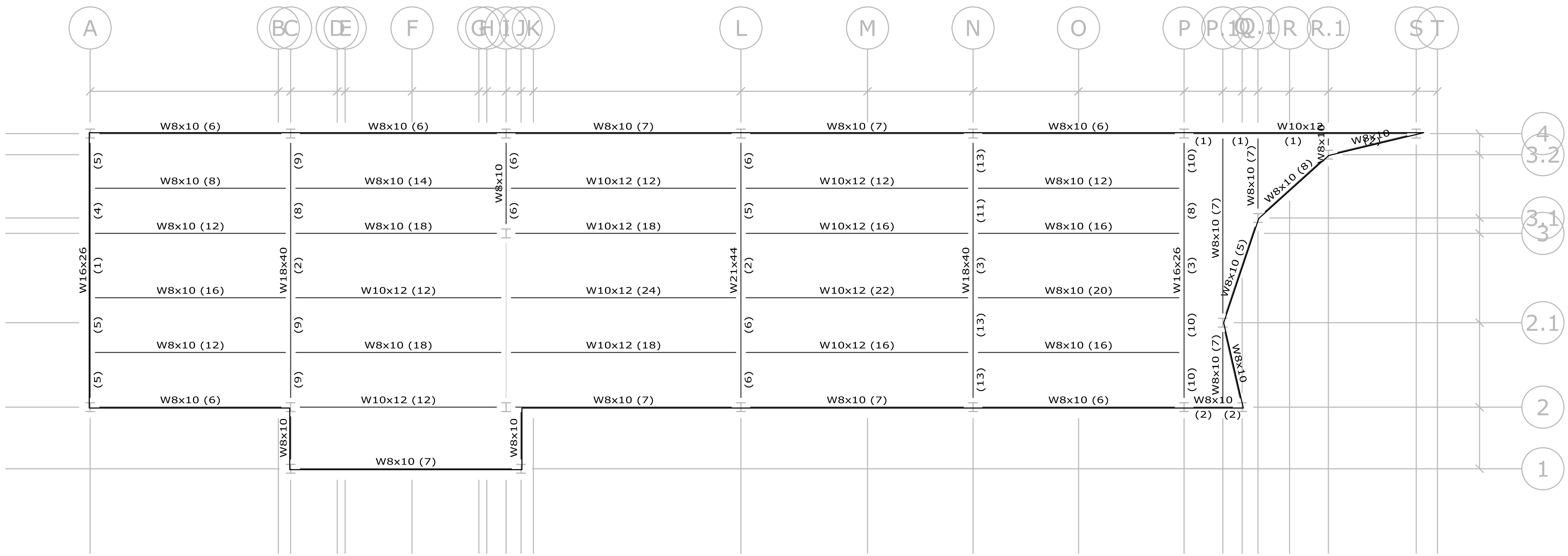
W14x22 (30)
W8x10 (6)
W8x10 (5)
W8x10 (9)
W18x35



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W8x10 (7)
W16x26
W18x40
W8x10
W21x44
W18x40
W16x26
W8x10
W8x10
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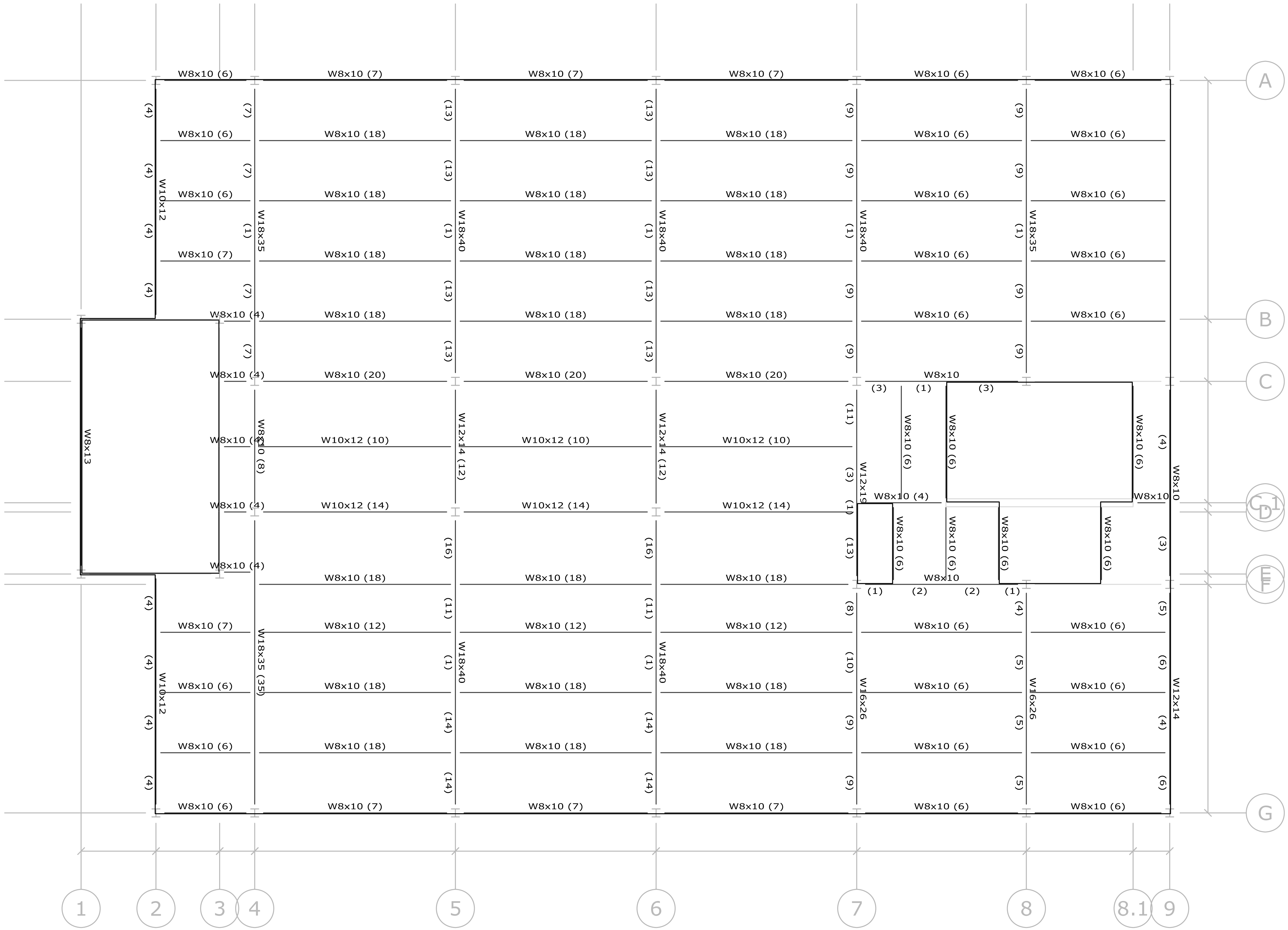
W8x10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W8x10 (6)
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W8x10 (20)
W8x10
W10x12 (10)
W10x12 (14)
W8x10 (12)
W10x12
W18x35
W18x40
W8x13
W8x10 (8)
W12x14 (12)
W12x19
W8x10 (6)
W18x35 (35)
W16x26
W12x14

Appendix C

Calculations of Lateral Loads

Wind Analysis

Simplified Method - ASCE 7 - 02 Sec. 6.4

Wind Load Factors			
mean building height (must be < 60'):	h (ft.) =	53.19	
Basic Wind Speed:	V (mph) =	90	From General Notes on Plans
Building Category:	Category	III	Table 1-1
Importance Factor:	I =	1.15	Table 6-1
Exposure Category:	Category	B	Sec. 6.5.6
Ht.& Exposure Adjustment Coeff.:	λ =	1.178	Fig. 6-2; by interpolation

Zone	p_{s30}	
A	12.8	Horizontal Pressures
B	-6.7	
C	8.5	
D	-4.0	
E	-15.4	Vertical Pressure
F	-8.8	
G	-10.7	
H	-6.8	

$p_s = \lambda * I * p_{s30}$
I = 1.15
p_{s30} = 12.8 - (-6.7)
λ: see below

height	λ	I	$p_{total} = \lambda * I * p_{s30}$ (psf)
15	1.00	1.15	21.42
20	1.00	1.15	21.42
25	1.00	1.15	21.42
30	1.00	1.15	21.42
35	1.05	1.15	22.16
40	1.09	1.15	22.74
45	1.12	1.15	23.19
50	1.16	1.15	23.78
55	1.19	1.15	24.22
60	1.22	1.15	24.66

Shears on West Wing & East Wing due to Wind Loads

		kips (n-s)		kips (e-w)	
Level	plf	West	East	West	East
Roof	170.0937	13.01	13.01	20.21	31.09
4th	311.0891	23.80	23.80	36.96	56.87
3rd	280.31	21.44	21.44	33.30	51.24
2nd	278.46	21.30	21.30	33.08	50.90
1st	0	0	0	0	0
Basement	------	------	------	------	------
Base Shear		79.56	79.56	123.55	190.10

Shears on Link due to Wind Loads

		kips (n-s)	kips (e-w)
Level	plf	Link	Link
Roof	146.36	18.69	5.15
3rd	280.31	35.80	9.87
2nd	278.46	35.56	9.80
1st	0	0.00	0.00
Basement	------	------	------
Base Shear		90.05	24.82

Building Information for Seismic Analysis:

Seismic Information	East & West Wings	Link
Building Location	Haverford, PA	Haverford, PA
# of stories	4	3
inner story ht.	13	13
Bldg. height	53	39
Seismic Use Group	II	II
Importance Factor	1.25	1.25
Site Classification	B	B
0.2s Acceleration	0.35	0.35
1.0s Acceleration	0.08	0.08
Site Class Factor:		
Fa	1.00	1.00
Fv	1.00	1.00
Adjusted Accelerations		
S_{ms}	0.35	0.35
S_{m1}	0.077	0.077
Spectral Response Accelerations		
S_{DS}	0.233	0.233
S_{D1}	0.051	0.051
Seismic Design Category	**B**	**B**

Seismic Analysis

East Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S}$ = 1+ ($T_{N\text{-}S}$ - 0.5)/(2.5 - 0.5) = 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	1297	53	55,579	0.263	73	-	3,860
4	2389	39	76,562	0.362	100	73	3,913
3	2389	26	52,162	0.247	68	173	1,777
2	2389	13	27,068	0.128	35	242	461
1						277	
	Σ = 8464		Σ = 211370	Σ = 1.000	Σ = 277		Σ = 10011

Link

Vertical Distribution of Seismic Forces

$k_{E\text{-}W}$ = 1+ ($T_{E\text{-}W}$ 0.5)/(2.5 - 0.5) = 0.906

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	561	39	15,516	0.333	37	-	1,432
3	1057	26	20,227	0.435	48	37	1,245
2	1057	13	10,794	0.232	26	85	332
1						110	
	Σ = 2674		Σ = 46537	Σ = 1.000	Σ = 110		Σ = 3009

West Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S}$ = 1+ ($T_{N\text{-}S}$ - 0.5)/(2.5 - 0.5) = 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	892	53	38,203	0.262	50	-	2,655
4	1650	39	52,873	0.363	69	50	2,703
3	1650	26	36,023	0.247	47	119	1,228
2	1650	13	18,693	0.128	25	167	319
1						191	
	Σ = 5841		Σ = 145792	Σ = 1.000	Σ = 191		Σ = 6904

Lateral Forces on Steel Braced Frames Due to Torsion

East Wing

L	162.67
W	76.5
centroid,y	81.335
centroid,x	38.25
y_{cr}	73.25
x_{cr}	38.25
$e_{acc,y}$	8.1335
$e_{acc,x}$	3.825
$e_{total,y}$	16.2185
$e_{total,x}$	3.825

E-W frames

			Frame 1				Frame 2				Frame 3, 4, 5, 6				
Story	Force	M	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	$\Sigma k_i*d_i^2$
4	86.5	2805.801	1	-44.3665	1968.386	**-21.16575**	1	59.8835	3586.034	**28.56838**	0.46	13.33	177.6889	**2.925273**	5881.367
3	34	1102.858	1	-44.3665	1968.386	**-8.319485**	1	59.8835	3586.034	**11.22919**	0.46	13.33	177.6889	**1.149818**	5881.367
2	17.5	567.6475	1	-44.3665	1968.386	**-4.282088**	1	59.8835	3586.034	**5.779731**	0.46	13.33	177.6889	**0.591818**	5881.367
1	0	0	1	-44.3665	1968.386	**0**	1	59.8835	3586.034	**0**	0.46	13.33	177.6889	**0**	5881.367

N-S frames

			Frame 1				Frame 2				Frame 3, 4, 5, 6				
Story	Force	M	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	$\Sigma k_i*d_i^2$
4	43.25	661.725	1	-44.3665	1968.386	**-4.816077**	1	59.8835	3586.034	**6.50048**	0.46	17.155	294.294	**0.856617**	6095.921
3	17	260.1	1	-44.3665	1968.386	**-1.893024**	1	59.8835	3586.034	**2.555102**	0.46	17.155	294.294	**0.336705**	6095.921
2	8.75	133.875	1	-44.3665	1968.386	**-0.974351**	1	59.8835	3586.034	**1.315126**	0.46	17.155	294.294	**0.173304**	6095.921
1	0	0	1	-44.3665	1968.386	**0**	1	59.8835	3586.034	**0**	0.46	17.155	294.294	**0**	6095.921

Lateral Forces on Steel Braced Frames Due to Torsion

Link

L	104
W	26
centroid,x	52
centroid,y	13
y_{cr}	13
x_{cr}	38.83
$e_{acc,x}$	5.2
$e_{acc,y}$	1.3
$e_{total,x}$	18.37
$e_{total,y}$	1.3

E-W frames

			Frame 7				**Frame 8**				**Frame 9**				
Story	**Force**	**M**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**$\Sigma k_i*d_i^2$**
4	18.5	48.1	1	14.3	204.49	**0.205108**	1	14.3	204.49	**0.205108**	0.74	-31.54	994.7716	**-0.334765**	3353.504
3	24	62.4	1	14.3	204.49	**0.266086**	1	14.3	204.49	**0.266086**	0.74	-31.54	994.7716	**-0.434289**	3353.504
2	13	33.8	1	14.3	204.49	**0.14413**	1	14.3	204.49	**0.14413**	0.74	-31.54	994.7716	**-0.23524**	3353.504
1	0	0	1	14.3	204.49	**0**	1	14.3	204.49	**0**	0.74	-31.54	994.7716	**0**	3353.504

N-S frames

			Frame 7				**Frame 8**				**Frame 9**				
Story	**Force**	**M**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**k**	**d_i**	**d_i^2**	**$F_{torsion}$**	**$\Sigma k_i*d_i^2$**
4	37	679.69	1	14.3	204.49	**2.898332**	1	14.3	204.49	**2.898332**	0.74	-31.54	994.7716	**-4.730483**	3353.504
3	48	881.76	1	14.3	204.49	**3.759998**	1	14.3	204.49	**3.759998**	0.74	-31.54	994.7716	**-6.136843**	3353.504
2	26	477.62	1	14.3	204.49	**2.036666**	1	14.3	204.49	**2.036666**	0.74	-31.54	994.7716	**-3.324123**	3353.504
1	0	0	1	14.3	204.49	**0**	1	14.3	204.49	**0**	0.74	-31.54	994.7716	**0**	3353.504

Lateral Forces on Steel Braced Frames Due to Torsion

West Wing

L	111
W	76
centroid,y	55.5
centroid,x	38
y_{cr}	63.17
x_{cr}	39.9
$e_{acc,y}$	5.55
$e_{acc,x}$	3.8
$e_{total,y}$	13.22
$e_{total,x}$	5.7

E-W frames

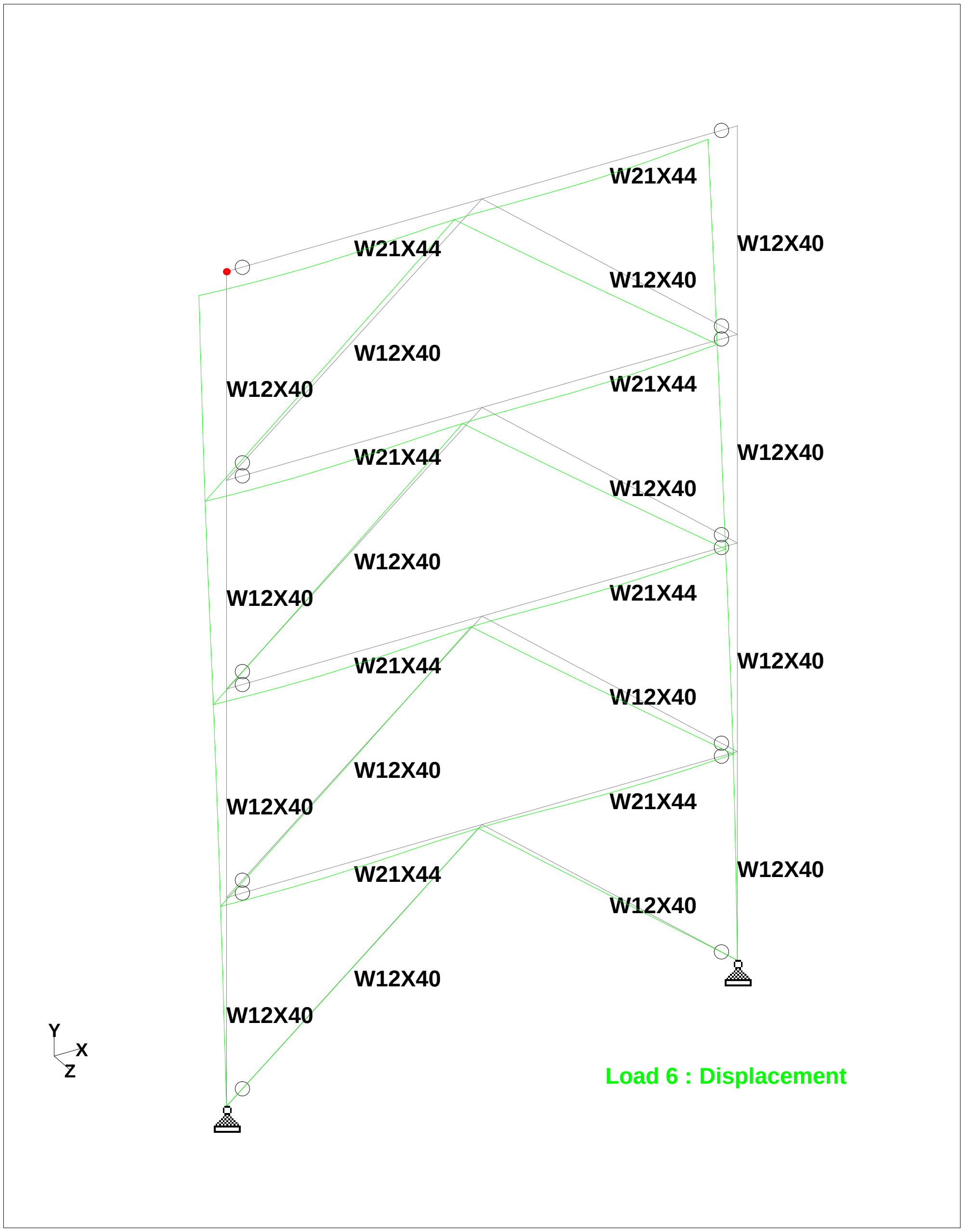
			Frame 10				Frame 11				Frame 12, 13, 14, 15				
Story	Force	M	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	$\Sigma k_i*d_i^2$
4	59.5	1573.18	0.85	-42.28	1787.598	**-9.390685**	1	68.72	4722.438	**15.26319**	0.48	20.93	438.0649	**2.231376**	7082.982
3	23.5	621.34	0.85	-42.28	1787.598	**-3.708926**	1	68.72	4722.438	**6.028321**	0.48	20.93	438.0649	**0.8813**	7082.982
2	12.5	330.5	0.85	-42.28	1787.598	**-1.972833**	1	68.72	4722.438	**3.206554**	0.48	20.93	438.0649	**0.468776**	7082.982
1	0	0	0.85	-42.28	1787.598	**0**	1	68.72	4722.438	**0**	0.48	20.93	438.0649	**0**	7082.982

N-S frames

			Frame 10				Frame 11				Frame 12, 13, 14, 15				
Story	Force	M	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	k	d_i	d_i^2	$F_{torsion}$	$\Sigma k_i*d_i^2$
4	29.75	678.3	0.85	-42.28	1787.598	**-4.048934**	1	68.72	4722.438	**6.580954**	0.48	20.93	438.0649	**0.962091**	7082.982
3	11.75	267.9	0.85	-42.28	1787.598	**-1.599159**	1	68.72	4722.438	**2.5992**	0.48	20.93	438.0649	**0.379986**	7082.982
2	6.25	142.5	0.85	-42.28	1787.598	**-0.850616**	1	68.72	4722.438	**1.382553**	0.48	20.93	438.0649	**0.20212**	7082.982
1	0	0	0.85	-42.28	1787.598	**0**	1	68.72	4722.438	**0**	0.48	20.93	438.0649	**0**	7082.982

Appendix D

STAAD Deflections & Diagrams of Proposed Steel Braced Frames

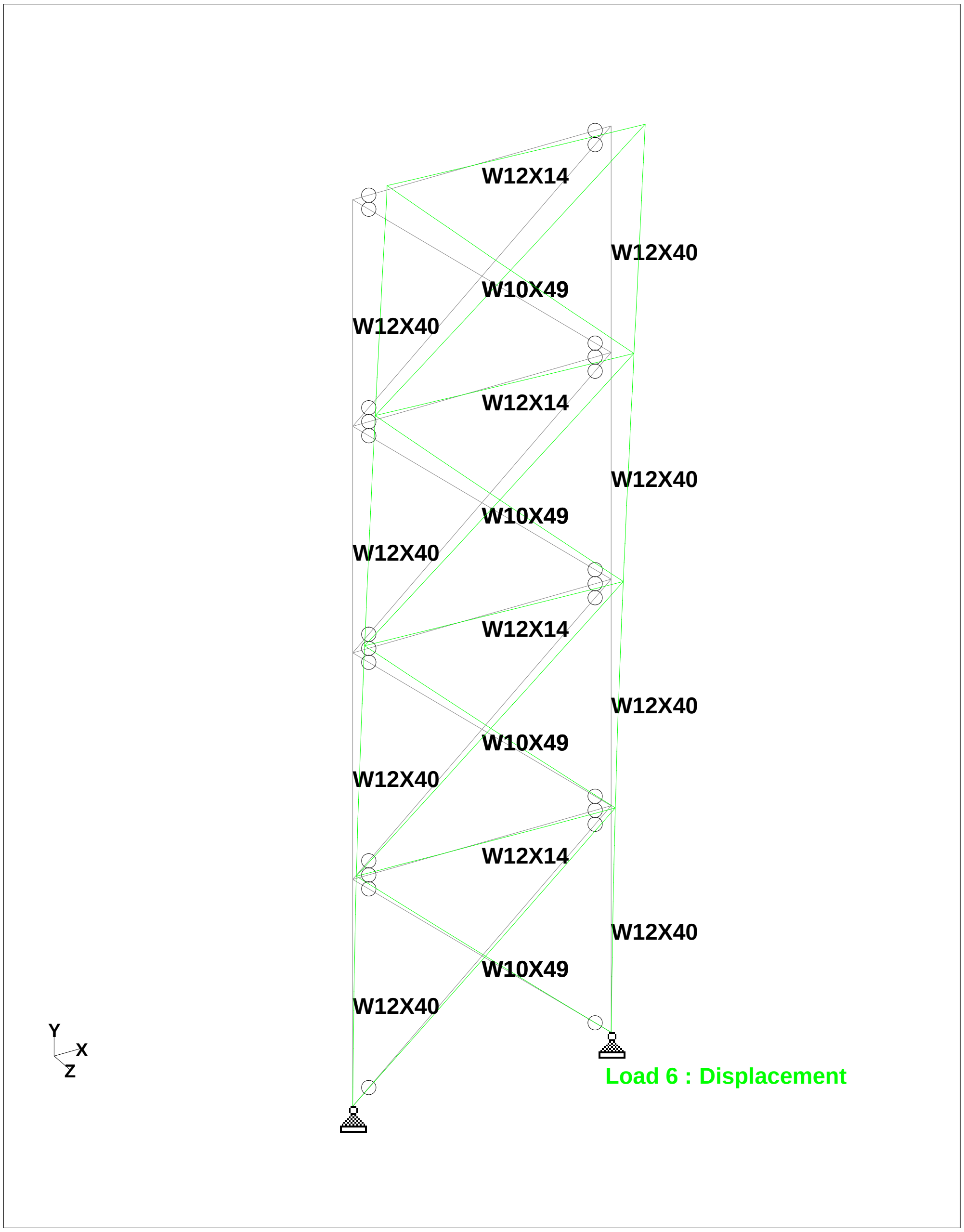


Whole Structure Displacements 0.5in:1ft 6 COMBINATION LOAD CASE 6

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	4	4:COMBINATI(	**0.033**	-0.163	0.000	0.166	0.000	0.000	-0.000
Min X	10	6:COMBINATI(	**-0.908**	-0.157	0.000	0.921	0.000	0.000	0.002
Max Y	10	3:EQ	-0.894	**0.160**	0.000	0.908	0.000	0.000	0.002
Min Y	14	4:COMBINATI(	-0.017	**-0.532**	0.000	0.533	0.000	0.000	0.000
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	10	6:COMBINATI(	-0.908	-0.157	0.000	0.921	0.000	0.000	**0.002**
Min rZ	3	4:COMBINATI(	0.000	0.000	0.000	0.000	0.000	0.000	**-0.000**
Max Rst	9	6:COMBINATI(	-0.850	-0.500	0.000	**0.986**	0.000	0.000	0.001

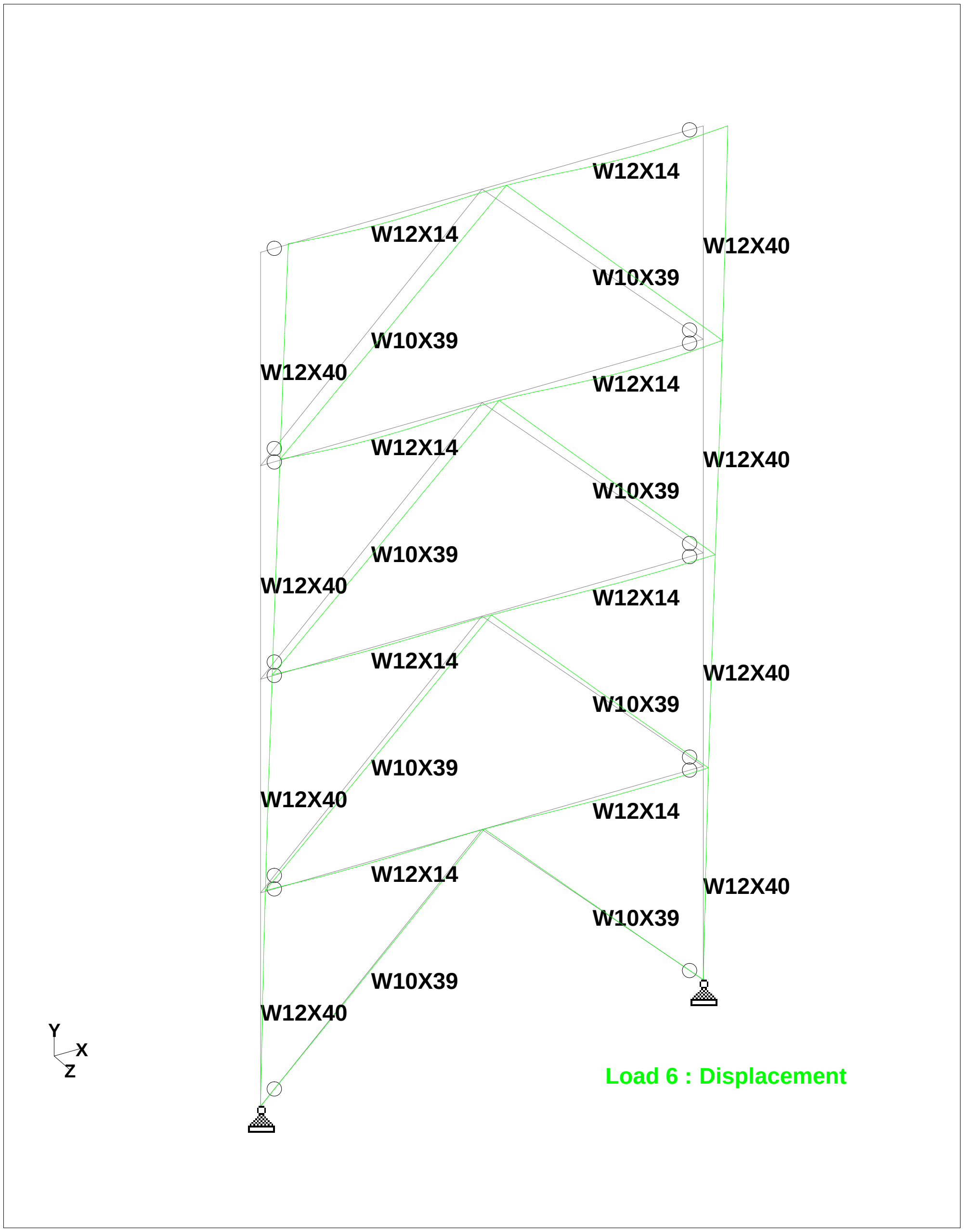
CHRIS SHELOW Software licensed to PENN STATE UNIVERSITY	Job No	Sheet No 1	Rev
	Part		
Job Title	Ref		
	By	Date 13-Mar-06	Chd
Client	File new East Wing NS x brac	Date/Time 02-Apr-2006 19:33	



Whole Structure Displacements 0.5in:1ft 6 1.2D+EQ+L

Node Displacement Summary

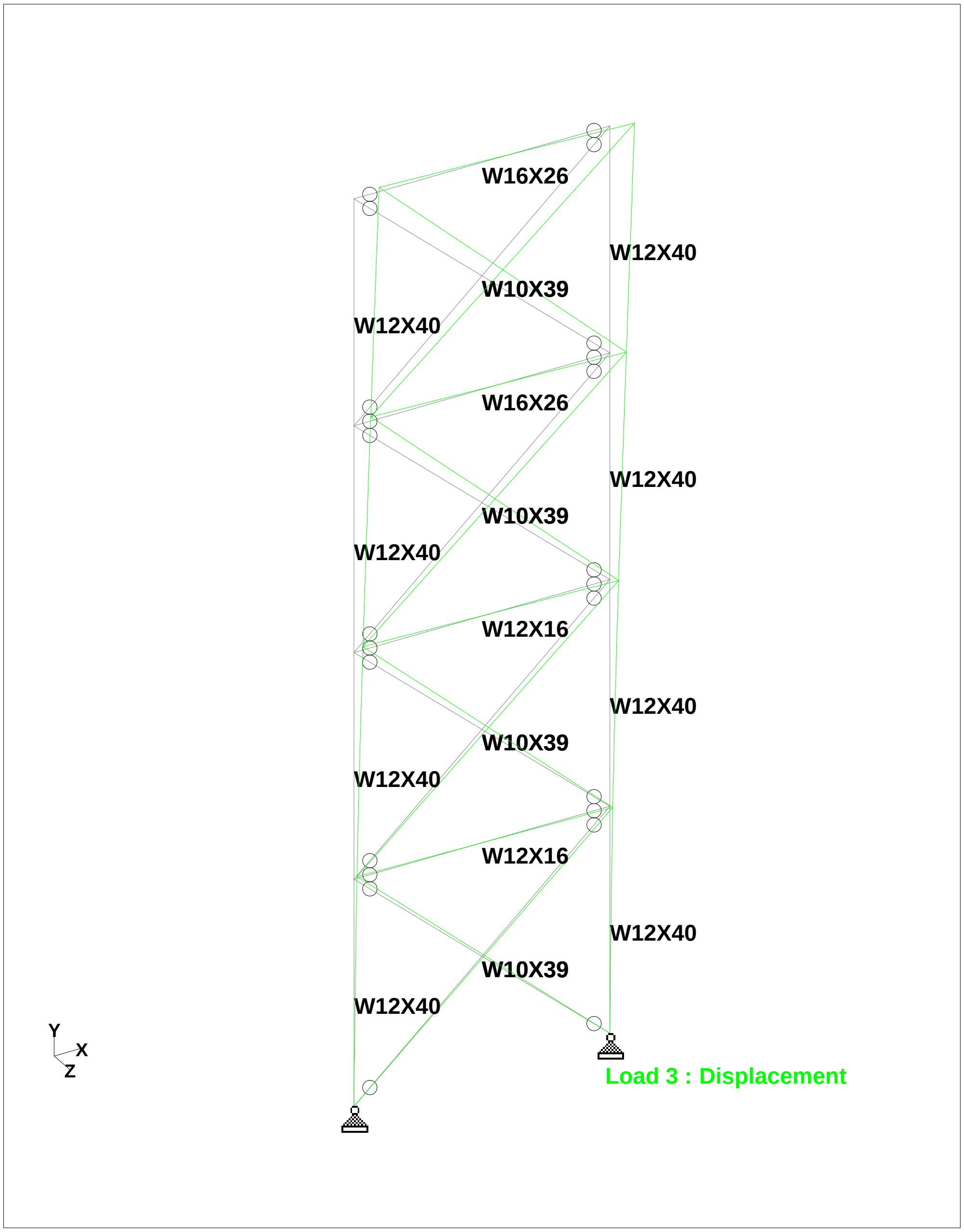
	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9	3:EQ	**0.982**	0.181	0.000	0.999	0.000	0.000	-0.002
Min X	2	4:1.2D+1.6L	**-0.008**	-0.017	0.000	0.019	0.000	0.000	0.000
Max Y	9	3:EQ	0.982	**0.181**	0.000	0.999	0.000	0.000	-0.002
Min Y	10	6:1.2D+EQ+L	0.965	**-0.233**	0.000	0.992	0.000	0.000	-0.002
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	1	4:1.2D+1.6L	0.000	0.000	0.000	0.000	0.000	0.000	**0.000**
Min rZ	9	6:1.2D+EQ+L	0.979	0.122	0.000	0.987	0.000	0.000	**-0.002**
Max Rst	9	3:EQ	0.982	0.181	0.000	**0.999**	0.000	0.000	-0.002



Whole Structure Displacements 0.5in:1ft 6 1.2D+L+E

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9	3:EQ	**0.831**	0.115	0.000	0.839	0.000	0.000	-0.001
Min X	5	4:1.2D+1.6L	**-0.023**	-0.083	0.000	0.086	0.000	0.000	0.000
Max Y	9	3:EQ	0.831	**0.115**	0.000	0.839	0.000	0.000	-0.001
Min Y	10	6:1.2D+L+E	0.738	**-0.211**	0.000	0.768	0.000	0.000	-0.001
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	10	4:1.2D+1.6L	0.000	-0.128	0.000	0.128	0.000	0.000	**0.000**
Min rZ	9	6:1.2D+L+E	0.831	0.019	0.000	0.831	0.000	0.000	**-0.002**
Max Rst	9	3:EQ	0.831	0.115	0.000	**0.839**	0.000	0.000	-0.001

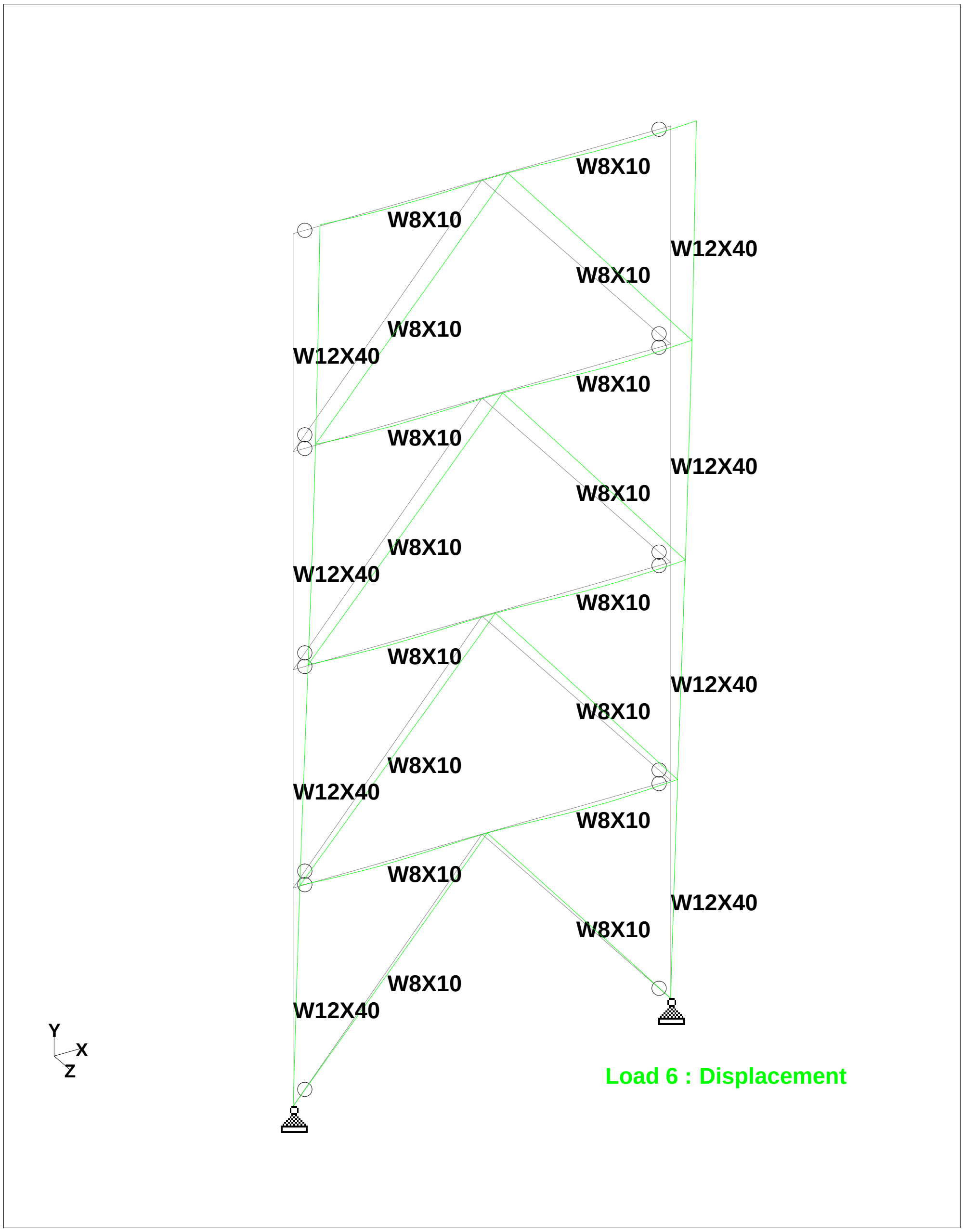


Whole Structure Displacements 0.5in:1ft 3 EQ

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9	3:EQ	**0.711**	0.126	0.000	0.722	0.000	0.000	-0.002
Min X	2	4:1.2D+1.6L	**-0.020**	-0.057	0.000	0.061	0.000	0.000	0.000
Max Y	9	3:EQ	0.711	**0.126**	0.000	0.722	0.000	0.000	-0.002
Min Y	10	6:1.2D+1.0L+1	0.702	**-0.225**	0.000	0.737	0.000	0.000	-0.001
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	1	4:1.2D+1.6L	0.000	0.000	0.000	0.000	0.000	0.000	**0.000**
Min rZ	9	6:1.2D+1.0L+1	0.710	0.022	0.000	0.710	0.000	0.000	**-0.002**
Max Rst	10	6:1.2D+1.0L+1	0.702	-0.225	0.000	**0.737**	0.000	0.000	-0.001

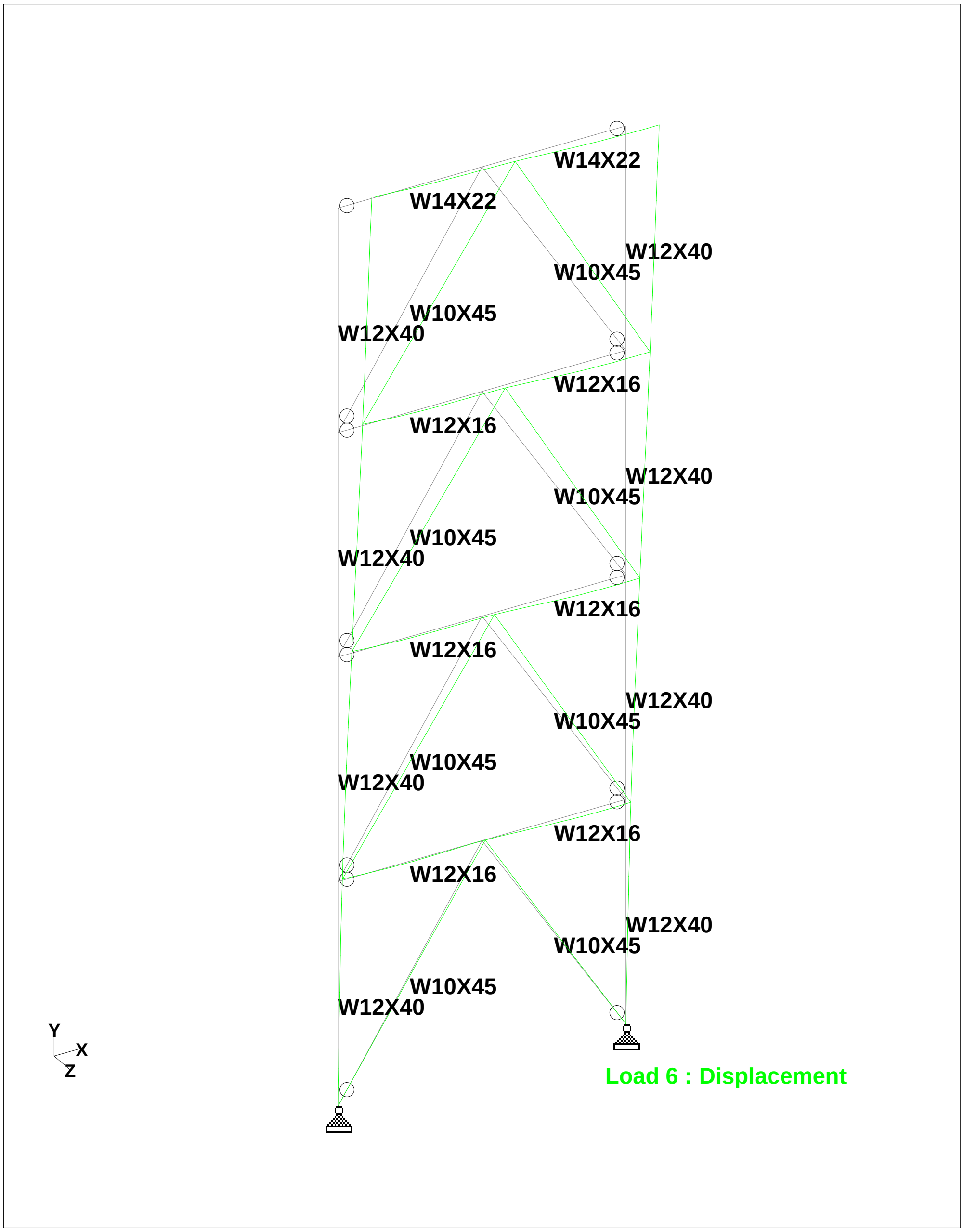
Chris Shelow	Job No	Sheet No 1	Rev
Software licensed to PENN STATE UNIVERSITY	Part		
Job Title	Ref		
	By	Date 01-Apr-06	Chd
Client	File Link E-W k brace.std	Date/Time 02-Apr-2006 19:35	



Whole Structure Displacements 0.5in:1ft 6 1.2D+E+L

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9	3:EQ	**0.787**	0.052	0.000	0.789	0.000	0.000	-0.001
Min X	5	4:1.2D+1.6L	**-0.007**	-0.020	0.000	0.021	0.000	0.000	-0.000
Max Y	9	3:EQ	0.787	**0.052**	0.000	0.789	0.000	0.000	-0.001
Min Y	10	6:1.2D+E+L	0.758	**-0.072**	0.000	0.762	0.000	0.000	-0.001
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	1	4:1.2D+1.6L	0.000	0.000	0.000	0.000	0.000	0.000	**0.000**
Min rZ	5	7:0.9D+E	0.444	0.042	0.000	0.446	0.000	0.000	**-0.002**
Max Rst	9	3:EQ	0.787	0.052	0.000	**0.789**	0.000	0.000	-0.001



Whole Structure Displacements 0.5in:1ft 6 1.2D+1.0L+1.0E

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9	3:EQ	**0.971**	0.141	0.000	0.981	0.000	0.000	-0.002
Min X	7	4:1.2D+1.6L	**-0.013**	-0.139	0.000	0.140	0.000	0.000	-0.000
Max Y	9	3:EQ	0.971	**0.141**	0.000	0.981	0.000	0.000	-0.002
Min Y	10	6:1.2D+1.0L+1	0.951	**-0.248**	0.000	0.983	0.000	0.000	-0.002
Max Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:DEAD	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:DEAD	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:DEAD	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	10	4:1.2D+1.6L	0.000	-0.148	0.000	0.148	0.000	0.000	**0.000**
Min rZ	5	3:EQ	0.407	0.127	0.000	0.426	0.000	0.000	**-0.002**
Max Rst	10	6:1.2D+1.0L+1	0.951	-0.248	0.000	**0.983**	0.000	0.000	-0.002

Appendix E

Calculations of Typical Steel Connections

Typical Steel Connection Calculations:

Beam-To-Girder Bolted-Bolted Connection:

Beam used: W10x12 Girder used: W21x44

V_u = **20.73 kips** M_u = 20.73*2" = **41.46"k**

use A325N bolts w/ ¾" diameter: ΦR_n = 15.9 kips

$C_{MIN} = V_u / \Phi R_n$ = 1.303 bolts; therefore **use 2 bolts**

Try an L3.5"x3.5"x0.5" with a length of 6"

Limit State Checks:

Angle Shear Yield:

$\Phi R_n = 0.9(0.6F_Y A_g) = 0.9*0.6*36*6*0.5$
= 58.32 kips > 20.73 kips; ok

Angle Shear Rupture:

$\Phi R_n = 0.75(0.6F_u A_n)$ = 55.46 kips > 20.73 Kips; ok

Angle Block Shear:

Tension Rupture = 46.2 K
Shear Yield = 72.9 k
Shear Rupture = 83.2 k
Tension Yield = 40.5 k

$\Phi R_n = 0.5(83.2+40.5)$ = 61.85 K > 20.73 K; ok

Angle Flexural Yield:

$\Phi M_n = 0.9(F_Y S_x)$ = 97.2"k > 41.46"k; ok

Angle Flexural Rupture:

S_{net} = 2.34
$\Phi M_n = \Phi F_u * S_{net}$ = 101.79"k > 41.46"k; ok

Beam Web Block Shear:

Tension Rupture = 51.8 K
Shear Yield = 101 k
Shear Rupture = 93.2 k
Tension Yield = 56.3 k

$\Phi R_n = 0.19(93.2+51.8)$ = 27.55 k >20.73 k; ok

Coped Beam Flexure:

ΦF_{bc} = 58.04 < 0.9*50 = 45; therefore use 45
$\Phi M_n = \Phi F_{bc} * S_{net}$ = 130.95"k > 41.46"k; ok

Bearing/T.O. & Bolt Shear:

Angle Bearing = $2.4F_u*t$ = 52.2 k

$T.O._{edge} = 1.2F_u*t*l_e$ = 29.4 k

$T.O._{other}$ = 76.1 k

Beam Bearing = 22.23 k

$T.O._{edge}$ = 12.5 k

ΦR_n = 0.75(12.5)+15.9 = 25.3 k > 20.73 k; ok

Girder-To-Column Bolted-Bolted Connection:

girder used: W21x44 Column used: W10x33

V_u = **68.56 kips** M_u =68.56*1.5"=**102.86"k**

use A325N bolts w/ ¾" diameter: ΦR_n = 15.9 kips

$C_{min} = V_u / \Phi R_n$ = 4.3 bolts; therefore **use 6 bolts**

Try an L3"x3"x0.5" with a length of 18"

Limit State Checks:

Angle Shear Yield:

$\Phi R_n = 0.9(0.6F_Y A_g)$ = 0.9*0.6*36*18*0.5

= 174.96 kips > 68.56 kips; ok

Angle Shear Rupture:

$\Phi R_n = 0.75(0.6F_u A_n)$ = 166.4 kips > 68.56 Kips; ok

Angle Block Shear:

Tension Rupture = 46.2 K

Shear Yield = 267 k

Shear Rupture = 305 k

Tension Yield = 40.5 k

ΦR_n = 0.5(305+40.5) = 172.8 K > 68.56 K; ok

Angle Flexural Yield:

$\Phi M_n = 0.9(F_Y S_x)$ = 874.8"k > 102.84"k; ok

Angle Flexural Rupture:

S_{net} = 19.3

$\Phi M_n = \Phi F_u * S_{net}$ = 839.6"k > 102.84"k; ok

Bearing/T.O. & Bolt Shear:

Angle Bearing = $2.4F_u*t$ = 52.2 k

$T.O._{edge} = 1.2F_u*t*l_e$ = 29.4 k

$T.O._{other}$ = 76.1 k

Beam Bearing = 40.95 k

$T.O._{edge}$ = 23.03 k

$T.O._{other}$ = 53.27 k

ΦR_n = 15.9*6 = 95.4 k > 68.56 k; ok

Appendix F

C.M. Cost Comparisons & Construction Schedules

Cost Breakdown for Existing Precast Concrete System

COLUMNS

Size	Length (total) ft.	Cost / ft Mat.	Labor	Equip.	Total	Total Cost
18X26 16X16 20X20	3172	74.5	19.55	10.7	104.75	332267

TOTAL = 332267.00

BEAMS

Size	Number #	Cost Info. Mat.	Labor	Equip.	Total	Total Cost
24x12	151	790.00	88.00	48.00	926.00	139826.00
12x20	60	780.00	88.00	48.00	916.00	54960.00
20x16	54	805.00	88.00	48.00	941.00	50814.00

TOTAL = 245600.00

PRECAST PLANK

Size	Area (total) sq. ft.	Cost Info. Mat.	Labor	Equip.	Total	Total Cost
10" hollow	91697.81	6.10	0.78	0.43	7.31	670310.99

TOTAL = 670311.00

CONCRETE TOPPING

Size	Area (total) sq. ft.	Cost / sq. ft. Mat.	Labor	Equip.	Total	Total Cost
2" topping	91697.81	1.04	0.67	0.27	1.98	181561.66

TOTAL = 181561.66

CMU BEARING WALLS

Size	Area (total) sq. ft.	Cost / sq. ft. Mat.	Labor	Equip.	Total	Total Cost
8"	360	1.81	2.76		4.57	1645.2
10"	3406	2.27	2.84		5.11	17404.66
12"	1911	2.46	3.58		6.04	11542.44
14"	1144	2.67	3.58		6.25	7150
					TOTAL =	37742.30

PC SHEAR WALLS

Size	Area (total) sq. ft.	Cost / sq. ft. Mat.	Labor	Equip.	Total	Total Cost
8" thick	6300	10.88	4.15	4	19.03	119889
					TOTAL =	119889.00

Building Cost Summary for Existing Precast System

Building Cost Breakdown	
Structure component	**System Cost**
P.C. Columns	332267.00
P.C. Beams	245600.00
Precast Planks	670311.00
Concrete Topping	181561.66
CMU Bearing Walls	37742.30
P.C. Shear Walls	119889.00
Total Cost:	**1587370.96**

Cost Breakdown for Proposed Structural Steel System

BEAMS

Size	Length (total) ft.	Cost Info. Mat.	Labor	Equip.	Total	Total Cost
W8X10	8759.87	10.45	3.63	2.38	16.46	144187.46
W10X12	6341.09	12.55	3.63	2.38	18.56	117690.63
W12X14	1155.76	14.65	2.48	1.62	18.75	21670.50
W12X16	272.92	17.45	2.48	1.62	21.55	5881.43
W12X19	111.83	23.00	2.48	1.62	27.10	3030.59
W14X22	1135.25	23.00	2.20	1.44	26.64	30243.06
W16X26	591.34	27.00	2.18	1.43	30.61	18100.92
W16X31	284.84	32.50	2.42	1.59	36.51	10399.51
W18X35	321.08	36.50	3.28	1.58	41.36	13279.87
W18X40	569.92	42.00	3.28	1.58	46.86	26706.45
W21X44	1182.83	46.00	2.96	1.42	50.38	59590.98
W21X50	63	52.50	2.96	1.42	56.88	3583.44
W24X55	63	57.50	2.84	1.37	61.71	3887.73
W24X62	63	65.00	2.84	1.37	69.21	4360.23
W24X76	84.25	79.50	2.84	1.37	83.71	7052.57
					TOTAL =	469665.36

COLUMNS

Size	Length (total) ft.	Cost Info. Mat.	Labor	Equip.	Total	Total Cost
W10X33	2379	34.50	3.96	2.59	41.05	97657.95
W10X39	312	40.68	3.96	2.59	47.23	14735.76
W10X45	208	46.90	3.96	2.59	53.45	11117.60
W10X49	13	51.00	3.96	2.59	57.55	748.15
W12X40	806	42.00	2.69	1.76	46.45	37438.70
W12X45	13	46.90	2.90	1.90	51.70	672.10
					TOTAL =	162370.26

LATERAL BRACES

Size	Length (total) ft.	Cost Info. Mat.	Labor	Equip.	Total	Total Cost
W10X39	921.1	40.68	3.96	2.59	47.23	43503.55
W10X45	123.17	46.90	3.96	2.59	53.45	6583.44
W10X49	627.17	51.00	3.96	2.59	57.55	36093.63
W12X22	326.75	23.00	2.48	1.62	27.10	8854.93

TOTAL = 95035.548

METAL DECK

Size	Area (total) sq. ft.	Cost / sq. ft. Mat.	Labor	Equip.	Total	Total Cost
gage	91697.81	1.14	0.26	0.02	1.42	130210.89

TOTAL = 130210.89

WWF SLAB REINFORCING

Size	Area (total) sq. ft.	Cost / 100 sq. ft Mat.	Labor	Equip.	Total	Total Cost
WWF 6X6 W1.4XW1.4	91697.81	12	18.05	0	30.05	27555.19191

TOTAL = 27555.19

COMPOSITE CONCRETE SLAB

Size	Area (total) sq. ft.	Cost / 100 sq. ft Mat.	Labor	Equip.	Total	Total Cost
4"+1.5" deck	91697.81	1.18	0.66	0.27	2.11	193482.38

TOTAL = 193482.38

SHEAR STUDS

Size	Number #	Cost / 100 sq. ft Mat.	Labor	Equip.	Total	Total Cost
3/4" DIA., 4" LONG	16663	0.46	0.69	0.28	1.43	23828.09
					TOTAL =	23828.09

FIREPROOFING

Component	Area (total) sq. ft.	Cost / 100 sq. ft Mat.	Labor	Equip.	Total	Total Cost
DECK	91697.81	0.62	0.54	0.09	1.25	114622.2625
BEAMS	73499.93	0.41	0.45	0.07	0.93	85278.9633
COLUMNS	14700.14	0.88	0.97	0.15	2	29400.28
					TOTAL =	229301.51

CONNECTIONS

Size	Area (total) sq. ft.	Cost / 100 sq. ft Mat.	Labor	Equip.	Total	Total Cost
L3.5X3.5X0.5	576	8	3.45	2.21	13.66	7868.16
L3X3X0.5	1722	7.5	3.45	2.21	13.16	22661.52
					TOTAL =	30529.68

Building Cost Summary for Existing Precast System

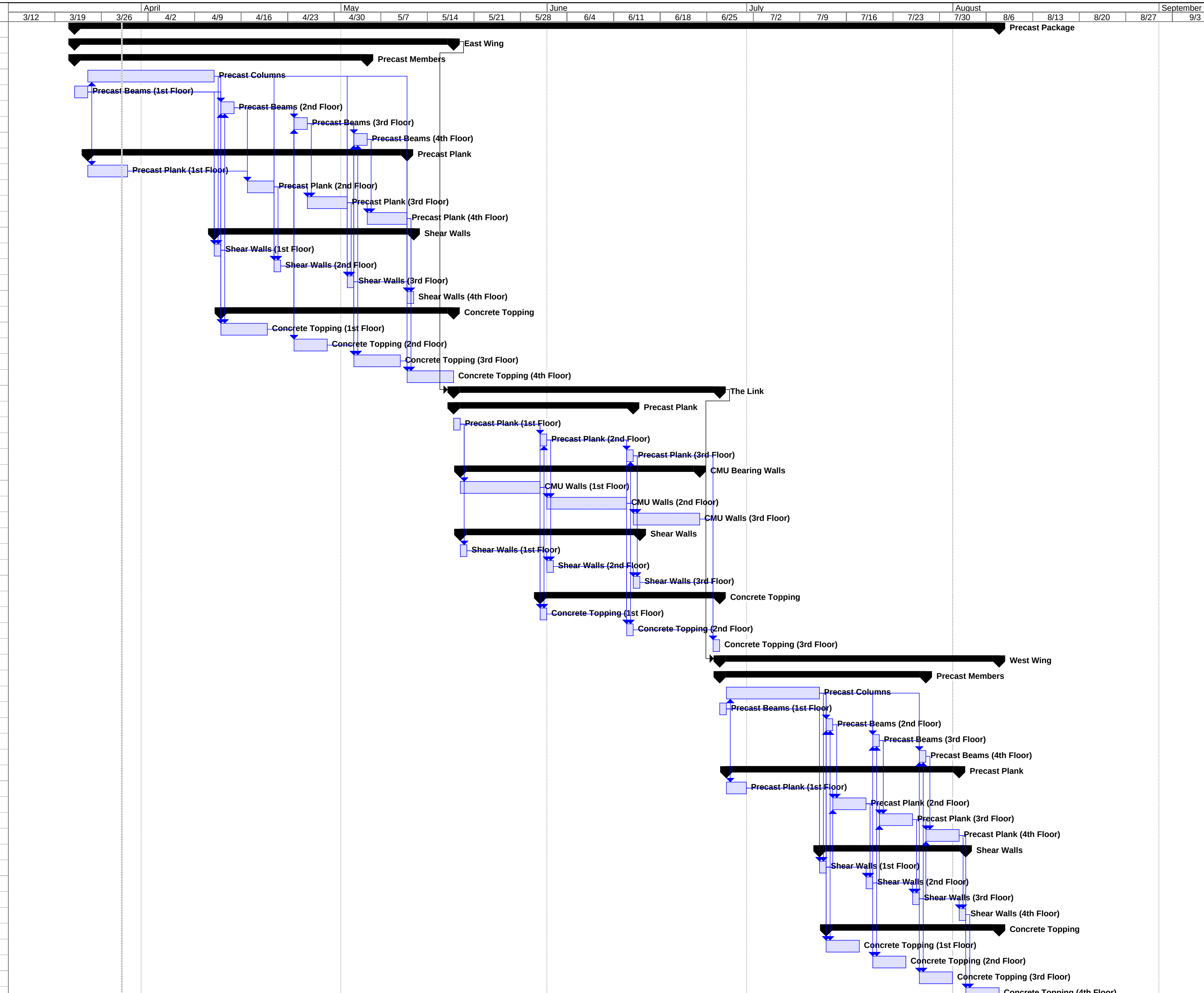
Building Cost Breakdown	
Structure component	**System Cost**
Structural Beams	469665.36
Structural Columns	162370.26
Lateral Braces	95035.548
Metal Deck	130210.89
WWF Slab Reinforcing	27555.19
Concrete Slab	193482.38
Fireproofing	229301.51
Connections	30529.68
Shear Studs	23828.09
Total Cost:	**1361978.902**

Building Cost Comparison of Both Structural System

Building Cost Breakdown		
Building System	**System Cost**	**Cost/sq. ft.**
Steel System	1361978.90	14.85
Precast System	1587370.96	17.31
Total Savings:	**225392.06**	
% Savings:	**14**	

Chris Shelow
Koshland Integrated Natural Science Center
Precast Package

ID		Task Name	Duration	Start	Finish	Predecessors
1		**Precast Package**	**99 days**	**Wed 3/22/06**	**Mon 8/7/06**	
2		**East Wing**	**41 days**	**Wed 3/22/06**	**Wed 5/17/06**	
3		**Precast Members**	**32 days**	**Wed 3/22/06**	**Thu 5/4/06**	
4		Precast Columns	13 days	Fri 3/24/06	Tue 4/11/06	5
5		Precast Beams (1st Floor)	2 days	Wed 3/22/06	Thu 3/23/06	
6		Precast Beams (2nd Floor)	2 days	Thu 4/13/06	Fri 4/14/06	5,4,10,15
7		Precast Beams (3rd Floor)	2 days	Mon 4/24/06	Tue 4/25/06	6,11,16
8		Precast Beams (4th Floor)	2 days	Wed 5/3/06	Thu 5/4/06	7,12,17
9		**Precast Plank**	**34 days**	**Fri 3/24/06**	**Wed 5/10/06**	
10		Precast Plank (1st Floor)	4 days	Fri 3/24/06	Wed 3/29/06	5
11		Precast Plank (2nd Floor)	4 days	Mon 4/17/06	Thu 4/20/06	10,6
12		Precast Plank (3rd Floor)	4 days	Wed 4/26/06	Mon 5/1/06	11,7
13		Precast Plank (4th Floor)	4 days	Fri 5/5/06	Wed 5/10/06	8,12
14		**Shear Walls**	**22 days**	**Wed 4/12/06**	**Thu 5/11/06**	
15		Shear Walls (1st Floor)	1 day	Wed 4/12/06	Wed 4/12/06	4,5,10
16		Shear Walls (2nd Floor)	1 day	Fri 4/21/06	Fri 4/21/06	4,6,11,15
17		Shear Walls (3rd Floor)	1 day	Tue 5/2/06	Tue 5/2/06	4,16,7,12
18		Shear Walls (4th Floor)	1 day	Thu 5/11/06	Thu 5/11/06	17,8,13,4
19		**Concrete Topping**	**25 days**	**Thu 4/13/06**	**Wed 5/17/06**	
20		Concrete Topping (1st Floor)	5 days	Thu 4/13/06	Wed 4/19/06	5,10,15
21		Concrete Topping (2nd Floor)	5 days	Mon 4/24/06	Fri 4/28/06	6,11,20,16
22		Concrete Topping (3rd Floor)	5 days	Wed 5/3/06	Tue 5/9/06	7,12,21,17
23		Concrete Topping (4th Floor)	5 days	Thu 5/11/06	Wed 5/17/06	8,13,22
24		**The Link**	**28 days**	**Thu 5/18/06**	**Mon 6/26/06**	**2**
25		**Precast Plank**	**19 days**	**Thu 5/18/06**	**Tue 6/13/06**	
26		Precast Plank (1st Floor)	1 day	Thu 5/18/06	Thu 5/18/06	
27		Precast Plank (2nd Floor)	1 day	Wed 5/31/06	Wed 5/31/06	26,30
28		Precast Plank (3rd Floor)	1 day	Tue 6/13/06	Tue 6/13/06	27,31
29		**CMU Bearing Walls**	**26 days**	**Fri 5/19/06**	**Fri 6/23/06**	
30		CMU Walls (1st Floor)	8 days	Fri 5/19/06	Tue 5/30/06	26
31		CMU Walls (2nd Floor)	8 days	Thu 6/1/06	Mon 6/12/06	27,30
32		CMU Walls (3rd Floor)	8 days	Wed 6/14/06	Fri 6/23/06	28,31
33		**Shear Walls**	**19 days**	**Fri 5/19/06**	**Wed 6/14/06**	
34		Shear Walls (1st Floor)	1 day	Fri 5/19/06	Fri 5/19/06	26
35		Shear Walls (2nd Floor)	1 day	Thu 6/1/06	Thu 6/1/06	27,34
36		Shear Walls (3rd Floor)	1 day	Wed 6/14/06	Wed 6/14/06	28,35
37		**Concrete Topping**	**19 days**	**Wed 5/31/06**	**Mon 6/26/06**	
38		Concrete Topping (1st Floor)	1 day	Wed 5/31/06	Wed 5/31/06	26,30,34
39		Concrete Topping (2nd Floor)	1 day	Tue 6/13/06	Tue 6/13/06	27,31,35,38
40		Concrete Topping (3rd Floor)	1 day	Mon 6/26/06	Mon 6/26/06	28,32,36,39
41		**West Wing**	**30 days**	**Tue 6/27/06**	**Mon 8/7/06**	**24**
42		**Precast Members**	**23 days**	**Tue 6/27/06**	**Thu 7/27/06**	
43		Precast Columns	10 days	Wed 6/28/06	Tue 7/11/06	44
44		Precast Beams (1st Floor)	1 day	Tue 6/27/06	Tue 6/27/06	
45		Precast Beams (2nd Floor)	1 day	Thu 7/13/06	Thu 7/13/06	43,44,49,54
46		Precast Beams (3rd Floor)	1 day	Thu 7/20/06	Thu 7/20/06	43,45,50,55
47		Precast Beams (4th Floor)	1 day	Thu 7/27/06	Thu 7/27/06	43,46,51,56
48		**Precast Plank**	**25 days**	**Wed 6/28/06**	**Tue 8/1/06**	
49		Precast Plank (1st Floor)	3 days	Wed 6/28/06	Fri 6/30/06	44
50		Precast Plank (2nd Floor)	3 days	Fri 7/14/06	Tue 7/18/06	45,54,49
51		Precast Plank (3rd Floor)	3 days	Fri 7/21/06	Tue 7/25/06	46,50,55
52		Precast Plank (4th Floor)	3 days	Fri 7/28/06	Tue 8/1/06	47,51,56
53		**Shear Walls**	**16 days**	**Wed 7/12/06**	**Wed 8/2/06**	
54		Shear Walls (1st Floor)	1 day	Wed 7/12/06	Wed 7/12/06	44,49,43
55		Shear Walls (2nd Floor)	1 day	Wed 7/19/06	Wed 7/19/06	50,54
56		Shear Walls (3rd Floor)	1 day	Wed 7/26/06	Wed 7/26/06	51,55
57		Shear Walls (4th Floor)	1 day	Wed 8/2/06	Wed 8/2/06	52,56
58		**Concrete Topping**	**18 days**	**Thu 7/13/06**	**Mon 8/7/06**	
59		Concrete Topping (1st Floor)	3 days	Thu 7/13/06	Mon 7/17/06	43,54,49
60		Concrete Topping (2nd Floor)	3 days	Thu 7/20/06	Mon 7/24/06	50,55
61		Concrete Topping (3rd Floor)	3 days	Thu 7/27/06	Mon 7/31/06	51,56
62		Concrete Topping (4th Floor)	3 days	Thu 8/3/06	Mon 8/7/06	52,57

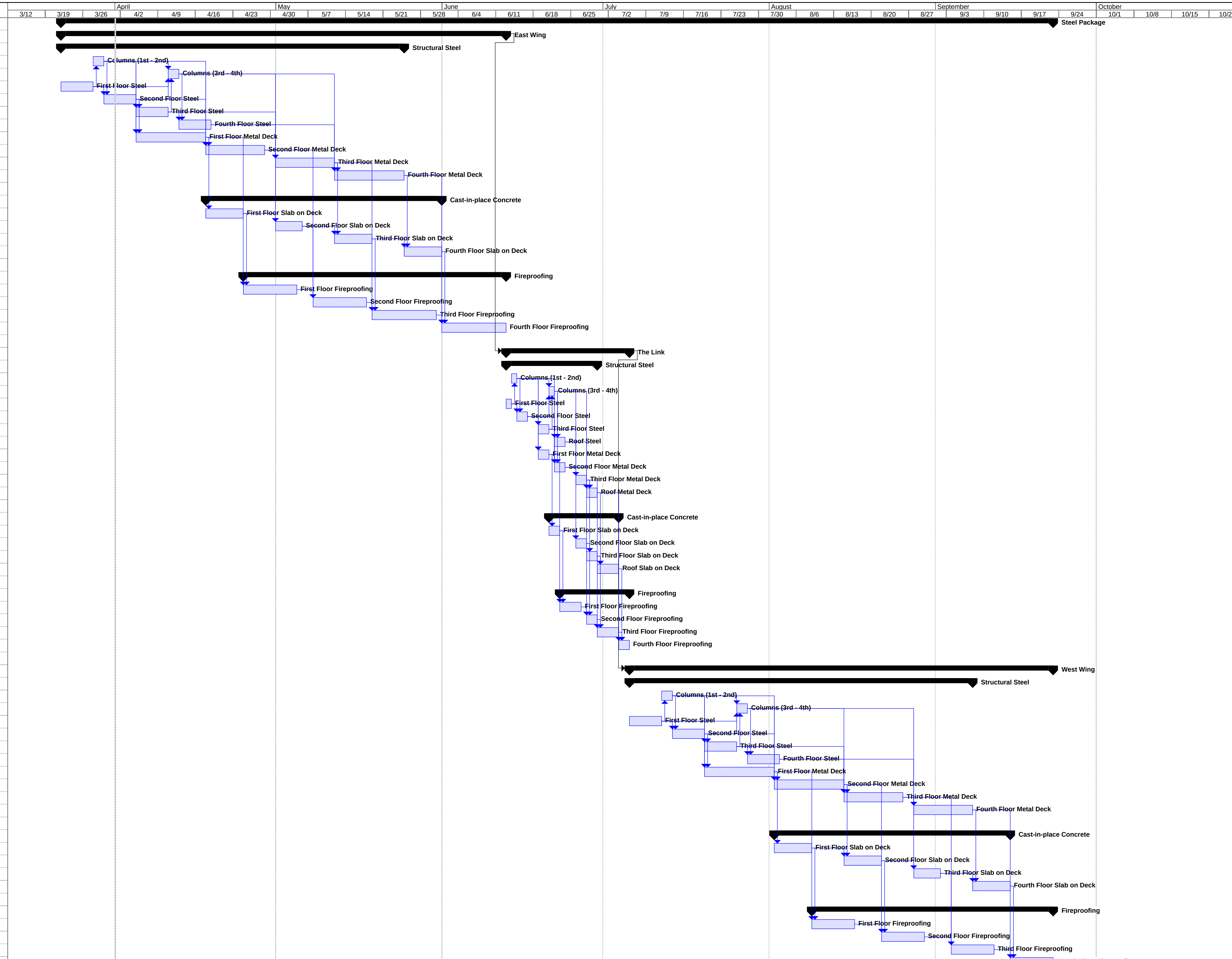


Project: Shelow Precast
Date: Wed 3/29/06

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

Chris Shelow
Koshland Integrated Natural Science Center
Steel Package Schedule

ID		Task Name	Duration	Start	Finish	Predecessors
1		**Steel Package**	**133 days**	**Wed 3/22/06**	**Fri 9/22/06**	
2		**East Wing**	**59 days**	**Wed 3/22/06**	**Mon 6/12/06**	
3		**Structural Steel**	**46 days**	**Wed 3/22/06**	**Wed 5/24/06**	
4		Columns (1st - 2nd)	2 days	Tue 3/28/06	Wed 3/29/06	6
5		Columns (3rd - 4th)	2 days	Tue 4/11/06	Wed 4/12/06	4,6,7,8
6		First Floor Steel	4 days	Wed 3/22/06	Mon 3/27/06	
7		Second Floor Steel	4 days	Thu 3/30/06	Tue 4/4/06	4,6
8		Third Floor Steel	4 days	Wed 4/5/06	Mon 4/10/06	7,4
9		Fourth Floor Steel	4 days	Thu 4/13/06	Tue 4/18/06	8,5
10		First Floor Metal Deck	9 days	Wed 4/5/06	Mon 4/17/06	4,6,7
11		Second Floor Metal Deck	9 days	Tue 4/18/06	Fri 4/28/06	4,5,7,8,10
12		Third Floor Metal Deck	9 days	Mon 5/1/06	Thu 5/11/06	5,8,9,11
13		Fourth Floor Metal Deck	9 days	Fri 5/12/06	Wed 5/24/06	9,12,5
14						
15		**Cast-in-place Concrete**	**32 days**	**Tue 4/18/06**	**Wed 5/31/06**	
16		First Floor Slab on Deck	5 days	Tue 4/18/06	Mon 4/24/06	10
17		Second Floor Slab on Deck	5 days	Mon 5/1/06	Fri 5/5/06	11,16
18		Third Floor Slab on Deck	5 days	Fri 5/12/06	Thu 5/18/06	12,17
19		Fourth Floor Slab on Deck	5 days	Thu 5/25/06	Wed 5/31/06	13,18
20						
21		**Fireproofing**	**35 days**	**Tue 4/25/06**	**Mon 6/12/06**	
22		First Floor Fireproofing	8 days	Tue 4/25/06	Thu 5/4/06	10,16
23		Second Floor Fireproofing	8 days	Mon 5/8/06	Wed 5/17/06	11,17,22
24		Third Floor Fireproofing	8 days	Fri 5/19/06	Tue 5/30/06	12,18,23
25		Fourth Floor Fireproofing	8 days	Thu 6/1/06	Mon 6/12/06	13,19,24
26						
27		**The Link**	**17 days**	**Tue 6/13/06**	**Wed 7/5/06**	**2**
28		**Structural Steel**	**13 days**	**Tue 6/13/06**	**Thu 6/29/06**	
29		Columns (1st - 2nd)	1 day	Wed 6/14/06	Wed 6/14/06	31
30		Columns (3rd - 4th)	1 day	Wed 6/21/06	Wed 6/21/06	29,31,32,33
31		First Floor Steel	1 day	Tue 6/13/06	Tue 6/13/06	
32		Second Floor Steel	2 days	Thu 6/15/06	Fri 6/16/06	29,31
33		Third Floor Steel	2 days	Mon 6/19/06	Tue 6/20/06	29,32
34		Roof Steel	2 days	Thu 6/22/06	Fri 6/23/06	30,33
35		First Floor Metal Deck	2 days	Mon 6/19/06	Tue 6/20/06	29,31,32
36		Second Floor Metal Deck	2 days	Thu 6/22/06	Fri 6/23/06	29,30,32,33,35
37		Third Floor Metal Deck	2 days	Mon 6/26/06	Tue 6/27/06	30,33,34,36
38		Roof Metal Deck	2 days	Wed 6/28/06	Thu 6/29/06	30,34,37
39						
40		**Cast-in-place Concrete**	**9 days**	**Wed 6/21/06**	**Mon 7/3/06**	
41		First Floor Slab on Deck	2 days	Wed 6/21/06	Thu 6/22/06	35
42		Second Floor Slab on Deck	2 days	Mon 6/26/06	Tue 6/27/06	41,36
43		Third Floor Slab on Deck	2 days	Wed 6/28/06	Thu 6/29/06	37,42
44		Roof Slab on Deck	2 days	Fri 6/30/06	Mon 7/3/06	38,43
45						
46		**Fireproofing**	**9 days**	**Fri 6/23/06**	**Wed 7/5/06**	
47		First Floor Fireproofing	2 days	Fri 6/23/06	Mon 6/26/06	35,41
48		Second Floor Fireproofing	2 days	Wed 6/28/06	Thu 6/29/06	36,42,47
49		Third Floor Fireproofing	2 days	Fri 6/30/06	Mon 7/3/06	37,43,48
50		Fourth Floor Fireproofing	2 days	Tue 7/4/06	Wed 7/5/06	38,44,49
57						
58		**West Wing**	**57 days**	**Thu 7/6/06**	**Fri 9/22/06**	**27**
59		**Structural Steel**	**46 days**	**Thu 7/6/06**	**Thu 9/7/06**	
60		Columns (1st - 2nd)	2 days	Wed 7/12/06	Thu 7/13/06	62
61		Columns (3rd - 4th)	2 days	Wed 7/26/06	Thu 7/27/06	60,62,63,64
62		First Floor Steel	4 days	Thu 7/6/06	Tue 7/11/06	
63		Second Floor Steel	4 days	Fri 7/14/06	Wed 7/19/06	60,62
64		Third Floor Steel	4 days	Thu 7/20/06	Tue 7/25/06	60,63
65		Fourth Floor Steel	4 days	Fri 7/28/06	Wed 8/2/06	61,64
66		First Floor Metal Deck	9 days	Thu 7/20/06	Tue 8/1/06	60,62,63
67		Second Floor Metal Deck	9 days	Wed 8/2/06	Mon 8/14/06	60,61,63,64,66
68		Third Floor Metal Deck	9 days	Tue 8/15/06	Fri 8/25/06	61,64,65,67
69		Fourth Floor Metal Deck	9 days	Mon 8/28/06	Thu 9/7/06	61,65,68
70						
71		**Cast-in-place Concrete**	**32 days**	**Wed 8/2/06**	**Thu 9/14/06**	
72		First Floor Slab on Deck	5 days	Wed 8/2/06	Tue 8/8/06	66
73		Second Floor Slab on Deck	5 days	Tue 8/15/06	Mon 8/21/06	67,72
74		Third Floor Slab on Deck	5 days	Mon 8/28/06	Fri 9/1/06	68,73
75		Fourth Floor Slab on Deck	5 days	Fri 9/8/06	Thu 9/14/06	69,74
76						
77		**Fireproofing**	**33 days**	**Wed 8/9/06**	**Fri 9/22/06**	
78		First Floor Fireproofing	6 days	Wed 8/9/06	Wed 8/16/06	66,72
79		Second Floor Fireproofing	6 days	Tue 8/22/06	Tue 8/29/06	67,73,78
80		Third Floor Fireproofing	6 days	Mon 9/4/06	Mon 9/11/06	68,74,79
81		Fourth Floor Fireproofing	6 days	Fri 9/15/06	Fri 9/22/06	69,75,80



Project: Shelow Steel shortened firepro
Date: Sat 4/1/06

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

Appendix G

ASHRAE Standard 90 Relevant Tables & Values

ASHRAE Standard 90 (2004) – Relevant Tables

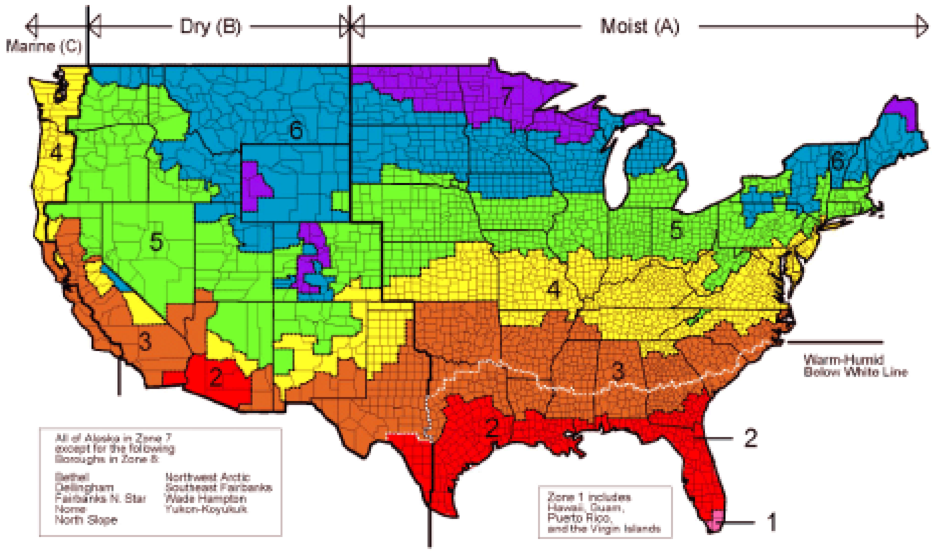


Figure B-1 Climate zones for United States locations.

TABLE B-1 U.S. Climate Zones

State / County	Zone
(North Dakota cont.)	
Adams	6A
Billings	6A
Bowman	6A
Burleigh	6A
Dickey	6A
Dunn	6A
Emmons	6A
Golden Valley	6A
Grant	6A
Hettinger	6A
LaMoure	6A
Logan	6A
McIntosh	6A
McKenzie	6A
Mercer	6A
Morton	6A
Oliver	6A
Ransom	6A
Richland	6A
Sargent	6A
Sioux	6A
Slope	6A
Stark	6A
Ohio (OH)	
Zone 5A Except	
Adams	4A
Brown	4A
Oregon (OR)	
Zone 4C Except	
Baker	5B
Crook	5B
Deschutes	5B
Gilliam	5B
Grant	5B
Harney	5B
Hood River	5B
Jefferson	5B
Klamath	5B
Lake	5B
Malheur	5B
Morrow	5B
Sherman	5B
Umatilla	5B
Union	5B
Wallowa	5B
Wasco	5B
Wheeler	5B
Pennsylvania (PA)	
Zone 5A Except	
Bucks	4A
Chester	4A
Delaware	4A
Montgomery	4A
Philadelphia	4A
York	4A
(South Dakota cont.)	
Jackson	5A
Mellette	5A
Todd	5A
Tripp	5A
Union	5A
Yankton	5A
Tennessee (TN)	
Zone 4A Except	
Chester	3A
Crockett	3A
Dyer	3A
Fayette	3A
Hardeman	3A
Hardin	3A
Haywood	3A
Henderson	3A
Lake	3A
Lauderdale	3A
Madison	3A
McNairy	3A
Shelby	3A
Tipton	3A
Texas (TX)	
Zone 3A Except	
Anderson	2A
Angelina	2A
Aransas	2A
(Texas cont.)	
Calhoun	2A
Cameron	2A
Chambers	2A
Cherokee	2A
Colorado	2A
Comal	2A
Coryell	2A
DeWitt	2A
Dimmit	2B
Duval	2A
Edwards	2B
Falls	2A
Fayette	2A
Fort Bend	2A
Freestone	2A
Frio	2B
Galveston	2A
Goliad	2A
Gonzales	2A
Grimes	2A
Guadalupe	2A
Hardin	2A
Harris	2A
Hays	2A
Hidalgo	2A
Hill	2A
Houston	2A

TABLE 5.5-4
Building Envelope Requirements For Climate Zone 4 (A,B,C)

		Nonresidential		Residential		Semiheated	
	Opaque Elements	Assembly Maximum	Insulation Min. R-Value	Assembly Maximum	Insulation Min. R-Value	Assembly Maximum	Insulation Min. R-Value
Roofs							
	Insulation Entirely above Deck	U-0.063	R-15.0 ci	U-0.063	R-15.0 ci	U-0.218	R-3.8 ci
	Metal Building	U-0.065	R-19.0	U-0.065	R-19.0	U-0.097	R-10.0
	Attic and Other	U-0.034	R-30.0	U-0.027	R-38.0	U-0.081	R-13.0
Walls, Above Grade							
	Mass	U-0.151[a]	R-5.7 ci[a]	U-0.104	R-9.5 ci	U-0.580	NR
	Metal Building	U-0.113	R-13.0	U-0.113	R-13.0	U-0.134	R-10.0
	Steel Framed	U-0.124	R-13.0	U-0.064	R-13.0 + R-7.5 ci	U-0.124	R-13.0
	Wood Framed and Other	U-0.089	R-13.0	U-0.089	R-13.0	U-0.089	R-13.0
Wall, Below Grade							
	Below Grade Wall	C-1.140	NR	C-1.140	NR	C-1.140	NR
Floors							
	Mass	U-0.107	R-6.3 ci	U-0.087	R-8.3 ci	U-0.322	NR
	Steel Joist	U-0.052	R-19.0	U-0.038	R-30.0	U-0.069	R-13.0
	Wood Framed and Other	U-0.051	R-19.0	U-0.033	R-30.0	U-0.066	R-13.0
Slab-On-Grade Floors							
	Unheated	F-0.730	NR	F-0.730	NR	F-0.730	NR
	Heated	F-0.950	R-7.5 for 24 in.	F-0.840	R-10 for 36 in.	F-1.020	R-7.5 for 12 in.
Opaque Doors							
	Swinging	U-0.700		U-0.700		U-0.700	
	Non-Swinging	U-1.450		U-0.500		U-1.450	
	Fenestration	Assembly Max. U (Fixed/ Operable)	Assembly Max. SHGC (All Orientations/ North-Oriented)	Assembly Max. U (Fixed/ Operable)	Assembly Max. SHGC (All Orientations/ North-Oriented)	Assembly Max. U (Fixed/ Operable)	Assembly Max. SHGC (All Orientations/ North-Oriented)
Vertical Glazing,% of Wall							
	0-10.0%	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-1.22	$SHGC_{all}$-NR
		U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-1.27	$SHGC_{north}$-NR
	10.1-20.0%	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-1.22	$SHGC_{all}$-NR
		U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-1.27	$SHGC_{north}$-NR
	20.1-30.0%	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-1.22	$SHGC_{all}$-NR
		U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-1.27	$SHGC_{north}$-NR
	30.1-40.0%	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-0.57	$SHGC_{all}$-0.39	U_{fixed}-1.22	$SHGC_{all}$-NR
		U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-0.67	$SHGC_{north}$-0.49	U_{oper}-1.27	$SHGC_{north}$-NR
	40.1-50.0%	U_{fixed}-0.46	$SHGC_{all}$-0.25	U_{fixed}-0.46	$SHGC_{all}$-0.25	U_{fixed}-0.98	$SHGC_{all}$-NR
		U_{oper}-0.47	$SHGC_{north}$-0.36	U_{oper}-0.47	$SHGC_{north}$-0.36	U_{oper}-1.02	$SHGC_{north}$-NR
Skylight with Curb, Glass,% of Roof							
	0-2.0%	U_{all}-1.17	$SHGC_{all}$-0.49	U_{all}-0.98	$SHGC_{all}$-0.36	U_{all}-1.98	$SHGC_{all}$-NR
	2.1-5.0%	U_{all}-1.17	$SHGC_{all}$-0.39	U_{all}-0.98	$SHGC_{all}$-0.19	U_{all}-1.98	$SHGC_{all}$-NR
Skylight with Curb, Plastic,% of Roof							
	0-2.0%	U_{all}-1.30	$SHGC_{all}$-0.65	U_{all}-1.30	$SHGC_{all}$-0.62	U_{all}-1.90	$SHGC_{all}$-NR
	2.1-5.0%	U_{all}-1.30	$SHGC_{all}$-0.34	U_{all}-1.30	$SHGC_{all}$-0.27	U_{all}-1.90	$SHGC_{all}$-NR
Skylight without Curb, All,% of Roof							
	0-2.0%	U_{all}-0.69	$SHGC_{all}$-0.49	U_{all}-0.58	$SHGC_{all}$-0.36	U_{all}-1.36	$SHGC_{all}$-NR
	2.1-5.0%	U_{all}-0.69	$SHGC_{all}$-0.39	U_{all}-0.58	$SHGC_{all}$-0.19	U_{all}-1.36	$SHGC_{all}$-NR
a	Exception to A3.1.3.1 applies.						