Andrew Simone

Æ Structural

The Pennsylvania State University

THE hub on Chestnut

PHILADELPHIA, PA

Senior Thesis 2007

FACULTY ADVISOR

Dr. Ali Memari

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SENIOR THESIS 2007

THE hub

Andrew Simone

The Pennsylvania State University
AE Structural Option

Philadelphia, PA

Project Overview
Nine Levels (Above-Grade)
110 Apartment Units
3 Levels of Retail
68,000 SF (Residential)
30,000 SF (Commercial)
$22.3 Million
Design-Build Delivery
June 2005 - December 2006

Structural
Caisson Foundation Support System
Post-Tensioned Concrete Floor Slabs
Steel Framed Stair/Elevator Shafts
Cast-in-Place Concrete
  5,000 PSI Columns/Slabs
  3,000 PSI Caissons/Foundation
Structural Metal Stud Wall Partitions

Project Team
Owner: Teres Holdings, LLC
Architects: Piatt Associates
  Brawer & Hauptman
CM/GC Agent: Domus, Inc.
Structural: O'Donnel & Naccarato
Civil: Vollmer Associates
MEP: AKF Engineers, LLC

Mechanical
Individual Heat Pumps
  with Fan Coils (208V/1PH)
Roof-Mounted Cooling Tower
  with Two Boilers
Water Source Heat Pump

Architecture
Mixed-Use Occupancy
Studio/Multi-Room Style Units
Double Height Retail Areas
Exposed Concrete Finishes
Aluminum Rainscreen System
  with Corrugated Metal and
  Wood Veneer Panels
EPDM Roof System

Lighting/Electrical
Main System:
  277/480 Volt, 3 Phase, 4W
  3,000 AMP Bus
Tenant System:
  120/208 Volt. 3 Phase, 4W
  4,000 AMP Bus
1000 KVA Transformer

Andrew Simone
Structural Option
Dr. Ali Memari, P.E.
The Hub on Chestnut
Philadelphia, PA
Senior Thesis 2007

EXECUTIVE SUMMARY

The following composition is the final product of a year long capstone project that is conducted by the Department of Architectural Engineering. Development begins by a 5-year AE student to select an existing building, to his/her interest, and use this structure as a model for technical research in accordance to their discipline. In the fall semester, each student analyzes and evaluates the existing design and submits three (3) technical assignments of the investigated material. A concluding report, based on the semester research, is submitted to the department with the students proposed redesign. The spring semester is dedicated to the individual developing the new design using an alternate system or by improving the existing conditions.

Within this report is the comparison of two structural designs. A newly constructed building, The HUB on Chestnut, was selected to provide a model and an existing structure. The existing structure is a flat plate, post-tensioned floor system support by a moment resisting concrete frame. The selected new design is precast hollow-core concrete slabs implemented on a steel girders supported by steel concentrically braced frames. Along with this design are two breadth topics that would help enhance the new structure. The first breadth is the application of a green roof and a recyclable gray water system. Second, is a cost and schedule comparison between the two structural systems.

The newly designed structure has proving to provide a much lighter structure as well as a more cost effective project. Load cases illustrated that the design was controlled by seismic loading over wind and the lateral system's members where sized based on LRFD strength design. All members were checked for strength as well as serviceability. Initial loading criteria's were provided to select effective member selection and avoid tedious and iterative applications. The RAM Structural System was used to aid analysis and provide redundant calculations.

PROJECT DESCRIPTION

EXISTING BUILDING AND CONDITIONS

The HUB on Chestnut is a mid-rise, mixed-use structure that has recently been completed in the University City section of Philadelphia, Pennsylvania. It is located at the northeast corner of Chestnut and 40th Streets. The building is predominantly a concrete structure that stands 9-levels with one sub-grade level covering a footprint of approximately 11,000 square-feet. The North/South width extends sixty-eight feet along Chestnut Street and the East/West length of the building extends one-hundred forty-eight feet down 40th Street. Construction began in the early summer of 2005 and the structure topped-out in the late summer of 2006. The HUB began accommodating occupants in the final weeks of 2006.

The Hub on Chestnut is designed to accommodate a mix-use of occupants. The building's main focus is to house University of Pennsylvania students in 110 units distributed over seven (7) above-grade levels. Units are also available to the public. A variety of different sized units are designed for single to multiple parties. The lower three (3) levels will provide commercial spaces directed to mercantile and retail clients. The residential space is approximately 68,000 square-feet and 30,000 square-feet are for commercial use. The street-level commercial space provides a large 40 foot oval opening in the floor to provide a double heighted space in the retail areas. The 9-level structure stands at a height of 100 feet above grade. The building footprint covers 10,500 square feet of the NE corner of 40th and Chestnut Streets. The street and second levels will have a floor-to-floor height of approximately 15 feet, while the above levels will be at 10 feet. All of the egresses are strategically placed to accommodate each occupancy type while isolating the commercial spaces from the residential. The north mid-section of the structure will house twin elevator shaft with a stair egress to serve the commercial spaces while a private elevator will serve the residents located in the south section of the building. The west end will provide an alternate stair egress and a central stair egress is placed to serve levels 3-9.

Interior spaces reveal urban and modern design schemes. An open HVAC system allows for an exposed mechanical system. All concrete is to be finished and be presented as the finished surface. Finished floors and ceilings are to be sanded and sealed concrete. The building is located in the University City section of Philadelphia. The prominent property owners of this area consist of The University of Pennsylvania and Drexel University. University City's community is a cultural area which exhibits a mix of architecture between historical, nostalgic buildings, and unique modern structures. The Hub on Chestnut will be the most noticeable structure in the community. The building envelope is complied of large prefabricated panels, a high-tech rain screen system, which is cladded with corrugated metal sheets, aluminum flat green panels, and multi-colored wood veneer panels. The roof envelope is an EPDM system to resist the elements and thermal resistance. Along with the vibrant cladding are three terrace levels located on Levels 3, 5, 6. A sequence of development includes the building itself and a series of renovations to create a complete promenade.

PRIMARY PROJECT TEAM

Owner: Teres Holdings, LLC

Construction Manager: Domus, Inc

General Contractor: Domus, Inc

Design Architect: Piatt Associates, Inc

Project Architect: Brawer & Hauptman Architects

Structural Engineer: O'Donnell & Naccarato

Civil Engineer: Barton & Martin Engineers (Vollmer Associates)

Geotechnical Engineer: McClymont & Rak

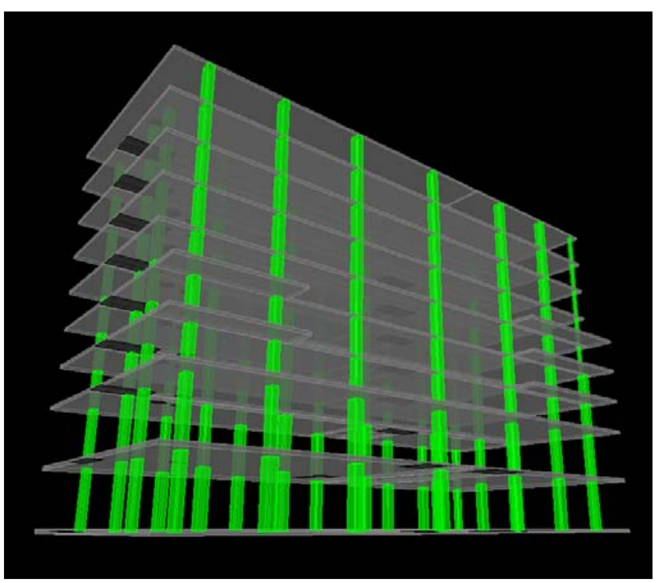
MEP Engineer: AKF Engineers

EXISTING STRUCTURAL SYSTEM

OVERVIEW

The basic structural system is classified as a 'moment resisting frame' consisting of post-tensioned, two-way flat slabs, and rectangular concrete columns. The superstructure is a system of exterior and interior concrete columns that support a concrete slab throughout each level. The structure is comprised of large 20"x 30" and 30"x 30" concrete columns that are monolithically poured with the flat slabs. The commercial space is designed using a thicker 12" two-way slab and the residential levels use a 9" post-tensioned slab with a mixed use of rectangular and round columns. A typical bay is sized as 25' x 25'. The HUB is designed with an ordinary concrete moment frame system, without the use of any shear walls or cross lateral bracing, to resists both vertical and lateral loads. The reinforced concrete, column size, and geometry of the structure all work in unison to resist the effects of lateral and gravity loading conditions. The foundation system is comprised of concrete caissons and spread footings. The sub-grade level perimeter consists of 12" thick cast-in-place concrete. The design of the building displays a sense of repetitive geometry and redundancy which allows for direct structural analysis and uniform performance by the structural elements throughout.

All gravity loads are taken from the applicable codes. The International Building Code 2003 was the main document used in designing The HUB on Chestnut. Lateral loading was also used in conjunction with the IBC 2003 Code to design resistance to wind and seismic forces.



FOUNDATION

The main foundation system is a grid of straight shaft caissons varying in size from 3'-6" to 4'-6" in diameter. All Caissons are constructed using a compressive strength of 3000 *PSI* concrete and bearing on undisturbed rock. The interior and exterior concrete columns are directly supported by caissons. All exterior walls are cast-in-place concrete placed on top of soil capable of supporting a load of 3000 *PSF*. A keyway system is oriented into the footing to resist lateral movement from the surrounding earth. The building footprint is classified as type *D* soil. Masonry walls, which are placed below grade, are constructed of Type N-1, ASTM C90 hollow grouted solid masonry units. All mortar is Type S, ASTM C270 with a minimum compressive strength of 1800 *PSI* after 28 days. Vertical reinforcement members of the masonry units are spaced at 16 inches on center. A 4" concrete slab-on-grade with 4" of crushed stone base and perforated pipe under drain system is placed at the lowest elevation of the structure. Finished floor elevation is 73.30' above sea-level. Also inlayed, is 6 x 6 welded wire fabric with a 8 mil vapor barrier.

COLUMNS

The main structural supports of the building are designed using three column lines forming six bays along each. Although the bays and column lines are unequally spaced throughout, the typical geometry is 25' x 25'. The columns are placed directly over one another from level to level to provide a stacked effect for transferring loads. At each level the columns are spliced by lapping the protruding rebar from the lower level to the newly formed column above. All columns are constructed of reinforced concrete having a minimum compressive strength of 5000 *PSI* after 28 days. The columns located on the lower levels are sized 30" x 30" while the upper floors (3-9) are sized 20" x 30". All reinforcement uses a #3 bar spaced twelve inches on center with varying rebar ranging from #7 to #10 bar.

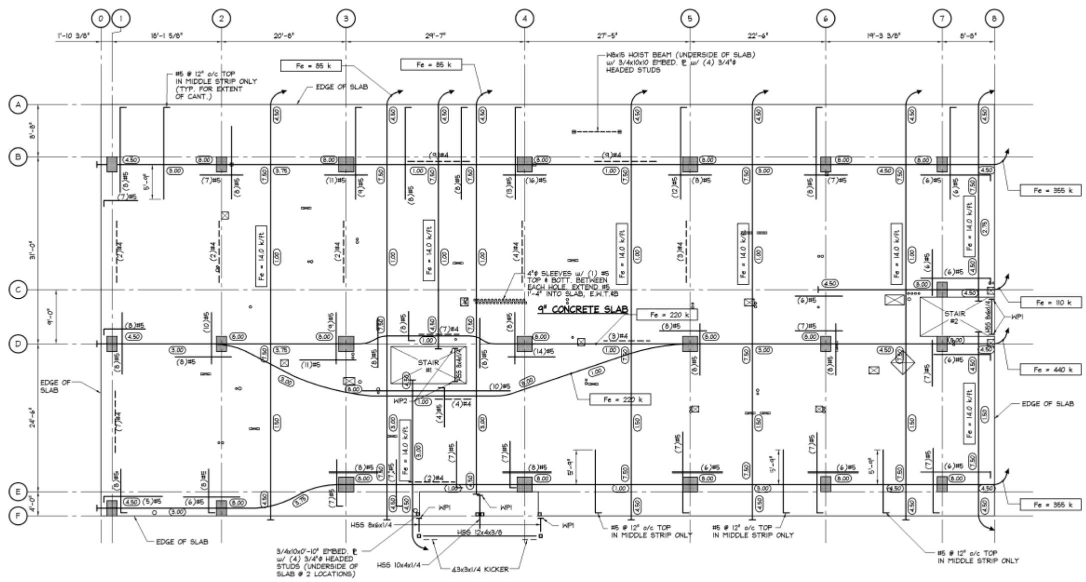
TWO-WAY SLABS

The ground level and second level are assigned to be flat two-way slab systems. These two (2) slabs located in the commercial space are at a depth of 12" compared to the 9" slabs located above in the structure. It is primarily reinforced in two directions using #6 rebar spaced sixteen-inches on center with additional rebar added in regions of needed higher strength. A large elliptical opening is placed on the ground level and the surrounding slab system is high reinforced.

The slabs are also highly reinforced around the support columns. No detailing of edge beams or dropped panels are integrated into the floor system.

POST-TENSIONED SLABS

All elevated slabs from level three to the roof are strengthened using post-tensioning. The process involves shoring the under layer of the slab, placing the conduits and tendons in accordance with its structural design, and then placing the concrete over the conduit layout. After the concrete has a reached a sustained strength, jacks or rams, are used to pull the tendons allowing the slab to carry the designed load. All tendons are designed to be ½ "Ø, 270 *KSI*, greased, and manufactured in a plastic sheath. Three main conduits are placed along each of the column lines. The two exterior tendon lines are symmetric in profile and in jacking force while the interior tendon line is ran around the central stair way and detailed with a much higher jacking force. The interior tendon profile also has an additional strand with a lesser post-tensioned force to accommodate the center stairway access.

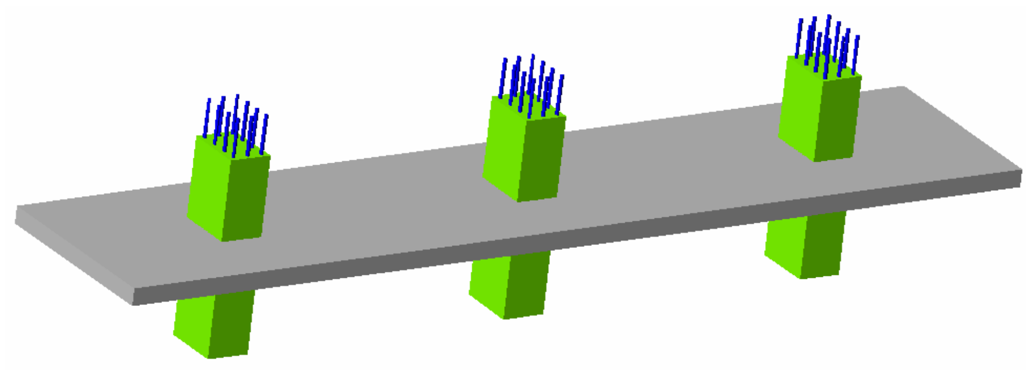


STEEL

The HUB has a predominantly concrete structure but does incorporate steel into the structural design. Located within the stairways and the elevator shafts are steel framing systems. A typical frame consists of several shapes. All wide-flanges are Gr 50 ASTM A992/A572, hollow rectangular/square steel Gr 50 ASTM A500 with yield strength of 46 *KSI*. All other steel members are ASTM A36 UNO. After fabrication, the steel was coated with a rust inhibitive paint and later the steel was to be sprayed with a layer of fibrous fireproofing material.

LATERAL SYSTEM

The lateral system is classified as a concrete moment resisting frame consisting of post-tensioned, two-way slabs and concrete columns. All floors are supported by twenty-two reinforced concrete columns. A typical bay sizing is 25' x 25'. Levels 4 through the roof are comprised of 20"x 30" concrete columns. The lower levels are supported by seven 20"x 30" columns and fifteen 30"x 30" columns. The typical reinforcing consists of (10-14) vertical bars ranging from #7- #10 in size. All ties are made up of #3 bars spaced at 12" on-center. All concrete columns and slabs are poured with high strength 5000 PSI concrete.



A monolithically poured slab and column is formed to create a simple moment connection. The design is based on this type of construction. No integration of shear walls or braced frames are used to distribute any of the lateral loads. The moment frames are designed to absorb, or resist, any and all forces that are distributed through the slab and the column itself.

In common design, the dimensioned columns are considered to be very large. The columns are uniform in dimension but are different in reinforcement throughout the structure. Therefore, as the elevation of the building increases the need for larger reinforcement decreases. This effect is caused by a lesser axial loading. The original designer may have considered several factors in selecting the uniform column sizes. First, it may be a request of the architectural. This aesthetic detail could be the main factor. Second, by choosing a uniform dimension construction can be greatly increased. The use of common concrete forms can greatly increase productivity and have less chance of error from story to story. Lastly, the current price of steel is significantly higher compared to concrete. The larger dimensions can add greater strengths to the concrete which will decrease reinforcement, hence lowering costs.

PROPOSAL & PROBLEM STATEMENT

PROPOSAL

In Technical Assignment 2, several floor systems had been designed to support the required loading conditions of The HUB on Chestnut. The hollow-core slab was chosen as the superior system over the others. Although post-tensioning is a very efficient design, the new system covers many issues that can be comparable to the original. Mainly, a concrete structure will be contrasted to a steel structure. Also, the lateral resistance system in each building can be evaluated and compared. The new design will stress the needs and requests of the owner as well as the requirements and desires of the architect.

Two breath topics are to be researched and incorporated into the building's overall design. A revamped mechanical system and issues relating to construction management will be oriented into design. The mechanical system will focus on two main feature; an application of a green roof and the collection of rainwater to be used as 'gray water' in the plumbing system. An investigation of material cost, scheduling, and project delivery will allow for comparison with the original plan. These two areas of research have been selected to work in conjunction with the proposed structural redesign to develop a new building package.

PROBLEM STATEMENT

After an in depth evaluation of the preliminary service conditions, floor system application, and lateral resisting system, it was evident that the initial design was very efficient and well designed. The architectural layout was very proficient in utilizing the spaces to accommodate each occupancy type. The two-way slab system provided an open floor plan that allows for the partitions to be placed at the owner and architects' desire. Post-tensioning was also very effective in the design. The University City section prohibits high-rise buildings to be constructed due to zoning regulations. Typical buildings range from low to mid-rise. The post-tensioned slab systems contributed to thin slabs which is can allow for an added level which increases occupancy. A moment frame lateral system eliminates the use braced frames and shear walls. These elements can obstruct the focus on maintaining a flexible floor plan. With the three structural features stated above, The HUB appears to be a justified design and stresses the needs that the owner wanted to implement into the building.

However, there are areas that can be enhanced. The use of reinforced concrete can be a very problematic material to work with. The reinforcement must be placed with care and accuracy to allow for proper serviceability. When the concrete is being placed around the reinforcement there are many places for failure to occur. The rebar may move affecting cover which can initiate spalling and voids may form hindering the bonding integrity. Constructing rebar cages and detailing rebar is very labor intensive and time consuming which can prolong the project duration. The main problem is the setting time that is needed to allow for the concrete to reach strength that is adequate for construction to continue. Post-tensioning further increases this delay because of the extra strength needed in concrete to avoid cracking, camber, and the possibility of strands 'blowing out'. A two-way slab floor system also involves tedious placement of multiple bars and the extra time needed in curing the concrete.

A moment frame can significantly increase the overall cost of a project. In concrete structures, the over sizing of columns to counter the effects of high moments can greatly increase the amount of concrete needed to complete the building. An increase in the amount of concrete that is used in the project will increase construction duration, the amount of traffic due to incoming and out going delivery trucks, and the most critical issue, cost.

OJECTIVES & GOALS

- To maintain the existing interior and exterior architectural design
- Meet the needs and requests of the owner and of the design architect
- Increase daily production and reduce the project duration
- Compare the superstructure of a steel vs. concrete frame system
- Reduce the overall weight of the structure to minimize the effective loading of a seismic occurrence

PROBLEM SOLUTION

The objective of the proposed building structure is to provide an alternate design to improve and redevelop the issues that were introduced in the problem statement. The proposed structure will comprise of a steel column and girder beam frame with a precast hollow-core concrete slab floor system. A composite action design between the beam and will be considered to achieve a shallow beam depth. A lateral system constructed of lateral braced frames will be designed to provide adequate resistance to shear and overturning moment induced by wind and seismic loading. The main ideas in selecting this design are constructability, loading, and serviceability.

As previously stated, concrete is a problematic material to work with. The use of steel and pre-cast concrete has many benefits over cast-in-place concrete. Constructability of this new system is the main advantage over the initial design. Concrete construction involves many steps compared to the erection of steel. The assembly of formwork, wiring rebar, detailing the steel, and placing the actual concrete is more time consuming than the picking up of steel and making connections. The steel is prefabricated and delivered to site. A small crew of men can erect several levels of steel in the time period needed to place and set concrete.

In both structures, the gravity loads are equivalent, with the exception of self weight, because they both provide identical occupancies. The building consists of commercial and residential spaces with the residential occupancy being 75% of total floor space. The residential live load is substantially less than the required live load in the commercial space. Therefore the pre-cast concrete elements can perform more efficiently with the long spans that are desired for an open floor plan.

In selecting precast, the concrete can be caste and cured in a controlled environment to provide maximum serviceability. The hollow-core slabs are reinforced with high strength tendons to provide adequate support under loading. When they are prefabricated the tendon placement can be placed with great precision to assure maximum performance. The compressive strength of cast-in-place concrete can be decreased because the weather conditions and temperature in the winter months can have negative effects in the chemical bonding of the mix.

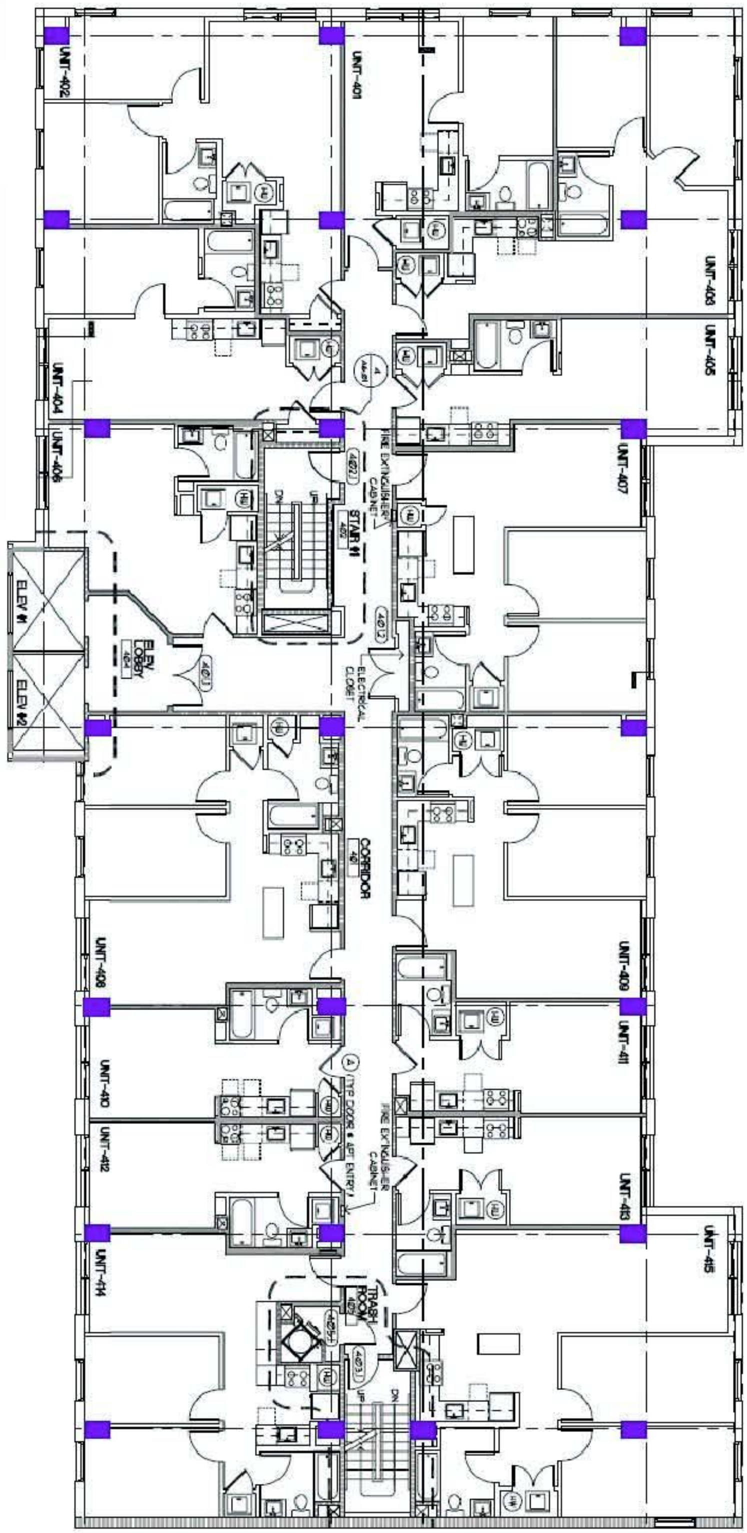
STRUCTURAL REDESIGN

OVERVIEW

The HUB on Chestnut has been redesigned as a steel framed structure supporting a precast floor system. The gravity system consists of precast hollow-core slab planks supported by steel girders that are orientated along three column lines. To increase strength, and reduce girder depth, the connections of the planks have been incorporated with shear studs to form a composite action design. All gravity columns are oriented to support the girder beams and maintain the building's geometry. All columns are continuously vertical and are supported by caisson foundations. Within the column lines are braced frames to function as lateral support against wind and seismic forces. A more thorough description of each system will be presented in the successive sections as well as in the Appendices.

My main focus in converting the structural system of The HUB, was to retain the interior design. The majority of the occupancy is dedicated to apartment units, which in turn produces an abundance of partitions. The existing system provided few columns and cantilevered slabs that provided open space for partitions and exterior wall openings. This design also had some negative effects. The moment frame system called for large columns to be designed to resist the effects of loading. The 20"x 30" and 30"x 30" concrete columns produced large masses that interrupted living spaces and could not be hidden within the partition walls. The redesigned system will allow the slender and compact W-shapes to be easily enclosed and hidden within partitions. The structural braced frames have been strategically placed among the columns and girders to preserve the partition layout as well.

The interior designs of the commercial spaces were also addressed in selecting the new structural system. In consideration of the owner and architect, the superstructure had to agree with the finishes. To impose the urban-modern scheme, the lateral system will be exposed and detailed for aesthetics. The braced frame system provides larger columns on these spaces. The cross bracing will include Chevron and X bracing constructed of HSS steel. All structural steel will be exposed and coated with a fireproofing material that is available in a variety of colors. The material is known as intumescent paint. When exposed to high temperatures the material expands forming a protective layer around the steel member. The following pages represent the existing structure overlaid by a transparency to compare the placement of structural members. The images designated a typical upper floor (Level 4) and the exterior North/South walls. Unfortunately, the window layouts on the North end undergo small design adjustments.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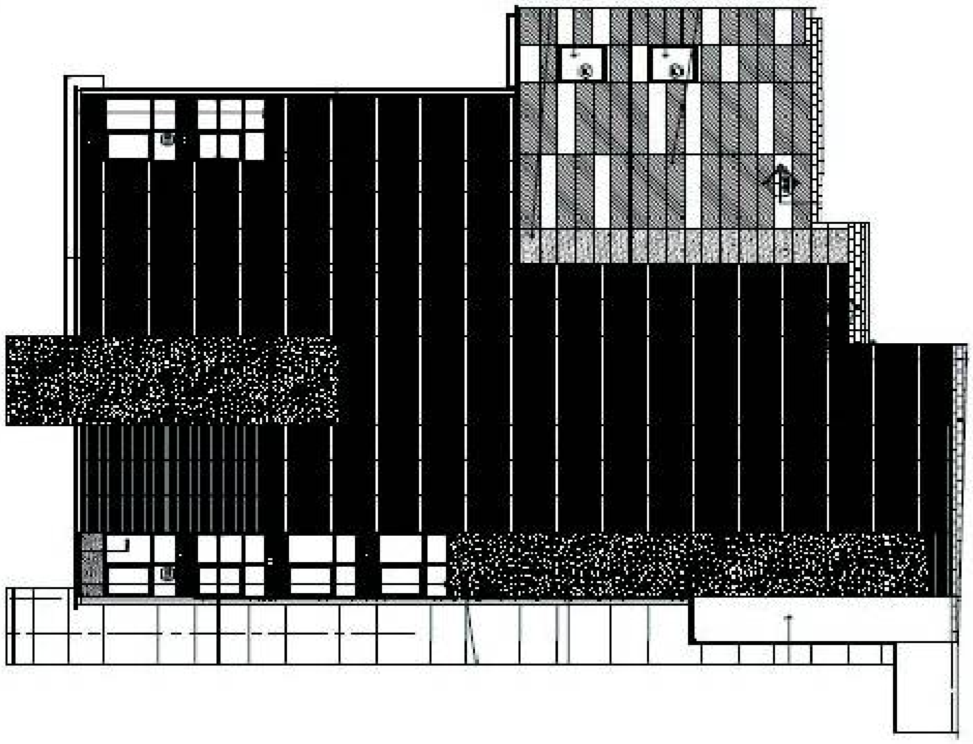
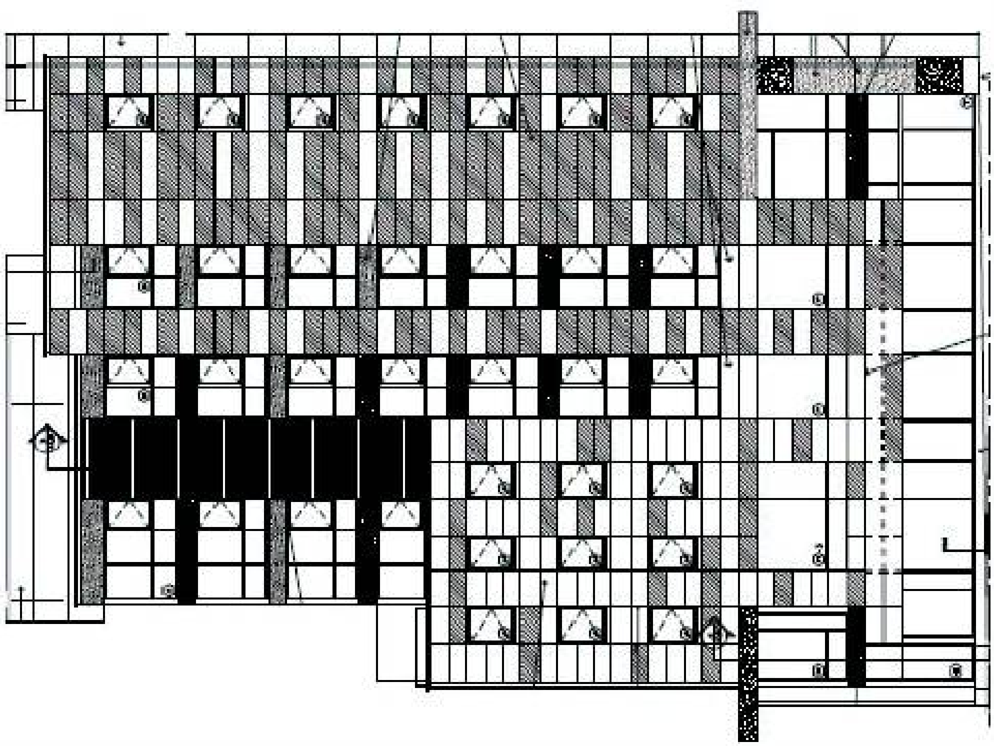
UNIT-415
STAIR #1
ELEV #1
ELEV #2
ELEV LOBBY
ELECTRICAL CLOSET
FIRE EXTINGUISHER CABINET
CORRIDOR
TYP DOOR & APT ENTRY
TRASH ROOM
UP
DN





CODES

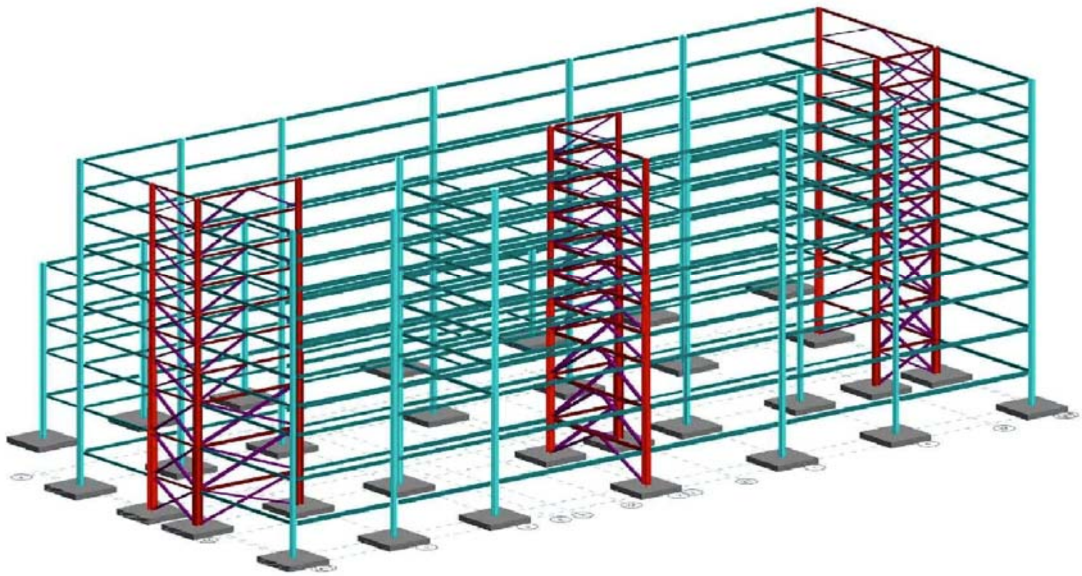
The International Building Code 2003 Edition (IBC 2003) was the main document that conducted the design of the existing structure. To account for the time that has since passed, and to familiarize myself with current design codes I have selected to use the most recent edition, International Building Code 2006 (IBC 2006), as a guide to redesign The HUB on Chestnut. Another leading code is the American Society of Civil Engineering Section 7, 2005 (SCE7-05). This code is continuously referenced in the IBC 2006 design sections. All steel members have been designed in accordance with the Manual of Steel Construction 2003 produced by AISC, using the Load and Resistance Factor Design method (LRFD). Other applicable industry codes include the PCI Handbook 2006 and the ACI 318-05. The goal of using these documents is to assure strength, serviceability, and analysis procedures.

All members and systems have been designed in accordance with the code documents stated above. The loading cases have been performed for both wind and seismic forces along with implementing all applicable loading combination to determine member strength. Please reference Appendix I for more information. The valid combinations are listed below.

IBC 2003 Edition	$1.4(D + F)$
	$1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$
	$1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
	$1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
	$1.2D + 1.0E + L + 0.2S$
	$0.9D + 1.6W + 1.6H$
	$0.9D + 1.0E + 1.6H$

RAM STRUCTURAL SYSTEM 11.0

A computer design program, known as RAM, was used to assist the redesign of The HUB. The parameters of this program allow the designer to construct a 3-dimensional model of the structure and generate data that can be used to design multiple members' strength and building serviceability. The program's software was updated to provide output that was compared to hand-calculated data based on current code editions. Although most programs can be referred to as a 'Black-Box', the output from RAM was very accurate to the calculation that had been produced by hand. This action verified that the design parameters within the program were acceptable. A 3-D model of The HUB is illustrated below.



GRAVITY SYSTEM

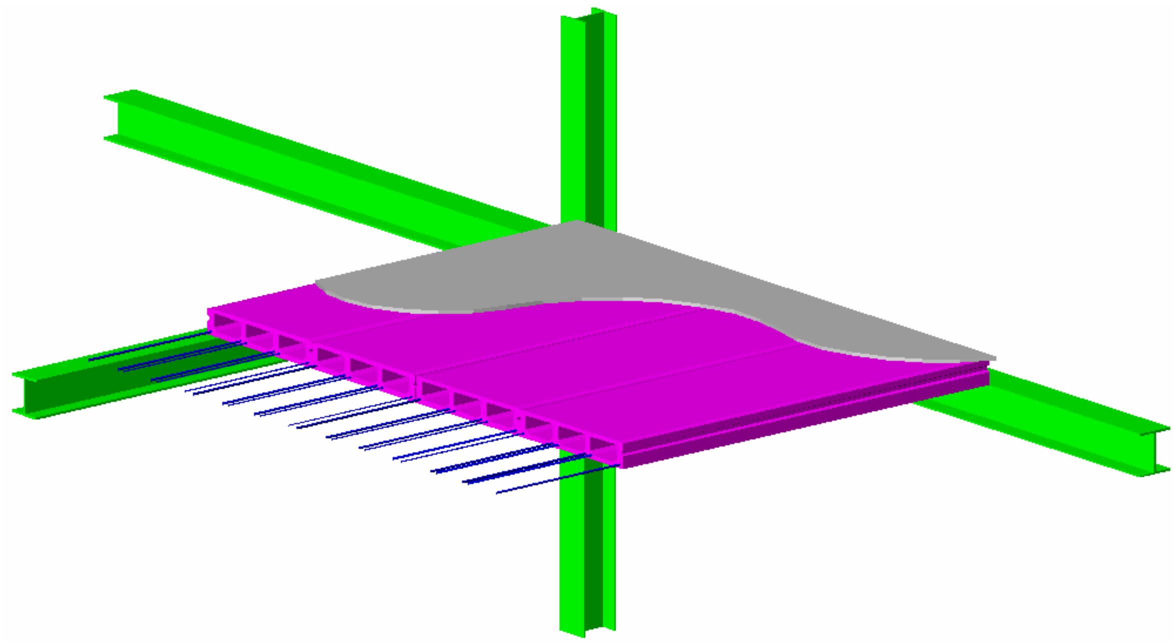
The selected floor system is precast hollow-core slabs. In general, precast slabs have several advantages. The most beneficial use of precast is the quick and steady installation. The product can arrive on-site and be put into place. No down time is required for concrete to be finished and set. The concrete planks, although hollow, can provide an adequate rigidity to resist lateral forces. Precast products provide the consumer with a quality product that is fabricated in a controlled working environment and can be installed in all weather conditions. Hand calculations were performed on the selected planks considering Class U members (uncracked).

In Technical Assignment 2, I chose a particular floor design based on a points system that I had developed. The group of systems chosen, including the existing, was compared based on fundamental design criteria, construction restraints, architectural aesthetics, and economical costs. Each item was scored in a particular section and issued a point value between 1 and 6. The most desirable design was given a 1, the next feasible design was issued a 2, and so on. Each system was then ranked by ascending order. No two systems can share a common value. After the numbers were tallied, the floor system with the least amount of points proved to be the paramount design. The diagram on the next page provides the Evaluation Table and points awarded to each system that was selected as an alternate floor system.

System	*I*	*II*	*III*	*IV*	*V*	*VI*
Economic Cost	5	1	2	4	5	6
Floor Depth	1	3	2	4	5	6
Loading Capacities	2	4	6	5	3	1
Fire Proofing (Rating)	4	3	2	1	5	6
Design Flexibility	2	6	1	4	5	3
Mechinical Placement	3	1	2	5	4	6
Constructability	5	1	2	3	6	4
Installation	4	1	2	3	6	5
Time Elapse	5	1	3	4	6	2
Weather Conditions	5	1	3	4	6	2
Quality	4	1	3	5	6	2
Aesthetics	2	3	1	4	6	5
Maintenance	3	2	1	4	5	6
Total	45	28	30	50	68	54

TABLE KEY		
	I	Post-Tensioned Flat Slab
	II	Hollow-Core Concrete Slab
	III	Two-Way Flat Slab
	IV	Two-Way Flat Slab with Drop Panels
	V	One-Way Concrete Joists
	VI	Composite Steel Beam

The floor system devised will feature a proprietary product known as SpanDeck® and will be provided by Old Castle Precast. The precast members are 8" x 96" hollow-core slabs spanning in the East/West direction of the building. The planks will be reinforced using multiple tendon sizes based on span and superimposed loading. The longest span is 29'-6" which is ideal for precast members. Due to slab irregularities and the effects of camber in the product, a 2" topping of concrete will be added to all floors. All slabs are caste using a concrete compressive strength of 5,000 PSI with multiple 7-wire, 270K low relaxation tendons placed in the lower section to resist positive bending moments. The 2-inch topping will be placed with 3,000 PSI concrete and the grout will reach strength of 4,000 PSI after 28 days. As previously stated in the proposal, the slabs with be integrated with shear studs to produce a composite floor system between the steel girder and the two bearing planks. The shear studs are sized at ½Ø and at a length of 4-inches. The *Connections* section will provide more detail on this topic. It was feasible to consider a non-composite system, but would cause the girder beams' depths to increase by approximately 2-3 inches. Although I strategically placed the girders so that they will be hidden in partitions and a chase duct, the composite system allowed for one major detail. The composite action from the 2-inch concrete topping and the grouted shear studs provides the floor diaphragm to be considered rigid. A rigid diaphragm is defined as when it is able to distribute the horizontal forces, from wind and seismic loading, to the vertical lateral elements in proportion to their relative stiffness. To provide more adequate rigidity, at a less economically approach, the slabs can include grouted shear keys or perimeter reinforcement. This option was excluded.



LOADING

The HUB floor system was designed for each level beginning at the roof and continuing to the base of the structure. For this new design, several loading conditions were investigated. The proposed roof is to support a green roof application, Levels 9-3 are for residential occupancy, and the lower levels are to accommodate retail occupancy. Also to be considered is the intermediate terrace levels located on Levels 6, 5, and 3. The dead, live, and snow loads were taken from the IBC 2006, ASCE7-05, and from my own intuition. To accommodate unrecognized loads a collateral load was placed on each floor based on occupancy. Collateral loads may be induced by mechanical fixture, electrical components, and excessive masses. With the loading criteria below, an additional load schedule is provided in Appendix I.

- The green roof is comprised of insulation, waterproofing material, and media soil. The media density was sized based on a fully saturated state to account for maximum loading. Also, the entire roof was designed to accommodate 100% cover of the 'green' material. A live load was considered in accordance with IBC 2006 - 1607.11.2.3, which states a landscaped roof used for special purposes. A snow load was also calculated based on the Philadelphia location. The planks were sized using an 8" depth with a 2" covering to provide strength and the resistance to water penetration through cracks and interfaces.

DEAD		LIVE		SNOW
Media/Sedum	23.5 lb/ft^2	Landscaped Roof	20 lb/ft^2	18 lb/ft^2
Root Barrier	0.5 lb/ft^2			
Insulation	1 lb/ft^2			
HDPE 80	1 lb/ft^2			
Collateral	6 lb/ft^2			
	32 lb/ft^2			

- Levels 3-9 have been loaded based on the residential section, hotels and multifamily houses. The partitions are sized using two sides of ½ inch gypsum with a non structural light gauge metal stud spaced at 16" o.c. throughout each level. After talking with the manufacturer of the exterior rain screen wall system, the representative concluded that the total weight was approximately 20 PSF. The live load was uniformly distributed in all areas including corridors based on code specifications. All planks were sized using 8" depths with 2" covering. The tendon sizes are based on the span between supporting girders.

DEAD		LIVE	
Partitions	20 lb/ft2	Hotel/Multi-Family	40 lb/ft2
Collateral	8 lb/ft2		
	28 lb/ft2		

- The retail and mercantile areas were sized based on the finishes and live load distribution. The owner and architect call for an open-atmosphere and only minimum partitions will be added. Therefore a minimum dead load was considered. Level 2 is considered as an upper level of a retail store and is only loaded with 75% of the live load.

DEAD		LIVE	
Collateral	10 lb/ft^2	Retail (1st Floor)	100 lb/ft^2
		Retail (2nd Floor)	75 lb/ft^2

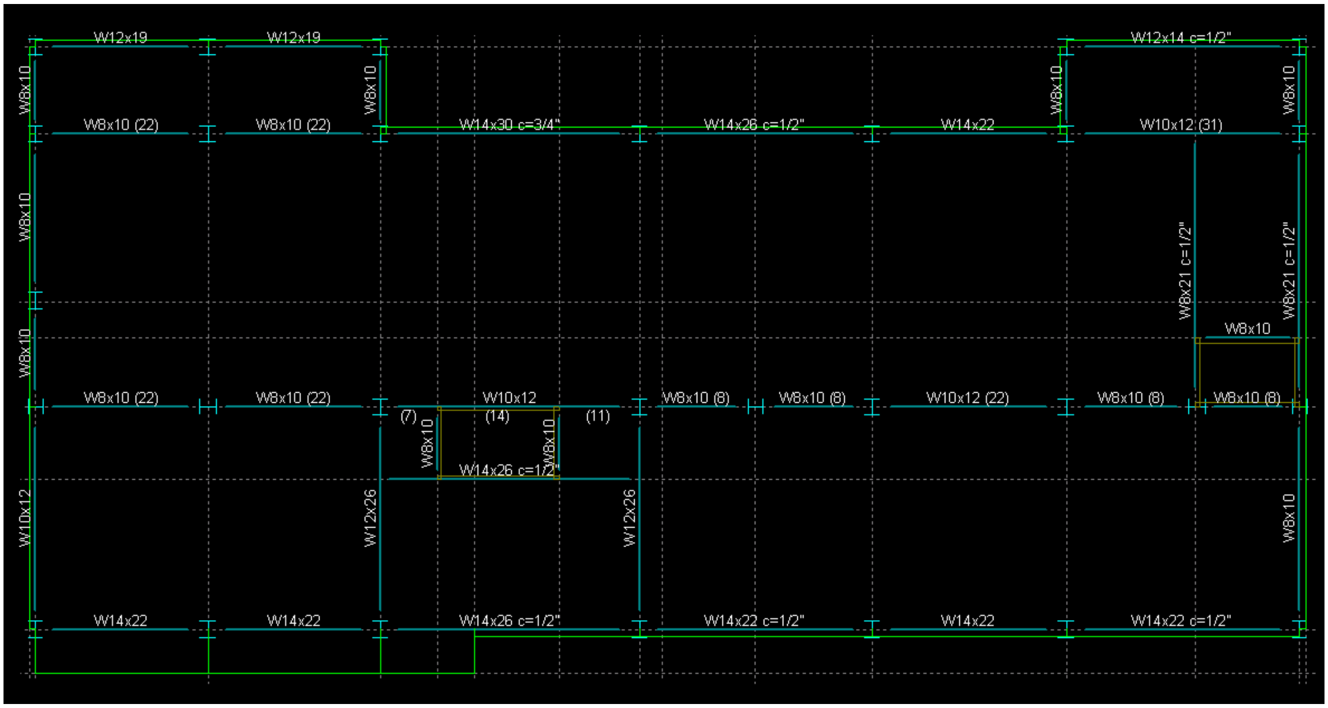
- The terrace areas have been supported by shorter slabs spans to accommodate the extra live loading specified by code. The terraces also have alternate dead loading requirements than the other occupancies.

DEAD		LIVE	
Insulation	1 lb/ft^2	Balconies	100 lb/ft^2
EPDM	3 lb/ft^2		
Partition	10 lb/ft^2		
Pavers	10 lb/ft^2		
Collateral	6 lb/ft^2		
	30 lb/ft^2		

BEAMS

The steel beams are considered to act as girders because they only support one-way uniformly loaded members. All beams are rolled from high-strength, low alloy ASTM A992 steel. The girders run perpendicular to the hollow-core planks along the three (3) continuous column lines that rise from the base to the roof. At the lower levels exterior girders are placed to support the projected areas on the North and South ends of the structure. All interior beams are designed with composite action, but the exterior beams are not. This was not needed to due to the rigidity of the shears studs of the interior and the inefficiency of placing additional grout at these locations. For serviceability requirements, I set a deflection limit on all beams, Live/360 and Total/240. These limits set the bar for initial beam sizing.

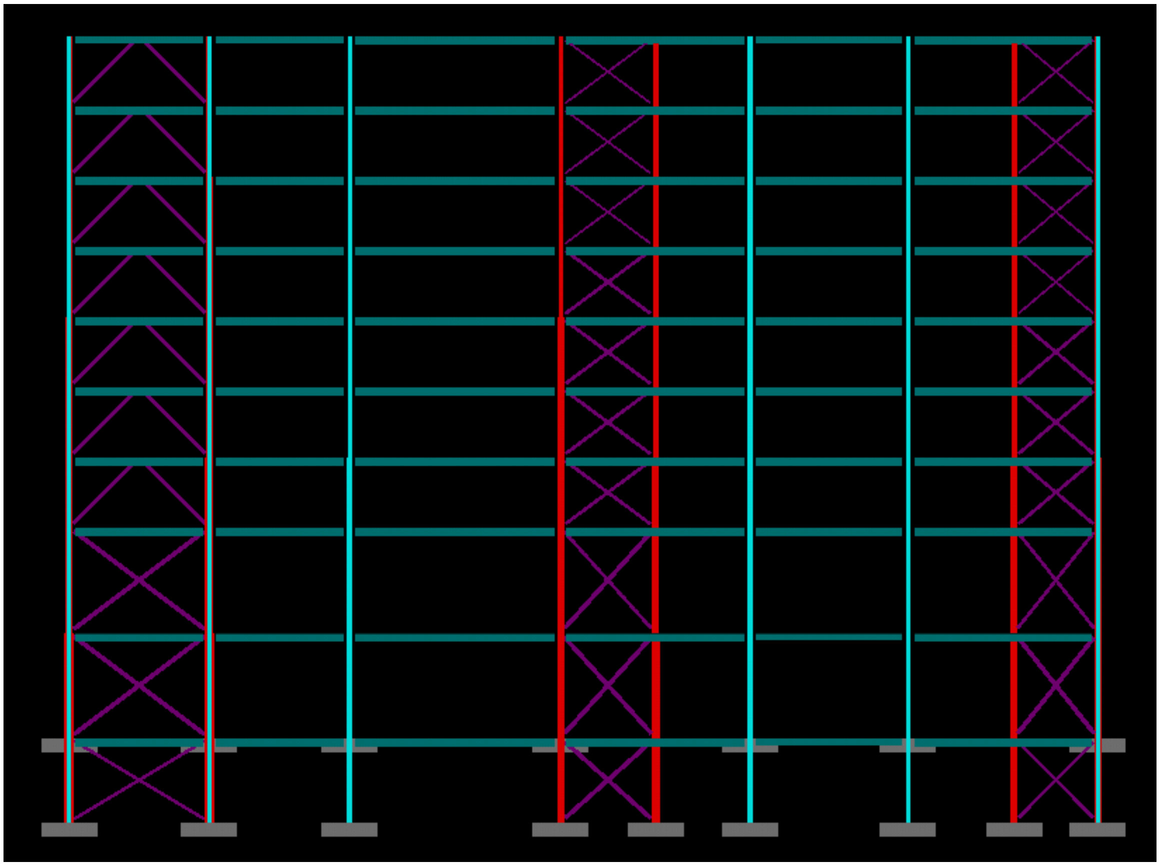
With the aid of RAM, three options were investigated; (1) non-composite (2) composite (3) size restriction. Option number one concluded that the beam depths were too deep and that windows, doors, and openings maybe altered. Option number 2 proved that the composite interaction provided much shallower beams. The third method chosen was placing a depth restriction of 14” on all beams. This was the beam design that proved most effective. As an example, a girder beam was selected from Level 4. The non-composite beam was sized as W14 x 26, as a composite beam the member became a W10 x 12 with (22) shear studs. The extra grout and studs provides a much more acceptable depth. A decrease of almost 4” was observed. The average girder beam is sized as a W12 x 16 with an upper limit of W14 x 22 and a lower limit of W8 x 12. Refer to the *Construction Management* section for a complete beam take-off, Appendix I. The following is an illustration of a typical floor represented by Level 4.



COLUMNS

The columns that are not integrated with the braced frames are considered to act as gravity columns. These members are rolled of high strength, low alloy ASTM A992 steel. All columns run continuously from the roof to the base. For an efficient erection sequence, all columns will carry two building stories and spliced approximately 3 feet above the second level. The axial loads were considered to be concentrically placed on each column. To accommodate the code provisions in the ASCE7-05, live load reductions, on both the roof and applicable floor areas, were implemented over a column's influenced area.

Initial calculations proved axial loads due to gravity were not substantial enough to greatly increase the column size as the elevation from the roof is decreased. A common column is sized as a W10 x 39. This column is oversized at the upper levels but was considered conservative because the AISC Steel Manual restricts its design strength tables to a minimum size of W10 shapes. For design purposes, all columns were considered to be braced in both the x-axis and y-axis. The girders and plate welded planks provided horizontal bracing at each intermediate level. An effective length value of 10' was used on upper levels, and 15' was issued on the lower levels. The largest gravity column was sized as a W10 x 49 at an interior bay. The *Construction Management* section, Appendix IV, will provide a take-off of all columns to provide the amount of steel used as compression members.



LATERAL SYSTEM

The lateral loads performed on The HUB were analyzed by standard practice and the guidelines recognized in the IBC 2006 along with ASCE 7-05. For both wind and seismic loading, the forces calculated were based on the size, geometry, type, and geologic location of the structure. The HUB on Chestnut does not exhibit any irregularities in geometry for both vertical and horizontal directions. The wind load analysis is performed on the main wind force-resisting system (MWFRS), which is guided by ASCE 7-05 Chapter 6 and the seismic loading is performed on the structural framing, which is guided by ASCE 7-05 Chapters 11-12. All applicable loading cases have been invested and the calculations demonstrate the building is controlled by the effects of a seismic occurrence. Available spreadsheets and calculations are provided in the following sections, as well as in Appendices I-III.

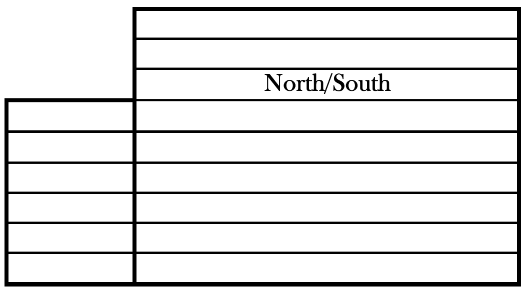
WIND

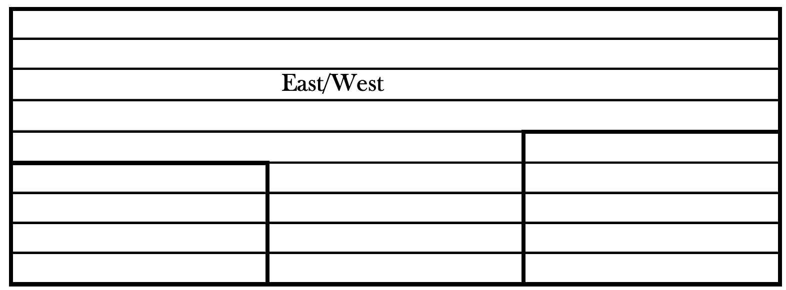
The HUB on Chestnut of Philadelphia is not located in a special wind region or subjected to hurricane winds. The applied wind velocity is valued at 90 mph and is distributed to both the North/South and East/West faces of the structure. Basic geometry includes two width values of 60' and 68' in the N/S face and a length of 148' in the E/W face. The HUB is classified as a mid-rise structure with a mean roof height of 100' above grade. The building is categorized with exposure B due to its urban setting and surrounding dwellings. No topographic adjustments have been included because the landscape is homogenous. Although, the building's glazing is not blast proof, I believe that the glazing is able to withstand most windborne debris therefore classifying the structure as fully enclosed. In the case of a 'breached' building envelope it is possible to increase the internal pressure by almost three times ($\pm 0.18 \rightarrow \pm 0.55$). The wind analysis will be designed considering a flexible structure because the lateral system is a braced frame and no moment connections have been implemented. With this assumption, the RAM structural model proved that the structure is considered flexible. Three (3) gust factors were calculated to determine more accurate pressures that will be exerted in each direction. Also, the four (4) wind loading cases provided in ASCE7-05 have been invested to find the controlling force orientation. A detailed and concise calculation is proved in Appendix II with other applicable information.

The analyzed data obtained has proven that the East-to-West wind direction is the most critical orientation because higher pressures are to be exerted on the structure. This conclusion is based on adding both the windward and leeward pressures and observing which produces higher result. Another observation is that when the interior pressure is negative the windward pressure is greatest. Contrary to this assumption, when the interior pressure is positive the leeward pressures are greatest. The calculated results have been summarized in the illustration and tables below.

North/South MWRS Forces

Level	Height (FT)	Tributary (FT)	Pressure	Height	Pressure	Height	Pressure	Height	Story Dist.	Cum Dist.	Story Shear	Cum Shear
Roof	100	5.00	21.07	5					105.36	1808.51	6.32	6.32
9	90	10.00	21.07	5	20.62	5			208.46	1703.15	12.51	18.83
8	80	10.00	20.62	5	20.12	5			203.73	1494.69	12.22	31.05
7	70	10.00	20.12	5	19.58	5			198.53	1290.96	11.91	42.96
6	60	10.00	19.58	5	18.73	5			191.58	1092.42	11.49	54.46
5	50	10.00	18.73	5	18.08	5			184.05	900.84	12.52	66.97
4	40	10.00	18.08	5	17.31	5			176.93	716.80	12.03	79.01
3	30	12.50	17.31	5	16.39	5	15.77	2.5	207.93	539.86	14.14	93.15
2	15	15.00	15.77	2.5	15.15	5	14.45	7.5	223.56	331.93	15.20	108.35
1	0	7.50	14.45	7.5					108.37	108.37	7.37	115.72





East/West MWRS Forces

Level	Height (FT)	Tributary (FT)	Pressure	Height	Pressure	Height	Pressure	Height	Story Dist.	Cum Dist.	Story Shear	Cum Shear
Roof	100	5.00	21.03	5					105.15	1862.53	15.56	15.56
9	90	10.00	21.03	5	20.65	5			208.41	1757.38	30.84	46.41
8	80	10.00	20.65	5	20.23	5			204.42	1548.96	30.25	76.66
7	70	10.00	20.23	5	19.77	5			200.03	1344.55	29.60	106.27
6	60	10.00	19.77	5	19.28	5			195.25	1144.52	28.90	135.16
5	50	10.00	19.28	5	18.71	5			189.94	949.28	28.11	163.27
4	40	10.00	18.71	5	18.06	5			183.85	759.34	27.21	190.48
3	30	12.50	18.06	5	17.27	5	16.73	2.5	218.48	575.49	32.34	222.82
2	15	15.00	16.73	2.5	16.21	5	15.61	7.5	239.95	357.01	35.51	258.33
1	0	7.50	15.61	7.5					117.06	117.06	17.32	275.65

SEISMIC

Seismic activity can be catastrophic to a building structure. A lateral load produced by an earthquake causes the structure to absorb a tremendous amount of force at its connections and distributes forces horizontally as well as vertical. To calculate the effects of seismic movement, The HUB on Chestnut has a framing system classified as ordinary steel concentrically braced frames. This denotes a response modification coefficient of 3¼ and an overstrength factor of 2. The framing system is placed in both directions to provide equal forces in the N/S and E/W axis of the building. After all preliminary criterion was gathered it was found to determine all seismic forces using the equivalent lateral force analysis procedure. From the data collected, seismic lateral loads are the controlling factor over wind. The HUB is not a very heavy structure, in regards to its gravity loads and self-weight, therefore it is less prone to damage from seismic activity. Although the building in not located in a very active seismic area, the structure must be designed to resist lateral movement in the event of an earthquake.

The approximate fundamental period of the structure was calculated in accordance with ASCE 7-05. The product of this value and the coefficient for upper limit produced a fundamental period of 1.036 seconds. This value was then inversed to determine the building's fundamental frequency, which was equal to 0.965 Hz (≤ 1.0 Hz). This value was confirmed by RAM Structural System and also validated the assumption of a flexible structure that was used in the wind analysis. The seismic calculations provide the effects of shear distribution through each floor as well as overturning. A more in depth analysis can be found in Appendix III and will provide more data.

North/South/East/West

Level	h_x(FT)	w_x(K)	$w_x h^k_x$	C_{vx}	F_x(K)	V_x(K)	M_x(FT-K)
Roof	100	1003	344565	0.211	73.96	73.96	739.59
9	90	952	286156	0.175	61.42	135.38	2093.39
8	80	952	246457	0.151	52.90	188.28	4715.78
7	70	952	208069	0.127	44.66	232.94	9878.17
6	60	994	178631	0.109	38.34	271.28	20139.76
5	50	1023	145911	0.089	31.32	302.60	40592.71
4	40	1023	109953	0.067	23.60	326.20	81421.42
3	30	1134	84638	0.052	18.17	344.37	164746.37
2	15	971	30095	0.018	6.46	350.83	329589.63
1	0	971	0	0	0	0	0
∑	100	9003	1634476	1	350.83		

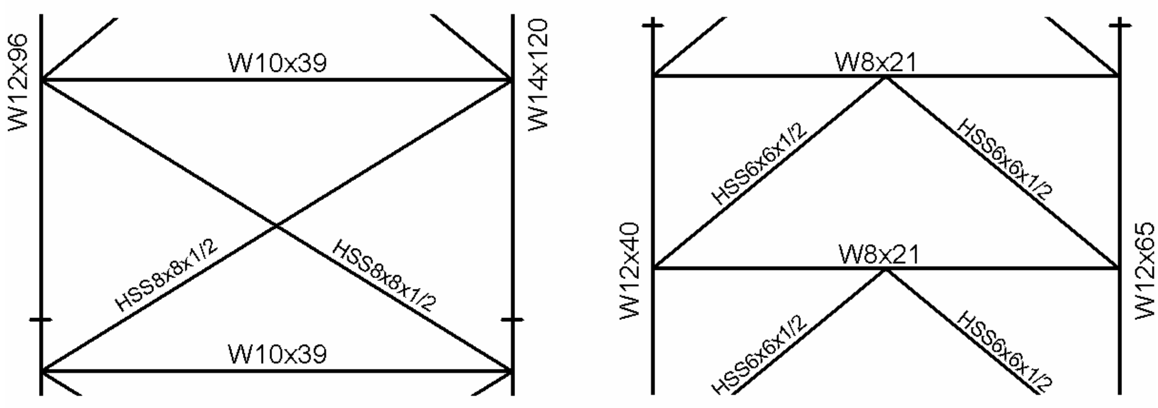
FRAMES

The HUB on Chestnut has been redesigned using the erection of concentrically braced frames to provide sufficient stability against lateral loads and maximize serviceability. It was found to be a very tedious and iterative process in selecting members and the geometrical orientation. The main objective was to maintain interior design as stated previously. In the final design, a combination of Chevron and X bracing was implemented to establish an efficient product. The complete design of all braced frames can be found in Appendix I.

In the first attempt I placed three (3) braced frames on the exterior perimeter of the building. This allowed for the interior spaces to not be disturbed. Unfortunately, the lack of rigidity in the structure forced column sizes to be steadily increased and maximized displacements. The window layout was also obstructed with this arrangement of frames. The next attempt provided two (2) exterior frames and three (3) interior frames placed in appropriate locations. This orientation satisfied both constructability requirements as well as member sizes based on strength. This attempt did lower column sizes but some members I felt could be downsized. In selecting members it was noticed that some shapes exceeded a slender ration. The overall displacement in the x-axis was not acceptable by my initial set criteria of H/400. The final attempt called for an additional frame located within the interior. After reviewing member sizes and story drifts I concluded that this was the most efficient design. The final braced frame system will be elaborated in the succeeding paragraphs.

The final framing system consists of six (6) braced frames. Each frame is designed based on an arrangement of Chevron and X braces constructed by W-shape columns, W-shape beams, and HSS-shape cross members. With my limited experience in selecting braces subjected to both compression and tensile axial loads, I feel that I have designed a very economical design.

The seismic calculations provided the horizontal force distribution to each floor which where then transferred to each frame. For each particular direction, the rigid floor diaphragms transfer loads to each frame that are arranged parallel to the axis that is being analyzed. Next, I chose the vertical and cross members that best resisted the forces based on member strength and stiffness. My initial instinct selected single and double angles to act as members in axial tension. This approach work well for the upper levels, but as forces increases down through the building they were becoming too large and failing due to shear. The attempt to reduce member sizes is to asses one of the main goals of this redesign, that is reproduce a more economical superstructure. The next approach was selecting hollow structural steel (HSS) shapes. These members worked very well in acting as tension members. The HSS sizes used in design are 8x8x½, 6x6x½, and 4x4x½. The use of square shapes provided symmetry in x-x and y-y axis for unbraced lengths. The upper levels, typically 7 to Roof, were assigned the smaller sizes while the lower to intermediate levels were braced using the larger members. An alternate design of using W-shapes as tension members may have been initiated but I felt that the HSS shapes would be more appealing as exposed steel on the lower levels.



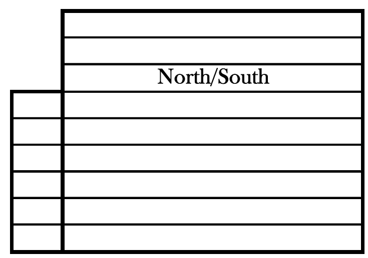
The frame designs also affected the lateral beams and columns. The lateral beams that were placed in X bracing sections were not modified because the truss effect from the braces only subjected them to flexure stresses from the floor system. However, beams that were placed within Chevron sections had to be modified. At the connection where the two members meet, an upward force caused the members to pass their yield point. For example, Frame 3 provides horizontal beam members that act as struts, not gravity beams. These members were first sized as W10x15 and had to be up-sized to W12x26.

The columns had the most effect on the frame designs. As the columns sizes increased so did the stiffness. All columns had first been sized by gravity loads. When the lateral loads had been issued most members began to fail. The columns on the upper levels were not affected much but the columns at the base needed to be greatly increased. On Level 2, the gravity loading called for a W10x49. When subjected as a lateral column the new size became a W14x109. This showed how significant these forces are on the building.

Located in ASCE 7-05, Chapter 12, is the allowable drift limits due to seismic loading. Analysis shows that seismic load effects are controlling in the North/South direction of the building. This is found by examining that the frames parallel to this direction are less stiff than the counter-axis. The story drift limit is set to allow any story to move 2% of the height of the story below from its center of mass. Accidental torsion moments were not considered on because The HUB is classified as Seismic Design Category B. For serviceability, I set a total building displacement of H/400. After a final design was completed the roof level was displaced at 4.16 inches. This represented criteria of approximately H/300. I accepted this value based on the fact that all members were sized due to strength design. To satisfy the initial limit most members would be well oversized.

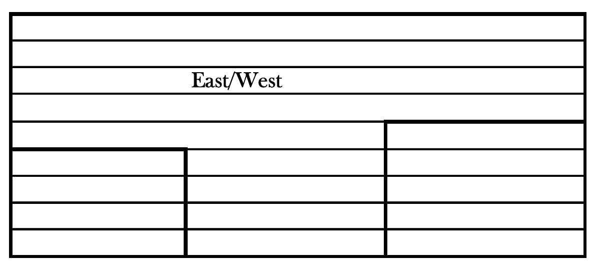
North/South *ASCE 7-05 [12.12.1]* $\Delta \leq \Delta a = 0.015 h_{sx}$

Level	Story Height Below (ft)	Displace (in)	Δ (in)	Δa (in)	
Roof	10	4.597	0.577	2.4	✓
9	10	4.02	0.594	2.4	✓
8	10	3.426	0.600	2.4	✓
7	10	2.826	0.563	2.4	✓
6	10	2.263	0.537	2.4	✓
5	10	1.726	0.490	2.4	✓
4	10	1.236	0.429	2.4	✓
3	15	0.807	0.494	3.6	✓
2	15	0.313	0.313	3.6	✓
1	0	0	0	0	



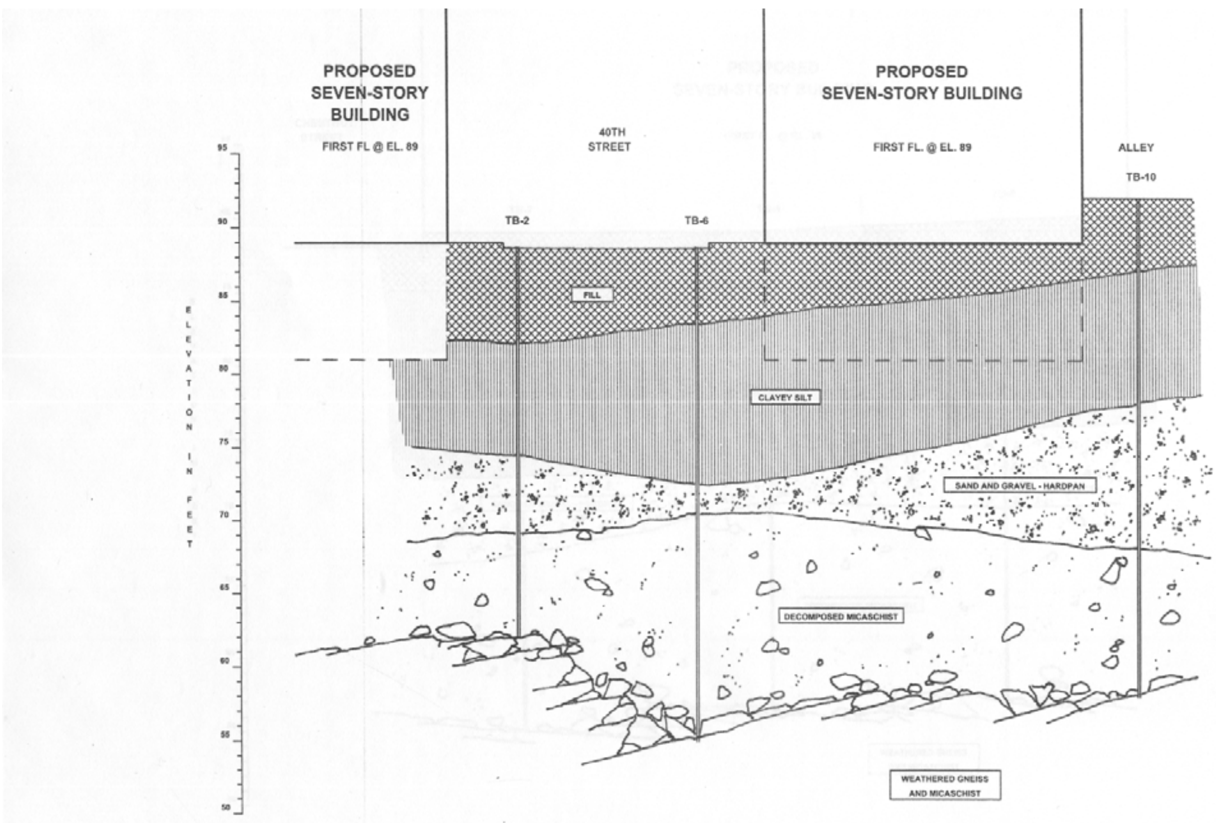
East/West *ASCE 7-05 [12.12.1]* $\Delta \leq \Delta a = 0.015 h_{sx}$

Level	Story Height Below (ft)	Displace (in)	Δ (in)	Δa (in)	
Roof	10	3.281	0.370	2.4	✓
9	10	2.911	0.399	2.4	✓
8	10	2.512	0.407	2.4	✓
7	10	2.105	0.367	2.4	✓
6	10	1.738	0.409	2.4	✓
5	10	1.329	0.346	2.4	✓
4	10	0.983	0.324	2.4	✓
3	15	0.659	0.386	3.6	✓
2	15	0.273	0.273	3.6	✓
1	0	0	0	0	



FOUNDATIONS

The geotechnical report provided the necessary information to assign footing to the steel columns. The report details that bedrock is located approximately 35 feet below five layers of soil (fill, clayey silt, sand, hardpan, and decomposed mica schist). Using the IBC 2006, the Site Class Definition is classified as D. This was also used in consideration for seismic analysis. The geotechnical engineer recommends drilled piers to support the 9-story structure. The original footings consisted of two concrete caissons sized at 3'-6 and 4'-6 diameters bearing on undisturbed rock. A typical caisson is constructed of 5,000 PSI concrete reinforced with (10) #10 vertical bars with a #4 circular tie spaced at 18 inches.



STRATIGRAPHIC CROSS-SECTION A-A

The new structure is does not weigh as much as the existing. This intern allows the footings to be much smaller. However, the depth of each caisson does need to bear on undisturbed rock. Most gravity columns exhibit an axial load of 25-45 kips. The most critical columns were the lateral columns. These members are designed as much higher W-shapes and reach axial loads of 600-1000 kips. The new footings have been designed to support the lateral columns located in frames 1-6. Reinforcement will include (10) #10 vertical bars with a #4 circular tie spaced at 18 inches.

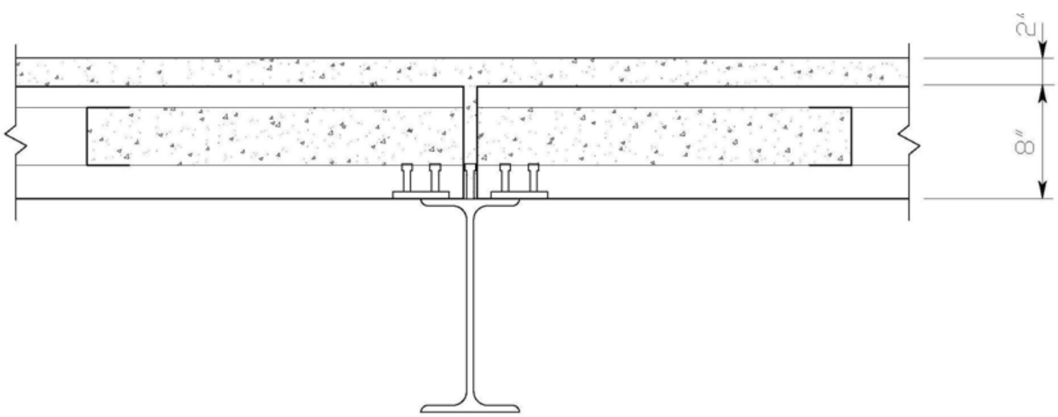
Caisson Design

Column	Axial Load (kips)	Caisson Ø	Depth (ft)*
1	607.65	36"	32'
2	689.37	36"	35'
3	937.44	42"	39'
4	452.21	36"	30'
5	864.13	42"	35'
6	686.13	36"	37'
7	328.72	36"	30'
8	459.73	36"	30'
9	393.28	36"	30'
All Gravity	< 500	36"	30'

**Depth is measured down vertical from grade level*

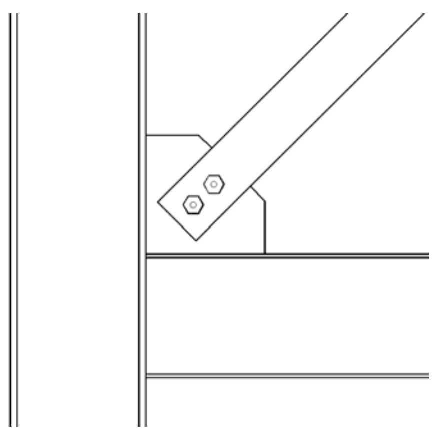
CONNECTIONS

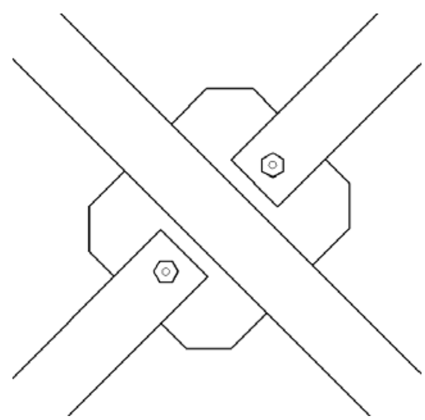
The redesign includes no moment or rigid connections. All connections are considered to be simple, or semi rigid at certain locations. Previous stated, the precast hollow-core slabs will be attached to the steel girders with a combination of grout and shear studs. Also, the blanks will be caste with a metal plate that will be field welded to the top flange of the W-shape.

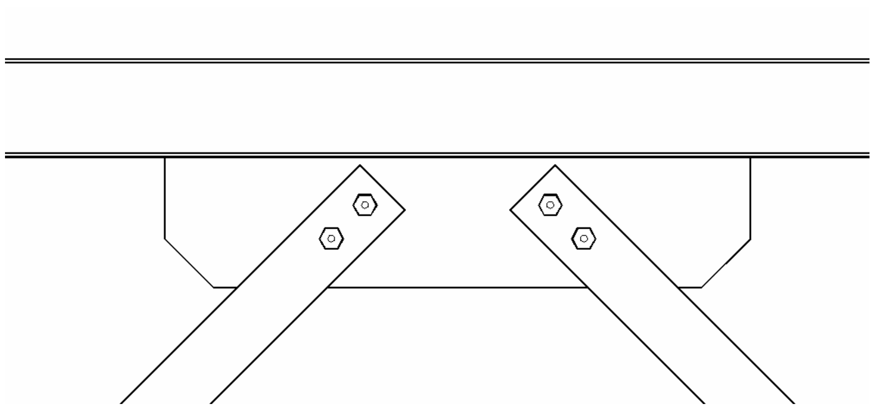


The steel girders will be connected to the vertical columns by a simple shear connection. The shear plates shall be bolted/bolted angle shapes attached to either the column flange or web based on orientation. The angles shall also have 1/8 inch tolerance to allow for easy erection. Typical holes patterns will include 3-4 holes based on the workable gauge and the beam depth to resist the effects of blockshear and tearout.

The braced frame connections are considered to be simple connections at all nodes of the vertical truss. To utilize maximum connection ability, most diagonal braces are placed at 39°-45° to the horizontal. The HSS members will be slotted to receive a steel plate that is attach to the horizontal W-shape member. For X-members, a steel plate will be attached to both sides of continuous diagonal HSS that can receive two short members at the node point. The following represents three typical connections located through the structure.







CONCLUSION

The preceding depth study has provided the investigated information to prove a well designed structure has been exhibited. The steel frame and precast slab combination offered an efficient design in both construction and delivery. The concentrically braced frames were constructed using HSS members as diagonal bracing. These braces provided adequate tensile loading to resist the effects of wind and seismic forces. Column sizes were limited to members with relative light weights to produce an economical structure. This criteria lead to adding an addition frame. However, the sixth frame provided more stiffness in the structure to accommodate a more serviceable drift limit. The initial limit was set to H/400. To satisfy this, members would have to be well oversized. I decided that sizing members based on strength was more proficient. The final building drift was approximately H/300.

In analyzing the existing structure last semester I concluded that the PT flat slap system was very effective. The only fault I would consider was the over massed columns that crowded simple apartments. The most effective use of the concrete moment frame was the ability to have an open space on the exterior. The use of braced frames and girder slabs did have a large effect on freedom to the exterior.

In the end I had achieved all five goals that I intended to satisfy. This accomplishment led me to believe that the system was well designed. Another goal in this 5th year thesis project was to finish my AE academic career with a more understanding of design. I had spent the fall semester analyzing an in depth concentration in a PT concrete system, and spent the spring semester analyzing an in depth concentration in steel design.

With more available time I would have liked to investigate a dual lateral system consisting of braced frames with moment connections. I feel that if I had initial went in this direction the redesign could have been more effect. Less material could have been used and lesser frames would be integrated in the structure.

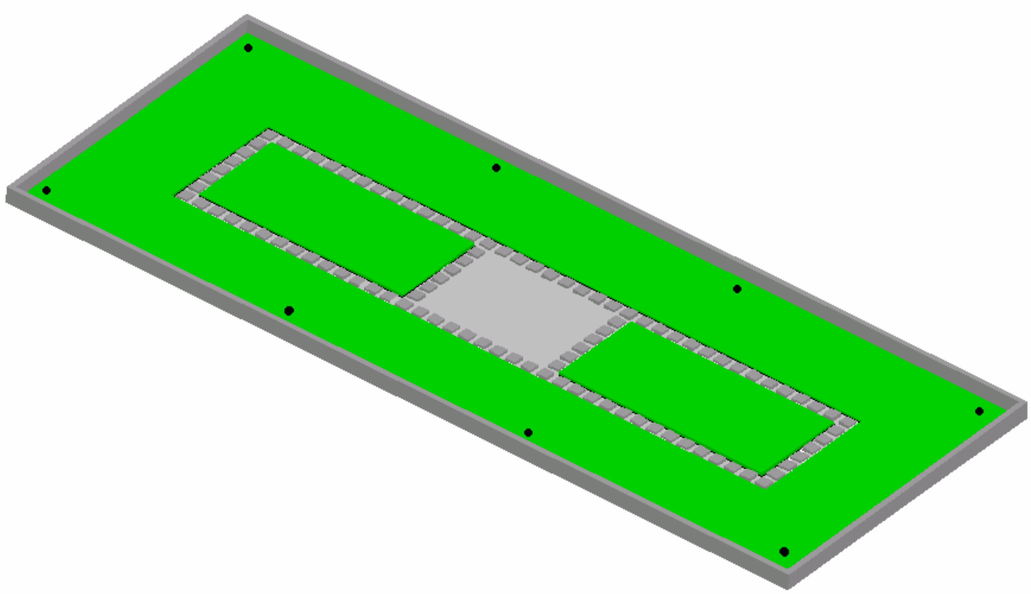
MECHANICAL BREADTH

GREEN ROOF & GRAY WATER

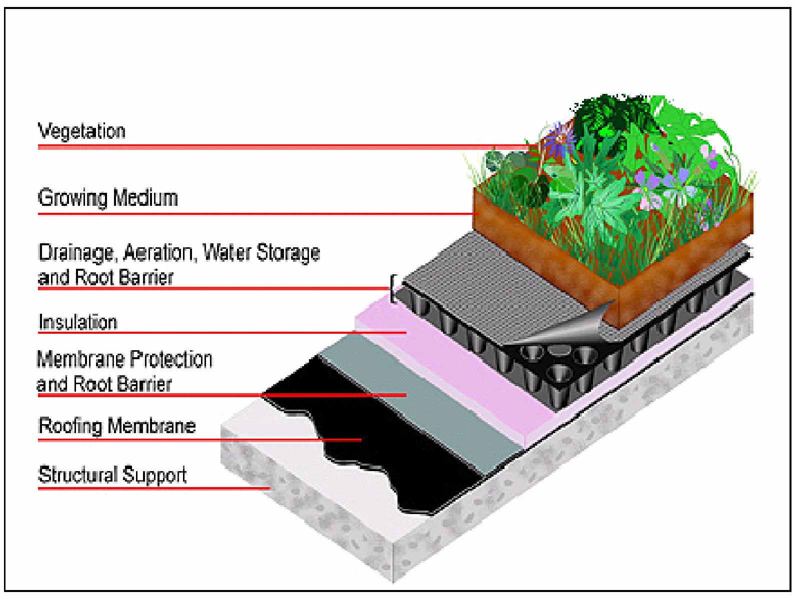
The first breath topic to be introduced is the application of a green roof system. Currently, everyday ideas and products are pushing to 'go green'. The building construction industry has advanced in the past few years in certifying structures to be classified as 'Green Structures'. Owners and architects continue to introduce new buildings to the public to enhance the design and claim the status of being environmental friendly. The main focus of placing a roof garden on The HUB is to promote a safe and sustainable building that will be beneficial to the owner, occupants, and the surrounding community. This design will also function with a recyclable gray water system. The goals to achieve in this design are as follows:

- Reduce the amount and slow flow of storm water run-off
- Provide additional water supply at a very low cost
- Reduce annual utility costs to almost 70%
- Provide a nature habitat for urban found wildlife

Several issues must be addressed in creating an efficient system. The roof must be designed to carry a fully saturated soil and vegetation state, a garden system must be selected, and the amount of water collected and supplied must be researched. The proposed design is to cover 65-85% of the roof area. In the image below, approximately 85% of the roof is covered. The black dots indicate the 5" PVC down spouts that are connected to a central flow system. Drainage has been directed to the roof perimeter to allow the piping to run vertical along the column lines.



The garden is created by a 3-inch mixture of materials referred to as media. This material consists of 25% peat moss and 75% perlite. Perlite is an amorphous volcanic glass that has a relatively high water content along with many nutrients that are ideal for plant growth. The nutrients also work as a natural filter for contaminated water. Its main characteristic is the light density. This allows for a more economical structural support than using traditional soils. Many nutrients include phosphorus, potassium, magnesium, and nitrate. Beneath the 3-inch layer of media is a plastic drainage mat to allow water to freely flow to the drainage system. This mat only allows water to pass so that media aggregate does not interrupt the water flow or clog piping. The next three layers include insulation, a root barrier, and a water resistant membrane. The insulation will be manufactured to create a 2% slope along the roof. The water resistant membrane shall be HDPE 80 (60 mil).



Green roof cross-section
(Source: American Wick Drain Corp.)

There are two types of green roof gardens, extensive and intensive. The HUB on Chestnut was designed using an extensive approach. This allowed for much lighter materials and was more applicable to the Philadelphia climate. The vegetation planted within the media is *Sedum Spurium*. This type of plant is ideal for its climate location, has very short root growth, is dormant in the winter, and has very low maintenance. The vegetation provides local city birds, such as sparrows and hawks, with an ideal place to feed and use raw materials to create a natural habitat.

The main idea of the redesign is to collect rainwater and gray water to be redistributed back into the water supply to be used as flushing water in toilets and water for washing machines. The vegetation will absorb, act as a primary filter, and drain the rainwater into a main collection tank within the lower level of the structure where it can be mechanically and chemically filtered. Research has shown that 30% of fresh water supplied to a building is used to flush toilets and in washing devices. Since the water system will already incorporate a filtration system the fresh water that is discarded can be routed into the roof drainage lines and be circulated back into the system. This large percent can be very efficient to the owner's water supply cost.

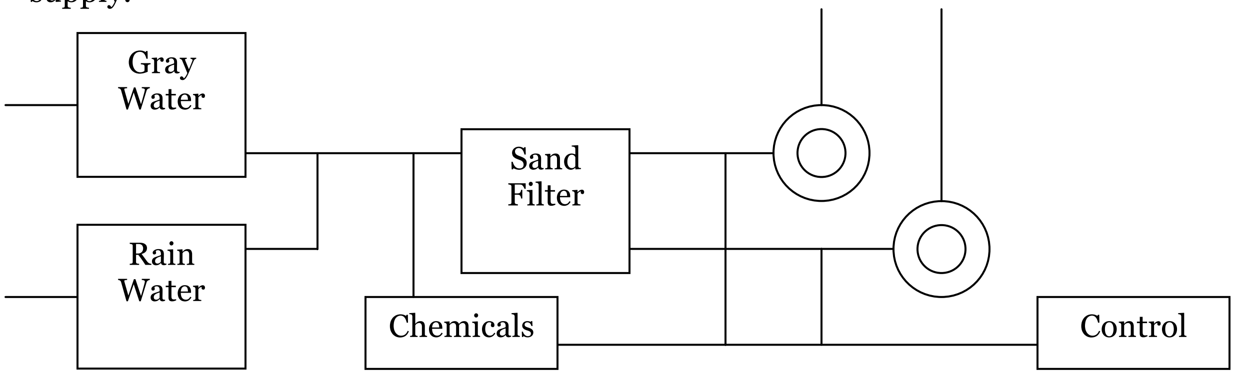
Total Discharge Per Capita (per occupant)

		Black Water		Gray Water	
	Fixture	*GPD*	*%GPD*	*GPD*	*%GPD*
Bathroom	Bath/Shower	8	32	8	47
	Toilet	6	24	-	-
	Hand Basin	4	16	4	24
Kitchen	Sink	2	8	-	-
	Dishwasher	3	12	3	18
	Laundry	2	8	2	12
Total		25	100	17	100

Supply Per Day	
(25 gal/person/day) x (146 persons)	3,650 gal/day
Black Water Discharge Per Day	
(8 gal/person/day) x (146 persons)	1,168 gal/day
Gray Water Discharge Per Day	
(17 gal/person/day) x (146 persons)	2,482 gal/day
Recycled to Supply Per Day	*2,482 gal/day*

The system will be a closed loop that will continuously filter the gray water that is retrieved within a 5,000 gallon sealed reservoir tank located on the sub grade. An everyday supply is estimate to be 3,650 gallon to service all 146 occupants. The discharged water from the kitchen sink and the toilet fixtures is considered black water and cannot be used again. This water is sent directly to the sanitary lines. The rainwater supply will be used to supply the loss 1,168 gallons. A 2,500 gallon tank will store the rainwater for several days. The rainwater will be collected in another tank and will refurbish the water supply as needed. Both tanks will have an outlet to discharge water for over fill and emergencies.

A commercial sized sand filter will be used to provide sufficient decontamination with the aid of additives and chemicals. An electrical control unit will regulate all water flow. Two pumps will be needed to supply the upper levels with their daily supply.



A benefit to the surrounding community is the storm water management. The HUB is placed in a historic section of the city with an out dated street drainage system. By collecting the water from the building's impervious footprint the local community can reduce the amount of water running into the local streets to avoid possible flooding and pollution.

The green roof also makes the upper levels more efficient in cooling and heating months. The increased thermal performance of the roof allows the units below to maintain a stable indoor air temperature and quality that eliminates the need for constant mechanical adjustment. In effect, less energy is used in the entire building lowering costs each year and through the life span of the whole building.

Green Roof	Initial	Area (ft^2)	Total Cost
	14.10/SF	8428	$118,835
5000 Gal tank	$0.50 per gal		$2,500
2500 Gal tank	$0.50 per gal		$1,250
Filtration System			$9,000
			$12,750

Water Usage per Day					
$21.14/1000 ft^3	→	3650 gal = 487 ft^3	→	$10.30	
Water Usage per year					
$10.30/Day	→	365 Days	→	$3,757.74	
Wihtout Rain Supply					
$21.14/1000 ft^3		1168 gal = 156 ft^3	→	$3.30	
Water Usage per year					
$3.30/Day	→	365 Days	→	$1,203.71	
			Savings	$2,554.03	per year

Payoff system in 5 years if no rainwater is used

CONSTRUCTION MANAGEMNET BREADTH

SCHEDULE & SUPERSTRUCTURE COST

The construction management research will cover a design that utilizes the cost of materials, scheduling, and project delivery tasks. The main cost analysis will be the selection of steel and concrete. Due to the current economy, steel prices are at an all time high which has many designers resorting to concrete structures. A recent article has shown that steel prices are down $76 since July 24, 2006. The current price of structural steel is fluctuating around $622 per ton. With a development that can save cost in other areas of construction such as productivity and project delivery time the difference in cost of the two materials may balance to an efficient design.

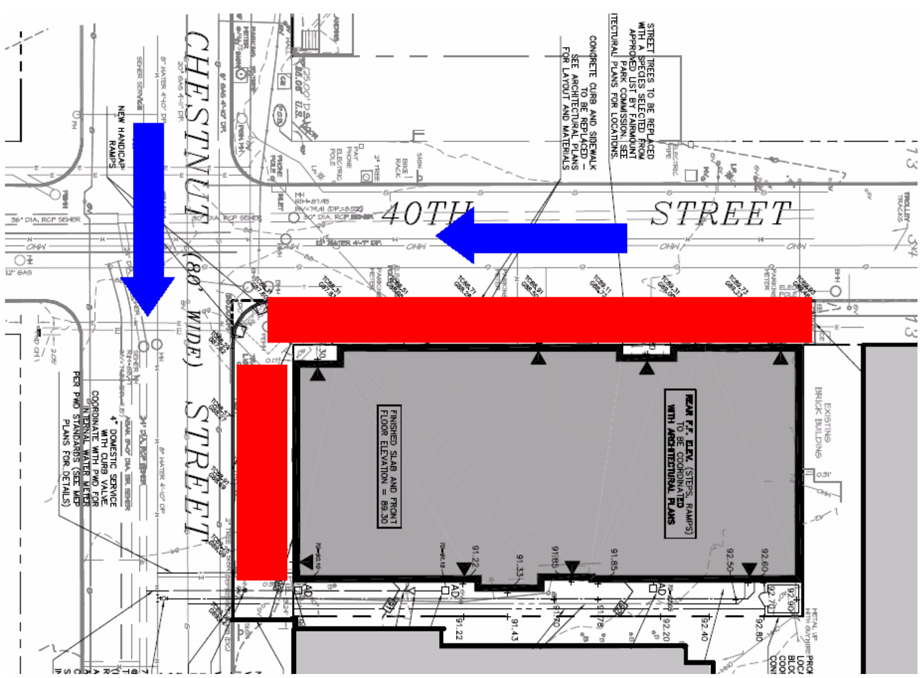
The use of pre-cast elements was selected to increase production. The corner of 40th and Chestnut Streets in Philadelphia is a very congested intersection. The streets in this part of the city are one-way directions and accommodate the traffic of local businesses and the commuters of the several schools and universities in the surrounding communities. Also, the limited space presents no staging areas to store materials onsite. The use of pre-cast concrete and steel can be very beneficial in shortening the project duration. The manufacturers of both materials can dispatch their delivery trucks to site when the material is needed. An onsite crane can unload the materials off the trucks and put them directly in place. The truck can leave and have the next truck follow in sequence.

In scheduling, the superstructure is placed on the critical path to assure that numerous activities and finishes can be completed. The HUB is completely constructed from concrete. The columns and floor systems are all cast-in-place. This type of construction calls for large crew sizes to pump, place, and finish the concrete. Another disadvantage is the post-tensioning system. The concrete needs to set and cure for a certain amount of time before the tendons can be stressed. Also, many pieces of formwork are needed to place the large masses of concrete. The advantage to switch to a steel structural is to increase daily output and productivity.

Steel erection is a quick and steady process. Members can be picked from the delivery truck and put in place immediately. Only a few bolts need to be used to allow the piece to stay. Most connections are left loose throughout an erection sequence due to the fact that the structure will have to be checked for racking later.

The precast planks are erected just as the steel. Pieces are lifted from the truck and put directly in place. Another crew comes behind and makes the connection. However, another step is involved. The placing of a 2" topping can cause trades to conflict and could cause tremendous delays.

The main idea in delivering this type of project is the coordination among trades. The project site is capable of providing areas for two cranes. Unfortunately, this leaves no area to take deliveries. Therefore, the site shall have one crane to lift for both the precasters and the steel erectors. The second area will need to be readily available for truck deliveries. The steel erection must frame enough steel so that the precast slabs can be placed efficiently. A representative from Old Castle quoted an output of 10,000 square feet a day. The HUB displays an average floor area of 10,000 square feet which would allow for one floor per day. This time of productivity can advance a project schedule significantly. The superstructure can be completed approximately 2 months earlier than the concrete post-tensioning system. The owner is then able to receive an extra two months of lease payments in the first year of occupancy.



Superstructure Cost

Steel Frame with Hollow-Core Slabs

Precast Slabs	$869,055.39
Steel Members	$48,586.14
Steel Columns	$164,168.20
Fire Resistance	$96,239.73
	$1,178,049.46

Flat Plate wih Post-Tensioning

Flate Plate	$1,263,529.70
Post-Tensioning	add 2%
CIP Columns	$412,695.00
	$1,701,495.29

	Steel Frame with Hollow-Core Slabs	Flat Plate wih Post-Tensioning
Cost per S.F.	*$13.00*	*$18.78*
Difference in Cost		$523,445.83

Above is the cost comparison between the two structures. It is clear that the use of precast and a steel frame is much less. The concrete structure was a very effect system but the initial cost comes in at a higher price. The owner will also gain two months of lease payments leading to another advantage. The extra expenses could be used for the green roof application and its upgraded water filtration system.

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- O'Donnell & Naccarato
  - Jason Ganister
- Barton & Martin Engineers (Vollmer Associates)
- McClymont & Rak
- AKF Engineers
- 5th AE Thesis peers

APPENDIX I

Properties and Schedules

LIVE LOAD

			IBC 2006 Edition
	Landscaped Roof	20 lb/ft^2	1607.11.2.3
	Hotel/Multi-Family	40 lb/ft^2	Table 1607.1
	Retail (1st Floor)	100 lb/ft^2	Table 1607.1
	Retail (2nd Floor)	75 lb/ft^2	Table 1607.1
	Balconies	100 lb/ft^2	Table 1607.1

DEAD LOAD

Roof	Media/Sedum	23.5 lb/ft^2	
	Root Barrier	0.5 lb/ft^2	
	Insulation	1 lb/ft^2	
	HDPE 80	1 lb/ft^2	
	Collateral	6 lb/ft^2	
		32 lb/ft^2	
Terrace	Insulation	1 lb/ft^2	
	EPDM	3 lb/ft^2	
	Partition	10 lb/ft^2	
	Pavers	10 lb/ft^2	
	Collateral	6 lb/ft^2	
		30 lb/ft^2	
Residential	Partitions	20 lb/ft^2	
	Collateral	8 lb/ft^2	
		28 lb/ft^2	
Commercial	Collateral	10 lb/ft^2	
Span Deck®	8"x 96" w/o Topping	59 lb/ft^2	***Old Castle Precast***
	8"x 96" w/ 2" Topping	84 lb/ft^2	
	10"x 96" w/ 2" Topping	91 lb/ft^2	

SNOW LOAD

		ASCE 7-05
P_g	25	
C_e	1	
C_t	1	
I_s	1	
$P_f = 0.7C_eC_tIP_g$	18 lb/ft^2	

8" x 96" SECTION
WITH 2" TOPPING

SPAN—DECK®

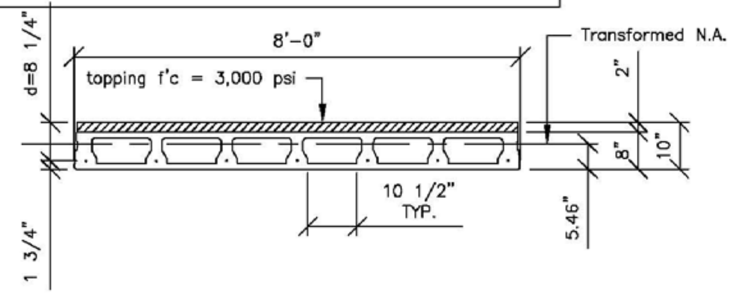
UNIFORMLY DISTRIBUTED SUPERIMPOSED* LOAD IN LBS. PER SQ. FT.

Standard Designation	7-WIRE P/S Strand Combination	P/S Strand Area Sq. In.	Ultimate Bending Moment ø Mn in Kip. Ft. per Unit	SIMPLE SPAN IN FEET																		ø Vcw in Kips per Unit
				16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	
T8S138	13-1/2	1.989	281.0							272	243	218	195	175	158	142	127	115	103	92	83	52.64
T8S128	12-1/2	1.836	263.2							251	224	200	179	160	143	128	115	103	92	82	73	51.60
T8S118	11-1/2	1.683	244.8						257	228	203	181	161	144	128	115	102	91	81	71	63	50.55
T8S108	10-1/2	1.530	225.7					263	232	205	182	161	143	127	113	100	89	78	69	60		49.51
T8S98	9-1/2	1.377	206.0				266	234	206	181	160	141	125	110	97	85	75	65	57			48.46
T8S88	8-1/2	1.224	185.5			268	233	204	178	156	137	120	105	92	81	70	61	52				47.42
T8S78	7-1/2	1.071	164.4		265	229	199	173	150	131	114	99	86	74	63	54						46.37
T8S52	5-1/2 & 2-3/8	.935	145.2	265	226	194	167	144	125	107	92	79	68	57								45.44

*INCLUDES THE LIVE LOAN PLUS ANY DEAD LOAD THAT IS ADDITIONAL TO THE WEIGHT OF THE BARE GROUTED SLABS IN PLACE

NOTES

1. Design Criteria: ACI 318-95
2. For complete and detailed calculations consult Oldcastle Precast, Inc.
3. For longer spans, heavier loads, or special conditions consult Oldcastle Precast, Inc.
4. The table indicates maximum safe loads, Camber and deflection must always be investigated by the architect, and/or engineer for the contemplated loading and span so that these factors are compatible with the contiguous materials in the proposed structure.



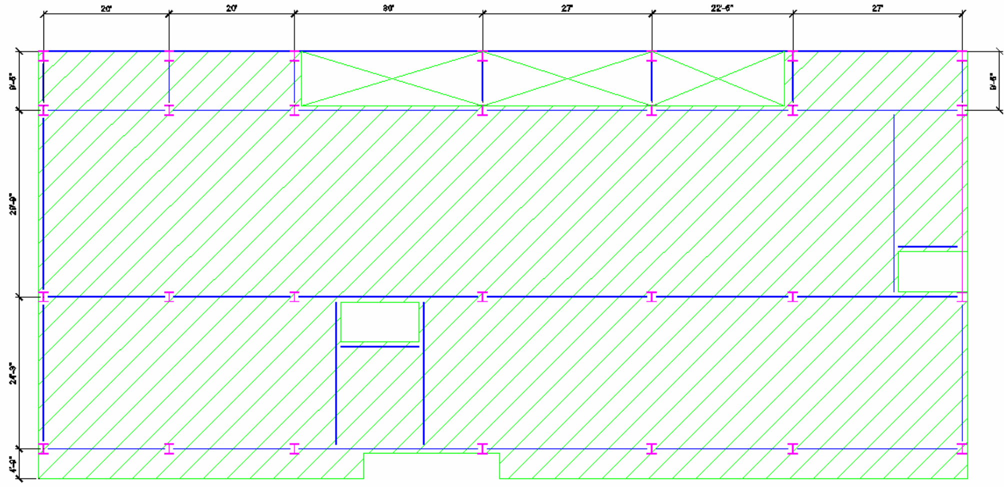
Grouted weight of slabs is 84 lbs. per sq. ft.

f'c = 5,000 psi   f'ci = 3,500 psi   Area = 364 in.²
f'pu = 270,000 psi   I = 5,860 in.⁴   bw = 22.50 in.

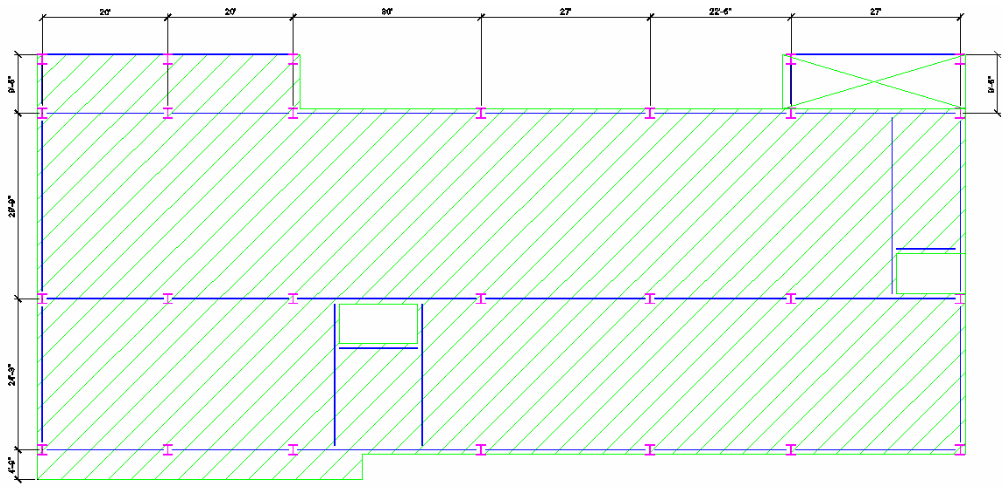
January, 1996

Old Castle Plank Schedule

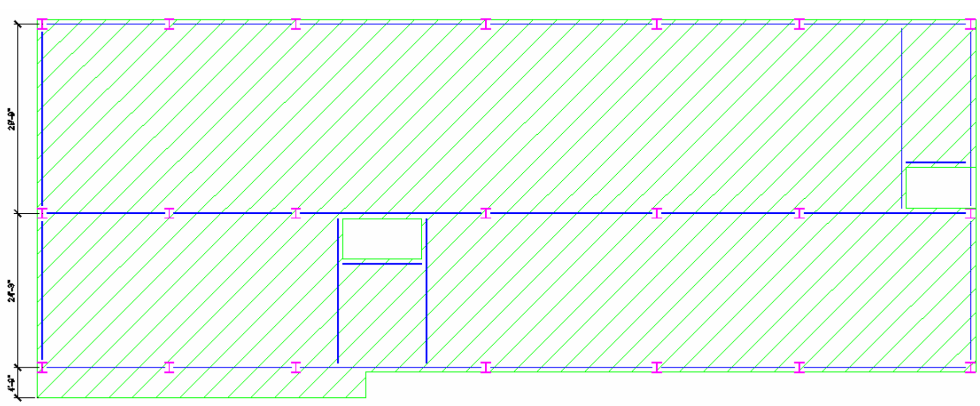
	Superimposed (D+L)	Length (ft)	Plank (ft)	Load (lb/ft^2)	Type
Roof	52 lb/ft^2	29'-9"	30	65	T8S98
	52 lb/ft^2	24'-3"	25	68	T8S52
	52 lb/ft^2	24'-3" w/c	25	68	T8S52
Level 9	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
Level 8	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
Level 7	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
Level 6	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
Terrace	130 lb/ft^2	9'-5"	16	265	T8S52
Level 5	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
	68 lb/ft^2	9'-5"	16	265	T8S52
Terrace	130 lb/ft^2	9'-5"	16	265	T8S52
Level 4	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
	68 lb/ft^2	9'-5"	16	265	T8S52
Level 3	68 lb/ft^2	29'-9"	30	78	T8S108
	68 lb/ft^2	24'-3"	25	86	T8S78
	68 lb/ft^2	24'-3" w/c	25	86	T8S78
	68 lb/ft^2	9'-5"	16	265	T8S52
Terrace	130 lb/ft^2	9'-5"	16	265	T8S52
Level 2	85 lb/ft^2	29'-9"	30	91	T8S118
	85 lb/ft^2	24'-3"	25	86	T8S78
	85 lb/ft^2	24'-3" w/c	25	86	T8S78
Level 1	110 lb/ft^2	29'-9"	30	115	T8S138
	110 lb/ft^2	24'-3"	25	125	T8S98
	110 lb/ft^2	24'-3" w/c	25	125	T8S98



LEVEL 3



LEVEL 5



LEVEL 7

BEAM SCHEDULE

Level 8

Size	Length	Studs	Wt (lbs)
W10x 12	24.25		291
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W12x 19	20.00		380
W14x 22	20.00		440
W8x 10	20.00	22	200
W12x 19	20.00		380
W12x 26	24.25		631
W14x 30	30.00		900
W14x 26	30.00		780
W10x 12	30.00	32	360
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W14x 26	27.00		702
W8x 21	29.75	8	625
W8x 10	11.92		119
W8x 10	11.92		119
W10x 12	24.25		291
W14x 26	29.75		774
		130	13188

Level 9

Size	Length	Studs	Wt (lbs)
W10x 12	24.25		291
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W12x 19	20.00		380
W14x 22	20.00		440
W8x 10	20.00	22	200
W12x 19	20.00		380
W12x 26	24.25		631
W14x 30	30.00		900
W14x 26	30.00		780
W10x 12	30.00	32	360
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W14x 26	27.00		702
W8x 21	29.75	8	625
W8x 10	11.92		119
W8x 10	11.92		119
W10x 12	24.25		291
W14x 26	29.75		774
		130	13188

Roof

Size	Length	Studs	Wt (lbs)
W10x 12	24.25		291
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	16	200
W8x 10	18.25		183
W12x 14	20.00		280
W12x 14	20.00		280
W8x 10	20.00	16	200
W12x 14	20.00		280
W14x 22	30.00		660
W12x 16	30.00	34	480
W14x 22	30.00		660
W8x 10	24.25		243
W12x 19	27.00		513
W8x 10	13.50	8	135
W14x 22	27.00		594
W8x 10	13.50	8	135
W12x 14	22.50		315
W8x 10	22.50	24	225
W12x 16	22.50		360
W12x 19	27.00		513
W8x 10	15.08	8	151
W14x 22	27.00		594
W8x 10	11.92	8	119
W8x 10	24.25		243
W10x 15	29.75		446
		122	8654

Level 7

Size	Length	Studs	Wt (lbs)
W10x 12	24.25		291
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W12x 19	20.00		380
W14x 22	20.00		440
W8x 10	20.00	22	200
W12x 19	20.00		380
W12x 26	24.25		631
W14x 30	30.00		900
W14x 26	30.00		780
W10x 12	30.00	32	360
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W14x 26	27.00		702
W8x 21	29.75	8	625
W8x 10	11.92		119
W8x 10	11.92		119
W10x 12	24.25		291
W14x 26	29.75		774
		130	13188

Level 6

Size	Length	Studs	Wt (lbs)
W8x 10	24.25		243
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W8x 10	9.50		95
W8x 10	20.00	22	200
W12x 19	20.00		380
W14x 22	20.00		440
W8x 10	20.00	22	200
W8x 10	20.00	22	200
W12x 19	20.00		380
W12x 26	24.25		631
W14x 26	30.00		780
W14x 26	30.00		780
W10x 12	30.00	32	360
W8x 10	9.50		95
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W14x 26	27.00		702
W8x 21	29.75		625
W8x 10	11.92	8	119
W8x 10	11.90		119
W8x 10	24.25		243
W8x 21	29.75		625
		174	13412

Level 5

Size	Length	Studs	Wt (lbs)
W8x 10	24.25		243
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W8x 10	9.50		95
W8x 10	20.00	14	200
W10x 12	20.00		240
W14x 22	20.00		440
W8x 10	20.00	22	200
W8x 10	20.00	14	200
W10x 12	20.00		240
W12x 26	24.25		631
W14x 26	30.00		780
W14x 26	30.00		780
W10x 12	30.00	32	360
W8x 10	9.50		95
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W8x 10	9.50		95
W12x 14	27.00	36	378
W14x 30	27.00		810
W8x 21	29.75		625
W8x 10	11.92	8	119
W8x 10	11.92		119
W8x 10	24.25		243
W8x 21	29.75		625
W8x 10	9.50		95
		194	13809

Level 4

Size	Length	Studs	Wt (lbs)
W10x 12	24.25		291
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W8x 10	9.50		95
W8x 10	20.00	22	200
W12x 19	20.00		380
W14x 22	20.00		440
W8x 10	20.00	22	200
W8x 10	20.00	22	200
W12x 19	20.00		380
W12x 26	24.25		631
W14x 26	30.00		780
W14x 26	30.00		780
W10x 12	30.00	32	360
W8x 10	9.50		95
W14x 30	30.00		900
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W14x 22	27.00		594
W8x 10	13.50	8	135
W14x 26	27.00		702
W8x 10	13.50	8	135
W14x 22	22.50		495
W10x 12	22.50	22	270
W14x 22	22.50		495
W14x 22	27.00		594
W8x 10	15.08	8	151
W8x 10	9.50		95
W10x 12	27.00	31	324
W12x 14	27.00		378
W8x 21	29.75		625
W8x 21	11.92	8	250
W8x 10	11.92		119
W8x 10	24.25		243
W8x 21	29.75		625
W8x 10	9.50		95
		205	13782

Level 3

Size	Length	Studs	Wt (lbs)
W8x 10	24.25		243
W14x 22	20.00		440
W8x 10	11.50		115
W8x 10	20.00	22	200
W8x 10	18.25		183
W8x 10	9.50		95
W8x 10	20.00	14	200
W10x 12	20.00		240
W14x 22	20.00		440
W8x 10	20.00	22	200
W8x 10	20.00	14	200
W10x 12	20.00		240
W12x 26	24.25		631
W14x 26	30.00		780
W14x 26	30.00		780
W10x 12	30.00	32	360
W12x 16	30.00	40	480
W14x 43	30.00		1290
W8x 10	7.92		79
W8x 10	7.92		79
W12x 26	24.25		631
W12x 40	27.00		1080
W8x 10	13.50	8	135
W12x 14	27.00	32	378
W14x 30	27.00		810
W8x 10	13.50	8	135
W14x 26	22.50		585
W10x 12	22.50	22	270
W10x 12	22.50	24	270
W14x 22	22.50		495
W12x 40	27.00		1080
W8x 10	15.08	8	151
W10x 12	27.00	31	324
W12x 14	27.00		378
W8x 21	29.75		625
W8x 10	11.92	8	119
W8x 10	11.92		119
W8x 10	24.25		243
W8x 21	29.75		625
W8x 10	9.50		95
		285	15821

Level 2

Size	Length	Studs	Wt (lbs)
W8x 13	24.25		315
W16x 26	20.00		520
W8x 10	11.50		115
W10x 12	20.00	22	240
W8x 10	18.25		183
W8x 10	9.50		95
W8x 10	20.00	20	200
W12x 14	20.00		280
W16x 26	20.00		520
W10x 12	20.00	22	240
W8x 10	20.00	20	200
W12x 14	20.00		280
W14x 68	30.00		2040
W12x 19	30.00	46	570
W12x 16	30.00	36	480
W16x 26	30.00		780
W14x 61	27.00		1647
W8x 10	13.50	10	135
W12x 14	27.00	28	378
W14x 22	27.00		594
W8x 10	13.50	10	135
W16x 31	22.50		698
W12x 14	22.50	24	315
W10x 12	22.50	22	270
W12x 19	22.50		428
W14x 61	27.00		1647
W8x 10	15.08	14	151
W12x 14	27.00	28	378
W14x 22	27.00		594
W8x 10	11.92	8	119
W8x 13	24.25		315
W8x 18	29.75		536
W8x 10	9.50		95
		310	15492

Frame 1

Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS4x4x½	8	15.25	21.5	328
HSS6x6x½	6	15.25	35.1	535
HSS6x6x½	2	16.50	35.1	579
HSS6x6x½	4	19.00	35.1	667
				2109

Frame 3

Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS6x6x½	14	15.75	35.1	553
HSS6x6x½	2	17.00	35.1	597
HSS8x8x½	4	19.25	48.7	937
				2087

Frame 5

Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS4x4x½	8	18.00	21.5	387
HSS6x6x½	6	18.00	35.1	632
HSS6x6x½	2	19.00	35.1	667
HSS6x6x½	4	21.00	35.1	737
				2423

Frame 2

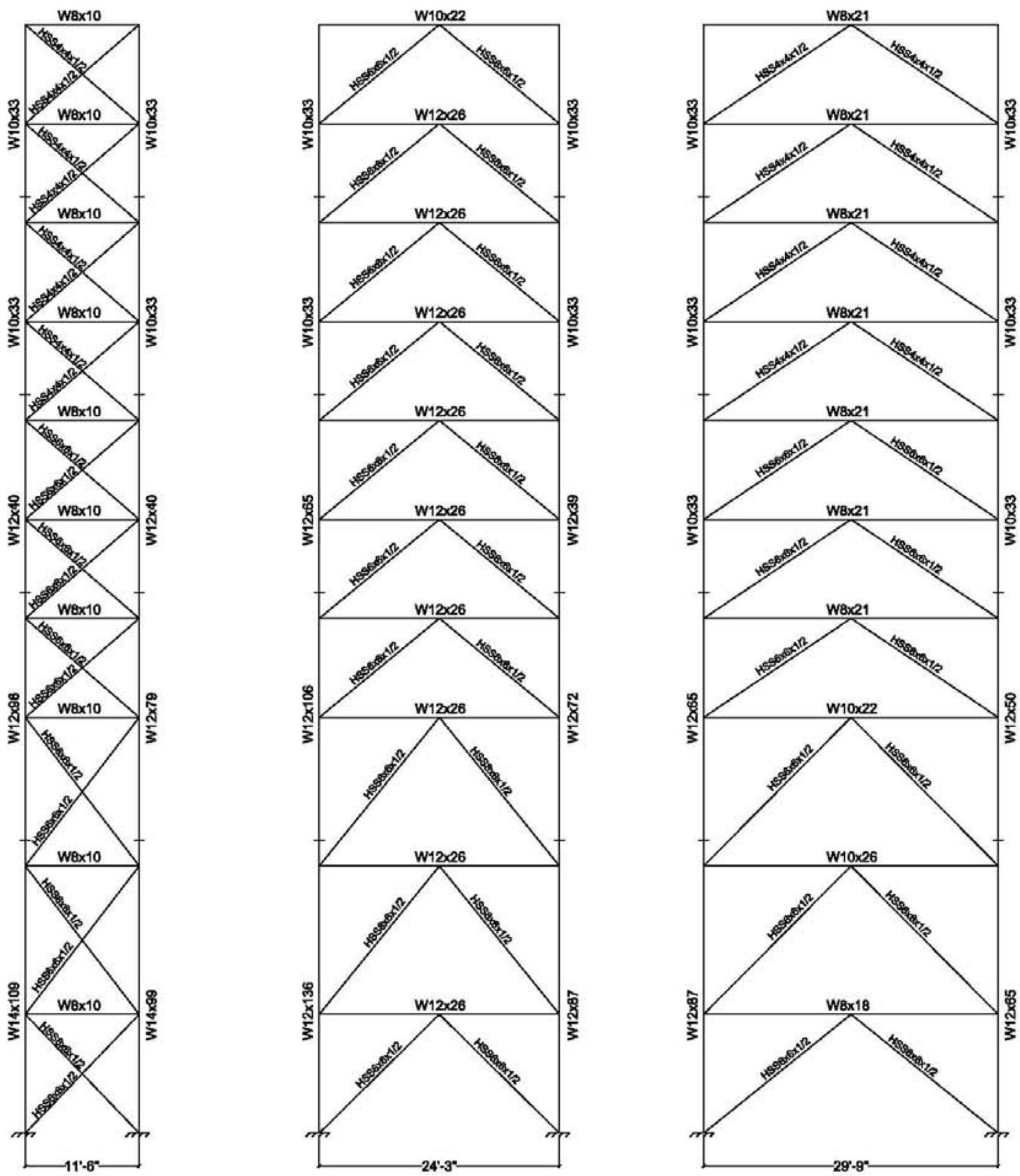
Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS6x6x½	14	15.75	35.1	553
HSS6x6x½	4	27.00	35.1	948
HSS6x6x½	2	28.50	35.1	1000
				2501

Frame 4

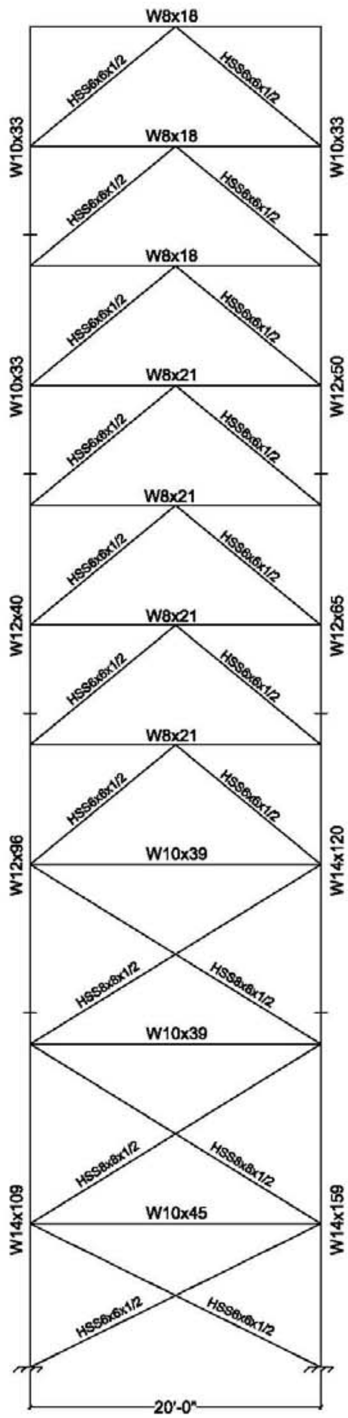
Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS4x4x½	6	16.75	21.5	360
HSS6x6x½	8	16.75	35.1	588
HSS8x8x½	4	18.00	48.7	877
HSS8x8x½	2	20.25	48.7	986
				2811

Frame 6

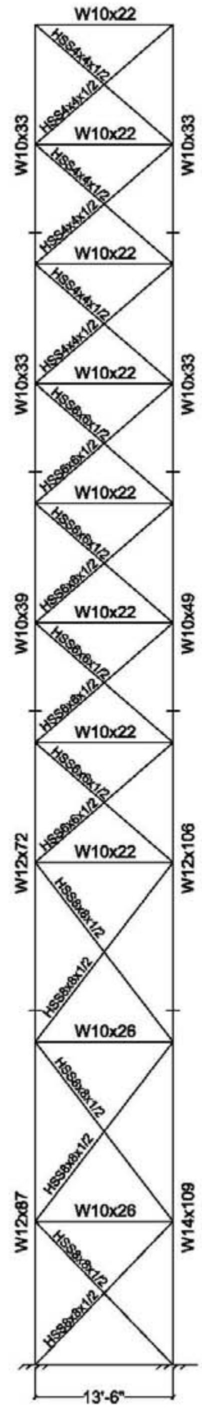
Size	Quanity	Length (ft)	Size (lb/ft)	Wt (lb)
HSS4x4x½	8	17.00	21.5	366
HSS6x6x½	6	17.00	35.1	597
HSS6x6x½	2	18.25	35.1	641
HSS6x6x½	2	18.25	35.1	641
HSS8x8x½	2	20.50	48.7	998
				3242



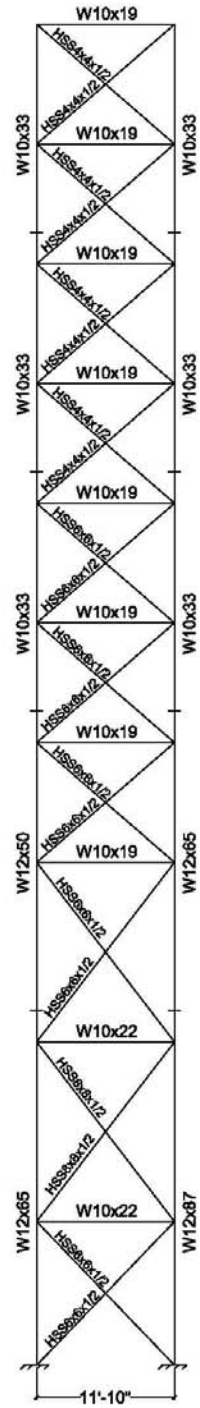
Frame 1 Frame 3 Frame 5



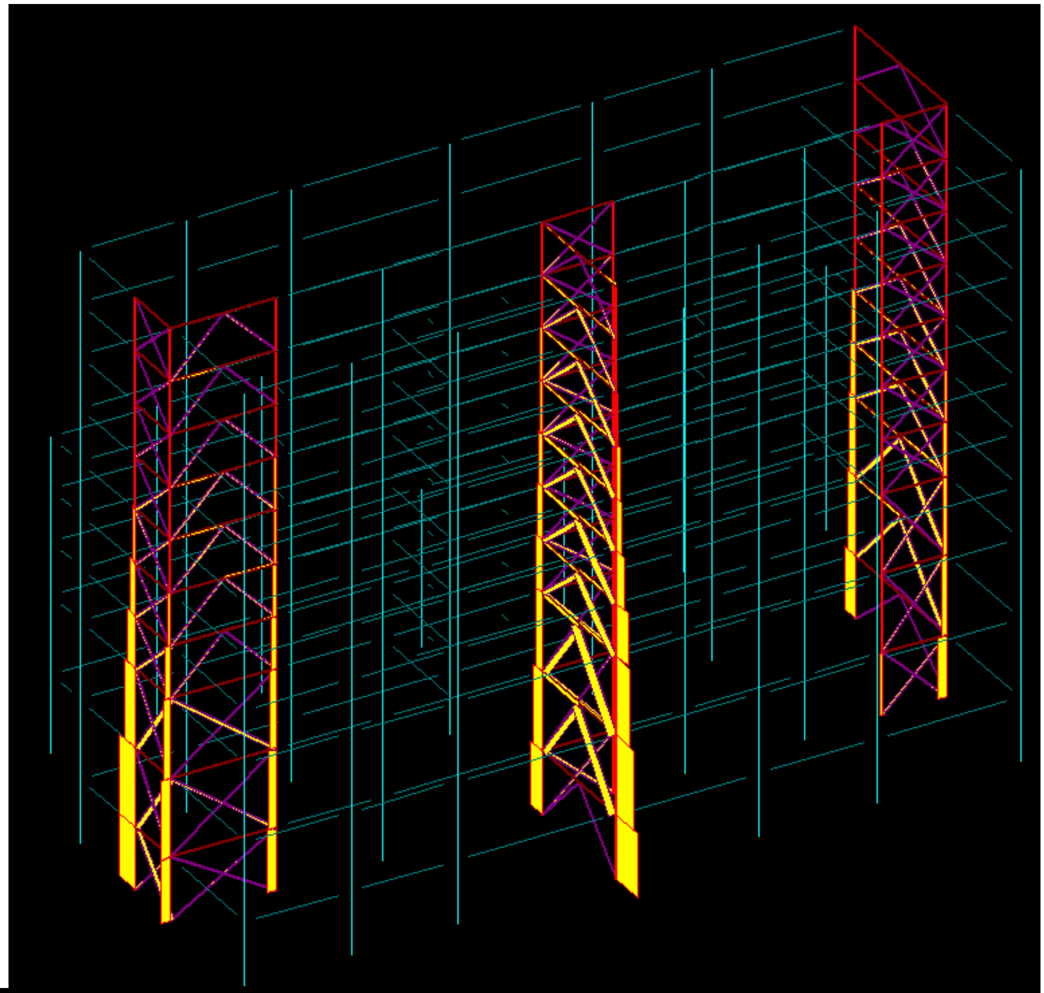
Frame 2

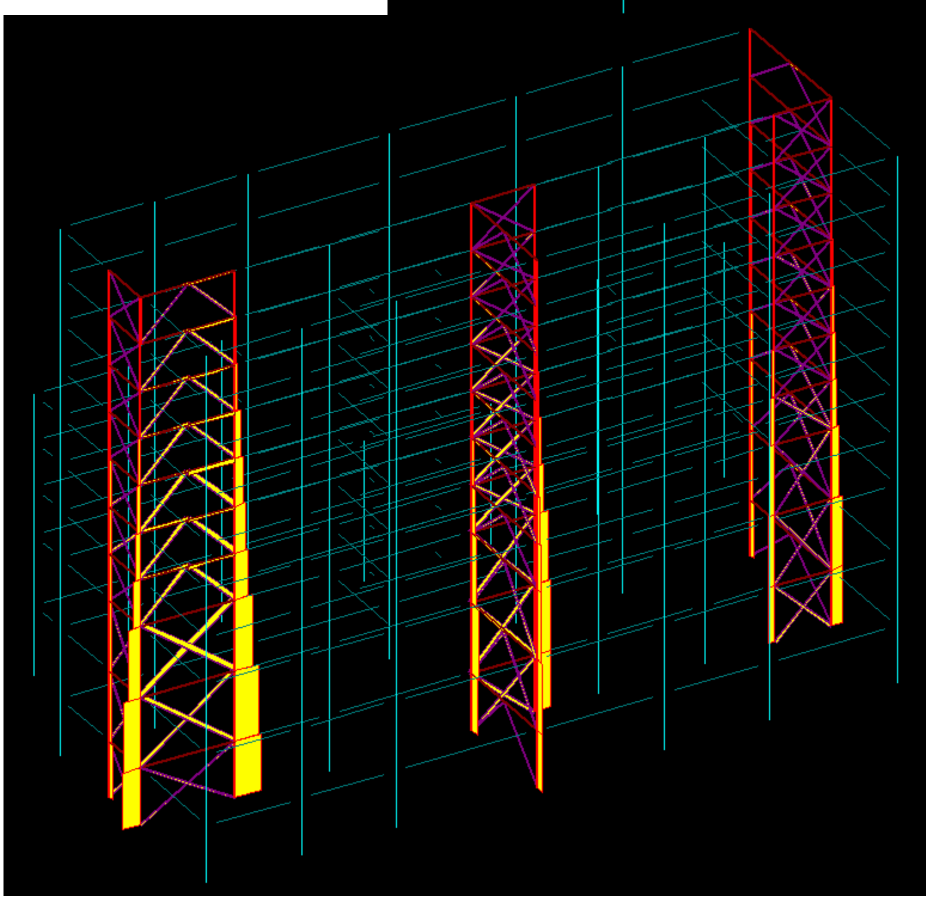


Frame 4



Frame 6





Existing Building Weight

	30x30	20x30	20Φ	w_c(lb)	Area (FT^2)	Load (lb/ft^2)	w_z(K)
Roof	0	21	1	134523	8428	130	1230
9	0	21	1	134523	8428	141	1323
8	0	21	1	134523	8428	141	1323
7	0	21	1	134523	8428	141	1323
6	0	21	1	134523	8791	141	1374
5	0	22	0	137500	9044	141	1413
4	0	22	0	137500	9044	141	1413
3	9	13	0	248442	10010	141	1660
2	15	7	0	276570	10010	160	1878
1	15	7	0	276570	9539	160	1803
							14739

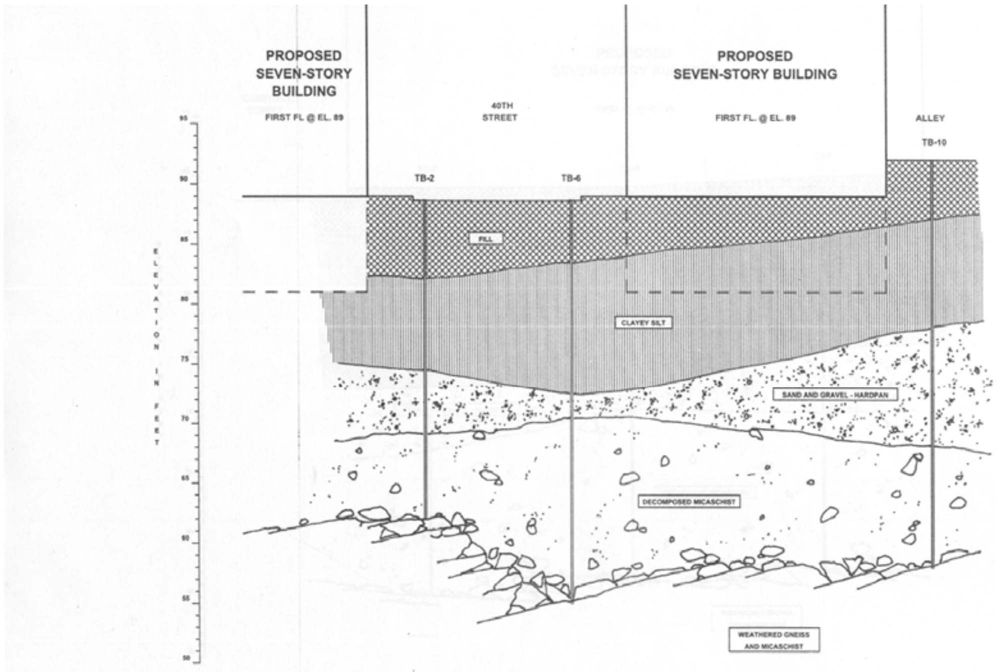
Redesign Building Weight

	Load (lb/ft^2)	Slab (lb/ft^2)	Steel (lb/ft^2)	Area (ft^2)	W_T (K)
Roof	32	84	3	8428	1003
9	28	84	3	8278	952
8	28	84	3	8278	952
7	28	84	3	8278	952
6	28	84	3	8641	994
5	28	84	3	8894	1023
4	28	84	3	8894	1023
3	28	84	3	9860	1134
2	10	84	3	10010	971
1	10	84	3	10010	971
					9974

Rigidity

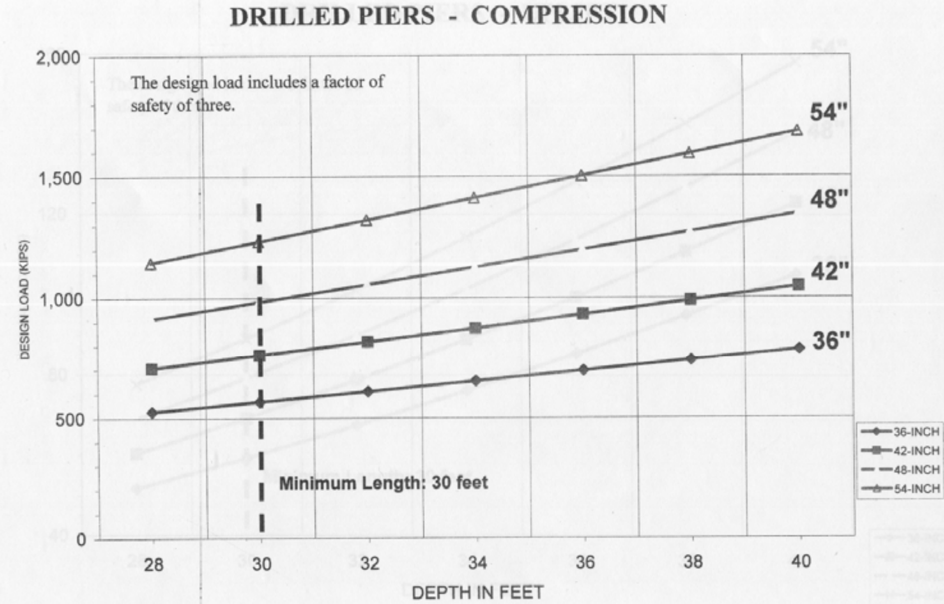
	Weight (K)	Xm (ft)	Ym (ft)	Xe (ft)	Ye (ft)
Roof	1003	72.12	31.02	7.4	2.98
9	952	71.75	31.06	7.4	2.98
8	952	71.75	31.06	7.4	2.98
7	952	71.75	31.06	7.4	2.98
6	994	69.27	32.56	7.4	3.45
5	1023	71.27	33.52	7.4	3.45
4	1023	71.27	33.52	7.4	3.45
3	1134	72.98	34.85	7.4	3.45
2	971	73.12	34.71	7.4	3.45
1	971	73.12	34.71	7.4	3.46

FOUNDATION AIDS



STRATIGRAPHIC CROSS-SECTION A-A

DRILLED PIERS - COMPRESSION



3938 - 40th Street Development, Philadelphia PA

APPENDIX II

Wind Analysis

WIND ANALYSIS

ASCE 7-05 Chapter 6

Location	Philadelphia, PA
Typography	Homogeneous
Framing	Braced Frames
Cladding	Rainscreen Panel Assembly
Frequency	Flexible
Enclosure Class	Enclosed

Velocity Pressure

q_z	$0.00256 K_z K_{zt} K_d V^2 I$
V_3	90
I_w	1.00
K_d	0.85
K_{zt}	1.00

Dimensions

N/S	B_1	=	*60*
N/S	B_2	=	*68*
E/W	B	=	*148*

Internal Pressure Coefficient

GC_{pi}	± 0.18

Gust Effect Factor

1/T ≤ 1	Flexible
North/South	East/West
1.100	0.929
1.085	

External Pressure Coefficients

North/South			East/West		
Wall	*Cp*		*Wall*	*Cp*	
	0.80	Windward		0.80	Windward
	-0.30	Leeward		-0.50	Leeward
	-0.70	Side		-0.70	Side
Roof	-0.95	0 to h/2	*Roof*	-1.04	0 to h/2
	-0.83	h/2 to h		-0.70	> h/2
	-0.57	h to 2h			

Wall Pressures

			Windward				Leeward				X	Y
			North/South		East/West		North/South		East/West		MWFRS	
Height (FT)	K_z	q_z	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*North/South*	*East/West*
0-15	0.57	10.12	5.65	11.92	4.38	10.65	-8.80	-2.53	-11.22	-4.95	14.45	15.61
20	0.62	10.93	6.35	12.62	4.99	11.26	-8.80	-2.53	-11.22	-4.95	15.15	16.21
25	0.66	11.63	6.96	13.23	5.51	11.78	-8.80	-2.53	-11.22	-4.95	15.77	16.73
30	0.70	12.36	7.59	13.86	6.05	12.32	-8.80	-2.53	-11.22	-4.95	16.39	17.27
40	0.76	13.41	8.51	14.78	6.83	13.10	-8.80	-2.53	-11.22	-4.95	17.31	18.06
50	0.81	14.29	9.27	15.54	7.49	13.76	-8.80	-2.53	-11.22	-4.95	18.08	18.71
60	0.85	15.05	9.93	16.20	8.05	14.32	-8.80	-2.53	-11.22	-4.95	18.73	19.28
70	0.89	15.72	10.70	16.97	8.55	14.82	-8.88	-2.61	-11.22	-4.95	19.58	19.77
80	0.93	16.34	11.24	17.51	9.01	15.28	-8.88	-2.61	-11.22	-4.95	20.12	20.23
90	0.96	16.90	11.74	18.01	9.43	15.70	-8.88	-2.61	-11.22	-4.95	20.62	20.65
100	0.99	17.41	12.19	18.46	9.81	16.08	-8.88	-2.61	-11.22	-4.95	21.07	21.03

Roof Pressures

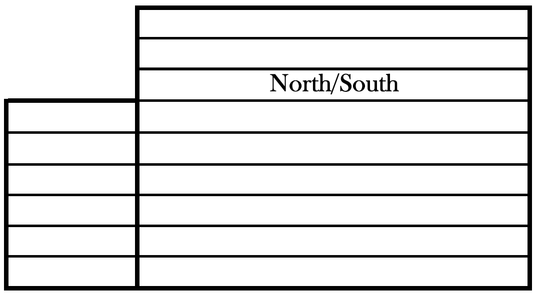
Zone	North/South		East/West	
	+0.18	*-0.18*	*+0.18*	*-0.18*
0 to h/2	-17.20	-10.93	-18.53	-10.93
h/2 to h	-15.42	-9.15	-13.50	-9.15
h to 2h	-11.57	-5.30		

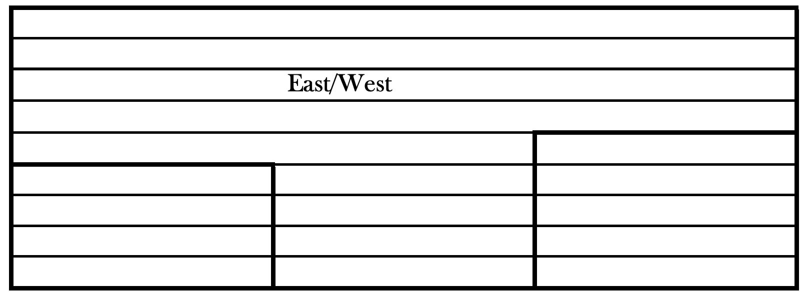
Parapet Pressures

$p_p = q_p GC_{pn}$	
Windward	Leeward
+1.50	*-1.00*
26.12	-17.41

North/South MWRS Forces

Level	Height (FT)	Tributary (FT)	Pressure	Height	Pressure	Height	Pressure	Height	Story Dist.	Cum Dist.	Story Shear	Cum Shear
Roof	100	5.00	21.07	5					105.36	1808.51	6.32	6.32
9	90	10.00	21.07	5	20.62	5			208.46	1703.15	12.51	18.83
8	80	10.00	20.62	5	20.12	5			203.73	1494.69	12.22	31.05
7	70	10.00	20.12	5	19.58	5			198.53	1290.96	11.91	42.96
6	60	10.00	19.58	5	18.73	5			191.58	1092.42	11.49	54.46
5	50	10.00	18.73	5	18.08	5			184.05	900.84	12.52	66.97
4	40	10.00	18.08	5	17.31	5			176.93	716.80	12.03	79.01
3	30	12.50	17.31	5	16.39	5	15.77	2.5	207.93	539.86	14.14	93.15
2	15	15.00	15.77	2.5	15.15	5	14.45	7.5	223.56	331.93	15.20	108.35
1	0	7.50	14.45	7.5					108.37	108.37	7.37	115.72

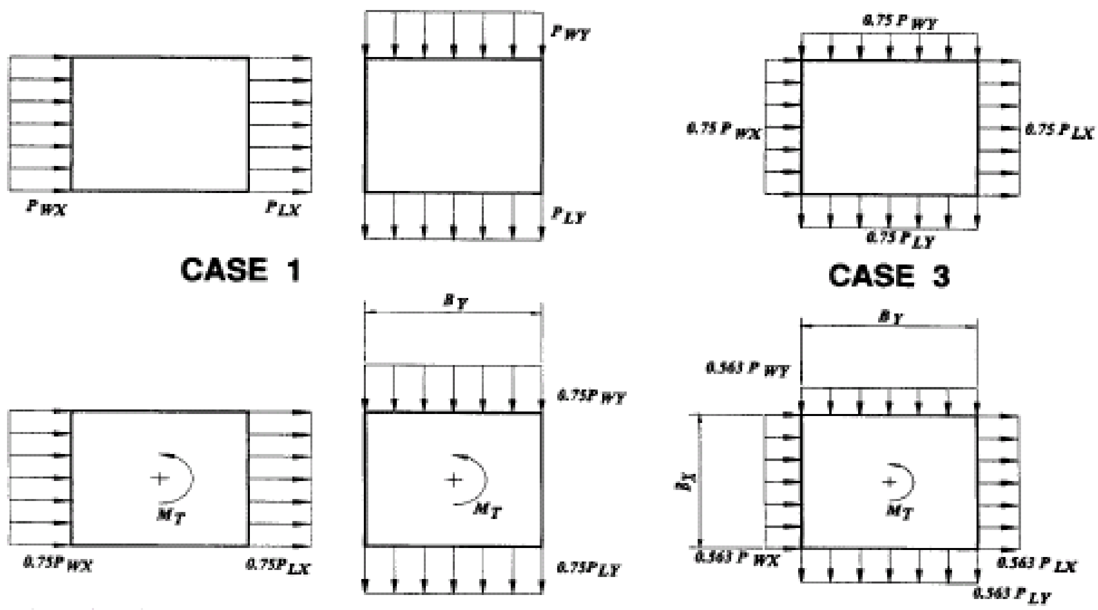




East/West MWRS Forces

Level	Height (FT)	Tributary (FT)	Pressure	Height	Pressure	Height	Pressure	Height	Story Dist.	Cum Dist.	Story Shear	Cum Shear
Roof	100	5.00	21.03	5					105.15	1862.53	15.56	15.56
9	90	10.00	21.03	5	20.65	5			208.41	1757.38	30.84	46.41
8	80	10.00	20.65	5	20.23	5			204.42	1548.96	30.25	76.66
7	70	10.00	20.23	5	19.77	5			200.03	1344.55	29.60	106.27
6	60	10.00	19.77	5	19.28	5			195.25	1144.52	28.90	135.16
5	50	10.00	19.28	5	18.71	5			189.94	949.28	28.11	163.27
4	40	10.00	18.71	5	18.06	5			183.85	759.34	27.21	190.48
3	30	12.50	18.06	5	17.27	5	16.73	2.5	218.48	575.49	32.34	222.82
2	15	15.00	16.73	2.5	16.21	5	15.61	7.5	239.95	357.01	35.51	258.33
1	0	7.50	15.61	7.5					117.06	117.06	17.32	275.65

Main Wind Force Resisting System– Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\,0.15\,B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\,0.15\,B_Y$

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\,0.15\,B_X$ $e_Y = \pm\,0.15\,B_Y$

CASE 2

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
$e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

Load Case 1

	X	Y
	Fx (K)	*Fy (K)*
Roof	6.32	15.56
9	12.51	30.84
8	12.22	30.25
7	11.91	29.60
6	11.49	28.90
5	12.52	28.11
4	12.03	27.21
3	14.14	32.34
2	15.20	35.51
	108.35	258.33

Load Case 3

	X + Y		X - Y	
	Fx (K)	*Fy (K)*	*Fx (K)*	*Fy (K)*
Roof	4.74	11.67	4.74	-11.67
9	9.38	23.13	9.38	-23.13
8	9.17	22.69	9.17	-22.69
7	8.93	22.20	8.93	-22.20
6	8.62	21.67	8.62	-21.67
5	9.39	21.08	9.39	-21.08
4	9.02	20.41	9.02	-20.41
3	10.60	24.25	10.60	-24.25
2	11.40	26.63	11.40	-26.63
	81.26	193.75	81.26	-193.75

Load Case 2

		X + e	*X - e*		*Y + e*	*Y - e*
	Fx (K)	M_T *(ft-K)*	M_T *(ft-K)*	*Fy (K)*	M_T *(ft-K)*	M_T *(ft-K)*
Roof	4.74	2560.13	-2560.13	11.67	38350.20	-38350.20
9	9.38	5065.61	-5065.61	23.13	76007.68	-76007.68
8	9.17	4950.66	-4950.66	22.69	74550.58	-74550.58
7	8.93	4824.39	-4824.39	22.20	72950.17	-72950.17
6	8.62	5979.55	-5979.55	21.67	71206.44	-71206.44
5	9.39	6510.41	-6510.41	21.08	69271.61	-69271.61
4	9.02	6258.76	-6258.76	20.41	67050.14	-67050.14
3	10.60	7355.33	-7355.33	24.25	79680.27	-79680.27
2	11.40	7908.01	-7908.01	26.63	87509.15	-87509.15
	81.26			193.75		

Load Case 4

			X +Y CW	*X +Y CCW*
	Fx (K)	*Fy (K)*	M_T *(ft-K)*	M_T *(ft-K)*
Roof	3.56	8.76	30710.02	-30710.02
9	7.04	17.37	60859.02	-60859.02
8	6.88	17.03	59678.93	-59678.93
7	6.71	16.67	58382.77	-58382.77
6	6.47	16.27	57940.95	-57940.95
5	7.05	15.83	56887.04	-56887.04
4	6.77	15.32	55030.55	-55030.55
3	7.96	18.20	65334.72	-65334.72
2	8.56	19.99	71626.49	-71626.49
	61.00	145.44		

APPENDIX III

Seismic Analysis

SEISMIC ANALYSIS

ASCE 7-05 Chapter 12

Location	Philadelphia, PA
Occupancy Category	II
Seismic Use Group	II
Importance Factor	1.00
Site Classification	D
Basic Structural Systen	Braced Frame
Seismic Resisting Syste	Ordinary Steel Concentrically Braced Frames
Frequency	Rigid Structure

I_E	1
S_S	0.320
S_1	0.082
Fa	1.54
Fv	2.40
S_{MS}	0.493
S_{M1}	0.197
S_{DS}	0.329
S_{D1}	0.131
R	3.25
Ω	2
Cd	3.25
Seismic Design Category B	

T_L	6
C_u	1.638
h_n	100
C_t	0.02
x	0.75
T_a	0.632
T	1.04

Cs	0.039
W (K)	9003
V (K)	351
k	1.268

1/*T*	0.965	? 1

Cs	0.1011	$S_{DS}/(R/I)$	12.8-2	
Cs	0.0390	$S_{D1}/T(R/I)$	12.8-3	*CONTROLS*
Cs		*> 0.01*	12.8-5	

North/South/East/West

Level	h_x(FT)	w_x(K)	$w_x h^k_x$	C_{vx}	F_x(K)	V_x(K)	M_x(FT-K)
Roof	100	1003	344565	0.211	73.96	73.96	739.59
9	90	952	286156	0.175	61.42	135.38	2093.39
8	80	952	246457	0.151	52.90	188.28	4715.78
7	70	952	208069	0.127	44.66	232.94	9878.17
6	60	994	178631	0.109	38.34	271.28	20139.76
5	50	1023	145911	0.089	31.32	302.60	40592.71
4	40	1023	109953	0.067	23.60	326.20	81421.42
3	30	1134	84638	0.052	18.17	344.37	164746.37
2	15	971	30095	0.018	6.46	350.83	329589.63
1	0	971	0	0	0	0	0
?	100	9003	1634476	1	350.83		

	X		Y	
	E_h	E_v	E_h	E_v
Roof	73.96	65.90	73.96	65.90
9	61.42	62.55	61.42	62.55
8	52.90	62.55	52.90	62.55
7	44.66	62.55	44.66	62.55
6	38.34	65.29	38.34	65.29
5	31.32	67.21	31.32	67.21
4	23.60	67.21	23.60	67.21
3	18.17	74.50	18.17	74.50
2	6.46	63.80	6.46	63.80

E_1 $E = E_h + E$ North/Sou controls

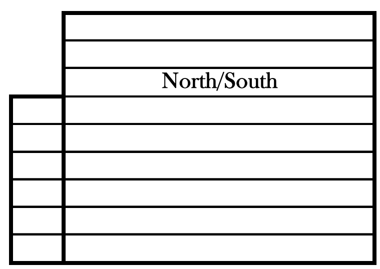
E_2 $E = E_h - E_v$ North/South

E_3 $E = E_h + E$ East/West

E_4 $E = E_h - E_v$ East/West controls

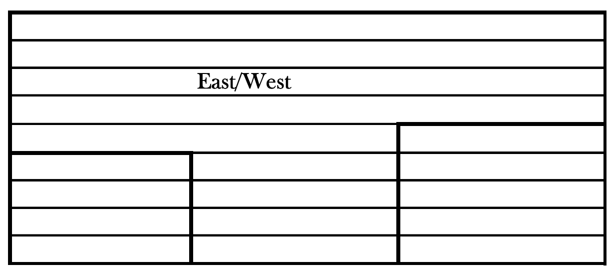
North/South *ASCE 7-05 [12.12.1]*

Level	Story Height Below (ft)	Displace (in)	Δ (in)	Δa (in)	Δ ≤ Δa=0.015hsx
Roof	10	4.597	0.577	2.4	✓
9	10	4.02	0.594	2.4	✓
8	10	3.426	0.600	2.4	✓
7	10	2.826	0.563	2.4	✓
6	10	2.263	0.537	2.4	✓
5	10	1.726	0.490	2.4	✓
4	10	1.236	0.429	2.4	✓
3	15	0.807	0.494	3.6	✓
2	15	0.313	0.313	3.6	✓
1	0	0	0	0	



East/West *ASCE 7-05 [12.12.1]*

Level	Story Height Below (ft)	Displace (in)	Δ (in)	Δa (in)	Δ ≤ Δa=0.015hsx
Roof	10	3.281	0.370	2.4	✓
9	10	2.911	0.399	2.4	✓
8	10	2.512	0.407	2.4	✓
7	10	2.105	0.367	2.4	✓
6	10	1.738	0.409	2.4	✓
5	10	1.329	0.346	2.4	✓
4	10	0.983	0.324	2.4	✓
3	15	0.659	0.386	3.6	✓
2	15	0.273	0.273	3.6	✓
1	0	0	0	0	



APPENDIX IV

Cost Analysis

Flat Plate

	Area (ft²)	Thick (in)	Material	Install	Total/S.F.	Cost
Roof	8428	9"	6.15	7.55	13.70	115463.60
9	8428	9"	6.15	7.55	13.70	115463.60
8	8428	9"	6.15	7.55	13.70	115463.60
7	8428	9"	6.15	7.55	13.70	115463.60
6	8791	9"	6.15	7.55	13.70	120436.70
5	9044	9"	6.15	7.55	13.70	123902.80
4	9044	9"	6.15	7.55	13.70	123902.80
3	10010	9"	6.15	7.55	13.70	137137.00
2	10010	12"	6.90	7.90	14.80	148148.00
1	10010	12"	6.90	7.90	14.80	148148.00
					$	1,263,529.70

Precast Hollow-Core Planks (2" Topping)

	Area (ft²)	Thick (in)	Material	Install	Total/S.F.	Cost
Roof	8428	8"	7.35	2.24	9.59	80824.52
9	8428	8"	7.35	2.24	9.59	80824.52
8	8428	8"	7.35	2.24	9.59	80824.52
7	8428	8"	7.35	2.24	9.59	80824.52
6	8791	8"	7.35	2.24	9.59	84305.69
5	9044	8"	7.35	2.24	9.59	86731.96
4	9044	8"	7.35	2.24	9.59	86731.96
3	10010	8"	7.35	2.24	9.59	95995.90
2	10010	8"	7.35	2.24	9.59	95995.90
1	10010	8"	7.35	2.24	9.59	95995.90
					$	869,055.39

Structral Steel

	Wt (lbs)	Material	Costs
Steel Girder	135027	$622.00 per ton	$41,993.43
Shear Studs	2026	$622.00 per ton	$630.09
Braces	15172	$622.00 per ton	$4,718.62
			$47,342.14

Cast-in-Place Concrete Columns

	Size	Wt (lb/ft)	Height (ft)	Each	Strength (psi)	Material	Install	Total/VLF	Cost
Roof	20x30	625	10	21	5000	46.50	106.00	153.00	32130.00
	20Φ	327	10	1	5000	28.00	75.50	103.50	1035.00
9	20x30	625	10	21	5000	46.50	106.00	153.00	32130.00
	20Φ	327	10	1	5000	28.00	75.50	103.50	1035.00
8	20x30	625	10	21	5000	46.50	106.00	153.00	32130.00
	20Φ	327	10	1	5000	28.00	75.50	103.50	1035.00
7	20x30	625	10	21	5000	46.50	106.00	153.00	32130.00
	20Φ	327	10	1	5000	28.00	75.50	103.50	1035.00
6	20x30	625	10	21	5000	46.50	106.00	153.00	32130.00
	20Φ	327	10	1	5000	28.00	75.50	103.50	1035.00
5	20x30	625	10	22	5000	46.50	106.00	153.00	33660.00
4	20x30	625	10	22	5000	46.50	106.00	153.00	33660.00
3	30x30	938	15	9	5000	67.00	134.00	201.00	27135.00
	20x30	625	15	13	5000	46.50	106.00	153.00	29835.00
2	30x30	938	15	15	5000	67.00	134.00	201.00	45225.00
	20x30	625	15	7	5000	46.50	106.00	153.00	16065.00
1	30x30	938	15	15	5000	67.00	134.00	201.00	45225.00
	20x30	625	15	7	5000	46.50	106.00	153.00	16065.00
									$ 412,695.00

Steel Column

Member Size	Length (ft)	Material	Install	Total/LF	Cost
W10x 33	21	38.00	9.40	47.50	997.50
W10x 39	1	48.00	9.40	57.40	57.40
W10x 45	111	57.20	7.05	64.55	7165.05
W10x 49	52	61.50	9.40	70.90	3686.80
W12x 40	40	50.50	7.05	57.55	2302.00
W12x 50	25	62.00	7.05	69.05	1726.25
W12x 65	172	83.50	9.40	92.90	15978.80
W12x 72	25	91.00	7.05	98.05	2451.25
W12x 79	25	94.50	7.05	101.55	2538.75
W12x 87	54	110.00	7.05	117.50	6345.00
W12x 96	25	110.00	7.05	117.50	2937.50
W12x 106	25	138.00	7.05	145.05	3626.25
W12x 136	27	186.00	9.40	195.40	5275.80
W14x 99	27	135.00	9.40	144.40	3898.80
W14x 109	27	138.00	7.05	145.05	3916.35
					$ 62,903.50

Gravity

Size	Length (ft)	Wt (lbs)
W10x 33	1688	55704
W10x 39	134	5226
W10x 45	111	4995
W10x 49	52	2548
		68473

Lateral

Size	Length (ft)	Wt (lbs)
W10x 33	280	9240
W10x 39	20	780
W12x 40	40	1600
W12x 50	25	1250
W12x 65	172	11180
W12x 72	25	1800
W12x 79	25	1975
W12x 87	54	4698
W12x 96	25	2400
W12x 106	25	2650
W12x 136	27	3672
W14x 99	27	2673
W14x 109	27	2943
		46861

Total Columns

		Material	Install	Total/LF	Cost
W10x 33	1968	$38.00	$9.40	$47.50	$93,480.00
W10x 39	154	$48.00	$9.40	$57.40	$8,839.60
W10x 45	111	$57.20	$7.05	$64.55	$7,165.05
W10x 49	52	$61.50	$9.40	$70.90	$3,686.80
W12x 40	40	$50.50	$7.05	$57.55	$2,302.00
W12x 50	25	$62.00	$7.05	$69.05	$1,726.25
W12x 65	172	$83.50	$9.40	$92.90	$15,978.80
W12x 72	25	$91.00	$7.05	$98.05	$2,451.25
W12x 79	25	$94.50	$7.05	$101.55	$2,538.75
W12x 87	54	$110.00	$7.05	$117.50	$6,345.00
W12x 96	25	$110.00	$7.05	$117.50	$2,937.50
W12x 106	25	$138.00	$7.05	$145.05	$3,626.25
W12x 136	27	$186.00	$9.40	$195.40	$5,275.80
W14x 99	27	$135.00	$9.40	$144.40	$3,898.80
W14x 109	27	$138.00	$7.05	$145.05	$3,916.35
					$164,168.20