AE Senior Thesis
Final Report

John Jay College Expansion Project

New York, NY

Michael Hopper

Structural Option

AE Consultant: Dr. Andres Lepage

April 7th, 2009

John Jay College Expansion Project

JOHN JAY COLLEGE OF CRIMINAL JUSTICE

New York, NY

Michael Hopper
Structural Option

Project Team:

Owner:
Dormitory Authority of the State of New York

Architect:
Skidmore, Owings, & Merrill LLP

Structural Engineer:
Leslie E. Robertson Associates, RLLP

MEP Engineer:
Jaros Baum & Bolles

Construction Management:
Turner Construction Company

Geotechnical Engineer:
Langan Engineering & Environmental Services, P.C.

Lighting:
SBLD Studio

Building Statistics:

Size: 620,000 s.f.
Stories: 14
Cost: $457 Million
Construction Dates:
February 2008 to Fall 2010
Site Restriction: Amtrak tracks cross the S-W corner of site

Architecture:

- 14 story tower containing office, classroom, and laboratory spaces.
- Grand Cascade connects existing building to the expansion project.
- Landscaped roof above cascade accommodates outdoor commons.
- Prefabricated curtain wall system with varying depth aluminum “fins”.
- Flat roof with parapets and single ply waterproofing membrane.

Mechanical:

- Cooling towers, water tanks, air-handling units, and mechanical exhaust fans are located at the penthouse level.
- Vertical mechanical shafts run through the 14 story service core to distribute air from the penthouse level.

Structural:

Foundation: Columns bear on a combination of concrete piers and caissons. The first floor system is a 6” thick two-way slab which is supported by grade beams.

Superstructure: Composite floor construction with 3” metal decking and 3 ½” lightweight concrete. Typical steel framing bay sizes are 30’-0” x 35’-9”. Perimeter plate hangers support the 6th -14th floors, which are hanging.

Lateral System: Steel braced frame core resists lateral loads. Trusses at the penthouse level support hanging floors 6-14 and transfer gravity loads to the braced frame.

Lighting & Electrical:

- 480 Y / 277 V electrical system with an emergency generator room at the penthouse level.
- Multistory cascade makes use of day lighting through skylights as well as recessed down lights to illuminate passageways.
- Walls of importance are accented with linear fluorescent wall washers.

CUNY The City University of New York

CPEP Website:
http://www.engr.psu.edu/ae/thesis/portfolios/2009/mwh174/

Dormitory Authority State of New York

Table of Contents

Executive Summary

This report is the culmination of a yearlong study performed on the John Jay College Expansion Project located in Manhattan. This academic building includes a midrise tower with classrooms, laboratories, and office spaces that reaches a maximum height of approximately 240 feet above 11th avenue. A 14 story tower is connected to the existing building with a 5 story podium which encases a "grand cascade". Amtrak tracks pass beneath the South-West corner of the site, which led to a unique structural solution of the midrise tower.

Levels 1 through 5 are transferred over the Amtrak tracks using built-up steel girders. Limited in depth to 3'-0" between the 1st floor and Amtrak tunnel, these built-up steel girders only have enough capacity to support the first 5 levels. To transfer the remaining 9 levels, trusses at the mechanical penthouse cantilever out from a braced frame core. Perimeter plates hang from the trusses and support floor framing of the 6th through 13th levels.

While this innovative solution creates some attractive architectural features, such as thin perimeter plates instead of columns, it is complicated to construct. Since the major transfer system is located at the top of the building, temporary supports and bracing must be used until the penthouse trusses are completed. Therefore, the main goal of this study was to design a more constructible transfer solution.

In this study, 6 architecturally exposed transfer trusses were designed for the 5th level. These trusses use custom built-up steel sections to transfer 55% of the perimeter columns of the midrise tower. The remaining 45% of the perimeter columns are supported at the foundation using concrete caissons embedded into bedrock.

In the existing design, heavy W14 sections are used for columns at the top of the braced frame core due to transferring gravity loads from the penthouse trusses. After new transfer trusses were design for the 5th level, the lateral force-resisting system was optimized. Lateral loads were calculated using ASCE 7-05, rather than using the NYC Building Code. Wind loads were determined to control for both strength and serviceability design considerations. Once lateral drifts were determined to be adequate for the lateral drift recommendations of ASCE 7-05, a separate analysis was performed to ensure the new design would meet the criteria - set forth by the NYC Building Code - the existing structure was designed for. By transferring gravity loads to the braced frame at the 5th level rather than at the penthouse, a savings of approximately 71 tons of steel was achieved in the braced frame columns.

Major architectural changes were made to the 5th level of the John Jay College Expansion Project to implement the transfer trusses. Floor-to-floor heights of the 5th level were increased from 20 feet to 30 feet to incorporate the interior trusses. These interior trusses were elevated 10 feet to allow building occupants to circulate beneath them. By exposing the transfer trusses using custom built-up sections, a new aesthetic was achieved for the 5th level dining and serving areas.

To determine if the ultimate goal of a more constructible structural solution was achieved, a construction management breadth study was completed. First, to

determine if the new transfer system was a viable option, a cost analysis was performed. This resulted in a total estimated cost for the transfer system of $ 6.74 million and $ 6.15 million for the existing transfer solution. These values were calculated based on the weight of steel used, and do not consider the premiums charged for the complicated construction of the existing hanging structure. Therefore, it was assumed that the two transfer systems cost about the same.

However, differences in constructability of the two transfer systems were seen when comparing construction schedules. Assuming that 40 pieces of steel could be erected per day, steel erection for the new transfer system was determined to take 3 weeks less than the existing design. While this decrease in erection time is important, it is more important to understand how this was achieved: by using less transfer trusses with less web members, and eliminating the need for temporary supports.

The difference in total superstructure construction time was increased when examining the placement of concrete decking. Due to the built-up steel girders above the Amtrak tracks having to support 14 levels of construction loads, concrete work cannot begin until penthouse trusses are complete and the tower is hanging. If concrete work begins before gravity loads of the upper levels are transferred using the penthouse trusses, the built-up steel girders would be overstressed. By transferring gravity loads at the 5^{th} level, conventional steel erection methods can be used and concrete work can be sequenced with the steel erection to approximately finish at the same time. Using the assumptions listed in this report to estimate the total superstructure construction time, the new transfer system was determined to take 6 weeks less than the existing design.

In conclusion, the new transfer system designed in this report is a viable solution. By exposing the transfer trusses at the 5^{th} level of the John Jay College Expansion Project, a more constructible design was achieved. Although there are many other considerations that may need to be taken into account when comparing construction methods, this study used time to compare constructability between the two transfer methods. This reduction in time was found to be caused by using less pieces of steel in the transfer trusses, as well as eliminating the need for temporary columns and supports, resulting in a simplified transfer solution.

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The writer is thankful to the following professionals, AE faculty, and students for their guidance throughout the year, as well as permission to use the John Jay College Expansion Project for this thesis project:

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Professor Robert Holland

AE Students

Jeremy McGrath

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Introduction

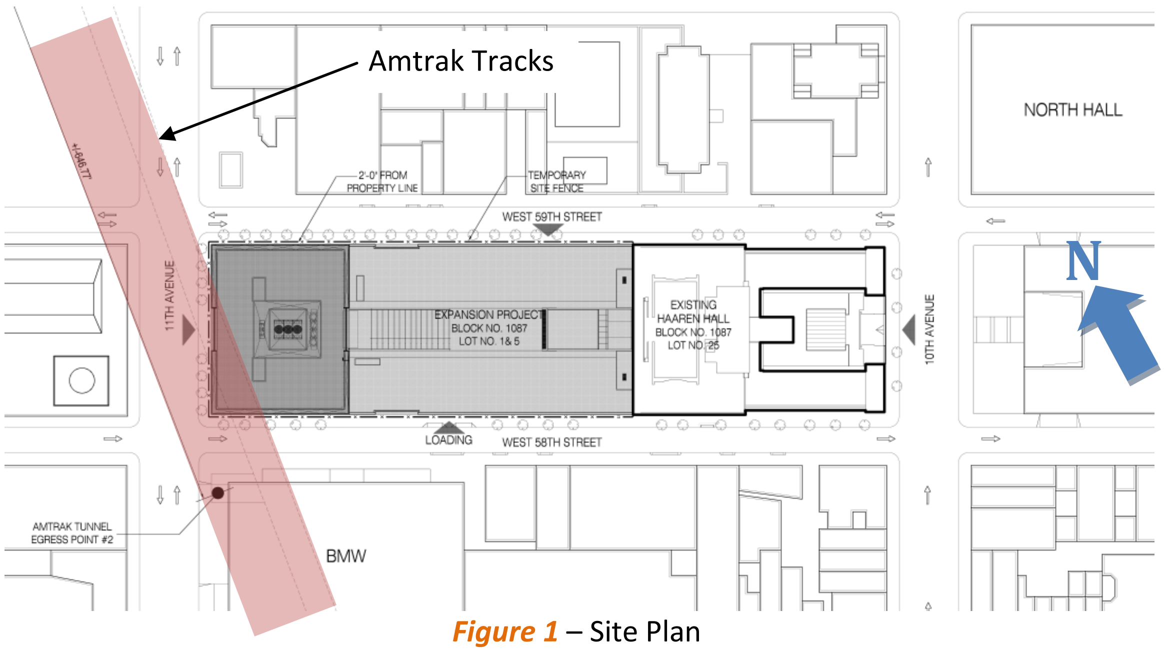


Figure 1 – Site Plan

This major expansion project in Manhattan will unify the City University of New York's John Jay College of Criminal Justice into a one block campus that will "demonstrate the transparency of justice". The design includes a mid-rise tower situated on the west side of the site, which will contain classrooms, forensic laboratories, department offices, department quads, several student lounge spaces, a "moot" courtroom, a café, and a student bookstore.

A low-rise structure connects the expansion to Haaren Hall (the existing building) and calls for a multi-level grand cascade (see Figure 2), which also serves as a main lounge space for students. The connection also contains classrooms, a black box theater, and two cyber cafes. A landscaped roof accommodates outdoor lounge and dining areas, and an outdoor commons.

Amtrak tracks cross the South-West corner of the site, which is beneath the mid-rise tower (see Figure 1 and 8). This restriction led to a unique structural solution to transfer gravity loads over the tracks. Floors 1 through 5 are transferred over the tracks using built-up steel transfer girders at the first level and floors 6 through 14 are hanging from perimeter plate hangers supported at the penthouse level by transfer trusses that are one-story tall. See Figure 5 for a load path diagram and Appendix H penthouse transfer truss information. These trusses then transfer the gravity loads to the lateral force-resisting system, which is a steel braced frame. This braced frame wraps around a centralized service core located in the 14 story tower. A braced frame is also utilized in the service core of the 5 story podium structure. See Figure 3 and Figure 4 for elevation views of the John Jay College Expansion Project.

The building envelope of the John Jay College Expansion Project consists of prefabricated curtain wall panels constructed with clear glass and frited vision glass, smooth aluminum mullions, and aluminum plate "fins" of varying depths. By varying the depth of each aluminum fin, pedestrians have a changing perspective of the façade as they pass by the building. The curtain wall is attached to the edge of the concrete slab on metal decking floor system. Typical roof construction consists of a sloped structural concrete slab with 3" rigid roof insulation covered in a water pervious fabric and topped with aggregate ballast.

Figure 2 – Rendering of the Grand Cascade

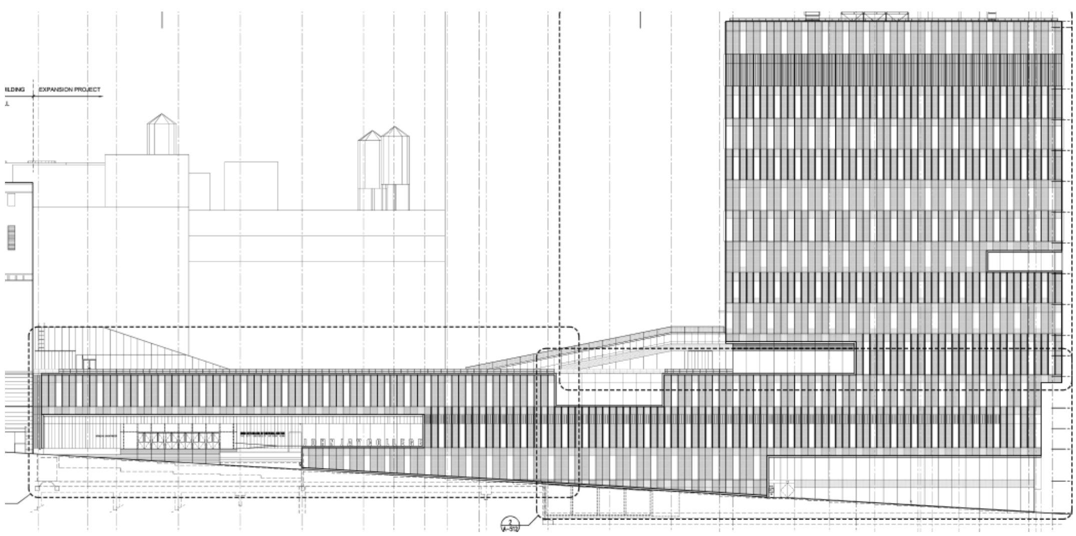


Figure 3 – North Elevation

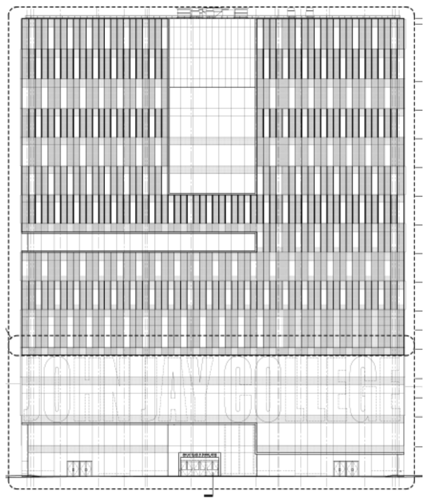


Figure 4 – West Elevation

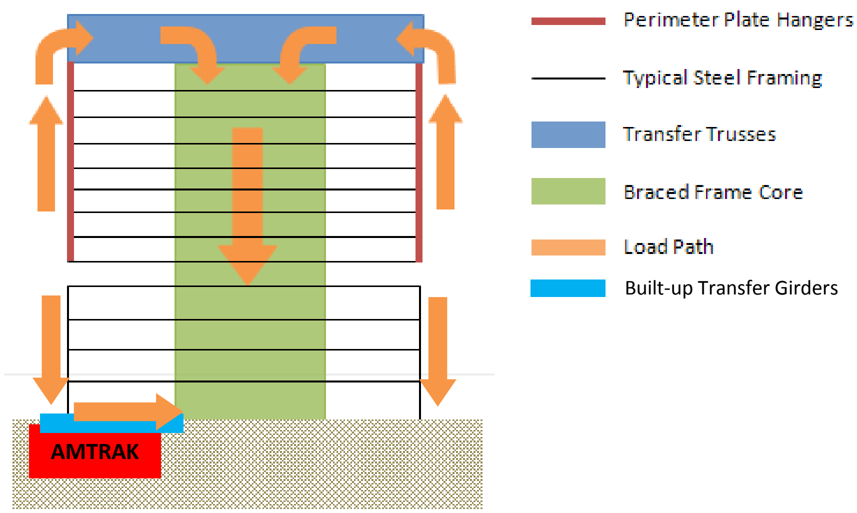


Figure 5 – Schematic Diagram of Existing Tower Design Load Path

Existing Structural Systems

Foundation

The existing design of the John Jay College Expansion Project uses two main types of foundation systems to support the structure. The Northern half and the South-Eastern corner of the building is primarily supported by drilled concrete caissons ranging from 18 to 36 inches in diameter. These caissons are embedded up to 14 feet into the bedrock below. On the South-Western corner of the site, columns are supported by reinforced concrete piers of dimensions ranging from 20 x 20 inches to 72 x 42 inches. These concrete piers are then supported by individual column footings ranging in sizes of 3 x 3 feet to 9 x 9 feet that are bearing on bedrock. See Figure 6 and 7 for locations of concrete piers and caissons.

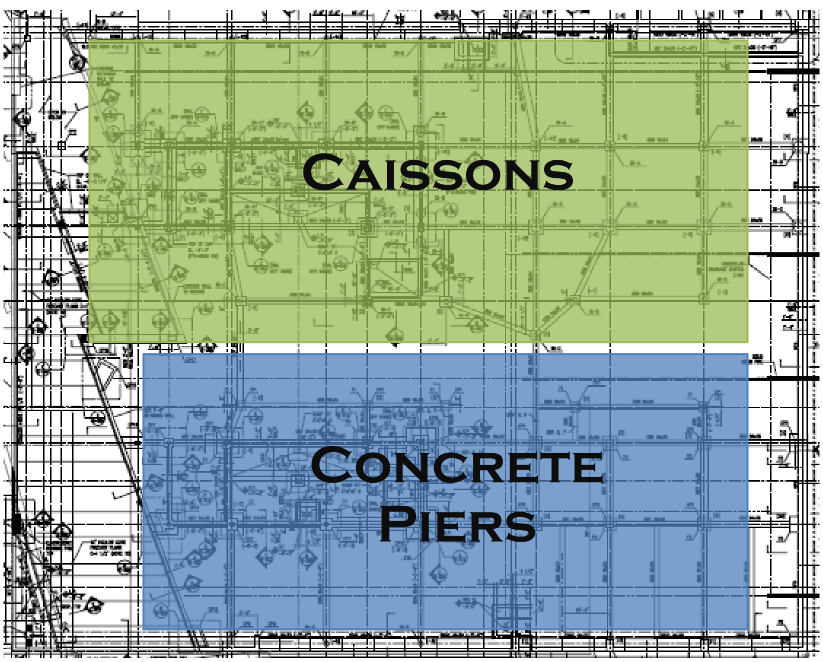


Figure 6 – West Foundation Plan

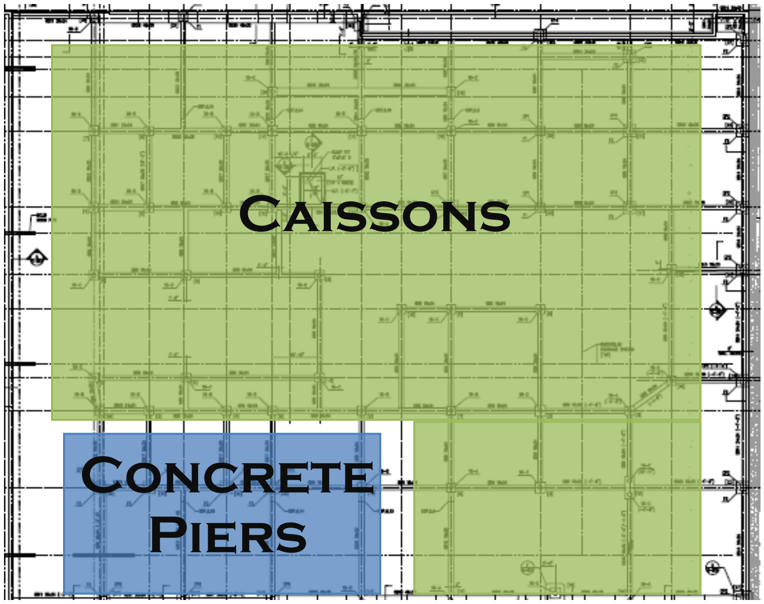


Figure 7 – East Foundation Plan

The first floor framing system is constructed using a two-way reinforced concrete slab-on-ground that is 6" thick. The slab is spanning to grade beams, which then frame into the concrete pier caps or concrete caisson caps. The perimeter of the building is enclosed with a reinforced concrete wall that varies in thickness from 12 to 20 inches. The Amtrak tracks beneath the first level of the expansion project are enclosed with 10 inch thick hollow core pre-cast planks, which have a short erection time and therefore minimize the amount of time the tracks are delayed for construction.

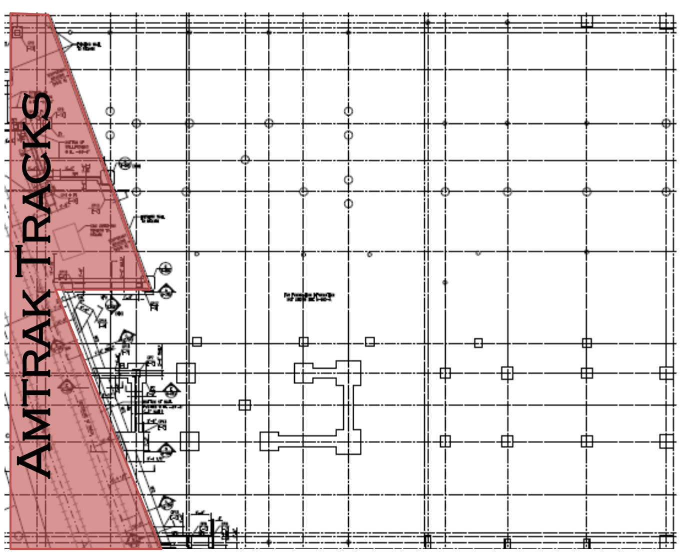


Figure 8 – Amtrak Foundation Plan

Floor System

The floor system of the John Jay College Expansion Project is a composite steel system with the most typical bay size being 30'-0" x 37'-10". 3 ½ inch light weight concrete and 3 inch metal decking typically span 12'-2" to W14x22 or W16x26 infill beams. ¾ inch diameter x 5 ½ inch long shear studs allow composite action between the concrete decking and steel beams. Infill beams span into W-shape girders of varying sizes or two back-to-back MC-shapes. Framing of the cascade, which connects the tower to the existing building (Haaren Hall), consists of W36 girders spanning 68'-4" with infill beams spaced typically at 11'-4" on center. See Appendix A for typical floor framing plans.

Columns

Typical gravity columns for the John Jay College Expansion Project are W14's. Lateral columns have a significantly heavier W14 section than the gravity columns due to the perimeter tensile loads transferring to the braced core at the penthouse level and of course due to resisting lateral loads. Perimeter plate hangers supporting the 6th through 14th floors range in size from 1 x 12 inches to 2 x 20 inches. Splices of the plate hangers occur at every two levels using 1 1/8 inch diameter A490 bolts.

Lateral Systems

The 14 story tower of the expansion project has a large centralized braced frame core (see Figure 9 and Appendix I). This braced frame surrounds the vertical shafts of the building, such as elevator shafts, stairwells, mechanical shafts, and plumbing. Columns of the braced frames are heavy W14 sections and the beams are typically W16 sections. HSS 6x6x3/8 are typically used for diagonal bracing at the 13th level and HSS 8x8x3/8 are used for the diagonal bracing at the 1st level. Reinforced concrete walls span between the caissons and concrete piers at the foundation of the lateral system.

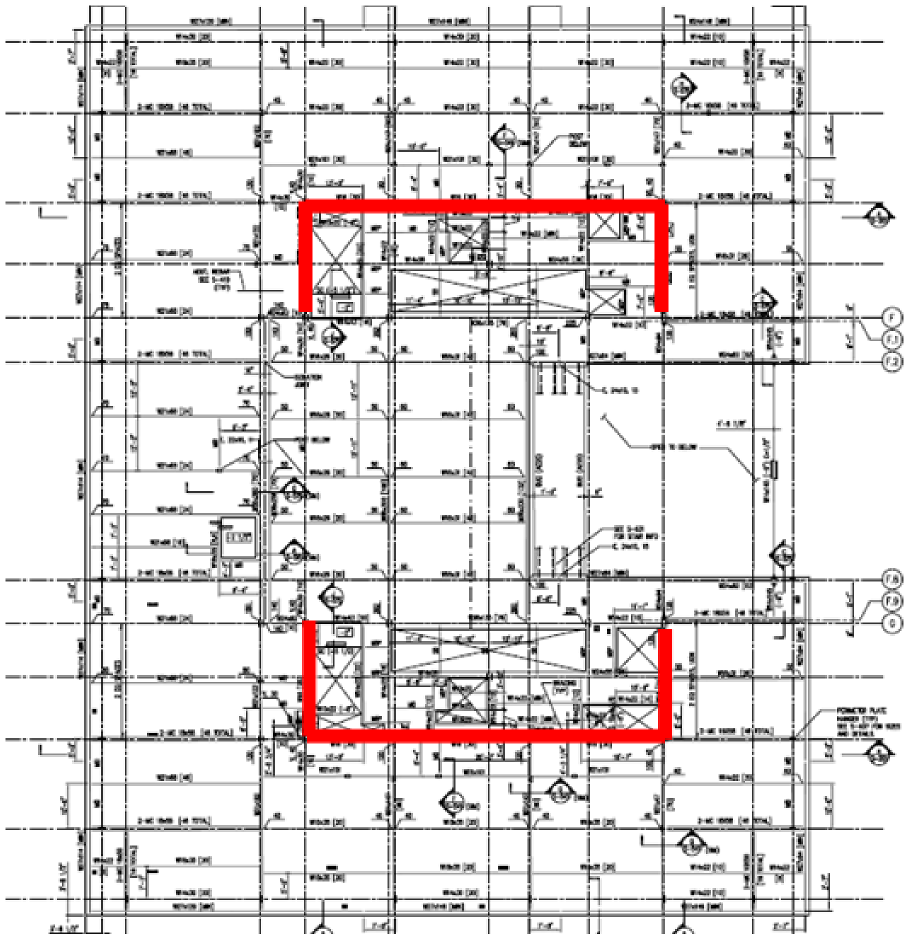


Figure 9 – Location of the Lateral Force-Resisting System (Braced Frames) in the 14 Story Tower

The lateral system for the 5 story cascade is also a braced frame which encases the buildings vertical circulation passages and shafts (see Figure 10). Columns of these braced frames are lighter W14 sections than the 14 story braced frame and the beams are W16x31's and W21x94's. Diagonal braces are typically 2 L6x4's with varying thicknesses.

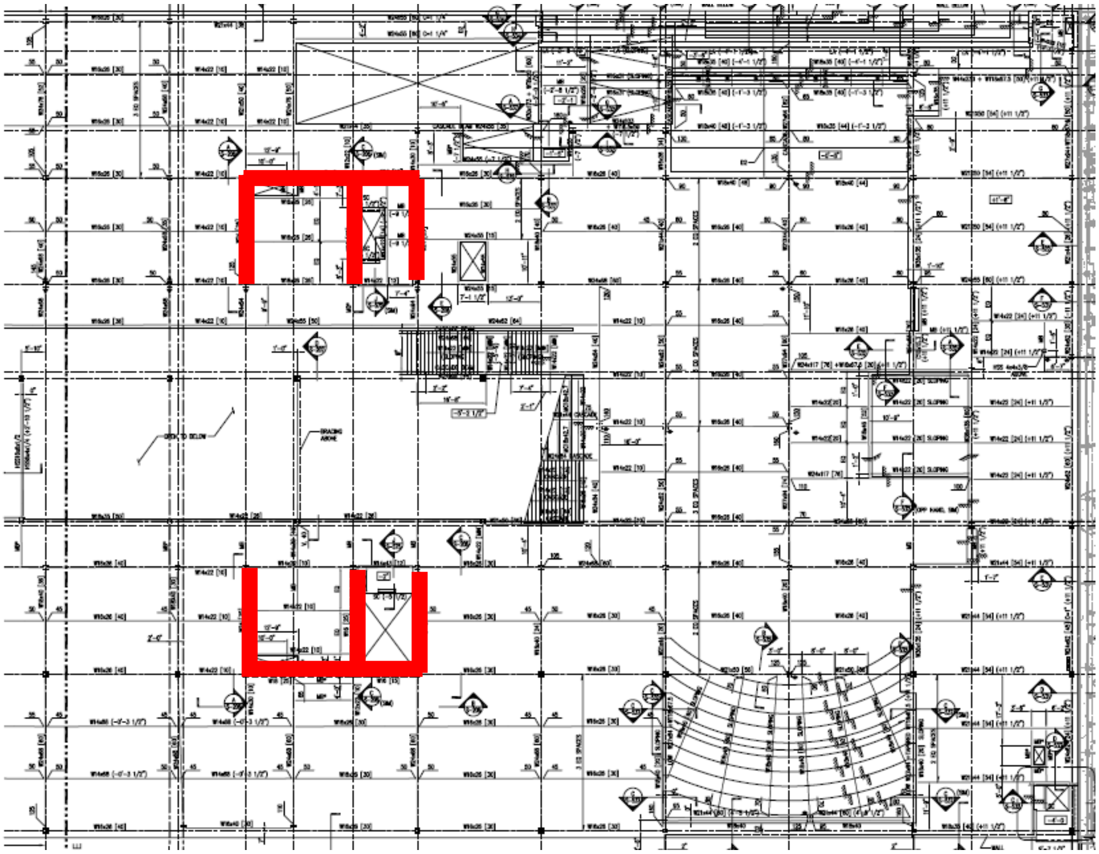


Figure 10 – Location of the Lateral Force-Resisting Systems (Braced Frames) in the 5 Story Cascade

Problem Introduction

The present design of the John Jay College Expansion Project calls for a unique solution to transfer gravity loads over the Amtrak tracks beneath the building. Built-up steel transfer girders at the first level are limited to 3 feet in depth and therefore can only support 5 levels of gravity loads. To transfer the additional 9 levels of gravity loads over the train tracks, trusses at the penthouse level, which cantilever from the braced frame core, are utilized. This allows the 6th through 13th levels to be supported by 1 to 2 inch thick steel plates at the perimeter of the building, which hang from the penthouse level trusses. Level 5 has no perimeter columns or plate hangers (see Figure 5).

While this solution to transfer gravity loads over the Amtrak tracks offers unique architectural features such as a column-less 5th level, it also creates several difficulties. One issue with the existing transfer system is the columns in the braced frame core. These columns support the gravity loads from their tributary areas, but they also must support the perimeter plate hanger gravity loads transferred from the trusses at the penthouse. This results in very heavy W14 column sections at the top and bottom of the braced frame core.

The John Jay College Expansion Project is also delayed and behind in construction. This is not entirely caused by the complicated structural system, but due to the transfer system being at the penthouse level, temporary columns and bracing must be used to construct each floor supported by plate hangers. Construction must also be closely monitored to ensure the built-up transfer girders at level one are not overstressed because they will be supporting all floors until the transfer trusses are built at the penthouse level.

Problem Solution

With the intentions of simplifying the construction process, a new transfer system will be studied in detail for the John Jay College Expansion Project. This new system includes removing the transfer trusses from the penthouse level, and redesigning them to fit on the 5th or 6th levels. These trusses will be partially exposed and will provide a new aesthetic to the spaces which they share. By moving the trusses to one of these levels (see Figure 11), there is no need to hang the above floors. This will simplify the construction process by eliminating the need for temporary columns and bracing, as well as eliminating the need to monitor the stresses in the built-up steel girders at the 1st level during construction.

After studying the behavior of the existing braced frames, it was found that the coupling action in the penthouse level caused by the transfer trusses was used to reduce the drift in the North-South direction. By moving the transfer trusses to the 5th or 6th level, the coupling action would be removed from the top of the braced frames and the lateral drifts would change. Therefore, members of the existing braced frames will also need to be resized by performing a detailed lateral analysis using ETABS.

Lateral columns will also be significantly reduced in size at the top of the braced frame core.

Breadth Studies

The structural system is not the only aspect of the John Jay College Expansion Project affected by redesigning the transfer system. Due to the change in location of the transfer trusses the steel erection sequence will be altered, and therefore a construction management breadth will be studied. This study will consist of creating and comparing a new construction schedule for erecting the structure to the existing schedule. Cost comparisons will also be made between the proposed structural system and the existing system.

Another area of study that will be necessary to implement the proposed structural system into the building will be an architectural breadth. The current 5th floor is used as a cafeteria with kitchen, student dining areas, and faculty dining areas. This space is primarily open, so the trusses will need to be designed to avoid interfering with the occupant circulation. The 6th floor primarily contains chemistry and physics laboratories. If the trusses are found adequate for this level, the floor plan will be arranged so that the trusses are mostly within the partitions. See Appendix B for the existing floor plans. Selecting the 5th or 6th floor to transfer loads will depend on which floor plan can successfully accommodate the trusses without having a negative impact on the architecture. Figure 11 displays a building section with the proposed transfer solution.

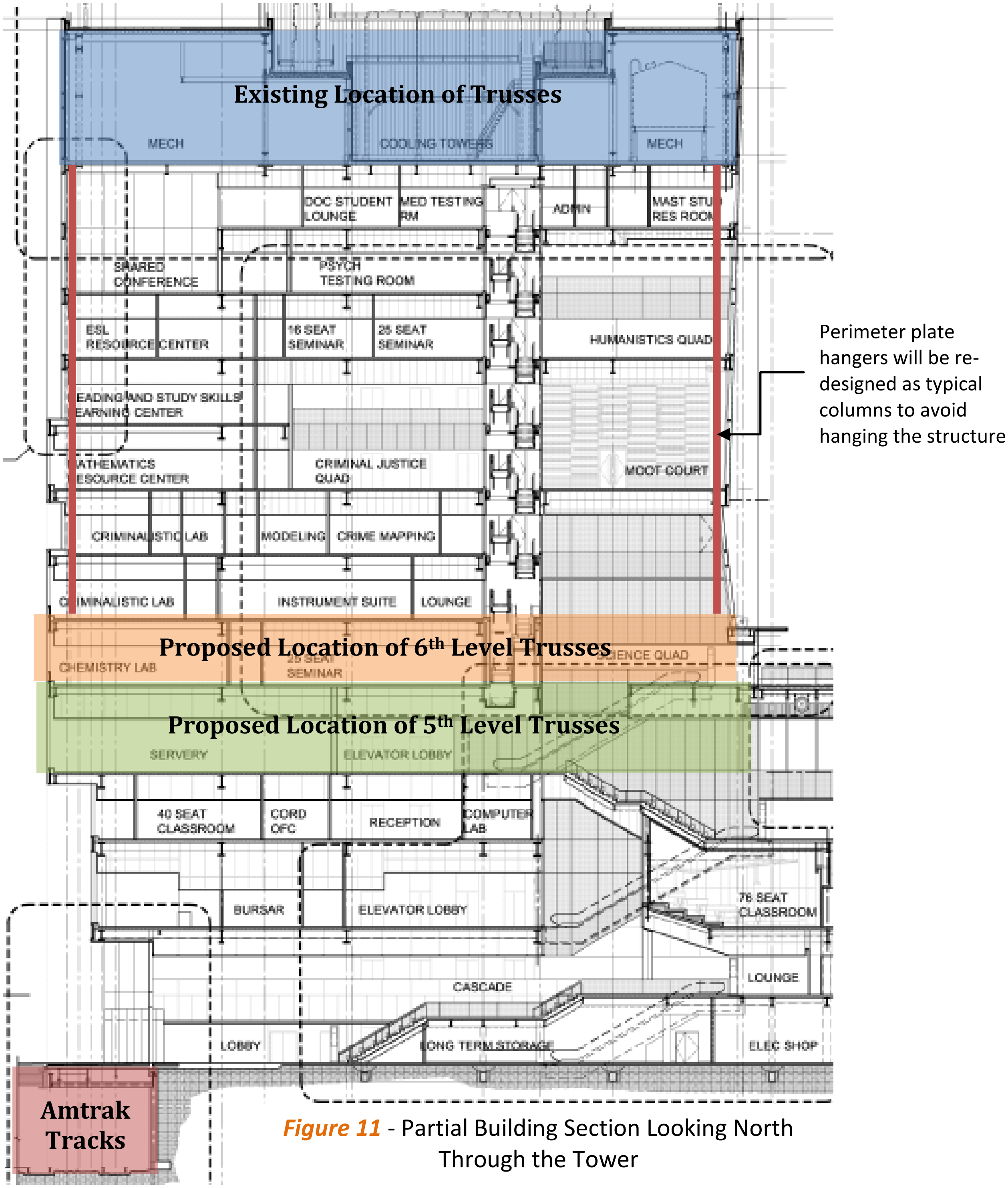


Figure 11 - Partial Building Section Looking North Through the Tower

Design Goals

The overall design goal of this project is to design architecturally exposed transfer trusses, which will allow typical steel framing construction to be used to efficiently transfer gravity loads over the Amtrak tracks beneath the building. Additional goals are as follows:

- Create a more constructible transfer solution than the existing design
- Design a series of transfer trusses which are architecturally exposed to building occupants
- Design custom built-up steel shapes for exposed truss members
- Develop transfer trusses that compliment the architecture of the building
- Ensure that transfer trusses do not interfere with the John Jay College Expansion Project core, such as elevators, means of egress, and mechanical shafts
- Use ETABS to perform an in-depth lateral analysis and to develop an efficient design for the braced frame core

Structural Depth Study

The Structural Depth Study includes the schematic design, analysis, and final design of a new transfer system for the John Jay College Expansion Project as defined in the problem statement. This required all perimeter columns of the tower to be designed and a detailed lateral analysis was performed to study the impact of a new transfer system on the existing braced frames. Final conclusions are based on the performance, constructability, cost, schedule, and architectural impacts of each transfer system.

Codes, References, and Criteria

Original Design Codes

National Model Code

The Building Code of the City of New York with latest supplements

Structural Standards

ASCE 7-02, Minimum Design Loads for Buildings and other Structures (used for cladding wind loads)

Design Codes

AISC –LRFD 1999, Load and Resistance Factor Design Specification for Structural Steel Buildings

AISC-ASD 1989, Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (used for the design of Braced Frames and Penthouse level Transfer Trusses)

ACI 318-95, Building Code Requirements for Structural Concrete

Original Deflection Criteria

Lateral Deflections

Total building sway deflection for wind loading is limited to H/500

Total building sway deflection for seismic loading is limited to H/260

Interstory shear deformation for wind loading is limited to (story H)/400

Interstory shear deformation for seismic loading is limited to (story H)/260

Thesis Codes

National Model Code

2006 International Building Code

Structural Standards

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Design Codes

Steel Construction Manual 13^{th} edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Thesis Deflection Limitations

Gravity Deflections

Gravity deflections of transfer trusses will be handled by cambering 80 percent of the dead load

Lateral Deflections

Total building drift and interstory drift for wind loading is limited to H/400

Total building drift and interstory drift for seismic loading is limited to $0.015h_{sx}$

Materials

Structural Steel:

- Wide Flanges and Tee Shapes……………………………….ASTM A572 or A992, Grade 50
- Channels and Built-Up Sections………………………………………..ASTM A572, Grade 50
- Pipes…………………………………………………ASTM A501 or A53, Types E or S, Grade B
- Tubes……………………………………………………………………………ASTM A500 Grade B
- Angles…………………………………………………………………………………………..ASTM A36
- Connection Plates…………………………………………………………………………ASTM A36

Metal Decking:

- 3” and 2” Composite Deck…………………………………..Fy = 40 ksi, 20 Gage Minimum

Headed Shear Studs:

- ¾” diameter……………………………………………………………………....ASTM A108, Type B

Welding Electrodes:

- E70XX……………………………………………………………………tensile strength of 70 ksi

High Strength Bolts:

- ¾” and 7/8” Bolts…………………………………………………………………………ASTM A325
- 1” and 1 1/8” Bolts……………………………………………………………………..ASTM A490

Cast-in-Place Concrete:

- Caisson Caps and Grade Beams…………………………………………………..f’c = 4000 psi
- Caissons and Piers…………………………………………………………………f’c = 6000 psi
- Slabs on Ground and Footings………………………………………………….f’c = 4000 psi
- Walls………………………………………………………………………………………..f’c = 4000 psi
- Slabs on Deck…….f’c = 4000 psi – light weight concrete unless noted on drawings

Reinforcement:

- Reinforcing Bars………………………………………………………...ASTM A615, Grade 60
- Caisson #18 Reinforcing Bars……………………………………………ASTM A615, Grade 75
- Welded Wire Fabric:
  - D4.0 and larger……………………………………………………..ASTM A497, Fy = 70 ksi
  - W4.0 and smaller………..ASTM A185 (Fy = 65 ksi $\geq$ W1.2, Fy=56 ksi < W1.2)
- Deformed Bar Anchors……………………………………………………ASTM A496, Fy = 70 ksi

Gravity Loads

The following gravity loads were determined by using ASCE 7-05. These loads were used to determine preliminary design forces for the perimeter columns, the transfer trusses, and the columns of the braced frames. Later in the design phase, more accurate gravity loads were used from the structural design criteria sheet provided by the structural engineer of record to directly compare transfer systems. These gravity loads are listed in Table 1.

Construction Dead Loads

Typical floor Construction	
3" Metal Decking: 20 Gage Minimum	3 psf
3 ½" Lightweight Concrete Slab (115 psf)	48 psf
Allowance for Self Weight of Steel Framing	7 psf
Total CDL for Floor System Design:	51 psf
Total CDL for Seismic Calculations:	58 psf

Mechanical and Mezzanine floor Construction	
3" Metal Decking: 20 Gage Minimum	3 psf
4 ½" Normal weight Concrete Slab	75 psf
Allowance for Self Weight of Steel Framing	7 psf
Total CDL for Floor System Design:	78 psf
Total CDL for Seismic Calculations:	85 psf

Superimposed Dead Loads

Typical floor Construction	
Fireproofing	2 psf
Finishes	5 psf
Partitions	20 psf
Ceiling	5 psf
Mech. & Electrical Distribution	5 psf
Total SDL:	37 psf

Live Loads

Classrooms	40 psf
Offices	50 psf
Lobbies & Corridors	100 psf
Cascade	100 psf
Stairs	100 psf
Assembly areas (moot court and quad spaces)	60 psf (fixed seats) 100 psf (movable seats)
Roof	20 psf

Heavy Mechanical Equipment Loads

6th, 7th, & 8th Floor: Increased loads in laboratory spaces	100 psf
Penthouse Mezzanine Level	63 kips (Total load)
Penthouse Level	853 kips (Total Load)

Wall Loads

Curtain Wall	15 psf
1'-6" Thick Reinf. Conc. Wall (@ Foundation)	225 psf

Table 1 – Original Design Gravity Loads (PSF)

OCCUPANCY CATEGORY	FIRST FL OVER AMTRAK (INSIDE THE BUILDING)	FIRST FL OVER AMTRAK (OUTSIDE THE BUILDING)	PRECAST PLANK	CLASSROOMS, MEETING ROOMS	TIERED CLASSROOMS	OFFICES	LOBBIES, CORRIDORS, CAFETERIA	LABORATORIES	CASCADE SEATING	CASCADE STEPS	MECHANICAL IN CORE, LIGHT STORAGE	TOILET ROOMS	MECHANICAL, MEZZANINE FLOORS (3RD AND 5TH FLOOR MEZZANINE)
CONSTRUCTION DEAD LOAD (PSF)	120	120	65	50	50	50	50	50	158	180	50	50	75
SUPERIMPOSED DEAD LOAD (PSF)													
FIREPROOFING	2	2		2	2	2	2	2	2	2	2	2	2
TOPPING SLAB	45	45	–	–	75	–	–	–	–	–	–	–	–
FINISH ALLOWANCE	13	13	–	2	2	2	13	2	2	2	–	25	–
PARTITIONS	L2	–	–	12	20	20	L2	L2	L2	L2	–	20	–
CEILING	–	–	L9	5	5	5	5	5	5	5	5	5	5
MECHANICAL/ ELECTRICAL	–	–	–	5	5	5	5	10	5	5	5	5	5
ROOFING/ INSULATION	15	15	–	–	–	–	–	–	–	–	–	–	–
SDL TOTAL (PSF)	75	75	–	26	109	34	25	19	14	14	12	57	12
TOTAL DEAD LOAD (PSF)	195	195	65	76	159	84	75	69	172	194	62	107	87
LIVE LOAD (PSF) (SEE NOTES)	100 L1	600 L4, L8	30	60 L1	60 L1	50 L1	100 L1	100 L1, L7	100 L1	100 L1	150 L3	40 L1	150 L3

OCCUPANCY CATEGORY	FIFTH FLOOR CAMPUS COMMONS	CAMPUS COMMONS GRAND STAIRS	PENTHOUSE LEVEL	PENTHOUSE LEVEL MECHANICAL AREAWAY	PENTHOUSE MEZZANINE	TOWER ROOF	METAL PAN STAIRS	CURTAIN WALL LOADS (OF VERTICAL WALL SURFACE)	CANOPIES	KITCHEN FREEZER ROOM	BALISTIC ROOM BLACK BOX THEATER	CHEMICAL SOLVENT STORAGE	MOOT COURT, HUMANITIES QUAD	FLOOR OVER SWITCHGEAR ROOM
CONSTRUCTION DEAD LOAD (PSF)	75	75	100	100	75	75	50	–	–	50	50	100	50	75/45
SUPERIMPOSED DEAD LOAD (PSF)														
FIREPROOFING	2	2	2	2	2	2	–	–	–	2	2	2	2	2
TOPPING SLAB	–	–	–	–	–	–	–	–	–	45	50	–	–	38
FINISH ALLOWANCE	SEE A/S-001	40	3	3	–	–	–	–	15	5	–	2	35	2
PARTITIONS	–	–	–	–	–	–	–	–	–	20	L2	L2	L2	L2
CEILING	5	5	5	5	–	–	–	–	10	5	15	5	5	5
MECHANICAL/ ELECTRICAL	5	5	5	5	30	30	–	–	–	10	10	10	5	10
ROOFING/ INSULATION	15	15	–	50	50	50	–	–	5	5	5	–	–	–
SDL TOTAL (PSF)	–	67	15	65	92	92	0	15	30	92	82	19	47	57
TOTAL DEAD LOAD (PSF)	–	142	115	165	167	167	50	15	30	142	132	119	97	132/102
LIVE LOAD (PSF) (SEE NOTES)	100 L1, L4	100 L1	150 L3	150 L3	150 L3	30 L1	100	–	30 L6	80 L1	80 L1	100 L5	100 L1	100 L1

Schematic Truss Design

The first step of designing a new transfer system for the John Jay College Expansion Project was to sketch several possible locations of truss configurations. After determining that either the 5th or 6th floors would be used as the transfer level, trusses were drawn in plan for each level to identify architectural constraints. Several considerations were taken into account early in the design process. These considerations were:

- amount of steel that would be exposed,
- disruptions of the floor plan,
- effect on the braced frame core,
- penetrations through the truss including doors and elevators,
- floor-to-floor heights, and
- the spaces the trusses would be visible in.

Blue represents the trusses and red represents the braced frame core in the schematic truss configuration plans presented in the following sections.

Column Location

The framing plans and plate hanger locations in the original design of the John Jay College Expansion project required two plate hangers in the corners to allow each floor to hang on the plate hangers (see Figure 12a). By using typical steel framing, there is no need for two columns in the corners of the building (see Figure 12b). Therefore, for this study one corner column will be used and the transfer system will be designed accordingly. Figure 18 displays the location of the perimeter columns of the 14 story tower. Columns shown in aqua must be transferred over the Amtrak tracks, which are represented in Figure 18 in orange. Green columns are permitted to extend down to the foundation.

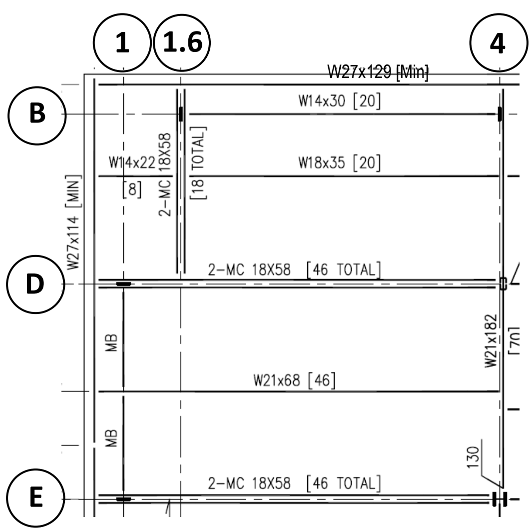


Figure 12a – Existing Design Corner Framing Plans

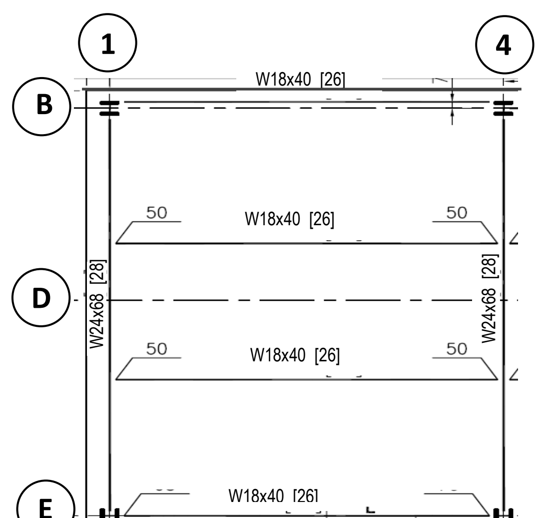


Figure 12b – New Design Corner Framing Plans

Truss Configurations

Truss Configuration 1

The first truss configuration was using the same layout of trusses at the 6th level that were used at the penthouse level (see Figure 13). Floor-to-floor heights for the 6th level are only 15 feet, and therefore axial forces in the top and bottom chords of the trusses would increase. Preliminary floor plan sketches were performed and only a few partitions would have to be moved to accommodate the trusses spanning from the braced frame core to the perimeter of the building. However, major complications arose when looking into the details of the escalators. Therefore, a truss cannot span across the East side of the core.

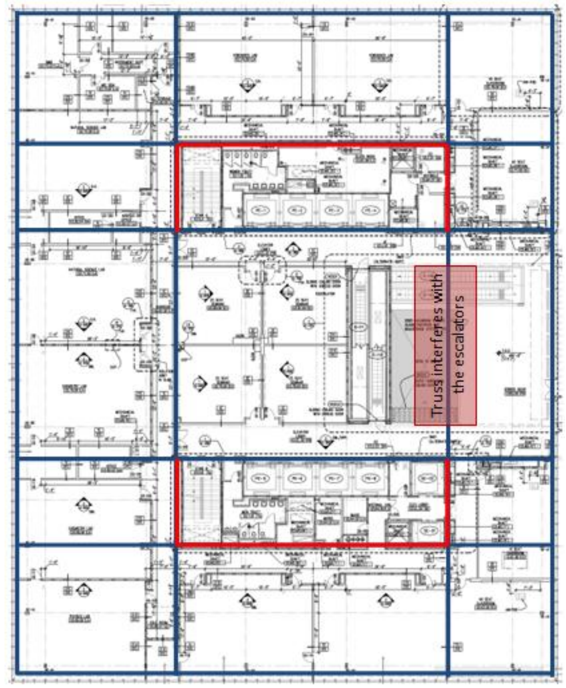


Figure 13 - Truss Configuration 1

Truss Configuration 2

Truss configuration 2 only carries the columns that must be transferred over the Amtrak tracks. This method reduces the amount of trusses needed at the 6th level to transfer the gravity loads to the core, reduces the gravity loads that must be carried by the columns of the braced frame core, and increases the amount of gravity columns for the entire height of the tower. This configuration also creates a large amount of bending in the braced frame core of the tower due to gravity loads only being transferred from the West side of the tower. One major downfall to using this transfer option is that the only trusses which could be architecturally exposed are the perimeter trusses in

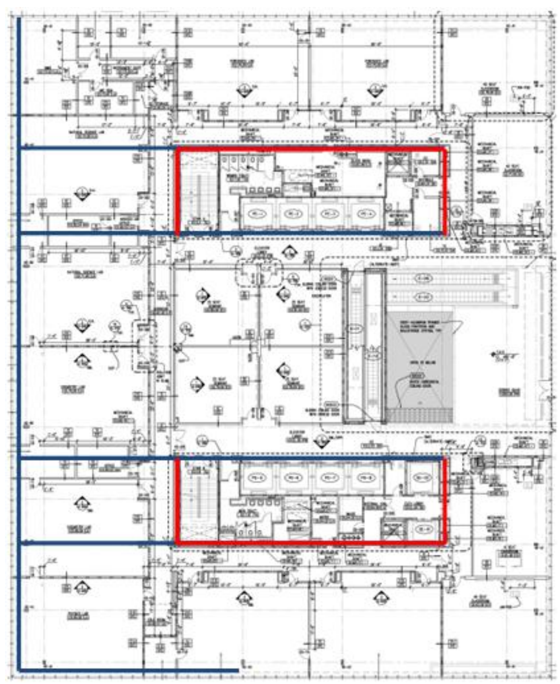


Figure 14 - Truss Configuration 2

Figure 14. With that being said, the spaces where this truss is visible are chemistry and physics laboratories and the view would only be available to those in the laboratory spaces.

Truss Configuration 3

Truss configuration 3 applies truss configuration 2 at the 5th level (see Figure 15). Level 5 is the student, faculty, and staff dining facility and has a greater opportunity to expose the truss. This space is shared by all students, faculty, and staff who will use the building, so this is the most rationale space to place an exposed transfer system. A preliminary gravity analysis was performed and it was determined that this configuration was not "balanced". By only transferring the columns to the West of the braced frame core, a substantial amount of bending was induced in the braced frames.

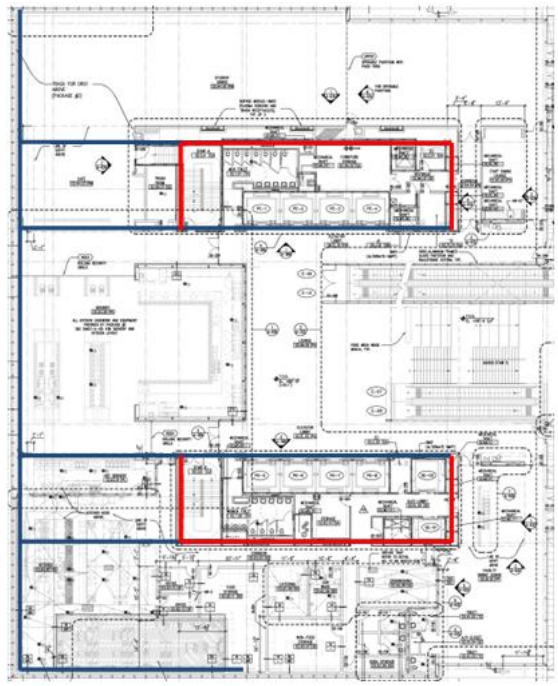


Figure 15 - Truss Configuration 3

Another downfall to this design is that the original floor plan is very open and this configuration requires interior trusses to span across the open space to support the perimeter truss. Pedestrian circulation within the student dining would be disrupted and would have a negative impact on the space.

Truss Configuration 4

Truss configuration 4 makes use of the 5th floor mezzanine level to allow interior trusses to span from the braced frame core to the perimeter trusses. Figure 16 displays the perimeter trusses at level 5, Figure 17 shows the perimeter trusses supported by trusses which span to the braced frame core at the 5th floor mezzanine level, and Section 1 shows a schematic section of the truss configuration. The 5th floor mezzanine level is 10 feet above the 5th level and therefore the trusses would be elevated 10 feet above the 5th level, allowing the building occupants to pass beneath. The space within the trusses spanning to the perimeter is occupied by the 5th floor mechanical mezzanine. The mezzanine is accessible through the stairs within the building's core and is bypassed by the elevators. Therefore, these trusses can span through the braced frame core without accommodating openings for the elevator doors. After performing a gravity analysis of truss configuration 3, it was determined that it was necessary to balance the load to reduce the bending induced into the braced frame core. Therefore, columns to the East of the braced frame core were also transferred.

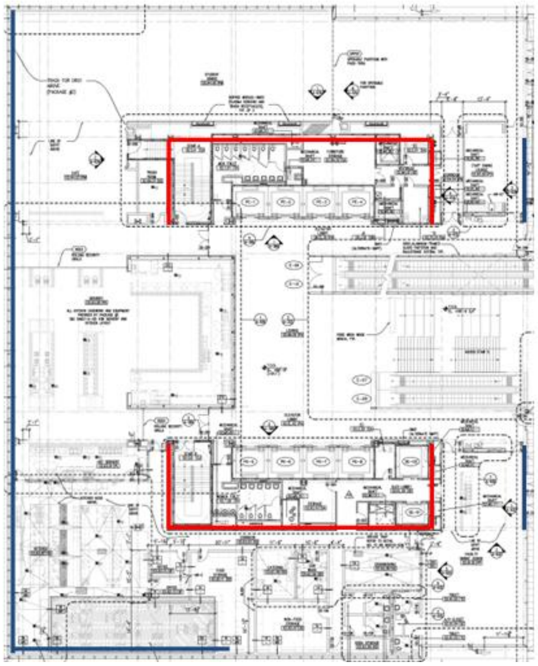


Figure 16 - Truss Configuration 4
5th Level

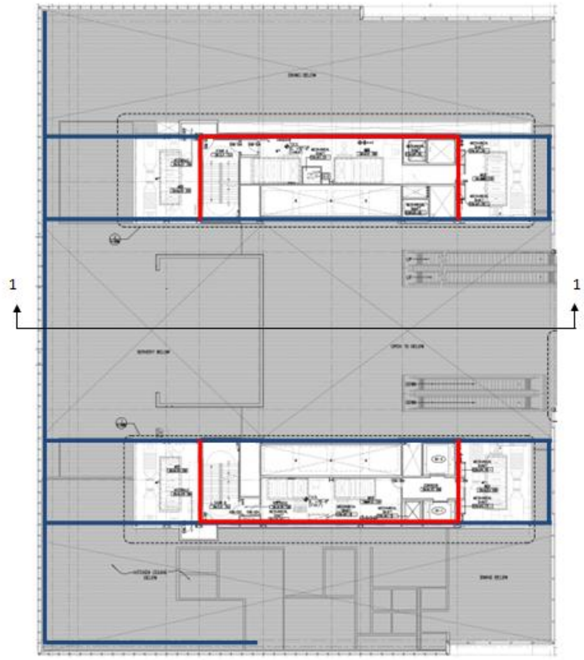
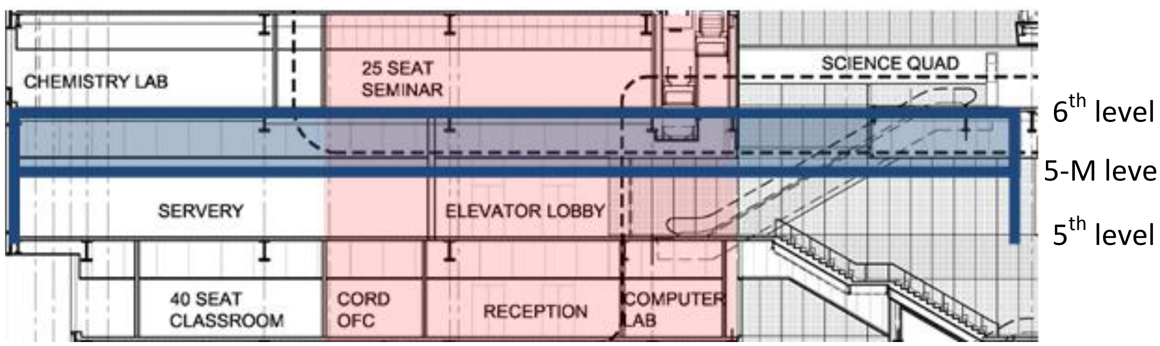


Figure 17 – Truss Configuration 4
5th Level Mezzanine



Section 1 - Looking North at Truss Configuration 4

Truss Configuration Conclusion

After evaluating each possible truss configuration based on the criteria listed above, it was determined that truss configuration 4 would be the best solution to efficiently transfer the gravity loads over the Amtrak tracks. Truss configuration 4 does not disrupt the flow of circulation through the 5th level and it is a public space where everyone can see the exposed trusses.

Gravity Analysis and Design

The new transfer system for the John Jay College Expansion Project will only be transferring columns which are directly above the Amtrak tracks beneath the West side of the building site. Figure 18 displays the new location of all perimeter columns for a typical level in the 14-story tower. Green rectangles represent columns that will now be supported at the foundation and the aqua rectangles represent columns which will be transferred to the braced frame core. Columns C1, C2, C3, C4, and C7 are the columns directly above the site restriction. Columns C5 and C6 are transferred in an attempt to "balance" the moment and shear induced into the braced frame core by the heavy transferring gravity loads.

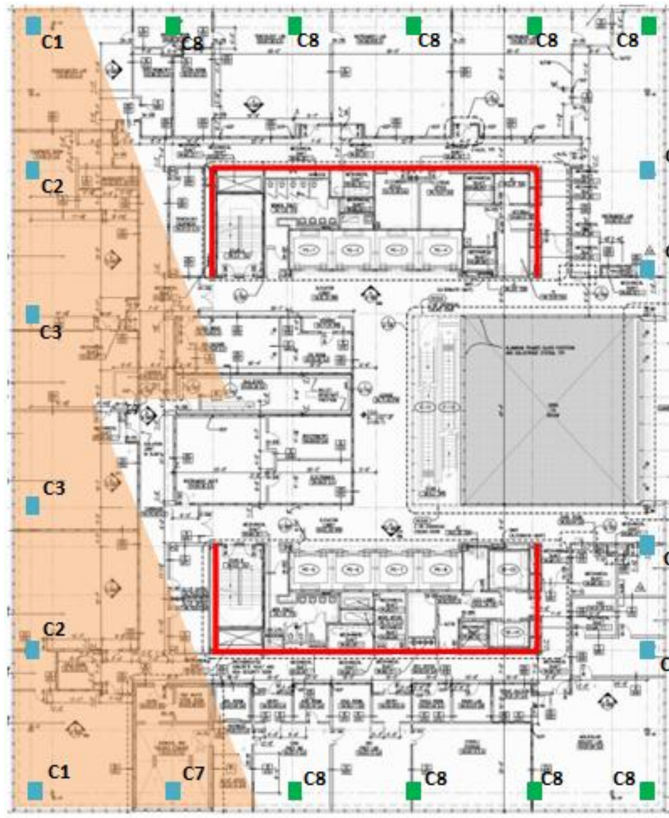


Figure 18 – Location of New Columns (Amtrak Tracks Shown in Orange)

Perimeter Column Design

By moving the transfer system to the 5th level typical steel framing can be used for all levels. Therefore, all of the perimeter columns had to be designed to replace the perimeter plate hangers. Aqua columns in Figure 18 are being transferred by the transfer trusses to the braced frame core and are designed for the 6th through roof levels, while the green columns are not being transferred and are designed to support the 1st through roof levels. A detailed gravity load takedown sample calculation is available in Appendix E. Columns were designed using live load reductions according to ASCE 7-05 Section 4.8. All columns were designed using LRFD with a controlling load combination of 1.2D + 1.6L. See Table 2 through Table 8 for design summaries and comparisons to the original plate hanger designs. Columns are assumed to be spliced at every two levels, which is how the original plate hangers and columns were designed. Columns were assumed to be laterally braced at each level.

Table 2 – Column C1 Design

C1		Thesis Design				Original Design			
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	99	15							
14	232	15	W14x48	332	OK	1x12	1.25x16	32	32
13	293	15							
12	353	15	W14x53	369	OK	1x12	1.25x16	32	32
11	414	15							
10	474	15	W14x61	543	OK	1x10	1x16	26	26
9	535	15							
8	624	15	W14x74	667	OK	1x10	1.5x10	25	25
7	713	15							
6	802	15	W14x90	1000	OK	0.75x8	0.75x10	13.5	14

Table 3 – Column C2 Design

C2		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	184	15						
14	429	15	W14x61	543	OK	1.5x18	27	92
13	535	15						
12	640	15	W14x74	667	OK	1.5x18	27	92
11	746	15						
10	851	15	W14x90	1000	OK	1.5x18	27	92
9	957	15						
8	1121	15	W14x109	1220	OK	1.75x12	21	71
7	1285	15						
6	1449	15	W14x132	1480	OK	1x14	14	48

Table 4 – Column C3 Design

C3		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	211	15						
14	493	15	W14x61	543	OK	2x20	40	136
13	615	15						
12	737	15	W14x90	1000	OK	2x20	40	136
11	859	15						
10	981	15	W14x90	1000	OK	2x20	40	136
9	1103	15						
8	1291	15	W14x120	1340	OK	2.75x12	33	112
7	1479	15						
6	1668	15	W14x159	1810	OK	1.25x16	20	68

Table 5 – Column C5 Design

C5		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	147	15						
14	343	15	W14x53	369	OK	1.25x18	23	77
13	428	15						
12	513	15	W14x61	543	OK	1.25x18	23	77
11	598	15						
10	683	15	W14x82	736	OK	1.25x16	20	68
9	767	15						
8	898	15	W14x90	1000	OK	1.25x14	18	60
7	1030	15						
6	1161	15	W14x109	1220	OK	1x10	10	34

Table 6 – Column C6 Design

C6		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	223	15						
14	520	15	W14x68	608	OK	1.5x18	27	92
13	647	15						
12	774	15	W14x90	1000	OK	1.5x18	27	92
11	901	15						
10	1028	15	W14x99	1100	OK	1.25x18	23	77
9	1155	15						
8	1353	15	W14x132	1480	OK	1.25x16	20	68
7	1552	15						
6	1750	15	W14x159	1810	OK	1.25x14	18	60

Table 7 – Column C7 Design

C7		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate (in. x in.)	A_s (in^2)	S.W. (plf)
Roof	164	15						
14	383	15	W14x61	543	OK	2x20	40	136
13	477	15						
12	572	15	W14x68	608	OK	2x20	40	136
11	667	15						
10	761	15	W14x90	1000	OK	2x20	40	136
9	856	15						
8	1002	15	W14x99	1100	OK	1.75x20	35	119
7	1149	15						
6	1295	15	W14x120	1340	OK	1.5x16	24	82

Table 8 – Column C8 Design

C8		Thesis Design				Original Design		
Level	Design Load (kips)	L_b (ft)	Section	Capacity (kips)	Design	Plate/Column (in. x in.)/(Section)	A_s (in^2)	S.W. (plf)
Roof	161	15						
14	377	15	W14x61	543	OK	1.75x18	31.5	107
13	470	15						
12	562	15	W14x68	608	OK	1.75x18	31.5	107
11	655	15						
10	748	15	W14x90	1000	OK	1.75x18	31.5	107
9	841	15						
8	985	15	W14x90	1000	OK	1.5x16	24	82
7	1129	15						
6	1273	15	W14x120	1340	OK	1x14	14	48
5	1417	15						
4	1502	15	W14x145	1650	OK	W14x74	--	74
3	1588	15						
2	1674	15	W14x159	1810	OK	W14x74	--	74

The majority of these perimeter columns require more steel than the original hanging design due to the need to resist buckling from the compressive forces versus having to resist a pure tensile force in the original design.

Truss Design

A gravity load takedown was performed for each of the new columns listed above for the design of the transfer trusses. Point load locations are shown in Figure 19 and are summarized in Table 9.

Table 9 – Gravity Load Takedown Summary

Load	DL (kips)	LL (kips)	P_u (kips)
P1	427	182	804
P2	748	345	1450
P3	866	393	1668
P4	470	195	876
P5	603	274	1162
P6	895	424	1753
P7	672	306	1296

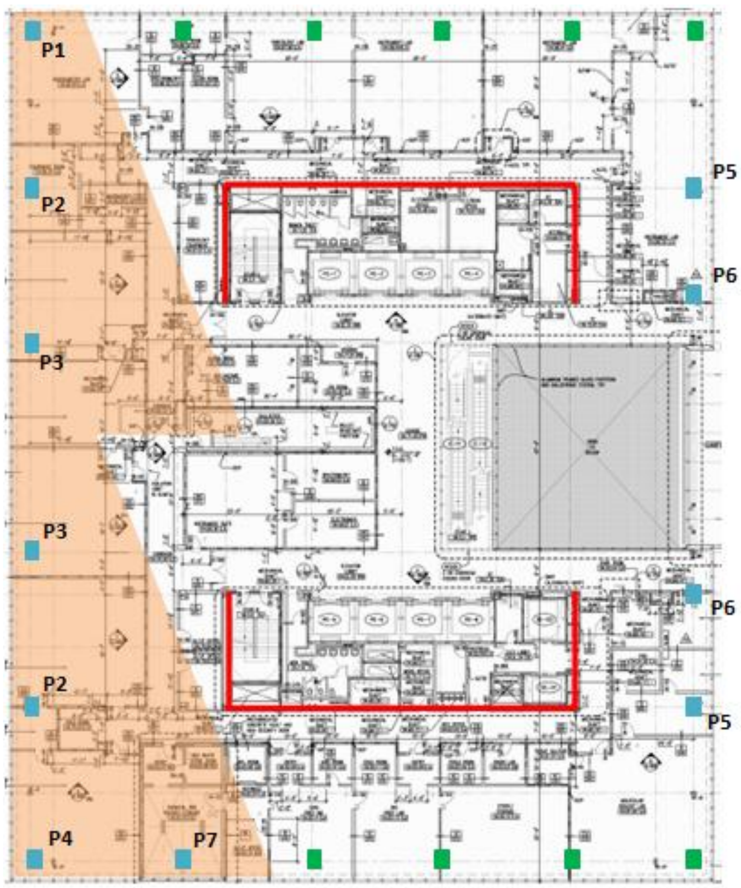


Figure 19 – Gravity Load Takedowns

Schematic Truss Elevations

Each of the following truss members were oriented to allow diagonal web members to resist major tensile forces. Placement of the truss columns were determined by the locations of columns being transferred. See Figure 20 for a plan view of the truss layout. Figure 21 is looking West at Truss 1, Figure 22 is looking North at Truss 2, Figure 23 is looking North at Truss 3, and Figure 24 is looking North at Truss 4. It should be noted that after analyzing the trusses shown below with a floor-to-floor height of 20 feet the axial forces in the top and bottom chords were extremely large and therefore the floor-to-floor height of level 5 was increased to 30 feet. For more information regarding this increase in floor-to-floor height, please see the Architectural Breadth Study.

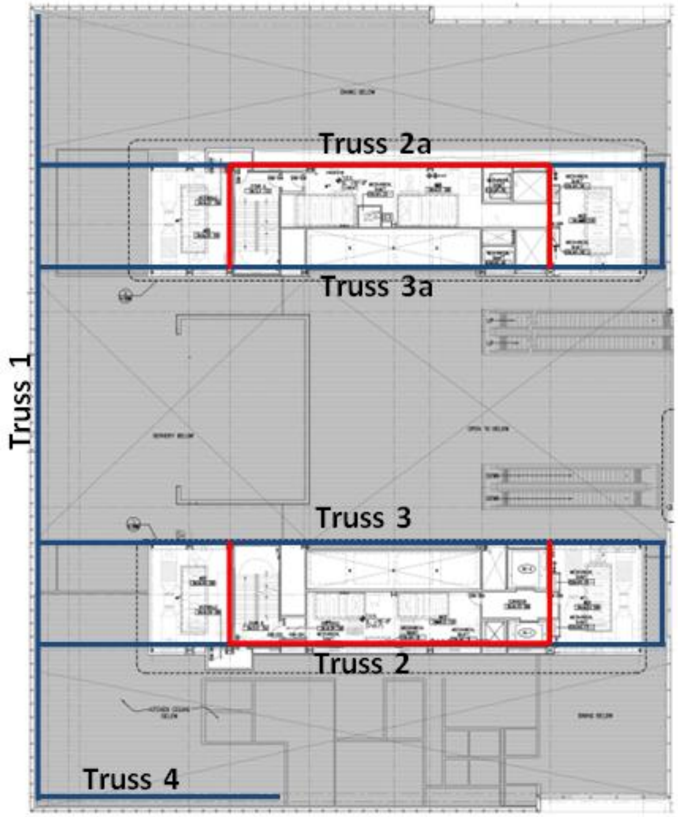


Figure 20 – Truss Nomenclature (Note: Truss 2a and Truss 3a designs are identical to Truss 2 and Truss 3, but have different cambers to control deflections.)

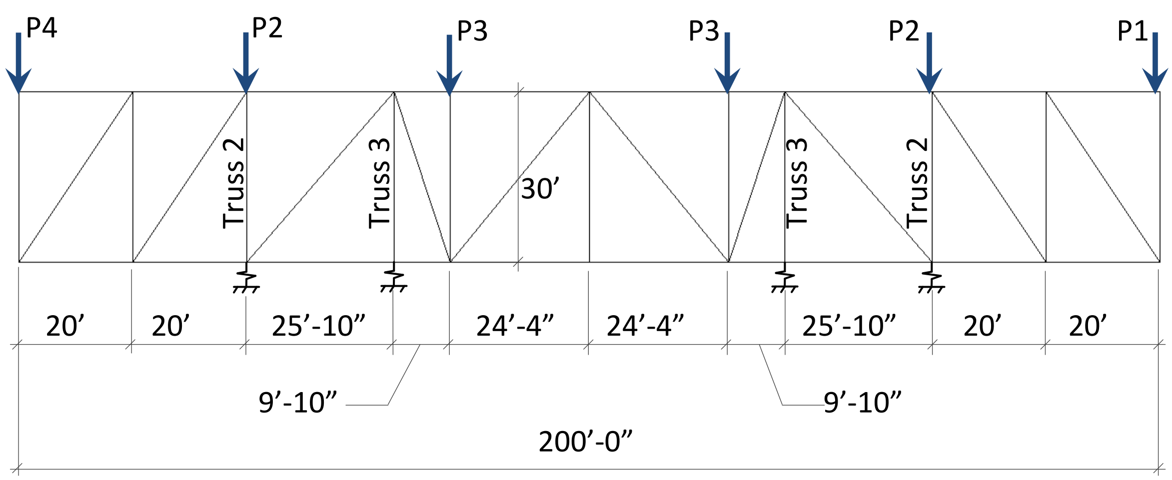


Figure 21 – Truss 1 Elevation

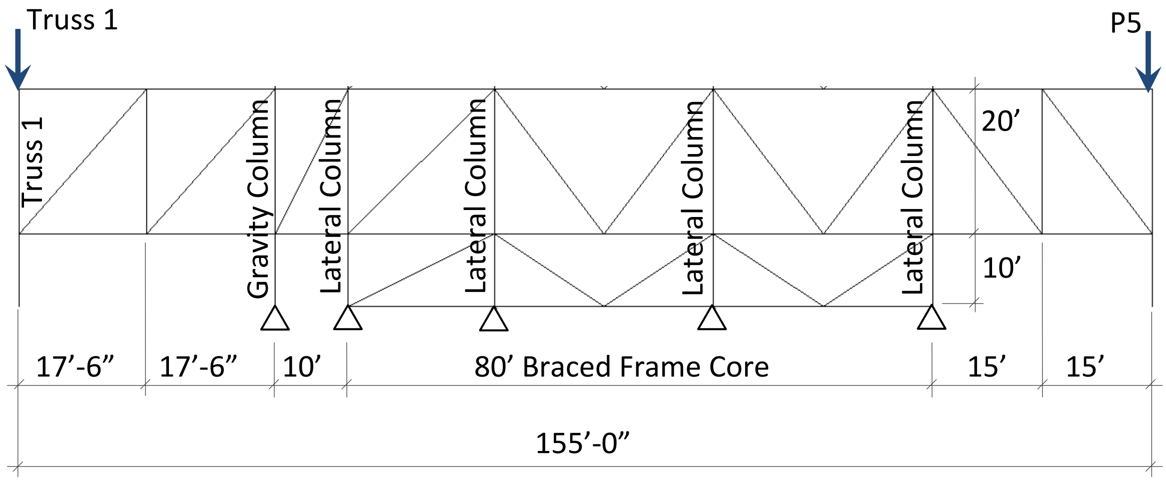


Figure 22 – Truss 2 Elevation

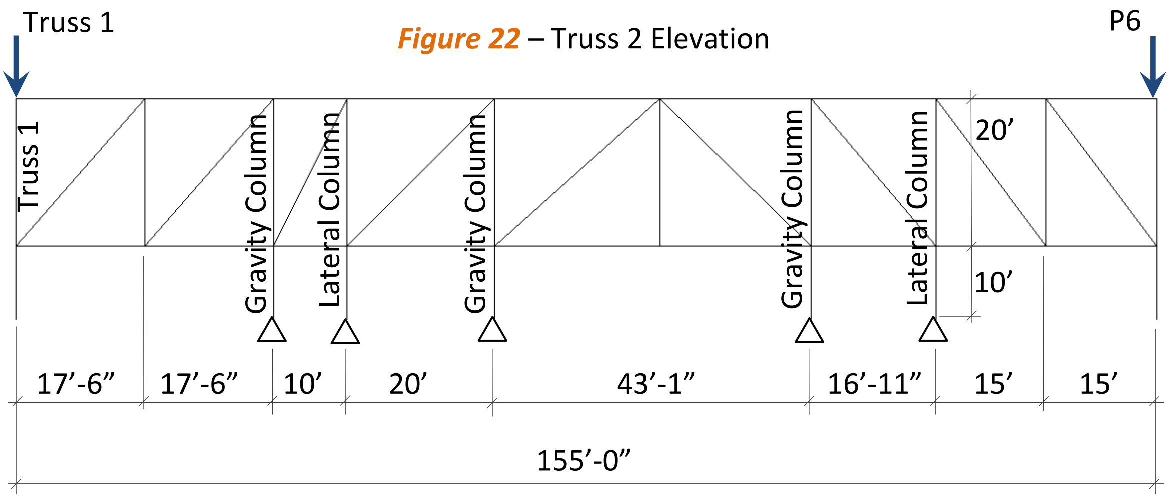


Figure 23 – Truss 3 Elevation

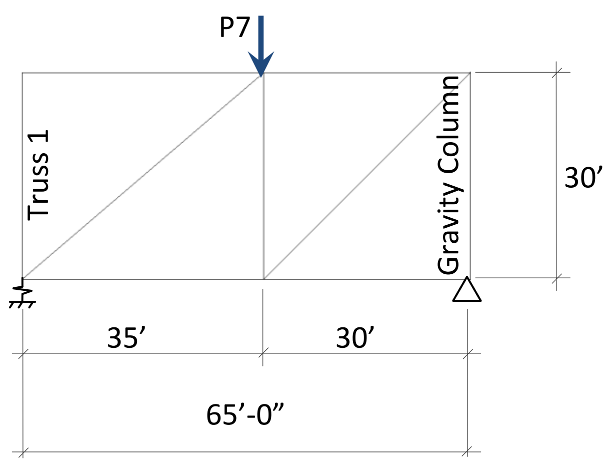
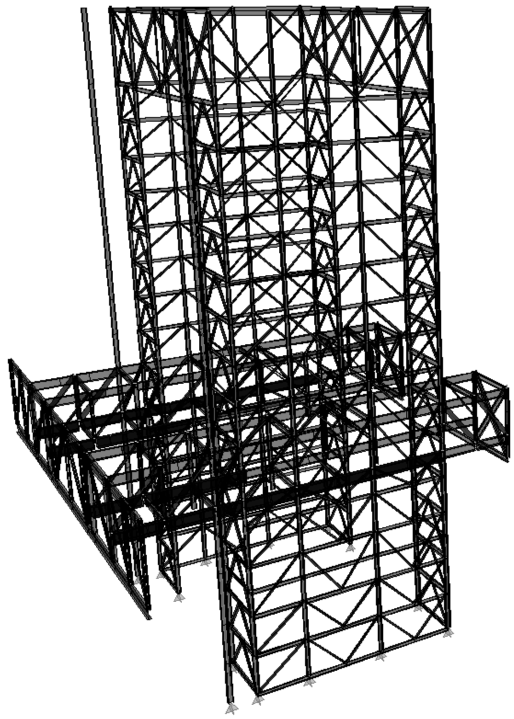


Figure 24 – Truss 4 Elevation

ETABS Gravity Model



An ETABS model was created to analyze the transfer system described above for gravity loads. The braced frame core of the tower was included, as well as the trusses shown above. The following assumptions were used in the ETABS gravity model:

- Web members are pinned at each end
- Top and bottom chords are continuous
- Axial forces in top and bottom chords DO NOT transfer into the floor diaphragms
- Un-braced lengths of chord members are taken as the length between column web members

This in-depth gravity analysis was also required to determine accurate deflections of each truss to determine the appropriate cambers.

Truss 1 was analyzed by hand to verify the axial forces obtained in the ETABS model (hand calculations are available upon request). Each member was assumed to be pinned at each end. Since Trusses 2 and 3 cantilever out from the braced frame core to Truss 1, support reactions for Truss 1 were taken directly from the ETABS model to simplify the analysis procedure. Forces were then distributed throughout the truss using the joint method. The results from the hand analysis of Truss 1 are comparable to the axial forces from the detailed gravity analysis using ETABS, which are available in Appendix F.

Truss Design Procedure

The new transfer option for the John Jay College Expansion Project was not only for structural reasons, but also for its architectural features. After researching several architecturally exposed transfer systems, and speaking with engineers in the industry, several possible sketches were produced displaying custom member shapes and connection details. Among these, parallel steel plates were the most appealing due to aesthetics, connections, fabrication of members, and the ability to maximize the capacity of the compressive members by increasing the space between the parallel plates while using a minimal amount of steel. Parallel plates in compression are provided with connector plates spaced at intervals to avoid buckling. Top and bottom chords were chosen to be a combination of built-up box sections and W-Shapes with filler plates welded between flanges to appear as a box-shape. See Figure 25 for a sketch of the desired truss connection.

Parallel plate compressive members were designed according to Specification Chapter E of the 13th Edition AISC Manual of Steel Construction. Table B4.1 was used to ensure all compressive members were compact. A modified KL/r factor was determined using equation E6-2:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\,\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2}$$

where:

$\left(\frac{KL}{r}\right)_o$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors
r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling
α = separation ratio, $h/(2r_{ib})$
h = distance between centroids of individual components perpendicular to the member axis of buckling

After determining the modified KL/r factor, the members were designed as a typical built-up column section according to Section E3 of the AISC Specification. Tension web members were designed using Chapter D of the AISC Specification. Tension member design was controlled by rupture when assuming a 6 inch diameter hole for the pin connection shown in Figure 25.

Built-up box sections were used where W-shapes did not provide enough capacity and were designed using Chapter F7 of the AISC Specification. Built-up box sections were used for chord members of Truss 2 and 3, and are subjected to bending and axial forces. Therefore, these members were designed according to Chapter H1 and meet the requirements of equation H1-1a or H1-1b. Table B4.1 was also used to ensure all compressive chord members were compact. All truss members were designed using Load and Resistant Factor Design with the governing ASCE 7-05 load combination equal to 1.2D + 1.6L.

It should be noted that the truss connections were not designed for this study. However, an extensive amount of consideration was given to the connection details when designing each truss. These connections have a large impact on the aesthetics of the exposed trusses, the shape (cross section) of individual web and chord members, and also impact the construction of the transfer system. After sketching several connection details, it was determined that the best connection solution for the transfer trusses was that shown in Figure 25. Single plates are used for the web members in tension and two parallel plates are used for the web members in compression. The compression web member plates are spaced so that the tension web members and a connection plate can fit between them. The connection plate is then welded to the top or bottom chords of the truss. Top and bottom chords were chosen to be W-shapes with filler plates welded between the flanges to create a box shape. W-shapes were chosen over built-up box members to allow the web transfer forces to the top and bottom chords (as opposed to the bottom flange of a built-up box section with stiffener plates). A large single pin can then be used to connect the web members to the plate. Although this custom connection will be expensive, it reduces on site construction time and also requires no field welds for web members.

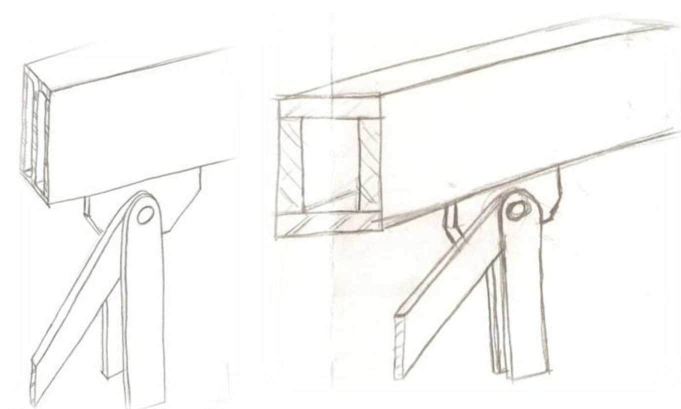


Figure 25 – Sketches of Desired Truss Connections

Truss 1 Design

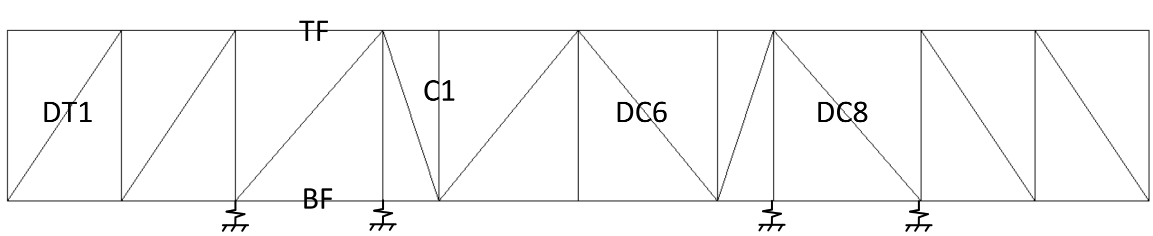


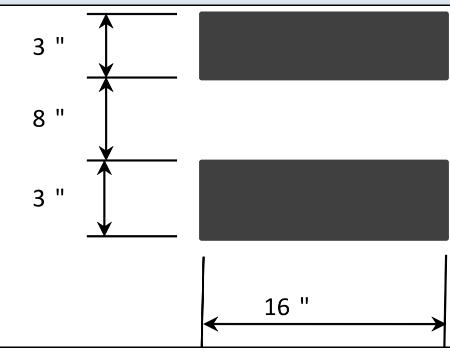
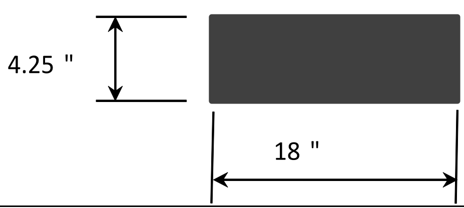
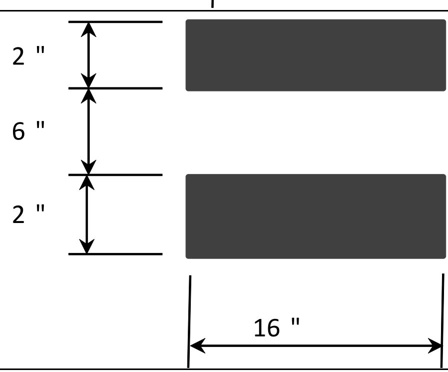
Figure 26 – Truss 1 Design Members

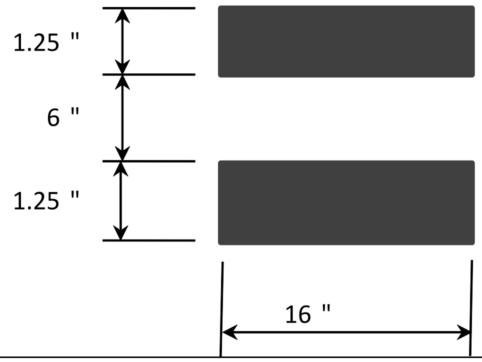
Table 10 – Truss 1 Design Forces

Member	Pu (k)	Mu (ft-k)
DT1	2430 (T)	0
C1	1960 (C)	0
DC6	900 (C)	0
DC8	450 (C)	0
TF	2700 (T)	1960
BF	2610 (C)	1620

Figure 26 displays the design members of Truss 1. Appendix F displays all design forces for Truss 1, and Appendix G shows sample calculations for the design of Truss 1 members. Table 10 provides the design forces for the members of Truss 1, and Table 11 summarizes the members designed for Truss 1.

Table 11 – Design Summary of Truss 1 Members

Member	Design
C1	 Stitching Spacing = 24 in P_u= 1960 kips ϕP_n= 2768 kips OK As= 96.0 in^2 S.W.= 327 PLF
DT1	 P_u= 2430 kips ϕP_n= 2486 kips OK As= 76.5 in^2 S.W.= 260 PLF
DC6	 Stitching Spacing = 12 in P_u= 900 kips ϕP_n= 1481 kips OK As= 64.0 in^2 S.W.= 218 PLF

DC8	 Stitching Spacing = 12 in P_u= 450 kips ϕP_n= 505 kips OK As= 40.0 in^2 S.W.= 136 PLF
TF	W40x362 w/ Filler Plates
BF	W40x362 w/ Filler Plates

Truss 2 Design

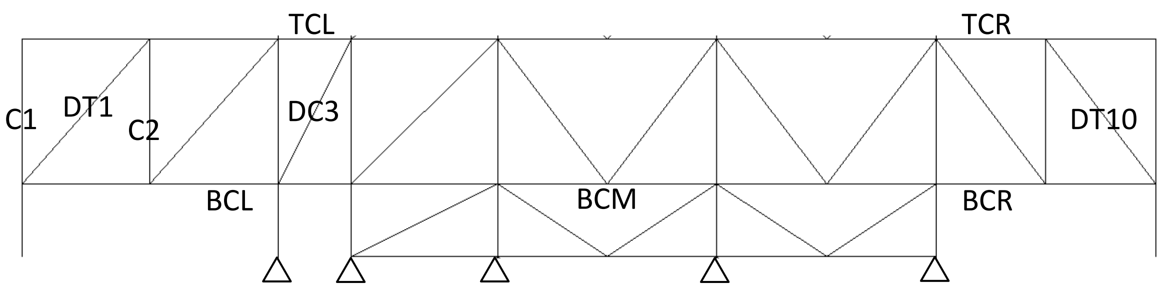


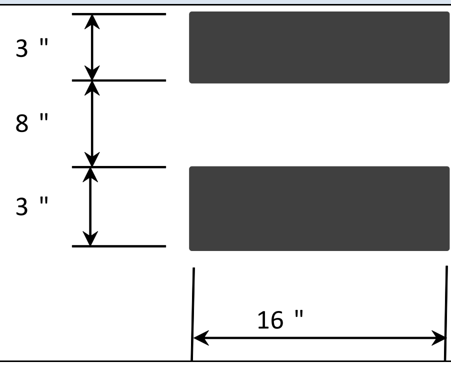
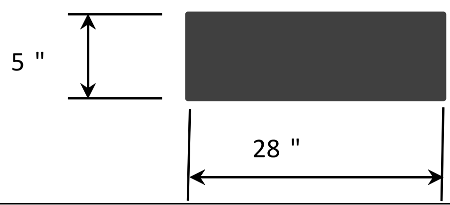
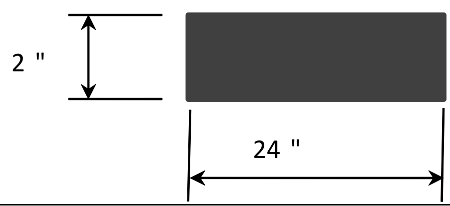
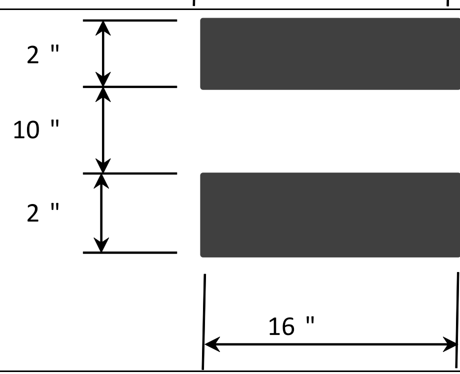
Figure 27 – Truss 2 Design Members

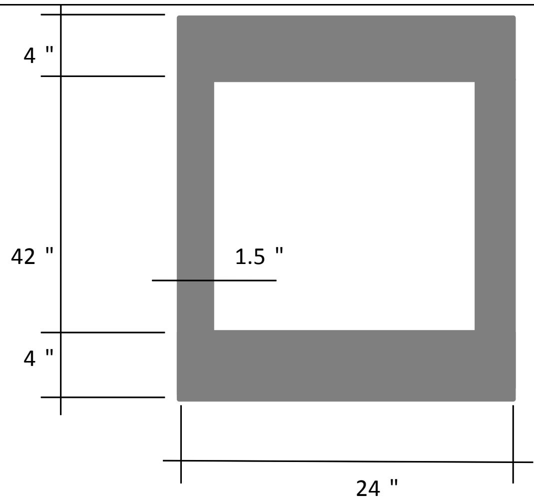
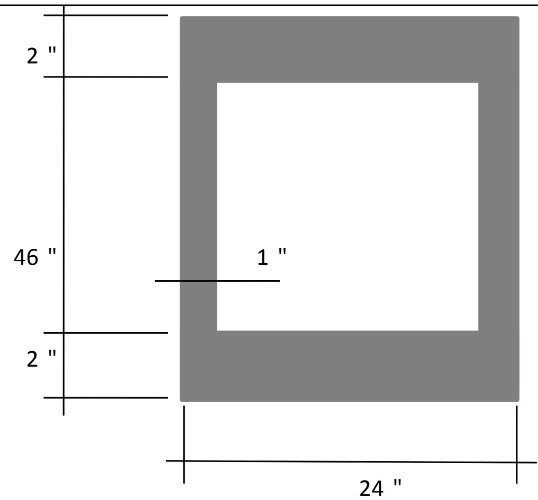
Table 12 – Truss 2 Design Forces

Member	Pu (k)	Mu (ft-k)
C1	3410 (C)	680
C2	3370 (C)	0
DC3	270 (C)	0
DT1	4850 (T)	0
DT10	1470 (T)	0
TCL	5800 (T)	4500
TCR	1200 (T)	1300
BCL	6000 (C)	9500
BCM	4000 (C)	1500
BCR	2500 (C)	2000

Truss 2 cantilevers out from the braced frame core, supports Truss 1 to the West, and supports Columns 5 and 6 to the East. Each end of Truss 2 is exposed (see Architectural Breadth Study). Appendix F displays all design forces for Truss 2, and Appendix G shows detailed calculations for the design of Truss 2 members. Table 12 provides the design forces for the members of Truss 2, and Table 13 summarizes the members designed for Truss 2.

Table 13 – Design Summary of Truss 2 Members

Member	Design
C2	 Stitching Spacing = 12 in P_u= 3370 kips ϕP_n= 4111 kips OK As= 96.0 in^2 S.W.= 327 PLF
DT1	 P_u= 4850 kips ϕP_n= 5363 kips OK As= 140.0 in^2 S.W.= 476 PLF
DT10	 P_u= 1470 kips ϕP_n= 1755 kips OK As= 48.0 in^2 S.W.= 163 PLF
DC3	 Stitching Spacing = 12 in P_u= 270 kips ϕP_n= 2294 kips OK As= 64.0 in^2 S.W.= 218 PLF

BCL		Pu= 6000 kips ϕPn= 13255 kips Mu = 9500 ft-k ϕMn= 23913 ft-k H1-1a = 0.81 OK As= 318 in^2 S.W.= 1082 PLF
BCM		Pu= 4000 kips ϕPn= 7904 kips Mu = 1500 ft-k ϕMn= 14008 ft-k H1-1a = 0.60 OK As= 188 in^2 S.W.= 640 PLF
TCL	W36x800 w/ Filler Plates	
BCR	W36x395 w/ Filler Plates	
TCR	W40x199 w/ Filler Plates	
C1	W36x441 w/ Filler Plates	

Truss 3 Design

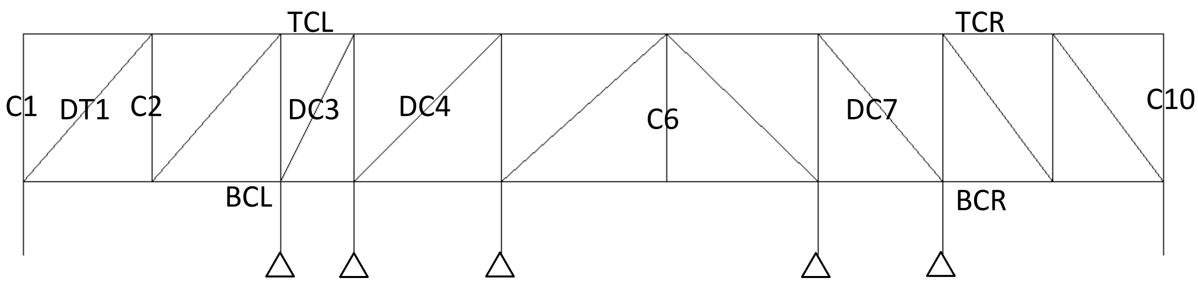


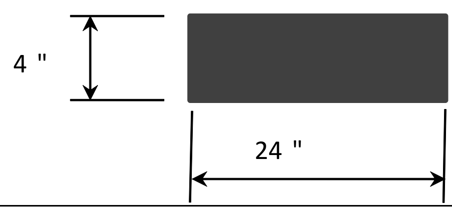
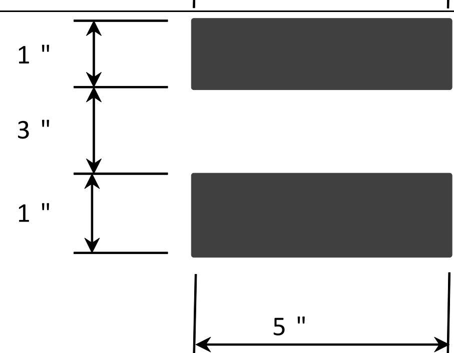
Figure 28 – Truss 3 Design Members

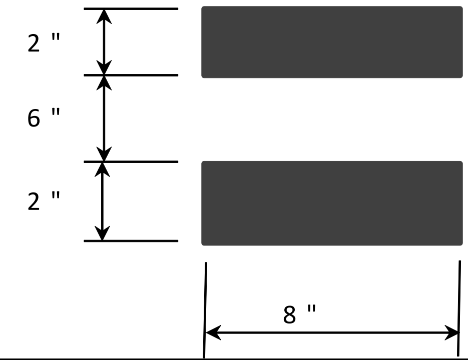
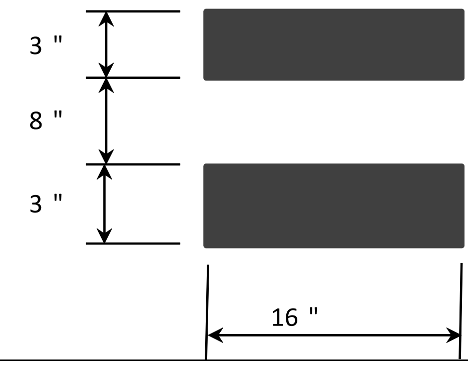
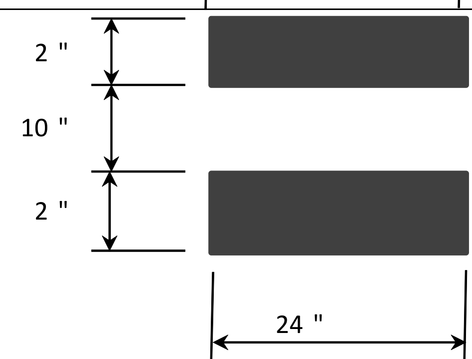
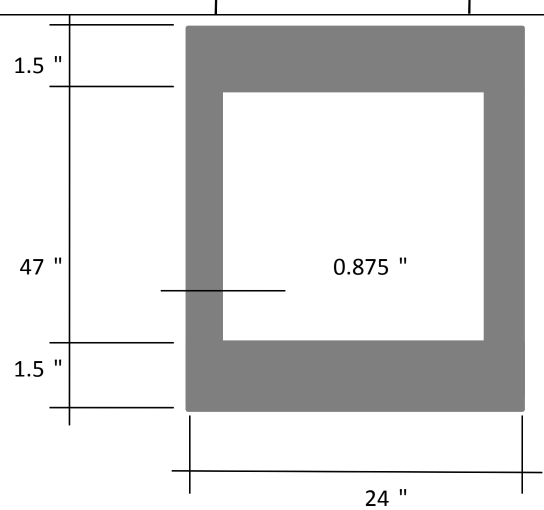
Table 14 – Truss 3 Design Forces

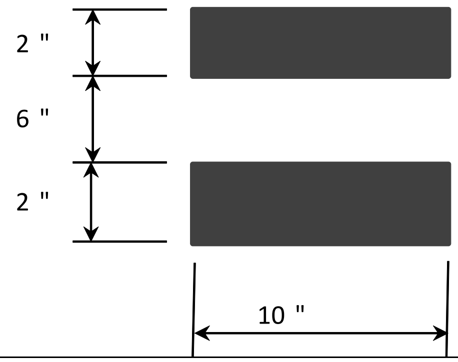
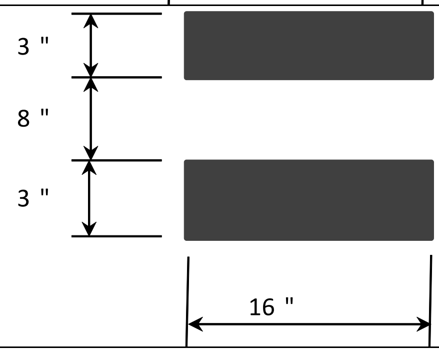
Member	Pu (k)	Mu (ft-k)
DT1	2700 (T)	0
DC7	30 (C)	0
C6	200 (C)	0
C10	2000 (C)	0
DC3	500 (C)	0
DC4	516 (C)	0
BCL	3300 (T)	5000
BCR	2500 (T)	2500
TCL	3200 (C)	1100
TCR	1300 (C)	2900
C2	1886 (C)	0
C1	1665 (C)	205

Truss 3 is very similar to Truss 2, but the truss does not pass through the braced frame core. It is supported by two braced frame columns and three gravity columns. Each end of the truss is exposed and the center of the truss is exposed above the elevator lobby of the 5th level (see Architectural Breadth Study). Appendix F displays all design forces for Truss 3, and Appendix G shows detailed calculations for the design of Truss 3 members. Table 14 provides the design forces for the members of Truss 3, and Table 15 summarizes the members designed for Truss 3.

Table 15 – Design Summary of Truss 3 Members

Member	Design	
DT1		P_u= 2700 kips ϕP_n= 3510 kips OK As= 96.0 in^2 S.W.= 327 PLF
DC7		Stitching Spacing = 24 in P_u= 30 kips ϕP_n= 54 kips OK As= 10.0 in^2 S.W.= 34 PLF

C6		Stitching Spacing = 12 in P_u= 200 kips ϕP_n= 1181 kips OK As= 32.0 in^2 S.W.= 109 PLF
C10		Stitching Spacing = 12 in P_u= 2000 kips ϕP_n= 4111 kips OK As= 96.0 in^2 S.W.= 327 PLF
DC3		Stitching Spacing = 12 in P_u= 500 kips ϕP_n= 3612 kips OK As= 96.0 in^2 S.W.= 327 PLF
BCL		Pu= 3300 kips ϕPn= 6506 kips Mu = 5000 ft-k ϕMn= 11302 ft-k H1-1a = 0.90 OK As= 154.25 in^2 S.W.= 525 PLF
BCR	W36x395 w/ Filler Plates	
TCL	W36x395 w/ Filler Plates	

TCR	W40x297 w/ Filler Plates	
DC4		Stitching Spacing = 12 in P_u= 516 kips ϕP_n= 653 kips OK As= 40.0 in^2 S.W.= 136 PLF
C2		Stitching Spacing = 24 in P_u= 1960 kips ϕP_n= 2768 kips OK As= 96.0 in^2 S.W.= 327 PLF
C1	W36x441 w/ Filler Plates	

Truss 4 Design

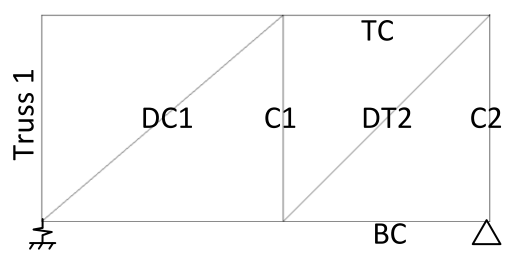
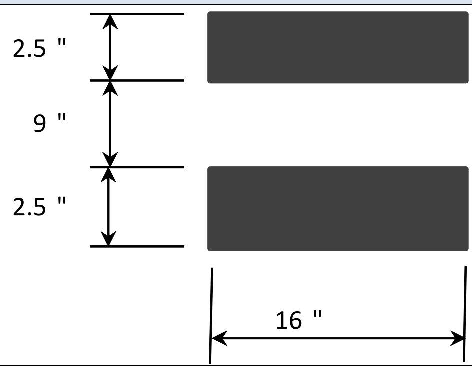
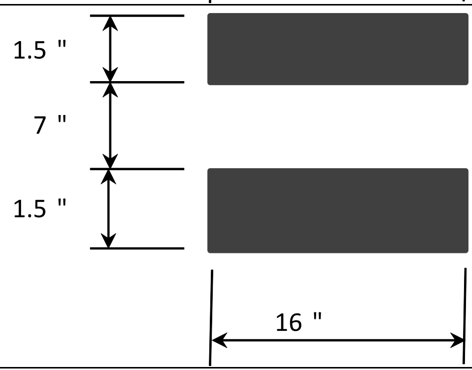
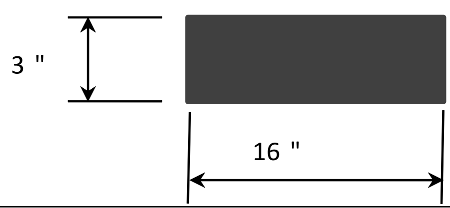


Figure 29 – Truss 4 Design Members

For member design forces, see Appendix F. Table 16 displays a summary for members of Truss 4 designed.

Table 16 – Design Summary of Truss 4 Members

Member	Design
DC1	 2.5 " / 9 " / 2.5 " ; 16 " Stitching Spacing = 12 in P_u= 1000 kips ϕP_n= 1260 kips OK As= 80.0 in^2 S.W.= 272 PLF
C1	 1.5 " / 7 " / 1.5 " ; 16 " Stitching Spacing = 12 in P_u= 900 kips ϕP_n= 1228 kips OK As= 48.0 in^2 S.W.= 163 PLF
DT2	 3 " ; 16 " P_u= 1300 kips ϕP_n= 1463 kips OK As= 48.0 in^2 S.W.= 163 PLF
TC	W40x294
BC	W40x264
C2	W14x311

Final Truss Designs

Truss 1

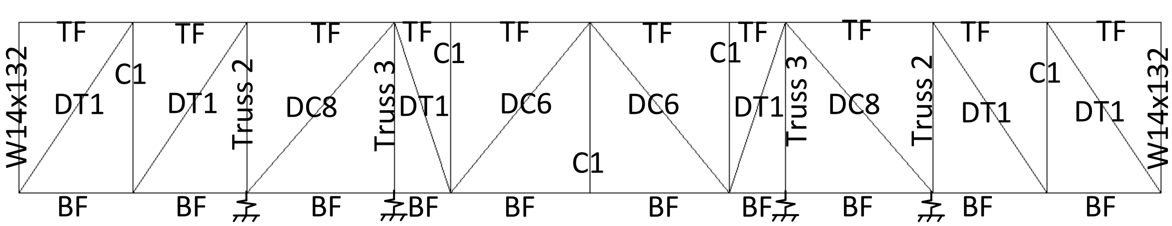


Figure 30 – Final Member Sizes for Truss 1

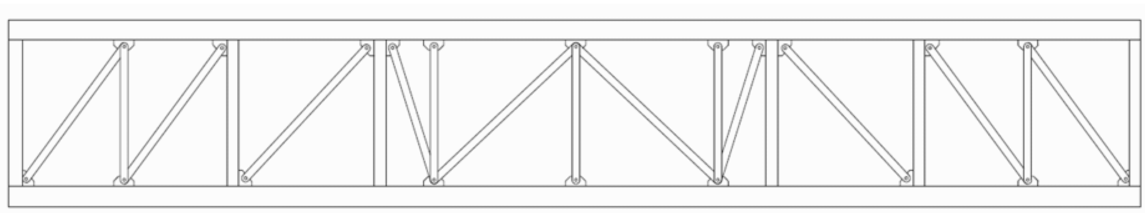


Figure 31 – Elevation View of Truss 1

Figure 30 displays a design summary of Truss 1. See Table 11 for cross-sections of Truss 1 member designs. Figure 31 is an elevation view of Truss 1.

Truss 2

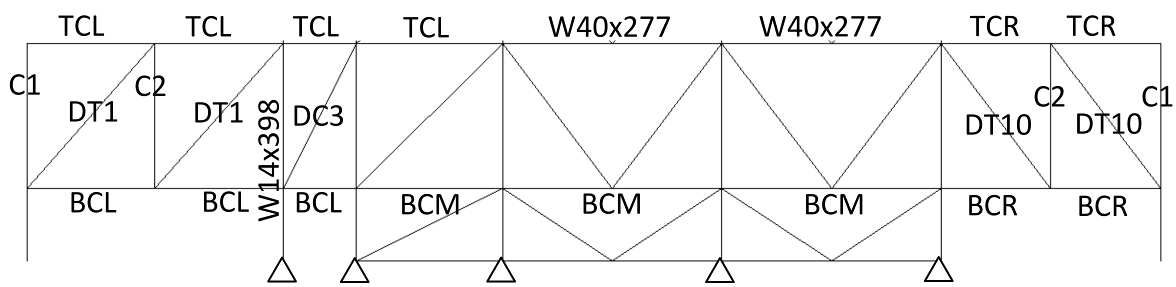


Figure 32 – Final Member Sizes for Truss 2

See Table 13 for cross-sections of Truss 2 member designs. For members not shown (LFRS members), see the Braced Frame Design section. An elevation view of Truss 2 is similar to an elevation view of Truss 3, shown in Figure 34.

Truss 3

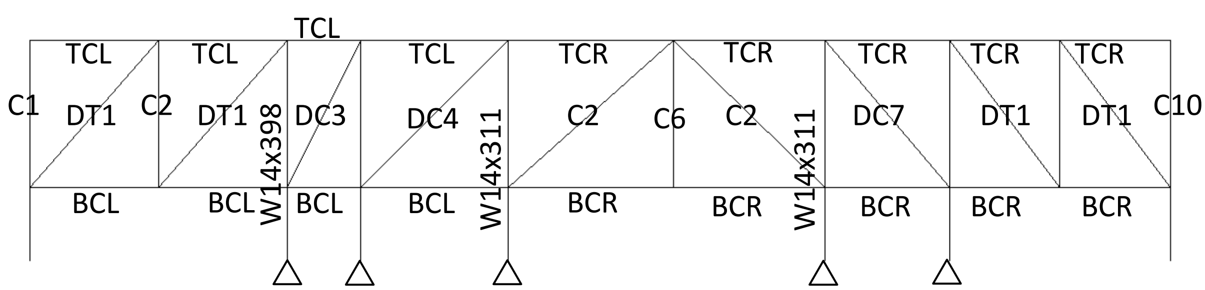


Figure 33 – Final Member Sizes for Truss 3

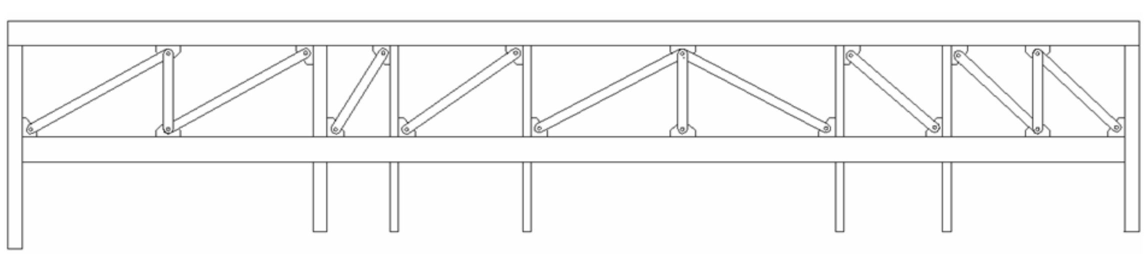


Figure 34 – Elevation View of Truss 3

See Table 15 for cross-sections of Truss 3 member designs. For members not shown, see the Braced Frame Design section.

Truss 4

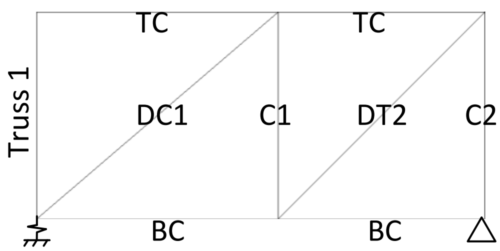


Figure 35 – Final Member Sizes for Truss 4

See Table 16 for cross-sections of Truss 4 members.

Truss Cambers and Truss Deflections

Due to the large gravity loads transferring through the trusses at the 5th level, deflections are difficult to control. Rather than sizing individual truss members for deflection, it is more efficient to camber each truss as necessary. For the trusses designed in this study, cambers were determined by applying 80 percent of the dead loads to the transfer system. Deflections were calculated using the ETABS model created. See Figure 36 through 41 for camber specifications of each truss. After cambers were determined, live load deflections were then calculated and compared to the limitations. AISC Design Guide 3 recommends a limit of L/180 for cantilevers supporting partitions, but a lower limit of L/250 was used for this study since the perimeter trusses will be supporting curtain walls. Live load deflections are available in Table 17.

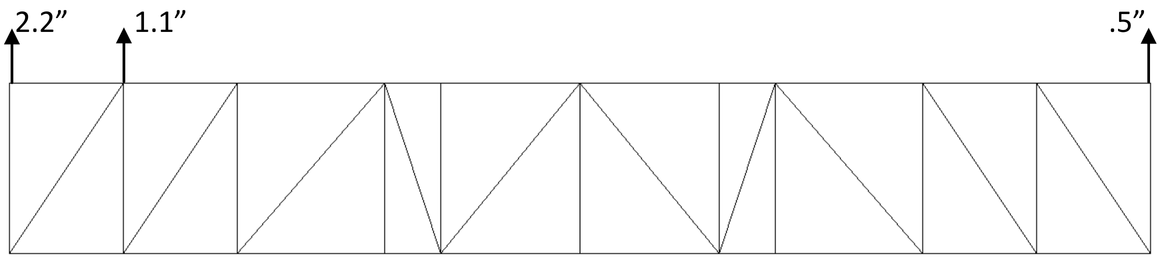


Figure 36 – Truss 1 Camber

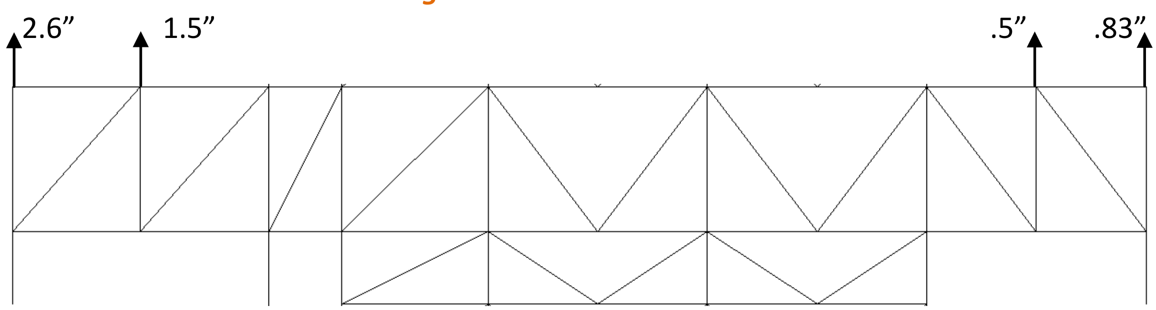


Figure 37 – Truss 2 Cambers

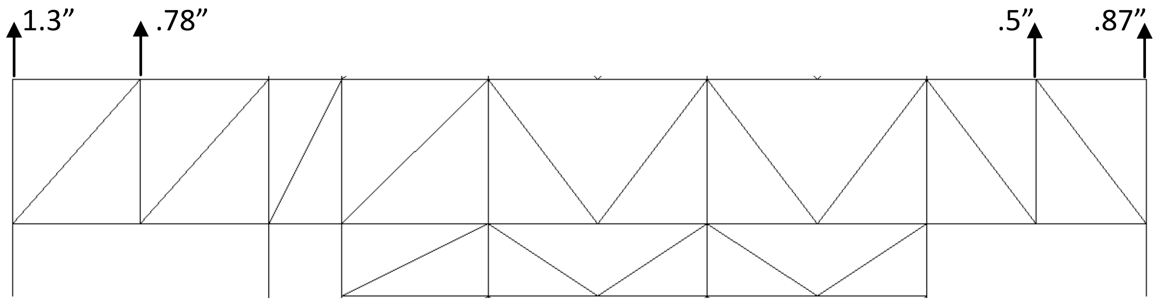


Figure 38 – Truss 2a Cambers

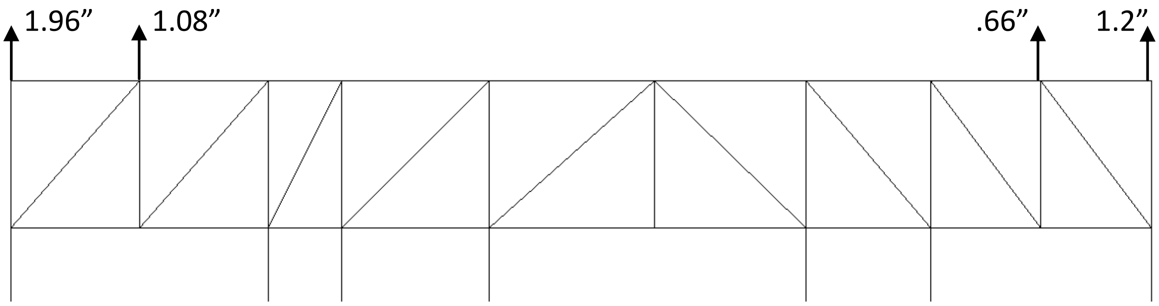


Figure 39 – Truss 3 Cambers

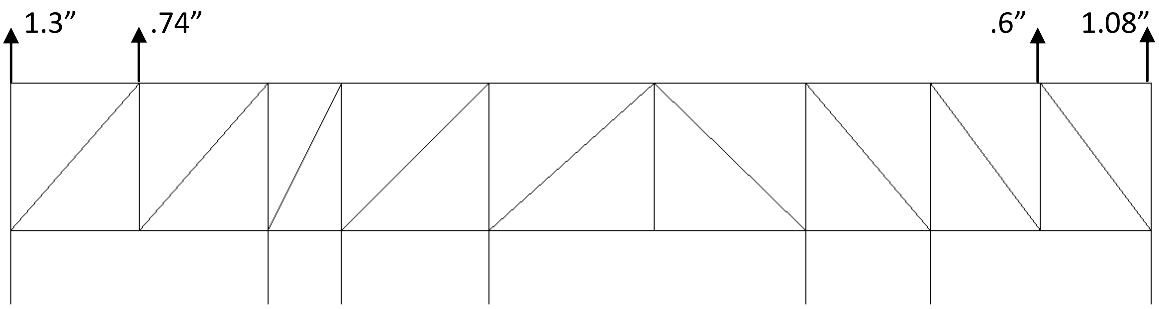


Figure 40 – Truss 3a Cambers

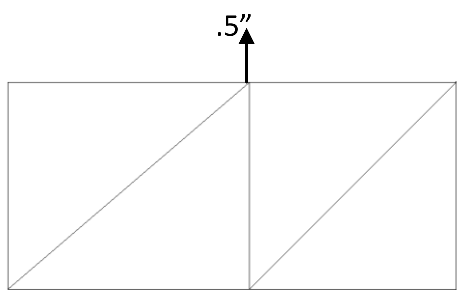


Figure 41 – Truss 4 Cambers

Table 17 – Maximum Live Load Deflections

Maximum Live Load Deflections			
Truss	L (ft)	$0.5\Delta_L$ (in)	(L/250)* (in)
1	40	1.41	1.92
2	35	0.73	1.68
2a	35	0.35	1.68
3	35	0.53	1.68
3a	35	0.36	1.68

* - Limit is for 50% live load. AISC Design Guide 3 recommends a limit of L/180.

Transfer Truss Comparison

The results of the gravity design for the new transfer system of the John Jay College Expansion Project conclude that the 5th level transfer trusses are a viable option. It was found that by using 6 trusses, only the necessary columns could be transferred over the Amtrak tracks to the braced frame core. By increasing the floor-to-floor height from 20 feet to 30 feet, interior trusses were designed with a height of 20 feet to avoid truss penetrations for elevators and doors within the braced frame core. Exterior truss 1 and 4 were designed with a height of 30 feet. All trusses use custom built-up steel sections and are architecturally exposed.

When comparing the transfer system designed in this study with the existing transfer system, several differences can be seen. The largest difference between the two transfer systems is the number of trusses used and the number of columns/plate hangers transferred. 100 percent of the perimeter plate hangers are transferred in the existing design, which requires a total of 10 trusses, and the new transfer system transfers 55 percent of the perimeter columns, which requires 6 trusses. The remaining 45 percent of the perimeter columns carry gravity loads to the foundation.

After performing a steel takeoff for each transfer system, it was determined that the transfer system designed for this study weighs 230 kips per truss and the existing transfer system weighs 152 kips per truss. The increase in weight for the new transfer trusses is caused by:

- the reduction of height in the interior trusses, which increases the axial design forces in the top and bottom chords,
- the reduction in the number of web members,
- the architectural depth requirements of the chord members,
- and by transferring an additional floor over the Amtrak tracks.

Typical built-up members were used where ever possible to allow for efficient fabrication and erection of steel members. Table 18 summarizes each transfer option.

Table 18 – Transfer System Comparison

Criteria	Thesis Transfer System	Existing Transfer System
Number of Transfer Trusses	6	10
Perimeter Columns Transferred	11/20 (55%)	24/24 (100%) *
Total Web Members	102	206
Avg. Web Members per Truss	17	21
Total Truss Weight (kips)	1380	1521
Avg. Truss Weight (kips)	230	152
Interior Truss Height	20'-0"	30'-0"
Perimeter Truss Height	30'-0"	30'-0"
Number of Levels Being Transferred w/ Trusses	11	10
Total Perimeter Column/Plate Hanger Weight (kips)	112	107

* - The original design uses perimeter plate hangers

Lateral Analysis and Design

Since the scope of this project is focused in the design of an alternate transfer system, it should be noted that the existing lateral force-resisting systems were analyzed and re-designed accordingly. This study was not performed to optimize the lateral force-resisting system by investigating other system types. Therefore, a detailed lateral analysis was performed to examine how moving the transfer trusses from the penthouse level to the 5th level and increasing the building height by 10 feet would affect the design of the braced frame core of the 14 story tower. An ETABS model was created to model the braced frames with the new transfer trusses. Each floor was assumed to be a rigid diaphragm. Wind loads were applied at the center of pressure for each level and seismic loads were applied at the center of mass for each level. These forces were then distributed to each braced frame based on the relative stiffness of each frame. Wind loads governed the design for strength and serviceability, which was expected due to the project being located in New York City. Braced Frames will be referred to as labeled in Figure 42.

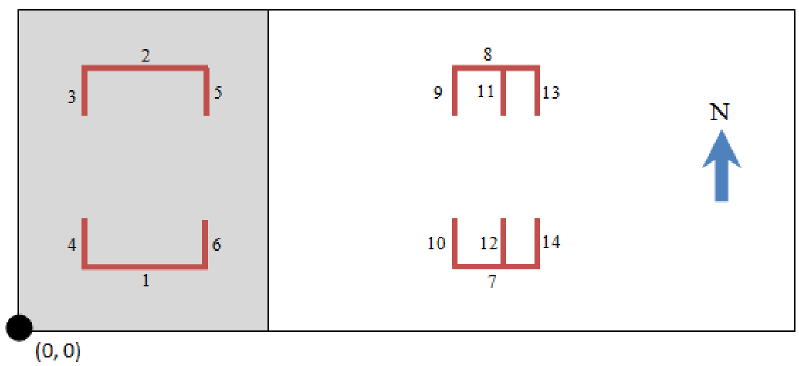


Figure 42 – Labeled Braced Frames

Lateral Loads

Wind Loads

Wind loads were calculated using Method 2 listed in Chapter 6 of ASCE 7-05. Table 19 displays calculated windward and leeward pressures for the North-South and East-West directions and Table 20 displays the calculated wind forces, story shears, and overturning moments for the North-South and East-West directions. See Appendix C for the assumptions used to calculate the wind loads.

Table 19 – Wind Pressures

	Level	Height Above ground (ft)	Kz	qz	Wind Pressures N-S (psf)	Wind Pressures E-W (psf)
Windward	T.O. Parapet	249.5	1.28	38.8	26.3	26.1
	Roof	246.67	1.274	38.6	26.2	25.9
	Penthouse	216.67	1.226	37.1	25.2	25.0
	13	201.67	1.203	36.4	24.7	24.5
	12	186.67	1.18	35.7	24.2	24.0
	11	171.67	1.15	34.8	23.6	23.4
	10	156.67	1.11	33.6	22.8	22.6
	9	141.67	1.1	33.3	22.6	22.4
	8	126.67	1.05	31.8	21.6	21.4
	7	111.67	1.02	30.9	21.0	20.8
	6	96.67	0.98	29.7	20.1	20.0
	5	66.67	0.87	26.3	17.9	17.7
	4	51.17	0.81	24.5	16.6	16.5
	3	31.17	0.71	21.5	14.6	14.5
	2	15.58	0.57	17.3	11.7	11.6
Leeward	All		1.28	38.8	-14.9	-16.3

Table 20 – Wind Forces

Level	Height Above ground (ft)	Building Dimensions (ft) H	B N-S	B E-W	Wind Forces Load (kips) N-S	Load (kips) E-W	Shear (kips) N-S	Shear (kips) E-W	Moment (ft-kips) N-S	Moment (ft-kips) E-W
Roof	236.67	15	165	200	102	127	0	0	25086	31246
Penthouse	206.67	22.5	165	200	149	186	102	127	57346	71461
13	191.67	15	165	200	98	122	251	312	77128	96132
12	176.67	15	165	200	97	121	349	435	95220	118706
11	161.67	15	165	200	95	119	446	556	111597	139152
10	146.67	15	165	200	93	117	541	675	126224	157428
9	131.67	15	165	200	93	116	634	791	139379	173868
8	116.67	15	165	200	90	113	727	907	150818	188180
7	101.67	15	165	200	89	111	818	1020	160733	200593
6	86.67	22.5	165	200	130	163	906	1131	173313	216357
5	66.67	22.75	500	200	373	155	1036	1295	198181	226670
4	51.17	17.75	500	200	280	116	1409	1449	212514	232624
3	31.17	17.795	500	200	263	109	1690	1566	220697	236035
2	15.58	15.585	375	200	156	87	1952	1675	223122	237389
Total	236.67				2108	1762	2108	1762	223122	237389

Wind Load Cases

Wind load cases 1, 2, 3, and 4 of Figure 6-9 in ASCE 7-05 were applied in the lateral analysis. This resulted in a total of 12 wind load cases. Each direction of wind loading was used to determine the maximum story drifts and total building drift, as well as determining the governing load combination for lateral member design.

Seismic Loads

The seismic loads were calculated using the equivalent lateral force procedure in Chapter 11 of ASCE 7-05. Table 21 displays story forces, story shears, and overturning moments for the seismic loads. The seismic loads did not govern the design of the lateral systems for strength or serviceability requirements. See Appendix D for the assumption used to calculate the seismic loads.

Table 21 – Seismic Story Forces, Story Shears, and Overturning Moments

Level	Story Weight w_x (Kips)	Height h_x (ft)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (kips)	Story Shear V_x (kips)	Moment M_x (ft-k)
Roof	3286	246.67	6493046	0.134	146	0	35895
Penthouse	6502	216.67	10746832	0.222	241	146	88081
13	2874	201.67	4303070	0.089	96	386	107530
12	2822	186.67	3797815	0.078	85	483	123419
11	3040	171.67	3645875	0.075	82	568	137446
10	2638	156.67	2789273	0.058	63	650	147240
9	3040	141.67	2798161	0.058	63	712	156124
8	2870	126.67	2264449	0.047	51	775	162552
7	2929	111.67	1942519	0.040	44	826	167414
6	3785	96.67	2057612	0.042	46	869	171872
5	12565	66.67	4093741	0.084	92	915	177989
4	8483	51.17	1919541	0.040	43	1007	180190
3	10119	31.17	1156578	0.024	26	1050	180998
2	10932	15.58	480610	0.010	11	1076	181166
Total	81866	246.67	48489122	1.000	1087	1087	181166

Seismic Load Cases

Seismic loads were applied in both the East-West and North-South directions at the center-of-mass. 4 additional seismic load cases were created to account for accidental torsion as required by Section 12.8.4.2 of ASCE 7-05. These load cases were also used to determine maximum story drifts and total building drifts, and to design lateral members. As mentioned above, wind loads controlled the design for both strength and serviceability.

Braced Frame Design

After designing a new transfer system for the 5^{th} level of the John Jay College Expansion Project, several members of the braced frames, which support the transfer trusses, had to be resized. ETABS was used to analyze and design the braced frames for the Expansion Project with the new transfer system.

Design Load Combinations

Load combinations 1 through 7 listed below were used to design the braced frame members. Load combinations 5 and 7 take vertical seismic load effects into account. Seismic vertical load effects are only used in load combinations 5 and 7 when S_{DS} is greater than 0.125, which is the case for this project (see Appendix D). Load combination 4 controlled the strength design for the majority of the lateral members.

1. 1.4D
2. $1.2D + 1.6L + 0.5L_r$
3. $1.2D + 1.6L_r + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5L_r$
5. $(1.2 + 0.2S_{DS})D + E + L$
6. $0.9D + 1.6W$
7. $(0.9 – 0.2S_{DS})D + E$

Design Assumptions

The following simplifying assumptions were used to design the braced frame members:

- Bracing members are pinned at each end
- Columns are continuous and pinned at the base
- Beams are simply supported
- All trusses, except Truss 2, were conservatively neglected when designing the braced frames for lateral forces

Since the top level is a mechanical penthouse, the opportunity to couple braced frames 3 to 4 and 5 to 6, was available. A preliminary analysis was completed without the coupling action in these frames, and the lateral drifts were much larger than the acceptable limits as defined by ASCE 7-05. Therefore, it was assumed that coupling the two slender frames together would be more efficient than increasing the appropriate braced frame member sizes to meet lateral drift requirements.

Braced Frame 1 and 2

Braced frames 1 and 2 have identical designs. The majority of the lateral members were controlled by load combination 4 listed above. Results from the design of braced frames 1 and 2 can be seen in Figure 43. Comparisons can be made to the existing design of braced frames 1 and 2 by looking at Appendix I. The biggest difference between the new and existing design is that the columns at the top of the new design are much lighter than those of the existing design.

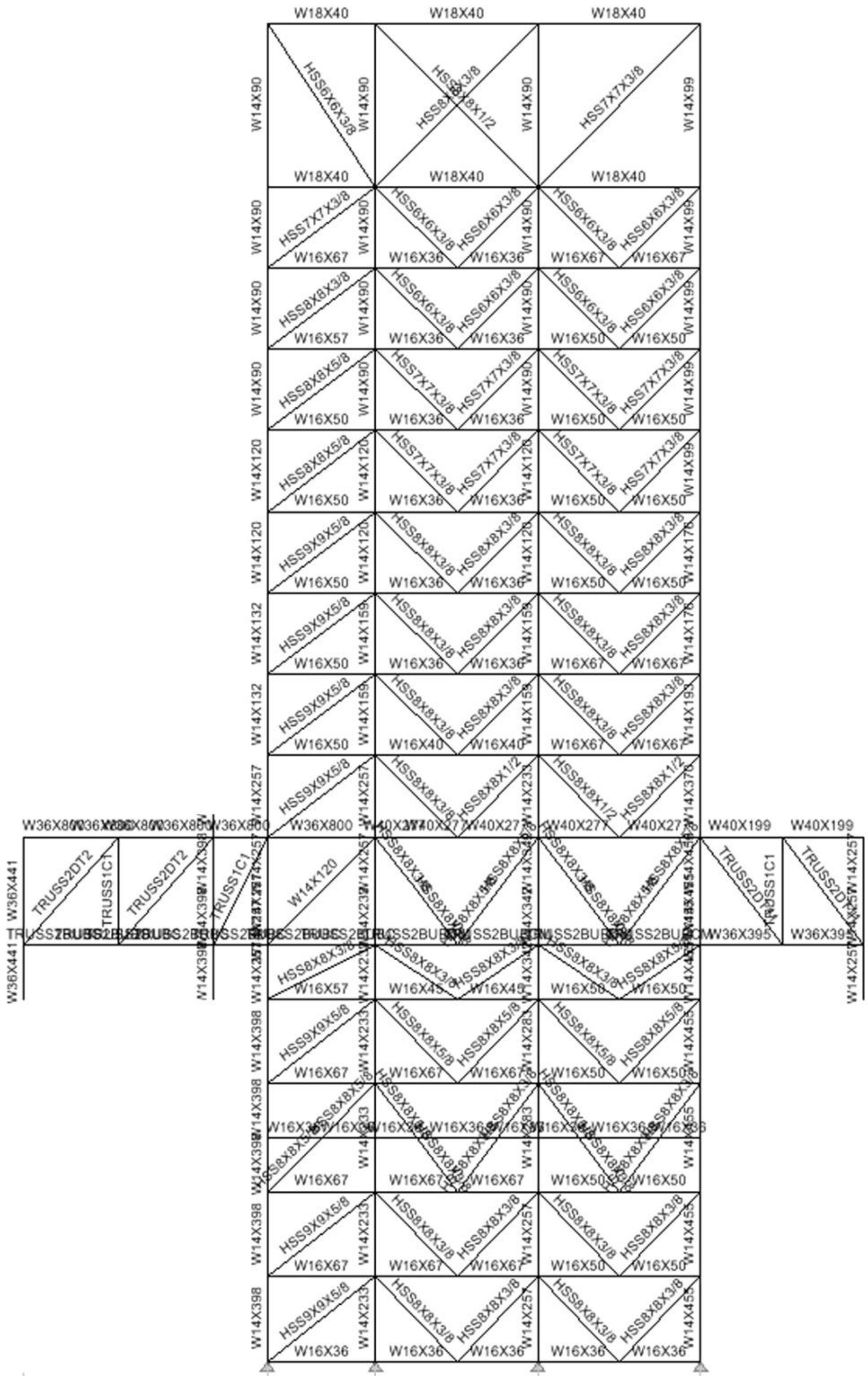


Figure 43 – Braced Frame 1 and 2 Design (for truss design members see Truss Design Sections)

Braced Frames 3, 4, 5, and 6

Braced frame 3 and 4 designs can be seen in Figure 44 and braced frame 5 and 6 can be seen in Figure 45. These designs can be compared to their existing design by looking at Appendix I. Figure 46 displays the interaction ratio if the members designed for each frame.

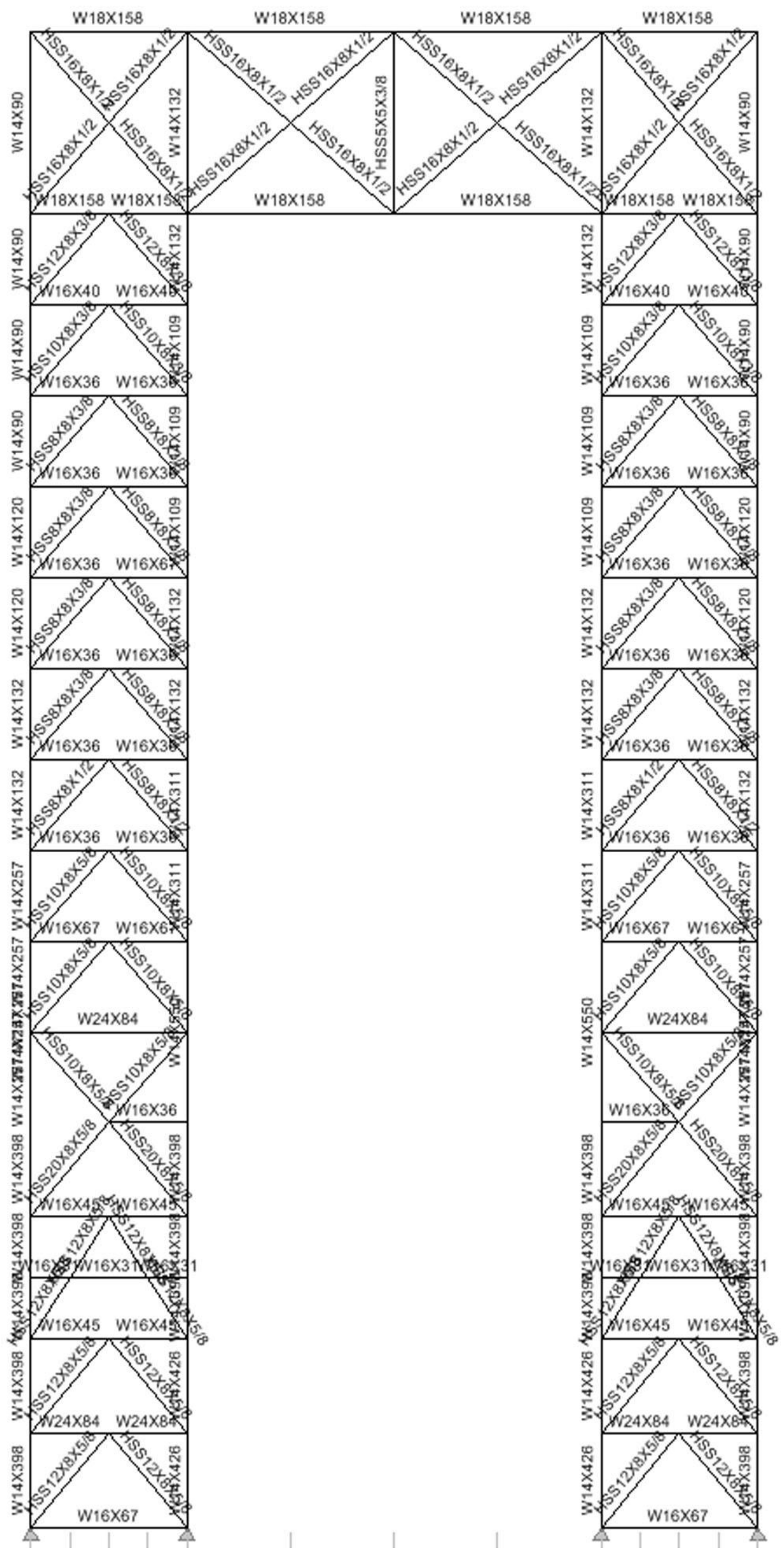


Figure 44 – Braced Frame 3 and 4 Design

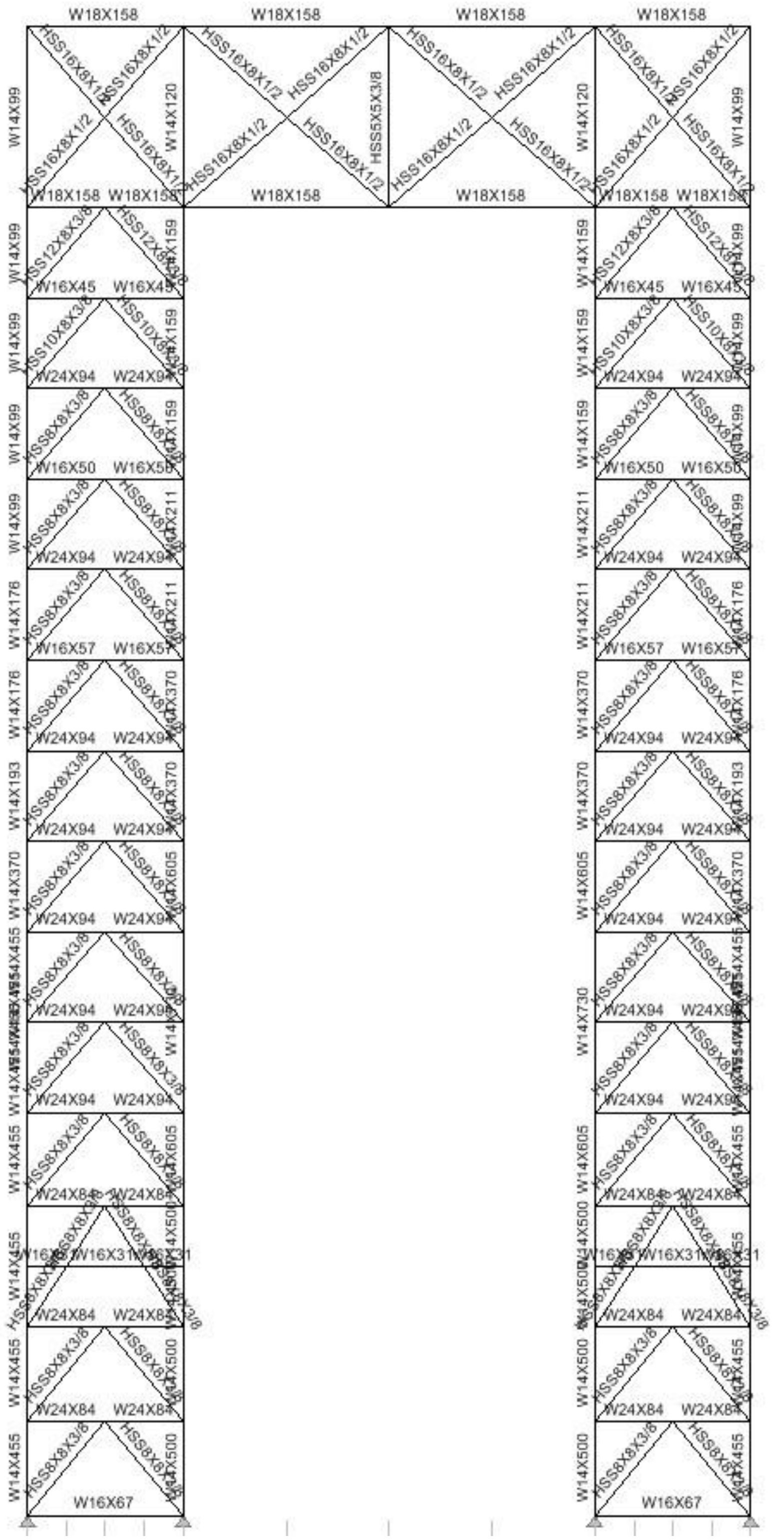
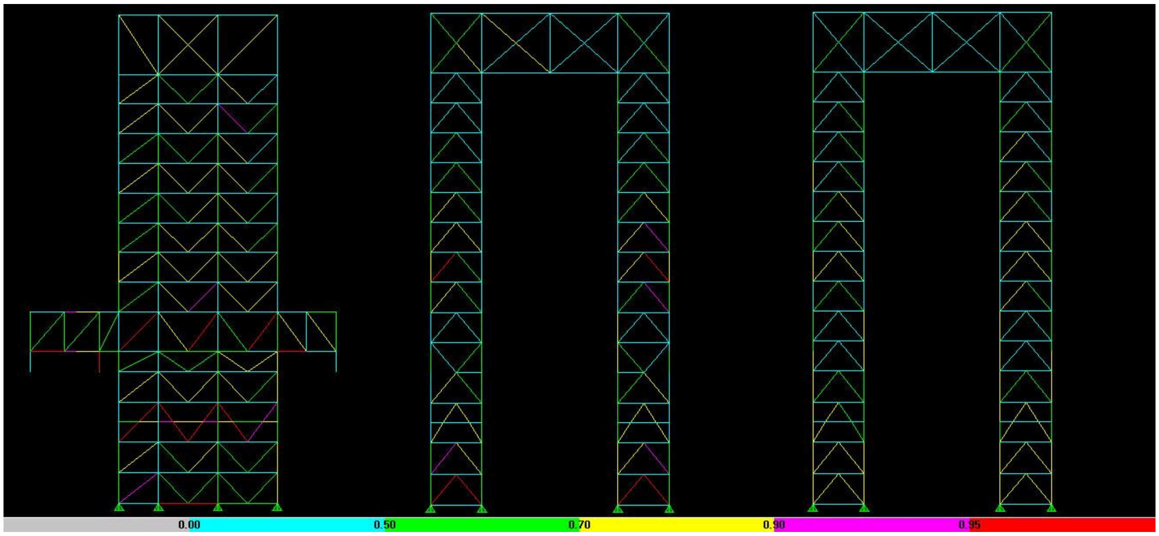


Figure 45 – Braced Frame 5 and 6 Design



Braced Frame 1 & 2 Braced Frame 3 & 4 Braced Frame 5 & 6

Figure 46 – Braced Frame Design Interaction Ratios

Lateral Drift Limitations

After the braced frames were resized for strength, lateral drifts were calculated for the applied wind and seismic loads. 70 percent of the East-West and North-South wind loads were applied to the ETABS model - as permitted by section CC.1.2 of ASCE 7-05 – to calculate the lateral drifts of each frame. Calculated lateral drifts due to wind were compared to the recommended allowable drift limitation in ASCE 7-05 of H/400. Table 22 summarizes the lateral drift study due to wind in the East-West direction and Table 23 summarizes the lateral drift study for the North-South direction. Chart 1 is a visual summary of the total building drift presented in Table 23, which produced maximum lateral drifts for the project.

Table 22 – East-West Wind Drift Summary

Wind Drift: East-West Direction									
Story	Story Height	Calculated Story Drift	Allowable Story Drift Δ_{wind} = H/400			Calculated Total Drift	Allowable Total Drift Δ_{wind} = H/400		
	(ft)	(in)	(in)			(in)	(in)		
Roof	246.67	0.109	<	0.900	Acceptable	1.30	<	7.40	Acceptable
14	216.67	0.072	<	0.450	Acceptable	1.20	<	6.50	Acceptable
13	201.67	0.082	<	0.450	Acceptable	1.12	<	6.05	Acceptable
12	186.67	0.084	<	0.450	Acceptable	1.04	<	5.60	Acceptable
11	171.67	0.091	<	0.450	Acceptable	0.96	<	5.15	Acceptable
10	156.67	0.091	<	0.450	Acceptable	0.87	<	4.70	Acceptable
9	141.67	0.095	<	0.450	Acceptable	0.78	<	4.25	Acceptable
8	126.67	0.099	<	0.450	Acceptable	0.68	<	3.80	Acceptable
7	111.67	0.091	<	0.450	Acceptable	0.58	<	3.35	Acceptable
6	96.67	0.176	<	0.900	Acceptable	0.49	<	2.90	Acceptable
5	66.67	0.082	<	0.465	Acceptable	0.32	<	2.00	Acceptable
4	51.17	0.114	<	0.600	Acceptable	0.23	<	1.54	Acceptable
3	31.17	0.058	<	0.468	Acceptable	0.12	<	0.94	Acceptable
2	15.58	0.062	<	0.467	Acceptable	0.06	<	0.47	Acceptable

Table 23 – North-South Wind Drift Summary

Wind Drift: North-South Direction										
Story	Story Height	Calculated Story Drift	Allowable Story Drift Δ_{wind} = H/400			Calculated Total Drift	Allowable Total Drift Δ_{wind} = H/400			
	(ft)	(in)	(in)			(in)	(in)			
Roof	246.67	0.090	<	0.900	Acceptable	4.80	<	7.40	Acceptable	
14	216.67	0.208	<	0.450	Acceptable	4.71	<	6.50	Acceptable	
13	201.67	0.271	<	0.450	Acceptable	4.50	<	6.05	Acceptable	
12	186.67	0.328	<	0.450	Acceptable	4.23	<	5.60	Acceptable	
11	171.67	0.365	<	0.450	Acceptable	3.90	<	5.15	Acceptable	
10	156.67	0.390	<	0.450	Acceptable	3.54	<	4.70	Acceptable	
9	141.67	0.405	<	0.450	Acceptable	3.15	<	4.25	Acceptable	
8	126.67	0.393	<	0.450	Acceptable	2.74	<	3.80	Acceptable	
7	111.67	0.365	<	0.450	Acceptable	2.35	<	3.35	Acceptable	
6	96.67	0.743	<	0.900	Acceptable	1.98	<	2.90	Acceptable	
5	66.67	0.776	>	0.465	Unacceptable	1.24	<	2.00	Acceptable	
4	51.17	0.245	<	0.600	Acceptable	0.47	<	1.54	Acceptable	
3	31.17	0.029	<	0.468	Acceptable	0.22	<	0.94	Acceptable	
2	15.58	0.193	<	0.467	Acceptable	0.19	<	0.47	Acceptable	

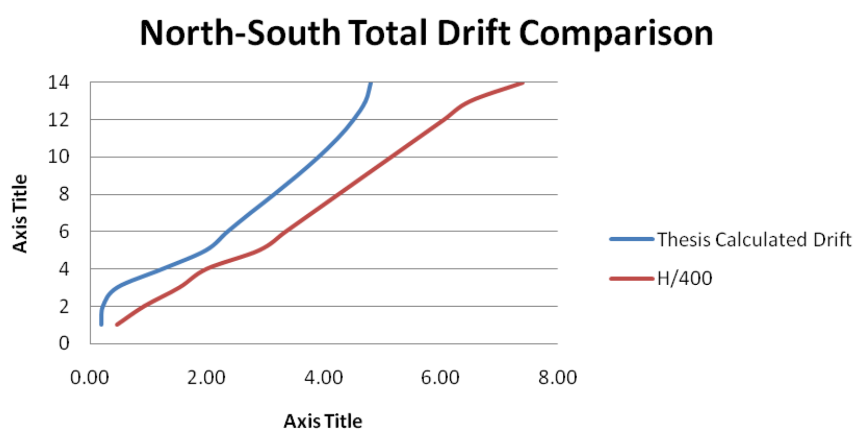


Chart 1 – North-South Total Building Drift Comparison

The design of the braced frame core was controlled by strength requirements, rather than serviceability requirements. It is visible in Chart 1 that the calculated total building drift is well below the recommended allowable of H/400.

Lateral drifts due to seismic loads were also calculated using the ETABS model. These calculated lateral drifts were multiplied by C_d to account for material nonlinearity effects and were reduced slightly by dividing by the importance factor, I, which is required by Section 12.8.6 of ASCE 7-05. Lateral drifts were then compared to the ASCE 7-05 allowable drift of $0.015h_{sx}$. Calculated lateral seismic drifts include accidental torsion effects. For ordinarily concentrically braced frames, C_d is equal to 3.25 and the importance factor, I, is equal to 1.25 for the John Jay College Expansion Project. Table 24 summarizes the seismic drift study performed for the East-West direction and Table 25 summarizes the seismic drift study performed for the North-South direction.

Table 24 – East-West Seismic Drift Summary

Seismic Drift: East-West Direction											
		Story Drift					Total Drift				
Story	Story Height	δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Total Drift $\Delta_{seismic} = 0.015h_{sx}$		
	(ft)	(in)	(in)	(in)			(in)	(in)	(in)		
Roof	246.67	0.40	1.04	<	5.40	Acceptable	3.31	8.60	<	44.40	Acceptable
14	216.67	0.25	0.65	<	2.70	Acceptable	2.91	7.57	<	39.00	Acceptable
13	201.67	0.27	0.70	<	2.70	Acceptable	2.66	6.92	<	36.30	Acceptable
12	186.67	0.26	0.66	<	2.70	Acceptable	2.39	6.21	<	33.60	Acceptable
11	171.67	0.26	0.68	<	2.70	Acceptable	2.14	5.55	<	30.90	Acceptable
10	156.67	0.25	0.64	<	2.70	Acceptable	1.87	4.87	<	28.20	Acceptable
9	141.67	0.25	0.64	<	2.70	Acceptable	1.62	4.22	<	25.50	Acceptable
8	126.67	0.24	0.62	<	2.70	Acceptable	1.38	3.59	<	22.80	Acceptable
7	111.67	0.21	0.54	<	2.70	Acceptable	1.14	2.96	<	20.10	Acceptable
6	96.67	0.37	0.97	<	5.40	Acceptable	0.93	2.42	<	17.40	Acceptable
5	66.67	0.16	0.41	<	2.79	Acceptable	0.56	1.45	<	12.00	Acceptable
4	51.17	0.20	0.53	<	3.60	Acceptable	0.40	1.04	<	9.21	Acceptable
3	31.17	0.10	0.26	<	2.81	Acceptable	0.20	0.51	<	5.61	Acceptable
2	15.58	0.09	0.25	<	2.80	Acceptable	0.09	0.25	<	2.80	Acceptable

Table 25 – North-South Seismic Drift Summary

Seismic Drift: North-South Direction											
		Story Drift					Total Drift				
Story	Story Height	δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Total Drift $\Delta_{seismic} = 0.015h_{sx}$		
	(ft)	(in)	(in)	(in)			(in)	(in)	(in)		
Roof	246.67	0.17	0.44	<	5.40	Acceptable	6.43	16.72	<	44.40	Acceptable
14	216.67	0.37	0.96	<	2.70	Acceptable	6.26	16.28	<	39.00	Acceptable
13	201.67	0.46	1.20	<	2.70	Acceptable	5.89	15.31	<	36.30	Acceptable
12	186.67	0.53	1.38	<	2.70	Acceptable	5.43	14.12	<	33.60	Acceptable
11	171.67	0.57	1.48	<	2.70	Acceptable	4.9	12.74	<	30.90	Acceptable
10	156.67	0.59	1.53	<	2.70	Acceptable	4.33	11.26	<	28.20	Acceptable
9	141.67	0.58	1.51	<	2.70	Acceptable	3.74	9.72	<	25.50	Acceptable
8	126.67	0.54	1.40	<	2.70	Acceptable	3.16	8.22	<	22.80	Acceptable
7	111.67	0.48	1.25	<	2.70	Acceptable	2.62	6.81	<	20.10	Acceptable
6	96.67	0.93	2.42	<	5.40	Acceptable	2.14	5.56	<	17.40	Acceptable
5	66.67	0.35	0.91	<	2.79	Acceptable	1.21	3.15	<	12.00	Acceptable
4	51.17	0.45	1.17	<	3.60	Acceptable	0.86	2.24	<	9.21	Acceptable
3	31.17	0.24	0.62	<	2.81	Acceptable	0.41	1.07	<	5.61	Acceptable
2	15.58	0.17	0.44	<	2.80	Acceptable	0.17	0.44	<	2.80	Acceptable

New York City Building Code Drift Limitations

Although this thesis project is using ASCE 7-05 for the determination of gravity and lateral loads, as well as drift limitations, the original design used the New York City Building Code. This code has different lateral load requirements and more strict allowable drift requirements. After the lateral drifts were determined to be sufficient for the re-designed braced frames using ASCE 7-05, a separate analysis was performed to ensure that the new design of the braced frames would also meet the drift requirements the original project was designed for. Table 26 displays the required North-South base shear for wind loads and the drift limitations for each code. Wind loads for ASCE 7-05 were factored by 0.7 and wind loads for the NYC Building code were factored by 1.0 when computing lateral drifts.

Table 26 – Design Drift Limitation Comparison

Design Drift Limitation Comparison		
Design	Base Shear (N-S)	Limit
Thesis	1476	H/400
Existing	1379	H/500

The New York City Building Code requires wind pressures of 20 psf for heights of 0 to 100 feet, and 25 psf for building heights of 100 to 300 feet. The lateral drift limitation due to wind loading is H/500 for the total building height and H/400 for interstory drift. After performing a separate lateral analysis for the requirements of the New York City Building Code, it was determined that the newly designed braced frames would also be sufficient for total building drift, but story drifts at levels 9 and 5 are unacceptable. Table 27 summarizes the drift study performed for the controlling North-South direction, along with a visual summary presented in Chart 2.

Table 27 – North-South Wind Drift Summary for the New York City Building Code

NYC Building Code Drift									
Story	Story Height	Calculated Story Drift	Allowable Story Drift Δ_{wind} = H/400			Calculated Total Drift	Allowable Total Drift Δ_{wind} = H/500		
	(ft)	(in)	(in)			(in)	(in)		
Roof	246.67	0.080	<	0.900	Acceptable	4.22	<	5.92	Acceptable
14	216.67	0.185	<	0.450	Acceptable	4.14	<	5.20	Acceptable
13	201.67	0.241	<	0.450	Acceptable	3.95	<	4.84	Acceptable
12	186.67	0.293	<	0.360	Acceptable	3.71	<	4.48	Acceptable
11	171.67	0.327	<	0.360	Acceptable	3.42	<	4.12	Acceptable
10	156.67	0.350	<	0.360	Acceptable	3.09	<	3.76	Acceptable
9	141.67	0.364	>	0.360	Unacceptable	2.74	<	3.40	Acceptable
8	126.67	0.354	<	0.360	Acceptable	2.38	<	3.04	Acceptable
7	111.67	0.330	<	0.360	Acceptable	2.02	<	2.68	Acceptable
6	96.67	0.672	<	0.720	Acceptable	1.69	<	2.32	Acceptable
5	66.67	0.550	>	0.372	Unacceptable	1.02	<	1.60	Acceptable
4	51.17	0.257	<	0.480	Acceptable	0.47	<	1.23	Acceptable
3	31.17	0.051	<	0.374	Acceptable	0.21	<	0.75	Acceptable
2	15.58	0.162	<	0.374	Acceptable	0.16	<	0.37	Acceptable

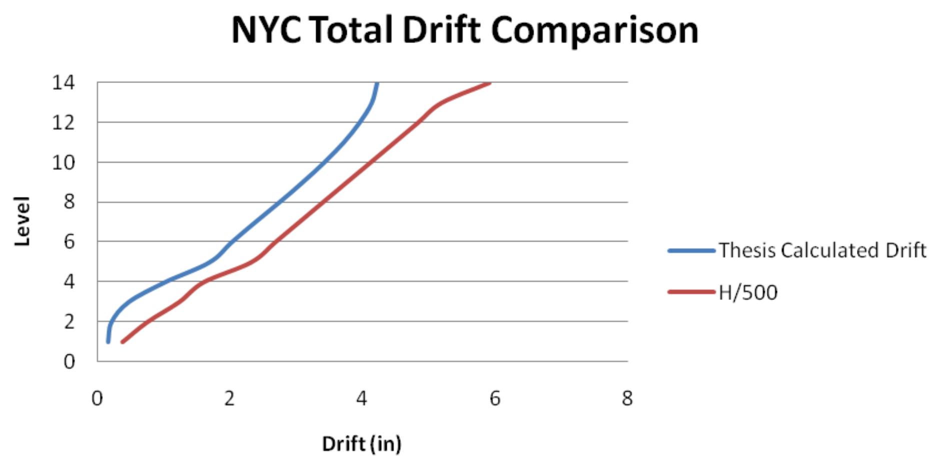


Chart 2 – NYC Building Code Total Building Drift Comparison

Base Shears and Relative Frame Stiffnesses

Table 28 displays the base shear present in each new braced frame of the John Jay College Expansion Project, and compares it with the total base shear. Table 29 displays the base shear present in the existing braced frames. Percentages of the total base shear are presented in both tables to display the relative stiffness of each braced frame.

Table 28 – Base Shear Summary for **New** Braced Frame Design

Tower Braced Frames: Base Shear						
	East-West Frames (kips)		North-South Frames (kips)			
	1	2	3	4	5	6
BASE	424	424	460	460	183	183
Total	1762	1762	2108	2108	2108	2108
Percentage	0.24	0.24	0.22	0.22	0.09	0.09

Cascade Braced Frames: Base Shear								
	East-West Frames (kips)		North-South Frames (kips)					
	7	8	9	10	11	12	13	14
BASE	453	452	175	175	125	125	112	112
Total	1762	1762	2108	2108	2108	2108	2108	2108
Percentage	0.26	0.26	0.08	0.08	0.06	0.06	0.05	0.05

Table 29 – Base Shear Summary for **Existing** Braced Frame Design

Tower Braced Frames: Base Shear						
	East-West Frames (kips)		North-South Frames (kips)			
	1	2	3	4	5	6
BASE	375	375	344	344	254	254
Total	1664	1664	1968	1968	1968	1968
Percentage	0.23	0.23	0.17	0.17	0.13	0.13

Cascade Braced Frames: Base Shear								
	East-West Frames (kips)		North-South Frames (kips)					
	7	8	9	10	11	12	13	14
BASE	448	448	198	198	118	118	71	71
Total	1664	1664	1968	1968	1968	1968	1968	1968
Percentage	0.27	0.27	0.10	0.10	0.06	0.06	0.04	0.04

Overturning Analysis

The recommended factor of safety against overturning is 3.0. When an overturning analysis was performed, all of the tower braced frames were determined to have a factor of safety that is less than 3.0, and therefore overturning is an issue (see Table 30). To ensure uplifting does not occur, exterior braced frame concrete pier footings need to be secured to the bedrock using rock anchors. For concrete caissons that are embedded into bedrock, overturning is not an issue.

An overturning analysis was performed for the existing design in Technical Report 3 using wind pressures from ASCE 7-05, which also resulted in the braced frame tower having a factor of safety less than 3.0 against overturning. The existing design does not use rock anchors to attach concrete pier footings to the bedrock, but it is assumed that this is because the lateral design forces of the New York City Building Code are less than ASCE 7-05. Therefore, overturning is an issue with both designs, and cannot be considered a flaw of the new transfer system.

Table 30 – Overturning Analysis of the Braced Frame Core

Tower Braced Frames: Overturning Check						
	East-West Frame		North-South Frames			
	1	2	3	4	5	6
Tensile Force at Edge Column from Wind (k)	703	703	1418	1418	1000	1000
Edge Column DL (k)	1417	1417	1430	1430	2310	2310
F.S.	2.02	2.02	1.01	1.01	2.31	2.31

Center-of-Rigidity Discussion

Strength requirements caused braced frames 3 and 4 to be stiffer than braced frames 4 and 5. This difference in stiffness caused the center-of-rigidity to move closer to the center-of-mass. This reduction in eccentricity between the center-of-mass and the center-of-rigidity reduces the shear due to torsion. Table 31 compares the COR and the resulting eccentricities for the new and existing braced frame designs. See Figure 47 for the location of the center-of-rigidity in the 14 story tower.

Table 31 – COR and COM Comparison

Level	New Braced Frame Design						Existing Braced Frame Design					
	Center of Rigidity		Center of Mass		Eccentricity		Center of Rigidity		Center of Mass		Eccentricity	
	Xr (ft)	Yr (ft)	Xm (ft)	Ym (ft)	Ex (ft)	Ey (ft)	Xr (ft)	Yr (ft)	Xm (ft)	Ym (ft)	Ex (ft)	Ey (ft)
14	93	100	78	100	15	0	103	100	78	100	26	0
13	93	100	78	100	15	0	103	100	78	100	26	0
12	94	100	78	100	16	0	104	100	78	100	27	0
11	95	100	78	100	17	0	105	100	78	100	28	0
10	96	100	78	100	18	0	107	100	78	100	29	0
9	97	100	78	100	19	0	108	100	78	100	30	0
8	98	100	78	100	20	0	110	100	78	100	32	0
7	100	100	78	100	22	0	113	100	78	100	35	0
6	104	100	78	100	27	0	118	100	78	100	40	0
5	113	100	78	100	35	0	129	100	78	100	51	0
4	175	100	243	100	-68	0	227	100	243	100	-17	0
3	196	100	243	100	-48	0	240	100	243	100	-3	0
2	220	100	243	100	-23	0	269	100	243	100	25	0

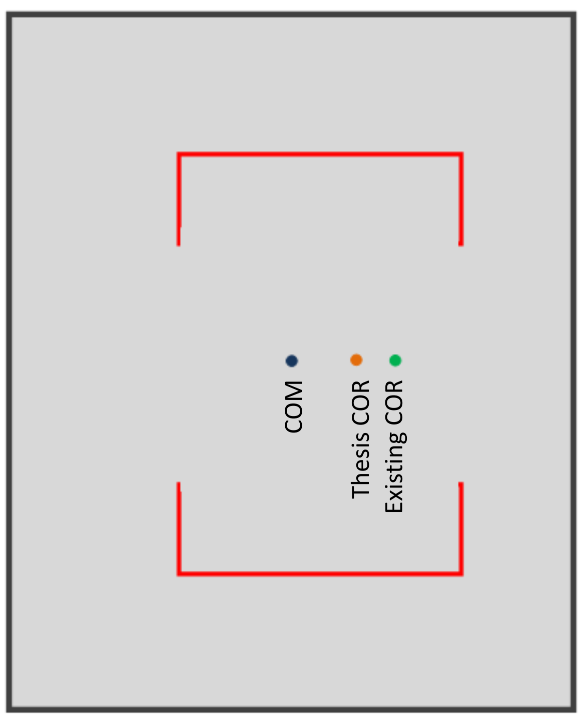


Figure 47 – Movement of the Center-of-Rigidity

Comparison between Existing and New Braced Frames

After re-designing and analyzing the braced frames of the John Jay College Expansion Project to incorporate transfer trusses at the 5th level, several differences from the original design are present. Some of the pros of using the 5th level transfer option instead of the penthouse level transfer option are:

- Lighter column sections at the top of the braced frame and
- The North-South braced frames 3 and 4 are stiffer than braced frames 5 and 6, which moves the center-of-rigidity closer to the center-of-mass, and therefore reducing shear due to torsion.

However, one con of using the 5th level transfer system is:

- The increase in height at the 5th level causes some lateral story drifts which are difficult to control.

It should be noted that the braced frames designed in this study had significantly larger braces than the original design. This is believed to be caused by the differences in wind pressures between the NYC Building Code and ASCE 7-05, and not caused by the transfer system at the 5th level.

Foundation Impacts

An additional study was performed to see how the perimeter columns (labeled C8 in Figure 18) that are not transferred over the Amtrak tracks affect the existing foundation design. The original design required columns supporting 5 floors of gravity load to rest on concrete caissons. These caissons are encased in a circular ½" thick steel shell and have vertical reinforcing bars, as well as #4 ties spaced at 12 inches. All bearing limit states were assumed to be adequate for this study, due to caissons and concrete piers resting on or embedded into bedrock.

Table 32 is the caisson schedule provided by the structural engineer of record. Caisson type 18-B was used in the existing design to support the perimeter columns which support 5 levels. As Table 30 displays, this caisson can support up to 720 kips. However, the new design requires caisson 18-B to support all 14 levels of gravity loads. The total design force for the new design of caisson 18-B is 1700 kips, which exceeds the capacity of 720 kips.

Caisson 18-B would have to be increased in size to a 36-C caisson. This requires more concrete and reinforcing steel, but the minimum embedment into the bedrock is the same as an 18-B caisson. See Figure 48 for the location of caissons which must increase in size to a type 36-C caisson (shown in red).

Table 32 – Existing Design Caisson Schedule

CAISSON SCHEDULE				
CAISSON TYPE	DIAMETER	VERT. REINF.	CAISSON CAPACITY (KIPS)	MIN. EMBEDMENT INTO BEDROCK
36–A	36"	17–#18	3,350	14'–0"
36–B	36"	9–#18	2,400	10'–0"
36–C	36"	11–#14	2,000	8'–0"
36–D	36"	8–#10	1,500	6'–0"
18–A	18"	6–#18	950	10'–0"
18–B	18"	7–#14	720	8'–0"
18–C	18"	7–#11	550	6'–0"

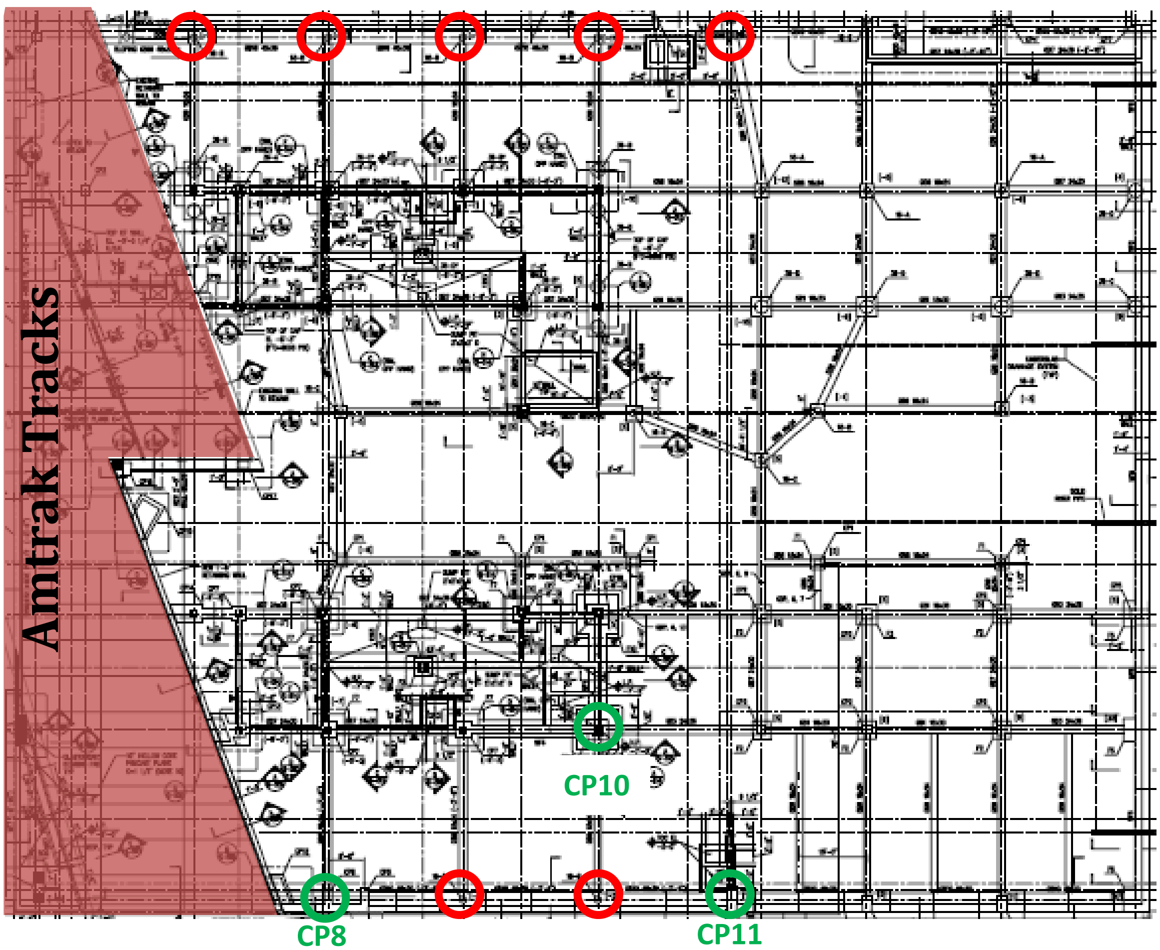


Figure 48 – Foundation Impacts of the New Transfer System. (Red circles represent concrete caisson 18-B which must be increased in size to a 36-C. Green circles are concrete piers, which were determined to be adequate for the new transfer system.)

Possible problematic reinforced concrete piers (shown in green of Figure 48) were also checked. Existing design CP8 was designed for 5 levels of gravity loads and now supports 14 levels of gravity loads, as well as the reaction from Truss 4. CP11 was also designed to support 5 levels of gravity loads, but now supports all 14 levels. CP10 supports a braced frame column, and was also checked for the new transfer system. PCA Column was used to determine the capacity of each concrete pier and each pier was determined to be adequate for the loading conditions of the new transfer system. For a summary of the concrete pier study, see Table 33.

Table 33 – Concrete Pier Study Summary

Concrete Pier	Size	Reinforcement	Design Force	Capacity	Design
CP8	20 x 60 in.	(12) #10	2610 kips	3600 kips	OK
CP10	72 x 42 in.	(24) #11	4710 kips	8000 kips	OK
CP11	24 x 60 in.	(14) #10	1700 kips	4300 kips	OK

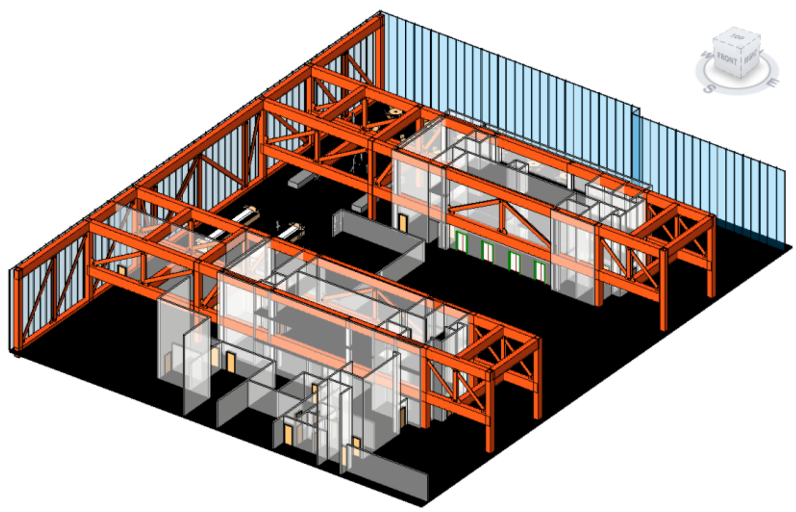
Structural Depth Conclusions

Based on the structural performance studied in this depth study, the transfer trusses at the 5th level of the John Jay College Expansion Project are a viable option to the existing design. By using a less number of trusses, the gravity loads of the tower were effectively transferred to the braced frame core at the 5th level. This transfer system was also analyzed to see how the braced frame core would be impacted. The outcome of this analysis was a more efficient braced frame design. Foundation impacts were minimal for the columns that now support all 14 levels of gravity loads. The remainder of this report studies the architectural and construction impacts of implementing the new transfer system.

Architectural Breadth Study

The 5th level of the John Jay College Expansion Project is used as the facilities dining area. The space is compromised of a kitchen with offices and support areas, a large servery, a café, and a student, faculty, and staff dining area. There is also access to the campus commons, which is a green space located on the roof of the low-rise cascade. The campus commons contain a wide variety of plants, and several areas for students to gather.

By changing the location of the transfer trusses of the John Jay College Expansion Project from the penthouse level to the 5th level, the architecture is dramatically changed. The original design called for 30 foot high transfer trusses which were hidden in the penthouse level, which is primarily mechanical equipment, and not for the public to see. Moving the transfer system to the 5th level requires the system to be exposed and open to the occupants of the expansion project. Not only does the new transfer system need to meet structural requirements, but it also needs to fit the architectural requirements of the 5th level and the entire building.



Since the 5th level is a public space and will be used by mostly everyone on the John Jay College of Criminal Justice campus, it was very important for the design allow pedestrians to circulate throughout the space without any obstruction from the trusses. To allow building occupants to circulate without any obstructions, trusses 2 and 3 are elevated 10 feet. This required trusses 2 and 3 to be 20 feet in depth to efficiently control the axial forces in the top and bottom chords of each truss.

Due to the heavy loads associated with transferring 10 levels of the expansion project using the transfer trusses at the 5th level, the truss members had to be custom steel sections. Parallel plates were designed for the web compression members and single plates were designed for the web tension members. Top and bottom chords are constructed using W-shapes with filler plates or built-up box sections. These custom shapes must be protected from fire using intumescent paint. For this study, a dark orange color was used to paint the trusses. This color was chosen because most of the accents in the building are dark orange.

Building Height Discussion

The original floor-to-floor height of the 5th level was 20 feet. There is also a 5th level mechanical mezzanine, which consists of four air-handling-units, inside of the braced frame core (see Appendix B). However, the new transfer system requires a total depth of 30 feet, which increases level 5 by 10 feet, as well as the total height of the building by 10 feet to 249'-10". The John Jay College Expansion Project is within C6-2 zoning requirements of Manhattan, which has no maximum building height and a maximum setback height of 85 feet. For C6-2 zones, the Sky Exposure Plane (see Figure 49) is defined by a vertical to horizontal distance ratio of 7.6 to 1. At the existing design height of 239'-10", the required setback is approximately 20' and the existing design only provides approximately 15'. Therefore, it is being assumed that a variance was obtained to override this setback requirement or the zoning was changed. This would allow increasing the building height by 10 feet.

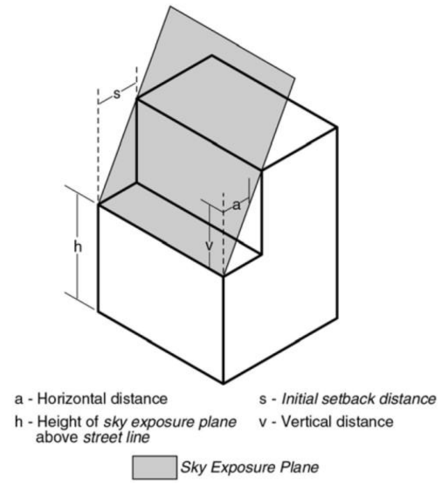


Figure 49 – Sky Exposure Plane

Architectural Plans

The architectural floor plans had to be slightly modified to incorporate the transfer trusses designed for the 5th level. Figure 50a displays the 5th level with the transfer trusses shown in orange. Truss 1 and 4 are the only trusses that span 30 feet from floor-to-ceiling, but they are along the edges of the building, and the floor plan remains open. Therefore, the original goal of permitting the building occupants to circulate throughout the 5th level without obstruction was met.

More architectural changes were made at the 5th floor mezzanine level (see Figure 50b), which was changed from a 10 foot floor-to-floor height to a 20 foot floor-to-floor height. The existing 5th floor mezzanine level has 2 air-handling-units to the East and West of the braced frame core (see Appendix B). To make use of the increased floor-to-floor height for the 5th level transfer trusses, these two air-handling-units were stacked on top of each other in the new 20 foot high mechanical mezzanine space. See Section 3 for a view through the 5th level mechanical mezzanine.

Trusses 2 and 3 span through the building's central core, which created two minor areas of concern. The first area of concern is that the trusses slightly reduce the cross-sectional area of the mechanical ducts running through the core. This reduction in cross-sectional area may cause some velocity problems with the air passing through the ducts, but this problem is not within the scope of this project. It should be noted that this is a consideration which should be examined early in the design process by the mechanical engineer.

The second area of concern is that the truss chords passing through the stairwells reduce the size of the stair landings. Each landing requires a 6 foot radius to be unobstructed for exiting purposes. The horizontal distance between Truss 2 and Truss 3 chords is 24 feet and the horizontal distance required for each stair riser is 10 feet. This allows 7 feet for each landing, which exceeds the minimum requirements.

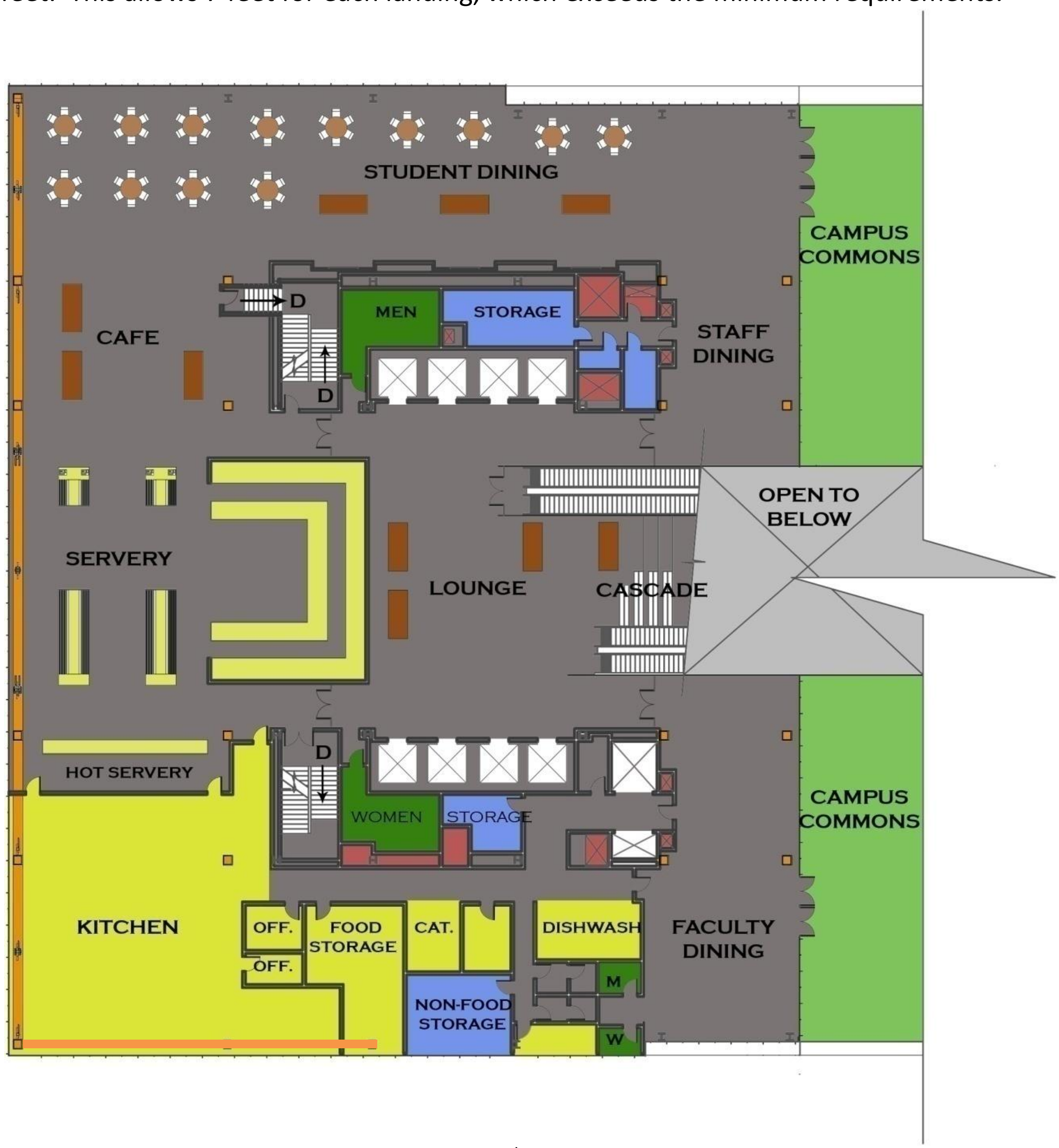


Figure 50a – 5th Level Floor Plan

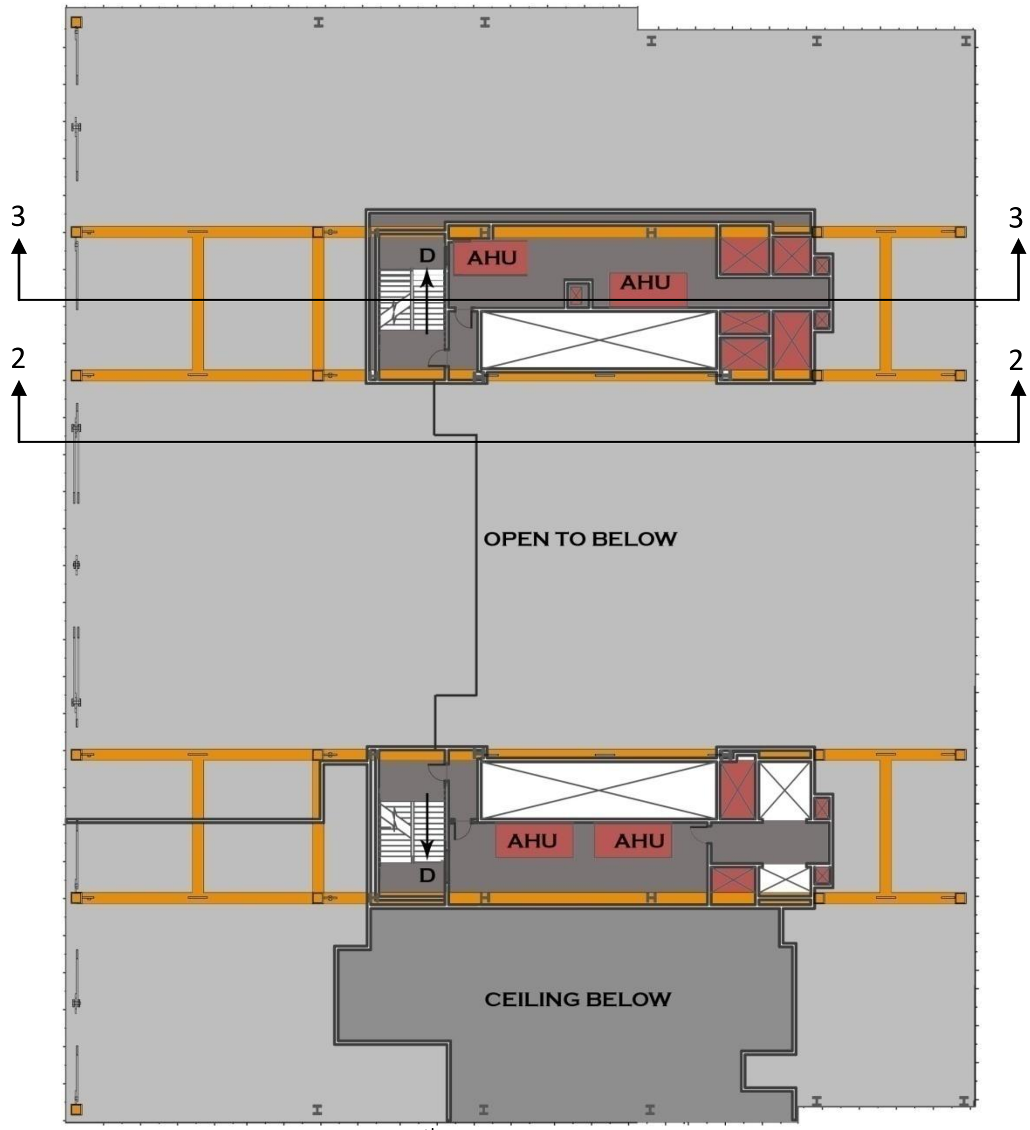
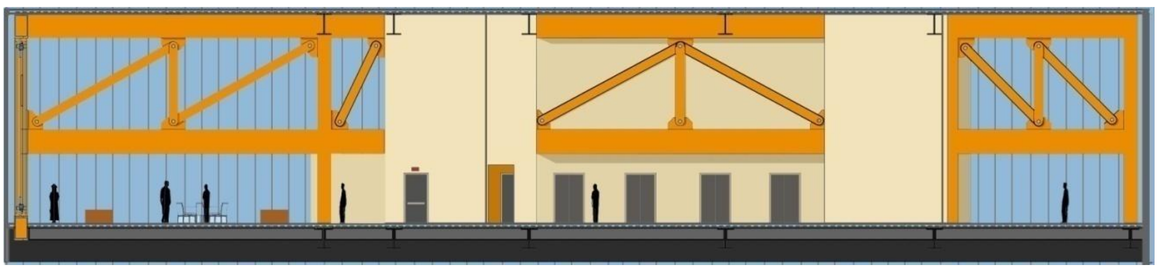


Figure 50b – 5th Level Mezzanine Floor Plan

Building Sections

After adjusting the floor plans of the 5th level and 5th level mezzanine to incorporate the transfer trusses, building sections were created to display how the trusses related 3-dimensionally to the space. Section 2 displays how Truss 3 fits within the architecture of the 5th level. Building occupants are able to pass underneath the truss with ample headroom. It also can be seen that by elevating Truss 2, the openings in the core of the building to access to the elevators, fire exits, and restrooms are avoided. Section 3 demonstrates how the 10 foot increase in floor height will be utilized. Each initial mezzanine had a floor-to-floor height of 10 feet and required 4 air-handling-units. By increasing the floor-to-floor height by 10 feet to a total of 20 feet, the air-handling-units could be stacked on top of each other using steel framing. This would have to be analyzed by the mechanical engineer and is not within the scope of this project.



Section 2 – Looking North Along Truss 2

Section 3 – Looking North Through Core

Renderings

An AutoDesk Revit model of the 5th level was constructed to ensure the trusses fit 3-dimensionally with the space and architecture. Figure 51 displays the points at which renderings were taken from the model.

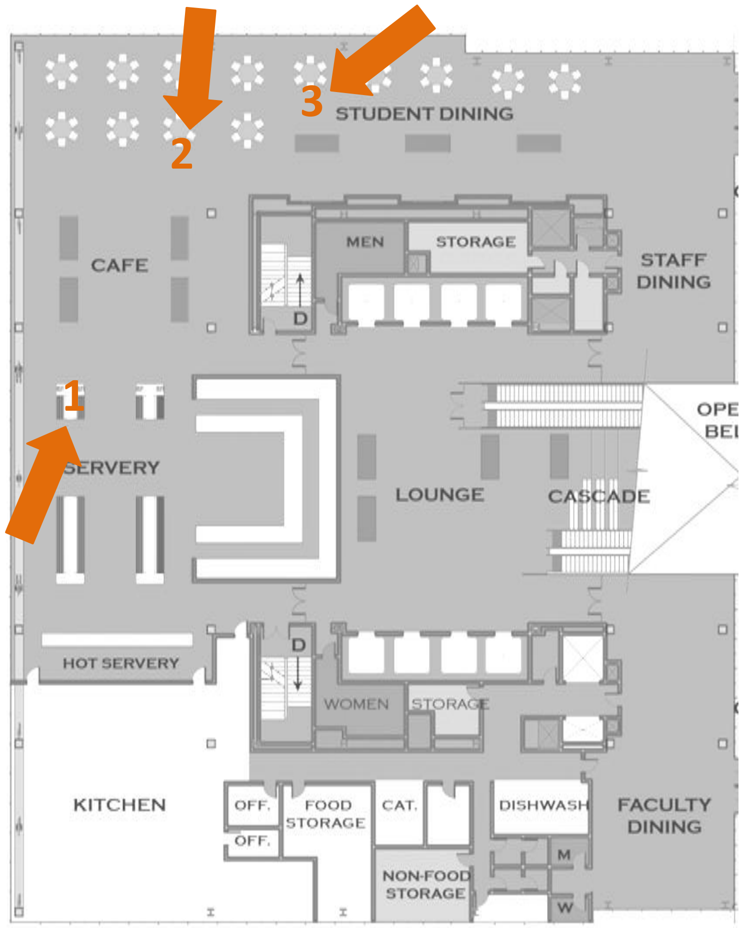


Figure 51 – Rendering Views

Rendering 1

Rendering 2

Rendering 3

The transfer trusses are also visible from the exterior of the building. To further expose Truss 1, the curtain wall was changed by removing the aluminum fins from the West side of the building at the 5th level only. This allows pedestrians from street level to view the trusses. Figure 52 displays the existing expansion project at night and was taken from SBLD Studio. Figure 53 shows the new transfer system with the modified curtain wall.

Figure 52 – Existing Exterior Rendering (From SBLD Studio)

Figure 53 – New 5th Level Transfer System Exterior Rendering

Architectural Breadth Conclusions

The new transfer trusses at the 5th level were successfully incorporated into the architecture of the 5th level. Pedestrian circulation was not disturbed by the interior trusses and the floor plan remains open. The trusses also add a sense of importance to the 5th level of the John Jay College Expansion Project, which is a nice feature because all students will use this space. One could argue that Truss 1 may block the views out of the 5th floor level. However, in Manhattan the view from the 5th floor of a building is typically of another building - and in this case the other building is a factory. After studying the architectural impacts of the new transfer system, it is still a possible option.

Construction Management Breadth

One of the main objectives of this thesis was to design a more constructible structure for the John Jay College Expansion Project. Therefore, the objective of this study is to verify that the transfer system discussed in this report has a simpler construction method than that of the original design. There are many differences between the erection sequence and construction management of the existing transfer system design and the transfer system discussed in this thesis. Two main differences will be examined in this breadth study to display the differences in constructability: cost and sequencing.

A cost analysis was completed using material, labor, overhead, and profit costs obtained from Turner Construction Company. Overall costs of each transfer system were calculated based on weight.

The existing transfer system requires a very experienced construction team due to the difficult nature of hanging the structure. Temporary supports and braces are required until the connection of plate hangers to floor girders can be made. Deflections of each floor must be closely monitored throughout the construction process and the stresses in the built-up girders at the first level must be closely monitored. Placing concrete decking cannot begin until the penthouse trusses are complete and temporary supports are removed because the built-up girders at the 1st level do not have the capacity to support 14 levels of full dead load.

The new transfer system was designed to simplify the construction process, and therefore a steel erection sequence was prepared to justify this goal. This erection sequence remains identical to the existing design until the 5th level, which is where the truss construction begins. Once the trusses are complete, typical steel framing is used until erection is complete. Temporary supports and bracing is not needed once truss construction is completed and there is no need to check stress levels in the 1st level built-up box sections because construction loads from levels 5 through 14 are directly transferred to the braced frame core from the trusses. Therefore, concrete decking of each level can be placed once the steel erection of the level above is complete, rather than waiting until steel erection of the entire tower is complete.

Cost Study

A few simplifying assumptions were made when performing the cost analysis for this study. They are as follows:

- Cost differentials due to fireproofing the truss members were neglected. Cementitious spray on fireproofing is used for the existing design, where intumescent paint is used for the exposed trusses designed in this report. It is assumed that fireproofing costs are about the same since the number of members needing intumescent paint are about half of those needing cementitous spray on fireproofing.
- A 10% premium was added to built-up and custom steel members

- Material and labor costs used were provided by Turner Construction Company from 2005 to allow for a direct cost comparison between transfer systems
- Unit costs were increased by 15 % to include overhead and profit
- Foundation changes were minimal and therefore were neglected

It should also be noted that two additional premiums are included in the original estimate which can be omitted from the new cost analysis. The first premium charged was for hanger construction. Using the new transfer system, there are no hanging members, which results in additional savings for the owner. Another premium was charged for the difficult construction of the tower. This premium can also be neglected since typical steel framing is used for the tower in the new transfer system.

Existing Transfer System Cost

Using the unit cost information for material and labor provided by Turner Construction Company, detailed steel takeoffs were performed for the various systems used to transfer gravity loads over the Amtrak tracks. These include the penthouse transfer trusses, perimeter plate hangers, the braced frame core within the tower, and the built-up girders beneath the 1st level (see Appendix J for takeoff values).

The total estimated structural cost of the existing transfer system is $6.15 million. See Table 34 and Appendix J for more information.

Table 34 – Existing Transfer System Cost Analysis

Existing Transfer System Cost Comparison						
System	Quanitity (tons)	Material ($/ton)	Labor ($/ton)	Total ($/ton)	Total w/ 15% O&P ($/ton)	Total Cost ($)
Trusses	761	1320	1980	3300	3795	2886098
Perimeter Plate Hangers	54	1452	1980	3432	3947	211154
Braced Frame Core	652	1320	1980	3300	3795	2474340
Built-Up Box Sections	147	1452	1980	3432	3947	580180
					TOTAL (Million):	**6.15**

Thesis Transfer System Cost

The same unit cost information provided by Turner Construction Company was applied to estimate the costs associated with the new transfer system. Appendix J includes steel takeoff information. Table 35 displays the cost of each contribution to the transfer system for a total structural cost of $ 5.91 million. An additional $ 820,000 is associated with the curtain wall costs of increasing the total building height by 10 feet. This brings the total cost of the new transfer system to $ 6.74 million.

Table 35 – Thesis Transfer System Cost Analysis

Thesis Transfer System Cost Comparison						
System	Quanitity (tons)	Material ($/ton)	Labor ($/ton)	Total ($/ton)	Total w/ 15% O&P ($/ton)	Total Cost ($)
Trusses	690	1452	1980	3432	3947	2723292
Perimeter Columns	56	1320	1980	3300	3795	212520
Braced Frame Core	662	1320	1980	3300	3795	2512290
Built-Up Box Sections	118	1452	1980	3432	3947	464144
					TOTAL (Million):	**5.91**

Steel Sequencing Study

According to the construction schedule from Turner Construction Company, the structural steel erection began in May of 2008 and is scheduled to be completed in July of 2009. This amounts to a total of 62 weeks to erect the steel superstructure. The steel erection sequence for the John Jay Expansion Project includes beginning erection at 11th avenue (to the West) and erect towards the existing building (to the East). Tower erection will then continue while steel framing in the cascade area is topped out.

Rather than using the schedule provided by Turner Construction Company, the assumptions listed in Table 36 were used to create a steel erection sequence for the existing design and the thesis design to allow for a direct comparison. Durations were determined by assuming 40 pieces of steel would be erected per day.

Table 36 – Steel Sequencing Assumptions

Activity	Thesis (Duration in Days/Level)	Existing (Duration in Days/Level)
Erect Columns	1	1
Erect Braced Frames	1	1
Erect Typical Floor Framing	7	7
Decking and Detailing	10	10
Erect Temporary Columns	1	1
Erect Reinforced Plate Hangers	N/A	1
Erect Truss Bottom Chords	3	4
Erect Truss Top Chords	2	4
Erect Truss Web Members	3	6
Detail and Plum Trusses	5	10
Remove Temporary Columns/Reinforced Plates	1[1]	5[1]
Placing Concrete Decking	10[2]	2[3]

[1] - Unit is Total Days
[2] – Includes duration of embeds, box outs, rebar, and placing concrete
[3] – Includes placing concrete

Existing Construction Schedule

Using the assumptions listed in Table 36, a steel erection sequence was created based on the original structural design. Steel erection was determined to take a total of 63 weeks, which verifies that the assumptions used are fairly accurate (Turner's schedule estimated 62 weeks). Appendix J displays the entire existing schedule created for this study and Figure 54 displays a summary of the existing schedule. When placing of the concrete decking was taken into account, the total superstructure construction time increased to 70 weeks, because concrete work cannot begin until the penthouse trusses are complete (see Figure 55). This duration was determined assuming that all embeds, box outs, and rebar were put in place while steel erection was topped out. Therefore, when the penthouse trusses are complete, each floor only takes 2 days to complete concrete work.

	Task Name	Duration	Start	Finish
1	East Foundation	190 days	Tue 4/1/08	Mon 12/22/08
2	Steel Erection Podium West	82 days	Mon 5/19/08	Tue 9/9/08
3	Steel Erection Podium East	93 days	Tue 12/23/08	Thu 4/30/09
4	Steel Erection Tower	182 days	Wed 9/10/08	Thu 5/21/09
5	Truss Erection	46 days	Fri 5/22/09	Fri 7/24/09
6	Temporary Support Removal	8 days	Mon 7/27/09	Wed 8/5/09

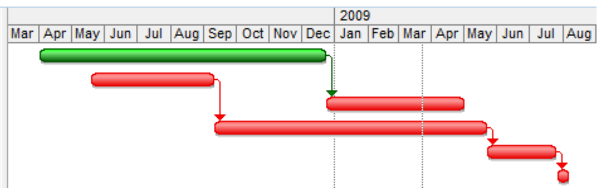


Figure 54 – Existing Design Steel Sequence Summary

	Task Name	Duration	Start	Finish
1	East Foundation	190 days	Tue 4/1/08	Mon 12/22/08
2	Steel Erection Podium West	82 days	Mon 5/19/08	Tue 9/9/08
3	Steel Erection Podium East	93 days	Tue 12/23/08	Thu 4/30/09
4	Steel Erection Tower	182 days	Wed 9/10/08	Thu 5/21/09
5	Truss Erection	46 days	Fri 5/22/09	Fri 7/24/09
6	Temporary Support Removal	8 days	Mon 7/27/09	Wed 8/5/09
7	Tower Concrete Deck	28 days	Thu 8/6/09	Mon 9/14/09

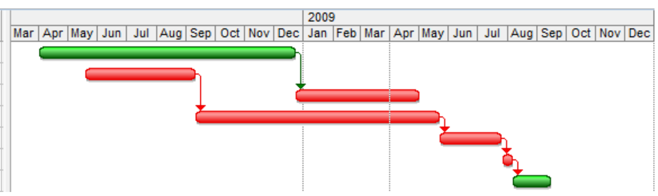


Figure 55 – Existing Design Steel Sequence Summary Including Concrete Decking

New Construction Schedule

A steel erection sequence for the new design was created using the assumptions presented in Table 36. Total steel erection time was found to be 60 weeks, which is 3 weeks less than the existing design. See Appendix J for a complete schedule of the new design and Figure 56 for a summary. Total superstructure construction time increased to 64 weeks when considering placing concrete decking, which is shown in Figure 57.

	Task Name	Duration	Start	Finish
1	East Foundation	190 days	Tue 4/1/08	Mon 12/22/08
2	Steel Erection Podium West	84 days	Mon 5/19/08	Thu 9/11/08
3	Truss Erection	39 days	Tue 8/19/08	Fri 10/10/08
4	Temporary Support Removal	1 day	Mon 10/13/08	Mon 10/13/08
5	Steel Erection Podium East	93 days	Tue 12/23/08	Thu 4/30/09
6	Steel Erection Tower	182 days	Wed 10/29/08	Thu 7/9/09

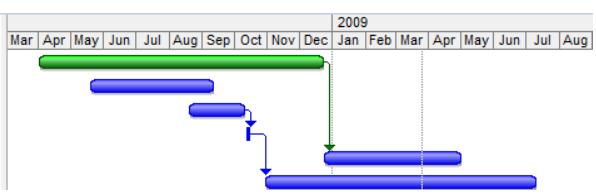


Figure 56 – Thesis Design Steel Sequence Summary

		Task Name	Duration	Start	Finish
1		East Foundation	190 days	Tue 4/1/08	Mon 12/22/08
2		Steel Erection Podium West	84 days	Mon 5/19/08	Thu 9/11/08
3		Truss Erection	39 days	Tue 8/19/08	Fri 10/10/08
4		Temporary Support Removal	1 day	Mon 10/13/08	Mon 10/13/08
5		Steel Erection Podium East	93 days	Tue 12/23/08	Thu 4/30/09
6		Steel Erection Tower	182 days	Wed 10/29/08	Thu 7/9/09
7		Tower Concrete Deck	150 days	Fri 12/26/08	Thu 7/23/09

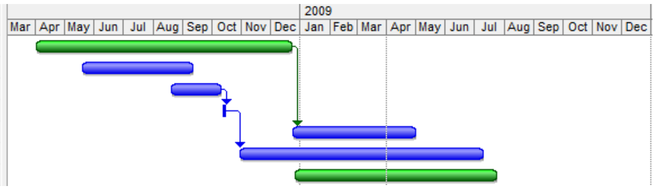


Figure 57 – Thesis Design Steel Sequence Summary Including Concrete Decking

Construction Management Conclusion

As displayed by Table 37, the cost study of the two transfer systems resulted in similar results. Total costs of the new transfer system, including the costs associated with increasing the curtain wall height 10 feet, are $ 6.74 million. The existing design was determined to cost $ 6.15 million. This cost does not include the expensive premiums charged for the difficult construction of the hanging structure of the existing design, and therefore the two transfer systems cost about the same.

Major differences can be seen between the construction schedules of the two transfer systems. When examining the steel erection sequence of each system, the transfer system designed in this thesis takes 60 weeks, where the existing design takes 63 weeks. This 3 week decrease in steel erection time of the new transfer system is created by using less pieces of steel for the transfer trusses than the existing design, as well as the need for only 1 level of temporary supports, where the existing design needs 9 levels of temporary supports.

These differences are increased when the placing of concrete decking is inspected. For the existing transfer system, concrete work cannot begin until the penthouse trusses are complete and the temporary columns are removed. This is because the built-up girders at the 1st level must support all 14 levels of steel framing during construction and any additional dead loads would overstress the girders. When the penthouse trusses are complete, the temporary supports are removed. This allows the building to hang, and gravity loads of levels 6 through the roof are transferred using the penthouse trusses. Now, only levels 1 through 5 are supported by the built-up girders at the 1st level, and the placement of concrete decking can begin. This increases the superstructure construction time to 70 weeks.

The new transfer system design simplifies the construction process of the superstructure. Traditional steel erection methods can be used to erect the structure, with the exception of building the transfer trusses at the 5th level. Concrete decking can be placed slightly after steel erection begins, given that the 5th through roof levels will be directly transferred over the Amtrak tracks using the trusses at level 5, rather than using the built-up girders at level 1. When including the placing of concrete decking, the total superstructure construction time amounts to 64 weeks, which is 6 weeks less than the existing design.

Table 37 – Construction Management Study Summary

	Thesis Transfer System	Existing Transfer System
Structural System Cost	$ 5.91 Million	$ 6.15 Million
Total Cost	$ 6.74 Million*	$ 6.15 Million
Steel Erection Schedule (Weeks)	60	63
Entire Superstructure Schedule (Weeks)	64	70

* - Includes increased cost of curtain wall

Final Conclusions and Recommendations

Many buildings in major cities are often are built over site restrictions. These site restrictions are challenging, but allow for architects and engineers to design creative solutions. This report focuses on the design of an alternative transfer option for the John Jay College Expansion Project. After studying the existing design, a few problematic areas were determined and were attempted to be corrected in this study. The existing system transfers gravity loads to the braced frame core at the penthouse level, which requires heavy column sections at the top of the braced frame core. These penthouse trusses are located at the top of the expansion project, and therefore untraditional construction methods must be used to erect the structure using temporary supports.

For this thesis, exposed steel trusses were designed at the 5th level of the John Jay College Expansion Project. Custom built-up steel sections were chosen for truss members due to the high forces being resisted and architectural considerations. A total of 6 trusses were used to transfer the necessary columns over the Amtrak tracks beneath the building, using approximately half of the number of members as the existing penthouse trusses.

These steel trusses were also designed to fit within the architecture of the 5th floor. Trusses had to be designed to avoid interfering with the function of the spaces on the 5th level. Therefore, interior trusses were elevated 10 feet to allow building occupants to pass beneath, as well as to avoid openings within the buildings core.

After the gravity design of the transfer trusses was complete, the braced frame core was re-designed. As expected, this resulted in reduced braced frame column sizes at the top of the building, due to gravity loads being transferred at the 5th level rather than the top. Bracing members increased in size, but this was accounted to the difference in the lateral design forces between the existing design and this thesis design. By using the new transfer system designed in this thesis report, a more efficient braced frame core can be used.

The main goal of designing an alternate transfer system was to design a more constructible structure. By using the transfer system designed in this thesis, conventional steel framing construction can be used. A steel erection sequence was created and it was determined that the alternative transfer system took 3 weeks less than the existing design. When placing concrete decking was taken into account, the differences were magnified with the new design taking 30 weeks less than the existing design. These decreases in construction time did not sacrifice any money for the owner, as each transfer system was determined to cost about the same. By using less pieces of steel for the transfer trusses and allowing traditional construction methods to be used, a more constructible structural system was design for the John Jay College Expansion Project.

The final recommendation is to use the transfer system designed in this project. Using transfer trusses at the 5th level of the John Jay College Expansion project does not have a negative impact on the space. In this study these transfer trusses were chosen as accents for the building, but they could also be painted a color which is less noticeable if desired by the owner. This transfer system was also proven to be more constructible than the existing design.

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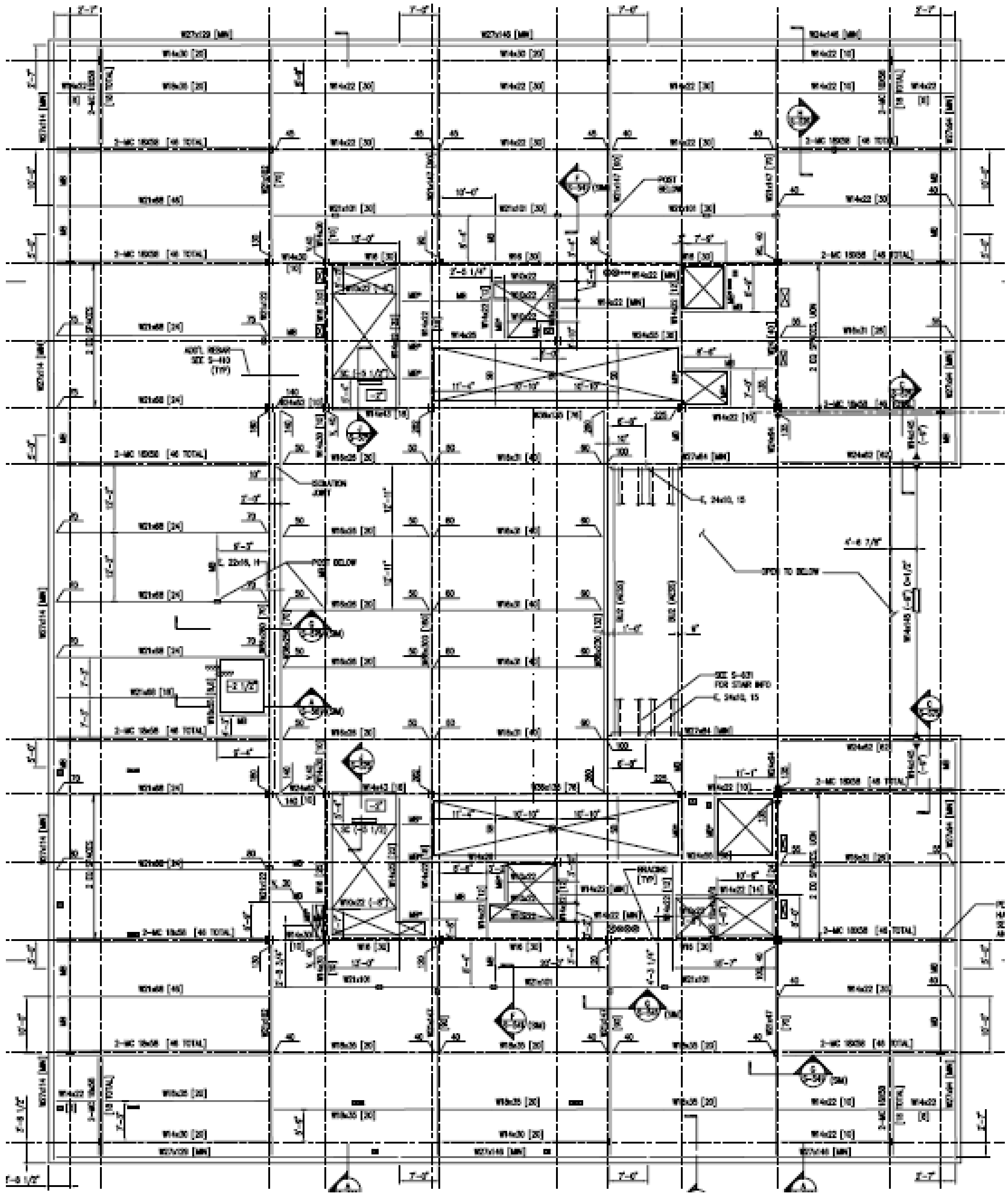
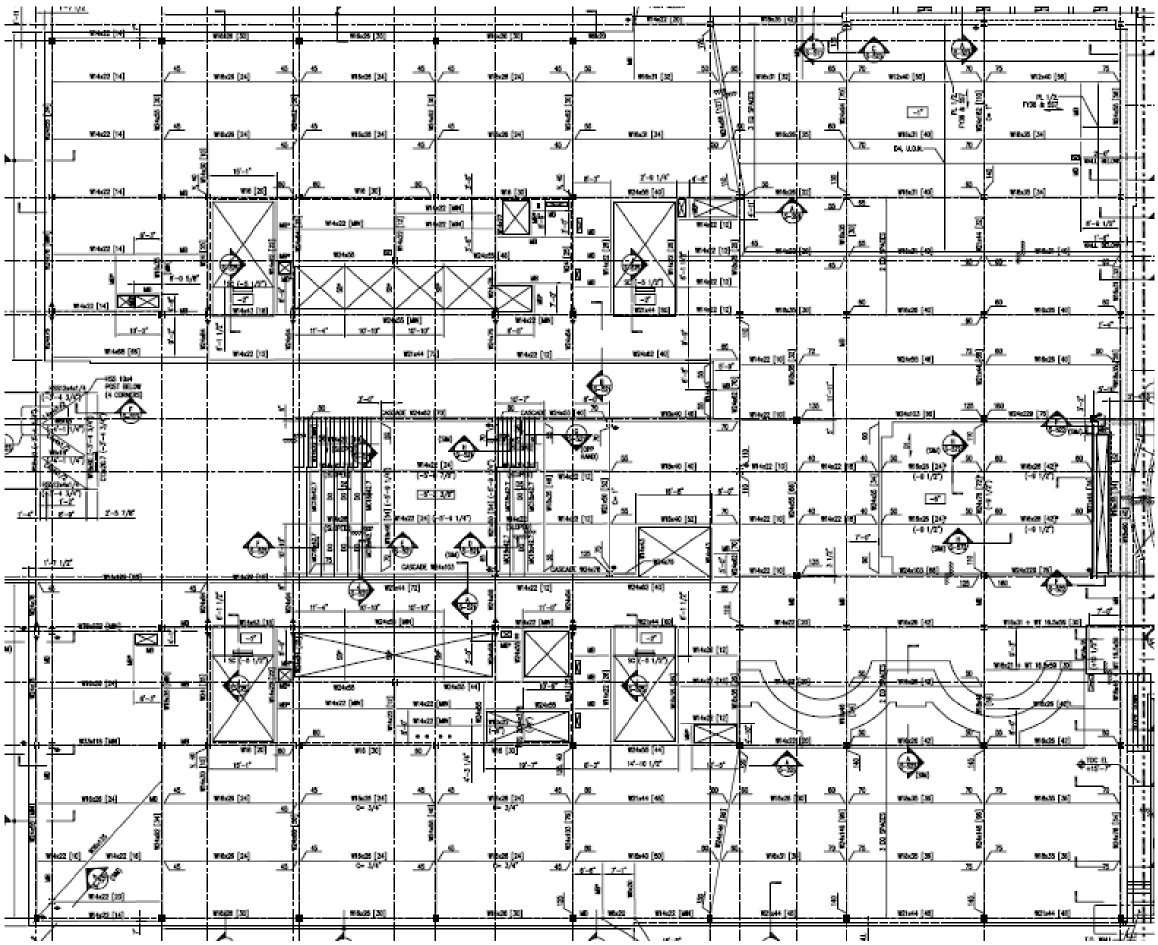
Post, Nadine M., "Newseum Builders Press on To Make the Media the Message". Engineering News Record, December 11th, 2006.

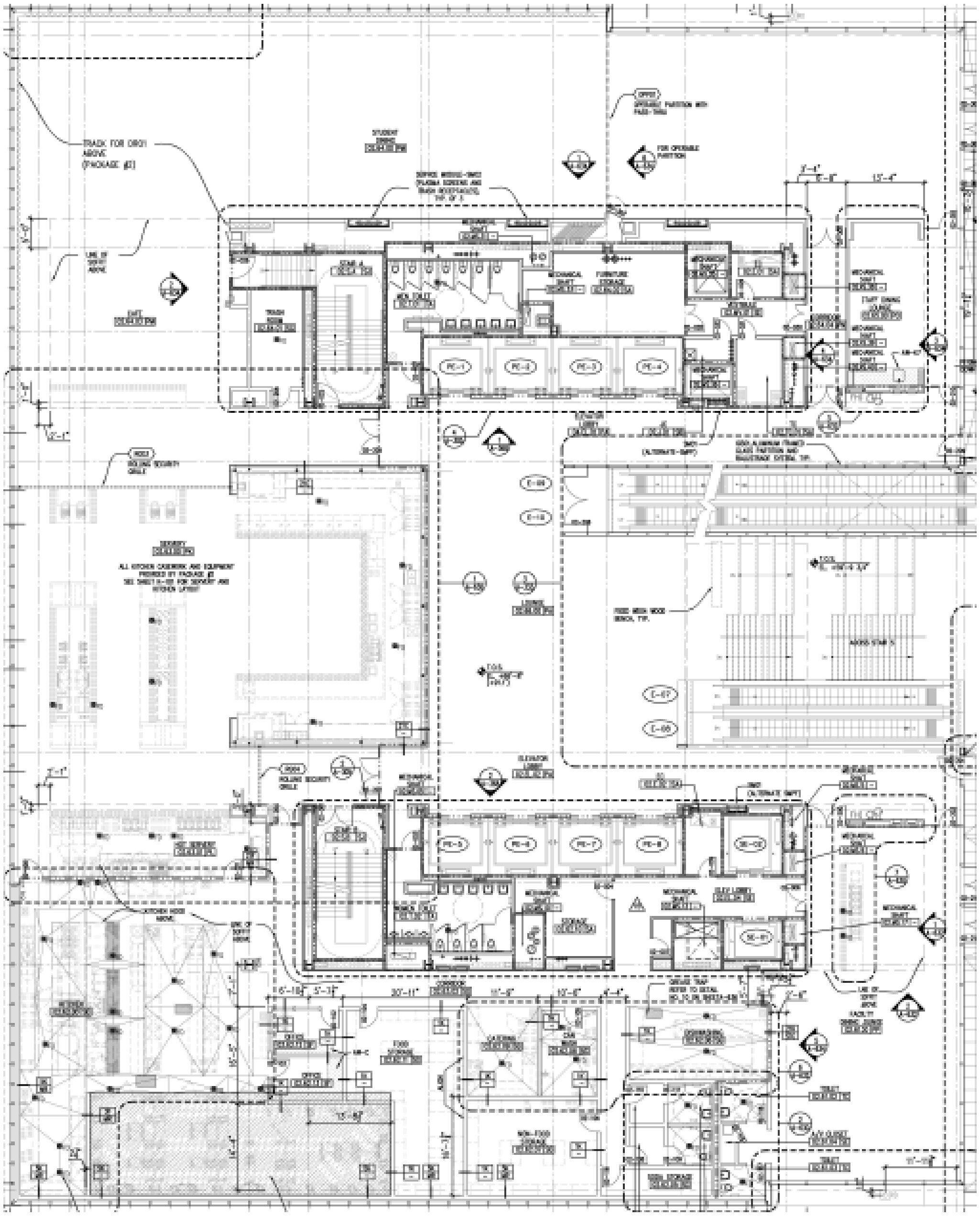
Appendix A – Typical Framing Plans



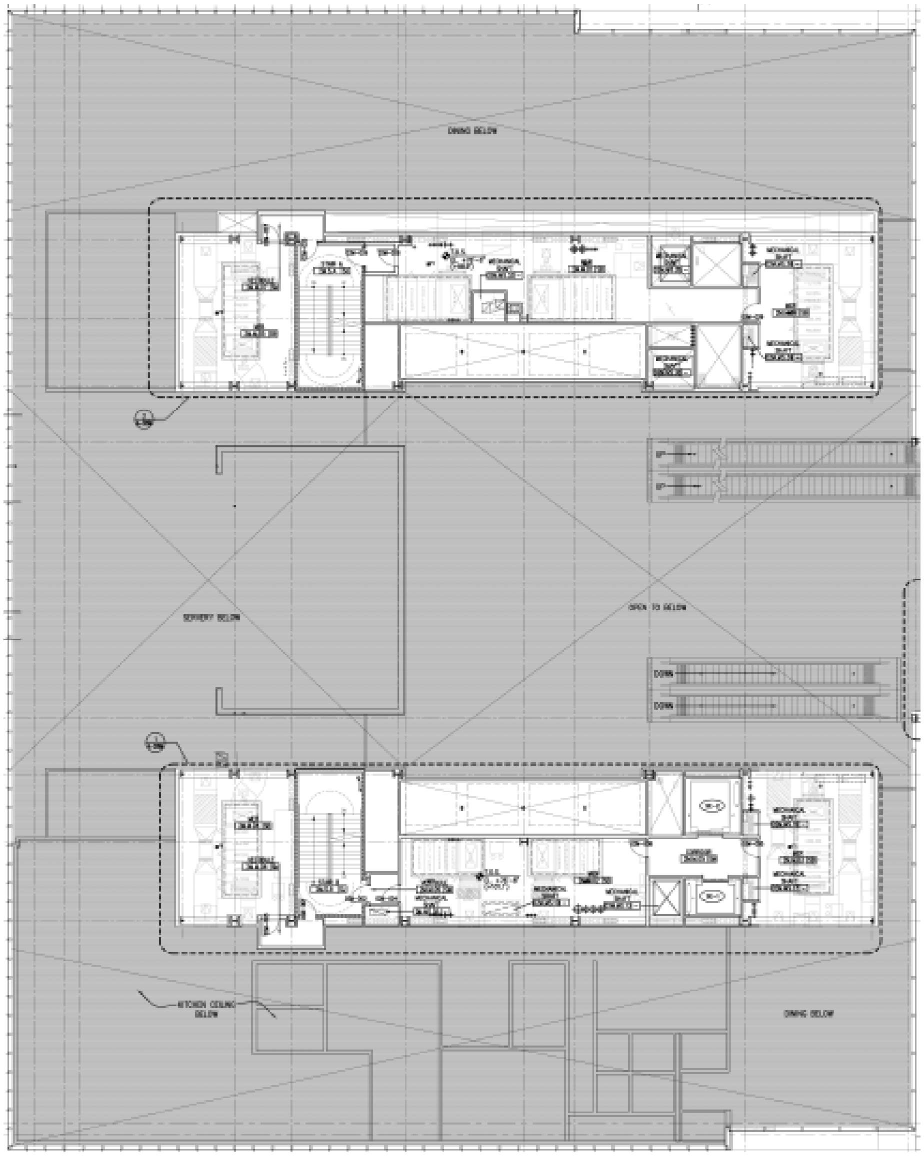
Typical Tower Framing Plan


Typical Cascade Area Framing Plan

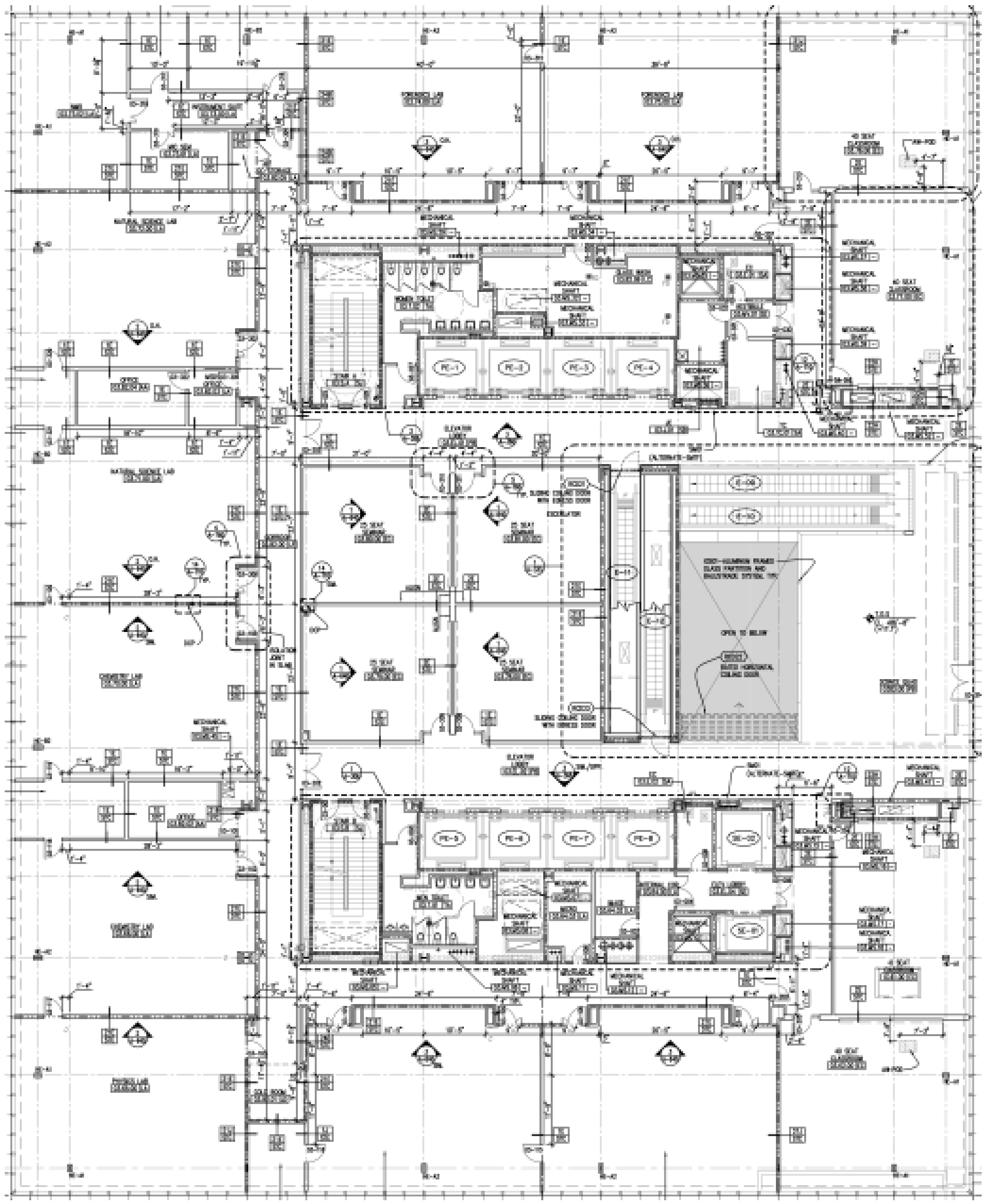
Appendix B – Existing Architectural Plans



Fifth Floor – Partial Floor Plan



Fifth Floor Mezzanine– Partial Floor Plan



Sixth Floor – Partial Floor Plan

Appendix C – Wind Load Calculations

V=	110	mph
K_d=	0.85	
I=	1.15	
K_{zt}=	1	
Exposure:	B	

C_p Value	N-S	E-W
Windward wall	0.8	0.8
Leeward Wall	-0.454	-0.5
Side Wall	-0.7	-0.7

Gust Effect Factors		
	N-S	E-W
B	163.33	200.67
L	200.67	163.33
h	249.5	249.5
n_1	0.40	0.40
Structure:	FLEXIBLE	FLEXIBLE
g_R	3.966	3.966
z	149.7	149.7
I_z	0.233	0.233
L_z	527	527
Q	0.806	0.798
V_z	105.95	105.95
N_1	1.994	1.994
R_n	0.089	0.089
R_h	0.204	0.204
n=	4.342	4.342
R_B	0.290	0.245
n=	2.842	3.492
R_L	0.082	0.100
n=	11.690	9.515
R	0.245	0.227
G_f	0.848	0.840

Appendix D – Seismic Load Calculations

The following table displays the assumptions used to calculate the seismic forces using ASCE 7-05.

S_s=	0.35	%g	
S_1=	0.06	%g	
Occupancy Category=	III		
Site Class=	C		
F_a=	1.2		
F_v=	1.7		
S_{ms}=	0.42		
S_{m1}=	0.102		
S_{DS}=	0.28		
S_{D1}=	0.068		
T_a=	1.256		
$0.8T_s$=	0.194	$< T_a$	
SDC=	B		Table 11.6-1
SDC=	B		Table 11.6-2
SDC=	**B**		Can use Equivalent Lateral Force Procedure
T_s=	0.243		
R=	3		Ordinary steel concentrically braced frames
I=	1.25		Occupancy Category III
T_a=	1.256		
T_b=	4.500		
T=	2.134		
C_u=	1.7		
T_L=	6	seconds	
C_s =	0.0133		
k=	1.38		
W=	81866	Kips	
V=	1087	Kips	

Appendix E – Gravity Load Takedowns

The following data is a sample calculation from the gravity load takedowns performed to use for the design of the transfer trusses and perimeter columns. More information is available upon request.

Column Takedowns Column: 1

Level	Floor Loads						Wall Loads	
	H	A_{trib}	CDL	SDL	LL	LL_{red}	Curtain Wall	Area
	ft	ft^2	psf	psf	psf	psf	psf	ft^2
Roof	30	361	75	92	30	30.0	15	525
14	15	361	89	26	150	120.0	15	787.5
13	15	361	58	30	50	22.4	15	525
12	15	361	58	30	50	22.4	15	525
11	15	361	58	30	50	22.4	15	525
10	15	361	58	30	50	22.4	15	525
9	15	361	58	30	50	22.4	15	525
8	15	361	58	19	100	80.0	15	525
7	15	361	58	19	100	80.0	15	525
6	15	361	58	19	100	80.0	15	525

Live Load Reductions

L_o= 100 psf

K_{LL}= 2 CORNER COLUMN

A_t= 361 ft^2

Factor = 0.45

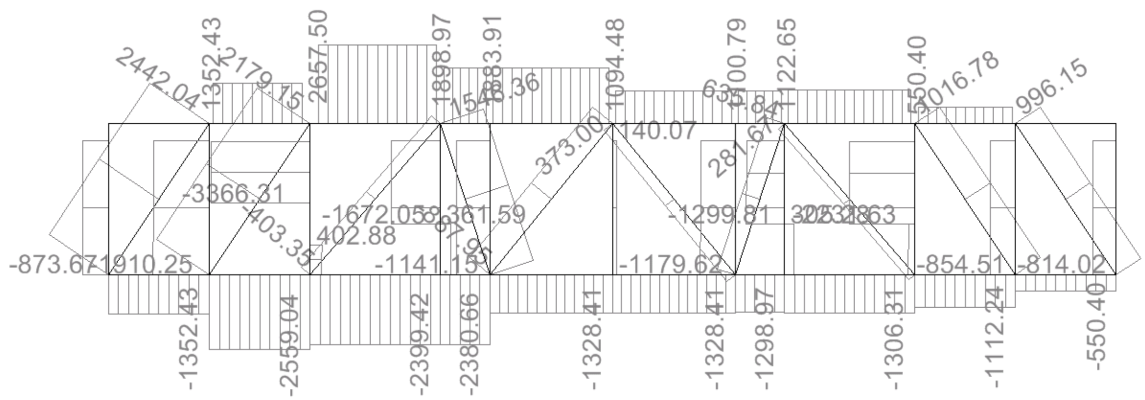
Level	SERVICE LOADS							
	CDL K	SDL K	DL K	LL K	LLred K	TL K	TLred K	WALL K
Roof	27.1	33.2	60.3	10.8	10.8	79.0	79.0	7.9
14	32.1	9.4	41.5	54.2	43.3	107.5	96.6	11.8
13	20.9	10.8	31.8	18.1	8.1	57.7	47.7	7.9
12	20.9	10.8	31.8	18.1	8.1	57.7	47.7	7.9
11	20.9	10.8	31.8	18.1	8.1	57.7	47.7	7.9
10	20.9	10.8	31.8	18.1	8.1	57.7	47.7	7.9
9	20.9	10.8	31.8	18.1	8.1	57.7	47.7	7.9
8	20.9	6.9	27.8	36.1	28.9	71.8	64.6	7.9
7	20.9	6.9	27.8	36.1	28.9	71.8	64.6	7.9
6	20.9	6.9	27.8	36.1	28.9	71.8	64.6	7.9
Level	**TAKEDOWN**							
Roof	27.1	33.2	60.3	10.8	10.8	79.0	79.0	7.9
14	59.2	42.6	101.8	65.0	54.2	186.5	175.6	19.7
13	80.1	53.4	133.6	83.0	62.2	244.2	223.4	27.6
12	101.1	64.3	165.3	101.1	70.3	301.9	271.1	35.4
11	122.0	75.1	197.1	119.1	78.4	359.5	318.8	43.3
10	143.0	85.9	228.9	137.2	86.5	417.2	366.5	51.2
9	163.9	96.7	260.6	155.2	94.5	474.9	414.2	59.1
8	184.8	103.6	288.4	191.3	123.4	546.7	478.8	66.9
7	205.8	110.5	316.2	227.4	152.3	618.5	543.3	74.8
6	226.7	117.3	344.0	263.5	181.2	690.3	607.9	82.7

Level	FACTORED LOADS					
	1.2DL	1.2WALL	1.6LL	1.6LLred	TL	TLred
	K	K	K	K	K	K
Roof	72	9	17	17	99	99
14	50	14	87	69	151	133
13	38	9	29	13	76	60
12	38	9	29	13	76	60
11	38	9	29	13	76	60
10	38	9	29	13	76	60
9	38	9	29	13	76	60
8	33	9	58	46	101	89
7	33	9	58	46	101	89
6	33	9	58	46	101	89
Level	**TAKEDOWN**					
Roof	72	9	17	17	99	99
14	122	24	104	87	250	232
13	160	33	133	100	326	293
12	198	43	162	112	403	353
11	237	52	191	125	479	414
10	275	61	219	138	556	474
9	313	71	248	151	632	535
8	346	80	306	197	733	624
7	379	90	364	244	833	713
6	413	99	422	290	934	802

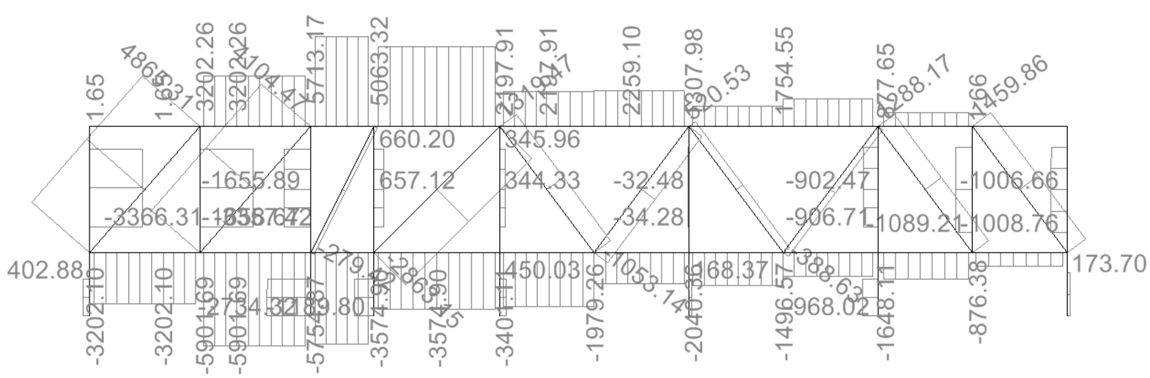
Appendix F – Truss Member Forces

The following diagrams were taken from the detailed gravity analysis performed using ETABS. The controlling load combination was 1.2D + 1.6L.

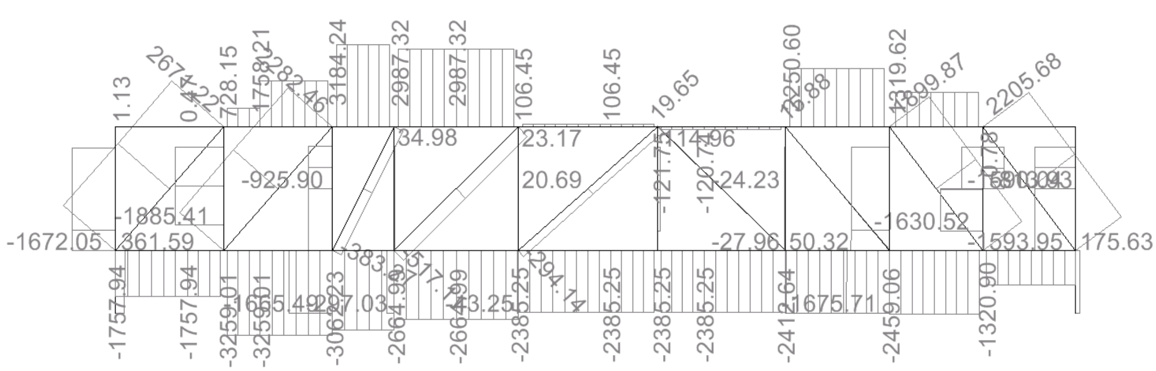
Axial Forces



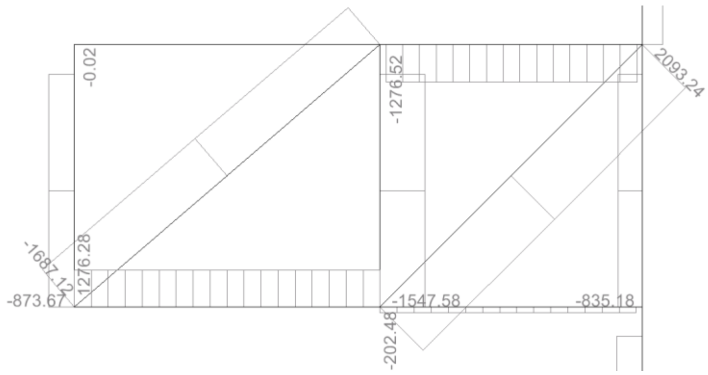
Truss 1 Axial Force (kips)



Truss 2 Axial Force (kips)

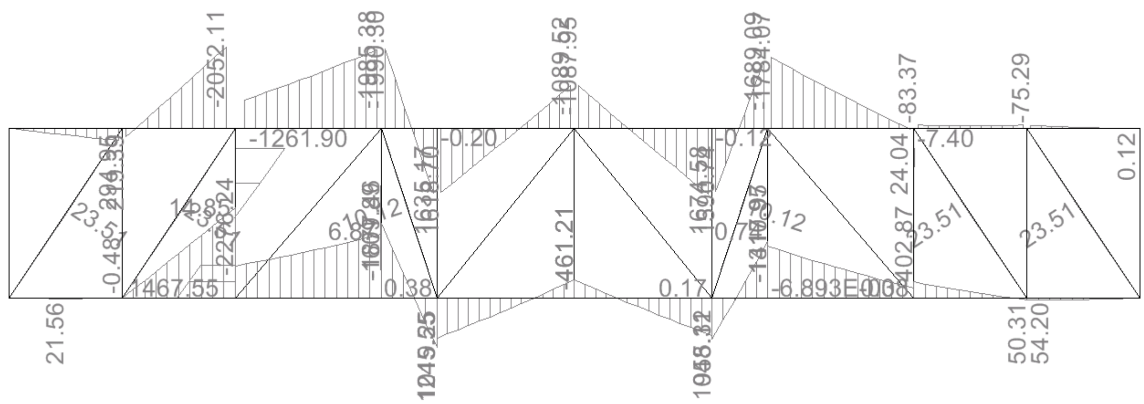


Truss 3 Axial Force (kips)

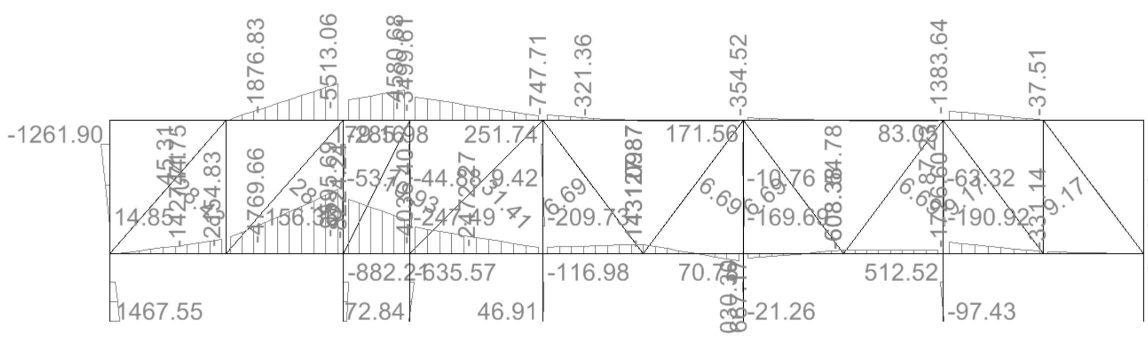


Truss 4 Axial Force (kips)

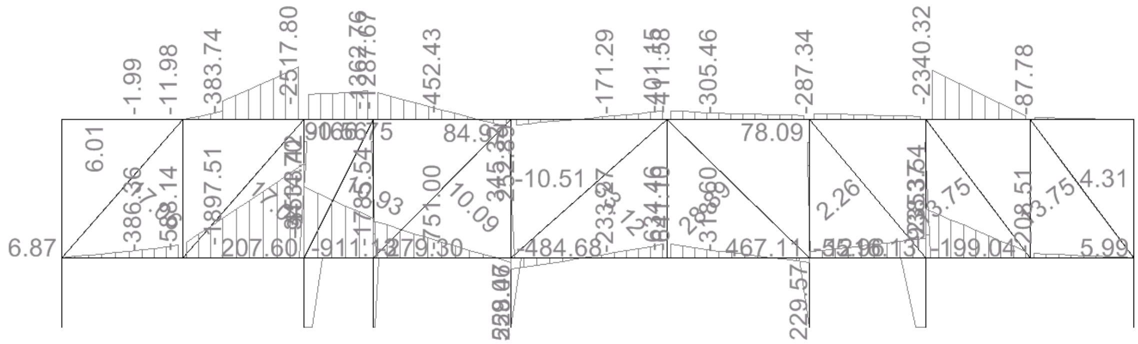
Moments



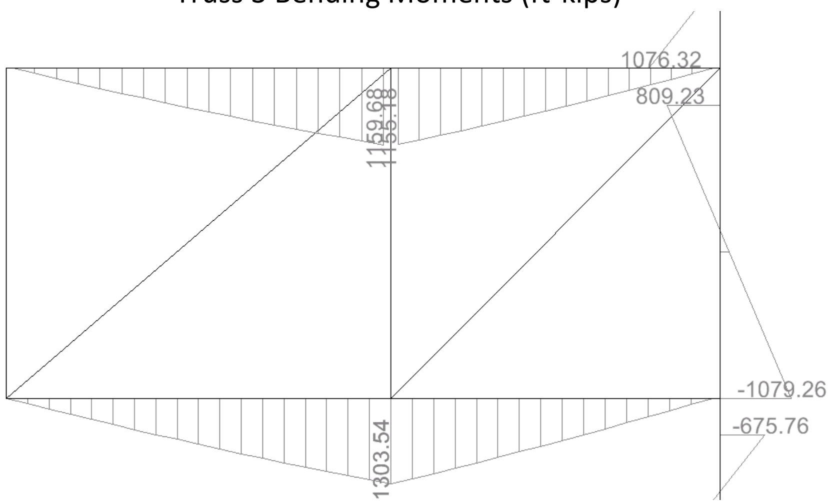
Truss 1 Bending Moments (ft-kips)



Truss 2 Bending Moments (ft-kips)



Truss 3 Bending Moments (ft-kips)



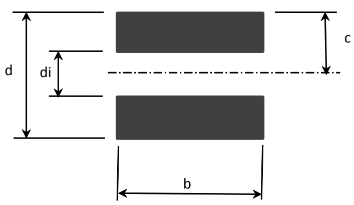
Truss 4 Bending Moments (ft-kips)

Appendix G – Truss Member Design Calculations

The following calculations are sample calculations for the design of various built-up sections used for architecturally exposed truss members. More information is available upon request.

Typical Web Compression Member

Truss #1
Design: C1



d=	14	in
di=	8	in
b=	16	in

plf= 327

	Strong		Weak
A=	96.0	in^2	
c=	7.0	in	8
I=	2976.0	in^4	2048.0
S=	425.1	in^3	256
r=	5.6	in	4.618802
Z=	528.0	in^3	384

Individual Plates

	Strong		Weak
A=	48	in^2	48
c=	1.5	in	8
I=	36	in^4	1024
S=	24	in^3	128
r=	0.866025	in	4.618802
Z=		in^3	

E=	29000	ksi
Fy=	50	ksi
phi=	0.9	

Pu=	1960	k	OK	OK
phiPn=	2767.982	k	Strength	Dimensions

K=	1		1
L=	360	in	360
a=	24	in	24
ri=	0.866025		4.618802
rib=	0.866025	in	4.618802
h=	11	in	11
α=	6.35		1.19
(KL/r)o=	64.66		77.94
(KL/r)m=	69.25		78.03
4.71√E/Fy=	113.43		113.43
Fe=	59.69		47.01
Fcr=	35.21293		32.03683

Dimensional Requirements

Ka/ri=	27.71281	OK
$3/4*kl/r_{gov}$=	48.49343	

Check elastic critical torsional buckling stress

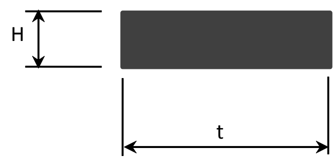
cw=	61952
J=	288
Fe=	669.2714

Check for slender flanges

b/t= 5.333333

Typical Web Tension Member

Truss #1
Design: DT1



Hmax=di=	8	in
H=	4.25	in
t=	18	in
phiy=	0.9	
phir=	0.75	
Ag=	76.5	in^2
A holes=	25.5	in^2
An=	51	in^2
Fy=	50	ksi
Fu=	65	ksi

Yield	phiPn=	3442.5	k
Rupture	phiPn=	2486.25	k

Pu=	2430	k	
phiPn=	2486	k	OK

plf= 260.3125
L= 28.3 ft

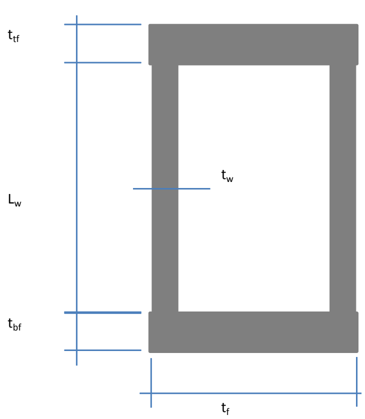
Typical Chord Built-Up Box Section

Pu =	4000	k
Mu =	1500	k-ft

E= 29000 ksi
Fy= 50 ksi
ϕ= 0.9

H1-1a =	0.60	OK

S.W.= 640 PLF



t_{tf}	2	in
t_{bf}	2	in
L_w	46	in
t_w	1	in
t_f	24	in

Flange	λ_p=	27.0 Compact
Web	λ_p=	58.3 Compact

A=	188	in2			
c=	25	in	c=	12	in
Ix=	71551	in4	Iy=	16783	in4
S=	2862	in3	S=	1399	in3
r=	19.5	in	r=	9.4	in
Z=	3362	in3	Z=	1634	in3

ϕMn= 14008 ft-k

K=	1	
L=	24	ft

KL/r= 30.5
Fe= 308.0
Fcr= 46.7

ϕPn= 7904 Kips

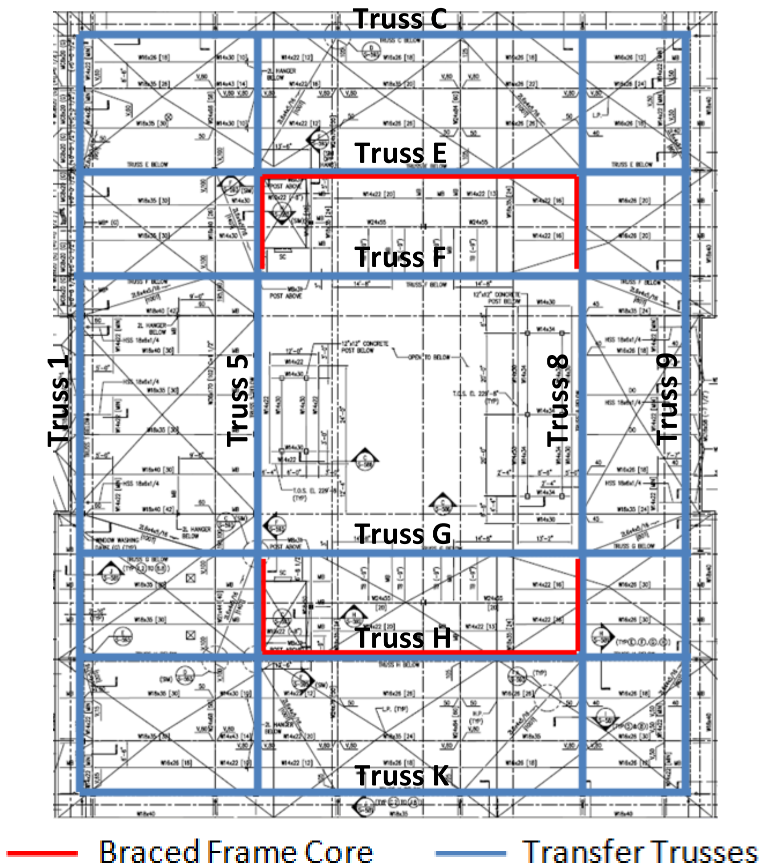
Combined Loading:

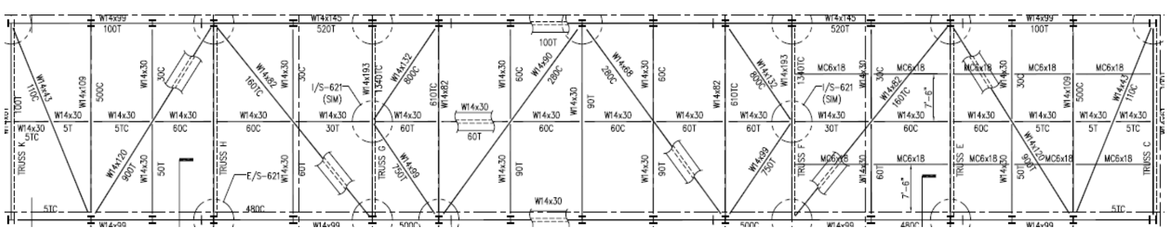
Pr/Pc= 0.51

H1-1a = 0.60 OK

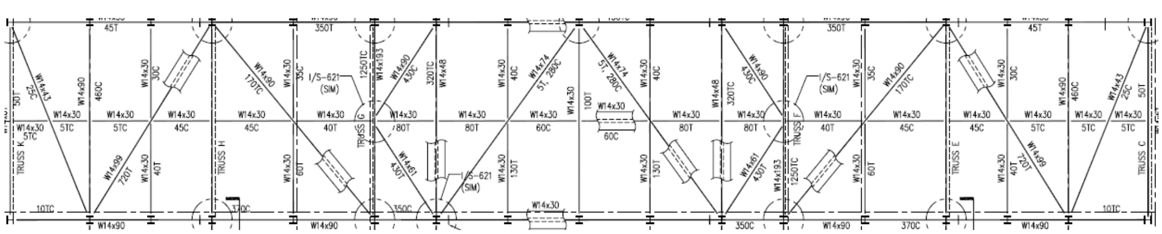
Appendix H – Existing Truss Designs

The following truss elevations were taken from the structural drawings.

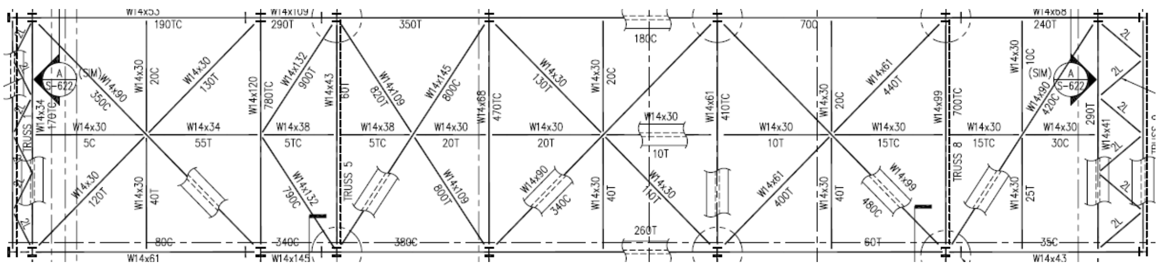




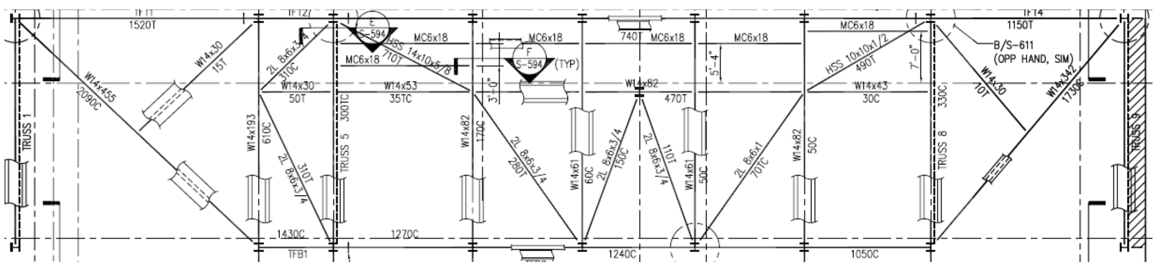
Truss 1 Elevation



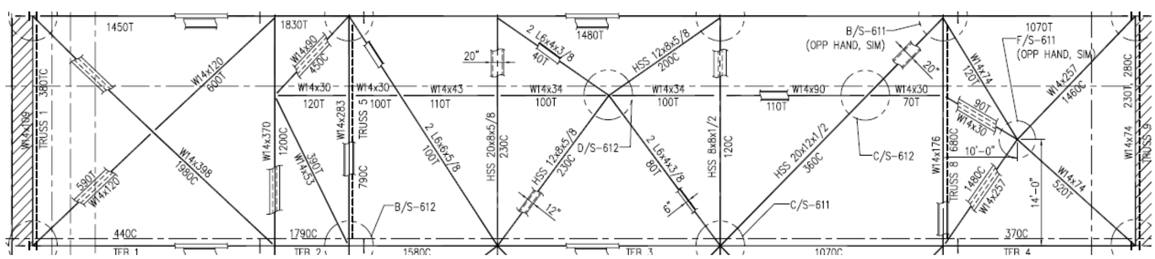
Truss 9 Elevation



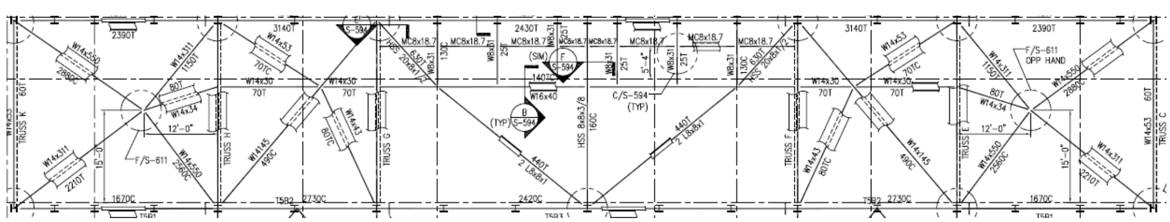
Truss C & K Elevation



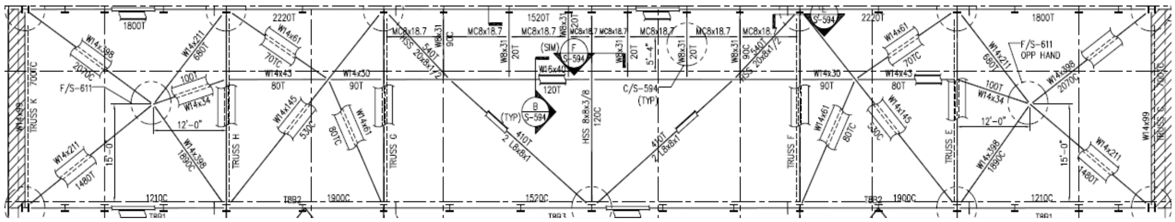
Truss F & G Elevation



Truss E & H Elevation



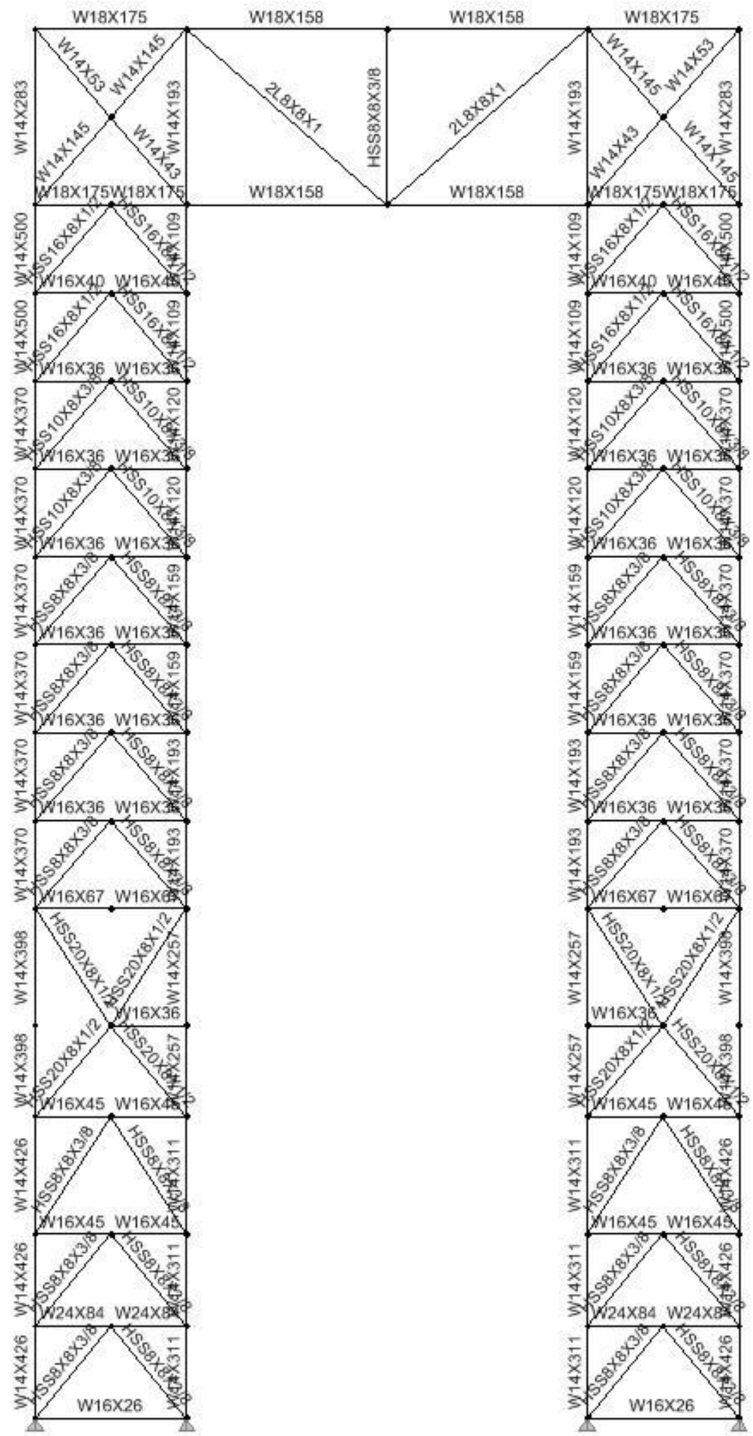
Truss 5 Elevation



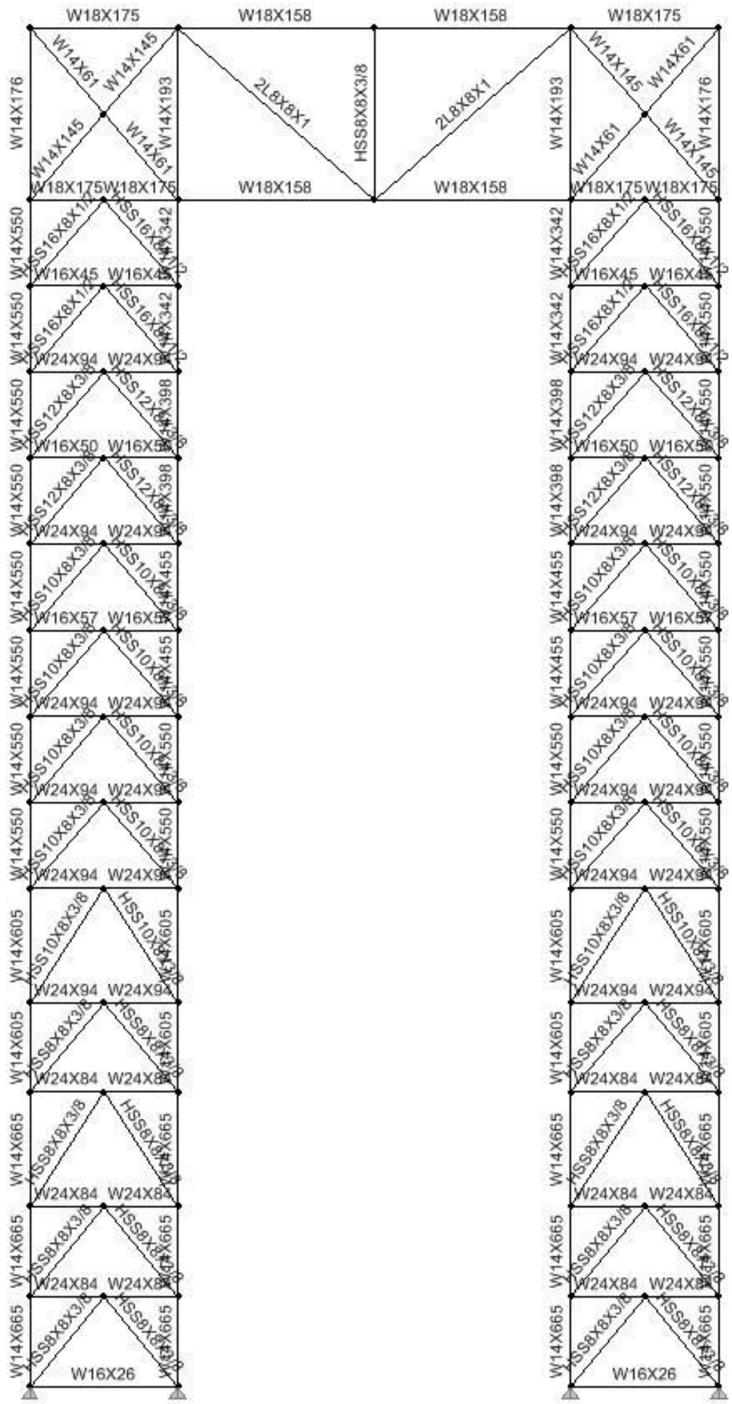
Truss 8 Elevation

Appendix I – Existing Lateral Systems (Tower Only)

Braced Frame 1 & 2



Braced Frame 3 Braced Frame 4



Braced Frame 5 Braced Frame 6

Appendix J – Construction Management

Steel Takeoff Information

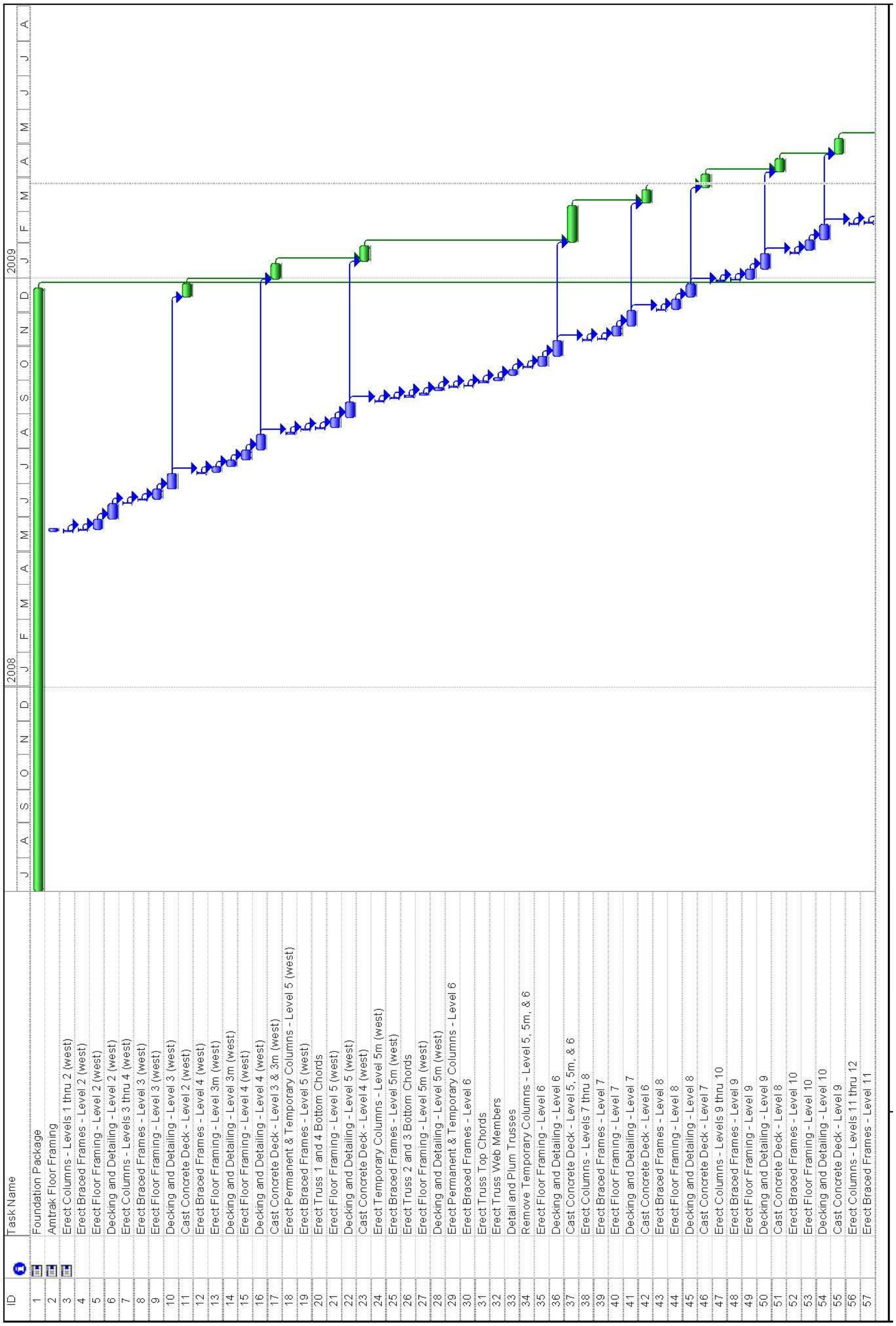
Thesis Transfer Truss Weights			
Truss	Quantity	Weight (kips/truss)	Total (kips)
1	1	333	333
2	2	263	525
3	2	225	450
4	1	72	72
		Total	1380

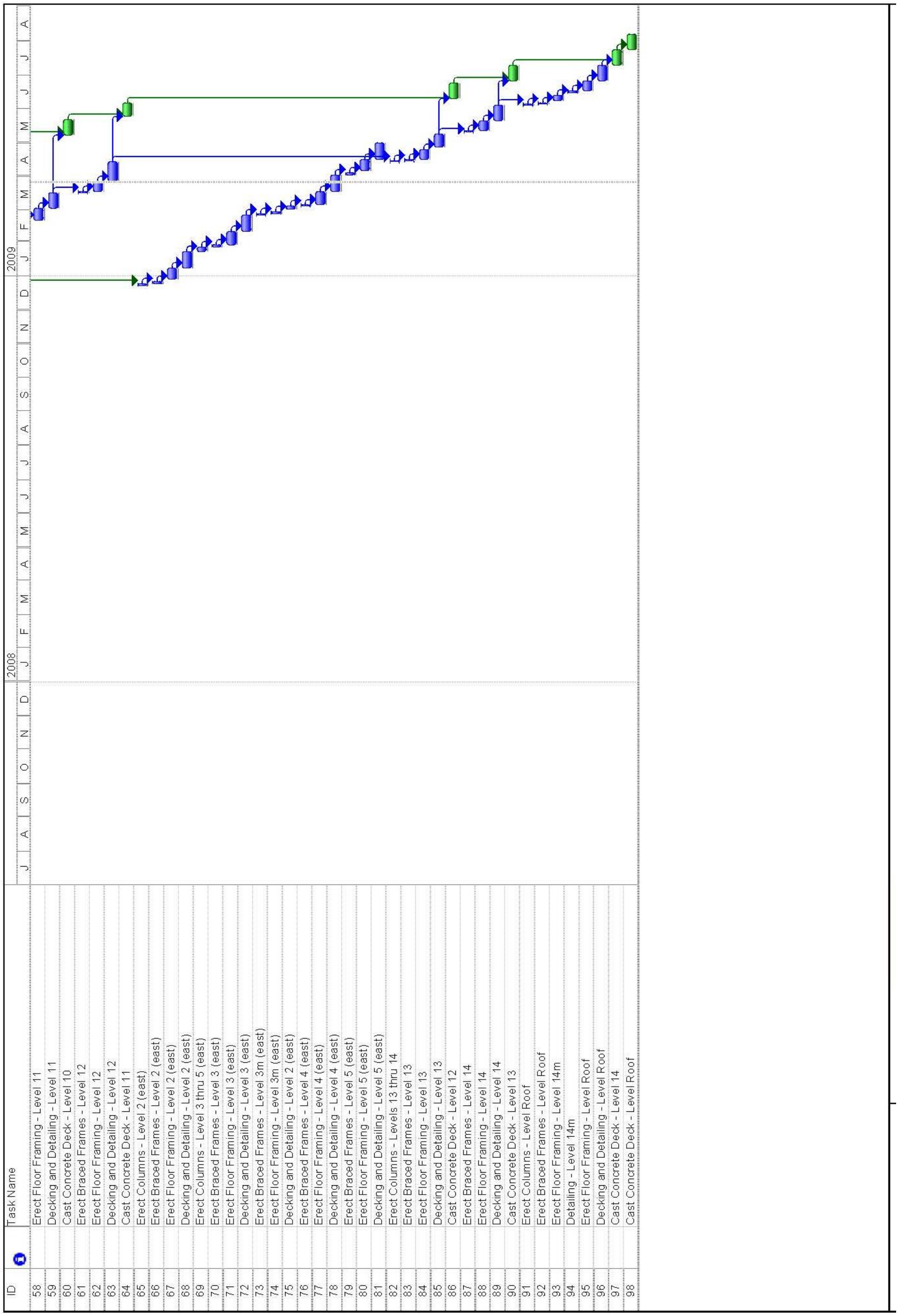
Existing Transfer Truss Weights			
Truss	Quantity	Weight (kips/truss)	Total (kips)
E & H	2	182	364
5	1	304	304
8	1	253	253
1	1	100	100
9	1	83	83
C & K	2	73	146
F & G	2	135	270
		Total	1521

Braced Frame Weights		
	Thesis (kips)	Existing (kips)
Columns	740	881
Bracing	296	209
Beams	288	214
TOTAL	1324	1304

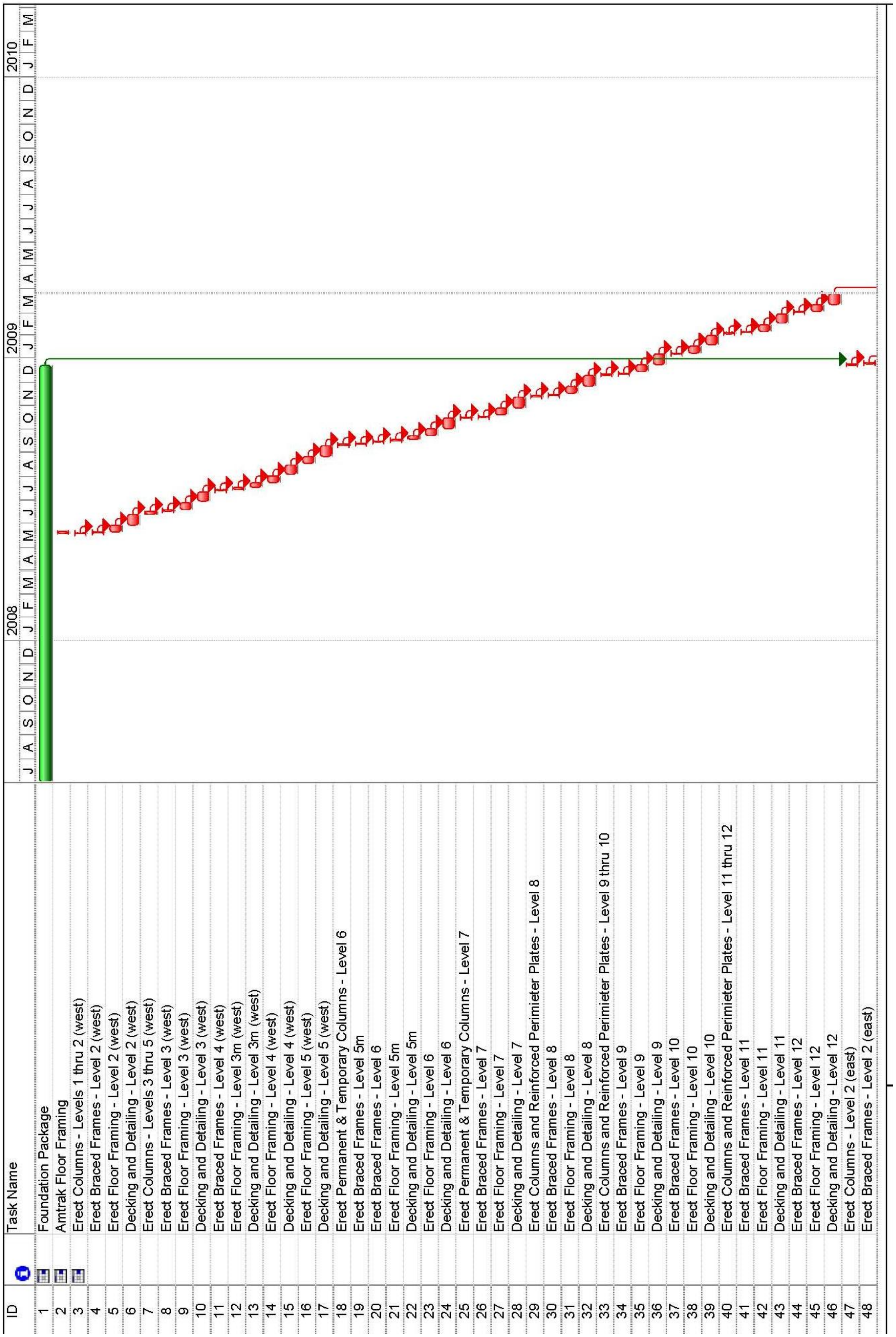
Transfer System Total Steel Weight Comparison		
System	Thesis (kips)	Existing (kips)
Trusses	1380	1521
Perimeter Columns/Plate Hangers	112	107
Braced Frame Core	1324	1304
Built-Up Box Sections	235	294
Total	3051	3226

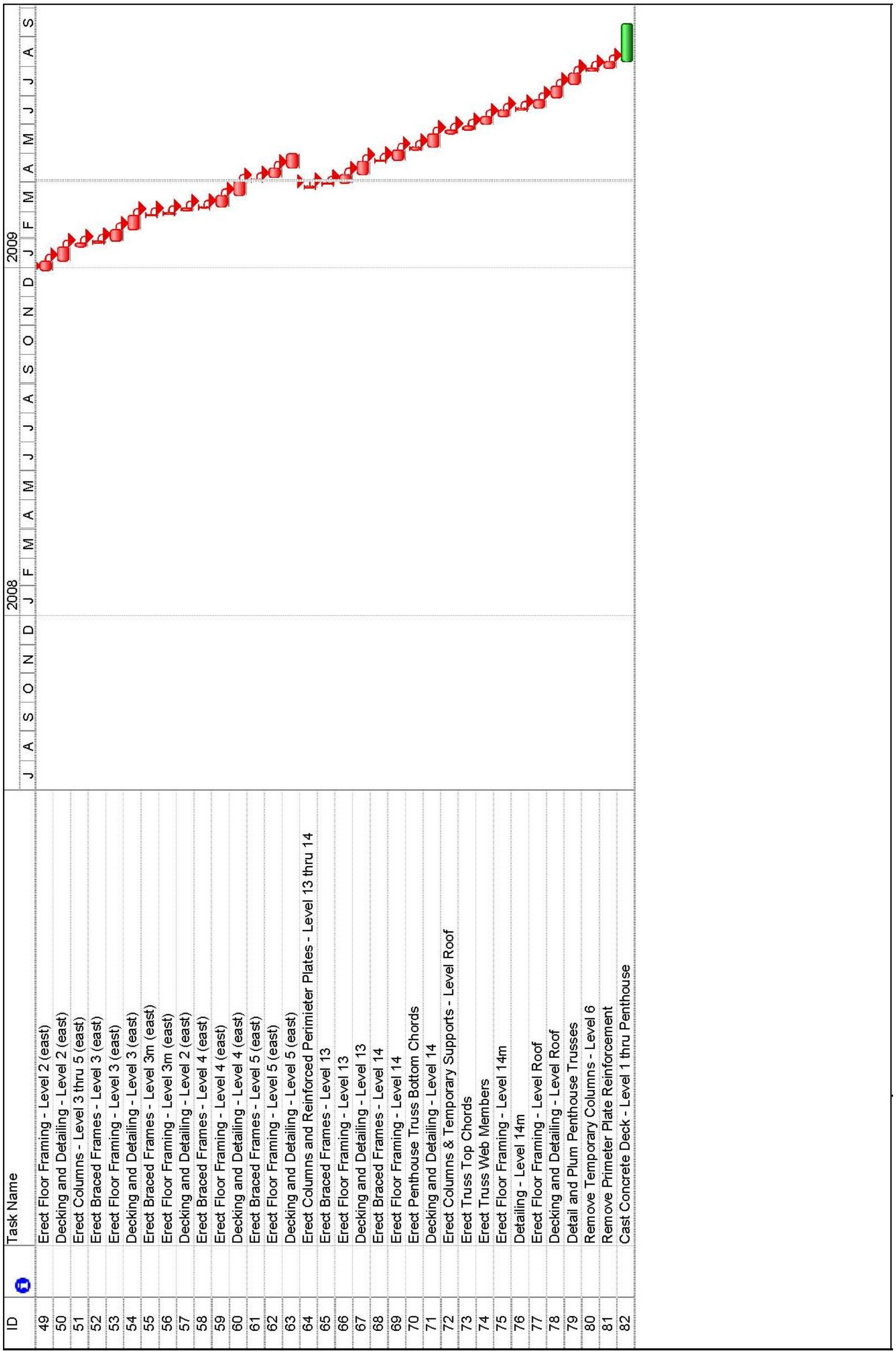
Thesis Transfer System Schedule


ID
Task Name
2008
2009
J A S O N D J F M A M J J A S O N D J F M A M J J A
1 Foundation Package
2 Amtrak Floor Framing
3 Erect Columns - Levels 1 thru 2 (west)
4 Erect Braced Frames - Level 2 (west)
5 Erect Floor Framing - Level 2 (west)
6 Decking and Detailing - Level 2 (west)
7 Erect Columns - Levels 3 thru 4 (west)
8 Erect Braced Frames - Level 3 (west)
9 Erect Floor Framing - Level 3 (west)
10 Decking and Detailing - Level 3 (west)
11 Cast Concrete Deck - Level 2 (west)
12 Erect Braced Frames - Level 4 (west)
13 Erect Floor Framing - Level 3m (west)
14 Decking and Detailing - Level 3m (west)
15 Erect Floor Framing - Level 4 (west)
16 Decking and Detailing - Level 4 (west)
17 Cast Concrete Deck - Level 3 & 3m (west)
18 Erect Permanent & Temporary Columns - Level 5 (west)
19 Erect Braced Frames - Level 5 (west)
20 Erect Truss 1 and 4 Bottom Chords
21 Erect Floor Framing - Level 5 (west)
22 Decking and Detailing - Level 5 (west)
23 Cast Concrete Deck - Level 4 (west)
24 Erect Temporary Columns - Level 5m (west)
25 Erect Braced Frames - Level 5m (west)
26 Erect Truss 2 and 3 Bottom Chords
27 Erect Floor Framing - Level 5m (west)
28 Decking and Detailing - Level 5m (west)
29 Erect Permanent & Temporary Columns - Level 6
30 Erect Braced Frames - Level 6
31 Erect Truss Top Chords
32 Erect Truss Web Members
33 Detail and Plum Trusses
34 Remove Temporary Columns - Level 5, 5m, & 6
35 Erect Floor Framing - Level 6
36 Decking and Detailing - Level 6
37 Cast Concrete Deck - Level 5, 5m, & 6
38 Erect Columns - Levels 7 thru 8
39 Erect Braced Frames - Level 7
40 Erect Floor Framing - Level 7
41 Decking and Detailing - Level 7
42 Cast Concrete Deck - Level 6
43 Erect Braced Frames - Level 8
44 Erect Floor Framing - Level 8
45 Decking and Detailing - Level 8
46 Cast Concrete Deck - Level 7
47 Erect Columns - Levels 9 thru 10
48 Erect Braced Frames - Level 9
49 Erect Floor Framing - Level 9
50 Decking and Detailing - Level 9
51 Cast Concrete Deck - Level 8
52 Erect Braced Frames - Level 10
53 Erect Floor Framing - Level 10
54 Decking and Detailing - Level 10
55 Cast Concrete Deck - Level 9
56 Erect Columns - Levels 11 thru 12
57 Erect Braced Frames - Level 11


ID
Task Name
2008
2009
J A S O N D J F M A M J J A S O N D J F M A M J J A
58 Erect Floor Framing - Level 11
59 Decking and Detailing - Level 11
60 Cast Concrete Deck - Level 10
61 Erect Braced Frames - Level 12
62 Erect Floor Framing - Level 12
63 Decking and Detailing - Level 12
64 Cast Concrete Deck - Level 11
65 Erect Columns - Level 2 (east)
66 Erect Braced Frames - Level 2 (east)
67 Erect Floor Framing - Level 2 (east)
68 Decking and Detailing - Level 2 (east)
69 Erect Columns - Level 3 thru 5 (east)
70 Erect Braced Frames - Level 3 (east)
71 Erect Floor Framing - Level 3 (east)
72 Decking and Detailing - Level 3 (east)
73 Erect Braced Frames - Level 3m (east)
74 Erect Floor Framing - Level 3m (east)
75 Decking and Detailing - Level 2 (east)
76 Erect Braced Frames - Level 4 (east)
77 Erect Floor Framing - Level 4 (east)
78 Decking and Detailing - Level 4 (east)
79 Erect Braced Frames - Level 5 (east)
80 Erect Floor Framing - Level 5 (east)
81 Decking and Detailing - Level 5 (east)
82 Erect Columns - Levels 13 thru 14
83 Erect Braced Frames - Level 13
84 Erect Floor Framing - Level 13
85 Decking and Detailing - Level 13
86 Cast Concrete Deck - Level 12
87 Erect Braced Frames - Level 14
88 Erect Floor Framing - Level 14
89 Decking and Detailing - Level 14
90 Cast Concrete Deck - Level 13
91 Erect Columns - Level Roof
92 Erect Braced Frames - Level Roof
93 Erect Floor Framing - Level 14m
94 Detailing - Level 14m
95 Erect Floor Framing - Level Roof
96 Decking and Detailing - Level Roof
97 Cast Concrete Deck - Level 14
98 Cast Concrete Deck - Level Roof

Existing Transfer System Schedule


ID
Task Name
1 Foundation Package
2 Amtrak Floor Framing
3 Erect Columns - Levels 1 thru 2 (west)
4 Erect Braced Frames - Level 2 (west)
5 Erect Floor Framing - Level 2 (west)
6 Decking and Detailing - Level 2 (west)
7 Erect Columns - Levels 3 thru 5 (west)
8 Erect Braced Frames - Level 3 (west)
9 Erect Floor Framing - Level 3 (west)
10 Decking and Detailing - Level 3 (west)
11 Erect Braced Frames - Level 4 (west)
12 Erect Floor Framing - Level 3m (west)
13 Decking and Detailing - Level 3m (west)
14 Erect Floor Framing - Level 4 (west)
15 Decking and Detailing - Level 4 (west)
16 Erect Floor Framing - Level 5 (west)
17 Decking and Detailing - Level 5 (west)
18 Erect Permanent & Temporary Columns - Level 6
19 Erect Braced Frames - Level 5m
20 Erect Braced Frames - Level 6
21 Erect Floor Framing - Level 5m
22 Decking and Detailing - Level 5m
23 Erect Floor Framing - Level 6
24 Decking and Detailing - Level 6
25 Erect Permanent & Temporary Columns - Level 7
26 Erect Braced Frames - Level 7
27 Erect Floor Framing - Level 7
28 Decking and Detailing - Level 7
29 Erect Columns and Reinforced Perimieter Plates - Level 8
30 Erect Braced Frames - Level 8
31 Erect Floor Framing - Level 8
32 Decking and Detailing - Level 8
33 Erect Columns and Reinforced Perimieter Plates - Level 9 thru 10
34 Erect Braced Frames - Level 9
35 Erect Floor Framing - Level 9
36 Decking and Detailing - Level 9
37 Erect Braced Frames - Level 10
38 Erect Floor Framing - Level 10
39 Decking and Detailing - Level 10
40 Erect Columns and Reinforced Perimieter Plates - Level 11 thru 12
41 Erect Braced Frames - Level 11
42 Erect Floor Framing - Level 11
43 Decking and Detailing - Level 11
44 Erect Braced Frames - Level 12
45 Erect Floor Framing - Level 12
46 Decking and Detailing - Level 12
47 Erect Columns - Level 2 (east)
48 Erect Braced Frames - Level 2 (east)
2008
2009
2010
J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M


ID
Task Name
49 Erect Floor Framing - Level 2 (east)
50 Decking and Detailing - Level 2 (east)
51 Erect Columns - Level 3 thru 5 (east)
52 Erect Braced Frames - Level 3 (east)
53 Erect Floor Framing - Level 3 (east)
54 Decking and Detailing - Level 3 (east)
55 Erect Braced Frames - Level 3m (east)
56 Erect Floor Framing - Level 3m (east)
57 Decking and Detailing - Level 2 (east)
58 Erect Braced Frames - Level 4 (east)
59 Erect Floor Framing - Level 4 (east)
60 Decking and Detailing - Level 4 (east)
61 Erect Braced Frames - Level 5 (east)
62 Erect Floor Framing - Level 5 (east)
63 Decking and Detailing - Level 5 (east)
64 Erect Columns and Reinforced Perimieter Plates - Level 13 thru 14
65 Erect Braced Frames - Level 13
66 Erect Floor Framing - Level 13
67 Decking and Detailing - Level 13
68 Erect Braced Frames - Level 14
69 Erect Floor Framing - Level 14
70 Erect Penthouse Truss Bottom Chords
71 Decking and Detailing - Level 14
72 Erect Columns & Temporary Supports - Level Roof
73 Erect Truss Top Chords
74 Erect Truss Web Members
75 Erect Floor Framing - Level 14m
76 Detailing - Level 14m
77 Erect Floor Framing - Level Roof
78 Decking and Detailing - Level Roof
79 Detail and Plum Penthouse Trusses
80 Remove Temporary Columns - Level 6
81 Remove Primeter Plate Reinforcement
82 Cast Concrete Deck - Level 1 thru Penthouse
2008
2009
J A S O N D J F M A M J J A S O N D J F M A M J J A S