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 13th 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.

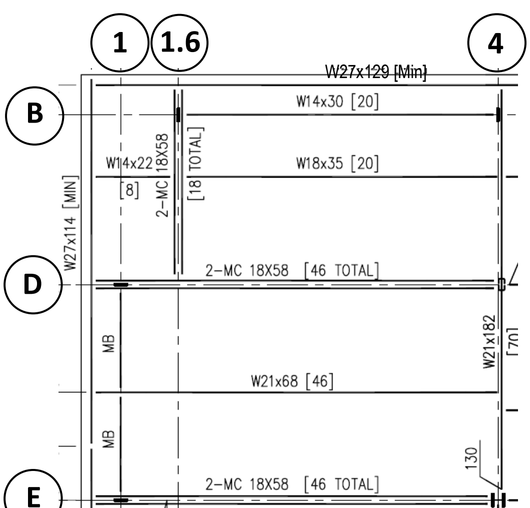


Figure 12a – Existing Design Corner Framing Plans

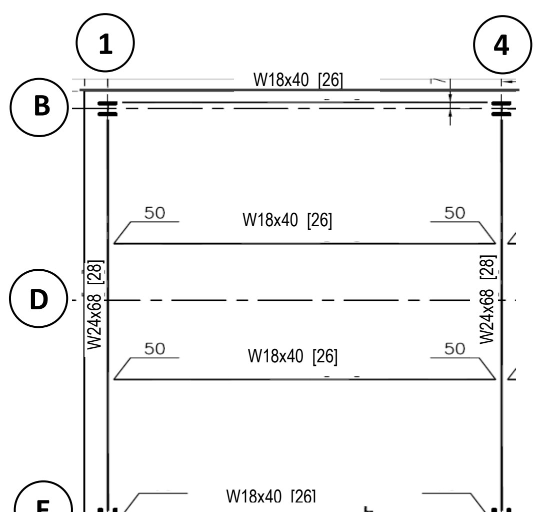


Figure 12b – New Design Corner Framing Plans

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.

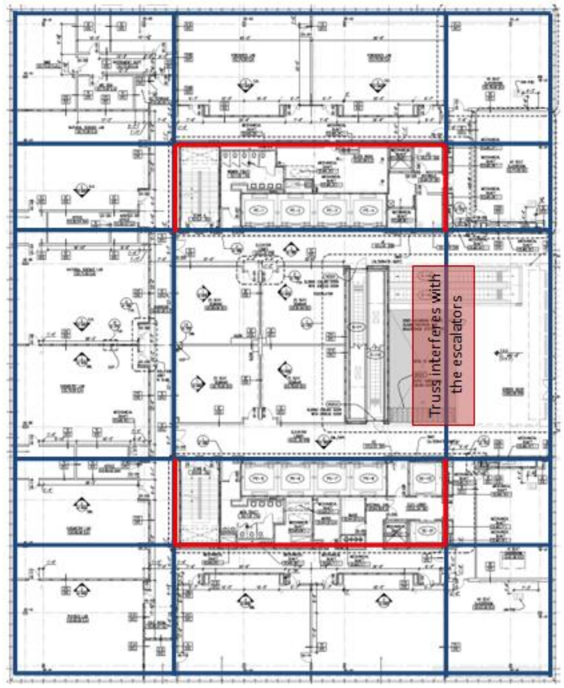


Figure 13 - Truss Configuration 1

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

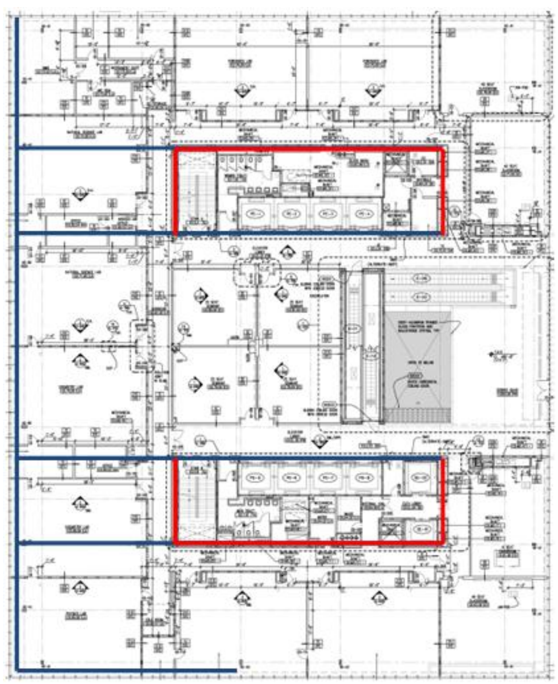


Figure 14 - Truss Configuration 2

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.

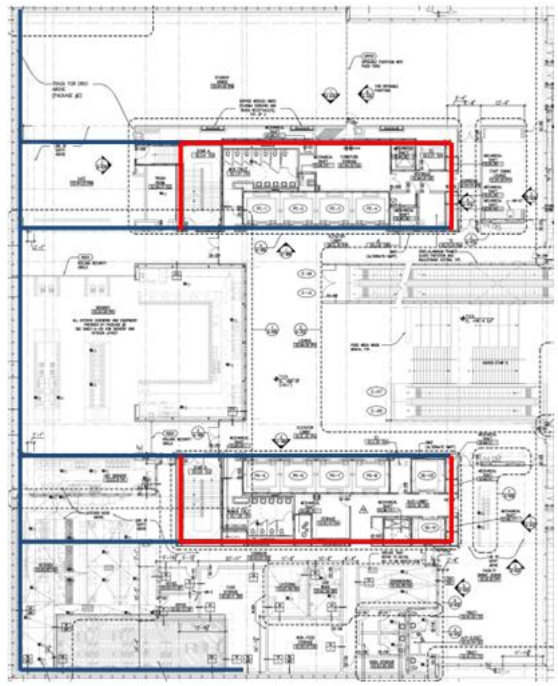


Figure 15 - Truss Configuration 3

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.

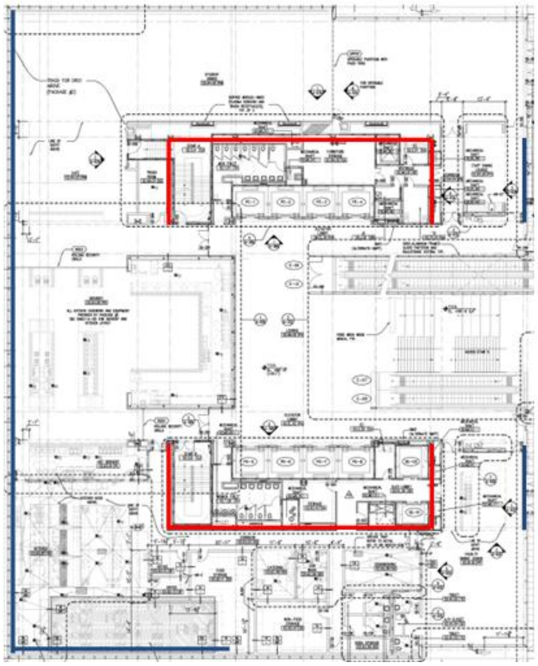


Figure 16 - Truss Configuration 4
5^{th} Level

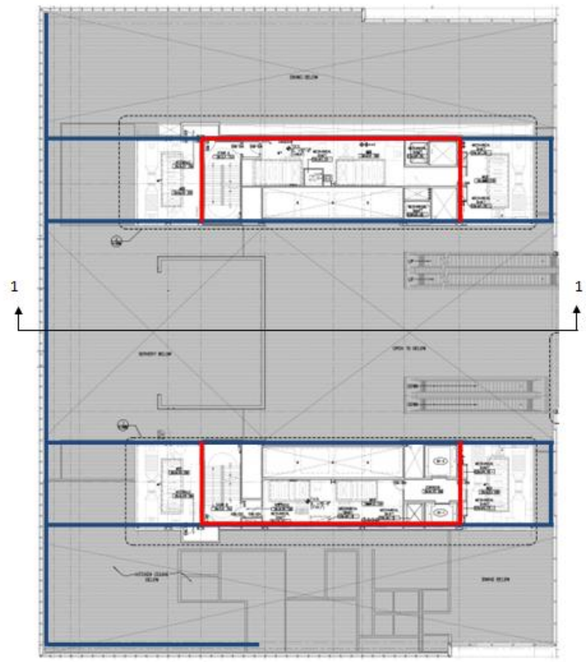
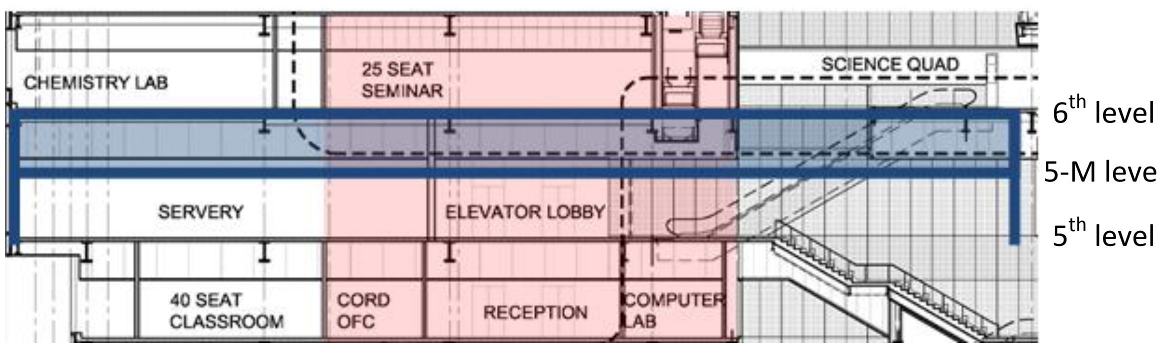


Figure 17 – Truss Configuration 4
5^{th} Level Mezzanine



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 5^{th} 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.

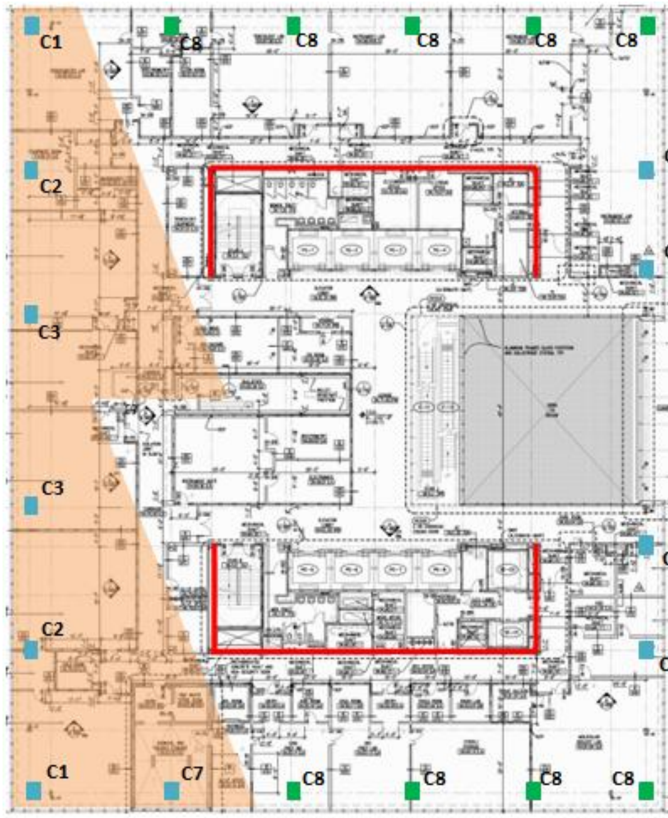


Figure 18 – Location of New Columns (Amtrak Tracks Shown in Orange)

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

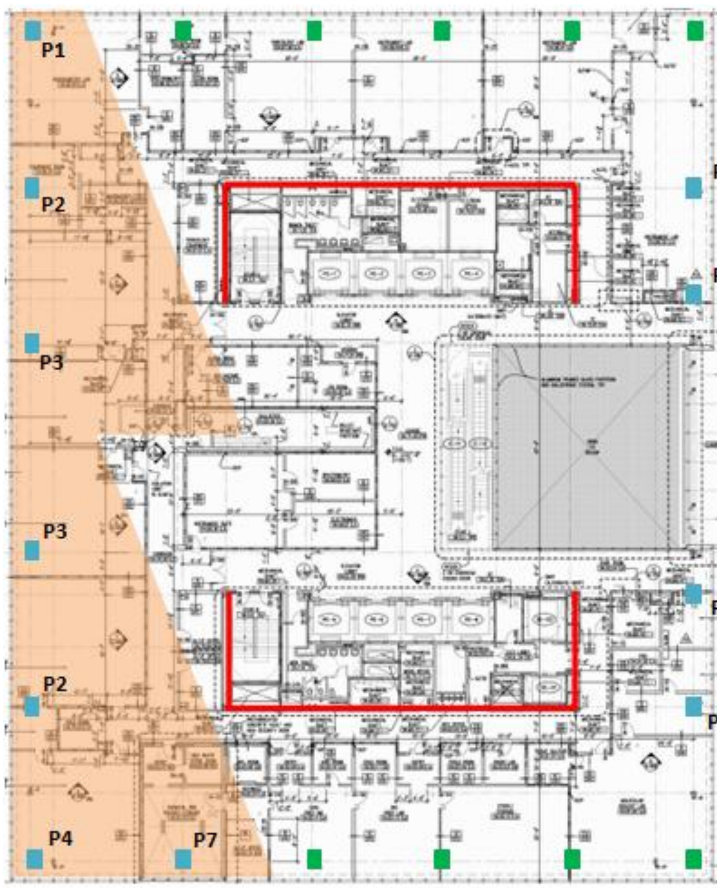


Figure 19 – Gravity Load Takedowns

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.

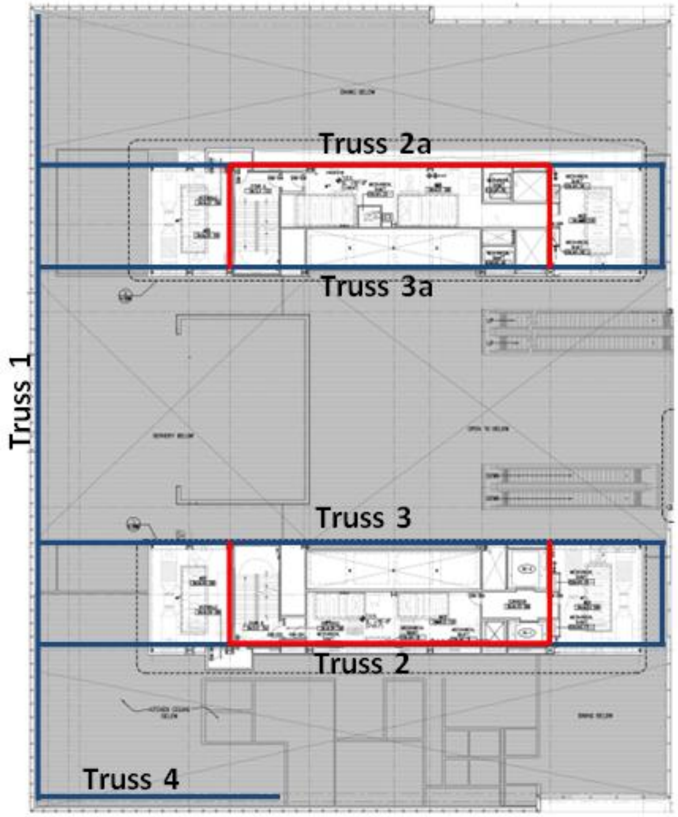


Figure 20 – Truss Nomenclature (Note: Truss 2a and Truss 3a designs are identical to Truss 2 and Truss 3, but have different cambers to control deflections.)

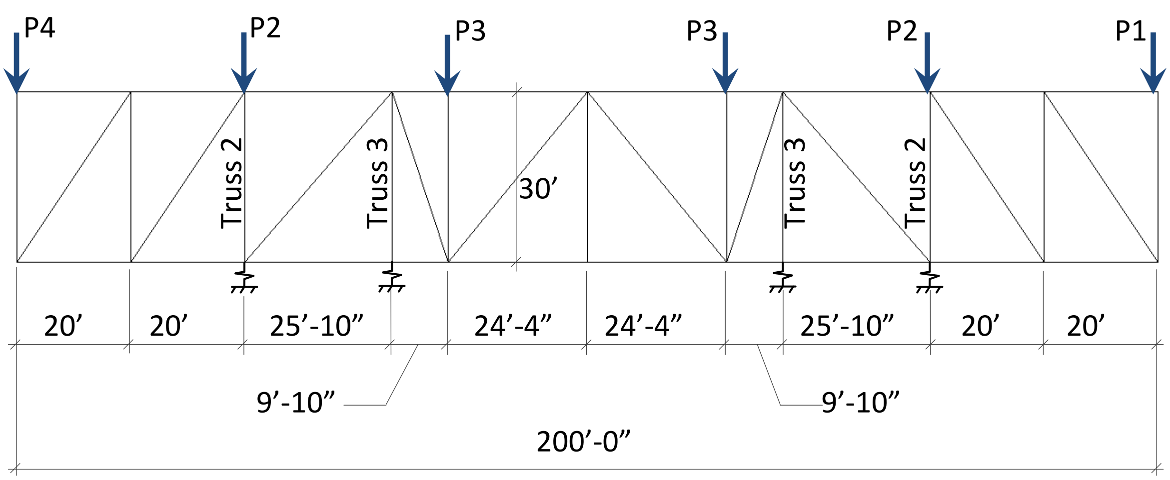


Figure 21 – Truss 1 Elevation

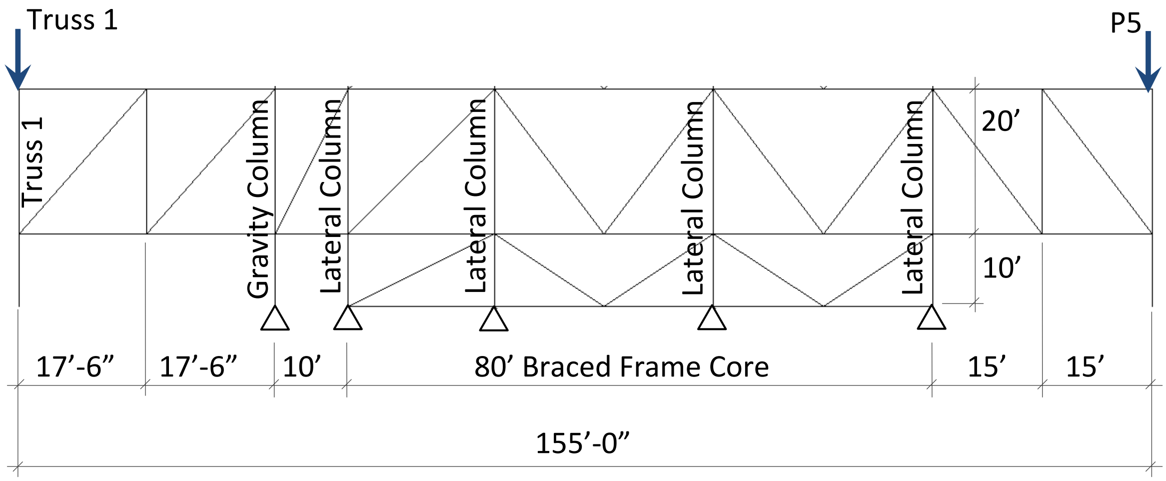


Figure 22 – Truss 2 Elevation

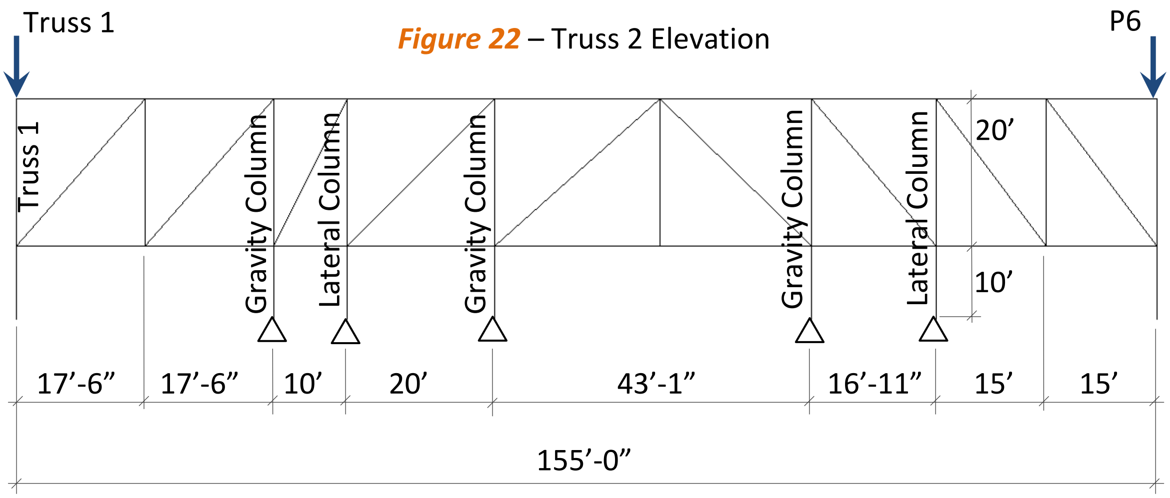


Figure 23 – Truss 3 Elevation

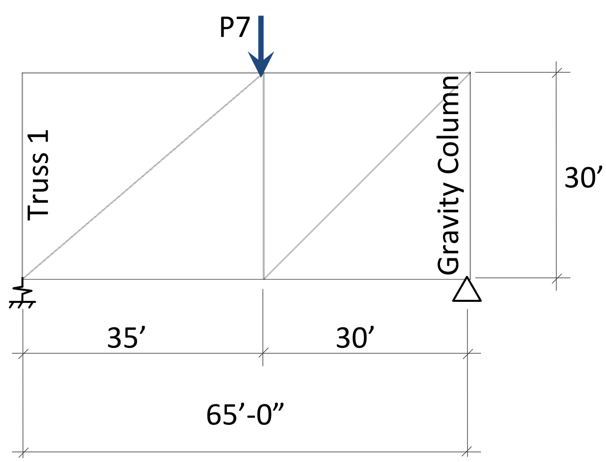
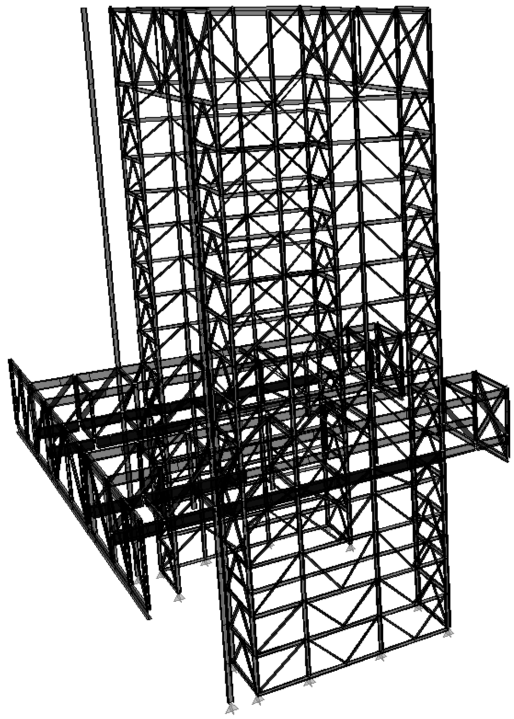


Figure 24 – Truss 4 Elevation

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.

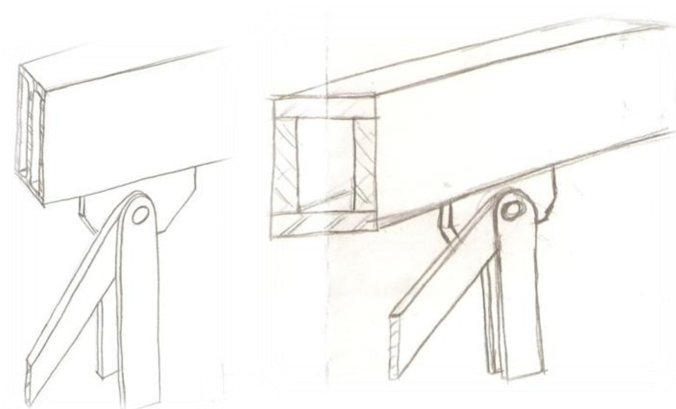


Figure 25 – Sketches of Desired Truss Connections

Truss 1 Design

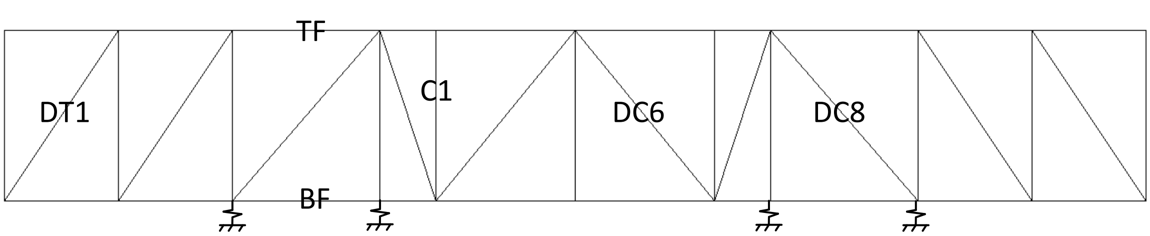


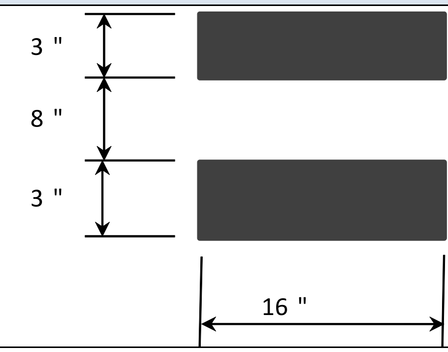
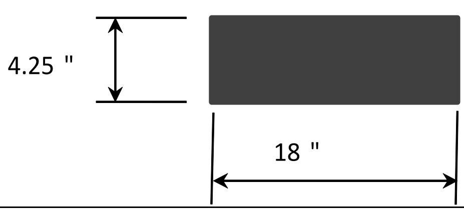
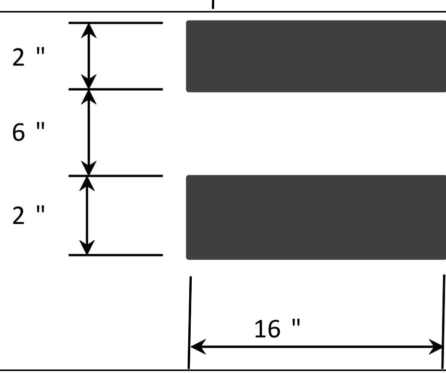
Figure 26 – Truss 1 Design Members

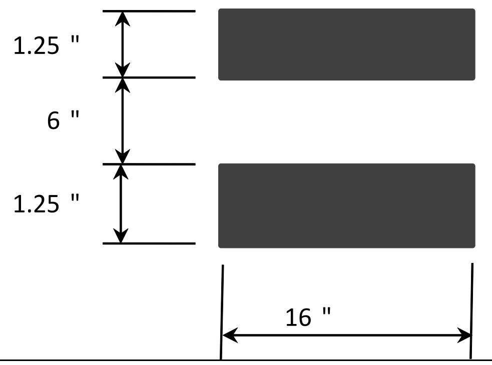
Table 10 – Truss 1 Design Forces

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

<table>
<tr><td>DC8</td><td>

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

Truss 2 Design

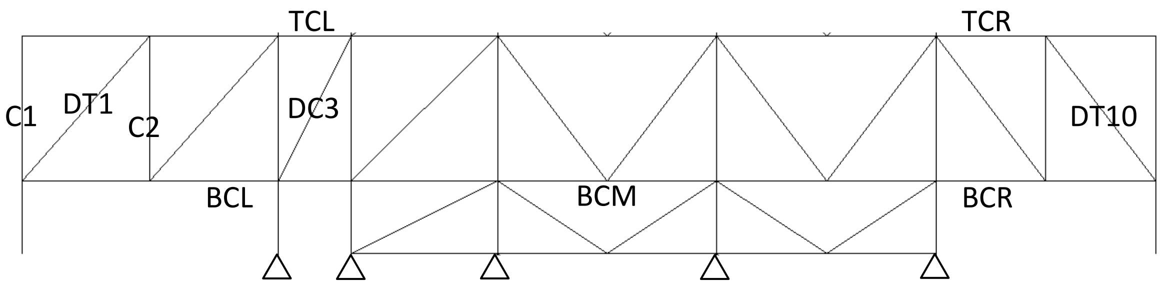


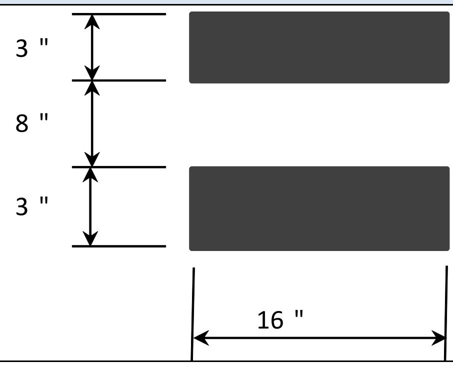
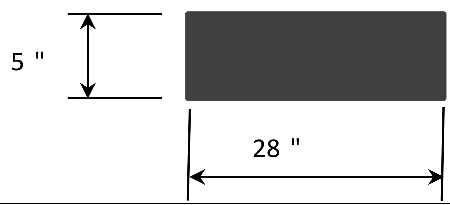
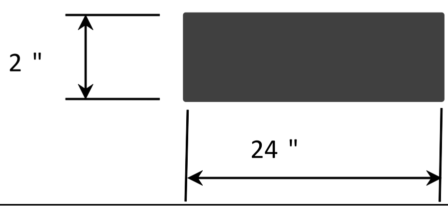
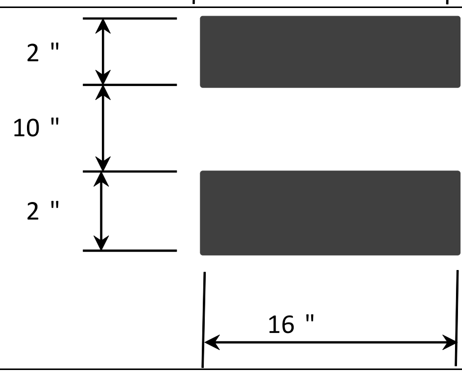
Figure 27 – Truss 2 Design Members

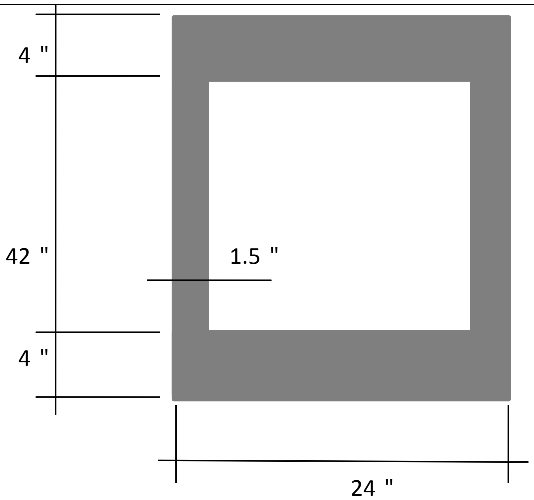
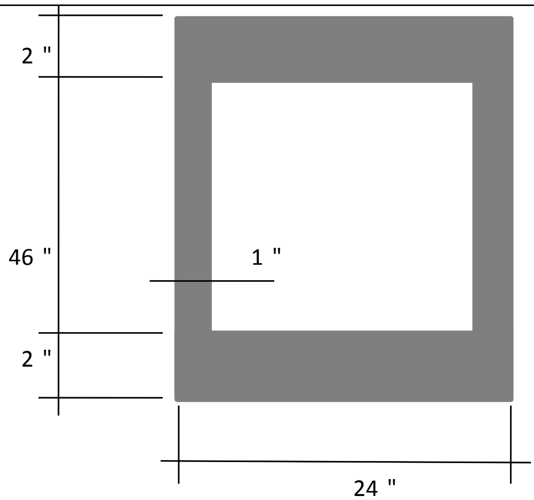
Table 12 – Truss 2 Design Forces

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

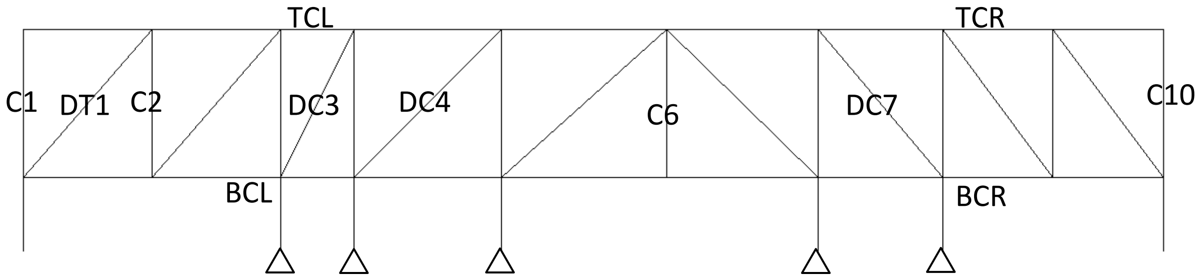


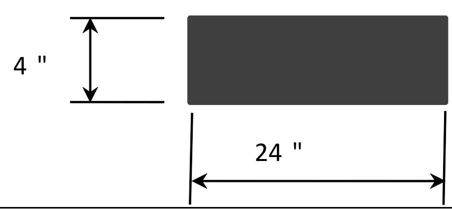
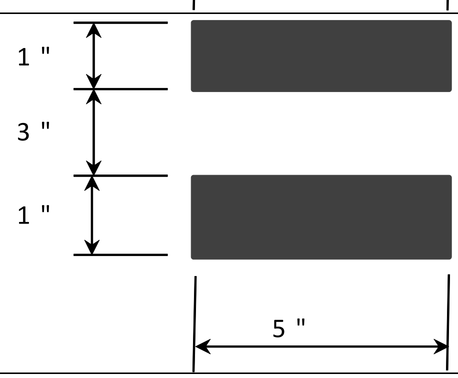
Figure 28 – Truss 3 Design Members

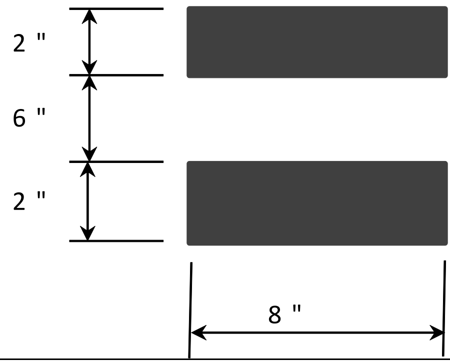
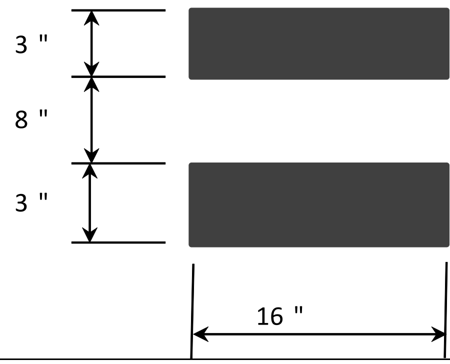
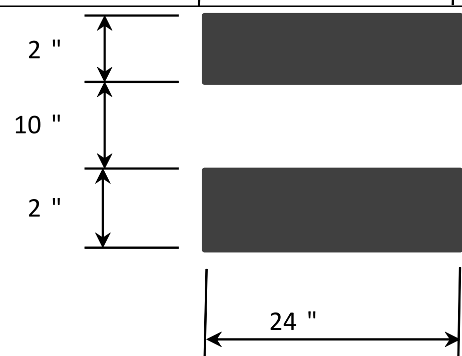
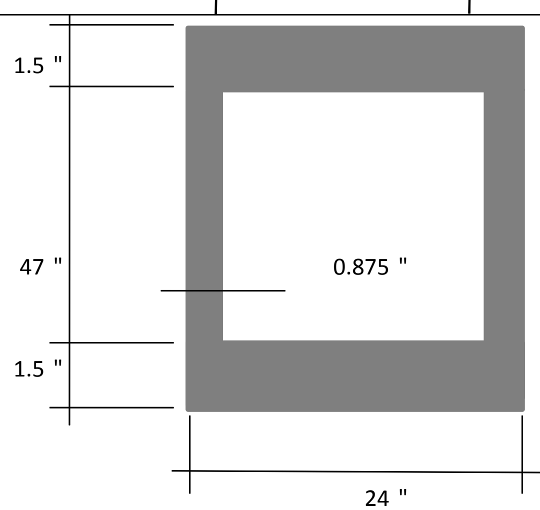
Table 14 – Truss 3 Design Forces

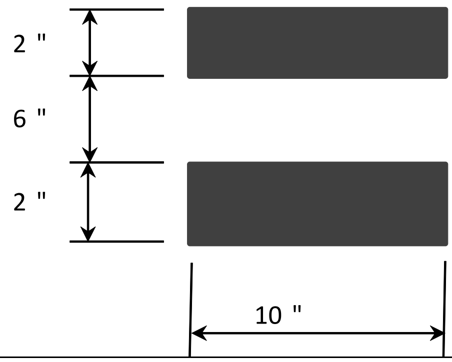
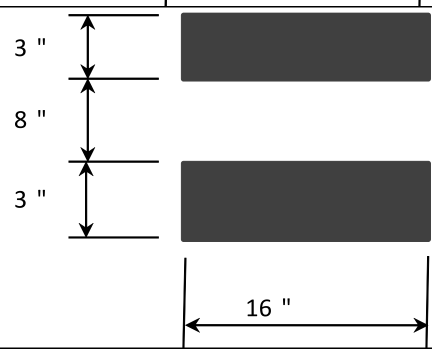
Member	Pu (k)	Mu (ft-k)
DT1	2700 (T)	0
DC7	30 (C)	0
C6	200 (C)	0
C10	2000 (C)	0
DC3	500 (C)	0
DC4	516 (C)	0
BCL	3300 (T)	5000
BCR	2500 (T)	2500
TCL	3200 (C)	1100
TCR	1300 (C)	2900
C2	1886 (C)	0
C1	1665 (C)	205

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

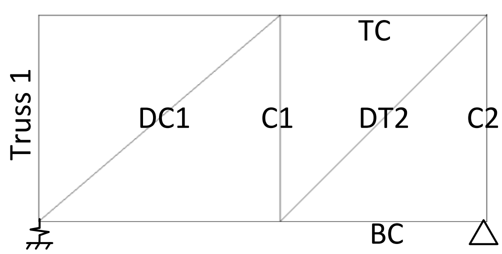


Figure 29 – Truss 4 Design Members

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

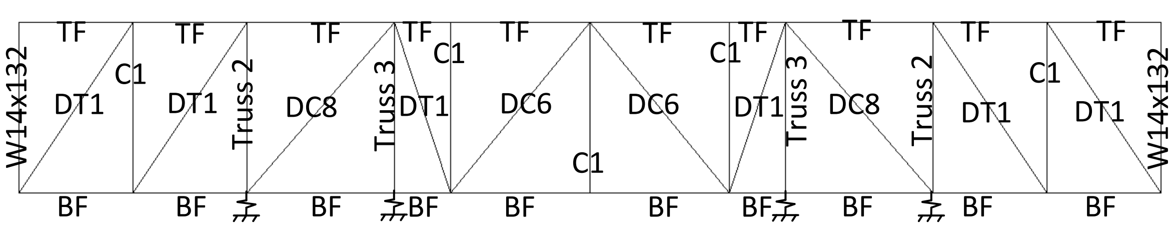


Figure 30 – Final Member Sizes for Truss 1

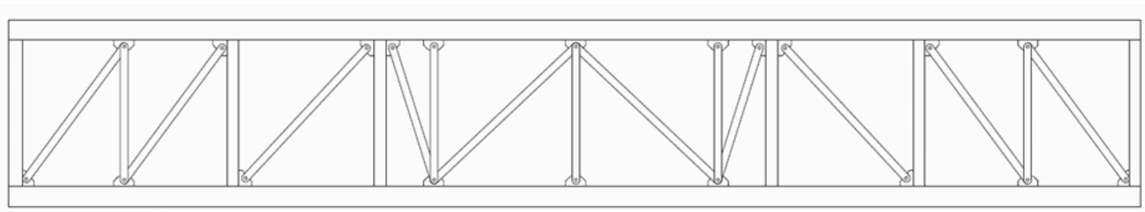


Figure 31 – Elevation View of Truss 1

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

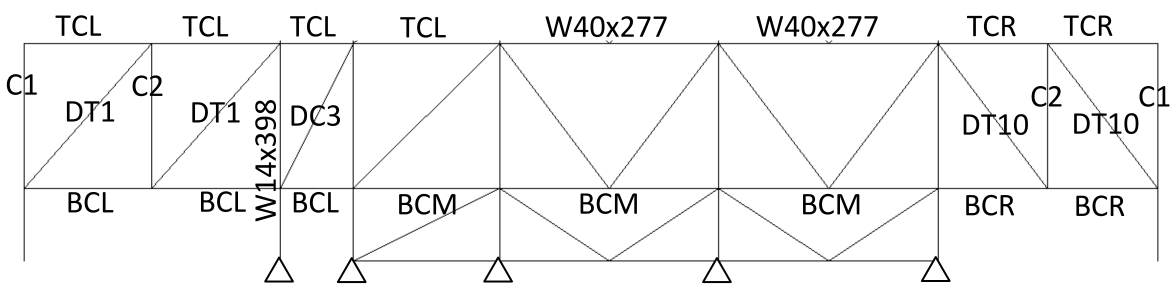


Figure 32 – Final Member Sizes for Truss 2

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

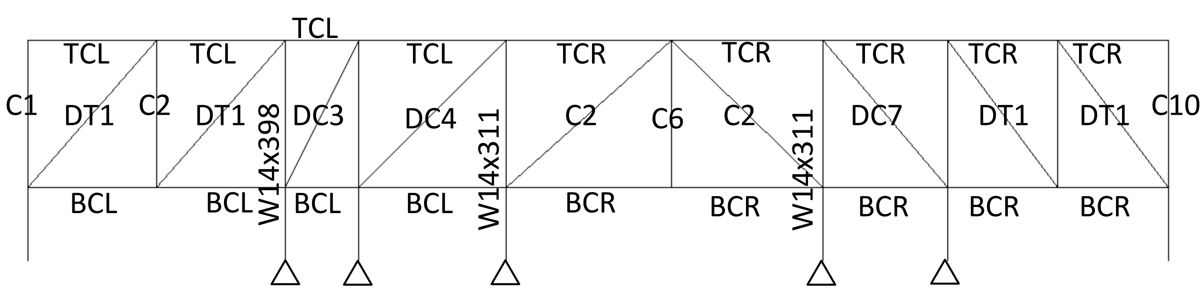


Figure 33 – Final Member Sizes for Truss 3

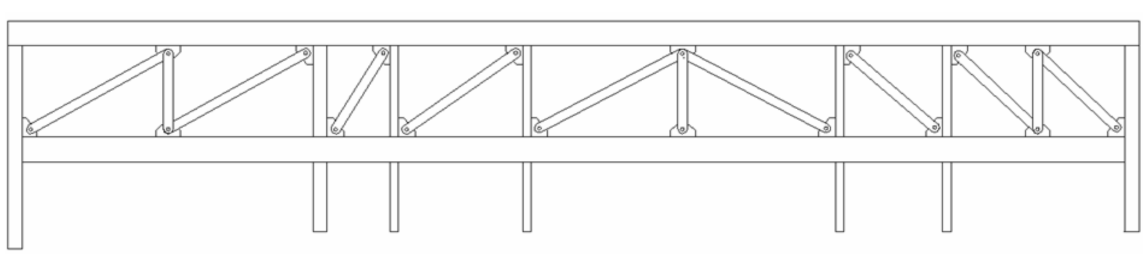


Figure 34 – Elevation View of Truss 3

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

Truss 4

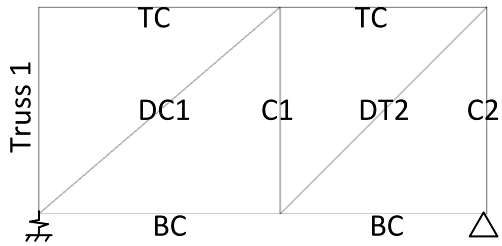


Figure 35 – Final Member Sizes for Truss 4

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.

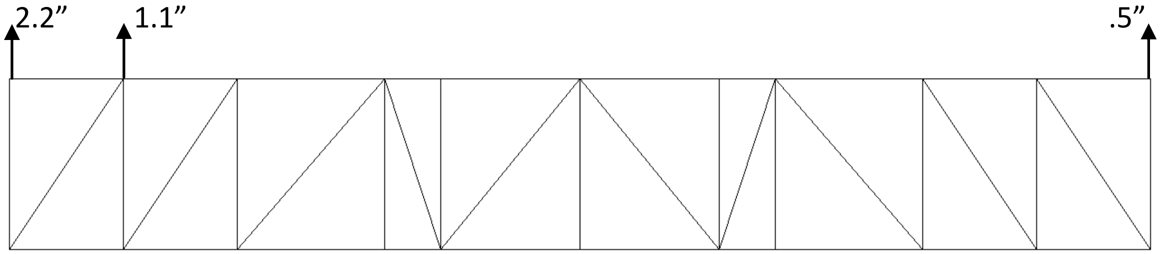


Figure 36 – Truss 1 Camber

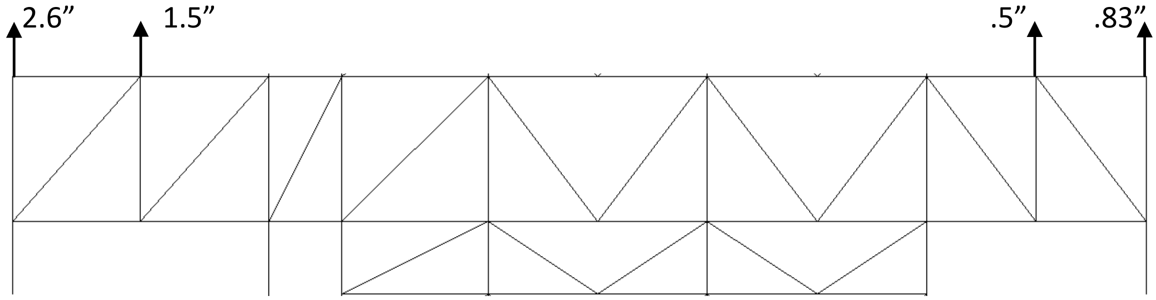


Figure 37 – Truss 2 Cambers

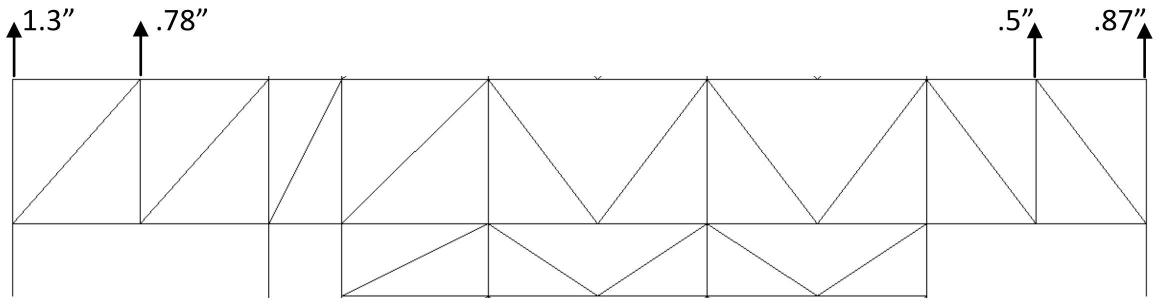


Figure 38 – Truss 2a Cambers

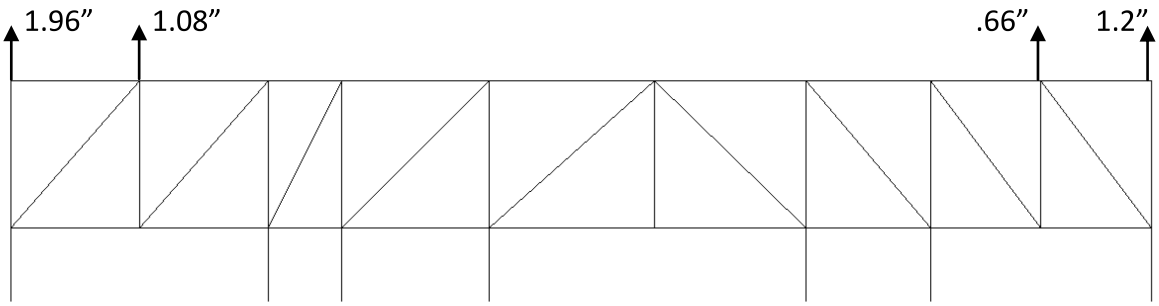


Figure 39 – Truss 3 Cambers

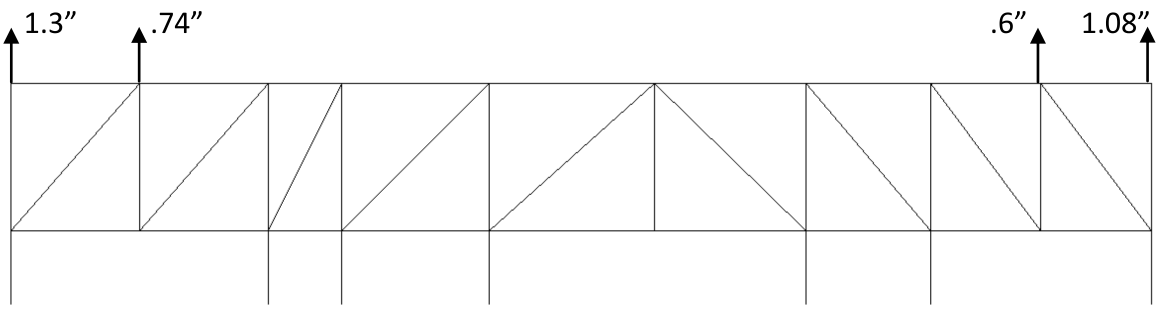


Figure 40 – Truss 3a Cambers

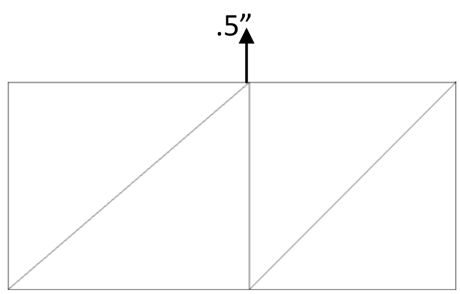


Figure 41 – Truss 4 Cambers

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 5^{th} 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.

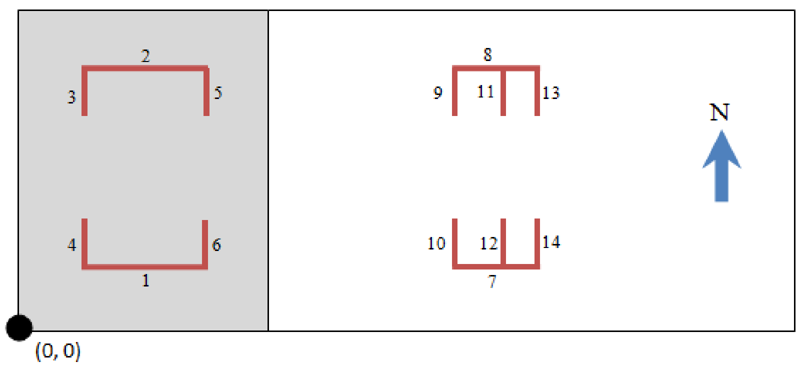


Figure 42 – Labeled Braced Frames

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)
	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
Windward	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 E-W	Shear (kips) N-S	Shear E-W	Moment (ft-kips) N-S	Moment 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.

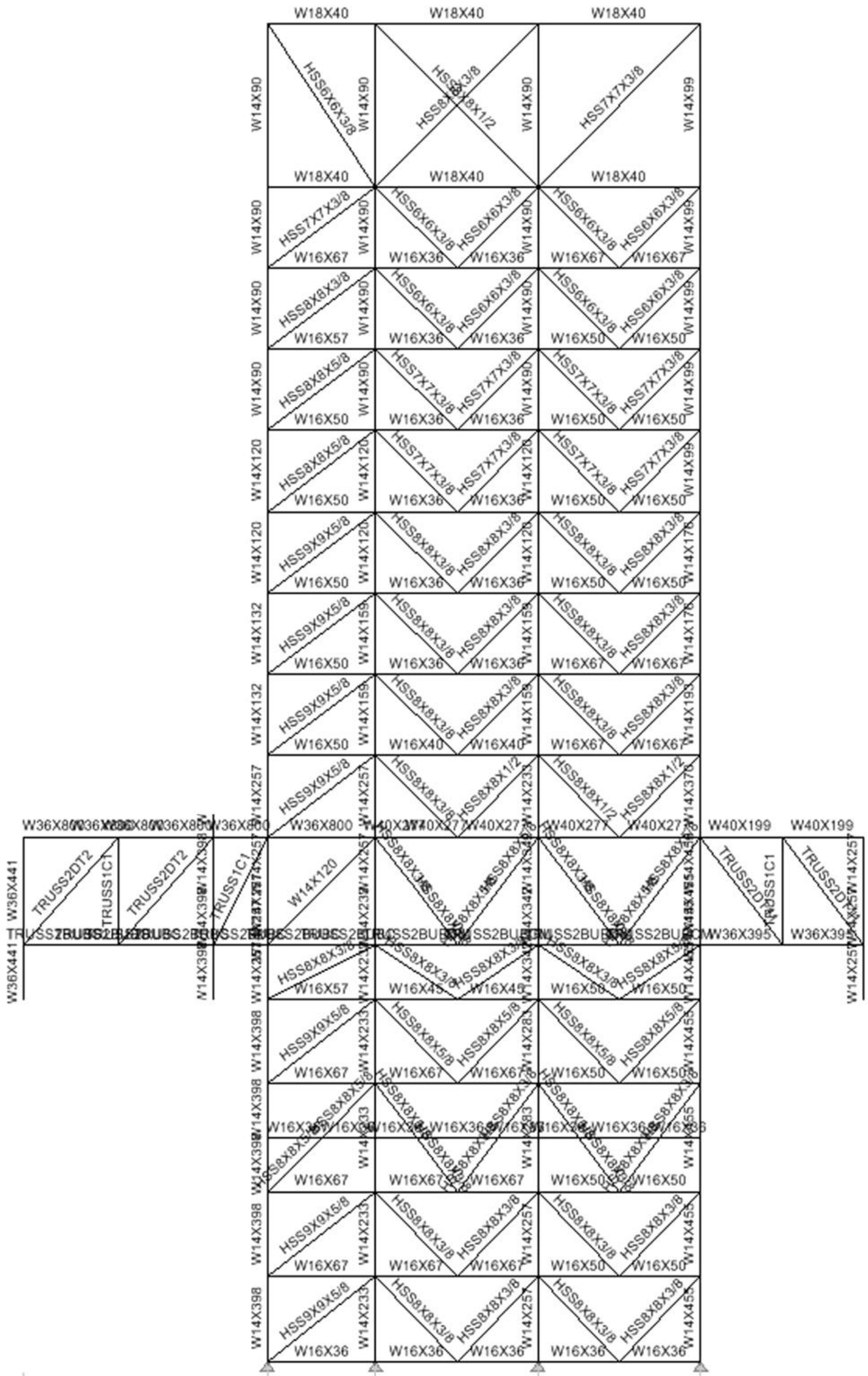


Figure 43 – Braced Frame 1 and 2 Design (for truss design members see Truss Design Sections)

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.

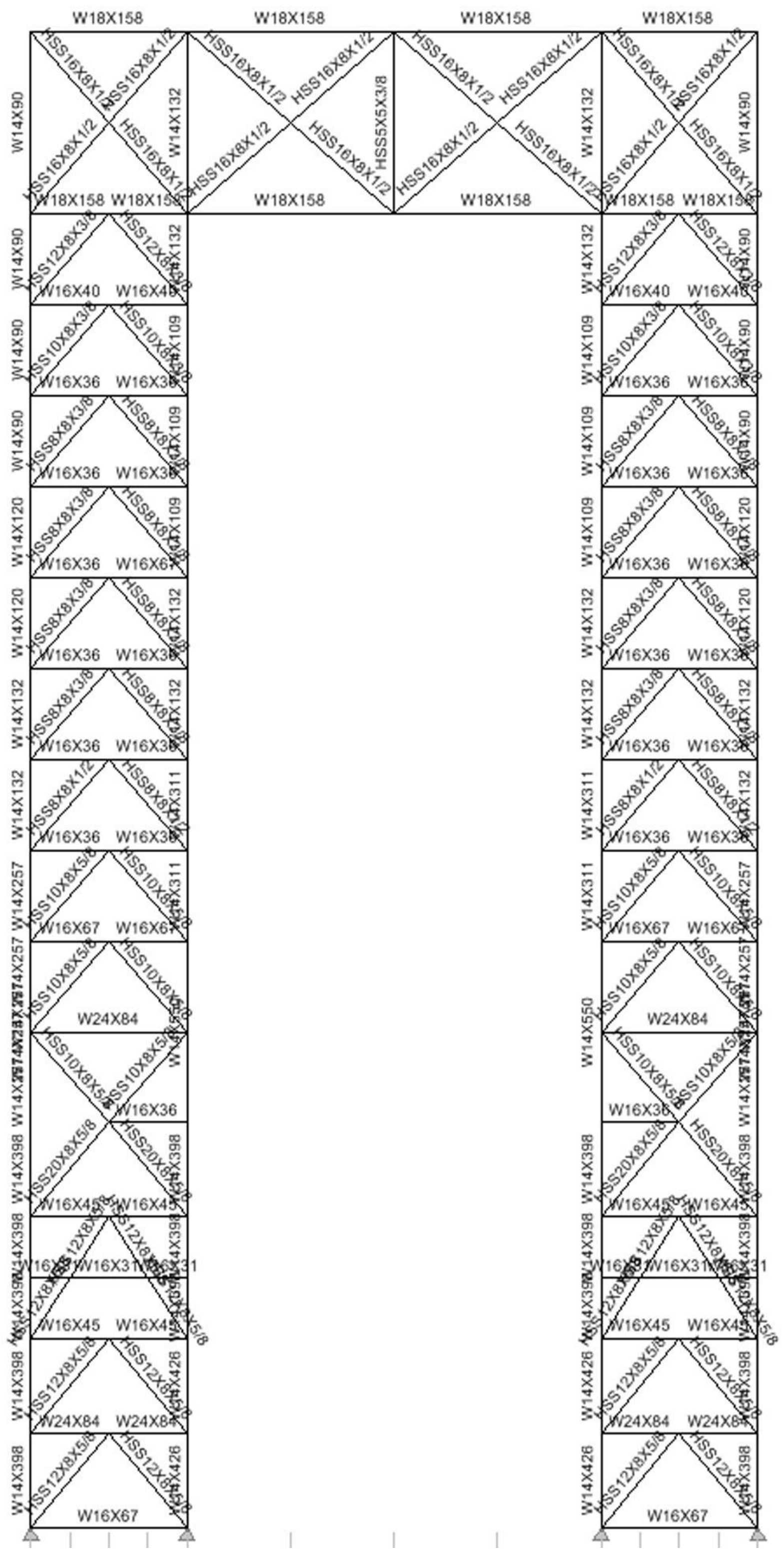


Figure 44 – Braced Frame 3 and 4 Design

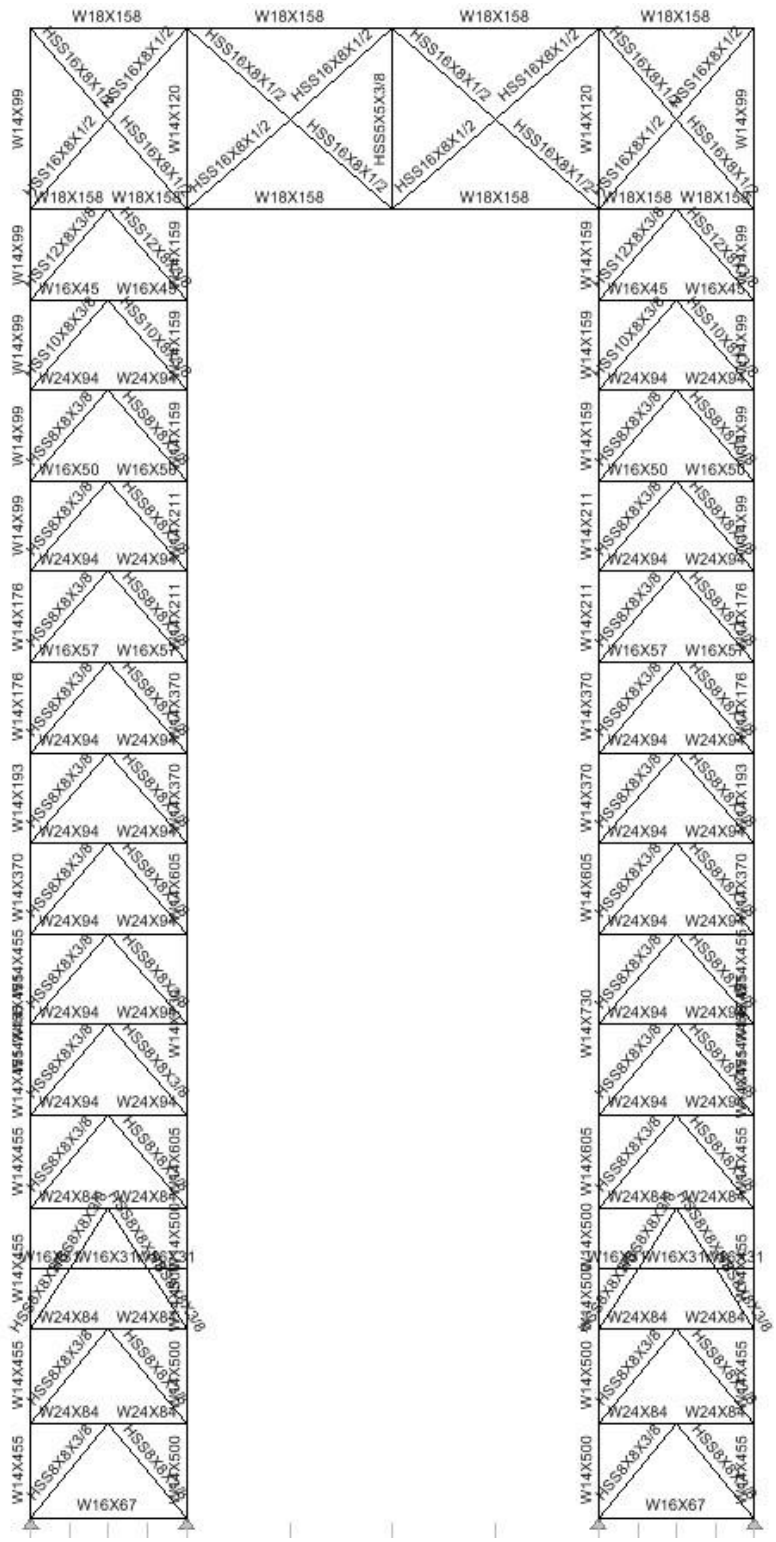
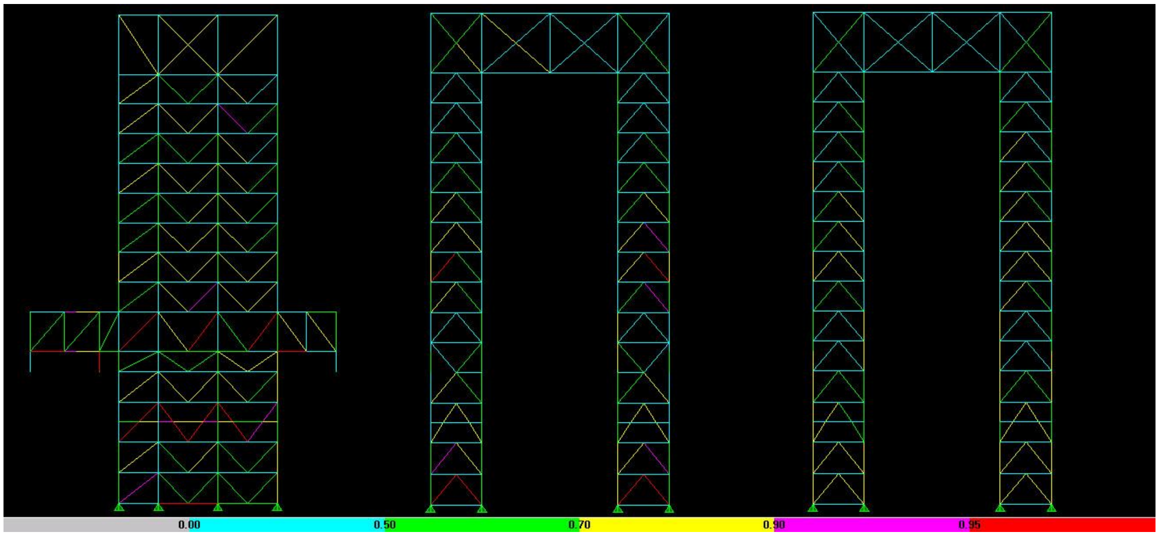


Figure 45 – Braced Frame 5 and 6 Design



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

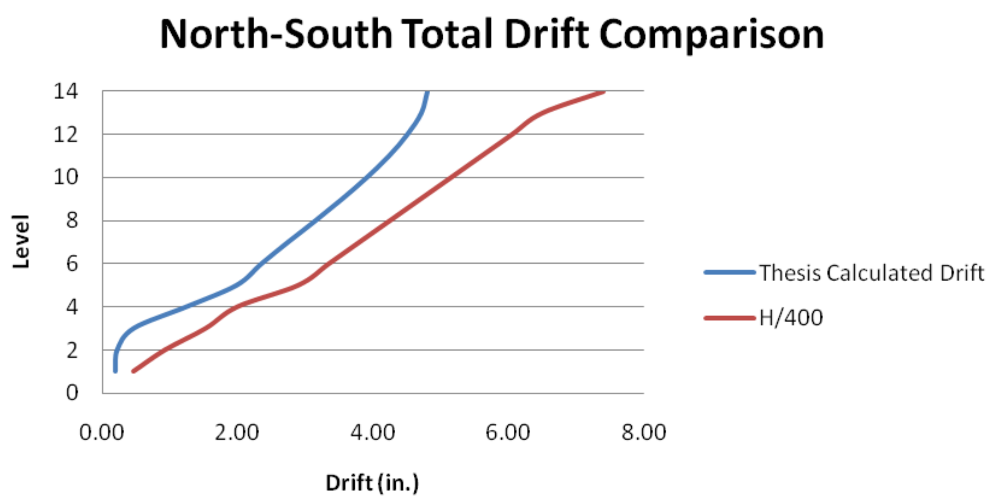


Chart 1 – North-South Total Building Drift Comparison

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	Story Height	Story Drift δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			Total Drift 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	Story Height	Story Drift δ_{xe}	Calculated Drift δ_{xe} (c_d/I)	Allowable Story Drift $\Delta_{seismic} = 0.015h_{sx}$			Total Drift 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

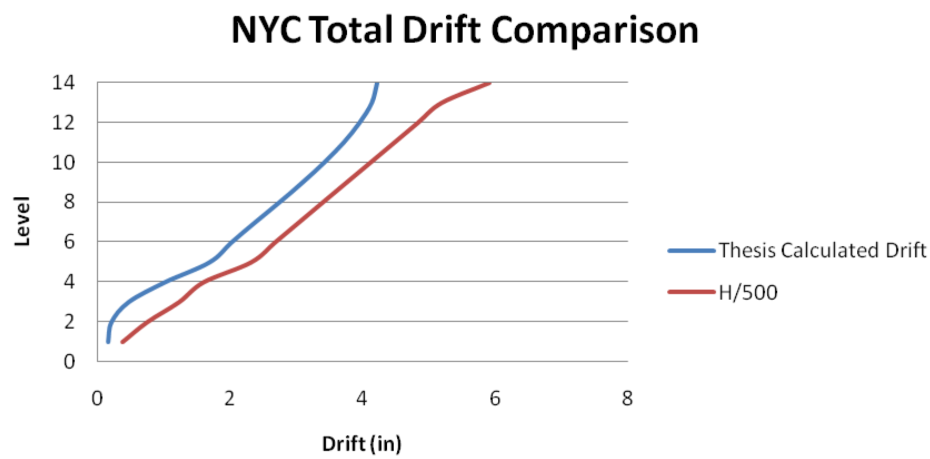


Chart 2 – NYC Building Code Total Building Drift Comparison

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

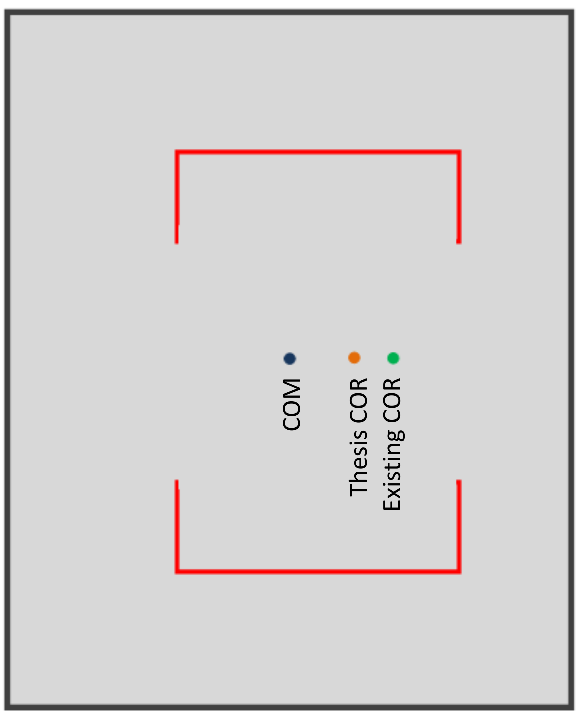


Figure 47 – Movement of the Center-of-Rigidity

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"

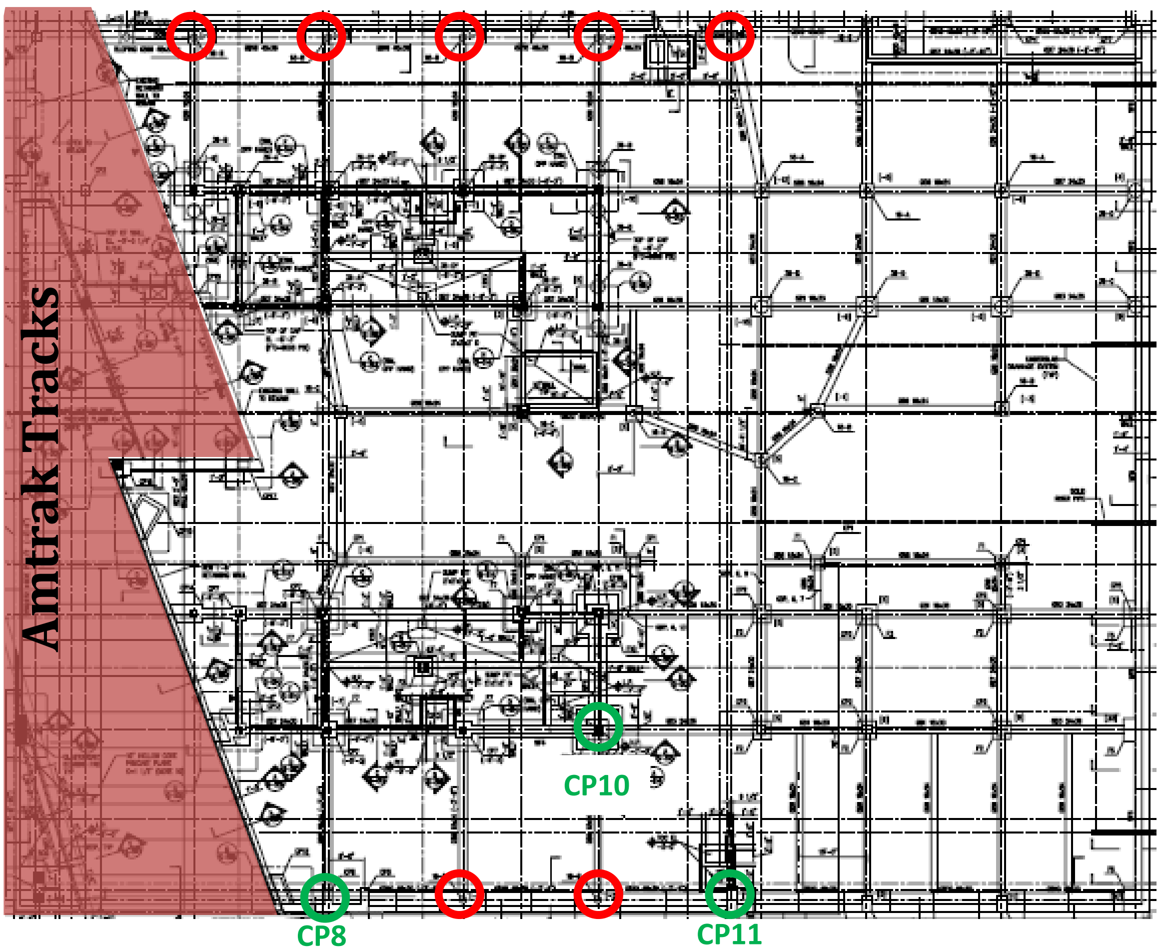


Figure 48 – Foundation Impacts of the New Transfer System. (Red circles represent concrete caisson 18-B which must be increased in size to a 36-C. Green circles are concrete piers, which were determined to be adequate for the new transfer system.)

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.