440 FIRST STREET, NW

WASHINGTON, D.C.

FINAL ◆ REPORT

YEMI OSITELU | STRUCTURAL OPTION

ADVISER | DR. ALY SAID

APRIL 8TH, 2016

440 FIRST STREET

YEMI A. OSITELU | STRUCTURAL OPTION

ADVISOR: **DR. ALY SAID**

GENERAL DESCRIPTION

LOCATION	WASHINGTON, D.C.
OCCUPANCY	OFFICE/ RETAIL
SIZE	141,929 SQUARE FT.
NUMBER OF STORIES	11 (ABOVE GRADE)
ACTUAL COST INFO.	$20,000,000 (RENO.)

PROJECT TEAM

NEW CONSTRUCTION

OWNER	FP FIRST STREET, LLC
GENERAL CONTRACTOR	SIGAL CONSTRUCTION
ARCHITECT	FOX ARCHITECTS
CIVIL ENGINEER	VIKA
STRUCTURAL ENGINEER	RGA
MEP ENGINEER	VANDERWEIL
LIGHTING CONSULTANT	C.M KLING & ASSOC.
SPECS. WRITER	BETHEL SPECS.
LEED CONSULTANT	LORAX
CODE CONSULTANT	AON RISK SOLUTIONS

EXISTING CONSTRUCTION

ARCHITECT	VLASTMIL KOUBEK, AIA
STRUCTURAL ENGINEER	BASKAM & JURCZYK
MECHANICAL & ELECTRICAL	THE OFFICE OF LEE KENDRICK

ARCHITECTURE

440 First Street, NW, is located between D and E Streets in downtown Washington, D.C. The existing 8-story building was constructed in 1982 and renovation was initiated in 2012. It has 10 stories + a mechanical penthouse, and there are two existing below grade parking garages, which were repaired and utilized as a valet parking facility. The new façade is a combined glass-and-metal curtain wall system, which allows for outstanding views and more importantly, natural daylighting.

STRUCTURAL SYSTEM

FRAMING SYSTEM

EXISTING *Cast-in-place concrete with two-way structural concrete slabs and reinforced concrete columns and beams.*

NEW *Composite steel framing with 5 1/4" slabs*

LATERAL SYSTEM

EXISTING *Slab-Column Concrete Frames*

NEW *Steel Moment Frames*

FOUNDATION

Walls and columns are supported by spread footings.

MECHANICAL SYSTEM

During the renovation of 440 First Street, the primary mechanical (DOAS) systems were replaced and resulted in a 25% reduction in energy usage. It consists of 3 mechanical rooms housed in the penthouse and 2 cooling towers on the penthouse roof. Openings were created in the steel beams and girders for ductwork and piping due to small ceiling heights

SUSTAINABILITY

- Majority of the building 's structural elements will be reused
- Green Roof will have local plants that require minimal watering and also reduces storm water overflow and minimizes "heat island" effect
- Recycled materials are used and are obtained regionally
- The building has achieved LEED Platinum Certification

LIGHTING/ELECTRICAL SYSYTEM

The curtain wall and the many windows in the façade provide the building with natural daylighting, improving energy efficiency.

The interiors are well lit with LED fixtures and other various energy efficient light fixtures

ALL IMAGES COURTSEY OF JEFF GOLDBERG OF ESTO PHOTOGRAPHY FOR FOX ARCHITECTS

TABLE OF CONTENTS

EXECUTIVE SUMMARY

440 First Street is a mixed-use building located in Washington, D.C. The existing 8-story building, constructed in the early 80's, began renovation in March 2012 and was completed in April 2013. The building also has two levels of below-grade parking. Three stories were added to the building, including a penthouse, resulting in a 20.6 foot increase in building height and a total gross square footage of about 142000 GSF. The new 10- story architectural design provided a seamless transformation of the existing building into a more modern, state-of-the-art building, well on its way to a platinum LEED certification.

The primary purpose of this report is to provide and design a structural steel solution for the building, while decreasing the construction cost and schedule.

Earlier reports showed the use of a composite steel joist framing system will provide a feasible design solution to the building. Through preliminary analysis and research, it was decided that the use of *ECOPSAN* composite steel joist along with non-composite beams on the column lines will yield the best design result. This system is a simple, inexpensive method for floor construction. Wind and seismic loads were taken into consideration and thus drove the design of the lateral systems used in the project. The use of moment frames were compared to the use of shear walls, and it was determined that moment frames will provide the best lateral solution without impacting the architecture or cost too negatively in comparison with the shear walls.

A breadth study was conducted into exploring the feasibility of solar thermal system to preheat ventilation (outdoor) air. Heating costs can be very expensive, however, this simple technology provides a very affordable way of utilizing useful solar energy to preheat the outdoor air while ultimately reducing the overall utility costs of the building and the annual energy consumption of the building. The entire penthouse houses the major mechanical equipment, hence, a study was conducted on the challenges of incorporating this new technology with the existing mechanical system. It was determined that this addition was practical and posed minor challenges.

A second breadth study was conducted to explore the impacts of these redesigns on the total construction cost and schedule of the project. It was determined that these redesigns were feasible, would not impact the schedule in too negative a way, and saw a 54% decrease in structural costs.

ACKNOWLEGEMENTS

The author of this report wishes to appreciate and sincerely thank the following individuals for their patience, understanding, and guidance to assist in the completion of this thesis study.

RATHGEBER/GOSS ASSOCIATES

Mike Goss,

Justin Domire

Paul White

Bill Duval

Brad Elhers

FOX ARCHITECTS

JP Spickler

THE PENNSYLVANIA STATE UNIVERSITY

Dr. Aly Said

Dr. Stephen Treado

M. Kevin Parfitt

This thesis project is dedicated to the Almighty God for His grace and favor throughout the entire process and family, friends and classmates for their much needed support.

INTRODUCTION

PURPOSE

The purpose of this report is to outline and describe the structural system and other design concepts behind it. The report includes an in-depth look into the structural systems used, specifically the gravity and lateral systems. Furthermore, there will be a description of the codes used in 440 First Street.

BUILDING OVERVIEW

First Potomac (FP) 440 First Street, NW, as seen in Figure 1, is located between D and E streets in downtown Washington, DC near the United States Capitol. The existing building was originally an 8-story building constructed in 1982 and had no major upgrades until the renovation began in 2012. The renovation comprised of adding three floors, an additional 34,500 SF, which resulted in a 32% increase in floor space over the existing 106,850 GSF. The building height was raised 20'- 8" and two floors as the existing roof (story height = 11'- 8") was removed through the use of Transfer Development Rights, thus allowing three 10'-9" stories within a total of 32'-3". The renovated building comprises of 10 stories above grade, which includes a penthouse level and 2 stories below grade.

Figure 1 | **View From Adjacent Building**

440 First Street is an office/retail building that has been re-constructed to fit the modern day requirements, while remaining aesthetically appealing.

STRUCTURAL DESIGN

This section offers a broad description of the overall structural design, including an in-depth look into the design criteria and the structural systems proposed for the renovation and addition.

OVERVIEW OF THE STRUCTURAL SYSTEM

Building Materials

The following ASTM standards and design stresses shall be used for the appropriate materials used in the construction of this project.

STRUCTURAL STEEL		
Member	Grade	Fy
Rolled Shapes	ASTM A992, Grade 50	50
Channels, Angles and Plates	ASTM A36	36
Structural Tubing	ASTM A500, Grade B	46
High Strength Bolts	ASTM A325-N	-
Expansion Anchors	HILTI KWIK Bolt TZ	-

MASONRY		
Use	Grade	Strength (PSI)
Load Bearing Concrete (Hollow and Solid)	ASTM C90	1900
Load Bearing Concrete (Brick)	ASTM C55	2000
Mortar	ASTM C270	-
Grout	ASTM C476	2000
Horizontal Joint Reinforcing	ASTM A82	-
Compressive Strength of Masonry	-	F'm = 1500 PSI

CONCRETE AND REINFORCING		
Use	Weight	Strength (PSI)
Slabs-on-grade (Interior)	145	3000
Slabs-on-grade(Exterior)	145	4500
Fill on metal deck	115	3500
Topping	145	3000
REINFORCEMENT		
Use	Grade	
Deformed Reinforcing Bars	ASTM A615, Grade 60	
Welded Wire Fabric (WWF)	ASTM A185	

EXISTING FRAMING

The existing building is a cast-in-place concrete structure consisting of two-way structural concrete slabs and reinforced concrete columns and edge beams. A concrete slab on grade is used at the lowest level of the garage. Furthermore, concrete columns and foundation walls are supported by spread footings.

EXISTING SLAB, GARAGE AND FRAMING RENOVATIONS

The existing roof slab and penthouse were removed and the existing slab edges were added to on all four sides for two reasons: increasing the net rentable space for each floor, and to provide a consistent location for new façade connections, as seen in Figure 2. Also, at the front of the building, slab edge and curtain wall at the corner column bays were extended to the property line, requiring cantilevered channel sections which were through bolted to the existing concrete columns, and support a new composite concrete slab.

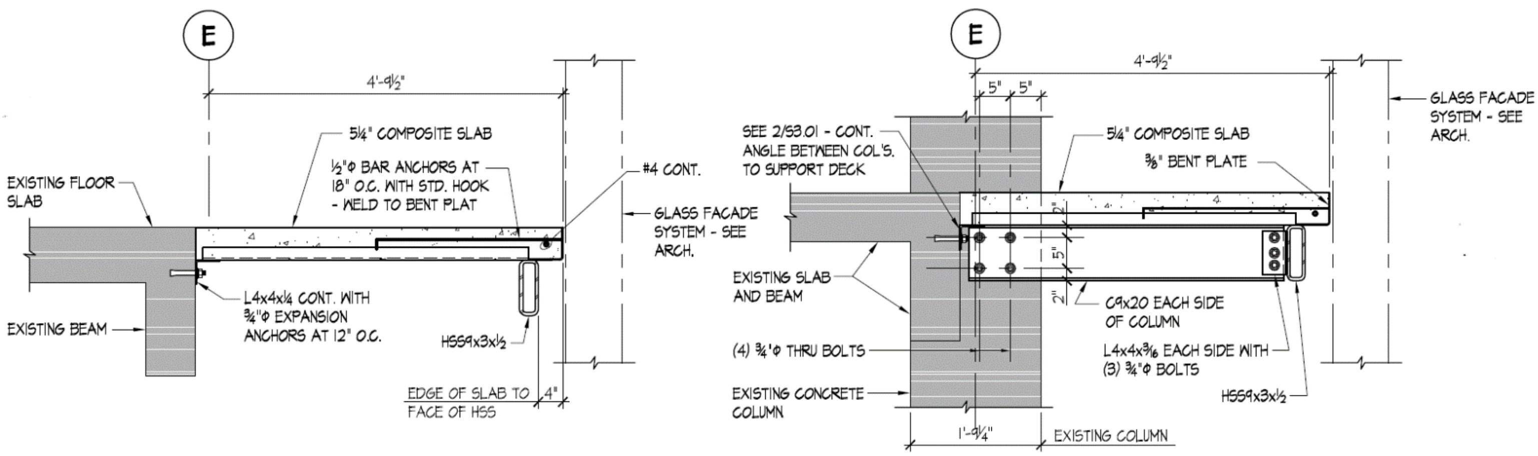


Figure 2 | **Slab Extension Details**

Slab extension at floors 2 through 8 will occur at the east side of the building toward the north, to match the new upper floors.

The existing garage levels had experienced serious deterioration due to road salts brought in on cars, and the design drawings contained repair plans and details. This work was performed first, and allowed parking for workers of all trades as the construction progressed.

FLOOR SYSTEM

As aforementioned, the floor system is comprised of steel reinforced cast-in-place concrete two-way slab system on typical floors (2-8). It consists of 5 ¼" lightweight concrete on a 2", 18 gage galvanized composite metal deck (total thickness = 7") reinforced with 6x6-W2.9xW2.9 WWF on typical floors, unless noted otherwise. Other slab thickness vary from 5 ¼" – 9 ½", as seen in Figure 3, depending on the location.

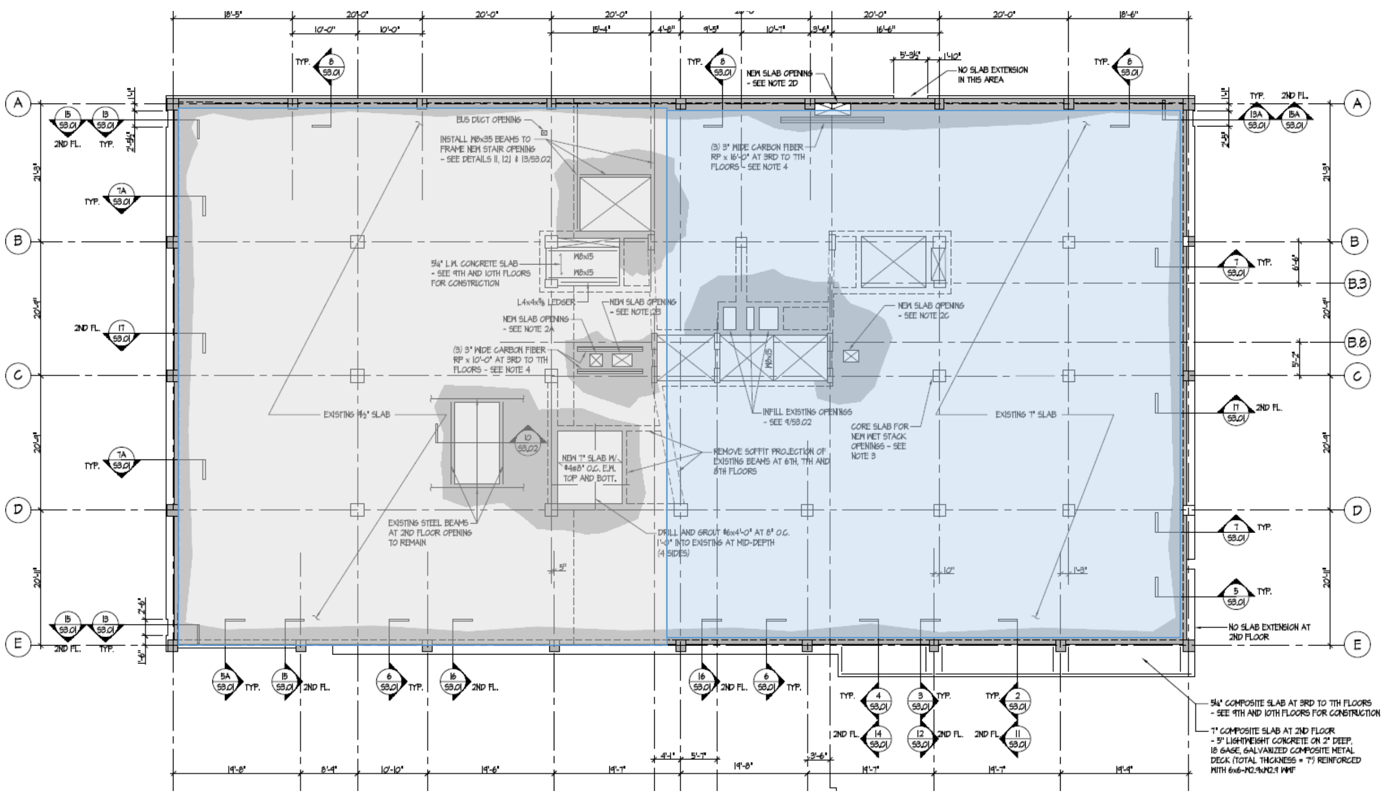


Figure 3 | **Typical floor plan of the building**

9 ½" slab 7" slab

ADDITION FRAMING SYSTEM OVERVIEW

There is an addition of three stories of steel framing (two new floors and a roof/penthouse) above the existing 8th floor. The new framed floors and roof are constructed using composite framing with a 5 ¼" thick structural slab (comprised of 3 ¼" of lightweight concrete fill on a 2" thick, 18 gage metal deck), reinforced with 6x6-W2.0xW2.0 WWF. Figures 4 and 5 show a typical and partial structural steel framing plan respectively, with beams spaced at 10'-0" on center and girders spanning 20'-0" between columns. Beam and girder sizes are typically W10's, W14's and W18's.

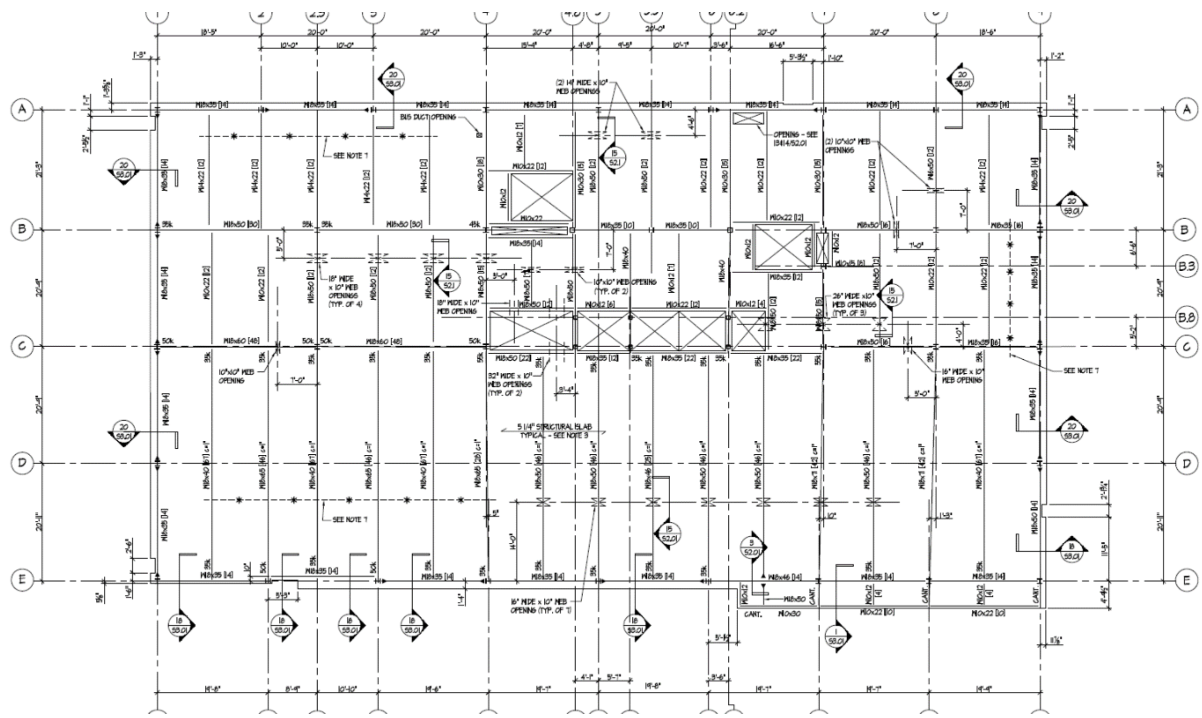


Figure 4 | **Typical Structural Framing Plan Of The Building**

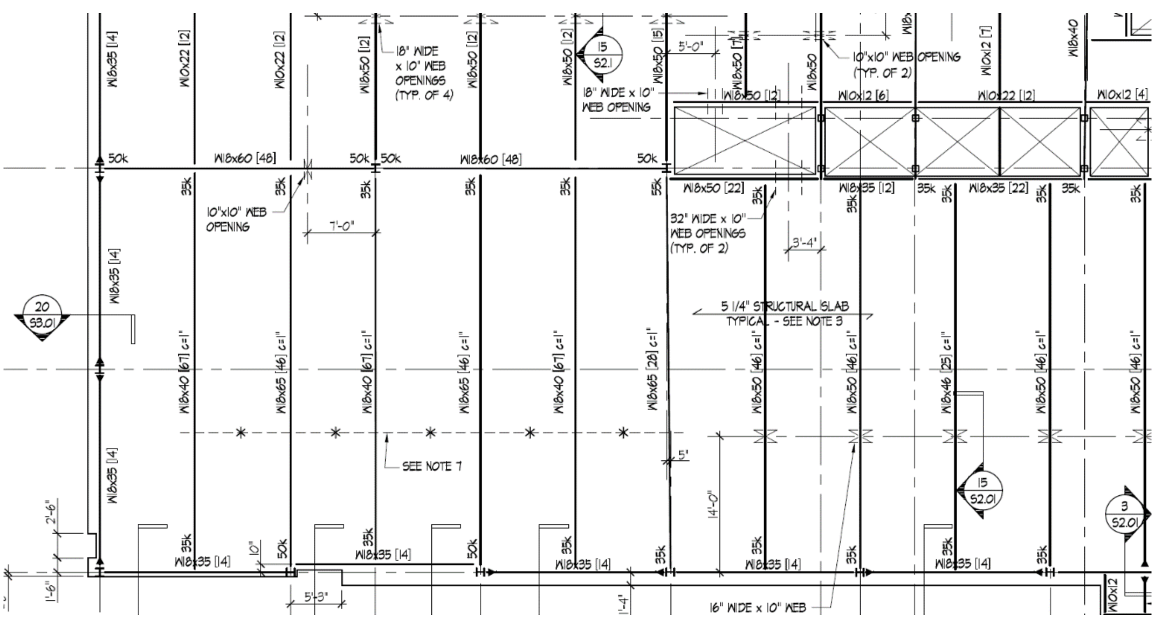


Figure 5 | **Partial Structural Framing Plan of the building**

A two hour fire rating is achieved by spraying fire-proofing the beams and girders.

ROOF SYSTEM

The roof framing system as hinted earlier, is a structural steel system. It can be broken down into two parts: the main roof/penthouse framing plan and the penthouse roof framing, as shown in Figures 6 and 7. The penthouse roof deck is a 1 ½" deep, wide rib, 20 gage galvanized metal deck.

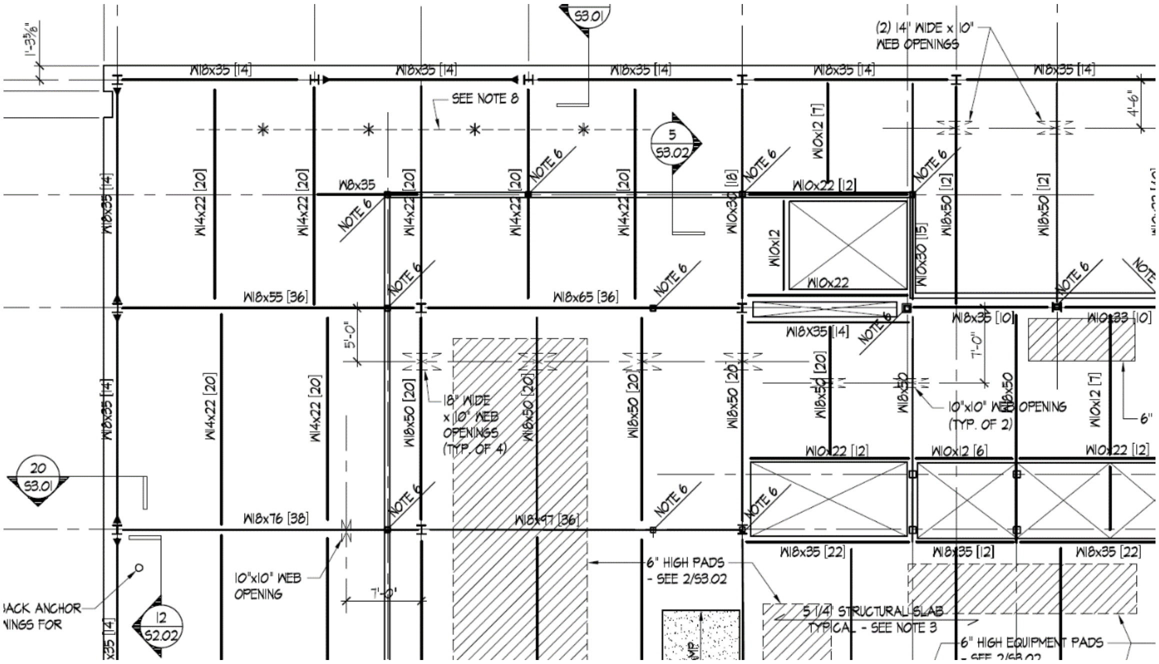


Figure 6 | **Partial Main Roof/Penthouse Framing Plan**

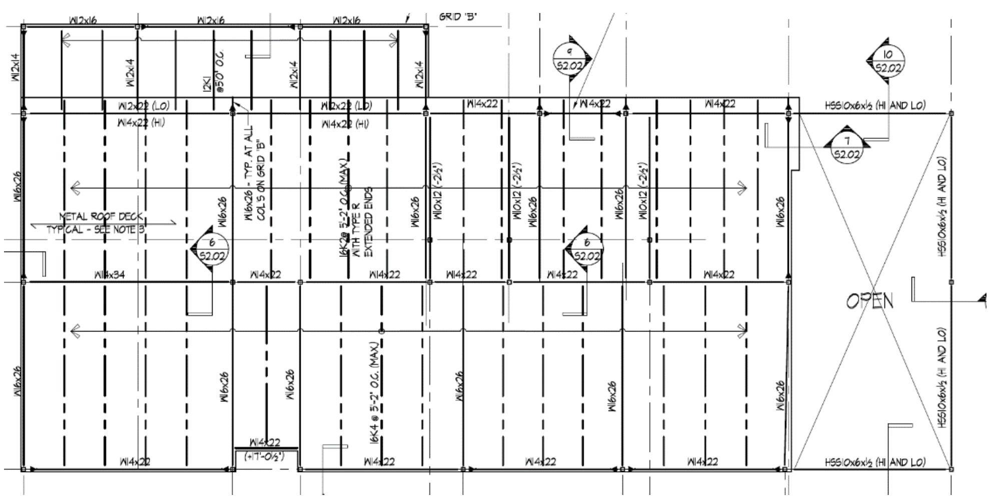


Figure 7 | **Penthouse Roof Framing Plan**

The penthouse floor framing plan includes an additional framing for the 12000 LBS cooling tower, as seen in Figure 8 and provides requirements for the 6" high equipment pads, as shown in Figure 9

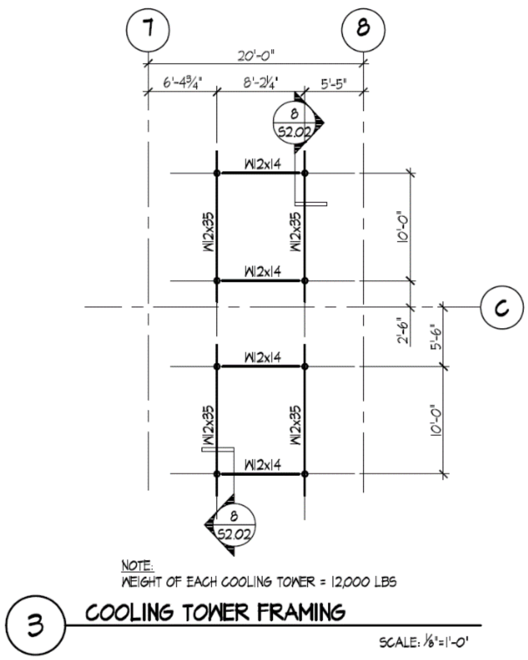


Figure 8 | **Cooling Tower Framing Plan**

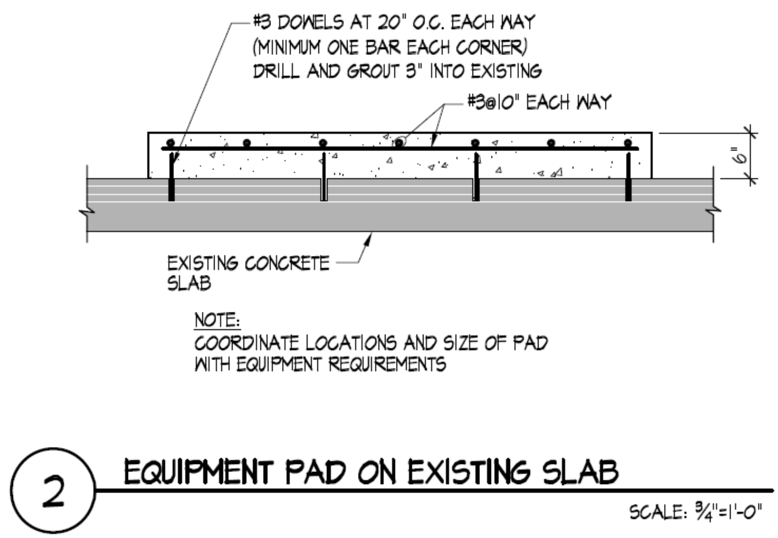


Figure 9 | **Equipment Pad Framing**

TYPICAL BAY – FLOOR SYSTEM

There are several bay sizes used in 440 First Street. A typical bay, 20' x 20'-11", was selected from the framing plans for floors 2 – 7, and is highlighted in Figure 10 below. Due to the different thicknesses of slabs (7" and 9 ½") on the typical floors, slab reinforcement varies.

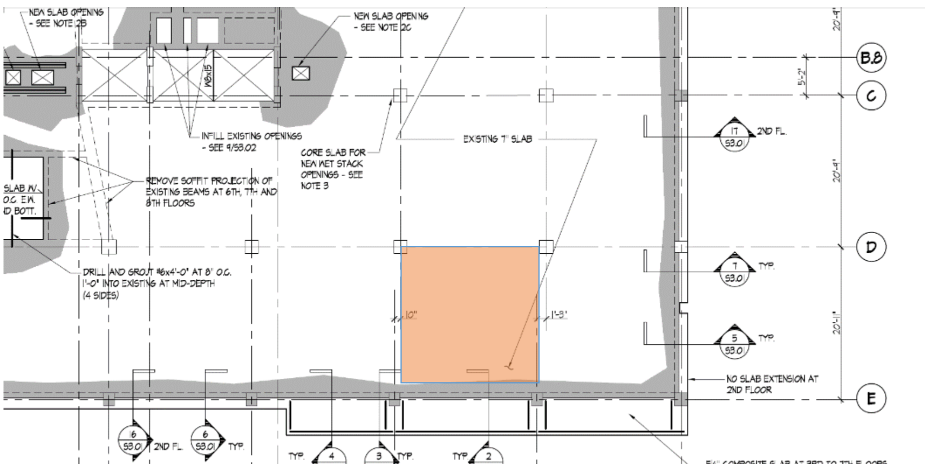


Figure 10 | **Typical Bay Shown**

For the new additional floors, typical girders will span 20 feet between columns and beams are spaced at 10 feet on center. Figure 11 shows a typical bay size, with beam sizes varying. The most common sizes are W10's, W14's and W18's as mentioned earlier.

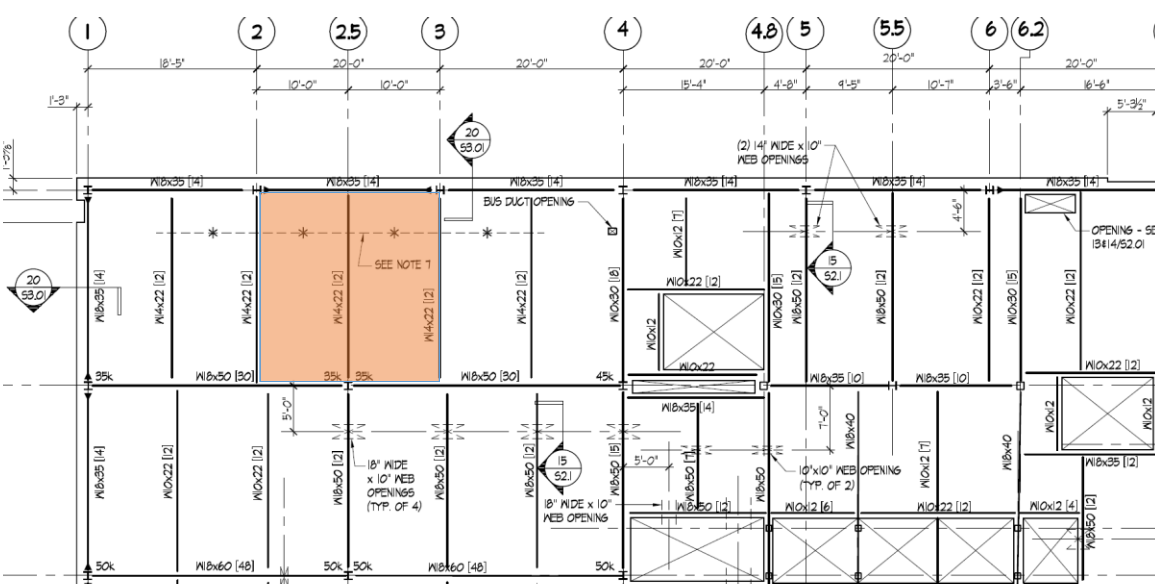


Figure 11 | **Typical Bay in Structural Framing**

COLUMNS

From the 8th floor, new steel columns were added and centered to the existing columns. The additional framing provides a column layout that creates interior column free space by eliminating the first interior columns on the east side of the building, as shown in Figure 11. The new columns will typically be 10” wide by 10” deep steel wide flange shapes.

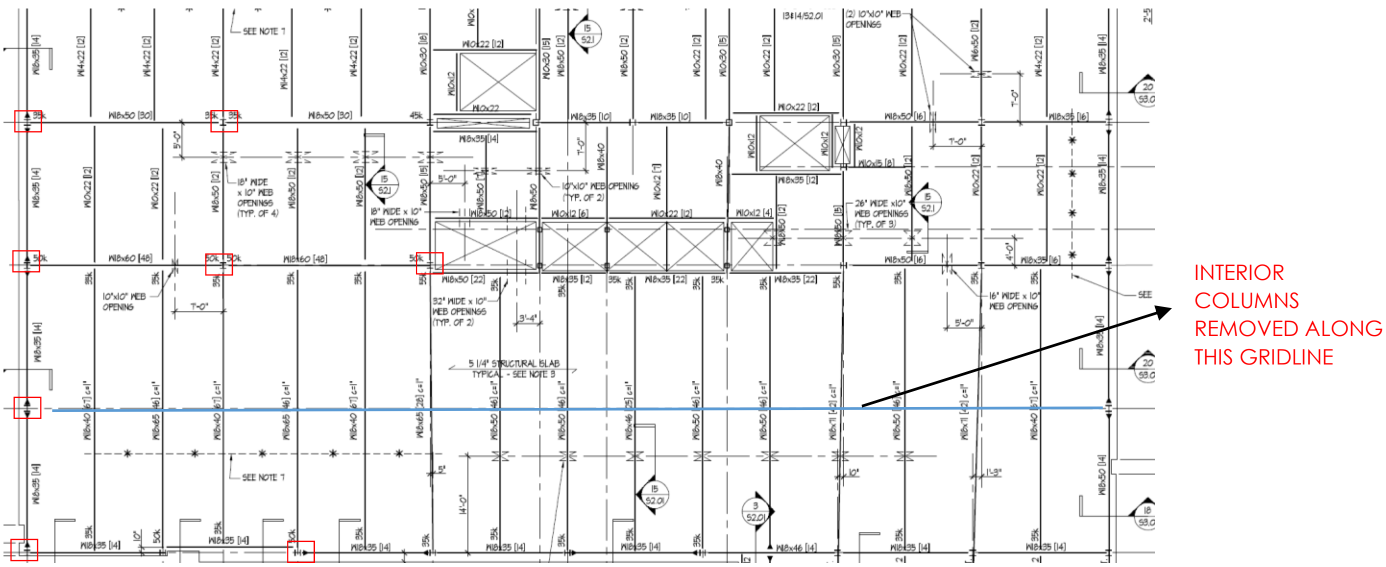


Figure 12 | **Steel Columns Highlighted in Red**

The rebar for the existing concrete column was to be retained for a height of 3’- 0” above the 8th floor slab, following the demolition of existing roof and penthouse removal, as shown in the column detail in Figure 13.

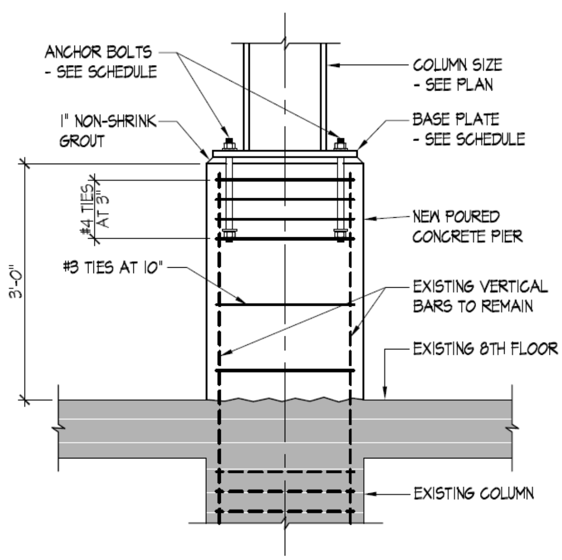


Figure 13 | **Column Base Detail**

A preliminary analysis indicated that removing the existing concrete roof and penthouse roof, in addition to removing the building facade on all 4 sides, provided a column load reduction that enabled the new totals to be comparable to the column loads on the existing base building drawings, after the new steel frame loads were added.

The new building façade consists of a state-of-the-art aluminum curtain wall at the east elevation and masonry walls at the other faces.

LATERAL SYSTEM

The lateral force resisting system consists of moment connections at the new steel framed levels, and will be used in conjunction with the slab-column frames at the existing levels.

The 2009 International Building Code chapter 34, Section 3403.4, which requires that an existing structure and its addition acting together as a single structure be shown to meet the requirements for wind and seismic design per 1609 and 1613. With that said, it allows an exception which states that load-carrying structural elements, columns and footings in this case, whose demand-capacity ratio with the addition is no more than 10 percent greater than its demand-capacity ratio with the addition shall be permitted unaltered.

Figure 14 and Figure 15 on the next page show the location of the steel moment frames on the new levels and the slab-column frames on the existing levels.

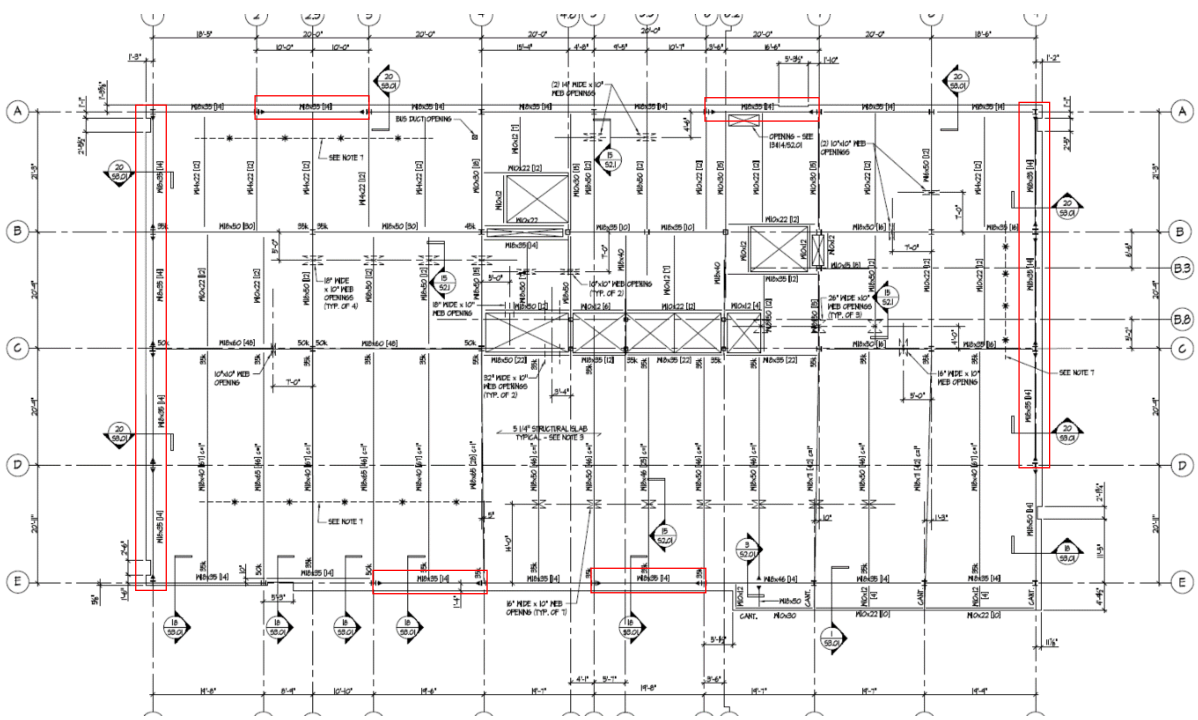


Figure 14 | **Steel Moment Frames Highlighted In Red**

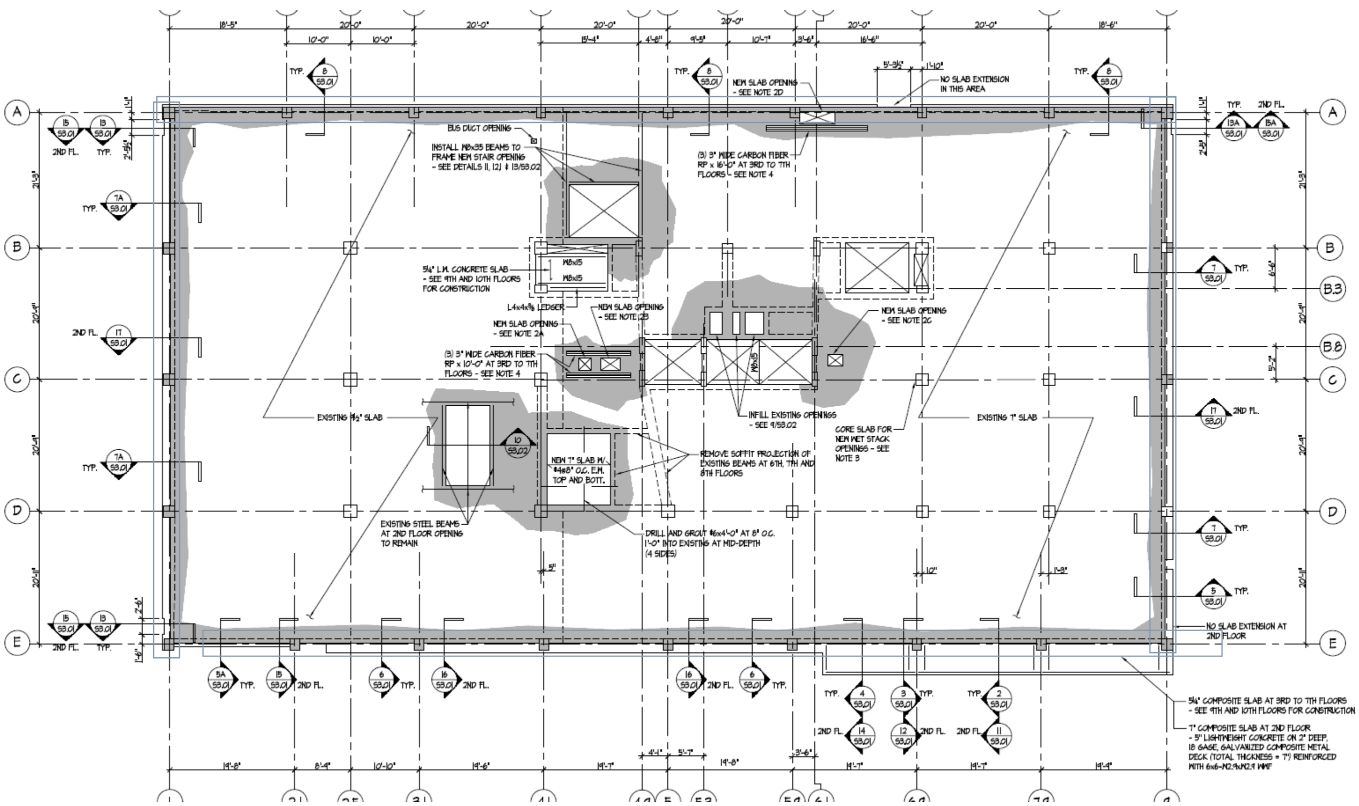


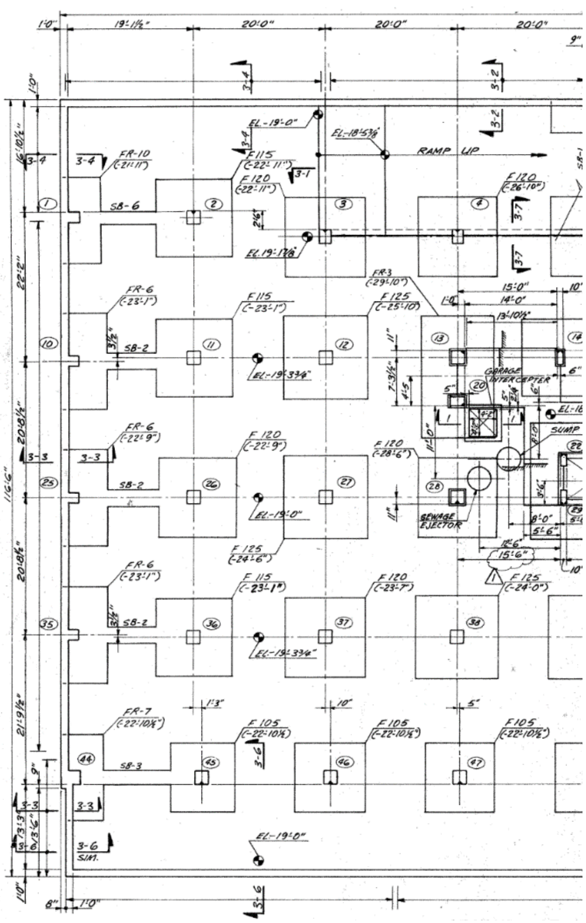
Figure 15 | **Slab-Columns Moment Frames Highlighted in Blue**

FOUNDATION SYSTEM

A geotechnical report was done by Schnabel Engineering Associates in the 1980's. They recommended foundation requirements for the support of the proposed building and floor slabs on grade, after an evaluation and analysis of subsurface conditions. The concrete columns and foundation walls are supported by spread footings.

Recommended design bearing values are 6000 PSF for the column footings and 4000 PSF for the wall footings. With the proposed addition of the new building, no new soil reports were performed since load reduction from removed components outweighed the additional loads from new floors.

A partial cellar plan and a typical footing detail are shown the Figures 16 and 17 respectively.



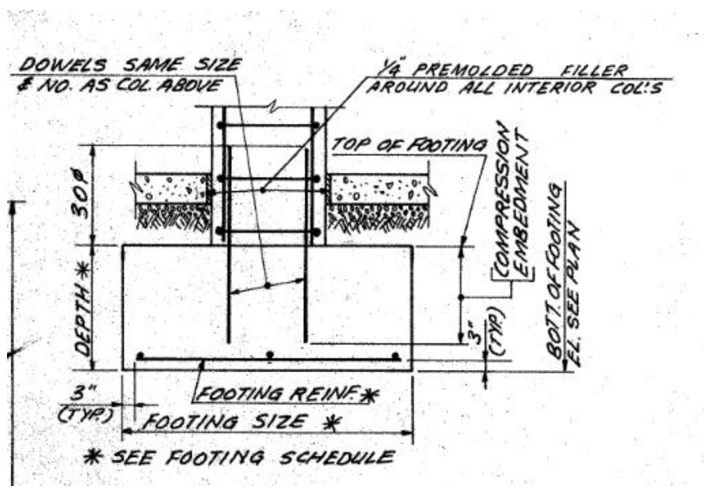


Figure 16 | **Partial Cellar Plan**

JOINT DETAILING AND DESIGN MODIFICATIONS

Connection detailing is key to the success of any steel structure. It is imperative that the various types of connections are correctly detailed to ensure proper load transfer between various members.

STEEL MOMENT CONNECTION DETAIL

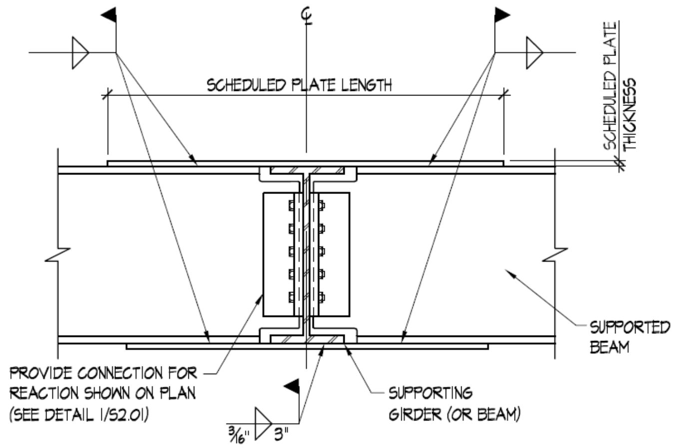


Figure 18 | **Moment Connection Detail Beam to Girder**

BEAM TO COLUMN CONNECTION DETAIL

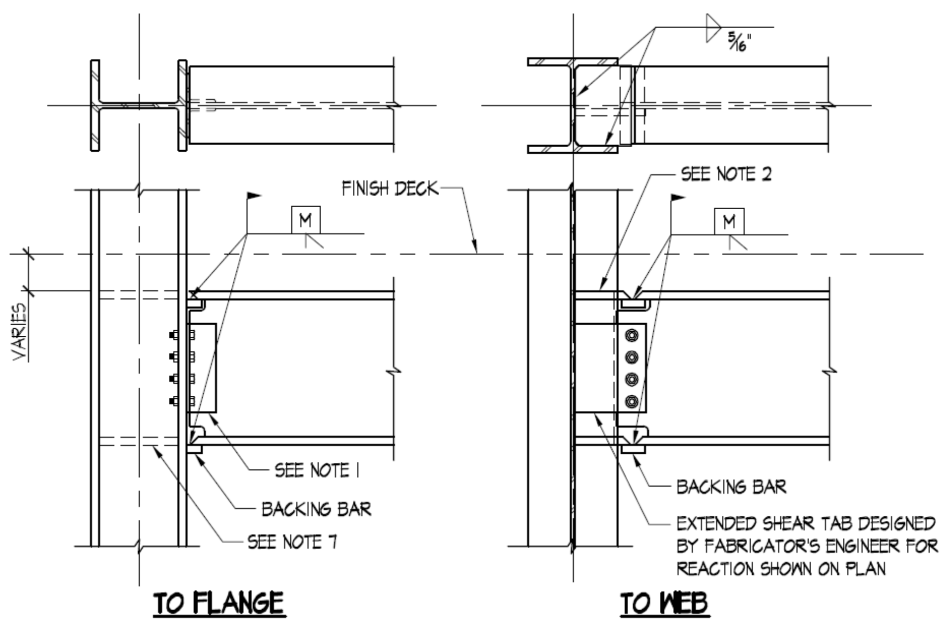


Figure 19 | **Beam to Column – Fully Restrained Moment Connection**

OTHER ADDITIONAL DETAILS

With ceiling heights of 8'-4", and a steel frame used to limit the added loads to the existing columns and footings, there was not enough room to accommodate ductwork under the structure. After careful consideration, it was decided to design the steel beams and girders with openings for ductwork and piping. A total of 99 openings were detailed, as shown below, and included in the design.

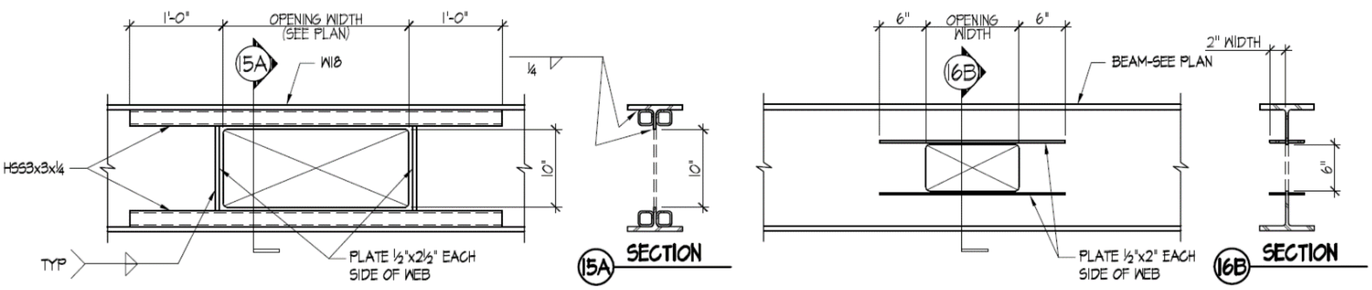


Figure 20 | **Web Opening Detail**

STRUCTURAL SYSTEM STUDY & REDESIGN

PROBLEM STATEMENT

The structural design of the current 440 First Street building consists of both concrete (Basement levels + Typical Floors) and Composite Steel (Newly added floors + the Penthouse). While analysis of the existing structure showed no major flaws, it was found, during the study of alternative systems in Technical Report III, a composite steel joist framing system might prove to be a possible alternative for the building. This system proves to more easily constructible than the original, and showed very comparable slap depth and overall cost. The overall weight of the building will also have a significant decrease due to the use of lightweight steel as a solution.

PROPOSED SOLUTION

The proposed solution to improve the constructability of the design will be to redesign the entire building in steel. The gravity system will look at the use of a composite steel joist framing with non-composite beams used along the column lines. The lateral systems will consists of steel moment frames systematically placed to ensure stability on the entire structure and limit twist. These systems were selected to provide the most economical solution.

The mechanical penthouse is also a point of interest. In an attempt to possibly increase the overall efficiency of the building, a mechanical breadth will be explored which involves the use of solar thermal energy to preheat the ventilation (outdoor) air. Implementation of this system can reduce utility bills and the annual energy consumption of the building.

IMPLICATIONS OF REDESIGN

The overall weight of the building should see a significant decrease with composite steel joists being very lightweight in nature, with the wind load cases will most likely controlling the design of the lateral systems. Additionally, the use of composite steel joist framing will allow for the mechanical ducts and piping to be passed underneath or through the open webs of the joists, which can also lead to a possible reduction in the overall floor-to-floor height of the structure. Furthermore, the design change should not see any significant impacts on the foundation, however, it will be considered. Construction cost and scheduling impacts will also be considered.

GRAVITY LOADS

The summary of the design gravity loads used for the design and member spot checks are as follows;

DEFLECTION CRITERIA

IBC 2012 – TABLE 1604.3 DEFLECTION LIMITS

Live load Deflection (Typ.)	L/360
Total Deflection (Typ.)	L/240

GRAVITY LOADS – FLOOR

FLOOR DEAD LOADS		
	DESIGN LOAD	REFERENCE
LIGHT WEIGHT CONCRETE	115 PCF	ACI 318 - 11
CEILING	5 PSF	STRUCTURAL DRAWINGS
MEP	15 PSF	STRUCTURAL DRAWINGS
SPRINKLERS	3 PSF	STRUCTURAL DRAWINGS
ROOF TOP CONCRETE PAVERS	25 PSF	STRUCTURAL DRAWINGS

FLOOR LIVE LOADS		
AREA	DESIGN LOAD	REFERENCE
OFFICE + PARTITIONS	100 PSF	STRUCTURAL DRAWINGS
LOBBIES/STAIRS/EXITS	100 PSF	ASCE 7-10
PENTHOUSE FLOOR	100 PSF	STRUCTURAL DRAWINGS
CORRIDORS ABOVE FIRST FLOOR	3 PSF	ASCE 7-10
PARKING	50 PSF	ASCE 7-10

GRAVITY LOADS – ROOF

ROOF LIVE LOADS		
AREA	DESIGN LOAD	REFERENCE
PENTHOUSE ROOF	30 PSF	STRUCTURAL DRAWINGS
MAIN ROOF	100 PSF	ASCE 7-10

GRAVITY LOADS – EXTERIOR WALL LOADS

EXTERIOR WALL LOADS		
AREA	DESIGN LOAD	REFERENCE
FACE MASONRY	39 PSF	INTERNATIONAL CODE COUNCIL
CURTAIN WALL SYSTEM	10 PSF	INTERNATIONAL CODE COUNCIL

LATERAL LOADS

Moment frames were the only lateral systems analyzed for this report. The wind loads are based on the building geometry and the seismic loads are based on the weight of the building. Furthermore, an R of 3 was used for the lateral system to avoid the necessity of seismically detailed connections. Below are the summarized wind and seismic loads used in the design of the steel moment frames. More detailed hand calculations can be seen in the Appendix B.

DEFLECTION CRITERIA

Allowable Building Deflection	***H/240 (WITH 1.0 WIND)****
Wind Allowable Inter-Story Drift	***H/240 (WITH 1.0 WIND)****
Seismic Allowable Story Drift	$\mathbf{0.02}h_x$

WIND LOADS

The design wind loads were calculated using the procedure in ASCE 7-10, Section 27. The tables below show the parameters used and a summary of the base shear and moment.

FACTOR	DESIGN VALUE	REFERENCE
Kzt	**1**	SEC. 26.8.2
Kd	**0.85**	SEC. 26.6
EXPOSURE CATEGORY	**B**	SEC. 26.7.3
V	**115**	SEC. 26.5
I	**1**	TABLE 1.5-2

TABLE 1: 440 FIRST STREET WIND FORCES IN BOTH DIRECTIONS							
STORY	HEIGHT	FORCE		SHEAR		MOMENT (FT-K)	
		N-S	E-W	N-S	E-W	N-S	E-W
PHR	127.25	45.68	96.92	0	0	5812.78	12333.07
MR	109.25	25.28	53.9	45.68	96.92	2761.84	5888.58
10	98.5	25.01	53.4	70.96	150.82	2463.49	5259.90
9	87.75	24.29	52.1	95.97	204.22	2131.45	4571.78
8	77	23.75	51.2	120.26	256.32	1828.75	3942.40
7	66.67	23.03	49.83	144.01	307.52	1535.41	3322.17
6	56.33	22.31	48.5	167.04	357.35	1256.72	2732.01
5	46	21.42	46.85	189.35	405.85	985.32	2155.10
4	35.67	20.34	44.86	210.77	452.7	725.53	1600.16
3	25.33	19.17	42.71	231.11	497.56	485.58	1081.84
2	15	25.58	57.93	250.28	540.27	383.70	868.95
G	0	0	0	**275.86**	**598.2**	0.00	0.00
						20370.56	**43755.94**

SEISMIC LOADS

Seismic design loads are calculated using ASCE 7-10, Chapter 12, using the Equivalent Lateral Force Procedure. The table below shows a summary of the base shear and moment for the lateral system (steel moment frames).

TABLE 2: VERTICAL DISTRIBUTION OF SEISMIC FORCES					
LEVEL	HEIGHT	STORY WT.	FORCE	STORY SHEAR	MOMENT
PHR	127.25	410	4.1	0	521.73
MR	109.25	1140	11.4	4.1	1245.45
10	98.5	1140	11.4	15.5	1122.90
9	87.75	1140	11.4	26.9	1000.35
8	77	1140	11.4	38.3	877.80
7	66.67	1140	11.4	49.7	760.04
6	56.33	1140	11.4	61.1	642.16
5	46	1140	11.4	72.5	524.40
4	35.67	1140	11.4	83.9	406.64
3	25.33	1140	11.4	95.3	288.76
2	15	1140	11.4	106.7	171.00
				118.1	**7561.23**

For this building, the wind loads control the lateral design. The applied wind load factor of 1.0 has greater magnitude than the applied seismic load factor of 1.0. Hence, the wind load governs and member checks are performed using the wind loads only.

LOAD COMBINATIONS

The following load combinations were considered for the gravity and lateral analysis.

1. **1.4*D***
2. **1.2*D* + 1.6*L* + 0.5(*Lr or S or R*)**
3. **1.2*D* + 1.6*(Lr or S or R) + (L or 0.5W)***
4. ***1.2D* + 1.0*W* + L + 0.5*(Lr or S or R)***
5. **1.2*D* + 1.0*E* + L + 0.2*S***
6. ***0.9D* + 1.0*W***
7. **0.9*D* + 1.0*E***

Gravity loads are usually governed by load combination case 2 and Lateral loads are usually governed by load combination cases 4 or 5, depending on the magnitude of the lateral load (wind or seismic).

DESIGN GOALS & CRITERIA

DESIGN GOALS

Due to the recent renovation of the entire structure of 440 First Street, there was a thin line as to how much more the building could be improved. Hen

- Redesign the entire building using lightweight structural steel and provide a solution that reduces the entire cost and weight of the building
- Shorten overall construction time by cutting the structural erection schedule
- Provide a solution that does not interfere with the existing architectural design

DESIGN CRITERIA

The structural gravity members were designed using the AISC steel manual (Strength Design). The lateral systems were designed using the calculated wind and seismic loads, with wind loads controlling the design. Below are a list of provisions used in the design of the lateral systems;

- None of the steel moment frames were seismically detailed (R = 3) to reduce cost
- All wind load cases are taken into account for the design
- There are no horizontal or vertical irregularities
- The lateral resisting system has a redundancy factor greater than 1, which is appropriate for building structures of SDC = A

DESIGN EVOLUTION

The placement of the lateral system was the driving force in the initial stages of the design process. The location of the moment frames were relatively convenient as they do not interfere with any openings in the building or the exterior façade. However, torsional issues created the greatest cause for concern, so steps were made to keep the center of mass (COM) and center of rigidity (COR) as close as possible. Minor changes were made to the gridline positions to ensure uniformity in the placement of the moment frames. The location of the steel moment frames in the existing and new design are denoted in red in the Figures 21 and 22.

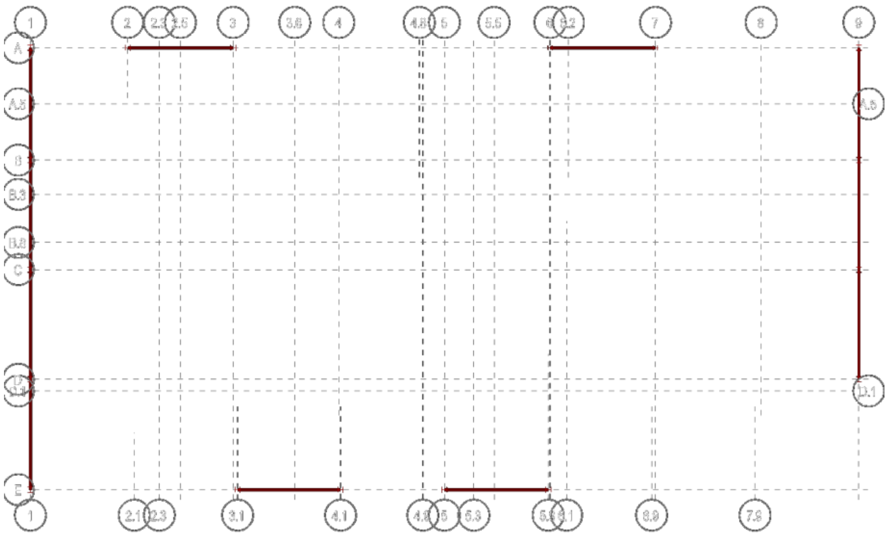


Figure 21 | **Layout Of Moment Frames In Original Design**

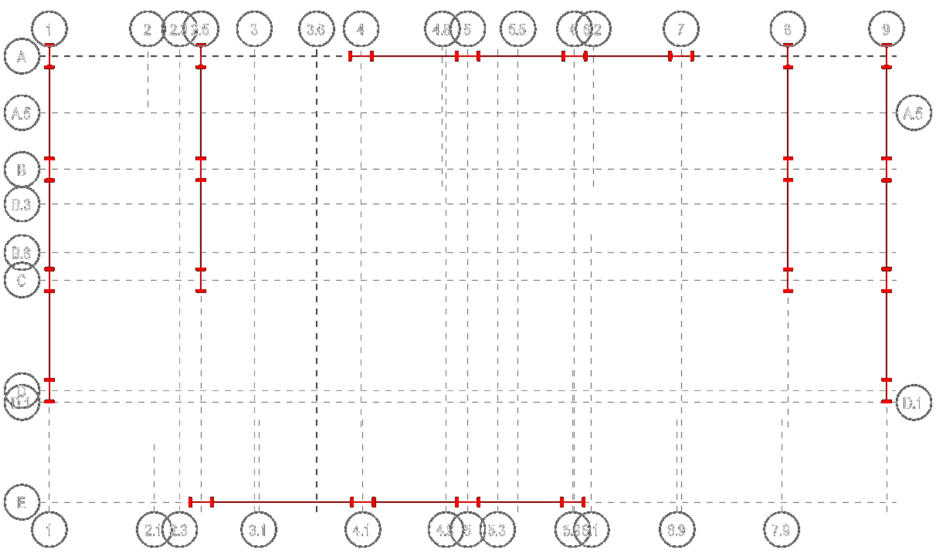


Figure 22 | **Layout Of Moment Frames In New Design**

The use of composite steel joists was another point of consideration. The selection of this system was based on limiting slap depth, reducing construction time and overall weight of the building. The original overall slab depth ranged between 7.25" and 9.5", with beam openings created to allow for passage of ductwork and to maintain ceiling heights of 8'-4". The incorporation of the new structural system allows for a total slab thickness of 4 ½". Furthermore, composite steel joist designs allowed the use of 12" joists with 14" deep wide flange beams used on the column lines to add stiffness to the structure. The composite steel joist designs were completed and governed by the requirements of the ECOSPAN Composite Floor System (http://www.ecospan-usa.com/design-span.html) and calculated using a spreadsheet provide by the manufacturers. A sample of the calculation can be seen in Appendix and the spreadsheet can be found at http://www.ecospan-usa.com/links/Ecospan-Specifications.pdf.

ECOSPAN composite steel joists are very inexpensive and can be readily found across the United States. Furthermore, they are easily constructible and its installation requires significantly less time than the other systems.

The table below shows the span capability for the *ECOSPAN* joists on for residential and commercial buildings.

TYPICAL LOADING	**RESIDENTIAL** Total load = 112 psf Live Load = 55 psf NC Dead Load = 42 psf Comp Dead Load = 15 psf	**COMMERCIAL** Total load = 152 psf Live Load = 95 psf NC Dead Load = 42 psf Comp Dead Load = 15 psf
Depth	Length	Length
10"	25'-0"	25'-0"
12"	30'-0"	30'-0"
14"	35'-0"	32'-8"
16"	40'-0"	37'-4"
18"	45'-0"	39'-0"
20"	46'-8"	43'-4"

Notes:
1. E-series joists are typically spaced at 4'-0" on center.
2. Shaded areas may require special chord or web members.
3. Tables assume 2 1/2" concrete above deck with 3000 psi concrete strength.

THE COMPUTER MODELING PROCESS

A RAM Structural System model was generated to design the gravity and lateral structural systems using previously calculated loads and the standard design criterion. The following modeling assumptions were accounted for:

I. Composite steel joists were modeled as non-composite steel joists, due to the inability of the software to account for the composite action of a joist. The equivalent joists were selected based on depth.

II. A rigid diaphragm was assumed on every level.

III. Accidental and inherent torsion was accounted for.

IV. The moment frame columns on the penthouse level do not line up with the lateral members below. Thus, the gravity members below the moment frames were changed to lateral members to create a pathway from the penthouse to the foundation, with the fixities on both ends of the members also changed.

V. All lateral members were fixed at both ends

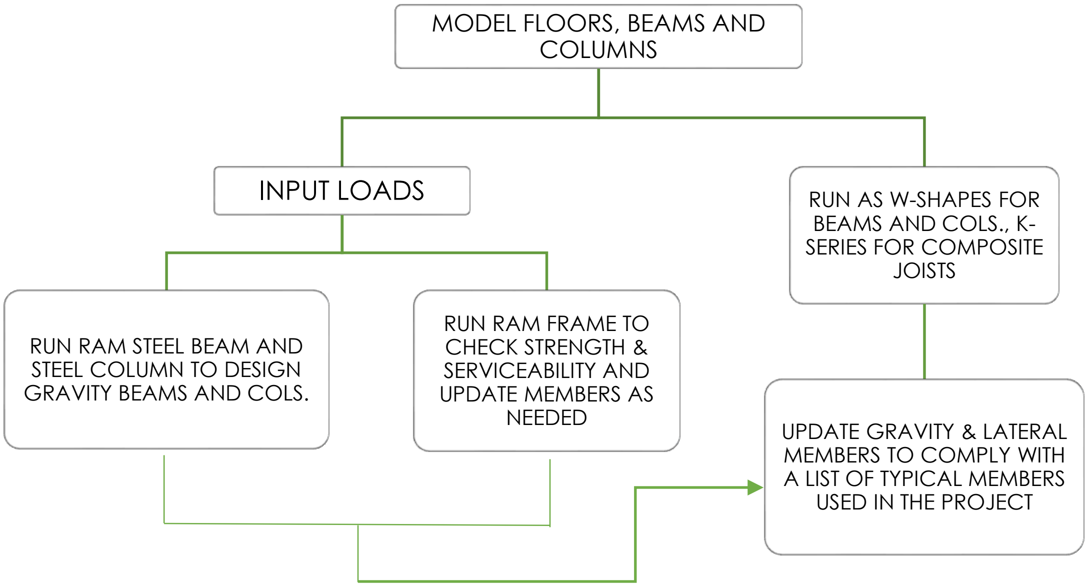
VI. P-Delta effects were taken into account.

VII. Load combinations were generated using IBC 2012/ASCE 7 -10.

VIII. Hand calculations were made for areas that required special attention.

IX. All members were updated to create a list of "typical" members for the projects for the ease of construction.

OUTLINE OF MODELING PROCESS



LATERAL FORCE RESISTING SYSTEM – MOMENT FRAME DESIGN

The steel moment frames layout as presented earlier in the report can be seen in the figures below. More specifically, Figure 23 shows a 3-Dimensional RAM model view of the moment frames incorporated with the entire structural system while Figure 24 shows a 3-Dimensional view of only the lateral system (moment frames).

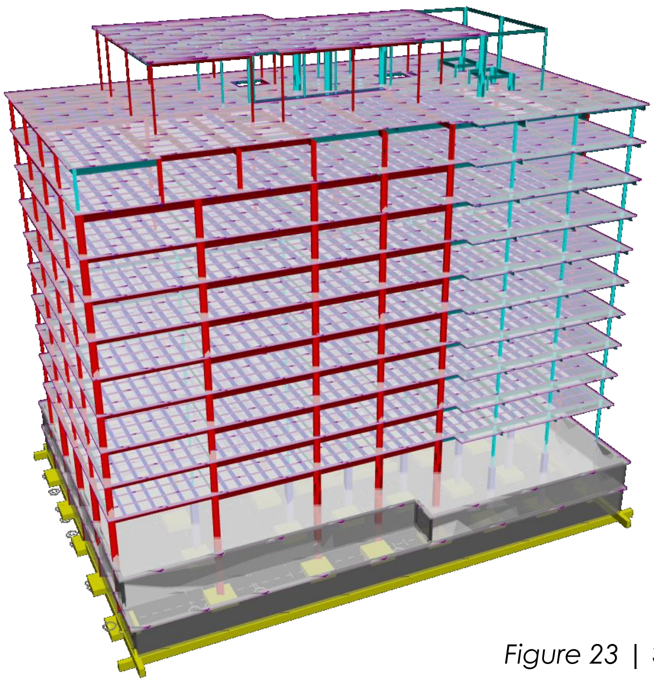


Figure 23 | **3-Dimensional View Of New Design**

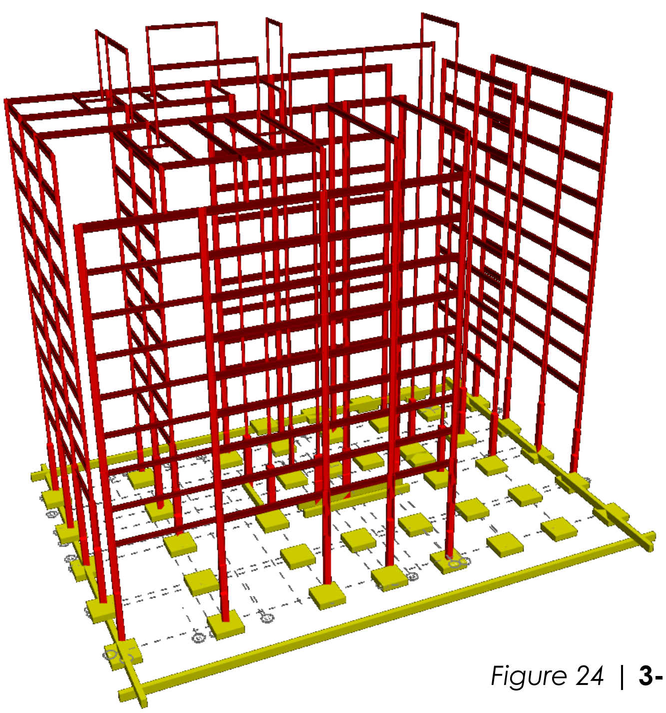


Figure 24 | **3-Dimensional View Of Moment Frame Layout**

As mentioned in the Modeling Process section, some of the gravity members on the main roof level needed to be changed to lateral members. This was performed to allow the frame members on the penthouse carry load to the foundation level. Furthermore, the bases of those frames are designed as pinned, to take only shear and not moment. Controlling the drift was a major key in this design.

The use of shear walls was also examined. However, that would have meant introducing walls into the existing building, at the stair and the elevators. Additionally, it would have involved cutting slabs, forming walls, doweling to the existing slabs and adding huge footings.

LATERAL SYSTEM CHECK – MOMENT FRAMES DESIGN CHECK

A number of checks were completed to check the efficiency and adequacy of the lateral force resisting elements as designed by RAM. The table below illustrates a summary of checks completed, with additional comments as needed.

TABLE 3: MOMENT FRAMES DESIGN CHECK

CHECK	REMARK	RESULT
STORY DRIFTS	THE ALLOWABLE STORY DRIFTS ARE MET FOR ALL LEVELS IN THE TWO ORTHOGONAL DIRECTIONS.	GOOD
TORSION	ACCIDDENTAL TORSION = 5%.	GOOD
MEMBER CHECKS	SOME OVERDESIGN BY THE RAM MODEL. CORRECTED AND VERIFIED USING HAND CALCULATIONS	GOOD
RAM MODAL PERIOD	__	GOOD
REDUNDANCY	__	GOOD

STORY DRIFTS

The tables below show the story drifts based on the wind loads that controlled the lateral system design in the RAM Structural System model. This allowable story drift is *h/240* for the overall and inter-story drifts. Case 1 wind (W1) controlled, with a factor of 1.0 used (***1.0W1***)

TABLE 4: N - S DIRECTION (STEEL MOMENT FRAMES) - H/240 LIMIT				
STORY	H_x (ft.)	STORY DRIFT	ALLOWABLE DRIFT	CHECK
P.H. ROOF	18.5	0.25	0.93	OK
MAIN ROOF	10.75	0.24	0.54	OK
TENTH FLOOR	10.84	0.23	0.54	OK
NINTH FLOOR	10.75	0.27	0.54	OK
EIGHTH FLOOR	10.33	0.3	0.52	OK
SEVENTH FLOOR	10.33	0.34	0.52	OK
SIXTH FLOOR	10.33	0.38	0.52	OK
FIFTH FLOOR	10.33	0.39	0.52	OK
FOURTH FLOOR	10.33	0.43	0.52	OK
THIRD FLOOR	10.33	0.48	0.52	OK
SECOND FLOOR	15	0.68	0.75	OK

TABLE 5: E - W DIRECTION (STEEL MOMENT FRAMES) - H/240 LIMIT				
STORY	H_x (ft.)	STORY DRIFT	ALLOWABLE DRIFT	CHECK
P.H. ROOF	18.5	0.22	0.93	OK
MAIN ROOF	10.75	0.19	0.54	OK
TENTH FLOOR	10.84	0.25	0.54	OK
NINTH FLOOR	10.75	0.31	0.54	OK
EIGHTH FLOOR	10.33	0.34	0.52	OK
SEVENTH FLOOR	10.33	0.38	0.52	OK
SIXTH FLOOR	10.33	0.43	0.52	OK
FIFTH FLOOR	10.33	0.44	0.52	OK
FOURTH FLOOR	10.33	0.46	0.52	OK
THIRD FLOOR	10.33	0.47	0.52	OK
SECOND FLOOR	15	0.64	0.75	OK

TORSIONAL EFFECTS

Diaphragms that are not modeled as flexible are required to account for inherent and accidental torsion, as per ASCE 7-10 Sections 12.8.4.1 and 12.8.4.2. All diaphragms were assumed rigid, with a G = 0.85.

INHERENT TORSION

The lateral forces are applied to the centers of mass (COM) and centers of rigidity (COR) on each level and are calculated in the RAM model. The RAM model automatically accounts for the inherent torsion, with the associated wind load cases also taken into account. The accuracies of the COM's and COR's were verified and documented in Table 9 below

TABLE 6: CENTERS OF MASS (COM) & CENTERS OF RIGIDITY (COR)				
LEVEL	**CENTERS OF RIGIDITY**		**CENTERS OF MASS**	
	X_r	Y_r	X_m	Y_m
P.ROOF	80.88	37.60	69.69	43.84
ROOF	78.43	44.66	80.30	41.95
10TH	78.33	42.24	79.83	41.37
9TH	78.19	42.12	79.82	41.40
8TH	78.00	42.13	79.59	41.43
7TH	77.75	42.11	79.60	41.39
6TH	77.39	42.03	79.60	41.39
5TH	76.83	41.87	79.60	41.39
4TH	75.90	41.55	79.58	41.39
3RD	74.37	40.84	79.56	41.40
2ND	71.51	39.14	79.23	41.46
GROUND	79.51	44.82	79.71	44.82
PARKING LEVEL 1	78.65	42.93	78.65	42.93

ACCIDENTAL TORSION

The calculations for accidental torsion are not required and hence neglected as the seismic loads do not control the design.

NEW STRUCTURAL FLOOR PLAN LAYOUTS & MOMENT FRAME ELEVATIONS

STRUCTURAL FLOOR PLAN LAYOUTS

A typical floor structural plan and a moment frame elevations are shown in the Figures below and on the following page. Member sizes are labeled and moment frame locations are highlighted in red. The typical sizes used are 12" deep *ECOSPAN* composite joists (highlighted in blue) with W14 beams used on column lines. However, the sizes differ in the moment frame locations and in other areas that required special framing. The detailed floor structural plans, along with the other moment frame elevations are included in Appendix A.

MOMENT FRAME 1 – ALONG COLUMN LINE 1 & 9

MOMENT FRAME 2 – ALONG COLUMN LINE 2.5 & 8

MOMENT FRAME 3 – ALONG COLUMN LINE A

MOMENT FRAME 4 – ALONG COLUMN LINE E

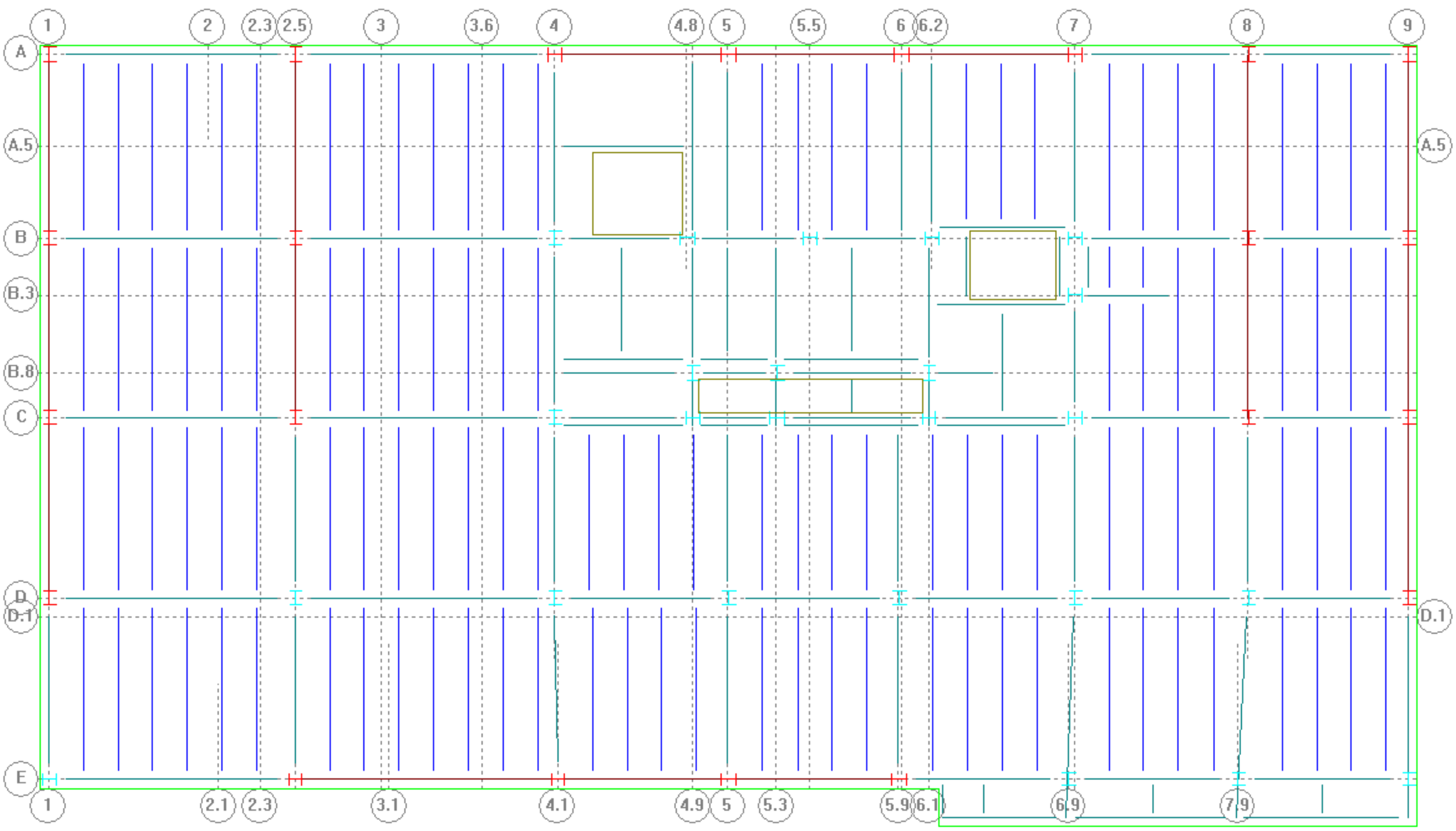
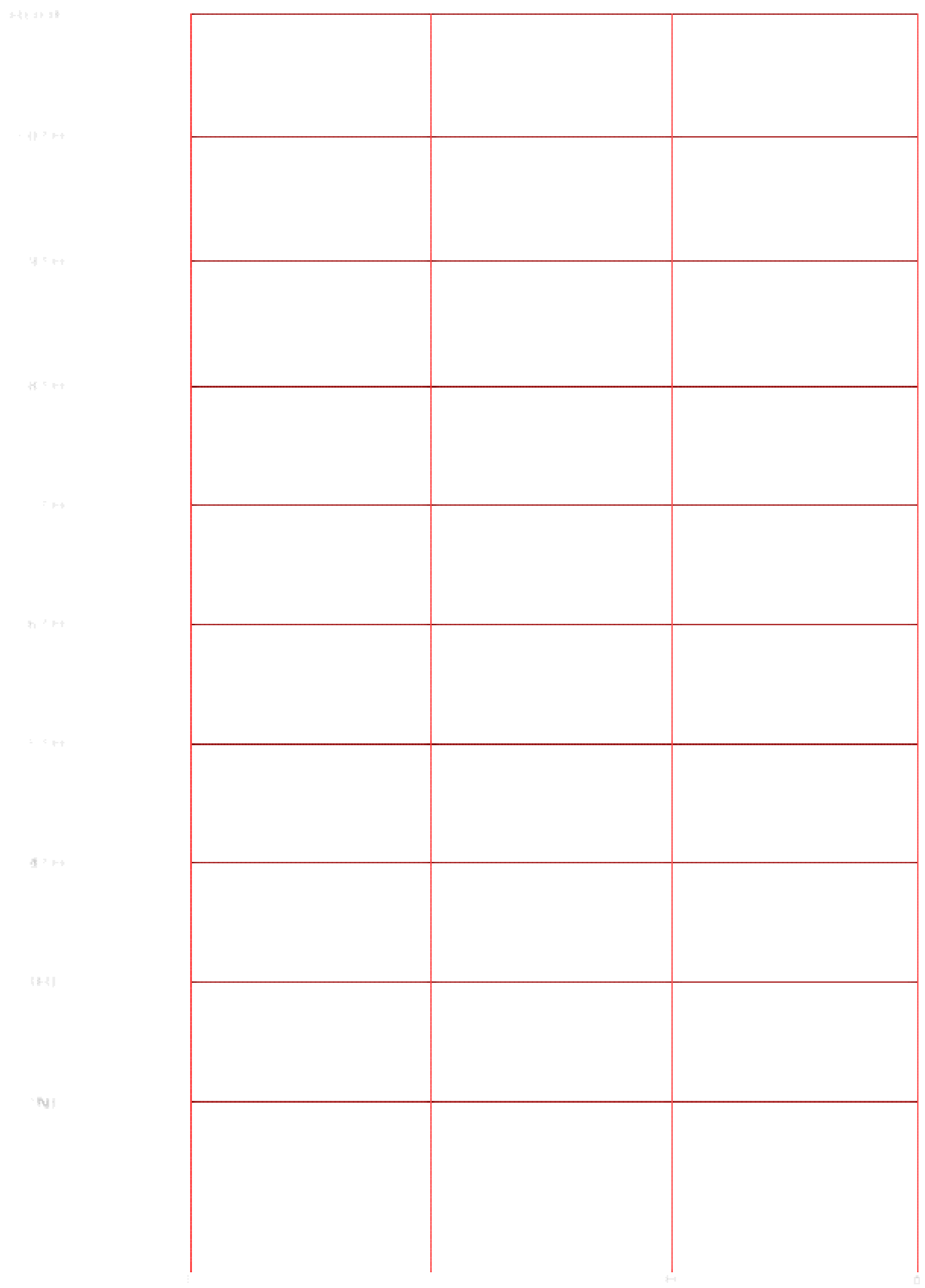


Figure 25 | **Typical Structural Floor Plan Layout**

JOISTS – 12" DEEP *ECOSPAN* JOISTS BEAMS – W12X26 (TYP.) COLUMNS – W21X93 (TYP.)

MOMENT FRAME BEAMS ARE W24X94 AND COLUMNS ARE W21X93



COLUMNS – W21X93 **BEAMS – W24X94**

Figure 26 | **Moment Frame 1 Elevation**

STANDARD JOISTS DETAILS

Standard connections are addressed below. They were taken from the standard details webpage of *ECOSPAN* Composite Floor System.

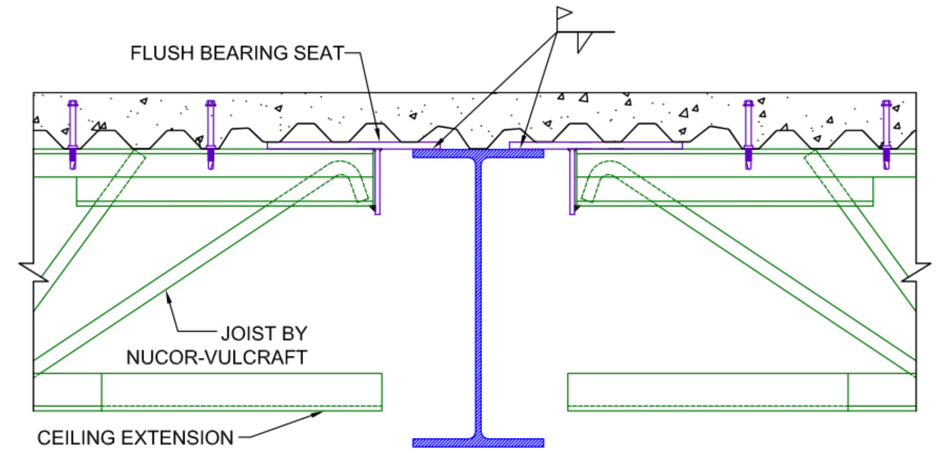


Figure 27 | **Flushing Bearing Seat on Beam**

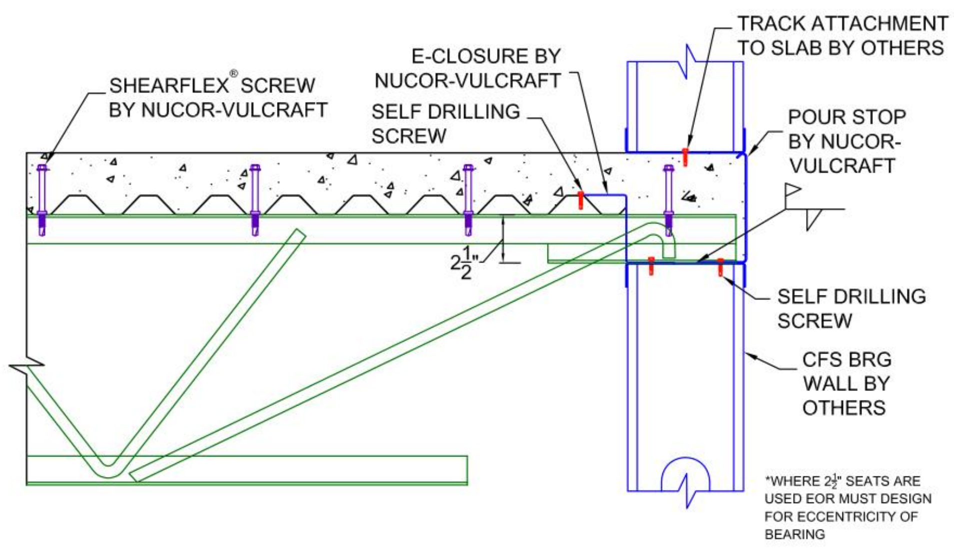


Figure 28 | **Standard 2 ½" Seat w/Full Bearing On CFS**

BREADTH #1 – USING SOLAR THERMAL ENERGY TO PREHEAT VENTILATION AIR

The penthouse is taken up entirely by the mechanical system in the original design. This is purposefully done so the building has a Dedicated Outdoor Air System (DOAS) with 100% fresh air to provide improved outdoor air supply and better ventilation. With that said, a mechanical breadth will be explored which involves the use of solar thermal panels as collectors to preheat outdoor ventilation air. This technology uses solar energy to preheat outdoor air when the building is in heating mode, which helps cut down overall heating costs in the building and saves energy by reducing the load on the building's heating system.

The solar thermal preheating system involves the use of the following components:

I. SOLAR COLLECTORS
II. PUMP
III. PIPING SYSTEM
IV. CONTROLLER
V. WATER TO AIR HEAT EXCHANGER ADDED TO THE AIR HANDLING UNIT (AHU)

DESIGN GOALS

- Create a mechanism that allows intake air to be collected by solar collectors and transferred into the building

- Design the transpired collectors to be mounted on the roof

DESIGN SOLUTION

USING SOLAR ENERGY TO HEAT INTAKE AIR THORUGH TRANSPIRED COLLECTORS

The transpired collector mechanism is relatively straightforward. The collector is usually a dark colored, perforated metal wall for maximum solar radiation and is usually installed on the walls that will receive the maximum exposure to sunlight in all seasons. Figure 29 shows an example of a transpired collector and how it operates. The transpired collectors preheat the ventilation (outdoor) air by using the building's ventilation fan to draw air through the perforated wall and up the plenum, and then into the building. This system allows air to be preheated by as much as 35 degrees Fahrenheit.

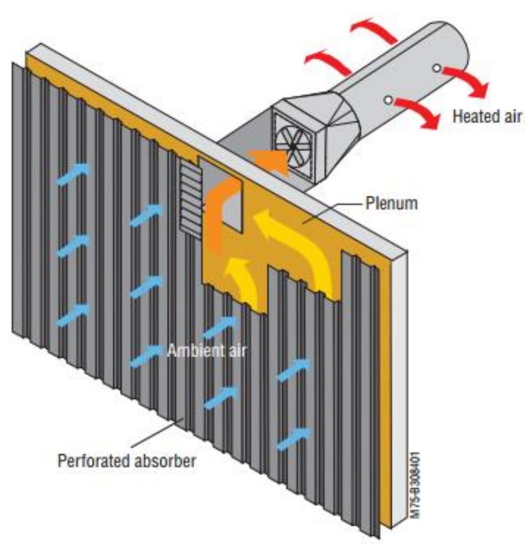


Figure 29 | **Transpired Air Collectors**

APPLICATION TO 440 FIRST STREET

As mentioned earlier, the mechanical penthouse houses the entirety of the major mechanical equipment used in the building. The schematic of this system makes it possible for this technology to be used, however, the use of this technology is significantly different for this project.

Solar thermal collectors will be mounted on the roof with a tilt angle equal to the site latitude + 10 degrees. A pump is connected to the system, which pumps water into the solar collector at a certain inlet fluid temperature (T_I) and leaves the collectors at an outlet temperature (T_O). A controller is installed as part of the system and records the outlet fluid temperature from the collectors. The controller activates the pump when the outlet fluid temperature is greater than the ventilation (outdoor) air temperature, that is when $T_I > T_a$. Hence, the pump only operates when the building is in heating mode. This system does not require a thermal storage tank and uses anti-freeze fluid for the pump, which makes the technology relatively simple. A schematic of the system, Figure 30, and how it operates is shown in the diagram below.

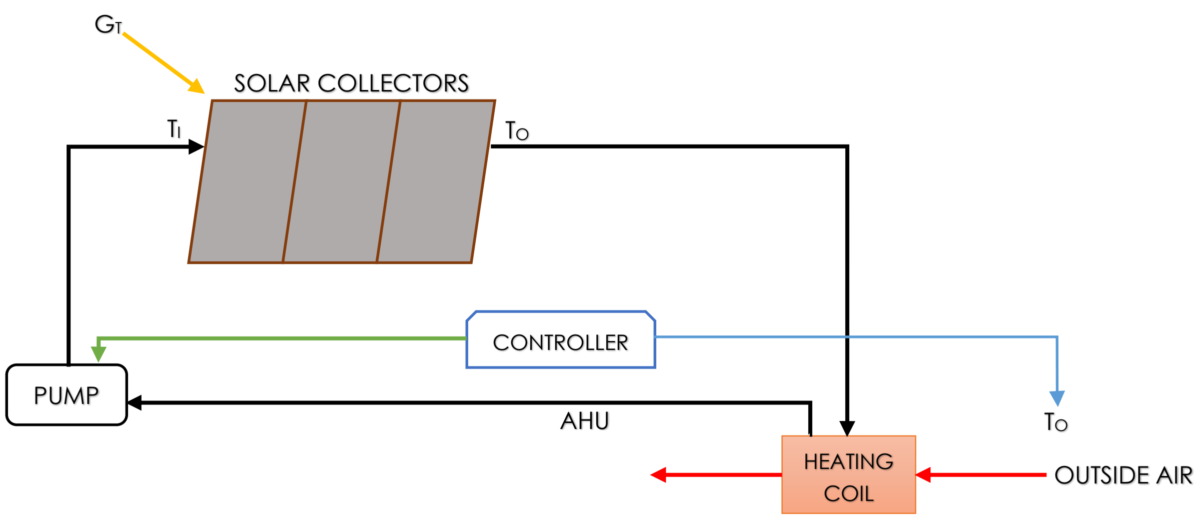


Figure 30 | **Schematic of Solar Collector System**

The useful solar gain, *Qu*, is calculated with respect to the area of the collector, *Ac*, and other factors. The equation for calculating the useful solar gain is:

$$Q_U = A_C\left[G_T.F_R(tr) - F_RU_L\left(T_I - T_a\right)\right]$$

Where Gt – **SOLAR IRRADIANCE (*w/m²*)** – *Use typical values i.e. 100 – 1000*

Tf – **INLET FLUID TEMP TO COLLECTORS** – *Assume 30 degrees Celsius*

To – **OUTLET FLUID TEMPERATURE FROM COLLECTORS**

F_RU_L – 0.83 **$F_R(tr)$ – 6.3 *w/m².c***

The useful solar gain per square meter of collector was calculated, changing the solar irradiance *(200, 400, 600, and 800)* and outdoor air temperature *(-10, -5, 0, 5, 10, 15)* values. Table 7 and the graph below show the relationship between the three variables.

TABLE 7: SOLAR IRRADIANCE X OUTSOOR AIR TEMP. X USEFUL SOLAR GAIN

SOLAR IRRADIANCE	OUTDOOR AIR TEMP.	USEFUL SOLAR GAIN
200	-10	-86
200	-5	-54.5
200	0	-23
200	5	8.5
200	10	40
200	15	71.5
400	-10	80
400	-5	111.5
400	0	143
400	5	174.5
400	10	206
400	15	237.5
600	-10	246
600	-5	277.5
600	0	309
600	5	340.5
600	10	372
600	15	403.5
800	-10	412
800	-5	443.5
800	0	475
800	5	506.5
800	10	538
800	15	569.5

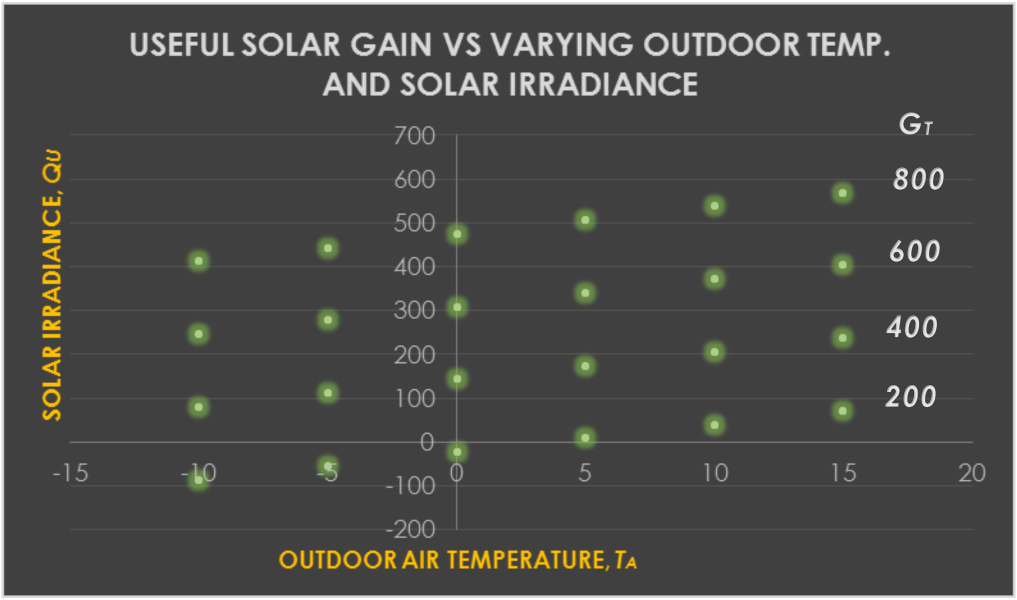


Figure 31 | **Graph of Solar Irradiance vs Outdoor Air Temperature**

As seen in the graph, the higher the outdoor temperatures and the solar irradiances are, the more useful solar gain the collectors receive. Furthermore, increasing the area of the solar collectors will yield in a larger useful solar gain as well.

BREADTH #2 – COST AND SCHEDULE ANALYSIS

The original building for 440 First Street was initially built in the 80s and has been since renovated began in 2012. The overall renovation schedule lasts from March 2012 – April 2013, which is 14 months in duration.

The total cost of the renovation is $20,000,000 and the structural cost is roughly about $2,582,000, which is about 13% of the overall renovation budget.

DESIGN GOALS

- Reduce the structural construction cost thus reducing the overall cost of the building
- Decrease the construction schedule of the structural system

COST ANALYSIS

Detailed quantity takeoffs were completed for the different structural elements used in the new design to determine its effect on the overall cost of the building. These costs are tabulated below and a more detailed structural cost breakdown can be seen in Appendix.

TABLE 8: ROUGH STRUCTURAL COST ESTIMATE

SUPERSTRUCTURE ESTIMATE	QTY	UNIT	LABOR RATE	MATERIAL RATE	EQUIP RATE	TOTAL RATE	TOTAL COST
FLOOR STRUCTURE							
STEEL BEAMS	167	TON	475	2750	131	3356	560452
ECOSPAN COMP. JOISTS	87	TON	875	175	76	1126	97962
STEEL COLUMNS	109	TON	425	850	131	1406	153254
METAL DECKING	142530	SF	2.24	0.45	0.04	2.73	389106.9
CONCRETE TOPPING	142530	SF	3.28	1.3	0.55	5.13	731178.9
STRUCTURE SUBTOTAL = 1,931,954							

Original Structural Cost: **$2,582,000**

New Structural Cost: **$1,931,954** (Rough Estimate), **$1,175,874** (Detailed Estimate)

Total Structural Savings: **$650,000** (Rough Estimate), **$1,406,126** (Detailed Estimate)

SCHEDULE ANALYSIS

The scheduling impact of the new design changes were also considered. The schedule below is based off of discussions with a representative in SIGAL Inc., and also Rathgeber/Goss Associates.

STRUCTURAL SCHEDULE PER FLOOR	
ITEM	DURATION IN DAYS
STEEL	7
CONCRETE	3
FABRICATION	75
DRAWING REVIEW	40
SHOP DRAWINGS	10

The durations are based on a floor per floor basis, excluding days for fabrication, drawing review and fabrication. The total structural duration for the new design is 95 days, with construction of each floor being on the critical path.

CONCLUSIONS AND RECOMMENDATIONS

The results of the redesign were compared to the established design goals in the report to evaluate the success of the redesign. Check marks indicate a successful design goal.

STRUCTURAL REDESIGN CONCLUSIONS

- ✓ REDESIGN THE ENTIRE BUILDING USING LIGHTWEIGHT STRUCTURAL STEEL
  - *The new design is a composite steel joist frame system (a very light-weight structural system) with non-composite beams, with steel moment frames as the lateral system.*

- ✓ PROVIDE A SOLUTION THAT DOES NOT INTERFERE WITH THE EXISTING ARCHITECTURAL DESIGN
  - *The architectural layout of this building was taken into full consideration due to the recent renovation that occurred in 2013. With that said, moment frames were used on the perimeters and along some interior column lines.*

- ✓ SHORTEN OVERALL CONSTRUCTION TIME BY CUTTING THE STRUCTURAL ERECTION SCHEDULE

USE OF SOLAR THERMAL COLLECTORS TO PREHEAT VENTILATION (OUTDOOR) AIR

- ✓ CREATE A MECHANISM THAT ALLOWS INTAKE AIR TO BE COLLECTED FROM THE SOLAR THERMAL COLLECTORS AND TRANSFERRED INTO THE BUILDING
  - *A schematic was created which involved mounting solar collectors on the roof and connecting to the necessary mechanical equipment in the mechanical penthouse.*

- ✓ DESIGN THE TRANSPIRED SOLAR THERMAL COLLECTORS TO BE MOUNTED ON THE ROOF
  - *Collectors were mounted on the roof with a tilt angle equal to the site latitude plus 10 degrees.*

Based on these design goals and criteria, the solar thermal collector addition was a success and will help reduce the energy requirements of the building.

COST AND SCHEDULE ANALYSIS

- ✓ REDUCE THE STRUCTURAL CONSTRUCTION COST THUS REDUCING THE OVERALL COST OF THE BUILDING
  - *The overall structural cost reduced from $2,582,000 to $1,175,874. This is approximately a 54% decrease in the cost. In a grander scheme, the new overall budget is now $18,593,874, which implies a 6% structural cost percentage of the overall budget.*

CODE AND USEFUL DOCUMENT ASSESSMENT

The following documents were used in the preparation of this report:

- ACI 318 – 11 *Building Code Requirements for Structural Concrete* by the American Concrete Institute.

- ASCE 7 – 10 *Minimum Design Loads for Buildings and Other Structures* by the American Society of Civil Engineers.

- AISC 13th Edition *Steel Construction Manual* by the American Institute of Steel Construction, Inc.

- IBC 2012 *International Building Code by the International Code Council, Inc*

- AE CLASS NOTES

- *ECOSPAN* Composite Steel Joist Design Guide

- First Edition *Standard Specifications for Composite Steel Joists* by the Steel Joist Institute (SJI)

- Solar Thermal Resources

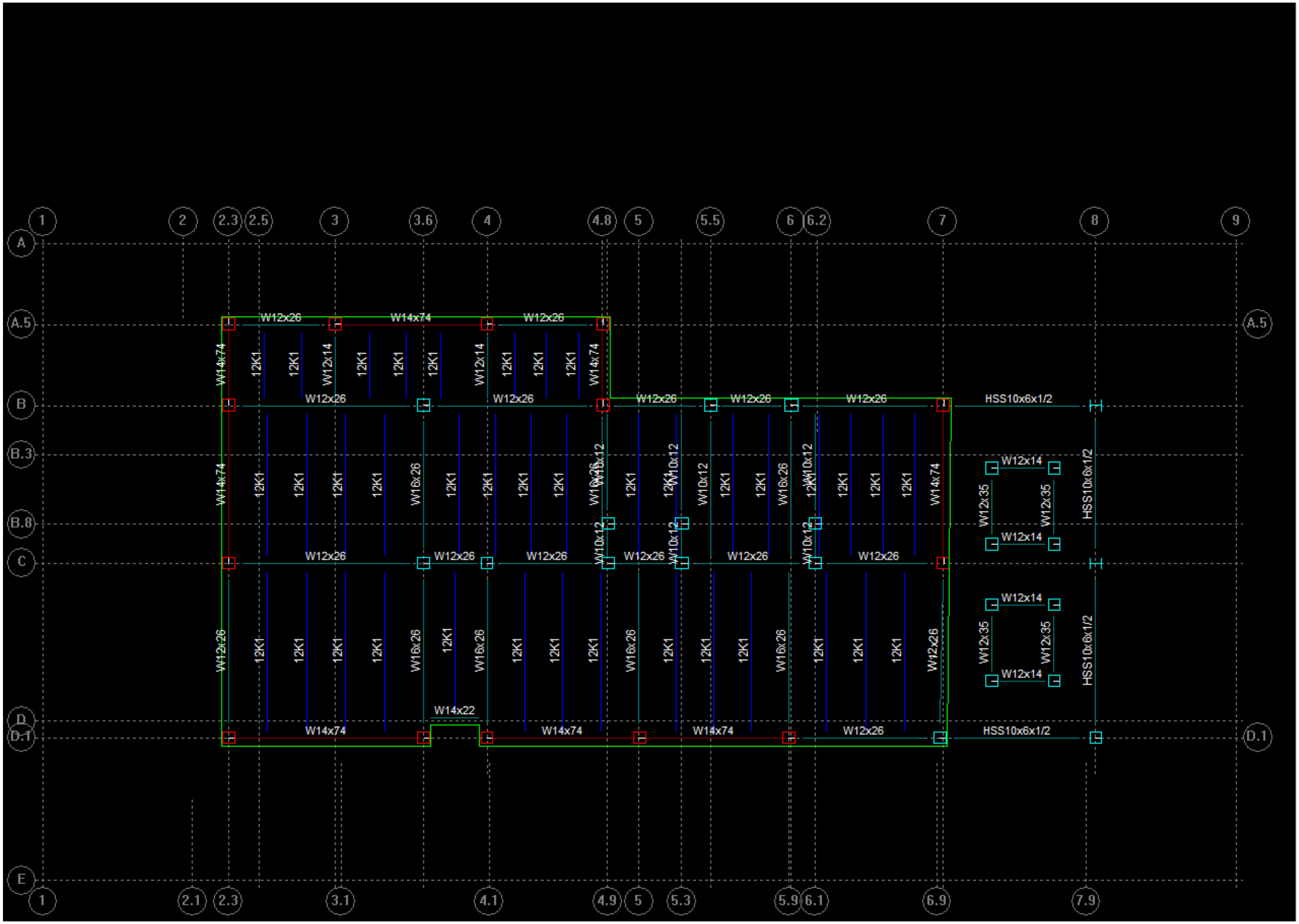
- 2016 RSMeans *Assemblies Cost Data*

- 2016 RSMeans Building Construction Cost Data

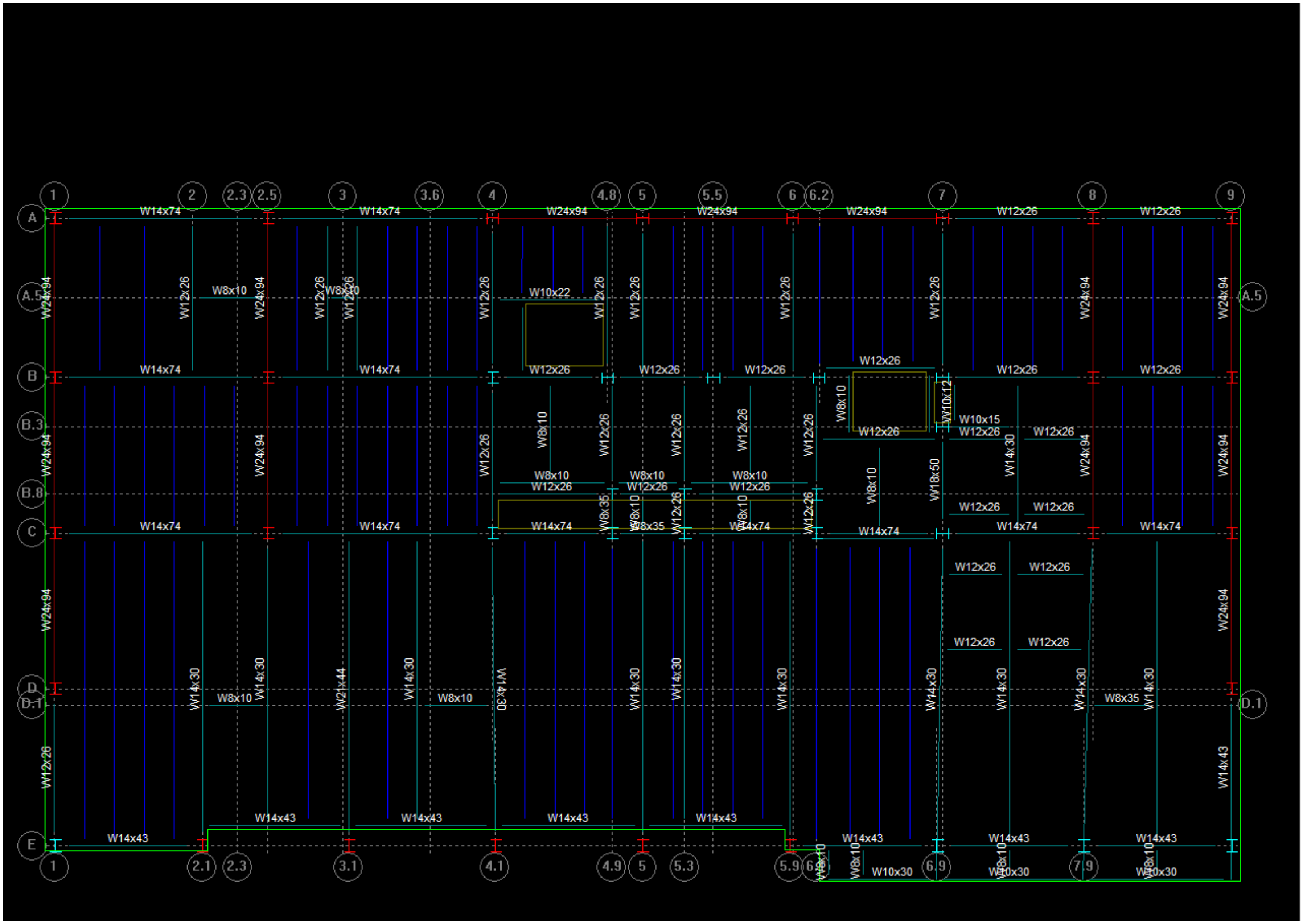
APPENDIXES

APPENDIX A

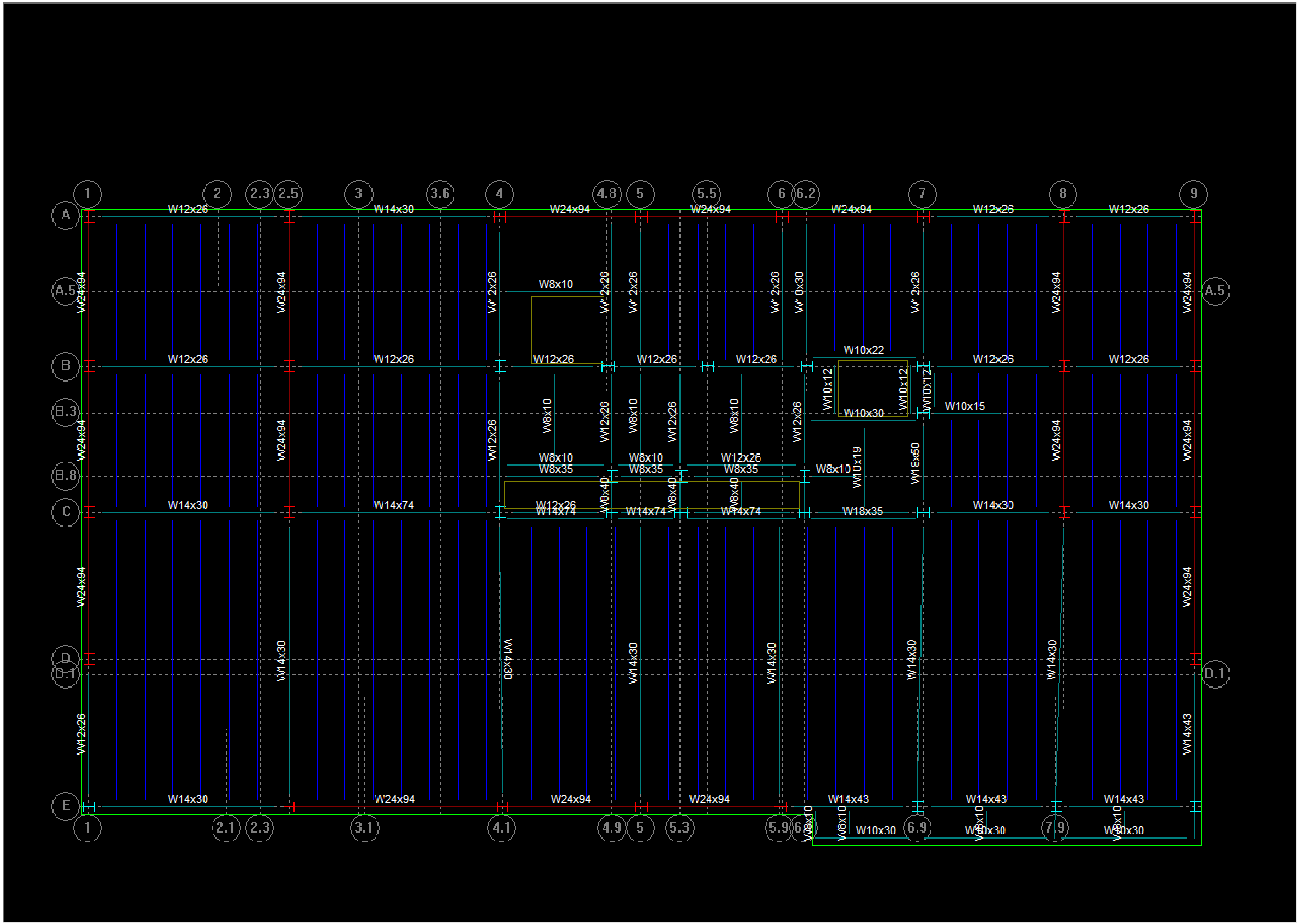
DETAILED FLOOR PLANS AND ELEVATIONS



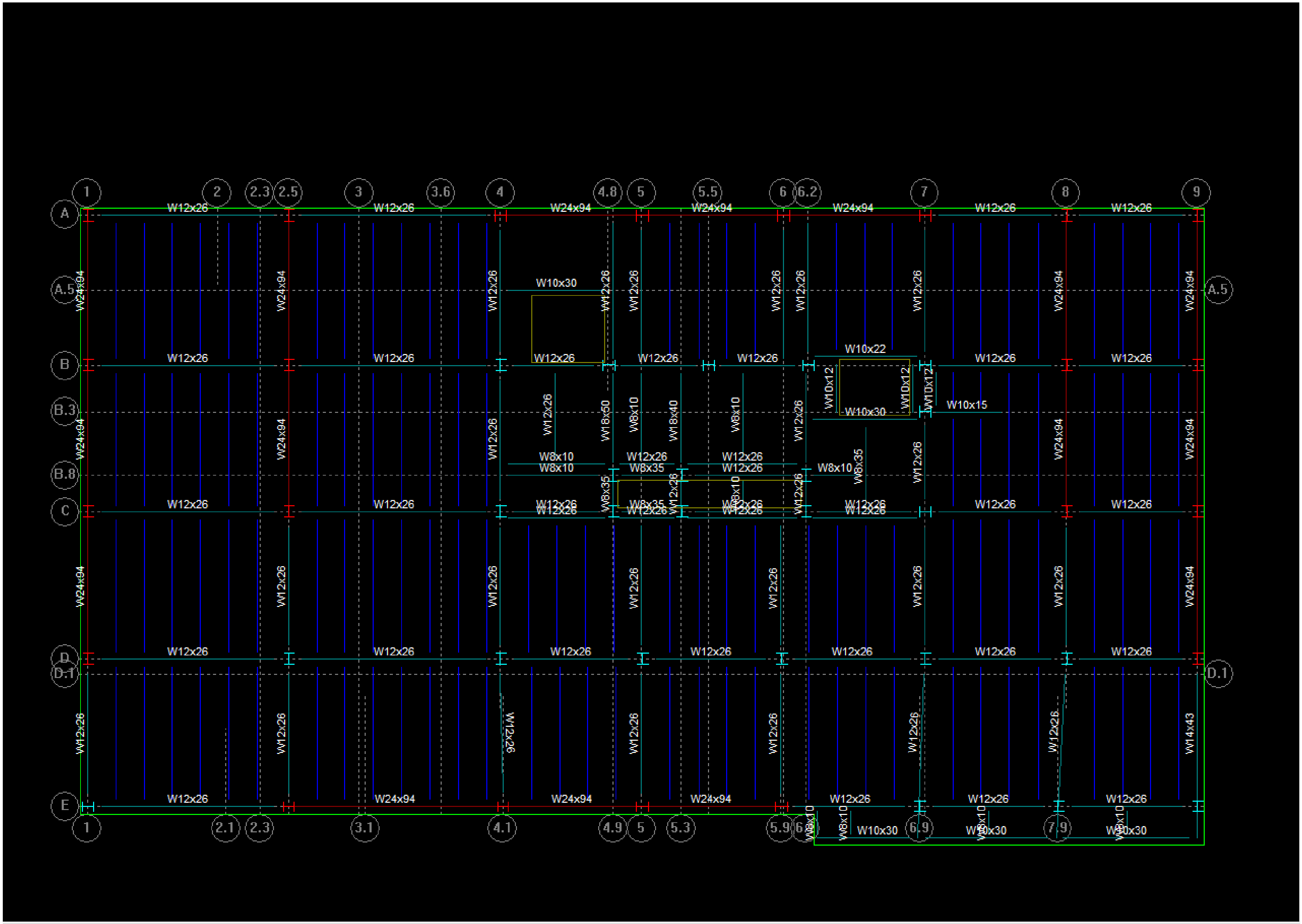
PENTHOUSE ROOF STRUCTUAL FLOOR PLAN



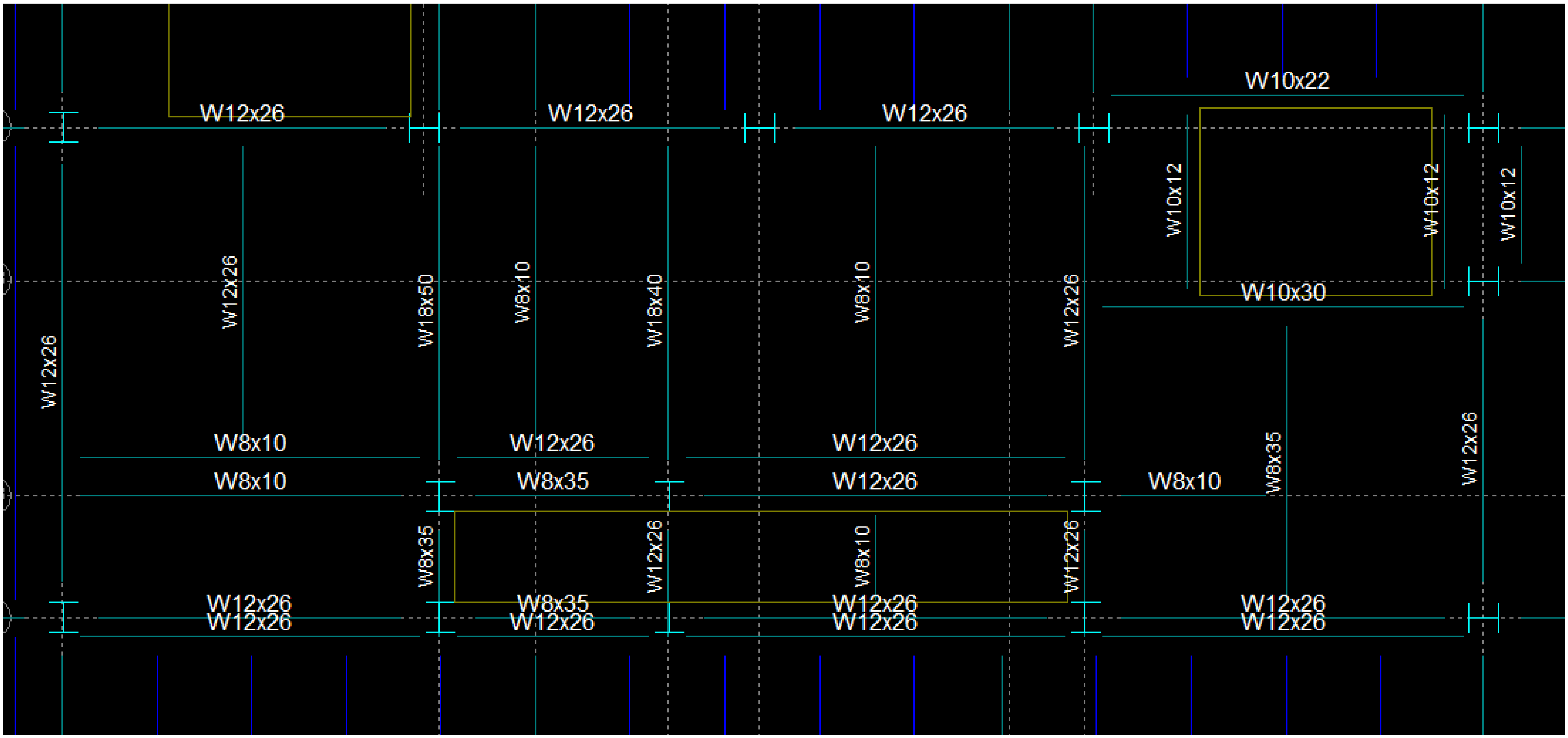
MAIN ROOF/ PENTHOUSE STRUCTUAL FLOOR PLAN



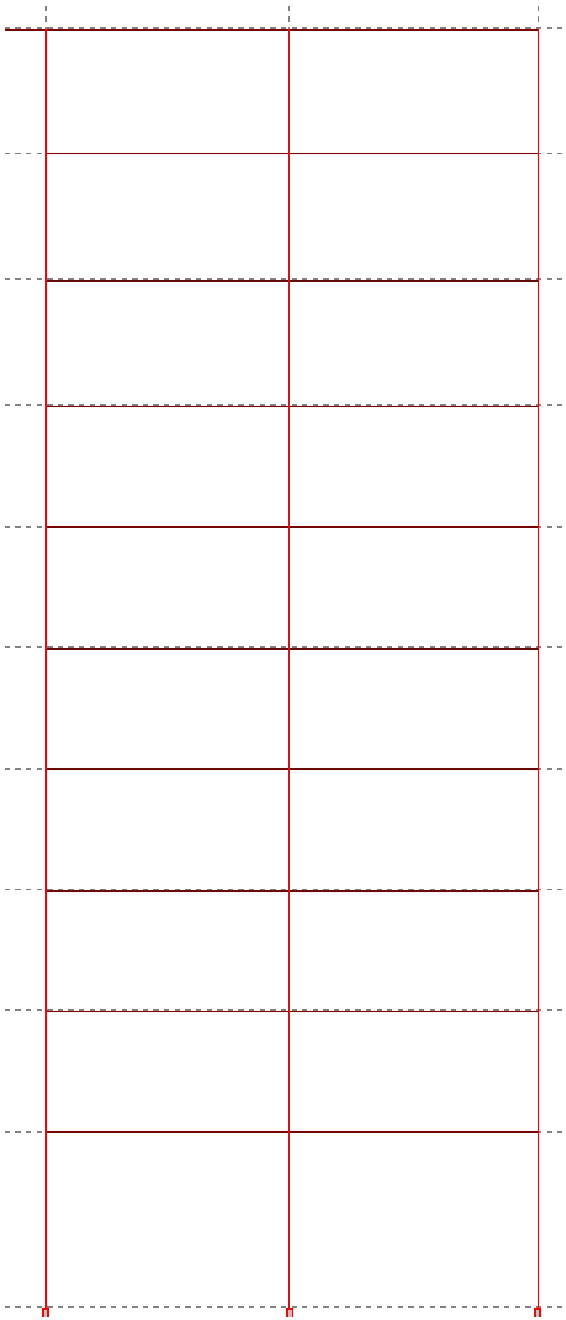
9TH & 10TH STRUCTUAL FLOOR PLAN



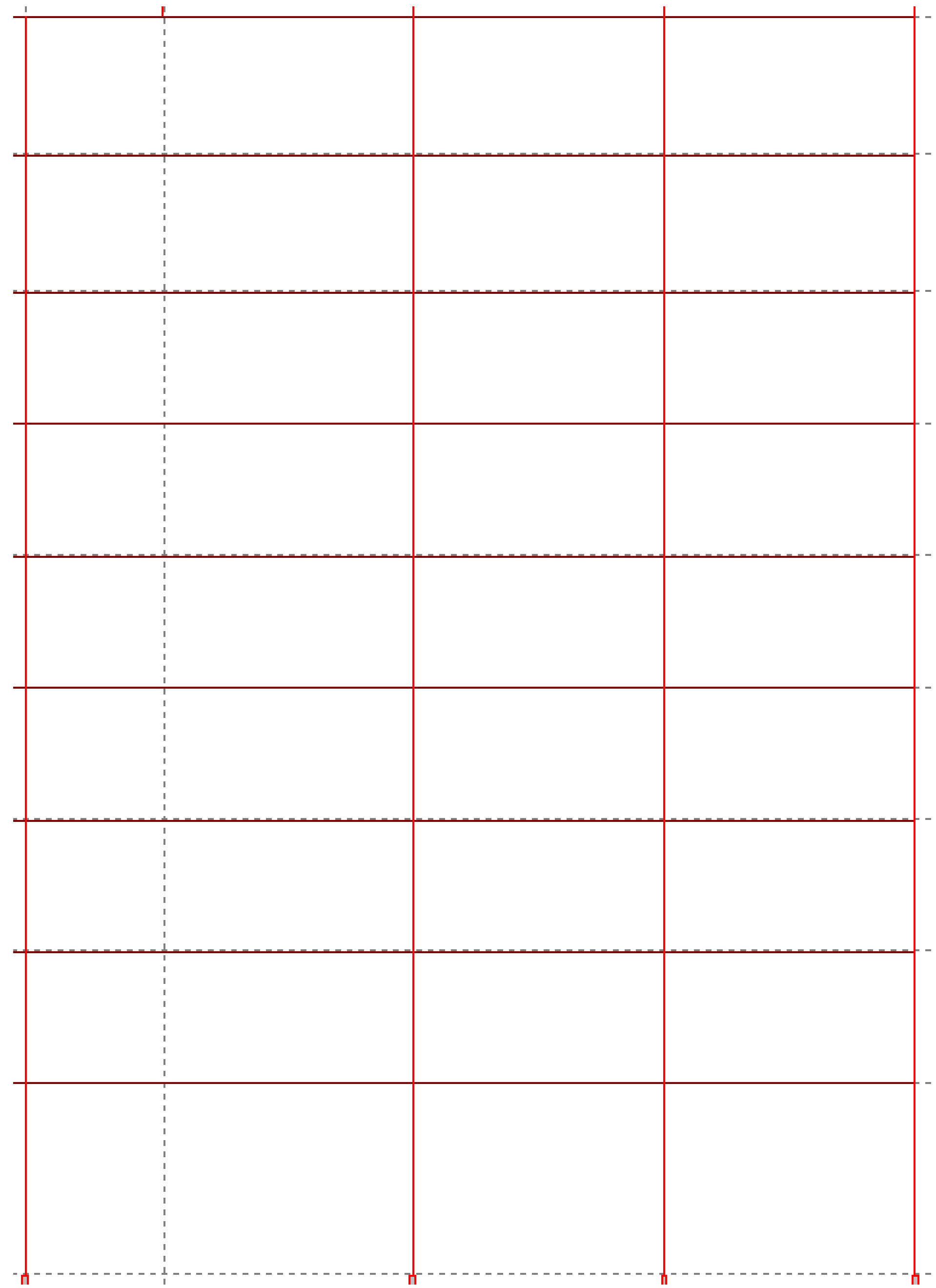
TYPICAL STRUCTUAL FLOOR PLAN (FLOORS 2 – 8)



MAGNIFIED VIEW OF INTERIOR BAY FRAMING



MOMENT FRAME 2

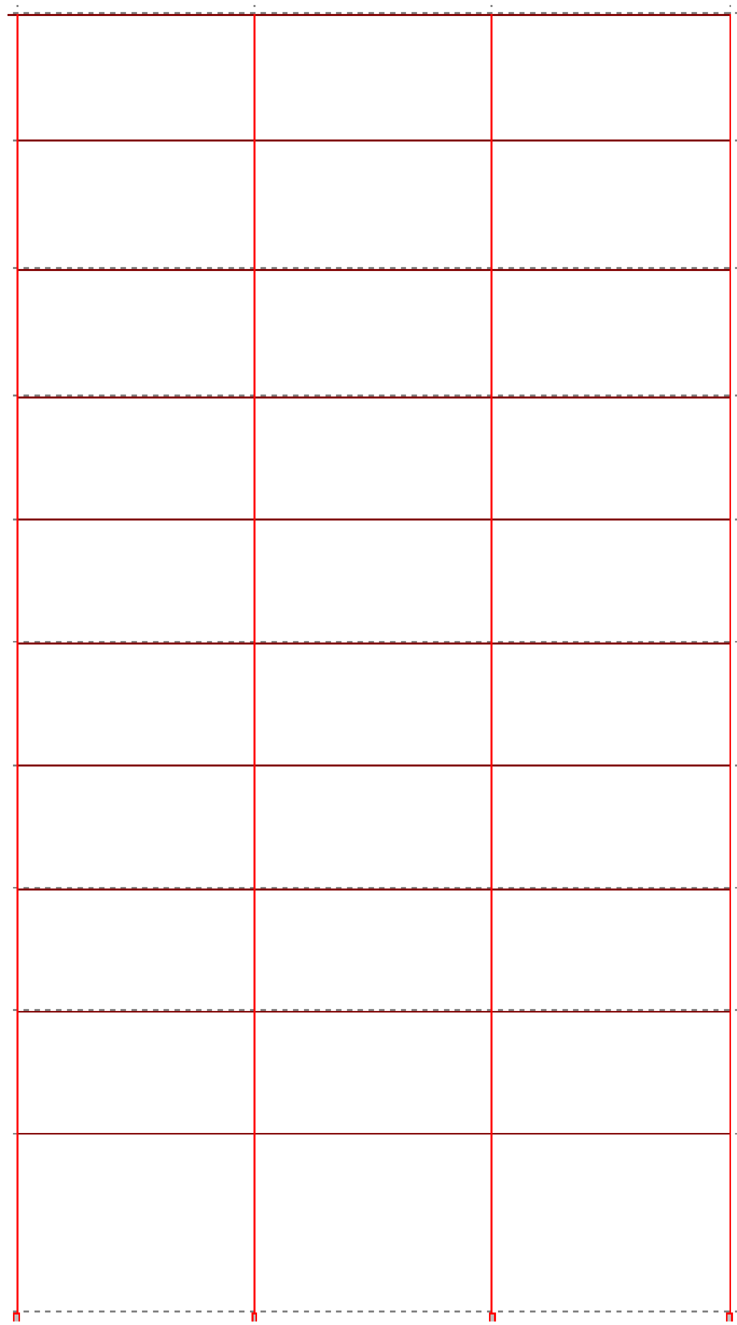


MOMENT FAME 4

MOMENT FRAMES

BEAMS - W24X94

COLUMNS - W21X93



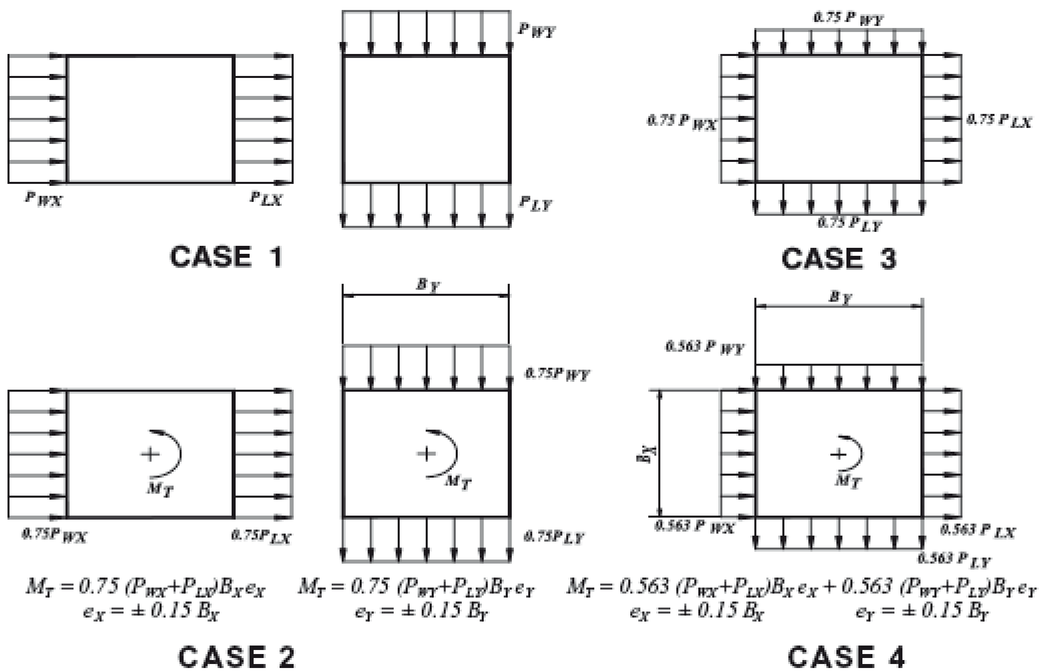
MOMENT FRAME 3

APPENDIX B

LATERAL LOAD CALCULATIONS

WIND LOAD CALCULATIONS

The load cases below were considered for the wind loading for the structure. They were extracted from ASCE 7-10 Figure 27.4-8.



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

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

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

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

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

WIND PRESSURES & FORCES IN NORTH – SOUTH DIRECTION

WIND PRESSURES (N - S)					
HEIGHT (FT.)	**Kz**	**qz**	**WINDWARD WALL (PSF)**	**LEEWARD WALL (PSF)**	**TOTAL (PSF)**
127.25	1.06	30.50	20.74	-8.43	**29.17**
109.25	1.01	29.07	19.7	-8.43	**28.13**
98.5	0.99	28.49	19.4	-8.43	**27.83**
87.75	0.95	27.34	18.6	-8.43	**27.03**
77	0.92	26.48	18	-8.43	**26.43**
66.67	0.88	25.32	17.2	-8.43	**25.63**
56.33	0.84	24.17	16.4	-8.43	**24.83**
46	0.79	22.73	15.4	-8.43	**23.83**
35.67	0.73	21.01	14.2	-8.43	**22.63**
25.33	0.66	18.99	12.9	-8.43	**21.33**
15	0.57	16.40	11.2	-8.43	**19.63**

SUMMARY (N - S)				
STORY	**HEIGHT (FT.)**	**FORCE (K)**	**SHEAR (K)**	**MOMENT (FT-K)**
PHR	127.25	45.68	0	5812.78
MR	109.25	25.28	45.96	2761.84
10	98.5	25.01	71.24	2463.49
9	87.75	24.29	96.25	2131.45
8	77	23.75	120.54	1828.75
7	66.67	23.03	144.29	1535.41
6	56.33	22.31	167.32	1256.72
5	46	21.42	189.63	985.32
4	35.67	20.34	211.05	725.53
3	25.33	19.17	231.39	485.58
2	15	25.58	250.56	383.70
			276.14	**20370.56**

WIND PRESSURES & FORCES IN EAST-WEST DIRECTION

WIND PRESSURES (E - W)					
HEIGHT (FT.)	**Kz**	**qz**	**WINDWARD WALL (PSF)**	**LEEWARD WALL (PSF)**	**TOTAL (PSF)**
127.25	1.06	30.50	20.7	-12.9	**33.6**
109.25	1.01	29.07	19.7	-12.9	**32.6**
98.5	0.99	28.49	19.4	-12.9	**32.3**
87.75	0.95	27.34	18.6	-12.9	**31.5**
77	0.92	26.48	18	-12.9	**30.9**
66.67	0.88	25.32	17.2	-12.9	**30.1**
56.33	0.84	24.17	16.4	-12.9	**29.3**
46	0.79	22.73	15.4	-12.9	**28.3**
35.67	0.73	21.01	14.2	-12.9	**27.1**
25.33	0.66	18.99	12.9	-12.9	**25.8**
15	0.57	16.40	11.2	-12.9	**24.1**

SUMMARY (E - W)				
STORY	**HEIGHT (FT.)**	**FORCE (K)**	**SHEAR (K)**	**MOMENT (FT-K)**
PHR	127.25	96.92	0	12333.07
MR	109.25	53.9	96.92	5888.58
10	98.5	53.4	150.82	5259.90
9	87.75	52.1	204.22	4571.78
8	77	51.2	256.32	3942.40
7	66.67	49.83	307.52	3322.17
6	56.33	48.5	357.35	2732.01
5	46	46.85	405.85	2155.10
4	35.67	44.86	452.7	1600.16
3	25.33	42.71	497.56	1081.84
2	15	57.93	540.27	868.95
			598.2	**43755.94**

SEISMIC LOAD CALCULATIONS

Below are the summaries of the seismic load factors from ASCE 7-10 and their references.

FACTOR	REFERENCE
SITE CLASS C	11.4.2
SS – 0.154	11.4.1
S1 – 0.050	11.4.1
IMPORTANCE FACTOR – 1.0	TABLE 1.5.2
OCCUPANCY CATEGORY II	
SDS – 0.123	11.4.4
SD1 – 0.057	11.4.4
SDC A	11.6
RESPONSE MODIFICATION FACTOR – 3	12.2.3.1
SEISMIC RESPONSE COEFFICIENT – 0.078	12.8.1.1

VERTICAL DISTRIBUTION OF SEISMIC FORCES

LEVEL	HEIGHT	STORY WT.	FORCE	STORY SHEAR	MOMENT
PHR	127.25	410	4.1	0	521.73
MR	109.25	1140	11.4	4.1	1245.45
10	98.5	1140	11.4	15.5	1122.90
9	87.75	1140	11.4	26.9	1000.35
8	77	1140	11.4	38.3	877.80
7	66.67	1140	11.4	49.7	760.04
6	56.33	1140	11.4	61.1	642.16
5	46	1140	11.4	72.5	524.40
4	35.67	1140	11.4	83.9	406.64
3	25.33	1140	11.4	95.3	288.76
2	15	1140	11.4	106.7	171.00
				118.1	**7561.23**

APPENDIX C

PRELIMINARY HAND CALCULATIONS

DETERMINATION OF VALUES USED IN CALCULATION

Decking – (2" x 18 GA DECK, 4 1/2" LWC)

1. Span Check
SDI Max Unshored span = 13' – 1" > 10" ✓ GOOD!

2. Superimposed load check
W_{LL} + Superimposed DL ≤ Superimposed load
123 ≤ 170 ✓ GOOD!

Construction Live Load
Construction live load should be estimated as follows:

$L_c = 20R_1$, where $12 \le L_c \le 20$ PSF

$A_t = 20 \times 20 = 400\ ft^2$

$\therefore R_1 = 1.2 - 0.001A_t$ for $200\ ft^2 < A_t < 600\ ft^2$

$R_1 = 0.8$; $L_c = 20\ PSF \times 0.8 = 16\ PSF$

Composite Live Load

Reduced as per ASCE 7-10, 4.7.2

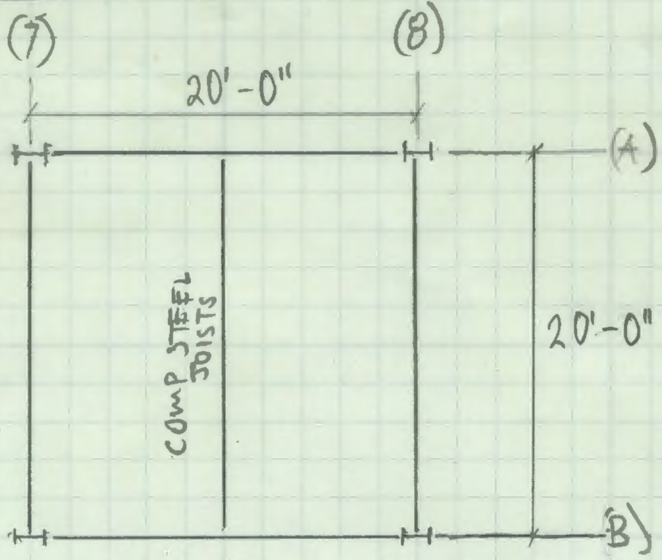
$L_0 = 100$ PSF ; $L = 100 \times \max\begin{cases} 0.5 \\ 0.25 + \dfrac{15}{\sqrt{2 \times 400}} = 0.78 \end{cases}$

$L = 100 \times 0.78 = 78$ PSF

ALTERNATIVE SYSTEM 2

STRUCTURAL STEEL FRAME W/ COMPOSITE JOISTS

TYPICAL BAY



ASSUMPTIONS AND CALCULATIONS

JOIST GEOMETRY

1. DEPTH — TO BE DETERMINED
2. SPAN — 20ft
3. ADJ. MEMBER SPACING — 10ft

CONCRETE AND DECK

1. TYPE OF FLOOR DECK — 2VLI18GA
2. DEPTH OF FLOOR DECK — 4.5"
3. SLAB THICKNESS ABOVE DECK — 2.5"
4. CONCRETE UNIT WT — 115 PCF
5. CONCRETE COMP. STRENGTH — 4 KSI

NOMINAL LOADS:

1. NON-COMPOSITE CONSTRUCTION DEAD LOAD
   a. Concrete + Deck — 35 PSF
   b. Joist and Bridging — 5 PSF (Assumed)

   40 PSF

   $\Rightarrow$ 40 PSF x 10' = 400 PLF

2. CONSTRUCTION LIVE LOAD [REDUCED]
   a) During Concrete Placement — 16 PSF

   $\Rightarrow$ 16 x 10 = 160 PLF

3. COMPOSITE DEAD LOAD

(a) MEP	15 PSF
(b) CEILING	5 PSF
(c) SPRINKLERS	3 PSF
	23 PSF

$\Rightarrow$ 23 X 10 = 230 PLF

4. COMPOSITE LIVE LOAD
(a) Live Load [Reduced] 78 PSF

$\Rightarrow$ 78 x 10 = 780 PLF

5. TOTAL FACTORED NON-COMPOSITE DEAD LOAD, 1.2 x (NCDL)
$\Rightarrow$ 1.2 x 40 = 48 PSF or 480 PLF

6. TOTAL FACTORED COMPOSITE DEAD LOAD, 1.2 x (CDL)
$\Rightarrow$ 1.2 x 23 = 27.6 PSF or 276 PLF

7. TOTAL FACTORED COMPOSITE LIVE LOAD, 1.6 x (CLL)
$\Rightarrow$ 1.6 x 78 = 124.8 PSF or 1248 PLF

8 TOTAL FACTORED COMPOSITE DESIGN LOAD,

$\Rightarrow$ 480 + 276 + 1248 = 2004 PLF

CAMBER AND DEFLECTION (Unfactored Load)

1. LOADS TO CAMBER FOR:

a) Non-composite Dead Load	40 x 100% = 40 PSF
b) Composite Dead Load	23 x 50% = 11.5 PSF
c) Composite Live Load	78 x 10% = 7.8 PSF

2. MAXIMUM ALLOWABLE LIVE LOAD DEFLECTION, L/360

$\Rightarrow$ (20 x 12) / 360 = 0.67 in

3. MAXIMUM DEFLECTION, L/240

$\Rightarrow$ (20 x 12) / 240 = 1 in

→ DETERMINE JOIST WT/FT, QUANTITY AND SIZE OF SHEAR STUDS, ANTICIPATED FLOOR DEFLECTIONS, NUMBER OF BRIDGING ROWS RQD AND MAXIMUM CIRCULAR DUCT OPENING

1). Assumed Joist Depth – $\boxed{14\text{in}}$

2). Joist Selection

* The proper joist shall be selected from the Design Guide LRFD Weight Table for Composite Steel Joists CJ-Series – LWC for a 20ft Joist, depth of 14in, a total factored composite design load of 2004 PLF, and a composite live load of 1248 PLF *

For 14in joist depth selected;

(a) W_t = 13.6 PLF
(b) W360 = 1384 PLF > 1248 PLF ✓ GOOD!
(c) N-ds = 16 — 5/8"

where N = Quantity of Shear Studs
ds = Types of Shear Studs

3. BRIDGING AND NOMINAL HORIZONTAL TOP CHORD FORCE (P_{br}) SELECTION

→ From the Design Guide LRFD Weight Bridging Table for Composite Steel Joists, CJ-Series – LWC

+ 1 row of horizontal bridging (1H) is required

Using Js = 10ft and Joist depth = 14in

Bridging Size – L 2.5 x 2.5 x 0.187 [Conservative Choice]
$\boxed{P_{br} = 750\text{ lbs}}$

4. Non-Composite Effective Moment of Inertia Selection

→ From the Design Guide LRFD Weight Bridging Table for Composite Steel Joists, CJ-Series – LWC

Using TL = 2000 PLF and Joist depth = 14in

$I_{non\text{-}comp\ eff} = 108\text{ in}^4$

DEFLECTION

$$\Delta_{NCDL} = \frac{5(W_{non\text{-}composite\,DL})(\text{Design Length})^4(1728)}{384 E_s I_{non\text{-}comp\,eff}}$$

where Design Length = Span − 4in
= 20 − 4/12 = 19.67ft

$$\Delta_{NCDL} = \frac{5(.400)(19.67)^4(1728)}{384(29000)(108)} = 0.43\text{in}$$

$$\Delta_{CDL} = \left[\frac{W_{COMP.DL}}{W_{360}}\right]\left[\frac{L}{360}\right] = \left[\frac{230}{1384}\right]\left[\frac{19.67\times12}{360}\right] = 0.11\text{in}$$

$$\Delta_{CLL} = \left[\frac{W_{COMP\,LL}}{W_{360}}\right]\left[\frac{L}{360}\right] = \left[\frac{780}{1384}\right]\left[\frac{19.67\times12}{360}\right] = 0.37\text{in}$$

$$\Delta_{TL} = \Delta_{Non\text{-}composite\,DL} + \Delta_{composite\,DL} + \Delta_{composite\,LL}$$

$$\Delta_{TL} = 0.43\text{in} + 0.11\text{in} + 0.37\text{in} = 0.91\text{in}$$

CAMBER

Camber joist for 100% × $\Delta_{NON\text{-}COMPOSITE\,DL}$ +
50% × $\Delta_{COMPOSITE\,DL}$ +
10% × $\Delta_{COMPOSITE\,LL}$

Joist Camber = 1.0 × 0.43 +
0.5 × 0.11 +
0.1 × 0.37
= 0.52in

EFFECTIVE MOMENT OF INERTIA SELECTION

→ From the Design Guide LRFD Weight Table for Composite Steel Joists, CJ-Series - LWC

Using TL = 2000PLF and 14in joist depth

→ $I_{eff} = 258\text{ in}^4$

Note: the published value of W360 takes into account the reductions in the effective transformed moment of inertia associated with web deformations and interfacial slippage. Hence, Ieff has been reduced by an assumed factor of 1.05 to account for these behaviors

⇒ $I_{e,\text{composite without slippage}} = 1.05\, I_{eff} = 1.05 \times 258\text{ in}^4 = 271\text{ in}^4$

Tech Report III | Yemi A. Ositelu

DESIGN SUMMARY

The Composite Steel Joist Used is $\boxed{\text{14 CJ 2004/1248/276}}$

Designations

14	—	Depth (in)
CJ	—	Composite Joist Series
2004	—	Total Factored Composite Design Load (PLF)
1268	—	Total Factored Composite Live Load (PLF)
276	—	Total Factored Composite Dead Load (PLF)

BRIDGING

Use 1 row of 2L's 2.5×2.5×0.187

JOIST WT = 13.6 PLF

DEFLECTIONS

$\Delta_{\text{Non-composite DL}} = 0.43\text{ in}$

$\Delta_{\text{composite DL}} = 0.11\text{ in}$

$\Delta_{\text{composite LL}} = 0.37\text{ in}$

CAMBER = 0.52 in

QUANTITY AND TYPE OF SHEAR STUDS

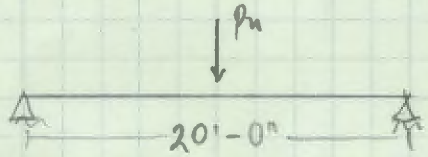
N — ds = 16 — 5/8"

GIRDER DESIGN (Interior)

(WEIGHT OF COMP. JOISTS)

Span = 20'-0" $DL = 35 + 23 + 13.6/20 = 58.6$ PSF
$LL = 78.0$ PSF [Reduced]

$P_u = [1.2(58.6) + 1.6(78)]\ 10 \times 20 = 39K$



$M_u = \dfrac{P_u L}{4} = \dfrac{39 \times 20}{4} = 195$ ftK

Check Bending
Using Zx tables (Table 3-2), $\boxed{\text{Try W16x31}}$

$\phi M_n = 203$ ftK $> M_u = 195$ ftK ✓ GOOD

Check Deflection , $I_x = 375$ in⁴

$\Delta_{LL} = \dfrac{0.78 \times 20 \times 20^3 \times 1728}{48 \times 29000 \times 375} = 0.41$ in $< \dfrac{L}{360} = 0.67$ in ✓

$\Delta_{DL,\,WET\ CONC} = \dfrac{0.357 \times 20 \times 20^3 \times 1728}{48 \times 29000 \times 375} = 0.19$ in $< \dfrac{L}{360} = 0.67$ in ✓

NO CAMBER

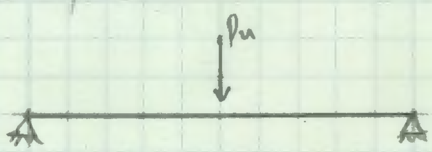
Exterior Girder

Span = 20'-0" DL = 58.6 PSF + Distributed Load (Wall)
LL = 100 PSF

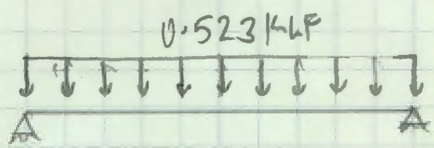
$P_u = [1.2(58.6) + 1.6(100)] \times 10 \times 10 = 23K$

$W_{u,wall} = 1.2(436\ PLF) = 0.523$ KLF

Super-position,



+



$M_u = \dfrac{23 \times 20}{4} + \dfrac{0.523 \times 20^2}{8}$

$M_u = 141.1$ ftK

Check Bending

Using Zx tables (Table 3-2), $\boxed{\text{Try W14X26}}$

$$\phi M_n = 151 \text{ ft k} > M_u = 141 \text{ ft k}$$

Check Deflection, $I_x = 245 \text{ in}^4$

$$\Delta_{LL} = \frac{P_{LL} l^3}{48EI} = \frac{.1 \times 10 \times 10 \times 20^3 \times 1728}{48 \times 29000 \times 245} = 0.4 \text{ in} < 0.67 \text{ in} \checkmark \text{ GOOD}$$

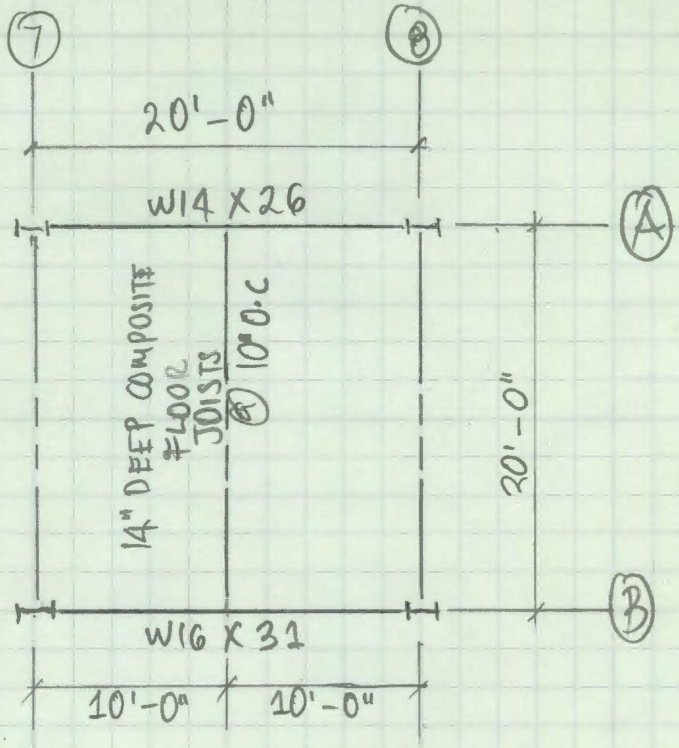
$$\Delta_{DL,\,wet\,conc} = \frac{P_{DL} l^3}{48EI} + \frac{5 w_{DL} l^4}{384EI}$$

$$= \frac{0.357 \times 10 \times 20^3 \times 1728}{48 \times 29000 \times 245} + \frac{5 \times 0.436 \times 20^4 \times 1728}{384 \times 29000 \times 245}$$

$$\Delta_{DL,\,wet\,conc} = 0.366 \text{ in} < 0.67 \text{ in} \checkmark \text{ GOOD!}$$

Hence, Interior Girder — W16 X 31
Exterior Girder — W14 X 26

DESIGN SUMMARY



APPENDIX D

STRUCTURAL COST INFORMATION

STRUCTURAL COST INFORMATION

ECOSPAN COMPOSITE JOISTS TAKEOFF				
SIZE	FLOOR	FLOOR AREA	COST/SF	COST
12" EJ	PHR	4567	1.2	5480.4
	MR	6171	1.2	7405.2
	10	6171	1.2	7405.2
	9	6171	1.2	7405.2
	8	12765	1.2	15318
	7	12765	1.2	15318
	6	12765	1.2	15318
	5	12765	1.2	15318
	4	12765	1.2	15318
	3	12765	1.2	15318
	2	12765	1.2	15318
				134922
18	MR	6594	1.2	7912.8
	10	6594	1.2	7912.8
	9	6594	1.2	7912.8
				23738.4

BEAM TAKEOFF			
SIZE	LENGTH (FT.)	COST/FT.	COST
W8X10	1272	9.17	11664.24
W12x26	7057	21.45	151372.65
W10x30	1075	26.4	28380
W14X30	1570	31.35	49219.5
W14X43	756	37.84	28607.04
W14X74	529	72.8	38511.2
W24X94	157	86.7	13611.9
			321366.53

COLUMN TAKEOFF			
SIZE	LENGTH (FT.)	COST/FT.	COST
W10X33	2210.4	19.67	43478.568
W21X93	2512.75	68.9	173128.475
HSS6X6X1/2	647.5	29.1	18842.25
			235449.293

CONCRETE TAKEOFF					
FLOOR	AREA	THICKNESS	VOLUME	COST/YD3.	COST
MAIN ROOF	14253	0.208	109.80	90	**9882.08**
10TH	14253	0.208	109.80	90	**9882.08**
9TH	14253	0.208	109.80	90	**9882.08**
8TH	14253	0.208	109.80	90	**9882.08**
7TH	14253	0.208	109.80	90	**9882.08**
6TH	14253	0.208	109.80	90	**9882.08**
5TH	14253	0.208	109.80	90	**9882.08**
4TH	14253	0.208	109.80	90	**9882.08**
3RD	14253	0.208	109.80	90	**9882.08**
2ND	14253	0.208	109.80	90	**9882.08**
					98820.8

STEEL DECK TAKEOFF			
FLOOR	AREA	COST/SF.	COST
P.ROOF	4567	1.15	**5252.05**
MAIN ROOF	14253	2.5	**35632.5**
10TH	14253	2.5	**35632.5**
9TH	14253	2.5	**35632.5**
8TH	14253	2.5	**35632.5**
7TH	14253	2.5	**35632.5**
6TH	14253	2.5	**35632.5**
5TH	14253	2.5	**35632.5**
4TH	14253	2.5	**35632.5**
3RD	14253	2.5	**35632.5**
2ND	14253	2.5	**35632.5**
			361577.05