Technical Report 1

Structural Concepts / Structural Existing Conditions Report

Columbia University Northwest Science Building

Broadway & 120th Street, New York, NY

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Executive Summary:

The overall objective of this technical report is to understand and analyze three conditions of the Columbia University Northwest Science Building's Structural Design. They are as follows:

- Gain practical knowledge of the building's structural design. Understand the gravity and lateral design systems and concepts, and how these elements work together.
- Calculate the wind, snow, and seismic loads and understand their effects on the structure.
- Provide several spot checks on critical members for a closer understanding of individual member design.

This report starts out with a basic introduction to the structural system and follows with a more in-depth breakdown of the structural components. The required design codes and material properties are then summarized before getting into more detailed calculations.

After the codes and materials are reviewed, the gravity and lateral loads are assessed. This section provides calculations on the wind, seismic, and snow loads. These loads are calculated using the required design codes. Diagrams of the loads are shown on the structure within the report. Furthermore, spreadsheet calculations are provided along with hand calculations that can be found at the end of the report in the appendix section.

The gravity and wind analysis was not able to be compared to the original building design. This building was originally designed using "New York City Building Code", which is different from the ASCE 7-05 and IBC 2006 codes used for load determination within this technical report. The loads determined made logical sense and will be analyzed further in Technical Reports 2 & 3, which are still underway.

Following the gravity and lateral load analysis, there are more specific spot check calculations of three critical elements within the structure. These elements consist of a beam/floor design, column design, and a truss member. The spot checks provide an initial discussion, the calculation data used, and conclusion on the calculation. The spot check calculations all confirmed adequate design. The detailed hand calculations on each spot check can be found at the end of the report in the appendix section.

This technical report will also illustrate that the Northwest Science Building has a very unique and intensive structural design. This building will most likely be seen as a great structural accomplishment within the engineering community upon completion in October 2010.

*Thank you to Turner Construction Company for providing the necessary documents, information, and images necessary for this Architectural Engineering Senior Thesis, Technical 1 Report.

Introduction to Structural System:

The structural system of Columbia University Northwest Building is a typical composite steel frame design. The steel framing consists mainly of wide flange shapes (beams and columns). All of the columns within the structure are W14's.

The floor system is composite. It uses wide flange beam members. On top of these members is corrugated metal decking and concrete slab (both normal weight and lightweight concrete is used throughout the structure). The concrete slab and metal decking is shear studded to the beam members creating a composite structure.

Castellated beams (cellular beams) are also used within the structure for larger clear spans of laboratory spaces. These beams provide great span to weight ratios.

The lateral system consists of horizontal, HSS shaped, girt members, and diagonal wide flange members. These members along with the composite floor system provide a safe and sound way for wind and seismic loads to reach the foundation and ultimately be distributed to the ground (Earth).

Part of the structure is very unique to the site and scope of the project. The design of this building calls for a 126-foot clear span over an existing structure, the Dodge Physical Fitness Center. This span is made possible by the use of heavy-duty steel trusses. The steel trusses consist of entirely W14 members ranging in weight per unit foot.

The building is 14 stories above grade reaching a maximum height of 226' 0". For more detailed information on the foundation, floor system, trusses, roof system, columns, lateral system, framing elevations, and structural sections see the following literature.

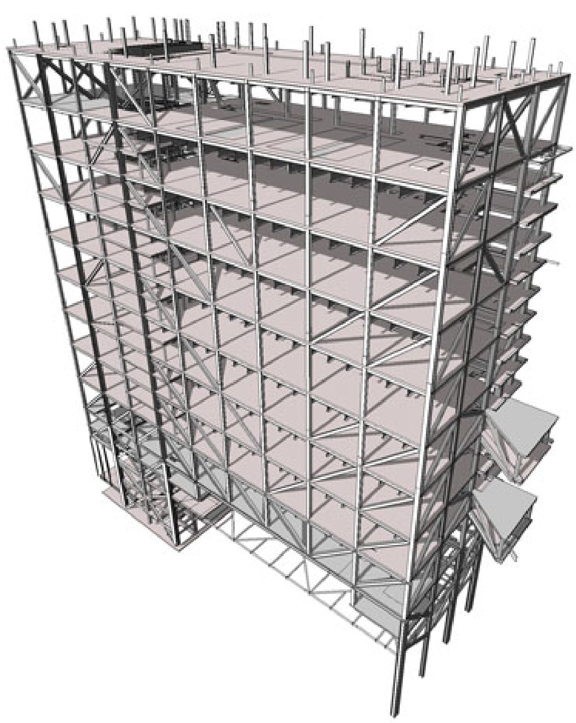


Figure 1: Structure Rendering

1. Foundation

The foundation consists of concrete piers, footings, column spread footings, and grade beams.

A. Concrete Piers

The concrete piers coincide with the sub-cellar and cellar foundation walls. These piers range in cross sectional size from 2'-0" x 3'-0" up to 5'-0" x 8'-0". See *Table 1*, "Pier Schedule", below, which breaks down the pier sizes and steel reinforcement used.

Table 1: Pier Schedule

PIER SCHEDULE				
COLUMN	SIZE (W x L)	REINFORCING		REMARKS
		VERTICAL	TIES	
A4	4'-9" x 6'-0"	44 - #10	#4@12	
C4	5'-0" x 8'-0"	44 - #10	#4@12	
D4	5'-0" x 8'-0"	44 - #10	#4@12	
A3	3'-0" x 5'-0"	26 - #9	#4@12	
A2	5'-0" x 5'-0"	28 - #10	#4@12	
A6.4	3'-0" x 3'-0"	16 - #9	#4@12	
E2, E3 & E4	2'-0" x 3'-0"	8 - #9	#4@12	

These piers are required to be normal weight concrete with a concrete compressive strength (f'c) of 6000 PSI. They support the exterior steel columns of the structure. See *Figure 2*, "Pier/Wall Section", below.

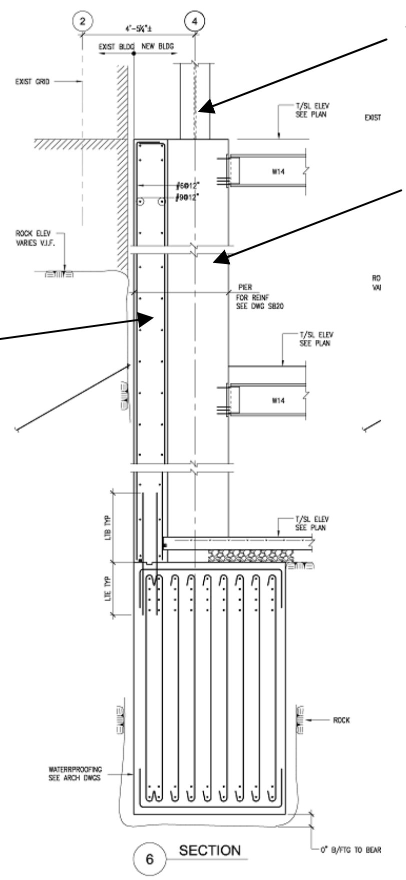


Figure 2: Pier/Wall Section

B. Footings and Column Spread Footings

The footings support the exterior foundation walls. These footings span the distance between the concrete piers.

The column spread footings support mainly interior columns and a few exterior columns. The spread footings vary in size. A large spread footing for this project is considered a 9'-0" x 9'-0" with a 5'-6" depth, while a smaller spread footing is 4'-6" x 4'-6" with a 2'-6" depth. See *Table 2*, "Footing Schedule", below. This schedule breaks down all footing sizes and shows the steel reinforcement used.

Table 2: Footing Schedule

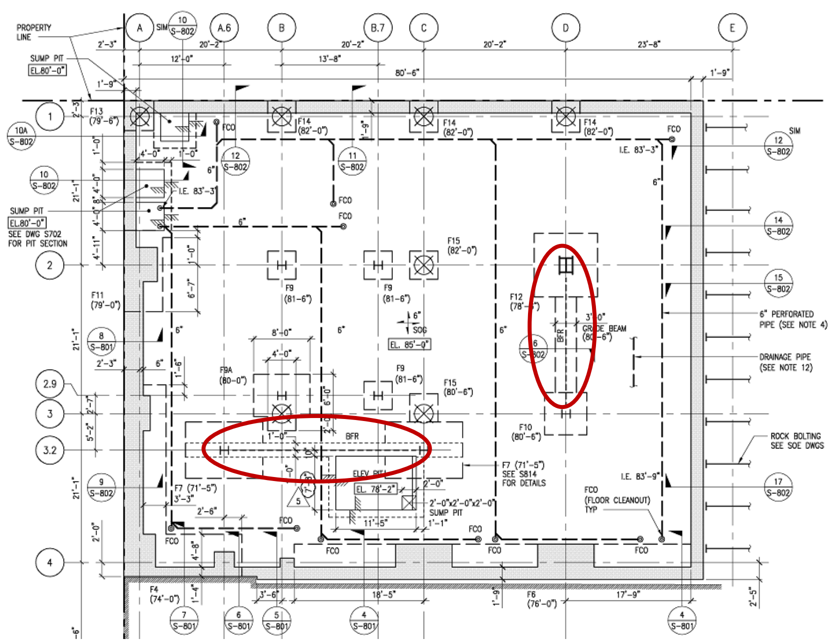
FOOTING SCHEDULE									
MARK	SIZE WxL	DEPTH	BOTTOM REINFORCING		TOP REINFORCING		DESIGN BEARING CAPACITY	NO. OF ROCK ANCHORS (TOTAL UPLIFT-KIPS)	REMARKS
			LONG WAY	SHORT WAY	LONG WAY	SHORT WAY			
F1	6'-0"x6'-6"	6'-0"	10-#9	10-#9			60 TSF	4 (400)	SKIN FRICTION=100 PSI W/ 6'-0" EMBED.
F2	3'-6"X5'-6"	2'-0"	6-#7	6-#7			44 TST		
F3	4'-0"x40'-2"	6'-0"	4 LAYERS 8-#10		3 LAYERS 8-#10		60 TSF	2 (400) @ C10 & D10	2x6 LEGS-#4@5" SHEAR REINFORCEMENT
F4	L-SHAPE (SEE PLAN)	10'-0"	2 LAYERS 10-#9	#9@6"	4 LAYER 10-#9	#9@6"	40 TSF	3 (400)	10 LEGS-#5@6" SHEAR REINF
F5	NOT USED								
F6	5'-0"x SEE PLAN	8'-0"	7 LAYERS 9-#10		2 LAYERS 9-#10		40 TSF	2 (400) @ C4 & D4	2x6 LEGS-#4@7" SHEAR REINFORCEMENT
F7	8'-0"X11'-0"	5'-0"	12-#9	16-#9	9-#9	12-#9	40 TSF	8 (1900) @ C-3.2 & A.6-3.2	SEE DWG S814 FOR DETAILS
F8	NOT USED								
F9	4'-6"X4'-6"	2'-6"	7-#8	7-#8			40 TSF	2 (400)	
F9A	SEE PLAN	4'-0"	20-#7	20-#7			40 TSF	2 (400)	COMBINED FOOTING FOR TEMP. SHORING
F10	6'-0"x6'-0"	3'-6"	8-#8	8-#8			40 TSF	4 (400)	
F11	5'-6"x12'-6"	5'-0"	3 LAYERS 11-#9	24-#8			40 TSF	2 (400)	
F12	9'-0"x9'-0"	5'-6"	17-#10	17-#10			40 TSF	4 (400)	
F13	4'-6"x4'-6"	4'-6"	9-#7	9-#7			40 TSF		FOOTING FOR TEMPORARY SHORING
F14	4'-0"x4'-6"	2'-0"	8-#8	9-#8			40 TSF		FOOTING FOR TEMPORARY SHORING
F15	4'-0"x4'-0"	2'-0"	8-#8	8-#8			40 TSF		FOOTING FOR TEMPORARY SHORING
F16	2'-0"x4'-0"	1'-4"	3-#6	5-#6			40 TSF		
F16A	2'-0"x4'-10"	1'-4"	3-#6	6-#6			40 TSF		

C. Grade Beams

Two grade beams are used in the foundation of the building. These grade beams are used to provide a resistance to lateral column base movement. One of the grade beams used is 80'-6" long, spans from grid lines 2 to 3, and has a cross section of 3'-0" x 3'-6". The other is smaller in cross section and length and spans in the opposite direction between grid lines A.6 and C. See *Figure 3*, "North End Foundation Plan", below showing both grade beams.

Figure 3: North End Foundation Plan

(Grade Beams Circled in Red)



2. Floor System

The building's floor system changes dramatically from level 500 to level 600. This is due to the buildings 126 foot clear span. The building spans over an existing structure, the Dodge Physical Fitness Center. This clear span allows for the continued use of the center with minimum demolition to its existing structure. Due to this dramatic change in floor area from level 500 to level 600, two floor plans of the structure will be discussed. These floor plans will be discussed as Typical Floor Plan 1 and Typical Floor Plan 2.

A. Typical Floor Plan 1 *(Levels 100 to 500)*

This floor system is a composite steel structure. The beam spanning consists of wide flange shapes. Spanning across from beam to beam is corrugated steel decking with concrete slabs, both shear studded to the wide flanges. The concrete slabs are designated a concrete compressive strength of 4000 PSI. Slab thickness and the use of normal and lightweight concrete vary throughout the structure. See *Figure 4* below, "Concrete Slab Notes", for information on slab types.

Figure 4: Concrete Slab Notes

SOG DENOTES 6" NORMAL WEIGHT CONCRETE SLAB ON GRADE

D1 DENOTES 4" NORMAL WEIGHT CONCRETE TOPPING ON 2"– 19 GA COMPOSITE METAL DECK (6" TOTAL SLAB THICKNESS REINF W/ 6 x 6 – W 2.0 x 2.0

D2 DENOTES 6" NORMAL WEIGHT CONCRETE TOPPING ON 2"–18 GA COMPOSITE METAL DECK (8" TOTAL SLAB THICKNESS) REINF W/ 4 x 4 – W 2.0 x 2.0

D3 DENOTES 5" NORMAL WEIGHT CONCRETE TOPPING ON 3"–16 GA COMPOSITE METAL DECK (8" TOTAL SLAB THICKNESS) REINF W/ # 5 @ 12 BOTTOM AND WWF 6 x 6 – W 2.9 x 2.9 TOP. SHORING REQUIRED FOR SPANS EXCEEDING 11'–6".

D4 DENOTES 5" LIGHT WEIGHT CONCRETE TOPPING ON 3"–16 GA COMPOSITE METAL DECK (8" TOTAL SLAB THICKNESS) REINF W/ # 6 @ 12 BOTTOM AND WWF 6 x 6 – W 2.9 x 2.9 TOP. SHORING REQUIRED FOR SPANS EXCEEDING 10'–6".

D5 DENOTES 3" LIGHT WEIGHT CONCRETE TOPPING ON 3"–18 GA COMPOSITE METAL DECK (6" TOTAL SLAB THICKNESS) REINF W/ WWF 6 x 6 – W 2.0 x 2.0 TOP.

D6 DENOTES 2½" LIGHT WEIGHT CONCRETE TOPPING ON 1½"– 18 GA COMPOSITE METAL DECK (4" TOTAL SLAB THICKNESS) REINF W/ WWF 6 x 6 – W 1.4 x 1.4 TOP.

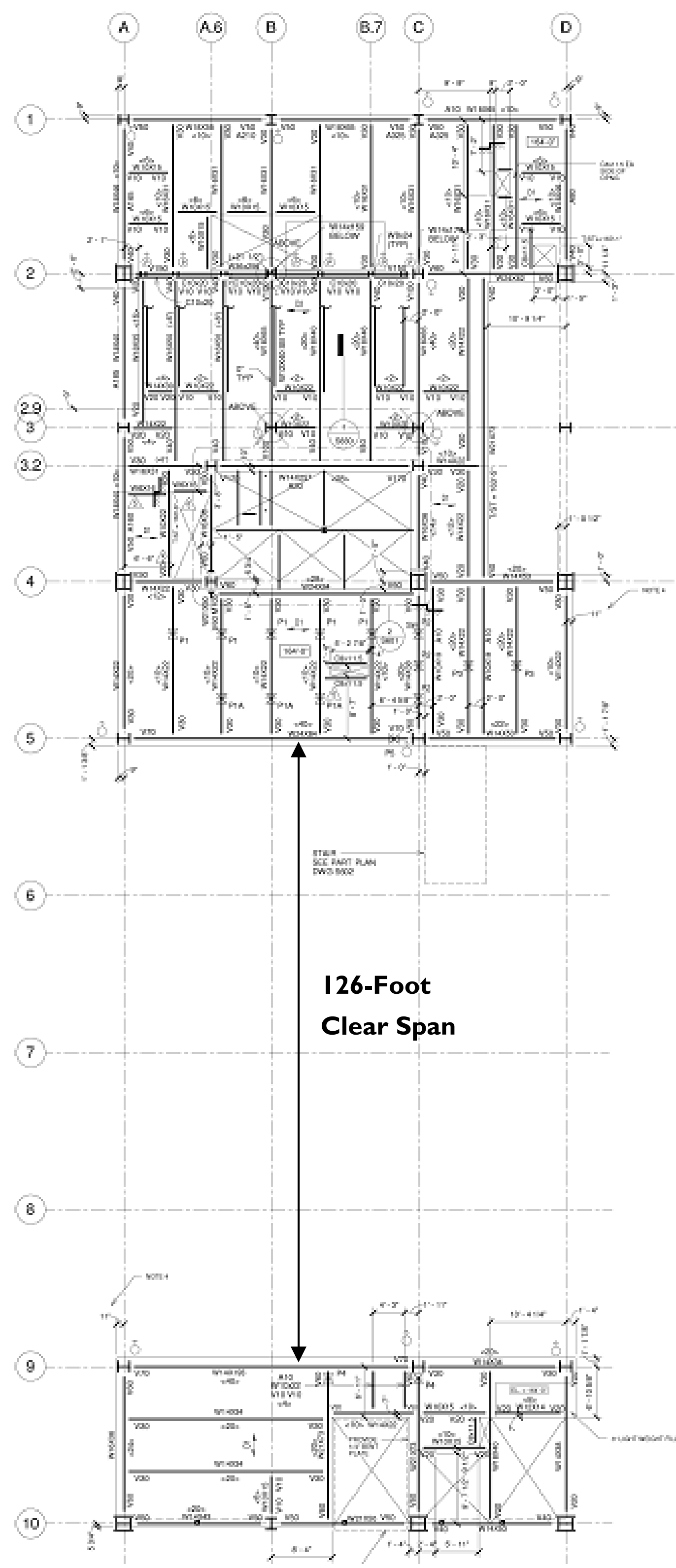
D7 DENOTES 3¼" LIGHT WEIGHT CONCRETE TOPPING ON 2"– 18 GA COMPOSITE METAL DECK (5¼" TOTAL SLAB THICKNESS) REINF W/ WWF 4 x 4 – W 2.0 x 2.0 TOP.

D8 DENOTES 6" LIGHT WEIGHT CONCRETE TOPPING ON 2"– 18 GA COMPOSITE METAL DECK (8" TOTAL SLAB THICKNESS) REINF W/ WWF 4 x 4 – W 2.0 x 2.0 TOP.

D9 DENOTES 2½" LIGHT WEIGHT CONCRETE TOPPING ON 2"– 18 GA COMPOSITE METAL DECK (4½" TOTAL SLAB THICKNESS) REINF W/WWF 6 x 6 – W 2.0 x 2.0 TOP.

All steel decking is to have a minimum yield stress of 33 KSI. All shear studs used will be nelson flux filled shear connectors. See *Figure 5*, "Typical Floor Plan 1", on the following page.

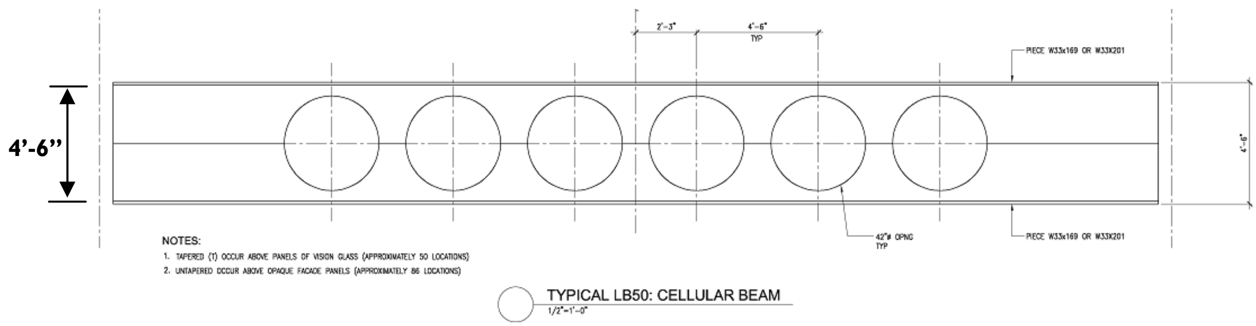
Figure 5: Typical Floor Plan 1



B. <u>Typical Floor Plan 2 *(Levels 600 to 1400)*</u>

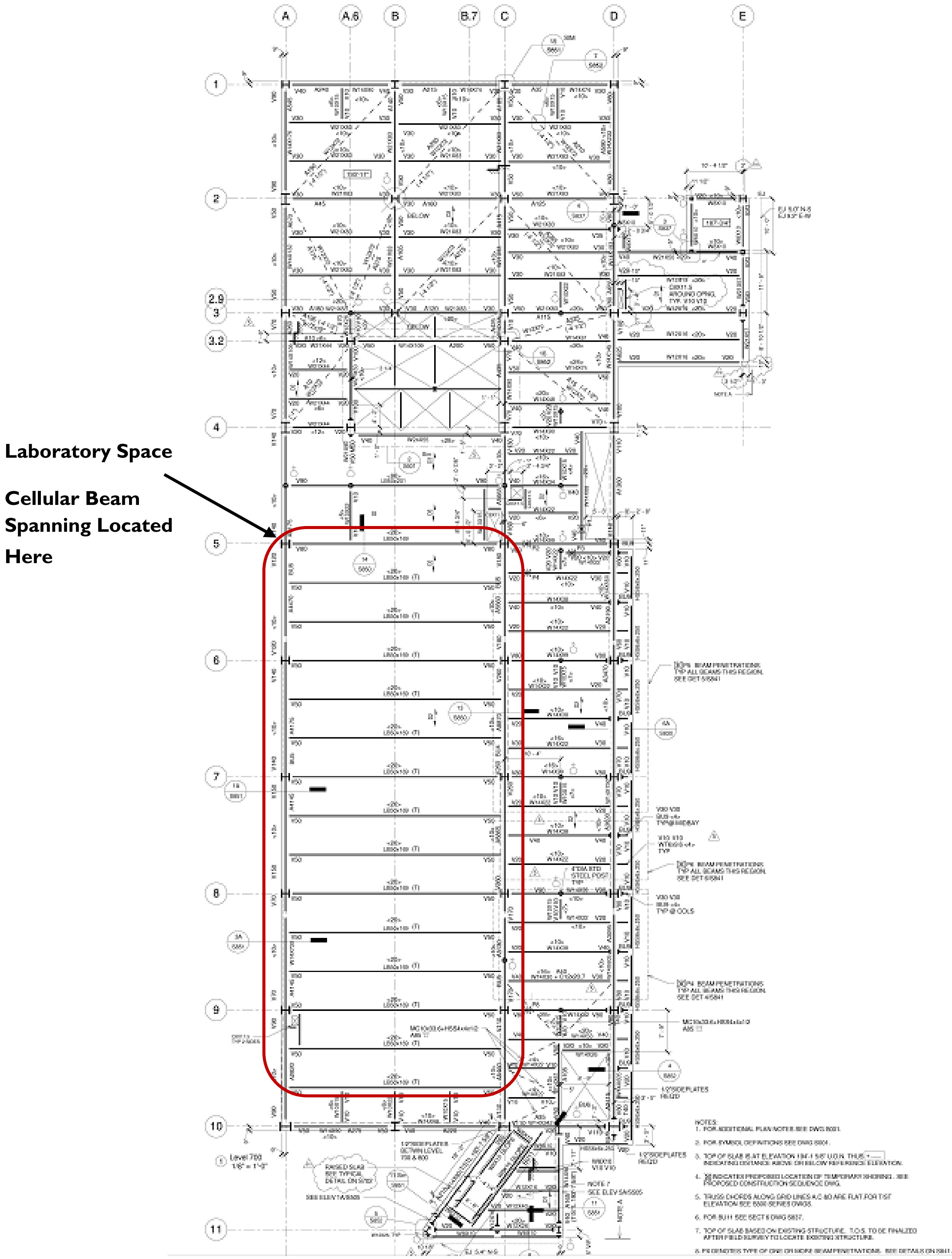
This floor system is also a composite steel structure and also uses wide flange shape spanning. However, another spanning member is introduced because of longer clear spans needed for large laboratory spaces. These members are castellated beams, also known as cellular beams. They are typically about five foot deep and allow for 40 feet clear spans in the labs. See *Figure 6*, "Typical Cellular Beam", below.

Figure 6: Typical Cellular Beam



"Typical Floor Plan 2" is shown on the following page as *Figure 7*.

Figure 7: Typical Floor Plan 2



3. Trusses

As mentioned before, the structure has a 126-foot clear span. In order to span over the existing fitness center three giant, heavy-duty steel trusses are used. These trusses are composed of at W14 diagonal members that are connected to the steel framing. These W14 members are very heavy. They are large enough to be comparable to a bridge truss steel structure. See *Figure 8*, "Truss Snapshot", below to get a better visual. These trusses direct the gravity load towards the ends of the structure to the edge columns, where then the loads can be directed towards the foundation and ultimately be distributed to the ground.

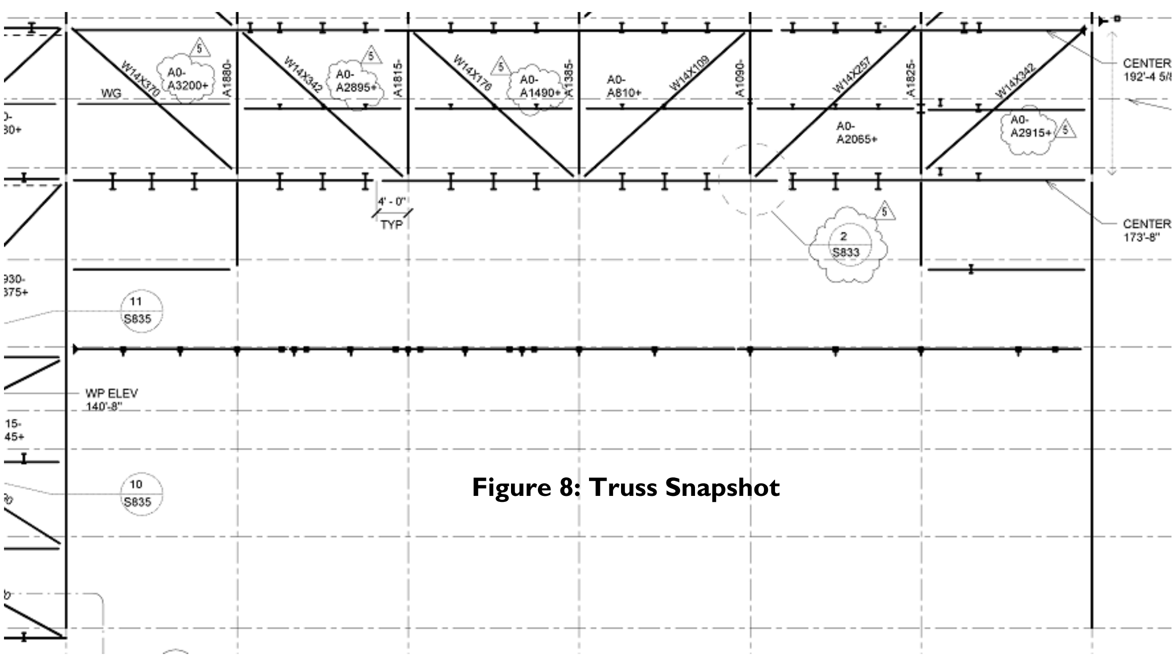
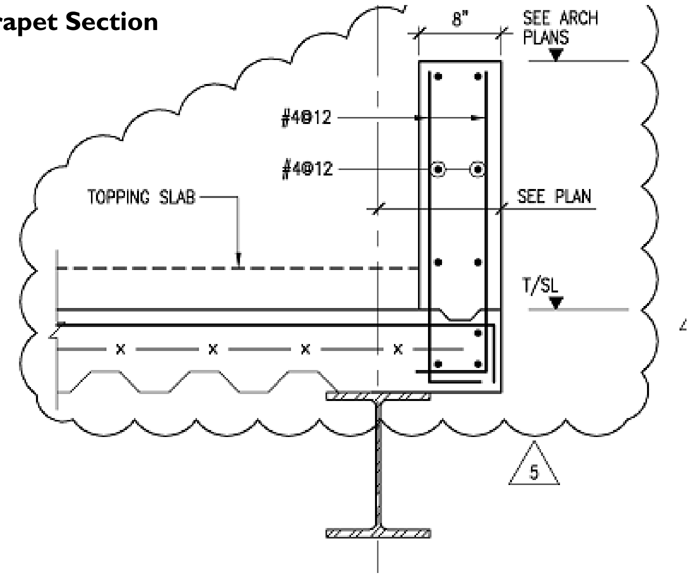


Figure 8: Truss Snapshot

4. Roof System

The roof system is a composite steel structure. It consists of wide flange spanning, steel decking, and concrete slab. Specifically, the floor system is 6" normal weight concrete topping on 2" – 18 GA composite metal deck, which is about an 8" total slab thickness. Above the concrete slab is a lightweight concrete topping with flashing and a roof membrane. The roof is surrounded by an 8" thick parapet wall. See *Figure 9*, "Roof Parapet Section", on the following page.

Figure 9: Roof Parapet Section



5. Columns

Level 600 and above typically contain 40 columns per level. Below level 600 the amounts of columns per level vary. However, an average of about 20 columns per level (levels 100-500) can be estimated. All of the columns above level 100 are wide flange shaped. Concrete columns/piers only exist as part of the foundation and have been discussed in the "Concrete Piers" section above on page 5.

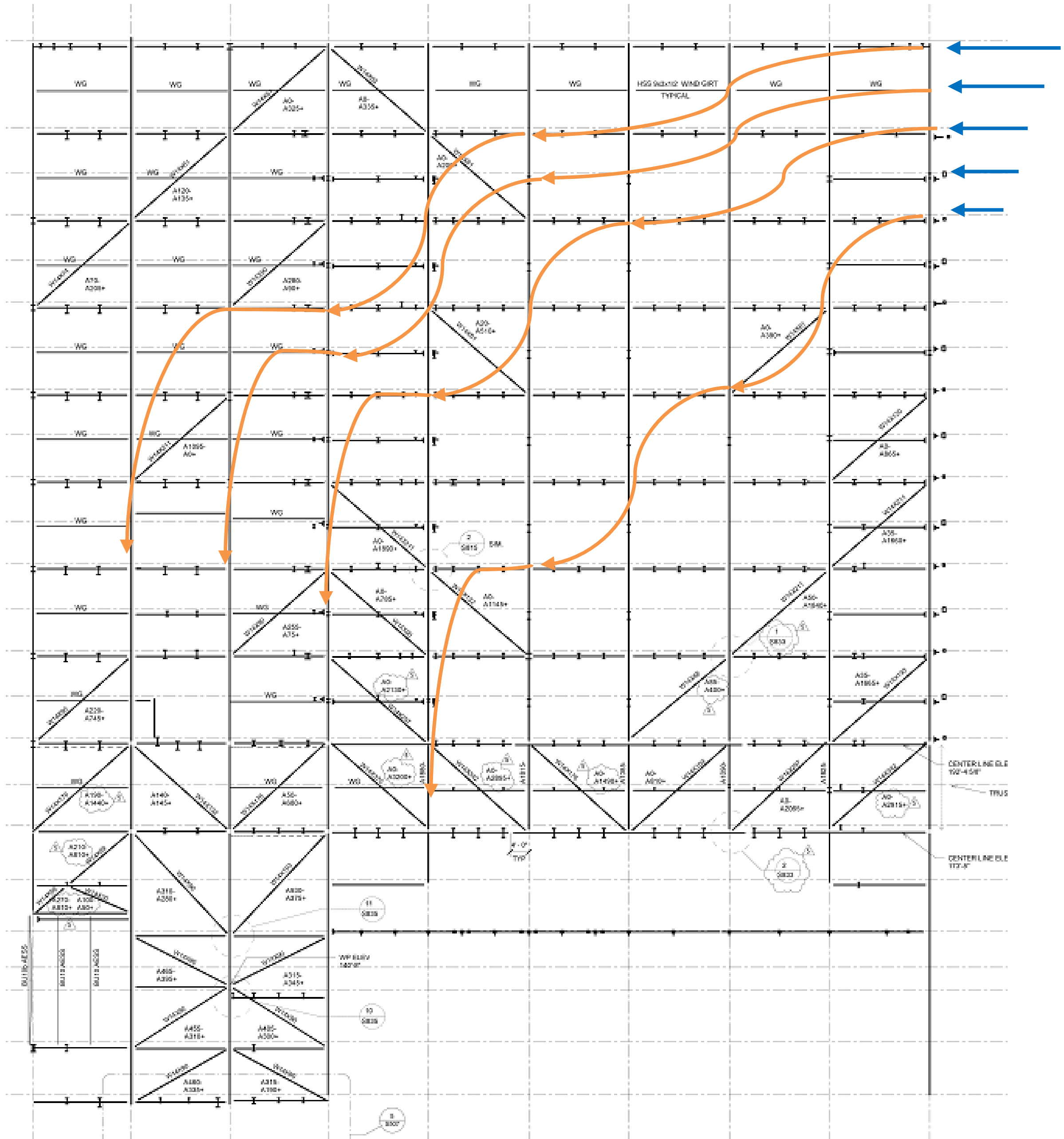
Every steel column used is a W14. The use of all W14's considerably increases column to column connection efficiency and labor. The weight per foot of the columns varies dramatically over the height of the structure. For example, level 200 contains column sizes of W14x605, W14x550, and W14x455, while level 1300 contains column sizes of W14x178, W14x109, and W14x61. The weight of the columns decreases with the height of the structure.

Usually, the gravity loads are directed downwards from column to column in typical structures until the load reaches the ground. This building follows this trend, with some exceptions due to its 126-foot clear span. Some of the gravity loads are directed upwards through the 126-foot truss system. These loads are directed diagonally until they are able to reach columns that run back down towards the ground foundation.

6. Lateral System

The lateral system is composed of diagonal wind bracing, wind girts, a composite floor system, moment connections, and wide flange beams and columns. The diagonal wind bracing elements are made up of W14 members and the wind girts are HSS shaped members. Due to its complexity with a large range of components, each taking part in the system, a lateral load path is illustrated below. See *Figure 10*, "Lateral Load Path Elevation", on the following page.

Figure 10: Lateral Load Path Elevation

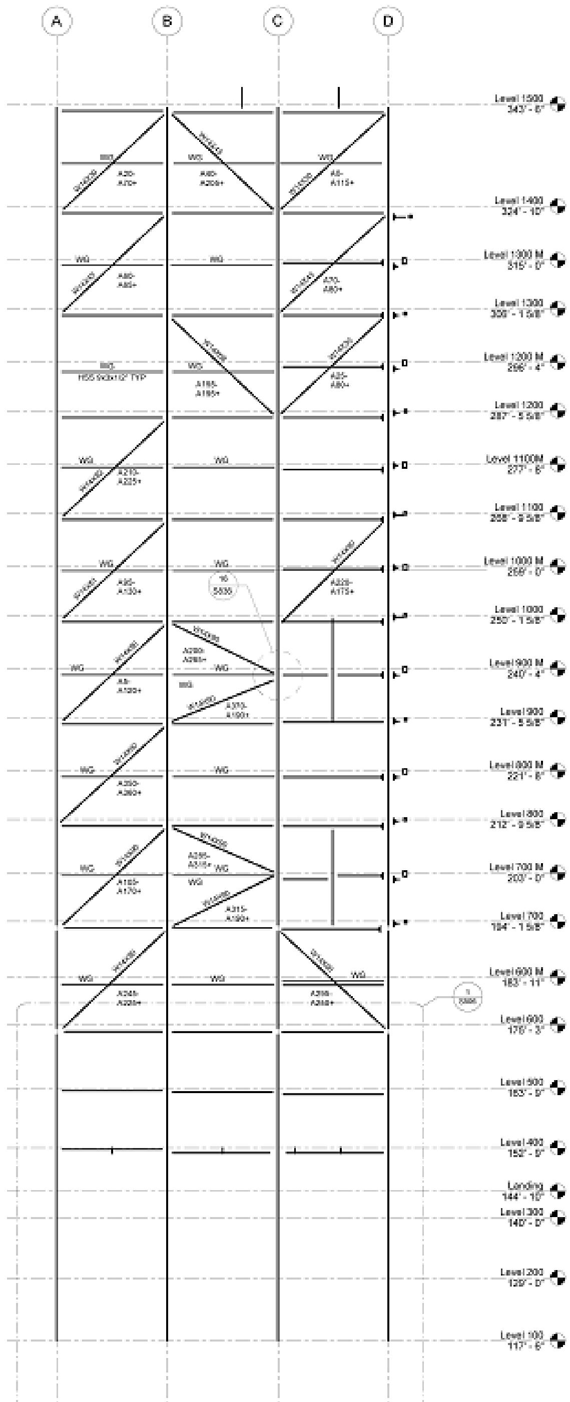


From the diagram above, notice how the lateral load first reaches the beam and wind girt elements. From these elements is it transferred to the composite floor system, and is then carried downwards by some diagonal bracing, columns, and moment connections.

7. Framing Elevations

A framing elevation in the North-South direction has already been shown in the lateral system description above. An additional elevation is shown of a typical East-West direction framing. See *Figure 11*, "Typical East-West Framing Elevation", below. Notice the wind girts, HSS members, provided in between the main levels. These girts provide an additional access for wind to be distributed within the structure.

Figure 11: Typical East-West Framing Elevation

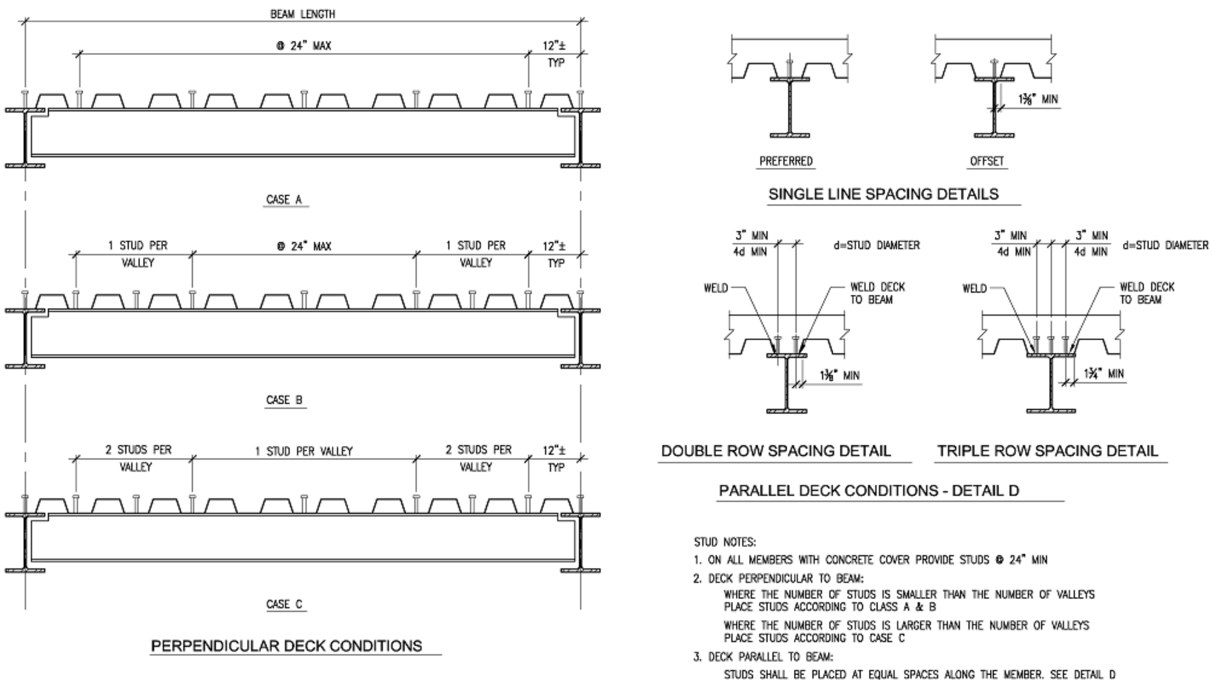


8. Structural Sections and Details

A. Shear Stud Connections

Due to the structural importunacy of the composite floor system a section detail of the shear stud connections is shown below, *Figure 12*.

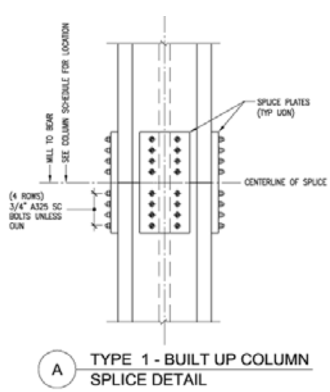
Figure 12: Typical Deck Connection Section Detail



B. Column Splice Connections

Due to the height of the structure a large amount of column splice connections are used. It is important to understand how the connection is made to adequately connect each column member and safely transfer the forces. Notice in *Figure 13* below, the plate and bolt connections provided on each flange member and web of each column. This section detail is very typical throughout the structure because the column size of W14 does not change.

Figure 13: Column Splice Connection Detail



9. Conclusions on Structural System

The structural system, as shown, can be very complex. Both the lateral and gravity systems can be very difficult to understand at first look. However, the complexity of this design can be understood because of its unique project requirements.

The use of steel for the entire frame of the structure is used most likely for two main reasons. The first is because New York City is known for steel structures and therefore reliable steel construction workers are attainable. The second reason is due to the required long spans. Both the laboratory spans and the 126-foot clear span over the fitness center call for a material that can provide a great strength to weight ratio.

The composite floor system design is used for two reasons. It was designed to allow for decreased steel sections for spanning. Decreasing the section sizes allows for more head room. The other reason is to use the floor system as a lateral component. The composite system allows lateral forces to be distributed to a stronger and stiffer structure.

II. Building Codes:

Codes used in the design of the structure are as follows:

- "International Building Code 2006" – International Code Council
- "ACI 318-05 Manual of Concrete Practice" – American Concrete Institute
- "Manual of Steel Construction 9th Edition" – American Institute of Steel Construction, Inc.
- "ASCE 7-05 Minimum Design Loads for Buildings and Other Structures" – American Society of Civil Engineers
- "New York City Building Code & Regulations"
- "New York City Construction Code"

Please note: From this point on all the research, calculations, interpretations, and findings of this Technical Report will be based solely on the International Building Code 2006, ASCE 7-05, and the Manual of Steel Construction 9th Edition. If calculations are not shown within the text please check the appendix section at the end of the report. Not all calculations are included in this report and can be provided upon request.

III. Materials:

1. Reinforced Concrete

Type	f'c (PSI)	Aggregate
Footings, Caissons	6000	Normal Weight
Slab on Grade	4000	Normal Weight
Walls and Columns	6000	Normal Weight
Beams and Slabs	6000	Normal Weight
Slab on Steel Deck	4000	Normal Weight
Equipment Pads and Curbs	4000	Normal Weight
Lean Concrete	4000	Lightweight

2. Structural Steel

Shape	Fy (KSI)
Wide Flanges	50
Fabricated / Plated Sections	50
Channels	50
Rectangular and Round HSS	46
Pipes	35
Angles – For Connections	36
Plates – For Connections	36
Tees	50

IV. Gravity and Lateral Loads:

The following gravity loads were determined from ASCE 7-05. When specific gravity loads could not be referenced, estimation was made with some basic structural research.

1. Floor Dead Loads:

Construction Dead Load

Load Type	PSF or PCF
Normal Weight Concrete	150 PCF
Lightweight Concrete	120 PCF
Steel	490 PCF
M.E.P	10 PSF
Finishes & Miscellaneous	5 PSF
Partitions	10 PSF
M.E.P	8 PSF
Façade (Aluminum Cladding)	0.75 PSF

2. Floor Live Loads:

Type of Space	PSF
Offices	50
Mechanical	150
Library – Stack Rooms	150
Library – Reading Rooms	60
Corridors above 1st Floor	80
Lobbies & 1st Floor Corridors	100
Roof	20
Classrooms	40
Laboratories	100
Stairs & Exit Ways	100

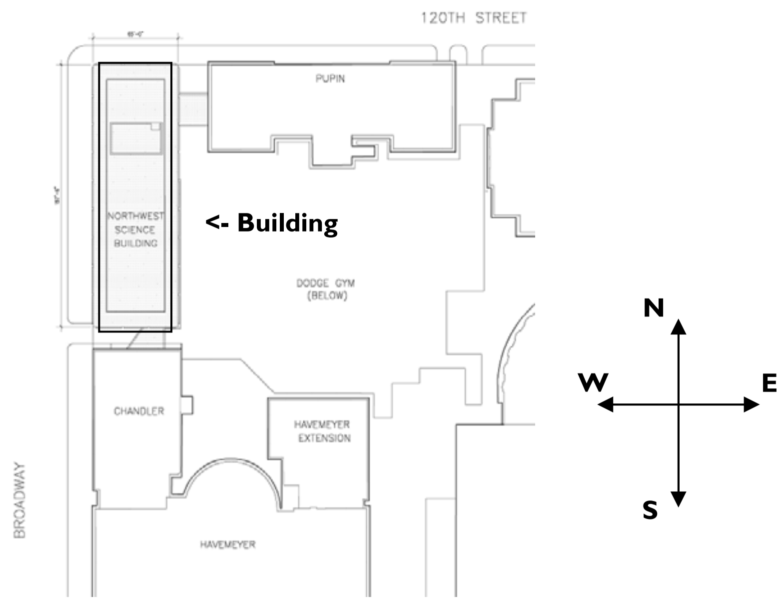
3. Wind Load Calculations and Diagrams:

Discussion

Wind load analysis is a critical factor in the design of the Columbia University Northwest Building. The wind analysis below obtained a base shear force of 725.41 kips for wind in the North-South direction and 2860.07 kips in the East-West direction. See *Figure 14,* "Site Map", below. The following literature, tables, and diagrams explain the wind analysis process and findings.

Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Figure 14: Site Map



Calculation Data

- Location: New York, NY
- Exposure: D (Building at Shoreline)
- Topography: Level (Not on a hill or ridge)
- Occupancy: III

Determine design wind pressures on Main Wind Force Resisting System (MWFRS)

- ASCE 7-05 – C6.5 Wind Design Method 2 – Analytical Procedure
    - Assume use of Analytical Procedure (Method 2) is efficient for wind study for Technical Report 1. The building is located in Manhattan where there are cluster of tall buildings. This cluster may cause a limitation on Method 2 Analytical Procedure, and a wind tunnel analysis could be used for more accuracy. However, due to time constraints and lack of research equipment availability, Method 2 Analytical Procedure will be applied.
- Occupancy Category → III (Bld. capacity greater than 500 for colleges)
- Basic Wind Speed (V) from Fig. 6-1 → V=110 mph (49 m/s)

Tables and Figures

Below is a bulleted list explaining the tables and figures to follow, regarding wind calculations and diagrams.

- Table 3: Basic Wind Pressure Parameters
  - Provides basic wind factors based upon location of site, topography of site, and additional building properties.
- Table 4: Gust Factor Parameters
  - Provides factors needed in finding the gust effect on the structure.
- Table 5: C_p, Gust Factor, GC_{pi} Factors
  - Summarizes the gust factors found for the leeward and windward sides of the building. Also provides the external pressure coefficient (C_p), and internal pressure coefficient (GC_{pi}) values.
- Figure 15: Wind North-South Direction Diagram
  - Provides a visual of the wind forces (windward and leeward) on the structure in PSF.
- Figure 16: Wind East-West Direction Diagram
  - Provides a visual of the wind forces (windward and leeward) on the structure in PSF.
- Tables 6A & 6B: Wind North-South Direction
  - Provides the excel spreadsheet wind analysis that was used in finding the wind forces acting on the structure. Also, provides the final base shear and overturning moment for the structure caused by wind.
- Tables 7A & 7B: Wind East-West Direction
  - Provides the excel spreadsheet wind analysis that was used in finding the wind forces acting on the structure. Also, provides the final base shear and overturning moment for the structure caused by wind.

Conclusions:

The wind calculations show a much larger base shear for the East-West wind direction. The East-West direction has base shear almost 4 times greater than the North-South wind direction. This is due to the large area of the East and West facades compared to the North and South facades. This additional area provides a large amount of space for the wind to act upon the building. In addition, the magnitudes of the wind forces are reasonable based upon the 226' height of the structure. Please reference the tables and diagrams for the actual values in question.

Table 3: Basic Wind Pressure Parameters

Basic Wind Speed (V)	110 MPH
Wind Exposure Category	C
Building Category	III
Importance Factor	1.15
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0

Number of Stories	14
Building Height (Feet)	226'-0"
N-S Building Length (Feet)	196.75'
E-W Building Length (Feet)	60.5'
L/B in N-S Direction	3.252
L/B in E-W Direction	0.307

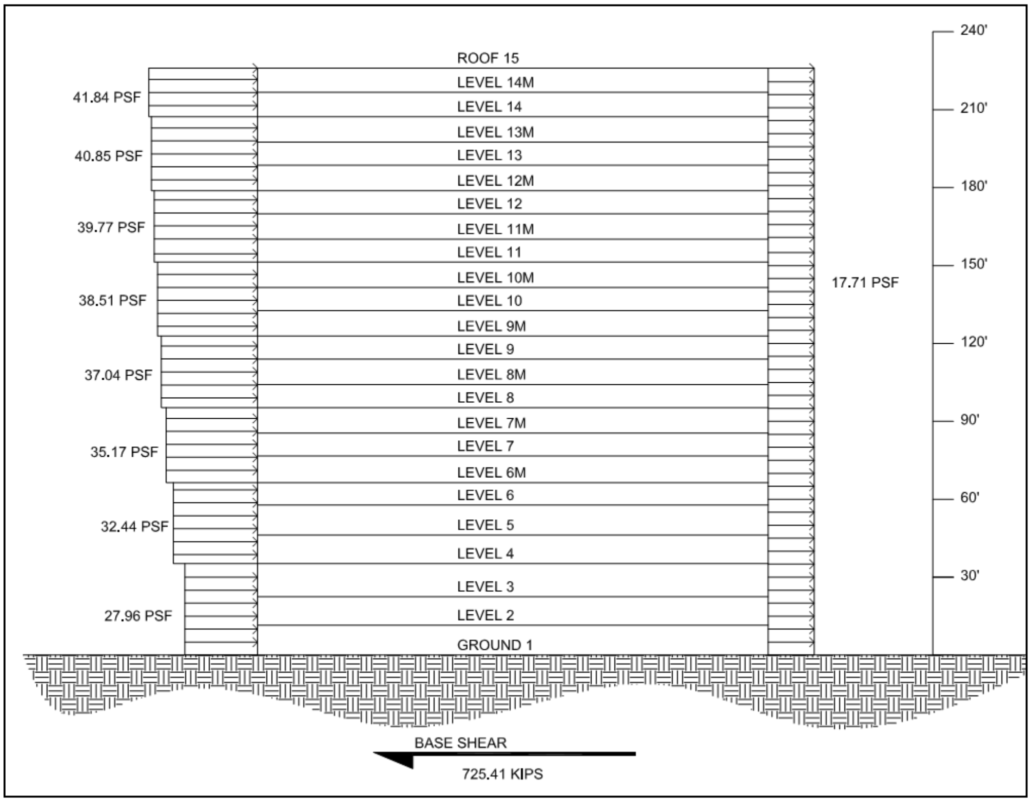
Table 4: Gust Factor Parameters

Gust Factor		
Variable	**Wind Direction**	
	N-S	**E-W**
Stiffness	**Flexible (n_1<1)**	**Flexible (n_1<1)**
n_1	0.4425	0.4425
B (Feet)	60.5196	196.75
L (Feet)	196.75	60.5
h (Feet)	226	226
I_z	0.005	0.005
L_z (Feet)	684.85	684.85
Q	0.856	0.826
g_r	3.99	3.99
g_Q & g_v	3.4	3.4
V_z	110.49	110.49
α	1/6.5	1/6.5
b	0.65	0.65
N_1	2.743	2.743
R_n	0.074	0.074
R_h	0.211	0.211
R_B	0.538	0.238
R_L	0.079	0.232
R	0.690	0.487
G_f	0.929	0.925

Table 5: C_p, Gust Factor, GC_{pi} Factors

Wind Direction	C_p (Windward)	C_p (Leeward)	Gust Factor (Windward)	Gust Factor (Leeward)	GC_{pi}
N-S Direction	0.8	-0.225	0.929	0.925	±0.18
E-W Direction	0.8	-0.5	0.929	0.925	±0.18

Figure 15: Wind North-South Direction Diagram



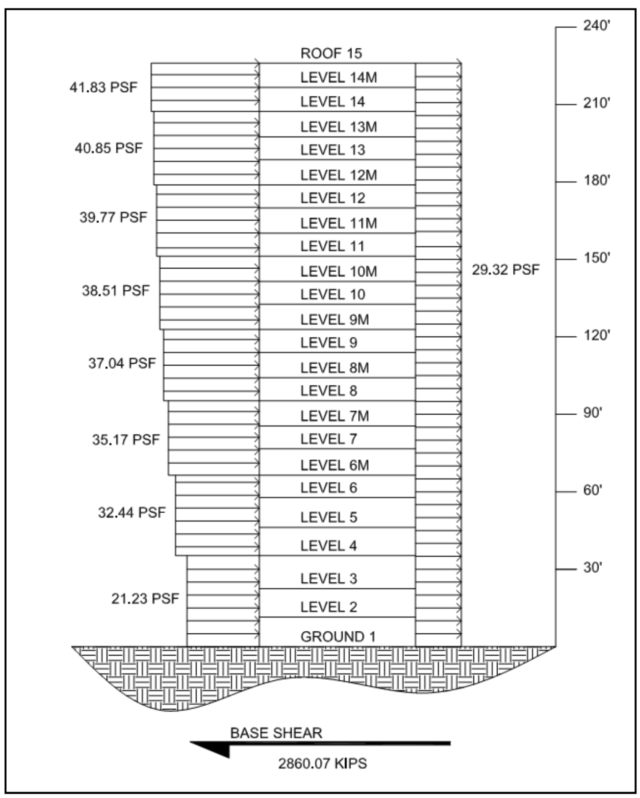


Figure 16: Wind East-West Direction Diagram

Table 6A: Wind North-South Direction

Level	Height (Feet)	Tributary Area (Feet)	K_z	$q_z = 0.00256K_zK_{zt}K_dV^2I$	K_h	$q_h = 0.00256K_hK_{zt}K_dV^2I$
Roof (15)	226.00	4.67	1.50	45.64	1.50	45.64
14M	216.67	9.34	1.49	45.24	1.50	45.64
14	207.33	9.59	1.48	44.82	1.50	45.64
13M	197.50	9.36	1.46	44.36	1.50	45.64
13	188.63	9.34	1.45	43.94	1.50	45.64
12M	178.83	9.33	1.43	43.45	1.50	45.64
12	169.97	9.33	1.42	42.98	1.50	45.64
11M	160.17	9.34	1.40	42.45	1.50	45.64
11	151.30	9.34	1.38	41.94	1.50	45.64
10M	141.50	9.84	1.36	41.36	1.50	45.64
10	132.63	9.84	1.34	40.80	1.50	45.64
9M	122.83	9.33	1.32	40.14	1.50	45.64
9	113.97	8.83	1.30	39.51	1.50	45.64
8M	104.17	8.84	1.28	38.77	1.50	45.64
8	95.30	9.34	1.25	38.05	1.50	45.64
7M	85.50	9.33	1.22	37.19	1.50	45.64
7	76.64	9.54	1.20	36.35	1.50	45.64
6M	66.42	9.45	1.16	35.27	1.50	45.64
6	57.75	10.09	1.13	34.25	1.50	45.64
5	46.25	11.25	1.08	32.68	1.50	45.64
4	35.25	11.88	1.02	30.86	1.50	45.64
3	22.50	11.88	0.92	28.08	1.50	45.64
2	11.50	11.25	0.85	25.78	1.50	45.64

Table 6B: Wind North-South Direction Continued

Level	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Roof (15)	42.14	17.71	59.85	16.91	16.91	0.00
14M	41.84	17.71	59.55	33.65	50.56	157.77
14	41.53	17.71	59.24	34.37	84.93	629.99
13M	41.19	17.71	58.90	33.35	118.28	1464.85
13	40.87	17.71	58.58	33.10	151.39	2514.02
12M	40.50	17.71	58.22	32.86	184.25	3997.62
12	40.16	17.71	57.87	32.67	216.92	5630.06
11M	39.76	17.71	57.48	32.48	249.40	7755.85
11	39.39	17.71	57.10	32.27	281.66	9968.00
10M	38.95	17.71	56.67	33.73	315.40	12728.29
10	38.53	17.71	56.25	33.49	348.88	15525.86
9M	38.05	17.71	55.76	31.48	380.36	18944.91
9	37.58	17.71	55.30	29.54	409.90	22314.89
8M	37.03	17.71	54.75	29.28	439.18	26331.91
8	36.50	17.71	54.21	30.63	469.81	30227.42
7M	35.86	17.71	53.57	30.24	500.05	34831.58
7	35.23	17.71	52.94	30.56	530.61	39262.03
6M	34.43	17.71	52.14	29.81	560.42	44684.85
6	33.67	17.71	51.38	31.36	591.78	49543.68
5	32.50	17.71	50.22	34.18	625.96	56349.19
4	31.15	17.71	48.87	35.12	661.09	63234.79
3	29.09	17.71	46.80	33.64	694.72	71663.65
2	27.38	17.71	45.09	30.69	725.41	79305.61
Ground (1)	27.40	17.71	45.12	0.00	**725.41**	**87647.88**

Table 7A: Wind East-West Direction

Level	Height (Feet)	Tributary Area (Feet)	K_z	$q_z = 0.00256K_zK_{zt}K_dV^2I$	K_h	$q_h = 0.00256K_hK_{zt}K_dV^2I$
Roof (15)	226.00	4.67	1.50	45.64	1.50	45.64
14M	216.67	9.34	1.49	45.24	1.50	45.64
14	207.33	9.59	1.48	44.82	1.50	45.64
13M	197.50	9.36	1.46	44.36	1.50	45.64
13	188.63	9.34	1.45	43.94	1.50	45.64
12M	178.83	9.33	1.43	43.45	1.50	45.64
12	169.97	9.33	1.42	42.98	1.50	45.64
11M	160.17	9.34	1.40	42.45	1.50	45.64
11	151.30	9.34	1.38	41.94	1.50	45.64
10M	141.50	9.84	1.36	41.36	1.50	45.64
10	132.63	9.84	1.34	40.80	1.50	45.64
9M	122.83	9.33	1.32	40.14	1.50	45.64
9	113.97	8.83	1.30	39.51	1.50	45.64
8M	104.17	8.84	1.28	38.77	1.50	45.64
8	95.30	9.34	1.25	38.05	1.50	45.64
7M	85.50	9.33	1.22	37.19	1.50	45.64
7	76.64	9.54	1.20	36.35	1.50	45.64
6M	66.42	9.45	1.16	35.27	1.50	45.64
6	57.75	10.09	1.13	34.25	1.50	45.64
5	46.25	11.25	1.08	32.68	1.50	45.64
4	35.25	11.88	1.02	30.86	1.50	45.64
3	22.50	11.88	0.92	28.08	1.50	45.64
2	11.50	11.25	0.80	24.38	1.50	45.64
Ground (1)	0.00	0	0.00	0.00	1.50	45.64

Table 7B: Wind East-West Direction Continued

Level	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Roof (15)	42.14	29.32	71.46	65.66	65.66	0.00
14M	41.84	29.32	71.16	130.77	196.42	612.59
14	41.53	29.32	70.85	133.68	330.10	2447.19
13M	41.19	29.32	70.51	129.85	459.95	5692.11
13	40.87	29.32	70.19	128.99	588.94	9771.91
12M	40.50	29.32	69.83	128.18	717.12	15543.55
12	40.16	29.32	69.48	127.55	844.68	21897.28
11M	39.76	29.32	69.09	126.96	971.63	30175.10
11	39.39	29.32	68.71	126.27	1097.90	38793.49
10M	38.95	29.32	68.27	132.18	1230.08	49552.92
10	38.53	29.32	67.86	131.38	1361.46	60463.76
9M	38.05	29.32	67.37	123.67	1485.13	73806.06
9	37.58	29.32	66.91	116.24	1601.37	86964.34
8M	37.03	29.32	66.36	115.41	1716.78	102657.77
8	36.50	29.32	65.82	120.96	1837.74	117885.62
7M	35.86	29.32	65.18	119.65	1957.39	135895.43
7	35.23	29.32	64.55	121.17	2078.55	153237.90
6M	34.43	29.32	63.75	118.53	2197.09	174480.73
6	33.67	29.32	62.99	125.05	2322.13	193529.47
5	32.50	29.32	61.83	136.85	2458.99	220234.01
4	31.15	29.32	60.48	141.36	2600.35	247282.87
3	29.09	29.32	58.41	136.53	2736.87	280437.29
2	26.34	29.32	55.66	123.20	2860.07	310542.89
Ground (1)	8.22	29.32	37.54	0.00	**2860.07**	**343433.70**

4. Seismic Load Calculations and Diagram:

Discussion

Seismic Loads on a structure can also be critical design loads, just like wind loads. It is necessary to find the seismic forces acting on the structure to prevent catastrophe during an earthquake occurrence. The location of the Northwest Building in not is a seismic zone that provides much concern. However, due to the amount of time, cost, and work put into the design, it is necessary to analyze the seismic loads and understand their impact on the structural design.

Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Calculation Data:

- Spectral Response Accelerations (S_s & S_1)
  - S_s = 0.365
  - S_1 = 0.071
- Soil Site Class B
- Seismic Design Category (SDC) B
- Calculated Total Building Weight (W) is 21,724.25 Kips

Tables and Figures:

- Figure 17: Seismic Load Diagram
  - Provides a visual of the seismic forces in kips per level. Both the story forces and story shear forces are given, along with the total base shear.
- Table 8: Total Building Weight Calculation
  - Provides the excel spreadsheet that documents the calculations made to find a reliable estimate of the total building weight. This total building weight is used in finding seismic loads.
- Table 9: Seismic Forces Calculation
  - Provides the excel spreadsheet that documents the calculations made in distributing the seismic loads to each floor of the structure.

Conclusions:

The seismic loads were expected to be a large fraction less than the calculated wind loads. Upon analysis, this is exactly what was determined. A base shear of 217.2 kips was found. Comparing this value to the wind base shears of 725.41kips and 2860.07 kips, it can be seen as less significant. Please reference the tables and diagrams for additional information.

Figure 17: Seismic Load Diagram

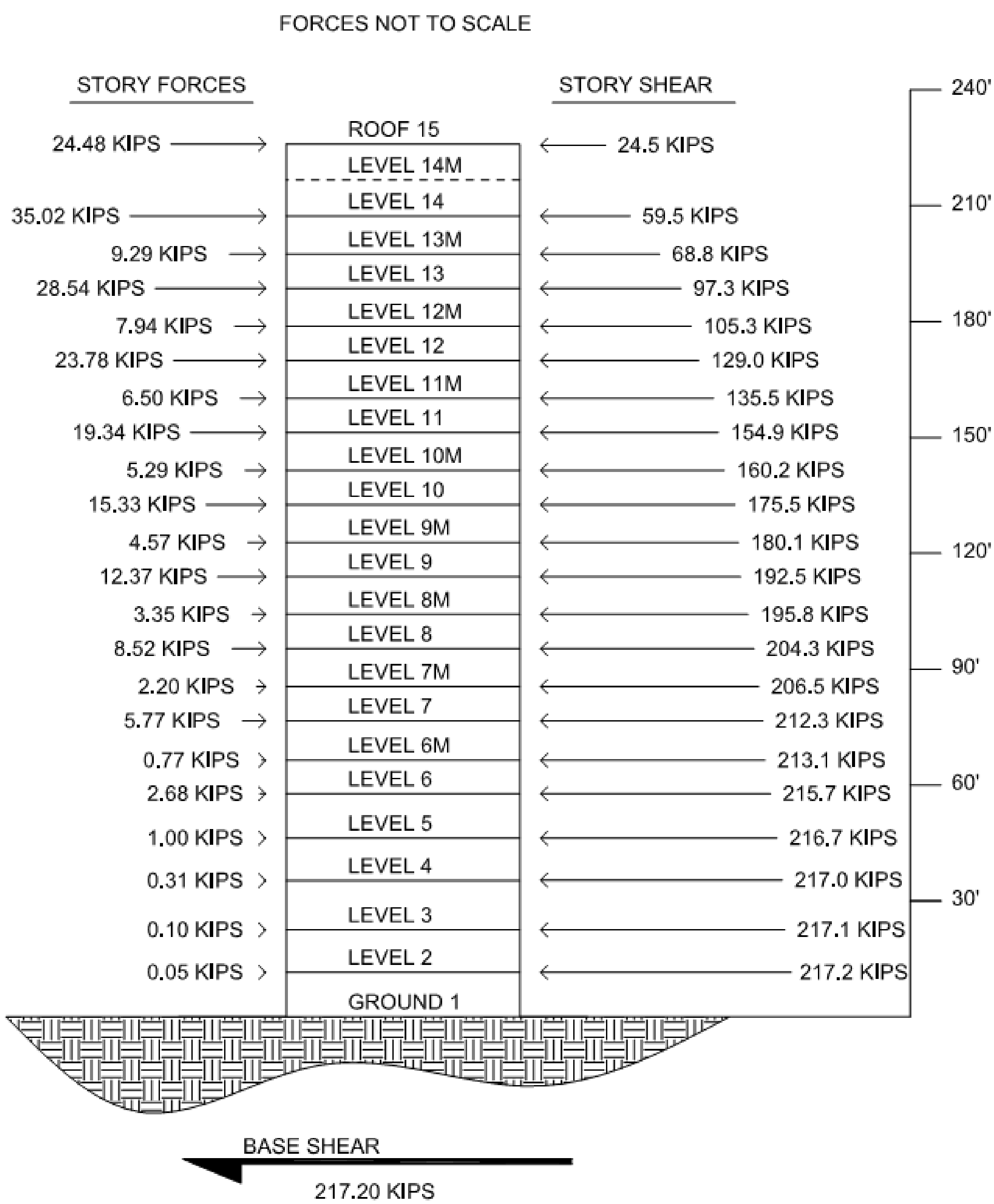


Table 8: Total Building Weight Calculation

LEVELS	Floor Slab (kips)	Partitions (10 PSF)	MEP (8 PSF)	Beams	Columns (kips)	Façade	AHUs (kips)	Mechanical Misc. Equipment (kips)
Level 1.5-2					36.49	104.65	185.5	21.176
Level 2	310.97	41.46	33.17	19.80				
Level 2-3					75.77			
Level 3	133.20	17.76	14.21	13.63				
Level 3-4					96.50			
Level 4	153.07	38.27	30.61	42.00				
Level 4-5					82.60			
Level 5	482.11	64.28	51.43	45.51				
Level 5-6					95.30			
Level 6	892.75	119.03	95.23	99.82				
Level 6M	119.26	21.68	17.35	12.73				
Level 6-7					214.20			
Level 7	1196.39	119.64	95.71	174.63				
Level 7M	307.85	43.90	35.12	23.26				
Level 7-8					213.20			
Level 8	1190.34	119.03	95.23	174.63				
Level 8M	331.85	47.90	38.32	25.38				
Level 8-9					211.70			
Level 9	1190.34	196.75	95.23	174.63				
Level 9M	335.85	48.90	38.32	26.89				
Level 9-10					211.70			
Level 10	1190.34	119.03	95.23	167.64				
Level 10M	307.85	43.90	38.32	20.41				
Level 10-11					169.36			
Level 11	1190.34	119.03	95.23	165.90				
Level 11M	307.85	43.90	38.32	20.14				
Level 11-12					149.20			
Level 12	1190.34	119.03	95.23	162.41				
Level 12M	307.85	43.90	38.32	20.14				
Level 12-13					149.20			
Level 13	1190.34	119.03	95.23	157.17				
Level 13M	307.85	43.90	38.32	20.14				
Level 13-14					125.22			
Level 14	1190.34	119.03	95.23	153.67				
Level 14-15					120.51			
Level 15	892.75	0.00	38.32	64.61				
TOTALS	14719.80	1649.38	1307.65	1785.1437	1950.95			
TOTAL BUILDING WEIGHT (KIPS)	21724.25							

Slabs	Weight (PSF)
D1	75
D2	100
D3	100
D4	80
D5	60
D6	40
D7	55
D8	80
D9	45

Table 9: Seismic Forces Calculation

LEVELS	Height (Feet)	Floor Weight	$w_x h_x^k$	$w_x h_x^k / \Sigma w_i h_i^k$	$w_x h_x^k / \Sigma w_i h_i^k * V$ (Story Force, kips)	Story Shear (kips)
Level 2	11.50	461.04	39283.32	0.0002	0.049	217.200
Level 3	22.50	273.72	79118.03	0.0005	0.099	217.151
Level 4	35.25	379.60	248399.40	0.0014	0.311	217.052
Level 5	46.25	745.08	799290.01	0.0046	1.002	216.740
Level 6	57.75	1329.09	2135882.63	0.0123	2.677	215.739
Level 6M	66.42	297.27	616209.02	0.0036	0.772	213.062
Level 7	76.64	1711.42	4603256.76	0.0266	5.769	212.290
Level 7M	85.50	535.88	1758922.93	0.0101	2.204	206.521
Level 8	95.30	1699.23	6795181.99	0.0392	8.516	204.317
Level 8M	104.17	568.45	2672912.85	0.0154	3.350	195.801
Level 9	113.97	1781.95	9868547.86	0.0569	12.367	192.451
Level 9M	122.83	574.96	3648989.41	0.0211	4.573	180.084
Level 10	132.63	1676.07	12232106.73	0.0706	15.329	175.511
Level 10M	141.50	514.31	4222820.88	0.0244	5.292	160.182
Level 11	151.30	1664.25	15435648.01	0.0891	19.344	154.890
Level 11M	160.17	503.96	5184833.30	0.0299	6.498	135.547
Level 12	169.97	1655.76	18979016.06	0.1095	23.784	129.049
Level 12M	178.83	503.96	6336341.03	0.0366	7.941	105.265
Level 13	188.63	1643.53	22771323.67	0.1314	28.537	97.324
Level 13M	197.50	491.97	7410913.73	0.0428	9.287	68.787
Level 14	207.33	1697.93	27941252.02	0.1612	35.016	59.500
Level 15	226.00	1014.84	19537716.91	0.1127	24.484	24.484
Totals		21724.25	173317967	1	217.2	

5. Snow Load Calculation:

Discussion

Snow load needs to be considered for both the gravity and lateral design of the structure. If the snow load is greater than 30 PSF, then it should be included in the total building weight of the structure when performing seismic calculations. Therefore, the snow load for the Northwest Building was calculated to see the effects on the gravity and lateral designs.

Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Calculation Data:

- Flat Roof
- Importance Factor = 1.15
- Ground Snow Load is 25 PSF.

Conclusions:

A snow load of 20 PSF was found for the Northwest Building. This load is not greater than 30 PSF therefore the snow load does not need to be considered in the seismic calculations. However, 20 PSF will impact the gravity system, especially sizing the members for the roof.

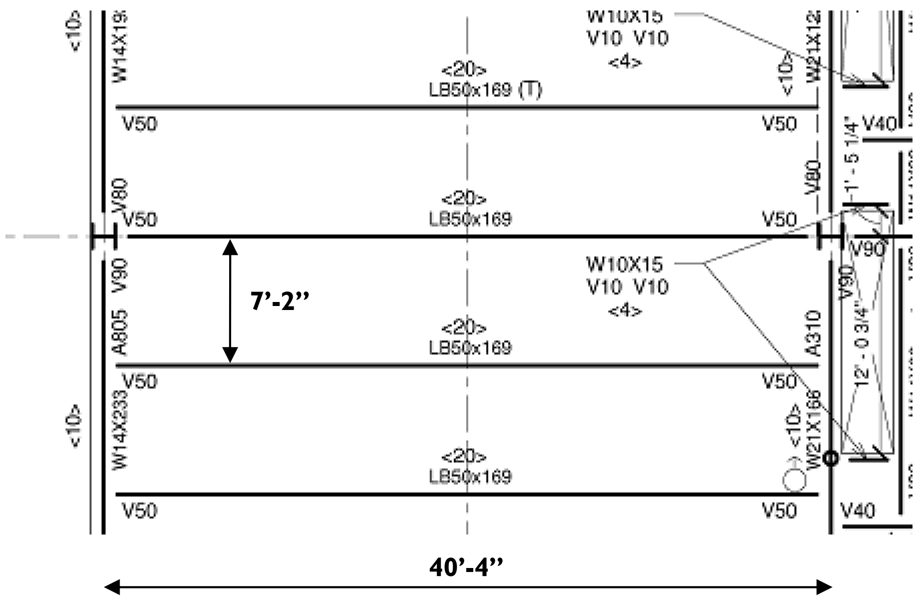
V. Spot-Checks of Typical Framing Elements:

1. Cellular Beam Check with 6" Normal Weight Concrete topping on Metal Decking (Level 900)

Discussion

Since, there is a large amount of laboratory space on almost every level above level 600; a spot check of one of the main cellular beams was performed. These beams span 40'- 4" and have 6" normal weight concrete topping on metal deck, spanning from beam to beam. See *Figure 18*, "Beam Spot Check Snapshot", below.

Figure 18: Beam Spot Check Snapshot



Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Calculation Data

- Tributary Length = 7'-2"
- Dead Load on Beam = 1015 lbs/ft (includes slab, MEP, partitions, & beam self weight)
- Live Load on Beam = 100 PSF = 717 lbs/ft
- Load Combination 1.2D + 1.6L

Conclusions

The moment capacity, deflection, and shear capacity were all spot checked for this calculation. Initially, all three spot checks show that this floor system was overdesigned. This can be due to conservative assumptions made during calculations. From the spot checks made, it can be concluded that shear governs the cellular beam design due to its small cross sectional area at its cells. However, the member connection may also be a governing factor in design, which was not part of the scope of this report.

2. Column C4 at Level 100

Discussion

This column is very critical due to the 126-clear span gravity load being distributed over to this edge column. The column in question is a W14x730. Due to its extremely large size, it is already seen that it takes a very large gravity load. See *Figures 19 & 20* below.

Figure 19: Snapshot Floor Location of Column C4

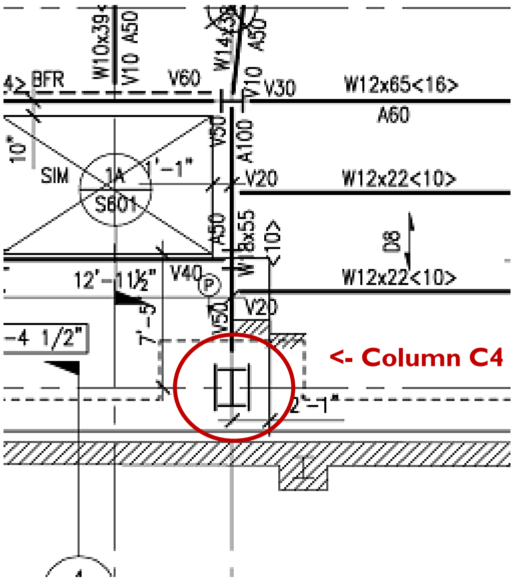
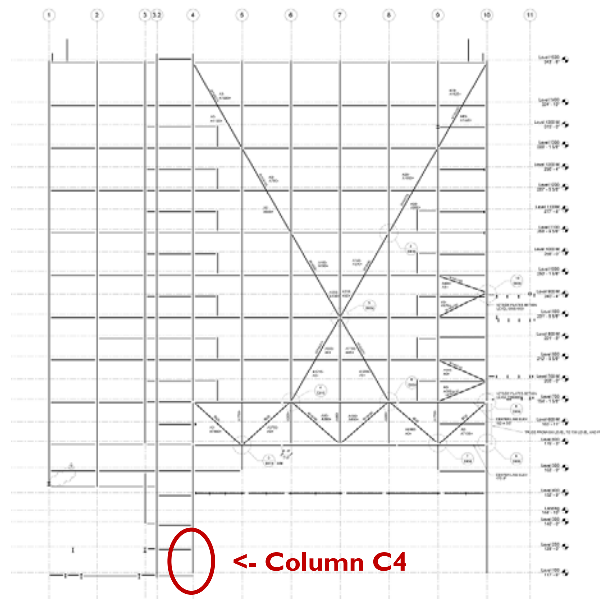


Figure 20: Elevation Location of Column C4



Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Calculation Data

- Height of Column = 11'-6"
- P = 7232 KIPS (total dead and live load force acting on column)
- Load Combination 1.2D + 1.6L

Conclusions

The compressive strength for flexural buckling was checked for this column. A nominal capacity of 9081.7 kips was found. When you compare this to the required capacity of 7232 kips it is an acceptable design. It has a safety value factor of 1.26.

3. Truss Diagonal Member on Frame Grid A

Discussion

The diagonal members of the trusses are very critical in design. These trusses have a large span and help support 10 additional stories above. Therefore, the truss design is very critical for support of the structure. One of these truss members has been chosen to be a spot check. A tensile strength and rupture spot check has been performed. See *Figures 21 & 22* for the location of this spot-check.

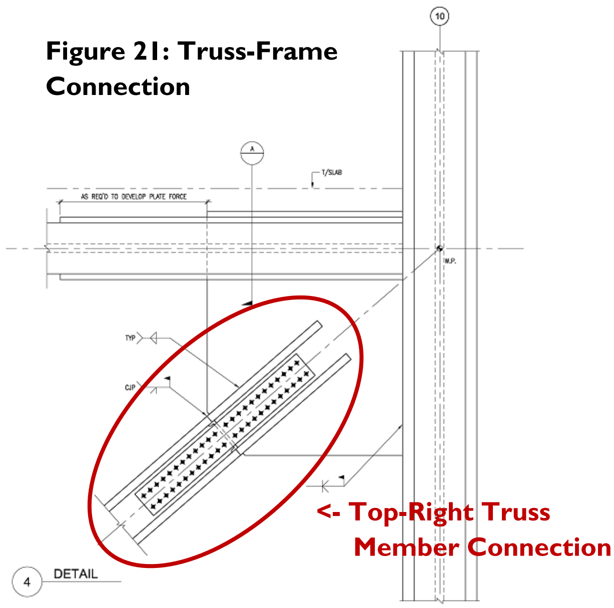


Figure 21: Truss-Frame Connection

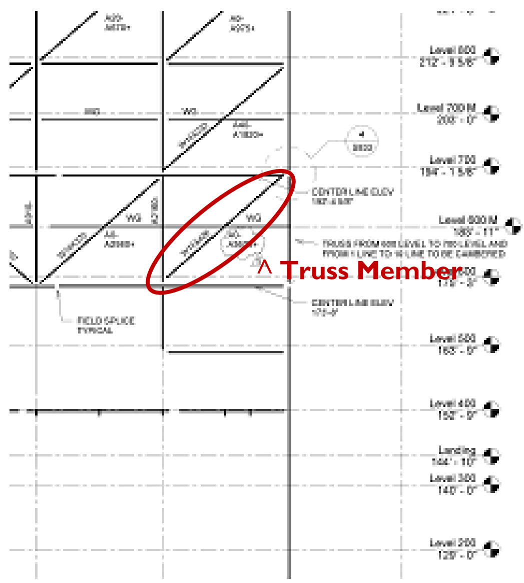


Figure 22: Elevation Snapshot of Truss Member Location

Please Note: Additional hand calculations are provided in the appendix section at the end of this report.

Calculation Data

- T = 2625 KIPS (total dead and live load axial force acting on member)
- Load Combination 1.2D + 1.6L
- ¾" Bolts Used

Conclusions

This spot check came up with a controlling tensile strength of 5625 kips. This tensile strength is a factor of 2.14 greater than the required capacity. This factor seems a bit high. This might be due to other governing factors of the connection design. Additional, in-depth connection strength calculations were not able to be performed due to lack of shop drawing information.

Evaluation and Summary:

For a summary on the structural system literature please see page 16.

The following evaluation and summary will conclude on the wind, seismic, and spot check calculations along with a brief conclusion on Technical Report 1.

Wind, Seismic, and Spot Checks

When comparing the final wind and seismic results and it is clear that wind design controls the lateral design of the structure. The buildings large 196 foot long East and West facades serve as a large wind collectors. The location of the structure in a coastal area causes an increase in wind speed, which is also a contributing factor. If the building was located on the west coast, perhaps seismic could govern the lateral design. However, the structure is lightweight due to its steel design and seismic still might not govern over wind.

Initially, the spot check calculations give a feeling that the structure has been overdesigned. All of the spot check calculations have came up with results of overdesign concerns. However, it is believed that this is not the case. Other contributing factors still need to be assessed. These factors deal with connection design and other limiting states. These spot check calculations did give great insight to composite floor, cellular beam, truss member, and column compressive design.

Technical Report 1

This report started out with a broad understanding of the structure's gravity and lateral system design. From this understanding, calculations were made to acknowledge the forces that have a contributing design impact upon the structure. Finally, spot-check calculations were made to get a closer look at what was involved in detailed design.

Appendix:

(Hand Calculations)

WIND PAGE 1

WIND CALCS: (INITIAL WIND PARAMETER FINDINGS)

- OCCUPANCY CATEGORY — III
- BASIC WIND SPEED — V = 110 MPH (NY, NY)
- $K_d = 0.85$
- $I = 1.15$
- $K_{zt} = 1.0$
- EXPOSURE CATEGORY C
- $G = 0.85$ (ASSUME RIGID STRUCTURE $I_c >>> I_b$)
- C_p → SEE EXCEL PRINTOUT

BUILDING ENCLOSED — YES

↳ PARAPET — YES

↳ VELOCITY PRESSURE q_p

$V = 110$ MPH
$K_d = 0.85$
$I = 1.15$
EXP. CAT C
$K_{zt} = 1.0$
q_z, K_z, K_H, q_h → TABULATED IN EXCEL SPREADSHEET

↳ $GC_{pn} = +1.5$ WINDWARD
$GC_{pn} = -1.0$ LEEWARD

↳ $p_p = q_p GC_{pn}$

$p_{p\,WINDWARD} = 1.5(45.64) = 68.46$
$p_{p\,LEEWARD} = -1.0(45.64) = -45.64$

WIND PAGE 2

→ NOT A LOW RISE BLD.

→ $n_1 = 100/226 = 0.4425 < 1$
FLEXIBLE

→ $q_z + q_h$ → TABULATED ON EXCEL SPREADSHEET

→ Cp

N-S — Cp WINDWARD = 0.8
Cp LEEWARD = −0.225

E-W — Cp WW = 0.8
Cp LEEWAR = −0.5

→ $GC_{pi} = \pm 0.18$

→ DESIGN WIND PRESSURES
→ TABULATED IN EXCEL

EXCEL PRINTOUTS
→ CONTAIN
↓
K_z, q_z, K_h, q_h, PSF WINDWARD, PSF LEEWARD, TOTAL PSF, STORY FORCE, STORY SHEAR, OVERTURNING MOMENT

FOR BOTH N-S + E-W DIRECTIONS.

SEISMIC

SEISMIC PAGE 1

SEISMIC GROUND MOTION VALUES:

USGA WEBPAGE

$S_S = 0.365$

$S_1 = 0.071$

$S_S > 0.15 \quad S_1 > 0.04$

$S_1 < 0.6$

SITE CLASS OF SOIL:

SITE CLASS B (ROCK)

S_{MS} + S_{M1}: $F_a = 1.0 \quad F_v = 1.0$

$S_{MS} = F_a S_S = (1)(0.365) = 0.365 = S_{MS}$

$S_{M1} = F_v S_1 = (1)(0.071) = 0.071 = S_{M1}$

S_{DS} + S_{D1}:

$S_{DS} = 2S_{MS}/3 = 2(0.365)/3 = 0.2433$

$S_{D1} = 2S_{M1}/3 = 2(0.071)/3 = 0.0473$

AVERAGE SHEAR WAVE VELOCITY: ($\bar{v}_s$)

2,500 to 5,000 FT/S $= \bar{v}_s$

SEISMIC PAGE 2

SEISMIC DESIGN CATEGORY:

$S_S = 0.365$ $S_1 = 0.071$

$S_{DS} = 0.2433$
$S_{D1} = 0.0473$

• OCCUPANCY CATEGORY III

∴ SEISMIC CATEGORY B = SDC

DIAPHRAGM FLEXIBILITY:

FLEXIBLE

12.8 Equivalent LATERAL FORCE PROCEDURE:

$S_S = 0.365$ $S_1 = 0.071$ $S_{DS} = 0.2433$ $S_{D1} = 0.0473$

RESPONSE MODIF. COEFF → $R = 3.0$

(ORDINARY STEEL + CONCRETE COMPOSITE BRACED FRAMES)

$I = 1.15$

FUND. PERIOD OF STRUCT. (T_a)

$C_t = 0.028$

$x = 0.8$

$T_a = C_t h_n^x$

$(0.028)226^{0.8}$

$T_a = 2.14$

$T_L = 6$ $T_a < T_L$

FIND CS:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.0473}{2.14(3/1.15)} = 0.00847 \le \frac{S_{DS}}{(R/I)} = 0.093$$

$C_S < 0.01 \rightarrow \therefore C_S = 0.01$

SEISMIC PAGE 3

DETERMINE SEISMIC WEIGHT:

W INCLUDES: ✓• TOTAL DEAD LOAD
✓• STORAGE AREAS → 25% OF FLOOR LIVE LOAD
✓• ACTUAL PARTITION WEIGHT OR 10PSF OF FLOOR AREA
✓• WEIGHT OF PERMANENT EQ.
✓• IF Ps exceeds 30PSF
↳ USE 20% OF SNOWLOAD.
↳ DOES NOT EXCEED

COLUMNS

LEVEL 1.5-2
W14x61 II
W14x605 II
W14x665 I
W14x500 II
W14x730 I
W14x550 II
W14x257 I
W14x455 I
W14x233 I
W14x730 I
W14x342 I

h(L1.5-2)
$6346\ lb/FT \cdot 5.75\ FT$
$= 36489.5\ lbs = 36.49^k$

LEVEL 2-3
W14x61
W14x550
W14x730
W14x500
W14x665
W14x500
W14x730
W14x342
W14x455
W14x233
W14x550
W14x730
W14x342
W14x500

h(L2-3)
$6888\ lb/FT \cdot 11\ FT$
$= 75768\ lbs = 75.77^k$

SEISMIC PAGE 4

COLUMNS

LEVEL 3-4:

W14X61 W16X665 W14X233
W14X550 W14X500 W14X233
W14X730 W14X730 W14X605
W14X500 W14X342 W14X730
W14X211 W14X426 W14X342
W14X211 W14X500

7569 lb/FT · 12.75 (L3-4)h = 96.5k

LEVEL 4-5:

7508 lb/FT · 11 = 82.6k (L4-5)

LEVEL 5-6:

580 500 730 132 730
43 665 342 193 342
43 342 176 233 500
730 500 126 605 159
43 426 43
43 43

8289 lb/F · 11.5 (L5-6)h = 95.3k

LEVEL 6-7:

43 233 730 283 283 176 211 193 730
43 233 500 109 730 159 211 176 342
550 283 665 120 342 233 605 233 500
311 311 342 109 193 159 311 455 233

11340 lb/FT (L6-7) × 18.89 = 214.2k

LEVEL 7-8:

11340 + 43 + 43 = 11426 lb/FT × 18.66 = 213.2k

LEVEL 8-9:

11340 × 18.67 = 211.7k

LEVEL 9-10: 211.7k

LEVEL 10-11: 169.36k

LEVEL 11-12: 149k

LEVEL 12-13: 149k

LEVEL 13-14: 125k

SEISMIC PAGE 5

$V = C_s W = 0.01(21724.25^k) = 217.2^k$

DISTRIBUTE FORCE TO FLOORS!

$k = 0.75 + 0.5(2.14) = 1.82$

$$F_x = \frac{w_x h_x^{1.82}}{\sum w_i h_i^{1.82}} V$$

SEE EXCEL SPREADSHEAT.

SNOW PAGE 1

SNOW LOAD CALCULATION:

$P_g \rightarrow$ (FIG 7-1) = 25PSF

FLAT ROOF SNOW LOAD

$P_f = 0.7 C_e C_t I p_g$

$I = 1.15$
$P_g = 25PSF$

$C_t \rightarrow$ TABLE 7-3 = 1.0
$C_e \rightarrow$ TABLE 7-2 = 0.9

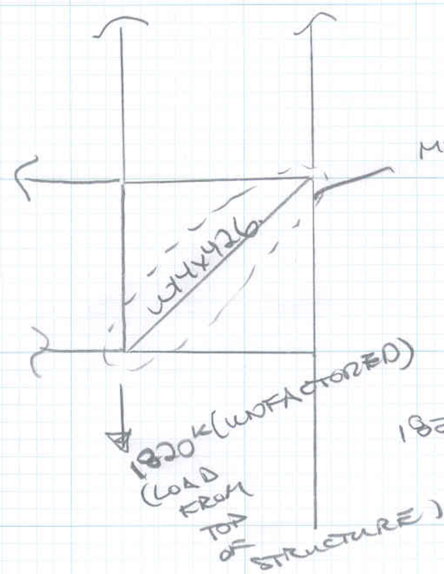
$P_f = 0.7(0.9)(1.0)(1.15)25 = 18.2PSF$

$P_f \approx 20PSF$

$P_f < 30PSF$ ∴ DO NOT NEED TO CONSIDER IN SEISMIC CALCULATIONS.

SC TRUSS PAGE 1

CHECK TRUSS CONNECTION OF DIAGONAL MEMBER:



MEMBER ∴ IN TENSION
1820k

ASSUME 40% D + 60% L.

1820

$1820(0.4)(1.2) + 1820(0.6)(1.6)$

$T = 2625^k$

TENSILE STRENGTH:

$P_n = F_y A_g = 50(125) = 6250^k(0.9) = \phi P_n$

$\phi P_n = 5625^{kips}$

TENSILE RUPTURE: ASSUME 3/4" BOLT HOLES.

$P_n = F_u A_e = 65(125 - 2(3/4 + 1/8)(1.75)) = 7925.9^{kips}$

$\phi P_n >> T$

FACTOR OF SAFETY = 2.14k

∴ MUST BE OTHER CONNECTION FACTORS DEALING W/ DESIGN.

WILL NEED SHOP DRAWINGS FOR OTHER CALCS.

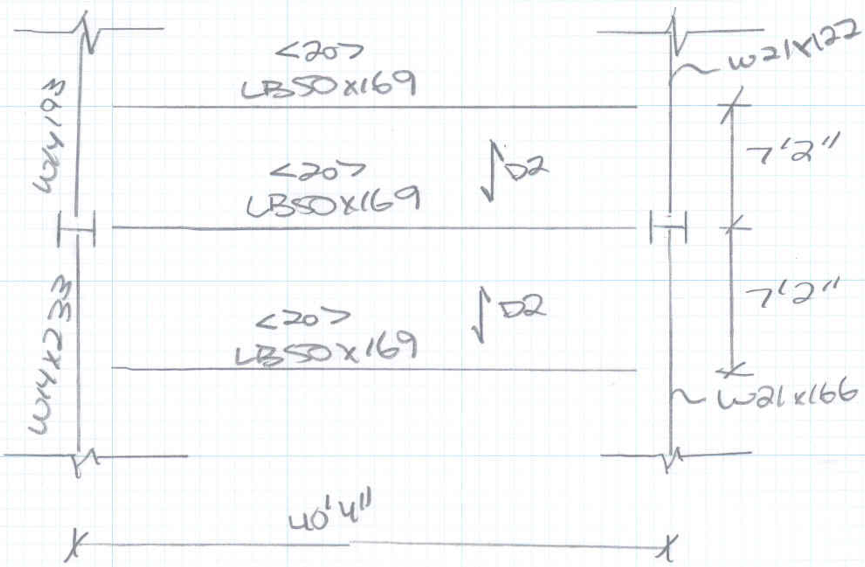
HOWEVER MEMBER TENSILE STRENGTH IS OKAY!

FLOOR SC PAGE 1

FLOOR SPOT CHECK:

LEVEL 9

GIVENS:



D2 SLAB: 6" NORMAL WEIGHT CONCRETE TOPPING ON 2"-18 GA COMPOSITE METAL DECK (8" TOTAL SLAB THICKNESS) REINF W/ 4x4 -W2.0x2.0

WEIGHT OF D2 = 100 PSF

TRIB AREA ON LB50x169 → 7'2"

DEAD LOAD = 100 PSF + 8 PSF + 10 PSF = 118 PSF
(SLAB) (MEP) (PARTITIONS)

$w_D = 846$ lbs/ft + WEIGHT OF LB50x169 = 1015 lbs/ft

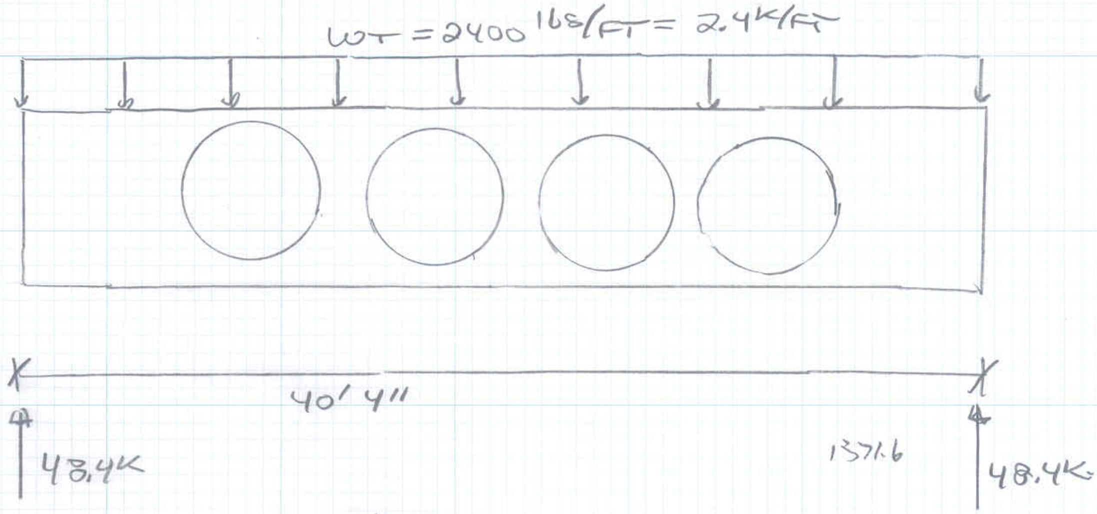
LIVE LOAD → 100 PSF x 7'2" = w_L = 717 lbs/ft

LOAD COMBO: 1.2D+1.6L (ASSUME WILL CONTROL FOR GRAVITY ANALYSIS)

$1.2(1015) + 1.6(717) = 2365.2 \rightarrow 2400$ lbs/ft $= w_T$

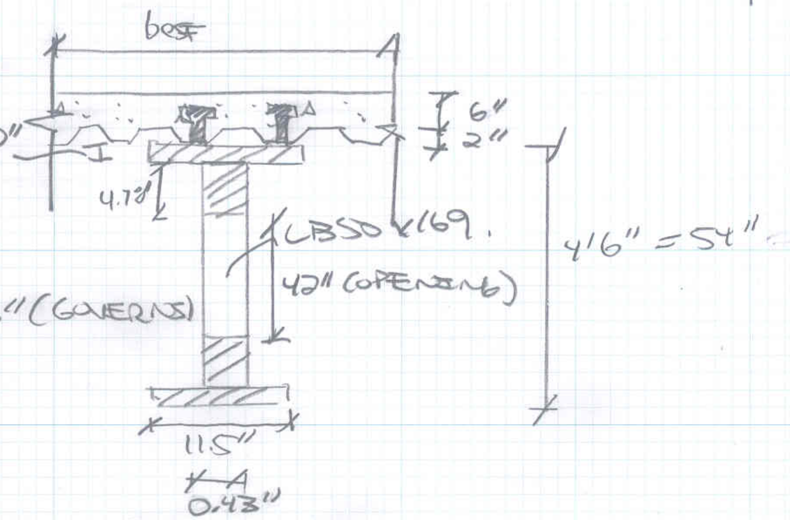
FLOOR SC PAGE 2

ELEVATION OF LB350X169 @ LEVEL 9



COMPOSITE

FIND beff: 1.22"



beff ≤ S
≤ 44
≤ 7.16' = 86" (GOVERNS)
≤ 121"

beff = 86"

STRENGTH OF CROSS SECTION:

$C_c = 0.85(f'_c)A_c = 0.85(6^{ksi})(6")(86") = 2631.6^k$

$T_s = A_s F_y = (50)[(4.78)(0.43)(2) + (1.22)(11.5)(2)] = 1608.5^k$

NA MOVES UP.

$0.85(6)(h)(86) = 1608.5^k$

$h = 3.67"$

FLOOR SC PAGE 3

$C_c = 0.85(6)(3.67'')(86) = 1608^k$

$T_s = 50A_s = 1608^k$

$M_n = 1608(6 - \frac{3.67}{2}) + 1608(54/2)$

$0.85(6)(86)a = 1608 \qquad a = 3.66$

$M_N = 50113.32^{IN \cdot K} = 4176.11^{FT \cdot K}$

$\phi M_N = 4740^{FT \cdot K}(0.9) = 4266.5^{FT \cdot K}$

$M_u = \frac{200(40.333 \times 12)^2}{8}$ (165/IN) $= 5856400^{lb \cdot IN} = 488^{K \cdot FT}$

$4176.11 >>> 488$ OKAY!!

∴ MUST NOT BE EXPECTING FULL COMPOSITE ACTION.

→ CHECK MEMBER ITSELF.

$M_{n\,LB\,W55 \times 169} = 701.5(54/2 - 1.22/2)^2 + 102.77(54/2 - 1.22 - 4.78/2)$

$M_N = 41832.75^{IN \cdot K}$

$C_{F_T} = 50(1.22)(11.5) = 701.5^k$

$C_{W_T} = 50(0.43)(4.78) = 102.77$

$3486.06 >>> 488$

– CHECK OTHER FACTORS →

FLOOR SC PAGE 4

DEFLECTION:

– DISREGARD COMPOSITE BEAM FOR CALCULATION — CONSERVATIVE

$$\Delta_{MAX} = \frac{5wl^4}{384EI} = \frac{5(200^{lbs/in})(40.33\times12)^4}{384(29000^{k/in^2})(29480 in^4)}$$

$$I = 2(1.22)(11.5)\left[\frac{51.56}{2} + \frac{1.22}{2}\right]^2 + 2(4.78)(0.43)\left[49.17\right]^2$$

$$19541.9 + 9938.6$$

$$I = 29480.5 in^4$$

$$\Delta_{MAX} = 0.2'' < l/360 \Rightarrow \text{okay!}$$

SHEAR:

$$V_{MAX} = 48.4^k$$

$$V_n = 0.6F_yA_wC_v$$

$$0.6(50)(4.78 \cdot 0.43 \cdot 2)(1)$$

$$V_n = 123.32^k$$

$$\phi V_n = 110^k > V_{MAX}$$

okay!

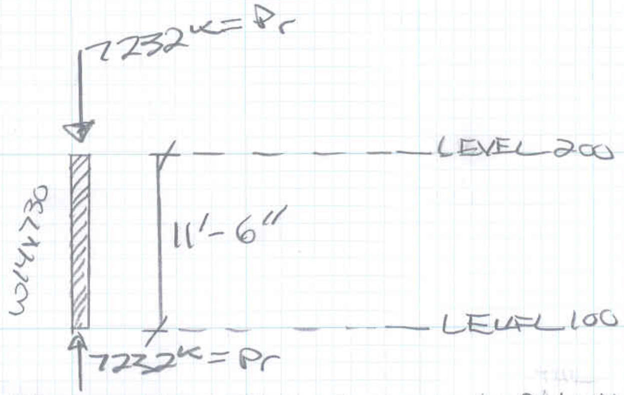
THIS FLOOR CHECK IS OKAY!

NOTE OTHER CONTRIBUTING FACTORS OF LATERAL DESIGN MAY CONTROL DESIGN. ALONG w/ CONNECTION DESIGN.

COLUMN SC PAGE 1

COLUMN C4 SPOTCHECK:

GIVENS:



DUE TO COMPLEXITY OF STRUCTURE AND LOAD PATH OF THE STRUCTURE THE TOTAL LOAD ON COLUMN C4 @ LEVEL 1 WAS TAKEN FROM THE DESIGN DOCUMENTS TO BE 7232ᴷ TOTAL.

NOTE IT WAS ALSO ASSUMED THAT 7232ᴷ WAS DETERMINED FROM A LOAD COMBO OF 1.2L + 1.6L.

$\phi P_N = 9080^k$ (AISC 4-10) $\quad \phi = 0.9$

COMPRESSIVE STRENGTH CHECK FOR FLEXURAL BUCKLING:

$P_n = F_{CR} A_g$ ASSUME K = 1.0

$$KL/r = \frac{(1.0)(11.5)(12)}{4.69} = 29.42 \leq 4.71\sqrt{\frac{29000}{50}} = 113.43$$

$$F_{CR} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = 46.93^{ksi}$$

$$F_e = \frac{\pi^2 (29000)}{(29.42)^2} = 330.68$$

$$P_N = (46.93^{ksi})(215^{IN^2}) = 10090.76^k$$

COLUMN SC PAGE 2

$\phi P_n = 0.9(10090.76) = 9081.7^k$

$\phi P_n \geq P_u$

$9081.7^k \geq 7232^k$ OKAY!

SAFETY FACTOR OF $\frac{9081.7}{7232} = \boxed{1.26}$