901 NEW YORK *Avenue*

Senior Thesis Final Report

Complete Analysis of 901 New York Avenue, Spring 2007

901 New York Avenue
Washington, D.C.

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(Structural Option)

901 NEW YORK Avenue

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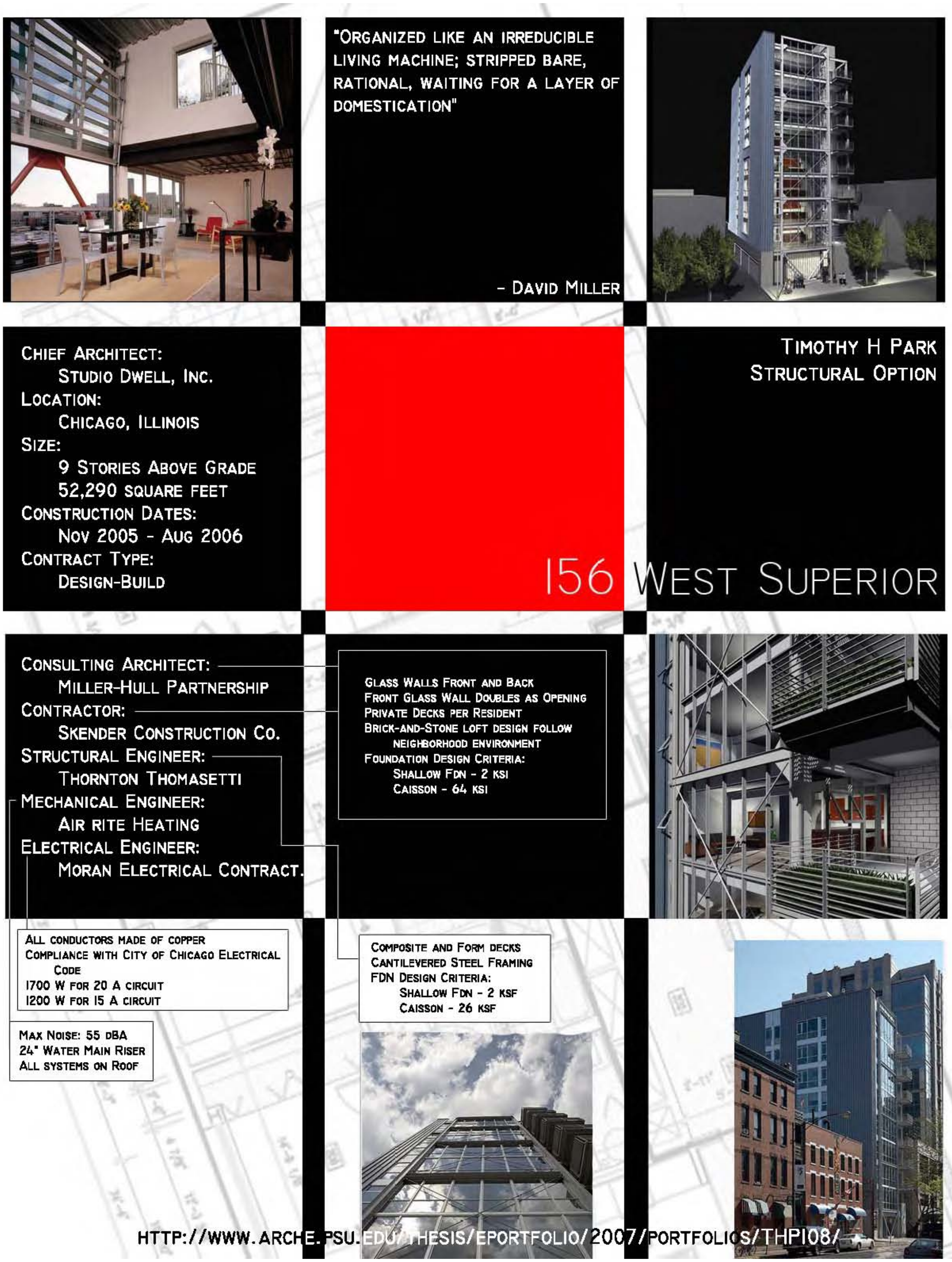
Last but not least, I must thank my parents and my family. Without any of your support, I wouldn't have made it this far. I am forever grateful that you will always be there for me. Thank you for all your prayers.



****Note:* *156 West Superior was the building of choice before 901 New York Avenue***


HTTP://WWW.ARCHE.PSU.EDU/THESIS/EPORTFOLIO/2007/PORTFOLIOS/THP108
Architect:
DCS Design, Ltd
Contractor:
Clark Construction
Structural Engineer:
SK&A Consulting
MEP Engineer:
Girard Engineering
Class A Office Building
Retail, Parking, Office Space
11 Stories Above Grade
4 Stories Below Grade
Custom-Built Glass/Steel Staircase
3-Story Atrium with Truss Facade
580,000 square feet
$54 million
Design-Bid-Build
DCS Design and SKB A&D Interior Designers
2-way Concrete Slab w/PT
20'-0" by 40'-0" bays
Moment Framing Lateral System
VAV Air Duct System
5 Cooling Towers (up to 191,890 cfm)
23 AHU's (up to 40,000 cfm)
3 Chillers (up to 750 tons)
3-phase Power using 4 - 4,000 A boards
2 server cores for west and east cores
Custom Lighting in staircases, atrium
901 NEW YORK Avenue
Timothy H Park
Structural Option

Executive Summary

In this report, 901 NYA has been reviewed for its current design, checked for gravity and lateral strength, and compared to several different alternative solutions, two of which were further analyzed and detailed for a more in-depth comparison with the current system. Furthermore, both alternatives were analyzed with construction in mind. 901 NYA was then also checked to see if there were possibilities of LEEDS certification.

Current Design: The current design is absolutely the best possible design for minimal floor thickness, large bay areas, and open floors. The extra costs that caused 901 NYA to be almost 150% of the typical cost for concrete buildings shows that it was more crucial to have an aesthetically pleasing building than a cost-efficient one.

Composite Alternative: Composite design shows that it is very possible to make steel work even with the tight criteria of long spans. A compromise of 4' shorter spans helps make members an even more realistic possibility. However, with some girders coming out to be almost 3' in depth, it really does depend on whether or not the interior designers would be able to work around those extremely deep beams. Also, even though construction time is cut almost by a quarter, costs may sky rocket. Current economy for steel in D.C. shows that it is very expensive to build with steel. Fabricators may not be available at time of construction as well. In the end, it will depend on the owner's personal desires and his/her concern for time constraint over money.

Shear Wall Alternative: Shear walls proved to be a very possible alternative to the current system. The new system creates a column size that is 38% of the current size, while also reducing the number of reinforcement. Of course, the catch is that those savings counter with the costs of building the shear wall itself. Even then, the proposed alternative saved more than $400,000. Without effecting the construction schedule at all, shear walls could have been a better solution than the current system. Once again, it is dependant on the owner as to whether or not they mind a 10" solid wall system used around their elevator shafts.

LEEDS Certification: 901 NYA was not built with the environment in mind. The current HVAC systems draw an immense amount of power, all rain water is sent directly to sewage, and there is no "greenery" to be seen except for the few isolated trees down New York Avenue itself. With its current system, 901 NYA cannot achieve even the lowest certification that USGBC offers. However, some mild changes (such as parking spots for carpoolers, showers installation for bike riders, etc.) can improve its points rating to being certified, and an extreme makeover (such as a DOAS, turning rain water into gray water, etc.) can allow 901 NYA to improve to even gold certification. It is to the owner's (and tenants) discretion as to how much money they are willing to spend and to what extent they desire to have a LEEDS-certified building.

Table of Contents

A Brief History of 901 New York Avenue

At the turn of the century, Washington D.C. wanted to revitalize the area around the newly built Convention Center, which was located at the heart of the city. In December 2000, Monument Realty sold the triangular-shaped property to Boston Properties for the purpose of constructing a multistory office building. Great controversy surrounded the building, as many thought the property should be used to build a much-needed hotel to support guests coming to the Convention Center. In the end, Boston Properties chose Davis, Carer, Scott to design an 11-story, 530,000 square-foot multi-use facility with a 4-level parking garage underground below the building.

Finnegan, Henderson, Farabow, Garrett & Dunner and Goodwin Procter, LLP are currently the two main tenants in 901 NYA. The main players of the construction of 901 New York Avenue (further annotated as 901 NYA) were the following:

Davis, Carter, Scott, Ltd. – Architect
Clark Construction Group, Inc. – General Contractor
Smislova, Kehnemui, & Associates – Structural Engineer
Girard Engineering, PC – MEP Engineer

The building has many great architectural and structural features. For one, its three-story atrium lobby houses a series of steel trusses for an old rustic feel to the interior entrance space. Architectural pre-cast concrete panels were used on the exterior façade. Complicated structural systems were utilized to create expansive 20'-0" by 40'-0" bays without compromising floor-to-floor thicknesses. Interior designers took advantage of the high ceiling spaces and created innovative and award-winning designs. More unnoticeable architectural features (such as differing heights at each corner of the building up to 8'-0" and creating usable space from acute corners of the building) had to be considered for design.




Goodwin Procter hired SKB Architecture & Design and Clark Interiors to develop their 96,000 square feet of interior space. Holding the floors from six through nine of the building, Goodwin Procter asked that the space offer integration of privacy and community between the workers' spaces. The space demonstrates the firm's concern and responsibility to the environment through the selection of materials used in all the spaces, as well as implying their desire to provide the healthiest environment for those who have the opportunity to work or visit the office space.






FHFGD hired Clark Interiors and DCS Design to design and fabricate their 250,000 square-foot interior space. An incredible 7-story glass-encased staircase is one of the greatest features of the space, which include custom-made light fixtures and full-height curved glass walls with steel and aluminum shoes. It was desired to have an outstanding interior space that showcased the characteristics of the firm. FHFGD's D.C. office has won several awards for interior design and layout.

Boston Properties also purchased a floor space of their own on the fourth floor. The ground level was reserved for retail use, such as white-cloth restaurants and other retail stores.



901 New York Avenue finished construction in 2005 and immediately opened for use.

General Building Summary

Architecture

The architecture of the building focuses on being the feature of the central D.C. area. As an up-and-coming location, the city had desired to revitalize the area with the Convention Center and subsequent hotels and buildings. It was important that 901 New York Avenue stood as a distinguished building in the Mount Vernon Square amongst others in the area.

Granite stone panels encompass the bottom exterior façade, while pre-cast panels face the rest of the building, with occasional aluminum-ribbed panels framing the windows. Canopies, located at the corners of the building and at the main entrances, are made of aluminum and pre-fabricated glass-and-steel components.

The architects had a few criteria that essentially “forced” the design of 901 NYA as what it is. First, they wanted high ceiling spaces (9’-0” per floor). Then, they also wanted large bay areas (20’ by 40’). Finally, they also wanted to have 11 stories above grade (as requested by the owner). For brevity purposes, preliminary assumptions were made that a concrete system had better flexibility than steel, post-tensioning would be required for such heavy loads at such long spans, and shear walls would obstruct too much of the floor space while over-designed columns could possibly take all the lateral loads (thus creating a moment-framing).

Lighting/Electrical Systems

Not much can be said about the lighting and electrical systems in the building. When Clark Construction had finished their part of the job as specified under the contract, it left the building essentially in its structural element plus its shell. Tenants had complete freedom to decide their interior layout and design. Some of the tenants even opted to install their own staircase apart from the building’s original. These staircases, along with many other spaces, contained several custom-built light fixtures and architectural features.

There are 4 main circuit boards in 901 NYA. Tenants requested a network system to fully support an in-house computer technical support center, so two server cores were installed at each west and east sides.

Roof Systems

The roof of 901 NYA houses the majority of all the MEP units. The roof was also designed for tenants to walk around. As such, a 5’-0” high parapet surrounds the perimeter of the building. At one corner of the roof is also a domed glass roof.

Zoning

901 New York Avenue is located in the heart of the city. By zoning classifications, it is square #372, district C-4 (PUD). The building takes up the full lot area, which is 53,252 square feet, and has a height limitation of 130’-0”.

Because the office facility is in the District of Columbia, it was built under the BOCA Building Code (1996). It is specified as a mixed-use facility, non-separated. The type of construction was specified as type 1B: high rise with automatic sprinkler system.

Existing Structural System

Loads and Codes

901 NYA is primarily used as office space for a number of law firms. As a result, the loads on the floors are office space and lobby/corridor loads. Also, to maximize space on each floor without clutter, typical bays were laid out to be 20' by 40'.

Dead Loads Superimposed		
	Finishes	15 psf
	MEP	5 psf
Dead Loads Self-Weight		
	11" slab	137.5 psf
	8" slab	100 psf
Live Loads	SOG	100 psf
	Parking	50 psf
	Office (w/partitions)	100 psf
	Lobby, Corridors	100 psf
	Heavy Mech.	150 psf
	Loading Truck Bay	250 psf

The model code used for the design of 901 NYA was BOCA 1996. Codes in addition to the BOCA code were:

ACI 318-95, 530-95	Reinforced Concrete, Masonry
AISC – 9th Edition	Structural Steel
AWS D1.1-98	Structural Steel Welding

This report shall use the ASCE/SEI 7-05 instead of the 1996 BOCA code for the purpose of the practice of current valid codes in the D.C. area. The following are associated references that are currently the most up-to-date information:

ACI 381-05, 530-05	Reinforced Concrete, Masonry
RS Means Construction Cost Data (2005)	Construction Cost Estimate
LRFD Steel Manual (13rd Edition)	Structural Steel
NDS 1991	Wood Construction
AWS D1.1-98	Structural Steel Welding

The 2005 RS Means Construction Cost Data was used instead of the most recent in order to accurately compare costs to that of the year of finished construction. The building cost $54 million in 2005, but the cost of construction today would be more costly than back then due to different availabilities in both steel and concrete companies.

Foundations

Sub-ground floor, the parking garage's column spacing does not usually span greater than 20'-0", but there are a few that span up to 40'-0". The rest of the building, from ground floor to the top, has a typical 20'-0" by 40'-0" column spacing.

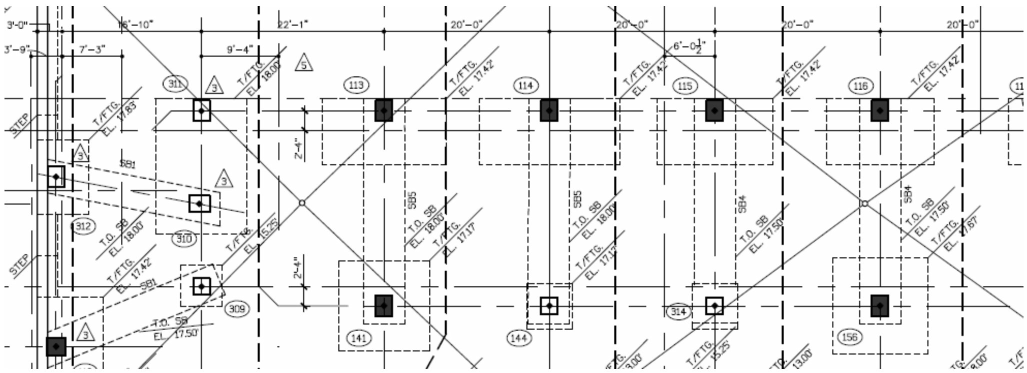


***Figure 1** – Foundation layout (incl. spread footings and strap beams)*

Spread footings were permissible for design of foundations. Pile driving or any other means of foundation support were not necessary due to sufficiently dense soils on site (as claimed in Geotechnical Report). The only consideration was groundwater control. The Geotechnical Report further suggests spread footings with a bearing capacity of 15,000 psf. Concrete for the foundations were poured with a 4,000-psi compressive strength requirement within 28 days. Strap beams were used to tie footings together, while footing sizes varied from 6'-0" by 6'-0 to 16'-0" by 16'-0". Further, the zone of influence is 100 feet, which neglects possible impact on nearby buildings, since the majority of the dewatering settlement will occur within the streets.

Foundation walls are 36" thick throughout the entire perimeter of the building (parking garage level). Slab-on-grade had a thickness of 5" using 6x6-W2.0xW2.0 WWF as reinforcement. Below the S.O.B. was the vapor barrier and 6" of crushed stone. A MAT foundation was found to be unnecessary.

Slabs

The parking garage had the following typical slab system of a 28-day compressive strength test of 5,000 psi at 8". Exceptions are at framed floor slab below columns, which was poured with 8,000 psi concrete. Certain portions of the slab also had a 4" concrete fill on top the typical slab (as noted in darker hatching), while some areas used lighter-weight concrete at 4" (as noted in lighter hatching).

The slabs above the parking garage are typically 11" slabs with the same 5,000 psi concrete. Truck bays have an increased 12" slab with a 4" topping above it. The center of the building had increased loads, so a thicker slab was laid, along with stronger columns. Post-tensioning was used primarily to minimize deflection in the slab.

Columns

Spacing of the building consisted of 20'-0" by 20'-0" in the parking garage and a typical spacing of 20'-0" by 40'-0" from the ground level up. From the fourth level of the parking garage (level P4) up to the second floor, the compressive strength of the columns are designed to be 8,000 psi. The third and fourth floors have strength of 6,000 psi. The fifth and sixth floors have strength of 5,000 psi. The rest of the stories have a compressive strength of 4,000 psi. The reason for the high-strength concrete is due to moment-resisting factors (described in lateral systems).

Typical sizes of the columns are a square 26" by 26", varying mostly only in reinforcement. The bays that actually span the full 20'-0" by 40'-0" spans have a typical 32" by 32" column design throughout the floors. The garage level columns are a bit larger, ranging from 24" by 30" to 24" by 36". They vary in height with an average of about 11'-0" (floor-to-floor height is roughly 11'-8" from the second story up).

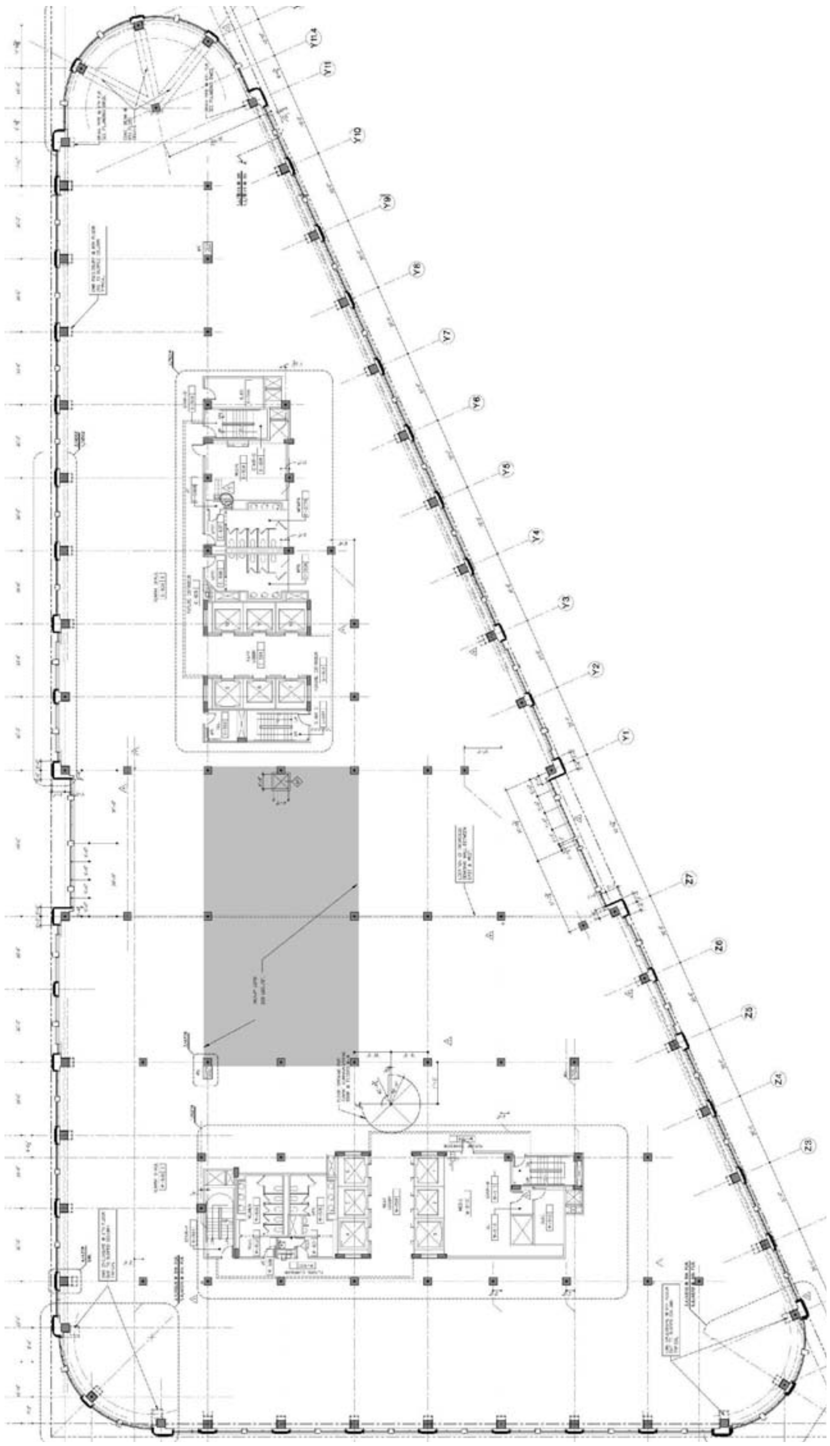


***Figure 2** – Typical Layout of Columns*

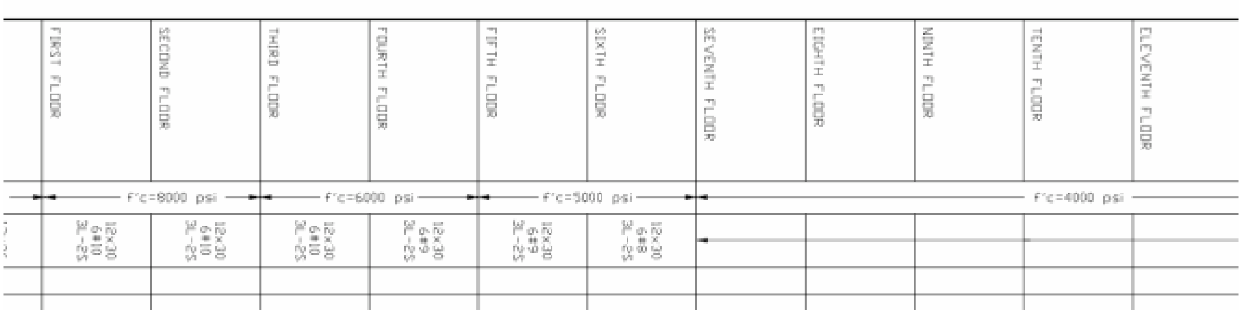


***Figure 3** – Designation of compressive strength of columns*

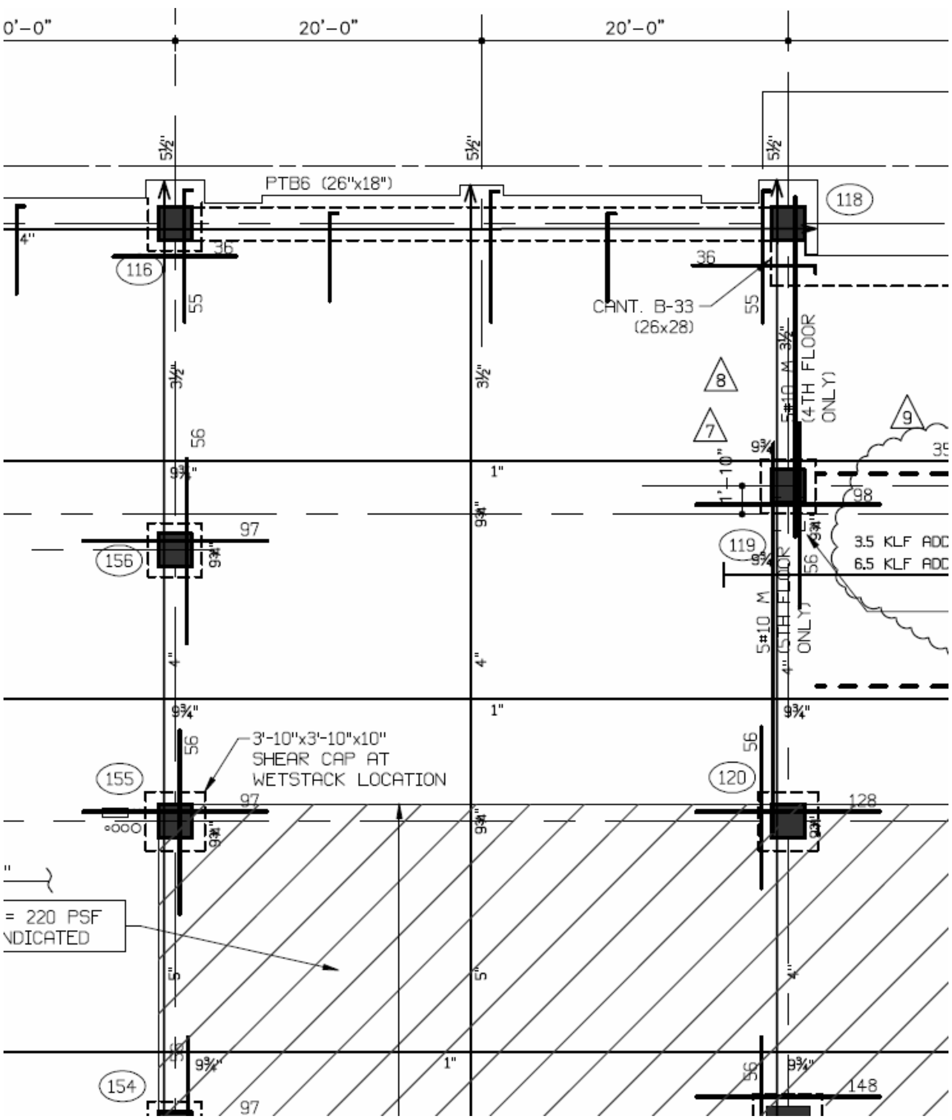


***Figure 4** – Detailed layout of columns and drop panels*

Lateral System
Although most concrete buildings have a set a shear walls to resist lateral loads (including wind and seismic forces), 901 NYA lacks such walls. It was assumed that the design could resist the lateral forces through moment-framing.

Moment framing is the designation of load being transferred from exterior façade to beam and column. Each connection between beam (or slab) and column is to resist the acting forces and moments. Such a system is created through pouring the column and slab monolithically. Columns are usually over designed to be able to withstand the extra moment force.

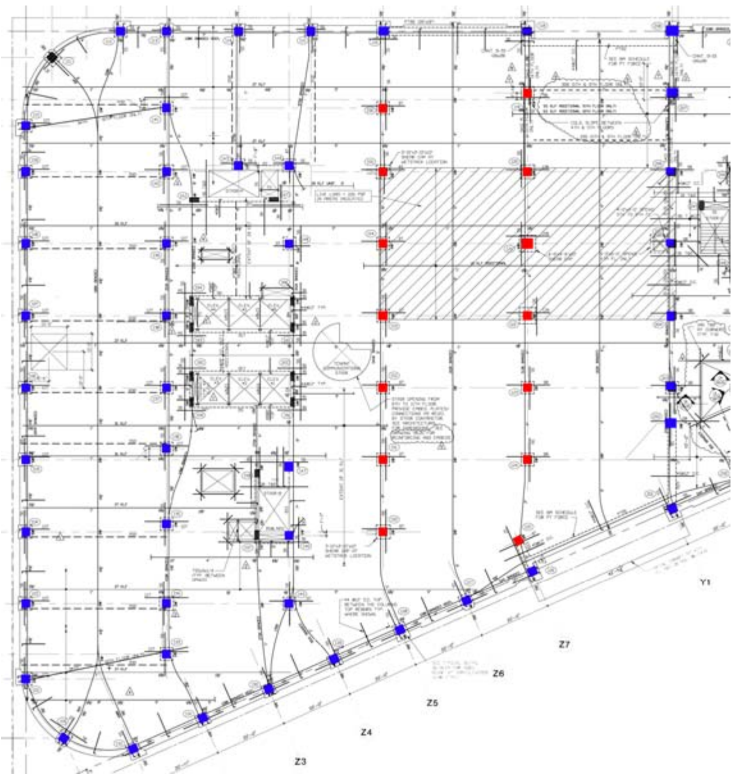


***Figure 5** – Designation of non-typical columns*

The red-spotted columns are higher dimensioned columns in order to take the majority of the central loads (as indicated in the hatched rectangle). All of the lateral forces need to be resisted by the columns, since the 11" slab will not have the "strong beam" classification. Most of the columns will be expected to take on lateral forces, except for those with either a 12"x30" or 14"x30" dimension. These columns are corner columns for the elevator shafts.

Wind and seismic loads will be further addressed in the following section addressed "Lateral Analysis."

Structure Summary

Foundation

Spread footing design – typical size of 8’-0” squares to 16’-0” squares
Columns – 8,000 psi, with occasional sloped columns (no effect on design)
5” S.O.G. with 6x6-W2.0xW2.0 WWF reinforcement
8” elevated slabs
36” perimeter walls

1st Floor – 11th Floor

Columns – from 4,000 psi to 8,000 psi; typical 26” square
11” elevated slabs; thicker slabs at heavier loads (designated at center of building)
Lateral system – resistance through moment-framing
Slab deflection – prevented through post-tensioning

Roof

Holds most of mechanical systems; assume 250 psf dead loads

Exterior Façade

Granite stone on bottom floors; pre-cast concrete on rest of building
Metal canopies on corners, doors, entrances
Mounted on slabs

Lateral Analysis

Wind Loads

Wind loads are a large factor in lateral analysis. It is always between wind and seismic loads that control lateral forces. As the curtain wall system will be connected to the building through the slab, the loads from wind will be transferred from curtain wall to slab to column, distributing to columns through tributary width of the area of the slab.

Wind loads were done without considering quartering winds. When the wind is analyzed on the building, the hypotenuse of the building (the side running alongside New York Avenue) is considered the leeward side of the building. As such, the short side is considered the side wall when the wind is parallel to the short side, and the long side is the side wall when the wind is parallel to the long side.

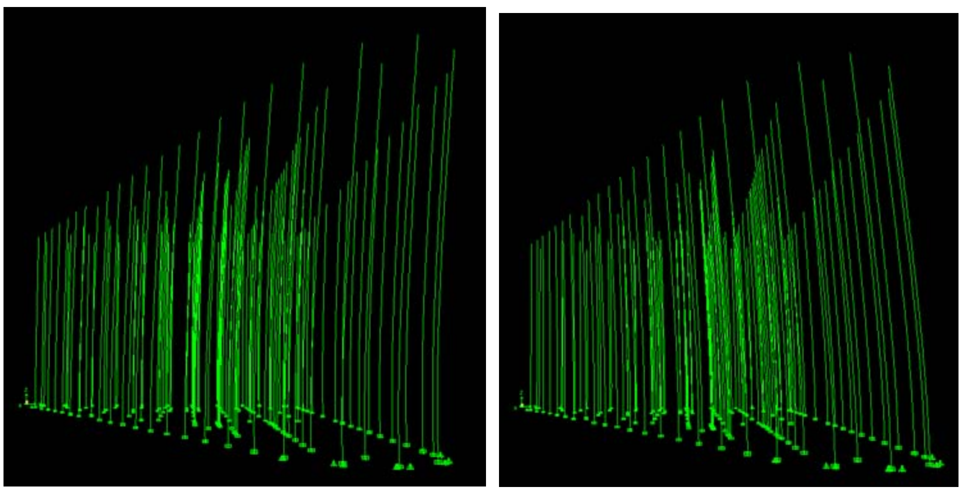


***Figure 6** – Deflection due to wind loads*

All values and calculations were based upon ASCE/SEI 7-05. Basic wind speed was found to be 90 mph, from both the drawing set and figure 6-1 in the code book. Wind forces were found using the following equations:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

C_p values were found to find the windward, leeward, and sidewall pressures. The values varied depending on whether the windward wall was the long side or the short side. Two trials were done to find the controlling pressures, and when the windward wall is the long side of the building controlled. The following tables are a summary of the second trial (windward wall is long side):

	MWFRS		C & C	
Height (ft)	Kz	qz (psf)	Kz	qz (psf)
0-15	0.57	10.0491	0.70	12.3410
20	0.62	10.9306	0.70	12.3410
25	0.66	11.6358	0.70	12.3410
30	0.70	12.3410	0.70	12.3410
40	0.76	13.3988	0.76	13.3988
50	0.81	14.2803	0.81	14.2803
60	0.85	14.9855	0.85	14.9855
70	0.89	15.6907	0.89	15.6907
80	0.93	16.3959	0.93	16.3959
90	0.96	16.9248	0.96	16.9248
100	0.99	17.4537	0.99	17.4537
120	1.04	18.3352	1.04	18.3352
140	1.09	19.2167	1.09	19.2167
Eave Height = 130'	1.07	18.7760	1.07	18.7760

Surface Type	**Surface Designation**	**Surface**	**Distance from Windward Edge**	**L/B or h/L**	***Cp***	**External Pressure @ q = 130' (psf)**
Walls	W2	WW	-	All	0.80	15.021
	W3	LW	-	0.56	-0.50	-9.388
	W1	Side	-	All	-0.70	-13.143
Roof			0 to h	0.34	-0.90	-16.898
			h to 2h	0.34	-0.50	-9.388
			> 2h	0.34	-0.30	-5.633

***Table Set 1** – Wind analysis*

Windward Pressures

qz (psf)	**Cp**	**External Pressure**	**Design Pressure (psf)**	
			(+GCpi)	**(-Gcpi)**
10.0491	0.80	6.83	3.45	10.21
10.9306	0.80	7.43	4.05	10.81
11.6358	0.80	7.91	4.53	11.29
12.3410	0.80	8.39	5.01	11.77
13.3988	0.80	9.11	5.73	12.49
14.2803	0.80	9.71	6.33	13.09
14.9855	0.80	10.19	6.81	13.57
15.6907	0.80	10.67	7.29	14.05
16.3959	0.80	11.15	7.77	14.53
16.9248	0.80	11.51	8.13	14.89
17.4537	0.80	11.87	8.49	15.25
18.3352	0.80	12.47	9.09	15.85
19.2167	0.80	13.07	9.69	16.45

***Table Set 2** – Wind analysis*

Leeward Pressures

qz (psf)	Cp	External Pressure (psf)	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	-0.5	-4.27	-7.65	-0.89
10.9306	-0.5	-4.65	-8.03	-1.27
11.6358	-0.5	-4.95	-8.32	-1.57
12.3410	-0.5	-5.24	-8.62	-1.87
13.3988	-0.5	-5.69	-9.07	-2.31
14.2803	-0.5	-6.07	-9.45	-2.69
14.9855	-0.5	-6.37	-9.75	-2.99
15.6907	-0.5	-6.67	-10.05	-3.29
16.3959	-0.5	-6.97	-10.35	-3.59
16.9248	-0.5	-7.19	-10.57	-3.81
17.4537	-0.5	-7.42	-10.80	-4.04
18.3352	-0.5	-7.79	-11.17	-4.41
19.2167	-0.5	-8.17	-11.55	-4.79

Table Set 3 *– Wind analysis*

Sidewall Pressures

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	Design Pressure (psf) (-GCpi)
10.0491	-0.70	-5.98	-9.36	-2.60
10.9306	-0.70	-6.50	-9.88	-3.12
11.6358	-0.70	-6.92	-10.30	-3.54
12.3410	-0.70	-7.34	-10.72	-3.96
13.3988	-0.70	-7.97	-11.35	-4.59
14.2803	-0.70	-8.50	-11.88	-5.12
14.9855	-0.70	-8.92	-12.30	-5.54
15.6907	-0.70	-9.34	-12.72	-5.96
16.3959	-0.70	-9.76	-13.14	-6.38
16.9248	-0.70	-10.07	-13.45	-6.69
17.4537	-0.70	-10.38	-13.76	-7.01
18.3352	-0.70	-10.91	-14.29	-7.53
19.2167	-0.70	-11.43	-14.81	-8.05

Roof

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	Design Pressure (psf) (-GCpi)
10.0491	-0.5	-4.27	-7.65	-0.89
10.9306	-0.5	-4.65	-8.03	-1.27
11.6358	-0.5	-4.95	-8.32	-1.57
12.3410	-0.5	-5.24	-8.62	-1.87
13.3988	-0.5	-5.69	-9.07	-2.31
14.2803	-0.5	-6.07	-9.45	-2.69
14.9855	-0.5	-6.37	-9.75	-2.99
15.6907	-0.5	-6.67	-10.05	-3.29
16.3959	-0.5	-6.97	-10.35	-3.59
16.9248	-0.5	-7.19	-10.57	-3.81
17.4537	-0.5	-7.42	-10.80	-4.04
18.3352	-0.5	-7.79	-11.17	-4.41
19.2167	-0.5	-8.17	-11.55	-4.79

Table Set 4 & 5 *– Wind Summary (3)*

Seismic Loads

Earthquake loads (or better stated as seismic) contribute to lateral effects on the building. Although the forces are still lateral, they act differently from wind loads. While wind loads vary from grade to top of building by pressure on the curtain wall, seismic loads transfer via columns. The loads still vary from story to story.

901 is a simple-use building, and does not represent a substantial hazard to human life, so **Occupancy Category II** was chosen. **Seismic Use Group I** was also chosen. Site Classification was designated in the GeoTechnical Report provided by the owner at **Class C** (very dense soil and hard rock). This classification is benefited to the fact that there is a 4-story parking garage below grade, which requires digging very deep into the earth. This also helps in building a solid foundation without the need for any caissons, pilings, etc. Since lateral forces were found to be resisted through concrete moment framing, R is valued at 5, and I is valued at 1.

Seismic base shear was founded with the following equation:

$V = C_S W$ where $Cs = S_{DS}/(R/I)$

$C_{Smax} = S_{D1}/[T(R/I)]$

and was found to be 0.00917*(3079+8426(9)+8548) = 802^K

The following is a summary of the story shear and overturning moments and their derivations:

Level	Height Above Shear Base, *h* (ft)	Weight *W* at Height *h* (kips)	Total Weight = **Σ** *W*	(Wxhx)^k	[(Wxhx)^k] ------------ [(Wihi)^k]	Lateral Seismic Force, *Fx* (kips)	Lateral Seismic Story Shear (kips)	Overturning Moment (kip-ft)
Roof	130	3,079	3,079	12,246,577	0.10900	87.42	87.42	-
11	118.86	8,426	11,505	269,486,085	0.24000	192.48	279.90	973.86
10	107.19	8,426	19,931	215,061,633	0.19100	153.18	433.08	3,266.43
9	95.52	8,426	28,357	167,191,752	0.14900	119.50	552.58	5,054.04
8	83.85	8,426	36,783	125,754,304	0.11200	89.92	642.50	6,448.61
7	72.18	8,426	45,209	90,629,510	0.08100	65.96	708.46	7,497.98
6	60.51	8,426	53,635	64,594,810	0.05500	44.11	752.57	8,267.73
5	48.84	8,426	62,061	38,496,793	0.03430	27.51	780.08	8,782.49
4	37.17	8,426	70,487	21,128,519	0.01900	15.24	795.32	9,103.53
3	25.5	8,426	78,913	9,190,626	0.00818	6.56	801.88	9,281.38
2	13.83	8,548	87,461	2,377,499	0.00212	1.70	802.00	9,357.94
1	-	-	-	-	-	-	-	11,091.66
Σ		87,461	497,421		1.00	802.00		11,091.66

Total Weight:	**87,461**
Base Shear:	**802 kips**
Total Overturning Moment:	**11091.66 ft-kips**

***Table Set 7** – Seismic Summary*

Figure 7 – Diagram of seismic loads

Analysis Process

ETabs v. 8.57 was used for analysis of the current lateral system. Although ETabs does not have an option for post-tensioning, there was no need to add post-tensioning to the model since the purpose of the model is to observe lateral activity, not gravity. As a result, a uniform slab at 5,000 psi was used throughout the entire model. Also, earthquake conditions used an outdated IBC 2000, and the wind conditions used ASCE 7-98. Both sets of code are outdated, but serve their purpose and use for the current analysis. Values from the hand calculations were added to the model for the seismic and wind. ETabs is also a great tool, since the user face allows for quick and easy development of the model. 3 separate grid patterns were used to create the model, since the irregular shape of a triangle creates a very difficult grid to work with. All columns needed to be placed exactly to its nearest inch in order to keep the model in correct shape. All three corners had to be radially calculated to find the distance to the columns and slab edges.

All columns and reinforcement as specified in the construction set were added to the model. Each column set (per floor) was also designated according to its compressive strength. This was done for a few reasons. If the model was built with just a typical 26" square column all throughout with 8-#9's for reinforcement, the building would be severely under-designed from the actual building. Columns located at the center of the building most definitely would fail with the combination of gravity and lateral loads applied. Also, 6,000 and 8,000 psi concrete columns are considerably stronger than 5,000 psi, which is still considered to be a high-strength concrete. It was necessary to build a model with as accurate a column schedule as the actual building. Column design and dimensions were found in the drawing set. Column sizes lessened going to higher stories as axial loads lessened going to higher stories.

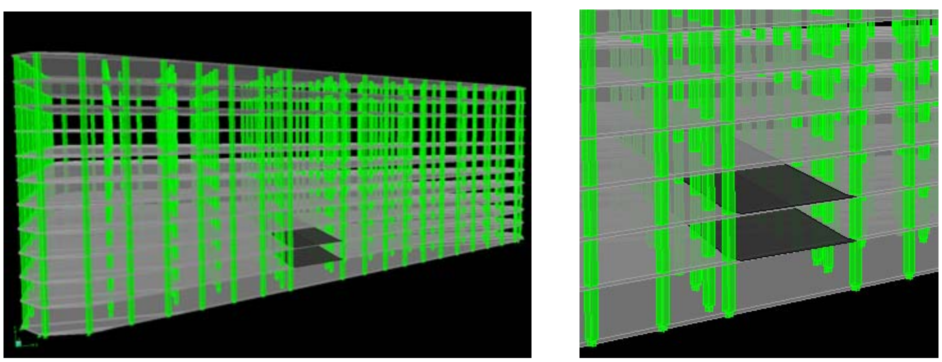


***Figure 8** – Designation of slab opening due to atrium space*

Slab openings were only considered in the major portions of the building, such as the 3-story atrium space in the center of the building. Small openings in the diaphragm (such as an opening for staircase, elevator shaft, etc.) do not have a significant effect on the model, whereas the opening for the atrium space cuts completely through the building to almost make two separate entities of the building.

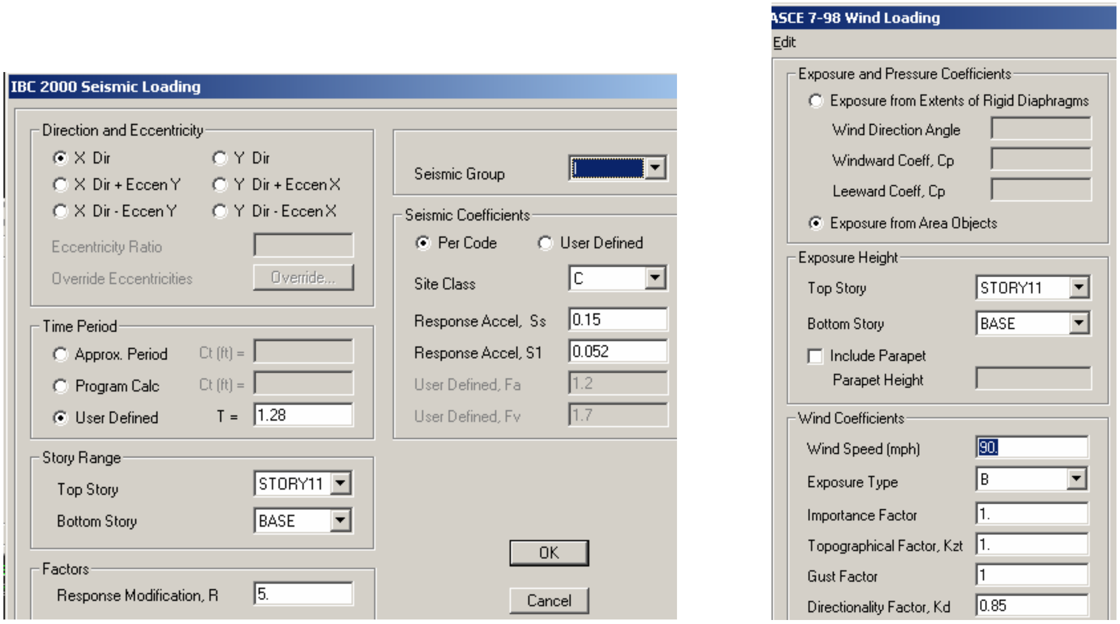


***Figure 9** – Seismic and wind custom input data (according to hand calculations)*

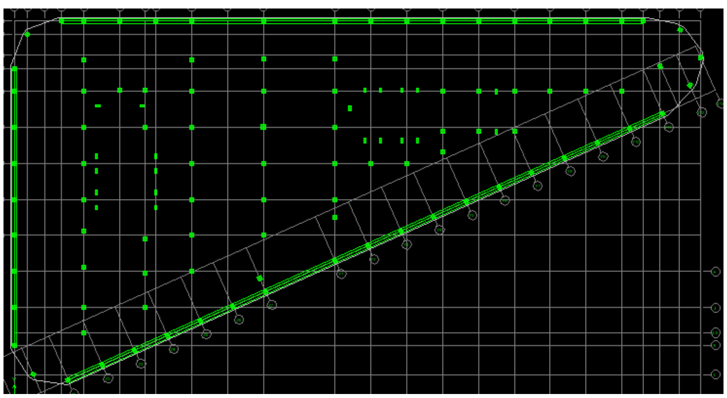


***Figure 10** – Designation of wind walls in ETabs*

As stated before, the loads were applied to the model according to the codes accepted by ETabs. A total of 10 load combinations were tried and checked to see which would control. Wind was considered to control over seismic.

Analysis of the model gave results for story drift and shear, support reactions, column interactive diagrams and forces, and center of rigidity (for mass).

Story Drift

The integrity of the building's lateral system will be assessed through its satisfaction in meeting the building's deflection requirements. Total drift of the building shall be assessed through the summation of each story's drift to get the entire building's drift. The allowable drift, or deflection, of the building is set at L/400, where L is the height of the building. Although there is no equation to derive the limiting value, L/400 was chosen as a general "rule of thumb" by the code. Using a value of L = (130')(12"/foot) = 1,560", L/400 was found to be 3.9". Below is a summary of the story drift.

Story	Item	Load	Point	X	Y	Z	DriftX *(in)*
2nd Floor	Max Drift X	09D10E	452	29.337	10.637	12.83	0.005791
3rd Floor	Max Drift X	09D10E	452	29.337	10.637	24.5	0.015941
4th Floor	Max Drift X	09D10E	452	29.337	10.637	36.17	0.024499
5th Floor	Max Drift X	09D10E	452	29.337	10.637	47.84	0.031857
6th Floor	Max Drift X	09D10E	452	29.337	10.637	59.51	0.037847
7th Floor	Max Drift X	09D10E	452	29.337	10.637	71.18	0.042518
8th Floor	Max Drift X	09D10E	452	29.337	10.637	82.85	0.046012
9th Floor	Max Drift X	09D10E	452	29.337	10.637	94.52	0.048439
10th Floor	Max Drift X	09D10E	452	29.337	10.637	106.19	0.049941
11th Floor	Max Drift X	09D10E	452	29.337	10.637	117.86	0.050718
Roof	Max Drift X	09D10E	452	29.337	10.637	130	0.051007
						Total:	**0.40457**

***Table 8** – Story drift summary*

0.41" is significantly smaller than the required building deflection limit of 3.9". Even if the limit had been L/600 (2.6") it still would have passed. This is an important aspect of the building, since the structural engineer designing 901 NYA had assumed that there was no need for shear walls. It is also important to note from the data from ETabs that earthquake loads controlled over wind loads. This is different than was expected; as information was given that the building was designed assuming that wind loads were the worst-case scenarios. However, it must be kept in mind that the codes used in 2000 and the codes used now for 2007 have gone through significant changes, from different coefficients to new equations and new diagrams. Even still, it is evident that the building was built to withstand any load combinations.

Center of Rigidity

The center of rigidity was found per floor for many reasons. It was important to state that each floor diaphragm was rigid, as it would be detrimental in a flat slab design if it weren't so. Also, in consideration of per floor drift, using the center of rigidity allowed a more accurate result of shear and torsion of the building. As viewed in the animation created by ETabs, the northeast side of the building sways more than the western portion. All of these factors were considered in the model, and the center of rigidity per floor was found for use of analysis.

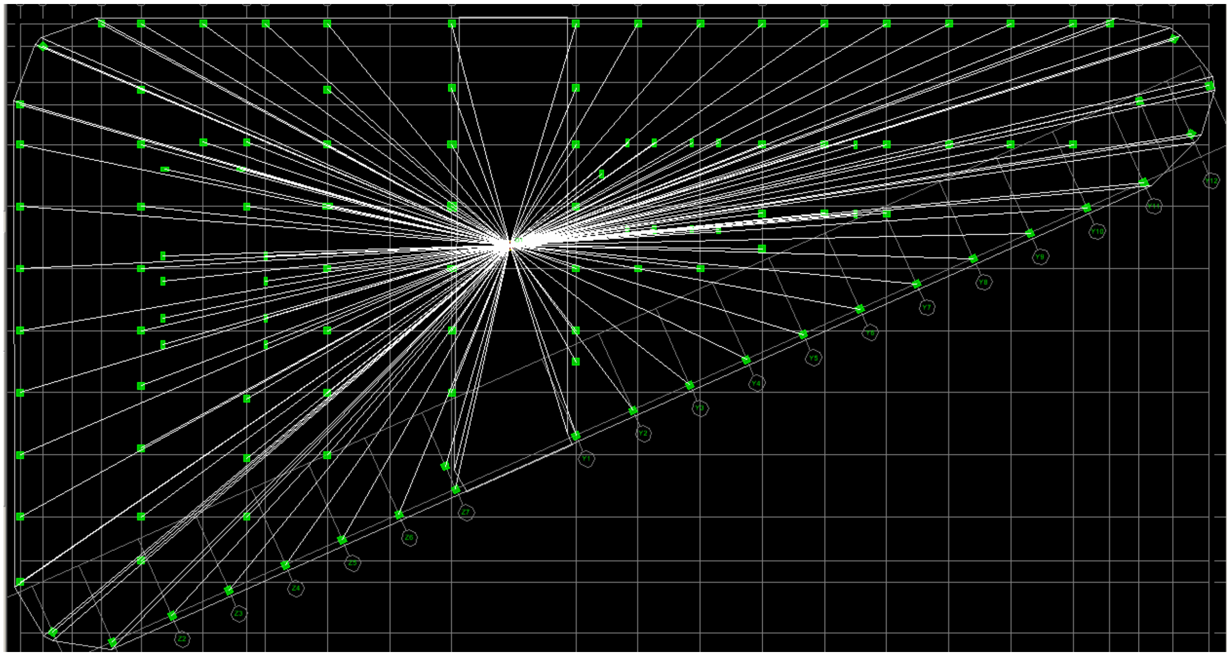


***Figure 11** – Location of center of rigidity of typical floor*

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR
STORY11	D1	226.9135	226.9135	149.382	139.499	226.9135	226.9135	149.382	139.499	154.498	140.052
STORY10	D1	240.7095	240.7095	149.636	139.541	467.623	467.623	149.513	139.521	154.489	140.063
STORY9	D1	240.1539	240.1539	149.627	139.539	707.7768	707.7768	149.551	139.527	154.479	140.076
STORY8	D1	240.0882	240.0882	149.63	139.535	947.865	947.865	149.571	139.529	154.469	140.092
STORY7	D1	240.0225	240.0225	149.632	139.531	1187.8875	1187.8875	149.584	139.529	154.46	140.109
STORY6	D1	240.0882	240.0882	149.63	139.535	1427.9757	1427.9757	149.591	139.53	154.456	140.129
STORY5	D1	240.1539	240.1539	149.627	139.539	1668.1296	1668.1296	149.596	139.532	154.459	140.153
STORY4	D1	240.172	240.172	149.646	139.54	1908.3016	1908.3016	149.603	139.533	154.474	140.179
STORY3	D1	240.1418	240.1418	149.641	139.538	2148.4434	2148.4434	149.607	139.533	154.491	140.197
STORY2	D1	240.2377	240.2377	149.611	139.544	2388.681	2388.681	149.607	139.534	154.481	140.173
STORY1	D1	241.7213	241.7213	149.629	139.551	2630.4024	2630.4024	149.609	139.536	154.456	140.136

***Table 9** – Center of rigidity summary*

The location of center of mass rigidity further explains the thicker slab and heavier reinforcing at the location of the center of the building. The application of heavier loads in the center also contributes to a more stable building.

Problem Statement

It is without question that 901 New York Avenue was built to be a very stable building. The building drifts only a meager 0.41" in worst load combinations (which was controlled laterally by seismic forces). Slabs are heavily reinforced with standard bars and post-tensioning to ensure minimum deflection throughout the 40'-0" spans. Although these "expensive" structural features seem unnecessary, it is important to note that the building is in the heart of Washington D.C. As such, there are zoning requirements that limit the building's height. As a result, in order to maintain 11 stories with 9'-0" high finished ceilings heights and stay within the limit of 130'-0", it was necessary for thin slabs. The architect also desired large bay areas in order to attract more interior designing possibilities. 11" for a slab thickness is then considered incredibly thin to span 40'-0".

It is then to question whether or not the current system was the best solution to the building's criteria. Could a steel-framed system have worked? Could one-way slabs have prevented extensive deflection and still leave room for 9'-0" finished ceiling heights? Could shear walls have resisted lateral forces and allowed for a smaller dimensioned column system?

Proposed Investigation

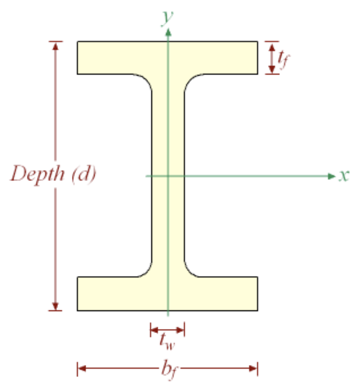
Initially, a few quick alternative systems were tried in order to see whether or not any other gravity systems could be a better alternative to the current system. The alternatives to be analyzed were: steel beam and column with metal deck and concrete slab; composite steel beam system with metal deck and concrete slab; pre-cast concrete slab panels on steel beams; one-way concrete slabs with joists. Then, a steel composite system was found to be the best possible alternative, so a more detailed analysis was done to conclude whether or not it would be a better solution. The steel building was then summarized in view of efficiency in cost, schedule, and feasibility.

Shear walls were also added to the building at the locations of the elevator shafts (locations of most shear and torsion of the building) to see if these locations would possibly take the lateral loads as necessary instead of the current moment framing. Then the alternative system was summarized in view of efficiency in cost, schedule, and feasibility.

Construction management and the possibility of turning 901 New York Avenue into a LEED-certified building will also be addressed as breadth options to the investigation.

Alternative System 1: Steel Beam and Column with Metal Deck and Concrete Slab

Description: The first alternative system to be analyzed was a steel-framed building, using wide-flanged beams and columns with metal form deck and a concrete slab. Structural steel has many benefits in design and construction, from strength in both compression and tension to very quick erection. Although typically composite systems are known to have stronger qualities, construction time on composite systems take a significantly longer time than a non-composite system. As a result, both systems were analyzed. The composite option will be described in the following alternative system.



in × lbf/ft	Area (in^2)	d (in)	b_f (in)	t_f (in)	t_w (in)	Ixx (in^4)	Zxx (in^3)	kxx (in)	Iyy (in^4)	Zyy (in^3)	kyy (in)
W24 × 76	22.4	23.92	8.990	0.680	0.440	2100	176	9.69	82.5	18.4	1.92

***Figure 4** – Dimension of a W24 x 76 beam*

The greatest factor will be the depth of the beams. Although steel opens up space in between beams and girders, the greatest depth of the beams will most likely control the floor-to-ceiling thickness (since you cannot cut through a steel beam without significantly losing the integrity of the beam).

Loads: Similar loads were used for the steel framing. It was assumed that this would only be a preliminary design, so lateral loads were, for the most part, not considered.

Live Load:	Lobby/Office Space	100 psf
Dead Load:	Metal Deck	3 psf
	Concrete Slab	(5.5" + 2"/2)*145 = 78.54 psf
	Beam Weight (assumed)	50 plf
	MEP and Finishes	20 psf

Bay Size: The same bay size was used as the original system at 20' by 40'. The metal decking spanned a complete distance of 8'-0", which also spread the beams out evenly within the bay at 8'. Sample design in RAM featured 3 bays horizontally (40' span) and 4 bays vertically (20' span). As already discussed, lateral loads were not considered. All beams and columns only take gravity loads.

Design: The metal decking used for design had to withstand at least 100 psf service loads. Vulcraft's catalog was used to find a suitable deck, and their 2C Conform deck was best fit for the 8' span.

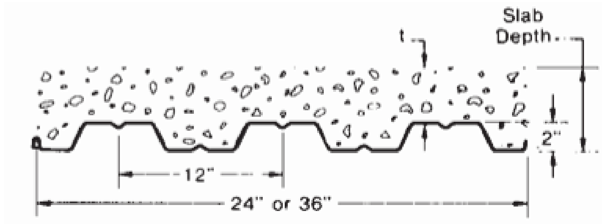


Figure 5 *– 2C Conform deck courtesy of Vulcraft*

Because of the deck's design, the total thickness slab is half the thickness of the deck and the cover on top of the deck. In this case, it was considered to be 5.5" + 2"/2 to make a total load of 78.54 psf by the slab and deck combination. It will be reinforced with 4x4-W2.9xW2.9 welded wire fabric.

Most of the beams that were spaced at 8' were typically designed at W14 x 22, while the girders were sized at W24 x 55 on the outside perimeter and W24 x 76 on the inside. Sample hand calculations were done to check the values of the RAM model. All calculations were done according to the LRFD Steel Manual (3rd Edition). The calcs showed that these estimated values are correct (see Appendix). Because there is nothing outside of the lateral system, the columns do not take on a heavy load. As a result, most of the columns were found to be either W10 x 33 or W10 x 39. Sample hand calculations show that these estimated values are also correct (see Appendix).

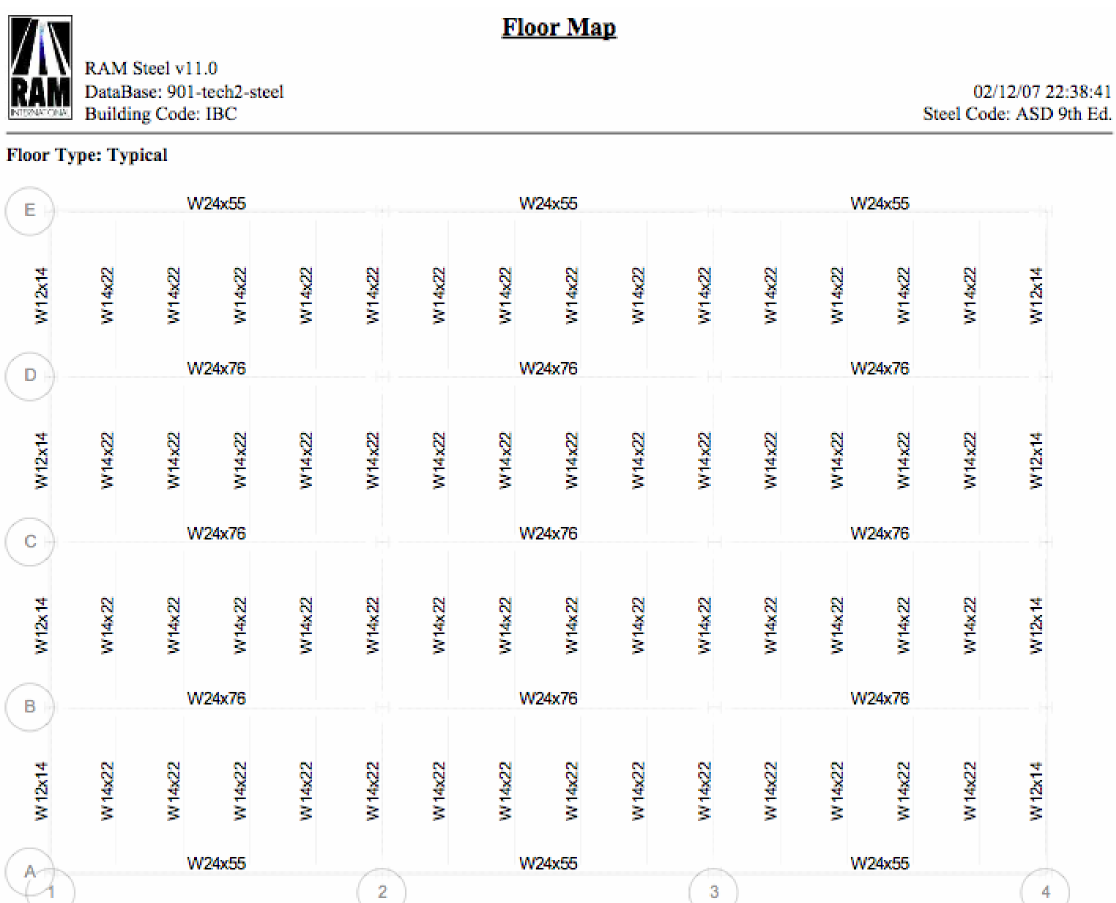


Figure 5 *– Beam Design of Steel System*

Gravity Column Design Summary

RAM Steel v11.0
DataBase: 901-tech2-steel
Building Code: IBC

02/12/07 22:44:48
Steel Code: ASD 9th Ed.

Column Line 1 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	18.1	8.8	1.8	1	0.25 Eq H1-3	0.0	50	W10X33
10	29.8	3.6	0.7	1	0.22 Eq H1-3	0.0	50	W10X33
9	40.5	3.3	0.7	3	0.24 Eq H1-1	0.0	50	W10X33
8	50.7	3.1	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
7	60.5	2.9	0.6	3	0.34 Eq H1-1	0.0	50	W10X33
6	70.1	2.8	0.6	3	0.38 Eq H1-1	0.0	50	W10X33
5	79.4	2.7	0.6	3	0.43 Eq H1-1	0.0	50	W10X33
4	88.7	2.7	0.5	3	0.48 Eq H1-1	0.0	50	W10X33
3	97.8	2.6	0.5	3	0.52 Eq H1-1	0.0	50	W10X33
2	106.8	2.6	0.5	3	0.57 Eq H1-1	0.0	50	W10X33
1	115.7	2.5	0.5	1	0.62 Eq H1-1	0.0	50	W10X33

Column Line 1 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33
10	47.7	5.8	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
9	65.7	5.3	0.5	3	0.38 Eq H1-1	0.0	50	W10X33
8	81.1	5.0	0.5	6	0.46 Eq H1-1	0.0	50	W10X33
7	98.0	4.8	0.5	6	0.55 Eq H1-1	0.0	50	W10X33
6	114.5	4.7	0.5	11	0.63 Eq H1-1	0.0	50	W10X33
5	133.2	4.7	0.5	6	0.73 Eq H1-1	0.0	50	W10X33
4	152.5	4.7	0.5	6	0.83 Eq H1-1	0.0	50	W10X33
3	171.8	4.7	0.5	6	0.94 Eq H1-1	0.0	50	W10X33
2	191.1	4.7	0.5	6	0.86 Eq H1-1	0.0	50	W10X39
1	210.5	4.7	0.5	6	0.93 Eq H1-1	0.0	50	W10X39

Column Line 1 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33
10	47.7	5.8	0.6	3	0.29 Eq H1-1	0.0	50	W10X33
9	65.7	5.3	0.5	3	0.38 Eq H1-1	0.0	50	W10X33
8	81.1	5.0	0.5	6	0.46 Eq H1-1	0.0	50	W10X33
7	98.0	4.8	0.5	6	0.55 Eq H1-1	0.0	50	W10X33
6	114.5	4.7	0.5	11	0.63 Eq H1-1	0.0	50	W10X33
5	133.2	4.7	0.5	6	0.73 Eq H1-1	0.0	50	W10X33
4	152.5	4.7	0.5	6	0.83 Eq H1-1	0.0	50	W10X33
3	171.8	4.7	0.5	6	0.94 Eq H1-1	0.0	50	W10X33
2	191.1	4.7	0.5	6	0.86 Eq H1-1	0.0	50	W10X39
1	210.5	4.7	0.5	6	0.93 Eq H1-1	0.0	50	W10X39

Column Line 1 - D

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	25.6	13.7	1.4	6	0.33 Eq H1-3	0.0	50	W10X33

Figure 6 *– Column Design of Steel System*

Advantages

- Off-site fabrication
- Very quick erection
- Tension/Compression benefits
- Longer lifetime integrity than concrete
- Different dead loads due to different materials could lead to different foundation and lateral system

Disadvantages:

- Fireproofing not included
- Moment framing much more complicated, otherwise braced framing needed
- Very thick beam-and-deck combination may not be a better solution. Beam itself is 24", and that doesn't include the 7.5" slab and deck.

Summary: It can be quickly assumed that a simple steel-framed building (no composite or other contribution to distributing loads) would not be in the best interest of the owner. A total floor thickness of 31.5" is more than acceptable, as the MEP systems have not even been considered. It is possible that perhaps a composite system may prove much more efficient for steel design. That option will be assessed in the next alternative system.

Alternative System 2: Steel Composite System w/Metal Deck and Concrete Slab

Description: In the previous alternative system, a steel system was proposed, but the sizes were coming out much too large to be considered as a true alternative. A composite system may help reduce the thickness of slab, deck, and beam.

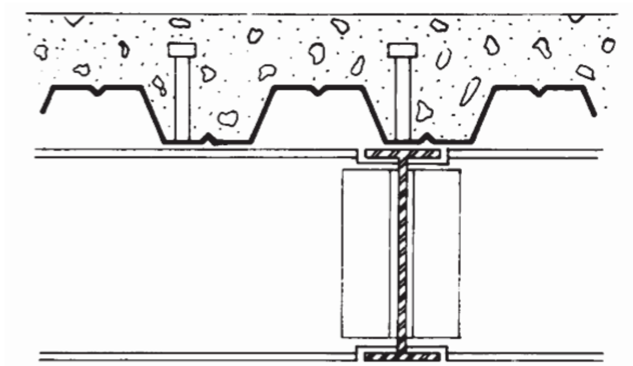


Figure 7 *– Example of Composite System*

A composite system works by distributing the loads of the beam to the deck along with itself. In this manner, stress on the beam is lessened, and a smaller beam is possible. There is a setback to this design, however. The deck and beam are connected through a mechanism called the shear stud, and the installation and application of these studs into the deck and beam is very time consuming. Also, the positive benefits of a composite system don't really come into effect until about 28' and more. This may actually be helpful in 901 NYA's case, as its span is as long as 40'-0".

Outside of these special conditions, a composite has mainly the same advantages and disadvantages of a regular steel system.

Loads: Once again, lateral loads were not considered for simplicity purposes. A composite steel framing considers the beam, slab, and deck weight, along with MEP and finishes. Live loads are still the same as the existing system:

Live Load:	Lobby/Office Space	100 psf
Dead Load:	Metal Deck and Slab (comb.)	2 psf
	Beam Weight (assumed)	50 plf
	MEP and Finishes	20 psf

Bay Size: Two different designs were considered for the composite system. Although the current bay length of 40'-0" is a positive benefit for composite beams, very long distances can still force the beam's depth to be too deep. As a result, composite beams at 32' and 40' were both analyzed through a RAM model. The current frame's short distance is 20', but for the fitting of the deck, a preferred distance would either be 7'-0" to 8'-0" between beams. As such, both distances were also tried, one at 3 beams @ 24'-0" and the other at 3 beams @ 21'-0".

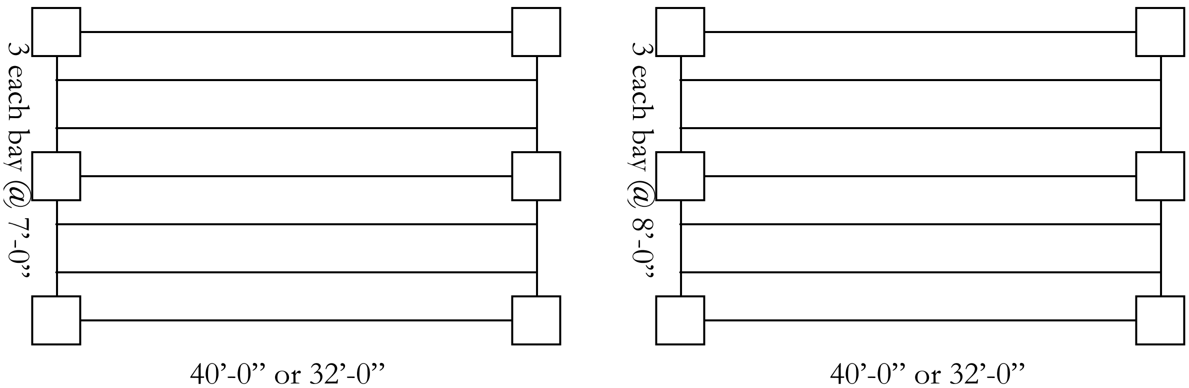


Figure 8 *– Bay designs for composite systems*

Design: Once again, the first step is to find the size of the deck. This time things are different from the previous design in that the length being spanned is 32'-0" and 40'-0" instead of 20'-0". Vulcraft's decking catalog also has a section for composite-use decks along with roof and non-composite decks. Distances of 7'-0" and 8'-0" were the span of the deck, with service loads as the considered loads in the tables. 1.5 VL/VLI was found to satisfy both distances, with the 7'-0" length needing 3.5" with 22-gage steel and the 8'-0" length requiring 3.5" with 21-gage steel. Already there is a significant difference from the regular steel framing. The non-composite system required 6.5" of slab and deck, whereas the composite system only requires 5" of slab and deck with a lower weight (1.97 psf).

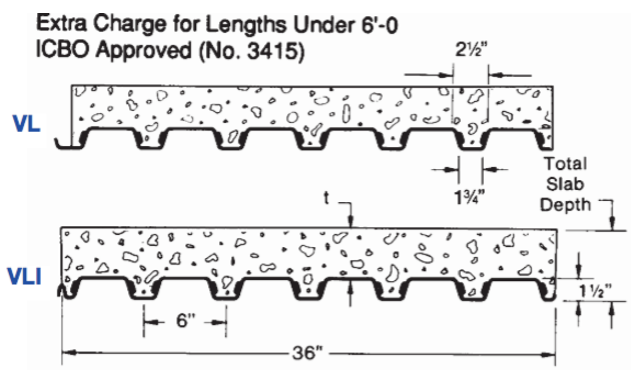


Figure 9 *– 1.5 VL/VLI deck courtesy of Vulcraft*

Several different designs were tried. Because it was desired to have a span of 7'-0" and 8'-0" for the deck, the bays had to be readjusted to 21' and 24' bay widths, respectively. This allows for 3 divisions in the bays.

The first trial in RAM of 40' x 7' (length by width) resulted in a typical layout with W16 x 26 beams throughout the frame (girders included). The lightest beams are found around the perimeter at W12 x 19. This is simply because of the fact that perimeter beams take half the load. It should be noted that there are numbers in parenthesis next to the beam size. These are the number of studs required for satisfactory design. The more studs, the better composite action, but longer construction time.

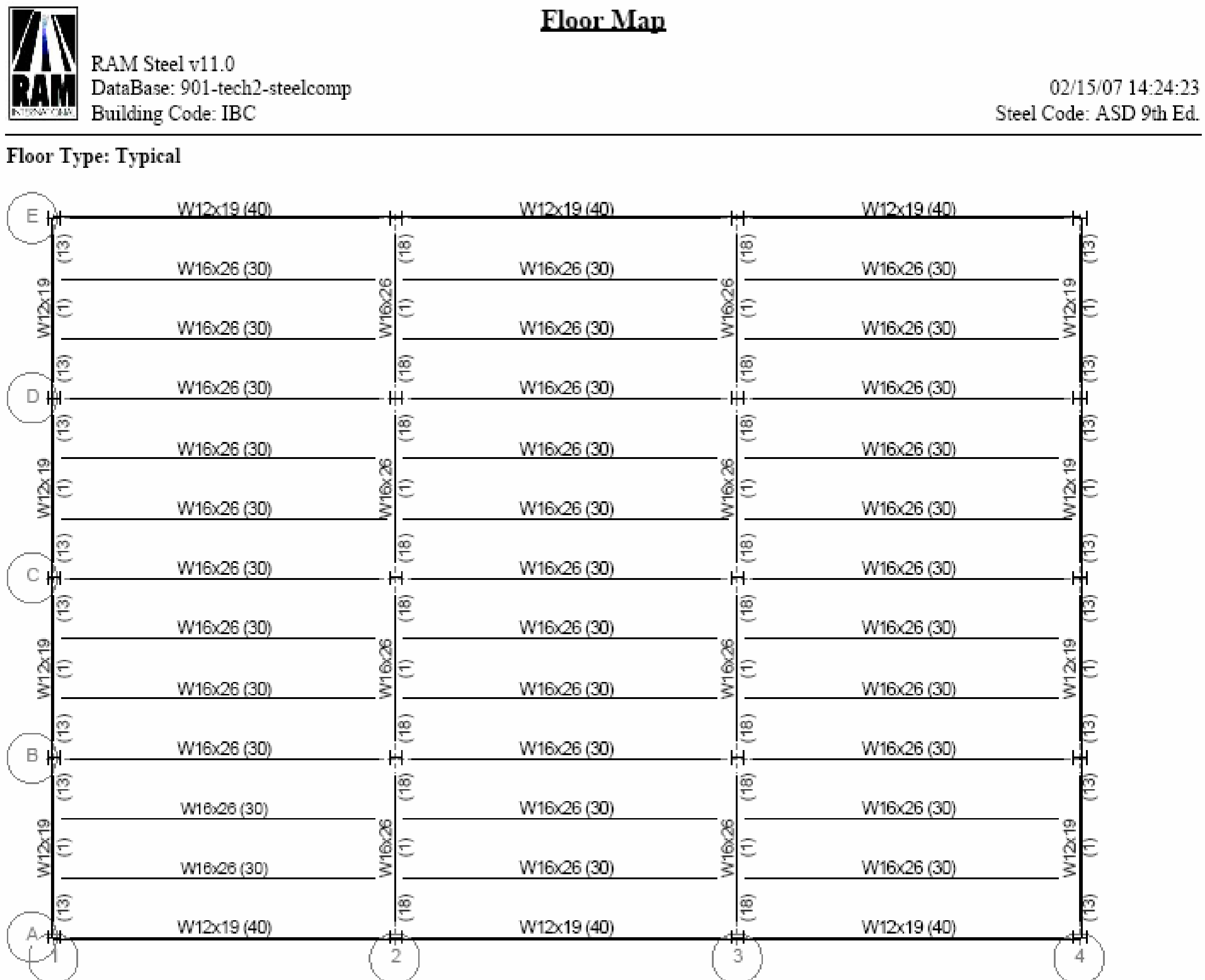


Figure 10 – Composite Layout with 21' by 40' bays

The second trial had a 24' by 40' bay using 8' divisions within the bay. This opened up the possibility of even larger bay spans than the current design. Although the beam sizes are the same (W16 x 26), it's observed that the wider bays require more studs. Not only that, but the girders are also larger sizes. In terms of fabrication and delivery to site, it is much easier to have pieces in the same size to reduce fabrication time. Also, an increase of shear studs can also greatly increase construction time. So far, the first trial is the better solution.

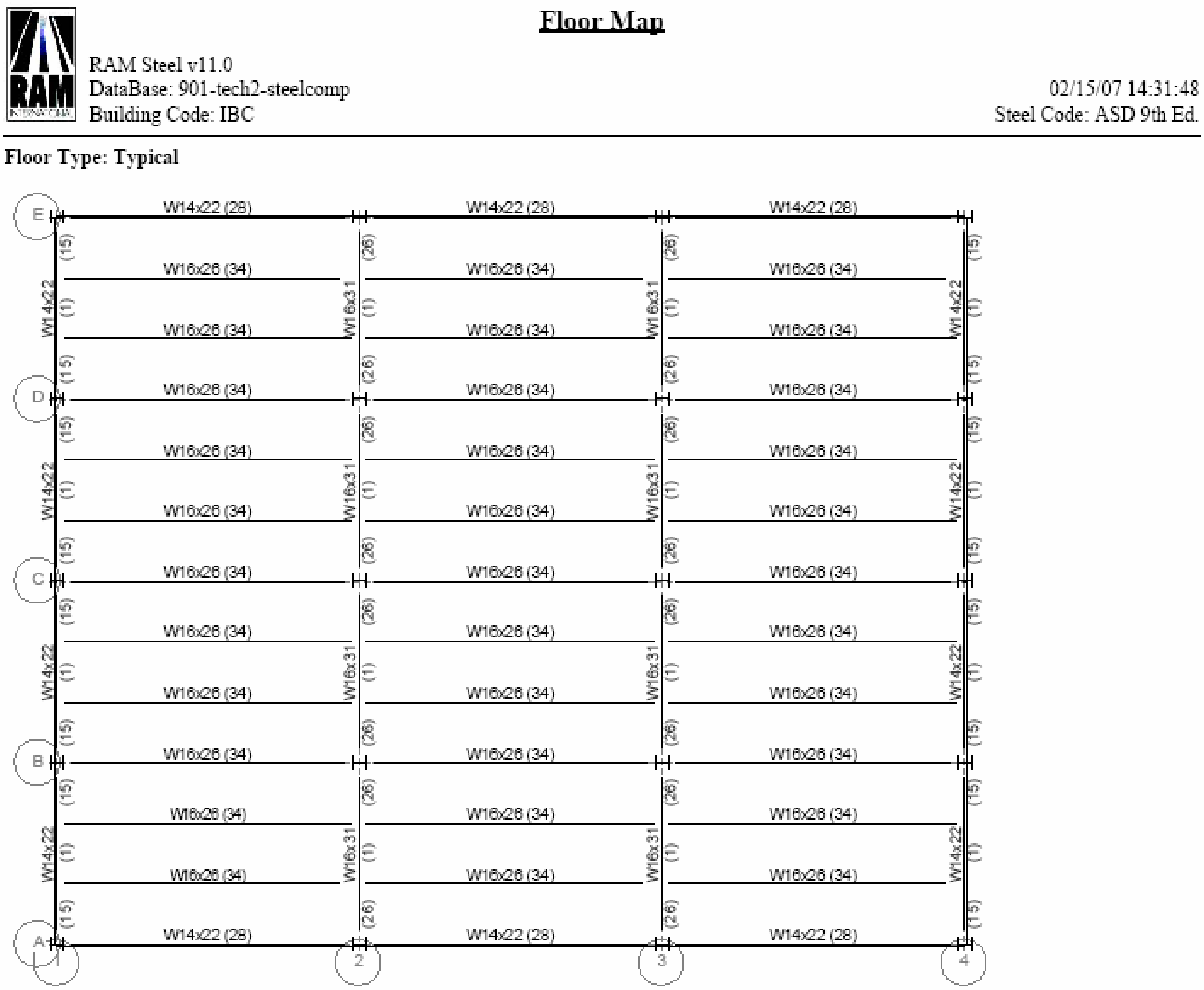


Figure 11 *– Composite Layout with 24' by 40' bays*

The final trial was an attempt to see if a smaller bay length would affect the size of the beams and the number of studs. The third trial had a bay size of 21' by 32'. Beams within the bay came out to about 2" smaller than the first trial and required less studs for composite action. However, it can be noted also that the girders stay the same size at W16 x 26. So even with the smaller beams, the total depth of the system is still 16". The first trial still has the best outcome (longer span, same sized beams and girders, average amount of studs).

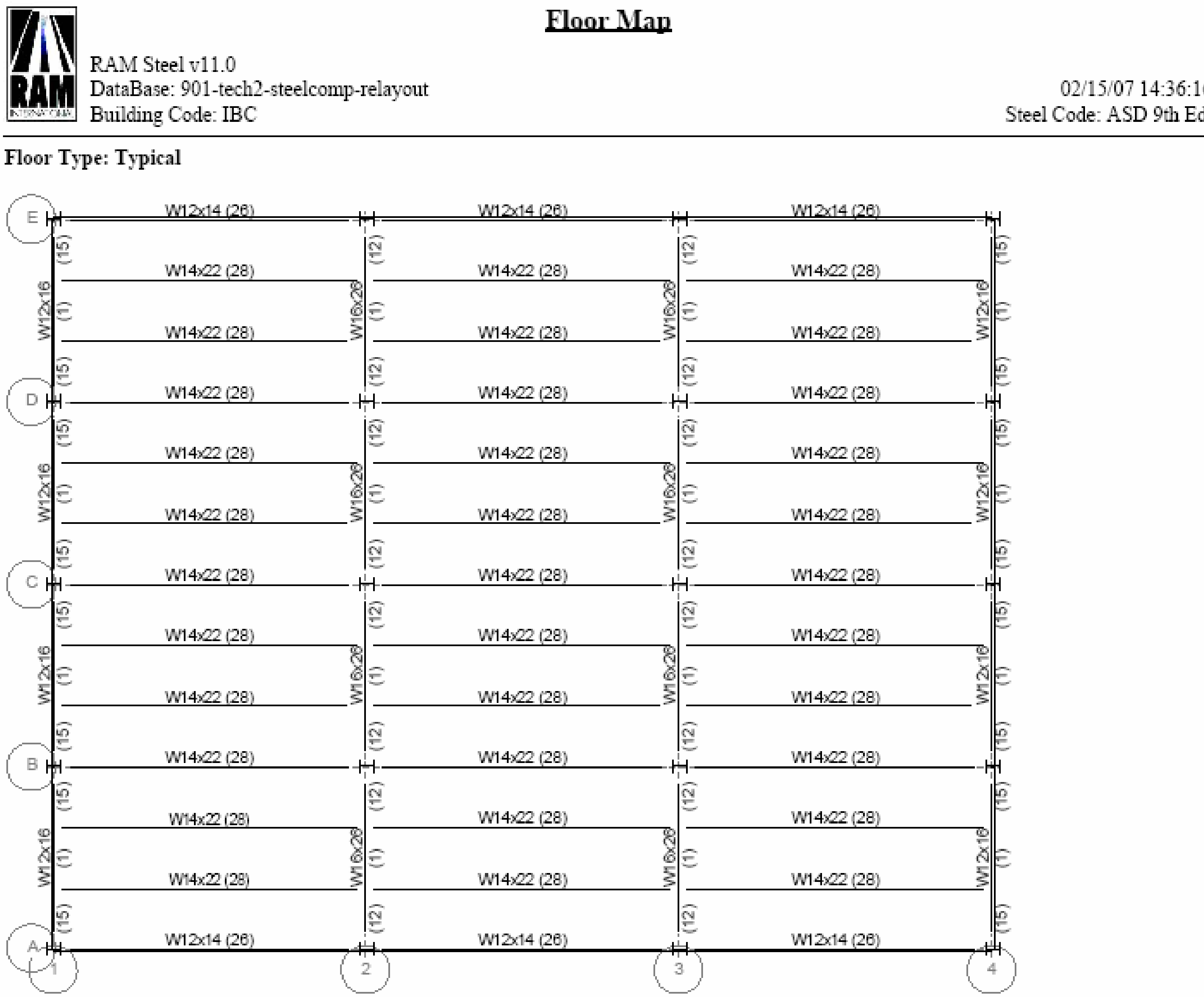


Figure 12 *– Composite Layout with 21' by 32" bays*

Advantages

- Much smaller sandwich system than the non-composite system
- Smaller slab and deck system than the non-composite system
- Much smaller beam at 16”
- Shoring is not needed
- A lighter system may lighten foundation design and lateral resistance necessities

Disadvantages

- Shear studs require much more construction time and work
- Same general disadvantages of steel structure as the non-composite system

Summary: Of all the steel systems, it seems that the 21’ by 40’ bay composite structure is the best solution. It is also important to note that the number of connections in the composite system is greatly decreased due to the fact that the beams run long-way instead of short-way in the non-composite alternative. There is still the setback of composite systems because of shear studs. But to my observation, if the owner was willing to pay extra cash for an extremely complicated post-tensioning system, extra money for a composite system would definitely be a possibility.

Another setback is that even the smallest system of 16” beams does not include the integration of the MEP system. So it can be assumed that the total depth of the system would be larger than the 16” of just the beam.

Alternative System 3: Pre-Cast Hollow Core Concrete Slab

Description: Another alternative system considered was a pre-cast, hollow-core concrete slab. Pre-cast (P/C) concrete is already used on the building for the outside façade. Because it is yet a young method of construction, pre-cast concrete brings in a great number of benefits atypical to steel and cast-in-place (CIP) concrete. Concrete is typically known for its time-consuming on-site construction and some tendencies of having unsatisfactory concrete batches (that would require re-pouring and a huge delay on many projects). P/C concrete benefits from CIP in the following ways: better controlled conditions, fire resistance, and durability (more benefits in the AS-1 summary). It is also just as shapeable as CIP concrete. These are the reasons as to why P/C concrete was considered.

Loads:

Bay Size: The slab is proposed to span 20'-0" (short direction) in the typical bay. Another option was to span the full 40'-0", but P/C slabs cannot be loaded to support more than 122 psf @ 40'-0". Thus the 20'-0" span was selected over the 40'-0" span.

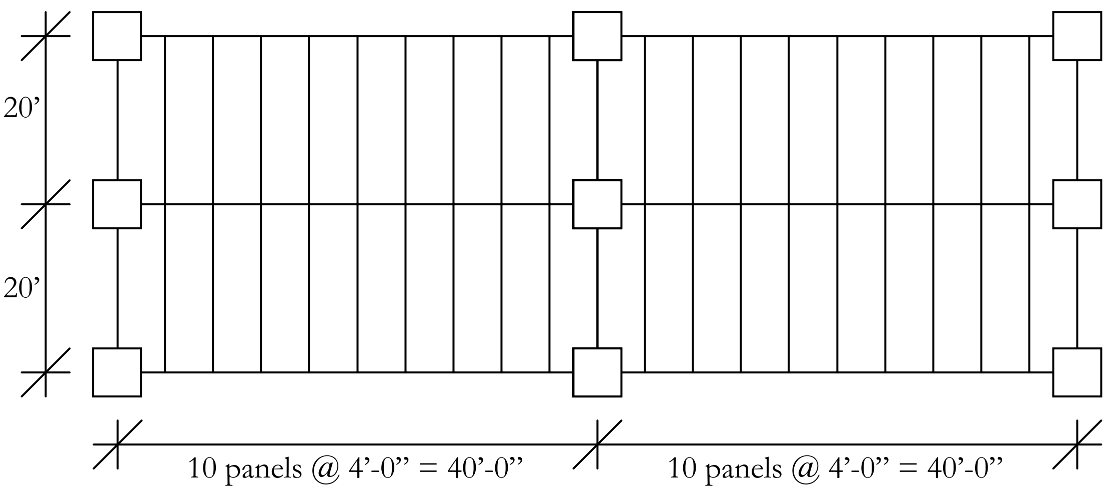


Figure 13 *– Layout of 4'-0" P/C Slabs*

Design: Factored loads included the pre-cast slab and 2" topping (73.75 psf), and the live load (160 psf) to get a total of 241 psf. Example slabs were found in the PCI Handbook (6th Edition), and a hollow core slab was found to best suit the current system (better long span conditions). Design guidelines were followed in conjunction with the PCI Handbook. 4HC6 + 2 was chosen, with 7-3/8" strands. Safe superimposed service loads come out to be 163 psf, with a camber of .3" during erection and 0.1" longtime camber. 4HC6 + 2 was chosen over 4HC6 because the deflection for 4HC6 was assumed to be 0.5". Although this is still in the acceptable range of deflection for its length,

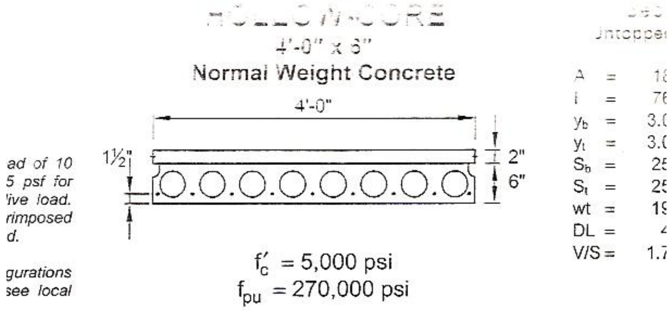


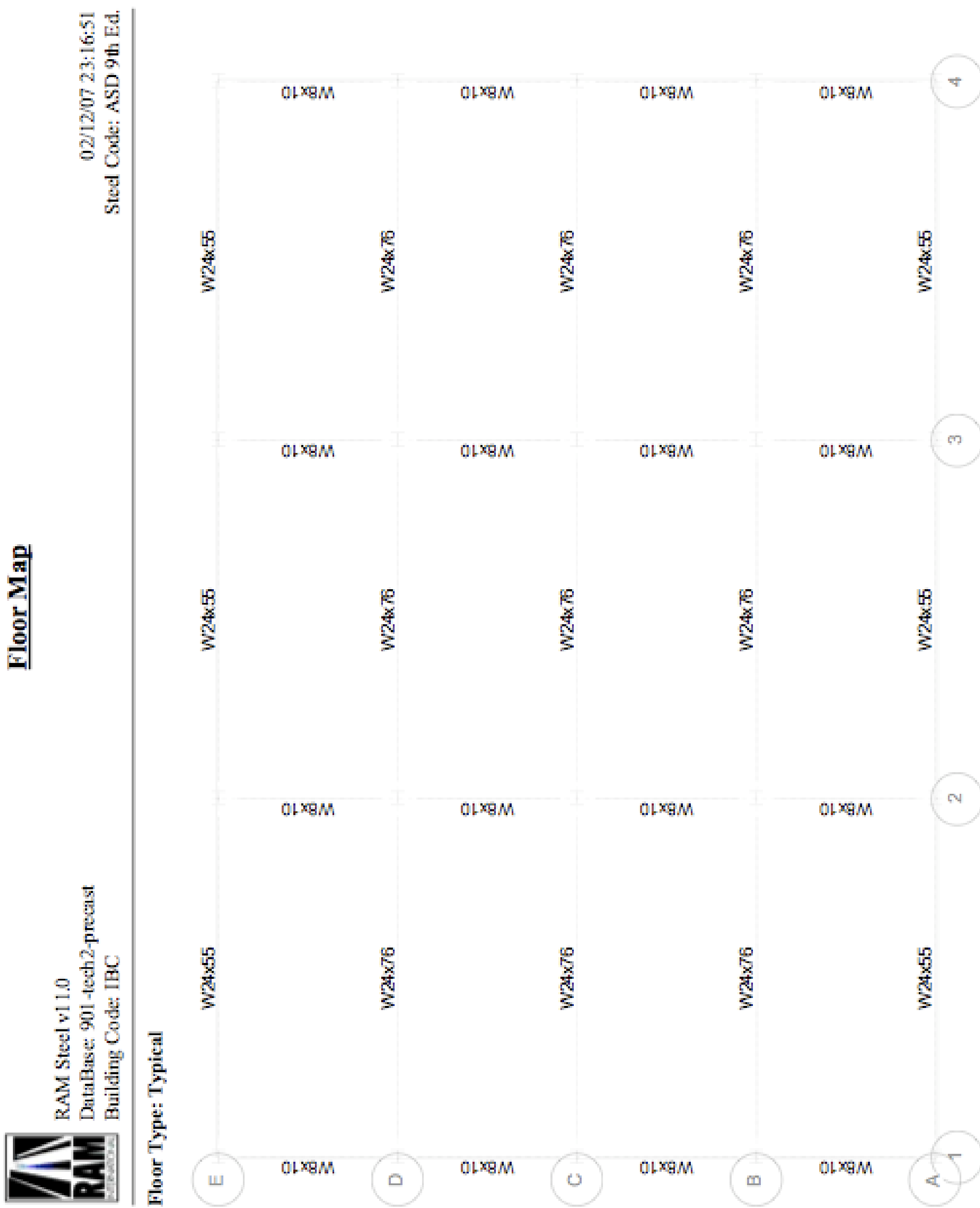
***Figure 14** – Sample of 6" Hollow Core Slab w/2" topping*

Although P/C beams can also be used, a concrete beam was assumed to be much too large at a 40' span, so both steel and concrete beams were considered. RAM Structural System was used to analyze the steel beams.

***Figure 15** – Steel beam layout for P/C panels*

Advantages
- Very quick to erect
- Off-site construction of panels
- Very quick scheduling
- Better integrity than CIP
- Lighter system may help lighten loads for foundation

Disadvantages
- Fireproofing not included for steel
- Lighter system may cause a whole new series of issues (different lateral system may control)
- Connections and details can become very complicated with hybrid systems
- Cannot "cut through" beams w/o losing significant strength


Floor Map
RAM Steel v11.0
DataBase: 901-tech2-precast
Building Code: IBC
02/12/07 23:16:51
Steel Code: ASD 9th Ed.
Floor Type: Typical
W24x55
W24x76
W8x10
1
2
3
4
A
B
C
D
E

Gravity Column Design Summary

RAM Steel v11.0
DataBase: 901-tech2-precast
Building Code: IBC

02/12/07 23:18:18
Steel Code: ASD 9th Ed.

Column Line 1 - A

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	17.6	10.4	0.2	1	0.21 Eq H1-3	0.0	50	W10X33
10	28.8	4.2	0.1	1	0.20 Eq H1-3	0.0	50	W10X33
9	39.1	3.8	0.1	3	0.22 Eq H1-1	0.0	50	W10X33
8	48.7	3.6	0.1	3	0.27 Eq H1-1	0.0	50	W10X33
7	58.1	3.4	0.1	3	0.31 Eq H1-1	0.0	50	W10X33
6	67.2	3.3	0.1	3	0.36 Eq H1-1	0.0	50	W10X33
5	76.1	3.2	0.1	3	0.40 Eq H1-1	0.0	50	W10X33
4	84.8	3.1	0.1	3	0.45 Eq H1-1	0.0	50	W10X33
3	93.5	3.0	0.1	3	0.49 Eq H1-1	0.0	50	W10X33
2	102.0	2.9	0.1	3	0.53 Eq H1-1	0.0	50	W10X33
1	110.4	2.9	0.1	1	0.58 Eq H1-1	0.0	50	W10X33

Column Line 1 - B

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33
10	45.8	6.8	0.0	3	0.28 Eq H1-1	0.0	50	W10X33
9	62.9	6.2	0.0	3	0.36 Eq H1-1	0.0	50	W10X33
8	79.0	5.8	0.0	6	0.44 Eq H1-1	0.0	50	W10X33
7	94.8	5.6	0.0	6	0.52 Eq H1-1	0.0	50	W10X33
6	110.3	5.4	0.0	6	0.59 Eq H1-1	0.0	50	W10X33
5	128.1	5.4	0.0	6	0.68 Eq H1-1	0.0	50	W10X33
4	146.4	5.4	0.0	6	0.78 Eq H1-1	0.0	50	W10X33
3	164.7	5.4	0.0	6	0.87 Eq H1-1	0.0	50	W10X33
2	183.0	5.4	0.0	6	0.96 Eq H1-1	0.0	50	W10X33
1	201.4	5.4	0.0	6	0.88 Eq H1-1	0.0	50	W10X39

Column Line 1 - C

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33
10	45.8	6.8	0.0	3	0.28 Eq H1-1	0.0	50	W10X33
9	62.9	6.2	0.0	3	0.36 Eq H1-1	0.0	50	W10X33
8	79.0	5.8	0.0	6	0.44 Eq H1-1	0.0	50	W10X33
7	94.8	5.6	0.0	6	0.52 Eq H1-1	0.0	50	W10X33
6	110.3	5.4	0.0	6	0.59 Eq H1-1	0.0	50	W10X33
5	128.1	5.4	0.0	6	0.68 Eq H1-1	0.0	50	W10X33
4	146.4	5.4	0.0	6	0.78 Eq H1-1	0.0	50	W10X33
3	164.7	5.4	0.0	6	0.87 Eq H1-1	0.0	50	W10X33
2	183.0	5.4	0.0	6	0.96 Eq H1-1	0.0	50	W10X33
1	201.4	5.4	0.0	6	0.88 Eq H1-1	0.0	50	W10X39

Column Line 1 - D

Level	P	Mx	My	LC	Interaction Eq.	Angle	Fy	Size
11	27.2	16.2	0.1	6	0.32 Eq H1-3	0.0	50	W10X33

Summary: The hollow core pre-cast system has many benefits. For one, the simplicity of design of erecting pre-cast panels instead of casting in place would save an immense amount of time. An 8" panel is sufficient to withstand gravity loads, which is thinner than the current system. The only setback is that if the same bay area is used, the depth of the beams becomes much too deep. If a concrete girder is used, it can be expected to exceed more than 42". Even a steel beam would be a depth of 33". An alternative to a simple girder is a pre-stressed concrete girder. This may help in the size of the beam.

Another setback is the fact that 901 NYA is not a simple rectangular building. The greatest benefit from pre-cast concrete is the repetition of panels. Because of so many different actually bay sizes and dimensions, pre-cast may not be the best alternative to the current system.

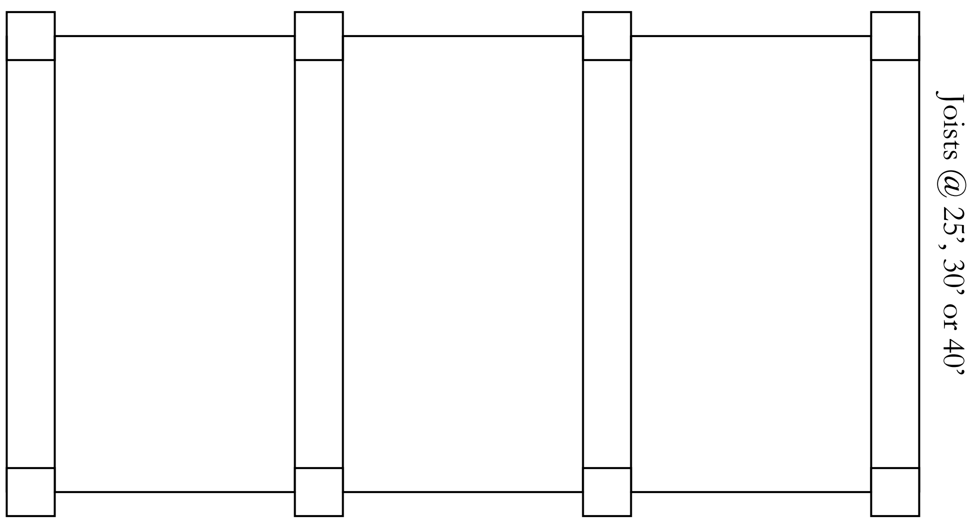
Alternative System 4: 1-way Concrete Slab w/Joists

Description: The final alternative system is the possibility of using a one-way slab supported on running joists. This is the only other concrete alternative that was assessed. One-way slab and joist systems are known for its low dead weight and need for reinforcement. It is also best suited at long distances, so it is beneficial that our current system uses a bar dimension of 20' by 40'.

Loads: Loads for the slab were first found before finding possible loads. Then the dead load of the slab was added to the total load to find the loading on the joists.

Live Load:	Lobby/Office Space	80 psf
Dead Load:	Slab (8")	100 psf
	Slab (5")	67.5 psf
	MEP and finishes	20 psf

Bay Size: Several different bay sizes were used to see what bay size might be best for a one-way joist. For initial calculations, I looked at a 13' and 20' slab span. For the 13' span, a 13' by 25' bay was selected (to maintain rectangular properties and not square). For the 20' span, a 20' by 30' bay and a 20' by 40' bay was selected.



Slab Spans 13' or 20' @ 5" and 8", respectively

***Figure 18** – 1-way joist dimensions*

Design: The CRSI Handbook was used to find acceptable sizes for different factored loads on a slab. At a 13'-0" span, the handbook allowed a 5" slab with #4's @ 10" OC on top and #3's @ 7" OC on the bottom. The slab is considered to be normal weight concrete, and the dead weight of the slab is 63 psf. At a 20'-0" span, the accepted design was an 8" slab with #5's @ 9" OC on top and #4's @ 8" OC on the bottom. All calculations can be found in the Appendix.

Advantages

- Simple design means simple construction and formwork
- Fireproofing is already implemented
- Generally about the same weight as current system; new foundation design wouldn't be necessary
- Much quicker construction than post-tensioning

Disadvantages

- Thinner slab brings new serviceable issues, like vibration
- At columns, the thickness of floor system ranges from 21" to 42", for 5" slab and 8", respectively
- Shear walls may need to be designed into building

Summary: Although the slab design came out beneficial for this alternative, the girders supporting the slabs were much too thick. Compared to the current building, it is a difference of 10"-31", which is perhaps more than permissible by the owner. As already explained, sacrificing ceiling space causes a "cramped" feel to the building floor, which would not be a comfortable environment to work in.

A joist-and-girder system has also been briefly viewed from the CRSI Handbook to see the possibilities of using a multi-joist system (8" deep rib + 3" slab is the smallest found in the handbook). The benefits of a joist-and-girder 1-way slab is it increases stiffness to the floor, MEP systems can be easily integrated into the floor system, and additional weight would factor out vibration as being an issue. The setbacks are that a new floor layout would be required, along with the fact that it will still be deeper than the current system. If a 1-way slab is to be considered for an alternative to the current system, it would be a 1-way joist-and-girder system.

Summary of Initial Investigation

	Steel Framing	Composite System	Hollow Core Pre-Cast Concrete	1-way Slab (w/ and w/o Joists)
Floor Depth	Slab and Deck: 7.5" Beam: 24" **Total: 31.5"**	Slab and Deck: 6.5" Beam: 16" **Total: 22.5"**	Panel: 8" Girder: 33" – 42" **Total: 41" – 50"**	Slab: 5" – 8" Girder: 16" – 34" **Total: 21" – 42"**
Floor Weight (psf)	~ 70 psf	~ 40 psf	~ 60 psf	~ 125 psf
Fireproofing	No	No	If concrete girder used	Yes
Vibration	Relatively light systems have vibration issues	No	Relatively light systems have vibration issues	Most likely no
Cost (RS Means)				
Lead Time	Yes	Yes	Yes	No
Feasibility of Design	Fabrication off-site, quick erection, braced framing, complicated connections, lighter weight may cause re-design of foundation	Fabrication off-site, semi-quick erection (shear studs), complicated connections, lighter wt may cause re-design of foundation	Fabrication off-site, quick erection, possible pre-stressed designs may help, perhaps not enough repeat of panels	Cast-in-place, long construction time, pre-stressed designs may help some, MEP implemented into floor system
General Comments	Not considered as an alternative	Possible consideration, but redesign of columns	Prestressed beams? Possible consideration for alternative	Possible consideration, but redesign of columns

In summary, there are many things to note. First, with the current criteria of 40'-0" spans, it will be near impossible to find any other realistic alternative to the current structural system without heavily losing ceiling height spaces. Any beam spanning that length would be susceptible to 800 square feet of loads, live and dead. The only possibility is the composite beam system, as it only compromises a loss of 10" (16" for deepest beam + 5" deck and slab). The next consideration would be whether or not a braced-frame or moment-frame system would be used. Moment frames would increase the sizes of the beams, as they would be required to resist moments created by lateral loads.
The foundation will also need to be re-designed, depending on the alternative system chosen. For example, a steel-framed building would have a total weight of 3,365 kips, while the current system has a total weight of 6,610 kips. Half the weight will change the size of footings, the need for strap beams, the effects of wind and seismic to a lighter building system, etc.

Finally, it is important to note and remember the fact that there is still a 4-level parking garage sub-grade. In my personal experience, I have yet to see a steel-framed parking garage. Most above-grade parking garages are usually made of pre-cast or cast-in-place concrete. Although it is possible to make a parking garage of steel, it is not usual practice to do so. However, since there is no post-tensioning in the sub-levels, it is still possible to create a concrete below-grade and steel grade-and-above building.

Overall, whatever system is chosen, it must meet the general criteria of the building. From building height limitations to desired floor-to-ceiling heights to exposed MEP systems, all of these must be considered before calling any other alternative system a true possibility.

The quick overview of all the systems above shows that the composite may be the best alternative to the current system. Steel buildings can be erected much faster than concrete buildings (from the lack of the need to wait for complete curing of slabs and columns to move on to next story). However, composite systems are complicated, from its connections to application of shear studs. Concrete still has better flexibility in terms of integration of MEP systems as well.

Alternative Gravity System Analysis – Steel Composite System

While the initial trials of the four alternatives proved to be a good preliminary analysis for a quick overview of each option, it is important to now further investigate the possibility of using a steel composite alternative. This investigation will require a more accurate model of the building, an application of lateral loads, and a re-calculation of foundation systems.

The first major change to the building is its loads, both gravity and lateral. The building will need to be re-analyzed for both.

Gravity Loads
Specific loads will be assessed in this analysis, since it will be directly compared to the current system in all aspects. The dead loads remained the same, besides the self-weight. The live loads will also remain the same for the most part, as well as the roof and snow loads.

Dead Loads

MEP Systems 15 psf
Floor finishes, etc. 15 psf

Live Loads

Public spaces, corridors 100 psf
Center Lobby 210 psf

Reducible Live Loads

Controlled by: $L = L_o[.25 + 15/(\sqrt{KA_t})]$

Roof Live Loads

Lr = $20R_1R_2$
= 5000 psf

Snow Loads

Pf = $0.7 * C_e * C_t * I * p_g$
= 15.75 psf

Design
ETabs will be used once again to model a sample building of 901 NYA in steel design. ETabs has an advanced modeling system in which iterative designs can be implemented into the building model. As a result, it will sometimes over-design and under-design some beams and columns for the benefit of repetitive sizes.

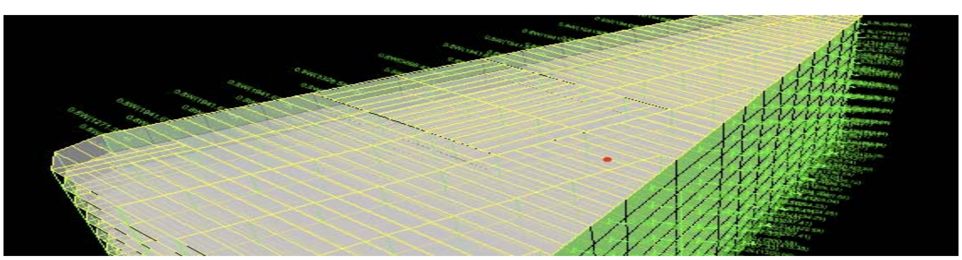


Figure 19 *– Rendering of 901 NYA as steel*

The first step requires a re-alignment of the columns from the current set-up. The same bay layout created in the preliminary designs will be used as a starting place for the re-organization of the columns. Previous designs suggested using 21'-0" by 36'-0" bays instead of the current 20'-0" by 40'-0". This is done because the greatest span that a steel deck from VulCraft at 1.5" is capable of spanning without shoring is 7'-0".

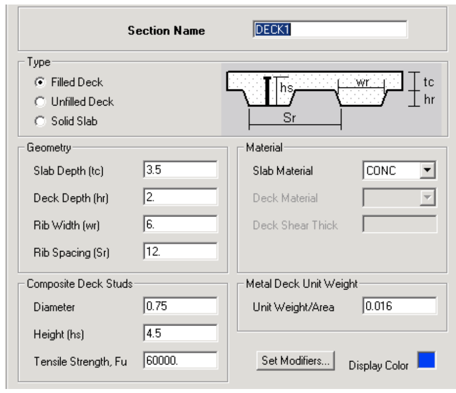
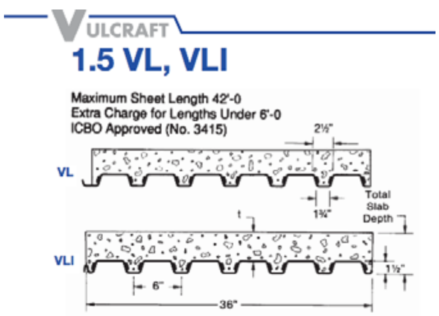


***Figure 20** – Deck data input information*



(N=9) NORMAL WEIGHT CONCRETE (145 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)											
		1 Span	2 Span	3 Span	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-
3 1/2"	1.5VL22	5'-2	6'-11	7'-0	314	259	230	206	186	169	154	141	130	120	111	10
	1.5VL21	5'-9	7'-8	7'-9	331	294	243	218	197	179	163	150	138	127	118	10
(t=2")	1.5VL20	6'-2	8'-3	8'-4	345	306	275	228	206	187	171	157	144	133	124	10
	1.5VL19	6'-10	9'-2	9'-4	372	330	296	268	223	203	186	171	157	145	134	11
	1.5VL18	7'-6	9'-11	10'-2	395	351	315	285	260	238	199	182	168	156	142	12
33 PSF	1.5VL17	8'-2	10'-6	10'-10	397	353	316	286	261	239	221	183	169	157	145	13
	1.5VL16	8'-8	11'-0	11'-5	397	353	316	286	261	239	221	205	169	156	145	13
4"	1.5VL22	4'-11	6'-6	6'-7	342	301	267	240	216	196	179	164	151	139	129	11
	1.5VL21	5'-5	7'-3	7'-4	385	318	283	253	229	208	190	174	160	148	137	12
(t=2 1/2")	1.5VL20	5'-10	7'-9	7'-11	400	356	295	264	239	217	198	182	167	155	143	13
	1.5VL19	6'-6	8'-8	8'-10	400	383	344	311	259	235	215	197	182	168	156	14
	1.5VL18	7'-1	9'-5	9'-7	400	400	365	330	301	251	229	211	194	180	167	15
39 PSF	1.5VL17	7'-8	10'-0	10'-4	400	400	366	331	302	277	230	211	195	180	168	15
	1.5VL16	8'-2	10'-6	10'-10	400	400	365	330	301	276	255	211	194	180	167	15
4 1/2"	1.5VL22	4'-8	6'-3	6'-4	392	345	307	275	248	225	205	188	173	159	147	13
	1.5VL21	5'-2	6'-11	7'-0	400	364	324	290	262	238	217	199	183	169	157	14
(t=3")	1.5VL20	5'-6	7'-5	7'-6	400	400	338	303	274	249	227	208	192	177	164	15
	1.5VL19	6'-2	8'-3	8'-4	400	400	393	328	296	269	246	226	208	193	179	16
	1.5VL18	6'-8	8'-11	9'-2	400	400	400	378	315	287	262	241	222	206	191	17
45 PSF	1.5VL17	7'-3	9'-6	9'-10	400	400	400	378	345	287	263	241	223	206	191	17
	1.5VL16	7'-9	10'-0	10'-4	400	400	400	377	344	315	262	240	222	205	190	17
5"	1.5VL22	4'-6	6'-0	6'-1	400	391	347	311	280	254	232	213	195	180	167	15
	1.5VL21	4'-11	6'-8	6'-9	400	400	366	328	297	269	246	225	207	191	177	16
(t=3 1/2")	1.5VL20	5'-3	7'-1	7'-2	400	400	382	343	310	281	257	236	217	200	186	17
	1.5VL19	5'-10	7'-11	8'-0	400	400	400	370	335	304	278	255	235	218	202	18
	1.5VL18	6'-4	8'-7	8'-9	400	400	400	394	356	324	297	272	251	233	216	20
51 PSF	1.5VL17	6'-11	9'-1	9'-5	400	400	400	400	357	325	297	273	251	233	216	20
	1.5VL16	7'-4	9'-7	9'-10	400	400	400	400	388	323	295	271	250	232	215	20

***Figure 21** – Diagram and loads table for* 1.5VL

The deck itself, with the assistance of the concrete (once fully cured), is capable of spanning longer distances, but shoring is a tedious process in construction, and, if possible, best to avoid. A complete 5" deck system spanning 7'-0" can withstand up to a full superimposed load of 280 psf. This means that this deck can be used in all locations of the building from the heavy loads at the center of the building @ 255 psf to the typical load of 100 psf. The deck was also arranged differently in the center bays than the rest of the building.

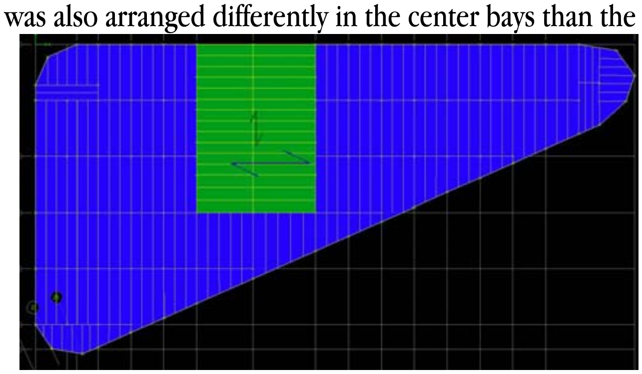


***Figure 22** – Designation of arranged deck*

The grid lines running from east to west have the spacing of 21'-0". The grid lines running from north to south have the spacing of 36'-0". Overall, the building then had a bay dimension of 36'-0" N-S and 21'-0" E-W. The center atrium area had both 36' and 40' spans. Loads are also heavier in the center bays, from 100 psf live load to 250 psf.

This new layout will also greatly decrease the depth of the beams and girders of the system, while still not seriously compromising the architectural layout of the building. 40'-0" bays remained at the center of the building, since the 3-story atrium lobby is in that location. There is no need to try to resize the entire space for a 4'-0" adjustment. As such, all columns were re-spaced according to the center columns.

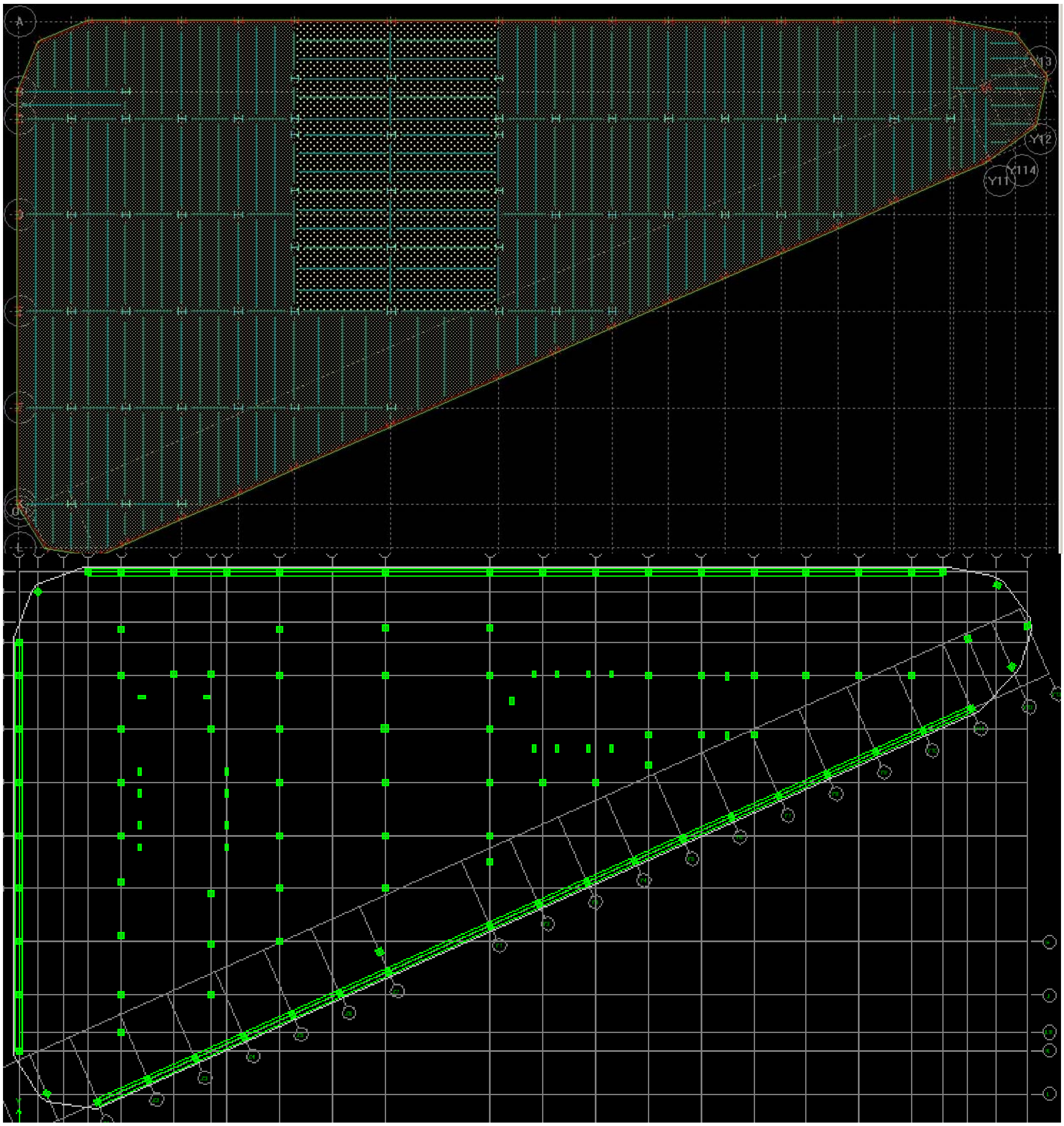


***Figure 23** – Direct comparison of proposed to current column layout*

A second model was also built in RAM, as it was the more "comfortable" computer modeling software than ETabs. So RAM was used as a check up to ensure that the ETabs model's results were generally in the same range.

Analysis Results

Since ETabs uses a system of continuous iterative processes, it will continue to design and analyze until the model reaches a uniform design throughout the floor. This has its benefits and setbacks. For one, it is most obvious that girders will need to be larger sizes than the minor beams (which span within the bays). This is one of the reasons that RAM was used to ensure a more complete design. ETabs concluded to assign a uniform size of W16x26 to all the beams on each floor.

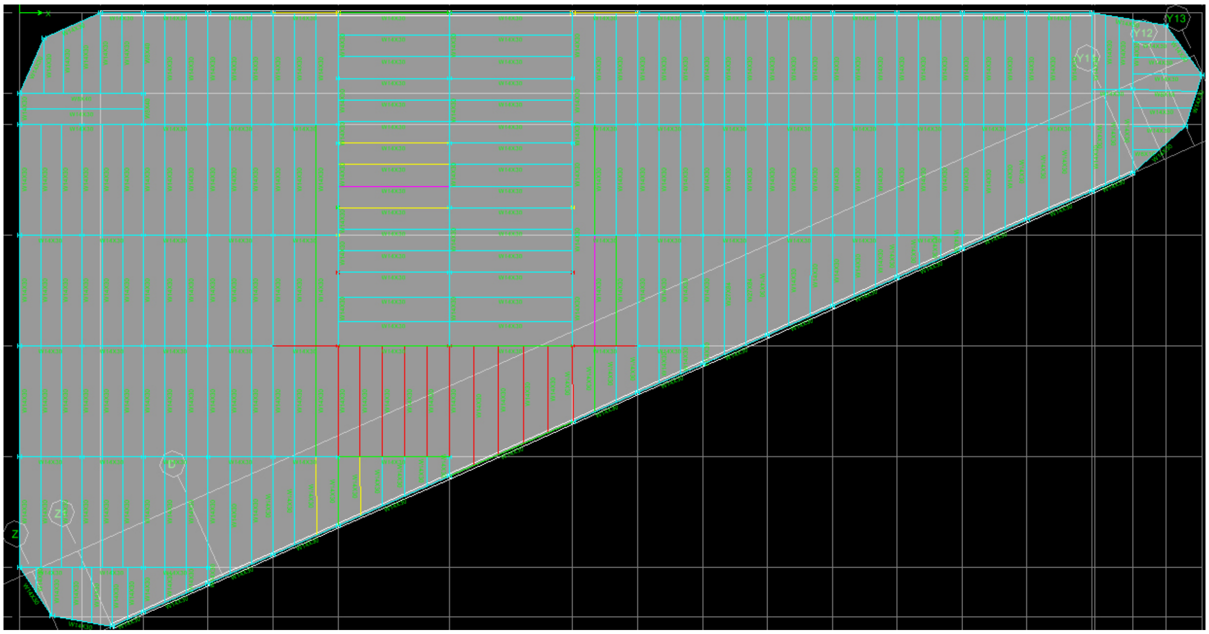


***Figure 24** – Etabs Results*

The light blue indicates that the designed beams are satisfactory to withstand all load combination possibilities. The red, pink, and yellow indicate higher-stressed beams, which suggest that perhaps larger sizes would be a better design for those areas. In order to ensure the design as safe, the ETabs model was compared to the results of the RAM model.

The RAM model showed some serious number differences than the ETabs model. There are several possibilities with the discrepancies. Since not a single beam or column was designed it left the possibility of hundreds of different solutions to the same answer. Running the deck either parallel or perpendicular to the beams and girders will also have an effect on the design. Finally, the process under which each software delineates to design a building is completely different. While ETabs analyzes the entire building at the same time (hence a much longer analysis rendering time), RAM allows you to design step-by-step, first dealing with gravity only loads, then moving onto creating a fully framed building, and then finally applying all possible load combinations (including lateral loads).

As a result, the RAM model showed typical W8x10's for interior bays and W12x19's in the center bays, while girders ranged from W18x40's to W30x90's (in the center bay). This is a considerable difference from the ETabs model. The results also seem a little more consistent as to what was expected.

***Figure 25** – RAM results*

Columns were also designed to range from W10x33 to W14x398 in the RAM model. However, the ETabs model has a general layout of W16x26's on the top 4 stories to be sufficient to withstand the combined lateral and gravity forces. Differing answers could be due to the fact that moment framing is analyzed with different codes and analysis process, just like the beams.

Overall, the final system of beams can be used without to much sacrifice in floor thicknesses. The architect wanted to give the interior designers at least 9'-0" throughout the building, and as long as the decks are running long-ways with the beams, most of the beams can remain within 12". Some adjustments to the MEP system may be required, as ceiling spaces will not be available in the center span of the center bays (due to an already 30" girder in the location) and also on the girders running east-west throughout the building.

Deflection, or story drift, of the building came to be a total of 1.76". The max allowable drift for the building is L/480, which is 3.25". Although the building drifts more than the concrete system, it is still satisfactory to the needs of the building.

More detailed take-offs and beam/column summaries are in the Appendices in the back of the report.

Conclusion

The final size of the floor thickness is 5" for deck and 12" for the wide-flange beam. As long as the MEP systems running alongside the girders, there shouldn't be any other serious issues. 17" will not affect the architect's desire for 9'-0" ceilings, since floor-to-floor heights are 11'-8", leaving a total space of 15" for the MEP systems to run through before dropping below the 9'-0" mark.

Moment frames that were placed on the exterior columns and beams resulted in a reasonable design. A story drift of 1.76" shows that the building will be stiff enough to resist all necessary lateral loads.

Although all results suggest that a steel system might have been a better solution, there are several points to take note of. For one, the current economy for steel in Washington, D.C. is not well known, and the construction of steel buildings in D.C. is not very common. Steel prices could be higher than usual especially with the additional costs of moment framing.

Secondly, it is important to note that with the change of the model from concrete to steel, the controlling lateral load was wind. This could be due to a lighter overall building, but further analysis should be done in order to ensure that this is the case.

The final point to note is that the building will be constructed essentially to watertight condition (meaning that the building will not have most of its interior finished). The space is personalized by each of the 3 tenants, and 2 of the 3 tenants desired to have room for a possible custom-made staircase in their space. In concrete, the process of creating a slab penetration and a new staircase isn't too difficult compared to a steel system. Steel has a tendency to lay out in a continuous pattern. For example, most bays have 2 interior beams to create 3 equal spaces. The installation of a staircase would require cutting through an entire bay, resizing the beams and layout, and the purchase of new beams to support the new deck opening. This process is incredibly tedious, difficult, and time-consuming.

Ultimately, in the end, even though steel framing allows the possibility of a faster construction pace, it does not compensate for the inconveniences that steel systems cause i.e. extra costs and little flexibility.

Alternative Lateral System Analysis – Shear Walls

The current lateral system of 901 New York Avenue is moment framing. This assumes that the lateral loads will be resisted through a column-and-beam interaction. It was concluded during the design phase of the building that shear walls would have been too expensive and time consuming in terms of construction. This section of the report will address this assumption and see whether or not shear walls could possible have been a better solution than the current system. ETabs will be used to model a proposed building and test the use of shear walls.

When first looking to design a shear wall system in a building, you must first find a place in the building to place the shear walls. There are several locations that seem to be a possible wall location. First, there are the elevator shafts on the west and east side of the building. There are also two set of staircases on each side of the building as well. The current model (the same used for initial lateral checks) did not have slab openings at these current locations. This was done in order to simplify the analysis process. Although slab openings have a huge effect on rigid diaphragm design, the current system had a series of support beams and columns directly surrounding the location of the slab openings, and so proper precautions were taken to ensure the rigidity of the openings. However, for the design of shear walls, the slab openings had to be accounted for, as the shear walls would surround the openings.

In addition to the openings in the plan view, openings were added to the areas of the elevator and staircase doors. This ensures that the model will be designed to the most detailed specifications.

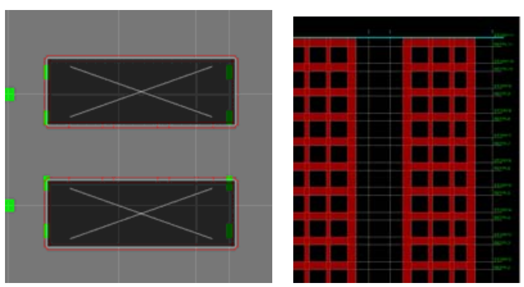


***Figure 26** – Slab and shear wall openings*

Design

Initially, the model was created to design the shear walls with a new column design for the building. Columns were standard at 5,000 psi concrete throughout all floors and stories. The shear wall was initially designed at 10” thick. Shear walls were also initially located at all 4 elevator shafts and one staircase on the east side. More stiffness was assumed to be required on the east side of the building as previous models had showed more deflection on that side of the building.

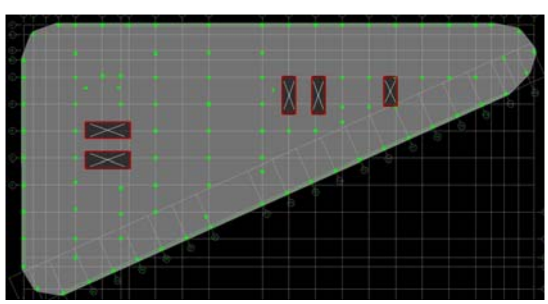


***Figure 27** – Shear wall designation (in red)*

The first model showed that with the use of 5 shear wall systems in one building, the 901 NYA would be incredibly stiff. Story drifts totaled to be 0.167" compared to the 0.41" using moment framing. Although this proves that shear walls would be a better solution, it must be noted that shear wall construction takes a great amount of time. As a result, a second model was created using only one of the elevator shafts on the west side and one elevator shaft on the east side. In addition to the deletion of two shear wall sections, columns were reset to a dimension of 16" square columns with 8-#8's instead of the previous 26" 8-#9's.

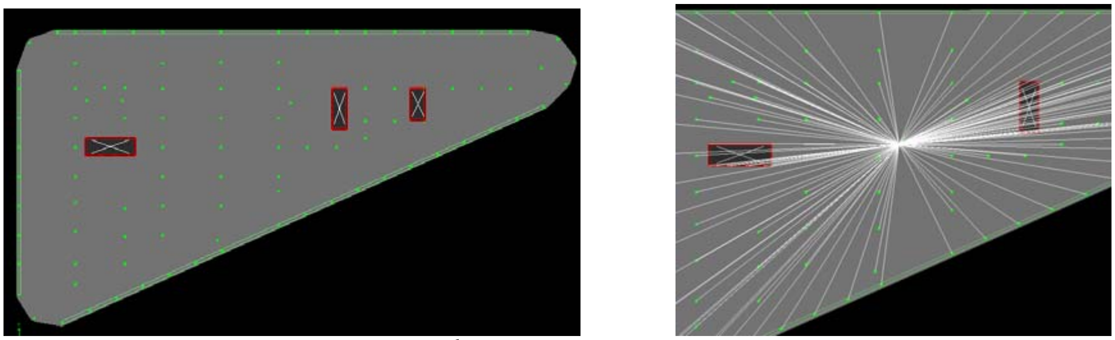


Figure 28 – 2nd shear wall designations (in red)

This second model had a story drift of 0.38", which is still a very reasonable drift. It is important to remember that although it is over twice as much drift as the previous model, there are 3 less shear wall sections to build (which run the entire height of the building). This decreases the time and effort in building the structural components of 901 NYA. Also, now all the columns are 16" squares with 8-#8's. Smaller bars mean cheaper costs for all reinforcement, and smaller columns with a standard 5,000 psi concrete means that only higher strength concrete will be needed at the locations sub-grade. (For future note, it was assumed for this model that the sub-level parking garage will not be changed by any means. This is because all sub-grade components essentially have no effect on the above-grade lateral changes. The following is an assessment of effects in reinforcing steel due to the change in shear walls.

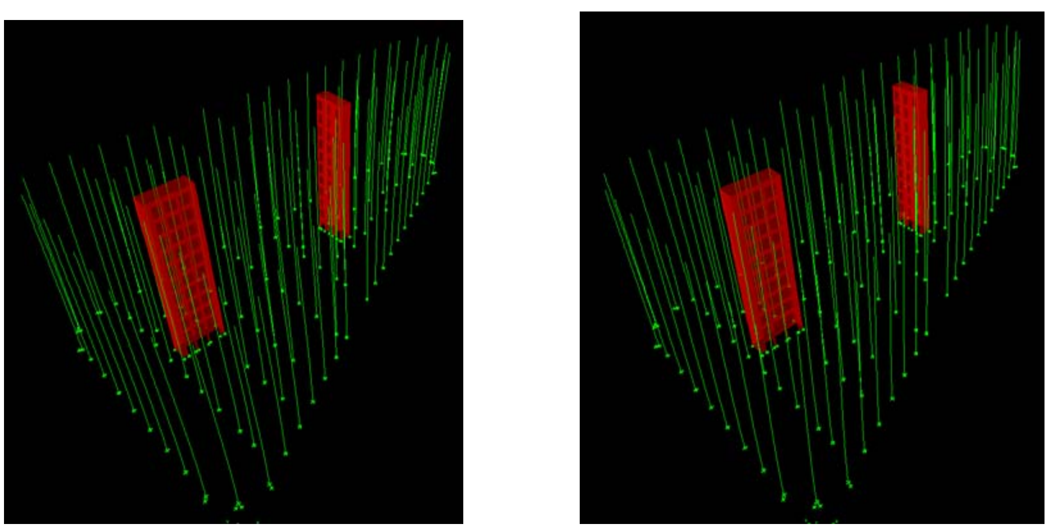


***Figure 29** – Drift effects on shear walls and columns*

Amount of rebar in columns before shear walls:	8-#9's per column, 255 columns 130'-0" length per rebar #9 = 3.4 lb per foot **Total Weight: 450.84 tons**
Amount of rebar in columns after shear walls:	8-#8's per column, 255 columsn 130'-0" length per rebar #8 = 2.67 lb per foot **Total Weight: 354.04 tons**

RS Means measures the cost of rebar in tons. The savings from using #8 bars instead of #9 bars is about 100 tons, or **$89,500**. That really isn't much, but that is just for the reinforcing. Savings also occurred for the concrete used. Not only were the columns smaller (from 26" to 16"), but all columns were switched from varying strengths to a single strength of 5,000 psi. The following will address only cost difference, but take note that constructability will improve incredibly, since only a single type of concrete needs to be poured.

Amount of concrete before shear walls:	26" square columns, 255 columns 130'-0" full height, 11" slab/floor deletion 4,000 psi: (2,382 CY)*($108) = $257,256 5,000 psi: (954 CY)*($114) = $108,756 6,000 psi: (954 CY)*($121) = $115,434 8,000 psi: (954 CY)*($128) = $122,112 **Total Cost: $603,558.00**
Amount of concrete after shear walls:	16" square columns, 255 columns 130'-0" full height, 11" slab/floor deletion 5,000 psi: (2033 CY)*($114) = $231,762 **Total Cost: $231,762.00**

The total savings then from using a shear wall system instead of a moment framing system is $461,296.00, or roughly half a million dollars. Now the cost of actually building the shear walls themselves must be taken into account.

The shear walls were designed at 10" thick with #5 bars at 12" o.c. using 5,000 psi concrete. The shear walls on the west side add up to 96'-0" long total. The shear walls on the east side add up to 76'-0" long total. The following is a summary of the costs of building these shear walls

Amount of rebar for shear wall:	#5's @ 12" o.c. = 172 bars (each side of wall) #5 = 1.043 lb per foot Total Weight: 24 tons @ $795 **Cost: $19,080.00**
Amount of concrete for shear wall:	10" wide by (96' + 76') long = 691 CY 5,000 psi: (691 CY) * ($114) = $78,774 **Cost: $78,774.00**
	Total Cost: $97,854.00

The final cost for building a shear wall system was roughly $100,000.00. Compared to the savings, it can be concluded that it would save about $400,000.00 if a shear wall system was used. Although building shear walls would require more work than just laying out the formwork for the columns, it shouldn't affect the scheduling of the building, as shear walls can be formed, poured, and cured at the same speed as columns in the building. As such, shear walls have no effect on the overall construction schedule.

The final concern is whether or not it would be worth it to go through the extra efforts to save $400,000 on the project. The addition of shear walls wouldn't complicate the construction schedule too much more than it already is. The required lateral resistance would only need to sets of shear wall systems in the building, and both locations would not have an effect on the rest of the building's layout. In fact, with smaller columns, the interior designers would have more flexibility with their spaces.

Conclusion

Although the current building model is fully capable of resisting lateral loads without the use of shear walls, it can now be suggested that shear walls could have saved costs and simplified the design of the building overall. Columns only need to withstand gravity loads (since deflection requirements already limit the building's overall drift), and the proposed location of shear walls would not interfere with any other architecture concepts of the building.

Breadth Option 1: Construction Management

Construction management is a crucial role in a building's erection process, as poor construction management can cause a delay in the building's schedule and inaccurate cost estimates. These are both crucial assets to a building, especially since 901 NYA is considered to be prime real estate inside the city.

Project Organization

The project is set up as a design-bid-build layout. David Carter Scott was the architect, and Clark Construction was the General Contractor. The GC had a lump sum agreement with the owner, and the other contracts are GMP arrangements. If money was saved during the project, 70% was returned to the owner, and 30% was to be given to the GC. Some noteworthy subcontractors are SK&A (structural engineering), Delong Hampton & Associates (civil engineering), Girard Engineering (mechanical engineering), and Goldin & Stafford (earth contractor). The organization of the project will not be effected to any changes due to the structural alternatives.

Site Layout

The site consists of 4 surrounding streets: K Street, New York Avenue, 10th Street, and 9th Street. Some aspects of the building design were used to help with excavation and construction. For one, the loading dock for 901 NYA is also to be used for one of the ramp locations. The openings for elevators and staircase (both on either side of the building) made a great spot for crane locations. As a result, the same location can be used for both concrete and steel erection. Two tower cranes were used in the construction of 901 NYA, and so two shall also be used for the construction of a proposed steel building.

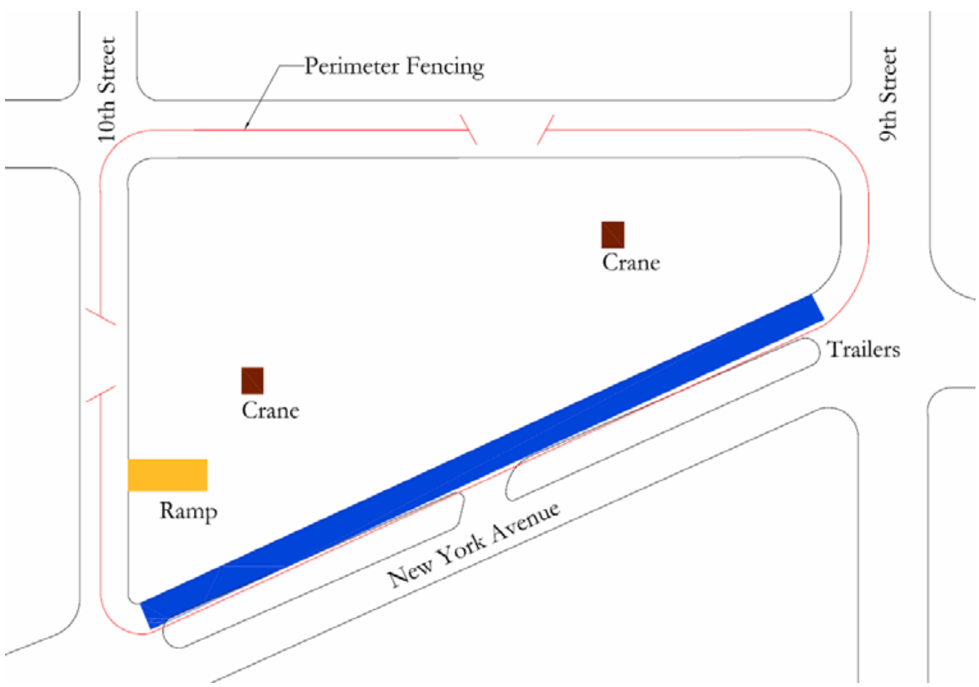


***Figure 30** – Construction Layout of 901 NYA*

The second ramp is located on the west side of the site on 10th Street. Dumpsters were placed on the three corners of the building for ease of access. Trailers were located along the New York Avenue side of the site and were eventually placed into the building as the fit-out phase began.

In conclusion, whether or not a steel or concrete building will be built will not have an effect on the building's current site layout. Crane locations are already stated and present, assembling locations can still remain the same, and there is plenty of room for the layout of steel members.

Scheduling
The official date of the Notice-to-Proceed was August 19, 2002. After mobilization to site and some site excavation, the area was cleared and ready for concrete pouring.

P4 Level Concrete Pour – about 2 months
    Footings and Grade Beams – 32 days
    Foundation Walls – 32 days
    Columns – 24 days
    Slab-on-Grade – 8 weeks
P3 Level Concrete Pour – about 2 months
    Foundation Wall Continuation – 27 days
    Elevated Slab – about 2 months
    Columns – about 2 months
P2 Level Concrete Pour – about 1.5 months
    Foundation Wall Continuation – 24 days
    Elevated Slab – about 1.5 months
    Columns – about 1.5 months
P1 Level Concrete Pour – about 1 month
    Foundation Wall Continuation – 29 days
    Elevated Slab – about 1 month
    Columns – about 1.5 months
First Floor Concrete Pour – about 2 months
    Foundation Wall Conclusion and PEPCO Vault – about 1 month
    Elevated Slab (and PT) – about 2 months
    Columns – about 2 months
Second Floor Concrete Pour – 1 month
    Elevated Slab (and PT) – 1 month
    Columns – 1 month
Third Floor Concrete Pour – about 1.5 months
    Elevated Slab (and PT) – about 1.5 months
    Columns – about 1 month
Fourth Floor Concrete Pour – 1 month
    Elevated Slab – 1 month
    Columns – 1 month
Fifth through Eleventh Floors Concrete Pour – about 2 months each floor
    Elevated Slab – about 2 months each floor
    Columns – about 1 month each floor
Pre-Cast Panels Installation – about 3 months
Curtain Walls Installation – about 2 months

Although there were some initial MEP systems installed into the building, final designs for interior designing and MEP layout was decided by a separate contract from the building construction. Another point to mention is that Clark Construction worked on a tight schedule, overlapping pours and framing on different floors at separate times. This was to enable all concrete pouring to be finished for all 11 floors in less than a year. The first pour began on November 4, 2002, and the final pour concluded on October 29, 2003. This is a great feat, as there are more than 45,000 square feet per floor and 16 floors to pour.

The proposed alternative of a composite steel building will greatly change the schedule of the building. Steel erection has a tendency to be more expensive but can be built at a much quicker pace than concrete buildings. To ensure that this assumption is correct, the building of a steel structure shall be tested to see if it can be built in a shorter time span than the current schedule.

The substructure will remain the same for the most part, so excavation and sub-grade work will not be a determining factor.

P4 Level Concrete Pour – about 2 months
P3 Level Concrete Pour – about 2 months
P2 Level Concrete Pour – about 1.5 months
P1 Level Concrete Pour – about 1 month

From historical content, RS Means has steel buildings capable of building per story columns in 1 day, depending on the number of crews and number of cranes. Fortunately for 901 NYA, since 2 cranes are available, we can assume a very quick schedule. However, the installation of deck and composite components take longer than non-composite decks, as it requires the addition of shear studs. 901 will also be unique in that the placement of abnormally-shaped decks would increase the amount of effort and time put into placing these decks.

Floor column erection and placement – about 1 full day
Floor beam erection and placement – about 1 full day
Composite deck installation – about 4 days
Concrete pour – 1 day (using crane and bucket with 2 cranes)

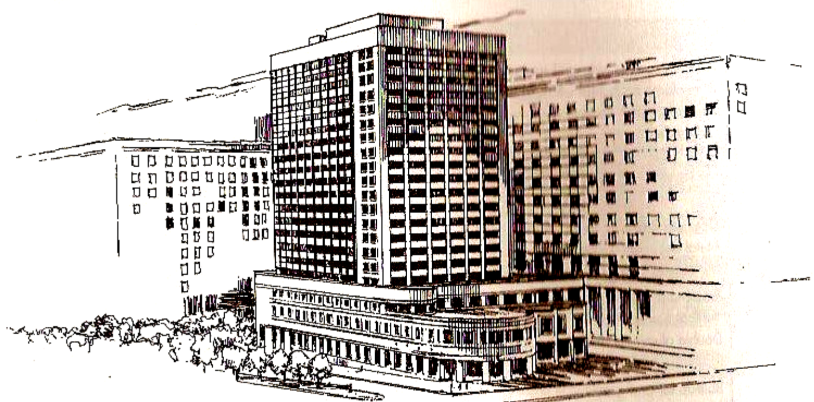
Once the deck has been placed, it would be a good idea to wait until higher stories have placed their decks before starting to pour concrete. As a result, concrete will not be poured onto each respective level until the deck on the third story above it has been installed. This repetitive process will continue from floor to floor, and in the same designation time as in the concrete construction process, construction of the shell of the building shall occur as well. The final length of construction is **54 days**, not including Saturday and Sunday. 54 days then is about 11 weeks, which is about 3 months.

In comparison to concrete, the steel building is built much more quickly (over ¼ of the time for concrete). However, some factors were not taken into account for specific scheduling. Although it is known the moment connections require more time and effort than simple connections, it wasn't configured as to how many days moment connections would delay over a simple system.

Cost Estimate

The final cost of 901 NYA was estimated to be about $54 million. In order to compare costs between the current concrete building and steel alternative, a square-foot cost estimate was done. RS Means 2005 was used for the estimate, as it would project the proper costs at the time of finished construction.

COMMERCIAL/INDUSTRIAL/ INSTITUTIONAL | **M.480** | **Office, 11-20 Story**



Costs per square foot of floor area

Exterior Wall	S.F. Area	120000	145000	170000	200000	230000	260000	400000	600000	800000
	L.F. Perimeter	420	450	470	490	510	530	600	730	820
Double Glazed Heat Absorbing Tinted Plate Glass Panels	Steel Frame	120.35	116.00	112.35	109.10	106.65	104.80	99.15	96.20	94.30
	R/Conc. Frame	120.90	115.50	111.15	107.25	104.30	102.10	95.55	92.00	89.75
Face Brick with Concrete Block Back-up	Steel Frame	119.85	115.20	111.50	108.15	105.60	103.75	98.20	95.10	93.20
	R/Conc. Frame	113.75	109.10	105.45	102.15	99.70	97.80	92.40	89.35	87.50
Precast Concrete Panel With Exposed Aggregate	Steel Frame	118.95	114.35	110.75	107.45	105.00	103.15	97.75	94.70	92.95
	R/Conc. Frame	113.10	108.55	105.00	101.75	99.40	97.55	92.25	89.25	87.45
Perimeter Adj., Add or Deduct	Per 100 L.F.	6.95	5.75	4.90	4.10	3.55	3.10	2.10	1.40	1.00
Story Hgt. Adj., Add or Deduct	Per 1 Ft.	2.80	2.50	2.25	1.90	1.80	1.55	1.25	.95	.80

For Basement, add $28.20 per square foot of basement area

***Figure 31** – RS Means excerpt*

For our purposes, we will use RS Means's "Office, 11-20 Story" example as our model. Out of the 3 exterior wall façade options, the "Precast Concrete Panel with Exposed Aggregate" seems to be of best fit for our building. Both steel and concrete framing prices will be used.

Since our building has an overall size of about 580,000 square feet, we will need to find a price between 400,000 and 600,000 square feet. For steel, it comes out to be $95.01/s.f. and for concrete $89.55/s.f. The actual building's cost is $93.10/s.f., but it must be noted that this is the price of the building with the interior unfinished. That is, the mechanical, electrical, and interior finishes are not included into the actual price of the building in the contract. If these prices were factored out of the estimate prices in RS Means, the adjusted prices are $61.75/s.f. for steel and $58.21/s.f. for concrete.

Now the prices must be adjusted due to perimeter, story height, and location factor. The building's perimeter (per floor) is 1021 linear feet and story height is 11'-8". The model's perimeter is 717 linear feet and a story height of 10'-0". The location is Washington D.C.

Steel:	[$61.75 + ($1.47)(3.04) + ($0.98)(1.67)] * 0.97 = $65.83
Concrete:	[$58.21 + ($1.47)(3.04) + ($0.98)(1.67)] * 0.97 = $62.40

In comparison to the usual price for a concrete-framed building, 901 NYA can be considered to be a very expensive building. There are several reasons as to why it cost so much more than the usual model concrete building. The greatest factor, however, is the complication of post-tensioning. Post-tensioning prices are not included in the square-foot estimate from RS Means, and the price to install the tendons and put tension in the tendons greatly increases the price. The current building system valued at $93.10/s.f. makes 901 NYA almost 150% of the usual cost for a concrete building.

It is also important to note the cost of the steel alternative. This method states nothing about possible moment framing of the building and the cost of all the complicated connections. It is difficult to estimate exactly how much it would cost to build a moment framed steel building over a normal steel building, so although steel costs more than a typical concrete building (and moment framing will increase the cost of construction), it cannot be determined whether or not it will be more costly than the current concrete post-tensioned system.

Shear Wall Alternative

The use of shear walls as an alternative to the current system has an effect on construction. In terms of price, it was previously stated that the suggested alternative system would save roughly $400,000. In terms of scheduling, shear walls should have no effect on the schedule, so it would neither delay nor accelerate the construction time. In terms of feasibility, shear walls do not take any particularly complicated method. They are built just like foundation walls (which are already built for 4 stories sub-grade) and have reinforcement throughout.

Even cheaper options can be found through a more in-depth analysis. It's possible that a shear wall with just one side of reinforcing can withstand the lateral effects (the proposed system drifts only a meager 0.47") and even further decrease the size of the columns. Lower strength concrete can even be used at higher stories due to progressive axial loads.

Conclusion

In view of all three aspects (feasibility, time, cost), it seems that the proposed composite system may not be in the best interest of the owner. Although floor thicknesses weren't entirely compromised, the re-assignment of the MEP systems and the cost of switching from a concrete building to a steel building suggest that steel might complicate matters, although overall saving time.

A shear wall alternative to the current building seems to be the better suggestion of the two alternatives. There is a definite cost saving in building the walls instead of depending on moment framing, and there is no effect on the project schedule.

Breadth Option 2: LEEDS Certification for an Existing Building

This breadth study shall observe the possibility of 901 NYA to receive the status of a LEEDS certified-building by the U.S. Green Building Council. LEEDS stands for Leadership in Energy and Environmental Design and is a building rating system nationally recognized "for the design, construction, and operation of high performance green buildings."

Green building design has become a popular topic in recent years, as many people are presently concerned with the effect of buildings (and the construction of them) to the environment. LEEDS certification goes through a process, depending on the status of your building, and analyzes different aspects of the building and its surrounding effects to see if the building is environmentally friendly.

Sustainable Sites	(14 Points Possible)
Water Efficiency	(5 Points Possible)
Energy and Atmosphere	(23 Points Possible)
Materials and Resources	(16 Points Possible)
Indoor Environment Quality	(22 Points Possible)
Innovation, Quality, Upgrades and Maintenance	(5 Points Possible)
Total Possible	**85 Points**

The accumulation of points in each category is an indication of the type of certification that a building can qualify for. Currently there are 4 classifications of certification.

Certified	32 - 39 points
Silver	40 - 47 points
Gold	48 - 63 points
Platinum	64 - 85 points

Certification is also divided into several building type categories. A building can be certified as new construction, existing building, existing historic building, commercial interiors projects, core and shell development, homes, and neighborhood development. 901 NYA will be considered to be under the existing building category.

There are several benefits of having a certified building. First, it is good for the tenants and the owner's reputation. It shows that the occupants and owner cares about the environment and is willing to put in the extra effort (and usually money) to actively participate in making a healthier environment. A green building is also good in the long run, as it assumes that the building will consume less energy and recycle more resources, such as rainwater.

901 NYA will be taken from an existing building perspective as it has already been constructed and been in use for more than 2 years. The following is a breakdown of the points that 901 NYA is eligible with its current condition.

Sustainable Sites	5 points (of 14)
Water Efficiency	1 point (of 5)
Energy and Atmosphere	4 points (of 23)
Materials and Resources	0 points (of 16)
Indoor Environment Quality	14 points (of 22)
Innovation, Quality, Upgrades and Maintenance	0 points (of 5)
Total Earned	**23 points (of 85)**

Since 23 points still does not qualify for even the lowest classification of certification, 901 cannot be considered a green building under the standards of USGBC. However, improvements to the building's current condition can make it a possible candidate for the gold certification.

In the "Sustainable Sites" category, points can be gained through the use of green spaces surrounding the building. This is usually difficult within the city, as there is little space to build a park to satisfy the green building requirements. However, 901 NYA has a special opportunity to do work in Mount Vernon Square. This square is essentially used for the purpose of walking from hotels to the nearby Convention Center. Currently there is artificial grass throughout the parking lot. A diversified use of green plants and the parking lot itself can benefit not only the general environment of the otherwise "artificial" lot, but also gain some points for certification**. Total Extra Points: 8 Points**

Collection of rain water could also help in improving 901 NYA's status as a green building. Since the drainage system on the roof already collects and removes all rain water from the building, a new system that allows the collection and storage of rain water allows the possibility of rain water used as "gray water." Gray water is used for things such as toilets, hand washing faucets, and any other system that does not allow the actual drinking of the water. There are setbacks to this suggestion, however. Location of storage tanks would be difficult, as the building is already 4 levels below grade (about 50'-0") and also located in the city. As a result, this suggestion will not be accounted for in the new analysis. **Total Extra Points: 4 Points**

As there is currently no points in the "Materials and Resources" category, it is only due to the fact that no specific information could be proposed for the addition of points. However, a simple documentation of recycling of resources (such as office waste i.e. paper, ink cartridges, cans and bottles, etc.) can gain several points in this category. Although specific numbers cannot be certain, 3 points will be allotted for now, since Washington, D.C. already mandates the recycling of all waste (non-participants are charged with a heavy fine). **Total Extra Points: 3 Points**

The greatest benefit of all the possible improvements would be a Dedicated Outdoor Air System (DOAS). An implementation of this system would allow a benefit of up to 21 points. This is due to its improvement in several categories and the improved use of all the HVAC systems (which tend to intake a great amount of energy throughout the year)**. Total Extra Points: 21 Points**

The final proposal is to change the layout of the parking lot in order to designated spots specifically for carpooling cars. This will gain two points in the "Sustainable Sites" category. **Total Extra Points: 2 Points**

Sustainable Sites	10 points (of 14)
Water Efficiency	1 point (of 5)
Energy and Atmosphere	11 points (of 23)
Materials and Resources	6 points (of 16)
Indoor Environment Quality	18 points (of 22)
Innovation, Quality, Upgrades and Maintenance	4 points (of 5)
Total Possible Points	**50 points (of 85)**

If all of these proposals are implemented, 901 NYA is capable of qualifying for Gold certification. It must be noted, however, that these upgrades to the facility and its surroundings will not be cheap. The owner and tenants will have to manage a schedule and see how long it will take before their loss will be regained in energy savings, and then decide whether or not the new upgrades (and the classification of a LEEDS-certified building) would be worthwhile. In our case, the tenants are 3 well-known and respected law firms, while the owner also has a good reputation throughout the country. Money could very well not be too much of an issue.

Other setbacks include the complications of the installation of these improvements. If these upgrades would ever require more than 50% of the inhabitants to not "live" in the building, then the building would need to be re-categorized as a new building construction. The installation of a DOAS system is extremely complicated, and many factors (structural supports, electrical and mechanical supports, etc.) will be need to be re-checked to see if it will be capable of supporting the new system. The adjacent lot might not be allowed to be renovated for the integration of green plants as well.

In conclusion, even though a Gold certification can be achieved with the suggested upgrades to the building, it is most likely than not that the owner and tenants would not want to lose valuable time due to construction and installation of new systems. At best, the most realistic solution would simply get 901 NYA a general certified classification.

Final Conclusion

In this report, 901 NYA has been reviewed for its current design, checked for gravity and lateral strength, and compared to several different alternative solutions, two of which were further analyzed and detailed for a more in-depth comparison with the current system. Furthermore, both alternatives were analyzed with construction in mind. 901 NYA was then also checked to see if there were possibilities of LEEDS certification.

Current Design: The current design is absolutely the best possible design for minimal floor thickness, large bay areas, and open floors. The extra costs that caused 901 NYA to be almost 150% of the typical cost for concrete buildings shows that it was more crucial to have an aesthetically pleasing building than a cost-efficient one.

Composite Alternative: Composite design shows that it is very possible to make steel work even with the tight criteria of long spans. A compromise of 4' shorter spans helps make members an even more realistic possibility. However, with some girders coming out to be almost 3' in depth, it really does depend on whether or not the interior designers would be able to work around those extremely deep beams. Also, even though construction time is cut almost by a quarter, costs may sky rocket. Current economy for steel in D.C. shows that it is very expensive to build with steel. Fabricators may not be available at time of construction as well. In the end, it will depend on the owner's personal desires and his/her concern for time constraint over money.

Shear Wall Alternative: Shear walls proved to be a very possible alternative to the current system. The new system creates a column size that is 38% of the current size, while also reducing the number of reinforcement. Of course, the catch is that those savings counter with the costs of building the shear wall itself. Even then, the proposed alternative saved more than $400,000. Without affecting the construction schedule at all, shear walls could have been a better solution than the current system. Once again, it is dependant on the owner as to whether or not they mind a 10" solid wall system used around their elevator shafts.

LEEDS Certification: 901 NYA was not built with the environment in mind. The current HVAC systems draw an immense amount of power, all rain water is sent directly to sewage, and there is no "greenery" to be seen except for the few isolated trees down New York Avenue itself. With its current system, 901 NYA cannot achieve even the lowest certification that USGBC offers. However, some mild changes (such as parking spots for carpoolers, showers installation for bike riders, etc.) can improve its points rating to being certified, and an extreme makeover (such as a DOAS, turning rain water into gray water, etc.) can allow 901 NYA to improve to even gold certification. It is to the owner's (and tenants) discretion as to how much money they are willing to spend and to what extent they desire to have a LEEDS-certified building.

Many things were learned from the study of 901 New York Avenue. The requirement to learn several new computer programs will better prepare me for my future profession. A better understanding of the study of lateral loads and its effects on buildings (through load combinations) also allows me to understand more fully the distribution of lateral loads in buildings depending on material (i.e. steel moment frame, concrete shear wall, concrete moment frame). The construction management study will help me in the future to make better decisions when choosing to begin the design of future buildings.

Appendix – Wind and Seismic

Seismic Supplemental Calculations

TA #3 | 901 NYA | Timothy H Park | 1

SEISMIC REDESIGN

OCCUPANCY GROUP	II	
SEISMIC USE GROUP	I	
SEISMIC DESIGN CATEGORY	TBD	
SITE CLASS	C	[TABLE 20.3-1]

$S_S = 15\%$ [FIG 22-1] $F_A = 1.2$ [TABLE 11.4-1]
$S_1 = 5.2\%$ [FIG 22-2] $F_V = 1.7$ [TABLE 11.4-2]

$S_{MS} = F_A S_S = 0.18$ $S_{DS} = 2/3\, S_{MS} = 0.12$
$S_{M1} = F_V S_1 = 0.088$ $S_{D1} = 2/3\, S_{M1} = 0.0587$

$\left.\begin{array}{l} S_{DS} < 0.167 \\ S_{D1} < 0.067 \end{array}\right]$ SDC : A [TABLE 11.6-1, 11.6-2]

$V = C_S W$

$C_S = \dfrac{S_{DS}}{R/I} = \dfrac{0.12}{5} = 0.024$ $R = 5$ [TABLE 12.2-1 : INT MOM. RES. FRAME]

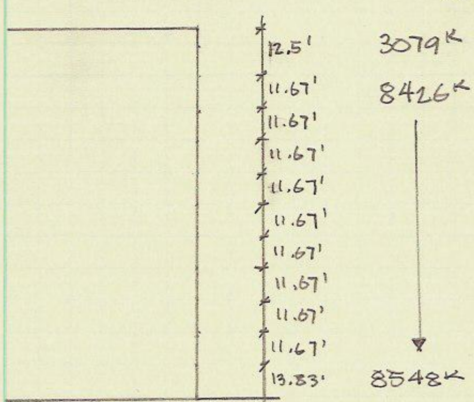
$C_{S_{max}} = \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.0587}{1.28(5)} = 0.00917$ ← CONTROLS

$T_a = C_t h_n^x = 1.28$ $C_t = 0.016$, $x = 0.9$, $h_n = 130'$ [TABLE 12.8-2]

$V = 0.00917(108693^K) = \cancel{996.7^K}$

$F_x = C_{Vx} V$ WHERE $C_{Vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

k: < 0.5 → 1; > 2.5 → 2;] 2.173 $= 1.84$



$V = 0.00917\left(3079 + 8426(9) + 8548\right) = 802^K$

TA #3 | 901 NYA | TIMOTHY H PARK | 2

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

$C_{vx} V$

2 $\frac{(8548)(13.33)^{2.173}}{1123383308} \times 802^k =$ 1.7^k

3 $\frac{(8426)(25)^{2.173}}{1123383308} \times 802^k =$ 6.56^k

4 $\frac{(8426)(36.67)^{2.173}}{1123383308} \times 802 =$ 15.24^k

5 $\frac{(8426)(48.33)^{2.173}}{1123383308} \times 802 =$ 27.51^k

6 $\frac{(8426)(60)^{2.173}}{1123383308} \times 802 =$ 44.11^k

7 71.67' : 0.081 (802) = 65.96^k

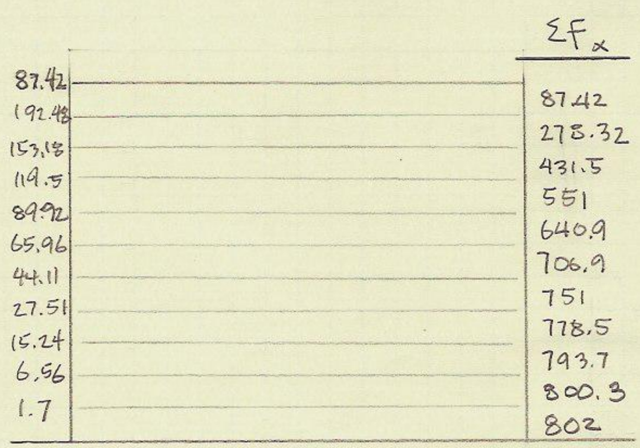
8 83.33' : 0.112 (802) = 89.82^k

9 95' : 0.149 (802) = 119.50^k

10 106.67' : 0.191 (802) = 153.18^k

11 118.34' : 0.24 (802) = 192.48^k

R 130.83' : 0.109 (802) = 87.42^k



(SEE EXCEL SPREADSHEET FOR REST)

Seismic Design for 901 New York Avenue

Level	Height Above Shear Base, *h*	Weight *W* at Height *h* (kips)	Total Weight = Σ*W*	(Wxhx)^k	[(Wxhx)^k] ------------- [(Wihi)^k]	Lateral Seismic Force, *Fx* (kips)	Lateral Seismic Story Shear (kips)	Overturning Moment (kip-ft)
Roof	130	3,079	3,079	12,246,577	0.10900	87.42	87.42	-
11	118.86	8,426	11,505	269,486,085	0.24000	192.48	279.90	973.86
10	107.19	8,426	19,931	215,061,633	0.19100	153.18	433.08	3,266.43
9	95.52	8,426	28,357	167,191,752	0.14900	119.50	552.58	5,054.04
8	83.85	8,426	36,783	125,754,304	0.11200	89.92	642.50	6,448.61
7	72.18	8,426	45,209	90,629,510	0.08100	65.96	708.46	7,497.98
6	60.51	8,426	53,635	64,594,810	0.05500	44.11	752.57	8,267.73
5	48.84	8,426	62,061	38,496,793	0.03430	27.51	780.08	8,782.49
4	37.17	8,426	70,487	21,128,519	0.01900	15.24	795.32	9,103.53
3	25.5	8,426	78,913	9,190,626	0.00818	6.56	801.88	9,281.38
2	13.83	8,548	87,461	2,377,499	0.00212	1.70	802.00	9,357.94
1	-	-	-	-	-	-	-	11,091.66
Σ		87,461	497,421		1.00	802.00		11,091.66

Total Weight:	87,461
Base Shear:	802 kips
Total Overturning Moment:	11091.66 ft-kips

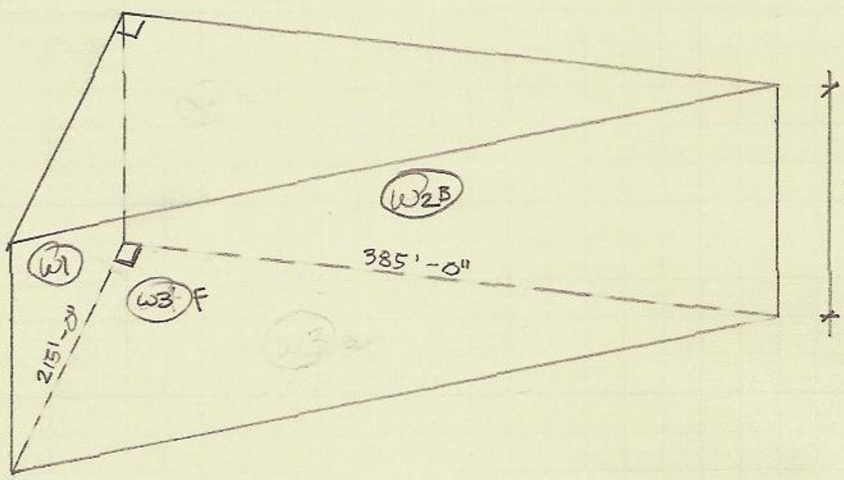
Wind Supplemental Calculations

WIND REDESIGN

NONSYMMETRICAL, CAN'T USE SIMPLIFIED METHOD

EXPOSURE CAT B [SECTION 6.5.6.3]

ASSUME TRIANGULAR BUILDING



$V = 90$ MPH [FIGURE 6-1]

$q_z = 0.00256 K_z K_{zT} K_D V^2 I$

$K_{zT} = 1.0$
$K_D = 0.85$
$V = 90$ MPH
$I = 1.0$

$q_z = 17.63 K_z$ K_z CASE 2 FOR MWFRS
CASE 1 FOR C-C

C_p: WW 0.8
SIDE -0.7
LW -0.5 FOR 11 TO 215'
-0.34 FOR 11 TO 385'

$h/L = 130/215 = 0.$
$130/385 = 0.$

Roof C_p: -0.9, -0.5, -0.3

(SEE EXCEL SHEET FOR MORE INFORMATION)

Wind Loads on 901 New York Avenue

	MWFRS		C & C	
Height (ft)	Kz	qz (psf)	Kz	qz (psf)
0-15	0.57	10.0491	0.70	12.3410
20	0.62	10.9306	0.70	12.3410
25	0.66	11.6358	0.70	12.3410
30	0.70	12.3410	0.70	12.3410
40	0.76	13.3988	0.76	13.3988
50	0.81	14.2803	0.81	14.2803
60	0.85	14.9855	0.85	14.9855
70	0.89	15.6907	0.89	15.6907
80	0.93	16.3959	0.93	16.3959
90	0.96	16.9248	0.96	16.9248
100	0.99	17.4537	0.99	17.4537
120	1.04	18.3352	1.04	18.3352
140	1.09	19.2167	1.09	19.2167
Eave Height = 130'	1.07	18.7760	1.07	18.7760

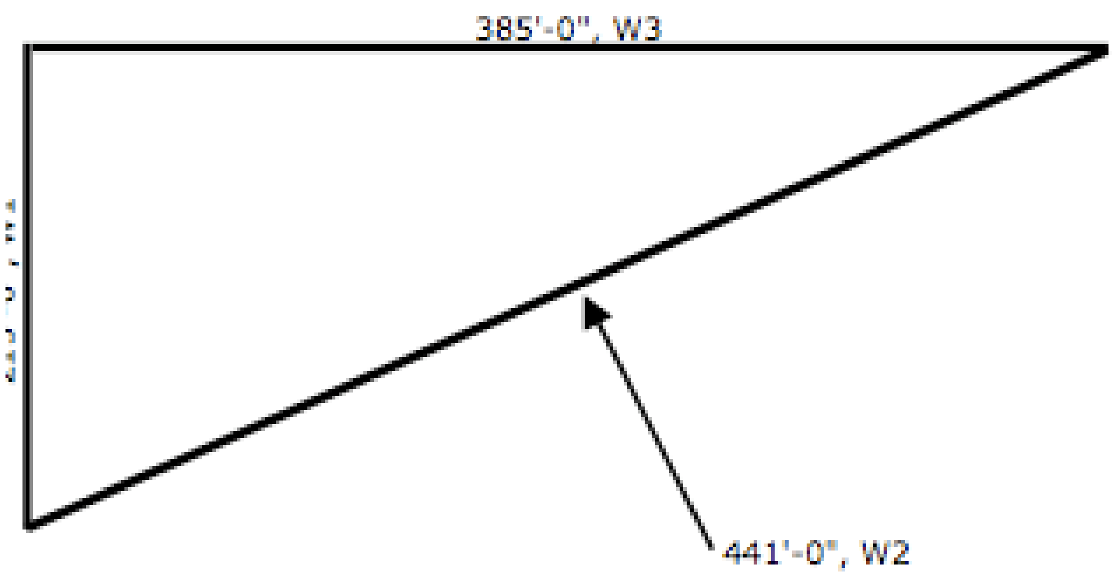


Figure 1: Walls Description for 901 NYA

Trial 1

Surface Type	Surface Designation	Surface	Distance from Windward Edge	L/B or h/L	Cp	External Pressure @ q = 130' (psf)
Walls	W1	WW	-	All	0.80	15.021
	W3	LW	-	1.80	-0.34	-6.384
	W2	Side	-	All	-0.70	-13.143
Roof			0 to h	0.34	-0.90	-16.898
			h to 2h	0.34	-0.50	-9.388
			> 2h	0.34	-0.30	-5.633

Windward Pressures

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	0.80	6.83	3.45	10.21
10.9306	0.80	7.43	4.05	10.81
11.6358	0.80	7.91	4.53	11.29
12.3410	0.80	8.39	5.01	11.77
13.3988	0.80	9.11	5.73	12.49
14.2803	0.80	9.71	6.33	13.09
14.9855	0.80	10.19	6.81	13.57
15.6907	0.80	10.67	7.29	14.05
16.3959	0.80	11.15	7.77	14.53
16.9248	0.80	11.51	8.13	14.89
17.4537	0.80	11.87	8.49	15.25
18.3352	0.80	12.47	9.09	15.85
19.2167	0.80	13.07	9.69	16.45

Leeward Pressures

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	-0.34	-2.90	-6.28	0.48
10.9306	-0.34	-3.16	-6.54	0.22
11.6358	-0.34	-3.36	-6.74	0.02
12.3410	-0.34	-3.57	-6.95	-0.19
13.3988	-0.34	-3.87	-7.25	-0.49
14.2803	-0.34	-4.13	-7.51	-0.75
14.9855	-0.34	-4.33	-7.71	-0.95
15.6907	-0.34	-4.53	-7.91	-1.15
16.3959	-0.34	-4.74	-8.12	-1.36
16.9248	-0.34	-4.89	-8.27	-1.51
17.4537	-0.34	-5.04	-8.42	-1.66
18.3352	-0.34	-5.30	-8.68	-1.92
19.2167	-0.34	-5.55	-8.93	-2.17

Sidewall Pressures

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	-0.70	-5.98	-9.36	-2.60
10.9306	-0.70	-6.50	-9.88	-3.12
11.6358	-0.70	-6.92	-10.30	-3.54
12.3410	-0.70	-7.34	-10.72	-3.96
13.3988	-0.70	-7.97	-11.35	-4.59
14.2803	-0.70	-8.50	-11.88	-5.12
14.9855	-0.70	-8.92	-12.30	-5.54
15.6907	-0.70	-9.34	-12.72	-5.96
16.3959	-0.70	-9.76	-13.14	-6.38
16.9248	-0.70	-10.07	-13.45	-6.69
17.4537	-0.70	-10.38	-13.76	-7.01
18.3352	-0.70	-10.91	-14.29	-7.53
19.2167	-0.70	-11.43	-14.81	-8.05

Roof

qz (psf)	Cp	External Pressure	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	-0.5	-4.27	-7.65	-0.89
10.9306	-0.5	-4.65	-8.03	-1.27
11.6358	-0.5	-4.95	-8.32	-1.57
12.3410	-0.5	-5.24	-8.62	-1.87
13.3988	-0.5	-5.69	-9.07	-2.31
14.2803	-0.5	-6.07	-9.45	-2.69
14.9855	-0.5	-6.37	-9.75	-2.99
15.6907	-0.5	-6.67	-10.05	-3.29
16.3959	-0.5	-6.97	-10.35	-3.59
16.9248	-0.5	-7.19	-10.57	-3.81
17.4537	-0.5	-7.42	-10.80	-4.04
18.3352	-0.5	-7.79	-11.17	-4.41
19.2167	-0.5	-8.17	-11.55	-4.79

Trial 2

Surface Type	Surface Designation	Surface	Distance from Windward Edge	L/B or h/L	Cp	External Pressure @ q = 130' (psf)
Walls	W2	WW	-	All	0.80	15.021
	W3	LW	-	0.56	-0.50	-9.388
	W1	Side	-	All	-0.70	-13.143
Roof			0 to h	0.34	-0.90	-16.898
			h to 2h	0.34	-0.50	-9.388
			> 2h	0.34	-0.30	-5.633

Windward Pressures *(same as Trial 1)*

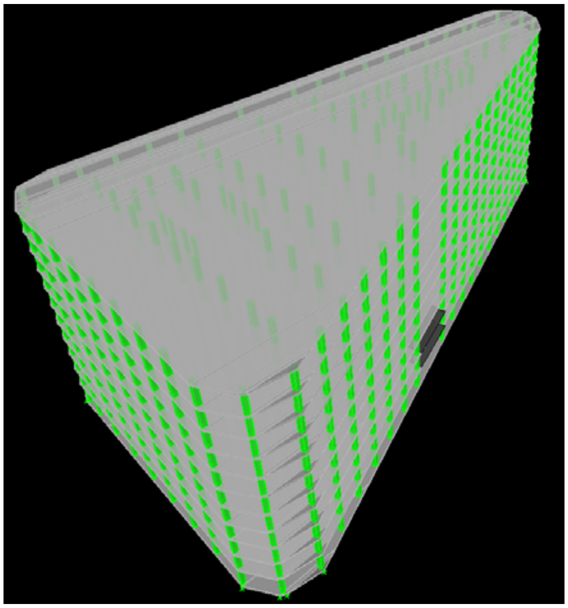
Leeward Pressures

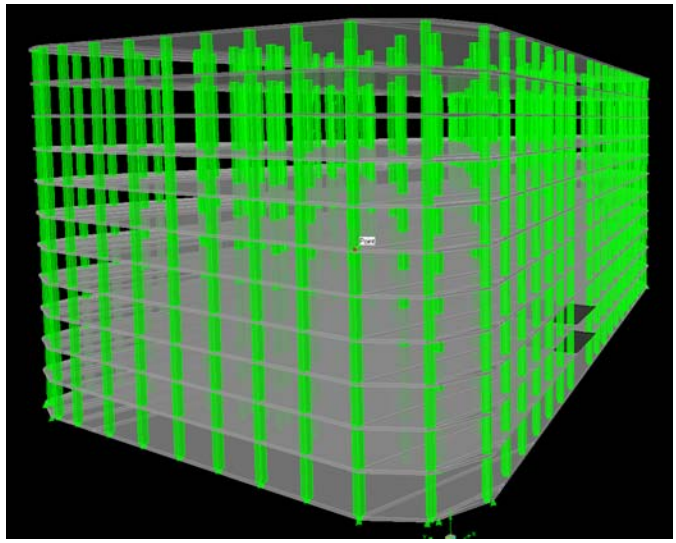
qz (psf)	Cp	External Pressure (psf)	Design Pressure (psf) (+GCpi)	(-GCpi)
10.0491	-0.5	-4.27	-7.65	-0.89
10.9306	-0.5	-4.65	-8.03	-1.27
11.6358	-0.5	-4.95	-8.32	-1.57
12.3410	-0.5	-5.24	-8.62	-1.87
13.3988	-0.5	-5.69	-9.07	-2.31
14.2803	-0.5	-6.07	-9.45	-2.69
14.9855	-0.5	-6.37	-9.75	-2.99
15.6907	-0.5	-6.67	-10.05	-3.29
16.3959	-0.5	-6.97	-10.35	-3.59
16.9248	-0.5	-7.19	-10.57	-3.81
17.4537	-0.5	-7.42	-10.80	-4.04
18.3352	-0.5	-7.79	-11.17	-4.41
19.2167	-0.5	-8.17	-11.55	-4.79

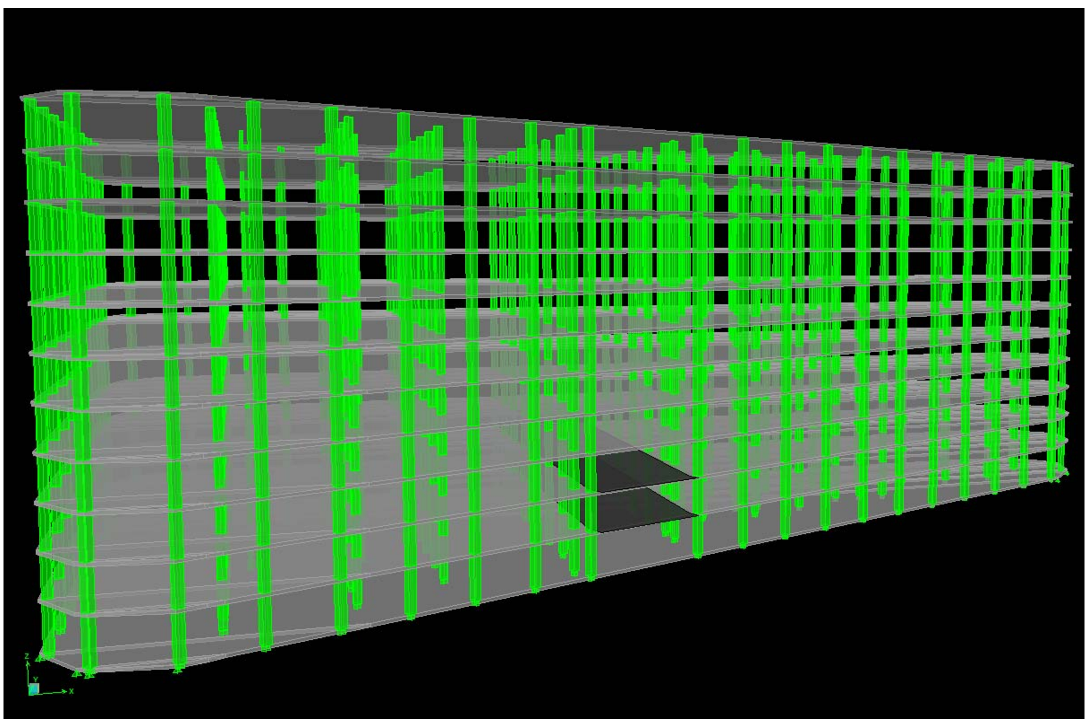
Sidewall Pressures *(same as Trial 1)*

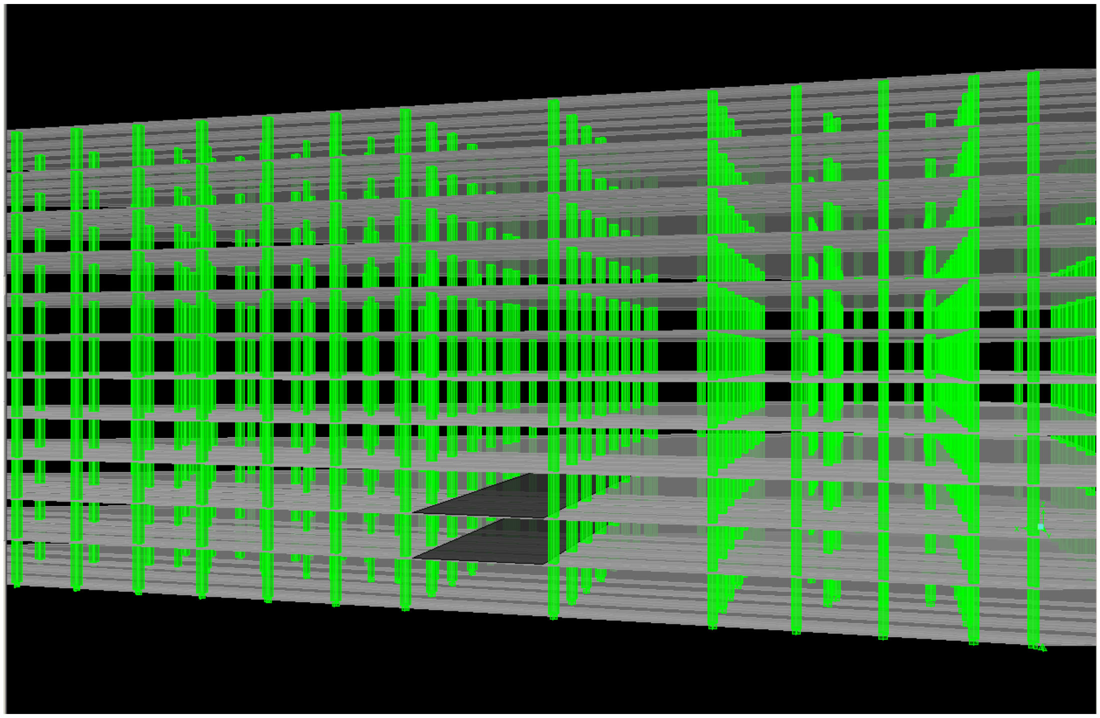
Roof *(same as Trial 1)*

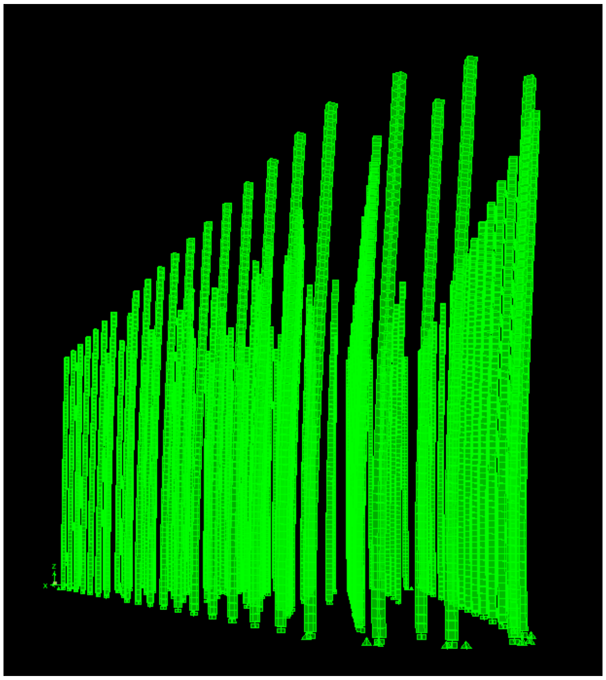
ETabs Renderings

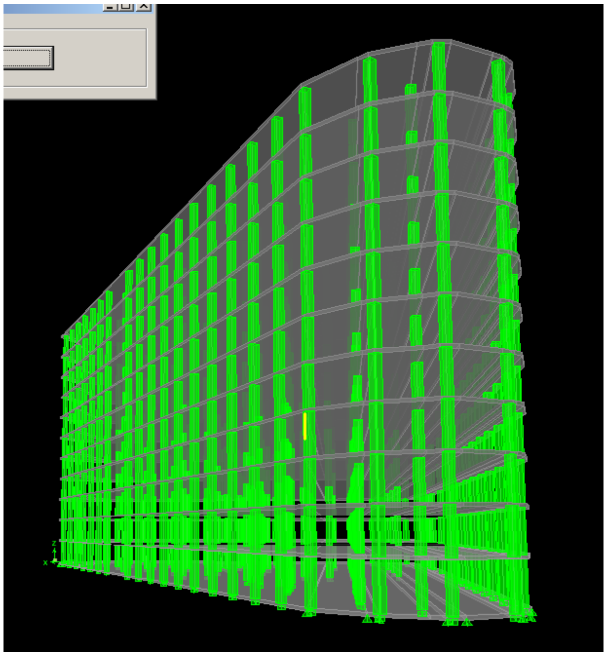


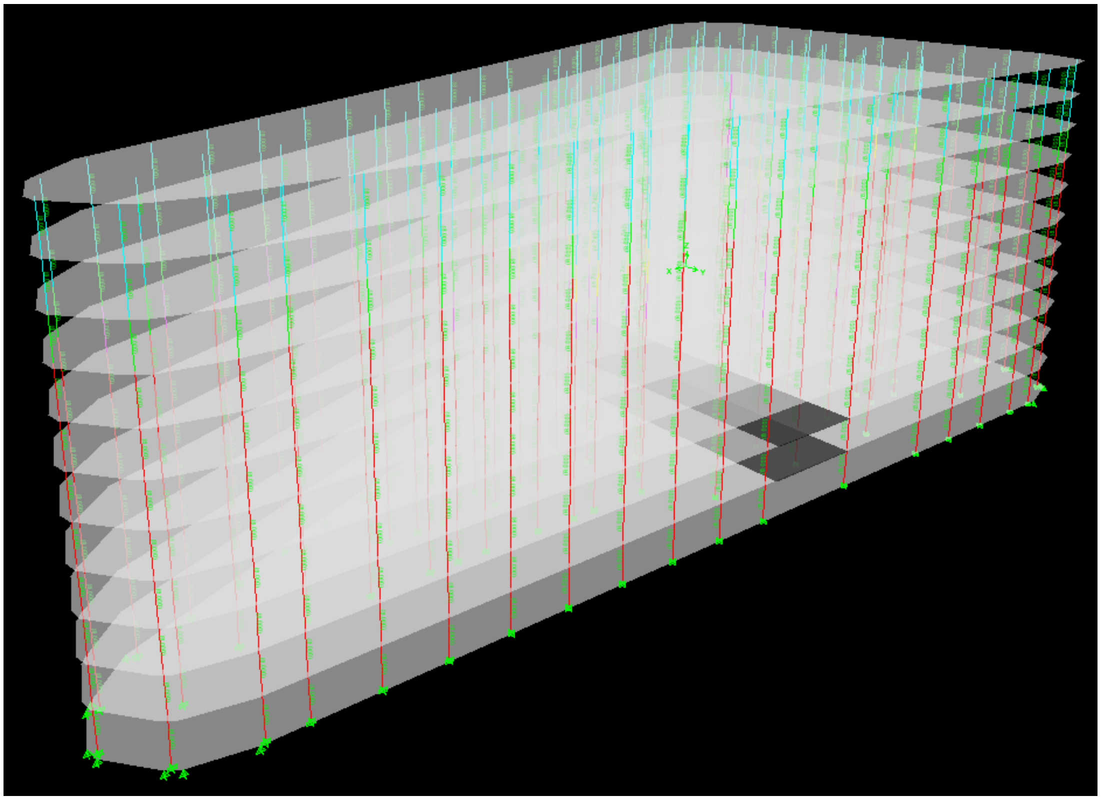


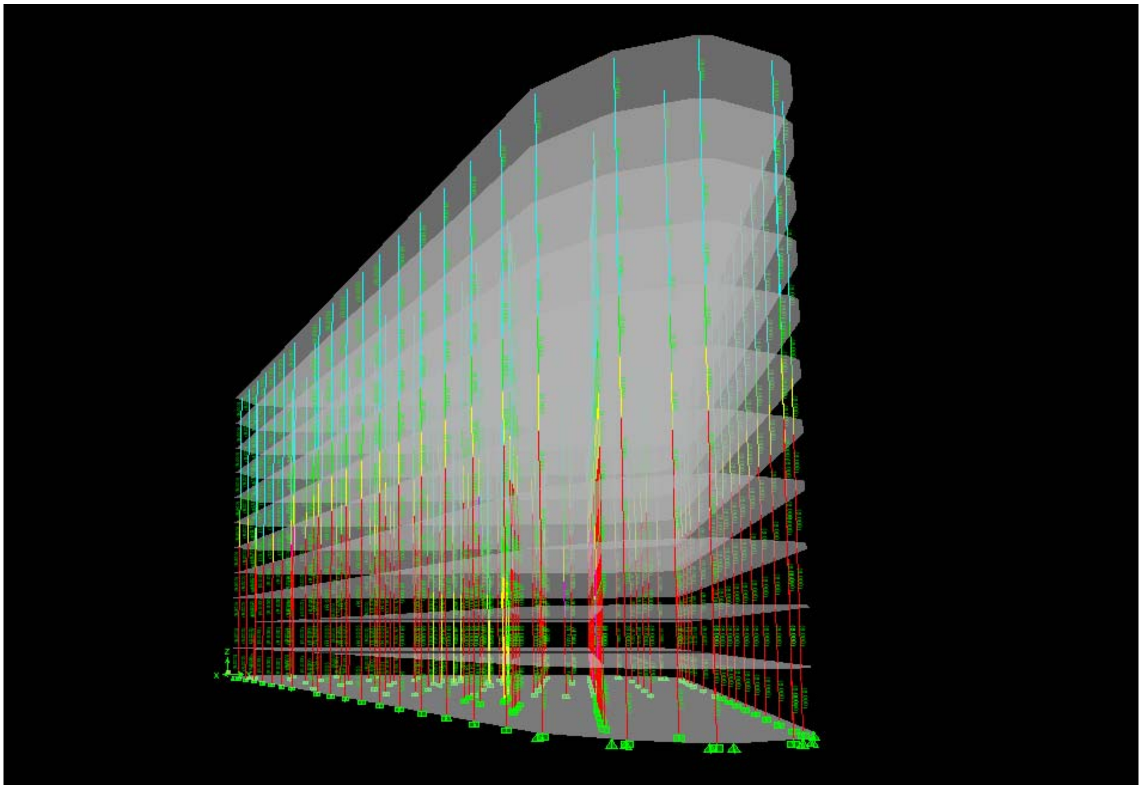












Story Drift Calculation Summary

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY1	Max Drift X	14D	459	370.019	210.904	12.83	0	
STORY1	Max Drift Y	14D	459	370.019	210.904	12.83		0
STORY1	Max Drift X	12D16L	459	370.019	210.904	12.83	0	
STORY1	Max Drift Y	12D16L	459	370.019	210.904	12.83		0
STORY1	Max Drift X	12D08W	452	29.337	10.637	12.83	0.001004	
STORY1	Max Drift Y	12D08W	456	384.696	191.023	12.83		0.001088
STORY1	Max Drift X	12D16WL05E	452	29.337	10.637	12.83	0.004903	
STORY1	Max Drift Y	12D16WL05E	456	384.696	191.023	12.83		0.002196
STORY1	Max Drift X	12D10E10L	452	29.337	10.637	12.83	0.005791	
STORY1	Max Drift Y	12D10E10L	456	384.696	191.023	12.83		0.00004
STORY1	Max Drift X	09D10E	452	29.337	10.637	12.83	0.005791	
STORY1	Max Drift Y	09D10E	456	384.696	191.023	12.83		0.00004
STORY1	Max Drift X	09D16W	452	29.337	10.637	12.83	0.002008	
STORY1	Max Drift Y	09D16W	456	384.696	191.023	12.83		0.002176
STORY2	Max Drift X	14D	484	176.178	214.626	24.5	0	
STORY2	Max Drift Y	14D	484	176.178	214.626	24.5		0
STORY2	Max Drift X	12D16L	484	176.178	214.626	24.5	0	
STORY2	Max Drift Y	12D16L	484	176.178	214.626	24.5		0
STORY2	Max Drift X	12D08W	452	29.337	10.637	24.5	0.002703	
STORY2	Max Drift Y	12D08W	456	384.696	191.023	24.5		0.002964
STORY2	Max Drift X	12D16WL05E	452	29.337	10.637	24.5	0.013377	
STORY2	Max Drift Y	12D16WL05E	456	384.696	191.023	24.5		0.00599
STORY2	Max Drift X	12D10E10L	452	29.337	10.637	24.5	0.015941	
STORY2	Max Drift Y	12D10E10L	456	384.696	191.023	24.5		0.000126
STORY2	Max Drift X	09D10E	452	29.337	10.637	24.5	0.015941	
STORY2	Max Drift Y	09D10E	456	384.696	191.023	24.5		0.000126
STORY2	Max Drift X	09D16W	452	29.337	10.637	24.5	0.005407	
STORY2	Max Drift Y	09D16W	456	384.696	191.023	24.5		0.005927
STORY3	Max Drift X	14D	459	370.019	210.904	36.17	0	
STORY3	Max Drift Y	14D	459	370.019	210.904	36.17		0
STORY3	Max Drift X	12D16L	459	370.019	210.904	36.17	0	
STORY3	Max Drift Y	12D16L	459	370.019	210.904	36.17		0
STORY3	Max Drift X	12D08W	452	29.337	10.637	36.17	0.004054	
STORY3	Max Drift Y	12D08W	456	384.696	191.023	36.17		0.004465
STORY3	Max Drift X	12D16WL05E	452	29.337	10.637	36.17	0.020357	
STORY3	Max Drift Y	12D16WL05E	456	384.696	191.023	36.17		0.00902
STORY3	Max Drift X	12D10E10L	452	29.337	10.637	36.17	0.024499	
STORY3	Max Drift Y	12D10E10L	456	384.696	191.023	36.17		0.00018
STORY3	Max Drift X	09D10E	452	29.337	10.637	36.17	0.024499	
STORY3	Max Drift Y	09D10E	456	384.696	191.023	36.17		0.00018
STORY3	Max Drift X	09D16W	452	29.337	10.637	36.17	0.008107	
STORY3	Max Drift Y	09D16W	456	384.696	191.023	36.17		0.00893
STORY4	Max Drift X	14D	459	370.019	210.904	47.84	0	
STORY4	Max Drift Y	14D	459	370.019	210.904	47.84		0
STORY4	Max Drift X	12D16L	459	370.019	210.904	47.84	0	
STORY4	Max Drift Y	12D16L	459	370.019	210.904	47.84		0
STORY4	Max Drift X	12D08W	452	29.337	10.637	47.84	0.005149	
STORY4	Max Drift Y	12D08W	456	384.696	191.023	47.84		0.005686
STORY4	Max Drift X	12D16WL05E	452	29.337	10.637	47.84	0.026228	
STORY4	Max Drift Y	12D16WL05E	456	384.696	191.023	47.84		0.011472
STORY4	Max Drift X	12D10E10L	452	29.337	10.637	47.84	0.031857	
STORY4	Max Drift Y	12D10E10L	456	384.696	191.023	47.84		0.000198
STORY4	Max Drift X	09D10E	452	29.337	10.637	47.84	0.031857	
STORY4	Max Drift Y	09D10E	456	384.696	191.023	47.84		0.000198

STORY4	Max Drift X	09D16W	452	29.337	10.637	47.84	0.010299	
STORY4	Max Drift Y	09D16W	456	384.696	191.023	47.84		0.011373
STORY5	Max Drift X	14D	459	370.019	210.904	59.51	0	
STORY5	Max Drift Y	14D	459	370.019	210.904	59.51		0
STORY5	Max Drift X	12D16L	459	370.019	210.904	59.51	0	
STORY5	Max Drift Y	12D16L	459	370.019	210.904	59.51		0
STORY5	Max Drift X	12D08W	452	29.337	10.637	59.51	0.005992	
STORY5	Max Drift Y	12D08W	456	384.696	191.023	59.51		0.00664
STORY5	Max Drift X	12D16WL05E	452	29.337	10.637	59.51	0.030908	
STORY5	Max Drift Y	12D16WL05E	456	384.696	191.023	59.51		0.013389
STORY5	Max Drift X	12D10E10L	452	29.337	10.637	59.51	0.037847	
STORY5	Max Drift Y	12D10E10L	456	384.696	191.023	59.51		0.000218
STORY5	Max Drift X	09D10E	452	29.337	10.637	59.51	0.037847	
STORY5	Max Drift Y	09D10E	456	384.696	191.023	59.51		0.000218
STORY5	Max Drift X	09D16W	452	29.337	10.637	59.51	0.011984	
STORY5	Max Drift Y	09D16W	456	384.696	191.023	59.51		0.01328
STORY6	Max Drift X	14D	459	370.019	210.904	71.18	0	
STORY6	Max Drift Y	14D	459	370.019	210.904	71.18		0
STORY6	Max Drift X	12D16L	459	370.019	210.904	71.18	0	
STORY6	Max Drift Y	12D16L	459	370.019	210.904	71.18		0
STORY6	Max Drift X	12D08W	452	29.337	10.637	71.18	0.00661	
STORY6	Max Drift Y	12D08W	456	384.696	191.023	71.18		0.007349
STORY6	Max Drift X	12D16WL05E	452	29.337	10.637	71.18	0.034479	
STORY6	Max Drift Y	12D16WL05E	456	384.696	191.023	71.18		0.014816
STORY6	Max Drift X	12D10E10L	452	29.337	10.637	71.18	0.042518	
STORY6	Max Drift Y	12D10E10L	456	384.696	191.023	71.18		0.000237
STORY6	Max Drift X	09D10E	452	29.337	10.637	71.18	0.042518	
STORY6	Max Drift Y	09D10E	456	384.696	191.023	71.18		0.000237
STORY6	Max Drift X	09D16W	452	29.337	10.637	71.18	0.013219	
STORY6	Max Drift Y	09D16W	456	384.696	191.023	71.18		0.014697
STORY7	Max Drift X	14D	459	370.019	210.904	82.85	0	
STORY7	Max Drift Y	14D	459	370.019	210.904	82.85		0
STORY7	Max Drift X	12D16L	459	370.019	210.904	82.85	0	
STORY7	Max Drift Y	12D16L	459	370.019	210.904	82.85		0
STORY7	Max Drift X	12D08W	452	29.337	10.637	82.85	0.007038	
STORY7	Max Drift Y	12D08W	456	384.696	191.023	82.85		0.007842
STORY7	Max Drift X	12D16WL05E	452	29.337	10.637	82.85	0.037082	
STORY7	Max Drift Y	12D16WL05E	456	384.696	191.023	82.85		0.015806
STORY7	Max Drift X	12D10E10L	452	29.337	10.637	82.85	0.046012	
STORY7	Max Drift Y	12D10E10L	456	384.696	191.023	82.85		0.000247
STORY7	Max Drift X	09D10E	452	29.337	10.637	82.85	0.046012	
STORY7	Max Drift Y	09D10E	456	384.696	191.023	82.85		0.000247
STORY7	Max Drift X	09D16W	452	29.337	10.637	82.85	0.014076	
STORY7	Max Drift Y	09D16W	456	384.696	191.023	82.85		0.015683
STORY8	Max Drift X	14D	459	370.019	210.904	94.52	0	
STORY8	Max Drift Y	14D	459	370.019	210.904	94.52		0
STORY8	Max Drift X	12D16L	459	370.019	210.904	94.52	0	
STORY8	Max Drift Y	12D16L	459	370.019	210.904	94.52		0
STORY8	Max Drift X	12D08W	452	29.337	10.637	94.52	0.007309	
STORY8	Max Drift Y	12D08W	456	384.696	191.023	94.52		0.008152
STORY8	Max Drift X	12D16WL05E	452	29.337	10.637	94.52	0.038837	
STORY8	Max Drift Y	12D16WL05E	456	384.696	191.023	94.52		0.016429
STORY8	Max Drift X	12D10E10L	452	29.337	10.637	94.52	0.048439	
STORY8	Max Drift Y	12D10E10L	456	384.696	191.023	94.52		0.00025
STORY8	Max Drift X	09D10E	452	29.337	10.637	94.52	0.048439	
STORY8	Max Drift Y	09D10E	456	384.696	191.023	94.52		0.00025
STORY8	Max Drift X	09D16W	452	29.337	10.637	94.52	0.014618	

STORY8	Max Drift Y	09D16W	456	384.696	191.023	94.52		0.016304
STORY9	Max Drift X	14D	459	370.019	210.904	106.19	0	
STORY9	Max Drift Y	14D	459	370.019	210.904	106.19		0
STORY9	Max Drift X	12D16L	459	370.019	210.904	106.19	0	
STORY9	Max Drift Y	12D16L	459	370.019	210.904	106.19		0
STORY9	Max Drift X	12D08W	452	29.337	10.637	106.19	0.007456	
STORY9	Max Drift Y	12D08W	456	384.696	191.023	106.19		0.008319
STORY9	Max Drift X	12D16WL05E	452	29.337	10.637	106.19	0.039883	
STORY9	Max Drift Y	12D16WL05E	456	384.696	191.023	106.19		0.016764
STORY9	Max Drift X	12D10E10L	452	29.337	10.637	106.19	0.049941	
STORY9	Max Drift Y	12D10E10L	456	384.696	191.023	106.19		0.000252
STORY9	Max Drift X	09D10E	452	29.337	10.637	106.19	0.049941	
STORY9	Max Drift Y	09D10E	456	384.696	191.023	106.19		0.000252
STORY9	Max Drift X	09D16W	452	29.337	10.637	106.19	0.014912	
STORY9	Max Drift Y	09D16W	456	384.696	191.023	106.19		0.016638
STORY10	Max Drift X	14D	459	370.019	210.904	117.86	0	
STORY10	Max Drift Y	14D	459	370.019	210.904	117.86		0
STORY10	Max Drift X	12D16L	459	370.019	210.904	117.86	0	
STORY10	Max Drift Y	12D16L	459	370.019	210.904	117.86		0
STORY10	Max Drift X	12D08W	452	29.337	10.637	117.86	0.007518	
STORY10	Max Drift Y	12D08W	456	384.696	191.023	117.86		0.008385
STORY10	Max Drift X	12D16WL05E	452	29.337	10.637	117.86	0.040394	
STORY10	Max Drift Y	12D16WL05E	456	384.696	191.023	117.86		0.016897
STORY10	Max Drift X	12D10E10L	452	29.337	10.637	117.86	0.050718	
STORY10	Max Drift Y	12D10E10L	456	384.696	191.023	117.86		0.000254
STORY10	Max Drift X	09D10E	452	29.337	10.637	117.86	0.050718	
STORY10	Max Drift Y	09D10E	456	384.696	191.023	117.86		0.000254
STORY10	Max Drift X	09D16W	452	29.337	10.637	117.86	0.015035	
STORY10	Max Drift Y	09D16W	456	384.696	191.023	117.86		0.01677
STORY11	Max Drift X	14D	459	370.019	210.904	130	0	
STORY11	Max Drift Y	14D	459	370.019	210.904	130		0
STORY11	Max Drift X	12D16L	459	370.019	210.904	130	0	
STORY11	Max Drift Y	12D16L	459	370.019	210.904	130		0
STORY11	Max Drift X	12D08W	452	29.337	10.637	130	0.007532	
STORY11	Max Drift Y	12D08W	456	384.696	191.023	130		0.008398
STORY11	Max Drift X	12D16WL05E	452	29.337	10.637	130	0.040568	
STORY11	Max Drift Y	12D16WL05E	456	384.696	191.023	130		0.016923
STORY11	Max Drift X	12D10E10L	452	29.337	10.637	130	0.051007	
STORY11	Max Drift Y	12D10E10L	456	384.696	191.023	130		0.000254
STORY11	Max Drift X	09D10E	452	29.337	10.637	130	0.051007	
STORY11	Max Drift Y	09D10E	456	384.696	191.023	130		0.000254
STORY11	Max Drift X	09D16W	452	29.337	10.637	130	0.015064	
STORY11	Max Drift Y	09D16W	456	384.696	191.023	130		0.016796

Appendix – Alternative Systems

TECH ASSIGNMENT #2 | 901 NYA | TIMOTHY H PARK

GIRDER DESIGN FOR HOLLOW CORE

- CONCRETE

USING $\boxed{4HC6+2}$ (DL = 74 PSF) ASSUME UNFACTORED

LOADS ON GIRDERS DL $20+74=94$, LL $~~1~~80 = 80$ $\Big]$ $1.2(94)+1.6(80) = 240.8$ PSF

$(240.8)(20') = 4.816$ KLF

$\frac{(4.816)(40)^2}{8} = 963.2'^{K} = 11558''^{K}$

$\rho = 0.6\,\rho_{max} = 6.6\,(0.0206) = 0.01236$

$f'_c = 5000$ PSI, $f_y = 60$ KSI $\Big]$

ρ	R
0.0120	659
0.0125	684

— $\rho = 0.0124$, $R = 679$

$M_u = \phi R b d^2 \quad bd^2 = \frac{M_u}{\phi R} = \frac{11558}{0.9(679)} = 18913 \text{ IN}^3$

TRY $27'' \times 27''$: $I = \frac{27 \times 27^3}{12} = 44287 \text{ IN}^4$

$\Delta = \frac{5 w l^4}{384 EI} = \frac{(5)(963)(40 \times 12)^4}{384(3.6E6)(44287)} = 4.17''$

$\Delta_{max} = \frac{l}{240} = 2'' \quad 2 < 4.17 \; \times$

$20'' \times 32''$: $I = 54613 \text{ IN}^4$, $\Delta = 3.39''$

$15'' \times 36''$ $I = 58320 \text{ IN}^4$, $\Delta = 3.17''$

$11'' \times 42''$ $I = 67914 \text{ IN}^4$, $\Delta = 2.72''$

W33×90
$\phi M_n = 1060'^{K}$

CONCRETE GIRDER NOT GOOD SOL FOR 40' SPAN

@ 28' : $M_u = 472'^{K} = 5664''^{K}$ $\quad bd^2 = 9268$

12×28 : $I = 16207 \text{ IN}^4$, $\Delta = 2.74''$

8" × 36" WORKS @ 28'-0" SPAN

$8'' \times 36''$: $I = 31105 \text{ IN}^4$, $\Delta = 1.4''$

TECH ASSIGNMENT #2 | 901 NYA | TIMOTHY H PARK

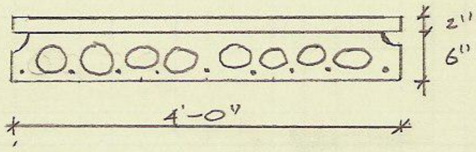
HOLLOW CORE SLAB DESIGN

FROM PCI DESIGN HANDBOOK 6TH EDITION

SPAN = 20'

SERVICEABLE LOADS: LL 80 PSF, DL 20 PSF $\Big]$ 1.2(20)+1.6(80) = 152 PSF

1)

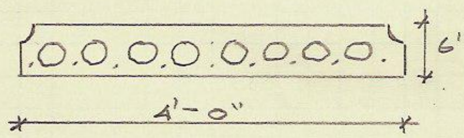


$\boxed{4HG6+2}$

w/ 76-S STRANDS

HOLLOW CORE PROP: MAX LOAD = 163 PSF
DL = 74 PSF
f'_c = 5000 PSI
STRANDS = 7 - 3/8'' STRANDS
Δ = 0.1''

2)



$\boxed{4HG6}$

w/ 96-S STRAND

HOLLOW CORE PROP: MAX LOAD = 157 PSF
DL = 49 PSF
f'_c = 5000 PSI
STRANDS = 9 - 3/8'' STRANDS
Δ = 0.5''

FROM RS MEANS

8'' HC SLAB : \$9.60/SF × (48070)(11) = \$5,076,192.00

6'' HC SLAB : \$9.50/SF × (48070)(11) = \$5,023,315.00

NOTE: $\Delta_{MAX} = \frac{l}{240} = \frac{20 \times 12}{240} = 1''$ BOTH SATISFY Δ REQUIREMENT

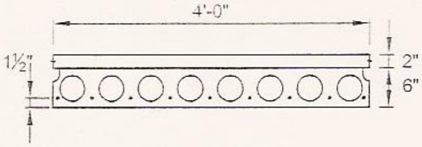
76-S
- S = straight
- Diameter of strand in 16ths
- No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 6"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

	Untopped	Topped
A =	187 in.^{2}	283 in.^{2}
I =	763 in.^{4}	1,640 in.^{4}
y_b =	3.00 in.	4.14 in.
y_t =	3.00 in.	3.86 in.
S_b =	254 in.^{3}	396 in.^{3}
S_t =	254 in.^{3}	425 in.^{3}
wt =	195 plf	295 plf
DL =	49 psf	74 psf
V/S =	1.73 in.	

4HC6

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **-2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

2500	40′ span, 12″ x 52″		20	3.600		4,175	146	92.50	4,413.50	4,925
2550	18″ x 52″		16	4.500		4,950	183	116	5,249	5,900
2600	24″ x 52″		12	6		5,425	243	154	5,822	6,575

03 41 05.15 Precast Columns

0010	**PRECAST COLUMNS** R034105-30									
0020	Rectangular to 12′ high, small columns	C-11	120	.600	L.F.	47	24.50	15.45	86.95	111
0050	Large columns		96	.750		82	30.50	19.30	131.80	165
0300	24′ high, small columns		192	.375		47	15.20	9.65	71.85	88.50
0350	Large columns		144	.500		82	20.50	12.85	115.35	140
0700	24′ high, 1 haunch, 12″ x 12″		32	2.250	Ea.	1,125	91.50	58	1,274.50	1,475
0800	20″ x 20″		28	2.571	"	1,975	104	66	2,145	2,425

03 41 05.25 Precast Joists

0010	**PRECAST JOISTS** R034105-30									
0015	40 psf L.L., 6″ deep for 12′ spans	C-12	600	.080	L.F.	7.80	2.90	1.21	11.91	14.40
0050	8″ deep for 16′ spans		575	.083		13	3.02	1.26	17.28	20.50
0100	10″ deep for 20′ spans		550	.087		23	3.16	1.32	27.48	31.50
0150	12″ deep for 24′ spans		525	.091		31	3.31	1.38	35.69	41

03 41 13 – Precast Concrete Hollow Core Planks

03 41 13.50 Precast Slab Planks

0010	**PRECAST SLAB PLANKS** R034105-30									
0020	Prestressed roof/floor members, grouted, solid, 4″ thick	C-11	2400	.030	S.F.	4.89	1.22	.77	6.88	8.40
0050	6″ thick		2800	.026		6.30	1.04	.66	8	9.50
0100	Hollow, 8″ thick		3200	.023		6.70	.91	.58	8.19	9.60
0150	10″ thick		3600	.020		7	.81	.51	8.32	9.70
0200	12″ thick		4000	.018		7.95	.73	.46	9.14	10.55

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SOLID ONE-WAY SLABS—INTERIOR SPAN — Top Steel for $-M_u$

$f_c' = 3{,}000$ psi — Grade 60 Bars — $\rho = 0.0050$

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Spacing (in.)	12	11	10	9	12	11	10	10	9	12	11	10	10
Bottom Bars	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	10	9	7	12	11	10	10	9	8	12	11	10	10
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel (in.²/ft) Top Interior	.200	.218	.240	.267	.310	.338	.372	.372	.413	.440	.480	.528	.528
Bottom	.132	.147	.189	.200	.218	.240	.240	.267	.300	.310	.338	.372	.372
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6'-0"	703	923											
6'-6"	689	775											
7'-0"	498	657	907										
7'-6"	425	562	778	986									
8'-0"	365	485	673	886									
8'-6"	315	420	586	747	935								
9'-0"	273	367	513	686	822								
9'-6"	238	321	452	579	727	894	960						
10'-0"	208	282	399	513	646	795	872						
10'-6"	181	243	317	410	539	661	779	882					
11'-0"	159	214	281	365	482	592	699	792	964				
11'-6"	139	189	249	326	432	532	629	713	870	994			
12'-0"	122	167	222	291	388	479	568	644	787	901			
12'-6"	107	148	197	261	349	433	514	583	715	819	967		
13'-0"	94	131	176	234	315	392	465	529	650	746	862		
13'-6"	82	116	157	210	285	355	423	481	593	681	806	959	
14'-0"	71	102	139	188	257	322	384	438	541	623	730	880	939
14'-6"	61	90	124	169	233	293	350	400	495	570	678	809	863
15'-0"	53	79	110	151	210	266	319	365	453	523	623	745	795
15'-6"	45	69	97	136	190	242	291	333	416	480	573	688	733
16'-0"		60	86	121	172	220	265	305	381	442	528	635	678
16'-6"		51	76	108	156	200	242	279	350	406	487	587	627
17'-0"		44	66	96	140	182	221	255	322	374	450	543	580
17'-6"			57	86	127	165	201	233	296	345	416	503	538
18'-0"			49	76	114	150	184	213	272	318	384	467	499
18'-6"			42	66	102	136	167	195	250	293	355	433	463
19'-0"				58	91	123	152	178	230	270	329	402	429
19'-6"				50	81	111	138	162	211	249	304	373	399
20'-0"				43	72	100	125	147	194	229	281	346	370

Note: See Fig. 7-1 for reinforcing bar details.

SOLID ONE-WAY SLABS—END SPAN — Top Steel for $-M_u$

$f'_c = 3{,}000$ psi — Grade 60 Bars — $\rho = 0.0050$

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Spacing (in.)	12	12	11	9	12	11	10	10	9	12	11	10	10
Bottom Bars	#4	#4	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6
Spacing (in.)	12	11	10	8	8	12	11	11	10	9	12	11	11
Top Bars Free End	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	12	12	12	12	12	12	12	12	12	12
T-S Bars	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5
Spacing (in.)	15	13	12	11	18	17	15	14	13	13	12	18	17
Areas of Steel ($in.^2/ft$) Top Interior	.200	.200	.218	.267	.310	.338	.372	.377	.413	.440	.480	.528	.528
Bottom	.200	.218	.240	.300	.300	.310	.338	.338	.372	.413	.440	.480	.480
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	**FACTORED USABLE SUPERIMPOSED LOAD (psf)**												
6'-0"	700	906											
6'-6"	586	761	967										
7'-0"	496	645	821										
7'-6"	423	552	704	988									
8'-0"	363	475	608	856	986								
8'-6"	314	412	528	747	861	975							
9'-0"	272	359	462	656	757	858							
9'-6"	237	314	405	579	669	759	916						
10'-0"	207	276	357	513	593	674	814	890					
10'-6"	158	191	248	364	481	591	722	790	957				
11'-0"	138	167	218	323	429	528	647	708	859	987			
11'-6"	120	146	192	287	383	473	582	636	774	890			
12'-0"	105	127	169	256	343	426	524	574	700	806	952		
12'-6"	91	111	149	228	308	383	473	518	634	731	865		
13'-0"	79	97	131	204	277	346	428	469	575	664	787	937	999
13'-6"	68	84	115	182	249	312	388	426	523	605	719	857	914
14'-0"	58	73	101	162	224	282	352	388	477	552	657	785	837
14'-6"	49	62	88	145	202	256	320	351	435	505	602	721	769
15'-0"	42	53	76	129	182	231	291	320	397	462	552	662	707
15'-6"		45	66	115	163	209	264	291	363	423	507	610	651
16'-0"			56	102	147	190	241	265	332	388	466	562	600
16'-6"			48	90	132	171	219	241	304	356	429	519	554
17'-0"			40	79	118	155	199	220	278	327	395	479	511
17'-6"				69	105	140	181	200	255	300	363	442	473
18'-0"				60	94	126	164	182	233	275	335	409	437
18'-6"				51	83	113	149	165	213	253	309	378	405
19'-0"				44	73	101	135	149	195	232	284	350	374
19'-6"					64	90	122	135	178	213	262	324	347
20'-0"					58	80	109	122	162	195	241	300	321

Note: See Fig. 7-1 for reinforcing bar details.

8-20

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS	30" Forms + 5" Rib @ 35" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)	f'_c = 4,000 psi f_y = 60,000 psi

8" Deep Rib + 3.0" Top Slab = 11.0" Total Depth

TOP BARS	Size @	# 4 12	# 4 12	# 4 11	# 4 9	# 5 11	End Span Defl. Coeff. (3)	# 4 12	# 4 12	# 4 10.5	# 4 8		Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6		# 3 # 3	# 3 # 4	# 4 # 4	# 4 # 5		
Steel (psf)		.50	.60	.72	.89	1.09		.56	.63	.78	1.00		
CLEAR SPAN		END SPAN						INTERIOR SPAN					
14'-0"		184 0	258 0	274* 346	285* 436	298* 464*	.450	194 0	302 0	312* 410	322* 538		.277
15'-0"		150 0	215 0	244* 292	253* 370	263* 428*	.593	159 0	253 0	280* 347	289* 459		.365
16'-0"		123 0	180 0	219* 247	226* 316	235* 382*	.767	131 0	213 0	253* 296	261* 394		.472
17'-0"		100 0	151 0	197* 210	203* 271	211* 339	.978	107 0	180 0	230* 254	237* 340		.602
18'-0"		81 0	126 0	178* 179	184* 233	190* 295	1.229	87 0	152 0	210* 218	216* 295		.756
19'-0"		65 0	105 0	153 0	167* 202	172* 257	1.525	71 0	129 0	188 0	198* 257		.939
20'-0"		51 0	88 0	131 0	152* 175	157* 224	1.873	56 0	109 0	162 0	181* 225		1.153
21'-0"			72 0	112 0	139* 151	143* 196	2.276	44 0	92 0	140 0	167* 197		1.401
22'-0"			59 0	95 0	128* 131	131* 172	2.742		77 0	121 0	154* 173		1.687
23'-0"			48 0	80 0	113 0	120* 151	3.276		64 0	104 0	143* 151		2.016
24'-0"				68 0	98 0	110* 132	3.884		52 0	89 0	132* 133		2.390
25'-0"				56 0	84 0	102* 116	4.572		42 0	76 0	116 0		2.814
26'-0"				46 0	72 0	94* 102	5.349			65 0	102 0		3.292
27'-0"					61 0	86* 89	6.221			55 0	89 0		3.828

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .36 CF/SF) (4)

NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.58	.58	.64	.78	.99		.58	.58	.67	.88		
STEEL % (UNIFORM)	1.03	1.03	1.12	1.37	1.75		1.03	1.03	1.18	1.54		
(TAPERED)	.55	.55	.60	.74	.94		.55	.55	.63	.83		
EFF. DEPTH, IN.	9.8	9.8	9.8	9.8	9.7		9.8	9.8	9.8	9.8		
−ICR/IGR	.208	.208	.222	.256	.298		.208	.208	.230	.278		
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.31	.40	.51	.62	.75		.22	.31	.40	.51		
STEEL %	.09	.12	.15	.18	.22		.06	.09	.12	.15		
EFF. DEPTH, IN.	9.8	9.8	9.7	9.7	9.6		9.8	9.8	9.8	9.7		
+ICR/IGR	.164	.207	.254	.303	.353		.121	.164	.207	.254		

CONCRETE REINFORCING STEEL INSTITUTE

Appendix – Gravity System Alternative

Gravity Column Design TakeOff

RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC

04/09/07 23:21:26
Steel Code: ASD 9th Ed.

Steel Grade: 50

I section

Size	#	Length (ft)	Weight (lbs)
W10X33	19	224.1	7404
W10X39	7	81.7	3197
W12X40	25	296.0	11784
W14X43	51	607.4	26042
W12X45	4	47.1	2102
W10X45	4	46.7	2113
W14X48	3	36.4	1747
W10X49	7	81.7	4003
W12X50	6	70.0	3479
W12X53	7	81.7	4336
W14X53	11	128.4	6814
W10X54	3	35.0	1882
W12X58	6	70.0	4050
W10X60	5	60.7	3634
W14X61	38	446.3	27183
W12X65	9	105.0	6826
W10X68	3	35.0	2383
W14X68	22	256.7	17473
W12X72	10	116.7	8379
W14X74	10	118.1	8761
W10X77	3	36.2	2782
W12X79	7	81.7	6449
W14X82	25	292.2	23865
W12X87	8	94.5	8234
W10X88	2	24.5	2159
W14X90	46	536.8	48407
W12X96	7	82.9	7950
W14X99	28	327.2	32403
W10X100	1	11.7	1167
W12X106	9	106.2	11274
W14X109	31	361.8	39393
W10X112	1	12.8	1437
W12X120	8	98.0	11773
W14X120	24	280.1	33643
W14X132	32	374.6	49458
W12X136	3	37.3	5069
W14X145	36	430.6	62564
W12X152	1	12.8	1952
W14X159	25	312.7	49689
W14X176	12	141.2	24889

Gravity Column Design TakeOff

RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC

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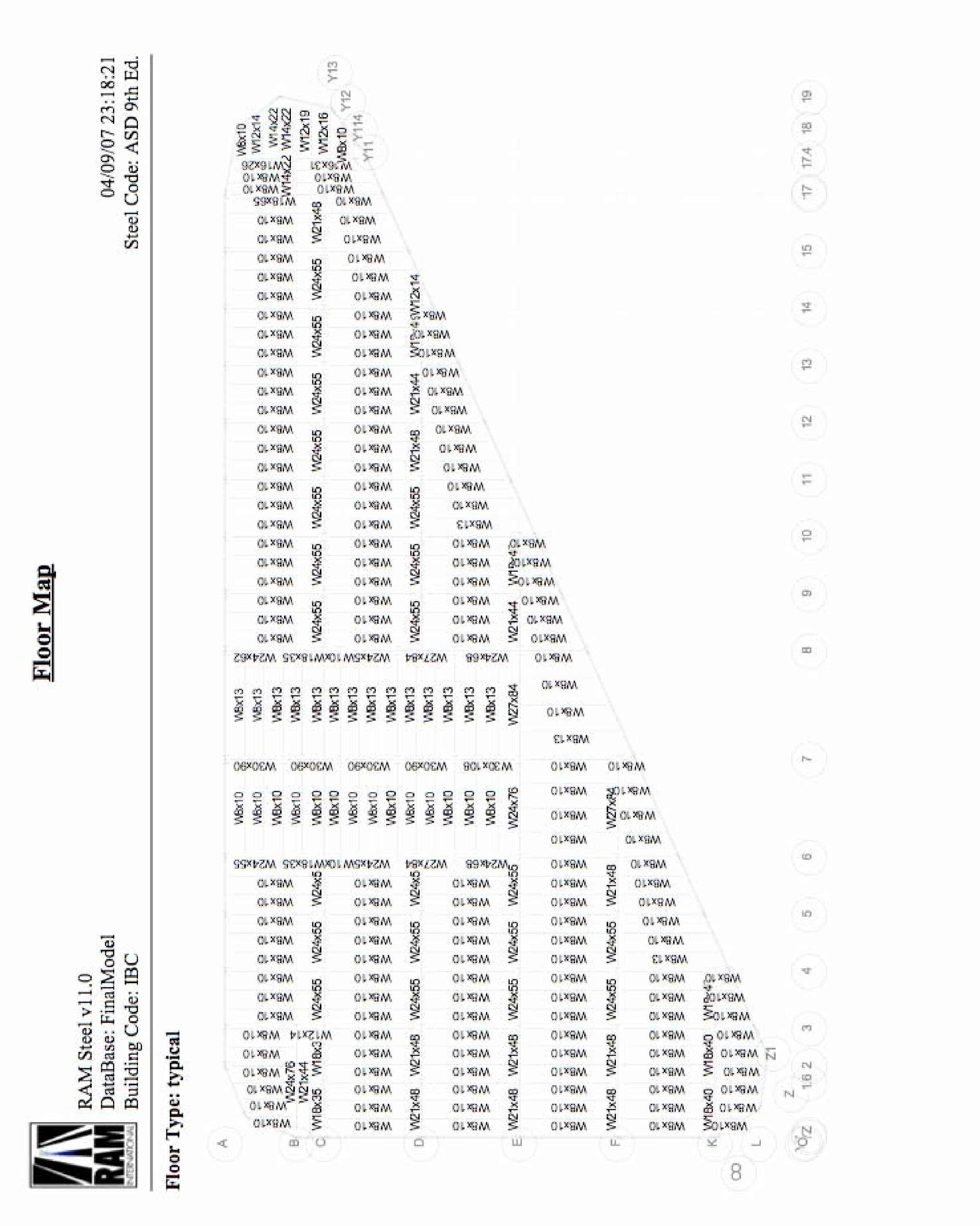
04/09/07 23:21:26
Steel Code: ASD 9th Ed.

W14X193	9	105.0	20300
W14X211	7	84.0	17725
W14X233	8	96.8	22575
W14X257	6	71.2	18312
W14X283	5	58.3	16539
W14X311	5	58.4	18148
W14X342	5	63.0	21653
W14X370	1	12.8	4760
	605		718158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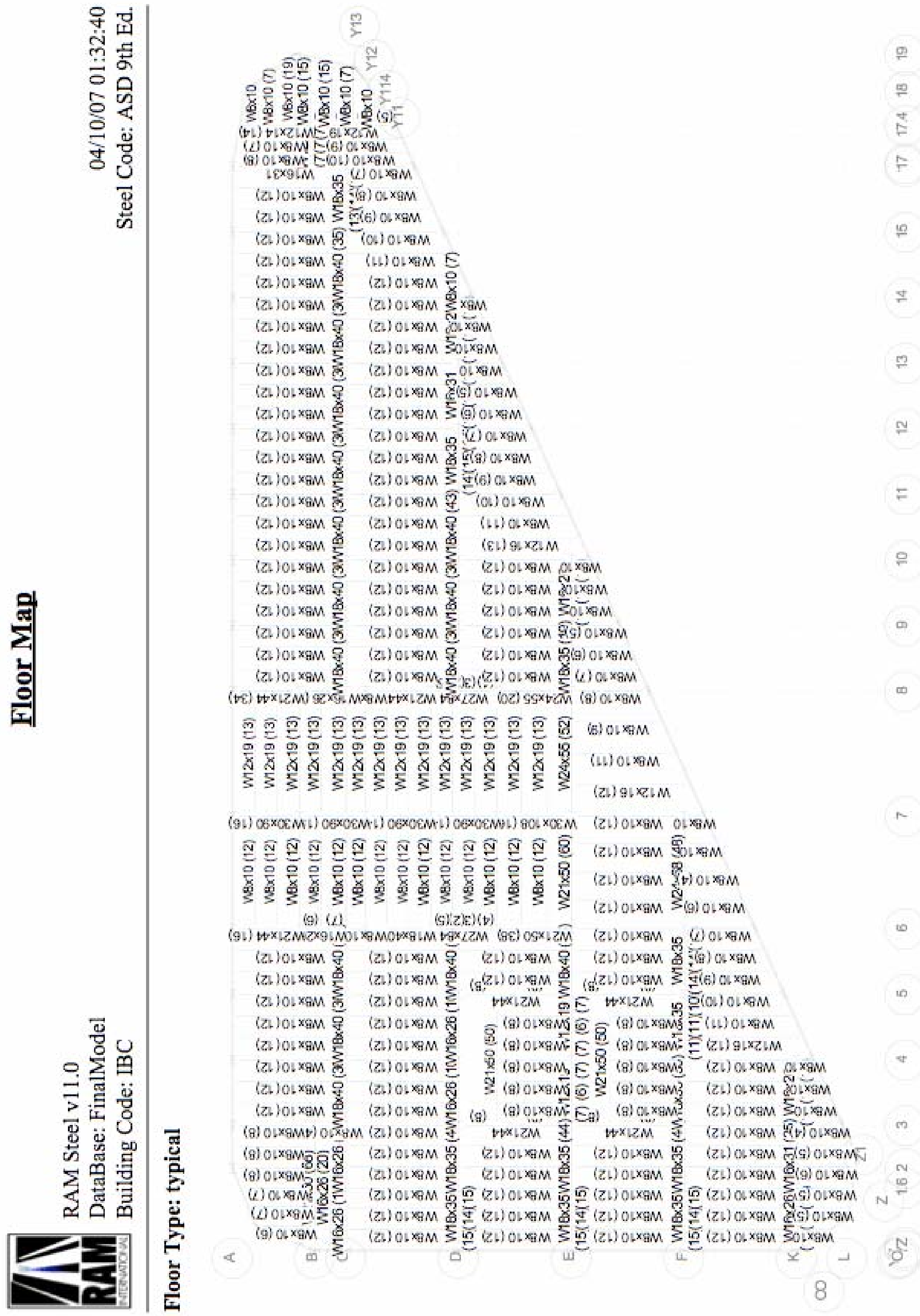
04/09/07 23:18:21
Steel Code: ASD 9th Ed.
RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC
Floor Type: typical


Floor Map
RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC
04/10/07 01:32:40
Steel Code: ASD 9th Ed.
Floor Type: typical

Gravity Beam Design Takeoff

RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC

04/10/07 02:02:20
Steel Code: ASD 9th Ed.

STEEL BEAM DESIGN TAKEOFF:

Floor Type: typical
Story Levels 1 to 11
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	186	5520.86	55607
W12X14	1	22.38	317
W12X16	4	127.78	2048
W12X19	17	629.13	11924
W16X26	11	234.67	6133
W16X31	3	77.08	2395
W18X35	12	245.75	8613
W18X40	16	336.00	13491
W21X44	7	207.50	9179
W21X50	5	183.83	9196
W24X55	2	63.92	3545
W24X68	1	36.00	2462
W27X84	2	42.00	3544
W30X90	4	84.25	7568
W30X108	1	23.92	2580
	-----		-------------
	272		**138603**

Total Number of Studs = **4193**

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	2046	60729.51	611681
W12X14	11	246.16	3484
W12X16	44	1405.62	22528
W12X19	187	6920.40	131166
W16X26	121	2581.35	67459
W16X31	33	847.92	26343
W18X35	132	2703.26	94746
W18X40	176	3696.00	148405
W21X44	77	2282.50	100969
W21X50	55	2022.17	101151
W24X55	22	703.09	38997
W24X68	11	396.00	27085

Gravity Beam Design Takeoff

RAM Steel v11.0
DataBase: FinalModel
Building Code: IBC

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04/10/07 02:02:20
Steel Code: ASD 9th Ed.

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W27X84	22	462.00	38988
W30X90	44	926.75	83253
W30X108	11	263.09	28379
	-----		-------------
	2992		**1524632**

Total Number of Studs = **46123**

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY11	Max Drift X	09D10E	368	102.917	-49.25	130	0.013489	
STORY10	Max Drift X	09D10E	399	170.917	-136.348	117.86	0.00703	
STORY9	Max Drift X	09D10E	44	178.965	-132.781	106.19	0.006064	
STORY8	Max Drift X	09D10E	44	178.965	-132.781	94.52	0.005021	
STORY7	Max Drift X	09D10E	44	178.965	-132.781	82.85	0.003895	
STORY1	Max Drift X	09D10E	162	102.917	-42.25	12.83	0.003701	
STORY6	Max Drift X	09D10E	44	178.965	-132.781	71.18	0.002816	
STORY2	Max Drift X	09D10E	123	0	-108.167	24.5	0.002057	
STORY5	Max Drift X	09D10E	44	178.965	-132.781	59.51	0.001778	
STORY3	Max Drift X	09D10E	132	20	-108.167	36.17	0.00164	
STORY4	Max Drift X	09D10E	137	102.917	-108.167	47.84	0.001501	
STORY11	Max Drift Y	09D10E	44	178.965	-132.781	130		0.015578
STORY10	Max Drift Y	09D10E	8	178.917	0	117.86		0.015741
STORY9	Max Drift Y	09D10E	44	178.965	-132.781	106.19		0.01462
STORY8	Max Drift Y	09D10E	173	178.917	-108.167	94.52		0.012811
STORY7	Max Drift Y	09D10E	173	178.917	-108.167	82.85		0.011045
STORY6	Max Drift Y	09D10E	173	178.917	-108.167	71.18		0.009234
STORY5	Max Drift Y	09D10E	173	178.917	-108.167	59.51		0.007392
STORY4	Max Drift Y	09D10E	173	178.917	-108.167	47.84		0.005527
STORY3	Max Drift Y	09D10E	44	178.965	-132.781	36.17		0.003652
STORY2	Max Drift Y	09D10E	334	74.917	-179.104	24.5		0.002161
STORY1	Max Drift Y	09D10E	139	60.917	-108.167	12.83		0.003228
STORY11	Max Drift X	09D16W	368	102.917	-49.25	130	0.013948	
STORY10	Max Drift X	09D16W	399	170.917	-136.348	117.86	0.007439	
STORY9	Max Drift X	09D16W	44	178.965	-132.781	106.19	0.00657	
STORY8	Max Drift X	09D16W	44	178.965	-132.781	94.52	0.005628	
STORY7	Max Drift X	09D16W	44	178.965	-132.781	82.85	0.004584	
STORY6	Max Drift X	09D16W	44	178.965	-132.781	71.18	0.003563	
STORY5	Max Drift X	09D16W	44	178.965	-132.781	59.51	0.002558	
STORY1	Max Drift X	09D16W	44	178.965	-132.781	12.83	0.002553	
STORY4	Max Drift X	09D16W	4	81.917	0	47.84	0.001656	
STORY3	Max Drift X	09D16W	5	102.917	0	36.17	0.001064	
STORY2	Max Drift X	09D16W	49	102.959	-166.493	24.5	0.001023	
STORY11	Max Drift Y	09D16W	44	178.965	-132.781	130		0.015595
STORY10	Max Drift Y	09D16W	8	178.917	0	117.86		0.015784
STORY9	Max Drift Y	09D16W	44	178.965	-132.781	106.19		0.014713
STORY8	Max Drift Y	09D16W	171	178.917	-84.25	94.52		0.012962
STORY7	Max Drift Y	09D16W	171	178.917	-84.25	82.85		0.011253
STORY6	Max Drift Y	09D16W	173	178.917	-108.167	71.18		0.009497
STORY5	Max Drift Y	09D16W	173	178.917	-108.167	59.51		0.007704
STORY4	Max Drift Y	09D16W	173	178.917	-108.167	47.84		0.005881
STORY3	Max Drift Y	09D16W	173	178.917	-108.167	36.17		0.004024
STORY2	Max Drift Y	09D16W	11	304.917	0	24.5		0.003336
STORY1	Max Drift Y	09D16W	182	304.917	-72.167	12.83		0.004917
STORY11	Max Drift X	12D10E10L	44	178.965	-132.781	130	0.166986	
STORY10	Max Drift X	12D10E10L	399	170.917	-136.348	117.86	0.124232	

STORY9	Max Drift X	12D10E10L	44	178.965	-132.781	106.19	0.104198	
STORY8	Max Drift X	12D10E10L	44	178.965	-132.781	94.52	0.087869	
STORY7	Max Drift X	12D10E10L	44	178.965	-132.781	82.85	0.071895	
STORY6	Max Drift X	12D10E10L	44	178.965	-132.781	71.18	0.057123	
STORY5	Max Drift X	12D10E10L	44	178.965	-132.781	59.51	0.043285	
STORY4	Max Drift X	12D10E10L	44	178.965	-132.781	47.84	0.030147	
STORY1	Max Drift X	12D10E10L	44	178.965	-132.781	12.83	0.024509	
STORY3	Max Drift X	12D10E10L	133	20	-144.167	36.17	0.018845	
STORY2	Max Drift X	12D10E10L	122	0	-144.167	24.5	0.015335	
STORY11	Max Drift Y	12D10E10L	44	178.965	-132.781	130		0.348082
STORY10	Max Drift Y	12D10E10L	8	178.917	0	117.86		0.278713
STORY9	Max Drift Y	12D10E10L	44	178.965	-132.781	106.19		0.245145
STORY8	Max Drift Y	12D10E10L	173	178.917	-108.167	94.52		0.210977
STORY7	Max Drift Y	12D10E10L	173	178.917	-108.167	82.85		0.178277
STORY6	Max Drift Y	12D10E10L	173	178.917	-108.167	71.18		0.146542
STORY5	Max Drift Y	12D10E10L	173	178.917	-108.167	59.51		0.115583
STORY4	Max Drift Y	12D10E10L	173	178.917	-108.167	47.84		0.085224
STORY3	Max Drift Y	12D10E10L	44	178.965	-132.781	36.17		0.055493
STORY2	Max Drift Y	12D10E10L	334	74.917	-179.104	24.5		0.038852
STORY1	Max Drift Y	12D10E10L	139	60.917	-108.167	12.83		0.05559
STORY11	Max Drift X	12D16W10L	44	178.965	-132.781	130	0.167316	
STORY10	Max Drift X	12D16W10L	399	170.917	-136.348	117.86	0.124641	
STORY9	Max Drift X	12D16W10L	44	178.965	-132.781	106.19	0.104704	
STORY8	Max Drift X	12D16W10L	44	178.965	-132.781	94.52	0.088475	
STORY7	Max Drift X	12D16W10L	44	178.965	-132.781	82.85	0.072585	
STORY6	Max Drift X	12D16W10L	44	178.965	-132.781	71.18	0.057869	
STORY5	Max Drift X	12D16W10L	44	178.965	-132.781	59.51	0.044064	
STORY4	Max Drift X	12D16W10L	44	178.965	-132.781	47.84	0.030941	
STORY1	Max Drift X	12D16W10L	44	178.965	-132.781	12.83	0.023475	
STORY3	Max Drift X	12D16W10L	133	20	-144.167	36.17	0.019692	
STORY2	Max Drift X	12D16W10L	122	0	-144.167	24.5	0.016308	
STORY11	Max Drift Y	12D16W10L	44	178.965	-132.781	130		0.348099
STORY10	Max Drift Y	12D16W10L	8	178.917	0	117.86		0.278755
STORY9	Max Drift Y	12D16W10L	44	178.965	-132.781	106.19		0.245238
STORY8	Max Drift Y	12D16W10L	173	178.917	-108.167	94.52		0.211128
STORY7	Max Drift Y	12D16W10L	173	178.917	-108.167	82.85		0.178485
STORY6	Max Drift Y	12D16W10L	173	178.917	-108.167	71.18		0.146805
STORY5	Max Drift Y	12D16W10L	173	178.917	-108.167	59.51		0.115895
STORY4	Max Drift Y	12D16W10L	173	178.917	-108.167	47.84		0.085578
STORY3	Max Drift Y	12D16W10L	44	178.965	-132.781	36.17		0.055864
STORY2	Max Drift Y	12D16W10L	334	74.917	-179.104	24.5		0.039255
STORY1	Max Drift Y	12D16W10L	139	60.917	-108.167	12.83		0.056267

Appendix – Lateral System Analysis

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY11	Max Drift X	09D10E	64	3346.999	2081.852	1560	0.009718	
STORY10	Max Drift X	09D10E	64	3346.999	2081.852	1414.32	0.009677	
STORY9	Max Drift X	09D10E	64	3346.999	2081.852	1274.28	0.009552	
STORY8	Max Drift X	09D10E	64	3346.999	2081.852	1134.24	0.009294	
STORY7	Max Drift X	09D10E	64	3346.999	2081.852	994.2	0.008861	
STORY6	Max Drift X	09D10E	64	3346.999	2081.852	854.16	0.008218	
STORY5	Max Drift X	09D10E	64	3346.999	2081.852	714.12	0.007336	
STORY4	Max Drift X	09D10E	64	3346.999	2081.852	574.08	0.006198	
STORY3	Max Drift X	09D10E	64	3346.999	2081.852	434.04	0.004796	
STORY2	Max Drift X	09D10E	64	3346.999	2081.852	294	0.003136	
STORY1	Max Drift X	09D10E	64	3346.999	2081.852	153.96	0.001133	
STORY9	Max Drift X	09D16W	452	352.041	127.643	1274.28	0.000078	
STORY8	Max Drift X	09D16W	452	352.041	127.643	1134.24	0.000078	
STORY10	Max Drift X	09D16W	452	352.041	127.643	1414.32	0.000077	
STORY7	Max Drift X	09D16W	452	352.041	127.643	994.2	0.000077	
STORY11	Max Drift X	09D16W	452	352.041	127.643	1560	0.000075	
STORY6	Max Drift X	09D16W	452	352.041	127.643	854.16	0.000075	
STORY5	Max Drift X	09D16W	452	352.041	127.643	714.12	0.000071	
STORY4	Max Drift X	09D16W	452	352.041	127.643	574.08	0.000065	
STORY3	Max Drift X	09D16W	452	352.041	127.643	434.04	0.000056	
STORY2	Max Drift X	09D16W	452	352.041	127.643	294	0.000043	
STORY1	Max Drift X	09D16W	452	352.041	127.643	153.96	0.000024	
STORY10	Max Drift X	12D08W	452	352.041	127.643	1414.32	0.00004	
STORY9	Max Drift X	12D08W	452	352.041	127.643	1274.28	0.00004	
STORY8	Max Drift X	12D08W	452	352.041	127.643	1134.24	0.00004	
STORY7	Max Drift X	12D08W	452	352.041	127.643	994.2	0.00004	
STORY11	Max Drift X	12D08W	452	352.041	127.643	1560	0.000039	
STORY6	Max Drift X	12D08W	452	352.041	127.643	854.16	0.000039	
STORY5	Max Drift X	12D08W	452	352.041	127.643	714.12	0.000036	
STORY4	Max Drift X	12D08W	452	352.041	127.643	574.08	0.000033	
STORY3	Max Drift X	12D08W	452	352.041	127.643	434.04	0.000028	
STORY2	Max Drift X	12D08W	452	352.041	127.643	294	0.000022	
STORY1	Max Drift X	12D08W	452	352.041	127.643	153.96	0.000012	
STORY11	Max Drift X	12D10E10L	64	3346.999	2081.852	1560	0.009718	
STORY10	Max Drift X	12D10E10L	64	3346.999	2081.852	1414.32	0.009677	
STORY9	Max Drift X	12D10E10L	64	3346.999	2081.852	1274.28	0.009552	
STORY8	Max Drift X	12D10E10L	64	3346.999	2081.852	1134.24	0.009294	
STORY7	Max Drift X	12D10E10L	64	3346.999	2081.852	994.2	0.008861	
STORY6	Max Drift X	12D10E10L	64	3346.999	2081.852	854.16	0.008218	
STORY5	Max Drift X	12D10E10L	64	3346.999	2081.852	714.12	0.007336	
STORY4	Max Drift X	12D10E10L	64	3346.999	2081.852	574.08	0.006198	
STORY3	Max Drift X	12D10E10L	64	3346.999	2081.852	434.04	0.004796	
STORY2	Max Drift X	12D10E10L	64	3346.999	2081.852	294	0.003136	
STORY1	Max Drift X	12D10E10L	64	3346.999	2081.852	153.96	0.001133	
STORY11	Max Drift X	12D16L	452	352.041	127.643	1560	0.000002	
STORY10	Max Drift X	12D16L	452	352.041	127.643	1414.32	0.000002	

STORY9	Max Drift X	12D16L	452	352.041	127.643	1274.28	0.000002	
STORY8	Max Drift X	12D16L	452	352.041	127.643	1134.24	0.000002	
STORY7	Max Drift X	12D16L	452	352.041	127.643	994.2	0.000002	
STORY6	Max Drift X	12D16L	452	352.041	127.643	854.16	0.000002	
STORY5	Max Drift X	12D16L	452	352.041	127.643	714.12	0.000002	
STORY4	Max Drift X	12D16L	452	352.041	127.643	574.08	0.000001	
STORY3	Max Drift X	12D16L	452	352.041	127.643	434.04	0.000001	
STORY2	Max Drift X	12D16L	452	352.041	127.643	294	0.000001	
STORY1	Max Drift X	12D16L	452	352.041	127.643	153.96	0	
STORY11	Max Drift X	12D16WL05E	64	3346.999	2081.852	1560	0.004859	
STORY10	Max Drift X	12D16WL05E	64	3346.999	2081.852	1414.32	0.004839	
STORY9	Max Drift X	12D16WL05E	64	3346.999	2081.852	1274.28	0.004776	
STORY8	Max Drift X	12D16WL05E	64	3346.999	2081.852	1134.24	0.004647	
STORY7	Max Drift X	12D16WL05E	64	3346.999	2081.852	994.2	0.004431	
STORY6	Max Drift X	12D16WL05E	64	3346.999	2081.852	854.16	0.004109	
STORY5	Max Drift X	12D16WL05E	64	3346.999	2081.852	714.12	0.003668	
STORY4	Max Drift X	12D16WL05E	64	3346.999	2081.852	574.08	0.003099	
STORY3	Max Drift X	12D16WL05E	64	3346.999	2081.852	434.04	0.002398	
STORY2	Max Drift X	12D16WL05E	64	3346.999	2081.852	294	0.001568	
STORY1	Max Drift X	12D16WL05E	64	3346.999	2081.852	153.96	0.000567	
STORY10	Max Drift X	DWAL2	452	352.041	127.643	1414.32	0.000003	
STORY9	Max Drift X	DWAL2	452	352.041	127.643	1274.28	0.000003	
STORY8	Max Drift X	DWAL2	452	352.041	127.643	1134.24	0.000003	
STORY7	Max Drift X	DWAL2	452	352.041	127.643	994.2	0.000002	
STORY6	Max Drift X	DWAL2	452	352.041	127.643	854.16	0.000002	
STORY5	Max Drift X	DWAL2	452	352.041	127.643	714.12	0.000002	
STORY4	Max Drift X	DWAL2	452	352.041	127.643	574.08	0.000002	
STORY3	Max Drift X	DWAL2	452	352.041	127.643	434.04	0.000001	
STORY2	Max Drift X	DWAL2	452	352.041	127.643	294	0.000001	
STORY1	Max Drift X	DWAL2	452	352.041	127.643	153.96	0	
STORY8	Max Drift X	DWAL3	452	352.041	127.643	1134.24	0.000063	
STORY10	Max Drift X	DWAL3	452	352.041	127.643	1414.32	0.000062	
STORY9	Max Drift X	DWAL3	452	352.041	127.643	1274.28	0.000062	
STORY7	Max Drift X	DWAL3	452	352.041	127.643	994.2	0.000062	
STORY11	Max Drift X	DWAL3	452	352.041	127.643	1560	0.000061	
STORY6	Max Drift X	DWAL3	452	352.041	127.643	854.16	0.00006	
STORY5	Max Drift X	DWAL3	452	352.041	127.643	714.12	0.000057	
STORY4	Max Drift X	DWAL3	452	352.041	127.643	574.08	0.000052	
STORY3	Max Drift X	DWAL3	452	352.041	127.643	434.04	0.000045	
STORY2	Max Drift X	DWAL3	452	352.041	127.643	294	0.000035	
STORY1	Max Drift X	DWAL3	452	352.041	127.643	153.96	0.000019	
STORY8	Max Drift X	DWAL4	452	352.041	127.643	1134.24	0.000059	
STORY7	Max Drift X	DWAL4	452	352.041	127.643	994.2	0.000059	
STORY10	Max Drift X	DWAL4	452	352.041	127.643	1414.32	0.000058	
STORY9	Max Drift X	DWAL4	452	352.041	127.643	1274.28	0.000058	
STORY6	Max Drift X	DWAL4	452	352.041	127.643	854.16	0.000057	
STORY11	Max Drift X	DWAL4	452	352.041	127.643	1560	0.000056	
STORY5	Max Drift X	DWAL4	452	352.041	127.643	714.12	0.000054	

STORY4	Max Drift X	DWAL4	452	352.041	127.643	574.08	0.00005	
STORY3	Max Drift X	DWAL4	452	352.041	127.643	434.04	0.000043	
STORY2	Max Drift X	DWAL4	452	352.041	127.643	294	0.000033	
STORY1	Max Drift X	DWAL4	452	352.041	127.643	153.96	0.000019	
STORY8	Max Drift X	DWAL5	452	352.041	127.643	1134.24	0.000064	
STORY10	Max Drift X	DWAL5	452	352.041	127.643	1414.32	0.000063	
STORY9	Max Drift X	DWAL5	452	352.041	127.643	1274.28	0.000063	
STORY7	Max Drift X	DWAL5	452	352.041	127.643	994.2	0.000063	
STORY11	Max Drift X	DWAL5	452	352.041	127.643	1560	0.000061	
STORY6	Max Drift X	DWAL5	452	352.041	127.643	854.16	0.000061	
STORY5	Max Drift X	DWAL5	452	352.041	127.643	714.12	0.000058	
STORY4	Max Drift X	DWAL5	452	352.041	127.643	574.08	0.000053	
STORY3	Max Drift X	DWAL5	452	352.041	127.643	434.04	0.000045	
STORY2	Max Drift X	DWAL5	452	352.041	127.643	294	0.000035	
STORY1	Max Drift X	DWAL5	452	352.041	127.643	153.96	0.000019	
STORY9	Max Drift X	DWAL6	452	352.041	127.643	1274.28	0.00006	
STORY8	Max Drift X	DWAL6	452	352.041	127.643	1134.24	0.00006	
STORY7	Max Drift X	DWAL6	452	352.041	127.643	994.2	0.00006	
STORY10	Max Drift X	DWAL6	452	352.041	127.643	1414.32	0.000059	
STORY11	Max Drift X	DWAL6	452	352.041	127.643	1560	0.000058	
STORY6	Max Drift X	DWAL6	452	352.041	127.643	854.16	0.000058	
STORY5	Max Drift X	DWAL6	452	352.041	127.643	714.12	0.000055	
STORY4	Max Drift X	DWAL6	452	352.041	127.643	574.08	0.000051	
STORY3	Max Drift X	DWAL6	452	352.041	127.643	434.04	0.000044	
STORY2	Max Drift X	DWAL6	452	352.041	127.643	294	0.000034	
STORY1	Max Drift X	DWAL6	452	352.041	127.643	153.96	0.000019	
STORY11	Max Drift X	DWAL7	64	3346.999	2081.852	1560	0.020444	
STORY10	Max Drift X	DWAL7	64	3346.999	2081.852	1414.32	0.020358	
STORY9	Max Drift X	DWAL7	64	3346.999	2081.852	1274.28	0.020095	
STORY8	Max Drift X	DWAL7	64	3346.999	2081.852	1134.24	0.019553	
STORY7	Max Drift X	DWAL7	64	3346.999	2081.852	994.2	0.018642	
STORY6	Max Drift X	DWAL7	64	3346.999	2081.852	854.16	0.017288	
STORY5	Max Drift X	DWAL7	64	3346.999	2081.852	714.12	0.015433	
STORY4	Max Drift X	DWAL7	64	3346.999	2081.852	574.08	0.013039	
STORY3	Max Drift X	DWAL7	64	3346.999	2081.852	434.04	0.01009	
STORY2	Max Drift X	DWAL7	64	3346.999	2081.852	294	0.006597	
STORY1	Max Drift X	DWAL7	64	3346.999	2081.852	153.96	0.002384	
STORY11	Max Drift X	DWAL8	64	3346.999	2081.852	1560	0.020444	
STORY10	Max Drift X	DWAL8	64	3346.999	2081.852	1414.32	0.020358	
STORY9	Max Drift X	DWAL8	64	3346.999	2081.852	1274.28	0.020095	
STORY8	Max Drift X	DWAL8	64	3346.999	2081.852	1134.24	0.019553	
STORY7	Max Drift X	DWAL8	64	3346.999	2081.852	994.2	0.018642	
STORY6	Max Drift X	DWAL8	64	3346.999	2081.852	854.16	0.017288	
STORY5	Max Drift X	DWAL8	64	3346.999	2081.852	714.12	0.015433	
STORY4	Max Drift X	DWAL8	64	3346.999	2081.852	574.08	0.013039	
STORY3	Max Drift X	DWAL8	64	3346.999	2081.852	434.04	0.01009	
STORY2	Max Drift X	DWAL8	64	3346.999	2081.852	294	0.006597	
STORY1	Max Drift X	DWAL8	64	3346.999	2081.852	153.96	0.002384	

STORY11	Max Drift X	DWAL9	64	3346.999	2081.852	1560	0.020845	
STORY10	Max Drift X	DWAL9	64	3346.999	2081.852	1414.32	0.020757	
STORY9	Max Drift X	DWAL9	64	3346.999	2081.852	1274.28	0.020489	
STORY8	Max Drift X	DWAL9	64	3346.999	2081.852	1134.24	0.019936	
STORY7	Max Drift X	DWAL9	64	3346.999	2081.852	994.2	0.019008	
STORY6	Max Drift X	DWAL9	64	3346.999	2081.852	854.16	0.017627	
STORY5	Max Drift X	DWAL9	64	3346.999	2081.852	714.12	0.015736	
STORY4	Max Drift X	DWAL9	64	3346.999	2081.852	574.08	0.013294	
STORY3	Max Drift X	DWAL9	64	3346.999	2081.852	434.04	0.010288	
STORY2	Max Drift X	DWAL9	64	3346.999	2081.852	294	0.006727	
STORY1	Max Drift X	DWAL9	64	3346.999	2081.852	153.96	0.002431	
STORY11	Max Drift X	DWAL10	64	3346.999	2081.852	1560	0.020845	
STORY10	Max Drift X	DWAL10	64	3346.999	2081.852	1414.32	0.020757	
STORY9	Max Drift X	DWAL10	64	3346.999	2081.852	1274.28	0.020489	
STORY8	Max Drift X	DWAL10	64	3346.999	2081.852	1134.24	0.019936	
STORY7	Max Drift X	DWAL10	64	3346.999	2081.852	994.2	0.019008	
STORY6	Max Drift X	DWAL10	64	3346.999	2081.852	854.16	0.017627	
STORY5	Max Drift X	DWAL10	64	3346.999	2081.852	714.12	0.015736	
STORY4	Max Drift X	DWAL10	64	3346.999	2081.852	574.08	0.013294	
STORY3	Max Drift X	DWAL10	64	3346.999	2081.852	434.04	0.010288	
STORY2	Max Drift X	DWAL10	64	3346.999	2081.852	294	0.006727	
STORY1	Max Drift X	DWAL10	64	3346.999	2081.852	153.96	0.002431	
STORY11	Max Drift X	DWAL11	64	3346.999	2081.852	1560	0.000019	
STORY10	Max Drift X	DWAL11	64	3346.999	2081.852	1414.32	0.000019	
STORY9	Max Drift X	DWAL11	64	3346.999	2081.852	1274.28	0.000019	
STORY8	Max Drift X	DWAL11	64	3346.999	2081.852	1134.24	0.000019	
STORY7	Max Drift X	DWAL11	64	3346.999	2081.852	994.2	0.000018	
STORY6	Max Drift X	DWAL11	64	3346.999	2081.852	854.16	0.000016	
STORY5	Max Drift X	DWAL11	64	3346.999	2081.852	714.12	0.000015	
STORY4	Max Drift X	DWAL11	64	3346.999	2081.852	574.08	0.000012	
STORY3	Max Drift X	DWAL11	64	3346.999	2081.852	434.04	0.00001	
STORY2	Max Drift X	DWAL11	64	3346.999	2081.852	294	0.000006	
STORY1	Max Drift X	DWAL11	64	3346.999	2081.852	153.96	0.000002	
STORY11	Max Drift X	DWAL12	64	3346.999	2081.852	1560	0.00002	
STORY10	Max Drift X	DWAL12	64	3346.999	2081.852	1414.32	0.00002	
STORY9	Max Drift X	DWAL12	64	3346.999	2081.852	1274.28	0.000019	
STORY8	Max Drift X	DWAL12	64	3346.999	2081.852	1134.24	0.000019	
STORY7	Max Drift X	DWAL12	64	3346.999	2081.852	994.2	0.000018	
STORY6	Max Drift X	DWAL12	64	3346.999	2081.852	854.16	0.000017	
STORY5	Max Drift X	DWAL12	64	3346.999	2081.852	714.12	0.000015	
STORY4	Max Drift X	DWAL12	64	3346.999	2081.852	574.08	0.000013	
STORY3	Max Drift X	DWAL12	64	3346.999	2081.852	434.04	0.00001	
STORY2	Max Drift X	DWAL12	64	3346.999	2081.852	294	0.000006	
STORY1	Max Drift X	DWAL12	64	3346.999	2081.852	153.96	0.000002