Pearl Condominiums

9th & Arch Street
Philadelphia, PA

AE Senior Thesis

Final Report

May 2, 2008

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Joseph G. Lichman Jr.

Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2008/jgl138/

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9th and Arch Street
Philadelphia, PA

The Project Team:

Owner - Parkway Corporation
Architect – Blackney Hayes Architects
Construction Manager – JJ Deluca Company Inc.
Structural Engineer – Pennoni Associates Inc.
Civil Engineer – Valimer Associates, LLP
Mechanical Engineer – M.P. Hershman, PE, Inc.
Electrical Engineer – DGW Electrical Engineers, Inc.
Geotechnical Engineer – David Blackmore and Associates

The Buidling:

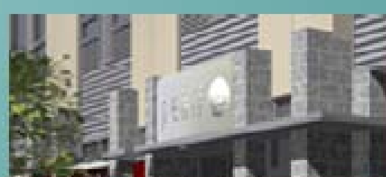
Size: 111,570 S.F.
Stories Above Grade: 6
Cost : $21.5 Million Dollars
Building Completion: October 2007
Occupancy: The Building Is A Mixed Use Developement Containing Retail and Housing
Delivery Method: Design-Bid -Build

Architecture:

"Modern" and "Sophisticated" Design
Combines the uses of Metal, Glass, Stucco And Masonry

Lighting\Electrical:

Primary Fluorescent Lighting
208/120 3 Phase, 4 Wire Main Sytem
Emergency Power: Diesel Generator 120/208V, 24hrs Capacity



Structural:

Foundation: 6" SOG With WWR Reinfrocing, Drilled Piers
SuperStructure: Structural Steel With Masonry And Steel Stud Load Bearing Walls
Floor System: 10" Precast Planks
Roof System: Steel Joists Supporting Steel Deck, Ruber Membrane And Rigid Insulation

Mechanical:

Main Components:
Engineered Air Make-Up Air RoofTop Units
3500 CFM, Cooling - 293,000 BTU
Heating - 341,000 BTU
GC Zoneline Terminal AC Units
310CFM, Cooling - 14,300 BTU
Heating 13,200 BTU

Design Consideration: Site Location Existing Above SEPTA Commuter Rail Tunnel

Joseph G. Lichman Jr. - Structural Option

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Executive Summary

Pearl Condominiums is located on 9th and Arch Street in Philadelphia, Pennsylvania. This structure is a mixed use development building. The building includes a retail floor at the ground level containing 10 units and five floors of housing above containing a total of 90 condominium units. One of the main design considerations for the site was the location of an existing SEPTA commuter rail tunnel which runs under the site

The objective of this report is to compare the existing structure of Pearl Condominiums building as a hybrid type of construction which uses load bearing wall and precast concrete planks to the redesigned structure which implements the use of the Flex Frame system that uses the combination of girder slab technology. The major factor influencing the redesign is the elimination of interior load bearing walls, thus resulting in a more flexible floor plan. The lateral system will also be changed to concrete shears walls to replace the concrete masonry shear walls present in the existing design.

The second topic that will be discussed in the paper is the foundation system. The analysis will focus on a possible alternative to the use of drilled piers, but will also include the effect on the train tunnel that runs underneath the site. This will help to determine if drilled piers were the most economical system for the foundation of Pearl Condominiums.

The paper will also briefly discuss two breadth topics concerning construction management and sustainability. It is proposed to analyze the effect of the Flex Frame system on construction cost and scheduling. Proposed for the issue of sustainability is the certification of the building for a LEED rating and how this will affect the building redesign.

Existing Gravity and Lateral Systems:

The gravity system of this building is comprised of load bearing walls and precast concrete planks. The main component in the lateral system is the use of concrete masonry units as shear walls in the stair towers and the elevator core. The ground floor contains moment frames to transfer lateral loads from the stair tower shear walls which end on the second floor. Finally, metal stud walls with metal strapping are used to help resist lateral load in the east to west direction of the building. From research and analysis performed during this semester the existing structural system used in Pearl Condominiums, was a very efficient and economical construction type to use for this type of building.

Conclusion:

After completing the analysis of the Flex Frame system, the conclusion that was reached was the Flex Frame is a viable alternative to the use of wall and plank construction. The cost and the time of construction required does not vary for this project therefore that is not a factor in limiting the use of the Flex Frame system. This system's main limitation occurs in the spans of the 8" precast planks because they are the only one the Flex Frame system uses. Overall both systems are very well designed for this type of construction.

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Acknowledgements

I would like to take this opportunity to acknowledge and thank all the people, design professionals and firms that gave up their time and effort to help in the completion of this senior thesis:

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Pennoni Associate Inc.
Mr. Keith Weitknecht
Mr. Mike Padula

The Pennsylvania State University
Dr. Linda Hanagan
The entire AE faculty and staff

Special thanks extended to my family and friends that have supported me through these last five years.

Building Background and Project Information

Pearl Condominiums is a mixed use development housing including 10 retail units on the ground floor and 90 condominium units on the upper floors. The gross floor area is 111,570 square feet and has 6 stories above grade. The start of construction was March 30, 2006 and the finish date is October 2007. The zoning is C-4 Commercial. Design considerations for the site included the site location existing above a SEPTA commuter rail tunnel. The Project Delivery method used was Design-bid-build and the total cost of the project was $22,646,674.

Primary project team:

- Owner - Parkway Corporation
- Architect – Blackney Hayes Architects
- Construction Manager – JJ Deluca Company Inc.
- Structural Engineer – Pennoni Associates Inc.
- Civil Engineer – Valimer Associates, LLP
- Mechanical Engineer – M.P. Hershman, PE, Inc.
- Electrical Engineer – DGW Electrical Engineers, Inc.
- Geotechnical Engineer – David Blackmore and Associates
- Plumbing and Fire Protection – Pan Am Consulting, Inc.

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Existing Structural System

Foundations:

The primary support for the foundation is the use of drilled piers. The drilled pier option was performed, so the loads from the building would be transferred from the pier to the soil below the SEPTA commuter train tunnel. The drilled piers range in size of diameter from 3'-0" to 3'-6" to 4'-0 To help distribute the load to the drilled piers the use of grade beams was employed. They range in width from 12" to 40" and in depth from 18" to 30". The slab on grade is 6" reinforced with 6x6 W2.9xW2.9 WWR over 6" crushed stone over 6 mil. Vapor retarder. (See Figure1)

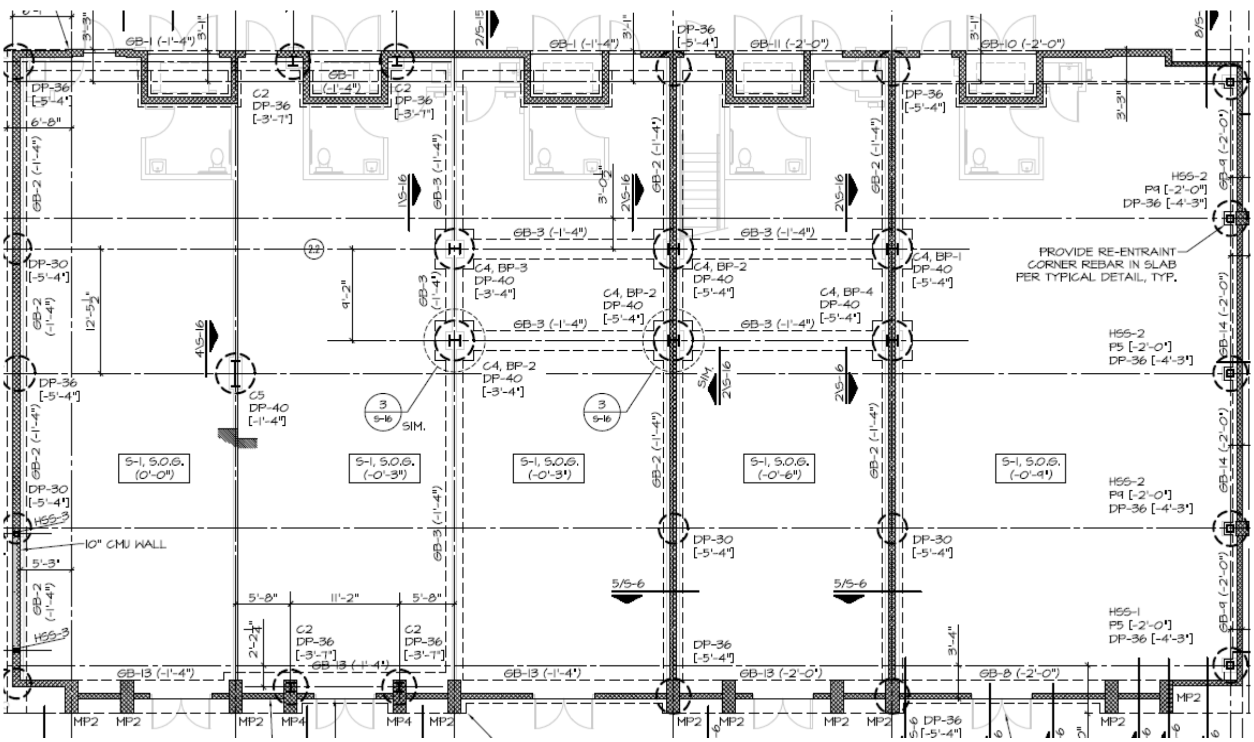


Figure 1 – South Side of Building Foundation Plan

Columns \ Load Bearing Walls:

The columns in Pearl Condominiums are used in two different types of loading. The HSS columns are used to take gravity loads, which occur at the ends of the building to support the precast concrete planks (Figure 2) and the Wide flange columns are used to resist lateral loads which occur on the ground floor in the moment frames (Figure 3)

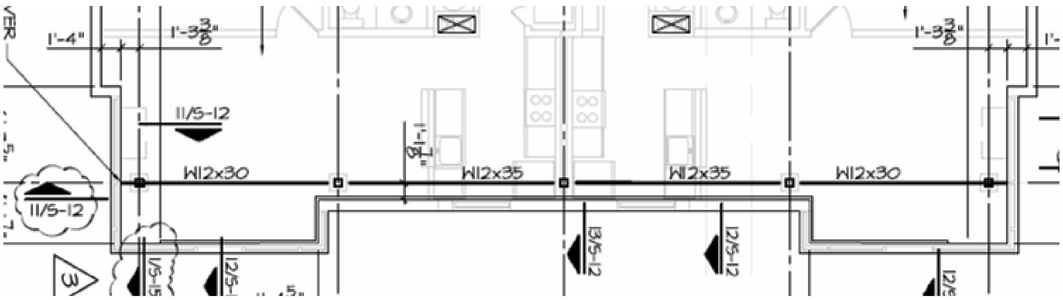


Figure 2 – HSS Columns at the south end of the building

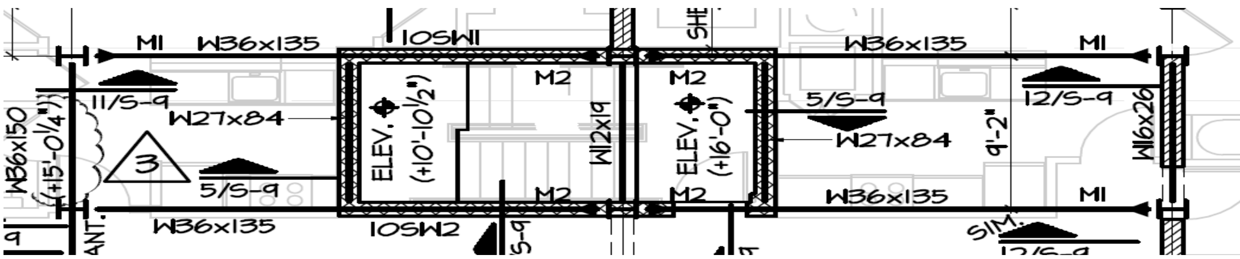


Figure 3 –Wide Flange Columns at the south side of the building

The interior bearing walls are comprised of 8 inch metal studs that are spaced at 12 inches and 16 inches on center depending on the floor location of the wall. (See Figure 4)

LOAD BEARING METAL STUD WALL SCHEDULE		
LEVEL	LOAD BEARING TYPE 1 STUD WALL	
	SIGMA STUDS	MARINOWARE STUDS
7TH - RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH - 7TH	800S6300-33 @ 16"	800S200-54 @ 12"
5TH - 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD - 5TH	800S6300-68 @ 16"	(2) 800S200-54 @ 12"
2ND - 3RD	800S6300-68 @ 12"	(2) 800S200-68 @ 12"
LEVEL	LOAD BEARING TYPE 2 STUD WALL	
7TH - RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH - 7TH	800S6300-33 @ 16"	(2) 800S200-43 @ 12"
5TH - 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD - 5TH	800S6300-68 @ 16"	(2) 800S200-68 @ 12"
2ND - 3RD	800S6300-68 @ 12"	(2) 800S200-97 @ 12"
LEVEL	LOAD BEARING TYPE 3 STUD WALL	
7TH - RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH - 7TH	800S6300-33 @ 16"	(2) 800S200-54 @ 12"
5TH - 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD - 5TH	800S6300-68 @ 16"	(2) 800S200-68 @ 12"
2ND - 3RD	800S6300-68 @ 12"	(2) 800S200-97 @ 12"

Figure 4 – Metal Stud Bearing Wall Schedule

Floor System:

The floor system for level 2 thru 6 is comprised of a 10” Precast Concrete Plank with a ¾” concrete thick topping. (See Figure 5) The concrete strength of the precast plank is f’c equals 5,000 psi.

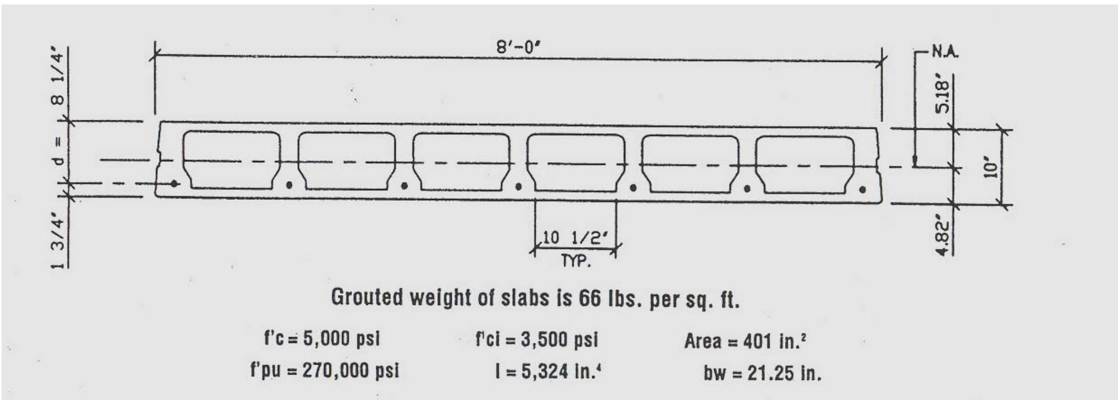


Figure 5 – Section Properties of Precast Planks

Level two acts as a transfer level, which requires the use of wide flange beam (W36). These transfer beams eliminate the need for load bearing walls to distribute the load to the foundations. The result of having these elements increases the available floor area for the retail units. (See Figure 6)

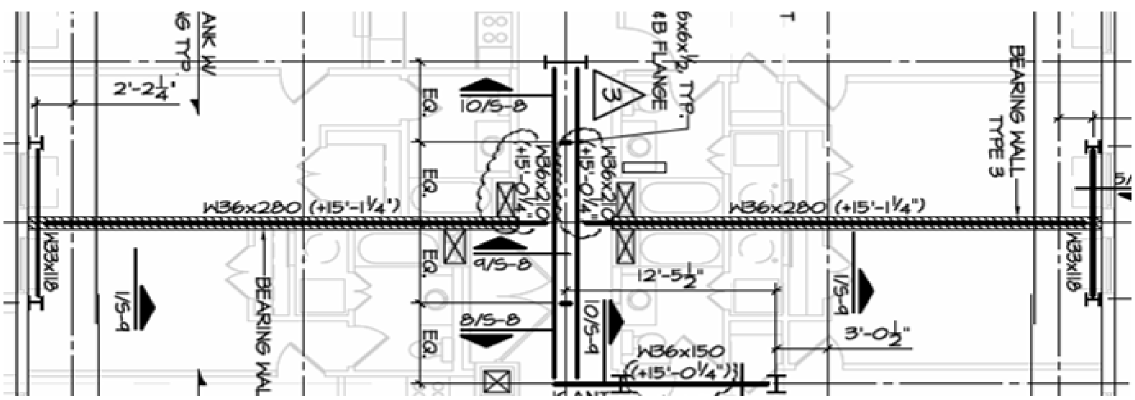


Figure 6 – Transfer Beams located on the Second Floor.

Typical Floor Plan

As shown in Figure 7, Pearl Condominiums shape and layout of the space is symmetric. This lends itself to the idea of using a structural system that is uses repetition in the design of the floor system. The floor system consisting of precast planks and metal stud bearing walls can easily duplicated from floor to floor.

Lateral Resisting System:

The Lateral System in the building is comprised of three types: concrete masonry unit (cmu) shear walls, moment frames and metal stud shear wall. (See Figure 7)

The concrete masonry unit shear walls are used around the elevator and stairway towers. These walls range from thickness of 10” in the stair areas and 12” in the elevators. The strength of the concrete masonry units (f’m) range from 1500 psi to 2000psi and 3000psi depending on the area they are used in.

The stair tower cmu walls end on the second floor, which results in the use of moment frames on the first floor to transfer the loads from the cmu shear walls on the second floor to the foundation below.

The metal stud shear walls are composed of 8” metal studs varying in thickness. The two heights of the studs are 13’-8” and 9’-0”. Metal diagonal straps connected by #12 screws to the metal studs and 7/8” diameter anchor bolts connected through different boot types help to resist the lateral forces applied to the metal studs. The metal studs are covered by gypsum wall board.

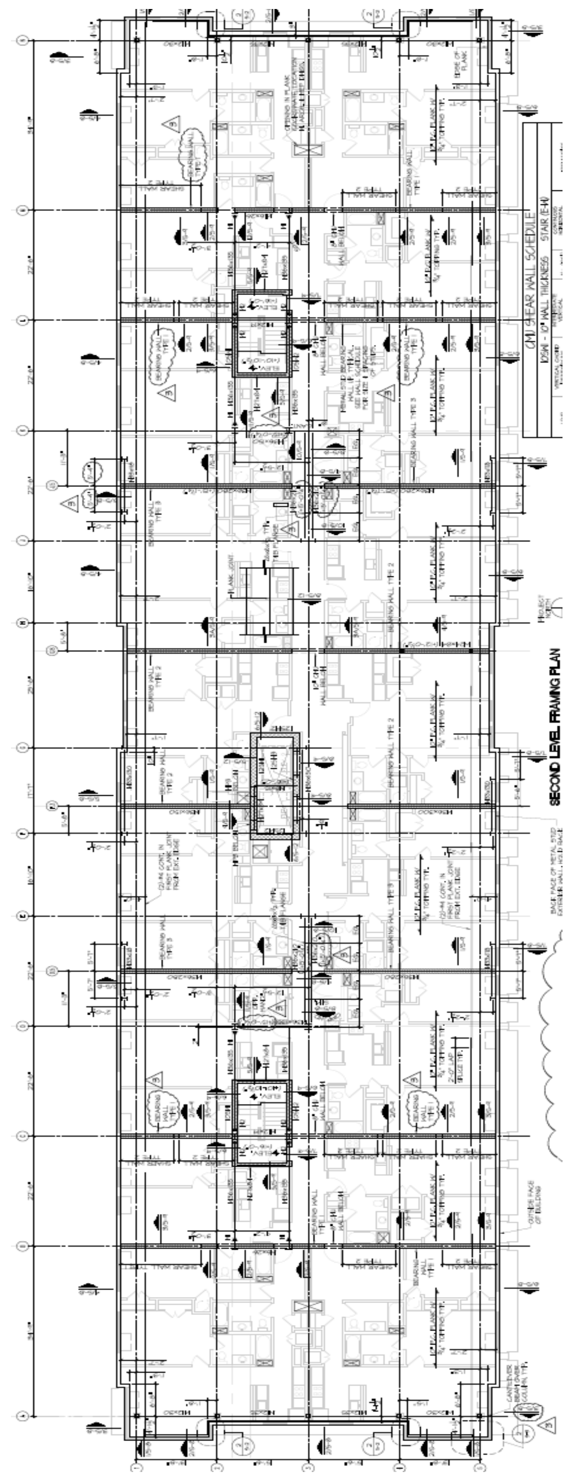


Figure 7 –Lateral Resisting System Present in Pearl Condominium

Problem Statement

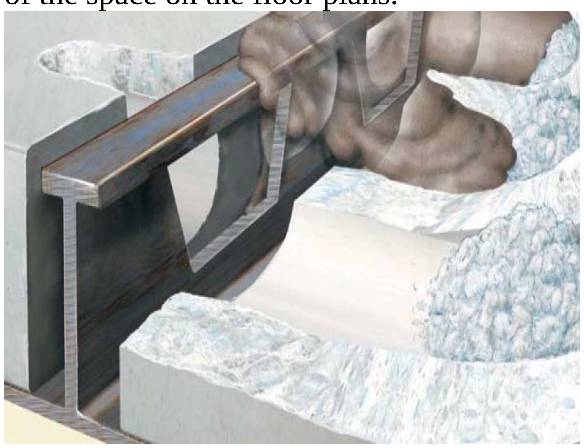
During the analysis of the gravity and the wind force resisting systems present in Pearl Condominiums, it is clear that the systems chosen by the design team is currently the most efficient possible. A possible issue arose during the analysis of the gravity system with the use of the load bearing wall, which limited the flexibility of the floor plans. For the redesign the floor system, the main objective will be to try and see if another structural system is possible to be able to create a more flexible floor plan by eliminating the need for the interior load bearing walls. Another aspect that will be investigated will be the lateral shear walls changing the material from concrete masonry units to reinforced concrete With respect to the foundation system, the objective will be to compare other possible systems to the existing drilled piers system to see if any are a viable alternative. This will prove if the current system was the most economical and efficient system used because of the impact of the SEPTA commuter train tunnel that runs underneath the site.

Problem Solution

Floor system:

The redesign of the system includes first and foremost the removal of the interior load bearing walls. To compensate for the loss of the interior load bearing walls, the proposed structural system to be implemented is the Flex Frame system. This floor system is comprised of precast concrete planks which are supported by special steel “d beams”. The precast planks are grouted solid around the “d beam” to create the beam to plank connection. (See Figure 3) The “d beam” is a specialty beam which is created by cutting a wide flange beam in half and adding a plate, with a smaller width than the bottom flange, to create a top flange.

With the Flex Frame system there are a few limiting factors such as the deepest member that is currently available is a nine inch “d beam”, this will decrease the possible span and tributary width that the beam can carry. From this the spans of the precast concrete planks will be reduced, in turn reducing the size of the typical bay. The overall geometry of the building will not be changed but the implementation of a column grid will be used to facilitate in the redesign. This column grid will be beneficial in seeing the impact of the new system on the layout of the space on the floor plans.



Structural Depth Study

The objective for the depth study is the analysis of the Flex Frame system in creating an alternative solution to the use of load bearing walls used in Pearl Condominiums. This will result in creating an open floor plan by eliminating the use of the load bearing walls.

To begin the redesign, the first area focused on was the roof, which required the transition from the use of the steel joist and load bearing wall system to a wide flange beam and girder system. This was done primarily to see the comparison in the size of steel joists to steel wide flange beams. With the steel joists the required depth to support the applied load was 24 inches and the steel beam maximum required depth designed is 14 inches.

With the removal of the load bearing walls the implementation of the HSS columns were required to pick up the load from the steel girders. The roof load was added to by the addition of the extensive green roof, in an effort to gain points for a LEED certification. The total load applied to the steel beam was 45 psf for dead and 30 psf for live load. The grid for the entire structural system was developed to limit the interference on the architectural plans. The columns followed the load path that the load bearing walls were designed. For the roof the largest span was 34’-9”, which resulted in a beam the size of W14x 61. The columns were spaced at 11’-8” on the column line. (See Figure 8 for Column Schedule) (See Figure 9 A&B for new framing plan of roof.

Column Schedule

All Columns on Line	Floor Level	Type of Column
A&N	Roof - 7	HSS 6x6x1/4
A&N	7 - 6	HSS 6x6x1/4
A&N	6 - 5	HSS 6x6x1/4
A&N	5 - 3	HSS 6x6x5/16
A&N	3 - 2	HSS 6x6x5/16
A&N	2 - Ground	HSS 6x6x1/2
B-M	Roof - 7	HSS 6x6x1/4
B-M	7 - 6	HSS 6x6x1/4
B-M	6 - 5	HSS 6x6x3/8
B-M	5 - 3	HSS 6x6x1/2
B-M	3 - 2	HSS 8x8x1/2
B,C,G.8,L	2 - Ground	HSS 10x10x1/2

Figure 8 – Column Schedule

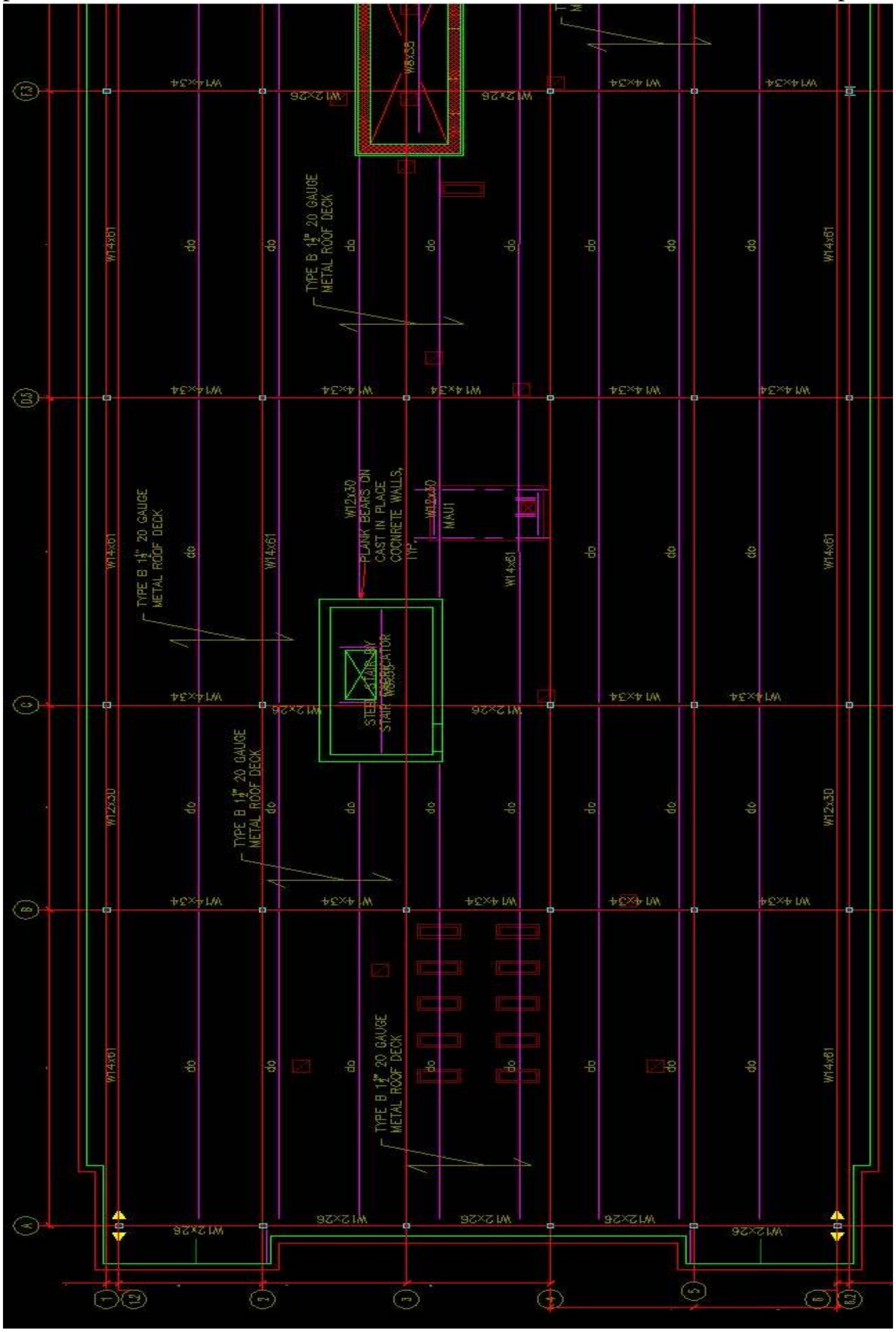


Figure 9A – North Side Roof Structural Framing Plan

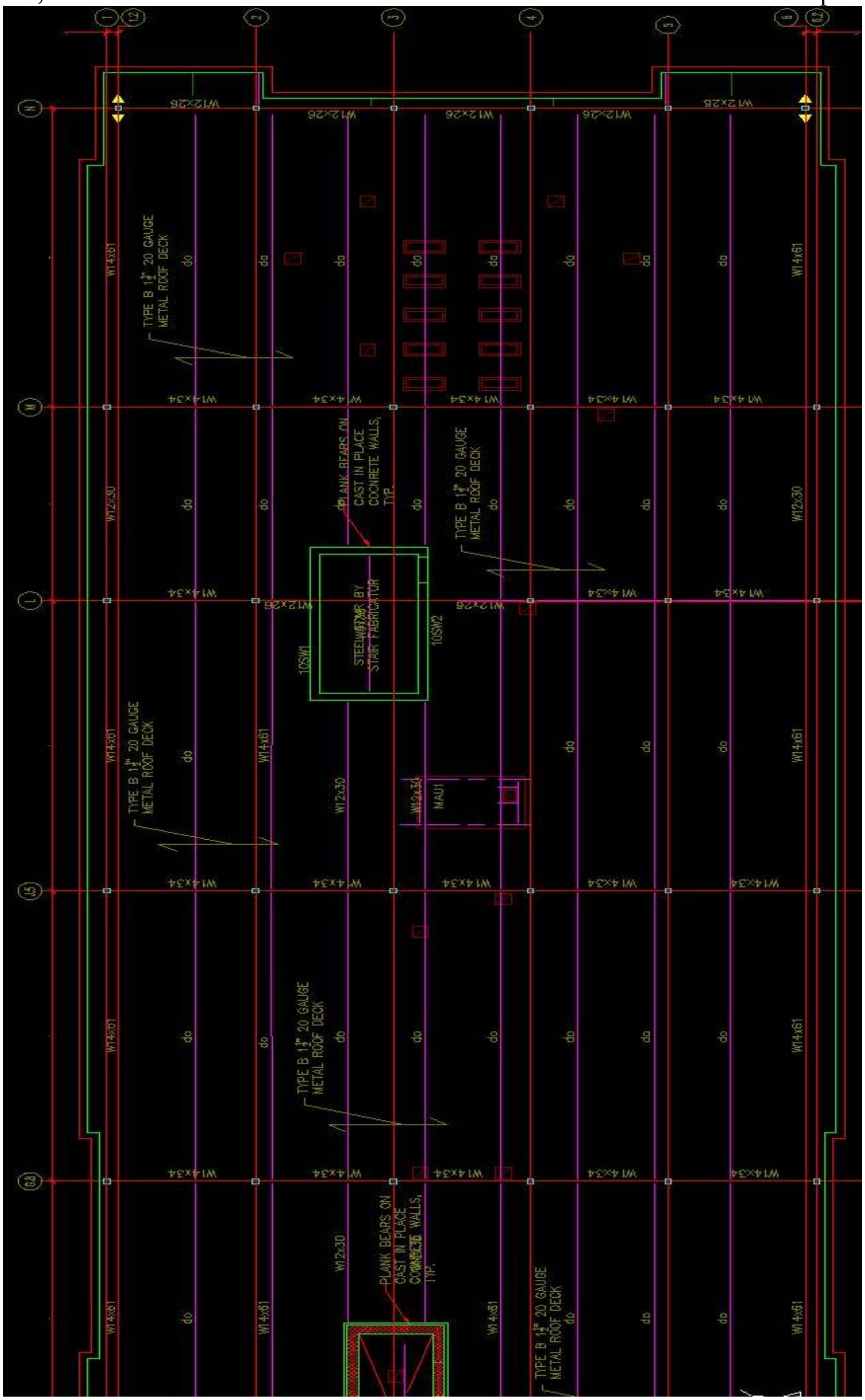


Figure 9B – South Side Structural Roof Framing Plan

For the redesign of floors 3, 5, 6, and 7 the framing was kept all the same to limit the effect on the architectural floor plan, and simplicity of construction with repetitive members. (See Figure11) This is were the implementation of the Flex Frame did occur The Dissymmetric beam that was chosen for this design was the DB 9x46, this is the largest size that is currently available. The limitation to the Flex Frame system that arose during the redesign is in the use of the precast planks. The only precast plank that can be used is the 8 inch thick precast plank. For the required spans that are present in this design the 8 inch precast plank with 2 inch topping will have to have a 1 inch camber before erection to limit the deflection after the load is applied. (See Figure 10 – 4HC8+2; 78S span 33'-34' chosen for design)

Strand Pattern Designation
76-S

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

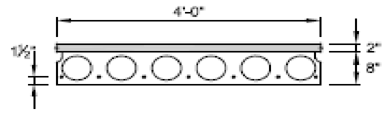
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

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

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

HOLLOW-CORE

4'-0" x 8"
Normal Weight Concrete



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

Section Properties

	Untopped		Topped	
A =	215	in.^{2}	311	in.^{2}
I =	1,666	in.^{4}	3,071	in.^{4}
y_b =	4.00	in.	5.29	in.
y_t =	4.00	in.	4.71	in.
S_b =	417	in.^{3}	581	in.^{3}
S_t =	417	in.^{3}	652	in.^{3}
wt =	224	plf	324	plf
DL =	56	psf	81	psf
V/S =	1.92	in.		

4HC8

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

Strand Designation Code	Span, ft																													
	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	−0.1	−0.2	−0.3	−0.5	−0.6						
	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	0.2	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9	−1.2	−1.4						
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.6	−0.8	−1.1	−1.4	−1.7	−2.0			
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	−0.4	−0.3	−0.5	−0.7	−0.9
	0.3	0.3	0.4	0.4	0.5	0.5	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	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4
68-S	476	430	393	361	332	309	286	269	253	236	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	−0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.8	−1.1	−1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	−0.3	−0.7

4HC8 + 2

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

Strand Designation Code	Span, ft																											
	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	106	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	−0.0	−0.1	−0.2	−0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.4	−0.6	−0.7	−0.9	−1.2	−1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	−0.0	−0.1	−0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9	−1.2	−1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	−0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	−0.1	−0.2	−0.4	−0.6	−0.9	−1.2	−1.5	−1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	196	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	−0.1	−0.3	−0.6	−0.9	−1.3	−1.7	−2.2

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.*

2–32

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Figure 10 – Precast Plank

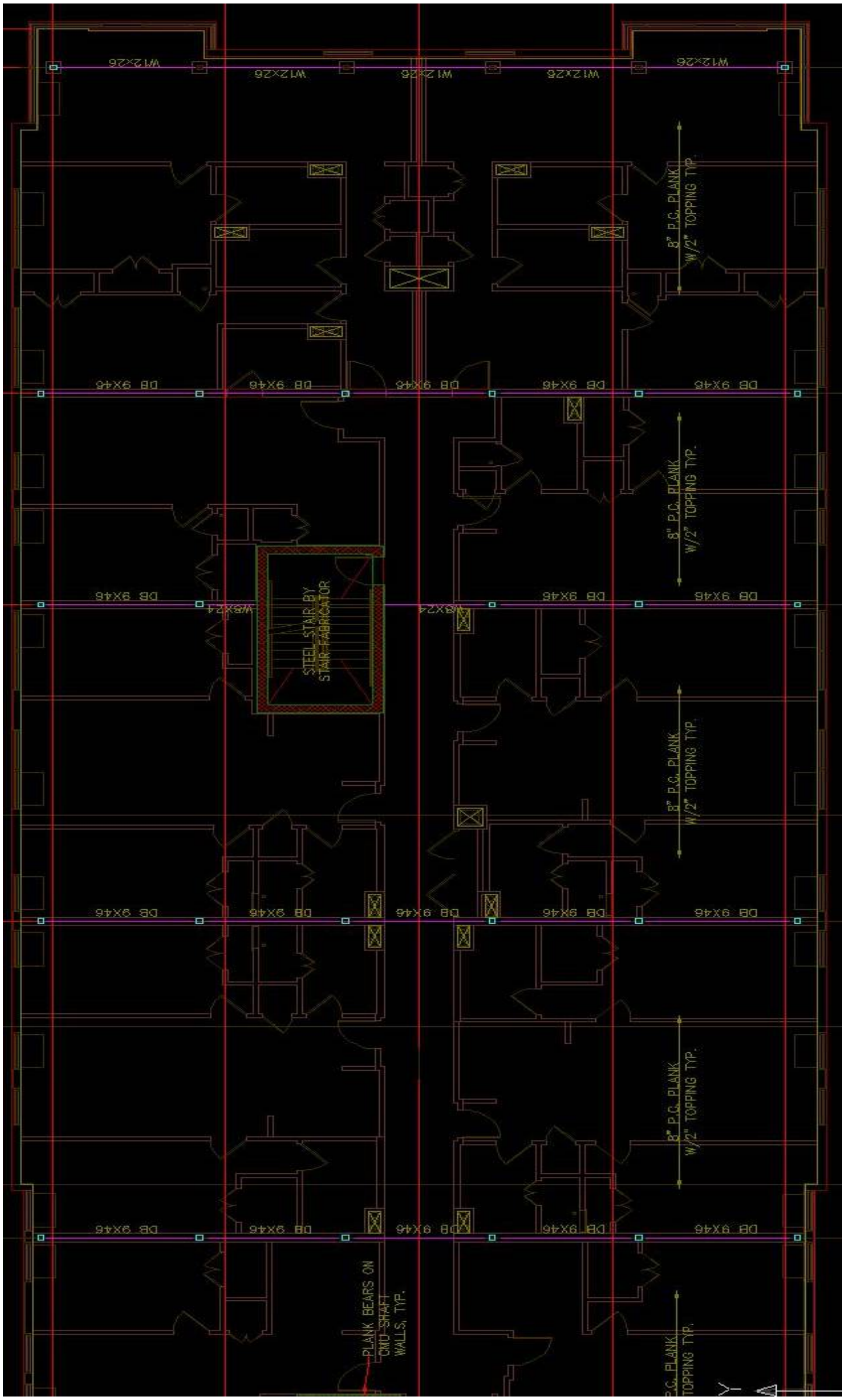
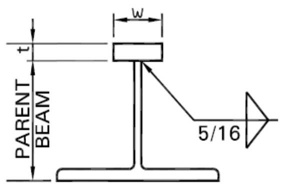


Figure 11 – Typical Architecture Floor Plan with Structural Framing

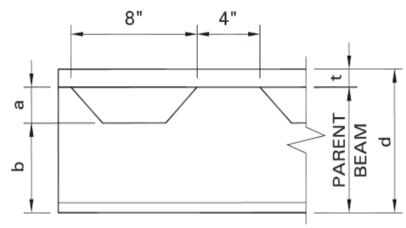
D-Beam® Properties Table



Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in^4	in	in	in^3	in^3	kft	in^4	in	in	in^3	in^3
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

D-Beam® Dimensions Table

Designation	Web Included		Depth	Web	Parent Beam			Top Bar w x t
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	
	lb/ft	in^2	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5



D-Beam® Reference Calculator is Available on Website. www.girder-slab.com

Figure 12 - D-Beam

The D-Beam chosen during the redesign is the DB9x46 (See Figure 12 for properties). The maximum span for this particular beam is 13 feet with the tributary width of 34 feet. (For calculations see Appendix) The D-Beams help to decrease the required depth of the floor system compared to wide flange beam system. The creation process for these d-beams, start with the parent beam, for the case of the D9x46 the parent beam is a W14x61. Next the parent beam is cut in half, during this step the notch the beam, this allows for the grout to pass through the beam and into the planks. Next a steel top bar is welded to the new beam, for this case the size of the steel plate is 3 inches by 1.5 inches

The assembling process for the girder slab system incorporates the use of grout which creates a composite action for the D-Beam and the precast planks increasing the strength which is also an advantage over the traditional load bearing walls with precast planks. The girders at the end of the building on Column lines A & N are the opposite; they are placed below the placed below the precast planks to support them. (See Figure 13A &B for typical floor framing)

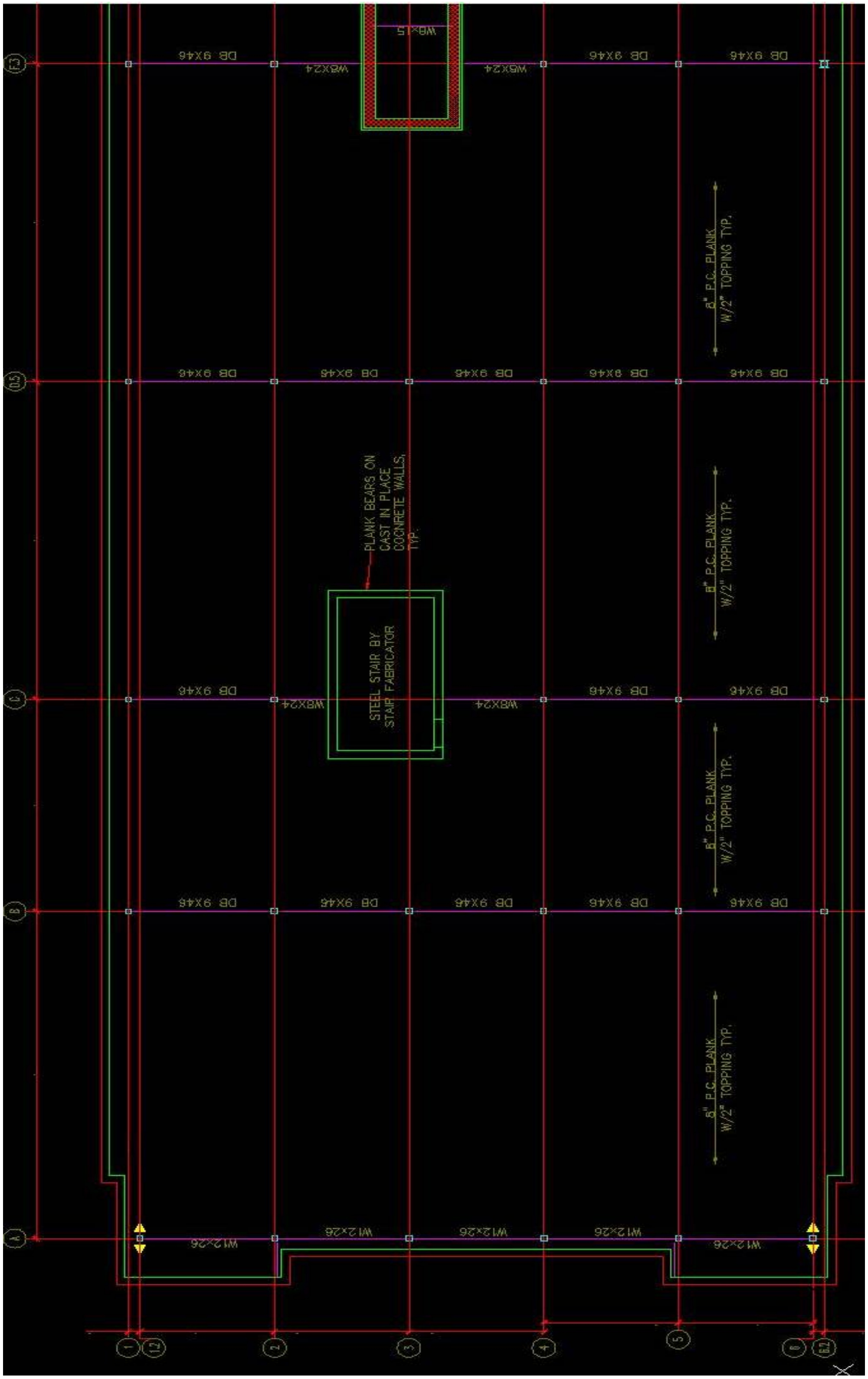


Figure 13A - Typical North Side Structural Floor Framing

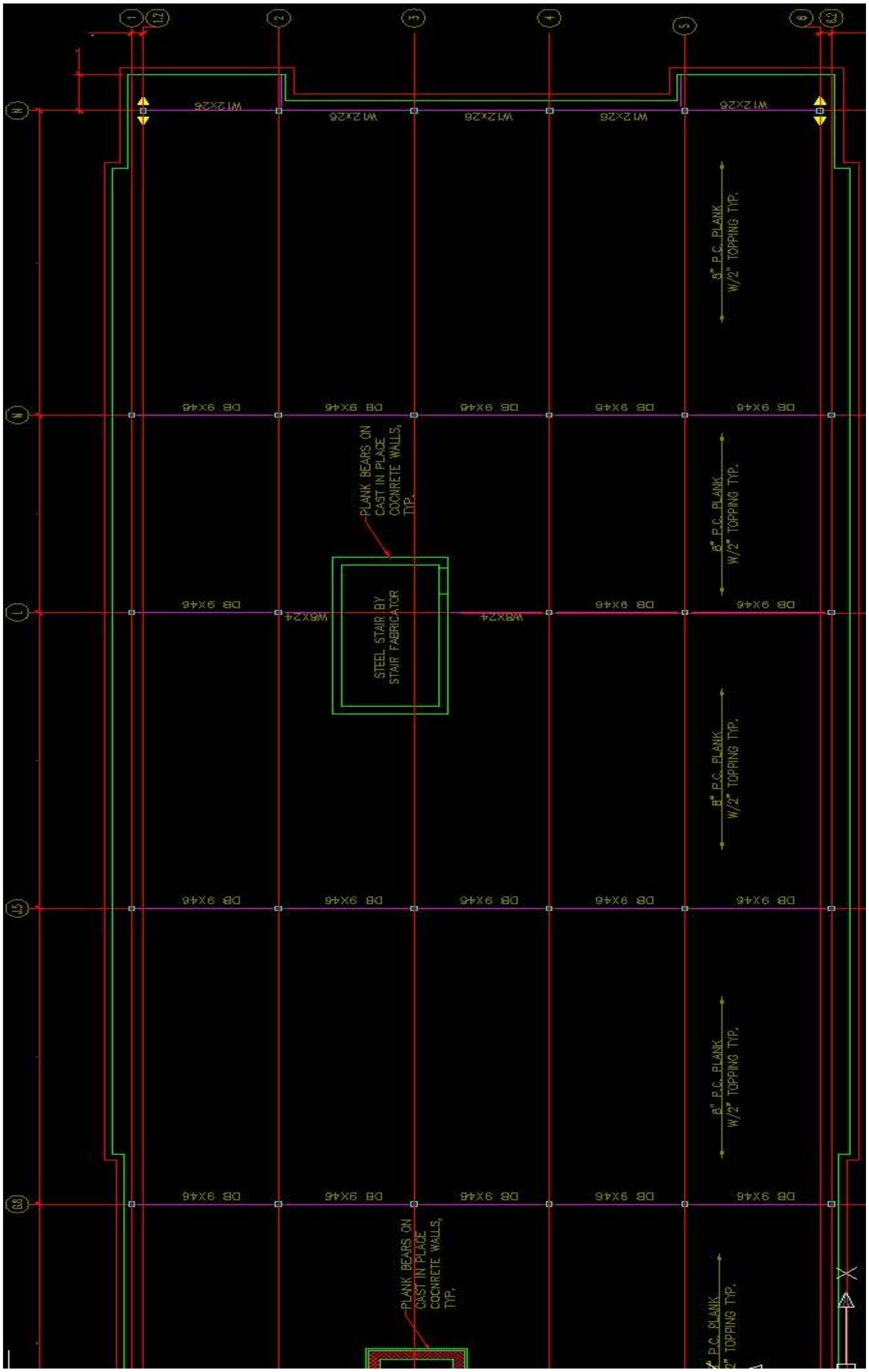


Figure 13B – Typical South Side Structural Floor Framing

The second floor framing remains the same because of trying to keep the same architectural floor plans that are now in the existing system. The major change is the removal of the masonry load bearing walls and now giving the possibility of expanding the retail space into another without having to worry about making changes to the walls. The plans are now more open.

The change from the concrete masonry shear walls to reinforced concrete shear walls did not affect the building structural design. With respect to the lateral system the size of the concrete wall required was only 8” with both horizontal and vertical reinforcement of #5 at 14” on center. The only major change is in the reduction of the width of the walls from 12” to 8”. (For calculations see Appendix) With the use of ETABS model the shear walls were checked and designed, this was done to compare the hand calculations to the computer modeling software. (ETABS design below Figure 14) (See Figure 15 for Lateral System floor plan)

Story	PierLbl	StnLoc	EdgeBar	EndBar	EndSpcng	ReqRatio	CurrRatio
STORY6	P1	Top	#4	#4	12	0.0025	0.0045
		Bot	#4	#4	12	0.0025	0.0045

Story Forces

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY6	P1	QUAKE	Top	0	5.46	0	0	0	-138.518
STORY6	P1	QUAKE	Bottom	0	5.46	0	0	0	72.922
STORY6	P1	WIND	Top	0	-1.12	0	0	0	28.3
STORY6	P1	WIND	Bottom	0	-1.12	0	0	0	-17.271

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY6	Max Drift X	QUAKE	106	3355.68	0	787.002	0.000001	
STORY6	Max Drift Y	QUAKE	107	3355.68	761.88	787.002		0.000008
STORY6	Max Drift X	WIND	106	3355.68	0	787.002	0.000003	
STORY6	Max Drift Y	WIND	107	3355.68	761.88	787.002		0.000001

Figure 14 – ETABS Design

With respect to the foundation system only one row of columns would be possible to have unrestricted foundation construction, which would allow for the use of piles without needing to reinforce the tunnel. To compare prices for the pile using HP 10x42 the cost per linear foot is $28 and the price for a drilled pier of 10” diameter is $18 per foot. The drilled pier for this project was the most economical and most efficient way. This type of construction of using the drilled pier creates the least amount of disturbance to the tunnel below and eliminates the need for reinforcing the tunnel, which would have greatly increased the cost of the project.

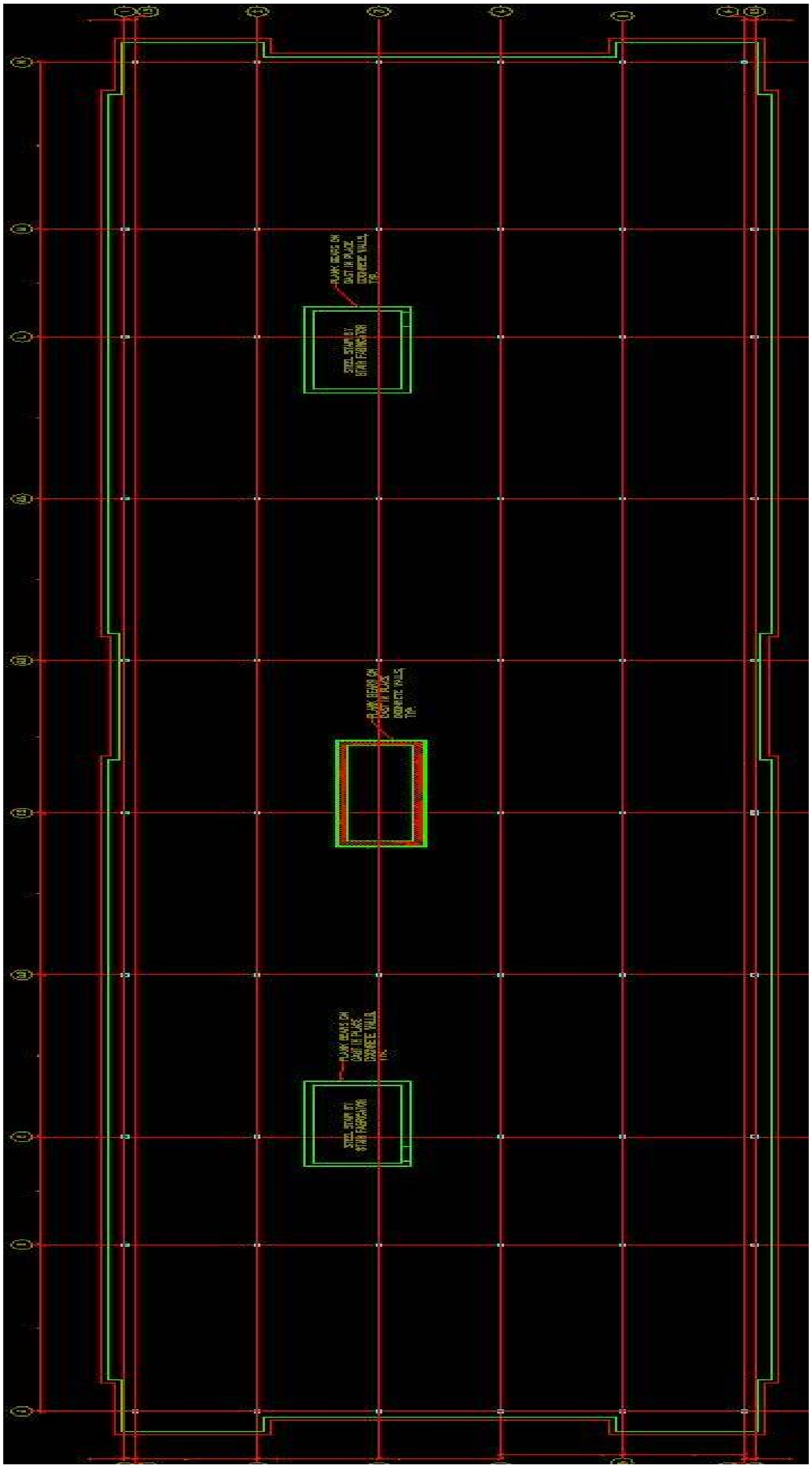


Figure 15 - Lateral System Floor Plan with Columns and Shear Walls

Breadth Study #1: Construction Cost and Schedule Analysis

The goal of this breadth study was to compare the existing structure of Pearl Condominiums to the redesign Flex Frame System. The analysis consisted of the structural system that started at the columns of the ground floor. The decision was made not to make any changes to the foundation system and analyze the impact of the new system on the floors above the ground floor. The use of RS Means Building Construction Data Cost book facilitated the estimation of the cost and time for construction for the new system.

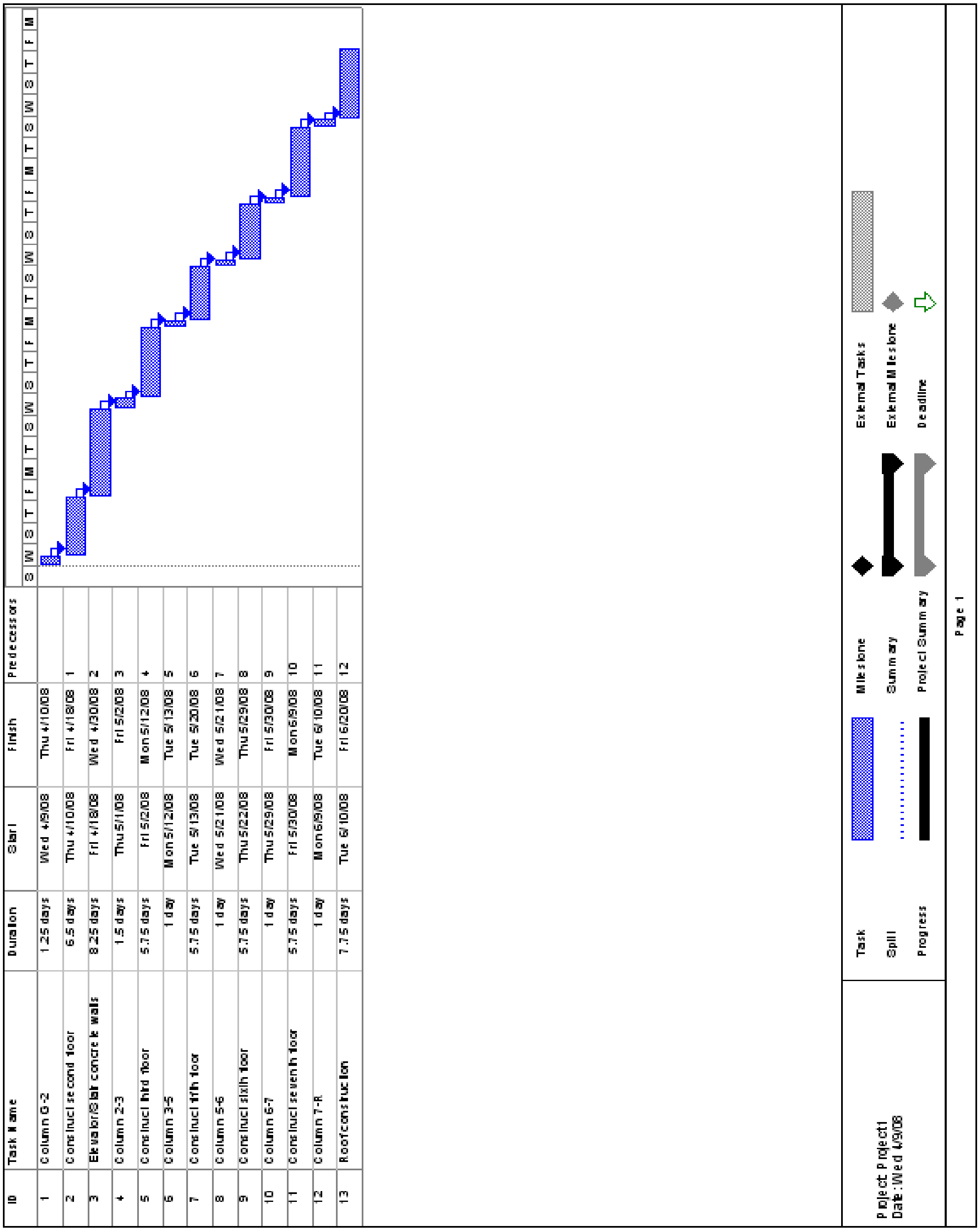
Starting the comparison, the base values of the existing load bearing wall system are 3 months for construction and the cost of construction was $1,754,524. The construction being compared is the precast planks, the load bearing walls that support them, the transfer level on the second floor and the roof framing.

The redesign using the Flex Frame resulted in the time of construction of 2 months and 12 days (See Figure 16 for schedule), along with the cost of construction reacting $1,760,136. The cost per floor and columns is as follows:

Roof – $232,011.37
Column from Roof to Level 7 – $24,510
Floor 7 - $206,167.20
Columns from Level 6 to 7 - $24,510
Floor 6 - $206,167.20
Columns from Level 5 to 6 - $24.510
Floor 5 - $206,167.20
Columns from Level 3 to 5 - $24,510
Floor 3 - $ 206,167.20
Columns from Level 2 to 3 - $43,075
Floor 2 - $327,867.53
Columns Ground to Level 2 - $342,492
Concrete Walls for Elevator and Stair Cores - $191,981

Conclusion:

After comparing the cost and time for construction, the Flex Frame system is a viable system to replace the load bearing wall and precast plank system. The difference in the time of construction and the cost is small enough that either system is a possible solution for this building. With the Flex Frame system the open-web dissymmetric beam has a added cost of $1.25 per pound for the added fabrication required to produce this beam.



ID	Task Name	Duration	Start	Finish	Predecessors
1	Column G-2	1.25 days	Wed 4/9/08	Thu 4/10/08	
2	Construct second floor	6.5 days	Thu 4/10/08	Fri 4/18/08	1
3	Elevator/Stair concrete walls	8.25 days	Fri 4/18/08	Wed 4/30/08	2
4	Column 2-3	1.5 days	Thu 5/1/08	Fri 5/2/08	3
5	Construct third floor	5.75 days	Fri 5/2/08	Mon 5/12/08	4
6	Column 3-5	1 day	Mon 5/12/08	Tue 5/13/08	5
7	Construct fifth floor	5.75 days	Tue 5/13/08	Tue 5/20/08	6
8	Column 5-6	1 day	Wed 5/21/08	Wed 5/21/08	7
9	Construct sixth floor	5.75 days	Thu 5/22/08	Thu 5/29/08	8
10	Column 6-7	1 day	Thu 5/29/08	Fri 5/30/08	9
11	Construct seventh floor	5.75 days	Fri 5/30/08	Mon 6/9/08	10
12	Column 7-R	1 day	Mon 6/9/08	Tue 6/10/08	11
13	Roof construction	7.75 days	Tue 6/10/08	Fri 6/20/08	12

Figure 16 - Schedule for Flex Frame System

Breadth Study #2: LEED Certification Study

The objective for this breadth study is the gaining of a LEED certification for this building, which requires obtaining a minimum of 26 out of a possible 69 points. When this building was in design the idea of LEED was still new, so it was not incorporated into the design. Some of the possible points to gain for certification this project are:

For the Sustainable Site, the highest possible obtainable amount of points that could be awarded would be 9 out of 14. One of the major categories in Sustainable Site reflects on the influence on the use of Alternative Transportation, this is beneficial for the location of the site, which is in Philadelphia. The availability of transportations such as the SEPTA Rails and Buses are close to the location of the site.

For the Material and Resources, the highest obtainable amount of points that could be awarded would be 10 out of 13. One of the major categories in Material and Resources is Waste Management and Recycled Content which reflects on the influence of construction management. This can be obtained by careful planning during the construction process. The Material and Resources category is where for this building can gain the most possible points, which can be accomplished by careful planning of what material to use and the waste management during the construction

There are also ways to gain credits by implementing low-flush toilets, the tenants of the units have control of their thermostat. The only part that is regulated temperature is by the owner are the hallways and the common areas. It is possible for this building to become LEED certified if planning to gain the certification happens during the design process, it is harder to go back in after the building is constructed to achieve a LEED certification, without major financial implications to the cost of the building.

Summary and Conclusion

After completing the analysis of the Flex Frame system, the conclusion that was reached was the Flex Frame is a viable alternative to the use of wall and plank construction. The cost and the time of construction required does not vary for this project therefore that is not a factor in limiting the use of the Flex Frame system. This system's main limitation occurs in the spans of the 8" precast planks because they are the only one the Flex Frame system uses. Overall both systems are very well designed for this type of construction.

Appendix

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Condominium Units w\ Partitions	60
Retail Units (first floor)	100
Stairs	100
Corridor above first floor	80
Corridor at first floor	100
Roof	30

Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Concrete Precast Plank	66
Roof	20

Superimposed Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Roof	20
Condominium Units w\ Partitions	25
Corridor above first floor	25
Corridor at first floor	25
Retail Units	25

Snow Loading	
Item	**Value**
Ground Snow Load (Pg)	25 psf
Exposure Factor	B
Roof Exposure	Fully Exposed
Exposure Factor (Ce)	0.9
Thermal Factor (Ct)	1.0
Occupancy Category	II
Importance Factor (Is)	1.0
Flat-Roof Snow Load Pf = 0.7CeCtIsPg	16 psf

<u>Wind Loads:</u>

North to South Direction

Windward Calculations							
Level	z	Kz	qz	Cp	Ext. Pressure	GCpi	P_{total} ($+GC_{pi}$) (psf)
1	0	0.57	10.05	0.80	6.83	+/- 0.55	-1.89
2	16	0.62	10.93	0.80	7.43	+/- 0.55	-1.29
3	25.92	0.70	12.34	0.80	8.39	+/- 0.55	-0.33
4	35.83	0.76	13.40	0.80	9.11	+/- 0.55	0.38
5	45.75	0.81	14.28	0.80	9.71	+/- 0.55	0.98
6	55.67	0.85	14.98	0.80	10.19	+/- 0.55	1.46
Roof	72.3	0.90	15.86	0.80	10.79	+/- 0.55	2.06

Leeward Calculations						
Level	z	Kz	q_h (psf)	C_p	External Pressure	P_{total} ($+GC_{pi}$) (psf)
1	0	0.57	15.86	-0.20	-2.70	-11.42
2	16	0.62	15.86	-0.20	-2.70	-11.42
3	25.92	0.70	15.86	-0.20	-2.70	-11.42
4	35.83	0.76	15.86	-0.20	-2.70	-11.42
5	45.75	0.81	15.86	-0.20	-2.70	-11.42
6	55.67	0.85	15.86	-0.20	-2.70	-11.42
Roof	72.3	0.90	15.86	-0.20	-2.70	-11.42

Negative Internal Pressure		
q_h (psf)	GC_{pi}	P_{neg} (psf)
15.86	-0.55	-8.7
Positive Internal Pressure		
q_z (psf)	GC_{pi}	P_{pos} (psf)
15.86	0.55	8.7

Total		
Level	P_{total} ($+GC_{pi}$) (psf)	Force at Floor (kips)
1	9.53	0
2	10.13	52
3	11.09	43
4	11.81	45
5	12.40	46
6	12.88	64
Roof	13.48	41

Seismic Loads:

Occupancy Category – II
Importance Factor – 1.0
Seismic Design Category – B
Response Modification Factor – 5.5
(Reinforced Masonry Shear Walls)
Site Class - D
Ss = 0.270g
S1 = 0.060g
F_a = 1.585
F_v = 2.4
S_{DS} = 0.287
S_{D1} = 0.096

Seismic Base Shear:
V = Cs*W

W = 11796 k
Cs = 0.0285
R = 4 (Reinforced Concrete Shear Wall)
V = 336k

Vertical Distribution of Forces:
Fundamental Period:
Ta = 0.496 sec
K = 1.0

Level	wx	hx	wx*hx^k	Cvx	Fx	Mx
2	2171	16.000	34736	0.0817	32.3	516.8
3	2080	25.917	53907.36	0.127	42.7	1106.7
4	2064	35.833	73959.312	0.174	58.5	2096.2
5	2064	45.750	94428	0.222	74	3385.5
6	2115	55.667	117735.705	0.277	93.1	5182.6
Roof	736.26	68.500	50434	0.119	40	2740
				Overturning Moment =		15028

Roof Structural System

- Loads
  - Green Roof
    - extensive type – 25 psf
  - Dead Load – 20 psf
  - Live Load – 30 psf

- Total Load $(25 + 20 + 30) = 75$ psf

Max Beam Span – 35'

Roof Metal Deck

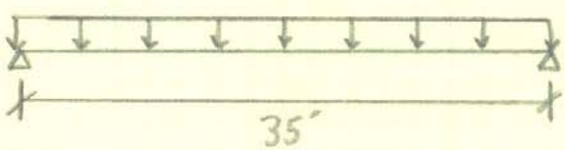
- Type B $1\frac{1}{2}$" – 20 GA
- Double Span Condition
- Span – 6'-6" 81 psf max uniform total load

Worst Case Beam

Span – 35'

Tributary width – 6'-6"

$w = (75\text{ psf})(6.5') = 487.5\text{ plf}$



$$M_a = \frac{0.488\text{ klf}(35)^2}{8} = 74.73\text{ K-ft} \qquad V = 0.488(35/2) = 8.54\text{ K}$$

$$S_{x_{req}} = \frac{(74.73\text{ K-ft})(12\text{ in/ft})}{(0.66)(50\text{ ksi})} = 27.17\text{ in}^3$$

Try 14x61

$$F_{cr} = \frac{1.14(\pi^2)E}{\left(\frac{35(12)}{2.78}\right)^2}\sqrt{1 + 0.078\left(\frac{2.19(1)}{92.1(13.2)}\right)\left(\frac{35(12)}{2.78}\right)^2}$$

$= 29.3$ ksi

Try W12x26

$$M_a = \frac{1.65\,klf\,(11.67)^2}{8} = 28.1\ k\text{-}ft$$

$L_b = 11'\text{-}8" > L_p = 5.33$

$\phi M_n = 1.14[92.8 - 3.61(11.67 - 5.33)]$

$= 79.7\ k\text{-}ft > 28.1\ k\text{-}ft$

Girder by Elevator

span – 8'-8"

tributary width – 35'

$w = 35'(75\,psf + 9.38\,psf) = 2.95\,klf$

$61\,plf/6.5'$

$$M_a = \frac{2.95\,klf\,(8.67')^2}{8} = 27.7\ k\text{-}ft$$

Use W12x26

Level 7 Columns

Worst Case
Tributary Area

$35' \times 12'\text{-}2" = 426\ sq\,ft$

$P_u = 426\,sq\,ft\,(75\,psf + 10\,psf) = 36.21^k$

steel beam/girder

Unbraced Length – 10'

Use HSS 6x6x1/4 for all Columns

Steel Manual Table 4-4

$KL = 10' \quad P_n/\Omega_c = 121^k > 36.21^k$

7th Floor

Loads

- Dead
  - 60 psf - 8" precast plank w/ 1" camber + 2" topping (add 25 psf)
- Live Load
  - 40 psf (typical room)
  - 20 psf (partition Loading)
  - 80 psf (corridors)

Open-Web Dissymmetric beam (D-Beam)

<u>Worst Case</u>

Dead Load - 60 psf

Live Load - 80 psf

Topping Load - 25 psf

DB span - 13 ft

Plank span - 34 ft

Grout f'_c - 4000 psi

Allowable $\Delta_{LL} = L/360$

Allowable $\Delta_{LL} = 0.43 in$

DB Properties

DB 9x46

Steel Section

I_s - 195 in^4

S_t - 33.7 in^3

S_b - 50.8 in^3

M_{scap} - 84.0 ft-k

t_w - 0.375 in

b - 5.75 in

Transformed Section

I_t - 356 in^4

S_t - 68.6 in^3

S_b - 80.6 in^3

Initial Load - Pre Composite

M_{DL}: $(34ft)(0.06\,ksf)(13ft)^2/8$

$= 43.1\,ft\text{-}k < 84.0\,ft\text{-}k$

$\therefore$ O.K.

$$\Delta_{DL} = \frac{5(34ft)(0.06\ ksf)(13ft)^4(1728in^3/ft^3)}{(384)(29000ksi)(195in^4)} = 0.23in$$

Δ Ratio $= L/673$

$13 \times 12 / 0.23 \approx 673$

Total Load - Composite

$M_{sup} = (34ft)(0.08 + 0.025\ ksf)(13ft)^2/8 = 75.4\ ft\text{-}k$

$M_{TL} = 43.1\ ft\text{-}k + 75.4\ ft\text{-}k = 118.5\ ft\text{-}k$

$S_{req} = (118.5ft\text{-}k)(12in/ft)/(0.60)(50ksi) = 47.4in^3 < 68.6in^3$ $\therefore$ O.K.

$$\Delta_{sup} = \frac{5(34ft)(0.08 + 0.025\ ksf)(13ft)^4(1728\ in^3/ft^3)}{384(29000\ ksi)(356\ in^4)}$$

$= 0.22in < 0.43in$ $\therefore$ O.K.

$\Delta_{TOT} = 0.23 + 0.22 = 0.45in$

$= L/344$

Compressive Stress on Concrete

$$N_{value} = \frac{E_{steel}}{E_{concrete}} = \frac{29{,}000\ ksi}{57{,}000(4000psi)^{1/2}} = \frac{29{,}000ksi}{3{,}605ksi} = 8.04$$

$S_{tc} = 8.04(68.6in^3) = 552in^3$

$f_c = (75.4\ k\text{-}ft)(12in/ft)/(552in^3) = 1.64ksi$

$F_c = (0.45)(4\ ksi) = 1.80\ ksi > 1.64\ ksi$ $\therefore$ O.K.

Bottom Flange Tension Stress (Total Load)

$$f_b = \frac{(43.1\ ft\text{-}k)(12in/ft)}{50.8in^3} + \frac{(75.4\ ft\text{-}k)(12in/ft)}{80.6in^3}$$

$= 10.18\ ksi + 11.23\ ksi = 21.4\ ksi$

$F_b = 0.9(50\ ksi) = 45ksi > 21.4ksi$ $\therefore$ O.K.

Check Shear

Total Load = (60 + 80 + 25 psf) = 165 psf

$w = (0.165\ \text{ksf})(34\ \text{ft}) = 5.61\ \text{klf}$

$R = (5.61\ \text{klf})(13\ \text{ft})/2 = 36.5\ \text{k}$

$f_v = (36.5\ \text{k}) / (0.375)(5.75\ \text{in}) = 16.9\ \text{ksi}$

$F_v = 0.4(50\ \text{ksi}) = 20\ \text{ksi} > 16.9\ \text{ksi}$ ∴ O.K.

USE DB 9x46
Same as on
Level 2, 3, 5, 6
as on 7

Worst Case Girder By Elevator

Span - 8'-8"

Tributary width - 35'

$w = 35'(60\ \text{psf} + 80\ \text{psf}) = 4.9\ \text{klf}$

$M_u = \dfrac{4.9(8.67')^2}{8} = 46\ \text{k-ft}$ Try W8x24

Same on Levels 2, 3, 5, 6
as on 7

$L_b = 8.67 > L_p = 5.69'$

$\phi M_n = 1.14[57.6 - 1.59(8.67 - 5.69)] = 60.3\ \text{k-ft} > 46\ \text{k-ft}$

Use W8x24

Girders on exterior

span - 11'-8"

Tributary width - 19.5'

$w - 19.5'(60\ \text{psf} + (40\ \text{psf} + 20\ \text{psf})) = 2.34\ \text{klf}$

$M_u = \dfrac{(2.34\ \text{klf})(11.67)^2}{8} = 39.8\ \text{k-ft}$ Try W12x26

$L_B = 11.67' > L_p = 5.33'$

$M_n = 1.14[92.8\ \ 3.61(11.67 - 5.33)]$
$= 79.7\ \text{ft-k} > 39.8\ \text{k-ft}$

USE W12x26

Same on levels
2, 3, 5, 6 as on 7

Columns

Level 6 - Worst Case

Square Feet - [35' x 12'-2"] = 426 sq ft

P_u = 426 (85 psf) + 426 (60 psf + 80 psf) = 95.85^k

roof Level 7

unbraced length - 10'

Level 6

End Columns

Square Feet - [35'/2 x 11'-8"] = 204.2 sq ft

P_u = 204.2 (85 psf) + 204.2 (60 psf + 60 psf) = 41.9^k

unbraced Length - 10'

USE HSS 6 x 6 x 1/4

P_n/Ω_c @ 10' = 121^k Table 4-4 Steel Manual

121^k > 95.85^k

Columns

Level 5 - 6

Square Feet - 426 sq ft

P_u = 426 (85 + 2(60 + 80)) = 156^k < 173^k

unbraced Length - 10' USE HSS 6x6x3/8

End Columns Line A & N

P_u = 204.2 (85 + 2 (120)) = 66.4^k < 121

Unbraced Length - 10' USE HSS 6x6x1/4

Columns

Level 3-5

$P_u = 426(85 + 3(140)) = 216^k < 221$

Unbraced Length - 10' USE HSS 6x6x 1/2

Columns Line A & N

$P_u = 204.2(85 + 3(120)) = 91^k < 148^k$

Unbraced Length - 10' USE HSS 6x6x 5/16

Columns

Level 2-3

$P_u = 426(85 + 4(140)) = 275^k < 334^k$

unbraced Length = 10' USE HSS 8x8x 1/2

Line A & N

$P_u = 204.2(85 + 4(120)) = 115^k < 148^k$

unbraced Length = 10' USE HSS 6x6x 5/16

Columns

Ground - Level 2

$P_u = 426(85 + 5(140)) = 334^k < 401^k$

Unbraced length 16' USE HSS 10x10x 1/2"

$P_u = 204.2(85 + 5(120)) = 140^k < 163^k$

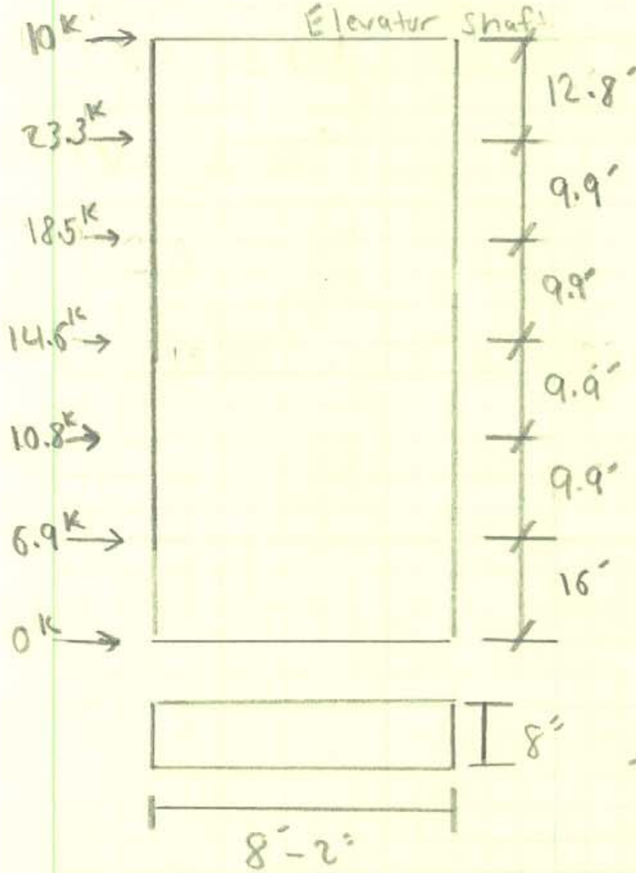
USE HSS 6x6x 1/2

Concrete Shear Wall

Story 5

East to West

Seismic Controls



$M_u = 10^K(68.5) + 23.3^K(55.67) + 18.5^K(45.75) + 14.6^K(35.83) + 10.80(25.9) + 6.9(16) = 3742$ k-ft

$V_u = 84.1^K$

$P_u = (8'-2")(17')(55psf + 5(60psf)) = 49.3^K$

Boundary Check

$A_g = (8'-2")(8") : 5.4 ft^2$

$I_g = \frac{(8/12)(8.167)^3}{12} = 30.3 ft^4$

$f_c : \frac{49.3}{5.4} + \frac{3742 + (\frac{8.167}{2})}{30.3} =$

$= 9.13 + 123.63 = 132.76$ Ksf

$= 132.76$ Ksf $= 0.92$ Ksi

$0.2(f'_c) : 0.2(5ksi) = 1$

$f_c = 0.92 < 1 \therefore$ No Boundary Element required

$f'_c = 5000 psi$

$C_v = \frac{49.3}{2} + \frac{3742}{8'-2"} = 482.8\ P_{uBE}$

Long + Transverse Reinforcement

$2 A_{cv} \sqrt{f'c} =$

$2(8 \times 12 \times 8.167')\sqrt{5000}/1000 = 111^K > 84.1^K \therefore$ Need one Curtain

$A_{cv} = 12" \times 8" = 96 in^2/ft$

$A_{s_{req}} = (0.0025)(96) = 0.24 in^2/ft$

$$\frac{0.24}{12} = \frac{0.31}{S}$$

$S = 15.5''$

$S_{req} = 14'' > 12'' \therefore$ o.k.

check shear

$$V_n = A_{cv}(\alpha_c \sqrt{f'c} + \rho_t f_y)$$

$$\frac{h_w}{l_w} = \frac{68.5}{8.167} = 12 > 2 \therefore \alpha_c = 2.0$$

$$A_{cv} = (8)(8'-2'')(12) = 784 in^2$$

$$\rho_t = \frac{0.31}{(12)(8)} = 0.0032$$

$$V_n = 784(2\sqrt{5000} + 0.0032(60{,}000))/1000 = 261.4^k$$

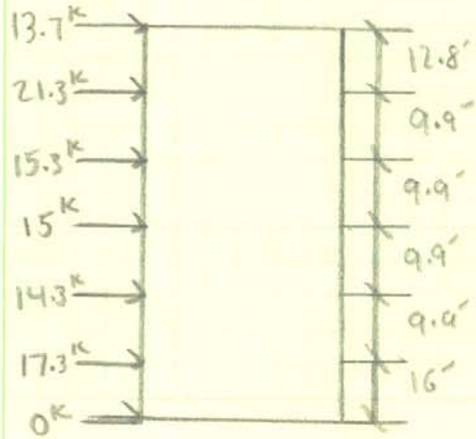
$$\phi V_n = 261.4(0.6) = 156.8 > 84.1^k \therefore 0.k.$$

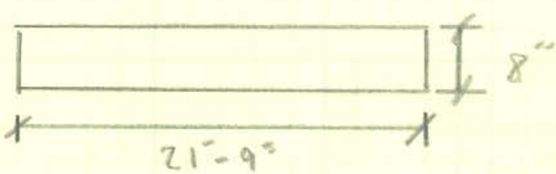
Use 8'' concrete shear wall with

#5 @ 14'' each way, $f'c = 5000$ psi

North to South

Wind Controls





$$M_u = 13.7^k(68.5) + 21.3^k(55.67) + 15.3^k(45.75) + 15(35.83) + 14.3(25.9) + 17.3(16)$$

$$= 4009 \text{ K-ft}$$

$V_u = 97^k$

$P_u = (63.75 \text{ sq ft})(55 \text{ psf} + 5(60 \text{ psf})) = 22.6^k$

Boundary Check

$C_v = \frac{22.6}{2} + \frac{4009}{21.75} = 195.6$ P_{uBE}

$A_g = (21''-9'')(8'') = 14.5 \text{ ft}^2$

$I_g = \frac{(8/12)(21.75')^3}{12} = 571.6 \text{ ft}^4$

$f_c = \frac{22.6}{14.5} + \frac{4009 + \frac{21.75}{2}}{571.6} =$

$1.56 + 7.03 = 7.59 \text{ ksf} = 0.05 \text{ ksi} < 1.0$ for 5 ksi

$\therefore$ No Boundary Element required

Long + Transverse Reinforcement

$2(8)(21.75 \times 12)\sqrt{5000}/1000 = 295^k > 97^k$

$\therefore$ Need One Curtain

$A_{cv} = (12'')(8'') = 96''$

$A_{s\,req} = (0.0025)(96'') = 0.24 \text{ in}^2/\text{ft}$

$$\frac{0.24 \text{ in}^2/\text{ft}}{12''} = \frac{0.31 \text{ in}^2/\text{ft}}{S}$$

$S = 15.5 \text{ in} \approx 14'' > 12''$

Check Shear

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{\ell_w} = \frac{68.5}{21.75} = 3.15 > 2$ $\therefore \alpha_c = 2.0$

$V_n = 2088(2\sqrt{5000} + 0.003(60000))/1000 = 671^k$

$A_{cv} = (8)(21.75 \times 12) = 2088 \text{ in}^2$

$\phi V_n = 671^k(0.6) = 403^k > 84.1^k$

$\rho_t = \frac{0.31}{(12)(8)} = 0.003$