Aric Heffelfinger
Fordham Place
Bronx, NY
Structural Option
Advisor - Dr. Hanagan

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Fordham Place

Project Overview

- Size: 174060 Ft^2
- 15 Stories
- Cost: $34.8 Million
- Owner/CM: Acadia Realty
- Architect: Greenberg Farrow
- Structural Engineer: M.G. McLaren
- Geotechnical Engineer: Soil Mechanics Drilling Corp.
- Surveyor: Control Point Associates

Mechanical Systems

- *Commercial*
    - 450 ton chiller located on roof
    - Space heating delivered from central boiler
- *Residential*
    - 450 ton water cooling tower located on the roof top
    - Space heating delivered from central boiler

Structural System

- ***Existing Building***
    - Concrete encased 14" Steel Columns
    - 18" Steel beams and double girders
    - 26'x22' typical bays
    - 2'x4' - 4'x8' exterior brick piers
    - 10'x10' step footings ~ 7' deep
- ***New Tower***
    - Steel beams/girders, composite metal deck with 6¼" concrete slab
    - 14" Steel Columns
    - Braced and Moment Frames for Lateral Resisting System
    - 150 Ton piles ~ 45' deep

Lighting/Electrical Systems

- 1200 Amp, 120/208V, 3 Phase 4-wire
- 12T5 Fluorescent Lighting
- Halogen lighting in retail area
- Special metal halide lighting for showcase

Name: Aric Heffelfinger
Option: Structural
http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/abh144/

Building: Fordham Place
Location: 400 East Fordham Road
Bronx, NY 10458

TABLE OF CONTENTS

Executive Summary

This report contains the results of the design and analysis of two different floor systems and Fordham Place, which is located in Bronx, NY. The two different floor systems that will be evaluated are a two way flat slab with drop panels, and the original design of a composite steel structure. All load cases involving dead, live, roof live, snow, wind, and seismic were evaluated.

Fordham Place is a 15 story office / retail / residential building comprised of a steel columns and beams that acts compositely with a concrete slab. Chevron style braced frames are the lateral force resisting system. This type of frame is very efficient because the only lateral drift is due to axial deformation of the cross members and columns. The location of the frames is so that there are minimal lateral forces induced in the building do to torsion.

By redesigning Fordham Place as an all concrete structure, adjustments in the lateral system, HVAC systems, construction schedule, and cost were re-examined. In this report you will see the lateral system changed to reinforced flanged concrete shear walls. While a single HVAC system will serve only one floor in an attempt to reduced large openings in the floor slab. Construction schedule and cost both increased with the change to an all concrete structure. After designing Fordham Place as an all concrete structure, it is very clear the original design is a better choice.

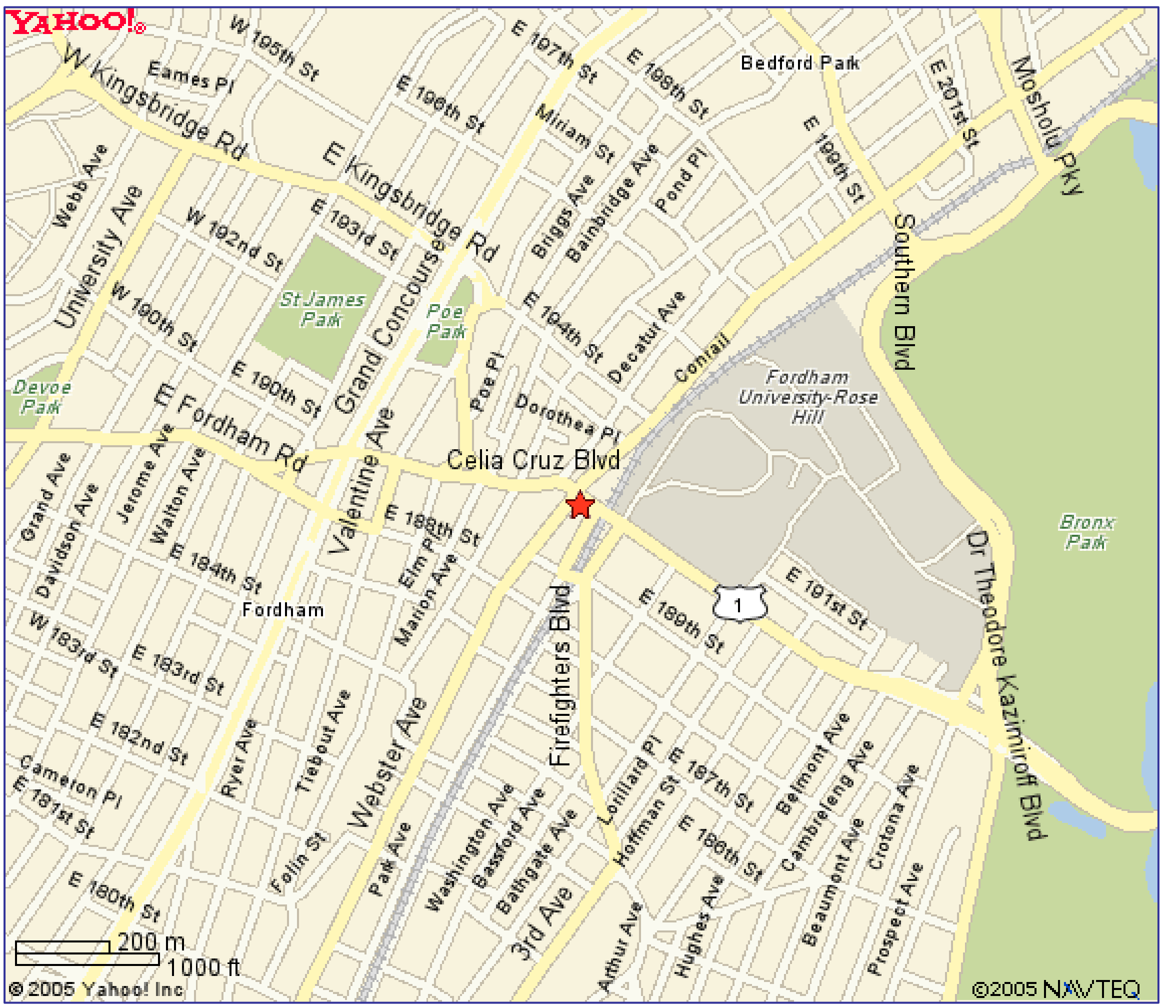
This report is solely used for educational purposes only, and should not be treated as a professional design. The purposed of this case study was to examine different structural systems and what effects they had on the rest of the building. If there are any question on this report, feel free to contact me at abh144@psu.edu

Introduction

Location of Site

400 East Fordham Road

Bronx, NY 10458



Project Team

Owner – Acadia Realty

CM – Acadia Realty

Architect – Greenberg Farrow

Structural Engineer – M.G. McLaren

Mechanical Engineer – Greenberg Farrow

Geotechnical Engineer – Soil Mechanics Drilling Corp.

Surveyor – Control Point Associates

Construction Information

Fordham Place is a $34.8 million design build project that was expected to break ground in the summer of 2006. However, due to a dramatic increase in steel, Acadia Realty, the owner decided to hold off on the construction of Fordham Place. There are now considerations of erecting only the first six stories until the cost of construction decreases. Currently, Acadia Realty's goal is to have at least the first six stories constructed by October. At which point they will review their funds and decide if it is feasible to construct the building in its entirety.

Building Function

Of the 15 stories that Fordham Place holds, the bottom two floors will be occupied by the retail industry. While the next six will be commercial offices and the remaining 7 will be residential condos. However parking will be and issue for these tenants since Fordham Place itself does not contain any parking.

Building Codes

New York City Building Code

Zoning

The area in the Bronx where Fordham Place is planned to be erected is zoned for both commercial and residential. If you click on the following link, it will direct you to a NYC zoning map of the Bronx.

http://www.nyc.gov/html/dcp/html/zone/bx_zonedex.shtml

Existing Building

Mechanical

The Buildings HVAC system basically consists of a Water Cooled Air Conditioning system in combination with a gas fired Central Boiler Plant. As part of the Core and Shell, a cooling tower mounted on the Building's roof will provide condenser water (supply and return) via common pumps and piping, to adequately size valved taps terminating within each of the tenant spaces. Space heating will be accomplished via a gas fired Modular Central Boiler Plant (located in a mechanical room in the cellar level) which will deliver hot water to the building via common and insulated hot water heating risers, where similarly to the condenser water, adequately sized valved taps terminating at each of the tenant spaces will be provided under the core and shell work.

Electrical

Retail Tenants - Each retail tenant will be provided with a dedicated and separately (direct to utility co) metered electric service feeder emanating from the building's main electric service room. Tenant's service feeder will terminate at a pull box within the tenants space.

Office Tenants - Each office floor will be provided with a separate feeder and floor panels sized to handle an above average office use type space complete with breakers and/or switches for future connection of both lighting and power loads. The floor electrical loads (including HVAC units) will be provided with electronic sub-metering furnished by the LL at tenant's expense for reading energy consumption.

Plumbing

The building will be provided with a few sanitary risers/stacks (with vents) complete with capped outlets at each retail tenant space and at each of the office floors. Domestic cold water to the building will be delivered from a master metered service to the various floors of the building via a common insulated riser. Separate valved outlets terminating at each of the retail tenant spaces will be provided under the core and shell work.

Fire Protection

Building will be provided with a fully automatic sprinkler system in accordance with Code requirements and standard occupancy uses. System coverage will consist of a riser, loop and minimum grid with upright heads. An automatic fire pump will be required and provided under the Core and Shell work. A

wet fire-standpipe system complete with hose rack stations, risers, fittings and devices will be required under the building core and shell work.

Transportation

Building will be equipped with two escalators, 5 passenger elevators, 1 freight elevator, and 4 stair towers. Both escalators will be side by side (one going up and one coming back down), located at the northwestern corner of the existing building, and serving transportation from the concourse floor to the ground floor and from the ground floor to the second floor. Both escalators are almost 3 feet wide and travel at a speed of 100 feet per minute. Of the five passenger elevators, one is located in the existing part of the building while the other four are clustered together serving all 15 floors of the tower part of the building. All five passenger elevators have at least a 3500 lb capacity while traveling at speed of 400 feet per minute. The Freight elevator is located at west side entrance of the existing building and has a capacity of 5000 lb traveling at speeds of 200 feet per minute. The 4 stair towers are strategically located to comply with code. Two of which serve the existing building while the other two stretch from the concourse floor the rooftop of the impressive 15 story tower.

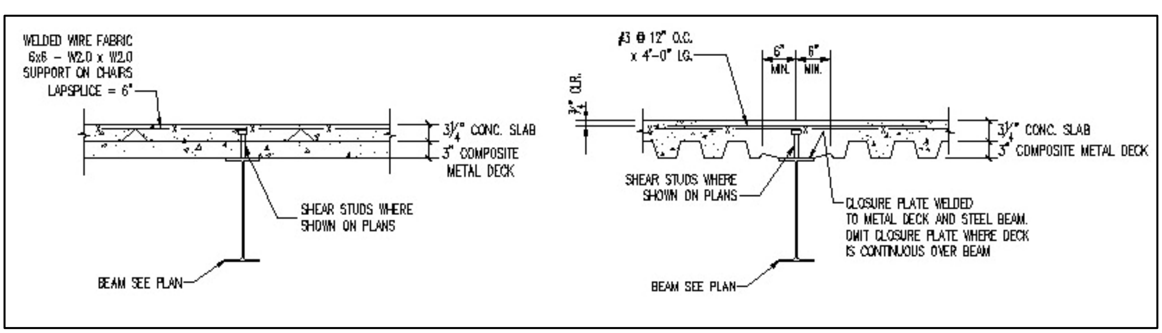
Telecommunication:

Fordham Place features a hi-tech, state of the art security system which consists of personal security at the lobby entrance with additional key cards for access of the building, and key card access of the elevators also. The reception desk will have computers with flat screens, telephones, and a concealed fire command station for security purposes. Pictures of visitors will also be taken upon entry of the building. Each tenant will be given an ample amount of roof space for use of satellite dishes, antennas, etc.

Structural

Floor System

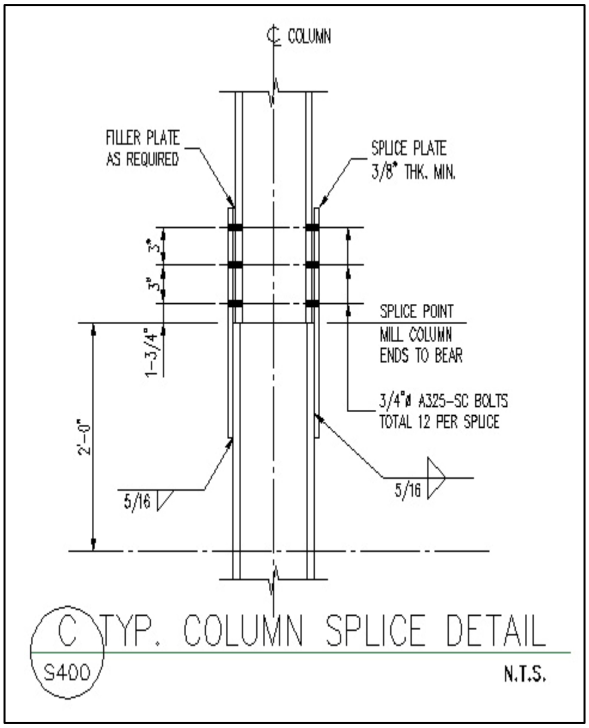
The floor system of Fordham Place consists of structural steel W sections that support metal deck and concrete slab. The W shape beams and girders are A992 grade 50 and support a light weight concrete (115pcf) slab of 6.25 in. The concrete's compressive strength is $f'_c = 3000$psi for all floors. Reinforcing of concrete is done with high strength billet deformed steel bars



with fy = 60,000psi as a minimum. All floor deck is 20 gage 3" deep galvanized composite deck and is continuous over 2 spans at the joints of the deck. All shear studs are headed studs of grade 1015 or 1020 cold finish carbon steel. Studs, at a maximum are spaced every 12".

Columns

Columns consist of rolled structural W14 shapes grade 50. However there are a few W10x39's that extend from the 14th floor to the roof at selected areas. Columns extend from the concourse floor to just above the second floor, extending 3 floors or 36'. From the second floor up to the roof, columns are spliced at every two floors or 27'. Column Splices consist of 2 – 3/8" plates applied to the flanges of the columns being spliced. The plates are then connected to the bottom column with a 5/16" fillet weld all around the plate. The top column is then connected to the splice plate with 12 - 3/4" Ø A325 S.C. bolts.



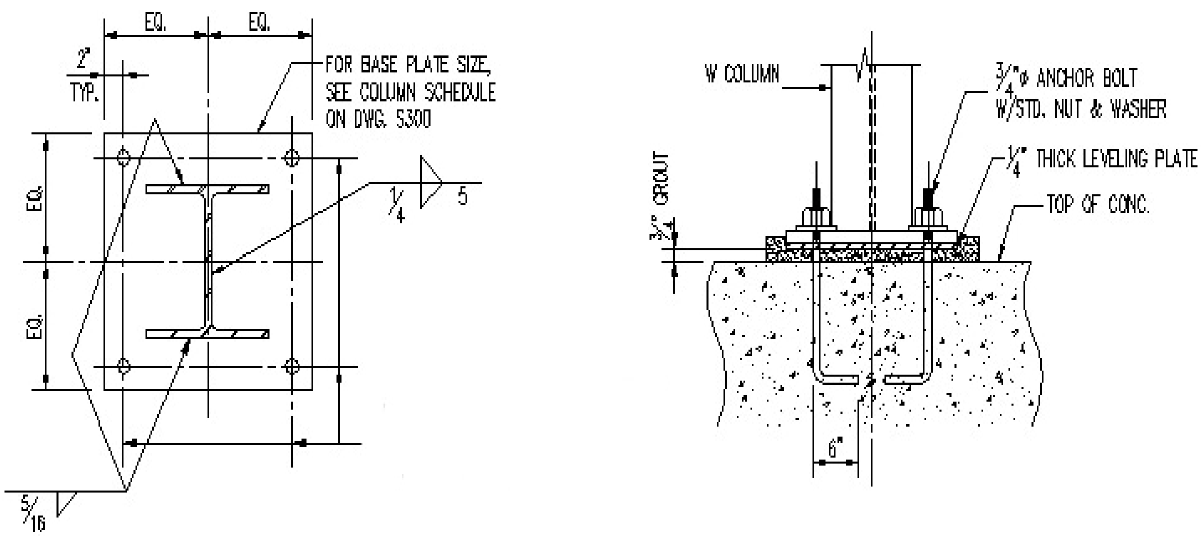
Roof

The roof consists of rolled structural steel W shapes supporting roof deck and a lightweight concrete slab. Structural steel members are grade 50 W16 shapes and typically span approximately 27' with spacing of 9'. Roof deck is 20 gage, 3" deep galvanized wide rib type NI and is continuous over 2 spans at the joints of the deck. The roof deck will span from beam to beam, 9ft., and the short direction of a typical roof bay. The roof deck will be connected to the structural steel with 5/8" puddle weld in a 12-6-12 in pattern. Compressive strength of concrete on the roof is f'c = 3500psi at a minimum. The top of the concrete slab is 3 ¼"above top of slab, totaling to a 6 ¼"concrete slab.

Foundations

The foundation system of Fordham Place is composed of 150 ton steel piles that extend approximately 45 – 50ft deep into bedrock. The piles are A992 grade 50 rolled W shapes and are capped with concrete caps that have a compressive strength of f'c = 3000psi. The pile caps will range in size depending on the number of piles it needs to contain, which is dependent on the load a given column transfers. The number of piles per pile cap

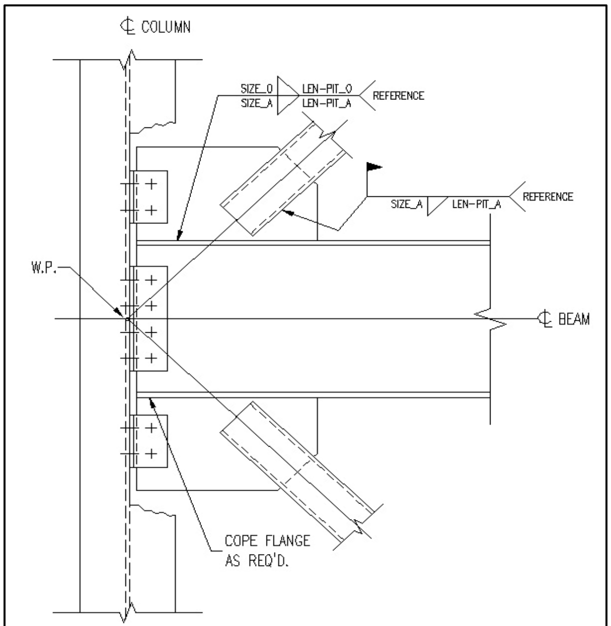
ranges from 4 (PC-4) to 13 (PC-13). Load is transferred from the columns to the pile caps via A36 1/4” steel base plates. The base plate is welded to the column using a 5/16” fillet weld on the exterior of the flanges and a ¼” fillet weld on the web and interior of the flanges. The base plate is connected to the pile cap with 4 - 3/4” Ø anchor bolts extending 12 inches into the pile cap before turning 180 degrees and extending 6 more inches. Flush with the pile cap will be a slab on grade with a compressive strength f’c = 4000psi.



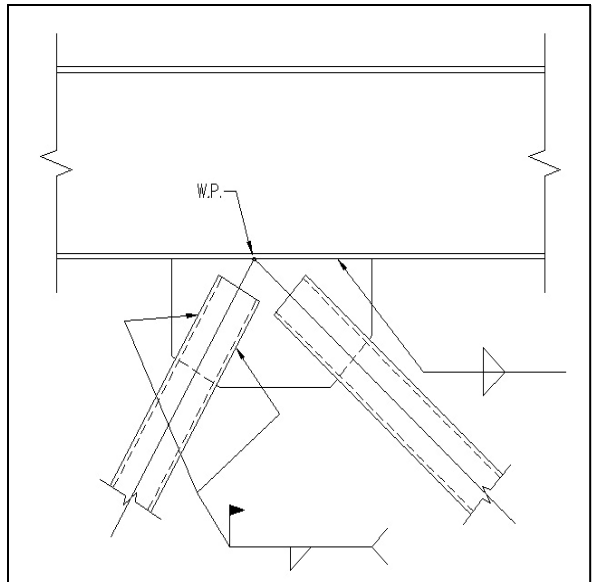
Base Plate Details

<u>Connections</u>

Throughout Fordham Place, there are many different connections, of which I have already talked about two; base plates and column splices. Other connections to consider are shear, moment, bracing connections to both columns and beams. Typical shear connections consist of double angles with the required number of A325 3/4"Ø S.C. bolts. Moment connections will be the same as a typical shear connection but will also have the top and bottom flanges of the beam welded with a 5/16" full penetration field weld. Bracing connections from the braced frames will be to beams and columns at different elevations of the building (See pictures below). Bracing to a column connections will compose of a gusset plate being welded to the underside of a beam and bolted to the column. Bracing members will be bolted to the gusset plate. Bracing to beam connections will occur at the midspan of the beam and will consist of a gusset plate welded to the underside of the beam. Bracing members will then be bolted to the gusset plate.



BRACING CONN. TO COL.



BRACING CONN. TO BEAM

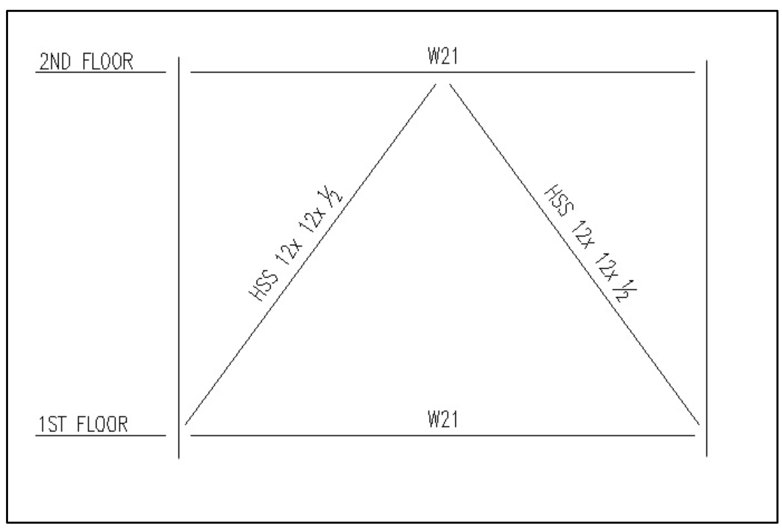
Enclosure

The building enclosure at Fordham Place consists of many different types. For the existing building, you will notice an older light brown brick wall with granite piers running the height of the building to interrupt the brick. At the base there currently is steel covering windows. But soon, when Fordham Place is finished with construction, it will return to display windows for retail stores. Playing off the older style building the existing structure brings, the new tower will match the light

brown brick in the façade. The façade will also have sunlight gleaming off the many blue tinted glass panes. Finally, on the lower 2 floors facing Fordham Road, the building will have a glass façade enclosing a two story lobby area.

Lateral System

The lateral system is composed of moment connections and braced frames. Moment connections are mostly located along the plane in which the existing building and new tower are connected. This is done so that each building can act independent of each other. The braced frames are "K" type braces utilizing A500 grade B HSS12x12x1/2" structural steel members. They are located in six different bents, all of which are centrally located near the core of the building and extend from the concourse floor to the roof. The bracing is located near the core of the building in order to avoid inducing any internal torsion. As discussed in the connections



part of this report, there is bracing connections to beams and columns. On each side of the bent, a bracing member will be framed from the bottom corner of the bent (column connection) to the midspan of the upper beam (beam connection). See picture to right.

Structural Design Code

The 2003 Building Code of New York City

Structural Design Specifications and Standards

Structural Concrete Design – American Concrete Institute, Building Code Requirements for Structural Concrete, ACI 318-02

Structural Steel Design – American Institute of Steel Construction, Steel Construction Manual, Allowable Stress Design Ninth Addition

Welding - American Welding Society, Structural Welding Code - Reinforcing Steel, AWS D1.4-79

Steel Deck - Design Manual for Floor Decks and Roof Decks, SDI

Masonry – American Concrete Institute, Specifications for masonry Structures, ACI 530.1

Project Material Strength

Concrete (28 day minimum compressive strength)

Footings: 3000psi

Slab on Grade: 4000psi

Piers: 4000psi

Footings: 4000psi

Steel Deck Slabs (lightweight): 3500psi

Lightweight Concrete: 115pcf

Normal weight Concrete: 145pcf

Steel Reinforcement

Reinforcing Bars – ASTM A615 or A706 Grade 60
(Fy = 60,000psi min)

Welded Wire Fabric – ASTM 185

Metal Deck

Roof Deck: ASTM A653, Grade 33

Floor Deck: ASTM A661, Grade C, D or E.

Structural Steel members

Columns, Beams, Girders: ASTM A992 or ASTM A572, Grade 50.

Structural Steel Plates and miscellaneous steel: ASTM A36

Cold-Formed Steel Tubing: ASTM A500, Grade B.

Structural Steel Pipe: ASTM A53 or A500, Type E or S, Grade B.

Connectors

Headed shear stud: ASTM A108, Grade 1015 or 1020

Anchor Rods: ASTM F1554 Grade 36,

Bolts: ASTM A325

Welding

All Welds: AWS E70XX Electrodes, minimum tensile strength = 70,000psi

Masonry

Concrete Masonry Units: ASTM C90, f'c = 3750psi

Grout: ASTM C476 f'c = 2500psi

Design Gravity Loads (ASCE 7-02)

Load Type	Existing Retail	Stairs	New Building Retail	Existing Building Community Areas
Dead Load	122	50	60	122
Superimposed Dead Load	20	-	30	20
Live Load	100	100	100/75	50
Truck Load	-	-	250	-

Load Type	New Building Community Areas	Existing Roof	New Roof	Penthouse
Dead Load	60	117	60	20
Superimposed Dead Load	30	10	20	60
Live Load	80	30	30	30
Truck Load	-	-	-	-

Table: Designer's gravity loads.

*Note: See PDF on next page for my gravity loads

GRAVITY

SNOW

TERRAIN CAT "B"
FULLY EXPOSED
$C_e = 0.9$
$C_t = 1.0$
$I = 1.0$
$p_g = 30$ psf
FLAT ROOF

$p_f = 0.7 C_e C_t I p_g$
$= 0.7(0.9)(1.0)(1.0)(30\text{psf})$

$p_f = 18.9$ psf

LIVE

LOBBY = 100 PSF
CORRIDOORS = 100 PSF (80 PSF ABOVE FIRST FLOOR)
OFFICES = 50 PSF (USED 60PSF BASED ON CLIENTS REQUEST)
RETAIL =
FIRST FLOOR = 100 PSF
UPPER FLOORS = 75 PSF

ROOF LIVE LOAD

$L_r = 20 R_1 R_2$ $\quad$ $R_2 = 1$ FLAT ROOF

$A_T = 258 \text{ ft}^2$

$R_1 = 1.2 - 0.001(258)$
$R_1 = 0.94$

$L_r = 20(0.94)(1.0)$

$L_r = 18.8$ PSF

DEAD LOAD

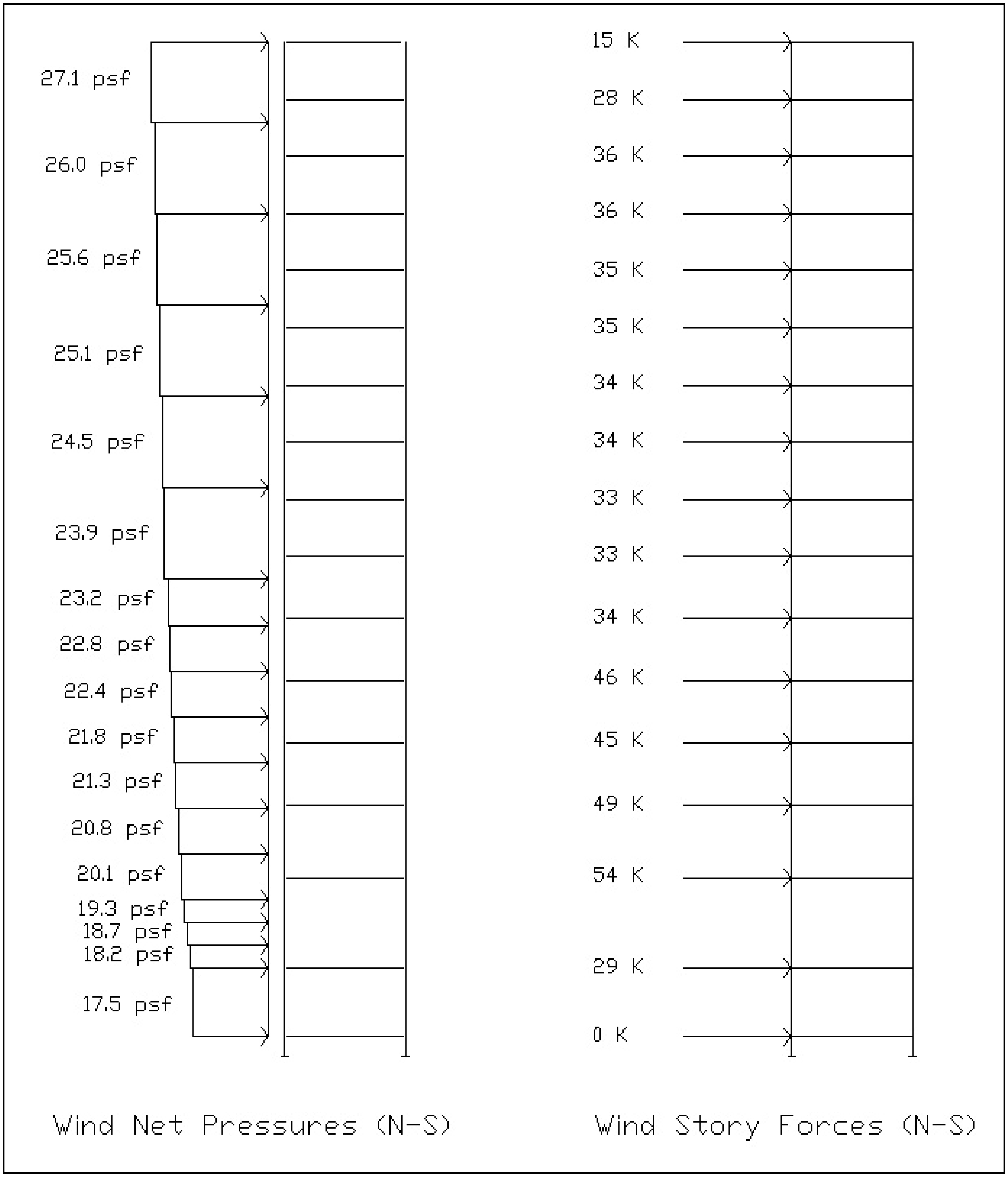
FOUND FOR EACH INDIVIDUAL CASE

Wind Loads

Height	K_z	q_h	q_v	$p_{leeward}$	$p_{windward}$	p_{net}
0-15	0.57	25.4592	12.4032	-9.8527104	7.680061	17.53277184
15-20	0.62	25.4592	13.4912	-9.8527104	8.353751	18.20646144
20-25	0.66	25.4592	14.3616	-9.8527104	8.892703	18.74541312
25-30	0.7	25.4592	15.232	-9.8527104	9.431654	19.2843648
30-40	0.76	25.4592	16.5376	-9.8527104	10.24008	20.09279232
40-50	0.81	25.4592	17.6256	-9.8527104	10.91377	20.76648192
50-60	0.85	25.4592	18.496	-9.8527104	11.45272	21.3054336
60-70	0.89	25.4592	19.3664	-9.8527104	11.99167	21.84438528
70-80	0.93	25.4592	20.2368	-9.8527104	12.53063	22.38333696
80-90	0.96	25.4592	20.8896	-9.8527104	12.93484	22.78755072
90-100	0.99	25.4592	21.5424	-9.8527104	13.33905	23.19176448
100-120	1.04	25.4592	22.6304	-9.8527104	14.01274	23.86545408
120-140	1.09	25.4592	23.7184	-9.8527104	14.68643	24.53914368
140-160	1.13	25.4592	24.5888	-9.8527104	15.22538	25.07809536
160-180	1.17	25.4592	25.4592	-9.8527104	15.76434	25.61704704
180-200	1.2	25.4592	26.112	-9.8527104	16.16855	26.0212608
200-250	1.28	25.4592	27.8528	-9.8527104	17.24645	27.09916416

Level	height range (ft)	Tributary Height (ft)	Tributary Width (ft)	Area Ave. Wind Pressure(psf)	F_x (k)
B		0.00	164	0.0	0
1	0-10	10.00	164	17.5	29
2	10-28	18.00	164	18.3	54
3	28-43	15.00	164	20.1	49
4	43-56.5	13.50	158	21.0	45
5	56.5-70	13.50	158	21.7	46
6	70-83.5	13.50	112	22.5	34
7	83.5-96.5	13.00	112	23.0	33
8	96.5-109	12.50	112	23.7	33
9	109-121.5	12.50	112	23.9	34
10	121.5-134	12.50	112	24.5	34
11	134-146.5	12.50	112	24.8	35
12	146.5-159	12.50	112	25.1	35
13	159-171.5	12.50	112	25.6	36
14	171.5-184	12.50	112	25.7	36
15	184-196.5	12.50	86	26.0	28
roof	196.5-203	6.50	86	26.5	15
	$\Sigma =$	203	$\Sigma =$	370	

North – South Direction



E

Height	K_z	q_h	q_v	$p_{leeward}$	$p_{windward}$	p_{net}
0-15	0.57	25.4592	12.4032	-5.94981504	7.729674	13.67948928
15-20	0.62	25.4592	13.4912	-5.94981504	8.407716	14.35753088
20-25	0.66	25.4592	14.3616	-5.94981504	8.950149	14.89996416
25-30	0.7	25.4592	15.232	-5.94981504	9.492582	15.44239744
30-40	0.76	25.4592	16.5376	-5.94981504	10.30623	16.25604736
40-50	0.81	25.4592	17.6256	-5.94981504	10.98427	16.93408896
50-60	0.85	25.4592	18.496	-5.94981504	11.52671	17.47652224
60-70	0.89	25.4592	19.3664	-5.94981504	12.06914	18.01895552
70-80	0.93	25.4592	20.2368	-5.94981504	12.61157	18.5613888
80-90	0.96	25.4592	20.8896	-5.94981504	13.0184	18.96821376
90-100	0.99	25.4592	21.5424	-5.94981504	13.42522	19.37503872
100-120	1.04	25.4592	22.6304	-5.94981504	14.10327	20.05308032
120-140	1.09	25.4592	23.7184	-5.94981504	14.78131	20.73112192
140-160	1.13	25.4592	24.5888	-5.94981504	15.32374	21.2735552
160-180	1.17	25.4592	25.4592	-5.94981504	15.86617	21.81598848
180-200	1.2	25.4592	26.112	-5.94981504	16.273	22.22281344
200-250	1.28	25.4592	27.8528	-5.94981504	17.35786	23.30768

Level	height range (ft)	Tributary Height (ft)	Tributary Width (ft)	Area Ave. Wind Pressure(psf)	F_x (k)
B		0.00	112	0.0	0
1	0-10	10.00	112	17.5	20
2	10-28	18.00	112	18.3	37
3	28-43	15.00	112	20.1	34
4	43-56.5	13.50	90	21.0	26
5	56.5-70	13.50	90	21.7	26
6	70-83.5	13.50	90	22.5	27
7	83.5-96.5	13.00	90	19.2	22
8	96.5-109	12.50	90	19.9	22
9	109-121.5	12.50	90	20.1	23
10	121.5-134	12.50	90	20.7	23
11	134-146.5	12.50	90	21.0	24
12	146.5-159	12.50	90	21.3	24
13	159-171.5	12.50	90	21.8	24
14	171.5-184	12.50	90	21.9	25
15	184-196.5	12.50	88	22.2	24
roof	196.5-203	6.50	88	22.7	13
	Σ =	203	Σ =	332	

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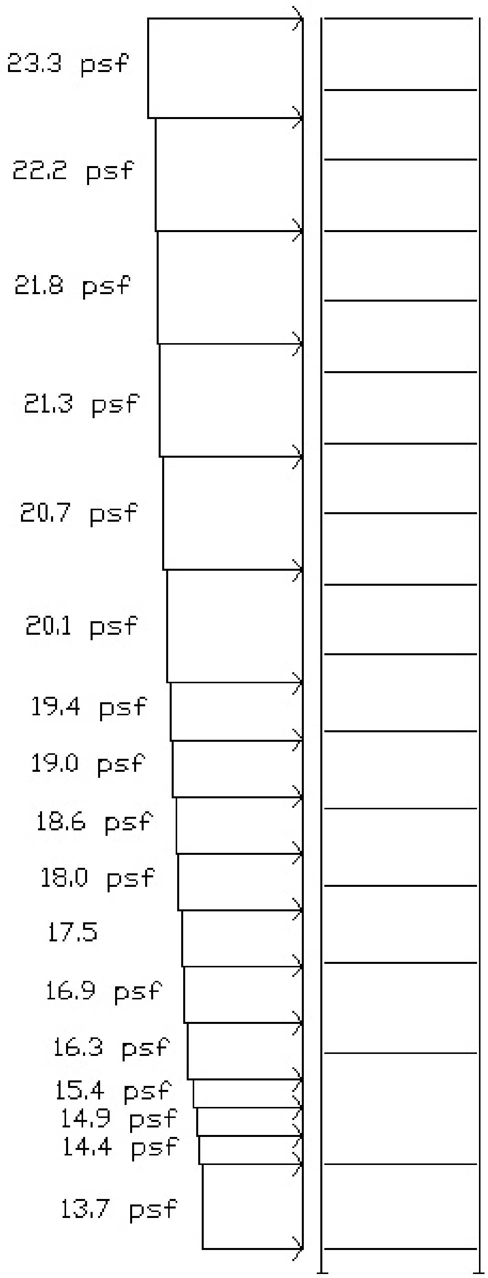
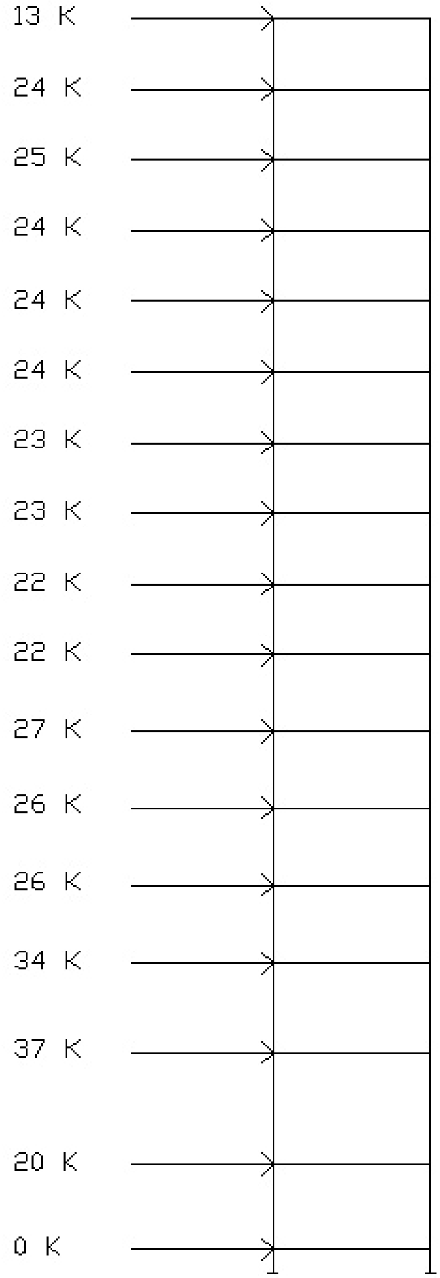




Wind Net Pressures (E-W) Wind Story Forces (E-W)

Seismic Loads

Assumptions:
Occupancy Category I (Table 1-1)
Seismic Use Group I (Table 9.1.3)
Importance Factor = 1.0 (Table 9.1.4)
Site Class D (Table 9.4.1.2)
Steel Concentrically Braced Frames

$S_s = 0.43$ (Figure 9.4.1.1a)
$S_1 = 0.095$ (Figure 9.4.1.1b)

$S_{ms} = 0.626$
$S_{m1} = 0.228$

$S_{ds} = 0.417$
$S_{d1} = 0.152$

$T = 1.725$
$C_s = 0.022$

Seismic Design Category B

Effective Seismic Weight of Structure (9.5.3)

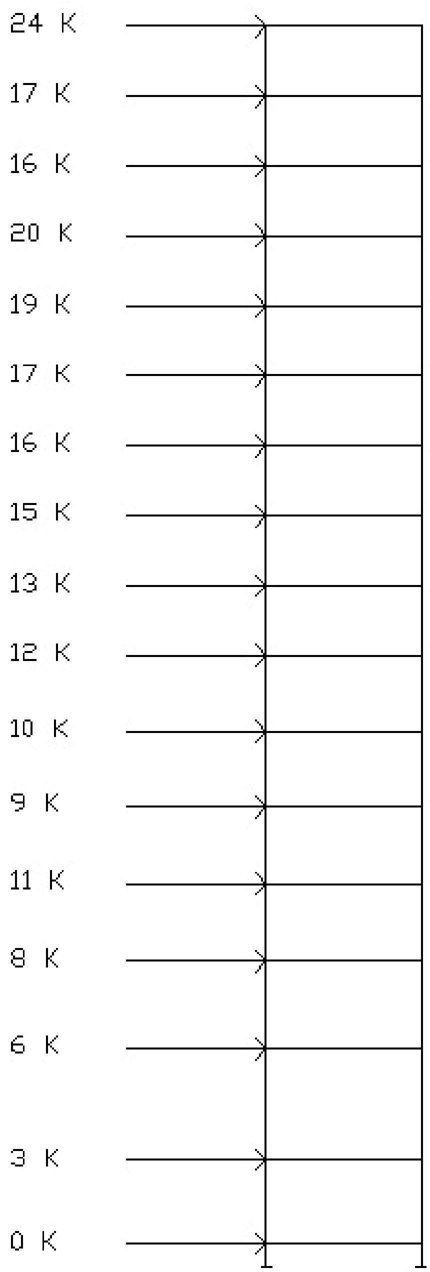
$W_{TOTAL} =$ 9921 k

Seismic Base Shear (9.5.5.2)

$V = C_s W$

$V =$ 218 k

Level	w_x (k)	h_x	$w_x h_x^k$	C_{vx}	F_x (k)
B	0	0	0	0	0
1	910	14.5	13195	0.012221	3
2	871	34.25	29831.75	0.027629	6
3	840	50	42000	0.038899	8
4	840	63.75	53550	0.049596	11
5	569	77.5	44097.5	0.040841	9
6	569	91	51779	0.047956	10
7	554	104.5	57893	0.053618	12
8	561	117	65637	0.06079	13
9	561	129.5	72649.5	0.067285	15
10	561	142	79662	0.07378	16
11	561	154.5	86674.5	0.080274	17
12	561	167	93687	0.086769	19
13	561	179.5	100699.5	0.093264	20
14	423	192	81216	0.075219	16
15	423	204.5	86503.5	0.080116	17
roof	556	217	120652	0.111743	24
Σ =	9921	Σ =	1079727		



Seismic Story Forces (E-W)

Problem Statement

After completing Technical Reports 1, 2, and 3, it was clear to me that the current design of Fordham Place is a complete efficient design. Technical report 1 was an exploration of the existing structural system and calculation of loads. For technical report 2, the existing floor system of concrete on composite metal deck supported by steel beams was compared to six other viable floor systems. It was obvious the existing system was the best and most efficient option, however two other options would be reasonable; two-way flat slab with drop panels and concrete on non-composite metal deck supported by steel beams. In technical report 3, a detailed analysis of the existing lateral system was done. It was determined the existing system, concentric steel chevron bracing, was also a great design for 2 reasons. One, chevron frames is a frame that is inexpensive compared to other lateral resisting systems such as moment frames. Two, the location of the frames throughout the building are located so that when lateral forces are applied to the building, very little torsional moment will be induced into the building. With this said, there was not an obvious system to change in Fordham Place. Therefore, I will redesign Fordham Place using a two way slab with drop panels to gain experience with a concrete floor system.

Structural Proposal

A viable solution will be to use an all concrete building as opposed to an all steel building. Due to architectural features, column locations will remain in the same locations; therefore leaving bay sizes the same. Considering the existing, 28' x 28' bay size, the only viable concrete system is a two way slab with drop panels. The new concrete floor system will require replacing the existing lateral force resisting system from concentric steel chevron braced frames to either concrete moment frames or shear walls. Both moment frames and shear walls will be further evaluated to determine which is better suited. Other design considerations will be floor to floor height, duct work / pipe / electrical paths, weight of building, and both story and total building drift.

The design of Fordham Place using a two-way flat slab with drop panels will done using the existing footprint and column locations of the building. A model of the building will be constructed using a finite element analysis computer program such as ADOSS or ETABS. Parameters such as slab thickness, gravity and lateral loads, concrete strength, etc. will be either hand calculated or assumed and inputted into the model. The modeling program will be used to design reinforcement; however spot checks will be done to assure a satisfactory design. Once, gravity loads are transferred throughout the building and slabs and columns are designed, the lateral system will

then be considered. Concrete shear walls and concrete moment frames will be considered as possible lateral resisting systems. With both systems, torsional effects can have a significant effect on the lateral design. However if they can be placed so that their center of rigidity is located near the geometric center of the building, the effects will be negligible. Floor deflection, story drift and total building drift will be checked. All designs of concrete elements will conform to ACI 02.

Breadth Work Proposal

Construction Management

With a switch from a steel building to a concrete building, impacts will be made on the construction schedule and methods used. Therefore, an analysis of each change in the depth work will be a crucial part in determining which solution is most economical and feasible. One analysis will compare the cost of a concrete slab on composite metal deck supported by steel beams to that of a two way flat slab with drop panels. This analysis will include the price difference due to a change in the lateral system from concentric chevron braced frames to either shear walls or concrete moment frames. Both shear walls and concrete moment frames will be researched to determine which a better option is. Another analysis will be of the construction schedule. This will clearly show critical paths and task durations for optimum construction processes for both the steel and concrete buildings. With all the changes made to the structure, there will be additional construction issues such as material availability, cost, constructability, and labor forces.

Mechanical

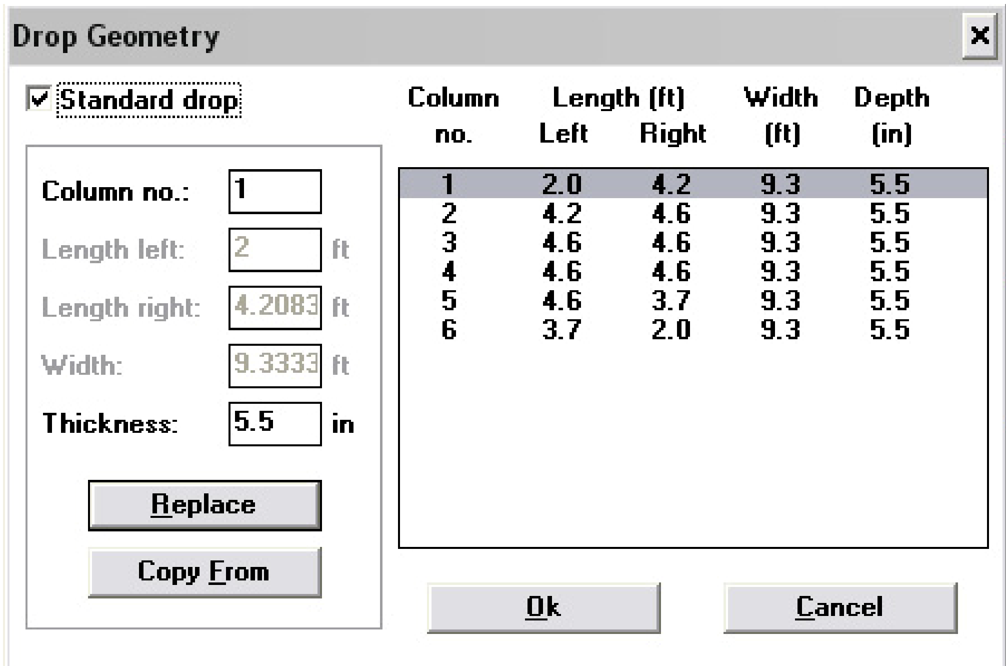
As a result of redesigning the structural system, mechanical system issues will arise. Changing the floor system to a two way flat slab will affect a number of things related to the current mechanical system design; such as routes of duct work, optimal mechanical systems used. Running duct work along walls may yield a smaller concrete slab rather than the current design of running it through interior sections of the floor. Also, it may be more efficient to use a totally different HVAC system such as individual units. This could possibly eliminate the need for punching large holes in the slab for duct work. Calculations will be performed to determine whether the current mechanical system is adequate to service the new structural systems. Adjustments to the mechanical system will be made as necessary.

Structural Redesign

Two way slab / Drop panels

My redesign of Fordham Place will comprise of a 9" flat slab with 5 ½" drop panels. Materials used for this redesign is normal weight concrete with compressive strength of 4ksi and steel rebar with a yield stress of 60 ksi. Floor slab thickness was determined by ACI 318-02 table 9.5(c) using exterior panels, without edge beams, but with drop panels to get minimum floor slab thickness of $\ell_n/36$. Where ℓ_n = 28' – 2' = 26', and the value $\ell_n/36$ = (26' x 12") / 36 = 8.67". At first I determined the drop projection of ¼ t_{slab} from ACI 318-02 section 13.3.7.2. Where ¼ t_{slab} = 9"/4 = 2.25". In order to form the drops with 2 x 4's or 2 x 6's, drop projection needs to be either 3.5" or 5.5". Therefore drop projections were 3.5". However, when analyzed in ADOSS, a 3.5" drop did not provide sufficient shear capacity. I then changed the drop projection to 5.5" and determined the slab had sufficient shear capacity.

In ADOSS, I used the standard drop tool which lets ADOSS determine the width of the panels. (see top left of picture below)



After doing a hand check for the drop widths, I determined that ADOSS calculated drop widths using ACI 13.3.7.1. This section states the minimum drop width shall be 1/6 span from center to center of supports in each direction. Where 1/6 span = 1/6 x 28' = 4.67'. This can also be seen in the above table. At this point I was able to analyze the floor system in ADOSS. Material properties, slab reinforcement data, geometry, loads, and load factors needed to be input into ADOSS. Flexural reinforcement is located 1.5" from the tension face with a minimum spacing of 6". #4 bar will be a minimum bar size.

Minimum reinforcement ratio is

$(A_s)_{min} = 0.0018\ A_g$ ACI 7.12.2.1

$= 0.0018 \times 9'' \times 12''$

$= 0.19\ in^2/ft$

Therefore the minimum flexural reinforcement will be #4's @ 12".

In order to simplify the design of the slab and columns, there was an assumption that shear walls would resist 100% of the lateral load; leaving the slab and columns to resist only gravity loads. Gravity loads that were considered were dead, live, roof live, and snow. The following is a list of the loads that were used in designing the concrete system.

Superimposed Dead = 30psf

Live = 80psf

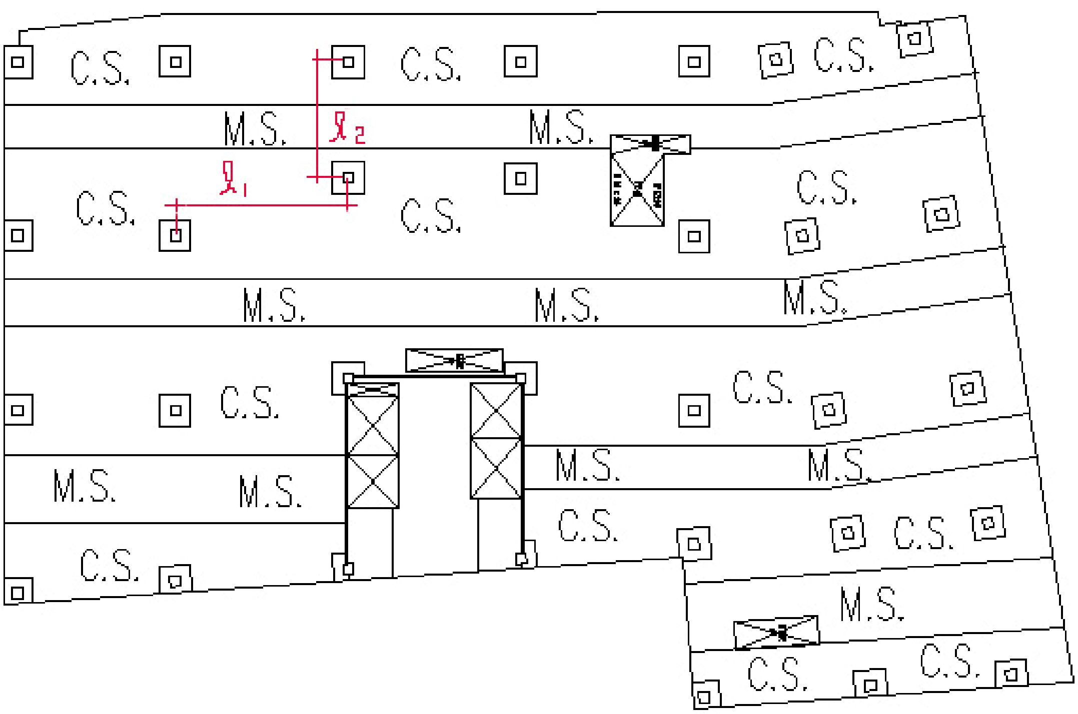
Roof live / snow = 30psf

Live loads were reduced to lesser values based on ASCE 7-02. (See Appendix for complete calculations)

After inputting this information into ADOSS, I was then able to design the system. The following is part of an ADOSS output file showing positive and negative reinforcement. Although ADOSS does design the number and spacing of bars, it was not very uniform throughout the different spans of the slab even though the total amount of steel required was similar. Therefore from the output file, I determined the amount of steel

per foot width and selected bar size and spacing. This was done for both column and middle strips. Having a more uniform steel layout throughout the building reduces the chance of a mistake in the field where a contractor may place the rebar incorrectly.

Because the column locations are staggered in two spans, it was a little difficult determining how I was going to analyze these spans. (See picture below)



Columns were determined using ACI 13.2.1. This section states the column strip shall be the lesser of $0.25\ell_1$ and $0.25\ell_2$. (See picture above for ℓ_1 and ℓ_2) Because of the staggered

columns, I decided to just make the area between those columns a big column strip. From the output file below, you can see the information I took from the output file to determine the area of steel per foot.

$$As = 3.84in^2 / 12.6ft$$

$$= 0.304\ in^2/ft$$

#5's @ 12"

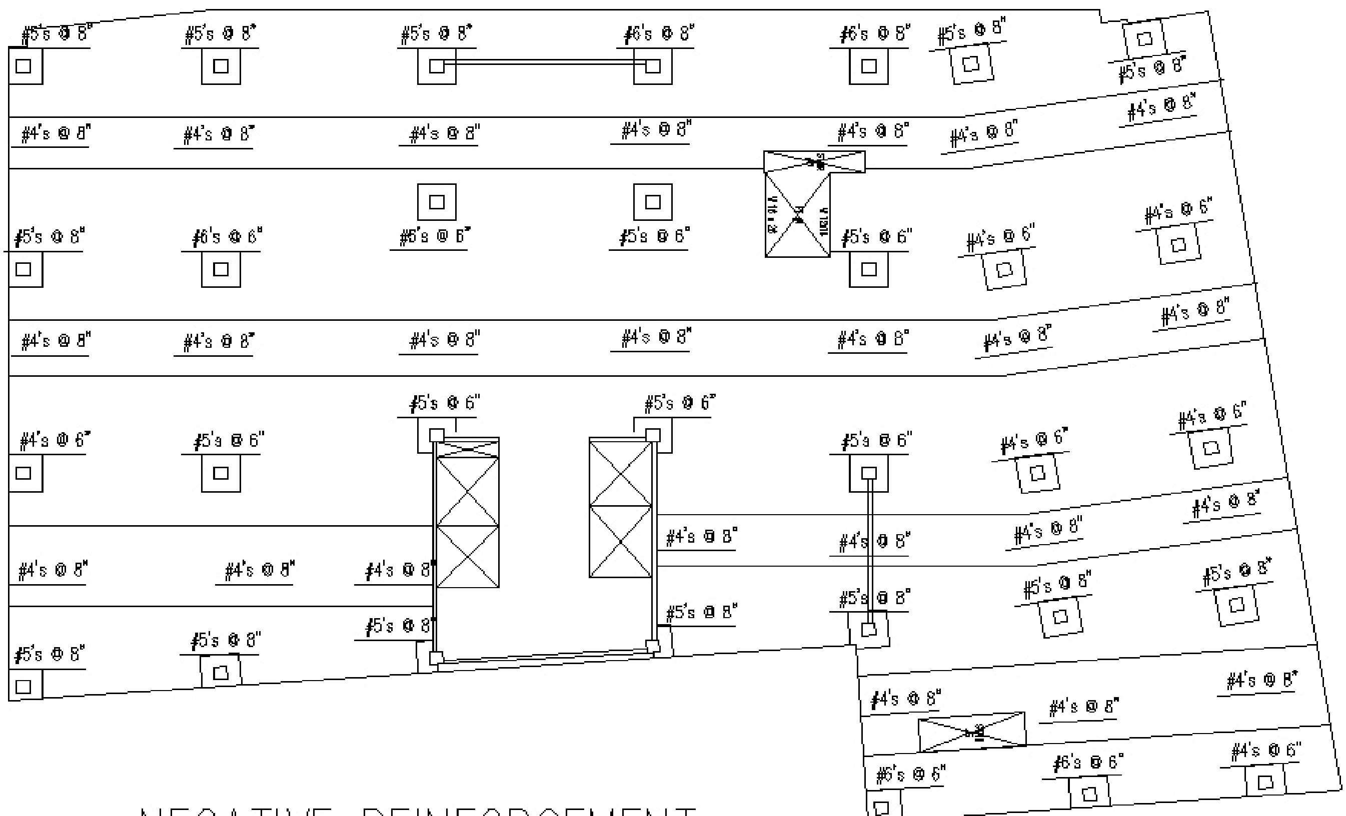
N E G A T I V E   R E I N F O R C E M E N T
****

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	4	|| R	231.2	3.84	12.6	3.32	15.4
2	4	L ||	-525.4	6.49	12.6	3.50	15.4
3	4	L ||	-516.4	6.38	13.9	3.45	14.1
4	4	|| R	533.3	6.59	13.9	3.56	14.1
5	4	L ||	-486.3	5.99	11.0	3.67	17.0
6	3	L ||	-146.9	3.48	11.0	3.67	17.0

P O S I T I V E   R E I N F O R C E M E N T
****

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	10.7	221.3	3.56	12.6	3.32	15.4
3	2	14.6	213.2	3.42	13.9	3.05	14.1
4	3	13.2	213.8	3.43	13.9	3.05	14.1
5	2	14.6	218.0	3.50	13.9	3.05	14.1
6	4	12.6	163.2	2.61	11.0	3.67	17.0

Once I knew that #5's @ 12" was a good rebar spacing and size, it just needs to be distributed over the entire column strip. The following is an example of a rebar plan in one direction. For complete rebar plans see appendix.



NEGATIVE REINFORCEMENT

The above rebar plan is showing both long and short bars. Half of the given bars are long bars and half are short bars. Extension of bars was done by ADOSS however it complies with figure 13.3.8 of ACI. This table can be viewed below.

222

CHAPTER 13

CODE

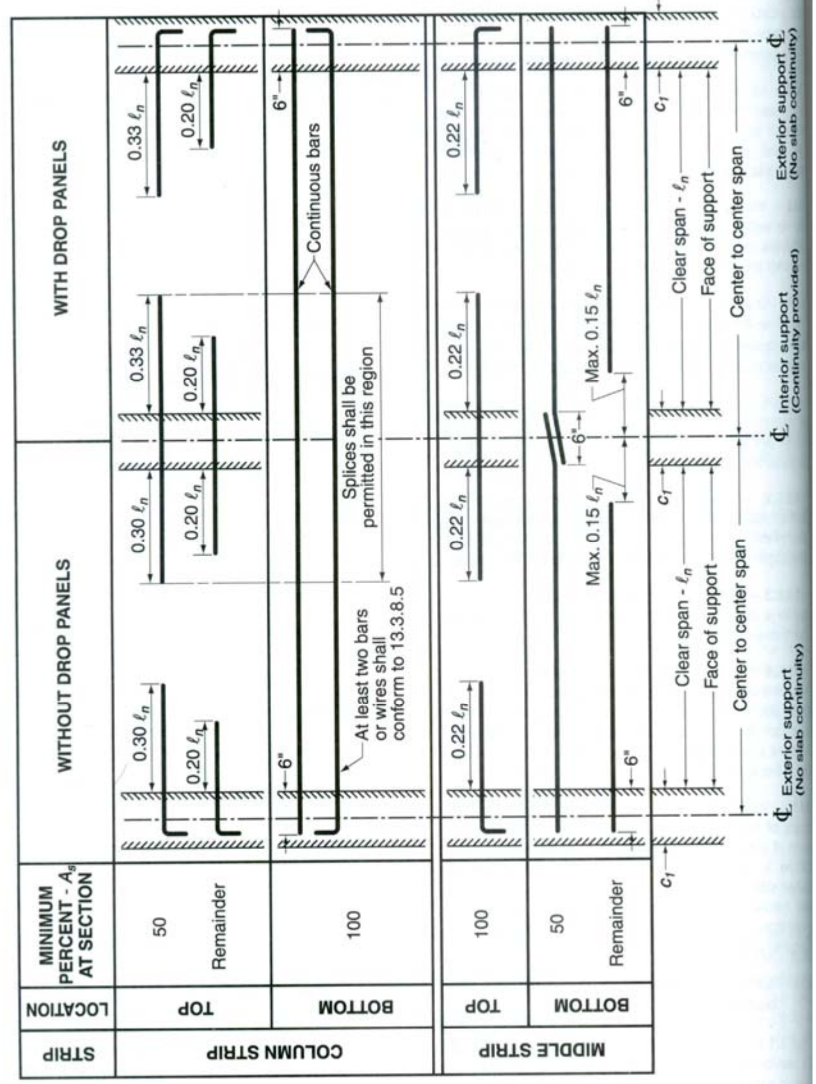


Fig. 13.3.8—Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

ACI 318 Building Code and Commentary

All other spans were analyzed using the same material properties, slab reinforcement data, and loads. The only thing that changed from span to span was its geometry.

Columns

The columns at Fordham Place are 26” x 26” normal weight concrete throughout the entire building. The concrete compressive strength is primarily 4ksi, however there are some 8ksi columns on the bottom 5 floors which support large tributary areas and in turn carry very large axial loads. The columns were designed by taking the unbalanced moment in each direction due to gravity loads and inputting them along with axial loads into PCA Column. Design moments were taken from the ADOSS output file. (see picture below)

N E G A T I V E   R E I N F O R C E M E N T

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	4	|| R	231.2	3.84	12.6	3.32	15.4
2	4	L ||	-525.4	6.49	12.6	3.50	15.4
3	4	L ||	-516.4	6.38	13.9	3.45	14.1
4	4	|| R	533.3	6.59	13.9	3.56	14.1
5	4	L ||	-486.3	5.99	11.0	3.67	17.0
6	3	L ||	-146.9	3.48	11.0	3.67	17.0

P O S I T I V E   R E I N F O R C E M E N T

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	10.7	221.3	3.56	12.6	3.32	15.4
3	2	14.6	213.2	3.42	13.9	3.05	14.1
4	3	13.2	213.8	3.43	13.9	3.05	14.1
5	2	14.6	218.0	3.50	13.9	3.05	14.1
6	4	12.6	163.2	2.61	11.0	3.67	17.0

Also, design axial loads were determined using an excel spreadsheet that multiplied tributary area by self weight, superimposed dead load, reduced live load, and roof live load. (See table below)

		roof	15th	14th	13th	12th	11th	10th	9th	8th	7th	6th
D.2 - 10.0	Tributary A	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3
		0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832
	LL + SDL (	48.87934	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449
	Slab Weig	75	125	125	125	125	125	125	125	125	125	125
	Drop Widt	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	Drop Leng	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	Drop Dept	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333
	Column W	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667
	Column Le	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667
	Pu (Kips)	123.7096	336.0698	548.43	760.7902	973.1504	1185.511	1397.871	1610.231	1822.591	2035.796	2249.002
	Mx (K-ft)											
	My (K-ft)											
	Unbraced (	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	13.5	13.5
	Column Si	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	20 - #11	20 - #11	20 - #11

The following is a list of other design criteria that was used for the concrete columns at Fordham Place:

- Minimum Reinforcement Ratio = 0.01
- Maximum Reinforcement Ration = 0.08
- Minimum Clear spacing between bars = 1.5”
- Minimum Clear cover = 0.75”
- Minimum bar size = #8
- Maximum Bar Size = #11

Longitudinal reinforcement in columns at a minimum is 12 - #11’s. This is the next smallest reinforcement ratio = 0.014 > 0.01.

Tie reinforcement was designed to conform to ACI 10.16.8.1 through 10.16.8.8. Bar sizes will be #3'and #4's where longitudinal reinforcement bar size is #8's and #11's, respectively. The spacing of ties was determined from the least of the following three criteria from ACI 10.16.8.5:

- $16 \times d_{\text{longitudinal bar}} = 16(1'') = 16''$
- $48 \times d_{\text{tie bar}} = 48(.375'') = 18''$
- $0.5 \times \text{column dimension} = 0.5(26) = 13''$

Since the maximum spacing of tie reinforcement was controlled by the column dimension, and the columns are sized the same throughout the entire building, ties throughout the columns will be spaced the same. Furthermore, since the maximum spacing is just 13", tie reinforcement will be spaced at 12" for convenience purposes.

<u>Shear Walls</u>

When a floor system is changed from composite steel beams to an all concrete structure, the original lateral system of braced frames need to be re-evaluated to some kind of concrete system such as shear walls or moment frames. I decided to treat my columns as supporting gravity load only, and therefore the shear walls will be the sole lateral force resisting system. The starting point for designing the lateral system was first to determine the weight and the seismic characteristics of Fordham Place. Then I was able to

compare the new seismic forces to the wind forces determined in Tech 1. The extra weight of the building caused the seismic loads to control the design. The following table shows the seismic characteristics determined in accordance with ASCE 7-02. For building weight see appendix.

Seismic Analysis

Assumptions:
Occupancy Category I (Table 1-1)
Seismic Use Group I (Table 9.1.3)
Importance Factor = 1.0 (Table 9.1.4)
Site Class D (Table 9.4.1.2)
Ordinary Reinforced Concrete Shear Walls

Ss = 0.43 (Figure 9.4.1.1a)
S1 = 0.095 (Figure 9.4.1.1b)

Sms = 0.626
Sm1 = 0.228

Sds = 0.417
Sd1 = 0.152

T = 1.07
Cs = 0.03551

Seismic Design Category B

Effective Seismic Weight of Structure (9.5.3)

W_{TOTAL} = 28004 k

Seismic Base Shear (9.5.5.2)

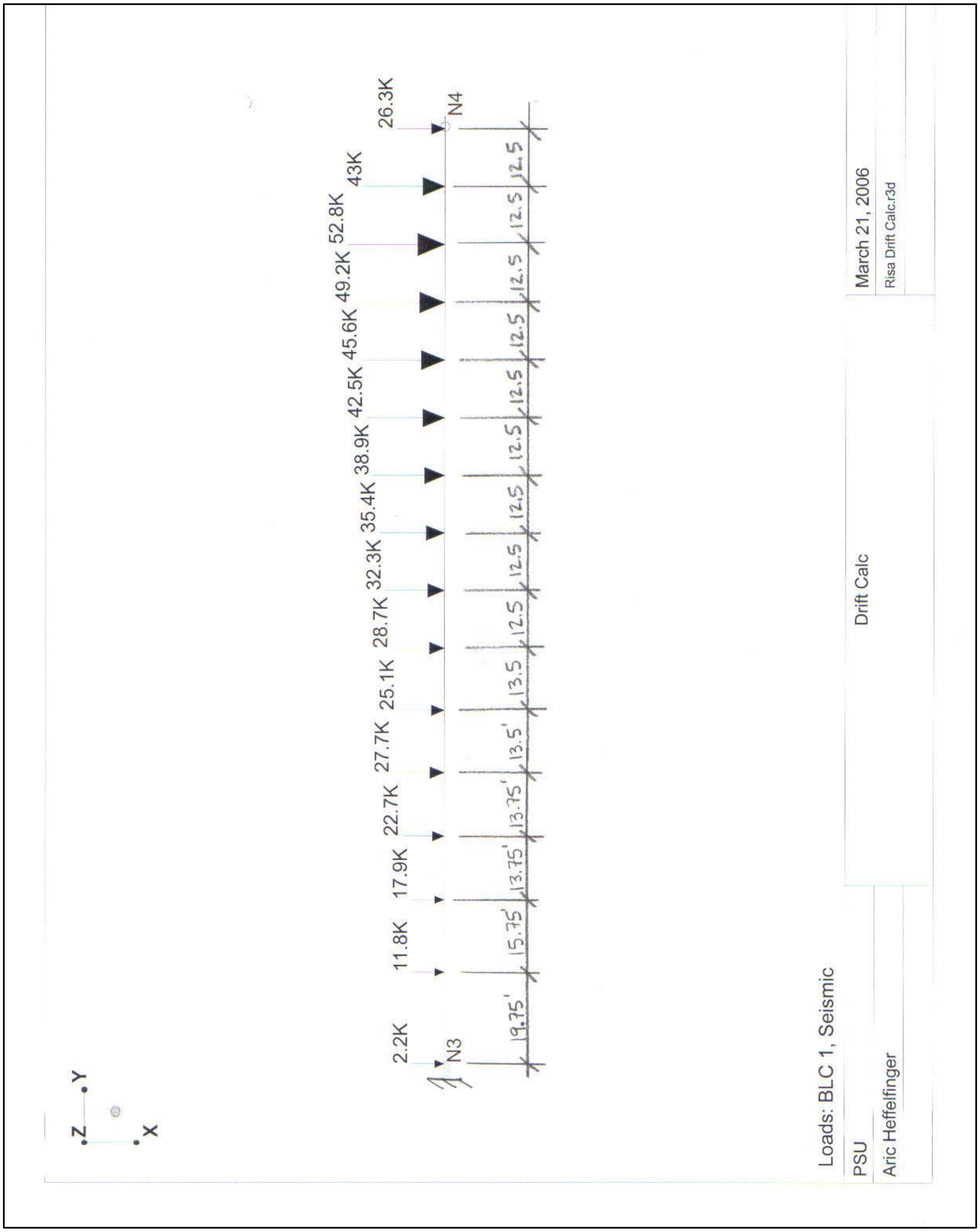
$V = C_sW$

V = 994 k

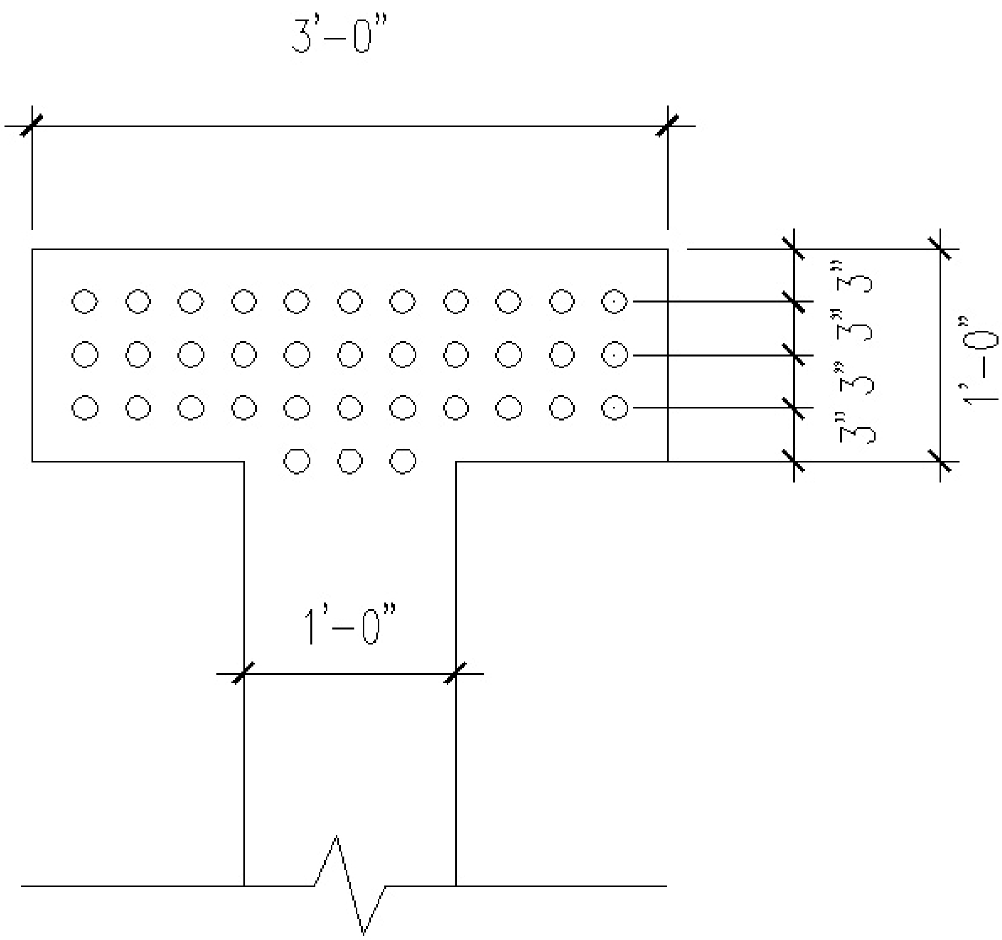
Level	w_x (k)	h_x	$w_xh_x^k$	C_{vx}	F_x (k)
B	0	0	0	0	0
Mezz.	1068	14.5	15486	0.00492	5
2	2477	34.25	84837.25	0.026953	27
3	2577	50	128850	0.040936	41
4	2286	63.75	145732.5	0.046299	46
5	2286	77.5	177165	0.056285	56
6	1691	91	153881	0.048888	49
7	1691	104.5	176709.5	0.056141	56
8	1691	117	197847	0.062856	63
9	1691	129.5	218984.5	0.069572	69
10	1691	142	240122	0.076287	76
11	1691	154.5	261259.5	0.083002	83
12	1691	167	282397	0.089718	89
13	1691	179.5	303534.5	0.096433	96
14	1691	192	324672	0.103149	103
15	1322	204.5	270349	0.08589	85
roof	764	217	165788	0.052671	52
Σ=	27999	Σ=	3147615		

After determining the story forces located at each floor level, lateral forces were distributed based on stiffness of each shear wall. Since my shear walls are not at the face of the building, the floor slab will have to axially transfer lateral loads on the building to the shear walls. Once the lateral forces reach the shear walls, they will act as point loads on the shear walls. To design the shear walls, I treated the wall as a gigantic cantilever beam with numerous point loads. The following is a diagram of the most severely loaded shear wall showing lateral forces on the wall. However, every shear wall will be designed the same for simplification purposes.

Aric Heffelfinger
Fordham Place
Bronx, NY
Structural Option
Advisor - Dr. Hanagan



At this point I was able to determine shear and moment diagrams. The max shear was determined to be 502k and was located at the fixed based of the "cantilever beam". The final design of shear reinforcement in the wall was #5's at 12" for the first 1/3 of the building height. The second third will contain #5's at 24", while the last third will not require shear reinforcement. When I move on to designing the flexural reinforcement, I discovered I would need a lot more steel than I had originally estimated. (A_s = 53.7in^2) With using a 12" shear wall, it was merely impossible to stuff this steel into the end of the wall with only 1ft width. From here I decided to use a flanged shear wall. The flanged shear wall consisted of the exact same design, but allowed me to fit all the steel in a reasonable configuration. The dimensions of the flanged section are 3ft flange width with a flange thickness of 1ft. There will be 3 rows of 11 - #11's within the flange while 1 row of 3 - #11's are just inside the web. See picture below



With 36 - #11's, this gives $A_s = 56.2in^2 > 53.7in^2$. See appendix for complete shear and flexural reinforcement calculations. Building drift calculations were determined by taking the most severely loaded shear wall and determining its deflection, and then extrapolating to get the drift of the building corner. This value was then compared to H / 400. To find the drift of the shear wall, I once again treated the shear wall as a cantilever beam, and then used the deflection equation from the Manual of Steel

Construction, Load and Resistance Factor Design, Third Edition, Table 5-17.

$\Delta = Pb^2(3\ell - b) / 6EI$

Where, P = Force on beam

b = distance from point load to fixed end

ℓ = length of beam

E = Modulus of Elasticity of concrete

I = Moment of inertia of cross section

Method of superposition was utilized by determining the deflection due to each load and then summing the total up. Calculations of deflections can be seen in the following table.

Load (K)	b (ft)	Δi (in)
8.85	19.75	0.003369325
13.425	35.5	0.016070886
17.025	49.25	0.038282572
20.775	63	0.074557751
18.825	76.5	0.097146053
21.525	90	0.149834326
24.225	102.5	0.213439935
26.55	115	0.287170105
29.175	127.5	0.378046531
31.875	140	0.485021138
34.2	152.5	0.600966102
36.9	165	0.738210665
39.6	177.5	0.890908581
32.25	190	0.807172183
19.725	202.5	0.54399496
	Δtotal =	5.324191114

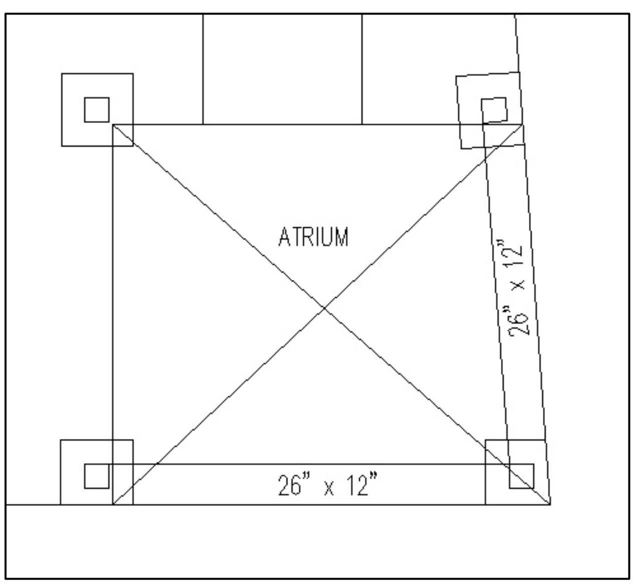
5.32 in < h/400
5.32 in < 6.07 in OK

Special Areas throughout Building

There are a couple different areas throughout the building that required a little extra attention and also a modification to the standard designs. These areas comprise of an atrium space on second floor, a mezzanine floor that resulted in columns with large unbraced lengths, and a large span in the floor slab.

Atrium space on the second floor

The problem with the atrium space is that it is at a corner of the building, which means there is no floor slab to laterally support the columns. To resolve this problem, I designed 26" x 12" beams to span from the corner column both adjacent columns. These beams reduce the unbraced length of the columns and in turn dramatically increase the capacity of the columns. This area can be seen on the following diagram.



Mezzanine floor / columns with large unbraced lengths

There is a mezzanine floor between the ground and second floors that covers only about ¼ of the building footprint. This makes about ¾ of the columns be designed with a large unbraced length. The typical 26" x 26" 4ksi column did not have the capacity to carry required loads with this large unbraced length. However since there were a few columns that carried extremely large axial loads and required 8ksi concrete, this gave me another option to look at. The question was then; would these columns have sufficient capacity using 8 ksi concrete? After running a few of the critical columns in PCA Column with 8ksi concrete, I was able to determine that yes, the 8ksi concrete did provide enough capacity for the given unbraced length.

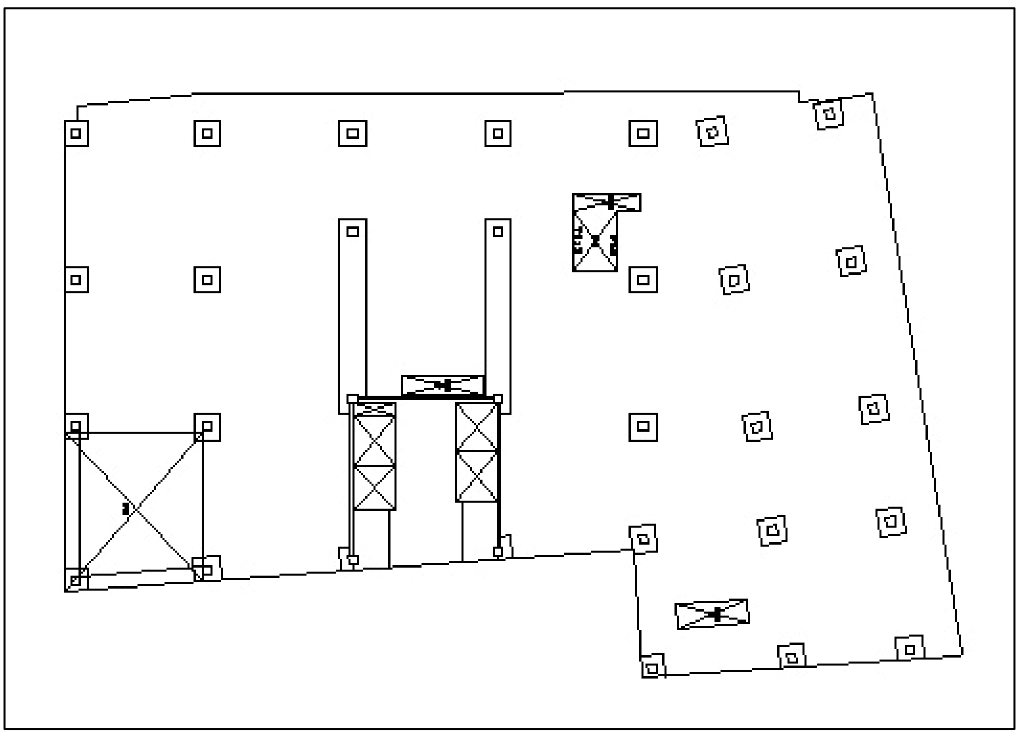
Large span in floor slab

There are two 32' – 2" spans on every floor that are larger than the typical 28' span. I could have just designed the entire building thicker slab that would be sufficient for a 32' span, however once you get over about 30', a two way slab is not very efficient. A common practice when there are one or two larger spans within the building is to use a continuous drop from column to column. This is precisely what I ended up doing. The contractors forming the concrete will just form the drop from one column to the next which will essentially make that part of the slab have a thickness of

t = 9" + 5.5"

= 14.5"

Reinforcement will be placed at 0.75in from tension face. Although this will require a bit more concrete, it is a far better solution than to just design the entire system based on a typical 32' span. See picture below for specified spans.



Foundations

Final designs of foundations were not completed for the original design, therefore will not be done as a redesign. However is understood that with and increased building weight, there will be a need for larger foundations and in turn be an increase in overall building cost.

Breadth Work

<u>Construction Management / Cost Analysis</u>

Both time and durations were compared for each of the composite steel and entire concrete structure. While the cost of the concrete and composite steel superstructures were comparable, the duration of the all concrete building needed nearly double the time as the composite steel. The total cost of each building is as follows:

All concrete = \$2.42 Million

Composite Steel = \$1.74 Million

Yielding a difference of 2.42 - 1.74 = \$0.68 = \$680,000. However these numbers are only taken from the differences that would between composite steel and all concrete building; and do not include the entire building. They are basically the superstructures of each building; columns, floor slabs, and lateral resisting systems. However material, labor, and equipment cost were taken into account for the entire superstructure. The material costs of the two structures were almost exactly the same, which means the labor costs of the concrete structure was a significant amount more. This can be seen in the following two tables.

Estimate Detail - Manhours - Standard Construction Project

Detail - Without Taxes and Insurance

Estimator :
Project Size : sqft

ItemCode	Description	Quantity	UM	Crew	MH/Unit	Lab.Unit	Mat.Unit	Eqp.Unit	Sub.Unit	Lab.Total	Mat.Total	Eqp.Total	Sub.Total	Tot.UnitCost	TotalCost
03220.010	6x6 W1.4/W1.4 MESH	1,873.87	SQS	C320	0.80	18.8640	8.200			35,348.72	15,365.75			27.064	50,714.47
03311.700	**CONC IN SLAB OVER MTL DECK**		****												
03311.726	4000 PSI W/CRANE	2,366.00	CUYD	C230	0.576	13.9420	56.000			32,986.77	132,496.00			69.942	165,482.77
03315.991	* SLAB OVER METAL DECK AREA *	170,352.00	SQFT												
05129.101	STEEL BEAMS		****												
05129.102	I BEAMS	705.60	CWT	C510	0.90	28.7300	35.000	5.000		20,271.89	24,696.00	3,528.00		68.730	48,495.89
05129.121	STEEL COLUMNS		****												
05129.122	I SHAPES	4,003.22	CWT	C510	0.90	28.7300	35.000	5.000		115,012.51	140,112.70	20,016.10		68.730	275,141.31
05129.181	BRACING		****												
05129.182	I BEAMS	4,564.63	CWT	C510	1.20	38.3067	35.000	5.000		174,855.76	159,761.91	22,823.13		78.307	357,440.80
05129.304	ASTM A572 50 KSI STEEL ADDER	4,003.22	CWT												
05129.310	TYPE B STEEL ADDER	4,564.63	CWT												
05129.404	SHEAR STUD, 3/4"	72.00	EACH	C509	0.01714	0.5434	0.717	0.300		39.12	51.61	21.60		1.560	112.33
05129.990	* STRUCTURAL STEEL WEIGHT *	463.67	TONS												
05129.101	STEEL BEAMS		****												
05129.102	I BEAMS	8,593.36	CWT	C510	0.90	28.7300	35.000	5.000		246,887.23	300,767.60	42,966.80		68.730	590,621.63
05129.404	SHEAR STUD, 3/4"	1,173.00	EACH	C509	0.01714	0.5434	0.717	0.300		637.41	840.81	351.90		1.560	1,830.11
05129.990	* STRUCTURAL STEEL WEIGHT *	429.67	TONS												
05310.019	3" METAL DECK	170,352.00	SQFT	C510	0.01393	0.4445	1.011			75,721.46	172,259.94			1.456	247,981.41
	Total Estimate									$701,761	$946,352	$89,708			$1,737,821

Estimate Detail - Manhours - Standard Construction Project

Detail - Without Taxes and Insurance

Estimator :
Project Size : sqft

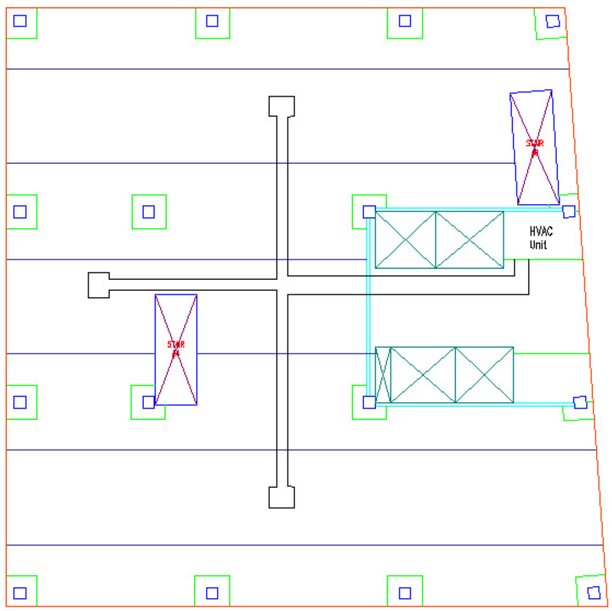
ItemCode	Description	Quantity	UM	Crew	MH/Unit	Lab.Unit	Mat.Unit	Eqp.Unit	Sub.Unit	Lab.Total	Mat.Total	Eqp.Total	Sub.Total	Tot.UnitCost	TotalCost
03111.118	WALL FORM 20'+ HIGH	72,912.00	SQFT	C311	0.14795	3.8747	1.600			282,512.13	116,673.78			5.475	399,185.91
03111.189	WALL FORM HARDWARE	36,456.00	SQFT				0.102				3,733.09			0.102	3,733.09
03111.203	WOOD COLUMN FORMS, 12'-16'	44,165.33	SQFT	C311	0.044	1.1524	1.227			50,896.13	54,182.03			2.379	105,078.16
03111.612	SLAB FORM W/2.6 BM/SF	176,587.00	SQFT	C311	0.0969	2.5380	1.263			448,177.81	223,047.04			3.801	671,224.85
03150.900	FORM RELEASING AGENT	117,077.33	SQFT	C311	0.008	0.2095	0.023			24,527.70	2,692.78			0.233	27,220.48
03150.650	SCREEDS FOR SLAB	21,190.44	LNFT	C311	0.0352	0.9219	0.320			19,535.47	6,780.94			1.242	26,316.41
03150.900	FORM RELEASING AGENT	176,587.00	SQFT	C311	0.008	0.2095	0.023			36,994.98	4,061.50			0.233	41,056.48
03210.150	COLUMN REBAR	3,322.62	CWT	C321	0.77778	24.7222	26.750			82,142.56	88,880.18			51.472	171,022.74
03210.130	SUPPORTED SLAB REBAR	5,886.23	CWT	C321	1.01818	32.3636	26.750			190,499.70	157,456.74			59.114	347,956.44
03310.550	**CONCRETE IN WALLS**		****												
03310.576	4000 PSI W/CRANE	1,350.22	CUYD	C230	0.68571	16.5977	56.000			22,410.58	75,612.44			72.598	98,023.03
03310.650	**CONCRETE IN COLUMNS**		****												
03310.676	4000 PSI W/CRANE	886.03	CUYD	C230	0.90	21.7845	56.000			19,301.78	49,617.84			77.785	68,919.63
03315.982	* CONCRETE WALL AREA *	36,456.00	SQFT												
03315.984	* NO. OF COLUMNS *	392.00	EACH												
03311.500	**CONC IN SUPPORTED SLAB**		****												
03311.526	4000 PSI W/CRANE	4,905.19	CUYD	C230	0.576	13.9420	56.000			68,388.22	274,690.89			69.942	343,079.11
03315.986	* SUPPORTED SLAB AREA *	176,587.00	SQFT												
03350.131	POINT & PATCH	117,077.33	SQFT	C276	0.00427	0.1102	0.013			12,901.92	1,498.59			0.123	14,400.51
03350.130	MACHINE TROWEL FINISH	176,587.00	SQFT	C276	0.0128	0.3304				58,344.34				0.330	58,344.34
03350.131	POINT & PATCH	176,587.00	SQFT	C276	0.00427	0.1102	0.013			19,459.89	2,260.31			0.123	21,720.20
03390.010	PROTECT & CURE	176,587.00	SQFT	C276	0.00427	0.1102	0.019			19,459.89	3,390.47			0.129	22,850.36
	Total Estimate									$1,355,553	$1,064,579				$2,420,132

With this said, it was no surprise to see that the concrete structure took almost twice as long as the composite steel structure. The composite steel structure needs structural steel crews, concrete crew, wire mesh, and miscellaneous steel crew. Because the steel erectors can work as fast as they can, there will be 2 crews to speed up the project. The total duration of the composite steel building is 40.2 calendar weeks. For the all concrete structure, formwork crews, reinforcing steel crews, concrete crew, and a finishing crew are needed. Since there is a tremendous amount of formwork to be place, there will be five formwork crews. There will also be two concrete and reinforcing steel crews. Even with all these crews, the total duration of the concrete superstructure is 78.3 weeks. A complete set of descriptions and calculations for both superstructures are in the appendix.

Mechanical / Duct Work Reroute

After reviewing the duct work and HVAC plans, the HVAC units and duct work routes were still sufficient. However, there is a better solution. Because a concrete floor system does not work well with large openings in the slab, one HVAC unit serving multiple floors is not a great idea. Therefore the new design will employ a single HVAC unit for each floor, eliminating the need for large duct work both through the floors and throughout each floor level. Having only one HVAC unit per floor gives you, the owner, the ability to rent each floor out to different tenants while keeping their utilities separated. The disadvantage to having an HVAC unit on each floor is that you need to have a place to store each unit on each floor, taking away from valuable square feet of floor space. Whereas with a single unit serving every couple floors, one can be put on the rooftop, one in the basement, and as they are needed throughout the building. With a composite steel building, the single unit serving multiple floors is a better option, but with a concrete system, it eliminates the need for very detailed engineering of floor slabs by using a single unit for each floor. The duct work for the new systems will be 20” wide x 6” deep and then decreases to 12 x 6 when it branches off. This may seem a little large however; it is only six inches deep. The large area that the air will travel through will also reduce the need to “force”

air through the duct work and in turn reduce noise produced by the airflow. An example of the duct work routes can be seen in the following diagram.



Diffusers also can be located at the end of the duct work. The large diffuser size of 36” x 42” also permits air to flow at a slower rate; reducing both noise and the sensation of sitting just below an air conditioner.

Final Conclusions

All concrete vs. Composite steel

When you compare two structural systems, there is a lot more to compare than just how long will the job take and how much will it cost. Other such factors are perceptibility to floor vibrations, constructability, floor depths, area of the country, resistance to lateral motion, and many others. These factors will be examined in this section and in turn I will rate each of the two structural systems. Cost and duration analysis can be examined in the construction management breadth work.

The constructability of each of the structures is similar; however the edge would have to go to the composite steel. Although there are many hours put towards placing formwork, it is not very difficult to do so. Moment connections are a very difficult and time consuming connection, but the engineers were able to limit the building to only a few. Shear connections are very easy and quick to erect. In a concrete building, there is a lag time on the erection of the building due to the need to let the concrete cure. With a steel building, as fast as the steel erectors can put up the steel is how quick the project will move along.

When comparing floor depths between a concrete structure and a steel structure, an obvious advantage goes to the concrete building. Maximum floor depth needs to be looked at when

comparing floor depths. The following is a table showing maximum floor depths for each of the two buildings.

	Slab	Beam	MEP	Total (in)
Composite steel	6.25	30	18	54.25
All Concrete	9	5.5	18	32.5

The higher floor depth essentially means a taller building.

In the area where this building is being built, New York, building height is not an issue. However if you were to proposed this design to a developer in the DC area, they would laugh at you. In Washington, DC all buildings need to be shorter than the capital building. Essentially the difference between a steel building and a concrete building in DC is an extra floor. With and extra floor, as an owner you can lease it out and make about 15% more profit than you would in a steel building.

The lateral stability of the steel and concrete structures is completely dependent on the type of lateral resisting system used within the building. For a steel building, moment frames or braced frames can be used. In a concrete building, moment frames or shear walls are used. Braced frames and shear walls have a much larger stiffness, therefore limiting the lateral drift of the building. Braced frames resist loads through axial

deformation while shear walls resist loads through shear deformation. Both of which are exponentially better than resisting loads through moment rotation. With this said, both the shear walls and braced frames are very comparable and great lateral resisting systems.

Typically, you will not experience noticeable floor vibrations in a normal weight concrete structure nor a composite steel structure. Floor vibrations are sometimes a serious issue with open web steel joist as a floor system. Other floor systems that will sometimes cause vibrations are lightweight concrete floors, non composite steel systems with a small concrete slab. The two major factors that affect floor vibrations are rigidity and weight of the floor system. The following is a table rating each of the two systems on the basis of 0 being the worst and 5 being the best.

	Composite steel	All Concrete
Cost	5	4
Duration	5	2
Vibration Issues	5	5
Constructibility	4	3
Floor depth	2	5
Area of country	4	1
Lateral Drift	5	5
OVERALL	4.29	3.57

Cost and Duration

When looking and cost and duration of the two systems in detail, it is easy to see the composite steel structure has a distinct advantage. The cost of the concrete superstructure was nearly 40% more than the cost of the steel. The duration of the concrete structure was nearly 2 times the duration of the steel structure. This can be seen in the following table.

	Composite steel	All Concrete	Concrete / Steel	
Labor Cost	701,761	1,355,553	1.93	%
Material / Equipment Cost	1,036,060	1,064,579	102.75	%
Total Cost	1,737,821	2,420,132	139.26	%
Duration (weeks)	40	78	195.00	%

Cost and durations were pulled off the construction assemblies in ICE 2000 Estimating. Then the software was used to analyze and compare differences in material cost, labor cost, and durations.

Recommendations

After having the opportunity to design Fordham Place as both an all concrete structure and a composite steel structure, it was easy to come to the conclusion that the composite steel structure is a far better solution. The composite structure was more advantageous on all design considerations, including cost, duration, efficiency of system, etc. Fordham Place as a composite system uses the different materials as efficiently as they can be. Steel is the best material to resist tension, while concrete is the best to resist compression; and that is exactly how a composite steel system works. There is compression in the top concrete flange while the bottom steel takes the tension. Lateral forces are resisted by cross members in the braced frames that are under axial tension loading. Because the materials at Fordham place are used as efficiently as possible, this is the least that will be spent in material cost. When you combine that with how easy it is to construct and compost steel system, the final result is a very stable and inexpensive building.

Acknowledgements

The past five years I have spent at Penn State have been a fun, stressful, but a very worthwhile learning experience that I will never forget. So many people have given me so much advice support, and encouragement. At this time I would like to thank the people who were there for me and made my thesis report and career at Penn State a memorable one.

First and foremost I would like to thank Mom, Dad, Jason, Erin, and the rest of my family for always being there for me and believing in my ability. Also for always pushing me to strive to do my best and reach my full potential. Without them, there is no doubt; I would not be the person I am or achieve all the success that I have been so blessed to have done to this point in my life.

Secondly, I would like to thank the entire Penn State Architectural Engineering Faculty and staff for all the professional experience they have provided me. I would also like to personally thank Dr. Hanagan for not only providing me the knowledge needed to become an Engineer, but also for providing me with both professional and social advice.

Finally I would also like to thank both my professional contacts and colleagues here at PSU. A special thanks to all the folks at M.G. for providing me with two summers of great engineering experience; also for providing me with all the information and contacts for a good thesis project.

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American Institute of Steel Construction. Manual of Steel Construction: LRFD 3rd Edition. Chicago, IL: American Institute of Steel Construction, 2001.

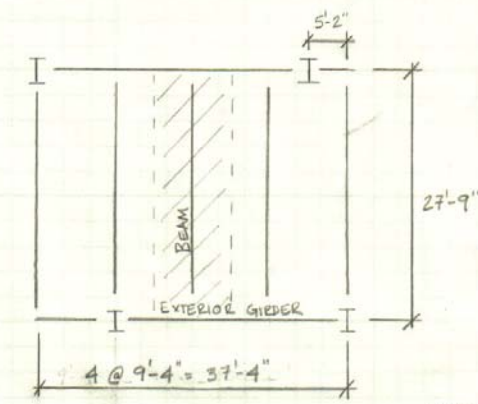
TABLE OF CONTENTS

Appendix

Gravity Load Spot Check

GRAVITY LOAD
SPOT CHECK

* THERE IS NO "TYPICAL" BAY SO I RANDOMLY SELECTED A BAY ON THE 9TH FLOOR



CONCRETE
3" - 20 GA COMP FLOOR DECK
+ 3¼" L.W. CONCRETE SLAB
TOTAL SLAB THK = 6¼"
f'_c = 3500 PSI
W_c = 115 PCF

STEEL
A992 f_y = 50 ksi

BEAM

SELF = 115 PCF (3¼" + 1.5")(9'-4")/12 + 26 PLF (W16×26)

= 451 plf

SUPERIMPOSED DEAD LOAD = 30 PSF (9'-4") = 279 plf

TOTAL DL = 451 plf + 279 plf = 730 plf

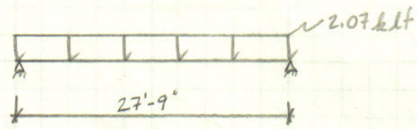
LIVE LOAD = 60 PSF$_{OFFICE}$ + 20 PSF$_{PARTITION}$ = 80 PSF (9'-4") = 747 plf

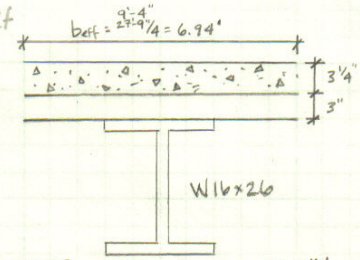
w_u = 1.2 D + 1.6 L
= 1.2(730) + 1.6(747)

w_u = 2.07 klf

GRAVITY LOAD
SPOT CHECK CONT'

BEAM CONT'





$M_u = \frac{w_u l^2}{8} = \frac{(2.07\ klf)(27.75)^2}{8}$

$M_u = 199^{ft\text{-}k}$

28 SHEAR STUDS   ASSUME 17.7^k/STUD

$\Sigma Q_n = 14(17.7)$
$= 247.8^k$

$0.85 f'_c ba = \Sigma Q_n$

$a = \frac{247.8}{0.85(3.5)(6.94)(12)}$

$a = 1.00$ IN

$T = A_s f_y = (50)(7.68\ in^2)$
$= 384^k$

$C_c = 0.85 f'_c ba = 0.85(3.5)(6.94')(12)(3.25)$
$= 805^k$

PNA IN CONC.

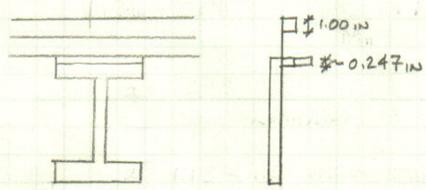
$A_s f_y = 247^k$

$A_s = 247.8^k/50$

$A_s = 4.956\ in^2$

$\frac{7.68 - 4.95}{2} = (A_s)_{comp} = 1.362\ in^2$

$1.362/b_{f} = 1.362/5.5 = 0.247$ IN   ALL IN FLANGE OF STEEL.



$M_n = 0.85 f'_c ba(6.25 - a/2) + A_s f_y(d/2) - (0.247)(5.5)(f_y)(0.247/2)2$
$= 0.85(3.5)(83.3)(1.0)(6.25 - 0.5) + (384^k)(15.7/2) - (0.247)(5.5)(50)(0.247/2)(2)$

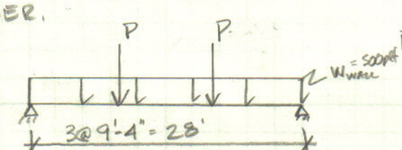
$M_n = 368^{k\text{-}ft}$

$\phi M_n = 0.9(368) = 331.7^{k\text{-}ft}$ $M_u = 199^{k\text{-}ft}$

I ATTRIBUTE THE DIFFERENCE IN ϕM_n AND M_u TO THE DIFFERENCE IN DESIGNING WITH ASD TO LRFD. IN ASD, COMPOSITE DESIGN IS TAKEN AS $0.76\ F_y$, WHERE AS IN LRFD, COMPOSITE DESIGN IS AT FULL ULTIMATE CAPACITY. ASD DESIGN WOULD YIELD A MUCH STRONGER MEMBER (MORE CONSERVATIVE) THAN LRFD, HENCE THE MOMENT CAPACITY, ϕM_n, OF THE CURRENT SYSTEM DESIGNED WITH ASD, AND ANALYZED WITH LRFD.

GRAVITY LOAD
SPOT CHECK CONT'

EXTERIOR GIRDER.



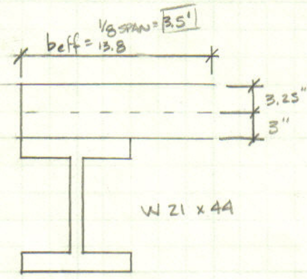
$P = \dfrac{w_u l}{2}$

$= \dfrac{(2.07\,klf)(27.75')}{2}$

$P = 28.7^k$

$M_u = Pa + w_{wall} l^2/8$
$= (28.7^k)(9.\bar{3}') + (0.5)(28^2)/8$

$M_u = 317^{k\text{-}ft}$



48 - 3" SHEAR STUDS (17.7^k/STUD)

$\Sigma Q_n = 24(17.7^k) = 425^k$

$C_c = 0.85(3.5)(42")(3.25 + 1.5)$
$= 594^k$

$T_s = (13\,in^2)(50\,ksi)$
$= 650^k$

$a = \dfrac{\Sigma Q_n}{0.85 f'_c b} = \dfrac{425^k}{(0.85)(3.5)(42")}$

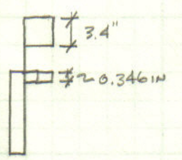
$a = 3.4\,in$

$A_s f_y = 425^k$
$A_s = 8.5\,in^2$

$(A_s)_c = \dfrac{13 - 8.5}{2} = 2.25\,in^2$

$\dfrac{(A_s)_c}{b_f} = 2.25/6.5 = 0.346\,in \le h_f = 0.45$

$\therefore$ ALL COMP. IN FLANGE



$M_n = 0.85 f'_c ba(6.25 - a/2) + A_s f_y(d/2) - (0.346)(6.5)(f_y)(0.346/2)(2)$

$= 0.85(3.5)(42)(3.4)(6.25 - 3.4/2) + (50)(13)(20.7/2) - (0.346)(6.5)(50)(0.346)$

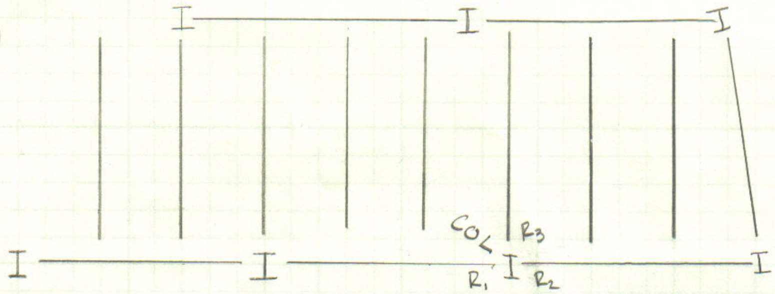
$M_n = 718$ ft-k

$\phi M_n = 0.9(718) = 647$ ft-k $\ge M_u = 317$ ft-k

I ATTRIBUTE THE DIFFERENCE IN M_u AND ϕM_n TO TWO THINGS. FIRST, I MADE THE ASSUMPTION THAT THE EXTERIOR GIRDER WAS SUPPORTING ONLY 1 STORY OF THE BRICK FACADE. WHEN THE NEW YORK CITY BUILDING CODE REQUIRES RELIEVING ANGLES AT LEAST EVERY 3 STORIES. THEREFORE I BELIEVE IT MAY BE DESIGNED FOR 3 STORIES. SECONDLY, SOME OF THE DIFFERENCE MAY BE COMING FROM DESIGNING WITH ASD AS OPPOSED TO LRFD AS WAS THE CASE WITH THE BEAM.

GRAVITY LOAD
SPOT CHECK CONT'

COLUMN



FROM PREVIOUS CALCULATIONS $R_1 = \frac{\omega_{wall}\,l}{2} + P$

$= \frac{(0.5\ k/l')(27.75')}{2} + 28.7^k$

$R_1 = 35.6^k$

$R_2 \approx 35.6^k$ – A LITTLE CONSERVATIVE

$R_1 = R_2 = 35.6^k \qquad R_3 = 28.7^k$

TOTAL REACTION AT COLUMN FROM LOADING ON FLOOR 9

$R_9 = 2R_1 + R_3 = 100.1^k$

SINCE SLAB THICKNESS, FRAMING MEMBERS, SUPERIMPOSED DEAD LOADS, AND LIVE LOADS ARE THE SAME THROUGHOUT THE REMAINDER OF THE BUILDING, EACH FLOOR WILL CONTRIBUTE 100.1^k TO THAT COLUMN LOCATED BETWEEN THE 8^{TH} & 9^{TH} FLOOR.
∴
TOTAL AXIAL LOAD ON COLUMN

P_u = 7 FLOOR LOAD + ROOF = $7(100.1^k) + 13.7^k$
$P_u = 714^k$

* ASSUME BRACED FRAME PICKS UP ENTIRE LATERAL LOAD.

∴ COLUMN IS PURE AXIAL LOAD, NOT A BEAM COLUMN.

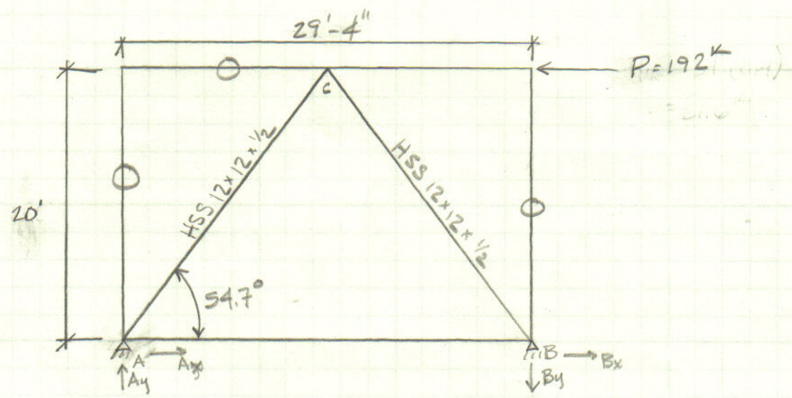
ASD DESIGN = W14 x 99 $\quad \phi P_n = 1090^k \quad P_u = 714^k$
$l_b = 12.5'$

THE DIFFERENCE BETWEEN ϕP_n AND P_u IS ATTRIBUTED TO THE ASSUMPTION THAT THE COLUMN IS PURELY AXIALY LOADED. WITH THE ADDITION OF BENDING MOMENT, THE COLUMN AXIAL CAPACITY, ϕP_n WOULD DECREASE CLOSER TO P_u AS ϕM_n WOULD HAVE TO REACH AT LEAST M_u.

Lateral Load Spot Check

LATERAL LOAD
SPOT CHECK

CHEVRON BRACING



ASSUMPTION: EACH OF 3 CHEVRON BRACED FRAMES WILL RECEIVE 40% OF LATERAL FORCE. 40% x 3 = 120% MAKING IT A CONSERVATIVE ESTIMATE.

TOTAL SHEAR AT GROUND LEVEL = 576^k

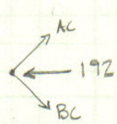
SHEAR PER FRAME = $576^k/3 = 192^k$

$\Sigma M_B = 0$

$A_y(29.\bar{3}) = 192^k(20')$

$A_y = 131^k \uparrow \quad B_y = 131^k \downarrow$

<u>Pt C</u>



$\Sigma F_y = 0$

$A_C = B_C$

$\Sigma F_x = 0$

$2\,A_C \cos(54.7) = 192$

$A_C = 166^k$

$P_u = 166^k$
$l_b = 24.8\,ft$

$\phi P_n = 620^k$

THERE IS QUITE A DIFFERENCE BETWEEN ϕP_n AND P_u FOR MY LATERAL LOAD CHECK. THIS COULD BE FOR A NUMBER OF REASONS; 1) CERTAIN SIZE MEMBER NEEDED TO MAKE THE CONNECTION BIG ENOUGH, 2) ME DISTRIBUTING THE LOAD VERY CONSERVATIVELY, 3) DID NOT INCORPERATE GRAVITY LOADS INTO THE COLUMNS.

Lateral Load Calculations (Composite Steel Structure)

WIND

$K_{zt} = (1 + K_1 K_2 K_3)^2$ NO TOPOGRAPHIC FRACTOR

$K_{zt} = 1.0$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$
$= (0.3)\left(\frac{33}{110.3}\right)^{1/6}$

$I_{\bar{z}} = 0.795$

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}}$
$= 320\left(110.3/33\right)^{0.\bar{3}}$
$= 478$

TABLE 6-2
$c = 0.3$
$\ell = 320$ ft
$\bar{\epsilon} = 0.\bar{3}$

$h = 183.9$ ft
$B = 164$
$L = 112$
} OPPOSITE FOR E-W

$\bar{z} = \left| \begin{array}{l} 0.6h = 0.6(183.9 \text{ ft}) = 110.3 \\ 30 \text{ ft} \end{array}\right.$ MAX

$Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$

$= \sqrt{\dfrac{1}{1 + 0.63\left(\frac{164+183.9}{478}\right)^{0.63}}}$

$= 0.81$

$G = 0.925\left(\dfrac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right)$ $g_v = g_Q = 3.4$

$= 0.925\left(\dfrac{1 + 1.7(3.4)(0.795)(0.81)}{1 + 1.7(3.4)(0.795)}\right)$

$G = 0.78$

$q_z = 0.00256\,(K_{zt} K_z K_d V^2 I)$
$= 0.00256\,(100)^2(1.0)(0.85)(1.0) K_z$

$q_z = 21.76 K_z$

$V = 100$ MPH
$K_d = 0.85$
CAT II : $I = 1.0$
K_z = VARIES

$GC_{pi} = \pm 0.18$
WINDWARD $C_p = 0.8$
LEEWARD C_p (N-S) : $L/B = 112/164 = 0.68$ $C_p = -0.5$
LEEWARD C_p (E-W) : $L/B = 164/112 = 1.46$ $C_p = -0.3$

$P = qGC_p - q_i(GC_{pi})$

* SEE SPREADSHEET FOR THE REST OF WIND

SEISMIC

SEISMIC USE GROUP I

$I = 1.0$

SITE CLASS "D"

$S_1 = 9.5 = 0.095$ $\quad$ $S_S = 43 = 0.43$

$F_V = 2.4$ $\quad$ $F_A = 1.456$

$S_{M1} = F_V S_1 = (2.4)(0.095)$ $\quad$ $S_{MS} = F_A S_S = (1.456)(0.43)$

$S_{M1} = 0.228$ $\quad$ $S_{MS} = 0.626$

$S_{D1} = 2/3\, S_{M1} = 2/3(0.228)$ $\quad$ $S_{DS} = 2/3\, S_{MS} = 2/3(0.626)$

$S_{D1} = 0.152$ $\quad$ $S_{DS} = 0.417$

$W_{roof} = (\underset{DL}{60\,\text{PSF}} + \underset{SNOW}{18.9\,\text{PSF}})(7045\,ft^2) = 556^k$

$W_{14-15} = (60\,\text{PSF})(7045\,ft^2) = 423^k$

$W_{8-13} = (60\,\text{PSF})(9343\,ft^2) = 561^k$

$W_7 = (60\,\text{PSF})(9226\,ft^2) = 554^k$

$W_{5-6} = (60\,\text{PSF})(9483\,ft^2) = 569^k$

$W_{3-4} = (60\,\text{PSF})(13994\,ft^2) = 840^k$

$W_2 = (60\,\text{PSF})(14516\,ft^2) = 871^k$

$W_1 = (60\,\text{PSF})(15174\,ft^2) = 910^k$

$W_{TOTAL} = 9921^k$

$R = 4.0$ STEEL CONCENTRICALLY BRACED FRAMES

$T = C_t h_n^x = (0.03)(207+15)^{0.75}$

$T = 1.725$ SEC

$C_S = \dfrac{S_{DS}}{R/I} = \dfrac{0.417}{4.0} = 0.104$ BUT NOT MORE THAN

$C_{SMAX} = \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.152}{(1.725)(4)} = 0.022$ AND NOT LESS THAN

$C_{SMIN} = 0.044\, I\, S_{DS} = 0.044(1.0)(0.417) = 0.018$

$C_S = 0.022$

SEISMIC CONT'

$V = C_s W$
$= (0.022)(9921)$

V = BASE SHEAR = 218^K

$K = 1 + \frac{(1.725 - 0.5)}{2}$

$K = 1.61$

SEE SPREADSHEET FOR THE REST OF SEISMIC

ADOSS Output

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN.
Page   1
6:08:47 PM Licensed to: ae, university park, PA

```
                pppppp    ccccc    aaaaa
                p     p  c     c  a     a
                p     p  c     c        a
                p     p  c         aaaaaa
                p     p  c     c  a     a
                p     p  c     c  a     a
                pppppp    ccccc    aaaaaa
                p
                p


         AAA    DDDDD      OOO     SSSSS    SSSSS
        A   A   D    D    O   O   S     S  S     S
       A     A  D     D  O     O  S        S
       AAAAAAA  D     D  O     O   SSSSS    SSSSS
       A     A  D     D  O     O        S        S   ( ttttt mm   mm  )
       A     A  D    D    O   O   S     S  S     S   (   t   m m m m  )
       A     A  DDDDD      OOO     SSSSS    SSSSS    (   t   m  m  m  )


*********************************************************

 Computer program for ANALYSIS AND DESIGN OF SLAB SYSTEMS

*********************************************************
```

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CEMENT ASSN. Page   2
 6:08:47 PM Licensed to: ae, university park, PA

FILE NAME            P:\THESIS\ADOSS\ADOSSF~1\GROUND\SLABS\13.ADS

PROJECT ID.          Ground
                     -----------------------------------
SPAN ID.             10.0 11.0
                     -------------------

ENGINEER             Aric Heffelfinger

DATE                 02/15/06
TIME                 14:10:35

UNITS                U.S. in-lb
CODE                 ACI 318-89

SLAB SYSTEM          FLAT SLAB SYSTEM
FRAME LOCATION       INTERIOR

DESIGN METHOD        STRENGTH DESIGN
MOMENTS AND SHEARS   NOT PROPORTIONED

NUMBER OF SPANS   7

  SOLID HEAD DIMENSIONS :       COMPUTED BY PROGRAM


CONCRETE FACTORS       SLABS         BEAMS         COLUMNS
  DENSITY(pcf  )       150.0         150.0         150.0
  TYPE              NORMAL WGT    NORMAL WGT    NORMAL WGT
  f'c     (ksi)          4.0           4.0           4.0
  fct     (psi)        423.7         423.7         423.7
  fr      (psi)        474.3         474.3         474.3


REINFORCEMENT DETAILS: NON-PRESTRESSED
  YIELD STRENGTH Fy  =  60.00 ksi
  DISTANCE TO RF CENTER FROM TENSION FACE:
       AT SLAB TOP    =   1.50 in  OUTER LAYER
       AT SLAB BOTTOM =   1.50 in  OUTER LAYER
  MINIMUM FLEXURAL BAR SIZE:
       AT SLAB TOP    =  # 4
       AT SLAB BOTTOM =  # 4
```

MINIMUM SPACING:
IN SLAB = 6.00 in

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN. Page 3
6:08:48 PM Licensed to: ae, university park, PA

SPAN/LOADING DATA
****

SPAN NUMBER	LENGTH L1 (ft)	Tslab (in)	WIDTH LEFT (ft)	L2*** RIGHT (ft)	SLAB SYSTEM	DESIGN STRIP (ft)	COLUMN STRIP** (ft)	UNIFORM LOADS S. DL (psf)	UNIFORM LOADS LIVE (psf)
1*	2.0	10.0	14.0	14.0	2	28.0	.0	30.0	80.0
2	25.3	10.0	14.0	14.0	2	28.0	12.6	30.0	80.0
3	27.8	10.0	14.0	14.0	2	28.0	13.9	30.0	80.0
4	27.8	10.0	14.0	14.0	2	28.0	13.9	30.0	80.0
5	27.8	10.0	14.0	14.0	2	28.0	13.9	30.0	80.0
6	22.0	10.0	14.0	14.0	2	28.0	11.0	30.0	80.0
7*	2.0	10.0	14.0	14.0	2	28.0	.0	30.0	80.0

* -Indicates cantilever span information.
** -Strip width used for positive flexure.
***-L2 widths are 1/2 dist. to transverse column.
"E"-Indicates exterior strip.

PARTIAL LOADING DATA

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Page 4
6:08:48 PM Licensed to: ae, university park, PA

COLUMN/TORSIONAL DATA
****

COLUMN NUMBER	COLUMN ABOVE SLAB C1 (in)	C2 (in)	HGT (ft)	COLUMN BELOW SLAB C1 (in)	C2 (in)	HGT (ft)	CAPITAL** EXTEN. (in)	DEPTH (in)	COLUMN STRIP* (ft)	MIDDLE STRIP* (ft)
1	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	12.6	15.4
2	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	12.6	15.4
3	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	13.9	14.1
4	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	13.9	14.1
5	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	11.0	17.0
6	26.0	26.0	10.0	26.0	26.0	15.5	.0	.0	11.0	17.0

Columns with zero "C2" are round columns.
* -Strip width used for negative flexure.
**-Capital extension distance measured from face of column.

COLUMN NUMBER	TRANSVERSE BEAM WIDTH (in)	DEPTH (in)	ECCEN (in)	DROP PANEL/SOLID HEAD LEFT (ft)	RIGHT (ft)	WIDTH (ft)	THICK (in)	SUPPORT FIXITY* %

```
|    1   |    .0    .0    .0 |     2.0       4.2       9.3    5.5 |
100%  |
|    2   |    .0    .0    .0 |     4.2       4.6       9.3    5.5 |
100%  |
|    3   |    .0    .0    .0 |     4.6       4.6       9.3    5.5 |
100%  |
|    4   |    .0    .0    .0 |     4.6       4.6       9.3    5.5 |
100%  |
|    5   |    .0    .0    .0 |     4.6       3.7       9.3    5.5 |
100%  |
|    6   |    .0    .0    .0 |     3.7       2.0       9.3    5.5 |
100%  |
|________|___________________|____________________________________|____
______|

* -Support fixity of 0% denotes pinned condition.
   Support fixity of 999% denotes fixed end condition.

02-20-** ADOSS(tm) 6.01  Proprietary Software of PORTLAND CEMENT ASSN.
Page   5
 6:08:48 PM Licensed to: ae, university park, PA

                           LATERAL LOAD/OUTPUT DATA
                           ************************

LATERAL LOADS ARE NOT SPECIFIED



OUTPUT DATA
PATTERN LOADINGS:                1 THRU 4
PATTERN LIVE LOAD FACTOR (1-3)  =    75%

LOAD FACTORS:
  U = 1.20*D + 1.60*L
  U =  .75( 1.20*D + 1.60*L + 1.70*W)
  U =  .90*D + 1.30*W


OUTPUT OPTION(S):
  Input Echo
  Centerline Moments and Shears
  Column Strip Distribution Fac
  Shear Table
  Reinforcing Required
  Bar Sizing
  Additional Information
  Deflections
  Material Quantities
```

**TOTAL UNFACTORED DEAD LOAD = 608.752 kips
LIVE LOAD = 301.280 kips

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN. Page 6
6:08:48 PM Licensed to: ae, university park, PA

---- STATICS PRINT-OUT FOR GRAVITY LOAD ANALYSIS ----

J O I N T   M O M E N T S (ft - kips)

JOINT NUMBER	PATTERN-1 LEFT	RIGHT	TOP	BOTTOM	PATTERN-2 LEFT	RIGHT	TOP	BOTTOM
1	-17.0	323.8	-182.7	-124.1	-17.0	175.4	-94.3	-64.1
2	-600.4	611.0	-6.3	-4.3	-453.7	564.6	-66.0	-44.8
3	-614.5	614.1	.2	.2	-546.1	415.6	77.7	52.8
4	-619.4	625.1	-3.4	-2.3	-420.6	556.4	-80.9	-55.0
5	-577.7	504.5	43.5	29.6	-544.2	395.4	88.6	60.2
6	-226.3	17.0	124.6	84.6	-108.6	17.0	54.5	37.0

JOINT NUMBER	PATTERN-3 LEFT	RIGHT	TOP	BOTTOM	PATTERN-4 LEFT	RIGHT	TOP	BOTTOM
1	-11.7	326.4	-187.4	-127.3	-18.8	335.2	-188.4	-128.0
2	-529.3	410.9	70.5	47.9	-657.0	651.9	3.0	2.1
3	-420.9	550.1	-76.9	-52.2	-646.2	645.3	.6	.4
4	-558.9	439.2	71.3	48.4	-654.6	665.3	-6.4	-4.4
5	-375.0	426.5	-30.7	-20.8	-614.3	549.3	38.7	26.3
6	-228.9	11.7	129.4	87.9	-225.4	18.8	123.0	83.6

J O I N T   S H E A R S (kips)

JOINT PATTERN-1 PATTERN-2 PATTERN-3 PATTERN-4

NUMBER	LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	-16.8	93.9	-16.8	57.4	-11.4	94.3	-18.6	100.9
2	-114.4	115.5	-79.5	113.2	-110.4	74.9	-126.4	125.2
3	-115.6	115.5	-111.9	75.1	-75.6	112.3	-124.8	124.7
4	-115.9	116.7	-75.5	113.0	-112.9	77.6	-125.3	126.8
5	-113.7	103.1	-112.1	72.6	-73.0	98.1	-123.2	113.7
6	-79.6	16.8	-46.5	16.8	-80.2	11.4	-84.3	18.6

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN. Page 7
6:08:48 PM Licensed to: ae, university park, PA

DESIGN MOMENT ENVELOPES AT CRITICAL SECTIONS FROM SUPPORTS
****

COL NUM	LOAD TYPE	CROSS SECTN	DESIGN MOMENT (ft-k)	DISTANCE CR.SECTN (ft)	LOAD PTRN	MAX.I.P. DISTANCE (ft)	LOAD PTRN
1	TOTL LEFT	TOP	-12.9	.350	4	2.000	1
		BOT	.0	.000	0	.000	0
	RGHT	TOP	231.2	1.083	4	3.787	2
		BOT	.0	.000	0	.000	0
2	TOTL LEFT	TOP	-525.4	1.083	4	7.575	2

			BOT	.0	.000	0	.000	0
		RGHT	TOP	521.7	1.083	4	8.325	3
			BOT	.0	.000	0	.000	0
3	TOTL	LEFT	TOP	-516.4	1.083	4	8.325	3
			BOT	.0	.000	0	.000	0
		RGHT	TOP	515.6	1.083	4	8.325	2
			BOT	.0	.000	0	.000	0
4	TOTL	LEFT	TOP	-524.2	1.083	4	8.325	2
			BOT	.0	.000	0	.000	0
		RGHT	TOP	533.3	1.083	4	8.325	3
			BOT	.0	.000	0	.000	0
5	TOTL	LEFT	TOP	-486.3	1.083	4	6.938	2
			BOT	.0	.000	0	.000	0
		RGHT	TOP	431.5	1.083	4	7.700	2
			BOT	.0	.000	0	.000	0
6	TOTL	LEFT	TOP	-146.9	1.083	3	3.300	2
			BOT	.0	.000	0	.000	0
		RGHT	TOP	12.9	.350	4	2.000	1
			BOT	.0	.000	0	.000	0

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN. Page 8
6:08:48 PM Licensed to: ae, university park, PA

DESIGN MOMENT ENVELOPES AT CRITICAL SECTIONS ALONG SPANS
****

SPAN NUM	LOAD TYPE	CRITICAL SECTION (ft)		DESIGN MOMENT (ft-k)	LOAD PTRN	MAX. I.P. DIST LEFT (ft)	LOAD PTRN	MAX. I.P. DIST RGHT (ft)	LOAD PTRN
2	TOTL	10.731	TOP	.0	0	.000	0	.000	0
			BOT	221.3	4	6.944	1	8.206	3
3	TOTL	14.569	TOP	.0	0	.000	0	.000	0
			BOT	213.2	2	7.631	1	6.244	1
4	TOTL	13.181	TOP	.0	0	.000	0	.000	0
			BOT	213.8	3	6.244	1	7.631	1
5	TOTL	14.569	TOP	.0	0	.000	0	.000	0
			BOT	218.0	2	7.631	1	6.244	1
6	TOTL	12.650	TOP	.0	0	.000	0	.000	0
			BOT	163.2	4	7.150	3	6.050	1

02-20-** ADOSS(tm) 6.01 Proprietary Software of PORTLAND CEMENT ASSN. Page 9
6:08:48 PM Licensed to: ae, university park, PA

DISTRIBUTION OF DESIGN MOMENTS AT SUPPORTS

COL NUM	CROSS SECTN		TOTAL MOMENT (ft-k)	TOTAL-VERT DIFFERENCE (ft-k) (%)	COLUMN STRIP MOMENT (ft-k) (%)	BEAM MOMENT (ft-k) (%)	MIDDLE STRIP MOMENT (ft-k) (%)
1	LEFT	TOP	-12.9	.0 (0)	-12.4 (96)	.0 (0)	-.5 (3)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT	TOP	231.2	.0 (0)	222.9 (96)	.0 (0)	8.3 (3)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
2	LEFT	TOP	-525.4	.0 (0)	-394.0 (75)	.0 (0)	-131.3 (25)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT	TOP	521.7	.0 (0)	391.2 (75)	.0 (0)	130.4 (25)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
3	LEFT	TOP	-516.4	.0 (0)	-387.3 (75)	.0 (0)	-129.1 (25)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)
	RGHT	TOP	515.6	.0 (0)	386.7 (75)	.0 (0)	128.9 (25)
		BOT	.0	.0 (0)	.0 (0)	.0 (0)	.0 (0)

```
   4  LEFT TOP  -524.2      .0 (   0)  -393.1 ( 75)      .0 (   0)  -
131.0 ( 25)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

      RGHT TOP   533.3      .0 (   0)   400.0 ( 75)      .0 (   0)
133.3 ( 25)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

   5  LEFT TOP  -486.3      .0 (   0)  -364.7 ( 75)      .0 (   0)  -
121.6 ( 25)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

      RGHT TOP   431.5      .0 (   0)   323.6 ( 75)      .0 (   0)
107.9 ( 25)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

   6  LEFT TOP  -146.9      .0 (   0)  -141.6 ( 96)      .0 (   0)    -
5.3 (   3)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

      RGHT TOP    12.9      .0 (   0)    12.4 ( 96)      .0 (   0)
.5 (   3)
           BOT      .0      .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)

-----------------------------------------------------------------------
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02-20-** ADOSS(tm) 6.01  Proprietary Software of PORTLAND CEMENT ASSN.
Page  10
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                    DISTRIBUTION OF DESIGN MOMENTS IN SPANS
                    ***************************************
-----------------------------------------------------------------------
---------
 SPAN CROSS     TOTAL    TOTAL-VERT   COLUMN STRIP     BEAM
MIDDLE STRIP
 NUM  SECTN     MOMENT   DIFFERENCE      MOMENT       MOMENT
MOMENT
                (ft-k)  (ft-k) ( % )  (ft-k) ( % )  (ft-k) ( % )  (ft-
k) ( % )
```

```
--------------------------------------------------------------------------
---------
  2  10.73 TOP      .0       .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)
           BOT   221.3       .0 (   0)   132.8 ( 60)      .0 (   0)
88.5 ( 39)

  3  14.57 TOP      .0       .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)
           BOT   213.2       .0 (   0)   127.9 ( 60)      .0 (   0)
85.3 ( 39)

  4  13.18 TOP      .0       .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)
           BOT   213.8       .0 (   0)   128.3 ( 60)      .0 (   0)
85.5 ( 39)

  5  14.57 TOP      .0       .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)
           BOT   218.0       .0 (   0)   130.8 ( 60)      .0 (   0)
87.2 ( 40)

  6  12.65 TOP      .0       .0 (   0)      .0 (   0)      .0 (   0)
.0 (   0)
           BOT   163.2       .0 (   0)    97.9 ( 60)      .0 (   0)
65.3 ( 39)

--------------------------------------------------------------------------
---------
02-20-** ADOSS(tm) 6.01  Proprietary Software of PORTLAND CEMENT ASSN.
Page  11
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                          S H E A R   A N A L Y S I S
                          ***************************

    NOTE--Allowable shear stress in slabs = 252.96 psi when ratio
          of col. dim. (long/short) is less than 2.0.

        --Wide beam shear (see "CODE") is not computed, check manually.

        --After the column numbers, C = Corner, E = Exterior, I =
Interior.

  D I R E C T   S H E A R     W I T H   T R A N S F E R   O F   M O M
E N T
  - - - - - - - - - - -   A R O U N D       C O L U M N   - - - - - - -
- - -
 COL. ALLOW.  PATT  REACTION  SHEAR   PATT  REACTION  UNBAL.  SHEAR
SHEAR
```

NO.	STRESS (psi)	NO.	(kips)	STRESS (psi)	NO.	(kips)	MOMENT (ft-k)	TRANSFR (ft-k)	STRESS (psi)
1E	252.96	4	115.7	81.04	4	115.7	250.0	100.0	140.42
2I	252.96	4	248.3	130.43	4	248.3	-5.1	-2.0	131.36
3I	252.96	4	246.2	129.28	4	246.2	-.9	-.4	129.45
4I	252.96	4	248.9	130.71	4	248.9	10.8	4.3	132.68
5I	252.96	4	233.6	122.67	4	233.6	-65.0	-26.0	134.59
6E	252.96	4	99.0	69.36	4	99.0	-149.5	-59.8	104.87

- - AROUND DROP/SOLID HEAD - -

COLUMN NUMBER	ALLOW. STRESS (psi)	PATT NO.	REACTION (kips)	SHEAR STRESS (psi)
1E	184.48	4	94.8	47.18
2I	172.22	4	215.5	63.45
3I	171.27	4	211.7	61.03
4I	171.27	4	214.4	61.81
5I	173.53	4	202.8	61.43
6E	187.33	4	80.2	41.89

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N E G A T I V E   R E I N F O R C E M E N T
****

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	4	|| R	231.2	3.84	12.6	3.32	15.4
2	4	L ||	-525.4	6.49	12.6	3.50	15.4
3	4	L ||	-516.4	6.38	13.9	3.45	14.1
4	4	|| R	533.3	6.59	13.9	3.56	14.1
5	4	L ||	-486.3	5.99	11.0	3.67	17.0
6	3	L ||	-146.9	3.48	11.0	3.67	17.0

P O S I T I V E   R E I N F O R C E M E N T

****

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	10.7	221.3	3.56	12.6	3.32	15.4
3	2	14.6	213.2	3.42	13.9	3.05	14.1
4	3	13.2	213.8	3.43	13.9	3.05	14.1
5	2	14.6	218.0	3.50	13.9	3.05	14.1
6	4	12.6	163.2	2.61	11.0	3.67	17.0

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Page 13
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D E S I G N   R E S U L T S

****

NOTE--The schedule given below is a guide for proper reinforcement placement and is based on reasonable engineering judgement. Unusual boundary and/or loading conditions may require modification of this schedule.

N E G A T I V E   R E I N F O R C E M E N T

****

COLUMN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)
1	10	# 4	2.00	8.70	9	# 4	2.00	5.70	17	# 4	2.00	6.16
2	11	# 5	9.53	9.92	10	# 5	6.20	6.20	18	# 4	9.17	9.92
3	11	# 5	9.92	9.92	10	# 5	6.20	6.20	18	# 4	9.92	9.92
4	11	# 5	9.92	9.92	11	# 5	6.20	6.20	18	# 4	9.92	9.92
5	7	# 6	9.53	9.53	7	# 6	6.20	6.20	19	# 4	8.54	9.30

```
    6       9   # 4  7.63   2.00    9   # 4  5.05   2.00   19   # 4
5.45   2.00


                  P O S I T I V E   R E I N F O R C E M E N T
                  ********************************************

        *    C O L U M N      S T R I P    *    M I D  D L E     S T
R I P
        *   LONG   BARS   *   SHORT  BARS   *   LONG   BARS   *   SHORT
BARS
 SPAN   * ---- B A R ---- * ---- B A R ---- * ---- B A R ---- * ---- B
A R ----
 NUMBER * NO  SIZE LENGTH * NO  SIZE LENGTH * NO  SIZE LENGTH * NO
SIZE LENGTH
        *           (ft)  *           (ft)  *           (ft)  *
(ft)
 -----------------------------------------------------------------------
---------
    2       9   # 4  21.51    9   # 4  21.51    9   # 4  24.92    8   #
4  20.88
    3       9   # 4  20.81    8   # 4  20.81    8   # 4  28.25    8   #
4  19.42
    4       9   # 4  20.81    8   # 4  20.81    8   # 4  28.25    8   #
4  19.42
    5       9   # 4  20.81    9   # 4  20.81    8   # 4  28.25    8   #
4  19.42
    6       7   # 4  18.75    6   # 4  18.75   10   # 4  21.67    9   #
4  18.12


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Page  14
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       A D D I T I O N A L   I N F O R M A T I O N   A T  S U P P O R T
S

*******************************************************************

        * REINF. SUMMARY* ADD'L R/F REQ'D DUE TO UNBALANCED (U.) MOMENT
TRANSFER
 COLUMN * --------------* ----------------------------------------------
---------
 NUMBER * W/O U. MOMENT * MAX.U. *GAMMA* FLEXURAL  *PATT* CRITICAL
SECTION
        * REQ'D - PROV'D* MOMENT * -f  * TRANSFER  *NO. * SLABW  - AREA
-  R/F
        *(sq.in) (sq.in)* (ft-k) *     *  (ft-k) *    * (ft)   (sq.in)
```

```
--------------------------------------------------------------------------
--------
   1       7.16     7.20   316.4     .60    189.8      4      6.0    3.10
7  # 4
   2       9.99    10.11  -118.4     .60    -71.1      3      6.0    1.14
0  # 5
   3       9.82    10.11  -130.5     .60    -78.3      2      6.0    1.26
0  # 5
   4      10.16    10.42   135.9     .60     81.5      2      6.0    1.31
0  # 5
   5       9.66     9.96  -148.8     .60    -89.3      2      6.0    1.44
0  # 6
   6       7.16     7.40  -217.2     .60   -130.3      3      6.0    2.11
2  # 4
```

NOTE: Zero transfer "CRITICAL SLABW" indicates no support dimensions given for transfer.
If beam(s) are present, transfer mode may be due to beam shear and/or torsion, check manually.

A D D I T I O N A L   I N F O R M A T I O N   F O R
I N - S P A N   C O N D I T I O N S

****

SPAN NUMBER	REINF. SUMMARY AT MIDSPAN REQ'D. (sq.in)	REINF. SUMMARY AT MIDSPAN PROV'D. (sq.in)	TOTAL FACTORED SPAN STATIC DESIGN MOMENT (W/O PARTIAL LOADS) (ft-k)
2	6.88	7.00	585.6
3	6.47	6.60	719.3
4	6.48	6.60	719.3
5	6.55	6.80	719.3
6	6.29	6.40	432.3

D E F L E C T I O N   A N A L Y S I S

****

NOTES--The deflections below must be combined with those of the analysis in the perpendicular direction. Consult users manual for method of combination and limitations.

--Spans 1 and 7 are cantilevers.

--Time-dependent deflections are in addition to those

shown and must be computed as a multiplier of the dead load(DL) deflection. See "CODE" for range of multipliers.

--Deflections due to concentrated or partialloads may be larger at the point of application than those shown at the centerline. Deflections are computed as from an average uniform loading derived from the sum of all loads applied to the span.

--Modulus of elasticity of concrete, Ec = 3834. ksi

SPAN NUMBER	DEAD LOAD Ieff. (in^4)	COLUMN STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)	MIDDLE STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)
1	60819.	-.013	-.007	-.019	-.013	-.007	-.020
2	44410.	.099	.070	.169	.045	.029	.074
3	44410.	.096	.105	.201	.045	.049	.095
4	44410.	.096	.105	.200	.045	.049	.094
5	44410.	.102	.103	.205	.051	.050	.101
6	44410.	.060	.034	.093	.020	.011	.032
7	60819.	-.008	-.004	-.013	-.008	-.004	-.013

Q U A N T I T Y   E S T I M A T E S
****

TOTAL QUANTITIES

CONCRETE	....	123.8 cu.yd
FORMWORK	....	3861. sq.ft
REINFORCEMENT (IN THE DIRECTION OF ANALYSIS)		
(NEGATIVE)	....	2749. lbs
(POSITIVE)	....	2424. lbs

```
SUMMARY OF QUANTITIES
---------------------
CONCRETE         ....        .89  cu.ft/sq.ft
FORMWORK         ....       1.03  sq.ft/sq.ft
REINFORCEMENT** ....       1.37  lbs / sq.ft

  **(IN THE DIRECTION OF ANALYSIS)

             * Program completed as requested *
```

PCA COL OutPut

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Page 1
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```
========================================================================
=
    Computer program for the Strength Design of Reinforced Concrete
Sections

========================================================================
=
```

Licensee stated above acknowledges that Portland Cement Association(PCA) is not and cannot be responsible for either the accuracy or adequacy of the material supplied as input for processing by the PCACOL(tm) computer program. Furthermore, PCA neither makes any warranty expressed nor implied with respect to the correctness of the output prepared by the PCACOL(tm) program. Although PCA has endeavored to produce PCACOL(tm) error free, the program is not and can't be certified infallible. The final and only responsibility for analysis, design and engineering documents is the licensees. Accordingly, PCA disclaims all responsibility in contract, negligence or other tort for any analysis, design or engineering documents prepared in connection with the use of the PCACOL(tm) program.

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02/22/06 PCACOL(tm)V2.30 Proprietary Software of PORTLAND CEMENT ASSN.
Page 2
15:28:25 Licensed to: ae, university park, PA


       General Information:
       ====================
         File Name:   P:\4KSI2011.COL
         Project:     Ground                   Code: ACI 318-89
         Column:      lSpan 3, Column Above    Units: US in-lbs
         Engineer:    Aric Heffelfinger        Date: 02/22/06  Time:
15:23:28

         Run Option: Design                    Short (nonslender) column
         Run Axis:   Biaxial                   Column Type: User-defined

       Material Properties:
       ====================
         f'c   = 4 ksi                              fy   = 60 ksi
         Ec    = 3834.25 ksi                        Es   = 29000 ksi
         fc    = 3.4 ksi                            erup = 0 in/in
         eu    = 0.003 in/in
         Stress Profile: Parabolic

       Geometry:
       =========
         Rectangular: Width = 26 in                 Depth = 26 in

         Gross section area, Ag =  676 in^2
         Ix =  38081.3 in^4                         Xo =  0 in
         Iy =  38081.3 in^4                         Yo =  0 in

       Reinforcement:
       ==============
         Rebar Database: ASTM
         Size     Diam     Area  Size     Diam     Area  Size     Diam
Area
         -------------------------------------------------------------
-----
            3     0.38     0.11     4     0.50     0.20     5     0.63
0.31
            6     0.75     0.44     7     0.88     0.60     8     1.00
0.79
            9     1.13     1.00    10     1.27     1.27    11     1.41
1.56
           14     1.69     2.25    18     2.26     4.00

         Confinement: User-defined; phi(c) = 0.7,  phi(b) = 0.9,  a =
0.8
```

#3 ties with #10 bars, #4 with larger bars.

Layout: Rectangular
Pattern: All Sides Equal [Cover to transverse reinforcement (ties)]

Total steel area, As = 31.20 in^2 at 4.62%

20-#11 Cover = 0.75 in

02/22/06 PCACOL(tm)V2.30 Proprietary Software of PORTLAND CEMENT ASSN. Page 3
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	Applied Loads			Computed Strength			Computed/
Pt.	P (kips)	Mx (ft-k)	My (ft-k)	P (kips)	Mx (ft-k)	My (ft-k)	Applied Ray length
1	2500	120	50	2276	108	45	0.911

Program completed as requested!

Seismic Calculations (All Concrete Structure)

Self Weight

	slab volume	drop volume	column volume	Shear wall volume	Weight
Roof	3680	0	507	911.25	764.7375
15th	5520	458.333333	1014	1822.5	1322.225
14th	7086	733.333333	1267.5	2187	1691.075
13th	7086	733.333333	1267.5	2187	1691.075
12th	7086	733.333333	1267.5	2187	1691.075
11th	7086	733.333333	1267.5	2187	1691.075
10th	7086	733.333333	1267.5	2187	1691.075
9th	7086	733.333333	1267.5	2187	1691.075
8th	7086	733.333333	1267.5	2187	1691.075
7th	7086	733.333333	1267.5	2187	1691.075
6th	7086	733.333333	1267.5	2187	1691.075
5th	10185	1100	1774.5	2187	2286.975
4th	10185	1100	1774.5	2187	2286.975
3rd	11750.25	1283.33333	1964.625	2187	2577.781
2nd	11175	1191.66667	1964.625	2187	2477.744
Mezz.	2651.25	320.833333	1964.625	2187	1068.556
Ground	11750.25	1283.33333	1964.625	2187	0
					28004.67

Seismic Analysis

Assumptions:
Occupancy Category I (Table 1-1)
Seismic Use Group I (Table 9.1.3)
Importance Factor = 1.0 (Table 9.1.4)
Site Class D (Table 9.4.1.2)
Ordinary Reinforced Concrete Shear Walls

Ss = 0.43 (Figure 9.4.1.1a)
S1 = 0.095 (Figure 9.4.1.1b)

Sms = 0.626
Sm1 = 0.228

Sds = 0.417
Sd1 = 0.152

T = 1.07
Cs = 0.03551

Ss = 0.43 (Figure 9.4.1.1a)
S1 = 0.095 (Figure 9.4.1.1b)

Sms = 0.626
Sm1 = 0.228

Sds = 0.417
Sd1 = 0.152

T = 1.07
Cs = 0.03551

Seismic Design Category B

Effective Seismic Weight of Structure (9.5.3)

W_{TOTAL} = 28004 k

Seismic Base Shear (9.5.5.2)

$V = C_sW$

V = 994 k

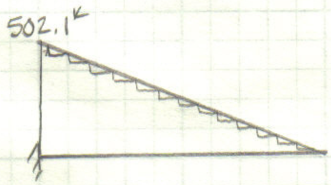
Level	w_x (k)	h_x	$w_x h_x^k$	C_{vx}	F_x (k)
B	0	0	0	0	0
Mezz.	1068	14.5	15486	0.00492	5
2	2477	34.25	84837.25	0.026953	27
3	2577	50	128850	0.040936	41
4	2286	63.75	145732.5	0.046299	46
5	2286	77.5	177165	0.056285	56
6	1691	91	153881	0.048888	49
7	1691	104.5	176709.5	0.056141	56
8	1691	117	197847	0.062856	63
9	1691	129.5	218984.5	0.069572	69
10	1691	142	240122	0.076287	76
11	1691	154.5	261259.5	0.083002	83
12	1691	167	282397	0.089718	89
13	1691	179.5	303534.5	0.096433	96
14	1691	192	324672	0.103149	103
15	1322	204.5	270349	0.08589	85
roof	764	217	165788	0.052671	52
Σ=	27999	Σ=	3147615		

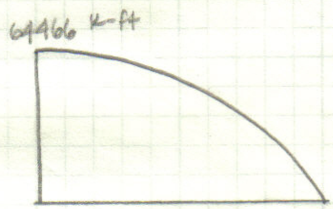
Shear wall Design

SHEAR WALL DESIGN

$V_u = 502.1^k$

$M_u = 64466$ k-ft





SHEAR DESIGN

$V_u \leq \phi V_n = \phi(V_c + V_s)$

$\frac{V_u}{\phi} \leq V_c + V_s$

$\frac{502.1}{0.75} \leq V_c + V_s$

$V_c = 2\sqrt{f'_c}\ hd$

$= 2\sqrt{4000}\,(8")(0.8)(27)(12)$

$V_c = 262.3^k$

$V_s \geq 669.5^k - 262.3^k$

$V_s \geq 407.2^k$

$V_s \geq 407.2^k = \dfrac{A_v f_y d}{s}$

$\dfrac{(407.2^k)(12'')}{(60)(27)(0.8)(12)} = A_v$

$A_v = 0.31 \text{ in}^2/\text{ft}$

USE #5's @ 12"

FLEXURAL DESIGN

$M_u = 64466^{k\text{-}ft} = A_s f_y (d - a/2)$

$A_s = 53.7 \text{ in}^2$

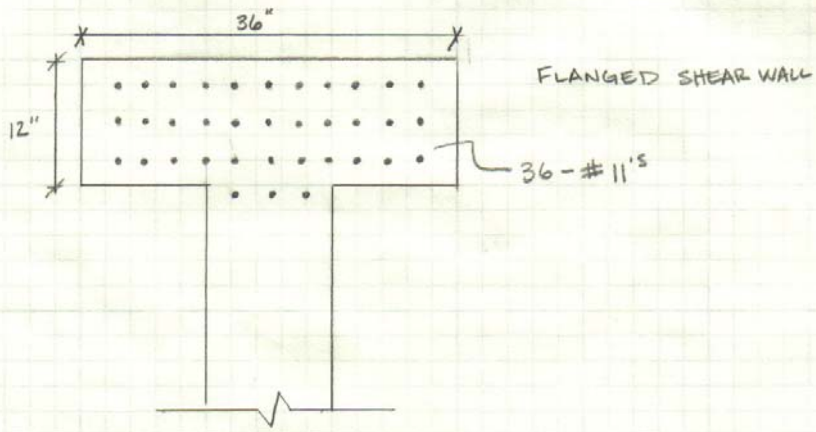
36 - #11's

$A_s = 56.2 \text{ in}^2$

$A_s f_y = 0.85 f'_c \, b \, a$

$(56.2 \text{ in}^2)(60) = 0.85(4)(12)\, a$

$a = 82.6 \text{ in} = 6.9 \text{ ft}$



**** Shear wall load distribution table can be viewed as an excel spreadsheet on my webpage. It is too large to fit on an 8.5" x 11" piece of paper.

Concrete structure cost breakdown

EDF Report - Standard Construction Project

Estimator : User
Project Size : sqft
Date : 3/28/2006
Time : 02:12 PM

Group 1: Divisions
Group 2: Major ItemCode Groups
Group 3: Minor ItemCode Groups
Group 4: Alternates

ItemCode	Description	Quantity	UM	Labor$	MH/Unit	Units/MH
Concrete						
Structural CIP forms						
Structural CIP forms						
Alternates Blank						
03111.118	WALL FORM 20'+ HIGH	72,912.00	SQFT	3.8747	0.147947	6.75918
03111.189	WALL FORM HARDWARE	36,456.00	SQFT			
03111.203	WOOD COLUMN FORMS, 12'-16'	44,165.33	SQFT	1.1524	0.044	22.72727
03111.612	SLAB FORM W/2.6 BM/SF	176,587.00	SQFT	2.5380	0.096904	10.3195
	**** Total Alternates Blank				$1,179,222.01	
	*** Total Structural CIP forms				$1,179,222.01	
	** Total Structural CIP forms				$1,179,222.01	
Concrete accessories						
Concrete accessories						
Alternates Blank						
03150.650	SCREEDS FOR SLAB	21,190.44	LNFT	0.9219	0.0352	28.40909
03150.900	FORM RELEASING AGENT	117,077.33	SQFT	0.2095	0.008	125.00
03150.900	FORM RELEASING AGENT	176,587.00	SQFT	0.2095	0.008	125.00
	**** Total Alternates Blank				$94,593.36	
	*** Total Concrete accessories				$94,593.36	
	** Total Concrete accessories				$94,593.36	
Reinforcing steel						
Reinforcing steel						
Alternates Blank						
03210.130	SUPPORTED SLAB REBAR	5,886.23	CWT	32.3636	1.018182	0.98214
03210.150	COLUMN REBAR	3,322.62	CWT	24.7222	0.777778	1.28571
	**** Total Alternates Blank				$518,979.18	
	*** Total Reinforcing steel				$518,979.18	
	** Total Reinforcing steel				$518,979.18	
Structural concrete						
Structural concrete						
Alternates Blank						
03310.550	**CONCRETE IN WALLS**	****				
03310.576	4000 PSI W/CRANE	1,350.22	CUYD	16.5977	0.685714	1.45833
03310.650	**CONCRETE IN COLUMNS**	****				
03310.676	4000 PSI W/CRANE	886.03	CUYD	21.7845	0.90	1.11111
03311.500	**C0NC IN SUPPORTED SLAB**	****				
03311.526	4000 PSI W/CRANE	4,905.19	CUYD	13.9420	0.576	1.73611
03315.982	* CONCRETE WALL AREA *	36,456.00	SQFT			
03315.984	* NO. OF COLUMNS *	392.00	EACH			
03315.986	* SUPPORTED SLAB AREA *	176,587.00	SQFT			
	**** Total Alternates Blank				$510,021.77	
	*** Total Structural concrete				$510,021.77	
	** Total Structural concrete				$510,021.77	
Finishing						
Finishing						
Alternates Blank						
03350.130	MACHINE TROWEL FINISH	176,587.00	SQFT	0.3304	0.0128	78.125
03350.131	POINT & PATCH	117,077.33	SQFT	0.1102	0.004267	234.375
03350.131	POINT & PATCH	176,587.00	SQFT	0.1102	0.004267	234.375
	**** Total Alternates Blank				$94,465.06	
	*** Total Finishing				$94,465.06	
	** Total Finishing				$94,465.06	

P:\THESIS\CE Estimating\Construction durations.est

Page 1

EDF Report - Standard Construction Project

Estimator : User
Project Size : sqft
Date : 3/28/2006
Time : 02:12 PM

Group 1: Divisions
Group 2: Major ItemCode Groups
Group 3: Minor ItemCode Groups
Group 4: Alternates

ItemCode	Description	Quantity	UM	Labor$	MH/Unit	Units/MH
Curing						
Curing						
Alternates Blank						
03390.010	PROTECT & CURE	**176,587.00**	SQFT	0.1102	0.004267	234.375
	******* Total Alternates Blank***				***$22,850.36***	
	****** Total Curing***				***$22,850.36***	
	***** Total Curing***				***$22,850.36***	
	**** Total Concrete***				***$2,420,131.74***	

Aric Heffelfinger
Fordham Place
Bronx, NY
Structural Option
Advisor - Dr. Hanagan

Concrete Labor Details

Labor Detail - Standard Construction Project

Estimator :
Project Size : sqft

Group 1: Divisions

Item Code	Description	Quantity	Hours	Base Rate	Fringe Rate	Total Rate	Prod. Factor	Total Labor Cost
Concrete								
03111.118	WALL FORM 20'+ HIGH	72,912.00 SQFT						
	(Crew C311) FORMWORK CREW needed for 245.16 DAY. Production: 297 SQFT/DAY							
	L040 - Carpenter		5,883.88	22.55	5.60	28.15	1.00	165,631.08
	L041 - Carpenter foreman		980.65	24.15	6.00	30.15	1.00	29,566.47
	L020 - Common laborer		3,922.58	17.83	4.43	22.26	1.00	87,316.71
03111.203	WOOD COLUMN FORMS, 12'-16'	44,165.33 SQFT						
	(Crew C311) FORMWORK CREW needed for 44.17 DAY. Production: 1,000 SQFT/DAY							
	L040 - Carpenter		1,059.97	22.55	5.60	28.15	1.00	29,838.10
	L041 - Carpenter foreman		176.66	24.15	6.00	30.15	1.00	5,326.34
	L020 - Common laborer		706.65	17.83	4.43	22.26	1.00	15,729.93
03111.612	SLAB FORM W/2.6 BM/SF	176,587.00 SQFT						
	(Crew C311) FORMWORK CREW needed for 388.91 DAY. Production: 454 SQFT/DAY							
	L040 - Carpenter		9,333.80	22.55	5.60	28.15	1.00	262,746.56
	L041 - Carpenter foreman		1,555.63	24.15	6.00	30.15	1.00	46,902.36
	L020 - Common laborer		6,222.54	17.83	4.43	22.26	1.00	138,513.64
03150.650	SCREEDS FOR SLAB	21,190.44 LNFT						
	(Crew C311) FORMWORK CREW needed for 16.95 DAY. Production: 1,250 LNFT/DAY							
	L040 - Carpenter		406.86	22.55	5.60	28.15	1.00	11,453.01
	L041 - Carpenter foreman		67.81	24.15	6.00	30.15	1.00	2,044.45
	L020 - Common laborer		271.24	17.83	4.43	22.26	1.00	6,037.75
03150.900	FORM RELEASING AGENT	293,664.33 SQFT						
	(Crew C311) FORMWORK CREW needed for 53.39 DAY. Production: 5,500 SQFT/DAY							
	L040 - Carpenter		1,281.44	22.55	5.60	28.15	1.00	36,072.66
	L041 - Carpenter foreman		213.57	24.15	6.00	30.15	1.00	6,439.26
	L020 - Common laborer		854.30	17.83	4.43	22.26	1.00	19,016.63
03210.130	SUPPORTED SLAB REBAR	5,886.23 CWT						
	(Crew C321) REINFORCING STEEL CREW needed for 107.02 DAY. Production: 55 CWT/DAY							
	L120 - Reinforcing rodman		5,137.08	21.55	9.95	31.50	1.00	161,817.91
	L121 - Reinforcing rodman foreman		856.18	22.91	10.59	33.50	1.00	28,682.01
03210.150	COLUMN REBAR	3,322.62 CWT						
	(Crew C321) REINFORCING STEEL CREW needed for 46.15 DAY. Production: 72 CWT/DAY							
	L120 - Reinforcing rodman		2,215.08	21.55	9.95	31.50	1.00	69,775.09
	L121 - Reinforcing rodman foreman		369.18	22.91	10.59	33.50	1.00	12,367.54
03310.576	4000 PSI W/CRANE	1,350.22 CUYD						
	(Crew C230) CONCRETE CREW, CRANE needed for 12.86 DAY. Production: 105 CUYD/DAY							
	L070 - Equipment operator		102.87	25.56	3.29	28.85	1.00	2,967.92
	L021 - Common laborer foreman		102.87	19.42	4.83	24.25	1.00	2,494.70
	L020 - Common laborer		617.24	17.83	4.43	22.26	1.00	13,739.86
	L052 - Vibrator operator		102.87	24.75	6.44	31.19	1.00	3,208.13
03310.676	4000 PSI W/CRANE	886.03 CUYD						
	(Crew C230) CONCRETE CREW, CRANE needed for 11.08 DAY. Production: 80 CUYD/DAY							
	L070 - Equipment operator		88.60	25.56	3.29	28.85	1.00	2,556.20
	L021 - Common laborer foreman		88.60	19.42	4.83	24.25	1.00	2,148.63
	L020 - Common laborer		531.62	17.83	4.43	22.26	1.00	11,833.86
	L052 - Vibrator operator		88.60	24.75	6.44	31.19	1.00	2,763.09
03311.526	4000 PSI W/CRANE	4,905.19 CUYD						
	(Crew C230) CONCRETE CREW, CRANE needed for 39.24 DAY. Production: 125 CUYD/DAY							
	L070 - Equipment operator		313.93	25.56	3.29	28.85	1.00	9,056.95
	L021 - Common laborer foreman		313.93	19.42	4.83	24.25	1.00	7,612.86
	L020 - Common laborer		1,883.59	17.83	4.43	22.26	1.00	41,928.82
	L052 - Vibrator operator		313.93	24.75	6.44	31.19	1.00	9,789.98
03350.130	MACHINE TROWEL FINISH	176,587.00 SQFT						
	(Crew C276) CONCRETE FINISHING CREW needed for 70.63 DAY. Production: 2,500 SQFT/DAY							
	L020 - Common laborer		565.08	17.83	4.43	22.26	1.00	12,578.65
	L050 - Concrete finisher		1,695.24	23.89	3.11	27.00	1.00	45,771.35
03350.131	POINT & PATCH	293,664.33 SQFT						
	(Crew C276) CONCRETE FINISHING CREW needed for 39.16 DAY. Production: 7,500 SQFT/DAY							
	L020 - Common laborer		313.24	17.83	4.43	22.26	1.00	6,972.77
	L050 - Concrete finisher		939.73	23.89	3.11	27.00	1.00	25,372.60
03390.010	PROTECT & CURE	176,587.00 SQFT						
	(Crew C276) CONCRETE FINISHING CREW needed for 23.54 DAY. Production: 7,500 SQFT/DAY							
	L020 - Common laborer		188.36	17.83	4.43	22.26	1.00	4,192.88
	L050 - Concrete finisher		565.08	23.89	3.11	27.00	1.00	15,257.12
	* Total Concrete		50,330.50					$1,355,524
	Total Estimate		50,330.50					$1,355,524

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Concrete Duration Calcs

- USE 5 FORMWORK CREWS
- USE 2 REINFORCING STEEL CREWS
- USE 2 CONCRETE CREWS

$$\text{TOTAL DURATION} = \frac{(245 + 44 + 389 + 17 + 53)}{5} + \frac{(107 + 46 + 13 + 11 + 39)}{2}$$

$$+ 71 + 39 + 24$$

$$= 392 \text{ WORK DAYS} \times \frac{1 \text{ WEEK}}{5 \text{ WORK DAYS}}$$

$$= 78.3 \text{ WEEKS}$$

Steel Cost Breakdown

EDF Report - Standard Construction Project

Estimator : User
Project Size : sqft
Date : 3/28/2006
Time : 02:04 PM

Group 1: Divisions
Group 2: Major ItemCode Groups
Group 3: Minor ItemCode Groups
Group 4: Alternates

ItemCode	Description	Quantity	UM	Labor$	MH/Unit	Units/MH
Concrete						
Welded wire fabric						
Welded wire fabric						
Alternates Blank						
03220.010	6x6 W1.4/W1.4 MESH	1,873.87	SQS	18.8640	0.80	1.25
	**** Total Alternates Blank				$50,714.47	
	*** Total Welded wire fabric				$50,714.47	
	** Total Welded wire fabric				$50,714.47	
Structural concrete						
Structural concrete						
Alternates Blank						
03311.700	**CONC IN SLAB OVER MTL DECK*	****				
03311.726	4000 PSI W/CRANE	2,366.00	CUYD	13.9420	0.576	1.73611
03315.991	* SLAB OVER METAL DECK AREA *	170,352.00	SQFT			
	**** Total Alternates Blank				$165,482.77	
	*** Total Structural concrete				$165,482.77	
	** Total Structural concrete				$165,482.77	
	* Total Concrete				$216,197.24	
Metals						
Structural steel						
Structural steel						
Alternates Blank						
05129.101	STEEL BEAMS	****				
05129.101	STEEL BEAMS	****				
05129.102	I BEAMS	705.60	CWT	28.7300	0.90	1.11111
05129.102	I BEAMS	8,593.36	CWT	28.7300	0.90	1.11111
05129.121	STEEL COLUMNS	****				
05129.122	I SHAPES	4,003.22	CWT	28.7300	0.90	1.11111
05129.181	BRACING	****				
05129.182	I BEAMS	4,564.63	CWT	38.3067	1.20	0.83333
05129.304	ASTM A572 50 KSI STEEL ADDER	4,003.22	CWT			
05129.310	TYPE B STEEL ADDER	4,564.63	CWT			
05129.404	SHEAR STUD, 3/4"	72.00	EACH	0.5434	0.017143	58.33333
05129.404	SHEAR STUD, 3/4"	1,173.00	EACH	0.5434	0.017143	58.33333
05129.990	* STRUCTURAL STEEL WEIGHT *	463.67	TONS			
05129.990	* STRUCTURAL STEEL WEIGHT *	429.67	TONS			
	**** Total Alternates Blank				$1,273,642.08	
	*** Total Structural steel				$1,273,642.08	
	** Total Structural steel				$1,273,642.08	
Steel deck						
Steel deck						
Alternates Blank						
05310.019	3" METAL DECK	170,352.00	SQFT	0.4445	0.013926	71.80556
	**** Total Alternates Blank				$247,981.41	
	*** Total Steel deck				$247,981.41	
	** Total Steel deck				$247,981.41	
	* Total Metals				$1,521,623.49	

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Page 1

Steel Labor Detail / Duration Calcs

Labor Detail - Standard Construction Project

Estimator :
Project Size : sqft

Group 1: Divisions

Item Code	Description	Quantity	Hours	Base Rate	Fringe Rate	Total Rate	Prod. Factor	Total Labor Cost
Concrete								
03220.010	6x6 W1.4/W1.4 MESH	1,873.87 SQS						
	(Crew C320) WIRE MESH CREW needed for 26.77 DAY. Production: 70 SQS/DAY							
	L020 - Common laborer		1,284.94	17.83	4.43	22.26	1.00	28,602.78
	L120 - Reinforcing rodman		214.16	21.55	9.95	31.50	1.00	6,745.94
03311.726	4000 PSI W/CRANE	2,366.00 CUYD						
	(Crew C230) CONCRETE CREW, CRANE needed for 18.93 DAY. Production: 125 CUYD/DAY							
	L070 - Equipment operator		151.42	25.56	3.29	28.85	1.00	4,368.58
	L021 - Common laborer foreman		151.42	19.42	4.83	24.25	1.00	3,672.03
	L020 - Common laborer		908.54	17.83	4.43	22.26	1.00	20,224.19
	L052 - Vibrator operator		151.42	24.75	6.44	31.19	1.00	4,722.16
	*** Total Concrete**		**2,861.91**					**$68,336**
Metals								
05129.102	I BEAMS	9,298.96 CWT						
	(Crew C510) STRUCTURAL STEEL CREW needed for 116.24 DAY. Production: 80 CWT/DAY							
	L160 - Steelworker		7,439.17	18.20	13.50	31.70	1.00	235,821.63
	L161 - Steelworker foreman		929.90	19.34	14.36	33.70	1.00	31,337.50
05129.122	I SHAPES	4,003.22 CWT						
	(Crew C510) STRUCTURAL STEEL CREW needed for 50.04 DAY. Production: 80 CWT/DAY							
	L160 - Steelworker		3,202.58	18.20	13.50	31.70	1.00	101,521.66
	L161 - Steelworker foreman		400.32	19.34	14.36	33.70	1.00	13,490.85
05129.182	I BEAMS	4,564.63 CWT						
	(Crew C510) STRUCTURAL STEEL CREW needed for 76.08 DAY. Production: 60 CWT/DAY							
	L160 - Steelworker		4,868.93	18.20	13.50	31.70	1.00	154,345.22
	L161 - Steelworker foreman		608.62	19.34	14.36	33.70	1.00	20,510.39
05129.404	SHEAR STUD, 3/4"	1,245.00 EACH						
	(Crew C509) MISCELLANEOUS METALS CREW needed for 0.89 DAY. Production: 1,400 EACH/DAY							
	L160 - Steelworker		21.34	18.20	13.50	31.70	1.00	676.57
05310.019	3" METAL DECK	170,352.00 SQFT						
	(Crew C510) STRUCTURAL STEEL CREW needed for 32.95 DAY. Production: 5,170 SQFT/DAY							
	L160 - Steelworker		2,108.81	18.20	13.50	31.70	1.00	66,849.16
	L161 - Steelworker foreman		263.60	19.34	14.36	33.70	1.00	8,883.35
	* Total Metals		19,843.26					$633,436
	Total Estimate		22,705.18					$701,772

USE 2 STRUCTURAL STEEL CREWS

$$\text{TOTAL DURATION} = \frac{(116+50+76)}{2} + 33 + 1 + 27 + 19$$

$$= 201 \text{ WORK DAYS} \times \frac{1 \text{ WEEK}}{5 \text{ WORK DAYS}}$$

$$= 40.2 \text{ WEEKS}$$

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