Table of Contents

Executive Summary:

Lexington II is a residential tower located as part of the Market Square North building complex in downtown Washington D.C. Due to strict height controls in the area, the structural design of Lexington II was dictated by its need to contain maximum usable floor levels and thin floor sandwiches. The structural system chosen was flat plate slab spanning small bays. Lateral load is resisted by a core of shear walls located at the building's center.

In designing Lexington II, the structural system was chosen following the common practice of using a flat plate slab or pre-cast systems when designing for the D.C. area. Other systems, such as steel and composite, are often over looked although the have advantages. This report compares the benefits of a steel system versus a concrete flat plate slab in the design of Lexington II had height requirements not been a factor. The building systems are evaluated on their structural advantages, construction ease, ability to integrate mechanical systems, and most importantly economy of the design.

After a brief evaluation of several systems, the final alternative system designed was composite deck on steel beams and columns. 2"-Lok decking with 2.5" slab (total depth of 4.5") was chosen from a decking manual and catalog. The remainder of the gravity system was designed using the finite element software RAM and resulted in W 12's. The total floor sandwich was 16 inches; double that of the flat plate slab.

The lateral system which would work best in Lexington II with composite flooring was braced frames. The braced frames had to be designed around the existing architecture. Chevron frames were used at the building core in both the N-S and W-E directions. Some member sizes were greatly increased from that needed to support the lateral load including columns in biaxial bending.

Other structural considerations to complete a total design were also evaluated. To better withstand subterranean conditions, the sub-grade structure was designed as cast in place joist floors with shear walls. Connections details for typical column to beam connections as well as heavy braced connections were calculated.

A construction management study verified that a composite system was feasible. A site layout was completed with ample area for all necessary spaces. Scheduling had no major conflicts, and most importantly there was no cost increase in the composite system.

Mechanical integration was also possible with a composite structural system. Fresh air requirements are met by the new window layout associated with the column grid, and simple redesign of the ductwork allows HVAC to reach all spaces. Similarly, simple changes in the sprinkler layout can be made to ensure adequate fire suppression within Lexington II. Acoustical differences between a concrete and steel system may require some greater attention to detailing but do not present any major problems.

There are several benefits to each type of structural system. In the design of Lexington II no system prevails greatly over the other. The small scale of the Lexington II prevents the full economy associated with most steel systems to be reached. Using flat plate slab enables Lexington II to meet the required height restriction without the tradeoff of compromising other building systems.

Building Summary:

Lexington II is the luxury apartment tower built as part of the Market Square North complex in Washington D.C. With 72,000 square feet of floor area, Lexington II has 11 residential stories, a ground level with retail, and 3 below grade parking and retail stories.

Key Players:

Lexington II was build by Square 407 LP, a joint venture of Gould Property Company and Boston Properties. The architecture of Lexington II was designed by Studios Architects. The structural engineer on the project was the Washington D.C. office of Thornton Thomasetti Group, formerly James Madison Cutts. Other engineers on the project include The Engineering Design Group as the MEP and The Clark Construction Group as the general contractor.

Architecture:

The Lexington was designed to be exclusive apartments located in downtown Washington D.C., and to compliment the surrounding architecture. Lexington II consists of 49 individual apartment units varying between one bedroom, two bedrooms, and studio apartments. All apartments feature over sized windows, walk in closets, and spacious ceramic baths. Interiors are finished in luxury materials including Italian marble, French limestone, granite and cherry. Some apartments also feature French balconies, terraces, and bay windows. A luxurious main lobby with full concierge service is provided. A reception room is also available.

Lexington II also includes three below grade levels that are utilized as parking and retail space. The below grade levels connect Lexington II via tunnel to the rest of the Market Square North development.

Building Envelope and Façade:

Lexington II has a non-load bearing exterior brick cavity wall featuring pre-cast stone trim and pre-cast concrete accents. Punched windows are in a grid like pattern along the two exposed exterior walls. The other two walls of Lexington II abut other buildings in the Market Square North complex.

A typical wall sandwich of Lexington II consists of facebrick, a 1 7/8" airspace, 15# building paper, 5/8" exterior gypsum sheathing, 3 5/8" galvanized metal studs located 16" on center, 3" batt. insulation with an R-value of 19, and a 1/2" gypsum wallboard. See appendix Figure A-1 for wall section.

A steel and glass canopy defines the entrance to Lexington II. The main entrance is a set of double glass doors opening up into a vestibule with a second set of doors leading into the lobby. The other building entrances are directly connected to adjoining buildings, the below grade parking areas, and retail spaces which opens exteriorly to the street.

The roof of Lexington II has no special features except for a mechanical penthouse that houses elevator equipment, a cooling tower, and a backup generator. Roof construction is a ballast, filter fabric, rigid insulation, separation sheet, and fluid applied membrane waterproofing. The penthouse enclosure surrounding the backup generator is made of 2” exterior insulation and finish system (EILF), #15 building paper, 5/8” exterior gypsum sheathing, 3 5/8” metal studs 16” on center and 1/2” gypsum wallboard. The cooling tower is enclosed by 2” EILF and an 8” CMU wall. See Appendix Figure A-2 for roof sections.

Zoning/ Site:

Lexington II is located in downtown Washington D.C. at the corner of 8th Street and E Street, a few streets back from Pennsylvania Avenue. This location places the Lexington in Washington D.C.’s Historic Penn Quarter. Being in the Historic Penn Quarter means that the Lexington is located close to many nationally significant sites; such as the White House, Capitol Building, Mall, Smithsonian, Shakespeare Theater, and MCI Arena as well as numerous other upscale restaurants, galleries, and theaters. Being such a historically rich area, the Historic Penn Quarter was declared a national historic site on September 30, 1965 by the Secretary of the Interior. October 15, 1966 the site was added to the National Registry of Historic Places. Currently the block on which Lexington II is located is governed by the Pennsylvania Avenue Development Corporation, established on October 27, 1972.

The District of Columbia Office of Zoning has designations to the block Lexington II is located on. Found on zoning map 10, Lexington II has been given a designation of DD/C-4. DD/C-4 refers to the downtown development district and the central business district of Washington D.C. Taken from the District of Columbia Office of Zoning, regulations for DD and C-4 are as follows:

DD – Downtown Development District

> Permits incentives and requirements for downtown sub-areas to a maximum FAR of 6.0 to 10.0, and a maximum height of one hundred-thirty feet.

C-4

> The downtown core comprising the retail and office centers for the District of Columbia and the metropolitan area, and allows office, retail, and housing and mixed uses to a maximum lot occupancy of 100%, a maximum FAR of 8.5 to 10.0, a maximum height of 110 feet and 130 feet on 110-foot adjoining streets.

Other systems:

Transportation: The vertical transportation system of Lexington II is located in a central core of the building. Two passenger elevators operate, both traveling the entire vertical length of the building. Two stairwells also run the vertical length of the building. One stairwell terminates on the underground concourse level while the other continues to the lowest parking level.

Mechanical: The mechanical system for The Lexington is a water source heat pump system. This system involves the use of a boiler located in the building’s basement, pumps, and a rooftop-cooling tower. The cooling tower is a 176-ton counter flow blow-thru tower. Fresh air requirements are met by operable windows in all residential units and fresh air intake units in the roof that provide 100% outside air to the corridor spaces. All residential units are equipped with kitchen, toilet, and washer/dryer exhausts.

Electrical/Lighting: Each apartment unit is provided with a voltage of 120/208V. This power is on a phase 1P 3-wire system. Located in the roof penthouse is an emergency generator. The incoming electricity is provided by a PEPCO vault located outside of The Lexington's parking levels. This incoming power is 120/280V and is 3 phase with 4 wires. Fluorescent lighting is used in both public and private spaces of Lexington II.

Fire Protection: The Lexington is provided with a 100% fully sprinklered, automatic wet and dry pipe system. This system utilizes a fire pump, jockey pump, wet pipe sprinkler system, dry pipe sprinkler system, and fire standpipe systems as its components.

Existing Structure:

The basic structural system of Lexington II is two-way flat plate slab supported by cast in place concrete columns. The existing structural system of Lexington II is complicated by offset columns in many locations. Lateral resistance is provided by concrete shear walls around the elevator shaft at the center of the building. The entire building is resting on a MAT foundation.

Gravity System:

The existing gravity system of The Lexington is two-way flat plate slab resting on concrete columns. Flat plate slab was chosen because of its ability to maintain a shallow floor sandwich, an important criterion when designing in an area with height restrictions on buildings. In order to achieve the shallowest floor sandwich possible, columns were placed close together creating small bays for the slab to span. The column layout was planned around the building architecture and often offset or turned columns were used to better fit into architectural partitions. Column layouts for the three floor plans used in Lexington II can be found on the next three pages (Figures 1-3). The average bay size is approximately 13.5' by 16.2'. The majority of the bays have 2-way flat-plate slabs with no edge beams. However, edge beams can be found on the lower levels where the live load is increased. Edge beams are also in place along the east exterior bays on some levels.

The 2-way slab floors are concrete with a compressive strength of 4000psi. The floors of the 3 level sub-structure are 10" thick while the superstructure has floors that are 8" thick. Exceptions to flat plate slab are 5" drop panels around the southern columns of the concourse level. The drop panels are bending drops which are in place to provide for the greater flexural and shear loads caused by an increased live load on the concourse level. Another exception is an increase in the 8" slab to 10" at the south end of the ground floor. This 10" thick slab, localized to the south end of the ground floor, is a loading dock for the retail space which will have the additional weight of trucks.

The 2-way slab is reinforced with a continuous bottom mat of #4 bars 12 inches on center. These bars are ASTM A216, grade 60. In addition to the #4 bars at 12" mat, there is top reinforcing in some locations. Typically the top reinforcing are #4 or #5 bars. The top reinforcement is often located by columns and shafts cut into the slab which creates a stronger moment in these locations. For reinforcement lay out, see framing plans in Appendix, Figures A-3, A-4, A-5, A-6, A-8 and A-8.

All of the columns throughout Lexington II are 5000psi compressive strength concrete with ASTM A615, grade 60 reinforcement. Columns range in size from 14" x 14" columns reinforced by 4 #9 bars to 42" x 14" columns reinforced with 18 #11 bars. As expected, the larger columns are in the lower stories of the building which carry the building's entire weight.

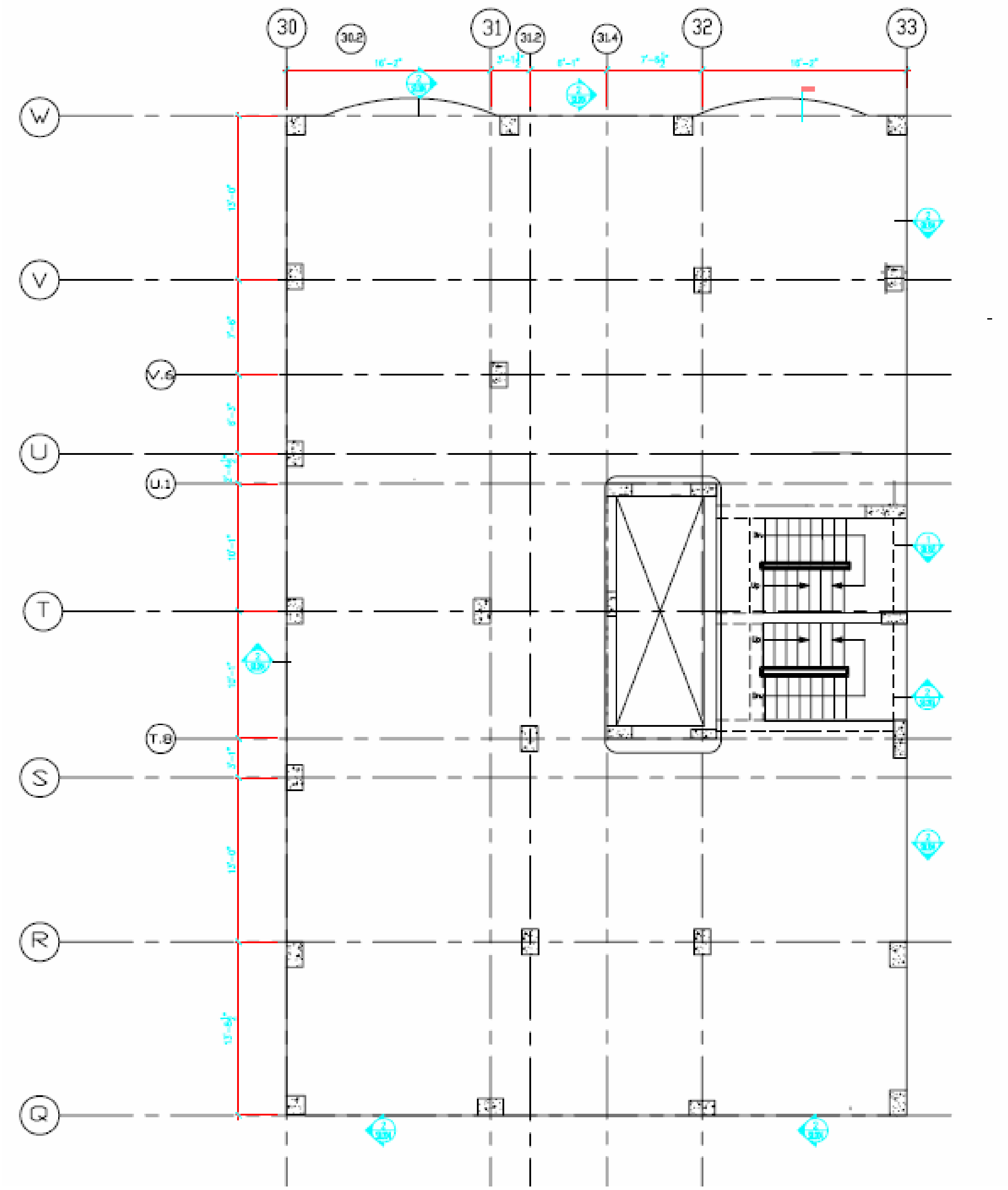


Figure 1
Column Layout for floors 12 to 8

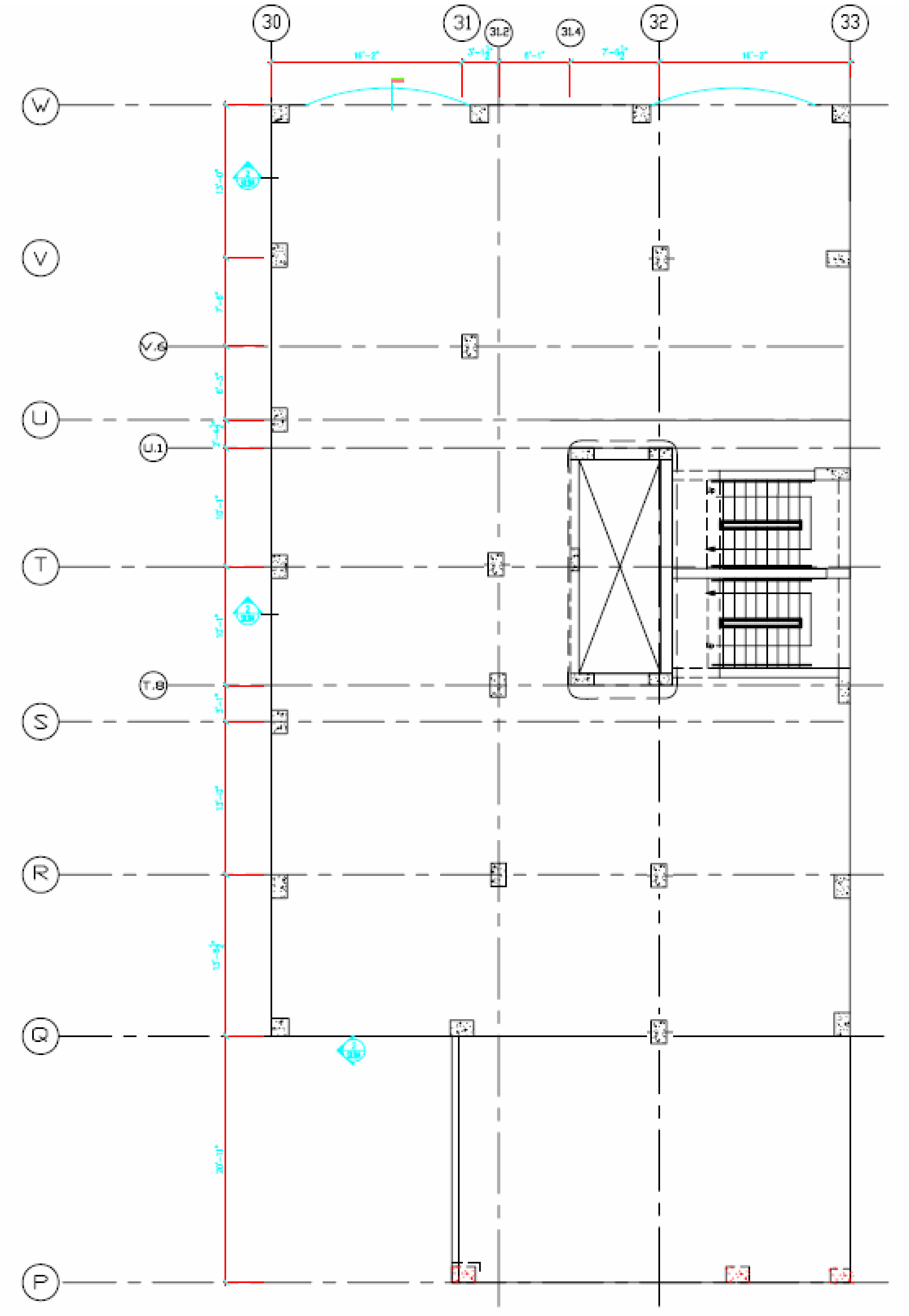


Figure 2
Column Layout for floors 7 to 2

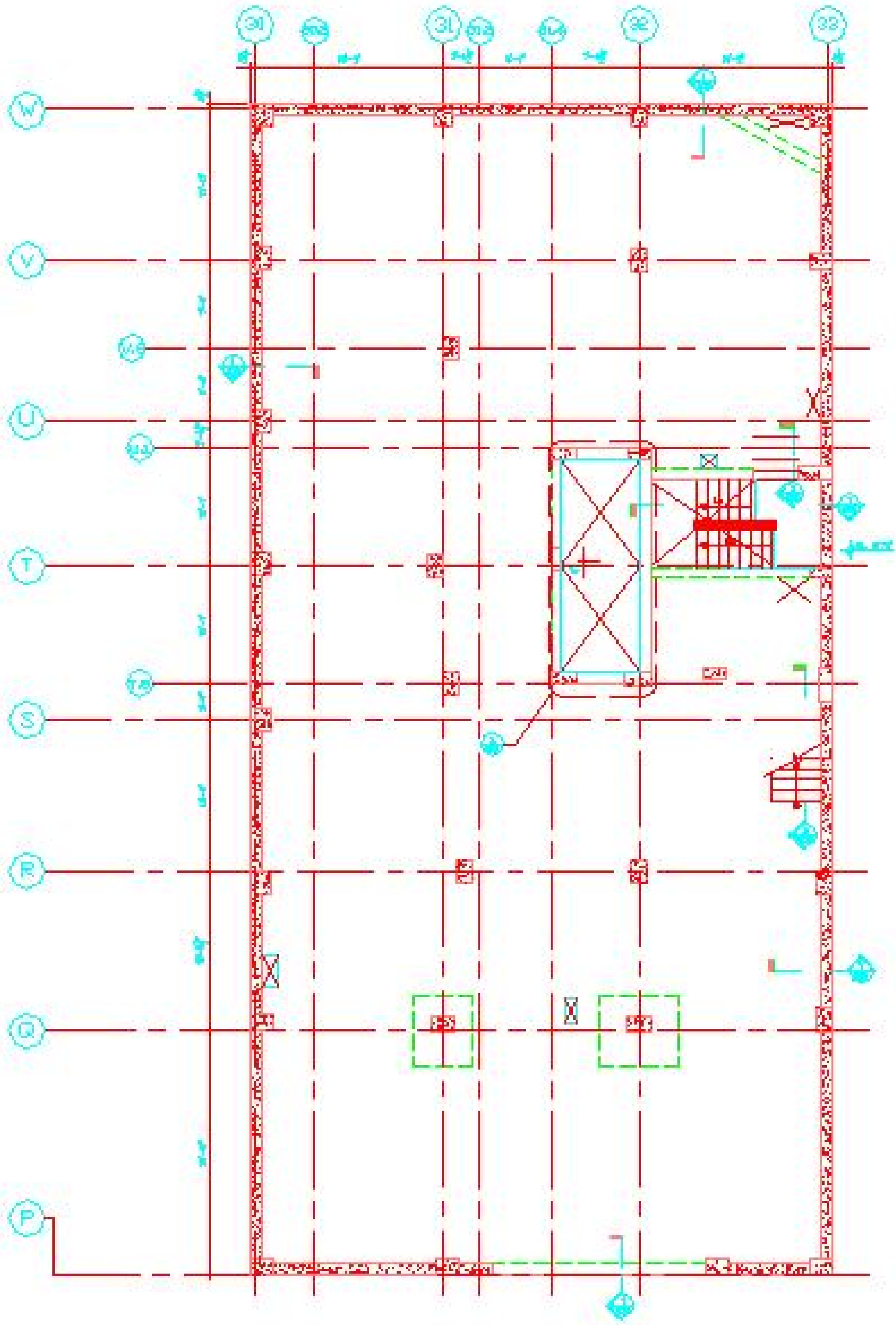


Figure 3
Column Layout for the ground floor, L-1, concourse, and P-1
Green areas represent drop plans and edge beams found on the concourse level

Lateral:

The lateral forces on Lexington are resisted by a core of shear walls located around the building's elevator shaft. See shear wall plan below, Figure 4. All shear walls are 12" thick, constructed of 4000psi concrete, and cast in place. Shear wall reinforcement includes #4 bars every 12" on center.

Since Lexington II's gravity system is monolithically poured, it naturally creates moment framing. However, contact with the structural engineer confirmed that the shear walls in Lexington II were designed with the intention of carrying the entire lateral load.

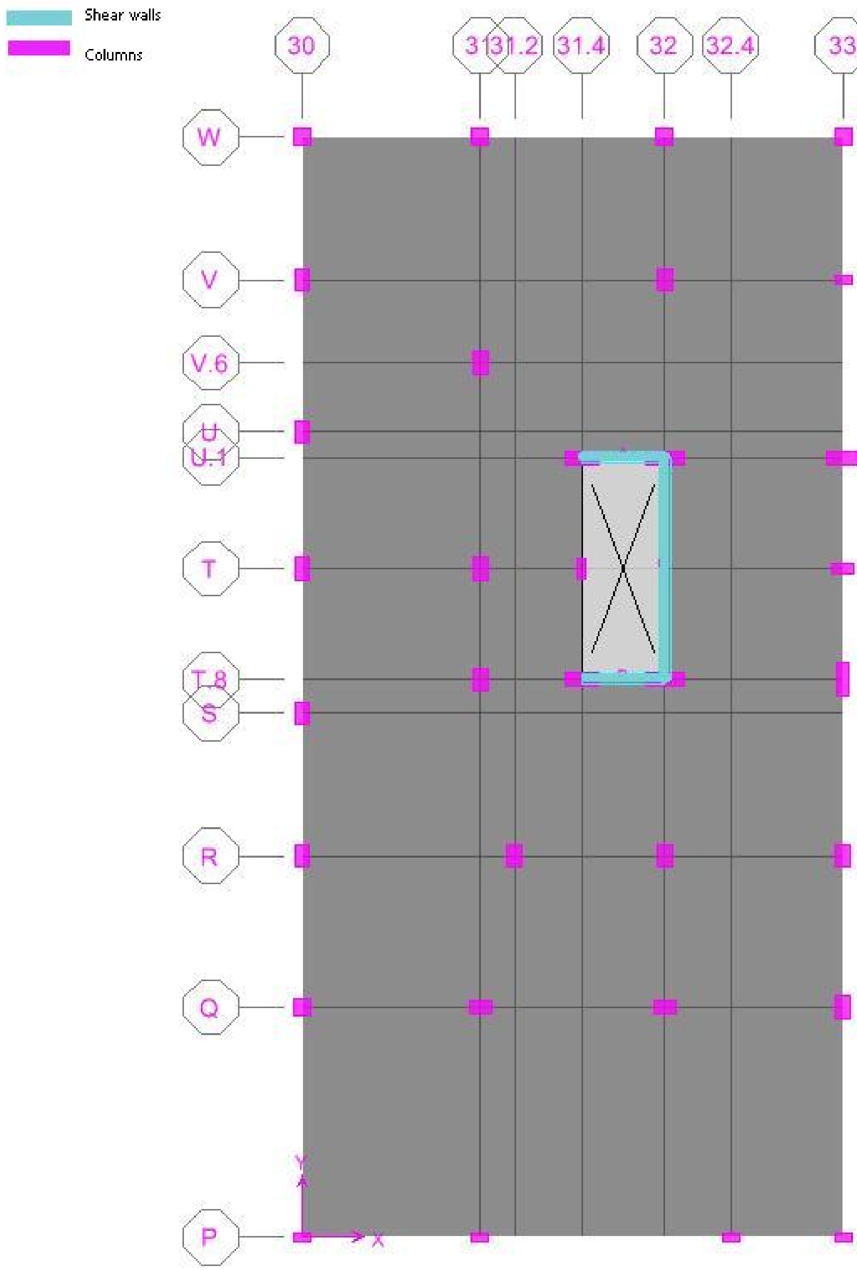


Figure 4
Shear Wall Plan

Foundation:

The foundation of Lexington II is a 3'-6" thick MAT foundation which is reinforced with deformed #8 bars located every 9" o.c. The MAT foundation is also reinforced with #11 top bars in some locations and designed in a 2-way slab formation. Below the MAT foundation is a 3" sub-grade working MAT. The foundation rests on original soil and structural fill with a compressive strength of 8000psf. Along the southern wall of Lexington II the foundation rests on HP 14 x 89 piles every five feet on center with one inch cap plates. The piles are in place because the pre-existing building to the south of Lexington II (which Lexington II abuts) is a story lower. Rather than undermining the existing building's foundation, piles were installed as an alternative to providing control fills stepped up to the new foundation level (which is more costly).

The below grade walls are reinforced concrete which is 14" thick from level P1 to the concourse level at which point they are reduced to 12" until they end at the ground level. Reinforcement in the retaining walls are #4 bars every 12" running in the longitudinal direction and #5 bars every 12" running vertically. Both the concrete walls and the MAT foundation have a compressive strength of 5000 psi. The reinforcing steel in both the MAT foundation and the below grade walls is ASTM A615, grade 60.

Summary of Structural System:

Floors 12 to 2:
- Concrete:
  - Columns……………………………..5000psi
  - 8" 2-way floor slab…………………..…4000psi
  - Beams………………………………..4000psi
  - Shear walls…………………………..4000psi
- Reinforcing steel:
  - Bar reinforcing…………ASTM A-615, grade 60, 60psi
  - Welded Wire Mesh……..ASTM A-185

Floors Ground to Concourse:
- Concrete:
  - Columns……………………………..5000psi
  - Basement Walls……………………...5000psi
  - 10" 2-way floor slabs…..……………4000psi
  - Shear walls…………………………..4000psi
  - Beams………………………………..4000psi
- Reinforcing steel:
  - Bar reinforcing…………ASTM A-615, grade 60, 60psi
  - Welded Wire Mesh……..ASTM A-185

Foundation:
- Concrete:
  - MAT foundation……………………5000psi
  - Basement Walls……………………...5000psi
- Reinforcing steel:
  - Bar reinforcing…………ASTM A-615, grade 60, 60psi
  - Welded Wire Mesh……..ASTM A-185

Codes and Loading:

The model code used to design the existing Lexington II, completed in 2002, was the 1996 edition of the BOCA codes. Other codes used while designing Lexington II include:

ACI 318-95	Reinforced Concrete
AISC- 9th Ed.	Structural Steel (design, fabrication, and erection)
AWS D1.1-98	Structural Welding
ACI 530-95/ ASCE 5-96	Masonry

Loading: (From ASCE7-02)

Dead Load- Superimposed:

Finishes……………………………15psf
Partitions……………………………included in live load, see below
<u>Mechanical/Lighting……………… 5psf</u>
Total Superimposed………………..20psf

Dead Load- Self Weight

Substructure Slab (10")…………….125psf (Appendix)
Superstructure Slab (8")……………100psf (Appendix)
Exterior Wall………………………30psf

Live Load:

Lexington II was designed following the loading as prescribed by the 1996 edition of the BOCA code. The engineers assumed the following live loads:

Roof……………………………….. 30psf
Ground, L1, and P1 level stairs……100psf
Mechanical Rooms………………...150psf
Lobbies……………………………100psf
Concourse level……………………225psf
Residential Levels…………………..60psf + 20psf (for partitions)

For my report, I will be using a more recent code, ASCE7-02. Live loads obtained from ASCE 7-02 are comparable with those used in the building's original design

Roof……………………………...20psf (Appendix)
Public Levels/ Stairs………………100psf (ASCE7-02)
Lobbies……………………………100psf (ASCE&-02)
Residential Levels…………………40psf + 20psf (for partitions)

Snow Load:

Snow Load…………………………15.75psf (Appendix)

Wind Loads:

N/S direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)
ground	21.22	281.75	5.98	139.07	0.00
1	21.22	497.15	10.55	133.09	121.32
2	22.30	430.71	9.60	122.54	194.89
3	22.78	430.78	9.81	112.93	285.34
4	23.50	430.96	10.13	103.12	383.53
5	24.10	430.78	10.38	92.99	484.45
6	24.58	430.71	10.58	82.61	587.02
7	25.06	430.76	10.79	72.03	693.43
8	25.53	430.71	11.00	61.24	803.29
9	25.89	430.83	11.16	50.24	912.89
10	26.25	430.96	11.31	39.08	1025.34
11	26.25	446.02	11.71	27.77	1164.17
12	26.85	414.30	11.12	16.06	1210.73
roof	26.85	183.75	4.93	4.93	573.99

moment total
8440.40

E/W direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)
ground	11.51	575.00	6.62	170.79	0.00
1	11.51	1014.60	11.67	164.18	134.24
2	12.58	879.00	11.06	152.51	224.45
3	13.06	879.15	11.48	141.44	333.97
4	13.78	879.50	12.12	129.96	459.10
5	14.38	879.15	12.64	117.84	590.06
6	14.86	879.00	13.06	105.20	724.42
7	15.34	879.10	13.49	92.13	866.44
8	15.82	879.00	13.91	78.65	1015.64
9	16.18	879.25	14.23	64.74	1164.06
10	16.54	879.50	14.55	50.52	1318.20
11	16.54	910.25	15.05	35.97	1496.69
12	17.14	845.50	14.49	20.92	1576.94
roof	17.14	375.00	6.43	6.43	747.61

moment total
10651.82

Table 1
For full wind load calculation, see Appendix Table A-1.

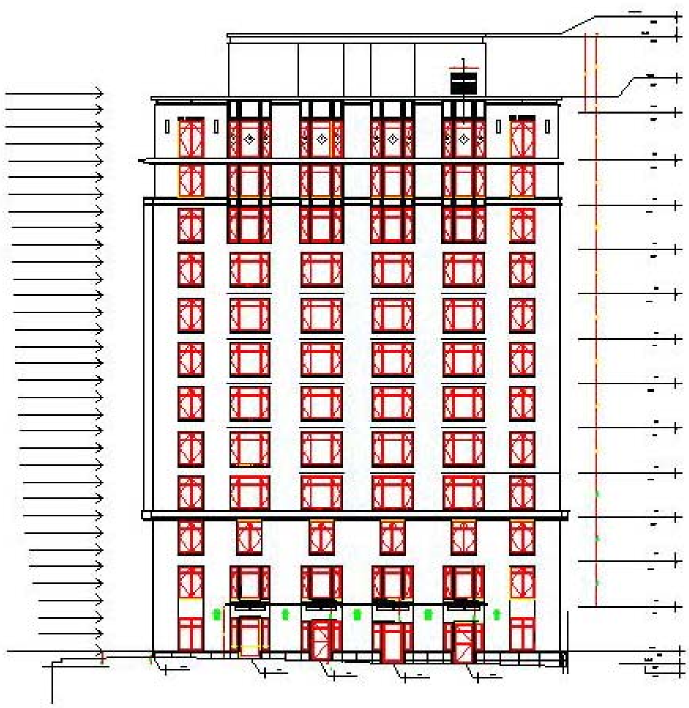


Figure 5
Wind hitting the building in the North South Direction

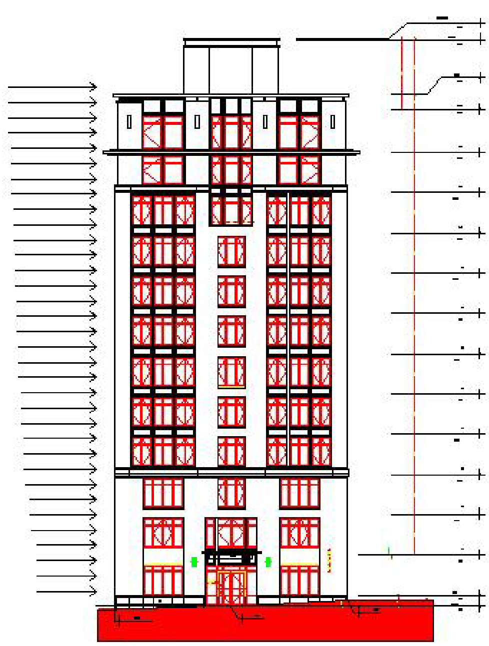
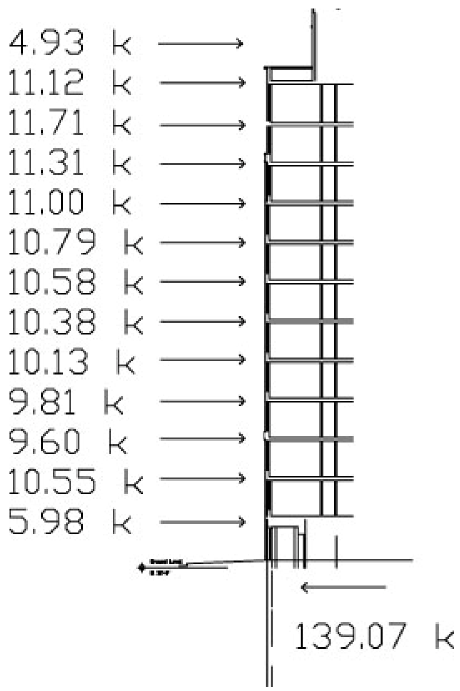
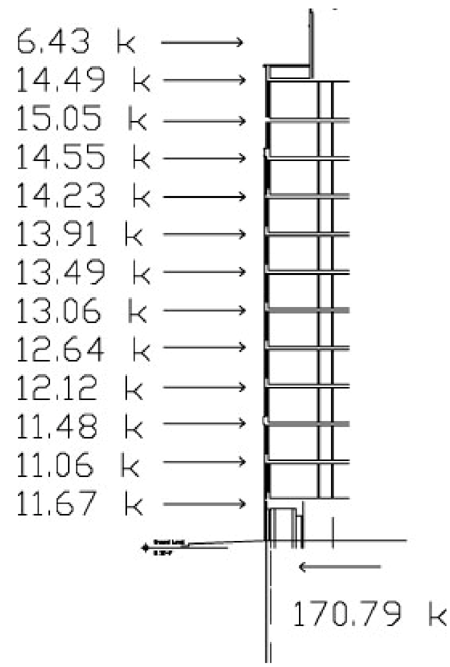


Figure 6
Wind hitting the building in the East West Direction





Wind (North-South Direction) Story Forces   Wind (East-West Direction) Story Forces

Figure 7

Seismic Loads:

Floor	height (ft)	Total Load (kips)	wx*hx^k	Cvx	Fx (kips)	Vx (kips)	Mx (kip ft)
roof	108.58	423.23	68449.38	0.14	14.88	0.00	1615.74
12	99.17	457.01	66987.79	0.14	14.56	14.88	1444.20
11	90.375	454.65	60253.93	0.12	13.10	29.44	1183.82
10	81.58	454.63	53916.66	0.11	11.72	42.54	956.22
9	72.79	454.61	47641.36	0.10	10.36	54.26	753.89
8	64	454.61	41432.54	0.09	9.01	64.62	576.47
7	55.21	534.65	41510.32	0.09	9.02	73.63	498.23
6	46.42	548.54	35284.38	0.07	7.67	82.65	356.07
5	37.625	548.56	28094.23	0.06	6.11	90.32	229.80
4	28.83	548.53	21044.17	0.04	4.57	96.43	131.90
3	20.042	548.52	14183.91	0.03	3.08	101.01	61.80
2	11.25	545.65	7540.78	0.02	1.64	104.09	18.44
Ground	0	540.29	0.00	0.00	0.00	105.73	0.00
			486339.46				

Total Building Weight (kips)	6513.4607
Overturning Moment	7826.58356

Table 2

For full seismic loading calculations, see Appendix Table A-2.

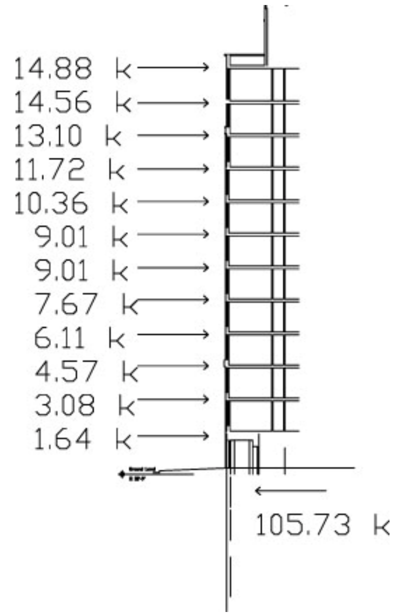


Figure 8
Seismic Story Forces

Load Combinations:

Taken from ASCE 7-02.

1.4D
1.2D + 1.6L + .5Lr
1.2D + 1.6Lr + (L or .8W)
1.2D + 1.6W + .5L + .5Lr
1.2D + E + .2S
.9D + 1.6W + 1.6H
.9D + 1E + 1.6H

The controlling load case is 1.2D + 1.6W +.5L + .5Lr. This was determined by running all load cases (psf) in an excel spread sheet. See Appendix Table A-3 for excel spread sheet and results.

Problem Statement:

Proposed Investigation:

Lexington II's design was greatly influenced by the demand for shallow floor sandwiches imposed by the Washington D.C. height restriction. It is customary for engineers in D.C. to design using two way flat plate slab or pre-stressed concrete (traditionally the shallowest sandwich depths) without thoroughly investigating other structural systems. Had Lexington II been located outside of the central Washington D.C. area other structural systems which employ the use of beams and the creation of a deeper floor sandwich would have been investigated further. Another structural system may have proved to be a more time and cost efficient design for The Lexington.

For my project, I am investigating the effect a steel structural system would have had on the overall design of Lexington II had it not been located in a height restricted area. The steel system will be analyzed on the basis of time and cost.

Investigation Method:

In order to investigate the effects a steel system would have on the Lexington II building project, I plan to design the Lexington II with a steel system and compare my final design to the actual concrete design of Lexington II.

To design The Lexington as a steel building I first plan to look at several systems. Through a brief analysis I will determine which system is the most appropriate for Lexington II. The system deemed the most feasible will then be used in a total building design of Lexington II. In order to complete the design, RAM steel as well as hand calculations are utilized.

The lateral system of Lexington II will also be designed in accordance with a steel structural system design. Alternatives to be considered for the lateral system are shear walls, braced frames, and moment frames. Member analysis and drift checks will be performed using finite element software, such as STAAD.

After the completion of both the gravity and lateral system, the building as a whole will be looked at. By looking at the systems and how they work together, other important details can be checked, including foundation and connection design.

The final step of the analysis is to compare my steel design to the current concrete design of Lexington II. This comparison will look at construction management criteria such as time of construction. The most important criterion that will be investigated is the cost of a steel design versus the concrete design.

Design- Gravity System:

Investigation:

To design the gravity system of Lexington II many types of floor systems were investigated. The system which proved to have the most benefits was then designed in further detail for Lexington II. The design of the gravity system includes floor slab, floor decking, beams, and columns.

Systems investigated for the Lexington II design include one-way slab, one-way joist, non-composite steel, composite steel, and pre-cast concrete with steel beams. These systems were looked at last semester and compared based on the design of an average bay. For most of these systems to be economical, the bay spans were increased from those of the existing two-way slab. Although height restriction was no longer a requirement, thinner floor systems were given preference incase a zoning variance was achievable.

Results of the initial comparison are below:

	Floor Depth	Weight[1]	Fireproofing	Vibration	General Comments	Feasibility
Existing System: Two-Way Flat Plate	8" floor slab with suspended ceiling for MEP space	100psf	No additional fireproofing is required			
One-Way Slab	6.5" slab + 20" beam = 26.5"	112psf	No additional fireproofing is required	Heavier then existing system, will dampen vibrations	• Works with existing column layout. • Rearranging bay sizes may help to reduce beam depth, however bay sizes are already very small • Simple formwork and construction	Increased weight and floor depth make more analysis unnecessary without alternating the column gird.
One-Way Joist	3" slab + 8" ribs =11"	75psf	No additional fireproofing is required	Joists add more stiffness	• Form work is easy to erect • Larger columns and punching shear will result	Should be investigated
Steel with Non-Composite Deck	8" slab + 16" beam = 24"	67.5psf	Additional fireproofing is required	Lighter system may cause vibration issues	• Lateral Bracing required • Complex connections • Possible foundation and lateral system redesign	Possible for investigating, however floor sandwich may become a problem
Composite with Composite Deck	4" slab +1.5" deck + 12" beam =17.5"	35psf	Additional fireproofing required on steel beams	Usually no vibration problems with composite	• No shoring required • Extra cost and labor of shear studs • Possible foundation and lateral system redesign	Should be investigated
Pre-Cast Slab with Steel Beams	4" slab + 18" beam = 22"	54psf	Additional fireproofing is required on beams	Lighter system could cause vibration problems	• Fast to construct, all pieces fabricated offsite	Possible for investigating

Table 3
Comparison of Floor Systems

The final system decided upon for an alternative design of Lexington II was a composite system of composite deck and steel beams. This system has a relatively shallow floor sandwich and should not effect vibration throughout the building. Fire proofing and shear studs will be required and may increase labor costs, but generally speaking steel buildings are considered to be more economical than concrete in a majority of cases.

Loads:

DEAD LOAD: (ASCE 7)

MEP	15 psf
Finishes[1]-luxury	15 psf
Cladding[2]-brick cavity wall	39 psf
TOTAL	30 psf (cladding will be added as a line load to the perimeter)

LIVE LOAD:

Public levels; Lobbies, retail, concourse	100 psf
Residential Levels	60 psf
Partitions	20 psf

Live Load Reduction: $L = Lo\left(.25 + \frac{15}{\sqrt{KAt}}\right)$, can not be determined until tributary area and K is known.

Roof Live Load:

$Lr = 20R_1R_2$

$R_1 = 1.2 - .001A_t$

$R_2 = 1$ for a flat roof

SNOW LOAD:

$P_f = .7CeCtIp_g$

p_g= .25 psf	(ASCE 7, Figure 7-1)
Ce = .9	(ASCE 7, Table 7-2)
Ct = 1	(ASCE 7, Table 7-3)
I = 1	(ASCE 7, Table 7-4)

P_f = 15.75 psf

[1] A large load was picked for finishes to account for the luxury materials used in Lexington II, such as limestone, granite, and cherry wood. Finishes also include acoustical ceiling and flooring.

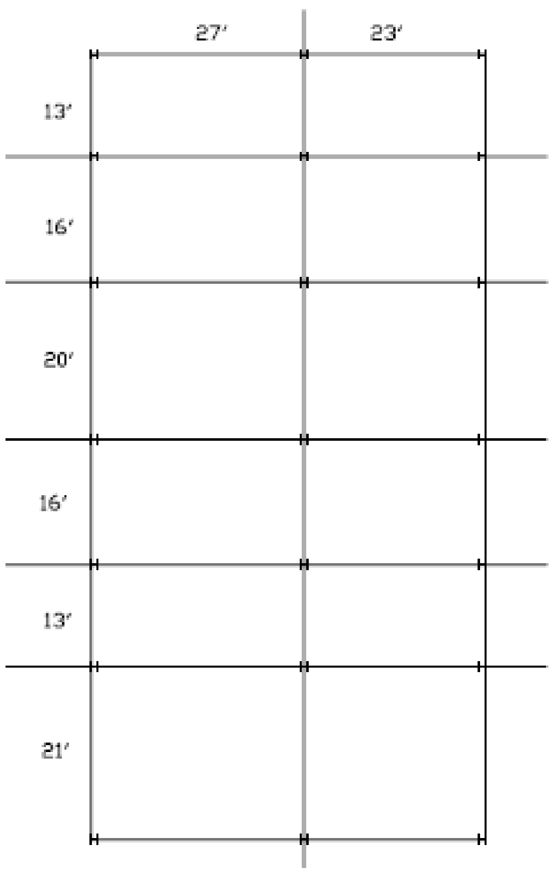
[2] Brick cavity wall with pre-cast trim, loads for 4"clay brick wythe from ASCE7 were used.

Solution:

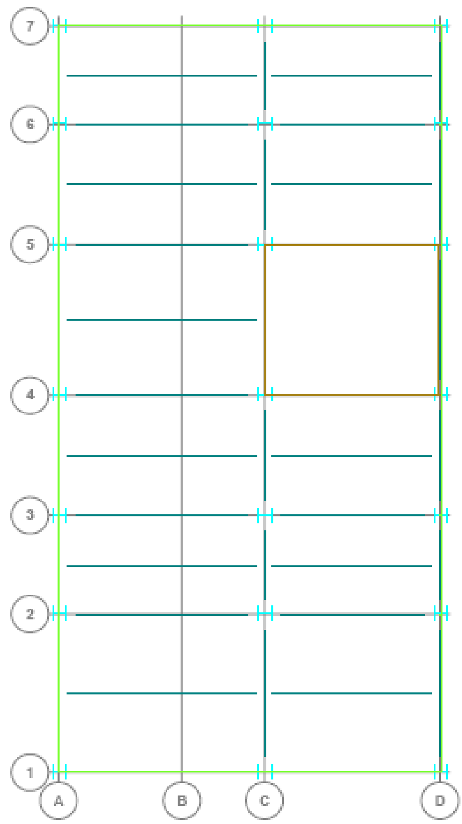
Column Grid:

Before a design was started, the column grid was looked at. The flat plate design of Lexington II used small bays sizes to create a shallow floor slab. Bays sizes as small as used in the flat plate slab design of The Lexington were impractical and uneconomic for alternative floor systems. Another problem with the existing column grid was the large number of offset columns which would create many difficult framing connections when used with a steel system.

When planning a new column grid, working around existing architecture became a main criterion. Many practical and evenly spaced grids placed columns in halls or rooms and therefore were unusable. The final column grid will require some slight change in the window layout along the west face of Lexington II. Other architecture affected by the new column grid is the placement of one closet door. All other columns line up with existing walls or mechanical shafts.



AutoCAD Grid with Spacing
Figure 8
Dimensions of Column Grid



RAM column layout
Figure 9
Column Grid with Beams

Flooring:

Once a column grid is established the floor can be designed. As determined earlier, the Lexington II design will feature composite deck. The largest bay size spans 21 feet which is too great a span for the decking. To shorten the decking span, beams were added bisecting each bay. The addition of beams changed the greatest span length to 10.5 feet.

Decking was designed using the *United Steel Deck; Steel Decks for Floors and Roof* design manual and catalog of products. Many various composite decks worked. The decking I chose is at follows:

Residential Levels:	2” Lok-Floor, 22 gage, 4.5” slab depth, unshored
Public Levels:	3” Lok-Floor, 22 gage, 5.5” slab depth, unshored

These designs were chosen because they were the minimum required deck and slab to span the lengths unshored. Had shoring been used, additional costs for the labor, materials, and time needed to shore may affect the construction price. Unshored construction may however require a slight amount of extra concrete to account for the immediate deflection of the slab under its own weight. The extra concrete would be used to even out and create a flat floor.

Beams:

Beams for Lexington II were designed using RAM. The gravity loads, decking, and slab were all input into RAM along with the framing plan of The Lexington. Through finite element analysis RAM is able to calculate the required beam sizes. For the composite construction of Lexington II, RAM is also able to calculate the number of shear shuts needed along each beam. All loads entered into RAM complied with ASCE 7, and RAM was set to design all steel in accordance with LRFD 3rd Edition. For full beam summary, see Appendix Table A-4.

Columns:

Columns were also designed using RAM. The column designs in RAM are for the gravity loads, and therefore the column designs given by RAM will only be used for columns that are not a part of the lateral force resisting system.

Full Beam and Column Designs are as follows.

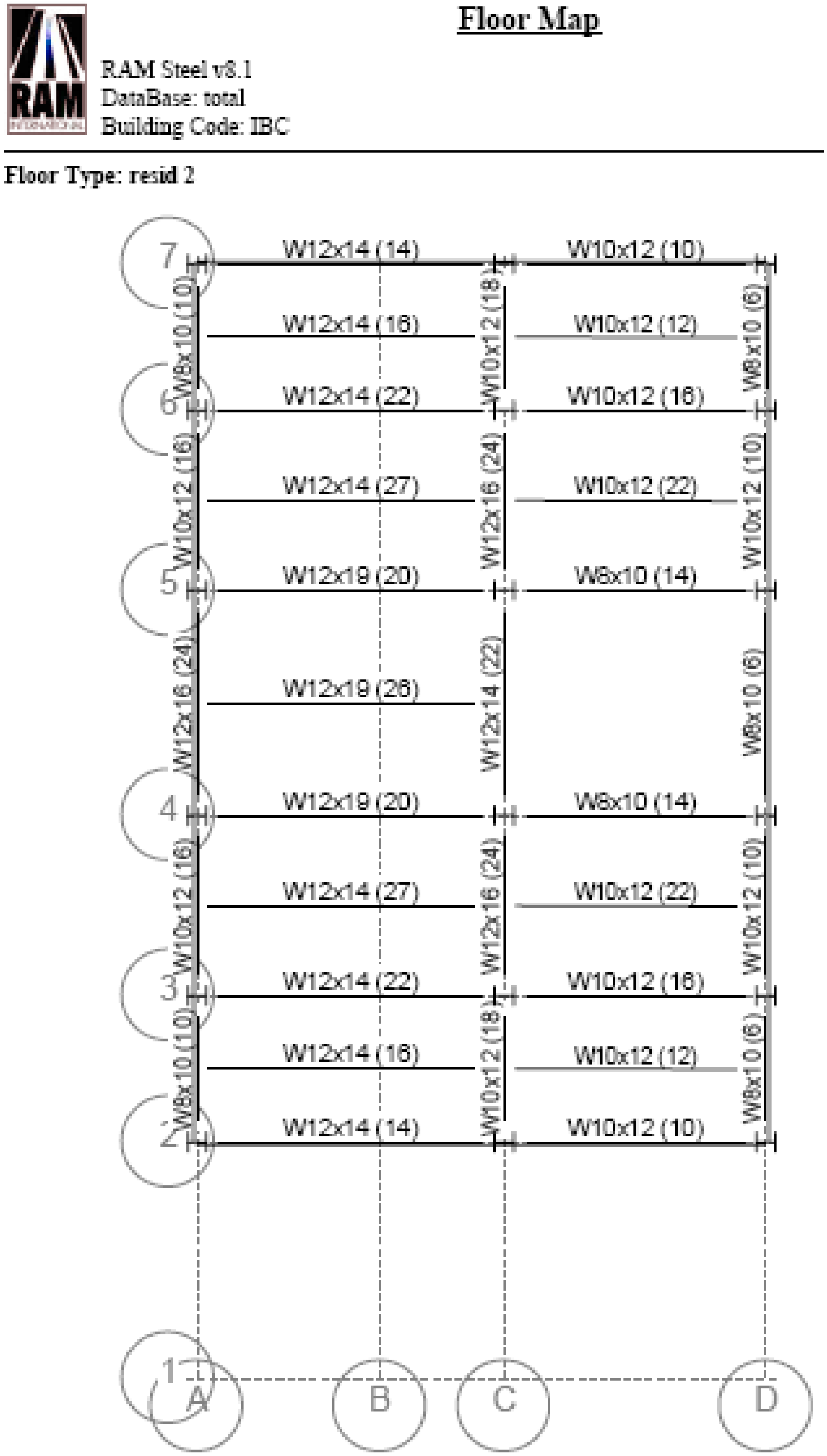


Figure 10
Beam Design for Levels 12-8

Floor Map

RAM Steel v8.1
DataBase: total
Building Code: IBC

Floor Type: resid 1

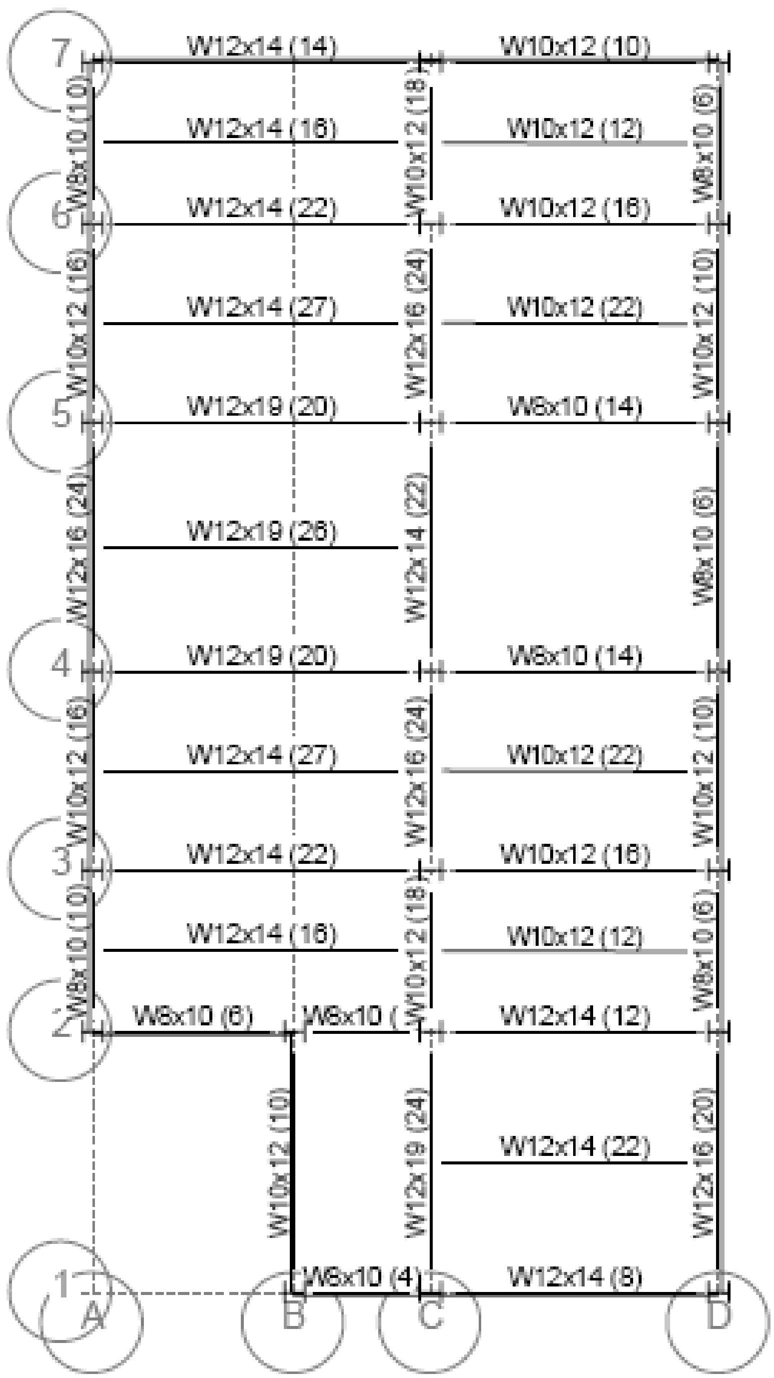


Figure 11
Beam Design for Levels 7-2

Floor Map

RAM Steel v8.1
DataBase: total
Building Code: IBC

Floor Type: Ground

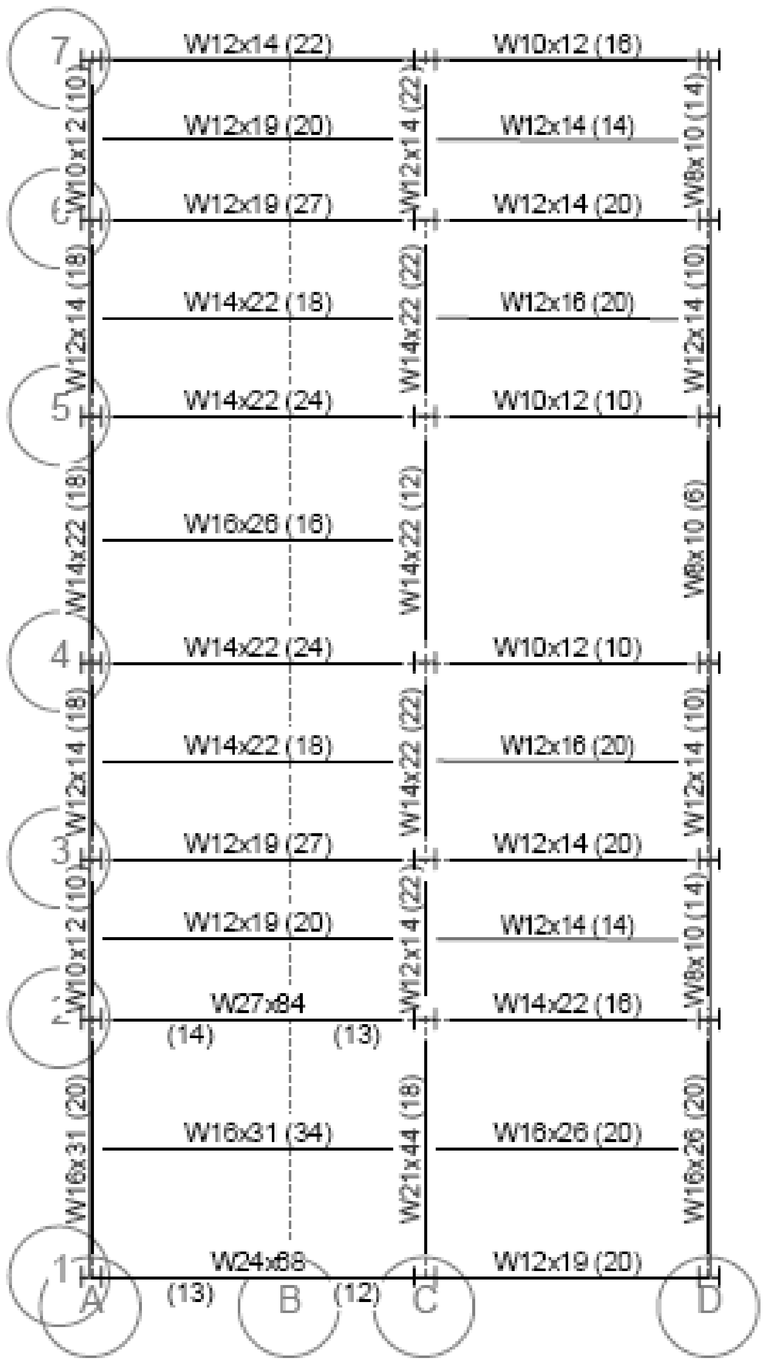


Figure 12
Ground Floor Beam Design

Floor Map

RAM Steel v8.1
DataBase: total
Building Code: IBC

Floor Type: L-1

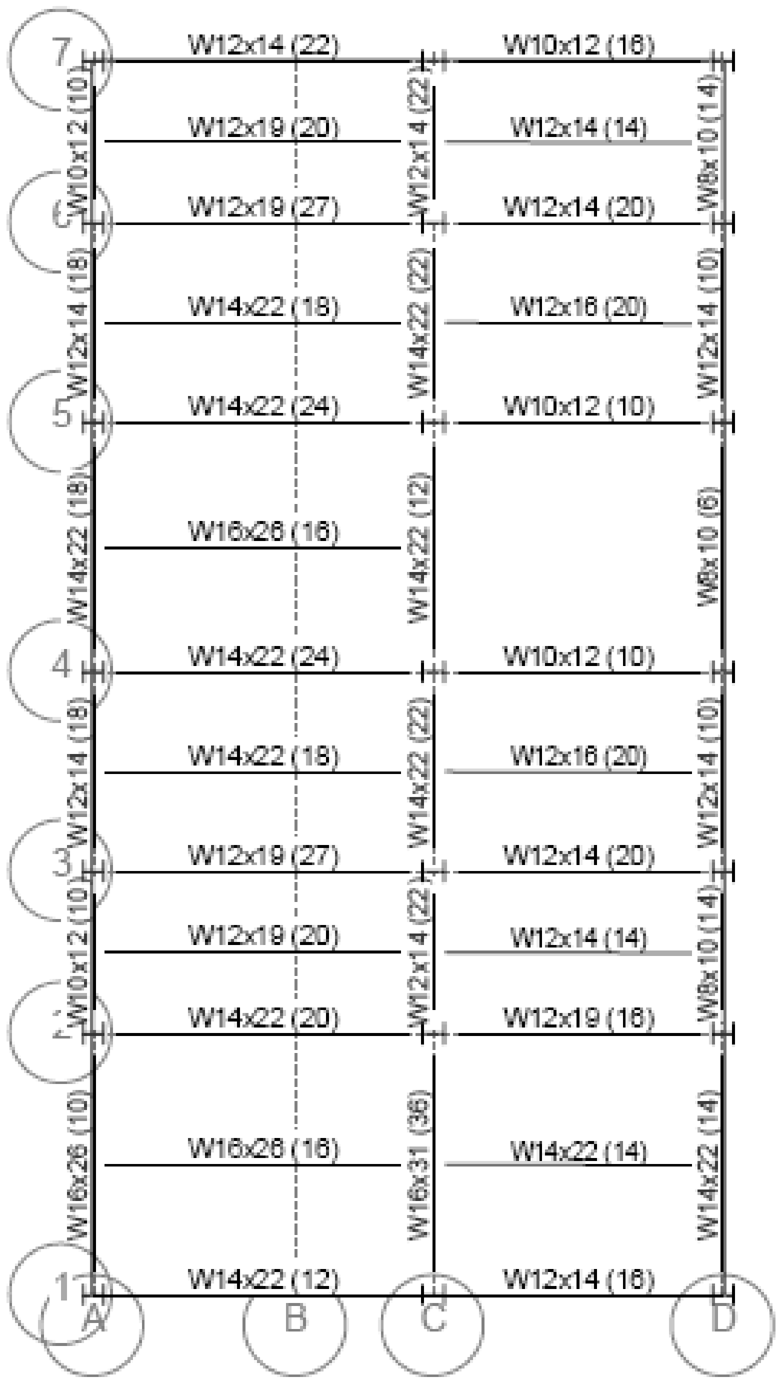


Figure 13
L-1 Beam Design

Floor Map

RAM Steel v8.1
DataBase: total
Building Code: IBC

Floor Type: Concourse

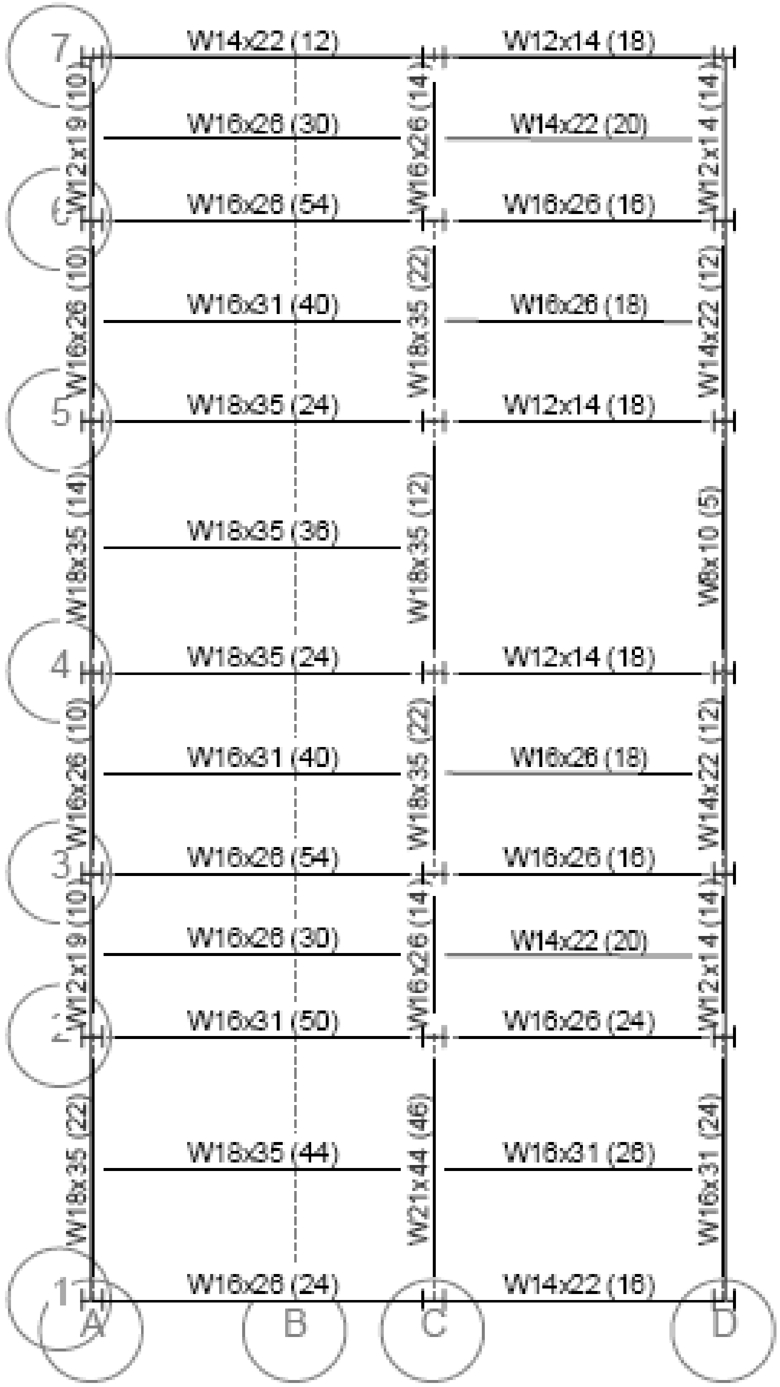


Figure 14
Concourse and P-1 Beam Design

Gravity Column Design Summary

RAM Steel v8.1
DataBase: total — 02/20/06 11:40:07
Building Code: IBC — Steel Code: AISC LRFD

Column Line A - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Ground	88.4	38.8	13.3	1	0.77 Eq H1-1a	0.0	50	W10X33
L-1	121.5	7.7	6.6	1	0.70 Eq H1-1a	0.0	50	W10X33
Concourse	171.3	9.1	7.7	1	0.81 Eq H1-1a	0.0	50	W10X33

Column Line A - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	25.1	8.4	5.9	1	0.21 Eq H1-1b	0.0	50	W10X33
11	48.2	4.2	2.9	1	0.16 Eq H1-1b	0.0	50	W10X33
10	69.1	3.9	2.7	1	0.28 Eq H1-1a	0.0	50	W10X33
9	89.7	3.8	2.6	1	0.34 Eq H1-1a	0.0	50	W10X33
8	110.0	3.7	2.6	1	0.40 Eq H1-1a	0.0	50	W10X33
7	125.5	2.2	2.5	1	0.43 Eq H1-1a	0.0	50	W10X33
6	141.0	2.2	2.5	1	0.48 Eq H1-1a	0.0	50	W10X33
5	156.4	2.2	2.5	1	0.53 Eq H1-1a	0.0	50	W10X33
4	171.8	2.1	2.5	1	0.57 Eq H1-1a	0.0	50	W10X33
3	187.0	2.1	2.4	1	0.62 Eq H1-1a	0.0	50	W10X33
2	202.3	27.4	4.8	2	0.86 Eq H1-1a	0.0	50	W10X33
Ground	301.2	20.8	2.9	2	0.87 Eq H1-1a	0.0	50	W10X45
L-1	344.1	8.4	4.4	2	0.87 Eq H1-1a	0.0	50	W10X49
Concourse	401.3	10.1	5.3	6	0.91 Eq H1-1a	0.0	50	W10X49

Column Line A - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	35.9	10.2	3.1	10	0.19 Eq H1-1b	0.0	50	W10X33
11	72.7	4.8	1.3	4	0.27 Eq H1-1a	0.0	50	W10X33
10	105.1	4.5	1.2	5	0.36 Eq H1-1a	0.0	50	W10X33
9	137.0	4.3	1.2	4	0.46 Eq H1-1a	0.0	50	W10X33
8	168.6	4.3	1.1	4	0.55 Eq H1-1a	0.0	50	W10X33
7	199.9	4.2	1.1	4	0.65 Eq H1-1a	0.0	50	W10X33
6	231.1	4.1	1.1	4	0.74 Eq H1-1a	0.0	50	W10X33
5	262.1	4.1	1.0	4	0.83 Eq H1-1a	0.0	50	W10X33
4	293.0	4.1	1.0	4	0.92 Eq H1-1a	0.0	50	W10X33
3	323.9	4.1	1.0	4	0.85 Eq H1-1a	0.0	50	W10X39
2	354.7	5.5	1.6	5	0.94 Eq H1-1a	0.0	50	W10X39
Ground	390.7	5.5	1.8	5	0.82 Eq H1-1a	0.0	50	W10X49
L-1	427.4	6.8	2.7	5	0.95 Eq H1-1a	0.0	50	W10X54
Concourse	476.0	8.4	3.3	10	0.95 Eq H1-1a	0.0	50	W10X54

Column Line A - 4

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	43.6	12.3	3.8	10	0.22 Eq H1-1b	0.0	50	W10X33
11	88.4	5.8	1.6	4	0.33 Eq H1-1a	0.0	50	W10X33
10	128.2	5.4	1.5	5	0.44 Eq H1-1a	0.0	50	W10X33

Gravity Column Design Summary

RAM Steel v8.1 — Page 2/8
DataBase: total — 02/20/06 11:40:07
Building Code: IBC — Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
9	167.5	5.3	1.5	4	0.56 Eq H1-1a	0.0	50	W10X33
8	206.3	5.2	1.4	4	0.68 Eq H1-1a	0.0	50	W10X33
7	244.9	5.1	1.4	4	0.79 Eq H1-1a	0.0	50	W10X33
6	283.4	5.1	1.3	4	0.91 Eq H1-1a	0.0	50	W10X33
5	321.7	5.1	1.3	4	0.86 Eq H1-1a	0.0	50	W10X39
4	359.9	5.1	1.3	4	0.95 Eq H1-1a	0.0	50	W10X39
3	398.1	5.1	1.3	4	0.90 Eq H1-1a	0.0	50	W10X45
2	437.8	6.8	2.3	5	0.86 Eq H1-1a	0.0	50	W10X49
Ground	484.1	6.9	2.3	5	0.93 Eq H1-1a	0.0	50	W10X54
L-1	530.8	8.7	3.4	5	0.93 Eq H1-1a	0.0	50	W10X68
Concourse	591.4	10.7	4.2	10	0.93 Eq H1-1a	0.0	50	W10X68

Column Line A - 5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	43.6	12.3	3.8	6	0.22 Eq H1-1b	0.0	50	W10X33
11	88.4	5.8	1.6	2	0.33 Eq H1-1a	0.0	50	W10X33
10	128.2	5.4	1.5	2	0.44 Eq H1-1a	0.0	50	W10X33
9	167.5	5.3	1.5	2	0.56 Eq H1-1a	0.0	50	W10X33
8	206.3	5.2	1.4	2	0.68 Eq H1-1a	0.0	50	W10X33
7	244.9	5.1	1.4	2	0.79 Eq H1-1a	0.0	50	W10X33
6	283.4	5.1	1.3	2	0.91 Eq H1-1a	0.0	50	W10X33
5	321.7	5.1	1.3	2	0.86 Eq H1-1a	0.0	50	W10X39
4	359.9	5.1	1.3	2	0.95 Eq H1-1a	0.0	50	W10X39
3	398.1	5.1	1.3	2	0.90 Eq H1-1a	0.0	50	W10X45
2	437.8	6.8	2.3	2	0.86 Eq H1-1a	0.0	50	W10X49
Ground	484.1	6.9	2.3	2	0.93 Eq H1-1a	0.0	50	W10X54
L-1	530.8	8.7	3.4	2	0.93 Eq H1-1a	0.0	50	W10X68
Concourse	591.4	10.7	4.2	6	0.93 Eq H1-1a	0.0	50	W10X68

Column Line A - 6

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	35.9	10.2	3.1	6	0.19 Eq H1-1b	0.0	50	W10X33
11	72.7	4.8	1.3	2	0.27 Eq H1-1a	0.0	50	W10X33
10	105.1	4.5	1.2	2	0.36 Eq H1-1a	0.0	50	W10X33
9	137.0	4.3	1.2	2	0.46 Eq H1-1a	0.0	50	W10X33
8	168.6	4.3	1.1	2	0.55 Eq H1-1a	0.0	50	W10X33
7	199.9	4.2	1.1	2	0.65 Eq H1-1a	0.0	50	W10X33
6	231.1	4.1	1.1	2	0.74 Eq H1-1a	0.0	50	W10X33
5	262.1	4.1	1.0	2	0.83 Eq H1-1a	0.0	50	W10X33
4	293.0	4.1	1.0	2	0.92 Eq H1-1a	0.0	50	W10X33
3	323.9	4.1	1.0	2	0.85 Eq H1-1a	0.0	50	W10X39
2	354.7	5.5	1.6	2	0.94 Eq H1-1a	0.0	50	W10X39
Ground	390.7	5.5	1.8	2	0.82 Eq H1-1a	0.0	50	W10X49
L-1	427.4	6.8	2.7	2	0.95 Eq H1-1a	0.0	50	W10X54
Concourse	476.0	8.4	3.3	6	0.95 Eq H1-1a	0.0	50	W10X54

Figure 15

Gravity Column Design Summary

RAM Steel v8.1 — Page 3/8
DataBase: total — 02/20/06 11:40:07
Building Code: IBC — Steel Code: AISC LRFD

Column Line A - 7

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	25.1	8.4	5.9	1	0.21 Eq H1-1b	0.0	50	W10X33
11	48.2	4.2	2.9	1	0.16 Eq H1-1b	0.0	50	W10X33
10	69.1	3.9	2.7	1	0.28 Eq H1-1a	0.0	50	W10X33
9	89.7	3.8	2.6	1	0.34 Eq H1-1a	0.0	50	W10X33
8	110.0	3.7	2.6	1	0.40 Eq H1-1a	0.0	50	W10X33
7	130.1	3.7	2.5	1	0.46 Eq H1-1a	0.0	50	W10X33
6	150.1	3.6	2.5	1	0.52 Eq H1-1a	0.0	50	W10X33
5	169.9	3.6	2.5	1	0.58 Eq H1-1a	0.0	50	W10X33
4	189.7	3.6	2.4	1	0.63 Eq H1-1a	0.0	50	W10X33
3	209.4	3.6	2.4	1	0.69 Eq H1-1a	0.0	50	W10X33
2	229.1	4.4	3.1	1	0.77 Eq H1-1a	0.0	50	W10X33
Ground	251.3	4.4	3.1	1	0.95 Eq H1-1a	0.0	50	W10X33
L-1	269.5	4.8	3.5	1	0.91 Eq H1-1a	0.0	50	W10X45
Concourse	300.4	5.9	4.3	1	0.85 Eq H1-1a	0.0	50	W10X45

Column Line B - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
7	17.5	1.3	8.1	1	0.19 Eq H1-1b	0.0	50	W10X33
6	35.0	0.7	4.0	1	0.14 Eq H1-1b	0.0	50	W10X33
5	50.6	0.7	3.9	1	0.16 Eq H1-1b	0.0	50	W10X33
4	65.5	0.7	3.7	1	0.20 Eq H1-1b	0.0	50	W10X33
3	80.2	0.7	3.7	1	0.31 Eq H1-1a	0.0	50	W10X33
2	94.7	0.7	3.6	1	0.35 Eq H1-1a	0.0	50	W10X33

Column Line B - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
7	25.0	3.9	8.0	12	0.22 Eq H1-1b	0.0	50	W10X33
6	49.1	1.8	3.7	3	0.16 Eq H1-1b	0.0	50	W10X33
5	70.7	1.7	3.5	3	0.28 Eq H1-1a	0.0	50	W10X33
4	92.0	1.7	3.5	3	0.35 Eq H1-1a	0.0	50	W10X33
3	113.1	1.6	3.4	3	0.41 Eq H1-1a	0.0	50	W10X33
2	134.0	1.3	3.3	1	0.47 Eq H1-1a	0.0	50	W10X33

Column Line C - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
7	37.5	7.9	10.9	8	0.32 Eq H1-1b	0.0	50	W10X33
6	69.6	3.6	5.0	2	0.32 Eq H1-1a	0.0	50	W10X33
5	100.6	3.5	4.8	2	0.41 Eq H1-1a	0.0	50	W10X33
4	131.2	3.4	4.6	2	0.49 Eq H1-1a	0.0	50	W10X33
3	161.4	3.4	4.5	2	0.58 Eq H1-1a	0.0	50	W10X33
2	191.4	21.2	9.6	4	0.87 Eq H1-1a	0.0	50	W10X33
Ground	208.2	14.8	7.3	1	0.91 Eq H1-1a	0.0	50	W10X45
L-1	258.7	3.6	9.6	4	0.93 Eq H1-1a	0.0	50	W10X49

Gravity Column Design Summary

RAM Steel v8.1 — Page 4/8
DataBase: total — 02/20/06 11:40:07
Building Code: IBC — Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Concourse	432.8	1.2	11.6	1	1.00 Eq H1-1a	0.0	50	W10X49

Column Line C - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	35.4	3.2	7.7	12	0.23 Eq H1-1b	0.0	50	W10X33
11	71.7	1.3	3.6	3	0.29 Eq H1-1a	0.0	50	W10X33
10	103.9	1.2	3.3	4	0.38 Eq H1-1a	0.0	50	W10X33
9	135.6	1.2	3.2	3	0.47 Eq H1-1a	0.0	50	W10X33
8	167.1	3.8	3.5	2	0.59 Eq H1-1a	0.0	50	W10X33
7	208.5	3.7	2.3	2	0.69 Eq H1-1a	0.0	50	W10X33
6	249.4	3.6	2.3	2	0.81 Eq H1-1a	0.0	50	W10X33
5	290.0	3.5	2.2	2	0.93 Eq H1-1a	0.0	50	W10X33
4	330.4	3.5	2.2	2	0.88 Eq H1-1a	0.0	50	W10X39
3	370.7	3.5	2.1	2	0.98 Eq H1-1a	0.0	50	W10X39
2	412.6	30.1	7.5	3	0.95 Eq H1-1a	0.0	50	W10X49
Ground	572.0	21.9	4.8	2	0.91 Eq H1-1a	0.0	50	W10X68
L-1	641.6	5.4	6.9	3	0.98 Eq H1-1a	0.0	50	W10X77
Concourse	737.1	1.6	8.6	6	0.88 Eq H1-1a	0.0	50	W10X88

Column Line C - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	47.1	4.6	4.3	16	0.19 Eq H1-1b	0.0	50	W10X33
11	106.6	2.0	1.9	4	0.36 Eq H1-1a	0.0	50	W10X33
10	154.5	1.8	1.7	4	0.51 Eq H1-1a	0.0	50	W10X33
9	201.8	1.7	1.6	4	0.65 Eq H1-1a	0.0	50	W10X33
8	248.6	1.7	1.6	4	0.79 Eq H1-1a	0.0	50	W10X33
7	295.1	1.6	1.5	4	0.92 Eq H1-1a	0.0	50	W10X33
6	341.7	1.6	1.5	4	0.89 Eq H1-1a	0.0	50	W10X39
5	390.6	1.6	1.5	4	0.87 Eq H1-1a	0.0	50	W10X45
4	439.6	1.6	1.5	4	0.98 Eq H1-1a	0.0	50	W10X45
3	488.5	1.6	1.8	4	0.93 Eq H1-1a	0.0	50	W10X49
2	537.5	2.7	2.9	4	0.94 Eq H1-1a	0.0	50	W10X54
Ground	596.5	2.8	3.0	4	0.88 Eq H1-1a	0.0	50	W10X68
L-1	655.9	4.5	4.7	4	0.99 Eq H1-1a	0.0	50	W10X77
Concourse	735.8	1.3	5.8	10	0.99 Eq H1-1a	0.0	50	W10X77

Column Line C - 4

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	46.9	8.9	4.9	15	0.23 Eq H1-1b	0.0	50	W10X33
11	101.0	4.0	2.1	3	0.37 Eq H1-1a	0.0	50	W10X33
10	146.3	3.8	2.0	3	0.50 Eq H1-1a	0.0	50	W10X33
9	190.9	3.7	1.9	3	0.63 Eq H1-1a	0.0	50	W10X33
8	235.2	3.7	1.9	3	0.76 Eq H1-1a	0.0	50	W10X33
7	279.1	3.6	1.8	3	0.89 Eq H1-1a	0.0	50	W10X33
6	322.9	3.6	1.8	3	0.86 Eq H1-1a	0.0	50	W10X39

Figure 16

Gravity Column Design Summary

RAM Steel v8.1 | Page 5/8
DataBase: total | 02/20/06 13:40:07
Building Code: IBC | Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
5	368.1	3.6	1.8	3	0.97 Eq H1-1a	0.0	50	W10X39
4	414.2	3.6	1.8	3	0.93 Eq H1-1a	0.0	50	W10X45
3	460.3	3.6	2.1	3	0.88 Eq H1-1a	0.0	50	W10X49
2	506.5	5.2	3.3	3	0.99 Eq H1-1a	0.0	50	W10X49
Ground	561.5	5.3	3.3	3	0.96 Eq H1-1a	0.0	50	W10X60
L-1	617.0	7.4	5.0	3	0.94 Eq H1-1a	0.0	50	W10X77
Concourse	692.6	6.6	6.2	6	0.95 Eq H1-1a	0.0	50	W10X77

Column Line C - 5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	46.9	8.9	4.9	16	0.23 Eq H1-1b	0.0	50	W10X33
11	101.0	4.0	2.1	4	0.37 Eq H1-1a	0.0	50	W10X33
10	146.3	3.8	2.0	4	0.50 Eq H1-1a	0.0	50	W10X33
9	190.9	3.7	1.9	4	0.63 Eq H1-1a	0.0	50	W10X33
8	235.2	3.7	1.9	4	0.76 Eq H1-1a	0.0	50	W10X33
7	279.1	3.6	1.8	4	0.89 Eq H1-1a	0.0	50	W10X33
6	322.9	3.6	1.8	4	0.86 Eq H1-1a	0.0	50	W10X39
5	368.1	3.6	1.8	4	0.97 Eq H1-1a	0.0	50	W10X39
4	414.2	3.6	1.8	4	0.93 Eq H1-1a	0.0	50	W10X45
3	460.3	3.6	2.1	4	0.88 Eq H1-1a	0.0	50	W10X49
2	506.5	5.2	3.3	4	0.99 Eq H1-1a	0.0	50	W10X49
Ground	561.5	5.3	3.3	4	0.96 Eq H1-1a	0.0	50	W10X60
L-1	617.0	7.4	5.0	4	0.94 Eq H1-1a	0.0	50	W10X77
Concourse	692.6	6.6	6.2	10	0.95 Eq H1-1a	0.0	50	W10X77

Column Line C - 6

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	47.1	4.6	4.3	15	0.19 Eq H1-1b	0.0	50	W10X33
11	106.6	2.0	1.9	3	0.36 Eq H1-1a	0.0	50	W10X33
10	154.5	1.8	1.7	3	0.51 Eq H1-1a	0.0	50	W10X33
9	201.8	1.7	1.6	3	0.65 Eq H1-1a	0.0	50	W10X33
8	248.6	1.7	1.6	3	0.79 Eq H1-1a	0.0	50	W10X33
7	295.1	1.6	1.5	3	0.92 Eq H1-1a	0.0	50	W10X33
6	341.7	1.6	1.5	3	0.89 Eq H1-1a	0.0	50	W10X39
5	390.6	1.6	1.5	3	0.87 Eq H1-1a	0.0	50	W10X45
4	439.6	1.6	1.5	3	0.98 Eq H1-1a	0.0	50	W10X45
3	488.5	1.6	1.8	3	0.93 Eq H1-1a	0.0	50	W10X49
2	537.5	2.7	2.9	3	0.94 Eq H1-1a	0.0	50	W10X54
Ground	596.5	2.8	3.0	3	0.88 Eq H1-1a	0.0	50	W10X68
L-1	655.9	4.5	4.7	3	0.99 Eq H1-1a	0.0	50	W10X77
Concourse	735.8	1.3	5.8	6	0.99 Eq H1-1a	0.0	50	W10X77

Column Line C - 7

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	35.4	3.2	7.7	12	0.23 Eq H1-1b	0.0	50	W10X33

Gravity Column Design Summary

RAM Steel v8.1 | Page 6/8
DataBase: total | 02/20/06 13:40:07
Building Code: IBC | Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
11	71.7	1.3	3.6	3	0.29 Eq H1-1a	0.0	50	W10X33
10	103.9	1.2	3.3	3	0.38 Eq H1-1a	0.0	50	W10X33
9	135.6	1.2	3.2	3	0.47 Eq H1-1a	0.0	50	W10X33
8	167.1	1.1	3.2	3	0.56 Eq H1-1a	0.0	50	W10X33
7	198.3	1.1	3.1	3	0.66 Eq H1-1a	0.0	50	W10X33
6	229.4	1.1	3.1	3	0.75 Eq H1-1a	0.0	50	W10X33
5	260.3	1.1	3.1	3	0.84 Eq H1-1a	0.0	50	W10X33
4	291.1	1.0	3.0	3	0.93 Eq H1-1a	0.0	50	W10X33
3	321.9	1.0	3.0	3	0.86 Eq H1-1a	0.0	50	W10X39
2	352.7	1.5	4.1	3	0.95 Eq H1-1a	0.0	50	W10X39
Ground	387.5	1.5	4.7	3	0.82 Eq H1-1a	0.0	50	W10X49
L-1	422.5	2.3	5.8	3	0.95 Eq H1-1a	0.0	50	W10X54
Concourse	472.0	0.9	7.1	1	0.94 Eq H1-1a	0.0	50	W10X54

Column Line D - 1

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
7	31.2	9.5	8.2	1	0.27 Eq H1-1b	0.0	50	W10X33
6	57.6	4.4	3.8	1	0.19 Eq H1-1b	0.0	50	W10X33
5	83.2	4.2	3.7	1	0.34 Eq H1-1a	0.0	50	W10X33
4	108.4	4.1	3.6	1	0.41 Eq H1-1a	0.0	50	W10X33
3	133.4	4.1	3.5	1	0.49 Eq H1-1a	0.0	50	W10X33
2	158.2	6.6	5.8	1	0.62 Eq H1-1a	0.0	50	W10X33
Ground	194.9	5.1	4.5	1	0.78 Eq H1-1a	0.0	50	W10X33
L-1	223.0	5.9	5.1	1	0.92 Eq H1-1a	0.0	50	W10X39
Concourse	263.5	7.1	6.4	1	0.92 Eq H1-1a	0.0	50	W10X39

Column Line D - 2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	21.8	7.2	5.3	1	0.18 Eq H1-1b	0.0	50	W10X33
11	42.6	3.6	2.6	1	0.14 Eq H1-1b	0.0	50	W10X33
10	61.0	3.4	2.4	1	0.18 Eq H1-1b	0.0	50	W10X33
9	79.1	3.3	2.4	1	0.30 Eq H1-1a	0.0	50	W10X33
8	96.9	4.8	2.5	3	0.36 Eq H1-1a	0.0	50	W10X33
7	128.0	4.4	1.9	2	0.44 Eq H1-1a	0.0	50	W10X33
6	160.8	4.3	1.8	3	0.54 Eq H1-1a	0.0	50	W10X33
5	193.2	4.2	1.8	2	0.64 Eq H1-1a	0.0	50	W10X33
4	225.5	4.2	1.8	2	0.73 Eq H1-1a	0.0	50	W10X33
3	257.5	4.1	1.7	2	0.83 Eq H1-1a	0.0	50	W10X33
2	289.5	6.9	3.6	3	0.97 Eq H1-1a	0.0	50	W10X33
Ground	334.4	5.7	2.5	3	0.88 Eq H1-1a	0.0	50	W10X45
L-1	371.6	6.9	3.7	3	0.92 Eq H1-1a	0.0	50	W10X49
Concourse	420.7	8.4	4.5	6	0.94 Eq H1-1a	0.0	50	W10X49

Column Line D - 3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size

Figure 17

Gravity Column Design Summary

RAM Steel v8.1
DataBase: total
Building Code: IBC

Page 7/8
02/20/06 13:40:07
Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	32.0	8.9	2.8	10	0.16 Eq H1-1b	0.0	50	W10X33
11	64.6	4.2	0.5	1	0.19 Eq H1-1b	0.0	50	W10X33
10	93.3	3.8	1.1	4	0.32 Eq H1-1a	0.0	50	W10X33
9	121.5	3.7	1.0	4	0.41 Eq H1-1a	0.0	50	W10X33
8	149.4	3.7	1.0	4	0.49 Eq H1-1a	0.0	50	W10X33
7	177.0	3.6	1.0	4	0.57 Eq H1-1a	0.0	50	W10X33
6	204.5	3.6	1.0	4	0.65 Eq H1-1a	0.0	50	W10X33
5	231.9	3.5	0.9	4	0.74 Eq H1-1a	0.0	50	W10X33
4	259.2	3.5	0.9	4	0.82 Eq H1-1a	0.0	50	W10X33
3	286.4	3.5	0.9	4	0.90 Eq H1-1a	0.0	50	W10X33
2	313.5	4.7	1.4	4	1.00 Eq H1-1a	0.0	50	W10X33
Ground	345.1	4.8	1.4	4	0.89 Eq H1-1a	0.0	50	W10X45
L-1	376.7	5.9	2.4	4	0.92 Eq H1-1a	0.0	50	W10X49
Concourse	417.7	7.2	2.9	10	0.91 Eq H1-1a	0.0	50	W10X49

Column Line D - 4

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	25.7	5.7	4.3	6	0.16 Eq H1-1b	0.0	50	W10X33
11	49.5	2.7	1.9	1	0.15 Eq H1-1b	0.0	50	W10X33
10	71.0	2.5	1.8	3	0.26 Eq H1-1a	0.0	50	W10X33
9	92.1	2.4	1.8	2	0.32 Eq H1-1a	0.0	50	W10X33
8	112.9	2.3	1.7	2	0.38 Eq H1-1a	0.0	50	W10X33
7	133.5	2.3	1.7	2	0.44 Eq H1-1a	0.0	50	W10X33
6	154.0	2.3	1.6	2	0.50 Eq H1-1a	0.0	50	W10X33
5	174.3	2.2	1.6	2	0.57 Eq H1-1a	0.0	50	W10X33
4	194.6	2.2	1.6	2	0.63 Eq H1-1a	0.0	50	W10X33
3	214.8	2.2	1.6	2	0.69 Eq H1-1a	0.0	50	W10X33
2	234.9	2.9	2.2	3	0.76 Eq H1-1a	0.0	50	W10X33
Ground	257.3	2.9	2.2	3	0.94 Eq H1-1a	0.0	50	W10X33
L-1	279.9	3.7	3.0	3	0.93 Eq H1-1a	0.0	50	W10X45
Concourse	312.0	4.5	3.8	6	0.87 Eq H1-1a	0.0	50	W10X45

Column Line D - 5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	25.7	5.7	4.3	10	0.16 Eq H1-1b	0.0	50	W10X33
11	49.5	2.7	1.9	1	0.15 Eq H1-1b	0.0	50	W10X33
10	71.0	2.5	1.8	4	0.26 Eq H1-1a	0.0	50	W10X33
9	92.1	2.4	1.8	4	0.32 Eq H1-1a	0.0	50	W10X33
8	112.9	2.3	1.7	4	0.38 Eq H1-1a	0.0	50	W10X33
7	133.5	2.3	1.7	4	0.44 Eq H1-1a	0.0	50	W10X33
6	154.0	2.3	1.6	4	0.50 Eq H1-1a	0.0	50	W10X33
5	174.3	2.2	1.6	4	0.57 Eq H1-1a	0.0	50	W10X33
4	194.6	2.2	1.6	4	0.63 Eq H1-1a	0.0	50	W10X33
3	214.8	2.2	1.6	4	0.69 Eq H1-1a	0.0	50	W10X33
2	234.9	2.9	2.2	4	0.76 Eq H1-1a	0.0	50	W10X33

Gravity Column Design Summary

RAM Steel v8.1
DataBase: total
Building Code: IBC

Page 8/8
02/20/06 13:40:07
Steel Code: AISC LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Ground	257.3	2.9	2.2	4	0.94 Eq H1-1a	0.0	50	W10X33
L-1	279.9	3.7	3.0	4	0.93 Eq H1-1a	0.0	50	W10X45
Concourse	312.0	4.5	3.8	10	0.87 Eq H1-1a	0.0	50	W10X45

Column Line D - 6

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	32.0	8.9	2.8	6	0.16 Eq H1-1b	0.0	50	W10X33
11	64.6	4.2	0.5	1	0.19 Eq H1-1b	0.0	50	W10X33
10	93.3	3.8	1.1	3	0.32 Eq H1-1a	0.0	50	W10X33
9	121.5	3.7	1.0	2	0.41 Eq H1-1a	0.0	50	W10X33
8	149.4	3.7	1.0	2	0.49 Eq H1-1a	0.0	50	W10X33
7	177.0	3.6	1.0	2	0.57 Eq H1-1a	0.0	50	W10X33
6	204.5	3.6	1.0	2	0.65 Eq H1-1a	0.0	50	W10X33
5	231.9	3.5	0.9	2	0.74 Eq H1-1a	0.0	50	W10X33
4	259.2	3.5	0.9	2	0.82 Eq H1-1a	0.0	50	W10X33
3	286.4	3.5	0.9	2	0.90 Eq H1-1a	0.0	50	W10X33
2	313.5	4.7	1.4	3	1.00 Eq H1-1a	0.0	50	W10X33
Ground	345.1	4.8	1.4	3	0.89 Eq H1-1a	0.0	50	W10X45
L-1	376.7	5.9	2.4	3	0.92 Eq H1-1a	0.0	50	W10X49
Concourse	417.7	7.2	2.9	6	0.91 Eq H1-1a	0.0	50	W10X49

Column Line D - 7

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
12	21.8	7.2	5.3	1	0.18 Eq H1-1b	0.0	50	W10X33
11	42.6	3.6	2.6	1	0.14 Eq H1-1b	0.0	50	W10X33
10	61.0	3.4	2.4	1	0.18 Eq H1-1b	0.0	50	W10X33
9	79.1	3.3	2.4	1	0.30 Eq H1-1a	0.0	50	W10X33
8	96.9	3.2	2.3	1	0.35 Eq H1-1a	0.0	50	W10X33
7	114.5	3.2	2.3	1	0.40 Eq H1-1a	0.0	50	W10X33
6	132.0	3.1	2.2	1	0.45 Eq H1-1a	0.0	50	W10X33
5	149.4	3.1	2.2	1	0.51 Eq H1-1a	0.0	50	W10X33
4	166.7	3.1	2.2	1	0.56 Eq H1-1a	0.0	50	W10X33
3	184.0	3.1	2.2	1	0.61 Eq H1-1a	0.0	50	W10X33
2	201.2	3.8	2.8	1	0.68 Eq H1-1a	0.0	50	W10X33
Ground	220.7	3.8	2.8	1	0.83 Eq H1-1a	0.0	50	W10X33
L-1	240.2	4.2	3.2	1	0.95 Eq H1-1a	0.0	50	W10X39
Concourse	263.3	5.0	4.0	1	0.87 Eq H1-1a	0.0	50	W10X39

Figure 18

Design- Lateral System:

Investigation:

When considering the design of the lateral load resisting system, first a look was taken at the existing shear walls. The existing lateral system in The Lexington consists of a core of shear walls designed as the elevator shaft of the building. Before a new lateral system was investigated, the existing shear walls were again considered as the main lateral force resisting system.

The elevator core is an architectural feature of the building and, as such, will remain unchanged in the new building design. The elevator shaft therefore lends itself nicely as shear walls which will run the entire height of the building uninterrupted. Logic shows that the existing shear wall specifications are adequate to carry the new lateral loads which may be applied to the building. By changing the building system to a steel frame, the floor sandwich depth of each level was doubled resulting in a total height change of approximately 8'. Although this change in height will add additional wind load to the building, it was the seismic lateral loads which were the controlling load case for the upper stories, the wind load on the lower stories will remain comparable to the current load. The seismic load associated with the steel redesign of The Lexington will also differ from the load applied to the existing building. As the Lexington gravity system was redesigned in steel, the weights of the building were changed altering the seismic load the building will receive. By converting the current concrete system to a composite steel system, the building weight will be reduced. In turn, this lighter weight will cause a reduction in the seismic load making the seismic load on the new building design less then the load on the original design. With this considered, the shear walls used in the original design of the Lexington will be able to carry the new load. The reinforcement and materials used in the current shear walls are already minimal and a reduction in the size and reinforcement of the existing shear wall is unnecessary.

To verify the above assumptions, the same ETABS model used when evaluating the existing shear walls was altered to evaluate the shear walls when used with the composite floor system. Changes to the ETABS model include increasing the building height by adding 8 ½" to each floor and reducing the dead load to 32 psf. The results of the shear wall analysis were very similar to the results calculated for the existing structure and are as predicted above.

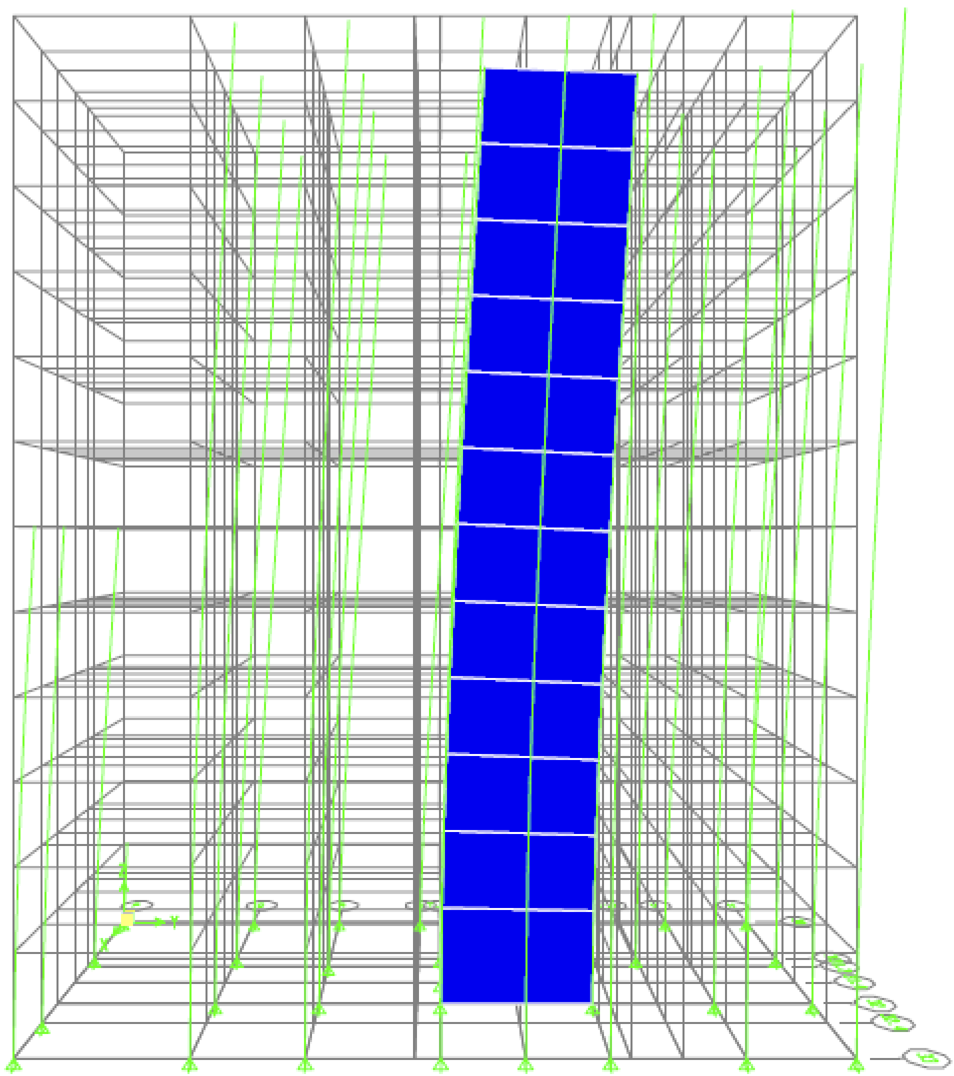


Figure 19
ETABS Model. Shear wall results auto scaled by 3000

Although the existing shear walls work, it may be in the interest of the designer to try other lateral force resisting systems as well. By considering alternatives a more cost efficient solution may be possible. The other systems considered in the redesign of The Lexington include moment and braced frames. Both moment and braced frames have advantages and disadvantages.

Moment frames were the first system I considered for The Lexington. The greatest benefit of using a moment frame in The Lexington is that moment frames are unobtrusive. This will allow for moment frames to be placed in bays that span living areas and other spaces that must remain open. The biggest disadvantage with moment frames in The Lexington, like in most buildings, is the cost associated with them. Moment frames would require very specific connection detailing and assembly.

Braced frames were also a possibility for use in The Lexington. Braced frames are easier to erect from a constructability aspect and have higher strength and stiffness than other lateral systems. The only obstacle in placing braced frames is to find locations where they will not interrupt the architecture of the building and can be concealed in walls without obstructing window or door placement.

The design decided on for The Lexington, was to use braced frames in place of the shear walls. The braced frames will be easier and more cost efficient to construct than the moment frames. Braced frames can be located in the same place as the exiting shear walls, around the building's elevators and stairwells, to avoid interference with the architecture. If additional braced frames are needed, they can be placed along the exterior walls of The Lexington which abut the adjoining building. I have also found several other frames in which braces can be added with minimal conflict with the existing architectural.

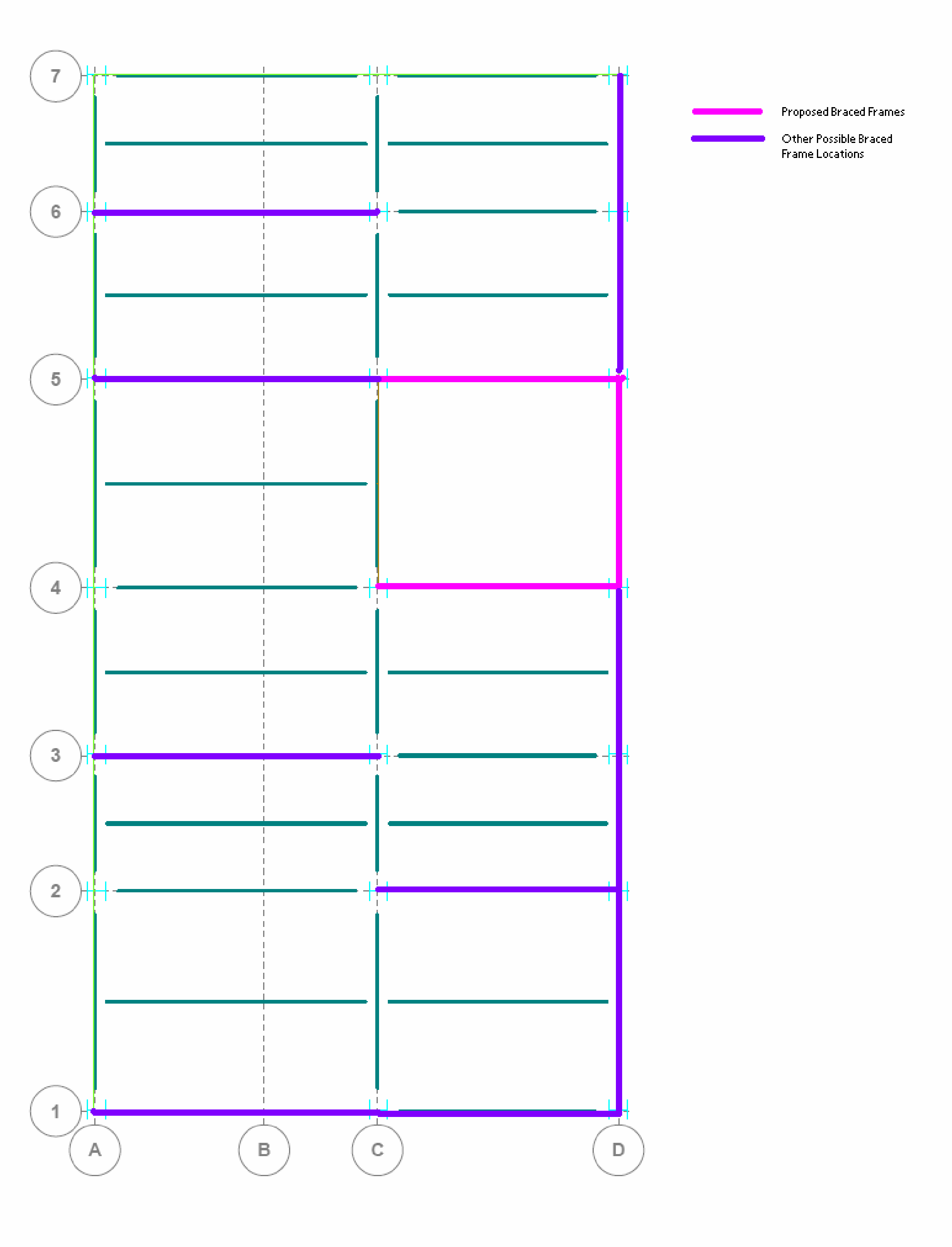


Figure 20
Proposed Brace Frame Locations

Loads:

Before the new lateral system can be designed, the lateral loads affecting the building must be calculated. The lateral loads on the redesign will differ from the loads acting on the current building because of changes in the height and weight of the building.

The height of the building will be increased to accommodate the new floor sandwich associated with the steel gravity system. The total depth of the new system includes 12" deep beams and 2" composite deck topped with a 2.5" concrete slab, or 16.5 total inches. In comparison, the existing building used 8" flat plate two way slab. The total height difference of the building is 8.5" per floor for 12 above grade floors, which add 8.5' to the building height. The new height information was input into the same excel spreadsheet used to calculate the wind loads on the actual building. Results of wind loading on the new height are as below:

N/S direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)	
ground	21.93	294.00	6.45	159.20	0.00	
1	21.93	526.75	11.55	152.75	138.63	
2	23.01	465.50	10.71	141.20	230.31	
3	24.21	465.50	11.27	130.49	349.40	
4	24.81	465.50	11.55	119.21	467.78	
5	24.81	465.50	11.55	107.66	577.51	
6	25.29	481.06	12.17	96.11	723.95	
7	25.77	496.13	12.79	83.95	890.38	
8	26.25	495.88	13.02	71.16	1038.20	
9	26.61	496.13	13.20	58.14	1186.64	
10	26.97	496.13	13.38	44.94	1338.18	
11	26.97	496.13	13.38	31.56	1473.67	
12	27.57	451.41	12.45	18.18	1496.71	
roof	28.17	203.35	5.73	5.73	736.45	moment total
						10647.84

E/W direction

Floor	P (net)	Trib Area (ft^2)	Fx (kips)	Vx (kips)	Mx (kip ft)	
ground	11.75	600.00	7.05	193.95	0.00	
1	11.75	1075.00	12.63	186.90	151.52	
2	12.83	950.00	12.18	174.27	261.97	
3	14.03	950.00	13.32	162.09	413.07	
4	14.63	950.00	13.89	148.76	562.74	
5	14.63	950.00	13.89	134.87	694.74	
6	15.11	981.75	14.83	120.98	882.41	
7	15.59	1012.50	15.78	106.14	1098.90	
8	16.07	1012.00	16.26	90.36	1296.65	
9	16.43	1012.50	16.63	74.10	1494.75	
10	16.79	1012.50	17.00	57.47	1699.60	
11	16.79	1012.50	17.00	40.48	1871.69	
12	17.39	921.25	16.02	23.48	1926.05	moment total
roof	17.99	415.00	7.46	7.46	959.54	13313.62

Table 4

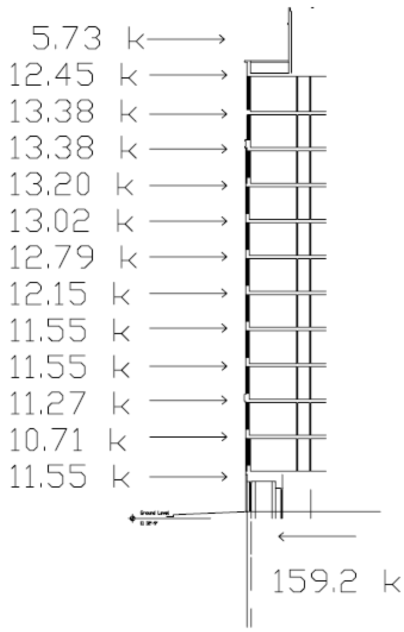
The other lateral load to be recalculated was the seismic load. Like the wind load, the excel spreadsheet used to calculate the load on the existing building was reused with proper adjustments. In the seismic case, data on the spread sheet was changed to reflect the building's new weight. The weight of the composite deck and slab was given from the decking catalog as 34 psf. The average weight of the steel framing system was determined by multiplying the weight of the beam in lb/in by the length of the beam in inches. The total weight for every beam on a floor was added together to find the framing system weight and then divided by the area of the floor to achieve the units of psf. This had to be done twice for the two varying floor plans used on residential levels. Along with the weights, other factors had to be changed to comply with the ASCE 7 code. These include the building height, response modification factor (R), and any variable which was affected by the type of lateral load resisting system used, such as C_t, x, W_o, and C_d. Results of the earthquake loading are as follows below:

Floor	height (ft)	Total Load (kips)	wx*hx^k	Cvx	Fx (kips)	Vx (kips)	Mx (kip ft)
roof	120.25	294.37	258349.75	0.16	4.35		523.40
12	110.125	333.25	258245.95	0.16	4.35	4.35	479.14
11	100	333.25	225301.95	0.14	3.80	8.70	379.58
10	89.875	333.25	193715.33	0.12	3.26	12.50	293.32
9	79.75	333.21	163555.00	0.10	2.76	15.76	219.75
8	69.635	333.25	135010.01	0.08	2.27	18.52	158.39
7	59.5	380.56	123414.56	0.08	2.08	20.79	123.72
6	50	386.63	98024.68	0.06	1.65	22.87	82.57
5	40.5	386.63	72751.46	0.04	1.23	24.52	49.64
4	31	386.63	49839.04	0.03	0.84	25.75	26.03
3	21.5	386.63	29695.68	0.02	0.50	26.59	10.76
2	12	388.90	13088.12	0.01	0.22	27.09	2.65
Ground	0	368.22	0.00	0.00	0.00	27.31	0.00
			1620991.54				

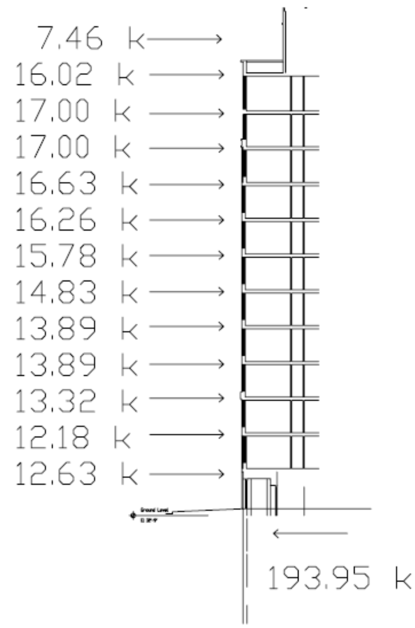
Total Building Weight (kips)	4644.75
Overturning Moment	2348.95

Table 5

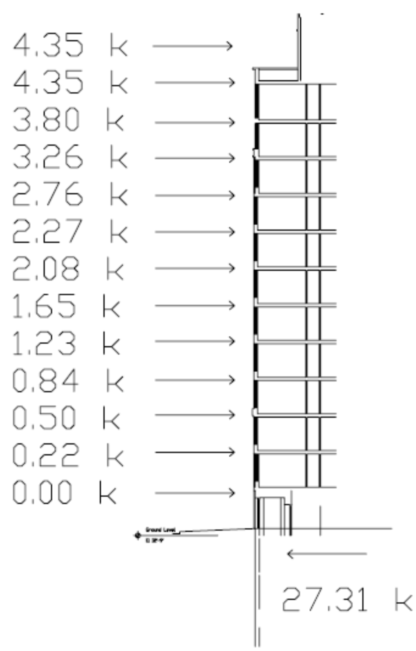
Story forces must be found. Story forces are the forces which will act on each floor and are called F_x in the wind and seismic load tables.



N-S wind forces



E-W wind forces



Earthquake Forces

Figure 21

To determine the worst case lateral load experienced by The Lexington, the load combinations below from ASCE 7 were analyzed.

$1.4D$
$1.2D + 1.6L + 1.6L_r$
$1.2D + 1.6L_r + (L \text{ OR } .8W)$
$1.2D + 1.6W + .5L + .5L_r$
$1.2D + 1E + .2S$
$.9D + 1.6W + 1.6H$
$.9D + 1E + 1.6H$

It is obvious from observation of the story forces, that the wind loading is the more critical loading case for The Lexington. In the above load combinations, the wind load will have an additional increase of 1.6 while the earthquake load would remain the same with a factor of 1, making the wind load case all the more critical. The controlling load combination for the new design of The Lexington in both directions is:

$$1.2D + 1.6W + .5L + .5L_r.$$

Distribution of Loads:

The loads were distributed to each frame based on rigidity. The rigidity of the lateral elements is affected by their member sizes, moments of inertia, and geometry. The first consideration in picking a braced frame shape is the building's architecture. However, the proposed frames will not obstruct any doors or windows and can be used with any frame geometry. A simple analysis in STAAD was run to compare the rigidity of braced frames. This resulted in the X brace as having the greatest stiffness, followed by the chevron, single diagonal and finally inverted chevron. Using an X or K frame will result in more connections and may therefore be more costly if the extra stiffness is not needed. The first braced frame model analyzed was for chevron frames in only the frames around the elevator core. The two frames spanning east to west will be identical to each other, and the north- south frame will only differ due to length. Distribution by rigidity also depends on the distance of the frame to the center of rigidity. Because there is only one frame spanning N-S, it will be the x coordinate of the center of rigidity. The y coordinate will be directly between the two walls spanning the e-w direction since they have the same stiffness as each other.

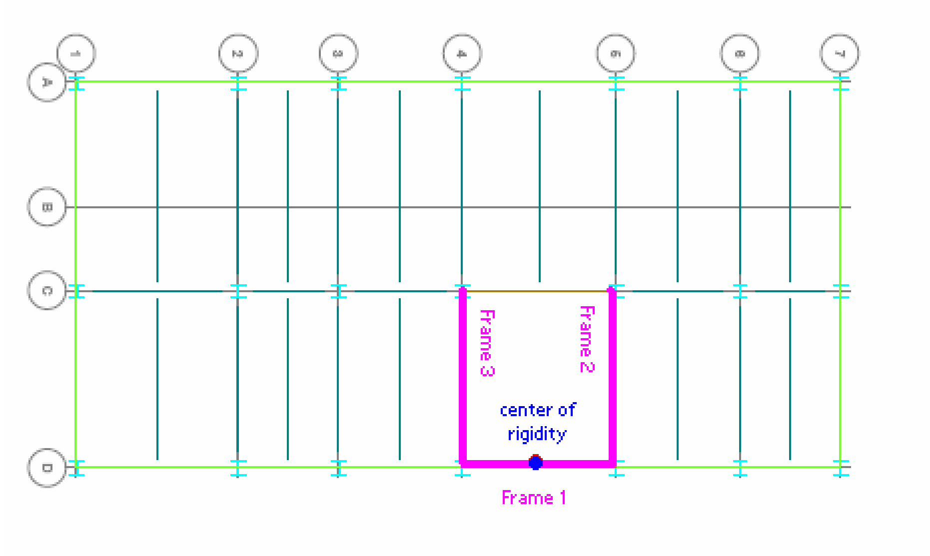


Figure 22

N-S

F_{direct}

Frame 1 = 100%$F_{n\text{-}s}$
Frame 2 = 0%$F_{n\text{-}s}$
Frame 3 = 0%$F_{n\text{-}s}$

$F_{torisonal}$

Frame 1 = 0
Frame 2 = 5%M
Frame 3 = 5%M

E-W

F_{direct}

Frame 1 = 0% $F_{e\text{-}w}$
Frame 2 = 50%$F_{e\text{-}w}$
Frame 3 = 50%$F_{e\text{-}w}$

$F_{torisonal}$

Frame 1 = 0
Frame 2 = 5%M
Frame 3 = 5%M

When wind loading is the controlling lateral load case, ASCE 7 prescribes 4 wind cases which should be checked for each building. These 4 load cases vary by percent of wind load acting on the building, and by eccentricity. All 4 cases have been checked; case 1 is the controlling wind case for frame 1, and case 3 is the controlling load case for frames 2 and 3.

Solution:

A rough estimate of the stiffness of the building can be computed based on allowable deflection. The deflection criteria used for buildings is called the drift index, where Δ/Story Height. It is common practice to use allowable Δ = H/400. For the new height of The Lexington,

$$H/400 = 120.25/ 400 = .3 \text{ ft or } 3.6 \text{ inches}$$

Using allowable drift, the stiffness needed in each frame can be calculated.

$$\Delta = \text{Story shear/ Stiffness}$$

$$K = AE \cos^2\theta /L$$

By solving for stiffness and then for area, a rough size for the bracing members was determined. Using STAAD, the frame was modeled and analyzed for the wind load case. The wind load was applied as a point load at each level. Because ASCE 7 wind load case one controlled, the story forces were as calculated in the excel wind load spreadsheet. (For the E-W frames, the story loads are taken at 75% but applied in both directions creating a larger moment to be resisted, in compliance with ASCE 7 wind load case 3).

To design the braced frames, the allowable stress on each member as well as the overall deflection of the frame must be considered. By running a model in STADD, the average stresses and hence axial loads in the columns can be found as approximately 1200 kips. The columns must be able to support both this wind load, as well as the load contributed by the gravity loading. The column is then sized by Table 4-2 of the LRFD manual, design strength in axial compression. This same method of sizing beams is applied to each member in the frame, starting at the top and working downward. The frame is then re-tested in STADD for the wind load. The final design is:

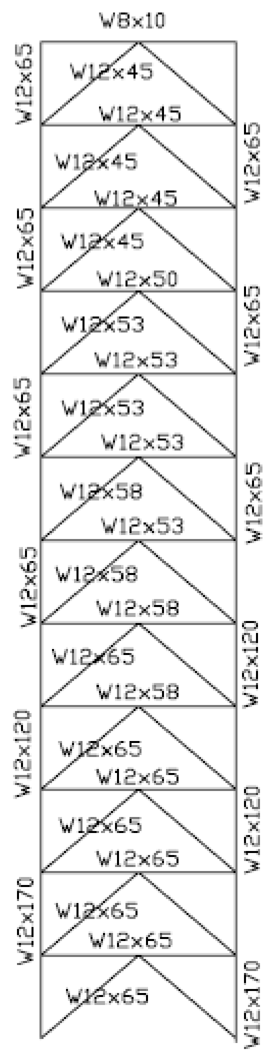


Figure 23
The frame is symmetric, any beam not labeled is the same as its counterpart.

The same frame design is also being used in the N-S direction. Although the N-S direction has smaller loads acting on the frame then the E-W direction, the change in member size is slight compared to the benefit of using repetitive members.

Before this frame can be used, the columns' members must be checked for biaxial bending since two of the columns are used in both E-W and N-S frames. In these columns, biaxial moment will control and there will be bending around the weak axis as well as the strong axis. To design these columns the AISC code was used with equation H1-1b: $\frac{Pu}{2\phi Pn} + \left(\frac{Mux}{\phi Mnx} + \frac{Muy}{\phi Muy} \right) \leq 1$. The final column designs for the biaxially loaded columns are much larger.

Story	Design
1	14 x 342
2	14 x 342
3	14 x 311
4	12 x 336
5	12 x 305
6	12 x 279
7	12 x 252
8	12 x 230
9	12 x 190
10	12 x 152
11	12 x 106
12	12 x 65

Table 6

The frame has total deflection of 2.7", which is less then the allowable 3.6".

Other Structural Considerations:

Connections:

The first connection designed was a typical beam to column connection. This connection will transfer both shear and moment from the beam into the column. The connection I designed was for the 7th story. The beam to column connection will be similar on levels 12 to 2 since all residential levels support the same gravity load.

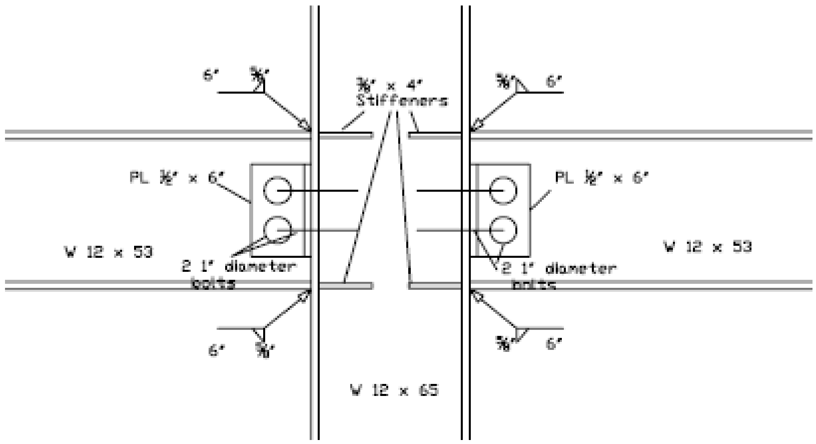


Figure 24

The final connection design is beams attached to the columns using T plates. Two 1" bolts connect the column to the plate. Two 1" bolts also connect plate to the beam. The moment in the beams is transferred to the column through tension and compression welds that connect the beam flanges to the column flange. Stiffener plates are required to prevent local flange bending in the column, local web yielding in the column, and local web crippling in the column. The welds connecting the stiffeners to the column are 3/8" fillet welds, not shown in above connection.

A sample connection has been designed for a lateral brace framing into a column and beam. The connection is for the bracing on the 7th story of Lexington II. The 7th story was designed to be an average connection for the building, with levels above 7 using less material and levels below 7 using more materials. Level 7 was also chosen because the column above and below the floor are the same size and therefore splicing will not be needed in the area. The connection includes two angles connecting the bracing member to a gusset plate. The gusset plate is then connected to both the beam and column by additional angles.

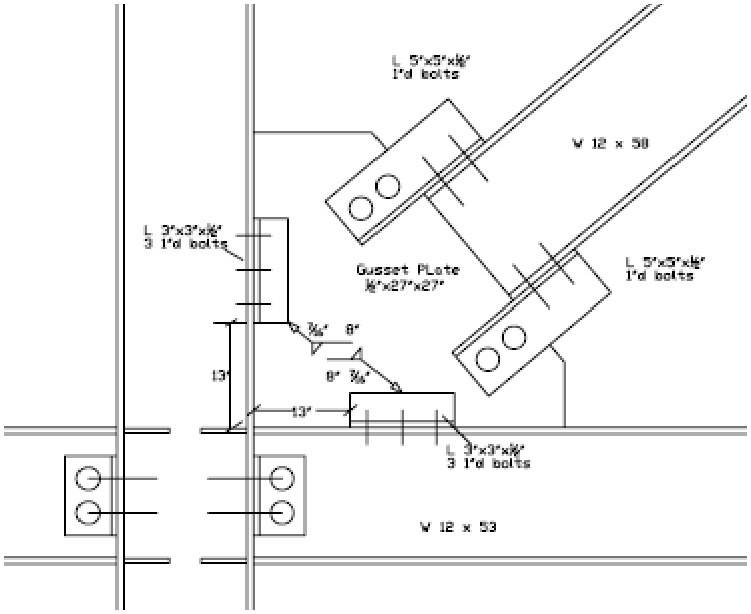


Figure 25

The Substructure:

Three levels of Lexington II are below grade levels. Although composite floor decking and beam sizes have been selected and designed in RAM, it is recommended that these floors remain concrete. Concrete is better able to withstand subterranean conditions, such as moisture.

Almost any concrete floor system will work. Floor sandwich depth is no longer an issue because the bottom three levels are below grade and can be dug deeper if needed. Based on a brief analysis of several concrete floor systems (Table 3), I decided to design a one way joist floor system. A one way joist floor was selected due to its ease of construction and its ability to work with the new column grid and larger bay sizes. One way joist girder systems can be designed using the CRSI handbook. Before using the handbook, it must be taken into consideration that the handbook is only valid when the larger of two adjacent spans does not exceed the smaller by more then 20%. With its new column layout, the Lexington no longer meets this criterion. Two other methods of analysis are possible, the first is moment distribution to find the maximum positive and negative moments experienced in each bay and design the joist floor using the determined moments. The other, less economical, method is to use the CRSI and design each bay as a single span. Once the one way joist system is designed, the girders that support it must be sized. It is common that the girders be the same depth as the joists to maintain a shallow floor sandwich. However, again, floor sandwich depth is less important for below grade floors.

Superimposed load: Dead (no self weight) = 30psf
Live (for below grade levels) = 100psf
Total Factored Load = 212psf

The design chosen for the one way joist floor was:

20" pans, 5" ribs, at 25" O.C.; this is with a 12" pan depth, 3" slab depth[3], 5" rib width, and 20" pan width. Two #6 bottom reinforcing bars are needed. This joist was chosen based on the maximum critical negative moment experienced by a single joist.

Calculated M = -22k < Table M = -22.1

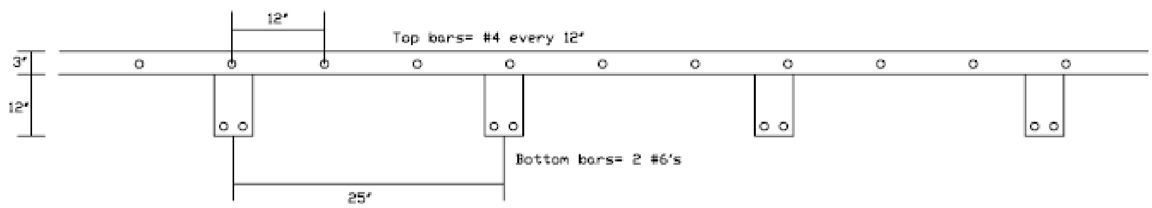


Figure 26
Floor Section

Girders were designed by ultimate moment to support the joist floor, and then checking shear. Designing for the most critical bay will give a beam size that will be conservative for girders with small tributary areas. By using one consistent girder size, formwork can be reused.

M= 506 ft-k V= 61.25 k

The girder depth is designed to be the same as the joists'. An assumption that the girders would be 24" wide was made. The final girder design is as follows[4]:

Top Steel = 7 #9's
Bottom Steel = 4 #8's
Shear:
#3 stirrups every 6.5" until 4.5 feet from the support
#3 stirrups every 4.5" until 3.2 feet from the support

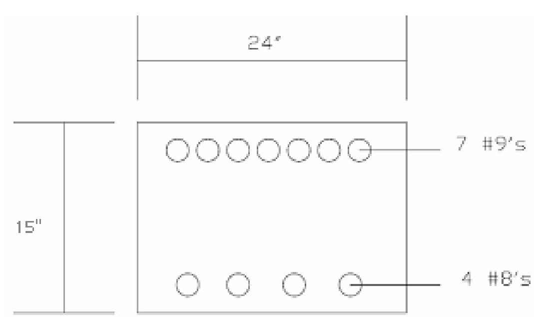


Figure 27
Girder Section with Reinforcement

[3] A 4.5" slab depth is required for 2 hour fire rating; this means self weight of the slab should include additional weight due to spray on fire proofing. This additional weight is added in assumed MEP superimposed dead load.
[4] For full girder design calculations see Appendix

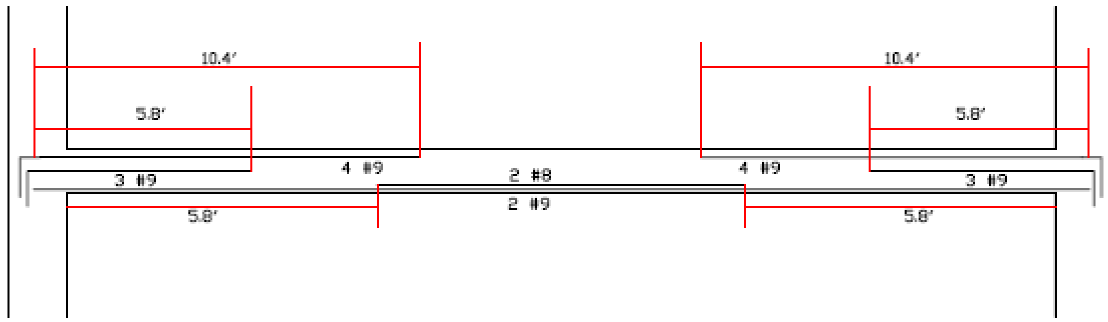


Figure 28
Bar cut offs in Girder

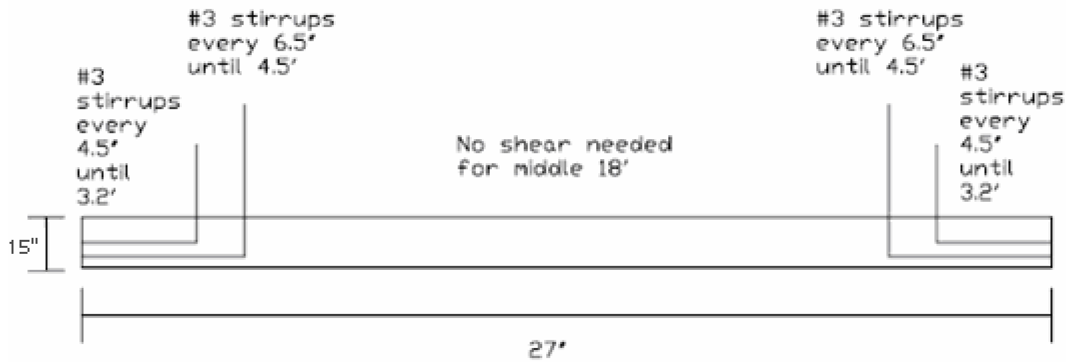


Figure 29
Girder with shear cut offs

The below grade columns must also be designed to carry the increased weight caused by the larger tributary areas of the new column grid. Because the floors on which concrete columns are located are below grade, there will be no wind loading (the controlling lateral load) on them. Also, since braced frames were designed to carry the entire lateral load of Lexington II there will not be lateral load transferred to the concrete columns from the above steel columns. The moments in each column were calculated using moment distribution from pattern loading on the beams which frame into each column. The calculated moments were small and almost negligible on column interaction diagrams, ρ= .05 for the worst case biaxial loading. However, each column must be designed to carry a minimum 1 inch eccentricity (this is approximately equivalent to the P-delta effect a column may experience). Column design was completed using a column strength interaction diagram and then checking for biaxial loading with the Load Contour Method. Full design calculations for the Concourse level are included in the Appendix.

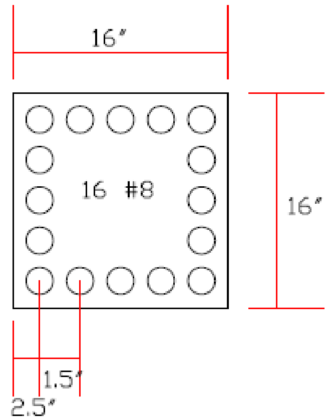
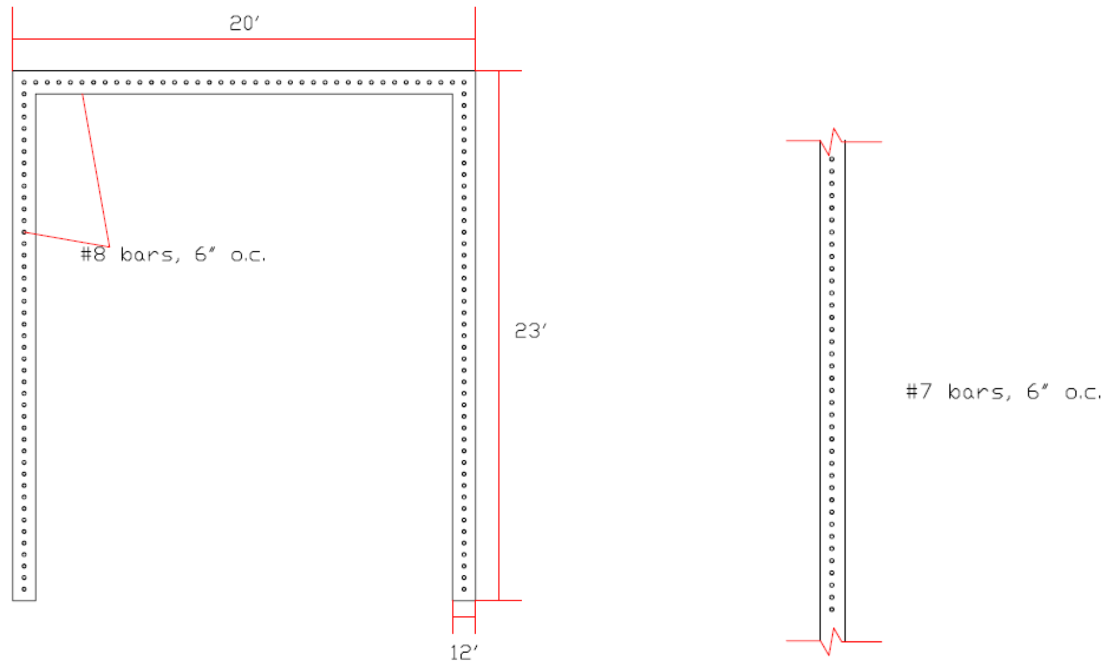


Figure 30
Column Section

Because girders frame into the columns, punching shear is not a concern and does not need to be considered. Additional strength must be added to the columns placed below the braced frames, as these will be carrying the lateral load on the building into the foundation. The two options to transfer the lateral load through the sub-grade levels are using shear walls or moment frames below the braced frame. For Lexington II I have decided to design the sub-grade levels with shear walls, similar to the original building design.

The shear walls were designed to meet the ACI building code. The wall design began by assuming a 12" thick wall. A 12" thick wall was assumed for reasons of practicality. Because the shear walls only run the length of three floors (approximately 30') the design of the shear walls was controlled by the shear resistance of the walls and not by flexure. For 12" thick shear walls, it was found that the shear capacity of the concrete was able to resist most of the shear and the steel only needed to resist a small portion of the shear load. The area of steel required for the shear wall design was .00923 square inches. Therefore, the steel design was governed by required actual instead of the code requirement of ρ=.0025. The final design of the shear walls were 12" thick shear walls with #6 bars every 6" both horizontally and #7 bars every 6" vertically.



Shear Wall Plan (in inches) Shear Wall Section

Figure 31

Connections from the steel super structure to the sub-structure must also be considered in this design. To connect the structures together steel base plates for the columns can be designed. These base plates will be sunk into the concrete floor slab at the ground level. Although this may increase floor thickness, it will keep the floor level so that the retail space on the ground floor will not have to avoid the area around columns.

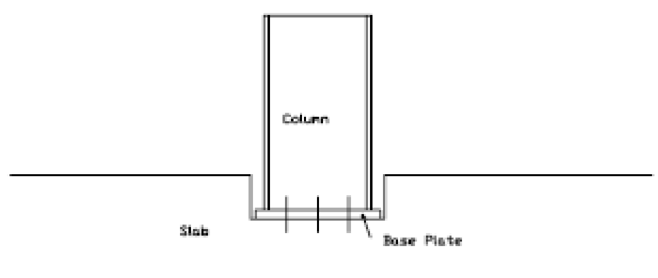


Figure 32

The size of the connection is dependent on the column size and vertical load on the column. A spreadsheet to calculate the base plate design is included in the (Appendix Table A-6). The average base plate size will be 20" x 18" x 3.5". The base plate size will be increased for the columns in the lateral braces and greatly increased for the columns in biaxially bending due to the braced frames.

Foundation:

The last item to be considered is the foundation. Due to time constraints, I have decided to use the existing foundation if it proves effective for the new design of The Lexington. In the original design of Lexington II, the foundation was a MAT foundation due to the columns' close spacing. It is possible that completely redesigning the foundation as spread footings or other shallow systems will result in a design with less material hence be less costly. Before the same foundation in the existing design can be considered it must be checked for punching shear. Punching shear of each column may have increased in load as spacing and tributary areas for each grew.

Overturning of the building must also be checked. In a simplified check, the moment caused by the lateral loading around the foundation (30' below grade) as well as the moment cause by the building weight was compared to the uplift needed on the opposite corner to create a resisting moment.

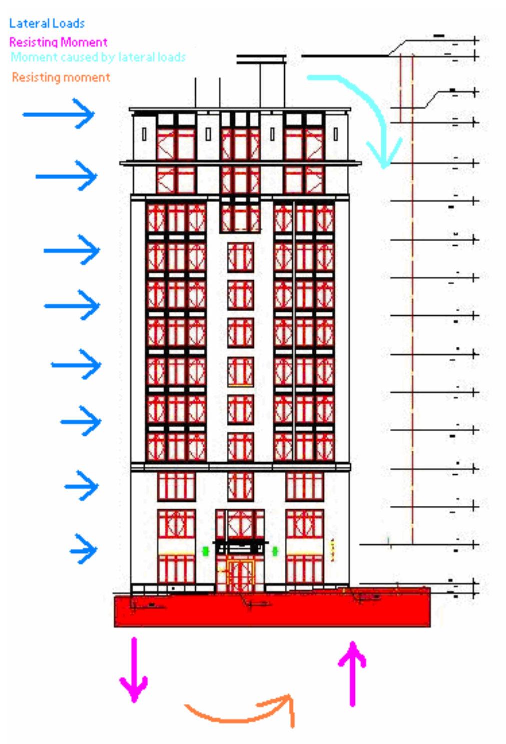


Figure 33
Forces effecting Overturning

For Lexington II, the uplift needed to resist the overturning moment must be less than ½ of the building weight.

$M_{n\text{-}s} = M_o + W(l/2) - x(l)$

$0 = 15424 + 4645*50 - x*100$

$X = 2476.74$

$M_{e\text{-}w} = M_o + W(l/2) - x(l)$

$0 = 19132 + 4645*20 - x*50$

$X = 2705.15$

Although the reaction needed at point x is less then ½ the building weight, this check works. The moment due to lateral loads was taken around the foundation (height + 30' below grade). However, the number used for building weight does not account for the additional weight of the sub-grade levels and of the foundation. Once these weighty floors have been included, the overturning check will pass.

Punching shear on the foundation was also checked. The actual punching shear on the foundation was much less then the shear capacity of the foundation. I believe the foundation was designed as a MAT because of the initially close column spacing, and that punching shear was always over designed which is why even with greater point loads created by columns, punching shear is still not a controlling design criterion.

Final Design Solution:

My final design of Lexington II is a composite structural system resting on a cast in place substructure. The superstructure will be composite floor decking connected to steel beams and columns through shear studs. Braced frames will resist the lateral load in both the N-S direction and the E-W direction. The design of the braced frames is controlled by the allowable stress and biaxial bending on each member, increasing the size of the members used in frames. The composite deck system will reduce the amount of concrete and form work needed to build the structure, and hopefully reduce the cost. The floor sandwiches are increased, but the system should still prove to be economical in any area without a height requirement.

The substructure of Lexington II was designed to be one way joist floors poured monolithically with girders framing into concrete columns. Using pans to construct the joist floors should reduce construction costs by eliminating time and labor involved with form work. Shear and lateral loads transferred from the superstructure will be carried to the foundation through shear walls.

Connections will play a large role in this structural system. Costs associated with the composite system include the extra material and labor used while installing shear studs. Bolts and welds to connect steel members will also greatly affect the cost of this building. Additional connections need to be specially designed to transfer loads from the steel superstructure to the concrete substructure.

While this new design should not greatly affect any of Lexington II's other building systems such as mechanical and electrical systems, it is important to note that fire proofing not required with the original design is now necessary.

Breadth: Construction Management

Construction management is an important part of the engineering of a building. While construction management has many items to take into consideration, I have decided to concentrate on three aspects; site layout, cost, and scheduling.

Site Layout:

Due to its location in downtown Washington D.C., site layout is very important for Lexington II. The site must accommodate site offices, trailers, cranes, and lay down areas as well as circulation paths around the site and maintain a safe work area. As in the original design of Lexington II, concrete buildings allow for a clearer site by eliminating the need for a crane and lay down area.

For my steel design of Lexington II, a crane will be needed for erection. The most logical type of crane for placement of steel would be a moving crawler crane. This crane can be located to the east of the building and move north to south depending on the stage of steel construction. To reach all areas of the building, the crane must reach a radius of 75'.

A unique feature of the Market Square North complex is that at the time of construction there was an open area in the center along Eighth Street. This open area allows space for offices, storage, lay down areas, parking, sanitary facilities and other necessary accompaniments.

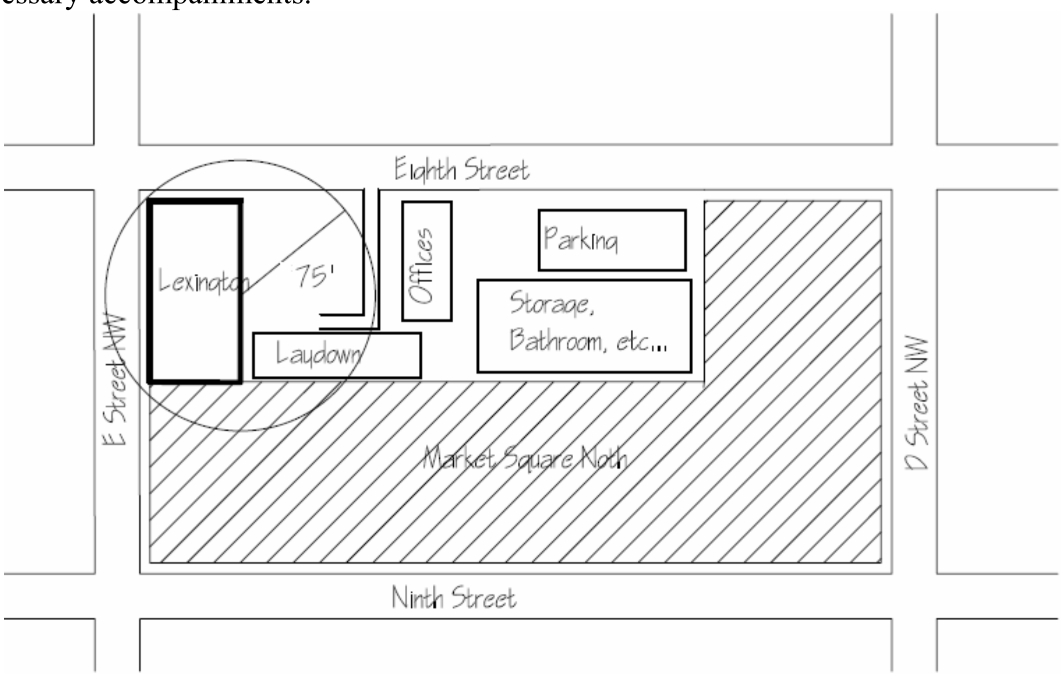


Figure 33
Site Layout

Cost:

Cost is always an important part of any building project. Cost is usually the determining factor to decide if a building will be a profitable venture to invest in. Steel and concrete buildings often differ greatly in cost. Besides accounting for material costs, concrete buildings require formwork, rebar, finishes, and accessories such as ties and chairs. Steel buildings also have many additional costs, usually caused by expensive labor intensive connections.

For a cost analysis on Lexington II, the R.S. Means Construction Estimating Guide was used. Each stage of the building was analyzed separately and added to find the total cost of the building materials, labor, and equipment. To simplify the analysis, many small items such as concrete accessories were not included. The façade and finishes of the building were also not included because these items will remain unchanged between a concrete and steel design.

Steel Design Cost:[5]

Excavation:[6]	$23,600
Foundation:	$159,000
Sub Grade Levels:[7]	$741,200
Steel Levels:[8]	$467,200
Braced Frames:	$156,400
Connections:	$51,300
Total:	**$1,455,600**

Concrete Design Cost:[5]

Excavation:[6]	$23,600
Foundation:	$159,000
Sub Grade Floors:	$671,000
Super Structure Floors:	$154,000
Columns:	$395,000
Shear Walls:	$123,900
Total:	**$1,526,000**

These totals seem reasonable. Steel is generally considered a cheaper material; however this concept is based on the economy of larger construction projects. For steel building projects, the cost of steel connections may be expensive. For a building the size of Lexington II the conclusion that there is no large price difference between a steel and concrete building is appropriate.

[5] All cast in place concrete costs from RS Means includes formwork, reinforcing steel, and finishes. Concrete costs have also been adjusted for 10% waste. Steel costs are as written in RS Mean and do not include any adjustment factors since over 99 tons of steel are in the building.

[6] Includes equipment, sheathing, and hauling costs

[7] Includes joist floor, grade walls, columns, and shear walls

[8] Includes composite decking, slab, shear studs, beams, and columns

Scheduling:

Scheduling is another important issue dealt with by the construction manager. The time it takes to erect a building can greatly affect the cost. An inefficient schedule can cause major setbacks in the construction of a building, and employing workers and equipment before they are needed is a great waste of capital.

To schedule Lexington II's construction, each stage of construction was looked at individually to ensure its completion before the next phase of construction began. Multiple crews were employed when needed and to limit certain tasks, such as pouring concrete, to a single day. The schedule is as follows:

Excavation:

Level 1- Backhoe (B-12A) ½ day
-Wood Sheathing (3 B-31) ½ day
-Hauling (4 trucks) 1 day
-Wood Sheathing (3 B-31) 3 days
Level 2- Backhoe (B-12A) ½ day
-Wood Sheathing (3 B-31) ½ day
-Hauling (4 trucks) 1 day
-Wood Sheathing (3 B-31) 3 days
Level 3- Backhoe (B-12A) ½ day
-Wood Sheathing (3 B-31) ½ day
-Hauling (4 trucks) 1 day
-Wood Sheathing (3 B-31) 3 days

Item Time: 12 days **Total Work Weeks: 2.4**

Foundation:

-Cast in Place, MAT (12 C-14C) 1 day
-Concrete Curing 4 days

Item Time: 5 days[9] **Total Work Weeks: 3**

[9] If building construction begins on a Monday, the final two curing days can be Saturday and Sunday, therefore these 2 curing days are no included in the work week schedule.

Sub-Grade Levels:

Level 3-Cast in Place, Columns (1 C-14A) 1.1 day
-Cast in Place, Grade Wall (1 C-14D) 2 days
-Cast in Place, Shear Wall (1 C-14D) 1/2 day
-Cast in Place, One Way Joist (2 C-14B) 5 days
-Concrete Curing 4 days
Level 2-Cast in Place, Columns (1 C-14A) 1.1 day
-Cast in Place, Grade Wall (1 C-14D) 2 days
-Cast in Place, Shear Wall (1 C-14D) 1/2 day
-Cast in Place, One Way Joist (2 C-14B) 5 days
-Concrete Curing 4 days
Level 1-Cast in Place, Columns (1 C-14A) 1.1 day
-Cast in Place, Grade Wall (1 C-14D) 2 days
-Cast in Place, Shear Wall (1 C-14D) 1/2 day
-Cast in Place, One Way Joist (2 C-14B) 5 days
-Concrete Curing 4 days

Item Time: 11 days/ floor
33 days **Total Work Weeks: 9.2**

Super Structure:[10]

-Level 1: Structural Steel Columns, (E-2 or E-5)1/2 day
-Level 1: Structural Steel Beams, (1 E-2) 1 day
-Level 1: Composite Deck, (1 E-4) 2 days
-Level 2: Structural Steel Columns, (E-2 or E-5)1/2 day
-Level 2: Structural Steel Beams, (1 E-2) 1 day
-Level 2: Composite Deck, (1 E-4) 2 days
-Level 3: Structural Steel Columns, (E-2 or E-5)1/2 day
-Level 3: Structural Steel Beams, (1 E-2) 1 day
-Level 3: Composite Deck, (1 E-4) 2 days
-Level 4: Structural Steel Columns, (E-2 or E-5)1/2 day
-Level 4: Structural Steel Beams, (1 E-2) 1 day
-Level 1: Slab,(1 C-8) 1 day
-Level 4: Composite Deck, (1 E-4) 2 days
-Level 1: Slab, (1 C-8) 1 day
-Level 1: Concrete Curing, 1 day
-Level 5: Structural Steel Columns, (E-2 or E-5)1/2 day
-Level 5: Structural Steel Beams, (1 E-2) 1 day
-Level 2: Slab, (1 C-8) 1 day
-Level 1: Concrete Curing, 1 day
-Level 5: Composite Deck, (1 E-4) 2 days
-Level 2: Slab, (1 C-8) 1 day

[10] Each level is built in the sequence of 1 day for columns and beams and 2 days for deck, and then the next level is started. The slabs were poured once the beam, column, and deck construction was a full 3 stories ahead.

  - Level 1: Concrete Curing, 3 day
    - Level 6: Structural Steel Columns, (E-2 or E-5)1/2 day
    - Level 6: Structural Steel Beams, (1 E-2) 1 day
    - Level 3: Slab,(1 C-8) 1 day
    - Level 2: Concrete Curing, 1 day
      - Level 6: Composite Deck, (1 E-4) 2 days
      - Level 3: Slab, (1 C-8) 1 day
- Level 7: Structural Steel Columns, (E-2 or E-5)1/2 day
- Level 7: Structural Steel Beams, (1 E-2) 1 day
- Level 4: Slab, (1 C-8) 1 day
- Level 3: Concrete Curing, 1 day
  - Level 7: Composite Deck, (1 E-4) 2 days
  - Level 4: Slab, (1 C-8) 1 day
  - Level 4: Concrete Curing, 1 day
    - Level 8: Structural Steel Columns, (E-2 or E-5)1/2 day
    - Level 8: Structural Steel Beams, (1 E-2) 1 day
    - Level 5: Slab,(1 C-8) 1 day
    - Level 4: Concrete Curing, 1 day
      - Level 8: Composite Deck, (1 E-4) 2 days
      - Level 5: Slab, (1 C-8) 1 day
      - Level 4: Concrete Curing, 2 days
      - Level 5: Concrete Curing, 3 days
- Level 9: Structural Steel Columns, (E-2 or E-5)1/2 day
- Level 9: Structural Steel Beams, (1 E-2) 1 day
- Level 6: Slab, (1 C-8) 1 day
- Level 5: Concrete Curing, 1 day
  - Level 9: Composite Deck, (1 E-4) 2 days
  - Level 6: Slab, (1 C-8) 1 day
  - Level 6: Concrete Curing, 1 day
    - Level 10: Structural Steel Columns, (E-2 or E-5)1/2 day
    - Level 10: Structural Steel Beams, (1 E-2) 1 day
    - Level 7: Slab,(1 C-8) 1 day
    - Level 6: Concrete Curing, 1 day
      - Level 10: Composite Deck, (1 E-4) 2 days
      - Level 7: Slab, (1 C-8) 1 day
      - Level 6: Concrete Curing, 2 days
      - Level 7: Concrete Curing, 3 days
- Level 11: Structural Steel Columns, (E-2 or E-5)1/2 day
- Level 11: Structural Steel Beams, (1 E-2) 1 day
- Level 8: Slab, (1 C-8) 1 day
- Level 7: Concrete Curing, 1 day
  - Level 11: Composite Deck, (1 E-4) 2 days
  - Level 8: Slab, (1 C-8) 1 day
  - Level 8: Concrete Curing, 1 day
    - Level 12: Structural Steel Columns, (E-2 or E-5)1/2 day
    - Level 12: Structural Steel Beams, (1 E-2) 1 day

-Level 9: Slab,(1 C-8) 1 day
-Level 8: Concrete Curing, 3 day
-Level 12: Composite Deck, (1 E-4) 2 days
-Level 9: Slab, (1 C-8) 1 day
-Level 9: Concrete Curing, (1 C-8) 1 day
-Level 10: Slab, (1 C-8) 2 days
-Level 9: Concrete Curing, 2 days
-Level 11: Slab, (1 C-8) 2 days
-Level 10: Concrete Curing, 2 days
-Level 9: Concrete Curing, 1 day
-Level 12: Slab, (1 C-8) 2 days
-Level 11: Concrete Curing, 2 days
-Level 10: Concrete Curing, 2 days
-Level 12: Concrete Curing, 4 days
-Level 11: Concrete Curing, 2 days

Item Time: 45 days **Total Work Weeks: 18**

The complete structural system of Lexington II will take 18 five day work weeks to complete. This time does not include interior construction, finishes, or façade.

Breadth: Mechanical

There are many mechanical systems which are including in building design and construction. Important systems designed by the mechanical engineering include HVAC, plumbing, fire suppression, transportation, and acoustics. For my breadth work I will briefly discuss how some of these systems can be integrated into a composite floor and steel frame structural design.

HVAC:

Fresh air requirements in the original design were provided by working windows in all apartment units. The new column layout will require some window placement be moved. Although the bay spacing is irregular, it is symmetrical about the geometric center of the residential levels forming a 13', 16', 20', 16', 13' pattern. This allows for the windows to be moved while maintaining a symmetric grid. No move is significant enough as too eliminate large working windows from each living space.

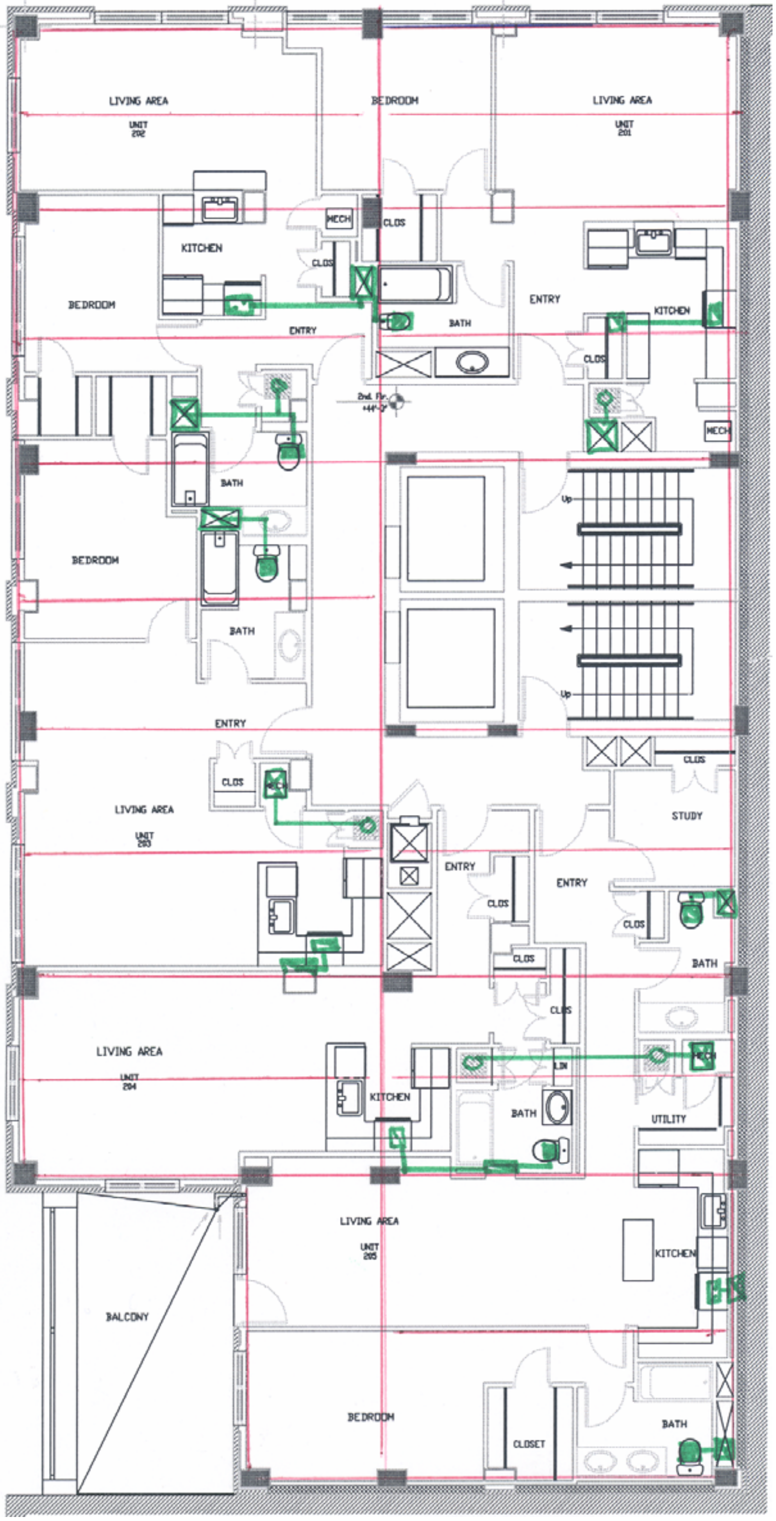
Existing Window Grid Adjusted Window Grid

Figure 35

To minimum the floor depth of Lexington II, the HVAC system was run though soffits along the top of interior partitions. Since the steel alternative design was developed around the existing architecture, no partitions were moved and the use of soffits can be maintained in the exact same manor as previously designed.

However, one advantage of a deeper floor sandwich is the ability to conceal the mechanical systems within the ceiling. Concealing the mechanical systems is usually more aesthetically pleasing to the tenant. The ductwork can be placed anywhere within the floor sandwich as long as it does not intersect a beam. If any ductwork intersects a beam, a hole cut into the beam would be necessary and the beam's structural integrity would be comprised.

I have mapped out a brief example of an alternative duct work design to verify that there are possible routes for which the duct work can be concealed within the ceiling (Figures 36 and 37). This design provides exhaust to each toilet room, utility (washer/dryer unit), and oven range. Supply ducts are routed into every room and were designed to maintain the same supply quantities and number of diffusers to each space as in the original design. The numerous spaces from which concrete columns in the original design were removed provided additional space for risers in the new duct layout.



Red lines represent beams which cannot be crossed.

Green represents duct work

Figure 36
Possible Exhaust Plenum Layout

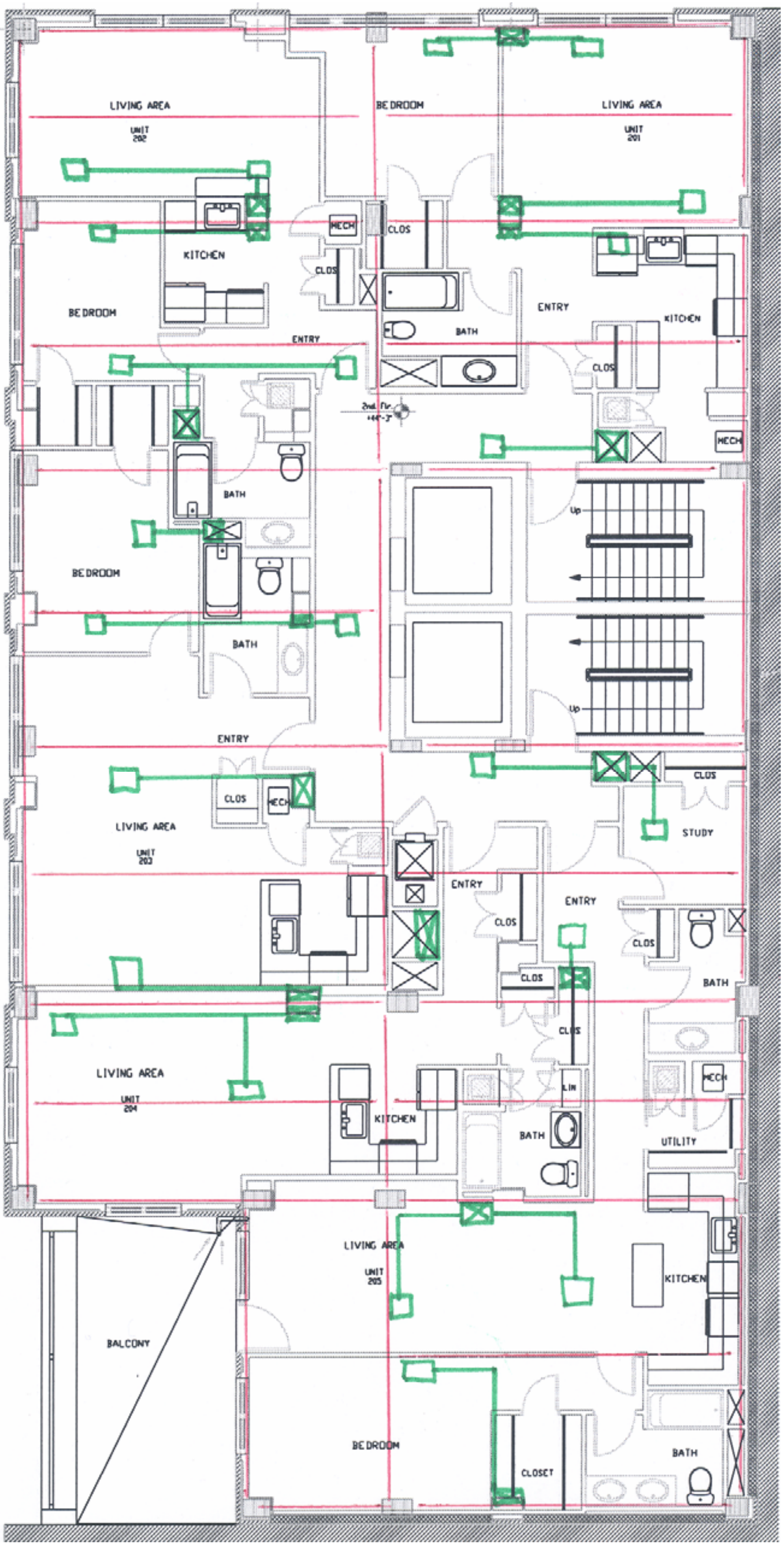


Figure 37
Possible Supply Diffuser Layout

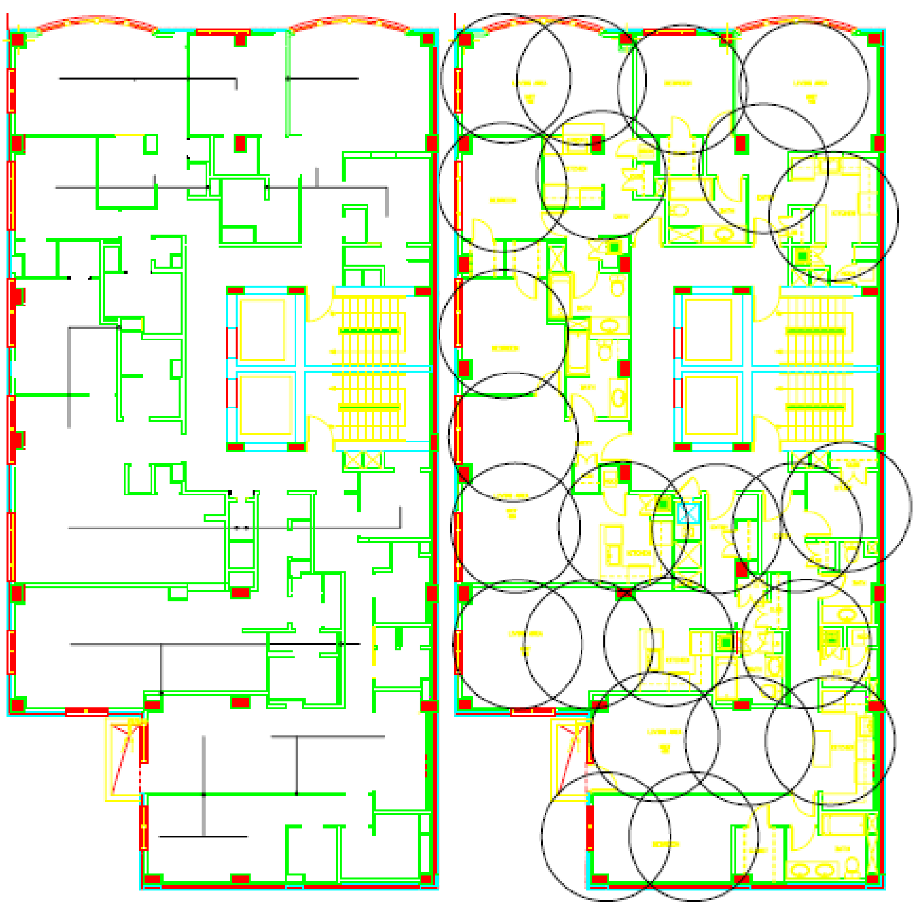
Plumbing:

All plumbing is concealed in existing walls. The steel design of Lexington II is sensitive to the existing architecture and wall partitions. Therefore, no changes to the plumbing layout are necessary.

Fire Suppression[11]:

Currently Lexington II is 100% fully sprinklered. Although I do not have a copy of the sprinkler layout, I believe the current system will still work with the new structural design. Like the other MEP systems, in the original design of Lexington II the sprinkler system would have been run through soffits. All soffits are still possible to construct since none of the interior architecture has been altered. However, like the HVAC system it is safe to assume that a new layout may be completed upon investigation and that standpipes may be run in the areas previously occupied by concrete columns.

Lexington II would be classified as a Light Hazard Occupancy. Using upright or pendant sprinklers, this means that each sprinkler has a protection area of 225 square feet and the maximum spacing for sprinklers is 15'. Sprinklers are normally not required for bathrooms 55 square feet or less and closest with the least dimension 3' or less.



Piping Layout (with stand pipes) Sprinkler coverage area

Figure 38

[11] All fire protection requirements are as listed in *Mechanical and Electrical Equipment for Buildings* 9th Edition by Benjamin Stein, final design should be checked and complete with the Washington DC codes.

Figure 38 shows a possible sprinkler layout, all stand pipes have been run through walls and are concealed. There is no piping which intersects a ceiling beam.

Additionally, fire proofing must be added to all steel components. The most commonly used fireproofing is cementitious spray on fireproofing. This popcorn like fire retardant material must be applied to the underside of the steel decking as well as all beams and columns. Other fireproofing may include using fire retardant materials as finishes such as suspended ceilings and wall boards.

Means of egress is also an important issue with fire protection. In residential sprinklered buildings 35 feet is the common path limit for means of egress from a suite exit. As seen in figure 38, all apartment units open to the same hall and have very short egress paths. Fire resistance construction should be applied to the stairwells to create enclosed means of egress paths.

Additional precautions should also be taken. Smoke and fire detectors will be placed through out the building as prescribed in local code requirements. Smoke management should also be considered. Some ideas for smoke management may be automatic controls of the HVAC system once an alarm has been activated, or opening the top of the elevator shafts to create a natural chimney for the smoke to escape from.

Transportation:

No changes are necessary to the vertical transportation elements in Lexington II. The elevators and stairwells are located as before. Although braced framing now surrounds the stairwells, using the inverted chevron braced frame, the door to the stairwell is uninterrupted by the framing members.

Figure 39

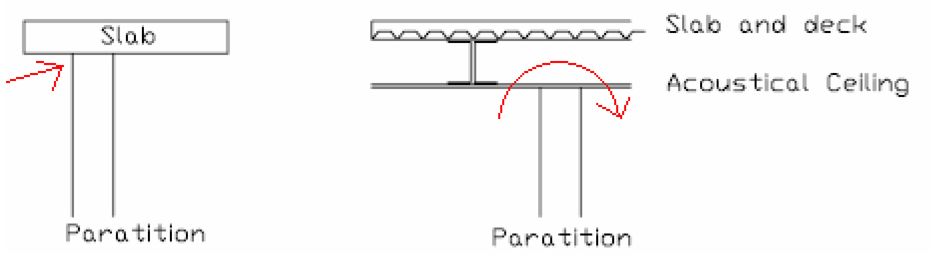
Acoustics:

Acoustics may be the mechanical system which differs the most between a steel and concrete building. Many items which affect the acoustic properties of each room will remain unchanged from one structural system to the next. Items in each room that are not altered by the structural system include interior partition materials and floor finishing materials. The greatest change to the acoustics will be reverberation and absorption associated with each room’s ceiling.

Concrete flat plate slab is a very hard and dense material. In the original design of Lexington II there was exposed concrete slab with sprayed acoustical sealant on it. Sprayed cellulose fibers can provide a noise reduction coefficient (NRC) of .75. The NRC is a single number rating of the sound absorption of a material averaged over the entire range of auditable sound frequencies.

For a composite steel decking system, it is possible to leave the decking and beams exposed as part of the ceiling system. In this case, there is also fireproofing exposed. Exposed sprayed fireproofing can provide a noise reduction coefficient (NRC) ranging from 0 to .75 depending on the product chosen. While this would cause little change from the concrete structural system, it is unlikely that a residential building would choose to leave such a system exposed. To be aesthetically pleasing, typically a suspended ceiling would be hung. This suspended ceiling is even more critical to hide the other MEP systems which have been moved from soffits to the floor sandwiches. Many suspended ceilings are designed to be acoustically sensitive and almost any required NRC can be specified. It is common for the NRC of acoustical tile to range from .5 to .95.

Noise infiltration can also differ between structural systems. Sound leaks are possible anywhere there is an interception of building partitions or materials. Although no specifics are known of the assembles existing between the concrete slab and wall partitions, it can be assumed that the partitions run the entire height of the room and connect to the concrete ceiling. For many reasons; aesthetical, thermal, etc, the owner and engineers will want to ensure that there is a solid connection at the ceiling and floor. When steel beams and a suspended ceiling are used, there is a much greater chance that the partition will not extend as far into the floor sandwich as needed to control noise leaks. Special attention should be paid to the design and construction of this detail.



Concrete Construction Steel Construction

Figure 40

Solutions to prevent noise infiltration are to continue the partition all the way to the ceiling or to use continuous gypsum in additional to the acoustic ceiling, Figure 41.

Figure 41

Final Recommendations:

When designing a building for maximum occupancy in areas with strict zoning requirements often factors other than economy dictate the final design. In the Lexington II building project, the design was required to meet local height requirements and certain structural systems became unfeasible while trying to achieve the maximum number of usable floor levels. To create the smallest possible floor sandwich flat plate slab was used with close column spacing.

By redesigning the building as a composite system the height requirement was no longer met, however other advantages presented themselves. Using a composite system was only economical once the bay sizes had been increased. Although the system I designed works with the existing architecture, larger bay sizes would also provide more architectural freedom to redesign the building interiors if desired. Fewer columns spread further apart will also alleviate congestion that can occur on the sub-grade parking levels. Using a composite system affected the weight of the building lowering the seismic load. A composite structure also has its advantages when integrating other systems. MEP systems are now able to fit into the floor sandwich with no major changes to the components used.

For reasons of practicality, the final design of the substructure was one way joist floors. Keeping the substructure concrete will protect the building from subterranean conditions. Using two types of structural systems results in specialized and costly connections, however when many other advantages are present connections should not be considered the controlling factor in deciding if the design is feasible.

Economy, however, is often the most critical criterion used when evaluating building systems. Cost analysis using *R.S. Means* showed that there is very little advantage to either system over the other. The cost of the concrete system begins to compile when an additional 10% for waste is accounted for. The biggest advantage of two way flat plate is its ability to maintain an acceptable building height. Steel which is often considered more economical did not prove to be greatly so. The economy of a steel system is dependent on the scale of the building project outweighing many other costs which accompany steel. When dealing with a building the size of Lexington II, the full advantage of economy through scale was not able to be reached.

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Jyutika Baheti of Studios Architecture

Appendix

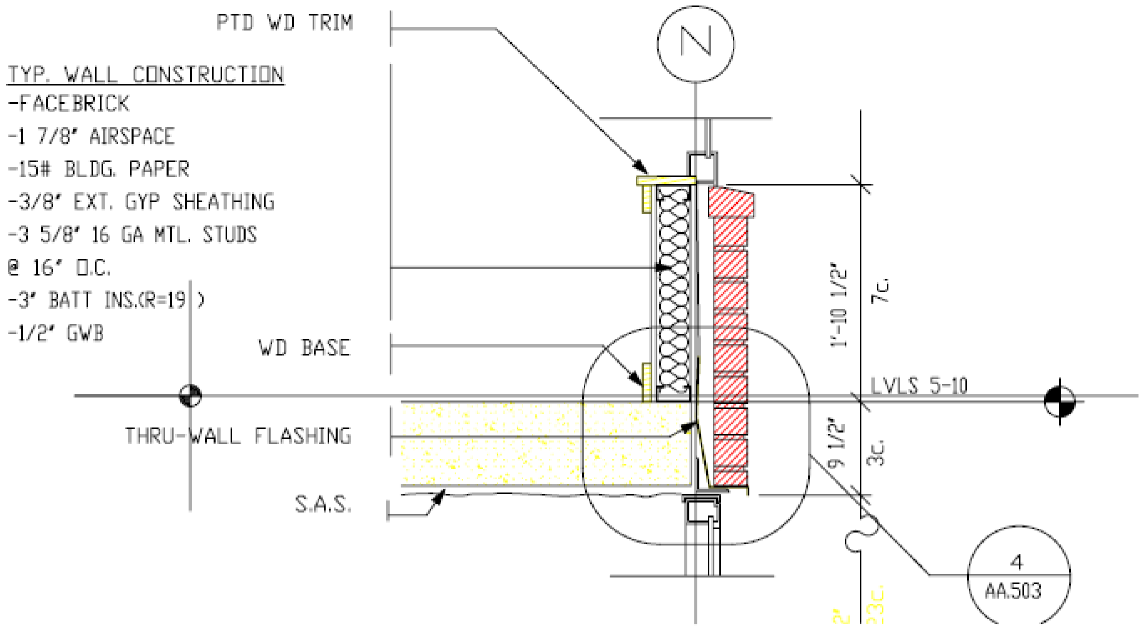


Figure A-1
Typical Wall Section

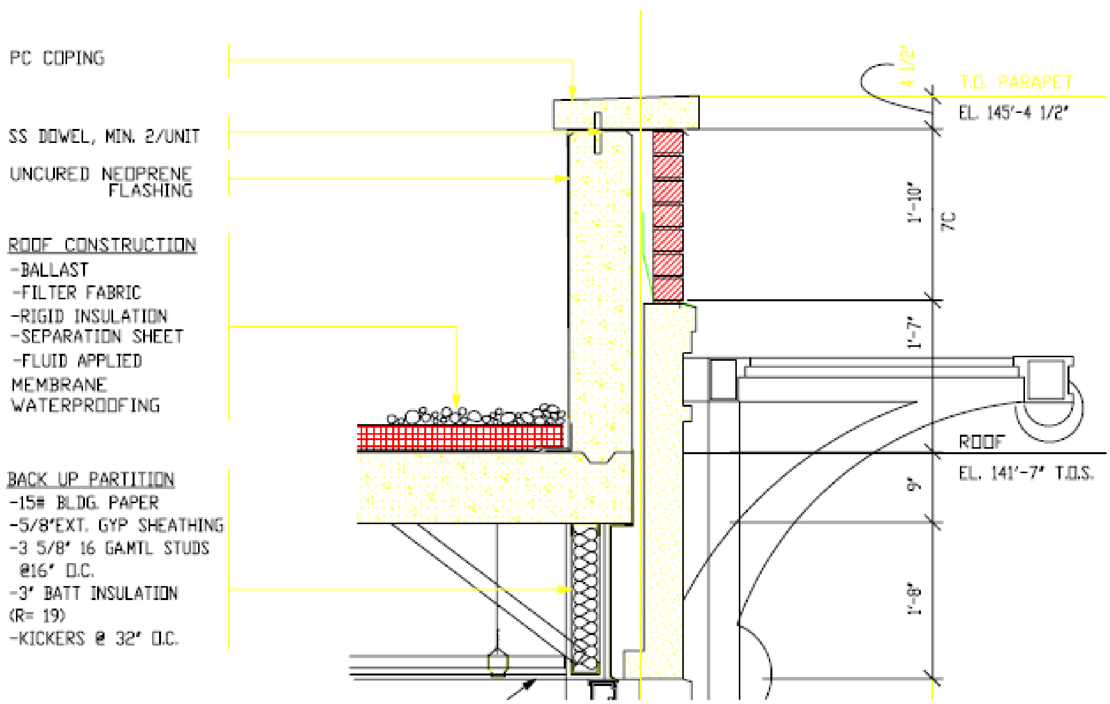


Figure A-2
Typical Roof Section

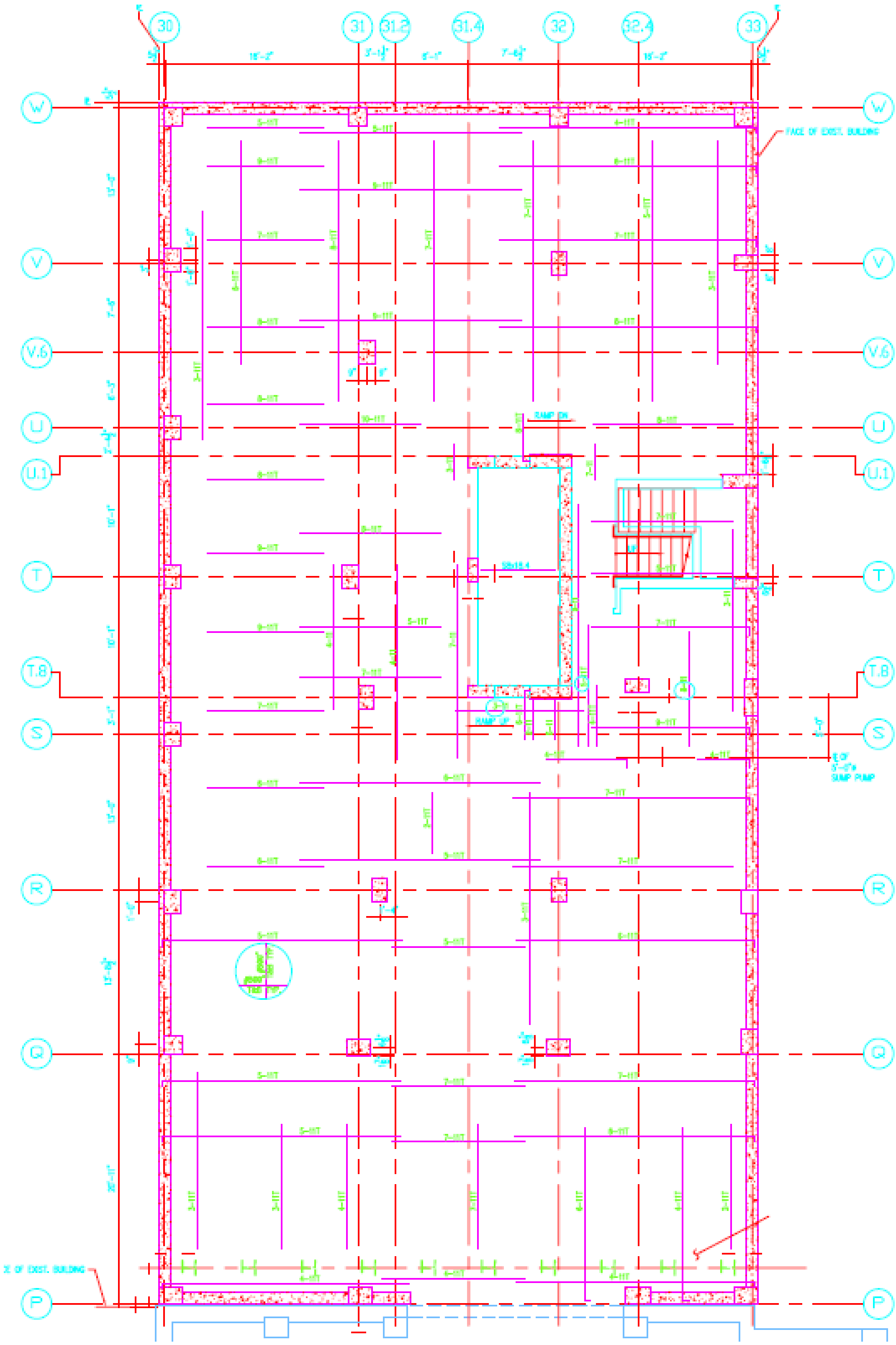


Figure A-3
Foundation Plan

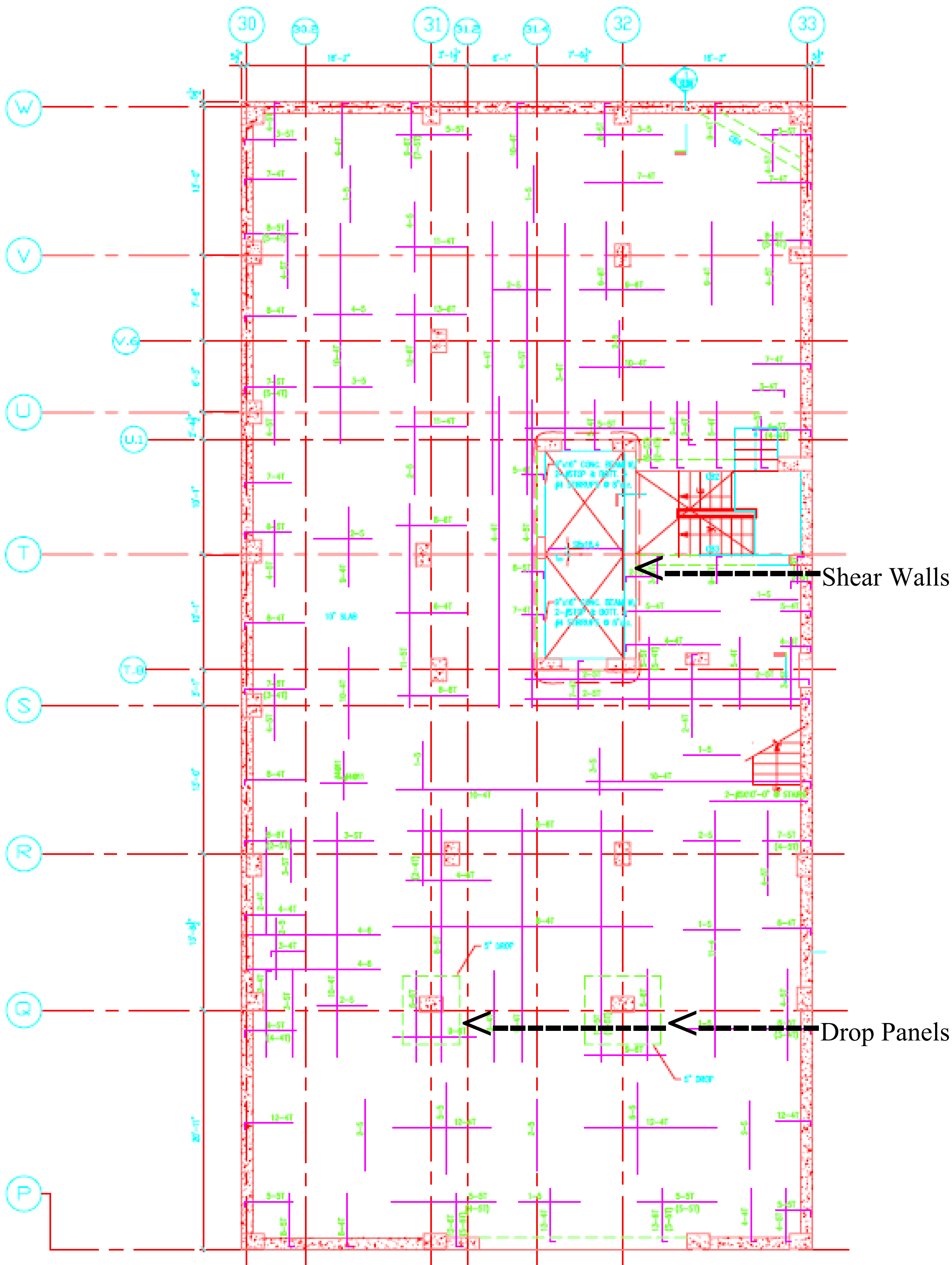


Figure A-4
Concourse Framing Plan
Purple= top rebar (with bar size in green)

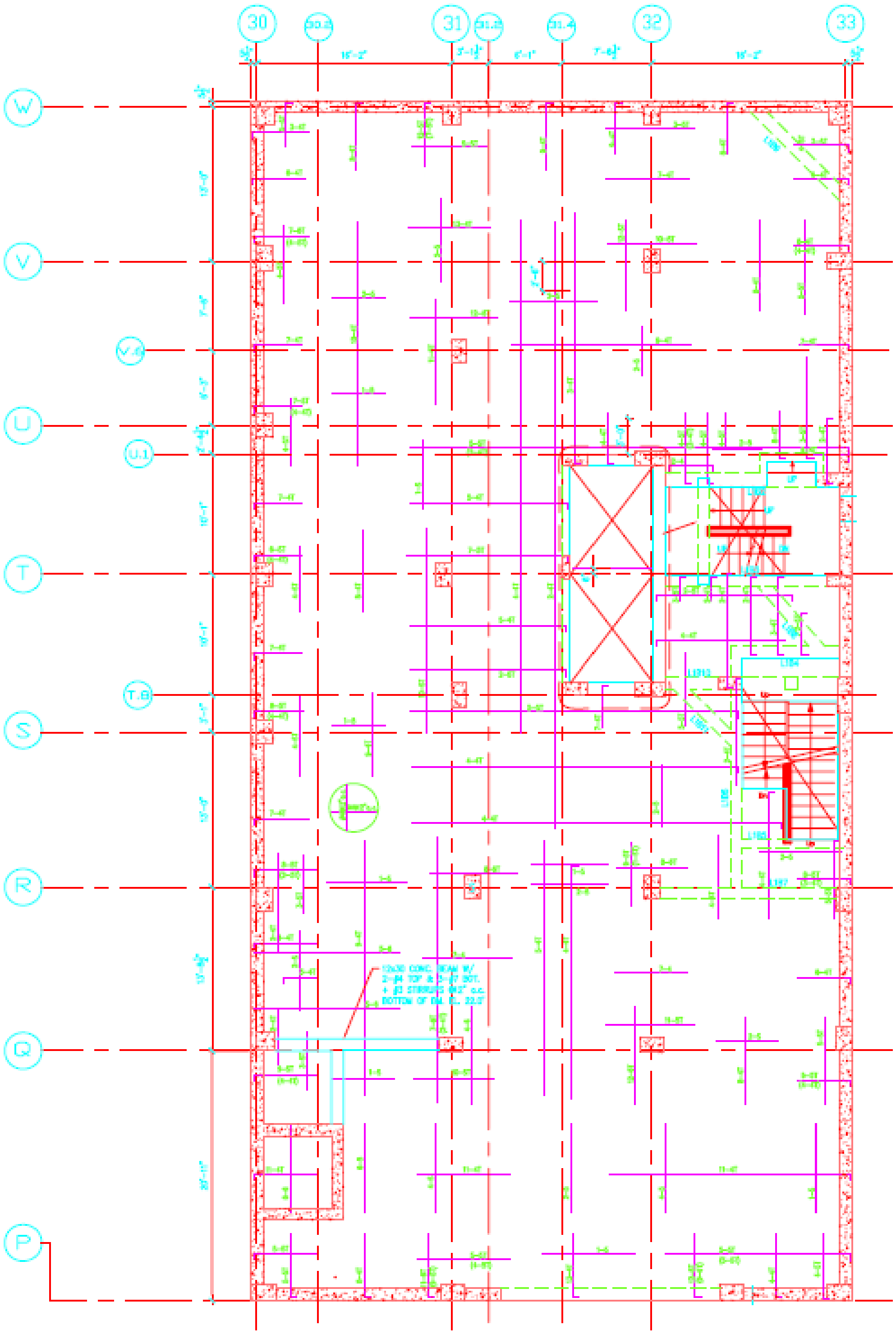


Figure A-5
L-1 Framing Plan

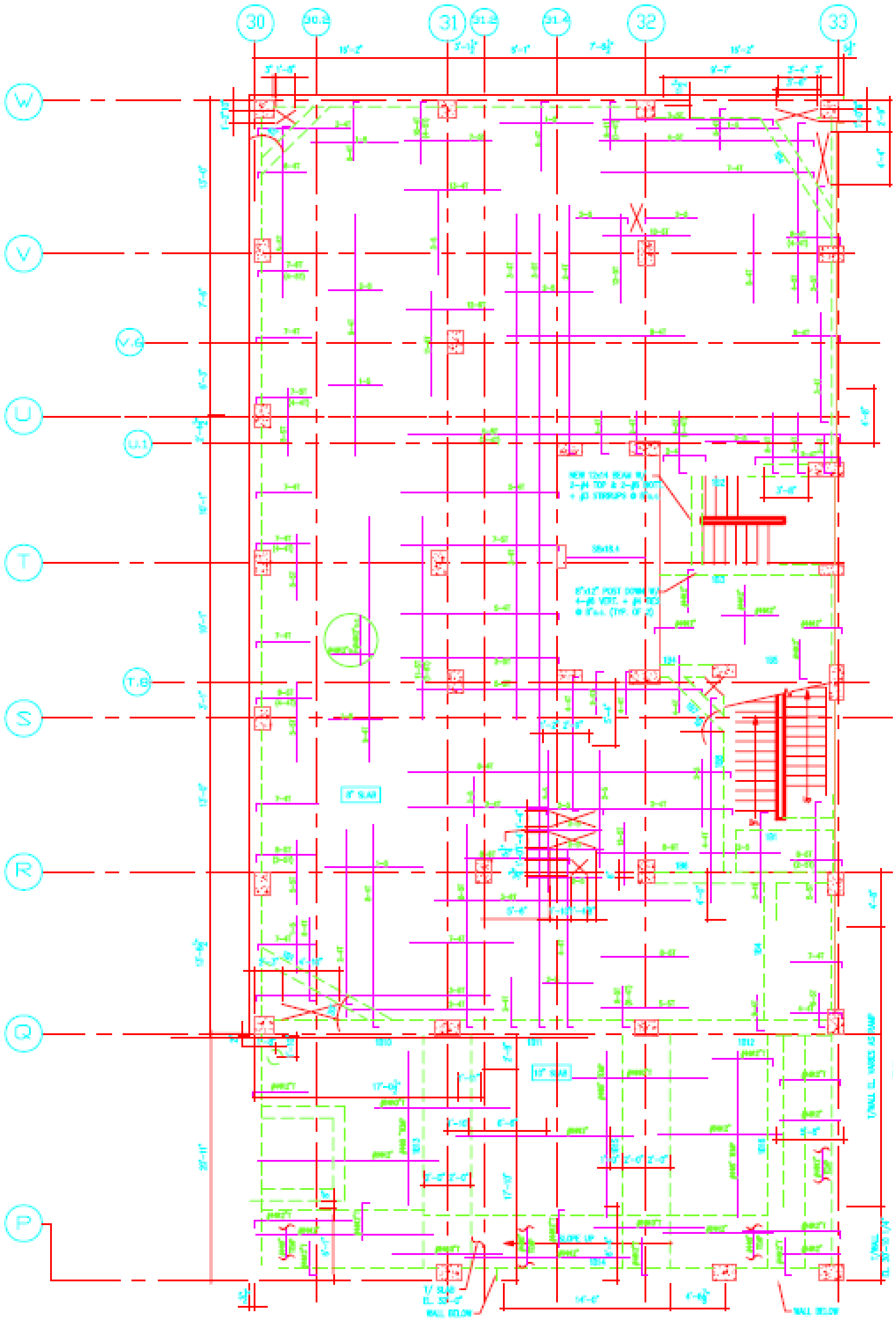


Figure A-6
Ground Level Framing Plan

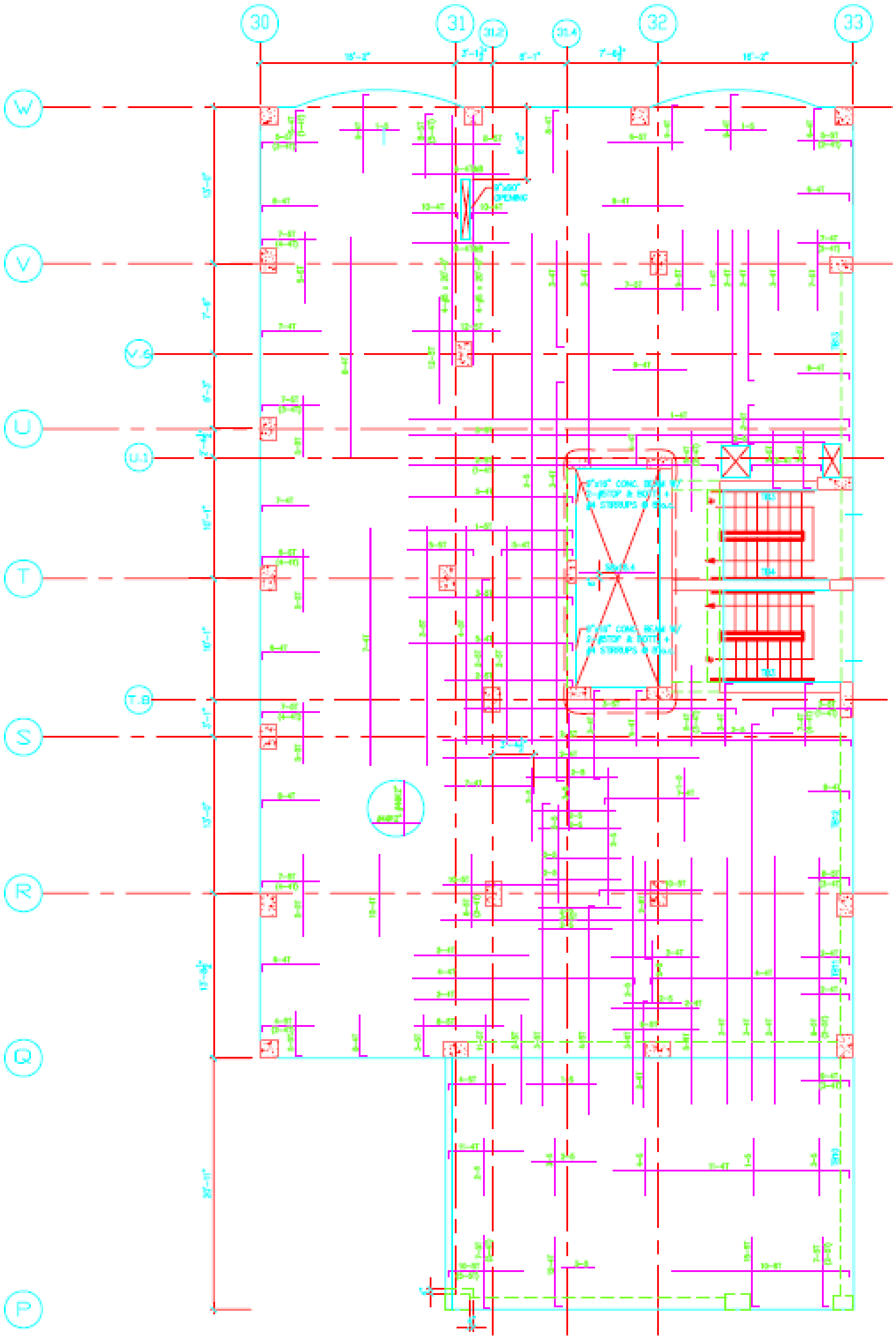


Figure A-7
Framing Plan Levels 2 to 7

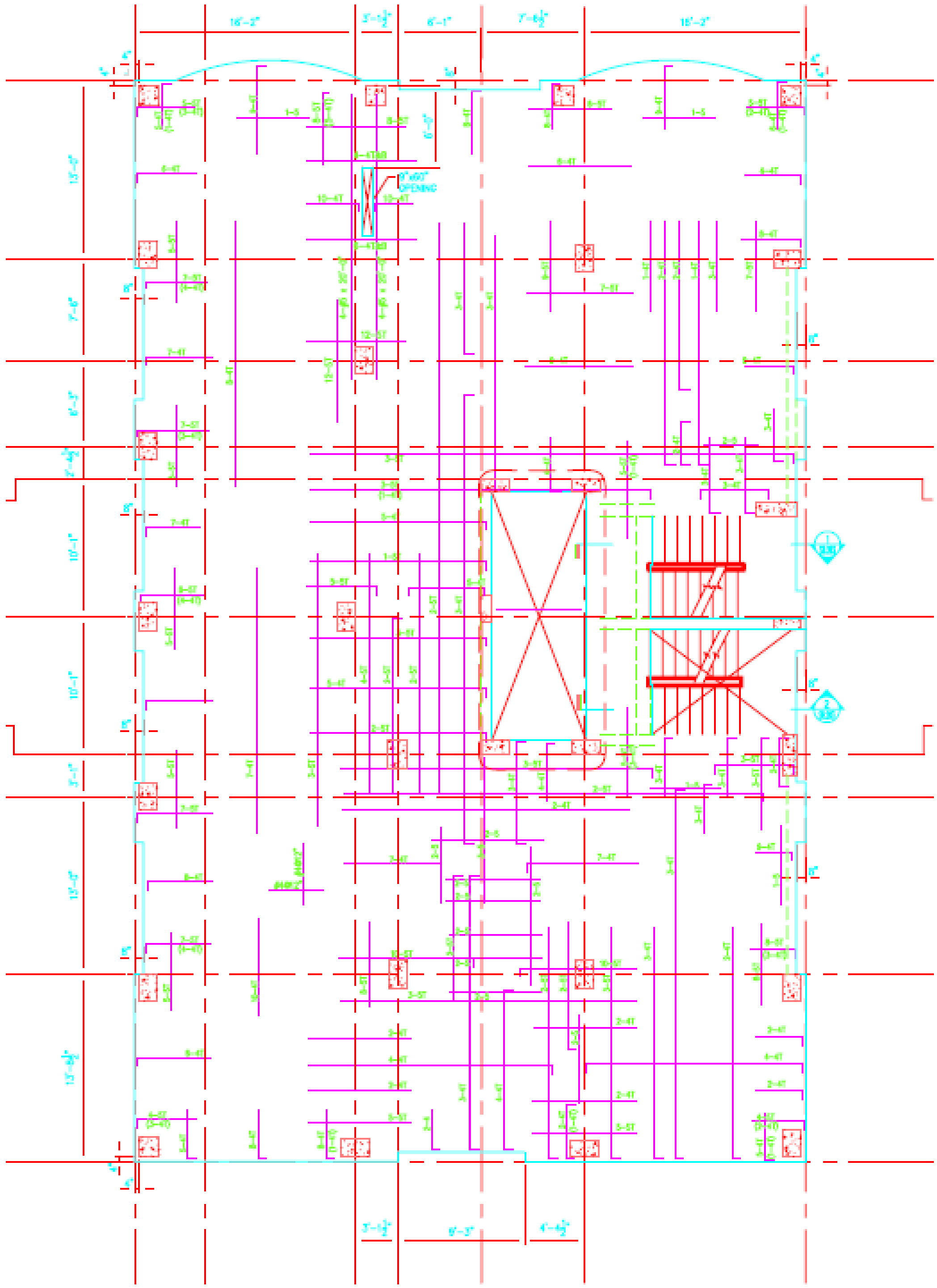


Figure A-8
Framing Plan Levels 8 to 12

Load Calculations:

Self Weight:

8" Slab: 150psf * 8"thick slab / 12" per foot = 100psf
10" Slab: 150psf*10" thick slab/ 12" per foot = 125psf

Roof Live Load:

A_t= 16.2' * 13' (for a typical bay)
= 208 ft^2
R_1= 1.2 - .001* A_t
= 1.2 - .001*208
= .992
F = 0 for a flat roof
R_2 =1

L_r=20*(.992) (.1)
= 20psf

Snow Load:

C_e = .9 (Table 7.2, B-urban, partially exposed)
C_t = 1 (Table 7.3)
I = 1 (Table 7.4, Category II)
P_g = 25psf (Fig. 7-1)

P_f= .7*(.9)(1)(1)(25) = 15.75psf

Wind Load:

WindCals

Basic Wind Speed (V)	90	Fig 6-1	L	50	build. Geo	qz factor	17.6256
Wind Directionality (kd)	0.85	Table 6-4	B	100	build. Geo	qh	18.330624
Importance Factor (I)	1	Table 6-4	H	116.33	build. Geo		
Topical Factor (kzt)	1		l	320	Table 6-2		
Cp windward	0.8	Fig 6-6					
Cp leeward	-0.5	Fig 6-7					
Cp leeward	-0.3	Fig 6-8					
Gcpi (internal pressure)	0.18	Fig 6-5					
Gust Factor	**0.849930408**						
Iz	0.264788883						
Z bar	69.798						
c	0.3						
Q (n/s)	0.865816967						
Lz	410.7645834						
gv	3.4						

Table A-1

Wind Load Continued:

Floor	Height	Kz values	qz
ground	0	0.57	10.05
1	11.5	0.57	10.05
2	20.292	0.66	11.63
3	29.08	0.7	12.34
4	37.875	0.76	13.40
5	46.67	0.81	14.28
6	55.458	0.85	14.98
7	64.25	0.89	15.69
8	73.04	0.93	16.39
9	81.83	0.96	16.92
10	90.625	0.99	17.45
11	99.42	0.99	17.45
12	108.83	1.04	18.33
roof	116.33	1.04	18.33

N/S direction **(lbs/ft^2)**

P (windward)	P (leeward)	P (net)
10.13	-11.09	21.22
10.13	-11.09	21.22
11.21	-11.09	22.30
11.69	-11.09	22.78
12.41	-11.09	23.50
13.01	-11.09	24.10
13.49	-11.09	24.58
13.97	-11.09	25.06
14.45	-11.09	25.53
14.80	-11.09	25.89
15.16	-11.09	26.25
15.16	-11.09	26.25
15.76	-11.09	26.85
15.76	-11.09	26.85

Floor	Height	Kz values	qz
ground	0	0.57	10.05
1	11.5	0.57	10.05
2	20.292	0.66	11.63
3	29.08	0.7	12.34
4	37.875	0.76	13.40
5	46.67	0.81	14.28
6	55.458	0.85	14.98
7	64.25	0.89	15.69
8	73.04	0.93	16.39
9	81.83	0.96	16.92
10	90.625	0.99	17.45
11	99.42	0.99	17.45
12	108.83	1.04	18.33
roof	116.33	1.04	18.33

E/W direction **(lbs/ft^2)**

P (windward)	P (leeward)	P (net)
10.13	-1.37	11.51
10.13	-1.37	11.51
11.21	-1.37	12.58
11.69	-1.37	13.06
12.41	-1.37	13.78
13.01	-1.37	14.38
13.49	-1.37	14.86
13.97	-1.37	15.34
14.45	-1.37	15.82
14.80	-1.37	16.18
15.16	-1.37	16.54
15.16	-1.37	16.54
15.76	-1.37	17.14
15.76	-1.37	17.14

Table A- 1 Continued

Seismic Load:

Seismic Cals

Seisimc Use Group		I	Table 9.1.3
Occupancy Category		II	Table 1
Impartance Factor	I	1	Table 9.1.4
Max Ground Motions			
	Ss	18.7	Fig 4.1.1
	Si	6.3	Fig 4.1.1
Site Class		C	9.4.2.4
Stie Class Factors			
	Fa	1	Table 9.4.1.3.4a
	Fv	1.3	Table 9.4.1.3.4b

height (ft)	108.58	
Ct	0.02	Table 9.5.5.3.2
x	0.75	Table 9.5.5.3.2

Sms	18.7
Smi	8.19

Sds	12.47
Sdi	5.46

Seismic Design Cat.		A	Table 9.4.21
Response Mod. Fact.	R (n/s)	5	Table 9.5.2.2
	R (e/w)	5	Table 9.5.2.2
Building Frame	Wo (n/s)	2.5	Table 9.5.2.2
	Wo (e.w)	2.5	Table 9.5.2.2
	Cd (n/s)	4.5	Table 9.5.2.2
	Cd (e/w)	4.5	Table 9.5.2.2
Structure Type	Ct	0.02	Table 9.5.5.3.2
	x	0.75	Table 9.5.5.3.2

Seismic Resp. Coef	Cs	0.025	9.5.5.2.1
	Cs (max)	0.016	
	Cs (min)	0.005	
	Cs	0.016	

Period	Ta	0.67	Eq 9.5.5.3.2-1
	k	1.09	9.5.4.4

Seismic Base Shear	V (kips)	105.73	Eq 9.5.5.2-1

exterior wall weight (ft^2)	30

Table A-2

Seismic Loading Continued:

Floor	height (ft)	Floor Area (fts)	Slab thickness (in)	Floor Load (kips)	Exterior Wall length (ft)	Exterior wall trib height (ft)	Wall Load (kips)
roof	108.58	3871.00	8.00	387.10	256	4.71	36.13
12	99.17	3871.00	8.00	387.10	256	9.10	69.91
11	90.38	3871.00	8.00	387.10	256	8.79	67.55
10	81.58	3871.00	8.00	387.10	256	8.79	67.53
9	72.79	3871.00	8.00	387.10	256	8.79	67.51
8	64.00	3871.00	8.00	387.10	256	8.79	67.51
7	55.21	4560.64	8.00	456.06	298	8.79	78.58
6	46.42	4699.34	8.00	469.93	298	8.79	78.60
5	37.63	4699.34	8.00	469.93	298	8.80	78.63
4	28.83	4699.34	8.00	469.93	298	8.79	78.60
3	20.04	4699.34	8.00	469.93	298	8.79	78.58
2	11.25	4560.64	8.00	456.06	298	10.02	89.59
Ground	0.00	4900.00	8.00	490.00	298	5.63	50.29

Floor	height (ft)	Total Load (kips)	wx*hx^k	Cvx	Fx (kips)	Vx (kips)	Mx (kip ft)
roof	108.58	423.23	68449.38	0.14	14.88		1615.74
12	99.17	457.01	66987.79	0.14	14.56	14.88	1444.20
11	90.375	454.65	60253.93	0.12	13.10	29.44	1183.82
10	81.58	454.63	53916.66	0.11	11.72	42.54	956.22
9	72.79	454.61	47641.36	0.10	10.36	54.26	753.89
8	64	454.61	41432.54	0.09	9.01	64.62	576.47
7	55.21	534.65	41510.32	0.09	9.02	73.63	498.23
6	46.42	548.54	35284.38	0.07	7.67	82.65	356.07
5	37.625	548.56	28094.23	0.06	6.11	90.32	229.80
4	28.83	548.53	21044.17	0.04	4.57	96.43	131.90
3	20.042	548.52	14183.91	0.03	3.08	101.01	61.80
2	11.25	545.65	7540.78	0.02	1.64	104.09	18.44
Ground	0	540.29	0.00	0.00	0.00	105.73	0.00
			486339.46				

Total Building Weight (kips)	6513.46
Overturning Moment	7826.58

Table A-2 Continued

Load Cases:

Case 1: 1.4D
Case 2: 1.2D + 1.6L + .5Lr
Case 3: 1.2D + 1.6Lr + (L or .8W)
Case 4: 1.2D + 1.6W + .5L + .5Lr
Case 5: 1.2D + E + .2S
Case 6: .9D + 1.6W + 1.6H
Case 7: .9D + 1E + 1.6H

Story	D (psf)	L (psf)	Lr (psf)	S (psf)	W (psf)	E (psf)
12	120	60	20	15.75	26.85	3.84
11	120	60	20	15.75	26.25	3.76
10	120	60	20	15.75	26.25	3.38
9	120	60	20	15.75	25.89	3.02
8	120	60	20	15.75	25.53	2.67
7	120	60	20	15.75	25.06	2.32
6	120	60	20	15.75	24.58	1.97
5	120	60	20	15.75	24.1	1.632
4	120	60	20	15.75	23.5	1.299
3	120	60	20	15.75	22.78	0.973
2	120	60	20	15.75	22.3	0.656
1	120	100	20	15.75	21.22	0.359

Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7
168	250	197.48	226.96	78.99	150.96	111.84
168	250	197	226	78.91	150	111.76
168	250	197	226	78.53	150	111.38
168	250	196.712	225.424	78.17	149.424	111.02
168	250	196.424	224.848	77.82	148.848	110.67
168	250	196.048	224.096	77.47	148.096	110.32
168	250	195.664	223.328	77.12	147.328	109.97
168	250	195.28	222.56	76.782	146.56	109.632
168	250	194.8	221.6	76.449	145.6	109.299
168	250	194.224	220.448	76.123	144.448	108.973
168	250	193.84	219.68	75.806	143.68	108.656
168	314	192.976	237.952	123.509	141.952	108.359

Table A-3

Beam Summary

RAM Steel v8.1
DataBase: total
Building Code: IBC

02/13/06 10:02:34
Steel Code: AISC LRFD

STEEL BEAM DESIGN SUMMARY:

Floor Type: resid 2

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
1	13.00	62.9	0.0	75.8	50.0	W8X10	10
16	27.00	92.8	0.0	132.3	50.0	W12X14	14
32	27.00	113.1	0.0	139.4	50.0	W12X14	16
2	16.00	95.2	0.0	113.6	50.0	W10X12	16
18	27.00	126.0	0.0	153.1	50.0	W12X14	22
31	27.00	136.9	0.0	161.7	50.0	W12X14	27
3	20.00	149.1	0.0	174.7	50.0	W12X16	24
20	27.00	151.1	0.0	185.6	50.0	W12X19	20
30	27.00	164.6	0.0	199.1	50.0	W12X19	26
4	16.00	95.2	0.0	113.6	50.0	W10X12	16
22	27.00	151.1	0.0	185.6	50.0	W12X19	20
29	27.00	136.9	0.0	161.7	50.0	W12X14	27
5	13.00	62.9	0.0	75.8	50.0	W8X10	10
24	27.00	126.0	0.0	153.1	50.0	W12X14	22
28	27.00	113.1	0.0	139.4	50.0	W12X14	16
26	27.00	92.8	0.0	132.3	50.0	W12X14	14
6	13.00	101.1	0.0	121.8	50.0	W10X12	18
17	23.00	67.2	0.0	93.4	50.0	W10X12	10
33	23.00	81.9	0.0	100.0	50.0	W10X12	12
7	16.00	152.9	0.0	181.9	50.0	W12X16	24
19	23.00	91.3	0.0	109.4	50.0	W10X12	16
34	23.00	100.6	0.0	122.4	50.0	W10X12	22
8	20.00	131.9	0.0	157.3	50.0	W12X14	22
21	23.00	53.7	0.0	79.4	50.0	W8X10	14
9	16.00	152.9	0.0	181.9	50.0	W12X16	24
23	23.00	53.7	0.0	79.4	50.0	W8X10	14
35	23.00	100.6	0.0	122.4	50.0	W10X12	22
10	13.00	101.1	0.0	121.8	50.0	W10X12	18
25	23.00	91.3	0.0	109.4	50.0	W10X12	16
36	23.00	81.9	0.0	100.0	50.0	W10X12	12
27	23.00	67.2	0.0	93.4	50.0	W10X12	10
11	13.00	54.8	0.0	65.3	50.0	W8X10	6
12	16.00	82.9	0.0	100.4	50.0	W10X12	10
13	20.00	23.4	0.0	56.3	50.0	W8X10	6
14	16.00	82.9	0.0	100.4	50.0	W10X12	10
15	13.00	54.8	0.0	65.3	50.0	W8X10	6

Floor Type: resid 1

Table A-4

Beam Summary

RAM Steel v8.1 — Page 2/6
DataBase: total — 02/13/06 10:02:34
Building Code: IBC — Steel Code: AISC LRFD

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
1	13.00	62.9	0.0	75.8	50.0	W8X10	10
21	16.13	32.9	0.0	60.1	50.0	W8X10	6
38	27.00	113.1	0.0	139.4	50.0	W12X14	16
2	16.00	95.2	0.0	113.6	50.0	W10X12	16
24	27.00	126.0	0.0	153.1	50.0	W12X14	22
37	27.00	136.9	0.0	161.7	50.0	W12X14	27
3	20.00	149.1	0.0	174.7	50.0	W12X16	24
26	27.00	151.1	0.0	185.6	50.0	W12X19	20
36	27.00	164.6	0.0	199.1	50.0	W12X19	26
4	16.00	95.2	0.0	113.6	50.0	W10X12	16
28	27.00	151.1	0.0	185.6	50.0	W12X19	20
35	27.00	136.9	0.0	161.7	50.0	W12X14	27
5	13.00	62.9	0.0	75.8	50.0	W8X10	10
30	27.00	126.0	0.0	153.1	50.0	W12X14	22
34	27.00	113.1	0.0	139.4	50.0	W12X14	16
32	27.00	92.8	0.0	132.3	50.0	W12X14	14
6	21.00	78.7	0.0	101.5	50.0	W10X12	10
19	10.88	6.5	0.0	57.7	50.0	W8X10	4
22	10.88	9.2	0.0	66.0	50.0	W8X10	6
7	21.00	173.0	0.0	209.0	50.0	W12X19	24
20	23.00	92.3	0.0	111.9	50.0	W12X14	8
39	23.00	127.4	0.0	152.4	50.0	W12X14	22
8	13.00	101.1	0.0	121.8	50.0	W10X12	18
23	23.00	107.0	0.0	127.4	50.0	W12X14	12
40	23.00	81.9	0.0	100.0	50.0	W10X12	12
9	16.00	152.9	0.0	181.9	50.0	W12X16	24
25	23.00	91.3	0.0	109.4	50.0	W10X12	16
41	23.00	100.6	0.0	122.4	50.0	W10X12	22
10	20.00	131.9	0.0	157.3	50.0	W12X14	22
27	23.00	53.7	0.0	79.4	50.0	W8X10	14
11	16.00	152.9	0.0	181.9	50.0	W12X16	24
29	23.00	53.7	0.0	79.4	50.0	W8X10	14
42	23.00	100.6	0.0	122.4	50.0	W10X12	22
12	13.00	101.1	0.0	121.8	50.0	W10X12	18
31	23.00	91.3	0.0	109.4	50.0	W10X12	16
43	23.00	81.9	0.0	100.0	50.0	W10X12	12
33	23.00	67.2	0.0	93.4	50.0	W10X12	10
13	21.00	143.0	0.0	168.9	50.0	W12X16	20
14	13.00	54.8	0.0	65.3	50.0	W8X10	6
15	16.00	82.9	0.0	100.4	50.0	W10X12	10
16	20.00	23.4	0.0	56.3	50.0	W8X10	6
17	16.00	82.9	0.0	100.4	50.0	W10X12	10
18	13.00	54.8	0.0	65.3	50.0	W8X10	6

Table A-4, Continued

Beam Summary

RAM Steel v8.1
DataBase: total
Building Code: IBC

Page 3/6
02/13/06 10:02:34
Steel Code: AISC LRFD

Floor Type: Ground

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
2	21.00	292.6	0.0	346.0	50.0	W16X31	20
20	27.00	778.6	0.0	930.5	50.0	W24X68	13, 12
39	27.00	320.3	0.0	383.7	50.0	W16X31	34
3	13.00	81.0	0.0	98.9	50.0	W10X12	10
22	27.00	1067.1	0.0	1259.1	50.0	W27X84	14, 13
38	27.00	151.6	0.0	185.4	50.0	W12X19	20
4	16.00	122.6	0.0	146.0	50.0	W12X14	18
24	27.00	168.8	0.0	199.0	50.0	W12X19	27
37	27.00	183.1	0.0	222.1	50.0	W14X22	18
5	20.00	192.0	0.0	229.4	50.0	W14X22	18
26	27.00	200.3	0.0	237.6	50.0	W14X22	24
36	27.00	217.6	0.0	272.5	50.0	W16X26	16
6	16.00	122.6	0.0	146.0	50.0	W12X14	18
28	27.00	200.3	0.0	237.6	50.0	W14X22	24
35	27.00	183.1	0.0	222.1	50.0	W14X22	18
7	13.00	81.0	0.0	98.9	50.0	W10X12	10
30	27.00	168.8	0.0	199.0	50.0	W12X19	27
34	27.00	151.6	0.0	185.4	50.0	W12X19	20
32	27.00	109.4	0.0	148.8	50.0	W12X14	22
8	21.00	472.0	0.0	559.2	50.0	W21X44	18
21	23.00	150.5	0.0	181.0	50.0	W12X19	20
40	23.00	240.8	0.0	287.3	50.0	W16X26	20
9	13.00	135.3	0.0	160.2	50.0	W12X14	22
23	23.00	181.0	0.0	215.2	50.0	W14X22	16
41	23.00	109.6	0.0	133.6	50.0	W12X14	14
10	16.00	205.1	0.0	243.9	50.0	W14X22	22
25	23.00	122.1	0.0	148.1	50.0	W12X14	20
42	23.00	134.8	0.0	162.2	50.0	W12X16	20
11	20.00	174.9	0.0	212.0	50.0	W14X22	12
27	23.00	69.1	0.0	93.3	50.0	W10X12	10
12	16.00	205.1	0.0	243.9	50.0	W14X22	22
29	23.00	69.1	0.0	93.3	50.0	W10X12	10
43	23.00	134.8	0.0	162.2	50.0	W12X16	20
13	13.00	135.3	0.0	160.2	50.0	W12X14	22
31	23.00	122.1	0.0	148.1	50.0	W12X14	20
44	23.00	109.6	0.0	133.6	50.0	W12X14	14
33	23.00	79.2	0.0	106.9	50.0	W10X12	16
14	21.00	251.8	0.0	298.5	50.0	W16X26	20
15	13.00	69.9	0.0	82.8	50.0	W8X10	14
16	16.00	105.9	0.0	128.4	50.0	W12X14	10
17	20.00	21.6	0.0	44.2	50.0	W8X10	6
18	16.00	105.9	0.0	128.4	50.0	W12X14	10

Table A-4, Continued

Beam Summary

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
19	13.00	69.9	0.0	82.8	50.0	W8X10	14

Floor Type: L-1

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
1	21.00	211.8	0.0	258.3	50.0	W16X26	10
19	27.00	156.2	0.0	199.2	50.0	W14X22	12
38	27.00	226.0	0.0	272.5	50.0	W16X26	16
2	13.00	81.0	0.0	98.9	50.0	W10X12	10
21	27.00	191.7	0.0	227.9	50.0	W14X22	20
37	27.00	151.6	0.0	185.4	50.0	W12X19	20
3	16.00	122.6	0.0	146.0	50.0	W12X14	18
23	27.00	168.8	0.0	199.0	50.0	W12X19	27
36	27.00	183.1	0.0	222.1	50.0	W14X22	18
4	20.00	192.0	0.0	229.4	50.0	W14X22	18
25	27.00	200.3	0.0	237.6	50.0	W14X22	24
35	27.00	217.6	0.0	272.5	50.0	W16X26	16
5	16.00	122.6	0.0	146.0	50.0	W12X14	18
27	27.00	200.3	0.0	237.6	50.0	W14X22	24
34	27.00	183.1	0.0	222.1	50.0	W14X22	18
6	13.00	81.0	0.0	98.9	50.0	W10X12	10
29	27.00	168.8	0.0	199.0	50.0	W12X19	27
33	27.00	151.6	0.0	185.4	50.0	W12X19	20
31	27.00	109.4	0.0	148.8	50.0	W12X14	22
7	21.00	332.7	0.0	394.1	50.0	W16X31	36
20	23.00	112.8	0.0	136.6	50.0	W12X14	16
39	23.00	169.5	0.0	208.1	50.0	W14X22	14
8	13.00	135.3	0.0	160.2	50.0	W12X14	22
22	23.00	143.4	0.0	174.3	50.0	W12X19	16
40	23.00	109.6	0.0	133.6	50.0	W12X14	14
9	16.00	205.1	0.0	243.9	50.0	W14X22	22
24	23.00	122.1	0.0	148.1	50.0	W12X14	20
41	23.00	134.8	0.0	162.2	50.0	W12X16	20
10	20.00	174.9	0.0	212.0	50.0	W14X22	12
26	23.00	69.1	0.0	93.3	50.0	W10X12	10
11	16.00	205.1	0.0	243.9	50.0	W14X22	22
28	23.00	69.1	0.0	93.3	50.0	W10X12	10
42	23.00	134.8	0.0	162.2	50.0	W12X16	20
12	13.00	135.3	0.0	160.2	50.0	W12X14	22
30	23.00	122.1	0.0	148.1	50.0	W12X14	20
43	23.00	109.6	0.0	133.6	50.0	W12X14	14
32	23.00	79.2	0.0	106.9	50.0	W10X12	16
13	21.00	183.1	0.0	219.6	50.0	W14X22	14
14	13.00	69.9	0.0	82.8	50.0	W8X10	14
15	16.00	105.9	0.0	128.4	50.0	W12X14	10

Table A-4, Continued

Beam Summary

RAM Steel v8.1
DataBase: total
Building Code: IBC

Page 5/6
02/13/06 10:02:34
Steel Code: AISC LRFD

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
16	20.00	21.6	0.0	44.2	50.0	W8X10	6
17	16.00	105.9	0.0	128.4	50.0	W12X14	10
18	13.00	69.9	0.0	82.8	50.0	W8X10	14

Floor Type: Concourse

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
1	21.00	368.3	0.0	438.5	50.0	W18X35	22
19	27.00	257.4	0.0	309.2	50.0	W16X26	24
38	27.00	402.2	0.0	474.7	50.0	W18X35	44
2	13.00	141.0	0.0	168.5	50.0	W12X19	10
21	27.00	345.5	0.0	408.0	50.0	W16X31	50
37	27.00	275.1	0.0	327.3	50.0	W16X26	30
3	16.00	213.7	0.0	264.5	50.0	W16X26	10
23	27.00	306.5	0.0	364.2	50.0	W16X26	54
36	27.00	331.0	0.0	392.8	50.0	W16X31	40
4	20.00	334.2	0.0	396.2	50.0	W18X35	14
25	27.00	360.2	0.0	427.5	50.0	W18X35	24
35	27.00	388.4	0.0	459.7	50.0	W18X35	36
5	16.00	213.7	0.0	264.5	50.0	W16X26	10
27	27.00	360.2	0.0	427.5	50.0	W18X35	24
34	27.00	331.0	0.0	392.8	50.0	W16X31	40
6	13.00	141.0	0.0	168.5	50.0	W12X19	10
29	27.00	306.5	0.0	364.2	50.0	W16X26	54
33	27.00	275.1	0.0	327.3	50.0	W16X26	30
31	27.00	173.2	0.0	205.8	50.0	W14X22	12
7	21.00	594.2	0.0	699.1	50.0	W21X44	46
20	23.00	186.5	0.0	221.6	50.0	W14X22	16
39	23.00	304.7	0.0	361.2	50.0	W16X31	26
8	13.00	245.7	0.0	290.1	50.0	W16X26	14
22	23.00	260.4	0.0	310.4	50.0	W16X26	24
40	23.00	199.3	0.0	238.4	50.0	W14X22	20
9	16.00	372.5	0.0	444.7	50.0	W18X35	22
24	23.00	222.4	0.0	280.9	50.0	W16X26	16
41	23.00	245.2	0.0	290.0	50.0	W16X26	18
10	20.00	317.1	0.0	382.8	50.0	W18X35	12
26	23.00	125.2	0.0	150.5	50.0	W12X14	18
11	16.00	372.5	0.0	444.7	50.0	W18X35	22
28	23.00	125.2	0.0	150.5	50.0	W12X14	18
42	23.00	245.2	0.0	290.0	50.0	W16X26	18
12	13.00	245.7	0.0	290.1	50.0	W16X26	14
30	23.00	222.4	0.0	280.9	50.0	W16X26	16
43	23.00	199.3	0.0	238.4	50.0	W14X22	20
32	23.00	125.0	0.0	150.5	50.0	W12X14	18
13	21.00	316.6	0.0	371.3	50.0	W16X31	24

Table A-4, Continued

Beam Summary

RAM Steel v8.1 | Page 6/6
DataBase: total | 02/13/06 10:02:34
Building Code: IBC | Steel Code: AISC LRFD

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
14	13.00	121.0	0.0	146.9	50.0	W12X14	14
15	16.00	183.6	0.0	219.4	50.0	W14X22	12
16	20.00	21.6	0.0	45.2	50.0	W8X10	5
17	16.00	183.6	0.0	219.4	50.0	W14X22	12
18	13.00	121.0	0.0	146.9	50.0	W12X14	14

* after Size denotes beam failed stress/capacity criteria.
\# after Size denotes beam failed deflection criteria.
u after Size denotes this size has been assigned by the User.

Table A-4, Continued

Girder Design:

Ultimate Moment

W= 1.2D + 1.6L = 270 psf
W= Trib. Width * W = 18.5' * 270 psf = 5 klf

Design Top Steel:

$$M= \frac{wl^{\wedge}2}{12} = \frac{5*27^{\wedge}2}{12}{}_2 = 303.75 \text{ ft-k}$$

$$Mu= \frac{M}{\phi} = \frac{303.75}{.9} = 337.5 \text{ ft-k}$$

Assume d= 12" + 3" slab= 15"
Assume b= 2' = 24"

$$As= \frac{Mn}{fy(d - \frac{a}{2})}$$

Assume $(d - \frac{a}{2})$ = .9d
Let d = 15- 2.5 = 12.5"

$$As= \frac{337.5}{60*.9*12.5} = 6 \text{ square inches}$$

From *Design of Concrete Structures by Nilson*
Table A.2, use 6 #9's as bottom steel in girder ρ=.0167

Check:
d= 15 – 1/2 – 1.5" cover = 13"

$$a_s= \frac{Asfy}{.85f'cbeff} = \frac{6*60}{.85*4*24} = 4.41$$

Mn= $Asfy\ (d - \frac{a}{2})$ = 6*60*(13 – 4.41/2) = 3886.2/12= 323.85 <337.5

This does not work.

Try 7 #9's ρ=.02
Check:
d= 15 – 1/2 – 1.5" cover = 13"

$$a_s= \frac{Asfy}{.85f'cbeff} = \frac{7*60}{.85*4*24} = 5.14$$

Mn= $Asfy\ (d - \frac{a}{2})$ = 7*60(13-5.14/2) = 4380.6/12 = 365 <337.5 OK!

Design Bottom Steel:

M= $\frac{wl\text{^}2}{24} = \frac{5*27\text{^}2}{24}{}^{2}$ = 151.875 ft-k

Mu= $\frac{M}{\phi} = \frac{151.875}{.9}$ = 168.75 ft-k

Assume d= 12" + 3" slab= 15"
Assume b= 2' = 24"

As= $\frac{Mn}{fy(d-\frac{a}{2})}$

Assume $(d-\frac{a}{2})$ = .9d
Let d = 15- 2.5 = 12.5"

As= $\frac{168.75}{60*.9*12.5}$ = 3 square inches

From *Design of Concrete Structures by Nilson*
Table A.2, use 4 #8's as bottom steel in girder

Check:
d= 15 – .79/2 – 1.5" cover = 13.1"

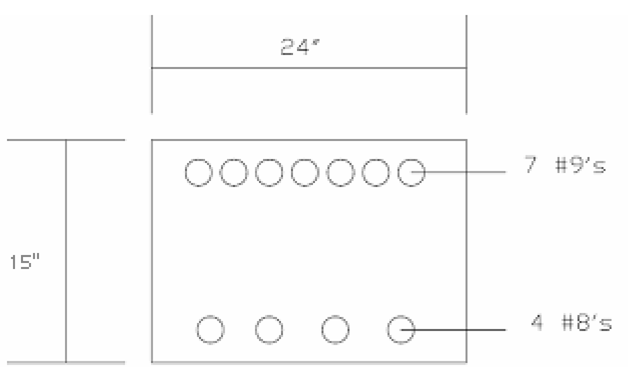
a_s= $\frac{Asfy}{.85f'cbeff} = \frac{3.16*60}{.85*4*24}$ = 2.32

Mn= $Asfy\ (d-\frac{a}{2})$ = 3.16*60*(13.1 – 2.32/2) = 2263.8/12= 188.6 <169

Ductility Check:
T=C
Asfy = (As' Fy') + (.85*f'c*b*a)
7*60= (60*3.16) + (.85*4*24*a)
A=2.8

C= a/β = 2.8 / .85 = 3.32

E = .003*(13-3.32) / 3.32 = .008 > .005 OK



Girder Cross Section
Figure A-9

Bar Cut Offs

Top Reinforcement, 7 #9's A=7 in^2

Development length: L_d= 62d = 62* 9/8 = 70 in = 5.8'

Theoretical cutoff: 3 #9 A=3 in^2

$$a_s = \frac{Asfy}{.85 f'cbeff} = \frac{3*60}{.85*4*24} = 2.2 \text{ in}$$

ΦMn=.9 $Asfy\ (d - \frac{a}{2})$ = .9*3*60*(13-2.2/2) = 160 ft-k

Point at which Mn= 160 ft kips is x = 2.33'

Therefore the cutoff is at x+12d_b=28+ 12*9/8 = 41.5"

Or cutoff = 62"

Or development length = 69.6" ← Controls

Point of inflection = .211l = .211*27'=68.36 in

Cutoff at 68.36 + 13 = 81.36"

Or 55.02" + 70" = 125.02" ← Controls

Bottom Reinforcement, 4 #8 A= 3.16 in^2

Not continuous, 2 bars need to be carried into supports

Theoretical cutoff: 2 #8's A=1.58 in2

$$a_s = \frac{Asfy}{.85 f'cbeff} = \frac{1.58*60}{.85*4*24} = 1.16 \text{ in}$$

ΦMn=.9 $Asfy\ (d - \frac{a}{2})$ = .9*1.58*60*(13-1.16/2) = 88.3 ft-k

Point at which Mu= 88.3 ft-k is at x = 8.5' ← Controls

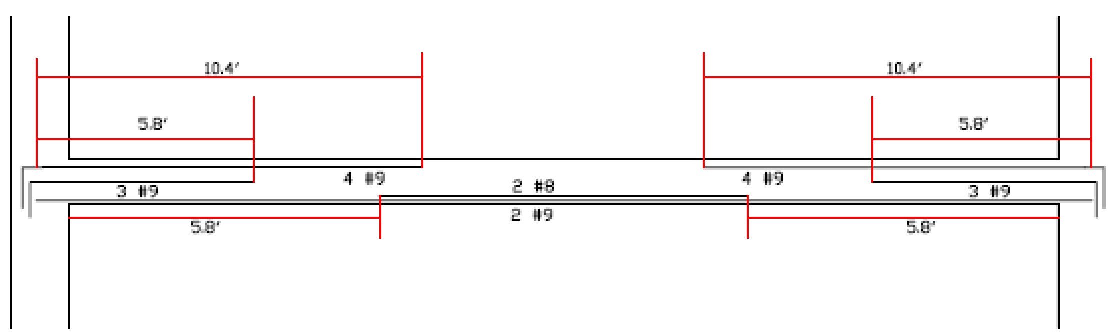
Point of inflection= 68.36"

Check 12.11.3 ACI

$$Ld \le \frac{Mn}{Vu} + La$$

28 < 88.3/ 42.5 +13 = 15 in

Not okay, bottom bars must be hooked at column centerline.



Reinforcement Cut Offs
Figure A-10

Shear Design

Vu at d from support = 61.25 kips (by similar triangles)

$$Vs = \frac{Vu}{\phi} - Vc$$

$$Vc = 2\sqrt{4000} * 24 * 13 = 39.5 \text{ kips}$$

Vs = 61.25/.8 – 39.5 = 37 kips

Max Spacing S= d/2 = 13/2 = 6.5"

Min Av = .75* $\sqrt{4000}$ *24* 6.5 / 60000 = .1233

Min Av = 50*24*6.5/ 60000 = .13 ← Controls

From *Design of Concrete Structures by Nilson*

Table A.3 Try #3 bars every 6.5" Av= .22"

Vs min = AvFy*d/s = .22*60*13/6.5 =26.4 kips

Spacing at supports

$$S = \frac{Avfyd}{Vs} = \frac{.22 * 60 * 13}{37} = 4.63" \approx 4.5"$$

Spacing Cut offs

Vu = Φ(Vc + Vs) = .8(39.5 + 26.4) = 52.72 kips

By similar triangles, cut off is at 10.5 ft

Vu=ΦVc/2 = .8*39.5/2 = 15.8

By similar triangles, cut off is at 3.16 ft

Solution:

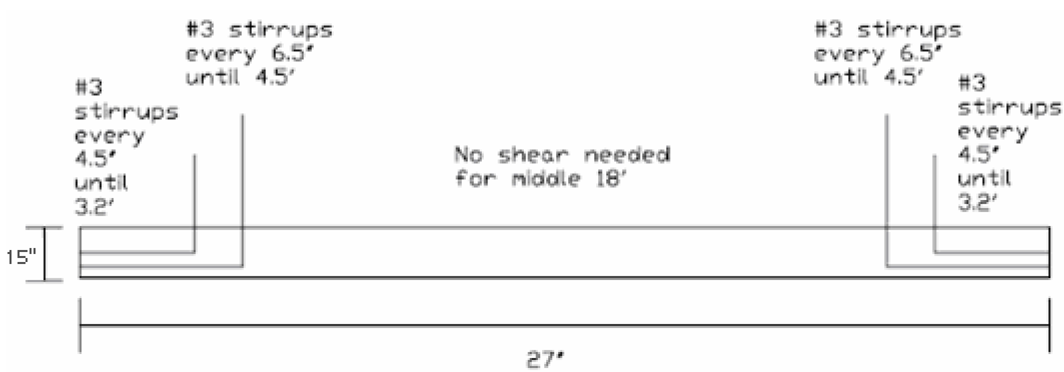
Top Steel = 7 #9's

Bottom Steel = 4 #8's

Shear:

#3 stirrups every 6.5" until 4.5 feet from the support

#3 stirrups every 4.5" until 3.2 feet from the support



Stirrup Spacing

Figure A-11

Column Design:

Assume a square column. Assume **b=16”** and #8 bars

Pick an interaction diagram based on γ:

$$\gamma = \frac{b - 2\left(\frac{d}{2} + \text{cov}\,er\right)}{b} = \frac{16 - 2\left(\frac{1}{2} + 2.5\right)}{16} = .667 \text{ round to } .7$$

Use graph A.5 in *Design of Concrete Structures by Nilson*

Kn= $\frac{Pu}{\phi f'cAg}$ See Excel for results   Rn= $\frac{Mu}{\phi f'cAgh}$ See Excel for results

Rn is very insignificant   Design for ρ=.04

As= 10.24   **Use 16 #8 As=12.566 ρ=.049**

Check with Load Contour Method:

Kn=1.11   Let ρ= .049

Therefore Rn=.125

φMn= 1331.2 in-k

$$\left(\frac{\phi Mnx}{\phi Mnxo}\right)^{\alpha} + \left(\frac{\phi Mny}{\phi Mnyo}\right)^{\alpha} > 1$$ All columns checked on Excel.

The above was done for both the actual moment on the columns, and the axial load offset by 1” in off directions.

assumed	
b	16

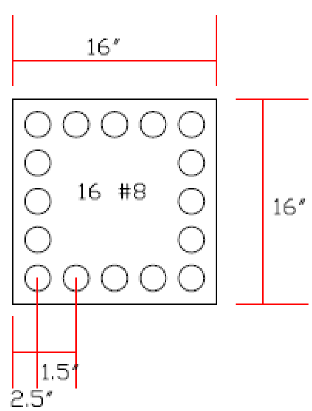
Column	Axial (kips)	Mux (ft-k)	Muy (ft-k)	Kn	Rnx	Rny	contour	< 1
A-1	171.3	9.1	7.7	0.26	0.01	0.01	0.102912	yes
A-2	401.3	10.1	5.3	0.60	0.01	0.01	0.093847	yes
A-3	476	8.4	3.3	0.72	0.01	0.00	0.068986	yes
A-4	591.4	10.7	4.2	0.89	0.01	0.00	0.091101	yes
A-5	591.4	10.7	4.2	0.89	0.01	0.00	0.091101	yes
A-6	476	8.4	3.3	0.72	0.01	0.00	0.068986	yes
A-7	300.4	5.9	4.3	0.45	0.01	0.00	0.058065	yes
B-1	94.7	0.7	3.6	0.14	0.00	0.00	0.022361	yes
B-2	134	3.3	3.3	0.20	0.00	0.00	0.035122	yes
C-1	432.8	1.2	11.6	0.65	0.00	0.01	0.080025	yes
C-2	737.1	1.6	8.6	1.11	0.00	0.01	0.060474	yes
C-3	735.8	1.3	5.8	1.11	0.00	0.01	0.039605	yes
C-4	692.6	6.6	6.2	1.04	0.01	0.01	0.075236	yes
C-5	692.6	6.6	6.2	1.04	0.01	0.01	0.075236	yes
C-6	735.8	1.3	5.8	1.11	0.00	0.01	0.039605	yes
C-7	472	0.9	7.1	0.71	0.00	0.01	0.046325	yes
D-1	263.5	7.1	6.4	0.40	0.01	0.01	0.079999	yes
D-2	420.7	8.4	4.5	0.63	0.01	0.01	0.076512	yes

D-3	417.7	7.2	2.9	0.63	0.01	0.00	0.058207	yes
D-4	312	4.5	3.8	0.47	0.01	0.00	0.045741	yes
D-5	312	4.5	3.8	0.47	0.01	0.00	0.045741	yes
D-6	417.7	7.2	2.9	0.63	0.01	0.00	0.058207	yes
D-7	263.3	5	4	0.40	0.01	0.00	0.050227	yes

Actual Moment

Column	Axial (kips)	Mux (in-k)	Muy (in-k)	Kn	Rnx	Rny	contour	< 1
A-1	171.3	171.3	171.3	0.26	0.02	0.02	3.296891	yes
A-2	401.3	401.3	401.3	0.60	0.04	0.04	8.775527	yes
A-3	476	476	476	0.72	0.04	0.04	10.67903	yes
A-4	591.4	591.4	591.4	0.89	0.06	0.06	13.70715	yes
A-5	591.4	591.4	591.4	0.89	0.06	0.06	13.70715	yes
A-6	476	476	476	0.72	0.04	0.04	10.67903	yes
A-7	300.4	300.4	300.4	0.45	0.03	0.03	6.289825	yes
B-1	94.7	94.7	94.7	0.14	0.01	0.01	1.667578	yes
B-2	134	134	134	0.20	0.01	0.01	2.485731	yes
C-1	432.8	432.8	432.8	0.65	0.04	0.04	9.572249	yes
C-2	737.1	737.1	737.1	1.11	0.07	0.07	17.6579	yes
C-3	735.8	735.8	735.8	1.11	0.07	0.07	17.62209	yes
C-4	692.6	692.6	692.6	1.04	0.07	0.07	16.4376	yes
C-5	692.6	692.6	692.6	1.04	0.07	0.07	16.4376	yes
C-6	735.8	735.8	735.8	1.11	0.07	0.07	17.62209	yes
C-7	472	472	472	0.71	0.04	0.04	10.57589	yes
D-1	263.5	263.5	263.5	0.40	0.02	0.02	5.409802	yes
D-2	420.7	420.7	420.7	0.63	0.04	0.04	9.265141	yes
D-3	417.7	417.7	417.7	0.63	0.04	0.04	9.189202	yes
D-4	312	312	312	0.47	0.03	0.03	6.569941	yes
D-5	312	312	312	0.47	0.03	0.03	6.569941	yes
D-6	417.7	417.7	417.7	0.63	0.04	0.04	9.189202	yes
D-7	263.3	5	263.3	0.40	0.00	0.02	2.730859	yes

1” eccentricity
Table A-5



Final Column Design
Figure A-12

Base Plate Design:

vertical load on column	V (kips)	350
concrete strenght	Fc (ksi)	4

Area of plate	A (in^2)	350

Depth of column	d (in)	10
width of column	b (in)	10

lenght of effective area	E (in)	4.986657

Plate length	L (in)	19.47331
Plate Width	W (in)	17.97331

Steel Strength	Fy (ksi)	36
Plate Thickness	t (in)	3.324438

Table A-6

Cost Estimates:

Excavation:

#	Item	Unit	Crew	Mat	Labor	Equip	Total/ unit	Total
424 0250	Backhoe	BCY	B-12A	0	0.66	0.9	1.56	1084.2
400 4000	Wood Sheathing	SF	B-31	1.83	3.92	0.45	6.2	18600
490 0540	Hauling	LCY		0	1.81	3.81	5.62	3905.9
								23590.1

Foundation:

#	Item	Unit	Crew	Mat	Labor	Equip	Total/ unit	Total
240 4050	Concrete, MAT	CY	C-14C	156	67	0.38	223.38	144750.2
								144750.2

Sub-Grade Levels:

#	Item	Unit	Crew	Mat	Labor	Equip	Total/ unit	Total
240 2500	One Way Joist	CY	C-14B	410	270	26.5	706.5	168853.5
240 0820	Columns	CY	C-14A	410	565	57.5	1032.5	14279.48
240 4260	Grade Walls	CY	C-14D	150	143	14.65	307.15	34124.37
240 4260	Shear Wall	CY	C-14D	150	143	14.65	307.15	7509.818
								224767.2
								per level

Super Structure Levels:

	#	Item	Unit	Crew	Mat	Labor	Equip	Total/ unit	Total
Levels 12-8	300 ####	Composite Deck	SF	E-4	1.97	0.38	0.03	2.38	9401
	240 3150	Elevated Slab	SF	C-8	1.16	0.66	0.27	2.11	8334.5
	840 ####	Shear Studs	Each	E-10	0.43	0.69	0.28	1.4	826
	640 0300	W 8 x 10	LF	E-2	10.45	3.63	2.38	16.46	855.92
	640 0600	W 10 x 12	LF	E-2	12.55	3.63	2.38	18.56	5085.44
	640 1200	W 12 x 16	LF	E-2	16	2.48	1.62	20.1	1045.2
	640 1100	W 12 x 14	LF	E-2	14.65	2.48	1.62	18.75	4425
	640 1250	W 12 x 19	LF	E-2	20	2.48	1.62	24.1	1952.1
	640 1560	W 12 x 50	LF	E-2	52.5	2.9	1.9	57.3	3781.8
									35706.96

Levels 7-2	300 ####	Composite Deck	SF	1 E-4	1.97	0.38	0.03	2.38	11075.33
	240 3150	Elevated Slab	SF	2 C-8	1.16	0.66	0.27	2.11	9818.885
	840 ####	Shear Studs	Each	1 E-10	0.43	0.69	0.28	1.4	943.6
	640 0300	W 8 x 10	LF	1 E-2	10.45	3.63	2.38	16.46	1407.33
	640 0600	W 10 x 12	LF	1 E-2	12.55	3.63	2.38	18.56	4807.04
	640 1200	W 12 x 16	LF	1 E-2	16	2.48	1.62	20.1	1467.3
	640 1100	W 12 x 14	LF	1 E-2	14.65	2.48	1.62	18.75	3937.5
	640 1250	W 12 x 19	LF	1 E-2	20	2.48	1.62	24.1	2458.2
	640 1560	W 12 x 50	LF	1 E-2	75	3.4	2.23	80.63	5321.58
									41236.77 Per level
Braced Frames	640 1550	W 12x 45	LF	1 E-2	52.5	2.9	1.9	57.3	4297.5
Diagonals	640 1555	W 12 x 53	LF	1 E-2	52.5	2.9	1.9	57.3	2887.92
	640 1580	W 12 x 58	LF	1 E-2	60.5	2.9	1.9	65.3	3291.12
	640 1590	W 12 x 65	LF	1 E-2	75	3.4	2.23	80.63	10159.38
Columns	641 1590	W 12 x 65	LF	1 E-2	75	3.4	2	150	12000
		W 12 x 120	LF	1 E-2	120	4	2	150	1500
		W 12 x 170	LF	1 E-2	170	4	2	150	3000
		W 12 x 106	LF	1 E-2	106	4	2	150	1500
		W 12 x 152	LF	1 E-2	153	4	2	150	1500
		W 12 x 190	LF	1 E-2	190	4	2	150	1500
		W 12 x 230	LF	1 E-2	230	4	2	150	1500
		W 12 x 252	LF	1 E-2	252	4	2	150	1500
		W 12 x 279	LF	1 E-2	279	4	2	150	1500
		W 12 x 305	LF	1 E-2	305	4	2	150	1500
		W 12 x 336	LF	1 E-2	336	4	2	150	1500
		W 14 x 311	LF	1 E-2	311	4	2	150	1500
		W 14 x 342	LF	1 E-2	342	4	2	150	1500
									52135.92 Per fram
Columns, not part of a frame	640 0740	W 10 x 33	LF	1 E-2	34.5	3.96	2.59	41.01	76278.6
		W 10 x 39	LF	1 E-2	39	3.96	2.59	45.55	7743.5
		W 10 x 45	LF	1 E-2	45	3.96	2.59	51.55	4124
	640 0900	W 10 x 49	LF	1 E-2	51	3.96	2.59	57.55	5179.5
		W 10 x 54	LF	1 E-2	54	3.96	2.59	60.55	1211
									94536.6