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.

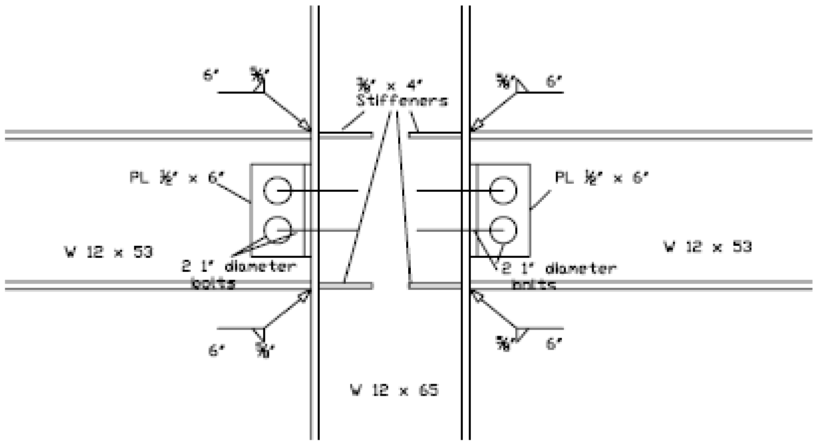


Figure 24

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.

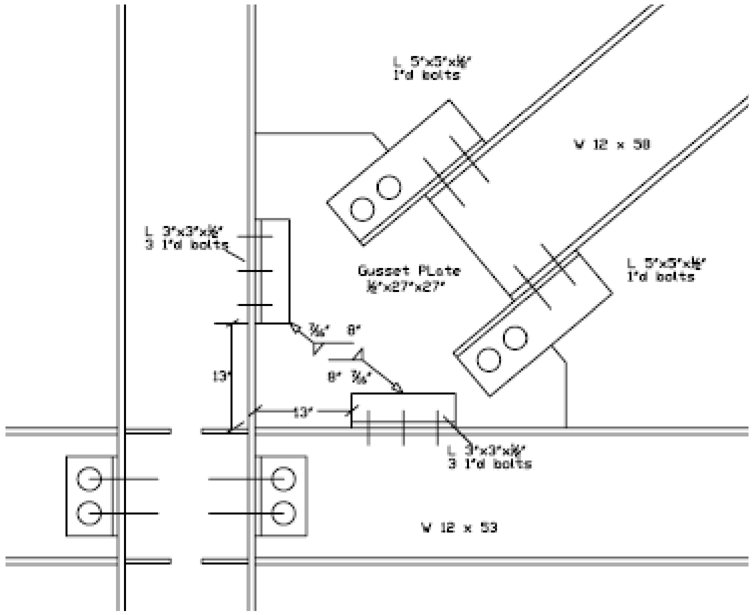


Figure 25

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[1], 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

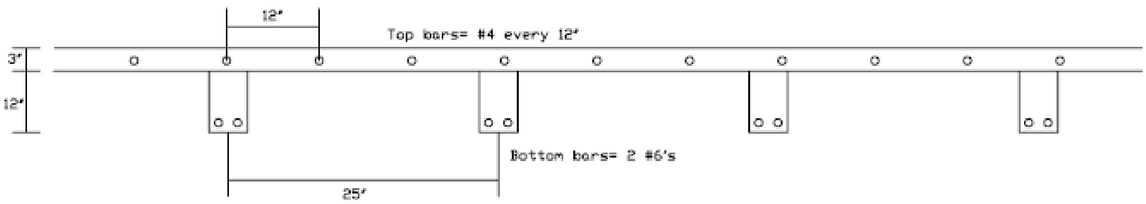


Figure 26
Floor Section

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[2]:

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

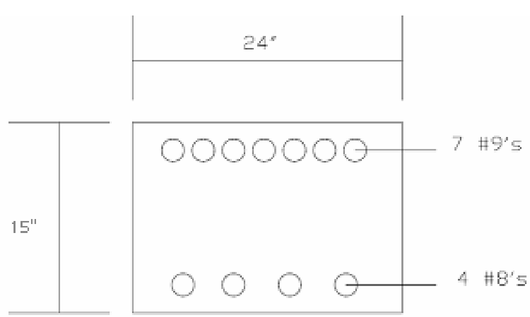


Figure 27
Girder Section with Reinforcement

[1] 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.

[2] For full girder design calculations see Appendix

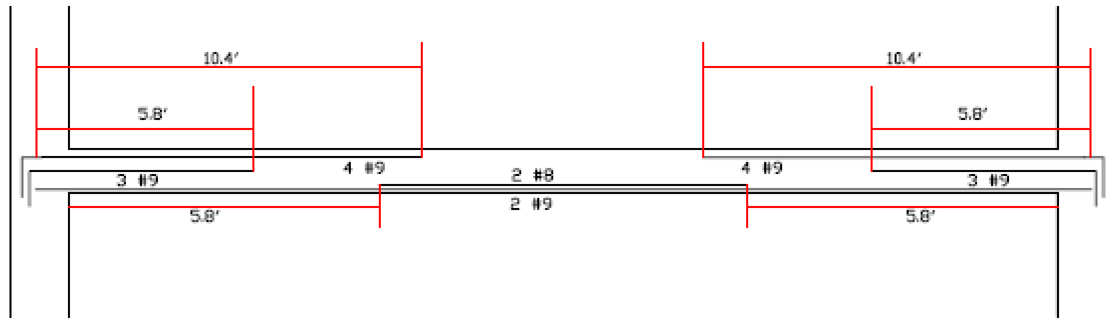


Figure 28
Bar cut offs in Girder

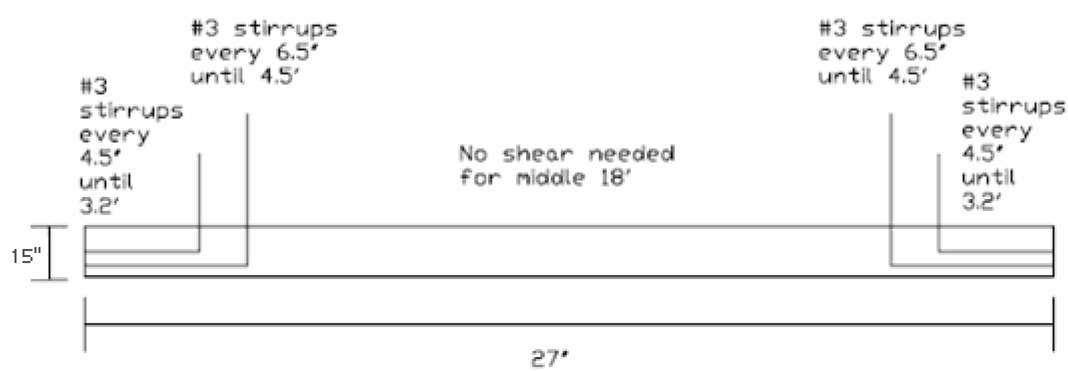


Figure 29
Girder with shear cut offs

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.

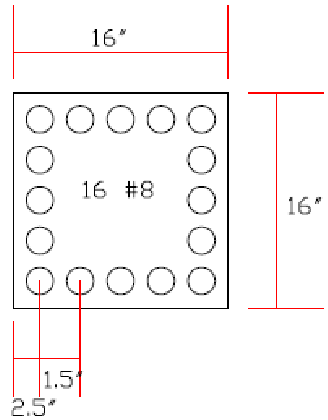
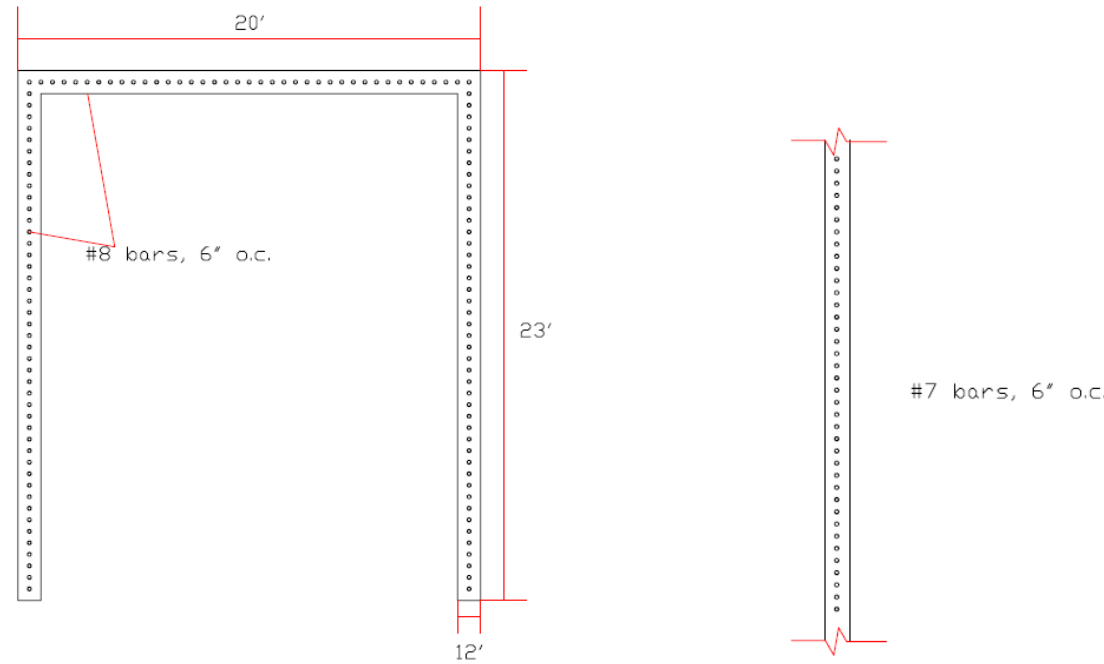


Figure 30
Column Section

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.

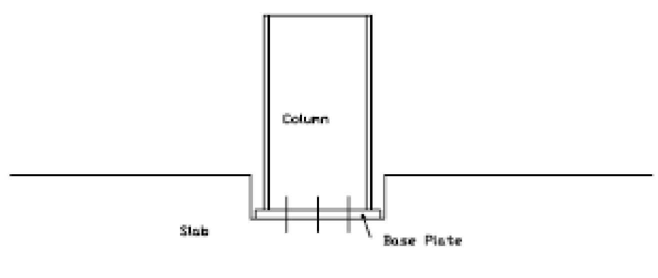


Figure 32

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.

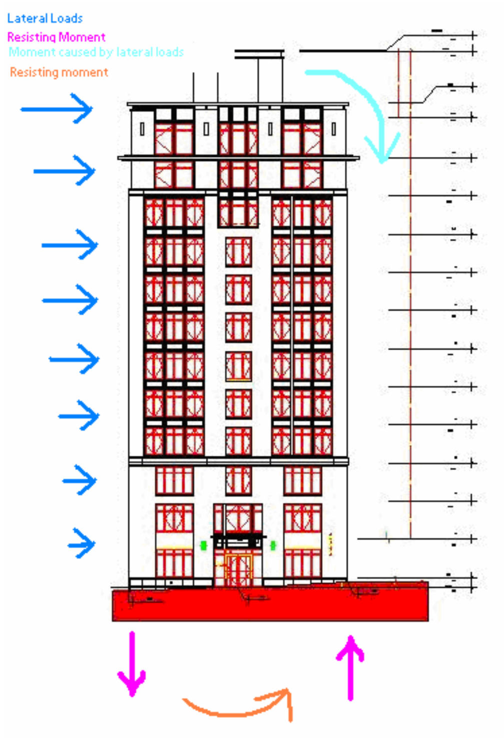


Figure 33
Forces effecting Overturning

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

Mn-s = Mo + W(l/2) – x(l)	Me-w = Mo + W(l/2) – x(l)
0 = 15424 + 4645*50 – x*100	0 = 19132 + 4645*20 – x*50
X= 2476.74	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.