Sallie Mae HQ

Frank Burke
Structural Option
Final Report- Spring 2006
Thesis Advisor – Thomas E. Boothby

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Sallie Mae HQ
Reston, VA

General Project Data

- Overall Project Cost : 41 million Dollars
- Start of Construction : April 2003
- End of Construction : September 2004
- Type of Project : Design-Bid-Build

Project Team

- Owner : Sallie Mae
- Architect : Boggs & Partners Architects
- Structural Engineer : Cagley and Associates
- M/E Engineer : GHT Limited Consulting Engineers

Structural

- 9 floors of steel framing above grade
- Concrete parking garage spans max: 28’
- Steel spans reach max: 42’
- 2” Deep x 18 Gage Composite Metal Deck with 3 1/4” Lightweight Concrete
- Soils conditions allow building to rest on spread footings
- Max load of 3250 kips is on 40” by 30” concrete

Architecture

- Office Building
- 9 stories office complex above grade, 5 level parking structure below grade
- 207,000 SF of office space
- 307,000 SF of parking space
- Post Modern Building featuring glass atrium extending 9 floors in stairwell
- 14’ average floor-to-floor height / 5’ Ceiling

Electrical

- Four 1500 KW diesel generators in a parallel configuration providing standby power to UPS and HVAC equipment
- 2 4000A and 1 3000A Main Switchboards running at 480/277V, 3 Phase 4 wire

Mechanical

- One 900 ton, 2230 GPM cooling tower at penthouse floor
- Electric Heating Coils reaching 9700 CFM
- 80 gallon and 40 gallon Domestic Water Heaters
- Triplex Domestic Water Booster Pump below grade

E portfolio website: http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/fmb112/

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Executive Summary

Sallie Mae's architecture focuses on the idea of Sallie Mae providing and encouraging higher education. The office consists of 9 floors above grade and 5 parking levels below grade. Sallie Mae's new corporate headquarters will be able to support 700 employees with its 207,000 SF of office space. The architect behind Sallie Mae was Boggs & Partners. And the main attraction of the building is the symbolic glass staircase on the outside of the building which is a metaphor to the "classic stairway to learning."

The building layout consists of rectangular bays with the long direction spanning 42' - 55' and the short direction varying between 18' and 28'. Post-tensioning was the best system other then steel to design with, and from analysis a one-way slab system would work best.

The purpose of this thesis was considering a post-tensioning design that is simplistic in nature, and examines constructability to the fullest degree. The way the tendons are layed out, the type of concrete used, and the design of the framing plan are all considerations that I examined fully.

Overall, my design is highly constructible and limits any possible mistakes in the building process to a minimum. All tendons used on the building are in bundles of four, and there are only four prestressed beam sizes. Furthermore, all jacking can occur from the exterior of the building, so the stressing process will be simplified greatly.

Comparing concrete vs. steel led me to believe that the steel process seems to be faster for erection; but that may not be true due to lead-in-time needed for steel systems. Surprisingly post-tensioning was still less expensive, even though I didn't consider lead-in-time. Also, this project was planned out before the big boom in steel prices and the cost data I compared post-tensioning with was from 2002. So post-tensioning seems to be a more worthy claim to investigate.

Overall the final cost of the building is cheaper with the post-tensioning system, but it takes longer to schedule. The longer the project is to schedule, the higher possibility that renting cost can come into play. But in Sallie Mae's case there are no tenants, so rent isn't a factor.

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Introduction:

Sallie Mae's architecture focuses on the idea of Sallie Mae providing and encouraging higher education. The office consists of 9 floors above grade and 5 parking levels below grade. Sallie Mae's new corporate headquarters will be able to support 700 employees with its 207,000 SF of office space. The architect behind Sallie Mae was Boggs & Partners. And the main attraction of the building is the symbolic glass staircase on the outside of the building which is a metaphor to the "classic stairway to learning."

Other Role Players were:

- Structural Engineers: Cagley and Associates
- GC: Hitt Contracting Inc.
- Electrical and MEP: GHT Limited Consulting Engineers

Building Statistics:

- Building name: Sallie Mae Headquarters
- Location and Site: Reston , Virginia
- Building Occupant Name: Sallie Mae

- Occupancy or function types:

    - Primary Occupancy: Office Building
    - Accessory Occupancy: Parking Garage

- Square feet:

    - Office Space: 207,183 SF
    - Parking Garage: 307,164 SF

- Construction:

    - Dates of construction (start-finish): April 2003 – September 2004
    - Overall Project Cost: 41 million
    - Project delivery method: Design-Bid-Build

- Codes

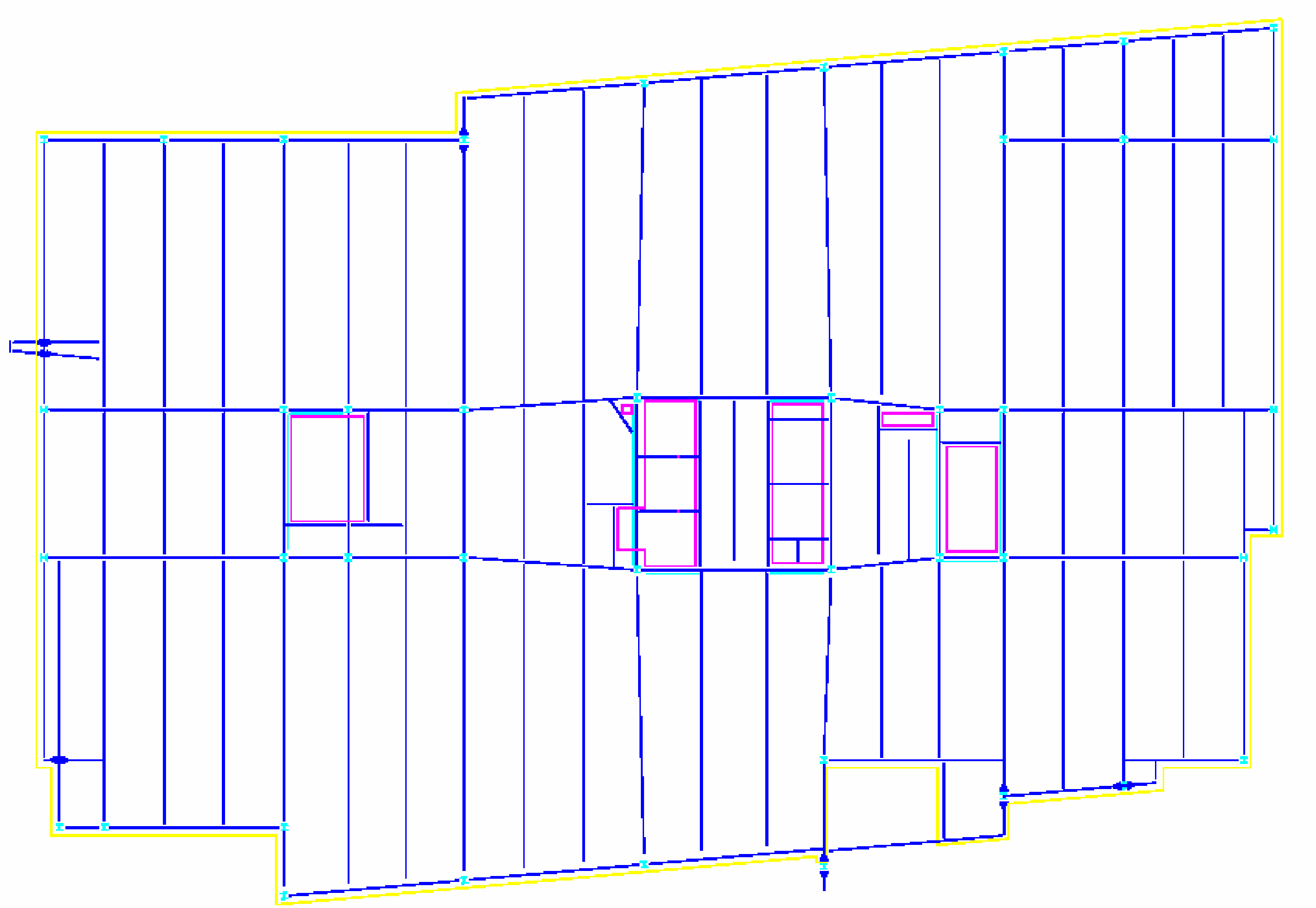
    - The Boca National Building Code – 1996
    - Virginia Uniform Statewide Building Code – 2000
    - ASCE 7-98
    - ACI 318-99

Overview of the Existing Office Structural System:

Gravity:

Sallie Mae HQ is a 9 story office building with a 5 story parking garage submerged underneath. The grade slopes around the building so there isn't one consistent ground level. The parking garage which is 75,000 SF in area consists of tight 28' by 28' bays. These bays are regular reinforced concrete with drops panels and a two-flat slab that primary is 9" to 10" deep.

The office portion of Sallie Mae consists of steel tower framing that extends 9 stories above grade. Typical floors in the office structure consist of lightweight concrete on metal decking, and are approximately 22,000 SF in area. The architecture controls the structural design based on the 42+ spans needed around the central core. To implement this requirement the structural firm Cagley and Associates needed to use cambered steel beams that were about 80lb/ft in weight; W24's and W27's. The floor-to-floor height was 14' with the exception of the two top floors which varied due to the penthouse and roof. The facade is primary glass and needs to be accountable during the design of the building. Most of the columns are leaning columns, and rely on the braced frames to take 100% of the lateral load. Also, the foundation mostly consists of shallow square footings which can be attributed to the rock at the bottom of the garage.

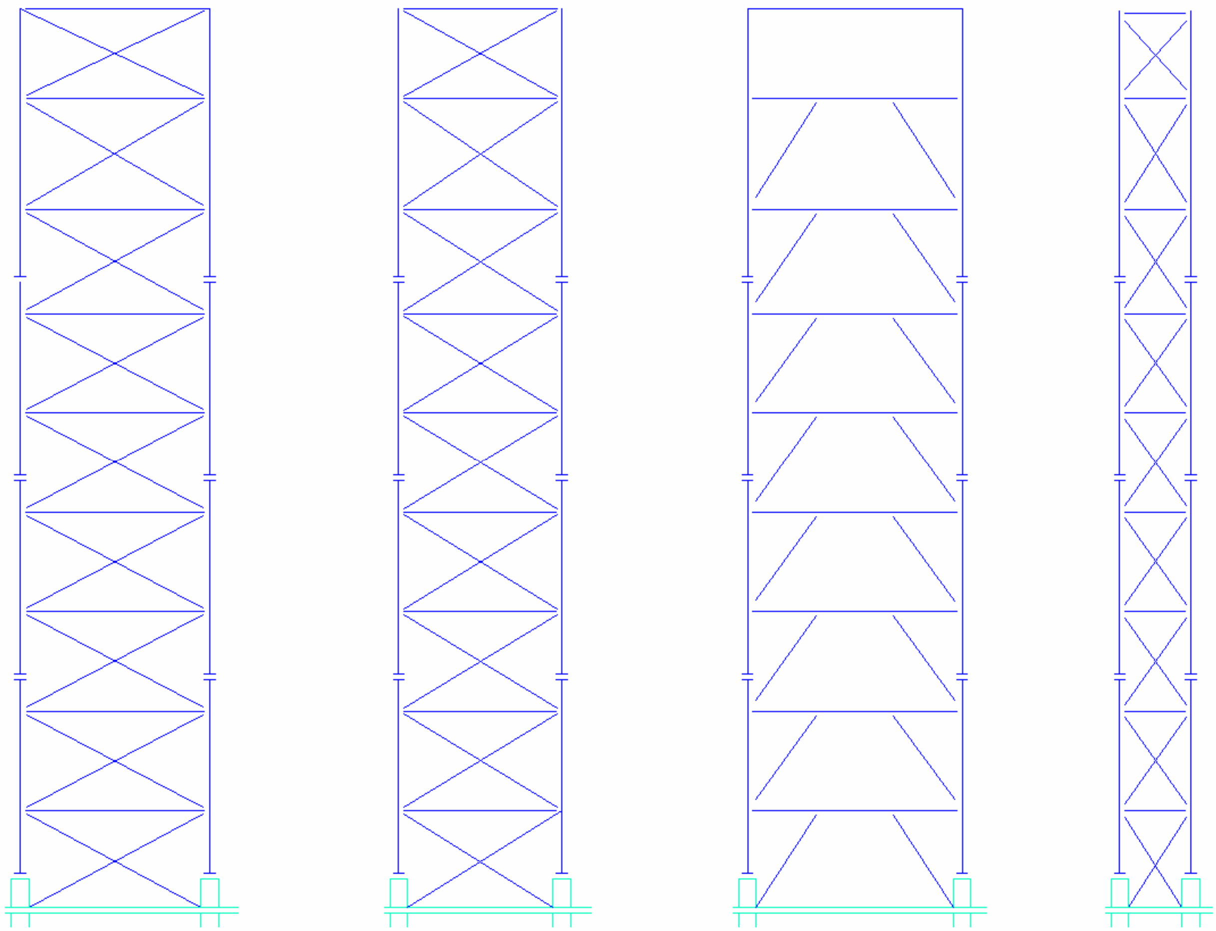


Typical Steel Framing Floor

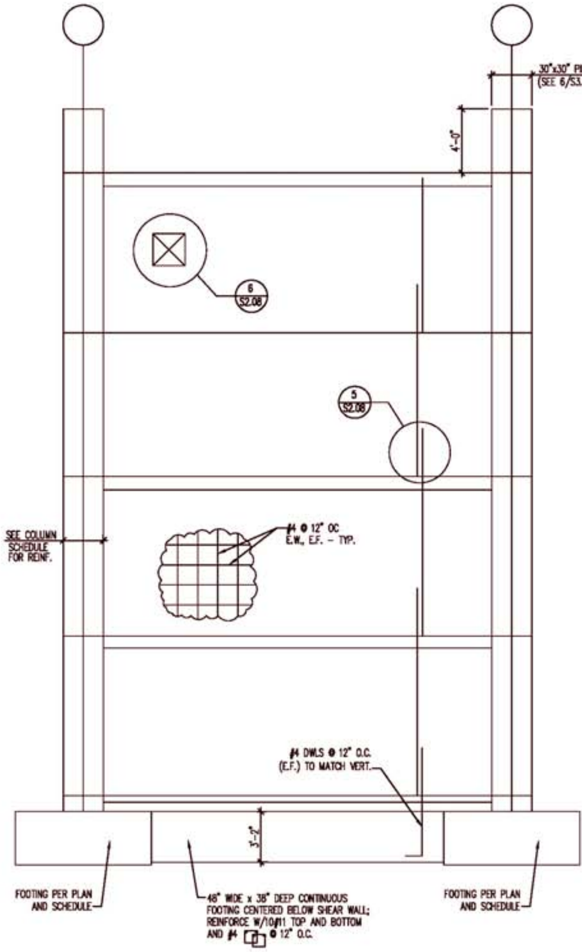
Lateral:

The office's structure is a steel framing system that features braced concentric frames in the E-W direction, and mixed eccentric and concentric frames in the N-S direction. Sallie Mae isn't in a strong seismic zone so the wind easily controls both directions.

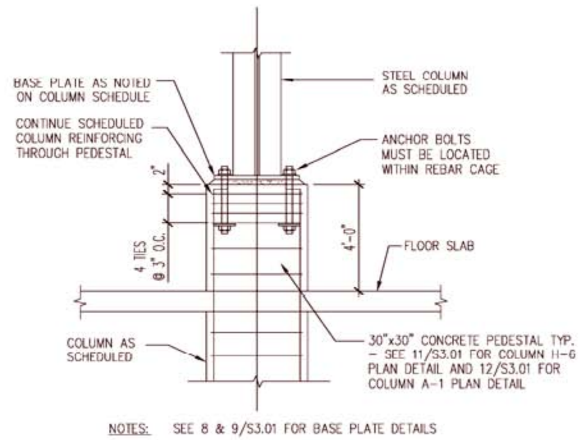
The braced frames are assumed to take 100% of the lateral load. As they progress to the ground level of the building, the braced frames line up with the parking garage shear walls. The connection between the shear walls and the braced frames are considered pinned connections and keep a continuous load path all the way down to the foundation.



Braced Frames



Resistance to overturning



Pin Connection

THESIS GOALS

- **TO CREATE A POST-TENSIONING DESIGN THAT CONSIDERS CONSTRUCTABILITY AND IS SIMPLISTIC IN NATURE**

- **INVESTIGATE THE SAVINGS IN POST-TENSIONING**

Lateral Redesign

Steel Structure

Level	Weight kips
ROOF	498.1
RF/PENT	2377.0
8TH	2224.7
7TH	2227.9
6TH	2201.5
5TH	2201.5
4TH	2201.5
3RD	2201.5
2ND	2151.0

$\sum = 18284$

Concrete Structure

Level	Weight Kips
RF	498
RF/PENT	4270
8TH	4120
7TH	4220
6TH	4150
5TH	4150
4TH	4150
3RD	4150
2ND	4050

$\sum = 33758$

- According to previous analysis wind controlled the design of the steel system by a large margin.

Load Case: E2 seis-x EQ_BOCA96/99_X_-E_F

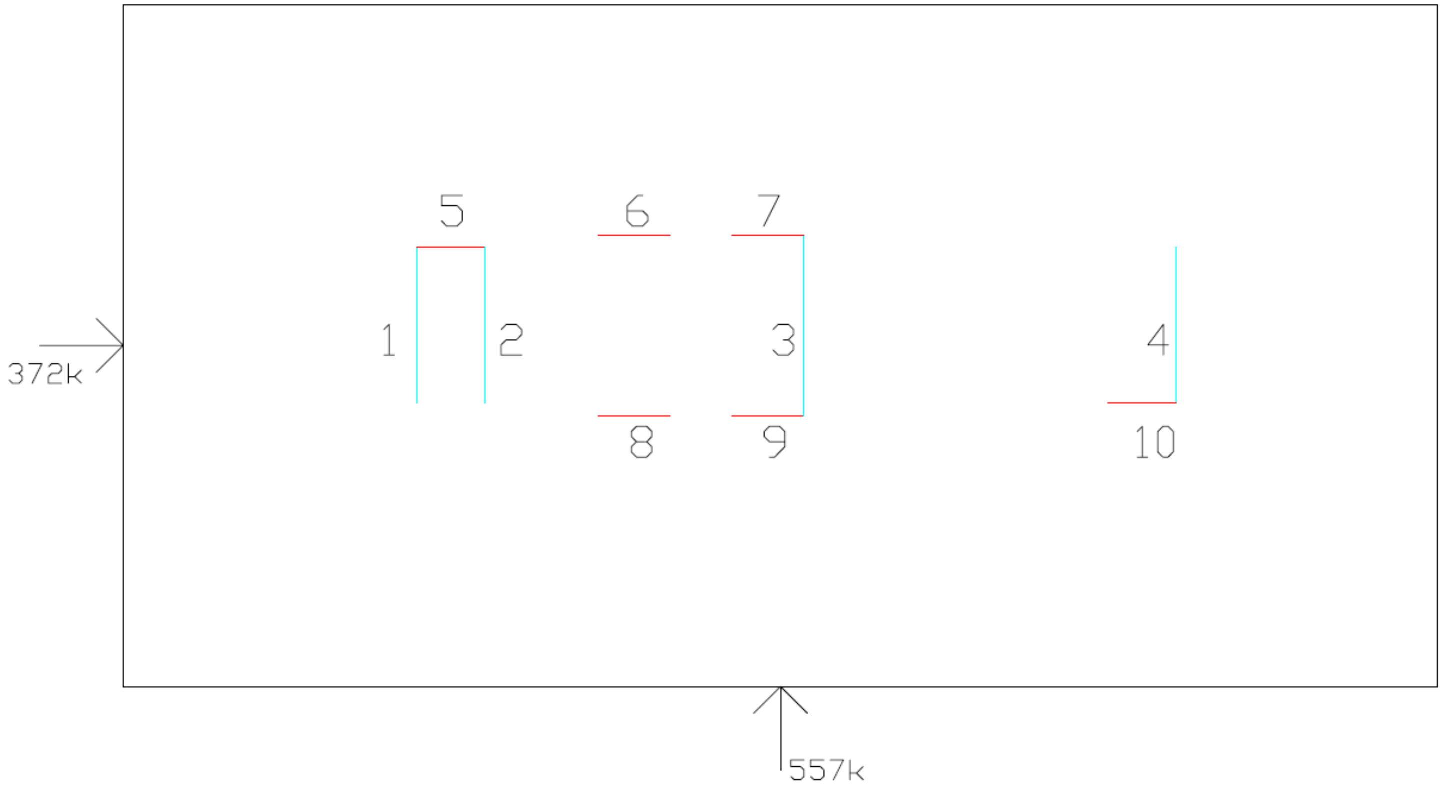
Level	Shear-X kips	Change-X kips
ROOF	10.22	10.22
RF/PENT	51.06	40.84
8TH	80.69	29.63
7TH	103.02	22.33
6TH	119.04	16.02
5TH	129.70	10.66
4TH	135.51	5.82
3RD	137.92	2.40
2ND	137.76	-0.15

Load Case: W1 wind-x Wind_BOCA96/99_X

Level	Shear-X kips	Change-X kips
ROOF	21.87	21.87
RF/PENT	88.49	66.61
8TH	172.41	83.93
7TH	249.57	77.15
6TH	321.48	71.91
5TH	389.23	67.75
4TH	451.02	61.79
3RD	507.10	56.08
2ND	556.73	49.63

137.76k vs. 556.73k

- The short direction of the building is the critical direction, and the R values of concentric braced frames and ordinary reinforced shear walls are the same. Therefore the seismic forces on the structure will increase by a factor of two due to the load. However, the wind will still control due to it being four times larger then the original seismic forces. It would be conservative in not worrying about the decrease of four feet of the building. The base shear will remain the same; however the load will distribute different among the shear walls compared with the braced frames.



Shearwall Plan

- The critical wall is 3. Shearwall 3 was checked to see if it could be properly reinforced to meet the loading criteria. Using PCA column 44 #6 bars on each wall, spaced at 7" apart will suffice. Therefore if the critical shearwall works, all the other shearwalls could be reinforced to meet there loading needs.
- These results prove that the lateral system works with 12" shearwalls at the same locations that the braced frames were located.

Critical Wall #3						
	Force at level	Height	Moment			
Roof	7.1	126	889.6			
9th	21.5	112	2409.1			
8th	27.1	98	2655.8			
7th	24.9	84	2092.4			
6th	23.2	70	1624.0			
5th	21.8	56	1220.8			
4th	20.0	42	837.9			
3rd	18.1	28	507.1			
2nd	16.0	14	224.4			
				Loads		
		Sum	12461.1	x1.6 =	19937.8	kft
				.9D =	576	k
				1.2D + L	1000	k

```
General Information:
====================
  File Name:  UNTITLED.COL
  Project:                              Code: ACI 318-89
  Column:                               Units: US in-lbs
  Engineer:                             Date: 04/03/06  Time: 11:28:17

  Run Option: Design                    Short (nonslender) column
  Run Axis:   X-axis                    Column Type: Structural

Material Properties:
====================
  f'c    = 6 ksi                             fy   = 60 ksi
  Ec     = 4695.98 ksi                       Es   = 29000 ksi
  fc     = 5.1 ksi                           erup = 0 in/in
  eu     = 0.003 in/in
  Stress Profile: Block                      Beta1 = 0.75

Geometry:
=========
  Rectangular: Width = 12 in                 Depth = 321 in

  Gross section area, Ag =  3852 in^2
  Ix =  3.30762e+007 in^4                    Xo =  0 in
  Iy =  46224 in^4                           Yo =  0 in

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

  Confinement: Tied; phi(c) = 0.7,  phi(b) = 0.9,  a = 0.8
  #3 ties with #10 bars, #4 with larger bars.

  Layout: Rectangular
  Pattern: Equal Bar Spacing  [Cover to transverse reinforcement (ties)]

  Total steel area, As = 38.72 in^2 at 1.01%

  88-#6   Cover = 1.5 in
■04/03/06 PCACOL(tm)V2.30 Proprietary Software of PORTLAND CEMENT ASSN. Page 3
11:30:32 Licensed to: ae, university park, PA


             Applied Loads   Computed Strength   Computed/
               P       Mx       P       Mx       Applied
   Pt.      (kips)   (ft-k)  (kips)   (ft-k)    Ray length
   ---------------------------------------------------------
    1         567    20000     905    32370      1.618
    2        1000    20000    1866    38058      1.903
```

Structural Depth

The parking garage was considered in my design; however, since there is real no architectural limitations on the garage, the existing system seemed to be the most economical. However, the office structure which was made of steel could have alternative designs that may be more economical, especially with the price fluctuations that occurred in the last few years with steel. Also have the parking garage and the post-tensioned floor as concrete will eliminate the need for having connections between the two; no braced frames to shearwalls; or no need for anchor bolts into concrete piers.

Through deep consideration, the one-way post-tensioned slab system seemed to be the best alternative system to steel. With this in mind I began my post-tensioning design by making a spreadsheet which could design post-tensioning for a three span continuous beam. Concrete is considered fixed at the supports, however, if it was assumed pinned at all the supports; the design would be considered conservative for the midspans of all the members. This was a good way to judge my initial beam sizes and understand the concepts of post-tensioning a little better. Before the initial beams sizes were chosen, I needed to pick a slab thickness. With the knowledge that the maximum slab span was 28' long, I performed a few hand checks, and I picked an 8" slab. While working with this spreadsheet my initial goals of the building was to lower the building as much as possible. This could be very cost efficient due to the facade savings. I analyzed all my beams in my building as T beams, and used the limits of 400psi- 600psi as good precompression limits to size my beams. I began with 18" beams, and the results were miserable. The spans of my building were causing the beams to become highly inefficient. An 18" beam needed a 48" web for the design to work. This could be done, but I didn't believe that the facade savings would make up for the extra cost of construction and material. Also I wasn't limited to a given amount of ceiling space. So I kept trying variations of size until I changed my design goals. Now I wanted the beam to have a minimum of 1 for its depth/width ratio. From this I determined that 24" deep beams may work with my design.

For construction reasons, I picked 6000psi concrete to work with and continued using lightweight concrete in my design. I also verified that RAM concept performs an estimated check on the amount of losses incurred during post-tensioning. The plan is approximately 120' by 200', and it would be easier if jacking occurred from one side of the building and occurred only in one process. The longest tendons run across the slab about two hundred feet. Do to these long spans, the estimated approach for the calculation of losses may be flawed. If more detailed analyses were performed, a few more tendons may have to be designed to consider the long lasting affect of losses which need to be considered eventually.

So the ground floor level will be a cast in place floor due to the denser column space in the floor below. Then the first elevated floor the second floor will be post-tensioned. All post-tensioned floors will be 8" slabs. And the post-tensioning running through these slabs were designed to be either the same or very similar between the floors. The best approach for doing this is to keep the bundles of unbonded tendons consistent through all the spans.

The 24" beams I previously chosen was picked with the assumption that I would be using lightweight concrete. Lightweight concrete is more fire protective, less-corrosive, and lighter in weight. However, it is also more-expensive, harder to work, and less resistant against deflections. After analyzing the problem more carefully and running a few floor calculations on RAM Concept, it was clear that lightweight concrete wasn't the way to go; mostly because of constructability reasons which I will continue discussing in my breadth analysis.

After switching back to normal concrete, the fact of repetition and consistency was a major concern in my design and led me to making the beams 26 " which allowed me to cut the width of the beams, and have less concrete area for the same strength. This could be attributed to the greater moment of inertia and to the increased drape on the post-tensioning tendons.

Regular reinforced area

The entire interior core of the building spanning from stairwell to stairwell is considered non-post-tensioned and will be designed with regular reinforcing. Do to the fact the post-tensioning is running parallel to these regular reinforced regions. Extra reinforcing needs to be specified to handle the shrinkage in the concrete that will be caused by the post-tensioning. Usually adding a wire mesh in the top and bottom of the slabs in the zone will suffice to handle all the strains that will occur between the zones, or number #4's @ 18.

Beams and Slabs

All slabs were designed as class U with the exception of a few that had to be changed to Class T due to regions of negative moment caused by conditions on the beams. Class T was verified to be allowed since my slab elements are one-way and are not-limited to the two-way design limitations as specified in the code.

The perimeter beams with the exception of one beam, are all regular reinforced 12 x 16 beams. The beams weren't needed in my design until the facade loads were added to the perimeter. The only beams that were designed for torsion were the edge beams that run east-west on the edges of the building. These beams throughout all the floors are 16 x 26 or 12 x 26 post-tensioned beams. All prestressed beams in the building with the exception of one beam were designed under Class T. The deflection was never an issue when designing under class U, so with Class T there were a minimal amount of problems that were clearly identified and adjusted. For the whole building only one critical beam was designed as Class C and special provisions, especially reinforcement for crack control need to be specified for that beam.

The most important elements of the entire structure are the main girders running east-west and supporting the entire structure. The fact that some of the girders were continuous greatly added to the strength of the design. Also the majority of the tendons were designed with bundles of 4 strands in mind so the minimum would be 8 strands in a beam, and a max would be 28 in a beam. The minimum cover at the bottom of the beams is 4" and the minimum cover at the top of the beams is 4" in interior spans, and 6" at the anchorage zones. The post-tensioning cables will be greased coated, and are the normal .153in^2 (7wire) strands.

Special Floors

The beams on the 7th floor need to be enlarged to support a file storage load. However, on the eighth floor the beams need to be maximized and the depth of a few beams need to be increased to 30" to counteract the point loads incurred from the column above. So these beams act as transfer girders. This wouldn't be much of a problem; however, the next floor is the penthouse which is host to all the mechanical and elevator equipment. The beams also need to have unique tendon profiles to keep the beams in T class design. However, one of the beams has to be designed as C class. The deflections on this floor are a major concern, and a large amount of beams need to be resized to account for the deflections. The penthouse floor which houses the mechanical equipment also had some large loads, but nothing compared with the eighth floor.

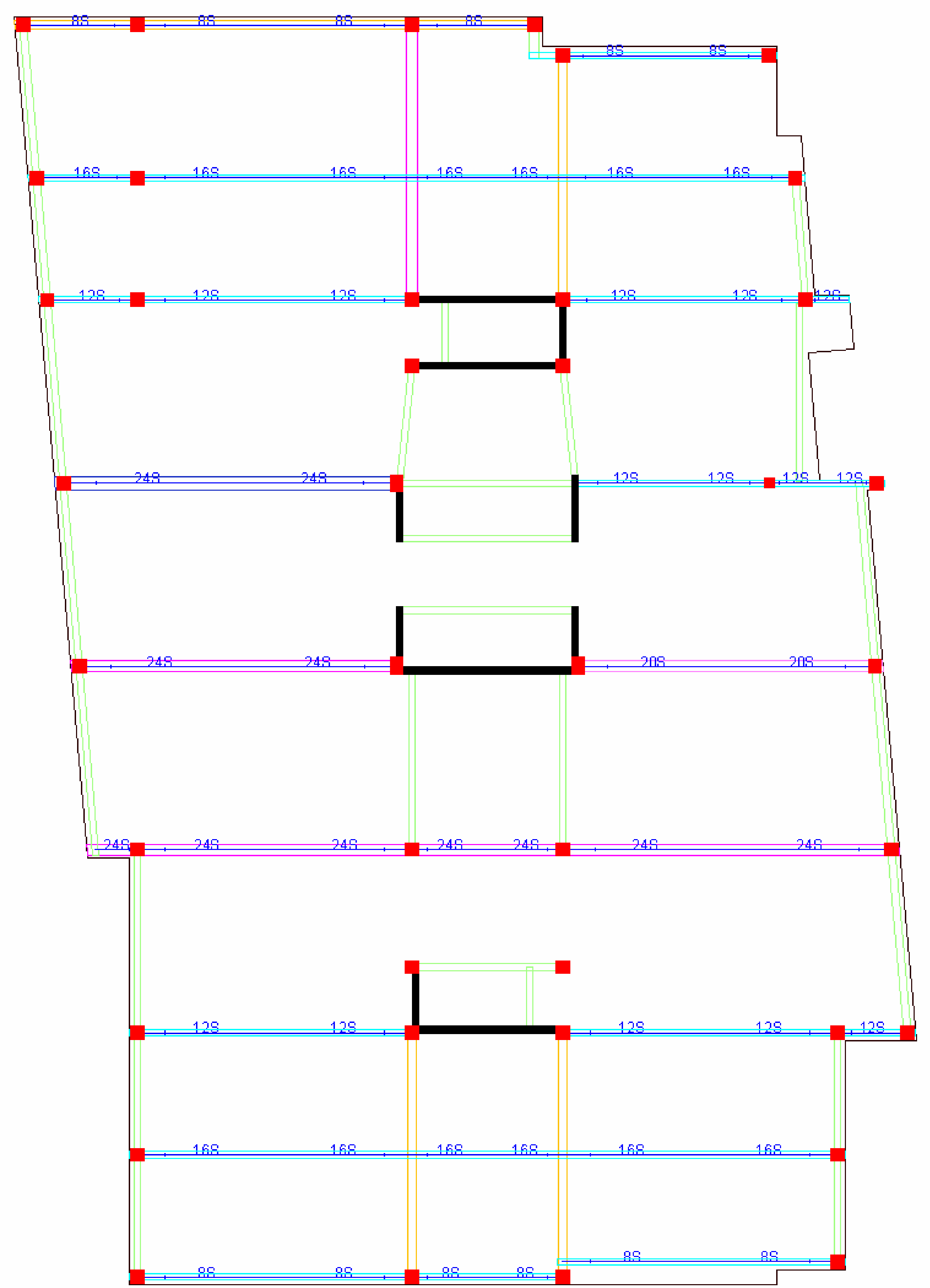
<u>Below is the layout of my typical floor, and a legend for the corresponding elements in that floor:</u>

Prestressed Beam Schedule		
Dimensions	Strands	Color
12 x 26	12 or 16	
16 x 26	16 or 20	
20 x 26	20 or 24	
24 x 26	24 or 28	
Nonprestressed		
Reg 12 x 16		
Reg 12 x 26		
Columns		
Walls		

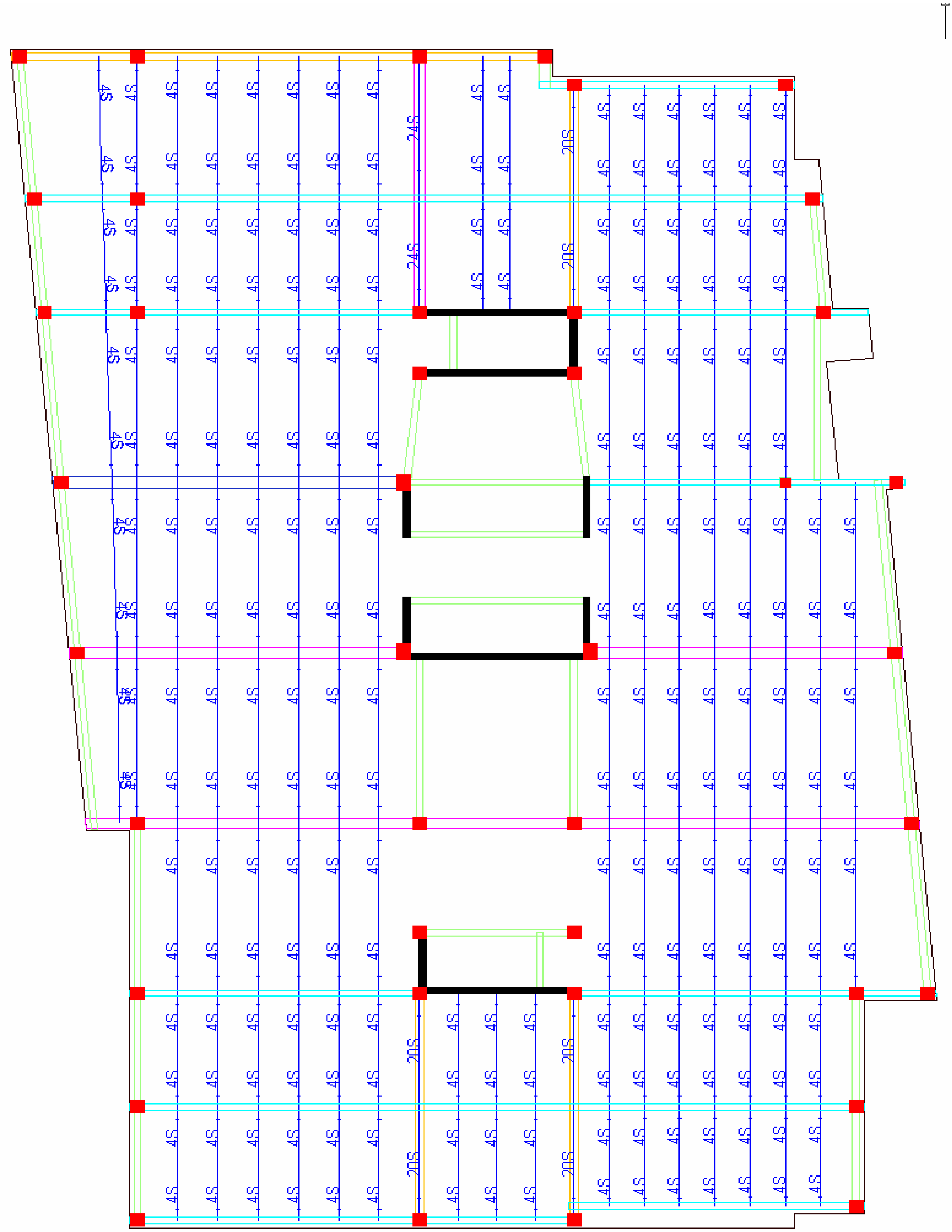
See Appendix 3 for check and Appendix 6 for sections

Column Schedule							
Types	Max Reinforcing	Factored Force		Factored X Moment		Factored Y Moment	
30 x 24	12 #8	2300	kip	200	ftkip	0	ftkip
24 x 24	10 #11	2100	kip	320	ftkip	75	ftkip
20 x 20	8 #9	700	kip	600	ftkip	75	ftkip

See Appendix 4 for check and Appendix 5 for sections



Latitude Tendon Plan



Longitudinal Tendon Plan

Construction Management Breadth

Designers need to make practical decisions regarding the design of the building. Here's a list of all the construction management considerations that went into my design:

- Type of system?
- Concrete strength?
- Normal or lightweight concrete?
- Unbonded or Bonded tendons?
- Design according to class U, T, or C?

- Impact on foundations
- Impact on building height
- Impact on schedule

The building layout consists of rectangular bays with the long direction spanning 42' - 55' and the short direction varying between 18' and 28'. Post-tensioning at this time was the system that I was going to design Sallie Mae with, and from previous analysis a two-way system was impractical. It would take a considerable of concrete, probably up to 14" thick to the span the slab up to 55'. The only solution was to use a one-way post-tensioned slab system.

To keep the amount of dead load minimum in the slabs I pushed for an 8" slab. From simple analysis, I came with the conclusion that this only would be efficient with concrete over 5000psi. Furthermore, the columns and beams would be much more efficient with a higher strength concrete; concrete columns in a 9 story building get large fast, and the beams have large spans and therefore need a higher modulus of elasticity to help with deflection. Looking at the prices of concrete in today's market, 6000psi seemed to be the best strength for the amount it was going for, 109LB/CY.

After determining the strength, I used lightweight concrete in my design. My perception was that the savings from the cost of a lighter structure on both the foundations and the floor system itself would make up for the extra cost imposed by lightweight concrete. However, lightweight concrete is very difficult to work with for large projects, and also is very hard to pump to elevated slabs. Also problems with the rehydrating of the concrete wouldn't really go well with the problems that are just caused by post-tensioning itself. Furthermore, even though lightweight concrete helps because of its decreased dead weight, it has problems because of its low modulus of elasticity. In post-tensioning you wouldn't think that would be a problem, but it is. Because of my 28' spans, even with the lightweight concrete I need at least a 26kip force per foot of slab. Also, the tendon needed its maximum drape. However, because the lightweight concrete is not very strong in deflection, the initial camber caused by the prestressing was too much is some places. This certainly was fixable with adjustments of the tendons, but it did cause some problems. Also, it caused the design to be flawed on basis of consistency; meaning I couldn't keep the same tendon profile through continuous spans that varied in length. Therefore, because of those factors I switched back to normal weight concrete.

Another decision that had to be made was between bonded or unbonded tendons. Bonded tendons work out because they can be considered as minimum bonded reinforcement as well as main flexural reinforcement. The downside of the bonded tendons is that it requires extra material in the form of ductwork and grout, and also the layout process is longer because of the grouting. On the other hand, unbonded tendons can me placed individually or grouped in bundles rather quickly because they don't need to be bonded with the concrete. The problem with unbonded tendons is that the slab and beams still need a large amount of bonded reinforcement throughout the structure, and this adds some cost to the structure. Also, unbonded tendons are naturally more dangerous to work with, and also cause problems cutting openings later in the life of a structure.

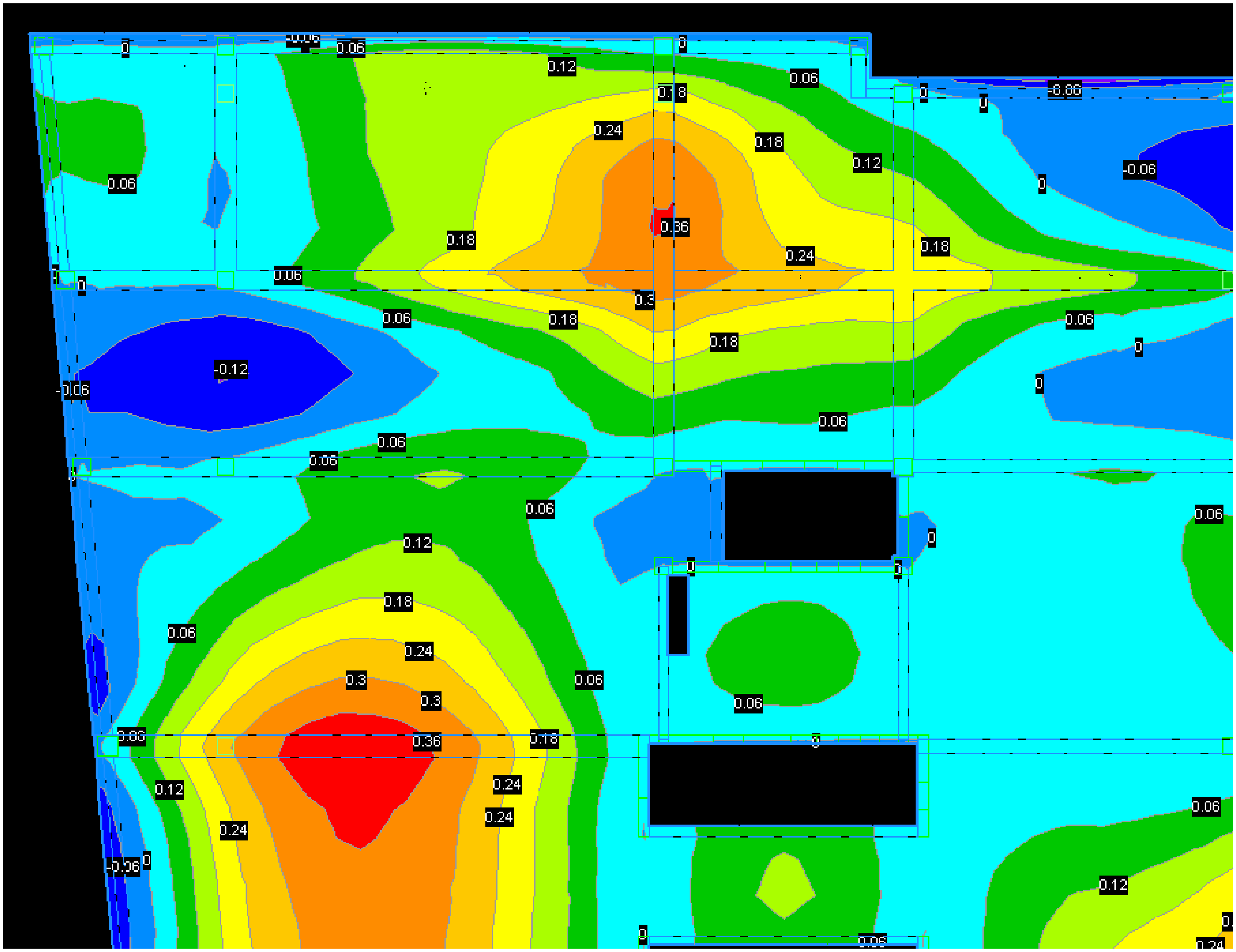
The last decision that must be made before designing of post-tensioning is what class you want to design it for. The classes are according to the ACI code 18.3.3 which states:

(a) Class U: $f_t \leq 7.5\sqrt{f_c'}$

(b) Class T: $7.5\sqrt{f_c'} < f_t \leq 12\sqrt{f_c'}$

(c) Class C: $\geq 12\sqrt{f_c'}$

Class U allows the gross section to be used in both the calculation of the service load stresses and in the calculation of deflection. Class T which allows a 60% increase in your maximum tension stress at serviceability requires that you calculate the deflection on a cracked section, bilinear process. Class C is a fully cracked section, and requires additional crack control, and side skin reinforcement, which in the construction process can be a real congestion problem and should be avoided. Before computers you probably would have to consider and pick what section you want to design for before you start, and go through a highly iterative process. However, with programs such as RAM Concept, which I used, it would seem reasonable to design all your beam sections as T class. In this regard you can minimize the post-tensioning and also the beam size. The slab should stay as U class because of its smaller section. Then the deflections can be examined and you can determine if T class was a good assumption. The graph below is a sustained service load deflection plan for my 8th floor of Sallie Mae. As you can barely see, the deflection in the red zones reaches a maximum at .36" which is acceptable for those spans.



My design is about 200% heavier then the steel design. The foundation underneath the parking garage makes up about 1/5th of the total footprint. It was assumed then that the foundation underneath the office building makes up about 2/5th of the total foundation cost. The footings were checked and needed to be redesigned to 150% the existing square footage size. This is the same as saying the foundations are 150% more expensive. So 2/5th the total foundation cost multiplied by the 150% increase came out to be around $188,000.

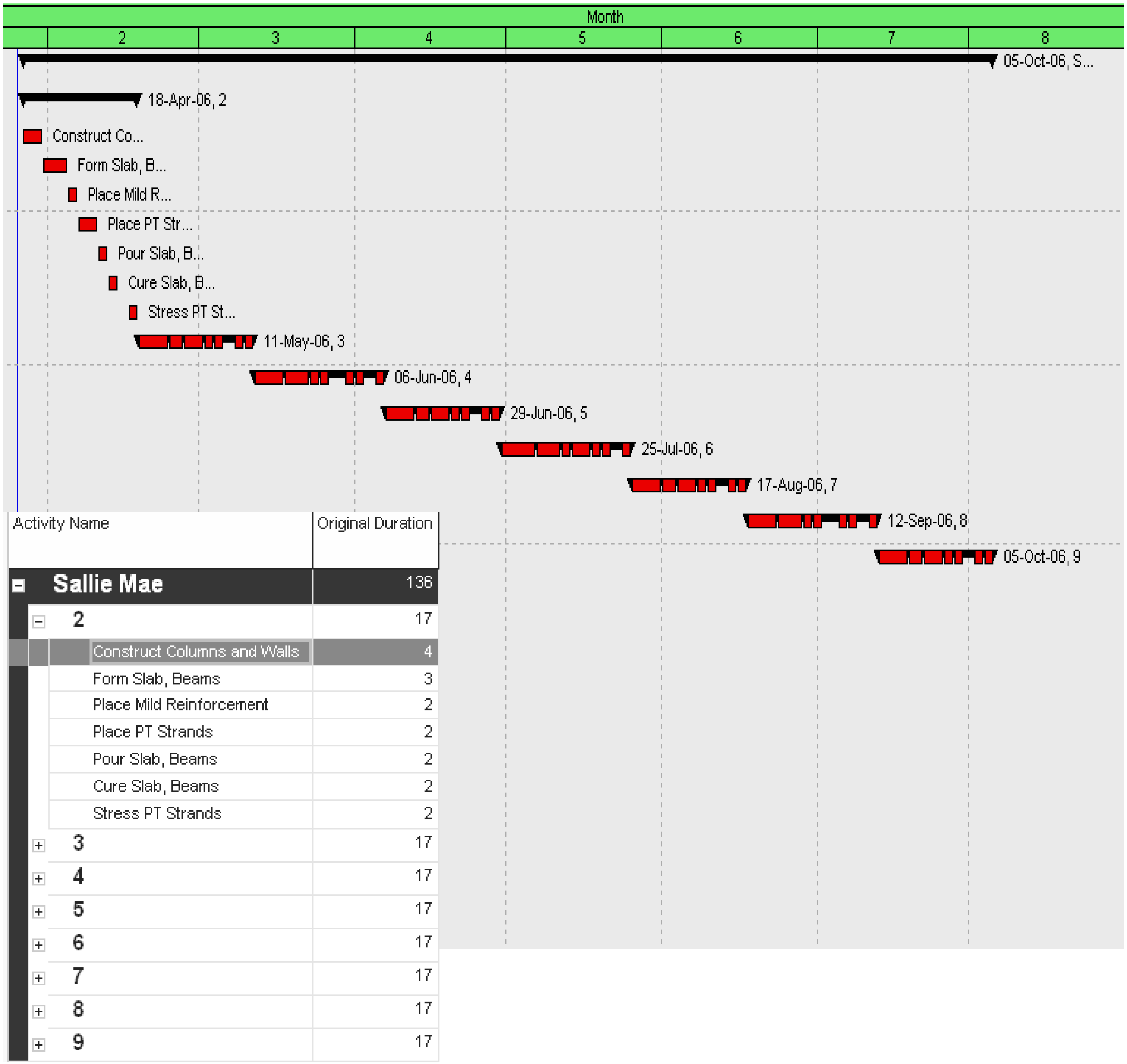
Due to the post-tensioning design the total building height was decreased by 4'. The approximate cost of the original facade was 5 million. Which entails that 4' of facade is $152,000 savings.

- The entire cost estimate is as follows:

Floors	$1,013,185		
Columns	$481,644		
Walls	$264,823		
Formwork	$946,248		
General Conditions Increase	$264,000		
Foundation Increase	$188,400		
Fascade Savings	-$152,672		
Total	$3,005,629		
	Cost Index	0.91	
		Total w / Index	$2,735,122
	SF analyzed	166,000	
Price/SF	$16.48		

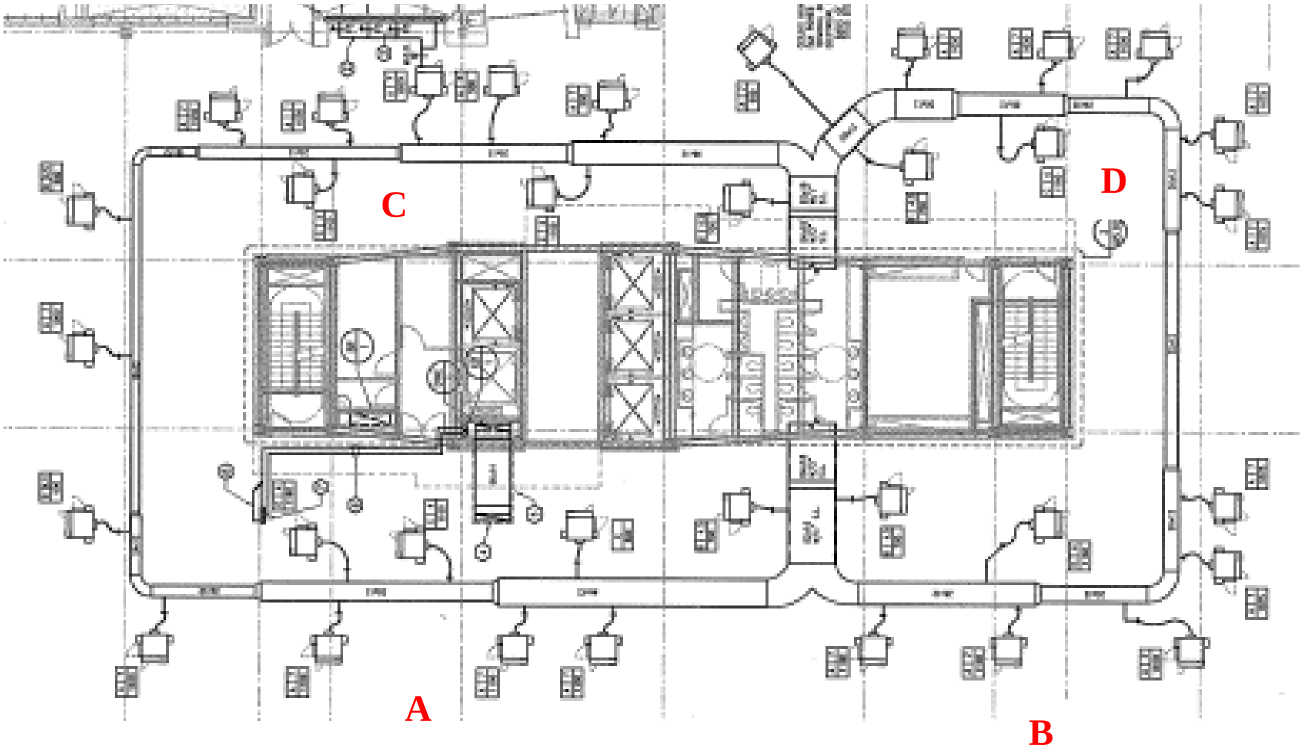
PT Concrete	Steel
$16.48/SF	$17.79/SF

According to the cost data in Appendix 1, I was able to formulate a time schedule for Sallie Mae. Each floor is very similar so the schedule remains consistent within all the floors. Furthermore, the schedule does reflect the belief that my post-tensioning design will enable them to layout the rebar and PT strands a day quicker for every floor. The schedule also reflects that the subs are familiar with concrete and therefore are very efficient at their work. The overall duration of the project is 136 days for the 8 post-tensioned floors. This in turn, since I'm using a 5 day workweek, comes out to be around 8.5 months. The steel erection was much quicker by a few months; however, that doesn't include the fact that they need lead-in-time for that project. Also, Sallie Mae doesn't have any tenant space and therefore doesn't lose any money from possible rent. Therefore the concrete design seems to be slightly cheaper in the end.


Month
2
3
4
5
6
7
8
05-Oct-06, S...
18-Apr-06, 2
Construct Co...
Form Slab, B...
Place Mild R...
Place PT Str...
Pour Slab, B...
Cure Slab, B...
Stress PT St...
11-May-06, 3
06-Jun-06, 4
29-Jun-06, 5
25-Jul-06, 6
17-Aug-06, 7
12-Sep-06, 8
05-Oct-06, 9
Activity Name
Original Duration
Sallie Mae 136
2 17
Construct Columns and Walls 4
Form Slab, Beams 3
Place Mild Reinforcement 2
Place PT Strands 2
Pour Slab, Beams 2
Cure Slab, Beams 2
Stress PT Strands 2
3 17
4 17
5 17
6 17
7 17
8 17
9 17

Mechanical Breadth

The new structure of Sallie Mae allows for a new floor thickness due to the post-tensioning system. The ductwork can be resized to a more efficient size, because of its current low aspect ratio.



Each of the four branches were calculated for all there characteristics of equivalent length, and equivalent diameter. Then each piece of every duct was resized with the same equivalent diameter. The total pressure drop for each new duct stayed reasonably close to that of the old duct. But most importantly the velocities of the new ducts are consistent with that of the old ones. Then with the new ducts known, a cost estimate was done for the 8th floor system. Each of the eight floors is similar so it is reasonably to times the savings by floor over 8 floors. The cost analysis is as such:

	Existing System	New System
Lbs/floor	4419	3119
Cost/Lb	4	4
Cost/floor	$17,366	$12,258
Total	$138,930	$98,061

Schedule Difference =	4 days
Difference =	$40,869

- The savings per floor is 5,000 so the overall savings is 40,000. The schedule also decreases by 4 days.

Summary and Conclusions

Comparison between post-tensioning and steel

The Lead-in-time is a big difference in potential cost between post-tensioned concrete and steel systems. The factor that makes the lead-in-time so important is the potential rent that the owner can get from a quicker erected building. Concrete only requires the actual design drawings, the rebar, concrete, and PT strands. On the other hand, steel needs to be called for in advance for fabrication, which can add up to a few months. This may not be a problem if foundations take a considerable long time. Also transportation of large members can be a problem, and the possibility of a mix up in the sizes is always possible.

Constructability is another concern between the two systems

There's know doubt that for my building steel is erected faster then post-tensioned concrete. This is because of the simplicity of the floor grid. This structure is not irregular, so post-tensioning doesn't have its advantage in that area. Steel can be brought on to site, dumped and arranged, and easily erected with a crane. Also, multiple floors can have construction going on them at once, and this allows for other trades to be brought in faster.

With all concrete construction, you can't pour one floor and start pouring the next right away. Time needs to be allowed for the concrete to cure and gain sufficient strength before the columns and the next floor can be poured. This delays the process, by around two months when it comes down to it. Post-tensioning also takes more time to layout, and may be slightly complicated, and possible delays can occur if the construction drawings aren't very detailed; errors in congested areas aren't uncommon.

Safety is another major concern

The tendons need to be laid out properly because if the radius of curvature of the tendons is too small then a chunk of concrete can be blasted out of the slab and possibly kill someone. Also while jacking the tendons, a special inspector needs to be on site, and usually the structural engineer should be on site for the first jack, because sometimes things can get really screwed up and may require a proper judgment call. These safety hazards may require the contractors to implement a higher insurance policy for their companies due to these extra risks.

Foundations

The concrete building has amount 200% the amount of dead load compared to the steel building. The office building was modified with the 200% of dead load, while the parking wasn't. A good example would be the column with the heaviest unfactored load of 1700k at the 2^{nd} floor level, then with the lobby and parking garage added to the 1700k, the total load at the foundation is 2060k. On the other hand the steel building had a load of 1000k in the office building plus the lobby and parking garage is 1360k. This is a 2060k/1360k = 150% of the original load. The original footing = 1360000/20000 = 68 SF needed = 8.5 x 8.5 footing. Now the footing needs to be 2060000Lb/20000psf (capacity of the soil) = 103SF = 10.5 x 10.5. The foundation cost underneath the office structure will therefore approximately increase by 150%.

The columns in the parking garage will also see about 150% percent of the factored load. The original parking garage design was conservative because the oversized columns at the lobby level needed to handle the base plate connection of the steel column. Since the base plate needed abdicable bearing and the anchorage bolts need to be within the stirrup caging, the column needed to be oversized. The columns in the parking garage were checked with the new heavier structure imposed on it (see Appendix 4 for PCA calculations). The column with the maximum load was satisfactory as a 30"x30" column with abdicable reinforcing. So the parking garage can remain the same with the new loads opposed on the columns.

Gravity System

The gravity systems for both steel and post-tensioned concrete are very efficient. The steel framing makes use of the rectangular bays, and the cambered ability of steel allows it to counteract deflection. Furthermore, the steel framing layout is very light, and requires not much material compared to concrete.

Concrete on the other hand requires twice the amount of material then steel, but concrete does cost less. The system is very heavy, and is harder to build, but the main savings comes in the form of just a few feet of building height. Furthermore, the cost analysis revealed that the concrete system may be slightly less, and to add to that this didn't compare prices that were equal in time. So the steel estimate should be increased by a factor. Overall, both systems seemed very efficient.

Lateral System

The shearwalls seem very inefficient in resisting the lateral load. The braced frames are lighter, and can easily be bridged over openings with eccentric bracing. This is more difficult to do with shearwalls and is a waste of extra capacity. Also the shearwalls are heavier, and due to the amount of reinforcing needed, they are more expensive then the braced frames. Overall, the shearwalls work, but aren't as effective as the braced frames.

MEP

The mechanical is affected by the alteration of the building by use of post-tensioned concrete. Each floor can be reduced by 6" because of the new system. The mechanical ductwork for the average floor was redesigned using the new plenum depth. The purpose of this was to see how much savings can occur from the more efficient use of sheet metal. The velocities for the new ductwork were held around that of the older ductwork during the redesign. The sheet metal savings reached about 40,000 dollars for the entire building.

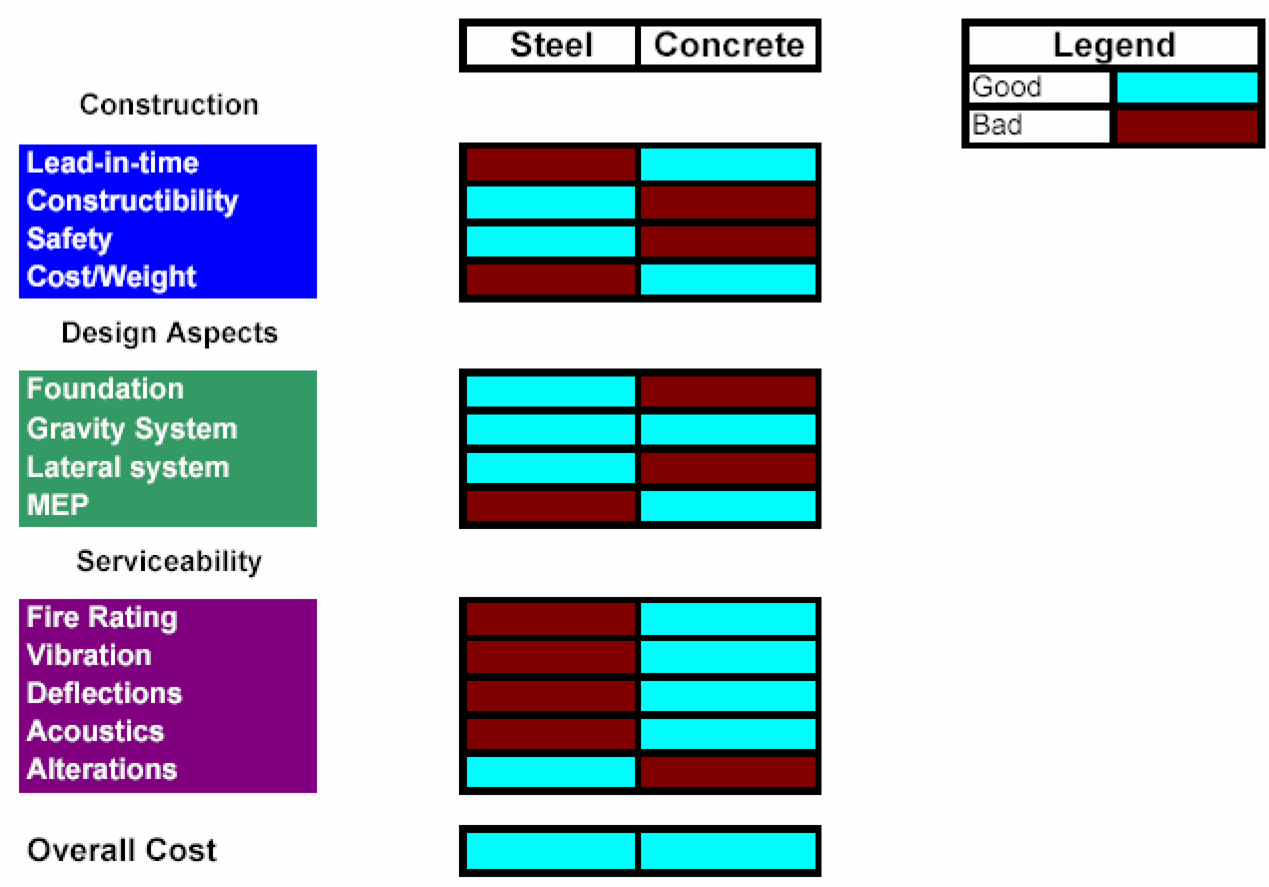
Serviceability

Concrete is naturally resistive to fire, and doesn't require fireproofing. Also do to the heavier weight of the structure concrete performs better against vibration, and it is also very resistive to noise transmission. Furthermore, the application of post-tensioned concrete is highly efficient in counteracting deflections. The only problem with using post-tensioning concrete is that openings in the slab are very dangerous to implement later in the life of the structure

Overall

The steel process seems to be faster for erection; however, the prefabrication process is much more of a problem. Also, this project was planned out before the big boom in steel prices, so post-tensioning which was considered before seems to be a more worthy a claim to investigate.

Overall the final cost of the building is cheaper with the post-tensioning system, but it takes longer to schedule. The longer the project is to schedule, the higher possibility that renting cost can come into play. But in Sallie Mae's case there are no tenants, so rent isn't a factor.


Steel
Concrete
Legend
Good
Bad
Construction
Lead-in-time
Constructibility
Safety
Cost/Weight
Design Aspects
Foundation
Gravity System
Lateral system
MEP
Serviceability
Fire Rating
Vibration
Deflections
Acoustics
Alterations
Overall Cost

Acknowledgements

Special Thanks:

Cagley and Associates

- Frank Malits

- Structural Mentors

AE Faculty

- Professor Boothby
- Professor Parfitt
- Professor Hanagan

Fellow Classmates

- Bryan Quinn
- Lourdes Diaz

- Family and Friends

Appendix 1 (CM Breadth Study)

	Crew	Daily Output	Units	Materials	Labor	Equipment	Total
Floors							
Concrete ready mix 6000 psi			CY	109 610926			109 555388
Early strength additive				0 61093			61093
6" to 10" pumped	C-20	160	CY		12 60634	5 23693	17 84327
Screed, float, hand trowel	1 Cefi	600sf		0 0	0 79401	0 0	0 79401
			Total	**672019**	**60634**	**23693**	**$780,208**
Floor Formwork							
4 use formwork /girders	C-2	395	SFCA	1 33486	4 144992	0 0	5 178479
2 use formwork /slabs	C-2	520	SFCA	2 327269	3 440501	0 0	6 767769
			Total	360755	585493	0	**$946,248**
Reinforcing for floors							
Stressing Tendons	C-4	1500	LB	0 61786	1 111741	0 2629	1 176156
Rebar	4 Rodm	2	Ton	835 134762	550 88765	0 0	1385 223527
Unloading & sorting	C-5	100	Ton	0 0	22 3470	7 1049	28 4519
Crane cost for handling /Average	C-5	92	Ton	0 0	24 3793	7 1138	31 4931
			Total	**134762**	**96028**	**2187**	**$232,977**

Columns							
20" x 20" Average	C-14A	15	CY	390 119262	483 134274	49 13692	922 **256247**
24" x 24" Average	C-14A	18	CY	370 113116	400 111170	41 11395	811 **225398**
			Total	232378	245444	25086	**481644**
Walls							
14' high and 12" thick	C-14D	40	CY	159	176	18	353
			Total	**131192**	**132018**	**13539**	**$264,823**

General Conditions

$88,000 per month

months added to the schedule compared with steel	3	**Total**	**$264,000**

Foundations

Total	**$188,400**

Fascade savings

Original Cost of fascade		4750000
	Sav/Ft	38168
Total		**$152,672**

Totals

Analysis was for 8 floors of my building

SF analyzed	166,000 SF

Final Cost **$2,762,098**

Concrete	Steel
16.64 $/SF	**17.79$/SF**

Difference

$191,042

Appendix 2 (Mechanical Breadth)

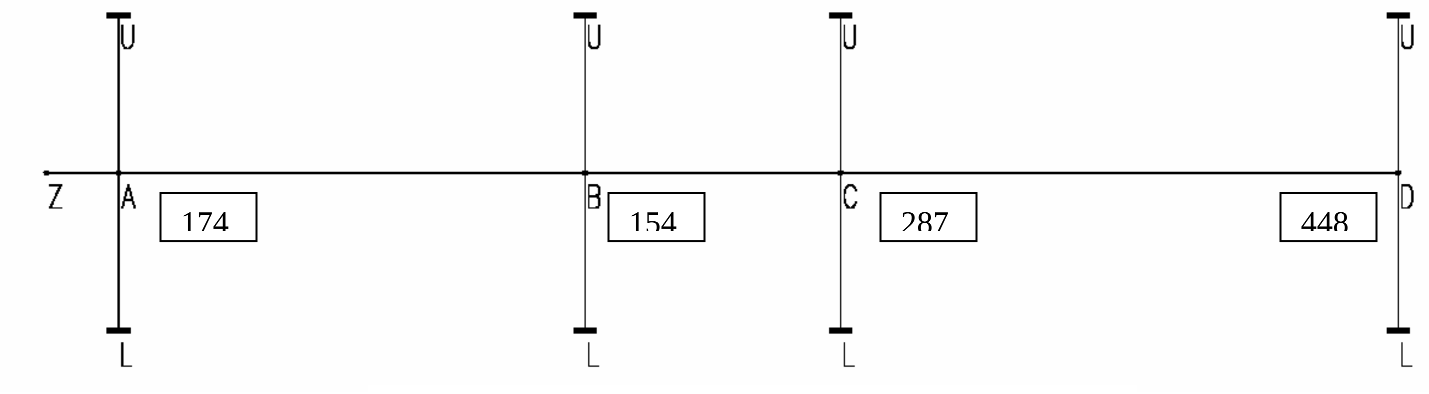
Existing Mechanical Ducts

	L (ft)	Size	Eq. Diameter	CFM	Velocity	Perimeter	Gauge	Sum 2-S	Lb/ft	Lbs
	22	18x12	16	1000	713	60	24	30	7	143
	10	24x12	18	1600	911	72	24	36	8	78
A	13	24x12	18	2700	1523	72	24	36	8	101
	12	30x12	20	2700	1233	84	24	42	9	108
	20	30x12	20	4960	2273	84	24	42	9	180
	37	44x12	24	9120	2900	112	22	56	14	518
	16	22x12	17.5	1000	600	68	24	34	7	117
	12	26x12	19	2075	1060	76	24	38	8	98
B	5	26x12	19	3150	1600	76	24	38	8	41
	12	26x12	19	4250	2350	76	24	38	8	98
	21	36x12	22	6510	2450	96	22	48	12	252
	4	36x12	22	7610	2900	96	22	48	12	48
	11	82x16	36.5	18130	2300	196	20	98	28	312
	8	84x18	40	18130	2100	204	20	102	30	237
	19	18x12	16	1000	730	60	24	30	7	124
	15	18x12	16	1600	1170	60	24	30	7	98
	7	22x12	17.5	1600	975	68	24	34	7	51
	15	22x12	17.5	2650	1600	68	24	34	7	110
C	7	22x12	17.5	4860	2950	68	24	34	7	51
	13	28x12	19.5	5910	2800	80	24	40	9	112
	11	28x12	19.5	6810	3250	80	24	40	9	95
	28	38x12	22.5	8870	3200	100	22	50	13	350
	16	22x12	17.5	1000	600	68	24	34	7	117
	12	26x12	19	2075	1060	76	24	38	8	98
	5	26x12	19	3150	1600	76	24	38	8	41
D	8	26x12	19	4200	2300	76	24	38	8	66
	15	36x12	22	6410	2450	96	22	48	12	180
	12	50x12	25	7460	2160	124	22	62	16	186
	2	50x12	25	9060	2460	124	22	62	16	31
	5	82x16	36.5	18630	2500	196	20	98	28	142
	8	84x18	40	18630	2150	204	20	102	30	237

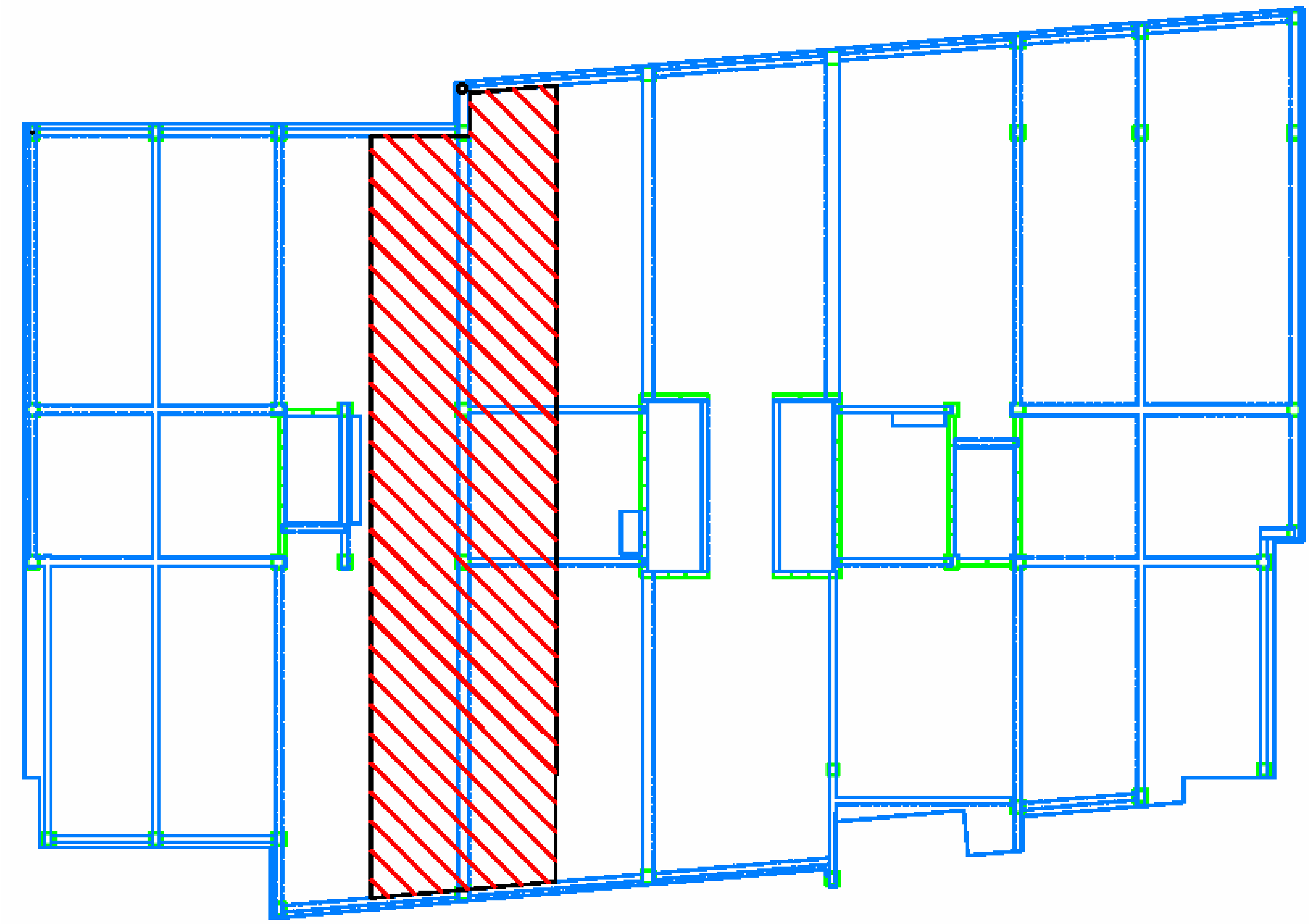
New Mechanical Ducts

	L (ft)	Size	Eq. Diameter	CFM	Velocity	Perimeter	Gage	Sum 2S	Lb/ft	Lbs
A	22	14x14	16	1000	732	56	26	28	4.7	103.4
	10	16x16	18	1600	906	64	26	32	5.3	53.0
	13	16x16	18	2700	1514	64	26	32	5.3	68.9
	12	18x18	20	2700	1196	72	26	36	6	72.0
	20	18x18	20	4960	2204	72	26	36	6	120.0
	37	26x18	24	9120	2803	88	24	44	9.5	351.5
B										
	16	16x16	17.5	1000	560	64	26	32	5.3	84.8
	12	16x16	19	2075	1168	64	26	32	5.3	63.6
	5	16x16	19	3150	1776	64	26	32	5.3	26.5
	12	16x16	19	4250	2396	64	26	32	5.3	63.6
	21	22x18	22	6510	2365	80	24	40	8.6	180.6
	4	22x18	22	7610	2766	80	24	40	8.6	34.4
	11	54x22	36.5	18130	2199	152	22	76	19	209.0
	8	58x24	40	18130	1876	164	20	82	23.8	190.4
	19	14x14	16	1000	730	56	26	28	4.7	89.3
C	15	14x14	16	1600	1183	56	26	28	4.7	70.5
	7	16x16	17.5	1600	906	64	26	32	5.3	37.1
	15	16x16	17.5	2650	1490	64	26	32	5.3	79.5
	7	16x16	17.5	4860	2729	64	26	32	5.3	37.1
	13	18x18	19.5	5910	2627	72	26	36	6	78.0
	11	18x18	19.5	6810	3023	72	26	36	6	66.0
	28	24x18	22.5	8870	2959	84	24	42	9	252.0
D	16	16x16	17.5	1000	560	64	26	32	5.3	84.8
	12	18x18	19	2075	923	72	26	36	6	72.0
	5	18x18	19	3150	1403	72	26	36	6	30.0
	8	18x18	19	4200	1865	72	26	36	6	48.0
	15	22x18	22	6410	2327	80	24	40	8.6	129.0
	12	28x18	25	7460	2131	92	24	46	9.9	118.8
	2	28x18	25	9060	2591	92	24	46	9.9	19.8
	5	54x22	36.5	18630	2258	152	22	76	19	95
	8	58x24	40	18630	1927	164	20	82	23.8	190.4

-Hand Check and compare column moments with Ram Concept



Ram Concept Moments at the lower column joint



Distributed Loads				
	ZA	AB	BC	CD
Length	6.5	42	23	50.2
SIDL	0.49	0.98	0.98	0.98
DL	1.94	3.34	3.34	3.34
LL	1.12	2.24	2.24	2.24
Balanced	0.000	3.880	0.24	2.89

Due to anchorages	
FEM added to balanced ZA	268
FEM added to balanced CD	-161

FEM chart				
	ZA	AB	BC	CD
SIDL	1.73	144.06	43.20	205.80
DL	109.27	491.27	147.33	701.83
LL	3.94	329.28	98.75	470.41
Balanced	268.00	-570.36	-10.71	-767.91

	A				B				C				D		
	AZ	AU	AL	AB	BA	BU	BL	BC	CB	CU	CL	CD	DC	DU	DL
I	29293	13333	27648	59925	59925	13333	27648	49488	49488	13333	27648	62689	62689	27648	27648
L	6.5	14	14	42	42	14	14	23	23	14	14	50.2	50.2	14	14
K	0	952	1975	1427	1427	952	1975	2152	2152	952	1975	1249	1249	1975	1975
DF	0.000	0.219	0.454	0.328	0.219	0.146	0.304	0.331	0.340	0.151	0.312	0.197	0.240	0.380	0.380
FEM	76.813			-635.334	635.334			-190.528	190.528			-907.634	907.634		
												-109.016	-218.032	-344.801	-344.801
	0.000	122.166	253.330	183.025	91.512			140.457	280.914	124.337	257.832	163.038	81.519		
				-74.213	-148.427	-99.073	-205.442	-223.834	-111.917			-9.791	-19.583	-30.968	-30.968
	0.000	16.233	33.661	24.319	12.160			20.693	41.386	18.318	37.985	24.020	12.010		
				-3.603	-7.205	-4.809	-9.973	-10.866	-5.433			-1.442	-2.885	-4.562	-4.562
	0.000	0.788	1.634	1.181	0.590			1.169	2.338	1.035	2.146	1.357	0.678		
				-0.193	-0.386	-0.258	-0.534	-0.582	-0.291			-0.081	-0.163	-0.258	-0.258
	0.000	0.042	0.087	0.063					0.127	0.056	0.116	0.073			
Dead	76.81	139.23	288.71	-504.75	583.58	-104.14	-215.95	-263.49	397.65	143.75	298.08	-839.48	761.18	-380.59	-380.59

	A				B				C				D		
	AZ	AU	AL	AB	BA	BU	BL	BC	CB	CU	CL	CD	DC	DU	DL
I	29293	13333	27648	59925	59925	13333	27648	49488	49488	13333	27648	62689	62689	27648	27648
L	6.5	14	14	42	42	14	14	23	23	14	14	50.2	50.2	14	14
K	0	952	1975	1427	1427	952	1975	2152	2152	952	1975	1249	1249	1975	1975
DF	0.000	0.219	0.454	0.328	0.219	0.146	0.304	0.331	0.340	0.151	0.312	0.197	0.240	0.380	0.380
FEM	3.943			-329.280	329.280										
												0.000	0.000	0.000	0.000
	0.000	71.161	147.564	106.611	53.306			0.000	0.000	0.000	0.000	0.000	0.000		
				-41.953	-83.907	-56.006	-116.138	-126.535	-63.267			0.000	0.000	0.000	0.000
	0.000	9.177	19.029	13.748	6.874			10.757	21.513	9.522	19.746	12.486	6.243		
				-1.933	-3.867	-2.581	-5.352	-5.831	-2.916			-0.750	-1.500	-2.372	-2.372
	0.000	0.423	0.877	0.634	0.317			0.623	1.246	0.552	1.144	0.723	0.362		
				-0.103	-0.206	-0.138	-0.285	-0.311	-0.155			-0.043	-0.087	-0.137	-0.137
	0.000	0.023	0.047	0.034					0.068	0.030	0.062	0.039			
Live AB	3.94	80.78	167.52	-252.24	301.80	-58.72	-121.78	-121.30	-43.51	10.10	20.95	12.46	5.02	-2.51	-2.51

	A				B				C				D		
	AZ	AU	AL	AB	BA	BU	BL	BC	CB	CU	CL	CD	DC	DU	DL
I	29293	13333	27648	59925	59925	13333	27648	49488	49488	13333	27648	62689	62689	27648	27648
L	6.5	14	14	42	42	14	14	23	23	14	14	50.2	50.2	14	14
K	0	952	1975	1427	1427	952	1975	2152	2152	952	1975	1249	1249	1975	1975
DF	0.000	0.219	0.454	0.328	0.219	0.146	0.304	0.331	0.340	0.151	0.312	0.197	0.240	0.380	0.380
FEM								-98.747	98.747						
												0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000			-16.789	-33.578	-14.862	-30.819	-19.488	-9.744		
				12.669	25.339	16.913	35.072	38.212	19.106			1.170	2.341	3.702	3.702
	0.000	-2.771	-5.746	-4.152	-2.076			-3.447	-6.895	-3.052	-6.328	-4.002	-2.001		
				0.606	1.211	0.809	1.677	1.827	0.913			0.240	0.481	0.760	0.760
	0.000	-0.132	-0.275	-0.198	-0.099			-0.196	-0.392	-0.174	-0.360	-0.228	-0.114		
				0.032	0.065	0.043	0.090	0.098	0.049			0.014	0.027	0.043	0.043
	0.000	-0.007	-0.015	-0.011					-0.021	-0.009	-0.020	-0.012			
Live BC	0.00	-2.91	-6.04	8.95	24.44	17.76	36.84	-79.04	77.93	-18.10	-37.53	-22.31	-9.01	4.50	4.50

	A				B				C				D		
	AZ	AU	AL	AB	BA	BU	BL	BC	CB	CU	CL	CD	DC	DU	DL
I	29293	13333	27648	59925	59925	13333	27648	49488	49488	13333	27648	62689	62689	27648	27648
L	6.5	14	14	42	42	14	14	23	23	14	14	50.2	50.2	14	14
K	0	952	1975	1427	1427	952	1975	2152	2152	952	1975	1249	1249	1975	1975
DF	0.000	0.219	0.454	0.328	0.219	0.146	0.304	0.331	0.340	0.151	0.312	0.197	0.240	0.380	0.380
FEM												-470.407	470.407		
												-56.501	-113.001	-178.703	-178.703
	0.000	0.000	0.000	0.000	0.000			89.585	179.170	79.303	164.448	103.987	51.994		
				-9.824	-19.647	-13.114	-27.194	-29.629	-14.814			-6.245	-12.490	-19.752	-19.752
	0.000	2.149	4.456	3.219	1.610			3.581	7.161	3.170	6.573	4.156	2.078		
				-0.569	-1.138	-0.760	-1.576	-1.717	-0.858			-0.250	-0.499	-0.789	-0.789
	0.000	0.124	0.258	0.187	0.093			0.188	0.377	0.167	0.346	0.219	0.109		
				-0.031	-0.062	-0.041	-0.085	-0.093	-0.047			-0.013	-0.026	-0.042	-0.042
	0.000	0.007	0.014	0.010					0.020	0.009	0.019	0.012			
Live CD	0.00	2.28	4.73	-7.01	-19.14	-13.92	-28.86	61.92	171.01	82.65	171.38	-425.04	398.57	-199.29	-199.29

	A				B				C				D		
	AZ	AU	AL	AB	BA	BU	BL	BC	CB	CU	CL	CD	DC	DU	DL
I	29293	13333	27648	59925	59925	13333	27648	49488	49488	13333	27648	62689	62689	27648	27648
L	6.5	14	14	42	42	14	14	23	23	14	14	50.2	50.2	14	14
K	0	952	1975	1427	1427	952	1975	2152	2152	952	1975	1249	1249	1975	1975
DF	0.000	0.219	0.454	0.328	0.219	0.146	0.304	0.331	0.340	0.151	0.312	0.197	0.240	0.380	0.380
FEM	268.000			570.360	-570.360			10.712	-10.712			767.910	-767.910		
												92.234	184.467	291.721	291.721
	0.000	-183.376	-380.258	-274.727	-137.363			-144.420	-288.840	-127.846	-265.107	-167.638	-83.819		
				92.269	184.538	123.176	255.425	278.291	139.146			10.068	20.135	31.842	31.842
	0.000	-20.182	-41.851	-30.236	-15.118			-25.369	-50.738	-22.458	-46.569	-29.448	-14.724		
				4.440	8.879	5.927	12.290	13.391	6.695			1.768	3.537	5.593	5.593
	0.000	-0.971	-2.014	-1.455	-0.727			-1.439	-2.878	-1.274	-2.642	-1.670	-0.835		
				0.238	0.475	0.317	0.658	0.717	0.358			0.100	0.201	0.317	0.317
	0.000	-0.052	-0.108	-0.078					-0.156	-0.069	-0.143	-0.091			
	0.000														
Balance	268.00	-204.58	-424.23	360.81	-529.68	129.42	268.37	131.88	-207.13	-151.65	-314.46	673.23	-658.95	329.47	329.47

Live Load Patterns

1		3.94	83.06	172.24	-259.25	282.65	-72.64	-150.63	-59.38	127.50	92.75	192.34	-412.59	403.59	-201.79	-201.79
2		3.94	77.87	161.48	-243.30	326.24	-40.96	-84.94	-200.34	34.42	-7.99	-16.57	-9.85	-3.99	2.00	2.00
3		0.00	-0.63	-1.31	1.94	5.30	3.85	7.98	-17.13	248.94	64.55	133.86	-447.35	389.56	-194.78	-194.78

Balanced	268.00	-204.58	-424.23	360.81	-529.68	129.42	268.37	131.88	-207.13	-151.65	-314.46	673.23	-658.95	329.47	329.47

1.2D + 1.6L +1B for Live Load Case 1															
	366.4845	95.39536	197.8168	-659.697	622.8623	-111.771	-231.774	-279.317	474.0528	169.2535	350.9728	-994.279	900.2108	-450.105	-450.105
1.2D + 1.6L +1B for Live Load Case 2															
	366.4845	87.0902	180.5948	-634.17	692.597	-61.0828	-126.664	-504.85	325.1252	8.060448	16.71456	-349.9	248.0802	-124.04	-124.04
1.2D + 1.6L +1B for Live Load Case 3															
	366.4845	87.0902	180.5948	-634.17	692.597	-61.0828	-126.664	-504.85	325.1252	8.060448	16.71456	-349.9	248.0802	-124.04	-124.04
	366.4845	95.39536	197.8168	-659.697	692.597	-111.771	-231.774	-504.85	474.0528	169.2535	350.9728	-994.279	900.2108	-450.105	-450.105
			At Column				At Column				At Column				At Column

Appendix 4 (PCA column)

General Information:
====================

File Name: UNTITLED.COL
Project:
Column:
Engineer:

Code: ACI 318-89
Units: US in-lbs
Date: 04/06/06 Time: 08:38:10

Run Option: Investigation
Run Axis: Biaxial

Short (nonslender) column
Column Type: Structural

Material Properties:
====================

f'c = 6 ksi
Ec = 4695.98 ksi
fc = 5.1 ksi
eu = 0.003 in/in
Stress Profile: Block

fy = 60 ksi
Es = 29000 ksi
erup = 0 in/in
Beta1 = 0.75

Geometry:
=========

Rectangular: Width = 24 in
Depth = 24 in

Gross section area, Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

Reinforcement:
==============

Rebar Database: ASTM

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

Confinement: Tied; phi(c) = 0.7, phi(b) = 0.9, a = 0.8
#3 ties with #10 bars, #4 with larger bars.

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

Total steel area, As = 15.60 in^2 at 2.71%

10-#11 Cover = 1.5 in

04/06/06 PCACOL(tm)V2.30 Proprietary Software of PORTLAND CEMENT ASSN. Page 3
8:41:27 Licensed to: ae, university park, PA

	Applied Loads			Computed Strength			Computed/
	P	Mx	My	P	Mx	My	Applied
Pt.	(kips)	(ft-k)	(ft-k)	(kips)	(ft-k)	(ft-k)	Ray length
1	2100	300	75	2125	314	79	1.014

Program completed as requested!

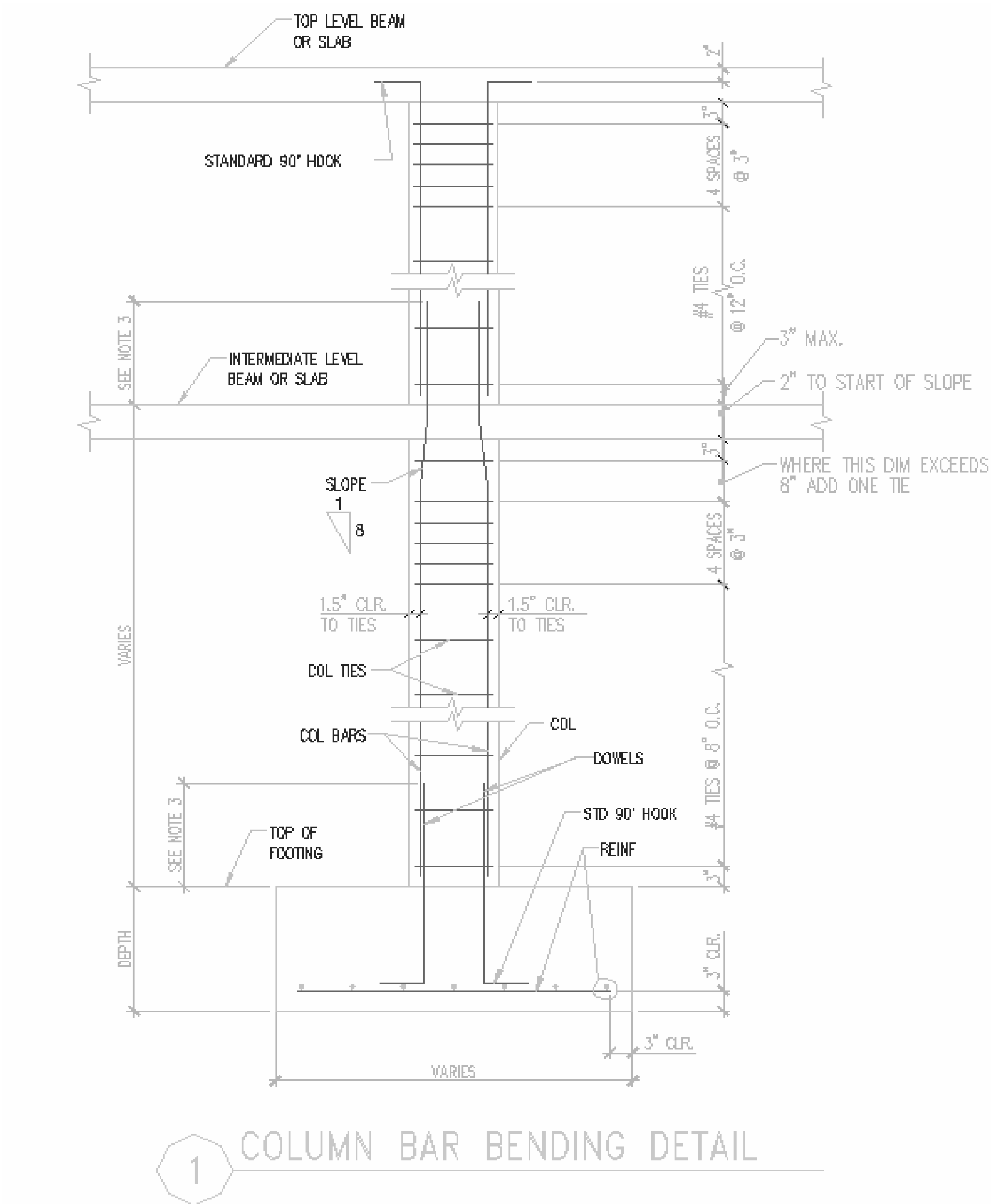
```
General Information:
====================
  File Name:  UNTITLED.COL
  Project:                                 Code: ACI 318-89
  Column:                                  Units: US in-lbs
  Engineer:                                Date: 04/06/06  Time: 08:38:10

  Run Option: Investigation                Short (nonslender) column
  Run Axis:   Biaxial                      Column Type: Structural

Material Properties:
====================
  f'c   = 6 ksi                                 fy   = 60 ksi
  Ec    = 4695.98 ksi                           Es   = 29000 ksi
  fc    = 5.1 ksi                               erup = 0 in/in
  eu    = 0.003 in/in
  Stress Profile: Block                         Beta1 = 0.75

Geometry:
=========
  Rectangular: Width = 24 in                    Depth = 24 in

  Gross section area, Ag =  576 in^2
  Ix =  27648 in^4                              Xo =  0 in
  Iy =  27648 in^4                              Yo =  0 in

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

  Confinement: Tied; phi(c) = 0.7,  phi(b) = 0.9,  a = 0.8
  #3 ties with #10 bars, #4 with larger bars.

  Layout: Rectangular
  Pattern: Equal Bar Spacing  [Cover to transverse reinforcement (ties)]

  Total steel area, As = 8.00 in^2 at 1.39%

  8-#9   Cover = 1.5 in
```

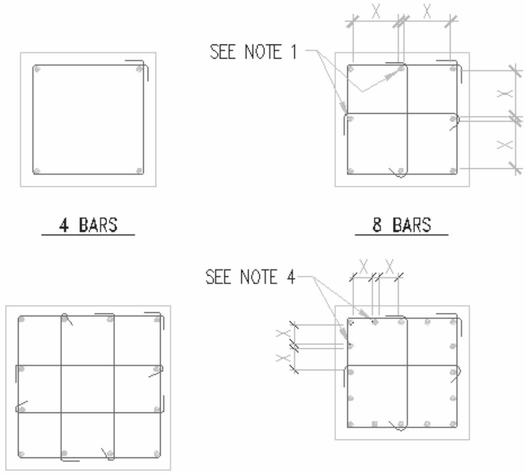

```
               Applied Loads         Computed Strength   Computed/
            P       Mx       My        P       Mx       My   Applied
   Pt.  (kips)  (ft-k)  (ft-k)    (kips)  (ft-k)  (ft-k) Ray length
   ----------------------------------------------------------------
   1       700     600      75       717     644      80    1.046


                   Program completed as requested!
```

```
General Information:
====================
  File Name:  T:\COLCHECK.COL
  Project:                                   Code: ACI 318-89
  Column:                                    Units: US in-lbs
  Engineer:                                  Date: 03/31/06  Time: 19:03:44

  Run Option: Design                         Short (nonslender) column
  Run Axis:   Biaxial                        Column Type: Structural

Material Properties:
====================
  f'c    = 5 ksi                                  fy   = 60 ksi
  Ec     = 4286.83 ksi                            Es   = 29000 ksi
  fc     = 4.25 ksi                               erup = 0 in/in
  eu     = 0.003 in/in
  Stress Profile: Block                           Beta1 = 0.8

Geometry:
=========
  Rectangular: Width = 30 in                      Depth = 30 in

  Gross section area, Ag =  900 in^2
  Ix =  67500 in^4                                Xo =  0 in
  Iy =  67500 in^4                                Yo =  0 in

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

  Confinement: Tied; phi(c) = 0.7,  phi(b) = 0.9,  a = 0.8
  #3 ties with #10 bars, #4 with larger bars.

  Layout: Rectangular
  Pattern: Equal Bar Spacing  [Cover to transverse reinforcement (ties)]

  Total steel area, As = 18.72 in^2 at 2.08%

  12-#11   Cover = 1.5 in
```


```
             Applied Loads          Computed Strength    Computed/
          P       Mx       My        P       Mx       My    Applied
  Pt.  (kips)  (ft-k)  (ft-k)    (kips)  (ft-k)  (ft-k)  Ray length
  ---------------------------------------------------------------
  1      2700     120        0     2726     117      -0    1.010


                     Program completed as requested!
```

Appendix 5 (Column and Shearwall details)





NOTES:
1) THESE BARS MUST BE TIED AS SHOWN BY DASHED LINES WHEN X DISTANCE IS GREATER THAN 6".
2) MINIMUM CONCRETE COVER IS 1.5" TO TIES.
3) PROVIDE 135° HOOKS FOR SEISMIC ZONES 2, 3 AND 4.
4) THESE BARS NEED NOT BE TIED WHEN DISTANCE X EQUALS 6" OR LESS.

MAXIMUM SPACING OF COLUMN TIES

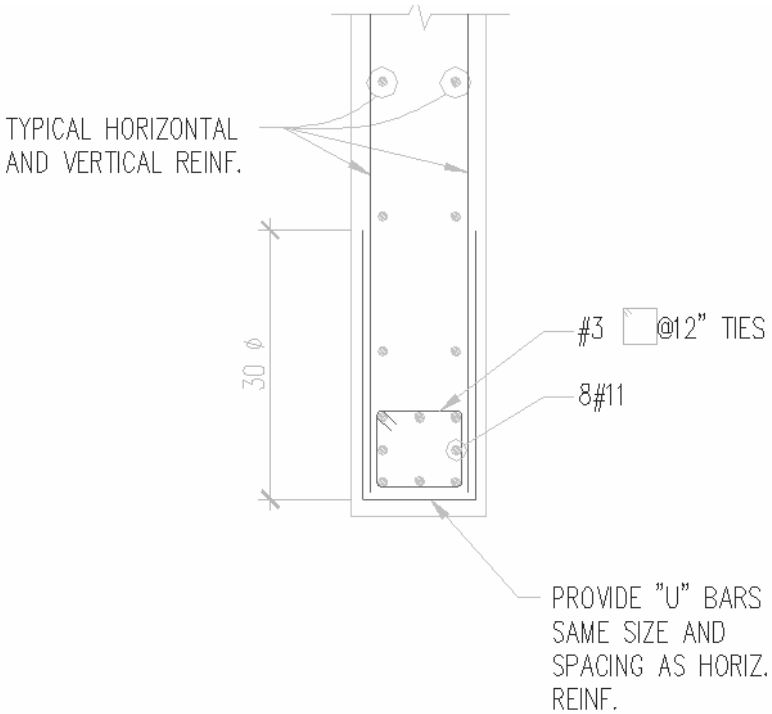
VERT BAR SIZE	SIZE & SPACING OF TIES (INCH)		
	#3	#4	#5
#5	10	–	–
#6	12	–	–
#7	14	–	–
#8	16	16	–
#9	18	18	–
#10	18	20	–
#11	–	22	22

MAXIMUM SPACING NOT TO EXCEED LEAST COLUMN DIMENSION.

3 SQUARE COLUMN TIE ARRANGEMENTS AND SPACING

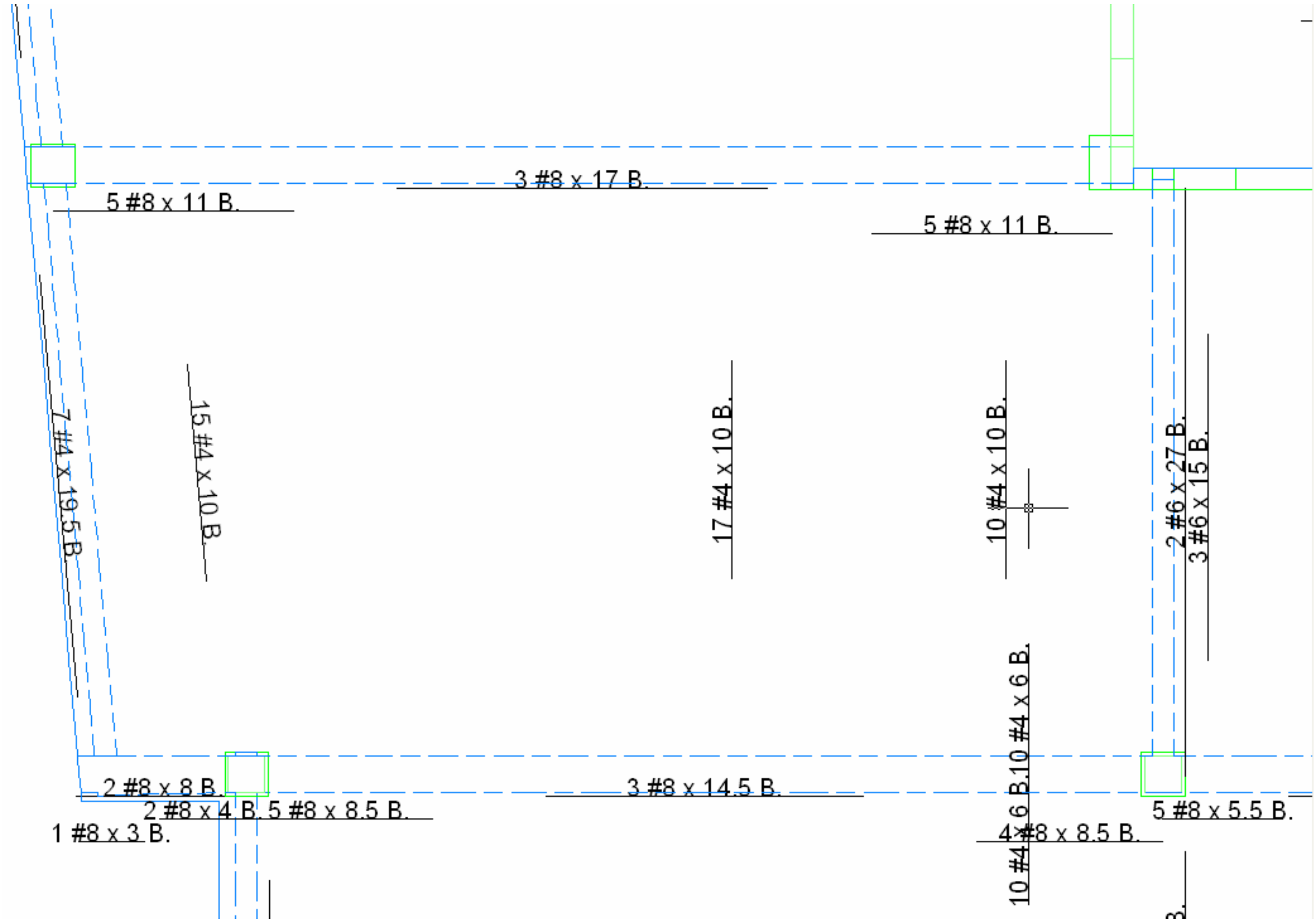
CENTER TO CENTER BAR SPACING	BAR SIZE	CONCRETE STRENGTH, PSI				
		4000	5000	5600	6000	7000
	#5	1'-11"	1'-9"	1'-8"	1'-7"	1'-6"
	#6	2'-4"	2'-1"	2'-0"	1'-11"	1'-10"
	#7	2'-9"	2'-5"	2'-4"	2'-3"	2'-1"
	#8	3'-3"	2'-11"	2'-9"	2'-8"	2'-6"
	#9	4'-2"	3'-8"	3'-6"	3'-5"	3'-2"
MORE THAN 5"	#10	5'-3"	4'-8"	4'-5"	4'-4"	4'-0"
MORE THAN 5?"	#11	6'-5"	5'-9"	5'-6"	5'-3"	4'-11"
5" OR LESS	#10	7'-4"	6'-7"	6'-3"	6'-1"	5'-7"
5?" OR LESS	#11	9'-0"	8'-1"	7'-8"	7'-5"	6'-10"

5 SPLICE LENGTHS FOR COLUMN VERTICAL BARS

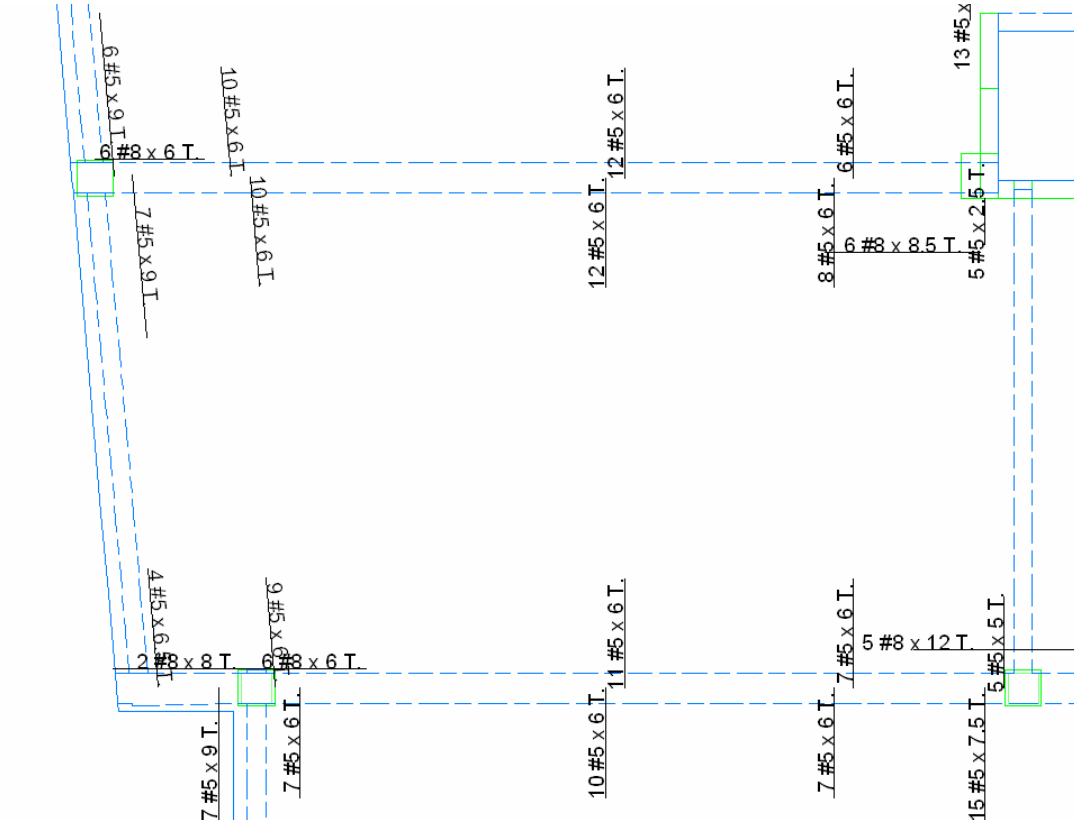


3 SHEAR WALL END

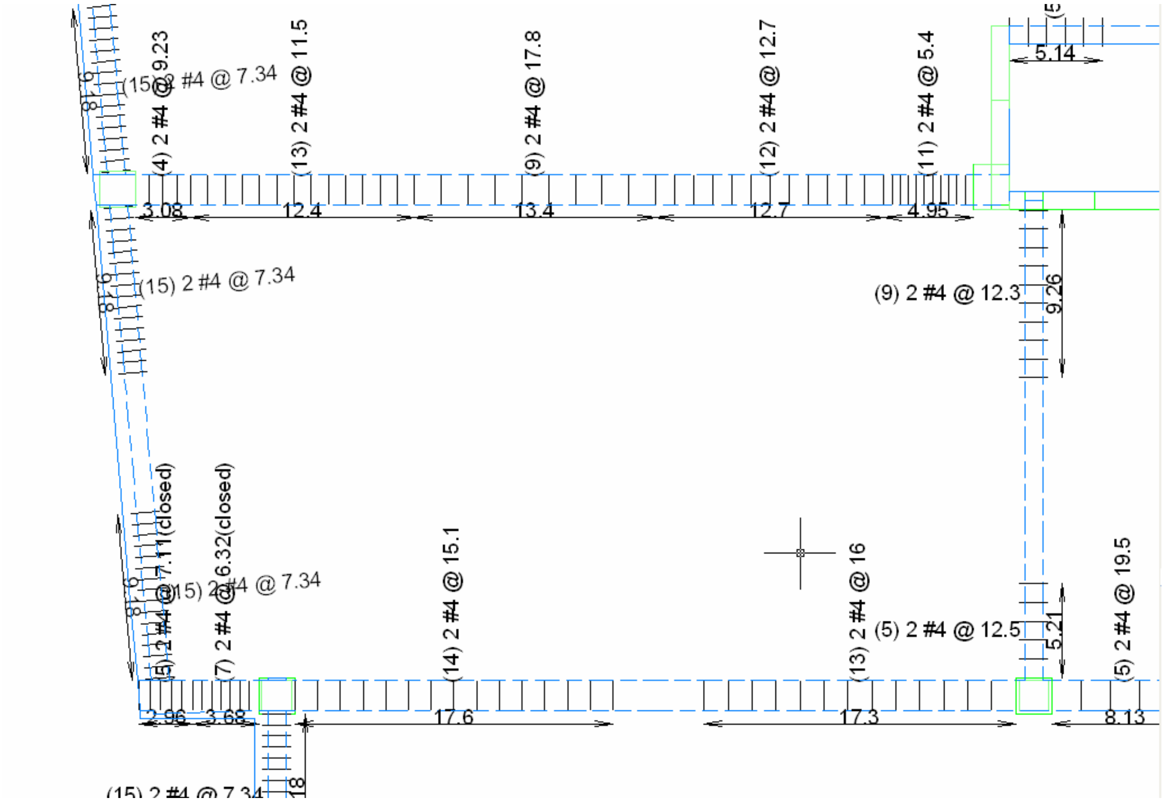
Appendix 6 (PT system details)



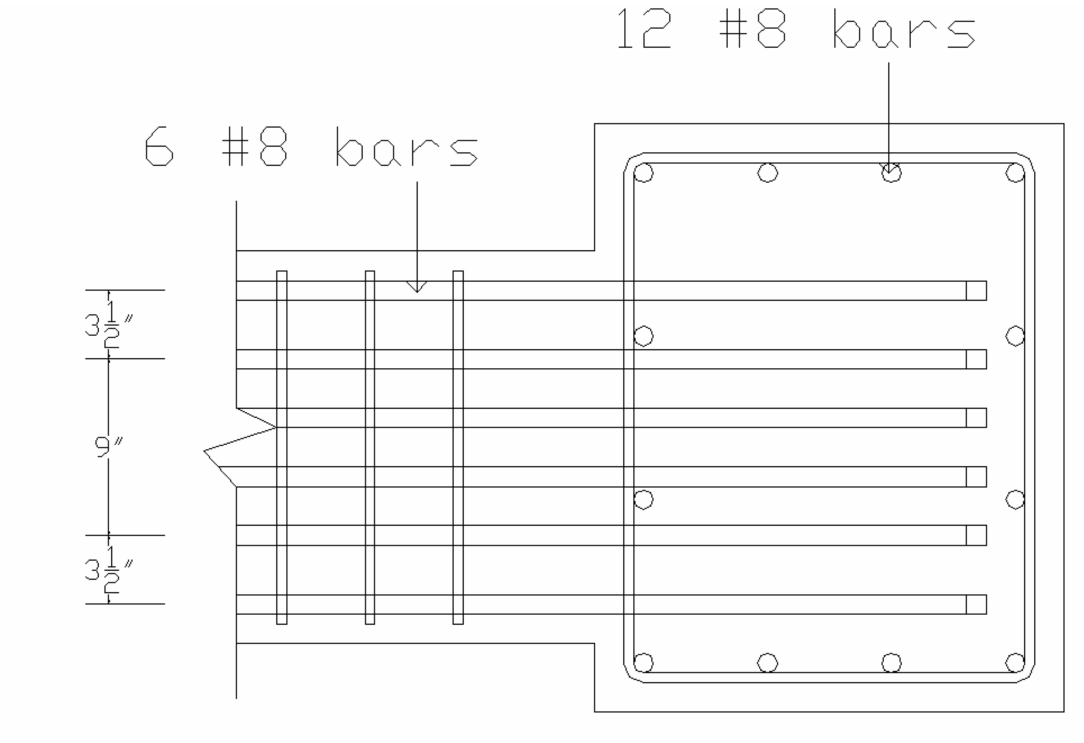
Bottom Reinforcing Plan



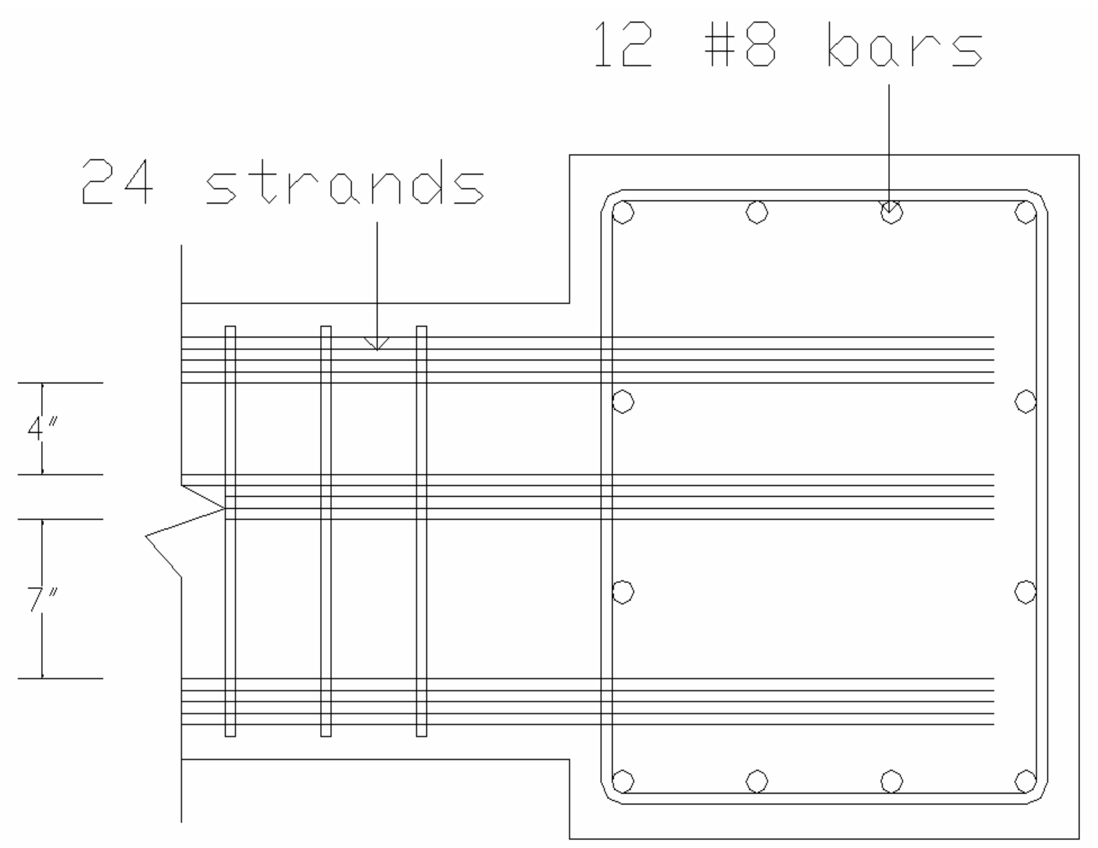
Top Reinforcing Plan



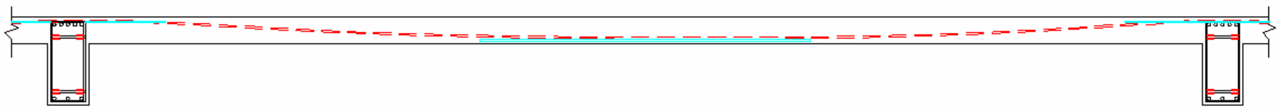
Shear Reinforcing Plan



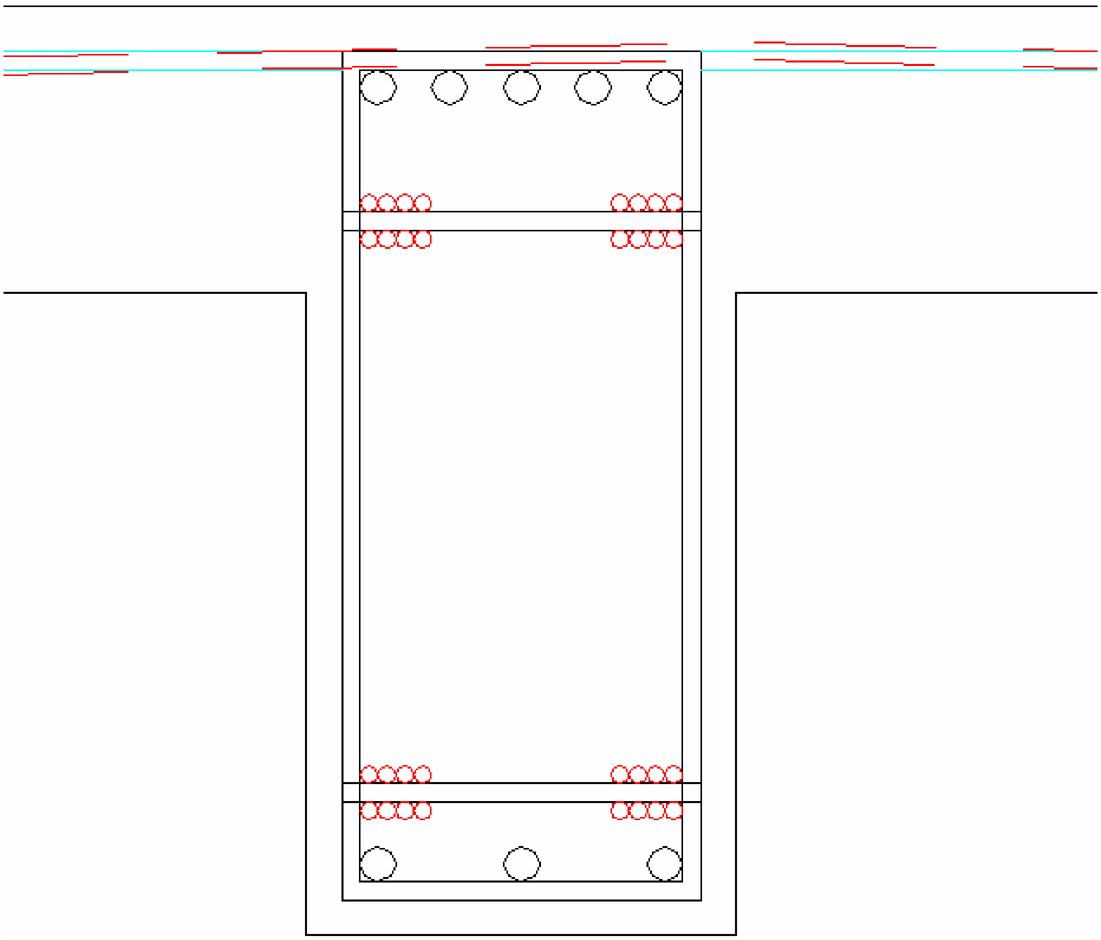
Rebar Congestion Detail



PT Congestion Detail



Slab PT Section



Beam PT Section