Architectural Engineering Senior Thesis Project
Structural Emphasis

Campus Square
Buildings C & D
Lehigh University

Matthew Kutzler

Spring 2004

Campus Square – Buildings C & D

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Structural

http://www.arche.psu.edu/thesis/2004/mpk149/

Lehigh University, Bethlehem, PA

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Building C

- Five stories (Two story bookstore; Three story housing)
- Size: 49,565 sq. ft. (gross); 18010 sq. ft. (multipurpose)

Building D

- Three stories, consisting mainly of quad apartments
- Size: 30,654 sq. ft.

Structural

- Continuous concrete strip footing along the perimeter and interior spread and combined footings
- Floors constructed of 8" precast concrete plank
- Building C: steel framing on second and third floors; load bearing masonry walls on fourth and fifth floors
- Building D: load bearing masonry walls on all three floors
- Shear wall lateral system

Construction Management

- Began in April 2001 and completed in July 2002
- Project delivery method : fast-track design and CM at risk
- Cost of Buildings C and D : $9.5 million dollars
- Somewhat similar, but smaller Buildings A and B built simultaneously with C and D as well as a parking garage

Electrical/Lighting

- 120/208V : indoor fans use 1 phase; indoor air handling units and condensing units use 3 phase
- 277/480V : horizontal unit heaters and pumps use 3 phase
- Standby generator rating : 80 kW, 100 kVA
- Electric snow melting system
- 6" x 4' recessed lighting fixtures placed throughout

Project Team

Owner : Lehigh University

Architect : Bohlin Cywinski Jackson

Mechanical/Plumbing/Electrical : Brinjac Engineering, Inc.

Civil and Structural : Barry Isett & Associates , Inc.

Construction Management : Alvin H. Butz, Inc.

Architecture

- Red brick face and gabled roof with asphalt shingles
- Connector bridge spans between the two buildings
- Aluminum storefront framing along the campus bookstore
- Second floor of bookstore is partially open to the first floor below
- Each quad apartment on first floor of Building D has it's own exterior access

Mechanical

- Main units for both buildings are found in Building C
- Combination of wet/dry pipe sprinkler system
- Modular indoor air-handling and split-system air-conditioning units
- Each room and corridor has an individual fan coil unit
- Dual heat exchangers convert steam from central utilities to hot water

Campus Square – Buildings C & D
Lehigh University, Bethlehem, PA

Advisor: Dr. Memari
Spring 2004

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Executive Summary

This report is an exploration into the existing structural system of Lehigh University's Campus Square and whether proposed alternatives can possibly replace the system. Campus Square is a multipurpose building complex consisting of four residence halls, a parking deck, and retail space.

For the purposes of the Senior Thesis project, only Buildings C and D were investigated in the fall semester. Building C is the focus of the spring semester redesign, though the proposed alternatives could be applied to the other buildings in the complex, including Building D.

First, an overview of Buildings C and D will be presented, including other systems and architecture. The existing structural system is composed of a steel frame and load bearing masonry walls along with precast concrete plank flooring.

A proposal was drawn up to replace this system, while still retaining some original criteria. Once the aspects for investigation were established, exploration began into two alternative systems. One is an all concrete building with pan joist flooring. The other system uses a steel frame with open-web steel joists and non-composite metal deck. Each was looked at in terms of framing, foundations, lateral system, floor vibration, and fire protection.

The two breadth areas included acoustics and construction management. A cost analysis was performed in addition to a discussion of other issues and some formwork design.

The conclusion was drawn that the existing and proposed concrete systems are both adequate solutions. Though the concrete system is a little more expensive, it has some added benefits. The proposed steel solution would most likely not be appropriate for the building studied.

Credits/Acknowledgements

I would like to thank those who helped me throughout the past year during my thesis project. Without them, this would not have been possible to complete.

- Barry Isett & Associates, particularly Ross Sotak and Jeff Thoms, for sponsoring me. They provided me with the drawings and resources I needed as well as answered many of my questions.

- Anthony Corallo and Lehigh University, for allowing me to use Campus Square.

- Alvin H. Butz, the construction manager

- The entire AE faculty for all of their support and help

- My fellow AE students, friends, and family for keeping my spirits up when times got tough

Building Background

Campus Square is a multipurpose building complex consisting of four residence halls, a parking deck, and retail space. It is constructed on the outskirts of Lehigh University's Asa Packer Campus in Bethlehem, PA, as seen in Figure 1.

The complex is intended to create a bridge between the University and the south side of Bethlehem. A parking lot was originally located at the site, creating a divide between the students and community. However, with the new presence of stores such as Barnes & Noble and the campus bookstore, Lehigh University hopes to end this abrupt split with Campus Square.

Figure 1: North quadrant of Asa Packer Campus

For the purposes of the Senior Thesis project, only Buildings C and D were investigated in the fall semester. As will be mentioned later, Building C is the focus of the spring semester redesign, though the proposed alternatives could be applied to the other buildings in the complex, including Building D.

The primary project team is as follows:

- Owner: Lehigh University
- Architect: Bohlin Cywinski Jackson
- Mechanical/Plumbing/Electrical: Brinjac Engineering, Inc.
- Civil and Structural: Barry Isett & Associates
- Construction Management: Alvin H. Butz, Inc.

The complex was constructed from April 2001 - July 2002 at a cost of $23 million. Buildings C and D commanded around $10 million of the total project cost. The delivery method involved was a fast-track design and the complex was constructed with the Construction Manager at risk.

The four residence halls are labeled simply with letters. Buildings C and D form an L shape, mirroring Buildings A and B in the process. Both building footprints are somewhat large, with Building C covering about 11,500 square feet and Building D having a footprint of nearly 10,250 square feet. However, Building C is the taller of two, rising nearly 70' above ground level to the roof peak. This makes it nearly 20' taller than Building D.

Below is a quick overview of the different systems within Campus Square, though the structural system is detailed later:

- *Electrical/Lighting*
  The electrical system derives its power from a 13.2 kv system already installed on campus. This power is distributed through transformers depending on the demand of the equipment. A 120/208V system is in place for loads such as indoor air handling units and condensers, which use a three phase system. It also supplies power to single phase indoor fans and other minor loads. Heavier loads receive power at 277/480V, such as three phase pumps and horizontal unit heaters. The complex also has an electric snow melting system and an 80 kW, 100 kVA standby generator in case of power outages. Most of the lighting consists of 6" x 4' recessed fixtures and smaller, circular units such as 2' x 2' recessed fixtures and wall washers.

- *Mechanical*
  As with the electrical system, main components of the mechanical system are linked to a central utility plant via underground piping. Steam from this plant is converted by dual heat exchangers into hot water for the plumbing system. A separate heat exchanger converts steam in the same manner for the heating system. Hot and chilled water are delivered to terminal units through a 2-pipe summer/winter changeover system. The terminal units then branch off to individual fan coil units in the dorm rooms, corridors, and other areas. Ventilation is provided through modular indoor air-handling units and split-system air-conditioning. Main mechanical units servicing both buildings can be found in Building C.

- *Fire Protection*
  A combination wet/dry sprinkler system services both buildings. Piping for this system runs underground and meets the central utility plant for the campus. A full fire alarm system is also in place. Most structural elements have a fire rating of one hour, with load bearing walls, shafts, and stairways/exits having a rating of two hours. In addition, all steel structural members have been coated with spray-on fire proofing.

- *Transportation*
  Building C has two elevators and Building D has one. Traveling at 150'/minute, the cab is 6.67' x 4.25' with a height of 8'. The bookstore elevator travels only to the second floor between the receiving area and stockroom. The other two elevators provide transportation for the disabled, as well as easy means for caretakers and residents to transport heavy items from floor to floor. Stairwells are present on the northern and southern ends of Building C and the western and eastern portions of Building D.

- *Telecommunications*
  Small rooms housing central areas for telecommunications data exist in both buildings. Each apartment has phone, cable, and internet access. This is provided via fiber optic cable.

Architecture

As mentioned before, the footprints are essentially in the shape of two L's that mirror each other. A large pedestrian plaza passes between the two L's. The entire complex has a total of 250 beds divided between apartments featuring two, three, or four bedrooms. Each apartment contains fully furnished living and dining areas as well as a private kitchen and bathroom. The apartments are separated from each other by a combination of masonry walls and full height partitions. Almost every floor possesses a different floor-to-ceiling height, but all are larger than the 8'.

Buildings C and D have a red brick exterior face that can be seen in Figure 2, as does the entire complex. Larger windows appear around the campus bookstore along with aluminum storefront framing. Both buildings have a gabled roof at a slope of 9:12 and covered with asphalt shingles.

Figure 2: The façade of Building C near its completion

On the outside, Buildings C and D appear similar in design and function, but a few differences exist in the interior. Building C houses the campus bookstore on its first two levels, along with a stockroom and office space, among other rooms. A portion of the second floor is open to the first floor below. Most of the bookstore is an open space, occasionally interrupted by columns every 20' in most cases. The three upper stories are comprised of double, triple, and quad apartments, as described above. Most means of egress are tied in with the bookstore below.

Building D contains three floors of quad apartments and no merchant space. See Figure 3 for the layout of both buildings. Each room on the first floor has its own exterior access, giving it the appearance of row homes. A connector bridge extends between Buildings C and D, providing overhead shelter for those walking between the two buildings.

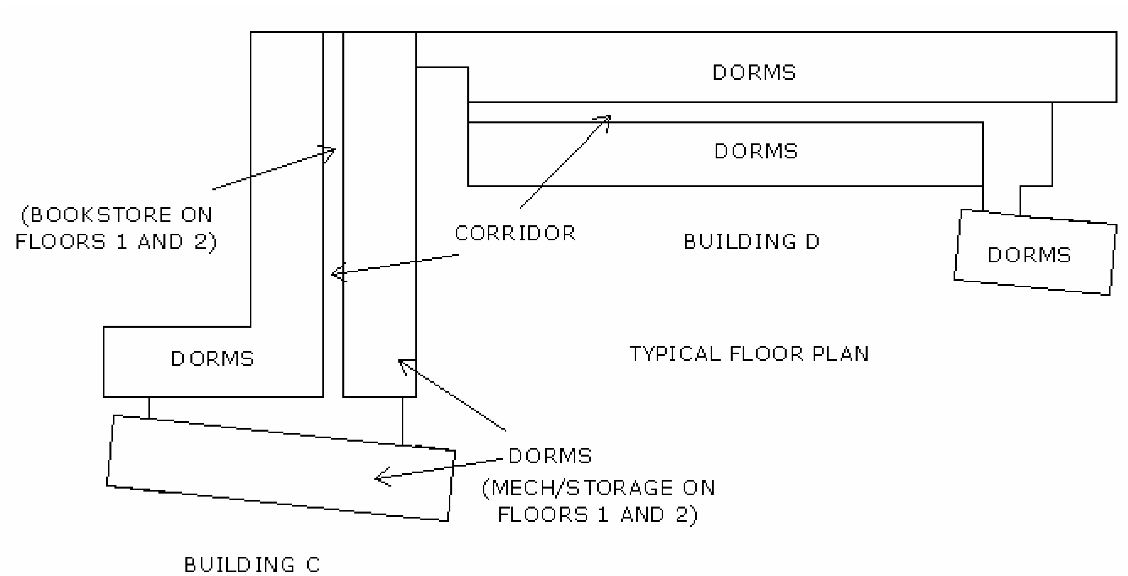


Figure 3: Function of Buildings C and D

Existing Structural System

Buildings C and D utilize two somewhat different framing systems, but are similar in most respects. Building C is composed of steel members on the second and third floors. Figure 4 is a rough framing plan with typical beam and girder sizes. Typical bays for Building C are 15' x 20' and 20' x 20'. Generally, W16's and W18's are used to support the floors in apartment areas. W14's are used for the corridors, though their longest span is only 6'. Most columns used in the building are W12 x 79. Floors four and five consist entirely of load bearing masonry walls. Typical bays are larger than the lower floors, spanning approximately 48' x 20'.

Figure 4: Building C – Typical Framing Plan

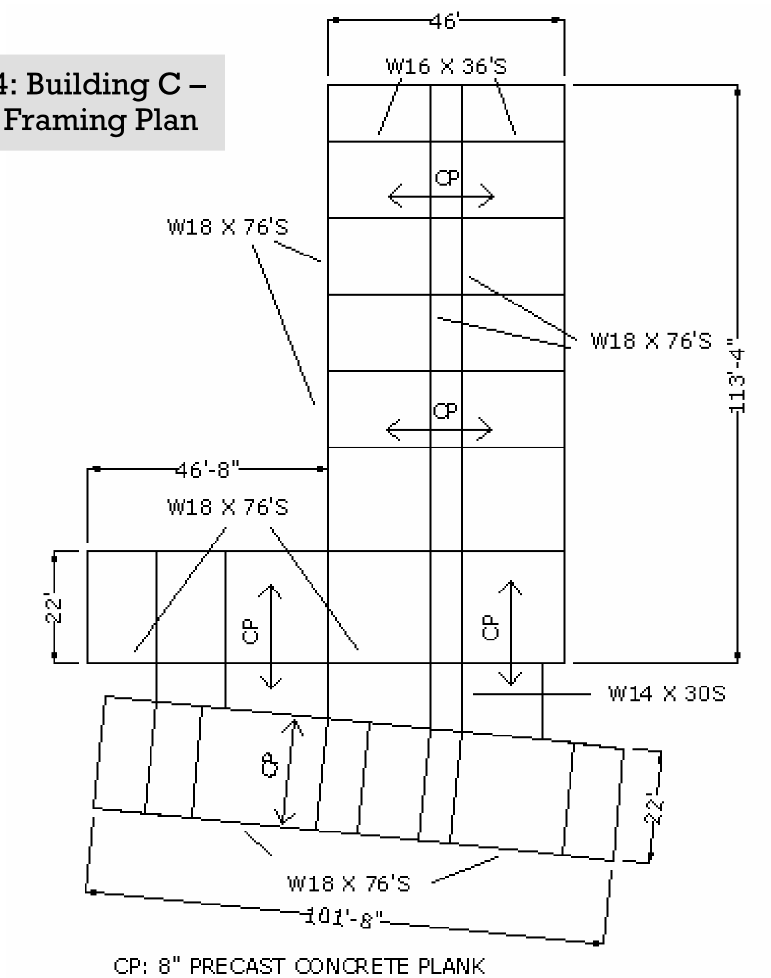


Figure 5: Building C – The steel frame on the first two floors coupled with load bearing masonry walls on the upper floors can be seen.

Figure 6: Building C – The steel frame as seen from the west face.

Building D has a framing system similar to that appearing on floors four and five of Building C (Figure 7). Load bearing masonry walls assume all of the gravity loads for floors two and three and most bays are again approximately 48' x 20'. Both buildings have gabled roofs supported by trusses ranging from 12' to 45' in length and spaced at 2' on center (o.c.).

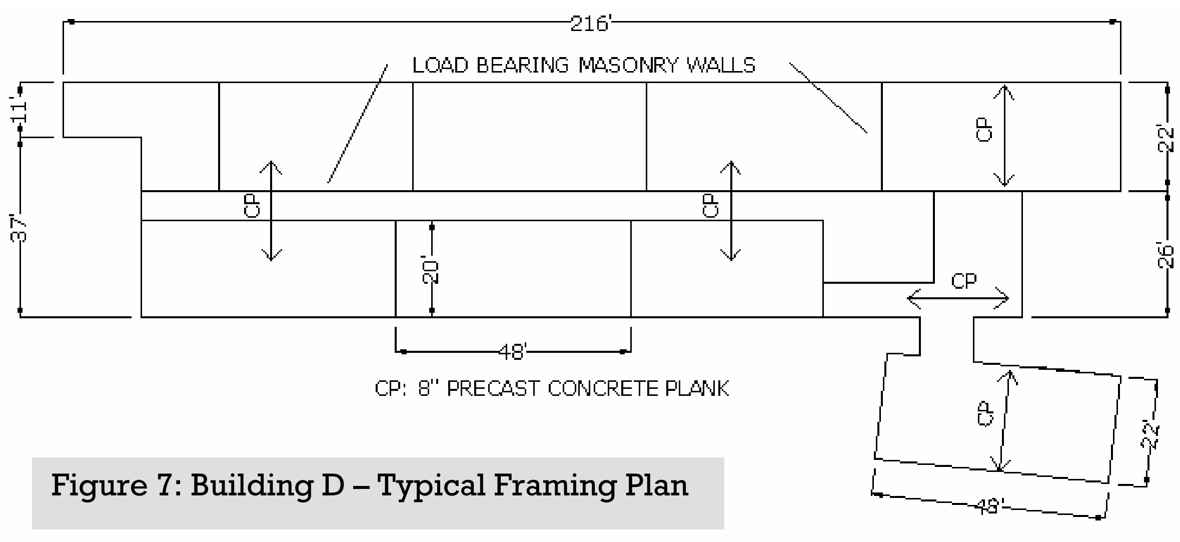


Figure 7: Building D – Typical Framing Plan

Concrete strip footings with strength of 4000 psi appear along the perimeter of each building. In addition to the strip footings, Building C employs spread and combined footings in the interior of the foundation. Spread footings are generally 7' x 7', while combined footings as large as 11' x 20' also appear. The footings range from 2' to almost 4' in depth. The buildings are resting on virgin soil with a bearing pressure of 3 ksi.

Both buildings have a 4" concrete slab-on-grade with strength of 4000 psi. With the exception of the ground floor in each building, all floors are constructed of an 8" prestressed hollow core concrete plank at 5000 psi. The strengths of the concrete masonry units and the structural steel are 1500 psi and 50 ksi, respectively.

Lateral loads are resisted by a series of concrete masonry unit shear walls in each building. Building C has shear walls on floors three through five and all three floors in Building D. The shear walls extend the entire height of each floor and are generally spaced 48' apart in both buildings. See Figure 8 for the shear wall layout.

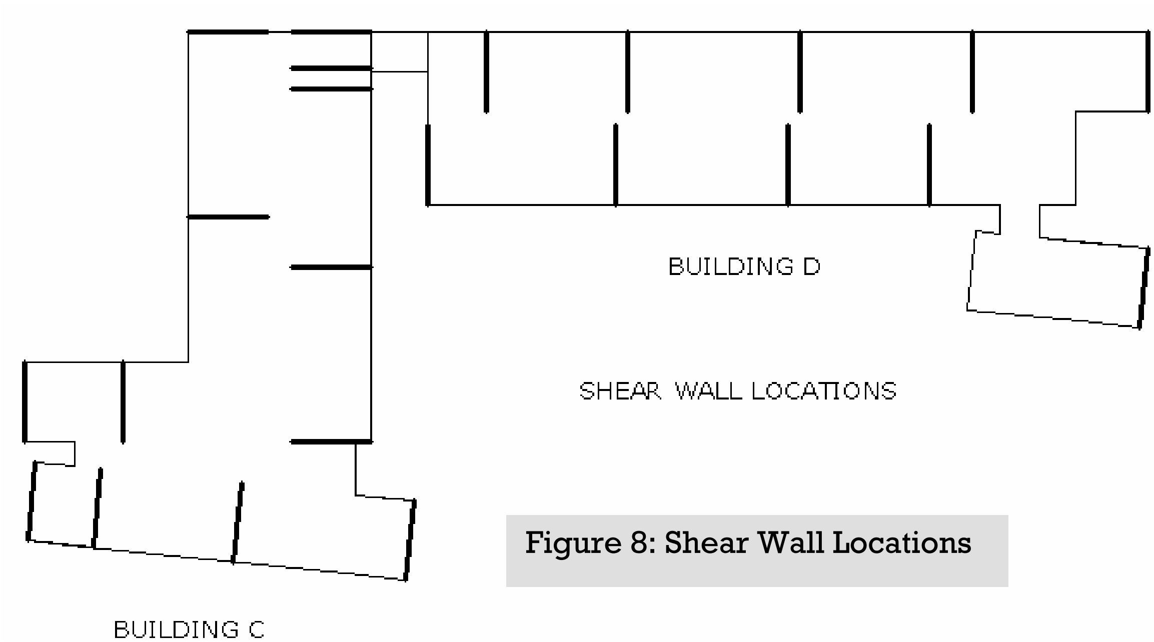


Figure 8: Shear Wall Locations

National Code

- BOCA National Building Code, 1996
- ASCE 7-98 Standard

Design Codes and Standards

- Masonry
    - American Concrete Institute (ACI 530) – Building Code Requirements for Concrete Masonry Structures
    - National Concrete Masonry Association (NCMA) – Specifications for the Design and Construction of Load-Bearing Concrete Masonry
- Steel
    - American Institute of Steel Construction (AISC)
    - Allowable Stress Design (ASD), 9th edition

Gravity Loads

Floor Live

- 40 psf residential dwelling units
- 20 psf partitions
- 100 psf corridors and stairwells

Roof Live

- 20 psf
- 30 psf snow

Floor Dead

- 58 psf 8" hollow core, precast concrete plank
- 6 psf M/E/P
- 4 psf Flooring
- 11 psf Steel framing

Roof Dead

- 25 psf truss system w/ finishes and MEP
- 5 psf asphalt shingles and insulation

Wall Dead

- 50 psf 8" masonry walls (grout is 48" o.c.)

Lateral Loads

Wind loads were calculated for each building in the North-South and East-West directions. They were then compared with the seismic loads, which control about half of the time (Table 1). Overall, the shear wall systems in Buildings C and D were found to be adequate to handle the lateral loads. The direct forces on each wall were equal for each floor due to the same height, strength, and length of the walls. As expected, those walls farthest from the center of rigidity carried the most torsional forces. However, torsion was a very minor factor in the total lateral force experienced by each shear wall. The drift check was acceptable along with the strength of the shear walls.

Building C - Lateral Load Comparison				
	North-South		East-West	
Level	Seismic (k)	Wind (k)	Seismic (k)	Wind (k)
Roof (flat)	1.42		1.42	
Roof (gable)	27.81	**42.00**	27.81	**37.20**
5	**72.40**	27.00	**72.40**	43.20
4	**56.66**	25.98	**56.66**	41.57
3	**46.97**	30.00	46.97	**48.00**
2	28.83	**38.01**	28.83	**60.82**
1	1.33	**30.51**	1.33	**48.82**
Total	234.00	193.50	234.00	309.60

Building D - Lateral Load Comparison				
	North-South		East-West	
Level	Seismic (k)	Wind (k)	Seismic (k)	Wind (k)
Roof	37.87	**84.00**	**37.87**	20.16
3	**80.07**	54.00	**80.07**	12.96
2	46.50	**54.00**	**46.50**	12.96
1	2.48	**46.98**	2.48	**11.28**
Total	167.00	238.98	167.00	57.36

Table 1: Lateral load comparisons

Proposal

Criteria to Maintain

In approaching any redesign, there are multiple options one could pursue in altering a building structurally. I found the easiest way to begin was to determine the existing conditions I wished to retain. As these conditions were gathered, a clearer picture of what course to take began to emerge. Some of the things I wanted to preserve were:

- *Floor-to-ceiling height:*
  The height of the residence floors was only 8' and I did not want to reduce them below this. As for increasing the height, it would put the building in danger of exceeding the 70' height restriction imposed by local building codes. This still could become a problem with larger plenum spaces in the proposed alternatives. The bookstore also possesses a height of 8' on the second floor, but ranges from 9' to 11' on the first floor. These first two floors are the only ones in the entire building with a finished ceiling and plenum space for other trades. The precast concrete plank provides a finished ceiling for the residence floors.

- *Current floor plan:*
  Since the building is a residence hall, little option exists for changing the existing floor plan. The shape of the building was mainly arrived at to serve the floor plan, which has a corridor down the center and rooms on each side. While a more efficient structural system might be possible, a better floor plan is unlikely. However, some apartment lengths may be increased or decreased if shear walls are moved to accommodate new lateral forces. Shear walls usually separate the apartments.

- *Architectural scheme:*
  It is inevitable that with any structural redesign come modifications to the overall architecture of a building. However, I want to keep this to a minimum to preserve the outside façade and interior finishes, mainly in the bookstore. Most of the changes will occur due to larger or smaller plenums resulting from the alternate systems. The greatest change will likely concern a roof alteration in order to keep the total building height below 70' to accommodate local codes.

Goals

Once these criteria were determined, I decided what aspects were the most critical for a redesign. Listed below are the areas in which I will be performing an analysis:

- *Framing:*
  This is the most involved portion of the redesign. The existing frame will be analyzed to find what is worth patterning and what could be improved upon to better suit the alternatives. This will dictate the remainder of the redesign, such as the location of lateral elements and foundations, among other things.

- *Foundations:*
  The foundations will be altered to accommodate the new frame layout as needed. They will consist of continuous strip footings along the exterior and a combination of spread and combined footings within the perimeter.

- *Lateral system:*
  While the shear wall system will be preserved, it will be investigated to see if some walls can be removed or relocated to be more efficient. Plus, walls may need to be added in order to compensate for increased lateral loads.

- *Vibration:*
  With new floor systems questions will arise as to how they perform with regards to impact vibration. Therefore, a study will be conducted into their adequacy in this area.

- *Fire protection:*
  It's important to maintain current fire protection ratings for crucial structural elements. Fire rated assemblies will be employed where applicable.

- *Acoustics:*
  Sound transmission is always a concern in buildings with multiple residents. As a result, an investigation will be done to ensure that the wall and floor assemblies are acceptable in terms of acoustics.

- *Construction Management:*
  To measure the effectiveness of alternate systems costs and time issues will be looked at. Design of concrete formwork will be performed as necessary.

Depth Study: Structural Redesign

As noted previously, Building C is the focus of the spring semester redesign, though the proposed alternatives could be applied to the other buildings in the complex, including Building D. The selection of alternative framing systems was begun by referring to Technical Assignment #2 from last semester. It included ventures into steel joists with non-composite deck, composite steel, flat slab with drop panels, and concrete pan joists. Through these rough design explorations, composite steel and a flat slab system were deemed inappropriate for the function of the building and its current layout.

When I originally approached my sponsor about using one of their projects for thesis, they gave me some insight into Campus Square along with some ideas to possibly pursue. They said they would have liked to design the buildings with steel frames in their entirety. This was especially true for Buildings B and C, which are a mixture of steel framing on the lower floors and load bearing masonry on the upper floors. Also, the firm was forced to employ precast plank flooring because anything thicker than one foot would have put the structure at risk for breaking the local height limitations.

As a result of these ideas, I wanted to eliminate the precast concrete plank and replace it with a poured-in-place concrete slab, whether on metal deck or with formwork. I knew this would create problems with the height limitations, but an alteration of the roof slope would not be a problem since this was an exploratory project. Furthermore, I decided to use a steel frame throughout Building C to coincide with an open-web steel joist flooring system. Since this was somewhat similar to the existing plan, a completely different system would be of interest to determine benefits and drawbacks of each design upon completion. This led to the selection of a one-way concrete pan joist system, which was deemed the most appropriate concrete solution among other possible selections.

One-way concrete pan joist system

General

- Normal weight concrete (150 pcf)
- f'_c = 3000 psi for the foundations
- f'_c = 4000 psi for the slabs, joists, and girders
- Grade 60 (f_y = 60 ksi) reinforcement

Design Codes

- BOCA National Building Code, 1996
- ASCE 7-98 Standard
- ACI 318-02

The live load is the same as that used in the existing structure and, along with the dead loads, can be found below. The superimposed load was used as a blanket load for such elements as floor tile and other miscellaneous loads. The wall weight would be multiplied by the floor-to-floor height to achieve a load in plf.

Live

- 40 psf residential dwelling units
- 20 psf partitions
- 100 psf corridors

Dead

- 25 psf superimposed
- 44 psf slab: w = (3.5 in)(150 pcf)(1/12)
- 50 psf wall weight

Total: 69 psf

A higher strength concrete was used because of the desire to maintain shallow floor members. Sizes of the elements were arrived at based on preliminary calculations.

- 40” pans
- 8” rib size
- 3.5” minimum concrete slab thickness needed for 1 hour fire rating
- 18” wide girders
- 18” wide columns

Framing – Concrete System

The existing column layout is pretty efficient. Some bays that were originally 15' x 20' were increased to 20' x 20', but little else changed. Exploration into 25' x 25' bays was done, but the depth of the concrete members became larger than I wanted. There was very little room for altering the exterior columns to preserve the architecture, but three were able to be eliminated. Another two columns were removed from the interior, bringing the total to five.

Once the bays were established, the slab was determined. It needed to carry moment from the live and dead loads as well as fulfill the shrinkage and temperature requirements for reinforcement steel. The largest live load, which was 100 psf for the corridor, was conservatively used for the entire building. The result was a 3.5" slab, which was the minimum needed to avoid a fire-rated floor assembly and still achieve a one-hour fire rating. The reinforcement will be #3's @ 12" o.c.

With the criteria used above, the pan joists will be placed at 4' o.c., resulting from 40" pans and 8" wide joists. The pan joists could have been designed for the predominant lengths of 10', 15', and 20', but would have done more harm than good. I wanted to keep all of the joist depths and reinforcement consistent for ease of construction. Pan joist systems are effective because of their redundancy and I wanted to take advantage of this. Plus, using the same reinforcement would reduce confusion on the job site and ensure placement of the correct bar size. The final size arrived at to accommodate all spans was a 12" x 8" joist. The steel consists of 2 - #6's on the top and bottom and #3's @ 12" to provide shear reinforcement.

Keeping the girders the same depth as the joists (12") was a top priority for ease of constructability once again. They are designed to resist shear and moment along their span and at the face of their supports. Unlike the joists, designing the girders for various span lengths and live loadings is needed because of their size and amount of load they will carry. Still, an overall size of 12" x 18" will be present throughout the entire floor. The span lengths and their reinforcement can be seen in Table 2 on the following page.

Girder Reinforcement					
Girder	Span (ft)	Tributary Width (ft)	Top	Bottom	Shear
a	20	10	5 - #7	4 - #5	#4 @ 12"
b	20	20	5 - #8	4 - #6	#4 @ 6"
c	20	6	7 - #6	2 - #6	#5 @ 12"
d	20	13	5 - #8	3 - #6	#5 @ 12"

Table 2: Girder Reinforcement

The final element designed was the columns. Axial and flexural loads were inputted into PCA column to determine if they fell within the interaction diagram. Reinforcement was also arrived at using this program. Though all of the columns are sized at 18" x 18", three different columns bays were investigated for any variation in reinforcement. They are listed below with the location they are predominantly used in.

- Column A: 20' x 10' … exterior
- Column B: 20' x 13' … corridor
- Column C: 20' x 20' … interior, other than the corridor

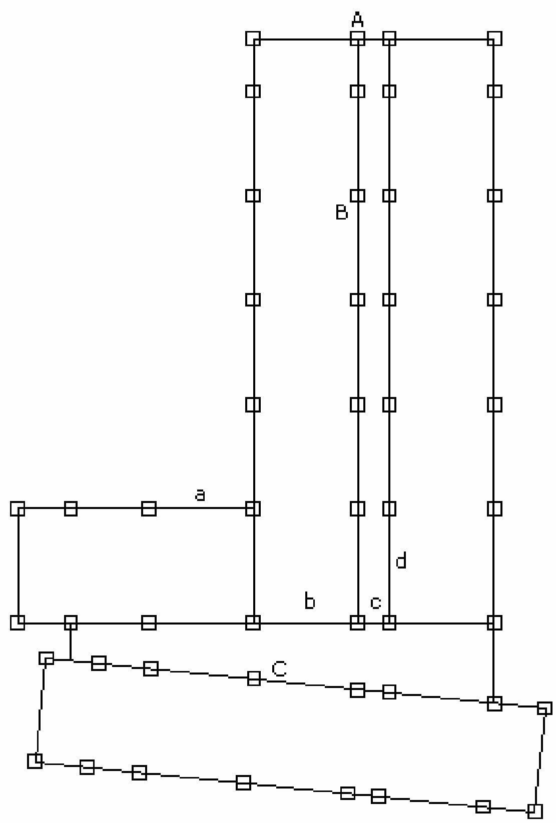


Figure 9: Column and Girder Layout

All of the 18" x 18" columns are reinforced with 4 - #9's and are non-slender. The columns carrying a smaller tributary area could probably use less reinforcement, but it was thought to be more efficient for cost and constructability to specify the same column properties. See Figure 9 for the girder and column layout.

In regards to the floor and building heights, their variation from the existing structure can been seen in Table 3. The lower floors experienced a decline in floor thickness because the steel members were replaced with shallower concrete members. However, the upper floors increased in thickness due to the presence of members supporting the concrete slab, instead of simply taking the concrete plank into account. The roof slope remained 9:12 and was able to stay below the local 70' height restriction.

Floor Height Comparison						
	Existing			Concrete		
Floor	Floor-to-Ceiling (ft)	Floor Thickness (ft)	Total Height (ft)	Floor-to-Ceiling (ft)	Floor Thickness (ft)	Total Height (ft)
Roof	-	-	19	-	-	19
5	8	-	8	8	-	8
4	8	0.67	8.67	8	1.46	9.46
3	8	0.67	8.67	8	1.46	9.46
2	8	3.5	11.5	8	2.55	10.55
1	9	5	14	9	4.3	13.3
		Total	69.84		**Total**	69.77

Roof slope:	
Existing - 9:12	
Concrete - 9:12	

Table 3: Floor Height Comparison

All of the calculations for the concrete system can be found in Appendix A.

Foundations – Concrete System

A concrete system will usually result in an increased foundation size, especially if using normal weight concrete, as has been done here. Since the column layout was altered slightly, the foundations will almost remain the same. Continuous strip footings will still be used along the perimeter of the building with spread and combined footings in the interior. Only one combined footing has been eliminated as a result of the change in the column grid.

In the redesign, the continuous strip footings were not looked at. The existing foundation has footings ranging from 14" to 32" in thickness and 36" to 54" wide. Reinforcement is generally either a #5 or #6 bar spaced at 12" o.c. These sizes are assumed to increase slightly with the concrete system. Instead, the spread and combined footings were concentrated on, particularly the larger ones, to get a good idea of the sizes that would most likely be used. Some footings were determined to only need steel on the bottom each way, but it is usually specified on the top and bottom to account for uplift forces.

The foundations were checked for a variety of things, such as bearing on the soil, shear, and bending. Ductility and shrinkage and temperature steel parameters were also looked at as well as overturning moment. Below is the footing sizes I arrived at supporting the columns specified in Figure 9. I've detailed the footings in Table 4 in addition to the sizes that would most likely be used when taking uplift into account. See Appendix B for calculations.

Typical Footings					
		Calculated		Specified	
Column	Size	Depth	Reinf.	Depth	Reinf.
A (spread)	9'-0" x 9'-0"	1'-6"	9 #7 E.W.	2'-4"	9 #7 E.W. (T + B)
B (comb.)	16'-0" x 12'-0"	2'-6"	#9 @ 12" E.W.	2'-6"	#9 @ 12" E.W. (T + B)
C (spread)	12'-0" x 12'-0"	2'-0"	12 #8 E.W.	2'-4"	12 #8 E.W. (T + B)

Table 4: Typical Footings

<u>Lateral – Concrete System</u>

One large advantage of a concrete system is that no added lateral resistance is needed. Therefore, the existing shear walls can be removed, since the concrete will act as its own lateral resistance system. This can be done since the columns, girders, and slab will be poured monolithically and act as moment connections. For low-rise buildings, such as the five-story structure I am analyzing, most of the drift is caused by something called bent action, which is the end rotation of beams and columns in rigid frames.

Wind and seismic loads were calculated for Building C in the North-South and East-West directions. Seismic forces control on nearly every floor (Table 5). For the purpose of analyzing the bents, the largest lateral load of the two cases on each floor was taken as the maximum lateral load.

Building C - Lateral Load Comparison				
	North-South		East-West	
Level	Seismic (k)	Wind (k)	Seismic (k)	Wind (k)
Roof (flat)	1.34		1.34	
Roof (gable)	**31.37**	31.20	31.37	**49.92**
5	**71.42**	30.33	**71.42**	48.53
4	**55.30**	31.50	**55.30**	50.40
3	**37.42**	31.89	37.42	**51.02**
2	20.14	**36.66**	20.14	**58.66**
1	0.00	**20.52**	0.00	**32.83**
Total	216.00	182.10	216.00	291.36

Table 5: Lateral load comparisons

The most common bent present is through two apartments and the corridor. Figure 10 contains an example from the 3rd floor. The lateral forces are added during the descent down the building. Referring to the calculations in Appendix C, the story drifts and total drifts are found for the building, treating the connections as moment connections. The total bent displacement is 0.515" at the top of the building, which is below the 1.53" allowable drift based on h/400. H/400 is an accepted industry standard based on experimentation and is similar to that specified in BOCA 1999. The successful transferring of lateral loads by this bent indicates that other areas of the building can perform as needed, also. This conclusion can be drawn since a greater moment of inertia will result from the 12" x 8" joists at 4' o.c. than a girder placed every 20' in a 20' wide bay. Overall, the concrete "moment frame" is adequate.

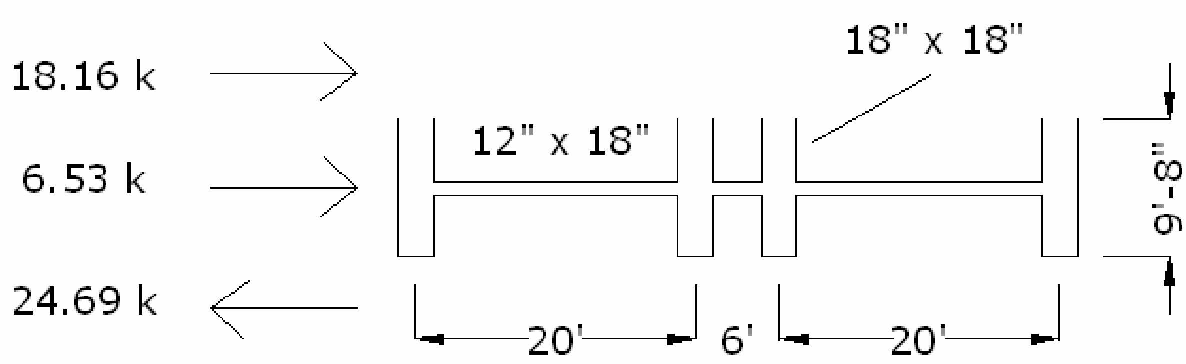


Figure 10: 3rd Floor Bent Elevation

Open-web steel joist system

General

- Normal weight concrete (150 pcf)
- f'_c = 3000 psi for the foundations and slabs
- Grade 50 steel (f_y = 50 ksi)

Design Codes

- BOCA National Building Code, 1996
- ASCE 7-98 Standard
- Load and Resistance Factor Design (LRFD), 3rd edition
- ACI 318-02

The live load is the same as that used in the existing structure and, along with the dead loads, can be found below. The superimposed load was used as a blanket load for such elements as floor tile and other miscellaneous loads. The wall weight would be multiplied by the floor-to-floor height to achieve a load in plf.

Live

- 40 psf residential dwelling units
- 20 psf partitions
- 100 psf corridors

Dead

- 25 psf superimposed
- 50 psf slab: w = (4 in)(150 pcf)(1/12)
- 50 psf wall weight

Total: 75 psf

A combination of RAM steel and hand checks were used to determine the framing of the steel system. It is important to remember that, if used improperly, a program like RAM steel can be a black box that provides sizes which may not be reasonable. Using the LRFD and New Columbia Joist Company manuals, I was able to determine if the output was consistent with the appropriate sizes.

<u>Framing – Steel System</u>

The existing column layout is pretty efficient. Some bays that were originally 15' x 20' were increased to 20' x 20', but little else changed. Exploration into 25' x 25' bays was done, but the depth of the steel members began to become larger than I wanted. There was very little room for altering the exterior columns to preserve the architecture, but three were able to be eliminated. Another two columns were removed from the interior, bringing the total to five.

Once the bays were established, the slab was determined. It needed to carry moment from the live and dead loads as well as fulfill the shrinkage and temperature requirements for reinforcement steel. A live load of 100 psf was used for the corridor and 60 psf for the dwelling units, which included 20 psf for the partition loading. The result was a 4" slab on UFS form decking, which is greater than the minimum slab thickness needed to achieve the required one-hour fire rating. However, as will be seen later in the fire protection section, a fire rated assembly is still needed for the steel members supporting the slab. The reinforcement of the slab will be wire mesh, 44 – W4.0 x 4.0.

With the criteria used above, the joists will be placed at 2' 10" o.c., creating 7 equal bays between joists for a 20' tributary width. Even though the column grid doesn't allow for spans over 20', the long span capabilities of open-web joists are still realized. I wanted to keep the joist and girder depths relatively equal and hold them to around 18", which I was able to do. Most joists are 18K3, though the ones spanning under the corridor are 16K3 @ 3' o.c.

The use of joists in the floor system should not be a problem in terms of plenum space whatsoever. The precast concrete plank served as the finished ceiling in the existing scheme on most levels and there was no plenum space allocated on the residence floors. Most piping or electrical work was run through the walls. Even where a plenum existed on the bookstore floors, the existing beams were 18" in depth. Ductwork and other trades appear below these beams, creating a plenum space of nearly 5' in some areas. Thus, the 18" joists will not be a problem.

Keeping the girders the same maximum depth as the joists (18"), I was able to hold the interior members to a W18 x 35 and smaller. The exterior members are W16 x 31. The members lining the corridor have two different sizes, depending on the orientation of the joists. When it acts as a girder, the resulting size is a W 16 x 31.

However, when it acts as a beam, it is a W 14 x 22. A rough layout of the girders, beams and joists can be seen in Figures 11-13, along with two typical bays.

The final element designed was the columns. The column layout would be the same as that used for the concrete system and can be found at Figure 9 in the concrete redesign section. All of the columns will be W 12 x 72, regardless of the tributary area. There are some variations where the tributary areas or live loads applied might be different, but it is simply easier to pick a uniform column for constructability and connection design. As previously mentioned, three exterior and two interior columns were able to be removed based on the bay resizing.

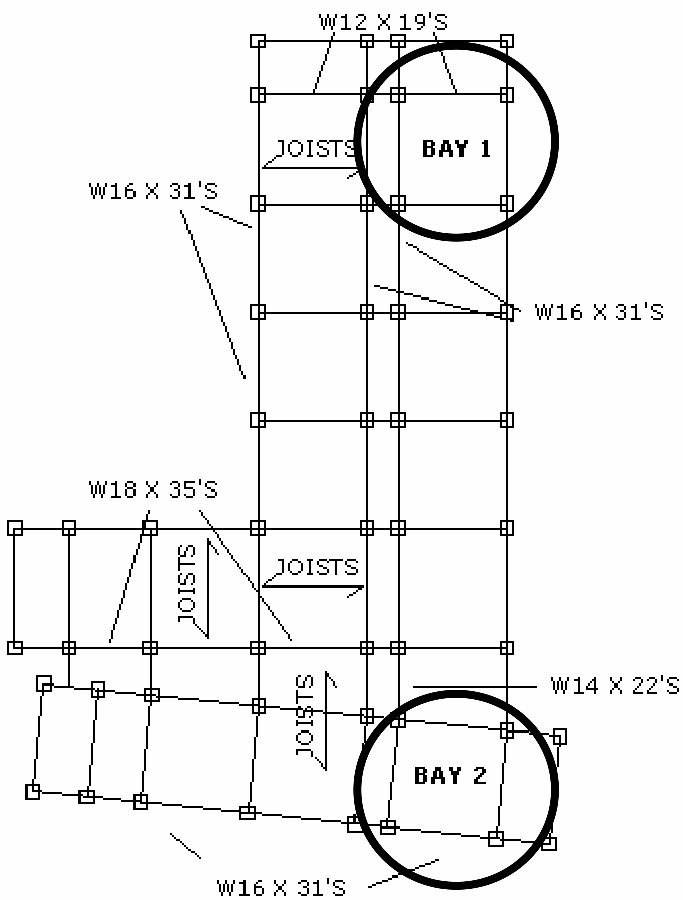


Figure 11: Steel Frame Layout

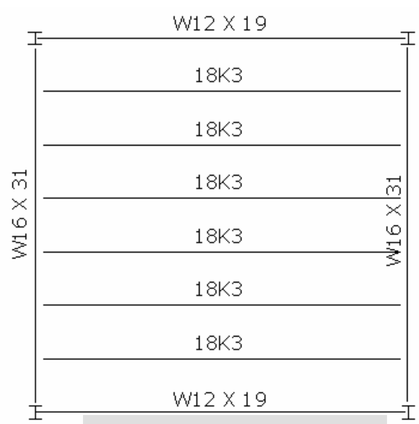


Figure 12: Bay 1

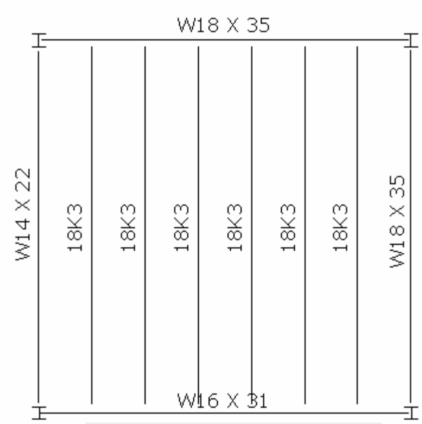


Figure 13: Bay 2

In regards to the floor and building heights, their variation from the existing structure can been seen in Table 6. The lower floors were about the same thickness because the existing and proposed member depths were very similar. However, the upper floors increased in thickness due to the presence of members supporting the concrete slab, instead of simply taking the concrete plank into account. To remain below the local 70' height restriction, the roof slope must be decreased from 9:12 to 7:12. This will result in a decrease of about 5.5' for the present roof height, but should not make a large difference overall. The wooden trusses supporting the roof would need to be designed for a different slope as a result, though.

Floor Height Comparison						
	Existing			**Steel**		
Floor	Floor-to-Ceiling (ft)	Floor Thickness (ft)	Total Height (ft)	Floor-to-Ceiling (ft)	Floor Thickness (ft)	Total Height (ft)
Roof	-	-	19	-	-	13.5
5	8	-	8	8	-	8
4	8	0.67	8.67	8	2.33	10.33
3	8	0.67	8.67	8	2.33	10.33
2	8	3.5	11.5	8	3.42	11.42
1	9	5	14	9	5.17	14.17
		Total	69.84		**Total**	67.75

Roof slope:	
Existing - 9:12	
Steel – 7:12	

Table 6: Floor Height Comparison

<u>Foundations – Steel System</u>

The steel system provides foundation sizes relatively similar to those of the existing building. The live loads are exactly the same and the dead load, due to a lighter slab, is smaller than the original design with the precast concrete plank. Since the column layout was altered slightly, the foundation layout will almost remain the same. Continuous strip footings will still be used along the perimeter of the building with spread and combined footings in the interior. Only one combined footing has been eliminated as a result of the change in the column grid.

In the redesign, the continuous spread footings were not looked at. The existing foundation has footings ranging from 14" to 32" in thickness and 36" to 54" wide. Reinforcement is generally either a #5 or #6 bar spaced at 12" o.c. These sizes are assumed to remain the same due to the similar loads between the original and proposed alternative. Instead, the spread and combined footings were concentrated on, particularly the larger ones, to get a good idea of the sizes that would most likely be used. Some footings were determined to only need steel on the bottom each way, but it is usually specified on the top and bottom to account for uplift forces.

The foundations were checked for a variety of things such as bearing on the soil, shear, and bending. Ductility and shrinkage and temperature steel parameters were also looked at as well as overturning moment. Below is the footing sizes I arrived at supporting the columns specified in Figure 9 on the concrete redesign section. I've detailed the footings I arrived at in Table 7 in addition to the footings that would most likely be used when taking uplift into account. See Appendix D for calculations.

Typical Footings					
		Calculated		Specified	
Column	Size	Depth	Reinf.	Depth	Reinf.
A (spread)	8'-0" x 8'-0"	1'-6"	8 #6 E.W.	2'-4"	8 #6 E.W. (T + B)
B (comb.)	15'-0" x 10'-0"	2'-4"	#9 @ 12" E.W.	2'-4"	#9 @ 12" E.W. (T + B)
C (spread)	11'-0" x 11'-0"	2'-0"	11 #8 E.W.	2'-4"	11 #8 E.W. (T + B)

Table 7: Typical Footings

Lateral – Steel System

Though the shear wall system was preserved, it was investigated to see if some walls could be removed or relocated to be more efficient. Figure 14 shows the framing and shear wall plan of the original design. The darkened lines represent the 20' long shear walls and the numbers within the circles indicate how many bedrooms appear in each apartment. I did this because the shear walls often separated apartments and, by moving some walls around, their capacities may have changed. Figure 15 shows the new shear wall plan. One wall toward the top has been removed, leaving six walls in each direction. Also, by comparing plans, it can be seen which walls were moved around. Though each apartment gained or lost a few feet in terms of their length, the same amount of bedrooms still exists on each floor. The only large change was taking the double and quad apartments near the bottom of the plan and making them two triples.

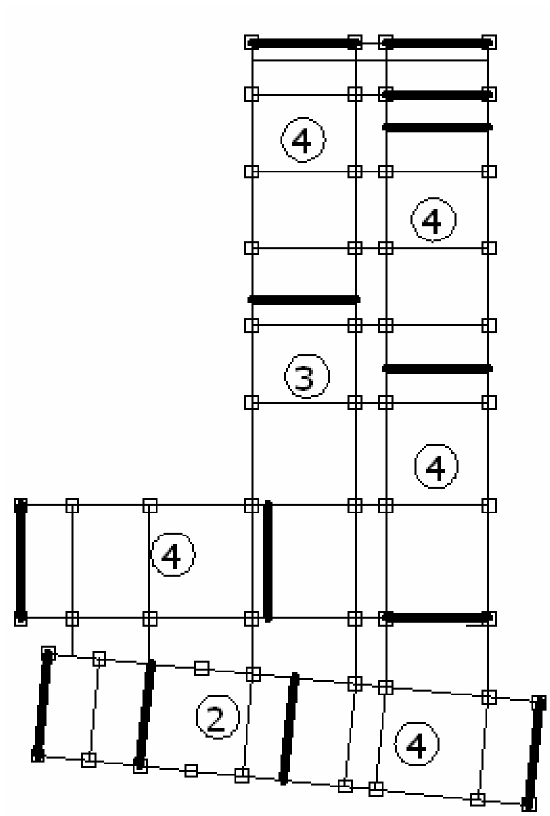


Figure 14: Existing Shear Wall Plan

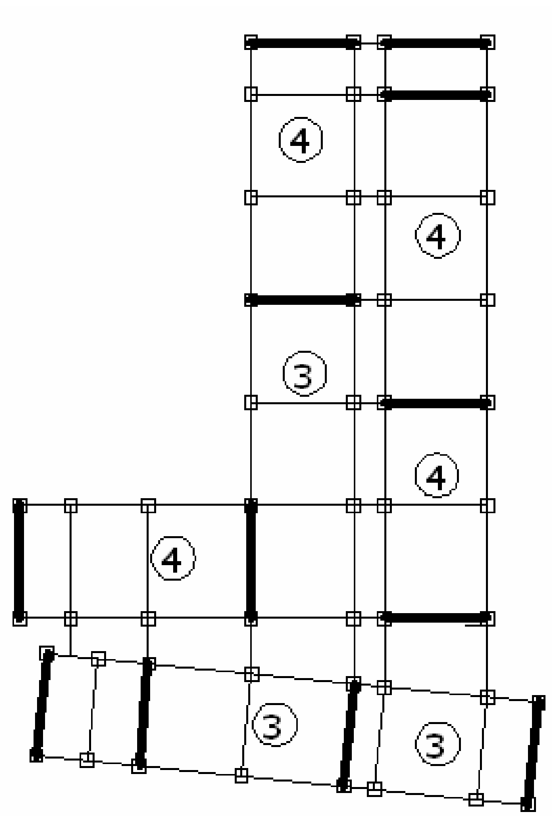


Figure 15: Steel Shear Wall Plan

Wind and seismic loads were calculated for Building C in the North-South and East-West directions. Seismic forces control on nearly every floor (Table 8). For the purpose of analyzing the shear walls, the largest lateral load of the two cases on each floor was taken as the maximum lateral load.

Building C - Lateral Load Comparison				
	North-South		East-West	
Level	Seismic (k)	Wind (k)	Seismic (k)	Wind (k)
Roof (flat)	1.34		1.34	
Roof (gable)	**31.37**	31.20	31.37	**49.92**
5	**71.42**	30.33	**71.42**	48.53
4	**55.30**	31.50	**55.30**	50.40
3	**37.42**	31.89	37.42	**51.02**
2	20.14	**36.66**	20.14	**58.66**
1	0.00	**20.52**	0.00	**32.83**
Total	216.00	182.10	216.00	291.36

Table 8: Lateral load comparisons

The results of the lateral force distribution reveal the performance expected of each shear wall in terms of how the direct and torsion forces affect them. All of the direct forces are consistent because, since all shear walls are the same size and strength, they possess an equal stiffness. Torsion loads are what increase the load on certain shear walls based on their locations relative to the center of rigidity. As expected, those walls farthest from the center of rigidity experience the greatest torsion force. With the torsion forces taken into consideration, the lateral forces on each shear wall vary by a few kips. The largest force experienced by a single shear wall was 32 kips. As for overturning moment, it wasn't a problem since the gravity loads and foundation are adequate compensation and the buildings are relatively short.

Calculations were performed for each shear wall to determine the story drift as well as the overall building drift. The story drifts were compared to an allowable drift of h/400, which is an accepted industry standard based on experimentation. The story drifts were 0.33" and 0.47" in the north-south and east-west directions, respectively. The total drift was less than that allowed, which was 1.64" for the entire structure. In

fact, the drifts appear to be somewhat low, but still reasonable. This could be due to the low floor-to-floor heights, which cause more displacement based on shear, rather than flexure. Also, the placement of numerous shear walls throughout the building may have played a part, rather than just around a core.

Additional calculations were conducted to determine if the shear walls could individually handle the loads placed on them. All were found to be adequate and require 4 - #8 bars at the jambs, having two on each end. There will also be #4 bars at 48" o.c. vertically. All lateral calculations can be found in Appendix E.

Floor Vibration

Floor vibration can dictate the amount of comfort a tenant feels while in his apartment. Noise generated from simply walking across a floor is enough to distract someone in the corridor or apartment below. Plus, a stiffer floor is always desirable, as opposed to one that feels shaky while walking on it.

Concrete systems are the most effective in terms of damping the floor vibration. Precast concrete plank responds well and is constructed to keep the effects at a minimum. The proposed concrete frame alternative is also expected to perform well. As a result, an analysis is rarely performed, if ever, for vibration on a concrete structure.

The open-web steel joist system needs to be analyzed, however, since steel is less reliable in damping vibration effects. The American Institute of Steel Construction published a design guide entitled *Floor Vibrations Due to Human Activity* that was used for the analysis. Two bays were analyzed, one under an apartment and other below the corridor. In both cases, the joists and girders passed the walking excitation criteria and found to be adequate to handle the floor vibrations of the applied live and dead loads. There was no need to check them for floor stiffness because the frequency in each case was found to be below 9 Hz, the maximum allowable frequency before calculations are required. See Appendix F for all calculations.

Fire Protection

Another important area to address in designing a structural system is fireproofing. Both national and local codes detail required fireproofing for various construction elements depending on where they are located in the building.

According to BOCA 1996, dormitories fall under building type R-2 and type 2B construction. Normally, this classification would command a two hour fire rating, but the building is equipped with an automatic sprinkler system. As a result, the required fire resistance rating between separation assemblies can be reduced by one hour.

The floors and walls will be the two main concerns in dealing with the one hour fire rating. The walls were untouched and, therefore, retain their one-hour fire rating. The 3.5" thick slab of the concrete pan joist system alone provides a fire rating of one hour. Had it been any thinner, spray-on fireproofing would have been needed for the underside of the slab between joists or some sort of fire-rated assembly.

The steel joist system, however, cannot command an acceptable fire rating on its own. While the 4" slab is more than adequate for the needed fire rating, the structural steel members do not meet the requirement on their own. Spray-on fireproofing was avoided due to the amount of open-web joists present. Therefore, a fire rated assembly was chosen from the Underwriters Laboratory's 2001 Fire Resistance Directory.

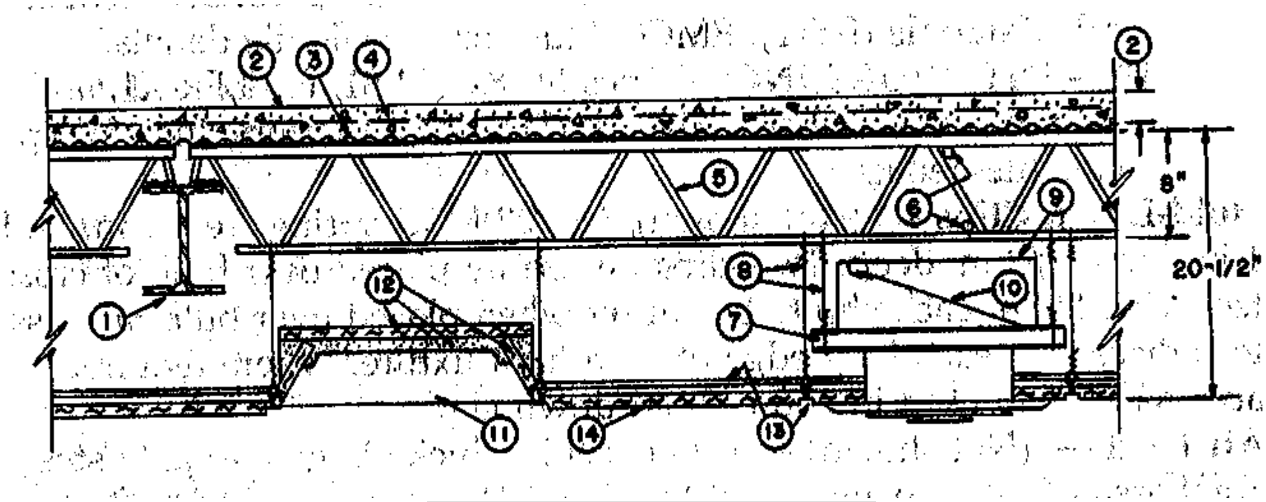


Figure 16: Design No. G205

Design No. G205, as seen in Figure 16, can achieve a rating of 1-3 hours based on a combination of different materials and components. Though it is tough to read the numbers, the basic layout of the assembly can be seen. Some important specifications included are:

5) Steel joists
- The minimum allowable size is 8J2 or 10K1 spaced at a maximum of 48" o.c.

6) Bridging
- 1/2" diameter bars welded to the top and bottom chords of each joist

14) Finished ceiling options
- Acoustical material ranging in size from 24"x 24" to 24" x 48" at 5/8" thick
- Gypsum board at a thickness of 1/2" and a size of 24" x 48"

Of course, there any many options for achieving a one hour fire rating. For Building C, this option can be utilized on the residence floors without the plenum space for lighting fixtures and ductwork. On the floors housing the bookstore, this assembly will provide a finished ceiling as well as space for other trades.

Breadth Study: Acoustics

The acoustical performance of any type of construction is important for buildings housing multiple residents. Sound transmission through walls and floors can often dictate the level of comfort a resident experiences. Since Buildings C is a college residences, acoustics is of particular concern with heightened noise levels from stereos, televisions, and the general rowdiness of students at times.

When analyzing acoustics, two aspects are taken into account, the sound transmission class (STC) and the impact insulation class (IIC). According to M. David Egan's *Architectural Acoustics,* the STC is a rating of the airborne sound transmission loss for a construction element over a frequency range of 125 Hz to 4000 Hz. The IIC is very similar to the STC, giving a rating of the impact sound transmission over the same frequency range. The higher the STC and IIC, the more efficient the construction element will be in reducing airborne and impact sound transmission, respectively. Typically, all elements have an STC rating, while only floor-ceiling assemblies will possess an IIC rating.

According to BOCA 1999, STC and IIC levels must each reach a level of 45. An STC of 45 is equivalent to a stereo playing at a moderate volume. While these are the minimums, a more desirable rating would be around 50 for each qualification. The architectural drawings list the existing CMU walls and partitions with an acceptable STC rating of 52. Molin Concrete Products report a rating of 58 for an 8" hollow core precast plank with vinyl tile, well above the required minimum. The plank also possesses an IIC of 50. The wall assemblies will be maintained with their present acoustical qualities. As a result, only the floor-ceiling construction of the proposed alternatives will be analyzed.

Steel joist system
With a concrete slab of 4" in thickness, only an STC of 44 and an IIC of 25 is attained. Although the STC is close, an acoustical assembly is required to raise these ratings above the BOCA minimums.

Concrete pan joist system
The slab in this system is smaller in thickness at 3.5". I was unable to locate ratings for a slab under 4", most likely because it provides little help with acoustics. It's obvious that an acoustical assembly is required.

Acoustical Floor Solution

A viable solution was found to be produced by the Huff Company, Inc, as seen in Figure 17. According to its website at www.soundcontrol.com, the assembly can achieve an STC of 60, which is above and beyond the BOCA minimum. Also, a rating of this magnitude is approaching a level where construction elements are close to reducing highway traffic noise. The IIC is also quite large at a rating of 59, again exceeding the BOCA recommendations.

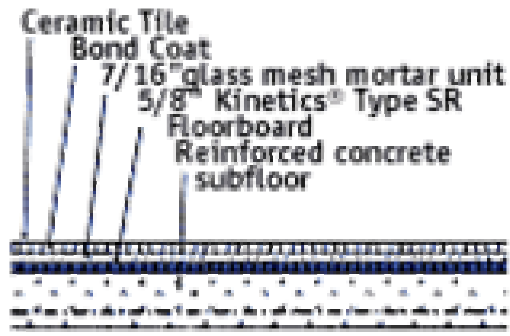


Figure 17: Acoustical Floor Solution

It does not specify a minimum concrete slab thickness, implying that the additional components produce the specified STC and IIC ratings. This is important since the slabs are relatively thin in thickness, especially the 3.5" slab of the concrete pan joist system.

Looking at the individual elements above the slab, the ceramic tile does very little for acoustical performance and can be interchanged with vinyl or other hard surfaced floors. Vinyl would be the most likely floor surface for a residence hall. The bond coat and 7/16" glass mesh mortar unit add a little more acoustical performance than the tile, but the largest gain is in the floorboard below them. The 5/8" Kinetics mortar unit substantially reduces sound transmission through the floor and is most efficient when combined with a hard floor surface, such as that present in the existing and proposed conditions.

Acoustics Comparison		
	STC	IIC
8" concrete plank	58	50
steel joist system	60	59
concrete pan joist system	60	59

Table 9: Acoustics Comparison

Summary

The STC and IIC ratings are either equal or have surpassed that of the existing hollow core precast plank. As a result, the redesigns are fine acoustically and do not need to be altered as long as the above assembly is used. Refer to Table 9 on the left for a comparison of the three systems.

Breadth Study: Construction Management

Cost Comparison

One major aspect of determining constructability is how much the various systems cost in terms of material and labor. It can make or break a project, depending on whether the benefits of something outweigh the costs. In comparing the three systems, a detailed cost estimate was performed for the framing and floor assemblies in question. This involved a material take-off of various materials. It is important to note that the estimates are performed using RS Means 2004, which could harm the accuracy of the numbers. However, it should not be of great consequence since it was used for all three estimates. The goal was for a comparison between the systems, not an intensely accurate assemblies estimate.

In comparing the systems, these criteria were estimated and taken into account:

- Framing: girders, columns, beams, joists
- Floor: precast plank, cast-in-place slab, metal decking, reinforcement
- Formwork: columns, slab, beams, and joists

Other areas of the building that remained unchanged were not estimated. Plus, this is a rough estimate and would fluctuate based on other elements involved, such as an increase or decrease in the foundation size. Calculations are in Appendix G.

Cost Comparison	
Existing	$595,800
Concrete	$774,457
Steel	$564,961

Figure 18: Cost Comparison

The resulting estimate is found in Figure 18. Based on the structural framing, the existing frame comes to a cost of $595, 800. The concrete structure is nearly $200,000 greater at a cost of $774,457. However, the steel structure, at $564.961, comes in over $30,000 cheaper. From this information, it is apparent that the concrete system may not be feasible in terms of cost. Remember, these estimates are for one building in a 5 building complex. Though a parking structure is involved and Buildings A and D are smaller in size, the concrete system may add over $500,000 to the overall project cost. Looking at the steel system, it could result in a savings of around $100,000. Considering the entire project is around $23 million, with Building C commanding around $6 million of that total cost, the steel system is a viable solution. However, the estimations for all three systems could

level a bit with the addition of steel surcharges. The concrete system cannot be entirely ruled out, but is less likely to be accepted based on the cost increase.

<u>Other Issues</u>
Money is not the only issue when choosing a structural system. Weather plays a huge factor, particularly in concrete systems. The outside temperature and other elements can decrease the quality of the concrete pour and delay a schedule.

Pan joists may have the advantage when it comes to deliveries and crane rentals. Long-bed trucks must bring in all of the steel and precast concrete planks, which takes time and could be cumbersome in a quiet, small college environment. Pan joists will still require a large amount of deliveries, but they can most likely be done quicker with less strain on the surroundings. Also, the pan joist system will not require a crane on-site for its framing. In contrast, the proposed steel system will need a crane longer than the existing structure for its steel frame. Still, a crane is needed for the precast concrete plank and, as a result, usage between the two steel systems will somewhat even out.

Time and scheduling also play a large role. As far as lead time, concrete requires none at all, unless precast elements are used. Steel must be decided upon months in advance and be ready for the job's start date. Problems with steel availability can delay a project for days or even weeks. However, once the project is underway, concrete takes much longer to erect, sometime two to three times that of steel. This could offset any advantage gained with steel fabrication lead time and delivery. Formwork is the primary reason for the longer erection time, although pan joist forms are somewhat faster than normal due to their redundancy and reusability.

<u>Formwork Design</u>
Formwork for the pan joist system was designed using Principals and Practices of Commercial Construction. The joist, girder, and slab formwork will be provided by the 40" pans. The column formwork was based on a few assumptions made beforehand. The recommended concrete pour rate is 18' in a 2 hour period. Taking this rate conservatively, I used a pour rate of 7 ft/hr. A concrete temperature of 50° F was also assumed. The result was ½" plywood forms, reinforced with 2 x 4 studs at 12" o.c. and 2 x 4 wales spaced at 18" o.c. This formwork will be removed after a short curing time and reused on every floor. However, the pan forms will need to be duplicated based on a longer curing time, but can still be reused. Calculations are in Appendix H.

Conclusions

Many things were looked at while analyzing the existing system and two proposed alternatives. Each has their advantages and disadvantages, but, in the end, not all of the systems can be equally applicable to Building C.

Which system has the advantage?			
	Existing - steel w/ precast plank	**Steel joists w/ cast-in-place slab**	**Concrete frame and pan joists**
Cost	x	x	
Labor Time	x		
Building Height	x		x
Weight	x	x	
Lead Time			x
Plenum Space	x		x
Lateral System			x
Crane Rental			x
Acoustics	x		x
Fire Protection	x		x
Vibration	x		x

Table 10: Advantages of Each System

Upon analyzing Table 10, it is apparent that the existing structure is a good structural solution. The cost and erection time are reasonable, while keeping the building below height limitations. It provides the needed plenum space for lower floors and is lightweight compared to a concrete building. The existing system also performs well in acoustics, vibration, and fire protection, mostly attributed to the precast concrete plank. Lead time of the steel and plank, as well as the need for a lateral system and a crane on sight, are some disadvantages of the existing system.

The proposed concrete system is also a viable solution, despite the added costs. Its advantages lay in other aspects than the existing system. The system provides a finished ceiling for the apartments, but still keeps the plenum thin enough to prevent any adjustment to the roof slope. It does not require any lead time, crane, or lateral

system. In addition, the concrete performs well in acoustics, fire protection, and vibration and does not require anything extra to enhance its performance in these areas. Still, the labor and cost are higher than the steel systems and increased building weight means larger foundations.

Though the proposed steel system saves a small amount of money compared to the existing system, it is lacking in too many categories. It needs help with acoustics and fire protection, requires changes to the roof slope, and does not perform as well in terms of vibration compared to the other systems. The labor time is increased with the placement of joists and a crane will be required to be on-site longer. There are too many disadvantages and this system must be ruled out as a result.

It appears as though the existing structure may be the best possible solution, though the concrete system is nearly as good. An unsightly crane will not be towering above campus and less room will be needed to store steel and planks on the crowded campus. Lehigh University may prefer the concrete solution and its benefits, if they are willing to spend a little extra money.

References

American Concrete Institute. (2002). *Building Code Requirements for Structural Concrete and Commentary.*

American Institute of Steel Construction, Inc. (2001). *Floor Vibrations Due to Human Activity.*

American Institute of Steel Construction, Inc. (2001). *Manual of Steel Construction – Load and Resistance Factor Design, 3rd Edition.*

American Society of Civil Engineers. (1999). *ASCE Standard 7-98 – Minimum Design Loads for Buildings and Other Structures.* Reston, Virginia.

Andres, Cameron K. (2001). *Principles and practices of commercial construction.* Upper Saddle River, N.J.: Prentice Hall.

Building Officials and Code Administrators International. (1999). BOCA National Building Code 1999.

Das, Braja M. (2004). *Principles of Foundation Engineering: Fifth Edition.* Pacific Grove, CA: Brooks and Cole.

Egan, M. David. (1988). Architectural Acoustics. New York, NY: McGraw Hill.

Gustaferro, Armand H. (2003). *Fire-Resistant Concrete*. http://www.precast.org/publications/mc/TechArticles/02_Spring_FireResistance.html.

Huff Company, Inc, The. http://www.soundcontrol.com/float3.htm

Naeim, Farzad. (2001). The Seismic Design Handbook: 2nd Edition. Norwell, Mass: Kluwer Academic Publishers.

Nicholas J. Bouras, Inc. (2002). *New Columbia Joist Company – Steel Joists and Joist Girders.*

Nilson, Arthur H., David Darwin, and Charles W. Dolan. (2004). *Design of Concrete Structures: Thirteenth Edition.* New York, NY: McGraw Hill.

Taly, Narendra. (2001). *Design of Reinforced Masonry Structures.* New York, NY: McGraw Hill.

Underwriters Laboratory. (1998). *Fire Resistance Directory 1998.*

United Steel Deck, Inc. (2003). *Deck Design Manual.*

APPENDIX A

Members – Concrete System

Assumptions

40" pans
8" rib size
3.5" slab
girder is 1.5 ft wide

Live

- 40 psf residential dwelling units
- 20 psf partitions
- 100 psf corridors

Dead

- 25 psf superimposed
- 44 psf slab: w = (3.5 in)(150 pcf)(1/12)
- 50 psf wall weight

Total: 69 psf

Slab Design

w_u = 1.2 (69 psf) + 1.6 (100 psf) = 0.243 k/ft

$M_u = w_u l_n^2/10$ = [0.243 k/ft (40"/12) ^{2}]/10 = 0.270 ft-k

If #3 bars used, A_s = 0.11 in^2/ft
$a = A_s f_y / (0.85 f'_c b)$ = 0.11 in^2/ft (60 ksi) / (0.85 * 4 ksi * 12 in) = 0.16 in

$\Phi M_n = 0.9 A_s f_y (d - a/2)$ = (0.9)(0.11 in^2/ft)(60 ksi)(1.75 in – 0.16 in/2) = 9.92 in-k/ft
ΦM_n = 9.92 in-k/ft / 12 in = 0.827 ft-k > 0.270 ft-k

Joist Design

$l_u/21$= [(20 ft – 1.5 ft)/21](12 in) = 10.58 in
try 12" deep pans; h = 15.5 in

w_u from slab = 0.243 k/ft^2 * 4 ft = 0.972 k/ft

Rib selfweight = 150 pcf (8 in)(12 in)(1/144) = 0.100 k/ft

Total w_u = 0.972 k/ft + 1.2 (0.100 k/ft) = 1.2 k/ft

L_n = 18.5 ft (1.5' girder is assumed)

<u>Positive moment</u>

Interior span: $M_u = w_u l_n^2 / 16$ = 1.2 k/ft (18.5 ft)^2 / 16 = 25.7 ft-k
End span: $M_u = w_u l_n^2 / 14$ = 1.2 k/ft (18.5 ft)^2 / 14 = 29.4 ft-k

<u>Negative moment</u>
Interior span: $M_u = w_u l_n^2 / 11$ = 1.2 k/ft (18.5 ft)^2 / 11 = 37.4 ft-k
End span: $M_u = w_u l_n^2 / 10$ = 1.2 k/ft (18.5 ft)^2 / 10 = 41.1 ft-k

<u>Positive Reinforcement</u>
Cover = 0.75 in
Assume a #5 bar

d = 15.5 in - 0.75 in - 0.3125 in = 14.44 in (Use 14.5 in)

$A_s = M_u / 4d$ = 25.7 ft-k / (4 * 14.5 in) = 0.45 in^2

Use 2 - #5, A_s = 0.61 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 0.61 in^2 * 60 ksi / (0.85 x 4 ksi x 8 in) = 1.35 in

$\Phi M_n = 0.9 A_s f_y (d – a/2)$ = 0.9(0.61 in^2)(60 ksi)(14.5 in – 1.35 in/2) = 456 in-k

ΦM_n = 456 in-k / 12 = 38.0 ft-k > 29.4 ft-k so it is OK

Ductility Check
c = a / 0.85 = 1.35 in / 0.85 = 1.59 in

ε_s = 0.003 (d - c) / c = 0.003 (14.5 in – 1.59 in) / 1.59 in = 0.025 > 0.005 so it is OK

<u>Negative Reinforcement</u>
d = 15.5 in - 0.75 in - 0.3125 in = 14.44 in (Use 14.5 in)

$A_s = M_u / 4d$ = 37.4 ft-k / (4 * 14.5 in) = 0.65 in^2

Use 2 - #6, A_s = 0.88 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 0.88 in^2 * 60 ksi / (0.85 x 4 ksi x 8 in) = 1.95 in

$\Phi M_n = 0.9 A_s f_y (d – a/2)$ = 0.9(0.88 in^2)(60 ksi)(14.5 in – 1.95 in/2) = 643 in-k
ΦM_n = 643 in-k / 12 = 53.6 ft-k > 41.1 ft-k so it is OK

Ductility Check
c = a / 0.85 = 1.95 in / 0.85 = 2.30 in

$\varepsilon_s = 0.003 (d - c) / c = 0.003 (14.5 \text{ in} – 2.30 \text{ in}) / 2.30 \text{ in} = 0.016 > 0.005$ so it is OK

<u>Shear Design</u>
$V_u = 1.15\ w_u l_n / 2 = 1.15 (1.2 \text{ k/ft})(18.5 \text{ ft}) / 2 = 12.8 \text{ k}$

$V_c = 2 \sqrt{f'_c}\ b\ d = 2 \sqrt{4000 \text{ psi}}\ (8 \text{ in})(14.5 \text{ in}) = 14.7 \text{ k}$

$0.5\ \mathbf{\Phi} V_c = 0.5 * 0.75(14.7 \text{ k}) = 5.5 \text{ k} < 12.8 \text{ k}$
Shear reinforcement is required

$S_{max} = d/2 = 14.5 \text{ in} / 2 = 7.25 \text{ in}$

$A_{vmin} = 50\ b_w\ S / f_y = 50(8 \text{ in})(7.25 \text{ in})/60{,}000 \text{ psi} = 0.049 \text{ in}^2$

Try #3 @ 12 in, $A_v = 0.22 \text{ in}^2$

$V_s = A_v\ f_y\ d / S = 0.22 \text{ in}^2 (60 \text{ ksi})(14.5 \text{ in}) / 7.25 \text{ in} = 26.4 \text{ k}$

$\mathbf{\Phi}(V_c + V_s) = 0.75 (14.7 + 26.4) = 30.9 \text{ k} > 12.8 \text{ k}$

Girder A
Live load
$w_u = 60 \text{ psf } (10 \text{ ft}) = 0.6 \text{ k/ft}$
$w_u = 100 \text{ psf } (10 \text{ ft}) = 1.0 \text{ k/ft}$

Dead load
superimposed = 25 psf (10 ft) = 0.25 k/ft
slab = 150 pcf (3.5 in)(1 ft/12 in)(10 ft) = 0.438 k/ft
joist = 150 pcf (8 in)(12 in)(1/144)(16 ft)(1/4 ft) = 0.4 k/ft
self weight = 150 pcf (1 ft)(1.5 ft) = 0.225 k/ft
wall weight = 50 psf (10 ft) = 0.5 k/ft

20 f t span moments (at face of support)
Supports: $M_u = 142$ ft-k
Midspan: $M_u = 55$ ft-k

<u>Reinforcement</u>
Supports: $M_u = 142$ ft-k
$d = 15.5 \text{ in} - 2 \text{ in} - 0.3125 \text{ in} = 13.19 \text{ in}$ (Use 13 in)

$A_s = M_u / 4d = 142 \text{ ft-k} / (4 * 13 \text{ in}) = 2.73 \text{ in}^2$

Try 5 - #7, $A_s = 3.0 \text{ in}^2$

$a = A_s f_y / (0.85 f'_c b) = 3.0 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 2.95 in$

$\Phi M_n = 0.9 A_s f_y (d – a/2) = 0.9(3.0 in^2)(60 ksi)(13 in – 2.95 in/2) = 1868 in-k$

$\Phi M_n = 1868 in-k / 12 = 156 ft-k > 142 ft-k$ so it is OK

Ductility Check
$c = a / 0.85 = 2.95 in / 0.85 = 3.51 in$

$\varepsilon_s = 0.003 (d - c) / c = 0.003 (13 in – 3.51 in) / 3.51 in = 0.0082 > 0.005$ so it is OK

Midspan: $M_u = 55$ ft-k
$d = 14.5 in - 2 in - 0.3125 in = 13.19 in$ (Use 13 in)

$A_s = M_u / 4d = 55 ft-k / (4 * 13 in) = 1.06 in^2$

Try 4 - #5, $A_s = 1.24 in^2$

$a = A_s f_y / (0.85 f'_c b) = 1.24 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 1.22 in$

$\Phi M_n = 0.9 A_s f_y (d – a/2) = 0.9(1.24 in^2)(60 ksi)(13 in – 1.22 in/2) = 830 in-k$

$\Phi M_n = 830 in-k / 12 = 70 ft-k > 55 ft-k$ so it is OK

Ductility Check
$c = a / 0.85 = 1.22 in / 0.85 = 1.44 in$

$\varepsilon_s = 0.003 (d - c) / c = 0.003 (13 in – 1.44 in) / 1.44 in = 0.024 > 0.005$ so it is OK

<u>Shear Design</u>
$V_u = 34.8$ k (at face of support)

$V_c = 2 \sqrt{f'_c} b d = 2 \sqrt{4000 psi} (18 in)(13 in) = 29.6 k$

$V_{s\,req.} = (V_u / \Phi) - V_c = (34.8 k / 0.75) – 29.6 k = 16.8 k$

$S_{max} = d/2 = 13 in / 2 = 6.5 in$

Try #4 @ 12 in, $A_v = 0.20 in^2$

$V_s = A_v f_y d / S = 0.20 in^2 (60 ksi)(12 in) / 6.5 in = 22.2 k$

Girder B

Live load

w_u = 60 psf (20 ft) = 1.2 k/ft
w_u = 100 psf (20 ft) = 2.0 k/ft

Dead load

superimposed = 25 psf (20 ft) = 0.5 k/ft
slab = 150 pcf (3.5 in)(1 ft/12 in)(20 ft) = 0.875 k/ft
joist = 150 pcf (8 in)(12 in)(1/144)(18.5 ft)(1/4 ft) = 0.463 k/ft
self weight = 150 pcf (1 ft)(1.5 ft) = 0.225 k/ft
wall weight = 50 psf (10 ft) = 0.5 k/ft

20 ft span moments (at face of support)

Supports: M_u = 113 ft-k
Midspan: M_u = 77 ft-k

<u>Reinforcement</u>

Supports: M_u = 183 ft-k
d = 15.5 in - 2 in - 0.3125 in = 13.187 in (Use 13 in)

$A_s = M_u / 4d$ = 183 ft-k / (4 * 13 in) = 3.52 in^2

Try 5 - #8, A_s = 3.95 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 3.52 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 3.95 in

$\Phi M_n = 0.9 A_s f_y (d - a/2)$ = 0.9(3.95 in^2)(60 ksi)(13 in – 3.45 in/2) = 2405 in-k

ΦM_n = 2405 in-k / 12 = 201 ft-k > 183 ft-k so it is OK

Ductility Check

c = a / 0.85 = 3.95 in / 0.85 = 4.65 in

ε_s = 0.003 (d - c) / c = 0.003 (13 in – 4.65 in) / 4.65 in = 0.0054 > 0.005 so it is OK

Midspan: M_u = 87 ft-k
d = 15.5 in - 2 in - 0.3125 in = 13.187 in (Use 13 in)

$A_s = M_u / 4d$ = 87 ft-k / (4 * 13 in) = 1.68 in^2

Try 4 - #6, A_s = 1.76 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 1.76 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 1.74 in

$\Phi M_n = 0.9\ A_s f_y\ (d - a/2) = 0.9(1.76\ in^2)(60\ ksi)(13\ in - 1.74\ in/2) = 1153\ in\text{-}k$

$\Phi M_n = 1153\ in\text{-}k / 12 = 97\ ft\text{-}k > 87\ ft\text{-}k$ so it is OK

Ductility Check
$c = a / 0.85 = 1.74\ in / 0.85 = 2.05\ in$

$\varepsilon_s = 0.003\ (d - c) / c = 0.003\ (13\ in - 2.05\ in) / 2.05\ in = 0.016 > 0.005$ so it is OK

<u>Shear Design</u>
$V_u = 49.8\ k$ (at face of support)

$V_c = 2\ \sqrt{f'_c}\ b\ d = 2\ \sqrt{4000\ psi}\ (18\ in)(12\ in) = 27.4\ k$

$V_{s\ req.} = (V_u / \Phi) - V_c = (49.8\ k / 0.75) - 27.4\ k = 39\ k$

$S_{max} = d/2 = 13\ in / 2 = 6.5\ in$

Try #4 @ 6 in, $A_v = 0.4\ in^2$

$V_s = A_v\ f_y\ d / S = 0.4\ in^2\ (60\ ksi)(12\ in) / 6.5\ in = 44.3\ k$

Girder C
Live load
$w_u = 60\ psf\ (20\ ft) = 1.2\ k/ft$
$w_u = 100\ psf\ (20\ ft) = 2.0\ k/ft$

Dead load
superimposed = 25 psf (20 ft) = 0.5 k/ft
slab = 150 pcf (3.5 in)(1 ft/12 in)(20 ft) = 0.875 k/ft
joist = 150 pcf (8 in)(12 in)(1/144)(18.5 ft)(1/4 ft) = 0.463 k/ft
self weight = 150 pcf (1 ft)(1.5 ft) = 0.225 k/ft

6 ft span moments (at face of support)
Support: $M_u = 157\ ft\text{-}k$
Midspan: $M_u = 10\ ft\text{-}k$

<u>Reinforcement</u>
Support: $M_u = 157\ ft\text{-}k$

$d = 15.5\ in - 2\ in - 0.3125\ in = 13.187\ in$ (Use 13 in)

$A_s = M_u / 4d = 15\ ft\text{-}k / (4 * 13\ in) = 3.02\ in^2$

Try 7 - #6, A_s = 3.08 in^2

a = $A_s f_y$ / (0.85 f'_c b) = 3.08 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 3.02 in

ΦM_n = 0.9 $A_s f_y$ (d – a/2) = 0.9(3.08 in^2)(60 ksi)(13 in – 3.02 in/2) = 1912 in-k

ΦM_n = 1912 in-k / 12 = 160 ft-k > 157 ft-k so it is OK

Ductility Check
c = a / 0.85 = 3.02 in / 0.85 = 3.56 in

ε_s = 0.003 (d - c) / c = 0.003 (12 in – 3.56 in) / 3.56 in = 0.0072 > 0.005 so it is OK

Midspan: M_u = 10 ft-k
d = 15.5 in - 2 in - 0.3125 in = 13.18 in (Use 13 in)

$A_s = M_u$ / 4d = 10 ft-k / (4 * 13 in) = 0.20 in^2

Try 2 - #6, A_s = 0.88 in^2

a = $A_s f_y$ / (0.85 f'_c b) = 0.88 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 0.87 in

ΦM_n = 0.9 $A_s f_y$ (d – a/2) = 0.9(0.88 in^2)(60 ksi)(13 in – 0.87 in/2) = 598 in-k

ΦM_n = 598 in-k / 12 = 50 ft-k > 10 ft-k so it is OK

Ductility Check
c = a / 0.85 = 0.87 in / 0.85 = 1.03 in

ε_s = 0.003 (d - c) / c = 0.003 (13 in – 1.03 in) / 1.03 in = 0.035 > 0.005 so it is OK

<u>Shear Design</u>
V_u = 71.2 k (at face of support)

$V_c = 2 \sqrt{f'_c}$ b d = 2 $\sqrt{4000}$ psi (18 in)(12 in) = 27.4 k

$V_{s\ req.}$ = (V_u / **Φ**) - V_c = (70.5 k / 0.75) – 27.4 k = 66.6 k

S_{max} = d/2 = 13 in / 2 = 6.5 in

Try #5 @ 12 in, A_v = 0.62 in^2

$V_s = A_v f_y$ d / S = 0.62 in^2 (60 ksi)(12 in) / 6.5 in = 68.7 k
Girder D

Live load
w_u = 60 psf (10 ft) = 0.6k/ft
w_u = 100 psf (3 ft) = 0.3 k/ft

Dead load
superimposed = 25 psf (13 ft) = 0.325 k/ft
slab = 150 pcf (3.5 in)(1 ft/12 in)(13 ft) = 0.569 k/ft
joist = 150 pcf (8 in)(12 in)(1/144)(18.5 ft)(1/4 ft) = 0.463 k/ft
self weight = 150 pcf (1 ft)(1.5 ft) = 0.225 k/ft
wall weight = 50 psf (10 ft) = 0.5 k/ft

20 ft span moments (at face of support)
Supports: M_u = 187 ft-k
Midspan: M_u = 66 ft-k

<u>Reinforcement</u>
Supports: M_u = 187 ft-k
d = 15.5 in - 2 in - 0.3125 in = 13.187 in (Use 13 in)

$A_s = M_u / 4d$ = 187 ft-k / (4 * 13 in) = 3.6 in^2

Try 5 - #8, A_s = 3.95 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 3.95 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 3.88 in

$\Phi M_n = 0.9 A_s f_y (d - a/2)$ = 0.9(3.95 in^2)(60 ksi)(13 in – 3.88 in/2) = 2360 in-k

ΦM_n = 2360 in-k / 12 = 197 ft-k > 187 ft-k so it is OK

Ductility Check
c = a / 0.85 = 3.88 in / 0.85 = 4.57 in

ε_s = 0.003 (d - c) / c = 0.003 (13 in – 4.57 in) / 4.57 in = 0.0056 > 0.005 so it is OK

Midspan: M_u = 66 ft-k
d = 15.5 in - 2 in - 0.3125 in = 13.187 in (Use 13 in)

$A_s = M_u / 4d$ = 66 ft-k / (4 * 13 in) = 1.27 in^2

Try 3 - #6, A_s = 1.32 in^2

$a = A_s f_y / (0.85 f'_c b)$ = 1.32 in^2 * 60 ksi / (0.85 x 4 ksi x 18 in) = 1.30 in

$\Phi M_n = 0.9 A_s f_y (d - a/2)$ = 0.9(1.32 in^2)(60 ksi)(13 in – 1.30 in/2) = 881 in-k

ΦM_n = 881 in-k / 12 = 74 ft-k > 66 ft-k so it is OK

Ductility Check
c = a / 0.85 = 1.30 in / 0.85 = 1.53 in

ε_s = 0.003 (d - c) / c = 0.003 (13 in – 1.53 in) / 1.53 in = 0.023 > 0.005 so it is OK

<u>Shear Design</u>
V_u = 43.3 k (at face of support)

$V_c = 2 \sqrt{f'_c}\, b\, d = 2 \sqrt{4000 \text{ psi}}\,(18 \text{ in})(12 \text{ in}) = 27.4$ k

$V_{s\ req.} = (V_u / \Phi) - V_c$ = (43.3 k / 0.75) – 27.4 k = 30.4 k

S_{max} = d/2 = 13 in / 2 = 6.5 in

Try #5 @ 12 in, $A_v = 0.31$ in^2

$V_s = A_v f_y d / S$ = 0.31 in^2 (60 ksi)(12 in) / 6.5 in = 34.4 k

Column Design
18"x 18" Column

<u>Column A (20' x 10' bay – exterior)</u>
Roof Dead load (assume 30 psf dead load + girder self weight)
P_d = 0.3 k/ft (20 ft) = 6 k

Floor Dead load
P_d = 1.32 k/ft (20 ft) = 26.4 k

Wall Dead load
P_d = 0.5 k/ft (20 ft) = 10 k

Roof Live load
P_r = 30 psf (20 ft)(10 ft) = 6 k

Floor Live load
P_l = 60 psf (20 ft)(10 ft) = 12 k

Column Self-weight (by floor)
2 - 18 in (18 in)(13')(0.15)/144 = 4.4 k
3 - 18 in (18 in)(10.167')(0.15)/144 = 3.5 k
4,5,6 - 18 in (18 in)(9.25')(0.15)/144 = 3.2 k

Column A (20' x 10' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	6	3.2	6
5	36.4	3.2	12
4	36.4	3.2	12
3	36.4	3.5	12
2	36.4	4.4	12
Total	151.6	17.5	54

P_u = 1.2 (151.6 + 17.5) + 1.6 (48) + 0.5 (6) = 283 k

<u>Column B (20' x 13' bay – corridor)</u>
Roof Dead load (assume 30 psf dead load + girder self weight)
P_d = 0.39 k/ft (20 ft) = 7.8 k

Floor Dead load
P_d = 1.59 k/ft (20 ft) = 31.8 k

Wall Dead load
P_d = 0.5 k/ft (20 ft) = 10 k

Roof Live load
P_r = 30 psf (20 ft)(13 ft) = 7.8 k

Floor Live load
P_l = 60 psf (20 ft)(10 ft) + 100 psf (20 ft)(3 ft)= 18 k

Column Self-weight (by floor)
2 - 18 in (18 in)(13')(0.15)/144 = 4.4 k
3 - 18 in (18 in)(10.167')(0.15)/144 = 3.5 k
4,5,6 - 18 in (18 in)(9.25')(0.15)/144 = 3.2 k

Column B (20' x 13' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	7.8	3.2	7.8
5	41.8	3.2	18
4	41.8	3.2	18
3	41.8	3.5	18
2	41.8	4.4	18
Total	175	17.5	79.8

P_u = 1.2 (175 + 17.5) + 1.6 (72) + 0.5 (7.8) = 351 k

Column C (20’ x 20’ bay – interior)
Roof Dead load (assume 30 psf dead load + girder self weight)
P_d = 0.6 k/ft (20 ft) = 12 k

Floor Dead load
P_d = 2.07 k/ft (20 ft) = 41.3 k

Wall Dead load
P_d = 0.5 k/ft (20 ft) = 10 k

Roof Live load
P_r = 30 psf (20 ft)(20 ft) = 12 k

Floor Live load
P_l = 60 psf (20 ft)(20 ft) = 24 k

Column Self-weight (by floor)
2 - 18 in (18 in)(13’)(0.15)/144 = 4.4 k
3 - 18 in (18 in)(10.167’)(0.15)/144 = 3.5 k
4,5,6 - 18 in (18 in)(9.25’)(0.15)/144 = 3.2 k

Column C (20' x 20' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	7.8	3.2	12
5	51.3	3.2	24
4	51.3	3.2	24
3	51.3	3.5	24
2	51.3	4.4	24
Total	213	17.5	108

P_u = 1.2 (213 + 17.5) + 1.6 (96) + 0.5 (12) = 437

Check Slenderness
Assume K = 1.0

r = 0.3h = 0.3 (18 in) = 5.4

l_u = 13.833 ft (12 in/ft) + 32 in – 14 in = 183.5 in

k l_u / r = 1.0 (183.5 in)/ 5.4 in = 33.8 < 34
Thus, it is non-slender

PCA Column input
Column A
P_u = 283 * 1.08 = 306 k
Mu = 142 * 1.08 = 154 ft-k

Column B
P_u = 351 * 1.08 = 379 k
Mu = 157 * 1.08 = 170 ft-k

Column C
P_u = 437 * 1.08 = 472 k
Mu = 187 * 1.08 = 202 ft-k

Column A – with live load reduction
$A_I = 4A_T$ = 4 * (20 ft)(10 ft) = 800 ft^2

RF = 0.25 + (15 / $\sqrt{A_I}$) = 0.25 + (15 / $\sqrt{800\ ft^2}$) = 0.78

P_l = 0.78 * 60 psf (20 ft)(10 ft) = 9.4 k

Column A (20' x 10' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	6	3.2	6
5	36.4	3.2	9.4
4	36.4	3.2	9.4
3	36.4	3.5	9.4
2	36.4	4.4	9.4
Total	151.6	17.5	43.6

P_u = 1.2 (151.6 + 17.5) + 1.6 (37.6) + 0.5 (6) = 267 k

Column B – with live load reduction
$A_I = 4A_T$ = 4 * (20 ft)(13 ft) = 1040 ft^2

RF = 0.25 + (15 / $\sqrt{A_I}$) = 0.25 + (15 / $\sqrt{1040\ ft^2}$) = 0.72

P_l = 0.72 * [60 psf (10 ft)(20 ft) + 100 psf (3 ft)(20 ft)] = 13 k

Column B (20' x 13' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	7.8	3.2	7.8
5	41.8	3.2	13
4	41.8	3.2	13
3	41.8	3.5	13

2	41.8	4.4	13
Total	175	17.5	59.8

P_u = 1.2 (175 + 17.5) + 1.6 (52) + 0.5 (7.8) = 319 k

<u>Column C – with live load reduction</u>
$A_I = 4A_T$ = 4 * (20 ft)(20 ft) = 1600 ft^2

RF = 0.25 + (15 / $\sqrt{A_I}$) = 0.25 + (15 / $\sqrt{1600 \text{ ft}^2}$) = 0.625

P_l = 0.625 * [60 psf (20 ft)(20 ft)] = 15 k

Column C (20' x 20' trib)			
Floor	P dead (k)	SW (k)	P live (k)
Roof	7.8	3.2	12
5	51.3	3.2	15
4	51.3	3.2	15
3	51.3	3.5	15
2	51.3	4.4	15
Total	213	17.5	72

P_u = 1.2 (213 + 17.5) + 1.6 (60) + 0.5 (12) = 379 k

PCA Column input
<u>Column A</u>
P_u = 267 * 1.08 = 288 k
Mu = 142 * 1.08 = 154 ft-k

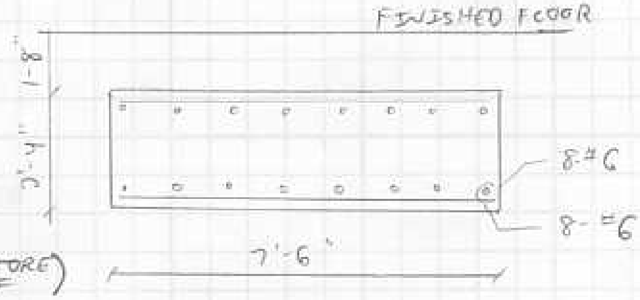
<u>Column B</u>
P_u = 319 * 1.08 = 344 k
Mu = 157 * 1.08 = 170 ft-k

<u>Column C</u>
P_u = 379 * 1.08 = 409 k
Mu = 187 * 1.08 = 202 ft-k

APPENDIX B

FOUNDATIONS

F10
- 7'-6" x 7'-6"
- DEPTH = 2'-4"
- 8-#6's E.W. (T+B)



EXISTING

(A) 20' x 10' TRIB (+ BOOKSTORE ROOF)

FLOOR LIVE: 60 PSF (10 FT)(20 FT) = 12^K
ROOF LIVE: 30 PSF (20 FT)(20 FT) = 12^K
ROOF DEAD: 30 PSF (20 FT)(20 FT) = 12^K
FLOOR DEAD: 94 PSF (10 FT)(20 FT) + 0.225 K/FT (30 FT) = 25.6^K
COLUMN DEAD: 17.5^K
WALL DEAD: 50 PSF (51 FT)(20 FT) = 51^K

LIVE: $4(12^K) + 12^K = 60^K$
DEAD: $4(25.6^K) + 12^K + 68.5^K = 183^K$ $\sim 243^K$

DEAD:
SUPERIMPOSED: 25 PSF
SLAB: 44 PSF
JOISTS: 25 PSF
GIRDER: 0.225 K/FT
COLUMNS: 17.5^K
WALL: 50 PSF

$q_{NET} = 3$ KSF = 21 PSI

$3 \text{ KSF} = \frac{243^K}{BL}$ $\quad B = L = 9'\text{-}0"$

$P_u = 1.2(183^K) + 1.6(60^K) = 316^K$
$q = \frac{316^K}{(9')^2} = 3.90$ KSF (27.1 PSI)

$V_c \leq \phi 4\sqrt{f'_c}$
$\leq 0.75(4)\sqrt{3000}$
≤ 165 PSI

$d^2[V_c + q/4] + d[V_c + q/2]w = q/4[BL - w^2]$
$d^2[165 \text{ PSI} + \frac{27.1 \text{ PSI}}{4}] + d[165 \text{ PSI} + \frac{27.1 \text{ PSI}}{2}](18 \text{ IN}) = \frac{27.1 \text{ PSI}}{4}[(108 \text{ IN})^2 - (18 \text{ IN})^2]$

$d^2(171.8) + d(3214) = 76829$
$d = 13.8$ IN
ASSUME #7's

$h = d + d_b + 3"$
$= 13.8" + 0.875" + 3"$
$= 17.7" \approx 18"$

$d = 18" - 3" - \frac{1}{2}(0.875") = 14.57"$

SHEAR
$V_u = 3.90\text{KSF}\left[\frac{108" - 18"}{2} - 14.57"\right]\frac{1}{12} = 9.9^K$

$\phi V_n = 0.75(2)\sqrt{3000}(12")(14.57") = 14.4^K$

$V_u \leq \phi V_n \therefore$ OK

F10 CONTINUED

BENDING

$\ell = \frac{108\text{ IN} - 18\text{ IN}}{2} = 45\text{ IN}$

$M_u = \frac{q\ell^2}{2} = \frac{3.90\text{ KSF}(3.75\text{ FT})^2}{2} = 27.5\text{ FT·K}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60\text{ KSI})A_s}{0.85(3\text{ KSI})(12\text{ IN})} = 1.96 A_s$

$27.5\text{ FT·K}(12\text{ IN/FT}) = 0.9(60\text{ KSI})A_s(14.57\text{ IN} - \frac{1.96A_s}{2})$

$6.1 = 14.57A_s - 0.98A_s^2$

$A_s \geq 0.44\text{ IN}^2/\text{FT}$

#7 @ 12 IN $\quad A = 0.60\text{ IN}^2/\text{FT}$

$\rho = \frac{0.60\text{ IN}^2/\text{FT}}{(12\text{ IN})(18\text{ IN})} = 0.0028 > 0.0018 \quad \therefore \text{OK}$

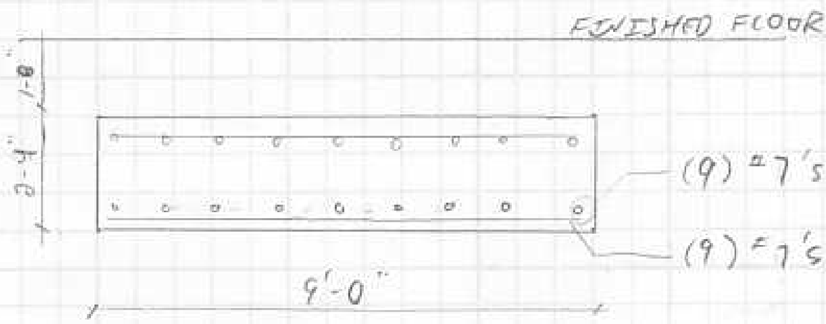
$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.39\text{ IN}}(14.57'' - 1.39'') = 0.029 > 0.005$

$a = 1.96(0.60\text{ IN}^2/\text{FT}) = 1.18\text{ IN} \qquad c = \frac{1.18\text{ IN}}{0.85} = 1.39\text{ IN}$

$S = 12'' \leq 3(14.57'') = 44'' \quad \therefore \text{OK}$

$\leq 18'' \quad \therefore \text{OK}$

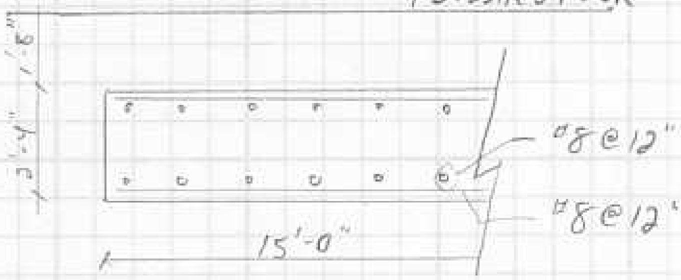
(9) #7's EW



9'-0" × 9'-0"

FOUNDATIONS

Fd2
- 15'-0" x 10'-0"
- DEPTH = 2'-4"
- #8 @ 12" EW (T+B)



EXISTING

(6) 20' x 13' TRIB (CORRIDOR + DORMS)

FLOOR LIVE: 100 PSF (6FT)(20FT) + 60 PSF (20FT)(20FT) = 36^K
ROOF LIVE: 30 PSF (26FT)(20FT) = 15.6^K
ROOF DEAD: 30 PSF (26FT)(20FT) = 15.6^K
FLOOR DEAD: 94 PSF (26FT)(20FT) + 0.225 K/FT (66FT) = 63.8^K
COLUMN DEAD: $2(17.5^K) = 35^K$
WALL DEAD: 50 PSF (51 FT)(40 FT) = 102^K
LIVE: $4(36^K) + 15.6^K = 160^K$
DEAD: $4(63.8^K) + 15.6^K + 137^K = 408^K$ ~ 568^K

DEAD:	
SUPERIMPOSED:	25 PSF
SLAB:	44 PSF
JOISTS:	25 PSF
GIRDER:	0.225 K/FT
COLUMNS:	17.5^K
WALL:	50 PSF

q_{NET} = 3 KSF = 21 PSI

$3 \text{ KSF} = \frac{568^K}{1.5B^2}$ B = 11.3 FT USE B = 12 FT

$3 \text{ KSF} = \frac{568^K}{(12FT)L}$ L = 15.8 FT USE B = 16 FT

$P_u = 1.2(408^K) + 1.6(160^K) = 746^K$
$q = \frac{746^K}{(16)(12)} = 3.89$ KSF (27.0 PSI)

$V_c = 0.75(4)\sqrt{3000}$
$= 165$ PSI

$d^2[165 \text{ PSI} + \frac{27.0 \text{ PSI}}{4}] + d[165 \text{ PSI} + \frac{27.0 \text{ PSI}}{2}](18 \text{ IN}) = \frac{27.0 \text{ PSI}}{4}[(192")(144") - (18")^2]$

$d^2(171.8) + d(3213) = 184437$
d = 24.8 IN
ASSUME #8's

h = 24.8 IN + 1 IN + 3 IN
= 28.8 IN ≈ 30 IN

d_1 = 30 IN − 3 IN − .5(1 IN) = 26.5" LONG
d_2 = 30 IN − 3 IN − 1.5(1 IN) = 25.5" SHORT

F22 CONTINUED

SHEAR

$$V_u = 3.89\text{ KSF}\left[\frac{144'' - 18''}{2} - 25.5''\right]\frac{1}{12} = 12.2^K$$

SHORT

$$\phi V_n = 0.75(2)\sqrt{3000}(12'')(25.5'') = 25.2^K$$

$$V_u = 3.89\text{ KSF}\left[\frac{192'' - 18''}{2} - 26.5''\right]\frac{1}{12} = 19.6^K$$

LONG

$$\phi V_n = 0.75(2)\sqrt{3000}(12'')(26.5'') = 26.2^K$$

BENDING (LONG)

$$l = \frac{192\text{ IN} - 18\text{ IN}}{2} = 87\text{ IN}$$

$$M_u = \frac{3.89\text{ KSF}(7.25\text{ FT})^2}{2} = 102.3\text{ FT}\cdot K$$

$$a = \frac{A_s(60\text{ KSI})}{0.85(3\text{ KSI})(12\text{ IN})} = 1.96 A_s$$

$$102.3\text{ FT}\cdot K(12\text{ IN/FT}) = 0.9(60\text{ KSI})A_s\left(26.5\text{ IN} - \frac{1.96A_s}{2}\right)$$

$$22.8 = 26.5A_s - 0.98A_s^2$$

$$A_s \geq 0.89\text{ IN}^2$$

#9's @12" $A_s = 1.0\text{ IN}^2$

$$\rho = \frac{1.0\text{ IN}^2}{(12\text{ IN})(30\text{ IN})} = 0.0028 > 0.0018 \therefore OK$$

$$\varepsilon_s = \frac{0.003(d-c)}{c} = \frac{0.003}{2.31\text{ IN}}(26.5\text{ IN} - 2.31\text{ IN}) = 0.032 > 0.005 \quad \therefore \phi = 0.9$$

$$a = 1.96(1.0\text{ IN}^2) = 1.96\text{ IN} \qquad c = \frac{1.96\text{ IN}}{0.85} = 2.31\text{ IN}$$

E22 CONTINUED

BENDING (SHORT)

$l = \frac{144'' - 18''}{2} = 63\ \text{IN}$

$M_u = \frac{3.89\ \text{KSF}(5.25\ \text{FT})^2}{2} = 53.7\ \text{FT·K}$

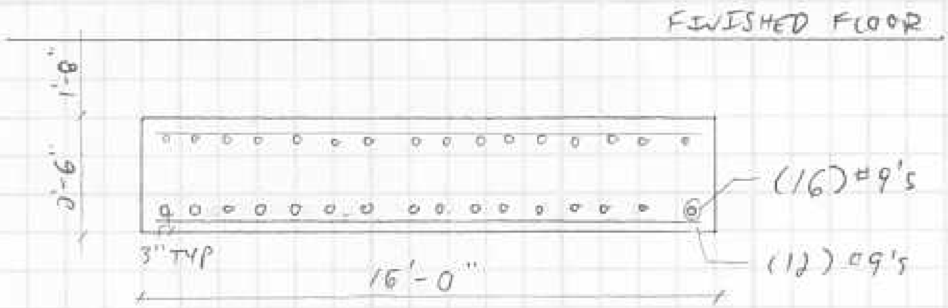
$a = 1.96 A_s$

$53.7\ \text{FT·K}(12\ \text{IN/FT}) = 0.9(60\ \text{KSI}) A_s (25.5\ \text{IN} - \frac{1.96 A_s}{2})$

$12 = 25.5 A_s - 0.98 A_s^2$

$A_s = 0.48\ \text{IN}^2$

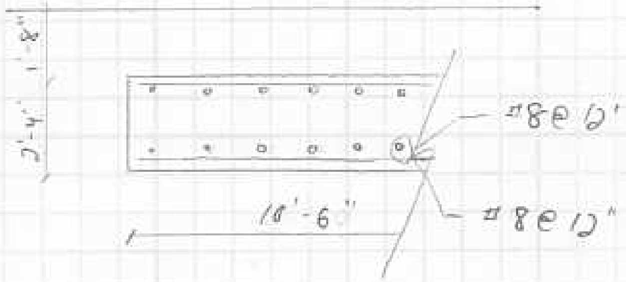
#9's @ 12 IN



16'-0" × 12'-0"

FOUNDATIONS

F16
- $10'-6" \times 10'-6"$
- DEPTH : $2'-4"$
- #8 @ 12" EW (T+B)



EXISTING

C 20' × 20' TRIB

FLOOR LIVE : 60 PSF (20 FT)(20 FT) = 24^K
ROOF LIVE : 30 PSF (20 FT)(20 FT) = 12^K
ROOF DEAD : 30 PSF (20 FT)(20 FT) = 12^K
FLOOR DEAD : 94 PSF (20 FT)(20 FT) + 0.225 ^K/FT (40 FT) = 46.6^K
COLUMN DEAD : 17.5^K
WALL DEAD : 50 PSF (51 FT)(30 FT) = 76.5^K
LIVE : 4(24^K) + 12^K = 108^K
DEAD : 4(46.6^K) + 12^K + 94^K = 293^K ~ 401^K

DEAD:
SUPERIMPOSED : 25 PSF
SLAB : 44 PSF
JOISTS : 25 PSF
GIRDER : 0.225 ^K/FT
COLUMNS : 17.5^K
WALL : 50 PSF

$q_{NET} = 3\text{ KSF} = 21\text{ PSI}$

$3\text{ KSF} = \dfrac{401^K}{BL}$ $\qquad B = L = 12\text{ FT}$

$P_u = 1.2(293) + 1.6(108^K) = 525^K$

$q = \dfrac{525^K}{(12')(12')} = 3.65\text{ KSF} \quad (25.4\text{ PSI})$

$V_c = 0.75(4)\sqrt{3000}$
$\doteq 165\text{ PSI}$

$d^2\left[165\text{ PSI} + \dfrac{25.4\text{ PSI}}{4}\right] + d\left[165\text{ PSI} + \dfrac{25.4\text{ PSI}}{2}\right](18\text{ IN}) = \dfrac{25.4\text{ PSI}}{4}\left[(144")(144") - (18")^2\right]$

$d^2(171.4) + d(3199) = 129617$
$d = 19.7"$
ASSUME #8's

$h = 19.7" + 3" + 1"$
$= 23.7" \approx 24"$

$d = 24" - 3" - \frac{1}{2}(1") = 20.5"$

SHEAR

$V_u = 3.65\text{ KSF}\left[\dfrac{144" - 18"}{2} - 20.5"\right]\dfrac{1}{12} = 13^K$

$\phi V_n = 0.75(2)\sqrt{3000}(12")(20.5") = 20.2^K$

$V_u < \phi V_n \therefore OK$

F23 CONTINUED

BENDING

$l = \frac{144 \text{ IN} - 18 \text{ IN}}{2} = 63 \text{ IN}$

$M_u = \frac{3.65 \text{ KSF} (5.25 \text{ FT})^2}{2} = 50.4 \text{ FT}\cdot\text{K}$

$a = 1.96 A_s$

$50.4 \text{ FT}\cdot\text{K} (12 \text{ IN/FT}) = 0.9 A_s (60 \text{ KSI})(20.5'' - \frac{1.96 A_s}{2})$

$11.2 = 20.5 A_s - 0.98 A_s^2$

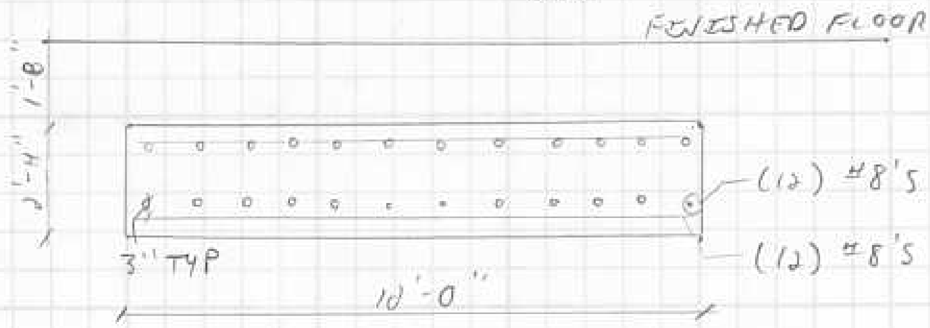
$A_s = 0.57 \text{ IN}^2$

#8 @ 12" $A_s = 0.79 \text{ IN}^2$

$\rho = \frac{0.79 \text{ IN}^2}{(12 \text{ IN})(24 \text{ IN})} = 0.0028 > 0.0018 \therefore \text{OK}$

$\varepsilon_s = \frac{0.003}{1.83''}(20.5'' - 1.83'') = 0.03 > 0.005 \therefore \text{OK}$

$a = 1.96(0.79 \text{ IN}^2) = 1.55 \text{ IN}$ $c = \frac{1.55 \text{ IN}}{0.85} = 1.83 \text{ IN}$



12'-0" x 12'-0"

APPENDIX C

Concrete lateral check

floor	trib height (ft)	height (ft)	total ht (ft)	V (k)	trib ratio	new V (k)	Σ V (k)
5	9.30	9.33	51.00	73.60	0.14	10.30	10.30
4	9.26	9.25	41.67	56.08	0.14	7.85	18.16
3	9.72	9.25	32.42	46.66	0.14	6.53	24.69
2	11.60	10.17	23.17	55.68	0.14	7.80	32.48
1		13.00					

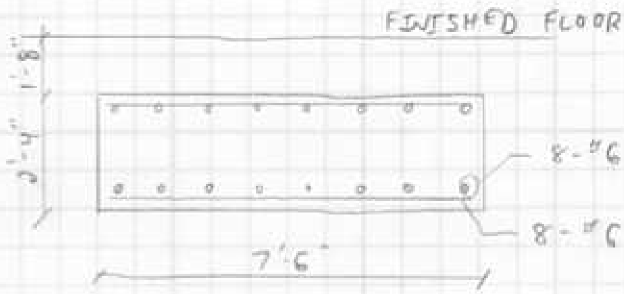
floor	$\Sigma (I_g/L_g)$ (in^3)	$\Sigma (I_c/L_c)$ (in^3)	Δ_{bi} (in)	bent disp. (in)	Δ_{allow} (in)
5	74.32	235.16	0.052	0.515	1.530
4	74.64	236.18	0.090	0.463	1.250
3	71.11	225.00	0.129	0.373	0.973
2	59.59	188.53	0.244	0.244	0.695

	size	I (in^4)
column	18" x 18"	8748
girder	12" x 18"	2592

APPENDIX D

FOUNDATIONS

F10
- 7'-6" × 7'-6"
- DEPTH = 2'-4"
- 8 #6's EW (T+B)



EXISTING

(A) 20' × 10' TRIB (+ BOOKSTORE ROOF)

FLOOR LIVE LOAD: 60 PSF (10 FT)(20 FT) = 12^K
FLOOR DEAD LOAD: 75 PSF (10 FT)(20 FT) = 15^K

ROOF LIVE LOAD: 30 PSF (20 FT)(20 FT) = 12^K
ROOF DEAD LOAD: 30 PSF (20 FT)(20 FT) = 12^K

WALL DEAD: 50 PSF (54 FT)(20 FT) = 54^K

LIVE: $4(12^K) + 12^K = 60^K$
DEAD: $4(15^K) + 12^K + 54^K = 126^K$ ~ 186^K

DEAD:
SUPERIMPOSED: 25 PSF
SLAB: 50 PSF
WALL: 50 PSF

q_{NET} = 3 KSF

$3\text{ KSF} = \frac{186^K}{BL}$ $B = L = 8'\text{-}0"$

$P_u = 1.2(126^K) + 1.6(60^K) = 248^K$

$q = \frac{248^K}{(8')^2} = 3.87\text{ KSF}$ (26.9 PSI)

$V_c \leq \phi 4\sqrt{f'_c}$
$\leq 0.75(4)\sqrt{3000}$
$\leq 165\text{ PSI}$

$d^2[V_c + q/4] + d[V_c + q/2]w = q/4[BL - w^2]$

$d^2[165\text{ PSI} + \frac{26.9\text{ PSI}}{4}] + d[165\text{ PSI} + \frac{26.9\text{ PSI}}{2}](12") = \frac{26.9\text{ PSI}}{4}[(96\text{ IN})^2 - (12\text{ IN})^2]$

$d^2(171.8) + d(2142) = 61010$

$d = 13.7\text{ IN}$

ASSUME #6's

$h = d + d_b + 3"$
$= 13.7" + 0.750" + 3"$
$= 17.5" \approx 18"$

$d = 18" - 3" - \frac{1}{2}(0.750") = 14.63"$

SHEAR

$V_u = 3.87\text{ KSF}\left[\frac{96" - 12"}{2} - 14.63"\right]\frac{1}{12} = 8.9^K$

$\phi V_n = 0.75(2)\sqrt{3000}(12")(14.63") = 14.5^K$

$V_u < \phi V_n$ ∴ OK

F10 CONTINUED

BENDING

$l = \frac{96\text{ IN} - 12\text{ IN}}{2} = 42\text{ IN}$

$M_u = \frac{q l^2}{2} = \frac{3.87\text{ KSF }(3.5\text{ FT})^2}{2} = 23.7\text{ FT}\cdot\text{K}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{60\text{ KSI } A_s}{0.85(3\text{ KSI})(12\text{ IN})} = 1.96 A_s$

$23.7\text{ FT}\cdot\text{K}(12\text{ IN/FT}) = 0.9(60\text{ KSI})A_s(14.63\text{ IN} - \frac{1.96A_s}{2})$

$5.27 = 14.63A_s - 0.98A_s^2$

$A_s = 0.37\text{ IN}^2/\text{FT}$

#6'S @ 12 IN $\quad A = 0.44\text{ IN}^2/\text{FT}$

$\rho = \frac{0.44\text{ IN}^2/\text{FT}}{(12\text{ IN})(18\text{ IN})} = 0.0021 > 0.0018 \quad \therefore \text{OK}$

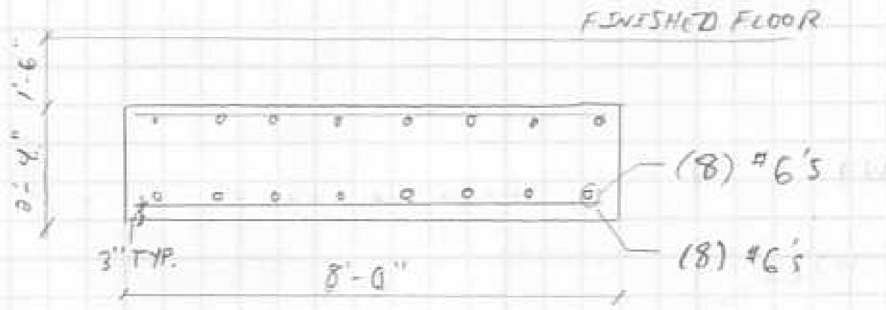
$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.02''}(14.63'' - 1.02'') = 0.040 > 0.005 \quad \therefore \phi = 0.9$

$a = 1.96(0.44) = 0.87'' \qquad c = \frac{0.87''}{0.85} = 1.02''$

$S = 12'' \le 3(14.63'') = 44'' \quad \therefore \text{OK}$

$\le 18 \quad \therefore \text{OK}$

(8) #6'S EW

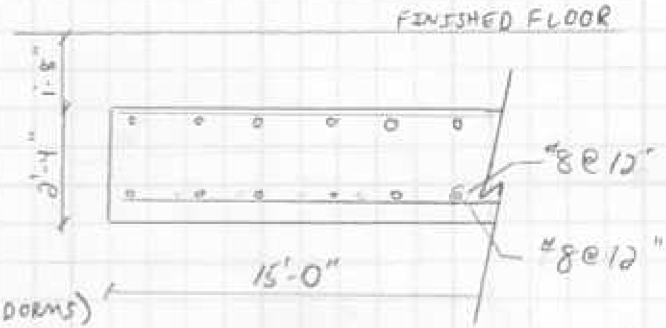


8'-0" x 8'-0"

FOUNDATIONS

F22

- 15'-0" x 10'-0"
- DEPTH = 2'-4"
- #8 @ 12" EW



EXISTING

(B) 20' x 13' TRIB (CORRIDOR + DORMS)

FLOOR LIVE LOAD: 100 PSF (6 FT)(20 FT) + 60 PSF (20 FT)(20 FT) = 36^K

DEAD LOAD: 75 PSF (26 FT)(20 FT) = 39^K

DEAD:

SUPERIMPOSED:	25 PSF
SLAB:	50 PSF
WALL:	50 PSF

ROOF LIVE LOAD: 30 PSF (26 FT)(20 FT) = 15.6^K

DEAD LOAD: 30 PSF (26 FT)(20 FT) = 15.6^K

WALL DEAD: 50 PSF (54 FT)(40 FT) = 108^K

LIVE: $4(36^K) + 15.6^K = 160^K$

DEAD: $4(39^K) + 15.6^K + 108^K = 280^K$ $\sim 440^K$

$q_{NET} = 3$ KSF

$3\text{ KSF} = \frac{440^K}{1.5B^2}$ $B = 9.9$ FT USE $B = 10$ FT

$3\text{ KSF} = \frac{440^K}{(10)L}$ $L = 14.7$ FT USE $L = 15$ FT

$P_u = 1.2(280^K) + 1.6(160^K) = 592^K$

$q = \frac{592^K}{(10)(15)} = 3.95\text{ KSF}$ (27.4 PSI)

$V_c = \phi 4\sqrt{f'_c}$
$= 0.75(4)\sqrt{3000}$
$= 165$ PSI

$d^2[V_c + q/4] + d[V_c + q/2]w = q/4[BL - w^2]$

$d^2[165\text{ PSI} + \frac{27.4\text{ PSI}}{4}] + d[165\text{ PSI} + \frac{27.4\text{ PSI}}{2}](12") = \frac{27.4\text{ PSI}}{4}[(120")(180") - (12")^2]$

$d^2(171.9) + d(2145) = 146974$

$d = 23.7$ in

ASSUME #8's

$h = d + d_b + 3"$
$= 23.7" + 1" + 3"$
$= 27.7" \approx 28"$

$d_1 = 28" - 3" - \frac{1}{2}(1") = 24.5"$ LONG

$d_2 = 28" - 3" - 1.5(1") = 23.5"$ SHORT

F22 CONTINUED

SHEAR

$V_u = 3.95KSF\left[\frac{60"-12"}{2} - 23.5"\right]\frac{1}{12} = 0.2^K$

SHORT

$\phi V_n = 0.75(2)\sqrt{3000}(12")(23.5") = 23.2^K$

$V_u = 3.95KSF\left[\frac{90"-12"}{2} - 24.5"\right]\frac{1}{12} = 4.8^K$

LONG

$\phi V_n = 0.75(2)\sqrt{3000}(12")(24.5") = 18.4^K$

BENDING (LONG)

$\ell = \frac{180"-12"}{2} = 84\ IN$

$M_u = \frac{q\ell^2}{2} = \frac{3.95KSF(7.0FT)^2}{2} = 96.8\ FT\cdot K$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s(60KSI)}{0.85(3KSI)(12IN)} = 1.96 A_s$

$96.8FT\cdot K(12IN/FT) = 0.9(60KSI)A_s(24.5IN - \frac{1.96A_s}{2})$

$21.5 = 24.5A_s - 0.98A_s^2$

$A_s \geq 0.91\ IN^2$

#9's @ 12" $A = 1.0\ IN^2$

$\rho = \frac{1.0 IN^2}{(12IN)(28IN)} = 0.0030 > 0.0018 \therefore OK$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{2.31IN}(24.5IN - 2.31IN) = 0.029 > 0.005 \therefore \phi = 0.9$

$a = 1.96(1.0\ IN^2) = 1.96\ IN$ $c = \frac{1.96 IN}{0.85} = 2.31\ IN$

F22 CONTINUED

BENDING (SHORT)

$\ell = \frac{120" - 12"}{2} = 54 \text{ IN}$

$M_u = \frac{q\ell^2}{2} = \frac{3.95 \text{KSF} (4.5 \text{ FT})^2}{2} = 40 \text{ FT}\cdot\text{K}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s (60 \text{KSI})}{0.85 (3 \text{KSI})(12 \text{ IN})} = 1.96 A_s$

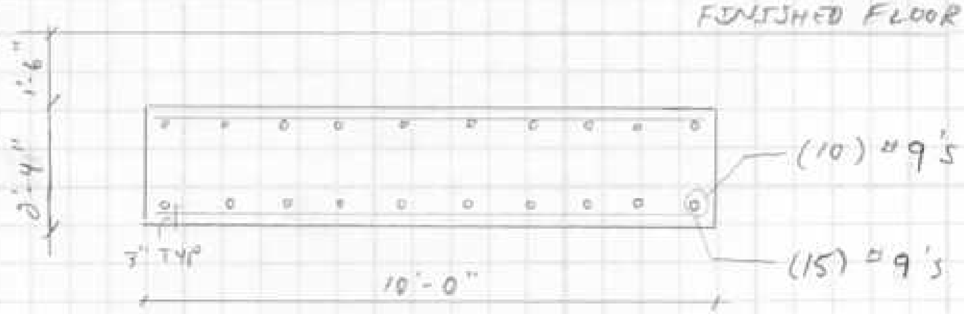
$40 \text{ FT}\cdot\text{K} (12 \text{ IN/FT}) = 0.9 (60 \text{ KSI}) A_s (25.5 \text{ IN} - \frac{1.96 A_s}{2})$

$8.89 = 25.5 A_s - 0.98 A_s^2$

$A_s \geq 0.36 \text{ IN}^2$

#9's @ 12"

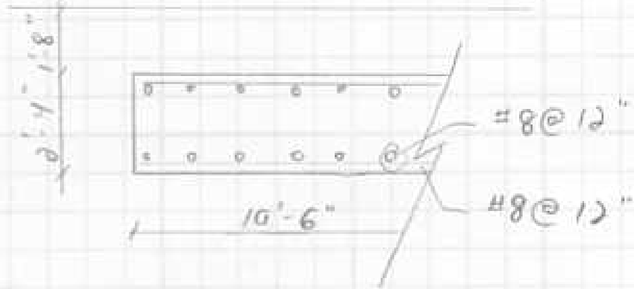
FINISHED FLOOR



15' x 10'

FOUNDATIONS

F1G
- 10'-6" × 10'-6"
- DEPTH: 2'-4"
- #8 @ 12" EW (T+B)



EXISTING

© 20' × 20' TRIB

FLOOR LIVE: 60 PSF(20 FT)(20 FT) = 24^K
DEAD: 75 PSF(20 FT)(20 FT) = 30^K

DEAD:
SUPERIMPOSED: 25 PSF
SLAB: 50 PSF
WALL: 50 PSF

ROOF LIVE: 30 PSF(20 FT)(20 FT): 12^K
DEAD: 30 PSF(20 FT)(20 FT): 12^K
WALL DEAD: 50 PSF(54 FT)(30 FT) = 81^K
LIVE: $4(24^K) + 12^K = 108^K$
DEAD: $4(30^K) + 12^K + 81^K = 213^K$ ~ 321^K

$q_{NET} = 3$ KSF

$3 \text{ KSF} = \frac{321^K}{LB}$ $B = L = 11 FT$

$P_u = 1.2(213^K) + 1.6(108^K) = 429^K$

$q = \frac{429^K}{(11')(11')} = 3.55 \text{ KSF}$ (24.7 PSI)

$V_c = \phi 4\sqrt{f'_c}$
$= 0.75(4)\sqrt{3000}$
$= 165$ PSI

$d^2\left[165 \text{ PSI} + \frac{24.7 \text{ PSI}}{4}\right] + d\left[165 \text{ PSI} + \frac{24.7 \text{ PSI}}{2}\right](12 \text{ IN}) = \frac{24.7 \text{ PSI}}{4}\left[(132")(132") - (12")^2\right]$

$d^2(171.2) + d(2129) = 106704^K$

$d = 19.6$ IN

ASSUME #8's

$h = d + d_b + 3"$
$= 19.6" + 1" + 3"$
$= 23.6" \approx 24"$

$d = 24" - 3" - \frac{1}{2}(1") = 20.5"$

SHEAR

$V_u = 3.55 \text{ KSF}\left[\frac{132" - 12"}{2} - 20.5"\right]\frac{1}{12} = 11.7^K$

$\phi V_n = 0.75(2)\sqrt{3000}(12")(20.5") = 20.2^K$

$V_u < \phi V_n \therefore$ OK

F23 CONTINUED

BENDING

$l = \frac{132" - 12"}{2} = 60$ IN

$M_u = \frac{q l^2}{2} = \frac{3.55\ KSF\ (4.5\ FT)^2}{2} = 36$ FT·K

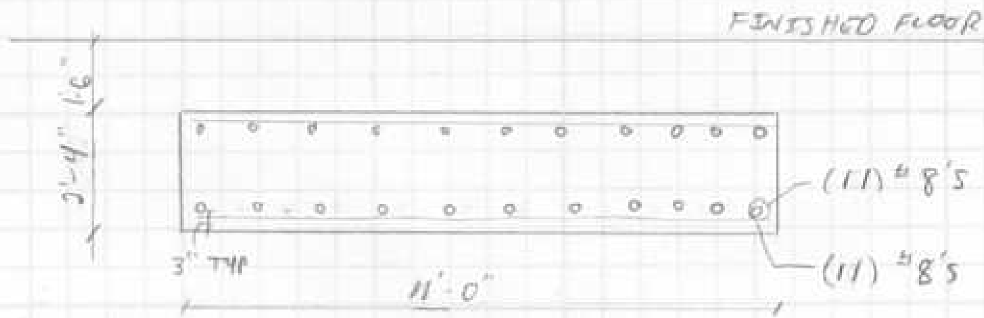
$a = 1.96 A_s$

$36\ FT \cdot K\ (12\ IN/FT) = 0.9(60\ KSI) A_s (19.5\ IN - \frac{1.96 A_s}{2})$

$8" = 19.5 A_s - 0.98 A_s^2$

$A_s \geq 0.42\ IN^2$

#7 @ 12 IN $A_s = 0.60\ IN^2$



11'x11'

$\rho = \frac{0.60\ IN^2}{(12\ IN)(24\ IN)} = 0.0021 > 0.0018$ ∴ OK

$\varepsilon_s = \frac{0.03}{1.39"}(20.5" - 1.39") = 0.042 > 0.005$ ∴ OK

$a = 1.96(0.60\ IN^2) = 1.18$ IN $\quad c = \frac{1.18\ IN}{0.85} = 1.39$ IN

APPENDIX E

Lateral – Steel System

Minimum height-to- length ratio

Building C: North to South: 8.67'/20' = 0.44 > 0.3
East to West: 0.44 > 0.3

Maximum height-to- length ratio

Building C: North to South: 13.67'/20' = 0.69 > 0.3
East to West: 0.69 > 0.3

Center of Rigidity

Building C:
x = [2(20')(0') + 20'(20') + 20'(55') + 20'(60') + 20'(96')] / [20'(6)]
= 39' from left
y = [20'(35') + 20'(75') + 20'(95') + 20'(135') + 2(20')(145') / [(20')(6)]
= 105' from bottom

Center of Mass

Building C:
x = 98'/2 = 49'
y = 146'/2 = 73'

Eccentricity

Building C:
x = 49' – 39' = 10'
y = 105' – 73' = 32'

Lateral – Steel System

Lateral Force Distribution										
Building C - floors 3-5						**North-South**				
				Shear wall	1	2	3	4	5	6
				%P (F_{tor})	0.0122	0.0122	0.0060	0.0050	0.0066	0.0179
Level	P (k)	%P (F_{dir})	F_{direct} (k) (per wall)		$F_{torsion}$ (k)					
Roof	31.37									
5	71.42	0.167	11.93		0.87	0.87	0.43	0.36	0.47	1.28
4	55.30	0.167	9.24		0.67	0.67	0.33	0.28	0.36	0.99
3	37.42	0.167	6.25		0.46	0.46	0.22	0.19	0.25	0.67
2	36.66									
1	20.52									
			27.41	Total	2.00	2.00	0.98	0.82	1.08	2.94
				F_{dir} + F_{tor}	29.41	29.41	28.39	28.23	28.49	30.35
Building C - floors 3-5						**East-West**				
				Shear wall	7	8	9	10	11	12
				%P (F_{tor})	0.0422	0.0181	0.0060	0.0181	0.0241	0.0241
Level	P (k)	%P (F_{dir})	F_{direct} (k) (per wall)		$F_{torsion}$ (k)					
Roof	49.92									
5	71.42	0.167	11.93		3.01	1.29	0.43	1.29	1.72	1.72
4	55.30	0.167	9.24		2.33	1.00	0.33	1.00	1.33	1.33
3	51.02	0.167	8.52		2.15	0.92	0.31	0.92	1.23	1.23
2	58.66									
1	32.83									
			29.68	Total	7.50	3.22	1.07	3.22	4.28	4.28
				F_{dir} + F_{tor}	32.92	28.64	26.49	28.64	29.70	29.70
$F_{direct} = P * (\%P (F_{dir}))$										
$F_{torsion} = P * (\%P (F_{tor}))$										

F_{direct} and $F_{torsion}$						
Building C - floors 3-5						
Wall	Direction	%P (F_{direct})	e (ft)	d_i (ft)	$d_i/(\sum d_i)^2$ (1/ft)	%P ($F_{torsion}$)
1	N-S	0.167	10	39.00	0.0012	0.0122
2	N-S	0.167	10	39.00	0.0012	0.0122
3	N-S	0.167	10	19.00	0.0006	0.0060
4	N-S	0.167	10	16.00	0.0005	0.0050
5	N-S	0.167	10	21.00	0.0007	0.0066
6	N-S	0.167	10	57.00	0.0018	0.0179
			Total	191.00		
7	E-W	0.167	32	70.00	0.0013	0.0422
8	E-W	0.167	32	30.00	0.0006	0.0181
9	E-W	0.167	32	10.00	0.0002	0.0060
10	E-W	0.167	32	30.00	0.0006	0.0181
11	E-W	0.167	32	40.00	0.0008	0.0241
12	E-W	0.167	32	40.00	0.0008	0.0241
			Total	220.00		
F_{direct} = P/n					N-S: n = 6	
$F_{torsion} = [d_i/(\sum d_i)^2]*(P*e)$					E-W: n = 6	

Drift									
Building C (North-South)									
Level	P (k)	h (ft)	E (psi)	I (ft^4)	A_w (ft^2)	$[(P*h^3)/(3*E*I)]*83.33$ (in)	$((2.78*P*h)/(A_w*E))*83.33$ (in)	Δ_i (in)	h/400 (in)
Roof	31.37	9.33	1350000	447	13.4	0.0082	0.0262	0.0344	0.28
5	71.42	10.5	1350000	447	13.4	0.0266	0.0672	0.0939	0.32
4	55.30	10.5	1350000	447	13.4	0.0206	0.0520	0.0727	0.32
3	37.42	10.75	1350000	447	13.4	0.0150	0.0361	0.0510	0.32
2	36.66	13.67	1350000	447	13.4	0.0302	0.0449	0.0751	0.41
					Total	0.1006	0.2265	0.3271	1.64
Building C (East-West)									
Level	P (k)	h (ft)	E (psi)	I (ft^4)	A_w (ft^2)	$[(P*h^3)/(3*E*I)]*83.33$ (in)	$((2.78*P*h)/(A_w*E))*12$ (in)	Δ_i (in)	h/400 (in)
Roof	49.92	9.33	1350000	447	13.4	0.0149	0.0477	0.0626	0.28
5	71.42	10.5	1350000	447	13.4	0.0304	0.0768	0.1073	0.32
4	55.30	10.5	1350000	447	13.4	0.0236	0.0595	0.0831	0.32
3	51.02	10.75	1350000	447	13.4	0.0233	0.0562	0.0795	0.32
2	58.66	13.67	1350000	447	13.4	0.0552	0.0822	0.1373	0.41
					Total	0.1475	0.3224	0.4698	1.64
$\Delta_i = (P*h^3)/(3*E*I) + (2.78*P*h)/(A_w*E)$									
$I = (t*d^3)/12$					t = 0.67 ft				
$A_w = t*d$					d = 20 ft				

SHEAR WALL CHECK

EAST - WEST : WALL 7 $F = 33^K$

THICKNESS = 8" HEIGHT: 30.4FT
LENGTH = 20'
REINFORCEMENT: #4 @ 48" O.C.

DEAD WEIGHT:
$(50\,PSF)(30.4\,FT)(20\,FT) = 30.4^K$

$M = Vh = 33^K(30.4\,FT)(12\,IN) = 12039\ K \cdot IN$

UNCRACKED SECTION

$S = \frac{tL^2}{6} = \frac{7.625\,IN\,(240\,IN)^2}{6} = 73{,}200\ IN^3$

$f_b = \frac{M}{S} = \frac{12039\,K \cdot IN\,(1000\,LB/K)}{73{,}200\,IN^3} = 165\ PSI$

AXIAL STRESS

$f_a = \frac{P}{A_e} = \frac{P}{tL}$

$= \frac{30.4^K}{(7.625\,IN)(240\,IN)}$

$= 16.7\ PSI < 165\ PSI$

$F_b = \frac{f'_m}{3} = \frac{1500\ PSI}{3} = 500\ PSI > f_b = 165\ PSI \quad \therefore OK$

FLEXURAL REINFORCEMENT

$d = 20\,FT(12\,IN/FT) - 8\,IN = 232\ IN$

$F_s = 0.5f_y = 0.5(50{,}000) = 25{,}000\ PSI$
USE 24,000 PSI
(MAXIMUM)

$A_s = \frac{M}{F_s j d} = \frac{12039\,K \cdot IN}{(24\,KSI)(0.9)(232\,IN)} = 2.41\ IN^2$

4-#8 BARS AT JAMBS (2 AT EACH END)

SHEAR STRESS

$f_v = \frac{V}{bjd} = \frac{33000\,LB}{(7.625\,IN)(0.9)(232\,IN)} = 20.8\ PSI$

$\frac{M}{Vd} = \frac{12039\,K \cdot IN}{33^K(232\,IN)} = 1.58 > 1.0 \quad \therefore F_v = 1.0\sqrt{f'_m} = \sqrt{1500} = 39\ PSI > 35\ PSI$

$\therefore$ USE 35 PSI

$f_v < F_v$ $\therefore$ NO SHEAR REINFORCEMENT NEEDED

BOND STRESS
$\Sigma_o = 6.3\ IN$

$u = \frac{V}{\Sigma_o j d} = \frac{33{,}000\,LB}{(6.3\,IN)(0.9)(232\,IN)} = 25.1\ PSI < 200\ PSI \quad \therefore OK$

MAXIMUM REINF IN A CELL
2 - #8 BARS: $1.57\ IN^2$

$\%_d = \frac{1.57}{(34.5\,IN^2)}(100) = 4.55\% < 6\% \quad \therefore OK$

APPENDIX F

VIBRATION CHECK - STEEL JOIST SYSTEM

DECK PROPERTIES:

CONCRETE: $w_c = 145\ PCF$

$f'_c = 4\ KSI$

FLOOR THICKNESS = 4 IN (1.5 IN RIBS)

SLAB + DECK WEIGHT = 39 PSF

20 FT SPAN	6 FT SPAN
JOIST PROPERTIES:	
18K3	16K3
WT = 6.6 PLF	WT = 6.3 PLF
$A = 1.20\ IN^2$	$A = 1.20\ IN^2$
$I_{CHORD} = 90.3\ IN^4$	$I_{CHORD} = 70.4\ IN^4$
D = 18 IN	D = 16 IN
$y_c = 10\ IN$	$y_c = 8.88\ IN$
$M_{ALL} = \frac{wL^2}{8} = \frac{463 PLF(20FT - 0.33FT)^2}{8} = 22.4\ K\cdot FT$	$M_{ALL} = \frac{410 PLF(20FT - 0.33FT)^2}{8} = 19.9\ K\cdot FT$
$A_{BOT} = \frac{M_{ALL}}{(d-1")f_{ALL}} = \frac{22.4K\cdot FT(12 IN/FT)}{(18"-1")(30KSI)} = 0.53\ IN^2$	$A_{BOT} = \frac{19.9K\cdot FT(12 IN/FT)}{(16"-1")(30\ KSI)} = 0.53\ IN^2$
$A_{TOP} = 1.25(0.53 IN^2) = 0.67\ IN^2$	$A_{TOP} = 1.25(0.53 IN^2) = 0.67\ IN^2$
$\bar{y} = 0.5" + \frac{0.67 IN^2(18"-1")}{1.20 IN^2} = 10\ IN$	$\bar{y} = 0.5" + \frac{0.67 IN^2(16"-1")}{1.20 IN^2} = 8.88\ IN$
$I = A_{TOP}(\bar{y}-0.5")^2 + A_{BOT}(d-\bar{y}-0.5")^2$ $= 0.67 IN^2(10"-0.5")^2 + 0.53 IN^2(18"-10"-0.5")^2$ $= 90.3\ IN^4$	$I = 0.67 IN^2(8.88"-0.5")^2 + 0.53 IN^2(16"-8.88"-0.5")^2$ $= 47.1\ IN^4 + 23.3\ IN^4$ $= 70.4\ IN^4$
GIRDER PROPERTIES	
W18×35	W12×14
$A = 10.3\ IN^2$	$A = 4.16\ IN^2$
$I = 510\ IN^4$	$I = 88.6\ IN^4$
d = 17.7 IN	d = 11.9 IN
BEAM MODE PROPERTIES	
$E_c = w^{1.5}\sqrt{f'_c} = (145 PCF)^{1.5}\sqrt{4 KSI} = 3492\ KSI$	$E_c = 3492\ KSI$
$n = \frac{E_S}{1.35E_c} = \frac{29{,}000\ KSI}{1.35(3492\ KSI)} = 6.16$	$n = 6.16$

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS

BEAM MODE PROPERTIES (CONT.)

$$\bar{y} = \frac{1.20''^2(1.5''+10'') - (18''/6.16)(2.5'')(2.5''/2)}{1.20''^2 + (18''/6.16)(2.5'')}$$

$$= \frac{13.8'' - 9.14''}{8.51''}$$

$$= 0.55''$$

$$I_{comp} = 90.3\,IN^4 + 1.20\,IN^2(1.5''+10''-0.55'')^2 + (18''/6.16)(2.5'')^3/12 + (18''/6.16)(2.5'')(0.55''+\tfrac{2.5}{2}'')^2$$

$$= 90.3 + 143.9 + 3.8 + 23.7$$

$$= 262\,IN^4$$

$$6 \le \frac{L_j}{d} = \frac{20\,FT(12\,IN/FT)}{18\,IN} = 13.4 \le 24$$

$$\therefore C_r = 0.9(1-e^{-0.28(L_j/D)})^{2.8}$$

$$= 0.9(1-e^{-0.28(13.4)})^{2.8}$$

$$= 0.85$$

$$\gamma = \frac{1}{C_r} - 1 = \frac{1}{0.85} - 1 = 0.18$$

$$I_j = \frac{1}{\frac{\gamma}{I_{theor}} + \frac{1}{I_{comp}}} = \frac{1}{\frac{0.18}{90.3\,IN^4} + \frac{1}{262\,IN^4}}$$

$$= 173\,IN^4$$

distributed loading

DL = 65 PSF

LL = 60 PSF

$$w_j = \left(\frac{40\,IN}{12\,IN/FT}\right)(60\,PSF + 65\,PSF)$$

$$= 417\,PLF$$

$$\Delta_j = \frac{5wL^4}{384EI}$$

$$= \frac{5(417\,PLF)(20\,FT)^4(1728\,IN^4/FT^4)}{384(29{,}000{,}000\,PSI)(173\,IN^4)}$$

$$= 0.3\,IN$$

frequency

$$f_j = 0.18\sqrt{\frac{g}{\Delta_j}} = 0.18\sqrt{\frac{386}{0.3\,IN}} = 6.5\,HZ$$

$$y = \frac{1.20\,IN^2(1.5''+8.88'') - (16''/6.16)(2.5'')(2.5''/2)}{1.20\,IN^2 + (16''/6.16)(2.5'')}$$

$$= \frac{12.46'' - 8.12''}{7.70''}$$

$$= 0.57''$$

$$I_{comp} = 70.4\,IN^4 + 1.20\,IN^2(1.5''+8.88''-0.57'')^2 + (16''/6.16)(2.5'')^3/12 + (16''/6.16)(2.5'')(0.57''+\tfrac{2.5}{2}'')^2$$

$$= 70.4 + 115.5 + 3.4 + 21.5$$

$$= 211\,IN^4$$

$$6 \le \frac{L_j}{d} = \frac{20\,FT(12\,IN/FT)}{16\,IN} = 15 \le 24$$

$$\therefore C_r = 0.9(1-e^{-0.28(15)})^{2.8}$$

$$= 0.87$$

$$\gamma = \frac{1}{0.87} - 1 = 0.15$$

$$I_j = \frac{1}{\frac{0.15}{70.4\,IN^4} + \frac{1}{211\,IN^4}}$$

$$= 146\,IN^4$$

DL = 65 PSF

LL = 100 PSF

$$w_j = \left(\frac{24\,IN}{12\,IN/FT}\right)(100\,PSF + 65\,PSF)$$

$$= 330\,PLF$$

$$\Delta_j = \frac{5(330\,PLF)(20\,FT)^4(1728\,IN^4/FT^4)}{384(29{,}000{,}000\,PSI)(146\,IN^4)}$$

$$= 0.28\,IN$$

$$f_j = 0.18\sqrt{\frac{386}{0.28\,IN}} = 6.7\,HZ$$

BEAM MODE PROPERTIES (CONT.)

$D_s = \frac{12 d_e^3}{12 n} = \frac{(3.25\,IN)^3}{6.16} = 5.58\ IN^4/FT$

$D_j = \frac{I_j}{S} = \frac{173\,IN^4(12)}{(40\,IN)} = 52\ IN^4/FT$

effective beam panel width

$B_j = C_j (D_s/D_j)^{1/4} L_j$

$= 2.0(5.58/52)^{1/4}(20\,FT) = 22.9\,FT$

$W_j = \left(\frac{W_j}{S}\right) B_j L_j = \left(\frac{417\,PLF}{3.33\,FT}\right)(22.9\,FT)(20\,FT)$

$= 57.4^K$

$D_s = \frac{(3.25\,IN)^3}{6.16} = 5.58\ IN^4/FT$

$D_j = \frac{146\,IN^4(12)}{24\,IN} = 73\ IN^4/FT$

$B_j = 2.0(5.58/73)^{1/4}(20\,FT) = 21.1\,FT$

$W_j = \left(\frac{330\,PLF}{2\,FT}\right)(21.1\,FT)(20\,FT)$

$= 69.7^K$

GIRDER MODE PROPERTIES

$0.4 L_g = 0.4(20\,FT)(12\,IN/FT) = 96\,IN$

AVG. CONC. DEPTH = 3.75 IN

$\bar{y} = \frac{10.3\,IN^2(0.75'' + 4'' + \frac{17.7''}{2}) - (\frac{96''}{6.16})(3.75'')(\frac{3.75''}{2})}{10.3\,IN^2 + (\frac{96''}{6.16})(3.75'')}$

$= \frac{140.1\,IN^3 - 109.6\,IN^3}{68.8\,IN^2}$

$= 0.45\,IN$

$I_g = 510\,IN^4 + 10.3\,IN^2(0.75'' + 4'' + \frac{17.7''}{2} - 0.45'')^2$
$+ (96''/6.16)(3.75'')^3/12$
$+ (96''/6.16)(3.75'')(0.45'' + \frac{3.75''}{2})^2$
$= 510 + 1781.1 + 68.5 + 316$
$= 2676\,IN^4$

account for reduced stiffness

$I_g = I_{nc} + (I_c - I_{nc})/4$
$= 510\,IN^4 + (2676\,IN^4 - 510\,IN^4)/4$
$= 1052\,IN^4$

$w_g = L_j (w_j/s) + \text{GIRDER WT.}$
$= 20\,FT(417\,PLF/3.33\,FT) + 35\,PLF$
$= 2540\,PLF$

$\Delta_g = \frac{5 w L^4}{384 E I}$
$= \frac{5(417\,PLF)(20\,FT)^4(1728)}{384(29{,}000{,}000\,PSI)(1052\,IN^4)}$
$= 0.050\,IN$

$0.4 L_g = 0.4(6\,FT)(12\,IN/FT) = 29\,IN$

AVG. CONC. DEPTH = 3.75 IN

$\bar{y} = \frac{4.16\,IN^2(0.75'' + 4'' + \frac{11.9''}{2}) - (\frac{29''}{6.16})(3.75'')(\frac{3.75''}{2})}{4.16\,IN^2 + (\frac{29''}{6.16})(3.75'')}$

$= \frac{44.6\,IN^3 - 33.1\,IN^3}{21.9\,IN^2}$

$= 0.53\,IN$

$I_g = 88.6\,IN^4 + 4.16\,IN^2(0.75'' + 4'' + \frac{11.9''}{2} - 0.53'')^2$
$+ (29''/6.16)(3.75'')^3/12$
$+ (29''/6.16)(3.75'')(0.53'' + \frac{3.75''}{2})^2$
$= 88.6 + 430.3 + 20.7 + 102.2$
$= 642\,IN^4$

$I_g = 88.6\,IN^4 + (642\,IN^4 - 88.6\,IN^4)/4$
$= 227\,IN^4$

$W_g = (20\,FT)(330\,PLF/2\,FT) + 14\,PLF$
$= 3314\,PLF$

$\Delta_g = \frac{5(330\,PLF)(6\,FT)^4(1728)}{384(29{,}000{,}000\,PSI)(227\,IN^4)}$

$= 0.002\,IN$

frequency

$f_g = 0.18\sqrt{\frac{g}{\Delta_g}} = 0.18\sqrt{\frac{386}{0.05}} = 15.9\,HZ$

$D_j = 52\ IN^4/FT$

$D_g = \frac{I_g}{L_j} = \frac{1052\,IN^4}{20\,FT} = 52.6\ IN^4/FT$

$B_g = C_g(D_j/D_g)^{1/4}L_g$
$= 1.6(52/52.6)^{1/4}(20\,FT) = 31.9\,FT$

$W_g = (w_g/L_j)B_g L_g$
$= (2540\,PLF/28\,FT)(31.9\,FT)(20\,FT)$
$= 57.9^K$

COMBINED MODE PROPERTIES

GIRDER SPAN < B_j

$\Delta_g' = \frac{L_g}{B_j}\Delta_g = \left(\frac{20\,FT}{22.9\,FT}\right)(0.05\,IN) = 0.05\,IN$

$f_n = 0.18\sqrt{g/(\Delta_j + \Delta_g')}$
$= 0.18\sqrt{386/(0.3\,IN + 0.05\,IN)}$
$= 5.98\ HZ$

$W = \frac{\Delta_j}{\Delta_j + \Delta_g}W_j + \frac{\Delta_g}{\Delta_j + \Delta_g}W_g$
$= \frac{0.3}{0.3+0.05}(57.4^K) + \frac{0.05}{0.3+0.05}(57.9^K)$
$= 49.2^K + 8.3^K$
$= 57.5^K$

$\beta W = 0.05(57.5^K) = 2.9^K$

WALKING EVALUATION

$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} = \frac{65 e^{-0.35(5.98)}}{2900}$
$= 0.0028$

0.28% < 0.5% ∴ OK
ACCELERATION LIMIT

$f_g = 0.18\sqrt{\frac{386}{0.002}} = 79.1\ HZ$

$D_j = 73\ IN^4/FT$

$D_g = \frac{I_g}{L_j} = \frac{227\,IN^4}{20\,FT} = 11.4\ IN^4/FT$

$B_g = 1.6(11.4/73)^{1/4}(6\,FT) = 6.1\,FT$

$W_g = (3314\,PLF/28\,FT)(6.1\,FT)(6\,FT)$
$= 4.4^K$

GIRDER SPAN > B_j

$\Delta_g' \neq$ (CANNOT BE REDUCED)

$f_n = 0.18\sqrt{386/(0.28\,IN + 0.002\,IN)}$
$= 6.66\ HZ$

$W = \frac{0.28}{0.28+0.002}(69.7^K) + \frac{0.002}{0.28+0.002}(4.4^K)$
$= 69.2^K + 0.1^K$
$= 69.3^K$

$\beta W = 0.05(69.3^K) = 3.5^K$

$\frac{a_p}{g} = \frac{65 e^{-0.35(6.66)}}{3500}$
$= 0.0018$

0.18% < 0.5% ∴ OK
ACCELERATION LIMIT

* $f_n < 9\,HZ$ SO NO NEED TO CHECK FLOOR STIFFNESS

APPENDIX G

Cost Analysis

Cost Analysis - Existing System										
Takeoff				Estimate						
					Unadjusted			Adjusted		
	Material	Quantity	Total Length (ft)	Units	Material Unit Cost	Overall Unit Cost	Overall Cost	Material Unit Cost	Overall Unit Cost	Overall Cost
	Steel									
Columns	W12x79	55	1925	lf	61.00	75.00	144,375	60.21	80.70	155,348
3rd Floor	W14x30	4	60	lf	21.00	28.50	1,710	20.73	30.67	1,840
Beams	W16x36	12	240	lf	28.00	37.50	9,000	27.64	40.35	9,684
	W21x44	5	100	lf	31.00	40.50	4,050	30.60	43.58	4,358
	W21x57	2	30	lf	43.50	54.50	1,635	42.93	58.64	1,759
	W21x83	38	560	lf	58.00	70.50	39,480	57.25	75.86	42,480
	W24x76	12	190	lf	53.50	64.50	12,255	52.80	69.40	13,186
	W24x117	6	110	lf	82.00	97.00	10,670	80.93	104.37	11,481
	W27x94	2	40	lf	66.00	78.00	3,120	65.14	83.93	3,357
2nd Floor	W14x30	13	110	lf	21.00	28.50	3,135	20.73	30.67	3,373
Beams	W16x36	16	320	lf	28.00	37.50	12,000	27.64	40.35	12,912
	W18x76	50	860	lf	53.50	66.00	56,760	52.80	71.02	61,074
	W21x44	2	20	lf	31.00	40.50	810	30.60	43.58	872
	W21x83	4	80	lf	58.00	70.50	5,640	57.25	75.86	6,069
	W30x90	1	20	lf	69.50	82.00	1,640	68.60	88.23	1,765
			Total S.F.							
	Concrete									
Precast plank			38000	sf	5.10	7.55	286,900	4.55	7.01	266,243
						Total	593,180			595,800

Cost Factors:		
	Material	Total
Formwork	1.017	1.082
Reinf.	1.026	1.037
Concrete	0.893	0.928
Steel	0.987	1.076

Cost Analysis

Cost Analysis - Proposed Concrete System										
Takeoff				Estimate						
					Unadjusted			Adjusted		
	Material	Quantity	Total Cubic Yards	Units	Material Unit Cost	Overall Unit Cost	Overall Cost	Material Unit Cost	Overall Unit Cost	Overall Cost
	Concrete									
	Columns	50	212.5	cy	197.00	905.00	192,313	175.92	839.84	178,466
4 Floors	Girders		40	cy	214.00	840.00	134,400	191.10	779.52	124723
	Joists		10.5	cy	214.00	840.00	35,280	191.10	779.52	32740
			Total S.F.							
	Slab		38000	sf	0.97	2.32	88,160	0.87	2.15	81,812
	Formwork (j + g)		38000	sf	2.14	7.35	279,300	2.18	7.95	302,203
	Formwork (col)		16200	sf	0.92	3.11	50,382	0.94	3.37	54,513
						Total	729,453			774,457

Cost Factors:		
	Material	Total
Formwork	1.017	1.082
Reinf.	1.026	1.037
Concrete	0.893	0.928
Steel	0.987	1.076

Cost Analysis - Proposed Steel System										
Takeoff				Estimate						
					Unadjusted			Adjusted		
	Material	Quantity	Total Length (ft)	Units	Material Unit Cost	Overall Unit Cost	Overall Cost	Material Unit Cost	Overall Unit Cost	Overall Cost
	Steel									
Columns	W12x72	50	2700	lf	50.50	63.50	171,450	49.84	68.33	184,480
4 Floors	W12x19	15	130	lf	15.45	22.50	11,700	15.25	24.21	12,589
of Beams	W14x22	6	80	lf	18.25	25.00	8,000	18.01	26.90	8,608
	W16x31	48	840	lf	21.50	29.50	99,120	21.22	31.74	106,653
	W18x35	10	160	lf	24.50	34.00	21,760	24.18	36.58	23,414
4 Floors	18K3	126	2520	lf	3.23	6.95	70,056	3.19	7.48	75,380
of Joists										
			Total S.F.							
	Concrete									
Slab			38000	sf	0.97	2.32	88,160	0.87	2.15	81,812
Wire Mesh			38000	sf	0.26	0.68	25,840	0.27	0.71	26,796
Form Deck			38000	sf	0.55	1.10	41,800	0.56	1.19	45,228
						Total	537,886			564,961

Cost Factors:		
	Material	Total
Formwork	1.017	1.082
Reinf.	1.026	1.037
Concrete	0.893	0.928
Steel	0.987	1.076

APPENDIX H

Formwork Design

Assumptions

R = 7 ft/hr (concrete pour rate)
T = 50° F (temperature of concrete)
½ " plywood forms
2 x 4 studs and wales

Column Formwork

p = 150 + ((9000 * R) / T)
= 150 + ((9000 * 7 ft/hr) / 50° F) = 1410 psf

Table 6-4 (plywood support spacing)
s = 6"
w = (8" / 12") * 1410 psf = 705 plf

Table 6-6 (wale spacing)
s = 18"