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S&T Bank
Corporate Headquarters
Indiana, PA

Senior Thesis Report

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Executive Summary

S&T Bank, located in Indiana, PA is the corporate headquarters for the company. To be sure of an economical design, the existing design for the building is compared to a new design considering the structural system.

The original system consists of a structural steel frame. The floor system consists of a 3″ normal weight concrete slab topping, reinforced with 6x6 W1.4 x W1.4 WWF placed on Bowman 28 Gage SF-1 galvanized deck. The concrete topping is rated at 3000psi. The decking is set on 24k4 joists that are spaced at 2′ apart. The foundation includes a 12″ masonry wall with concrete piers placed intermittently. The building is supported on spread footings which sit on soil that can support a maximum of 6000psf. The frame resists lateral forces with moment connections placed throughout many of the column-girder connections.

The proposed building incorporates a two-way flat slab system. The slab is 10″ thick and has 5.5″ drop panels at most of the columns. 20″ x 20″ and 24″ x 24″ concrete columns support the slab system. The columns rest on spread footings which are slightly larger than the spread footings for the original design. This is due to the added building weight. The monolithically poured concrete frame is determined to be sufficient to resist the lateral loads without the assistance of shear walls.

The proposed concrete system costs $622,311 less than the existing steel system, but takes 95 more days to construct. The added duration is a result of setting and removing formwork as well as the additional time to allow the concrete to cure.

Existing lighting in the research room on the first floor of the bank uses (2) 32 watt T8 lamps in a direct lighting fixture to light the room. The room doesn't meet the recommended minimum 50fc light intensity for the use prescribed. The existing light levels are 32.7fc – 46.1fc. A new indirect system, which also uses (2) 32 watt T8 lamps, is proposed to replace the existing lighting scheme. The new lighting scheme produces light levels of 41.8fc – 59.3fc.

It is noted that even though the concrete system is $622,311 less than the steel system, further investigation is required to specify which system is more economical. The proposed lighting scheme is recommended over the existing lighting system because the indirect meets recommended lighting levels and will reduce most of the glare on the workspace. These better working conditions will promote a more productive environment.

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Table of Contents

Table of Contents (cont.)

Appendix A

Appendix B

Appendix C

Appendix D

Appendix E

Appendix F

Appendix G

Introduction

The goal of this report is presenting a specific building in such a way that describes the existing conditions of the building. The report will then present an argument which will lead to an extensive study that will produce a redesign of the building in question. In this case, the building in question is S&T Bank Corporate Headquarters, located in Indiana, PA. After the depth of the report is finished, two smaller breadth topics will be discussed. For the overall report, some general building information is first provided so that the reader can familiarize themselves with the case-study at hand.

In the first main section of the report, findings of the initial "existing structural conditions" study will be provided. Items that were analyzed in depth during the study include soil conditions via a geotechnical report provided by Triad Engineering Inc., foundation walls and footings, structural steel framing members, lateral resisting elements, as well as floor and roof construction.

The next section of the report will propose a problem or argument that will justify the purpose of this report. Issues of the argument will be based primarily on economics, i.e. cost, time, availability of materials etc.

The third section of the report will be similar to the first section in that it will be a "proposed structural conditions" study. This study uses common design techniques of today's construction engineering industry to make a new design of the foundation, framing members, and floor system.

Following the proposed design section of the report, the first breadth topic is discussed. A cost comparison and a schedule comparison will be provided for both the existing design and the proposed design. This study

will assist in the determination of the conclusion to the proposed problem/argument.

The final topic to be discussed is a proposed lighting redesign in a particular room of the bank. For the sake of comparison each lighting scheme (existing and proposed) will be shown through the rendering program AGI. The comparison will be based on aesthetics as well as issues concerning a better design will be addressed.

Since the report will include many topics of debate and provide a large amount of information on each topic, a comparison summary will be provided at the end of the report. Along with direct comparison, recommendations on why one system is better than the other are mentioned. To complete the report, a conclusion is included to summarize what was discussed in the report and also to provide recommendations on which system is better and why.

General Building Information

About the Company:

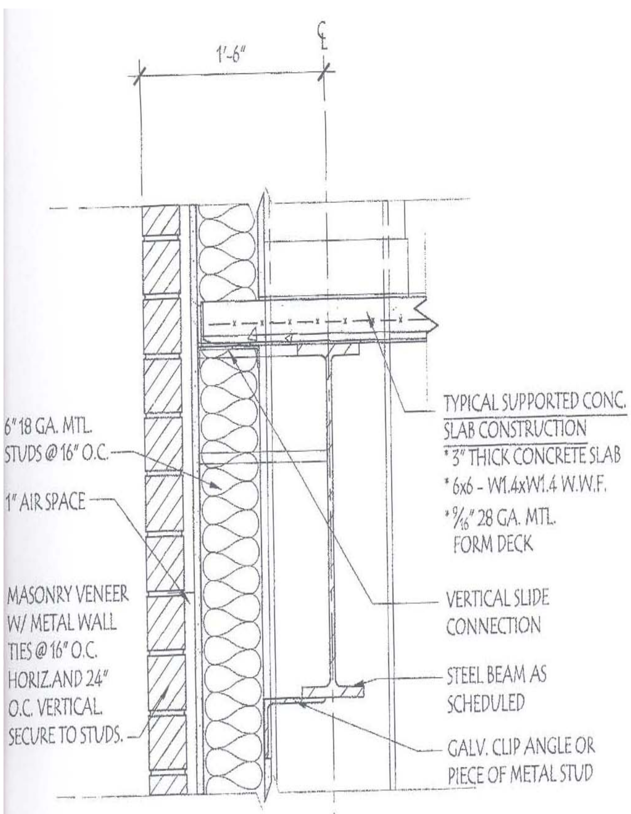
S&T Bancorp, a $3.2 billion financial holding company headquartered in Indiana, Pennsylvania, has two wholly owned subsidiaries, S&T Bank and 9th Street Holdings, Inc. Founded in 1902 with a single location in Indiana, S&T Bank has grown to 51 offices that provide a full range of financial services to individuals and businesses throughout Western Pennsylvania. The mission of S&T Bank is to achieve consistent superior financial performance, which creates value for their shareholders by: identifying and satisfying customer needs with quality products and services which exceed their expectations; providing a stimulating and challenging team-oriented work environment, which encourages, develops, and rewards excellence; and diligently serving our communities with integrity and pride.

About the Building:

Construction of the project began in June 2005 and project completion is projected for August 2006. Primarily the building is a corporate office for S&T Bank employees. The building is 4 stories above ground rising to almost 60 feet with a one-story basement underground. On the first floor, a bank branch is available for customers. The rest of the floors except the fourth floor comprise of some offices, however there are large lobby areas designated for different facilities of the bank (i.e. finance dept., loan dept., etc.). The 4th floor is reserved for future plan layouts, which are dependent on the growth of the company. There are two

entrances; the more grand entrance in the north-east corner of the building, and another accessible entrance on the south end. There is an elevator bay, containing two elevators, in the central core of the building. Stairs are provided at the south and west ends of the building. One set of restrooms are provided on each floor adjacent to the elevators in the center of the building.

Concerning the exterior façade of the building, the clearest idea of what the building looks like can be gathered from the architectural rendering at the top of the page. However, to describe the exterior with words and technical jargon, the envelope is as follows. A typical exterior wall consists of a 4″ nominal masonry veneer, 1″ air space, ½″ glass mat faced gypsum sheathing with a weather resistive barrier, 6″ metal studs @ 16″ O.C., R-21 fiberglass batt insulation with vapor barrier and 5/8 gypsum wallboard at the interior. The color of the brick veneer used to clad the building envelope is cranberry velour. The Pre-cast Concrete (used as an architectural detail) is a limestone color with a smooth finish. Metal coping is used as an accent which is to match the champagne aluminum curtain wall and door systems.



Figure#1: Typical Wall Section

EIFS (exterior insulation finish system) is used near the top of the building (4th floor) which is matched to the pre-cast color (limestone) or a senergy metallic finish patina green (used on the soffit/fascia). There are also large spandrel glass sections used to envelope the staircases on both sides of the building. Now that you have a general idea of S&T Bank and the building taken to be its corporate headquarters, the next section will describe, in detail, the structural components of the building.

Existing Structural Conditions

For this section of the report, the following paragraphs will completely describe the existing structural system including the foundation, framing system, floor system, and the roof system. As reference a plan of the typical layout is provided below in figure #2.

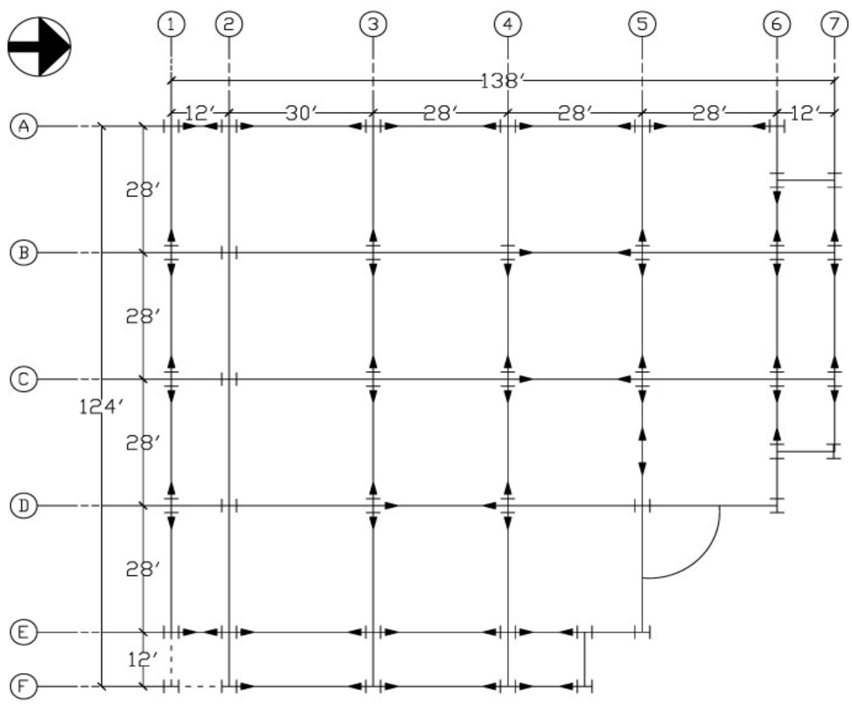


Figure #2: Typical Framing Layout

Foundation:

A geotechnical report provided by Triad Engineering Inc. establishes that the bearing on the soil below the building can be no more than 6000psf. The foundation of the building rests on spread footings, which have a concrete strength of f'_c=4000psi at 28 days. The footings are as

small as 5′x 5′ and 1′ thick or can be as large as 10′x10′ and 2′-6″ thick. A typical footing is 7′-6″x 7′6″ and is 2′ thick. Concrete piers are used throughout the masonry wall; these piers are typically 1′-8″ by 2′-4″. Both spread footings and piers have reinforcing steel ranging from #5′s to #9′s. The columns are attached to the concrete with A36 steel base plates and anchor bolts. The basement exterior wall is a typical 12″ Ivany block except for the section under the rotunda entrance, which is reinforced concrete. This wall extends to the 2nd floor and is then replaced by a curtain wall. Basement floor construction consists of a 4″ concrete slab, reinforced with 6x6 w1.4 x w1.4 WWF on a 6mil vapor barrier placed on a minimum of 4″ compacted stone.

Framing System:

S&T Bank Corporate Headquarters is a steel frame building. The steel frame is four stories high and has a typical layout on every floor. The story heights are 15′-4″, 13′-4″, 13′-4″ and 13′-4″ for the 1st through 4th floors respectively. The building footprint is 141 ft. in the North-South direction by 127 ft. in the East-West direction. In general, differences in floor framing layout consists of the sizes of the beams and the addition/subtraction of shafts which typically appear near the staircases. However in the S-E corner of the first floor only, there is a bank vault. On the floors above a 12′ x 12′ section is taken out of the S-E corner. There are 6 bays in both the N-S direction and the E-W direction. The first and the last bays in either direction are 12′ bays. Typically the central bays are 28′ wide, but can be as large as 30′ or as small as 16′. All of the structural columns, beams, and girders are A992 steel and have yield strengths of 50ksi. The arrows near the columns designate moment connections that resist lateral load in the

direction of the arrow. These moment connections are attached by "wind clips" which are angles welded to the top and bottom of the connection. Figure #3 shows a detail of the moment connection. Due to this connection they are not as rigid as full moment connections.

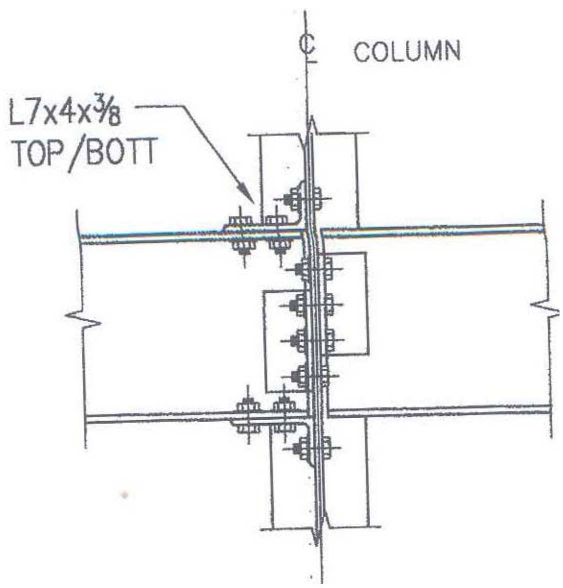


Figure #3: Moment Connection Detail

The building's columns range from W10x33 to W12x87, while a typical column size used is W12x53. As you can see in figure #1, every beam forms into a column. Therefore, beams and girders cannot be considered in their normal sense. Girders will be considered those running in the E-W direction and beams will be considered to be running in the N-S direction. Girder sizes range from W16x26 up to W24x76 with a typical girder size of W24x55. Beams that run in the N-S direction are much smaller than the girders that frame in from the E-W direction. Beams running N-S range from W12x16 up to W16x26 with a typical beam size of W14x22. Any other structural components, such as angles or base plates, are A36 steel and have yield strengths of f_y=36ksi. All of the structure's

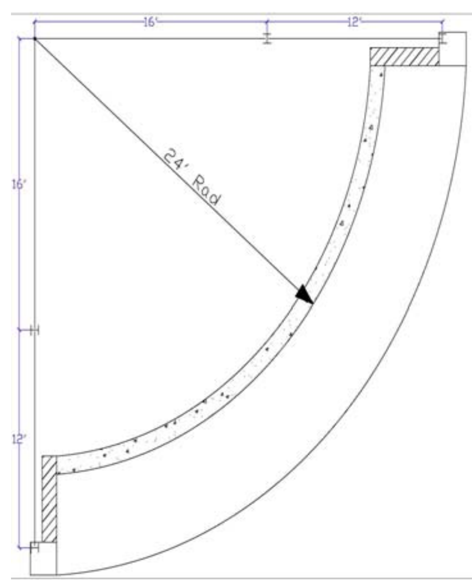


Figure #4: Non-Typical Wall Section

walls, girders, and beams run orthogonal except for a small section on the North-East corner of the building. This corner is rounded at the foundation with a 24′-0″ radius semi-circle as shown in Figure #4.

Floor System:

The deck system consists of a 3″ normal weight concrete slab topping, reinforced with 6x6 W1.4 x W1.4 WWF placed on Bowman 28 Gage SF-1 galvanized deck. This non-composite decking is set on 24k4 joists that are spaced at 2′ apart. The concrete topping is rated at 3000psi. The roof decking is relatively the same as the floors below except when placed under the AHU, the decking then sits on 24k6 joists. The depth of the Bowman deck with the concrete is 3″. The joists are 24″ deep which gives a total floor system depth of 27″. A typical floor construction detail can be seen in Figure #5 and a layout of the floor system in a typical bay can be seen in Figure#6.

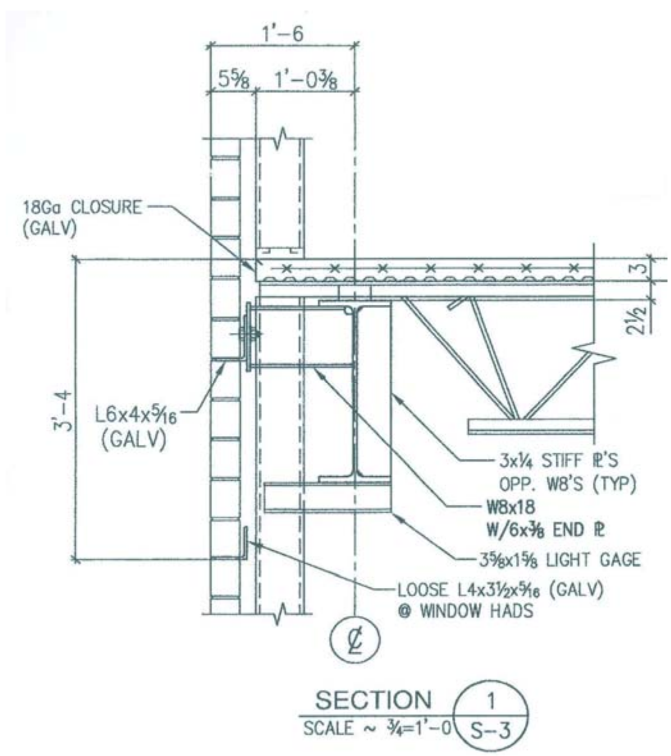
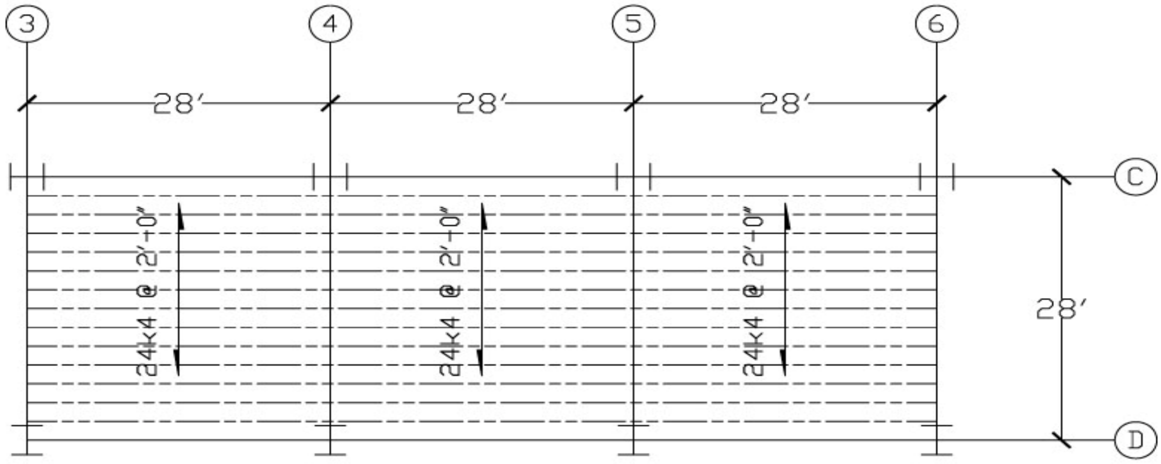


Figure #5: Typical Floor Detail



Figure#6: Typical Floor Layout

Roof System:

The roof system used is a built-up roof which is slightly sloped to the different areas of the roof for draining purposes. A detailed section of the roof can be referenced in Figure #7. Interior drainage is provided so that the architectural façade will not be interrupted by drain pipes. This built-up roofing system utilizes a stone ballast system that sits upon ½" glass sheathing roof membrane, R20 roof insulation, and 1½" metal decking (typically). There are several roof overhangs placed at the 2nd floor, which are above entrances and various windows. The overhang roofs are sloped up at 45° and are flat on top using the same draining techniques as the larger roof (Figure #8).

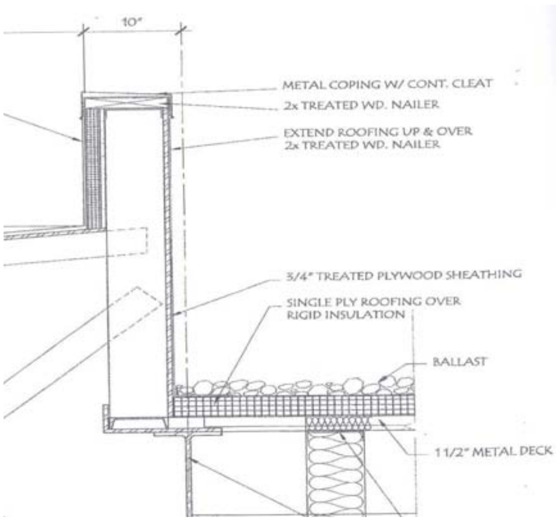


Figure #7: Roof Construction Detail

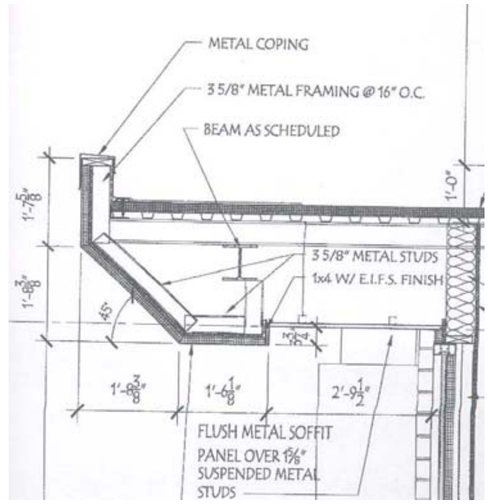


Figure #8: Roof Overhang Detail

Problem Statement

Analysis of S&T Bank through previous reports on the building has shown that the existing building structure is a relatively simple design. Due to the simplicity of the design and the straightforwardness of the building layout, a creative redesign that is appropriate for such a building is hard to discern. Though the current floor system is 27" deep, the building is still 15 feet under the 75 foot height restriction, so floor-to-floor height is not much of a concern. The columns are not exceptionally large, and the foundation consists of spread footings (which are a simple foundation design). The lateral system does utilize moment connection which can get expensive. However this building uses wind-clip moment connections which are one of the cheapest moment connections available.

Steel and concrete prices are constantly fluctuating. Due to the recent demand in the Far East, both prices and availability have been a concern. Currently things are looking better concerning these building materials. To make an economic decision on which building material is best, a comparative study of sorts would need to be conducted. Since S&T Bank uses steel as its primary building material it could be speculated whether or not concrete would be a more economic solution as the primary building material. Which is a more economic and efficient design material for this building, steel or concrete?

Proposed Solution

To accurately compare the current steel system to a concrete system for the same building, a structural system using concrete must be completely designed. From a previous study of alternate floor systems, it was determined that a two-way flat slab system with drop panels would be the most efficient concrete system. Two-way flat slabs with drop panels provide the ability to compensate for longer spans and heavier loads, while keeping the slab system itself thin. Depending on the exterior slab loads and moments, a perimeter beam may be required. The design of the slab system will be controlled by allowable floor loads and both dead and live load deflection criteria. This system will be best designed with ADOSS, a Portland Cement Association concrete design program. This new floor system will require the design of reinforced concrete columns to replace the existing steel columns. The size of the columns will be controlled by strength, however the top floors will induce a much smaller load on the columns than those on the lower floors, thus the column size may be controlled by punching shear at the top levels. Since the amount of concrete will create a larger building weight, the footings will need to be checked and redesigned if necessary. Along with a new footing design, the added building weight will create greater seismic loads, so these new loads will also need to be determined.

After the design of the new structural system is complete, a breadth pertaining to construction management issues will be conducted. A cost and schedule estimate will be developed so as to make an accurate comparison to the original system. To complete the cost estimate of the new design, a complete take off of the concrete, rebar, and formwork must

be prepared. Once these values are determined, an aid such as R.S. Means will be utilized to turn these quantities into an overall cost. However to obtain an accurate estimate, labor costs must also be addressed. Again using R.S. Means, labor crews appropriate for each job will be designated and used to determine labor costs, as well as a time schedule for the required work. Depending on the returning values from the cost and schedule estimate, one design will be noted as more practical and efficient than the other.

As for the second breadth, present lighting conditions surveyed in the research room on the first floor will be checked for optimal performance. The existing lighting conditions will be analyzed using the lighting program AGI. Issues concerning light placement, lamp output, foot candle levels and glare will justify an optimal lighting scheme. After the existing design is represented, and the conditions are analyzed, a new better performing system will be developed.

Structural Depth

The following section of this report will go into detail about the proposed solution, specifying what the particular system is and how it was designed. First, the various building codes used during design are specified. Then the loads applied to the structural system are presented. Next the structural system is presented in the same fashion as it was designed. Starting with the *slab system*, the following section will present detailed information on *column design*, *edge beam design*, and *footing design*. At the end of each system design, a summary with diagrams of the system layout will be provided.

Load Determination:

Building Codes

IBC 2003- International Building Code
(In accordance with ASCE 7-05)

Design Loads:

Dead Loads

Superimposed DL:	12 psf
Floor Loads: (Slab Self-weight)	125 psf

Live Loads

Floors 1, 2, 3, & 4 (Lobby area)	100 psf

Snow Loads

P_f = 20psf

Wind Loads

Roof	14.66 kips
4th Floor	29.3 kips
3rd Floor	29.3 kips
2nd Floor	31.5 kips
1st Floor	16.9 kips

Seismic Loads

Roof	23.98 kips
4th Floor	57.94 kips
3rd Floor	57.94 kips
2nd Floor	66.52 kips
1st Floor	0 kips

Design Loads Used

Roof	23.98 kips
4th Floor	57.94 kips
3rd Floor	57.94 kips
2nd Floor	66.52 kips
1st Floor	16.9 kips (wind case controls at base)

From IBC 2003 different load combinations were analyzed to check which would control design. The load combinations looked at, are as follows…

1.4D
1.2D+1.6L+ (0.5L or 0.8W)
1.2D+1.6W+0.5L+0.5S
1.2D+1.0E+0.5L+0.5S
0.9D+ (1.6W or 1.0E)

The controlling case is

$$1.2D + 1.0E + 0.5L + 0.5S$$

Though it is not readily apparent, after some minor calculations it was determined that $1.6W < 1.0E$, hence the controlling case is chosen.

<u>Slab System:</u>

Ultimately, the computer program ADOSS will assist in designing the proposed slab system. Before ADOSS can be used, a preliminary design of slab dimensions must be sought out. The CRSI Handbook was an efficient tool used to decipher an initial two-way slab for the present spans and load conditions. Compared to the one-way joist system, the two-way flat slab system will have a smaller depth. The initial flat slab system analyzed has a total depth of 18″, according to the CRSI manual, as compared to 20.5″ of that in a one-way joist system. Also the two-way slab system with drop panels will require less formwork than the one-way joist concrete system. Entering the table on page 10-25 of the CRSI Handbook with a span of 28′ and a factored load of 186.8psf, an initial two-way flat slab system was chosen. The following information is provided by the CRSI. Appropriate supporting work can be found in Appendix B-15.

Initial Panel Specs:

Slab:	10.5″ thick		
Drop Panel:	9′-4″ by 9′4″		
	7.5″ thick		
Reinforcement:	Top:	15-#6 ———> Column Strip	
	Bottom:	12-#6	
	Top:	13-#5————> Middle Strip	
	Bottom:	11-#5	
Total Steel:	3.07psf	f'_c:	4000psi
Total Depth:	18″	f_y:	60ksi

The next step is to take portions of this data and enter them into ADOSS to determine an accurate slab design. Through trial and error, the system that works was found to be a 10″ reinforced slab with 5.5″ drop panels. According to ACI 318-05 the thinnest slab allowable is controlled by table 9.5(c). Since there is an edge beam and drop panels, the thickness of the slab is restricted to L/36 which is equivalent to 10″. Therefore the slab is acceptable according to this criterion. The partial output for a single span can be seen in Appendix B-1. This particular output is for the E-W span along column line 3. An equivalent frame representation of this span is shown in figure #9. With the applied loads in both the horizontal and vertical directions, the deflections over this span are as follows…

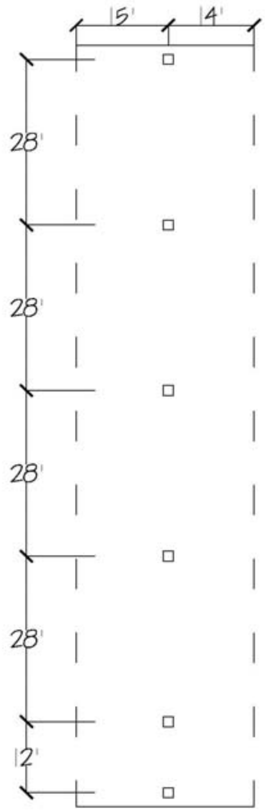


Figure #9: Equivalent Frame

	E-W Span	N-S Span	Total Deflection
Dead	0.163″	0.110″	0.273″
Live	0.236″	0.112″	0.348″
Total	0.400″	0.222″	0.622″

The allowable live load deflection limit is L/360 (0.93″).

0.348″ < 0.933″ → Deflection is OK!

The allowable total load deflection limit is L/480 (0.7″).

0.622″ < 0.700″ → Deflection is OK!

There are critical shear stresses pointed out by the ADOSS on the exterior span of the building. To address this problem, an edge beam is designed in the *Edge Beam Design* section that will resist the shear stresses at and around the columns. Another concern of this slab system would be the punching stresses caused by the columns. This will be addressed in the *Column Design* section. A section view of the proposed slab dimensions is shown below in Figure #10.

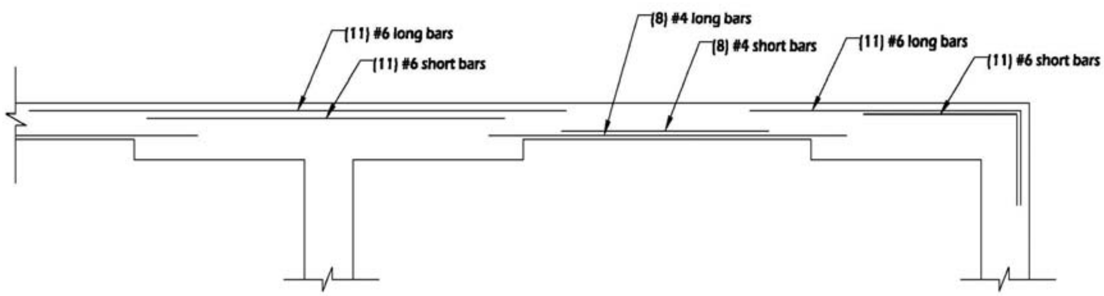


Figure # 10: Proposed Slab Detail

A complete reinforcing plan in the N-S and E-W directions can be referenced in Appendix B-17, B-18. To make sure that the design from ADOSS is accurate, and there were no human errors, the output was verified with the direct-design method as specified in ACI 318-05. The comparison can be seen in detail in Appendix B-16. The results from the direct-design method proved that ADOSS gave a reliable output. Using the steel specified from ADOSS it was determined that the bending capacity ØM_n=531.2 ft-k is greater than the critical moment at this section M_n= 520.5 ft-k (i.e. 531.2 ft-k > 520.5 ft-k). Since M_n is not much greater than ØM_n, it

can be concluded that the steel specified in the slab from ADOSS is an accurate output.

<u>Traverse Beam Design:</u>

The purpose of an edge beam is to assist the slab in resisting torsion induced on and exterior span. Interior spans have torsion on either side of the span which cancels each other out. Edge beams also provide resistance against shear failure around the exterior of the slab. The beam dimensions are based upon the slab geometry. Beyond each column, there is an 18″ overhang. For the beam to run flush with the interior face of the column (20″ x 20″), the edge beam will need to be 38″, with a depth of 18″. For the design of the reinforced concrete beam, critical areas of flexure, torsion, and shear must be considered. For a concrete strength of f'_c=4000psi and a steel yield strength of f_y= 60 ksi, the following design values are determined.

Flexure:

At Interior Support: $ØM_n$ =178 ft-k > M_u =160.25 ft-k

At Mid-span: $ØM_n$ =108.9 ft-k > M_u= 96.45 ft-k

Torsion:

$ØT_n$ = 85.9 ft-k > T_u = 66 ft-k

Shear:

No Additional Shear Reinforcing Required

To best describe the complete design of the edge beams, details showing reinforcing placement and beam dimensions are shown in Figure #11 and Figure #12. The calculations for this design can be referenced in Appendix C-1.

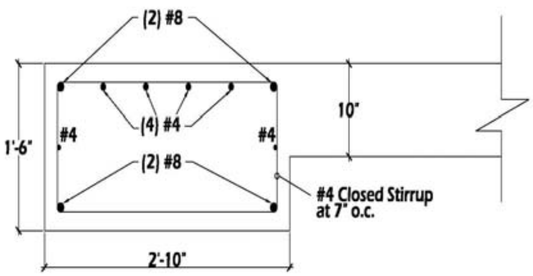


Figure #11: At Support

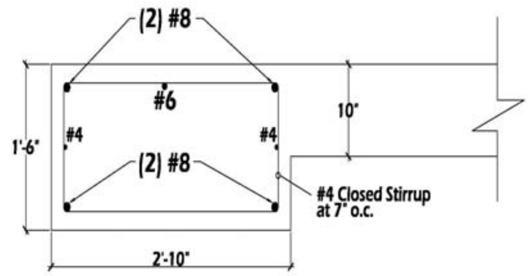


Figure #12: At Mid-span

Column Design:

Now that everything that is supported by the columns is designed, the columns themselves are ready to be designed. The columns were initially designed using the interaction diagram in figure #13.

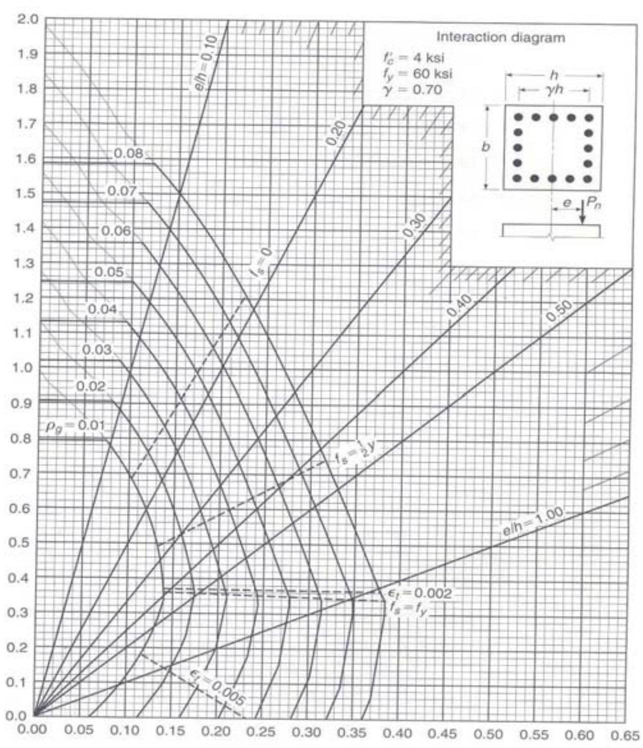


Figure #13: Column Interaction Diagram

This diagram shows the relationship between the amount of axial load and the amount of bending moment a column can support. After determining the values in the table in Appendix E-5, the following columns were *initially* designed.

4th, 3rd, and 2nd Floors:	16″ x 16″ with (8) #6 reinforcing bars
1st Floor Exterior Columns:	16″ x 16″ with (10) #8 reinforcing bars
1st Floor Core Columns:	18″ x 18″ with (10) #8 reinforcing bars
Ground Floor Exterior Columns:	20″ x 20″ with (10) #8 reinforcing bars
Ground Floor Core Columns:	24″ x 24″ with (10) #8 reinforcing bars

Ground floor column dimensions are based on bearing capacity at footing

Based on a punching shear analysis, 16″x16″ columns are okay to use with a 5.5″ drop panel. This analysis can be seen in Appendix B-19

$$\text{Ø}V_c = 318.8 \text{ kips} > V_u = 260 \text{ kips}$$

But as will be pointed out in the *Footing Design* section, the majority of columns will need to be at least 20″x20″ unless a higher strength concrete is used. This control is due to the bearing pressure at the footing. A larger area is needed to distribute the load more evenly and prevent bearing failure.

However, it is not practical to step column sizes throughout the height of the building. It is more common to either increase the amount of reinforcing or to increase the strength of concrete before changing the size of the columns. To increase bearing strength of the footings, the strength of the concrete in the columns and footings need to be increased. This

would eliminate the need to step the column sizes. If there was a need to keep the column sizes small, potentially the strength of the columns and footings could be increased to 6000psi concrete. Were this to be the case, a 4000psi and a 6000psi concrete would need to be ordered to the job site. Though changing the strength to 6000psi is more expensive per cubic yard ($20/cy) than 4000psi concrete, the overall cost would be lower ($10,000 increase). This is because the use of the 4000psi concrete for the columns would require much more concrete, which gives an added value of $48,000. From this aspect it is more practical to use a smaller column and a higher strength. However, ordering two types of concrete to a job site leaves room for concrete placing errors. For this reason alone, 20" x 20" and 24" x 24" columns will be called out through the entire height of the building.

All columns, except for the center core, will be specified as 20" x 20" with (10) #8 reinforcing, f'_c=4000psi and can be seen below in figure #14. The core columns will be 24"x 24" with (10) #8 reinforcing, f'_c=6000psi and can be seen below in figure #15.

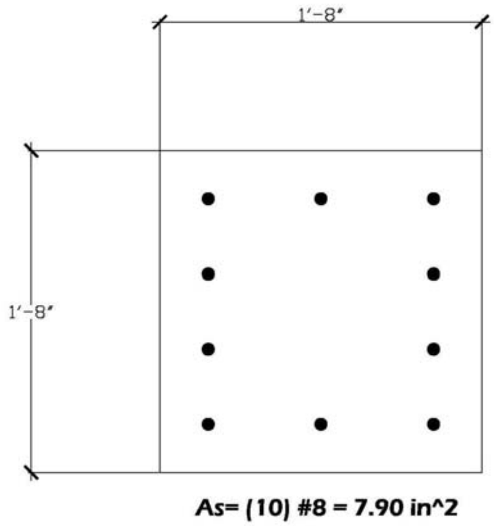


Figure #14: Column Design

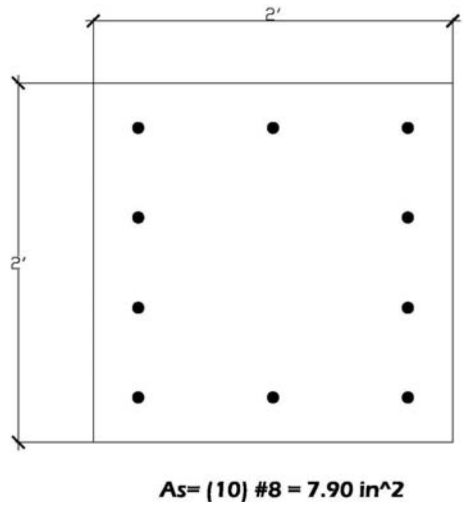


Figure #15: Core Column Design

Footing Design:

To first decide which type of footing would best suit this building, allowable soil bearing pressure was used to determine the smallest size of the simplest footing type, a spread footing. Since the building is so uniform, it was found that only three different types of spread footings really needed to be designed. Footings 1, 2, and 3 are [10′-6″ x 10′-6″], [11′-6″ x 11′-6″], and [8′-6″ x 8′-6″], respectively. The layout plan of these footings can be seen in Appendix D-6. To determine the area of these footings, divide the total load, P, by the allowable bearing pressure, in this case 6000psf. The next thing to do is to determine what area of steel is required for bending and for shear. Finally to check that the spread footing can hold the column above it without failing, bearing calculations must be performed. The results are as follows…

Footing #1:

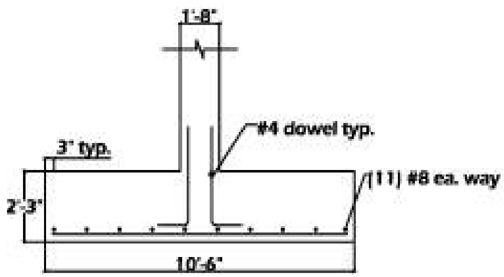


Figure #16: Footing 1 detail

Footing #2:

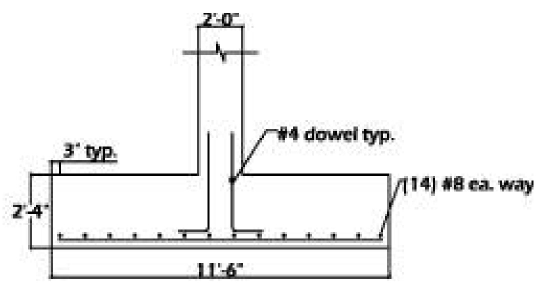


Figure #17: Footing 2 detail

Footing #3:

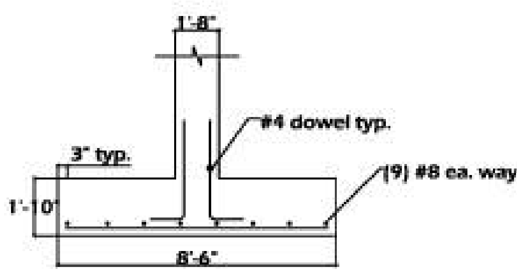


Figure #18: Footing 2 detail

This section wraps up the proposed building design section of the report. Any and all supporting calculations can be found in the appendices following the report. The next section discusses construction management issues such as cost and schedule to erect the proposed design.

Construction Management Issues

A cost estimate and a time schedule are presented to verify which system is the more efficient design for this application. Only the structural system is used for the takeoff and analysis of these estimates. This includes the concrete and reinforcement in the footings, columns, and slabs. The concrete is 4000psi concrete and the steel is 60ksi steel.

Existing Cost Data:

For the existing structure, information concerning cost was provided by Jay Deluca at R. W. Larson Associates.

Structural Costs:	***$ 1,320,000***
Labor Costs:	***$ 1,870,000***
TOTAL COSTS:	***$ 3,190,000***

Proposed Cost Data:

To calculate the proposed structural costs, R.S. Means 2005 was used. First a takeoff of the material used was performed. Then the total cost was determined and is as follows…

Structural Costs:	***$ 1,045,340***
Labor Costs:	***$ 1,478,349***
TOTAL COSTS:	***$ 2,523,689***

SAVINGS OF… **$ 666,311**

Existing Schedule Data:

For the existing structure, information concerning scheduling was provided by Jay Deluca at R. W. Larson Associates.

Existing Structural Schedule: 102 days (5.1 months)

This information is an estimate due to the fact that the building has not yet completed its construction phase.

Proposed Schedule Data:

The values are determined using R. S. Means construction data. The gross duration of the erection of the structure is estimated at 303 days. In determining the actual schedule however certain tasks can be overlapped with other tasks, giving a shorter construction time.

Proposed Structural Schedule: 197 days (9.85 months)

As compared to the existing system, the proposed concrete building would take 95 days longer. This is a significant difference in time, though it is expected. Usually concrete buildings take more time to erect because the concrete takes time to cure and there is a lot of formwork that needs to be placed before any concrete can be poured. It is difficult to compare these numbers directly because different areas are sometimes more efficient with one trade over another. Next the report will provide insight into an alternate lighting design for one of the rooms located in S&T Bank.

Interior Lighting Design

Lighting design is an essential element of the building process. Not only does lighting bright up a room and make things look attractive, they are functional too. In this section of the report, the lighting scheme for the research room on the first floor is presented. Issues concerning lighting levels, appearance, and optimal functionality are used to propose a new lighting scheme for the room.

The reason the research room was analyzed is because the performance of the workers in the space is important, and lighting has a large impact on worker output. Typical tasks in the research room would be computer work and reading. As noted from the IES Handbook, the minimum lighting levels for the performance of visual tasks of high contrast and small size is 50 foot-candles. These levels are based on healthy younger individuals. Since it is possible that older people will use this workspace, a slightly higher level of light would be ideal. Thus, target lighting levels are 50-57 foot-candles (slightly higher than the minimum).

Existing Lighting Scheme:

The existing light assembly that is used is shown below in figure #19. This particular fixture uses two T8, 32 watt fluorescent lamps. The light distribution for this light is 100% direct lighting, as is shown in the photometry report provided in Appendix G-2. To determine the lighting levels in the research room, the computer program AGI was used. Figure #20 shows the lighting levels on each

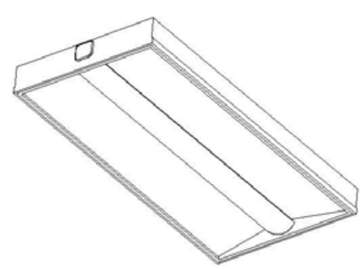
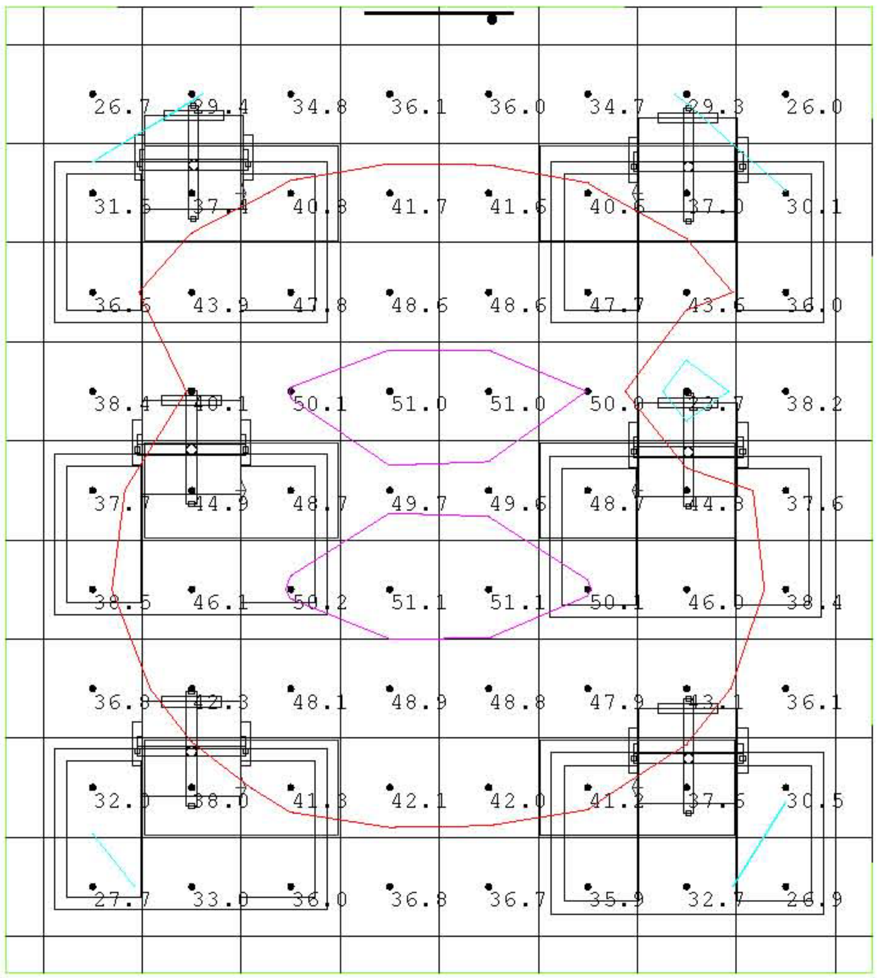


Figure #19: Existing Lighting

of the work planes. The work plane is set at 33 inches from the floor. As can be seen, the existing lighting levels range from 32.7fc – 46.1fc on the desktops. These values are less than the levels set by the IES Handbook and are less than the specified ideal levels for the room use.



<u>Figure #20: Existing Foot-Candle Levels</u>

For this workspace, direct lighting will produce glare on a computer screen and even a glossy magazine article. Fatigue can set in during a long workday when harsh conditions exist, and low lighting levels along with bright glare are some of these conditions. To actually grasp what the space

looks like, a rendering is provided in figure #21. The visual presentation of this space is good. There aren't many shadows on the walls or on the floor. Concerning the negative effects of the current lighting scheme and the goal to have the most productive workspace possible, a new lighting design is proposed.

Figure #21: Existing Layout Rendering

Proposed Lighting Conditions:

The new lighting scheme will incorporate an indirect light system to help reduce glare on the work plane. If the ceiling is lit too brightly however, this can also produce unwanted glare. As limited by the IES Handbook, ceiling levels should be no higher than 100fc. The proposed lights also incorporate two T8, 32 watt lamps. Below in Figure #22 shows a profile of the newly proposed lights.

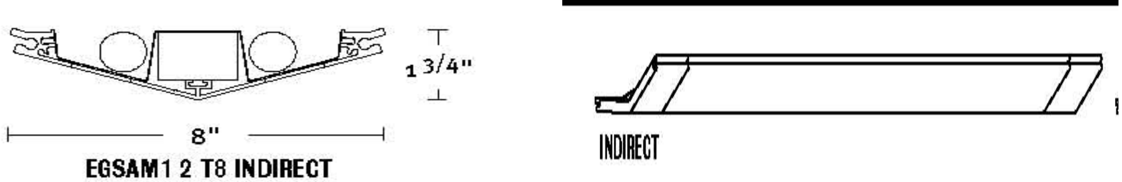


Figure #22: Proposed Light Assembly

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Indiana, PA

Figure #23 is an output from AGI that shows the actual lighting levels on the work plane. As mentioned before, the ideal range for the research room is 50-57 foot-candles. The lighting levels for this space average around 54.8 foot-candles, which is within the range specified for this space.

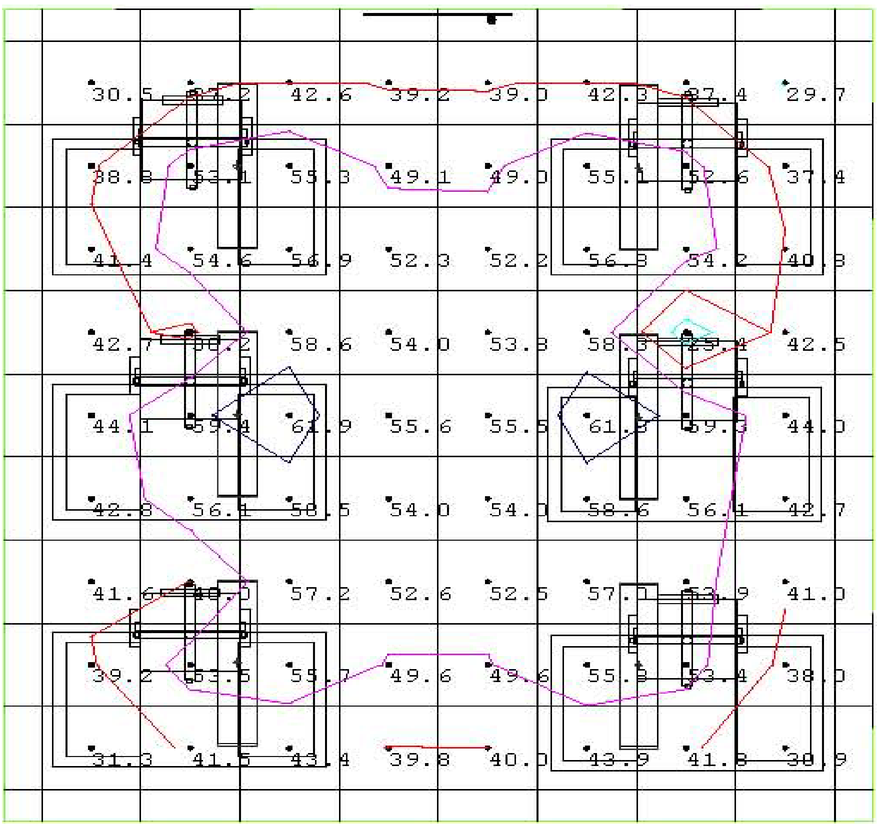


Figure #23: Proposed Foot-Candle Levels

The rendered image of the proposed space shown in figure #24 indicates that the visual representation of this space is also good. The room is slightly better visually than the existing lighting because there are practically no shadows on the wall.

Figure #24: Proposed Layout Rendering

Now that the lighting discussion is complete, recommendations and conclusions are presented. The following conclusions are based solely on the information provided in the report.

Conclusions and Recommendations

Through previous reports, it was determined that the current design is a basic system and an efficient design as well. To determine how efficient a system it is, a comparison with another system design needed to be done. The overall idea is to compare the current steel building to a proposed concrete building and see which building is a more efficient design.

The concrete structural system incorporates a two-way flat slab system with drop panels at the columns. The slab is 10″ thick and each of the drop panels is 5.5″ thick. Due to the new larger weight of the building, the footings needed to be redesigned. The footings for the proposed concrete building are spread footings which can be as big as 11′-6″ square or as small as 8′-6″ square. Typically the footings are 27″ thick at the base. Due to bearing on the footings, the columns need to be 24″ x 24″ for the central core columns and 20″ x 20″ for the rest of the columns. The columns do not need to be this large toward the top of the building but it is unreasonable to change the dimensions of the column throughout the height of the building. All concrete specified has a strength of f'_c=4000psi. The reinforcement steel is 60ksi steel. All calculations to support the design for the new proposed system can be referenced in the Appendix.

Through an investigation of cost and schedule for the proposed concrete building, a direct comparison can be made between it and the existing steel building. Through this investigation it was determined that the total cost of the steel building (including labor) was $3,190,000 and would require 102 days to complete the construction of the structural system. The proposed concrete structure would cost $2,523,689. There would be a savings of $666,311 were the concrete building to be erected.

The proposed design would require 197 days to complete construction of the structural system. Therefore the original steel design would take 95 less days to complete construction.

While evaluating the current system, it became apparent that there was a room dedicated for research on the first floor of the building. Research room workers continually perform tasks essential for the maintenance and growth of the company that include reading and computing. Since worker output is so important, an investigation of the lighting design was conducted to ensure the lighting was optimal for performance. The existing design utilized a direct lighting scheme which incorporated (2) 32 watt T8 lamps. The lighting levels on the work plane in the research room are currently 32.7fc – 46.1fc. Ideal lighting intensities range from 52fc - 57fc, as determined from the IES handbook. The proposed lighting design presents indirect fixtures which also incorporate (2) 32 watt T8 lamps. The lighting levels on the work plane in the research room that are proposed are 41.8fc – 59.3fc.

<u>Recommendations:</u>

Although the concrete design is over a half of a million dollars less expensive, it takes 95 more days to construct. During this time the client could potentially move into the space. The longer the client is not able to operate, the more money they forfeit to make. One could do a study to see how much potential profit the client would make in 95 days to accurately determine which is more cost beneficial. All in all, without further investigation neither system can be classified better than the other. The longer a building is in construction, the more opportunity there is for things to go wrong and for problems to arise. However, if money is a critical issue,

it would be advisable to use concrete for the structural system of the building.

The lighting scheme that exists is moderately acceptable. The light intensity does not meet the recommended levels at the work plane for the intended tasks. It is recommended to change the lighting system to an indirect system like the one presented in this report. With indirect lighting, there is minimal to no glare observed on computer screens or high-gloss papers such as magazines.

References:

"Manual of Steel Construction: Load and Resistance Factor Design", 3rd Edition. American Institute of Steel Construction, 2001.

"Building Code Requirements for Structural Concrete (ACI 318-05)" American Concrete Institute, December 2004.

"CRSI Design Handbook, 9th Edition" Concrete Reinforcing Steel Institute, 2002.

"Minimum Design Loads for Buildings and Other Structures" American Society of Civil Engineers, 2002.

"International Building Code 2003." International Code Council, Inc. 2002.

"Building Construction Cost Data, 62nd Annual Edition" Phillip, Waier P.E. R.S. Means Co. Inc. 2004.

"IES Lighting Handbook; the standard lighting guide" 7th Edition. Illuminating Engineering Society, New York, 1987.

Thank You!!

Thank you to the Pennsylvania State University for enabling me the opportunity to gain an education that will forever open doors that would normally be locked.

Thank you to Dr. Linda Hanagan for being supportive of the work that I was doing, and for being unwavering in her guidance and availability during the school year. She made it possible to ask questions without feeling intimidated.

Thank you to R.W. Larson Associates for always providing information in a timely manner, and for coordinating the acquisition of a set of drawings for S&T Bank.

Thank you to all of the professors who have answered random questions throughout the school year and for being flexible in class deadlines around busy thesis deadlines.

Above all, thank you to S&T Bank for permitting me to conduct my thesis research. Without them none of this could be possible.

Appendix A

TABLE C3-1
MINIMUM DESIGN DEAD LOADS*

Component	Load (psf)	Component	Load (psf)
CEILINGS		Decking, 2-in. wood (Douglas fir)	5
✓Acoustical Fiber Board	1	Decking, 3-in. wood (Douglas fir)	8
Gypsum board (per mm thickness)	0.55	Fiberboard, 1/2-in.	0.75
Mechanical duct allowance	4	Gypsum sheathing, 1/2-in.	2
Plaster on tile or concrete	5	Insulation, roof boards (per inch thickness)	
Plaster on wood lath	8	Cellular glass	0.7
Suspended steel channel system	2	Fibrous glass	1.1
Suspended metal lath and cement plaster	15	Fiberboard	1.5
Suspended metal lath and gypsum plaster	10	Perlite	0.8
Wood furring suspension system	2.5	Polystyrene foam	0.2
COVERINGS, ROOF, AND WALL		Urethane foam with skin	0.5
Asbestos-cement shingles	4	Plywood (per 1/8-in. thickness)	0.4
Asphalt shingles	2	Rigid insulation, 1/2-in.	0.75
Cement tile	16	Skylight, metal frame, 3/8-in. wire glass	8
Clay tile (for mortar add 10 psf)		Slate, 3/16-in.	7
Book tile, 2-in.	12	Slate, 1/4-in.	10
Book tile, 3-in.	20	Waterproofing membranes:	
Ludowici	10	Bituminous, gravel-covered	5.5
Roman	12	Bituminous, smooth surface	1.5
Spanish	19	Liquid applied	1
Composition:		Single-ply, sheet	0.7
Three-ply ready roofing	1	Wood sheathing (per inch thickness)	3
Four-ply felt and gravel	5.5	Wood shingles	3
Five-ply felt and gravel	6	FLOOR FILL	
Copper or tin	1	Cinder concrete, per inch	9
Corrugated asbestos-cement roofing	4	Lightweight concrete, per inch	8
Deck, metal, 20 gage	2.5	Sand, per inch	8
Deck, metal, 18 gage	3	Stone concrete, per inch	12

(continued)

Appendix A

TABLE C3-1 — continued
MINIMUM DESIGN DEAD LOADS*

Component				Load (psf)
FLOORS AND FLOOR FINISHES				
Asphalt block (2-in.), 1/2-in. mortar				30
Cement finish (1-in.) on stone-concrete fill				32
Ceramic or quarry tile (3/4-in.) on 1/2-in. mortar bed				16
Ceramic or quarry tile (3/4-in.) on 1-in. mortar bed				23
Concrete fill finish (per inch thickness)				12
Hardwood flooring, 7/7-in.				4
Linoleum or asphalt tile, 1/4-in.				1
Marble and mortar on stone-concrete fill				33
Slate (per mm thickness)				15
Solid flat tile on 1-in. mortar base				23
Subflooring, 3/4-in.				3
Terrazzo (1-1/2-in.) directly on slab				19
Terrazzo (1-in.) on stone-concrete fill				32
Terrazzo (1-in.), 2-in. stone concrete				32
Wood block (3-in.) on mastic, no fill				10
Wood block (3-in.) on 1/2-in. mortar base				16
FLOORS, WOOD-JOIST (NO PLASTER) DOUBLE WOOD FLOOR				
Joist sizes (inches):	12-in. spacing (lb/ft^2)	16-in. spacing (lb/ft^2)	24-in. spacing (lb/ft^2)	
2 × 6	6	5	5	
2 × 8	6	6	5	
2 × 10	7	6	6	
2 × 12	8	7	6	
FRAME PARTITIONS				
Movable steel partitions				4
Wood or steel studs, 1/2-in. gypsum board each side				8
Wood studs, 2 × 4, unplastered				4
Wood studs, 2 × 4, plastered one side				12
Wood studs, 2 × 4, plastered two sides				20
FRAME WALLS				
Exterior stud walls:				
2 × 4 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding				11
2 × 6 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding				12
Exterior stud walls with brick veneer				48
Windows, glass, frame and sash				8

Component					Load (psf)
Clay brick wythes:					
4 in.					39
8 in.					79
12 in.					115
16 in.					155
Hollow concrete masonry unit wythes:					
Wythe thickness (in inches)	4	6	8	10	12
Density of unit (16.49 kN/m^3)					
No grout	22	24	31	37	43
48″ o.c.		29	38	47	55
40″ o.c. grout		30	40	49	57
32″ o.c. spacing		32	42	52	61
24″ o.c.		34	46	57	67
16″ o.c.		40	53	66	79
Full Grout		55	75	95	115
Density of unit (125 pcf):					
No grout	26	28	36	44	50
48″ o.c.		33	44	54	62
40″ o.c. grout		34	45	56	65
32″ o.c. spacing		36	47	58	68
24″ o.c.		39	51	63	75
16″ o.c.		44	59	73	87
Full Grout		59	81	102	123
Density of unit (21.21 kN/m^3)					
No grout	29	30	39	47	54
48″ o.c.		36	47	57	66
40″ o.c. grout		37	48	59	69
32″ o.c. spacing		38	50	62	72
24″ o.c.		41	54	67	78
16″ o.c.		46	61	76	90
Full Grout		62	83	105	127
Solid concrete masonry unit wythes (incl.					
Wythe thickness (in mm)	4	6	8	10	12
Density of unit (105 pcf):	32	51	69	87	105
Density of unit (125 pcf):	38	60	81	102	124
Density of unit (135 pcf):	41	64	87	110	133

* Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft^2 for each face plastered. Values given represent averages. In some cases, there is a considerable range of weight for the same construction.

Appendix A

to the lateral stiffness of the runway beam and supporting structure.

4.10.4 Longitudinal Force. The longitudinal force on crane runway beams, except for bridge cranes with hand-geared bridges, shall be calculated as 10% of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam in either direction parallel to the beam.

SECTION 4.11 REFERENCES

Ref. 4-1 ANSI. (1988). "American National Standard Practice for the Inspection of Elevators, Escalators, and Moving Walks (Inspectors' Manual)." *ANSI A17.2.*

Ref. 4-2 ANSI/ASME. (1993). "American National Standard Safety Code for Elevators and Escalators." *ANSI/ASME A17.1.*

TABLE 4-1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_O, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform psf (kN/m²)	Conc. lbs (kN)
Apartments (see residential)		
Access floor systems		
Office use	50 (2.4)	2000 (8.9)
Computer use	100 (4.79)	2000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)	
Lobbies	100 (4.79)	
Movable seats	100 (4.79)	
Platforms (assembly)	100 (4.79)	
Stage floors	150 (7.18)	
Balconies (exterior)	100 (4.79)	
On one- and two-family residences only, and not exceeding 100 ft.² (9.3 m²)	60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwellings (see residential)		
Elevator machine room grating (on area of 4 in.² (2580 mm²))		300 (1.33)
Finish light floor plate construction (on area of 1 in.² (645 mm²))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders		See Section 4.4
Garages (passenger vehicles only)	40 (1.92)	Note (1)
Trucks and buses	Note (2)	

(*continued*)

Appendix A

TABLE 4-1 — continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_O, AND MINIMUM CONCENTRATED LIVE LOADS

Occupancy or Use	Uniform psf (kN/m²)	Conc. lbs (kN)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	100 (4.79) Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals		
Operating rooms, laboratories	60 (2.87)	1000 (4.45)
Private rooms	40 (1.92)	1000 (4.45)
Wards	40 (1.92)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Hotels (see residential)		
Libraries		
Reading rooms	60 (2.87)	1000 (4.45)
Stack rooms	150 (7.18) Note (3)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Manufacturing		
Light	125 (6.00)	2000 (8.90)
Heavy	250 (11.97)	3000 (13.40)
Marquees and canopies	75 (3.59)	
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100 (4.79)	2000 (8.90)
Offices	50 (2.40)	2000 (8.90)
Corridors above first floor	80 (3.83)	2000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79) Note (4)	
Roofs	See Sections 4.3 and 4.9	

(continued)

Appendix A

Wind Load Analysis | S&T Bank

$h \leq 60'$ → use simplified wind load method

$V = 90$ mph (40 m/s)
$I_w = 1.0$
Exposure Category B
$\lambda = 1.22$
$P_{s30} = -6.7$ psf

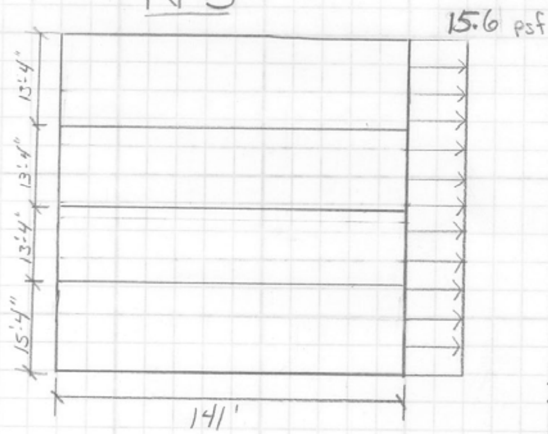
$P_s = \lambda I_w P_{s30}$

$= 1.22(1.0)(-6.7)$

$P_s = -8.174$ psf

Must check wind Loads @ each surface (IBC2003-1609.6.2.1)

N-S

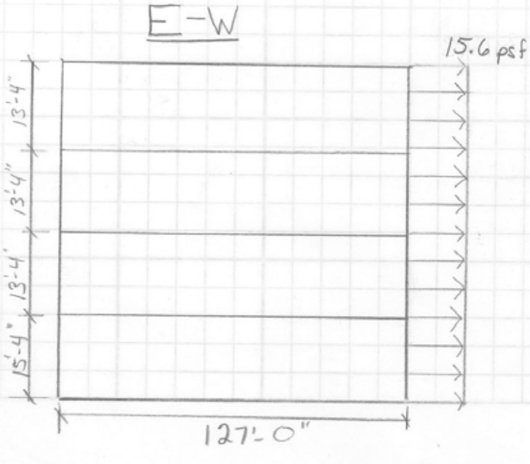


MWFRS

Zone	Adjusted Pressure P_s
A	15.6 psf
B	-8.2 psf
C	10.37 psf
D	-4.5 psf
E	-18.8 psf
F	10.7 psf
G	-13.1 psf
H	-8.3 psf

Zone A is the significant wall loading

E-W



Cladding

Zone	Adjusted Pressure
Wall	17.8 psf -19.3 psf
Corner	17.8 psf -23.4 psf

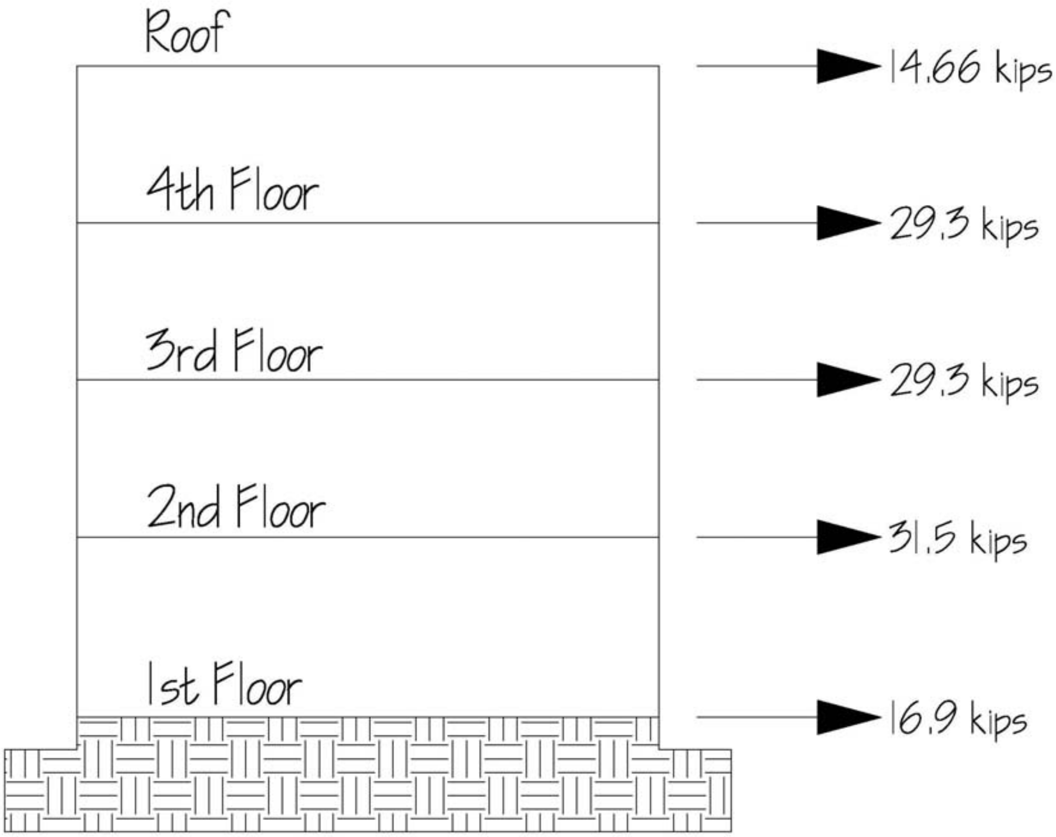
Appendix A

Wind Load Analysis

Floor	Tributary Area
1^{st}	$(141)(15.33/2) = 1080.76$ SF
2^{nd}	$(141)\left(\frac{13.33+15.33}{2}\right) = 2020.53$ SF
3^{rd}	$(141)(13.33) = 1879.53$ SF
4^{th}	$(141)(13.33) = 1879.53$ SF
Roof	$(141)(13.33)(.5) = 939.76$ SF

Floor	Wind Load
1^{st}	$1080.76(15.6)/1000 = 16.9$ Kips
2^{nd}	$2020.53(15.6)/1000 = 31.5$ Kips
3^{rd}	$1879.53(15.6)/1000 = 29.3$ Kips
4^{th}	$1879.53(15.6)/1000 = 29.3$ Kips
Roof	$939.76(15.6)/1000 = 14.66$ Kips

Appendix A



Roof
14.66 kips
4th Floor
29.3 kips
3rd Floor
29.3 kips
2nd Floor
31.5 kips
1st Floor
16.9 kips

Appendix A

Proposed Seismic Loads

Seismic Use Group : 1
Importance Factor : 1
$S_s = 12.7$
$S_1 = 5.4$
Site Class C

$F_a = 1.2$
$F_v = 1.7$

$S_{ms} = F_a S_s$
$= 1.2(12.7) = 15.24\%$

$S_{m1} = F_v S_1$
$= 1.7(5.4) = 9.18\%$

$S_{ds} = 2/3 (S_{ms}) = 10.16\%$

$S_{D1} = 2/3 (S_{m1}) = 6.12\%$

$S_A = 10.16$

$T_a = .1(\#\text{stories}) = .1(4) = .4$

$T_0 = \dfrac{.2(S_{D1})}{(S_{DS})} = 0.121$

$T_s = S_{D1}/S_{DS} = 6.21/10.16 = .611$

$T_0 < T_a < T_s \therefore S_a = S_{DS}$

Design Category A :
Ordinary

$R^a = 6 \quad \Omega = 2.5 \quad C_d = 5$

$C_s = S_{DS}/(R/I)$
$= .1016/(6/1.0)$
$= .0169$

concrete

$W = (2950\,cy \times 27\,ft^3/cy \times 150)$
$+ (250{,}000\text{ lbs})$
Rebar

$= 11{,}950{,}000\text{ lbs} + 250{,}000\text{ lbs}$
$= 12{,}200\text{ Kips}$

$V = C_s W$
$= .0169(12{,}200)$
$V = 206.2\text{ kips}$

Base Shear : $V = 206.2$ kips

$T_a = 0.1N$
$= .1(4) = .4 \rightarrow k = 1$

Appendix A

Proposed Seismic Loads

Roof = 1,525,000 lbs @ 9.67' $\qquad w_r h_r^k = 14{,}746{,}750$#

4th = 2,668,750 lbs @ 13.33' $\qquad w_4 h_4^k = 35{,}574{,}438$#

3rd = 2,668,750 lbs @ 13.33' $\qquad w_3 h_3^k = 35{,}574{,}438$#

2nd = 2,668,750 lbs @ 15.33' $\qquad w_2 h_2^k = 40{,}911{,}938$#

$F_x = C_{v_x} V$, $k = 1$ $\qquad \Sigma w_i h_i^k = 126{,}808$ kips

$$C_{v_x} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$$

$C_{v_r} = 0.1163$ $\qquad F_r = (0.1163)(206.2 \text{ kip}) = 23.98$ kips

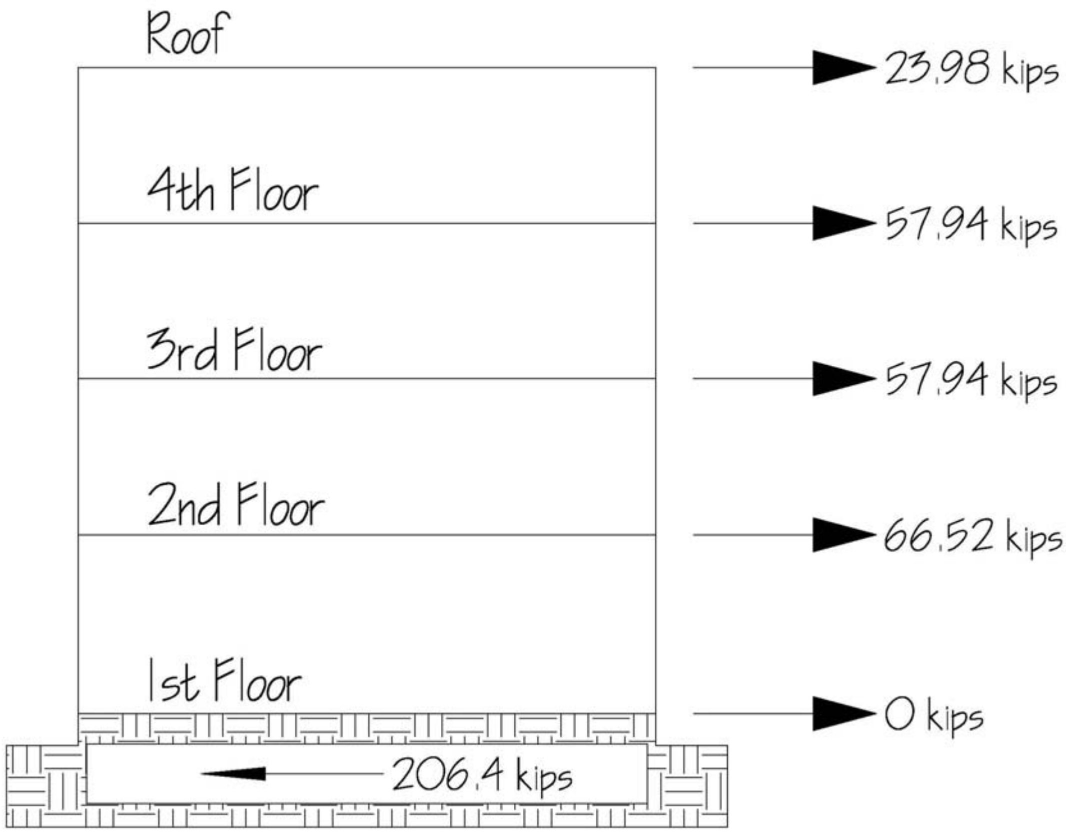
$C_{v_4} = 0.2810$ $\qquad F_4 = (0.2810)(206.2 \text{ kips}) = 57.94$ kips

$C_{v_3} = 0.2810$ $\qquad F_3 = (0.2810)(206.2 \text{ kip}) = 57.94$ kips

$C_{v_2} = 0.3226$ $\qquad F_2 = (0.3226)(206.2 \text{ kip}) = 66.52$ kips

$C_{v_1} = 0$ $\qquad F_1 = 0$

Appendix A


Roof
23.98 kips
4th Floor
57.94 kips
3rd Floor
57.94 kips
2nd Floor
66.52 kips
1st Floor
0 kips
206.4 kips

Appendix B

```
FILE NAME            P:\STBANK3.ADS

PROJECT ID.          S&T Bank
                     -----------------------------------
SPAN ID.             3 West-East 3rd fl
                     -------------------

ENGINEER             Daniel Hancock

DATE                 02/09/06
TIME                 12:49:19

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

SLAB SYSTEM          FLAT SLAB SYSTEM
FRAME LOCATION       INTERIOR

DESIGN METHOD        STRENGTH DESIGN
MOMENTS AND SHEARS   NOT PROPORTIONED

NUMBER OF SPANS  7

  SOLID HEAD DIMENSIONS :       COMPUTED BY PROGRAM


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


REINFORCEMENT DETAILS: NON-PRESTRESSED
  YIELD STRENGTH Fy  =  60.00 ksi
  DISTANCE TO RF CENTER FROM TENSION FACE:
       AT SLAB TOP    =   1.50 in  INNER LAYER
       AT SLAB BOTTOM =   1.50 in  INNER LAYER
  MINIMUM FLEXURAL BAR SIZE:
       AT SLAB TOP    =  # 4
       AT SLAB BOTTOM =  # 4
  MINIMUM SPACING:
       IN SLAB =   6.00 in
```

Appendix B

SPAN/LOADING DATA
****

SPAN NUMBER	LENGTH L1 (ft)	Tslab (in)	WIDTH LEFT (ft)	L2*** RIGHT (ft)	SLAB SYSTEM	DESIGN STRIP (ft)	COLUMN STRIP** (ft)	UNIFORM LOADS S. DL (psf)	LIVE (psf)
1*	1.5	10.0	15.0	14.0	2	29.0	.0	12.0	100.0
2	12.0	10.0	15.0	14.0	2	29.0	6.0	12.0	100.0
3	28.0	10.0	15.0	14.0	2	29.0	14.0	12.0	100.0
4	28.0	10.0	15.0	14.0	2	29.0	14.0	12.0	100.0
5	28.0	10.0	15.0	14.0	2	29.0	14.0	12.0	100.0
6	28.0	10.0	15.0	14.0	2	29.0	14.0	12.0	100.0
7*	1.5	10.0	15.0	14.0	2	29.0	.0	12.0	100.0

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

PARTIAL LOADING DATA
****

PARTIAL LOADINGS ARE NOT SPECIFIED

Appendix B

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

LATERAL LOADS ARE SPECIFIED AS BEING CAUSED BY WIND

LATERAL LOAD FROM FLOORS ABOVE (Pa) =     81.92 kips

LATERAL LOAD AT THIS FLOOR     (Pb) =     57.94 kips

NOTE: The analysis procedure adopted by the program is approximate.

LATERAL LOADS DISTRIBUTED TO THE COLUMN AND MIDDLE STRIPS ACCORDING TO
CODE DISTRIBUTION FACTORS.



OUTPUT DATA
PATTERN LOADINGS:                1 THRU 8
PATTERN LIVE LOAD FACTOR (1-3)  =    75%

LOAD FACTORS:
  U = 1.40*D + 1.70*L
  U =  .75( 1.40*D + 1.70*L + 1.70*W)
  U =  .90*D + 1.30*W

OUTPUT OPTION(S):
  Input Echo
  Centerline Moments and Shears
  Column Strip Distribution Fac
  Shear Table
  Reinforcing Required
  Bar Sizing
  Additional Information
  Deflections
  Material Quantities
**DROP NOT SPECIFIED AT COLUMN  1

**SPECIFIED DROP DEPTH AT COLUMN  2 GREATER THAN 1/4TH THE SUPPORT-DROP
EDGE
DISTANCE, EXCESS DEPTH ON SPAN  2 SIDE IGNORED FOR REINFORCEMENT
CALCULATIONS.
**DROP NOT SPECIFIED AT COLUMN  6

**SLAB SPAN  2 IS NOT A TWO WAY SYSTEM.
THE SLAB DESIGN MUST BE PERFORMED MANUALLY.

**TOTAL UNFACTORED DEAD LOAD =         541.607 kips
                   LIVE LOAD =         368.300 kips
```

Appendix B

---- STATICS PRINT-OUT FOR GRAVITY/LATERAL LOAD ANALYSIS ----
****

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

JOINT NUMBER	PATTERN-5 LEFT	RIGHT	TOP	BOTTOM	PATTERN-6 LEFT	RIGHT	TOP	BOTTOM
1	-6.8	-272.7	100.4	179.2	-6.8	258.9	-86.6	-165.4
2	-434.7	114.3	107.7	212.8	86.8	323.6	-149.4	-261.0
3	-409.8	169.7	85.5	154.6	-179.0	400.6	-76.9	-144.7
4	-375.4	141.9	82.4	151.1	-144.6	372.8	-80.0	-148.2
5	-413.7	204.3	71.3	138.2	-180.1	432.3	-91.1	-161.1
6	-278.9	6.8	118.2	153.9	-38.5	6.8	33.6	-2.0

JOINT NUMBER	PATTERN-7 LEFT	RIGHT	TOP	BOTTOM	PATTERN-8 LEFT	RIGHT	TOP	BOTTOM
1	-12.1	-278.1	106.5	183.8	-12.1	243.3	-76.9	-154.2
2	-634.8	374.2	80.7	179.9	-123.3	579.5	-171.4	-284.8
3	-754.2	507.7	89.0	157.5	-527.8	734.1	-70.3	-136.0
4	-679.2	446.9	82.4	150.0	-452.8	673.3	-76.9	-143.6
5	-761.8	582.5	57.8	121.5	-532.6	806.2	-101.5	-172.1
6	-464.0	12.1	208.4	243.4	-228.2	12.1	125.5	90.5

Appendix B

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

JOINT	PATTERN-5		PATTERN-6		PATTERN-7		PATTERN-8	
NUMBER	LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT
1	-9.0	-34.0	-9.0	53.7	-16.0	-24.8	-16.0	61.3
2	-81.5	42.1	6.2	57.8	-124.6	99.6	-38.5	115.0
3	-63.2	45.3	-47.4	61.8	-126.7	107.0	-111.3	123.2
4	-60.0	42.9	-43.5	59.5	-119.3	101.9	-103.1	118.2
5	-62.3	49.8	-45.7	66.5	-124.4	117.2	-108.1	133.6
6	-56.5	9.0	-39.8	9.0	-110.3	16.0	-93.9	16.0

** - Negative moment encountered in span, analyze manually.

Appendix B

DESIGN MOMENT ENVELOPES AT CRITICAL SECTIONS FROM SUPPORTS

COL NUM	LOAD TYPE		CROSS SECTN	DESIGN MOMENT (ft-k)	DISTANCE CR.SECTN (ft)	LOAD PTRN	MAX.I.P. DISTANCE (ft)	LOAD PTRN
1	TOTL	LEFT	TOP	-11.0	.262	4	1.500	1
			BOT	.0	.000	0	.000	0
		RGHT	TOP	216.1	.833	6	6.600	6
			BOT	-253.8	.833	7	6.000	5
	VERT	LEFT	TOP	-11.0	.262	4	1.500	1
			BOT	.0	.000	0	.000	0
		RGHT	TOP	.0	.000	0	4.800	4
			BOT	-52.8	.833	2	.000	0
2	TOTL	LEFT	TOP	-533.8	.833	7	6.600	7
			BOT	80.2	.833	6	5.400	6
		RGHT	TOP	520.5	.833	4	7.000	6
			BOT	.0	.000	0	4.200	7
	VERT	LEFT	TOP	-418.6	.833	4	.000	0
			BOT	.0	.000	0	.000	0
		RGHT	TOP	520.5	.833	4	5.600	2
			BOT	.0	.000	0	.000	0
3	TOTL	LEFT	TOP	-726.3	.833	4	8.400	3

			BOT	.0	.000	0	5.600
8							
		RGHT	TOP	703.9	.833	4	9.800
2							
			BOT	.0	.000	0	5.600
7							
		VERT LEFT	TOP	-726.3	.833	4	8.400
3							
			BOT	.0	.000	0	.000
0							
		RGHT	TOP	703.9	.833	4	9.800
2							
			BOT	.0	.000	0	.000
0							
	4	TOTL LEFT	TOP	-635.0	.833	4	8.400
2							
			BOT	.0	.000	0	5.600
8							
		RGHT	TOP	628.4	.833	4	8.400
3							
			BOT	.0	.000	0	5.600
7							

Appendix B

DESIGN MOMENT ENVELOPES AT CRITICAL SECTIONS FROM SUPPORTS

COL NUM	LOAD TYPE	CROSS SECTN		DESIGN MOMENT (ft-k)	DISTANCE CR.SECTN (ft)	LOAD PTRN	MAX.I.P. DISTANCE (ft)	LOAD PTRN
	VERT	LEFT	TOP	-635.0	.833	4	8.400	2
			BOT	.0	.000	0	.000	0
		RGHT	TOP	628.4	.833	4	8.400	3
			BOT	.0	.000	0	.000	0
5	TOTL	LEFT	TOP	-737.6	.833	4	9.800	3
			BOT	.0	.000	0	7.000	8
		RGHT	TOP	790.3	.833	4	8.400	2
			BOT	.0	.000	0	7.000	7
	VERT	LEFT	TOP	-737.6	.833	4	9.800	3
			BOT	.0	.000	0	.000	0
		RGHT	TOP	790.3	.833	4	8.400	2
			BOT	.0	.000	0	.000	0
6	TOTL	LEFT	TOP	-375.7	.833	7	7.000	5
			BOT	.0	.000	0	2.800	8
		RGHT	TOP	11.0	.262	4	1.500	1
			BOT	.0	.000	0	.000	0
	VERT	LEFT	TOP	-352.9	.833	4	4.200	2

```
                    BOT          .0        .000      0         .000
0

              RGHT  TOP        11.0        .262      4        1.500
1
                    BOT          .0        .000      0         .000
0

 ----------------------------------------------------------------------
--------
```

Appendix B

S H E A R   A N A L Y S I S
****

D I R E C T   S H E A R   W I T H   T R A N S F E R   O F   M O M E N T

- - - - - - - - - - -   A R O U N D   C O L U M N   - - - - - - - - - -

COL. NO.	ALLOW. STRESS (psi)	PATT NO.	REACTION (kips)	SHEAR STRESS (psi)	PATT NO.	REACTION (kips)	UNBAL. MOMENT (ft-k)	SHEAR TRANSFR (ft-k)	SHEAR STRESS (psi)
1E	252.96	8	75.9	63.31	7	.0	-293.8	-117.5	201.20
2I	252.96	4	248.8	153.76	8	151.3	456.2	182.5	208.02
3I	252.96	4	309.2	191.07	7	231.5	-246.5	-98.6	204.95
4I	252.96	4	292.0	180.44	7	219.0	-232.3	-92.9	193.63
5I	252.96	4	319.3	197.31	8	239.6	273.6	109.4	216.70
6E	252.96	4	155.7	129.83	4	155.7	-382.9	-153.2	260.95*

* - Shear stress exceeded.

- - AROUND DROP/SOLID HEAD - -

COLUMN NUMBER	ALLOW. STRESS (psi)	PATT NO.	REACTION (kips)	SHEAR STRESS (psi)
1	Not applicable, drop dimensions not specified.			
2I	176.95	4	218.4	70.95
3I	170.36	4	266.3	75.21
4I	170.36	4	249.1	70.35
5I	170.36	4	276.4	78.06
6	Not applicable, drop dimensions not specified.			

Appendix B

T R A N S V E R S E   B E A M   S H E A R   A N D   T O R S I O N
R E Q U I R E M E N T S (kips, ft-k, SQ.in, /,in.)

****

LEFT SIDE

BEAM No.	PATT. NO.	Vu@d SHEAR	Vc@d SHEAR	Tu@d TORSION	Tc@d TORSION	Av/s @d	At/s @d	Atot/s @d	Al @d
1	7	26.4	13.5	118.7	60.8	.030*	.024	.066	2.64
2	* *	Transverse beam not specified							* *
3	* *	Transverse beam not specified							* *
4	* *	Transverse beam not specified							* *
5	* *	Transverse beam not specified							* *
6	4	50.8	16.8	182.1	60.3	.043	.047	.137	4.39

RIGHT SIDE

BEAM No.	PATT. NO.	Vu@d SHEAR	Vc@d SHEAR	Tu@d TORSION	Tc@d TORSION	Av/s @d	At/s @d	Atot/s @d	Al @d
1	7	24.2	11.6	127.2	61.1	.030*	.027	.071	2.53
2	* *	Transverse beam not specified							* *
3	* *	Transverse beam not specified							* *
4	* *	Transverse beam not specified							* *
5	* *	Transverse beam not specified							* *
6	4	44.8	14.0	195.1	60.7	.039	.051	.142	4.81

NOTES: 1.) Deep beam analysis not considered.
2.) Loads assumed applied from above beam.
3.) Moment and shear at concentrated load must be checked manually if located along transverse beam.

4.) Symbols following Av/s values:
    * - Minimum shear 50.*bw/Fyv - based on beam dimensions.
    x - Vs exceeds 4*Vc, increase member section.
5.) Symbols following At/s values:
    * - Minimum torsion 50.*bw/Fyv - based on beam dimensions.
    x - Ts exceeds 4*Tc, increase member section.
6.) Symbols following Atot/s values:
    * - Minimum torsion 50.*bw/Fyv - based on beam dimensions.
7.) Redistribution of torque is not considered.
8.) Detail first stirrup @ 3 inches.

Appendix B

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

COLUMN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LEFT (ft)	RIGHT (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LEFT (ft)	RIGHT (ft)
1**	5	# 7	1.50	7.47	5	# 7	1.50	4.18	25	# 4	1.50	7.31
2**	4	# 9	9.52	9.52	4	# 9	6.10	6.10	25	# 4	8.25	8.65
3	11	# 6	10.05	11.45	10	# 6	6.10	6.55	25	# 4	10.05	11.45
4	9	# 6	10.05	10.05	9	# 6	6.10	6.10	22	# 4	10.05	10.05
5	12	# 6	11.45	10.05	11	# 6	6.55	6.10	27	# 4	11.45	10.05
6	11	# 6	8.73	1.50	11	# 6	6.10	1.50	16	# 4	8.65	1.50

** - Positive reinforcement required, design manually.

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

SPAN NUMBER	COLUMN STRIP LONG BARS NO	SIZE	LENGTH (ft)	COLUMN STRIP SHORT BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP LONG BARS NO	SIZE	LENGTH (ft)	MIDDLE STRIP SHORT BARS NO	SIZE	LENGTH (ft)
2**	4	# 4	11.42	3	# 4	10.17	13	# 4	11.92	12	# 4	9.87
3	12	# 4	21.00	12	# 4	21.00	8	# 4	28.50	8	# 4	19.60
4	12	# 4	21.00	11	# 4	21.00	8	# 4	28.50	8	# 4	19.60
5	12	# 4	21.00	11	# 4	21.00	8	# 4	28.50	8	# 4	19.60
6	10	# 5	27.42	9	# 5	24.17	10	# 4	27.92	9	# 4	23.47

** - Negative reinforcement required, design manually.

Appendix B

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

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

--Spans 1 and 7 are cantilevers.

--Time-dependent deflections are in addition to those shown and must be computed as a multiplier of the dead load(DL) deflection. See "CODE" for range of multipliers.

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

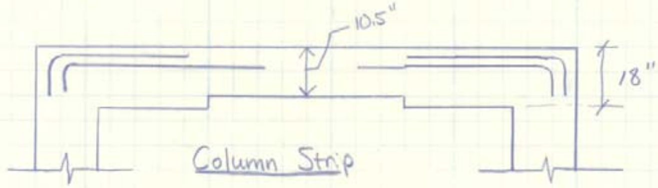
--Modulus of elasticity of concrete, Ec = 3834. ksi

SPAN NUMBER	DEAD LOAD Ieff. (in^4)	COLUMN STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)	MIDDLE STRIP DEFLECTION DUE TO: DEAD (in)	LIVE (in)	TOTAL (in)
1	29000.	.001	.001	.002	.001	.001	.002
2	37498.	.002	.001	.002	-.007	-.005	-.012
3	45996.	.111	.116	.227	.060	.060	.120
4	45996.	.089	.126	.215	.039	.056	.094
5	45996.	.083	.123	.206	.033	.052	.084
6	37498.	.163	.236	.400	.087	.107	.194
7	29000.	-.015	-.011	-.026	-.015	-.011	-.026

* Program completed as requested *

Appendix B

Two-Way Solid Flat Slabs - w/ drop Panels   CRSI Handbook



Superimposed DL = 12 psf $w_u = 1.4DL + 1.7LL$ (for CRSI tables)

LL = 100 psf $= 1.4(12) + 1.7(100)$

$= \underline{186.8 \text{ psf}}$

Enter tables w/ span = 28', load = 186.8 psf

⟹ use slab: 10.5"

Drop Panel: 9'-4" x 9'-4"
depth: 7.5"

column: 18" x 18"

Reinforcement:
Top = 15 - #6
Bottom = 12 - #6 } column Strip

Top = 13 - #5
Bottom = 11 - #5 } middle Strip

total steel: 3.07 psf

Total depth: 18"

$f'_c = 4000$ psi

$f_y = 60$ ksi

Appendix B

Direct Design Method 1

$f_y = 60{,}000$ psi   Interior Panel w/ drop Panels

min thickness $l_n/36 = 9.33$ in $< 10"$ ok ✓

Self-weight : $(150\text{pcf})(10/12) + 6\text{psf} = 131$ psf   *Note 6psf allowance for drop panel dist over entire bay.

$w_u = 1.2(131+12) + 1.6(100)$

$= .332$ ksf

$M_o = \dfrac{w_u l_2 l_n^2}{8}$ $l_2 = l_1 = 28'$ $l_n = 28' - \frac{20"}{12} = 26.33'$

$= \dfrac{.332(28')(26.33)^2}{8}$

$= 800.7$ ft·kips

Interior Support $(65\% M_u) = .65(800.7) = 520.5$ ft·k (neg reinf)

Midspan $(35\% M_u) = .35(800.7) = 280.3$ ft·k (pos reinf)

Column Strip Moment @ support $= (.75)(.65)M_o = 390.3$ ft·k

Column Strip Moment @ Midspan: $(.60)(.35)M_o = 168.2$ ft·k

$A_s = 20\#6\text{'s} = 8.8 \text{ in}^2$

$d: 10" + 5.5" - .75(\text{cover}) - .5(\text{outer layer}) - .375(\tfrac{1}{2} b_d) = 13.875$

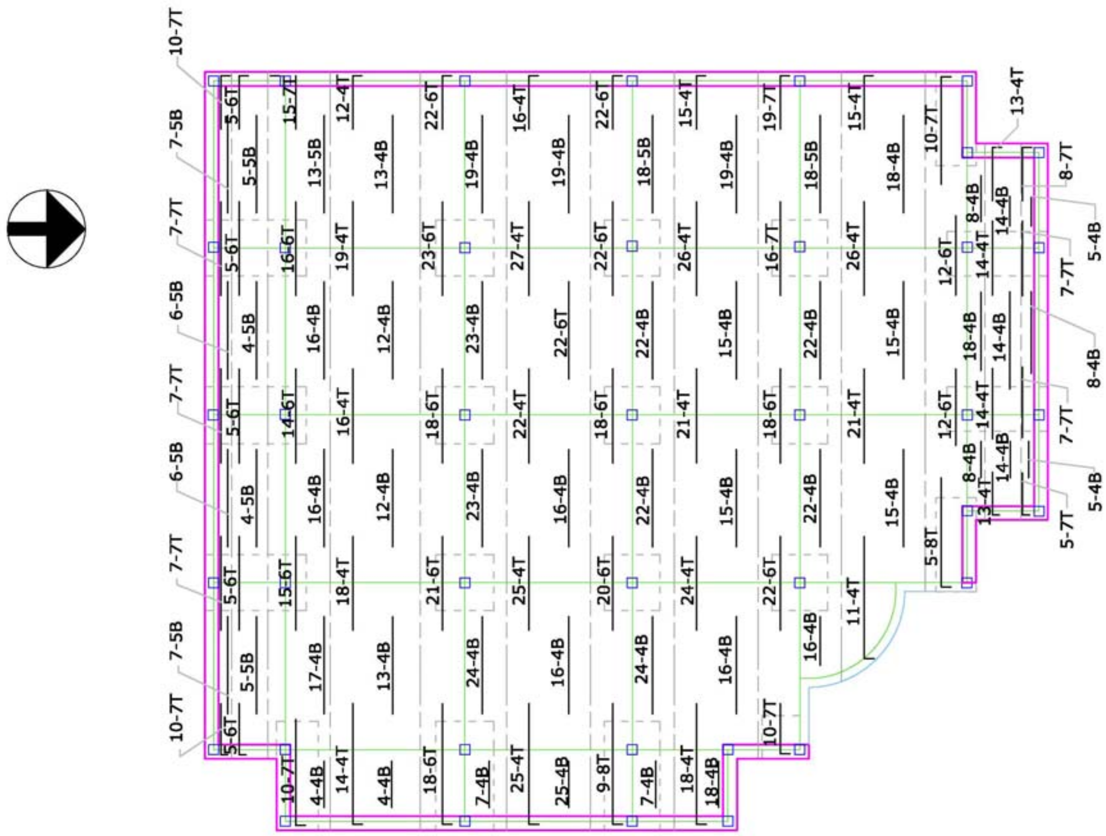
$a: \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{8.8(60)}{.85(4)(168 in)} = .924$

$M_n = A_s f_y (d - a/2) = 8.8(60)(13.875 - .462) = 590.2$ ft·k

$\phi M_n = 531.2$ ft·k > 520.5 ft·k ✓ok

ADOSS OUTPUT is ACCURATE!

Appendix B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7-7T
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19-4B
23-4B
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24-4B
7-4B
16-4T
27-4T
22-4T
25-4T
25-4T
19-4B
22-6T
16-4B
16-4B
25-4B
22-6T
22-6T
18-6T
20-6T
9-8T
18-5B
22-4B
22-4B
24-4B
7-4B
15-4T
26-4T
21-4T
24-4T
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15-4B
15-4B
16-4B
18-4B
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22-4B
22-4B
16-4B
15-4T
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21-4T
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15-4B
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14-4T
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13-4T
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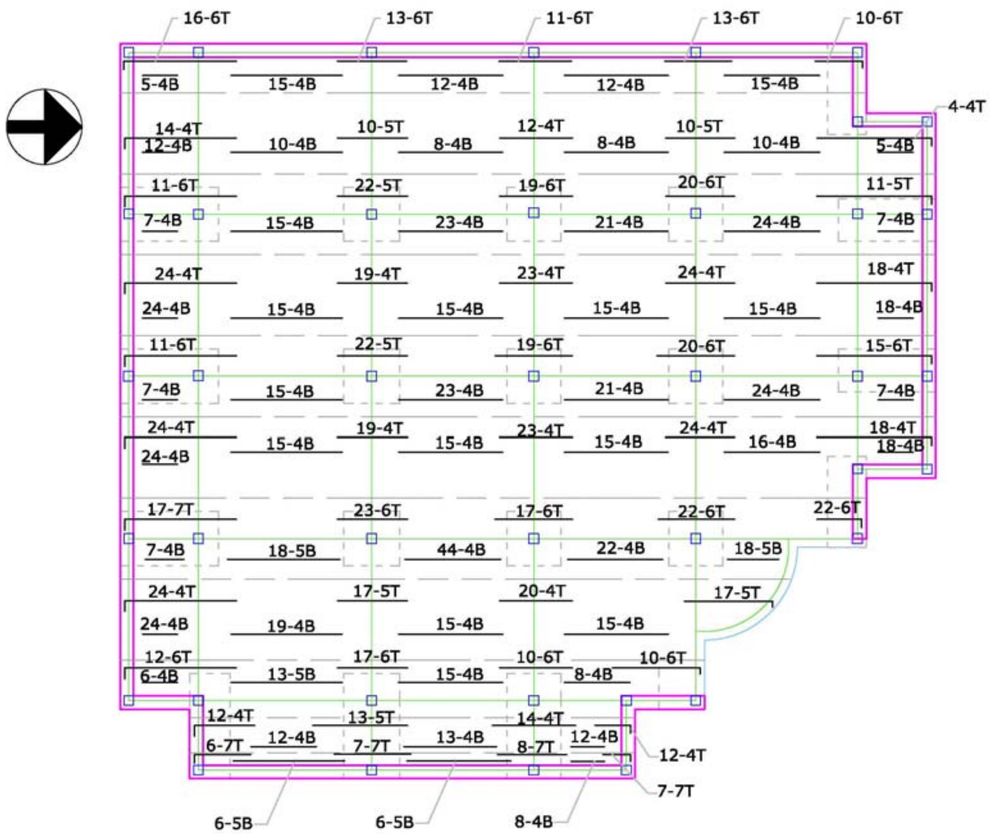
5-4B
7-7T
8-4B
7-7T
5-4B
5-7T

Appendix B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13-4B
8-7T
12-4B
12-4T
7-7T
6-5B
6-5B
8-4B

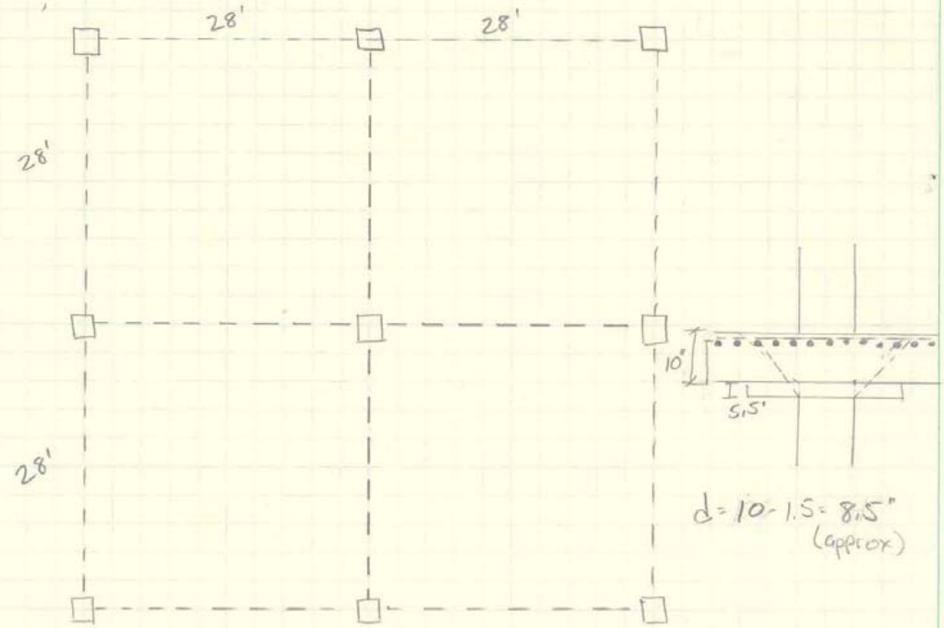
Appendix B

Punching Shear

1

DL: Self-weight $= 150\text{pcf}\,(10''/12) = 125\text{ psf}$ LL $= 100$ psf (lobby)

Superimposed $=$ 12 psf



$d = 10 - 1.5 = 8.5''$ (approx)

$W_u = 1.2(137) + 1.6(100) = .325\text{ ksf}$

Interior Panel w/ drops

$t \geq l_n/36 = (28 \times 12)/36 = 9.33\text{ in} \rightarrow$ ok used 10"

Size of Panel 9'-4" square $b_o = 4(16+14) = 120\text{ in}$

$$V_u = 1.2\left(\left(.125\text{ ksf} + .012\text{ ksf}\right)\left(28^2 - 2^2\right) + (.069)\left(9.33^2 - 2^2\right)\right) + 1.6\left(\left(.1\text{ ksf}\right)\left(28^2 - 2^2\right)\right) = 260\text{ kips}$$

$$\phi V_c = .75\,(4)\left(\sqrt{4000}\right)(120\text{ in})(14\text{ in}) = 318.8\text{ kips}$$

Punching Shear is not a problem w/ 16×16 columns w/ 5½" drop Panels + 10" slab. However increase to 20"×20" column for consistency throughout height of bldg

Appendix C

Torsional Member Design

Design for flexure, shear, + Torsion

Dead Load

Self-weight: $(150\,pcf)(10/12) = 125\,psf$
Superimposed: $= 12\,psf$
Total: $137\,psf$

$W_u = 1.2(137) + 1.6(100\,psf) = 325\,psf$

Check use of Direct Design Method

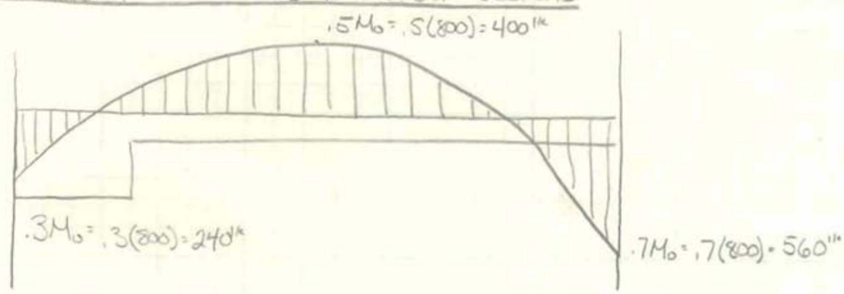
1. Ratio of longer span/shorter span $= 28'/28' = 1.0 \leq 2.0 \rightarrow$ 2-way action
2. More than 3 panels in each direction. ✓
3. $W_D < 2W_L$: $137 < 200$ ✓

Determine Static Moment

$l_1 = 28'$ $l_2 = 28'$ $l_n = (28' - 18/12) = 26.5' > .65\,l_1$

$$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{.325(28')(26.5)^2}{8} = 798.81 \rightarrow \text{say } 800^{1k}$$

Determination of Moments to Critical Sections



* The Ext. Factored negative moment (240^{1k}) is transmitted to the edge beam in the form of Torsion

"Short Span"
Interior Support $M_u^- = .65\,M_o = .65(421) = 274^{1k}$
Midspan Support $M_u^+ = .35\,M_o = .35(421) = 147^{1k}$

$$M_o = \frac{.325(14.75')(26.5)^2}{8} = 421^{1k}$$

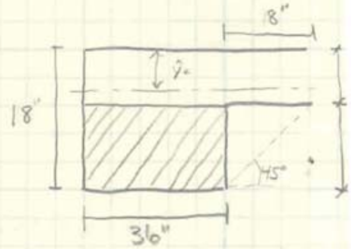
Appendix C

Torsional Member Design

Lateral Distribution of Moments to Column + Middle Strip

$I_{slab} = \frac{bh^3}{12} = \frac{(177)(10)^3}{12} = 14{,}750\ in^4$

$[177" = (28'/12)/2 + \frac{18"}{2}]$



Beam Moment of Inertia $\quad y = \left(\frac{\Sigma A\bar{y}}{\Sigma A}\right)$

$y_c = [(18)(36)(9) + (8)(10)(5)] / [(36 \times 18) + (10 \times 8)]$

$= 8.56$

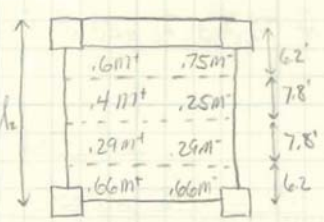
$I_b = \frac{(36)(18)^3}{12} + (36)(18)(9 - 8.56)^2 + \frac{(8)(10)^3}{12} + (8)(10)(8.56 - 5)^2$

$I_b = 19{,}302\ in^4 \qquad I = \frac{bh^3}{12} + bh(y - \bar{y})^2$

$\alpha_{f_1} = \frac{E_{cb} I_b}{E_{cs} I_s} = \frac{I_b}{I_s} = \frac{19302}{14750} = 1.309$

$l_2/l_1 = 1.0 \qquad \frac{\alpha_{f_1} l_2}{l_1} = 1.309(1.0) = 1.309 > 1.0$

∴ Portion of Interior ⊖ factored moment in C.S. $= .75 - (1.309 - 1)(.75 - .45) = .66$
Portion of Int ⊕ factored moment in C.S $= .66$



Neg. Moment for edge C.S. $= .66(274) = 165^{1k}$

Pos span Moment for edge C.S $= .66(147) = 97^{1k}$

$\frac{\alpha_{f_1} l_2}{l_1} > 1.0 \Rightarrow$ beam shall resist 85% of CS factored moment.

$M^- = .85(165) = 140.25^{1k}$

$M^+ = .85(97) = 82.45^{1k}$

Appendix C

Torsional Member Design

Account for weight of Projected beam stem below slab:

$1.2(150\text{pcf})(8)(36)/144 = 360 \text{ plf}$

$M_u^- = \dfrac{w_u \ell^2}{11} = \dfrac{.360(24.75)^2}{11} = 20^{'k}$

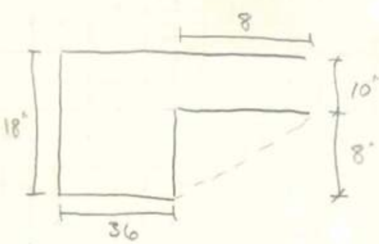
$M_u^+ = \dfrac{w_u \ell^2}{16} = 14^{'k}$

Total beam Design Moments: $M_u^- = 140.25 + 20 = 160.25^{'k}$

$M_u^+ = 82.45 + 14 = 96.45^{'k}$

"Long Span"

$I_{slab} = \dfrac{(28' \times 12'')(10)^3}{12} = 28{,}000 \text{ in}^4$



$C = \Sigma[1 - .63(\frac{x}{y})]x^3 y(\frac{1}{3})$

$= [1 - .63(\frac{36}{18})](36^3)(18)(\frac{1}{3})$

$+ [1 - .63(\frac{8}{10})](8^3)(10)(\frac{1}{3})$

$C = -72783.36 + 846.5 = -71{,}936.86$

$\beta_t = \dfrac{E_{cb} C}{2 E_{cs} I_s} = \dfrac{71{,}936.86}{2(28000)} = +1.28 \qquad \ell_2/\ell_1 = 1.0 \qquad \alpha_{f1} = 0$

$\therefore$ Portion of Ext. ⊖ factored moments in CS. $= 1 - (1.28)(1 - .75)/2.5$

$= .872$

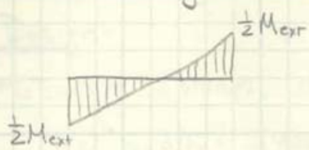
"Long Span"

Location	Strip	Total M	Total Width	M/ft of width
Int. Support ($560^{'k}$)	CS 75%	420	14	30
	MS 25%	140	14	10
Ext. Support ($240^{'k}$)	CS 87%	210	14	15
	MS 13%	31	14	2.2
Int Span ($400^{'k}$)	CS 60%	240	14	17.1
	MS 40%	160	14	11.4

Appendix C

Torsional Member Design

Torsion



$T = 210/2 = 105^{1k}$

$d = 18 - 1.5 - .5 - .375 = 15.625\,in$

$T_u @ d = 105\left(1 - \frac{15.625}{168}\right) = \underline{\underline{95.23^{1k}}}$

Factored Shear

$\frac{\alpha_{f1} l_2}{l_1} = 1.309 > 1.0 \therefore$ Trib area for shear =

$A_{trib} = (14)(28)/2 + (9''/12)(24')$

$= 214 ft^2$

Factored Shear from 2-way Slab

$V_u = (.325)[214/2 - (9'')(15'')/144] = 34.5^k$

to account for drop in shear @ column face

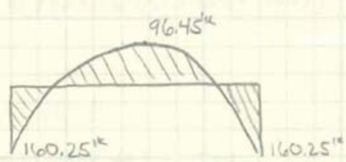
Factored Shear from factored weight of beam stem

$V_u = \frac{.36(26.5)}{2} = 4.77^k$ Total design Shear $V_u = 39.27^k$

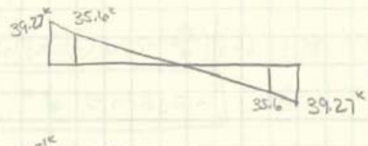
$V_u @ d = 39.27\,(1 - 15.625/168) = 35.62^k$

Summary of Factored Moments, Shear + Torsion

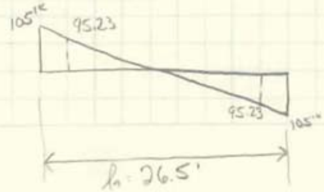
Flexure:



Shear:



Torsion



Appendix C

Torsional Member Design

Flexure Design:

Assuming #6 bars + #4 stirrups, $d = 18 - 1.5 - .5 - .375 = 15.625$ in

Min Reinf. $3(f'_c)^{1/2} b_w d/f_y = 3(4000)^{1/2}(36)(15.625)/60000 = 1.78 in^2$

$200 b_w d/f_y = 200(36)(15.625)/60{,}000 = 1.875 in^2$ ← c

At Interior Support

Assume $a = 2in$

$A_s = M_u/(0.9 f_y (d - a/2))$

$= (160.25 \times 12)/[(.9)(60)(15.625-1)]$

$A_s = 2.43 in^2$

Try (6)#6 $A_s = 2.64$

$a = (2.64)(60)/(.85)(4)(36) = 1.29$

$\phi M_n = .9(2.64)(60)(15.625 - 1.29/2)/12 = 178$ ft-k $> 160.25^{1k}$ ✓ok

Check $c/d = (1.29)/(.9 \times 15.625) = .092 < .375$ ✓ok

Spacing $= (36 - 5(1.5) - 6(.75))/2 = 14.5in > 1"$ ✓ok

$\boxed{\therefore \text{ use (6) \#6 @ support}}$ $A_s = 2.64 in^2$

At Midspan:

assume $a = .5in$

$A_s = ((96.45)(12))/[(.9)(60)(15.625 - .25)]$

$= 1.39 in^2$

Try (4)#6 $A_s = 1.76 in^2$

$a = (1.76)(60)/(.85)(4)(44) = .706$ $b_{eff} = 36 + 8 = 44$ in

$\phi M_n = .9(1.76)(60)(15.625 - \frac{.706}{2})/(12) = 108.86^{1k} > 96.45^{1k}$

$\boxed{\therefore \text{ use (4) \#6 @Midspan}}$ $A_s = 1.76 in^2$

Appendix C

Torsional Member Design

Torsion + Shear:

$A_{cp} = (18")(36") + (8)(10) = 728 \text{ in}^2$

$P_{cp} = 2(36+18+10) = 128 \text{ in}$

$(\phi f'_c)^{.5}(A_{cp}^2/P_{cp}) = .75(4000)^{.5}(728^2/128)//2000 = 16.4 < 105$ ∴ Torsion must be con

$\phi 4(f'_c)^{.5}(A_{cp}^2/P_{cp}) = 4(16.4) = 65.6 < 105$ ∴ reduction possible.

$T_u = 66'^k$

$y_1 = 18 - 2(1.5 + .25) = 14.5 \text{ in}$ ∴ $p_h = 2(y_1 + x_1) = 2(14.5 + 32.5) = 94 \text{ in}$

$x_1 = 36 - 2(1.5 + .25) = 32.5 \text{ in}$

$p_h/8 = 11.75 < 12"$ max spacing = 11.75 in (torsion)

$d/2 = 15.625/2 = 7.8 \text{ in} < 24$ max spacing for shear - 7.8 in

$S_{max} = 7 \text{ in}$

$V_c = 2(f'_c)^{.5} b_w d = 2(4000)^{.5}(36)(15.625)/1000 = 71.15^k$

$V_s = V_u/\phi - V_c = (39.27/.75) - 71.15 = -19^k$ ← neg. so no reinf. req'd for sh

$T_n = (66)/.75 = 88'^k$

$A_{oh} = x_1 y_1 = 14.5 \times 32.5 = 471.25 \text{ in}^2$

$A_o = .85 A_{oh} = 401 \text{ in}^2$

$\theta = 45°$ $\cot\theta = 1$

$A_t = T_n s/(2 A_o f_v \cot\theta) = (88)(12)(7)/\{2(401)(60)(1)\}$

$= .154 \text{ in}^2/\text{leg} \rightarrow 2A_t = .31 \text{ in}^2$

Try #4 closed stirrups $A_s = .4 \text{ in}^2 > .31 \text{ in}^2$ ✓ok

$A_{s\,min} = 50(36)(7)/60000\text{psi} = .21 \text{ in}^2 < .4$ ✓ok

$\boxed{\text{Use \#4 closed stirrups @ 7" spacing}}$

Appendix C

Torsional Member Design

Longitudinal torsional Reinforcement

$A_\ell = p_h \cot^2\theta A_t/s = (94)(1.0)(.154)/7 = 2.1 in^2$

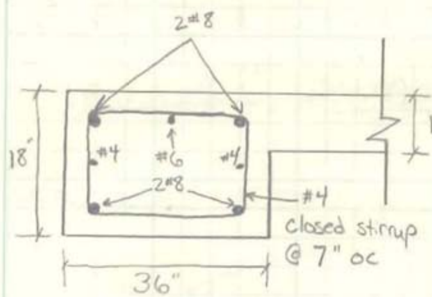
$A_{\ell min} = 5(f'_c)^{.5} A_{cp}/f_y - (A_t/s)p_h =$

$= (5)(4000)^{.5}(728)/60000 - (.154/7)(94)$

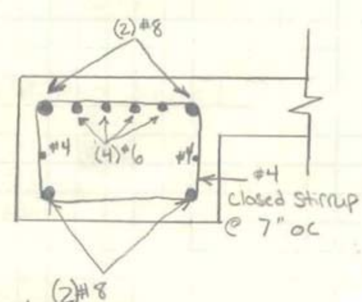
$A_{\ell min} = 1.768 in^2 < 2.1 in^2$

$A_\ell = 2.1 in^2$ $[1/4 A_\ell = .525 in^2]$ to top corners + flexural
to bottom corners + flexural

$A_{corner} = .44 + .2625 = .7025 in^2$ must bump reinf @ corners to #8's



At Mid span



At Support

Increased Size to a 38" wide beam because of exterior column Size increase to 20"

∴ 18" over hang + 20" column = 38 in ∴

Appendix D

Footing Design

Existing: Typical 7.5'×7.5'
Largest 10'×10'
Smallest 5'×5'

$q_{allow} = 6000psf$ ← soil bearing Capacity

$q_a = \frac{P}{A}$

$$\boxed{B_{req} = \sqrt{\frac{P^k}{6^{ksf}}}}$$

3 types of loading situations (may want to design all ftgs)

1: $P = 615^k$ (2^{nd} row from Ext)
2: $P = 770^k$ (Interior Core)
3: $P = 406^k$ (Span A-(2→5))
} Unfactored for use of $q_a = \frac{P}{A}$

$A_T = 406\ ft^2$ $D_L = 146\ psf$ (134 psf)

$L_r = 100(.25 + \frac{15}{\sqrt{1624}}) = 62.2\ psf$

$L_e = 50(.25 + \frac{15}{\sqrt{1624}}) = 31.1\ psf$

$67.1^k + 84.6^k + 84.6^k + 84.6^k + 84.6^k = 405.5^k \rightarrow 406^k$
(83.5) · 111.5 $P_u = 532^k$

Footing #1

$B = \sqrt{\frac{615^k}{6}} = 10.12' \rightarrow$ [10'-6"×10'-6"]

Footing #2:

$B = \sqrt{\frac{770}{6}} = 11.34' \rightarrow$ [11'-6"×11'-6"]

Footing #3:

$B = \sqrt{\frac{406}{6}} = 8.23' \rightarrow$ [8'-6"×8'-6"]

Appendix D

Footing Design

Footing #1

$P = 615^k$ $10\text{-}6'' \times 10\text{-}6''$

$q_a = 6\ \text{ksf}$

$P_u = 1.2D + 1.6L$

$P_u = 803^k$ $q = \frac{803}{10.5^2} = 7.3\ \text{ksf} = 50.6\ \text{psi}$

$V_c = \phi 4 \sqrt{f'_c}$

$= .75(4)\sqrt{6000} = V_c = 2324\ \text{psi}$

*2-way shear stress controls by Inspection

$$d^2\left(V_c + \frac{q}{4}\right) + d\left(V_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$

$$d^2\left(190 + \frac{50.6}{4}\right) + d\left(190 + \frac{50.6}{2}\right)(20) = \frac{50.6}{4}\left(126^2 - 20^2\right)$$

$$202.6\, d^2 + 4306\, d = 195771.4$$

$$d = 22.2'' \rightarrow h = d + 3'' + d_b = 22.2 + 3 + 1''$$

$$h = 27''$$

$d = 27 - 1'' - 3''$
$= 23''$

$$l = \frac{10.5' - 1.6}{2} = 4.5' \quad M_u = \frac{q l^2}{2} = \frac{7.3\ \text{ksf}\ (4.5)^2}{2} = 74'^k$$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{A_s (60)}{.85(4)(27)} = .63 A_s$$

$$M_u = \phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$74'^k(12) = .9 A_s (60)\left(23 - \frac{.63 A_s}{2}\right)$$

$$16.44 = 23 A_s - .315 A_s^2$$

$$A_s = .72\ \text{in}^2$$

Use #8 @ 12" oc $A_s = .79\ \text{in}^2$

Appendix D

Footing Design

footing #1 cont.

$$\rho = \frac{A_s}{bh} = \frac{.79}{27(12)} = .0024 \geq .0018$$

$$a = .63 A_s = .63(.79) = .5 \text{ in}$$

$$c = a/.85 = .586$$

$$\varepsilon_s = \frac{.003}{c}(d-c) = \frac{.003}{.5}(23-.5)$$

$$\varepsilon_s = .135 \geq .005 \therefore \phi = .9$$

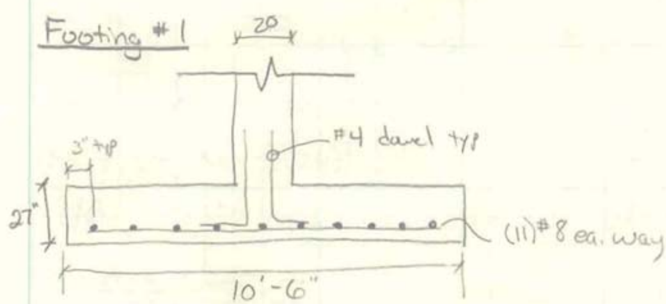
use (11) #8 each way

$$\phi B_n = \phi\, .85 f'_c A_1$$

$$= .65(.85)(4)(20)^2$$

$$= 884^k \geq P_u = 803^k$$

Footing #1



Appendix D

Footing Design

Footing #2 *Same Procedure as before

$P = 770^k$ 11'-6" x 11'-6"
$q_a = 6$ ksf

$P_u = 1.2D + 1.6L$

$P_u = 1003$ $q = 7.58$ ksf $= 52.7$ psi → use 24" x 24" column

$V_c = 190$ psi

$203.18\,d^2 + 5192.4\,d = 243316$

$l = \frac{11.5 - 2}{2} = 4.75$ $d = 24.11 \rightarrow$ say $d = 24"$

$h = 28"$

$M_u = \frac{7.58(4.75)^2}{2} = 86^{1k}$ $a = .63A_s$

$19.11 = 24A_s - .315A_s^2$

$A_s = .805\ in^2/ft$ use #8 @ 10" oc $A_s = .948\ in^2/ft$

$\rho = .0028 \geq .0018$ ✓

$a = .597$
$c = .7026$

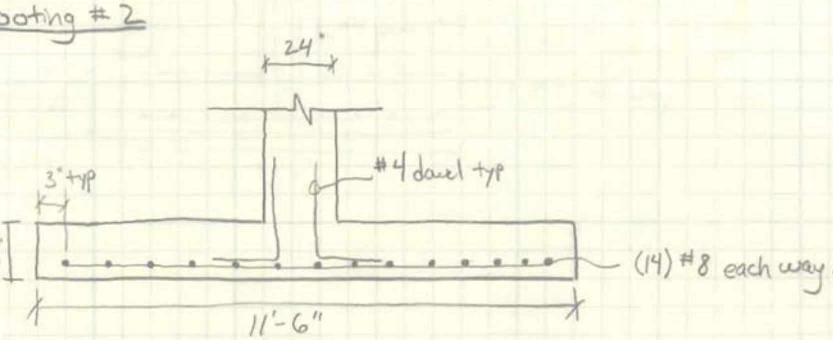
$\varepsilon_s = \frac{.003}{.7026}(24 - .7026)$
$= .0994 \geq .005 \therefore \phi = .9$ ✓

use (14) #8 ea. way

$\phi B_n = (65)(.85)(4)(24)^2$

$\phi B_n = 1273^k > 1003^k$ ✓ ok w/ 24" x 24" column @ ground

Footing #2



Appendix D

Footing Design

Footing #3

$P = 406\ ft^2$

$q_a = 6ksf \quad 8'\text{-}6" \times 8'\text{-}6"$

$P_u = 532^k \quad q = 7.36\ ksf = 51.13\ psi$ 18"x18" column

$V_c = 190 psi \quad 202.8d^2 + 3880.2d = 128848$

$d = 17.4 \quad h = 21.025$ say $22" = h$ assume #8

$l = \frac{8.5 - 1.5}{2} = 3.5$

$d = 22 - 3 - 1 \quad d = 18$

$M_u = \frac{(3.5)^2\, 7.36}{2} = 45.1^{1k} \quad a = .802 A_s$

$11.36 = 18A_s - .401A_s^2$

$A_s = .64$ use #8 @ 12" oc $A_s = .79 in^2$

$\rho = .0029 \geq .0018$ ✓

$a = .802(.79) = .633$

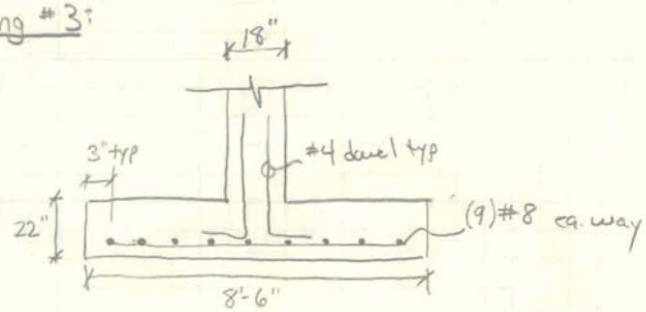
$c = .745$

$\varepsilon_s = \frac{.003}{.745}(18 - .745) = .069 \geq .005 \therefore \phi = .9$

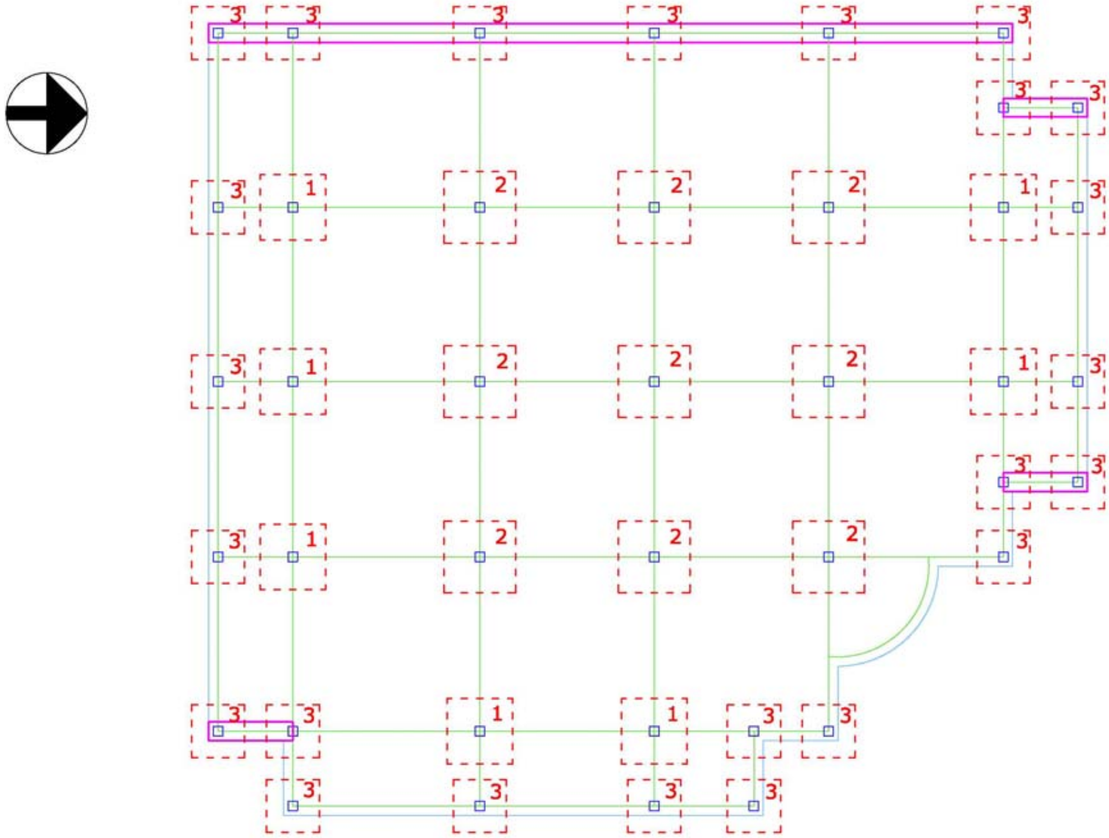
Use (9) #8 each way

$\phi B_n = 716^k > 532^k$ ✓ok w/ 18"x18" column

Footing #3:



Appendix D



Appendix E

Biaxially Loaded Columns

Find P_n → Loads, Trib Area

Find M_{nx}, M_{ny} } ADOSS, 2 frames for data

* Check Critical Moments w/ RAM ADVANSE

Checked w/ ADOSS

Use Interaction Diagrams

Worst Case Column: Span 3 col. 5 $M_L = -771.4^{1k}$, $M_R = 821^{1k}$ } Worst Mome

Typ. Center Columns { Span B col. 3 $M_L = -714.4^{1k}$, $M_R = 686^{1k}$ } Worst Mome

Typ. Center Columns { Span 3 col. 2 $M_L = -742.4^{1k}$, $M_R = 724^{1k}$

Span E col. 3 $M_L = -385.3^{1k}$, $M_R = 396^{1k}$

① { $M_x = 10.4^{1k}$ (Span E col 3), $M_y = 49.4^{1k}$ (span 3 col 5) }

② { $M_x = 28.4^{1k}$ (span B col 3), $M_y = 18.4^{1k}$ (span 3 col 2) } Typ Column (Most Axial load)

Axial Loads

	4th ↑	3rd ↑	2nd ↑	1st ↑	Basement ↑
①	130k	298k	466k	634k	803k
②	164k	373k	583k	793k	1003k

L_{roof}: 50 psf (Snow + L_r

Conc. 132 lbs/sf } ⟹ ADOSS

Reinf 1.97 lbs/sf } ⟹ ADOSS

DL: 12 psf

LL: 100 psf*

$L_R = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$

$A_I = K_{LL} A_T$

$K_{LL} = 4$ → column

$A_{T_1} = 638\ ft^2$ *$L_1 = 54.7$ psf $L_r = 27.35$ psf

$A_{T_2} = 812\ ft^2$ *$L_2 = 51.3$ psf $L_r = 25.65$ psf

Appendix E

Biaxially Loaded Columns

Column 1:

$K_n = \frac{P}{\phi A_g f'_c}$ $R_n = \frac{M_u (12")}{\phi A_g h f'_c}$ $f'_c = 4\,ksi$ $\gamma = \frac{h - 2cover}{h}$ $\phi = .65$

4th Floor: Try 16"×16" column

$\gamma = .7$ $\rho_g = .01$ $A_s = .01(256in^2) = 2.56in^2$

$K_n = \frac{111}{(.65)(256in^2)(4)} = .167$ use (6) #6's $A_s = 2.64in^2$

$R_n = \frac{60"(12")}{(.65)(256)(16)(4)} = .068$

Load Contour Method

$\phi M_{nxo} = \phi M_{nyo} = (.068)(.65)(4)(16)^3 = 724$ in-k

$$\left(\frac{\phi M_{nx}}{\phi M_{nxo}}\right)^{1.15} + \left(\frac{\phi M_{ny}}{\phi M_{nyo}}\right)^{1.15} \le 1.0$$

$$\left(\frac{10.4^{'k}(12)}{724^{"k}}\right)^{1.15} + \left(\frac{49.4^{'k}(12)}{724^{"k}}\right)^{1.15} \le 1.0$$

$.132 + .794 = .926 \le 1.0$ ✓ok for moment

* Assume moment differences do NOT change significantly as the floors change. Otherwise this case is for 3rd floor.

Axial Loading:

$P_o = .85 f'_c (bh - A_s) + f_y A_s$ $-.85 f'_c A_s + f_y A_s = P_o - .85 f'_c bh$

$A_s \ge \frac{P_o - .85 f'_c bh}{-.85 f'_c + f_y}$ $\frac{(-.85 f'_c + f_y)}{P_o - .85 f'_c bh} = \frac{1}{A_s}$

$P_o = .85 f'_c bh - .85 f'_c A_s + f_y A_s$

Try 16"×16" w/ (6) #6's $A_s = 2.64in^2$ $P_o - .85 f'_c bh = -.85 f'_c A_s + f_y A_s$

$P = .85(4)(16^2 - 2.64) + (60 \times 2.4)$ $\frac{-.85 f'_c + f_y}{P_o - .85 f'_c bh}$

$P = 1020$ Kips $> 1000^k$ ✓

$\boxed{\text{16"×16" w/(6) #6's is adequate for all columns!}}$

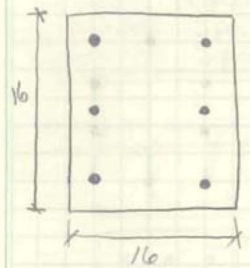
Pure Axial Load, check Balanced P+M

Appendix E

Biaxially Loaded Columns

$P_b = .85 f'_c b \beta_1 c + A_{s1} f_{s1} + A_{s2} f_{s2} + A_s f_{s3}$

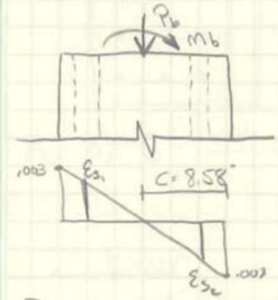
$M_b = .85 f'_c b \beta_1 c \,(h/2 - a/2) + \sum (A_{s1} f_{s1} (h/2 - d_i))$



(6) #6's $\quad A_s = 2.64 in^2$

$C = \frac{.003}{.003+\varepsilon_t}(d_t)$

$C = \frac{.003}{.003+.00172}(13.5) = C = 8.58 in$



$\varepsilon_{s1} = \frac{.003}{8.58}(8.58 - 2.5) = .00212$

$f_{s1} = .00212(29{,}000) \cdot 60 \text{ ksi} \rightarrow$ yeilds

$\varepsilon_{s2} = \frac{.003}{8.58}(8.58 - 13.5) = -.00172$

$f_{s2} = -.00172(29000) = -49.89$ ksi

$P_b = .85(4)(16)(.85)(8.58) + 1.32(60)(3) - 3(1.32)(+49.89)$

$P_b = 436.77$ kips $\rightarrow$ MUST Increase Column Design

* Good for 3rd + 4th floor

Try 20"x20" w/ (10) #8

$C = 11.12$

$\varepsilon_{s1} = \frac{.003}{11.12}(11.12 - 2.5) = .0023$

$f_{s1} = 29000(.0023) : 60 \text{ ksi} \rightarrow$ yeilds

$\varepsilon_{s2} = \frac{.003}{11.12}(11.12 - 10) = .000302$

$f_{s2} = 29000(.000302) = 8.758$

$\varepsilon_{s3} = \frac{.003}{11.12}(11.12 - 17.5) = -.00172$

$f_{s3} = 29000(-.00172) = -49.92$

$P_b = .85(4)(20)(.85)(11.12) + 3.16(4)(6$
$+(2)(8.758)(1.58) - 3.16(4)(+49.92)$

$= 797.8$ kips

Bigger Still!

* Good for 1st + 2nd Floors

(↑ invalid since M_u is little)

* Use Interaction diagrams In Book for design

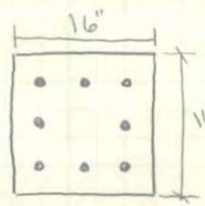
Appendix E

Column Design Summary

Column Values used for design of columns : Col 3-B
Col 3-E

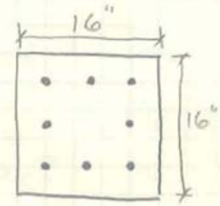
All exterior Columns	Span 3, B-D
Columns Span 2	Span 4, B-D
Columns Span E	Span 5, B-D
Columns Span 6	

4th, 3rd, 2nd Floors



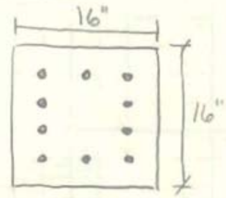
$A_s = (8)\ \#6 = 3.52 in^2$

4th, 3rd, 2nd Floors



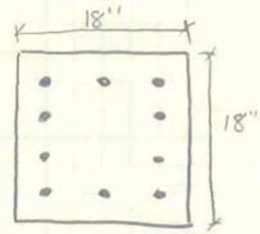
$A_s = (8)\ \#6 = 3.52 in^2$

1st Floor



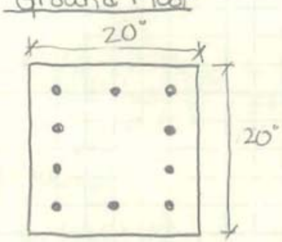
$A_s = (10)\ \#8 = 7.90 in^2$

1st Floor



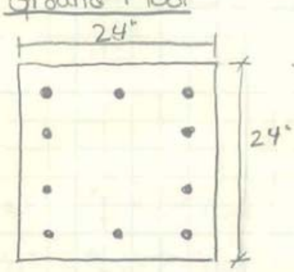
$A_s = (10)\ \#8 = 7.90 in^2$

Ground Floor



$A_s = (10)\ \#8 - 7.90 in^2$

Ground Floor



$A_s = (10)\ \#8 = 7.90 in^2$

* Ground floor column dimensions based on allowable bearing capacity

Appendix E

Column Loads	in Kips						UNFACTORED				
	4th	3rd	2nd	1st	ground						
1	130	298	466	634	803		103	231	359	487	615
2	164	373	583	793	1003		130	290	450	610	770

Load Contour	Method									
	4th		3rd		2nd		1st		ground	
	Mnx	Mny	Mnx	Mny	Mnx	Mny	Mnx	Mny	Mnx	Mny
1	124.8	592.8	124.8	592.8	124.8	592.8	124.8	592.8	124.8	592.8
2	340.8	220.8	340.8	220.8	340.8	220.8	340.8	220.8	340.8	220.8
	Rn	Kn	Rn	Kn	Rn	Kn	Rn	Kn	Rn	Kn
1	0.0450721	0.130695	0.045072	0.298556	0.045072	0.466417	0.045072	0.63528	0.035613	0.633417
2	0.0353065	0.164127	0.035306	0.373344	0.035306	0.583563	0.027896	0.627185	0.022596	0.643201

Rho	4th	3rd	2nd	1st	ground
1	0.01	0.01	0.01	0.01	0.01
2	0.01	0.01	0.01	0.01	0.01

As Req'd	4th	3rd	2nd	1st	ground
1	2.56	2.56	2.56	2.56	2.56
2	2.56	2.56	2.56	2.56	2.56

As Provided	4th	3rd	2nd	1st	ground
1	(8) #6	(8) #6	(8) #6	(8) #6	(8) #6
2	(8) #6	(8) #6	(8) #8	(8) #10	(8) #12

Appendix F

CM Breadth — Material Quantities

SPAN 1

Conc. 28.3 cy
Rebar 2338+1005 lbs
fmwk 898 SF

SPAN 2

Conc. 84.6 cy
Rebar 3203+2030 lbs
Fmwk 2664 SF

SPAN 3

Conc. 119.4 cy
Rebar 3996+2938 lbs
Fmwk 3750

SPAN 4

Conc. 112.7 cy
Rebar 4056+2794 lbs
Fmwk 3541. SF

SPAN 5

Conc. 106.9 cy
Rebar 3346+2263 lbs
Fmwk 3019 SF

SPAN 6

Conc. 52.7 cy
Rebar 1758+1155 lbs
Fmwk 1665 SF

SPAN 7

Conc. 15.7 cy
Rebar 1321+408 lbs
Fmwk 499 SF

Totals

Concrete: 520.2 cy
Rebar: 32612 lbs
Fmwk: 16,036 SF

SPAN A

Conc. 65 cy
Rebar 2874+1656 lbs
Fmwk 2043 SF

SPAN B

Conc. 126 cy
Rebar 4145+2643 lbs
Fmwk 3948 SF

SPAN C

Conc. 126 cy
Rebar 4145+2643 lbs
Fmwk 3948 SF

SPAN D

Conc. 117.1 cy
Rebar 4403+3158 lbs
FMWK 3679 SF

SPAN E

Conc. 66.5 cy
Rebar 3205+1935 lbs
Fmwk 2071 sf

SPAN F

Conc. 19.7 cy
Rebar 1794+672 lbs
Fmwk 600 SF

Totals

Concrete: 520.3 cy
Rebar: 33,279 lbs
Fmwk: 16,289 SF

* Use most Critical Case * *

** Totals are from ADOSS output for Third Floor Only **

*** Half value used for Roof ***

Appendix F

CM Breadth

Footing Materials

Footing #1

Conc. 9.2 cy
rebar 587.4 lbs
Fmwk 94.5 SF

Footing #2

Conc. 11.4 cy
Rebar 822.4 lbs
Fmwk 107.3 SF

Footing #3

Conc. 4.9 cy
Rebar 384.5 lbs
Fmwk 62.3 SF

Total : All footings

Conc : 284.6 cy
Rebar : 20,741.4 lbs
Fmwk : 3122.4 SF

Totals: 7 footings

Conc 64.4 cy
Rebar 4111.8 lbs
Fmwk 661.5 SF

Totals: 9 footings

Conc 102.6 cy
Rebar 7401.6 lbs
Fmwk 965.7 SF

Totals: 24 footings

Conc 117.6 cy
Rebar 9228 lbs
Fmwk 1495.2 SF

Appendix F

CM Breadth — Column Materials / Beam Materials

Floor	Height	Size	Concrete	Rebar	Formwork
4th	13.33'	16x16	.877 cy	160.17 lbs	71 SF
3rd	13.33'	16x16	.877 cy	160.17 lbs	71 SF
2nd	13.33'	16x16	.877 cy	160.17 lbs	71 SF
1st	15.33'	16x16	1 cy	409.3 lbs	81.75 SF
Ground	13.33'	20x20	1.37 cy	356. lbs	89 SF
Totals for Column #1 : (most ext. columns)			5 cy × 31 = 155 cy	1245.8 lbs × 31 = 38,619.8 lbs	383.75 SF × 31 = 11,896.25 SF

Floor	Height	Size	Concrete	Rebar	Formwork
4th	13.33'	16x16	.877 cy	160.17 lbs	71 SF
3rd	13.33'	16x16	.877 cy	160.17 lbs	71 SF
2nd	13.33'	16x16	.877 cy	160.17 lbs	71 SF
1st	15.33'	18x18	1.27 cy	409.3 lbs	92 SF
Ground	13.33'	24x24	1.97 cy	356 lbs	106.7 SF
Totals for Column #2 : (Interior Columns)			5.87 cy × 9 = 52.8 cy	1248.8 lbs × 9 = 11,239.2 lbs	411.7 SF × 9 = 3705.3 SF

Edge Beams

Size	length	Concrete	Rebar	Formwork
36"x18"	174 ft	29 cy × 4	2587.3 lbs × 4	1566 SF × 4
	Totals:	116 cy	10,349.2 lbs	6264 SF

Appendix F

Materials Cost Estimate

MATERIALS

Item	Unit	Quantity	Price per unit	Total Cost
Concrete	cy			
Slab		2,341.35	264.15	$618,467.60
Footing		284.60	242.26	$68,947.20
Column		207.80	262.25	$54,495.55
Beam		116.00	337	$39,092.00
TOTAL		**2,949.75**		**$781,002.35**
Formwork	sf			
Slab		73,300.50	1.56	$114,348.78
Footing		3,122.40	0.84	$2,622.82
Column		15,601.60	1.64	$25,586.62
Beam		6,264.00	1.27	$7,955.28
TOTAL		**98,288.50**		**$150,513.50**
Rebar	lbs			
Slab		166,395.00	0.47	$78,205.65
Footing		20,741.40	0.44	$9,126.22
Column		49,859.00	0.44	$21,937.96
Beam		10,349.20	0.44	$4,553.65
TOTAL		**247,344.60**		**$113,823.47**

MATERIALS COST

TOTAL: $1,045,339.32

Appendix F

Labor Cost Estimate

LABOR COST
TOTAL: $1,478,348.96

LABOR

Task	Crew	Labor hrs. per unit	Price per unit	Price per Labor hr.	Quantity	Total Cost
Concrete						
Slab	C14-B	4.079	$265.85	$63.12	2,341.35	$602,819.14
Footing	C14-C	1.382	$102.40	$56.42	284.60	$22,190.96
Column	C14-A	10.46	$642.50	$63.12	207.80	$137,196.87
Beam	C14-A	10.782	$663.00	$63.12	116.00	$78,944.94
TOTAL						**$841,151.92**
Formwork						
Slab	C-1	0.088	$5.19	$57.20	73,300.50	$368,965.40
Footing	C-2	0.084	$4.70	$55.53	3,122.40	$14,564.50
Column	C-1	0.076	$4.41	$57.20	15,601.60	$67,823.28
Beam	C-1	0.081	$4.93	$57.20	6,264.00	$29,022.36
TOTAL						**$480,375.53**
Rebar						
Slab	4 Rodm	0.006	$0.41	$62.80	166,395.00	$62,697.64
Footing	4 Rodm	0.004	$0.35	$62.80	20,741.40	$5,210.24
Column	4 Rodm	0.009	$0.77	$62.80	49,859.00	$28,180.31
Beam	4 Rodm	0.095	$0.73	$62.80	10,349.20	$61,743.33
TOTAL						**$157,831.51**

Appendix F

Structural Schedule

Task	Crew	Daily Output	Quantity	Total Duration
Concrete				
Slab	C14-B	50.99	2,341.35	45.92
Footing	C14-C	81.04	284.60	3.51
Column	C14-A	19.82	207.80	10.48
Beam	C14-A	18.55	116.00	6.25
TOTAL				**66.17**
Formwork				
Slab	C-1	544	73,300.50	134.74
Footing	C-2	485	3,122.40	6.44
Column	C-1	420	15,601.60	37.15
Beam	C-1	440	6,264.00	14.24
TOTAL				**192.56**
Rebar				
Slab	4 Rodm	5800	166,395.00	28.69
Footing	4 Rodm	7200	20,741.40	2.88
Column	4 Rodm	4600	49,859.00	10.84
Beam	4 Rodm	5400	10,349.20	1.92
TOTAL				**44.32**

STRUCTURAL SCHEDULE

TOTAL: 303 Days

Appendix G

Lighting Breadth

Research Room - First Floor

32' x 38' x 15'-4" high

work plane : 30" high

Materials:

Ceiling - Wood Works Tegular acoustical Panel .82 reflectance
Walls - Painted assume .5 reflectance
Floor - Carpet assume .20 reflectance

luminaires:

2x4 (64 W) recessed (2) F32/T8 curved acrylic lense

Ledalite Architectural Products Pure/Fx 9424D1

35' x 39' x 10'-8"
Room

Window
2.8' x 2.4'

FIXTURE SPEC

BALLAST SPEC

LAMP SPEC

use desi Lumens

$$RCR = \frac{5h(w+l)}{(w \times l)} = 5.82$$

RSDD = .97 *
LLD = 0.8 *
LDD = .88

Existing

Category V
assume Clean Cond. based on 12 month cleaning
LDD = .88

if direct/indirect Cat II
LDD .95

if Para Cat IV
LDD .85

Aim 50 fc on Desks

if Indirect Cat VI
LDD .85

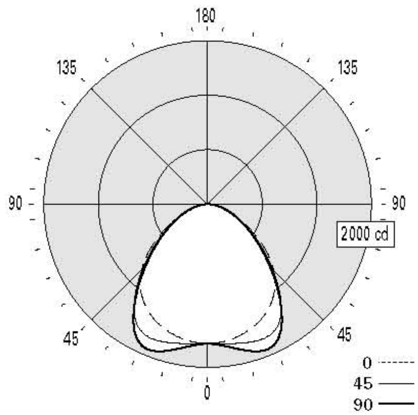
parabolic troffer try indirect

Appendix G

9424D1	2 Lamp	T8

Photometry

2 T8 32W



CANDELA DISTRIBUTION

Vert. Angle	Horizontal Angle 0	22.5	45	67.5	90	Zonal Lumens
0	1761	1761	1761	1761	1761	
5	1752	1757	1771	1786	1790	173
15	1693	1730	1815	1893	1920	513
25	1587	1663	1820	1906	1943	819
35	1421	1488	1577	1534	1552	944
45	1146	1124	1104	1022	1049	839
55	760	703	689	646	669	617
65	392	380	387	374	391	382
75	136	146	180	182	194	182
85	19	25	45	39	45	43
90	0	0	0	0	0	

Report #	2101751
Efficiency	76.4%
Spacing Criteria	1.3 @ 0° along
	1.4 @ 90° across

COEFFICIENTS OF UTILIZATION (%)

Ceiling	80				70			50			0
Wall	70	50	30	10	70	50	30	50	30	10	0
RCR											
0	91	91	91	91	89	89	89	85	85	85	76
1	84	81	78	75	82	79	76	76	74	72	66
2	77	71	67	63	75	70	66	67	64	61	56
3	71	64	58	53	69	62	57	60	56	52	49
4	65	57	51	46	64	56	50	54	49	45	42
5	60	51	45	40	59	50	44	49	43	39	37
6	56	46	40	35	54	46	39	44	39	35	33
7	52	42	36	31	51	41	35	40	35	31	29
8	48	39	32	28	47	38	32	37	32	28	26
9	45	35	29	25	44	35	29	34	29	25	24
10	42	33	27	23	41	32	27	32	26	23	21

Based on a floor reflectance of 0.2

AVERAGE LUMINANCE (cd/m^2)

Vert. angle	Horizontal angle 0	45	90
55	1894	1717	1667
65	1326	1309	1323
75	751	994	1072
85	312	738	738