MQP No.______________

Re-Design and Method Comparison of *Institute Hall, Fall 1994.*

submitted to the Faculty

of the

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

by

Ricardo Fabrega

Pranas Lukauskis

George Pissimissis

Date: December, 1994

Approved:

____

Prof. Leonard Albano

Statement of Authorship

All three members of this Major Qualifying Project (MQP) participated in the development of the abstract, introduction, background, methodology, conclusions, bibliography, and appendices sections of this project report.

Individually, they participated as follows to make this project a reality:

Ricardo Fabrega: Developed the Cost Estimating chapter for both the Steel and Reinforced Concrete Designs.

Pranas Lukauskis: Developed the Gravity and Lateral Loads chapter and the Steel Design chapter.

George Pissimissis: Developed the Reinforced Concrete Design chapter and the Fire-protection chapter.

Acknowledgements

Every attempt has been made to give credit to quoted sources. We accept responsibility for any errors or omissions.

Our thanks to: Prof. Leonard Albano, our project advisor, for his assistance in giving us recommendations to improve our project. Prof. P. Jayachandran, Prof. R. Fitzgerald, and Prof. T. El-Korchi, who granted us with their respective knowledge in the frame computer analysis, fire-protection, concrete and building assembly areas of Civil Engineering.

Abstract

This MQP focuses on providing an understanding of the building design process. It shows a redesign of Institute Hall's structure using two different types of construction materials: Steel and Reinforced Concrete. Each structural design is accompanied with its respective construction cost estimate and schedule. Fire protection and heating analysis issues are also considered. These factors are the basis for comparison between the two designs to determine which is the most adequate for future projects of similar type around the Worcester area.

Table of Contents

CHAPTER 1

Introduction

1. *Introduction*

It takes four years, on average, to receive a bachelors degree in **Civil Engineering**. If we neglect the new trend of classifying **Environmental Engineering** as Civil Engineering, and put the **Urban Development Majors** in their own category; Civil Engineering consists of one thing. **Constructing !**

Constructing, or construction is a multifaceted word ; it encompasses a multitude of actions. Construction itself has many meanings, some of these meanings are, to erect, to form, to build, and to devise. Just with these meanings we can see that a lot goes into construction, or rather that it is based on many disciplines.

What are the disciplines needed to construct something, let us say a building? For one we must have a **manager**, someone who oversees the process, or it will turn chaotic. We have to know how much it will cost, so the manager must also be able to estimate the cost, as well as the time span of the construction.

A building will need **designs**, it must be based on something before construction begins. So architects and engineers must design it. They may decide to build it from **Steel**, or maybe from **Concrete**. Each has its strengths, and weaknesses, as well as its construction and life-cycle costs. Therefore the designers must have a firm understanding of the building materials to be used.

They must also know how to use the State Codes pertaining to their building, and how to utilize these codes in his/her design.

What about protecting the building from fire? Yes, there must be someone disciplined in this form of engineering known as **Fire Protection**. Someone has to be able to design the structure for its resistance to fire, so that it may be safe for human use. If it is not safe, the same engineer must know how to make it so, either by treatment, or with devices (i.e. sprinklers, fire alarms)

The building must also be built upon ground, and the **foundations** must be sturdy and stable enough to keep it from collapsing or toppling over. Not only must the Engineer know how to design in order to build foundations, but know how to analyze the soil as well. Therefore the design must also involve geo-technical engineers to get the foundations right.

While this may seem a strange introduction to an MQP, we could think of no better way to start our project off. This project is not only about a re-design of Institute Hall, it is about the entire process necessary for the construction of a building.

In our project we are taking **Institute Hall** and designing it twice. Once out of **Steel**, and once out of **Reinforced Concrete**. We are then analyzing the **fire protection** for each building, the **heating** necessary for this particular type of building, and doing a **cost analysis** of each design. In conclusion we intend to compare the two structures and state which would be the better choice.

The summary that was just given is the mechanics of the project, but we wish to show more than mechanics. Our goal is also educational, to show all of the knowledge that a civil engineer must have; as well as to give a summary to beginning students of all that will be learned in their future classes.

CHAPTER 2

Background

2. Background

2.1. *History of Institute Hall*

Institute Hall is the newest of all the dormitory buildings at the Worcester Polytechnic Institute (WPI). It is located on the corners of Institute Road and Boynton Street in Worcester, Massachusetts. Its original name was the Princess Apartments, and it served as a residential apartment building until it was purchased by the Lambda Chi Alpha Fraternity. This fraternity chapter would later sell the lease of the property to WPI to ease financial troubles. However, in 1989 the fraternity lost its national recognition and vacated the property.

In 1990, the WPI freshman class of 1990 was projected at 700 students, and the school's campus was not equipped to handle such a student population. The reason for the problem was that the Institute guaranteed housing accommodation for all freshmen, and even with the addition of Founders Hall in 1985, the Institute was still about 100 beds short of the amount required. Therefore, after some review, and balancing of alternatives, Institute Hall became the newest dormitory at the expense of less needed office space. The name, as it reflects, is a symbol of the Institute's new acquisition.

The new building's renovation was quoted approximately at the expense of $1,000,000.00, and opened its doors in the fall of 1989. The resident population of the dormitory is 70 students, accommodated in either double or triple bedrooms. Also, the building provides laundry machinery, leisure, studying, and recreational facilities, housed in its basement. One of the latest events in this young building's history, is the planting of a memorial tree on October 29, 1990. The memorial was dedicated to Andrew Heitman "93, who unfortunately died days before he was scheduled to move in to the new dormitory. We would like to dedicate this project to his memory.[1]

2.2. Institute Hall Information Gathering

We began gathering Information on Institute Hall from the Plant Services department. In the basement of Plant Services are housed all the designs, and documents pertaining to WPI's buildings. It is here that all of the blue-prints, and specifications for construction were found.

Several days were spent copying designs and taking down information. All of our design dimensions were taken from the blue-prints and we received CAD prints of Institute's floor plans from the Plant Service secretary. The CAD drawings are located in Appendix A.

Even though some information was received this way, much of it depended on going to Institute Hall and looking at its actual makeup. We had planned on designing the building again using the original architectural layouts. We needed the buildings basic dimensions and space distribution to proceed with the steel frame and reinforced concrete frame designs.

---

[1] Institute Hall's history was compiled using an IQP project, WPI Revisited: The Growth of the WPI Campus, done in the fall semester of 1990 by Michael Schorr & Christopher Riley.

We went to Institute Hall several times, took pictures of it as well as of other construction sites around Worcester. Some of these pictures are contained in this project report. The purpose of these visits was to see how the students lived, and the ways in which the building's components were being used. This would be important to us later on, in the design of the rooms, and recreation areas; as well as the stairs, and elevator.

Once we knew all the space distribution requirements we proceeded to develop the steel and reinforced concrete structural designs. Each design is accompanied with its respective construction cost estimate and schedule. Fire protection and heating analysis issues are also considered. These factors are the basis for comparison between the two designs to determine which is the most adequate for future projects of similar type around the Worcester area.

CHAPTER 3

Methodology

3. Methodology

The purpose of this chapter is to give the reader a general overview of the steps taken to develop the various parts of the project such as the steel and reinforced concrete designs, the fire -protection issues, and the cost estimating.

3.1 Steel Design

The first step taken in the steel design was to obtain the gravity and lateral loads acting on the building, presented in chapter 4, in accordance with the Massachusetts Building Code 1990 and the Canadian NBC Code 1977. We decided to compare both codes and use the one with the most conservative values for our calculations. Then we made a thorough review of the space distribution along the four stories of the building. The purpose of the review was to develop a structural layout which coincides with the architectural drawings of the building, avoiding problems like placing columns in the middle of entrances or beams running across stairways.

Knowing where columns and beams could be placed, the next step was to create a structural layout for the building. This layout was to include the columns, girders, beams, and steel deck plates present in the building.

As far as the design went, it was performed in accordance with the top to bottom sequence of loading or, in other words, load transmission from roof to foundation. The roof and floor decks were designed to be composite concrete and steel decking, in accordance with the United Steel Deck, Inc. (from 1992

SWEET's Catalog, file 2). Once the roof and floors were designed to carry their particular live and dead loads, the design of the structural members such as beams, girders, and columns could proceed since all the uniform dead and live loads acting on them were known.

The design of these members was done by using John Smith's Structural Steel Design: LRFD Approach ,1991 edition textbook , in conjunction with AISC's LRFD [1] manual of steel construction. The LRFD complies with the provisions of the Massachusetts Building Code 1990.

All structural members were designed to be as wide-flange sections since it is the most common shape used in structural steel design as a *beam* (bending member), a *column* (axial compression member), and a *beam-column* (bending plus axial compression member). [2] They were designed for gravity loads by using the tributary area method, checking for local buckling, shear, and deflections.

The girders and columns designed to make up the buildings main frames were then analyzed for wind and seismic loading conditions under the computer Program *FRAME: Stiffness Analysis of Plane Frames*.© 1988, by John F.Fleming.

Lastly, the basement slab and was designed according to the WPI General Specifications on Institute Hall and in accordance with the

[1] AISC stands for American Institute of Steel Construction and LRFD stands for Load and Resistance Factor Design.
[2] Smith, John. Structural Steel Design: LRFD Approach, 1991 edition, pgs.2-8.

Massachusetts Building Code 1990. The foundations for the building were developed with the aid of Peck, Hanson, and Thornburn's Foundation Engineering, 1974 edition textbook and in accordance with the Mass Building Code 1990.

3.2 Reinforced Concrete Design

The first step taken in the reinforced concrete design was to determine the gravity and lateral loads acting on the building, presented in chapter 4, in accordance with the Massachusetts Building Code 1990 and the Canadian NBC Code 1977. Then we had to make a thorough review of the space distribution along the four stories of the building. The purpose of the review was to develop a structural layout that coincides with the architectural drawings of the building, avoiding problems like placing columns in the middle of entrances or beams running across stairways.

Knowing where columns, and slabs would be placed, the next step was to create a structural layout for the building. This layout was to include the columns, and slabs present in the building. As far as the design went, it was performed in accordance with the top to bottom sequence of loading or, in other words, load transmission from roof to foundation.

The design process followed for the two-way slabs was in accordance with the Direct Design Method, given in, " Reinforced Concrete, Mechanics And Design", 2nd edition textbook. The Slabs were designed with Two-Way reinforcement, and were only supported by columns. Once the slab design was complete, the concentrated loads acting on the columns could be calculated.

Design of the columns was based on the methods given in, " Reinforced Concrete, Mechanics and Design" . The columns, and their reinforcement were designed under ACI specifications, subject to several ACI checks for shear, and deflection.

Upon completion of the columns, drop panels were designed for their tops, and bottoms. The drop panels were designed under ACI guidelines, using methods form, "Reinforced Concrete, Mechanics and Design".

The slabs and columns designed to make up the buildings main frames were then analyzed for wind and seismic loading conditions under the computer Program *FRAME: Stiffness Analysis of Plane Frames.*© 1988, by John F.Fleming. Lastly the basement slab was designed according to the WPI General Specifications on Institute Hall and in accordance with the Massachusetts Building Code 1990.

3.3 Fire Protection

The first step taken in the fire protection design, and analysis, was to establish the minimum fire protection rating for each building design option. These were found using the 1990 Massachusetts State Buildings Codes, for minimum fire protection ratings.

The next step taken was to organize all the data that had been designed in the Steel, and Reinforced Concrete Designs. The data was organized by types, and material of members.

The "Fire Resistance Directory Vol. I" ,Underwriters Laboratories, and ," Calculation Methods for Fire Resistance", were then used to analyze, and design the members for our given fire protection rating. The fire protection designs were then tabulated for easier reference.

Methodology for Cost Estimating

The methodology used for the construction estimate of this project was based on the bottom to top philosophy. In other words, the foundations are first, then the basement, and so on. In this case, there are six levels of construction: Foundations, Basement Level, First Floor Level, Second Floor Level, Third Floor Level, and Roof Level.

All quantities obtained for the estimate were taken directly out of the design drawings provided. All prices were obtained from the Means Construction Costs Data 1994. Conversion factors were obtained from page 523 in this book and were used to convert project cost from a National Average Cost to Worcester cost. So, individual trades were multiplied by their respective factors to accentuate any location benefits in one design or the other..

These factors, which are also mentioned at the start of the steel estimate, are the following:

Activity or Trade	Cost Factor
2. Sitework	106.1
3.1. Formwork	123.6
3.2. Reinforcing	134.5
3.3. Cast in Place Concrete	114.9
3. Concrete in General	121.1
4. Masonry	131.9
5. Metals	112.0
7. Thermal & Moisture Protection	116.3
9.2 Lath, Plaster & Gypsum	122.3
9.5 Acoustical Ceilings	116.9
Worcester Average	109.8

In order to obtain real costs these factors are divided by 100, thus, obtaining the multiplier as a percent, how it appears in the later table, which is then multiplied times the cost of the respective activity transforming its cost to real cost in Worcester.

These prices represent the unit cost of the activity. The unit price of the activity is obtained after the contractor's profit is introduced into the process. This profit is taken as 8% of total project cost. After it is tabulated as such, it will be prorated amongst all the activities increasing their price by 8% which then becomes the unit price. This final unit price is the one which appears in all the cost reports for both designs.

Furthermore, another important factor in the determination of a final project cost is time. The construction schedule provides an in depth look at what the project represents in a real profile. This is so because a project's base cost does not include interests paid on loans or overheads for the project. Therefore, a comparison based on base cost is not very accurate or wise. However, once an overhead of $2,700 weekly and an 11% biweekly interest are included, the situation changes dramatically and, as future graphs will show, may influence decision making a big deal.

A mock schedule is provided in the appendix for both designs. However, this schedule only lists the critical activities in sequential order. Its real importance is that it shows the duration of construction which will be used to determine interests and overhead.

Final project reports are presented in several different ways for both designs. In the cost estimate sections of each design, several different tables will be introduced detailing cost reports by activity, subdivision, division, budget (materials, labor, and equipment), by floor, and so on. Furthermore, in the cost analysis part of the document a new set of tables will be introduced. These tables will depict weekly cash flow and interest payments of both designs. It will also include 11 graphs with hopes to further detailing the differences between designs, help point out any advantages or disadvantages any design might offer, and help in the later decision making process.

CHAPTER 4

Gravity and Lateral Loads

4. Gravity and Lateral Loads

This chapter directly applies to both the steel and reinforced concrete structural designs. After reviewing the architectural layout[1] of Institute Hall and knowing the general dimensions (lengths, widths, and heights) of the building, the next step in the design process was to determine the various gravity and lateral loads that would be acting on the building. Live loads were obtained in accordance with the Massachusetts Building Code - 1990 edition. Since Massachusetts is located in the northeastern region of the United States, we decided to obtain the wind and earthquake forces acting on the building by comparing both the Canadian Code - NBC 1977 edition and the Massachusetts Code - 1990 edition, and using the one producing the greatest levels of loading.

4.1 Gravity Loads

Institute Hall is, basically, a four story college dormitory composed of a basement with recreational facilities and three floors of apartments. The following are the gravity loads for the building in accordance with the 1990 Massachusets Building Code.

4.1.1 Snow Load

Figure 1111.1 C [2], which is a map of Massachusetts showing snow loads distribution, locates Worcester in zone 3 with a corresponding snow load of 35

[1] see Building Drawings in Appendix A.
[2] 1990 Massachusetts Building Code, p. 11-13 in Appendix B.

psf. Since the building to be designed has a flat roof, the basic snow load is not modified to account for snow drifting or roof slope.

4.1.2 Uniformly Distributed Live Loads

Since the building layout consists of a recreational basement and three apartment floors, the minimum live load distribution[3] is the following:

Live Load for Roof: **35 psf (snow)**

Live Load for Private Apartments: **40 psf**.

Live Load for Public or Entertainment Rooms: **100 psf**.

Live Load for Corridors: **100 psf**.

Live Load for Stairs: **100 psf**.

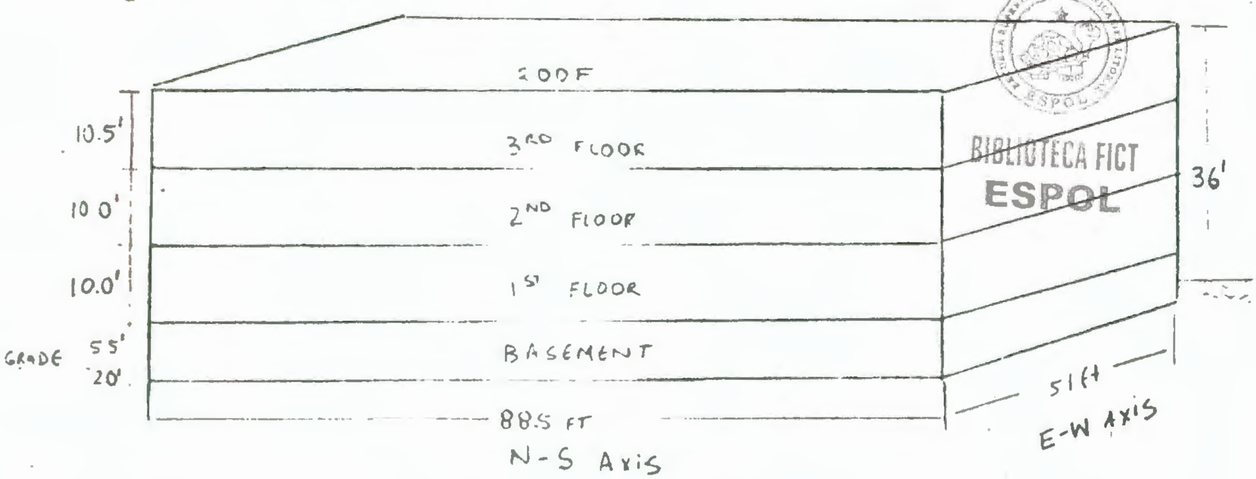
Concentrated Load for Elevators (includes impact effects): **300 k.**

Construction Load: **20 psf.**

4.2 Lateral Loads

The following are the lateral loads acting along the North-South axis and East-West axis of the building. The building's shape is that of a rectangle with an 88.5 ft by 51 ft base. Its roof height is 36 feet height above the ground.

Figure 4.1 Basic Dimensions of Institute Hall



[3] In accordance with the 1990 Mass Code Section 1106, Table 1106, p. 11-3

4.2.1 Wind Load

Our calculations[4] yielded the following wind loads for both.the steel and reinforced concrete designs of the building, since no frame material factor is needed for wind calculations.

4.2.1.1 Reference Pressure

The reference pressure **(P)** is the wind pressure, in lb/ft^2, along the surface of the building where the wind is acting.

Using the Canadian Code: **P = 28.86 psf**

Using the 1990 Mass Code: **P= 17 psf**

The reason for this difference is due to the use of a 2.0 gust effect factor by the Canadian Code. As stated in the introduction to this chapter, we decided to use the more conservative approach for our calculations. Therefore, the calculations for individual wind forces acting on each floor were performed in accordance with the Canadian Code.

4.2.1.2 Wind Force Applied per Floor

The following wind force values per floor[5] were obtained using the tributary area method:

---

[4] Calculations for Wind Loading can be found in Appendix C

[5] In accordance to NBC 1977 Code.

N-S Axis of Building (wind acts along a 51 ft. span)

Floor	*Height Above Ground (ft)*	*Tributary Height (ft)*	*Wind Force (Kips)*	*Moments (ft-K)*
Roof	*36.0*	*5.25*	*7.7*	*277.2*
Third	*25.5*	*10.25*	*15.1*	*385.1*
Second	*15.5*	*10.00*	*14.7*	*227.9*
First	*5.5*	*7.75*	*11.4*	*62.7*
			V= 48.9	*M= 952.8*

V = Shear of the Building
M = Overturning Moment

E-W SIDE OF BUILDING (wind acts along an 88.5 ft.)

Floor	*Height Above Ground (ft)*	*Tributary Height (ft)*	*Wind Force (Kips)*	*Moments (ft-K)*
Roof	*36.0*	*5.25*	*13.4*	*482.4*
Third	*25.5*	*10.25*	*26.2*	*668.1*
Second	*15.5*	*10.00*	*25.5*	*395.3*
First	*5.5*	*7.75*	*19.8*	*108.9*
			V= 84.9	*M=1654.7*

V = Shear of the Building
M = Overturning Moment

4.2.1.3 Wind Force Applied per Frame

The wind force values calculated on the previous section were acting on the entire N-S or E-W projected area of the building. However, there are four set of frames present on each direction. Therefore, the wind forces on the previous section must be distributed per frame to obtain the values of the tributary forces on each frame and be able to use the plane (2 dimensional) structural analysis program *FRAME* to check for maximum deflections.

N-S Side of the Building

Largest tributary width of the 4 frames = **13.25 ft.**

Length of the E-W side (where the wind is acting) = **51 ft.**

Largest Width Ratio = $\frac{13.25}{51.0}$ = **0.26**

N-S Axis of Building (wind acts along a 51 ft. span)

Floor	*Height Above Ground (ft)*	*Wind Force Per Floor (Kips)*	*Width Ratio*	*Wind Force Per Frame (Kips)*
Roof	*36.0*	*7.7*	*0.26*	***2.0***
Third	*25.5*	*15.1*	*0.26*	***3.9***
Second	*15.5*	*14.7*	*0.26*	***3.8***
First	*5.5*	*11.4*	*0.26*	***3.0***

Note: The values in the "wind force per frame" column are used for the plane frame computer analysis.

E-W Side of the Building

Largest tributary width of the 4 frames = **31 ft.**

Length of the E-W side (where the wind is acting) = **88.5 ft.**

Largest Width Ratio = $\frac{\mathbf{31.0}}{\mathbf{88.5}}$ = **0.35**

E-W Axis of Building (wind acts along an 88.5 ft. span)

Floor	*Height Above Ground (ft)*	*Wind Force Per Floor (Kips)*	*Width Ratio*	*Wind Force Per Frame (Kips)*
Roof	*36.0*	*7.7*	*0.35*	***4.7***
Third	*25.5*	*15.1*	*0.35*	***9.2***
Second	*15.5*	*14.7*	*0.35*	***8.9***
First	*5.5*	*11.4*	*0.35*	***6.9***

Note: The values in the "wind force per frame" column are used for the plane frame computer analysis.

4.2.2 Seismic Loading for the Steel Frame

Unlike the wind calculations, the seismic calculations[6] were done independently for the steel and reinforced concrete frames since a **K** factor for each particular type of frame had to be applied. For the steel frame the value of **K** was **0.7**, while the **K** value for the reinforced concrete frame was **1.3**. This made a significant difference when calculating the base shear **V** for each design option.

[6] Calculations for Seismic Loading can be found in Appendix C.

4.2.2.1 Base Shear

The base shear **(V)** is the total lateral force of the seismic force acting at the ground level of the building.

Using the Canadian Code:

North-South Axis (acting along 51 ft): **V = 75.9 kips.**

East-West Axis (acting along 88.5 ft): **V= 68.9 kips.**

Using the 1990 Mass Code:

North-South Axis (acting along 51 ft): **V = 24.20 kips.**

East-West Axis (acting along 88.5 ft): **V = 21.98 kips.**

The reason for such a substantial difference in the values obtained using the Massachusetts Code to those obtained by using the Canadian Code is the 1/3 factor used for the base shear equation of the Massachusetts Code[7]. Besides, we are comparing one country's code to a state's code. The Canadian Code is then more conservative, having a larger value for **V,** since it is designed to cover from coast to coast. As stated before, we decided to use the more conservative method in our calculations. Therefore, the calculations for individual seismic forces acting per floor were done using the Canadian Code.

4.2.2.2 Seismic Force Acting per Floor

The following seismic force values per floor were obtained in accordance with the Canadian Code:

[7] See shear Formula on seismic loading in Appendix B.

North-South Axis of Building [8](seismic force acting along a 51 ft span.)

Floor	*Height (Hx) Above Base level (ft)*	*Wx (Kips)*	*WxHx (Ft-Kips)*	*Fx (ft-K)*	*Moments (Ft-Kips)*
Roof	*36.0*	*158*	*5688*	*27.0*	*972.0*
Third	*25.5*	*212*	*5406*	*25.7*	*655.4*
Second	*15.5*	*212*	*3286*	*15.6*	*241.8*
First	*7.5*	*212*	*1590*	*7.6*	*57.0*
			∑= 15970	*V=75.9*	*M=1926.2*

V = Base Shear of the Building ***M*** = Overturning Moment

East-West Axis of Building (seismic force acting along an 88.5 ft span.)

Floor	*Height (Hx) Above Base level (ft)*	*Wx (Kips)*	*WxHx (Ft-Kips)*	*Fx (ft-K)*	*Moments (Ft-Kips)*
Roof	*36.0*	*158*	*5688*	*24.5*	*882*
Third	*25.5*	*212*	*5406*	*23.3*	*594*
Second	*15.5*	*212*	*3286*	*14.2*	*220*
First	*7.5*	*212*	*1590*	*6.9*	*52*
			∑= 15970	*V=68.9*	*M=1748*

V = Base Shear of the Building ***M*** = Overturning Moment

[8] For reference see **Figure 4.1**.

4.2.2.3 Seismic Force Applied per Steel Frame

The seismic force values calculated on the previous section were acting on the whole N-S or E-W side area of the building. However, there are four set of frames present on each direction of the steel design. Therefore, the wind forces on the previous section must be distributed per frame to be able to use the plane (2 dimensional) structural analysis program *FRAME*.

N-S Side of the Building

Largest tributary width of the 4 frames = **13.25 ft.**

Length of the E-W side (where the seismic force is acting) = **51 ft.**

Largest Width Ratio = $\frac{\mathbf{13.25}}{\mathbf{51.0}}$ = **0.26**

N-S Axis of Building (seismic force acts along a 51 ft. span)

Floor	*Height Above Ground (ft)*	*Seismic Force Per Floor (Kips)*	*Width Ratio*	*Seismic Force Per Frame (Kips)*
Roof	*36.0*	*27.0*	*0.26*	***7.0***
Third	*25.5*	*25.7*	*0.26*	***6.7***
Second	*15.5*	*15.6*	*0.26*	***4.1***
First	*5.5*	*7.6*	*0.26*	***2.0***

Note: The values in the "seismic force per frame" column are used for the plane frame computer analysis.

E-W Side of the Building

Largest tributary width of the 4 frames = **31 ft.**

Length of the E-W side (where the seismic force is acting) = 88.5 **ft.**

Largest Width Ratio = $\frac{31.0}{88.5}$ = **0.35**

E-W Axis of Building (seismic force acts along an 88.5 ft. span)

Floor	*Height Above Ground (ft)*	*Seismic Force Per Floor (Kips)*	*Width Ratio*	*Seismic Force Per Frame (Kips)*
Roof	*36.0*	*24.5*	*0.35*	***8.6***
Third	*25.5*	*23.3*	*0.35*	***8.2***
Second	*15.5*	*14.2*	*0.35*	***5.0***
First	*5.5*	*6.9*	*0.35*	***2.4***

Note: The values in the "seismic force per frame" column are used for the plane frame computer analysis.

4.2.3 Seismic Loading for the Reinforced Concrete Frame

The **K** factor for the reinforced concrete frame was **1.3**. Following are the calculations for the seismic loads acting on the frame.

4.2.3.1 Base Shear

The base shear **(V)** is the total lateral force of the seismic force acting at the ground level of the building.

<u>Using the Canadian Code</u>:

North-South Axis (acting along 51 ft): **V = 140.9 kips.**

East-West Axis (acting along 88.5 ft): **V= 128.9 kips.**

<u>Using the 1990 Mass Code</u>:

North-South (acting along 51 ft): **V = 47.0 kips.**

East-West (acting along 88.5 ft): **V = 42.7 kips.**

The reason for such a big difference in the values obtained using the Massachusetts Code to those obtained by using the Canadian Code is the 1/3 factor used for the base shear equation of the Massachusetts Code[9], as explained in section 4.2.2.1.

4.2.3.2 Seismic Force Acting per Floor

The following seismic force values per floor were obtained for the reinforced concrete design in accordance with the Canadian Code:

[9] See shear Formula on seismic loading in Appendix B).

North-South Axis of Building [10](seismic force acting along a 51ft span.)

Floor	*Height (Hx) Above Base level (ft)*	*Wx (Kips)*	*WxHx (Ft-Kips)*	*Fx (ft-K)*	*Moments (Ft-Kips)*
Roof	36.0	158	5688	50.2	1807.2
Third	25.5	212	5406	47.7	1216.4
Second	15.5	212	3286	29.0	450.0
First	7.5	212	1590	14.0	105.0
			$\sum$= 15970	V=140.9	M=3578.6

V = Base Shear of the Building *M* = Overturning Moment

East-West Axis of Building (seismic force acting along an 88.5 ft span.)

Floor	*Height (Hx) Above Base level (ft)*	*Wx (Kips)*	*WxHx (Ft-Kips)*	*Fx (ft-K)*	*Moments (Ft-Kips)*
Roof	36.0	158	5688	45.6	1641.6
Third	25.5	212	5406	43.3	1104.2
Second	15.5	212	3286	26.3	407.6
First	7.5	212	1590	12.7	95.3
			$\sum$= 15970	V=128.0	M=3248.7

V = Base Shear of the Building *M* = Overturning Moment

[10] For reference see **Figure 4.1**.

4.2.3.3 Seismic Force Applied per Reinforced Concrete Frame

The seismic force values calculated on the previous section were acting on the entire N-S or E-W side area of the building. However, there are six sets of frames present on the N-S side and four on the E-W side. Therefore, the wind forces on the previous section must be distributed per frame to be able to calculate the maximum deflection per frame by using the plane (2 dimensional) structural analysis program *FRAME*.

N-S Side of the Building

Largest tributary width of the 6 frames = **13.25 ft.**

Length of the E-W side (where the seismic force is acting) = **51 ft.**

Largest Width Ratio = $\dfrac{13.25}{\mathbf{51.0}}$ = **0.26**

N-S Axis of Building (seismic force acts along a 51 ft. span)

Floor	*Height Above Ground (ft)*	*Seismic Force Per Floor (Kips)*	*Width Ratio*	*Seismic Force Per Frame (Kips)*
Roof	*36.0*	*50.2*	*0.26*	***13.1***
Third	*25.5*	*47.7*	*0.26*	***12.4***
Second	*15.5*	*29.0*	*0.26*	***7.5***
First	*5.5*	*14.0*	*0.26*	***3.6***

Note: The values in the "seismic force per frame" column are used for the plane frame computer analysis.

E-W Side of the Building

Largest tributary width of the 4 frames = **31 ft.**

Length of the E-W side (where the seismic force is acting) = **88.5 ft.**

Largest Width Ratio = $\frac{\mathbf{31.0}}{\mathbf{88.5}}$ = **0.35**

E-W Axis of Building (seismic force acts along an 88.5 ft. span)

Floor	*Height Above Ground (ft)*	*Seismic Force Per Floor (Kips)*	*Width Ratio*	*Seismic Force Per Frame (Kips)*
Roof	*36.0*	*45.6*	*0.35*	***16.0***
Third	*25.5*	*43.3*	*0.35*	***15.2***
Second	*15.5*	*26.3*	*0.35*	***9.2***
First	*5.5*	*12.7*	*0.35*	***4.4***

Note: The values in the "seismic force per frame" column are used for the plane frame computer analysis.

4.3 SUMMARY

For both the wind and seismic calculations, the force values obtained from the Canadián Code were higher than those obtained from the Massachusetts Building Code. We preferred to use the Canadian Code to be more conservative in our calculations. Based on the results, one can also see that the wind or seismic force values obtained were greater for the East-West axis than for the

North-South axis, and that the overturning moment was greater for the seismic force than for the wind force.

Therefore, the structural members were designed for gravity loads and for a mix of both wind and seismic loads as the lateral forces. This design was based on the values obtained by using the Canadian Code. Once, the structural members for the steel and reinforced concrete designs (chapters 5 & 6), are designed for gravity loads, they will be analyzed under the influence of wind and seismic forces by performing a computer analysis with the structural design program *FRAME*[11].

[11] A printout of this analysis for the steel and reinforced concrete frames is located in Appendices F &G.

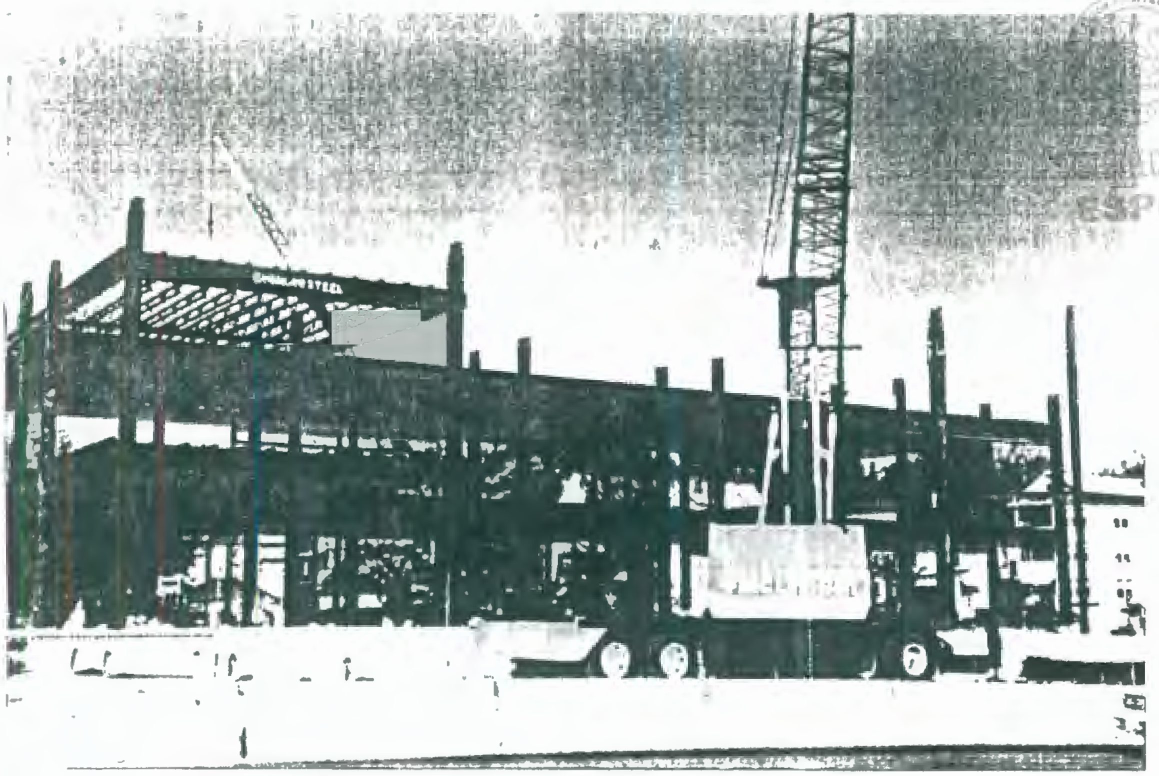
CHAPTER 5

Steel Design

5. Steel Design

At present, as we can see from new construction and renovation projects throughout the Worcester area and here at Worcester Polytechnic Institute, a steel frame with thin concrete slabs placed over a corrugated steel deck, and drywalls connected to metal studs, are becoming a trend in the construction business. Two major features of this system are its fast assembly and ability to use a wide variety of suspended ceilings, HVAC and electrical systems.

Figure 5.1 Steel frame for a construction project in front of Webster Square. Construction began in September 1994.



This chapter summarizes the steel design of a lateral load sytem for the roof, floors, beams, girders, columns, and foundations of Institute Hall. Furthermore, the steel design will be compared to a reinforced concrete design, presented in chapter 6, in terms of estimated cost and life cycle issues such as heating and fire-safety, extremely important in the case of a college dormitory. We will then see which design performs better to be used in future projects.

The structural frame of a building, acting as a skeleton, supports all the loads acting on it. For our building, in general, gravity loads (dead, live, and snow) govern over lateral loads (wind and seismic). The main structural members such as girders, beams, and columns will be designed for gravity loads but will be checked for wind and seismic forces using FRAME, a computer analysis program[1]. As far as the design goes, it will be performed in accordance with the top to bottom sequence of loading or, in other words, load transmission from roof to foundation.

5.1 Introduction to the Design

After finding the gravity and lateral loads acting on the building, presented in chapter 4, our next step was to make a thorough review of the space distribution along the four stories of the building. In this case, space distribution implies the way in which the layout of the building is configured. In other words, size and location of living quarters, bathrooms, corridors, stairways, elevators, and so on[2]. The purpose of the review was to develop a structural

---

[1] For a thorough computer analysis on the steel frame see appendix F.
[2] CAD drawings of Institute Hall, obtained from plant services, can be seen in appendix A.

layout that coincided with the architectural drawings of the building, avoiding problems like placing columns in the middle of entrances or beams running across stairways.

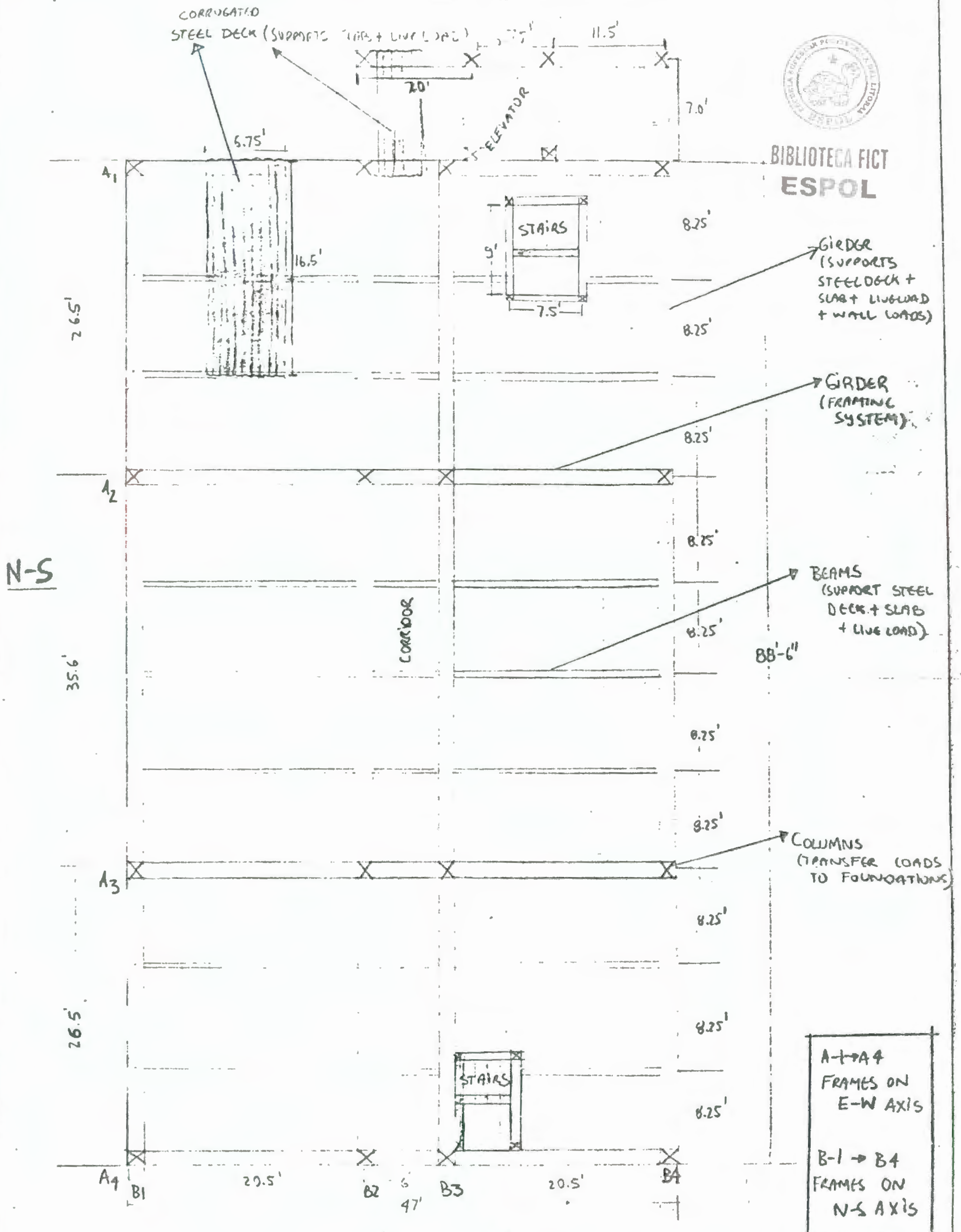
5.2 Distribution and Layout of Structural Members

Knowing where columns and beams could be placed, the next step was to create a structural layout of the building. This layout was to include the columns, girders, beams, and steel deck plates present in the building. They were distributed along the building using two factors as the governing criteria:

1. Creating a distribution that would standardize spacing and sizes of members.
2. Using a type of material that would be cost efficient for the construction site region, such as A36 steel.

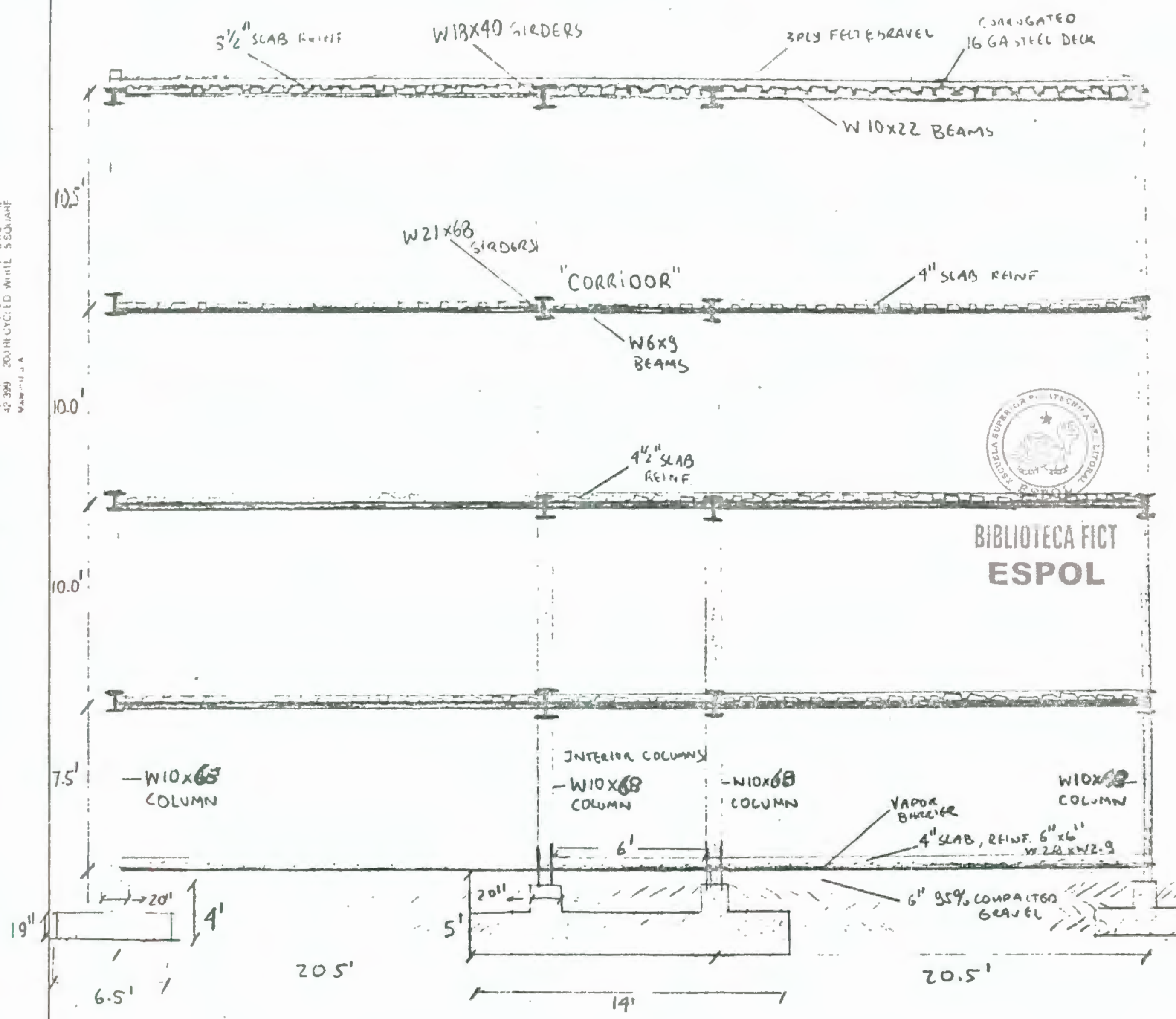
Figures 5.4 and **5.5**, following, depict a top and side view of the distribution and layout of structural members for the steel design.

STEEL DESIGN (DIMENSIONS)



"TOP VIEW" (BEFORE DESIGN)

E-W



E-W VIEW @ CENTER FRAME

SIDE VIEW OF FRAMES A3 OR A2 ON FIGURE 5.4

5.3 Roof Design

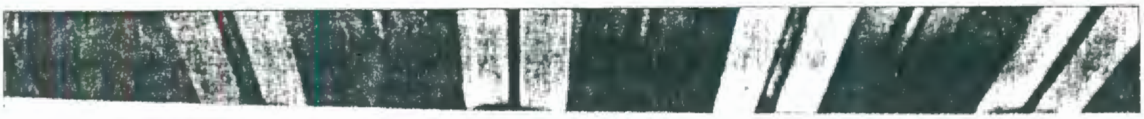
The type of roof designed was a reinforced concrete slab on an inverted, "B" steel deck (1.5" standard & galvanized) to be covered by a 3 ply felt and gravel, and supported by beams[3].

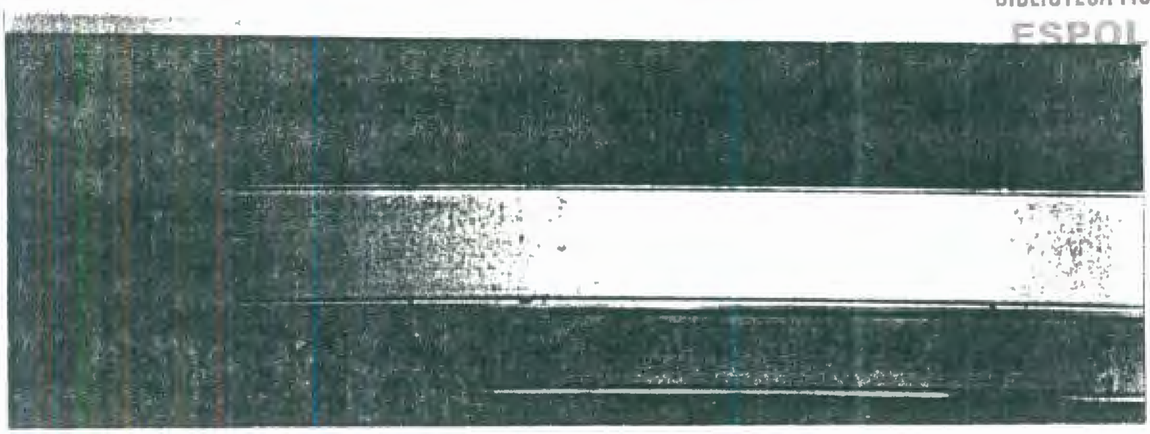
5.3.1 General Information

1. Steel deck runs in the opposite direction to the beams length as seen in **figure 5.6** and designed for a two-span, and is to be welded to the structural supports using 5/8" dia-puddle welds, spaced not more than 12 " on center.

2. Reinforcement is a WWC (Welded Wire Fabric) supplying the minimum A_s permitted by ACI code for Temperature, shrinkage and structural reinforcement.

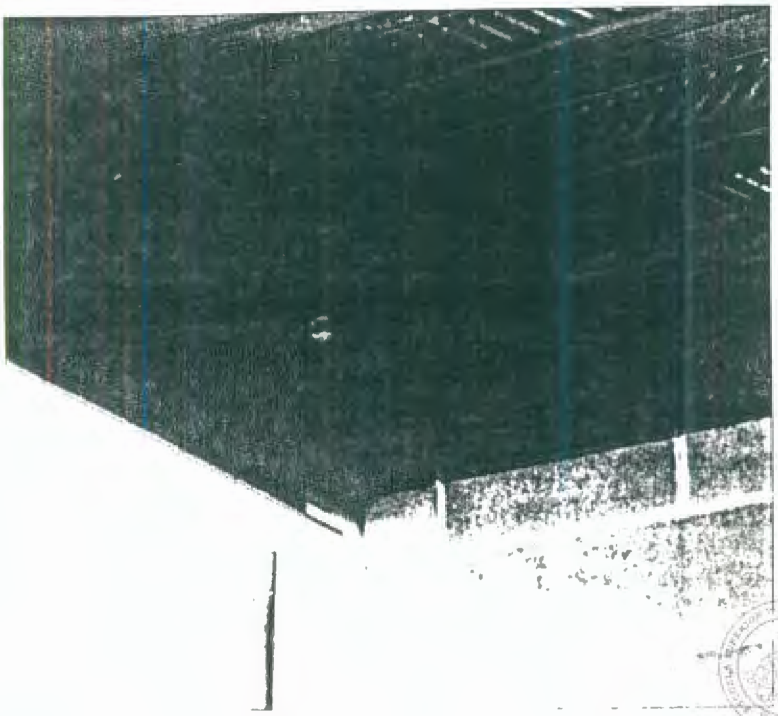
Figure 5.6 Layout of beams supporting the steel deck in the Royal Worcester Apartments parking garage.





[3] The roof and floor decks were designed in accordance with the United Steel Deck, Inc. (from 1992 SWEET's Catalog, file 2). All design details and calculations can be found in Appendix D under Steel Design.

Figure 5.7 Steel deck of Royal Worcester Apartments' parking garage. It shows connections of columns- beams and the steel deck.



5.3.2 Load and Design Values for Roof's Slab

Reinforced concrete values	Loads acting on Roof
$f_c' = 3000$ psi	35 psf snow load
$f_c = 1350$ psi	20 psf construction load
$f_s = 30,000$ psi	

5.3.3 Roof's Steel Deck

The steel deck was designed by checking the load **w** by using two equations from the Sweet's Catalog[4]. These two equations were the following:

1. Negative moment (governs)

$$-M = \frac{wL^2}{8}$$

2. Deflection

$$\Delta = 5\, w_1 L^4 / 384EI$$

On these two equations,

$w = w_1 + w_{2\,(psf)}$

w_1= Weight of wet concrete slab + Deck (psf)

w_2=Snow load (psf)

L=length of deck's span

E= Steel's modulus of elasticity

I= Moment of Inertia

Then the value of w obtained was used to enter a table of different steel deck gauges such that the value from the table was greater than the value obtained from $w_1 + w_2$. This gave us the required steel deck and slab to support the loads.[5]

The following design was obtained for the roof's corrugated steel deck plates:

---

[4] see pg.14 on Sweet's Catalog, file 2, under steel decks.
[5] See calculations in appendix D.

GAUGE:	**16 GA**
SPAN:	**TWO @ 8.25 ft = 16.5 ft.**
WIDTH:	**6.75" (to simplify design)**
WELD:	**$^5/_8$ " dia-puddle welds** **SPACED: 12" o.c.** **Side laps of sheets to be welded**

Concrete slab is of composite construction with the steel deck:

SLAB SIZE:	**3.5"**
REINFORCEMENT:	**66-44 Mesh** **in direction of deck span**

5.4 Floor Design

The type of floor designed was a reinforced concrete slab on an inverted, "B" steel deck (1.5" standard & galvanized). This design is to be repetitive for the first, second, and third floors of the building because they all have the same space distribution[6].

5.4.1 General Information

1. Steel deck runs in the opposite direction to the beams length as seen in **figure 5.6** and designed for a two-span, and is to be welded to the structural supports using 5/8" dia-puddle welds, spaced not more than 12 " on center.

2. Reinforcement is a WWC (Welded Wire Fabric) supplying the minimum A_s permitted by ACI code for Temperature, shrinkage and structural reinforcement.

[6] See building's drawings in appendix A.

5.4.2 Load and Design Values for Floors

Reinforced concrete values	Loads acting on Floors
f'_c = 3000 psi	40 psf (residential apartments)
f_c = 1350 psi	100 psf (corridors)
f_s = 30,000 psi	20 psf construction load

5.4.3 Floor's Steel Deck

The floor slab and steel deck were obtained using the same equations shown in section 5.3.3. [7] The following design was obtained for the corrugated steel deck plates:

5.4.3.1 Design for Apartments Area

GAUGE: **16 GA**

SPAN: **TWO @ 8.25 ft = 16.5 ft.**

WIDTH: **6.75" (to simplify design)**

WELD: **$^5/_8$ " dia-puddle welds**
SPACED: 12" o.c.
Side laps of sheets to be welded

Concrete slab is of composite construction with the steel deck:

SLAB SIZE: **4.0"**
REINFORCEMENT: **66-44 Mesh**
in direction of deck span

[7] See Floor Deck calculations in appendix D.

5.4.3.2 Design for Corridors Area

GAUGE:	**16 GA**
SPAN:	**TWO @ 8.25 ft = 16.5 ft.**
WIDTH:	**6.75" (to simplify design)**
WELD:	**5/8 " dia-puddle welds** **SPACED: 12" o.c.** **Side laps of sheets to be welded**

Concrete slab is of composite construction with the steel deck:

SLAB SIZE:	**4.5"**
REINFORCEMENT:	**44-44 Mesh** **in direction of deck span**

5.5 Structural Members Design

Once the roof and floor slab/decking systems were designed to carry their particular live and dead loads, the design of the structural members such as beams, girders, and columns could proceed since all the uniform dead and live loads acting on them were known.

The design of these members was done by using John Smith's Structural Steel Design: LRFD Approach ,1991 edition textbook , in conjunction with AISC's LRFD [8] manual of steel construction. The design was in accordance to the Mass Building Code 1990, which says that the design of steel structural members is to be done using the AISC's LRFD manual.

[8] AISC stands for American Institute of Steel Construction and LRFD stands for Load and Resistance Factor Design.

Value-engineering is a topic commonly used today in the construction business. It is basically an analytical study of the possible options for a project and choosing the one which will optimize the project both in terms of efficiency and economy. Therefore, the design of all beams, girders, and columns for the building were based on ***A 36 steel*** since this carbon steel is commonly used for general purposes, especially for the design of buildings.

All structural members were designed to be ***W sections*** since it is the most common shape used in structural steel design as a *beam* (bending member), a *column* (axial compression member), and a *beam-column* (bending plus axial compression member). [9]

The structural steel design of the building members was done in sequence, starting from the design of the roof beams, girders, and columns all the way down to the design of the foundations supporting all live and dead loads of the building; where the columns and girders would be making up resisting frames both for wind and seismic forces.

5.5.1 Design of Roof and Floor Beams

The first structural members to be designed were the roof and floor beams since they only support the steel deck and concrete slab, which carry the roof live load for snow (35 psf) or the floor live load for private apartments (40 psf). The beams design did not require calculation of dead load values for walls or other members acting over them. The design was based on calculating the

[9] Smith, John. Structural Steel Design: LRFD Approach, 1991 edition, pgs.2-8.

largest tributary area that each beam would be supporting[10]. Knowing this, the total live and dead loads acting on the beam could be calculated.

These loads were multiplied by the following factors required by the AISC:

$$\mathbf{W_u = 1.2\,D + 1.6\,L}$$

Using this uniform load (Wu) we found the reactions @ supports, the ultimate shear V_u, and the ultimate moment M_u (for simple spans):

$$M_u = \frac{W_u l^2}{8}$$

Then we searched through the beam tables in the LRFD manual for the lightest A36 W section with the required span (see layout sketch of structural members on **Figures 5.4 and 5.5**) and whose designed moment multiplied by a safety factor (ϕ_b) of .85 would be greater than the ultimate moment:

$$\phi_b\, M_p \geq M_u$$

This W section was then checked for local buckling, shear, and deflection limit.

5.5.1.1 Check for Local Buckling

Both the flange and the web of the W section were checked for local buckling using the following AISC equations.

<u>Check for the Flange:</u>

$$\lambda_p = \frac{65}{\sqrt{f_y}} \geq \frac{0.5\, b_f}{t_f}$$

where: f_y = yield strength of steel
b_f = Flange's base width
t_f = Flange's thickness

[10] For reference, see appendix D.

Check for the Web:

$$\lambda_{p=}\ \frac{65}{\sqrt{f_y}} \geq \frac{h_c}{t_w}$$

where:

$f_{y=}$ yield strength of steel
$t_{w=}$ Web's thickness

5.5.1.2 Check for shear

To check the W section for shear we used the following AISC equation:

$$\phi V_n \geq V_u$$

where:

$\phi_=$0.9 safety factor
$V_{n=}$ **0.6 F_y t_w d**
$V_{u=}$ required shear strength
$f_{y=}$ yield strength of steel
$t_{w=}$ Web's thickness
d= section's depth

5.5.1.3 Check for Deflection

The maximum deflection had to be within the limits of the limiting deflection using the following AISC equation:

$$\Delta_{lim.=}\ \frac{\text{Span (in)}}{360} \geq \Delta_{max.=}\ \frac{5wL^4}{384EI}$$

Where:

w = service live load
L= span of the beam
E = Steel's Modulus of elasticity
I = Section's moment of inertia

If the W section satisfied all the requirements for local buckling, shear, and deflection, then it was chosen as an appropriate member for the design of the building, if not, the next W section with a higher. ϕ_b M_p was selected and checked for all the requirements again.

After performing the calculations, see appendix D on Steel Design, the following members were chosen as roof beams and floor beams, which came out to be the same since the 40 psf for private apartment does not make a big difference compared to the 35 psf for snow load.

Beams required for both Roof and Floors

Member	*Location*	*Length(ft)*
W 10x22	Apartments area	20.5
W 6x9	Corridors area	6.0

5.5.2 Design of Roof and Floor Girders

The roof and floor girders run along the perimeter, corridor, and main divisions of the building[11].

Roof girders were designed to support all the live and dead loads transferred from the roof beams, and as well as the columns support all lateral loads. The girder design was done following the same procedures as those taken in the beam design (section 5.5.1), thus selecting a W section that would satisfy all requirements for local buckling, shear, and allowable deflections. In the case of the floor girders, they had to support the vertical loads transferred from the walls and columns. In order to design the girders we first had to choose the type of outer wall system we wanted on the building. We decided to use 8" hollow concrete blocks with heavy aggregate as our outer wall system[12].

[11] See Figures 5.4 and 5.5 for layout of girders.
[12] For calculations refer to appendix D.

After performing the calculations for the girders, see appendix D, the following members were chosen as roof or floor girders supporting the largest tributary area of either 35 psf for snow loads, 40 psf for private apartment ,and 100 psf for corridors.

Girders Required for Roof and Floors

Member	*Location*	*Lengths(ft)*
W 18x40	Roof	35.5, 26.5, 20.5, & 6.0
W 21x68	1st, 2nd & 3rd Floors	35.5, 26.5, 20.5, & 6.0

5.5.3 Design of Columns

After designing all the horizontal elements of the steel frame such as the metal decking, the beams, and the girders, we could then proceed to design the columns, which are the vertical elements transferring all live and dead loads from the roof to the foundations, and which as well resist lateral loads due to wind and earthquakes.

The column design was based on W shapes and done by using Smith's textbook and in accordance to the LRFD manual[13]. The benefit of choosing a W section as a column is since it is a doubly symmetric cross section, it only twists when the column buckles (torsional mode of buckling). To have a balance between safety and economy, and since the building consists of only four floors, we decided to standardize the columns by designing the basement level columns

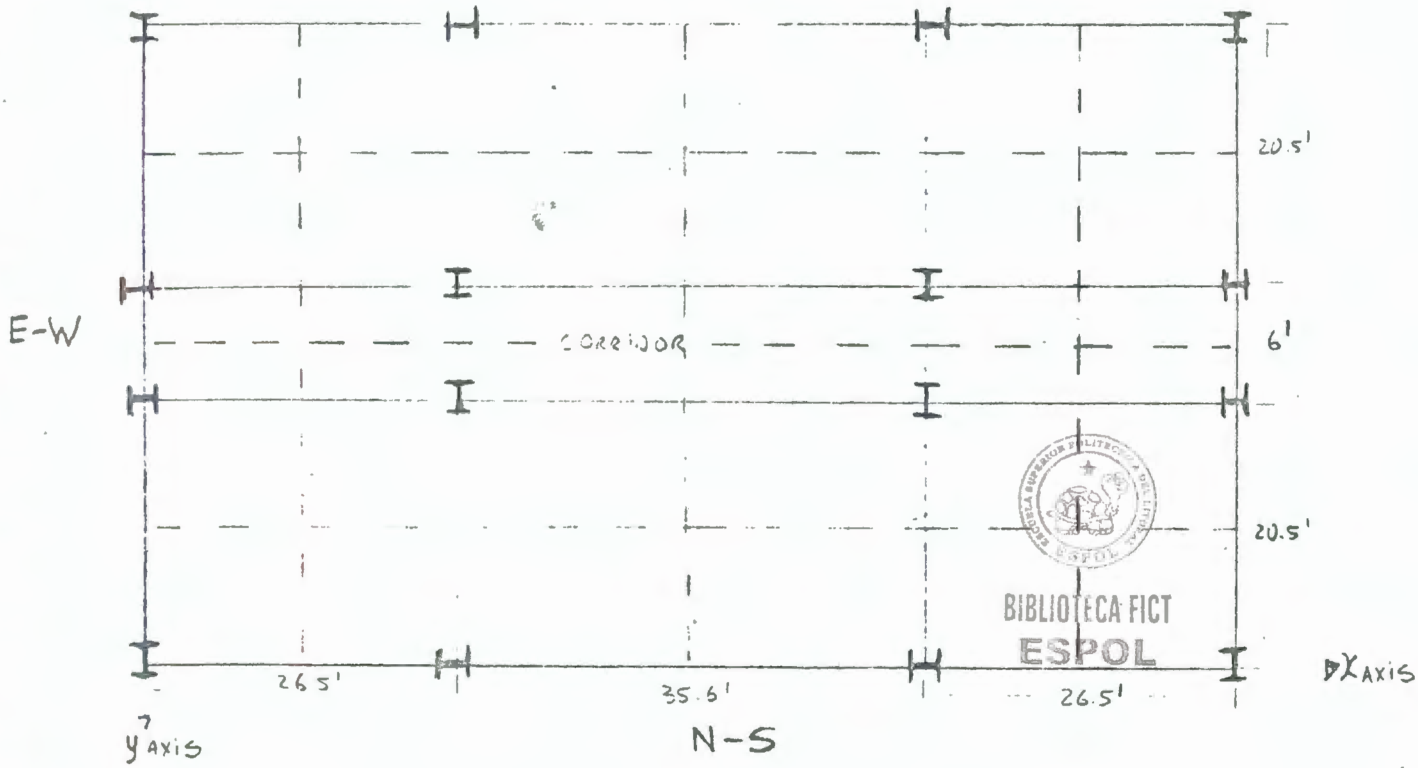
[13] Smith, J.C. Structural Steel Design: LRFD approach. New York: John Wiley and Sons, 1991 edition.
LRFD (Load & Resistance Factor Design: Manual of Steel Construction). AISC, 1986 edition.

and applying this design for higher floors columns. Four types of columns were designed based on their tributary areas. These four types were: corner columns, N-S edge columns, E-W edge columns, and interior columns.

<u>Type of columns</u> (refer to **Figure 5.8**)

1. Corner columns are labeled: 1,4,16,13
2. N-S edge columns are labeled: 8,9,5,12
3. E-W edge columns are labeled: 2,3,14,15
4. Interior columns are labeled: 6,7,10,11

Figure 5.8 Sketch of columns layout.



Note: Shaded areas are the respective tributary areas for each column. As one can see, the corner columns have the smallest tributary area while the interior columns have the largest tributary area.

From the sketch one can also note that each cross section in the corners of the building is rotated 90° with respect to its neighboring cross sections. This was done deliberately to provide some major principal axis column bending strength for resisting sidesway buckling in both directions (X and Y)[14].

5.5.3.1 Obtaining the Concentrated Load at each Column

After finding the tributary areas for all columns, see **figure 5.8**, all the dead and live loads, including exterior walls, were calculated for each type of column at the basement level. All these live and dead loads, usually in pounds per square feet, were multiplied by the tributary areas to obtain a concentrated load, P_u, acting at the center of each type of column.[15]

Concentrated Loads Acting on Columns @ basement level

TYPE OF COLUMN	*P_U (kips)*
Corner columns	**110**
N-S edge columns	**197**
E-W edge columns	**142**
Interior columns	**287**

5.5.3.2 Selection of Column Sections

The first step in selecting a w shape for a particular column was to:

A. Check the Y-Y axis:

To be safe, assume column pin connected at top and bottom with sidesway inhibited.

[14] Smith, John. Structural Steel Design, 1991 edition, pg.144
[15] see calculations on "Column Design" section of Appendix D.

From Table C-C2.1 in the LRFD for condition (d), K=1.0

Therefore, the effective length for the columns is:

$$Kl_y = \text{length of column} \times \text{K factor}$$

Then we entered the column tables at this effective length and selected a W shape whose ϕP_u was greater than the previously calculated P_u from the tributary area; where ϕ = **0.85**.

B. Check the X-X axis

From properties section in column tables in the LRFD, the r_x/r_y ratio was found for the W section previously selected for the Y-Y axis.

If the effective length **(Kl)** of the column divided by the r_x/r_y ratio was less than the original **Kl**, then the w section was acceptable. If not, the check for the Y-Y and X-X axes with the next lightest w section having a greater ϕP_u had to be done over again[16].

Preliminary W Sections For Columns

COLUMN TYPE	*W SECTION*
Corner columns	**W 8x24**
N-S edge columns	**W 8x31**
E-W edge columns	**W 8x24**
Interior columns	**W 10x45**

[16] See appendix D for all column calculations.

5.5.4 Preliminary Wind and Seismic Computer Analysis For the Steel Frame

All the girders and columns designed in sections 5.5.2 through 5.5.3 are assembled together into frames which make up the building's skeleton. Since the designs of both girders and columns were based on live and dead loads only, they must be exposed to a computer analysis for wind and seismic forces[17] to check if the frame would be acceptable under such circumstances. We did four computer runs to verify the frame:

Run 1. N-S wind analysis.
Run 2. N-S earthquake analysis.
Run 3. E-W wind analysis.
Run 4. E-W earthquake analysis.

Note: Each run had the following load combinations: (**1.2 DL + 1.6 L.L + 1.0 W or 1.0 E**), and the concentrated load $\mathbf{P_u}$ for the columns.

5.5.4.1 *FRAME © :* Structural Analysis Computer Program

In order to perform the wind and seismic analysis we used the computer program ***FRAME***[18]. This two dimensional (plane) structural analysis program is composed of program running files, a data input file, and an output file.

The first step was to enter all the member and joint data for the frame to be analyzed. This was done by editing a sample input data file from the program called "plfrm.dat." To access this file one must go into **DOS ©** and at the prompt type: "**edit plfrm.dat**".

---

[17] see Wind and Seismic calculations on chapter 4.
[18] Program *FRAME: Stiffness Analysis of Plane Frames*.© 1988, by John F.Fleming.

Once in this file, one makes all the necessary changes for one's particular frames' materials, members, and joints data. When all the changes are made, save it under a different name such as "**example.dat**".

Then one must run the file: "**frame**". This file will ask for the input file; in this case "**example.dat**", and for an output file such as "**example.out**".

Finally when the message "**stop-Program Terminated**" appears on the screen, one can go over and analyze the computer analysis of the frame by typing the following at the prompt command:

"**edit example.out**"

If one is satisfied with the frame analysis one can print it by typing:

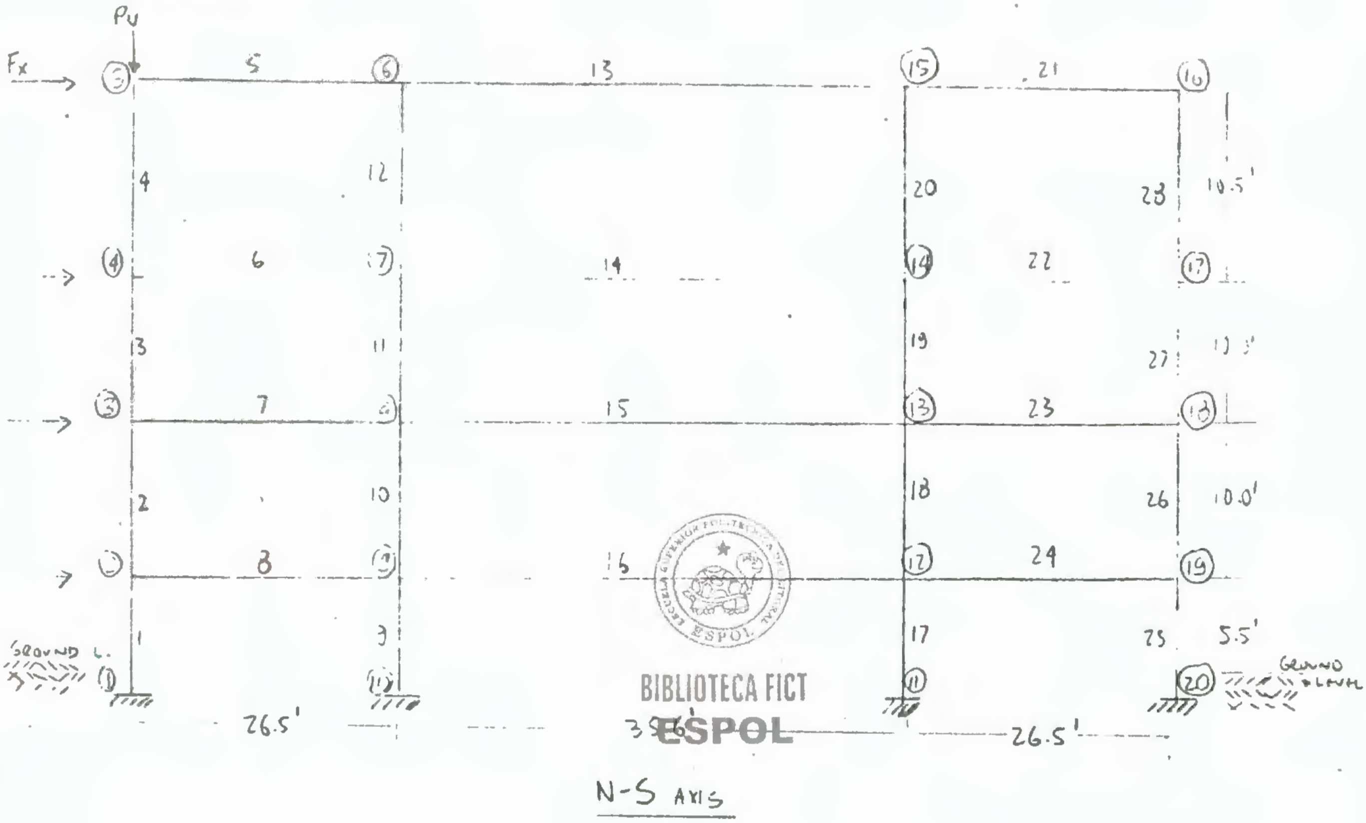
"**print example.dat**"

5.5.4.2 Required Data for the Preliminary Computer Analysis

The computer analysis[19] required to number all the joints and members in the N-S and E-W frames and then enter them with all their required data in tabular form:

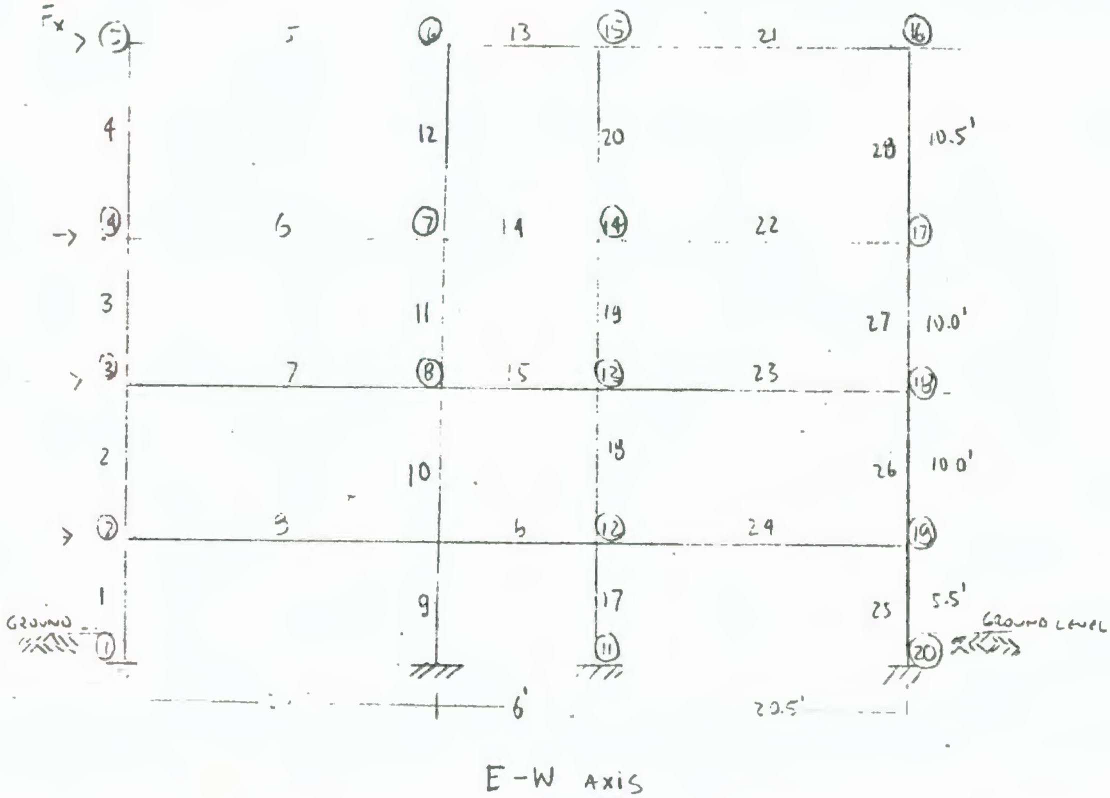
[19] For a printout of the computer analysis on the Steel Frame see Appendix F.

Figure 5.9 N-S view



EX. ① = JOINT REFERENCE NUMBER
1 = MEMBER REFERENCE NUMBER
F_x = WIND OR SEISMIC FORCE

Figure 5.10 E-W view



Summary of Preliminary Members for the Steel Design of Institute Hall

	Design of Members for the Main Section of the Building			
Member	*Type*	*Location*	*Length(ft)*	*Quantity*
W 18x40	Girder	Roof	35.5	4
W 18x40	Girder	Roof	26.5	8
W 18x40	Girder	Roof	20.5	8
W 18x40	Girder	Roof	6.0	4
W 21x68	Girder	3rd, 2nd, & 1st Floors	35.5	12
W 21x68	Girder	3rd, 2nd, & 1st Floors	26.5	24
W 21x68	Girder	3rd, 2nd, & 1st Floors	20.5	24
W 21x68	Girder	3rd, 2nd, & 1st Floors	6.0	12
W 10x22	Beam	Roof, 3rd, 2nd, & 1st Floors	20.5	56
W 6x9	Beam	Roof, 3rd, 2nd, & 1st Floors	6.0	28
W 10x45	Interior column	Roof	10.5	4
W 10x45	Interior column	3rd, & 2nd Floors	10.0	8
W 10x45	Interior column	1st Floor	7.5	4
W 8x31	N-S columns	Roof	10.5	4
W 8x31	N-S columns	3rd, & 2nd Floors	10.0	8
W 8x31	N-S columns	1st Floor	7.5	4
W 8x24	E-W & Corner columns	Roof	10.5	8
W 8x24	E-W & Corner columns	3rd, & 2nd Floors	10.0	16
W 8x24	E-W & Corner columns	1st Floor	7.5	8
W 8x24	Stairway columns	3rd, & 2nd Floors	10.0	16
W 8x24	Stairway columns	1st Floor	7.5	8
W 12x14	Stairway beams	3rd,2nd, & 1st Floors	9.0	12
W 12x14	Stairway beams	3rd,2nd, & 1st Floors	7.5	12
	Design of Members for the Elevator Section of the Building			
Member	*Type*	*Location (below)*	*Length(ft)*	*Quantity*
W 18x40	Girder	Roof	11.50	1
W 18x40	Girder	Roof	7.00	4
W 18x40	Girder	Roof	6.75	2
W 21x68	Girder	3rd, 2nd, & 1st Floors	7.00	12
W 21x68	Girder	3rd, 2nd, & 1st Floors	6.75	6
W 8x24	Elevator waiting area	Roof	10.5	1
W 8x24	Elevator waiting area	3rd, & 2nd Floors	10.0	3
W 8x24	Elevator waiting area	1st Floor	7.5	1
W 8x24	Elevator shaft columns	Roof	10.5	4
W 8x24	Elevator shaft columns	3rd, & 2nd Floors	10.00	8
W 8x24	Elevator shaft columns	1st Floor	7.50	4

Column and Girder Properties for the Preliminary Steel Frame Computer Analysis on the N-S side of Institute Hall

All steel members are A36; Fy=36 ksi; Modulus of elasticity (E)= 29,000 ksi
All section properties were obtained from the LRFD tables
All members, loads, and forces were calculated in appendix D

N-S side Columns of Building

#	W Section	AX (in^2)	IY (in^4)	IX (in^4)	Length (ft)	Pu (Kips)
1	W 8x24	7.08	18.3	82.8	5.5	110.0
2	W 8x24	7.08	18.3	82.8	10.0	78.0
3	W 8x24	7.08	18.3	82.8	10.0	46.3
4	W 8x24	7.08	18.3	82.8	10.5	14.6
9	W 8x31	9.13	37.1	110	5.5	197.0
10	W 8x31	9.13	37.1	110	10.0	141.8
11	W 8x31	9.13	37.1	110	10.0	86.7
12	W 8x31	9.13	37.1	110	10.5	31.6
17	W 8x31	9.13	37.1	110	5.5	197.0
18	W 8x31	9.13	37.1	110	10.0	141.8
19	W 8x31	9.13	37.1	110	10.0	86.7
20	W 8x31	9.13	37.1	110	10.5	31.6
25	W 8x24	7.08	18.3	82.8	5.5	110.0
26	W 8x24	7.08	18.3	82.8	10.0	78.0
27	W 8x24	7.08	18.3	82.8	10.0	46.3
28	W 8x24	7.08	18.3	82.8	10.5	14.6

N-S side Girders of Building

#	W Section	AX (in^2)	IX (in^4)	Length (ft)	Wu (k/ft)	($Wu\ l^2/12$) Mz (k-ft)
5	W 18x40	11.8	612	26.5	1.31	76.7
6	W 21x68	20.0	1480	26.5	2.66	155.7
7	W 21x68	20.0	1480	26.5	2.66	155.7
8	W 21x68	20.0	1480	26.5	2.66	155.7
13	W 18x40	11.8	612	35.5	1.31	137.6
14	W 21x68	20.0	1480	35.5	2.66	279.4
15	W 21x68	20.0	1480	35.5	2.66	279.4
16	W 21x68	20.0	1480	35.5	2.66	279.4
21	W 18x40	11.8	612	26.5	1.31	76.7
22	W 21x68	20.0	1480	26.5	2.66	155.7
23	W 21x68	20.0	1480	26.5	2.66	155.7
24	W 21x68	20.0	1480	26.5	2.66	155.7

Note: Wu =1.2 D +1.6 L

Joint Data for the Steel Frame Computer Analysis on the N-S side of Institute Hall

Joint #	X-cor. (ft)	X-cor. (in)	Y-cor. (ft)	Y-cor. (in)	Pu (k)	Wind Fx (k) Per Frame	Earthquake Fx (k) Per Frame	(+) Moment Mz (ft-k)	(-) Moment Mz (ft-k)	Net Moment Mz (ft-k)
1	0.0	0.0	0.0	0.0						
2	0.0	0.0	5.5	66.0	108.9	3.0	2.0	0.0	155.7	-155.7
3	0.0	0.0	15.5	186.0	77.8	3.8	4.1	0.0	155.7	-155.7
4	0.0	0.0	25.5	306.0	46.3	3.9	6.7	0.0	155.7	-155.7
5	0.0	0.0	36.0	432.0	13.5	2.0	7.0	0.0	76.7	-76.7
6	26.5	318.0	36.0	432.0	30.5	0.0	0.0	76.7	137.6	-60.9
7	26.5	318.0	25.5	306.0	86.1	0.0	0.0	155.7	279.4	-123.7
8	26.5	318.0	15.5	186.0	141.2	0.0	0.0	155.7	279.4	-123.7
9	26.5	318.0	5.5	66.0	196.3	0.0	0.0	155.7	279.4	-123.7
10	26.5	318.0	0.0	0.0						
11	62.0	744.0	0.0	0.0						
12	62.0	744.0	5.5	66.0	196.3	0.0	0.0	279.4	155.7	123.7
13	62.0	744.0	15.5	186.0	141.2	0.0	0.0	279.4	155.7	123.7
14	62.0	744.0	25.5	306.0	86.1	0.0	0.0	279.4	155.7	123.7
15	62.0	744.0	36.0	432.0	30.5	0.0	0.0	137.6	76.7	60.9
16	88.5	1062.0	36.0	432.0	13.5	0.0	0.0	76.7	0.0	76.7
17	88.5	1062.0	25.5	306.0	46.3	0.0	0.0	155.7	0.0	155.7
18	88.5	1062.0	15.5	186.0	77.8	0.0	0.0	155.7	0.0	155.7
19	88.5	1062.0	5.5	66.0	108.9	0.0	0.0	155.7	0.0	155.7
20	88.5	1062.0	0.0	0.0						

Column and Girder Properties
for the Preliminary Steel Frame Computer Analysis
on the E-W side of Institute Hall

All steel members are A36; Fy=36 ksi; Modulus of elasticity (E)= 29,000 ksi

All section properties were obtained from the LRFD tables

All members, loads, and forces were calculated in appendix D

E-W side Columns of Building						
#	**W Section**	**AX (in^2)**	**IY (in^4)**	**IX (in^4)**	**Length (ft)**	**Pu (Kips)**
1	W 8x24	7.08	18.3	82.8	5.5	110.0
2	W 8x24	7.08	18.3	82.8	10.0	78.0
3	W 8x24	7.08	18.3	82.8	10.0	46.3
4	W 8x24	7.08	18.3	82.8	10.5	14.6
9	W 8x24	7.08	18.3	82.8	5.5	142.0
10	W 8x24	7.08	18.3	82.8	10.0	100.7
11	W 8x24	7.08	18.3	82.8	10.0	60.6
12	W 8x24	7.08	18.3	82.8	10.5	18.4
17	W 8x24	7.08	18.3	82.8	5.5	142.0
18	W 8x24	7.08	18.3	82.8	10.0	100.7
19	W 8x24	7.08	18.3	82.8	10.0	60.6
20	W 8x24	7.08	18.3	82.8	10.5	18.4
25	W 8x24	7.08	18.3	82.8	5.5	110 0
26	W 8x24	7.08	18.3	82.8	10.0	78.0
27	W 8x24	7.08	18.3	82.8	10.0	46.3
28	W 8x24	7.08	18.3	82.8	10.5	14.6

E-W side Girders of Building						**(Wu l ^{2}/12)**
#	**W Section**	**AX (in^2)**	**IX (in^4)**	**Length (ft)**	**Wu (k/ft)**	**Mz (k-ft)**
5	W 18x40	11.8	612	20.5	1.31	45.9
6	W 21x68	20.0	1480	20.5	2.66	93.2
7	W 21x68	20.0	1480	20.5	2.66	93.2
8	W 21x68	20.0	1480	20.5	2.66	93.2
13	W 18x40	11.8	612	6.0	1.31	3.9
14	W 21x68	20.0	1480	6.0	2.66	8.0
15	W 21x68	20.0	1480	6.0	2.66	8.0
16	W 21x68	20.0	1480	6.0	2.66	8.0
21	W 18x40	11.8	612	20.5	1.31	45.9
22	W 21x68	20.0	1480	20.5	2.66	93.2
23	W 21x68	20.0	1480	20.5	2.66	93.2
24	W 21x68	20.0	1480	20.5	2.66	93.2
			Note: Wu =1 2 D +1.6 L			

Joint Data for the Steel Frame Computer Analysis on the E-W side of Institute Hall

Joint #	X-cor. (ft)	X-cor. (in)	Y-cor. (ft)	Y-cor. (in)	Pu (kips)	Wind Fx (k) Per Frame	Earthquake Fx (k) Per Frame	(+) Moment Mz (ft-k)	(-) Moment Mz (ft-k)	Net Moment Mz (ft-k)
1	0.0	0.0	0.0	0.0						
2	0.0	0.0	5.5	66.0	108.9	6.9	2.4	0.0	93.2	-93.2
3	0.0	0.0	15.5	186.0	77.8	8.9	5.0	0.0	93.2	-93.2
4	0.0	0.0	25.5	306.0	46.3	9.2	8.2	0.0	93.2	-93.2
5	0.0	0.0	36.0	432.0	13.5	4.7	8.6	0.0	45.9	-45.9
6	20.5	246.0	36.0	432.0	17.3	0.0	0.0	45.9	3.9	42.0
7	20.5	246.0	25.5	306.0	58.5	0.0	0.0	93.2	8.0	85.2
8	20.5	246.0	15.5	186.0	100.1	0.0	0.0	93.2	8.0	85.2
9	20.5	246.0	5.5	66.0	141.3	0.0	0.0	93.2	8.0	85.2
10	20.5	246.0	0.0	0.0						
11	26.5	318.0	0.0	0.0						
12	26.5	318.0	5.5	66.0	141.3	0.0	0.0	8.0	93.2	-85.2
13	26.5	318.0	15.5	186.0	100.1	0.0	0.0	8.0	93.2	-85.2
14	26.5	318.0	25.5	306.0	58.5	0.0	0.0	8.0	93.2	-85.2
15	26.5	318.0	36.0	432.0	17.3	0.0	0.0	3.9	45.9	-42.0
16	47.0	564.0	36.0	432.0	13.5	0.0	0.0	45.9	0.0	45.9
17	47.0	564.0	25.5	306.0	46.3	0.0	0.0	93.2	0.0	93.2
18	47.0	564.0	15.5	186.0	77.8	0.0	0.0	93.2	0.0	93.2
19	47.0	564.0	5.5	66.0	108.9	0.0	0.0	93.2	0.0	93.2
20	47.0	564.0	0.0	0.0						

5.5.5 Deflection Check for the Steel Frame

The data in the previous section (5.5.4) was entered into ***FRAME.*** The program output gives shear, moments, and deflections at each joint. In order for the previously designed members to satisfy the computer analysis, it was required that the lateral deflection at the roof level be within the following limit :

$$\Delta_{max.}\ \textbf{(range)} = \frac{H}{400} \leftrightarrow \frac{H}{500}$$

where:

H = Height of the building

To prevent any unnacceptable deflections, we chose the lower limit :

$$\frac{H}{500} = \frac{(36' \times 12'')}{500} = \boxed{0.864\ in.}$$

After running the computer analysis for wind and earthquake forces on both the N-S and E-W sides, we found that earthquake governed with maximum deflections of [20]:

$$\Delta\ \textbf{N-S} = \textbf{1.027 in.}$$

$$\Delta\ \textbf{E-W} = \textbf{1.671 in.}$$

Both were greater than $\Delta_{max.} =$ ***0.864 in.*** Therefore we checked the calculations and saw that the problem was the **W 8x24** columns designed for the corner columns since they were only designed for D.L and L.L, but they were critical since they are corner columns directly exposed to wind and earthquake

[20] For a printout of the computer analysis for the Steel Frame see Appendix F.

forces. To solve the problem we increased the Ix (moment of inertia) of these columns to a **W 10x45**, which was the W section designed for the interior columns. Doing this, not only solved the Δ (deflection) problem, but also standardized all columns to a **W 10x45**.

After running the computer analysis for earthquake on the N-S and E-W sides of the building using the new **W 10x45** members for corner and edge columns, we found the following maximum deflections:

Δ **N-S = 0.525 in.**

Δ **E-W = 0.748 in.**

Both were less than $\Delta_{max.}$ = ***0.864 in.*** Therefore, the designed structural members making up the frame were acceptable for deflection. Following is the data, presented in tabular form, which was needed for the deflection correction of columns:

Column and Girder Properties to Correct the Deflection for the Steel Frame on the N-S side of Institute Hall

All steel members are A36; Fy=36 ksi; Modulus of elasticity (E)= 29,000 ksi
All section properties were obtained from the LRFD tables
All members, loads, and forces were calculated in appendix D

N-S side Columns of Building						
#	W Section	AX (in^2)	IY (in^4)	IX (in^4)	Length (ft)	Pu (Kips)
1	W 10x45	13.3	53.4	248	5.5	110.0
2	W 10x45	13.3	53.4	248	10.0	78.0
3	W 10x45	13.3	53.4	248	10.0	46.3
4	W 10x45	13.3	53.4	248	10.5	14.6
9	W 10x45	13.3	53.4	248	5.5	197.0
10	W 10x45	13.3	53.4	248	10.0	141.8
11	W 10x45	13.3	53.4	248	10.0	86.7
12	W 10x45	13.3	53.4	248	10.5	31.6
17	W 10x45	13.3	53.4	248	5.5	197.0
18	W 10x45	13.3	53.4	248	10.0	141.8
19	W 10x45	13.3	53.4	248	10.0	86.7
20	W 10x45	13.3	53.4	248	10.5	31.6
25	W 10x45	13.3	53.4	248	5.5	110.0
26	W 10x45	13.3	53.4	248	10.0	78.0
27	W 10x45	13.3	53.4	248	10.0	46.3
28	W 10x45	13.3	53.4	248	10.5	14.6

N-S side Girders of Building						($Wu\ l^2/12$)
#	W Section	AX (in^2)	IX (in^4)	Length (ft)	Wu (k/ft)	Mz (k-ft)
5	W 18x40	11.8	612	26.5	1.31	76.7
6	W 21x68	20.0	1480	26.5	2.66	155.7
7	W 21x68	20.0	1480	26.5	2.66	155.7
8	W 21x68	20.0	1480	26.5	2.66	155.7
13	W 18x40	11.8	612	35.5	1.31	137.6
14	W 21x68	20.0	1480	35.5	2.66	279.4
15	W 21x68	20.0	1480	35.5	2.66	279.4
16	W 21x68	20.0	1480	35.5	2.66	279.4
21	W 18x40	11.8	612	26.5	1.31	76.7
22	W 21x68	20.0	1480	26.5	2.66	155.7
23	W 21x68	20.0	1480	26.5	2.66	155.7
24	W 21x68	20.0	1480	26.5	2.66	155.7

Note: Wu =1.2 D +1.6 L

Column and Girder Properties to Correct the Deflection for the Steel Frame on the E-W side of Institute Hall

All steel members are A36; Fy=36 ksi; Modulus of elasticity (E)= 29,000 ksi

All section properties were obtained from the LRFD tables

All members, loads, and forces were calculated in appendix D

E-W side Columns of Building

#	W Section	AX (in^2)	IY (in^4)	IX (in^4)	Length (ft)	Pu (Kips)
1	W 10x45	13.3	53.4	248	5.5	110.0
2	W 10x45	13.3	53.4	248	10.0	78.0
3	W 10x45	13.3	53.4	248	10.0	46.3
4	W 10x45	13.3	53.4	248	10.5	14.6
9	W 10x45	13.3	53.4	248	5.5	142.0
10	W 10x45	13.3	53.4	248	10.0	100.7
11	W 10x45	13.3	53.4	248	10.0	60.6
12	W 10x45	13.3	53.4	248	10.5	18.4
17	W 10x45	13.3	53.4	248	5.5	142.0
18	W 10x45	13.3	53.4	248	10.0	100.7
19	W 10x45	13.3	53.4	248	10.0	60.6
20	W 10x45	13.3	53.4	248	10.5	18.4
25	W 10x45	13.3	53.4	248	5.5	110.0
26	W 10x45	13.3	53.4	248	10.0	78.0
27	W 10x45	13.3	53.4	248	10.0	46.3
28	W 10x45	13.3	53.4	248	10.5	14.6

E-W side Girders of Building

#	W Section	AX (in^2)	IX (in^4)	Length (ft)	Wu (k/ft)	($Wu\ l^2/12$) Mz (k-ft)
5	W 18x40	11.8	612	20.5	1.31	45.9
6	W 21x68	20.0	1480	20.5	2.66	93.2
7	W 21x68	20.0	1480	20.5	2.66	93.2
8	W 21x68	20.0	1480	20.5	2.66	93.2
13	W 18x40	11.8	612	6.0	1.31	3.9
14	W 21x68	20.0	1480	6.0	2.66	8.0
15	W 21x68	20.0	1480	6.0	2.66	8.0
16	W 21x68	20.0	1480	6.0	2.66	8.0
21	W 18x40	11.8	612	20.5	1.31	45.9
22	W 21x68	20.0	1480	20.5	2.66	93.2
23	W 21x68	20.0	1480	20.5	2.66	93.2
24	W 21x68	20.0	1480	20.5	2.66	93.2

Note: Wu =1.2 D +1.6 L

5.6 Final Check for Columns using the Interaction Equation

Since we had designed our columns based only on axial loading $\mathbf{P_u}$, we had to check them for the $\mathbf{M_u}$, composed of the superimposed moments $\mathbf{M_{lt}} + \mathbf{M_{nt}}$ obtained from the computer analysis [21].

To perform this check, we took the most critical case which was: using the largest concentrated load @basement level for the interior columns. This $\mathbf{P_u}$= 287 k.

We found that the most critical moment for basement columns from the computer analysis was 393 in-k.

From Chapter H in the LRFD,

if $\dfrac{P_u}{\phi P_n} \geq 0.2$ **then use:** $\dfrac{P_u}{\phi P_n} + \dfrac{8}{9}\left((M_{ux}/\phi_b M_{nx}) + (M_{ux}/\phi_b M_{nx})\right)$

which must be **less than or equal to 1.0.**

After performing the calculations required (see appendix d), we found that the W10x45 was not acceptable:

$\dfrac{P_u}{\phi P_n}$ **was : 0.67 $\geq$ 0.2;** but the value for the interaction equation was **1.55 > 1.0**

Increasing the member to a W 10x68 solved the problem. Checking this member for the interaction equation gave us a .92 less than 1.0. Therefore, a W 10x68 is the final member for all columns.

Following is the data in tabular form needed to perform the computer analysis for this new member:

---

[21] For a printout of the computer analysis for the steel frame see appendix F.

Column and Girder Properties to Correct for the Interaction Equation on the Steel Frame

All steel members are A36; Fy=36 ksi; Modulus of elasticity (E)= 29,000 ksi

All section properties were obtained from the LRFD tables

All members, loads, and forces were calculated in appendix D

E-W side Columns of Building

#	W Section	AX (in^2)	IY (in^4)	IX (in^4)	Length (ft)	Pu (Kips)
1	W 10x68	20.0	134	394	5.5	110.0
2	W 10x68	20.0	134	394	10.0	78.0
3	W 10x68	20.0	134	394	10.0	46.3
4	W 10x68	20.0	134	394	10.5	14.6
9	W 10x68	20.0	134	394	5.5	142.0
10	W 10x68	20.0	134	394	10.0	100.7
11	W 10x68	20.0	134	394	10.0	60.6
12	W 10x68	20.0	134	394	10.5	18.4
17	W 10x68	20.0	134	394	5.5	142.0
18	W 10x68	20.0	134	394	10.0	100.7
19	W 10x68	20.0	134	394	10.0	60.6
20	W 10x68	20.0	134	394	10.5	18.4
25	W 10x68	20.0	134	394	5.5	110.0
26	W 10x68	20.0	134	394	10.0	78.0
27	W 10x68	20.0	134	394	10.0	46.3
28	W 10x68	20.0	134	394	10.5	14.6

E-W side Girders of Building

#	W Section	AX (in^2)	IX (in^4)	Length (ft)	Wu (k/ft)	($Wu\ l^2/12$) Mz (k-ft)
5	W 18x40	11.8	612	20.5	1.31	45.9
6	W 21x68	20.0	1480	20.5	2.66	93.2
7	W 21x68	20.0	1480	20.5	2.66	93.2
8	W 21x68	20.0	1480	20.5	2.66	93.2
13	W 18x40	11.8	612	6.0	1.31	3.9
14	W 21x68	20.0	1480	6.0	2.66	8.0
15	W 21x68	20.0	1480	6.0	2.66	8.0
16	W 21x68	20.0	1480	6.0	2.66	8.0
21	W 18x40	11.8	612	20.5	1.31	45.9
22	W 21x68	20.0	1480	20.5	2.66	93.2
23	W 21x68	20.0	1480	20.5	2.66	93.2
24	W 21x68	20.0	1480	20.5	2.66	93.2

Note: Wu =1.2 D +1.6 L

5.7 Design of Basement's Floor[22]

The basement slab as given in the general specifications on Institute Hall was designed to carry a 100 psf live load for social areas or other recreational facilities. This was done in accordance to the Massachusetts State Code 1990.

According to the WPI General Specifications on Institute Hall

"Slabs on ground shall be placed on a minimum **6" layer of 95% compacted gravel** and placed in alternate panels not exceeding 1000 ft^2.

They shall be reinforced with minimum **6" x 6" -w2.9 x w2.9 welded wire fabric, lapped 12" on sides and ends.**"

According to the Massachusetts Code 1990 (section 1509)

"The **minimum slab thickness** for a slab is 3.5". However, since we are designing a slab for a college dormitory, the live load acting on it is 100 psf (social areas). Therefore, to be safe this slab was designed to be **6.0" thick**, like the slab designed for the corridor areas.

An approved **vapor barrier** with **joints lapped not less than 6"** shall be placed between one base course and the concrete slab."

5.8 Design of Foundations

The foundations for the steel frame were designed by using the Mass Code 1990 and Peck, Hanson, and Thornburn's, Foundation Engineering.[23]

---

[22] see design of basement's slab in appendix D.

[23] Peck, Hanson, and Thornburn. Foundation Engineering, New York: John Wiley and Sons, Inc, 1974 edition.

5.8.1 General Information for Foundations

From Mass Code 1990

SECTION 1205.1 Frost Protection

"All permanent supports of buildings and structures shall extend a minimum of four (4) feet below finished grade except when erected upon sound bedrock..."

SECTION 1206.1 Footing Design

"The loads to be used in computing the pressure upon bearing materials directly underlying foundations shall be the live and dead loads of the structure, as specified in section 1115.0, including the weight of the foundations and any immediately overlying material..."

SECTION 1206.2 Pressure due to Lateral Loads

"Where the pressure on the bearing material due to wind or any other lateral loads is less than one-third (1/3) of that due to dead and live loads, it may be neglected for the foundations design..."

In the case of Institute Hall, wind and earthquake loads are less than 1/3 dead load plus live loads.

SECTION 1209.0 Concrete Footings

"In plain concrete footings, the edge thickness shall be not less than 8 inches for footings on soil..."

TABLE 1201 on
(Allowable bearing pressure for foundation materials)

For type of soil @ Worcester, MA.

Material Class	*Description*	*Consistency*	*Max. allowable Net bearing Pressure (tons/ft^2)*
7	Gravel, widely graded sand & gravel; & abrasion till	Dense-Medium	6

From "Foundations Engineering", **1974 edition.**

Pg.265

"As a general rule, a factor of Safety of 3 should be provided against the loads specified by the building code. It should not be less than 2 even if the type of soil and maximum loads are known exceptionally well."

Note: Since we did not know the type of soil for Institute Hall exceptionally well, we used a factor of safety of 3.

Pg. 114, Table 5.3

For a till, gravel, or sandy type of soil with a medium to dense relative density, the range of number of blows (N) per foot for the penetration test is 30-50.

As an average value, we used N=40.

Pg. 309

In order to follow Peck, Hanson, and Thornburn's design method for footings we used Terzaghi's figure 19.3 pg.309. This figure is a chart proportioning shallow footings. It was designed taking a maximum value settlement of 1".

5.8.2 Design of Footings[24]

All footings must have:

4 feet or more below finished grade (footing depth for Massachusetts)
Thickness ≥ 8 inches
P_u's x 3 safety factor
Average number of blows N=40
Max. differential settlement = 1"

The first step in the footing design was to obtain the $\mathbf{P_u}$**'s** from each particular type of column and multiply them by the S.F of **3**.

Next, we assumed a reasonable footing width **b** for the particular

[24] Foundation calculations for the Steel Frame are in Appendix D.

load acting on it and find the ratio $\frac{D_f}{b}$

where:

D_f = Depth of footing from ground level

Using Terzaghi's chart in figure 19.3, pg.309 on Peck, Hanson, and Thornburn's "Foundation Engineering", we entered this ratio and N=40 to obtain the **Q_{all}.** (allowable bearing strength).

Then this **Q_{all}**., **4.4 TSF** (tons per square foot) in our case, was compared with the **Q_{act}**, which is calculated as:

$$Q_{act} = \frac{P_u}{b^2}$$

If $Q_{all} \geq Q_{act}$, then the foundation is acceptable.

Finally, the thickness of the foundation was estimated as $\frac{b}{4}$.

If **Q_{all}** is less than **Q_{act}** then one must start over again choosing a larger valuefor **b** until requirements were satisfied.

After performing all calculations (see Appendix D), the following footings were obtained:

Location	*Type of foundation*
For corner columns	A plain concrete footing w/ a 6.5' x 6.5' base, thickness = 1.6'
For N-S edge columns	A plain concrete footing w/ a 8.5' x 8.5' base, thickness = 2.0'
For E-W edge columns	A plain concrete combined footing w/ a 10' x 10 base, thickness = 2.5'
For Interior columns	A plain concrete combined footing w/ a 14'x14' base, thickness =3.5'

5.9 Summary of Final & Complete Steel Design

The following tables summarize all the final members and construction materials developed during the steel structural design of Institute Hall:

Construction Materials for the Steel Design of Institute Hall

Section	*Building Materials*	*Description*
ROOF		
	Concrete Slab	**3.5"** thick, reinforced w /**66-44** Mesh
		(rolled in the direction of deck span)
	Corrugated Steel Deck	1.5" depth, Inverted "B" deck, galv.
		16 Gauge, span:**16.5'**, width:**6.75'**
		5/8" puddle welds spaced approx.
		12" on center.
	3 ply felt & gravel	For insulation & to protect slab from rain
ROOMS AREA		
(floors1,2,3)	Concrete Slab	**4.0"** thick, reinforced w /**66-44** Mesh
		(rolled in the direction of deck span)
	Corrugated Steel Deck	1.5" depth, Inverted "B" deck, galv.
		16 Gauge, span:**16.5'**, width:**6.75'**
		5/8" puddle welds spaced approx.
		12" on center.
CORRIDORS AREA		
(floors 1,2,3)	Concrete Slab	**4.5"** thick, reinforced w /**44-44** Mesh
		(rolled in the direction of deck span)
	Corrugated Steel Deck	1.5" depth, Inverted "B" deck, galv.
		16 Gauge, span:**16.5'**, width:**6.0'**
		5/8" puddle welds spaced approx.
		12" on center.
ELEVATORS	Concrete Slab	**4.5"** thick, reinforced w /**44-44** Mesh
WAITING AREA		(rolled in the direction of deck span)
	Corrugated Steel Deck	1.5" depth, Inverted "B" deck, galv.
		16 Gauge, span:**7'**, width:3.75'
		5/8" puddle welds spaced approx.
		12" on center.

BASEMENT	Concrete Slab	**6.0"** thick, reinforced w/ 6" x 6"-w2.9 x w2.9
		welded wire fabric, lapped 12" on ends.
	Vapor Barrier	lapped not less than **6"**
	Compacted Gravel	**6"** 95 % compacted gravel
EXTERIOR	Concrete Blocks	**8"** Hollow concrete block w/ heavy aggregate
CORRIDOR &		
ELEVATOR BOX		
WALLS		
FOUNDATIONS	For corners and	A plain concrete footing w/ a **6.5' x 6.5'** base,
(all bases reinforced	elevator waiting area	thickness = **1.6"**; and with a **20" x 20"** & **2.4'**
w/ 6"x6"-w2.9xw2.9	columns	deep vertical section supporting the column.
welded wire fabric		
lapped 12" on ends)	For N-S intermediate	A plain concrete footing w/ a **8.5' x 8.5'** base,
	columns	thickness = **2.0'**; and with a **20" x 20"** & **2.0'**
		deep vertical section supporting the column.
	For E-W edge/corridor	A plain concrete combined footing w/ a
	columns	**10' x 10'** base, thickness = **2.5'**; and with two
		20" x 20" & **1.5'** deep vertical sections
		supporting the columns.
	For interior columns	A plain concrete combined footing w/ a
		14' x 14' base, thickness = **3.5'**; and with two
		20" x 20" & **1.5'** deep vertical sections
		supporting the columns.
	For Stairways columns	A plain concrete combined footing w/ a
		9.5' x 9.5' base, thickness = **2.5'**; supporting
		all four stairway columns.
	For Elevator shaft columns	A plain concrete combined footing w/ a
		9.5' x 9.5' base, thickness = **2.5'**; supporting
		all four elevator shaft columns.

	Design of Members for the Main Section of the Building			
Member	*Type*	*Location*	*Length(ft)*	*Quantity*
W 18x40	Girder	Roof	35.5	4
W 18x40	Girder	Roof	26.5	8
W 18x40	Girder	Roof	20.5	8
W 18x40	Girder	Roof	6.0	4
W 21x68	Girder	3rd, 2nd, & 1st Floors	35.5	12
W 21x68	Girder	3rd, 2nd, & 1st Floors	26.5	24
W 21x68	Girder	3rd, 2nd, & 1st Floors	20.5	24
W 21x68	Girder	3rd, 2nd, & 1st Floors	6.0	12
W 10x22	Beam	Roof, 3rd, 2nd, & 1st Floors	20.5	56
W 6x9	Beam	Roof, 3rd, 2nd, & 1st Floors	6.0	28
W 10x68	Interior column	Roof	10.5	4
W 10x68	Interior column	3rd, & 2nd Floors	10.0	8
W 10x68	Interior column	1st Floor	7.5	4
W 10x68	N-S columns	Roof	10.5	4
W 10x68	N-S columns	3rd, & 2nd Floors	10.0	8
W 10x68	N-S columns	1st Floor	7.5	4
W 10x68	E-W & Corner columns	Roof	10.5	8
W 10x68	E-W & Corner columns	3rd, & 2nd Floors	10.0	16
W 10x68	E-W & Corner columns	1st Floor	7.5	8
W 8x24	Stairway columns	3rd, & 2nd Floors	10.0	16
W 8x24	Stairway columns	1st Floor	7.5	8
W 12x14	Stairway beams	3rd,2nd, & 1st Floors	9.0	12
W 12x14	Stairway beams	3rd,2nd, & 1st Floors	7.5	12
	Design of Members for the Elevator Section of the Building			
Member	*Type*	*Location (below)*	*Length(ft)*	*Quantity*
W 18x40	Girder	Roof	11.50	1
W 18x40	Girder	Roof	7.00	4
W 18x40	Girder	Roof	6.75	2
W 21x68	Girder	3rd, 2nd, & 1st Floors	7.00	12
W 21x68	Girder	3rd, 2nd, & 1st Floors	6.75	6
W 10x68	Elevator waiting area	Roof	10.5	1
W 10x68	Elevator waiting area	3rd, & 2nd Floors	10.0	3
W 10x68	Elevator waiting area	1st Floor	7.5	1
W 8x24	Elevator shaft columns	Roof	10.5	4
W 8x24	Elevator shaft columns	3rd, & 2nd Floors	10.00	8
W 8x24	Elevator shaft columns	1st Floor	7.50	4

CHAPTER 6

Reinforced Concrete Design

6. Reinforced Concrete Design

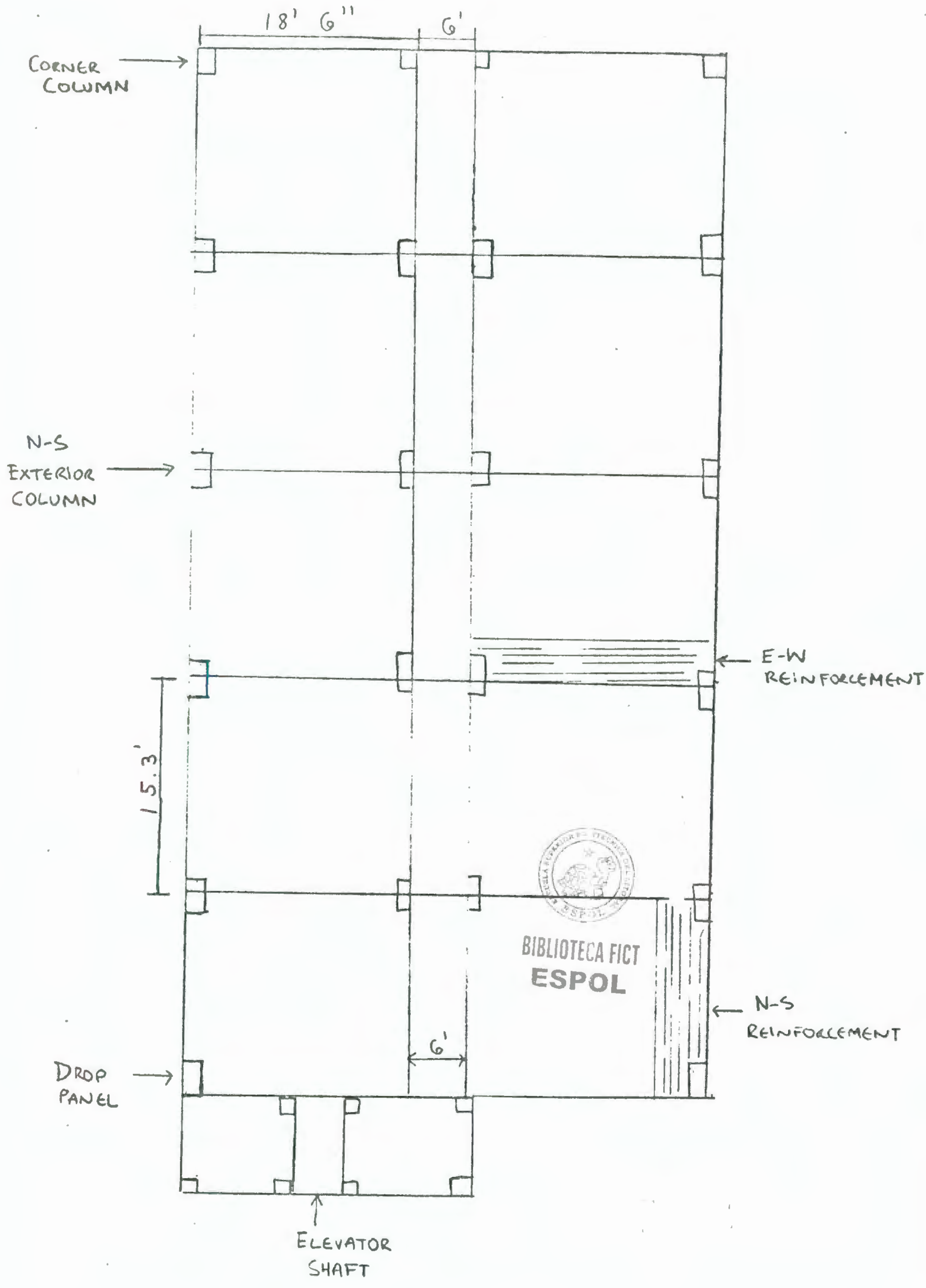
Reinforced concrete has been used in many construction projects throughout this century. Before the advent of pre-cast concrete , and the mass fabrication of steel members, it was the building material and method of choice. In many designs that do not require a, " fast-track" schedule of construction, it is still used today.

One reason for the use of reinforced concrete is the ease in which concrete structures can be built; if the design fits all of the specified codes, you need only pour it into your desired forms, and let it cure. This is unlike steel structures, which have to be designed with **pre-shaped** members. Therefore concrete can be utilized more easily as a building material.

While being very strong under **compression**, concrete is weaker under **tension**, and in the past this made steel a safer choice for structures undergoing large lateral loads, (**i.e. earthquake, wind**). Today with the advent of reinforced concrete, in which concrete members are reinforced with **steel bars** of varying diameters, we can construct concrete buildings to withstand substantial lateral loading.

The following design shows the method used in the design of the reinforced concrete members. The building we chose to re-design was Institute Hall, its plans and dimensions are given in **Appendix E**, and the floor plan is shown in **figure 6.1**. It is hoped that the following method will help in the

FIGURE 6.1



18' 6''
6'
CORNER COLUMN
N-S EXTERIOR COLUMN
E-W REINFORCEMENT
15.3'
N-S REINFORCEMENT
6'
DROP PANEL
ELEVATOR SHAFT

understanding of reinforced concrete design, and an appreciation of its usefulness.

6.1. Preliminaries to the Design of the Structural Members

The first order of operations in the design of a structure, is to pick the type of structure you wish to design. For example should it be a building made with columns, girders, and beams? Should it have a one way or two way slab?

Our structure is a three story dormitory, with relatively light live loads. There were a number of different ways we could of designed the dormitory, after comparing a few different methods (i.e., two way slab with beams, one way slab),

we decided to design with a **frame** composed of **columns**, and **two way slabs**.

We decided not to use beams in the design of the frame. We concluded that they would not be necessary to hold up the structure, the reason for this is due to the small bay sizes of the structure. As can be seen in **figure 6.1**, the size of the room bays, (area between corridors ,and walls) is not very large. The maximum bay area for a room is only **18.5 by 15.3 feet** which is relatively small, and could be held up with only columns.

The problem with having no beams is often the absence of **lateral support**, but this is not a problem when using a two way slab. When we say two way slab we mean a slab reinforced in 2 directions, in the **North-South**, and **East-West** directions. It is this reinforcement which allows the slab to withstand the lateral stresses.

How is this accomplished? When **lateral loads** act on A building they create **moments** in the frame. These moments cause both **compressive** , and **tensile forces** to act on the slab; therefore something is pushing, and pulling on the concrete at the same time. The steel reinforcement is what deals with the tensile stresses, and because it is 2-way reinforcement it can deal with stresses coming from a number of directions. At the same time the concrete can deal with the compressive forces acting on it.

Another way to look at it is to divide the slab into **strips**, see figure 6.3. Now consider each strip acting as a beam, if we were to take several reinforced beams, and put them together; we would have a strong member. This is exactly what a reinforced concrete slab is, and because we have strips going in 2 directions, it is like having two sets of beams. One going in the N-S direction, the other going in the E-W direction. Now you can begin to see the strength of this design, and why beams are not necessary.

Therefore we chose to design the frame using only columns, and a 2 way slab. In order to decrease the chances of fracture due to moments and shear, we fitted the columns with drop panels. Which are a small extension of the columns' thickness at the top and bottom.

6.1.1. Distribution and Layout of Structural Members

In order to comprehensively design the structure thus mentioned, we must have a precise measurement of the size of the building, and its needed members. Through study of the original building plans, the floor plan shown in figure 6.1 was contrived.

The first measurement needed, was the distance between the columns; because no beams are to be used, it was decided that the distance should be shorter than a normal beam span length. This would help reduce any damage due to **tension** or **flexure** strains.

A distance of 15.3 ft was picked for the span length between the columns in the East - West direction. The span lengths in the North South direction were complicated by the existence of the buildings main corridor. We decided on putting a column on each side of the corridor, with a 6 foot span in between, as shown in figure 6.1. Each of these columns would then be 18.5 ft from the exterior columns.

6.2. Roof Design

The Method Used to design the Roof Slab is known as the Direct Design Method. The requirements for this method are given below[1]

(a) Minimum of three consecutive spans each way- therefore OK.

(b) Longer span/shorter span ≤ 2: $20.0/17.5 < 2$ - therefore OK.

(c) Successive span lengths differ by not more than one-third of the longer span. Thus short span/long span $\geq .667$: $17.75/20 = .89$ - therefore OK.

(d) Columns offset up to 10% - OK.

[1] Reinforced Concrete, Mechanics & Design second edition, pg581.

(e) All loads are uniformly distributed gravity loads. Strictly speaking, the wall load is not uniformly distributed, but use DDM.

(f) Unfactored live load not greater than three times the unfactored dead load. Using Table A - 24, estimate the slab thickness as L/36 = 20 x 12/36 $\cong$ 6.5 in. The approximate dead load = 6.5/12 x 150 + 25 = 106psf. This exceeds one-third of the live load - therefore OK.

(g) No beams - therefore, ACI Sec. 13.6.1.6 does not apply and it is not necessary to make this check.

Therefore use the Direct Design Method.

The first operation done in the Direct Design Method is to calculate the slab thickness. We start by finding the largest span between columns. This is 18.5 ft, or 222 inches, then by using the equation;

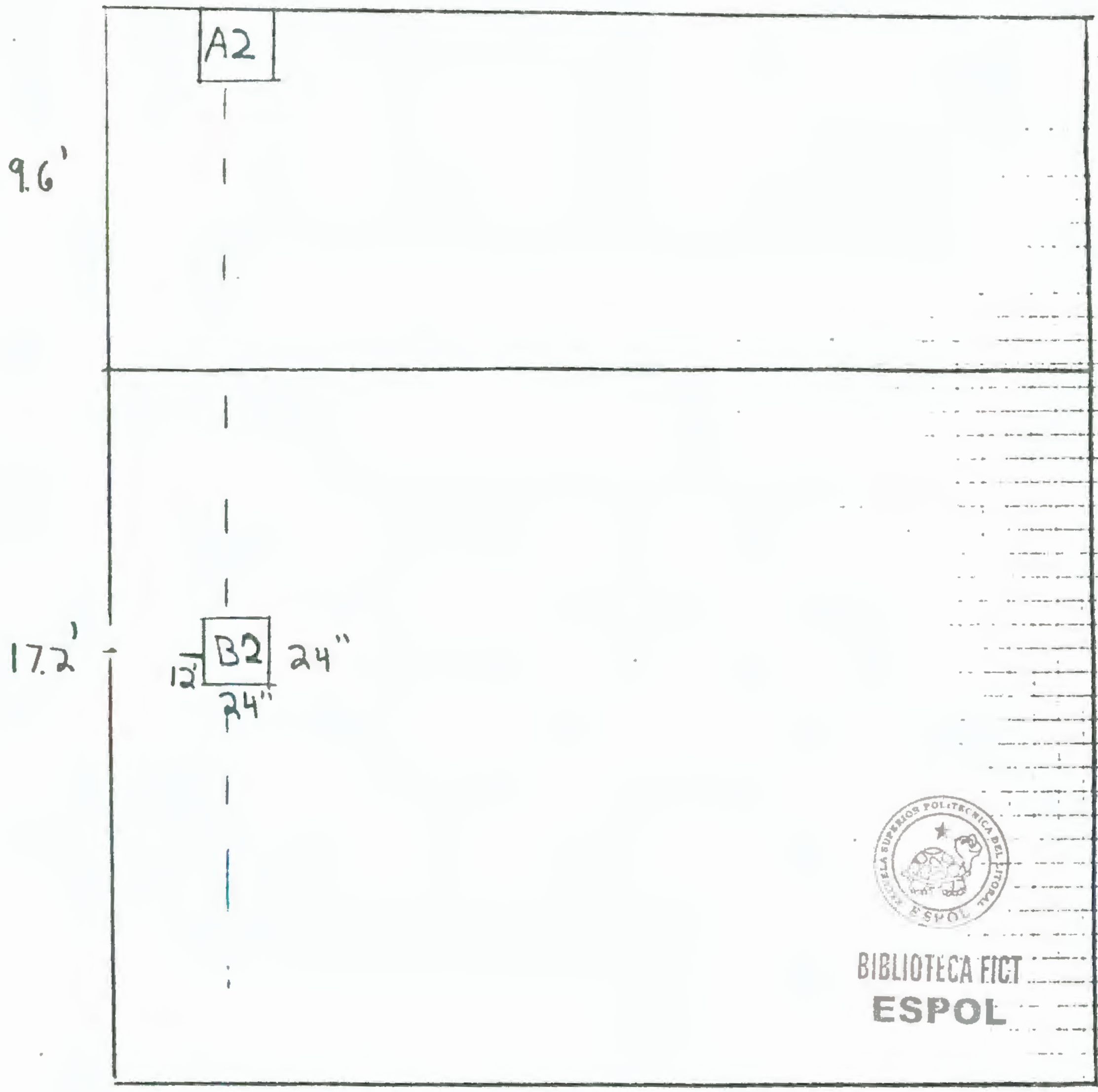
$$\textbf{min h:} = L_n/33$$

we get a slab with a thickness of 6 75 inches.

6.2.1. Check thickness for shear:

To check the slab for shear, first we must calculate the loads that affect it, for the roof this will be its roofing, insulation, and suspended lighting. This comes to a dead load of 10 lbs/ft^2. This must then be added to the roof dead load to calculate its total dead load. A slab of 6.75 inches will have a calculated dead load of 84.38 lbs/ft^2, to this we add the roof fixture's dead load, and calculate a total dead load of 94.38 lbs/ft^2.

FIGURE 6.2. Calculation of Constant



For Column B2.

b_o = Perimeter of critical section
$= 2(24+24) = 96$ in

d = effective depth => assume 5 in

β_c = ratio of long side to short side of Column
$= 2/2 = 1$

According to ACI equation.(9-2) the Total Factored Load is calculated by the equation:

$$W_u = 1.4DL + 1.7SL$$

which gives us a load of 192 psf.

This factored load is then used to check for shear using several ACI equations. Along with the factored load certain constants pertaining to the column design must also be found; these are b_o, B_c, and d. These constants are shown, and explained in figure 6.2.

The first equation used shows us the amount of shear that the slab will be subjected to at a given column, it is;

$$V_u = W_u \left[\left(l_n + l_b / 2 \right) \times l_c - \text{area column} \right]$$

This result is then compared to three other equations which give the amount of shear that the member can withstand at that given column. The three equations are

$$Vc = .85 \left(2 + 4/ B_c \right) \sqrt{f_c'} b_o d$$

$$Vc = .85 \left(A_s d/b_o + 2 \right) \sqrt{f_c'} b_o d$$

$$Vc = .85 \sqrt{f_c'}\, b_o d$$

if V_u is larger than the largest of the V_c, than the slab is adequate. For all our columns, interior, and exterior, our slab is adequate.

6.2.2. Calculation of negative and positive moments:

The first step in calculating the moments in the slab, is to break up the slab into strips, (see figure 6.3). This is done by cutting the area between columns in half, and then cutting that area in half again. We then analyze each strip to find its maximum negative, and positive moments.

In order to calculate the moments affecting the strip, we use the following equation:

$$Mo = W_u l_2 l_n^2/8$$

Now the basic moment is factored. This is done by multiplying Mo by three factors, -0.26,0.52, and -0.70. The factored moments are then distributed throughout the strip; the negative moments at the strips edges , and the positive moments to the center.

Each strip must now be designed for reinforcement. The strips are drawn in two directions, one in the East - West , the other in the North - South direction; as shown in figure 6.3. In the figure only a part of the floor plan is shown, this is because the floor is symmetrical, and the moments calculated for these strips will be the same as those located elsewhere in the slab. We can interchange the values with other symmetrical strips.

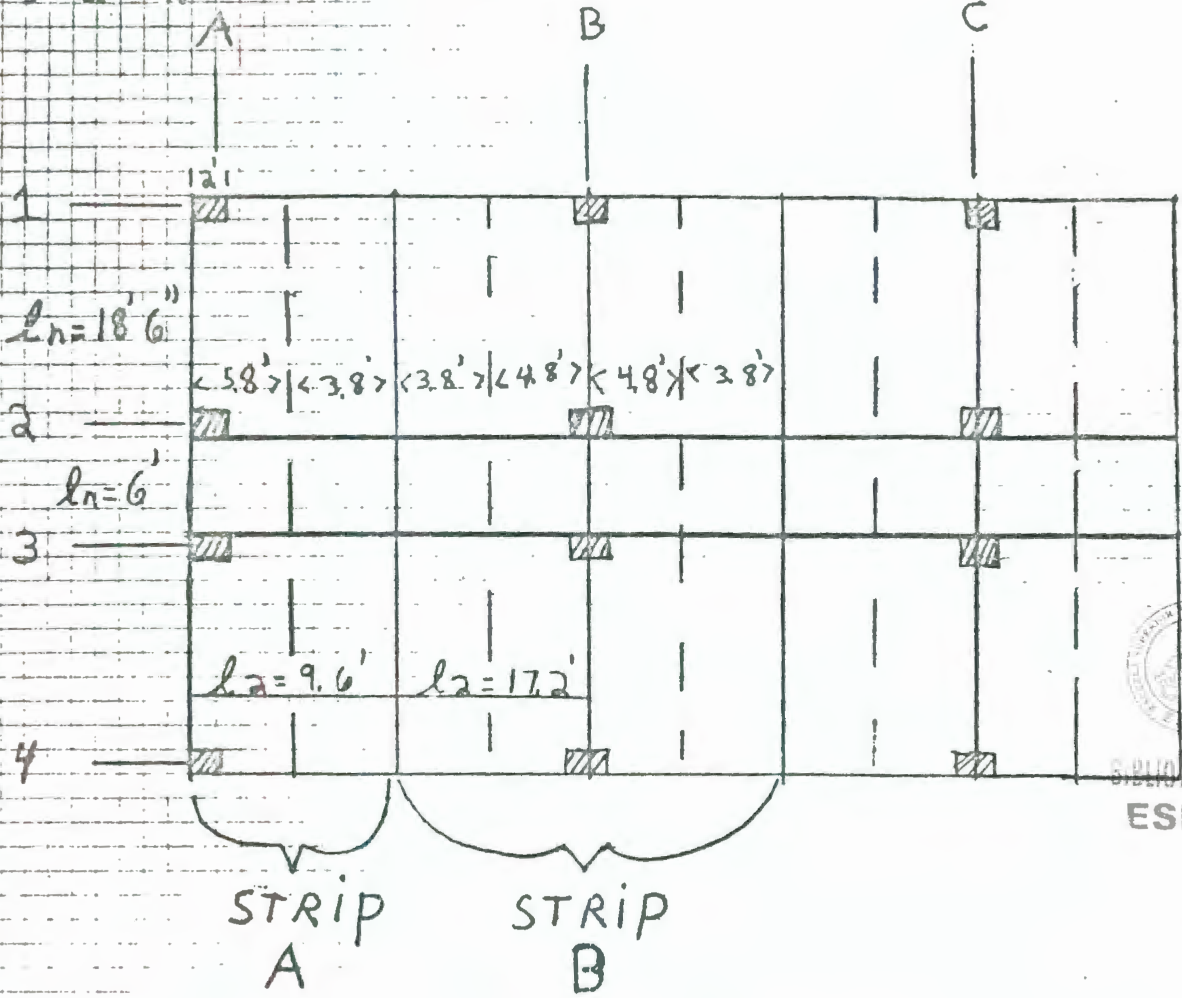
The spans between the columns are now labeled, and we may begin our moment calculations. Table 6.1 gives an example of the moment calculations for slab strip B. After the moments have been calculated, we then go about the

Table 6.1
Calculation of negative and positive moments in slab B.

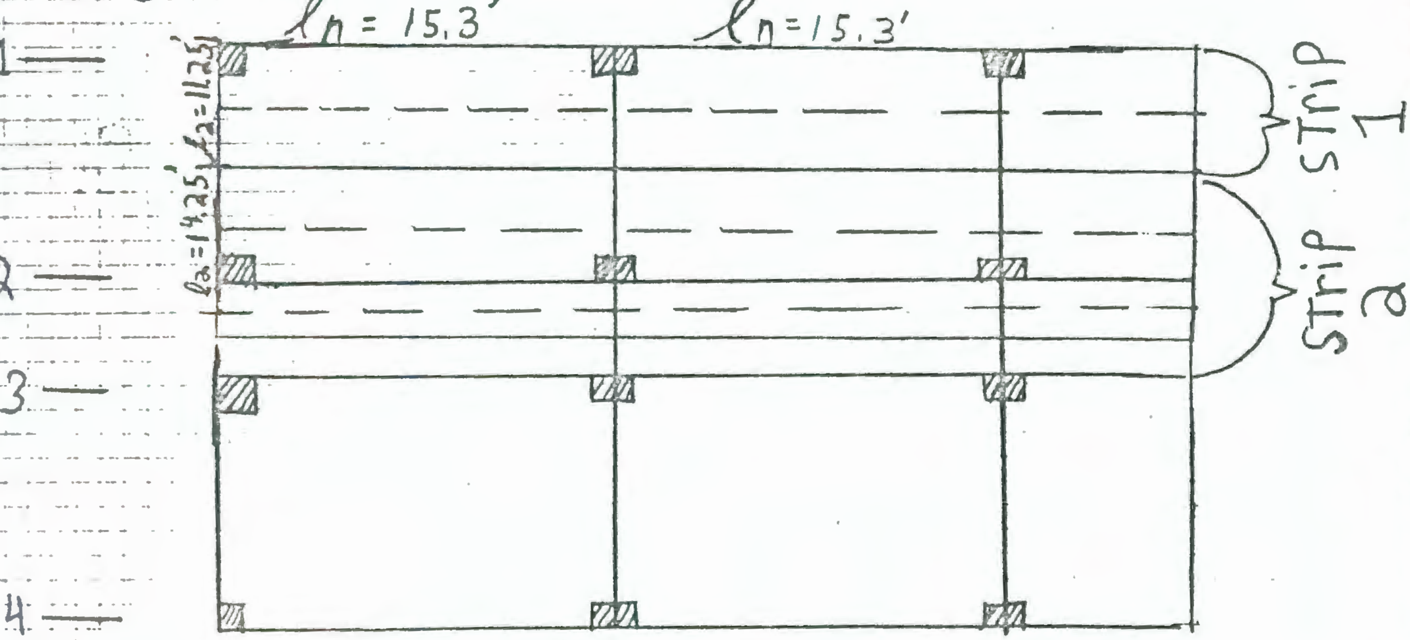
	B1			B2			B3			B4
l1		21.5'			7'			21.5'		
ln		18.6'			6'			18.6'		
l2		17.2'			17.2'			17.2'		
wu		0.192			,192			0.192		
mo = wul2ln^2/8		141.2			14.8			141.2		
mo coefficients	-0.26	0.52	-0.7	-0.65	0.35	-0.65	-0.7	0.52	-0.26	
moments	-36.7	73.4	-98.8	-9.62	5.18	-9.62	-98.8	73.4	-36.7	
sum of column moments	36.7			11			11			

FIGURE 6.3 (E-W; N-S STRIPS)

a) E-W



b) N-S



process of distributing them across the slab strips, and designing the reinforcement.

6.2.3. Design of Slab Reinforcement

The preliminary step in the design of the reinforcement is to check for deflection and calculate the necessary constants. The first constant that must be found is d, the distance between reinforcement. If we assume a 1 inch diameter bar for a 6.75 in slab, then we come to a d of 5.94 inches (see appendix).

Next we check the Ratio of Depth of Rectangular Stress Block for Balanced Failure to Effective depth, other wise known as, **a/d**. If the ratio that we calculate is greater than the ratio given in Table A-5[2], then we must re-check our design.

We begin by finding the maximum moment that our slab undergoes. Assuming j to equal .925, we use the following equation:

$$A_s(req'd) = M_o / .9 \times f_y \times j \times d$$

Now we compute a , and a/d, a is found by

$$a = A_s\, f_y / .85\, f'_c\, b$$

which comes out to be .28 Dividing a by **d** we get **.05**. Using these results to look up values for ρ, and $.75\rho_b$ in Table A-5 of the reinforced concrete text, we find $.75\,\rho_b$ =.437.

This is less than the a/d we calculated, so we may continue with our design.

[2] Reinforced Concrete, Mechanics and Design
2nd Edition, pf 799

TABLE 6.2
REINFORCEMENT
DESIGN

FIRST INTERIOR NEGATIVE MOMENTS	EDGE COLUMN STRIP	MIDDLE STRIP		COLUMN STRIPS		MIDDLE STRIP		EDGE COLUMN STRIP
	B4			B3	B2			B1
SLAB MOMENTS	-27.6			-35	-35			-27.6
MOMENT COFFICIENTS	0.75	0.25	0.125	0.75	0.75	0.125	0.25	0.75
MOMENT TO COLUMNS AND MIDDLE STRIPS;	-20.7	-6.9	-4.4	-26.25	-26.25	-4.4	-6.9	-20.7
WALL MOMENT								
TOTAL MOMENT	-20.7		-11.3	-26.25	-26.25		-11.3	-20.7
AS REQUIRED	1.28		0.7	1.63	1.63		0.7	1.28
MIN AS	1.07		1.8	1.3	1.3		1.8	1.07
CHOOSE STEEL	7 # 4 BARS		9 # 4 BARS	9 # 4 BARS	9 # 4 BARS		9 # 4 BARS	7 # 4 BARS
AS PROVIDED	1.4		1.8	1.8	1.8		1.8	1.4
INTERIOR POSITIVE MOMENTS								
SLAB MOMENT	14.9			18.8	18.8			14.9
MOMENT COEFFICIENTS	0.6	0.4	0.2	0.6	0.6	0.2	0.4	0.6
MOMENT TO COLUMN AND MIDDLE STRIPS;	8.94	5.96	3.76	11.28	11.28	3.76	5.96	8.94
WALL MOMENTS								
TOTAL MOMENT	8.94		9.72	11.28	11.28		9.72	8.94
AS REPUIRED	0.55		0.6	0.7	0.7		0.6	0.55
MIN AS	1.07		1.8	1.3	1.3		1.8	1.07
CHOOSE STEEL	6 # 4 BARS		9 # 4 BARS	7 # 4 BARS	7 # 4 BARS		9 # 4 BARS	6 # 4 BARS
AS PROVIDED	1.2		1.8	1.4	1.4		1.8	1.2

The last step before beginning our actual design is to calculate the constants used in our design equations, our first equation is;

$$A_s = M_u / .9\, f_y\, jd$$

$$jd = d - a/2$$

the equation used for calculating the area of steel reinforcement is $A_s = .062M_u$. The equation which gives us the minimum area of steel need not be calculated, it is given in Sec 13.4.2 of the ACI codes as; A_s **min = .002bh.**

With the preliminaries completed we can now set up tables to design the reinforcement in the east, and west directions. Table 6.2 is an example of one of these tables, and shows the process used. It should be noted that the tables used to calculate the steel bars come from the ACI codes, but were taken from the Reinforced Concrete Textbook.

6.3. *Floor Designs*

The process used to design the roof, is the exact same process used to design the floors. The only difference is in the loading. Unlike the roof there is a substantial effect caused by live load and by wall loads.

The equation for factored loading in the case of the floor is;

$$W_u = 1.4\, DL + 1.7\, LL$$

which brings us to our first problem, we have two different live loads. The live load for the rooms is 40 lbs/ft^2, while the live load for the corridors is 100 lbs/ft^2, (in case the corridors get jammed with people during an emergency). So two different factored loads must now be calculated, and two separate checks must be done for shear.

During the moment calculation phase the corridors factored loads must be taken into consideration, as well as the loads caused by the walls. In Table 6.3 we give an example of how this is done, and the effect of these loads on the moment calculations. It should be stated that I use a dead load of .42 ksf for the wall load, instead of .62 ksf used in the Steel Design Chapter.

I did this, because at first I was using the the Reinforced Concrete textbook to help me with my design, and it only used a value of .42 ksf for its walls. Later on we referred to the LRFD Steel Design Manual[3], which gave a value of .62 ksf for the walls we were using, (see cost analysis for comprehensive description of the walls, and interior design). We decided that for future calculations we would use the .62 ksf load, but we kept the slab design that had utilized the .42 ksf load.

The reinforcement design is done with the same procedure used for the roof design, the only difference being the addition of the wall moments. **Table 6.3** is an example of how the moments are distributed with the addition of wall moments.

6.4. *Column Design*

6.4.1. Preliminary Calculations:

[3] Smith, John, Structural Steel Design: LRFD Approach, 1991 edition,

Design of Columns : Basement level

Calculate Tributary Area : Find worst case PU.

a) Edge columns, Exterior columns (N-S)

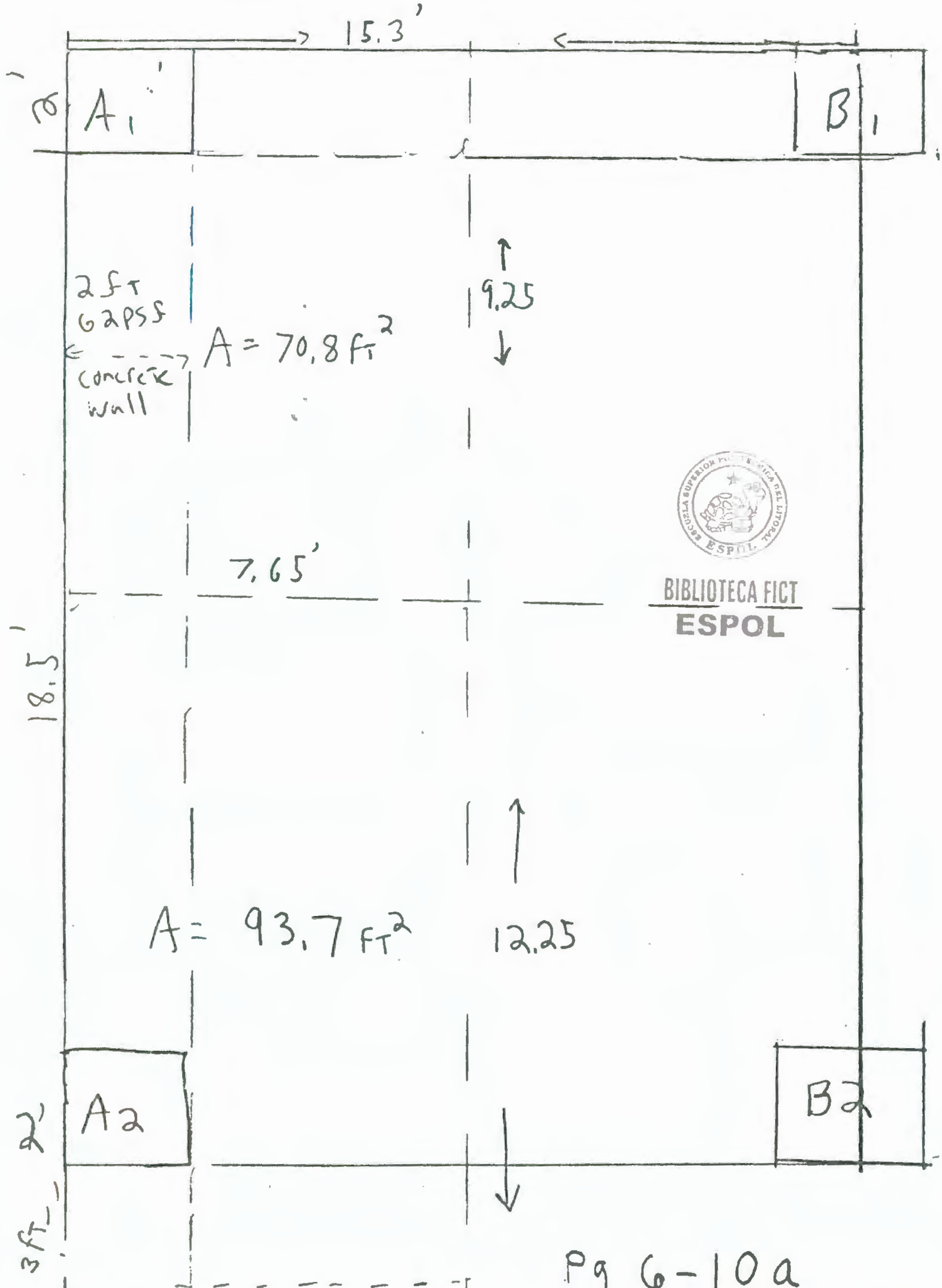


FIGURE 6.3
CALCULATION OF POSITIVE AND NEGATIVE MOMENTS FOR STRIP A

	A1			A2			A3			A4
L1		15.5			4			15.5		
LN		18.5			6			18.5		
L2		9.6			9.6			9.6		
Wu		0.192			0.294			0.192		
Mo		78.9			12.7			78.9		
Mo coefficients	-0.26	0.52	-0.7	-0.65	0.35	-0.65	-0.7	-0.52	-0.26	
Positive and Negative moments	-20.5	41	-55.2	-8.3	4.45	-8.3	-20.5	41	-55.2	
wall load		0.42			0.42			0.42		
wall Mo		18			2			18		
Moments from wall	-4.68	9.36	-12.6	-1.3	0.7	-1.3	-12.6	9.3 6	-4.68	
sum of column moments	20.5			37			37			55.2

When dealing with a building the size of ours, dead and live load govern the design of the columns. This is the case because the building lacks, considerable height, and bending moments due not play a significant part in design. This does not mean that the building is not checked for deflection due to wind, and seismic load ! It is, and later on in the text the result of this frame analysis for deflection will be shown.

To find the dead load affecting each column, we must first calculate its **Tributary Area**. This is the amount of the floor span that each column supports. Figure 6.4 shows how the tributary area was broken up for each column. It should be stated that in analyzing tributary area I took into account both the geometry of the span, and the distribution of loads. After finding the tributary area, we multiply the area by the factored load, $\mathbf{W_u}$. This will give us the concentrated load on the column, or the $\mathbf{P_u}$. The following is the summary of the P_u calculations.

Columns	**P_u (kips)**
Exterior Corridor Columns	102
Interior Corridor Columns	179.1
N-S Exterior Columns	131.5
Corner Columns	78

6.4.2. Design Method:

We decided to design the columns from the bottom up, in other words, to find the concentrated loads on the basement columns, design for those loads, and then use those columns for the rest of the building. This is designing for the worst case, since the **worst case columns** in our building,(due to **gravity load**) are the basement columns.

The first step in the design process is to calculate the trial area of the column, $\mathbf{A_g}$. This is given by the equation:

$$\mathbf{A_g > P_u/.45(f'_c + f_y\rho_t)}$$

After the A_g is calculated the preliminary shape, and dimensions of the column may be calculated. Now we have to check our preliminary design.

First we calculate the property **e**, which is the ratio of the column moment to the concentrated load; M_u/P_u. After e is calculated it is divided by h. If the ratio is .2 or greater the bar arrangement is rectangular, and the column is a square. If the ratio is less than .2, then the reinforcement is a spiral, and the column is circular. In our case all our columns are square.

Our next step is to choose the reinforcement for the column. We start by calculating the area of steel needed, given by the equation:

$$\mathbf{A_{st} = \rho_t A_g}$$

We assume ρt to equal .01, then using our values for Ag, we calculate the area of the steel. Using the tables given in the reinforced concrete handbook, we choose the type, and number of reinforcing bars; now our design is complete. An example, and summary of the column design is given in **Table 6.4.**

TABLE 6.4
DESIGN OF BASEMENT COLUMNS

	A1	A2	B1	B2
Pu	78	102	131.5	179.1
Mu	20.5	8.3	36.7	14.8
Ag(trial)	37	49	62.6	86.5
select dimensions	8x8	8x8	12x12	12x12
e = Mu/Pu	0.26	0.082	0.28	0.082
e/h	78	0.082	0.28	0.082
Y	0.75	0.75	0.75	0.75
Pu/Ag	0.52	0.7	0.9	1,2
Mu/Agh	0.144	0.06	0.252	0.1032
pt	0.01	0.01	0.01	0.01
Ast = ptAg	1.44	1.44	1.44	1.44
choose steel	4 # 6 bars	4 # 6 bars	4 # 6 bars	4 # 6 bars
As provided	1.76	1.76	1.76	1.76

6.5. Lateral load check

Up to this point we have done all of our designs with gravity loads governing (ie. dead, and live load). Now we must check if our members, and frame are capable of resisting the lateral loads placed on them, (ie. wind, and seismic load).

To check for lateral loading we chose to use a program called Frame. This FORTRAN based program calculates all of the joint moments in the frame, as well as the deflection at each joint. We then can use this information to check our frame, but first some preliminary steps must be taken.

6.5.1. Member Data

Before the FORTRAN program may be run, we must first enter some input. We must find the moment of inertia of each column and beam, its effective area, and its edge moments. An immediate problem is that we have no beams; this is easily remedied by considering the column strips, used in the slab design, as beams. **Tables 6.4 - 6.8** summarize the values for all of our members, and **Figure 6.5** demonstrates how the joints, and members are categorized.

6.5.2. FRAME © : Structural Analysis Computer Program

FRAME[4] is a two dimensional (plane) structural analysis program is composed of program running files, a data input file, and an output file.

The first step was to enter all the members and joints data for the frame to be analyzed by using a sample input data file from the program called "plfrm.dat." To access this file one must go into **DOS** © and at the prompt type :

"edit plfrm.dat"

Once on this file, one makes all the necessary changes for one's particular frames' materials, members, and joints data. When all the changes are made, save it under a different name such as "**example.dat**".

Then one must run the file: "**frame**". This file will ask for the input file; in this case "**example.dat**", and for an output file such as "**example.out**".

Finally when the message "**stop-Program Terminated**" appears on the screen, one can go over and analyze the computer analysis of the frame by typing the following at the prompt command:

"edit example.out"

If one is satisfied with the frame analysis one can print it by typing:

"print example.dat"

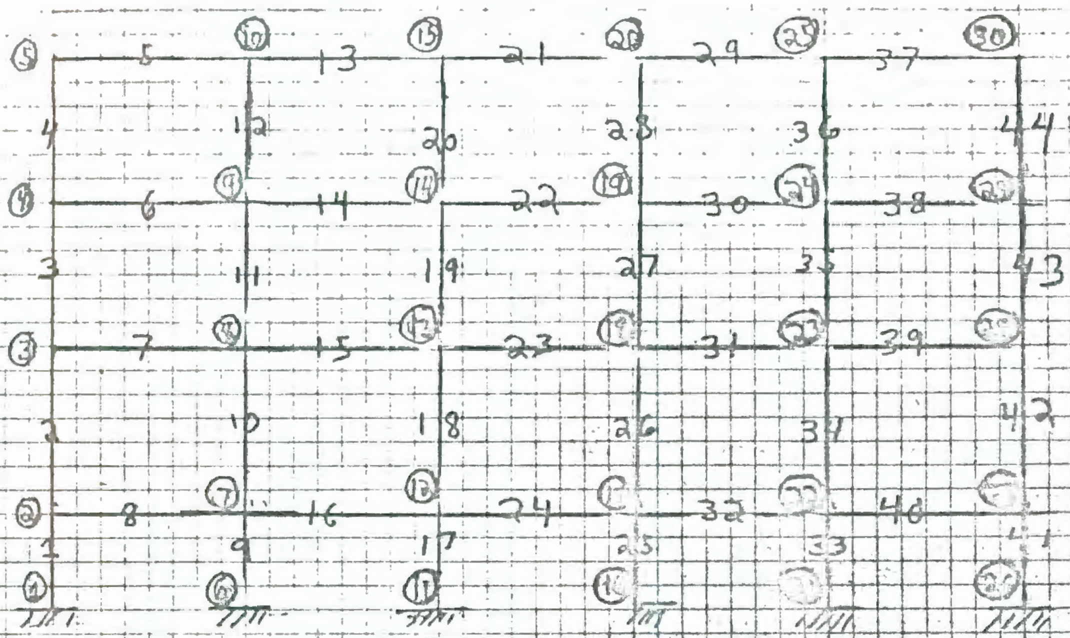
6.5.3. Required Data for the Preliminary Computer Analysis

The computer analysis[5] require to number all the joints and members in the N-S and E-W frames and then enter them with all their required data in tabular form:

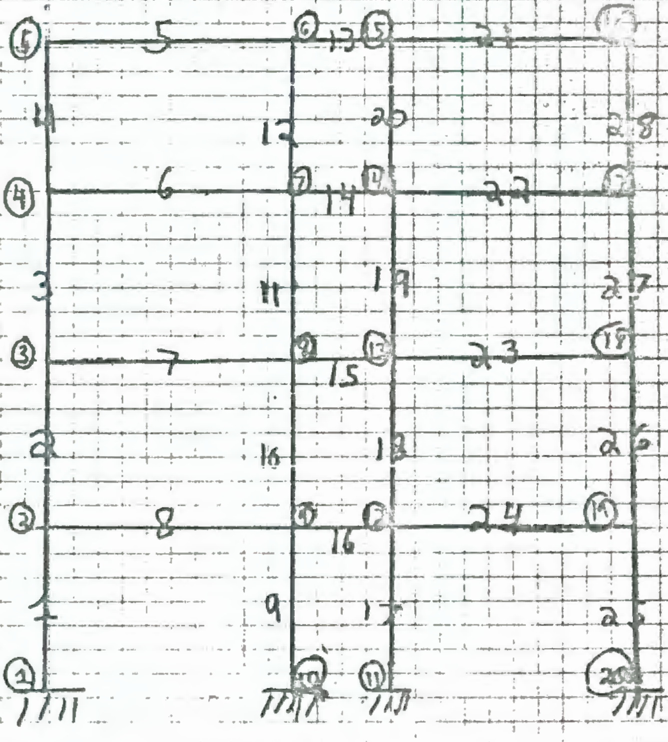
---

[4] Program <u>*FRAME: Stiffness Analysis of Plane Frames.*</u>© 1988, by John F.Fleming.

FIGURE 6-5



NORTH SOUTH



East — WEST

Pg 6-13a

Column and Girder Properties for the Preliminary
Table 6.5
Reinforced Concrete Computer Analysis
on the N-S side of institute Hall

N-S side columns of Building					
#	Section	Ax (in^2)	Iz(in^4)	Length (ft)	Pu (kips)
1	8x8	64	3840	7.5	78
2	8x8	64	5120	10	55.2
3	8x8	64	5120	10	34.4
4	8x8	64	5376	10.5	13.6
9	12x12	144	12960	7.5	131.5
10	12x12	144	17280	10	95.4
11	12x12	144	17280	10	61.2
12	12x12	144	18144	10.5	27
17	12x12	144	12960	7.5	131.5
18	12x12	144	17280	10	95.4
19	12x12	144	17280	10	61.2
20	12x12	144	18144	10.5	27
25	12x12	144	12960	7.5	131.5
26	12x12	144	17280	10	95.4
27	12x12	144	17280	10	61.2
28	12x12	144	18144	10.5	27
33	12x12	144	12960	7.5	131.5
34	12x12	144	17280	10	95.4
35	12x12	144	17280	10	61.2
36	12x12	144	18144	10.5	27
41	8x8	64	3840	7.5	78
42	8x8	64	5120	10	55.2
43	8x8	64	5120	10	34.4
44	8x8	64	5376	10.5	13.6

Table 6.5 Continued

	N-S side Sections of Building					
#	Section	Ax (in^2)	Iz (in^4)	ength (f	Wu (k/ft)	Mz (k-ft)
5	slab strip 6.75"	336	2952.5	15.3	0.192	4
6	slab strip 6.75"	336	2952.5	15.3	0.254	5
7	slab strip 6.75"	336	2952.5	15.3	0.254	5
8	slab strip 6.75"	336	2952.5	15.3	0.254	5
13	slab strip 6.75"	336	2952.5	15.3	0.192	4
14	slab strip 6.75"	336	2952.5	15.3	0.254	5
15	slab strip 6.75"	336	2952.5	15.3	0.254	5
16	slab strip 6.75"	336	2952.5	15.3	0.254	5
21	slab strip 6.75"	336	2952.5	15.3	0.192	4
22	slab strip 6.75"	336	2952.5	15.3	0.254	5
23	slab strip 6.75"	336	2952.5	15.3	0.254	5
24	slab strip 6.75"	336	2952.5	15.3	0.254	5
29	slab strip 6.75"	336	2952.5	15.3	0.192	5
30	slab strip 6.75"	336	2952.5	15.3	0.254	5
31	slab strip 6.75"	336	2952.5	15.3	0.254	5
32	slab strip 6.75"	336	2952.5	15.3	0.254	5
37	slab strip 6.75"	336	2952.5	15.3	0.192	4
38	slab strip 6.75"	336	2952.5	15.3	0.254	5
39	slab strip 6.75"	336	2952.5	15.3	0.254	5
40	slab strip 6.75"	336	2952.5	15.3	0.254	5

Table 6.6
Column and Girder Properties for the Preliminary
Reinforced Concrete Computer Analysis
on the E-W side of Institute Hall

	East - West side Columns of Building					
#	Section	Ax (in^2)	Iz (in^4)	Length (ft)	Pu (kips)	
1	8x8	64	3840	7.5	78	
2	8x8	64	5120	10	55.2	
3	8x8	64	5120	10	34.4	
4	8x8	64	5376	10.5	13.6	
9	8x8	64	3840	7.5	102	
10	8x8	64	5120	10	74.4	
11	8x8	64	5120	10	46.8	
12	8x8	64	5376	10.5	18	
17	8x8	64	3840	7.5	102	
18	8x8	64	5120	10	74.4	
19	8x8	64	5120	10	46.8	
20	8x8	64	5376	10.5	18	
25	8x8	64	3840	7.5	78	
26	8x8	64	5120	10	55.2	
27	8x8	64	5120	10	34.4	
28	8x8	64	5376	10.5	13.6	
	E-W	Girders	of building			
	#	Section	Ax (in^2)	Iz (in^4)	Length (ft)	Wu (k/ft)
	5	slab section 6.75"	387	3460	18.5	0.192
	6	slab section 6.75"	387	3460	18.5	0.192
	7	slab section 6.75"	387	3460	18.5	0.192
	8	slab section 6.75"	387	3460	18.5	0.192
	13	slab section 6.75"	387	1845.3	6	0.294
	14	slab section 6.75"	387	1845.3	6	0.294
	15	slab section 6.75"	387	1845.3	6	0.294
	16	slab section 6.75"	387	1845.3	6	0.294
	21	slab section 6.75"	387	3460	18.5	0.192
	22	slab section 6.75"	387	3460	18.5	0.192
	23	slab section 6.75"	387	3460	18.5	0.192
	24	slab section 6.75"	387	3460	18.5	0.192

Table 6.7 North-South
lateral moment distribution

Joint	x cor. (ft)	x cor.(in)	y cor. (ft)	y cor.(in)	Pu (k)	Wind Fx (k)	Earthquake Fx (k)	Mu(ft-k)
1	0	0	0	0	85.2	0	0	0
2	0	0	7.5	90	78	3	3.64	-5
3	0	0	17.5	210	55.2	3.8	7.54	-5
4	0	0	27.5	330	34.4	3.9	12.4	-5
5	15.3	183.6	38	456	13.6	2	13.1	-4
6	15.3	183.6	0	0	138.7	0	0	0
7	15.3	183.6	7.5	90	131.5	0	0	0
8	15.3	183.6	17.5	210	95.4	0	0	0
9	15.3	183.6	27.5	330	61.2	0	0	0
10	15.3	183.6	38	456	27	0	0	0
11	30.6	367.2	0	0	138.7	0	0	0
12	30.6	367.2	7.5	90	131.5	0	0	0
13	30.6	367.2	17.5	210	95.4	0	0	0
14	30.6	367.2	27.5	330	61.2	0	0	0
15	30.6	367.2	38	456	27	0	0	0
16	45.9	550.8	0	0	138.7	0	0	0
17	45.9	550.8	7.5	90	131.5	0	0	0
18	45.9	550.8	17.5	210	95.4	0	0	0
19	45.9	550.8	27.5	330	61.2	0	0	0
20	45.9	550.8	38	456	27	0	0	0
21	61.2	734.4	0	0	138.7	0	0	0
22	61.2	734.4	7.5	90	131.5	0	0	0
23	61.2	734.4	17.5	210	95.4	0	0	0
24	61.2	734.4	27.5	330	61.2	0	0	0
25	61.2	734.4	38	456	27	0	0	0
26	76.5	918	0	0	85.2	0	0	0
27	76.5	918	7.5	90	78	0	0	5
28	76.5	918	17.5	210	55.2	0	0	5
29	76.5	918	27.5	330	34.4	0	0	5
30	76.5	918	38	456	13.6	0	0	4

Table 6.8
East - West
Lateral moment distribution

Joints	x cor.(ft)	x cor.(in)	y cor.(ft)	x cor.(in)	Pu (k)	Wind Fx (k)	Earthquake Fx (k)	Mu (ft-k)
1	0	0	0	0	85.2	0	0	0
2	0	0	7.5	90	78	6.93	4.4	-5.5
3	0	0	17.5	210	55.2	8.93	9.2	-5.5
4	0	0	27.5	330	34.4	9.17	15.2	-5.5
5	0	0	38	456	13.6	4.69	16	-5.5
6	18.5	222	0	0	85.2	0	0	4.62
7	18.5	222	7.5	90	78	0	0	4.62
8	18.5	222	17.5	210	55.2	0	0	4.62
9	18.5	222	27.5	330	34.4	0	0	4.62
10	18.5	222	38	456	13.6	0	0	-4.9
11	24.5	294	0	0	85.2	0	0	-4.62
12	24.5	294	7.5	90	78	0	0	-4.62
13	24.5	294	17.5	210	55.2	0	0	-4.62
14	24.5	294	27.5	330	34.4	0	0	-4.62
15	24.5	294	38	456	13.6	0	0	4.9
16	43	516	0	0	85.2	0	0	0
17	43	516	7.5	90	78	0	0	5.5
18	43	516	17.5	210	55.2	0	0	5.5
19	43	516	27.5	330	34.4	0	0	5.5
20	43	516	38	456	13.6	0	0	5.5

6.5.4. Preliminary Wind and Seismic Computer Analysis For the Reinforced Concrete Frame

Now we expose our frame to a computer analysis for wind and seismic forces[6] to check if the frame is acceptable under the given circumstances. We did four computer runs to verify the frame:

Run 1. N-S wind analysis.

Run 2. N-S earthquake analysis.

Run 3. E-W wind analysis.

Run 4. E-W earthquake analysis.

Note: Besides wind or earthquake forces, each run had the following vertical loads: for slab strips (**1.4 DL + 1.7 L.L**), and for columns the concentrated load $\mathbf{P_u}$.

It is here that a major problem occurred. We could not get the N-S analysis to work. We went over the program several times, checking to make sure no mistake had been done. None came to our attention. The only theory we had was that since the program had successfully analyzed all the sides composed of 4 frames, but not the one composed of six frames, that the program could not analyze structures composed of more than 4 frames. This is only a theory though, and we leave this problem for a future MQP to solve. So for our case we will take the E-W deflections to govern, just as they did for the Steel Frame.

---

[5] For a printout of the computer analysis on the Concrete Frame see Appendix G.
[6] see Wind and Seismic calculations on chapter 4.

6.5.5. Deflection Check for the Reinforced Concrete Frame

In order for the previously designed members to satisfy the computer analysis, it is required that the lateral deflection at the roof level be within the following limit :

$$\Delta_{max.}\ (\text{range}) = \frac{H}{400} \leftrightarrow \frac{H}{500}$$

where:

H = Height of the building

This range is between

$$: \frac{H}{500} = \frac{(36' \times 12")}{500} = \mathbf{\mathit{0.864\ in.}}$$

$$: \frac{H}{400} = \frac{(36' \times 12")}{400} = \mathbf{\mathit{1.08\ in.}}$$

After running the computer analysis for wind and earthquake forces on the E-W side, we found that the earthquake load governed with maximum deflection of [7]:

$$\Delta\ \text{E-W} = .86316\ \text{in.}$$

Which is within the range of Δ_{max} = **0.864 - 1.08 in.** Therefore, the designed structural members making up the frame are acceptable for deflection.

---

[7] For a printout of the computer analysis for the Reinforced Concrete Frame see Appendix G

6.5.6. Beam-Column Analysis

The Frame has checked for both gravity, and lateral loads; now we must check it for the combined effect of these two loads.

The first step is to pick the column you wish to check. It is wise to pick the worst case column, because if it checks, then all of the columns will check. In our case this is a basement exterior corridor column. Now we can calculate our needed data, and check our member.

Step 1) Calculate ρ_t ; and ε_y

$$\rho_t = A_{st}/A_g \quad , \varepsilon_y = f_y/E_s$$

Step 3) Compute the concentric axial load capacity, and Max axial load capacity.

a) from Eq 11-1 (Nominal Concentric load)

$$P_o = (.85fc')(A_g - A_{st}) = f_y(A_{st})$$

b) max allowable load

$$\Phi P_n = .8\phi P_o \text{ , where } \phi = .70$$

Step 4) Compute ΦP_n, and ΦM_n, for balanced failure ($\varepsilon_{s1} = -\varepsilon_y$)

a) Determine c, and the strains in reinforcement

(Eq 11- 6) $c = [.003/ .003 - (-\varepsilon_y)]\, d_1$

(Eq 11 - 7) $\varepsilon_{s2} = (c - d_2/c).003$

b) Compute the stresses in the reinforcement layers.

(Eq 11-8) $fs_2 = \varepsilon_{s2}E_s$ but $-f_y \leq f_{s2} \leq f_y$

c) Compute the forces in the concrete and steel.

$$(\text{Eq } 11 - 9) \quad C_c = (.85f'_c)(ab)$$

If the distance d_1(distance between reinforcement) exceeds a. Then the layer of steel lies outside the compression stress block, and does not displace concrete included in the area (ab) when computing C_c. Thus

$$F_{s1} = f_{s1}A_{s1}$$

If a exceeds d_2 then reinforcement layer 2 lies in the compression zone. Hence we must allow for the stress in the concrete displaced by the steel when we compute f_{s2}. From equation 11-10b,

$$F_{s2} = (f_{s2} - .85f'_c)A_{s2}$$

e) Compute P_n

$$P_n = C_c + \Sigma f_{si}$$

f) Compute M_n

$$M_n = C_c(h/2 - a/2) + F_{s1}(h/2 - d_1) + F_{s2}(h/2 - d_2)$$

P_n = 145.4 kips , and M_n = 102.8 ft-kips is the final result. These are the maximum combined loads our column can withstand. The actual combined load on the column is P_u = 102 kips, and M_z = 52.3 ft - kips. Therefore the member checks, and because this is the worst case member all the members check.

6.6. Design of Basement's Floor[8]

The basement slab as given in the general specifications on Institute Hall was designed to carry a 100 psf live load for social areas or other recreational facilities. This was done in accordance to the Massachusetts State Code 1990.

<u>According to the General Specifications on Institute Hall</u>

"Slabs on ground shall be placed on a minimum **6" layer of 95% compacted gravel** and placed in alternate panels not exceeding 1000 ft^2.

They shall be reinforced with minimum **6" x 6" -w2.9 x w2.9 welded wire fabric, lapped 12" on sides and ends.**"

<u>According to the Massachusetts Code 1990</u> (section 1509)

"The **minimum slab thickness** for a slab is 3.5". However, since we are designing a slab for a college dormitory, the live load acting on it is 100 psf (social areas). Therefore, to be safe this slab was designed to be **6.0" thick**, like the slab designed for the corridor areas.

An approved **vapor barrier** with **joints lapped not less than 6"** shall be placed between one base course and the concrete slab."

6.7. Design of Foundations

The foundations for the reinforced concrete frame were designed by using the Mass Code 1990 and Peck's, Hanson's, and Thornburn's, <u>Foundation</u>

[8] see design of basement's slab in appendix D.

Engineering[9], the textbook used in Worcester Polytechnic Institute's foundation engineering course.

6.7.1. General Information for Foundations

From Mass Code 1990

SECTION 1205.1 Frost Protection

"All permanent supports of buildings and structures shall extend a minimum of four (4) feet below finished grade except when erected upon sound bedrock..."

SECTION 1206.1 Footing Design

"The loads to be used in computing the pressure upon bearing materials directly underlying foundations shall be the live and dead loads of the structure, as specified in section 1115.0, including the weight of the foundations and any immediately overlying material..."

SECTION 1206.2 Pressure due to Lateral Loads

"Where the pressure on the bearing material due to wind or any other lateral loads is less than one-third (1/3) of that due to dead and live loads, it may be neglected for the foundations design..."

In the case of Institute Hall, wind and earthquake loads are less than 1/3 dead load plus live loads.

SECTION 1209.0 Concrete Footings

"In plain concrete footings, the edge thickness shall be not less than 8 inches for footings on soil..."

TABLE 1201 on

---

9 Hanson, Peck, and Thornburn. Foundation Engineering, New York: John Wiley and Sons, Inc, 1974 edition.

(Allowable bearing pressure for foundation materials)

For type of soil @ Worcester, MA.

TABLE 1201

Material Class	*Description*	*Consistency*	*Max. allowable Net bearing Pressure (tons/ft^2)*
7	Gravel; widely graded sand & gravel; & abrasion till	Dense-Medium	6

<u>From "Foundations Engineering</u>", 1974 edition.

Pg.265

"As a general rule, a <u>factor of Safety of 3</u> should be provided against the loads specified by the building code. It should not be less than 2 even if the type of soil and maximum loads are known exceptionally well."

Note: Since we did not know the type of soil for Institute Hall exceptionally well, we used a factor of safety of 3 for $\mathbf{P_u}$**'s**.

Pg. 114, Table 5.3

For a till, gravel, or sandy type of soil with a medium to dense relative density, the range of number of blows (N) per foot for the penetration test is 30-50.

As an average value, we used <u>N=40.</u>

Pg. 309

In order to follow Peck's, Hanson's, and Thornburn's design method for footings we used Terzaghi's figure 19.3 pg.309. This figure is a chart proportioning shallow footings. It was designed taking a <u>maximum value settlement of 1".</u>

6.7.2. Design of Footings[10]

<u>All footings must have:</u>

[10] Foundation calculations for the Steel Frame are in Appendix D.

4 feet or more below finished grade (footing depth for Massachusetts)
Thickness $\geq$ 8 inches
P_u's x 3 safety factor
Average number of blows N=40
Max. differential settlement = 1"

The first step in the footing design was to obtain the **P_u's** from each particular type of column and multiply them by the S.F of **3**.

Next, we assumed a reasonable footing width **b** for the particular load acting on it and find the ratio $\frac{D_f}{b}$

where:

D_f = Depth of footing from ground level

Using Terzaghi's chart in figure 19.3, pg.309 on "Foundation Engineering", we entered this ratio and N=40 to obtain the Q_{all}. (allowable bearing strength).

Then this Q_{all}., **4.4 TSF** (tons per square foot) in our case, was compared with the Q_{act}, which is calculated a

$$Q_{act} = \frac{P_u}{b^2}$$

If $Q_{all} \geq Q_{act}$, then the foundation is acceptable.

Finally, the thickness of the foundation was estimated as $\frac{b}{4}$.

If Q_{all} is less than Q_{act} then one must start over again choosing a larger value for **b** until requirements were satisfied.

After performing all calculations (see Appendix D), the following footings were obtained:

Location	*Type of foundation*
Corner Columns	A plain concrete footing with a 6'x6' base, thickness
Footing for Edge Columns	A plain concrete footing with a 7' x 7' base, thickness = 1.75'
Footing for Interior Columns	A plain concrete combined footing with a 11.5' x 11.5' base, thickness = 3/0'

6.8. *Drop Panel Design*

6.8.1. Design Method

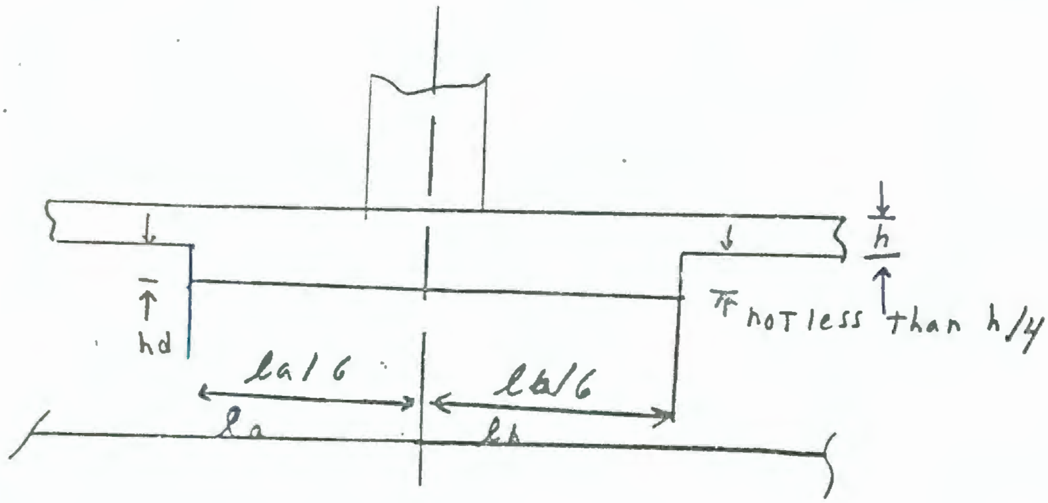
Figure 6.6 illustrates the minimum design requirements specified by **ACI sections 9.5.3.2, and 13.4.7**. The drop panels in this building were designed using this method. It was decided that the drop panels would be uniform for all columns, both to save on cost, and for added support. The panels were designed for the worst case columns, which are the interior columns.

6.9. *Summary of Final & Complete Reinforced Concrete Design*

The following, Table 6.9 ,summarizes the Reinforced Concrete Frame Design.

Drop Panel design: FIGURE 6.6

min size of drop panels (ACI sec. 9.5.3.2 and 13.4.7)



a) Design of Drop panel for column B2

East - West direction dimension

$l_a = l_b = 7.65'$ (use distance from column to edge of Tributary Area)

$l_a/6 = 7.65'/6 = 1.3'$

North south direction dimension

$l_a = l_b = 9.25'$

$l_a/6 = 9.25'/6 = 1.54'$

Sketch:

$h = 2.0''$

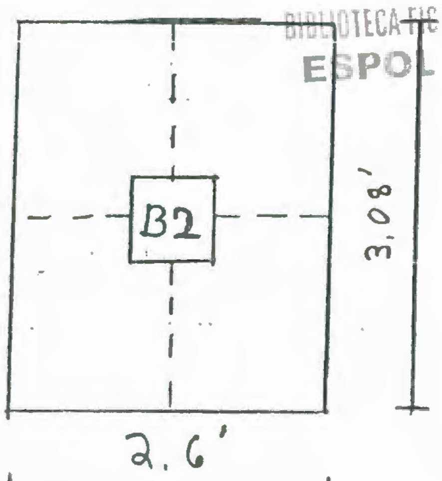


Table 6.9
Summary of Reinforced Concrete Design

Structural Members	Members	Discription
Roof:		
	6.75 inch reinforced concrete **slab**	reinforced in **N-S**, and **E-W** directions with no. 4,5, and 6 bars see figures 6.6, and 6.7 for reinforcement design
	3 ply felt and gravel roofing	
1st,2nd, and 3rd Floors Design:	**6.75** inch reinforced concrete **slab**	reinforced in **N-S**, and **E-W** directions withe no. 4,5,6, and 7 bars. see figures 6.4, and 6.5 for reinforcement design
	5 corner columns	**8x8**,reinforced with 4 no.6 bars
	2 exterior corrdidor columns	**8x8**,reinforced with 4 no.6 bars
	10 interior corridor columns	**12x12**, reinforced with 4 no.6 bars
	9 N-S exterior columns	**12x12**, reinforced with 4 no.6 bars
Elevator Shaft: roof	**6.75** in reinforced concrete **slab**	
	3 ply felt and gravel roofing	
columns	4 columns	**8x8**,reinforced with 4 no.6 bars
Basement:	**6.0** inch **slab**	
	5 corner columns	**8x8**,reinforced with 4 no.6 bars
	2 exterior corrdidor columns	**8x8**,reinforced with 4 no.6 bars
	10 interior corridor columns	**12x12**, reinforced with 4 no.6 bars
	9 N-S exterior columns	**12x12**, reinforced with 4 no.6 bars

Table 6.9

Summary of Reinforced Concrete Design

	4 elevator shaft columns	8x8,reinforced with 4 no.6 bars

CHAPTER 7

Cost Estimating

Chapter 7 Cost Estimating

This chapter in our MQP is dedicated to the cost estimating aspect of the re-design of Institute Hall. In it, we will present how the estimate was assembled from start to finish, including all quantity take-off, pricing from the Means Construction Cost Data 1994[1], and a final cost analysis of both designs. Furthermore, we will depict some of the relevant advantages and disadvantages that each design offers to the owner, such as, fire protection, savings, construction schedules, and so on. As for heat loss in the building, the wall components were made the same to eliminate differences.[2] However, this does not eliminate differences in heat loss through the slabs.

Chapter 7 . 1 Steel Design

One of the two design methods for the structure of Institute Hall is a steel frame. In previous chapters loading and design parameters have been covered. In this section of chapter 7, we will cover all the quantity take-off procedures that allowed the estimate to be constructed. Also, we will include the prices used in each estimate, including a description of the items used in construction. Then, the cost estimate will be performed activity per activity, floor by floor, bottom to top. Some assumptions have been made in order to achieve the different stages of the estimate and they will be thoroughly described in the following sections.

---

[1] Means Construction Costs Data 1994, R.S. Means Publishers

[2] Both designs specified a 2' exterior wall left to the discretion of the estimator. The walls chosen for the project are 8" concrete blocks with insulated cores and a drywall system as an inner wall.

Chapter 7 . 1 . 1 Quantity Take-Off

Often considered the backbone of the estimate, it is vital for the quantities involved to be as accurate as possible to maintain the reliability of the estimate. After all, we are price quoting something which is unique in its nature and has not been constructed yet. Therefore, it involves a large amount of risk, which can be considerably reduced through an accurate cost estimate.

The quantities were taken out of the design drawings one level of construction at a time. First, the foundations were calculated, followed by the basement level, floors levels, and finally the roof level. Some simplifications in the design stage allowed for the quantities to be somewhat uniform from level to level. For example, the first and second floors are identical making their quantities equal. The third floor's quantities only vary where height is involved because there is a differential of .5' from the other floors.[3] This is the case for both designs.

For each level, only relevant quantities were calculated, meaning that quantities were calculated once and then repeated when needed in the estimate. These quantities were excavation volumes, surface contact areas, wall areas, floor areas, member surface areas for fireproofing, and so on.

Some assumptions were made during quantity takeoff, as well as, suggestions[4] which were left to the discretion of the estimator. Here is a list of the assumptions and suggestions that were made.

1. Foundations Level:

[3] First and Second floor heights are 10 feet, whereas the third floor is 10.5 feet in height.
[4] Suggestions include the exterior walls, concrete curing methods, finishes, concrete block types, insulators, and others.

- All dimensions for the footings were increased one foot for the excavation quantities, to allow for forming of the footings. For example, a 6x6 footing was taken as 7x7 to calculate the amount of excavation.
- Waste material was taken as the difference between the volume of excavated material and the volume of backfill material, in other words, the amount of waste is equal to the amount of concrete in the footings.
- Suggested forms for all formwork are to be 4 use forms.
- A 40% allowance was included for excavation shoring.
- A 10% allowance was used for compaction of backfill
- A 25% swell factor allowance was used for disposal of material.
- An 11% waste factor was used for concrete waste in footings and an 18% factor for the reinforcement.

2. <u>Basement Level:</u>

- Edge forming for the slab was taken as an edge form running along the perimeter of the building.
- The floor slab was specified as 6" deep.
- A 6% waste of concrete allowance was used for the slab on this level and an 14% allowance for waste and overlapping of the reinforcing wire mesh.
- A 3% connection allowance was added to the total quantity of each W shaped member.

3. <u>All levels:</u>

- The floor slabs were specified as 3.5" deep in the bedrooms and 4.5" deep in other areas.
- The wall area was taken as the perimeter of the floor multiplied by the height of the floor.
- In the case of metal edge forms, they were calculated as one around the perimeter of each panel.

- Fireproofing specified a 2 hour minimum retarding on all structural members and on the roof panels only. A sprayed mineral or cementitious fiber was selected and its quantities were taken as areas represented by the surface perimeter of the W shapes times their respective lengths.
- A 6% waste of concrete allowance was used for the slab on this level and an 14% allowance for waste and overlapping of the reinforcing wire mesh.
- A 3% connection allowance was added to the total quantity of each W shaped member.

Chapter 7 . 1 . 1 . 1 Quantities for the Building Foundations

Excavation	No. of Footings	Length (FT)	Width (FT)	Depth (FT)	Shoring	Volume (CY)	Total (CY)
Corner Columns	6	7.50	7.50	4.00	20.0	8.33	70.0
Edge Columns (N-S)	4	9.50	9.50	4.00	21.4	13.37	74.9
Edge Columns (E-W)	4	11.00	11.00	4.00	28.7	17.93	100.4
Interior Columns	2	15.00	15.00	5.00	33.3	41.67	116.7
Elevator Columns	1	10.50	10.50	4.00	6.5	16.33	22.9
Waiting Area Columns	1	7.50	7.50	4.00	3.3	8.33	11.7
Totals	**18**	**61.00**	**61.00**	**25.00**	**113.3**	**105.96**	**396.5**

Concrete for Footings	No. of Footings	Length (FT)	Width (FT)	Depth (FT)	Waste	Volume (CY)	Total (CY)
Corner Columns	6	6.50	6.50	1.60	1.7	2.50	16.7
Edge Columns (N-S)	4	8.50	8.50	2.00	2.4	5.35	23.8
Edge Columns (E-W)	4	10.00	10.00	2.50	4.1	9.26	41.1
Interior Columns	2	14.00	14.00	3.50	5.6	25.41	56.4
Elevator Columns	1	9.50	9.50	2.50	0.9	8.36	9.3
Waiting Area Columns	1	6.50	6.50	1.60	0.3	2.50	2.8
Totals	**18**	**55.00**	**55.00**	**13.70**	**14.9**	**53.38**	**150.01**
Concrete for Column Rests	**No. of Rests**	**Length (FT)**	**Width (FT)**	**Depth (FT)**	**Waste**	**Volume (CY)**	**Total (CY)**
Corner Columns	6	1.67	1.67	2.40	0.2	0.25	1.6
Edge Columns (N-S)	4	1.67	1.67	2.00	0.1	0.21	0.9
Edge Columns (E-W)	4	1.67	1.67	1.50	0.1	0.15	0.7
Interior Columns	4	1.67	1.67	1.50	0.1	0.15	0.7
Elevator Columns	4	1.67	1.67	1.50	0.1	0.15	0.7
Waiting Area Columns	1	1.67	1.67	2.40	0.0	0.25	0.3
Totals	**23**	**10.00**	**10.00**	**11.30**	**0.5**	**1.16**	**4.89**

Backfill Material	No. of Footings	Excavation (CY)	Footings (CY)	Column Rests (CY)	Sub-Total (CY)	Total (CY)
Corner Columns	6	8.33	2.28	0.22	5.83	38.50
Edge Columns (N-S)	4	13.37	4.87	0.19	8.32	36.60
Edge Columns (E-W)	4	17.93	8.42	0.14	9.37	41.22
Interior Columns	2	41.67	23.10	0.14	18.43	40.54
Elevator Columns	1	16.33	7.60	0.14	8.60	9.46
Waiting Area Columns	1	8.33	2.28	0.22	5.83	6.42
Totals	**18**	**105.96**	**48.53**	**1.06**	**56.38**	**172.73**

Edge Formwork	Type	# of Footings	Length (FT)	Width (FT)	Perimeter (FT)	Depth (FT)	Linear Footage (FT)	Total SFCA
Corner Columns	4 Use	6	6.50	6.50	26.00	1.60	41.60	262.08
Edge Columns (N-S)	4 Use	4	8.50	8.50	34.00	2.00	68.00	285.60
Edge Columns (E-W)	4 Use	4	10.00	10.00	40.00	2.50	100.00	420.00
Interior Columns	4 Use	2	14.00	14.00	56.00	3.50	196.00	411.60
Elevator Columns	4 Use	1	9.50	9.50	38.00	2.50	95.00	99.75
Waiting Area Columns	4 Use	1	6.50	6.50	26.00	1.60	41.60	43.68
Totals		**18**	**55.00**	**55.00**	**220.00**	**13.70**	**542.20**	**1522.71**

Steel Reinforcement	Type	Size	# of Footings	Length (FT)	Width (FT)	Area (SF)	Total Area (CSF)
Corner Columns	Wire Mesh	66-66	6	6.50	6.50	42.25	2.99
Edge Columns (N-S)	Wire Mesh	66-66	4	8.50	8.50	72.25	3.41
Edge Columns (E-W)	Wire Mesh	66-66	4	10.00	10.00	100.00	4.72
Interior Columns	Wire Mesh	66-66	2	14.00	14.00	196.00	4.63
Elevator Columns	Wire Mesh	66-66	1	9.50	9.50	90.25	1.06
Waiting Area Columns	Wire Mesh	66-66	1	6.50	6.50	42.25	0.50
Totals			**18**	**55.00**	**55.00**	**543.00**	**17.31**

Chapter 7 . 1 . 1 . 2 Quantities for the Basement Level

Gravel Fill & Vapor Barrier	Length (FT)	Width (FT)	Area (SF)	Area (CSF)
Main Section	87.00	51.00	5324.40	48.81
Elevator Section	7.00	28.00	235.20	2.16
Totals	**94.00**	**79.00**	**5559.60**	**50.96**

Forms, Reinforcement, & Concrete	Length (FT)	Width (FT)	Perimeter (FT)	Area (SF)	Area (CSF)	Depth (FT)	Volume (CY)
Main Section	87.00	51.00	289.80	5058.18	50.58	0.50	99.29
Elevator Section	7.00	28.00	73.50	223.44	2.23	0.50	4.39
Totals	**94.00**	**79.00**	**363.30**	**5281.62**	**52.82**	**1.00**	**103.68**

Columns	Type	Quantity	Size (FT)	Length (FT)
Interior Columns	W10x68	4	7.50	30.90
N-S Columns	W10x68	4	7.50	30.90
E-W Columns	W10x68	4	7.50	30.90
Corner Columns	W10x68	6	7.50	46.35
Elevator Columns	W8x24	4	7.50	30.90
Waiting Area Columns	W10x68	1	7.50	7.73
Totals		**23.00**	**45.00**	**177.68**

Chapter 7 . 1 . 1 . 3 Quantities for the First, Second & Third Floors

Girders	Type	Quantity	Size (FT)	Length (FT)	No. Of Stories	Total (FT)
Size "A"	W21x68	4	35.50	146.26	3.00	438.78
Size "B"	W21X68	8	26.50	218.36	3.00	655.08
Size "C"	W21x68	8	20.50	168.92	3.00	506.76
Size "D"	W21x68	4	6.00	24.72	3.00	74.16
Size "E"	W21x68	4	7.00	28.84	3.00	86.52
Size "F"	W21x68	2	6.75	13.91	3.00	41.72
Totals		**30**	**102.25**	**601.01**	**18.00**	**1803.02**

Beams	Type	Quantity	Size (FT)	Length (FT)	No. Of Stories	Total (FT)
Size "A"	W10x22	14	20.50	295.61	3.00	886.83
Size "B"	W6x9	7	6.00	43.26	3.00	129.78
Stairway Size "A"	W12x14	4	9.00	37.08	3.00	111.24
Stairway Size "B"	W12x14	4	7.50	30.90	3.00	92.70
Totals		29	**43.00**	**406.85**	**12.00**	**1220.55**

Steel Decking (16 ga. Panels)	Length of Floor (FT)	Width of Floor (FT)	Area of Floor (SF)	Floor Area (CSF)	Floor Depth (FT)	Volume (CY)
Rooms Area Left	87.00	22.50	2055.38	20.55	0.29	22.84
Corridors Area	87.00	6.00	548.10	5.48	0.38	7.83
Elevator Area	7.00	28.00	205.80	2.06	0.38	2.94
Rooms Area Right	87.00	22.50	2055.38	20.55	0.29	22.84
Totals	**268.00**	**79.00**	**4864.65**	**48.65**	**1.33**	**56.45**

Panel Edge Forms & Welding	# of Panels Per Floor	Panel Length (FT)	Panel Width (FT)	Panel Perimeter (FT)	Length of Welds (FT)	Welding per Floor (FT)
Rooms Area Left	18	16.50	6.75	46.50	0.08	78.66
Corridors Area	6	16.50	6.00	45.00	0.08	22.84
Elevator Area	8	7.00	3.75	21.50	0.08	15.45
Rooms Area Right	18	16.50	6.75	46.50	0.08	78.66
Totals	**50**	**56.50**	**23.25**	**159.50**	**0.08**	**195.61**

Columns for First, Second & Third Floors	Quantity	First Floor Size (FT)	Total First Floor (FT)	Second Floor Size (FT)	Total Second Floor (FT)	Third Floor Size (FT)	Total Third Floor (FT)
Interior Columns	4	10.00	41.20	10.00	41.20	10.50	43.26
N-S Columns	4	10.00	41.20	10.00	41.20	10.50	43.26
E-W Columns	4	10.00	41.20	10.00	41.20	10.50	43.26
Corner Columns	6	10.00	61.80	10.00	61.80	10.50	64.89
Elevator Columns	4	10.00	41.20	10.00	41.20	10.50	43.26
Waiting Area Columns	1	10.00	10.30	10.00	10.30	10.50	10.82
Totals	**23**		**236.90**		**236.90**		**248.75**

Chapter 7 . 1 . 1 . 4 Quantities for the Roof Level

Girders	Quantity	Size (FT)	Length (FT)
Size "A"	4	35.50	146.26
Size "B"	8	26.50	218.36
Size "C"	8	20.50	168.92
Size "D"	4	6.00	24.72
Size "E"	1	11.50	11.85
Size "F"	4	7.00	28.84
Size "G"	2	6.75	13.91
Totals	**29**	**107.00**	**612.85**

Beams	Quantity	Size (FT)	Length (FT)
W10x22	14	20.50	295.61
W6x9	7	6.00	43.26
Totals	**21**	**26.50**	**338.87**

Concrete for Slabs	Panels Area (SF)	# of Panels	Total Area (SF)	Depth (FT)	Volume (CY)
Rooms Area Left	111.38	18	2055.38	0.38	30.83
Corridors Area	99.00	6	548.10	0.38	8.22
Elevator Area	26.25	8	205.80	0.38	3.09
Rooms Area Right	111.38	18	2055.38	0.38	30.83
Totals	**348.00**	**50**	**4864.65**	**1.50**	**72.97**

Chapter 7 . 1 . 1 . 5 Quantities for Fireproofing

Columns	Type	Width (bf) (FT)	Depth (d) (FT)	Thickness (tw) (FT)	Perimeter (FT)	Length (FT)	Quantity	Surface Area (SF)
Basement	W10x68	0.84	0.87	0.04	0.42	7.50	19	62.74
	W8x24	0.66	0.54	0.02	0.31	7.50	4	9.67
First & Second Floors	W10x68	0.84	0.87	0.04	0.42	10.00	19	83.65
	W8x24	0.66	0.54	0.02	0.31	10.00	4	12.90
Third Floor	W10x68	0.84	0.87	0.04	0.42	10.50	19	87.83
	W8x24	0.66	0.54	0.02	0.31	10.50	4	13.54
Totals		**4.52**	**4.22**	**0.18**	**2.18**	**56.00**	**69**	**270.34**

Beams	Type	Flange (bf)	Web (d)	Thickness (tw)	Perimeter (FT)	Length (FT)	Quantity	Surface Area (SF)
All Floors	W10x22	0.85	0.48	0.02	0.36	20.50	14	108.19
	W6x9	0.49	0.33	0.01	0.22	6.00	7	9.54
	W12x14	0.99	0.33	0.02	0.38	9.00	4	14.48
	W12x14	0.99	0.33	0.02	0.38	7.50	4	12.07
Roof	W10x22	0.85	0.48	0.02	0.36	20.50	14	108.19
	W6x9	0.49	0.33	0.01	0.22	6.00	7	9.54
Totals		**4.66**	**2.28**	**0.10**	**1.92**	**69.50**	**50**	**262.01**

Girders	Type	Flange (bf)	Web (d)	Thickness (tw)	Perimeter (FT)	Length (FT)	Quantity	Surface Area (SF)
All Floor Levels	W21x68	1.76	0.69	0.04	0.70	35.50	4	103.75
	W21x68	1.76	0.69	0.04	0.70	26.50	8	154.89
	W21x68	1.76	0.69	0.04	0.70	20.50	8	119.82
	W21x68	1.76	0.69	0.04	0.70	6.00	4	17.54
	W21x68	1.76	0.69	0.04	0.70	7.00	4	20.46
	W21x68	1.76	0.69	0.04	0.70	6.75	2	9.86
Totals		10.57	4.14	0.04	4.18	102.25	30	426.32
Roof Level	W18x40	1.49	0.50	0.03	0.58	35.50	4	85.94
	W18x40	1.49	0.50	0.03	0.58	26.50	8	128.30
	W18x40	1.49	0.50	0.03	0.58	20.50	8	99.25
	W18x40	1.49	0.50	0.03	0.58	6.00	4	14.53
	W18x40	1.49	0.50	0.03	0.58	11.50	1	6.96
	W18x40	1.49	0.50	0.03	0.58	7.00	4	16.95
	W18x40	1.49	0.50	0.03	0.58	6.75	2	8.17
Totals		**10.44**	**3.51**	**0.03**	**4.03**	**113.75**	**31**	**360.10**

Steel Decking	Length (FT)	Width (FT)	Panel Area (SF)	Length of Floor (FT)	Width of Floor (FT)	Area of Floor (SF)
Rooms Area Left	16.50	6.75	111.38	87.00	22.50	2055.38
Corridors Area	16.50	6.00	99.00	87.00	6.00	548.10
Elevator Area	7.00	3.75	26.25	7.00	28.00	205.80
Rooms Area Right	16.50	6.75	111.38	87.00	22.50	2055.38
Totals	**56.50**	**23.25**	**348.00**	**268.00**	**79.00**	**4864.65**

Chapter 7 . 1 . 1 . 6 Quantities for the Exterior Walls

Surface Area & Volume of Wall	Length (FT)	Width (FT)	Perimeter (FT)	Height (FT)	Wall Area (SF)	Waste (5%)	Wall Area w/ Waste	Mortar (CF)
Basement	87.00	51.00	276.00	7.50	2070.00	103.50	2173.50	137.99
	7.00	28.00	70.00	7.50	525.00	26.25	551.25	35.00
First Floor	87.00	51.00	276.00	10.00	2760.00	138.00	2898.00	183.99
	7.00	28.00	70.00	10.00	700.00	35.00	735.00	46.66
Second Floor	87.00	51.00	276.00	10.00	2760.00	138.00	2898.00	183.99
	7.00	28.00	70.00	10.00	700.00	35.00	735.00	46.66
Third Floor	87.00	51.00	276.00	10.50	2898.00	144.90	3042.90	193.19
	7.00	28.00	70.00	10.50	735.00	36.75	771.75	49.00
Totals	**376.00**	**316.00**	**1384.00**	**76.00**	**13148.00**	**657.40**	**13805.40**	**876.48**

Chapter 7 . 1 . 2 Prices for Activities from Means Construction Costs 1994[5]

This section presents the unit prices used in the estimate. They are entered by the section in which they appear in the book. First, earthwork, forming, reinforcing, cast in place concrete, masonry, steel, and so on.

Prices include the value for material, labor, and equipment were they apply.

Earthwork								
Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
022-250-2060	45	Excavation for Spread Footings on Common Earth with a 2 C.Y. Hydraulic Backhoe	200	CY	$0.00	$1.85	$4.68	$6.53
022-204-1300	41	Dozer Backfilling, bulk, up to 300' haul, no compaction	400	CY	$2.74	$6.82	$4.58	$14.14
022-266-1255	48	Hauling Earth, 20 CY dump trailer, 20 miles roundtrip, and medium traffic (20% increase)	96	CY	$0.00	$1.64	$5.15	$6.79

Formwork								
Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
031-142-5150	84	Plywood forms for 10" x 10" columns, 4 use	220	SFCA	$0.78	$3.29	$0.12	$4.19
031-142-5650	84	Plywood forms for 12" x 12" columns, 4 use	225	SFCA	$0.81	$3.21	$0.12	$4.14
031-150-1150	85	Flat plate form for elevated slabs up to 15' high, 4 use	560	SF	$0.51	$2.00	$0.07	$2.58
031-150-7000	85	Edge forms to 6" high on elevated slab, 4 use	500	LF	$0.31	$1.45	$0.06	$1.82
031-158-5150	86	Spread footing, 4 use	414	SFCA	$0.51	$1.75	$0.07	$2.33
031-170-3000	87	Slab on grade up to 6" High, 4 use	600	LF	$0.27	$1.21	$0.05	$1.53
031-192-1000	89	Vertical shores up to 10' high, erect & strip by hand	55	Ea	$0.00	$6.90	$0.00	$6.90
031-192-1500	89	Reshoring	1400	SF	$0.14	$0.27	$0.00	$0 41
053-104-7200	141	Sheet metal edge closure form, 12" wide with 2 bends, 16 ga.	360	LF	$2.25	$0.63	$0.21	$3.09

---

[5] See footnote #1, Referring to R.S. Means Publishers. For reference purposes, all of the activity's locations are provided by means of their respected page numbers listed next to the division code of each activity.

Reinforcement

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
032-107-0200	93	Reinforcing in place A615-G60, Columns, #3 to #7	1.5	TON	$480.00	$565.00	$0.00	$1,045.00
032-107-0600	93	Reinforcing in place A615-G60, Elevated Slabs #4 to #7	2.9	TON	$480.00	$291.00	$0.00	$771.00
032-109-0110	94	Splicing #6 reinforcing bars by butt welds, incl. holding bars in place	150	Ea	$1.35	$9.60	$3.28	$14.23
032-207-0300	95	Welded Wire Fabric 6 x 6 - #6/6 (W2.9/W2.9)	29	CSF	$12.90	$14.55	$0.00	$27.45
032-207-0400	95	Welded Wire Fabric 6 x 6 - #4/4 (W4/W4)	27	CSF	$17.25	$15.65	$0.00	$32.90
032-207-0650	95	Welded Wire Fabric 4 x 4 - #4/4 (W4/W4)	25	CSF	$26.50	$16.90	$0.00	$43.40

Cast-In-Place Concrete

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
033-126-0350	97	Concrete, ready mix, 4500 psi, high early strength (10% inc.)	NA	CY	$57.20	$0.00	$0.00	$57.20
033-134-0330	100	Curing with sprayed membrane curing compound	95	CSF	$2.05	$3.20	$0.00	$5.25
033-172-0400	101	Placing and vibrating concrete, columns, square up to 12", pumped to place	45	CY	$0.00	$29.00	$13.85	$42.85
033-172-1500	101	Placing and vibrating concrete, Elevated Slabs 6" to 10", pumped	130	CY	$0.00	$10.05	$4.80	$14.85
033-172-2600	101	Placing and vibrating concrete, spread footings over 5 CY, direct chute	110	CY	$0.00	$8.75	$0.62	$9.37
033-172-4600	101	Placing and vibrating concrete, slab on grade 6" & under, chute	165	CY	$0.00	$5.85	$0.41	$6.26
033-454-0100	102	Finishing floors, float finish	725	SF	$0.00	$0.23	$0.04	$0.27
033-458-0010	103	Breaking patches and voids	540	SF	$0.01	$0.34	$0.00	$0.35
033-458-0600	103	Float finishing walls, ceilings, includes columns	300	SF	$0.02	$0.62	$0.00	$0.64

Concrete Unit Masonry

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
041-024-0200	107	Mortar, with masonry cement, type N, 1:3 Mix	NA	CF	$2.81	$1.09	$0.00	$3.90
042-220-0200	112	Simulated brick concrete blocks, embossed both sides, not reinforced, 8" x 16", 8" thick	340	SF	$2.50	$2.65	$0.00	$5.15
042-252-0250	115	Plant installed styrofoam inserts for insulation, 8" x 16" x 8" concrete blocks	Add to above	SF	$0.66	$0.00	$0.00	$0.66
071-108-0300	169	Masonry Insulation Vermiculite or perlite, poured in cores of block, 8" thick wall, .258CF/SF	2400	SF	$0.29	$0.15	$0.00	$0.44
015-254-0090	11	Exterior scaffolding, steel tubular, rent, 1 use per mo , no plank, 1 to 5 stories high	16.8	CSF	$23.50	$34.00	$0.00	$57.50

Steel Members

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
050-575-2200	128	Continuous fillet welding, incl. equipment, 5 to 6 passes, .75" thick, 1.3#/LF	19	LF	$1.18	$12.00	$3.98	$17.16
051-220-6800	131	Wide flange, A36 steel, 2 tier, W8x24 column	1080	LF	$11.90	$1.34	$0.90	$14.14
051-220-7050	131	Wide flange, A36 steel, 2 tier, W10x68 column	984	LF	$33.50	$1.47	$0.99	$35.96
051-250-0100	133	Wide flange, A36 steel, W6x9, incl. bolts and erection	600	SF	$4.45	$2.41	$1.62	$8.48
051-250-0700	134	Wide flange, A36 steel, W10x22, incl. bolts and erection	660	SF	$10.90	$2.41	$1 62	$14.93
051-250-1100	134	Wide flange, A36 steel, W12x14, incl. bolts and erection	880	SF	$6.95	$1.64	$1.10	$9.69
051-250-3500	134	Wide flange, A36 steel, W18x40, incl. bolts and erection	960	SF	$19.80	$2.19	$1.16	$23.15
051-250-4700	134	Wide flange, A36 steel, W21x68, incl. bolts and erection	1036	SF	$32.00	$2.02	$1.07	$35.09
053-104-3050	140	Open type, 1.5" deep galvanized steel deck, 16 ga., under 50 CSF	3700	SF	$1.45	$0.23	$0.03	$1.71

Waterproofing

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
071-104-0600	167	Membrane waterproofing on slabs, 3 ply, felt	2100	SF	$0.33	$0.53	$0.07	$0.93
071-922-0900	168	.006" Polyethilene vapor barrier, standard	37	Sq	$4.00	$5.15	$0.00	$9.15

Drywalls & Partitions

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
095-106-0810	237	Complete suspended ceiling, mineral fiber panels, T-bar suspension, 2 x 4 x .625 bds.	380	SF	$1.10	$0.50	$0.00	$1.60
092-612-2100	231	Metal studs, drywal, 10' high, NLB galvanized 25 ga. studs, 1.625", @ 24" O.C.	520	SF	$0.18	$0.37	$0.00	$0.55

Fireproofing

Code	Page #	Description of Activity	Output	Units	Material	Labor	Equip.	Total
072-554-0400	155	Sprayed mineral or cementitious fiber for fireproofing beams	1500	SF	$0.36	$0.33	$0.17	$0.86
072-554-0500	155	Sprayed mineral or cementitious fiber for fireproofing corrugated or flutted steel decks	1250	SF	$0.37	$0.40	$0.20	$0.97
072-554-0700	155	Sprayed mineral or cementitious fiber for fireproofing columns to 1.125" thick	1100	SF	$0.41	$0.45	$0.23	$1.09
072-554-0850	155	For tamping, add 10%		SF				
072-554-0900	155	For canvas protection, add	5000	SF	$0.05	$0 10	$0.05	$0.20

Chapter 7 . 1 . 3 Cost Estimate for Steel Design

The cost estimate follows a straightforward approach defined by the previous two sections, quantities are all tabulated by level and then multiplied by their respective unit prices. Some specifications entitled the estimator to suggest the item best suited for the purpose of construction. Such cases were interior walls, insulation, concrete placing, curing, and finishing. For each case the following suggestions were made:

- Interior walls are $^5/_8$" gypsum drywall panels over aluminum studs.
- Insulation is provided by a vermiculite concrete fill inside the cores of the concrete blocks and 6" board inserts between the wall and the drywall giving the wall a heat loss rating of R-41.[6]
- Concrete placing applies to each individual case, whereas curing is achieved with a sprayed membrane curing compound and all finishes are float finishes.
- Also, all concrete used is 4500 psi concrete, in compliance with design requirements. This suggestion, in no way, alters the loading criterion of the structure because a safety factor for this was included in the design calculations.
- A number of factors were used to make the national average prices equal to Worcester prices. These factors were obtained from page 523 of Means Construction Costs Data 1994.

Cost Factors for the City of Worcester in 1994 (p. 523)

Division	Factor	Division	Factor
2. Sitework	1.061	4. Masonry	1.319
3.1 Formwork	1.236	5. Metals	1.120
3.2 Reinforcement	1.345	7. Thermal & Moisture Prot.	1.163
3.3 Cast in Place	1.149	9.2 Gypsum, Lather & Plaster	1.223
3. Concrete	1.211	9.5 Accoustical Elements	1.169

[6] According to Table 11-10, on page 214 of Estimating Home Building Costs by W.P. Jackson, 6" blankets provide an R-19 and 6" Cellulosic Fiber fill provides an R-22, adding up to an R-41 for the entire wall.

These factors were included into the unit prices of each activity, along with an 8% allowance for contractor's profit. Also, upon consultation with Dr. Guillermo Salazar, a $2,500 weekly overhead ($10,000/month) fee was used in totalling the cost of the project.

In order to determine the time cost of money, a very important factor because of the considerable differences in the schedules of both designs, an analyst from the Bank of Boston was contacted and an interest rate of 11% annually was determined and to be computed over the total balance every two week payment period. The purpose of all these factors is to adjust the total cost of both designs accordingly and, therefore, make a comparison a reliable, feasible and accurate one.

Chapter 7 . 1 . 3 . 1 Cost Report for Structural Steel Design by Activity

Activity	Quantity	Output	Duration	Unit Price	Cost
Excavation for Footings	396.5	200	1.98	$7.48	$2,966.85
Forming for Footings	1522.7	414	3.68	$3.11	$4,735.63
Reinforcing for Footings	17.3	29	0.60	$39.87	$689.82
Pouring of Concrete for Footings	154.9	110	1.41	$11.10	$1,719.70
Curing of Concrete for Footings	15.9	95	0.17	$6.87	$109.03
Backfill for Footings	172.7	400	0.43	$16.20	$2,798.23
Disposal for Footings	279.7	96	2.91	$7.78	$2,176.07
Erection of 8x24 Basement Columns	30.9	1080	0.03	$17.10	$528.39
Erection of 10x68 Basement Columns	146.8	984	0.15	$43.50	$6,384.93
Gravel Base Course	5559.6	8600	0.65	$0.37	$2,038.60
Vapor Barrier for Basement Slab	51.0	37	1.38	$11.49	$585.53
Edge Forming for Basement Slab	363.3	600	0.61	$2.04	$741.99
Reinforcing for Basement Slab	52.8	29	1.82	$39.87	$2,105.93
Pouring for Basement Slab	103.7	165	0.63	$7.77	$805.59
Curing for Basement Slab	48.6	95	0.51	$6.86	$333.71
Float Finishing for Basement Slab	4633.0	725	6.39	$0.35	$1,636.04
Fireproof Basement Columns	72.4	6100	0.01	$1.45	$104.99
Canvas Protection for Basement Floor	72.4	5400	0.01	$0.25	$18.10
Erection of Basement Exterior Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation for Basement Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Basement Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Erection of First Floor 21x68 Girders	601.0	1036	0.58	$42.44	$25,506.86

Erection of First Floor 6x9 Beams	43.3	600	0.07	$10.26	$443.85
Erection of First Floor 10x22 Beams	295.6	660	0.45	$18.06	$5,338.34
Erection of First Floor 12x14 Beams	68.0	880	0.08	$11.72	$796.64
Erection of First Floor Metal Deck	4864.7	3700	1.31	$2.07	$10,062.12
Welding of First Floor Metal Deck	195.6	19	10.30	$20.76	$4,060.86
Edge Forming for First Floor Slab	363.3	360	1.01	$3.74	$1,356.96
Reinforcing for First Floor Slab Halls	7.5	25	0.30	$63.04	$475.32
Reinforcing for First Floor Slab Rooms	41.1	27	1.52	$47.79	$1,964.17
Pouring First Floor Slab	56.5	130	0.43	$18.43	$1,040.37
Curing First Floor Slab	48.6	95	0.51	$6.86	$333.71
Float Finish of First Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Erection of First Floor Scaffolding	35.2	16.8	2.09	$81.91	$2,881.32
Fireproof First Floor Beams & Girders	570.6	6650	0.09	$1.19	$679.04
Erection of Basement Ceiling	4864.7	540	9.01	$2.11	$10,280.68
Erection of First Floor Exterior Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation for First Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of First Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Erection of First Floor 8x24 Columns	41.2	1080	0.04	$17.10	$704.52
Erection of First Floor 10x68 Columns	195.7	984	0.20	$43.50	$8,512.95
Fireproof First Floor Columns	96.6	6100	0.02	$1.35	$130.34
Canvas Protection for First Floor	667.2	5400	0.12	$0.25	$166.80
Erection of Second Floor 21x68 Girders	601.0	1036	0.58	$42.44	$25,506.86
Erection of Second Floor 6x9 Beams	43.3	600	0.07	$10.26	$443.85
Splicing for Second Floor Columns	295.6	660	0.45	$18.06	$5,338.34
Erection of Second Floor 12x14 Beams	68.0	880	0.08	$11.72	$796.64
Erection of Second Floor Metal Deck	4864.7	3700	1.31	$2.07	$10,062.12
Welding of Second Floor Metal Deck	195.6	19	10.30	$20.76	$4,060.86
Edge Forming for Second Floor Slab	363.3	360	1.01	$3.74	$1,356.96
Reinforcing for Second Floor Slab Halls	7.5	25	0.30	$63.04	$475.32
Reinforcing for Second Floor Slab Rooms	41.1	27	1.52	$47.79	$1,964.17
Pouring Second Floor Slab	56.5	130	0.43	$18.43	$1,040.37
Curing Second Floor Slab	48.6	95	0.51	$6.86	$333.71
Float Finish of Second Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Erection of Second Floor Scaffolding	35.2	16.8	2.09	$81.91	$2,881.32
Fireproof Second Floor Beams & Girders	570.6	6650	0.09	$1.19	$679.04
Erection of First Floor Ceiling	4864.7	540	9.01	$2.11	$10,280.68
Erection of Second Floor Exterior Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation for Second Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Second Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Erection of Second Floor 8x24 Columns	41.2	1080	0.04	$17.10	$704.52
Erection of Second Floor 10x68 Columns	195.7	984	0.20	$43.50	$8,512.95
Fireproof Second Floor Columns	96.6	6100	0.02	$1.35	$130.34
Canvas Protection for Second Floor	667.2	5400	0.12	$0.25	$166.80
Erection of Third Floor 21x68 Girders	601.0	1036	0.58	$42.44	$25,506.86
Erection of Third Floor 6x9 Beams	43.3	600	0.07	$10.26	$443.85
Erection of Third Floor 10x22 Beams	295.6	660	0.45	$18.06	$5,338.34
Erection of Third Floor 12x14 Beams	68.0	880	0.08	$11.72	$796.64
Erection of Third Floor Metal Deck	4864.7	3700	1.31	$2.07	$10,062.12

Welding of Third Floor Metal Deck	195.6	19	10.30	$20.76	$4,060.86
Edge Forming for Third Floor Slab	363.3	360	1.01	$3.74	$1,356.96
Reinforcing for Third Floor Slab Halls	7.5	25	0.30	$63.04	$475.32
Reinforcing for Third Floor Slab Rooms	41.1	27	1.52	$47.79	$1,964.17
Pouring Third Floor Slab	56.5	130	0.43	$18.43	$1,040.37
Curing Third Floor Slab	48.6	95	0.51	$6.86	$333.71
Float Finish of Third Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Erection of Third Floor Scaffolding	35.2	16.8	2.09	$81.91	$2,881.32
Fireproof Third Floor Beams & Girders	570.6	6650	0.09	$1.19	$679.04
Erection of Second Floor Ceiling	4864.7	540	9.01	$2.11	$10,280.68
Erection of Third Floor Exterior Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation for Third Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Third Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Erection of Third Floor 8x24 Columns	43.3	1080	0.04	$17.10	$739.75
Erection of Third Floor 10x68 Columns	205.5	984	0.21	$43.50	$8,938.82
Fireproof Third Floor Columns	96.6	6100	0.02	$1.35	$130.34
Canvas Protection for Third Floor	667.2	5400	0.12	$0.25	$166.80
Erection of Roof Floor 18x40 Girders	612.9	960	0.64	$28.00	$17,159.80
Erection of Roof Floor 6x9 Beams	43.3	600	0.07	$10.26	$443.85
Erection of Roof Floor 10x22 Beams	295.6	660	0.45	$18.06	$5,338.34
Erection of Roof Floor Metal Deck	4864.7	3700	1.31	$2.07	$10,062.12
Welding of Roof Floor Metal Deck	195.6	19	10.30	$20.76	$4,060.86
Edge Forming for Roof Floor Slab	363.3	360	1.01	$3.74	$1,356.96
Reinforcing for Roof Floor Slab	52.8	29	1.82	$39.89	$2,105.93
Pouring Roof Floor Slab	73.0	130	0.56	$18.43	$1,344.84
Curing Roof Floor Slab	48.6	95	0.51	$6.86	$333.71
Float Finish of Roof Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Erection of Third Floor Ceiling	4864.7	540	9.01	$2.11	$10,280.68
Waterproof Membrane over Roof Slab	5096.3	2100	2.43	$1.17	$5,953.07
Fireproof Roof Floor Beams & Girders	477.8	6650	0.07	$1.19	$568.62
Fireproof Roof Floor Metal Deck	4864.7	6375	0.76	$1.29	$6,278.60
Canvas Protection for Roof	5342.5	5400	0.99	$0.25	$1,335.63
Idle Materials					
4500 psi Concrete Ready Mix	500.9			$70.98	$35,553.88
1:3 Mix Ratio Masonry Mortar	876.5			$5.56	$4,873.23
Sub-Total					$493,718.16
Job Overhead					$29,700.00
GRAND TOTAL					**$523,418.16**

Chapter 7 . 1 . 3 . 2 Cost Report of Structural Steel Design by Subdivision

Item Code	Description	Quantity	Unit	Unit Price	Total Price
Division 1: General Requirements					
	Subdivision 1.1: Overhead				
1.101	Job Overhead	2.75	MONTH	$10,800.00	$29,700.00
				Overhead =	**$29,700.00**
				Division 1: General Requirements =	**$29,700.00**
Division 2: Sitework					
	Subdivision 2.1: Sitework				
2.101	Structural Excavation with a Backhoe	396.50	CY	$7.48	$2,966.85
2.102	Backfill with 10% Compaction	172.73	CY	$16.20	$2,798.23
2.103	Disposal of Waste Material Using Dump Trucks	279.71	CY	$7.78	$2,176.07
2.104	Gravel Fill Under Slab	5,559.60	SF	$0.37	$2,038.60
				Sitework =	**$9,979.75**
				Division 2: Sitework =	**$9,979.75**
Division 3: Concrete					
	Subdivision 3.1: Formwork				
3.105	Forms for Spread Footings	1,522.71	SFCA	$3.11	$4,735.63
3.106	Edge Forms Under 7" for Slabs on Grade	363.30	LF	$2.04	$741.99
3.107	Sheet Metal Edge Closure Form	1,453.20	LF	$3.74	$5,427.84
				Formwork =	**$10,905.46**
	Subdivision 3.2: Reinforcement				
3.203	WWF 6x6-#6/6 (W2.9xW2.9)	122.92	CSF	$39.88	$4,901.68
3.204	WWF 6x6-#4/4 (W4xW4)	123.30	CSF	$47.79	$5,892.51
3.205	WWF 4x4-#4/4 (W4xW4)	22.62	CSF	$63.04	$1,425.96
				Reinforcement =	**$12,220.16**
	Subdivision 3.3: Concrete in Place				
3.301	Concrete Ready Mix 4500psi, High Early Strength	500.90	CY	$70.98	$35,553.88
3.303	Placing Concrete on Elevated Slabs 6" to 10"	242.32	CY	$18.43	$4,465.96
3.304	Placing Concrete over Spread Footings	154.90	CY	$11.10	$1,719.70
3.305	Placing Concrete on Slab on Grade	103.68	CY	$7.77	$805.59
				Concrete in Place =	**$42,545.13**
	Subdivision 3.4: Miscellaneous				
3.401	Float Finishing Floors	23,165.00	SF	$0.35	$8,180.20
3.404	Concrete Curing with Sprayed Compound	258.95	CSF	$6.86	$1,777.58
				Concrete In Place =	**$9,957.78**
				Division 3: Concrete =	**$75,628.53**

			Division 3: Concrete =	**$75,628.53**

Division 4: Masonry

Subdivision 4.1: Mortar

4.101	Mortar with Masonry Cement, 1:3 Mix	876.48 CF	$5.56	$4,873.23
			Mortar =	**$4,873.23**

Subdivision 4.2: Concrete Blocks

4.201	Simulated Brick Concrete Blocks 8"x16"x8"	13,148.00 SF	$8.17	$107,356.89
			Concrete Blocks =	**$107,356.89**

Subdivision 4.3: Miscellaneous

4.301	Tubular Scaffolding for Wall Erection	105.53 CSF	$81.91	$8,643.96
			Miscellaneous =	**$8,643.96**
			Division 4: Masonry =	**$120,874.08**

Division 5: Metals

Subdivision 5.1: W Shaped Columns

5.101	W 8x24 Columns with 3% for Connections	156.56 LF	$17.10	$2,677.18
5.102	W10x68 Columns with 3% Connections	743.67 LF	$43.50	$32,349.65
			W Shaped Columns =	**$35,026.83**

Subdivision 5.2: W Shaped Members

5.201	W 6x9 Beams with 3% Connections	173.04 LF	$10.26	$1,775.40
5.202	W 10x22 Beams with 3% Connections	1,182.40 LF	$18.06	$21,353.35
5.203	W 12x14 Beams with 3% Connections	203.90 LF	$11.72	$2,389.92
5.204	W 18x40 Girders with 3% Connections	612.85 LF	$28.00	$17,159.80
5.205	W 21x68 Girders with 3% Connections	1,803.03 LF	$42.44	$76,520.59
			W Shaped Members =	**$119,199.06**

Subdivision 5.3: Metal Decking

5.301	Open Type Metal Deck, 1.5" Deep, 16 Gauge	19,458.60 SF	$2.07	$40,248.48
			Metal Decking =	**$40,248.48**

Subdivision 5.4: Connections

5.401	Continuous Fillet Welding of Deck	782.44 LF	$20.76	$16,243.44
			Connections =	**$16,243.44**
			Division 5: Metals =	**$210,717.81**

Division 7: Thermal & Moisture Protection

Subdivision 7.1 Insulators

7.101	Masonry Insulation Poured in Block Cores	13,805.40 SF	$0.55	$7,593.08
			Insulators =	**$7,593.08**

Subdivision 7.2: Fireproofing				
7.201	Sprayed Fireproofing for Beams & Girders	2,189.69 SF	$1.19	$2,605.74
7.202	Sprayed Fireproofing for Columns	362.06 SF	$1.37	$496.02
7.203	Sprayed Fireproofing for Metal Deck	4,864.70 SF	$1.29	$6,278.60
7.204	Canvas Protection	7,416.45 SF	$0.25	$1,854.13
			Fireproofing =	**$11,234.49**
Subdivision 7.3: Waterproofing				
7.301	Membrane Waterproofing	5,096.30 SF	$1.17	$5,953.07
7.302	Polyethylene Vapor Barrier	50.96 Sq.	$11.49	$585.53
			Waterproofing =	**$6,538.60**
			Division 7: Thermal & Moist. Prot. =	**$25,366.17**
Division 9: Gypsum & Lather				
Subdivision 9.1 Gypsum				
9.101	10' High Drywall with Aluminum Studs	13,805.40 SF	$0.73	$10,029.10
9.102	Suspended Ceiling, T Bar Shaped Suspension	19,458.60 SF	$2.11	$41,122.72
			Gypsum =	**$51,151.82**
			Division 9: Gypsum & Lather =	**$51,151.82**
			*****Bid Total*** =**	**$523,418.16**

Chapter 7 . 1 . 3 . 3 Cost Report of Structural Steel Design by Division

Code	Description	Total Cost	Profit	Total Price	Percentage
1	General Requirements	$27,500.00	$2,200.00	$29,700.00	5.67%
2	Sitework	$9,240.51	$739.24	$9,979.75	1.91%
3	Concrete	$70,026.42	$5,602.11	$75,628.53	14.45%
4	Masonry	$111,920.44	$8,953.64	$120,874.08	23.09%
5	Metals	$195,109.08	$15,608.73	$210,717.81	40.26%
7	Thermal & Moisture Prot.	$23,487.19	$1,878.98	$25,366.17	4.85%
9	Gypsum & Lather	$47,362.80	$3,789.02	$51,151.82	9.77%
	*****Bid Total*****	**$484,646.44**	**$38,771.72**	**$523,418.16**	**100.00%**

Chapter 7 . 1 . 3 . 4 Cost Report for Structural Steel Design by Budget

Code	Description	Total Cost	Profit	Total Price	Percentage
0	Office Expenses	$27,500.00	$2,200.00	$29,700.00	5.67%
1	Materials	$302,443.61	$24,195.49	$326,639.10	62.41%
2	Labor	$133,956.52	$10,716.52	$144,673.04	27.64%
3	Equipment	$20,746.31	$1,659.70	$22,406.01	4.28%
	*****Bid Total*****	**$484,646.44**	**$38,771.72**	**$523,418.16**	**100.00%**

Chapter 7 . 1 . 3 . 5 Cost Report for Structural Steel Design by Item

Code	Description	Total Cost	Profit	Total Price	Percentage
1.101	Job Overhead	$27,500.00	$2,376.00	$29,700.00	5.67%
2.101	Structural Excavation with a Backhoe	$2,747.08	$237.35	$2,966.85	0.57%
2.102	Backfill with 10% Compaction	$2,590.95	$223.86	$2,798.23	0.53%
2.103	Disposal of Waste Material Using Dump Trucks	$2,014.88	$174.09	$2,176.07	0.42%
2.104	Gravel Fill Under Slab	$1,887.59	$163.09	$2,038.60	0.39%
3.105	Forms for Spread Footings	$4,384.84	$378.85	$4,735.63	0.90%
3.106	Edge Forms Under 7" for Slabs on Grade	$687.03	$59.36	$741.99	0.14%
3.107	Sheet Metal Edge Closure Form	$5,025.78	$434.23	$5,427.84	1.04%
3.203	WWF 6x6-#6/6 (W2.9xW2.9)	$4,538.60	$392.13	$4,901.68	0.94%
3.204	WWF 6x6-#4/4 (W4xW4)	$5,456.03	$471.40	$5,892.51	1.13%
3.205	WWF 4x4-#4/4 (W4xW4)	$1,320.34	$114.08	$1,425.96	0.27%
3.301	Concrete Ready Mix 4500psi, High Early Strength	$32,920.26	$2,844.31	$35,553.88	6.79%
3.303	Placing Concrete on Elevated Slabs 6" to 10"	$4,135.15	$357.28	$4,465.96	0.85%
3.304	Placing Concrete over Spread Footings	$1,592.31	$137.58	$1,719.70	0.33%
3.305	Placing Concrete on Slab on Grade	$745.92	$64.45	$805.59	0.15%
3.401	Float Finishing Floors	$7,574.26	$654.42	$8,180.20	1.56%
3.404	Concrete Curing with Sprayed Compound	$1,645.91	$142.21	$1,777.58	0.34%
4.101	Mortar with Masonry Cement, 1:3 Mix	$4,512.25	$389.86	$4,873.23	0.93%
4.201	Simulated Brick Concrete Blocks 8"x16"x8"	$99,404.53	$8,588.55	$107,356.89	20.51%
4.301	Tubular Scaffolding for Wall Erection	$8,003.67	$691.52	$8,643.96	1.65%
5.101	W 8x24 Columns with 3% for Connections	$2,478.87	$214.17	$2,677.18	0.51%
5.102	W10x68 Columns with 3% Connections	$29,953.38	$2,587.97	$32,349.65	6.18%
5.201	W 6x9 Beams with 3% Connections	$1,643.89	$142.03	$1,775.40	0.34%
5.202	W 10x22 Beams with 3% Connections	$19,771.62	$1,708.27	$21,353.35	4.08%
5.203	W 12x14 Beams with 3% Connections	$2,212.89	$191.19	$2,389.92	0.46%
5.204	W 18x40 Girders with 3% Connections	$15,888.70	$1,372.78	$17,159.80	3.28%
5.205	W 21x68 Girders with 3% Connections	$70,852.40	$6,121.65	$76,520.59	14.62%

5.301	Open Type Metal Deck, 1.5" Deep, 16 Gauge	$37,267.11	$3,219.88	$40,248.48	7.69%
5.401	Continuous Fillet Welding of Deck	$15,040.22	$1,299.48	$16,243.44	3.10%
7.101	Masonry Insulation Poured in Block Cores	$7,030.63	$607.45	$7,593.08	1.45%
7.201	Sprayed Fireproofing for Beams & Girders	$2,412.72	$208.46	$2,605.74	0.50%
7.202	Sprayed Fireproofing for Columns	$459.28	$39.68	$496.02	0.09%
7.203	Sprayed Fireproofing for Metal Deck	$5,813.52	$502.29	$6,278.60	1.20%
7.204	Canvas Protection	$1,716.78	$148.33	$1,854.13	0.35%
7.301	Membrane Waterproofing	$5,512.10	$476.25	$5,953.07	1.14%
7.302	Polyethylene Vapor Barrier	$542.16	$46.84	$585.53	0.11%
9.101	10' High Drywall with Aluminum Studs	$9,286.20	$802.33	$10,029.10	1.92%
9.102	Suspended Ceiling, T Bar Shaped Suspension	$38,076.59	$3,289.82	$41,122.72	7.86%
	*****Bid Total*****	**$484,646.44**	**$41,873.45**	**$523,418.16**	**100.00%**

Chapter 7 . 1 . 3 . 6 Cost Report for Structural Steel Design by Floor Level

Code	*Description*	*Total Cost*	*Profit*	*Total Price*	*Percentage*
1	Foundations	$24,242.11	$1,939.37	$26,181.48	5.30%
2	Basement Floor	$60,543.72	$4,843.50	$65,387.22	13.24%
3	First Floor	$104,492.19	$8,359.38	$112,851.56	22.86%
4	Second Floor	$104,492.19	$8,359.38	$112,851.56	22.86%
5	Third Floor	$104,919.13	$8,393.53	$113,312.66	22.95%
6	Roof Floor	$58,457.11	$4,676.57	$63,133.68	12.79%
	*****Bid Total*****	**$457,146.44**	**$36,571.72**	**$493,718.16**	**100.00%**

Chapter 7 . 2 Reinforced Concrete Design

The second design method used was a reinforced concrete design. The changes in methodology for the concrete estimate are very little because the same dimensions and floor levels were used from the steel design. However, the changes that did happen will be addressed in their respective sections.

Chapter 7 . 2 . 1 Quantity Take-Off

Almost everything follows unchanged from the previous section except, slab thickness at the upper levels which now becomes 6.75" . Because heating is a major issue in a college dormitory and its operational cost, we decided to maintain the exterior walls in both designs equal to simplify the issue. We repeated the concept of using 8" simulated brick concrete blocks with vermiculite insulated cores, 6" insulating board inserts, and a gypsum drywall over aluminum studs[7].

It turns out, that by doing this we do not utilize the natural ability of concrete to act as an insulator by itself but, on the other hand, we shortened the schedule and reduced cost because a 2' concrete wall or concrete block wall is more expensive and requires more time to construct.

The prices used in the concrete estimate are already discussed in the previous section[8]. Therefore, we will skip that section and move on to the estimated cost for the reinforced concrete design.

---

[7] Design calculations were made assuming 2' of wall to be present. By selecting the mentioned system we only use about 18 to 20 inches of the allowed 24 inches, therefore complying with both design requirements.

[8] See section 7.1.2 Prices for Activities from Means Construction Cost Data 1994.

The columns in the concrete design were originally specified as 8" by 8", but after review using the F.R.A.M.E. program they were changed to 10" by 10". This change is reflected in the estimate respectively.

Chapter 7 . 2 . 1 . 1 Quantities for Foundations Level

Excavation	No. of Footings	Length (FT)	Width (FT)	Depth (FT)	Shoring	Volume (CY)	Total (CY)
Corner Columns	5	7.00	7.00	4.00	14.52	7.26	50.81
E-W Exterior Columns	2	8.00	8.00	4.00	7.59	9.48	26.55
Interior Columns	5	12.50	12.50	5.00	57.87	28.94	202.55
N-S Exterior Columns	9	8.00	8.00	4.00	34.13	9.48	119.47
Elevator Columns	1	10.50	10.50	4.00	6.53	16.33	22.87
Totals	**22**	**46.00**	**46.00**	**21.00**	**120.64**	**71.49**	**422.24**

Concrete for Footings	No. of Footings	Length (FT)	Width (FT)	Depth (FT)	Waste	Volume (CY)	Total (CY)
Corner Columns	5	6.00	6.00	1.50	1.10	2.00	11.10
E-W Exterior Columns	2	7.00	7.00	1.75	0.70	3.18	7.05
Interior Columns	5	11.50	11.50	3.00	8.08	14.69	81.55
N-S Exterior Columns	9	7.00	7.00	1.75	3.14	3.18	31.73
Elevator Columns	1	9.50	9.50	2.50	0.92	8.36	9.28
Totals	**22**	**41.00**	**41.00**	**10.50**	**13.94**	**31.40**	**140.71**

Concrete for Column Rests	No. of Rests	Length (FT)	Width (FT)	Depth (FT)	Factor (CF to CY)	Volume (CY)	Total (CY)
Corner Columns	5	1.67	1.67	2.50	0.14	0.26	1.43
E-W Exterior Columns	2	1.67	1.67	2.25	0.05	0.23	0.51
Interior Columns	10	1.67	1.67	2.00	0.23	0.21	2.28
N-S Exterior Columns	9	1.67	1.67	2.25	0.23	0.23	2.31
Elevator Columns	4	1.67	1.67	1.50	0.07	0.15	0.69
Totals	**30**	**8.33**	**8.33**	**10.50**	**0.72**	**1.08**	**7.22**

Backfill Material	No. of Footings	Excavation (CY)	Footings (CY)	Column Rests (CY)	Sub-Total (CY)	Total (CY)
Corner Columns	5	7.26	1.82	0.23	5.21	26.04
E-W Exterior Columns	2	9.48	2.89	0.21	6.38	12.77
Interior Columns	5	28.94	13.36	0.19	15.39	76.95
N-S Exterior Columns	9	9.48	2.89	0.21	6.38	57.45
Elevator Columns	1	16.33	7.60	0.14	8.60	8.60
Totals	**22**	**71.49**	**28.55**	**0.98**	**41.96**	**181.80**

Edge Formwork	Type	# of Footings	Length (FT)	Width (FT)	Perimeter (FT)	Depth (FT)	Linear Footage (FT)	Total SFCA
Corner Columns	4 Use	5	6.00	6.00	24.00	1.50	126.00	189.00
E-W Exterior Columns	4 Use	2	7.00	7.00	28.00	1.75	58.80	117.60
Interior Columns	4 Use	5	11.50	11.50	46.00	3.00	241.50	1207.50
N-S Exterior Columns	4 Use	9	7.00	7.00	28.00	1.75	264.60	2381.40
Elevator Columns	4 Use	1	9.50	9.50	38.00	2.50	39.90	39.90
Totals		**22**	**41.00**	**41.00**	**164.00**	**10.50**	**730.80**	**3935.40**

Steel Reinforcement	Type	Size	# of Footings	Length (FT)	Width (FT)	Area (SF)	Total Area (CSF)
Corner Columns	Wire Mesh	66-66	5	6.00	6.00	42.48	2.12
E-W Exterior Columns	Wire Mesh	66-66	2	7.00	7.00	57.82	1.16
Interior Columns	Wire Mesh	66-66	5	11.50	11.50	156.06	7.80
N-S Exterior Columns	Wire Mesh	66-66	9	7.00	7.00	57.82	5.20
Elevator Columns	Wire Mesh	66-66	1	9.50	9.50	106.50	1.06
Totals			**22**	**41.00**	**41.00**	**420.67**	**17.35**

Chapter 7 . 2 . 1 . 2 Quantities for Basement Level

Gravel Fill & Vapor Barrier	Length (FT)	Width (FT)	Area (SF)	Area (CSF)
Main Section	87.00	51.00	5324.40	48.81
Elevator Section	7.00	28.00	235.20	2.16
Totals	**94.00**	**79.00**	**5559.60**	**50.96**

Forms, Reinforcement & Concrete	Length (FT)	Width (FT)	Perimeter (FT)	Area (SF)	Area (CSF)	Depth (FT)	Volume (CY)
Main Section	87.00	51.00	281.10	5058.18	50.58	0.50	99.29
Elevator Section	7.00	28.00	72.80	223.44	2.23	0.50	4.39
Totals	**94.00**	**79.00**	**353.90**	**5281.62**	**52.82**	**1.00**	**103.68**

Columns	Type	Quantity	Width (FT)	Size (FT)	Area (SF)	Concrete (CY)	Perimeter (LF)	Area (SFCA)
Corner Columns	10x10	5	0.83	7.50	3.47	1.14	3.33	131.25
E-W Exterior Columns	10x10	2	0.83	7.50	1.39	0.46	3.33	52.50
Interior Columns	12x12	10	1.00	7.50	10.00	3.28	4.00	315.00
N-S Exterior Columns	12x12	9	1.00	7.50	9.00	2.95	4.00	283.50
Elevator Columns	10x10	4	0.83	7.50	2.78	0.91	3.33	105.00
Totals		**30**			**26.64**	**8.73**	**18.00**	**887.25**

Reinforcement for Columns	Type	Quantity	Bars per Column	Size of Bars (FT)	Wt. of Bars (#/LF)	Wt. of Steel (TONS)
Corner Columns	No 6	5	4	8.50	1.502	0.15
E-W Exterior Columns	No. 6	2	4	8.50	1.502	0.06
Interior Columns	No. 6	10	4	8.50	1.502	0.29
N-S Exterior Columns	No 6	9	4	8.50	1.502	0.26
Elevator Columns	No. 6	4	4	8.50	1.502	0.12
Totals		**30**		**42.50**	**1.502**	**0.87**

Chapter 7 . 2 . 1 . 3 Quantities for Slabs & Columns on All Levels

Forms, Curing, Finish, and Concrete for Slabs	Length (FT)	Width (FT)	Depth (FT)	Surface Area (SF)	Perimeter (LF)	Concrete Volume (CY)	Surface Area (CSF)
Main Section	87.00	51.00	0.56	4658.85	289.80	104.82	44.37
Elevator Section	7.00	28.00	0.56	205.80	73.50	4.63	1.96
Totals		**79**	**1.13**	**4864.65**	**363.30**	**109.45**	**46.33**

Reinforcing of Floor & Roof Slabs N-S Strips	Type	Quantity	Size (FT)	Length (FT)	Wt. of Bar (#/LF)	Total Wt. (TONS)
Edge Column Strip	No. 4	14	17.16	240.19	0.668	0.09
	No. 4	18	22.50	405.00	0.668	0.15
	No. 4	5	6.00	30.00	0.668	0.01
	No. 5	18	14.25	256.50	1.043	0.15
Between Edge & Interior Columns	No. 4	14	17.16	240.19	0.668	0.09
	No. 4	22	22.50	495.00	0.668	0.19
	No. 4	7	6.00	42.00	0.668	0.02
	No. 5	14	14.25	199.50	1.043	0.12
Strip Along Lines B and C	No. 4	26	17.16	446.06	0.668	0.17
	No. 4	8	6.00	48.00	0.668	0.02
	No. 5	20	22.50	450.00	1.043	0.27
	No. 6	24	14.25	342.00	1.502	0.29
Strip Between Lines B and C	No. 4	14	17.16	240.19	0.668	0.09
	No. 4	20	22.50	450.00	0.668	0.17
	No. 4	7	6.00	42.00	0.668	0.02
	No. 5	14	14.25	199.50	1.043	0.12
Totals		**245**	**239.63**	**4126.13**		**1.97**

Reinforcing of Floor & Roof Slabs E-W Strips	Type	Quantity	Size (FT)	Length (FT)	Wt. of Bar (#/LF)	Total Wt. (TONS)
Edge Column Strips Along Lines 1 and 4	No. 4	24	13.33	319.95	0.668	0.12
	No. 4	42	17.40	730.80	0.668	0.28
	No. 4	60	17.40	1044.00	0.668	0.40
Middle Strips Between Lines 1,2 and 3,4	No. 4	40	13.33	533.25	0.668	0.20
	No. 4	54	17.40	939.60	0.668	0.36
	No. 4	40	17.40	696.00	0.668	0.27
	No. 4	54	17.40	939.60	0.668	0.36
Column Strip Along Lines 2 and 3	No. 4	14	13.33	186.64	0.668	0.07
	No. 4	27	17.40	469.80	0.668	0.18
	No. 4	35	17.40	609.00	0.668	0.23
Totals		**390**	**161.79**	**6468.64**		**2.46**

First & Second Floor Columns	Type	Quantity	Width (FT)	Size (FT)	Area (SF)	Concrete (CY)	Area (SFCA)
Corner Columns	10x10	5	0.83	10.00	3.47	1.52	175.00
E-W Exterior Columns	10x10	2	0.83	10.00	1.39	0.61	70.00
Interior Columns	12x12	10	1.00	10.00	10.00	4.37	420.00
N-S Exterior Columns	12x12	9	1.00	10.00	9.00	3.93	378.00
Elevator Columns	10x10	4	0.83	10.00	2.78	1.21	140.00
Totals		**30**		**50.00**	**26.64**	**11.64**	**1183.00**

Third Floor	Type	Quantity	Width (FT)	Size (FT)	Area (SF)	Concrete (CY)	Area (SFCA)
Corner Columns	10x10	5	0.83	10.50	3.47	1.59	183.75
E-W Exterior Columns	10x10	2	0.83	10.50	1.39	0.64	73.50
Interior Columns	12x12	10	1.00	10.50	10.00	4.59	441.00
N-S Exterior Columns	12x12	9	1.00	10.50	9.00	4.13	396.90
Elevator Columns	10x10	4	0.83	10.50	2.78	1.27	147.00
Totals		**30**		**52.50**	**26.64**	**12.22**	**1242.15**

Reinforcement First & Second Floor Columns	Type	Quantity	Bars per Column	Size of Bars (FT)	Wt. of Bars (#/LF)	Wt. of Steel (TONS)
Corner Columns	No. 6	5	4	11.00	1.502	0.19
E-W Exterior Columns	No. 6	2	4	11.00	1.502	0.08
Interior Columns	No. 6	10	4	11.00	1.502	0.38
N-S Exterior Columns	No. 6	9	4	11.00	1.502	0.34
Elevator Columns	No. 6	4	4	11.00	1.502	0.15
Totals		**30**		**55.00**	**1.502**	**1.13**

Reinforcement Third Floor Columns	Type	Quantity	Bars per Column	Size of Bars (FT)	Wt. of Bars (#/LF)	Wt. of Steel (TONS)
Corner Columns	No. 6	5	4	11.50	1.502	0.20
E-W Exterior Columns	No. 6	2	4	11.50	1.502	0.08
Interior Columns	No. 6	10	4	11.50	1.502	0.39
N-S Exterior Columns	No. 6	9	4	11.50	1.502	0.35
Elevator Columns	No. 6	4	4	11.50	1.502	0.16
Totals		**30**		**57.50**	**1.502**	**1.18**

Chapter 7 . 2 . 1 . 4 Quantities for Exterior Wall All Levels

Surface Area & Volume of Wall	Length (FT)	Width (FT)	Perimeter (FT)	Height (FT)	Wall Area (SF)	Waste (5%)	Wall Area w/ Waste	Mortar (CF)
Basement	87.00	51.00	276.00	7.50	2070.00	103.50	2173.50	137.99
	7.00	28.00	70.00	7.50	525.00	26.25	551.25	35.00
First Floor	87.00	51.00	276.00	10.00	2760.00	138.00	2898.00	183.99
	7.00	28.00	70.00	10.00	700.00	35.00	735.00	46.66
Second Floor	87.00	51.00	276.00	10.00	2760.00	138.00	2898.00	183.99
	7.00	28.00	70.00	10.00	700.00	35.00	735.00	46.66
Third Floor	87.00	51.00	276.00	10.50	2898.00	144.90	3042.90	193.19
	7.00	28.00	70.00	10.50	735.00	36.75	771.75	49.00
Totals	**376.00**	**316.00**	**1384.00**	**76.00**	**13148.00**	**657.40**	**13805.40**	**876.48**

Chapter 7 . 2 . 1 . 5 Quantities for Miscellaneous Items All Levels

Shoring for All Levels	Shore Tributary Area (SF)	Length (FT)	Width (FT)	Floor Area (SF)	No. of Shores per Floor	Reshoring Third & Roof
Main Section	36.00	87.00	51.00	4437.00	124.0	4658.85
Elevator Section	36.00	7.00	28.00	196.00	6.0	205.8
Totals	**542.00**	**489.00**	**1809.00**	**4633.00**	**130**	**4864.65**

Breaking Patches & Voids	Columns	Slabs	Ceilings	Total
Basement	887.25	4633.00	4633.00	10153.25
First Floor	1183.00	4633.00	4633.00	10449.00
Second Floor	1183.00	4633.00	4633.00	10449.00
Third Floor	1242.15	4633.00	4633.00	10508.15
Totals	**4495.40**	**18532.00**	**18532.00**	**41559.40**

Chapter 7 . 2 . 2 Cost Estimate for Reinforced Concrete Design

The methodology for the estimate is identical to the steel design except for the logical sequence of the activities in the schedule. In steel construction, it is standard practice to begin a new floor level as soon as the columns of the previous level have been erected. However, in concrete there is a waiting period of 5 to 7 days before another floor can be started.

Since, the float finishing and curing activities on each slab combined for 7 days of duration, this lag time was absorbed efficiently and construction of each new level was allowed to continue without delays. However, we were forced to use two sets of shores because concrete slabs need 14 days of curing time to be able to support themselves and, as mentioned before, a new level was started after float finishing was concluded allowing the concrete only 7 days to cure.

Chapter 7 . 2 . 2 . 1 Cost Report for Reinforced Concrete Design by Activity

Activity	Quantity	Output	Duration	Unit Price	Cost
Excavation for Footings	422.2	200	2.11	$7.48	$3,159.91
Forming for Footings	3935.4	414	9.51	$3.11	$12,240.16
Reinforcing for Footings	17.4	29	0.60	$39.87	$693.80
Pouring of Concrete for Footings	147.9	110	1.34	$11.63	$1,719.70
Curing of Concrete for Footings	14.7	95	0.15	$6.87	$101.06
Backfill for Footings	181.8	400	0.45	$16.20	$2,945.16
Disposal for Footings	300.6	96	3.13	$7.78	$2,338.67
Splicing Rebars for Basement Columns	30.0	150	0.20	$20.67	$620.12
Reinforcement for Basement Columns	0.9	1.5	0.58	$1,517.97	$1,320.63
Forming for Small Basement Columns	288.8	220	1.31	$5.59	$1,614.11
Formwork for Large Basement Columns	598.5	225	2.66	$5.53	$3,309.71
Pouring of Concrete for Columns	8.7	45	0.19	$53.17	$464.17
Curing of Concrete for Columns	8.9	95	0.09	$8.67	$76.92
Float Finishing of Basement Columns	887.3	300	2.96	$0.83	$736.42
Gravel Base Course	5559.6	8600	0.65	$0.37	$2,038.60
Vapor Barrier	51.0	37	1.38	$11.49	$585.53
Edge Forming for Basement Slab	363.3	600	0.61	$2.04	$741.13
Reinforcing for Basement Slab	52.8	29	1.82	$39.87	$2,105.93

Pouring for Basement Slab	103.7	165	0.63	$7.77	$805.59
Curing for Basement Slab	48.6	95	0.51	$6.86	$333.71
Float Finishing of Basement Slab	4633.0	725	6.39	$0.35	$1,636.04
Shoring for First Floor Slab	130.0	55	2.36	$9.02	$1,172.60
Plate Forming for First Floor Slab	4864.7	560	8.69	$3.44	$16,753.81
Edge Forming for First Floor Slab	363.3	500	0.73	$2.43	$882.63
Reinforcing for First Floor Slab	4.4	2.9	1.53	$1,119.95	$4,961.38
Pouring for First Floor Slab	109.5	130	0.84	$18.43	$2,017.16
Curing for First Floor Slab	48.6	95	0.51	$6.87	$333.71
Float Finishing of First Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Splicing for First Floor Columns	30.0	150	0.20	$20.67	$620.12
Reinforcement for First Floor Columns	1.1	1.5	0.75	$1,517.97	$1,715.31
Forming for Small First Floor Columns	385.0	220	1.75	$5.59	$2,152.15
Forming for Large First Floor Columns	798.0	225	3.55	$5.53	$4,412.94
Pouring for First Floor Columns	11.6	45	0.26	$53.17	$618.90
Curing for First Floor Columns	11.8	95	0.12	$8.67	$102.57
Float Finishing of First Floor Columns	1183.0	300	3.94	$0.83	$981.89
Erection of Exterior Basement Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation on Basement Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Basement Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Shoring for Second Floor Slab	130.0	55	2.36	$9.02	$1,172.60
Plate Forming for Second Floor Slab	4864.7	560	8.69	$3.44	$16,753.81
Edge Forming for Second Floor Slab	363.3	500	0.73	$2.43	$882.63
Reinforcing for Second Floor Slab	4.4	2.9	1.53	$1,119.95	$4,961.38
Pouring for Second Floor Slab	109.5	130	0.84	$18.43	$2,017.16
Curing Second Floor Slab	48.6	95	0.51	$6.87	$333.71
Float Finishing of Second Floor Slab	4633.0	725	6.39	$0.35	$1,636.04
Splicing for Second Floor Columns	30.0	150	0.20	$20.67	$620.12
Reinforcement for Second Floor Columns	1.1	1.5	0.75	$1,517.97	$1,715.31
Forming for Small Second Floor Columns	385.0	220	1.75	$5.59	$2,152.15
Forming for Large Second Floor Columns	798.0	225	3.55	$5.53	$4,412.94
Pouring for Second Floor Columns	11.6	45	0.26	$53.17	$618.90
Curing for Second Floor Columns	11.8	95	0.12	$8.67	$102.57
Float Finishing of Second Floor Columns	1183.0	300	3.94	$0.83	$981.89
Erection of Scaffolding for First Floor Wall	35.2	16.8	2.10	$81.86	$2,881.32
Erection of Exterior First Floor Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation on First Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of First Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Shoring for Third Floor Slab	4864.7	1400	3.47	$0.54	$2,626.91
Plate Forming for Third Floor Slab	4864.7	560	8.69	$3.44	$16,753.81
Edge Forming for Third Floor Slab	363.3	500	0.73	$2.43	$882.63
Reinforcing for Third Floor Slab	4.4	2.9	1.53	$1,119.95	$4,961.38
Pouring for Third Floor Slab	109.5	130	0.84	$18.43	$2,017.16
Curing for Third Floor Slab	48.6	95	0.51	$6.87	$333.71
Float Finishing Third Floor Slabs	4633.0	725	6.39	$0.35	$1,636.04
Float Finishing of Basement Ceiling	4633.0	725	6.39	$0.83	$3,845.39
Breaking Patches and Voids Basement	10153.3	540	18.80	$0.46	$4,702.05
Splicing Rebars for Third Floor Columns	30.0	150	0.20	$20.67	$620.12

Reinforcement for Third Floor Columns	1.2	1.5	0.79	$1,517 97	$1,791.21
Forming of Small Third Floor Columns	404 3	220	1.84	$5.59	$2,259.76
Forming for Large Third Floor Columns	837.9	225	3.72	$5 53	$4,633.59
Pouring for Third Floor Columns	12.2	45	0.27	$53.17	$649 74
Curing for Third Floor Columns	12.4	95	0.13	$8.67	$107.69
Float Finishing Third Floor Columns	1242.2	300	4.14	$0.84	$1,038.95
Erection of Scaffolding for Second Floor Wall	35.2	16.8	2.09	$81.90	$2,881.32
Erection of Exterior Second Floor Wall	3287.0	340	9.67	$8.17	$26,839 22
Insertion of Insulation on Second Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Second Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Shoring for Roof Floor Slab	4864.7	1400	3.47	$0.54	$2,626.91
Plate Forming for Roof Floor Slab	4864.7	560	8.69	$3.44	$16,753.81
Edge Forming for Roof Floor Slab	363.3	500	0.73	$2.43	$882.63
Reinforcing for Roof Floor Slab	4.4	2.9	1.53	$1,119.95	$4,961.38
Pouring for Roof Floor Slab	109.5	130	0.84	$18.43	$2,017.16
Curing for Roof Floor Slab	48.6	95	0.51	$6.87	$333.71
Float Finishing for Roof Slab	4633.0	725	6.39	$0.35	$1,636.04
Waterproof Membrane on Roof Floor	5096.3	2100	2.43	$1.17	$5,953.07
Float Finishing of First Floor Ceiling	4633.0	725	6.39	$0.83	$3,845.39
Breaking Patches and Voids First Floor	10449.0	540	19.35	$0.45	$4,702.05
Float Finishing of Second Floor Ceiling	4633.0	725	6.39	$0.83	$3,845.39
Breaking Patches and Voids Second Floor	10449.0	540	19.35	$0.45	$4,702.05
Erection of Scaffolding for Third Floor Wall	35.2	16.8	2.09	$81.90	$2,881.32
Erection of Exterior Third Floor Wall	3287.0	340	9.67	$8.17	$26,839.22
Insertion of Insulation on Third Floor Wall	3451.4	2400	1.44	$0.55	$1,898.27
Erection of Third Floor Inner Wall	3451.4	520	6.64	$0.73	$2,507.28
Float Finishing Third Floor Ceiling	4633.0	725	6.39	$0.83	$3,845.39
Breaking Patches and Voids Third Floor	10508.2	540	19.46	$0.25	$2,627.05
Idle Materials					
4500 psi Concrete Ready Mix	733.6			$70.98	$52,070.93
1:3 Mix Ratio Masonry Mortar	876.5			$5.56	$4,873.23
Sub-Total					$414,468.82
Job Overhead					$78,300.00
GRAND TOTAL					**$492,768.82**

Chapter 7 . 2 . 2 . 2 Cost Report for Reinforced Concrete Design by Subdivision

Item Code	Description	Quantity	Unit	Unit Price	Total Price
Division 1: General Requirements					
	Subdivision 1.1: Overhead				
1.101	Job Overhead	7.25	MONTH	$10,800.00	$78,300.00
				Overhead =	**$78,300.00**
				Division 1: General Requirements =	**$78,300.00**
Division 2: Sitework					
	Subdivision 2.1: Sitework				
2.101	Structural Excavation with a Backhoe	422.24	CY	$7.48	$3,159.91
2.102	Backfill with 10% Compaction	181.80	CY	$16.20	$2,945.16
2.103	Disposal of Waste Material Using Dump Trucks	300.55	CY	$7.78	$2,338.67
2.104	Gravel Fill Under Slab	5,559.60	SF	$0.37	$2,038.60
				Sitework =	**$10,482.34**
				Division 2: Sitework =	**$10,482.34**
Division 3: Concrete					
	Subdivision 3.1: Formwork				
3.101	Plywood Forms for 10x10 Columns	1,463.00	SFCA	$5.59	$8,178.17
3.102	Plywood Forms for 12x12 Columns	3,032.40	SFCA	$5.53	$16,769.18
3.103	Flat Plate Forms for Elevated Slabs	19,458.60	SFCA	$3.44	$67,015.23
3.104	Edge Forms Under 7" for Slabs	1,453.20	LF	$2.43	$3,530.52
3.105	Forms for Spread Footings	3,935.40	SFCA	$3.11	$12,240.16
3.106	Edge Forms Under 7" for Slabs on Grade	363.30	LF	$2.04	$741.13
				Formwork =	**$108,474.39**
	Subdivision 3.2: Reinforcement				
3.201	Rebars for Columns #3 to #7	4.31	TON	$1,517.97	$6,542.46
3.202	Rebars for Slabs #3 to #7	17.72	TON	$1,119.95	$19,845.51
3.203	WWF 6x6-#6/6 (W2.9xW2.9)	70.22	CSF	$39.87	$2,799.73
3.204	Splicing Columns Rebars by Butt Welds	120.00	EA	$20.67	$2,480.46
				Reinforcement =	**$31,668.16**
	Subdivision 3.3: Concrete in Place				
3.301	Concrete Ready Mix 4500psi, High Early Strength	733.64	CY	$70.98	$52,070.93
3.302	Placing Concrete in Columns up to 12"	44.23	CY	$53.17	$2,351.71
3.303	Placing Concrete on Elevated Slabs 6" to 10"	437.80	CY	$18.43	$8,068.65
3.304	Placing Concrete over Spread Footings	147.93	CY	$11.63	$1,719.70

3.305	Placing Concrete on Slab on Grade	103.68 CY	$7.77	$805.59
			Concrete In Place =	**$65,016.59**
	Subdivision 3.4: Miscellaneous			
3.401	Float Finishing Floors	23,165.00 SF	$0.35	$8,180.20
3.402	Float Finishing Columns & Ceilings	23,027.40 SF	$0.83	$19,120.71
3.403	Breaking Patches & Voids	41,559.40 SF	$0.40	$16,733.20
3.404	Concrete Curing With Sprayed Compound	302.71 SF	$7.13	$2,159.34
3.405	Vertical Shores up to 10' High	260.00 EA	$9.02	$2,345.20
3.406	Reshoring of Vertical Shores	9,729.30 SF	$0.54	$5,253.82
			Miscellaneous =	**$53,792.47**
			Division 3: Concrete =	**$258,951.62**
Division 4: Masonry				
	Subdivision 4.1: Mortar			
4.101	Mortar with Masonry Cement, 1:3 Mix	876.48 CF	$5.56	$4,873.23
			Mortar =	**$4,873.23**
	Subdivision 4.2: Concrete Blocks			
4.201	Simulated Brick Concrete Blocks 8"x16"x8"	13,148.00 SF	$8.17	$107,356.89
			Concrete Blocks =	**$107,356.89**
	Subdivision 4.3: Miscellaneous			
4.301	Tubular Scaffolding for Wall Erection	105.56 CSF	$81.89	$8,643.96
			Miscellaneous =	**$8,643.96**
			Division 4: Masonry =	**$120,874.08**
Division 7: Thermal & Moisture Protection				
	Subdivision 7.1 Insulators			
7.101	Masonry Insulation Poured in Block Cores	13,805.40 SF	$0.55	$7,593.08
			Insulators =	**$7,593.08**
	Subdivision 7.3: Waterproofing			
7.301	Membrane Waterproofing	5,096.30 SF	$1.17	$5,953.07
7.302	Polyethylene Vapor Barrier	50.96 Sq.	$11.49	$585.53
			Waterproofing =	**$6,538.60**
			Division 7: Thermal & Moist. Prot. =	**$14,131.68**
Division 9: Gypsum & Lather				
	Subdivision 9.1 Gypsum			
9.101	10' High Drywall with Aluminum Studs	13,805.40 SF	$0.73	$10,029.10
			Gypsum =	**$10,029.10**
			Division 9: Gypsum & Lather =	**$10,029.10**
			*****Bid Total*** =**	**$492,768.82**

Chapter 7 . 2 . 2 . 3 Cost Report for Reinforced Concrete Design by Division

Code	Description	Total Cost	Profit	Total Price	Percentage
1	General Requirements	$72,500.00	$5,800.00	$78,300.00	15.89%
2	Sitework	$9,705.87	$776.47	$10,482.34	2.13%
3	Concrete	$239,770.02	$19,181.60	$258,951.62	52.55%
4	Masonry	$111,920.44	$8,953.65	$120,874.08	24.53%
7	Thermal & Moisture Prot.	$13,084.89	$1,046.79	$14,131.68	2.87%
9	Gypsum & Lather	$9,286.20	$742.90	$10,029.10	2.04%
	Bid Total	$456,267.43	$36,501.40	$492,768.82	100.00%

Chapter 7 . 2 . 2 . 4 Cost Report for Reinforced Concrete Design by Budget

Code	Description	Total Cost	Profit	Total Price	Percentage
0	Office Expenses	$72,500.00	$5,800.00	$78,300.00	15.89%
1	Materials	$156,043.46	$12,483.48	$168,526.94	34.20%
2	Labor	$214,536.94	$17,162.96	$231,699.90	47.02%
3	Equipment	$13,187.02	$1,054.96	$14,241.98	2.89%
	Bid Total	$456,267.42	$36,501.39	$492,768.82	100.00%

Chapter 7 . 2 . 2 . 5 Cost Report of Reinforced Concrete Design by Item

Code	Description	Total Cost	Profit	Total Price	Percentage
1.101	Job Overhead	$72,500.00	$6,264.00	$78,300.00	15.89%
2.101	Structural Excavation with a Backhoe	$2,925.84	$252.79	$3,159.91	0.64%
2.102	Backfill with 10% Compaction	$2,727.00	$235.61	$2,945.16	0.60%
2.103	Disposal of Waste Material Using Dump Trucks	$2,165.44	$187.09	$2,338.67	0.47%
2.104	Gravel Fill Under Slab	$1,887.59	$163.09	$2,038.60	0.41%
3.101	Plywood Forms for 10x10 Columns	$7,572.38	$654.25	$8,178.17	1.66%
3.102	Plywood Forms for 12x12 Columns	$15,527.02	$1,341.53	$16,769.18	3.40%
3.103	Flat Plate Forms for Elevated Slabs	$62,051.14	$5,361.22	$67,015.23	13.60%
3.104	Edge Forms Under 7" for Slabs	$3,269.00	$282.44	$3,530.52	0.72%
3.105	Forms for Spread Footings	$11,333.48	$979.21	$12,240.16	2.48%

3.106	Edge Forms Under 7" for Slabs on Grade	$686.23	$59.29	$741.13	0.15%
3.201	Rebars for Columns #3 to #7	$6,057.83	$523.40	$6,542.46	1.33%
3.202	Rebars for Slabs #3 to #7	$18,375.48	$1,587.64	$19,845.51	4.03%
3.203	WWF 6x6-#6/6 (W2.9xW2.9)	$2,592.35	$223.98	$2,799.73	0.57%
3.204	Splicing Columns Rebars by Butt Welds	$2,296.72	$198.44	$2,480.46	0.50%
3.301	Concrete Ready Mix 4500psi, High Early Strength	$48,213.82	$4,165.67	$52,070.93	10.57%
3.302	Placing Concrete in Columns up to 12"	$2,177.51	$188.14	$2,351.71	0.48%
3.303	Placing Concrete on Elevated Slabs 6" to 10"	$7,470.98	$645.49	$8,068.65	1.64%
3.304	Placing Concrete over Spread Footings	$1,592.31	$137.58	$1,719.70	0.35%
3.305	Placing Concrete on Slab on Grade	$745.92	$64.45	$805.59	0.16%
3.401	Float Finishing Floors	$7,574.26	$654.42	$8,180.20	1.66%
3.402	Float Finishing Columns & Ceilings	$17,704.36	$1,529.66	$19,120.71	3.88%
3.403	Breaking Patches & Voids	$15,493.70	$1,338.66	$16,733.20	3.40%
3.404	Concrete Curing With Sprayed Compound	$1,999.39	$172.75	$2,159.34	0.44%
3.405	Vertical Shores up to 10' High	$2,171.48	$187.62	$2,345.20	0.48%
3.406	Reshoring of Vertical Shores	$4,864.65	$420.31	$5,253.82	1.07%
4.101	Mortar with Masonry Cement, 1:3 Mix	$4,512.25	$389.86	$4,873.23	0.99%
4.201	Simulated Brick Concrete Blocks 8"x16"x8"	$99,404.53	$8,588.55	$107,356.89	21.79%
4.301	Tubular Scaffolding for Wall Erection	$8,003.67	$691.52	$8,643.96	1.75%
7.101	Masonry Insulation Poured in Block Cores	$7,030.63	$607.45	$7,593.08	1.54%
7.301	Membrane Waterproofing	$5,512.10	$476.25	$5,953.07	1.21%
7.302	Polyethylene Vapor Barrier	$542.16	$46.84	$585.53	0.12%
9.101	10' High Drywall with Aluminum Studs	$9,286.20	$802.33	$10,029.10	2.04%
	Bid Total	**$456,267.43**	**$39,421.51**	**$492,768.82**	**100.00%**

Chapter 7 . 2 . 2 . 6 Cost Report for Reinforced Concrete Design by Floor

Code	Description	Total Cost	Profit	Total Price	Percentage
1	Foundations	$31,219.25	$2,497.54	$33,716.79	8.14%
2	Basement Floor	$60,246.28	$4,819.70	$65,065.99	15.71%
3	First Floor	$84,115.49	$6,729.24	$90,844.73	21.93%
4	Second Floor	$84,115.49	$6,729.24	$90,844.73	21.93%
5	Third Floor	$84,049.36	$6,723.95	$90,773.31	21.92%
6	Roof Floor	$39,743.77	$3,179.50	$42,923.28	10.36%
	Bid Total	**$383,489.65**	**$30,679.17**	**$414,168.82**	**100.00%**

Chapter 7 . 3 Cost & Graphic Analysis for Both Designs

In the cost analysis of the designs, we compared base costs, total costs (profit and overhead), real cost (interests on construction loan), costs per level, and construction schedules. However, we used a the national average costs of each activity to perform the estimate, therefore it was necessary to multiply the results by a conversion factor to calculate the project's cost in the City of Worcester. These factors were taken out Means Construction Costs Data 1994[9] and were outlined in a table in section 7.1.

The base cost of both designed offered a substantial advantage to the concrete design but is misleading because several significant factors are left out of the comparison process. Some of these are length of schedule, time value of money, overhead, and contractor's profit.

Following tables will summarize that, when these factors are added to the equation, there is a significant increase in the cost of the concrete design, whereas, there is not that much of an increase in the steel design. The factors which affect this comparison the most are project overhead and interest payments. As mentioned previously, overhead was $2,500 per week and interests were 11%. These factors are time or schedule of construction dependent and, as the schedule in the appendix shows, steel design construction lasts 11 weeks and concrete design construction lasts 29 weeks or more than twice as long. Therefore, it is very important to include these factors in order to make an accurate comparison of both designs.

The scope of the project limited us to only account for phases and aspects of the project in which both design would differ. Therefore, mass excavation, internal construction, mechanical systems, electrical systems, H.V.A.C. systems, and so on were left out of the analysis because their requirements would be identical in both designs.

---

[9] See footnote 1.

An important aspect of this analysis is the real value of money comparison. This was a new experience because we were forced to take the side of the owner, instead of the contractor, and use a cash flow analysis to determine what payments had to be made, when, what the retainage would be, what the outstanding balance on the construction loan would be from week to week, and so on. After the cash flow and all other variables were obtained, we plotted some graphs to illustrate the findings from the analysis. These are presented next along with their respective data tables.

Chapter 7 . 3 . 1 . 1 Cost Analysis Tables

Cash Flow for Concrete Design					
Period	Week 1	Week 2	Base Cost	w/ Overhead	Payments
1	$17,882.35	$13,195.37	$31,077.72	$60,796.20	$0.00
2	$16,530.20	$9,739.91	$26,270.11	$52,783.52	$54,716.58
3	$9,725.75	$23,245.20	$32,970.95	$63,951.58	$47,505.17
4	$19,880.58	$13,827.50	$33,708.08	$65,180.13	$57,556.43
5	$10,459.60	$30,977.68	$41,437.28	$78,062.13	$58,662.12
6	$26,755.04	$13,940.73	$40,695.77	$76,826.28	$70,255.92
7	$19,624.58	$50,547.44	$70,172.02	$125,953.37	$69,143.66
8	$38,846.11	$21,920.37	$60,766.48	$110,277.47	$113,358.03
9	$4,025.36	$9,660.81	$13,686.17	$31,810.28	$99,249.72
10	$598.16	$2,895.82	$3,493.98	$14,823.30	$28,629.26
11	$10,534.03	$13,163.96	$23,697.99	$48,496.65	$13,340.97
12	$13,591.35	$7,711.79	$21,303.14	$44,505.23	$43,646.99
13	$5,527.95	$4,899.08	$10,427.03	$26,378.38	$40,054.71
14	$2,236.02	$1,572.17	$3,808.19	$15,346.98	$23,740.55
15	$953.91		$953.91	$10,589.85	$13,812.29
16					$92,109.00
TOTAL	**$197,170.99**	**$217,297.83**	**$414,468.82**	**$825,781.37**	**$825,781.37**

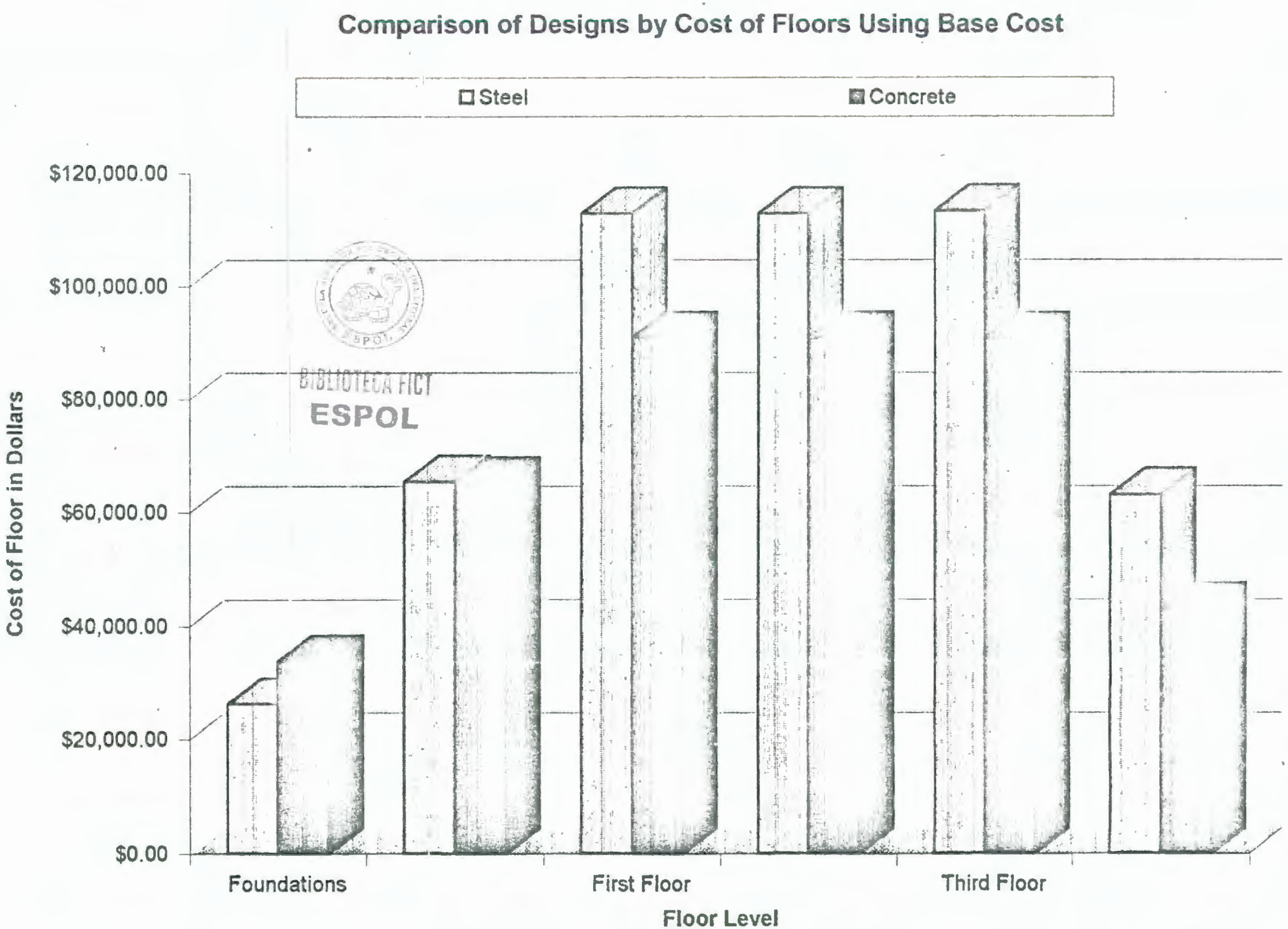
Cash Flow for Steel Design					
Period	Week 1	Week 2	Base Cost	w/ Overhead	Payments
1	$7,544.92	$13,662.26	$21,207.18	$44,345.30	$0.00
2	$171,078.30	$78,195.29	$249,273.59	$424,455.98	$39,910.77
3	$31,618.88	$65,854.66	$97,473.54	$171,455.90	$382,010.39
4	$25,725.37	$52,456.05	$78,181.42	$139,302.37	$154,310.31
5	$40,343.82	$5,664.39	$46,008.21	$85,680.35	$125,372.13
6	$1,574.22		$1,574.22	$11,623.70	$77,112.32
7					$98,147.69
TOTAL	**$277,885.51**	**$215,832.65**	**$493,718.16**	**$876,863.60**	**$876,863.60**

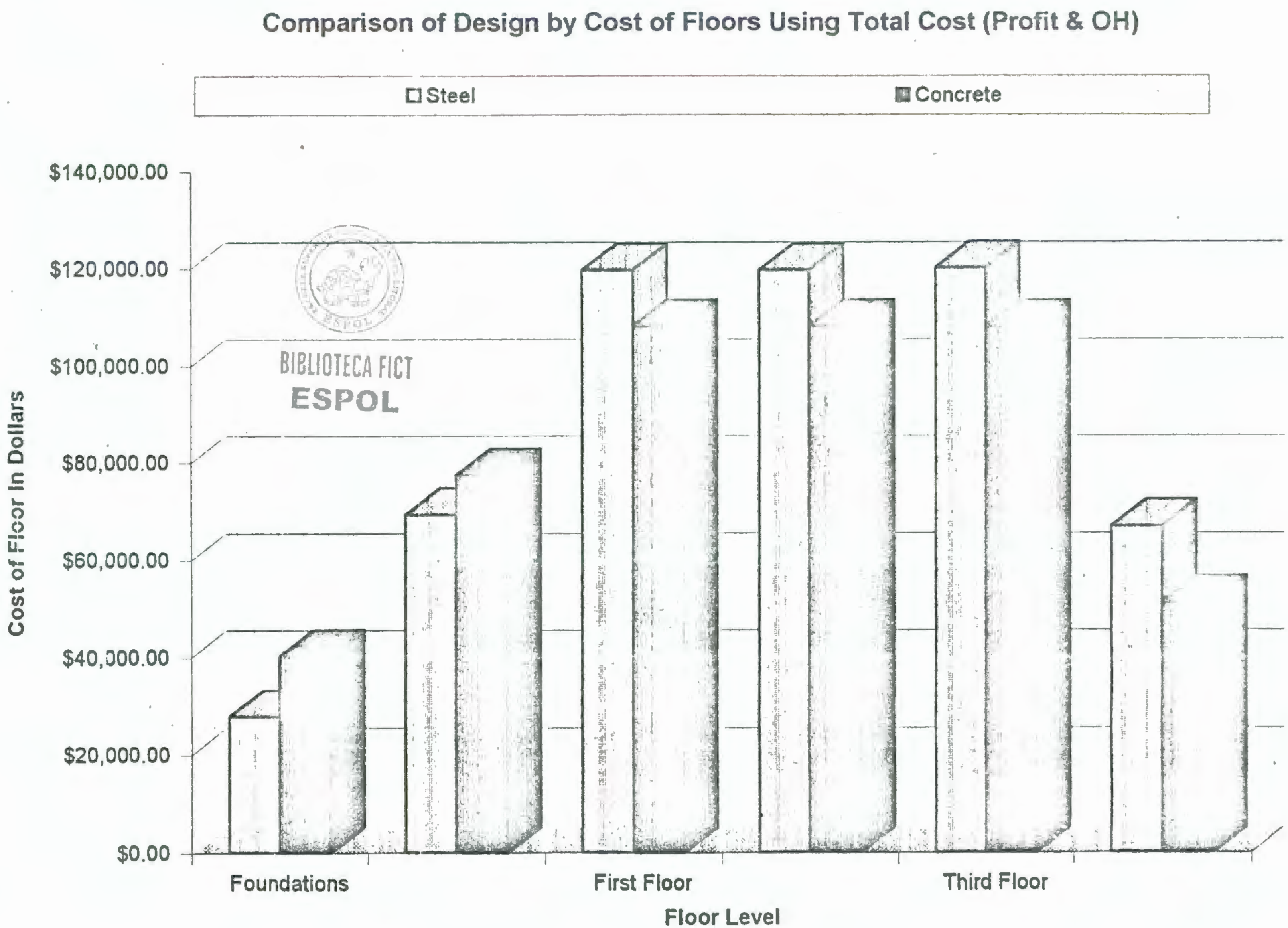
Interest Payments for Both Designs				
	Steel Design		Concrete Design	
Weeks	Balance	Interests	Balance	Interests
1	$39,910.77	$168.85	$54,716.58	$231.49
2	$422,090.01	$1,785.77	$102,453.24	$433.46
3	$578,186.08	$2,446.17	$160,443.12	$678.80
4	$706,004.39	$2,986.94	$219,784.04	$929.86
5	$786,103.64	$3,325.82	$290,969.81	$1,231.03
6	$887,577.16	$1,877.57	$361,344.49	$1,528.77
7	$889,454.72		$476,231.29	$2,014.82
8			$577,495.83	$2,443.25
9			$608,568.34	$2,574.71
10			$624,484.02	$2,642.05
11			$670,773.06	$2,837.89
12			$713,665.65	$3,019.35
13			$740,425.55	$3,132.57
14			$757,370.41	$3,204.26
15			$852,683.67	$1,803.75
TOTAL	**$889,454.72**	**$12,591.12**	**$854,487.42**	**$28,706.05**

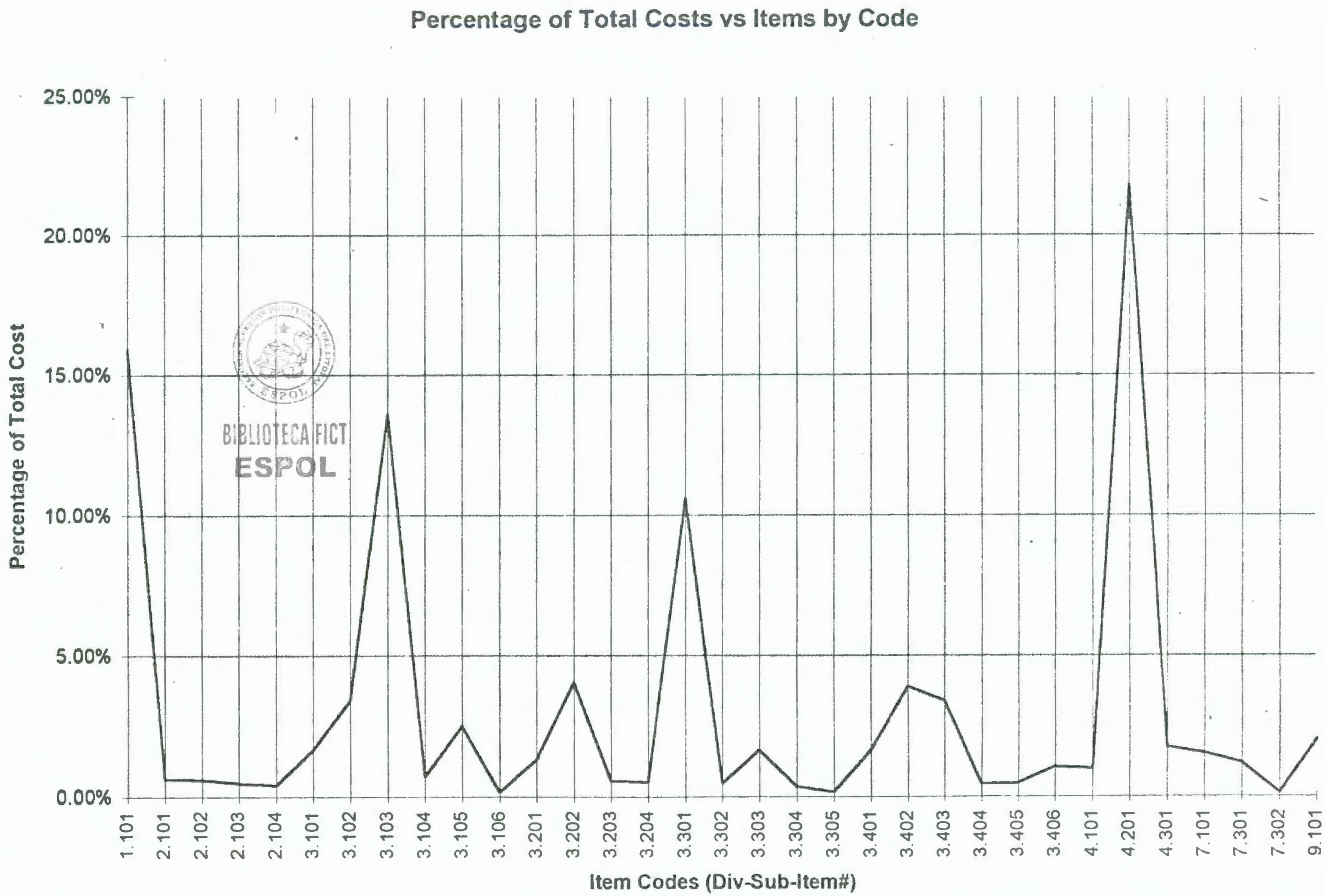
Chapter 7 . 3 . 1 . 2 Cost Analysis Graphs

In the following pages, some of the above tables will be further detailed and explained by converting their information into a visual aid or graph. These graphs make any differences between the design much easier to appreciate and, more importantly, help to make the best choice amongst the two designs for the project and its requirements. These requirements are a constructable design which is as economic and time saving as possible. Also, a design which is not very expensive to operate in the long run and which can be flexible to future changes.

Differences in cost and interest payments between designs are illustrated by the differences in height of the lines at the end of each graph. These are the graphs we constructed:

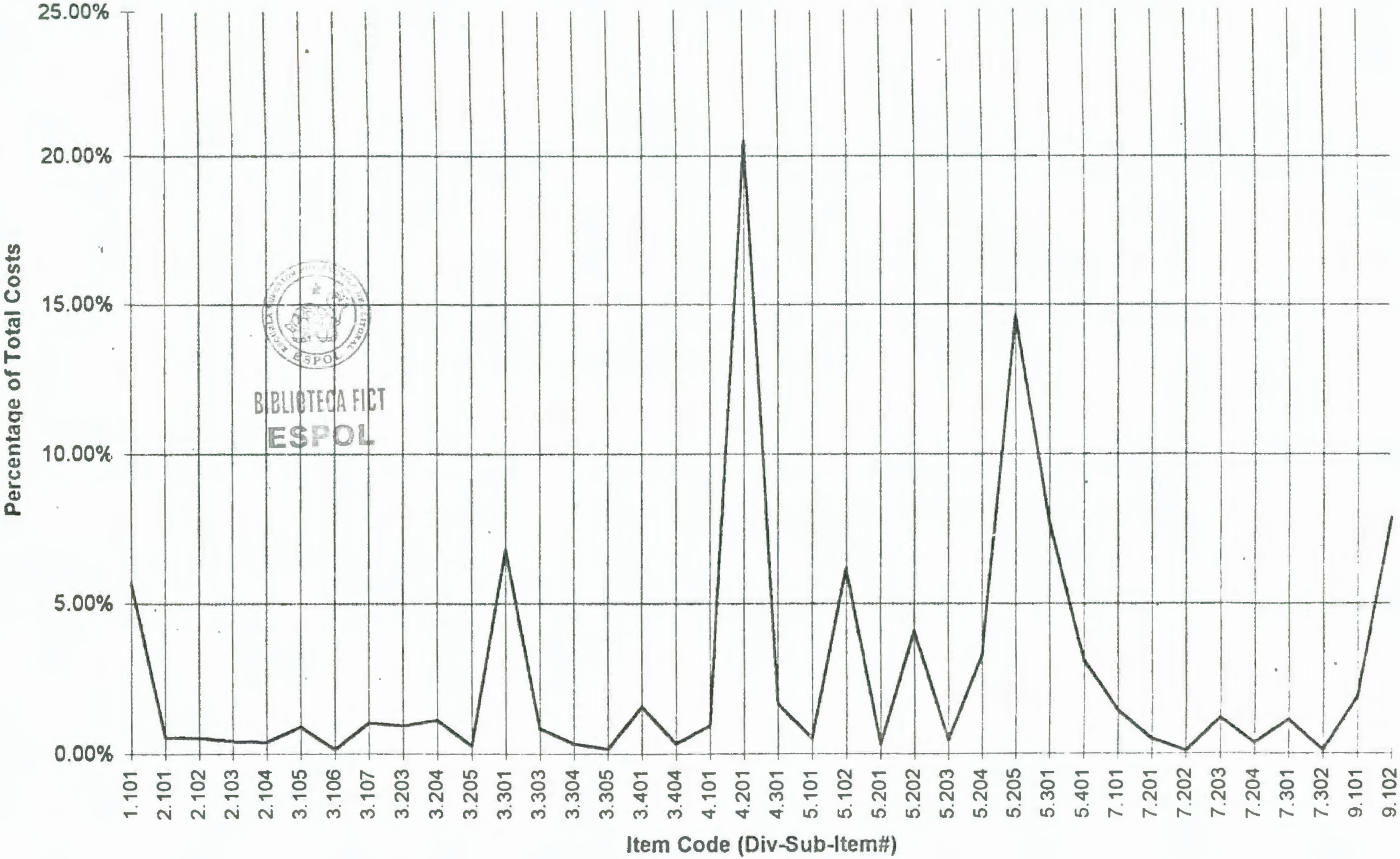

Comparison of Designs by Cost of Floors Using Base Cost
Steel
Concrete
$120,000.00
$100,000.00
$80,000.00
$60,000.00
$40,000.00
$20,000.00
$0.00
Cost of Floor in Dollars
Foundations
First Floor
Third Floor
Floor Level


Comparison of Design by Cost of Floors Using Total Cost (Profit & OH)
Steel
Concrete
$140,000.00
$120,000.00
$100,000.00
$80,000.00
$60,000.00
$40,000.00
$20,000.00
$0.00
Cost of Floor in Dollars
Foundations
First Floor
Third Floor
Floor Level

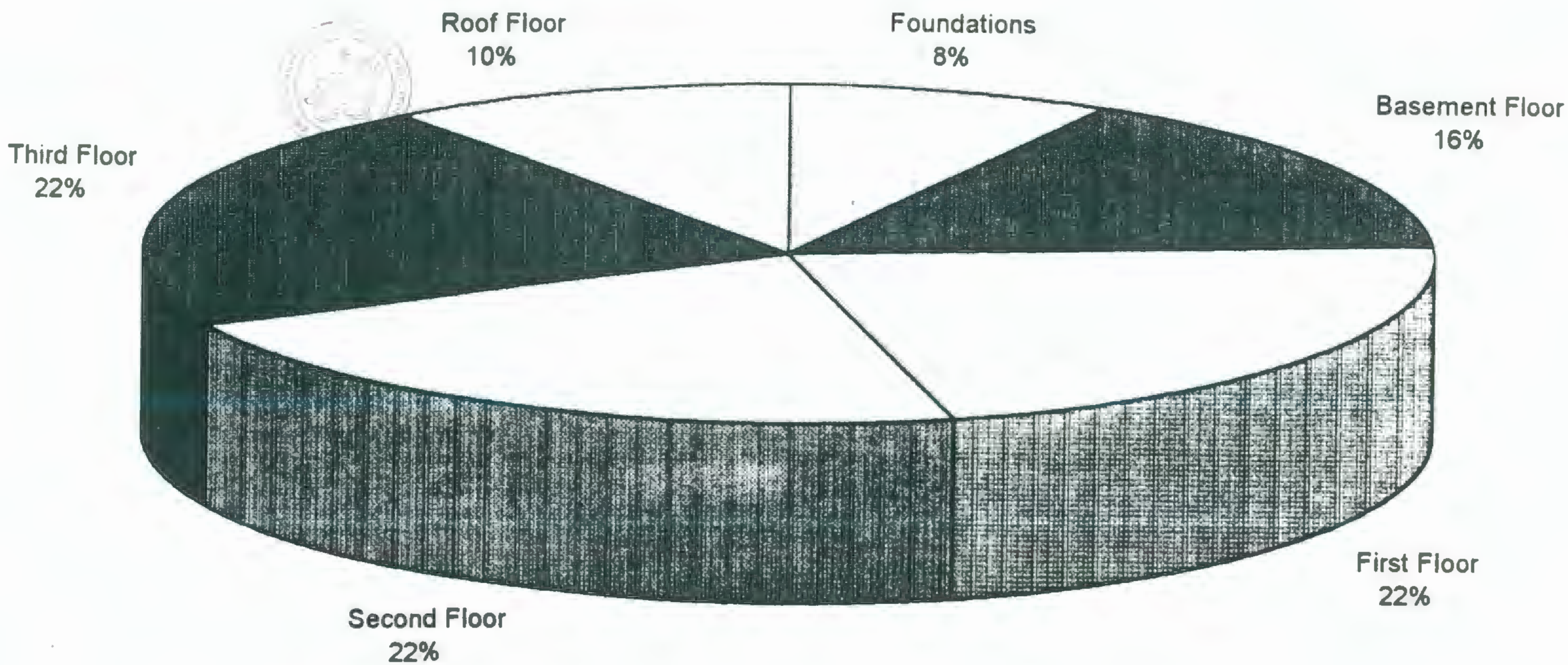


Percentage of Total Costs vs Items by Code
25.00%
20.00%
15.00%
10.00%
5.00%
0.00%
Percentage of Total Cost
1.101
2.101
2.102
2.103
2.104
3.101
3.102
3.103
3.104
3.105
3.106
3.201
3.202
3.203
3.204
3.301
3.302
3.303
3.304
3.305
3.401
3.402
3.403
3.404
3.405
3.406
4.101
4.201
4.301
7.101
7.301
7.302
9.101
Item Codes (Div-Sub-Item#)

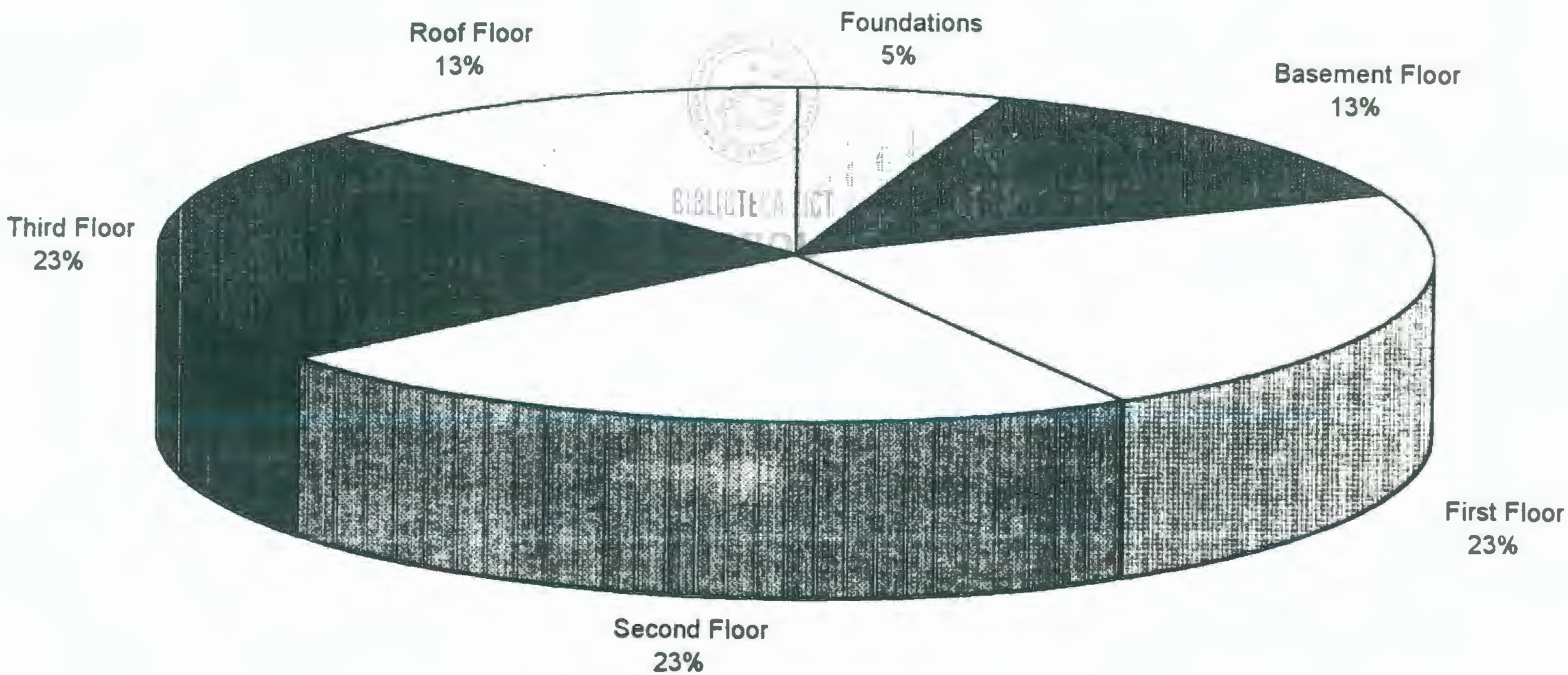
Percentage of Total Costs vs Item Codes

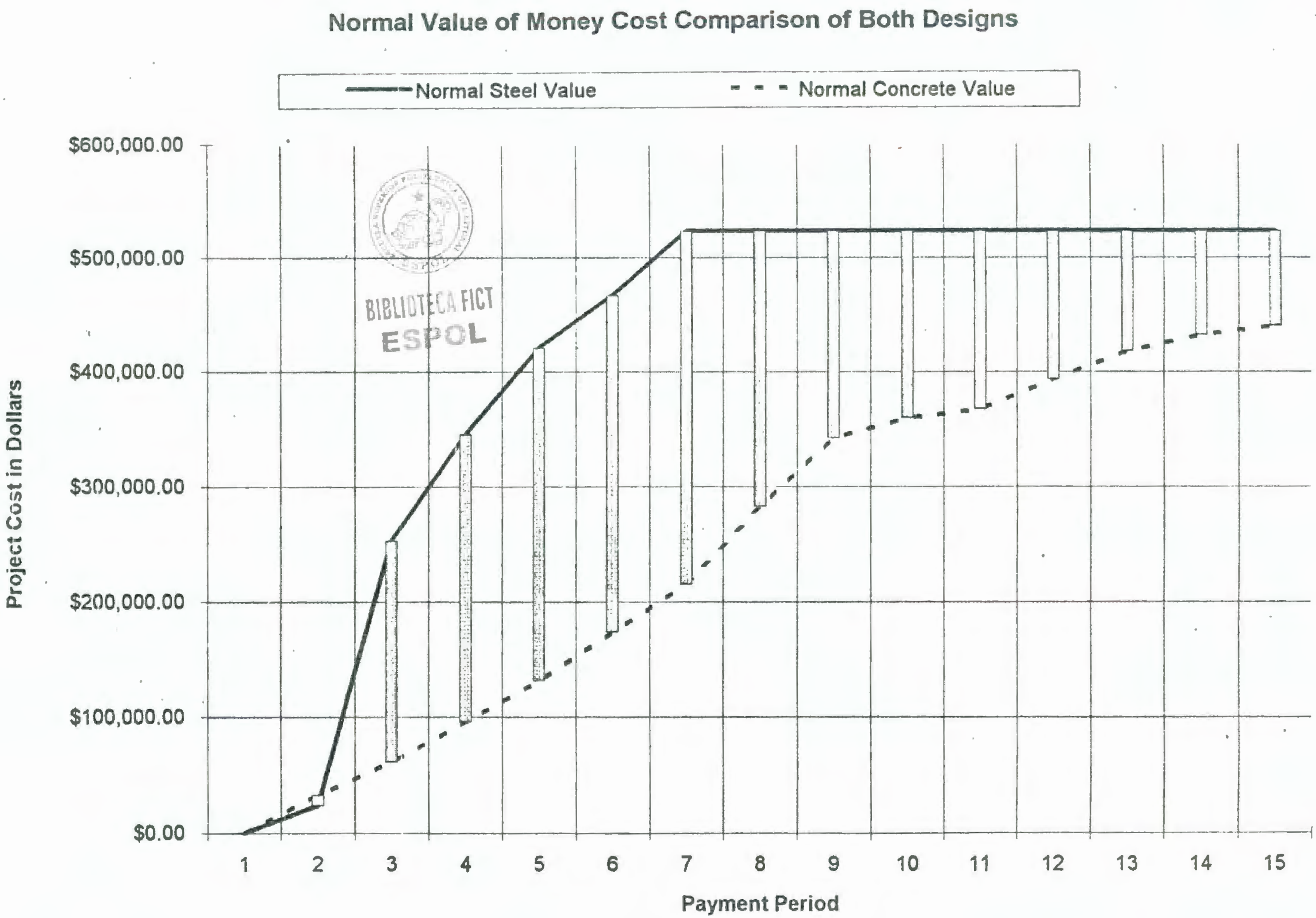


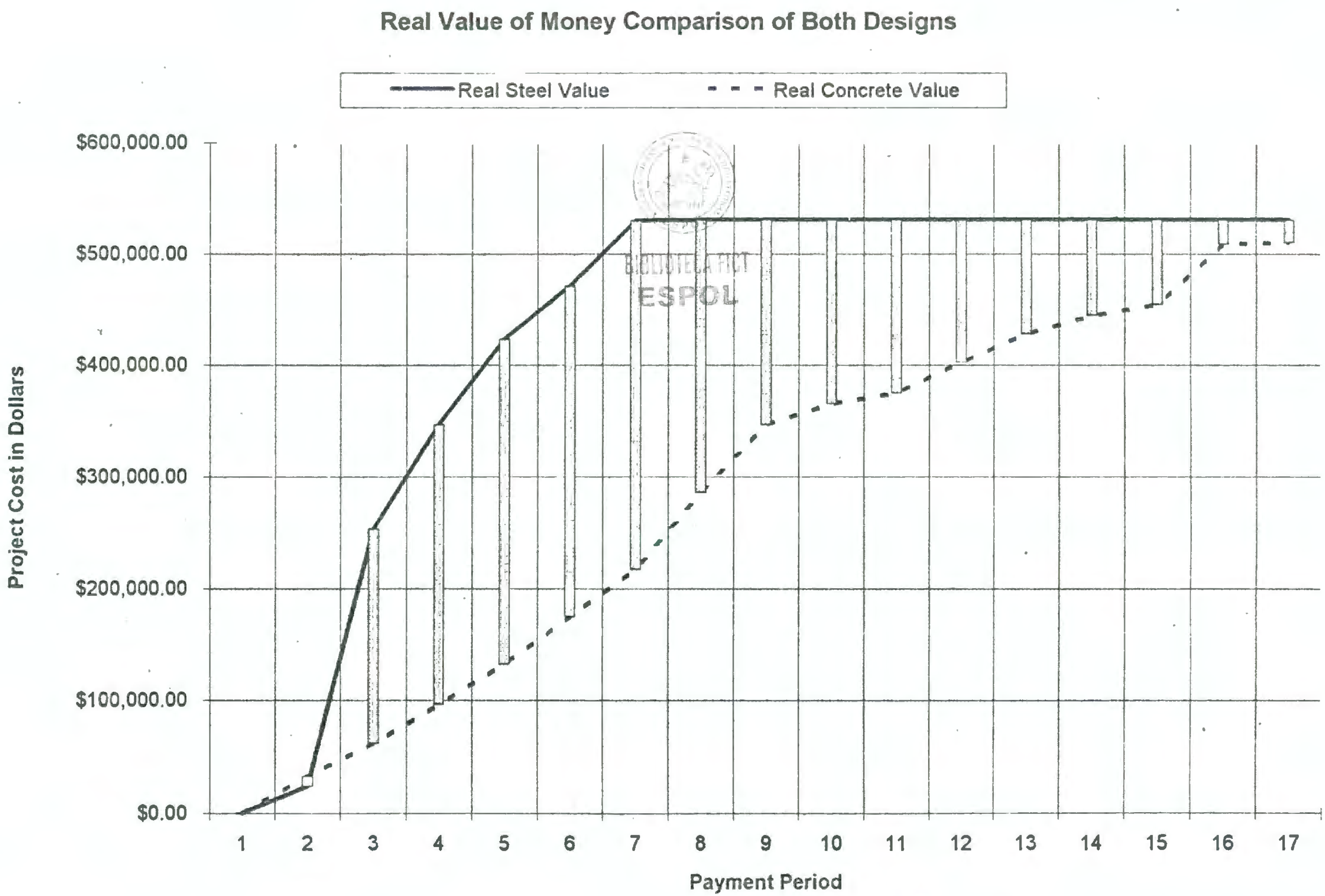
Percentage of Total Costs of Each Floor Level



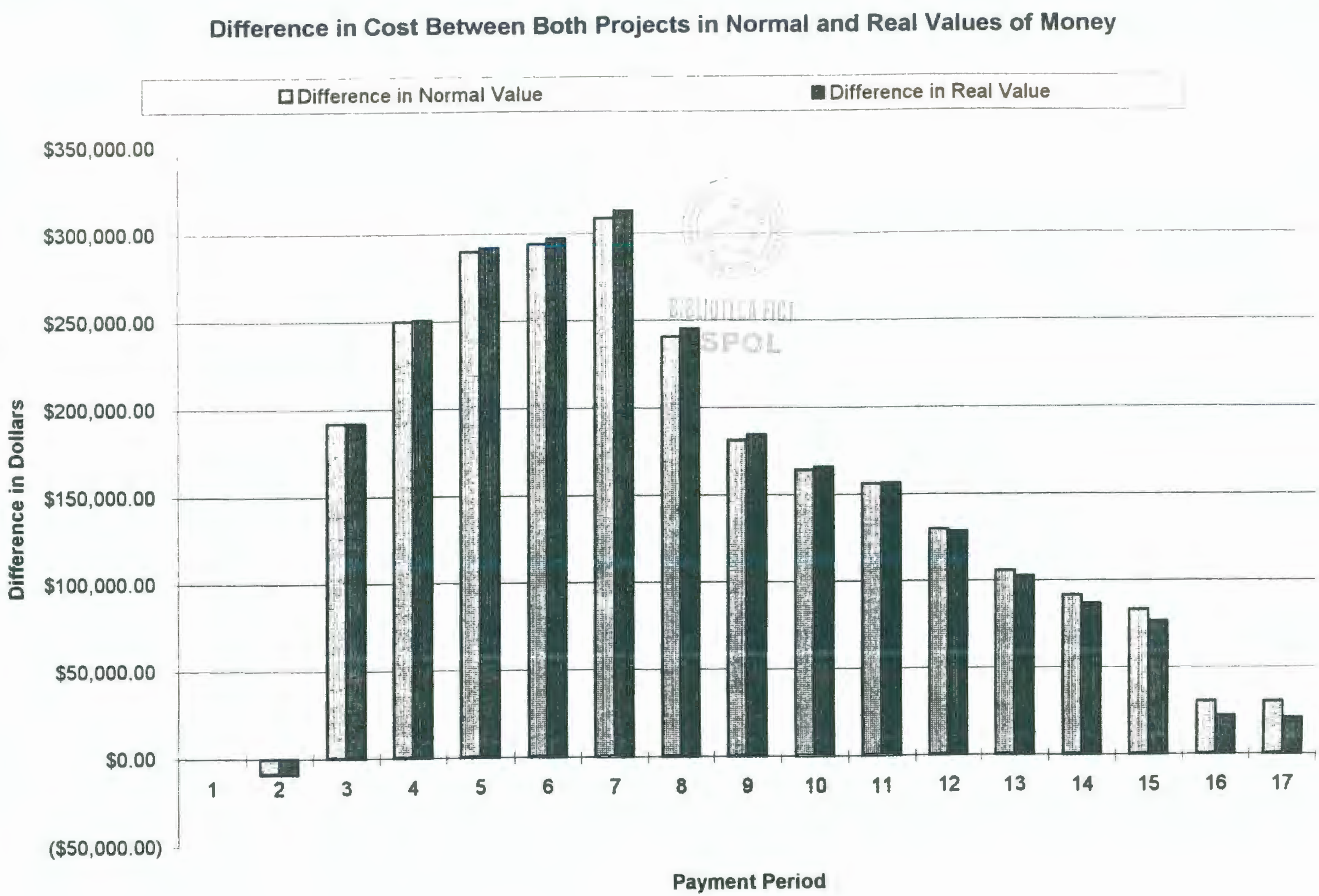
Percentage of Total Costs of Each Floor Level



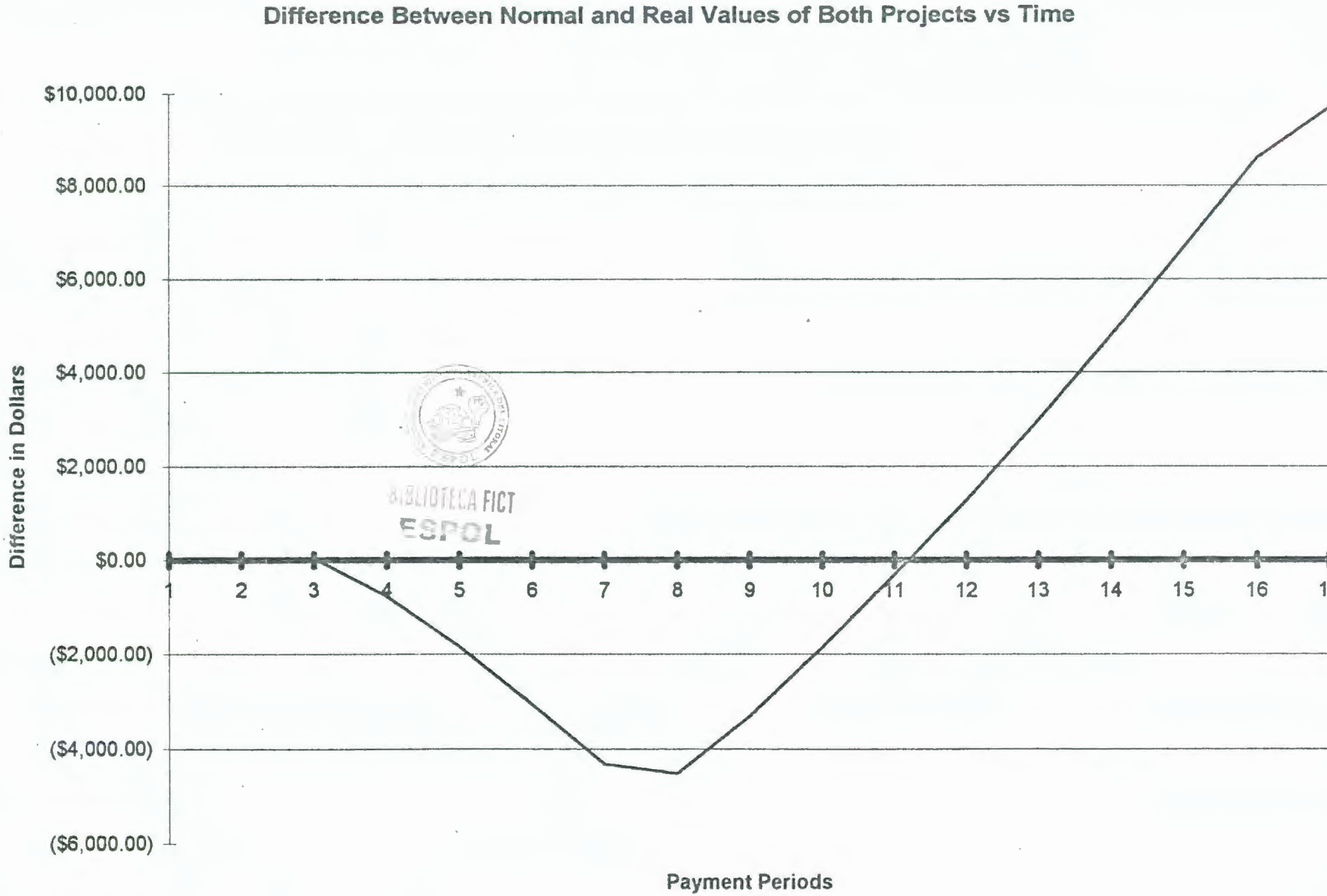

Normal Value of Money Cost Comparison of Both Designs
Normal Steel Value
Normal Concrete Value
$600,000.00
$500,000.00
$400,000.00
$300,000.00
$200,000.00
$100,000.00
$0.00
Project Cost in Dollars
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Payment Period

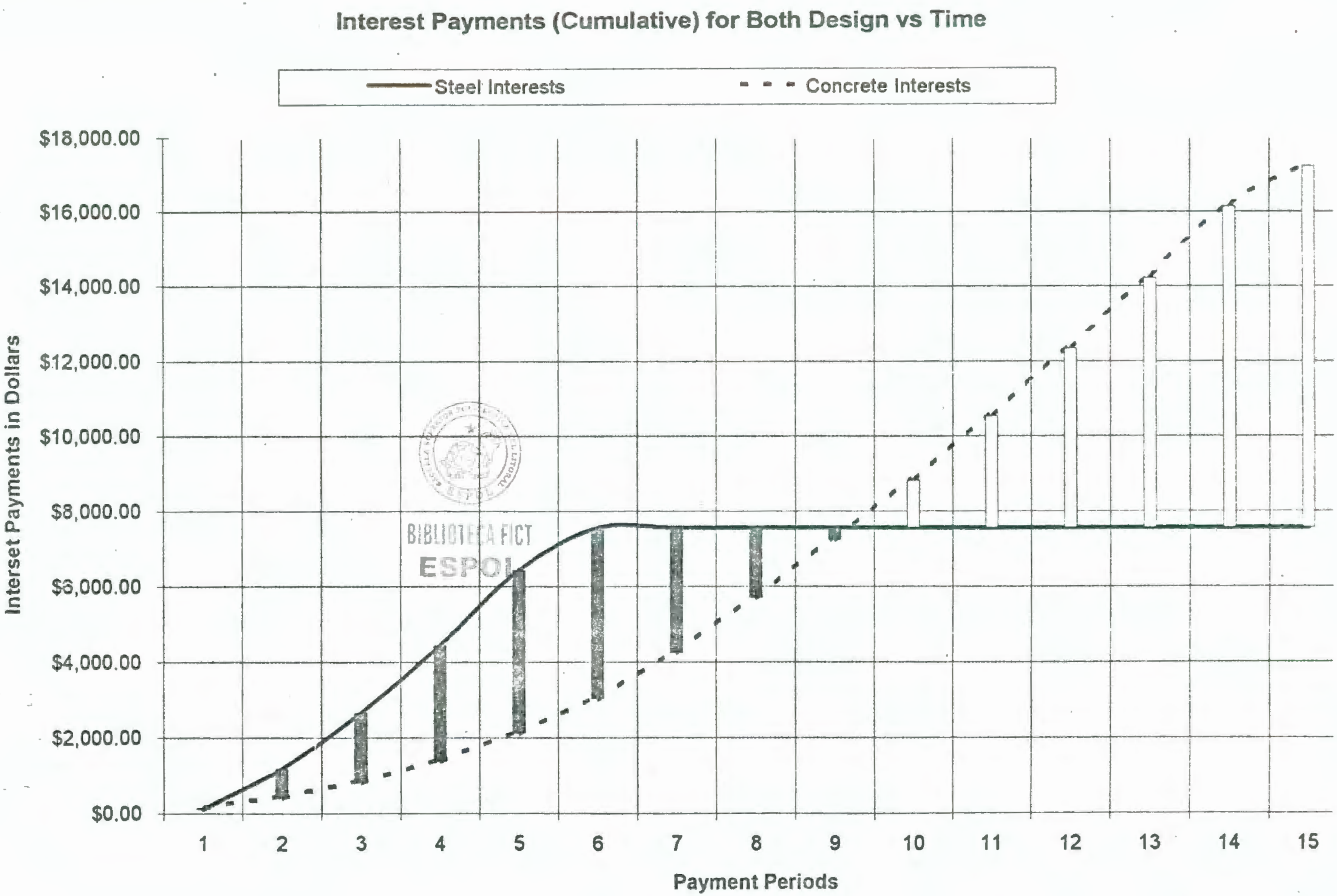


Real Value of Money Comparison of Both Designs
Real Steel Value
Real Concrete Value
$600,000.00
$500,000.00
$400,000.00
$300,000.00
$200,000.00
$100,000.00
$0.00
Project Cost in Dollars
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Payment Period



Difference in Cost Between Both Projects in Normal and Real Values of Money
Difference in Normal Value
Difference in Real Value
$350,000.00
$300,000.00
$250,000.00
$200,000.00
$150,000.00
$100,000.00
$50,000.00
$0.00
($50,000.00)
Difference in Dollars
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Payment Period


Difference Between Normal and Real Values of Both Projects vs Time
$10,000.00
$8,000.00
$6,000.00
$4,000.00
$2,000.00
$0.00
($2,000.00)
($4,000.00)
($6,000.00)
Difference in Dollars
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Payment Periods



Interest Payments (Cumulative) for Both Design vs Time
Steel Interests
Concrete Interests
$18,000.00
$16,000.00
$14,000.00
$12,000.00
$10,000.00
$8,000.00
$6,000.00
$4,000.00
$2,000.00
$0.00
Interset Payments in Dollars
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Payment Periods

CHAPTER 8

Life Cycle Issues

8. Fire Protection Analysis

Fire protection analysis is the study of how well a building will resist fire. There are two ways we can go about this. The first is to study the entire building, its resistance to fire, and suitable protection systems. The second is to study each member in the building and analyze its resistance to fire, and suitable protection for it. We chose to study each member individually.

Our reasoning for this is that we wished to do more of a frame comparison of our buildings, as opposed to a general building comparison. In this respect most interiors, (partitions, ceilings, etc.), of the two buildings, will be the same; thus the fire protection analysis for the interiors will also be the same. So it was decided only to study the frames, one of **Reinforced Concrete**, the other of **Steel**.

8.1. Fire resistance; defined:

Fire resistance is rated in hours. Each hour signifies the amount of time a member can undergo a certain temperature before yielding. Most ratings are derived from materials being tested in furnaces. The average furnace temperature at which these ratings are derived are as follows; 1000 degrees F at 5 min., 1400 degrees F at 15 min., 1550 degrees F at 30 min., 1700 degrees F at 60 min., 1850 degrees F at 120 min., 1925 degrees F at 180 min., and 2000 degrees F at 240 min.[1]

---

[1] Fire Resistance - Vol. I, pg 6 paragraph
Underwriters Laboratories Inc.

The Massachusetts State Building Code gives us the Fire grading in hours for different types, and uses of structures[2]. For residential apartments, and hotels it specifies a minimum rating of 2hrs. This being the closest to the structure we are designing, we chose to use 2hrs as our fire protecting rating.

8.2. Fire Protection Analysis

The Fire Resistance Directory is a book published by **Underwriters Laboratories Inc**. It is used to find out if a given member is suitable in its fire resistance rating, and if not it gives protection systems that may will be implemented to up its rating. By fire protection systems we mean different types of coating which can be sprayed on the members. It is this coating which will increase the time a member can be subjected to fire.

The directory is quite easy to use; first you look up the code for the type of member you wish to check on.[3] Then you open up to the section of the book corresponding to the code letter. The letters, and their corresponding members are as follows[4]:

---

[2] Massachusetts State Building Code
Section 902.0 Fire Hazard Classification.
902.1, Table 902; pg 780

[3] Fire Resistance Directory
pg 7

[4] Fire Resistance Directory
pg 7

Code	Members
A	Floor-ceiling Designs - Concrete with Cellular Steel Floor Units and Beam Support
D	Floor-Ceiling Designs - Concrete with Steel Floor Units and Beam Support
G	Floor-Ceiling Design - Concrete and Steel Joists
J or K	Floor-Ceiling Design - Precast and Field Poured Concrete
L	Floor-Ceiling Designs - Wood or combination Wood and Steel Joists Assemblies
N	Beam Designs - For Floor-Ceiling Assemblies
P	Roof-Ceiling Designs
S	Beam Designs - For Roof-ceiling Assemblies
U	Wall and Partition Designs
X	Column Designs

8.3. Fire Protection Design

After the proper code group is found, we then begin searching for a grouping of members similar to ours. Let us say we are looking up a W18x40 girder. The code for this member is N, because it is a beam for a floor assembly. Now we look through the N's until a suitable member pops up.

Starting at Design No. N745[5] we find several beam slab systems whose sketch is s similar to our own (refer to figure 8.1 on following page) . The next step is to go through the description of the beam given in the text below the sketch. After reading through several of these it is found that N753 most closely resembles our member[6].

---

[5] Fire Resistance Directory
pg 599

[6] Fire Resistance Directory
pg 604

FIGURE 8.1

min and density of 15/14 pcf respectively. For method of density determination, refer to Design Information Section.

Restrained and Unrestrained Beam Ratings, Hr.	Min Thickness, In.
1	1-1/8
1-1/2	1-3/4
2	2-1/4
3	2-7/8

Construction Products Div. W. R. Grace & Co. of Canada Ltd.—Types MK-4, MK-5.
Southwest Vermiculite Co., Inc.—Types 4, 5.
Vermiculite Products, Inc.—Types MK-4, MK-5.
Zonolite Construction Products Div. W. R. Grace & Co.—Types MK-4, MK-5, MK-6/ED, MK-6/CBF, RG and RG1.

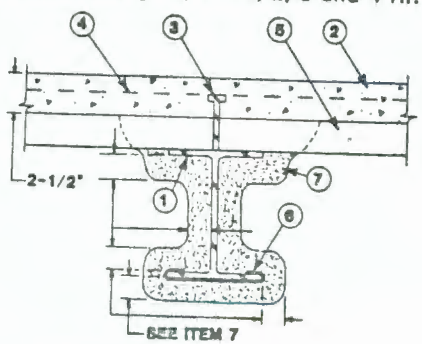
7. **Metal Lath**—(Optional)—Metal lath may be used to facilitate the spray application of cementitious mixture on steel bar joists and trusses. The diamond mesh, 3/8 in. expanded steel lath, 1.7 to 3.4 lb/sq yd is secured to one side of each steel joist with No. 18 SWG galv steel wire at joist web and bottom chord members, spaced 15 in. O.C. max. When used, the metal lath is to be fully covered with cementitious mixture with no min thickness requirements for material applied onto the lath between chords and betwen web members.

7A. **Non-Metallic Fabric Mesh**—(Optional)—As an alternate to metal lath, glass fiber fabric mesh, weighing approximately 2.5 oz/sq yd, polypropylene fabric mesh, weighing approximately 1.25 oz/sq yd or equivalent, may be used to facilitate the spray application. The mesh is secured to one side of each joist web member. The method of attaching the mesh must be sufficient to hold the mesh and the spray-applied cementitious mixture in place during application until it has cured. An acceptable method to attach the mesh is by stapling the mesh in min 1/4 in. long beads of hot-melted glue. The beads of glue shall be spaced a max of 12 in. O.C. along the top chord of the bar joist. Another method to secure the mesh is by 1-1/4 in. long by 1/2 in. wide hairpin clips formed from No. 18 SWG or heavier steel wire.

*Bearing the UL Classification Marking

Design No. N737

Restrained Beam Rating—1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)
Unrestrained Beam Rating—1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)



1. **Steel Beam**—W8X28, minimum size.
2. **Normal Weight or Lightweight Concrete**—Compressive strength, 3500 psi. For normal weight concrete either carbonate or siliceous aggregate may be used. Unit weight is 148 pcf. For lightweight concrete, unit weight is 112 pcf.
3. **Shear Connector**—(Optional)—Studs, 3/4 in. diameter headed type of equivalent per AISC specifications, welded to top flange of beam through the steel floor units.
4. **Welded Wire Fabric**—6x6, W1.4XW1.4.
5. **Steel Floor and Form Units***—1-1/2 to 3 in. corrugated, fluted or cellular units in any combination.
6. **Metal Lath**—(Optional—See tables in Item 7)3.4 lb. per sq. yd. galvanized or painted expanded steel applied only to bottom flange of beam. Secured by bending tight around flange a minimum of 1-1/2 in. toward web of beam.
7. **Cementitious Mixture***—Prepared by mixing with water according to instructions on each bag of material. Mixture is sprayed onto beam to a final minimum thickness as shown below. Crest areas above the beam shall be sealed with cementitious mixture. Beam surfaces must be free of dirt, oil or loose scale. The minimum average density shall be 50 pcf with a minimum individual density of 45 pcf. For method of density determination, see design information section. The thicknesses shown on the following table are for normal weight or lightweight concrete. Metal lath, Item 6, is required.

Rating-Hr.	Minimum Thickness - In. Restrained Beam	Unrestrained Beam
1	5/16	
1-1/2		

	Minimum Thickness - In.	
		Unrestrained Beam
		1-1/2
		2-13/16

The thicknesses shown on the following table are for normal weight or lightweight concrete. Metal lath, Item 6, is optional.

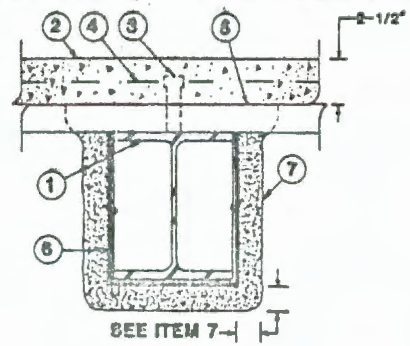
Rating-Hr.	Minimum Thickness - In. Restrained Beam	Unrestrained Beam
1	9/16	9/16
1-1/2	5/8	3/4
2	13/16	1
3	1-5/16	1-3/4

Carboline Co.—Type 240. Investigated for exterior use.

*Bearing the UL Classification Marking

Design No. N738

Restrained Beam Rating—1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)
Unrestrained Beam Rating—1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)



1. **Steel Beam**—W8X28, minimum size.
2. **Normal Weight or Lightweight Concrete**—Compressive strength, 3500 psi. For normal weight concrete either carbonate or siliceous aggregate may be used. Unit weight is 148 pcf. For lightweight concrete, unit weight is 112 pcf.
3. **Shear Connector**—(Optional)—Studs, 3/4 in. diameter headed type of equivalent per AISC specifications, welded to top flange of beam through the steel floor units.
4. **Welded Wire Fabric**—6x6, W1.4XW1.4.
5. **Steel Floor and Form Units***—1-1/2 to 3 in. corrugated, fluted or cellular units in any combination.
6. **Metal Lath**—3.4 lb. per sq. yd. galvanized or painted expanded steel. Lath lapped in and tied together with No. 18 SWG galvanized steel wire spaced 6 in. OC.
7. **Cementitious Mixture***—Prepared by mixing with water according to instructions on each bag of material. Mixture is sprayed onto beam to a final minimum thickness as shown below. Crest areas above the beam shall be sealed with cementitious mixture. Beam surfaces must be free of dirt, oil or loose scale. The minimum average density shall be 50 pcf with a minimum individual density of 45 pcf. For method of density determination, see design information section.

Rating-Hr.	Minimum Thickness - In. Restrained Beam	Unrestrained Beam
1	5/16	5/16
1-1/2	3/8	1/2
2	9/16	3/4
3	1-1/16	1-1/2
4	1-7/8	2-13/16

Carboline Co.—Type 240. Investigated for exterior use.

*Bearing the UL Classification Marking

Now we go through the table given with each of these design, that tells us what type, and how much of a certain for protection we must use for our given rating. An example of one of these tables is given below:

6. Cementitious mixture[7] - Prepared by mixing with water and liquid concentrate according to instructions on each bag of material and spray - applied to beam surfaces which are free of dirt, loose scale, and oil. The crests of corrugated and fluted units above the beam shall be 25 pcf. For method of density determination, refer to Design Information Section.

	Minimum Thickness, In.	
Rating, Hr	**Unrestrained Beams**	**Restrained Beams**
1	7/16	7/16
1-1/2	11/16	1/2
2	15/16	3/4
3	1-7/16	1-1/4
4	—	1-11/16

In certain cases a coating is not given in order to increase the rating. Such cases are for members that have concrete already in them. In these cases it may just be necessary to increase the concrete thickness. Such is the case with concrete beams, slabs, and columns; as well as steel decking with a concrete slab, or steel beams encased in concrete.

8.4. Fire Protection Design For Steel Structure

The following are the fire protection designs for the given steel members. Included are the design number given in the Fire Resistance Directory, and the type and amount of protection to be used.

[7] Fire Resistance Directory
pg 605

8.4.1. Roof Design:

Corrugated Steel Deck:

1.5" depth, inverted "B" deck, alv. 16 Gauge, span: 16.5", width: 6.75"
5/8"
puddle welds spaced approx. 12" on center

Concrete Steel Deck:

3.5" thick, reinforced w/ 66-44 Mesh (rolled in the direction of deck span)

3 ply felt & gravel:

For insulation & to protect slab form rain

Fire Protection:

Design No. P002[8]. Use Acoustical Material; 12 by 12 in. tile with T&G and Kerfed dedge detail. Border tile supported by channels made of 26 MSG painted steel formed 1-5/16 in. high by 3/4 in. wide at the top and 1 in. wide at the bottom flange. Flat splines formed of 30 MSG galv steel, 11 in. long by 7/8 in. wide inserted into kerfed tile joints perpendicular to zee splines. (S) = surface perforations, (V) = ventilating slots.

8.4.2. Rooms Area:

Concrete Slab:

4.0" thick, reinforced w/ 66-44 mesh (rolled in the direction of deck span)

---

[8] Fire Resistance Directory
pg 636

Corrugated Steel Deck:

1.5" depth, inverted "B" deck, galv. 16 Gauge, span:16.5', width:6.0'. 5/8' puddle
welds spaced approx. 12" on center

Fire Protection:

Design No. D010. The minimum concrete topping on the steel decking must be 2-1/4 inch for a 2 hr rating. Our slab is 4" thick, and thus satisfies the requirements.

8.4.3. Corridors Area: Elevators Waiting Area:

Concrete Slab:

4.5" thick reinforced w /44-44 Mesh (rolled in the direction of deck span)

Corrugated Steel Deck:

1.5" depth, inverted "B" deck, falv. 16 Gauge, span:16.5', width: 6.0'. 5/8" puddle welds spaced approx. 12" on center.

Fire Protection:

D010. See Rooms Area.

The following Table, Table 8.1 summarizes the Fire Protection Design for the Beams, Girders, and Columns.

Fire Protection Desegn For Steel Members
Table 8.1

Fire Protection for steel members					
2hr rating					
Member	Type	Design No.	Location	Fire Protection Type	Thickness
W 18x40	Girder	S722	Roof	Cementatious Mixture	1-5/8"
W 21x68	Girder	N 753	3rd, 2nd, & 1st Floors	Cementatious Mixture	3/4"
W 10x22	Beam	N 753	3rd, 2nd, & 1st Floors	Cementatious Mixture	3/4"
W 10x22	Beam	S722	Roof	Cementatious Mixture	1-5/8"
W 6x9	Beam	N 753	3rd, 2nd, & 1st Floors	Cementatious Mixture	3/4"
W 6x9	Beam	S722	Roof	Cementatious Mixture	1-5/8"
W 10x45	Interior Column	X738	Roof, 3rd, 2nd, & 1st Floors	Cementatious Mixture	1-3/4"
W 8x31	N-S columns	X 723	Roof, 3rd, 2nd, & 1st Floors	Cementatious Mixture	1-3/8"
W 8x24	E-W & Corner columns	X 722	Roof, 3rd, 2nd, & 1st Floors	Cementatious Mixture	1-11/16"
W 12x14	Stairway beams	N 753	3rd, 2nd, & 1st Floors	Cementatious Mixture	3/4"

8.5. Fire Protection Design for The Reinforced Concrete Frame

While the Fire Resistance Directory was suitable for the analysis of all the steel members, it came up short for the analysis on reinforced concrete columns. Actually there was no information on reinforced concrete columns in the book, so I needed to find a different source.

The source that I found is "**Calculation Methods for Fire Resistance**" .On pg. 37 of this document is table 2.6, which gives the fire protection requirements for reinforced concrete columns. The table is given below:

MINIMUM CONCRETE COLUMN SIZE

Concrete Aggregate Type	**Minimum column dimensions**				
	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Siliceous	8x8	9x9	10x10	12x12	14x14
Carbonate	8x8	9x9	10x10	11x11	12x12
Sand-lightweight	8x8	8.5x8.5	9x9	10.5x10.5	12x12

Table 2.6 gives no reference to fire protection, that is because none is needed. Concrete is the fire protection, in this case it is the protection for the reinforcement. If there was no reinforcement the concrete would act as its own fire protection. Thus instead of adding anything to the members, a minimum value is given to their sizing.

The reasoning for this is that concrete can withstand extremely high temperatures, for an extended amount of time. It is also a good insulator, this is why it is used to protect steel members form fire. So concrete needs nothing added to it, to increase its resistance to fire, but must be sized adequately to decrease the chances of failure.

The following question will most likely ; what shall happen if a small member is needed, and it cannot be sized any larger to fit the fire protection requirements? The solution would be to add extra concrete to the outside of the member after construction, until its thickness fits the minimum necessary size.

The following is the design analysis of the Reinforced Concrete Members, included are the design number, or table used:

8.5.1. Concrete Slab:

6.75" reinforced in N-S, and E-W directions

Fire Protection:

Design No.J900G. The slab will have a 2hr rating if it has a minimum thickness of 4 -1/2 inches of reinforced concrete. We have 6.75 inches, thus we fit the requirement.

8.5.2. Columns:

8x8 column, reinforced with **4 no. 6 bars:**

Fire Protection:

According to table 2.6[9], for a column to have a 2 hr rating it must be 10x10. Therefore all of the 8x8 columns should be changed to 10x10.

[9] Calculation Methods for Fire Resistance

12x12 column, reinforced with **4 no. 6 bars:**

Fire Protection:

According to table 2.6, this design is satisfactory for 2 hr fire protection.

8.6 Conclusion

Both buildings, after the fire protection design, now have a 2hr minimum fire rating. The comparison that now can be made, is one of cost. The Steel structure must have nearly all its structural members treated, to increase their ratings. The Reinforced Concrete building on the other hand, only has to increase its 8x8 columns, to 10 x 10.

The total effect this has, on the the cost of the structures, will be analyzed in the next chapter. It is safe to say though, that the **fire protection** for the **Steel Structure** will be considerably more expensive than that, needed for the **Reinforced Concrete** structure.

CHAPTER 9

Conclusions
&
Recommendations

Chapter 9 Conclusion

In this chapter of the MQP, we will present our conclusions and observations on both designs. We will evaluate both designs based on their properties. These properties are base cost, real cost, duration, risk, insulation, flexibility of design to changes, and operational costs. Through these properties we will compare both designs and decide on one of the two designs for recommendation.

The first property on our list is base cost. As previous tables and charts have shown, the reinforced concrete design will cost approximately $383,489.65, while the structural steel design's approximate base cost will be $457,146.44. This means that the difference between them is $73,656.79 which is a very large and considerable difference. Therefore, this evaluation yields an advantage to the reinforced concrete design.

However, this evaluation is inconclusive because it does not take into account any effect of construction loans, interests, and overhead. Therefore, a more accurate perception needs to be established, and this is achieved by evaluating the designs based on real cost which includes these parameters. This real cost could also be considered the final construction cost of the designs. In this way, the reinforced concrete design's real cost is $492,768.82 which represents an increase of $109,369.17 which reflects the effect of the contractor's 10% profit and overhead, respectively, $30,679.17 and $78,600. On the other hand, structural steel has a real cost of $523,418.16, representing an increase of $66,271.72 which includes $36,571.72 of profit and $29,700 of overhead. This property is judged by the amount of increase in the design's cost, therefore, structural steel has an advantage in this property.

The construction schedule or duration is very important in judging between designs because it influences other factors such as risk, overhead, interests on construction loans, owner's access time to his/her property, etc. So, with respect to this property, the construction schedules are 11 and 29 weeks for structural steel and reinforced concrete respectively. That means a difference of 18 weeks or 4.5 months in favor of the structural steel design. The importance of this difference, as mentioned before, is measured by the effect it has on other properties which will be discussed later.

The evaluation of risk is a very complex and complicated task because of the number of variables or unknowns involved. Some of these are weather conditions, working environment (job site congestion, project manager performance, worker productivity, schedule control, etc.), and change orders in the future. So, we will only evaluate the risk involving money which is influenced by the schedule control. In this context, the difference of 18 weeks in construction time represents an increase in risk if the concrete design is implemented. This increase in risk contemplates the larger duration of the construction loan which continues to generate interests throughout its duration and the increased probability of delays due to change orders on the project or weather delays. Therefore, it is clear that the steel design is less risky to the owner than the reinforced concrete design.

The other properties can be summarized briefly because they depend solely on the design. First, there is insulation which offers an advantage to reinforced concrete because, although the walls are the same in both designs, the floor slab thickness is greater in the reinforced concrete design. This means that there is less heat loss through the floor slabs giving the advantage to the reinforced concrete design.

Next, there is the flexibility of design which depends on the material used in construction. Steel, because it can be cut or fitted to attain different lengths and shapes, offers a complete advantage in flexibility over concrete which is already in place. However, before concrete is put in place the fact that it is formed into place would nullify this advantage but, most of the time, changes in project scope occur when its elements are already in place. Therefore, using reality as a measuring stick, steel offers an advantage in this very important property of a design.

Finally, there is the issue of operational costs. These costs include maintenance and future changes. As far as maintenance goes, the steel design has an acoustical ceiling which allows better access to the "veins and arteries" of the mechanical systems which are all the different parts of these systems. On the other hand, the reinforced concrete design, which lacks this ceiling, would have to supply extra space for these systems and this represents hidden costs which would favor a steel design. However, the extra thickness of the slabs in the reinforced concrete design means an increase in noise control within the building and, since this is a college dormitory, this is an advantage which steel does not offer.

The other factor, the ability to accommodate future changes, is similar in both designs because the interior layouts are identical in both designs. This of course, as mentioned before, excludes H.V.A.C. systems because steel can accommodate more options for its layout because of the acoustical ceiling. Therefore, both designs offer exclusive advantages which could be interpreted either way but, since the final evaluation has to do with the owner needs, flexibility for allocation of H.V.A.C. outweighs the small advantage of noise control which gives the advantage to the structural steel design.

So, in conclusion, after these evaluations have been performed the design we are inclined to recommend is **the structural steel design** because, inspite of its higher final cost, its construction schedule is shorter and this allows for a sooner occupation of the building by the owner, which translates into a sooner generation of income.

Also, a structural steel offers a reduction in construction and financial risks over reinforced concrete. This can be attributed to the regional preference of steel over concrete in Massachusetts which can influence labor productivity, building permits, insurance, labor availability, and so on.

Another relevant factor, with respect to the schedule, is the avoidance of weather related delays. Steel is relatively independent of weather with only a small decrease in productivity due to bad weather. On the other hand, concrete depends on nature to provide suitable temperatures for proper curing or the mixing plant to include in the mix additives to achieve it. Concrete admixtures are very expensive and only God controls mother nature. Therefore, we can expect a significant reduction of this risk with the implementation of a steel design because its construction schedule, due to the shorter length, can be manipulated to concur with the end of spring and beginning of the summer seasons to assure suitable weather conditions for construction. Meanwhile, the reinforced concrete schedule is larger and less flexible meaning that some of the construction would occur under more uncertain conditions increasing risk.

So, with this in mind, we believe that the **structural steel** design is the smartest choice amongst the two choices offered and have attempted to provide our conclusions and ideas to explain why this is so.

Bibliography

AISC LRFD Manual
Published; 1986.

Canadian NBC Code
Published; 1977.

Fire Resistance Directory- Vol. I
Publisher; Underwriters Laboratories Inc.,1993

Massachusetts State Building Code
Published; 1990.

Means Construction Cost Data
Published; 1994.

Sweets Catalog, file 2
Published; 1992.

" Calculation Methods for Fire Resistance"

" Foundations Engineering", **Hanson, Peck, and Thornburn**
Publisher; New York Wiley and Sons Inc.

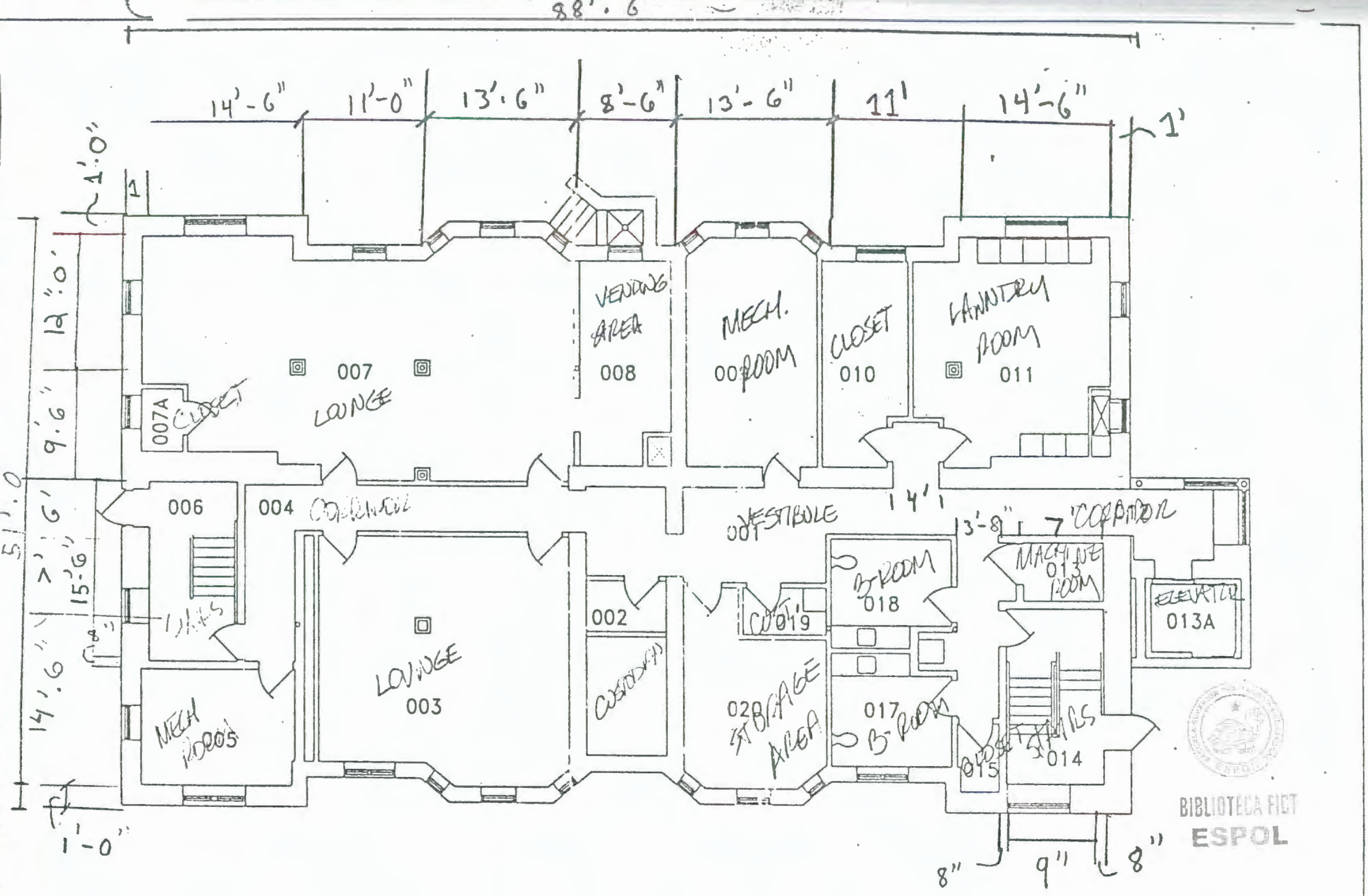
" Reinforced Concrete; Mechanics and Design", **James G. Macgregor**
Published; 1992.

" Structural Steel Design: LRFD Approach ", **John Smith**
Published; 1991.

" The Growth of the WPI Campus - IQP", **Michael Schorr and Cristopher Riley**.
Published; 1990.

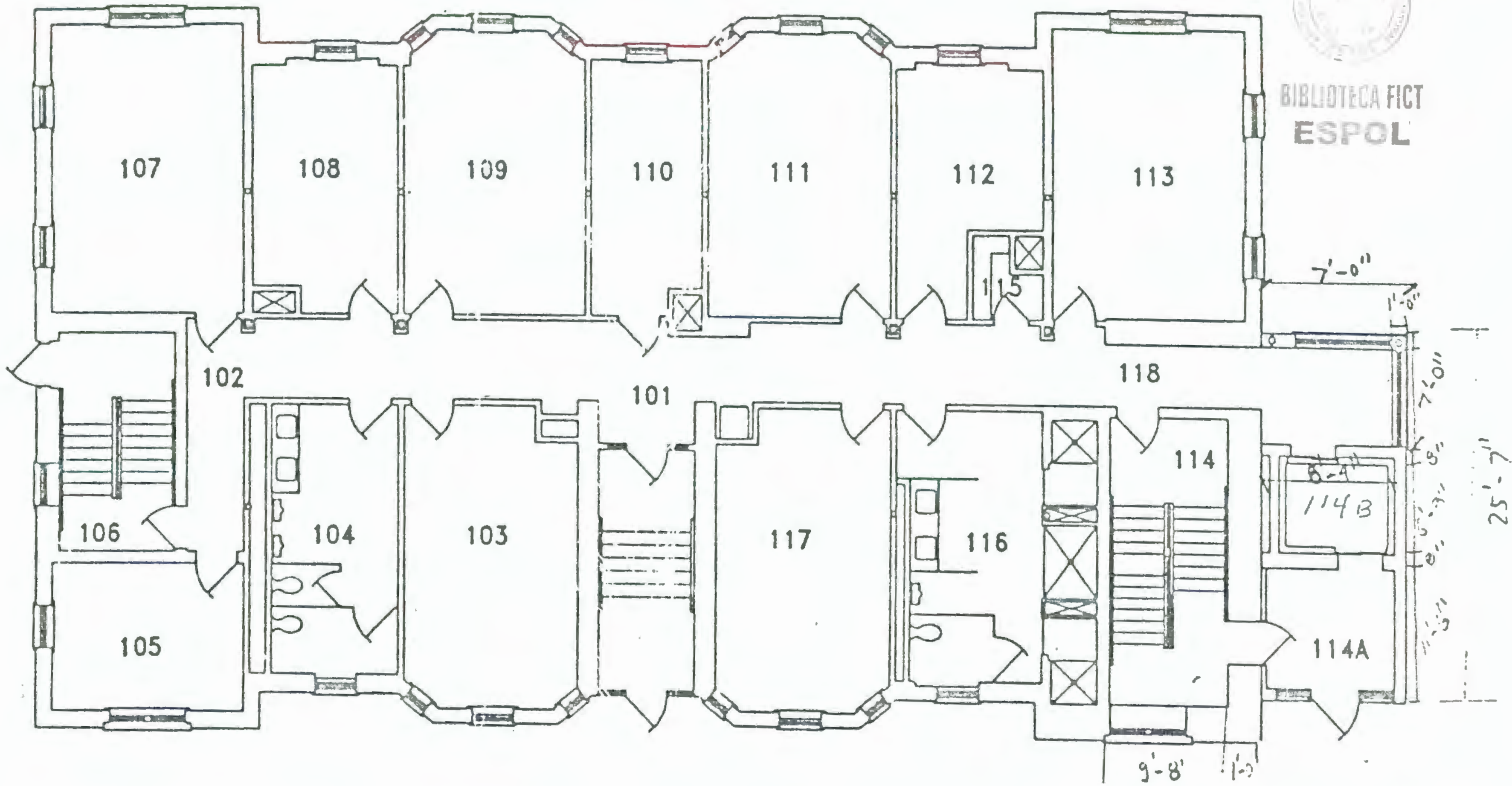
Appendix A

Building Drawings



WORCESTER
POLYTECHNIC
INSTITUTE

Institute Hall Dormitory BASEMENT	DRAWN BY W. BARRY	REV. 8/26/94	SHEET 1 OF 4
	DATE 7/30/93	SCALE 1" = 12'	



WORCESTER POLYTECHNIC INSTITUTE

Institute Hall Dormitory FIRST FLOOR	DRAWN BY W. BARRY	REV.	SHEET 2 OF 4
	DATE 7/30/93	SCALE 1" = 12'	

Appendix B

Mass Code
&
Canadian Code

Massachusetts State Building Code 1990

(Relevant Pages)

901.2 Penetrations: Plans for buildings more than two stories in height shall indicate where penetrations will be made for electrical, mechanical, plumbing and communications conduits, pipes and systems, and shall also indicate the materials and methods for maintaining the required structural integrity, fireresistance rating and firestopping.

SECTION 902.0 FIRE HAZARD CLASSIFICATION

902.1 General: The degree of fire hazard of buildings and structures for each specific use group as defined by the fire grading in Table 902 shall determine the requirements for fire walls, fire separation walls, and horizontal and vertical assemblies separating mixed uses as prescribed in Section 313.0 and all structural members supporting such elements unless otherwise provided for in this code.

902.2 Unclassified uses: The building official shall determine the fire hazard classification of a building or structure designed for a use not specifically provided in Table 902 in accordance with the fire characteristics and potential fire hazard of the use group which it most nearly resembles; or its designation shall be fixed by the approved rules.

Table 902
FIRE GRADING OF USE GROUPS

Use Group	Description	Fire grading on hours
A-1	Assembly, theaters	3
A-2	Assembly, night clubs	3
A-3	Assembly, recreation centers, lecture halls, terminals, restaurants	2
A-4	Assembly, churches	1½
B	Business	2
E	Educational	1½
F	Factory and industrial	3
H	High hazard	4
I-2	Institutional, incapacitated	2
I-3	Institutional, restrained	3
M	Mercantile	3
R-1	Residential, hotels	2
R-2	Residential, multi-family dwellings	1½
R-3	Residential, 1- and 2- family dwellings	1
S-1	Storage, moderate hazard	3
S-2	Storage, low hazard	2

safe in all its parts, adequate for its existing use, and the public safety is not endangered thereby.

SECTION 1106.0 UNIFORMLY DISTRIBUTED LIVE LOADS

1106.1 Uniform live load: The plans for all buildings and structures intended for other than R-3 and R-4 use groups shall specify the live and partition loads for which each floor or part thereof has been designed. The minimum uniformly distributed live load in pounds per square foot (psf) shall be as provided in Table 1106, and for all concentrated loads wherever they occur as provided in Section 1107.0.

Table 1106
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS

Occupancy and use	Live load (psf)
Apartments (see Residential)	
Armories and drill rooms	150
Assembly halls and other places of assembly:	
Fixed seats	60
Movable seats	100
Platforms (assembly)	100
Balcony and Open decks	60 (or as required by occupancy load)
Bowling alleys, poolrooms, and similar recreational areas	75
Cornices	75
Court rooms	100
Corridors:	
First floor	100
Other floors, same as occupancy served except as indicated	
Dance halls and ballrooms	100
Dining rooms and restaurants	100
Dwellings (see Residential)	
Elevator machine rooms	150
File and computer rooms in all building types	Unit load based on anticipated occupancy
Fire escapes:	100
On multi or single family residential buildings only	40
Garages (passenger cars only)	50
(For trucks and buses use AASTHO[a] lane loads)	
(see Table 1107 for concentrated load requirements)	
(see Section 1110.1 for roofs)	

Table 1106 (continued)
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOAD

Occupancy or use	Live load (psf)
Grandstands (see reviewing stands)	
Gymnasiums, main floors and balconies	100
Hospitals:	
Operating rooms, laboratories	100
Private rooms	40
Wards	40
Corridors, above first floor	80
Hotels (see residential)	
Libraries:	
Reading rooms	60
Stack rooms (books & shelving at 40 pcf) but not less than	150
Corridors, above first floor	80
Manufacturing:	
Light	125
Heavy	150 (min) or occupancy load, if greater
Marquees	75
Office buildings:	
Offices	50
Lobbies	100
Corridors, above first floor	80
Open parking structures (passenger cars only)	50
Penal institutions:	
Cell blocks	40
Corridors	100
Residential:	
Multi family:	
Private apartments	40
Public rooms	100
Corridors, balconies, open decks	80
1 & 2 family dwellings:	
First floor	40
Second floor and habitable attics	30
Unhabitable attics[b]	20
Open decks or balconies	60
Hotels:	
Guest rooms	40
Public rooms	100
Corridors, balconies or open decks serving public rooms	100

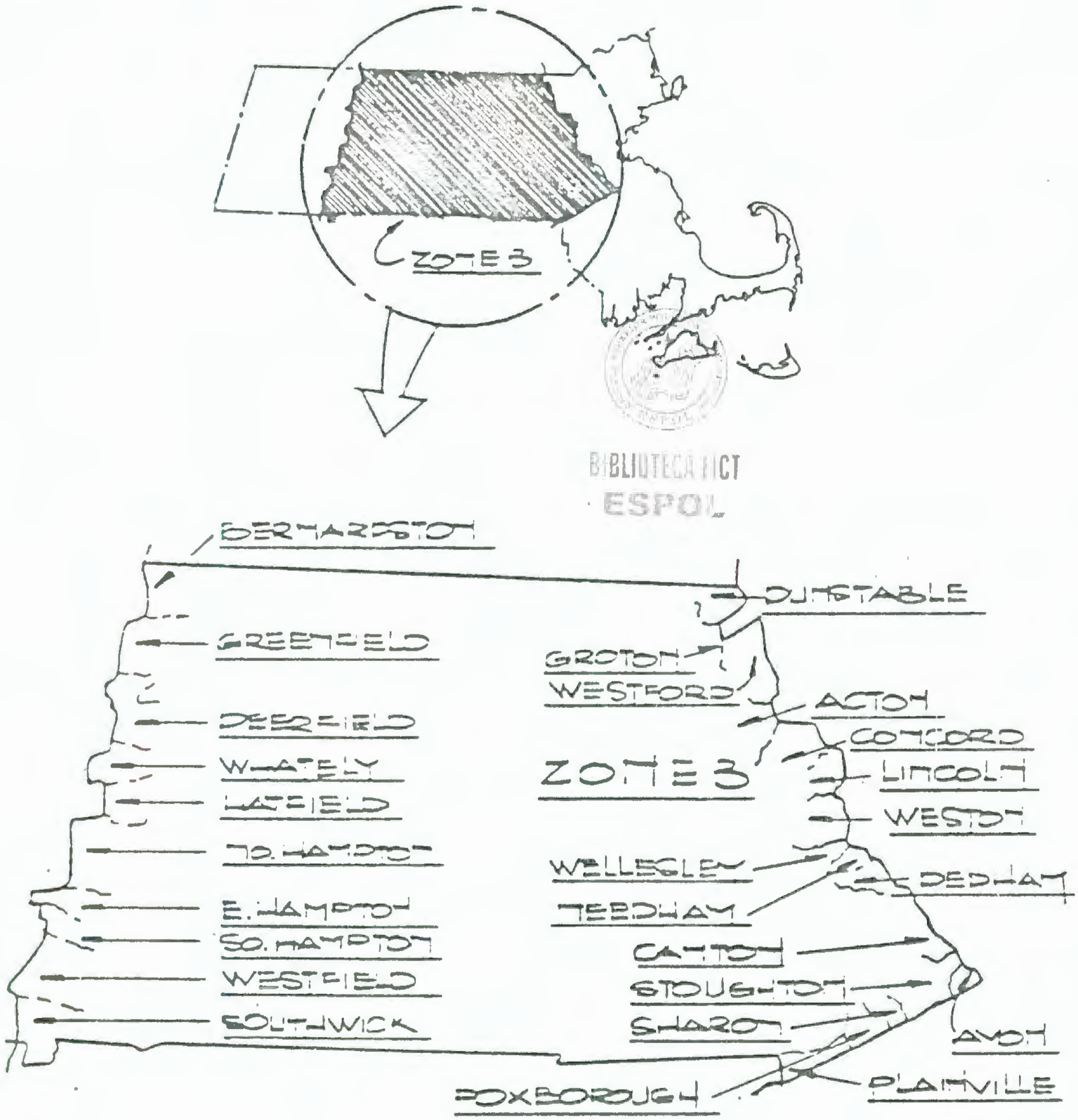


Figure 1111.1C
SNOW LOAD MAP - ZONE 3

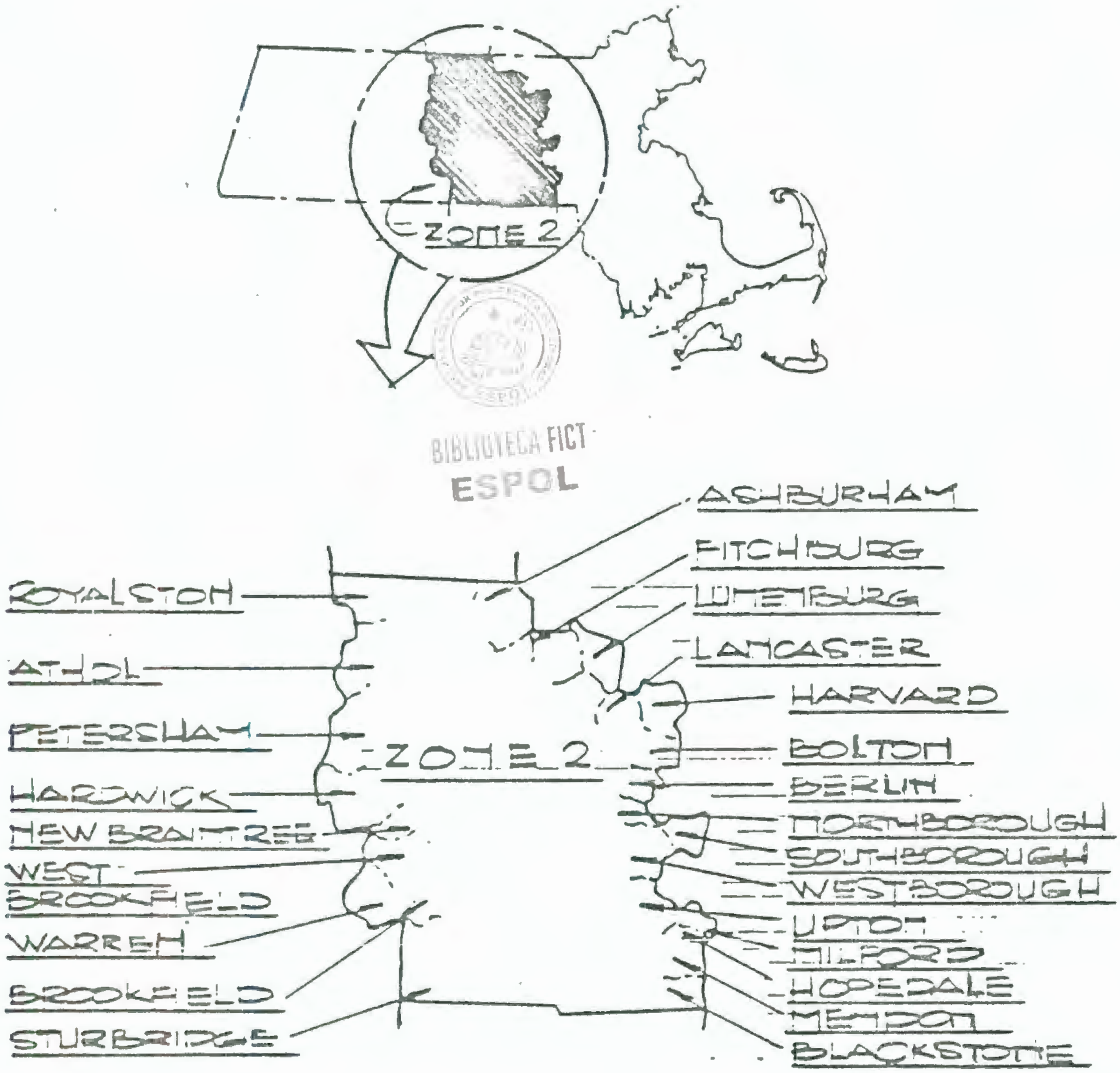


Figure 1112.1B
WIND LOAD MAP - ZONE 2

Table 1112.1
REFERENCE PRESSURE (POUNDS PER SQUARE FOOT)

H (feet) Height above grade	Zone 1 Exposure			Zone 2 Exposure			Zone 3 Exposure		
	A	B	C	A	B	C	A	B	C
0-50	11	12	12	11	17	17	14	21	21
50-100	11	12	18	11	17	24	14	21	31
100-150	11	16	22	14	21	29	18	26	37
150-200	13	18	25	17	24	33	22	30	41
200-250	15	20	27	20	27	36	25	34	45
250-300	17	22	29	22	30	39	28	37	48
300-400	19	25	31	25	33	42	32	41	52
400-500	22	28	34	29	37	46	26	46	57
500-600	24	30	37	33	41	49	41	51	61
600-700	27	33	39	36	44	52	45	55	65
700-800	29	35	41	39	47	55	48	58	68
800-900	31	37	43	41	49	57	52	62	72
900-1000	33	39	45	44	52	59	55	65	74

See Table 1112.1a, next page, for empirical wind pressure formulas.

the direction of each of the main axes of the structure in accordance with the following formula:

$$V = \frac{1}{3} KCSW$$

1113.4.1.1 C factor: The value of C shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

For all one- and two-story buildings or structures the value of C shall be zero point one (0.1). The maximum value of C need not exceed zero point one (0.1).

T is the fundamental period of vibration of the structure in seconds in the direction under consideration. Properly substantiated technical data for establishing the period T may be submitted. In the absence of such data, the value for T for buildings shall be determined by the following formula:

$$T = \frac{0.05 h_n}{\sqrt{D}}$$

Exception: In all buildings which the lateral force resisting system consists of a moment-resisting space frame which resists one hundred (100) percent of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements would tend to prevent the frame from resisting lateral forces.

$$T = 0.010N$$

1113.4.1.2 K factor: The horizontal force factors K for structures meeting the requirements of Section 1113.5 are set forth in Table 1113.1.

Table 1113.1
HORIZONTAL FORCE FACTORS "K" FOR STRUCTURES

Type of arrangement of existing element	Value of K
Buildings with a box system as defined in Section 201.0	1.33
Buildings with a dual bracing system as defined in Section 201.0	0.80
Buildings with a moment-resisting space frame designed to resist the total required lateral force	0.67
Other buildings	1.00
Elevated tanks plus full contents on four (4) or more cross-braced legs and not supported by a building[2]	3.00[3]
Structures other than buildings and other than those set forth in Table 1113.1	2.00

Note 1. Where wind load would produce higher stresses, the wind load shall be used in lieu of the loads resulting from earthquake forces.
Note 2. The minimum value of KC shall be zero point twelve (0.12) and the maximum value of KC need not exceed zero point twenty-five (0.25).
Note 3. The tower shall be designed for accidental torsion of five (5) percent as specified in Section 1113.4.3. Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described above shall be designed in accordance with Section 1113.4.5 using $C_p = 0.2$.

1113.4.1.3 S Factor: The S Factor shall have the following values according to the types of soil sites as defined below.

Soil Site S_1, S = 1
Soil Site S_2, S = 1.2
Soil Site S_3, S = 1.5

Values other than those tabulated may be used provided they are based on studies by a registered professional engineer and are not less than 1.0. The values of CS need not exceed zero point twelve (0.12).

For the purposes of determining the S Factor, the following types of soils sites are defined according to the materials encountered below the foundation level.

Soil Site S_1: Bedrock of any type including material Classes 1 through 4 of Table 1201.

Stiff soil conditions where the soil depth below foundation level is less than 200 ft. and the soil types overlying bedrock consist of glacial till; gravel or well graded sand and gravel, sands that are not susceptible to liquefaction in accordance with Section 1113.8, clay having an undrained shear strength of at least one thousand (1,000) psf, dense silts and compacted granular fill provided that fill soils are compacted throughout as required in Section 1201.3.1.

Soil Site S_2: Soil sites that cannot be classified as Soil Sites S_1 or S_3.

Soil Site S_3: Soil profiles that contain 30 ft or more of soft clays having an undrained shear strength smaller than 1,000 psf, loose silts, organic soils, loose sands, or miscellaneous fill.

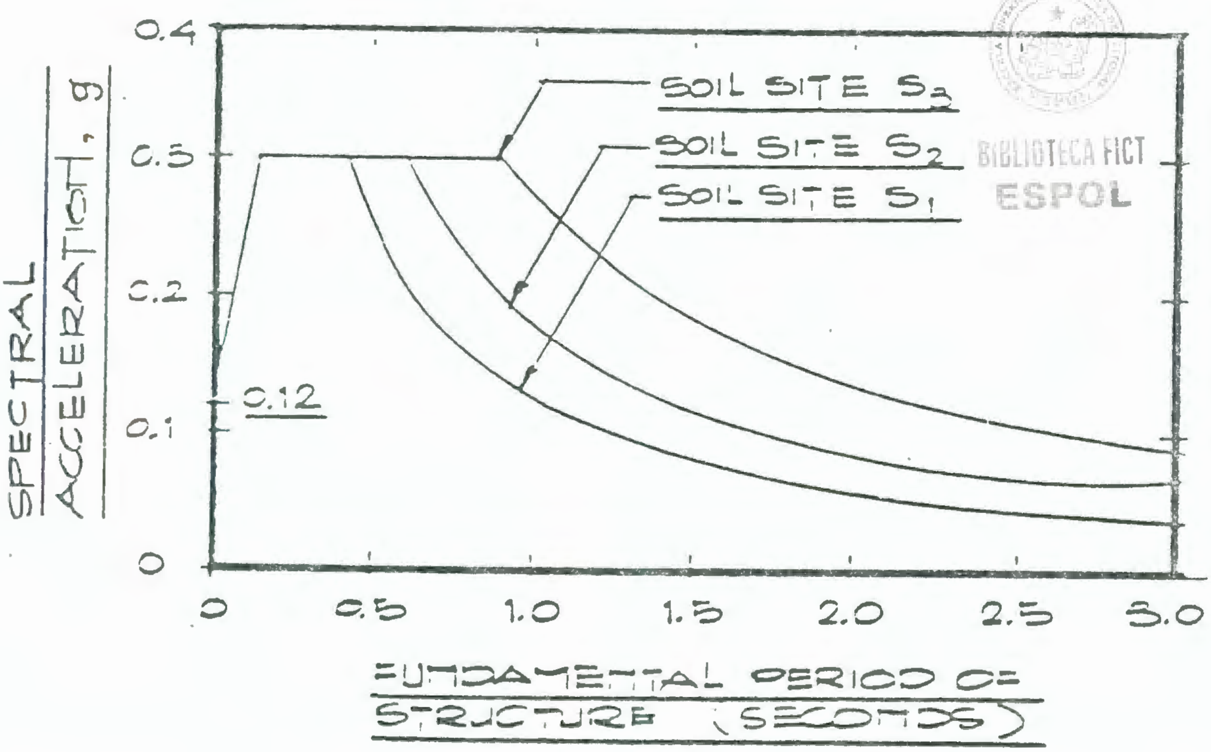


Figure 1113.1
DESIGN RESPONSE SPECTRUM

Table 1201
ALLOWABLE BEARING PRESSURES FOR FOUNDATION MATERIALS

Material Class	Description	Notes	Consistency in Place[1]	Allowable Net Bearing Pressure (tons/ft^2)
1a	Massive bedrock: Granite, diorite gabbro, basalt, gneiss	3	Hard, sound rock , minor jointing	100
1b	Quartzite, well cemented conglomerate	3	Hard, sound rock moderate jointing	60
2	Foliated bedrock: slate, schist	3	Medium hard rock, minor jointing	40
3	Sedimentary bedrock: cementation shale, siltstone, sandstone, limestone, dolomite, conglomerate	3,4	Soft rock, moderate jointing	20
4	Weakly cemented sedimentary bedrock: compaction shale or other similar rock in sound condition	3	Very soft rock	10
5	Weathered bedrock: any of the above except shale.	3,5	Very soft rock, weathered and/or major jointing and fracturing	8
6	Slightly cemented sand and/or gravel, glacial till (basal or lodgement), hardpan	7,8	Very dense	10

Material Class	Description	Notes	Consistency in Place[1]	Allowable Net Bearing Pressure (tons/ft^2)
7	Gravel, widely graded sand and gravel; and granular ablation till	6,7,8	Very dense Dense Medium dense Loose Very loose	8 6 4 2 Note 11
8	Sands and nonplastic silty sands with little or no gravel (except for Class 9 materials)	6,7,8,9	Dense Medium dense Loose Very loose	4 3 2 Note 11
9	Fine sand, silty fine sand, and nonplastic inorganic silt	6,7,9	Dense Medium dense Loose Very loose	3 2 1 Note 11
10	Inorganic sandy or silty clay, clayey sand, clayey silt, clay, or varved clay; low to high plasticity	5,6,10	Hard Stiff Medium Soft	4 2 1 Note 11
11	Organic soils: peat, organic silt, organic clay	11	---------	Note 11

Notes on Table 1201:

1. Refer to commentary in Appendix D regarding typical index test values that may be helpful as guides for evaluation of consistency in place.
2. Refer to Section 1206.0 for determination of design loads and for special cases.
3. The allowable bearing pressures may be increased by an amount equal to ten (10) percent for each foot of depth below the surface of sound rock; however, the increase shall not exceed two (2) times the value given in the table.
4. *For limestone and dolomite, the bearing pressures given are acceptable only if an exploration program performed under the direction of a registered professional engineer demonstrates that there are no cavities within the zone of influence of the foundations. If cavities exist, a special study of the foundation conditions is required.*
5. Weathered shale and/or weathered compaction shale shall be included in Material Class 10. Other highly weathered rocks and/or residual soils shall be treated as soil under the appropriate description in Material Classes 6 to 10. Where the transition between residual soil and bedrock is gradual, a registered professional engineer shall make a judgment as to the appropriate bearing pressure.

minus the weight of the excavated material, induce a maximum stress greater than three hundred (300) pounds per square foot at mid-depth of the underlying soft clay layer.

Settlement analysis will be based on a computation of the new increase in stress that will be induced by the structure and realistically appraised live loads, after deducting the weight of excavated material under which the soil was fully consolidated. The effects of fill loads within the building area or fill and other loads adjacent to the building shall be included in the settlement analysis. The appraisal of the live loads may be based on surveys of actual live loads of existing buildings with similar occupancy. The soil compressibility shall be determined by a registered professional engineer.

1204.6 Disturbance of bearing materials: Whenever the bearing materials are disturbed from any cause, for example, by the inward or upward flow of water and/or by construction activities, the extent of the disturbance shall be evaluated by a registered professional engineer and appropriate remedial measures taken satisfactory to the building official.

SECTION 1205.0 DEPTH OF FOOTING

1205.1 Frost protection: All permanent supports of buildings and structures shall extend a minimum of four (4) feet below finished grade except when erected upon sound bedrock or when protected from frost, or when the foundation grade is established by a registered professional engineer and as approved by the building official. The engineer shall show supporting data including the type and extent of free-draining foundation material, ground water levels, and climatic records.

1205.2 Isolated footing: Footings on granular soil of Classes 7 to 9 of Table 1201 and compacted fill shall be so located that the line drawn between the lower edges of adjoining footings shall not have a steeper slope than thirty (30) degrees with the vertical, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner.

1205.3 Depth of spread foundations: The bottom surface of any footing resting on material of Classes 5 to 10 of Table 1201, inclusive, shall be at least eighteen (18) inches below the lowest ground surface or the surface of a floor slab bearing directly on the soil immediately adjacent to the footing.

SECTION 1206.0 FOOTING DESIGN

1206.1 Design loads: The loads to be used in computing the pressure upon bearing materials directly underlying foundations shall be the live and dead loads of the structure, as specified in Section 1115.0 including the weight of the foundations and of any immediately overlying material, but deducting from the resulting pressure per square foot the total weight of a one-(1) foot-square column of soil, including the water in its voids, which extends from the lowest immediately adjacent surface of the soil to the bottom of the footing, pier or mat. Foundations shall be constructed so as to resist the maximum probable hydrostatic pressures.

1206.2 Pressure due to lateral loads: Where the pressure on the bearing material due to wind or other lateral loads is less than one-third (⅓) of that due to dead and live loads, it may be neglected in the foundation design. Where this ratio exceeds one-third (⅓), foundations shall be so proportioned that the pressure due to combined dead, live, wind loads, and other lateral loads shall not exceed the allowable bearing pressures by more than one-third (⅓).

1206.3 Earthquake loads: Special provisions shall be made in the foundation design to comply with the provisions of Section 1113.0.

1206.4 Vibratory loads: Where machinery or other vibrations may be transmitted through the foundations, consideration shall be given in the design of the footings to prevent detrimental disturbances of the soil.

1206.5 Varying unit pressures: Footings shall be so designed that the unit soil pressure under the dead load shall be as uniform as possible under all parts of the building structure. When necessary for stability in the structure due to settlement or varying soil conditions, approved variations are permitted in the unit pressure under different footings.

1206.6 Eccentric loads: Eccentricity of loadings in foundations shall be fully investigated, and the maximum pressure on the basis of straight-line distribution shall not exceed the allowable bearing pressures.

SECTION 1207.0 TIMBER FOOTINGS AND WOOD FOUNDATIONS

1207.1 Timber footings: Timber footings are permitted for buildings of Type 5 construction and as otherwise approved. Such footings shall be treated in accordance with AWPA C2 or C3 listed in Appendix A. Treated timbers are not required when placed entirely below permanent water level, or when used as capping for wood piles which project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported upon piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the NFoPA *National Design Specification for Wood Construction* listed in Appendix A.

1207.2 Pole buildings: Pole-type buildings shall be designed and erected in accordance with AWPI *Pole Building Design* listed in Appendix A. The poles shall be treated in accordance with AWPA C2 or C4 listed in Appendix A.

1207.3 Wood foundations: Wood foundation systems shall be designed and installed in accordance with NFoPA TR7 listed in Appendix A. All lumber and plywood shall be treated in accordance with AWPB-FDN listed in Appendix A and shall be identified as to conformance with such standards by an approved agency.

SECTION 1208.0 STEEL GRILLAGES

1208.1 General: All steel grillage beams shall be separated with approved steel spacers and shall be entirely encased in at least 3 inches of concrete and the spaces between the beams shall be completely filled with concrete or cement grout. When used on yielding soils, steel grillages shall rest on approved concrete beds not less than 6 inches thick.

SECTION 1209.0 CONCRETE FOOTINGS

1209.1 Concrete strength: Concrete in footings shall have a specified compressive strength of not less than 2,500 psi at 28 days.

1209.2 Design: Concrete footings shall comply with Article 15 and ACI 318 listed in Appendix A.

1209.3 Thickness: The thickness of concrete footings shall comply with Section 1209.3.1 and 1209.3.2.

1209.3.1 Plain concrete: In plain concrete footings, the edge thickness shall be not less than 8 inches for footings on soil; except that for buildings of Use Group R-3 and buildings less than two stories in height of Type 5 construction, the required edge thickness shall be reduced to 6 inches provided the footing does not extend beyond 4 inches on either side of the supported wall.

1209.3.2 Reinforced concrete: In reinforced concrete footings, the thickness above the bottom reinforcement shall be not less than 6 inches for footings on soil, nor less than 12 inches for footings on piles. The clear cover on reinforcement where the concrete is cast against the earth shall not be less than 3 inches. Where concrete is exposed to soil after it has been cast, the clear cover shall be not less than 1½ inches for reinforcement of No. 5 bars or ⅝ inch diameter wire or smaller, nor less than 2 inches for larger reinforcement.

1209.4 Deposition: Concrete footings shall not be poured through water unless otherwise approved. When poured under or in the presence of water, the concrete

ARTICLE 15

CONCRETE

SECTION 1500.0 CONCRETE DESIGN STANDARDS

1500.1 Reinforced and prestressed concrete: Structural members of reinforced concrete, including prestressed concrete, shall be designed and constructed in accordance with the provisions of this article and ACI 318 listed in Appendix A, hereafter referred to in this article as ACI 318.

1500.2 Plain concrete: Structural members of plain concrete shall be designed and constructed in accordance with the provisions of this article and ACI 318.1 listed in Appendix A. Concrete that is either unreinforced or contains less reinforcement than the minimum specified for reinforced concrete by ACI 318 shall be classified as plain concrete. Plain concrete shall not be used for structural members where special design considerations are required for earthquake or blast forces, unless specifically approved.

SECTION 1501.0 SEISMIC DESIGN PROVISIONS

See Section 1113.0 for all seismic design criteria.

SECTION 1502.0 MATERIALS

1502.1 General: Materials used to produce concrete and admixtures for concrete shall comply with the requirements of this section and ACI 318.

1502.2 Cements: Cement shall conform to ASTM C 150 listed in Appendix A, or to such other cements listed in ACI 318.

1502.3 Aggregates: Concrete aggregates shall conform to ASTM C33 or to ASTM C330 listed in Appendix A.

1502.3.1 Special tests: Aggregates failing to meet the specifications listed in Section 1502.3 shall not be used unless approved and shown by special test or actual service to produce concrete of adequate strength and durability.

reinforcing and when it is demonstrated that adequate encasement of the bars at the splice can be achieved.

1508.5 Rebound: Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be reused as aggregate.

1508.6 Joints: Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless all edges are sloped to a thin edge. For structural elements which will be under compression and for construction joints shown on the approved plans, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

1508.7 Damage: Any in-place shotcrete which exhibits sags or sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced.

1508.8 Curing: During the curing periods specified herein, shotcrete shall be maintained above 40 degrees F. (4 degrees C.) and in a moist condition.

1508.8.1 Initial curing: Shotcrete shall be kept continuously moist for 24 hours after shotcreting is completed or shall be sealed with an approved curing compound.

1508.8.2 Final curing: Final curing shall continue seven days after shotcreting, or for three days if high-early-strength cement is used, or until the specified strength is obtained. Final curing shall consist of either the initial curing process or the shotcrete shall be covered with an approved moisture-retaining cover.

1508.8.3 Natural curing: Natural curing shall not be used in lieu of that specified above unless the relative humidity remains at or above 85 percent, and is authorized by the design architect/engineer, and approved by the building official

1508.9 Strength test: Strength test of shotcrete shall be made in accordance with the quality assurance provisions of ACI 506.2 listed in Appenaix A.

SECTION 1509.0 MINIMUM SLAB THICKNESS

1509.1 General: The thickness of concrete floor slabs supported directly on the ground shall be not less than 3 ½ inches. An approved vapor barrier with joints lapped not less than 6 inches shall be placed between the base course or subgrade and the concrete floor slab.

ARTICLE 18

STEEL

SECTION 1800.0 GENERAL

1800.1 Scope: The provisions of this article shall govern the materials, design, construction and quality of steel structural members.

SECTION 1801.0 STRUCTURAL STEEL CONSTRUCTION

1801.1 General: Structural steel construction used in all buildings and structures shall be fabricated from materials of uniform quality, free from defects that would vitiate the strength or stability of the structure. All structural steel shall be designed and constructed in accordance with either the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* or the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* listed in Appendix A.

1801.2 Temporary and special stresses: Due provision shall be made in the design for temporary stresses occurring during erection, and for the influence of special loads producing impact or vibrations as provided in Article 11. Stresses caused by eccentric loading shall be fully provided for and eccentric details shall be shown on the design and shop drawings.

1801.3 Shop drawings: Complete shop drawings shall be prepared in conformance with the best modern practice in advance of the actual fabrication. Such drawings shall clearly distinguish between shop and field rivets, bolts and welds in all connections and details.

1801.4 Painting and special protection: All painting shall comply with the requirements contained in either of the AISC design specifications referenced in Section 1801.1. When exposed to highly corrosive fumes or vapors, or subject to destruction from other highly hazardous industrial processes, all structural steelwork shall be protected by an approved method.

Canadian NBC Code 1977

(Relevant Pages)

SUBSECTION 4.1.8. EFFECTS OF WIND

External pressure or suction

4.1.8.1.(1) The specified external pressure or suction due to wind on a *building* as a whole or on cladding shall be calculated from

$$p = qC_eC_gC_p$$

where p = the specified external pressure acting statically and in a direction normal to the surface either as a pressure (directed towards the surface) or as a suction (directed away from the surface),

q = the reference velocity pressure as provided for in Sentence (3),

C_e = the exposure factor as provided for in Sentence (4),

C_g = the gust effect factor as provided for in Sentence (5),

C_p = the external pressure coefficient for the cladding location considered or the shape factor for the *building* as a whole. The shape factor is equal to the algebraic difference of the external pressure coefficients for the windward and leeward sides of the *building*.

(Information on pressure coefficients can be found in the Commentary on Wind Loads in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Internal pressure or suction on cladding

(2) The net specified pressure due to wind on cladding shall be the algebraic difference of the external pressure or suction as provided for in Sentence (1) and the specified internal pressure or suction due to wind calculated from either

(a) $p_i = qC_eC_{pi}$

or

(b) $p_i = qC_eC_gC_{pi}$

where p_i = the specified internal pressure acting statically and in a direction normal to the cladding either as a pressure (directed outwards) or as a suction (directed inwards),

q, C_e, C_g are as provided for in Sentences (3), (4) and (5) respectively, except that C_e shall be evaluated at the *building* mid-height instead of the height of the element considered, and

C_{pi} = the internal pressure coefficient.

Formula (b) shall be used if the *building* has large openings such that the effects of wind gusts are transmitted to the internal air space of the *building*. In the design of cladding adequate allowance shall be made for regions of high local external pressures or suctions."

(Information on pressure coefficients can be found in the Commentary on Wind Loads in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(3) The reference velocity pressure q is the appropriate value specified in the Table of Climatic Data in Part 2 of this Bylaw for the following conditions:

(a) the reference velocity pressure q for the design of cladding shall be based on a probability of being exceeded in any one year of 1 in 10,

(b) the reference velocity pressure q for the design of structural members for deflection and vibration shall be based on a probability of being exceeded in any one year of 1 in 10,

(c) for all *buildings*, except those listed in Clause (d), the reference velocity pressure q for the design of structural members for strength shall be based on a probability of being exceeded in any one year of 1 in 30, and

(d) the reference velocity pressure q for the design of structural members for strength for *post-disaster buildings* shall be based on a probability of being exceeded in any one year of 1 in 100.

(4) The exposure factor C_e shall be

(a) the value shown in Table 4.1.8.A. for the appropriate height of the surface or part of the surface,

(b) the value of the function: $(h/30)^{1/5}$ but not less than 1.0 where h is the height above *grade* in feet of the surface or part of the surface, or

(c) if a dynamic approach to the action of wind gusts is used, an appropriate value depending on both height and shielding.

(Information on a dynamic approach can be found in the Commentary on Wind Loads in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Table 4.1.8.A.

Forming Part of Sentence 4.1.8.1.(4)

Height, ft	Exposure Factor
0 to 40	1.0
Over 40 to 60	1.1
" 60 to 90	1.2
" 90 to 130	1.3
" 130 to 190	1.4
" 190 to 270	1.5
" 270 to 420	1.6
" 420 to 740	1.8
" 740 to 1,200	2.0
Column 1	2

Gust effect factor

(5) The gust effect factor C_g is one of the following values:
(a) 2.0 for structural members,
(b) 2.5 for small elements including cladding, or
(c) if a dynamic approach to the action of wind gusts is used, an appropriate value depending on the turbulence of the wind and the size and natural frequency of the structure.

(Information on a dynamic approach to the action of wind gusts can be found in the Commentary on Wind Loads in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Dynamic effects of wind

4.1.8.2.(1) *Buildings* whose height is greater than 4 times their minimum effective width or greater than 400 ft and other *buildings* whose light weight, low frequency and low damping properties make them susceptible to vibration shall be
(a) designed by experimental methods for the danger of dynamic overloading and vibration and the effects of fatigue, or
(b) designed using a dynamic approach to the action of wind gusts.

(Information on dynamic approach to the action of wind gusts can be found in the Commentary on Wind Loads in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Full and partial loading

4.1.8.3.(1) *Buildings* and structural members shall be capable of withstanding the effects of
(a) the full wind load over the entire area, or
(b) 0.75 times the full wind load acting over any portion of the area and full load on the rest of the area,

whichever produces the greatest effect on the *building* or member concerned.

Interior walls and partitions

4.1.8.4.(1) In the design of interior walls and *partitions* due consideration shall be given to differences in air pressure on opposite sides of the wall or *partition* which may result from
(a) pressure differences between the windward and leeward sides of a *building*,
(b) stack effects due to a difference in air temperature between the exterior and interior of the *building*, and
(c) air pressurization by the mechanical services of the *building*.

SUBSECTION 4.1.9. EFFECTS OF EARTHQUAKES

Types of analysis

4.1.9.1.(1) The specified loading due to earthquake motion shall be determined
(a) by the analysis given in this Subsection, or
(b) by a dynamic analysis provided that the acceleration ratio, A, is not less than that given in the Table of Climatic Data in Part 2 of this Bylaw and provided that the dynamically determined value of V is not less than 90 per cent of that determined by the analysis of Clause (a).

(Information on an appropriate dynamic approach consistent with the specified acceleration ratio including a recommended response spectrum and ductility factors can be found in the Commentary on Dynamic Analysis for the Seismic Response of Buildings in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Nomenclature

(2) In this Subsection

A = acceleration ratio = the ratio of the specified horizontal ground acceleration to the acceleration due to gravity.

D = the dimension of the *building* in a direction parallel to the applied forces.

D_n = plan dimension of the *building* in the direction of the computed eccentricity.

D_s = the dimension of the lateral force-resisting system in a direction parallel to the applied forces.

e = computed eccentricity between the centre of mass and centre of rigidity at the level being considered.

e_x = design eccentricity at level x.

F = foundation factor as given in Sentence 4.1.9.1.(9).

F_t = portion of V to be concentrated at the top of the structure as defined in Sentence 4.1.9.1.(11).

F_x = lateral force applied to level x.

h_i, h_n, h_x = the height above the base (i = 0) to level "i", "n" or "x", respectively.

I = importance factor of the structure as described in Sentence 4.1.9.1.(8).

J = numerical reduction coefficient for base overturning moment as defined in Sentence 4.1.9.1.(14).

J_x = numerical reduction coefficient for moment at level "x" as defined in Sentence 4.1.9.1.(15).

K = numerical coefficient that reflects the material and type of construction, damping, ductility and/or energy-absorptive capacity of the structure as given in Sentence 4.1.9.1.(7).

Level i = any level in the *building*, i = 1 first level above the base.

Level n = that level which is uppermost in the main portion of the structure.

Level x = that level which is under design consideration.

M_{tx} = torsional moment at level x.

N = the total number of *storeys* above exterior *grade* to level "n". (N is usually numerically equal to n.)

S = seismic response factor for the structure as defined in Sentence 4.1.9.-1.(5).

S_p = horizontal force factor for part or portion of a structure, as given in Table 4.1.9.C.

T = fundamental period of vibration of the *building* or structure in seconds in the direction under consideration.

V = minimum lateral seismic force at the base of the structure.

V_p = lateral force on a part of the structure.

W = *dead load* including the following:
25 per cent of the design snow load specified in Subsection 4.1.7.; for areas used for storage, the full design live load modified according to Sentence 4.1.6.3.(4); the full contents of any tanks.

W_i, W_x = that portion of W which is located at or is assigned to level "i" or "x", respectively.

W_p = the weight of a part or portion of a structure, e.g. cladding, *partitions* and appendages.

(3) Earthquake forces shall be assumed to act in any horizontal direction. Except where required otherwise by the *authority having jurisdiction*, independent design about each of the principal axes shall be considered to provide adequate resistance in the structure for earthquake forces applied in any direction. Direction of forces

(4) The minimum lateral seismic force, V, assumed to act nonconcurrently in any direction on the *building* shall be equal to the product of Lateral seismic force

$$A \cdot S \cdot K \cdot I \cdot F \cdot W$$

where A is the acceleration ratio, given in the Table of Climatic Data in Part 2 of this Bylaw, and the value of this ground acceleration is assumed constant within each seismic zone as defined in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977."

(Within seismic Zone 3, peak horizontal ground accelerations corresponding to 1 in 100 probability of annual exceedance can be larger than the assumed constant value. Information concerning the calculation of acceleration for special sites is contained in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(5) The seismic response factor, S, shall be equal to $0.5/(T^{1/3})$ but need not exceed 1.00. Seismic response factor

(6) Except where technical data proves otherwise, the fundamental period, T, in Sentence (5) shall be equal to $0.05h_n/\sqrt{D}$ where h_n and D are in feet, except that where the lateral force-resisting system consists of a moment-resisting space frame which resists 100 per cent of the required lateral forces and the frame is not enclosed by or adjoined by more rigid elements that would tend to prevent the frame from resisting lateral forces, the fundamental period, T, shall equal 0.1 N. Fundamental period

(7) Values of the numerical coefficient, K, shall conform to Table 4.1.9.A. Types of construction

(8) The importance factor, I, shall equal 1.3 for all *post-disaster buildings* and schools, and 1.0 for all other *buildings*. Importance factor

(9) The foundation factor, F, shall conform to Table 4.1.9.B., except that the product FS need not exceed 1. Foundation factor

Table 4.1.9.A.

Forming Part of Sentence 4.1.9.1.(7)

Case[1]	Type or Arrangement of Resisting Elements	Value of K
1	*Buildings* with a ductile moment-resisting space frame[2],[3] with the capacity to resist the total required force.	0.7
2	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and ductile flexural walls[4] designed in accordance with the following criteria: The frames and ductile flexural walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the flexural walls and frames. In this analysis the maximum shear in the frame must be at least 25 per cent of the total base shear.	
3	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and shear walls[5] or steel bracing designed in accordance with the following criteria: (a) The shear walls or steel bracing acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force, but in no case shall the ductile moment-resisting space frame have a lower capacity than that required in accordance with the relative rigidities.	0.8
4	*Buildings* with ductile flexural walls[4] and *buildings* with ductile framing systems not otherwise classified in this Table as Cases 1, 2, 3 or 5.	1.0
5	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame with masonry infilling designed in accordance with the following criteria: (a) The wall system comprising the infilling and the confining elements acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.	1.3
6	*Buildings* (other than Cases 1, 2, 3, 4 and 5) of (a) continuously reinforced concrete, (b) structural steel, and (c) reinforced masonry shear walls.	1.3[6]
7	*Buildings* of unreinforced masonry and all other structural systems except Cases 1 to 6 inclusive and those set forth in Table 4.1.9.C.	2.0
Col. 1	2	3

Table 4.1.9.A. (Cont'd)

Case	Type or Arrangement of Resisting Elements	Value of K
8	Elevated tanks plus full contents, on 4 or more cross-braced legs and not supported by a *building*, designed in accordance with the following criteria: (a) The minimum and maximum value of the product SKI shall be taken as 1.2 and 2.5 respectively. (b) For overturning, the factor J as set forth in Sentence 4.1.9.1.(14) shall be 1.0. (c) The torsional requirements of Sentence 4.1.9.1.(15) shall apply.	3.0
Column 1	2	3

Notes to Table 4.1.9.A.:

(1) Explanatory notes on the various cases can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977."

(2) A space frame is a 3 dimensional structural system composed of interconnected members laterally supported so as to function as a complete self-contained unit with or without horizontal diaphragms.

(3) A ductile moment-resisting space frame is a space frame that is designed to resist the specified seismic forces and that, in addition, has adequate ductility or energy-absorptive capacity.
(Information on ductile moment-resisting space frames can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(4) A ductile flexural wall is a ductile flexural member cantilevering from the *foundation* consisting of a ductile reinforced concrete wall designed and detailed according to CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.

(5) Shear walls may be either flexural walls or shear walls as defined in CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.

(6) Except as required by Sentence 4.1.9.3.(1).

Weight of structure

(10) The weight, W, of the structure shall be calculated in accordance with the following formula:

$$W = \sum_{i=1}^{n} W_i$$

Distribution of lateral seismic force

(11) The total lateral seismic force, V, shall be distributed as follows:

(a) a portion F_t shall be assumed to be concentrated at the top of the structure and equal to $0.004V(h_n/D_s)^2$, except that F_t need not exceed 0.15 V and may be considered as zero for $(h_n/D_s) \leqslant 3$.

(b) the remainder, $V - F_t$, shall be distributed along the height of the *building* including the top level in accordance with the following formula:

$$F_x = (V - F_t)\, W_x h_x / \left(\sum_{i=1}^{n} W_i h_i\right), \text{ and}$$

(c) the total shear in any horizontal plane shall be distributed to the various elements of the lateral force-resisting system in proportion to their rigidities with due regard to the capacities and stiffnesses of the nonstructural elements.

Table 4.1.9.B.

Forming Part of Sentence 4.1.9.1.(9)

Type and Depth of *Soil*[1]	F
Rock, dense and very dense coarse-grained *soils,* very stiff and hard fine-grained *soils;* compact coarse-grained *soils* and firm and stiff fine-grained *soils* from 0 to 50 ft deep	1.0
Compact coarse-grained *soils,* firm and stiff fine-grained *soils* with a depth greater than 50 ft; very loose and loose coarse-grained *soils* and very soft and soft fine-grained *soils* from 0 to 50 ft deep	1.3
Very loose and loose coarse-grained *soils,* and very soft and soft fine-grained *soils* with depths greater than 50 ft	1.5[2][3]
Column 1	2

Notes to Table 4.1.9.B.:

[1] *Soil* depth shall be measured from *foundation* or *pile* cap level. Descriptive terminology relating to the *soils* is as defined in Section 4.2.

[2] Where *soil* deposits are of the order of 300 ft or more, amplification factors greater than those given in the Table may arise in the case of tall *buildings.*

[3] The possibility of ground failure beneath the structure due to excessive settlement or liquifaction of very loose sands and loss of strength of sensitive clays shall be considered.

(12) Parts of *buildings* as described in Table 4.1.9.C. and their anchorage shall be designed for a lateral force, V_p, equal to AS_pW_p, distributed according to the distribution of mass of the element under consideration.

(13) The values of S_p in Sentence (12) shall conform to Table 4.1.9.C.

Overturning

(14) The overturning moment, M, at the base of the structure shall be multiplied by a reduction coefficient, J, where

(a) $J = 1$ where T is less than 0.5,

(b) $J = (1.1 - 0.2T)$ where T is at least 0.5, but not more than 1.5, and

(c) $J = 0.8$ where T is greater than 1.5.

(15) The overturning moment M_x at any level x shall be multiplied by J_x where

$$J_x = J + (1\text{-}J)\,(h_x/h_n)^3$$

The incremental changes in the design overturning moments, in the *storey* under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided. Where a vertical-resisting element is discontinuous, the overturning moment carried by the lowest *storey* of that element shall be carried down as loads to the *foundation.*

Table 4.1.9.C.

Forming Part of Sentence 4.1.9.1.(13)

Category	Part or Portion of *Building*	Direction of Force	Value of S_p
1	All exterior and interior walls except those of category 2 and 3	Normal to flat surface	2
2	Cantilever parapet and other cantilever walls except retaining walls	Normal to flat surface	10
3	Exterior and interior ornamentations and appendages	Any direction	10
4	Towers, *chimneys,* smokestacks, all when less than 10 ft high above the *building,* machinery, fixtures and equipment, pipes, tanks plus contents and penthouses, all when connected to or forming part of a *building*	Any direction	2[(1),(2)]
5	Towers, *chimneys* and smokestacks more than 10 ft high above the *building*	Any direction	3[(3)]
6	Tanks plus contents when resting on the ground	Any direction	1[(2)]
7	Floors and roofs acting as diaphragms	Any direction	1[(4)]
8	Connections for exterior and interior walls and elements, except those forming part of the main structural system	Any direction	25
Column 1	2	3	4

Notes to Table 4.1.9.C.:

(1) When h/D of any *building* is equal to or greater than 5 to 1, increase value by 50 per cent.

(2) The value shall be increased 50 per cent for pipes and containers for toxic or explosive materials, for materials having a flash point below 100°F or for firefighting fluids.

(3) Lower values of S_p may be used if they can be proven by analysis.

(4) Floors and roofs acting as diaphragms shall be designed for a minimum force corresponding to a value of $S_p = 1$ applied to loads tributary from that *storey,* unless a greater force F_x is assigned to the level under consideration as in Sentence 4.1.9.1.(11).

Torsional moments

(16) Torsional moments in the horizontal plane of the *building* shall be computed in each *storey* using the following formula:

$$M_{tx} = \left(V - \sum_{i=1}^{x} F_i\right) e_x$$

(Severe modal coupling may occur in symmetrical or nearly symmetrical structures when the fundamental lateral and torsional periods are nearly equal. Information on this phenomenon is given in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Design eccentricity

(17) The design eccentricity, e_x, in Sentence (16) shall be computed by one of the following equations, whichever provides the greater stresses:

(a) $e_x = 1.5e + 0.05D_n$, or
(b) $e_x = 0.5e - 0.05D_n$

(18) When the maximum design eccentricity exceeds $0.25D_n$,

(a) a dynamic analysis shall be made, or
(b) the adverse effects of torsion as computed in Sentence 4.1.9.1.(16) shall be doubled.

(Information on a dynamic analysis can be found in the Commentary on Dynamic Analysis for the Seismic Response of Buildings in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Setbacks

(19) The *building* design shall take full account of the possible effects of setbacks.

(A definition of setback together with a recommended design procedure for *buildings* having setbacks is contained in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

General provisions

4.1.9.2.(1) Lateral deflections of a *storey* relative to its adjacent *storeys* shall be considered in accordance with accepted practice.

(2) Lateral deflections of a *storey* relative to its adjacent *storeys* obtained from an elastic analysis using the loads given in Sentence 4.1.9.1. (11) shall be multiplied by 3 to give realistic values of anticipated deflections.

(3) All portions of the structure shall be designed to act as integral units in resisting horizontal forces, unless separated by adequate clearances which permit horizontal deflections of the structure consistent with values of deflections calculated in accordance with Sentence 4.1.9.2.(2).

(4) The nonstructural components shall be designed so as not to transfer to the structural system any forces unaccounted for in the design, and any interaction of rigid elements such as walls and the structural system shall be designed so that the capacity of the structural system is not impaired by the action or failure of the rigid elements.

(5) To prevent collision of *buildings* in an earthquake, adjacent structures shall either be separated by twice the sum of their individual deflections obtained from an elastic analysis using the loads given in Sentence 4.1.9.1.(11) or shall be connected to each other.

(6) The method of connection in Sentence (5) shall take into account the mass, stiffness, strength, ductility and anticipated motion of the connected *buildings* and the character of the connection.

(7) The connected *buildings* in Sentence (5) shall be assumed to have a K value equal to that of the least ductile of the *buildings* connected, unless a lower value can be justified by rational analysis.

(8) Except in seismic Zone 0, *pile* footings of every *building* or structure shall be interconnected continuously by ties in at least 2 directions, designed to carry by tension or compression a horizontal force equal to 10 per cent of the larger *pile* cap loading, unless it can be demonstrated that equivalent restraints can be provided by other means.

Special Provisions

4.1.9.3.(1) *Buildings* more than 3 *storeys* in height in seismic Zones 2 and 3 shall have a structural system as described in Cases 1, 2, 3, 4, 5 and 6 of Table 4.1.9.A. In addition, for *buildings* in seismic Zone 3 more than 200 ft in height and with a structural system of Case 6 the value of K shall be increased to 2.0.

(2) The design for any structural system which has an assigned value of K of 1 or less, shall ensure that when any member yields the remaining members of the structure shall be capable of resisting 25 per cent of the design seismic force including the effects of torsion.

(3) For *buildings* in Zones 2 and 3 in which discontinuities in columns or shear walls occur, special design provisions shall be made to ensure that failure at the point of discontinuity will not occur before the capacity of the remaining portion of the structure has been realized.

(4) In seismic Zones 2 and 3, reinforcement conforming to Clause 3.1.19. of CSA S304-1976, "Masonry Design and Construction in Buildings" shall be provided for masonry construction in
- (a) *loadbearing* and lateral load-resisting masonry,
- (b) masonry enclosing elevator shafts and stairways, or used as *exterior cladding*, and
- (c) masonry *partitions*, except for *partitions* which
  - (i) do not exceed 40 lb per sq ft in weight, and
  - (ii) do not exceed 10 ft in height and are laterally supported at the top.

SUBSECTION 4.1.10. OTHER EFFECTS

Loads on guards

4.1.10.1.(1) The minimum specified load applied horizontally and normal to the span at the top of every required *guard* shall be
- (a) 40 lb/lineal ft for exterior balconies of individual residential units and a concentrated load of 200 lb applied concurrently,
- (b) 100 lb/lineal ft for *exits* and stairs,
- (c) 150 lb/lineal ft for *assembly occupancies*, except for grandstands and stadia,
- (d) 250 lb/lineal ft for grandstands and stadia including ramps,
- (e) 300 lb/lineal ft for vehicle guard rails for parking garages applied 21 in. above the roadway and minimum total load of 2,500 lb uniformly distributed over each vehicle space applied 21 in. above the roadway, and
- (f) a 125-lb concentrated load applied at any point for industrial catwalks and other areas where crowding by many people is very improbable.

(2) Individual elements within the *guard*, including solid panels and pickets, shall be designed for 20 psf or 100 lb of concentrated load at any point in the element, whichever results in the more critical loading condition. The loads need not be considered to act simultaneously with the loads provided for in Sentence (1) and (3).

(3) The minimum specified load applied vertically at the top of every required *guard* shall be 100 lb/lineal ft and need not be considered to act simultaneously with the horizontal load provided for in Sentence (1).

Inertia sway forces

4.1.10.2. The floor assembly and other structural elements that support fixed seats in any building used for *assembly occupancies* to accommodate large numbers of people at one time, such as grandstands, stadium and theatre balconies, shall be designed to resist a horizontal force equal to at least 20 lb per lineal foot of seats acting parallel to each row of seats, and at least 10 lb per lineal foot acting at right angles to each row of seats, assuming such forces to be acting independently of each other.

Impact and vibrations

4.1.10.3. The minimum specified load due to equipment, machinery or other objects or persons that may produce impact, is the total of the weight of the equipment or machinery plus its maximum lifting capacity, or the appropriate *live load*, multiplied by an appropriate factor listed in Table 4.1.10.A. Where dynamic effects such as resonance and fatigue are likely to be important as a result of vibration of equipment or machinery, a dynamic analysis shall be carried out.

Table 4.1.10.A.

Forming Part of Article 4.1.10.3.

Impact Due to	Factor
Operation of motor driven cranes	1.25
Operation of hand driven cranes	1.10
Live loads on hanger supported floors and stairs	1.33
Operation of elevators	See CSA B44-1975, Clause 2.6.2.
Supports for light machinery, shaft or motor driven	1.20
Supports for reciprocating machinery or power driven units	1.50
Column 1	2

Horizontal crane loads

4.1.10.4.(1) The minimum horizontal specified loads on crane runway rails are
(a) the lateral force which shall be
  (i) for power-operated crane trolleys, 20 per cent, and for hand operated trolleys, 10 per cent, of the sum of the weights of the lifted loads and of the crane trolley excluding other parts of the crane,
  (ii) applied at the top of the rail, one-half on each side of the runway, and
  (iii) considered as acting in either direction normal to the runway rail, and
(b) the longitudinal force which shall be
  (i) 10 per cent of the maximum wheel loads of the crane, and
  (ii) applied at the top of the rail.

SECTION 4.2 FOUNDATIONS

SUBSECTION 4.2.1. GENERAL

4.2.1.1. This Section applies to excavations and *foundation* systems for *buildings*.

SUBSECTION 4.2.2. DOCUMENTS, APPROVAL AND INSPECTION

Subsurface investigation

4.2.2.1. Unless exempted by the *authority having jurisdiction* a *subsurface investigation*, including *groundwater* conditions, shall be carried out under the direction of a

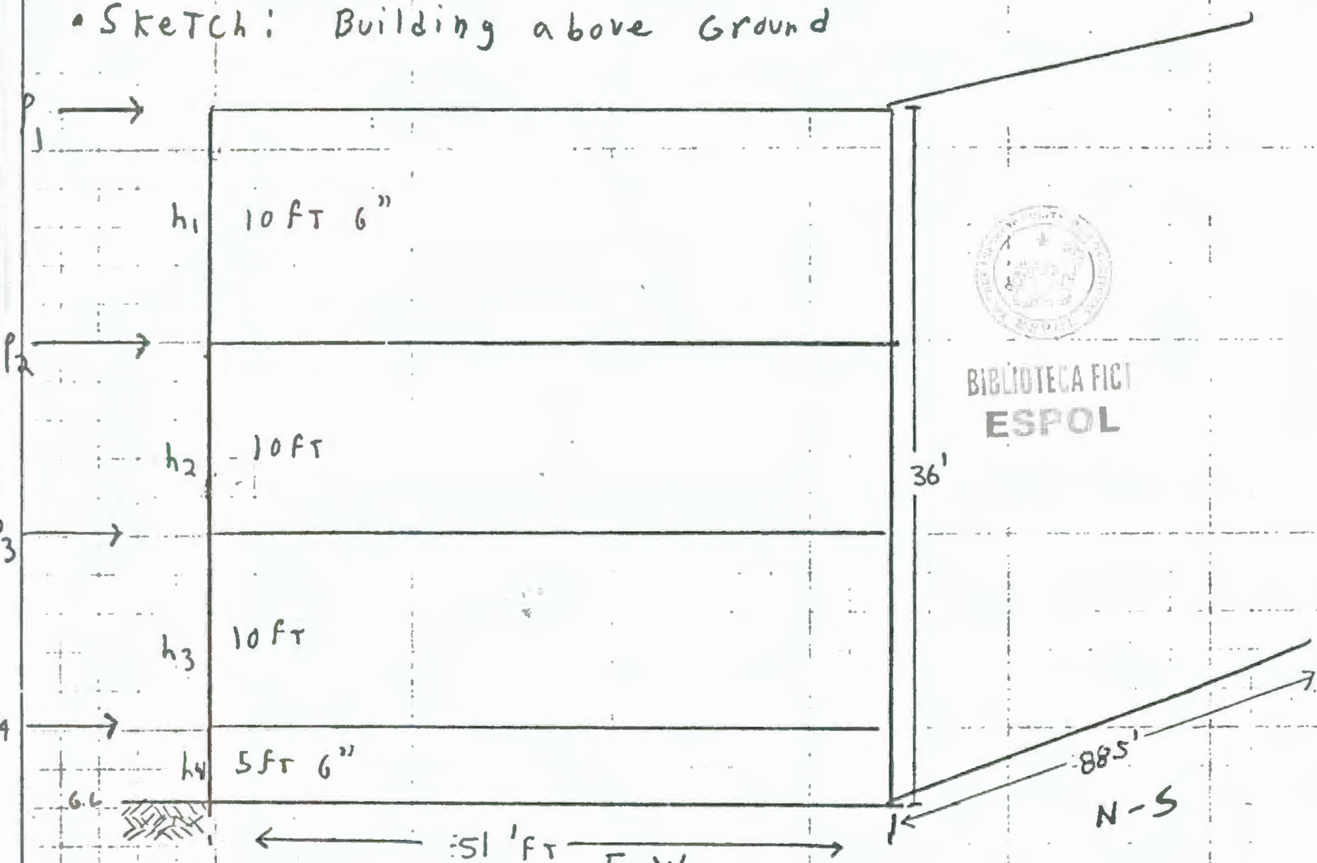
Appendix C

Wind & Earthquake Calculations

Wind loading for Institute Hall ((CANADIAN CODE)

I) Compute wind load for E-W (SHORT SIDE)

• Sketch: Building above Ground



"CANADIAN CODE"

• Calculations:

$P = C_e C_g C_p q$

New England, Worcester location,

Mass Building Codes 11-33 gives us Zone 2.

Figure 1112.1A $V_{fastest} = 80$ mph

$q = \frac{1}{2}\rho \bar{V}_{30}^2 = .0027\,\bar{V}_{30}^2$

$\bar{V}_{30} = \dfrac{\bar{V}_{fastest}}{1.25} = \dfrac{80\text{mph}}{1.25} = 64$ mph

$q = .0027(64\text{mph})^2 = 11.1$ psf

$C_E = 1$

C_g = gust effect factor = 2.0 (small building < 5 stories)

C_p = Pressure coefficient = .8 + .5 (drag coefficient) = 1.3

$p = (1)(2)(1.3)(11.1\,psf) = 28.86\,psf$

- Wind loadings, Tributary method (short side)

$F_1 = h_{T_1} \times w \times P = (5.25')(88.5')(28.86\,psf) = \underline{13.4\,k}$

$F_2 = h_{T_2} \times w \times P = (10.25')(88.5')(28.86\,psf) = \underline{26.2\,k}$

$F_3 = h_{T_3} \times w \times P = (10.0')(88.5')(28.86\,psf) = \underline{25.5\,k}$

$F_4 = h_{T_4} \times w \times P = (7.75')(88.5')(28.86\,psf) = \underline{19.8\,k}$

<u>Shear</u>: $V = F_1 + F_2 + F_3 + F_4 = 13.4 + 26.2 + 25.5 + 19.8 = \underline{84.9}\,k$

- <u>Overturning Moment:</u>

$M_o = \Sigma P_i \times h_i = (13.4 \times 36') + (26.2 \times 25.5') + (25.5 \times 15.5') + (19.8 \times 5.5')$

$= 482.4\,k\text{-}ft + 668.1\,k\text{-}ft + 395.3\,k\text{-}ft + 108.9\,k\text{-}ft$

$= \boxed{1654.7\,k\text{-}ft}$

2) Compute wind loads for N-S side

<u>Loadings: Tributary area method. (long side)</u>

$F_1 = h_{T_1} \times w \times P_1 = (5.25')(51')(28.86\,psf) = \underline{7.7\,k}$

$F_2 = h_{T_2} \times w \times P = (10.25')(51')(28.86\,psf) = \underline{15.1\,k}$

$F_3 = h_{T_3} \times w \times P = (10.0')(51')(28.86\,psf) = \underline{14.7\,k}$

$F_4 = h_{T_4} \times w \times P = (7.75')(51')(28.86\,psf) = \underline{11.4\,k}$

<u>Shear:</u>

$V = F_1 + F_2 + F_3 + F_4 = 7.7\,k + 15.1\,k + 14.7\,k + 11.4\,k = \underline{48.9\,k}$

<u>Overturning Moment:</u>

$M_0 = \Sigma P_i \times h_i = (7.7k \times 36') + (15.1k \times 25.5') + (14.7 \times 15.5') + (11.4k \times 5.5')$

$= 277.2 \text{ k-ft} + 385.05 \text{ k-ft} + 227.85 \text{ k-ft} + 62.7 \text{ k-ft}$

$= \boxed{952.8 \text{ k-ft}}$

SUMMARY

VALUES NEEDED FOR COMPUTER ANALYSIS

E-W (SIDE)

	Fx ACTING ON E-W
@ ROOF	13.4 k
@ 3rd FLOOR	26.2 k
@ 2nd FLOOR	25.5 k
@ 1st FLOOR	19.8 k

THESE FORCES ARE DISTRIBUTED WITHIN 4 SETS OF FRAMES → LARGEST TRIBUTARY WIDTH OF THESE 4 FRAMES = 31 ft

	Fx	WIDTH RATIO	Fx (PER FRAME)
@ ROOF	13.4k	0.35	4.69k
@ 3rd FLOOR	26.2k	0.35	9.17k
@ 2nd FLOOR	25.5K	0.35	8.93K
@ 1st FLOOR	19.8K	0.35	6.93K

- LENGTH OF N-S SIDE = 88.5 ft
- WIDTH RATIO $\frac{31}{88.5} = 0.35$

N-S (SIDE)

	Fx ACTING ON N-S
@ ROOF	7.7k
@ 3RD FLOOR	15.1k
@ 2ND FLOOR	14.7k
@ 1st FLOOR	11.4k

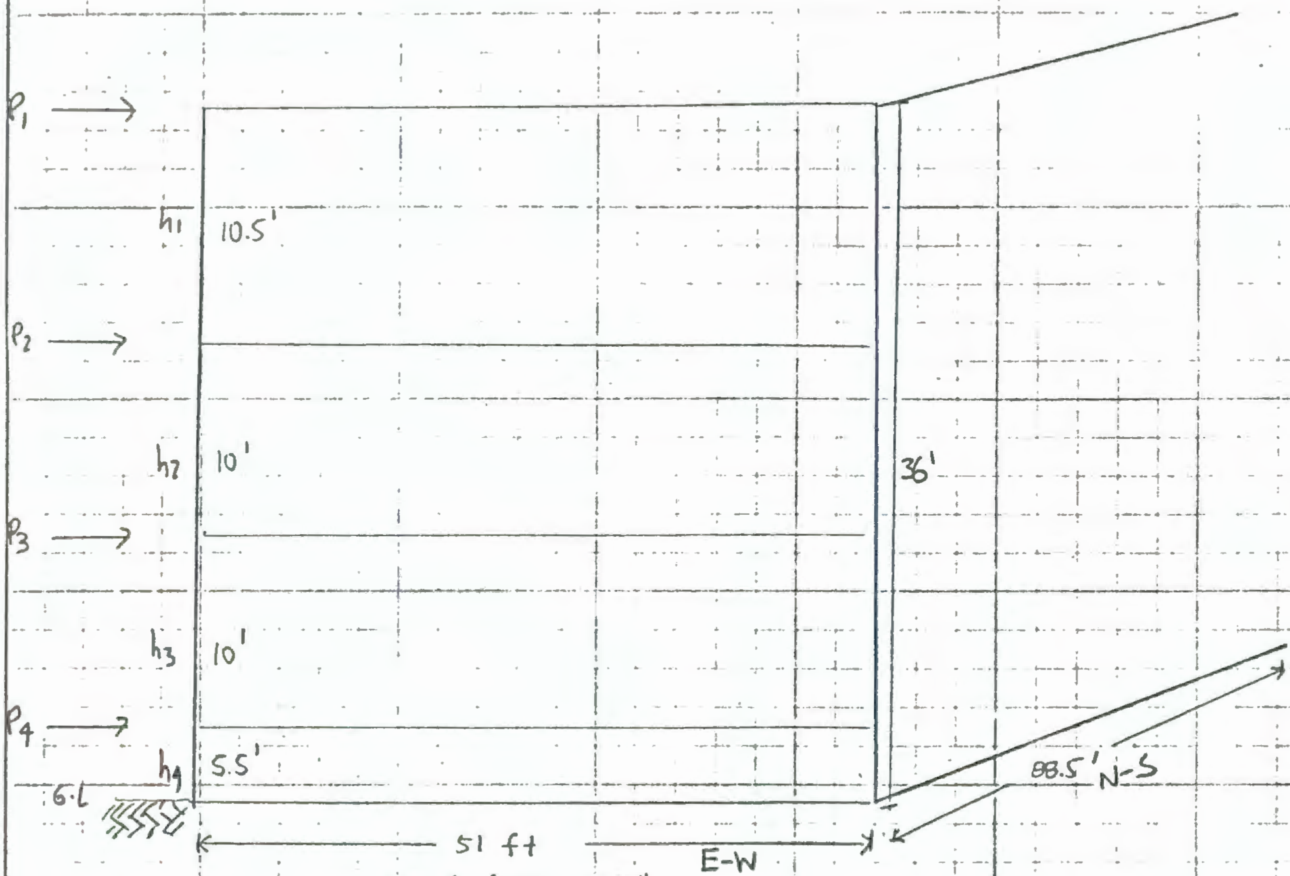
THESE FORCES ARE DISTRIBUTED WITHIN 4 SETS OF FRAMES → • LARGEST TRIBUTARY WIDTH OF THESE 4 FRAMES = 13.25 ft

	Fx	WIDTH RATIO	Fx (PER FRAME)
@ ROOF	7.7K	0.26	2.0K
@ 3RD FLOOR	15.1K	0.26	3.9K
@ 2ND FLOOR	14.7K	0.26	3.8K
@ 1st FLOOR	11.4K	0.26	3.0K

- LENGTH OF E-W SIDE = 51 ft
- WIDTH RATIO $\frac{13.25}{51} = 0.26$

WIND LOADING FOR INSTITUTE HALL ("IN ACCORDANCE WITH MASSACHUSETTS STATE BUILDING CODE" 1990)

• SKETCH (BUILDING ABOVE GROUND)



"MASS CODE - 1990"

• WIND SPEED

(FIGURE 1112.1B) Pg. 11-34

For Zone 2 (Worcester) V_{30} (MPH) = 80 MPH

• EXPOSURE

FROM TABLE 1112.1 FOR ZONE 2 Pg. 11-37

EXPOSURE B (SINCE LESS 50% OF BUILDINGS AROUND NEIGHBORHOOD ARE LESS 4 STORIES)

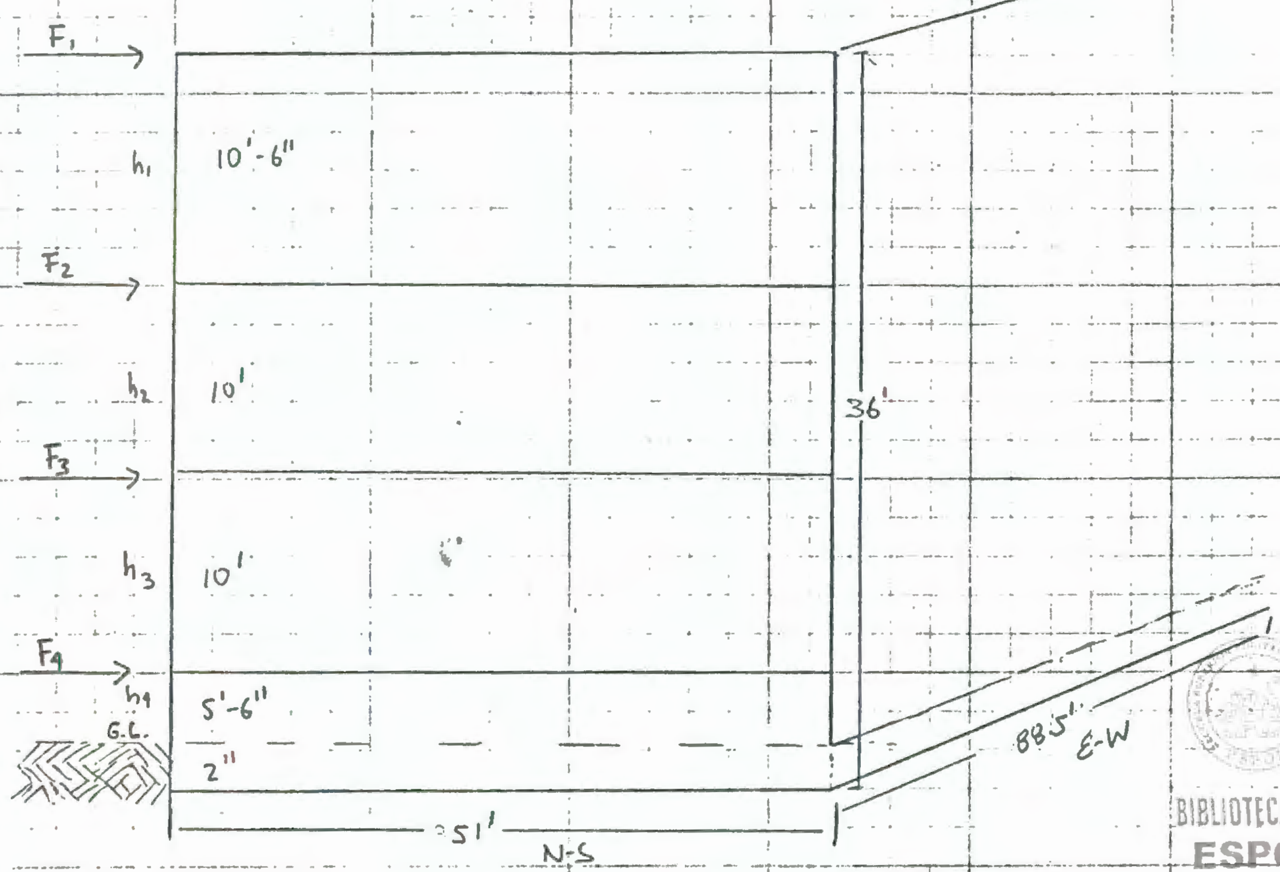
REFERENCE PRESSURE = 17 PSF

• COMPARED TO CANADIAN CODE = 28.36 PSF, 17 PSF IS VERY LOW SINCE THE CANADIAN CODE USES A 2.0 FACTOR FOR GUSTS WINDS IN CASE OF SEVERE WINDS
∴ TO BE SAFER IN OUR STRUCTURAL DESIGN CALCULATIONS WE PREFERED USING THE CANADIAN CODE.

EARTHQUAKE LOADING FOR INSTITUTE HALL (CANADIAN CODE)

LIVE LOADS
- corridors = 100 psf
- Snow load: 35 psf
- LL private APTS = 40 psf
- LL Public ROOMS (BASEMENT) = 100 psf

• SKETCH (BUILDING ABOVE GROUND)



1) • N-S SIDE CALCULATIONS

• BASE SHEAR ⟹

$V = ASKIFW$

$= (0.1)(0.87)(0.7)(1.0)(1.0)(1246)$

$= \boxed{75.9 \text{ kips}}$

where
- $A = 0.1$ (New England)
- $S = 0.5/(T^{1/3}) = 0.5/(0.191)^{1/3} = 0.8682$
- $T = 0.05\, h_n/\sqrt{D} = 0.05(36)/\sqrt{88.5} = 0.191$
- $k = 0.7$ (ductile moment resisting frame)
- $I = 1.0$ (APT)
- $F = 1.0$ (rock)

$$W = \frac{\overset{\text{roof L.L}}{(88.5 \times 51)(35)}}{1000} + 6\frac{\overset{\text{APTS AREA L.L}}{(88.5 \times 22.5)(40)}}{1000} + 3\frac{\overset{\text{CORRIDORS}}{(88.5 \times 6)(100)}}{1000}_{\text{L.L}} + \frac{\overset{\text{BASEMENT}}{(88.5 \times 51)(100)}}{1000}_{\text{L.L}}$$

$W = 158\text{k} + 478\text{k} + 159\text{k} + 451\text{k}$

$\underline{W = 1246\text{k}}$

• LOADING

$$F_x = \frac{(V - F_t)\, W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

If $\frac{h_n}{D} \le 3$ $F_t = 0$

$\frac{10.5}{88.5} = 0.12$; $\frac{10}{88.5} = 0.11$; $\frac{7.5}{88.5} = 0.08$

For all floors

$h_n/D \le 3 \therefore F_t = 0$

EARTHQUAKE CALCULATIONS (STEEL FRAME)

LEVEL	AREA x	LIVELOAD =	Wx	hx (FT)	Wx hx
ROOF	(88.5 x 51)	(35) =	158 K	36'	5688 FTK
3rd	APTS 2(88.5 x 22.5) x	(40) =	159 K +	212K	25.5'	5406 FTK
	+ CORRIDOR (88.5 x 6) x	(100) =	53K		
2nd	2(88.5 x 22.5) x	(40) =	159 K	212K	15.5'	3286 FTK
	+ (88.5 x 6) x	(100) =	53 k		
1st	2(88.5 x 22.5) x	(40) =	159k	212k	7.5'	1590 FTK
	+ (88.5 x 6) x	(100) =	53k		
					$\Sigma = 15970$ FT-K

• $F_x = \dfrac{(V-F_t)\,W_x h_x}{\sum_{i=1}^{n} w_i h_i}$

$F_{ROOF} = \dfrac{75.9\,(5688)}{15970} = \boxed{27.0\text{ k}}$

$F^{3rd} = \dfrac{75.9\,(5406)}{15970} = \boxed{25.7\text{ k}}$

$F^{2nd} = \dfrac{75.9\,(3286)}{15970} = \boxed{15.6\text{ k}}$

$F^{1st} = \dfrac{75.9\,(1590)}{15970} = \boxed{7.6\text{ k}}$

• OVERTURNING MOMENT

J = since T < 0.5

$M = F_{ROOF} Z_1 + F^{3rd} Z_2 + F^{2d} Z_3 + F^{1st} Z_4$

$M = (27.0 \times 36) + (25.7 \times 25.5) + (15.6 \times 15.5) + (7.6 \times 7.5)$

$M = 972 + 655.4 + 241.8 + 57$

$\boxed{M = 1926.2 \text{ FT-K}}$

2) SHORT SIDE (E-W)

• BASE SHEAR

$V = ASKIEW$

$= (0.1)(0.79)(0.7)(1.0)(1.0)(1246)$

$V = \boxed{68.9\text{K}}$

where
$A = 0.1$ (New England)
$S = 0.5/(T^{1/3}) = 0.5/(0.252)^{1/3} = 0.79$
$T = 0.05\,h_n/\sqrt{D} = 0.05(36)/\sqrt{51} = 0.252$
$K = 0.7$ (ductile moment resisting frame - steel frame)
$I = 1.0$ (APTS)
$F = 1.0$ (ROCK)
W (from long side calculations) = 1246K

• LOADING

$F_x = \dfrac{(V-F_t)\,W_x h_x}{\sum_{i=1}^{n} w_i h_i}$

$W_x h_x$, $w_i h_i$ } from long-side calculations.

• E-W SIDE CALCULATIONS

$F_{ROOF} = \frac{68.9(5688)}{15970} = \boxed{24.5k}$

$F_{3^{rd}} = \frac{68.9(5406)}{15970} = \boxed{23.3k}$

$F_{2^{nd}} = \frac{68.9(3286)}{15970} = \boxed{14.2k}$

$F_{1^{st}} = \frac{68.9(1590)}{15970} = \boxed{6.9k}$

• OVERTURNING MOMENT (SHORT SIDE)

$J = 1$ since $T < 0.5$

$M = F_{ROOF} \cdot Z_1 + F_{3^{rd}} \cdot Z_2 + F_{2^{nd}} \cdot Z_3 + F_{1^{st}} \cdot Z_4$

$M = (24.5 \times 36) + (23.3 \times 25.5) + (14.2 \times 15.5) + (6.9 \times 7.5)$

$M = 882 + 594 + 220 + 52$

$M = \boxed{1748 \text{ FT-K}}$

SUMMARY

VALUES NEEDED FOR COMPUTER ANALYSIS

E-W (SIDE)

	F_x ALONG ON E-W
@ ROOF	24.5k
@ 3RD FLOOR	23.3k
@ 2ND FLOOR	14.2k
@ 1ST FLOOR	6.9k

THESE FORCES ARE DISTRIBUTED WITHIN 4 SETS OF FRAMES →

• LARGEST TRIBUTARY WIDTH OF THESE 4 FRAMES ON E-W SIDE = 31 ft

• LENGTH OF N-S SIDE 88.5 ft

• WIDTH RATIO 31/88.5 = 0.35 ft

	FX	WIDTH RATIO	FX (PER FRAME)
@ ROOF	24.5k	0.35	8.6k
@ 3RD FLOOR	23.3k	0.35	8.2k
@ 2ND FLOOR	14.2k	0.35	5.0k
@ 1ST FLOOR	6.9k	0.35	2.4k

• N-S (SIDE)

	F_x ACTING ON N-S
@ ROOF	27 k
@ 3RD FLOOR	25.7 K
@ 2ND FLOOR	15.6K
@ 1ST FLOOR	7.6K

THESE FORCES ARE DISTRIBUTED WITHIN 4 SETS OF FRAMES → • LARGEST TRIBUTARY WIDTH OF THESE 4 FRAMES = 13.25 ft

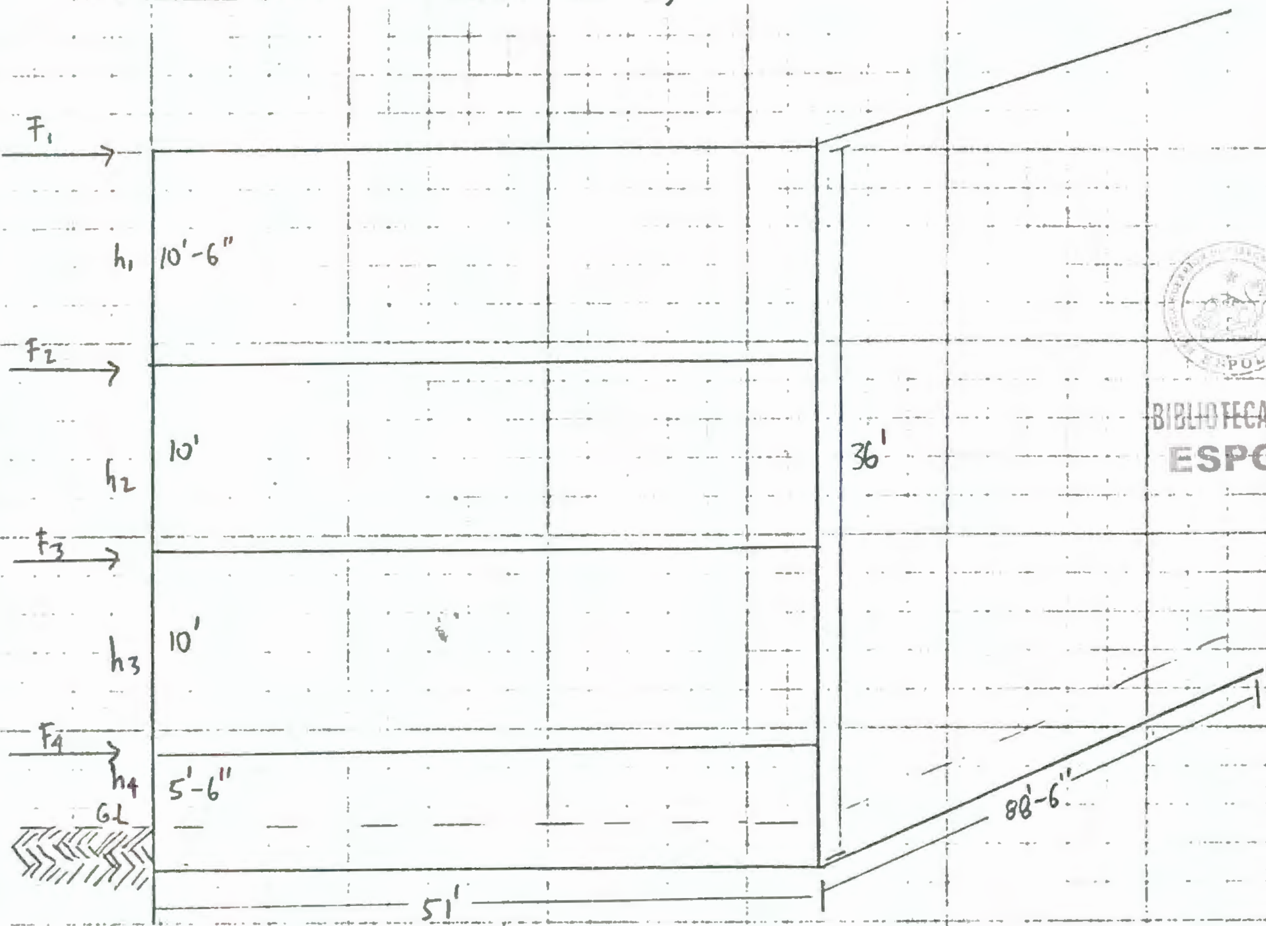
• LENGTH OF E-W SIDE = 51 ft

• WIDTH RATIO $\frac{13.25}{51} = 0.26$

	Fx	WIDTH RATIO	Fx (PER FRAME)
@ ROOF	270K	0.26	7.0 K
@ 3rd FLOOR	25.7K	0.26	6.7 K
@ 2ND FLOOR	15.6K	0.26	4.1 K
@ 1st FLOOR	7.6K	0.26	2.0 K

EARTHQUAKE LOADING FOR INSTITUTE HALL ("IN ACCORDANCE WITH MASSACHUSETTS STATE BUILDING CODE") 1990

• SKETCH (BUILDING ABOVE GROUND)



• EARTHQUAKE LOADS "MASS. BUILDING CODE - 1990"

+ TOTAL LATERAL FORCE (1113.4.1 Pg 11-47)

$$V = \tfrac{1}{3}\, KCSW$$

where

$C = \dfrac{0.05}{\sqrt[3]{T}}$; $T = \dfrac{0.05 h_n}{\sqrt{D_s}}$; D = DIMENSION OF BUILDING

K = 0.67 (TABLE 1113.1) FOR BUILDINGS WITH A MOMENT RESISTING SPACE FRAME DESIGNED TO RESIST THE TOTAL REQUIRED LATERAL FORCE (STEEL FRAME)

S = 1 (1113.4.1.3) SINCE SOIL DEPTH BELOW FOUND. LEVEL IS LESS THAN 200 FT

"MASSACHUSETTS CODE -1990"

• N-S SIDE

$T = \frac{0.05(36)}{\sqrt{88.5}} = 0.191$; $C = \frac{0.05}{\sqrt[3]{T}} = \frac{0.05}{\sqrt[3]{0.191}} = 0.087$

+ LATERAL FORCE

LIVE LOADS

CORRIDORS = 100 PSF
SNOW LOAD = 35 PSF
L.L. PRIVATE APTS = 40 PSF
LL PUBLIC ROOMS (BASEMENT) = 100 PSF

$V = 1/3\, KCSW$

$W = \frac{(88.5 \times 51)(35)}{1000} + \frac{6(88.5 \times 22.5)(40)}{1000} + \frac{3(88.5 \times 6)(100)}{1000} + \frac{(88.5 \times 51)(100)}{1000}$

ROOF AREA, APTS AREA, CORRIDORS AREA, BASEMENT AREA

$W = 158\,k + 479\,k + 159\,k + 451\,k$

$W = 1246\ k$

$V = 1/3(0.67)(0.087)(1.0)(1246)$

$\boxed{V = 24.2\,K}$

• E-W SIDE

$T = \frac{0.05(36)}{\sqrt{51}} = 0.25$; $C = \frac{0.05}{\sqrt[3]{0.25}} = 0.079$

+ LATERAL FORCE

$V = 1/3\, KCSW$
$= 1/3(0.67)(0.079)(1.0)(1246)$
$\boxed{V = 21.98\,K}$

NOTE: COMPARING THE V VALUES WITH THOSE OBTAINED WITH THE CANADIAN CODE → V(LONG SIDE) = 75.9 K > 24.2 K (MASS CODE)
V(SHORT SIDE) = 68.9 K > 21.98 K (MASS CODE)

THE MASSACHUSETTS CODE VALUES WERE LOWER DUE TO THE 1/3 FACTOR.
∴ TO BE SAFER WE PREFERRED TO USE THE CANADIAN CODE FOR OUR STRUCTURAL DESIGN CALCULATIONS.

1) N-S side Calculations (Reinforced Concrete)

• Base Shear => where

$V = A_S S K I F W$
$= (0.1)(.87)(1.3)(1.0)(1.0)(1246)$

$= \boxed{140.9 \text{ kips}}$

$A_S = 0.1$ (New England)
$S = .5/(T^{1/3}) = .5/(.191)^{1/3} = .8682$
$T = .05\, h_n/\sqrt{D} = .05(36)/\sqrt{88.5}$
$= .191$
$k = 1.3$ (ductile moment resisting frame)
$I = 1.0$ (APT)
$F = 1.0$ (rock)

$W = 1246\,K$ (previous calculations)

• Loading

$F_x = \dfrac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$

If $\dfrac{h_n}{D} \le 3$ $F_t = 0$

$\dfrac{10.5}{88.5} = .12$; $\dfrac{10}{88.5} = .11$; $\dfrac{7.5}{88.5} = .08$

For all Floors
$h_n/D \le 3 \therefore F_t = 0$

level	Area	×	Liveload	=	W_x	h_x (FT)	$W_x h_x$
roof	(885 x 51)		(35)		158 K	36'	5688 Ft K
3rd	APT 2(88.5 x 22.5)	x	(40)	=	159 K		
	+				+		
	corr (88.5 x 6)	x	(100)	=	53 K } 212 K	25.5'	5406 ft K
2nd	2(88.5 x 22.5)	x	(40)	=	159 K		
	+ (88.5 x 6)	x	(100)	=	53 K } 212 K	15.5'	3286 FT K
1ST	2(88.5 x 22.5)	x	(40)	=	159 K		
	+ (88.5 x 6)	x	(100)	=	53 K } 212 K	7.5'	1590 FT K

$\Sigma = 15970$ FT-K

• $F_x = \dfrac{(V - F_t) W_x h_x}{\sum_{i=1}^{n} W_i h_i}$

$F_{roof} = \dfrac{(140.9)(5688)}{15970} = \boxed{50.2\,K}$

$F_{3rd} = \dfrac{(140.9)(5406)}{15970} = \boxed{47.7\,K}$

$F_{2nd} = \dfrac{(140.9)(3286)}{15970} = \boxed{29.0\,K}$

$F_{1ST} = \dfrac{(140.9)(1590)}{15970} = \boxed{14.0\,K}$

: Overturning Moment

$J = 1$ since $T < 0.5$

$M = F_{roof} Z_1 + F^{3rd} Z_2 + F^{2nd} Z_3 + F^{1ST} Z_4$

$m = (50.2 \times 36) + (47.7 \times 25.5') +$
$(29.0 \times 15.5) + (14 \times 7.5) =$

$m = 1807.2 + 1216.4 + 450 + 10$

$\boxed{m = 3578.6 \text{ FT-K}}$

a)- Short side (E-W)

- Base shear

$V = ASKIFW$
$= (.1)(.79)(1.3)(1.0)(1.0)(1246)$
$= 128\ K$

where $A = .1$ (New England)
$S = .5/T^{1/3} = .5/(.252)^{1/3} = .74$
$T = .05hn/\sqrt{D} = .05(36)/\sqrt{51}$
$= .252$
$K = 1.3$ (ductile, moment resisting Frame, reinforced Concrete)
$I = 1.0$ (APTS)
$F = 1.0$ (ROCK)
W (From long side calculations)

- Loading

$$Fx = \frac{(V-Ft)\,w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

$\left.\begin{matrix} w_x h_x \\ w_i h_i \end{matrix}\right\}$ From Long-side Calculations

- E-W side Calculations

$$F_{roof} = \frac{128(5688)}{15970} = \boxed{45.6\ K}$$

$$F_{3^{rd}} = \frac{128(5406)}{15970} = \boxed{43.3\ K}$$

$$F_{2^{nd}} = \frac{128(3286)}{15970} = \boxed{26.3\ K}$$

$$F_{1^{st}} = \frac{128(1590)}{15970} = \boxed{12.7\ K}$$

- Overturning Moment (short side)

$J = 1$ since $T < .5$

$M = F_{roof} \cdot Z_1 + F_{3^{rd}} \cdot Z_2 + F_{2^{nd}} \cdot Z_3 + F_{1st} \cdot Z_4$

$M = (45.6)(36) + (43.3)(25.5) + (26.3)(15.5) + (12.7)(7.5)$

$= 1641.6 + 1104.2 + 407.6 + 95.3$

$$\boxed{M = 3248.7\ Ft\text{-}K}$$

Summary

Values Needed for Computer Analysis

E-W (side)

	F_x
@ roof	45.6 K
@ 3rd Floor	43.3 K
@ 2nd Floor	26.3 K
@ 1st Floor	12.7 K

These forces are distributed within 4 sets of Frames → largest Tributary width of these 4 Frames on E-W side = [illegible]

- Length of N-S side 88.5 ft
- width ratio $31/88.5 = .35$ ft

	Fx	width Ratio	Fx [Per Frame]
@ roof	45.6	.35	16 K
@ 3rd floor	43.3	.35	15.2 K
@ 2nd floor	26.3	.35	9.2 K
@ 1st floor	12.7	.35	4.4 K

• N-S (Side)

	F_x acting on N-S
@ Roof	50.2 K
@ 3rd floor	47.7 K
@ 2nd floor	29.0 K
@ 1st floor	14.0 K

These forces are distributed within 4 sets of Frames → • Largest Tributary width of these 4 Frames = 13.25 ft

- Length of E-W side = 51 ft

	Fx	width Ratio	F_x (Per Frame)
@ Roof	50.2 K	.26	13.1 K
@ 3rd floor	47.7 K	.26	12.4 K
@ 2nd floor	29.0 K	.26	7.54 K
@ 1st floor	14.0 K	.26	3.64 K

- width Ratio

$13.25/51 = .26$

• Earthquake loads "Mass Building Code - 1990"

+ Total Lateral Force (1113-4.1 pg 11-47)

$V = 1/3\, KCSW$

Where

$c = \frac{.05}{\sqrt[3]{T}}$; $T = \frac{.05\, h_n}{\sqrt{D_s}}$; D = Dimension of Building

$K = 1.3$

$s = 1$ (113.4.1.3)

N-S side

$T = \frac{.05(36)}{\sqrt{88.5}} = .191$; $c = \frac{.05}{\sqrt[3]{T}} = \frac{.05}{\sqrt[3]{.191}} = .087$

$V = 1/3\, KCSW$

W = 1246 K (Previous calculations)

$V = 1/3 (1.3)(.087)(1)(1246)$

$\boxed{V = 47.0\ K}$

E-W side

$T = \frac{.05(36)}{\sqrt{51}} = .25$; $C = \frac{.05}{\sqrt[3]{.25}} = .079$

+ Lateral Force

$V = 1/3\, KCSW$

$= 1/3 (1.3)(.079)(1.0)(1246)$

$\boxed{= 42.7 K}$

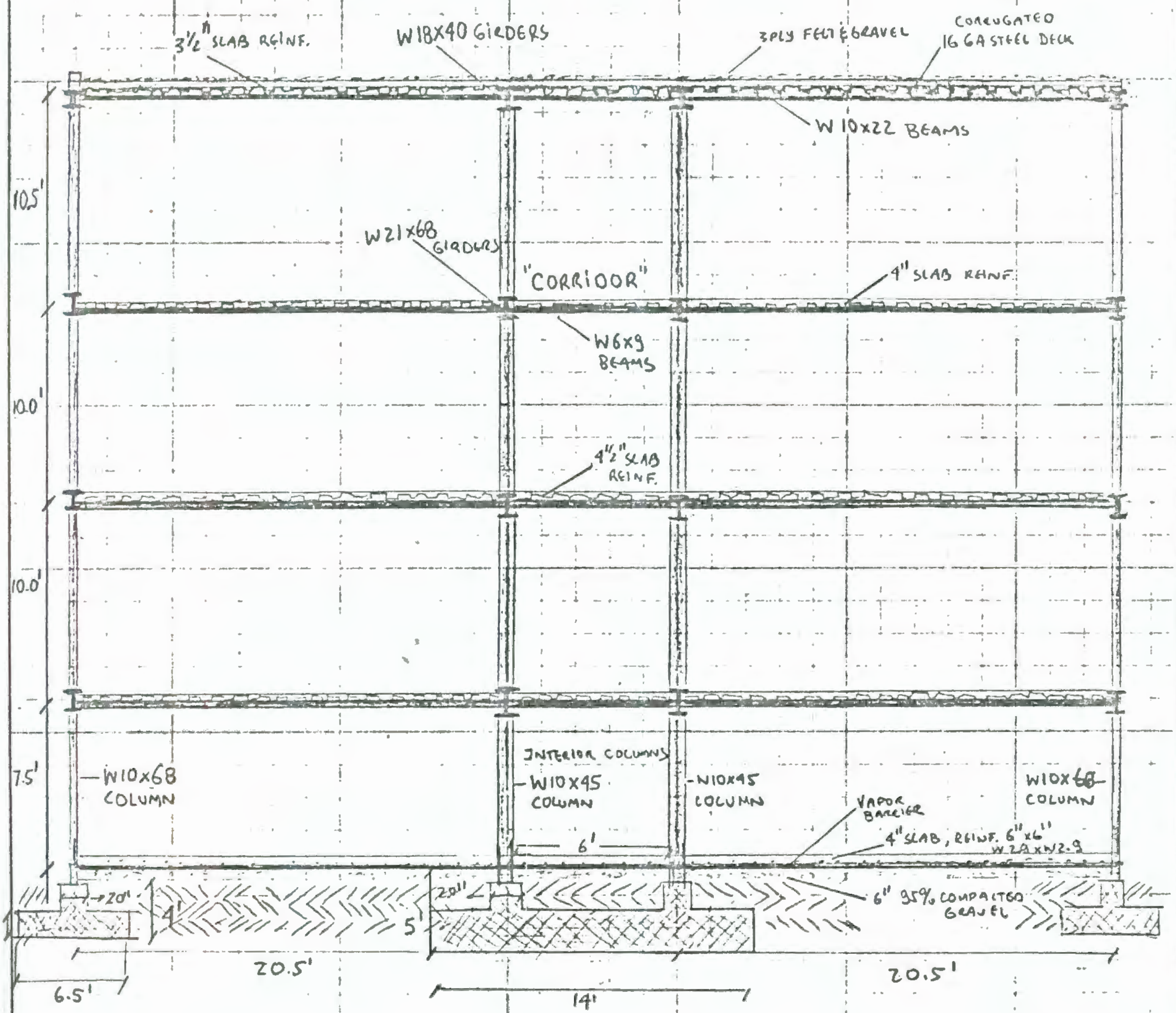
Comparison of codes

	Canadian		Mass
N-S	140.9	>	47.0
E-W	128	>	42.7

We choose to use the Canadian code

Appendix D

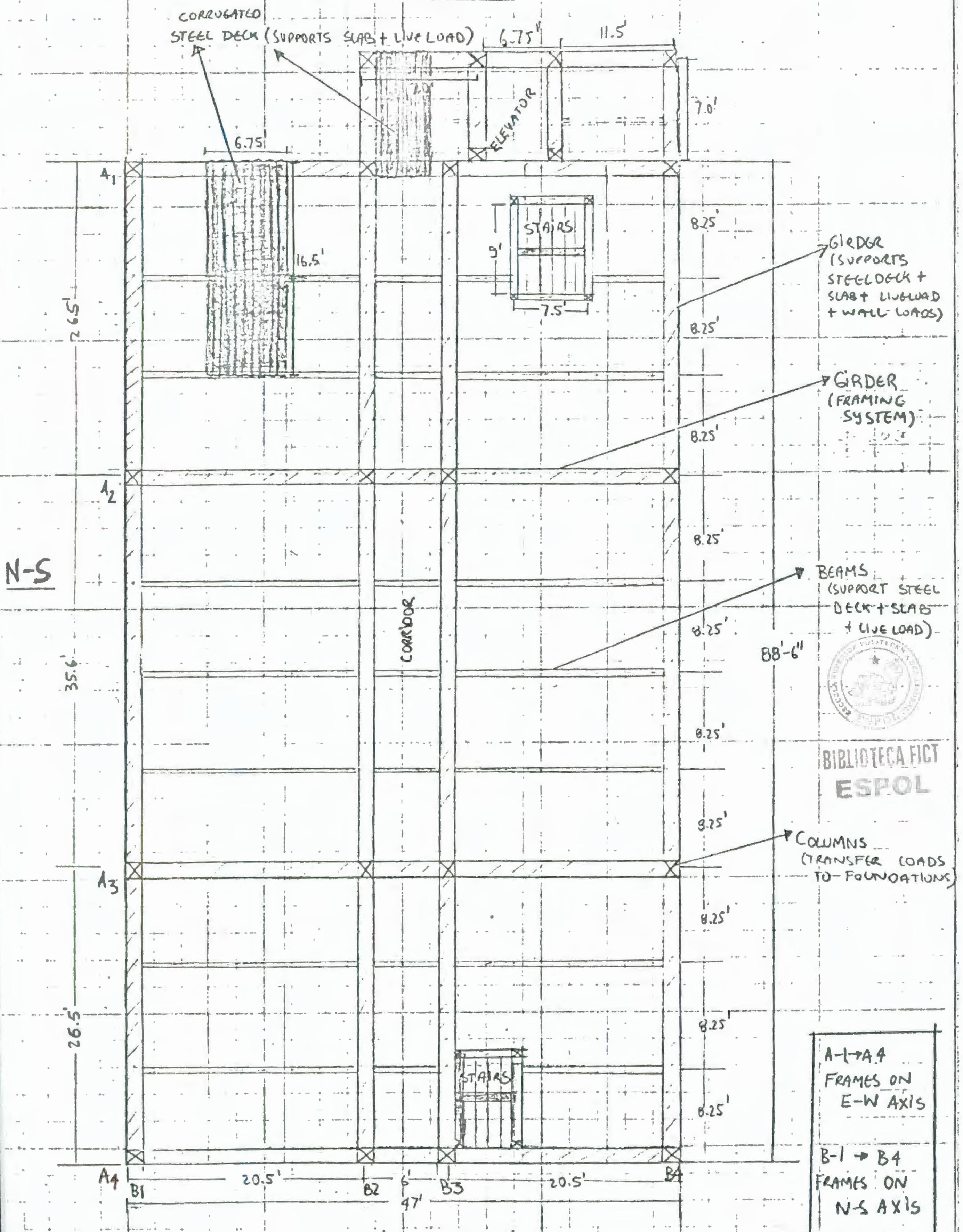
Steel Design Calculations



E-W VIEW @ CENTER FRAME

SIDE VIEW OF FRAMES A3 OR A2 ON FIGURE 5.4

STEEL DESIGN (DIMENSIONS)



"TOP VIEW" (BEFORE DESIGN)

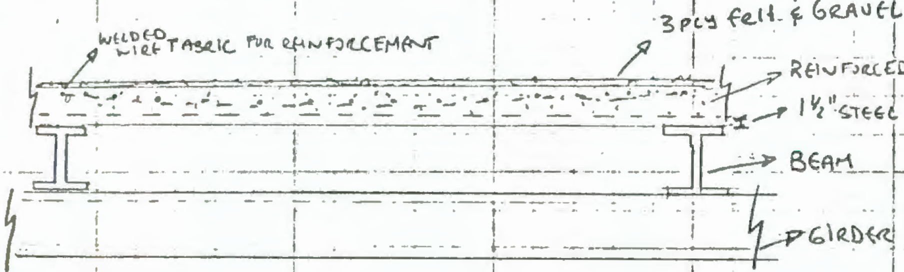
E-W

ROOF DECK DESIGN

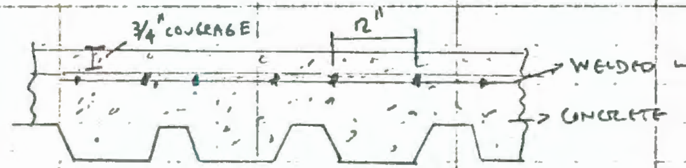
"IN ACCORDANCE WITH THE UNITED STEEL DECK, INC. (FROM 1992 SWEET'S CATALOG FILE 2)

- TYPE OF ROOF: REINFORCED CONCRETE FLOOR SLABS ON INVERTED "B" DECK (GALV)"

- SKETCHES:



TYPICAL INTERIOR SPAN (SIDE VIEW)



FRONTAL VIEW OF DECK-SLAB

- GENERAL INFORMATION

1. STEEL DECK IS TO BE TYPE "B" INSTALLED IN THE INVERTED POSITION (SEE SKETCH) AND IS TO BE WELDED TO THE STRUCTURAL SUPPORTS USING 5/8" dia puddle welds, SPACED NOT MORE THAN 12" ON CENTER.
2. REINFORCEMENT (WELDED WIRE FABRIC) SHOWN FROM TABLE IN SWEETS CATALOG IS THE NEAREST POPULAR SIZE SUPPLYING THE MINIMUM A_s PERMITTED BY ACI CODE FOR TEMPERATURE, SHRINKAGE AND STRUCTURAL REINFORCEMENT $A_s = 0.0018\ bt$.
   ITS TENSILE FIBER STRESS IS TAKEN AT 30000 psi, ACI 318-63.

- LOADS AND VALUES

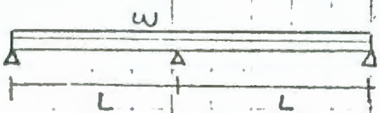
$f'_c = 3000$ psi
$f_c = 1350$ psi
$f_s = 30{,}000$ psi
$n = 9$

- ROOF LIVE LOADS: 35 PSF (SNOW), 20 PSF (CONSTRUCTION) — LARGEST GOVERNS

- DESIGN FOR A TWO SPAN (SEE SWEETS CATALOG, FILE 2, ON "STEEL DECKS")

TWO SPAN

$w = w_1 + w_2$



W_1 = Wet Concrete + Deck Weight PSF

W_2 = SNOW LOAD = 35 PSF

CHOOSING A $3\frac{1}{2}''$ THICK SLAB WITH AN $8.25'$ SPAN.

$$W_2 = 37\,PSF + 5.5\,PSF\,(3\,PLY\ FELT\ \&\ GRAVEL) = 42.5\,PSF$$

$$\therefore\ W = W_1 + W_2 = 42.5\,PSF + 35\,(SNOW\ LOAD) = 77.5\,PSF$$

- EQUATIONS FROM SWEETS CATALOG (P6.14) ON STEEL DECK SECTION.
  - NEGATIVE MOMENT GOVERNS

$$-M = .125\,wl^2$$
$$-M = .125\,(77.5)\,(8.25')^2$$
$$-M = 659.35\ FT\,LB$$

- DEFLECTION:

$$\Delta = .0054\,w_1 l^4/EI$$
$$\Delta = .0054\,(35)\,(8.25')^4/30000 \times 825'$$
$$\Delta = 5.19 \times 10^{-5}\ FT \le 1/180\ SPAN\ (DEFLECTION\ LIMIT)$$
$$= 1/180\,(8.25) = 4.58 \times 10^{-2}\ \therefore\ OK ✓$$

- ROOF DECKING

CHOOSING A 16 GA (GAUGE), WITH A $3\frac{1}{2}''$ SLAB, A 66-44 MESH, AND A $8.25'$ SPAN, gives US A $W = 78\,PSF > 77.5\,PSF$ ACTUAL $\therefore$ OK. IT ALSO SATISFIES THE DEFLECTION LIMIT AND TEMPERATURE, SHRINKAGE, & STRUCTURAL REINFORCEMENT REQUIREMENTS.

- SUMMARY:

GAUGE: 16 GA
SPAN: $(8.25')\times 2 = 16.5'$; WIDTH: $6.75''$ (TO SIMPLIFY THE DESIGN)
SLAB: $3\frac{1}{2}''$ W/ 66-44 MESH (ROLLED IN DIRECTION OF DECK SPAN)
WELD: $5/8''$ dia. PUDDLE WELDS SPACED AT $12''$ ON CENTER
SIDE LAPS OF SHEETS TO BE WELDED OR BOLTED

- ROOF AREA $= 47' \times 88.5' = 4159.5\ FT^2$
- AREA SHEET: $16.5' \times 6.75' = 111.4\ FT^2$
- STAIRS AREA $= 2(9' \times 7.5') = 135\ FT^2$
- NET AREA $4159.5 - 135 = 4024.5\ FT^2$

$$\#\ SHEETS = \frac{4024.5}{111.4} = 36.13 \approx \boxed{37\ SHEETS}$$

TO BE SAFE

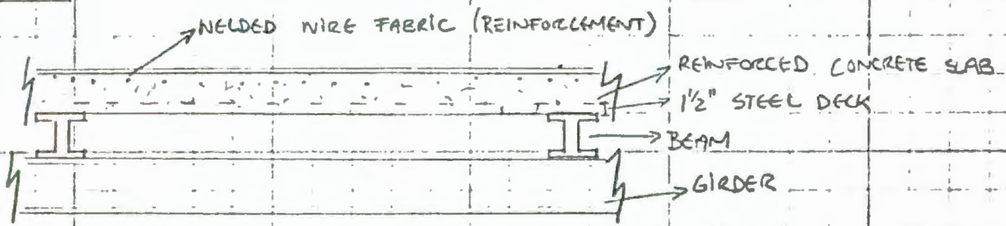
FLOOR DECK DESIGN

"IN ACCORDANCE WITH THE UNITED STEEL DECK, INC (FROM 1992 SWEETS CATALOG FILE 2)

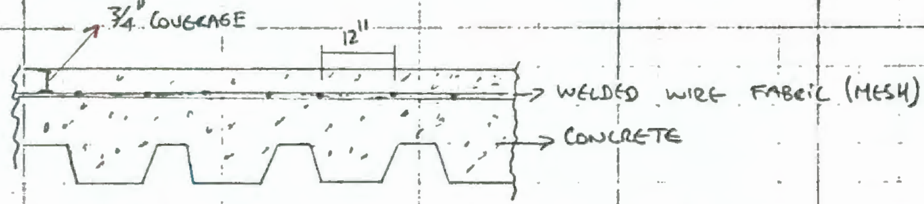
NOTE: SINCE THE 3rd, 2nd, and 1st FLOORS HAVE THE SAME SPACE DISTRIBUTION (SAME # APTS., BATHROOMS, ETC) THE FLOOR DESIGN WILL BE THE SAME FOR EACH ONE.

• TYPE OF FLOOR:
REINFORCED CONCRETE FLOOR SLABS ON INVERTED "B" DECK (GALV)

• SKETCHES:



TYPICAL INTERIOR SPAN (SIDE VIEW)



FRONTAL VIEW OF DECK-SLAB

GENERAL INFORMATION

1. STEEL DECK IS TO BE TYPE "B" INSTALLED IN THE INVERTED POSITION (SEE SKETCH) AND IS TO BE WELDED TO THE STRUCTURAL SUPPORTS USING 5/8" dia. PUDDLE WELDS, SPACED NOT MORE THAN 12" ON CENTER.
2. REINFORCEMENT (WELDED WIRE FABRIC) SHOWN FROM TABLE IN SWEETS CATALOG IS THE NEAREST POPULAR SIZE SUPPLYING THE MINIMUM AS PERMITTED BY ACI CODE FOR TEMPERATURE, SHRINKAGE AND STRUCTURAL REINFORCEMENT $A_s = 0.0018\,bt$.
   ITS TENSILE FIBER STRESS IS TAKEN AT 30,000 Psi ACI 318-63.

• LOADS AND VALUES

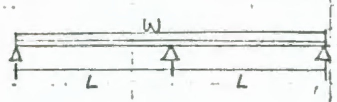
$f'_c = 3000$ Psi
$f_c = 1350$ Psi
$f_s = 30{,}000$ psi
$n = 9$

• FLOOR LIVE LOADS
40 PSF APTS & BATHROOMS
100 PSF CORRIDORS
20 PSF CONSTRUCTION

• DESIGN FOR A TWO SPAN (SEE SWEETS CATALOG, FILE 2 ON "STEEL DECKS"

TWO SPAN

$W = W_1 + W_2$



W_1 = WET CONCRETE + DECK WEIGHT PSF
W_1 = 43 PSF FOR A 4" SLAB
50 PSF FOR A 4½" SLAB

+ APARTMENTS W_2

LIVELOADS : 40 PSF (RESIDENTIAL) > LARGEST GOVERNS
20 PSF (CONST)

W_2 = 40 PSF + 3 PSF (SUSPENDED LIGHTING AND AIR DISTRIBUTION SYSTEMS)
W_2 = 43 PSF

∴ $W = W_1 + W_2$ = 43 PSF + 43 PSF = 86 PSF

+ CORRIDORS

W_2 LIVELOADS: 100 PSF (CORRIDORS) > LARGEST GOVERNS
20 PSF (CONSTRUCTION)

W_2 = 100 PSF + 3 PSF (SUSPENDED SYSTEMS)
W_2 = 103 PSF
∴ $W = W_1 + W_2$ = 50 PSF + 103 PSF = 153 PSF

• EQUATIONS FROM SWEETS CATALOG (PG. 14) "ON STEEL DECK SECTION."

• NEGATIVE MOMENT GOVERNS:

$-M = .125\, w\, l^2$

+ APARTMENTS

$-M = .125\,(86) \times 8.25'^2 =$
$-M =$ 732 FT-LB

+ CORRIDORS

$-M = .125\,(153) \times 8.25'^2$
$-M =$ 1302 FT-LB

• DEFLECTION:

$\Delta = .0054\, w_1\, l^4 / EI$

+ APARTMENTS + CORRIDORS

$\Delta = .0054\,(43)\,(8.25)^4 / 30000 \times 8.25$
$\Delta = 0.0040$ FT $\leq$ 1/180 SPAN (DEFLECTION LIMIT)
$\leq 1/180\,(8.25) = 4.58 \times 10^{-2}$
$4.09 \times 10^{-3} \leq 4.58 \times 10^{-2}$ ∴ OK ✓

- FLOOR DECKING

+ APARTMENTS' AREA

CHOOSING A 16 GA (GAUGE) DECK WITH A 4" SLAB, A 66-44 MESH, AND A TWO-8.25' SPAN, GIVES A W= 95 PSF > 86 PSF ACTUAL ∴ OK.
IT ALSO SATISFIES THE DEFLECTION LIMIT AND TEMPERATURE, SHRINKAGE, & STRUCTURAL REINFORCEMENT REQUIREMENTS.

+ CORRIDORS' AREA

CHOOSING A 16 GA (GAUGE), WITH A 4½" SLAB, A 44-44 MESH, AND A TWO-8.25' SPAN, GIVES A W= 165 PSF > 153 PSF ACTUAL ∴ OK.
IT ALSO SATISFIES THE DEFLECTION LIMIT AND TEMPERATURE, SHRINKAGE, & STRUCTURAL REINFORCEMENT REQUIREMENTS.

- SUMMARY:

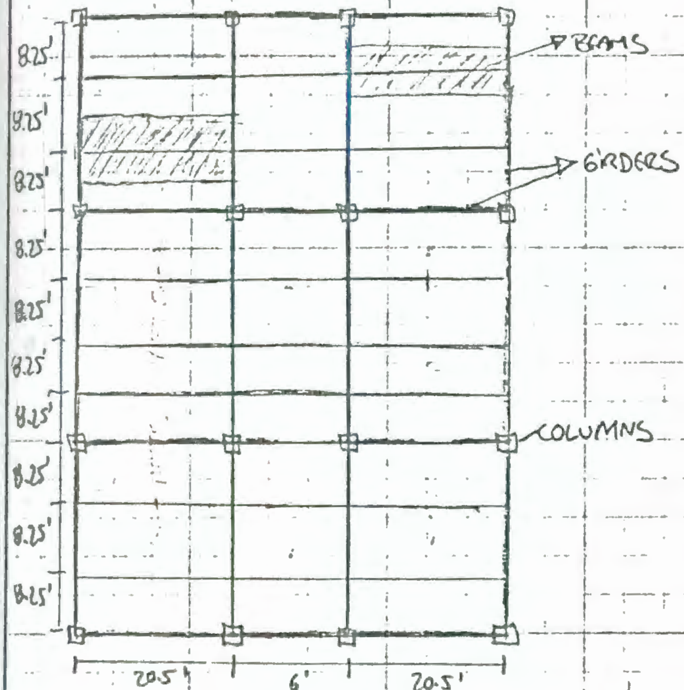
> - APARTMENTS AREA (CORRUGATED STEEL SHEETS)
>   GAUGE: 16 GA
>   SPAN: 2X8.25' ; WIDTH: 6.75' (TO SIMPLIFY THE DESIGN)
>   SLAB: 4" W/ 66-44 MESH (ROLLED IN DIRECTION OF DECK SPAN)
> - CORRIDORS AREA (CORRUGATED STEEL SHEETS)
>   GAUGE: 16 GA
>   SPAN: 2X8.25', WIDTH 6.0' (TO SIMPLIFY THE DESIGN)
>   SLAB: 4½" W/ 44-44 MESH (ROLLED IN DIRECTION OF DECK SPAN)

APARTMENTS AREA

- PER FLOOR
  $2(20.5' \times 88.5') = 3628.5\ FT^2$
- AREA SHEET
  $16.5' \times 6.75' = 111.4\ FT^2$
- AREA STAIRS
  $2(9' \times 7.5') = 135\ FT^2$
- NET AREA
  $3628.5 - 135 = 3493.5\ FT^2$
- # SHEETS
  $\frac{3493.5}{111.4} = 31.35$
  $\approx \boxed{32\ SHEETS}$

CORRIDORS AREA

- PER FLOOR
  $(6' \times 88.5') = 531\ FT^2$
- AREA SHEET
  $16.5' \times 6.0' = 99\ FT^2$
- # SHEETS $\frac{531}{99} = 5.4$
  $\times 3$ FLOORS $= 16.2$
  $\approx \boxed{17\ SHEETS}$



- LOADS ACTING ON ROOF BEAMS

- LIVE LOADS:
  SNOW = $\boxed{35\ PSF}$
- DEAD LOADS:
  + 3½" SLAB (REINFORCED)
  $150\ PSF \times \frac{3.5''}{12} = 43.8\ PSF$
  + STEEL DECK PAINTED FOR FIREPROOF
  16 GAUGE ≈ 3 PSF
  + 3 ply FELT & GRAVEL
  5.5 PSF
  + SUSPENDED LIGHTNING AND AIR-DISTRIBUTION SYSTEMS
  3 PSF

TOTAL DEAD LOAD = $\boxed{55.3\ PSF}$

- BEAMS TRIBUTARY AREA (LARGEST)

$20.5' \times 8.25' = 169\ FT^2$

- LOADS ON EACH BEAM

= TRIB AREA x WEIGHT (PSF)

- LIVE LOAD

$169\ ft^2 \times 35\ \frac{lbs}{ft^2} = 5915\ lbs$

$l = \frac{5915\ lbs}{20.5'} = 288\ lbs/FT = \boxed{0.29\ K/FT}$

- DEAD LOAD

- EXTERNAL $169\ FT^2 \times 55.3\ \frac{LBS}{FT^2} = 9346\ lbs$

$\frac{9346}{20.5} = 456\ lbs/FT$

$\approx 0.46\ K/FT$

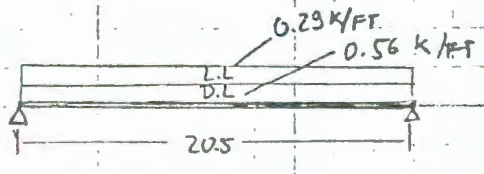
- SELF WEIGHT

(ESTIMATED) = 100 lbs/FT (TO BE SAFE)

= .1 K/FT

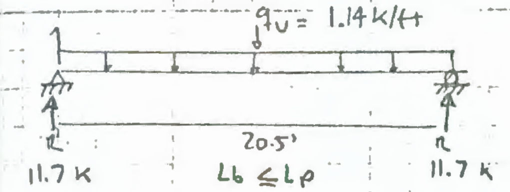
TOTAL DEAD LOAD = $\boxed{0.56\ K/FT}$

SAMPLE BEAM

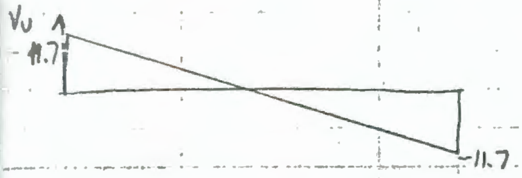


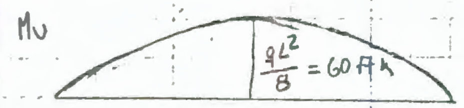
- FROM LRFD A.4.1

$$W_U = 1.2D + 1.6L = 1.2(0.56) + 1.6(0.29) = \underline{1.14\ K/FT}$$



$R = \dfrac{1.14 \times 20.5}{2} = 11.7$ kips





$$\frac{qL^2}{8} = \frac{1.14(20.5)^2}{8} = \underline{60\ ft\,k}$$

- <u>BEAM</u> <u>DESIGN</u> (USING STRUCTURAL STEEL DESIGN (J.C. SMITH) TEXTBOOK & LRFD 1986)

  - SPAN = <u>20.5 ft</u>
  - assume COMPRESSION FLANGE CAN BE LATERALLY BRACED $\underline{L_b \le L_p}$
  - LIMITING DEFLECTION DUE TO SERVICE LIVE LOAD SPAN/360

$$\Rightarrow \frac{20.5' \times 12''}{360''} = \underline{0.68\ in}$$

  - SELECT LIGHTEST W SECTION FOR $F_y = 36$ KSI STEEL

$$\phi M_{nx} = 0.9 M_{px} \le M_{ux} = 60\ ft\,k$$

- FROM LRFD 3-16

<u>W 10x22</u> (Lightest W section)

$\phi M_{px} = 70.2$ ft k $\ge (M_U = 60)$; $L_p = 5.5$; $Z_x = 26.0$; $L_r = 16.9$

- FROM LRFD 1-34

Area = 6.49 in^2 ; d = 10.17" ; t_w = 0.24" ; I_x = 118 in^4
b_f = 5.75" , t_f = 0.36" ; h_c/t_w = 36.9

- <u>CHECK</u> <u>LOCAL</u> <u>BUCKLING</u>

  - FLANGE $\left[\dfrac{0.5 b_f}{t_f} = \dfrac{0.5(5.75)}{0.36} = 7.98\right] \le \left[\lambda_p = \dfrac{65}{\sqrt{36}} = 10.8\right]$ FLANGE OK ✓

  - WEB $\left[\dfrac{h_c}{t_w} = 36.9\right] \le \left[\lambda_p = \dfrac{640}{\sqrt{36}} = 106.7\right]$ WEB OK ✓

- CHECK SHEAR (SEE LRFD F2)

$$\left(h_c/t_w = 36.9\right) \leq \left(187 \times \sqrt{K/F_{yw}}\right)$$
$$\leq 187 \times \sqrt{5/36} = 69.7$$

$$\phi V_n = 0.9 \times 0.6 \times f_{yw} \times A_w \rightarrow (d \times t_w)$$
$$= 0.9 \times 0.6 \times 36 \times (10.17 \times 0.24) = 47.5_k > V_u = 11.7_k$$

OK ✓

- CHECK DEFLECTION (SEE LRFD P63-130)

Service live load, $w = 0.29$ K/FT ; limiting deflection : 0.68 in

$$\Delta max = \frac{5wL^4}{384\,EI}$$

$$\Delta max = \frac{5 \times (0.29 \times 20.5^4 \times (12\,in/ft)^3}{384\,(29000)\,(118)} = 0.337\,in \leq \text{limiting deflection } 0.68\,in$$

OK ✓

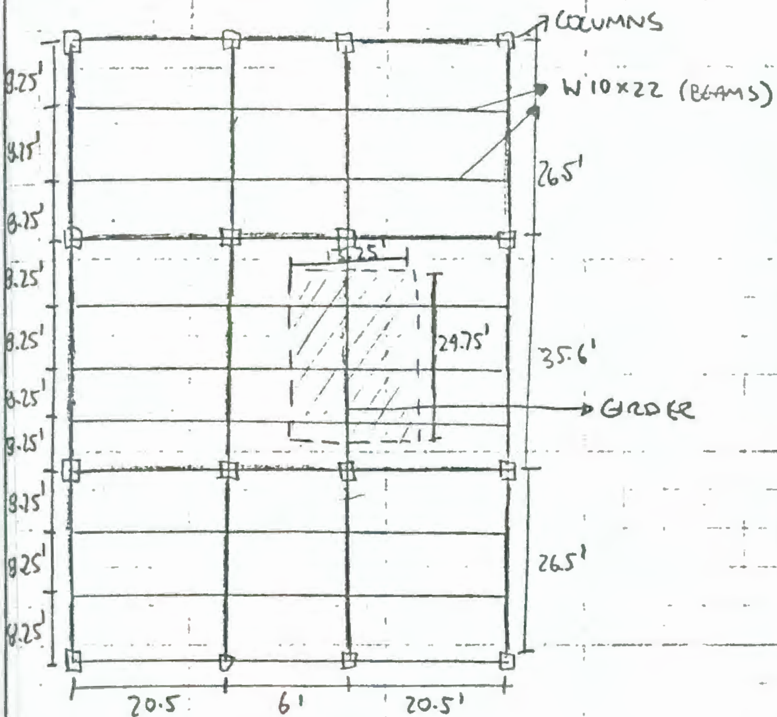
* ALL DESIGN REQUIREMENTS ARE SATISFIED

USE $\boxed{W\,10 \times 22}$ FOR ROOF BEAMS

NOMINAL WEIGHT = 22 lb/ft
d = 10 1/8"

- FOR APTS AREA
  NEED 14 $\boxed{W\,10 \times 22}$ WITH A LENGTH OF 20.5'
- FOR CORRIDOR AREA (SEE DESIGN FOR CORRIDOR BEAMS IN FLOORS SECTION)
  NEED 7 $\boxed{W\,6 \times 9}$ WITH A LENGTH OF 6'

* NOTE : THE DESIGN FOR W 6x9 6' LONG , CAN BE SEEN IN PAGES 15-16 OF THIS APPENDIX.



LOADS ACTING ON ROOF GIRDERS

- LIVE LOADS:
  SNOW: $\boxed{35\ PSF}$
- DEAD LOADS:
  + $3\frac{1}{2}$" SLAB (REINFORCED)
  $150\ PSF \times \frac{3.5"}{12} = 43.3\ PSF$
  + STEEL DECK PAINTED FOR FIREPROOF
  16 GAUGE $\approx$ 3 PSF
  + 3 PLY FELT & GRAVEL
  5.5 PSF
  + SUSPENDED LIGHTING AND AIR DISTRIBUTION SYSTEMS
  3 PSF

TOTAL DEAD LOAD = 55.3 PSF

- GIRDERS TRIBUTARY AREA
  (DESIGN FOR LARGEST AREA TO BE SAFER)

$(24.75' \times 13.25') = 334.13$
$\approx 335\ FT^2$

- LOADS ACTING ON EACH GIRDER

= TRIB AREA × WEIGHT (PSF)

- LIVE LOAD

$335\ FT^2 \times 35\ \frac{lbs}{FT^2} = 11725\ lbs$

$\frac{11725}{l = 35.5} = 330\ lbs/ft$

$= \boxed{0.33\ K/FT}$

- DEAD LOAD

- EXTERNAL • $335\ FT^2 \times 55.3\ \frac{lbs}{FT^2} = 18526\ lbs$

• 3 (W10x22 BEAMS)
$3(22\ lbs/FT \times 13.25') = 874.5\ lbs$

+

19400.5 lbs

$\frac{19400.5}{l = 35.5} = 546\ lbs/ft$

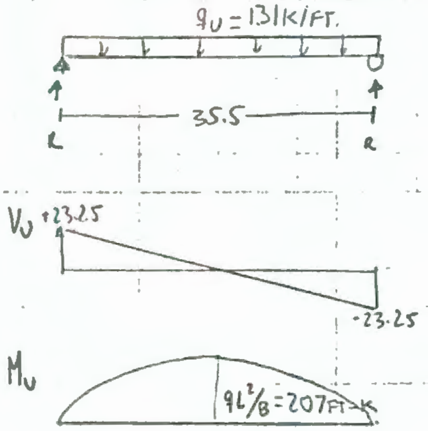
$= 0.55\ k/FT$

- SELF WEIGHT (ESTIMATED)
= 100 lbs/FT (TO BE SAFE)
= 0.1 k/FT

TOTAL DEAD LOAD = $\boxed{0.65\ K/FT}$

- FROM LRFD A.4.1

$$W_u = 1.2D + 1.6L = 1.2(0.65) + 1.6(0.33) = \underline{1.31 \text{ K/FT}}$$



$$R = \frac{1.31 \times 35.5}{2} = \underline{23.25 \text{ K}}$$

$$\frac{qL^2}{8} = \frac{1.31\,(35.5^2)}{8} = 206.4 \text{ FT-K}$$
$$\approx \underline{207 \text{ FTK}}$$

- GIRDER DESIGN (USING STRUCTURAL STEEL DESIGN (J.C SMITH) TEXTBOOK & LRFD 1986)

  - SPAN = 35.5 FT
  - ASSUME $L_b \le L_p$
  - LIMITING DEFLECTION DUE TO SERVICE LIVE LOAD

$$\text{SPAN}/360 \Rightarrow \frac{35.5' \times 12''}{360''} = \underline{1.18 \text{ in}''}$$

  - SELECT LIGHTEST W SECTION FOR $F_y = 36$ ksi STEEL

$$\phi M_{nx} = 0.9\, M_{px} \le M_{ux} = 207 \text{ FTK}$$

- FROM LRFD 3-16

W 18x40 (LIGHTEST W SECTION)

$\phi M_{px} = 212$ FTK $> M_u = 207$ FTK ; $L_p = 5.3$; $Z_x = 78.4$; $L_r = 15.7$

- FROM LRFD 1-26
  AREA $= 11.8$ in^2 ; $d = 17.90''$; $t_w = 0.315''$; $I_x = 612$ in^4
  $b_f = 6.015''$, $t_f = 0.525''$; $h_c/t_w = 51$

  - CHECK LOCAL BUCKLING

  - FLANGE $\left[\frac{0.5 b_f}{t_f} = 0.5\left(\frac{6.015}{0.525}\right) = 5.73\right] \le \left[\lambda_p = \frac{65}{\sqrt{36}} = 10.8\right]$ FLANGE OK ✓

• WEB $\left[h_c/t_w = 51 \right] \leq \left[\lambda_p = \frac{640}{\sqrt{36}} = 106.7 \right]$ WEB OK ✓

- CHECK SHEAR (SEE LRFD F2)

$$(h_c/t_w = 51) \leq \left(187\sqrt{k/F_{yw}}\right)$$
$$\leq 187 \times \sqrt{5/36} = 69.7$$

$$\phi V_n = 0.9 \times 0.6 \times f_{yw} \times A_w \rightarrow (d \times t_w)$$
$$= 0.9 \times 0.6 \times 36 \times (17.9'' \times 0.315'') = \underline{109.6\,k > V_u = 23.25} \checkmark OK$$

- CHECK DEFLECTION (SEE LRFD P6-130)

SERVICE LIVE LOAD, $w = 0.33\,k/FT$; LIMITING DEFLECTION = $\underline{1.18''}$

$$\Delta max = \frac{5wL^4}{384\,EI}$$

$$\Delta max = \frac{5 \times (0.33 \times 35.5^4 \times (12in/ft)^3}{384\,(29000)\,(612)} = \underline{0.66''} \leq \text{LIMITING DEFECTION } 1.18''$$

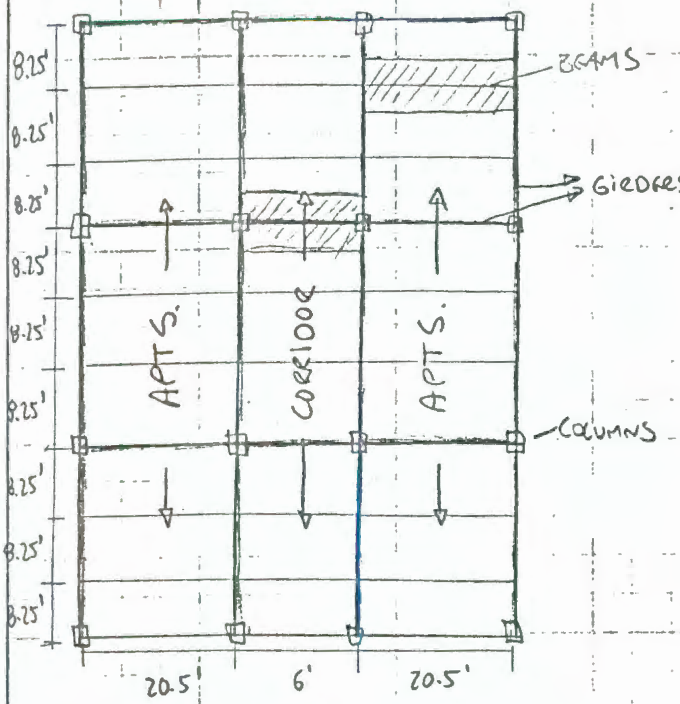
OK ✓

* ALL DESIGN REQUIREMENTS ARE SATISFIED

USE $\boxed{W\,18 \times 40}$ FOR ROOF GIRDERS

NOMINAL WEIGHT = 40 lb/ft
d = 17.9"

- NEED 8 $\boxed{W18 \times 40}$ WITH A LENGTH OF 26.5'
- NEED 4 $\boxed{W18 \times 40}$ WITH A LENGTH OF 35.5'
- NEED 8 $\boxed{W18 \times 40}$ WITH A LENGTH OF 20.5'
- NEED 4 $\boxed{W18 \times 40}$ WITH A LENGTH OF 6'



NOTE: SINCE THE FIRST, SECOND, AND THIRD FLOORS CARRY THE SAME LOADS, THE DESIGNED BEAMS WILL APPLY TO THE THREE FLOORS

- LIVE LOADS ACTING ON FLOOR

  40 PSF (PRIVATE APARTMENTS)
  100 PSF (CORRIDORS)

1. DESIGN OF BEAMS FOR APARTMENTS AREA

- LIVE LOAD

  40 PSF (PRIVATE APTS)

- DEAD LOADS

  + 4" SLAB (REINFORCED)

  $150 \text{ PSF} \times \frac{4''}{12''} = \underline{50 \text{ PSF}}$

  + STEEL DECK PAINTED FOR FIREPROOF 16 GAUGE ≈ 3 PSF

  + SUSPENDED LIGHTNING AND AIR DISTRIBUTION SYSTEMS = 3 PSF

  TOTAL DEAD LOAD = $\boxed{56 \text{ PSF}}$

- BEAM TRIBUTARY AREA

  (DESIGN FOR LARGEST TRIB AREA, TO BE SAFER)

  $20.5' \times 8.25' = \underline{169 \text{ FT}^2}$

- LOADS ON EACH BEAM

= TRIB. AREA × WEIGHT (PSF)

+ LIVE LOAD

$169\ FT^2 \times 40\ \frac{LBS}{FT^2} = 6760\ lbs$

$\ell = \frac{6760\ LBS}{20.5'} = 330\ lbs/FT$

$= \boxed{0.33\ K/FT}$

+ DEAD LOAD

- $169\ FT^2 \times 56\ \frac{LBS}{FT^2} = 9464\ LBS$

$\ell = \frac{9464}{20.5} = 462\ lbs/ft$

$\approx 0.46\ k/FT$

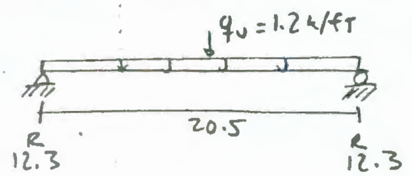
- SELFWEIGHT (ESTIMATED) = 100 lbs/FT (TO BE SAFE)

$\approx .1\ k/FT$

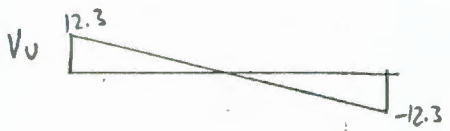
TOTAL DEAD LOAD = $\boxed{0.56\ K/FT}$

– FROM LRFD A.4.1

$W_u = 1.2D + 1.6L = 1.2(0.56) + 1.6(0.33) = \underline{1.2\ k/FT}$



$R = \frac{1.2 \times 20.5}{2} = 12.3\ KIPS$



$\frac{qL^2}{8} = \frac{1.2(20.5)^2}{8} = \underline{63\ FT\text{-}K}$

- DESIGN (USING STRUCTURAL STEEL DESIGN (JC SMITH) TEXTBOOK & LRFD 1986)

  - SPAN = 20.5 ft
  - ASSUME $L_b < L_p$
  - LIMITING DEFLECTION DUE TO SERVICE LIVE LOAD

$SPAN/360 = \frac{20.5' \times 12''}{360''} = 0.68\ in$

- SELECT LIGHTEST W SECTION FOR $F_y = 36\ KSI$ STEEL

$\phi M_{nx} = 0.9 M_{px} \geq M_{ux} = 63\ ftk$

- FROM LRFD 3-16

$\underline{W\,10 \times 22}$ (LIGHTEST W SECTION)

$\phi M_{px} = 70.2$ ft k $\geq (M_u = 63$ k$)$; $L_p = 5.5$; $Z_x = 26.0$; $L_r = 16.9$

- FROM LRFD 1-34

Area $= 6.49$ in^2 ; $d = 10.17"$; $t_w = 0.24"$; $I_x = 118$ in^4

$b_f = 5.75"$, $t_f = 0.36"$; $h_c/t_w = 36.9$

- CHECK LOCAL BUCKLING

- FLANGE $\left[\dfrac{0.5 b_f}{t_f} = \dfrac{0.5(5.75)}{0.36} = 7.98\right] \leq \left[\lambda_p = \dfrac{65}{\sqrt{36}} = 10.8\right]$ FLANGE OK ✓

- WEB $\left[h_c/t_w = 36.9\right] \leq \left[\lambda_p = \dfrac{640}{\sqrt{36}} = 106.7\right]$ WEB OK ✓

- CHECK SHEAR (SEE LRFD F2)

$$(h_c/t_w = 36.9) \leq (187 \times \sqrt{k/F_{yw}})$$

$$\leq 187 \times \sqrt{5/36} = 69.7$$

$$\phi V_n = 0.9 \times 0.6 \times f_{yw} \times A_w \rightarrow (d \times t_w)$$

$$= 0.9 \times 0.6 \times 36 \times (10.17 \times 0.24) = \underline{47.5\text{ k} > V_u = 12.3\text{ k}}$$

OK ✓

- CHECK DEFLECTION (SEE LRFD P63-130)

SERVICE LIVE LOAD , $w = 0.33$ k/FT ; LIMITING DEFLECTION $= \underline{0.68\text{ in}}$

$$\Delta_{MAX} = \frac{5wL^4}{384\,EI}$$

$$\Delta_{MAX} = \frac{5 \times (0.33 \times 20.5^4 \times 12\text{ in/ft}^3}{384\,(29000)\,(118)} = 0.38" \leq \underline{0.68"}$$

OK ✓

* ALL DESIGN REQUIREMENTS ARE SATISFIED

∴ FOR APTS AREA

NEED 14 / PER FLOOR $\boxed{W10 \times 22}$ WITH A LENGTH OF 20.5'

FOR 3 FLOORS ; NEED 42 W10x22

* NOTE : THE BEAM DESIGN FOR THE APTS FLOOR AREAS CAME TO BE THE SAME AS THE ROOF, WHERE A W 10x22 WAS ALSO THE LIGHTEST W SECTION.

2. DESIGN OF BEAMS FOR CORRIDORS AREA

- LIVE LOAD

  100 PSF (CORRIDORS)

- DEAD LOADS

  + 4.5" SLAB (REINFORCED)

    $150\text{ PSF} \times \frac{4.5''}{12''} = \underline{56.25\text{ PSF}}$

  + STEEL DECK PAINTED FOR FIREPROOF 16 GAUGE ≈ 3PSF

  + SUSPENDED LIGHTNING AND AIR DISTRIBUTION SYSTEMS ≈ 3PSF

    TOTAL DEAD LOAD = 62.25 ≈ $\boxed{63\text{ PSF}}$

- BEAM TRIBUTARY AREA

  (DESIGN FOR THE LARGEST TRIB AREA, TO BE SAFER)

  $6' \times 8.25' = \underline{50\text{ FT}^2}$

- LOADS ON EACH BEAM

  = TRIB. AREA x WEIGHT (PSF)

  + LIVE LOAD

    $50\text{ FT}^2 \times 100\text{ LBS/FT}^2 = 5000\text{ LBS}$

    $\frac{5000}{\ell = 6'} = 833\text{ lbs/FT}$

    $= \boxed{0.84\text{ lbs/FT}}$

  + DEAD LOAD

    - $50\text{ FT}^2 \times 63\ \frac{\text{LBS}}{\text{FT}^2} = 3150\text{ LBS}$

    $\frac{3150}{\ell = 6} = 525\text{ lbs/FT}$

    $\approx \underline{0.53\text{ k/FT}}$

    - SELFWEIGHT

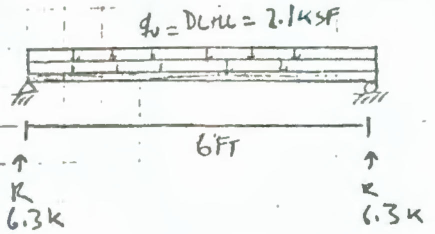
      (ESTIMATED) = 100 lbs/FT (TO BE SAFE)

      = .1 k/FT

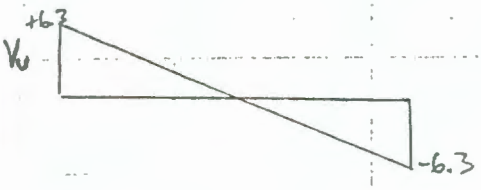
    TOTAL DEAD LOAD = $\boxed{0.63\text{ k/FT}}$

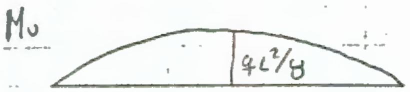
- FROM LRFD 4.4.1

  $W_u = 1.2D + 1.6L = 1.2(0.63) + 1.6(0.84) = \underline{2.1\text{ k/FT}}$



$R = \frac{2.1 \times 6}{2} = 6.3\text{ k}$





$\frac{qL^2}{8} = \frac{2.1 \times 6^2}{8} = 9.45$
$\approx$ 10 FT-K

- DESIGN (USING STRUCTURAL STEEL DESIGN J.C SMITH TEXTBOOK & LRFD 1986)
  - SPAN = 6 ft
  - ASSUME: $L_b \leq L_p$
  - LIMITING DEFLECTION DUE TO SERVICE LIVE LOAD

$$\text{SPAN}/360 = \frac{6' \times 12''}{360''} = 0.2''$$

- SELECT LIGHTEST W SECTION FOR $F_y = 36$ ksi STEEL

$$\phi M_{nx} = 0.9\, M_{px} \leq M_{ux} = 10 \text{ ftk}$$

- FROM LRFD 3-17
  W 6x9 (LIGHTEST W SECTION)

$\phi M_{px} = 16.8$ ftk $> M_{ux} = 10$ ftk ; $L_p = 3.8$; $Z_x = 6.23$; $L_r = 12.0$

- FROM LRFD 1-36

$A_{req} = 2.68$ in^2 ; $d = 5.9''$; $t_w = 0.170''$; $I_x = 16.4$ in^4
$b_f = 3.94''$, $t_f = 0.215''$; $h_c/t_w = 29.2$

- CHECK LOCAL BUCKLING

  - FLANGE $\left[\frac{0.5 b_f}{t_f} = \frac{0.5(3.94)}{0.215} = 9.16\right] \leq \left[\lambda_p = \frac{65}{\sqrt{36}} = 10.8\right]$ FLANGE OK✓

- CHECK SHEAR (SEE LRFD F2)

$$(h_c / t_w) = 29.2 \le (137 \times \sqrt{K/F_{yw}}$$
$$\le 137 \times \sqrt{5/36} = 69.7 \checkmark$$

$$\phi V_n = 0.9 \times 0.6 \times f_{yw} \times A_w \rightarrow (d \times t_w)$$
$$= 0.9 \times 0.6 \times 36 \times (5.9 \times 0.215) = 24.66\,k \geq V_u = 6.3\,k$$

OK ✓

- CHECK DEFLECTION (SEE LRFD P63-130)

SERVICE LIVE LOAD $W = 0.84\ k/FT$; LIMITING DEFLECTION: $0.2''$

$$\Delta max = \frac{5 w L^4}{384 EI}$$

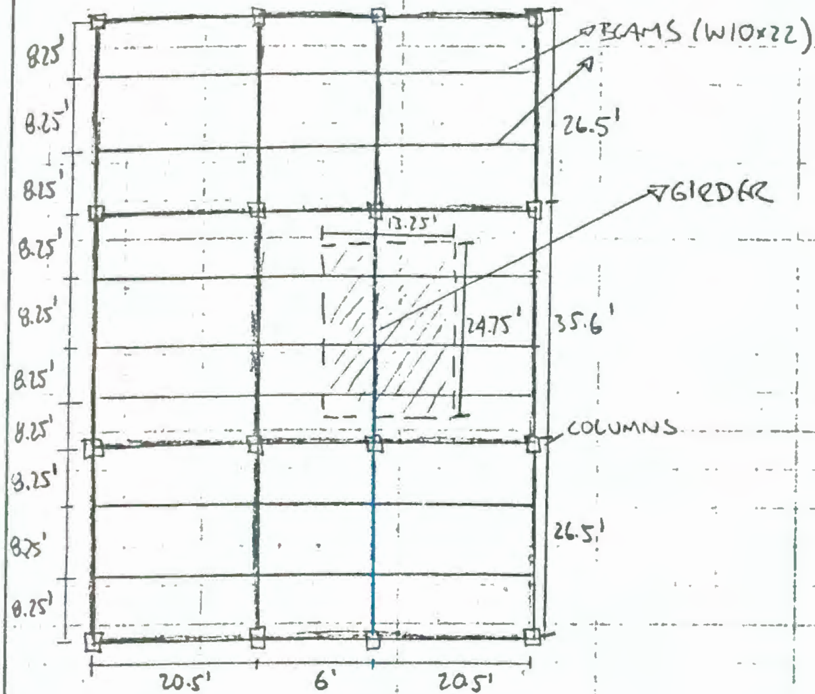
$$\Delta max = \frac{5 (0.84) (6)^4 \times 12 in/ft^3}{384 (29000)(16.4)} = 0.05'' \le 0.21'' \text{ OK} \checkmark$$

* ALL DESIGN REQUIREMENTS ARE SATISFIED

∴ FOR CORRIDORS AREA
NEED 7 PER FLOOR $\boxed{W6 \times 9}$ WITH A 6' LENGTH

FOR 3 FLOORS + ROOF ; NEED 28 W6x9 6' long

NOTE: THIS DESIGN WILL APPLY TO ALL THREE FLOORS, SINCE DESIGN IS. THIS DESIGN IS TO WITHSTAND THE HEAVIEST LOADS.



- GIRDERS TRIBUTARY AREA
(DESIGN FOR LARGEST AREA, TO BE SAFER)

$(24.75' \times 13.25') = 334.13$

$\approx \underline{335\ FT^2}$

LOADS ACTING ON FLOOR GIRDERS

- LIVE LOADS: (DESIGN FOR THE HIGHEST LL (CORRIDORS)

CORRIDORS = 100 PSF

- DEAD LOADS:

HORIZONTAL

T $4\frac{1}{2}''$ SLAB (REINFORCED)

$+\ 150\ PSF \times \frac{4.5''}{12} = \underline{56.25\ PSF}$

+ STEEL DECK PAINTED FOR FIREPROOF 16 GAUGE $\approx$ $\underline{3\ PSF}$

+ SUSPENDED LIGHTNING AND AIR DISTRIBUTION SYSTEMS $\underline{3\ PSF}$

$= \underline{62.25\ PSF}$

VERTICAL

- WALLS
- 8" HOLLOW CONCRETE BLOCK w/ HEAVY AGGREGATE $\underline{55\ PSF}$
- STEEL PARTITIONS $\underline{4\ PSF}$
- PLASTER 1" GYPSUM $\underline{5\ PSF}$

$+\ \overline{64\ PSF}$

- LOADS ACTING ON EACH GIRDER

- LIVE LOAD

$335\ FT^2 \times 100\ \frac{lbs}{FT^2} = 33500\ lbs$

$\frac{33500}{l = 35.6} = 941\ lbs/ft$

$\approx \boxed{.94\ k/ft}$

- DEAD LOAD
  - HORIZONTAL

$335\ FT^2 \times 62.25\ \frac{LBS}{FT^2} = \underline{20854\ lbs}$

+ WEIGHT BEAMS
$\approx 3(W10 \times 22)$

$3(22\ lbs/ft \times 13.25') = \underline{874.5\ lbs}$

$21729\ lbs$

$\frac{21729}{l = 35.6} = 611\ lbs/ft$

$= \boxed{0.61 K/ft}$

- VERTICAL

WALLS $64\ \frac{LBS}{FT^2} \times 10.5'(\text{HEIGHT}) = 672\ lbs/ft$

$\approx \boxed{0.68 k/ft}$

- SELF WEIGHT (ESTIMATED)

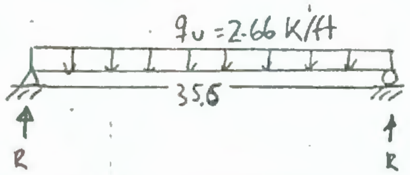
$= 100\ lbs/ft = \boxed{0.1\ k/ft}$

TOTAL DEAD LOAD

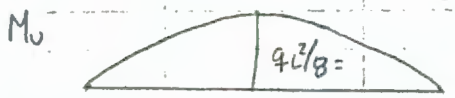
$0.6 + 0.68 + 0.1 = \boxed{1.4\ k/ft}$

-FROM LRFD A.4.1

$$W_u = 1.2D + 1.6L = 1.2(1.4) + 1.6(0.61) = \underline{2.66\ K/FT}$$



$$R = \frac{2.66 \times 35.6}{2} = \underline{47.35\ K}$$



$$\frac{qL^2}{8} = \frac{2.66(35.6)^2}{8} = 421.39$$
$$\approx \underline{422\ FTK}$$

- GIRDER DESIGN (USING STRUCTURAL STEEL DESIGN J.C SMITH TEXTBOOK & LRFD 1986)

  - SPAN : 35.6 FT
  - ASSUME $L_b \leq L_p$
  - LIMITING DEFLECTION DUE TO SERVICE LIVE LOAD

$$SPAN/360 \Rightarrow \frac{35.5' \times 12''}{360''} = \underline{1.18\ in''}$$

- SELECT LIGHTEST W SECTION FOR $F_y = 36\ ksi$ STEEL

$$\phi M_{nx} = 0.9 M_{px} \leq M_{ux} = 422\ FT\text{-}K$$

- FROM LRFD 3-15

  W 21x68 (lightest W section)

$$\phi M_{px} = 432\ FTK > M_u = 422\ FT\text{-}K;\ L_p = 5.6;\ Z_x = 134;\ L_r = 16.6$$

- FROM LRFD 1-24
  AREA = 20.0 in²; $d = 21.13''$; $t_w = 0.43''$; $I_x = 1480\ in^4$
  $b_f = 8.27''$; $t_f = 0.685''$; $h_c/t_w = 43.6$

- CHECK LOCAL BUCKLING
  - FLANGE $\left[\frac{0.5 b_f}{t_f} = 0.5\left(\frac{8.27}{0.685}\right) = 6.03\right] \leq \left[\lambda_p = \frac{65}{\sqrt{36}} = 10.9\right]$ FLANGE OK ✓
  - WEB $\left[h_c/t_w = 43.6\right] \leq \left[\lambda_p = \frac{640}{\sqrt{36}} = 106.7\right]$ WEB OK ✓

- CHECK SHEAR (SEE LRFD F2)

$(h_c/t_w = 43.6) \leq \left(187\sqrt{k/F_{yw}}\right)$
$\leq 187 \times \sqrt{5/36} = 69.7$

$\phi V_n = 0.9 \times 0.6 \times f_{yw} \times A_w \rightarrow (d \times t_w)$
$= 0.9 \times 0.6 \times 36 \times (21.13'' \times 0.43'') = \underline{176.6k > V_u = 47.35}$

OK ✓

- CHECK DEFLECTION (SEE LRFD PG 63-130)

SERVICE LIVE LOAD, $w = 0.94$ k/ft ; LIMITING DEFLECTION = $\underline{1.18''}$

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

$$\Delta_{max} = \frac{5 \times (0.94 \times 35.6^4 \times (12in/ft)^3}{384\ (29000)\ (1480)} = \underline{0.79'' \leq 1.18''}$$

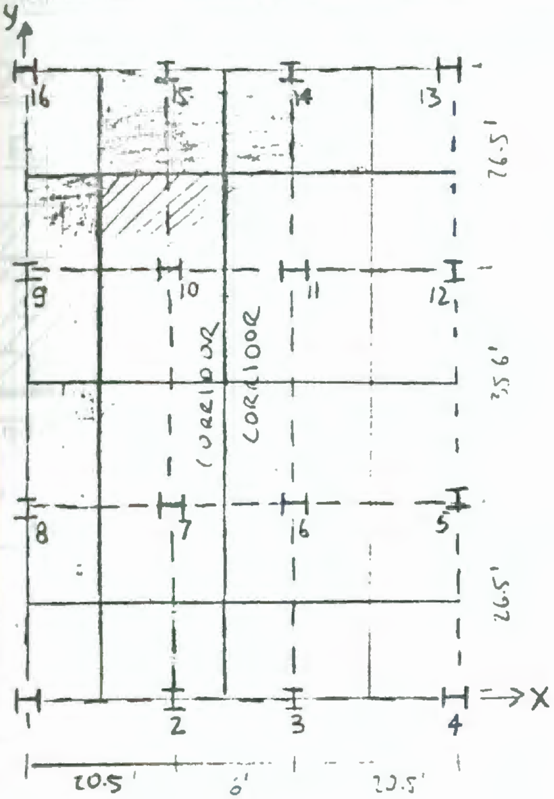
OK ✓

* ALL DESIGN REQUIREMENTS ARE SATISFIED

USE $\boxed{W21 \times 68}$ FOR FLOOR GIRDERS

FOR EACH FLOOR

8 W21 x68 WITH A LENGTH OF 26.5' X 3 FLOORS = 24
4 W 21 x 68 WITH A LENGTH OF 35.5' X 3 " = 12
8 W 21 x 68 WITH A LENGTH OF 20.5' X 3 " = 24
4 W 21 x 68 WITH A LENGTH OF 6' X 3 " = 12



- The column design will be based on W-shapes in accordance to the AISC [illegible]. The benefit of choosing a W [illegible] as a column [illegible] since it is a doubly symmetric cross section only twists when the column buckles (torsional mode of column buckling), we just have to [illegible] for column buckling.

- To [illegible] a balance between safety and economy we decided to standardize the columns by designing the basement columns and applying them all the way up, since the building consists of only four floors. However we want to have some economy, therefore we decided to design for four types of columns, based on the dead and live loads [illegible].
  1. Corner columns (1, 4, 16, 13)
  2. N-S edge columns (8, 9, 5, 12)
  3. E-W edge columns (2, 3, 14, 15)
  4. Interior columns (6, 7, 10, 11)

- Note that each column cross section in the corner of the building is rotated 90° with respect to the [illegible] cross sections. This was done on purpose to [illegible] major principal axis [illegible] bending strength [illegible] directions (X and Y)
  - See [illegible] Structural Steel Design Textbook, 1991) Pg. 144

- LOADS ACTING ON THE COLUMNS

- Roof Loads

LL - snow . 35 PSF

[illegible] $= \frac{[illegible]}{2} = 43.5$ PSF

[illegible]

[illegible] $\approx$ 30 PSF

\+ [illegible] . 5.5 PSF

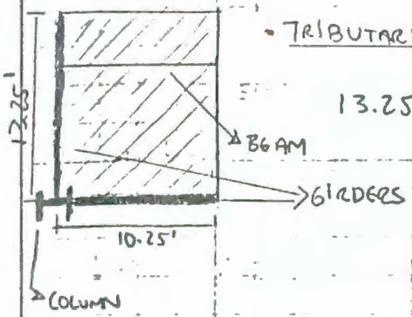
\+ [illegible] $\approx$ 30 PSF

Total = 90.3 PSF

NOTE: THE COLUMN DESIGN WILL BE MADE FOR THE BASEMENT COLUMNS AND WILL BE REPETITIVE FOR THE FLOORS ABOVE.

① CORNER COLUMNS (1, 4, 16, 13) (REFER TO SKETCH PG. 21)

Ex: COLUMN 1



• TRIBUTARY AREA

$$13.25' \times 10.25' = \underline{135.8\ FT^2} \approx \underline{136\ FT^2}$$

• ROOF LOADS (FROM PG.21)

+ LIVE & DEAD UNIFORM LOADS → $90.3\ \frac{lbs}{FT^2} \times 136\ FT^2 = \underline{12281\ lbs}$

+ 1 W 10x22 (BEAM)
  22 lbs/ft x 10.25 ft = = = = = = 225 lbs

+ 2 W 18 x 40 (GIRDERS)
  40 lbs/ft x 10.25 ft = 410 lbs
  40 lbs/ft x 13.25 ft = 530 lbs
  + 940 lbs ... = 940 lbs

TOTAL ROOF LOADS = 13,446 lbs → ≈ $\boxed{13.5\ \text{KIPS}}$

• FLOOR LOADS

+ LIVE LOAD . . . . 40 PSF

+ 4" SLAB (REINFORCED)
  150 PSF x $\frac{4.0''}{12}$ ..... 50 PSF

+ STEEL DECK PAINTED FOR FIRE PROOF 16 GAUGE ..... 3 PSF

+ SUSPENDED LIGHTING AND AIR DISTRIBUTION SYSTEMS ..... 3 PSF
+

• WALLS
- 8" HOLLOW CONCRETE BLOCK W/ HEAVY AGGREGATE .. 55 PSF
- STEEL PARTITIONS . . . . 4 PSF
- PLASTER 1" GYPSUM . . . . 5 PSF
  64 PSF

$64\ lbs/ft^2 \times 10.5'$ (Height) ....
672 lbs/ft

• TOTAL UNIFORM LOADS = 96 PSF x 136 ft² (TRIB. AREA) = 13056 lbs

+ TOTAL WALL LOADS
  672 lbs/ft x 10.25 ft . . . . = 6888 lbs
  672 lbs/ft x 13.25 ft = 8904 lbs

+ 1 W 10 x 22 (BEAM)
  22 lbs/ft x 10.25 ft . . . . = 225.5 lbs

+ 2 W 21 x 68 (GIRDERS)
  68 lbs/ft x 10.25 ft . . . . = 697 lbs
  68 lbs/ft x 13.25 ft . . . . = + 901 lbs
  30,671 lbs

TOTAL FLOOR LOADS = 30671 lbs
⇒ $\boxed{30.7\ \text{Kips}}$

- TOTAL LOAD ACTING
  ON CORNER COLUMNS (IN BASEMENT)

= ROOF LOAD + 3 FLOORS LOAD − (WEIGHT OF COLUMNS ESTIMATED) ≈ 100 lb/ft

= 13.5 + 3(30.7) − (100 lb/ft × 38') → HEIGHT OF ALL COLUMNS

= 13.5 + 92.1 − 3.8 k

= 109.4 k ⇒ $\boxed{P_u = 110 \text{ KIPS}}$

- LENGTH OF COLUMNS
  DESIGN FOR
  LONGEST COLUMNS = 10.5 FT

a. CHECK Y-Y AXIS

ASSUME COLUMNS PIN CONNECTED AT TOP AND BOTTOM WITH SIDESWAY INHIBITED.
FROM TABLE C-C2.1 IN THE COMMENTARY (AISC) for condition (d)
K = 1.0 :

- EFFECTIVE LENGTH $(KL)_y$ = 10.5 FT

- SINCE ALL SECTIONS OF A 36 STEEL SATISFY LRFD TABLE B5.1
  SELECT A SECTION THAT SATISFIES THE DESIGN REQUIREMENTS
  FOR THE Y-AXIS

∴ $\phi P_n > (P_u = 110 \text{ k})$ FROM PG. 2-29 AISC

For $(KL)_y = 10$ ⇒ W8x24 = $\phi P_n = 162$ k
$(KL)_y = 11$ ⇒ W8x24 : $\phi P_n = 152$ K

For $(KL)_y = 10.5$ ⇒ W8x24 : $\phi P_n = 157\text{k} > 110\text{K}$ ✓ OK

b. CHECK X-X AXIS

ASSUME SIDESWAY UNINHIBITED AND PIN CONNECTED AT BASE

FROM TABLE C-C2.1 (AISC) for CONDITION (f):
K = 2.0 :

- EFFECTIVE LENGTH
  $(KL)_x = 2 \times 10.5 = 21$ ft

- FROM AISC 2-29 FOR A W8x24
  $r_x/r_y = 2.12$

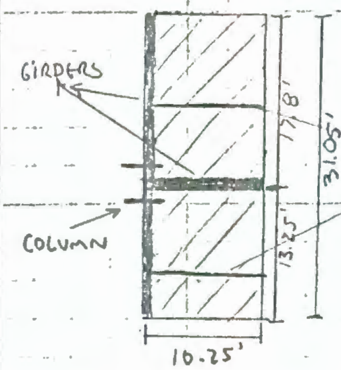
CORRESPONDING EFFECTIVE LENGTH RELATIVE TO THE y-y AXIS.

$\frac{21.0}{2.12} = 9.9 \text{ ft} < 10.5 \text{ ft}$ ✓ OK ∴ EFFECTIVE LENGTH FOR y-y AXIS IS CRITICAL

THE $\boxed{W 8 \times 24}$ ✓ OK

② N-S EDGE COLUMNS DESIGN (8, 9, 5, 12) (REFER TO SKETCH PG.21 OF THIS APPENDIX)

EX : COLUMN 8



• TRIBUTARY AREA

$(31.05' \times 10.25') = 318.26\,FT^2$
$\approx \underline{319\,FT^2}$

• ROOF LOADS (FROM PG.22 ON THIS APPENDIX)

+ LIVE & DEAD UNIFORM LOADS $\Rightarrow 90.3\ \frac{lbs}{FT^2} \times \overset{TRIB.\ AREA}{319\,FT^2} = \underline{28806\ lbs}$

+ 2 W 10x22 (BEAMS)
$2 \times (22\,lb/ft \times 10.25\,ft)$ .... = $\underline{451\ lbs}$

+ 2 W 18x40 (GIRDERS)
$(40\,lb/ft \times 17.8\,ft)$ .... = $\underline{712\ lbs}$
$(40\,lb/ft \times 13.25\,ft)$ .... = + $\underline{530\ lbs}$

TOTAL ROOF LOADS = $\underline{30\,499\ lbs} \Rightarrow \boxed{30.5\ K}$

• FLOOR LOADS

+ LIVE LOAD .... 40 PSF
+ 4" SLAB (REINFORCED)
$150\ PSF \times \frac{4.0''}{12}$ .... 50 PSF
+ STEEL DECK PAINTED FOR FIREPROOF 16GA .... 3 PSF
+ SUSPENDED LIGHTNING AND AIR DIST. SYSTEMS + 3 PSF

• TOTAL UNIFORM LOADS $96\,PSF \times \overset{TRIB.\ AREA}{319\,FT^2} = \underline{30624\ lbs}$

+ WALL LOADS
$672\ lbs/ft \times 31.05' =$ .... = $\underline{20866\ lbs}$

+ 2 W 10 x 22 (BEAMS)
$2 \times (22\ lb/ft \times 10.25') =$ .... = $\underline{451\ lbs}$

+ 2 W 21 x 68 (BEAMS)
$68\ lb/ft \times 17.8' =$ .... = $\underline{1210\ lbs}$
$68\ lb/ft \times 13.25' =$ .... = + $\underline{901\ lbs}$
54,052 lbs

TOTAL FLOOR LOADS = 54,052 lbs $\approx \boxed{54.1\ K}$

• TOTAL LOAD ACTING
ON INTERMEDIATE COLUMNS (@ BASEMENT)

= ROOF LOAD + 3FLOOR LOAD + SELF WEIGHT OF COLUMNS (ESTIMATED ≈ 100 lb/ft)

→ HEIGHT OF THE COLUMN

$= 30.5 + 3(54.1) + (100\ lb/ft \times 33\ ft)$

$= 30.5k + 162.3k + 3.3k$

$= 196.6 \Longrightarrow \boxed{P_U = 197\ \text{kips}}$

LENGTH OF COLUMN

DESIGN FOR LONGEST COLUMN = 10.5 FT

a. CHECK Y-Y AXIS

ASSUME COLUMN PIN CONNECTED AT TOP AND BOTTOM WITH SIDESWAY INHIBITED FROM TABLE C-C2.1 IN THE COMMENTARY (AISC) FOR CONDITION (d) K=1.0:

• EFFECTIVE LENGTH $(KL)_y$ = 10.5 FT

• SINCE ALL SECTIONS OF A36 STEEL SATISFY LRFD TABLE 3-[illegible] SELECT A SECTION THAT SATISFIES THE DESIGN REQUIREMENTS FOR THE Y-AXIS.

$\therefore \phi P_n > (P_u = 197k)$ FROM PG. 2-28 AISC

FOR $(KL)_y = 10 \Rightarrow$ W8x31 $= \phi P_n = 232k$
FOR $(KL)_y = 11 \Rightarrow$ W8x31 $- \phi P_n = 223k$

$\therefore$ FOR $(KL)_y = 10.5 \Rightarrow$ W8x31 $= \phi P_n = 227k$ ✓OK

b. CHECK X-XAXIS

ASSUME SIDESWAY UNINHIBITED AND PIN CONNECTED AT BASE

FROM TABLE C-C2.1 (AISC) FOR CONDITION (f)
$K = 2.0$

• EFFECTIVE LENGTH
$(KL)_x = 2 \times 10.5 = 21.0\ ft$

• FROM PG 2-28 FOR A W8x31
$r_x / r_y = 1.72$

CORRESPONDING EFFECTIVE LENGTH RELATIVE TO THE Y-Y AXIS:
$\frac{21.0}{1.74} = 12.0\ ft > 10.5\ ft$

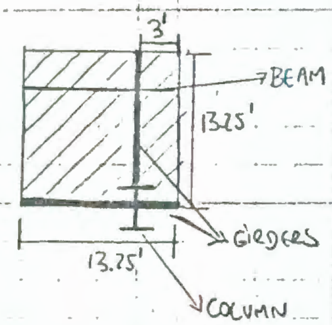
$\therefore$ EFFECTIVE LENGTH FOR X-XAXIS IS CRITICAL

ENTER COLUMN TABLE WITH AN EFF LENGTH OF 12 ft

THE $\boxed{\text{W8x31}}$ @ 12 ft $= \phi P_n = 214k > 196k$ ✓OK

③ E-W1 EDGE COLUMNS DESIGN (2, 3, 14, 15) (REFER TO SKETCH Pg. 21 OF THIS APPENDIX)

Ex: COLUMN 2



• TRIBUTARY AREA

$13.25' \times 13.25' = 175.6\ FT^2$

$\approx 176\ FT^2$

• ROOF LOADS (FROM Pg. 22 ON THIS APPENDIX)

+LIVE & DEAD UNIFORM LOADS → $90.3 \frac{lbs}{FT^2} \times 176 FT^2 =$ 15893 lbs

+ 1 W 10x22 (BEAM)
$22\ lb/ft^2 \times 13.25\ ft$ = 292 lbs

+ 2 W 18x40 (GIRDERS)
$2(40\ lb/ft^2 \times 13.25 ft)$ .... = + 1060 lbs

TOTAL ROOF LOADS = 17245 lbs → $\boxed{17.3k}$

• FLOOR LOADS

• LIVE + 40 PSF $(10.25' \times 13.25')$ + 100 (CORRIDORS) $(3' \times 13.25')$.... = 9409 lbs

• DEAD + $4\frac{1}{2}$" SLAB (REINFORCED)
$150\ lb/ft^2 \times \frac{4.5"}{12}$ ...... 56 PSF

+ STEEL DECK PAINTED
FOR FIREPROOF 16GA...... 3 PSF

+ SUSPENDED LIGHTNING
AND AIR DIST. SYSTEMS. + 3 PSF

• TOTAL UNIFORM LOADS 62 SF × 176 ft² (TRIB. AREA) = 10912 lbs

+ WALL LOADS
$2(672\ lbs/ft \times 13.25\ FT)$ .... = 17808 lbs

+ 1 W 10x22 (BEAMS)
$22\ lb/ft \times 13.25\ ft$ .... = 292 lbs

+ 2 W 21x68 (BEAMS)
$2(68\ lb/ft \times 13.25\ ft)$ .... = + 1802 lbs

40222 lbs

TOTAL FLOOR LOADS = 40222 lbs

$\approx \boxed{40.2K}$

• TOTAL LOAD ACTING ON CORNER COLUMNS (@ BASEMENT)

= ROOF LOAD + 3 FLOORS LOAD + SELF-WEIGHT OF COLUMNS (ESTIMATED 100 lb/ft)

HEIGHT OF ALL COLUMNS

= 17.2 + 3(42.2) + (100 lb/ft × 33 ft)

= 17.2 k + 120.6 k + 3.3 k

= 141.7 k => $\boxed{P_u = 142\ k}$

• LENGTH OF COLUMNS

$K = 1.0$

EFFECTIVE LENGTH

• DESIGN FOR BASEMENT COLUMN = 10.5 FT $\Rightarrow (KL)_y = 10.5$ ft

a. CHECK Y-Y AXIS

• SINCE ALL SECTIONS OF A 36 STEEL SATISFY LRFD TABLE B5.1 SELECT A SECTION THAT SATISFIES THE DESIGN REQUIREMENTS FOR THE Y-AXIS.

∴ ($\phi P_n > P_u = 142$ k) FROM Pg. 2-29

For $(KL)_y = 10 \Rightarrow W8\times24 = \phi P_n = 162$ k

$(KL)_y = 11 \Rightarrow W8\times24 = \phi P_n = 152$ k

∴ For $(KL)_y = 10.5 \Rightarrow W8\times24 = \phi P_n = 157\text{k} > 142\text{k}$ ✓OK

b. CHECK X-X AXIS

ASSUME COLUMNS UNBRACED AND PIN CONNECTED AT BASE

FROM TABLE (C-C2.1 AISC) FOR CONDITION (f)

$K = 2.0$

• EFFECTIVE LENGTH

$(KL)_x = 2 \times 10.5 = 21$ ft

• FROM AISC 2-29, FOR A W8×24

$r_x/r_y = 2.12$

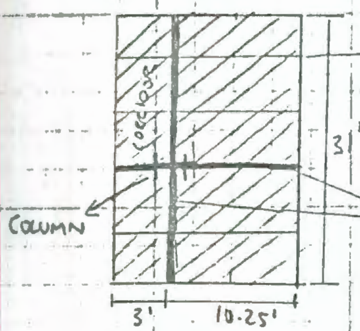
CORRESPONDING EFFECTIVE LENGTH RELATIVE TO THE Y-Y AXIS:

$\frac{21.0}{2.12} = 9.9 \text{ ft} \leq 10.5 \text{ ft}$ ✓OK. ∴ EFFECTIVE LENGTH FOR Y-Y AXIS IS CRITICAL.

THE $\boxed{W8\times24}$ ✓OK

④ INTERIOR COLUMNS DESIGN (6,7,10,11)

Ex 6



• TRIBUTARY AREA

$(13.25' \times 31') = 410.75\ FT^2$
$\approx 411\ FT^2$

• ROOF LOADS (FROM PG.22 ON THIS APPENDIX)

+ LIVE LOAD & DEAD LOAD
(UNIFORM LOADS) $\rightarrow 90.3\ \frac{lbs}{ft^2} \times \overset{TRIB. AREA}{411 ft^2} =$ 37113 lbs

+ 3W 10x22 (BEAMS)
$3\times(22 lb/ft \times 13.25ft)$ .... = 875 lbs

+ 2W 18x40 (GIRDERS)
$(40 lb/ft \times 13.25')$ .... = 530 lbs
$(40 lb/ft \times 31')$ .... = + 1240 lbs

TOTAL ROOF LOADS = 39758 lbs $\Rightarrow$ $\boxed{39.8K}$

• FLOOR LOADS

·LIVE $40\ PSF(10.25' \times 31') + \overset{CORRIDOR}{100\ PSF}(3' \times 31')$ .... = 22010 lbs

·DEAD + 4½" SLAB (REINFORCED)
$150\ PSF \times \frac{4.5''}{12} =$ .... 56 PSF

+ STEEL DECK .... 3 PSF
+ SUSPENDED CEILING
& AIR DIST. SYSTEMS .. + 3 PSF

·TOTAL UNIFORM DEAD LOADS $62\ PSF \times \overset{TRIB. AREA}{411 ft^2}$ ...... = 25482 lbs

+ WALL LOADS
$672 lb/ft \times 13.25'$ .... = 8904 lbs
$672 lb/ft \times 31'$ .... = 20832 lbs

+ 3W 10x22 BEAMS
$3(22 lb/ft \times 13.25')$ .... = 875 lbs

+ 2W 21x68 BEAMS
$(68 lb/ft \times 13.25')$ .... = 901 lbs
$(68 lb/ft \times 31')$ .... = + 2108 lbs
81112 lbs

TOTAL FLOOR LOADS = 81,112 lbs
$\approx \boxed{81.1k}$

• TOTAL LOAD ACTING ON INTERIOR COLUMNS (@ BASEMENT)

= ROOF LOAD + [illegible] FLOOR LOADS + SELF WEIGHT OF COLUMNS (ESTIMATED [illegible])

= 33.3k + [illegible] (31.1) + ([illegible]) → HEIGHT OF COLUMNS

= [illegible].3k + [illegible]k + 3.3k

= 236.3k ⇒ P_u = $\boxed{237k}$

• [illegible]

K=1.0

• [illegible] = 12.5 FT = KL_y = 12.5 ft (EFFECTIVE LENGTH)

1. CHECK Y-Y AXIS

• SINCE [illegible] STEEL SATISFY LRFD TABLE B.5.1 [illegible] SATISFIES THE DESIGN REQUIREMENTS FOR [illegible]

[illegible] 237k) FROM Pg. 2-23

KL_y = [illegible] ⇒ W8 × 40 = ϕP_n = 233k

KL_y = [illegible] ⇒ W8 × [illegible] = ϕP_n = 237k

KL_y = 12.5 ⇒ W8×40 = ϕP_n = 293k ✓ OK

b. CHECK X-X AXIS

ASSUME [illegible] PIN CONNECTED AT BASE

FROM TABLE ([illegible] CONSTRUCTION [illegible])

K = [illegible]

• EFFECTIVE LENGTH [illegible] = 2 × 12.5 = 25 ft

• FROM AISC 2-[illegible] : W8×40

[illegible] = 1.73

CORR[illegible] LENGTH RELATIVE TO THE Y-Y AXIS

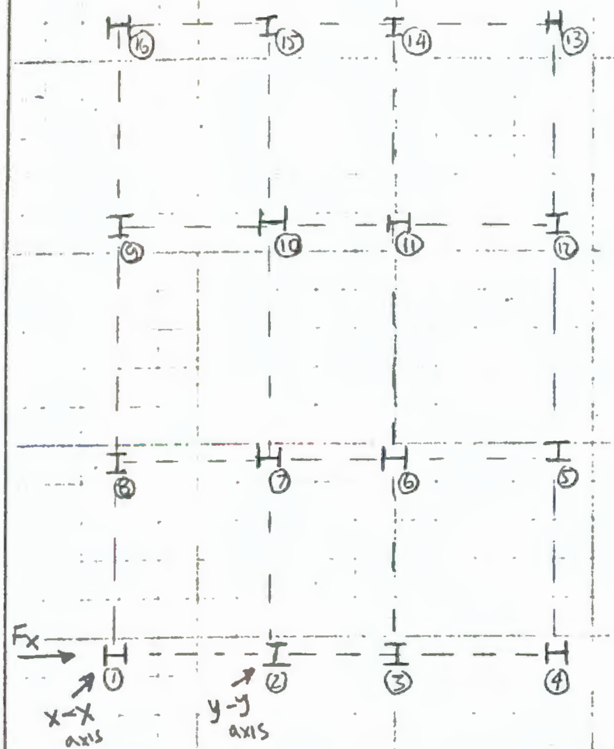
[illegible] EFFECTIVE LENGTH FOR X-Y AXIS [illegible] CRITICAL

W8×40 @ [illegible] = ϕP_n = 275 < 287 NOT OK

GO TO NEXT W SECTION

@ 12 ft = W10 × 45 = ϕP_n = 311 > 237 OK

∴ USE A $\boxed{W10 \times 45}$



DESIGN SUMMARY

o CORNER COLUMNS (1,4,13,16)

TO SUPPORT:	QTY.	SECTION		LENGTH
ROOF	4	W8x24	≃	10.5'
3rd & 2nd FLOORS	8	W8x24	≃	10.0'
1st FLOOR	4	W8x24	≃	7.5'

o N-S EDGE COLUMNS (8,9,5,12)

TO SUPPORT:	QTY.	SECTION		LENGTH
ROOF	4	W8x31	≃	10.5'
3rd & 2nd FLOORS	8	W8x31	=	10.0'
1st FLOOR	4	W8x31	=	7.5'

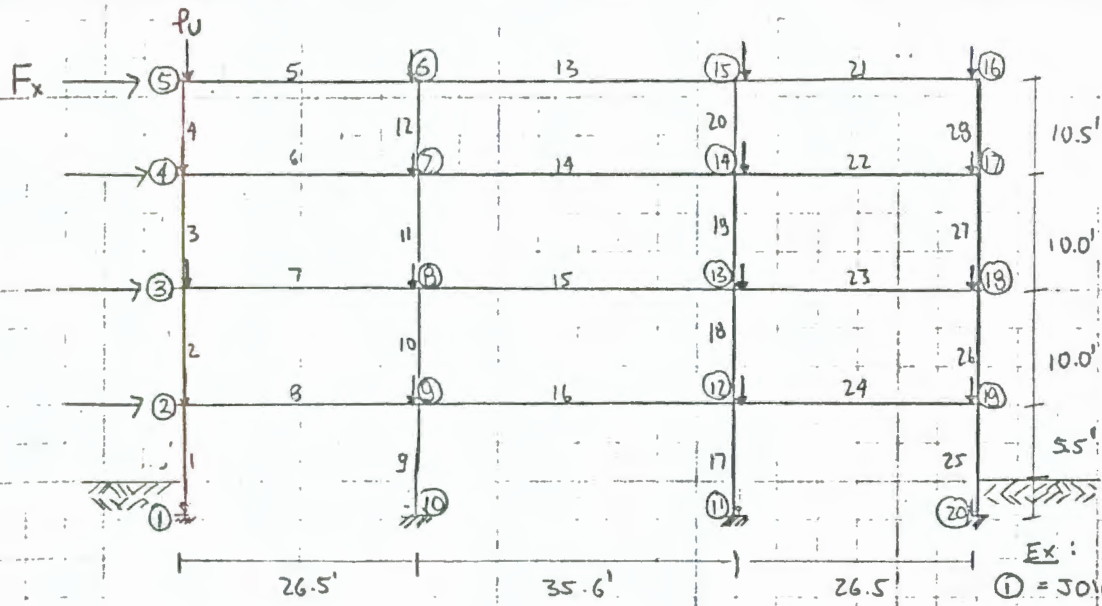
o E-W EDGE COLUMNS (2,3,14,15)

TO SUPPORT:	QTY.	SECTION		LENGTH
ROOF	4	W8x24	≃	10.5'
3rd & 2nd FLOORS	8	W8x24	=	10.0'
1st FLOOR	4	W8x24	=	7.5'

o INTERIOR COLUMNS (6,7,10,11)

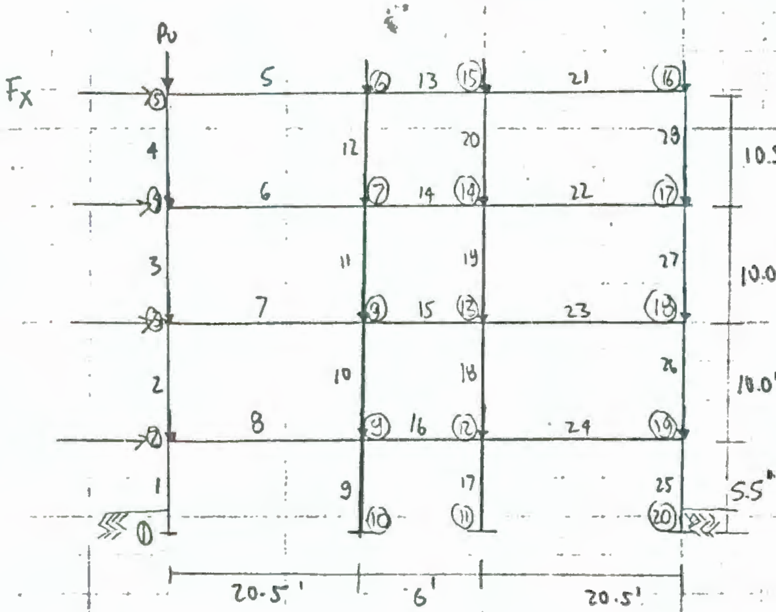
TO SUPPORT:	QTY.	SECTION		LENGTH
ROOF	4	W10x45	=	10.5'
3rd & 2nd FLOORS	8	W10x45	=	10.0'
1st FLOOR	4	W10x45	=	7.5'

GRAVITY & LATERAL CONCENTRATED LOADS ACTING ON PERIMETER FRAMES



N-S

Ex:
① = JOINT REF. #
1 = MEMBER REF #
↓ = CONCENTRATED LOAD (Pu) OBTAINED FROM COLUMN DESIGN
→ = CONCENTRATED LOAD (Fx) OBTAINED FROM WIND OR EARTHQUAKE ANALYSIS

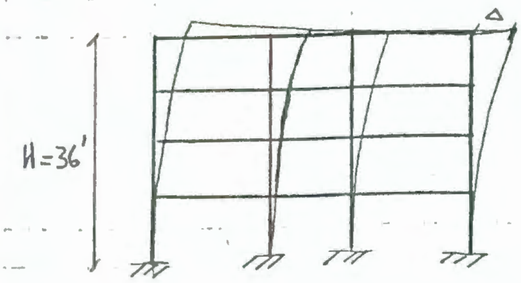


E-W

FOR THE COMPUTER ANALYSIS WE ENTERED THE VALUES OF GRAVITY & LATERAL LOADS AT EACH JOINT.

THE DESIGN OF THE FRAME'S COLUMNS & GIRDERS IS

OK IF $\Delta = \leq \left(\frac{H}{400} - \frac{H}{500}\right)$



$\Delta_{MAX\ USED} = \frac{H}{500} = \frac{36' \times 12''}{500} = \underline{0.864''}$

· AFTER RUNNING THE COMPUTER ANALYSIS ON BOTH THE N-S AND E-W DIRECTIONS, FOR BOTH WIND & EARTHQUAKE, (SEE APPENDIX D), WE FOUND THAT EARTHQUAKE GOVERNED

WITH MAX DEFLECTIONS OF:

Δ N-S = 1.397"
Δ E-W = 1.867"
BOTH WERE GREATER THAN Δ ALLOW $= \frac{H}{500} = 0.864''$

· THEREFORE WE CHECKED CALCULATIONS AND SAW THAT THE PROBLEM WERE THE W8x24 COLUMNS, SINCE THEY WERE ONLY DESIGNED FOR D.L. + L.L. BUT NOT FOR M_z.
WE INCREASED THE Ix (MOMENT OF INERTIA) OF THIS COLUMNS TO A W10x45, THE DESIGN VALUE FOR THE INTERIOR COLUMNS.

DOING THIS, WE DID NOT ONLY SOLVE THE Δ PROBLEM, BUT ALSO, STANDARDIZE ALL COLUMNS TO A W10x45.

AFTER A COMPUTER ANALYSIS WE FOUND THAT EARTHQUAKE MAX DEFLECTIONS:

Δ N-S = 0.783"
Δ E-W = 0.725"
BOTH WERE LESS THAN Δ ALL = 0.864"

$\therefore$ DESIGN CHECKS ✓

INTERACTION EQUATION FOR COLUMNS AFTER COMPUTER ANALYSIS

W10X45 COLUMNS N-S (@ BASEMENT COLUMNS)

FROM THE DESIGN CALCULATIONS AND THE COMPUTER ANALYSIS WE OBTAINED THE FOLLOWING

- LARGEST P_u = 237 K (FROM INTERIOR COLUMNS)
- LARGEST MOMENT = 393 in-k (FROM FINAL COMPUTER ANALYSIS)

W10X45 COLUMNS E-W (@ BASEMENT COLUMNS)

- LARGEST P_u = 237 K (FROM INTERIOR COLUMNS)
- LARGEST MOMENT = 393 in-k (FROM FINAL COMPUTER ANALYSIS)

USE INTERACTION EQUATION FOR THE WORST CASE:

USING $\boxed{\begin{array}{l} P_u = 237\text{ K (FROM INTERIOR COLUMNS)} \\ M = 393\text{ in-k (FROM FINAL COMPUTER ANALYSIS)} \end{array}}$

- FROM LRFD MANUAL CHAPTER H MEMBERS UNDER TORSION AND COMBINED FORCES.

H1. FOR SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE.

$$\frac{P_u}{\phi P_n} = \frac{237}{0.9\,(F_y\,A_g)} = \frac{237\text{K}}{0.9\,(36\text{ksi} \times 13.3\text{ in}^2)} = \underline{0.67}$$

FOR W10X45

for $\dfrac{P_u}{\phi P_n} \geq 0.2$ $\qquad \dfrac{P_u}{\phi P_n} + \dfrac{8}{9}\left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$

$0.67 \geq 0.2$

where M_u = required flexural strength
M_n = nominal flexural strength

$$M_u = B_1 M_{nt} + B_2 M_{lt}$$

where M_{nt} = required flexural strength in MEMBER, kip-in. (NO TRANSLATION)
M_{lt} = required flexural strength from translation (COMPUTER ANALYSIS)

$B_1 = \dfrac{C_m}{(1 - P_u/P_e)} \geq 1$, $C_m = 0.85$ for MEMBERS WHOSE ENDS ARE RESTRAINED

$P_e = A_g F_y / \lambda_c^2$

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u}{\Sigma P_e}}$$

$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}}$;

k = effective length factor
E = modulus of elasticity, ksi
l = unbraced length of members, in.
Fy = specified yield stress, ksi
r = governing radius of gyration about plane of buckling, in.

• $M_{ux} = B_1 M_{nt} + B_2 M_{lt}$ (x-x axis)

1. $\lambda_c = \frac{kl}{r_x\pi}\sqrt{\frac{F_y}{E}} = \frac{75 \times 12''}{(4.32)(\pi)}\sqrt{\frac{36}{29000}} = 0.2336$ (BASEMENT COLUMN)

2. $P_e = A_g F_y/\lambda_c^2 = 13.3\ in \times 36 ksi / 0.2336^2 = 8744$ K (BASEMENT COLUMN)

3. $B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1 = \frac{0.85}{(1 - 237k/8744)} = 0.88 \approx 1.0$

4. $B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_e}}$; $\Sigma P_u = 237 + 206 + 125 + 40 = 658K$

$P_e = A_g F_y / \lambda_c^2$ (FOR 3 FLOORS)

$\lambda_c^2 = \frac{kl}{r_x\pi}\sqrt{\frac{F_y}{E}}$

$\lambda_c^2 = \frac{10' \times 12''}{(4.32)\pi}\sqrt{\frac{36}{29000}} = 0.3115$

$P_e = A_g F_y / \lambda_c^2$

$= 13.3 \times 36 / 0.3115^2 = 4934$ K

$\Sigma P_e = 8744 + 3(4934) = 23546$ K

$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_e}} = \frac{1}{1 - \frac{658}{23546}} \approx 1.03$

FROM COMPUTER ANALYSIS (Pg. 27 APPENDIX F)

MAX M_{nt} @ BASEMENT LEVEL = 43 in-k (NO LATERAL LOADS)
MAX M_{lt} @ BASEMENT LEVEL = 393 in-k (w/ " ")

$M_{ux} = B_1 M_{nt} + B_2 M_{lt} = 1.0(43.0\ in\text{-}k) + 1.03(393\ in\text{-}k) = 448\ in\text{-}k$

$\boxed{M_{ux} = 448\ in\text{-}k}$

• $M_{uy} = B_1 M_{nt} + B_2 M_{lt}$

1. $\lambda_c = \frac{kl}{r_y\pi}\sqrt{\frac{F_y}{E}} = \frac{75 \times 12''}{2.01\pi}\sqrt{\frac{36}{29000}} = 0.502$ (BASEMENT COLUMN)

2. $P_e = A_g F_y / \lambda_c^2 = 13.3 \times 36 ksi / 0.502^2 = 1900$ k (BASEMENT)

3. $B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1 = \frac{0.85}{(1 - 237K/1900K)} \approx 1.0$

4. $B_2 = \dfrac{1}{1-\Sigma \frac{P_u}{P_e}} =$ $\quad \Sigma P_u = 287 + 206 + 125 + 40 = \underline{658\,k}$

$P_e = A_g\, F_y / \lambda_c^2$ (For 3 Floors)

$$\lambda_c^2 = \frac{Kl}{r_y \pi}\sqrt{\frac{F_y}{E}} = \frac{10' \times 12''}{2.01\,\pi}\sqrt{\frac{36}{29000}} = 0.6696$$

$P_e = 13.3 \times 36 / 0.6696^2 = 1068\,k$

$\Sigma P_e = 1900\,k + (1068 \times 3) = 5104\,k$

$$B_2 = \frac{1}{1-\frac{\Sigma P_u}{\Sigma P_e}} = \frac{1}{1-\frac{658}{5104}} = 1.15$$

Max M_{nt} @ basement level = 43 in-k (no lateral loads)

Max M_{lt} @ basement level = 393 in-k (w/ lateral loads)

$M_{uy} = B_1 M_{nt} + B_2 M_{lt} = 1.0(43.0\,\text{ft-k}) + 1.15(393\,\text{ft-k}) = 495$ in-k

$\boxed{M_{uy} = 495\ \text{in-k}}$

- $M_n = C_b \left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$

$C_b = 1.0$ since basement column only be treated as an unbraced cantilever due to the foundation

M_1 = Smaller moment from computer analysis = 206 in-k

M_2 = Largest moment from computer analysis = 393 in-k

L_p = (For I shapes) $= \dfrac{300\, r_y}{\sqrt{F_{yf}}} = \dfrac{300(2.01)}{\sqrt{36}} = \underline{101\ \text{in}} = \underline{8.4\ \text{ft}}$

- From Sectional Properties For A W 10x45

$L_r = 35.1\ \text{ft}$; L_b = Longest unbraced length = 10.5'

$M_{rx} = (F_{yw} - F_r) S_x = (36 - 16.5)\, 49.1 = \underline{958\ \text{in-k}}$; $M_{ry} = (36 - 16.5)\,13.3 = \underline{260\ \text{in-k}}$

M_{px} = plastic moment $= F_y Z_x = 36 \times (54.9) = \underline{1977\ \text{in-k}}$

M_{py} = plastic moment $= F_y Z_y = 36 \times (20.3) = \underline{731\ \text{in-k}}$

- $M_{nx} = 1.0\left[M_{px} - (M_{px} - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_{px}$

$= 1.0\left[1977 - (1977 - 958)\left(\frac{10.5 - 8.4}{35.1 - 8.4}\right)\right] \le M_{px} = 1977\ \text{in-k}$

$\boxed{M_{nx} = 1897\ \text{in-k}} \le M_{px} \le 1977\ \text{in-k}$

- $M_{ny} = 1.0\left[M_{py} - (M_{py} - M_{ry})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_{py}$

$= 1.0\left[731 - (731 - 260)\left(\frac{10.5 - 8.4}{35.1 - 8.4}\right)\right] \le 731\ \text{in-k} \rightarrow \boxed{M_{ny} = 694\ \text{in-k}}$

- Interaction Equation

$\dfrac{P_u}{\phi P_n} = \dfrac{287}{0.9(36 \times 13.3)} = 0.67 \ge 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

$$= 0.67 + \frac{8}{9}\left(\frac{448\ \text{in-k}}{0.9(1897)} + \frac{495\ \text{in-k}}{0.9(694)}\right) \le 1.0$$

$$= 0.67 + \frac{8}{9}(0.262 + 0.79)$$

W10x45

FROM OUR PREVIOUS CHECK FOR THE "INTERACTION EQUATION", THE W10 x 45 WAS NOT ACCEPTABLE.

ONE CAN NOTICE THAT THE MAJOR PROBLEM WAS IN THE $\frac{\phi M_{uy}}{\phi b M_{ny}}$ SECTION OF THE FORMULA WHERE WE GOT A 0.78 RATIO.

THEREFORE WE WANT A W-SECTION THAT HAS ABOUT TWICE THE Z_y OF THE W10 x 45 = 20.3 in^3, SO THAT WE GET A LOWER RATIO THAN 0.78.

LOOKING AT THE COLUMN TABLES IN THE LRFD WE SEE THAT A W10 x 68 HAS A $Z_y = 40.1\ in^3$ about TWICE AS W10x45 = 20.3 in^3.

W10x68

$A = 20.0\,in^2$ $S_x = 75.7\ in^3$

$Z_y = 40.1\,in^3$ $S_y = 26.4\ in^3$

$Z_x = 85.3\,in^3$ $L_p = 10.8'$

$L_r = 53.7'$

SINCE $L_b = 10.5 - L_p = 10.8$, IS ALMOST 0, ASSUME

$M_{nx} = M_{px}$; WHICH ACTUALLY WOULD BE SAFER

$M_{ny} = M_{py}$

$M_{px} = f_y \times Z_x = 36(85.3) = 3071$ in-k

$M_{py} = f_y \times Z_y = 36(40.1) = 1444$ in-k

FROM PREVIOUS CALCULATIONS & FROM THE COMPUTER ANALYSIS:

$M_{ux} = B_1 M_{nt} + B_2 M_{lt} = 1.0(43) + 1.03(393\ in\text{-}k) = 448\ in\text{-}k$

$M_{uy} = B_1 M_{nt} + B_2 M_{lt} = 1.0(43) + 1.15(393\ in\text{-}k) = 495\ in\text{-}k$

$P_u = 287\,K$ (INNER COLUMNS)

INTERACTION EQUATION

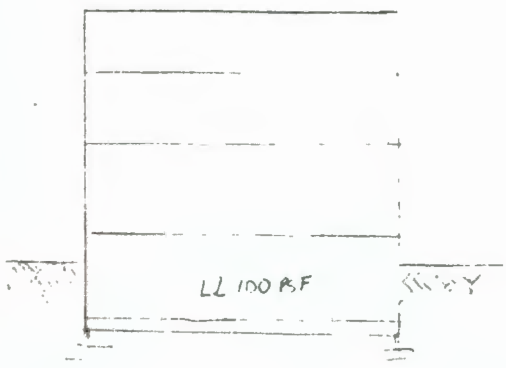
$$\text{if } \frac{P_u}{\phi P_n} \geq 0.2 \qquad \frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi b M_{nx}} + \frac{\phi M_{uy}}{\phi b M_{ny}}\right) \leq 1.0$$

$$\frac{P_u}{\phi P_n} = \frac{287}{\underset{\substack{F_y \times A_g \\ W10\times68}}{0.9(36\times20.0)}} = 0.44 + \frac{8}{9}\left(\frac{448}{0.9(3071)} + \frac{495}{0.9(1444)}\right)$$

$$= 0.44 + \frac{8}{9}(0.16 + 0.38)$$

$$= \underline{\underline{0.92 \leq 1.0}}$$

$\therefore$ W10 x 68 SATISFIES THE REQUIRMENTS.

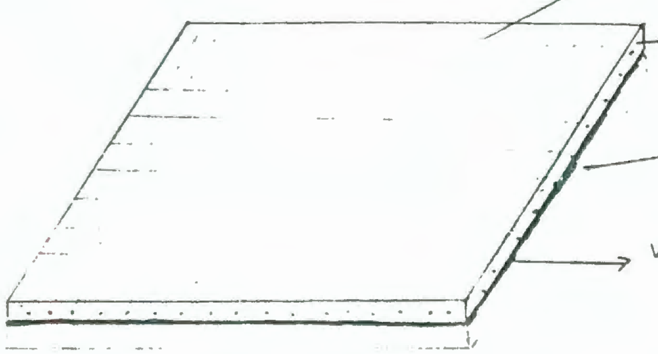


<u>NOTE</u> THE DESIGN OF THE BASEMENT SLAB WAS SHOWN IN THE GENERAL SPECIFICATIONS ON INSTITUTE HALL. THIS WAS DONE IN ACCORDANCE TO THE MASSACHUSETTS STATE CODE (990)

- ACCORDING TO THE GENERAL PROJECT SPECIFICATIONS ON INSTITUTE HALL, THE SLABS ON GROUND SHALL BE PLACED ON MINIMUM 6" LAYER OF 95% COMPACTED GRAVEL AND PLACED IN ALTERNATE PANELS NOT EXCEEDING [illegible] SQUARE FEET. (SEE ARTICLE 6 ON FOLLOWING PAGE)

- ACCORDING TO THE GENERAL PROJECT SPECIFICATIONS ON INSTITUTE HALL, ALL SLABS PLACED ON GROUND SHALL BE REINFORCED WITH MINIMUM 6x6 - W2.9 x W2.9 WELDED WIRE FABRIC LAPPED 12" ON SIDES AND ENDS UNLESS OTHERWISE SHOWN ON THE DRAWINGS

- ACCORDING [illegible] MASSACHUSETTS STATE CODE THE MINIMUM SLAB THICKNESS FOR A SLAB IS 3.5" SECTION 1509 ON MASS CODE (990) SINCE WE ARE DESIGNING A SLAB FOR A COLLEGE DORMITORY, THE LIVE LOAD ACTING ON IT IS 100 PSF (FOR SOLID AREAS). THEREFORE TO BE SAFE WE DESIGNED THE SLAB TO BE 6" THICK, JUST LIKE THE SLAB DESIGNED FOR THE CORRIDOR.
  AN APPROVED VAPOR BARRIER WITH JOINTS LAPPED NOT LESS THAN 6" SHALL BE PLACED BETWEEN THE BASE COURSE OR SUBGRADE AND THE CONCRETE SLAB.



MASS CODE 1980

SECTION 1205.1 FROST PROTECTION

[illegible] (4) [illegible]
[illegible] BEDROCK

SECTION 1206.1 FOOTING DESIGN

[illegible] BEARING MATERIALS [illegible] AND [illegible]

SECTION [illegible] LATERAL LOADS

[illegible] ONE-THIRD (1/3) OF [illegible] BE INTEGRATED BY THE [illegible]

[illegible] SUBSTITUTE [illegible]

[illegible] 1/3 OF LIVE + DEAD LOADS

[illegible]

TYPE OF SOIL IN WORCESTER

ACCORDING TO [illegible] ALLOWABLE BEARING PRESSURE FOR FOUNDATION [illegible]

MATERIAL CLASS	DESCRIPTION	CONSISTENCY	MAX ALLOW. NET BEARING PRESSURE (tons/ft²)
7	[illegible] SAND AND GRAVEL AND GRANULAR ABLATION TILL	DENSE MEDIUM	6

FROM "FOUNDATION ENGINEERING", 1974 edition by PECK, HANSON, & THORNBURN Pg. 265

"AS A GENERAL RULE A FACTOR OF SAFETY OF 3 SHOULD [illegible] AGAINST THE LOADS SPECIFIED BY THE [illegible]" → CONTINUES

"THE FACTOR OF SAFETY SHOULD NOT ORDINARILY BE LESS THAN 2 EVEN IF THE MAXIMUM LOADS ARE KNOWN EXCEPTIONALLY WELL".

NOTE: SINCE WE DON'T KNOW THE SOIL TYPES EXCEPTIONALLY WELL, WE USE A FACTOR OF SAFETY OF 3. FOR Pu'S.

- FROM "FOUNDATION ENGINEERING", 1974 EDITION by PECK, HANSON & THORNBURN PG. 114 TABLE 5.3

TYPE	RELATIVE DENSITY	# BLOWS (PENETRATION TEST) Per ft, N
FOR SANDS/GRAVEL	DENSE	30-50

- FOR AVERAGE # OF BLOWS USE N = 40.

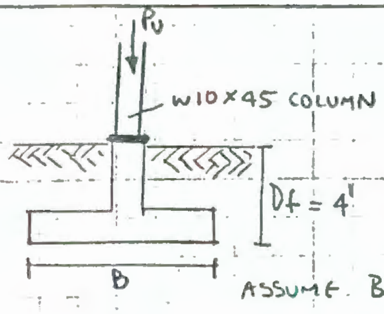
- TO USE PECK'S, HANSON'S, & THORNBURN DESIGN METHOD FOR FOOTINGS WE USED TERZAGHI'S FIGURE 19.3 PG. 309 ON "FOUNDATION ENGINEERING". THIS FIGURE IS A CHART FOR PROPORTIONING SHALLOW FOOTINGS. IT IS WAS DESIGNED TAKING A MAXIMUM VALUE SETTLEMENT OF 1".

DESIGN OF FOOTINGS

ALL FOOTINGS MUST BE:

- 4' BELOW FINISHED GRADE (D_f) (MASSACHUSETTS)
- THICKNESS ≥ 8"
- DL + LL × 3 SAFETY FACTOR
- AVERAGE N OF BLOWS = N = 40
- MAX. DIFFERENTIAL SETTLEMENT = 1"

1. FOOTING FOR CORNER COLUMNS



ASSUME B = 6'

$D_f/B = \frac{4'}{6'} = 0.67$

Pu FROM CORNER COLUMNS ARE DL + LL OF ALL STORIES

$P_U = 110K \times 3SF = 330K$

FROM Fig. 19.3 (Pg 309 ON PECK, HANSON & THORNBURN) FOR N = 40 ; $D_f/B = 0.67$

$q_{all} \approx 4.4\ TSF$

$q_{actual} = \frac{P_u}{A} = \frac{330}{6^2} = 9.17\ KSF$ (A = AREA)

1 TSF = 2 KSF

9.17/2 = 4.85 TSF

4.85 TSF (q_{actual}) > 4.4 TSF (q_{all}) ∴ NOT OK

TRY $B = 6.5'$ FROM FIG. 19.3, Pg. 309 ON PECK

$\frac{Df}{B} = \frac{4'}{6.5'} = 0.62$ FOR $N = 40$; $Df/B = 0.62$

$q_{all} \approx 4.4$ TSF

$q_{ACTUAL} = \frac{P_u}{A} = \frac{330}{6.5^2} = 7.8$ KSF

$= \frac{7.8 \text{ KSF}}{2} = 3.9$ TSF

$q_{ACTUAL} < q_{ALLOWABLE}$

3.9 TSF < 4.4 TSF OK

- FOOTING THICKNESS

IS APPROXIMATELY $B/4 = \frac{6.5'}{4} = 1.6' \rightarrow \approx 19''$

- LOAD OF SLAB $= 1.6' \times 0.15 K/FT^3 / 2 = +0.12$ TSF

WT. CONCRETE

- SURCHARGE LOAD SOIL $= 1.6' \times 0.115 K/FT^3/2 = -0.09$ TSF

WT SOIL

- TOTAL q ACTUAL = +3.9 TSF
+ 0.12 TSF
− 0.09 TSF

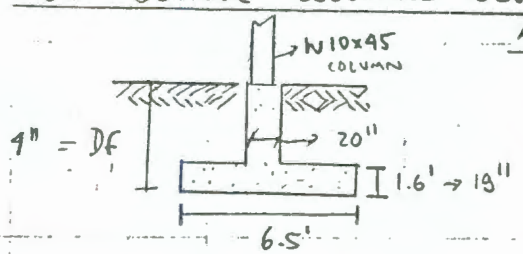
3.93 TSF < 4.4 TSF OK ✓

$q_{ACT} < q_{ALL}$

∴ FOR CORNER COLUMNS USE:

A 6.5' x 6.5'

SQUARE FOOTING



2. FOOTING FOR N-S (INTERMEDIATE COLUMNS)

$P_u = 197K \times 3$ S.F. $= 591K$

Df FOR MASSACHUSSETS = 4'

ASSUME $B = 8'$ ∴ $Df/B = \frac{4'}{8'} = 0.5$

FROM FIG. 19.3 (Pg 309 ON PECK)

FOR $N = 40$; $Df/B = 0.5$

$q_{ALL} = 4.4$ TSF

$q_{ACTUAL} = \frac{P_U}{A} = \frac{591}{B^2} = 9.23\ KSF$

$= \frac{9.23}{2} = 4.62\ TSF > 4.4\ TSF \therefore$ NOT OK
$\quad q_{ACT.} \qquad q_{ALL}$

• TRY $B = 8.5'$

$\frac{D_f}{B} = \frac{4}{8.5} = 0.47$ FROM FIG. 19.3

FOR $N = 40$; $D_f/B = 0.47$

$q_{ALL} \approx 4.4\ TSF$

$q_{ACTUAL} = \frac{P_U}{A} = \frac{591}{8.5^2} = 8.2\ KSF$

$\frac{8.2}{2} = 4.1\ TSF < 4.4\ TSF$ ✓ OK.
$\quad q_{ACT.} \qquad q_{ALL}$

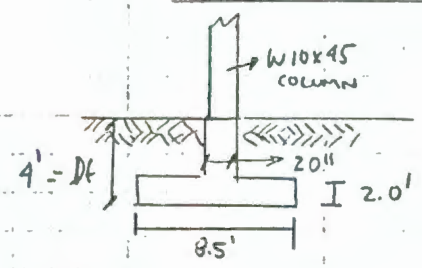
- FOOTING THICKNESS
  is APPROXIMATELY $B/4 = \frac{8.5'}{4} \approx 2'$

• LOAD OF SLAB (CONCRETE) $= 2.0' \times 0.15\ K/FT^3 / 2 = +0.15\ TSF$ 0.26 1.21

• SURCHARGE LOAD SOIL $= 2.0' \times 0.115\ K/FT^3 / 2 = -0.115\ TSF$

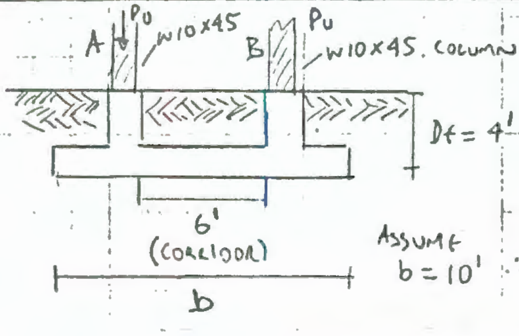
• TOTAL q ACTUAL $= +4.1\ TSF$
$+0.15\ TSF$
$-0.115\ TSF$
$4.14\ TSF < 4.4\ TSF$ OK ✓
$q_{ACT.} < q_{ALL}$

$\therefore$ FOR N-S (INTERMEDIATE COLUMNS) USE:



A $8.5' \times 8.5'$
SQUARE FOOTING

3. FOOTING FOR E-W (CORRIDOR COLUMNS) (COMBINED FOOTING)



$P_U = 142K$

$2P_U = 284K \times 3\ S.F. = 852K$ @ CENTROID OF FOUNDATION. 3 FT FROM A & 3 FT FROM B

FROM FIG 19.3 (Pg. 309 ON DECK)

ASSUME $b = 10'$ $\therefore \frac{D_f}{b} = \frac{4}{10} = 0.4$

FOR $N = 40$; $D_f/B = 0.4$

$q_{ALL} \approx 4.4\ TSF$

$q_{actual} = \frac{852}{10^2} = 8.52\ KSF$

$\frac{8.52\ KSF}{2} = 4.26\ TSF < q_{ALL} = 4.4\ TSF$ ✓OK

• FOOTING THICKNESS IS APPROXIMATELY $b/4 = \frac{10'}{4} = 2.5'$

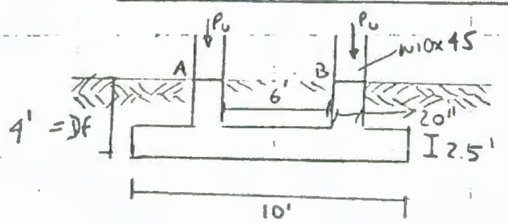
• LOAD OF SLAB = $2.5' \times 0.15\ K/FT^3/2 = +0.19\ TSF$ (CONCRETE)

• SURCHARGE LOAD (SOIL) = $2.5' \times 0.115\ K/FT^3/2 = -0.115\ TSF$

• TOTAL q_{ACTUAL} = +4.26 TSF
+ 0.19 TSF
− 0.115 TSF

4.34 TSF < q_{ALL} = 4.4 TSF ✓OK

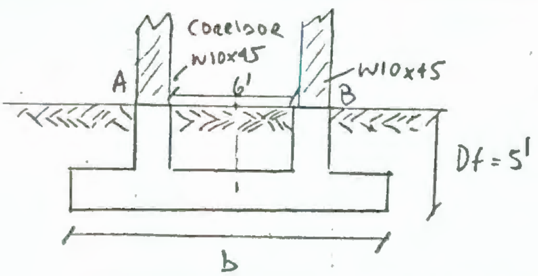
∴ FOR E-W (CORRIDOR COLUMNS)



A 10'x10' SQUARE FOOTING

4. FOOTING FOR INTERIOR COLUMNS (COMBINED FOOTING)

E-W VIEW



$P_U = 287\ k$

$\Sigma P_U = 568k \times 3\ S.F = 1704k$ @ CENTROID OF FOUNDATION (3' from A & 3' FROM B)

ASSUME b = 14'

$\therefore \frac{Df}{b} = \frac{5}{14} = 0.36$

FROM FIG. 19.3 (Pg. 309 ON PECK)

FOR N = 40; $Df/B = 0.36$

$q_{ALL} \approx 4.4\ TSF$

$q_{ACTUAL} = \frac{P_U}{A} = \frac{1704}{14^2} = 8.69\ KSF$

$\frac{8.69}{2} = 4.34\ TSF < q_{ALL} = 4.4\ TSF$ ✓OK

• FOOTING THICKNESS IS APPROXIMATELY $b/4 = \frac{14'}{4} = 3.5'$

• LOAD OF SLAB = $3.5' \times 0.15\ K/FT^3 / 2 = +0.26$

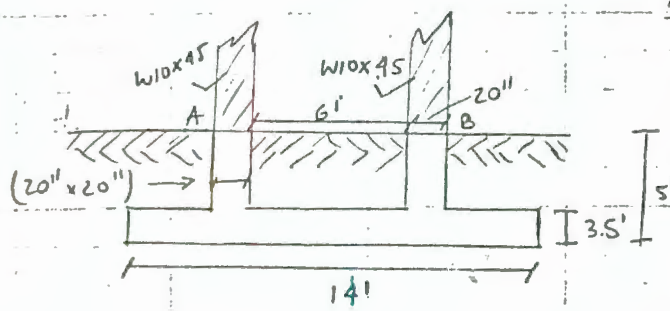
• SURCHARGE LOAD SOIL = $35' \times 0.115\ K/FT^3 / 2 = -0.20$

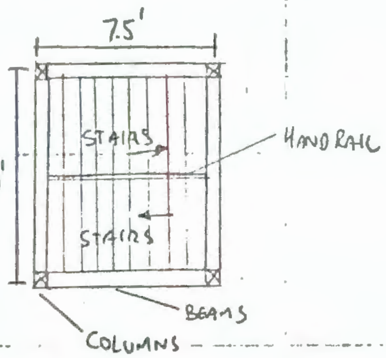
• TOTAL q_{ACTUAL} = +4.34 TSF
+ 0.26 TSF
− 0.20 TSF

4.40 TSF = 4.4 TSF ✓OK.
q_{ALL}

∴ For Interior Columns

A 14' x 14' SQUARE FOOTING



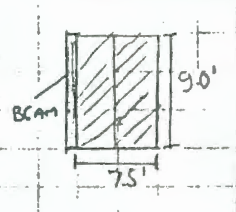


- BEAM DESIGN
  FOR STAIRWAY

LIVE LOAD = 100 PSF (FROM MASS. STATE CODE 1990)

APROX. DEAD LOAD OF STEEL/CONCRETE STAIRS $\simeq$ 70 PSF

- TRIBUTARY AREA (DESIGN FOR LARGEST BEAM, COVERING WHOLE AREA OF STAIRS)



TRIB AREA = $\frac{1}{2}(9' \times 7.5') =$
$\approx$ 67.5 FT^2

- LOAD PER BEAM

- LIVE LOAD x TRIB AREA

$100\,PSF \times 67.5\,FT^2 = 6750\ lbs$

$\frac{6750\,lbs}{9.0\,FT} = 750\ lbs/ft$ ← length of beam

$= \boxed{0.75\ k/ft}$

- DEAD LOAD x TRIB AREA

$70\,PSF \times 67.5\,FT^2 = 4725\ lbs$

$\frac{4725}{9.0} = 525\ lbs/ft$

$= \boxed{0.53\ k/ft}$

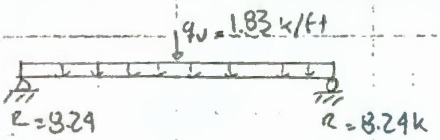
- ESTIMATED SELF WEIGHT (BEAM)

$\approx 50\,lbs/ft \approx .05\ k/ft$

- TOTAL DEAD LOAD = $\boxed{0.58\ k/ft}$

- FROM LRFD A.4.1

$W_u = 1.2\,D + 1.6\,L$
$= 1.2(0.75) + 1.6(0.58)$
$= 1.83\ K/ft$



$R = \frac{1.83 \times 9.0}{2} = 8.24\ K$

$\frac{ql^2}{8} = \frac{(1.83)(9.0)^2}{8}$
$= 18.5\ k\text{-}ft$

$M_u \approx \boxed{19\ k\text{-}ft} \times 2.0\ S.F.$ (for Stairs) $= \boxed{38\ K\text{-}ft}$

USING STRUCTURAL STEEL DESIGN (J.C. SMITH TEXTBOOK) & LRFD 1996)

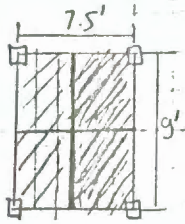
- SPAN = 9.0 ft
- LIMITING DEFLECTION SPAN/360 = $\frac{9.0' \times 12''}{360} = 0.3''$
- SELECT LIGHTEST W SECTION FOR $F_y = 36$ KSI STEEL

$\phi M_x = 0.9\,M_{px} \leq M_{ux} = 38\ ftk$

FROM LRFD 3-17 W12X14 (LIGHTEST W SECTION)

$\phi M_x = 47.0 > 38.0\ ftk$ OK ✓

∴ FOR STAIRWAY'S FRAME ∴ USE $\boxed{W12 \times 14\ BEAMS}$



TRIB AREA ½ (STAIRS AREA) TO BE SAFE

$\frac{1}{2}(9 \times 7.5) = 33.8\ FT^2 \approx \underline{34\ FT^2}$

• ROOF LOADS

+ LIVE & DEAD UNIFORM LOADS → 100 + 70 = 170 PSF

170 PSF × 34 FT² = 5780 lbs

+ 2W 12x14 BEAMS

$14\ \frac{lb}{FT} \times 9.0'$ .... = 126.0 lbs

$14\ lb/FT \times \frac{7.5'}{2}$ .... = 52.5 lbs

ROOF LOADS = 5959 lbs ≈ $\boxed{6.0\ k}$

• FLOOR LOADS

LIVE - 100 PSF × 34 FT² .... 3400 lbs

DEAD (STAIRS) 70 PSF × 34 FT² .... 2380 lbs

• 2W 12x14 BEAMS

$14 \times 9.0'$ .... 126 lbs

$14 \times \frac{7.5}{2}$ .... 52.5 lbs

5959 lbs ≈ $\boxed{6.0\ k}$

• TOTAL LOAD ACTING ON COLUMN (@ BASEMENT)

= ROOF + 3 FLOORS + SELF WEIGHT OF COLUMNS (ESTIMATED 50 lb/ft)

= 6.0K + 3(6.0)k + 3×(1.9k) (50 × 38') ≈ 1.9k

= 29.7

29.7k × 2.0 (SAFETY FACTOR FOR STAIRS) = 59.4k
→ 100%

$\boxed{P_u\ @\ \text{BASEMENT} = 60\ K}$

• LENGTH OF COLUMNS K = 1.0

EFFECTIVE LENGTH

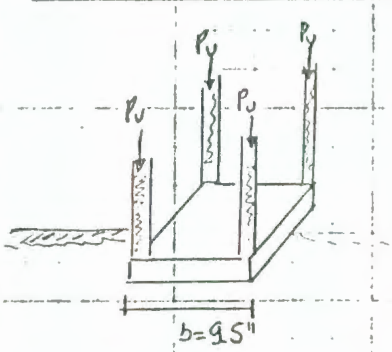
DESIGN FOR LARGEST = 10.5 ft $KL_y = 10.5\ ft$

FROM LRFD P_y 2.30 LIGHTEST W8 SECTION @ 11.0 ft

A W8x24 = 152 K > 60 k ∴ $\boxed{OK}$

USE $\boxed{W\ 8 \times 24}$ AS COLUMNS.
FOR STAIRS

FOUNDATIONS DESIGN FOR STAIRWAY COLUMNS (COMBINED FOOTING)



$P_U = 60K$

$4P_U = 240K \times 3\ S.F = 720K$

FROM FIG. 19.3 ON PECK, THOMSON

For $N = 40 \approx q_{all} = 4.4\,TSF$

$q_{ACTUAL} = \dfrac{720K}{9.5^2} = 8.0\,KSF$

$\dfrac{8.0}{2}KSF = 4.0\,TSF$

$4.0_{TSF} < 4.4\,TSF$

$q_{act} < q_{ALL}$ ✓OK.

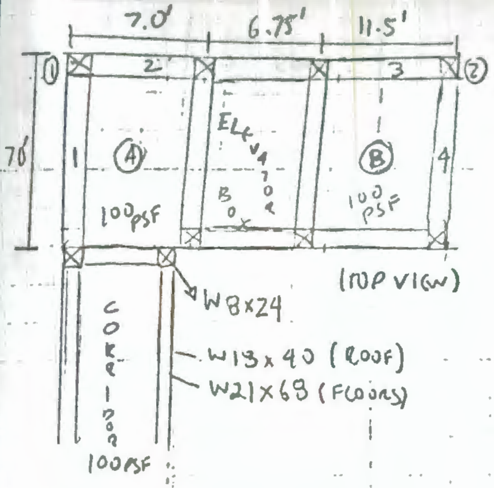
• FOOTING THICKNESS $\approx b/4 = \dfrac{9.5}{4} = 2.5'$

• LOAD OF SLAB $= 2.5' \times 0.15\ K/ft^3/2 = +0.19\,TSF$

• SUCCIACGF LOAD SOIL $= 2.5' \times 0.115\ K/ft^3/2 = -0.14\,TSF$

• TOTAL q ACTUAL $= 4.0$
$+ 0.19$
$- 0.14$
$4.05\,K < q_{ALL} = 4.4$ ✓OK

USE A $9.5' \times 9.5'$
W/ $2.5'$ THICKNESS
AND 6" x 6" - W2.9xW2.9 WELDED WIRE FABRIC



AREAS A & B

Since they are similar areas outside an elevator then

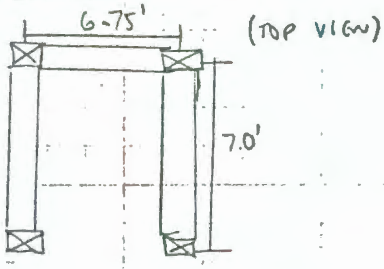
$LL = 100$ PSF.

The corridor beams & girders shown in the diagram were designed for 100 PSF and even a larger trib. area than area (B).

∴ Since the trib areas for area (A) & (B) are smaller than the trib areas for the corridor design, the members designed for the corridor check for shear, bending, and deflection for these areas.

- Steel Deck
  - Use 16 GA steel deck with a 7.0' span and 7.0' width.
- Slabs
  - 3.5" for roof (reinforced w/ 66-44 mesh)
  - 4.5" for floors (reinforced w/ 44-44 mesh)

ELEVATOR SHAFT



According to Mass Code design elevator shaft for a live load of 300 psf with a 100% safety factor

∴ $300 \times 2 = 600$ PSF

- Beam Design
  - Tributary area (design for largest beam)

$= \frac{1}{2}$ (area of elevator shaft)

$= \frac{1}{2}(6.75' \times 7.0')$

$= 23.63$

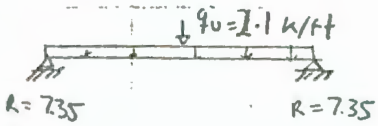
$\approx \underline{24\ ft^2}$

- Load per girder

$L.L = 600\ psf \times 24\ ft^2 = 14400\ lbs.$ D.L from

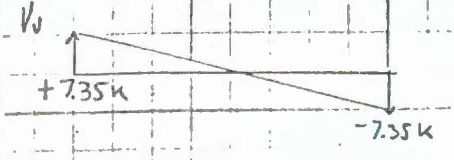
$\frac{14400\ lbs}{7'} = 2057\ lbs/ft$

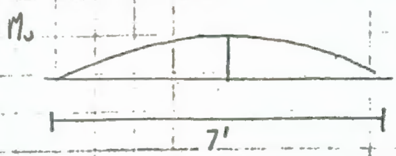
$q_u \approx \underline{2.1\ K/ft}$



$R = \frac{2.1 \times 7}{2} = 7.35\ K$

BEAM DESIGN CONT.





$$M_u = \frac{ql^2}{8} = \frac{(2.1)(7^2)}{8} = 12.9 \text{ k-ft}$$

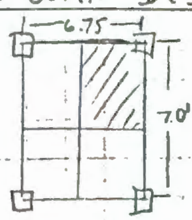
$$M_u = 13 \text{ k-ft}$$

∴ TO SUPPORT ELEVATOR LOADS + LOADS TRANSFERED FROM ELEVATOR SHAFT WALLS, THE PREVIOUSLY DESIGNED GIRDERS

W 18x 40 (ROOF)
W 21 x 68 (FLOORS)

ARE OVER DESIGNED FOR THE ELEVATOR SHAFT TO SUPPORT ALL LOADS INCLUDING WIND & EARTHQUAKE LOADS.

COLUMN DESIGN (ELEVATOR SHAFT)



TRIB AREA $= \frac{1}{4}(6.75 \times 7.0') = 11.8 \approx 12 \text{ ft}^2$

• ROOF LOADS

ELEVATOR LL. FROM pg. 21 (SNOW + ROOF D.L)

+ LIVE + DEAD LOADS = $600 + 90.3 \approx 691$ PSF $\times$ 12 FT^2 = 8292 lbs

+ 2 W18x40 (GIRDERS)

40 lbs/ft x 7.0' = 280 lbs
40 lbs/ft x 6.75' = 270 lbs
550 lbs

TOTAL ROOF LOADS = 8292 + 550 = 8842 lbs

$\approx \boxed{8.9 \text{ kips}}$

• FLOOR LOADS

+ 2 W 21x68 GIRDERS

68 lbs/ft x 7.0' = 476 lbs
68 lbs/ft x 6.75' = 459 lbs
935 lbs

+ ELEVATOR LIVE LOAD

600 PSF x 12 ft^2 = 7200 lbs

+ LOADS FROM WALLS

672 lbs/ft x 7.0' = 4704
672 lbs/ft x 6.75' = 4536
9240 lbs

∴ TOTAL FLOOR LOADS
= 935
+ 7200
+ 9240
17,375 lbs $\approx \boxed{17.4 \text{ kips}}$

• TOTAL LOAD ACTING ON ELEVATOR COLUMNS @ BASEMENT

= ROOF LOAD + 3 FLOORS LOAD + SELF WEIGHT OF COLUMNS
↳ ESTIMATED ≈ 100 lbs/ft

$= 8.9 + 3(17.4) + (100\ lb/f \times 36')$ → HEIGHT OF COLUMNS

$= 8.9 + 52.2 + 3.6\ k$

$= 64.7$ KIPS → $\boxed{P_u = 64.7 \text{ KIPS}}$

• LENGTH OF COLUMNS
LONGEST COLUMN = 10.5 ft

$k = 1.0$

• Effective LENGTH = 10.5 ft

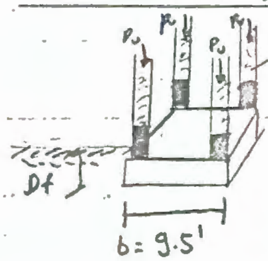
$\phi P_n > P_u = 64.7k$ FROM Pg. 2-29 AISC THE LIGHTEST & SMALLER W8 SECTION IS:

FOR $(kL)_y = 11$ ft → W8x24 $= \phi P_n = 162k$

$162k > 64.7k$ ∴ OK

$\boxed{W\ 8 \times 24}$ OK FOR ELEVATOR SHAFT COLUMNS

FOUNDATIONS DESIGN FOR ELEVATOR COLUMNS (COMBINED FOOTING)



$P_u = 64.7k$ ↗ SAFETY FACTOR FOR FOUNDATIONS

$4P_u = 259\ k \times 3.S.F = 777k$

FROM Fig 19.3 ON PECK, THOMSON
FOR $N = 40 \approx q_{all} \approx$ 4.4 TSF

• ASSUME $b = 9.5'$

$q_{ACTUAL} = \frac{777}{9.5^2} = 8.6$ KSF

$\frac{8.6\ KSF}{2} = 4.3$ TSF

$4.3 < 4.4$ TSF

$q_{act} < q_{all}$ ✓ OK.

• FOOTING THICKNESS $\approx b/4 = \frac{9.5}{4} \approx 2.5'$

• LOAD OF SLAB $= 2.5' \times 0.15\ k/ft^3/2 = +0.19$ TSF

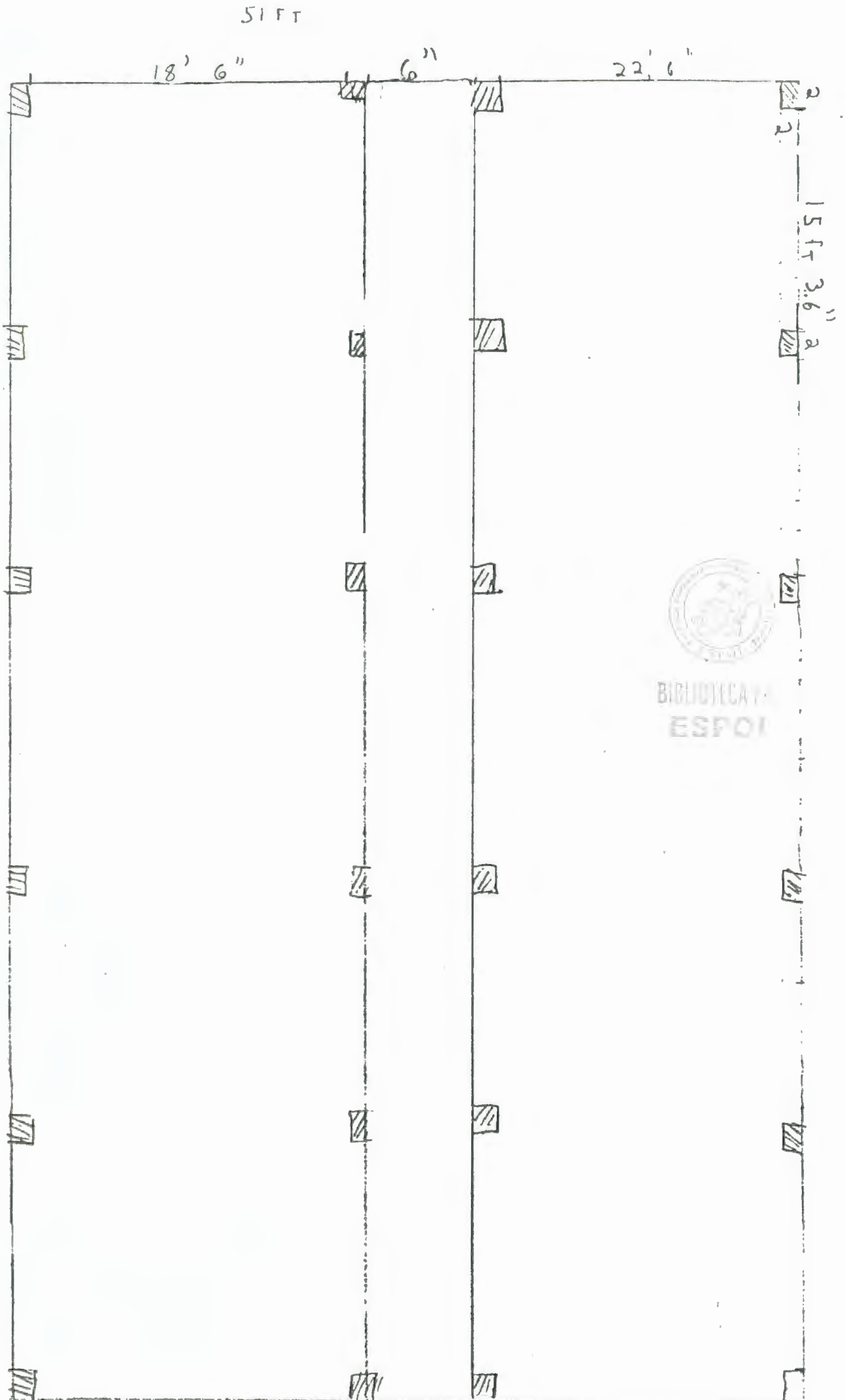
• SURCHARGE LOAD SOIL $= 2.5' \times 0.115\ k/ft^3/2 = -0.14$ TSF

• TOTAL q ACTUAL = 4.3
+0.19
−0.14
$= 4.35\ k < q_{all} = 4.4$ ✓ OK

USE A 9.5' x 9.5'
w/ 2.5' THICKNESS
AND 6" x 6" - W2.9 x W2.9 WELDED WIRE FABRIC

Appendix E

Reinforced Concrete Design Calculations


51 FT
18' 6"
6"
22' 6"
2
15 FT 3.6"
2

1) Select thickness.

a) Determine thickness to limit deflection: Panel 1-2

$\max l_n = 18\,ft\ 6\,in = 222\,in$

$\min h = \dfrac{l_n}{33} = \dfrac{222\,in}{33} = 6.73 \approx 6.75\,in$

b) Check Thickness for shear.

1) Calculate Dead load

Roof deadload:

3 ply felt and gravel roofing	$5.5\ lb/ft^2$
Fiber glass batts insulation	$.5\ lb/ft^2$
suspended steel channels	$1\ lb/ft^2$
Suspended lighting	$3\ lb/ft^2$
	$10\ lb/ft^2$

Slab dead load

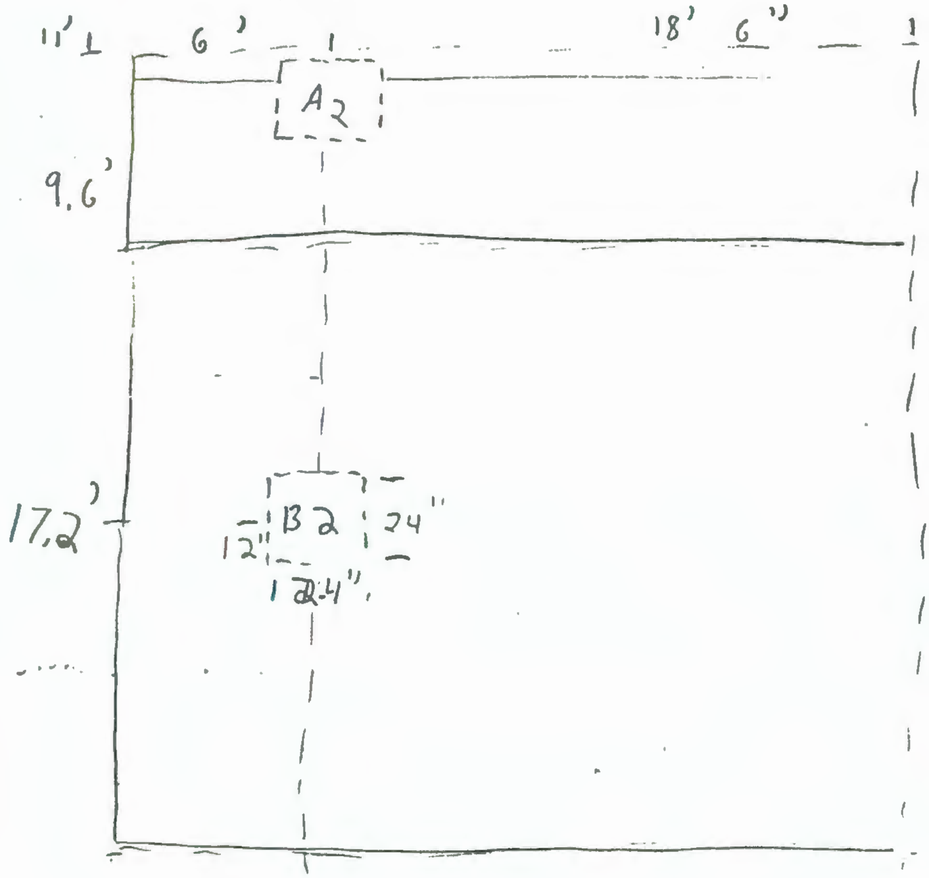
$$\left(\frac{6.75\,in}{12}\right)\left(150\ lb/ft^3\right) = 84.375\ lb/ft^2$$

Total dead load $= 94.375\ lb/ft^2$

ACI Eq. (9-2)

$$W_u = 1.4DL + 1.7S = 1.4(94.375) + 1.7(35\,psf)$$
$$= 132.125 + 59.5 = 191.6\,psf$$
$$\approx 192\,psf$$

Columns A2, B2



Column B2

$b_o = 2(24 + 24) = 96$ in

$$V_u = 192\left[\left(\frac{6' + 18'.5}{2}\right) \times 17.2' - (4\,ft^2)\right]$$

$= 39{,}686.4$ lb.

From Eq 13-18

$$\phi V_c = .85\left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\, b_o d$$

assume $d = 5$ in

$$\beta_c = \frac{2}{2} = 1$$

$$\phi V_c = .85(2 + 4)\sqrt{4000}\,(96\text{ in})(5\text{ in}) =$$

$= 154{,}825$ lbs.

From Eq. 13-19

$$\phi V_c = .85\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$$

$\alpha_s = 40$ for interior columns from (ACI Eq. 11-37)

$$= .85\left(\frac{40 \times 5}{96} + 2\right)\sqrt{4000} \times 96 \times 5 = 105{,}369.1 \text{ lbs}$$

From 13-20

$$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d = .85 \times 4\sqrt{4000} \times 96 \times 5 = 103{,}216.8$$

$\phi V_c > V_u$ Therefore thickness is OK.

Column A2, For Exterior columns ϕV_c should be $> 1.8-2\ V_u$

$b_o = 24 + 24 + 24 = 72$ in.

$$V_u = 192\left[\left(24.5 \times \frac{9.6}{2}\right) - \left(4 ft^2\right)\right] = 21{,}811 \text{ lb}$$

ϕV_c is the smallest of

$$\phi V_c = .85\left(2 + \frac{4}{1}\right)\sqrt{4000} \times 72 \times 5 \text{ in} = 116{,}118.8 \text{ lb}$$

Exterior column $\alpha_s = 30$

$$\phi V_c = .85\left(\frac{30 \times 5}{72} + 2\right)\sqrt{4000} \times 72 \times 5 = 79{,}025 \text{ lb}$$

$$\phi V_c = .85\left(4\sqrt{4000} \times 72 \times 5\right) = 77412.6 \text{ lb}$$

$$\phi V_c = 77412.6 = 3.55\ V_u$$

Thickness checks for shear;

(5

Calculation of Negative and Positive Moments for slab strip B

	B_1	B_2		B_3	B_4	
l_1	21' 6"		7'		21' 6"	
l_n	18' 6"		6'		18' 6"	
l_2	17.2'		17.2'		17.2'	
w_u	.192		.192		.192	
$M_0 = w_u l_2 l_n^2/8$	141.2		14.8		141.2	

M_0 coefficient	-.26	.52	-.70	-.65	.35	-.65	-.26	.52	-.70
Moments	-36.7	73.4	-98.8	-9.62	5.18	-9.62	-36.7	73.4	-98.9

	B_1	B_2	B_3	B_4
Effect of Pattern loads	—	—		—
Sum of column moments	36.7	11	11	36.7

Calculation of positive and Negative moments for strip 1

	A_1	B_1	C_1	D_1
l_1		12.3	13.3	13.3
l_n		15.3	15.3	15.3
l_2		11.25	11.25	11.25
w_u		.192	.192	.192
M_0		42.5	42.5	42.5

M_0 coefficient	-.26	.52	-.70	-.65	.35	-.65	-.65	.35	-.65
Moments	-11.1	22.1	-29.75	-27.6	14.9	-27.6	-27.6	14.9	-27.6

	A_1	B_1	C_1	D_1
Sum of column moments	11.1	21	21	21

Calculation of Negative and Positive moments for slab strip A

	A_1	A_2	A_3	A_4
l_1	15.5	4		15.5
l_n	18.5	6		18.5
l_2	9.6	9.6		9.6
W_u	.192	.192		.192
M_o	78.9	5.6		78.9

M_o coefficient	-.26	.52	-.70	-.65	.35	-.65	-.70	.52	-.26
Moments	-20.5	41	-55.2	-3.64	1.96	-3.64	-55.2	41	-20.5

Column moments from slab load: 20.5 7.7 7.7 20.5

Calculation of Negative and Positive moments for slab strip 2

	A2	B2	C3	D4
l_1	12.3	13.3		13.3
l_n	15.3'	15.3'		15.3'
l_2	14.25	14.25		14.25
W_u	.129	.129		.129
M_o	53.8	53.8		53.8

M_o coefficient	-.26	.52	-.76	-.65	.35	-.65	-.70	.52	-.26
Moments	-14	28	-37.7	-35	18.8	-35	-37.7	28	-14

Column moments from slab load: 14 10.1 10.1 10.1

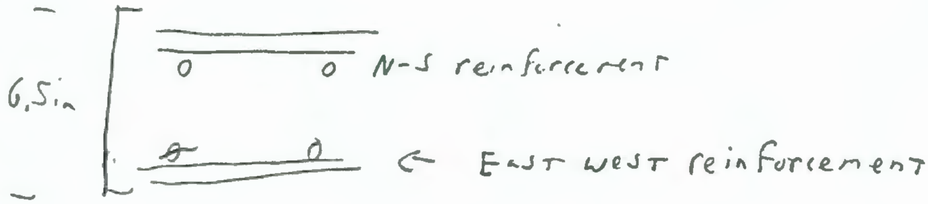
Calculate required reinforcement:

Calculate: As (req'd)

$$As = \frac{Mu}{\phi f_y j d}$$

a) Compute d,

With a slab size of 6.75 in Take reinforcement of 1 in diameter



$d = 6.75 - 3/4 \text{ in} - 1/2$ bar diameter $= 5.5$ in

b) $M_{o\,max} = -37.7$ ft-kips   Take $j = .925$

$$As(req'd) = \frac{37.7 \times 12000}{.9 \times 40{,}000 \times .925 \times 5.94} = 2.29 \text{ in}^2$$

c) Compute a and a/d and check $\rho \leq .75\rho b$.

$$a = \frac{As f_y}{.85 f'_c b} = \frac{2.29 \times 40{,}000}{.85 \times 4000\ (8 \times 12)} = .28$$

$$\frac{a}{d} = \frac{.28}{5.5} = .05$$

From Table A-5 [Reinforced Concrete Text Author James Macgregor 2nd Edition

$.75\rho b = .437$

$\rho < .75\rho b$ so OK ✓

d) Compute jd and the constant for computing As.

$$jd = d - \frac{a}{2} = 5.5 - \frac{.28}{2} = 5.36 \text{ in}$$

$$A_s = \frac{M_u \times 12000}{.9 \times 40{,}000 \times 5.36} = .062\, M_u =$$

Find A_s min. From ACI Sec 13.4.1 $A_s(min) = .002\, bh$

	Edge Column Strip	Middle Strip	Column Strips		Middle Strip	Edge Column Strip
Exterior Negative Moments	6.6	10.9	8	8	10.9	6.6
1. Slab moment	A_4 -11.1		A_3 -14	A_2 -14		A_1 -11.1
2. Moment Coefficients	0 1.00	0 0	1.0	1.0	0 0	1.0
3. Moment to Column and Middle strips	-11.1	0	-	-14	0	-11.1
4. Wall moment	-	-	-	-		-
5. Total moment	-11.1	0	-14	-14	0	-11.1
6. As required	.69	0	.87	.87	0	.69
7. As (min)	1.07	1.8	1.3	1.3	1.8	1.07
8. Choose Steel	6#4	10#4	7#4		10#4	6#4
9. AS provided	1.20	2.00	1.4		2.00	1.20

End span Positive Moments						
1. Slab moment	22.2		28	28		22.2
2. Moment Coefficients	.2 .6	.4 .2	.6	.6	.2 .4	.6
3. Moment to Column and middle strips (ft-kip)	13.3	8.9 5.6	16.8	16.8	5.6 8.9	13.3
4. Wall Moment (ft-kip)						
5. Total Moment in Strip	13.3	14.5	16.8	16.8	14.5	13.3
6. As required ($in.^2$)	.83	.9	1.04	1.04	.9	.83
7. Min As (in^2)	1.07	1.8	1.3	1.3	1.8	1.07
8. Choose Steel	6#4	10#4	7#4	7#4	10#4	6#4
9. As provided	1.2	2.0	1.4	1.4	2.0	1.20

First Interior Negative Moments

	Edge Column Strip (B4)	Middle Strip	Column Strips (B3)	Column Strips (B2)	Middle Strip	Edge Column Strip (B1)
1. Slab Moment (ft-kip)	-27.6		-35	-35		-27.6
2. Moment coefficients	.75	.25 .125	.75	.75	.125 .25	.75
3. Moment to column and middle strips:	-20.7	-6.9 -4.4	-26.25	-26.25	-4.4 -6.9	-20.7
4. Wall Moment (ft-kip)						
5. Total Moment in strip	-20.7	-11.3	-26.25	-26.25	-11.3	-20.7
6. As required (in^2)	1.28	.7	1.63	1.63	.7	1.28
7. Min As (in^2)	1.07	1.8	1.3	1.3	1.8	1.07
8. Choose steel	7#4 bars	9#4	9#4		9#4	7#4
9. As provided	1.40	1.80	1.80	1.80	1.80	1.40

Interior Positive Moments

	Edge Column Strip	Middle Strip	Column Strip	Column Strip	Middle Strip	Edge Column Strip
1. Slab moment (ft-kip)	14.9		18.8	18.8		14.9
2. Moment coefficients	.6	.4 .2	.6	.6	.2 .4	.6
3. Moment to column and middle strips (ft-kip)	8.94	5.96 3.76	11.28	11.28	3.76 5.96	8.94
4. Wall Moment						
5. Total Moment	8.94	9.72	11.28	11.28	9.72	8.94
6. As required (in^2)	.55	.6	.7	.7	.6	.55
7. Min As (in^2)	1.07	1.8	1.3	1.3	1.8	1.07
8. Choose steel.	6#4 1.20	9#4 1.80	7#4 1.40		9#4 1.80	6#4 1.20

Division of Moment to column and Middle Strips:
North-South Strips -.

	5.8	7.6	9.6	7.6	9.6
Exterior Negative Moments	A1		B1		C1
1. Slab moment (S--")	-20.5		-36.7		-36.7
2. Moment Coefficients	1.00	0	1.00	0	1.00
3. Moment to column and middle strips	-20.5	0	-36.7	0	-36.7
4. Wall moment					
5. Total moment	-20.5	0	-36.7	0	-36.7
6. As required	1.35	0	2.42	0	2.42
7. As (min)	.94	1.23	1.56	1.23	1.56
8. Choose steel	7#4	7#4	13#4 bars	7#4	13#4
9. As provided	1.40	1.40	2.60	1.40	2.60

End Span Positive moment					
1. Slab Moment	41		73.4		73.4
2. Moment Coefficients	.6	.4 .2	.6	.2 .2	.6
3. Moment To Column	24.6	16.4 14.7	44.04	14.7 14.7	44.04
4. Wall Moment					
5. Total moment	24.6	31.1	44.04	29.4	44.04
6. As required (in^2)	1.62	2.07	2.91	1.94	2.91
7. As (min)	.94	1.23	1.56	1.23	1.56
8. Choose Steel	9#4	11#4	10#5	10#4	10#5
9. As Provided	1.80	2.20	3.10	2.00	3.10

First interior Negative Moments	A2		B2		C3
1. Slab moment	-55.2		-98.8		-98.8
2. Moment Coefficients	.75	.25 .125	.75	.125 .125	.75
3. Moment to column and middle strips	-41.4	-13.8 -12.35	-74.1	-12.35 -12.35	-74.1
4. Wall moments					
5. Total Moment in strip.	-41.4	-26.15	-74.1	-24.7	-74.1
6. As required (in^2)	2.7	1.7	4.89	1.63	4.89
7. As min	1.62	2.07	2.91	1.94	2.91
8. Choose steel	9#5	7#5	12#6	7#5	12#6
9. As Provided	2.79	2.17	5.28	2.17	5.28

Interior positive Moments	5.8	7.6	9.6	7.6	9.6
1. slab Moment	1.96		5.18		5.18
2. Moment Coefficients	.6	.4 / .2	.6	.2 / .2	.6
3. Moment to Column and middle strips	1.18	.78 / 1.04	3.1	1.04 / 1.04	3.1
4. Wall moments					
5. Total moment in strip (ft-kip)	1.18	1.82	3.1	2.08	3.1
6. As required (in^2)	.08	.12	.2	.14	.2
7. As min (in^2)	.94	1.23	1.56	1.23	1.56
8. Choose steel.	5#4	7#4	8#4	7#4	8#4
9. As Provided	1.00	1.40	1.60	1.40	1.60

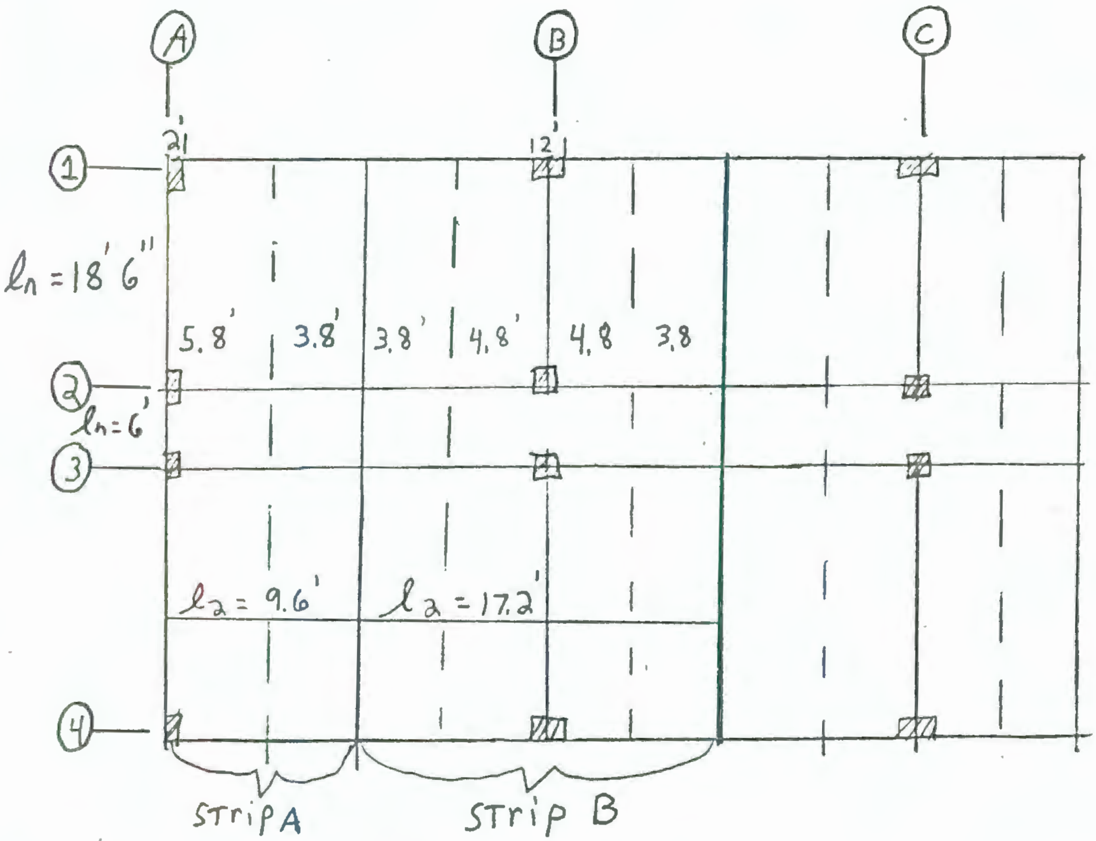
Design of Floor span (Floors 1 & 2)

Direct design Method:

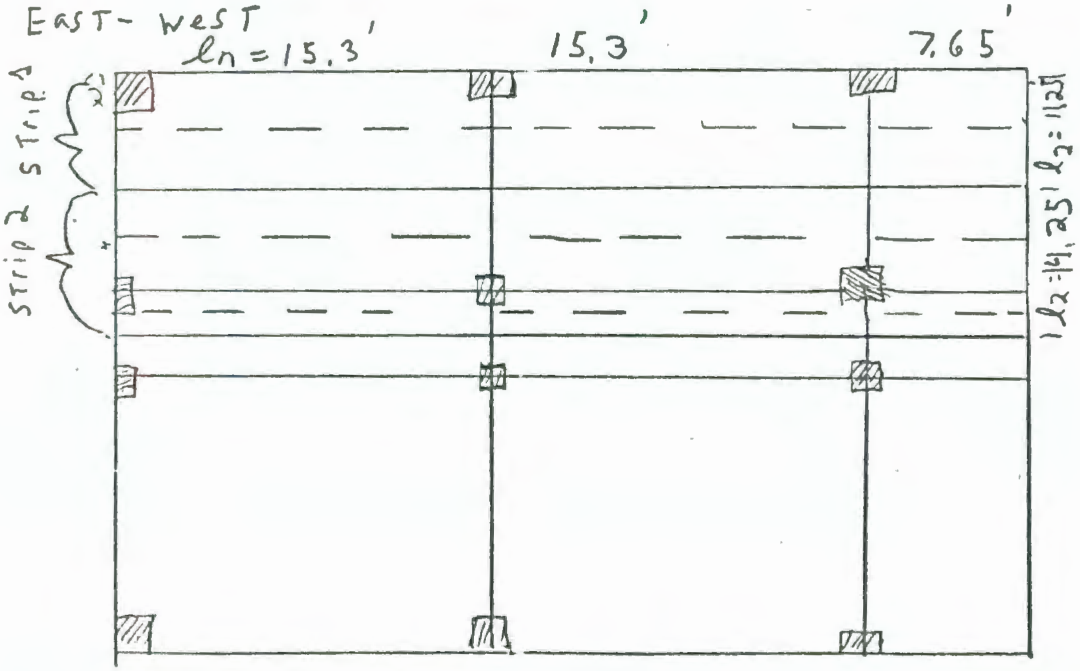
Diagram

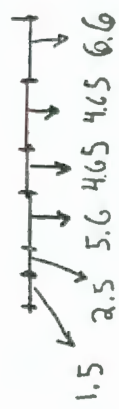
a) North - South

span = 44,25''



b) EAST - WEST





1) select thickness

a) Determine thickness to limit Deflection;

max l_n = 18 FT 6 in = 222 in

$$\min h = \frac{l_n}{33} = \frac{222}{33} = 6.73 \approx 6.75 \text{ in}$$

b) Check thickness for shear.

1b) Calculate Dead load

Floor deadload:

Slab load $\left(\frac{6.75 \text{in}}{12}\right)\left(150 \frac{lb}{FT^3}\right)$	=	84.375 lb/FT^2
Suspended steel channels	=	1 lb/FT^2
Suspended lighting	=	3 lb/FT^2
		88.38 lb/FT^2

Live load:

Corridors	=	100 lb/FT^2
Apartments	=	40 lb/FT^2

$W_u = 1.4 DL + 1.7 LL$

For corridors

$W_u = 1.4(88.38) + 1.7(100) = 123.7 + 170 = 293.7 \, psf \approx 294 \, psf$

$W_u = 1.4(88.38) + 1.7(40) = 123.7 + 68 = 191.7 \approx 192 \, psf$

Column B2:

*Assume a 2'x2' Area for calculation of column properties.

$b_o = 2(24+24) = 96 \text{ in}$

$V_u(\text{corridor}) = 294\left[\left(\frac{6'+18.5'}{2}\right) \times 17.2' - (4 ft^2)\right]$

$= 60769.8 \, lb$

$$V_u\,(\text{Apartments}) = 192\left[\left(\frac{6+18.5}{2}\right)\times 17.2' - (4ft^2)\right]$$

$$= 39,686.4$$

From: "Reinforced concrete, Mechanics and Design", 2nd Edition
Author James Macgregor

Shear checks:

Eq. 13-18

$$\phi V_c = .85\left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d$$

Assume, d = 5in

$\beta_c = \frac{2}{2} = 1$ $\qquad \phi V_c = .85(2+4)\sqrt{4000}\,(96in)(5in) = 154,825\ lbs$

EQ 13-19

$$\phi V_c = .85\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d$$

$\alpha_s = 40$ for interior colums (ACI Eq. 11-37)

$$= .85\left(\frac{40\times 5}{96}+2\right)\sqrt{4000}\times 96\times 5 = 103,216.8\ lbs$$

$\phi V_C > V_{u\,corridor}$ and $\phi V_C > V_{u\,apartments}$

Therefore thickness is OK ✓.

Column A2: For Exterior Columns $\phi V_c > 1.8-2\,V_u$

$b_o = 24+24+24 = 72$ in.

$$V_{u\,apartment} = 192\left[\left(24.5\times\frac{9.6}{2}\right)-(4ft^2)\right] = 21,811\ lb$$

$$V_{u\,corridor} : 294\left[\left(24.5\times\frac{9.6}{2}\right)-(4ft^2)\right] = 33,398.4\ lb$$

ϕV_c is the smallest of

$$\phi V_c = .85\left(\frac{30\times 5}{72} + 2\right)\sqrt{4000} \times 72 \times 5 = 79,025\ lbs$$

$$\phi V_c = .85\left(2 + \frac{4}{1}\right)\sqrt{4000} \times 72 \times 5 = 116,118.8\ lb$$

$$\phi V_c = .85\left(4\sqrt{4000} \times 72 \times 5\right) = 77412.6\ lb$$

$\phi V_c = 77412.6 = 3.55\ V_u$ apartments OK for shear ✓

$\phi V_c = 77412.6 = 2.3\ V_u$ corridor OK for shear ✓

Calculation of Negative and Positive Moments for Slab B.

	B_1			B_2			B_3 / B_4		
l_1		21'6"			7'			21'6"	
l_n		18'6"			6'			18'6"	
l_2		17.2'			17.2'			17.2'	
W_u		.192			.294			.192	
$M_o = W_u l_2 l_n^2/8$		141.2			22.7			141.2	
M_o coefficient	-.26	.52	-.70	-.65	.35	-.65	-.70	.52	-.26
moments	-36.7	73.4	-98.84	-14.8	7.9	-14.8	-98.84	73.4	-36.7
Σ Column moments	36.7								36.7

Calculation of positive and Negative Moments for Slab Strip A:

A₁ A₂ A₃ A₄

l_1	15.5	4	15.5
l_n	18.5	6	18.5
l_2	9.6	9.6	9.6
W_u	.192	.294	.192
M_o	78.9	12.7	78.9

M_o coefficients	-.26	.52	-.70	-.65	.35	-.65	-.70	.52	-.26
Positive Neg Moments	20.5	41	-55.2	-8.3	4.45	-8.3	-20.5	41	-55.2

Wall load	.42	.42	.42
Wall M_o	18	2	18

Moments from wall	-4.68	9.36	-12.6	-1.3	.7	-1.3	-12.6	9.36	-4.68

Column moment from slab load	20.5	37	37	55.2

Calculation of positive and Negative for slab strip 1

A₁ B₁ C₁ D₁

l_1	12.3	13.3	13.3
l_n	15.3	15.3	15.3
l_2	11.25	11.25	11.25
W_u	.192	.192	.192
M_o	63.2	63.2	63.2

M_o Coefficients	-.26	.52	-.70	-.65	.35	-.65	-.26	.52	-.70
Negative and Positive Moments	-16.4	32.9	-44.24	-41.1	22.12	-41.1	-16.4	32.9	-44.24

Sum of column moments	16.4	0	0	0

Wall load	.42	.42	.42
Wall M_o	12.3	12.3	12.3

Neg, Pos M_o	-3.2	6.4	-8.61	-8	4.3	-8	-3.2	6.4	-8.61

Σ column M_o [illegible]

Calculation of Positive and Negative Moments for Slab Strip 2

	A_2		B_2		C_2		D_2		
l_1		12.3			13.3		13.3		
l_n		15.3			15.3		15.3		
l_2		14.25			14.25		14.25		
W_u		.294			.294		.294		
M_o		122.6			122.6		122.6		
M_o Coefficients	-.26	.52	-.70	-.65	.35	-.65	-.26	52	-.70
Positive and Neg Moments	-31.9	63.8	-85.8	-80	42.9	-80	-31.9	63.8	-85.8
Sum of column moments	31.9		0			0		0	

Calculate required reinforcement:

$$As = \frac{Mu}{\varphi f_y j d}$$

a) Compute d.

Slab size = 6.75 in. Take reinforcement to be 1 in.

$$d = 6.75'' - 3/4'' - 1/2'' = 5.5 \text{ in.}$$

b) M_u max = -36.7 ft-kips Take J = .925

$$As\,(\text{req'd}) = \frac{36.7 \times 12000}{.9 \times 40{,}000 \times .925 \times 5.5} = 2.4 \text{ in}^2$$

c) Compute a and a/d and check $\rho \leq .75\rho_b$

$$a = \frac{As\,f_y}{.85 f'_c b} = \frac{2.4 \times 40{,}000}{.85 \times 4000 (8 \times 12)} = .29$$

$$\frac{a}{d} = \frac{.29}{5.5} = .05$$

From Table A-5 [reinforced Concrete Text, Author James Macgregor]

$$.75\rho_b = .437$$

$$\rho < .75\rho_b \quad \checkmark \text{ ok}$$

d) Compute jd and the constant for computing As.

$$jd = d - \frac{a}{2} = 5.5 - \frac{.29}{2} = 5.36$$

$$\underset{(\text{req'd})}{As} = \frac{Mu \times 12000}{.9 \times 40{,}000 \times 5.36} = .062\,Mu$$

Find As min. From ACI code sec 13.4.1 As (min) = .002 bh

Division of Moment to Column and middle Strips.
East-West strips:

Exterior Negative Moments

		Edge Column Strip	Middle Strip	Column Strips		Middle Strip	Edge Column Strip	
		6.6	10.9	8	8	10.9	6.6	
		A4		[illegible]	A2		A1	
1. Slab moment		-16.4		-32	-32		-16.4	
2. Moment Coefficients	0	1.00	0.0 0.0	1.00	1.00	0.0 0.0	1.00	0.0
3. Moment to Column and middle Strips		-16.4	0	-32	-32	0	-16.4	
4. Wall moment		-3.2					-3.2	
5. Total moment in Strip		-19.6	0	-32	-32	0	-19.6	
6. As required		1.22	[illegible]	2.0	2.0	0	1.22	
7. Min As		1.1	1.81	1.3	1.3	1.81	1.1	
8. Choose steel		7#4	6#5	11#4	11#4	6#5	7#4	
9. As provided		1.4	1.86	2.2	2.2	1.86	1.4	

End Span Positive Moments

1. Slab Moment (FT-kips)	32.9		63.8	63.8		32.9
2. Moment Coefficients	.6	.4 .2	.6	.6	.2 .4	.6
3. Moment to Column and middle Strips	19.7	13.2 12.8	38.3	38.3	12.8 13.2	19.7
4. Wall moment	6.4					6.4
5. Total moment in Strip	26.1	26	38.3	38.3	26	26.1
6. As (req'd)	1.62	1.61	2.4	2.4	1.61	1.62
7. Min As	1.1	1.8	1.3	1.3	1.8	1.1
8. Choose steel	6#5	6#5	8#5	8#5	6#5	6#5
9. As provided	1.86	1.86	2.48	2.48	1.86	1.86

First Interior Negative moments

	Edge Column strip	Middle strip		Column strips		middle strip		Edge column strip
	6.6	10.9		8	8	10.9		6.6
	B4			B3	B2			B1
1. Slab moment (ft-kip)	-41.1			-80	-80			-41.1
2. Moment coefficients	.75	.125	.125	.75	.75	.125	.25	.75
3. Moment to column and middle strips	-30.8	-10.3	-10	-60	-60	-10	-10.3	-30.8
4. Wall moment	-8							-8
5. Total moment	-38.8	-20.3		-60	-60	-20.3		-38.8
6. As required	2.4	1.26		3.72	3.72	1.26		2.4
7. As min	1.1	1.8		1.3	1.3	1.8		1.1
8. Choose steel	6#6	7#4		7#7	7#7	7#4		6#6
9. As provided	2.64	1.4		4.2	4.2	1.4		2.64

Interior Positive Moments

	Edge Column strip	Middle strip		Column strips		middle strip		Edge column strip
1. Slab Moment	22.1			42.9	42.9			22.1
2. Moment coefficients	.6	.40	.20	.6	.6	.2	.4	.6
3. Moment to column and Middle strips	13.3	8.84	8.58	25.7	25.7	8.58	8.84	13.3
4. Wall moment	4.3							4.3
5. Total moment	17.6	17.42		25.7	25.7	17.42		17.6
6. As required.	1.1	1.08		1.6	1.6	1.08		1.1
7. Min As	1.1	1.8		1.3	1.3	1.8		1.1
8. Choose steel	6#4	9#4		8#4	8#4	9#4		6#4
9. As provided	1.20	1.80		1.6	1.6	1.8		1.2

Division of Moment To column and middle Strips:
North — South Strips

Exterior Negative moments	5.8 A1	7.6	9.6 B1	7.6	9.6 C1	
1. Slab Moment	-20.5		-36.7		-36.7	
2. Moment Coefficients	1.00	0	1.0	0	1.00	0
3. Moment To column and middle strips:	-20.5	0	-36.7	0	-36.7	
4. Wall moment	-4.68					
5. Total moment	25.18	0	-36.7	0	-36.7	
6. As required	1.56	0	2.28	0	2.28	
7. Min As	.93	1.23	1.56	1.23	1.56	
8. Choose Steel	5#4	7#4	12#4	7#4	12#4	
9. As provided	1.0	1.40	2.4	1.40	2.4	

End span Positive Moments						
1. Slab moment	41		73.4		73.4	
2. Moment Coefficients	.6	.4 .2	.6	.2 .2	.6	.2
3. Moment To column and Middle strips	24.6	16.4 14.7	44	14.7 14.7	44	
4. Wall moment	9.36					
5. Total Moment	34	31.1	44	29.4	44	
6. As required	2.1	1.93	2.7	1.82	2.7	
7. Min As	.93	1.23	1.56	1.23	1.56	
8. Choose Steel	7#5	7#5	9#5	6#5	9#5	
9. As provided.	2.17	2.17	2.79	1.86	2.79	

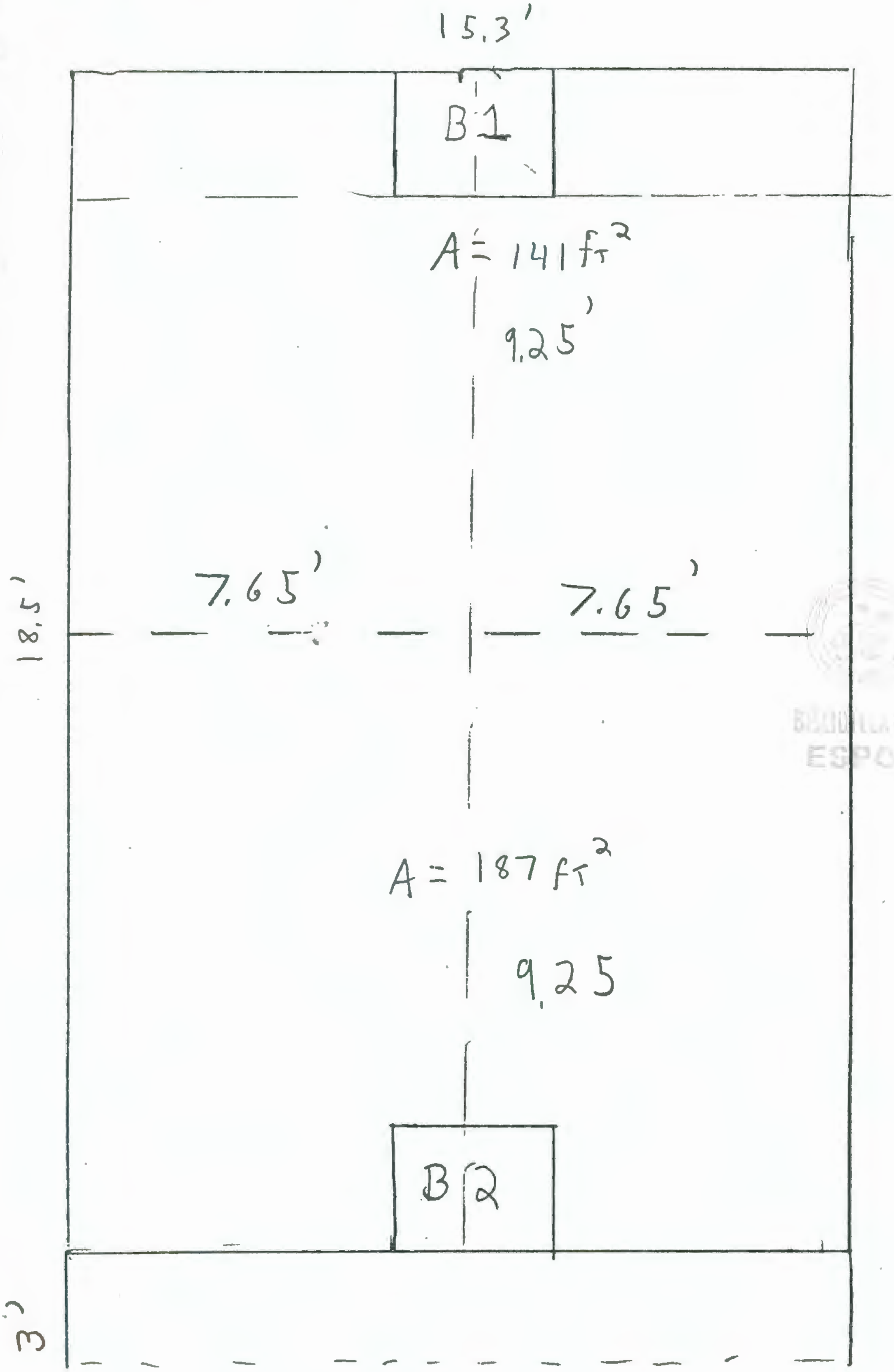
Exterior negative moments

	5.8	7.6		9.6	7.6		9.6
	A2			B2			C2
1. Slab moment	-8.3			-14.8			-14.8
2. Moment Coefficients	.75	.125	.125	.75	.125	.125	.75
3. Moment to column and middle strip	-6.23	-2.1	-1.9	-11.1	-1.9	-1.9	-11.1
4. Wall Moment	-1.3						
5. Total moment	-7.53	-4		-11.1	-3.8		-11.1
6. As required	.47	.25		.69	.24		.69
7. Min As	.93	1.23		1.56	1.23		1.56
8. Choose steel	5#4	7#4		8#4	7#4		8#4
9. As provided	1.0	1.4		1.6	1.4		1.6

Interior Positive moments

	A2			B2			C2
1. Slab moment	4.45			7.9			7.9
2. Moment coefficients	.6	.4	.2	.6	.2	.2	.6
3. Moment to column and middle strips	2.67	1.78	1.58	4.74	1.58	1.58	4.74
4. Wall moment	.7						
5. Total moment	3.37	3.36		4.74	3.16		4.74
6. As required	.2	.2		.29	.2		.29
7. Min As	.93	1.23		1.56	1.23		1.56
8. Choose steel	5#4	7#4		8#4	7#4		8#4
9. As provided	1.0	1.4		1.6	1.4		1.6

b) Exterior, Interior colums (E-W)



Tributary Area:

			Column
(E-W)	Exterior Columns	= 141 ft^2	B1
	Interior columns	= 187 ft^2	B2
(N-S)	Edge columns	= 70.8 ft^2	A_1
	Exterior column	= 93.7 ft^2	A_2

Calculate Pu:

a) Compute loads

loads

roof load = 192 psf
= .192 ksf

1st, 2nd, 3rd Floor loads
Corridors = 294 psf = .294 ksf
Apartments = 192 psf = .192 ksf

Column load:
Assume 2'x2' area, and 12' length

Weight = $2 \times 2 \times 12 \times 150\ \frac{lb}{ft^3}$ = 7.2 kips

b) Compute Pu for Column B1

A = 141 ft^2
wall load = 30.6 × .062 = 1.9 kips
roofload = 141 × .192 = 27 kips
1st floor = 141 × .192 = 27 kips
2nd floor = 141 × .192 = 27 kips
3rd floor = 141 × .192 = 27 kips
Colum load = 3 × 7.2 kips = 21.6 kips

Total load = 131.5 kips

c) Compute Pu for Column B2

(1st, 2nd, 3rd) Floors = 45.9 ft^2 × .294 ksf (Corridor) + 141 ft^2 × .192 ksf (Apartment)
13.5 kips + 27 kips = 40.5
= 3 × 40.5 = 121.5
roof load = 187 × .192 = 35.9 ≈ 36 kips
Column load = 21.6 kips

Total load = 179.1 kips

D) compute Pu for column A1

wall load	=	33.8 x .062	=	2.09
roof load	=	70.8 x .192	=	13.6
Floor loads	=	3 x 70.8 x .192	=	40.8
Column loads	=	3 x 7.2	=	21.6

13.6
+7.2
14.4
58.2

TOTAL load = 78 kips

E) compute Pu for column A2

Floors loads: corridor 23 x .294 + 70.75 x .192
6.8 + 13.6 = 20.4 kips

= 3 x 20.4 = 61.2

wall load	=	24.5 x .062	=	1.5
roof load	=	93.7 x .192	=	18
Column loads	=	3 x 7.2	=	21.6

TOTAL load = 102 kips

F) Pu listing

A1 = 78 kips
A2 = 102 kips

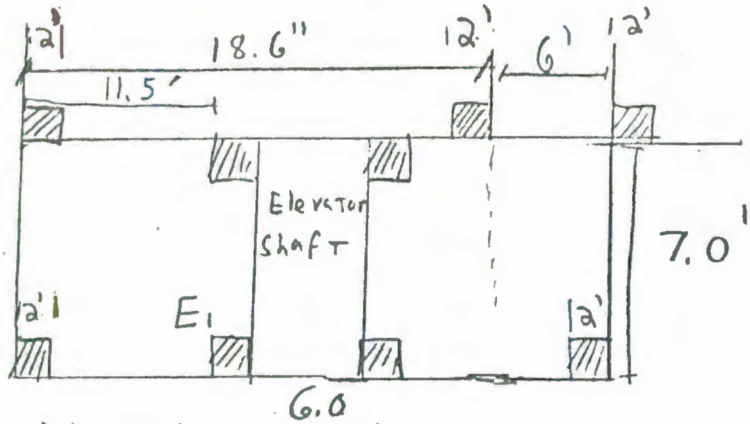
B1 = 131.5 kips
B2 = 179.1 kips

b) Thickness design

$$h = 6.75 \quad , \frac{h}{4} = \frac{6.75}{4} = 1.7'' \approx 2.0''$$

Since B2 is the worst case for Drop Panel size, all drop panels will be designed under its specifications.

Design of Elevator shaft.



First Floor column sketch:

roof slab design

1) Select Thickness.

a) same as rest of slab 6.75"

b) check thickness for shear

$$W_u = 1.4\,DL + 1.7\,LL$$
$$= 1.4(88.38) + 1.7(125) = 123.7 + 212.5 = 336.2\,psf$$

Column E1

$$b_o = 72\text{ in}$$

$$V_u = 336.2\left[\left(\frac{6 + 11.5}{2}\right) \times 7.0 - 4\,ft^2\right] =$$

$$= 19{,}247.5\,lb.$$

From Eq 13-18

$$\phi V_c = .85\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\,b_o\,d$$
$$= .85\left(2 + \frac{4}{1}\right)\sqrt{4000}\,(72)(5) = 116118.8\,lb$$

From Eq 13-19

$$\phi V_c = .85\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$$

$$= .85\left(\frac{40 \times 5}{72} + 2\right)\sqrt{4000}\,(72)(5) = 92{,}465 \text{ lb}$$

From Eq 13-20.

$$\phi V_c = \phi 4\sqrt{f_c'}\, b_o d = .85 \times 4\sqrt{4000} \times 72 \times 5 = 77412.6$$

Therefore $\phi V_c = 77412.6$, and $\phi V_c > V_u$. Thickness is OK.

c) Calculation of Negative and positive moments for Elevator shaft

N-S

l_1		7	
l_n		6	
l_2		7	
w_u		.336	
$M_0 = w_u l_2 l_n^2/8$		12.35	
Moment Coefficients	-.65	.35	-.65
Negative and Positive moments (ft-kips)	-8.03	4.3	-8.03
	8.03		8.03

E-W

l_1			
l_n		7	
l_2		6.75	
W_u		.336	
M_0		13.9	
Moment Coefficients	-.65	.35	-.65
Negative and positive moments	-9.	4.9	-9
	-9		-9

Calculate required reinforcement:

Calculate As(req'd)

$As = \frac{Mu}{\phi f_y jd} =$

a) Compute As : Assume No. 5 bars.

$d = 6.75 - .75 - 1.5(.625) = 5.06$ in

largest Mu = 73.4

$As(req'd) = \frac{73.4 \times 12000}{.9 \times 40{,}000 \times .925 \times 5.06} = 5.23\ in^2$

b) Compute a and a/d and check $\rho \leq .75$

$a = \frac{As\, f_y}{.85 f'_c b} = \frac{5.23 \times 40{,}000}{.85(4000)(9.6 \times 12)} = .53$ in

$\frac{a}{d} = \frac{.53}{5.06} = .105$

$\rho < .75 \rho_b$.

c) compute jd and the constant for computing As

$jd = d - a/2 = 5.06 - \frac{.53}{2} = 4.795 \approx 4.8$ in

$As = \frac{Mu \times 12000}{.9 \times 40{,}000 \times 5.06} = .066\ Mu$

As(min) = .002bh:

Reinforcement Design North - South

Negative Moments

	☐	6 feet	☐
1. Slab moment	- 8,03		- 8,03
2. Moment Coefficients	1,0		1,0
3. Moment to column and middle strip	- 8,03		- 8.03
4. As required	.52		.52
5. Min As	1,09		1,09
6. Choose steel	6 No 4 bars		6 No 4 bars
7. As provided	1,2		1,2

Positive moments

1. Slab moment	4,3			4,3
2. Moment Coefficients	,6	.2	,2	,6
3. Moment To column and middle strip	2.58	,86	,86	2.58
4. Total moment	2.58	1.72		2.58
5. As required	,168	.112		,168
6. Min As	1,09	1,09		1,09
7. Choose steel	6 No 4	6 No 4		6 No 4
8. As provided	1.2	1,2		1,2

East - West

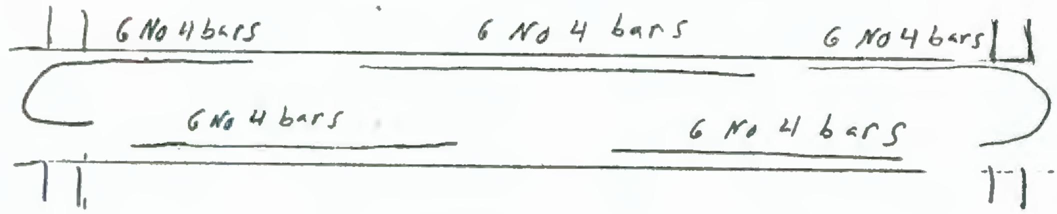
Negative moments

1. Slab moment	- 9	- 9
2. Moment Coefficients	1,0	1,0
3. Moments To column and middle strip	-.9	-.9
4. Total moment	-.9	-.9
5. As required	,35	,35
6. Min As	1,09	1,09
7. Choose steel	6 No 4 bars	6 No 4 bars
8. As provided	1. 2	1.2

positive moments

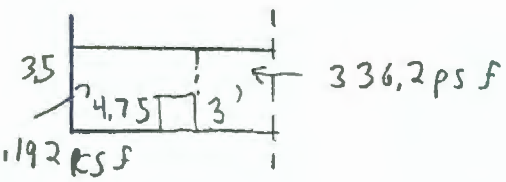
1. Slab moment	4.9			4.9
2. Moment coefficients	.6	.2	.2	.6
3. Moment to column and middle strip	2.94	.98	.98	2.94
4. Total moment	2.94	1.96		2.94
5. As req	.195	.127		.195
6. Min As	1.09	1.09		1.09
7. Choose steel	6 No4	6 No4		6 No4
8. As provided	1.2	1.2		1.2

Reinforcement Design East-West (North-South)



Column design:

Tributary area



Load:

load 1 $3.5 \times 4.75 \times .192\,ksf = 3.2\,K$

load 2 $3 \times 3.5 \times .336\,ksf = 3.5\,K$

Total Tributary Area load = 6.7 K

Basement column load.

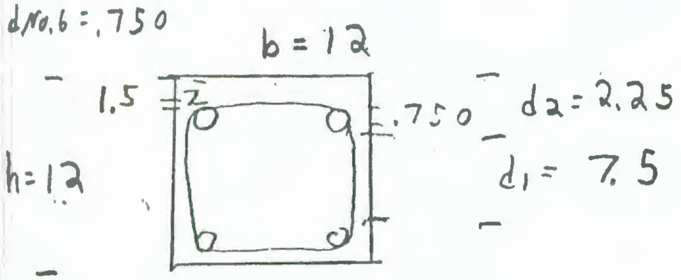
Slab = 6.7 K

3 columns = $3 \times 7.2K = 21.6K$ use 4 8x8 columns as calculated

for rest of building. This is the

Interaction check for combined moment and P_u.

1) Pick column:
Choose worst condition.
: Interior Basement column



Given
$A_{st} = 1.76$
$f_y = 60{,}000\,psi$
$f_c = 4000$

$A_g = 144$

1,a) Calculate ρt, and ε_y

$$\rho_t = A_{st}/A_g = 1.76/144 = .0122 \qquad \varepsilon_y = f_y/E_s = \frac{60{,}000}{29{,}000{,}000} = .00$$

2) Compute the concentric axial load capacity, and Max axial load capacity.

a) from Eq 11-1 (Nominal concentric load)

$$P_o = (.85 f_c')(A_g - A_{st}) + f_y(A_{st})$$
$$= (.85 \times 4{,}000)(144 - 1.76) + 60\,ksi \times 1.76$$
$$= 483.7 + 105.6 = 589.3\ kips$$

b) max allowable load

$$\phi P_n = .80\,\phi P_o \Rightarrow \text{where } \phi = .70$$
$$= 330\ kips$$

3) Compute ϕ and ϕm_n f balanced failure $(\varepsilon_{s1} = -\varepsilon_y)$

a) Determine c and the strains in reinforcement

$$(\text{Eq } 11\text{-}6) \quad c = \frac{.003}{.003 - (-1\varepsilon_y)} d_1 = \frac{.003}{.003 + .00207}(7.5\,in) = 4.4\,in$$

$$(\text{Eq } 11\text{-}7) \quad \varepsilon_{s2} = \left(\frac{c - d_2}{c}\right).003 = \left(\frac{4.4 - 2.25}{4.4}\right).003 = .0015$$

$$\varepsilon_{s1} = -.00207$$

b) Compute the stresses in the reinforcement layers.

Eq (11-8) $f_{s2} = \epsilon_{s2} E_s$ but $-f_y \le f_{s2} \le f_y$

$\epsilon_{s2} E_s = .0015 (29{,}000 \text{ ksi}) = 43.5 \text{ ksi}$, $-60 \le 43.5 \le 60$ OK

There fore $f_{s2} = 59.8 \text{ ksi}$,

c) Compute a, a = depth of equivalent rectangular stress block

$a = \beta_1 c$

$\beta_1 = 1.05 - .05 \left(\frac{f'_c}{1000 psi} \right) = .85$, $a = \beta_1 c = .85 \times 4.4 = 3.74$ in.

d) Compute the forces in the concrete and steel.

Eq (11-9) $C_c = (.85 f'_c)(ab)$

$= .85 \times 4 \text{ ksi} \times 3.74 \times 12 = 152.6 \text{ kips}$

The distance $d_1 = 7.5$ in to reinforcement layer 1 exceeds $a = 6.39$ in. Hence this layer of steel lies outside the compression stress block and does not displace concrete included in the area (ab) when computing C_c. Thus

$F_{s1} = f_{s1} A_{s1} = -60 \text{ ksi} \times 2 \text{ in}^2 = -120 \text{ kips (Tension)}$

Reinforcement layer 2 lies in the compression zone, since $a = 3.74$ exceeds $d_2 = 2.25$. Hence we must allow for the stress in the concrete displaced by the steel when we compute f_{s2}. From Eq 11-10b,

$F_{s2} = (f_{s2} - .85 f'_c) A_{s2}$

$= (59.8 - .85 \times 4) \times 2 = 112.8$

e) Compute P_n

$P_n = C_c + \Sigma F_{si} = 152.6 + 112.8 - 120 = 145.4 \text{ kips}$

Thus $P_n = P_b$

f) Compute M_n

$$M_n = C_c \left(\frac{h}{2} - \frac{a}{2} \right) + F_{s1} \left(\frac{h}{2} - d_1 \right) + F_{s2} \left(\frac{h}{2} - d_2 \right)$$

$$= 152.6 \left(\frac{12}{2} - \frac{3.74}{2} \right) + (-120) \left(\frac{12}{2} - 7.5 \right) + 112.8 \left(\frac{12}{2} - 2.25 \right)$$

$$= 630.2 + 180 + 423 = 1233.2 \text{ in kip} = 102.8 \text{ ft-kips}$$

$\phi M_n = .7 \times 102.8 = 71.96 \text{ ft-kips}$

MASS CODE 1990

SECTION 1205.1 FROST PROTECTION

"ALL PERMANENT SUPPORTS OF BUILDINGS AND STRUCTURES SHALL EXTEND A MINIMUM OF (4) FEET BELOW FINISHED GRADE EXCEPT WHEN ERECTED UPON SOUND BEDROCK..."

SECTION 1206.1 FOOTING DESIGN

"THE LOADS TO BE USED IN COMPUTING THE PRESSURE UPON BEARING MATERIAL DIRECTLY UNDERLYING FOUNDATIONS SHALL BE THE LIVE AND DEAD LOADS OF THE STRUCTURE, AS SPECIFIED IN SECTION 1115.0 INCLUDING THE WEIGHT OF THE FOUNDATIONS AND OF ANY IMMEDIATELY OVERLYING MATERIAL."

SECTION 1206.2 PRESSURE DUE TO LATERAL LOADS

"WHERE THE PRESSURE ON THE BEARING MATERIAL DUE TO WIND OR ANY OTHER LATERAL LOADS IS LESS THAN ONE-THIRD (1/3) OF THAT DUE TO DEAD AND LIVE LOADS, IT MAY BE NEGLECTED FOR THE FOUNDATIONS DESIGN."

THIS IS TRUE IN THE INSTITUTE HALL SINCE WIND & EARTHQUAKE LOADS ARE NOT MUCH FOR A 4 STORY BUILDING. THEY ARE LESS THAN 1/3 OF LIVE-DEAD LOADS.

SECTION 1209.0 CONCRETE FOOTINGS

1209.3.1 PLAIN CONCRETE

"IN PLAIN CONCRETE FOOTINGS, THE EDGE THICKNESS SHALL BE NOT LESS THAN 8 inches for FOOTINGS ON SOIL..."

TYPE OF SOIL IN WORCESTER

ACCORDING TO MASS CODE 1990 TABLE 1201 ON (ALLOWABLE BEARING PRESSURE FOR FOUNDATION MATERIALS)

MATERIAL CLASS	DESCRIPTION	CONSISTENCY	MAX ALLOW. NET BEARING PRESSURE (tons/ft²)
7	GRAVEL, WIDELY GRADED SAND AND GRAVEL, AND GRANULAR ABLATION TILL	DENSE MEDIUM	6

FROM "FOUNDATION ENGINEERING", 1974 edition by PECK, HANSON, & THORNBURN pg. 265

"AS A GENERAL RULE, A FACTOR OF SAFETY OF 3 SHOULD BE PROVIDED AGAINST THE LOADS SPECIFIED BY THE BUILDING CODE" → CONTINUES

• "THE FACTOR OF SAFETY SHOULD NOT ORDINARILY BE LESS THAN 2 EVEN IF THE MAXIMUM LOADS ARE KNOWN EXCEPTIONALLY WELL".

• NOTE: SINCE WE DON'T KNOW THE SOIL TYPES EXCEPTIONALLY WELL; WE USE A FACTOR OF SAFETY OF 3 FOR PU'S.

• FROM "FOUNDATION ENGINEERING", 1974 EDITION BY PECK, HANSON & THORNBURN PG. 114 TABLE 5.3.

TYPE	RELATIVE DENSITY	# BLOWS (PENETRATION TEST) PER ft, N
FOR SANDS/GRAVEL TILL	DENSE	30-50

• FOR AVERAGE # OF BLOWS WE USE N=40

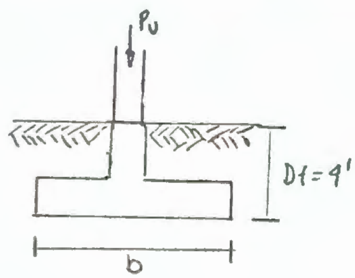
• TO USE PECK'S, HANSON'S & THORNBURN DESIGN METHOD FOR FOOTINGS WE USED TERZAGHI'S FIGURE 193 Pg. 309 ON "FOUNDATION ENGINEERING". THIS FIGURE IS A CHART FOR PROPORTIONING SHALLOW FOOTINGS. IT WAS DESIGNED TAKING A MAXIMUM VALUE SETTLEMENT OF 1".

DESIGN OF FOOTINGS

ALL FOOTINGS MUST BE:

- 4' BELOW FINISHED GRADE (Df) MASSACHUSETTS.
- THICKNESS ≥ 8"
- DL + LL x 3 SAFETY FACTOR
- AVERAGE N OF BLOWS = N=40
- MAX DIFFERENTIAL SETTLEMENT = 1"

1. FOOTING FOR CORNER COLUMNS



PU FROM CORNER COLUMNS ARE DL + LL OF ALL STORIES

$$P_U = \underline{101\ K} \times 3\,SF = \underline{303\ k}$$

FROM FIG. 19.3 (Pg 309 ON PECK, HANSON & THORNBURN
FOR N=40; $D_f/B = 0.67$

ASSUME B=6'

$$D_f/B = \frac{4'}{6'} = 0.67$$

$$q_{actual} = \frac{P_U}{A} = \frac{303}{6^2} = 8.42\ TSF$$

1 TSF = 2 KSF

$8.42/2 = 4.21\ TSF < 4.4\ TSF$

$q_{actual} < q_{all}$ OK ✓

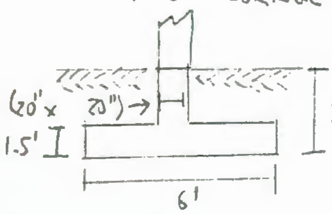
• FOOTING THICKNESS IS APPROX $b/4 = \frac{6}{4} = 1.5'$

• LOAD OF SLAB (CONCRETE) $= 1.5' \times 0.15\ K/FT^3 / 2 = +\ 0.113\ TSF$

• SURCHARGE LOAD (SOIL) $= 1.5' \times 0.115\ K/FT^3 / 2 = -\ 0.09\ TSF$

• TOTAL q ACTUAL = +4.21 TSF
+ 0.113 TSF
− 0.09 TSF
+ 4.23 TSF $< q_{ALL} = 4.4$ TSF ✓ OK

∴ FOR CORNER COLUMNS



A 6' x 6' SQUARE FOOTING

2. FOOTING FOR EDGE COLUMNS

$P_v = 129\ K \times 3\ S.F. = 387\ K$

Df = for MASSACHUSSETTS = 4'

ASSUME B = 7'

∴ $Df/B = \frac{4'}{7'} = 0.57$

FROM FIG. 19.3 (Pg 309 on PECK)

For N = 40 ; $Df/B = 0.57$

$q_{ALL} \simeq 4.4$ TSF

$q_{ACTUAL} = \frac{P_v}{A} = \frac{387}{7^2} = 7.90$ KSF

$= \frac{7.9\ KSF}{2} = 3.95$ TSF < 4.4 TSF

$q_{ACTUAL} < q_{ALL}$ OK

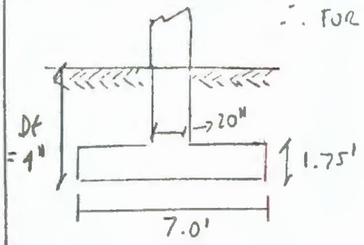
• FOOTING THICKNESS IS APPROXIMATELY $B/4 = \frac{7'}{4} = 1.75'$

• LOAD OF SLAB (CONCRETE) $= 1.75' \times 0.15\ K/FT^3/2 = +0.13$ TSF

• SURCHARGE LOAD $= 1.75' \times 0.115\ K/FT^3/2 = -0.10$ TSF

• TOTAL q ACTUAL = +3.95 TSF
+ 0.13 TSF
− 0.10 TSF
+ 3.98 TSF < 4.4 TSF ✓ OK.
$q_{ACT} < q_{ALL}$

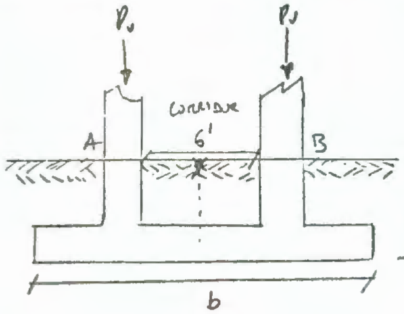
∴ FOR EDGE COLUMNS



A $7.0' \times 7.0'$
SQUARE FOOTING

3. FOOTING FOR INTERIOR COLUMNS (COMBINED FOOTING)

E-W VIEW



$P_u = 179\,k$

$2P_u = 358k \times 3\,SF = 1074\,k$ @ CENTROID OF FOUNDATION (3' FROM A AND 3' FROM B)

FROM FIG. 13.3 (P_g 309 ON Peck)

Assume $b = 11.5'$

$\therefore \frac{Df}{b} = \frac{5}{11.5} = 0.43$

FOR $N = 40$; $Df/B = 0.36$

$q_{all} \approx 4.4$ TSF

- $q_{ACTUAL} = \frac{P_u}{A} = \frac{1074}{11.5^2} = 8.12$ KSF

$\frac{8.12}{2} = 4.06$ TSF $< q_{all} = 4.4$ TSF ✓ OK

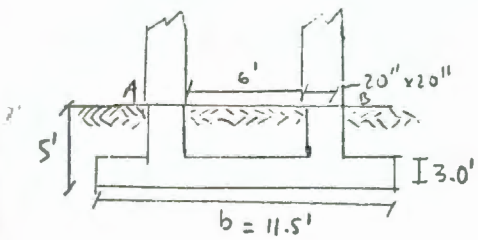
- FOOTING THICKNESS IS APPROXIMATELY $b/4 = \frac{11.5'}{4} = 2.9' \approx 3.0'$

- LOAD OF SLAB = $3.0' \times 0.15\,k/ft^3/2 = +0.225$
- SURCHARGE LOAD = $3.0' \times 0.115\,k/ft^3/2 = -0.0173$
SOIL

- TOTAL q ACTUAL = + 4.06 TSF
+ 0.225 TSF
− 0.173 TSF
4.112 TSF < 4.4 TSF
$q_{ACT} < q_{all}$ ✓ OK.

∴ For Interior Columns

A $11.5' \times 11.5'$
SQUARE FOOTING



Appendix F

Computer Analysis for Steel Frame

PLANE FRAME ANALYSIS FOR WIND STEEL DESIGN (N-S side)
Preliminary

UMBER OF JOINTS = 20
BER OF MEMBERS = 28
BER OF MATERIALS = 1
UMBER OF SUPPORT JOINTS = 4
UMBER OF LOADED JOINTS = 16

OINT DATA

OINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	318.000	432.000	0	0	0
7	318.000	306.000	0	0	0
8	318.000	186.000	0	0	0
9	318.000	66.000	0	0	0
10	318.000	.000	1	1	1
11	744.000	.000	1	1	1
12	744.000	66.000	0	0	0
13	744.000	186.000	0	0	0
14	744.000	306.000	0	0	0
15	744.000	432.000	0	0	0
16	1062.000	432.000	0	0	0
17	1062.000	306.000	0	0	0
18	1062.000	186.000	0	0	0
19	1062.000	66.000	0	0	0
20	1062.000	.000	1	1	1

EMBER DATA

EMBER	J1	J2	AX	IZ	E
1	1	2	7.080	18.300	29000.0
2	2	3	7.080	18.300	29000.0
3	3	4	7.080	18.300	29000.0
4	4	5	7.080	18.300	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	9.130	110.000	29000.0
10	8	9	9.130	110.000	29000.0
11	7	8	9.130	110.000	29000.0
12	6	7	9.130	110.000	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
7	11	12	9.130	110.000	29000.0
8	12	13	9.130	110.000	29000.0
19	13	14	9.130	110.000	29000.0
20	14	15	9.130	110.000	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	7.080	18.300	29000.0
26	18	19	7.080	18.300	29000.0
27	17	18	7.080	18.300	29000.0
8	16	17	7.080	18.300	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	3.000	-108.900	-155.70
3	3.800	-77.800	-155.70
4	3.900	-46.300	-155.70
5	2.000	-13.500	-76.70
6	.000	-30.500	-60.90
7	.000	-86.050	-123.70
8	.000	-141.150	-123.70
9	.000	-196.250	-123.70
12	.000	-196.250	123.70
13	.000	-141.150	123.70
14	.000	-86.050	123.70
15	.000	-30.500	60.90
16	.000	-13.500	76.90
17	.000	-46.300	155.70
18	.000	-77.800	155.70
19	.000	-108.900	155.70

JOINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.05272	-.07793	-.00035
3	.27620	-.15683	-.00049
4	.41713	-.19104	-.00057
5	.47809 ΔMAX.	-.19911	-.00064
6	.47619	-.29850	-.00032
7	.41517	-.28388	-.00037
8	.27431	-.23078	-.00051
9	.05123	-.11353	-.00045
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.05007	-.11372	-.00025
13	.27304	-.23119	-.00018
14	.41374	-.28439	.00002
15	.47405	-.29902	.00009
16	.47365	-.20289	.00063
17	.41356	-.19468	.00057
18	.27290	-.15983	.00051
19	.04988	-.07934	.00037
20	.00000	-.00000	.00000

MEMBER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	242.419	.911	32.89
1	2	-242.419	-.911	27.23
2	2	135.002	.638	38.90

F-

2	3	-135.002	-.638	37.71
3	3	58.546	.287	17.55
3	4	-58.546	-.287	16.85
4	4	13.150	-.047	-2.68
4	5	-13.150	.047	-3.29
5	5	2.047	-.350	-73.41
5	6	-2.047	.350	-37.88
6	4	3.566	-.904	-169.86
6	7	-3.566	.904	-117.64
7	3	3.448	-1.344	-210.96
7	8	-3.448	1.344	-216.56
8	2	2.728	-1.483	-221.83
8	9	-2.728	1.483	-249.70
9	9	455.437	4.823	137.17
9	10	-455.437	-4.823	181.13
10	8	258.696	3.664	218.45
10	9	-258.696	-3.664	221.22
11	7	117.168	1.951	120.64
11	8	-117.168	-1.951	113.51
12	6	30.717	.333	22.33
12	7	-30.717	-.333	19.64
13	6	1.714	-.133	-45.36
13	15	-1.714	.133	-11.15
14	7	1.948	-.503	-146.35
14	14	-1.948	.503	-68.11
15	8	1.736	-.966	-239.10
15	13	-1.736	.966	-172.39
16	9	1.569	-.992	-232.39
16	12	-1.569	.992	-190.19
17	11	456.210	5.590	196.31
17	12	-456.210	-5.590	172.61
18	12	259.197	4.380	260.93
18	13	-259.197	-4.380	264.62
19	13	117.361	2.904	169.15
19	14	-117.361	-2.904	179.32
20	14	30.755	1.283	78.90
20	15	-30.755	-1.283	82.72
21	15	.432	.122	-10.66
21	16	-.432	-.122	49.45
22	14	.327	.052	-66.41
22	17	-.327	-.052	83.03
23	13	.260	-.280	-137.68
23	18	-.260	.280	48.75
24	12	.359	-.229	-119.65
24	19	-.359	.229	46.85
25	19	246.835	1.376	48.41
25	20	-246.835	-1.376	42.44
26	18	137.706	1.018	61.71
26	19	-137.706	-1.018	60.44
27	17	59.626	.758	45.73
27	18	-59.626	-.758	45.25
28	16	13.378	.432	27.45
28	17	-13.378	-.432	26.94

..CTIONS

JOINT	RX	RY	MZ
1	-.911	242.419	32.89
10	-4.823	455.437	181.13

11	-5.590	456.210	196.31
20	-1.376	246.835	42.44

PLANE FRAME ANALYSIS FOR EARTHQUAKE STEEL DESIGN (N-S side

Preliminary

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	318.000	432.000	0	0	0
7	318.000	306.000	0	0	0
8	318.000	186.000	0	0	0
9	318.000	66.000	0	0	0
10	318.000	.000	1	1	1
11	744.000	.000	1	1	1
12	744.000	66.000	0	0	0
13	744.000	186.000	0	0	0
14	744.000	306.000	0	0	0
15	744.000	432.000	0	0	0
16	1062.000	432.000	0	0	0
17	1062.000	306.000	0	0	0
18	1062.000	186.000	0	0	0
19	1062.000	66.000	0	0	0
20	1062.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	7.080	18.300	29000.0
2	2	3	7.080	18.300	29000.0
3	3	4	7.080	18.300	29000.0
4	4	5	7.080	18.300	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	9.130	110.000	29000.0
10	8	9	9.130	110.000	29000.0
11	7	8	9.130	110.000	29000.0
12	6	7	9.130	110.000	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	9.130	110.000	29000.0
18	12	13	9.130	110.000	29000.0
19	13	14	9.130	110.000	29000.0
20	14	15	9.130	110.000	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	7.080	18.300	29000.0
26	18	19	7.080	18.300	29000.0
27	17	18	7.080	18.300	29000.0
28	16	17	7.080	18.300	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	2.000	-108.900	-155.70
3	4.100	-77.800	-155.70
4	6.700	-46.300	-155.70
5	7.000	-13.500	-76.70
6	.000	-30.500	-60.90
7	.000	-86.050	-123.70
8	.000	-141.150	-123.70
9	.000	-196.250	-123.70
12	.000	-196.250	123.70
13	.000	-141.150	123.70
14	.000	-86.050	123.70
15	.000	-30.500	60.90
16	.000	-13.500	76.90
17	.000	-46.300	155.70
18	.000	-77.800	155.70
19	.000	-108.900	155.70

OINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.08207	-.07715	-.00035
3	.49395	-.15499	-.00047
4	.82007	-.18864	-.00056
5	1.02727 DMAX.	-.19653	-.00064
6	1.02115	-.29815	-.00060
7	.81670	-.28356	-.00066
8	.49191	-.23053	-.00086
9	.08110	-.11342	-.00071
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.08029	-.11383	-.00050
13	.49052	-.23146	-.00052
14	.81423	-.28473	-.00027
15	1.01593	-.29939	-.00018
16	1.01511	-.20543	.00063
17	.81393	-.19705	.00058
18	.49037	-.16164	.00053
19	.08010	-.08012	.00038
20	.00000	.00000	.00000

BER END LOADS

EMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	239.992	1.565	54.43
1	2	-239.992	-1.565	48.87
2	2	133.194	1.337	80.78

2	3	-133.194	-1.337	79.65
3	3	57.569	.973	58.78
3	4	-57.569	-.973	58.00
4	4	12.867	.419	26.71
4	5	-12.867	-.419	26.05
5	5	6.581	-.632	-102.75
5	6	-6.581	.632	-98.36
6	4	6.145	-1.598	-240.41
6	7	-6.145	1.598	-267.91
7	3	3.736	-2.175	-294.13
7	8	-3.736	2.175	-397.46
8	2	1.772	-2.102	-285.35
8	9	-1.772	2.102	-383.11
9	9	455.020	7.688	219.49
9	10	-455.020	-7.688	287.92
10	8	258.379	7.022	417.38
10	9	-258.379	-7.022	425.25
11	7	117.005	5.176	315.67
11	8	-117.005	-5.176	305.43
12	6	30.675	2.389	152.12
12	7	-30.675	-2.389	148.93
13	6	4.192	-.457	-114.66
13	15	-4.192	.457	-80.02
14	7	3.359	-1.319	-320.39
14	14	-3.359	1.319	-241.49
15	8	1.890	-1.951	-449.05
15	13	-1.890	1.951	-382.00
16	9	1.106	-1.710	-385.32
16	12	-1.106	1.710	-343.29
17	11	456.647	8.496	304.52
17	12	-456.647	-8.496	256.25
18	12	259.538	7.729	464.36
18	13	-259.538	-7.729	463.09
19	13	117.542	6.115	360.19
19	14	-117.542	-6.115	373.56
20	14	30.804	3.310	206.31
20	15	-30.804	-3.310	210.73
21	15	.882	-.153	-69.81
21	16	-.882	.153	21.09
22	14	.554	-.631	-214.69
22	17	-.554	.631	13.95
23	13	.276	-1.105	-317.58
23	18	-.276	1.105	-33.69
24	12	.338	-.851	-253.62
24	19	-.338	.851	-17.05
25	19	249.240	2.050	70.68
25	20	-249.240	-2.050	64.62
26	18	139.489	1.712	103.40
26	19	-139.489	-1.712	102.06
27	17	60.584	1.437	86.40
27	18	-60.584	-1.437	85.99
28	16	13.653	.882	55.81
28	17	-13.653	-.882	55.35

...CTIONS

JOINT	RX	RY	MZ
1	-1.565	239.992	54.43
10	-7.688	455.020	287.92

11	-8.496	456.647	304.52
20	-2.050	249.240	64.62

PLANE FRAME ANALYSIS FOR WIND STEEL DESIGN (E-W side)

Preliminary

MBER OF JOINTS = 20
BER OF MEMBERS = 28
.BER OF MATERIALS = 1
MBER OF SUPPORT JOINTS = 4
MBER OF LOADED JOINTS = 16

INT DATA

INT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	246.000	432.000	0	0	0
7	246.000	306.000	0	0	0
8	246.000	186.000	0	0	0
9	246.000	66.000	0	0	0
10	246.000	.000	1	1	1
11	318.000	.000	1	1	1
12	318.000	66.000	0	0	0
13	318.000	186.000	0	0	0
14	318.000	306.000	0	0	0
15	318.000	432.000	0	0	0
16	564.000	432.000	0	0	0
17	564.000	306.000	0	0	0
18	564.000	186.000	0	0	0
19	564.000	66.000	0	0	0
20	564.000	.000	1	1	1

MBER DATA

MBER	J1	J2	AX	IZ	E
1	1	2	7.080	82.800	29000.0
2	2	3	7.080	82.800	29000.0
3	3	4	7.080	82.800	29000.0
4	4	5	7.080	82.800	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	7.080	18.300	29000.0
10	8	9	7.080	18.300	29000.0
11	7	8	7.080	18.300	29000.0
12	6	7	7.080	18.300	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	7.080	18.300	29000.0
18	12	13	7.080	18.300	29000.0
19	13	14	7.080	18.300	29000.0
20	14	15	7.080	18.300	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	7.080	82.800	29000.0
26	18	19	7.080	82.800	29000.0
~7	17	18	7.080	82.800	29000.0
8	16	17	7.080	82.800	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	6.900	-108.900	-93.20
3	8.900	-77.800	-93.20
4	9.200	-46.300	-93.20
5	4.700	-13.500	-45.90
6	.000	-17.300	42.00
7	.000	-58.850	85.20
8	.000	-100.050	85.20
9	.000	-141.250	85.20
12	.000	-141.250	-85.20
13	.000	-100.150	-85.20
14	.000	-58.850	-85.20
15	.000	-17.300	-42.00
16	.000	-13.500	45.90
17	.000	-46.300	93.20
18	.000	-77.800	93.20
19	.000	-108.900	93.20

'OINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.15832	-.07564	-.00154
3	.84806	-.15236	-.00168
4	1.29208	-.18611	-.00113
5	1.49155 ΔMAX.	-.19420	-.00091
6	1.48891	-.26549	.00015
7	1.28969	-.25530	.00016
8	.84585	-.21112	.00019
9	.15645	-.10562	.00016
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.15602	-.09801	.00009
13	.84532	-.19766	.00019
14	1.28912	-.24191	.00018
15	1.48828	-.25277	.00016
16	1.48665	-.21372	-.00020
17	1.28759	-.20505	-.00045
18	.84387	-.16831	-.00107
19	.15495	-.08332	-.00112
20	.00000	.00000	.00000

" BER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	235.318	10.773	411.55
1	2	-235.318	-10.773	299.48
2	2	131.264	8.283	499.70

2	3	-131.264	-8.283	494.23
3	3	57.737	4.594	264.74
3	4	-57.737	-4.594	286.56
4	4	13.185	1.024	60.23
4	5	-13.185	-1.024	68.84
5	5	3.676	-.315	-114.74
5	6	-3.676	.315	37.22
6	4	5.629	-1.747	-439.99
6	7	-5.629	1.747	10.13
7	3	5.212	-4.273	-852.17
7	8	-5.212	4.273	-199.07
8	2	4.410	-4.847	-892.37
8	9	-4.410	4.847	-299.89
9	9	328.564	3.581	119.43
9	10	-328.564	-3.581	116.90
10	8	180.525	2.619	157.29
10	9	-180.525	-2.619	156.96
11	7	75.590	1.714	102.67
11	8	-75.590	-1.714	102.99
12	6	16.596	.696	43.77
12	7	-16.596	-.696	43.87
13	6	2.980	-1.019	-38.99
13	15	-2.980	1.019	-34.38
14	7	4.617	-1.604	-71.46
14	14	-4.617	1.604	-44.00
15	8	4.307	.611	24.00
15	13	-4.307	-.611	20.02
16	9	3.448	1.943	108.70
16	12	-3.448	-1.943	31.18
17	11	304.914	3.524	115.54
17	12	-304.914	-3.524	117.03
18	12	170.493	2.603	155.75
18	13	-170.493	-2.603	156.63
19	13	75.712	1.718	103.13
19	14	-75.712	-1.718	103.04
20	14	17.692	.702	44.32
20	15	-17.692	-.702	44.11
21	15	2.278	-.627	-51.74
21	16	-2.278	.627	-102.54
22	14	3.595	-2.434	-188.56
22	17	-3.595	2.434	-410.17
23	13	3.421	-4.758	-364.97
23	18	-3.421	4.758	-805.53
24	12	2.527	-4.886	-389.16
24	19	-2.527	4.886	-812.75
25	19	259.205	11.821	349.30
25	20	-259.205	-11.821	430.89
26	18	145.419	9.294	558.66
26	19	-145.419	-9.294	556.65
27	17	62.861	5.874	364.77
27	18	-62.861	-5.874	340.06
28	16	14.127	2.278	148.44
28	17	-14.127	-2.278	138.60

..CTIONS

OINT	RX	RY	MZ
1	-10.773	235.318	411.55
10	-3.581	328.564	116.90

11	-3.524	304.914	115.54
20	-11.821	259.205	430.89

PLANE FRAME ANALYSIS FOR EARTHQUAKE STEEL DESIGN (E-W side)
Preliminary

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	246.000	432.000	0	0	0
7	246.000	306.000	0	0	0
8	246.000	186.000	0	0	0
9	246.000	66.000	0	0	0
10	246.000	.000	1	1	1
11	318.000	.000	1	1	1
12	318.000	66.000	0	0	0
13	318.000	186.000	0	0	0
14	318.000	306.000	0	0	0
15	318.000	432.000	0	0	0
16	564.000	432.000	0	0	0
17	564.000	306.000	0	0	0
18	564.000	186.000	0	0	0
19	564.000	66.000	0	0	0
20	564.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	7.080	82.800	29000.0
2	2	3	7.080	82.800	29000.0
3	3	4	7.080	82.800	29000.0
4	4	5	7.080	82.800	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	7.080	18.300	29000.0
10	8	9	7.080	18.300	29000.0
11	7	8	7.080	18.300	29000.0
12	6	7	7.080	18.300	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
7	11	12	7.080	18.300	29000.0
18	12	13	7.080	18.300	29000.0
19	13	14	7.080	18.300	29000.0
20	14	15	7.080	18.300	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	7.080	82.800	29000.0
26	18	19	7.080	82.800	29000.0
7	17	18	7.080	82.800	29000.0
8	16	17	7.080	82.800	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	2.400	-108.900	-93.20
3	5.000	-77.800	-93.20
4	8.200	-46.300	-93.20
5	8.600	-13.500	-45.90
6	.000	-17.300	42.00
7	.000	-58.850	85.20
8	.000	-100.050	85.20
9	.000	-141.250	85.20
12	.000	-141.250	-85.20
13	.000	-100.150	-85.20
14	.000	-58.850	-85.20
15	.000	-17.300	-42.00
16	.000	-13.500	45.90
17	.000	-46.300	93.20
18	.000	-77.800	93.20
19	.000	-108.900	93.20

OINT DISPLACEMENTS

OINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.13142	-.07532	-.00141
3	.79447	-.15113	-.00176
4	1.32706	-.18409	-.00140
5	1.67113 ΔMAX.	-.19187	-.00128
6	1.66676	-.26741	.00020
7	1.32494	-.25691	.00020
8	.79326	-.21195	.00022
9	.13066	-.10569	.00015
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.13050	-.09794	.00009
13	.79297	-.19686	.00021
14	1.32443	-.24035	.00022
15	1.66573	-.25090	.00021
16	1.66304	-.21599	-.00056
17	1.32305	-.20702	-.00072
18	.79216	-.16951	-.00115
19	.13014	-.08364	-.00100
20	.00000	.00000	.00000

BER END LOADS

EMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	234.313	8.502	331.92
1	2	-234.313	-8.502	229.20
2	2	129.715	7.882	479.92

2	3	-129.715	-7.882	465.95
3	3	56.388	5.714	335.73
3	4	-56.388	-5.714	349.99
4	4	12.684	2.521	156.44
4	5	-12.684	-2.521	161.23
5	5	6.079	-.816	-207.13
5	6	-6.079	.816	6.39
6	4	5.005	-2.596	-599.63
6	7	-5.005	2.596	-39.01
7	3	2.833	-4.472	-894.88
7	8	-2.833	4.472	-205.35
8	2	1.780	-4.302	-802.31
8	9	-1.780	4.302	-255.97
9	9	328.799	3.007	100.46
9	10	-328.799	-3.007	97.99
10	8	181.813	2.524	151.69
10	9	-181.813	-2.524	151.15
11	7	76.915	2.052	123.05
11	8	-76.915	-2.052	123.16
12	6	17.114	1.169	73.65
12	7	-17.114	-1.169	73.66
13	6	4.909	-1.002	-38.04
13	15	-4.909	1.002	-34.10
14	7	4.129	-1.645	-72.50
14	14	-4.129	1.645	-45.96
15	8	2.361	.376	15.70
15	13	-2.361	-.376	11.37
16	9	1.297	1.433	89.57
16	12	-1.297	-1.433	13.63
17	11	304.689	2.957	96.84
17	12	-304.689	-2.957	98.30
18	12	169.242	2.508	149.96
18	13	-169.242	-2.508	151.03
19	13	74.418	2.055	123.25
19	14	-74.418	-2.055	123.36
20	14	17.187	1.173	74.00
20	15	-17.187	-1.173	73.86
21	15	3.736	-1.115	-81.76
21	16	-3.736	1.115	-192.56
22	14	3.241	-3.264	-236.60
22	17	-3.241	3.264	-566.44
23	13	1.906	-4.950	-370.85
23	18	-1.906	4.950	-846.91
24	12	.849	-4.369	-347.08
24	19	-.849	4.369	-727.73
25	19	260.199	9.734	284.80
25	20	-260.199	-9.734	357.61
26	18	146.930	8.885	530.05
26	19	-146.930	-8.885	536.13
27	17	64.180	6.978	427.34
27	18	-64.180	-6.978	410.06
28	16	14.615	3.736	238.46
28	17	-14.615	-3.736	232.29

REACTIONS

JOINT	RX	RY	MZ
1	-8.502	234.313	331.92
10	-3.007	328.799	97.99

11	-2.957	304.689	96.84
20	-9.734	260.199	357.61

PLANE FRAME ANALYSIS FOR EARTHQUAKE STEEL DESIGN (N-S side
Final Design

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	318.000	432.000	0	0	0
7	318.000	306.000	0	0	0
8	318.000	186.000	0	0	0
9	318.000	66.000	0	0	0
10	318.000	.000	1	1	1
11	744.000	.000	1	1	1
12	744.000	66.000	0	0	0
13	744.000	186.000	0	0	0
14	744.000	306.000	0	0	0
15	744.000	432.000	0	0	0
16	1062.000	432.000	0	0	0
17	1062.000	306.000	0	0	0
18	1062.000	186.000	0	0	0
19	1062.000	66.000	0	0	0
20	1062.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	13.300	53.400	29000.0
2	2	3	13.300	53.400	29000.0
3	3	4	13.300	53.400	29000.0
4	4	5	13.300	53.400	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	13.300	248.000	29000.0
10	8	9	13.300	248.000	29000.0
11	7	8	13.300	248.000	29000.0
12	6	7	13.300	248.000	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	13.300	248.000	29000.0
18	12	13	13.300	248.000	29000.0
19	13	14	13.300	248.000	29000.0
20	14	15	13.300	248.000	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	13.300	53.400	29000.0
26	18	19	13.300	53.400	29000.0
~7	17	18	13.300	53.400	29000.0
3	16	17	13.300	53.400	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	2.000	-108.900	-155.70
3	4.100	-77.800	-155.70
4	6.700	-46.300	-155.70
5	7.000	-13.500	-76.70
6	.000	-30.500	-60.90
7	.000	-86.050	-123.70
8	.000	-141.150	-123.70
9	.000	-196.250	-123.70
12	.000	-196.250	123.70
13	.000	-141.150	123.70
14	.000	-86.050	123.70
15	.000	-30.500	60.90
16	.000	-13.500	76.90
17	.000	-46.300	155.70
18	.000	-77.800	155.70
19	.000	-108.900	155.70

OINT DISPLACEMENTS

OINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.04287	-.04113	-.00044
3	.24747	-.08267	-.00057
4	.41386	-.10065	-.00060
5	.52590 ΔMAX.	-.10488	-.00065
6	.51969	-.20479	-.00050
7	.41058	-.19477	-.00056
8	.24550	-.15837	-.00073
9	.04193	-.07793	-.00059
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.04106	-.07780	-.00042
13	.24423	-.15811	-.00046
14	.40808	-.19444	-.00025
15	.51360	-.20442	-.00017
16	.51223	-.11009	.00046
17	.40767	-.10557	.00041
18	.24407	-.08655	.00033
19	.04078	-.04286	.00019
20	.00000	.00000	.00000

" BER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	240.378	1.839	70.94
1	2	-240.378	-1.839	50.44
2	2	133.500	1.553	94.85

2	3	-133.500	-1.553	91.50	
3	3	57.787	1.036	62.60	
3	4	-57.787	-1.036	61.70	
4	4	12.949	.310	20.08	
4	5	-12.949	-.310	18.94	
5	5	6.691	-.551	-95.64	
5	6	-6.691	.551	-79.49	
6	4	5.974	-1.463	-237.48	
6	7	-5.974	1.463	-227.64	
7	3	3.583	-2.087	-309.81	
7	8	-3.583	2.087	-353.71	
8	2	1.714	-2.022	-300.99	
8	9	-1.714	2.022	-341.95	
9	9	455.397	6.756	158.82	
9	10	-455.397	-6.756	287.08	
10	8	258.557	6.219	364.72	
10	9	-258.557	-6.219	381.56	
11	7	117.010	4.367	271.87	
11	8	-117.010	-4.367	252.20	
12	6	30.653	1.803	117.12	
12	7	-30.653	-1.803	110.09	
13	6	4.887	-.398	-98.53	
13	15	-4.887	.398	-71.03	
14	7	3.410	-1.155	-278.02	
14	14	-3.410	1.155	-214.04	
15	8	1.731	-1.689	-386.92	
15	13	-1.731	1.689	-332.76	
16	9	1.177	-1.433	-322.13	
16	12	-1.177	1.433	-288.30	
17	11	454.660	8.160	* 315.10	*
17	12	-454.660	-8.160	223.43	MAX MOMENT ALL @ BASEMENT COLUMN FOR INTERACTION EQUATION
18	12	258.118	7.507	452.81	
18	13	-258.118	-7.507	448.06	
19	13	116.766	6.063	351.01	
19	14	-116.766	-6.063	376.52	
20	14	30.571	3.409	210.54	
20	15	-30.571	-3.409	219.01	
21	15	1.478	-.327	-87.08	
21	16	-1.478	.327	-16.80	
22	14	.756	-1.010	-249.32	
22	17	-.756	1.010	-71.97	
23	13	.287	-1.487	-342.61	
23	18	-.287	1.487	-130.26	
24	12	.525	-1.142	-264.23	
24	19	-.525	1.142	-98.82	
25	19	250.466	3.045	105.01	
25	20	-250.466	-3.045	95.99	
26	18	140.424	2.521	152.98	
26	19	-140.424	-2.521	149.52	
27	17	61.137	2.234	135.12	
27	18	-61.137	-2.234	132.98	
28	16	13.827	1.478	93.70	
28	17	-13.827	-1.478	92.55	

CTIONS

OINT	RX	RY	MZ
1	-1.839	240.378	70.94
10	-6.756	455.397	287.08

F-

11	-8.160	454.660	315.10
20	-3.045	250.466	95.99

PLANE FRAME ANALYSIS FOR EARTHQUAKE   STEEL DESIGN (E-W side)
Final Design

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	246.000	432.000	0	0	0
7	246.000	306.000	0	0	0
8	246.000	186.000	0	0	0
9	246.000	66.000	0	0	0
10	246.000	.000	1	1	1
11	318.000	.000	1	1	1
12	318.000	66.000	0	0	0
13	318.000	186.000	0	0	0
14	318.000	306.000	0	0	0
15	318.000	432.000	0	0	0
16	564.000	432.000	0	0	0
17	564.000	306.000	0	0	0
18	564.000	186.000	0	0	0
19	564.000	66.000	0	0	0
20	564.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	13.300	248.000	29000.0
2	2	3	13.300	248.000	29000.0
3	3	4	13.300	248.000	29000.0
4	4	5	13.300	248.000	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	13.300	53.400	29000.0
10	8	9	13.300	53.400	29000.0
11	7	8	13.300	53.400	29000.0
12	6	7	13.300	53.400	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	13.300	53.400	29000.0
18	12	13	13.300	53.400	29000.0
19	13	14	13.300	53.400	29000.0
20	14	15	13.300	53.400	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	13.300	248.000	29000.0
26	18	19	13.300	248.000	29000.0
?7	17	18	13.300	248.000	29000.0
?8	16	17	13.300	248.000	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	2.400	-108.900	-93.20
3	5.000	-77.800	-93.20
4	8.200	-46.300	-93.20
5	8.600	-13.500	-45.90
6	.000	-17.300	42.00
7	.000	-58.850	85.20
8	.000	-100.050	85.20
9	.000	-141.250	85.20
12	.000	-141.250	-85.20
13	.000	-100.150	-85.20
14	.000	-58.850	-85.20
15	.000	-17.300	-42.00
16	.000	-13.500	45.90
17	.000	-46.300	93.20
18	.000	-77.800	93.20
19	.000	-108.900	93.20

JOINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.05985	-.04023	-.00107
3	.34545	-.08064	-.00134
4	.58440	-.09817	-.00102
5	.74840 ΔMAX.	-.10231	-.00089
6	.74359	-.14144	.00009
7	.58218	-.13584	.00009
8	.34424	-.11187	.00010
9	.05897	-.05564	.00007
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.05881	-.05273	.00000
13	.34395	-.10570	.00006
14	.58166	-.12882	.00007
15	.74252	-.13441	.00006
16	.73999	-.11487	-.00055
17	.58035	-.11008	-.00069
18	.34316	-.09010	-.00105
19	.05856	-.04443	-.00086
20	.00000	.00000	.00000

MEMBER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	235.082	7.357	359.47
1	2	-235.082	-7.357	126.08
2	2	129.908	7.029	438.06

2	3	-129.908	-7.029	405.37
3	3	56.329	4.862	272.12
3	4	-56.329	-4.862	311.34
4	4	12.662	1.906	112.62
4	5	-12.662	-1.906	127.59
5	5	6.693	-.838	-173.49
5	6	-6.693	.838	-32.58
6	4	5.244	-2.633	-517.16
6	7	-5.244	2.633	-130.58
7	3	2.833	-4.221	-770.69
7	8	-2.833	4.221	-267.72
8	2	2.072	-3.726	-657.35
8	9	-2.072	3.726	-259.26
9	9	325.165	3.961	132.35
9	10	-325.165	-3.961	129.06
10	8	180.740	3.176	190.93
10	9	-180.740	-3.176	190.21
11	7	77.020	2.681	160.77
11	8	-77.020	-2.681	160.93
12	6	17.162	1.606	101.20
12	7	-17.162	-1.606	101.21
13	6	5.087	-.976	-26.62
13	15	-5.087	.976	-43.67
14	7	4.169	-1.625	-46.20
14	14	-4.169	1.625	-70.80
15	8	2.339	-.551	1.06
15	13	-2.339	.551	-40.73
16	9	1.287	-.551	21.90
16	12	-1.287	.551	-61.58
17	11	308.123	3.801	125.44
17	12	-308.123	-3.801	125.43
18	12	170.260	3.107	185.61
18	13	-170.260	-3.107	187.23
19	13	74.314	2.642	158.44
19	14	-74.314	-2.642	158.65
20	14	17.134	1.569	99.02
20	15	-17.134	-1.569	98.67
21	15	3.518	-1.143	-96.99
21	16	-3.518	1.143	-184.07
22	14	3.095	-3.295	-272.07
22	17	-3.095	3.295	-538.53
23	13	1.874	-4.755	-390.14
23	18	-1.874	4.755	-779.56
24	12	.593	-3.938	-334.66
24	19	-.593	3.938	-633.96
25	19	259.630	9.081	* 206.19
25	20	-259.630	-9.081	* 393.13
26	18	146.793	8.488	497.54
26	19	-146.793	-8.488	520.97
27	17	64.238	6.614	418.43
27	18	-64.238	-6.614	375.22
28	16	14.643	3.518	229.97
28	17	-14.643	-3.518	213.31

MAX MOMENT MEt @ BASEMENT COLU FOR INTERACTION EQUATION

..CTIONS

JOINT	RX	RY	MZ
1	-7.357	235.082	359.47
10	-3.961	325.165	129.06

11	-3.801	308.123	125.44
20	-9.081	259.630	393.13

F

PLANE FRAME ANALYSIS to obtain Mn(t) for Interaction Equation

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	246.000	432.000	0	0	0
7	246.000	306.000	0	0	0
8	246.000	186.000	0	0	0
9	246.000	66.000	0	0	0
10	246.000	.000	1	1	1
11	318.000	.000	1	1	1
12	318.000	66.000	0	0	0
13	318.000	186.000	0	0	0
14	318.000	306.000	0	0	0
15	318.000	432.000	0	0	0
16	564.000	432.000	0	0	0
17	564.000	306.000	0	0	0
18	564.000	186.000	0	0	0
19	564.000	66.000	0	0	0
20	564.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	13.300	248.000	29000.0
2	2	3	13.300	248.000	29000.0
3	3	4	13.300	248.000	29000.0
4	4	5	13.300	248.000	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	13.300	53.400	29000.0
10	8	9	13.300	53.400	29000.0
11	7	8	13.300	53.400	29000.0
12	6	7	13.300	53.400	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	13.300	53.400	29000.0
18	12	13	13.300	53.400	29000.0
19	13	14	13.300	53.400	29000.0
20	14	15	13.300	53.400	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	13.300	248.000	29000.0
26	18	19	13.300	248.000	29000.0
27	17	18	13.300	248.000	29000.0
28	16	17	13.300	248.000	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	.000	-108.900	-93.20
3	.000	-77.800	-93.20
4	.000	-46.300	-93.20
5	.000	-13.500	-45.90
6	.000	-17.300	42.00
7	.000	-58.850	85.20
8	.000	-100.050	85.20
9	.000	-141.250	85.20
12	.000	-141.250	-85.20
13	.000	-100.150	-85.20
14	.000	-58.850	-85.20
15	.000	-17.300	-42.00
16	.000	-13.500	45.90
17	.000	-46.300	93.20
18	.000	-77.800	93.20
19	.000	-108.900	93.20

JOINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.00011	-.04234	-.00010
3	-.00008	-.08539	-.00014
4	.00001	-.10415	-.00016
5	.00067	-.10862	-.00016
6	.00007	-.13789	.00002
7	-.00001	-.13229	.00001
8	-.00002	-.10875	.00002
9	.00001	-.05417	.00003
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	-.00001	-.05418	-.00003
13	.00000	-.10878	-.00002
14	-.00002	-.13231	-.00001
15	-.00011	-.13791	-.00002
16	-.00071	-.10862	.00016
17	-.00004	-.10416	.00016
18	.00007	-.08539	.00014
19	-.00011	-.04234	.00010
20	.00000	.00000	.00000

MEMBER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	247.408	-.969	-20.97
1	2	-247.408	.969	-43.01
2	2	138.384	-.738	-41.85

2	3	-138.384	.738	-46.76
3	3	60.309	-.891	-52.57
3	4	-60.309	.891	-54.33
4	4	13.662	-.837	-52.47
4	5	-13.662	.837	-53.04
5	5	.837	.162	7.14
5	6	-.837	-.162	32.80
6	4	.053	.346	13.60
6	7	-.053	-.346	71.60
7	3	-.152	.275	6.13
7	8	.152	-.275	61.59
8	2	.231	.124	-8.33
8	9	-.231	-.124	38.87
9	9	316.561	.074	3.23
9	10	-316.561	-.074	1.63
10	8	175.432	.033	1.73
10	9	-175.432	-.033	2.18
11	7	75.650	.017	.93
11	8	-75.650	-.017	1.12
12	6	17.140	.016	1.07
12	7	-17.140	-.016	.90
13	6	.822	.002	8.13
13	15	-.822	-.002	-7.95
14	7	.052	.006	11.77
14	14	-.052	-.006	-11.32
15	8	-.168	.008	20.76
15	13	.168	-.008	-20.20
16	9	.190	.003	40.92
16	12	-.190	-.003	-40.71
17	11	316.617	-.074	-1.64
17	12	-316.617	.074	-3.25
18	12	175.495	-.033	-2.21
18	13	-175.495	.033	-1.78
19	13	75.630	-.018	-1.17
19	14	-75.630	.018	-.98
20	14	17.135	-.016	-.95
20	15	-17.135	.016	-1.11
21	15	.838	-.163	-32.94
21	16	-.838	.163	-7.18
22	14	.054	-.348	-71.95
22	17	-.054	.348	-13.70
23	13	-.153	-.278	-62.04
23	18	.153	.278	-6.24
24	12	.231	-.125	-39.03
24	19	-.231	.125	8.28
25	19	247.414	.970	43.04
25	20	-247.414	-.970	20.97
26	18	138.389	.739	46.81
26	19	-138.389	-.739	41.88
27	17	60.311	.892	54.38
27	18	-60.311	-.892	52.63
28	16	13.663	.838	53.08
28	17	-13.663	-.838	52.52

MAX. M at moment @ BASEMENT COLUMNS FOR INTERACTION EQUATION

CTIONS

OINT	RX	RY	MZ
1	.969	247.408	-20.97
10	-.074	316.561	1.63

F-

11	.074	316.617	-1.64
20	-.970	247.414	20.97

PLANE FRAME ANALYSIS FOR EARTHQUAKE STEEL DESIGN (E-W side)
ACCEPTABLE DESIGN AFTER INTERACTION EQUATION

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	66.000	0	0	0
3	.000	186.000	0	0	0
4	.000	306.000	0	0	0
5	.000	432.000	0	0	0
6	246.000	432.000	0	0	0
7	246.000	306.000	0	0	0
8	246.000	186.000	0	0	0
9	246.000	66.000	0	0	0
10	246.000	.000	1	1	1
11	318.000	.000	1	1	1
12	318.000	66.000	0	0	0
13	318.000	186.000	0	0	0
14	318.000	306.000	0	0	0
15	318.000	432.000	0	0	0
16	564.000	432.000	0	0	0
17	564.000	306.000	0	0	0
18	564.000	186.000	0	0	0
19	564.000	66.000	0	0	0
20	564.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	20.000	394.000	29000.0
2	2	3	20.000	394.000	29000.0
3	3	4	20.000	394.000	29000.0
4	4	5	20.000	394.000	29000.0
5	5	6	11.800	612.000	29000.0
6	4	7	20.000	1480.000	29000.0
7	3	8	20.000	1480.000	29000.0
8	2	9	20.000	1480.000	29000.0
9	9	10	20.000	134.000	29000.0
10	8	9	20.000	134.000	29000.0
11	7	8	20.000	134.000	29000.0
12	6	7	20.000	134.000	29000.0
13	6	15	11.800	612.000	29000.0
14	7	14	20.000	1480.000	29000.0
15	8	13	20.000	1480.000	29000.0
16	9	12	20.000	1480.000	29000.0
17	11	12	20.000	134.000	29000.0
18	12	13	20.000	134.000	29000.0
19	13	14	20.000	134.000	29000.0
20	14	15	20.000	134.000	29000.0
21	15	16	11.800	612.000	29000.0
22	14	17	20.000	1480.000	29000.0

23	13	18	20.000	1480.000	29000.0
24	12	19	20.000	1480.000	29000.0
25	19	20	20.000	394.000	29000.0
26	18	19	20.000	394.000	29000.0
27	17	18	20.000	394.000	29000.0
28	16	17	20.000	394.000	29000.0

JOINT LOADS

JOINT	WX	WY	MZ
2	2.400	-108.900	-93.20
3	5.000	-77.800	-93.20
4	8.200	-46.300	-93.20
5	8.600	-13.500	-45.90
6	.000	-17.300	42.00
7	.000	-58.850	85.20
8	.000	-100.050	85.20
9	.000	-141.250	85.20
12	.000	-141.250	-85.20
13	.000	-100.150	-85.20
14	.000	-58.850	-85.20
15	.000	-17.300	-42.00
16	.000	-13.500	45.90
17	.000	-46.300	93.20
18	.000	-77.800	93.20
19	.000	-108.900	93.20

JOINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.03714	-.02691	-.00076
3	.21178	-.05395	-.00096
4	.35952	-.06565	-.00071
5	.46208	-.06840	-.00060
6	.45687	-.09076	-.00004
7	.35706	-.08714	-.00004
8	.21046	-.07169	-.00004
9	.03615	-.03568	-.00002
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.03600	-.03641	-.00009
13	.21017	-.07308	-.00009
14	.35654	-.08897	-.00008
15	.45582	-.09281	-.00009
16	.45369	-.07589	-.00038
17	.35544	-.07273	-.00050
18	.20948	-.05951	-.00077
19	.03588	-.02935	-.00061
20	.00000	.00000	.00000

$\Delta MAX \leq 0.86$ OK!

MEMBER END LOADS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	236.516	5.682	319.84
1	2	-236.516	-5.682	55.14
2	2	130.671	5.624	356.51

F-31

2	3	-130.671	-5.624	318.33
3	3	56.537	3.749	200.65
3	4	-56.537	-3.749	249.25
4	4	12.689	1.353	75.75
4	5	-12.689	-1.353	94.78
5	5	7.247	-.811	-140.68
5	6	-7.247	.811	-58.89
6	4	5.804	-2.451	-418.21
6	7	-5.804	2.451	-184.86
7	3	3.126	-3.666	-612.17
7	8	-3.126	3.666	-289.64
8	2	2.342	-3.055	-504.85
8	9	-2.342	3.055	-246.73
9	9	313.553	5.731	187.66
9	10	-313.553	-5.731	190.57
10	8	174.030	4.598	275.37
10	9	-174.030	-4.598	276.40
11	7	74.670	3.824	229.42
11	8	-74.670	-3.824	229.44
12	6	16.704	2.211	139.38
12	7	-16.704	-2.211	139.19
13	6	5.035	-1.407	-38.49
13	15	-5.035	1.407	-62.79
14	7	4.190	-3.336	-98.54
14	14	-4.190	3.336	-141.67
15	8	2.351	-4.355	-129.97
15	13	-2.351	4.355	-183.61
16	9	1.209	-4.782	-132.13
16	12	-1.209	4.782	-212.18
17	11	319.995	5.347	181.86
17	12	-319.995	-5.347	171.04
18	12	177.213	4.413	264.56
18	13	-177.213	-4.413	264.96
19	13	76.834	3.686	220.90
19	14	-76.834	-3.686	221.45
20	14	17.646	2.073	130.89
20	15	-17.646	-2.073	130.27
21	15	2.963	-1.061	-109.48
21	16	-2.963	1.061	-151.42
22	14	2.578	-2.999	-295.87
22	17	-2.578	2.999	-441.77
23	13	1.625	-4.126	-387.46
23	18	-1.625	4.126	-627.52
24	12	.275	-3.251	-308.62
24	19	-.275	3.251	-491.01
25	19	257.936	7.440	139.14
25	20	-257.936	-7.440	351.92
26	18	145.785	7.165	414.78
26	19	-145.785	-7.165	445.07
27	17	63.859	5.541	358.95
27	18	-63.859	-5.541	305.94
28	16	14.561	2.963	197.32
28	17	-14.561	-2.963	176.02

CTIONS

INT	RX	RY	MZ
1	-5.682	236.516	319.84
10	-5.731	313.553	190.57

11	-5.347	319.995	181.86
20	-7.440	257.936	351.92

Appendix G

Computer Analysis for Reinforced Concrete Frame

PLANE FRAME ANALYSIS FOR WIND REINF CONC DESIGN (E-W side)

NUMBER OF JOINTS = 20
UMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	90.000	0	0	0
3	.000	210.000	0	0	0
4	.000	330.000	0	0	0
5	.000	456.000	0	0	0
6	222.000	456.000	0	0	0
7	222.000	330.000	0	0	0
8	222.000	210.000	0	0	0
9	222.000	90.000	0	0	0
10	222.000	.000	1	1	1
11	294.000	.000	1	1	1
12	294.000	90.000	0	0	0
13	294.000	210.000	0	0	0
14	294.000	330.000	0	0	0
15	294.000	456.000	0	0	0
16	516.000	456.000	0	0	0
17	516.000	330.000	0	0	0
18	516.000	210.000	0	0	0
19	516.000	90.000	0	0	0
20	516.000	.000	1	1	1

EMBER DATA

EMBER	J1	J2	AX	IZ	E
1	1	2	64.000	3840.000	3600.0
2	2	3	64.000	5120.000	3600.0
3	3	4	64.000	5120.000	3600.0
4	4	5	64.000	5376.000	3600.0
5	5	6	387.000	3460.000	3600.0
6	4	7	387.000	3460.000	3600.0
7	3	8	387.000	3460.000	3600.0
8	2	9	387.000	3460.000	3600.0
9	9	10	64.000	3840.000	3600.0
10	8	9	64.000	5120.000	3600.0
11	7	8	64.000	5120.000	3600.0
12	6	7	64.000	5376.000	3600.0
13	6	15	387.000	1845.300	3600.0
14	7	14	387.000	1845.300	3600.0
15	8	13	387.000	1845.300	3600.0
16	9	12	387.000	1845.300	3600.0
17	11	12	64.000	3840.000	3600.0
18	12	13	64.000	5120.000	3600.0
19	13	14	64.000	5120.000	3600.0
20	14	15	64.000	5376.000	3600.0
21	15	16	387.000	3460.000	3600.0
22	14	17	387.000	3460.000	3600.0

23	13	18	387.000	3460.000	3600.0
24	12	19	387.000	3460.000	3600.0
25	19	20	64.000	3840.000	3600.0
26	18	19	64.000	5120.000	3600.0
27	17	18	64.000	5120.000	3600.0
28	16	17	64.000	5376.000	3600.0

JOINT LOADS

JOINT	WX	WY	MZ
2	6.900	-78.000	-5.50
3	8.900	-55.200	-5.50
4	9.200	-34.400	-5.50
5	4.700	-13.600	-5.50
6	.000	-18.400	-4.90
7	.000	-47.100	4.62
8	.000	-74.700	4.62
9	.000	-102.000	4.62
12	.000	-102.000	4.62
13	.000	-74.700	-4.62
14	.000	-47.100	-4.62
15	.000	-18.000	4.90
16	.000	-13.600	5.50
17	.000	-34.400	5.50
18	.000	-55.200	5.50
19	.000	-85.200	5.50

OINT DISPLACEMENTS

OINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.07745	-.06797	-.00113
3	.23961	-.11941	-.00109
4	.36268	-.14342	-.00069
5	.43241	-.15056	-.00040
6	.43168	-.19849	-.00039
7	.36152	-.18841	-.00061
8	.23845	-.15581	-.00091
9	.07673	-.08780	-.00087
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.07657	-.09994	-.00084
13	.23822	-.17624	-.00086
14	.36131	-.21060	-.00056
15	.43149	-.21992	-.00027
16	.43122	-.17060	-.00025
17	.36111	-.16236	-.00064
18	.23798	-.13531	-.00103
19	.07629	-.07773	-.00108
20	.00000	.00000	.00000

EMBER END LOADS

EMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	174.001	6.067	446.36
1	2	-174.001	-6.067	99.63
2	2	98.761	3.683	215.70

2	3	-98.761	-3.683	226.21
3	3	46.103	2.050	60.86
3	4	-46.103	-2.050	185.11
4	4	13.059	.152	-35.59
4	5	-13.059	-.152	54.74
5	5	4.549	-.541	-60.24
5	6	-4.549	.541	-59.87
6	4	7.303	-1.356	-155.02
6	7	-7.303	1.356	-145.99
7	3	7.267	-2.543	-292.56
7	8	-7.267	2.543	-271.88
8	2	4.516	-2.760	-320.83
8	9	-4.516	2.760	-291.78
9	9	224.770	8.554	251.34
9	10	-224.770	-8.554	518.53
10	8	130.577	7.030	415.61
10	9	-130.577	-7.030	428.03
11	7	62.597	4.082	291.13
11	8	-62.597	-4.082	198.77
12	6	18.432	.816	84.71
12	7	-18.432	-.816	18.09
13	6	3.732	-.509	-29.74
13	15	-3.732	.509	-6.89
14	7	4.035	-4.291	-158.61
14	14	-4.035	4.291	-150.37
15	8	4.318	-9.262	-337.88
15	13	-4.318	9.262	-329.01
16	9	2.992	-10.566	-382.97
16	12	-2.992	10.566	-377.80
17	11	255.845	8.806	525.56
17	12	-255.845	-8.806	266.99
18	12	146.500	7.606	459.50
18	13	-146.500	-7.606	453.25
19	13	65.964	4.796	242.13
19	14	-65.964	-4.796	333.42
20	14	17.053	2.050	83.65
20	15	-17.053	-2.050	174.60
21	15	1.683	-1.456	-162.82
21	16	-1.683	1.456	-160.33
22	14	1.291	-2.480	-271.32
22	17	-1.291	2.480	-279.32
23	13	1.508	-3.426	-370.99
23	18	-1.508	3.426	-389.70
24	12	1.793	-3.222	-344.08
24	19	-1.793	3.222	-371.12
25	19	198.984	6.274	116.02
25	20	-198.984	-6.274	448.61
26	18	110.562	4.481	277.14
26	19	-110.562	-4.481	260.59
27	17	51.936	2.973	238.65
27	18	-51.936	-2.973	118.05
28	16	15.056	1.683	165.83
28	17	-15.056	-1.683	46.17

ACTIONS

OINT	RX	RY	MZ
1	-6.067	174.001	446.36
10	-8.554	224.770	518.53

11	-8.806	255.845	525.56
20	-6.274	198.984	448.61

PLANE FRAME ANALYSIS FOR WIND REINF CONC DESIGN (E-W side)

NUMBER OF JOINTS = 20
NUMBER OF MEMBERS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

JOINT DATA

JOINT	X	Y	RESTRAINTS		
1	.000	.000	1	1	1
2	.000	90.000	0	0	0
3	.000	210.000	0	0	0
4	.000	330.000	0	0	0
5	.000	456.000	0	0	0
6	222.000	456.000	0	0	0
7	222.000	330.000	0	0	0
8	222.000	210.000	0	0	0
9	222.000	90.000	0	0	0
10	222.000	.000	1	1	1
11	294.000	.000	1	1	1
12	294.000	90.000	0	0	0
13	294.000	210.000	0	0	0
14	294.000	330.000	0	0	0
15	294.000	456.000	0	0	0
16	516.000	456.000	0	0	0
17	516.000	330.000	0	0	0
18	516.000	210.000	0	0	0
19	516.000	90.000	0	0	0
20	516.000	.000	1	1	1

MEMBER DATA

MEMBER	J1	J2	AX	IZ	E
1	1	2	64.000	3840.000	3600.0
2	2	3	64.000	5120.000	3600.0
3	3	4	64.000	5120.000	3600.0
4	4	5	64.000	5376.000	3600.0
5	5	6	387.000	3460.000	3600.0
6	4	7	387.000	3460.000	3600.0
7	3	8	387.000	3460.000	3600.0
8	2	9	387.000	3460.000	3600.0
9	9	10	64.000	3840.000	3600.0
10	8	9	64.000	5120.000	3600.0
11	7	8	64.000	5120.000	3600.0
12	6	7	64.000	5376.000	3600.0
13	6	15	387.000	1845.300	3600.0
14	7	14	387.000	1845.300	3600.0
15	8	13	387.000	1845.300	3600.0
16	9	12	387.000	1845.300	3600.0
17	11	12	64.000	3840.000	3600.0
18	12	13	64.000	5120.000	3600.0
19	13	14	64.000	5120.000	3600.0
20	14	15	64.000	5376.000	3600.0
21	15	16	387.000	3460.000	3600.0
22	14	17	387.000	3460.000	3600.0

23	13	18	387.000	3460.000	3600.0
24	12	19	387.000	3460.000	3600.0
25	19	20	64.000	3840.000	3600.0
26	18	19	64.000	5120.000	3600.0
27	17	18	64.000	5120.000	3600.0
28	16	17	64.000	5376.000	3600.0

JOINT LOADS

JOINT	WX	WY	MZ
2	4.400	-78.000	-5.50
3	9.200	-55.200	-5.50
4	15.200	-34.400	-5.50
5	16.000	-13.600	-5.50
6	.000	-18.400	-4.90
7	.000	-47.100	4.62
8	.000	-74.700	4.62
9	.000	-102.000	4.62
12	.000	-102.000	4.62
13	.000	-74.700	-4.62
14	.000	-47.100	-4.62
15	.000	-18.000	4.90
16	.000	-13.600	5.50
17	.000	-34.400	5.50
18	.000	-55.200	5.50
19	.000	-85.200	5.50

OINT DISPLACEMENTS

JOINT	X-DISP	Y-DISP	Z-ROT
1	.00000	.00000	.00000
2	.12500	-.06433	-.00190
3	.41970	-.11203	-.00212
4	.68400	-.13380	-.00160
5	.86582	-.14002	-.00097
6	.86364	-.18513	-.00090
7	.68204	-.17531	-.00139
8	.41849	-.14493	-.00176
9	.12464	-.08204	-.00149
10	.00000	.00000	.00000
11	.00000	.00000	.00000
12	.12454	-.10571	-.00146
13	.41825	-.18715	-.00171
14	.68168	-.22374	-.00135
15	.86316	-.23333	-.00078
16	.86254	-.18109	-.00081
17	.68131	-.17194	-.00154
18	.41802	-.14267	-.00204
19	.12428	-.08136	-.00185
20	.00000	.00000	.00000

EMBER END LOADS

EMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	164.682	9.008	696.92
1	2	-164.682	-9.008	113.85
2	2	91.577	6.888	446.86

2	3	-91.577	-6.888	379.65
3	3	41.810	5.288	237.83
3	4	-41.810	-5.288	396.68
4	4	11.379	2.338	50.08
4	5	-11.379	-2.338	244.53
5	5	13.664	-2.221	-250.03
5	6	-13.664	2.221	-243.02
6	4	12.250	-3.969	-452.25
6	7	-12.250	3.969	-428.90
7	3	7.600	-5.433	-622.98
7	8	-7.600	5.433	-583.24
8	2	2.279	-4.894	-566.20
8	9	-2.279	4.894	-520.30
9	9	210.017	13.115	361.48
9	10	-210.017	-13.115	818.87
10	8	120.746	12.641	716.39
10	9	-120.746	-12.641	800.48
11	7	58.327	9.511	627.63
11	8	-58.327	-9.511	513.65
12	6	17.964	4.291	345.21
12	7	-17.964	-4.291	195.48
13	6	9.366	-2.657	-107.09
13	15	-9.366	2.657	-84.19
14	7	7.034	-10.706	-389.61
14	14	-7.034	10.706	-381.24
15	8	4.470	-17.714	-642.18
15	13	-4.470	17.714	-633.22
16	9	1.805	-17.624	-637.04
16	12	-1.805	17.624	-631.87
17	11	270.625	13.381	826.54
17	12	-270.625	-13.381	377.76
18	12	156.361	13.211	831.57
18	13	-156.361	-13.211	753.77
19	13	70.254	10.214	556.37
19	14	-70.254	-10.214	669.37
20	14	17.529	5.516	260.56
20	15	-17.529	-5.516	434.48
21	15	3.855	-3.128	-345.38
21	16	-3.855	3.128	-348.92
22	14	2.335	-5.082	-553.30
22	17	-2.335	5.082	-574.83
23	13	1.472	-6.307	-681.54
23	18	-1.472	6.307	-718.58
24	12	1.635	-5.359	-572.84
24	19	-1.635	5.359	-616.95
25	19	208.275	9.297	133.58
25	20	-208.275	-9.297	703.11
26	18	117.716	7.661	430.45
26	19	-117.716	-7.661	488.88
27	17	56.209	6.189	449.07
27	18	-56.209	-6.189	293.62
28	16	16.728	3.855	354.42
28	17	-16.728	-3.855	131.25

EACTIONS

JOINT	RX	RY	MZ
1	-9.008	164.682	696.92
10	-13.115	210.017	818.87

11	-13.381	270.625	826.54
20	-9.297	208.275	703.11

Appendix H

Construction Schedule

Critical Task Project Schedule
Redesign of Institute Hall : Structural Steel Design
Lukauskis, Fábrega, and Pissimissis

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
1	**Start of Project**	51.59d	3/28/94	6/7/94		
2	Excavation for Footings	1.98d	3/28/94	3/29/94		

ID	*Successor Name*	*Type*	*Lag*
3	*Forming for Footings*	*FS*	*0d*
4	*Reinforcing for Footings*	*FS*	*0d*
8	*Disposal for Footings*	*FS*	*0d*

3	Forming for Footings	3.68d	3/29/94	4/4/94	2	

ID	*Successor Name*	*Type*	*Lag*
5	*Pouring of Concrete for Footings*	*FS*	*0d*

5	Pouring of Concrete for Footings	1.41d	4/4/94	4/6/94	4,3	

ID	*Successor Name*	*Type*	*Lag*
6	*Curing of Concrete for Footings*	*FS*	*0d*

6	Curing of Concrete for Footings	0.17d	4/6/94	4/6/94	5	

ID	*Successor Name*	*Type*	*Lag*
7	*Backfill for Footings*	*FS*	*6d*
9	*Erection of 8x24 Basement Columns*	*FS*	*6d*
10	*Erection of 10x68 Basement Columns*	*FS*	*6d*

10	Erection of 10x68 Basement Columns	0.15d	4/14/94	4/14/94	6FS+6d	

ID	*Successor Name*	*Type*	*Lag*
18	*Fireproof Basement Columns*	*FS*	*0d*
23	*Erection of First Floor 21x68 Girders*	*FS*	*0d*
42	*Erection of First Floor 10x68 Columns*	*FS*	*0d*

42	Erection of First Floor 10x68 Columns	0.2d	4/14/94	4/14/94	10	

ID	*Successor Name*	*Type*	*Lag*
43	*Fireproof First Floor Columns*	*FS*	*0d*
45	*Erection of Second Floor 21x68 Girders*	*FS*	*0d*
64	*Erection of Second Floor 10x68 Columns*	*FS*	*0d*

64	Erection of Second Floor 10x68 Columns	0.2d	4/14/94	4/14/94	42	

ID	*Successor Name*	*Type*	*Lag*
65	*Fireproof Second Floor Columns*	*FS*	*0d*
67	*Erection of Third Floor 21x68 Girders*	*FS*	*0d*
86	*Erection of Third Floor 10x68 Columns*	*FS*	*0d*

67	Erection of Third Floor 21x68 Girders	0.58d	4/14/94	4/15/94	64,63	

ID	*Successor Name*	*Type*	*Lag*
68	*Erection of Third Floor 6x9 Beams*	*FS*	*0d*
69	*Erection of Third Floor 10x22 Beams*	*FS*	*0d*
70	*Erection of Third Floor 12x14 Beams*	*FS*	*0d*
80	*Fireproof Third Floor Beams & Girders*	*FS*	*0d*

69	Erection of Third Floor 10x22 Beams	0.45d	4/15/94	4/15/94	67	

ID	*Successor Name*	*Type*	*Lag*
71	*Erection of Third Floor Metal Deck*	*FS*	*0d*
80	*Fireproof Third Floor Beams & Girders*	*FS*	*0d*

<table>
<tr><th>ID</th><th>Task Name</th><th>Duration</th><th>Start</th><th>Finish</th><th>Predecessors</th><th>Resource Names</th></tr>
<tr><td>71</td><td>Erection of Third Floor Metal Deck</td><td>1.31d</td><td>4/15/94</td><td>4/19/94</td><td>68,69,70</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>72</td><td>Welding of Third Floor Metal Deck</td><td>FS</td><td>0d</td></tr><tr><td>81</td><td>Erection of Second Floor Ceiling</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>72</td><td>Welding of Third Floor Metal Deck</td><td>10.3d</td><td>4/19/94</td><td>5/3/94</td><td>71</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>73</td><td>Edge Forming for Third Floor Slab</td><td>FS</td><td>0d</td></tr><tr><td>74</td><td>Reinforcing for Third Floor Slab Halls</td><td>FS</td><td>0d</td></tr><tr><td>75</td><td>Reinforcing for Third Floor Slab Rooms</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>75</td><td>Reinforcing for Third Floor Slab Rooms</td><td>1.52d</td><td>5/3/94</td><td>5/4/94</td><td>72</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>76</td><td>Pouring Third Floor Slab</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>76</td><td>Pouring Third Floor Slab</td><td>0.43d</td><td>5/4/94</td><td>5/5/94</td><td>73,75,74</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>77</td><td>Curing Third Floor Slab</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>77</td><td>Curing Third Floor Slab</td><td>0.51d</td><td>5/5/94</td><td>5/5/94</td><td>76</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>78</td><td>Float Finish of Third Floor Slab</td><td>FS</td><td>0d</td></tr><tr><td>79</td><td>Erection of Third Floor Scaffolding</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>78</td><td>Float Finish of Third Floor Slab</td><td>6.39d</td><td>5/5/94</td><td>5/16/94</td><td>77</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>82</td><td>Erection of Third Floor Exterior Wall</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>82</td><td>Erection of Third Floor Exterior Wall</td><td>9.67d</td><td>5/16/94</td><td>5/27/94</td><td>79,78</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>84</td><td>Erection of Third Floor Inner Wall</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>84</td><td>Erection of Third Floor Inner Wall</td><td>6.64d</td><td>5/27/94</td><td>6/7/94</td><td>82,83</td><td></td></tr>
<tr><td></td><td colspan="6"><table><tr><th>ID</th><th>Successor Name</th><th>Type</th><th>Lag</th></tr><tr><td>104</td><td>Completion of Project</td><td>FS</td><td>0d</td></tr></table></td></tr>
<tr><td>104</td><td>Completion of Project</td><td>0d</td><td>6/7/94</td><td>6/7/94</td><td>8,19,22,37,44,40,59,66,</td><td></td></tr>
</table>

Critical Tasks Project Schedule
Redesign of Institute Hall: Reinforced Concrete Design
Lukauskis, Fábrega, and Pissimissis

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
1	**Start of Project**	**143.93d**	**3/28/94**	**10/13/94**		
3	Forming for Footings	9.51d	3/28/94	4/8/94		

ID	*Successor Name*	*Type*	*Lag*
5	*Pouring of Concrete for Footings*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
5	Pouring of Concrete for Footings	1.34d	4/8/94	4/11/94	4,3	

ID	*Successor Name*	*Type*	*Lag*
6	*Curing of Concrete for Footings*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
6	Curing of Concrete for Footings	0.15d	4/11/94	4/11/94	5	

ID	*Successor Name*	*Type*	*Lag*
7	*Backfill for Footings*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
7	Backfill for Footings	0.45d	4/12/94	4/12/94	6	

ID	*Successor Name*	*Type*	*Lag*
16	*Gravel Base Course*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
16	Gravel Base Course	0.65d	4/12/94	4/13/94	7	

ID	*Successor Name*	*Type*	*Lag*
17	*Vapor Barrier*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
17	Vapor Barrier	1.38d	4/13/94	4/14/94	16	

ID	*Successor Name*	*Type*	*Lag*
18	*Edge Forming for Basement Slab*	*FS*	*0h*
19	*Reinforcing for Basement Slab*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
19	Reinforcing for Basement Slab	1.82d	4/14/94	4/18/94	17	

ID	*Successor Name*	*Type*	*Lag*
20	*Pouring for Basement Slab*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
20	Pouring for Basement Slab	0.63d	4/18/94	4/18/94	19,18	

ID	*Successor Name*	*Type*	*Lag*
21	*Curing for Basement Slab*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
21	Curing for Basement Slab	0.51d	4/18/94	4/19/94	20	

ID	*Successor Name*	*Type*	*Lag*
22	*Float Finishing of Basement Slab*	*FS*	*0d*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
22	Float Finishing of Basement Slab	6.39d	4/19/94	4/27/94	21	

ID	*Successor Name*	*Type*	*Lag*
23	*Shoring for First Floor Slab*	*FS*	*0d*
37	*Erection of Exterior Basement Wall*	*FS*	*0d*
38	*Insertion of Insulation on Basement Wall*	*FS*	*0d*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
23	Shoring for First Floor Slab	2.36d	4/27/94	5/2/94	22	

ID	*Successor Name*	*Type*	*Lag*
24	*Plate Forming for First Floor Slab*	*FS*	*0h*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
24	Plate Forming for First Floor Slab	8.69d	5/2/94	5/12/94	23	

ID	*Successor Name*	*Type*	*Lag*
25	*Edge Forming for First Floor Slab*	*FS*	*0h*
26	*Reinforcing for First Floor Slab*	*FS*	*0d*

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
26	Reinforcing for First Floor Slab	1.53d	5/12/94	5/16/94	24	
27	Pouring for First Floor Slab	0.84d	5/16/94	5/17/94	26,25	
28	Curing for First Floor Slab	0.51d	5/17/94	5/17/94	27	
40	Shoring for Second Floor Slab	2.36d	5/25/94	5/30/94	28FS+6d	
41	Plate Forming for Second Floor Slab	8.69d	5/30/94	6/9/94	40	
43	Reinforcing for Second Floor Slab	1.53d	6/9/94	6/13/94	41	
44	Pouring for Second Floor Slab	0.84d	6/13/94	6/14/94	43,42	
45	Curing for Second Floor Slab	0.51d	6/14/94	6/14/94	44	
46	Float Finishing of Second Floor Slab	6.39d	6/14/94	6/23/94	45	
58	Shoring for Third Floor Slab	3.47d	6/23/94	6/28/94	46	
59	Plate Forming for Third Floor Slab	8.69d	6/28/94	7/11/94	58	

Successors of 26 Reinforcing for First Floor Slab:

ID	Successor Name	Type	Lag
27	Pouring for First Floor Slab	FS	0d

Successors of 27 Pouring for First Floor Slab:

ID	Successor Name	Type	Lag
28	Curing for First Floor Slab	FS	0d

Successors of 28 Curing for First Floor Slab:

ID	Successor Name	Type	Lag
29	Float Finishing of First Floor Slab	FS	0d
40	Shoring for Second Floor Slab	FS	6d

Successors of 40 Shoring for Second Floor Slab:

ID	Successor Name	Type	Lag
41	Plate Forming for Second Floor Slab	FS	0d

Successors of 41 Plate Forming for Second Floor Slab:

ID	Successor Name	Type	Lag
42	Edge Forming for Second Floor Slab	FS	0d
43	Reinforcing for Second Floor Slab	FS	0d

Successors of 43 Reinforcing for Second Floor Slab:

ID	Successor Name	Type	Lag
44	Pouring for Second Floor Slab	FS	0d

Successors of 44 Pouring for Second Floor Slab:

ID	Successor Name	Type	Lag
45	Curing for Second Floor Slab	FS	0d

Successors of 45 Curing for Second Floor Slab:

ID	Successor Name	Type	Lag
46	Float Finishing of Second Floor Slab	FS	0d
47	Splicing for For Second Floor Columns	FS	6d

Successors of 46 Float Finishing of Second Floor Slab:

ID	Successor Name	Type	Lag
58	Shoring for Third Floor Slab	FS	0d
65	Float Finishing of Basement Ceiling	FS	0d
66	Breaking Patches and Voids Basement	FS	0d
74	Erection of Scaffolding for Second Floor Wall	FS	0d
86	Float Finishing of First Floor Ceiling	FS	0d
90	Erection of Scaffolding for Third Floor Wall	FS	0d

Successors of 58 Shoring for Third Floor Slab:

ID	Successor Name	Type	Lag
59	Plate Forming for Third Floor Slab	FS	0d

Successors of 59 Plate Forming for Third Floor Slab:

ID	Successor Name	Type	Lag
60	Edge Forming for Third Floor Slab	FS	0d
61	Reinforcing for Third Floor Slab	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
61	Reinforcing for Third Floor Slab	1.53d	7/11/94	7/12/94	59	

ID	Successor Name	Type	Lag
62	Pouring for Third Floor Slab	FS	0d

62	Pouring for Third Floor Slab	0.84d	7/12/94	7/13/94	61,60	

ID	Successor Name	Type	Lag
63	Curing for Third Floor Slab	FS	0d

63	Curing for Third Floor Slab	0.51d	7/13/94	7/14/94	62	

ID	Successor Name	Type	Lag
64	Float Finishing of Third Floor Slab	FS	0d
67	Splicing for For Third Floor Columns	FS	6d

64	Float Finishing of Third Floor Slab	6.39d	7/14/94	7/22/94	63	

ID	Successor Name	Type	Lag
67	Splicing for For Third Floor Columns	FS	0d

67	Splicing for For Third Floor Columns	0.2d	7/22/94	7/22/94	63FS+6d,64	

ID	Successor Name	Type	Lag
68	Reinforcement for Third Floor Columns	FS	0d

68	Reinforcement for Third Floor Columns	0.79d	7/22/94	7/25/94	67	

ID	Successor Name	Type	Lag
69	Forming of Small Third Floor Columns	FS	0d
70	Forming for Large Third Floor Columns	FS	0d

70	Forming for Large Third Floor Columns	3.72d	7/25/94	7/29/94	68	

ID	Successor Name	Type	Lag
71	Pouring for Third Floor Columns	FS	0d

71	Pouring for Third Floor Columns	0.27d	7/29/94	7/29/94	70,69	

ID	Successor Name	Type	Lag
72	Curing for Third Floor Columns	FS	0d

72	Curing for Third Floor Columns	0.13d	7/29/94	7/29/94	71	

ID	Successor Name	Type	Lag
73	Float Finishing of Third Floor Columns	FS	3d

73	Float Finishing of Third Floor Columns	4.14d	8/3/94	8/9/94	72FS+3d	

ID	Successor Name	Type	Lag
78	Shoring for Roof Floor Slab	FS	0d
96	Completion of Project	FS	0d

78	Shoring for Roof Floor Slab	3.47d	8/9/94	8/15/94	73	

ID	Successor Name	Type	Lag
79	Plate Forming for Roof Floor Slab	FS	0d
87	Breaking Patches and Voids First Floor	FS	0d

79	Plate Forming for Roof Floor Slab	8.69d	8/15/94	8/25/94	78	

ID	Successor Name	Type	Lag
80	Edge Forming for Roof Floor Slab	FS	0d
81	Reinforcing for Roof Floor Slab	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
81	Reinforcing for Roof Floor Slab	1.53d	8/25/94	8/29/94	79	

ID	Successor Name	Type	Lag
82	Pouring for Roof Floor Slab	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
82	Pouring for Roof Floor Slab	0.84d	8/29/94	8/30/94	81,80	

ID	Successor Name	Type	Lag
83	Curing for Roof Floor Slab	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
83	Curing for Roof Floor Slab	0.51d	8/30/94	8/30/94	82	

ID	Successor Name	Type	Lag
84	Float Finishing for Roof Slab	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
84	Float Finishing for Roof Slab	6.39d	8/30/94	9/8/94	83	

ID	Successor Name	Type	Lag
85	Waterproof Membrane on Roof Floor	FS	0d
88	Float Finishing of Second Floor Ceiling	FS	0d
94	Float Finishing Third Floor Ceilings	FS	6d
95	Breaking Patches and Voids on Third Floor	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
88	Float Finishing of Second Floor Ceiling	6.39d	9/8/94	9/16/94	84	

ID	Successor Name	Type	Lag
89	Breaking Patches and Voids Second Floor	FS	0d
96	Completion of Project	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
89	Breaking Patches and Voids Second Floor	19.35d	9/16/94	10/13/94	88	

ID	Successor Name	Type	Lag
96	Completion of Project	FS	0d

ID	Task Name	Duration	Start	Finish	Predecessors	Resource Names
96	Completion of Project	0d	10/13/94	10/13/94	8,15,36,38,39,56,57,66,	