SEISMIC CONSIDERATIONS IN THE DESIGN OF REINFORCED CONCRETE MULTI-STORY STRUCTURES

by

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INTRODUCTION

Earthquakes are one of nature's greatest hazards to life on this planet. Throughout historic time they have caused the destruction of countless cities and towns on nearly every continent. They are the least understood of the natural hazards and in early days were looked upon as supernatural events. It has been due to careful observation and in-depth study of the earth and the damages caused by earthquakes that we have been able to recognize earthquakes for what they are - a dynamic rebalancing of internal forces in parts of the earth crust which have been subjected to stress through differential uplift, subsidence or lateral shifting.

The purpose of this report is to explain the procedures currently being used in the design of a multi-story, reinforced concrete structure which is located in an area of high seismic risk. Primary emphasis is made in the determination of earthquake forces and incorporating their effects in design.

The same structure is also being analyzed and designed without any seismic consideration in order to compute the percentage increase in cost when seismic considerations are included.

CHAPTER 1

LITERATURE REVIEW

1.1 Occurrence of Earthquakes:

Earthquakes are a normal and inevitable feature of the geology of many countries of the world. In the United States there are 39 states (6) which are regarded to be located in regions of reasonable seismic danger.

The causes of earthquakes are not completely understood. It is said to be a phenomenon produced by a sudden change inside the earth. The origin of an earthquake is called the hypocenter, and the point vertically above it on the earth's surface is called the epicenter(6). The wave motions radiating from the epicenter and the ground motion of the earth's surface produced by these waves are called seismic waves and the earthquake ground motion, respectively. The hypocenter is not limited to a point but it has considerable length or volume. Its depth could be several hundred miles but in severe earthquakes it can be less than 30 miles.

1.2 Theories of Earthquake Mechanism:

There are two theories of Earthquake Mechanism:

1) The Dilatational Source Theory (15)
2) The Elastic Rebound Theory (15)

1.2.1 The Dilatational Source Theory

According to this theory earthquakes are caused due to explosive sources. This theory, which has now been eclipsed as the mechanical and elastic properties of materials have been more thoroughly understood, indicated a natural link between earthquakes and volcanic eruption.

1.2.2 The Elastic Rebound Theory

This theory is attributed to H. F. Reid and it was developed in 1906 around the time of the San Francisco earthquake. According to this theory, distortions occur gradually in the earth's crust. As a result, stresses build up with the passage of time until the stress at some location becomes great enough to fracture the rock or to cause it to slip along some previously existing fault planes. Slippage at one location causes an increase in the stress in the adjacent rock, so that the slippage propagates rapidly along the fault plane. The result is a sudden rebound of the elastic strain, and the strain energy that had accumulated in the rock is suddenly released and propagated in all directions from the source in a series of shock waves.

1.3 Earthquake Measurement

The most commonly used system for describing the intensity of earthquakes in the USA is the Modified Mercalli Intensity Scale. This indicates an arbitrary measure of the effects due to an earthquake as shown in Table 1.1 (15).

Earthquake magnitudes are measured on a seismograph using the scale developed by Charles F. Richter,

$$ME = \log_{10} \frac{A}{Ao}$$

where,

ME is the magnitude of the earthquake;

A is the maximum amplitude on the seismograph; and

A_o is an amplitude of one thousandth of a millimeter.

Richter also correlated the Modified Mercalli Intensity with the Richter magnitude scale as shown in Table 1.2 (15).

1.4 Earthquake Instrumentation

Earthquake Recording Instruments

In active seismic zones i.e. Zone No. 3 and Zone No. 4 according to the Uniform Building Code (7) - every building over 6 stories in height with an aggregate floor area of 60,000 square feet or more, and every building over 10 stories in height regardless of floor area shall be provided with three approved recording accelerographs. These instruments shall be located in the basement, midportion and top of the structure. They shall be properly maintained and easily accessable.

The accelographs measure and record the absolute acceleration of the ground and the structure over the required period of time which is useful in acquiring important and basic data for earthquake engineering. (15)

1.5 Seismic Risk

The Uniform Building Code (7) contains a seismic zone map of the United States, shown in Fig. 1.1. In this map four different zones are shown. Zone No. 1 has the minimum chance of earthquakes and Zone No. 4 has the maximum chance of earthquakes. This zoning has been based on the recorded distribution of earthquakes in the United States as well as other geological and statistical considerations. The use of this map in determining earthquake loadings for structures is explained in Section 2.2.1 of this report.

California is regarded as being the only region where major earthquakes are likely to occur in the U.S. Some of the largest earthquakes recorded in this country have occurred near New Madrid, Missouri and Charleston, South Carolina. The absence of recorded destructive earthquakes in any region is no reason to assume that it is immune, since major earthquakes can occur 100 or more years apart (15).

This suggests that even in regions that appear to be seismically inactive, the earthquake force should not be totally ignored in structural design.

1.6 Causes of Damage

Structural damage may be caused from any of several different effects of an earthquake such as tsunamis, landslides or soil - liquefaction. However "the principal loading mechanism recognized by the seismic design requirement in building codes is the response to the earthquake ground motions applied at the base of the structure." (15) The ground motions will be both vertical and horizontal but it is customary to neglect vertical components since most multi-story structures have more than adequate strength in the vertical directions due to safety factor requirements.

TABLE 1.1. Abridged Modified-Mercalli Intensity Scale

I.	Detected only by sensitive instruments.
II.	Felt by a few persons at rest, especially on upper floors; delicate suspended objects may swing.
III.	Felt noticeably indoors, but not always recognized as a quake; standing autos rock slightly, vibration like passing truck.
IV.	Felt indoors by many, outdoors by a few; at night some awaken; dishes, windows, doors disturbed; motor cars rock noticeably.
V.	Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects.
VI.	Felt by all; many are frightened and run outdoors; falling plaster and chimneys; damage small.
VII.	Everybody runs outdoors; damage to buildings varies depending on quality of construction; noticed by drivers of autos.
VIII.	Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of autos disturbed.
IX.	Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked; underground pipes broken.
X.	Most masonry and frame structures destroyed; ground cracked; rails bent; landslides.
XI.	New structures remain standing; bridges destroyed; fissures in ground; pipes broken; landslides; rails bent.
XII.	Damage total; waves seen on ground surface; lines of sight and level distorted; objects thrown up into air.

TABLE 1.2. Correlation of Richter Magnitude and Modified Mercalli Intensity

Richter Magnitude	Mercalli Intensity
2	I-II
3	III
4	V
5	VI-VII
6	VII-VIII
7	IX-X
8	XI

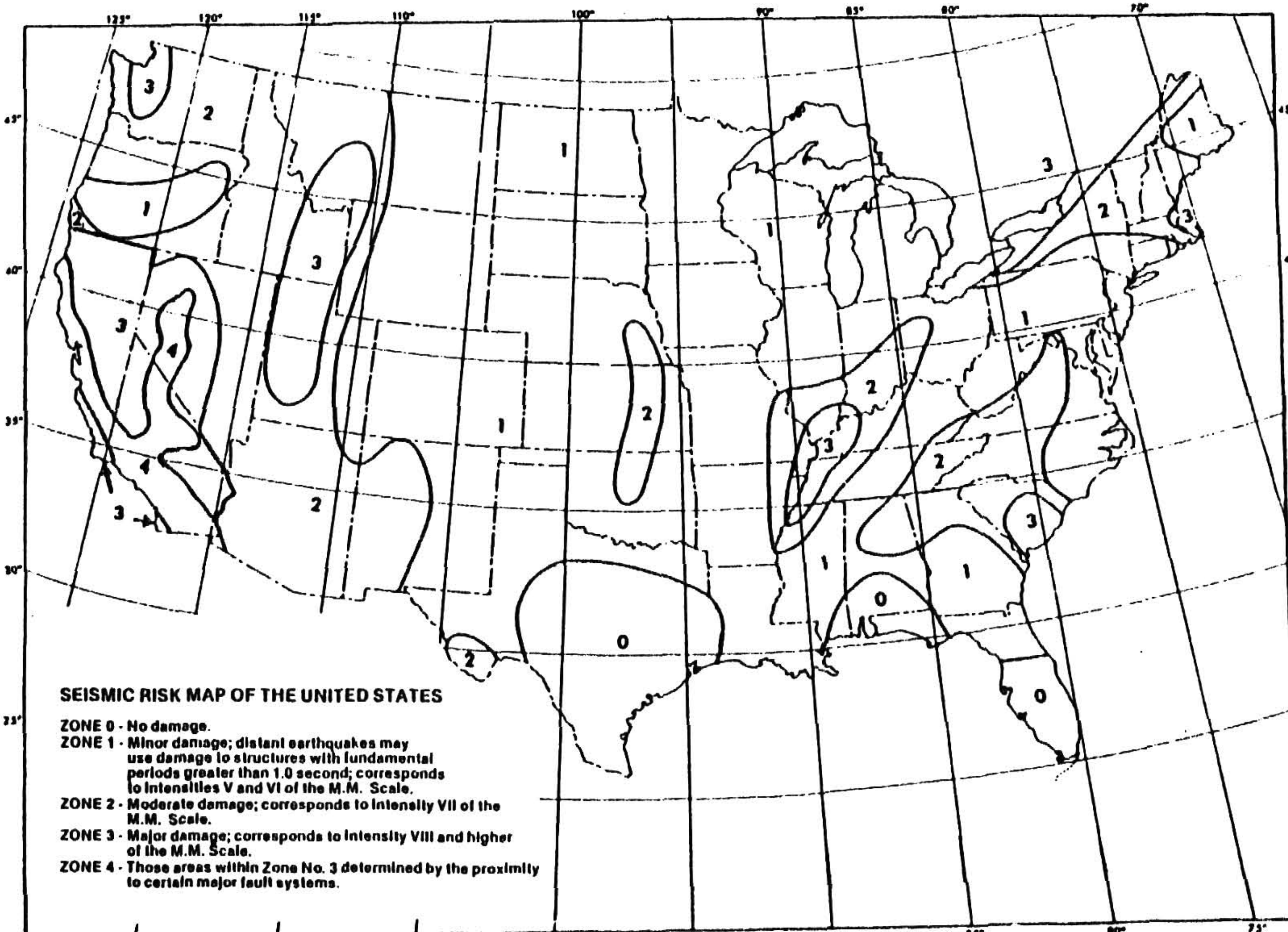


FIG. 1.1 Seismic Zone Map of the United States (7)

CHAPTER II

REVIEW OF THE SEISMIC CODE

2.1 Introduction

The seismic design considerations in this report are based on the Structural Engineers Association of California, (SEAOC) Seismology Committee's (1980) recommended lateral force requirements and commentary (Seismic Code). (12)

The main function of any building code is to provide minimum standards to assure public safety. Requirements of various codes are intended to safeguard against major failures and loss of lives. Absolute damage control is not the purpose of earthquake provisions in building codes, the purpose is to minimize damage and to protect the occupants.

The SEAOC recommendations provide criteria to fulfill life safety concepts. Structures designed in conformance with the provisions and principles of this code, as regard to earthquakes shall be able to:

1) Resist minor earthquakes without damage,

2) Resist moderate earthquakes without structural damage but with some non-structural damage.

3) Resist major earthquakes without collapse, but with some structural as well as non-structural damage.

The structural damage even in a major earthquake is expected to be repairable.

2.2 Minimum Earthquake Forces

According to the Seismic Code every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the structure in accordance with the formula

$$V = ZIKSCW \qquad \text{2-1}$$

except as provided in section 2.6 of this report and when lateral forces have been computed by dynamic analyses. (12)

2.2.1 The Zoning Factor "Z"

The term Z in Eq. 2-1 is a Zoning Factor, which is intended to take into account that geographic areas generally differ from each other with regard to the likelihood of earthquakes and also with regard to their probable frequency and intensity, if earthquakes have occurred in the past. The seismicity of an area, for zoning purposes, is determined primarily by the historical record of earthquakes and the location, length and estimated activity of earthquake faults in the region.

The value of Z is unity for the most active zones. The seismic code (12) and the UBC (7) have four zones and the values of Z for each zone are given in Table 2.1.

TABLE 2.1 THE ZONING FACTOR

	Zone No.	Z
1	Zone #1	3/16
2	Zone #2	3/8
3	Zone #3	3/4
4	Zone #4	1

2.2.2 Increment for Essential Facilities "I"

The term I establishes higher seismic design factors for facilities deemed essential to public welfare and which should remain functional for use after a major earthquake.

The seismic code establishes a maximum value for I of 1.5 for essential facilities and a value of 1.0 for other structures. No values of I are established for specific facilities. Studies on this coefficient are being continued by the SEAOC Seismology Committee. The UBC (7) contains an intermediate value of 1.25 also.

2.2.3 The Ductility Factor "K"

The value of K depends on the type of lateral resisting system as shown in Table 2.2. The K values are largely based on the actual observed performance of buildings in earthquake. Types of construction which have performed well in the past have been assigned lower values of K and, conversely, structures which have not peformed well and appeared to be inherently weak in resisting earthquake forces, have been assigned higher values of K.

These K values are really judgement factors since no way has been developed to measure and specify the ductility of a building structure as it actually performs. The ductility of a single structural member can be measured; it is the ratio of total deflection to yield deflection.

The ductility of concrete frames shall be dealt with in section 2.7 of this report.

2.2.4 Site Factor "S"

This term represents the site-structure interaction. From accelerograph records it is known that there may be significant differences in the ground motions at sites a relatively short distance apart. This has generally been attributed to differing conditions of the underlying soil. However, recent studies indicate that a determination of the so-called soil effect, or site effect is a complex problem. It involves not only the soil conditions at the site but also the source mechanism of the earthquake, the distance from source to site, surface and subsurface topography and the travel path of the earth virbrations from source to site.

TABLE 2.2
HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS
OR OTHER STRUCTURES

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	Value of K
All building framing systems except as hereinafter classified.	1.00
Building with a box system as defined in Section 1(B) of the seismic code. EXCEPTION: Buildings not more than three stories in height with stud-wall framing and using horizontal diaphragms and vertical shear panels for the lateral force system, may use K = 1.0.	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames designed in accordance with the following criteria: 1. The frame and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. 2. The shear walls or braced frames acting independently of the ductile moment resisting space frame shall resist the total required lateral force. 3. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.	0.80
Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force.	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building.	2.5
Structures other than buildings and other than those set forth in Table 2.3 on page 16.	2.0

The influence of the soil condition at the site is presumed to be directly related to the ratio of the fundamental vibration period T in seconds of the structure on the site, to an inherent natural vibration period T_s in seconds of the site itself. According to the Seismic Code the period T shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis such as the formula

$$T^* = 2\Pi \sqrt{\sum_{i=1}^{n} W_i \delta_i^2 \div (g \sum_{i=1}^{n} f_i \delta_i)} \qquad \text{2-2}$$

Where f_i represents any lateral force distributed approximately in accordance with the formulas (1-5) to (1-7) of the code or any other rational distribution. The elastic deflection δ_i shall be calculated using the applied lateral forces f_i.

In the absence of period determination as indicated above, the value of T may be determined by the formula

$$T = \frac{0.05\, h_n}{\sqrt{D}} \qquad \text{2-3}$$

(In this equation h_n is the height of the structure in ft and D is the lateral dimension of the structure in ft) T is obtained in sec.

Or, for buildings in which the lateral force resisting system consists of moment resisting space frames capable of resisting 100 percent of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces, T may be determined by the formula

$$T = 0.10N \qquad \text{2-4}$$

(N is the total number of stories above base)

According to the seismic code the value of S shall be determined by the following formulas but shall not be less than 1.0

For $\frac{T}{T_s} = 1$ or less,

$$S = 1.0 + T/Ts - .05\,(T/Ts)^2 \qquad \text{2-5}$$

*In the expression n is the total number of levels of the structure and i represents the respective levels whereas W_i is thier respective weight.

For $\frac{T}{T_s}$ greater than 1.0

$$S = 1.2 + 0.6 \frac{T}{T_s} - 0.3 \left[\frac{T}{T_s}\right]^2 \quad \text{------------} \quad 2\text{-}6$$

T in formulas (2-5) and (2-6) shall be established by a properly substantiated analysis but T shall not be taken less than 0.3 seconds.

The range of values of T_s may be established from properly substantiated geotechnical data, except that T_s shall not be taken as less than 0.5 second nor more than 2.5 seconds.

When T_s is not properly established, the value of S shall be 1.5.

2.2.5 The Seismic Coefficient "C"

The seismic coefficient C relates the response of the structure to the fundamental period of vibration T in seconds.

According to the code the value of C shall be determined in accordance with the formula

$$C = \frac{1}{15\sqrt{T}} \quad \text{--------------------} \quad 2\text{-}7$$

The value of C shall not exceed 0.12 and also the value of CS should not exceed 0.14.

2.2.6 The Effective Mass "W"

The term W in Eq. 2-1 is defined as the effective mass, but is actually a load and according to the code W is the total dead load and applicable portions of other loads such as: partitions, permanent equipment, snow, and in storage and warehouse occupancies, a minimum of 25 percent of the floor live load.

The combination of the factors (ZIKSC) applied to the effective mass (W) can result in an upper limit of 0.28 (.42 for essential facilities). According to the code, in any other region where earthquake provisions are applicable, the product of these factors should not be less than 0.015.

2.3 Distribution of Lateral Forces

2.3.1 Structures having Regular Shapes or Framing Systems

The total lateral force V shall be distributed over the height of the structure in accordance with the formula:

$$V = F_t + \sum_{i=1}^{n} F_i \qquad \text{2-8}$$

The concentrated force at the top, F_t shall be determined by the formula

$$F_t = 0.07\,T\,V \qquad \text{2-9}$$

F_t need not exceed 0.25V and may be considered as zero where T is 0.7 seconds or less. The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to the formula

$$F_x = \frac{(V - F_t)\,W_x h_x}{\sum_{i=1}^{n} W_i h_i} \qquad \text{2-10}$$

(In this expression h_x is the height and W_x is the weight of the structure at level x respectively.)

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level. The term F_x is the applied lateral force at the level or floor of the structure under consideration..

2.3.2 Structures having Irregular Shapes or Framing Systems

The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.

2.4 Overturning

Every structure shall be designed to resist the overturning effects caused by the wind forces and related requirements, or the earthquake forces specified in the seismic code, whichever governs.

The effects of the design overturning moment shall be taken into consideration as indicated in Section 1-F of the code.

2.5 Drift Provisions

As stated in Section 1-H of the code lateral deflections or drift of a story relative to its adjacent stories shall not exceed 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces shall be multiplied by $(1.0/K^{*})$ to obtain the drift. This ratio shall not be less than unity.

2.6 Lateral Force on Non-Structural Elements

Parts or portions of structures and their anchorage to the main structural system shall be designed for lateral forces in accordance with the formula

$$F_p = ZIC_pW_p \qquad \text{- - - - - - - - - - - - - - - -} \qquad 2\text{-}11$$

The values of C_p are set forth in Table 2.3 and the value of the I coefficient shall be the value used for the structure with some exceptions as indicated in Section (1-G) of the code. The term W_p is the weight of a portion of the strucure.

2.7 Concrete Ductile Moment Resisting Space Frames

2.7.1 Ductile Concrete Design

Ductile concrete differs from ordinary reinforced concrete in the amount and location of the reinforcing steel that is required. Ductile concrete is designed to ensure that in flexural members shear failure and compression failure in the concrete cannot occur prior to

*K is the ductility factor defined in section 2.2.3.

TABLE 2.3
HORIZONTAL FORCE FACTOR "C_p"
FOR ELEMENTS OF STRUCTURES

PART OR PORTION OF BUILDINGS	HORIZONTAL DIRECTION OF FORCE	VALUE OF C_p (1)
Cantilevered Elements a. Parapets	Normal to flat Surfaces	.8
b. Chimneys or stacks	Any Direction	
All other walls, partitions and similar elements	Any Direction	.3
Exterior and interior ornamentations and appendages	Any Direction	.8
When connected to, part of, or housed within a building: a. Penthouses, anchorage and supports for tanks including contents, chimneys and stacks. b. Storage racks plus contents. c. Suspended ceilings (3). d. All equipment or machinery.	Any Direction	$.3^{(2)(4)}$
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly.	Any Direction	$0.3^{(4)}$

(1) C_p for elements laterally self supported only at ground level may be 2/3 of the value shown.

(2) For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

(3) Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.

(4) The force shall be resisted by positive anchorage and not by friction.

stretching of the tensile bars, and in compression shear failure cannot occur and any concrete that fails in compression will be confined.

The capacity of a structure, based on its ability to absorb energy, is greatly increased by the use of a ductile rather than brittle material.

This difference is illustrated in Fig. 2.1 (15)

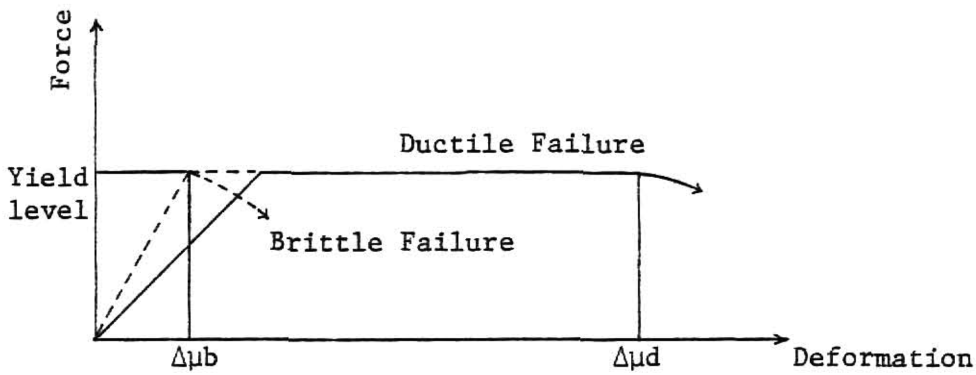


Fig. 2.1 DUCTILE VERSUS BRITTLE BEHAVIOUR

2.7.2 General Requirements of the Seismic Code

It is stated in Section (2-A) of the code that design and construction of cast-in-place, monolithic reinforced concrete framing members and their connections in ductile moment resisting space frames shall conform to the requirements of ACI Building Code, ACI 318, and all requirements of Section 2 of this code.

All lateral load resisting frame members shall be designed by the strength design method except that the alternate design method may be used provided that it is shown that the factor of safety is equivalent to that achieved with the strength design method.

The load factors given in ACI 318, for earthquake loading shall be modified to be

$U = 1.4\ (D + L + E)$ - - - - - - 2-12

and $U = 0.9D + 1.4E$ - - - - - - - 2-13

2.7.3 Seismic Code Requirements for Flexure, Shear and Axial Loading

a) Flexural Members:

Flexural members shall not have a width-depth ratio of less than 0.3, nor shall the width be less than ten inches nor more than the supporting column width plus a distance on each side of the column of three-fourths the depth of the flexural member. All flexural members shall have a minimum reinforcement ratio for top and bottom reinforcement, of $200/f_y$ throughout their length. The reinforcement ratio shall not exceed 0.02.

b) Shear:

Vertical web reinforcement of not less than No. 3 bar shall be provided in accordance with ACI 318 except that

i) Stirrups shall be spaced at no more than d/2 throughout the length of the member.

ii) Stirrup-ties at a maximum spacing of not over d/4, 8 bar diameters, 24 stirrup-tie diameters or twelve inches may be provided at specific locations.

c) Column Subjected to Direct Stress and Bending:

The ratio of minimum to maximum column thickness shall not be less than 0.4 nor shall any dimension be less than twelve inches.

The reinforcement ratio shall not be less than 0.01 nor greater than 0.06.

2.8 Physical Requirements for Concrete and Steel

Concrete: The minimum specified 28-day strength of the concrete, f_c', shall be 3000 psi. The maximum specified 28-day strength for light-weight concrete shall be 4000 psi.

Steel: All longitudinal reinforcing steel in columns and beams shall comply with ASTM AG15 Grade 40 or 60, or ASTM A706. Reinforcing conforming to ASTM AG15 shall comply with some additional requirements as stated by the Seismic Code.

Chapter III

ANALYSIS WITH SEISMIC CONSIDERATION

In order to explain the procedures as stated in Chapter 2 of this report, a structure will be analyzed and designed. This is shown in Fig. 3.1.

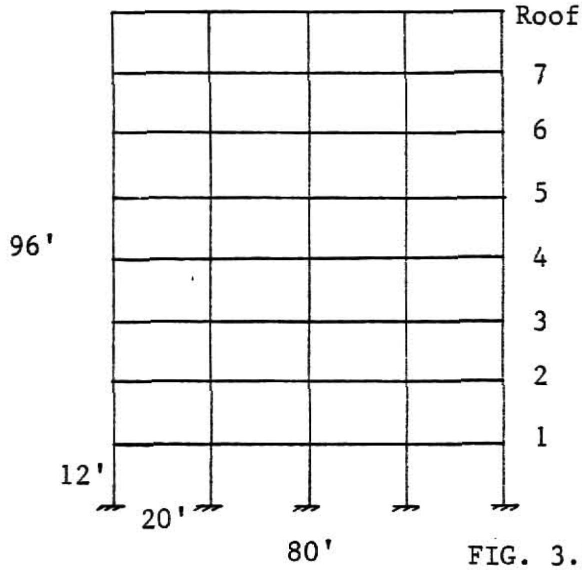


FIG. 3.1 THE ASSUMED STRUCTURE

3.1 Initial Assumption of Dimensions

The structure assumed is 8 stories high and consists of 4 bays in each direction. The total height is 96 feet, each floor being 12 feet high. The width in each direction is 80 feet, each bay being 20 feet wide.

This is a beam-column structure, the beams being 12" x 18"* and spanning 20 ft. The columns are assumed to be square (15" x 15")* in cross-section. The slab thickness is 6".

3.2 Dead and Live Loads

The live load is assumed to be 75 lbs/ft^2. The unit weight of reinforced concrete is 150 lbs/ft^3.

* These were initial assumptions and were changed later.

3.3 Wind Loads (5)

The velocity of wind is assumed to be 95 mph. The drag coefficient C_d is assumed to be 1.1.

3.4 Snow Load

The snow load is assumed to be 25 lbs/ft^2 for the design of structural members and is assumed as 15 lbs/ft^2 for the computation of effective mass.

3.5 *Material Assumption*

The ultimate compressive strength of concrete f'_c = 4 ksi. The yield stress of steel f_y = 60 ksi.

3.6 Computing The Effective Mass W

Weight of Structure Per Floor (Floors 1 to 7)

Weight of Slab	= 80 ft x 80 ft x .5 ft x .15 K/ft^3 = 480 kips
Weight of Beams*	= 10 x 80 ft x 1.5 ft x 1 ft x .15 K/ft^3 = 180 kips
Weight of Columns*	= 25 x 12 ft x 1.25 ft^2 x .15 K/ft^3 = 70 kips
Percentage of live load	= $\frac{25}{100}$ x 80 ft x 80 ft x .075 K/ft^2 = 120 kips
Walls and partitions etc. (There are five lines of these along each axis)	= 2 x 5 x 12 ft x 80 ft x .33 ft x .15 K/ft^3 = 480 kips

TOTAL WEIGHT OF STRUCTURE PER FLOOR = 1330 kips

Weight of roof (including snow load)

Snow Load 15 lbs/ft^2	= 80 ft x 80 ft x .025 K/ft^2	= 96 kips
Weight of Slab	= 80 ft^2 x .5 ft x .15 K/ft^2	= 480 kips
	TOTAL	= 646 kips

THUS TOTAL WEIGHT OF STRUCTURE = 9956 kips

*The slight increase in effective mass due to changes made in the dimension of columns and beams later is being neglected.

3.7 Computing the Lateral Forces

To determine the period T of the building use is made of Eq. 2-3

$T = 0.05\ h_n/\sqrt{D}$ (As explained on pg. 12, the value of T obtained is in secs when h_n and D are in feet.)

$T = 0.05 \times 96'/\sqrt{80}$

$T = 0.537$ secs > 0.3 secs O.K.

Computing the co-efficient C from Eq. 2-7

$C = 1/15\ \sqrt{T}$

$C = 1/15\ \sqrt{.537}$

$C = 0.091 < .12$ O.K.

In order to compute the T/T_s ratio a value of 1.5 seconds has been assumed for the characteristic site period T_s

$$T/T_s = \frac{.537}{1.5} = .358$$

Since this ratio is less than 1.0, Eq. 2-5 is used to find the numerical co-efficient for site-structure resonance S.

$S = 1.0 + T/T_s - 0.5\ (T/T_s)^2$

$S = 1.0 + .358 - 0.5\ (.358)^2$

$S = 1.294$

This value of S is greater than the minimum required value of 1.0.

The product of CS is .1178 which is less than the required maximum of 0.14.

The structure is assumed to be located in Earthquake Zone IV. The code specifies that the earthquake co-efficient Z is 1.0 for this location.

The structure is assumed not to be an essential facility and the value of I is taken as 1.0.

The value of K is assumed as 0.8.

The product of the combination of the factors should not be less than .015 according to the code:

$$ZIKSC = 1.0 \times 1.0 \times .8 \times 1.294 \times .091$$

$$= 0.0942032 > .015 \qquad \text{O.K}$$

According to Eq. 2-1

$$V = ZIKSCW$$

$$V = .0942032 \times 9956 \text{ kips}$$

$$V = 937.8 \text{ kips}$$

The portion of V considered concentrated at the top of the structure F_T is computed from Equation 2-9

But $T = 0.537$ secs < 0.7 secs

Thus F_T is assumed to be zero

The lateral force F_x applied at any level x is found using Eq. 2-10

$$F_x = (V - F_T)\ (W_x)\ (h_x) / \sum_{i=1}^{n} (W_i)(h_i)$$

$$F_x = (937.8 - 0)\ (W_x)(h_x) / 508896$$

The term W_i indicates the weight of the floors for i=1 to i=8 and similarly the term h_i indicates the height of each floor from the base.

$$\begin{aligned}\Sigma h_i W_i &= 12 \times 1330 + 24 \times 1330 + 36 \times 1330 + 48 \times 1330 \\ &+ 60 \times 1330 + 72 \times 1330 + 84 \times 1330 + 96 \times 646 \\ &= 508896 \text{ K-ft}\end{aligned}$$

The lateral forces for all of the floors and the roof can now be determined. The results of these computations are given in Table 3.1.

TABLE 3.1 COMPUTATION OF LATERAL FORCE FOR EACH FRAME

FLOOR NO. x	HEIGHT FT *	WEIGHT KIPS	TOTAL FORCE KIPS	FORCE FOR EACH FRAME KIPS
2	12	1,330	29.43	5.886
3	24	1,330	58.86	11.772
4	36	1,330	88.29	17.658
5	48	1,330	117.72	23.544
6	60	1,330	147.15	29.430
7	72	1,330	176.58	35.316
8	84	1,330	206.01	41.202
ROOF	96	646	113.29	22.658
			Σ 937.80	

*This height is the height of the floor from the base.

3.8 Portal Method For Lateral Loads (9)

The most common approximate method of analyzing building frames for lateral loads is the portal method. This method was presented by Albert Smith in the Journal of Western Society of Engineers in 1915 and is said to be satisfactory for most buildings up to 25 stories in height.

Three assumptions must be made for each individual portal or for each girder. In the portal method, the frame is theoretically divided into independant portals and the following three assumptions are made.

1. The columns bend in such a manner that there is a point of inflection at mid-depth.
2. The girders bend in such a manner that there is a point of inflection at their center lines.
3. The horizontal shears on each level are arbitrarily distributed between the columns. One commonly used distribution (and the one illustrated in this report) is to assume the shear divides among the columns in the ratio of one part to exterior columns and two parts to interior columns.

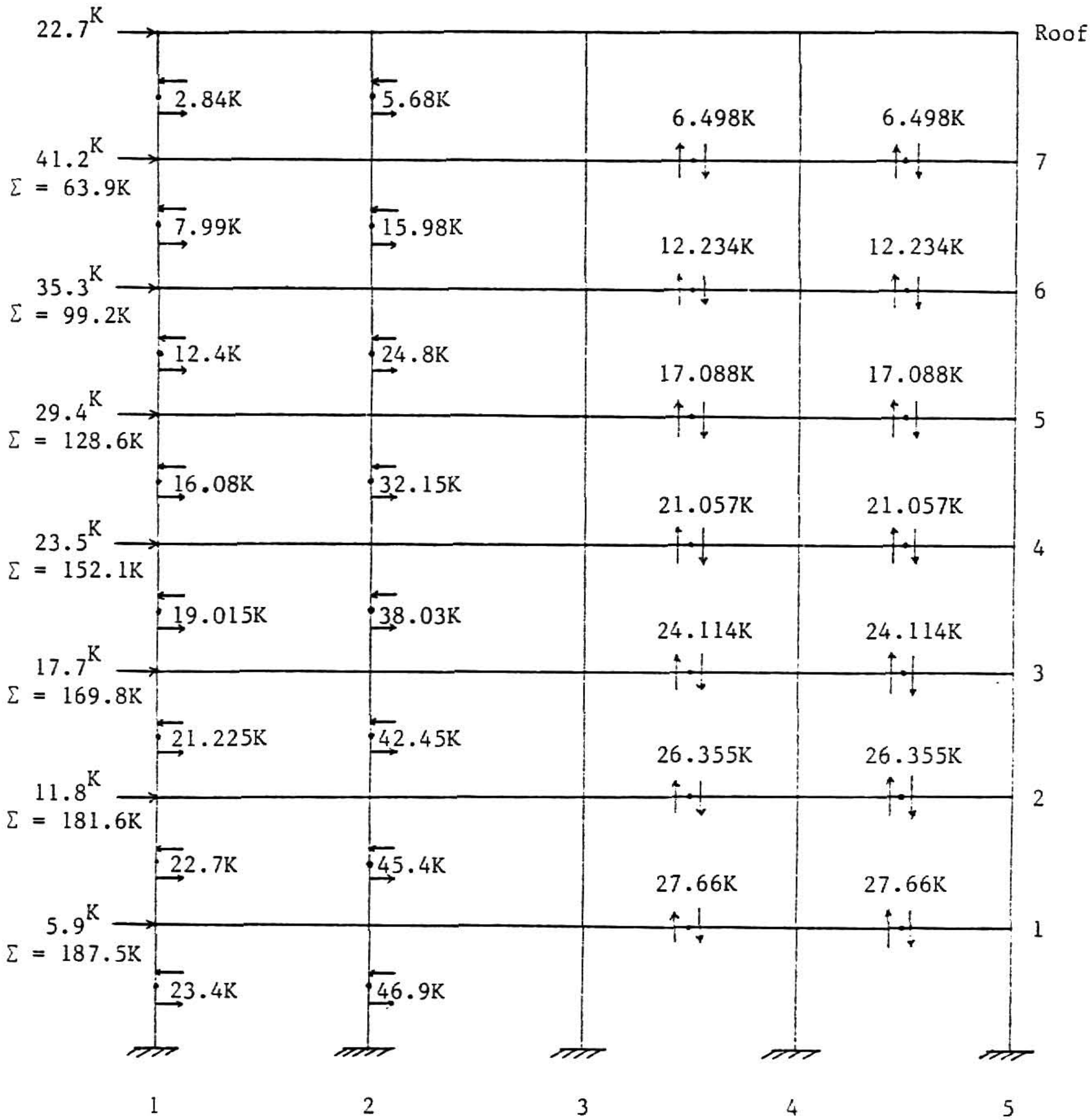
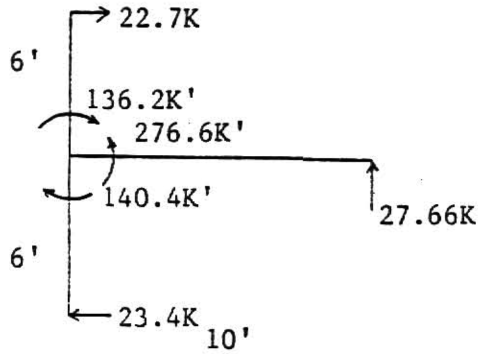


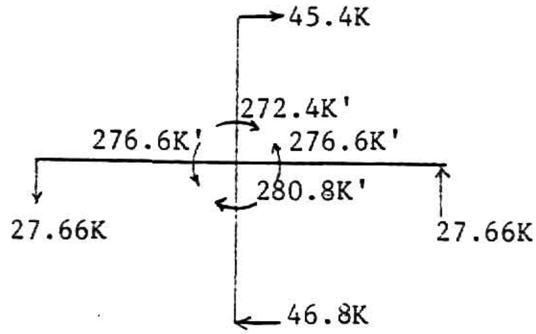
FIG. 3.2 FRAME ANALYSIS BY PORTAL METHOD
FOR SEISMIC LOADING

In the following computation, the shear forces are acting at center points of columns and beams which are 6' and 10' from the joints respectively (shown in figure for Joints 11, 51).

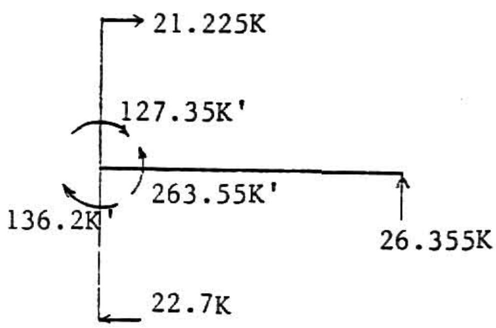
Joints 11, 51*



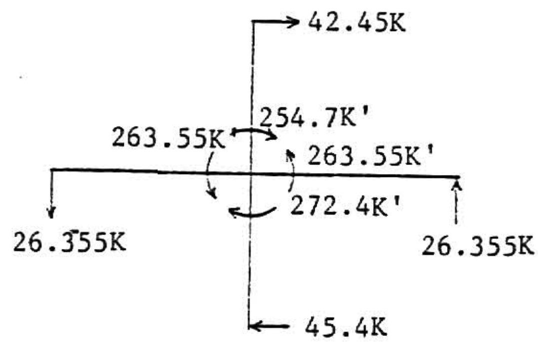
Joints 21, 31, 41*



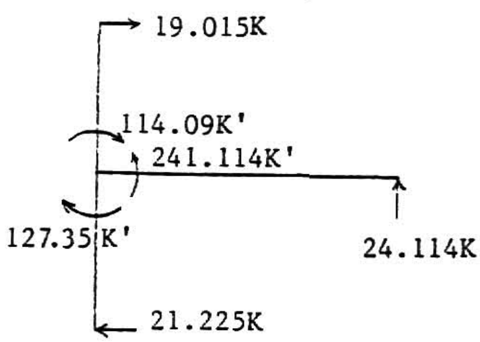
Joints 12, 52



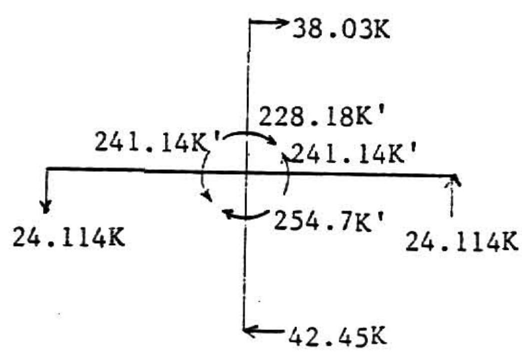
Joints 22, 32, 42



Joints 13, 53

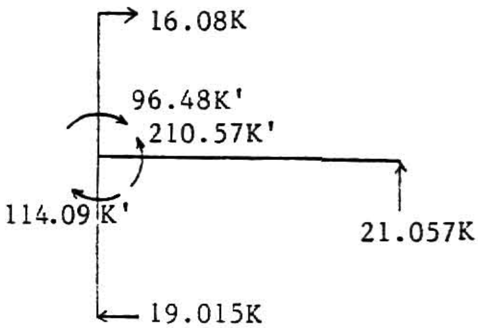


Joints 23, 33, 43

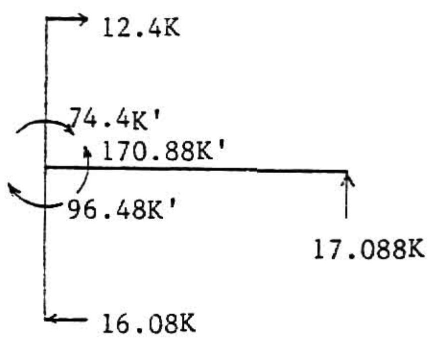


*The joints have been assigned these numbers in accordance with figure 3.2. The first number indicates the beam level and the second number indicates the column number. Numbers 1 and 5 columns are the corner columns and number 2, 3 and 4 are the intermediate columns.

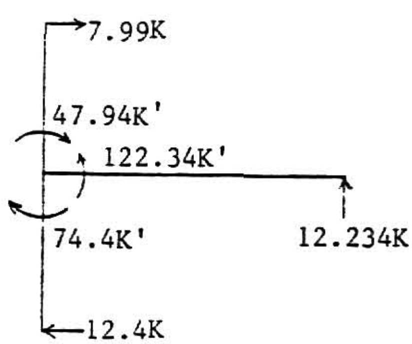
Joints 14, 54



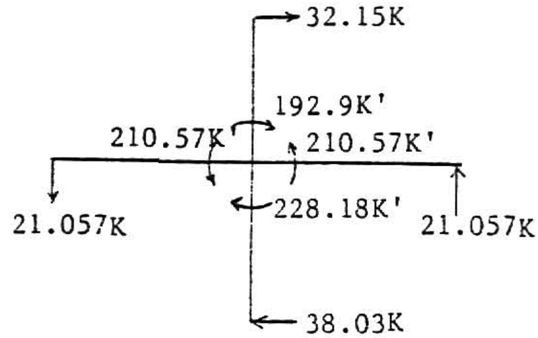
Joints 15, 55



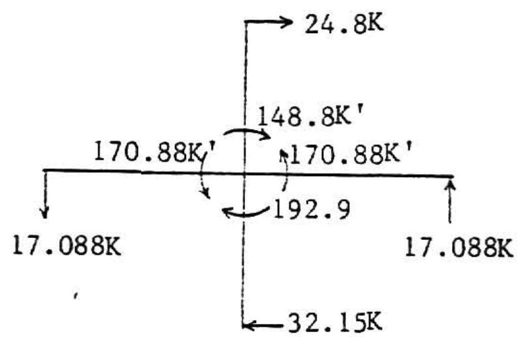
Joints 16, 56



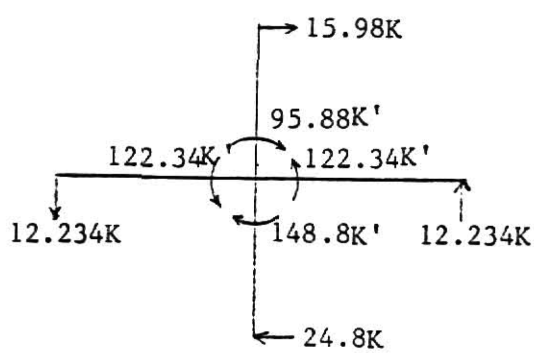
Joints 24, 34, 44



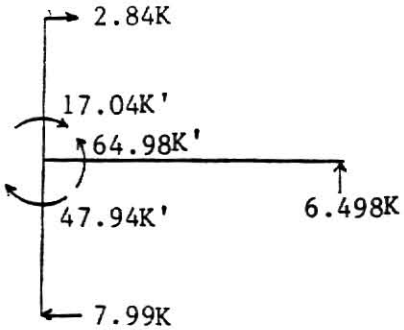
Joints 25, 35, 45



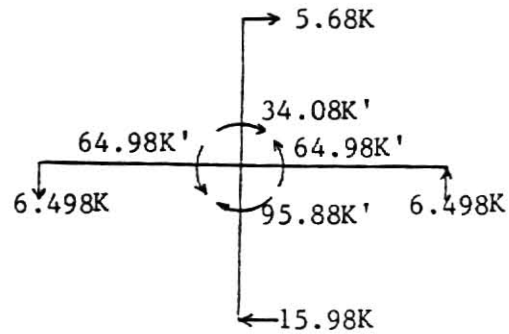
Joints 26, 36, 46



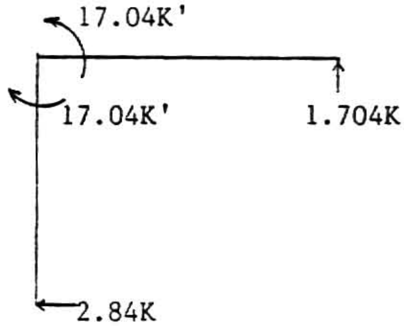
Joints 17, 57



Joints 27, 37, 47



Joints 1R, 5R



Joints 2R, 3R, 4R

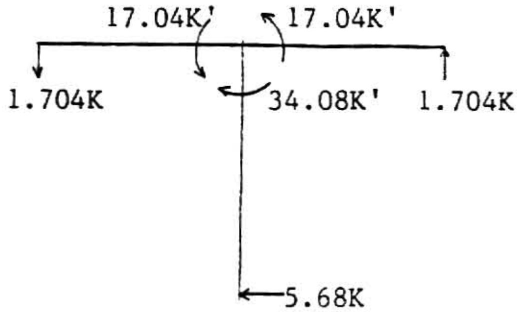
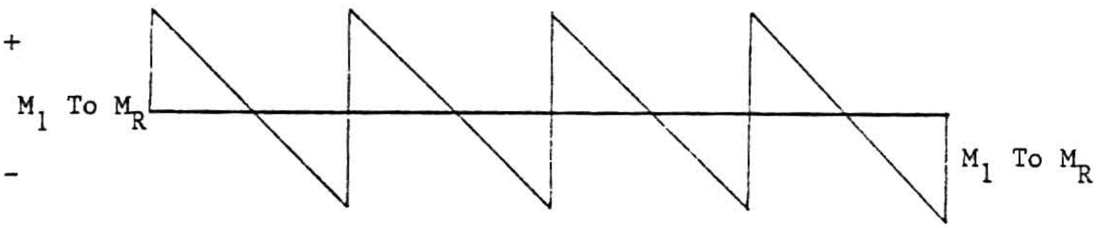


FIG. 3.3 BENDING MOMENT DIAGRAM FOR BEAMS



M_1 = 276.6 K'

M_2 = 263.55 K'

M_3 = 241.114 K'

M_4 = 210.57 K'

M_5 = 170.88 K'

M_6 = 122.34 K'

M_7 = 64.98 K'

M_R = 17.04 K'

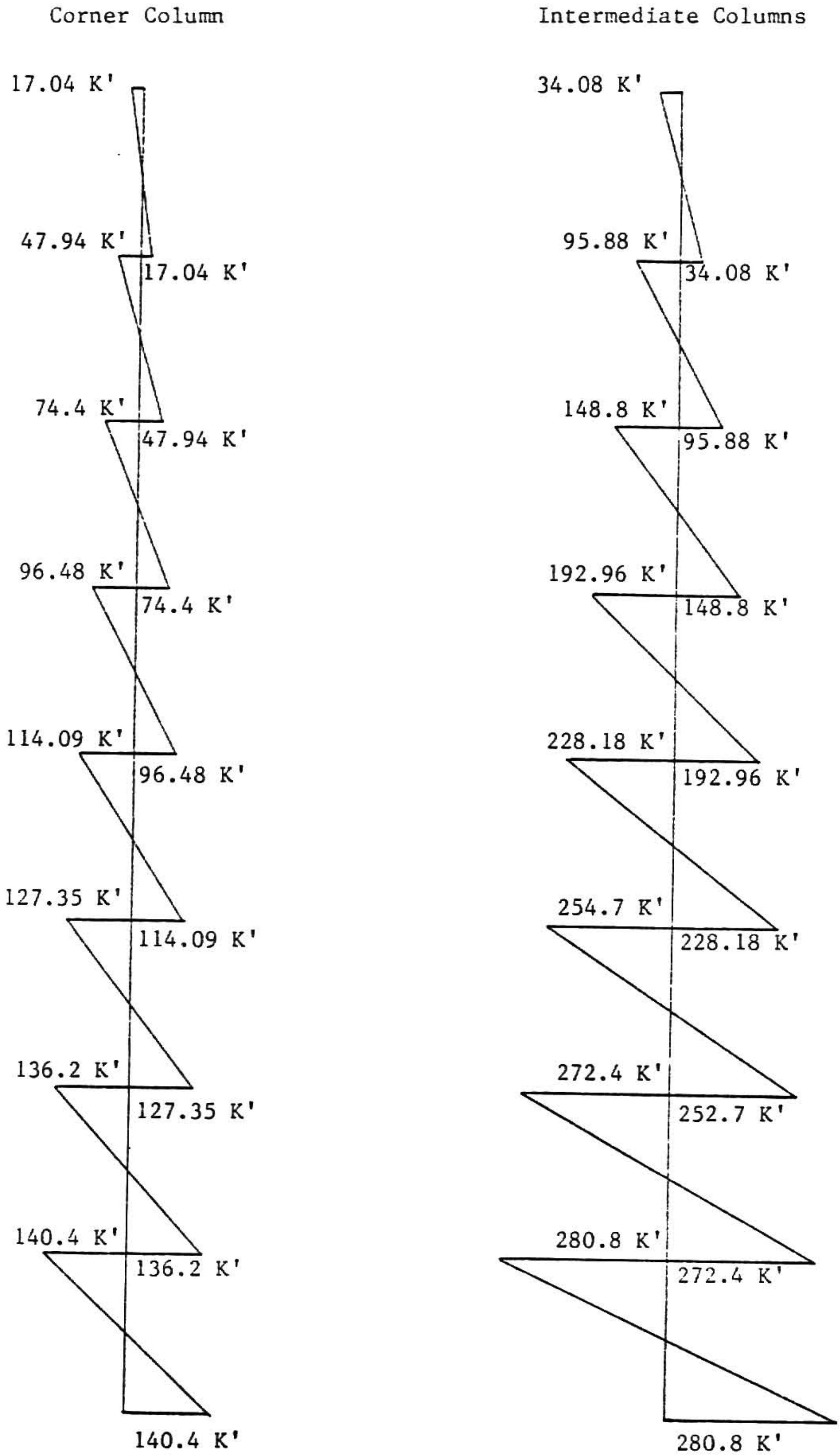


FIG. 3.4 MOMENT DIAGRAMS FOR COLUMNS

CHAPTER 4

NON-SEISMIC ANALYSIS AND DESIGN

4.1 Material Assumptions

f'_c = 4 ksi $\qquad E_c = 57000\sqrt{f'_c}$ (1)

f_y = 60 ksi $\qquad E_c = 3.605 \times 10^3$ ksi

ρ_B = .0285

$\rho_A = .5\,\rho_B = .0143$

ρ_{min} = .0033

ϕ = 0.9

Unit weight of concrete = 150 lbs/ft^3

$$\frac{M_u^*}{\phi b d^2} = \rho f_y \left(1 - 0.59\,\rho \frac{f_y}{f'_c}\right) = 0.75 \text{ ksi}$$

4.2 Loading

a) Dead Load

Slab load = 20 ft x 0.5 ft x .150 K/ft^3 = 1.5 K/ft

Beam load = $\dfrac{15 \text{ in} \times 18 \text{ in} \times .15\ K/ft^3}{144}$ = .28125 K/ft

W_D = 1.78125 K/ft

b) Live Load

Live load (Floors 1 - 7) = 75 lbs/ft^2

Live load (Roof) = 25 lbs/ft^2

c) Live load for floor beams = 1.5 k/ft $\qquad W_L$ = 1.5 K/ft

Live load for roof beams = 0.5 k/ft

4.3 Initial Member Size Assumptions

1. Slab Thickness 6"
2. Beam Size (breadth x depth) = 15" x 18"
3. Column Size 15" x 15"

*The usable flexural strength is given by the expression $M_u = \phi A_s f_y$ (d - a/2) and the depth of the stress block $a = A_s f_y / 0.85 f'_c b$. The substitution of expression for a in the expression for M_u results in this alternate expression for M_u.(8)

4.4 Computing Moment and Shear Using ACI Coefficient

$W_u = 5.05$ k/ft Case 1:(1) $W_u = 1.4\ W_D + 1.7\ W_L$

$W_u l_n^2 = 1775.39$ k ft $l_n = 18.75$ ft

Positive Moment Floor Beams

Ends	$M = 1/14\ W_u l_n^2$	= 126.81 K'	= 1521.76 K"
Interior	$M = 1/16\ W_u l_n^2$	= 110.96 K'	= 1331.54 K"

Negative Moment Floor Beams

Interior	$M = 1/16\ W_u l_n^2$	= 110.96 K'	= 1331.54 K"
Exterior	$M = 1/10\ W_u l_n^2$	= 177.54 K'	= 2130.47 K"
Other	$M = 1/11\ W_u l_n^2$	= 161.4 K'	= 1936.79 K"

Shear Floor Beams

End Members Interior	$V = 1.15\ (0.5\ W_u l_n) = 54.45$ K
End Members Exterior	$V = 0.85\ (0.5\ W_u l_n) = 40.24$ K
Other Supports	$V = 0.5\ W_u l_n$

M-Diagrams

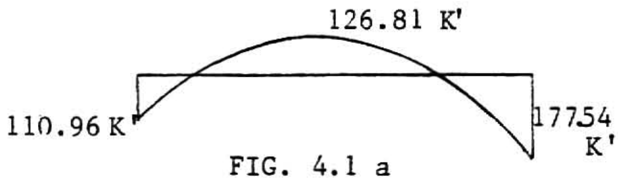


FIG. 4.1 a

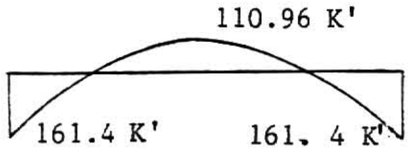


FIG. 4.1 b

V-Diagrams

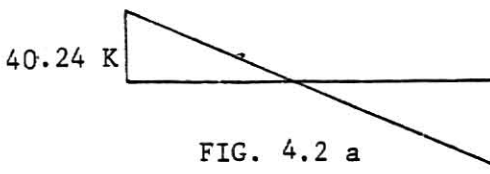


FIG. 4.2 a

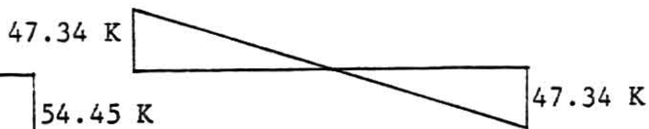


FIG. 4.2 b

4.5 Computing Moments and Shears Due to Wind Load

$p = 0.002558\ C_d X^2$ (5) $X = 95$ mph $C_d = 1.1$ (5)

$p = 25.3$ lb/ft^2 Area = 12 ft x 20 ft = 240 ft^2

(X is the velocity of wind and C_d is the shape factor)

Lateral Load Per Floor = .0253 K/ft^2 x 240 ft^2 = 6^K

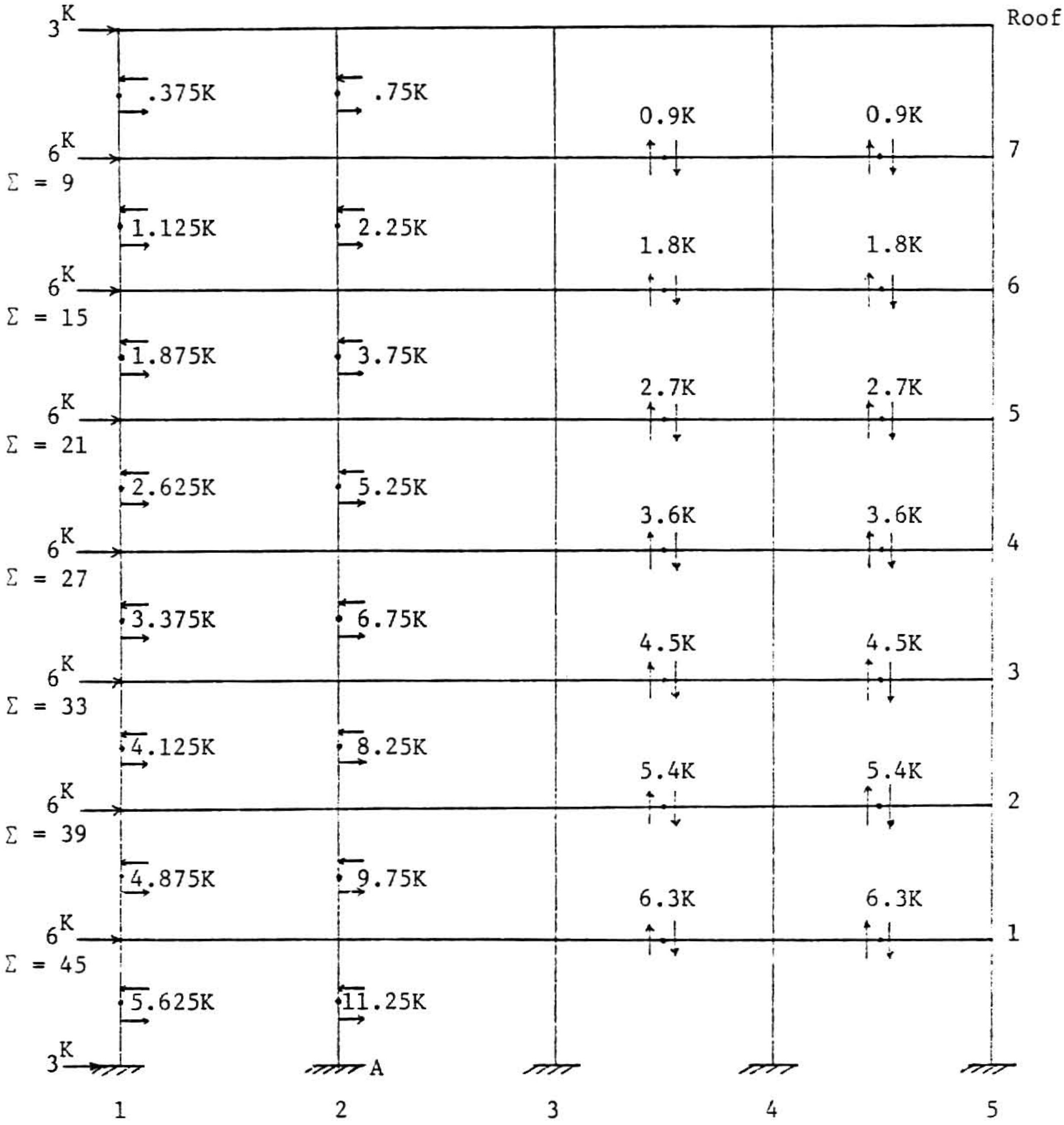
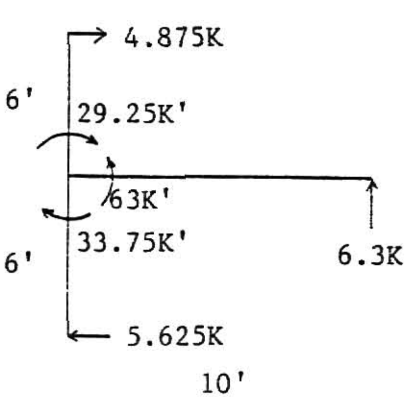


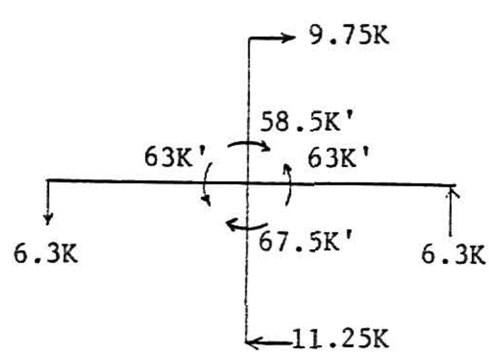
FIG. 4.3 FRAME ANALYSIS BY PORTAL METHOD

In the following computations the shear forces are acting at center points of columns and beams which are 6' and 10' from the joints respectively (shown in the figure for joints 11, 51).

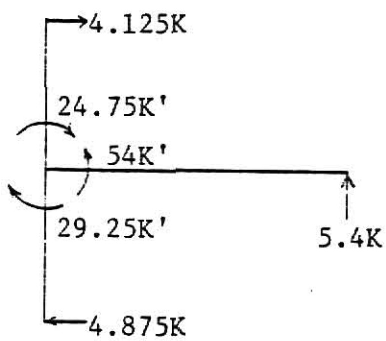
Joints 11, 51*



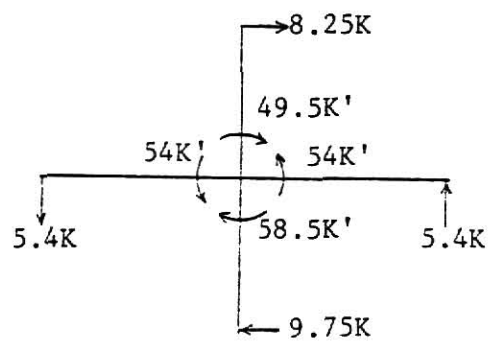
Joints 21, 31, 41*



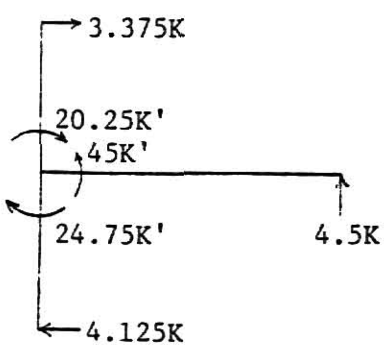
Joints 12, 52



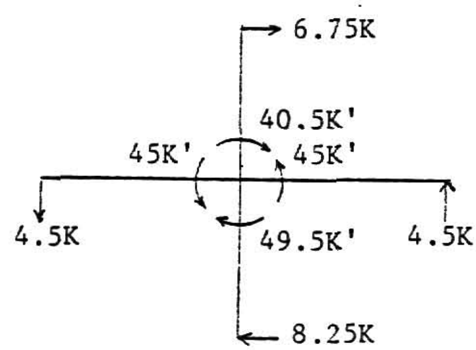
Joints 22, 32, 42



Joints 13, 53

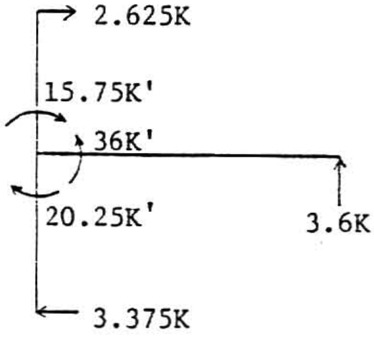


Joints 32, 33, 43

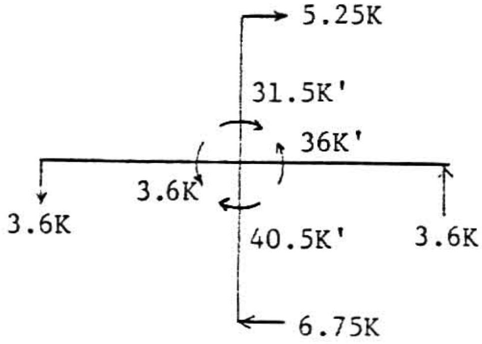


*The joints have been assigned these numbers in accordance with Figure 4.3. The first number indicates the beam level and the second number indicates the column number. Numbers 1 and 5 columns are the corner columns and numbers 2, 3 and 4 are the intermediate columns.

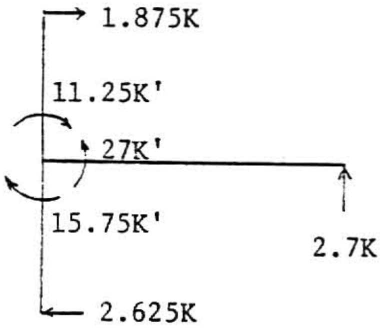
Joints 14, 54



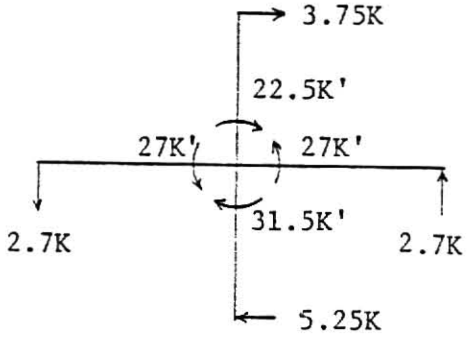
Joints 24, 34, 44



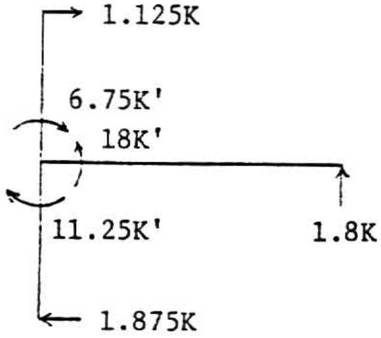
Joints 15, 55



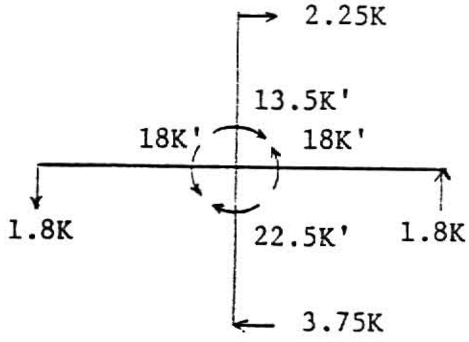
Joints 25, 35, 45



Joints 16, 56



Joints 26, 36, 46



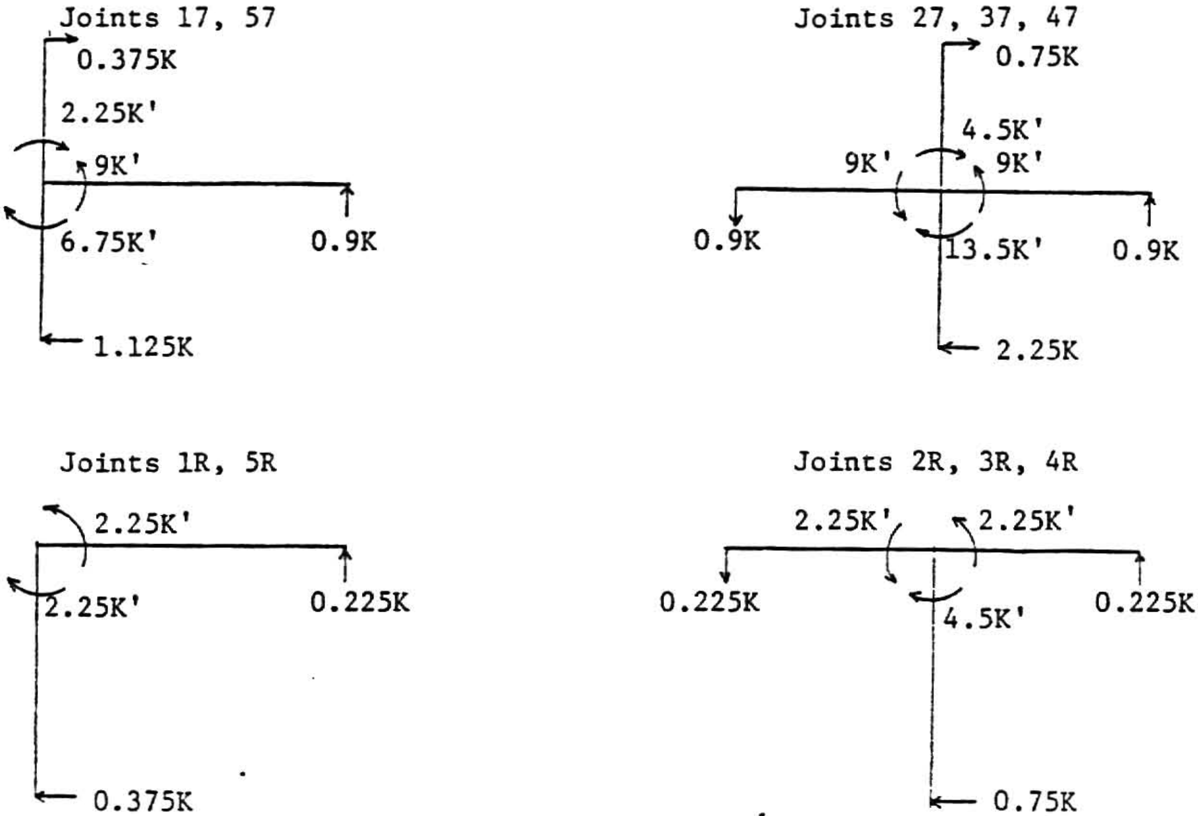
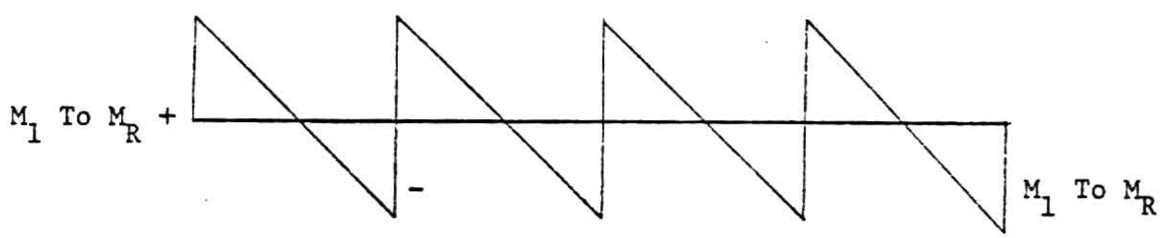


FIG. 4.4 BENDING MOMENT DIAGRAM FOR BEAMS



M_1 = 63K'

M_2 = 54K'

M_3 = 45K'

M_4 = 36K'

M_5 = 27K'

M_6 = 18K'

M_7 = 9K'

M_R = 2.25K'

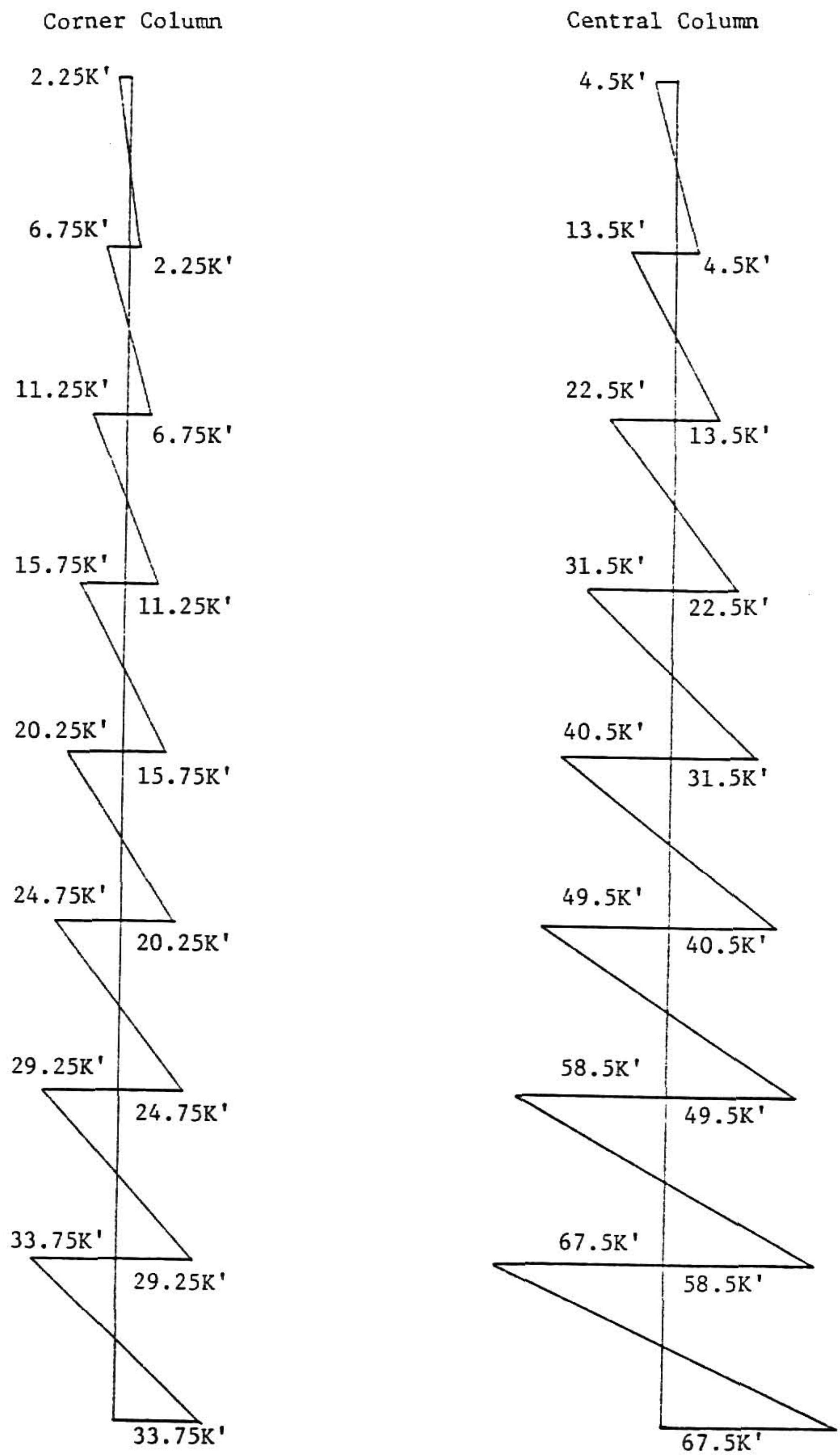


FIG. 4.5 BENDING MOMENT DIAGRAMS FOR COLUMNS
(Due to Wind Load)

4.6 ACI Load Combinations

a) 1.4 D + 1.7 L

b) 0.75 (1.4 D + 1.7 L + 1.7 W)

c) 0.75 (1.4 D + 1.7 W)

d) 0.9 D + 1.3 W

The four loading combinations have been used in computing moment and shear at various locations for the 8 floor beams.

4.6.1 Critical Moments for Beam #1 (Refer to Figs. 4.1 and 4.4)

a) End Span Positive = 126.81 K'

Int Span Positive = 110.96 K'

End Span Ext. Negative = 177.54 K'

End Span Int. Negative = 110.96 K'

Int Span Negative = 161.4 K'

b) End Span Positive = .75 x 126.81 = 95.11 K'

Int Span Positive = .75 x 110.96 = 83.22 K'

End Span Ext Negative = .75 x (177.54 + 1.7 x 63)
= 213.48 K'

End Span Int Negative = .75 x (110.96 + 1.7 x 63)
= 163.55 K'

Int Span Negative = .75 x (161.4 + 1.7 x 63)
= 201.375 K'

c) Ratio between 1.4 D and 1.4 D + 1.7 L = 0.494

End Span Positive = .75 (126.81 x .494) = 46.97 K'

Int Span Positive = .75 (110.96 x .494) = 41.09 K'

End Span Ext Negative = .75 (177.54 x .494 + 1.7 x 63)
= 146.08 K'

End Span Int Negative = .75 (110.96 x .494 + 1.7 x 63)
= 121.42 K'

Int Span Negative = .75 (161.4 x .494 + 1.7 x63)
= 140.10 K'

d) Ratio between 0.9 D and 1.4 D + 1.7L = 0.31784

End Span Positive	= .31784 x 126.81	= 40.26 K'
Int Span Positive	= .31784 x 110.96	= 35.2 K'
End Span Ext Negative	= .31784 x 177.54 + 1.3 x 63	= 138.26 K'
End Span Int Negative	= .31784 x 110.96 + 1.3 x 63	= 117.12 K'
Int Span Negative	= .31784 x 161.4 + 1.3 x 63	= 133.14 K'

4.6.2 Summary for Critical Moments for Beam #1

End Span Positive	= 126.81 K'	(a)
Int Span Positive	= 110.96 K'	(a)
End Span Ext Negative	= 213.48 K'	(b)
End Span Int Negative	= 163.55 K'	(b)
Int Span Negative	= 201.375 K'	(b)

Similar procedure was carried out for Beam #2 to roof beam and the critical moments were obtained. The critical moments for different level beams are shown in Table 4.1.

TABLE 4.1 CRITICAL DESIGN MOMENTS FOR BEAMS

FLOOR LEVEL	END SPAN INTERIOR NEGATIVE K-ft	END SPAN POSITIVE K-ft	END SPAN EXTERIOR NEGATIVE K-ft	INTERMEDIATE SPAN NEGATIVE K-ft	INTERMEDIATE SPAN POSITIVE K-ft
1	163.550	126.81	213.480	201.375	110.96
2	152.075	126.81	202.005	189.900	110.96
3	140.600	126.81	190.530	178.425	110.96
4	129.125	126.81	179.005	166.950	110.96
5	117.650	126.81	177.540	161.400	110.96
6	110.960	126.81	177.540	161.400	110.96
7	110.960	126.81	177.540	161.400	110.96
ROOF	73.610	84.12	117.540	107.070	73.61

4.7 Beam Design

M_{max} = 213.48 k-ft

$$bd^2 = \frac{M_u}{\phi x .75} = \frac{213.48 \times 12}{.9 \times .75} = 3795.2 \text{ in}^3$$

d^2 = 253.01 in^2 d = 15.91" , b = 15 in.

d provided = 15.75" (d = 18 - 1.5" - .375" - .75/2" = 15.75")

$$\rho = \frac{1 - \sqrt{1 - \frac{2.36 M_u}{\phi b d^2 f'_c}}}{1.18 f_y/f'_c} = .0146 \cong .0143 \quad \text{O.K}$$

Ast = 3.45 in^2

Provide 8 # 6 bars   Ast = 3.52 in^2

$$\rho_{provided} = \frac{3.52}{15 \times 15.75} = .0149 \cong .0146 \quad \text{O.K}$$

$$M_d = \phi A_s f_y (d - a/2)$$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{3.52 \times 60}{.85 \times 4 \times 15} = 4.141"$$

$$M_d = .9 \times 3.52 \times 60 \left(15.75 - \frac{4.141}{2}\right) \times \frac{1}{12} = 216.7 \text{ K-ft}$$

> 213.48 K-ft O.K

4.8 Design for Shear *

4.8.1 Beams on Floors 1 - Floor 7 W_u = 5.05 K/ft (Case a)

a) End Span

$V_{u_{max}}$ at a distance d from right support

$$54.45 - \frac{16}{12} \times 5.05 = 47.72^K$$

$V_{u_{max}}$ at a distance d from left support

$$40.24 - \frac{16}{12} \times 5.05 = 33.51^K$$

$$V_c = 2 \sqrt{f'_c}\, b_w\, d = 2 \frac{\sqrt{4000}}{1000} 15 \times 16 = 30.36^K$$

*The three loading combinations used for computation of maximum shear forces are shown in Table 4.3   d is approximated as 16" for shear design.

TABLE 4.2 FLEXURAL REINFORCEMENT FOR BEAMS
Required Areas and Bars Provided

FLOOR LEVEL	END SPAN INT. NEG in^2 BARS PROVIDED	END SPAND POSITIVE in^2 BARS PROVIDED	END SPAN EXT. NEG in^2 BARS PROVIDED	INT. SPAN NEGATIVE in^2 BARS PROVIDED	INT. SPAN POSITIVE in^2 BARS PROVIDED
1	2.62 6 # 6	1.93 5 # 6	3.45 8 # 6	3.23 8 # 6	1.70 4 # 6
2	2.44 6 # 6	1.93 5 # 6	3.24 8 # 6	3.05 7 # 6	1.70 4 # 6
3	2.26 5 # 6	1.93 5 # 6	3.07 7 # 6	2.86 7 # 6	1.70 4 # 6
4	2.07 5 # 6	1.93 5 # 6	2.87 7 # 6	2.68 6 # 6	1.70 4 # 6
5	1.89 5 # 6	1.93 5 # 6	2.85 7 # 6	2.59 6 # 6	1.70 4 # 6
6	1.78 4 # 6	1.93 5 # 6	2.85 7 # 6	2.59 6 # 6	1.70 4 # 6
7	1.78 4 # 6	1.93 5 # 6	2.85 7 # 6	2.59 6 # 6	1.70 4 # 6
ROOF	1.18 3 # 6	1.93 5 # 6	1.89 5 # 6	1.72 4 # 6	1.70 4 # 6

TABLE 4.3 COMPUTATION OF MAXIMUM SHEAR FORCES NEAR FACE OF COLUMNS

FLOOR LEVEL	CASES Maximum	END SPAN RIGHT SIDE kips	END SPAN LEFT SIDE kips	INTERMEDIATE SPAN LEFT & RIGHT SIDES kips
1	a	54.45	40.24	47.34
	b	49.10	38.21	43.54
	c	25.48	20.96	23.22
	Max	54.45	40.24	47.34
2	a	54.45	40.24	47.34
	b	47.52	37.07	42.39
	c	24.31	19.79	22.05
	Max	54.45	40.24	47.34
3	a	54.45	40.24	47.34
	b	46.58	35.92	41.24
	c	23.14	18.62	20.88
	Max	54.45	40.24	47.34
4	a	54.45	40.24	47.34
	b	45.43	34.77	40.10
	c	21.97	17.45	19.71
	Max	54.45	40.24	47.34
5	a	54.45	40.24	47.34
	b	44.28	33.62	38.95
	c	16.28	16.28	18.54
	Max	54.45	40.24	47.34
6	a	54.45	40.24	47.34
	b	43.13	32.48	37.80
	c	19.63	15.11	17.37
	Max	54.45	40.24	47.34
7	a	54.45	40.24	47.34
	b	41.99	31.33	36.65
	c	18.46	13.94	16.20
	Max	54.45	40.24	47.34
ROOF	a	36.05	26.64	31.35
	b	27.32	20.27	23.80
	c	17.57	13.06	15.32
	Max	36.05	26.64	31.35

Loading Combinations

$a = 1.4\ V_D + 1.7\ V_L$ $\quad$ $b = .75\ (1.4\ V_D + 1.7\ V_L + 1.7\ V_W)$

$c = .9\ V_D + 1.3\ V_W$

i) From Right Support

$V_u - \phi V_c = 47.72 - 85 \times 30.36 = 21.92^K < 4 \phi V_c$

S_{max} Use #3 Stirrups

$d/2 = 8"$: $24"$: $S_{max} = \dfrac{.22 \times 60000}{50 \times 15} = 17.6"$

$S_{max} = 8"$ Governs $S_{max} = \dfrac{A_v f_y}{50\, b_w}$

$$S_{req'd} = \frac{A_v f_y d}{V_u/\phi - V_c} = \frac{.22 \times 60 \times 16}{\frac{47.72}{.85} - 30.36} = 8.19"$$

Thus $S_{max} = 8"$ Governs

Provide #3 Stirrups 1 @ 2", 15 @ 8" (throughout)

ii) From Left Support

$V_u - \phi V_c = 33.51 - .85 \times 30.36 = 7.8^K < 60.72$

$S_{max} = 8"$

$$S_{req'd} = \frac{22 \times 60 \times 16}{\frac{33.51}{.85} - 30.36} = 23" > S_{max}$$

Thus $S_{max} = 8"$ Governs

Provide #3 Stirrups 1 @ 2", 15 @ 8" (throughout)

b) Intermediate Spans

$V_{u_{max}}$ at a distance d from either support

$47.34 - \frac{16}{12} \times 5.05 = 40.61^K$

$V_c = 30.36^K$ $V_u - \phi V_c = 40.61 - .85 \times 30.36 = 14.81^K < 60.72^K$

$S_{max} = 8"$

$$S_{req'd} = \frac{.22 \times 60 \times 16}{\frac{40.61}{.85} - 30.36} = 12" > S_{max}$$

Provide #3 Stirrup 1 @ 2", 15 @ 8" from either support.

4.8.2. Roof Beams $W_u = 3.344$ K/ft

a) End Span $V_u = 1.15 \times .5 \times 3.344 \times 18.75 = 36.05$ K(Int)
$V_u = .85 \times .5 \times 3.344 \times 18.5 = 26.64$ K (Ext)

i) $V_{u_{max}}$ at a distance d from right support

$$36.05 - 3.44 \times \frac{16}{12} = 31.59^K$$

$V_c = 30.36^K$

Provide #3 Stirrups 1 @ 2", 15 @ 8"

ii) $V_{u_{max}}$ at a distance d from left support

$$26.64 - 3.344 \times \frac{16}{12} = 22.18^K$$

$1/2\phi V_c = 12.9^K$ Stirrups are Reqd.

Provide #3 Stirrups 1 @ 2", 15 @ 8"

b) Intermediate Span $V_u = .5 \times 3344 \times 18.75 = 31.35$ K

$V_{u_{max}}$ at a distance d from either support

$$31.35 - 3.344 \times \frac{16}{12} = 26.90^K > 12.9^K = \frac{1}{2}\phi V_c$$

Stirrups are Reqd.

Provide #3 Stirrups 1 @ 2", 15 @ 8"

4.9 Moments Due to Dead and Live Load For Columns

4.9.1 Columns Floor 1 - Floor 7

Case 1

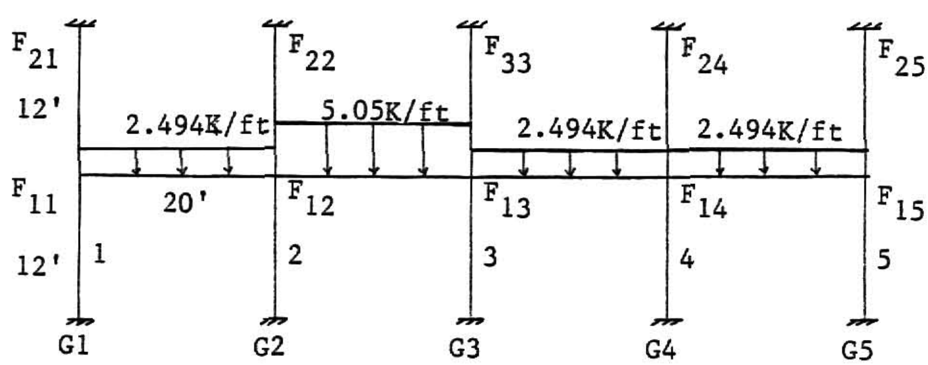


FIG. 4.6a MOMENT DISTRIBUTION Case 1

Stiffnesses: *

Columns: $I/L = 60.750\ in^3$

Beam: $I/L = 30.375\ in^3$

Distribution Factors:

Corner Supports Columns = 0.4 Beams = 0.2

Intermediate Supports Columns = 0.333 Beams = 0.167

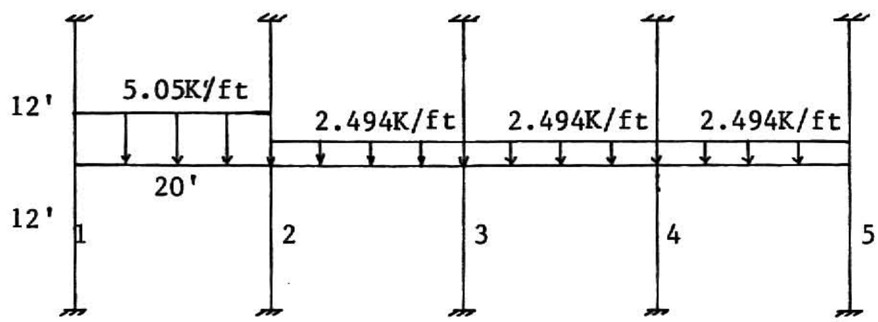
TABLE 4.4 Moment Table Case 1 (Dead and Live Load)

COLUMN NO.	UPPER END K-ft	LOWER END K-ft
#1	+30.41	+15.20
#2	+28.43	+14.22
#3	-31.19	-15.59
#4	+ 5.34	+ 2.67
#5	-33.76	-16.88

TABLE 4.5 Moment Table Case 2 (Dead and Live Load)

COLUMN NO.	UPPER END K-ft	LOWER END K-ft
#1	+65.05	+32.53
#2	-33.97	-16.99
#3	+ 2.60	+ 1.30
#4	+ 2.58	+ 1.29
#5	-33.53	-16.77

Case 2



4.6b MOMENT DISTRIBUTION Case 2

*For Columns $I = \frac{18^4}{12} = 8748\ in^4$ $L = 144''$ Thus $I/L = 60.75\ in^3$

For Beams $I = \frac{15 \times 18^3}{12} = 7290\ in^4$ $L = 240''$ Thus $I/L = 30.375\ in^3$

4.9.2. Column Floor 7 - Roof

Case 1

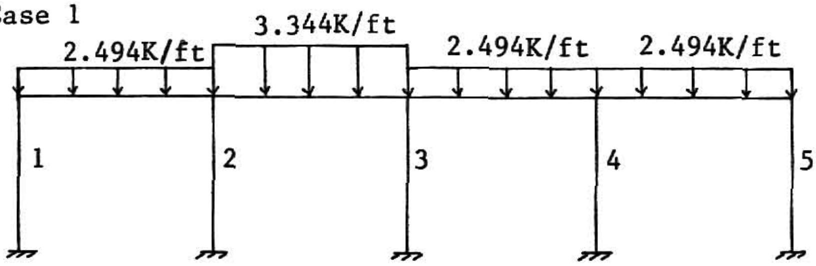


FIG. 4.7a MOMENT DISTRIBTION ROOF
Case 1

Distribution Factor:

Corner Supports Columns = 0.667 Beams = 0.333

Intermediate Supports Columns = 0.50 Beams = 0.25

Case 2

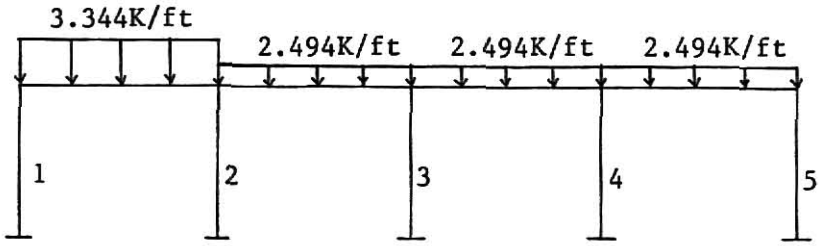


FIG. 4.7b MOMENT DISTRIBUTION ROOF
Case 2

TABLE 4.6

Moment Table (Roof): Case 1

COLUMNS NO.	UPPER END K-ft	LOWER END K-ft
#1	+53.92	+26.96
#2	+ 9.54	+ 4.77
#3	-16.40	- 8.20
#4	+ 8.92	+ 4.46
#5	-56.88	-28.44

TABLE 4.7

Moment Table (Roof): Case 2

COLUMNS NO.	UPPER END K-ft	LOWER END K-ft
#1	+78.21	+39.10
#2	-24.20	-12.10
#3	+ 2.06	+ 1.03
#4	+ 6.71	+ 3.35
#5	-56.58	-28.29

The computations of moments obtained from moment distribution (shown in Table 4.1 to Table 4.7) are shown in Appendix C.

4.10 Column Design

Columns have been designed in accordance with the new specifications of the ACI code (1). An initial assumption of 15" x 15" was made for the column dimension which is realized at this stage to be insufficient. Column dimensions are assumed as 18" x 18" except for the two top floors where the column dimensions remain unchanged. To use the alignment charts (8) for a particular column, ψ factors are computed at each end of the column. The ψ factor at one end of the column equals the sum of the stiffnesses ($\Sigma EI/l$) of the column meeting at the joint, including the column in question, divided by the sum of all the stiffnesses ($\Sigma EI/l$) of the beams meeting at the joint. Should one end of the column be pinned, ψ is theoretically equal to ∞, and if fixed $\psi = 0$. As a perfectly fixed end is practically impossible to have ψ is usually taken as 1.0 at the base connection. One of the two values is called ψ_A and the other ψ_B. After these values are computed, the effective length factor k is obtained by placing a straight edge between ψ_A and ψ_B.

It can be observed that the ψ factors used to enter the alignment charts and thus the resulting effective length factors are dependent on the relative stiffnesses of the compression and flexural members. This brings up the question as to what moment of inertia should be used in determining the ψ values. One acceptable practice is to use gross moments of inertia for the columns and 50% of gross moment of inertia for the flexural members (8).

According to ACI - 10.11.3, the radius of gyration r is equal to 0.3 times the overall dimension in the direction stability is being considered for rectangular compression members. Also slenderness effects shall be considered when $k\ell_u/r > 22$.

4.10.1 Moment Magnification

Compression members shall be designed using the factored axial load P_u and a magnified factored moment M_c

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \qquad 4\text{-}1$$

where
$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} > 1.0 \qquad 4\text{-}2$$

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{\phi \Sigma P_c}} > 1.0 \qquad 4\text{-}3$$

and
$$P_c = \frac{\Pi^2 EI}{(K l_u)^2} \qquad 4\text{-}4$$

In lieu of a more accurate calculation EI in Eq. 4-4 P_c may be computed conservatively as

$$EI = \frac{E_c I_g/2.5}{1 + \beta_d} \qquad 4\text{-}5$$

β_d is the absolute value of the ratio of the maximum factored dead load moment to the maximum factored total load moment.

For non sidesway moment magnification computation β_d is assumed to be equal to 0.4 and for wind load and seismic load moment magnification $\beta_d = 0$ because these loads are not sustained loads (11).

TABLE 4.8 k VALUES AT VARIOUS LOCATION

COLUMN LOCATION	COLUMN DIMENSION	INTERMEDIATE COLUMNS			CORNER COLUMNS		
		ψ_A	ψ_B	k	ψ_A	ψ_B	k
Between G F & 1st F	18" x 18"	1.00	4.00	1.60	1.00	8.00	1.8
Between 1st F & 2nd F	18" x 18"	4.00	4.00	2.00	8.00	8.00	2.6
Between 2nd F & 3rd F	18" x 18"	4.00	4.00	2.00	8.00	8.00	2.6
Between 3rd F & 4th F	18" x 18"	4.00	4.00	2.00	8.00	8.00	2.6
Between 4th F & 5th F	18" x 18"	4.00	4.00	2.00	8.00	8.00	2.6
Between 5th F & 6th F	18" x 18"	4.00	2.96	1.92	8.00	5.93	2.5
Between 6th F & 7th F	15" x 15"	2.96	1.93	1.70	5.93	3.86	2.1
Between 7th F & Roof	15" x 15"	0.96	1.44	3.86	3.86	1.93	1.8

For the computation of k values the alignment charts (8) for unbraced columns were used.

TABLE 4.9 AXIAL LOAD ON INTERMEDIATE COLUMNS

Column Location	$1.4\ P_D^*$ kips	$1.7\ P_L^*$ kips	P_W kips	Case 1 $1.4\ P_D + 1.7\ P_L$ kips	Case 2 $.75\ (1.4\ P_D + 1.7\ P_L + 1.7\ P_W)$ kips	Case 3 $.9\ P_D + 1.3\ P_W$ kips
Between G F & 1st F	430.50	374	0	804.50	603.38	276.75
Between 1st F & 2nd F	376.69	323	0	699.69	524.77	242.16
Between 2nd F & 3rd F	322.88	272	0	594.88	446.16	207.57
Between 3rd F & 4th F	269.06	221	0	490.06	367.55	172.97
Between 4th F & 5th F	215.25	170	0	385.25	288.94	138.38
Between 5th F & 6th F	161.44	119	0	280.44	210.33	103.78
Between 6th F & 7th F	107.63	68	0	175.63	131.72	69.19
Between 7th F & Roof	53.81	17	0	70.81	53.11	34.59

*The computation of axial load on columns are shown in section 4.12, pg. 54.

TABLE 4.10 AXIAL LOAD ON CORNER COLUMNS

Column Location	$1.4\ P_D$ kips	$1.7\ P_L$ kips	P_W kips	Case 1 $1.4\ P_D + 1.7\ P_L$ kips	Case 2 $.75\ (1.4\ P_D + 1.7\ P_L + 1.7\ P_W)$ kips	Case 3 $.9\ P_D + 1.3\ P_W$ kips
Between G F & 1st F	231.00	187.00	25.43	418.00	321.41	156.69
Between 1st F & 2nd F	202.13	161.33	19.13	363.46	279.48	136.96
Between 2nd F & 3rd F	173.25	135.84	13.73	309.09	237.56	117.23
Between 3rd F & 4th F	144.38	110.33	9.23	254.71	195.62	97.50
Between 4th F & 5th F	115.50	84.84	5.63	200.34	153.70	77.76
Between 5th F & 6th F	86.63	59.33	2.93	145.96	111.77	58.03
Between 6th F & 7th F	57.75	33.84	1.13	91.59	69.84	38.30
Between 7th F & Roof	28.88	8.33	0.23	37.21	28.19	18.86

The computation of axial load on columns are shown in section 4.12, pg. 54.

Computation of ψ-values for Intermediate Columns

Between G F and 1st F

$\psi_A = 1.0$ (Fixed) $\qquad \psi_B = \dfrac{2 \times 8748/144}{.5 \times 2 \times 7290/240} = 4$

Between 1st F & 5th F

$\psi_A = 4.0 \qquad \psi_B = 4.0$

Between 5th F & 6th F

$\psi_A = 4.0 \qquad \psi_B = \dfrac{8748 + 4218.75/144}{2 \times .5 \times 7290/240} = 2.96$

Between 6th & 7th F

$\psi_A = 4.0 \qquad \psi_B = 1.93$

Between 7th F & Roof

$\psi_A = 1.93 \qquad \psi_B = .96$

Similarly ψ-values have been computed for corner columns and the resulting k-values are shown in Table 5.8.

4.11 Spacing of Ties

When tied columns are used, the ties should not be less than #3 bars provided the longitudinal bars are #10 or smaller. The minimum size for ties is #4 bars when the longitudinal bars used are larger than #10 bars. The center-to-center spacing of ties shall not be more than 16 times the diameter of longitudinal bars, 48 times the diameter of the ties, nor the least lateral dimension of the column. The ties must be arranged so that every corner and alternate longitudinal bar have lateral support provided by the corner of a tie have an included angle not greater than 135°. No bars can be located a greater distance than 6" clear on either side from such a laterally supported bar. These requirements are stated in section 7.10.5 of the ACI Code.

48-φ	Ties:	16-φ	Longitudinal Bars:
#3	18"	#7	14.0"
		#8	16.0"
#4	24"	#9	18.0"
		#10	20.0"
		#11	22.5"

Least Lateral Dimension:

Floor 1 - Floor 6 = 18"

Floor 7 - Roof = 15"

4.12 Computation of Axial Load on Columns

4.12.1. Column Between Ground Floor and First Floor

The computations of Axial Load on Intermediate and corner columns between Ground Floor and First Floor are shown below. In a similar manner the axial loads on other stories columns were computed and the results are shown in Tables 4.9 and 4.10 on pages 51 and 52 respectively.

The column dimensions were initially assumed as 15" x 15" for the computation of dead loads. At the design stage the lower stories columns were increased to 18" x 18". This slight increase in axial loads was neglected. The beam dimensions are 15" x 18" and the slab is 6" thick.

a) Intermediate Columns:

i) Weight of Slab = 8 x 20 ft x 20 ft x 0.5 ft x .15 K/ft^3 = 240 K

ii) Weight of Column = 8×1.25 ft $\times 1.25$ ft $\times 12$ ft $\times .15$ K/ft^3 = 22.5 K

iii) Weight of Beam = 8×20 ft $\times 1.5$ ft $\times 1.25$ ft $\times .15$ K/ft^3 = 45 K

P_D = 307.5 K

1.4 P_D = 430.5^K (Refer to table 4.9, pg. 51)

iv) Live Load = 7×20 ft $\times 20$ ft $\times 0.075$ K/ft^2
= 210 K

v) Snow load = 1×20 ft $\times 20$ ft $\times 0.025$ K/ft^2
= 10 K

P_L = 220K

1.7 P_L = 374 K (Refer to Table 4.9, pg. 51)

Since the shear force has opposite signs at interior supports P_W = 0.

b) Corner Columns

i) Weight of Slab = 8×20 ft $\times 10$ ft $\times 0.5$ ft $\times .15$ K/ft^3 = 120 K

ii) Weight of Columns = 8×1.25 ft $\times 1.25$ ft $\times 12$ ft $\times .15$ K/ft^3 = 22.5 K

iii) Weight of beams = 8×10 ft $\times 1.25$ ft $\times 1.5$ ft $\times .15$ K/ft^3 = 22.5 K

P_D = 165 K

1.4 P_D = 231 K (Refer to Table 4.10, pg. 52)

iv) Live Load = 7×20 ft $\times 10$ ft $\times .075$ K/ft^2
= 105 K

v) Snow Load = 1×20 ft $\times 10$ ft $\times .025$ K/ft^2
= 5 K

P_L = 110 K

1.7 P_L = 187 K (Refer to Table 4.10, pg. 52)

From Fig. 4.3

P_W = 6.3 + 5.4 + 4.7 + 3.6 + 2.7 + 1.8 + 0.9 + .23
= 25.43 K

P_W = 25.43 K (Refer to Table 4.10, pg. 52)

TABLE 4.11 COMPUTATION OF M_c FOR INTERMEDIATE COLUMNS

Case 1. $M_{2b} = 1.4\ M_D + 1.7\ M_L$, $M_{2s} = 0$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	33.97	0	1.60	9.01×10^6	NOT REQD	804.50	3249.18	1531.6	NOT REQD	2.10	NOT REQD	71.34
Between 1st F & 2nd F	33.97	0	2.00	9.01×10^6	"	699.69	2826.0	980.2	"	3.40	"	115.50
Between 2nd F & 3rd F	33.97	0	2.00	9.01×10^6	"	594.88	2402.82	980.2	"	2.50	"	84.93
Between 3rd F & 4th F	33.97	0	2.00	9.01×10^6	"	490.06	1979.6	980.2	"	2.00	"	67.94
Between 4th F & 5th F	33.97	0	2.00	9.01×10^6	"	385.25	1556.43	980.2	"	1.65	"	56.05
Between 5th F & 6th F	33.97	0	1.92	9.01×10^6	"	280.44	1133.24	1063.6	"	1.40	"	47.56
Between 6th F & 7th F	33.97	0	1.70	4.34×10^6	"	175.63	710.06	653.6	"	1.40	"	47.56
Between 7th F & Roof	24.20	0	1.44	4.34×10^6	"	70.81	286.85	910.8	"	1.09	"	26.38

Refer to Appendix C for Sample Design Calculation

TABLE 4.12 COMPUTATION OF M_c FOR INTERMEDIATE COLUMNS

Case 2. $M_{2b} = .75(1.4\ M_D + 1.7\ M_L)$ $M_{2s} = 1.275\ M_W$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in²	EI_{2s} K/in²	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	25.48	86.06	1.60	9.01×10^6	12.61×10^6	603.38	2452.96	1531.6	10717.70	1.65	1.30	153.9
Between 1st F & 2nd F	25.48	74.59	2.00	9.01×10^6	12.61×10^6	524.77	2133.27	980.2	6859.30	2.15	1.45	162.9
Between 2nd F & 3rd F	25.48	63.11	2.00	9.01×10^6	12.61×10^6	446.16	1813.60	980.2	6859.30	1.84	1.36	132.7
Between 3rd F & 4th F	25.48	51.64	2.00	9.01×10^6	12.61×10^6	367.55	1493.89	980.2	6859.30	1.60	1.29	107.4
Between 4th F & 5th F	25.48	40.16	2.00	9.01×10^6	12.61×10^6	288.94	1174.22	980.2	6859.30	1.42	1.21	84.8
Between 5th F & 6th F	25.48	28.69	1.92	9.01×10^6	12.61×10^6	210.33	854.53	1063.6	7442.85	1.25	1.13	64.3
Between 6th F & 7th F	25.48	17.21	1.70	4.34×10^6	6.08×10^6	131.72	534.84	653.5	4557.5	1.25	1.13	51.3
Between 7th F & Roof	18.15	5.74	1.44	4.34×10^6	6.08×10^6	53.11	215.71	910.8	6379.8	1.06	1.04	25.2

Refer to Appendix C for Sample Design Calculations

TABLE 4.13 COMPUTATION OF M_c FOR INTERMEDIATE COLUMNS

Case 3. $M_{2b} = .9\ M_D$ $M_{2s} = 1.3\ M_W$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	10.80	87.75	1.60	9.01×10^6	12.61×10^6	276.75	1143.6	1531.6	10717.70	1.22	1.12	111.5
Between 1st F & 2nd F	10.80	76.05	2.00	9.01×10^6	12.61×10^6	242.16	1000.4	980.2	6859.30	1.33	1.17	103.3
Between 2nd F & 3rd F	10.80	64.35	2.00	9.01×10^6	12.61×10^6	207.57	857.2	980.2	6859.30	1.27	1.14	87.1
Between 3rd F & 4th F	10.80	52.65	2.00	9.01×10^6	12.61×10^6	172.97	713.9	980.2	6859.30	1.21	1.12	72.0
Between 4th F & 5th F	10.80	40.95	2.00	9.01×10^6	12.61×10^6	138.38	570.7	980.2	6859.30	1.16	1.09	57.2
Between 5th F & 6th F	10.80	29.25	1.92	9.01×10^6	12.61×10^6	103.78	427.4	1063.6	7442.85	1.11	1.06	43.0
Between 6th F & 7th F	10.80	17.55	1.70	4.34×10^6	6.08×10^6	69.19	284.2	653.5	4557.50	1.12	1.07	30.9
Between 7th F & Roof	11.61	5.85	1.44	4.34×10^6	6.08×10^6	34.59	141.5	910.8	6379.80	1.04	1.03	18.1

Refer to Appendix C for Sample Design Calculations

TABLE 4.14 COMPUTATION OF M_c FOR CORNER COLUMNS

Case 1. $M_{2b} = 1.4\ M_D + 1.7\ M_L \quad M_{2s} = 0$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	65.05	0	1.8	9.01×10^6	NOT REQD	418.00	NOT REQD	1210.14	NOT REQD	1.53	NOT REQD	99.5
Between 1st F & 2nd F	65.05	0	2.6	9.01×10^6	"	363.46	"	580.00	"	2.68	"	174.3
Between 2nd F & 3rd F	65.05	0	2.6	9.01×10^6	"	309.09	"	580.00	"	2.14	"	139.2
Between 3rd F & 4th F	65.05	0	2.6	9.01×10^6	"	254.71	"	580.00	"	1.78	"	115.8
Between 4th F & 5th F	65.05	0	2.6	9.01×10^6	"	200.34	"	580.00	"	1.53	"	99.5
Between 5th F & 6th F	65.05	0	2.5	9.01×10^6	"	145.96	"	627.30	"	1.30	"	84.6
Between 6th F & 7th F	65.05	0	2.1	4.34×10^6	"	91.59	"	428.30	"	1.27	"	82.6
Between 7th F & Roof	78.21	0	1.8	4.34×10^6	"	37.21	"	582.90	"	1.07	"	83.7

Refer to Appendix C for Sample Design Calculations

TABLE 4.15 COMPUTATION OF M_c FOR CORNER COLUMNS

Case 2. $M_{2b} = 0.75(1.4\ M_D + 1.7\ M_L)$ $M_{2s} = 1.275\ M_W$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	48.7	43.03	1.8	9.01×10^6	12.61×10^6	321.41	2452.96	1210.14	8468.3	1.36	1.41	126.9
Between 1st F & 2nd F	48.7	37.30	2.6	9.01×10^6	12.61×10^6	279.48	2133.27	580.00	4058.8	1.93	2.11	172.7
Between 2nd F & 3rd F	48.7	31.60	2.6	9.01×10^6	12.61×10^6	237.56	1813.60	580.00	4058.8	1.69	1.81	139.5
Between 3rd F & 4th F	48.7	25.8	2.6	9.01×10^6	12.61×10^6	195.62	1493.89	580.00	4058.8	1.51	1.58	114.3
Between 4th F & 5th F	48.7	20.1	2.6	9.01×10^6	12.61×10^6	153.70	1174.22	580.00	4058.8	1.36	1.41	94.6
Between 5th F & 6th F	35.7	14.34	2.5	9.01×10^6	12.61×10^6	111.77	854.53	627.30	4390.0	1.22	1.24	77.2
Between 6th F & 7th F	48.7	8.6	2.1	4.34×10^6	6.08×10^6	69.84	534.84	428.30	3000.0	1.19	1.22	68.4
Between 7th F & Roof	58.8	2.9	1.8	4.34×10^6	6.08×10^6	28.19	215.71	582.9	4083.1	1.05	1.06	64.8

Refer to Appendix C for Sample Design Calculations

TABLE 4.16 COMPUTATION OF M_c FOR CORNER COLUMNS

Case 3. $M_{2b} = .9\ M_D \quad M_{2s} = 1.3\ M_W$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	20.6	43.88	1.8	9.01×10^6	12.61×10^6	156.69	1143.6	1210.14	8468.3	1.15	1.16	74.6
Between 1st F & 2nd F	20.6	38.03	2.6	9.01×10^6	12.61×10^6	136.96	1000.4	580.00	4058.8	1.31	1.33	77.6
Between 2nd F & 3rd F	20.6	32.18	2.6	9.01×10^6	12.61×10^6	117.23	857.2	580.00	4058.8	1.25	1.27	66.6
Between 3rd F & 4th F	20.6	26.33	2.6	9.01×10^6	12.61×10^6	97.50	713.9	580.00	4058.8	1.20	1.21	56.6
Between 4th F & 5th F	20.6	20.48	2.6	9.01×10^6	12.61×10^6	77.76	570.7	580.00	4058.8	1.15	1.16	47.4
Between 5th F & 6th F	20.6	14.63	2.5	9.01×10^6	12.61×10^6	58.03	427.4	627.30	4390.00	1.10	1.11	38.9
Between 6th F & 7th F	20.6	8.78	2.1	4.34×10^6	6.08×10^6	38.30	284.2	428.30	3000.00	1.10	1.10	32.3
Between 7th F & Roof	37.6	2.93	1.8	4.34×10^6	6.08×10^6	18.86	141.5	582.9	4083.10	1.03	1.04	41.8

Refer to Appendix C for Sample Design Calculations

TABLE 4.17 DESIGN OF INTERMEDIATE COLUMNS

Column Location	CASE 1			CASE 2			CASE 3			ρ_{CRIT}	As Reqd in^2 bars provided	Ties and Spacing
	P_u-K	M_c-K'	ρ_{REQD}	P_u-K	M_c-K'	ρ_{REQD}	P_u-K	M_c-K'	ρ_{REQD}			
Between G F & 1st F (18" x 18")	804.50	71.34	.0439	603.38	153.9	.0439	276.75	111.5	.0053	.0439	14.2 4 # 18	#4 @ 18" c/c
Between 1st F & 2nd F (18" x 18")	699.69	115.50	.0329	524.77	162.9	.0329	242.16	103.3	.0053	.0329	10.7 6 " 14	#4 @ 18" c/c
Between 2nd F & 3rd F (18" x 18")	594.88	84.93	.0263	446.16	132.7	.0209	207.57	87.1	.0053	.0263	8.5 4 " 14	#4 @ 18" c/c
Between 3rd F & 4th F (18" x 18")	490.06	67.94	.0107	367.55	107.7	.0053	172.97	72.0	.0053	.0107	3.6 4 # 9	#4 @ 18" c/c
Between 4th F & 5th F (18" x 18")	385.25	56.05	.0053	288.94	84.8	.0053	138.38	57.2	.0053	.0100	3.24 4 # 9	#4 @ 18" c/c
Between 5th F & 6th F (18" x 18")	280.44	47.56	.0053	210.33	64.3	.0053	103.78	43.0	.0053	.0100	3.24 4 # 9	#4 @ 18" c/c
Between 6th F & 7th F (15" x 15")	175.63	47.56	.0053	131.72	51.3	.0053	69.19	30.9	.0053	.0100	2.25 4 # 8	#4 @ 15" c/c
Between 7th F & Roof (15" x 15")	70.81	26.38	.0053	53.11	25.2	.0053	34.59	18.1	.0053	.0100	2.25 4 # 8	#4 @ 15" c/c

TABLE 4.18 DESIGN OF CORNER COLUMNS

Column Location	CASE 1			CASE 2			CASE 3			ρ_{CRIT}*	As Reqd in^2 bars provided	Ties and Spacing
	P_u-K	M_c-K'	ρ_{REQD}	P_u-K	M_c-K'	ρ_{REQD}	P_u-K	M_c-K'	ρ_{REQD}			
Between G F & 1st F (18" x 18")	418.00	99.5	.0053	321.41	126.9	.0080	156.69	74.6	.0053	.0100	3.24 4 # 9	#4 @ 18" c/c
Between 1st F & 2nd F (18" x 18")	363.46	174.3	.0107	279.48	172.7	.0107	136.96	77.6	.0053	.0107	3.60 4 # 9	#4 @ 18" c/c
Between 2nd F & 3rd F (18" x 18")	309.09	139.2	.0080	237.56	139.5	.0053	117.23	66.6	.0053	.0100	3.24 4 # 9	#4 @ 18" c/c
Between 3rd F & 4th F (18" x 18")	254.71	115.8	.0053	195.62	114.3	.0053	97.50	56.6	.0053	.0100	3.24 4 # 9	#4 @ 18" c/c
Between 4th F & 5th F (18" x 18")	200.34	99.5	.0053	153.70	94.6	.0053	77.76	47.4	.0053	.01	3.24 4 # 9	#4 @ 18" c/c
Between 5th F & 6th F (18" x 18")	145.96	84.6	.0053	111.77	77.2	.0053	58.03	38.9	.0053	.01	3.24 4 # 9	#4 @ 18" c/c
Between 6th F & 7th F (15" x 15")	91.59	82.6	.0053	69.84	68.4	.0053	38.30	32.3	.0053	.01	2.25 4 # 8	#4 @ 15" c/c
Between 7th F & Roof (15" x 15")	37.21	83.7	.0053	28.19	64.8	.0053	18.86	41.8	.0053	.01	2.25 4 # 8	#4 @ 15" c/c

* $\rho_{min} = .01$ (ACI 10.9.1)

CHAPTER 5

DESIGN WITH SEISMIC CONSIDERATIONS

5.1 Load Combinations

As stated in Art 2.7.2 of this report, the following two load combinations have been used.

a) 1.4 (D + L + E)   b) .9 D + 1.4 E

5.2 Loading

The live load is assumed to be the same as it was for non-seismic design.(75 lbs/ft^2) The snow load for the roof beam design has also been kept unchanged.(25 lbs/ft^2) Since the design of slabs has not been dealt in this report the slab thickness has also not been changed (6") for seismic design considerations.

As an initial trial the member dimension have been kept the same as they were for non-seismic design. The major portion of dead and live load moments and shears is due to the self-weight of the slab and the assumed live load. Therefore the slight increase in these moments and shears due to increase in members dimensions has been neglected.

5.3 Computation of Design Moments for Beams

5.3.1. Critical Moments for Beam #1

a) Ratio between 1.4 D + 1.4 L and 1.4 D + 1.7 L = 0.91

End Span Int. Negative = $.91 \times 110.96^* + 1.4 \times 276.6$
$= 488.18$ K'

End Span Positive = $.91 \times 126.81^* = 115.35$ K'

End Span Ext. Negative = $.91 \times 177.54^* + 1.4 \times 276.6$
$= 548.74$ K'

Int. Span Negative = $.91 \times 161.4^* + 1.4 \times 276.6$
$= 534.06$ K'

*Refer to pg. 32 for the computations of these dead load and live load moments.

Int. Span Positive = .91 x 110.96* = 100.94 K'

End Support Positive = -.91 x 110.96* + 1.4 x 276.6
= 286.27 K'

Int. Support Positive = -.91 x 161.4* + 1.4 x 276.6
= 240.37 K'

b) Ratio between 0.9 D and 1.4 D + 1.7 L = 0.31745

End Span Int. Negative = .31745 x 110.96* + 1.4 x 276.6
= 422.46 K'

End Span Positive = .31745 x 126.81* = 40.26 K'

End Span Ext. Negative = .31745 x 177.54* + 1.4 x 276.6
= 443.6 K'

Int. Span Negative = .31745 x 161.4* + 1.4 x 276.6
= 438.48 K'

Int. Span Positive = .31745 x 110.96* = 35.22 K'

End Support Positive = -.31745 x 100.96* + 1.4 x 276.6
= 352.01 K'

Int. Support Positive = -.31745 x 161.4* + 1.4 x 276.6
= 336.0 K'

The critical moments for each location are shown in Table 5.1. Similar procedure was carried out for Beam #2 to Roof Beam.

5.4 Beam Design

M_u = 548.74 K-ft (Negative) $\phi = 0.9$ $M_n = M_u/\phi$

b = 15" M_n = 548.74/0.9 = 609.71 K-ft

d = 22 " (d = 25 - 1.5 - 0.625 - 1.41/2 = 22.17")

M_n = 609.71 K-ft Total depth = 25" d' = 3"

$\rho_{max} = .0143$ $A_{s1} = .0143 \times 15 \times 22 = 4.72\ in^2$

$\frac{M_{u1}}{\phi bd^2} = .75$ $M_{u1} = .75 \times .9 \times 15 \times 22^2 = 408.4$ K'

$M_{n1} = 453.75$ K'

$M_{n2} = M_n - M_{n1} = 155.96$ K'

*Refer to pg. 32 for the computations of these dead load and live load moments.

TABLE 5.1 CRITICAL DESIGN MOMENTS FOR BEAMS

FLOOR	END SPAN INTERIOR NEG K-ft	END SPAN POSITIVE K-ft	END SPAN EXTERIOR NEG K-ft	INTERMEDIATE NEG K-ft	INTERMEDIATE POSITIVE K-ft	END SUPPORT POSITIVE K-ft	INT SUPPORT POSITIVE K-ft
1	488.18	115.35	548.74	534.06	100.94	352.01	336.00
2	469.91	115.35	530.47	515.79	100.94	333.74	317.73
3	438.5	115.35	499.06	484.38	100.94	302.33	286.32
4	395.74	115.35	456.30	441.62	100.94	259.57	243.56
5	340.17	115.35	400.73	386.05	100.94	204.00	187.99
6	272.22	115.35	332.78	318.10	100.94	136.35	120.04
7	191.91	115.35	252.47	237.79	100.94	55.74	39.73
Roof	94.17	80.35	136.13	126.13	70.31	---	---

Check if compression steel has yielded

$$a = \frac{4.72 \times 60}{.85 \times 4 \times 15} = 5.55'' \qquad c = 6.53''$$

$$\varepsilon'_s = \frac{6.53 - 3.0}{6.53} \times .003 = .00162 < .00207''$$

Therefore compression steel has not yielded.

$$f'_s = \frac{.00162}{.00207} \times 60 = 46.96 \text{ ksi}$$

$$A'_s \text{ reqd} = \frac{155.96 \times 12}{46.96\ (22-3)} = 2.10 \text{ in}^2$$

$$A_{s2} = \frac{2.10 \times 46.96}{60} = 1.64 \text{ in}_2$$

$A_s = 6.36 \text{ in}_2$ $\qquad A'_s = 2.10 \text{ in}_2$

(4.72 + 1.64)

Provide 5 # 10 bars at top

5 # 6 bars at bottom.

(Table 5.2 contains the reinforcement requirements at various locations for beams.)

5.5 Design For Shear (Refer to pg. 32 for dead and live load shears)

The two loading combinations being used are

a) $1.4\ V_D + 1.4\ V_L + 1.4\ V_E$ and b) $0.9\ V_D + 1.4\ V_E$

The resulting maximum shear forces are shown in Table 5.3.

TABLE 5.2 FLEXURAL REINFORCEMENT FOR BEAMS

FLOOR LEVEL (bxd)	END SPAN INT. NEG M A_s, REQD (in^2) STEEL PROVIDED	END SPAN POSITIVE M A_s, REQD (in^2) STEEL PROVIDED	END SPAN EXT. NEG M A_s, REQD (in^2) STEEL PROVIDED	INT SPAN NEG M A_s, REQD (in^2) STEEL PROVIDED	INT SPAN POSITIVE M A_s, REQD (in^2) STEEL PROVIDED	END SUPPORT POSITIVE M A_s, REQD (in^2) STEEL PROVIDED	INT SUPPORT POSITIVE M A_s, REQD (in^2) STEEL PROVIDED
1 (15x22)	A_s = 5.65 4 # 11 A_s' = 1.19 3 # 6	A_s = 1.33 3 # 6	A_s* = 6.36 5 # 10 A_s' = 2.10 5 # 6	A_s* = 6.19 5 # 10 A_s' = 1.88 5 # 6	A_s = 1.16 2 # 7	A_s = 3.98 4 # 9	A_s = 3.80 4 # 9
2 (15x22)	A_s = 5.44 4 # 11 A_s' = 0.92 2 # 6	A_s = 1.33 3 # 6	A_s = 6.15 5 # 10 A_s' = 1.82 5 # 6	A_s* = 5.98 6 # 9 A_s' = 1.61 4 # 6	A_s = 1.16 2 # 7	A_s = 3.77 4 # 9	A_s = 3.59 4 # 9
3 (15x22)	A_s = 5.07 5 # 9 A_s' = 0.45 2 # 4	A_s = 1.33 3 # 6	A_s = 5.78 5 # 10 A_s' = 1.36 3 # 6	A_s = 5.61 4 # 11 A_s' = 1.14 2 # 7	A_s = 1.16 2 # 7	A_s = 3.42 6 # 7	A_s = 3.24 6 # 7
4 (15x22)	A_s = 4.72 5 # 9	A_s = 1.33 3 # 6	A_s = 5.28 4 # 11 A_s' = 0.72 2 # 6	A_s = 5.11 4 # 10 A_s' = 0.50 2 # 5	A_s = 1.16 2 # 7	A_s = 2.93 5 # 7	A_s = 2.75 5 # 7
5 (15x22)	A_s = 3.93 4 # 9	A_s = 1.33 3 # 6	A_s = 4.62 6 # 8	A_s = 4.43 5 # 9	A_s = 1.16 2 # 7	A_s = 2.31 4 # 7	A_s = 2.13 4 # 7
6 (15x22)	A_s = 3.14 4 # 8	A_s = 1.33 3 # 6	A_s = 3.83 4 # 9	A_s = 3.66 4 # 9	A_s = 1.16 2 # 7	A_s = 1.54 4 # 6	A_s = 1.36 3 # 6
7 (15x22)	A_s = 2.21 3 # 8	A_s = 1.33 3 # 6	A_s = 2.91 4 # 8	A_s = 2.74 4 # 8	A_s = 1.16 2 # 7	A_s = 0.80 2 # 6	A_s = 0.80 2 # 6
Roof (15x16)	A_s = 1.42 4 # 6	A_s = 1.2 3 # 6	A_s = 2.05 5 # 6	A_s = 1.9 5 # 6	A_s = 1.06 2 # 7		

*Steel may have to be provided in two layers at these locations. With the bars provided the clear cover on each side is 1.29" with #4 stirrups.

TABLE 5.3 COMPUTATION OF MAXIMUM SHEAR FORCES NEAR FACE OF COLUMNS

FLOOR LEVEL	CASES MAXIMUM	END SPAN RIGHT SIDE KIPS	END SPAN LEFT SIDE KIPS	INTERMEDIATE SPAN LEFT & RIGHT SIDE KIPS
1	a	88.27	75.34	81.80
	b	56.01	51.49	52.40
	Max	88.27	75.34	81.80
2	a	86.45	73.52	79.98
	b	54.19	49.67	50.58
	Max	86.45	73.52	79.98
3	a	83.31	70.38	76.84
	b	51.05	46.53	47.44
	Max	83.31	70.38	76.84
4	a	79.03	66.10	72.56
	b	46.77	42.25	43.16
	Max	79.03	66.10	72.56
5	a	73.47	60.54	67.00
	b	41.21	36.69	37.60
	Max	73.47	60.54	67.00
6	a	66.68	53.75	60.21
	b	34.42	29.90	30.81
	Max	66.68	53.75	60.21
7	a	58.65	45.72	52.18
	b	26.39	21.87	22.78
	Max	58.65	45.72	52.18
Roof	a	37.00	27.96	32.48
	b	18.97	14.64	16.81
	Max	37.00	27.96	32.48

5.5.1 Beams on Floor 1 - Floor 4 $W_u = 4.60$ K/ft
($1.4 \times 1.78125 + 1.4 \times 1.5$)

a) End Span

$V_{u_{max}}$ at a distance d from right support

$$V_u = 88.27\text{ K} - \frac{22''}{12} \times 4.60\text{ K/ft} = 79.84\text{ K}$$

$V_{u_{max}}$ at a distance d from left support

$$V_u = 75.34\text{ K} - \frac{22''}{12} \times 4.60\text{ K/ft} = 66.91\text{ K}$$

$$V_c = 2\ \frac{\sqrt{4000} \times 15''}{1000} \times 22'' = 41.74\text{ K}$$

i) From Right Support

$$V_u - \phi V_c = 79.84\text{ K} - .85 \times 41.74\text{ K} = 44.36\text{ K} < 83.48\text{ K}$$

$$S_{max} = d/2 \qquad S = \frac{A_v\text{ in}^2 \times f_y\text{ ksi} \times d\text{ in}}{V_u\text{ K}/\phi - V_c\text{ K}}$$

$$S_{reqd} = \frac{.396'' \times 60\text{ ksi} \times 22''}{79.84\text{ K}/.85 - 41.74\text{ K}} = 10''$$

$V_{u_{max}}$ at 42" from right support

$$V_u = 88.27\text{ K} - \frac{42''}{12} \times 4.6\text{ K/ft} = 72.17\text{ K} \quad :$$

$$S_{reqd} = \frac{.396'' \times 60\text{ ksi} \times 22''}{72.17\text{ K}/.85 - 41.74\text{ K}} = 12''$$

Provide # 4 Stirrups 1 @ 2", 4 @ 10", 7 @ 11"
(throughout)

ii) From Right Support

$$V_u - \phi V_c = 66.91\text{ K} - .85 \times 41.74\text{ K} = 31.43\text{ K} < 83.48\text{ K}$$

$$S_{reqd} = \frac{.396'' \times 60\text{ ksi} \times 22''}{66.91\text{ K}/.85 - 41.74\text{ K}} = 14'' > 11''$$

Provide # 4 Stirrups 1 @ 2", 11 @ 11"
(throughout)

b) Intermediate Spans

$V_{u_{max}}$ at a distance d from either support

$$V_u = 81.80\text{ K} - \frac{22''}{12} \times 4.6\text{ K/ft} = 73.37\text{ K}$$

$$V_u = \phi V_c = 73.37\text{ K} - .85 \times 41.74\text{ K} = 37.9\text{ K} < 83.48\text{ K}$$

$$S_{reqd} = \frac{.396'' \times 60\text{ ksi} \times 22''}{73.37\text{ K}/.85 - 41.74\text{ K}} = 11.7'' > 11''$$

Provide # 4 Stirrups 1 @ 2", 11 @ 11"
from either supports

5.5.2 Beam on Floor 5 - Floor 7

a) End Span

i) $V_{u_{max}}$ at a d from right support

$$V_u = 73.47\ K - \frac{22''}{12} \times 4.6\ K/ft = 65.04\ K$$

$$S_{reqd} = \frac{.22'' \times 60\ ksi \times 22''}{65.04\ K/.85 - 41.74\ K} = 8.35''$$

$V_{u_{max}}$ at 42" from right support = 57.37 K

$$S_{reqd} = \frac{.22'' \times 60\ ksi \times 22''}{65.04\ K/.84 - 41.74\ K} = 8.35''$$

$V_{u_{max}}$ at 42" from right support = 57.37 K

S_{reqd} = 11.27"

Provide # 3 Stirrups 1 @ 2", 5 @ 8", 7 @ 11"
(throughout)

ii) $V_{u_{max}}$ at d from left support

$$V_u = 60.54\ K - \frac{22''}{12} \times 4.6\ K/ft = 52.11\ K$$

S_{reqd} = 14.8" > 11"

Provide # 3 Stirrups 1 @ 2", 11 @ 11"
(throughout)

b) Intermediate Span

$V_{u_{max}}$ at d from supports = 67 K - 8.43 K = 58.57 K

S_{reqd} = 10.7"

Provide # 3 Stirrups 1 @ 2", 3 @ 10", 8 @ 11"
from either side

5.5.3 Roof Beam

a) End Span

i) From Left Support W_u = 3.19 K/ft
(1.4 x 1.78125 + 1.4 x .5)

V_c = 30.36 K

$V_{u_{max}}$ = 37 K - 4.25 K = 32.75 K

$$S_{reqd} = \frac{.22'' \times 60\ ksi \times 16''}{32.75\ K/.85 - 30.36\ K} = 25'' > 8''$$

Provide # 3 Stirrup 1 @ 2", 15 @ 8"
(throughout)

ii) From Right Support

$V_{u_{max}} = 27.96 \text{ K} - 4.25 \text{ K} = 23.71 \text{ K}$

$1/2\phi V_c = 12.9 \text{ K} < 23.71 \text{ K}$

Provide # 3 Stirrups 1 @ 2", 15 @ 8" (throughout)

b) Intermediate Spans

$V_{u_{max}} = 32.48 \text{ K} - 4.25 \text{ K} = 28.23 \text{ K}$

$1/2\phi V_c = 12.9 \text{ K} < 28.23 \text{ K}$

Provide # 3 Stirrups 1 @ 2", 15 @ 8" (throughout)

5.5.4 Special Considerations in Shear Design

Maximum stirrup spacing of d/2 is required throughout in order to provide effective nominal web reinforcement at all sections so that shear failures will not unexpectedly occur in these areas. Inclined stirrups, which are effective in one direction only, should not be used unless the analysis indicates that such shear stress reversal will not occur. Stirrup-ties, rather than U-shaped stirrups, provide additional confinement and must be used at specific locations. They provide the confinement necessary to develop effective inelastic hinges. During the formation of inelastic hinges, yielding occurs in the tensile reinforcement and cracks open up in concrete. Under reversed moments, the stresses in the reinforcement become compressive.

5.6 Column Design

Columns have been designed for the two cases of seismic load combinations as stated in section 5.1.

The design of columns have been carried out according to the new provision of the ACI Code explained in section 5.10 of this report. The value of β_d is assumed as 0.3 for non-side-sway moments in this case.

As an initial trial the lower four story columns have been assumed as 20" x 20" because of excessive moments in these floors due to seismic loading. Other story columns have been assumed to have the same dimensions as they had in non-seismic loading. The slight increase in self-weight of columns have been neglected.

5.6.1 Computations for ψ-Values

a) Moment of Inertias

Columns	20" x 20"	I_g =	13333.3 in^4
Columns	18" x 18"	I_g =	8748.0 in^4
Columns	15" x 15"	I_g =	4218.75 in^4
Beams	15" x 25"	I_g =	19531.25 in^4
Beams	15" x 18"	I_g =	7290.0 in^4

b) k-Value Between G F and 1st F

$_A = 1.0$ (fixed)

$$_B = \frac{2 \times 13333.3/144}{2 \times .5 \times 19531.25/240} = 2.27$$

k-value obtained from charts (8) = 1.5

Similar k-values have been obtained for other locations and have been tabulated in Table 5.6.

TABLE 5.4 AXIAL LOAD ON INTERMEDIATE COLUMNS

COLUMN LOCATION	1.4 P_D KIPS	1.4 P_L KIPS	1.4 P_E KIPS	CASE 1 1.4 (P_D + P_L + P_E) KIPS	CASE 2 .9 P_D + 1.4 P_E KIPS
Between G F & 1st F	430.50	308	0	738.50	276.75
Between 1st F & 2nd F	376.69	266	0	642.69	242.16
Between 2nd F & 3rd F	322.88	224	0	546.88	207.57
Between 3rd F & 4th F	269.06	182	0	451.06	172.97
Between 4th F & 5th F	215.25	140	0	355.25	138.38
Between 5th F & 6th F	161.44	98	0	259.44	103.78
Between 6th F & 7th F	107.63	56	0	163.63	69.19
Between 7th F & Roof	53.81	14	0	67.81	34.59

The computation of axial load on columns are shown on pg. 54.

TABLE 5.5 AXIAL LOAD ON CORNER COLUMNS

COLUMN LOCATION	1.4 P_D KIPS	1.4 P_L KIPS	1.4 P_E KIPS	CASE 1 1.4 $(P_D + P_L + P_E)$ KIPS	CASE 2 .9 P_D + 1.4 P_E KIPS
Between G F & 1st F	231.00	153.87	189.57	574.44	338.07
Between 1st F & 2nd F	202.13	132.86	152.67	487.66	282.61
Between 2nd F & 3rd F	173.25	111.87	115.77	400.89	227.15
Between 3rd F & 4th F	144.38	90.86	82.01	317.25	174.83
Between 4th F & 5th F	115.50	69.87	52.53	237.90	126.78
Between 5th F & 6th F	86.63	48.86	28.61	164.10	84.30
Between 6th F & 7th F	57.75	27.86	11.48	97.09	48.61
Between 7th F & Roof	28.88	6.86	2.39	38.13	20.96

The computations of axial load on columns are shown on pg. 54.

TABLE 5.6 k-VALUES AT VARIOUS LOCATIONS

COLUMN LOCATION	COLUMN DIMENSION	INTERMEDIATE COLUMNS			CORNER COLUMNS		
		Ψ_A	Ψ_B	k	Ψ_A	Ψ_B	k
Between G F & 1st F	20" x 20"	1.00	2.27	1.50	1.00	1.29	1.35
Between 1st F & 2nd F	20" x 20"	2.27	2.27	1.76	1.29	1.29	1.40
Between 2nd F & 3rd F	20" x 20"	2.27	2.27	1.74	1.29	1.29	1.40
Between 3rd F & 4th F	20" x 20"	2.27	2.27	1.74	1.29	1.29	1.40
Between 4th F & 5th F	18" x 18"	2.13	1.69	1.55	1.06	0.85	1.29
Between 5th F & 6th F	18" x 18"	1.69	1.69	1.50	0.85	0.85	1.25
Between 6th F & 7th F	15" x 15"	1.25	0.81	1.30	0.63	0.41	1.15
Between 7th F & Roof	15" x 15"	0.81	0.96	1.28	0.41	0.48	1.14

For the computation of k values the alignment charts (8) for unbraced columns were used. (Refer pg. 73 for ψ calculations.)

TABLE 5.7 COMPUTATION OF M_c FOR INTERMEDIATE COLUMNS

Case 1 $M_{2b} = 1.4\ M_D + 1.4\ M_L$ $M_{2s} = 1.4\ M_E$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	30.91	393.12	1.50	14.79×10^6	19.23×10^6	738.5	3364.4	3153.7	20502	1.31	1.20	512.2
Between 1st F & 2nd F	30.91	381.36	1.74	14.79×10^6	19.23×10^6	642.7	2903.5	2343.7	15237	1.38	1.24	515.[illegible]
Between 2nd F & 3rd F	30.91	353.78	1.74	14.79×10^6	19.23×10^6	546.9	2442.5	2343.7	15237	1.30	1.19	461.2
Between 3rd F & 4th F	30.91	319.45	1.74	14.79×10^6	19.23×10^6	451.1	1987.9	2343.7	15237	1.24	1.15	405.2
Between 4th F & 5th F	30.91	270.14	1.55	9.7×10^6	12.61×10^6	355.3	1541.7	1937.1	12591	1.22	1.14	345.7
Between 5th F & 6th F	30.91	208.32	1.50	9.7×10^6	12.61×10^6	259.4	1106.4	2068.4	13444	1.14	1.09	262.3
Between 6th F & 7th F	30.91	134.23	1.50	4.7×10^6	6.08×10^6	163.6	685.0	1334.3	8630	1.14	1.09	181.5
Between 7th F & Roof	23.24	47.71	1.28	4.7×10^6	6.08×10^6	67.8	279.6	1310.0	8473	1.05	1.03	73.6

TABLE 5.8 COMPUTATION OF M_c FOR INTERMEDIATE COLUMNS

Case 2 $M_{2b} = .9\ M_D$ $M_{2s} = 1.4\ M_E$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	10.80	393.12	1.50	14.79×10^6	19.23×10^6	276.8	1506.6	3153.7	20502	1.10	1.08	436.5
Between 1st F & 2nd F	10.80	381.36	1.74	14.79×10^6	19.23×10^6	242.2	1291.8	2343.7	15237	1.12	1.09	427.8
Between 2nd F & 3rd F	10.80	353.78	1.74	14.79×10^6	19.23×10^6	207.6	1077.2	2343.7	15237	1.10	1.08	393.9
Between 3rd F & 4th F	10.80	319.45	1.74	14.79×10^6	19.23×10^6	173.0	868.6	2343.7	15237	1.08	1.06	350.3
Between 4th F & 5th F	10.80	270.14	1.55	9.70×10^6	12.61×10^6	138.4	669.0	1937.1	12591	1.08	1.06	298.1
Between 5th F & 6th F	10.80	208.32	1.50	9.70×10^6	12.61×10^6	103.8	480.0	2068.4	13444	1.05	1.04	227.9
Between 6th F & 7th F	10.80	134.23	1.30	4.70×10^6	6.08×10^6	69.2	304.8	1334.3	8630	1.05	1.04	150.9
Between 7th F & Roof	11.61	47.71	1.28	4.70×10^6	6.08×10^6	34.6	145.8	1310.0	8473	1.03	1.02	60.6

TABLE 5.9 COMPUTATION OF M_c FOR CORNER COLUMN

Case 1 $M_{2b} = 1.4\ M_D + 1.4\ M_L$ $M_{2s} = 1.4\ M_E$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi \Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	59.19	393.12	1.35	14.79×10^6	19.23×10^6	574.4	3364.4	3893.5	25312	1.17	1.15	521.4
Between 1st F & 2nd F	59.19	383.36	1.40	14.79×10^6	19.23×10^6	487.7	2903.5	3620.3	23536	1.16	1.14	505.7
Between 2nd F & 3rd F	59.19	353.78	1.40	14.79×10^6	19.23×10^6	400.9	2442.5	3620.3	23536	1.13	1.12	463.1
Between 3rd F & 4th F	59.19	319.45	1.40	14.79×10^6	19.23×10^6	317.3	1987.9	3620.3	23536	1.10	1.09	413.3
Between 4th F & 5th F	59.19	270.14	1.29	9.70×10^6	12.61×10^6	237.9	1541.7	2796.6	18178	1.09	1.09	359.0
Between 5th F & 6th F	59.19	208.32	1.25	9.70×10^6	12.61×10^6	164.1	1106.4	2978.4	19360	1.06	1.06	283.6
Between 6th F & 7th F	59.19	134.23	1.15	4.70×10^6	6.08×10^6	97.1	685.0	1705.1	11028	1.06	1.07	206.4
Between 7th F & Roof	75.10	47.71	1.14	4.70×10^6	6.08×10^6	38.1	279.6	1651.5	11223	1.02	1.03	126.0

TABLE 5.10 COMPUTATION OF M_c FOR CORNER COLUMNS

Case 2 $M_{2b} = 0.9\ M_D$ $M_{2s} = 1.4\ M_E$

Location of Column	M_{2b} K'	M_{2s} K'	k	EI_{2b} K/in^2	EI_{2s} K/in^2	P_u kips	ΣP_u kips	ϕP_c kips	$\phi\Sigma P_c$ kips	δ_b	δ_s	M_c K'
Between G F & 1st F	20.6	393.12	1.35	14.79×10^6	19.23×10^6	338.1	1506.6	3893.5	25312	1.10	1.06	439.4
Between 1st F & 2nd F	20.6	383.36	1.40	14.79×10^6	19.23×10^6	282.6	1291.8	3620.3	23536	1.08	1.06	428.6
Between 2nd F & 3rd F	20.6	353.78	1.40	14.79×10^6	19.23×10^6	227.2	1077.2	3620.3	23536	1.07	1.05	393.5
Between 3rd F & 4th F	20.6	319.45	1.40	14.79×10^6	19.23×10^6	174.8	868.6	3620.3	23536	1.05	1.04	353.5
Between 4th F & 5th F	20.6	270.14	1.29	9.70×10^6	12.61×10^6	126.8	669.0	2796.6	18178	1.05	1.04	302.6
Between 5th F & 6th F	20.6	208.32	1.25	9.70×10^6	12.61×10^6	84.3	480.0	2978.4	19360	1.03	1.03	235.8
Between 6th F & 7th F	20.6	134.23	1.15	4.70×10^6	6.08×10^6	48.6	304.8	1705.1	11028	1.03	1.03	159.5
Between 7th F & Roof	37.6	47.71	1.14	4.70×10^6	6.08×10^6	21.0	145.8	1651.5	11223	1.01	1.01	86.2

TABLE 5.11 DESIGN OF INTERMEDIATE COLUMNS

COLUMN LOCATION	CASE 1			CASE 2			ρ_{CRIT}	AS REQD in^2	TIES & SPACING
	P_{u_K}	$M_{c_{K'}}$	ρ_{REQD}	P_{u_K}	$M_{c_{K'}}$	ρ_{REQD}			
Between* G F & 1st F	738.5	512.2	.0278	276.8	436.5	.0185	.0278	16.00 8 # 14	#4 @ 12" c/c
Between* 1st F & 2nd F	642.7	515.5	.0278	242.2	427.8	.0208	.0278	16.00 8 # 14	#4 @ 12" c/c
Between* 2nd F & 3rd F	546.9	461.2	.0185	207.6	393.9	.0185	.0185	10.66 8 # 11	#4 @ 12" c/c
Between* 3rd F & 4th F	451.1	405.7	.0107	173.0	350.3	.0196	.0196	11.29 8 # 11	#4 @ 12" c/c
Between** 4th F & 5th F	355.3	345.7	.0300	138.4	298.1	.0279	.0300	12.00 8 # 11	#4 @ 12" c/c
Between** 5th F & 6th F	259.4	262.3	.0196	103.8	227.9	.0196	.0196	7.84 6 # 10	#4 @ 12" c/c
Between 6th F & 7th F	163.6	181.5	.0329	69.2	150.9	.0329	.0329	8.47 6 # 11	#4 @ 12" c/c
Between 7th F & Roof	67.8	73.6	.0080	34.6	60.6	.0107	.0107	2.41 4 # 8	#4 @ 12" c/c

* These column dimensions should be 24" x 24"

** These column dimensions should be 20" x 20"

TABLE 5.12 DESIGN OF CORNER COLUMNS

COLUMN LOCATION	CASE 1			CASE 2			ρ_{CRIT}	AS REQD in^2	TIES & SPACING
	P_{u_K}	$M_{c_{K'}}$	ρ_{REQD}	P_{u_K}	$M_{c_{K'}}$	ρ_{REQD}			
Between * G F & 1st F	574.4	521.4	.0247	338.1	439.3	.0185	.0247	14.22 4 # 18	#4 @ 12" c/c
Between * 1st F & 2nd F	487.7	505.7	.0196	282.6	428.6	.0185	.0186	11.29 8 # 11	#4 @ 12" c/c
Between * 2nd F & 3rd F	400.9	463.1	.0147	227.2	393.5	.0147	.0147	8.47 6 # 11	#4 @ 12" c/c
Between * 3rd F & 4th F	317.3	413.3	.0148	174.8	353.9	.0147	.0148	8.52 6 # 11	#4 @ 12" c/c
Between ** 4th F & 5th F	237.9	359.0	.0300	126.8	302.6	.0300	.0300	12.00 8 # 11	#4 @ 12" c/c
Between ** 5th F & 6th F	164.1	283.6	.0226	84.3	235.8	.0226	.0226	9.04 6 # 11	#4 @ 12" c/c
Between 6th F & 7th F	97.1	206.4	.0353	48.6	159.5	.0353	.0353	7.94 6 # 11	#4 @ 12" c/c
Between 7th F & Roof	38.1	126.0	.0183	21.0	86.2	.0167	.0183	4.18 4 # 9	#4 @ 12" c/c

*These column dimensions should be 24" x 24"

**These column dimensions should be 20" x 20"

CHAPTER 6

CONCLUSIONS

The specifications and recommendations given by the seismic code dealt in this report provide minimum design criteria in specific categories but in broad general terms. Reliance must be placed on the experienced structural engineer to interpret and adapt the basic principles to each specific site. Because of the great number of problems and the complexity of the problem it is beyond the scope of this report to go to such detail as to cover specifically all the variations in response such as the dynamic characteristics of the structure, the variables in ground motion, the intensity of the earthquake, the distance to the epicenter of the seismic disturbance and the types of soil. It does not always follow that increasing the design lateral force results in a safer structure, in some circumstances this may do more harm than good. (1)

Where damage control is desired for non-structural members the design must provide not only sufficient strength to resist ground motion, but also must recognize the importance of providing the proper stiffness or rigidity to limit the lateral deflection or the drift. Overturning effects should be considered in foundation design.

In this report two separate design procedures were carried out. First the structure was designed without seismic considerations and then for a second time the structure was designed with seismic considerations but with wind considerations. The increase in dimensions of the structural members were computed and the increase in reinforcing steel for flexure, shear and axial compression for different members at all locations were computed. Some of these are shown in Tables 6.1 and 6.2.

There was an increase of 38.8 percent in the beams cross-sectional areas for seismic loading at all floor levels except the roof beam for which the same dimensions were sufficient for both designs. The columns cross-sectional areas for the lower four floors were increased by 77.7 percent and there was an increase of 23.5 percent in the cross-sectional areas of floor 5 and floor G columns. However, the column dimensions for the top two floors remain unchanged.

As is evident from Table 6.2, the increase in reinforcing steel was much higher in lower stories than in the top stories. The maximum increase in flexural steel is about 115 percent for the first floor beam as compared to an increase of 21 percent for the roof beam. Similar patterns can be observed for the increase in shear and column reinforcement.

TABLE 6.1 COMPARISON BETWEEN THE TWO DESIGNS (MEMBER DIMENSIONS)

STRUCTURAL MEMBERS	NON-SEISMIC DESIGN	SEISMIC DESIGN	PERCENTAGE INCREASE
a) Beams (1-7)	15" x 18"	15" x 25"	38.3
b) Beam (Roof)	15" x 18"	15" x 18"	0
c) Columns (1-4)	18" x 18"	24" x 24"	77.7
d) Columns (5-6)	18" x 18"	20" x 20"	23.5
e) Columns (7-Roof)	15" x 15"	15" x 15"	0

TABLE 6.2 COMPARISON BETWEEN THE TWO DESIGNS (REINFORCING STEEL)

DESCRIPTION	NON-SEISMIC DESIGN	SEISMIC DESIGN	PERCENTAGE INCREASE
1: Flexural Reinforcement			
i) Beam #1			
a) End Span Ext. Neg	3.45 in^2	6.36 in^2	85
b) End Span Int. Neg	2.62 in^2	5.65 in^2	115
c) Int. Span Neg.	3.23 in^2	6.19 in^2	90
ii) Beam #4			
a) End Span Ext. Neg.	2.87 in^2	5.28 in^2	84
b) End Span Int. Neg.	2.07 in^2	4.72 in^2	130
c) Int. Span Neg.	2.68 in^2	5.11 in^2	90
iii) Beam (Roof)			
a) End Span Ext. Neg.	1.89 in^2	2.05 in^2	8.5
b) End Span Int. Neg.	1.18 in^2	1.42 in^2	21
c) Int. Span Neg.	1.72 in^2	1.90 in^2	10.5
2: Shear Reinforcement			
i) Beam #1	# 3 @ 8" c/c	# 4 @ 11" c/c	30
ii) Beam #4	# 3 @ 8" c/c	# 4 @ 11" c/c	30
iii) Beam Roof	# 3 @ 8" c/c	# 3 @ 8" c/c	0
3: Column Reinforcement			
i) Between G F and 1st F			
a) Corner Column	13.42 in^2	14.22 in^2	330
b) Intermediate Column	14.20 in^2	16.00 in^2	13
ii) Between 4th F and 5th F			
a) Corner Column	3.24 in^2	12.00 in^2	270
b) Intermediate Column	3.24 in^2	12.00 in^2	270
iii) Between 7th F and Roof			
a) Corner Column	2.25 in^2	4.18 in^2	86
b) Intermediate Column	2.25 in^2	2.41 in^2	7
4: Lateral Ties for Columns	# 4 @ 18" c/c	# 4 @ 12" c/c	50

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APPENDIX A LIST OF NOTATIONS USED

a = Depth of equivalent stress block

A = Maximum amplitude on seismograph

A_o = Amplitude of one thousandth of a millimeter

A_s = Area of tension reinforcement

A'_s = Area of compression reinforcement

A_{st} = Total area of longitudinal reinforcement

A_v = Area of shear reinforcement

b = Width of the member

β_d = Absolute value of ratio of maximum factored dead load moment to maximum factored total moment

c = Distance from extreme compression fiber to neutral axis

C = Seismic co-efficient

C_d = The drag co-efficient

C_m = A factor relating actual moment diagram to an equivalent uniform moment diagram

C_p = Horizontal force factor for non-structural components

d = Distance from extreme compression fiber to centroid of tension reinforcement

D = The dimension of building in feet

δ_b = Moment magnification factor for M_{2b}

δ_i = Deflection at level i relative to the base

δ_s = Moment magnification factor for M_{2s}

Δ_{ub} = Ultimate deflection for brittle material

Δ_{ud} = Ultimate deflection for ductile material

E_c = Modulus of elasticity of concrete

EI = Flexural stiffness of compression member

EI_{2b} = Flexural stiffness of compression member for non-sidesway case

EI_{2s} = Flexural stiffness of compression member for sidesway case

ε_s' = Strain in steel

f_c' = Specified compressive strength of concrete

f_s = Tensile stress in steel

f_y = Specified yield strength of reinforcement

F_i F_n F_x = Lateral force applied at level i, n, or x respectively

F_p = Lateral force on a part of the structure and in the direction under consideration

F_t = That portion of V considered concentrated at the top of the structure in addition to F_v

g = acceleration due to gravity

h_i h_n h_x = Height in feet above the base to level i, n, or x respectively

I = Occupancy importance co-efficient

I_g = Gross moment of inertia of the section

k = Effective length factor for compression member

K = Ductility factor (used in Chapter 2 and 3)

K = kips (used from Chapter 3 onward)

ℓ_n = Clear span length and for computation of moments and shear

ℓ_u = Unsupported length of compression member

Level i = Level of the structure referred to by subscript i, i=1 designates the first floor above the base

Level n = Level which is upper most in the main portion of the structure

Level x = Level which is under consideration

ME = Magnitude of earthquake

M = Moment of section

M_c = Factored moment to be used for design of compression members

M_d = The design moment at a section

M_n = The nominal moment capacity of a section

M_u = Factored moment at any section

M_{2b} = Value of larger factored end moment on compression member due to loads that result in no appreciable sidesway

M_{2s} = Value of larger factored end moment on compression member due to loads that result in appreciable sidesway

N = The total number of stories above the base to level n

P = Pressure per ft^2 due to wind velocity

P_c = Critical load on compression member

P_u = Factored axial load at a given eccentricity for compression member

ϕ = Strength reduction factor

Π = Constant having a value of 3.14159

r = Radius of gyration of cross section of a compression member

ρ = Ratio of tension reinforcement

ρ_A = Allowable ratio of tension reinforcement

ρ_B = Balanced ratio of tension reinforcement

ρ_{crit} = Critical ratio of tension reinforcement

ρ_{max} = Maximum ratio of reinforcement

ρ_{min} = Minimum ratio of reinforcement

ρ_{req} = Required ratio of reinforcement

s = Spacing of shear reinforcement

S = Site factor

ψ_A or ψ_B = Ratio of total EI/ℓ of compression member to total EI/ℓ of flexural members in a plane at each end of the compression member

T = Fundamental elastic period of vibration of the structure

T_s = Characteristic site period

V = Total lateral force or shear at base

V_c = Nominal shear strength provided by concrete

V_u = Factored shear force

W = Total dead load and applicable portion of other loads

W_i W_x = That portion of W which is located at or assigned to level i or x respectively

W_u = Factored load per unit length of beam

W_p = The weight of a portion of a structure

X = Velocity of wind in miles per hour

Z = Numerical co-efficient related to the seismicity of a region

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APPENDIX C COLUMN DESIGN COMPUTATIONS
PART 1

In this report design of columns have been carried out according to the new specifications of the ACI Code (1). The design of intermediate column between ground floor and first floor for three loading cases is explained in this appendix.

Case 1

P_u = 804.5 K (Table 4.9), M_{2b} = 33.97 K' & M_{2s} = 0 (Table 4.11)

k = 1.6 (Table 4.8), $E_c = 3.605 \times 10^3$ ksi (pg. 31)

$I_g = 8748\ in^4$, l_u = 126"

According to Eq. 4-5 $EI = \dfrac{E_c I_g/2.5}{1 + \beta_d}$

For non-sidesway moments $\beta_d = 0.4$ and for sidesway moments $\beta_d = 0$
(Refer sec. 4.10.1)

$$EI_{2b} = \frac{3.605 \times 10^3 \text{ ksi} \times 8748 \text{ in}^4/2.5}{1 + 0.4} = 9.01 \times 10^6 \text{ ksi}$$

According to Eq 4-4 $P_c \doteq \dfrac{\Pi^2 EI}{(K\ l_u)^2}$

$$P_c = \frac{\Pi^2 \times 9.01 \times 10^6 \text{ ksi}}{(1.6 \times 126")^2} = 2188 \text{ K}$$

ϕP_c = 0.7 x 2188 K = 1531.6 K

According to Eq. 4-2 $\delta_b = \dfrac{C_m}{1 - P_u/\phi P_c}$

C_m = 1 for unbraced columns.

$$\delta_b = \frac{1}{1 - P_u/\phi P_c} = \frac{1}{1 - 804.5^K/1531.6^K} = 2.1$$

According to Eq. 4-1 $M_c = M_{2b}\ \delta_b + M_{2s}\ \delta_s$

M_c = 33.97 K' x 2.10 + 0 = 71.34 K'

PCA Charts (13) for tied columns were used to compute the ρ_{reqd}. These charts give the actual capacity of the section. Thus P_u and M_c have to be divided by ϕ and then the ρ_{reqd} is obtained from the charts for 18" x 18" columns and material properties $f_y = 60$ ksi and $f'_c = 4$ ksi

$$M_c' = \frac{71.34\ K'}{.7} = 101.9\ K'$$

$$P_u' = \frac{804.5\ K}{.7} = 1150\ K$$

The ρ_{reqd} from charts = 0.0439 (Refer Table 4.17, Case 1)

Case 2

$P_u = 603.38^K$, $M_{2b} = 25.48\ K'$ & $M_{2s} = 86.06^K$ (Table 4.12)

According to Eq. 4-5

$$EI_{2b} = \frac{3.605 \times 10^3 \text{ ksi} \times 8748 \text{ in}^4/2.5}{1 + 0.4} = 9.01 \times 10^6 \text{ ksi}$$

$$EI_{2s} = \frac{3.605 \times 10^3 \text{ ksi} \times 8748 \text{ in}^4/2.5}{1 + 0} = 12.61 \times 10^6 \text{ ksi}$$

According to Eq. 4-4

$$P_c = \frac{\Pi^2 \times 9.01 \times 10^6 \text{ ksi}}{(1.6 \times 126")^2} = 2188\ K \qquad \phi P_c = 1531.6\ K$$

$$\Sigma P_c = \frac{5 \times \Pi^2 \times 12.61 \times 10^6 \text{ ksi}}{(1.6 \times 126")^2} = 15311\ K \qquad \phi \Sigma P_c = 10717\ K$$

$$\Sigma P_u = 3 \times 603.38^K + 2 \times 321.41^K = 2452.96^K$$

According to Eq. 4-2 $\delta_b = \dfrac{1}{1 - 603.38^K/1531.6^K} = 1.65$

According to Eq. 4-3 $\delta_s = \dfrac{1}{1 - 2452.96^K/10717.7^K} = 1.30$

$$M_c = M_{2b}\ \delta_b + M_{2s}\ \delta_s$$

$$M_c = 25.48^{K'} \times 1.65 + 86.06^{K'} \times 1.3 = 153.9^{K'}$$

For PCA Charts

$$M_c' = \frac{153.9}{0.7} = 219.9 \text{ K}' \qquad P_u' = \frac{603.38}{0.7} = 862 \text{ K}$$

$\rho_{reqd} = 0.0439$ (Refer Table 4.17, Case 2)

<u>Case 3</u>

$P_u = 276.65$ K $\qquad M_{2b} = 10.80$ K' & $M_{2s} = 87.75$ K' (Refer Table 4.13)

$EI_{2b} = 9.01 \times 10^6$ ksi $\qquad EI_{2s} = 12.61 \times 10^6$ ksi

$P_c = 2188$ K $\qquad \phi P_c = 1531.6$ K

$\Sigma P_c = 15311$ K $\qquad \phi P_c = 10717.7$ K

$\Sigma P_u = 3 \times 276.5 + 2 \times 156.69 = 1143.6$ K

$\delta_b = 1.22 \qquad \delta_s = 1.12$

$$M_c = 10.80 \times 1.22 + 87.75 \times 1.12 = 111.5 \text{ K}'$$

For PCA Charts

$$M_c' = \frac{111.5}{0.7} = 159.3^{K'} \qquad P_u' = 395.2 \text{ K}$$

From charts $\rho_{reqd} = 0.0053$ (Refer Table 4.17, Case 3)

APPENDIX C — MOMENT DISTRIBUTION FOR COMPUTATION OF DEAD & LIVE LOAD MOMENT FOR COLUMNS

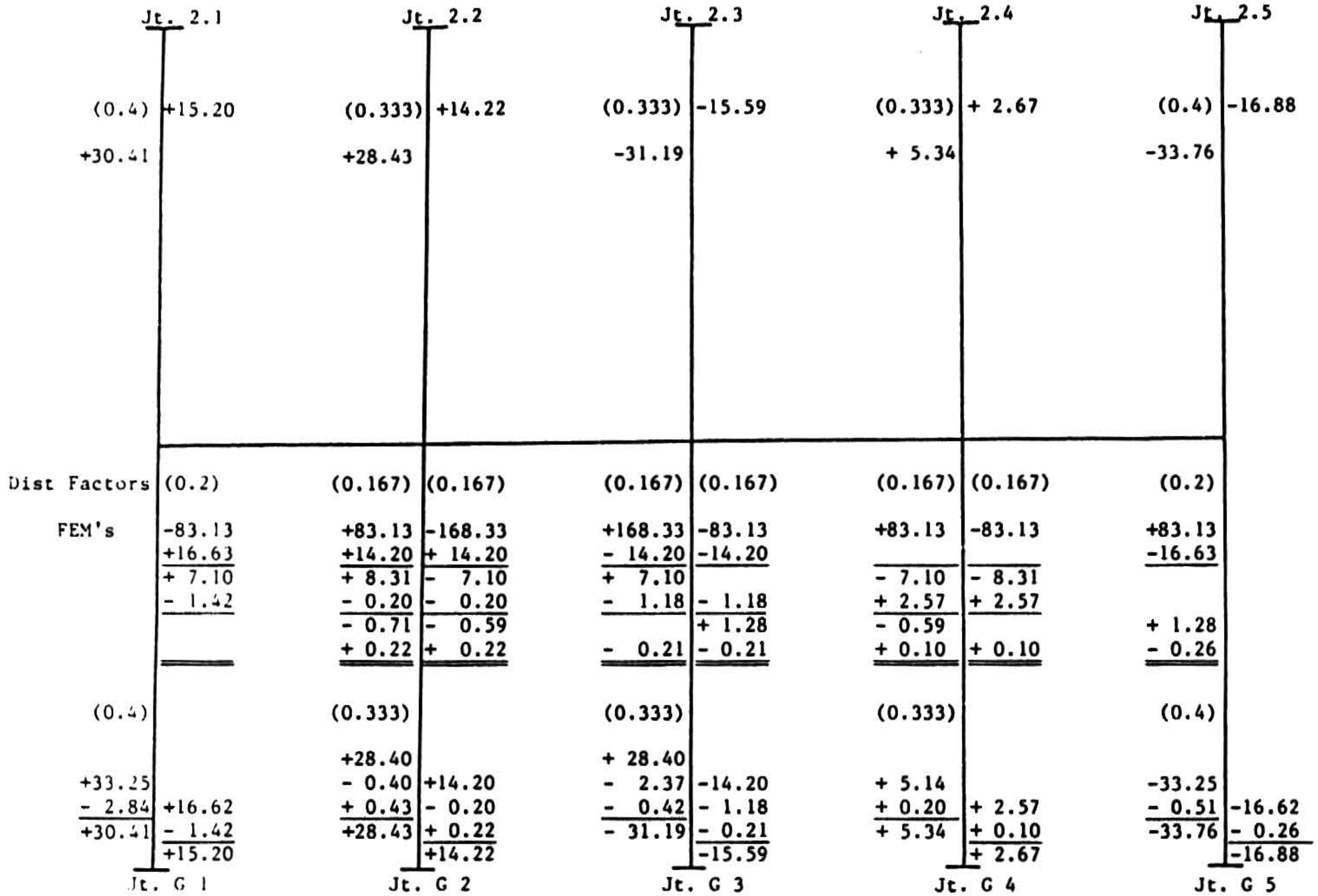
Part 2(a) — (Refer Fig. 4.7a and Table 4.6)

	Jt. R1	Jt. R2	Jt. R2	Jt. R3	Jt. R3	Jt. R4	Jt. R4	Jt. R5
Dist Factors	(0.333)	(0.25)	(0.25)	(0.25)	(0.25)	(0.25)	(0.25)	(0.333)
FEM's*	-83.13	+83.13	-111.47	+111.47	-83.13	+83.13	-83.13	+83.13
	+27.71	+ 7.09	+ 7.09	- 7.09	- 7.09			-27.72
	+ 3.54	+13.86	- 3.54	+ 3.54		- 3.54	-13.86	
	- 1.18	- 2.58	- 2.58	- 0.89	- 0.89	+ 4.35	+ 4.35	
	- 1.29	- 0.59	- 0.45	- 1.29	+ 2.18	- 0.45		+ 2.18
	+ 0.43	+ 0.26	+ 0.26	- 0.22	- 0.22	+ 0.11	+ 0.11	- 0.73

	Jt. R1	Jt. 71	Jt. R2	Jt. 72	Jt. R3	Jt. 73	Jt. R4	Jt. 74	Jt. R5	Jt. 75
Dist Factors	(0.67)		(0.5)		(0.5)		(0.5)		(0.67)	
	+55.42		+14.18		- 14.18					
	- 2.36	+27.71	- 5.16	+ 7.09	- 1.77	- 7.09	+ 8.70		-55.42	
	+ 0.86	+ 1.18	+ 0.52	- 2.58	- 0.45	- 0.89	+ 0.22	+ 4.35	- 1.46	-27.71
	+53.92	+ 0.43	+ 9.54	+ 0.26	- 16.40	- 0.22	+8.92	+ 0.11	-56.88	- 0.73
		+26.96		+ 4.77		- 8.20		+ 4.46		-28.44

Note: All moments are in K'

*Fixed End Moments are equal to $wl^2/12$



APPENDIX C MOMENT DISTRIBUTION FOR COMPUTATION OF DEAD & LIVE LOAD MOMENTS FOR COLUMNS
Part 2(b) (Refer Fig. 4.6a and Table 4.4)

SEISMIC CONSIDERATIONS IN THE DESIGN OF
REINFORCED CONCRETE MULTI-STORY STRUCTURES

by

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AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

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ABSTRACT

Although the incidence of earthquakes of destructive intensity has been confined to a relatively few areas of the world, the catastrophic consequences attending the few that have struck near centers of population has focused attention on the need to provide adequate safety against this most awesome of nature's quirks.

The satisfactory performance of a large number of reinforced concrete structures subjected to severe earthquakes in different areas of the world has demonstrated that it is possible to design such structures to successfully withstand earthquakes of major intensity.

To get an idea of the increase in member dimensions and reinforcing steel -- thus the increase in cost, a multi-storied reinforced concrete structure was designed with and without seismic considerations. This structure was assumed to be located in an area of high seismic risk and was designed according to the Lateral Force Requirements of the Seismology Committee, Structural Engineers Association of California.

The increase in cost due to Earthquake Resistant Design is analogous to any mishap coverage plan (medical insurance etc.) and should not be overlooked.