DESIGN AND ANALYSIS OF A THREE DIMENSIONAL FRAME TYPE BUILDING

by

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ABSTRACT

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The purpose of this thesis was to design a six story office building using the recommendations and requirements of Uniform Building Code. This process of design took the structural engineer through every stage of design to the final product which is the building itself. It started with manual calculations, using short-cut methods, and it was supplemented by computer calculations in two (2) directions for more accurate solutions. The final stage was the computer-aided three (3) dimensional design. This was in turn supplemented by a dynamic analysis of the building, so the entire process of designing was completed.

ACKNOWLEDGEMENTS

The author wishes to expres his gratitude to Dr. JAVED ALAM fcr his valuable advice and encourgement throughout this work. Sincere thanks are also due to Dr.JACK BAKOS,Jr. and Prof.JACK RITTER for serving on the thesis review committee.

This thesis is dedicated to my daughters MONICA and LIANA DRAGOMAN.

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LIST OF SYMBOLS

SYMBOL	DEFINITION
A	Cross sectional area of column or girder
C	Code lateral force coefficient
C_1	Lateral force coefficient equal to V/W
F	Lateral forces
F_t	Lateral force at top of structure
F_x	Lateral force at any level
f	Natural frequency of vibration
g	Acceleration of gravity
h	Height in feet above the base
I	Occupancy importance factor as specified in table 23-K (U.B.C.)
I_x	Moment of inertia for x-x axis
I_y	Moment of inertia for y-y axis
E	Modulus of elasticity of steel at 29,000 Kips per square inch
K	Numerical coefficient as set forth in table number 23-I (U.B.C.)
l	Girder length (ft.)
m	Mass
N	Total number of stories above the base
S	Numerical coefficient for site-structure resonance
P	Vertical load kips
T	Period of vibration of the structure in seconds
V	Lateral force or shear at base of structure
Z	Seizmicity zone factor used in lateral force formula
W	The total dead load in kip

LIST OF SYMBOLS, continued

M_x	Bending moment
M_y	Bending moment (kip-ft) along y-y axiz
B_x	Bending factor for x-x axis
B_y	Bending factor for y-y axis
P_{eq}	Equivalent oxial load due to bending component
P_t	Total oxial load kip
Δ	Lateral displacement
Δ_c	Lateral displacement due to column distortion
Δ_g	Lateral displacement due to girder distortion
ν	Poisson's ratio, may be taken as 0.3 for steel
ξ	Dumping coefficient
D.T.	Time step seconds
N.F.	Number of frequency
Σ	Summation of the mathematical terms that follow

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CHAPTER I

1.1 Introduction

The structural analysis and design of multi-story buildings have existed for many years back. The new era of a modern technology, the computer era, however, gives the engineer a powerful tool in analyzing and designing all kinds of building structures.

Designing and analyzing the multiple-bay rigid frame type of building, for instance, has now become a comparatively simple problem by using the computer.

The complicated job of solving simultaneous equations has been eased by the advent of modern electronic computing machines. The latter, nevertheless, represent an investment that few engineers can afford. Adding to the difficulties of solving the equations is the necessity of knowing all the sizes of the various members of the structure to be analyzed before the equations can be set up. In view of the difficulties surrounding the initial stages of elastic analyses, the structural engineer, who has knowledge of a good assortment of reliable short-cut methods of analysis, is fortunate indeed. Some of the most reliable short-cut methods are:

- the portal method - short-cut method of seismic-stress analysis or wind-stress analysis

- Hardy-Cross method of bending moment distribution

1.2 Objective and Scope of Study

In the predimensional stage of the structural frame, these two short-cut methods will be used. With sizes of the elements of the structure (computed in two dimensions) obtained from using these two methods, a static analysis using the computer will bring it closer to the final sizes of the elements of structure.

This predesign stage in two dimension was chosen due to the way the frames of the building are set up.

Since an exact evaluation of the bending moments in two directions was difficult to apply on frames, the final sizes of the elements of the building were obtained using a design in three dimensions by computer.

This final analysis by computer in three dimensions presents the possibility of a clear view of how the entire elements of the building are behaving under the loading conditions.

An intense earthquake constitutes the most severe loading to which most civil engineering structures might possibly be subjected. More and more often today, a designer is asked to perform a dynamic seismic analysis of the building. An optimum engineering approach and understanding of seismism will result in a good earthquake design.

With all sizes given and the mass of the building computed, a seismic analysis of the building using computers is possible. This will be seen in Chapter IV of this project.

The design of a six story office building is based on the recommendation and requirements of the Uniform Building Code[9] and American Institute of Steel Construction (AISC) Manual.[4]

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CHAPTER II

2.1 Load Evaluation for the Building

The process of designing a building is often thought of as arranging elements or forms in such a way that the expected loads are carried safely. Other than the functional and economical requirements, the ability of the building to support load is one of the most important requirements in design.

The loads for which a building must be designed may be classified into dead loads, vertical live loads, and lateral live loads. Dead loads include the weight of the fixed components of the building, such as floors, beams, girders, roofs, columns, fixed walls and partitions etc. All vertical loads other than dead load may be included under vertical live loads. Lateral live loads are external forces due to the action of wind, earth, and hydrostatic pressure. The effects of seismic disturbance are included by applying lateral loads at the different levels of the building. The office building is located in seismic zone 4 and it is subjected to a wind velocity of 80 m.p.h. For preliminary design purposes, it was decided that the building will have two ductile moment frames in direction east-west. It will also have five ductile moment frames in direction south-north. The story height was chosen as 11'-0".

The building is sidesway uninhibited. The columns are considered to be braced in the X direction and in the Y direction at each floor and roof elevations.

Material specifications: - steel frame A36
high-strength bolts . . . A325
welding electrodes. . . . E70.

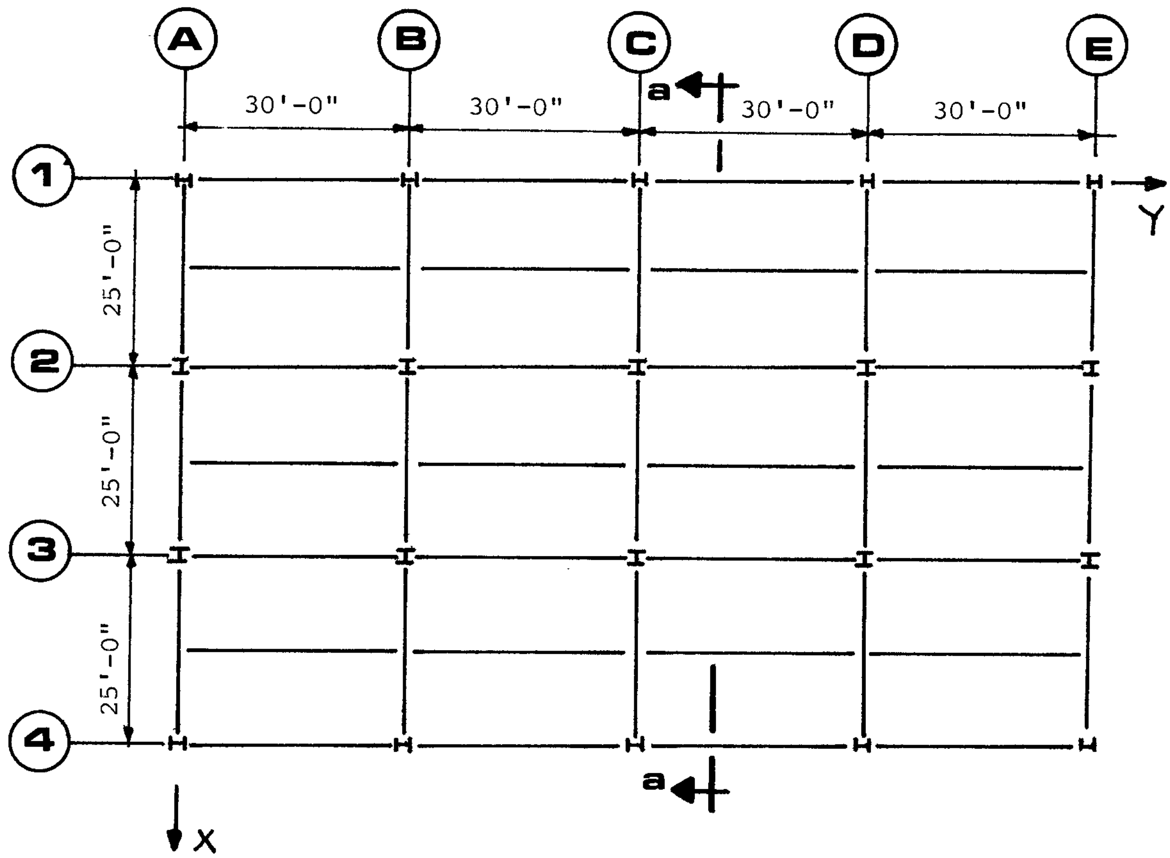

Fig. 2.1 - Typical floor framing

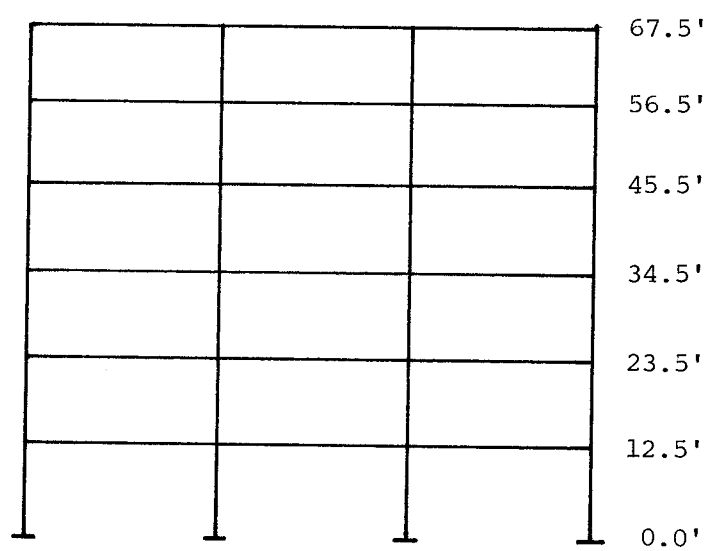

Fig. 2.2 - Typical section **a-a**

Loads: Roof loading

Roofing and insulation	= $7\#/ft^2$
Metal deck	= $3\#/ft^2$
3 1/4" lightweight concrete	= $44\#/ft^2$
Ceiling	= $1\#/ft^2$
Misc.	= $4\#/ft^2$
Steel Framing	= $7\#/ft^2$
	$66\#/ft^2$
Live load	$=20\#/ft^2$
	Total load = 86 psf

Floor loading

Metal deck	= $3\#/ft^2$
3 1/4 lightweight couc.	$=44\#/ft^2$
Ceiling	= $1\#/ft^2$
Misc.	= $4\#/ft^2$
Partition	$=20\#/ft^2$
Steel framing	$=11\#/ft^2$
	83 p.s.f. dead load
Live load for office building	= $50\#/ft^2$
Total	= $133\#/ft^2$
Curtain wall =	= $15\#/ft^2$

2.2 Load Evaluation for Frame 1 and 4 of the Building

Roof-Dead Load

Joint (A,1) and (E,1). See Figure 2.1. (The letter indicates column line and the number indicates the row line).

(66 x 12.5 x 30 x 1/2) x 1/2 = 6187.5#

cutain wall .15 x (11/2 + 4) x 25 x 1/2 = 1781.25 #

7968.75 #.= 7970#

at joint (B,1), (C,1), (D,1)

66 x 12.5 x 30 x 1/2 = 12375#

uniform load

w = 66 x 12.5 x 1/2 = 412.5 #/ft.

curtain wall 15 x (11/2 + 4) = 142.5 #/ft

555 #/ft.

floors (1,2,3,4,5)

at joint (A,1) and (E,1)

(83 x 12.5 x 30 x 1/2) x 1/2 = 7781.25#

curtain wall

15 x 11 x 25 x 1/2 = 2062.5 #

9843.75 #

uniform load

w = 83 x 12.5 x 1/2 = 418.75 #/ft

curtain wall 15 x 11 = 165.00 #/ft

683.75 = 684#/ft.

at joint (B,1); (C,1); (D,1)

(Letter indicates column line and number indicates row line)

83 x 12.5 x 30 x 1/2 = 15562# (see Figure 2.1)

Live Load

Roof

at joint (A,1) & (E,1)

(20 x 12.5 x 30 x 1/2) x 1/2 = 1875#

uniform load

20 x 12.5 x 1/2 = 125#/ft.

at joint (B,1); (C,1); (D-1)

20 x 12.5 x 30 x 1/2 = 3750#

floors (1,2,3,4,5)

at joint (A,1) & (E,1)

(50 x 12.5 x 30 x 1/2) x 1/2 = 4688#

at joint

(B,1); (C,1); (D,1)

50 x 12.5 x 30 x 1/2 = 9375#

uniform load

50 x 12.5 x 1/2 = 313 #/ft.

Wind Load

Roof -	15 x (11/2 + 4) x 75/2	=	5344#
Floor 5	15 x 11 x 75/2	=	6188#
Floor 4	15 x 11 x 75/2	=	6188#
Floor 3	14 x 11 x 75/2	=	5775#
Floor 2	10 x 11 x 75/2	=	4125#
Floor 1	10 x (11/2 + 12.5/2) 75/2	=	4406#

In this direction (east-west) the lateral load is taken only by the frames 1 and 4. The wind load is multiplied by 75'-0"/2

2.3 Load Evaluation for Frames A & E of the Building

Roof

At joint (A,1) & (A,4)

(E,1) & (E,4)

dead load

(66 x 12.5 x 1/2 x 30 x) 1/2 = 6187.5 #

cutain 15 x (11/2 + 4) x 30 x 1.2 = 2137.5 #

8325#

Uniform load

15 x (11/2 + 4) = 143 #/ft.

at joint (A,2); (A,3)

(E,2); (E,3)

and at the middle of the beams

66 x 12.5 x 30 x 1/2 = 12,375#

Floors

at joint (A,1) & (A,3)

(E,1) & (E,4)

(83 x 12.5 x 1/2 x 30) x 1/2 = 7,781.25#

cutain wall 15 x 11 x 30 x 1/2 = 2,475#

10,256#

Unif. load

15 x 11 = 165#/ft.

at joints (A,2); (A,3)

(E,2); (E,4)

and in the middle of beams

83 x 12.5 x 30 x 1/2 = 15,562#

Wind Load

Roof	15 x (11/2 + 4) x 30 x 1/2	=	2137#
Floors 5	15 x 11 x 30 x 1/2	=	2475#
4	15 x 11 x 30 x 1/2	=	2475#
3	14 x 11 x 30 x 1/2	=	2310#
2	10 x 11 x 30 x 1/2	=	1650#
1	10 x (11/2 + 12.5/2) x 30 x 1/2	=	1762#

Live Load

Roof - at joints (A,1); (A,4)

(E,1); (E,4)

(20 x 12.5 x 1/2 x 30) x 1/2 = 1875#

At joints (A,2); (A,3)

(E,2); (E,4)

and in the middle

20 x 12.5 x 30 x 1/2 = 3750#

floors

at joints (A,1); (A,4)

(E,1); (E,4)

(50 x 12.5 x 1/2 x 30) x 1/2 = 4688#

at joints (A,2); (A,3)

(E,2); (E,3) and in the middle

50 x 12.5 x 30 x 1/2 = 9375#

2.4 Load Evaluation for Frames B, C, D of the Building

Dead Load

Roof

at joints (B,1) & (B,4)

66 x 12.5 x 1/2 x 30 =	12,375#
curtain wall	
15 x (11/2 + 4) x 30 =	4,275#
	16,650#

at joints (B,2); (B,3) and in the middle

66 x 12.5 x 30 = 24,750#

Floors at joints (B,1) & (B,4)

83 x 12.5 x 1/2 x 30 = 15,562.5 #

curtain wall 15 x 11 x 30= 4,950. #

20,512#

at joints (B,2) & B,3)

83 x 12.5 x 30 = 31,125#

Live Load

Roof at joints (B,1) & (B,4)

20 x 12.5 x 1/2 x 30 = 3750#

at joints (B,2) & (B,3)

20 x 12.5 x 30 = 7500#

Floors

at joints (B,1) & (B,4)

50 x 12.5 x 1/2 x 30 = 9375#

at joints (B,2) & (B,3) and in the middle

50 x 12.5 x 30 = 18750#

Wind Load

Roof	15 x (11/2 + 4) x 30	=	4275#
5	15 x 11 x 30	=	4950#
4	15 x 11 x 30	=	4950#
3	14 x 11 x 30	=	4620#
2	10 x 11 x 30	=	3300#
1	10 x (11/2+12.5/2) x 30	=	3450#
			25545#

The same loads are good for frame C & D.

2.5 Seismic Forces Evaluation for Frame 1 & 4 of the Building

Except as provided in section 2314 (g) and (i) of U.B.C.[9] 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 following formula:

$V = ZIKCSW$

since the building is situated in zone 4

$Z = 1$ (Numerical coefficient dependent upon the seismic zone)

$K = 0.67$ (Horizontal force factor for buildings).

C & S will be computed but the value CxS need not exceed 0.14

$I = 1.00$ (Occupancy Importance Factor).

$V = 1.00 \times 1.00 \times 0.67 \times C \times S \times W = 0.67\ (C \times S) \times W$

Conform with U.B.C. $T = 0.10N$

Practice has shown that this formula yields a reasonable period estimate for buildings in the 40 story range but is not accurate for short buildings. In this case will be used the basic period formula for constant drift.

$$T = 0.25 \sqrt{\frac{\Delta}{C_1}}$$

where: T = period of building in sec.

Δ = lateral deflection at the top of building

C_1 = lateral force coefficient by which the total weight of the building is multiplied in order to obtain the seismic lateral force due to the building's response to a given base motion.

U.B.C. for a moment frame limit the Δ (drift) to 0.005 h.

Δ = 0.005 x (12.5 + 5 x 11) = 0.005 x 67.5 = 0.3375 ft.

Δ = 0.3375 x 12 = 4.05"

C_1 = ZICS = 1.00 x 1.00 x CS

$$T = 0.25 \sqrt{\frac{4.05}{CS}}$$

$$C = \frac{1}{15\sqrt{T}} \qquad T = 0.25 \sqrt{\frac{4.05}{\frac{1}{15\ T}} \times S}$$

$$T^2 = 0.25^2 \times \frac{4.05}{\frac{1}{15\ T}} \times S$$

S is a site structure coefficient $1 < S < 1.5$

$$\frac{T^2 \times 1}{15\ \ T} = \frac{0.25^2 \times 4.05}{S} \qquad \text{Consider } S = 1.00$$

T^3 = 15 x 0.25^2 x 4.05 = 3.796875

T^3 = 14.41626

T = 2.43 seconds

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{2.43}} = 0.042$$

Dead load evaluation

Roof	120 x 75 x 66	= 594,000#
Curtain Wall	15 x (240 + 150) x (11/2 + 4)	= 55,575#
Floors	120 x 75 x 83	= 747,000#
Curtain Wall	15 x 390 x 11	= 64,350#

Roof dead load = 649,575# = 650K

Floors = 811,350# = 811K

The total dead load of the building will be:

W = 649,575 + 5 x 811,350 = 4,706,325# = 471K

V = 0.67 (CS) x W = 0.67 x 0.042 x 1.00 x 4,706,325 = 132,435#

which is the total lateral force. The distribution of this force over the height of the building will be:

$$F_x = \frac{(V-F_t)\ W_x\ h_x}{\sum W_i\ U_i}$$

F_t need not be taken into consideration when

$$\frac{h_n}{D_s} \leq 3 \qquad 67.5 \times \frac{1}{120} = 0.56$$

in this case $F_x = \dfrac{V\ W_x\ h_x}{\sum W_i\ h_i}$

Table 2.1

Distribution of Earthquake Forces and Story Shears

Floor Level	h_x (Ft)	W_x (K)	$W_x h_x$ $\times 10^{-2}$	$\frac{W_x h_x}{\sum W_1 h_1}$	F_x (KIPS)	V_x (KIPS)	$F_x \times 1.5$ (KIPS)	$V_x \times 1.5$ (KIPS)
R	67.5	650	438	0.238	31.42		47.13	
6	56.5	811	458	0.249	32.86	31.42	49.29	47.13
5	45.5	811	369	0.201	26.54	64.28	39.82	96.42
4	34.5	811	280	0.153	20.19	90.82	30.28	136.23
3	23.5	811	191	0.105	13.86	111.01	20.79	166.51
2	12.5	811	101	0.054	7.13	124.87	10.69	187.30
1						132.00		198.00

Because in this kind of structure the drift criteria determine the size of the girders and columns, it is possible to verify the building period (T) by U.B.C. formula.

$$T = 2\pi \sqrt{\sum_{i=1}^{u} W_1\ \delta_1^2 \quad g \left[\sum_{i=1}^{n-1} F_i\ \delta 1 \quad (F_t + F_n)\ \delta n\right]}$$

Table 2.2

Verification of Building Period by U.B.C.

	h_x (ft.)	Drift Index	$\delta_x = 0.005\,h_x$	W_x (K)	F_x (K)	$W_x \delta_x^2$	$F_x \delta_x$
R	67.5	0.005	0.3375	650	47.13	74.03	15.90
6	56.5	0.005	0.2825	811	49.29	64.72	13.92
5	45.5	0.005	0.2275	811	39.82	41.97	9.05
4	34.5	0.005	0.1725	811	30.28	24.13	5.22
3	23.5	0.005	0.1175	811	20.79	11.19	2.44
2	12.5	0.005	0.0625	811	10.69	3.16	0.67
1							
						219.2	47.2

$$T = 2\pi \sqrt{219.2 \times \frac{1}{32.2 \times 47.2}} = 2.39 \text{ seconds}$$

This is in agreement with the result obtained from basic period formula

$$T = 0.25 \sqrt{\frac{\Delta}{C_1}} = 2.43 \text{ seconds}$$

2.6 Distribution of seismic forces

Although the center of mass and rigidity coincide, U.B.C. requires designing for a minimum torsional excentricity e equal to 5% of the maximum building dimension.

$$e = 0.05 \times 120 = 6.0'$$

This torsion will be taken by the moment frames in the north-south direction and the moment frames in the west - east direction.

Assume the relative rigidity of the moment frames as a function of tributary area for north-south direction

$$A_a = A_e = 67.5 \times 15.0 = 1012.5 \text{ ft}^2$$

$$A_b = A_c = A_d = 67.5 \times 30 = 2025 \text{ ft}^2$$

in west-east direction are two moment frames which have to take the seismic forces. It is assumed that the other two do not take any lateral forces.

$$A_1 = A_4 = 67.5 \times 37.5 = 2531.5 \text{ ft}^2$$

if considered R_a rigidity = 1 = R_e

then $R_b = \dfrac{2.25}{1012.5} = 2$ R_b, R_c, R_d

$$R_4 = R_1 = \frac{2531.5}{1012.5} = 2.5$$

$R_a = 1$

$R_b = 2$

$R_c = 2$

$R_d = 2$

$R_e = 1$

$R_1 = 2.5$

$R_4 = 2.5$

<u>2.6.1 Distribution of earthquake forces in direction east-west for frames 1 and 4</u>

In this case shear distribution for seismic forces in west-east direction will be:

$$V_{ix} = R_1 \left[\frac{V_x}{\sum REW} \pm \frac{(V \times e)\ d}{\sum R_y\ (d)^2} \right]$$

where

d = distance from the center of rigidity to frame

R_{E-W} = relative rigidity of all frames in west-east direction

R_y = total earthquake shear on building at story x

V_{yx} = earthquake shear on frame referred to that frame on column line y at story x

$\sum R_{ew} = 2 \times (2.5) = 5$

$\sum R_{yd}^2 = 2 \times (2.5 \times 37.5^2 + 2 \times (1.00) \times 60^2 + 2 \times (2.00) \times 30^2$

$\times (2.00) \times 0^2 =$

$= 7031.25 + 7200 + 3600 = 17831$

$$V_x = 2.5 \left[\frac{V_x}{2.5 \times 2} \pm \frac{(V_x\ 6.0) \times 37.5}{17831} \right]$$

$= 0.5\ V_x \pm 0.012\ V_x = 0.512\ V_x$

In this case the shear distribution forces from earthquake will be

$V_{1,r}$ (roof) $= V_{4,r} = 0.512 \times 0 = 0$

$V_{1,6} = V_{4,6} = 0.512 \times 31.42 = 15.93$ Kip

$V_{1,5} = V_{4,5} = 0.512 \times 64.28 = 32.59$ Kip

$V_{1,4} = V_{4,4} = 0.512 \times 90.82 = 46.05$ Kip

$V_{1,3} = V_{4,3} = 0.512 \times 111.01 = 56.29$ Kip

$V_{1,2} = V_{4,2} = 0.512 \times 124.87 = 63.32$ Kip

$V_{1,1} = V_{4,1} = 0.512 \times 132.00 = 66.93$ Kip

From shear distribution we can find the value of the forces acting at each level

at 12.50 $P_1 = 66.93 - 63.32 = 3.61$ Kip.

23.50 $P_2 = 63.32 - 56.29 = 7.03$ Kip.

34.50 $P_3 = 56.29 - 46.65 = 10.24$ Kip.

45.50 $P_4 = 46.05 - 32.59 = 13.46$ Kip.

56.50 $P_5 = 32.59 - 15.93 = 16.66$ Kip.

67.50 $R = 15.93 - 0 = \underline{15.93}$ Kip.

66.93 Kip

For drift design accordingly with U.B.C. the shear force has to be multiplied with $1/K = 1/0.67 = 1.5$. In this case the shear forces at each level will be

$P_1 = 3.61 \times 1.5 = 5.415$ K

$P_2 = 7.03 \times 1.5 = 10.545$ K

$P_3 = 10.24 \times 1.5 = 15.36$ K

$P_4 = 13.46 \times 1.5 = 20.19$K

$P_5 = 16.66 \times 1.5 = 24.99$ K

$R = 15.93 \times 1.5 = 23.895$ K

2.6.2 Distribution of earthquake forces in direction north-south

Frame A and E

$$V_{ax} = R_a \left[\frac{V_x}{\sum Rew} \pm \frac{(V_x e)\ d}{\sum R_y\ (d)^2} \right]$$

$\sum REW - 1 + 2 + 2 + 2 + 1 = 8$

$\sum R_y\ (d^2) = 17831$

$$V_{ax} = 1_x \left[\frac{V_x}{8} \pm \frac{V_x\ 6 \times 60}{17.831} \right] =$$

$$= 1_x\ (0.125\ V_x + 0.02\ V_x) = 0.145\ V_x$$

$$V_{ax} = V_{ex} - 0.145\ V_x$$

See value of V_x on Table 2.1.

The shear distribution forces from an earthquake in north-south direction will be

V_{ar} (roof) $= 0.145 \times 0 = 0$

$V_{a6} = 0.145 \times 31.42 = 5.55$ Kip

$V_{a5} = 0.145 \times 64.28 = 9.32$ Kip

$V_{a4} = 0.145 \times 90.82 = 13.16$ Kip

$V_{a3} = 0.145 \times 111.01 = 16.096$ Kip

$V_{a2} = 0.145 \times 124.87 = 18.106$ Kip

$V_{a1} = 0.145 \times 132.00 = 19.14$ Kip

From shear distribution the value of the forces acting at each level are:

at 12.5' $P_1 = 19.14 - 18.104 = 1.036$ Kip

at 23.5' $P_2 = 18.104 - 16.096 = 2.008$ Kip

34.5' $P_3 = 16.06 - 13.16 = 2.936$ Kip

45.5' $P_4 = 13.16 - 9.32 = 3.840$ Kip

56.5' $P_5 = 9.32 - 5.55 = 3.77$ Kip

67.5' $R = 5.55 - 0 = 5.55$ Kip

For drift design these forces have to be multiplied by $1/K = 1.5$

$P_1 = 1.554$ Kip

$P_2 = 3.012$ Kip

$P_3 = 4.404$ Kip

$P_4 - 5.76$ Kip

$P_5 = 5.65$ Kip

$R = 8.325$ Kip

2.6.3 Distribution of earthquake forces in direction north-south (Frame B, C, D)

$$V_{bx} = R_b \left[\frac{V_x}{R_{ew}} \pm \frac{V_x * \ell * d}{R_{yd}^{2}} \right] =$$

$$= 2 \left[\frac{V_x}{8} \pm \frac{V_x * 6 \times 30}{17{,}831} \right] =$$

$$= 2\ (0.125 \pm 0.010) = 2 \times 0.135 = 0.270$$

$$V_{bx} = V_{dx} = 0.270$$

$$V_{cx} = 2 \left[\frac{V_x}{8} \pm 0 \right] = 2 \times 0.125 = 0.250$$

The shear distribution forces from an earthquake in north south direction for frame B & D

V_b (Roof) $= 0.270 \times 0 = 0 = 0$

$V_{b6} = 0.270 \times 31.42 = 8.483$ Kip

$V_{b5} = 0.270 \times 64.28 = 17.355$ Kip

$V_{b4} = 0.270 \times 90.82 = 24.52$ Kip

$V_{b3} = 0.270 \times 111.01 = 29.97$ Kip

$V_{b2} = 0.270 \times 124.87 = 33.71$ Kip

$V_{b1} = 0.270 \times 132.00 = 35.64$ Kip

From this shear distribution the value of the forces acting at each level in this case will be

at 12.5' $P_1 = 35.64 - 33.71 = 1.93$ Kip

$P_2 = 33.71 - 29.97 = 3.74$ Kip

$P_3 = 29.97 - 24.52 = 5.45$ Kip

$P_4 = 24.52 - 17.355 = 7.165$ Kip

$P_5 = 17.355 - 8.483 = 8.872$ Kip

$R = 8.483 - 0 = \underline{8.483}$ Kip

3.564 Kip

For frame C

$V_{cr} = 0.250 \times 0 = 0$

$V_{c6} = 0.250 \times 31.42 = 7.855$ Kip

$V_{c5} = 0.250 \times 64.28 = 16.07$ Kip

$V_{c4} = 0.250 \times 90.82 = 22.75$ Kip

$V_{c3} = 0.250 \times 111.01 = 27.75$ Kip

$V_{c2} = 0.250 \times 124.87 = 31.217$ Kip

$V_{c1} = 0.250 \times 132.00 = 33.00$ Kip

The lateral forces will be

P_1 = 33.00 - 31.217 = 1.783 Kip

P_2 = 31.217 - 27.75 = 3.467 Kip

P_3 = 27.75 - 22.705 = 5.065 Kip

P_4 = 22.705 - 16.07 = 6.635 Kip

P_5 = 16.07 - 7.855 = 8.215 Kip

R = 7.885 - 0 = 7.855 Kip

33 Kip

Now that all the forces acting on building have been determined, the next stage is the structural design.

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CHAPTER III

Structural Design of the Building

In its "Commentary on Plastic Design", the American Society of Civil Engineers[10] stated:

> An engineering structure is satisfactorily designed if it can be built with needed economy and if, throughout its useful life, it carries its intended loads and otherwise performs its intended function.

The types of loading acting on the structure are mutually dependent. The selecting of loading conditions follows the determination of the type and magnitude of the loads that act on the structure. It involves a decision as to which loads act in combination with the dead load. The codes specify the loading conditions that must be considered, but basically it is part of the designer's responsibility to specify the loading conditions.

The structural form has a very significant influence on the loading conditions that must be considered. The Uniform Building Code requirements are used to select the different load cases.

The following loading combination will be applied:

- (dead load + live load)
- (dead load + live load + seismic load) x 0.75

In the preliminary design stage, manual calculation by short-cut methods are used. These methods are supplemented by computer calculations in 2 (two) directions to generate more accurate solutions.

With the information obtained in the previous step, an analysis in three directions using the computer is made to determine the required section modulus of each member. This step consists of the final or revised selection of size and shape of members to be used in the design. The selection is based on an analysis of structural strength, which leads to the required section property.

3.1 Preliminary Design of the Building

As was indicated in Chapter II, the entire building is a combination of the following frames:

- direction north-south; frames 1 and 4
- in direction east-west; frames A,B,C,D,E

This step of preliminary design will be started with the development of moment and shear diagram due to seismic forces.

As can be seen, the seismic forces for frame C are very close in magnitude with that of frame B and D. For this reason only seismic forces for frame B will be analyzed. To obtain the shear and moment diagram due to seismic forces the portal method will be used, since in this method it is not necessary to have the element sizes.

Since a comparatively simple method of analysis is being applied to a staticly indeterminate structure, some simplifying assumptions, which are not quite true, must be made. They are as follows:

- the points of contraflexure are at the mid-points of the columns and girders
- the steel bent resists all the seismic forces (with no help from the walls, floors and partitions)
- the sum of the seismic loads above a given story is distributed as shear among the columns of that story.

These assumptions determined the shear forces and bending moments at each level of the frame.

First determine the shear force and moment diagram of the frame 1 which is the same as frame 4. Frame 1 and frame 4 resist all the seismic loading in direction east-west.

To determine the relative rigidity of each column, it can be assumed that the rigidity of each column is a function of the tributary area (see Fig. 2.1.).

Column (A,1) and (E,1)

Tributary area = 30 x 1/2 x67.5 = 1012.5 ft^2

Column (B,1) ; (C,1) and (D,1)

Tributary area = 30 x 67.5 = 2025 ft^2

In this case if column A is assumed to have a relative rigidity equal to 1 , then the rigidity of the other columns will be as:

$$R_b = \frac{2025}{1012.5} = 2$$

$$R_c = \frac{2025}{1012.5} = 2$$

$$R_d = \frac{2025}{1012.5} = 2$$

$$R_e = \frac{1012.5}{1012.5} = 1$$

With the relative rigidities of the columns established , determining shear forces and the bending moment for the column and girder at each level can be accomplished.

It starts at the roof level. From Chapter II the seismic load acting on Frame 1 at elev. 67.5' is equal with P =23.895Kip. With the rigidity of the columns calculated above , the shear force acting on each column will be :

Column (A,1) and (E,1)

$$P_1 = R_1 \times \frac{P}{\sum R} = 1 \times \frac{23.895}{1+2+2+2+1} = 2.986 \text{ Kip}$$

Column (B,1); (C,1); and (D,1)

$$P_{2,3,4} = \frac{2 \times 23.895}{8} = 5.973 \text{ Kip}$$

With shear force distributed to the column at the last level, the bending moment in each column and girder will be:

$$M_1 = -P_1 \times h/2 = -2.986 \times 11/2 = -16.42 \text{ K-ft.}$$

$$M_2 = -P_2 \times h/2 = -5.973 \times 11/2 = -32.85J \text{ K-ft.}$$

$$M_3 = -P_3 \times h/2 = -5.973 \times 11/2 = -32.855 \text{ K-ft.}$$

$$M_4 = -P_4 \times h/2 = -5.973 \times 11/2 = -32.855 \text{ K-ft.}$$

$$M_5 = -P_5 \times h/2 = -2.986 \times 11/2 = -16.42 \text{ K-ft}$$

Looking at the Figure 3.1 below will give a better understanding of the shear force and bending moment.

The first joint of the Frame 1 at the roof elevation was taken separately.

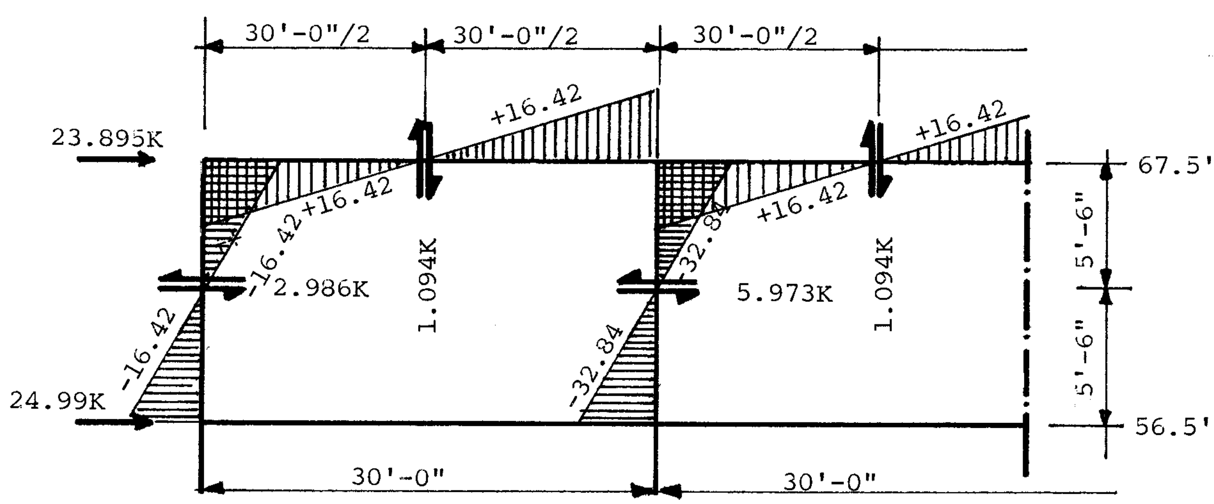

Fig.3.1. Portal Method for joint 1. Moment diagram and shear forces.

The joint has to be in equilibrium which means

$F_x = 0$

$F_y = 0$

$M = 0$

From the condition $M = 0$ the moment in the girder is found

$-16.42 + M_g = 0 \qquad M_g = 16.42$ K-ft.

Which will give us the shear force in the girder

$$M = V \times 30/2; \; V = \frac{16.42}{30} \times 2 = 1.092 \text{ Kip}$$

From $F_x = 0$ the axial force in column will be

$P + 1.094 = 0 \qquad P = -1.094$ Kip

Which will produce tension

$F_y = 0 \qquad 23.895 - P - 2.986 = 0$

$P = 20.909$ Kip

This force induces compression in girder.

Each joint is taken separately and the bending moment and shear forces are found for each element of the frame.

The entire bending moment and shear forces for the frame 1 are indicated on Figure 3.2. The seismic forces acting on frame 1 and 4 are extensive. For this reason a drift design for the frame is recommended by U.B.C.

For frames A,B,C,D,E (see Fig. 2.1) the bending moment and shear forces diagram is built in the same way as for frame 1 and 4 with the exception of the fact that the rigidity of the columns is assumed differently. After consulting Fig. 2.1 it can be seen that the extreme columns are oriented in their weak direction in the plane of acting forces. For this reason it can be assumed that the rigidity of the columns can be taken as a ratio of moments of inertia.

Assume for instance that it is used W12x58 column

$I_x = 476 \text{ in}^4$

$I_y = 107 \text{ in}^4$

for extreme column if the rigidity is = 1 for the middle column it will be :

$R_2 = \frac{476}{107} = 4.44 \quad \text{use} \quad R_2 = 4$

$R_3 = 4$

$R_4 = 1$

With this rigidity of the columns assumed the bending moment diagram and shear forces can be built as was indicated for frame 1 and 4.

The results are shown in Fig.3.3 and 3.4 for frame A and E and frame B , C ,D respectively.

It was indicated at the beginning of Chapter III ,that the loading condition for the predesign step will be :

1. (DL + LL)
2. (DL + LL + Seismic)x1/1.33

As a procedure in predesign , the loading condition of DL + LL will be used and verified by the second loading condition (DL + LL + Seismic)x1/1.33 It will start with the frames in the direction of north-south. The following frames are in this direction.

Frame A , and Frame E

Frame B , Frame C and Frame D

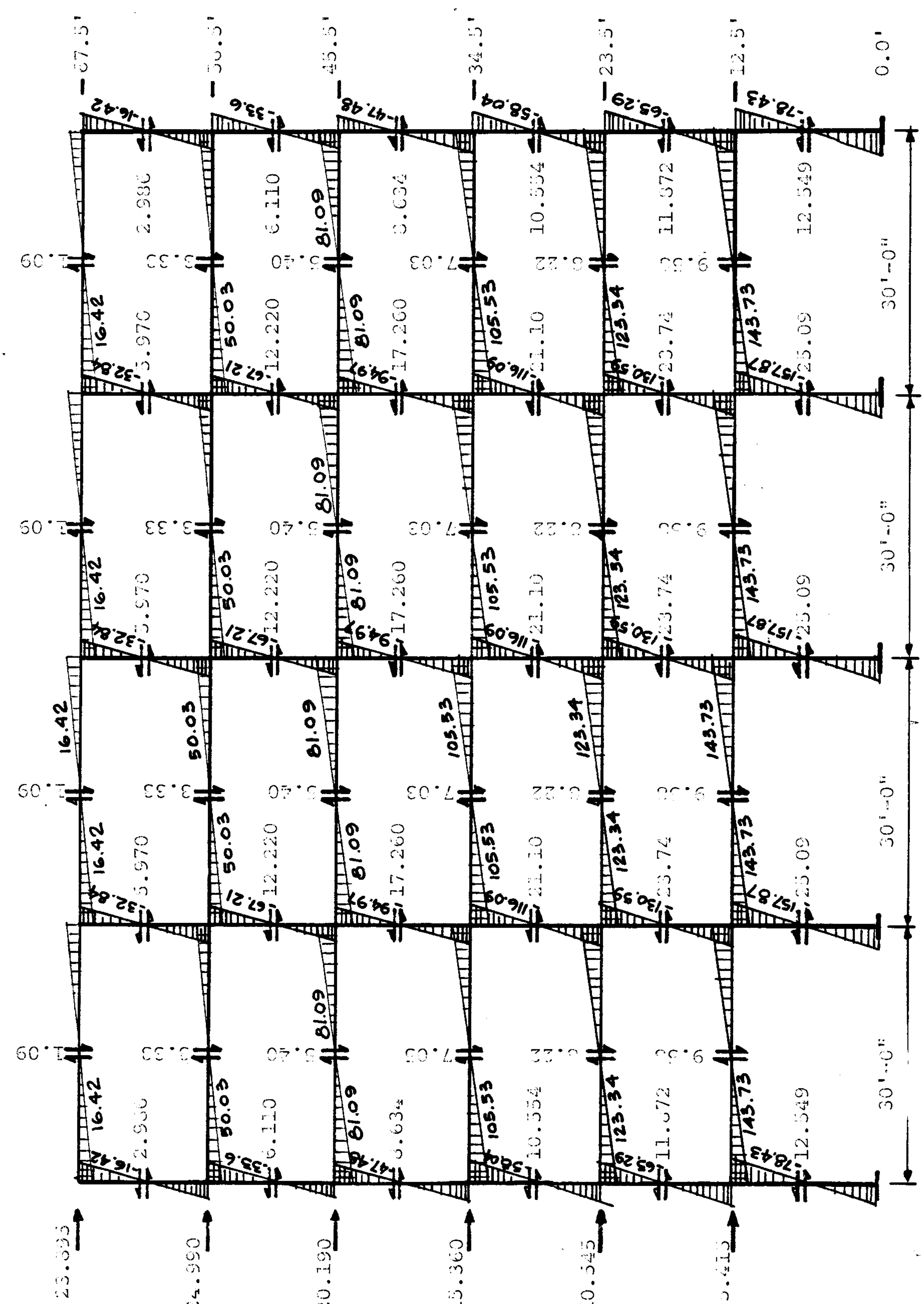

Fig. 3.2. Seismic forces -Moment diagram and shear forces for frame 1 & 4

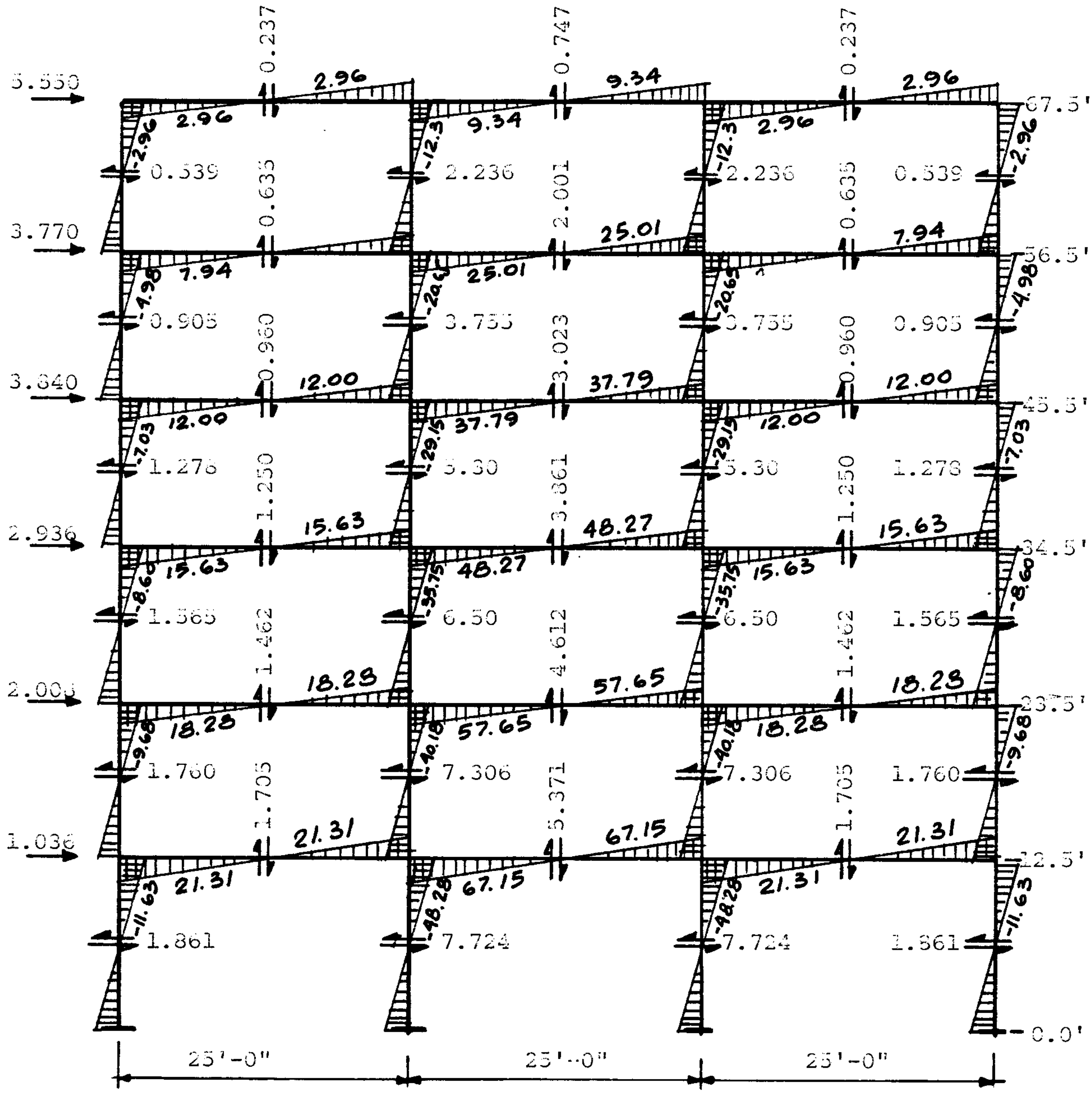

Fig. 3.3. Seismic forces- Moment diagram and shear forces for frame A & E.

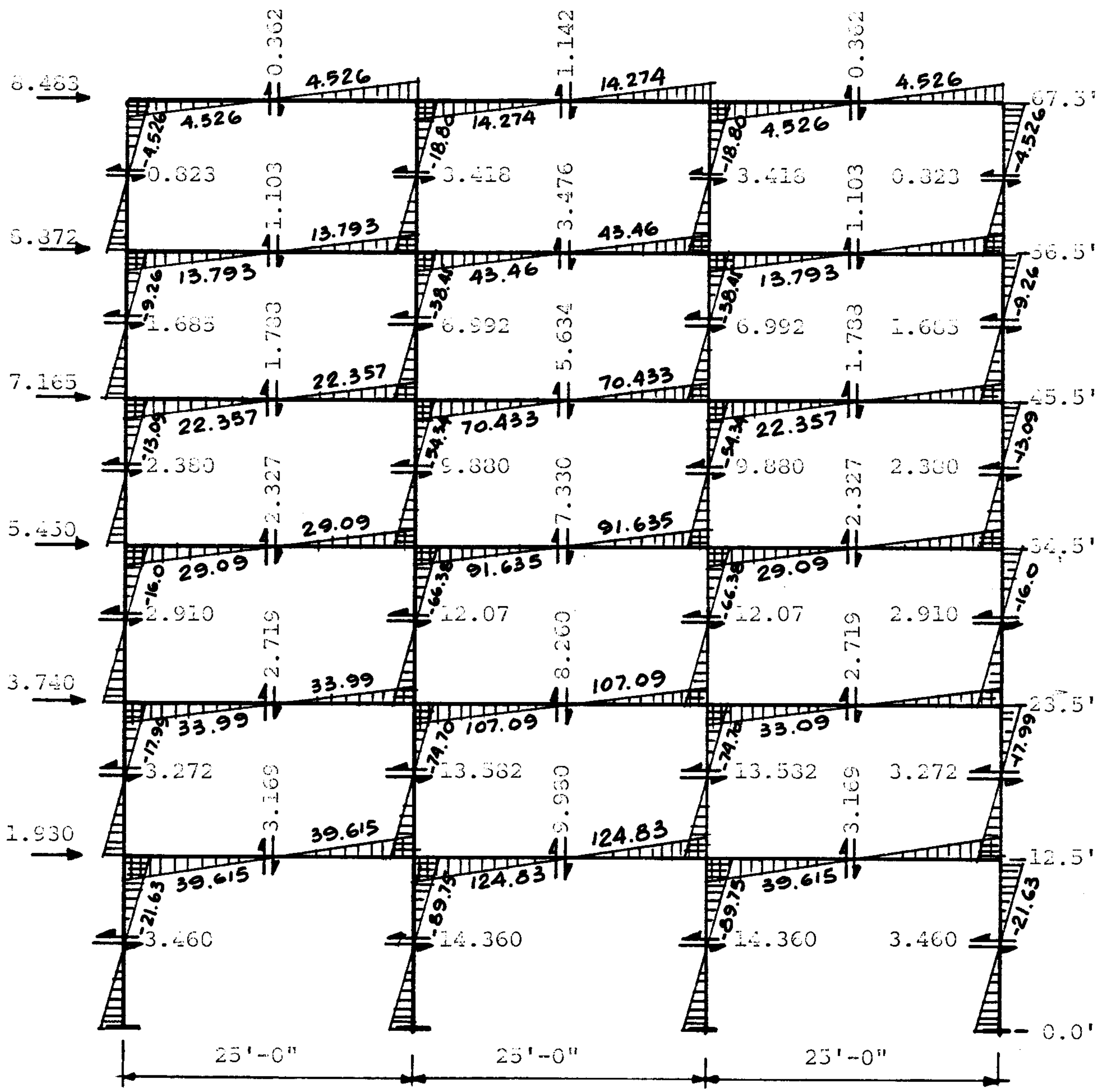

Fig. 3.4. Seismic forces -Moment diagram and shear forces for frame B ,C and D.

3.2 Predesign - Frame A and Frame E of the Building

The predesign of the frame will be divided in three heights

- From elevation +0.00 to 23.5'
- From elevation 23.5' to 45.5'
- From elevation 45.5' to 67.5'

Elevation 0.00 to 23.5' - Since the frame is a rigid frame, all beams of the frame will have a rigid connection to the column.

First it is necessary to evaluate the fixed end forces from beam loading.

Dead Load Roof

$R_1 = 12.375 \times 1/2 = 6.1875$ Kip

$R_2 = 0.142 \times 25 \times 1/2 = \underline{1.7875}$ Kip

$R_t = 7.975$ Kip.

$M_1 = \dfrac{12.375 \times 25}{8} = 38.67$ K-ft.

$M_2 = \dfrac{0.143 \times 25^2}{12} = 7.45$ K-ft.

$M_t = 46.12$ K-ft.

Live Load

$R_1 = 3.750 \times 1/2 = 1.875$ Kip

$M_1 = 3.750 \times 25 \times 1/8 = 11.72$ K-ft

Dead load + live load will be

$R_t = 7.975 + 1.875 = 9.85$ Kip

$M_t = 46.12 + 11.72 = 57.84$ K-ft.

For Floor Dead Load

$F_1 = 15.562 \times 1/2 = 7.781$ Kip

$$F_2 = 0.165 \times 25 \times 1/2 = \underline{2.062} \text{ Kip}$$

$$F_t = 9.843$$

$$M_1 = 15.562 \times 25 \times 1/8 = 48.63 \text{ K-ft}$$

$$M_2 = 0.165 \times 25^2 \times 1/12 = \underline{8.59} \text{ K-ft}$$

$$M_t = 57.22 \text{ K-ft}$$

<u>Live load</u>

$$F_1 = 9.375 \times 1/2 = 4.6875 \text{ Kip}$$

$$M_1 = 9.375 \times 25 \times 1/8 = 29.296 \text{ K-ft}$$

<u>For dead load + live load will be</u>

$$F = 9.843 + 4.687 = 14.53 \text{ Kip}$$

$$M = 57.22 + 29.29 = 86.51 \text{ K}$$

For frame A & E from elevation 0.00 to 23.5' , the simplified moment distribution method will be used to find the moments in columns and in beams

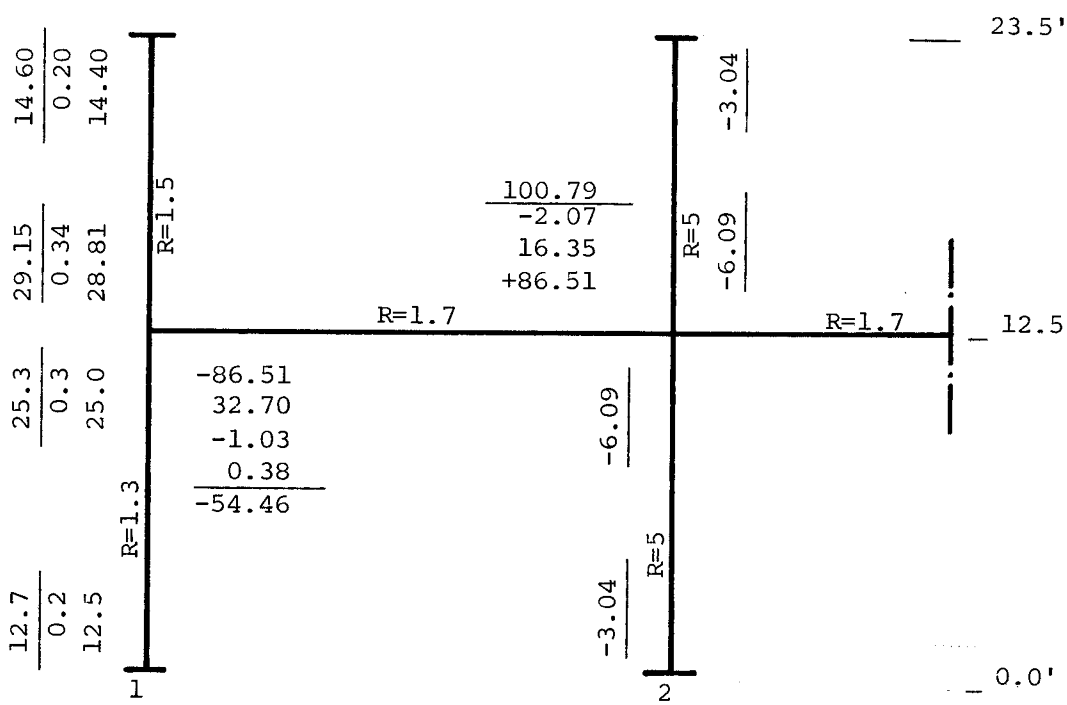

Fig 3.5. Simplified moment distribution method
Frame A & E.

Since it is a symmetrical frame and loaded symmetrically , only half of the frame will be considered. With the moments computed from moment distribution method , the beams and the columns can be design.

For beam between column 1 and 2 , the maximum moment is 100.79 K-ft. A W18x35 beam will satisfy this bending moment. Between column 2 and 3 the maximum bending moment is 88.58 K-ft, a W16x31 will satisfy the bending moment.

Column 1 (Fig.3.5)

The bending moment for column 1 is 25.3 K-ft. This column is oriented in the weak direction for this bending moment so in this case the equivalent axial load from bending moment will be :

$$P_{eq} = M_x\, B_y = 25.3 \text{x} 12 \text{x} 0.655 = 198.85 \text{ Kip.}$$

The axial load from dead load + live load will be the axial load from roof plus five floors.

Roof $\quad P = 9.85 + 8.325 + 1.875 = 20.5$ Kip.

Floor $\quad P = 14.53 + 10.256 + 4.687 = 29.473$ Kip.

$$P_{total} = 20.5 + 29.473 \text{x} 5 = 167.865 \text{ Kip.}$$

The total load acting on column is :

$$P = 198.85 + 167.865 = 366.715 \text{ Kip.}$$

With this axial load of 366.715 Kip and with an effective length of column of 12.5' , from AISC Manual the size of the column is obtained.

Use W12x72 with an allowable load of 390 Kip.

Column 2 (Fig.3.5)

The bending moment in this column is M = 6.09 K-ft.

$$P_{eq} = 6.09 \times 12 \times 0.218 = 15.93 \text{ Kip.}$$

The vertical load acting on this column will be :

at roof $P = 2 \times 9.85 + 12.375 + 3.750$

$= 35.825$ Kip

at floor $P = 2 \times 14.53 + 15.562 + 9.375$

$= 53.997$ Kip

the total vertical load is:

$P_t = 35.825 + 53.997 \times 5 =$

$= 305.81$ Kip

The total axial load for design is

$P = 305.81 + 15.93$ Kip $= 321.74$ Kip

use W12 x 72 $\quad P_{all} = 390$ Kip

<u>Elev. = 23.5' to 45.5'</u>

<u>Col. Row 1</u>

D.L. + L.L.

$P_{total} = P_{roof} + 3 \times P\ (floor)$

$= 20.5 + 29.473 \times 3 = 109.91$ Kip

$M = 23.5\ K_{ft} \quad P_{eq} = 198.85$ Kip

$P_{design} = 109.81 + 198.85 = 307.86$ Kip

use W12 x 65 $\quad P_{allow} = 350$ Kip

<u>Col. row 2</u>

D.L. + L.L.

$P_{total} = P\ (roof) + 3P\ (floors)$

$= 35.825 + 3 \times 53.997 = 197.816$

$P_{design} = 197.816 + 15.93$ Kip $= 213.74$ Kip

use W12 x 50 $\quad P_{all} = 230$ Kip

Elev. 45.5' - 67.5'

Col. Row 1

D.L. & L.L.

P_{total} = P (roof) + I X P (floor) =

= 20.5 + 29.473 = 49.97 Kip

M = 25.3 K_{ft} P_{eq} = 198.85 Kip

P_{design} = 49.97 + 198.85 = 248.82 Kip

use W12 x 53

P_{all} = 271 Kip

Col. Row 2

D.L. + L.L.

P_{total} = P (roof) + 1 - P (floor) =

= 35.825 + 53.995 = 89.82 Kip

M = 6.09 P_{eq} = 15.93

P_{design} = 89.82 + 15.93 = 105.75 Kip

use W12 x 40 P_{all} = 185 Kip

The frames A & E now have to be checked using the following combination of loading (See Fig 3.3 for moment) (D.L. + L.L. + seismic) x 1/1.33

Elev. 0.00 to 23.5'

Col. Row 1

0.75 (D.L. + L.L.) = 0.75 x 366.715 = 275.03K

0.75 x $P_{(seismic)}$

M_x = 11.63 K_{ft}

P_{eq} = 11.63 x 12 x 0.655 x 0.75 = 68.55

P_{design} = 275.03 + 68.55 = 343.58

W12 x 72 is OK

Col. Row 2

(D.L. + L.L.) x 0.75 = 321.74 x 0.75 = 241.305Kip.

$P_{(seismic)}$ x 0.75 = 48.275 x 12 x 0.218 x 0.75 x 94.71

P_{design} = 241.305 + 94.71 = 336.02 Kip

W12 x 72 is OK

Elev. 23.5' to 45.5'

Col. Row 1

(D.L. + L.L.) x 0.75 = 307.86 x 0.75 = 230.89 Kip.

$P_{(seismic)}$ x 0.75 = (8.60 x 12 x 0.655 x 0.75) = 50.69 Kip.

P_{design} = 281.87 Kip

W12 x 65 is OK

Col. row 2

(D.L. + L.L.) x 0.75 = 213 x 0.75 = 159.75 Kip

$P_{(seismic)}$ x 0.75 = (35.75 x 12 x 0.218 x 0.75) = 70.14Kip.

P_{design} = 229.89 Kip

W12 x 50 is OK

Elev 45.5 to 67.5

Col. Row 1

(D.L. & L.L.) x 0.75 = 248.82 x 0.75 = 187.36Kip.

$P_{(seismic)}$ x 0.75 = (4.98 x 12 x 0.655 x 0.75) = 29.29 Kip.

P_{design} = 216.65K

w12 x 53 is OK

Col. Row 2

(D.L. + L.L.) x 0.75 = 105.75 x 0.75 = 79.31Kip.

$P_{(seismic)} \times 0.75 = (20.65 \times 12 \times 0.655 \times 0.75) = 121.73$ Kip

$P_{design} = 201.04$ Kip

use W12 x 45 $P_{all} = 220$ Kip

For floor beams check the design with (D.L. + L.L. + seismic) x 0.75

At 12.5'

Beam between low 1 and 2

(D.L. + L.L.) 0.75 = 100.79 x 0.75 = 75.59 Kip.

$P_{(seismic)} \times 0.75 = 0.75 \times 21.31 =$ 15.98 Kip.

91.57 Kip.

W18 x 35 beam is good between row col. 2 and 3

(D.L. + L.L.) x 0.75 = 88.58 x 0.75 = 66.43 Kip.

$P_{(seismic)} \times 0.75 = 67.148 \times 0.75$ 50.36 Kip.

116.79 Kip.

In this case the size of the beam has to be increased from W16 x 31 to W18 x 35. The same size of beams will be used for all five floors.

The sizes of columns and girders for frame A and frame shown in Fig. 3.6. Having obtained the sizes of all the elements of the frame and the forces acting on it will make it now possible to use the computer.

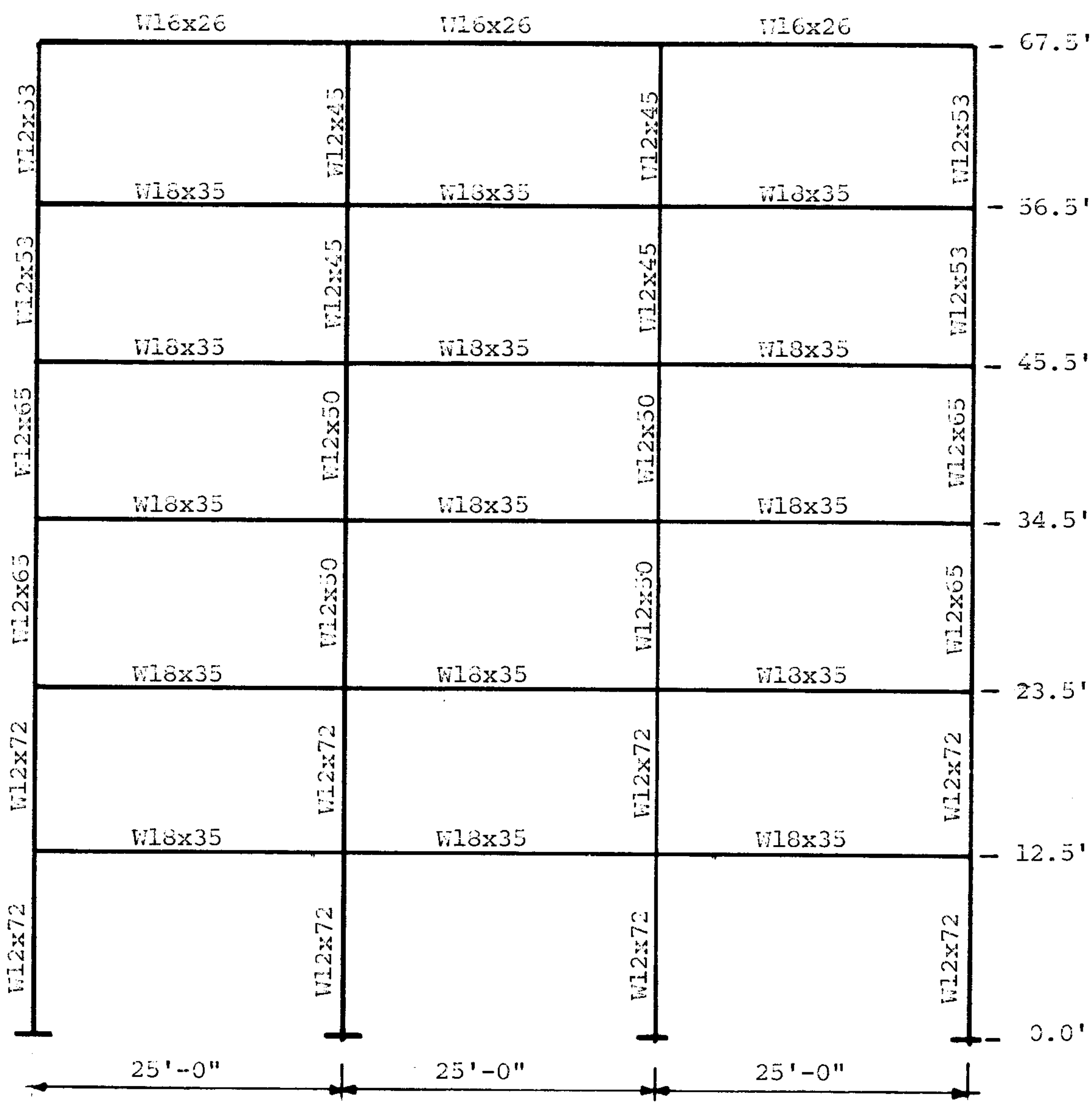

Fig.3.6. Sizes of columns and girders for preliminary design Frame A & E

3.3 Predesign of Frame B,C, and D of the Building

The frames B, C, and D are rigid frames. When designing these frames, the same loading conditions as were used on frames A & E will be duplicated.

1. Dead load + Live load
2. Dead load + Live load + Seismic) x 1/1.33

The frames will be divided in 3 levels

1. The frame from elev. 0.00 to 23.5'
2. The frame from elev. 23.5 to 45.5'
3. The frame from elev. 45.5 to 67.5'

Before going into designing the frame, it is necessary to evaluate the forces acting on the frame. These are forces acting on the joints of frames and forces acting on the beams of the frames.

The forces acting on the beams will produce a bending moment which will be distributed to all the elements of the joint as a function of their rigidity. In this case, it will be the following fix-end actions:

Roof Dead Load

$R = 24.750 \times 1/2 = 12.375$ Kip

$M = \dfrac{24.750 \times 25}{8} = 77.344$ K-ft

Live load

$R = 7.500 \times 1/2 = 3.750$ Kip

$M = \dfrac{7.500 \times 25}{8} = 23.43$ K-ft

The fix end forces are (DL + LL)

$R = 12.375 + 3.750 = 16.125$ Kip

$M = 77.344 + 23.43 = 100.77$ K-ft

The total forces acting on a joint will be:

For exterior joint

R_e = 16.125 + 16.650 + 3.750

= 36.525 Kip

M_{max} = 100.77 K-ft.

For an interior joint

R_i = 16.125 x 2 + 24.750 + 7.500

= 64.5 Kip

Floor

Dead load

P = 31.125 x 1/2 = 15.562 Kip

M = 31.125 x 25 x 1/8 = 97.26 K-ft.

Live load

P = 18.750 x 1/2 = 9/375 Kip

M = 18.750 x 25 x 1/8 = 58.59 K-ft.

The fix end forces for beams will be

(D.L. + L.L.) P = 15.562 + 9.375 = 24.937 Kip

M = 97.26 + 58.59 = 155.85 K-ft.

The total forces acting on each joint will be:

for exterior joint

P = 24.937 + 20.512 + 9.375 = 54.82 Kip

M_{max} = 155.85 K-ft.

for interior joint

P = 24.937 x 2 + 31.125 + 18.750 = 99.749 Kips.

Recapitulation

Roof	interior joint	P = 64.5 Kip
	exterior joint	P = 36.525 Kip

Floor interior joint P = 99.750 Kip

exterior joint P = 54.824 Kip

M = 155.85 K-ft.

Now, it can start designing the frames B,C,D from elevation 0.00' to 23.5'. To find the moment in beams and columns, the simplified moment distribution moment is used as indicated in the sketch below. Since the frame is symmetrical only half is used.

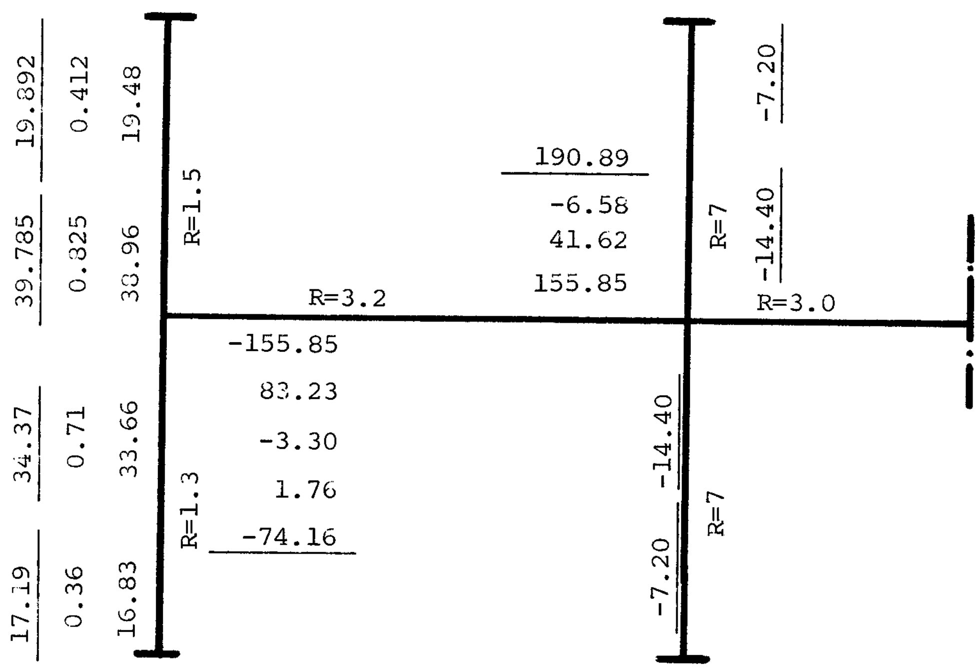

Fig. 3.7. Simplified moment distribution method for frame B,C and D

From this simplified moment distribution, the sizes of the elements of the frame are obtained.

For beam with a bending moment of M = 190.89 K-ft use W21x50 which has a bending moment of 189 K-ft which is very close to its allowable value.

The beam with bending moment of M = 162.05 K-ft use W21x44 which has an allow. bending moment of 163.2 K-ft. The same size beams will be used for all floors.

Elev. 0.00 to 12.5'.

Column Row 1 (See Fig. 3.7)

D.L. + L.L.

P_{total} = P (roof) + 5 P (floors) =

= 36.525 + 5 x 64.5 = 359.025 Kip.

from bending moment the equivalent axial load will be

P = 34.37 x 12 x 0.631 = 294.61 Kip

P_{design} = 359.025 + 294.61 = 653.64 Kip

use W12 x 120 P_{all} = 654 Kip

Column Row 2 (See Fig. 3.7)

(D.L. + L.L.)

P_{total} = P (roof) + 5 P (floors)

= 64.5 = 5 x 99.750 = 563.25 Kip

from bending moment the equivalent axial forces will be

P_{eq} = 14.41 x 12 x 0.217 = 37.52 Kip

P_{design} = 563.25 + 37.52 = 600.77 Kip

use W12 x 120 P_{all} = 654 Kip

Elev 23.5' - 45.5'

Col. Row 1

(D.L. + L.L.)

P = P (roof) + 3 P (floors) =

= 36.525 + 3 x 64.5 = 230.025 Kip

$P_{equivalent}$ from bending moment is

P_{eq} = 34.37 x 12 x .637 = 262.72 Kip

$P_{design} = 492.75$ Kip

use W12 x 96

$P_{all} = 514$ Kip

Col. Row 2

(D.L. + L.L.)

$P = P \text{ (roof)} + 3 P \text{ (floors)} =$

$= 64.5 + 3 \times 99.750 = 363.75$ Kip.

the equivalent axial load from moment will be

$P_{eq} = 14.41 \times 12 \times 0.25 = 37.52$ Kip.

$P_{design} = 363.75 + 37.52 = 401.27$ Kip.

use W12 x 79 with $P_{all} = 439$ Kip.

Elev 45.5 to 67.5'

(D.L. + L.L.)

Col. Row 1

$P_{total} = P \text{ (roof)} + 1 P \text{ (floor)} =$

$= 36.525 + 1 \times 64.5 = 101.25$ Kip

$P_{equiv.}$ from bending moment

$P = 34.37 \times 12 \times 0.657 = 270.97$ Kip.

$P_{design} = 101.25 + 270.97 = 372.22$ Kip

use W12 x 65

Col. Row 2

(D.L. + L.L.)

$P_{total} = P \text{ (roof)} + P \text{ (floor)} =$

$= 64.5 + 99.750 = 164.25$ Kip

P_{eq} from bending moment

$P_{eq} = 14.41 \times 12 \times 0.227 = 39.25$ Kip.

P_{design} = 164.25 + 39.25 = 203.50 Kip.

use W12 x 45

P_{all} = 220 Kip

With loading condition (D.L. + L.L.) satisfied, it is now possible to verify the loading condition for design (D.L. + L.L. + seismic) x 1/1.33.

(D.L. + L.L. + seismic) x 1/1.33

EL 0.00' to 23.5'

<u>Col. Row 1</u>

(D.L. + L.L.) x 0.75 = 653.64 x 0.75 = 490.23 Kip

P (seismic) x 0.75 = 21.65 x 12 x 0.631 x 0.75 = <u>121.00</u> Kip.

611.23 Kip.

W12 x 120 is good

<u>Col. row 2</u>

(D.L. + L.L.) x 0.75 = 600.77 x 0.75 = 450.58 Kip

P (seismic) x 0.75 = 89.75 x 12 x 0.217 x 0.75 = <u>175.28</u> Kip.

625.86 Kip.

W12 x 120 is good

<u>Elev. 23.5' to 45.5'</u>

<u>Col. Row 1</u>

(D.L. + L.L.) x 0.75

P = 492.75 x 0.75 = 369.56 Kip

P (seismic) x 0.75 = 16.60 x 12 x 0.637 x 0.75 = <u>91.72</u> Kip.

P =461.28 Kip.

W12 x 96 is good

<u>Col. Row 2</u>

(D.L. + L.L.) 0.75 = 401.28 x 0.75 = 300.95 Kip

P (seismic) x 0.75 = 66.38 x 12 x 0.217 x 0.75 = 129.66 Kip.

P_{total} = 430.59 Kip.

W12 x 79 is good

El. 45.5' to 67.5'

Col. Row 1

(D.L. + L.L.) x 0.75 = 372.22 x 0.75 = 279.16 Kip.

P (seismic) x 0.75 = 9.26 x 12 x 0.657 x 0.75 = 73.00 Kip.

W12 x 65 is good 352.165 Kip.

Col. Row 2

(D.L. + L.L.) x 0.75 = 203.56 = .75 = 152.625 Kip

P_q (seismic) x 0.75 = 34.45 x 12 x 0.227 x 0.75 = 70.38 Kip.

P_{design} = 223.00 Kip > 220 Kip

is not good and in this case pick up the next size column

use W12 x 50

P_{all} = 246 Kip

Check for the Beam Sizes

1. (D.L. + L.L.) M = 190.89 K-ft.

seismic M = 39.615 K-ft.

M_t=230.505 K-ft.

for design M = 230.505 x 0.75 = 172.87 K-ft.

W21 x 50 is good

2. (D.L. + L.L.) M = 162.06 K-ft.

seismic M = 124.83 K-ft.

286.89 x 0.75 = 215.16 K-ft.

M design = 215.16 K-ft.

The same size of beam will be used as in the first span. W21 x 50. All the sizes of columns and girders of frame B,C,D, are shown on Fig. 3.8.

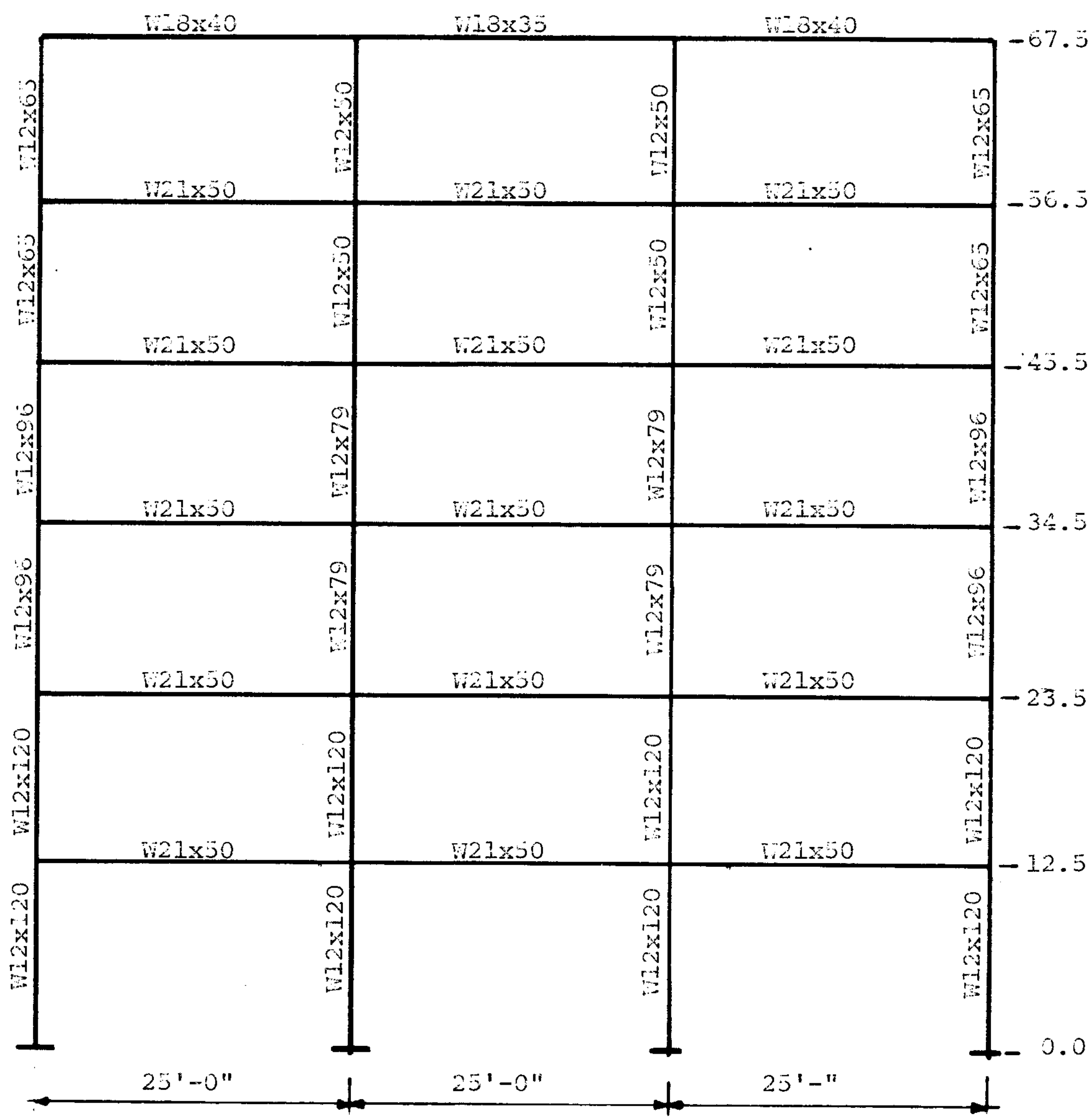

Fig.3.8. Sizes of columns and girders for preliminary design
Frames B,C&D

Having all sizes of the frames, A,B,C,D,E it is now possible to compute by computer for each frame the bending moments and shear forces for each element of the frame and to adjust the sizes , if necessary - in this predesign step.

Table 3 shows the initial sizes of frame A as a result of designing using short cut methods.

To verify the columns, an equivalent axial force has to be found from bending moment.

For this it is used:

$$P_{eq} = B_x M_x + B_y M_y$$

With the equivalent axial load and the actual axial forces in the column, the total axial force has to be smaller than the capacity of the column listed in AISC column chapter .

From analyzing Table 3.1, it can be seen that the following element fell short in sizes :

columns:

8,11,15,16,17,18,29,32,36,39 and beams

40 and 42

With the bending moments and axial forces from Table 3.1 using AISC the correct sizes of the elements under-designed from short-cut methods, can be determined. Introducing the new sizes of the elements of the frame in computer a new bending moments, axial forces and shear forces will be determined. All these are indicated in Table 3.2. Looking at Table 3.2 it can be seen that all the elements of the frame have the corresponding capacity. All these calculations were done on the frame as a two dimensional model.

Table 3.1. Member sizes from short-cut method frame A & E.

Elem. No.	Size	Element Capacity M. Allow Kft.	Element Capacity P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	P.axial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
1	W12x72		390.5	0.655	23.65	186.0	160.2	346.2	8.0	62.88	112.2	175.08
2	W12x72		390.5	0.218	3.41	8.9	313.5	322.4	43.60	114.05	234.4	348.40
3	W12x72		390.5	0.218	3.41	8.9	313.5	322.4	41.51	108.60	235.9	344.50
4	W12x72		390.5	0.655	23.65	186.0	160.2	346.2	27.65	217.32	128.0	345.32
5	W18x35	116.0			97.04				97.36			
6	W18x35	116.0			87.36				91.54			
7	W18x35	116.0			97.04				90.53			
8	W12x72		401.0	0.655	36.43	286.3	132.2	418.13	14.74	115.85	93.12	208.97
9	W12x72		401.0	0.218	6.27	16.40	258.0	274.40	33.39	87.34	193.00	280.34
10	W12x72		401.0	0.218	6.27	16.40	258.0	274.40	23.07	60.35	194.00	254.35
11	W12x72		401.0	0.655	36.43	286.30	132.2	418.53	39.80	312.82	105.10	417.92
12	W18x35	116.0			93.81				100.95			
13	W18x35	116.0			86.88				97.14			
14	W18x35	116.0			93.89				81.58			
15	W12x65		361.0	0.657	32.11	253.15	103.8	356.95	5.11	40.78	74.26	114.54
16	W12x50		246.0	0.228	1.96	5.40	202.9	208.30	45.52	124.54	151.70	276.24
17	W12x50		246.0	0.228	1.96	5.40	202.9	208.30	44.44	121.58	152.40	273.98
18	W12x65		361.0	0.657	32.11	253.15	103.8	356.95	46.14	363.76	81.12	444.80
19	W18x35	116.0			91.61				90.93			
20	W18x35	116.0			86.90				87.80			
21	W18x35	116.0			91.69				72.16			
22	W12x65		361.0	0.657	37.66	296.41	75.35	372.25	24.28	191.42	54.64	246.06

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
23	W12x87		485.0	0.216	30.76	79.72	106.80	186.52	55.78	144.58	89.06	233.64
24	W21x50	189.0			79.21				112.39			
25	W21x50	189.0			75.28				109.87			
26	W21x50	189.0			75.28				109.78			
27	W21x50	189.0			79.21				106.41			
28	W12x87		485.0	0.216	34.26	88.80	77.84	166.64	82.44	213.68	53.65	267.33
29	W12x120		671.0	0.217	2.44	6.35	147.40	153.75	50.40	131.24	110.40	241.64
30	W12x120		671.0	0.217	0	0	146.20	146.20	48.65	126.68	109.70	236.38
31	W12x120		671.0	0.217	2.44	6.35	147.40	153.75	47.76	124.36	110.20	234.56
32	W12x87		485.0	0.216	34.26	88.80	77.84	166.64	50.78	131.62	63.50	195.12
33	W16x50	162.0			78.73				96.23			
34	W16x50	162.0			74.64				94.20			
35	W16x50	162.0			74.64				94.10			
36	W16x50	162.0			78.73				90.60			
37	W12x65		361.0	0.218	32.96	86.22	48.77	134.99	65.78	172.08	34.36	206.44
38	W12x106		593.0	0.216	0.57	1.47	92.11	93.58	34.27	88.82	69.01	157.83
39	W12x106		593.0	0.216	0	0	91.41	91.41	34.54	89.52	68.60	158.12
40	W12x106		593.0	0.216	0.57		92.11	93.58	35.11	91.00	68.92	159.92
41	W12x65		361.0	0.218	32.96	86.22	48.77	134.99	44.92	117.51	38.99	156.00
42	W16x36	113.0			77.82				81.30			
43	W16x36	113.0			74.94				69.51			
44	W16x36	113.0			74.94				78.89			

Table 3.2. Member sizes from computer output (2D) frame A & E.

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
1	W12x87		472.0	0.643	25.72	198.45	163.10	361.55	9.31	71.83	114.20	186.03
2	W12x72		390.5	0.218	2.66	6.95	310.60	317.55	41.70	109.08	232.80	341.88
3	W12x72		390.5	0.218	2.66	6.95	310.60	317.55	40.22	015.21	233.20	338.41
4	W12x87		472.0	0.643	25.72	198.45	163.10	361.55	29.53	227.85	130.30	358.15
5	W18x35	116.0			94.73				95.51			
6	W18x35	116.0			87.21				90.42			
7	W18x35	116.0			94.73				71.43			
8	W12x87		485.0	0.643	38.70	298.60	134.80	433.40	15.34	118.36	94.99	213.35
9	W12x72		401.0	0.218	4.85	12.68	255.40	268.08	32.00	83.71	191.40	275.11
10	W12x72		401.0	0.218	4.85	12.68	255.40	268.08	23.77	62.18	191.60	253.78
11	W12x87		485.0	0.643	30.70	298.60	134.80	433.40	37.17	286.80	107.20	395.00
12	W18x35	116.0			90.83				98.22			
13	W18x35	116.0			87.94				96.13			
14	W18x35	116.0			90.83				83.79			
15	W12x87		485.0	0.643	35.81	276.30	106.10	382.40	8.61	66.43	75.85	142.28
16	W12x53		282.0	0.221	0.81	2.10	200.60	202.70	44.48	116.35	150.30	266.65
17	W12x53		282.0	0.221	0.81	2.10	200.60	202.70	46.10	120.60	150.30	170.89
18	W12x87		485.0	0.643	35.81	276.30	106.10	382.40	49.84	384.56	82.97	467.53
19	W18x35	116.0			88.20				88.16			
20	W18x35	116.0			86.59				87.41			
21	W18x35	116.0			88.20				76.93			
22	W12x87		485.0	0.643	39.74	306.60	77.26	383.80	26.85	207.17	55.92	263.10

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
23	W12x53		282.0	0.221	0.94	2.50	146.00	148.50	10.32	27.36	109.60	136.96
24	W12x53		282.0	0.221	0.94	2.50	146.00	148.50	6.72	17.82	109.60	127.42
25	W12x87		485.0	0.643	39.74	306.60	77.26	383.80	32.88	253.70	60.19	313.89
26	W18x35	116.0			85.36				76.65			
27	W18x35	116.0			86.61				75.59			
28	W18x35	116.0			85.36				64.00			
29	W12x65		361.0	0.657	33.76	266.16	48.41	314.57	19.21	151.45	35.16	186.61
30	W12x45		220.0	0.227	0.24	0.65	91.39	92.04	12.58	34.26	68.57	102.83
31	W12x45		220.0	0.227	0.24	0.65	91.39	92.04	13.55	36.91	68.51	105.42
32	W12x65		361.0	0.657	33.76	266.16	48.41	314.57	32.08	252.91	37.45	290.36
33	W18x35	116.0			88.12				75.77			
34	W18x35	116.0			86.43				74.82			
35	W18x35	116.0			88.12				62.58			
36	W12x65		361.0	0.657	39.95	314.96	19.66	334.29	25.40	200.25	14.37	214.62
37	W12x45		220.0	0.227	2.88	7.84	36.67	44.51	10.70	29.14	27.56	56.70
38	W12x45		220.0	0.227	2.88	7.84	36.67	44.51	6.13	16.69	27.46	44.15
39	W12x65		361.0	0.657	39.95	314.96	19.33	334.29	34.26	247.59	15.09	262.68
40	W16x31	94.0			75.73				61.35			
41	W16x26	77.0			57.53				47.05			
42	W16x31	94.0			75.73				52.49			

Table 3.3. Member sizes from short-cut method frame B , C ,D ,

Elem. No.	Size	Element Capacity M. Allow Kft.	Element Capacity P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
1	W12x120		654.0	0.631	41.80	316.50	296.50	613.00	12.62	95.93	209.90	305.83
2	W12x120		654.0	0.217	5.77	15.02	577.40	592.40	79.40	205.71	432.00	637.71
3	W12x120		654.0	0.217	5.77	15.02	577.40	592.40	75.76	197.28	434.00	631.28
4	W12x120		654.0	0.631	41.80	316.50	296.50	613.00	49.85	377.46	237.50	614.96
5	W21x50	187.0			174.88				174.92			
6	W21x50	187.0			172.04				165.12			
7	W21x50	187.0			174.88				120.12			
8	W12x120		671.0	0.631	64.62	489.30	244.40	733.70	26.50	200.65	174.20	374.85
9	W12x120		671.0	0.217	11.63	30.28	474.90	505.18	56.01	145.85	355.50	501.35
10	W12x120		671.0	0.217	11.63	30.28	474.90	505.18	41.31	107.57	356.90	464.47
11	W12x120		671.0	0.631	64.62	489.30	244.40	733.70	70.28	532.16	191.10	723.26
12	W21x50	187.0			169.51				170.30			
13	W21x50	187.0			156.88				162.90			
14	W21x50	187.0			169.51				127.18			
15	W12x96		534.0	0.637	54.88	419.50	191.80	610.00	20.50	156.70	138.10	294.80
16	W12x79		439.0	0.217	3.88	10.10	373.00	383.00	46.43	120.90	279.30	400.20
17	W12x79		439.0	0.217	3.88	10.10	373.00	383.00	40.46	105.35	280.20	385.50
18	W12x96		534.0	0.637	54.88	419.50	191.80	610.00	61.58	470.71	152.30	623.01
19	W21x50	187.0			166.26				161.84			
20	W21x50	187.0			156.84				156.37			
21	W21x50	187.0			166.26				121.22			
22	W12x96		534.0	0.637	62.55	478.13	139.10	617.23	29.90	228.55	101.50	330.05

Elem. No.	Size	Element Capacity M.Allow Kft.	Element Capacity P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
23	W12x79		439.0	0.217	5.91	15.38	271.10	286.48	41.09	106.99	203.10	310.09
24	W12x79		439.0	0.217	5.91	15.38	271.10	286.48	32.04	83.43	203.60	287.03
25	W12x96		534.0	0.637	62.55	478.13	139.10	617.23	63.73	487.15	109.80	596.95
26	W12x50	187.0			166.21				153.56			
27	W12x50	187.0			157.05				148.15			
28	W12x50	187.0			166.21				109.07			
29	W12x65		361.0	0.657	48.57	371.26	86.54	457.80	23.57	185.82	63.60	249.42
30	W12x50		246.0	0.228	3.30	9.02	169.10	178.12	28.52	78.03	126.70	204.73
31	W12x50		246.0	0.228	3.30	9.02	169.10	178.12	23.44	64.13	126.90	191.03
32	W12x65		361.0	0.657	48.57	371.26	86.54	457.80	48.53	382.61	67.49	450.10
33	W12x50	187.0			167.26				143.42			
34	W12x50	187.0			157.20				136.76			
35	W12x50	187.0			167.26				107.83			
36	W12x65		361.0	0.657	56.59	446.15	34.36	480.51	36.07	275.71	25.24	300.95
37	W12x50		246.0	0.228	8.33	22.79	66.66	89.45	19.14	49.84	50.03	99.87
38	W12x50		246.0	0.228	8.33	22.79	66.66	89.45	6.41	16.69	49.99	66.68
39	W12x65		361.0	0.657	56.59	446.15	34.36	480.51	48.60	383.16	26.20	409.36
40	W18x40	137.0			145.18				115.09			
41	W18x35	116.0			99.17				80.55			
42	W18x40	137.0			145.18				102.56			

Table 3.4. Member sizes from computer output (2D) frame B , C , D ,

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
1	W12x136		741.0	0.620	44.37	330.11	300.60	630.71	13.93	103.63	212.20	315.83
2	W12x120		654.0	0.217	4.92	12.81	573.30	586.11	77.01	200.53	430.30	630.83
3	W12x120		654.0	0.217	4.92	12.81	573.30	586.11	74.06	192.85	430.60	623.45
4	W12x136		741.0	0.620	44.37	330.11	300.60	630.71	52.44	390.15	240.50	630.65
5	W21x50	187.0			172.20				172.75			
6	W21x50	187.0			157.29				163.80			
7	W21x50	187.0			172.20				124.97			
8	W12x136		759.0	0.620	67.31	500.78	248.20	748.98	28.28	210.40	173.30	383.70
9	W12x120		671.0	0.217	9.96	25.93	471.20	497.13	54.58	142.12	354.00	496.12
10	W12x120		671.0	0.217	9.96	25.93	471.20	497.13	42.72	111.24	353.70	464.94
11	W12x136		759.0	0.620	67.31	500.78	248.20	748.98	69.53	517.30	197.80	715.10
12	W21x50	187.0			165.77				167.29			
13	W21x50	187.0			156.29				161.35			
14	W21x50	187.0			165.77				133.87			
15	W12x120		671.0	0.631	60.36	457.04	195.10	652.14	23.26	176.12	139.90	316.02
16	W12x79		439.0	0.217	2.09	5.44	369.70	375.14	43.74	113.89	278.00	391.89
17	W12x79		439.0	0.217	2.09	5.44	369.70	375.14	40.71	106.00	277.40	383.00
18	W12x120		671.0	0.631	60.36	457.04	195.10	652.14	66.92	506.71	154.60	661.31
19	W21x50	187.0			160.97				157.74			
20	W21x50	187.0			156.35				154.38			
21	W21x50	187.0			160.97				130.56			
22	W12x120		671.0	0.631	66.28	501.87	141.70	643.57	33.04	250.17	102.80	352.97

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
23	W12x79		439.0	0.217	2.51	6.53	268.50	275.03	38.15	99.34	202.10	301.44
24	W12x79		439.0	0.217	2.51	6.53	268.50	275.03	34.08	88.74	201.50	290.24
25	W12x120		671.0	0.631	66.28	501.87	141.70	643.57	68.21	516.48	111.16	627.64
26	W21x50	187.0			158.88				148.60			
27	W21x50	187.0			156.21				146.01			
28	W21x50	187.0			158.88				120.26			
29	W12x106		593.0	0.635	59.61	454.22	88.31	542.53	29.00	220.98	64.52	285.50
30	W12x50		246.0	0.228	0.40	1.0	167.30	168.30	25.38	69.43	126.10	195.53
31	W12x50		246.0	0.228	0.40	1.0	167.30	168.30	24.29	66.45	125.60	192.05
32	W12x106		593.0	0.635	59.61	454.22	88.31	542.53	56.23	428.47	68.60	497.07
33	W21x50	187.0			157.83				137.66			
34	W21x50	187.0			155.74				134.45			
35	W21x50	187.0			157.83				105.52			
36	W12x106		593.0	0.635	64.64	492.55	34.96	527.51	39.07	297.71	25.49	323.20
37	W12x50		246.0	0.228	2.72	7.44	66.07	80.44	15.34	41.97	49.87	91.84
38	W12x50		246.0	0.228	2.72	7.44	66.07	80.44	9.60	26.26	49.50	75.76
39	W12x106		593.0	0.635	64.64	492.55	34.96	527.57	53.27	405.91	26.67	432.88
40	W21x44	163.0			136.90				112.39			
41	W18x35	116.0			100.49				80.78			
42	W21x44	163.0			136.90				97.89			

54

For frames B,C,D the initial sises of the elements are indicated on Table 3.3.

From analyzing Table 3.3 a few elements of the frame are underdesign as:

1° column - elem. 8,11,15,18,22,25,29,32,36,39

2° beams - elem - 40,42

With the bending moments and axial forces from Table 3.3 using AISC Manual the sizes of the frame elements are tabulated in Table 3.4.

Examining Fig. 2.1, it can be seen that all the frames in direction south-north are designed and verified with correct sizes by computer in 2D. The exterior columns of all the frames A,B,C,D,E are the columns of the frames 1 and 4.

The frames in the east-west direction will be designed for drift and verified with loading conditions (DL+LL) and (DL+LL+S)x1/1.33 to conform with the U.B.C. recommendations.

3.4 Predesign of Frame 1 and 4

Frames 1 and 4 in direction east-west are rigid frames. The entire seismic force on wind forces in this direction will be taken by these two frames.

U.B.C. states that this kind of frame should be designed for drift. The code provides that all lateral forces must be multiplied by 1/K which means 1/0.67 = 1.5.

The drift limit of the building by code is Δ =0.005 Δ = 0.005 x 67.5 x 12 = 4.05" and drift limit of the story of the building Δ =0.005 x 11 x 12 = 0.65".

The story drift can be determined by the following relationship.

$$\Delta_s = \Delta_c + \Delta_g$$

where

$$\Delta_c = \frac{Fh^3}{12E_{1c}} \text{ and } \Delta g = \frac{Feh^2}{12E_1 I_g}$$

In these formulas the notation are as follows:

F - column shear (see Fig. 5)

h = story height

I_c = moment of inertia of column

I_g = moment of inertia of girder

l = girder length

Δ_s = story drift

Δ_g - contribution of girder to drift

Δ_c = contribution of column to drift

The columns of frames 1 and 4 are the exterior columns of frames A,B,C,D,E from north-south direction and were designed before and in this case from the drift formula it can be determined the moment of inertia of the girder. -

From 0.00 to 12.5'

Δ_s = 0.005 x 12.5 x 12 = 0.75"

The shear forces are determined using portal methods and are as indicated in Fig. 3.2.

$$0.75'' = \frac{25.098 \times 12.5^2 \times 1728}{12 \times 29{,}000 \times 1240} + \frac{25.098 \times 12.5^2 \times 30 \times 1728}{12 \times 29{,}00 \times I_g}$$

$$0.75'' = 0.196'' + \frac{584.17}{I_g}$$

$$I_g = \frac{584.17}{0.554} = 1054 \text{ in}^4$$

use W21 x 57 $\quad I_g = 1140 \text{ in}^4$

the drift will be

$$\Delta_s = 0.195'' + \frac{584.17}{1140} = 0.70''$$

<u>from 12.5' to 23.5'</u>

$$\Delta_s = 0.005 \times 11 \times 12 = 0.65''$$

$F = 23.745$ Kip

$$0.66'' = \frac{23.745 \times 11^3 \times 1728}{12 \times 29{,}000 \times 1240} + \frac{23.745 \times 11^2 \times 30 \times 1728}{12 \times 29{,}000 \times I_g}$$

$$- 0.126'' + \frac{427.99}{I_g}$$

$$I_g = \frac{427.99}{0.534} = 801.47 \text{ in}^4$$

use W21 x 50 $\quad I_g = 984 \text{ in}^4$

$$\Delta_s = 0.126'' + \frac{427.99}{984} = 0.56''$$

<u>from elev. 23.5' to 34.5'</u>

$F = 21.108$ Kip

$$0.66'' = \frac{21.108 \times 11^3 \times 1728}{12 \times 29{,}000 \times 1070} \times \frac{21.108 \times 11^2 \times 30 \times 1728}{12{,}29{,}000 \times I_g}$$

$$0.66'' = 0.130'' + \frac{380.46}{I_g}$$

$$I_g = \frac{380.46}{0.529} = 719 \text{ in}^4$$

use W21 x 50 $\quad I_g = 984 \text{ in}^4$

$$\Delta_s = 0.130'' + \frac{380.46}{984} = 0.51''$$

<u>from elev. 34.5' to 45.5'</u>

$F = 17.268$ Kip

$$0.66'' = \frac{17{,}268 \times 11^3 \times 1728}{12 \times 29{,}000 \times 1070} + \frac{17268 \times 11^2 \times 30 \times 1728}{12 \times 29{,}000 \times I_g}$$

$$0.66'' = 0.106'' + \frac{311.25}{I_g}$$

$$I_g - \frac{311.25}{0.553} = 562 \text{ in}^4$$

use W16 x 50 $\quad I_g = 659 \text{ in}^4$

$$\Delta_s = 0.106 + \frac{311.25}{659} = 0.67''$$

<u>from elev 45.5 to 56.6'</u>

$F = 12.221$ Kip

$$0.66'' = \frac{12.221 \times 11^3 \times 1728}{12 \times 29{,}000 \times 933} + \frac{12.221 \times 11^2 \times 30 \times 1728}{12 \times 29{,}000 \times I_g}$$

$$0.66'' = 0.086 + \frac{220.28}{I_g}$$

$$I_g = \frac{220.28}{0.573} = 338 \text{ in}^4$$

use W16 x 36 $\quad I_g = 448 \text{ in}^4$

$$\Delta_s = 0.086'' + \frac{220.28}{448} = 0.57''$$

<u>from elev. 56.5' to 67.5'</u>

$F = 3.973$ Kip

$$0.66'' = \frac{3.973 \times 11 \times 1728}{12 \times 29{,}000 \times 933} + \frac{3.973 \times 11 \times 30 \times 1728}{12 \times 29{,}000 \times I_g}$$

$$0.66'' = 0.017 + \frac{107.66}{I_g}$$

$$I_g = \frac{107.66}{0.642} = 167.69 \text{ in}^4$$

use W14 x 30 $\quad I_g = 291 \text{ in}^4$

$$\Delta_s = 0.017^4 + \underline{107.66} = 0.38''$$

The total drift will be

$$\Delta_s = 0.70 + 0.56 + 0.51 + 0.57 + 0.57'' + 0.38$$

$$= 3.29''$$

$3.29'' < 4.05''$ OK

This drift will be calculated by computer for a more exact value.

Table 3.5. Member sizes from drift design frame 1 , 2 ,

Elem. No.	Size	Element Capacity		Bx/ By	Loading Condition (DL+LL)				Loading Condition (DL+LL+S).75			
		M. Allow Kft.	P. Allow Kips.		Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
1	W12x87		472.0	0.216	23.85	61.81	164.40	226.22	57.80	149.81	107.30	257.11
2	W12x136		741.0	0.214	1.26	3.23	313.90	317.13	87.85	255.89	235.30	460.89
3	W12x136		741.0	0.214	0	0	310.60	310.60	97.53	250.45	233.20	483.65
4	W12x136		741.0	0.214	1.26	3.23	313.90	317.13	97.95	251.53	234.10	485.63
5	W12x87		472.0	0.216	23.85	61.81	164.40	226.22	78.64	203.83	140.70	344.53
6	W21x57	222.0			81.90				119.92			
7	W21x57	222.0			75.86				114.17			
8	W21x57	222.0			75.86				114.34			
9	W21x57	222.0			81.90				110.42			
10	W12x87		485.0	0.216	34.06	88.28	135.70	223.98	97.43	252.53	89.73	342.26
11	W12x136		759.0	0.214	4.85	12.45	258.20	270.65	72.61	186.46	193.50	379.96
12	W12x136		759.0	0.214	0	0	255.90	255.90	68.27	175.31	192.10	367.41
13	W12x136		759.0	0.214	4.85	12.45	258.20	270.65	66.45	170.64	192.80	363.44
14	W12x87		485.0	0.216	34.06	88.28	135.70	223.98	64.26	166.56	114.90	281.46
15	W21x50	189.0			80.05				116.67			
16	W21x50	189.0			74.98				113.42			
17	W21x50	189.0			74.98				113.04			
18	W21x50	189.0			80.05				110.17			
19	W12x87		485.0	0.216	30.76	79.72	106.80	186.52	68.55	177.68	71.83	249.51
20	W12x120		671.0	0.217	1.60	4.16	202.80	206.96	59.67	155.38	151.90	307.28
21	W12x120		671.0	0.217	0	0	201.00	201.00	59.67	155.38	151.90	307.28
22	W12x120		671.0	0.217	1.60	4.16	202.80	206.96	58.86	153.27	150.90	304.17

Elem. No.	Size	Element Capacity M. Allow Kft.	P. Allow Kips.	Bx/ By	Loading Condition (DL+LL) Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Loading Condition (DL+LL+S).75 Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
23	W12x50		246.0	0.228	3.13	8.56	147.90	156.46	10.12	27.68	110.70	138.38
24	W12x50		246.0	0.228	3.13	8.56	147.90	156.46	3.60	9.84	111.40	121.24
25	W12x65		361.0	0.657	37.66	296.91	75.35	372.25	31.71	250.00	58.60	308.60
26	W18x35	116.0			92.65				79.94			
27	W18x35	116.0			87.08				76.21			
28	W18x35	116.0			92.65				59.84			
29	W12x53		282.0	0.813	26.27	256.30	47.09	303.40	14.68	143.21	34.27	177.48
30	W12x45		220.0	0.227	2.53	6.89	92.70	99.59	15.64	42.60	69.30	111.90
31	W12x45		220.0	0.227	2.53	6.89	92.70	99.59	12.77	34.78	69.76	104.54
32	W12x53		282.0	0.813	26.27	256.30	47.09	303.38	25.14	245.26	36.36	281.62
33	W18x35	116.0			94.99				81.01			
34	W18x35	116.0			87.14				76.64			
35	W18x35	116.0			94.99				61.38			
36	W12x53		282.0	0.813	32.55	317.55	19.10	336.65	20.65	201.46	14.07	215.13
37	W12x45		220.0	0.227	6.73	18.33	37.14	54.47	13.95	37.99	27.80	65.79
38	W12x45		220.0	0.227	6.73	18.33	37.14	55.47	3.63	9.88	27.90	37.78
39	W12x53		282.0	0.813	32.55	317.55	19.10	336.65	27.96	272.77	14.70	287.47
40	W16x26	77.0			83.13				66.11			
41	W16x26	77.0			56.93				47.40			
42	W16x26	77.0			83.13				58.88			

Elem. No.	Size	Element Capacity		Bx/ By	Loading Condition (DL+LL)				Loading Condition (DL+LL+S).75			
		M. Allow Kft.	P. Allow Kips.		Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.	Mz. Kft.	Peq. Kips.	Paxial Kips.	Ptotal Kips.
45	W16x36	113.0			77.82				75.48			
46	W12x65		361.0	0.218	42.42	110.97	19.63	130.60	22.52	58.91	14.03	72.94
47	W12x106		593.0	0.216	0.38	1.00	36.93	37.93	24.42	63.29	27.65	90.94
48	W12x106		593.0	0.216	0	0	36.53	36.53	21.67	56.16	27.42	83.58
49	W12x106		593.0	0.216	0.38	1.00	36.93	37.93	19.09	49.48	27.62	77.10
50	W12x65		361.0	0.218	42.48	110.97	19.63	130.60	44.07	115.28	15.51	130.79
51	W14x30	84.0			59.57				54.92			
52	W14x30	84.0			51.00				49.36			
53	W14x30	84.0			51.00				48.94			
54	W14x30	84.0			59.57				46.75			

Frame 1 & 4 were designed from drift point of view and with the elements sizes obtain a check from (D.L. + L.L.) and (D.L. + L.L. + S) x 1/1.33 loading can be seen in Table 3.5. The sizes of the girder are overdesigned because of ridigity imposed by drift condition.

A clear picture of the drift for frame 1 and 4 is illustrated in Table 3.5 and Fig. 3.9. The value of the drift was calculated by:

- short cut method
- by computer in two dimensional frame
- by computer in three dimensional frame restricting all rotation at floors elevation - and free translation
- by computer in three dimension with free rotations and free translation at floor elevation.

Table 3.6 Drift

Drift Frame 1 (Direction East-West)

		Manual	2D	Free 3D Transl.	Free Rot. 3D Free Tran.
Elev.	0.00	0.00"	0.00"	0.00"	0.00"
	12.50	0.70"	0.40"	0.50"	0.69"
	23.50	1.26"	0.94"	0.81"	1.38"
	34.50	1.77"	1.47"	1.15"	2.03"
	45.50	2.35"	2.00"	1.44"	2.75"
	56.60	2.91"	2.47"	1.66"	3.19"
	67.60	3.21"	2.88"	1.78"	3.56"

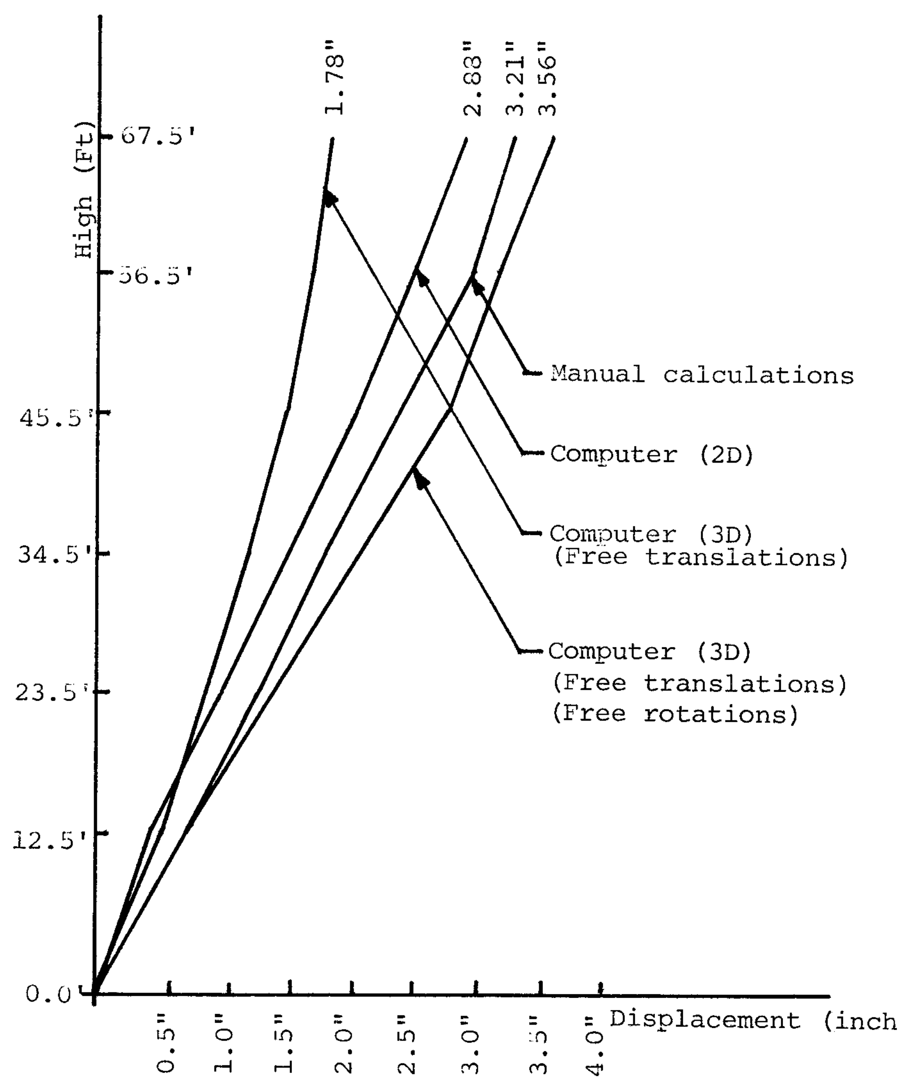

Fig. 3.9 - Displacement of Col. (D,1)

The only member sizes left to be found are the sizes of secondary beams. To predesign secondary beams, consider the beams as continuous beams over four spans and use the maximum bending moment for determining their sizes.

For roof; $M_{max} = 0.116\ Wl^2$

$W = (D.L. + L.L.) = (66 + 20) \times 12.5 = 1075\#/ft.$

use $W = 1.1$ Kip/ft.

$M_{max} = 0.116 \times 1.1 \times 30^2 = 114.84$ K-ft.

Use W18 x 35 with a moment capacity of 115 K-ft.

for floors

$W = (D.L. + L.L.) = (83 + 50) \times 12.5 = 1662.5\#/ft$

Use $W = 1.7$ Kip/ft.

$M_{max} = 0.116 \times 1.7 \times 30^2 = 177.48$ K-ft.

Use W21 x 50 with a moment capacity of 187 K-ft.

With this last calculation, the predesign step is completed. Before going to the next step in design, it will be better to take a look back and draw a few conclusions.

- the short-cut methods used in predesign are fairly adequate methods.

- the member sizes obtained by using the computer in two dimensions were very close to those computed manually.

- the deflection computed manually is very close to the deflection obtained using the computer and having the rotation free and translation free.

The deflections obtained from computer calculations are slightly bigger, which gives the conclusion that the building is fairly elastic. This notice is very important in the last step of the design because the stiffness of the building is one of the dynamic characteristics of the building and will have an influence on how the building responds to the seismic forces.

With these last remarks, the last step of design can be taken: a three dimension analysis by computer and the final member sizes.

3.5 Design of the Building in Three Dimensions

The design of the building in three dimensions will be the most accurate design and will give the possibility of imposing conditions which can be closer to reality.

Since the floors of the building are steel deck filled with 3" concrete, a first check can be made by restricting the rotation of the columns to each floor level. In this case the translational degrees of freedom are allowed.

To this first restriction imposed on the building, the question may be asked: is a 3" concrete fill slab able to restrict the rotations of the columns at each floor? Since a definite answer is not possible, letting the rotation free and translation free will provide a more accurate condition. Add to this that each joint to be considered is as a rigid joint imposed on columns carrying bending moments in two directions.

From the computer output sheets and using the restriction of rotation assumption, it can be seen that the bending moments in columns are small. Using free rotation and free translation contrary to the above restriction, columns are subjected to bigger bending moments in both directions which, in the end, providethe final member sizes.

Since the building is symmetrical in both directions ,the lateral forces can act from every direction. For this reason the four columns from the north-west of the building are considered with all the adjacent beams and they are analyzed and extended to the entire building.

In finalizing the sizes of beams and columns the following were taken in consideration :

Beams - the compression flange is continuousally braced by the floor or roof deck filled with 3½ concrete.

Columns - the effective length factor in the X direction (K_x) and the effective length factor in the Y direction (K_y) were taken as 1 (one). This is based on the fact that G_a and G_b from AISC Manual , Fig. C1.8.2 is equal to zero since the beams are continuousally braced.

For the computer input the X axis is along the column line A and the Y axis is along the row line 1 (one) See Fig 2.1.

The results are shown in the Table 3.1

With all final sizes of members , the structural design is completed. The secondary design will be the calculations and details of connections, column buckling,etc.

Table 3.7. Final column sizes from three dimension design (3D) by computer

El. No.	Size	P all	By/ Bx	Loading Condition (DL+LL) M_2	M_3	Peq	P ax	P tot	Loading Cond. (DL+LL+S)0.75 M_2	M_3	Peq	P ax	P tot	Final Size
Column (E,4) - 3D (Free rotations & free translations) X direction														
16	W12x87	472.0	0.643		17.60	135.80	161.2			13.2	101.8	128.6		
			0.216	60.17		155.96		452.9	57.2		148.2		378.7	
67	W12x87	485.0	0.643		27.46	211.88	134.4			20.6	158.9	106.6		
			0.216	71.61		185.61		531.8	66.1		171.4		437.0	
118	W12x87	485.0	0.643		26.57	205.01	106.2			19.9	153.8	83.5		
			0.216	41.88		108.55		419.7	45.3		117.6		355.0	
169	W12x87	485.0	0.643		31.15	240.35	77.6			23.3	180.4	60.5		
			0.216	46.59		120.76		438.7	46.5		120.6		361.6	
220	W12x65	361.0	0.657		28.62	225.64	48.8			21.4	271.8	37.6		
			0.218	42.20		110.39		384.9	41.4		108.3		417.9	
271	W12x65	361.0	0.657		38.57	304.00	19.6			28.9	228.5	15.1		
			0.218	52.41		125.00		449.2	44.7		117.0		360.6	
3D (Free rotations & free translation) Y direction														
16	W12x87	472.0	0.643							51.0	393.5	135.6		
			0.216						45.0		116.7		645.8	W12x120
67	W12x87	485.0	0.643							51.0	393.5	111.5		
			0.216						53.5		138.6		643.6	W12x120
118	W12x87	485.0	0.643							49.0	378.7	86.7		
			0.216						31.1		80.8		546.3	W12x120
169	W12x87	485.0	0.643							46.0	355.6	62.1		
			0.216						34.7		89.9		507.7	W12x120

El. No.	Size	P all	By/ Bx	Loading Condition (DL+LL) M2	M3	Peq	P ax	P tot	Loading Cond. (DL+LL+S)0.75 M2	M3	Peq	P ax	P tot	Final Size
220	W12x65	361.0	0.657							37.6	296.9	38.28		
			0.218						31.3		81.8		417.0	W12x87
271	W12x65	361.0	0.657							38.0	299.9	15.2		
			0.218						39.0		102.2		417.3	W12x87
	Column (E,3) - 3D (Free rotations + free translations) X direction													
15	W12x72	390.0	0.655		8.05	63.27				29.1	228.7	216.0		
			0.218	27.15		71.02	289.5	423.7	20.3		53.2		498.0	W12x96
66	W12x72	401.0	0.655		7.40	58.16				18.4	144.9	176.8		
			0.218	46.03		120.41	236.7	415.2	34.5		90.3		412.0	W12x96
117	W12x53	282.0	0.813		3.24	31.60				22.9	223.9	138.4		
			0.221	28.77		76.29	185.3	293.1	21.5		57.2		419.6	W12x79
168	W12x53	282.0	0.813		1.53	14.92				20.5	200.0	100.4		
			0.221	32.61		85.68	134.4	235.0	24.4		64.8		365.2	W12x79
219	W12x45	220.0	1.065		2.68	34.28				15.1	194.0	62.6		
			0.227	20.07		54.67	83.8	172.7	15.0		40.9		297.6	W12x58
270	W12x45	220.0	1.065		0.70	8.94				8.8	113.6	25.0		
			0.227	24.28		66.13	33.5	108.5	18.2		49.6		188.2	W12x58
	3D (Free rotation + free translation) Y direction													
15	W12x72	390.5	0.655							6.1	48.4			
			0.218						23.8		62.2	219.6	330.3	
66	W12x72	401.0	0.655							5.6	44.7			
			0.218						37.9		99.3	179.5	323.5	
117	W12x53	282.0	0.813							2.2	22.1			

El. No.	Size	P all	By/ Bx	Loading Condition (DL+LL) M_2	M_3	Peq	P ax	P tot	Loading Cond. (DL+LL+S)0.75 M_2	M_3	Peq	P ax	P tot	Final Size
			0.221						24.3		64.6	140.3	227.0	
168	W12x53	282.0	0.813							0.8	8.6			
			0.221						2.6		7.0	101.7	117.4	
219	W12x45	220.0	1.065							1.8	23.5			
			0.227						17.3		47.3	63.3	134.1	
270	W12x45	220.0	1.065							0.3	4.8			
			0.227						19.3		52.6	25.3	82.7	
	Column (D,4) - 3D (Free rotations + free translations) X direction													
17	W12x136	741.0	0.620		2.87	21.35	317.8			2.1	16.0	252.9		
			0.214	61.20		157.16		496.3	64.3		165.3		434.2	
68	W12x136	759.0	0.620		5.40	40.17	262.2			4.0	30.1	207.8		
			0.214	75.82		194.70		497.0	86.0		220.9		458.8	
119	W12x120	671.0	0.631		3.22	24.38	206.3			2.4	18.2	162.5		
			0.217	75.65		196.99		427.6	83.4		217.7		398.0	
170	W12x120	671.0	0.631		3.87	29.30	150.2			2.8	21.7	117.2		
			0.217	75.48		196.54		376.0	75.5		196.8		335.7	
221	W12x106	539.0	0.635		2.20	16.76	92.9			1.4	11.2	71.7		
			0.216	89.70		232.50		342.2	87.3		226.3		309.3	
272	W12x106	593.0	0.635		4.93	37.56	37.9			3.7	28.1	29.1		
			0.216	91.10		236.13		311.5	78.3		202.9		260.3	

El. No.	Size	P all	By/ Bx	Loading Condition (DL+LL) M_2	M_3	Peq	P ax	P tot	Loading Cond. (DL+LL+S)0.75 M_2	M_3	Peq	P ax	P tot	Final Size
			3D (Free rotations + free translations) Y direction											
17	W12x136	741.0	0.620							74.6	555.2			
			0.214						46.0		118.1	237.5	910.8	W12x170
68	W12x136	759.0	0.620							54.0	042.1			
			0.214						61.3		157.4	195.9	755.5	W12x170
119	W12x120	671.0	0.631							49.0	371.4			
			0.217						56.9		148.1	154.3	673.8	W12x120
170	W12x120	671.0	0.631							39.8	301.8			
			0.217						58.4		152.0	112.4	566.2	W12x120
221	W12x106	593.0	0.635							26.9	205.1			
			0.216						67.1		173.9	69.5	448.6	W12x106
272	W12x106	593.0	0.635							14.7	112.1			
			0.216						68.3		177.2	28.4	317.8	W12x106
	Column (D,3) (Free rotation + free translation) X direction													
14	W12x120	654.0	0.631		0.52	3.93				58.2	440.8			
			0.217	2.60		6.77	582.4	593.1	1.9		4.9	434.8	880.0	W12x170
65	W12x120	671.0	0.631		3.36	25.44				44.4	336.3			
			0.217	8.55		22.26	479.3	527.0	6.4		16.6	357.9	710.9	W12x170
116	W12x79	439.0	0.649		2.45	19.08				39.8	309.9			
			0.217	5.65		14.71	376.6	410.3	4.2		11.0	281.3	591.2	W12x120
167	W12x79	439.0	0.649		5.11	39.79				38.4	299.5			
			0.217	9.21		23.90	273.7	337.4	6.9		17.9	204.6	522.3	W12x120

El. No.	Size	P all	By/ Bx	Loading Condition (DL+LL) M_2	M_3	Peq	P ax	P tot	Loading Cond. (DL+LL+S)0.75 M_2	M_3	Peq	P ax	P tot	Final Size
218	W12x50	246.0	1.050		10.60	133.56				29.1	366.9			
			0.228	3.11		8.50	170.9	312.9	2.3		6.4	128.0	501.3	W12x106
269	W12x50	246.0	1.050		3.57	44.98				15.0	189.1			
			0.228	4.67		12.77	65.5	125.2	3.5		9.5	50.7	249.5	W12x106
			3D (Free rotations + free translations) Y direction											
14	W12x120	654.0	0.631							0.2	2.0			
			0.217						9.0		23.5	437.1	462.6	
65	W12x120	671.0	0.631							2.2	17.1			
			0.217						0.2		1.0	359.8	377.9	
116	W12x79	439.0	0.649							2.0	16.1			
			0.217						2.2		5.8	282.8	304.8	
167	W12x79	439.0	0.649							4.0	31.7			
			0.217						1.6		4.4	205.6	241.7	
218	W12x50	246.0	1.050							8.2	104.2			
			0.227						0.8		2.1	128.5	234.8	
269	W12x50	246.0	1.050							2.7	34.0			
			0.227						2.0		5.4	51.1	90.6	

Table 3.8. Final beams sizes from three dimension design (3D) by computer

Element No.	Size	M Cap.	DL + LL M	(DL+LL+S)1/133 X Dir. M.	(DL+LL+S)1/133 Y Dir. M.	Obs.
39	W21x50	187.0	60.44	100.46	108.63	
40	W21x50	187.0	156.82	154.12	117.43	
41	W21x50	187.0	146.08	109.55	101.13	
42	W18x35	115.0	90.50	82.00	67.92	
43	W18x35	115.0	96.71	72.09	72.78	
44	W21x57	222.0	86.73	65.02	94.63	
45	W21x50	187.0	159.73	143.91	119.56	
46	W21x57	222.0	78.78	59.06	115.90	
90	W21x50	187.0	134.17	100.62	108.72	
91	W21x50	187.0	155.95	154.77	116.77	
92	W21x50	187.0	147.50	110.62	102.55	
93	W18x35	115.0	89.34	83.53	67.05	
94	W18x35	115.0	86.38	86.62	67.38	
95	W21x50	187.0	84.13	63.06	93.98	
96	W21x50	187.0	154.19	157.27	115.36	
97	W21x50	187.0	76.82	57.59	108.44	
141	W21x50	187.0	135.76	101.81	108.93	
142	W21x50	187.0	155.81	149.08	116.76	
143	W21x50	187.0	150.50	112.88	105.95	
144	W18x35	115.0	87.13	82.49	65.40	

Element No.	Size	M Cap.	DL + LL M	(DL+LL+S)1/133 X Dir. M.	(DL+LL+S)1/133 Y Dir. M.	Obs.
145	W18x35	115.0	85.90	85.58	64.03	
146	W21x50	187.0	83.05	62.23	89.50	
147	W21x50	187.0	153.74	156.35	110.92	
148	W21x50	187.0	76.91	57.65	102.06	
192	W21x50	187.0	138.23	103.66	109.25	
193	W21x50	187.0	156.33	141.96	116.35	
194	W21x50	187.0	151.50	113.63	107.77	
195	W18x35	115.0	86.91	78.38	65.23	
196	W18x35	115.0	86.51	81.88	64.46	
197	W16x50	162.0	82.24	61.63	76.58	
198	W21x50	187.0	160.88	153.69	120.96	
199	W16x50	162.0	76.42	57.29	89.86	
243	W21x50	187.0	142.39	106.78	110.40	
244	W21x50	187.0	154.65	130.82	116.04	
245	W21x50	187.0	151.35	113.82	109.39	
246	W18x35	115.0	86.61	73.31	64.99	
247	W18x35	115.0	87.34	75.83	63.96	
248	W16x36	113.0	80.81	60.51	64.88	
249	W21x50	187.0	170.35	141.95	112.78	
250	W16x36	113.0	75.96	55.49	75.29	
294	W18x35	115.0	87.20	65.42	66.95	

Element No.	Size	M Cap.	DL + LL M	(DL+LL+S)1/133 X Dir. M.	(DL+LL+S)1/133 Y Dir. M.	Obs.
295	W18x35	115.0	102.50	81.14	88.97	
296	W18x35	115.0	137.66	103.24	101.38	W18x40
297	W16x26	77.0	88.64	46.88	43.94	W16x31
298	W16x31	94.0	63.37	49.03	47.76	

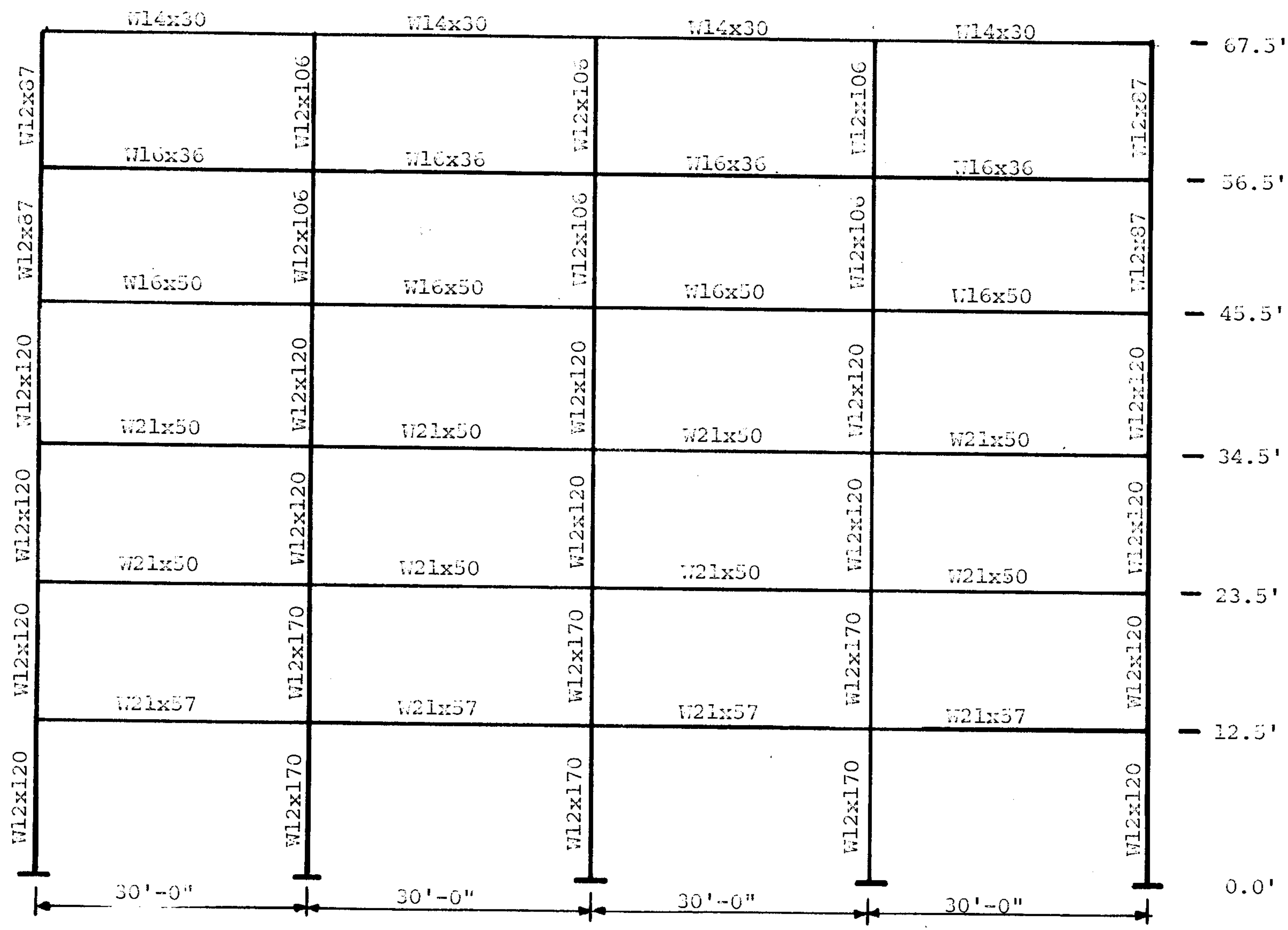

Fig.3.10. Line row 1 - Final member sizes.

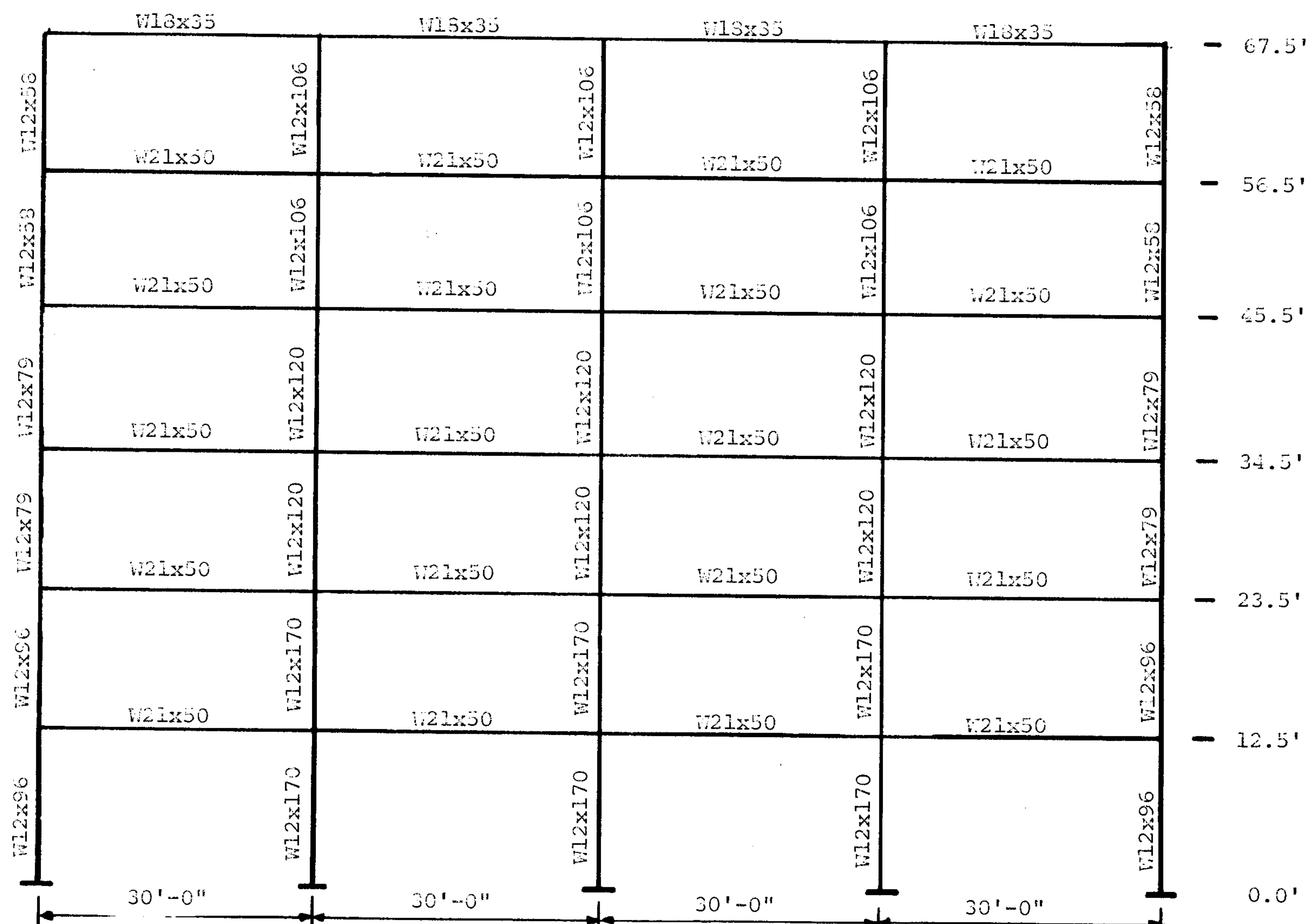

Fig.3.11. Line row 2 -Final member sizes.

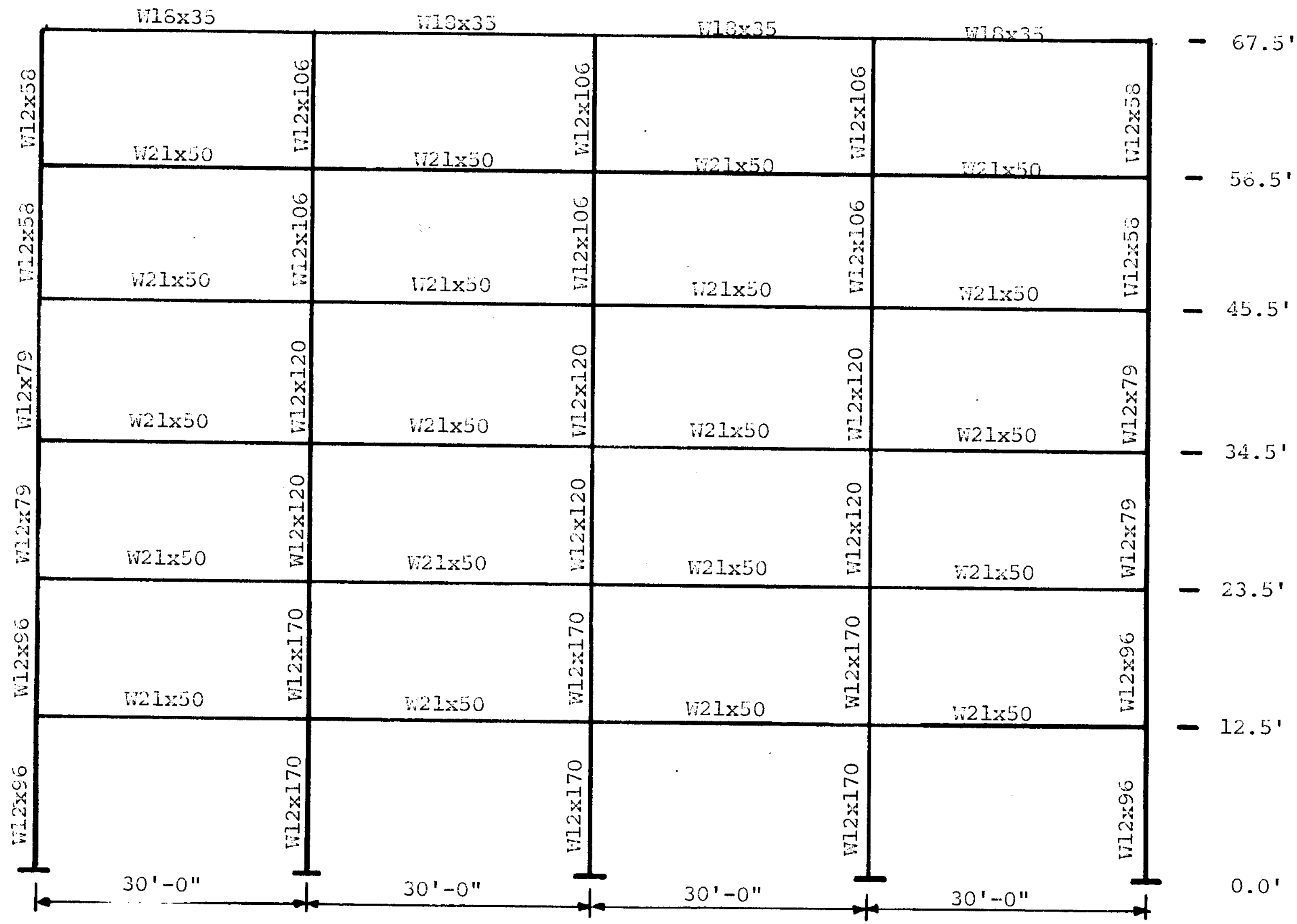

Fig.3.12. Line row 3 - Final member sizes.

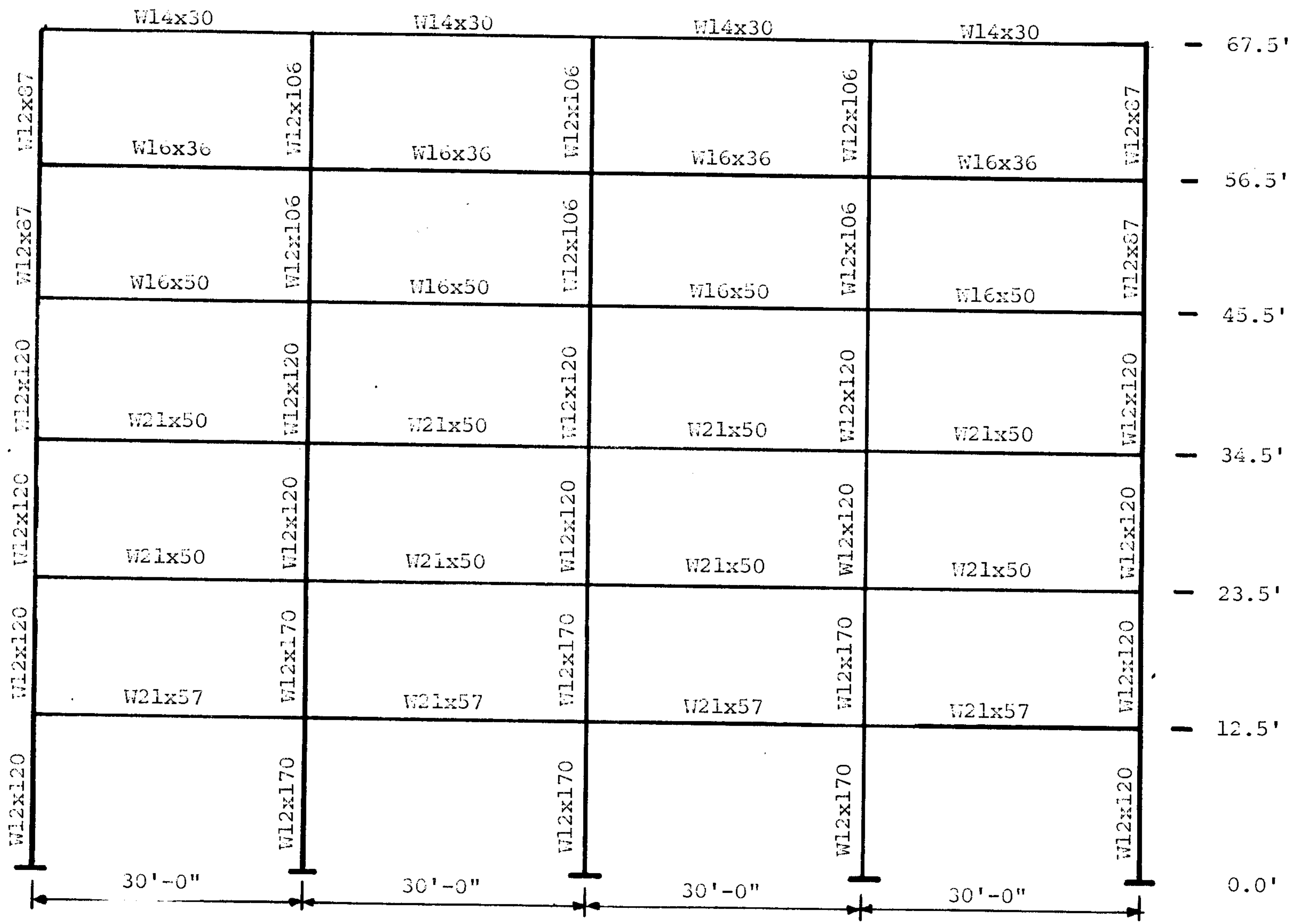

Fig. 3.13. Line row 4 - Final member sizes.

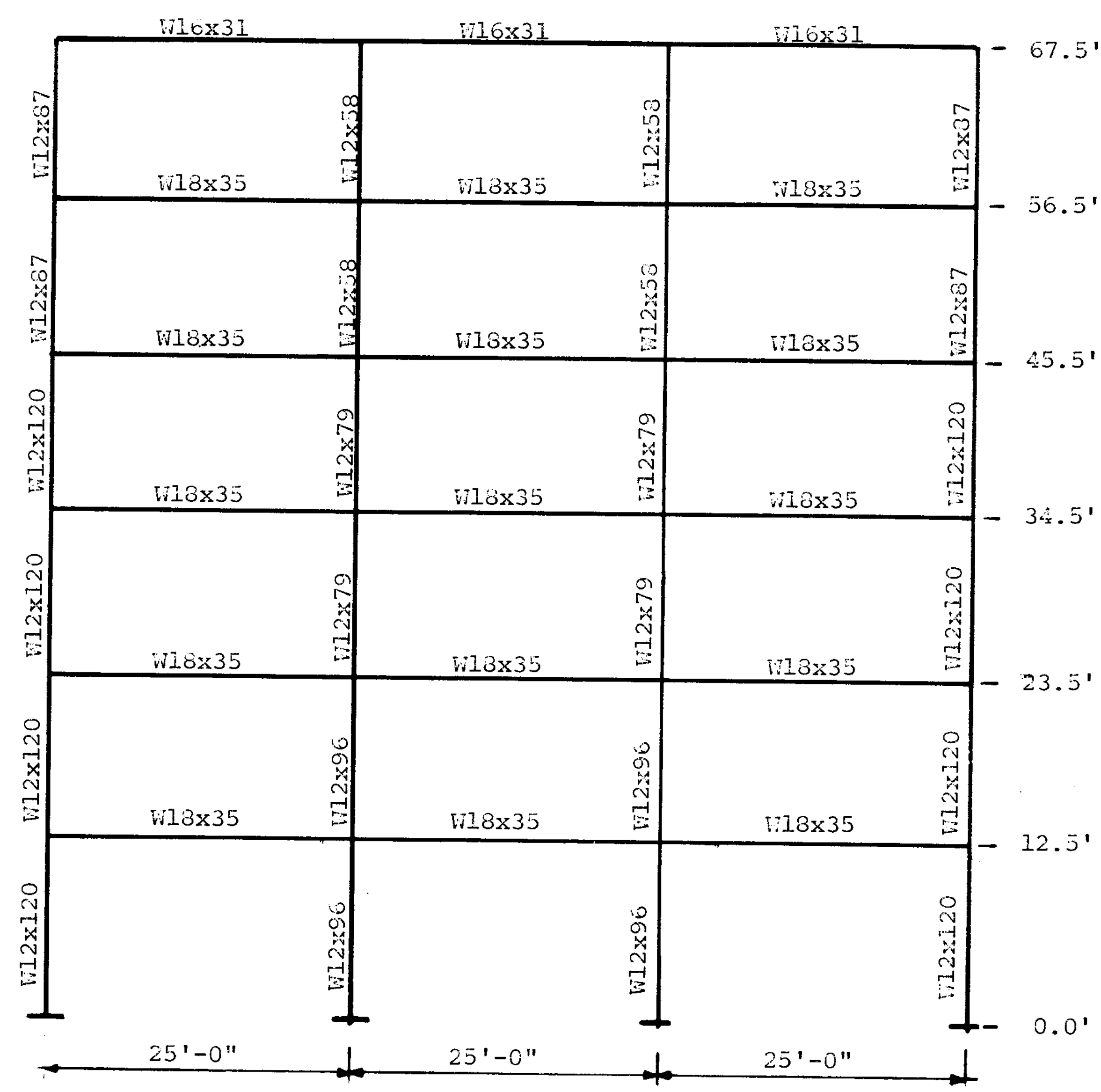

Fig.3.14. Column line A -Final member sizes

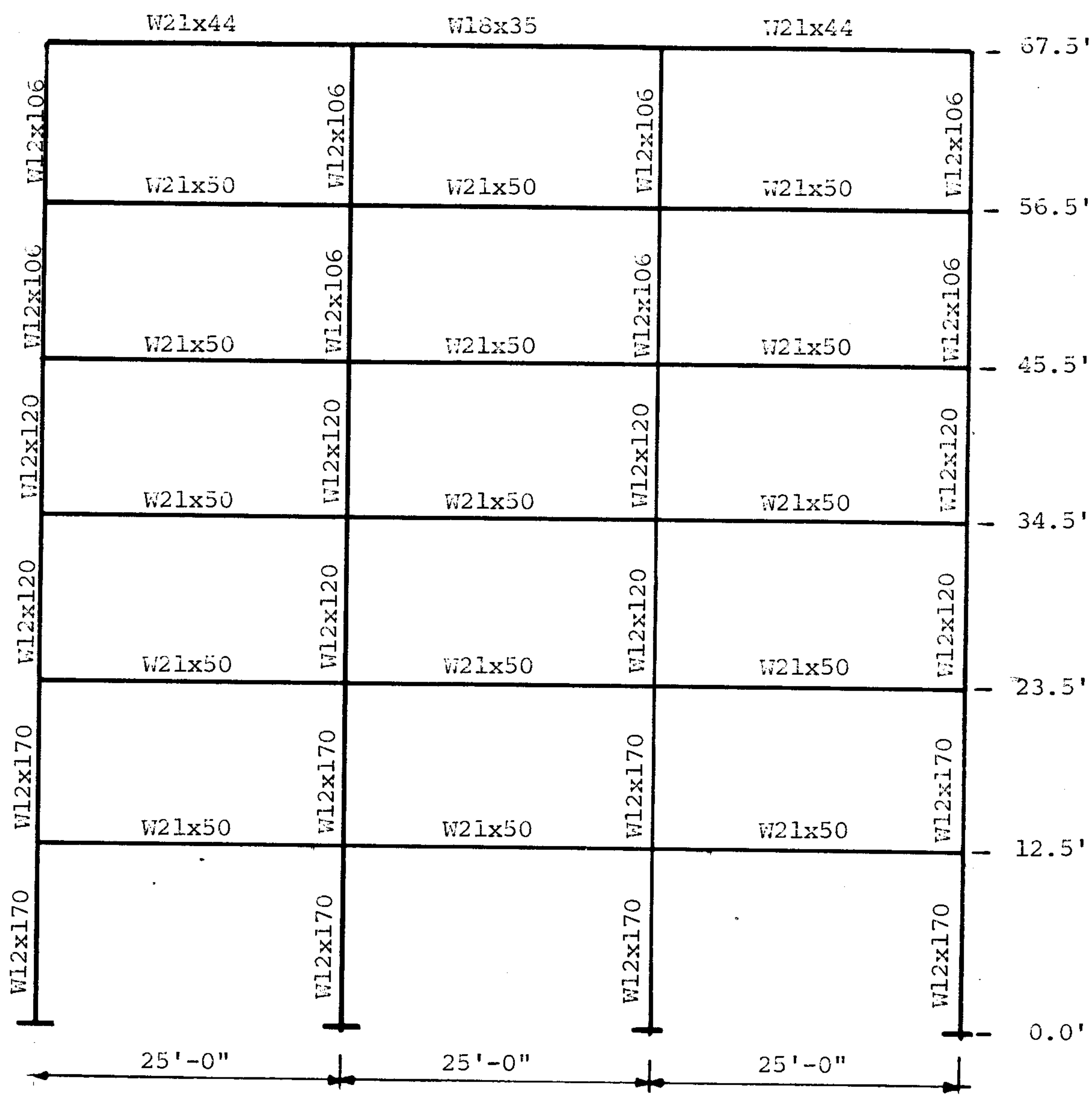

Fig. 3.15. Column line B -Final member sizes

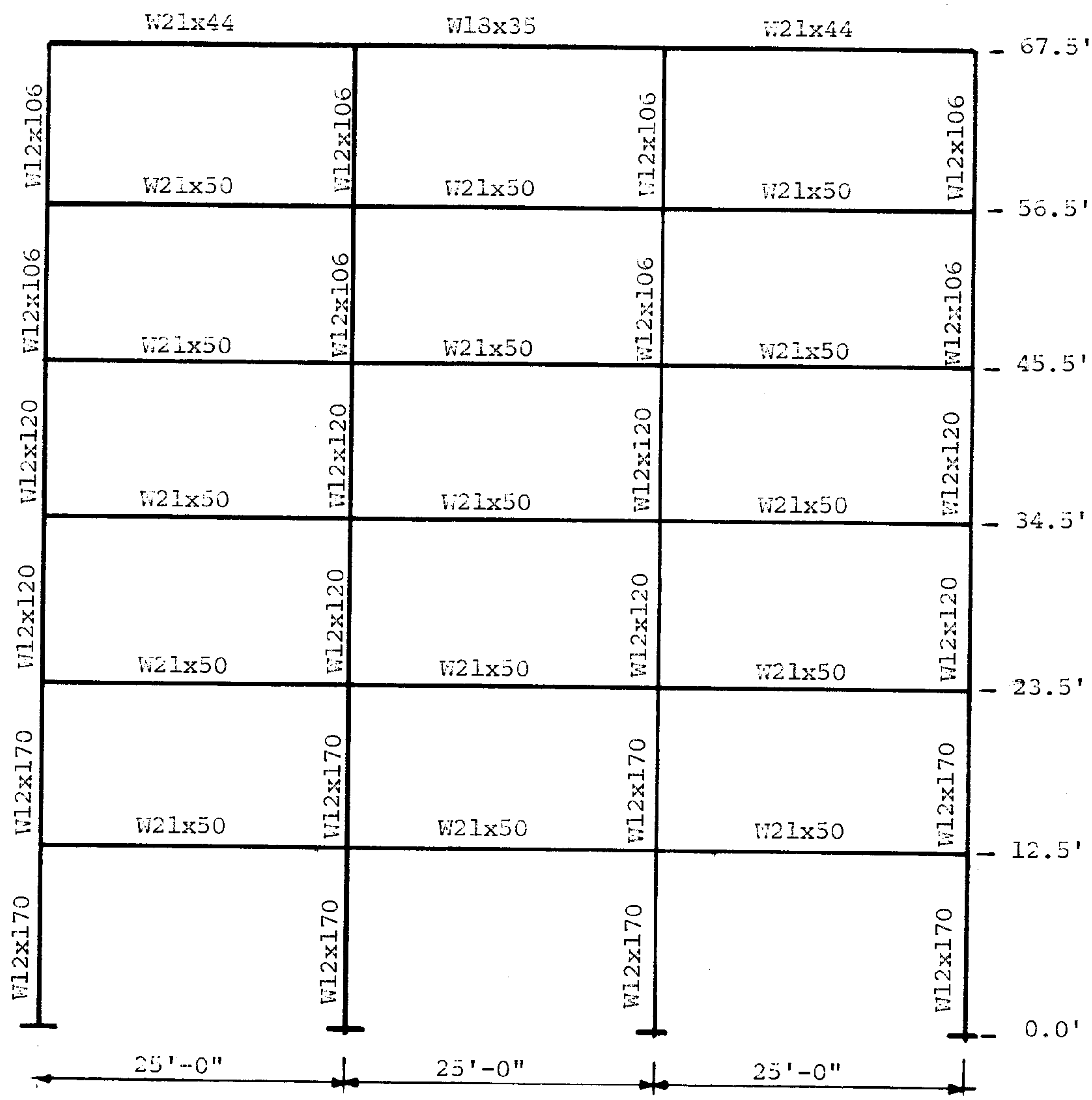

Fig.3.16. Column line C -Final member sizes.

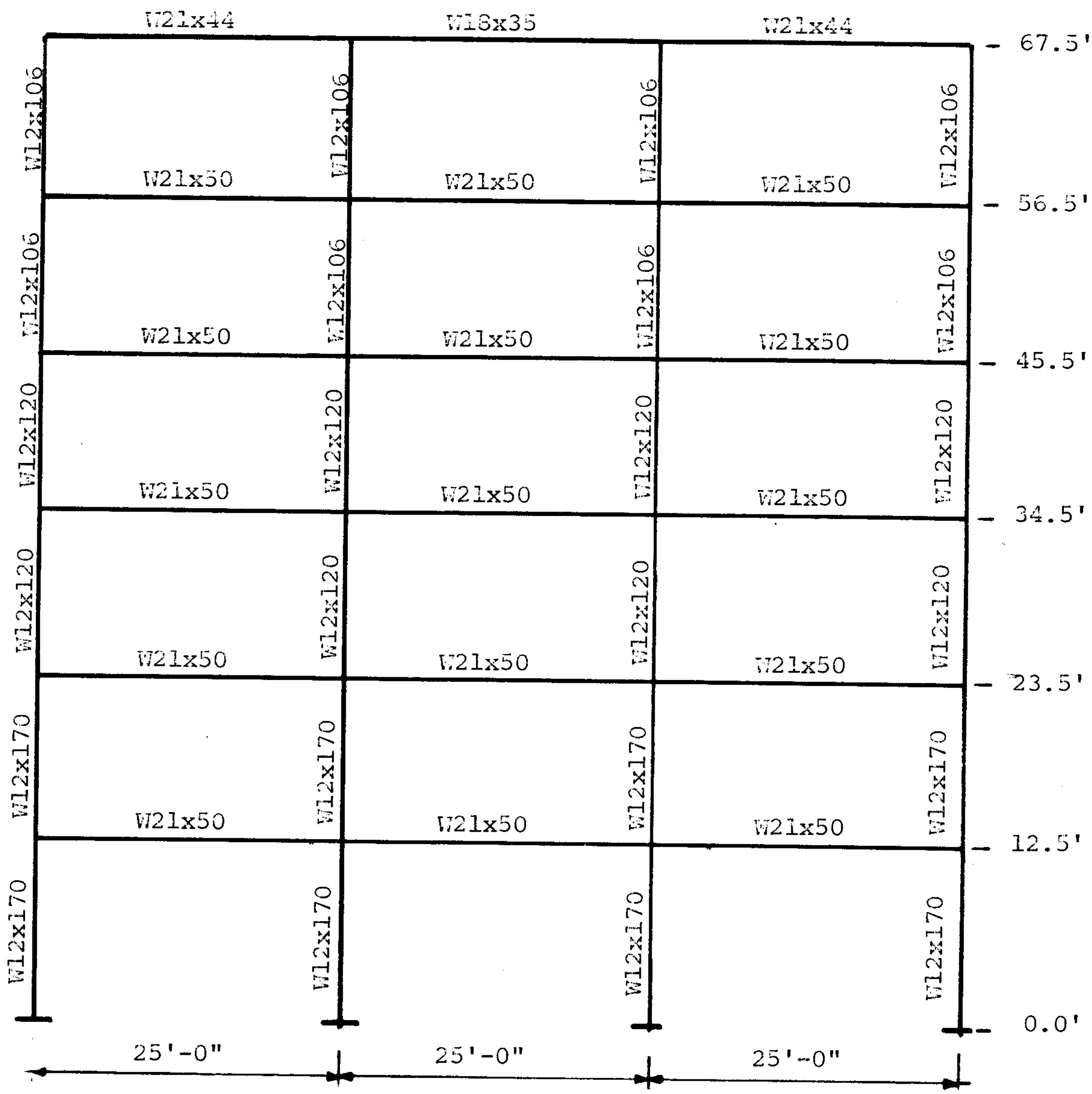

Fig.3.17. Column line D -Final member sizes

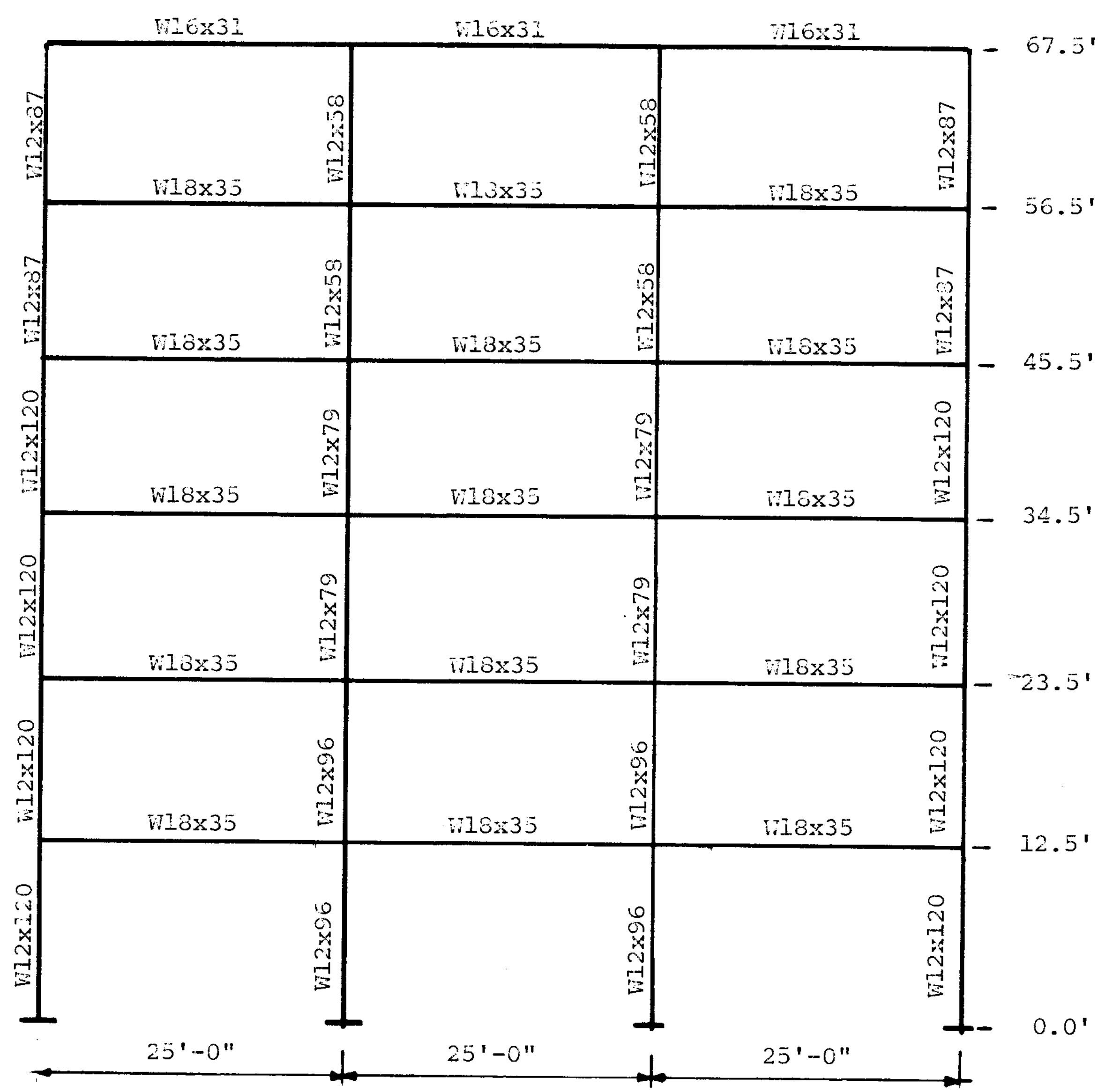

Fig. 3.18. Column line E -Final member sizes.

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CHAPTER IV

Dynamic Behavior of the Building

All parts of a structure exert static vertical loads on the structure due to their stationary dead weight, but they also exert dynamic loads due to inertia forces, if they are in motion. Dynamic forces are functions of the dynamic characteristics of structure and vary with every change in motion, at any fixed point in time, generally depending on the preceeding motion as well as motion at the instant considered. Every structure will vibrate with the laws of harmonic motion as determined by its own dynamic characteristics, which are functions of its weight and stiffness. A building response to the motion of its base is determined by those dynamic characteristics.

The computer program SAP IV was used to do the dynamic analysis. It uses the model superposition method to determine the dynamic response.

To determine the period of vibration, two different cases were tried. One case with rotation fixed for which the period of vibration is 1.7 and the second case was rotation free for which period of vibration is 2.1 seconds.

The first case seems to be more realistic, because it incorporates the rigidity of the floors. Therefore, the for subsequent analysis all rotations were fixed. It also helps in reducing the degrees of freedom, which in time results in saving computer time and storage.

This dynamic analysis is done with the sizes of members from predesign stage giving the possibility of a good judgement in choosing the final member sizes in the final design.

4.1.1 The Frequency and Time Period of the Building.

From this case the following results were obtained.

Table 4.1 for the frequency and time period.

Mode No.	Circular Freque. Rad/Sec.	Frequency Cycles/Sec.	Time period Secs.
1	3.684	0.5864	1.7050
2	4.161	0.6623	1.5010
3	6.135	0.9765	1.0240
4	6.285	1.0000	0.9996
5	7.878	1.2540	0.7975
6	8.075	1.2850	0.7781
7	8.185	1.3030	0.7677
8	8.319	1.3240	0.7553
9	8.358	1.3300	0.7518
10	8.487	1.3510	0.7403

4.1.2 Mode Shape

Column (B,3) was selected to plot the mode shape.

From computer output the following results were obtained ;

Joint	Mode 1	Mode 5	Mode 8
132	$+0.1915 \times 0.00$	$+0.2052 \times 0.00$	-0.1138×10^{-4}
109	$+0.1281 \times 0.00$	$+0.4760 \times 10^{-1}$	$+0.1756 \times 10^{-4}$
92	$+0.1039 \times 0.00$	-0.1150×0.00	-0.1690×10^{-4}
69	$+0.1887 \times 10^{-1}$	-0.1730×0.00	-0.3462×10^{-5}
52	$+0.4908 \times 10^{-1}$	-0.1373×0.00	$+0.1600 \times 10^{-4}$
29	$+0.2970 \times 10^{-1}$	-0.9140×10^{-1}	$+0.1670 \times 10^{-4}$
12	0.00	0.00	0.00

With this displacement from the output, the mode shapes can be easy built. The mode shapes were computed in three dimensions and a plot in 3D will show the real mode shape of the building.

Fig. 4.1. shows the mode shape in Y direction of column (B,3) (see Fig.2.1.), which does not give a real picture of the way the displacement is working.

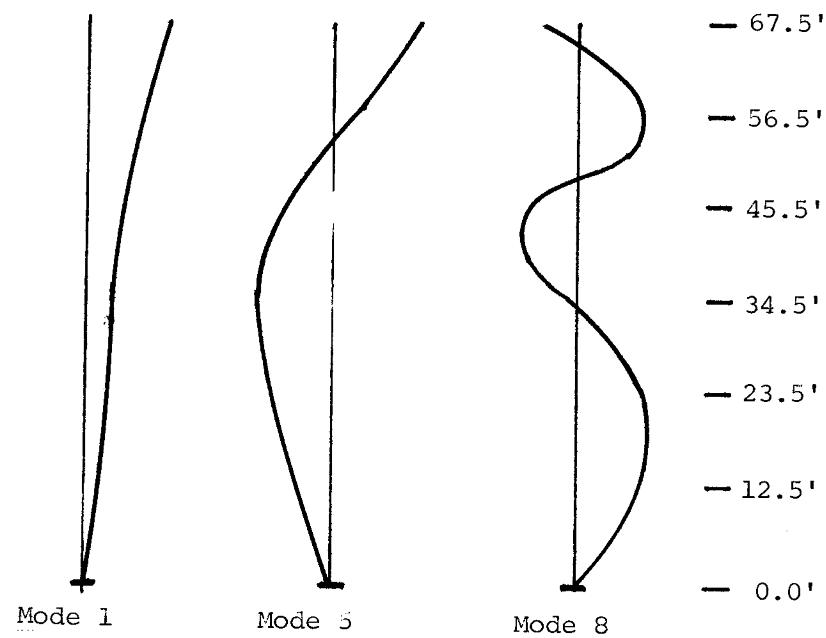

Fig 4.1 - Mode Shape

4.1.3 Computer Time Comparation

Using the case with fixed rotations and fixed translation in X and Z direction and D.T. = 0.1 the Eigen solution time is:

NF = 1	1.09 second
NF = 5	1.99 second
NF = 10	4.00 second
NF = 15	7.34 second
NF = 20	13.34 second

For the case of free translations and D1 = 0.1 Eigen solution time is:

NF = 1 5.47 second

NF = 5 9.36 second

NF = 10 21.43 second

With all these values from computer sheets it can be built the following graph.

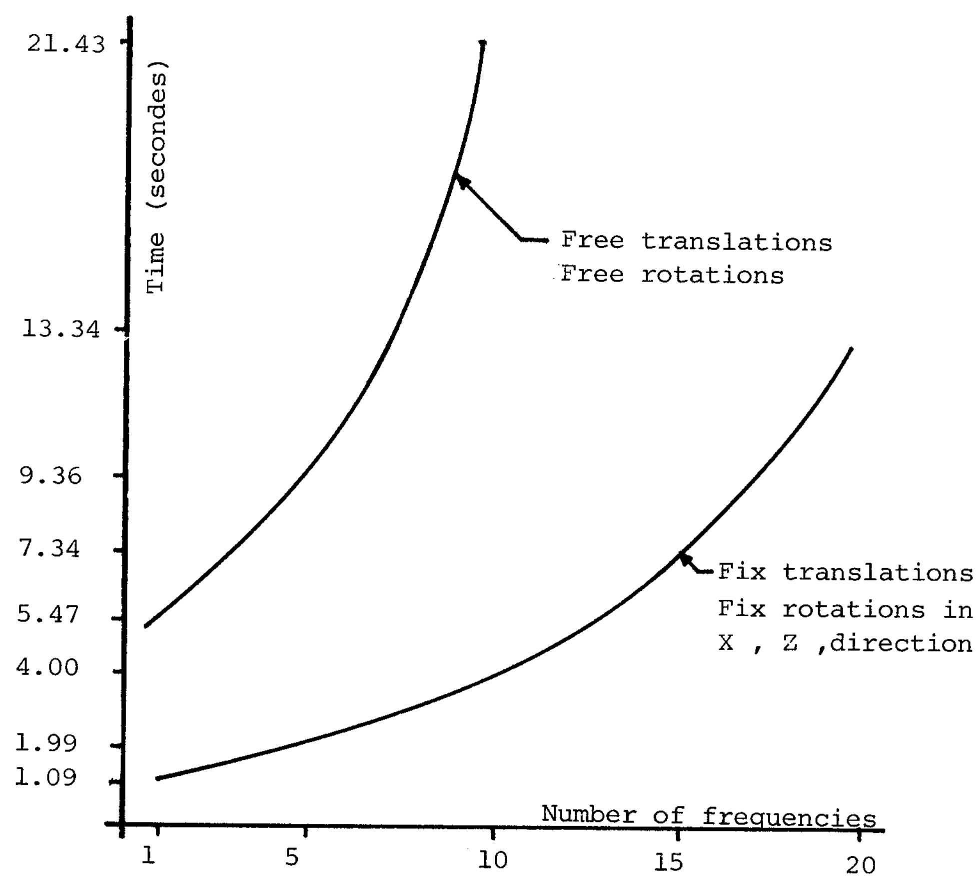

Fig. 4.2 - Computer time comparation

4.2. Earthquake Response of the Building

The three dimensional dynamic response of the building was evaluated by applying a ground acceleration at the base of the building. The data from the El Centro - 1940 earthquake were used to make the analysis more realistic. The acceleration versus time plot for north-south component of this earthquake is shown in Fig. 4.3.and the numerical data is given in Table 4.2. Using the adequate number of frequency (NF) and time step (DT) will result in saving computer time which generally means saving money.

Considering the case where besides fixing all rotations, translations will be fixed also in direction X and Z yields the following results :

Number of Frequency	Maximum Story Disple.
NF = 1	0.924'
NF = 5	0.915'
NF = 10	0.918'
NF = 15	0.918'
NF = 20	0.918'

From the above results it is evident that going beyond NF = 10 will be wasting computer time. Choosing the right time step in an earthquake response is also very important.

With NF = 10 choosing the same case of fix all rotation fix translation in direction X and Z with diff. DT. the result is:

DT	Max. Displacement
DT = 1.0	= 6.57'
DT = 0.10	= 0.918'
DT = 0.05	= 0.687'
DT = 0.025	= 0.628'

As it can be seen from the above, choosing DT = 0.025 will give a displacement of 0.628'.

Table 4.2. El Centro 1940 north-south component data

Time Value	Function	Time Value	Function	Time Value	Function
1	2	3	4	5	6
0.0	0.1080D-01	0.04200	0.1000D-02	0.09700	0.1590D-01
0.26300	0.1000D-03	0.29100	0.5900D-02	0.33200	-0.1200D-02
0.47100	0.7600D-2	0.58100	0.4250D-01	0.62300	0.9400D-02
0.72100	-0.2560D-01	0.78900	-0.3870D-01	0.79000	-0.5680D-01
0.94100	-0.4020D-01	0.94200	-0.6030D-01	0.99700	-0.7890D-01
1.09400	-0.4290D-01	1.16800	0.8970D-01	1.31500	-0.1696D 00
1.44000	-0.9450D-01	1.48100	-0.8850D-01	1.50900	-0.1080D 00
1.70300	0.2355D 00	1.80000	0.1428D 00	1.85500	0.1777D 00
2.21500	0.2952D 00	2.27000	0.2634D 00	2.32000	-0.2984D 00
2.51900	-0.4690D-01	2.57500	0.1516D 00	2.65200	0.2.77D 00
2.89300	0.1033D 00	2.97600	-0.8030D-01	3.06800	0.5200D-01
3.25300	-0.2060D 00	3.38600	0.1927D 00	3.41900	-0.9370D-01
3.66800	0.3650D-01	3.73800	-0.7360D-01	3.83500	0.3110D-01
4.05600	-0.4350D-01	4.10600	0.2160D-01	4.22200	-0.1972D 00
4.47100	-0.4700D-02	4.61800	0.2572D 00	4.66500	-0.2045D 00
4.97000	0.1779D 00	5.03900	0.3010D-01	5.10800	0.2183D 00
5.30200	0.1290D 00	5.33000	0.1089D 00	5.34300	-0.2390D-01
5.60600	0.1410D-01	5.69000	-0.1949D 00	5.77300	-0.2420D-01
5.86900	-0.5730D-01	5.88300	-0.3270D-01	5.92500	0.2160D-01
6.08500	-0.6650D-01	6.13200	0.1400D-02	6.17400	0.4930D-01
6.22900	-0.3810D-01	6.27900	0.2070D-01	6.32600	-0.5800D-02
6.40900	0.2000D-01	6.45900	-0.1760D-01	6.47800	-0.3300D-02
6.56200	-0.9900D-02	6.57500	-0.1700D-02	6.60300	-0.1700D-01

Time Value	Function	Time Value	Function	Time Value	Function
1	2	3	4	5	6
6.71400	0.3850D-01	6.72800	0.9000D-03	6.76900	-0.2880D-01
6.85200	0.2200D-02	6.90800	0.9200D-02	6.99100	-0.9960D-01
7.14300	-0.2770D-01	7.14900	0.2600D-02	7.17100	0.2720D-01
7.37000	0.2970D-01	7.40600	0.1090D-01	7.42500	0.1860D-01
7.57200	0.3600D-02	7.60000	-0.6280D-01	7.64100	-0.2800D-01
7.75200	-0.5400D-02	7.79400	-0.6030D-01	7.83500	-0.3570D-01
7.98700	-0.5600D-02	8.00100	0.2220D-01	8.07000	0.4680D-01
8.19500	-0.1280D-01	8.22300	0.6610D-01	8.27800	0.3050D-01
8.45800	-0.3690D-01	8.53300	-0.3440D-01	8.59600	-0.1040D-01
8.8100	-0.2800D-02	8.86000	0.2330D-01	8.88200	-0.2610D-01
9.05300	0.1260D 00	9.09500	0.3200D-01	9.12300	0.9550D-01
9.28900	-0.4510D-01	9.42700	0.1301D 00	9.44100	-0.1657D 00
9.70400	0.8160D-01	9.81500	-0.8810D-01	9.89800	0.6400D-02
10.02200	-0.7130D-01	10.05000	-0.4480D-01	10.05100	-0.2210D-01
10.18800	0.5100D-01	10.27200	-0.1243D 00	10.38200	0.5870D-01
10.45600	0.1164D 00	10.50700	-0.3740D-01	10.53400	-0.5720D-01
10.71400	0.5150D-01	10.77000	0.9030D-01	10.83900	-0.19000D-02
10.96400	-0.7940D-01	10.99100	-0.1200D-01	11.07400	0.6080D-01
11.20700	0.2930D-01	11.20800	0.5520D-01	11.22700	0.7560D-01
11.43400	0.1180D 00	11.57300	-0.9900D-02	11.65600	-0.1247D 00
11.7800	-0.1183D 00	11.80800	0.0	11.87700	0.7620D-01
12.04300	0.6730D-01	12.11300	0.8650D-01		

Time Value	Function	Time Value	Function
7	8	9	10
0.15100	0.1000D-03	0.22100	0.1890D-01
0.37400	0.2000D-01	0.42900	-0.2370D-01
0.66500	0.1380D-01	0.72000	-0.8800D-02
0.87200	-0.2320D-01	0.87300	-0.3430D-01
1.06600	-0.6660D-01	1.06700	-0.3810D-01
1.38400	-0.8280D-01	1.41200	-0.8280D-01
1.53700	-0.1280D 00	1.62800	0.1144D 00
1.92400	-0.2610D 00	2.00700	-0.3194D 00
2.39500	0.5400D-02	2.45000	0.2865D 00
2.70800	0.1097D 00	2.76900	-0.3250D-01
3.12900	-0.1547D 00	3.21200	0.6500D-02
3.53000	0.1703D 00	3.59900	-0.3590D-01
3.90400	-0.1833D 00	4.01400	0.2270D-01
4.31400	-0.1762D 00	4.41600	0.1460D 00
4.75600	0.6080D-01	4.83100	-0.2733D 00
5.19900	0.2670D-01	5.23300	0.1252D 00
5.45400	0.1723D 00	5.51000	-0.1021D 00
5.80000	-0.5000D-02	5.80900	-0.2750D-01
5.99000	0.1080D-01	6.01300	0.2350D-01
6.18800	0.1490D-01	6.18900	-0.2000D-01
6.36800	-0.6030D-01	6.38200	-0.1620D-01
6.52000	0.4300D-02	6.53400	-0.4000D-02
6.64500	0.3730D-01	6.68600	0.4570D-01
6.77000	0.1600D-02	6.81100	0.1130D-01
7.07400	0.3600D-01	7.12100	0.7800D-02

Time Value	Function	Time Value	Function
7	8	9	10
7.22600	0.5760D-01	7.29500	-0.4920D-01
7.46100	-0.2530D-01	7.52500	-0.3470D-01
7.66900	-0.1960D-01	7.69100	0.6800D-02
7.87700	-0.7160D-01	7.96000	-0.1400D-01
8.12600	0.2600D-01	8.12700	-0.3350D-01
8.33400	0.2460D-01	8.40300	0.3470D-01
8.63800	-0.2600D-01	8.73500	0.1534D 00
8.91500	-0.2200D-02	8.95600	-0.1849D 00
9.15000	0.1246D 00	9.25300	-0.3280D-01
9.51000	0.4190D-01	9.63500	-0.9360D-01
9.93900	-0.6000D-03	9.99500	0.5860D-01
10.10500	0.9300D-02	10.10600	0.2400D-02
10.42400	0.1330D-01	10.45200	0.3860D-01
10.64500	0.3080D-01	10.70100	0.2230D-01
10.92200	0.4710D-01	10.92300	-0.6770D-01
11.08800	-0.2690D-01	11.11600	-0.4160D-01
11.26800	0.4310D-01	11.32400	0.2080D-01
11.72500	-0.2094D 00	11.72600	-0.1419D 00
11.91900	0.5700D-01	11.99800	0.1354D 00

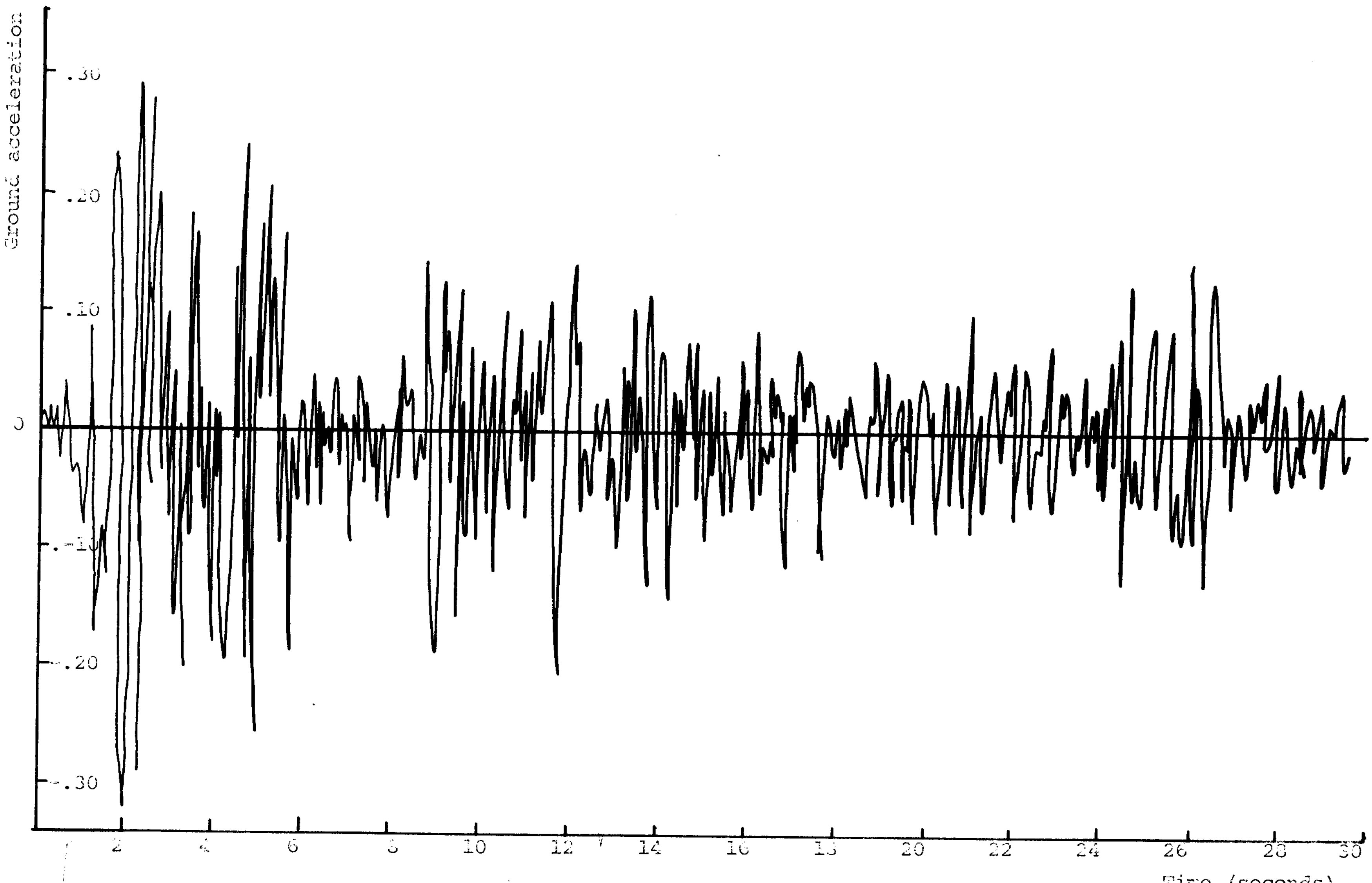

Fig. 4.3. 1940 El Centro earthquake ground motion acceleration in north-south direction

The way the system is chosen with two frames along the Y direction and five frames in the X direction, the larger response to the earthquake should be in the Y direction.

Using full system with DT = 0.1 and NF = 10 and with an earthquake directed along the Y direction, the maximum displacement in Y direction is 0.916 ft.

The same system used with the earthquake along the X direction, the maximum displacement was found to be 0.3124 ft. As was assumed, the largest displacement occurred along the Y direction.

Responses along the X and Y directions are independent of each other. Using the member sizes obtained from predesign, it is possible to see the response of the building using a 3 (three) dimensional earthquake excitation. From the output, it was found that in this case maximum displacement is of 0.6301 ft. at 8.97 seconds.

From these results it can be concluded that the building is elastic. The 0.6301 ft. maximum displacement can be reduced in two ways

- by reducing the mass (which in this case is not adequate)
- in increasing the rigidity of the members of the building

Since this analysis was done with the sizes of the members obtained from predesign, in order to determine the final size of members this aspect of the problem was taken into consideration.

The final selection of the members of the building is done in the last part of Chapter III. With the new size of the members:

- using free translation on X,Y,Z
- three dimensions earthquake excitation
- damping coefficient = 0.05
- DT = 0.025
- NF = 10

The results including the frequencies , are shown in Table 4·3.

Table 4.3

Print of Frequencies

Mode Number	Circular Frequency (Rad/Sec)	Frequency (Cycles/Sec)	Period (Sec)
1	0.4926D 01	0.7840D 00	0.1276D 01
2	0.5353D 01	0.8520D 00	0.1174D 01
3	0.6787D 01	0.1080D 01	0.9257D 00
4	0.6932D 01	0.1103D 01	0.9064D 00
5	0.9391D 01	0.1495D 01	0.6691D 00
6	0.9499D 01	0.1512D 01	0.6614D 00
7	0.9612D 01	0.1530D 01	0.6537D 00
8	0.9960D 01	0.1585D 01	0.6308D 00
9	0.1030D 02	0.1639D 01	0.6101D 00
10	0.1276D 02	0.2030D 01	0.4926D 00

The response of all the top story nodes can be seen on the output sheet. From all the nodes, only node 132 was chosen. Node 132 has the largest displacement and is shown on the graph in X,Y,Z direction.

From the computer output of the earthquake response, it can be seen that:

- Maximum displacement due to earthquake excitation is of 0.5265.' which seems to be very reasonable in comparison with the intensity of the earthquake.

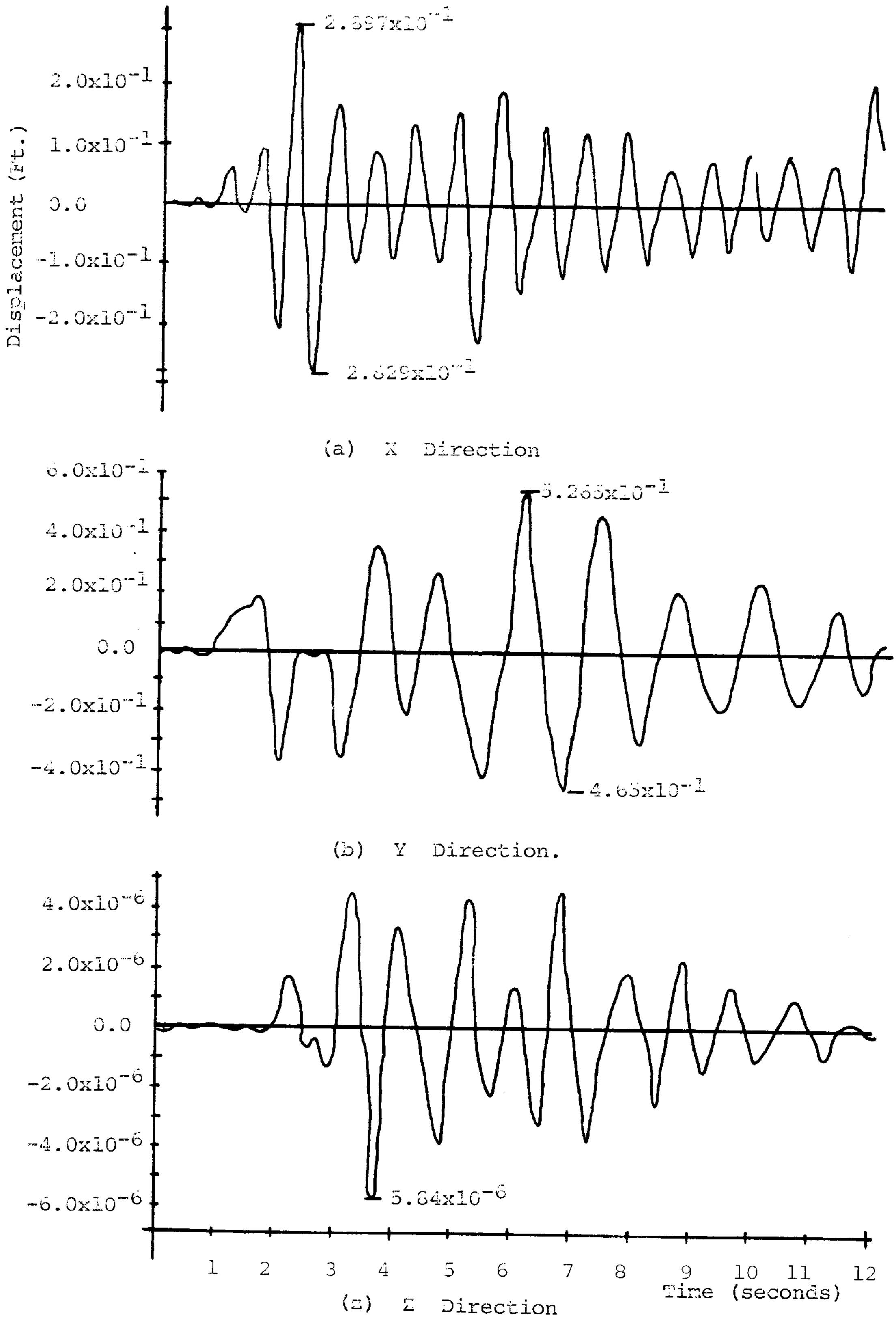

Fig.4 .4. Dynamic response of joint 132

(a) X Direction.

(b) Y Direction.

(c) Z Direction.

- Due to vertical loads, the size of the columns towards the roof gets smaller, but from earthquake excitation we learn that reduction in size of the column needs to be done more conservatively. This will be one way to reduce the top displacement.

- The orientation of the columns has an important role in controlling the displacement.

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CHAPTER V

Conclusions

Designing a building from an engineering point of view can become very involved and challenging depending on the way loads are carried down to the foundation. The safety of the building for all kinds of loading is a main concern. Earthquake-resistant design is a wide and immature subject with difficulty in deciding what design criteria and analytical methods should be applied for achieving a minimum destruction in the event of an earthquake.

Designing an office building with ductile moment frames in both directions give the possibility of seeing the pros and cons of this kind of structure. Conclusions drawn from such of an analysis include :

\- Since the columns were subjected to biaxial bending rigid joints were assumed and thus , their size had to be increased.

\- It was noticed that the selection of the columns seems to be determined by the stresses in columns from the second level.

\- More rigid columns should be used towards the top of the building even if calculations from dead and live loadings indicate that it is not necessary.

\- Special attention should be given to the corner columns of the building.

-The way the system is modelled has an important impact on how the building behaves.

-At Frame A and Frame E (see Fig.2.1) ,the orientation of the columns on Row Line 2 and 3 should be changed. In fact , instead of using a rigid frame ,a braced frame should be employed.

- The floor beams are slightly larger in size than the effective stress required with the exception to line row 1 and line row 4 where the beams were designed from a drift point of view which imposed the rigidity of the beams as a deciding factor for their sizes.

-The only underdesign elements of the floor and roof were two roof beams. This shows that a more rigid roof, even to the extent of using larger than required beams sizes, will be helpful for the deflection and the dynamic behavior of the building in an earthquake.

- These conclusions are applicable for buildings up to 10 (ten) story height ; beyond this height more careful attention should be accorded to the relative rigidity because the results are very sensitive especially to the lateral loading.

REFERENCES

1 John E. Lothers, "Advanced Design in Structural Steel".

2 Beedle, Blackmon, Cooper, Driscoll, Fisher, Galambos, Lu, Slutter, Tall, Yen, "Structural Steel Design".

3 Salvadori and Levy, "Structural Design in Architecture".

4 AISC, "Manual of Steel Construction".

5 D. J. Dowrick, "Earthquake Resistant Design".

6 Department of the Army, the Navy and the Air Force, April 1973, "Seismic Design for Buildings".

7 Klauss-Jurgen Bathe, Edward L. Wilson & Fred E. Peterson, "SAP IV: A Structural Analysis Program for Static and Dynamic Response of Linear Systems.

8 Edward J. Teal. "Seismic Design Practice for Steel Buildings".

9 Uniform Building Code, 1980 edition.

10 American Society of Civil Engineers, March 1976, "Commentary on Plastic Design"