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DESIGN OF A MULTI-STORY STEEL FRAME

by

MUKESH N. JHAVERI

B. E. (Civil) M. S. University of Baroda, India, 1970

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Peter B. Cooper
Major Professor

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1. INTRODUCTION

1.1 Problem

The behavior of a multi-story rigid frame structure is very complex, and there is no specific point at which one can start the analysis and design. An exact analysis of a frame depends on the prior knowledge of the member cross-section properties. In order to select the frame members one must know the maximum values of the internal forces caused by the application of different loading conditions; that is, the results of an analysis must be available. In brief, analysis and design of a rigid frame structure are not independent steps, and it is a problem for a designer to determine how to start the analysis and design process.

1.2 Purpose

The purpose of this report is to demonstrate by example one method of carrying out a preliminary analysis and design of a multi-story, steel, rigid frame structure.

1.3 Scope

The design of the beams and columns of an interior bent of a multi-story steel frame presented in this report is based on the following limitations and assumptions:

1. Rigid frame construction is assumed.
2. The members consist of ASTM A36 steel (1) W-shapes.

3. Lateral loads are resisted by frame action; that is, no bracing is provided.
4. Lateral load analysis is done by the portal method.
5. The allowable stress design approach will be used.

2. DESIGN CONDITIONS AND ASSUMPTIONS

2.1 Frame Geometry

The dimensions of the frame considered in this report are similar to those of the "Braniff Building," located in Dallas, Texas (2). Figure 1 shows a typical intermediate bent. The bents are spaced 25 feet center to center and there are 11 bays in the structure.

2.2 Special Features of the Building

The floor to ceiling height is to be fixed at 9' 6". This leaves a total vertical space of 2' 6" for the floor, floor girders and floor beams, ceiling and utilities. A maximum floor girder depth of 18" is consistent with these conditions (Figure 2).

The exterior walls, consisting of precast concrete masonry panels clip-bolted to the steel frame, are assumed to be located relative to the columns such that the center of gravity of the wall is in the same plane as the exterior surfaces of the column flanges.

Reinforced concrete foundations are assumed, and it is further assumed that the columns are rigidly connected to the foundations. This affects the strong axis effective length factors used in the column design for level 9 - 10.

Wind bracing is provided by using fully welded beam-to-column connections. The intermediate floor beam connections

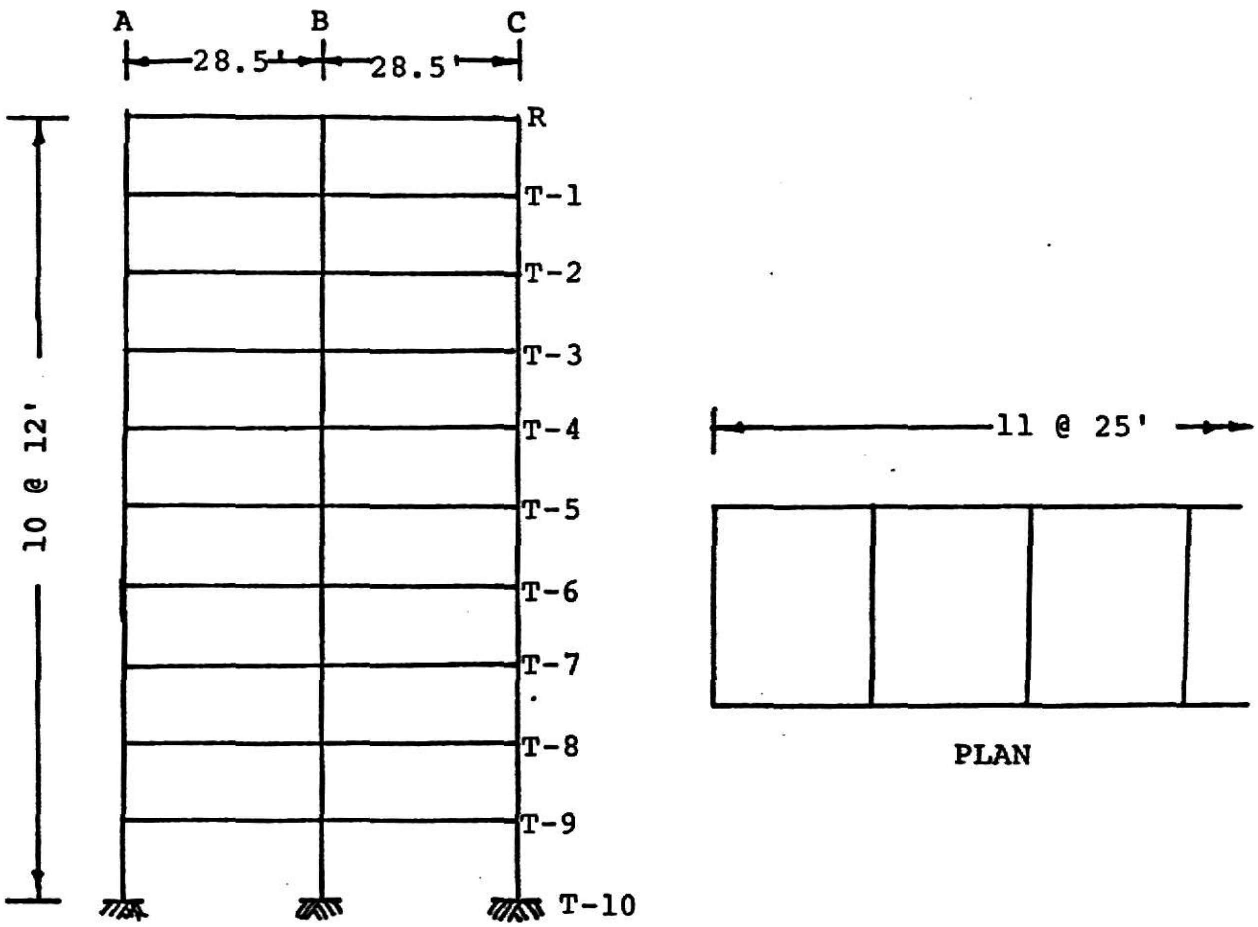

Figure 1. Elevation of Typical Bent

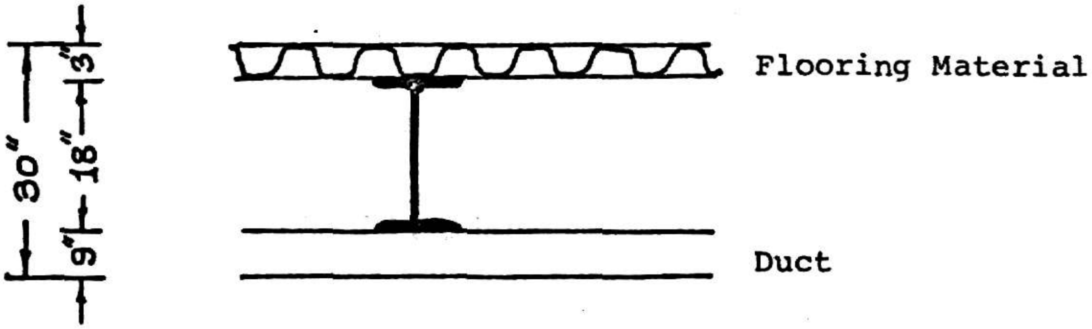

Figure 2. Cross Section of the Floor

between bents are assumed to be bolted, which justifies the weak axis effective length factors considered in the column design.

2.3 Design Specifications

The design calculations presented in this report conform to Part One of the AISC Specifications (3), and appropriate use has been made of the design aids in the "Manual of Steel Construction" (4) during the design process.

2.4 Loads

The loads for which a building must be designed may be classified as dead loads, vertical live loads and lateral live loads.

Dead loads include the weight of the permanent equipment and the weight of the fixed components of the building, such as floors, beams, girders, roofs, columns, walls, fixed partitions and the like.

The vertical live loads to be assumed in the design of buildings and other structures shall be the greatest loads that probably will be produced by the intended use of occupancy. Occupancy loads include personnel, furniture, machinery and stored materials. In most building designs, they are regarded as uniformly distributed loads.

The lateral live loads are those due to wind only.

It is assumed that the building is not located in an earthquake zone, so no precautions are taken for earthquake loading.

For the current design, the following loading data is assumed:

1.	Roof loads:	Dead load	60 lbs/s.f.
		Live load	60 lbs/s.f.
2.	Floor loads:	Dead load	60 lbs/s.f.
		Live load	100 lbs/s.f.
3.	Average weight of the wall over entire surface		60 lbs/s.f.
4.	Average column weight including fireproofing		300 lbs/ft.
5.	Wind load		25 lbs/s.f.

2.5 Live Load Reduction

Except in structures for storage and certain types of manufacturing and warehousing, the maximum loading on each floor is not likely to occur at any one time. In recognition of this fact, most building codes allow a reduction in live loading. According to recommendations of the American Standard Building Code (5):

A. No reduction shall be applied to the roof live loads.

B. For a live load of 100 pounds or less per square foot, the design live load on any member supporting 150 sq. ft. or more may be reduced at the rate of 0.08% per sq. ft. of area supported by the member, except that no reduction shall be made for areas to be occupied as places of public assembly. The reduction shall exceed neither R as determined by the following, nor 60%:

$$R = 100 \frac{D + L}{4.33\ L}$$

in which:

R = Reduction in %

D = Dead load per sq. ft. of the area supported by the member

L = Design live load per sq. ft. of the area supported by the member.

For live loads exceeding 100 pounds per sq. ft. no reduction shall be made except that the design live loads may be reduced by 20%. These criteria will be utilized in the design calculations presented in this report.

2.6 Distribution of Loads to the Frame (Figures 3 and 4)

The floor system is supported on floor beams which, in turn, are supported by the floor girders. The weight of the exterior wall is carried on spandrel beams framing to the outer faces of the exterior columns at each floor level.

The uniformly distributed wind load on the vertical surfaces of the building is assumed to be applied as horizontal concentrated loads at each floor level.

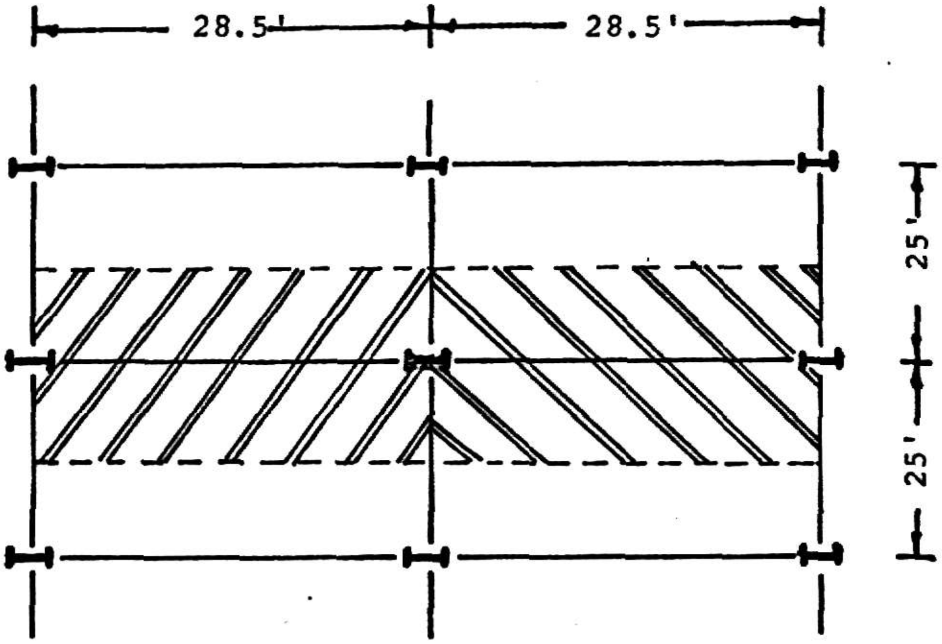

Figure 3. Distribution of Floor Loads to Girders

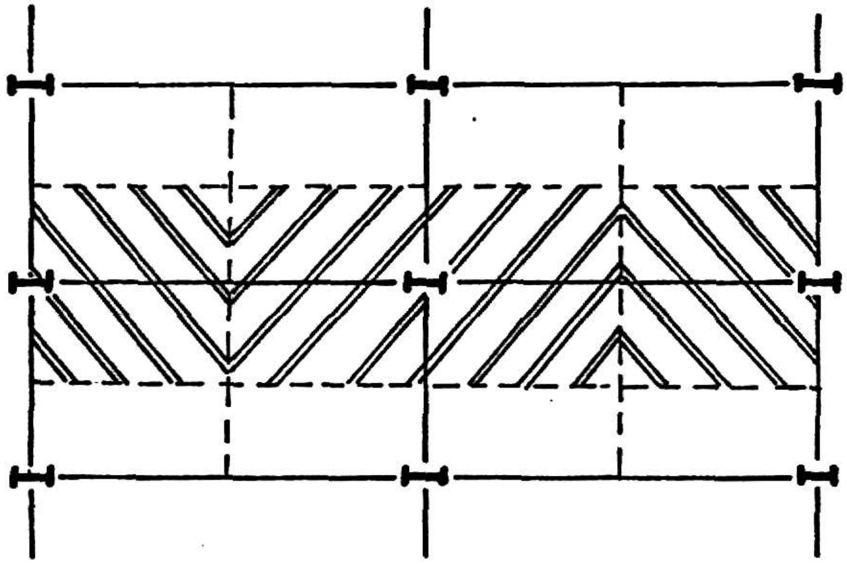

Figure 4. Distribution of Floor Loads to Columns

All the floor loads are assumed to be carried by the main girders as uniformly distributed loads, even though the floor beams framing into the girders would cause some concentrated loads. It should be noted that the share of the floor load carried by the spandrel beams is also assigned to the main girders. Thus the procedure of assigning all the floor loads to the main girders reduces some of the arithmetic in calculating girder moments. Figures 3 and 4 show the distribution of the floor loads to the girders and the columns, respectively. The weight of the column is assumed to be applied at the top of each column segment. No parapet wall is assumed at the roof.

3. METHODS OF PRELIMINARY ANALYSIS

3.1 Loading Conditions and Load Combinations

The structure will be analyzed for the following loading conditions:

1. Dead load + live load
2. Dead load + checkerboard live load
3. Wind load

The loading pattern shown in Figure 5 is termed the "checkerboard" pattern of live loading (6). This may produce the critical design conditions, particularly for maximum moments in the interior columns.

The design will be based on the stresses from the following combinations of the loading conditions:

1. Dead load + live load
2. Dead load + checkerboard live load
3. 0.75 (Dead load + live load + wind load)
4. 0.75 (Dead load + checkerboard live load + wind load)

3.2 Wind Load Analysis

As was mentioned earlier the wind loads are reduced to a series of concentrated loads applied at each floor level. The wind load analysis with unknown member sizes can be carried out quite rapidly using the portal or cantilever method. In this report the portal method (7) will be used.

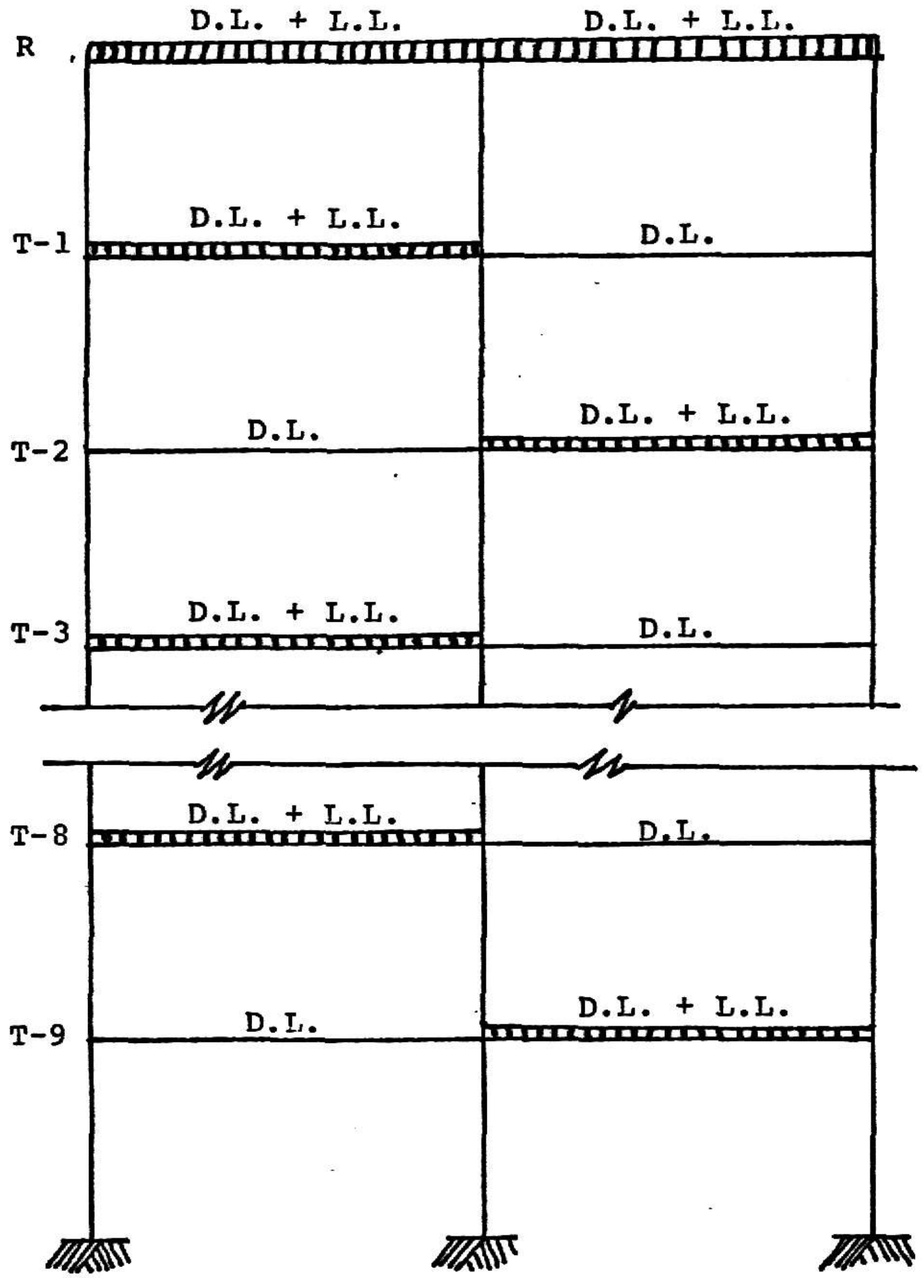

Figure 5. "Checkerboard" Pattern of Loading

The following assumptions are made in this method:

1. Points of inflections are located at the mid-point of each girder and column.

2. Each column in a story resists a percentage of the total horizontal shear on the story proportional to the width of aisle the column supports.

3.3 Gravity Load Analysis

With rigid framing, the moment distribution in the girders as well as in the columns is indeterminate. By examining a number of known limiting cases for the moments in indeterminate beams, an approximation can be made which will result in the selection of reasonable member sizes for the girders. Figure 6 shows the moment diagram for a member with ends perfectly fixed against rotation. In this case member sizes will be controlled by the negative moments at supports and inflection points will occur at 0.21 L from the ends. Figure 7 shows the moment diagram for the case of optimum redistribution of the moments as obtained from plastic analysis. This case will give the smallest possible member sizes. The inflection points will occur at 0.146 L from the ends.

The difference between the moment diagrams in Figure 6 and Figure 7 represents the effect of moment redistribution which could occur from rotation of ends due to either elastic bending of the adjacent columns or due to the development of plastic hinges in the ends of the girder. Further rotation

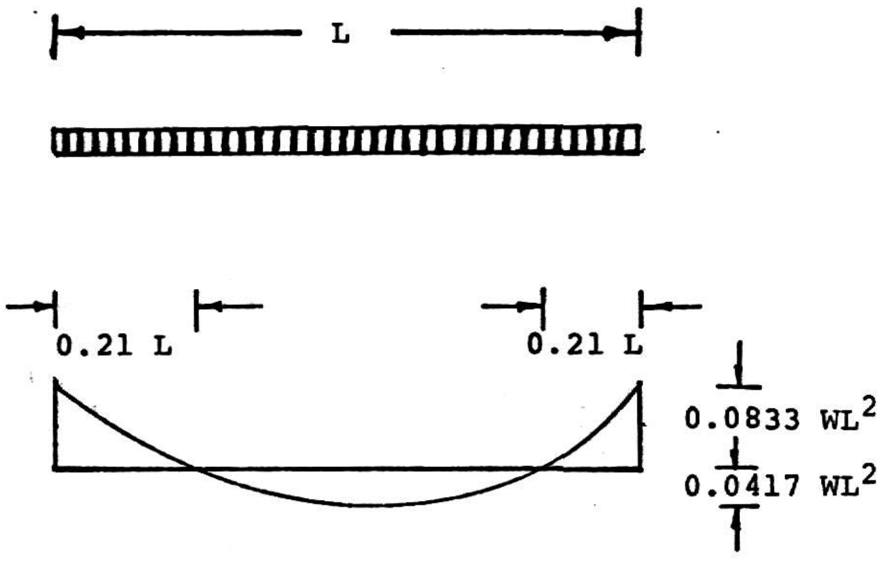

Figure 6. Fixed Ends

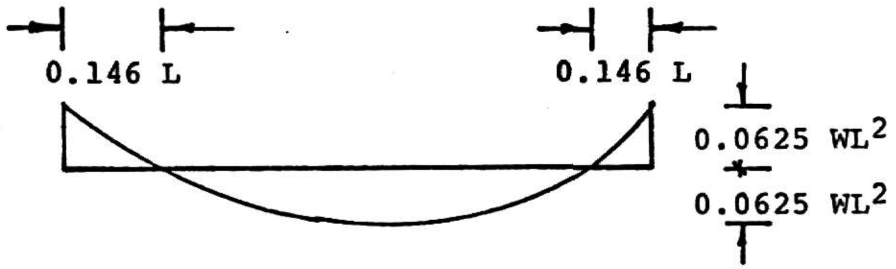

Figure 7. Complete Redistribution

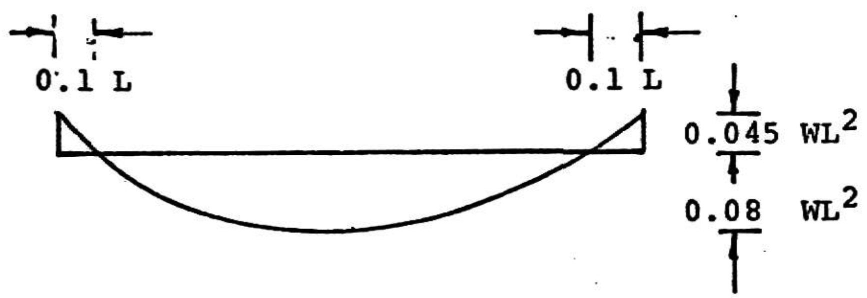

Figure 8. Design Approximation

of the end joints could result in a distribution of moments similar to that in Figure 8. Here the moment controlling member size is the maximum positive moment, and the inflection points occur at 0.1 L from the ends. It is found that the moment controlling member size has decreased from 0.0833 WL^2 to 0.625 WL^2 and increased back up to 0.08 WL^2 as the inflection points move. A member selected on the basis of the approximation given in Figure 8 should be strong enough to carry the loads, no matter where the inflection points occur (6). For this reason the girders will be designed based on the moment diagram given in Figure 8.

Bending moments in the columns can be induced by girder shear forces applied to the outer flanges of the columns at each floor level if the shear at the two faces of the column are unequal as shown in Figure 9a. The unbalanced moment will be equal to the difference in shears times one-half the column depth.

$$M = (V_1 - V_2)\ D_c/2$$

The proportion of column moments to be distributed above and below the joint depend on the stiffnesses of the two column segments. When selecting the member for column segment AG, Figure 9b, it is generally conservative to assume that half of the column moment acts on this segment, if both the columns are of equal length. Since column section BC is likely to be

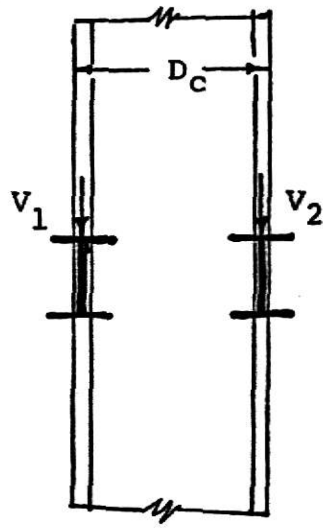

Figure 9a. Unbalanced Shears on Column

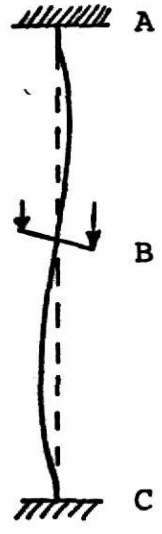

Figure 9b. Deflected Shape of Column

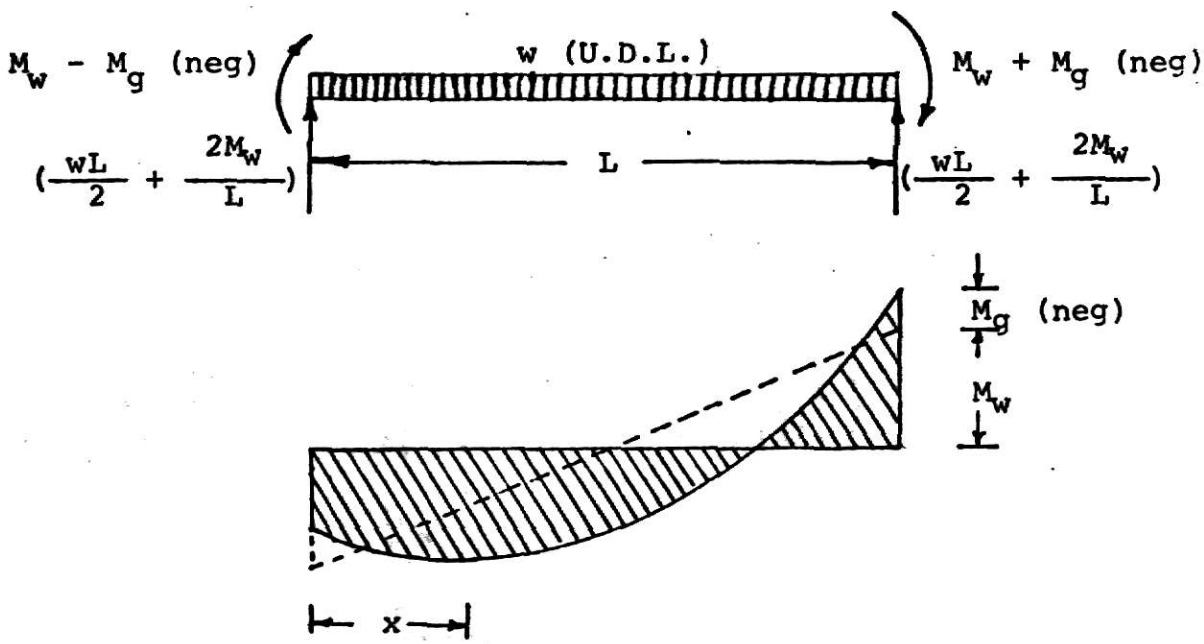

Figure 10. Moments on Girder Subjected to Wind plus Gravity Loads

heavier than AB, the actual moment in AB would be less than half the total. In this report it is assumed that the columns above and below a joint are of the same cross-section and length.

Due to the girder moments, additional moments beyond those caused by the unbalanced girder shears will be applied to the columns. These will be distributed one-half above and one-half below the joint in the same manner as described above for the girder shears.

3.4 Superposition of Wind Plus Gravity Loads

With combinations of wind and gravity loads, a 33.3% increase in allowable stress is permitted by the AISC Specification (3). This is usually provided in design by multiplying the loads by 3/4 and designing with the resulting moments, etc., for the basic allowable stresses. Negative moments are determined by adding the moments at the support due to the wind and gravity loadings. The maximum positive moment will vary in magnitude and location depending on the size of the wind moment.

The moments and reaction acting on the girder are shown in Figure 10. The positive moment, M, at any point, x, on the girder is equal to

$$M = [M_w - M_g(\text{neg})] + x(wL/2 - 2M_w/L) - wx^2/2$$

where M_w is the wind moment in the girder, M_g (neg) is the negative moment in the girder, w is the uniform load per unit length, and L is the length of the girder. The maximum positive moment will occur at

$$x = L/2 - 2M_w/wL$$

and its magnitude will be

$$M = wL^2/8 + 2M_w^2/wL^2 - M_g(\text{neg})$$

The design moment using normal allowable stresses will be three-quarters of this value.

4. PRELIMINARY ANALYSIS

4.1 Wind Load Analysis (wind from left)

Calculation of wind shear: (Figure 11)

Concentrated wind load = wind intensity x area x 0.75

= 25 psf x 12' x 25' x 0.75

= 5.6 kips

For roof story = 1/2 (5.6)

= 2.8 kips

Portion of the story shear taken by each column:

Column	A	B	C
Aisle wodth (ft)	14.25	28.5	14.25
% of total shear	25%	50%	25%

Analysis for T-1 story: (Figure 12)

Shear above floor T-1 = 2.8 kips

Shear below floor T-1 = 8.4 kips

Column moment = column shear x 1/2 story height

Moments for columns above floor T-1:

Column A	0.7 k x 6' = 4.2 k'
Column B	1.4 k x 6' = 8.4 k'
Column C	0.7 k x 6' = 4.2 k'

Moments for columns below floor T-1:

Column A	2.1 k x 6' = 12.6 k'
Column B	4.2 k x 6' = 25.2 k'
Column C	2.1 k x 6' = 12.6 k'

Girder moment ($\Sigma M = 0$ at joint):

$= 4.2 + 12.6 = 16.8$ k'

The frame is symmetrical so girders G-1 and G-2 will have the same moments.

$$\text{Girder shear} = \frac{\text{girder moment}}{1/2 \text{ span length}}$$

$= 16.8/14.25$

$= 1.2$ kips

Column axial load increments ($\Sigma V = 0$ at joint):

Column A = 1.5 k

Column B = 0.0 k

Column C = -1.5 k

Analysis for T-8 story: (Figure 13)

Shear above floor T-8 = 42.2 kips

Shear below floor T-8 = 47.8 kips

Moments for columns above floor T-8:

Column A	10.55 k x 6' =	63.3 k'
Column B	21.1 k x 6' =	126.6 k'
Column C	10.55 k x 6' =	63.3 k'

Moments for columns below floor T-8:

Column A	11.95 k x 6' =	71.7 k'
Column B	23.9 k x 6' =	143.4 k'
Column C	11.95 k x 6' =	71.7 k'

Girder moment ($\Sigma M = 0$ at joint):

$= 71.7 + 63.3 = 135.0$ k'

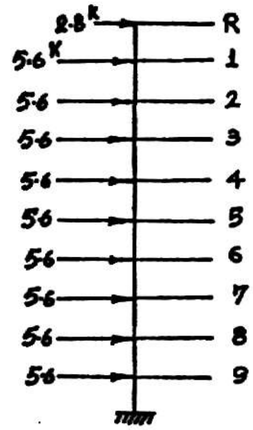

Figure 11. Wind Shear Acting on Each Story

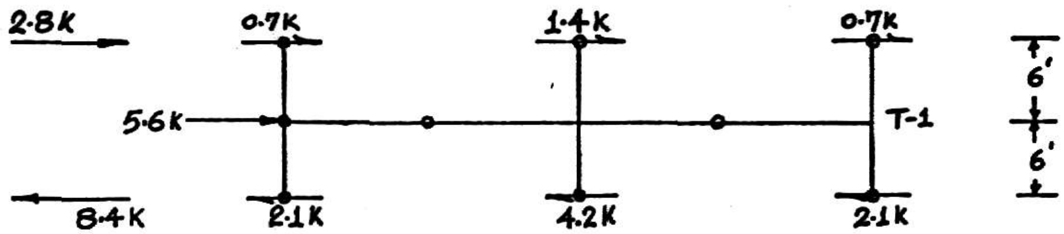

Figure 12. Distribution of Shears for T-1 Story

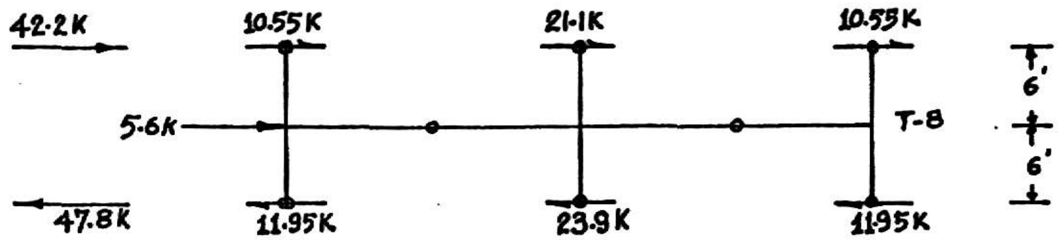

Figure 13. Distribution of Shears for T-8 Story

Girder shear = 135.0 k'/14.25' = 9.5 kips

Column axial load increments:

Column A = 9.5 k

Column B = 0.0 k

Column C = -9.5 k

The analysis for each story is carried out in the same manner and the results are shown in Table 1 and Figure 14.

Table 1. WIND LOAD ANALYSIS

Operation	Shear-kips	Column				
		A		B		C
<u>Roof Story</u>	2.8					
Column shear - k		0.7		1.4		0.7
Column moment - k'		4.2		8.4		4.2
Girder moment - k'			4.2		4.2	
Girder shear - k			0.3		0.3	
Column axial load - k		0.3		0.0		- 0.3
<u>T-1 Story</u>	8.4					
Column shear - k		2.1		4.2		2.1
Column moment - k'		12.6		25.2		12.6
Girder moment - k'			16.8		16.8	
Girder shear - k			1.2		1.2	
Column axial load - k		1.5		0.0		- 1.5

Table 1 (Continued)

Operation	Shear-kips	Column				
		A		B		C
<u>T-2 Story</u>	14.1					
Column shear - k		3.5		7.0		3.5
Column moment - k'		21.0		42.0		21.0
Girder moment - k'			33.6		33.6	
Girder shear - k			2.4		2.4	
Column axial load - k		3.9		0.0		- 3.9
<u>T-3 Story</u>	19.7					
Column shear - k		4.9		9.9		4.9
Column moment - k'		29.6		59.0		29.6
Girder moment - k'			50.6		50.6	
Girder shear - k			3.5		3.5	
Column axial load - k		7.4		0.0		- 7.4
<u>T-4 Story</u>	25.3					
Column shear - k		6.3		12.6		6.3
Column moment - k'		38.0		76.0		38.0
Girder moment - k'			67.6		67.6	
Girder shear - k			4.7		4.7	
Column axial load - k		12.1		0.0		-12.1

Table 1 (Continued)

Operation	Shear-kips	Column A		Column B		Column C
<u>T-5 Story</u>	31.0					
Column shear - k		7.7		15.5		7.7
Column moment - k'		46.4		93.0		46.4
Girder moment - k'			84.4		84.4	
Girder shear - k			5.9		5.9	
Column axial load - k		18.0		0.0		-18.0
<u>T-6 Story</u>	36.6					
Column shear - k		9.2		18.4		9.2
Column moment - k'		54.9		109.8		54.9
Girder moment - k'			101.3		101.3	
Girder shear - k			7.1		7.1	
Column axial load - k		25.1		0.0		-25.1
<u>T-7 Story</u>	42.2					
Column shear - k		10.6		21.1		10.6
Column moment - k'		63.4		126.7		63.4
Girder moment - k'			118.3		118.3	
Girder shear - k			8.3		8.3	
Column axial load - k		33.4		0.0		-33.4

Table 1 (Continued)

Operation	Shear-kips	Column				
		A		B		C
<u>T-8 Story</u>	47.9					
Column shear - k		12.0		23.9		12.0
Column moment - k'		72.0		143.4		72.0
Girder moment - k'			135.2		135.2	
Girder shear - k			9.5		9.5	
Column axial load - k		42.9		0.0		-42.9
<u>T-9 Story</u>	53.5					
Column shear - k		13.4		26.8		13.4
Column moment - k'		80.3		160.6		80.3
Girder moment - k'			152.1		152.1	
Girder shear - k			10.7		10.7	
Column axial load - k		53.6		0.0		-53.6

4.2 Girder Analysis

4.2.1 Gravity Load Analysis

Roof loads:

live load w = 60 psf x 25' = 1.5 k/ft.

dead load w = 60 psf x 25' = 1.5 k/ft.

Floor loads:

live load w = 100 psf x 25' = 2.5 k/ft.

dead load w = 60 psf x 25' = 1.5 k/ft.

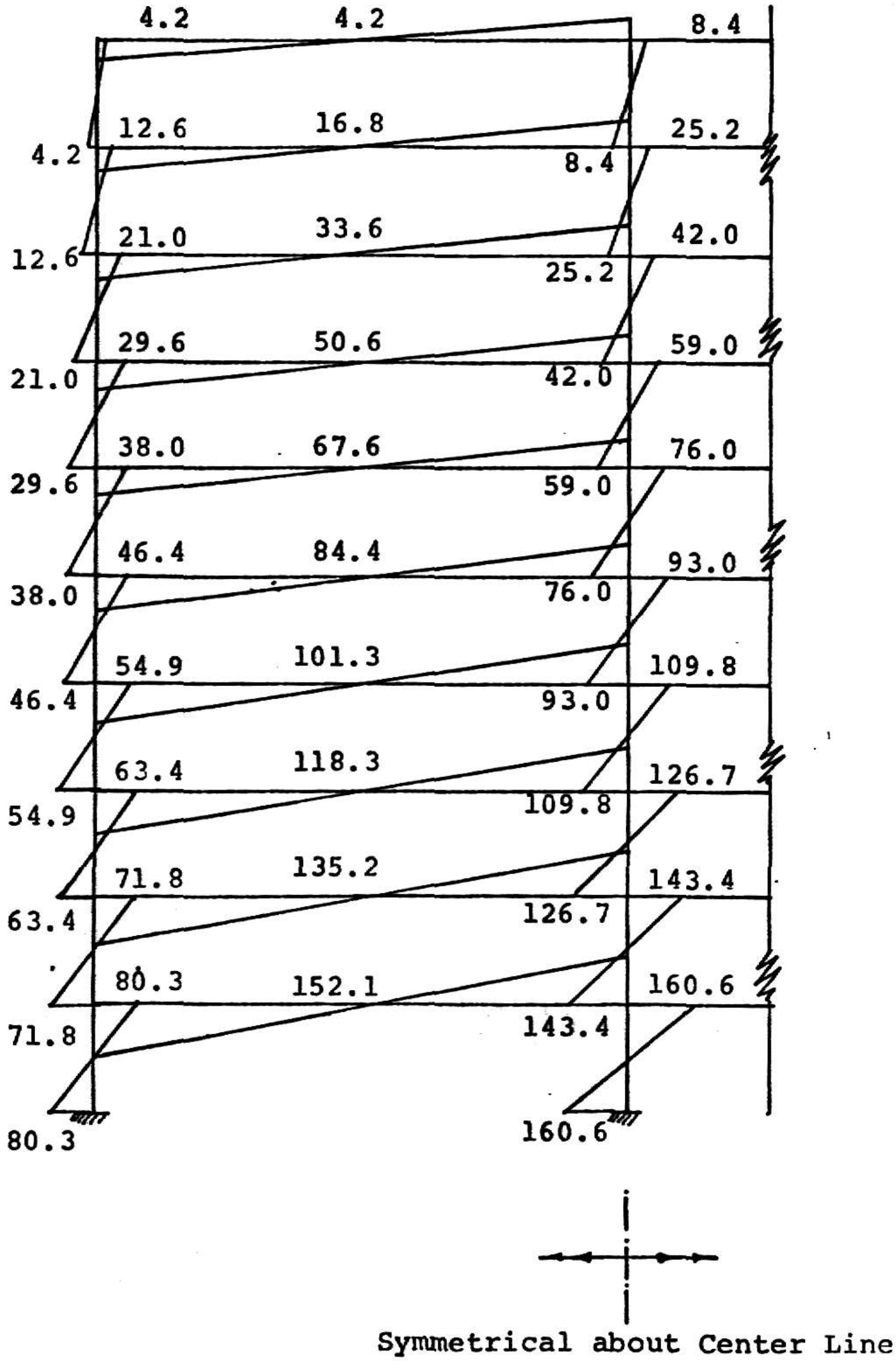

Figure 14. Moment (k.ft) Diagram Due to Wind Load
(Moments plotted on tension side)

Live load reduction: minimum from (1), (2) or (3)

(1) 60%

(2) 0.08 x supporting area

= 0.08 x 712.5

= 57.0%

(3)
$$R = 100 \times \frac{D + L}{4.33\ L}$$

= 100 x 160/433

= 37%

Girder No.	G-1	G-2
Floor area served (sq. ft.)	712.5	712.5
% reduction in live load	37.0%	37.0%

Column No.	A	B	C
Floor area served (sq. ft.)	356.25	712.5	356.25
% reduction at roof story	0.0 %	0.0%	0.0 %
T-1 story	28.5 %	37.0%	28.5 %
T-2 story	37.0 %	37.0%	37.0 %
All other stories	37.0 %	37.0%	37.0 %

Column dead load 300 lbs/ft. x 12' = 3.6 k/story

Wall load on exterior column

60 psf x 25' x 12' = 18 kips

Table 2. GRAVITY LOAD ANALYSIS

As the frame is symmetrical about the center line and the loadings are identical on both the girders, G-1 and G-2, the values of the moments and shears will be the same for both the girders.

Step No.	Operation		Units	Girder
	Roof Story			
1	Span - L		ft.	28.5
2	Live load - W_L		k/ft	1.5
3	Dead load - W_D		k/ft	1.5
4	Live load reaction - W_L.L.1/2		k	21.4
5	Dead load reaction - W_D.L.1/2		k	21.4
6	% live load used (1 - Red.)		%	100.0
7	% (W_L x L^2)		k.ft	1,220.0
8	W_D x L^2		k.ft	1,220.0
9	(7) + (8)		k.ft	2,440.0
	Gravity Moments			
10	Rigid framing	+ve = 0.08 x (9)	k.ft	195.0
11		-ve = 0.045 x (9)	k.ft	110.0
	T-1 Story			
12	Span		ft	28.5
13	Live load		k/ft	2.5
14	Dead load		k/ft	1.5
15	Live load reaction		k	35.6

Table 2 (Continued)

Step No.	Operation		Units	Girder
16	Dead load reaction		k	21.4
17	% live load used		%	63%
18	% (W_L x L^2)		k.ft	1,280.0
19	W_D x L^2		k.ft	1,220.0
20	(18) + (19)		k.ft	2,500.0
	Gravity Moments			
21	Rigid framing	+ ve = 0.08 x (20)	k.ft	200.0
22		- ve = 0.045 x (20)	k.ft	112.5

Note: The steps shown above for T-1 story are the same for all other floors because floor loads, % reduction and span remain the same.

Table 3. ANALYSIS FOR CHECKERBOARD LIVE LOAD

The final moments due to checkerboard loading will not govern the design values because they will be smaller in magnitude than those resulted from full gravity loads. However, the values of the moments may affect the design of the interior columns.

Step No.	Operation	Units	Girders	
			G-1	G-2
23	Live load	k/ft	2.5	0.0
24	Dead load	k/ft	1.5	1.5

Table 3 (Continued)

Step No.	Operation		Units	Girders	
				G-1	G-2
25	Live load reaction		k	35.6	0.0
26	Dead load reaction		k	21.4	21.4
27	% live load used		%	63.0	100.0
28	$(27) \times (23) \times (12)^2$		k.ft	1,280.0	0.0
29	$(24) \times (12)^2$		k.ft	1,220.0	1,220.0
30	(28) + (29)		k.ft	2,500.0	1,220.0
	Gravity Moments				
31	Rigid framing	+ ve = 0.08 x (30)	k.ft	200.0	97.6
32		- ve = 0.045 x (30)	k.ft	112.5	54.9

4.2.2 Wind Plus Gravity Load Analysis

Table 4. WIND PLUS GRAVITY LOAD ANALYSIS

Step No.	Operation	Units	Girder
	<u>T-9 Story</u>		
33	Wind moment - M_w	k.ft	152.1
34	0.09375 x (20)	k.ft	234.2
35	$\frac{2.67 \times (33)^2}{(20)}$	k.ft	24.8
36	(34) + (35)	k.ft	259.0
37	-0.75 x (22)	k.ft	-84.4

Table 4 (Continued)

Step No.	Operation	Units	Girder
38	Rigid framing + ve moment = (36) + (37)	k.ft	174.6
39	0.75 x (22)	k.ft	84.4
40	Rigid framing - ve moment = (33) + (39)	k.ft	236.5
41	Design moment + ve (38) or (21)	k.ft	200.0
42	Design moment - ve (40) or (22)	k.ft	236.5
	T-8 Story		
43	Wind moment	k.ft	135.2
44	0.09375 x (20)	k.ft	234.2
45	$\frac{2.67 \times (43)^2}{(20)}$	k.ft	19.6
46	(44) + (45)	k.ft	253.8
47	-0.75 x (22)	k.ft	-84.4
48	Rigid framing + ve moment = (46) + (47)	k.ft	169.4
49	0.75 x (22)	k.ft	-84.4
50	Rigid framing - ve moment = (49) + (43)	k.ft	219.6
51	Design moment + ve (48) or (21)	k.ft	200.0
52	Design moment - ve (50) or (22)	k.ft	219.6

4.2.3 Summary of Girder Design Forces

Table 5. SUMMARY OF GIRDER DESIGN FORCES

Level	Design moment - k.ft	Controlling load condition
Roof story	+ 195.0	Dead load + live load

Table 5 (Continued)

Level	Design moment - k.ft	Controlling load condition
T-1 to T-6	+ 200.0	Dead load + live load
T-7 story	- 202.7	0.75 (D.L. + L.L. + W.L.)
T-8 story	- 219.6	0.75 (D.L. + L.L. + W.L.)
T-9 story	- 236.5	0.75 (D.L. + L.L. + W.L.)

4.3 Column Analysis

4.3.1 Gravity Load Analysis for Exterior Columns

(A) Axial loads:

Roof loads:

Dead load, 60 psf x 14.25' x 25'	≈	21.4 k
Live load, 60 psf x 14.25' x 25'	≈	21.4 k
Column dead load	≈	3.6 k
Total		46.4 kips

Floor loads:

Dead load, 60 psf x 14.25' x 25'	=	21.4 k
Live load, 100 psf x 14.25' x 25' x (1 - 0.285)	=	25.5 k
Column dead load	=	3.6 k
Average wall load, 60 psf x 25' x 12'	=	18.0 k
Total	=	68.5 kips

Increment for each column below column 1-2:

Dead load = 21.4 k

Live load,

100 psf x 14.25' x 25' x 0.63 = 22.4 k

Column dead load = 3.6 k

Average wall load,

60 psf x 25' x 12' = 18.0 k

Total 65.4 kips/story

Axial load increment for two stories = 2 x 65.4

= 130.8 kips

(B) Moments:

Rigid framing moment at the floor level, M_g (neg)

= 112.5 k'

$1/2[M_g \text{ (neg)}]$ = 56.3 k'

Simple framing moment due to:

(1) floor girder live load

= red. factor x reaction x $D_c/4$

= 0.63 x 35.6' x $(D_c/4)'$ = 22.4 $D_c/4$ k'

(2) floor girder dead load

= reaction x $D_c/4$ = 21.4 $D_c/4$ k'

(3) spandrel dead load (negative moment)

= -18.0 $D_c/4$ k'

Total 25.8 $D_c/4$ k'

where D_c is the depth of the column = 14"

$$\text{Total moment} = \frac{25.8\ \text{k} \times (14/12)'}{4} = 7.5\ \text{kips.ft}$$

$$\text{Net moment for 14" column} = 56.3\ \text{k}' + 7.5\ \text{k}'$$

$$= 63.8\ \text{kips.ft.}$$

Calculation for axial loads on columns:

Column No.	Loads	Axial forces (kips)
1- 2	Roof loads	46.4
	Floor dead loads	43.0
	Floor live load	25.5
		114.9 kips
3- 4	Dead loads from column 1-2	89.4
	Floor live load	22.4
	Two-story increment	130.8
		242.6 kips
5- 6	Loads from column 3-4	242.6
	Two-story increment	130.8
		373.4 kips
7- 8	Loads from column 5-6	373.4
	Two-story increment	130.8
		504.2 kips
9-10	Loads from column 7-8	504.2
	Two-story increment	130.8
		635.0 kips

4.3.2 Summary of Design Forces on Exterior Columns

Table 6. DESIGN FORCES ON EXTERIOR COLUMNS DUE TO LOADING CONDITIONS 1 AND 3

Col. No.	P_g (k)	M_g (k')	P_w (k)	M_w (k')	$0.75P_g$ (k)	$0.75M_g$ (k')	$0.75P_g+P_w$ (k)	$0.75M_g+M_w$ (k')
1- 2	114.9	63.8	1.5	12.6	86.1	48.0	87.6	60.6
3- 4	242.6	63.8	7.4	29.6	182.0	48.0	189.4	77.6
5- 6	373.4	63.8	18.0	46.4	280.0	48.0	298.0	94.4
7- 8	504.2	63.8	33.4	63.4	378.0	48.0	411.4	111.4
9-10	635.0	63.8	53.6	80.3	476.0	48.0	529.6	128.3

4.3.3 Gravity Load Analysis for Interior Columns

Gravity load will cause equal and opposite moments which will cancel each other.

Roof loads:

Dead load, 60 psf x 28.5' x 25'		= 42.8 k
Live load, 60 psf x 28.5' x 25'		= 42.8 k
Column dead load		= 3.6 k
	Total	89.2 kips

Floor loads:

Dead load, 60 psf x 28.5' x 25'		= 42.8 k
Live load, 100 psf x 28.5' x 25' x 0.63		= 44.8 k
Column dead load		= 3.6 k
	Total	91.2 kips

Axial load increment for two stories = 2 (floor loads)

= 2 x 91.2

= 182.4 kips

4.3.4 Summary of Design Forces on Interior Columns

Table 7. DESIGN FORCES ON INTERIOR COLUMNS DUE TO LOADING CONDITIONS 1 AND 3

Col. No.	P_g (k)	M_g (k')	P_w (k)	M_w (k')	$0.75P_g$ (k)	$0.75P_g+P_w$ (k)	$0.75M_g+M_w$ (k')
1- 2	180.4	0.0	0.0	25.2	135.2	135.2	25.2
3- 4	362.6	0.0	0.0	59.0	272.0	272.0	59.0
5- 6	545.0	0.0	0.0	93.0	409.0	409.0	93.0
7- 8	727.4	0.0	0.0	126.7	545.0	545.0	126.7
9-10	909.8	0.0	0.0	160.6	681.0	681.0	160.6

4.3.5 Checkerboard Load Analysis for Interior Columns

(A) Axial loads:

Roof loads:

Dead load, 60 psf x 28.5' x 25'	=	42.8 k
Live load, 60 psf x 28.5' x 25'	=	42.8 k
Column dead load	=	3.6 k
	Total	89.2 kips

Floor loads:

Dead load, 60 psf x 28.5' x 25'	=	42.8 k
Live load, 100 psf x 14.25' x 25' x 0.715	=	25.5 k
Column dead load	=	3.6 k
	Total	71.9 kips

Axial load increment for two stories

= 2 (dead load + live load + column dead load)

= 2 (85.6 + 44.8 + 7.2)

= 137.6 kips

(B) Moments:

Net rigid framing moment at floor level M_g (neg)

= 112.5 - 54.9

= 57.6 kips.ft

$1/2\ M_g$ = 28.8 kips.ft

Simple framing moment due to

(1) floor girder live load

= red. factor x reaction x $D_c \cdot 1/4$

= 0.63 x 35.6 k x 1/4 D_c

= 22.4 x 1/4 $D_c k'$

For 14" column, simple framing moment

= 22.4 x 1/4 x (12/14)'

= 6.53 kips.ft

Net girder end moment

= 28.8 + 6.53

= 35.33 kips.ft

4.3.6 Summary of Design Forces on Interior Columns

Table 8. DESIGN FORCES ON INTERIOR COLUMNS
DUE TO LOADING CONDITIONS 2 AND 4

Col. No.	P_g (k)	M_g (k')	P_w (k)	M_w (k')	$0.75P_g$ (k)	$0.75M_g$ (k')	$0.75P_g+P_w$ (k')	$0.75M_g+M_w$ (k')
1- 2	161.1	35.3	0.0	25.2	121.0	26.5	121.0	51.7
3- 4	298.7	35.3	0.0	59.0	224.0	26.5	224.0	85.5
5- 6	436.3	35.3	0.0	93.0	327.0	26.5	327.0	119.5
7- 8	573.9	35.3	0.0	126.7	430.0	26.5	430.0	153.2
9-10	711.5	35.3	0.0	160.6	534.0	26.5	534.0	187.1

5. DESIGN

5.1 Design of Girders

The maximum girder depth is 18". The selection of flexural members is done on the basis of allowable bending stresses in accordance with Section 1.5.1.4 of the AISC Specifications (3). Only compact, W-shaped sections will be used; therefore, the allowable bending stress, $F_b = 0.66\ F_y$, where F_y (yield stress) is equal to 36 ksi. The compression flanges of the girders are assumed to be continuously supported by the floor slab, which provides an unbraced length of the compression flange equal to zero. The shear stress is checked according to Section 1.5.1.2 of the AISC Specifications (3). A 3% overstress is assumed to be acceptable.

Calculations of girder shears:

Shears due to gravity loads = dead load reaction
+ live load reaction x reduction factor
= 21.4 k + 35.6 k x 0.63
= 43.8 kips

Shears due to gravity plus wind loads (this will be maximum for T-9 story as the wind moment is maximum)
= 0.75 x gravity shears + wind shear
= 0.75 x 43.8 k + 10.7 k
= 43.5 kips

Shears due to gravity loads control.

Table 9. DESIGN OF GIRDERS

Step No.	Level	Units	Roof to T-8	T-9
1	Design moment - M	k.ft	219.6	236.5
2	S req. = M x 12/24	in^3	110.0	118.0
3	Trial section		W 18 x 60	W 18 x 64
4	Web area*	in^2	7.0	6.6
5	S provided	in^3	108.0**	118.0
	Compact section criteria			
6	F_y'	ksi	> 36 ksi	> 36 ksi
7	F_y''	ksi	> 36 ksi	> 36 ksi
	Check for shear			
8	Shear	kips	43.8	43.8
9	Shear stress	ksi	6.25	6.64
10	Allowable shear stress = 0.4 F_y	ksi	14.5	14.5
11	O.K. or N.G.		O.K.	O.K.

*Web area = (web depth - 2 x thickness of flange) x web thickness

**% over-stress = (110 - 108) x 100/110 = 1.8% less than 3% so O.K.

5.2 Design of Columns

In the column design examples shown in Tables 10 and 11, Steps 1 to 3 show the loading conditions and design forces. Steps 4 through 9 show the trial section and its section properties. Steps 10 through 14 show the calculations for the effective length factor K. The effective length, KL, is the actual unbraced length, in feet, multiplied by the factor K, which depends upon the restraint at the ends of the unbraced length and the means available to resist lateral movements. Table C 1.8.1 in the Commentary on the AISC Specifications (3) is used as a guide in the selection of the K factors. The intermediate floor beam connections between bents are assumed to be bolted so the effective length factor for weak axis buckling is 1.

Steps 15 through 32 are computed in accordance with the Section 1.6 of the AISC Specifications (3). Values for the allowable bending stress, F_b, are computed in accordance with Section 1.5.1.4 of the AISC Specifications (3). Steps 21 and 22 show the computed stresses respectively. Formulas (1.6 - 1a) and (1.6 - 1b) of the AISC Specifications (3) are checked in Steps 23 through 32.

A 3% overstress is assumed to be acceptable.

Typical designs of exterior columns 5-6 and 9-10 and interior column 7-8 are shown in Table 10 and Table 11 respectively.

5.2.1 Typical Designs of Exterior Columns

Table 10. DESIGN OF COLUMNS

Step No.	Operation	Units	Column 5-6		Column 9-10	
1	Load case		1	3	3	1
2	P	k	373.4	298.0	529.6	635.0
3	M	k.ft	63.8	94.4	128.3	63.8
4	Section		W 14 x 95		W 14 x 142	
5	A	in^2	27.9		41.8	
6	I_{xx}	in^4	1,060.0		1,670.0	
7	S_x	in^3	151.0		227.0	
8	r_x	in^2	6.17		6.32	
9	r_y	in^2	3.71		3.97	
10	I_c/L_c	in^4/ft	177.0		278.0	
11	I_g/L_g	in^4/ft	34.6		37.0	
12	G_T		5.1		7.52	
13	G_B		5.1		1.0	
14	K		2.24		1.82	
15	Kl_x/r_x		52.1		41.5	
16	l_x/r_y		38.8		36.3	
17	F_a	ksi	18.16		19.07	
18	F_e'	ksi	55.0		86.74	
19	L_c	ft	15.4		16.4	
20	F_b	ksi	22.0		24.0	
21	f_a	ksi	13.4	10.7	12.65	15.2

Table 10 (Continued)

Step No.	Operation	Units	Column 5-6		Column 9-10	
22	f_b	ksi	5.05	7.5	6.77	3.38
23	f_a/F_e'		0.244	0.194	0.146	0.175
24	$\frac{C_m \times f_b}{(1 - f_a/F_e')F_b}$		0.258	0.36	0.281	0.145
25	f_a/F_a		0.74	0.588	0.664	0.797
26	(24) + (25)		0.998	0.948	0.945	0.942
27	O.K. or N.G.		O.K.	O.K.	O.K.	O.K.
28	$f_a/0.6\ F_y$		0.609	0.486	0.575	0.691
29	f_b/F_b		0.230	0.341	0.282	0.141
30	(28) + (29)		0.839	0.827	0.857	0.832
31	O.K. or N.G.		O.K.	O.K.	O.K.	O.K.

5.2.2 Typical Design of Interior Columns

Table 11. DESIGN OF INTERIOR COLUMNS

Step No.	Operation	Units	Column 7-8			
1	Load case		3	4	2	1
2	P	K	545.0	430.0	573.9	727.4
3	M	k.ft	126.7	153.2	35.3	0.0
4	Section		W 14 x 150			
5	A	in^2	44.1			
6	I_{xx}	in^4	1,790.0			

Table 11 (Continued)

Step No.	Operation	Units	Column 7-8			
7	S_x	in^3	240.0			
8	r_x	in	6.37			
9	r_y	in	3.99			
10	I_c/L_c	in^4/ft	298.0			
11	I_g/L_g	in^4/ft	69.2			
12	$G_T = G_B$		4.31			
13	K		2.1			
14	Kl_x/r_x		47.5			
15	l_y/r_y		36.1			
16	F_a	ksi	18.57			
17	F_e'	ksi	66.21			
18	F_b	ksi	24.0			
19	f_a	ksi	12.35	9.75	13.0	16.5
20	f_b	ksi	6.34	7.67	1.77	0.0
21	f_a/F_e'		0.187	0.147	0.196	
22	$\dfrac{C_m \times f_b}{(1 - f_a/F_e')F_b}$		0.276	0.319	0.078	
23	f_a/F_a		0.666	0.526	0.7	0.89
24	(22) + (23)		0.942	0.845	0.778	0.89
25	O.K. or N.G.		O.K.	O.K.	O.K.	O.K.
26	f_b/F_b		0.264	0.32	0.074	
27	$f_a/0.6\ F_y$		0.561	0.443	0.591	

Table 11 (Continued)

Step No.	Operation	Units	Column 7-8		
28	(26) + (27)		0.825	0.763	0.665
29	O.K. or N.G.		O.K.	O.K.	O.K.

5.3 Summary of Results of Preliminary Design

The member sizes selected in the preliminary design process are summarized in Figure 15.

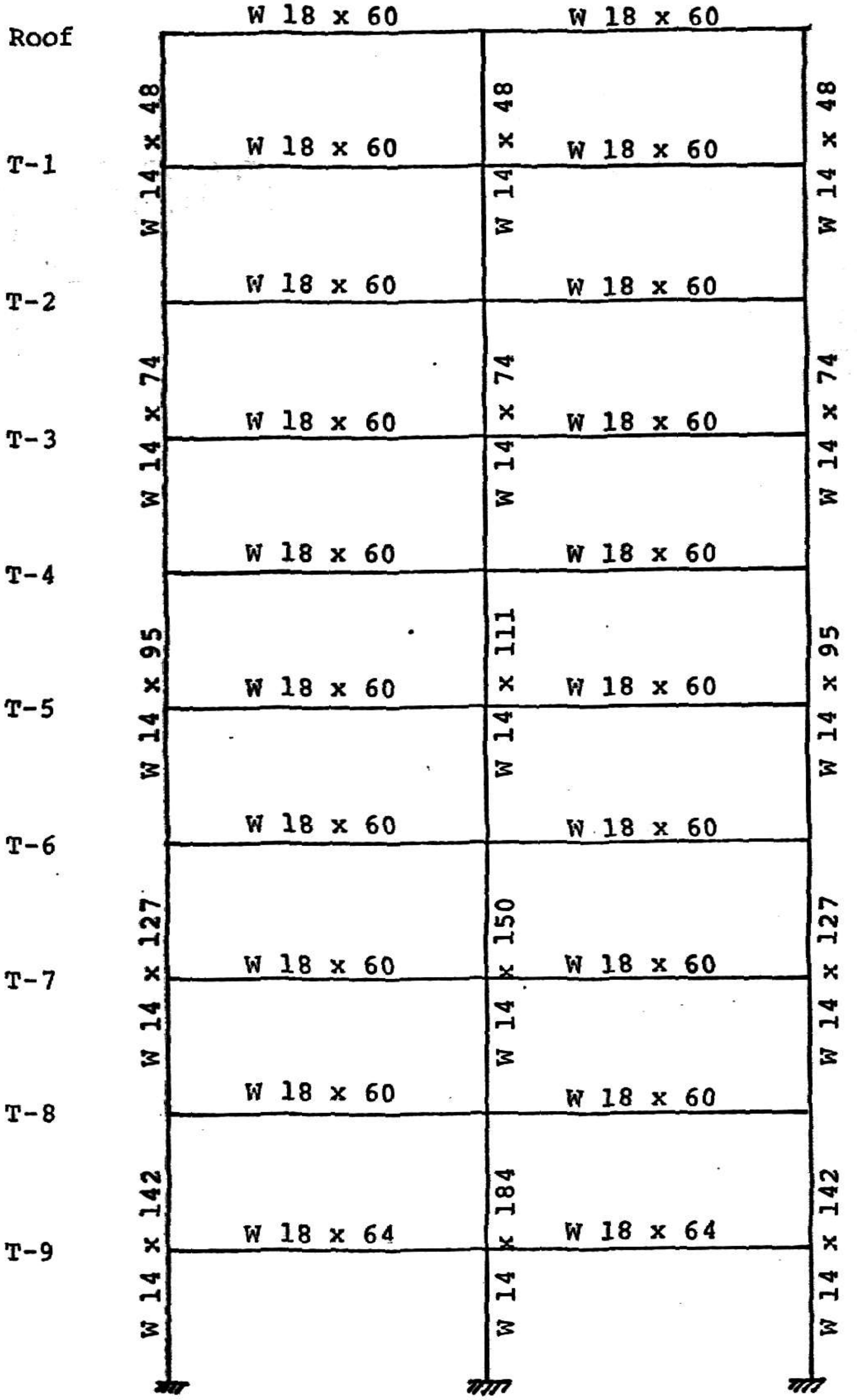

Figure 15. Member Sizes from Preliminary Design

6. COMPUTER ANALYSIS

6.1 Introduction

Once the members have been selected based on the preliminary, approximate analysis, a more accurate analysis can be performed. In this report a stiffness analysis has been carried out through the use of the "STRUDL" program (8).

The STRUDL language has a free format input, that is to say, no consideration has to be given to punching a given word or figure starting at a particular column. However, there are some general rules which must be followed (8).

It was assumed in formulating the STRUDL program that all joints are rigid, so that all the members meeting at a joint have the same rotation and displacement at that point. If this is not the actual condition (if there is a hinge, for example) releases have to be introduced.

The geometry of the structure is specified by means of the coordinates of its joints with respect to a global set of cartesian coordinate axes. For a plane frame, only two axes, x and y, are used according to the standard STRUDL convention. Once the geometry of the structure is defined, each joint is given a number or name. The numbering scheme of the joints affects the program execution time. Once the joints are defined, the position of each member is given by indicating the joints that occur at the member ends. Figure 16 shows the frame geometry, joints and member numbering.

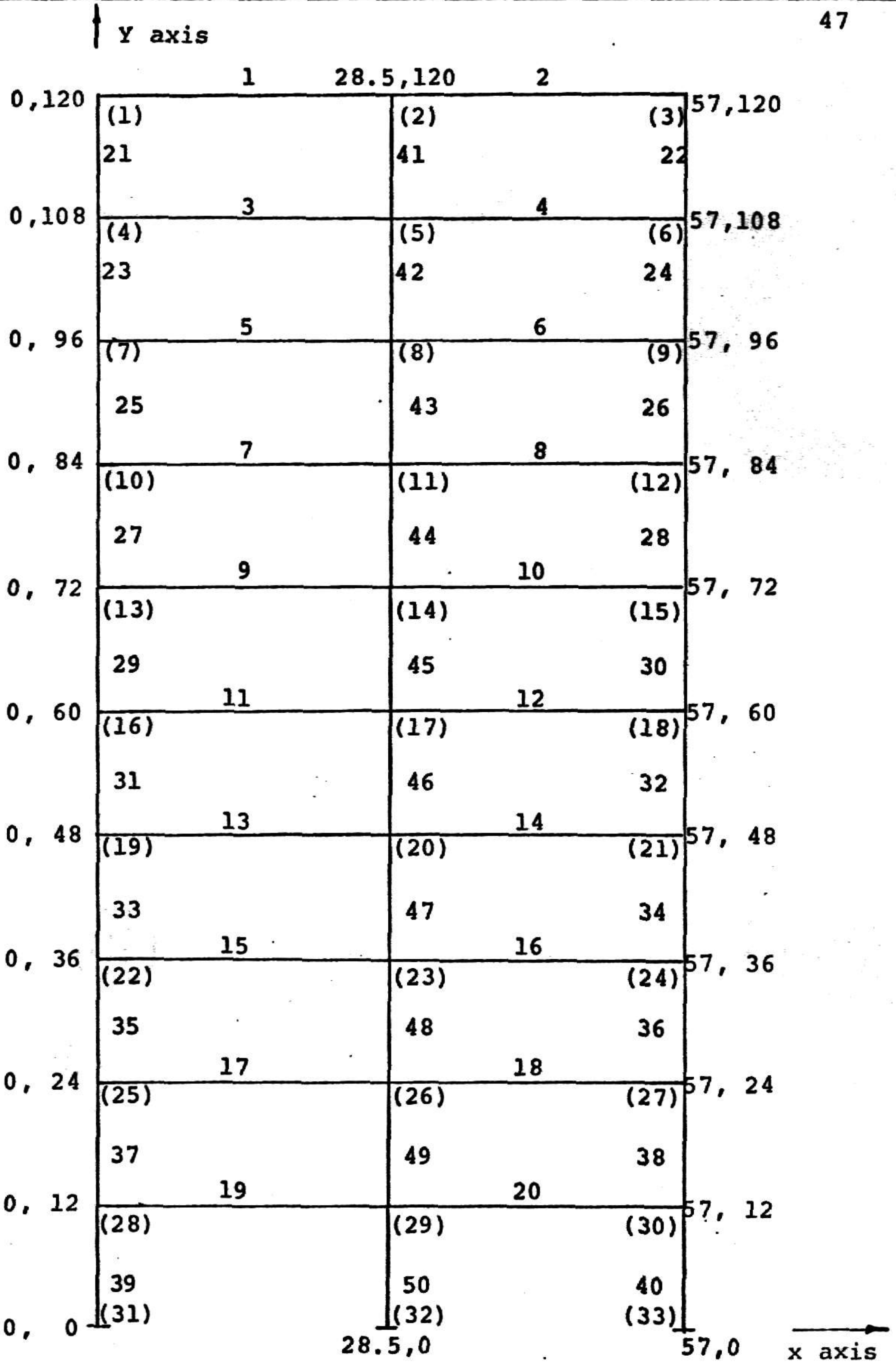

Figure 16. Joint Coordinates, Joint Numbering (x) and Member Numbering - x for STRUDL Analysis

6.2 Input Loading Data

The structure is analyzed for the following loading conditions:

1. Dead load
2. Live load
3. Checkerboard live load
4. Wind load

The design forces are based on the following combinations of the loading conditions:

1. Dead load + live load
2. Dead load + checkerboard live load
3. 0.75 (Dead load + live load + wind load)
4. 0.75 (Dead load + checkerboard live load + wind load)
5. 0.75 (Wind load)

Calculations for different loading conditions:

Dead loads:

Member load:

60 psf x 25' = 1.5 k/ft

Joint loads:

Average wall load, 60 psf x 25' x 12' = 18.0 k

Column dead load = 3.6 k

Moment due to wall load, 18 k x 7/12' = 10.5 k.ft

Figure 17a shows the application of dead loads to the various members and joints.

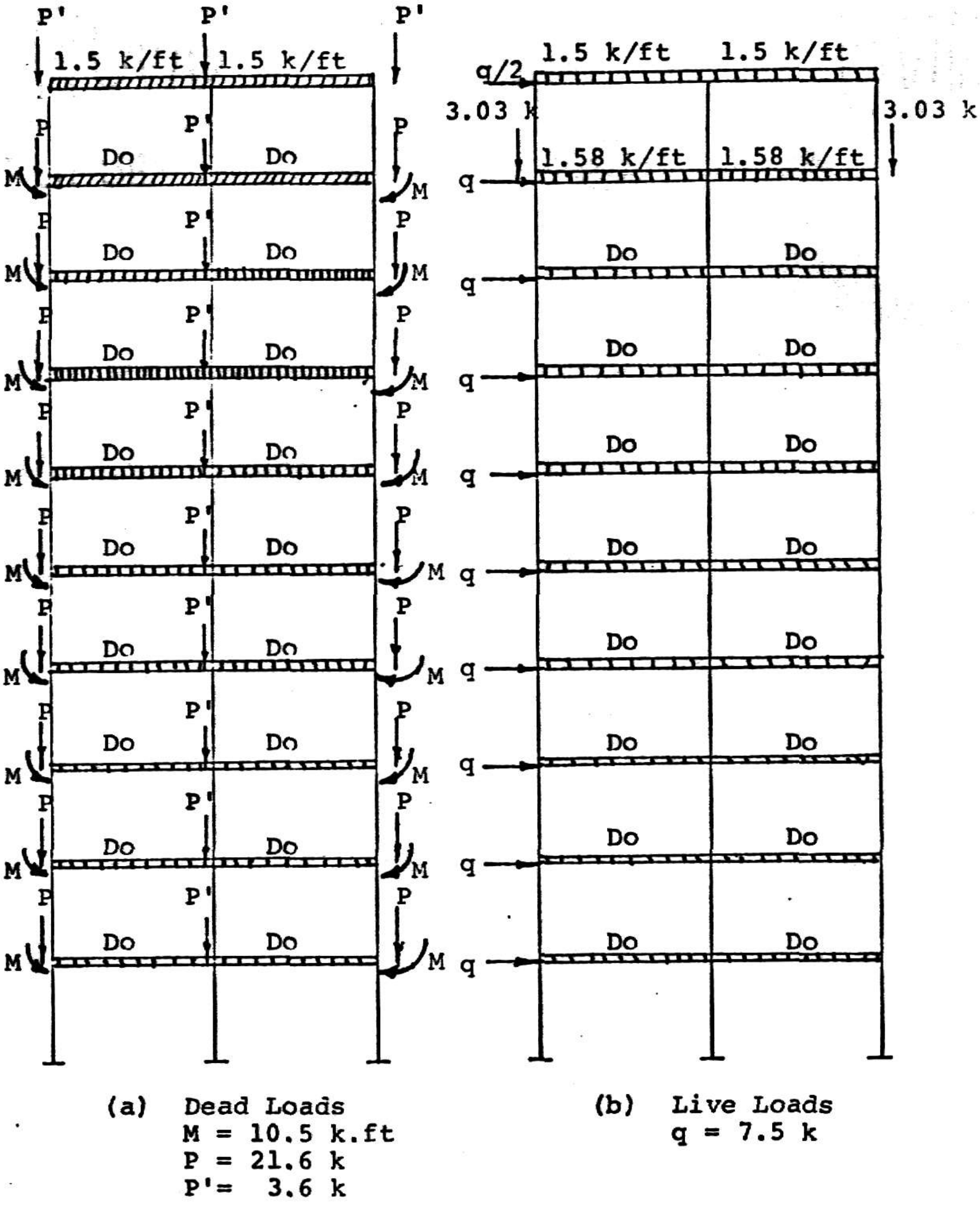

Figure 17. Loading Conditions for Computer Analysis

Live loads:

Member loads:

Roof level, 60 psf x 25' = 1.5 k/ft

Floor level, 100 psf x 25' x 0.63 = 1.58 k/ft

Joint load:

Exterior column at T-1 level (due to difference in reduction factors)

= (0.715 - 0.63) x 2.5 k/ft x 14.25 ft = 3.03 k

Wind load:

Joint loads:

Roof level (25 psf x 25' x 12') 1/2 = 3.75 k

Floor level (25 psf x 25' x 12') = 7.5 k

Figure 17b shows the application of live and wind loads to the different members and joints.

6.3 Results of the Computer Analysis

6.3.1 Girder Design Forces

The maximum magnitude of a force is considered as the design force, and it is found that negative moment, that is, the moment at a girder end, is the controlling design force.

Table 12. SUMMARY OF GIRDERS DESIGN MOMENTS (k.ft)

Level	Loading Condition 1	2	3	4
R	229	120	177	95
1	209	194	174	162

Table 12 (Continued)

Level	Loading Condition 1	2	3	4
2	210	191	191	172
3	209	191	206	192
4	211	196	225	211
5	212	198	242	232
6	212	200	259	247
7	212	201	272	263
8	214	203	280	267
9	217	208	268	261

6.3.2 Summary of Column Design Forces

Table 13. SUMMARY OF COLUMN DESIGN FORCES

Col. No.	Loading Condition 1		2		3		4	
	P-k	M-k'	P-k	M-k'	P-k	M-k'	P-k	M-k'
Exterior Columns								
1- 2	110	79	91	76	85	77	67	67
3- 4	240	89	198	88	188	104	141	85
5- 6	370	90	305	90	296	123	227	101
7- 8	500	92	412	92	408	138	323	129
9-10	629	84	519	93	521	192	419	172

Table 13 (Continued)

Col. No.	Loading Condition 1		2		3		4	
	P-k	M-k'	P-k	M-k'	P-k	M-k'	P-k	M-k'
				Interior Columns				
1- 2	189	00	121	23	142	26	90	43
3- 4	375	00	262	36	281	61	196	88
5- 6	560	00	402	42	420	98	301	129
7- 8	745	00	542	44	559	126	406	159
9-10	931	00	682	62	699	230	512	251

7. COMPARISON OF RESULTS OF PRELIMINARY AND STRUDL ANALYSIS

7.1 Comparison of Wind Forces

Moments due to wind load are compared in Table 14. A comparison is made between the maximum moment acting on a member based on the approximate analysis and based on the computer analysis. The percent difference has been calculated with respect to the computer analysis results. For the girder moments, the difference is within 10% for the top seven stories excluding the roof level. For the bottom three stories, the moments from the approximate analysis are larger than the moments from the computer analysis.

The percent differences for the exterior column moments are more than 10% for all levels except level 7-8, but the percent differences between the moments in the interior columns are within 10% except at the first floor level.

Table 15 shows a comparison of the axial forces acting on the columns due to wind loads. The difference is within 10% for all levels.

Table 14. COMPARISON OF MOMENTS

Level	Girder Moments			Exterior Columns			Interior Columns		
	C. A.	A. A.	% Diff.*	C. A.	A. A.	% Diff.	C. A.	A. A.	% Diff.
R	6.2	4.2	-32.2						
1	18.8	16.8	-10.6	17.6	12.7	-28.0	26	25.2	- 3.1
2	36.7	33.7	- 7.9						
3	53.0	50.6	- 4.55	37.1	29.6	-20.2	60.5	59.0	- 2.5
4	70.5	67.6	- 4.1						
5	86.4	84.4	- 2.3	55.1	46.4	-15.8	98	93	- 5.1
6	103.3	101.3	- 1.94						
7	116.3	118.3	+ 1.74	68.7	63.4	- 7.7	126	127	- 0.8
8	122.0	135.2	+11.9						
9	108.6	152.1	+40.0	163	80.3	-50.8	230	161	-30.0

* % Difference is calculated with respect to computer analysis results.

Table 15. COMPARISON OF COLUMN AXIAL LOAD DUE TO WIND FORCES

Col. No.	C. A.	A. A.	% Difference with respect to computer analysis
1- 2	1.6	1.5	-6.6
3- 4	7.7	7.4	-3.9
5- 6	18.5	18.0	-2.2
7- 8	33.7	33.4	-0.9
9-10	49.6	53.6	+8.1

The intermediate columns do not have any axial load due to wind load analysis.

7.2 Comparison of Girder Moments - Combined Loadings

Table 16 presents a comparison of the maximum moments acting on the girders for three loading combinations. The percent differences for loading conditions 1 and 2 are within 10% except at the roof level for loading condition 1.

In the last column of Table 16, the percent differences are shown between the controlling girder design moments. The differences are within 10% when the design moments are governed by gravity loads. However, when the combination of wind and gravity loads controls the design, the differences are larger than 10%.

Table 16. COMPARISON OF GIRDER MOMENTS

Level	Loading Condition 1			Loading Condition 2			Loading Condition 3		% Diff. in design moments
	C. A.	A. A.	% Diff.*	C. A.	A. A.	% Diff.	C. A.	A. A.	
R	229	195	-14.8	120	195	+62.5	177	201	-14.8
1	209	201	- 3.83	194	201	+ 3.6	174	201	- 3.83
2	210	201	- 4.3	191	201	+ 5.25	191	201	- 4.3
3	209	201	- 3.83	191	201	+ 5.25	206	201	- 3.83
4	211	201	- 4.75	196	201	+ 2.55	225	201	-10.6
5	212	201	- 5.2	198	201	+ 1.52	242	201	-17.0
6	212	201	- 5.2	200	201	+ 0.05	259	201	-22.4
7	212	201	- 5.2	201	201	0.0	272	201	-26.0
8	214	201	- 6.1	203	201	- 1.0	280	220	-21.7
9	217	201	- 8.0	208	201	- 3.35	268	236	-11.9

* % difference is calculated with respect to computer analysis results

7.3 Comparison of Column Forces - Combined Loadings

A comparison of column axial loads and moments is shown in Tables 17 and 18. Table 17 shows the comparison for the exterior columns where the percent differences for the axial loads are within 5%. The differences between the moments are quite large, however, ranging from 19 to 33.5%.

Table 18 presents the comparison for the interior columns. The axial loads and moments for loading conditions 1 and 3 are within 10%. For loading conditions 2 and 4, the axial loads from the approximate analysis are at many levels substantially greater than those obtained from the computer analysis. The column moments for loading condition 4 are within a reasonable percent difference except for column 9-10. For loading condition 2 the percent difference is more than 10% for all levels except level 3-4.

Table 17. COMPARISON OF FORCES FOR EXTERIOR COLUMNS

Table 17a. Loading Condition - 1

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	110	79	114.9	63.8	-4.45	-19.0
3- 4	240	89	242.6	63.8	-1.08	-28.0
5- 6	370	90	372.4	63.8	-0.9	-29.0
7- 8	500	92	504.2	63.8	-0.8	-30.5
9-10	629	84	635	63.8	-0.95	-24.0

Table 17b. Loading Condition - 3

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	85	77	87.6	60.6	4.3	-21.0
3- 4	188	104	189.4	77.6	0.75	-25.0
5- 6	296	123	298	94.4	0.68	-26.0
7- 8	408	138	411.4	111.4	0.83	-19.6
9-10	521	192	529.6	128.3	1.6	-33.5

Table 18. COMPARISON OF FORCES FOR INTERIOR COLUMNS

Table 18a. Loading Condition - 1

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	189	0.0	180.4	0.0	- 4.55	0.0
3- 4	375	0.0	362.6	0.0	- 3.2	0.0
5- 6	560	0.0	545.0	0.0	- 2.7	0.0
7- 8	745	0.0	724.4	0.0	- 2.7	0.0
9-10	931	0.0	909.8	0.0	- 2.3	0.0

Table 18b. Loading Condition - 2

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	121	23	161.1	35.3	+33	-52.0
3- 4	262	36	298.7	35.3	+14.2	- 1.94
5- 6	402	42	436.3	35.3	+ 8.5	-16.6
7- 8	542	44	573.9	35.3	+ 5.9	-20.5
9-10	682	62	711.5	35.3	+ 4.4	-45.0

Table 18c. Loading Condition - 3

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	142	26	135.2	25.2	- 4.9	- 3.2
3- 4	281	61	272	59.0	- 3.2	- 4.9
5- 6	420	98	409	93.0	- 2.6	- 5.1
7- 8	559	126	545	126.7	- 2.5	- 0.56
9-10	699	230	681	160.6	- 2.6	-30.0

Table 18d. Loading Condition - 4

Col. No.	C. A.		A. A.		% Difference	
	P - k	M - k'	P - k	M - k'	P	M
1- 2	90	43	121	51.7	+34.5	+18.6
3- 4	196	88	224	85.5	+14.3	- 2.85
5- 6	301	129	327	119.5	+ 8.7	- 7.0
7- 8	406	159	430	153.2	+ 5.9	- 3.9
9-10	512	251	534	187.1	+ 4.3	-25.5

8. SUMMARY AND CONCLUSIONS

For the wind load analysis, it is found that the moments in the girders obtained from the approximate and computer analysis are reasonably close. The moments on the interior columns are also within reasonable limits of variation, but the moments on the exterior columns are beyond an acceptable limit of variation. It is also found that the first assumption of the portal method, that inflection points are located at the mid-lengths of the members, is not satisfied in the results of the computer analysis. This is one of the reasons for the differences in the results. The axial loads in the columns due to wind load compared quite well.

The maximum moments in the girders due to gravity loads compared reasonably well, but with the combination of wind and gravity loads, the results did not compare within reasonable limits. This means that a substantial change in the grider sizes would be required. For the column forces due to combined loading, the magnitudes of the axial loads are within reasonable limits of variation as are the moments on the interior columns. However, the exterior column sizes in some cases would be changes significantly in a redesign.

9. SUGGESTIONS FOR FURTHER STUDY

1. It is suggested that a wind load analysis, based on the cantilever method (7), should be carried out and the results compared with the results of a computer analysis to find out if this method would be better suited for the preliminary analysis.

2. A redesign should be carried out using the design forces from the computer analysis, and with the resulting member properties a second computer analysis should be conducted.

3. It is also suggested that similar approximate and computer analyses should be conducted with frames having different geometry and loading to determine if the conclusions made in this report are applicable to these other design conditions.

NOTATION

The notation used in this report is the standard notation used by most design engineers. The notation used in the beam and column design is that given in the AISC Specification (3)

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DESIGN OF A MULTI-STORY STEEL FRAME

by

MUKESH N. JHAVERI

B. E. (Civil) M. S. University of Baroda, India, 1970

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ABSTRACT

In this report, one method of carrying out an approximate preliminary analysis and design of a multi-story, steel, rigid frame structure is demonstrated with the help of a design example. The structure used in the example is a ten story, two bay frame. Allowable stress design is used for the selection of members. The resulting member properties are used to carry out a stiffness analysis with the help of the STRUDL computer program. The results of the STRUDL program are compared with the results of the approximate analysis.