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RELIABILITY ANALYSIS OF CODE-DESIGNED STRUCTURES UNDER NATURAL HAZARDS

by

Howard H.M. Hwang
Center for Earthquake Research and Information
Memphis State University
Memphis, TN 38152

Hideharu Ushiba
Department of Civil Engineering and Engineering Mechanics
Columbia University
New York, NY 10027

Masanobu Shinozuka
Department of Civil Engineering and Operations Research
Princeton University
Princeton, NJ 08544

Technical Report NCEER-88-0008

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NOTICE

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1 Associate Research Professor, Center for Earthquake Research and Information, Memphis State University
2 Graduate Student, Columbia University, on leave from Shimizu Construction Company, Tokyo, Japan
3 Renwick Professor of Civil Engineering, Columbia University (Professor of Civil Engineering, Princeton University, as of Februrary, 1988.)

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
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ABSTRACT

This paper presents a preliminary study to assess the structural integrity of low-rise buildings which are designed according to appropriate provisions of ANSI A58.1-1982 and ACI Code 318-83. The main purpose of this study is to demonstrate how the knowledge of different scientific and engineering disciplines can be combined and synthesized to assess the actual degree of protection against natural hazards. The low-rise buildings considered in this paper are a shear wall structure and a flat-plate structure supposed to be located in New York City. These structures are designed to resist earthquake and wind forces separately. For the reliability assessment, seismic and wind hazards in the New York City area are estimated. The structural response to these hazards is then evaluated by using formulas specified in ATC 3-06. The variability of the structural response is quantified. In addition, the variability of the structural capacity is also assessed. The structural integrity is measured in terms of the annual limit state probability which provides a quantitative measure for comparing the relative extent of risk due to different natual hazards such as wind and earthquake.

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SECTION 1
INTRODUCTION

Conventional structures, in particular, low-rise buildings, are usually designed according to provisions specified in building codes and standards such as the Uniform Building Code (UBC) [1], Standard Building Code (SBC) [2] and American National Standard ANSI A58.1 [3]. The code provisions are intended to achieve the satisfactory performance of a building under loads imposed by users or nature such as wind or earthquake. However, building codes usually employ simplified formulas in the provisions in order to facilitate the design process. For example, equivalent static design forces are stipulated in building codes to represent wind or seismic forces which are dynamic and random in nature. Seismic hazards in the United States are grossly divided into several seismic zones to represent different degrees of seismic hazard and a typical peak ground acceleration (PGA) value is assigned to each zone. Furthermore, some building codes, e.g., New York City building laws, have provisions only for wind design without any provisions for aseismic design. Concern has been raised as to whether or not a building designed only for wind loads is safe under potential seismic hazards. There is no doubt that building codes should utilize simplified rules to facilitate the design process. However, the validity of these rules and their impact on building safety should be investigated.

This paper presents a preliminary study to assess the structural integrity of low-rise buildings which are designed according to appropriate provisions of ANSI A58.1-1982 [3] and ACI Code 318-83 [4]. The main purpose of this study is to demonstrate how knowledge of different scientific and engineering disciplines can be combined and synthesized to assess the actual degree of protection against natural hazards. The low-rise buildings considered in this paper are a shear wall structure and a flat-plate structure located in New York City. These structures are designed to resist earthquake and wind forces separately. Seven design cases are listed in Table 1-I. For the reliability assessment, seismic and wind hazards in the New York City area are estimated. The structural response to these hazards is evaluated by using formulas specified in ATC 3-06 [5]. The variability of the structural response is quantified. In addition, the variability of the structural capacity is also assessed. The structural integrity is measured in terms of the annual limit state probability, which is the probability per year that a limit state (failure criterion) will be reached. While, as is well known, the accuracy and interpretation of such a probability is still open to discussion, it

Table 1-I Design Cases

Case	Notation	Loading Condition
1	E - 2 - S_1	Earthquake, Zone 2, S_1
2	E - 2 - S_2	Earthquake, Zone 2, S_2
3	E - 2 - S_3	Earthquake, Zone 2, S_3
4	E - 1 - S_1	Earthquake, Zone 1, S_1
5	E - 1 - S_2	Earthquake, Zone 1, S_2
6	E - 1 - S_3	Earthquake, Zone 1, S_3
7	Wind	Wind

still provides a quantitative measure for comparing relatively the extent of risk to which a structure is subjected under different natural hazards; wind and earthquake in the present case.

SECTION 2
DESIGN OF SHEAR WALL STRUCTURE

The first building designed for this study is a five-story office building located in New York City. Appendix A shows the detail of the design, while the essential part of the design is summarized in this section. Figure 2-1 shows a typical floor plan and cross-section of the building. A reinforced concrete frame system is used to resist vertical loads, i.e., dead and live loads. The two reinforced concrete shear walls in the north-south direction as shown in Fig. 2-1 are used to resist all the lateral forces due to wind or earthquake loads in that direction. This study focuses on the design and reliability assessment of these two shear walls.

Four types of loads, i.e., dead, live, wind and earthquake loads are considered to act on the building. The design values of these loads are specified according to the provisions of American National Standard ANSI A58.1-1982 [3].

2.1 Dead and Live Loads

The dead and live loading conditions are listed below.

a) Dead Load

* Roof:

  5″ slab and 1″ finish — 63 + 12 = 75 psf

* 5th thru. 2nd Floor:

  5″ slab and 1.5″ finish — 63 + 18 = 81 psf

* Girder: Assuming 16″× 27″

  16″× (27″ - 5″) × 155 pcf/144 = 379 plf

* Beam: Assuming 12″× 23″

  12″× (23″ - 5″)× 155 pcf/144 = 233 plf

* Column:

  3rd-5th Floors — 20″× 20″× 155 pcf/144 = 431 plf

  1st-2nd Floors — 22″× 22″× 155 pcf/144 = 521 plf

* Exterior Walls: = 15 psf

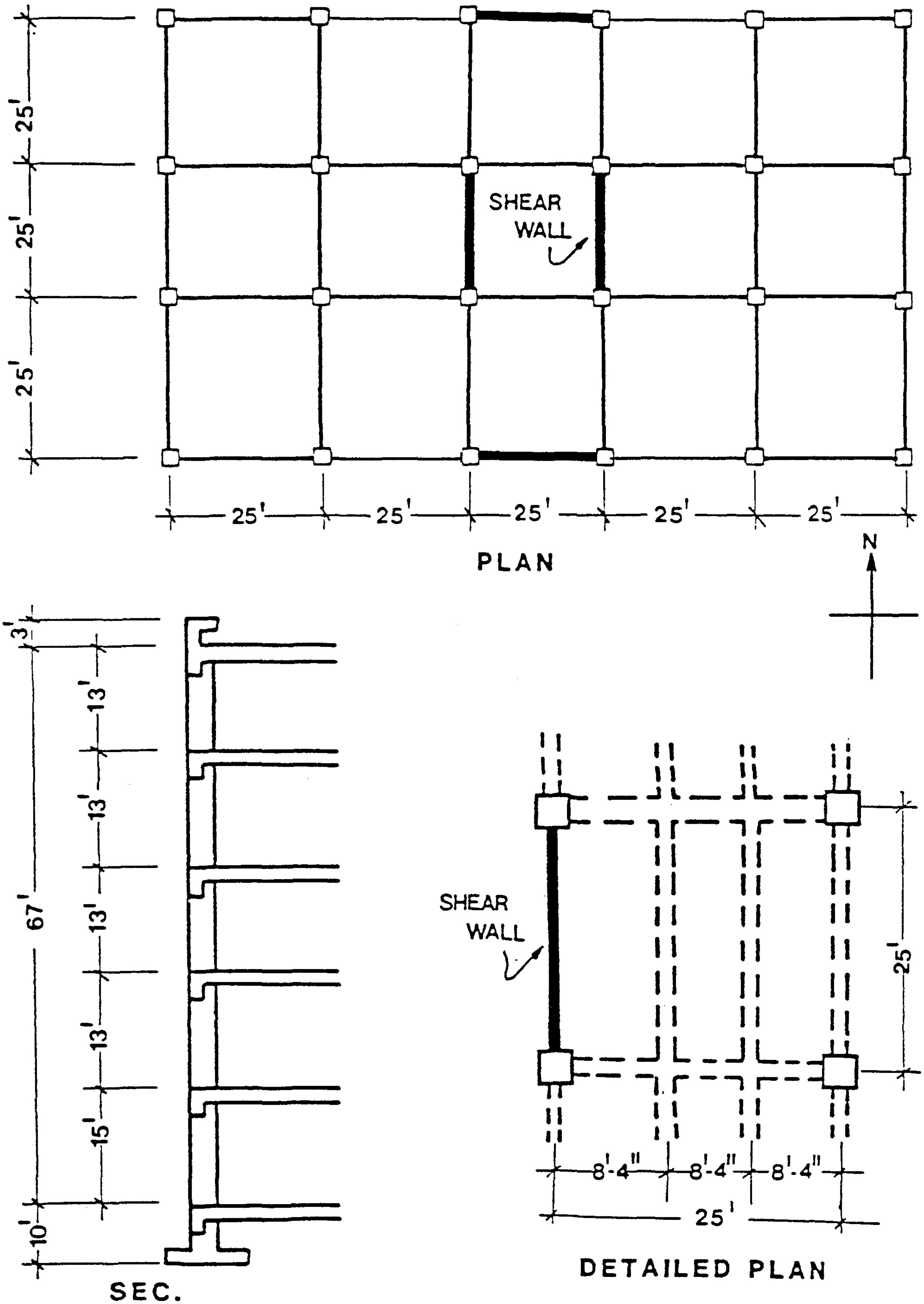

Fig. 2-1 Plan and Section of Office Building

* Shear Walls:

6″ thickness and 3″ finish	76 + 36 = 112 psf
5″ thickness and 3″ finish	64 + 36 = 100 psf

b) Live load

Roof:	20 psf
2nd-5th Floors:	50 psf

The analysis of frame system due to dead and live loads follows a conventional procedure.

2.2 Wind Load

The wind velocity pressure q_z specified in ANSI A58.1-1982 is

$$q_z = 0.00256\ k_z(IV)^2 \tag{2.1}$$

where V is the basic wind speed at a reference height of 33 ft for exposure C. From the map of basic wind speeds in ANSI A58.1-1982, $V = 80\ mph$ in New York City for a return period of 50 years. The importance factor I is chosen to be 1.05 (Category I at hurricane ocean line). The velocity pressure coefficient k_z varies with height. For exposure B considered here, k_z and q_z are listed in Table 2-I.

The design wind pressure P_z is determined by the following formula:

$$P_z = q_z G_h C_{p(W)} - q_h G_h C_{p(L)} \tag{2.2}$$

where G_h is the gust response factor at a height of h ft. For exposure B at 70 ft, $G_h =$ 1.36. q_h is the wind pressure for a leeward wall and roof evaluated at mean roof height. $C_{p(W)}$ and $C_{p(L)}$ are the wall pressure coefficients for the windward and leeward walls, respectively. In this case, $C_{p(W)} = 0.8$ and $C_{p(L)} = -\ 0.5$. The design wind pressure P_z is also shown in Table 2-I and plotted in Fig. 2-2. For design convenience, the design wind pressure is converted into a concentrated lateral load at each floor level, as shown in Fig. 2-2. The lateral loads acting on each shear wall are computed as follows:

$$H_1 = 23.32 \times (125 \times 9.5)/(2 \times 1000) = 13.82\ kips$$

$$H_2 = [23.32 \times (125 \times 10.5) + 21.35 \times (125 \times 2.5)]/(2 \times 1000) = 18.64\ kips$$

Table 2-I Design Wind Pressure (Office Building)

Height (ft)	k_z	q_z (psf)	P_z (psf)
50 - 70	0.73	13.19	23.32
30 - 50	0.63	11.38	21.35
15 - 30	0.50	9.03	18.80
0 - 15	0.37	6.68	16.24

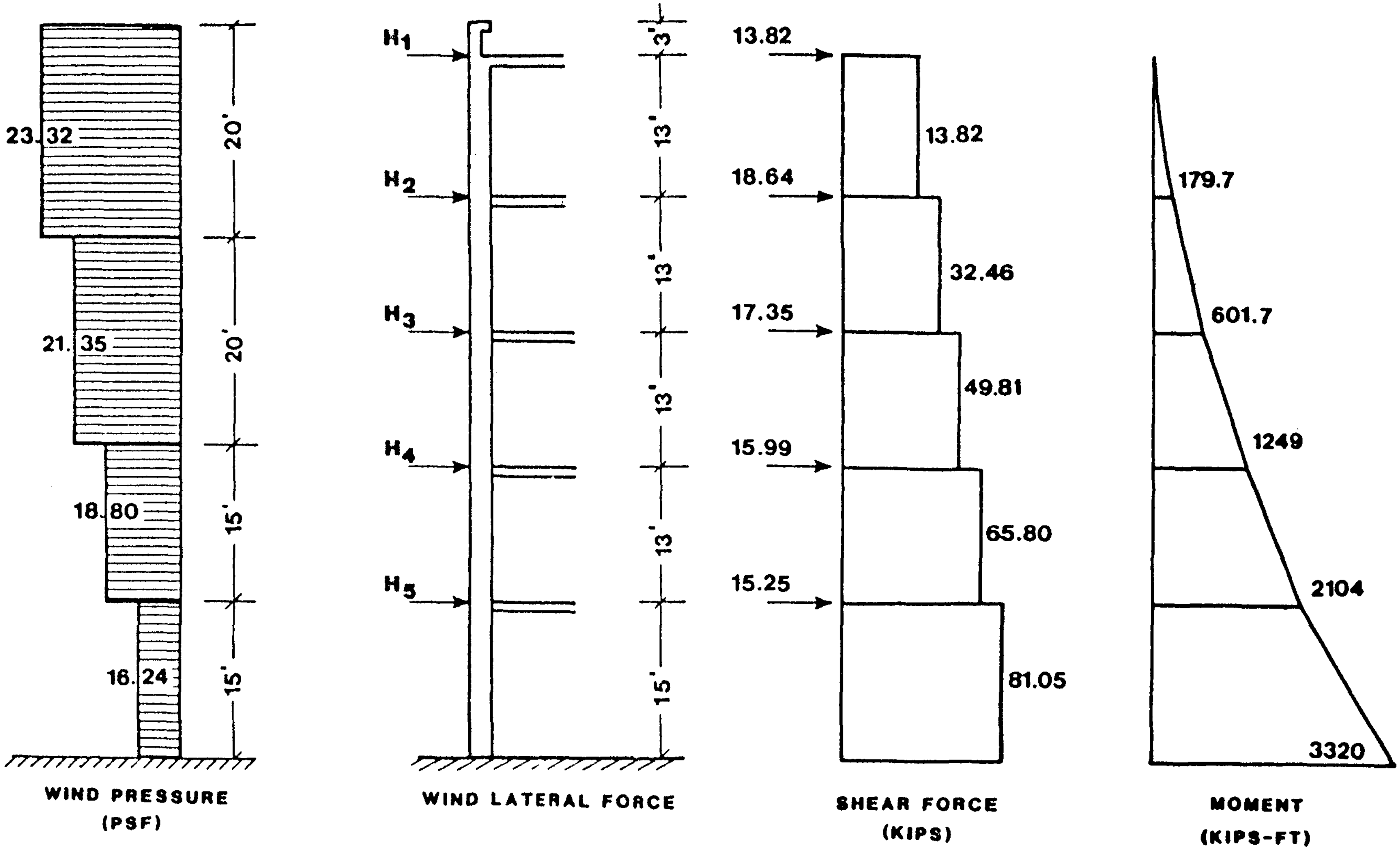

Fig. 2-2 Design Wind Pressure and Forces (Office Building)

$$H_3 = 21.35 \times (125 \times 13.0)/(2 \times 1000) = 17.35\ kips$$
$$H_4 = [21.35 \times (125 \times 4.5) + 18.80 \times (125 \times 8.5)]/(2 \times 1000) = 15.99\ kips$$
$$H_5 = [18.80 \times (125 \times 6.5) + 16.24 \times (125 \times 7.5)]/(2 \times 1000) = 15.25\ kips$$

The shear force and overturning moment due to these concentrated lateral loads can be determined and shown in Fig. 2-2.

2.3 Seismic Load

The design base shear Q due to earthquake specified in ANSI A58.1-1982 is

$$Q = ZIKCSW \tag{2.3}$$

where Q = total shear force at the base, Z = zone factor, I = importance factor, K = building system factor, C = numerical coefficient, S = soil factor and W = total dead load of the building.

New York City is located in seismic zone 2 according to the map for seismic zones in ANSI A58.1-1982. In this study, however, zone 1 is also used to design the shear wall in order to evaluate the effect of seismic zones on the safety of buildings. For seismic zones 1 and 2, Z is 3/16 and 3/8, respectively. The importance factor I and building system factor K are taken as 1.0. The value of C is determined by

$$C = \frac{1}{15\sqrt{T}} \tag{2.4}$$

in which T is the fundamental period of the building in seconds and is computed by the following formula:

$$T = \frac{0.05h_n}{\sqrt{D}} \tag{2.5}$$

where h_n is the building height from the base and D is the dimension of the building in the direction parallel to the applied seismic forces. For the building under consideration, h_n = 77 ft and D = 75 ft, thus, T is 0.45 sec and C is equal to 0.10.

In ANSI A58.1-1982, three types of soil are defined and denoted as S_1, S_2 and S_3. In this study, all three types of soil are considered. Thus, the soil factor S is 1.0, 1.2 and 1.5 for

S_1, S_2 and S_3, respectively. Furthermore, ANSI A58.1-1982 also specifies that the product of C and S need not exceed 0.14. Hence, in this study, for the soil type of S_3, CS is taken as 0.14 instead of 0.15. Dead load of the building W is calculated in Table 2-II. For seismic zone 2 and S_1 soil condition, the total seismic base shear Q determined by Eq. 2.3 is

$$Q = 3/8 \times 1.0 \times 1.0 \times 0.10 \times 8224.3 = 308.4\ kips$$

The seismic base shear coefficient, i.e. $ZIKCS$, and the seismic base shears under various design conditions are tabulated in Table 2-III. The base shear is distributed over the height of the structure by using the following formula.

$$F_x = \frac{(Q - F_t)W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{2.6}$$

where

F_x = Lateral force applied at level x.
F_t = Additional concentrated lateral force at top of structure.
h_x, h_i = Height from the base to levels x or i, respectively.
W_x, W_i = Weight located or assigned to level x or i, respectively.
N = Number of stories.

According to ANSI A58.1-1982, F_t may be considered as zero when T is 0.7 second or less. In this case, T = 0.45 sec., thus, $F_t = 0$. The calculation of F_x is shown in Table 2-IV for Zone 2 and S_1 soil condition. Given the lateral force, the shear force and overturning moment at each floor level can be determined. For seismic zone 2 and all three soil conditions, the shear force and overturning moment for each shear wall are shown in Fig. 2-3. For seismic zone 1, the shear force and moment are one-half those shown in Fig. 2-3.

2.4 Design of Shear Wall

The shear wall is designed according to ACI Code 318-83. The purpose of a structural design is to provide the structure or its components with sufficient resisting capacity against all postulated combinations of load effects (axial force, shear force, moment, etc). The design formulas specified in ACI Code 318-83 are

Table 2-II Calculation of Total Dead Load (Office Building)

Item	Calculation	Weight (kips)
Roof		
Roof	$75 \times 125 \times 75$	703.1
Girder	$379 \times (125 \times 4 + 75 \times 6)$	360.1
Beam	$233 \times (75 \times 2 \times 5)$	174.8
Column	$431 \times 6.5 \times 24$	67.2
Exterior Walls	$15 \times 6.5 \times 350$	34.1
Shear Walls	$100 \times 6.5 \times 100$	65.0
Subtotal		1404.3
5th and 4th Floors		
Floor	$81 \times 125 \times 75$	759.4
Girder	$379 \times (125 \times 4 + 75 \times 6)$	360.1
Beam	$233 \times (75 \times 2 \times 5)$	174.8
Column	$431 \times 13.0 \times 24$	134.5
Exterior Walls	$15 \times 13.0 \times 350$	68.3
Shear Walls	$100 \times 13.0 \times 100$	130.0
Subtotal		1627.1
3rd Floor		
Floor	$81 \times 125 \times 75$	759.4
Girder and Beam	$360.1 + 174.8$	534.9
Column	$(431 + 521) \times 6.5 \times 24$	148.5
Exterior Walls		68.3
Shear Walls	$(100 + 112) \times 6.5 \times 100$	137.8
Subtotal		1648.9
2nd Floor		
Floor	$81 \times 125 \times 75$	759.4
Girder and Beam	$360.1 + 174.8$	534.9
Column	$521 \times 14.0 \times 24$	175.1
Exterior Walls	$15 \times 14.0 \times 350$	73.5
Shear Walls	$112 \times 14.0 \times 100$	156.8
Subtotal		1699.7
1st Floor		
Column	$521 \times 7.5 \times 24$	93.8
Exterior Walls	$15 \times 7.5 \times 350$	39.4
Shear Walls	$112 \times 7.5 \times 100$	84.0
Subtotal		217.2
Total Dead Load W		8224.3

Table 2-III Seismic Base Shear (Office Building)

Case	Earthquake	Base Shear Coeff.	Total Base Shear ($kips$)
1	E - 2 - S_1	0.0375	308.4
2	E - 2 - S_2	0.0450	370.1
3	E - 2 - S_3	0.0525	431.8
4	E - 1 - S_1	0.01875	154.2
5	E - 1 - S_2	0.0225	185.1
6	E - 1 - S_3	0.02625	215.9

Table 2-IV Seismic Lateral Force (E - 2 - S_1)

Level	W_x $(kips)$	h_x (ft)	$W_x h_x$	F_x $(kips)$
Roof	1404.3	77	108131	82.8
5th Floor	1627.1	64	104134	79.8
4th Floor	1627.1	51	82982	63.6
3rd Floor	1648.9	38	62658	48.0
2nd Floor	1699.7	25	42493	32.6
1st Floor	217.2	10	2172	1.7
$\sum W_1 h_1$			402570	

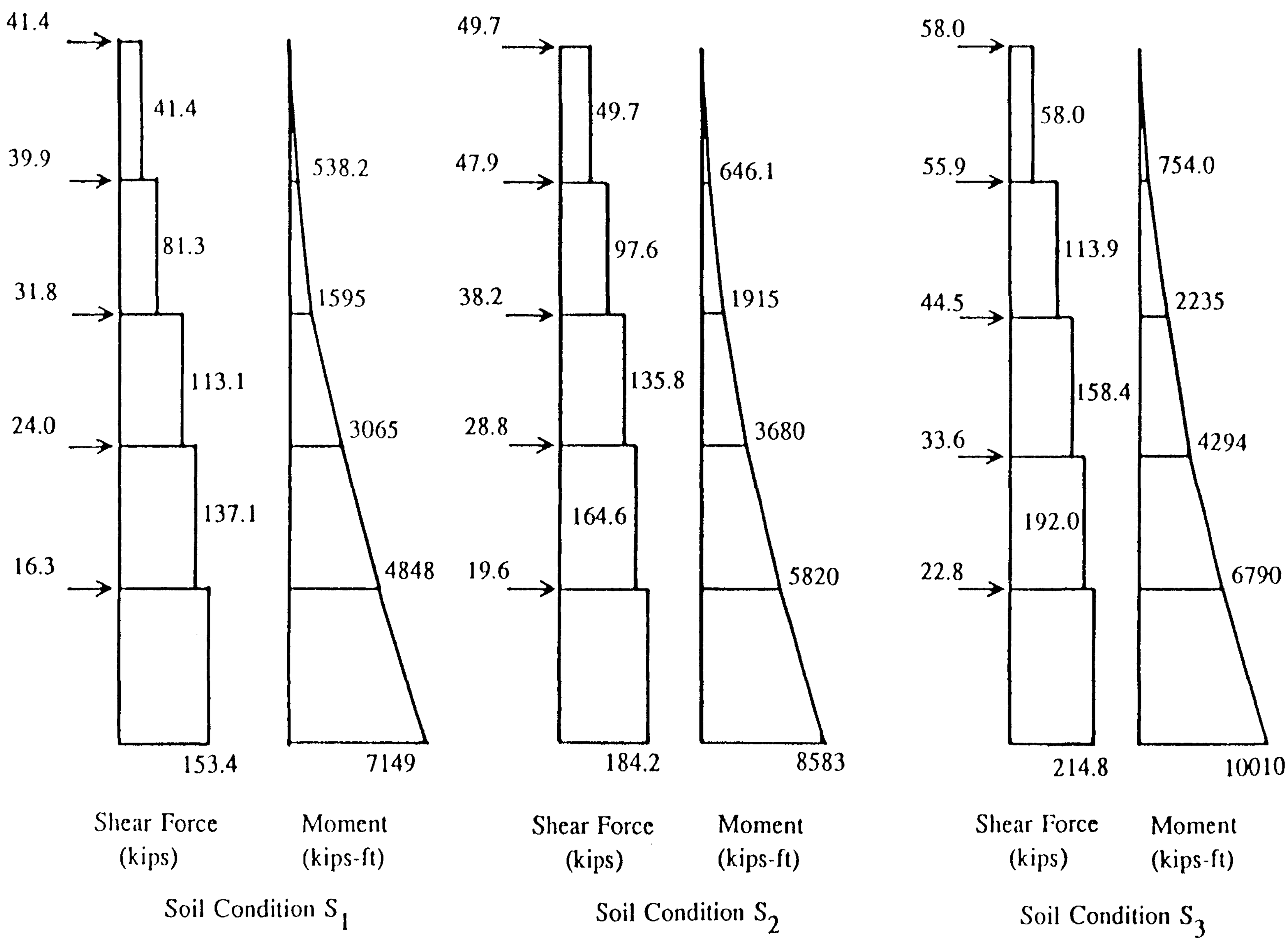

Fig. 2-3 Seismic Forces (Seismic Zone 2)

$$\phi R_n \geq \begin{cases} 1.4D + 1.7L \\ 0.75(1.4D + 1.7L + 1.7W) \\ 0.9D + 1.3W \\ 0.75(1.4D + 1.7L + 1.87E) \\ 0.9D + 1.43E \end{cases} \tag{2.7a - 2.7e}$$

where D = dead load effect, L = live load effect, W = load effect due to wind (not to be confused with the W used for dead weight in Eq. 2.3), E = load effect due to earthquake, ϕ = strength reduction factor and R_n = nominal capacity. It is noted that the shear wall in this study is designed separately for wind and earthquake (zone 2 or 1) in order to evaluate the integrity of the shear wall with respect to these two different types of natural hazards.

For wind load, the shear wall is designed according to Eqs. 2.7b and 2.7c. It is assumed that frame structures resist vertical loads and the overturning moment due to lateral force is resisted by end columns; thus, the shear wall is designed only for shear force. Furthermore, it is assumed that the critical section to be designed is at the bottom of the shear wall. Under these assumptions, Eqs. 2.7b and 2.7c become

$$\phi V_n \geq 1.3 Q_W \tag{2.8}$$

where V_n = nominal shear capacity of shear wall and Q_W = design shear force at the bottom of the shear wall. From Fig. 2-2, Q_W = 81.05 kips, and hence, $1.3Q_W$ = 105.4 kips.

The nominal shear capacity V_n specified in ACI Code 318-83 is

$$V_n = V_c + V_s \tag{2.9}$$

where V_c and V_s are the shear strength provided by concrete and reinforcement, respectively.

$$V_c = 2\sqrt{f'_c}\ td \tag{2.10}$$

$$V_s = \frac{A_v f_y d}{s_2} \tag{2.11}$$

where f_c' is the compressive strength of concrete and $f_c' = 3000$ psi in this study. f_y is the yield strength of the reinforcement, and for #3 and #4 rebars, f_y is specified as 40,000 psi. A_v is the area of horizontal shear reinforcement within a vertical distance of s_2. t is the thickness of the shear wall and $d = 0.8\ell_w$ in which ℓ_w is the length of the shear wall. Assuming the wall thickness is 5″ and the cross-section of end columns is 22″ × 22″, then, the shear strength provided by concrete is

$$V_c = 2 \times \sqrt{3000} \times 5 \times 0.8 \times (25 \times 12 - 22)/1000 = 121.8 \ kips$$

The minimum horizontal reinforcement ratio ρ_h required by ACI 318-1983 is 0.0025. For one layer of #3 rebars ($A_v = 0.11$ sq. in.) with yield strength $f_y = 40{,}000$ psi, the maximum spacing of s_2, i.e., $s_{2,max}$ to meet this minimum reinforcement requirement is

$$s_{2,max} = \frac{A_v}{t\rho_h} = 8.8''$$

Hence, s_2 is taken to be 8″ in this study. This produces the shear strength provided by steel reinforcement equal to 122.3 kips (Eq. 2.11), and the nominal shear capacity V_n equal to 244.1 kips (Eq. 2.9). The strength reduction factor ϕ for shear is 0.85 as specified in the ACI code. Thus, $\phi V_n = 207.5$ kips which is much larger than the factored design shear 105.4 kips. This apparently excessive over-capacity is resulted from the minimum reinforcement requirement specified by code. The design of the shear wall to wind load is summarized in Table 2-V.

The design formulas for earthquake load are Eqs. 2.7d and 2.7e. Since again the vertical loads are resisted by frame structures, Eqs. 2.7d and 2.7e become

$$\phi V_n \geq 1.43 \ Q_E \tag{2.12}$$

where Q_E is the design shear force due to earthquake at the bottom of the shear wall. The shear capacity for resisting earthquake forces is provided in the same way as that for wind loads. The results are also summarized in Table 2-V.

Table 2-V Design of Shear Wall

Case	t (in)	Horizontal Reinforcement	Vc (kips)	Vs (kips)	ϕVn (kips)	$1.43Q_E$ or $1.3Q_W$ (kips)
1	5	#3@7in	121.8	139.8	222.4	219.4
2	5	#3@5in	121.8	195.7	269.9	263.4
3	5	#4@7in	121.8	254.2	319.6	307.2
4	5	#3@8in*	121.8	122.3	207.5	109.7
5	5	#3@8in*	121.8	122.3	207.5	131.7
6	5	#3@8in*	121.8	122.3	207.5	153.6
7	5	#3@8in*	121.8	122.3	207.5	105.4

* Minimum reinforcement required by ACI 318-83.

SECTION 3
DESIGN OF FLAT PLATE STRUCTURE

The second building designed for this study is a five-story apartment building which consists of two-way flat plates and columns as shown in Fig 3-1. The building is also assumed to be located in New York City. The design of this flat-plate structure is limited to a typical interior frame in the north-south direction. The detail of the design is shown in Appendix B.

Similar to the first building, four loads, i.e., dead load, live load, wind and earthquake are assumed to act on the structure. Dead load is computed from the weight of the structure. for example, it is assumed that the roof through second floor slabs are made of 8 in. reinforced concrete slab. Thus, the weight of the slab is 100 psf. According to ANSI A58.1-1982, the live load acting on the roof is 20 psf and the live load on the floor is 50 psf, in which the weight of partitions is included. The analysis for dead and live loads follows the conventional procedure.

Wind load on the five-story flat plate structure is analyzed following the same procedure as that described in Section 2.2. Using Eqs. 2.1 and 2.2, the design wind pressure is calculated and shown in Table 3-I. The lateral wind load acting on each floor of a typical frame is shown in Fig. 3-2.

The total seismic base shear is determined using Eq. 2.3:

$$Q = ZIKCSW$$

For this flat-plate structure, the values of Z, I and S are the same as those used in Section 2.3. The value for K is taken as 1.0. The dimension of the building in the N-S direction is 60 ft. and the total height above the base is 70 ft. Hence, the fundamental period of the structure is estimated as 0.45 sec. (Eq. 2.5), and the value of C is determined as 0.10 (Eq. 2.4). As mentioned in Section 2.3, the product of C and S is limited to 0.14. This limitation applies to soil condition S_3 in this case, $CS = 0.14$ intsead of 0.15. The total dead load of the apartment building is shown in Table 3-II. For seismic zone 2, the total seismic base shear for soil conditions S_1, S_2, and S_3 are 138.2 kips, 165.8 kips and 193.5

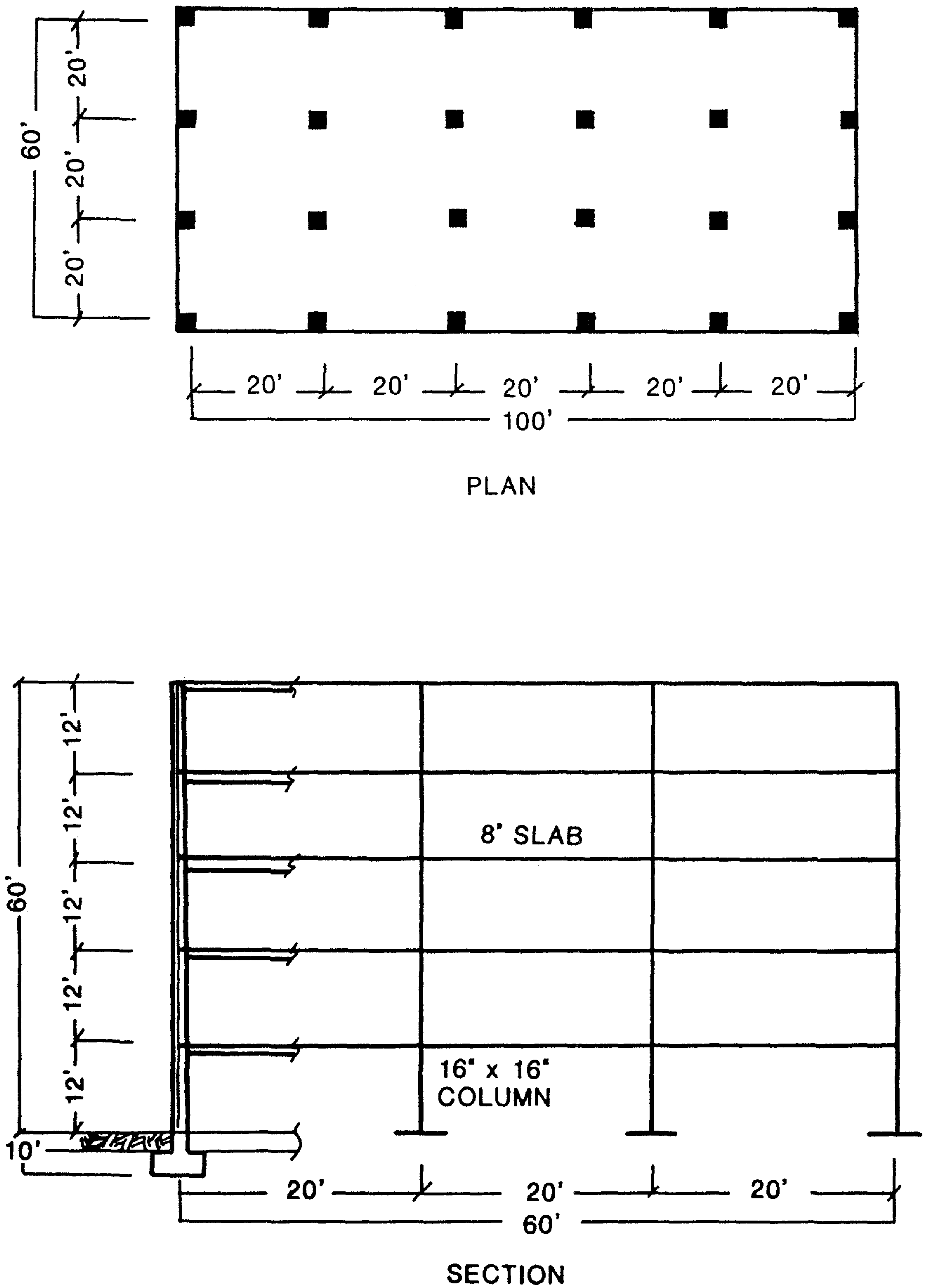

Fig. 3-1 Plan and Section of Apartment Building

Table 3-I Design Wind Pressure (Apartment Building)

Height (ft)	k_z	q_z (psf)	P_z (psf)
50 - 60	0.68	12.28	22.20
30 - 50	0.63	11.38	21.19
15 - 30	0.50	9.03	18.58
0 - 15	0.37	6.68	15.97

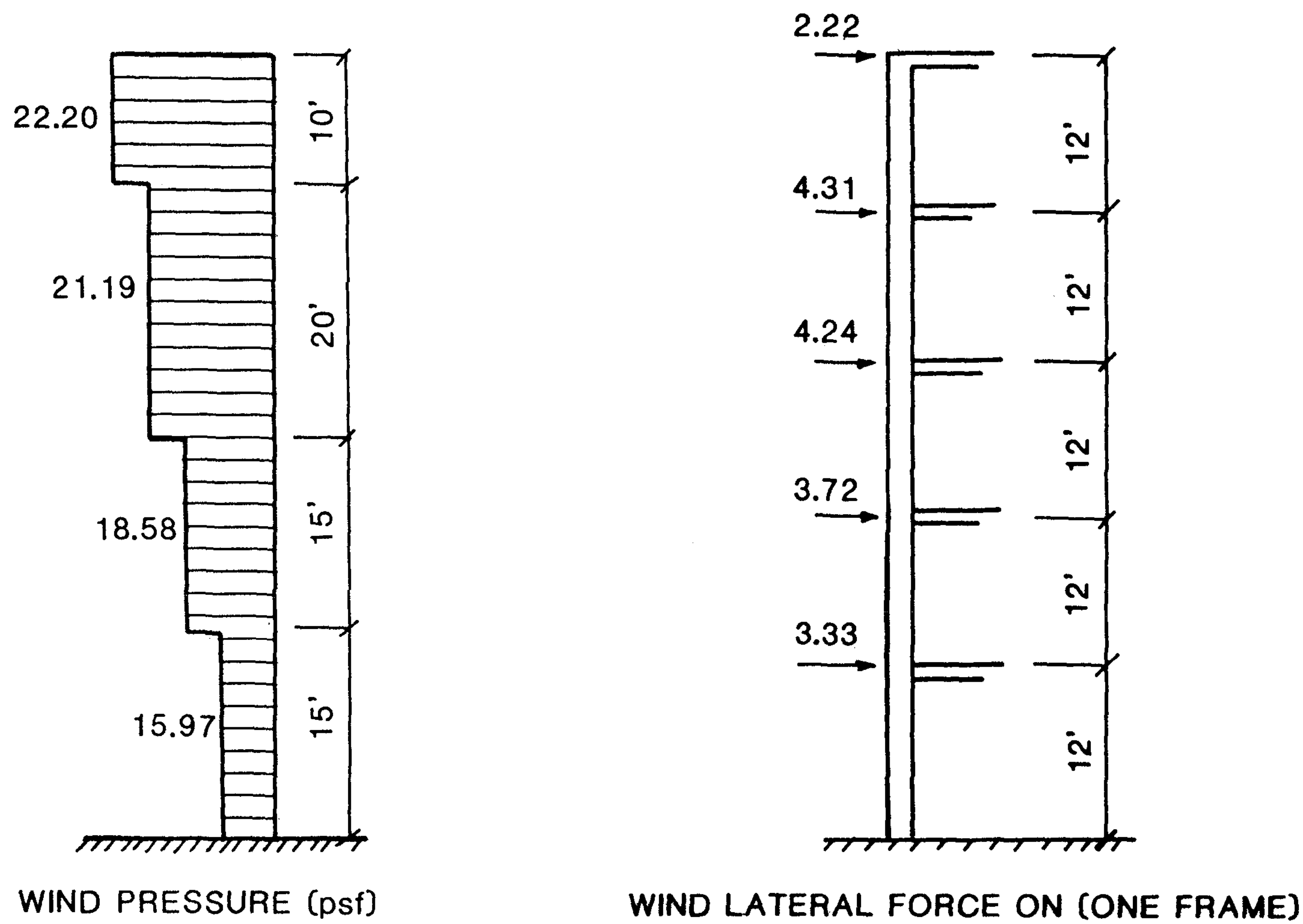

Fig. 3-2 Design Wind Pressure and Forces (Apartment Building)

Table 3-II Calculation of Total Dead Load (Apartment Building)

Item	Calculation	Weight ($kips$)
Roof:		
Roof	$100 \times 100 \times 60$	600.0
Column	$276 \times 6.0 \times 24$	39.7
Exterior Walls	$15 \times 6.0 \times 320$	28.8
Subtotal		668.5
2nd-5th Floors:		
Floor	$100 \times 100 \times 60$	600.0
Column	$276 \times 12.0 \times 24$	79.5
Exterior Walls	$15\times 12.0 \times 320$	57.6
Subtotal		$737.1 \times 4 = 2948.4$
1st Floor:		
Column	$276 \times 6.0 \times 24$	39.7
Exterior Walls	$15 \times 6.0 \times 320$	28.8
Subtotal		68.5
Total Dead Load W		3685.4

kips, respectively. The total base shear is distributed over the building height. Under the assumption that all six frames share the seismic load equally, the seismic force acting on each floor is determined and tabulated in Table 3-III. The seismic force for zone 1 is one-half the value shown in Table 3-III.

The detail of the design of the flat-plate and columns is shown in Appendix C. The design is also based on ACI 318-83 [4]. For lateral loads, the most critical section is the flat plate at the ends of the column strips. The design of the flat plate (column strip) is summarized in Table 3-IV. In addition, the design of the columns is summarized in Table 3-V.

Table 3-III Lateral Seismic Force (per frame, Zone 2)

Level	S_1	S_2	S_3
Roof	6.52	7.82	9.12
5th Floor	5.95	7.15	8.33
4th Floor	4.72	5.67	6.62
3rd Floor	3.48	4.18	4.88
2nd Floor	2.25	2.70	3.17

Seismic Force for Zone 1 is one-half the value shown in the Table

Table 3-IV Design of Flat-Plate (Column Strip)

Case	At Face of Exterior Column			At Face of Interior Column		
	Mu (*ft-kips*)	ϕMn (*ft-kips*)	Rebars	*Mu* (*ft-kips*)	ϕMn (*ft-kips*)	Rebars
1	80.3	85.6*	14-#4	132.1	132.2	22-#4
2	90.5	91.5	15-#4	142.3	143.6	24-#4
3	100.8	103.3	17-#4	152.5	154.9	26-#4
4	54.8	85.6*	14-#4	106.5	120.7*	20-#4
5	59.9	85.6*	14-#4	111.6	120.7*	20-#4
6	65.0	85.6*	14-#4	116.8	120.7*	20-#4
7	63.5	85.6*	14-#4	115.3	120.7*	20-#4

*Governed by gravity loading

Table 3-V Main Reinforcement of Columns

Case	Exterior Column	Interior Column
1	4-#8	4-#8
2	4-#8	6-#8
3	4-#8	6-#9
4	4-#8	4-#8
5	4-#8	4-#8
6	4-#8	4-#8
7	4-#8	4-#8

Note: 1. Column size is 16 in. $\times$ 16 in.

2. 4-#8, ($\rho_g = 0.012$) is minimum requirement of ACI 318-83

SECTION 4
PROBABILISTIC CHARACTERISTICS OF STRUCTURAL CAPACITY AND LOADS

The nominal structural capacity (resistance) and design loads are specified by simplified deterministic formulas in building codes. The single values determined by such formulas are for design purposes. In reality, the actual structural capacity and loads are random in nature and also involve modeling as well as parametric uncertainty. For example, we not only cannot predict in advance the occurrence of an earthquake but also cannot precisely estimate its intensity and duration. Similarly, the structural resistance cannot be determined precisely since basic parameters such as material strength always exhibit statistical variation. In addition, the failure mechanism of a structure, which is needed to define the structural resistance, is usually very complicated and cannot be defined with certainty. Furthermore, structural behavior is always idealized to simplify the analysis. In view of the randomness and uncertainty in loads, structural resistance and structural behavior etc., a probabilistic approach for the assessment of structural integrity is a rational choice, since the theory of probability provides a framework for the formal treatment of uncertainties.

An important ingredient for reliability analysis is the identification of limit states. A limit state represents a state of undesirable structural behavior. It is identified with the aid of experimental observations and analytical predictions of the actual behavior of a structure under all conceivable loading conditions. For a structural system, it is likely that more than one limit state has to be considered. Also, limit states must be specified in terms of the response quantities obtained by the selected structural analysis. In this paper, the limit state is defined in terms of base shear. It is recognized that other limit states such as those in terms of displacement ductility or energy absorption may be important and should be considered. However, the present study is a preliminary analysis which intends to illustrate how reliability analysis can be used to access the integrity of code-designed structures and to identify factors which are significant in the reliability assessment process. Thus, a simple limit state involving only base shear is considered.

In this study, it is assumed that the key parameters of structural capacity and structural responses can be treated as random variables whose variability represents a combination of randomness and uncertainty. Furthermore, it is assumed that these parameters are log-normally distributed. A log-normal variable X can be described by its median value $\tilde{X}$

and the logarithmic standard deviation β_X, i.e., the standard deviation of $\ell n\ X$. If the coefficient of variation (COV) is not very large, say, less than about 0.4, β_X is approximately equal to the COV value of random variable X.

4.1 Structural Capacity

The structural capacity is affected by variations in material strength, structural geometry and workmanship. For low-rise shear wall structures, Ellingwood and Hwang [6] estimate that the median ultimate shear capacity of a shear wall $\tilde{Q}_R^*$ is about 1.70 times the nominal capacity V_n and the COV is 0.18. On the basis of these estimations, the capacities of shear walls designed for various conditions are summarized in Table 4-I. For flat-plate structures, the median ultimate capacity is derived based on the plastic analysis shown in Appendix D. Table 4-II lists the structural capacity statistics for all cases in terms of base shear. In Table 4-II, β_{QR} is taken as 0.25. This value follows from the engineering judgement that the difference between the median ultimate capacity obtained from plastic analysis (Appendix D) and that computed in accordance with the design code (Appendix B) is approximatly three times the standard duration of the random ultimate capacity. In this connection, the ultimate capacity evaluated in accordance with the design code is assumed to be the minimum value of the random ultimate capacity.

4.2 Base Shear Due to Wind

The probabilistic model for wind pressure P^* is

$$P^* = 0.00256\ C_p^* K_z^* G_h^* (V^*)^2 \tag{4.1}$$

where V^* is the wind speed at reference height 10 m. From the analysis of observation data (1947-1977) at LaGuardia Airport in New York City, Simiu et al. [7] estimate that the annual extreme wind speed follows a Type I extreme-value distribution with expected value equal to 50.3 mph and standard deviation equal to 7.23 mph (COV = 0.14). In this study, it is assumed that the median value $\tilde{V}^*$ is the same as the mean, i.e., 50.3 mph and $\beta_V = 0.14$.

The statistics of C_p^*, K_z^* and G_h^* are described by Ellingwood et al. [8]. The median values of these factors, $\tilde{C}_p^*$, $\tilde{K}_z^*$ and $\tilde{G}_h^*$ are taken to be the same as the design values. Thus,

Table 4-I Statistics of Shear Wall Capacity

Case	Wall Thickness (in)	Horizontal Reinforcement	V_n (kips)	$\tilde{Q}_R^* = 1.7V_n$ (kips)	β_{QR}	Distribution
1	5	#3@7in	261.6	444.7		
2	5	#3@5in	317.5	539.8		
3	5	#4@7in	376.0	639.2		
4	5	#3@8in	244.1	415.0	0.18	Log-normal
5	5	#3@8in	244.1	415.0		
6	5	#3@8in	244.1	415.0		
7	5	#3@8in	244.1	415.0		

Table 4-II Statistics for Capacity of Flat Plate Structures

Case	$\tilde{Q}^*_{RE}$ (kips)	$\tilde{Q}^*_{RW}$ (kips)	β_{QR}	Distribution
1	82.4	98.4		
2	87.6	104.9		
3	92.7	110.7	0.25	Log-normal
4	80.1	95.8		
5	80.1	95.8		
6	80.1	95.8		
7		95.8		

$\tilde{C}_p^* = 1.3$, $\tilde{G}_h^* = 1.36$ and $\tilde{K}_z^*$ varies with height as shown in Table 2-I or Table 3-I. In addition, $\beta_{Cp} = 0.12$, $\beta_{Kz} = 0.16$ and $\beta_{Gh} = 0.11$ as indicated in Ref. 8 are also used in this study. Therefore, for one shear wall and an equivalent frame of the flat-plate structure, the median wind pressure $\tilde{P}^*$ and β_p are shown in Tables 4-III and 4-IV, respectively. The base shear Q_W^* due to wind is a product of the wind pressure and the exposed area of the building. The dimensions of the building are assumed to be deterministic. Thus, the variation of the base shear is the same as that of the wind pressure. Tables 4-III and 4-IV also show the median base shear due to wind $\tilde{Q}_W^*$ and β_{QW} for shear wall and flat-plate structures.

4.3 Seismic Base Shear

The total seismic base shear acting on the entire building, Q_{ET}^*, is determined by the following expression in ATC 3-06.

$$Q_{ET}^* = \frac{1.2\ S^* W^*}{R^* (T^*)^{2/3}}\ A^* \tag{4.2}$$

In Eq. 4.2, A^* is the annual extreme peak ground acceleration (PGA), which is usually assumed to follow Type II extreme-value distribution [8]:

$$F_{A^*}(a) = exp[-(\frac{a}{\mu})^{-\alpha}] \tag{4.3}$$

The parameters μ and α are estimated to be $\mu = 0.0135$ and $\alpha = 3.14$ for the New York City area [9]. Thus, Eq. 4.3 gives a COV of A^* equal to 0.6255 and $\tilde{A}^* = 0.01517$. In this study, A^* is assumed to follow a log-normal distribution with the same median $\tilde{A}^* = 0.01517$ and $\beta_{A^*} = 0.5746$ corresponding to COV = 0.6255. Figure 4-1 is a plot of the seismic hazard curves, in which the seismic hazard curve with $F_{A^*}(a)$ given by Eq. 4.3 is shown by a dashed curve and the seismic hazard curve corresponding to log-normal distribution by a solid curve. The log-normal assumption gives an unconservative estimate of the seismic hazards for extremely high values of A^*. However, it produces a conservative estimate of seismic hazards in the range of A^* where the structural capacity is primarily located. W^* is the weight of the structure. Ellingwood et al. [8] recommended that β_W be 0.10 and the median of $\tilde{W}^*$ be 1.05 times the design value. $1.2/(T^*)^{2/3}$ is a factor for linear dynamic response amplification. Based on the data collected by Haviland [10], the

Table 4-III Statistics of Base Shear Due to Wind (Shear Wall Structure)

Height (ft)	$\tilde{C}_p^*$	$\tilde{K}_Z^*$	$\tilde{G}_h^*$	$\tilde{V}^*$ (mph)	$\tilde{P}^*$ (psf)
50 - 70	1.3	0.73	1.36	50.25	8.34
30 - 50	1.3	0.63	1.36	50.25	7.20
15 - 30	1.3	0.50	1.36	50.25	5.71
0 - 15	1.3	0.37	1.36	50.25	4.23

$$\begin{aligned}\tilde{Q}_W^* &= (8.34 \times 20 + 7.20 \times 20 + 5.71 \times 15 + 4.23 \times 15) \times 125/(2 \times 1000) \\ &= 28.7\ kips\end{aligned}$$

$$\begin{aligned}\beta_{QW} = \beta_P &= (\beta_{Cp}^2 + \beta_{Kz}^2 + \beta_{Gh}^2 + 4\beta_V^2)^{1/2} \\ &= \left[(0.12)^2 + (0.16)^2 + (0.11)^2 + 4(0.14)^2\right]^{1/2} \\ &= 0.36\end{aligned}$$

Table 4-IV Statistics of Base Shear Due to Wind (Flat-Plate Structure)

Height (ft)	$\tilde{C}_p^*$	$\tilde{K}_Z^*$	$\tilde{G}_h^*$	$\tilde{V}^*$ (mph)	$\tilde{P}^*$ (psf)
50 - 60	1.3	0.68	1.39	50.25	7.94
30 - 50	1.3	0.63	1.39	50.25	7.36
15 - 30	1.3	0.50	1.39	50.25	5.84
0 - 15	1.3	0.37	1.39	50.25	4.32

$$\begin{aligned} \tilde{Q}_W^* &= (7.94 \times 10 + 7.36 \times 20 + 5.84 \times 15 + 4.32 \times 15) \times 100/(6 \times 1000) \\ &= 6.3\ kips \end{aligned}$$

$$\begin{aligned} \beta_{QW} = \beta_P &= (\beta_{Cp}^2 + \beta_{Kz}^2 + \beta_{Gh}^2 + 4\beta_V^2)^{1/2} \\ &= \left[(0.12)^2 + (0.16)^2 + (0.11)^2 + 4(0.14)^2\right]^{1/2} \\ &= 0.36 \end{aligned}$$

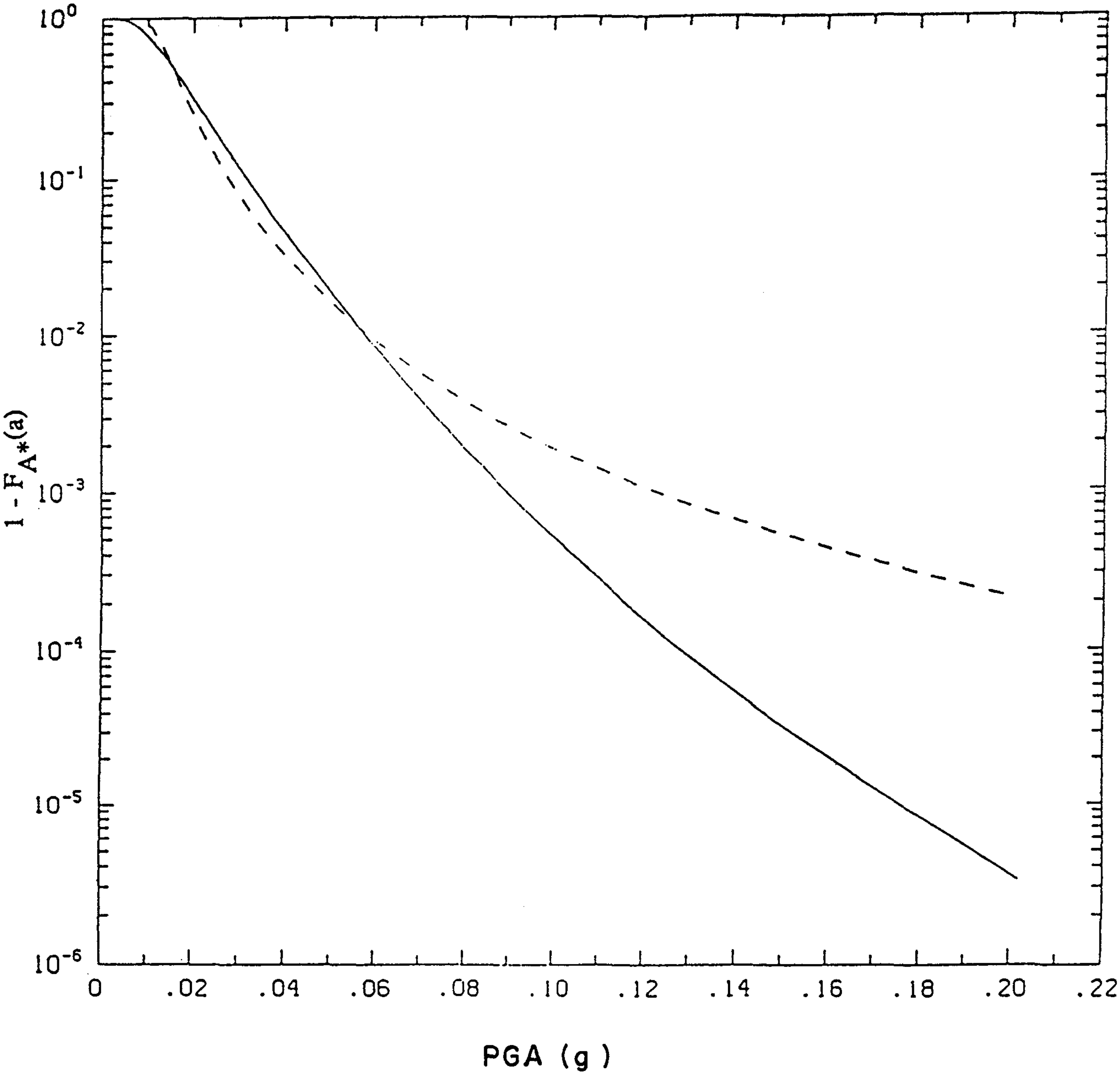

Fig. 4-1 Seismic Hazard Curve

median of period $\tilde{T}^*$ is taken to be 0.91 times the computed value, and β_T is 0.34. R^* is the (nonlinear) response modification factor. The median value $\tilde{R}^*$ for response modification factor R^* is assumed to be 7.0 and 3.0 for shear wall and flat-plate structures, respectively. For both structures, β_R is taken as 0.4. Finally, the median soil factor $\tilde{S}^*$ is taken to be the same as the design value, which depends on the soil type. β_S is assumed to be 0.3 for all soil conditions.

From Eq. 4.2 and the properties of the log-normal variable, the median of the total seismic base shear $\tilde{Q}^*_{ET}$ is

$$\tilde{Q}^*_{ET} = \frac{1.2\ \tilde{S}^*\tilde{W}^*}{\tilde{R}^*(\tilde{T}^*)^{2/3}}\ \tilde{A}^* \tag{4.4}$$

For each shear wall, the median seismic base shear, $\tilde{Q}^*_E$, is equal to one-half $\tilde{Q}^*_{ET}$. For soil types S_1, S_2 and S_3, $\tilde{Q}^*_E$ is 20.2 kips, 24.2 kips and 30.3 kips, respectively. For a typical frame of the flat-plate structure, the median seismic base shear Q^*_E is 7.0 kips, 8.4 kips and 10.6 kips respectively for S_1, S_2 and S_3 soil conditions. Furthermore, β_{QE} and β_{QET} are the same and, under the assumed independence of the random variables involved, can be determined as follows

$$\beta_{QE} = \beta_{QET} = [\beta_S^2 + \beta_W^2 + \beta_R^2 + (\frac{2}{3})^2\beta_T^2 + \beta_A^2]^{1/2} \tag{4.5}$$

From the data described above, β_{QE} is determined to be 0.80.

SECTION 5
RELIABILITY ANALYSIS

The limit state probability is used as a measure of the structural integrity. The limit state probability under earthquake load $P_{f,E}$ can be defined as:

$$P_{f,E} = P_r(\frac{Q_R^*}{Q_E^*} \leq 1) \tag{5.1}$$

Since Q_R^* and Q_E^* are log-normally distributed, Eq. 5.1 becomes

$$P_{f,E} = \Phi[\frac{-\ \ell n(\tilde{Q}_R^*/\tilde{Q}_E^*)}{(\beta_{QR}^2 + \beta_{QE}^2)^{1/2}}] \tag{5.2}$$

where $\Phi[\cdot]$ is the standardized normal distribution function. Similarly, the limit state probability under wind load $P_{f,W}$ is

$$P_{f,W} = \Phi[\frac{-\ \ell n(\tilde{Q}_R^*/\tilde{Q}_W^*)}{(\beta_{QR}^2 + \beta_{QW}^2)^{1/2}}] \tag{5.3}$$

Furthermore, disregarding the joint occurrence probability of earthquake and severe wind, the total limit state probability P_f is approximated by

$$P_f = P_{f,E} + P_{f,W} \tag{5.4}$$

The annual limit state probabilities values for shear wall and flat plate structures are summarized in Tables 5-I and 5-II, respectively. These limit state probability values are extremely small and must be interpreted as notional. They are meaningful only for comparison purposes. Under these circumstances, we may wish to utilize the safety index β for the comparison purposes.

$$P_{f,E}\ or\ P_{f,W} = 1 - \Phi(\beta) \tag{5.5}$$

The ranges of the safety index β corresponding to $P_{f,E}$ and $P_{f,W}$ are also indicated in these tables.

Table 5-I **Annual Limit State Probability (Shear-Wall Structure)**

Case	$P_{f,E}$	$P_{f,W}$	P_f
1	8.2×10^{-5}	3.9×10^{-12}	8.2×10^{-5}
2	7.5×10^{-5}	1.4×10^{-13}	7.5×10^{-5}
3	1.0×10^{-4}	5.0×10^{-15}	1.0×10^{-4}
4	1.1×10^{-4}	1.1×10^{-11}	1.1×10^{-4}
5	2.6×10^{-4}	1.1×10^{-11}	2.6×10^{-4}
6	7.1×10^{-4}	1.1×10^{-11}	7.1×10^{-4}
7		1.1×10^{-11}	
β	$3.2 \sim 3.8$	$6.7 \sim 7.8$	

Table 5-II Annual Limit State Probability (Flat -Plate Structure)

Case	$P_{f,E}$	$P_{f,W}$	P_f
1	1.64×10^{-3}	1.89×10^{-10}	1.64×10^{-3}
2	2.56×10^{-3}	7.01×10^{-11}	2.56×10^{-3}
3	4.80×10^{-3}	3.23×10^{-11}	4.80×10^{-3}
4	1.81×10^{-3}	2.69×10^{-10}	1.81×10^{-3}
5	3.57×10^{-3}	2.69×10^{-10}	3.57×10^{-3}
6	7.98×10^{-3}	2.69×10^{-10}	7.98×10^{-3}
7		2.69×10^{-10}	2.69×10^{-10}
β	$2.4 \sim 3.0$	$6.2 \sim 6.6$	

SECTION 6
CONCLUSIONS

This study presents the design and reliability assessment of low-rise buildings which are designed according to appropriate provisions of ANSI A58.1-1982 and ACI 318-83. The low-rise buildings considered in this paper are a shear-wall office building and a flat-plate apartment building located in New York City. Code specified wind and earthquake loads are considered for design of these buildings. For the reliability analysis, hazard curves due to wind or earthquake are established, probabilistic structural response is evaluated, limit state is defined and annual limit state probabilities are estimated. This work represents a preliminary study to demonstrate how knowledge of different scientific and engineering disciplines can be utilized to assess the actual integrity of structures under natural hazards.

The limit state probability values summarized in Tables 5-I and 5-II can be used primarily for comparative purposes. On the basis of the analytical formulation and data used in this study, the following conclusions are drawn.

1. Seismic hazard appears to be more serious than the hazard imposed by wind, even when a zone 2 design is implemented. Thus, a low-rise structure designed for wind loading without consideration for earthquake loading may require further strengthening for horizontal seismic force.

2. The seismic hazard curve approaches zero very slowly. Consequently, the limit state probability due to earthquake is rather insensitive to changes in the structural capacity. Thus, modeling of the seismic hazard needs special attention. Also, the seismic hazard curve used in the present paper expresses the seismic input only in terms of PGA. This is obviously not an adequate indicator of the seismic input.

3. The annual limit state probabilities due to wind are quite small as shown in Tables 5-I and 5-II. Thus, if the design is to be made for an ultimate limit state such as collapse of the structure, the load and resistance factors specified in ACI 318-83 associated with the load combination involving wind may be reduced for low-rise RC buildings.

The above conclusions obviously depend on the accuracy and credibility of the various assumptions made in the present study. Some of the factors that influence the probability values are delineated below.

(a) The ultimate limit state considered in this study may not be the most appropriate for comparison between the extent of seismic and wind hazard. Limit states that describe less severe structural damage but more functionally significant states of building response may need to be considered.

(b) The uncertainty associated with wind and seismic hazard curves will have a significant impact on the limit state probability. Thus, such uncertainty should be included in the analysis.

(c) All random variables are assumed to be log-normal. This may or may not be an appropriate assumption. A more rigorous analytical study is recommended in this connection.

(d) Equations 4.1 and 4.2, which are primarily devised for design, are used for estimating actual forces that will act on a building. Hence, both equations may oversimplify reality. Particularly, Eq. 4.2 involves rather bold simplification of the effects of the nonlinearity, soil properties, and dynamic characteristics of the building. These problems must be investigated by experts to provide simple yet scientifically sound solutions for the purpose of more accurate reliability and risk assessment.

(e) Details of local conditions such as interaction of the building with others in the vicinity are disregarded with respect to wind pressure distributions. Similarly, in dealing with the seismic effect, local geological and topographical peculiarities are not considered. The effect of soil conditions is grossly represented by S_1, S_2 and S_3. The dynamic interaction of the building with others in the vicinity is again not considered. Whether or not such detail should be accounted for in a study such as this depends on its purpose; for example, if the study is to be used for the overall risk assessment of a stock of shear-wall type buildings in a city area, such detail may not have to be addressed, indeed may be impossible to address.

(f) Building frames are assumed not to provide lateral resistance. This may be a conservative assumption. Also, the effects of possible torsional vibration of the building may be considerable. These problems must be investigated.

SECTION 7
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APPENDIX A

DESIGN OF RC FRAMING STRUCTURE

WITH RC SHEAR WALLS

APPENDIX A : RC FRAME STRUCTURE WITH RC SHEAR WALLS

A-1 Design Model

As shown in Fig. A-1, this design model consists of a five-story building in which the vertical elements of the laterally resistive system for wind and earthquake loading in each direction are two reinforced concrete shear walls . The structure of this building is a reinforced concrete frame structure with RC shear walls.

The materials used in this building are as follows:

Concrete : fc'= 3,000 psi

Reinforcement : fy = 40,000 psi (for #3 and #4), fy = 60,000 psi (for #5 or bigger)

The following discussion is limited to the design of two shear walls in the north-south direction.

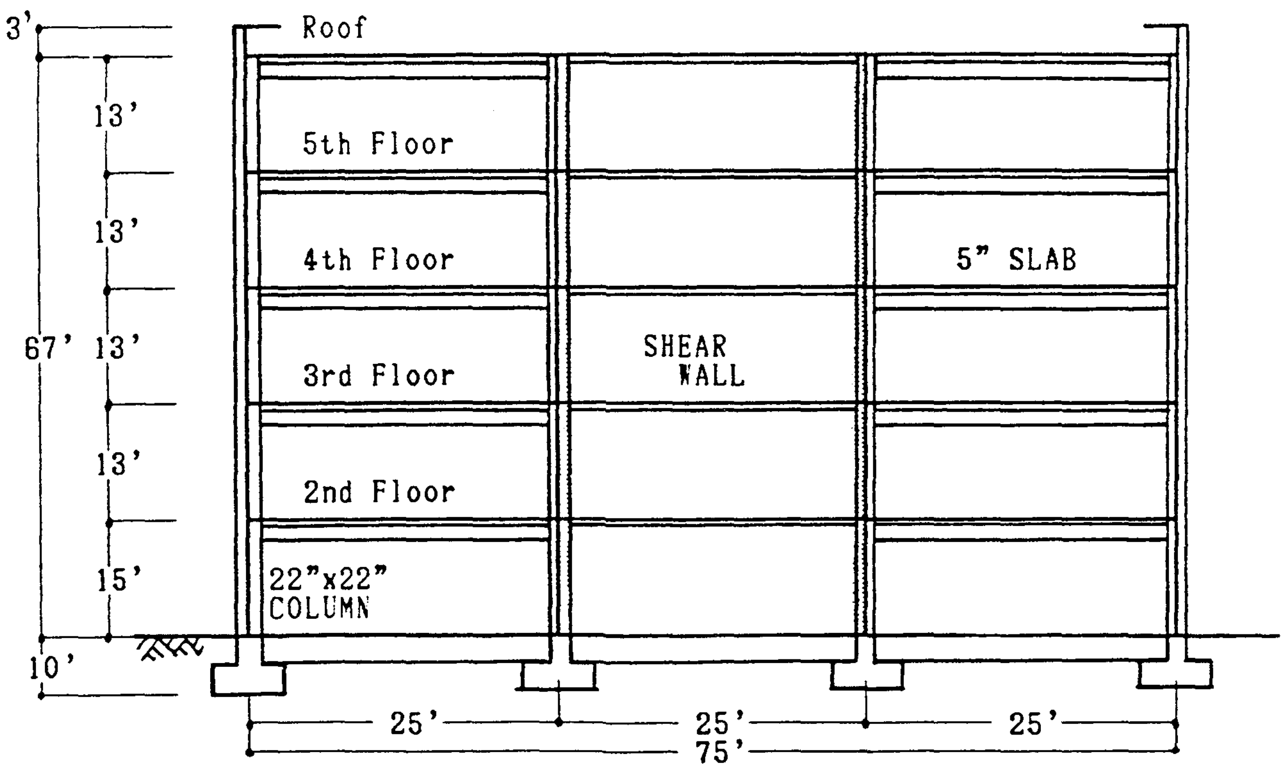


Fig. A-1a Cross-Section of Shear Wall

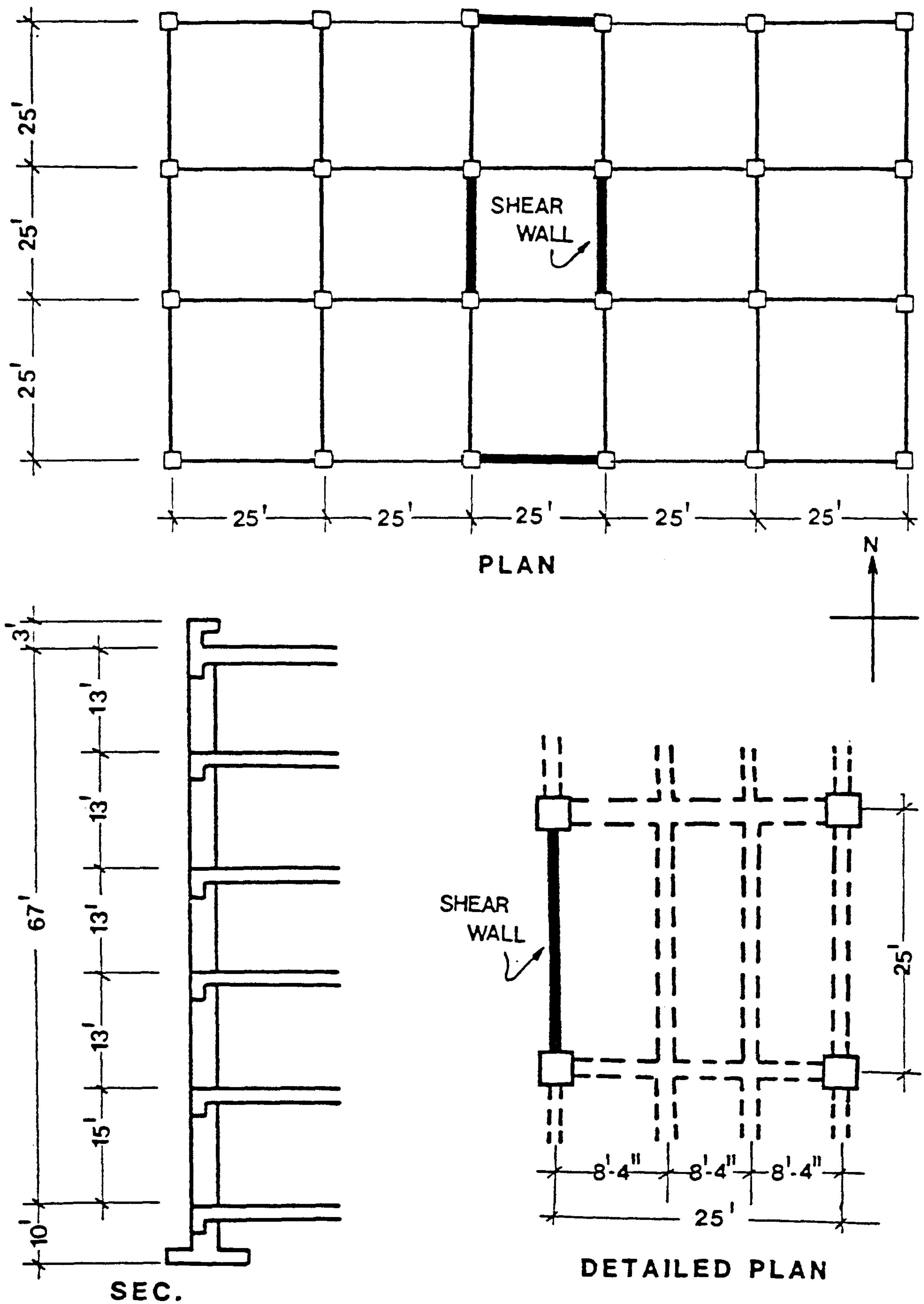


Fig. A-1 Plan and Cross-Section of Building

A-2 Loading Condition

The loads acting on the design model in Section A-1 are herein summarized, assuming a site of New York City.

a) Dead Load

* Roof :
  5" slab and 1" finish   63 + 12   = 75 psf

* 2nd through 5th Floors:
  5" slab and 1.5" finish   63 + 18   = 81 psf

* Main Beam : Assuming 16" by 27"
  16" x (27" - 5") x 155 pcf /144   = 379 plf

* Sub Beam : Assuming 12" by 23"
  12" x (23" - 5") x 155 pcf /144   = 233 plf

* Column :
  3rd through 5th Floors   20" x 20" x 155 pcf / 144   = 431 plf
  1st and 2nd Floors   22" x 22" x 155 pcf / 144   = 521 plf

* Exterior Walls :   15 psf

* Shear Walls :
  6" thickness and 3" finish   76 + 36   = 112 psf
  5" thickness and 3" finish   64 + 36   = 100 psf

b) Live load

Roof :   20 psf
2nd through 5th Floors: 50 psf (see ANSI A58.1)

c) Wind Load

Assuming a basic wind speed of 80 mph (New York City), the design wind pressure p can be calculated based on ANSI A58.1-1982[1].

Location : New York City
Basic wind pressure q_z : $q_z = 0.00256K_z(IV)^2$ (psf)

where
$I = 1.05$ (Category I at hurricane ocean line)
$V = 80$ mph
K_z: use Exposure B

0.37 for 0 to 15 ft height
0.50 for 15 to 30 ft height
0.63 for 30 to 50 ft height
0.73 for 50 to 70 ft height

Table A-1 Basic Wind Pressure

Height (ft)	q_z (psf)
0 to 15	6.683
15 to 30	9.032
30 to 50	11.38
50 to 70	13.19

The design wind pressure p is given by following formula and is shown in Table A-2.

$$p = q \times G_h \times C_p^* = q_z \times G_h \times C_{p(W)} - q_h \times G_h \times C_{p(L)}$$

where
$h = 70$ ft
$G_h = 1.36$ (Exposure B at 70 ft height)
$C_{p(Windward)} = 0.8$
$C_{p(Leeward)} = -0.5$

Table A-2 Design Wind Pressure

Height (ft)	$q_z \times G_h \times C_{p(W)}$	$q_h \times G_h \times C_{p(L)}$	p (psf)
0 to 15	7.271	-8.969	16.24
15 to 30	9.827	-8.969	18.80
30 to 50	12.381	-8.969	21.35
50 to 70	14.351	-8.969	23.32

d) Earthquake Load

Assuming seismic zone 2 (New York City), the total lateral base shear is calculated in this section based on ANSI A58.1-1982[1].

$$Q = ZIKCSW$$

where

Q = total lateral shear force at base (lbs)
Z = numerical coefficient due to zoning
K = numerical coefficient due to building system
I = occupancy importance factor
$C = 1/15\sqrt{T}$ but not more than 0.12
S = soil factor
W = total dead load (lbs)

Let the location = New York City (seismic zone 2)

$Z = 3/8$
$I = 1.0$ (Category I)
$K = 1.0$ (Building frame system with shear walls designed by ACI 318)
$T = 0.05\ hn / \sqrt{D}$ (sec.)

$hn = 77$ ft
$D = 75$ ft (in north-south direction)
$T = 0.05 \times 77/\sqrt{75} = 0.4446$ sec.

$C = 1/15\sqrt{0.4446} = 0.10$
$S = 1.0$ (Soil Profile Type S1 : Rock)
1.2 (" S2 : Stiff Clay)
1.5 (" S3 : Soft Clay)

But CS need not be greater than 0.14.

i) Under soil condition S1 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.10 \times W = 0.0375\ W$

ii) Under soil condition S2 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.12 \times W = 0.0450\ W$

iii) Under soil condition S3 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.14 \times W = 0.0525\ W$

e) Load Combination for Design

Load combinations are determined based on ACI 318-83[8].

Case 1 : $U = 1.4D + 1.7L$	
Case 2 : $U = 0.75\ (1.4D + 1.7L + 1.7W)$	for Wind
Case 3 : $U = 0.9D + 1.3W$	for Wind
Case 4 : $U = 0.75\ (1.4D + 1.7L + 1.87E)$	for Earthquake
Case 5 : $U = 0.9D + 1.43E$	for Earthquake

where

U = Required strength for design
D = Dead load
L = Live load
W = Wind load
E = Earthquake load

A-3 Design for Wind

The design of the building for wind load is based on ACI 318-83[8]. The design wind pressure p is shown in Fig. A-2, and the lateral load for each floor H_1, H_2, H_3, H_4, H_5 (Fig. A-2) can be calculated as follows.

H_1 = 23.32 x (125x9.5) = 27693 lbs = 27.63 kips

H_2 = 23.32 x (125x10.5) + 21.35 x (125x2.5) = 37279 lbs = 37.28 kips

H_3 = 21.35 x (125x13.0) = 34649 lbs = 34.69 kips

H_4 = 21.35 x (125x4.5) + 18.80 x (125x8.5) = 31984 lbs = 31.98 kips

H_5 = 18.80 x (125x6.5) + 16.24 x (125x7.5) = 30500 lbs = 30.50 kips

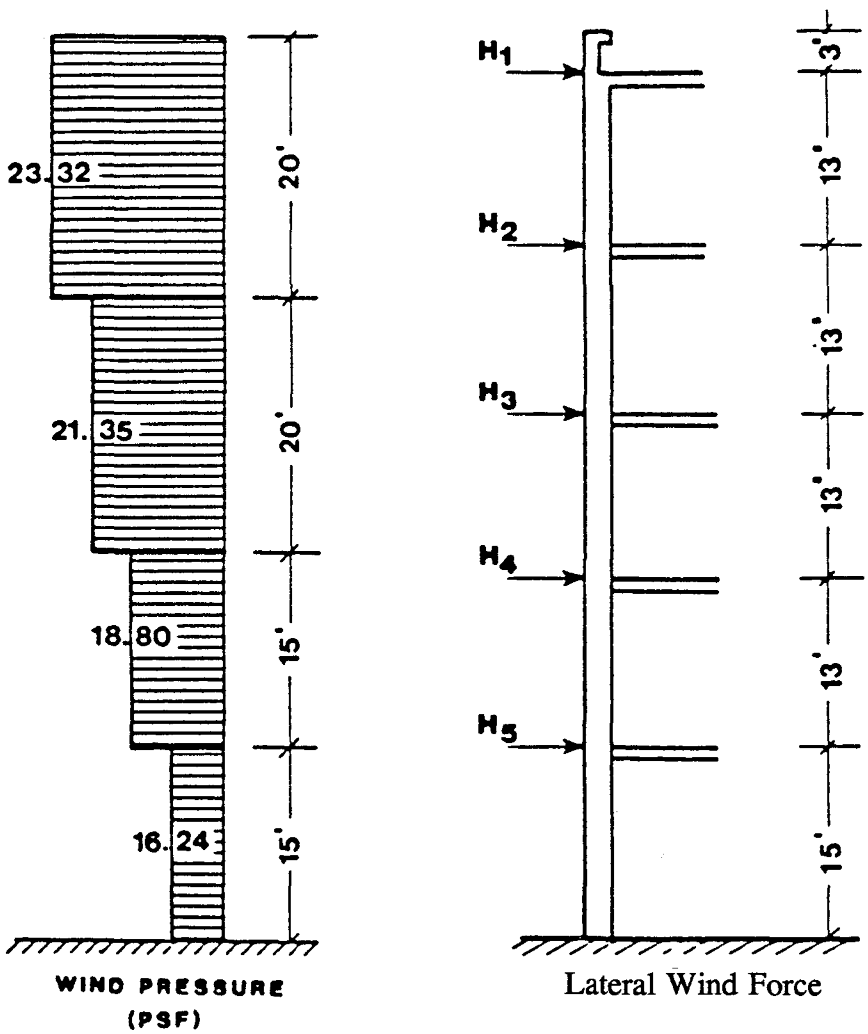


Fig. A-2 Wind Forces

a) Design of Shear Walls

Assuming 100% of the lateral load is to be resisted by the two shear walls, the shear walls are designed based on ACI 318-83[8]. A shear force diagram is shown in Fig. A-3.

* Required Shear Strength : Vu

$Vu = 1.3 \times 81.05 = 105.4$ kips (Using load combination case 3)

* Design Criteria of Shear Wall :

$Vu \leq \phi\, Vn$ (ACI 318-83, Eq.11-1)

where

$Vn = Vc + Vs$ (ACI 318-83, Eq.11-2)

$\phi = 0.85$ (Capacity reduction factor)

$Vc = 2 \sqrt{fc'}\; h\, d$ (ACI 318-83, Eq.11-3)

$Vs = Av\; fy\; d \,/\, s_2$ (ACI 318-83, Eq.11-17)

where

- fc' = 3,000 psi (concrete strength)
- h = thickness of shear wall (in.)
- d = 0.8 lw (lw : length of shear wall, in.)
- Av = area of horizontal shear reinforcement within vertical distance s_2 (sq.in.)
- s_2 = vertical distance between horizontal reinforcement (in.)
- fy = 40,000 psi (yield strength of reinforcement)

* 1st Floor Shear Wall

Assuming the axial force caused by the overturning moment due to lateral force is resisted by the end columns, the shear walls are designed only for shear force.

Assuming h = 5",

$Vc = 2 \times \sqrt{3000} \times 5 \times (25'\text{x}12 - 22") \times 0.8 = 121.8$ kips

Provide #3 rebars @ 8 in. for horizontal reinforcement.

$Vs = 0.11 \times 40000 \times 222.4 \,/\, 8 = 122.3$ kips

$\rho_h = 0.00275 \;>\; \text{min. req. } \rho_h = 0.0025$

$\phi\, Vn = \phi\,(Vc + Vs) = 0.85 \times (121.8 + 122.3) = 207.5 \text{ kips} \;>\; Vu = 105.4 \text{ kips}$

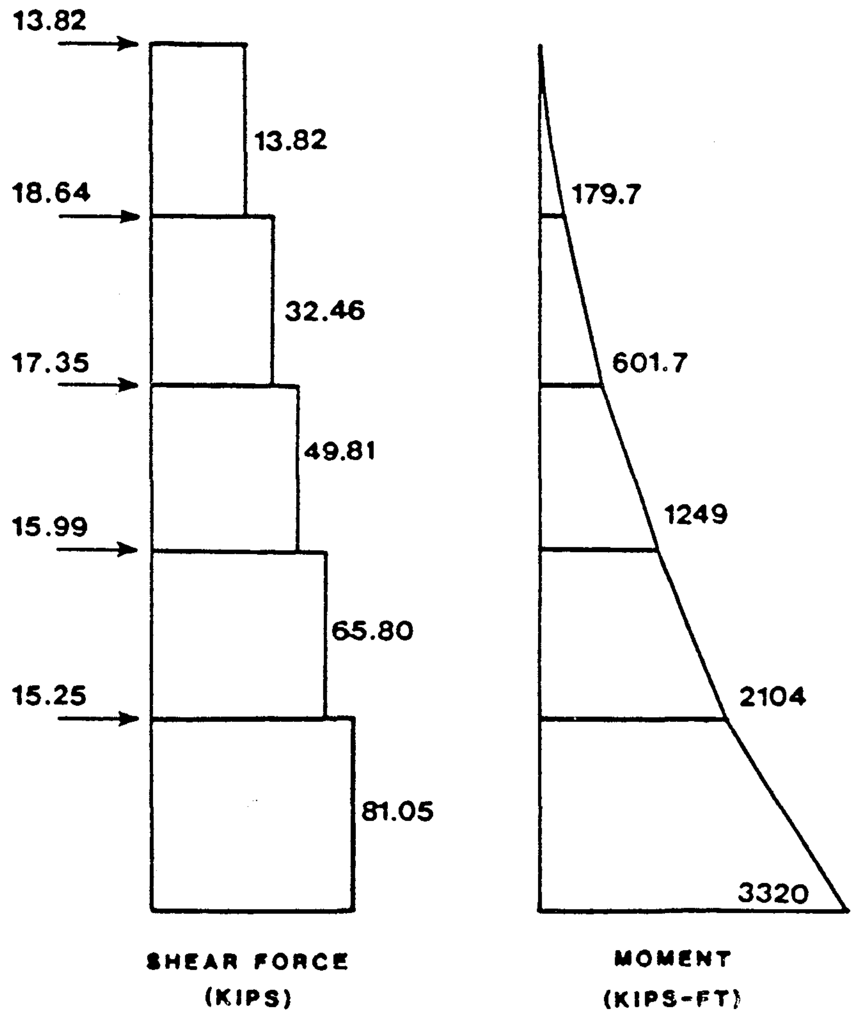


Fig. A-3 Shear Force and Moment Diagrams

b) Design of Shear Wall End Column

* Calculation of Dead Load of Column

Tributary area of column = 25' x 25' = 625 sq.ft.

5th Floor

Roof	75 x 625	= 46.9 kips
Main beam	379 x (25' + 25'/2)	= 14.2 kips
Sub beam	233 x 25' x 2	= 11.7 kips
Shear wall	100 x 11.6' x 13'	= 15.1 kips
Column	431 x 13'	= 5.6 kips
Subtotal		93.5 kips

3rd and 4th Floors

Floor	81 x 625	= 50.6 kips
Beam, Wall and Column		46.6 kips
Subtotal		97.2 kips

2nd Floor

Floor	81 x 625	= 50.6 kips
Beam	14.2 + 11.7	= 25.9 kips
Shear Wall		15.1 kips
Column	521 x 13'	= 6.8 kips
Subtotal		98.4 kips

1st Floor

Floor	81 x 625	= 50.6 kips
Beam	14.2 + 11.7	= 25.9 kips
Shear wall	100 x 11.6' x 15'	= 17.4 kips
Column	521 x 15'	= 7.8 kips
Subtotal		101.7 kips

* Calculation of Live Load of Column

5th Floor

Roof	20 x 625	= 12.5 kips

1st through 4th Floors

Floor	50 x 625	= 31.3 kips

* Load Combinations

Axial load due to overturning moment P_W at 1st floor :

$P_W = 3320.4 / 23.2 = 143.1$ kips

Axial load due to dead load P_D at 1st floor :

$P_D = 93.5 + 97.2 \times 2 + 98.4 + 101.7 = 488.0$ kips

Axial load due to live load P_L at 1st floor :

$P_L = 12.5 + 31.3 \times 4 = 137.7$ kips

Required Strength of Axial Load Pu :

Case 1 : $Pu = 1.4 \times 488.0 + 1.7 \times 137.7 = 917.3$ kips

Case 2 : $Pu = 0.75\ (1.4\times488.0 + 1.7\times137.7 \pm 1.7\times143.1) = 870.4$ or 505.5 kips

Case 3 : $Pu = 0.9 \times 488.0 \pm 1.3 \times 143.1 = 625.2$ or 253.2 kips

Therefore, design load Pu shall be determined by load combination case 1 :
Pu = 917.3 kips (for Dead and Live load).

* Design of Column (at 1st floor shear wall end)

$Pu \leq \phi\ Pn$

where $\phi = 0.70$

$\phi\ Pn = 0.80\ \phi\ [\ 0.85\ fc'\ Ac + fy\ Ast\ ]$ (ACI 318-83, Eq.10-2)

Ac : Gross sectional area of concrete column (sq.-in.)

Ast : Total area of longitudinal reinforcement (sq.-in.)

Use fc' = 3000 psi, and assume a 22 in. by 22 in. square columns with 10 #8 rebars for longitudinal reinforcement.

$\phi\ Pn = 0.80 \times 0.70 \times [\ 0.85\times3\times22\times22 + 60\times0.79\times10\ ] = 956.6$ kips $>$ $Pu = 917.3$ kips

A-4 Design for Earthquake (Seismic Zone 2)

a) Calculation of Total Dead Load

Roof :		
Roof	75 x 125 x 75	= 703.1 kips
Main Beam	379 x (125 x 4 + 75 x 6)	= 360.1 kips
Sub Beam	233 x (75 x 2 x 5)	= 174.8 kips
Column	431 x 6.5 x 24	= 67.2 kips
Exterior Walls	15 x 6.5 x 350	= 34.1 kips
Shear Walls	100 x 6.5 x 100	= 65.0 kips
Subtotal		1404.3 kips
4th and 5th Floors :		
Floor	81 x 125 x 75	= 759.4 kips
Main Beam	379 x (125 x 4 + 75 x 6)	= 360.1 kips
Sub Beam	233 x (75 x 2 x 5)	= 174.8 kips
Column	431 x 13.0 x 24	= 134.5 kips
Exterior Walls	15 x 13.0 x 350	= 68.3 kips
Shear Walls	100 x 13.0 x 100	= 130.0 kips
Subtotal		1627.1 kips x 2 = 3254.2 kips
3rd Floor :		
Floor	81 x 125 x 75	= 759.4 kips
Beam	360.1 + 174.8	= 534.9 kips
Column	(431 + 521) x 6.5 x 24	= 148.5 kips
Exterior Walls		68.3 kips
Shear Walls	(100 + 112) x 6.5 x 100	= 137.8 kips
Subtotal		1648.9 kips
2nd Floor :		
Floor	81 x 125 x 75	= 759.4 kips
Beam	360.1 + 174.8	= 534.9 kips
Column	521 x 14.0 x 24	= 175.1 kips
Exterior Walls	15 x 14.0 x 350	= 73.5 kips
Shear Walls	112 x 14.0 x 100	= 156.8 kips
Subtotal		1699.7 kips
1st Floor :		
Column	521 x 7.5 x 24	= 93.8 kips
Exterior Walls	15 x 7.5 x 350	= 39.4 kips
Shear Walls	112 x 7.5 x 100	= 84.0 kips
Subtotal		217.2 kips
Total Dead Load :		W = 8224.3 kips

b) Calculation of Base Shear Q

Under soil condition S1 : Q = 0.0375W = 0.0375 x 8224.3 = 308.4 kips
Under soil condition S2 : Q = 0.0450W = 0.0450 x 8224.3 = 370.1 kips
Under soil condition S3 : Q = 0.0525W = 0.0525 x 8224.3 = 431.8 kips

c) Distribution of Earthquake Forces

Assuming Ft = zero, since T=0.4446 sec., and the bottom of the base is 10 feet below the ground floor, the lateral force at each floor, Fx, can be calculated based on ANSI A58.1-1982[1]. The earthquake loads applied to the shear walls are shown in Tables A-3, A-4 and A-5 and in Fig. A-4.

$$F_x = Q\, W_x\, h_x \,/\, (\Sigma W_i\, h_i)$$

Table A-3 Lateral Force Fx : Soil Condition S1 (Q=308.4 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	1404.3	77	108131	82.8
5th Floor	1627.1	64	104134	79.8
4th Floor	1627.1	51	82982	63.6
3rd Floor	1648.9	38	62658	48.0
2nd Floor	1699.7	25	42493	32.6
1st Floor	217.2	10	2172	1.7
$\Sigma w_i h_i$			402570	

Table A-4 Lateral Force Fx : Soil Condition S2 (Q=370.1 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	1404.3	77	108131	99.4
5th Floor	1627.1	64	104134	95.7
4th Floor	1627.1	51	82982	76.3
3rd Floor	1648.9	38	62658	57.6
2nd Floor	1699.7	25	42493	39.1
1st Floor	217.2	10	2172	2.0
$\Sigma w_i h_i$			402570	

Table A-5 Lateral Force Fx : Soil Condition S3 (Q=431.8 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	1404.3	77	108131	116.0
5th Floor	1627.1	64	104134	111.7
4th Floor	1627.1	51	82982	89.0
3rd Floor	1648.9	38	62658	67.2
2nd Floor	1699.7	25	42493	45.6
1st Floor	217.2	10	2172	2.3
$\Sigma w_i h_i$			402570	

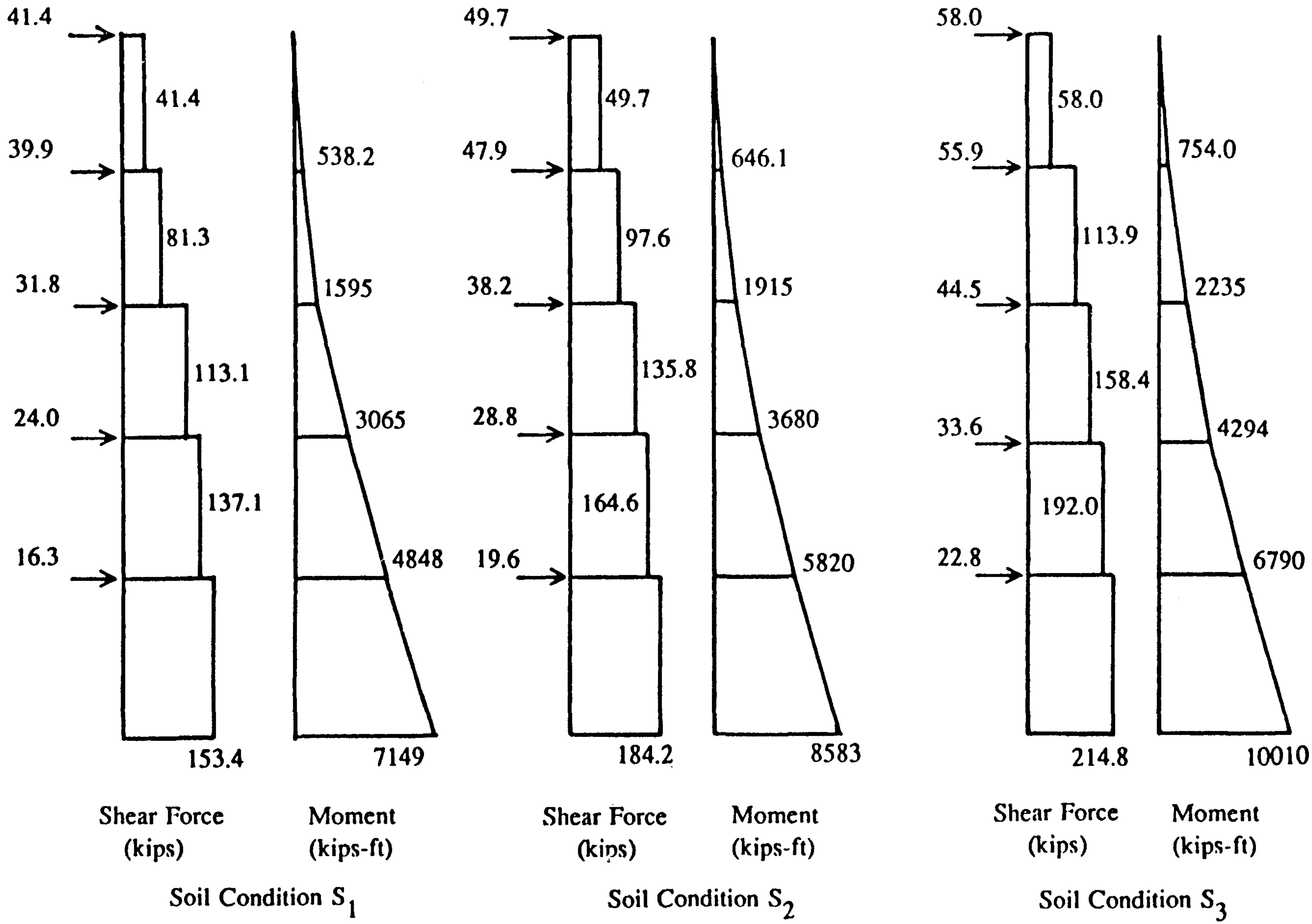


Fig. A-4 Seismic Forces (Seismic Zone 2)

d) Design of shear walls at 1st floor

d-1) Under Soil Condition S1 :

* Required Shear Strength Vu

$$Vu = 1.43 \times 153.4 = 219.4 \text{ kips} \quad \text{(using load combination case 5)}$$

* Shear wall design

Assuming the axial force caused by the overturning moment due to lateral force is resisted by the end columns, the shear walls are designed only for shear force.

Assuming h = 5",

$$Vc = 2 \times \sqrt{3000} \times 5 \times (25' \times 12 - 22") \times 0.8 = 121.8 \text{ kips}$$

Provide #3 rebars @ 7 in. for horizontal reinforcement.

$$Vs = 0.11 \times 40000 \times 222.4 / 7 = 139.8 \text{ kips}$$

$$\phi\, Vn = \phi\, (Vc + Vs) = 0.85 \times (121.8 + 139.8) = 222.4 \text{ kips} > Vu = 219.4 \text{ kips}$$

d-2) Under Soil Condition S2

* Required shear strength Vu

$$Vu = 1.43 \times 184.2 = 263.4 \text{ kips} \quad \text{(using load combination case 5)}$$

* Shear wall design

Assuming the axial force caused by overturning moment due to lateral force is resistive by end columns, shear walls are designed only for shear force.

Assuming h = 5",

$$Vc = 2 \times \sqrt{3000} \times 5 \times (25' \times 12 - 22") \times 0.8 = 121.8 \text{ kips}$$

Provide #3 rebars @ 5 in. for horizontal reinforcement.

$$Vs = 0.11 \times 40000 \times 222.4 / 5 = 195.7 \text{ kips}$$

$$\phi\, Vn = \phi\, (Vc + Vs) = 0.85 \times (121.8 + 195.7) = 269.9 \text{ kips} > Vu = 263.4 \text{ kips}$$

d-3) Under Soil Condition S3 :

* Required shear strength Vu

$$Vu = 1.43 \times 214.8 = 307.2 \text{ kips} \quad \text{(using load combination case 5)}$$

* Shear wall design

Assuming the axial force caused by the overturning moment due to lateral force is resisted by the end columns, the shear walls are designed only for shear force.

Assuming h = 5",

$$Vc = 2 \times \sqrt{3000} \times 5 \times (25' \times 12 - 22") \times 0.8 = 121.8 \text{ kips}$$

Provide #4 rebars @ 7 in. for horizontal reinforcement.

$$Vs = 0.20 \times 40000 \times 222.4 / 7 = 254.2 \text{ kips}$$

$$\phi\, Vn = \phi\, (Vc + Vs) = 0.85 \times (121.8 + 254.2) = 319.6 \text{ kips} > Vu = 307.2 \text{ kips}$$

e) Design of Shear Wall End Column at 1st Floor

e-1) Under Soil Condition S1 :

* Load Combinations

Axial load due to overturning moment P_E at 1st floor :

P_E = 7148.7 / 23.2 = 308.1 kips
P_D = 488.0 kips
P_L = 137.7 kips

Required strength of axial load Pu :

Case 1 : Pu = 1.4 x 488.0 + 1.7 x 137.7 = 917.3 kips
Case 4 : Pu = 0.75 (1.4x488.0 + 1.7x137.7 ± 1.87x308.1) = 1120.1 or 255.9 kips
Case 5 : Pu = 0.9 x 488.0 ± 1.43 x 308.1 = 879.8 or -1.4 kips

Therefore, design load Pu shall be determined by load combination case 4 :
Pu = 1120.1 kips (for positive P_E).

* Design of Column (at 1st floor shear wall end)

Use fc' = 3000 psi, and assume 22 in. by 22 in. square columns with 14 # 9 rebars for longitudinal reinforcement.

$$\phi\, Pn = 0.80 \times 0.70 \times [0.85 \times 3 \times 22 \times 22 + 60 \times 1.0 \times 14] = 1161.6 \text{ kips} > Pu = 1120.1 \text{ kips}$$

e-2) Under Soil Condition S2 :

* Load Combination

Axial load due to overturning moment P_E at 1st floor :

P_E = 8583.1 / 23.2 = 370.0 kips
P_D = 488.0 kips
P_L = 137.7 kips

Required strength of axial load Pu :

Case 1 : Pu = 1.4 x 488.0 + 1.7 x 137.7 = 917.3 kips
Case 4 : Pu = 0.75 (1.4x488.0 + 1.7x137.7 ± 1.87x370.0) = 1206.9 or 169.1 kips
Case 5 : Pu = 0.9 x 488.0 ± 1.43 x 370.0 = 968.3 or -89.9 kips

Therefore, design load Pu shall be determined by load combination case 4 :
Pu = 1206.9 kips (for positive P_E).

* Design of Column (at 1st floor shear wall end)

Use fc' = 3000 psi, and assume 22 in. by 22 in. square columns with 16 # 9 rebars for longitudinal reinforcement.

ϕ Pn = 0.80 x 0.70 x [0.85x3x22x22 + 60x1.0x16] = 1228.8 kips > Pu = 1206.9 kips

e-3) Under Soil Condition S3 :

* Load Combination

Axial load due to overturning moment P_E at 1st floor :

P_E = 10011.9 / 23.2 = 451.5 kips
P_D = 488.0 kips
P_L = 137.7 kips

Required strength of axial load Pu :

Case 1 : Pu = 1.4 x 488.0 + 1.7 x 137.7 = 917.3 kips
Case 4 : Pu = 0.75 (1.4x488.0 + 1.7x137.7 ± 1.87x451.5) = 1321.2 or 54.7 kips
Case 5 : Pu = 0.9 x 488.0 ± 1.43 x 451.5 = 1084.8 or -206.4 kips

Therefore, design load Pu shall be determined by load combination case 4 :
Pu = 1321.2 kips (for positive P_E).

* Design of Column (at 1st floor shear wall end)

Use fc' = 3000 psi, and assume 22 in. by 22 in. square columns with 20 # 9 rebars for longitudinal reinforcement.

ϕ Pn = 0.80 x 0.70 x [0.85x3x22x22 + 60x1.0x20] = 1363.2 kips > Pu = 1321.2 kips

A-5 Comparison of Wind and Earthquake Designs

A summary of the required shear force, factored shear strength and nominal strength of the shear wall at the first floor is shown in Table A-6. A summary of the columns of the shear wall end at the 1st floor is also shown in Table A-7. In these tables, 'W' indicates a building designed for wind load only, and 'E-1', 'E-2' and 'E-3' indicate buildings designed for earthquake load under soil conditions S1, S2 and S3, respectively, in addition to wind load.

With regard to this design model, the shear forces due to earthquake loads based on ANSI A58.1 are greater than that due to wind load (see Table A-6). Since the wind load is relatively small, the design load Pu of the end columns under wind loading (W) is determined by load combination Case 1 (Dead + Live loads). However, the design loads Pu under earthquake loading (E-1, E-2 and E-3) are determined by load combination case 4 (Dead + Live + Earthquake loads).

Table A-6 Summary of Shear Walls at 1st Floor
(Seismic Zone 2)

	W	E-1(S1)	E-2(S2)	E-3(S3)
Q (kips)	81.05	153.4	184.2	214.8
Vu (kips)	105.4	219.4	263.4	307.2
ϕ Vn (kips)	207.5*	222.4	269.9	319.6
Section	h = 5"	h = 5"	h = 5"	h = 5"
	#3 @ 8 in.	#3 @ 7 in.	#3 @ 5 in.	#4 @ 7 in.
	(horiz.)	(horiz.)	(horiz.)	(horiz.)

Notes:

Q : Shear force of each shear wall due to lateral load

Vu : Required factored strength based on ACI 318

ϕ Vn : Reduced nominal shear strength based on ACI 318

$$\phi \text{ Vn} = \phi \text{ (Vc + Vs)}$$

* This value is determined by minimum reinforcement in ACI 318.

Table A-7 Summary of Columns at 1st Floor
(Seismic Zone 2)

	W	E-1(S1)	E-2(S2)	E-3(S3)
Pu (kips)	917.3	1120.1	1206.9	1321.2
ϕ Pn (kips)	956.6	1161.6	1228.8	1363.2
Section	22" x 22"	22" x 22"	22" x 22"	22" x 22"
	10 - #8	14 - #9	16 - #9	20 - #9
	(longitud.)	(longitud.)	(longitud.)	(longitud.)

Notes:

Pu : Required factored strength based on ACI 318

ϕ Pn : Reduced nominal axial strength based on ACI 318

A-6 Design for Earthquake (Seismic Zone 1)

To obtain the general trend between wind and earthquake loads, the following cases are studied for reference.

* Let Seismic Zone (in ANSI A58.1) = 1 ; Z = 3/16
* Base Shear Q becomes one-half the values in Section 3-4.
  - Under soil condition S1 : Q = 0.01875W = 154.2 kips
  - Under soil condition S2 : Q = 0.02250W = 185.1 kips
  - Under soil condition S3 : Q = 0.02625W = 215.9 kips

A summary of the shear force and required factored shear strength and nominal strength of the shear wall at the first floor is indicated in Table A-8. In this table, 'W' indicates a building designed for wind load only, and 'E-1', 'E-2' and 'E-3' indicate buildings designed for earthquake load under soil conditions S1, S2 and S3, respectively, in addition to wind load.

With regard to this design model, design forces of shear wall due to "Seismic Zone 1" are still greater than those due to wind. However, the capacity of shear walls was determined by the minimum reinforcement requirement of ACI 318.

Table A-8 Summary of Shear Walls at 1st Floor
(Seismic Zone 1)

	W	E-1(S1)	E-2(S2)	E-3(S3)
Q (kips)	81.05	76.7	92.1	107.4
Vu (kips)	105.4	109.7	131.7	153.6
ϕ Vn (kips)	207.5*	207.5*	207.5*	207.5*
Section	h = 5"	h = 5"	h = 5"	h = 5"
	#3 @ 8 in. (horiz.)	#3 @ 8 in. (horiz.)	#3 @ 8 in. (horiz.)	#3 @ 8 in. (horiz.)

Notes:

Q : Shear force of each shear wall due to lateral load

Vu : Required factored strength based on ACI 318

ϕ Vn : Reduced nominal shear strength based on ACI 318

$$\phi \, Vn = \phi \, (Vc + Vs)$$

* This value is determined by the requirement of minimum reinforcement.

APPENDIX B

DESIGN OF RC SOLID FLAT PLATE STRUCTURE

APPENDIX B : RC SOLID FLAT PLATE STRUCTURE

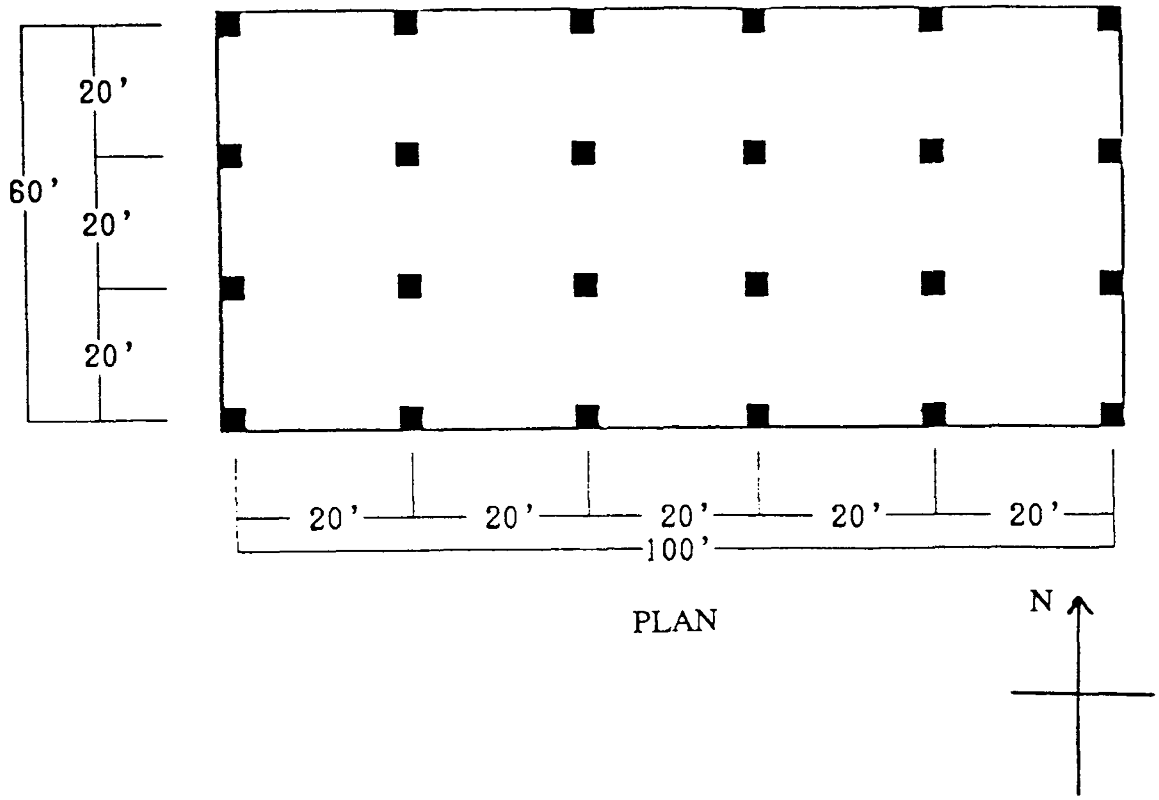
B-1 Design Model

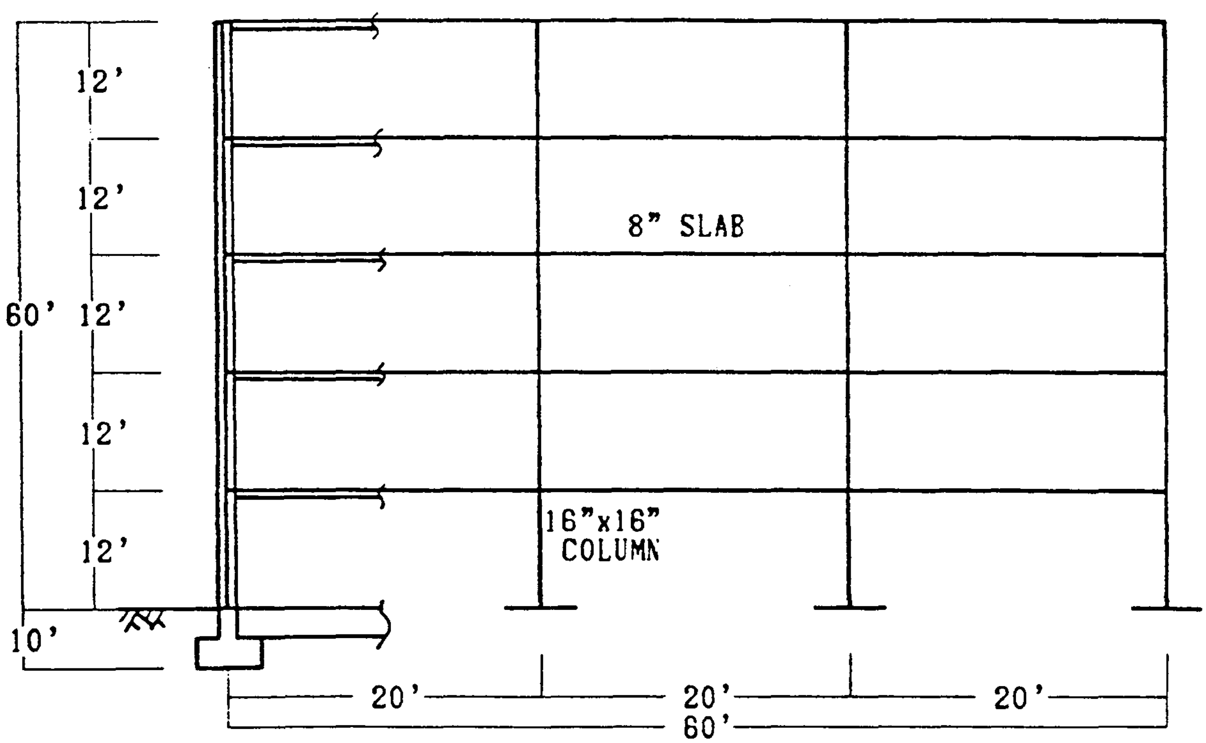
As shown in Fig. B-1, this design model is a five-story building which consists of two way solid flat plates and columns. This building is supposed to be an apartment house or a dormitory or small hotel.

The materials used in this building are as follows:

Concrete : fc'= 4,000 psi
Reinforcement : fy = 60,000 psi

The following discussion is limited to the design in the north-south direction.





SECTION

Fig. B-1 Plan and Cross-Section of Building

B-2 Loading Condition

Loads acting on the design model in Section B-1 are herein summarized, assuming a site of New York City.

a) Dead Load

* Roof through 2nd Floor :
  8" slab 100 psf

* Column :
  16" x 16" x 155 pcf / 144 = 276 plf

* Exterior Walls : 15 psf

b) Live load

Roof : 20 psf
Floor : 50 psf
where 40 psf (use Dwellings or Hotels in ANSI A58.1)
10 psf (other super-imposed live load, such as partitions)

c) Wind Load

Assuming a basic wind speed of 80 mph (New York City), design wind pressure p can be calculated based on ANSI A58.1-1982[1].

Location : New York City
Basic wind pressure q_z : $q_z = 0.00256K_z(IV)^2$ (psf)

where
I = 1.05 (Category I at Hurricane ocean line)
V = 80 mph
K_z: use Exposure B

0.37 for 0-15 ft height
0.50 for 15-30 ft height
0.63 for 30-50 ft height
0.68 for 50-60 ft height

Table B-1 Basic Wind Pressure

Height (ft)	q_z (psf)
0 to 15	6.683
15 to 30	9.032
30 to 50	11.38
50 to 60	12.28

Design wind pressure p is given by the following formula and is shown in Table B-2.

$$p = q \times G_h \times C_p^* = q_z \times G_h \times C_{p(W)} - q_h \times G_h \times C_{p(L)}$$

where

$h = 60$ ft

$G_h = 1.39$ (Exposure B at 60 ft height)

$C_{p(Windward)} = 0.8$

$C_{p(Leeward)} = -0.5$

Table B-2 Design Wind Pressure

Height (ft)	$q_z \times G_h \times C_{p(W)}$	$q_h \times G_h \times C_{p(L)}$	p (psf)
0 to 15	7.431	-8.535	15.97
15 to 30	10.04	-8.535	18.58
30 to 50	12.65	-8.535	21.19
50 to 60	13.66	-8.535	22.20

d) Earthquake Load

Assuming seismic zone 2 (New York City), the total lateral base shear is calculated in this section based on ANSI A58.1-1982[1].

$$Q = ZIKCSW$$

where

Q = total lateral shear force at base (lbs)
Z = numerical coefficient due to zoning
K = numerical coefficient due to building system
I = occupancy importance factor
$C = 1/15\sqrt{T}$ but not more than 0.12
S = soil factor
W = total dead load (lbs)

Let the location = New York City (seismic zone 2)

$Z = 3/8$
$I = 1.0$ (Category I)
$K = 1.0$ (Assumption)
$T = 0.05\ hn / \sqrt{D}$ (sec.)

$hn = 70$ ft
$D = 60$ ft (in north-south direction)
$T = 0.05 \times 70/\sqrt{60} = 0.4518$ sec.

$C = 1/15\sqrt{0.4518} = 0.10$
$S = 1.0$ (Soil Profile Type S1 : Rock)
1.2 (" S2 : Stiff Clay)
1.5 (" S3 : Soft Clay)

But CS need not be greater than 0.14.

i) Under soil condition S1 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.10 \times W = 0.0375\ W$

ii) Under soil condition S2 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.12 \times W = 0.0450\ W$

iii) Under soil condition S3 :
$Q = 3/8 \times 1.0 \times 1.0 \times 0.14 \times W = 0.0525\ W$

e) Load Combinations for Design

Load combinations are determined based on ACI 318-83[8].

Case 1 :	$U = 1.4D + 1.7L$	
Case 2 :	$U = 0.75\ (1.4D + 1.7L + 1.7W)$	for Wind
Case 3 :	$U = 0.9D + 1.3W$	for Wind
Case 4 :	$U = 0.75\ (1.4D + 1.7L + 1.87E)$	for Earthquake
Case 5 :	$U = 0.9D + 1.43E$	for Earthquake

where

U = Required strength for design
D = Dead load
L = Live load
W = Wind load
E = Earthquake load

B-3 Calculation of Lateral Loads

a) Wind Load

The wind load is calculated based on ANSI A58.1[1]. The design wind pressure p is shown in Fig. B-2, and the lateral load for each floor H_1, H_2, H_3, H_4, H_5 (Fig. B-2) can be calculated as follows.

H_1 = 22.20 x (100x6.0) = 13320 lbs = 13.32 kips

H_2 = 22.20 x (100x4.0) + 21.19 x (100x8.0) = 25832 lbs = 25.83 kips

H_3 = 21.19 x (100x12.0) = 25428 lbs = 25.43 kips

H_4 = 18.58 x (100x12.0) = 22296 lbs = 22.30 kips

H_5 = 18.58 x (100x3.0) + 15.97 x (100x9.0) = 19947 lbs = 19.95 kips

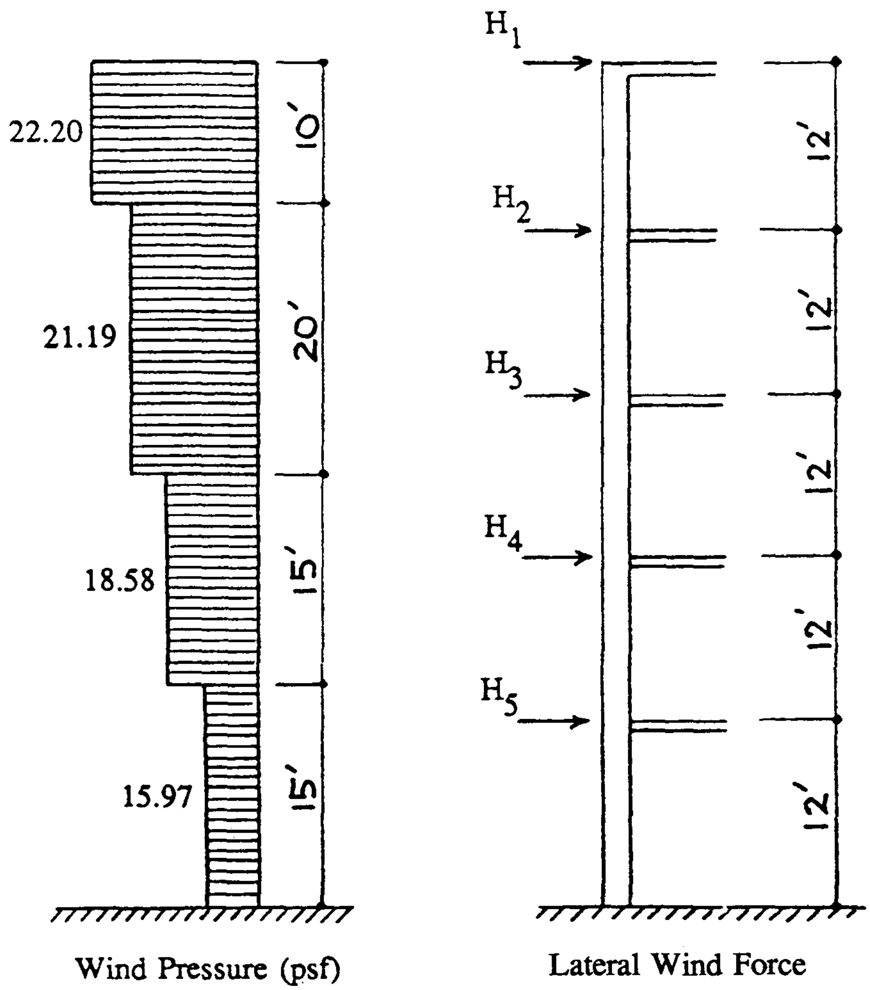


Fig. B-2 Wind Forces

b) Earthquake Load (Seismic Zone 2)

* Calculation of Total Dead Load

Roof :		
Roof	100 x 100 x 60	= 600.0 kips
Column	276 x 6.0 x 24	= 39.7 kips
Exterior Walls	15 x 6.0 x 320	= 28.8 kips
Subtotal		668.5 kips
2nd through 5th Floors		
Floor	100 x 100 x 60	= 600.0 kips
Column	276 x 12.0 x 24	= 79.5 kips
Exterior Walls	15 x 12.0 x 320	= 57.6 kips
Subtotal		737.1 kips x 4 = 2948.4 kips
1st Floor :		
Column	276 x 6.0 x 24	= 39.7 kips
Exterior Walls	15 x 6.0 x 320	= 28.8 kips
Subtotal		68.5 kips
Total Dead Load :		W = 3685.4 kips

* Calculation of Base Shear Q

Under soil condition S1 : Q = 0.0375W = 0.0375 x 3685.4 = 138.2 kips
Under soil condition S2 : Q = 0.0450W = 0.0450 x 3685.4 = 165.8 kips
Under soil condition S3 : Q = 0.0525W = 0.0525 x 3685.4 = 193.5 kips

* Distribution of Earthquake Forces

Assuming Ft = zero, since T=0.4518 sec., and the bottom of the base is 10 feet below the ground floor, the lateral force at each floor, Fx, can be calculated based on ANSI A58.1-1982[1]. The earthquake loads applied to the buildings are shown in Tables B-3, B-4, B-5 and Fig. B-3.

$$F_x = Q\, W_x\, h_x / (\Sigma W_i\, h_i)$$

Table B-3 Lateral Force Fx : Soil Condition S1 (Q=138.2 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	668.5	70	46795	39.1
5th Floor	737.1	58	42752	35.7
4th Floor	737.1	46	33907	28.3
3rd Floor	737.1	34	25061	20.9
2nd Floor	737.1	22	16216	13.5
1st Floor	68.5	10	685	0.6
$\Sigma w_i h_i$			165416	

Table B-4 Lateral Force Fx : Soil Condition S2 (Q=165.8 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	668.5	70	46795	46.9
5th Floor	737.1	58	42752	42.9
4th Floor	737.1	46	33907	34.0
3rd Floor	737.1	34	25061	25.1
2nd Floor	737.1	22	16216	16.2
1st Floor	68.5	10	685	0.7
$\Sigma w_i h_i$			165416	

Table B-5 Lateral Force Fx : Soil Condition S3 (Q=193.5 kips)

Level	w_x (kips)	h_x (ft)	$w_x h_x$	F_x (kips)
Roof	668.5	70	46795	54.7
5th Floor	737.1	58	42752	50.0
4th Floor	737.1	46	33907	39.7
3rd Floor	737.1	34	25061	29.3
2nd Floor	737.1	22	16216	19.0
1st Floor	68.5	10	685	0.8
$\Sigma w_i h_i$			165416	

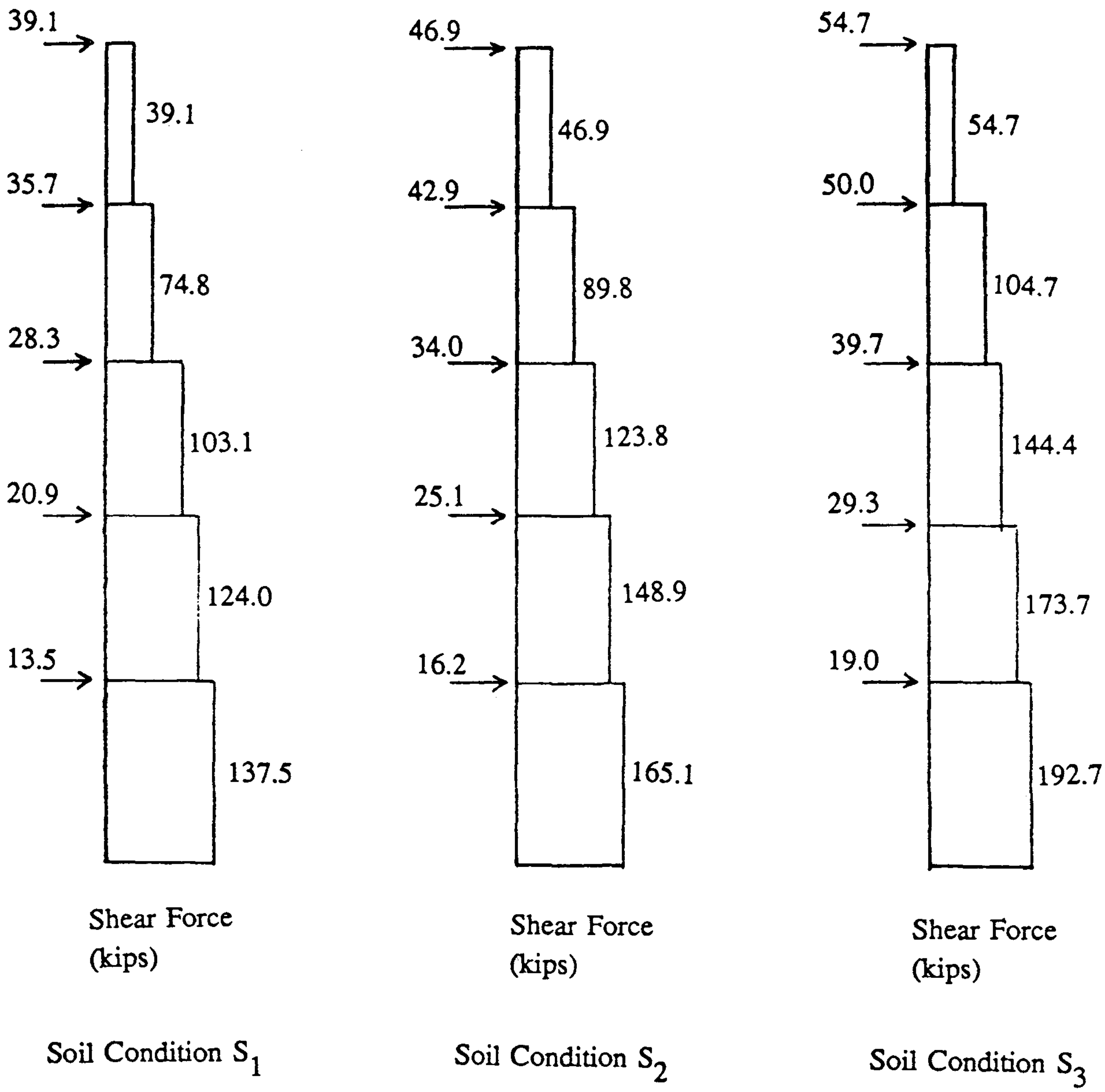


Fig. B-3 Seismic Forces (Seismic Zone 2)

B-4 Summary of Design Results

From detailed calculation based on ACI 318-83[8] (see Appendix C), the most critical area of lateral loading are the slab ends of the column strips. A summary of the design results on the column strip slab (see Fig. C-1) is indicated in Tables B-6 and B-7 (see section C-4 f also). A summary of the design results for columns is indicated in Tables B-8 and B-9 (see section C-5 also).

Table B-6 Summary of Column Strip Design Considering Lateral Load (Seismic Zone 2)

	W	S1	S2	S3
	At Face of Exterior Column			
Mu (ft-kips)	63.5	80.3	90.5	100.8
ϕ Mn (ft-kips)	85.6*	85.6	91.5	103.3
Reinforcement	14 - #4	14 - #4	15 - #4	17 - #4
	At Face of Interior Column			
Mu (ft-kips)	115.3	132.1	142.3	152.5
ϕ Mn (ft-kips)	120.7*	132.2	143.6	154.9
Reinforcement	20 - #4	22 - #4	24 - #4	26 - #4

Table B-7 Summary of Column Strip Design Considering Lateral Load (Seismic Zone 1)

	W	S1	S2	S3
	At Face of Exterior Column			
Mu (ft-kips)	63.5	54.8	59.9	65.0
ϕ Mn (ft-kips)	85.6*	85.6*	85.6*	85.6*
Reinforcement	14 - #4	14 - #4	14 - #4	14 - #4
	At Face of Interior Column			
Mu (ft-kips)	115.3	106.5	111.6	116.8
ϕ Mn (ft-kips)	120.7*	120.7*	120.7*	120.7*
Reinforcement	20 - #4	20 - #4	20 - #4	20 - #4

Notes;
W = is designed for Wind only
S1 : designed for Earthquake under S1
S2 : designed for Earthquake under S2
S3 : designed for Earthquake under S3
Sections : b = 10 ft. and h = 8"
Mu : Required factored strength
ϕ Mn : Reduced nominal moment strength

$$\phi \text{ Mn} = \phi \text{ [As fy (d - a / 2)]}$$

*) This value was determined by gravity loading.

Table B-8 Summary of Column Designs
(Seismic Zone 2)

	W	S1	S2	S3
	Exterior Columns			
Section	16" x 16"	16" x 16"	16" x 16"	16" x 16"
Main Bars	4 - #8*	4 - #8*	4 - #8*	4 - #8*
Provided ρg	0.012	0.012	0.012	0.012
	Interior Columns			
Section	16" x 16"	16" x 16"	16" x 16"	16" x 16"
Main Bars	4 - #8*	4 - #8*	6 - #8	6 - #9
Provided ρg	0.012	0.012	0.0185	0.023

Table B-9 Summary of Column Designs
(Seismic Zone 1)

	W	S1	S2	S3
	Exterior Columns			
Section	16" x 16"	16" x 16"	16" x 16"	16" x 16"
Main Bars	4 - #8*	4 - #8*	4 - #8*	4 - #8*
Provided ρg	0.012	0.012	0.012	0.012
	Interior Columns			
Section	16" x 16"	16" x 16"	16" x 16"	16" x 16"
Main Bars	4 - #8*	4 - #8*	4 - #8*	4 - #8*
Provided ρg	0.012	0.012	0.012	0.012

Notes;

W : designed for Wind only

S1 : designed for Earthquake under S1

S2 : designed for Earthquake under S2

S3 : designed for Earthquake under S3

ρg : Ratio of longitudinal column reinforcements to column sectional area

*) This reinforcement was determined from minimum requirement in ACI 318-83

B-5 Summary of Ultimate Structural Capacity

From detailed calculations (see Appendix D), since the ultimate capacity is dependent on the external load, the ultimate structural capacity of each frame can be derived as follows, using plastic analysis[11].

Table B-10 Summary of Ultimate Capacity of Each Frame

	W	S1	S2	S3
	Seismic Zone 2			
Q_{uE} (kips)	---	82.4	87.6	92.7
Q_{uW} (kips)	95.8	98.4	104.9	110.7
	Seismic Zone 1			
Q_{uE} (kips)	---	80.1	80.1	80.1
Q_{uW} (kips)	95.8	95.8	95.8	95.8

Notes;

W : designed for Wind only

S1 : designed for Earthquake under S1

S2 : designed for Earthquake under S2

S3 : designed for Earthquake under S3

Q_{uE} : Ultimate Capacity of Each Frame at 1st Floor for Earthquake Loading

Q_{uW} : Ultimate Capacity of Each Frame at 1st Floor for Wind Loading

APPENDIX C

DETAILED CALCULATION OF RC SOLID FLAT PLATE STRUCTURE

APPENDIX C : Detailed Calculation of RC Solid Flat-Plate Structure

C-1 Design of Two-way Solid Flat-Plate Slab for Dead and Live Loads

ACI 318-83 Code[8] allows a Direct Design Method (Section 13.6) and Equivalent Frame Method (Section 13.7) for two-way slab systems. In this appendix, the Equivalent Frame Method (Fig. C-1) will be used, since the Direct Design Method is basically simplified and its a practical method for first trial. The following discussions are limited to the design of interior equivalent frame in north-south direction.

a) Plate thickness (h) : Assume h = 8"

Minimum requirement of ACI 318-83 (9.5.3.1.(a)) is 5-in.
In Eq. 9-11, 9-12, let l_n = 20 - 16/12 = 18.67 ft. and fy = 60,000 psi, the following equations are given.

From Eq. 9-11	min. h = l_n/32.73 = 6.8"	(square interior panel)
	min. h = l_n/28.7 = 7.8"	(square edge panel)
	min. h = l_n/27.7 = 8.1"	(square corner panel)
From Eq. 9-12	min. h = l_n/41.8 = 5.4"	(square interior panel)
	min. h = l_n/36.98 = 6.1"	(square edge panel)
	min. h = l_n/35.95 = 6.2"	(square corner panel)

Therefore, the assumption of h = 8" will be adopted.

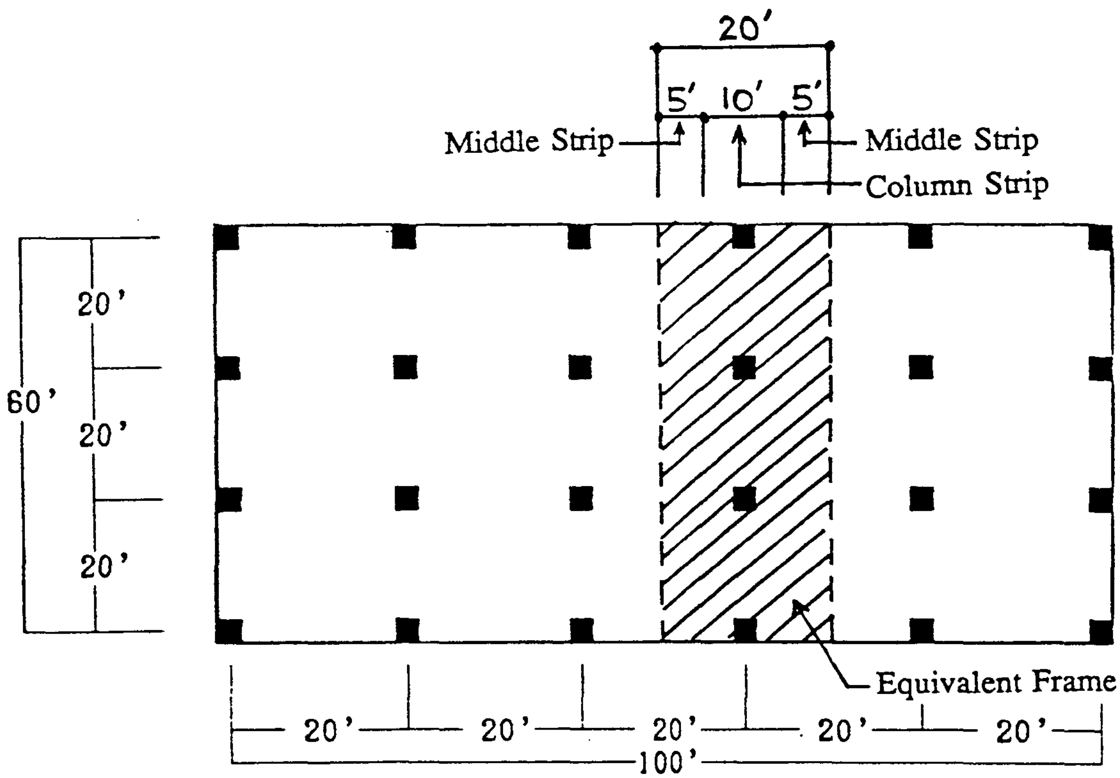


Fig. C-1 Equivalent Frame

b) Determine column stiffness (Kc) and slab stiffness (Ks) :

$Kc = 4\ E\ Ic\ /\ (\ l_c - 2h)$, $\qquad Ks = 4\ E\ Is\ /\ (\ l_1 - 0.5 \times c_1)$

where

Ic, Is	= gross section moment of inertia of column and slab (in^4)	
l_c	= story height (in.)	
l_1	= center-to-center span (in.)	
c_1	= column dimension (in.)	
Ic	$= 16 \times 16^3 / 12$	$= 5460\ in^4$
Kc / E	$= 4 \times 5460 / (144 - 2x8)$	= 171
Is	$= 240 \times 8^3 / 12$	$= 10240\ in^4$
Ks / E	$= 4 \times 10240 / (240 - 0.5x16)$	= 176

* Check of ratio of column-to-slab stiffness (α_{min}) : (ACI 318-83,13.6.10)

The ratio of dead load to live load : β_a

$\beta_a = 100 / 50 = 2.0 \qquad$ and $\alpha = 0$

From Table 13.6.10 (ACI 318-83), $\alpha_{min} = 0$

$\alpha_c = 2 \times 171 / 176 = 1.945 \quad > \quad \alpha_{min} = 0$

Therefore, it is not necessary to consider the moment multiplication factor δs.

c) Stiffness of equivalent column (Kec)

$Kec = \Sigma\ Kc\ /\ (\ 1 + \Sigma\ Kc\ /\ Kt\ )$ (ACI 318-83 Commentary 13.7.4)

where Kt is the torsional stiffness of the plate at the side of the column.

$Kt = \Sigma\ 9\ E\ C\ /\ [l_2(1 - c_2/l_2)^3]$ (ACI 318-83, Eq.13-6)

$C = \Sigma\ (1 - 0.63\ x/y\ )\ x^3 y\ /\ 3$ (ACI 318-83, Eq 13-7)

where x = 8 in., y = 16 in., $\Sigma = 1$ in this model.

$C = 1 \times (1 - 0.63 \times 8/16\ ) \times 8^3 \times 16 / 3 = 1870$

$Kt / E = 2 \times 9 \times 1870 / (240 \times (1 - 0.0667)^3) = 172$

Therefore,

$Kec / E = 2 \times 171 / (1 + 2 \times 171/172) = 114$

d) Moment distribution factors (DF)

DF(ext.) = Ks / (Ks + Kec) = 176 / 290 = 0.607

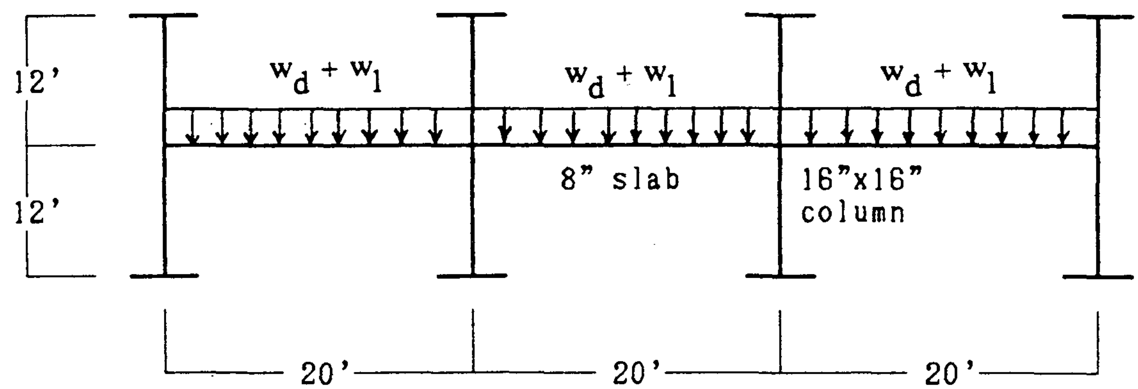
DF(int.) = Ks / (2Ks + Kec) = 176 / 466 = 0.378

e) Design moment for moment distribution

Based on ACI 318-83,13.7.6.2, the equivalent frame can be analyzed neglecting the full-design gravity load on the spans and pattern loading.

$$w_d = 1.4 \times 100 \times 20 \times 10^{-3} = 2.80 \text{ klf}$$
$$w_l = 1.7 \times 50 \times 20 \times 10^{-3} = 1.70 \text{ klf}$$
$$FEM = (2.80 + 1.70) \times 20^2 / 12 = 150 \text{ ft.-kips}$$
$$Mo = (2.80 + 1.70) \times 20^2 / 8 = 225 \text{ ft.-kips}$$
$$V = (2.80 + 1.70) \times 20 / 2 = 45.0 \text{ kips}$$

f) Moment distribution analysis



DF	0.607	0.378	0.378	0.378	0.378	0.607
FEM	-150	150	-150	150	-150	150
D1	91	-	-	-	-	-91
C1	-	46	-	-	-46	-
D2	-	-17	-17	17	17	-
C2	-9	-	9	-9	-	9
D3	5	-3	-3	3	3	-5
C3	-2	3	2	-2	-3	2
D4	1	-2	-2	2	2	-1
Mu,neg.	-64	177	-161	161	-177	64
Mu,pos.	105		64		105	
Vo	45.0 -5.7	45.0 5.7	45.0 -	45.0 -	45.0 5.7	45.0 -5.7
Vu	39.3	50.7	45.0	45.0	50.7	39.3

Fig. C-2 Moment Distribution Analysis

g) Check of slab shear at exterior column

Calculation of shear force (Vcs) and moment (Mcs) at shear center of perimeter is based on ACI 318-83 Commentary 11.12.2.4, and also see Fig. C-3.
In this figure, $c_1 = c_2 = 16$ in. and $d = 8 - 3/4 - 3/4 = 6.5$ in. and b_0 (perimeter length) $= 19.25 \times 2 + 22.5 = 61$ in.

$$Vcs = 39.3 + (2.80 + 1.70)\ x8/12 = 42.3 \text{ kips}$$
$$Mcs = 64 - (2.80 + 1.70) \times 8/12 \times 4/12 - Vcs \times e = 64 - 1 - 42.3 \times 5.18/12$$
$$= 44.7 \text{ ft-kips}$$

Critical design shear stress of perimeter AB is given by following formula:

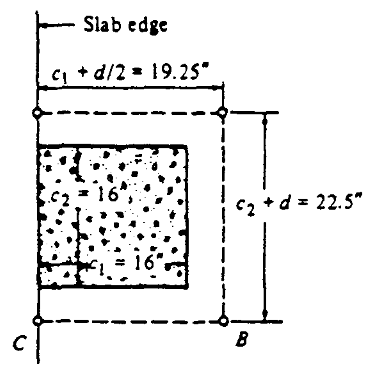
$$v_{AB} = Vcs / Ac + k\ Mcs\ C_{AB} / Jc \qquad \text{(ACI 318-83 Commentary 11.12.2.4)}$$

where

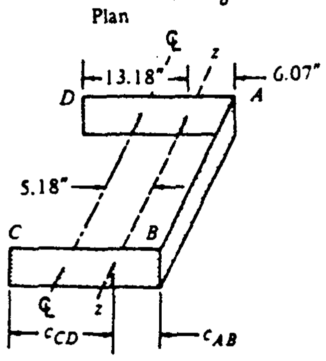
Ac = shear section = $b_0 d = 61 \times 6.5 = 396$ in^2
Jc = polar moment of inertia of shear section = 17,160 in^4
$C_{AB} = 6.07"$
k = fraction of moment between slab and column
$= 1 - 1 / (1 + 2/3 \times [(c_1 + d/2) / (c_2 + d)]^{1/2})$
$= 1 - 1 / (1 + 2/3 \times [(16 + 6.5/2) / (16 + 6.5)]^{1/2}) = 0.381$

$$v_{AB} = 42300 / 396 + 0.381 \times 44700 \times 12 \times 6.07 / 17160 = 179 \text{ psi}$$

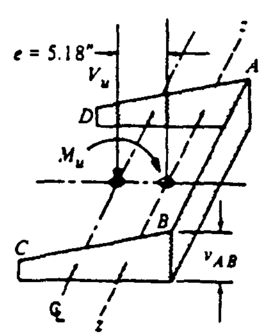
Allowable stress $\quad v_a = \phi \times 4\sqrt{fc'} = 0.85 \times 4\sqrt{4000} = 215$ psi $> v_{AB} = 179$ psi
(ACI 318-83,11.11.2.1)



Perimeter A-B-C-D = b_o
Plan



Shear section, A_c
Dimensions



V_u, M_u, v_{AB}

Fig. C-3 Shear Force

h) Check of slab shear at interior column (see Fig. C-4)

$$V_{cs} = 50.7 + 45.0 = 95.7 \text{ kips}$$
$$M_{cs} = 177 - 161 = 16 \text{ ft-kips}$$

Let $A_c = 585$ in^2, $J_c = 50389$ in^4, $C_{AB} = 11.25$ in. for interior column,

$v_{AB} = 95700 / 585 + 0.40 \times 16000 \times 12 \times 11.25 / 50389 = 181$ psi $< v_a = 215$ psi

i) Check of beam shear : (ACI 318-83,11.11.1.1) (see Fig. C-5)

$b_0 = 240$ in.

$V_u = 50.7 - (2.8 + 1.7) \times (8 + 6.5)/12 = 45.3$ kips

$\phi V_c = \phi \times 2\sqrt{f_c'}\, b_0\, d = 0.85 \times 2 \times \sqrt{4000} \times 240 \times 6.5 = 167.7$ kips $>$ $V_u = 45.3$ kips

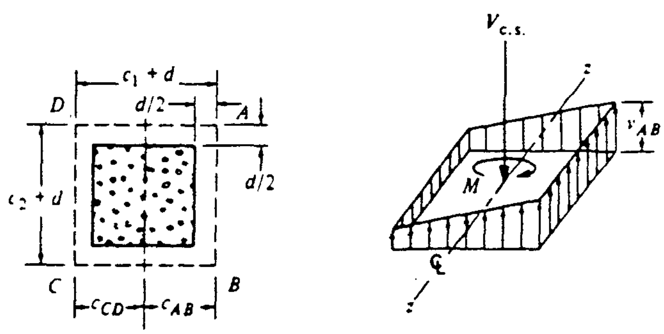


Fig. C-4 Shear at Interior Column

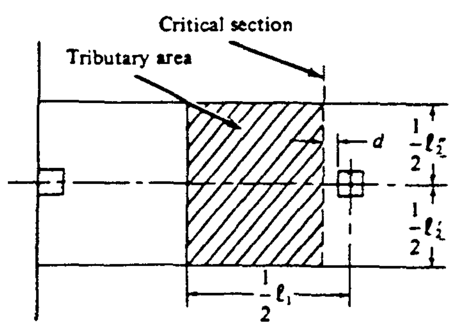


Fig. C-5 Beam Shear

j) Distribution of panel moments to column and middle strips

The face moments of the exterior and interior columns can be calculated and distributed to the column and middle strips of slab based on ACI 318-83,13.6.4.1 & 13.6.4.2 & 13.6.4.4.

Face moment at exterior column :

$$M = -64 + (39.3 + (39.3 - 4.5 \times 8/12)) / 2 \times 8/12 = -39 \text{ ft-kips}$$

Face moment at interior column :

$$M = -177 + (50.7 + (50.7 - 4.5 \times 8/12)) / 2 \times 8/12 = -144 \text{ ft-kips}$$

If $\alpha_1 = 0$, $\beta_t = 0$, $l_2 / l_1 = 1.0$, we obtain the following :

Table C-1 Distribution of Panel Moments

Location	Panel Moments (ft-kips)	Column Strip (%)	Middle Strip (%)
Face of ext. column	-39	100	0
Mid-span of ext. panel	105	60	40
Face of int. column	-144	75	25
Mid-span of int. panel	64	60	40

Table C-2 Design Moments of Panels

Location	Column Strip (ft-kips)	Column Strip* (within c + 3h) (ft-kips)	Middle Strip (ft-kips)
Face of ext. column	-39	-39.6	0
Mid-span of ext. panel	63	--	42
Face of int. column	-108	-9.6	-36
Mid-span of int. panel	38	--	26

Notes:

* Design moment within c + 3h width is calculated as :

$Mu = M_{unbalance} \times \gamma_f$ (ACI 318-83,13.3.3.2)

$\gamma_f = 0.619$ (for exterior column)

$\gamma_f = 0.600$ (for interior column)

k) Design of slabs of column strip

* Column strip at face of exterior column

  $Mu = 39$ ft-kips, assume $j = 0.98$

  $As, req = Mu / \phi\, fy\, j\, d = 39 \times 12 / (0.9 \times 60 \times 0.98 \times 7) = 1.26\ in^2$ ($\rho = 0.0015$)

  but from ACI 318-83,7.12, $\rho_{min} = 0.0018$, and

  $As\ min = 1.33 \times 1.26 = 1.68\ in^2$ ($\rho = 0.002$) (ACI 318-83,10.5.2)

  prov. 10 - #4 @ 120"/10 = 12.0 in. ($As = 2.0\ in^2$, $\rho = 0.0024$)

  $a = As\ fy / (0.85 \times fc' \times b) = 2.0 \times 60 / (0.85 \times 4 \times 120) = 0.294$ in.

  $\phi\ Mn = \phi\ [As\ fy\ (d - a/2)] = 0.9 \times [2.0 \times 60 \times (7 - 0.147)] / 12 = 61.7$ ft-kips

* Column strip at face of exterior column within c + 3h width

  $Mu = 39.6$ ft-kips, assume $j = 0.92$

  $As\ req = Mu / \phi\, fy\, j\, d = 39.6 \times 12 / (0.9 \times 60 \times 0.92 \times 7) = 1.37\ in^2$ ($\rho = 0.0049$)

  prov. 7 - #4 @ 40"/7 = 5.7 in. ($As = 1.4\ in^2$)

  $a = 1.4 \times 60 / (0.85 \times 4 \times 40) = 0.618$ in.

  $\phi\ Mn = 0.9 \times [1.4 \times 60 \times (7 - 0.309)] / 12 = 42.2$ ft-kips $>$ $Mu = 39.6$ ft-kips

* Mid span of exterior panel

  $Mu = 63$ ft-kips, assume $j = 0.96$

  $As\ req = Mu / \phi\, fy\, j\, d = 63 \times 12 / (0.9 \times 60 \times 0.96 \times 7) = 2.08\ in^2$ ($\rho = 0.0025$)

  but from ACI 318-83,10.5.1, $\rho_{min} = 200/fy = 0.0033$ $As\ min = 2.77\ in^2$

  prov. 14 - #4 @ 120"/14 = 8.6 in. ($As = 2.8\ in^2$)

  $a = 2.8 \times 60 / (0.85 \times 4 \times 120) = 0.412$ in.

  $\phi\ Mn = 0.9 \times [2.8 \times 60 \times (7 - 0.206)] / 12 = 85.6$ ft-kips $>$ $Mu = 63$ ft-kips

* Column strip at face of interior column

  $Mu = 108$ ft-kips, assume $j = 0.92$

  $As\ req = Mu / \phi\, fy\, j\, d = 108 \times 12 / (0.9 \times 60 \times 0.92 \times 7) = 3.73\ in^2$ ($\rho = 0.0044$)

  prov. 20 - #4 @ 120"/20 = 6 in. ($As = 4.0\ in^2$)

  $a = 4.0 \times 60 / (0.85 \times 4 \times 120) = 0.59$ in.

  $\phi\ Mn = 0.9 \times [1.4 \times 60 \times (7 - 0.295)] / 12 = 120.7$ ft-kips $>$ $Mu = 108$ ft-kips

* Mid span of interior panel

  $Mu = 38$ ft-kips, assume $j = 0.96$

  $As\ req = Mu / \phi\, fy\, j\, d = 38 \times 12 / (0.9 \times 60 \times 0.96 \times 7) = 1.27\ in^2$ ($\rho = 0.0015$)

  but from ACI 318-83,10.5.2 $As\ min = 1.33 \times 1.27 = 1.69\ in^2$ ($\rho = 0.002$)

  prov. 10 - #4 @ 120"/10 = 12.0 in. ($As = 2.0\ in^2$ $\rho = 0.0024$)

  $a = 2.0 \times 60 / (0.85 \times 4 \times 120) = 0.294$ in.

  $\phi\ Mn = 0.9 \times [2.0 \times 60 \times (7 - 0.147)] / 12 = 61.7$ ft-kips $>$ $Mu = 38$ ft-kips

Table C-3 Summary of Column Strip Design Due to Gravity Load

	Face of ext. column	Mid-span of ext. panel	Face of int. column	Mid-span of int. column
Mu (ft-kips)	39	63	108	38
ϕ Mn (ft-kips)	85.6	85.6 (*1)	120.7	61.7 (*1)
Section	14 #4 (*2)	14 #4	20 #4	10 #4
	b = 10 ft.	b = 10 ft.	b = 10 ft.	b = 10 ft.
	h = 8"	h = 8"	h = 8"	h = 8"

Notes;

Mu = Required factored strength

ϕ Mn = Reduced nominal moment strength

$$\phi \text{ Mn} = \phi \text{ [As fy (d - a/2)]}$$

*1 This strength is determined by minimum reinforcement requirement in ACI 318-83,7.12.

*2 7 #4 bars should be provided within 3.3' width around column.

$$\phi \text{ Mn} = 0.9 \times [\ 2.8 \times 60 \times (7 - 0.206)] / 12 = 85.6 \text{ ft-kips}$$

C-2 Analysis of Frames Under Dead and Live Loading

a) For Gravity and Full Live Loading (ACI 318-83,13.7.6.2)

From Fig. C-2,

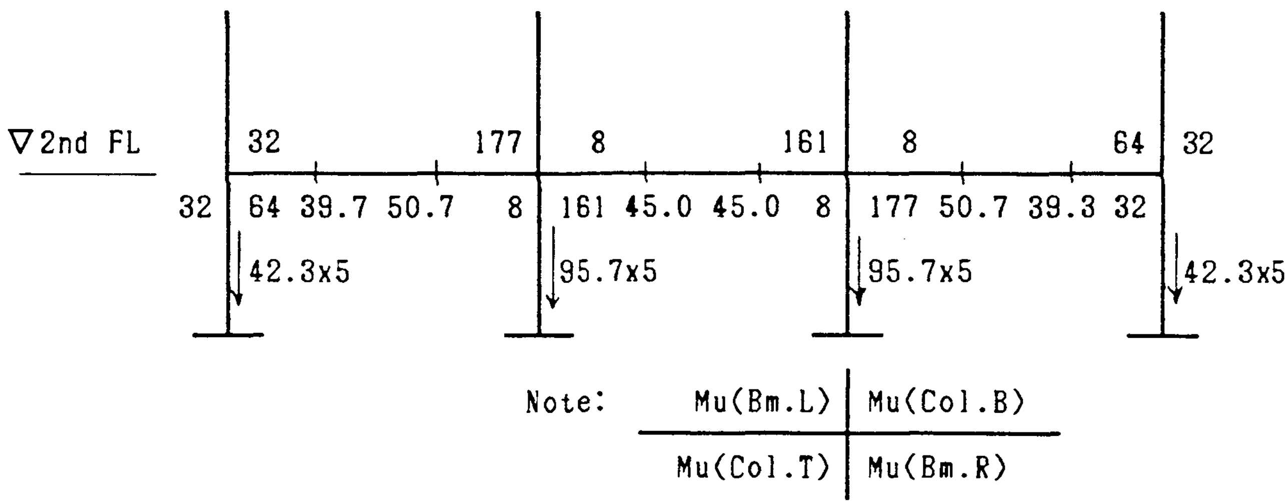
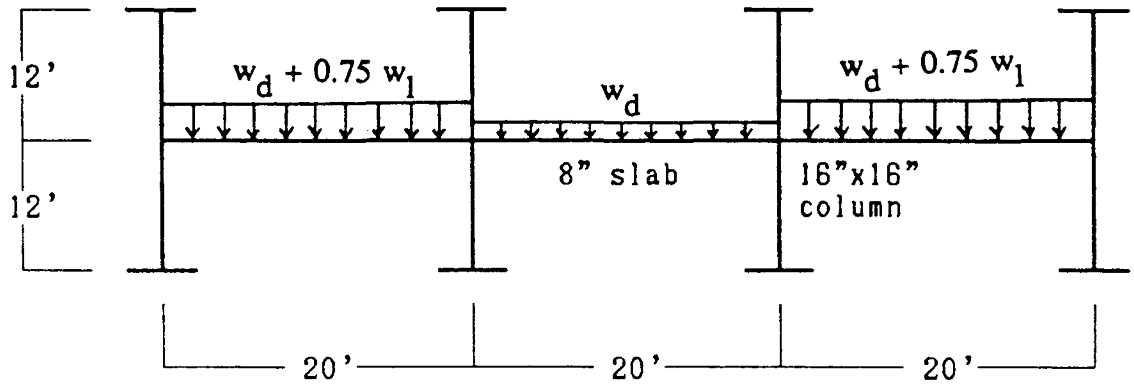


Fig. C-6 Moment, Shear and Axial Forces Under Gravity Load

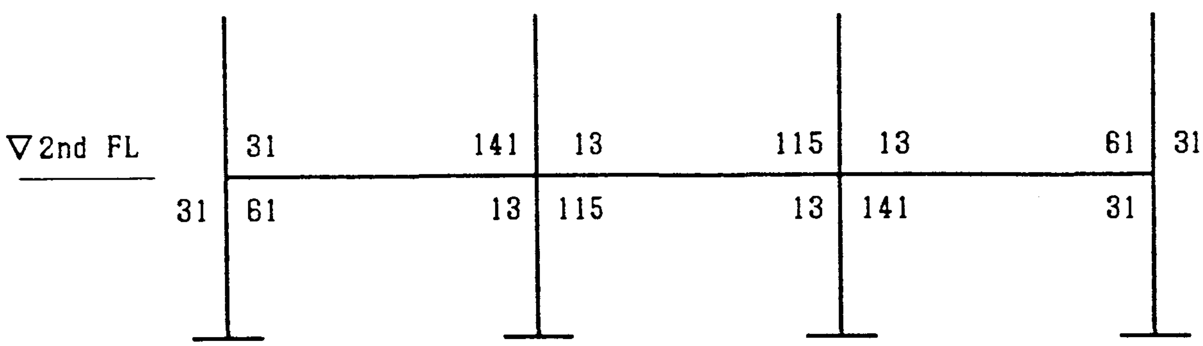
b) For pattern loading (ACI 318-83,13.7.6.3),

$$FEM(w_d) = 2.80 \times 20^2 / 12 = 93.3 \text{ ft-kips}$$

$$FEM(w_d + 0.75w_l) = (2.80 + 1.275) \times 20^2 /12 = 135.8 \text{ ft-kips}$$



DF	0.607	0.378	0.378	0.378	0.378	0.607
FEM	-135.8	135.8	-93.3	93.3	-135.8	135.8
D1	82.4	-16.1	-16.1	16.1	16.1	-82.4
C1	-8.1	41.2	8.1	-8.1	-41.2	8.1
D2	4.9	-18.6	-18.6	18.6	18.6	-4.9
C2	-9.3	2.5	9.3	-9.3	-2.5	9.3
D3	5.6	-4.5	-4.5	4.5	4.5	-5.6
C3	-2.3	2.8	2.3	-2.3	-2.8	2.3
D4	1.4	-1.9	-1.9	1.9	1.9	-1.4
Mu,neg.	-61	141	-115	115	-141	61



Note:

Mu(Bm.L)	Mu(Col.B)
Mu(Col.T)	Mu(Bm.R)

Fig. C-7 Moment Due to Pattern Loading

C-3 Analysis of Frame for Lateral Loadings

In the analysis, the Approximate Method (Portal Approximation) will be used and only the column strip will be considered effective as a part of the unbraced frame.

In the analysis for lateral loading, the following assumptions are used.

1. Lateral loads due to wind or earthquake in the north-south direction are equally resisted by the six frames.

2. The total horizontal shear in all columns of a given story is equal and opposite to the sum of all horizontal loads acting above that story.

3. The horizontal shear is the same in both exterior columns, and the horizontal shear in each interior column is twice that of an exterior column.

4. The inflection points of all columns except bottom columns are located midway between the joints. The inflection points of bottom columns are assumed to be at a distance of 0.6 times story height from bottom.

5. The inflection points of all beams are located at midway between the supports.

a) Analysis for wind loading

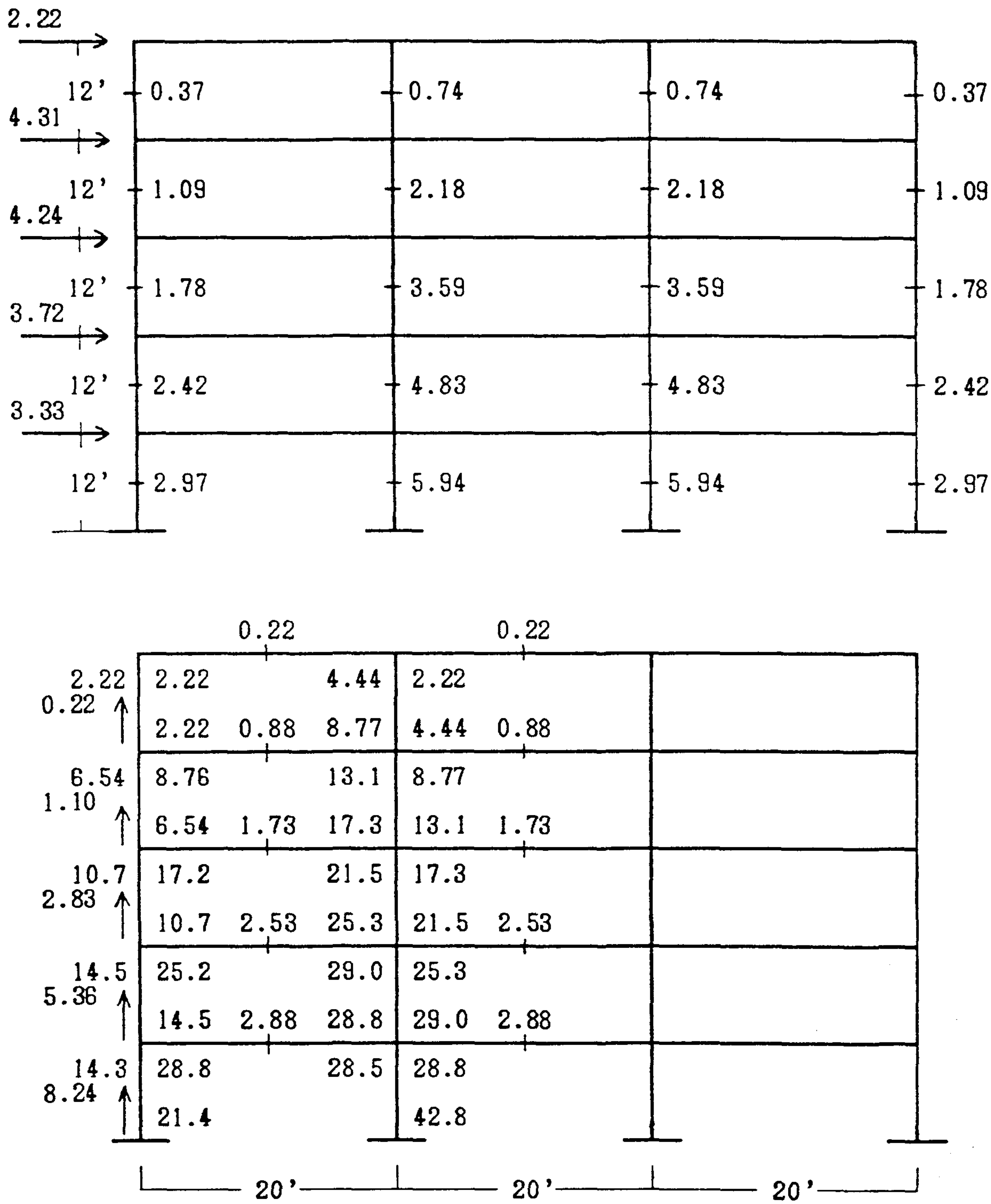


Fig. C-8 Wind Load Analysis

Note:

Mu(Bm.L)	Mu(Col.B)
Mu(Col.T)	Mu(Bm.R)

b) Analysis for earthquake loading (Seismic Zone 2)

Under soil condition S1 :

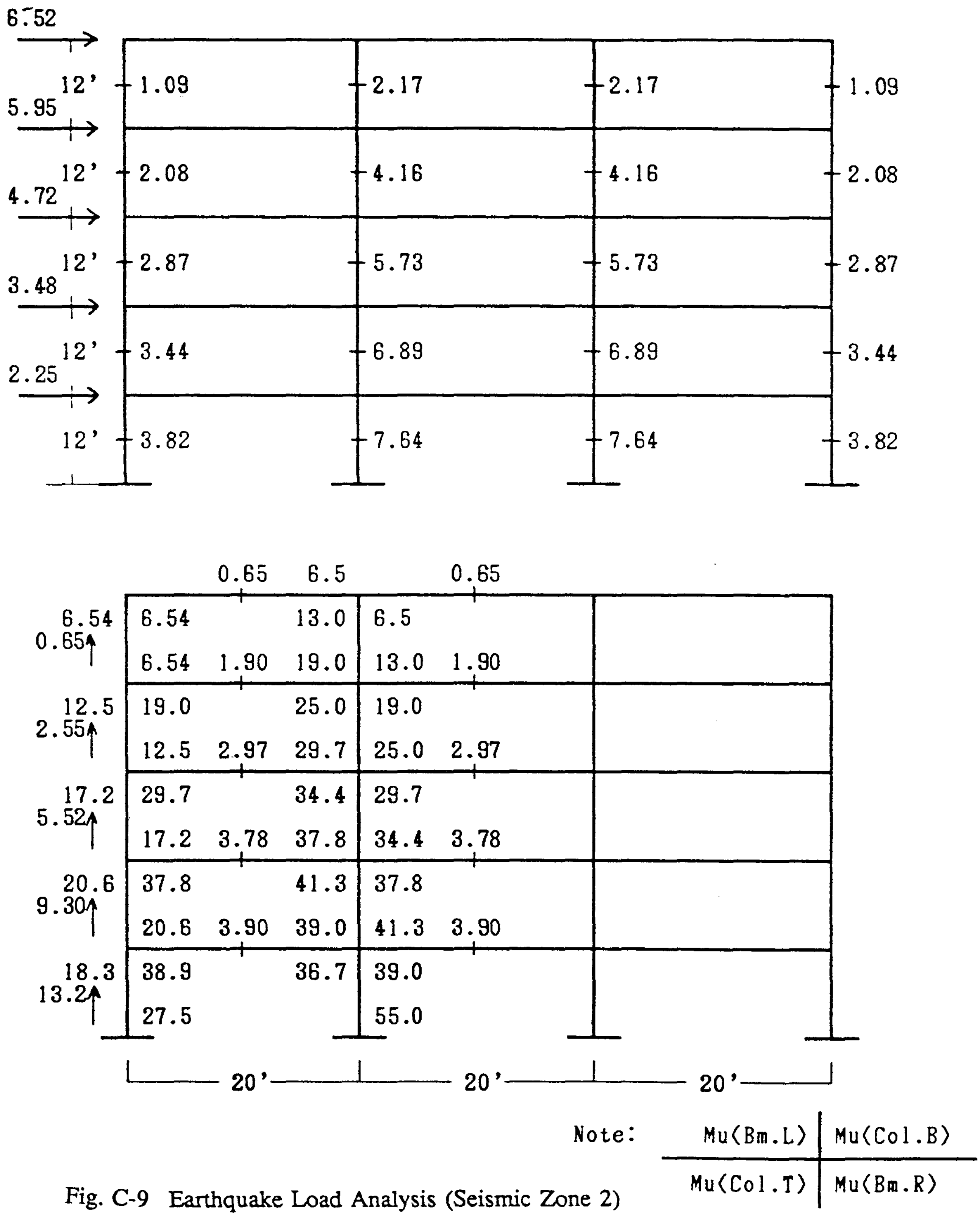


Fig. C-9 Earthquake Load Analysis (Seismic Zone 2)

C-4 Check of Design Moment of Slab Considering Lateral Load

a) Wind loading

* Column strip at face of exterior column at 2nd floor

Face moment due to wind M_W

$M_W = 28.8 - 2.88 \times 8/12 = 26.9$ ft-kips

$Mu = 0.75 \times (39.0 + 26.9 \times 1.7) = 63.5$ ft-kips $< \phi\, Mn = 85.6$ ft-kips

* Column strip at face of interior column at 2nd floor

$Mu = 0.75 \times (108 + 26.9 \times 1.7) = 115.3$ ft-kips $< \phi\, Mn = 120.7$ ft-kips

* Check of slab shear at interior column at 2nd floor

$v_{AB} = 0.75 \times (181 + 2880 / 585 \times 1.7) = 142$ psi $< v_a = 215$ psi

Therefore, no change of design for wind load is required.

b) Earthquake Loading (soil condition S1) - seismic zone 2

* Column strip at face of exterior column at 2nd floor

Face moment due to earthquake M_E

$M_E = 39.0 - 3.90 \times 8/12 = 36.4$ ft-kips

$Mu = 0.75 \times (39.0 + 36.4 \times 1.87) = 80.3$ ft-kips $< \phi\, Mn = 85.6$ ft-kips

* Column strip at face of interior column at 2nd floor

$Mu = 0.75 \times (108 + 36.4 \times 1.87) = 132.1$ ft-kips $> \phi\, Mn = 120.7$ ft-kips

prov. 22 - #4 @ 120"/22 = 5.5 in. ($As = 4.4$ in^2)

$a = 4.4 \times 60 / (0.85 \times 4 \times 120) = 0.647$ in.

$\phi\, Mn = 0.9 \times [4.4 \times 60 \times (7 - 0.324)] / 12 = 132.2$ ft-kips $> Mu = 132.1$ ft-kips

* Check of slab shear at interior column at 2nd floor

$v_{AB} = 0.75 \times (181 + 3900 / 585 \times 1.87) = 145$ psi $< v_a = 215$ psi

No change of design for shear is required.

c) Earthquake loading (soil condition S2) - seismic zone 2

* Column strip at face of exterior column at 2nd floor

Face moment due to earthquake M_E

$M_E = 36.4 \times 165.8 / 138.2 = 43.7$ ft-kips

$Mu = 0.75 \times (39.0 + 43.7 \times 1.87) = 90.5$ ft-kips > ϕ Mn = 85.6 ft-kips

prov. 15 - #4 within 120" width ($As = 3.0$ in^2)

$a = 3.0 \times 60 / (0.85 \times 4 \times 120) = 0.441$ in.

$\phi Mn = 0.9 \times [3.0 \times 60 \times (7 - 0.221)] / 12 = 91.5$ ft-kips > $Mu = 90.5$ ft-kips

* Column strip at face of interior column at 2nd floor

$Mu = 0.75 \times (108 + 43.7 \times 1.87) = 142.3$ ft-kips

prov. 24 - #4 @ 120"/24 = 5 in. ($As = 4.8$ in^2)

$a = 4.8 \times 60 / (0.85 \times 4 \times 120) = 0.706$ in.

$\phi Mn = 0.9 \times [4.8 \times 60 \times (7 - 0.353)] / 12 = 143.6$ ft-kips > $Mu = 142.3$ ft-kips

d) Earthquake loading (soil condition S3) - seismic zone 2

* Column strip at face of exterior column at 2nd floor

Face moment due to earthquake M_E

$M_E = 36.4 \times 193.5 / 138.2 = 51.0$ ft-kips

$Mu = 0.75 \times (39.0 + 51.0 \times 1.87) = 100.8$ ft-kips

prov. 17 - #4 within 120" width ($As = 3.4$ in^2)

$a = 3.4 \times 60 / (0.85 \times 4 \times 120) = 0.500$ in.

$\phi Mn = 0.9 \times [3.4 \times 60 \times (7 - 0.250)] / 12 = 103.3$ ft-kips > $Mu = 100.8$ ft-kips

* Column strip at face of interior column at 2nd floor

$Mu = 0.75 \times (108 + 51.0 \times 1.87) = 152.5$ ft-kips

prov. 26 - #4 @ 120"/26 = 4.6 in. ($As = 5.2$ in^2)

$a = 5.2 \times 60 / (0.85 \times 4 \times 120) = 0.765$ in.

$\phi Mn = 0.9 \times [5.2 \times 60 \times (7 - 0.382)] / 12 = 154.9$ ft-kips > $Mu = 152.5$ ft-kips

e) Earthquake loading - seismic zone 1

Since coefficient $Z = 3/16$ in seismic zone 1, base shear Q becomes one-half the values of seismic zone 2. Therefore, design moment and shear force of slab due to earthquake load also become one-half the values in section C-4, b, c and d.

* Soil condition S3 (which is the most critical case in Zone 1)

i) Column strip face moment at exterior column at 2nd floor : M_E

$M_E = 51.0 / 2 = 25.5$ kips-ft

$Mu = 0.75 \times (39.0 + 25.5 \times 1.87) = 65.0$ kips-ft

Mu = 65.0 kips-ft is smaller than $\phi Mn = 85.6$ kips-ft which is the reduced nominal moment of the section determined by gravity load.
Therefore, the section in zone 1 is the same as that for wind.

ii) Column strip face moment at interior column at 2nd floor : M_E

$M_E = 51.0 / 2 = 25.5$ kips-ft

$Mu = 0.75 \times (108 + 25.5 \times 1.87) = 116.8$ kips-ft

Mu = 116.8 kips-ft is smaller than $\phi Mn = 120.7$ kips-ft which is the reduced nominal moment of the section determined by gravity load.
Therefore, the section in zone 1 is the same as that for wind.

f) Summary of design of column strip of slab

Table C-4 Summary of Column Strip Design Considering Lateral Load (Seismic Zone 2)

	W	E-1 (S1)	E-2 (S2)	E-3 (S3)
	At Face of Exterior Column			
Mu (ft-kips)	63.5	80.3	90.5	100.8
ϕ Mn (ft-kips)	85.6*	85.6	91.5	103.3
Reinforcement	14 - #4	14 - #4	15 - #4	17 - #4
	At Face of Interior Column			
Mu (ft-kips)	115.3	132.1	142.3	152.5
ϕ Mn (ft-kips)	120.7*	132.2	143.6	154.9
Reinforcement	20 - #4	22 - #4	24 - #4	26 - #4

Table C-5 Summary of Column Strip Design Considering Lateral Load (Seismic Zone 1)

	W	E-1 (S1)	E-2 (S2)	E-3 (S3)
	At Face of Exterior Column			
Mu (ft-kips)	63.5	54.8	59.9	65.0
ϕ Mn (ft-kips)	85.6*	85.6*	85.6*	85.6*
Reinforcement	14 - #4	14 - #4	14 - #4	14 - #4
	At Face of Interior Column			
Mu (ft-kips)	115.3	106.5	111.6	116.8
ϕ Mn (ft-kips)	120.7*	120.7*	120.7*	120.7*
Reinforcement	20 - #4	20 - #4	20 - #4	20 - #4

Notes;

W : designed for Wind only

E-1 (S1) : designed for Earthquake under S1 in addition to wind

E-2 (S2) : designed for Earthquake under S2 in addition to wind

E-3 (S3) : designed for Earthquake under S3 in addition to wind

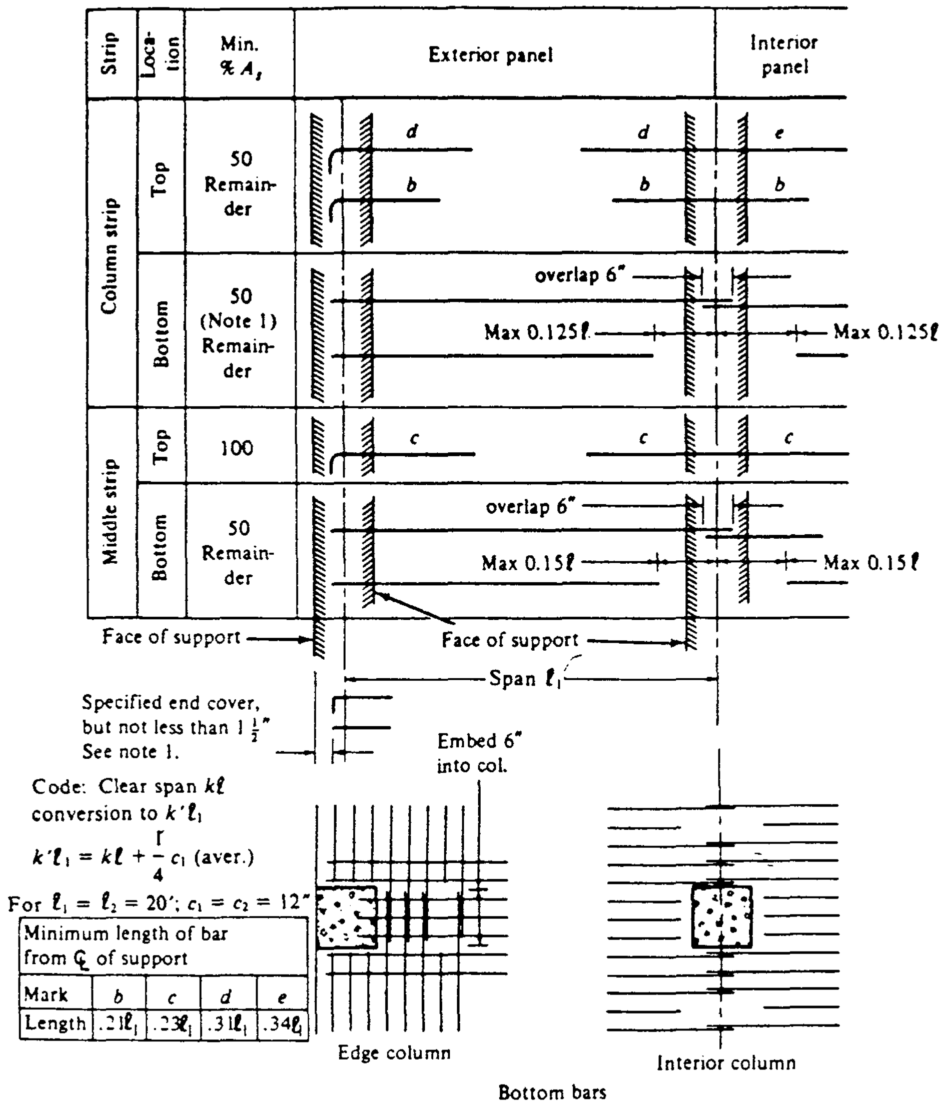
Sections : b = 10 ft. and h = 8"

Mu : Required factored strength

ϕ Mn : Reduced nominal moment strength

$$\phi \text{ Mn} = \phi \text{ [As fy (d - a / 2)]}$$

*) This value was determined by gravity loading.



Notes:

1. Place alternate lengths of column strip bottom bars except within the width of the column verticals. Use the shorter length bars in this width to clear the column verticals as shown, except at edge columns, embed 6" into edge columns.
2. Place half of the column strip top bars within the middle third of the column strip.
3. Place alternate lengths of middle strip bottom bars.

Fig. C-10 Flat Plate Reinforcement Detail

C-5 Design of Columns at 1st Floor

Since this model frame is not braced sideways, both magnification factors δ_b and δ_s should be considered.

a) Check of slenderness ratio of column ($k\,l_u$ / r) (ACI 318-83,10.11.4.2 & 10.11.4.3)

$k \geq 1.0$, assume $k = 1.0$ (only for the purpose of checking slenderness ratio)
$l_u = 12' \times 12 - 8 = 136$ in.
$r = 0.30 \times 16 = 4.8$ in. (ACI 318-83,10.11.3)
$k\,l_u / r = 1.0 \times 136 / 4.8 = 28 > 22$

Therefore, slenderness effects should be considered.

b) Calculation of column stiffness

$E\,I = (Ec\,Ig / 5 + Es\,Ise) / (1 + \beta_d)$ (ACI 318-83, Eq.10-10)

where

$Ec\,Ig / 5 = (3.6 \times 10^6 \times 16^4 / 12) / 5 = 3.93 \times 10^9$ lb-in^2

Assume 4 #9 located at 2.5" from the face of column,

$Es\,Ise = 29 \times 10^6 \times 4 \times 1.0 \times 5.5^2 = 3.51 \times 10^9$ lb-in^2

$E\,I = (3.93 + 3.51) \times 10^9 / (1 + \beta_d) = 7.44 \times 10^9 / (1 + \beta_d)$

c) Calculation of δ_b

Although this frame is unbraced sideways, This frame can be assumed as a braced frame for gravity loading (for the calculation of δ_b) based on ACI 318-83, Commentary 10.11.5.1, because the frame and loadings are symmetric. Therefore, k = 1.0 can be used for the calculation of δ_b (ACI 318-83,10.11.2.1).

$\beta_d = w_d / (w_d + w_l) = 2.8 / (2.8 + 1.7) = 0.62$

From ACI 318-83, Eq.10-9,

$Pc = \pi^2 E\,I / (k\,l_u)^2 = (3.14^2 \times 7.44 \times 10^9 / 1.62) / 136^2 = 2451$ kips

$Pu = 95.7 \times 5 = 478.5$ kips (From Fig. C-6)

$\delta_b = Cm / (1 - Pu / \phi\,Pc) \geq 1.0$ (ACI 318-83, Eq.10-7)

where $Cm = 1.0$ (ACI 318-83,10.11.5.3), $\phi = 0.7$.

$\delta_b = 1.0 / (1 - 478.5 / (0.7 \times 2451)) = 1.39$

Assume the same value of δ_b for both interior and exterior columns which gives conservative results for convenience of calculation.

d) Calculation of M_{2b}

M_{2b} is defined as a factored moment due to gravity loading in ACI Commentary 10.11.5.1 but shall be greater than the value based on a minimum eccentricity of (0.6 + 0.03h) in. (ACI 318-83,10.11.5.4).

* Exterior column

M_{2b} = 32 ft-kips
Pu = 42.3 x 5 = 211.5 kips
e = 32 x 12 / 211.5 = 1.82 in. > 0.6 + 0.03h = 1.08 in.

* Interior column

Pu = 478.5 kips
M_{2b} = (0.6 + 0.03 h) x 478.5 / 12 = 43.1 ft-kips

e) Calculation of δ_s (under wind load)

In order to calculate the k value for δ_s of an unbraced frame (ACI 318-83,10.11.2.2), we will use a value of 0.5Ig for the slab column strip as a flexural member and ACI 318-83, Eq.10-10 with $\beta_d = 0$ for the columns only for calculation of the k value (ACI Commentary 10.11.2).

* Slab column strip EI :

EI = 0.5 Ig Ec = 0.5 x (120 x 8^3 / 12) x 3.6 x 10^6 = 9.22 x 10^9 lb-in^2.

* k value for δ_s at 1st floor

Calculating ψ (= {Σ EI/l of columns} / {Σ EI/l of flexural members}), k value for δ_s can be derived using Fig. 10.11.2 in ACI Commentary 10.11.2.

i) Exterior column

ψ(bottom) = 0 (Fixed base)
ψ(top) = (2 x 7.44 / 12) x (20 / 9.22) = 2.69
k(ext.) = 1.33

ii) Interior column

ψ(top) = (2 x 7.44 / 12) x (20 / (2x9.22)) = 1.35
k(int.) = 1.18

* β_d value for δ_s at 1st floor

i) Exterior column

Mu(1.4D) = 32 x w_d / (w_d + w_l) = 32 x 0.62 = 19.8
Mu(0.75x(1.4D + 1.7L + 1.7W)) = 0.75 x (32 + 1.7 x 14.3) = 42.2
β_d = 19.8 / 42.2 = 0.47

ii) Interior column

Mu(1.4D) = 8 x w_d / (w_d + w_l) = 8 x 0.62 = 5.0
Mu(0.75x(1.4D + 1.7L + 1.7W)) = 0.75 x (8 + 1.7 x 28.5) = 42.3
β_d = 5.0 / 42.3 = 0.12

* Calculation of Σ Pc and Σ Pu at 1st floor

$EI(ext.) = 7.44 \times 10^9 / (1 + 0.47) = 5.06 \times 10^9$
$EI(int.) = 7.44 \times 10^9 / (1 + 0.12) = 6.64 \times 10^9$

$\Sigma Pc = [5.06 / (1.33x136)^2 + 6.64 / (1.18x136)^2] \times 3.14^2 \times 2 \times 10^6 = 8142$ kips

$\Sigma Pu = (42.3 \times 5 + 95.7 \times 5) \times 2 = 1380$ kips

* δ_s at 1st floor (ACI 318-83, Eq.10-8)
$\delta_s = 1 /[1 - \Sigma Pu / (\phi \Sigma Pc)] = 1 / [1 - 1380 / (0.7 \times 8142)] = 1.32$

f) Design of column at 1st floor (under wind load)

* Exterior column

Case 1 : 1.4 D + 1.7 L

$Pu = 42.3 \times 5 = 211.5$ kips
$Mc = \delta_b M_{2b} = 1.39 \times 32 = 44.5$ kips-ft

Pu / Ag = 0.82, Mc / (Ag h) = 0.13, req. $\rho_g = 0.01$ (from Fig. A-11[10]).
Provide minimum reinforcement for column, 4-#8 (3.16 in^2, $\rho_g = 0.012$).

Case 2 : 0.75 x (1.4 D + 1.7 L + 1.7 W)

$Pu = 0.75 \times (211.5 - 8.24 \times 1.7) = 148.1$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 44.5 \times 0.75 + 1.32 \times 21.4 \times 1.7 \times 0.75 = 69.4$ kips-ft

Pu / Ag = 0.58, Mc / (Ag h) = 0.20, req. $\rho_g = 0.01$ (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

Case 3 : 0.9 D + 1.3 W

$Pu(D) = 211.5 \times w_d / (w_d + w_l) / 1.4 = 211.5 \times 0.62 / 1.4 = 93.7$ kips
$Mu(D) = 32 \times w_d / (w_d + w_l) / 1.4 = 32 \times 0.62 / 1.4 = 14.2$ kips-ft
$M_{2b} = 0.9 \times 14.2 = 12.8$ kips-ft , $e = 12.8 \times 12 / 93.7 = 1.64$ in $> e_{min} = 1.08$ in

$Pu = 0.9 \times 93.7 - 1.3 \times 8.24 = 73.6$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39 \times 12.8 + 1.32 \times 21.4 \times 1.3 = 54.5$ kips-ft

Pu / Ag = 0.29, Mc / (Ag h) = 0.16, req. $\rho_g = 0.01$ (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

* Interior column

Case 1 : 1.4 D + 1.7 L

Pu = 95.7 x 5 = 478.5 kips
Mc = δ_b M_{2b} = 1.39 x 43.1 = 60.0 ft-kips

Pu / Ag = 1.87, Mc / (Ag h) = 0.18, req. ρ_g = 0.01 (from Fig. C-11[10]).
Provide minimum reinforcement for column, 4-#8 (3.16 in^2, ρ_g = 0.012).

Case 2 : 0.75 x (1.4 D + 1.7 L + 1.7 W)

Pu = 0.75 x 478.5 = 358.9 kips
Mc = δ_b M_{2b} + δ_s M_{2s} = 60.0 x 0.75 + 1.32 x 42.8 x 1.7 x 0.75 = 117.0 ft-kips

Pu / Ag = 1.40, Mc / (Ag h) = 0.34, req. ρ_g = 0.01 (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, ρ_g = 0.012).

Case 3 : 0.9 D + 1.3 W

Pu(D) = 478.5 x w_d / (w_d + w_l) / 1.4 = 478.5 x 0.62 / 1.4 = 211.9 kips
M_{2b} = 211.9 x (0.6 + 0.03 x 16) / 12 x 0.9 = 17.2 kips-ft

Pu = 0.9 x 211.9 = 190.7 kips
Mc = δ_b M_{2b} + δ_s M_{2s} = 1.39 x 17.2 + 1.32 x 42.8 x 1.3 = 97.4 kips-ft

Pu / Ag = 0.74, Mc / (Ag h) = 0.29, req. ρ_g = 0.01 (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, ρ_g = 0.012).

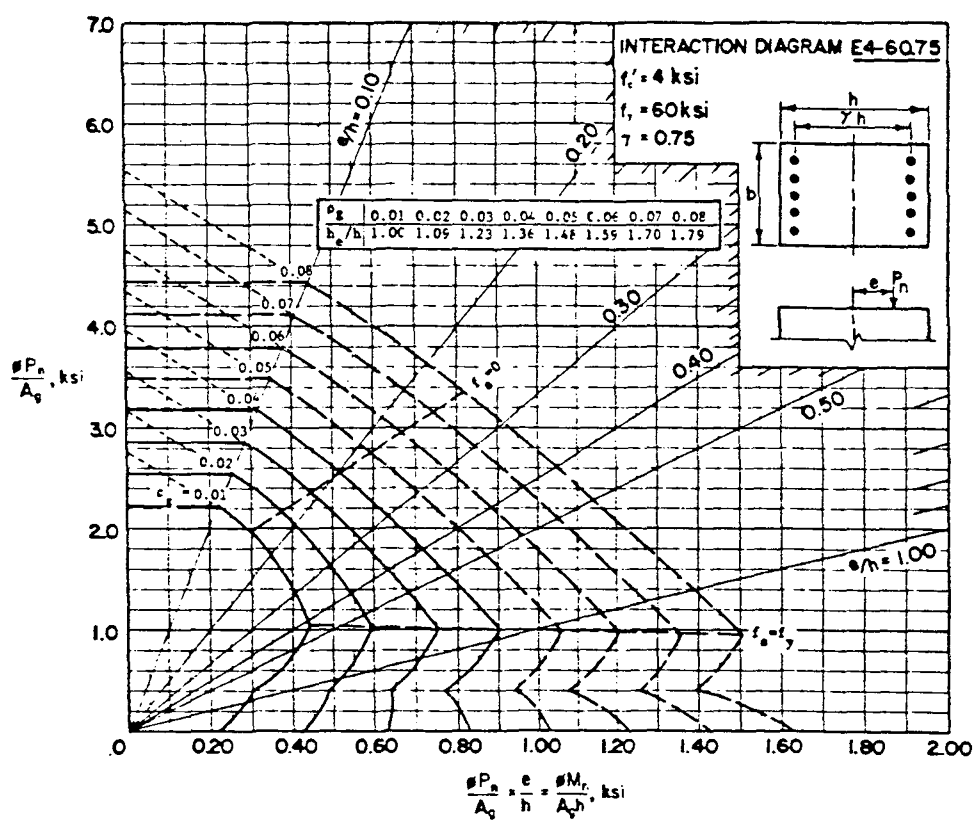


Fig. C-11 Load-Moment Strength Interaction Diagram for Column [10]

g) Calculation of δ_s (under earthquake load - seismic zone 2)

In order to calculate the k value for δ_s of an unbraced frame (ACI 318-83,10.11.2.2), we will use a value of 0.5Ig for the slab column strip as a flexural member and ACI 318-83, Eq.10-10 with $\beta_d = 0$ for the columns only for calculation of the k value (ACI Commentary 10.11.2).

* Slab column strip EI :

$EI = 0.5\ Ig\ Ec = 0.5 \times (120 \times 8^3 / 12) \times 3.6 \times 10^6 = 9.22 \times 10^9$ lb-in^2.

* k value for δ_s at 1st floor

Calculating ψ (= {Σ EI/l of columns} / {Σ EI/l of flexural members}),the k value for δ_s can be derived using Fig. 10.11.2 in ACI Commentary 10.11.2.

i) Exterior column

ψ(bottom) = 0 (Fixed base)

ψ(top) = (2 x 7.44 / 12) x (20 / 9.22) = 2.69

k(ext.) = 1.33

ii) Interior column

ψ(top) = (2 x 7.44 / 12) x (20 / (2x9.22)) = 1.35

k(int.) = 1.18

* β_d value for δ_s at 1st floor (use soil condition S1)

i) Exterior column

$Mu(1.4D) = 32 \times w_d / (w_d + w_l) = 32 \times 0.62 = 19.8$

$Mu(0.75x(1.4D + 1.7L + 1.87E)) = 0.75 \times (32 + 1.87 \times 18.3) = 49.7$

$\beta_d = 19.8 / 49.7 = 0.40$

ii) Interior column

$Mu(1.4D) = 8 \times w_d / (w_d + w_l) = 8 \times 0.62 = 5.0$

$Mu(0.75x(1.4D + 1.7L + 1.87E)) = 0.75 \times (8 + 1.87 \times 36.7) = 57.5$

$\beta_d = 5.0 / 57.5 = 0.09$

* Calculation of Σ Pc and Σ Pu at 1st floor

$EI(ext.) = 7.44 \times 10^9 / (1 + 0.40) = 5.31 \times 10^9$

$EI(int.) = 7.44 \times 10^9 / (1 + 0.09) = 6.83 \times 10^9$

$\Sigma Pc = [5.31 / (1.33 \times 136)^2 + 6.83 / (1.18 \times 136)^2] \times 3.14^2 \times 2 \times 10^6 = 8430$ kips

$\Sigma Pu = (42.3 \times 5 + 95.7 \times 5) \times 2 = 1380$ kips

* δ_s at 1st floor (ACI318-83, Eq.10-8)

$\delta_s = 1 / [1 - \Sigma Pu / (\phi\ \Sigma Pc)] = 1 / [1 - 1380 / (0.7 \times 8430)] = 1.31$

h) Design of column at 1st floor (under earthquake load - seismic zone 2)

h-1) Soil condition S1

* Exterior column

Case 1 : 1.4 D + 1.7 L

Pu = 42.3 x 5 = 211.5 kips
$Mc = \delta_b M_{2b}$ = 1.39 x 32 = 44.5 ft-kips

Pu / Ag = 0.82, Mc / (Ag h) = 0.13, req. ρ_g = 0.01 (from Fig. C-11[10]).

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

Pu = 0.75 x (211.5 - 13.2 x 1.87) = 140.1 kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s}$ = 44.5 x 0.75 + 1.31 x 27.5 x 1.87 x 0.75 = 83.9 kips-ft

Pu / Ag = 0.55, Mc / (Ag h) = 0.25, req. ρ_g = 0.01 (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, ρ_g = 0.012).

Case 5 : 0.9 D + 1.43 E

Pu(D) = 211.5 x $w_d / (w_d + w_l)$ / 1.4 = 211.5 x 0.62 / 1.4 = 93.7 kips
Mu(D) = 32 x $w_d / (w_d + w_l)$ / 1.4 = 32 x 0.62 / 1.4 = 14.2 kips-ft
M_{2b} = 0.9 x 14.2 = 12.8 kips-ft , e = 12.8 x 12 / 93.7 = 1.64 in > e_{min} = 1.08 in

Pu = 0.9 x 93.7 - 1.43 x 13.2 = 65.5 kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s}$ = 1.39 x 12.8 + 1.31 x 27.5 x 1.43 = 69.3 kips-ft

Pu / Ag = 0.26, Mc / (Ag h) = 0.20, req. ρ_g = 0.01 (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, ρ_g = 0.012).

* Interior column

Case 1 : 1.4 D + 1.7 L

Pu = 95.7 x 5 = 478.5 kips
$Mc = \delta_b M_{2b}$ = 1.39 x 43.1 = 60.0 ft-kips

Pu / Ag = 1.87, Mc / (Ag h) = 0.18, req. ρ_g = 0.01 (from Fig. C-11[10]).

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

Pu = 0.75 x 478.5 = 358.9 kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s}$ = 60.0 x 0.75 + 1.31 x 55.0 x 1.87 x 0.75 = 146.1 ft-kips

Pu / Ag = 1.40, Mc / (Ag h) = 0.43, req. ρ_g = 0.012 (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, ρ_g = 0.012).

Case 5 : 0.9 D + 1.43 E

$Pu(D) = 478.5 \times w_d / (w_d + w_l) / 1.4 = 478.5 \times 0.62 / 1.4 = 211.9$ kips
$M_{2b} = 211.9 \times (0.6 + 0.03 \times 16) / 12 \times 0.9 = 17.2$ kips-ft

$Pu = 0.9 \times 211.9 = 190.7$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39 \times 17.2 + 1.31 \times 55.0 \times 1.43 = 127.0$ kips-ft

$Pu / Ag = 0.74$, $Mc / (Ag\ h) = 0.37$, req. $\rho_g = 0.01 < \rho_g = 0.012$

h-2) Soil condition S2

* Exterior column

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

$Pu = 0.75 \times (211.5 - 13.2 \times 1.2 \times 1.87) = 136.4$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 44.5x0.75 + 1.31x27.5x1.2x1.87x0.75 = 94.0$ kips-ft

$Pu / Ag = 0.53$, $Mc / (Ag\ h) = 0.28$, req. $\rho_g = 0.01$ (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

Case 5 : 0.9 D + 1.43 E

$Pu = 0.9 \times 93.7 - 1.43 \times 13.2 \times 1.2 = 61.7$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39x12.8 + 1.31x27.5x1.2x1.43 = 79.6$ kips-ft

$Pu / Ag = 0.24$, $Mc / (Ag\ h) = 0.23$, req. $\rho_g = 0.01$ (from Fig. A-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

* Interior column

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

$Pu = 0.75 \times 478.5 = 358.9$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 60.0x0.75 + 1.31x55.0x1.2x1.87x0.75 = 166.3$ ft-kips

$Pu / Ag = 1.40$, $Mc / (Ag\ h) = 0.49$, req. $\rho_g = 0.018$ (from Fig. C-11[10]).
Provide 6 #8 (4.74 in^2, $\rho_g = 0.0185$).

Case 5 : 0.9 D + 1.43 E

$Pu = 0.9 \times 211.9 = 190.7$ kips
$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39x17.2 + 1.31x55.0x1.2x1.43 = 147.5$ kips-ft

$Pu / Ag = 0.74$, $Mc / (Ag\ h) = 0.43$, req. $\rho_g = 0.013 < \rho_g = 0.018$

h-3) Soil condition S3

* Exterior column

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

$Pu = 0.75 \times (211.5 - 13.2 \times 1.4 \times 1.87) = 132.7$ kips

$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 44.5x0.75 + 1.31x27.5x1.4x1.87x0.75 = 104.1$ kips-ft

Pu / Ag = 0.52, Mc / (Ag h) = 0.30, req. $\rho_g = 0.01$ (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

Case 5 : 0.9 D + 1.43 E

$Pu = 0.9 \times 93.7 - 1.43 \times 13.2 \times 1.4 = 57.9$ kips

$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39x12.8 + 1.31x27.5x1.4x1.43 = 89.9$ kips-ft

Pu / Ag = 0.23, Mc / (Ag h) = 0.26, req. $\rho_g = 0.01$ (from Fig. C-11[10]).
Provide 4 #8 (3.16 in^2, $\rho_g = 0.012$).

* Interior column

Case 4 : 0.75 x (1.4 D + 1.7 L + 1.87 E)

$Pu = 0.75 \times 478.5 = 358.9$ kips

$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 60.0x0.75 + 1.31x55.0x1.4x1.87x0.75 = 186.5$ ft-kips

Pu / Ag = 1.40, Mc / (Ag h) = 0.55, req. $\rho_g = 0.022$ (from Fig. C-11[10]).
Provide 6 #9 (6.00 in^2, $\rho_g = 0.023$).

Case 5 : 0.9 D + 1.43 E

$Pu = 0.9 \times 211.9 = 190.7$ kips

$Mc = \delta_b M_{2b} + \delta_s M_{2s} = 1.39x17.2 + 1.31x55.0x1.4x1.43 = 168.2$ kips-ft

Pu / Ag = 0.74, Mc / (Ag h) = 0.48, req. $\rho_g = 0.015 < \rho_g = 0.022$

i) Consideration of earthquake load - seismic zone 1

In seismic zone 1, the base shear force Q becomes one-half the values in seismic zone 2. Even in the most critical case in seismic zone 1, which is soil condition S3, the effect due to lateral load is smaller than the one due to load in seismic zone 2, soil condition S1. Since the column reinforcements in seismic zone 2, soil condition S1 are 4 #8 which are determined by the minimum reinforcement requirements of ACI 318-83, column reinforcements in seismic zone 1 will result in 4 #8 ($\rho_g = 0.012$).

Since the dimensions of the columns were not determined by the design stress of the columns but by punching the shear of a slab (see section C-1,g and C-1,h), no change of column dimension even in seismic zone 1 will be required.

j) Summary of design of columns at 1st floor

Table C-6 Summary of Columns Design
(Seismic Zone 2)

	W	E-1 (S1)	E-2 (S2)	E-3 (S3)
	Exterior Columns			
Section Main Bars Provided ρg	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012
	Interior Columns			
Section Main Bars Provided ρg	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 6 - #8 0.0185	16" x 16" 6 - #9 0.023

Table C-7 Summary of Columns Design
(Seismic Zone 1)

	W	E-1 (S1)	E-2 (S2)	E-3 (S3)
	Exterior Columns			
Section Main Bars Provided ρg	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012
	Interior Columns			
Section Main Bars Provided ρg	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012	16" x 16" 4 - #8* 0.012

Notes;

W : designed for Wind only

E-1 (S1) : designed for Earthquake under S1 in addition to wind

E-2 (S2) : designed for Earthquake under S2 in addition to wind

E-3 (S3) : designed for Earthquake under S3 in addition to wind

ρg : Ratio of longitudinal column reinforcements to column sectional area

*) This reinforcement was determined by minimum requirement in ACI 318-83.

*) This reinforcement was determined from minimum requirement in ACI 318-83.

APPENDIX D

PLASTIC ANALYSIS OF RC SOLID FLAT PLATE STRUCTURE

APPENDIX D : Plastic Analysis of RC Solid Flat-Plate Structure

D-1 Assumption

In the plastic analysis for lateral loading, the following assumptions are used.

1. Lateral loads due to wind or earthquake in the north-south direction are equally resisted by the six frames.

2. Only the column strip of the slab will be considered effective as a part of the laterally resistive frame.

3. The ultimate capacity of the frame can be given by lateral loads of failure mechanism[11].

4. For the calculation of the ultimate capacity, the virtual work method[11] will be used based on Upper Bound Theorm. This method gives the upper limit of the ultimate capacity.
$$\Sigma P_i \delta_i = \Sigma Mp_i \theta_i$$

5. A bending failure mechanism will be assumed in this section. The assumed failure mechanism is shown on the next page.

6. The lateral loads on failure mechanism are assumed to be proportional to the external loads due to wind or earthquake[11]. The calculated external work is indicated in Table D-1.

7. The plastic moment Mp is equal to the nominal moment capacity Mn without the capacity reduction factor ϕ, and plastic hinges at the face of the supports.

Table D-1 External Works

Position	P_i		δ_i	$P_i \delta_i$	
	Wind	Earthquake		Wind	Earthquake
Roof	0.67 P	2.90 P	60 θ	40 Pθ	174 Pθ
5th floor	1.29 P	2.64 P	48 θ	62 Pθ	127 Pθ
4th floor	1.27 P	2.10 P	36 θ	46 Pθ	76 Pθ
3rd floor	1.12 P	1.55 P	24 θ	27 Pθ	37 Pθ
2nd floor	1.00 P	1.00 P	12 θ	12 Pθ	12 Pθ

$W_{ext} = \Sigma P_i \delta_i =$ 187 Pθ 426 Pθ

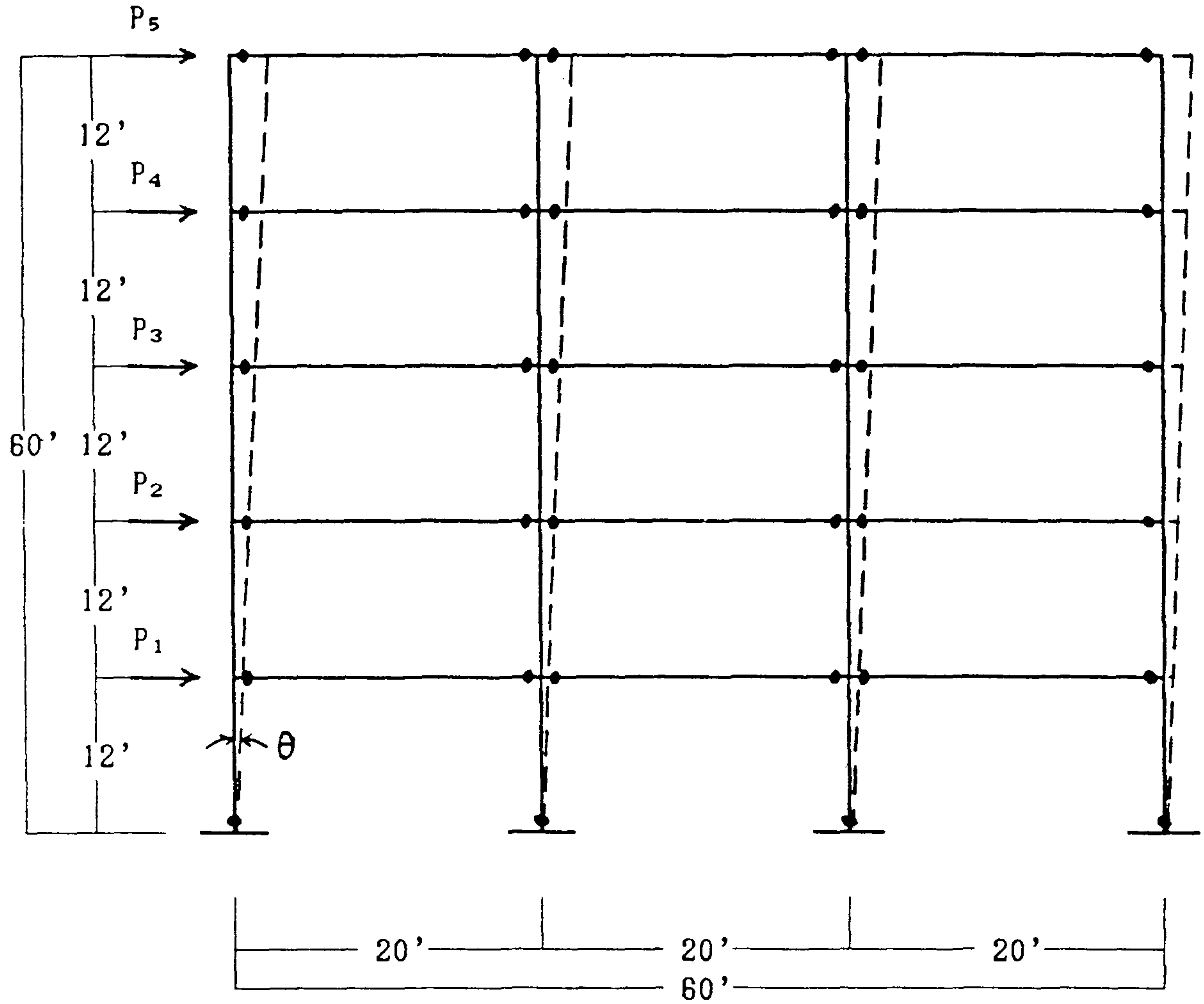


Fig. D-1 Assumed Mechanism

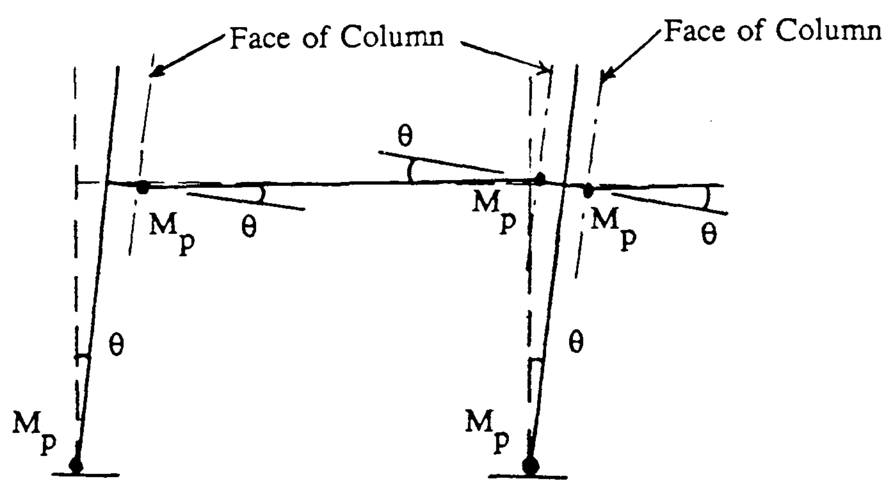


Fig. D-1a Internal Works

D-2 Calculation of Ultimate Capacity for Wind

* Negative plastic moment Mp of slab ends

i) at exterior slab ends : Mp = 85.6 / 0.9 = 95.1 kips-ft
ii) at interior slab ends : Mp = 120.7 / 0.9 = 134.1 kips-ft

* Positive plastic moment Mp of slab ends

Assuming one half of mid span reinforcements to be extended until interior supports and 100% of mid span reinforcements to be extended until exterior supports (see Fig. C-10), the positive plastic moment Mp of slab ends can be calculated as follows :

i) At exterior slab ends : $l_{d(prov.)} / l_{d(full)} = 14 / 12 > 1.0$

Mp = 85.6 / 0.9 = 95.1 kips-ft

ii) At interior slab ends : $l_{d(prov.)} / l_{d(full)} = 11 / 12 < 1.0$

Mp = 61.7 / 2 / 0.9 x 11/12 = 31.4 kips-ft

* Calculation of plastic moment of column

Assuming the additional axial force due to the failure mechanism can be calculated from the assumed mechanism (Fig. D-2) and can be estimated without the load factor, Mp of the column will be derived from the column interaction diagram in Fig. C-11 where ρ_g of a column = 0.012 (see Section C-5 f).

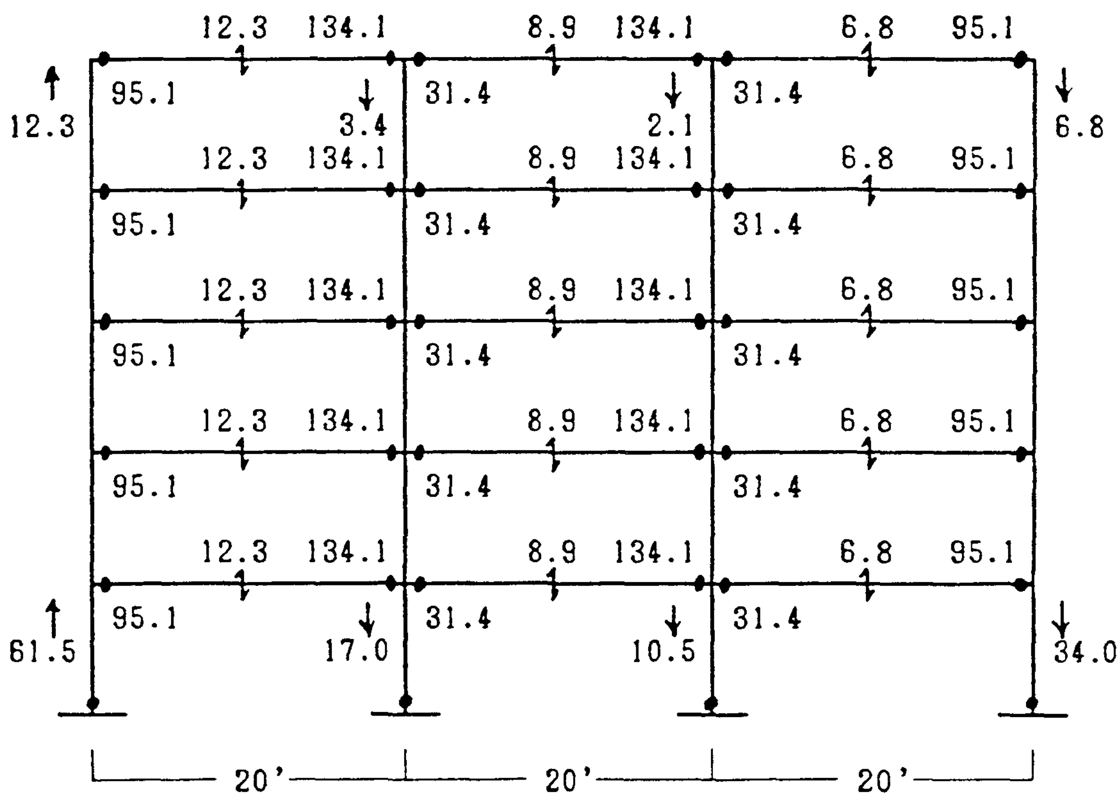


Fig. D-2 Assumed Mechanism

i) 1st floor exterior column (windward)

$Pu = 0.75 \times 211.5 - 61.5 = 97.1$ kips

$Pu / Ag = 97.1 / (16x16) = 0.38$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, $\phi\ Mn / Ag\ h = 0.35$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.38 = 0.71$

$Mp = Mn = 0.35 \times Ag\ h / \phi = 0.35 \times 16^3 / 0.71 / 12 = 168.3$ kips-ft

ii) 1st floor interior column

$Pu = 358.9 + 17.5 = 376.4$ kips

$Pu / Ag = 376.4 / (16x16) = 1.47$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, $\phi\ Mn / Ag\ h = 0.41$ ksi, $\quad \phi = 0.7$

$Mp = Mn = 0.41 \times Ag\ h / \phi = 0.41 \times 16^3 / 0.7 / 12 = 199.9$ kips-ft

iii) 1st floor exterior column (leeward)

$Pu = 0.75 \times 211.5 + 34.0 = 192.6$ kips

$Pu / Ag = 192.6 / (16x16) = 0.75$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, $\phi\ Mn / Ag\ h = 0.44$ ksi, $\quad \phi = 0.7$

$Mp = Mn = 0.44 \times Ag\ h / \phi = 0.44 \times 16^3 / 0.7 / 12 = 214.6$ kips-ft

iv) 5th floor exterior cclumn

$Pu = 0.75 \times 42.3 - 12.3 = 19.4$ kips

$Pu / Ag = 19.4 / (16x16) = 0.08$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, $\phi\ Mn / Ag\ h = 0.27$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.08 = 0.86$

$Mp = Mn = 0.27 \times Ag\ h / \phi = 0.27 \times 16^3 / 0.86 / 12 = 107.2$ kips-ft > 95.1

v) 5th floor interior column

$Pu = 0.75 \times 95.7 + 2.1 = 73.9$ kips

$Pu / Ag = 73.9 / (16x16) = 0.29$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, $\phi\ Mn / Ag\ h = 0.32$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.29 = 0.755$

$Mp = Mn = 0.32 \times Ag\ h / \phi = 0.32 \times 16^3 / 0.755 / 12 = 144.7$ kips-ft

* Internal work

$$\Sigma\ Mp\ \theta = (95.1x10 + 134.1x8 + 31.4x8 + 144.7x2 + 199.9x2 + 168.3 + 214.6)\ \theta$$
$$= 3347.1\ \theta$$

* Calculation of base shear capacity Qu of each frame for wind loading

$W_{ext} = W_{int}$ $\qquad 187\ P\theta = 3347.1\ \theta$ $\qquad P = 17.9$ kips

$Qu = \Sigma\ P_i = (0.67 + 1.29 + 1.27 + 1.12 + 1.0) \times 17.9 = 95.8$ kips

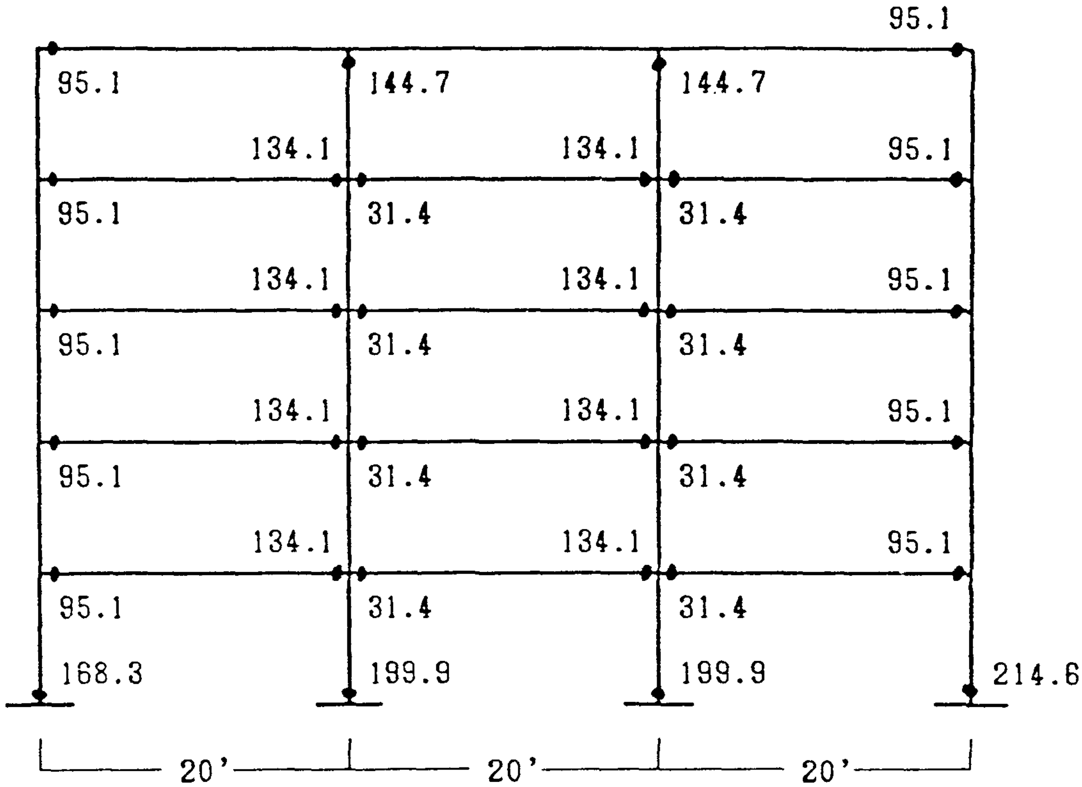


Fig. D-3 Final Mechanism

D-3 Calculation of Ultimate Capacity for Earthquake (Seismic Zone 2)

a) Soil condition S1

* Negative plastic moment Mp of slab ends

i) at exterior slab ends : Mp = 85.6 / 0.9 = 95.1 kips-ft
ii) at interior slab ends : Mp = 132.2 / 0.9 = 146.9 kips-ft

* Positive plastic moment Mp of slab ends

Assuming one-half of the mid-span reinforcements are extended to the interior supports and 100% of the mid-span reinforcements are extended to the exterior supports (see Fig. C-10), the positive plastic moment Mp of the slab ends can be calculated as follows :

i) At exterior slab ends : $l_{d(prov.)} / l_{d(full)} = 14 / 12 > 1.0$

Mp = 85.6 / 0.9 = 95.1 kips-ft

ii) At interior slab ends : $l_{d(prov.)} / l_{d(full)} = 11 / 12 < 1.0$

Mp = 61.7 / 2 / 0.9 x 11/12 = 31.4 kips-ft

* Calculation of plastic moment of column

Assuming additional axial force due to the failure mechanism can be calculated from the assumed mechanism (Fig. D-4) and can be estimated without the load factor, Mp of a column will be derived from the column interaction diagram in Fig. C-11 where ρ_g of a column = 0.012 (see Section C-5,h).

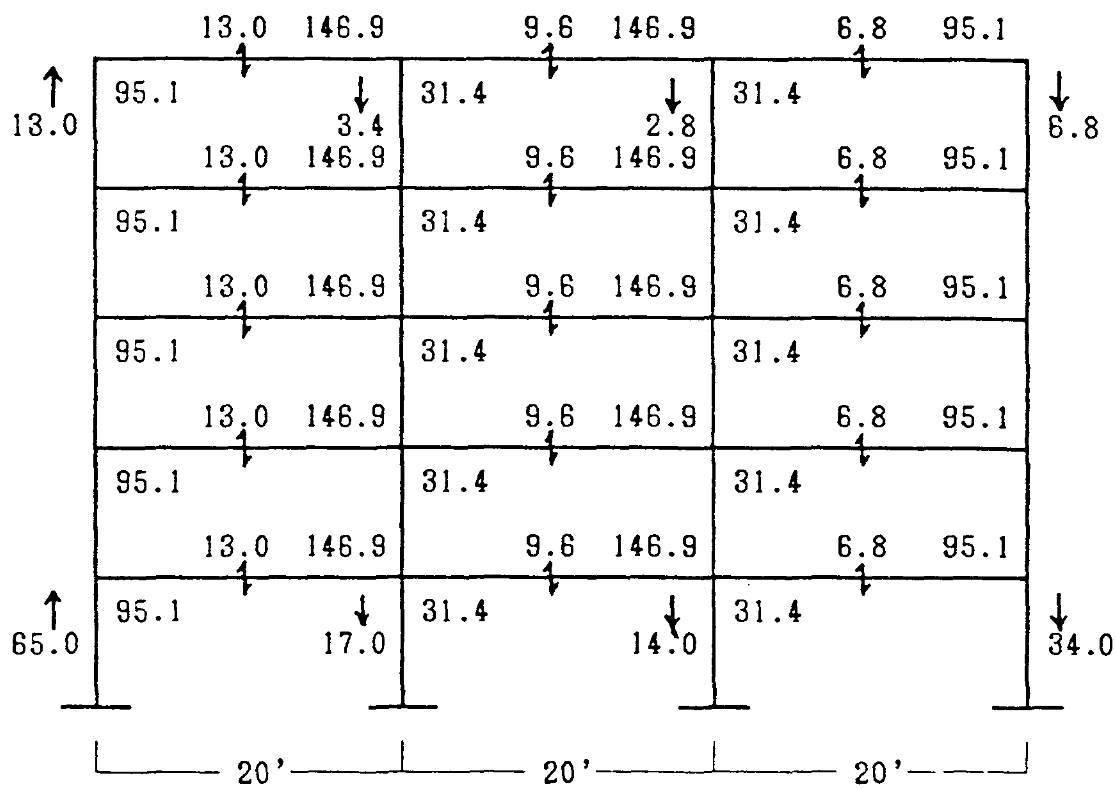


Fig. D-4 Assumed Mechanism

i) 1st floor exterior column (windward)

$Pu = 0.75 \times 211.5 - 65.0 = 93.6$ kips

$Pu / Ag = 93.6 / (16x16) = 0.37$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, ϕ Mn / Ag h = 0.35 ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.37 = 0.72$

$Mp = Mn = 0.35 \times Ag\ h / \phi = 0.35 \times 16^3 / 0.72 / 12 = 165.9$ kips-ft

ii) 1st floor interior column

$Pu = 358.9 + 17.0 = 375.9$ kips

$Pu / Ag = 375.9 / (16x16) = 1.47$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, ϕ Mn / Ag h = 0.41 ksi, $\quad \phi = 0.7$

$Mp = Mn = 0.41 \times Ag\ h / \phi = 0.41 \times 16^3 / 0.7 / 12 = 199.9$ kips-ft

iii) 1st floor exterior column leeward)

$Pu = 0.75 \times 211.5 + 34.0 = 192.6$ kips

$Pu / Ag = 192.6 / (16x16) = 0.75$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, ϕ Mn / Ag h = 0.44 ksi, $\quad \phi = 0.7$

$Mp = Mn = 0.44 \times Ag\ h / \phi = 0.44 \times 16^3 / 0.7 / 12 = 214.6$ kips-ft

iv) 5th floor exterior column

$Pu = 0.75 \times 42.3 - 13.0 = 18.7$ kips

$Pu / Ag = 18.7 / (16x16) = 0.07$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, ϕ Mn / Ag h = 0.26 ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.07 = 0.87$

$Mp = Mn = 0.26 \times Ag\ h / \phi = 0.26 \times 16^3 / 0.87 / 12 = 102.0$ kips-ft > 95.1

v) 5th floor interior column

$Pu = 0.75 \times 95.7 + 2.8 = 74.6$ kips

$Pu / Ag = 74.6 / (16x16) = 0.29$ ksi $\quad (\rho_g = 0.012)$

From Chart of Column, ϕ Mn / Ag h = 0.32 ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.29 = 0.755$

$Mp = Mn = 0.32 \times Ag\ h / \phi = 0.32 \times 16^3 / 0.755 / 12 = 144.7$ kips-ft

* Internal work

$$\Sigma Mp\ \theta = (95.1x10 + 146.9x8 + 31.4x8 + 144.7x2 + 199.9x2 + 165.9 + 214.6)\ \theta$$
$$= 3447.1\ \theta$$

* Calculation of base shear capacity Qu of each frame for earthquake loading

$W_{ext} = W_{int} \qquad 426\ P\theta = 3447.1\ \theta \qquad P = 8.09$ kips

$Qu = \Sigma P_i = (2.90 + 2.64 + 2.10 + 1.55 + 1.0) \times 8.09 = 82.4$ kips

* Calculation of base shear capacity Qu of each frame for wind loading

$W_{ext} = W_{int}$ $\quad$ $187\, P\theta = 3447.1\, \theta$ $\quad$ $P = 18.4$ kips

$Qu = \Sigma P_i = (0.67 + 1.29 + 1.27 + 1.12 + 1.0) \times 18.4 = 98.4$ kips

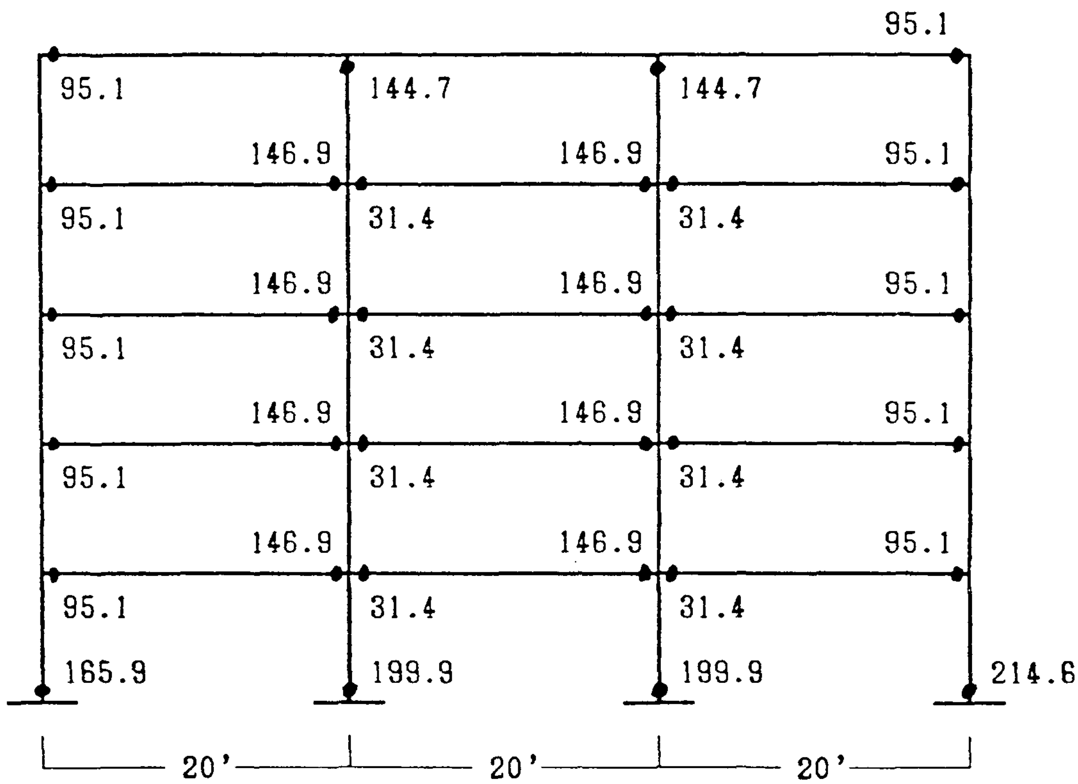


Fig. D-5 Final Mechanism

b) Soil condition S2

* Negative plastic moment Mp of slab ends

i) At exterior slab ends : Mp = 91.5 / 0.9 = 101.7 kips-ft
ii) At interior slab ends : Mp = 143.6 / 0.9 = 159.6 kips-ft

* Positive plastic moment Mp of slab ends

Assuming one-half of the mid-span reinforcements are extended to the interior supports and 100% of the mid-span reinforcements are extended to the exterior supports (see Fig. C-10), the positive plastic moment Mp of the slab ends can be calculated as follows :

i) At exterior slab ends : $l_{d(prov.)} / l_{d(full)} = 14 / 12 > 1.0$

Mp = 85.6 / 0.9 = 95.1 kips-ft

ii) At interior slab ends : $l_{d(prov.)} / l_{d(full)} = 11 / 12 < 1.0$

Mp = 61.7 / 2 / 0.9 x 11/12 = 31.4 kips-ft

* Calculation of plastic moment of column

Assuming the additional axial force due to the failure mechanism can be calculated from the assumed mechanism (Fig. D-6) and can be estimated without the load factor, Mp of a column will be derived from the column interaction diagram in Fig. C-11 where ρ_g of a column=0.012 (see Section C-5 h).

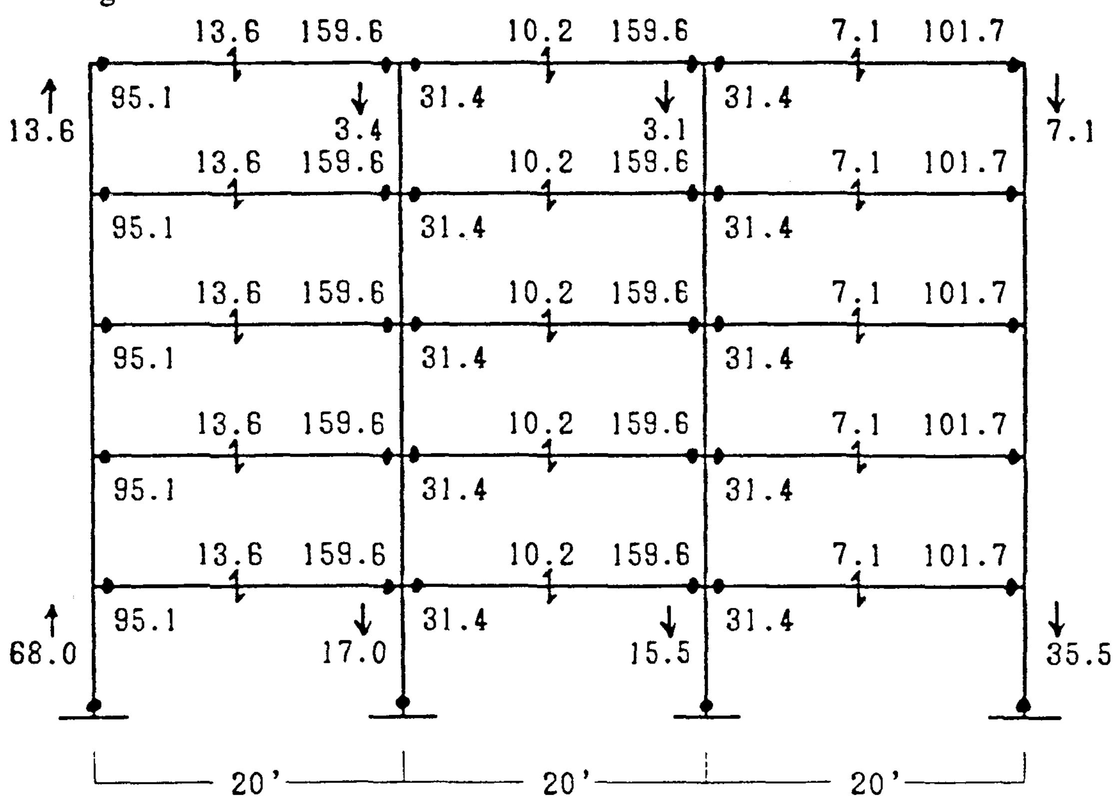


Fig. D-6 Assumed Mechanism

i) 1st floor exterior column (windward)

$Pu = 0.75 \times 211.5 - 68.0 = 90.6$ kips

$Pu / Ag = 90.6 / (16x16) = 0.35$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.32$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.35 = 0.725$

$Mp = Mn = 0.32 \times Ag\, h / \phi = 0.32 \times 16^3 / 0.725 / 12 = 150.7$ kips-ft

ii) 1st floor interior column

$Pu = 358.9 + 17.0 = 375.9$ kips

$Pu / Ag = 375.9 / (16x16) = 1.47$ ksi $(\rho_g = 0.0185)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.51$ ksi, $\phi = 0.7$

$Mp = Mn = 0.51 \times Ag\, h / \phi = 0.51 \times 16^3 / 0.7 / 12 = 248.7$ kips-ft

iii) 1st floor exterior column (leeward)

$Pu = 0.75 \times 211.5 + 35.5 = 194.1$ kips

$Pu / Ag = 194.1 / (16x16) = 0.76$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.44$ ksi, $\phi = 0.7$

$Mp = Mn = 0.44 \times Ag\, h / \phi = 0.44 \times 16^3 / 0.7 / 12 = 214.6$ kips-ft

iv) 5th floor exterior column

$Pu = 0.75 \times 42.3 - 13.6 = 18.1$ kips

$Pu / Ag = 18.1 / (16x16) = 0.07$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.27$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.07 = 0.865$

$Mp = Mn = 0.27 \times Ag\, h / \phi = 0.27 \times 16^3 / 0.865 / 12 = 106.5$ kips-ft > 101.7

v) 5th floor interior column

$Pu = 0.75 \times 95.7 + 3.1 = 74.9$ kips

$Pu / Ag = 74.9 / (16x16) = 0.29$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.32$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.29 = 0.755$

$Mp = Mn = 0.32 \times Ag\, h / \phi = 0.32 \times 16^3 / 0.755 / 12 = 144.7$ kips-ft

* Internal work

$$\Sigma\, Mp\, \theta = (95.1x5 + 101.7x5 + 159.6x8 + 31.4x8 + 144.7x2 + 248.7x2 + 150.7 + 214.6)\, \theta = 3664.1\, \theta$$

* Calculation of base shear capacity Qu of each frame for earthquake loading

$W_{ext} = W_{int}$ $426\, P\theta = 3664.1\, \theta$ $P = 8.60$ kips

$Qu = \Sigma\, P_i = (2.90 + 2.64 + 2.10 + 1.55 + 1.0) \times 8.60 = 87.6$ kips

* Calculation of base shear capacity Qu of each frame for wind loading

$W_{ext} = W_{int}$ $\quad$ $187\,P\theta = 3664.1\,\theta$ $\quad$ $P = 19.6$ kips

$Qu = \Sigma P_i = (0.67 + 1.29 + 1.27 + 1.12 + 1.0) \times 19.6 = 104.9$ kips

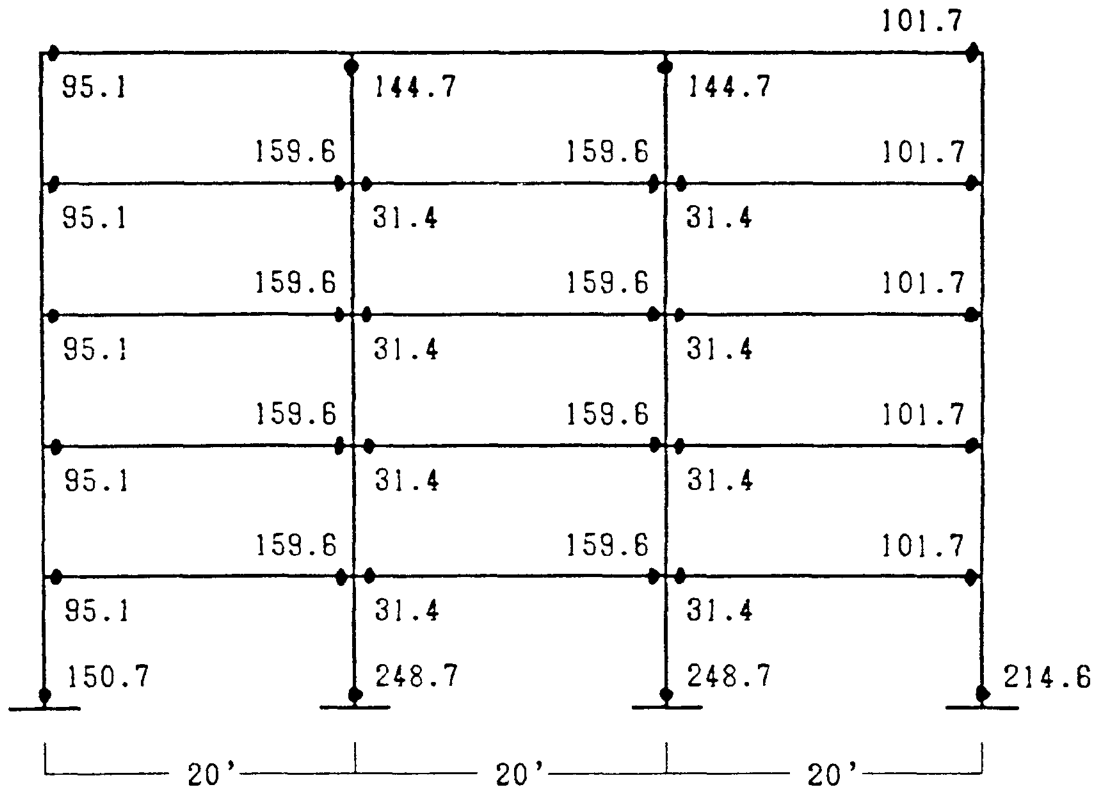


Fig. D-7 Final Mechanism

c) Soil condition S3

* Negative plastic moment Mp of slab ends

i) At exterior slab ends : Mp = 103.3 / 0.9 = 114.8 kips-ft
ii) At interior slab ends : Mp = 154.9 / 0.9 = 172.1 kips-ft

* Positive plastic moment Mp of slab ends

Assuming one-half of the mid-span reinforcements are extended to the interior supports and 100% of the mid-span reinforcements are extended to the exterior supports (see Fig. C-10), the positive plastic moment Mp of the slab ends can be calculated as follows :

i) At exterior slab ends : $l_{d(prov.)} / l_{d(full)} = 14 / 12 > 1.0$

Mp = 85.6 / 0.9 = 95.1 kips-ft

ii) At interior slab ends : $l_{d(prov.)} / l_{d(full)} = 11 / 12 < 1.0$

Mp = 61.7 / 2 / 0.9 x 11/12 = 31.4 kips-ft

* Calculation of plastic moment of column

Assuming the additional axial force due to the failure mechanism can be calculated from the assumed mechanism (Fig. D-8) and can be estimated without the load factor, Mp of a column will be derived from the column interaction diagram in Fig. C-11 using ρ_g of column =0.012 (see section C-5 h).

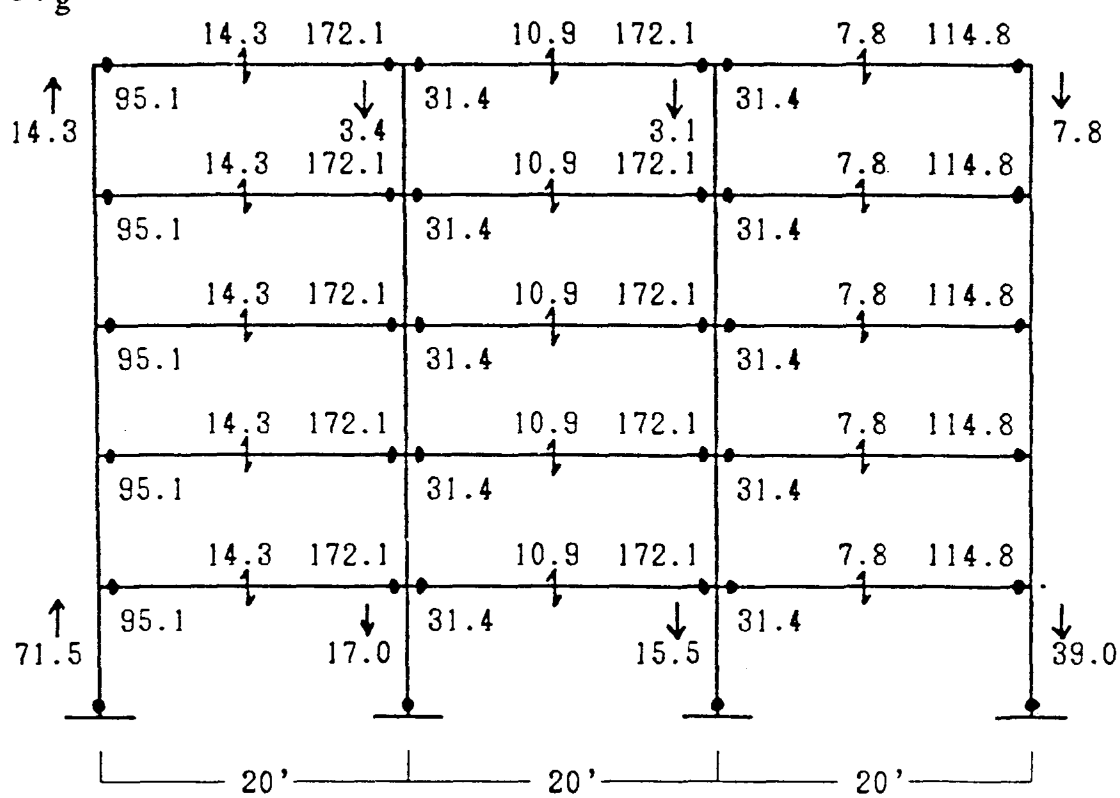


Fig. D-8 Assumed Mechanism

i) 1st floor exterior column (windward)

$Pu = 0.75 \times 211.5 - 71.5 = 87.1$ kips

$Pu / Ag = 87.1 / (16x16) = 0.34$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.33$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.34 = 0.73$

$Mp = Mn = 0.33 \times Ag\, h / \phi = 0.33 \times 16^3 / 0.73 / 12 = 154.3$ kips-ft

ii) 1st floor interior column

$Pu = 358.9 + 17.0 = 375.9$ kips

$Pu / Ag = 375.9 / (16x16) = 1.47$ ksi $(\rho_g = 0.023)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.57$ ksi, $\phi = 0.7$

$Mp = Mn = 0.57 \times Ag\, h / \phi = 0.57 \times 16^3 / 0.7 / 12 = 277.9$ kips-ft

iii) 1st floor exterior column (leeward)

$Pu = 0.75 \times 211.5 + 39.0 = 197.6$ kips

$Pu / Ag = 197.6 / (16x16) = 0.77$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.43$ ksi, $\phi = 0.7$

$Mp = Mn = 0.43 \times Ag\, h / \phi = 0.43 \times 16^3 / 0.7 / 12 = 209.7$ kips-ft

iv) 5th floor exterior column

$Pu = 0.75 \times 42.3 - 14.3 = 17.4$ kips

$Pu / Ag = 17.4 / (16x16) = 0.07$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.27$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.07 = 0.865$

$Mp = Mn = 0.27 \times Ag\, h / \phi = 0.27 \times 16^3 / 0.865 / 12 = 106.5$ kips-ft

v) 5th floor interior column

$Pu = 0.75 \times 95.7 + 3.1 = 74.9$ kips

$Pu / Ag = 74.9 / (16x16) = 0.29$ ksi $(\rho_g = 0.012)$

From Chart of Column, $\phi\, Mn / Ag\, h = 0.32$ ksi

$\phi = 0.9 - 0.2 / 0.4 \times 0.29 = 0.755$

$Mp = Mn = 0.32 \times Ag\, h / \phi = 0.32 \times 16^3 / 0.755 / 12 = 144.7$ kips-ft

* Internal work

$$\Sigma Mp\, \theta = (95.1x5 + 114.8x4 + 172.1x8 + 31.4x8 + 144.7x2 + 277.9x2 + 106.5 + 154.3 + 209.7)\, \theta = 3878.4\, \theta$$

* Calculation of base shear capacity Qu of each frame for earthquake loading

$W_{ext} = W_{int}$ $\quad 426\, P\theta = 3878.4\, \theta$ $\quad P = 9.10$ kips

$Qu = \Sigma P_i = (2.90 + 2.64 + 2.10 + 1.55 + 1.0) \times 9.10 = 92.7$ kips

* Calculation of base shear capacity Qu of each frame for wind loading

$W_{ext} = W_{int}$ $187\,P\theta = 3878.4\,\theta$ $P = 20.7$ kips

$Qu = \Sigma P_i = (0.67 + 1.29 + 1.27 + 1.12 + 1.0) \times 20.7 = 110.7$ kips

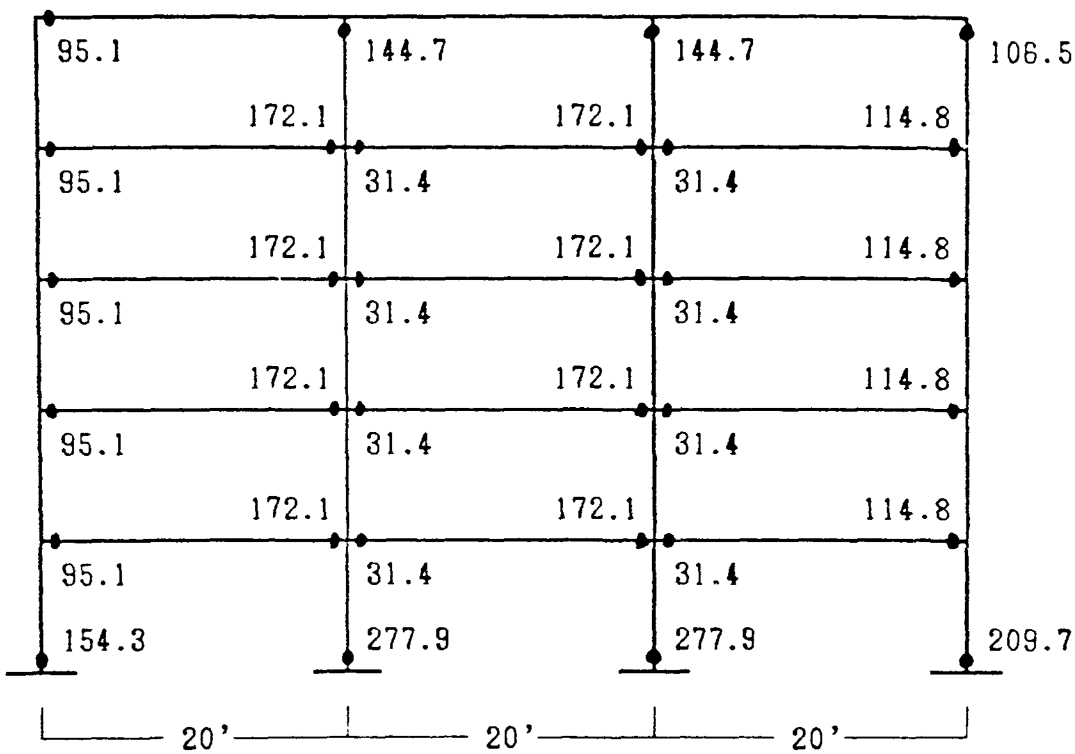


Fig. D-9 Final Mechanism

D-4 Calculation of Ultimate Capacity for Earthquake (Seismic Zone 1)

Since structural members in seismic zone 1 are equal to those under wind loading (see Tables C-4, C-5, C-6 and C-7), internal work and mechanism of collapse in zone 1 for soil conditions S1, S2 and S3 are same as those under wind loading (see Section D-2).

* Internal work

From section D-2, $\Sigma Mp\ \theta = 3347.1\ \theta$

* Calculation of base shear capacity Qu of each frame for earthquake loading

$W_{ext} = W_{int}$ $426\ P\theta = 3347.1\ \theta$ $P = 7.86$ kips

$Qu = \Sigma P_i = (2.90 + 2.64 + 2.10 + 1.55 + 1.0) \times 7.86 = 80.1$ kips

* Calculation of base shear capacity Qu of each frame for wind loading

$W_{ext} = W_{int}$ $187\ P\theta = 3347.1\ \theta$ $P = 17.9$ kips

$Qu = \Sigma P_i = (0.67 + 1.29 + 1.27 + 1.12 + 1.0) \times 17.9 = 95.8$ kips

D-5 Summary of Ultimate Structural Capacity

Since the ultimate capacity is dependent on the external load, the structural ultimate capacity of each frame can be derived as in the following table, using plastic analysis[11].

Table D-2 Summary of Ultimate Capacity of Each Frame

	W	E-1 (S1)	E-2 (S2)	E-3 (S3)
	Seismic Zone 2			
Q_{uE} (kips)	---	82.4	87.6	92.7
Q_{uW} (kips)	95.8	98.4	104.9	110.7
	Seismic Zone 1			
Q_{uE} (kips)	---	80.1	80.1	80.1
Q_{uW} (kips)	95.8	95.8	95.8	95.8

Notes;

W : designed for Wind only

E-1 (S1) : designed for Earthquake under S1 in addition to wind

E-2 (S2) : designed for Earthquake under S2 in addition to wind

E-3 (S3) : designed for Earthquake under S3 in addition to wind

Q_{uE} : Ultimate Capacity of Each Frame at 1st Floor for Eathquake Loading

Q_{uW} : Ultimate Capacity of Each Frame at 1st Floor for Wind Loading

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