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PROGRESSIVE COLLAPSE

OF

MULTISTORY BUILDINGS

- A CASE STUDY -

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TABLE OF CONTENTS

LIST OF FIGURES

— • —

LIST OF TABLES

— • —

ABSTRACT

In this case study, analyses based on a linear finite element model are used to investigate the progressive collapse behaviour of a braced, 10-story steel building. The building is 96ft by 78ft in plan and 123ft in height.

Formulations for strength of the individual structural elements of the building (columns, beams, braces, etc.) are taken from the 1986 AISC-LRFD specifications. Six cases, with one column removed in each, are investigated under the collapse loading.

The results show that the progressive collapse behaviour of a multistory building may be greatly enhanced by adequate bracing. They also indicate that floor slabs can add a significant amount of stiffness to the whole structure, and aid in redistributing loads after an initial failure.

I. INTRODUCTION

In 1968, one corner of the 22-story Ronan Point apartment building in London, England, collapsed almost completely, following a gas explosion in the kitchen of an apartment on the 18th floor. The building was constructed using precast reinforced concrete panels, and the explosion blew out an exterior wall panel (figure 1.1a). This resulted in a loss of support of the stories above, which caused a collapse chain reaction upwards to the roof (figure 1.1b). Debris falling from the stories above caused the stories below to collapse in a similar chain reaction almost to the ground level (figure 1.1c). This type of chain reaction, where failure of members propagates through a major portion of a structure, following damage to a relatively small portion of it, has been termed "progressive collapse" [4].

A report [9] was issued after the collapse, by the Commission of Inquiry, in which it was concluded that the building met governing building standards but was not "an acceptable building". According to the Commission, the structure lacked the capability to redistribute loads in case of an abnormal loading, i.e., "alternate paths". The gas explosion initiating the collapse was termed an abnormal loading because it is a loading type not considered in regular design.

Following the report on the collapse, a circular [10] was issued by the Ministry of Housing and Local Government which required multistory buildings to be designed to provide either an alternate load path in the event of the loss of a single critical member, or sufficient local resistance to withstand the effects of a 5 psi pressure (based on a gas-type explosion). Local resistance means that a structural element possesses enough reserve capacity to withstand abnormal loads in its immediate vicinity. The design pressure of 5 psi was selected with the intention of preventing the initial failure required for initiation of a progressive collapse. The alternate path approach, on the other hand, would allow the initial failure to occur but would prevent the spread of damage such that progressive collapse could not occur. Both approaches were heavily criticized within and outside the U.K. for being too conservative and costly. Particularly the alternate path approach was seen as being too complex [11] and illogical. It was argued that in case of an abnormal event (such as a gas explosion), more than one critical load carrying member could be removed (damaged). Some critics argued that the alternate path method was too severe a requirement since statically determinate structures with no alternate path have long been designed and built by engineers, and are still standing [12].

Notwithstanding these criticisms, a progressive collapse design requirement was incorporated into the Canadian code in 1970 [13], and The City of New York amended its building code in 1973 [14] to require that progressive collapse resistance be provided by either the local resistance or the alternate path method. The American National Standards Institute has included guidelines for general structural integrity in its standard ANSI 58.1-1982 [15].

The ANSI 58.1-1982 standard describes "General Structural Integrity" as follows: "... structures ... may suffer local damage ... In recognition of this, buildings and structural systems shall possess general structural integrity, which is the quality of being able to sustain local damage with the structure as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage. ... through an arrangement ... that gives stability ... combined with the provision of sufficient continuity and energy absorbing capacity (ductility) ... to transfer loads from any locally damaged region to adjacent regions capable of resisting these loads without collapse." The standard gives guidelines, as to how general structural integrity can be achieved, in its appendix A1.3.

The purpose of the reported investigation is to analyse the progressive collapse behaviour of a multi-story building using the Alternate Path Method [4]. The building used in this case study is a ten-story steel frame consisting of 3 bays (30-24-24 feet) in the short direction and 4 bays (24 feet each) in the long direction. All bents in both directions are braced through the full height in one bay. A detailed discussion of the building frame is given in chapter 3. Three-dimensional analyses are done and the structure is evaluated within the framework of the regulations of the 1986 AISC-LRFD specifications [6].

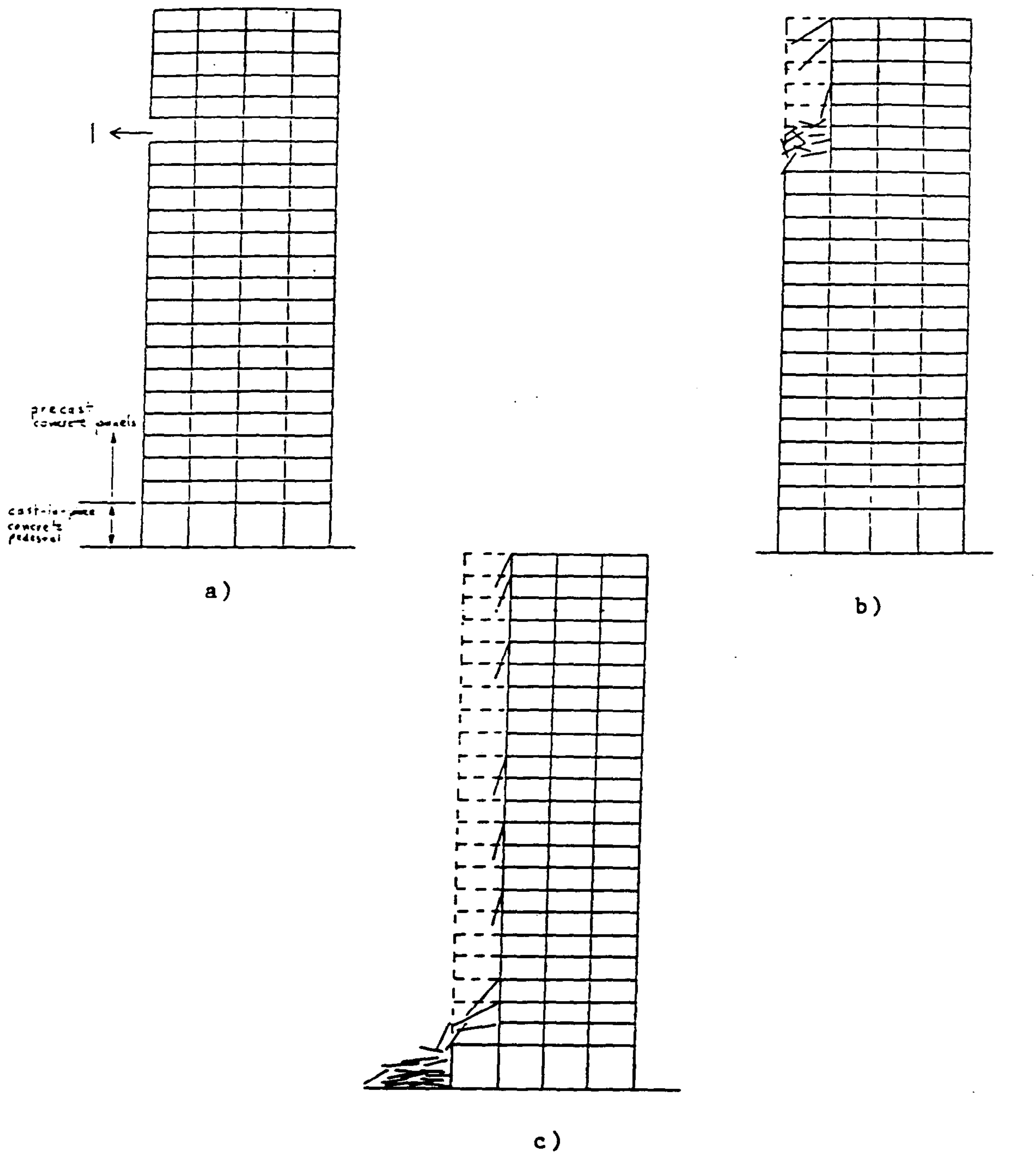


Fig.1.1 : Collapse of one corner of the Ronan Point apartment building in London, England.

II. PROGRESSIVE COLLAPSE AND THE ALTERNATE PATH METHOD

Progressive collapse has been defined by Leyendecker and Ellingwood [4] as "a chain reaction or propagation of failures following damage to a relatively small portion of a structure". In other words, for a collapse to be classified as progressive collapse, following criteria must be met:

- There must be an initiating event (abnormal load) which results in the failure of one or more primary structural elements,
- the initial damage must be small compared to the whole structure,
- a chain reaction of failures must follow,
- the result must be the destruction of either the complete structure or a large portion of it.

The ANSI 58.1-1982 [15] standard realizes that "... it is impractical for a structure to be designed to resist general collapse caused by gross misuse of a large part of the system or severe abnormal loads acting directly on a large portion of it. However, precautions can be taken in the design of structures to limit the effects of local collapse, that is, to prevent progressive collapse ...".

The standard also gives examples to distinguish between general collapse and limited local collapse. An instance of a general collapse would be the demolition of an entire building by a high-energy bomb. The failure of a column in a 1-, 2-, 3-, or even 4-column structure could also lead to a general collapse, since the failed column would be a significant part of the structure and not just a small portion of it.

Examples of limited local collapse are given in [15] as "... the containment of damage to adjacent bays and stories following the destruction of one or two neighboring columns in a multibay structure [or] the restriction of damage to portions of two or three stories of a higher structure following the failure of a section of bearing wall in one story ...".

Contrary to the general opinion that prevailed within and outside the U.K. after the Ronan Point collapse — that both the alternate path method and the local resistance method were too conservative and costly — the ANSI 58.1-1982 states: "Experience has demonstrated that the principle of taking precautions in design to limit the effects of local collapse is realistic and can be satisfied economically. From a public-safety viewpoint it is reasonable to expect all multistory buildings to possess general structural integrity comparable to that of properly designed, conventional framed structures ...".

Two approaches exist that can be used to satisfy the performance requirement stated by Ellingwood and Leyendecker [4], which requires that structures should have inherent capability to limit the spread of the local damage regardless of the cause. The approaches, which are also adopted in the ANSI 58.1-1982 standard, are [4,15]:

- direct design: explicit consideration of resistance to progressive collapse during the design process through,
  1) alternate path method:
     allows local failure to occur but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted,
  2) specific local resistance method:
     seeks to provide sufficient strength to a structural member to resist failure from abnormal loads in its immediate vicinity due to accidents or misuse.

- indirect design: implicit consideration of resistance to progressive collapse during the design process through the provision of minimum levels of strength, continuity and ductility.

The latter approach is also sometimes called "General Structural Integrity" for which ANSI 58.1-1982 provides basic guidelines.

For the case study at hand, the alternate path method was selected as the method with which to check the performance of the building. It should be noted that for the alternate path method to be applied, there is no need to know the nature of the abnormal load that causes the initial damage to the critical primary structural member. However, in order to predict the actual extent of the initial damage, a thorough understanding of the nature and magnitude of abnormal loads is essential.

For our case, the initial damage is simulated by simply removing one of the critical primary structural members. The structure is then repeatedly analysed under the progressive collapse load combination, each time with one of the critical members removed, and examined if there are any other members in the structure that can not bear the forces induced by the redistributed loads due to the initial damage. The results of the analyses will indicate if the structure possesses enough continuity and ductility to redistribute the forces without the initiation of a progressive collapse sequence. This method is also suggested in [4] as follows: "Compliance may be determined by assuming that the primary structural elements are incapable of carrying load, one element at a time, and evaluating the resulting structural behaviour." Primary structural elements are defined in [4] as:

- major load carrying beams
- floor slabs between supports
- columns
- bearing wall panels

Similar definitions are given in [16] and [17]. Here, attention will be focused on columns. Of particular interest are the columns at the lower floor levels. One column is removed at a time, then the structure is analysed and the forces in the members checked against their capacity.

By comparing the force in each member with its capacity, a "response ratio" can be established. The response ratio is defined as the ratio of force acting on the member to strength of the member. A generic equation can be written, that can be used in conjunction with any type of structural action, as:

$$RR = \frac{F_u}{\phi R'}$$

where,

$RR =$ response ratio,

$F_u =$ factored (ultimate) load acting on the member,

$\phi =$ resistance factor (≤ 1.0),

$R' =$ nominal resistance.

Members with a response ratio greater than one will most probably fail if the collapse sequence is allowed to go further. This would cause further redistribution of loads, and again additional members would fail, which would then represent a progressive collapse sequence. If, however, all members have a response ratio less than one, progressive collapse is less likely to occur.

The formulation of the design loads to be used in the alternate path method has been derived in [4] by using statistical methods to take into account duration and frequency of occurrence of different types of load. Equations 2.1 and 2.2 represent the criteria for the remaining structural members to be considered safe, once a critical member is damaged. The right sides correspond to F_u mentioned above. When gravity and wind load effects have the same sense,

$$\phi R' \quad \geq \quad D + 0.5 \cdot L_{ANSI} + 0.2 \cdot W_{ANSI} \tag{2.1}$$

When they oppose one another,

$$\phi R' \quad \geq \quad D - 0.3 \cdot W_{ANSI} \tag{2.2}$$

where D = design dead load

L_{ANSI} = design live load as specified in ANSI 58.1

W_{ANSI} = design wind load as specified in ANSI 58.1

and ϕ and R' are defined as before. Note that if both sides of the equations are divided by ϕR', it can be seen that the criteria for safety is a response ratio less than one.

The ϕ factor depends on the material being used for a specific member, and on the governing structural action (bending, shear, axial force, etc.). Thus, the formulas are not limited to one particular material or type of member, but are rather general formulas.

III. SELECTION AND MODIFICATION OF A FRAME FOR THE ANALYSIS

III.1. Selection Of The Frame

One of the key tasks in the initial phases of this analysis was to find an appropriate building frame that could be used. In order to have a realistic approach to the concept of progressive collapse, use was made of a frame that had been designed in 1965 by the faculty of the Civil Engineering department at Lehigh University [1,2]. The lecture notes "Plastic Design of Multi-Story Frames" included three frames, ranging from 2 to 24 stories, of which frame "B" (10 stories) was chosen to be used in this study. Four versions of the frame had been designed using combinations of ultimate strength/working stress methods and braced/unbraced frames. The braced frame, designed using the ultimate strength concept, was selected for this analysis. Figure 3.1 shows the frame with abbreviations for the final member sizes that had been used in the original design. The full designations of the rolled steel sections are listed in table 3.1.

The assumptions underlying the analysis of the frame were following [1,2]:

- The bent spacing between successive frames is 24 ft.
- The beam-to-column connections are moment connections, and they are capable of transmitting all forces and moments acting on them.
- The weight of the exterior walls is assumed to be applied at the centroid of the exterior columns.
- All columns are adequately braced in the weak (out-of-plane) direction.
- Braces act only in tension.
- All columns, beams and braces are rolled sections made of A36 steel (F_y=36ksi).

The service loads used, were:

Dead load:	Floors	$\omega = 80$ psf
	Roof	$\omega = 60$ psf
	Exterior walls dead weight = 45 psf	
Live load:	Floors	$\omega = 80$ psf
	Roof	$\omega = 30$ psf
Wind load:	Lateral pressure = 20 psf	

In order to use the 1986 AISC-LRFD specifications and the wide flange sections listed in the 1986 AISC manual, it was necessary to redesign the frame using today's load factors and design requirements. The next section describes the procedure followed.

III.2. Redesigning The Frame For LRFD Compatibility

III.2.1. Software and load combinations used

In order to make use of a more realistic design conforming to the 1986 AISC-LRFD specifications, the plane frame was redesigned using load combinations as specified by the LRFD, rather than the combinations originally used for the allowable stress design in 1965 [1,2]. The geometry and service loads, however, were used exactly as given.

The new design was made using SODA [3], a state-of-the-art software package capable of designing planar steel frames and trusses according to 1986 AISC-LRFD specifications. The following load combinations were used:

1.) $1.4 \cdot D$ (A4-1)

2.) $1.2 \cdot D + 1.6 \cdot L + 0.5 \cdot L_r$ (A4-2)

3.) $1.2 \cdot D + 1.3 \cdot W_l + 0.5 \cdot L + 0.5 \cdot L_r$ (A4-4)

4.) $1.2 \cdot D + 1.3 \cdot W_r + 0.5 \cdot L + 0.5 \cdot L_r$ (A4-4) where. D = dead load

L = live load

L_r = roof live load

W_l = wind load from left

W_r = wind load from right

The magnitude of the different loads is given in section III.1.

The new design of the plane frame and the design of a 3-D frame was accomplished in three steps as explained in the following sections.

III.2.2. Redesigning plane frame "B" of the 1965 Lehigh conference

The sections that had been selected for the final design of the frame in 1965 were used as initial member sizes in this design, which were then checked by the SODA package and, if found inadequate, replaced by appropriately sized beams, columns or braces. Where a given size didn't exist in the 1986 AISC manual for the initial guess, the member was replaced by the nearest possible member from the AISC manual.

The grouping of the different member sizes was kept the same as it was in the 1965 design. In other words, column sizes change only at every other floor level (two-story tier construction) and the beams change their size only once at the seventh floor level in the braced bay (see figures 3.1 and 3.2). As an example, the columns of the first and second floor on column line A had been chosen the same as the columns of the third and fourth floor on column line B (14 WF 119). This kind of grouping was kept, although the member size in any given group was allowed to change as needed for strength. An exception was made for the braces. Originally, all braces were the same size. Here, the braces of the first floor were chosen to be of a different size than the upper floors, thereby allowing a reduction in the size of the upper story braces.

Out of these considerations, a preliminary design of the short direction plane frame was arrived at, as shown in figure 3.2. The member sizes for each group are listed in table 3.2.

III.2.3. Design of braces for the long direction

The next step towards a "three-dimensional design" was to select braces for the long direction of the 3-D frame. The long direction of the 3-D frame was chosen to consist of four bays of 24 ft width each. Braces were provided in one bay. Figure 3.3 shows the long direction with dimensions and column-line numbering.

It is a common design practice to assume that all dead and live load is carried by a succession of plane frames (in this case, the short-direction frames), and that in the long direction the beams do not carry any gravity load. The beams in this direction are usually attached to the columns by shear connections, and they are all chosen to be the same size since they do not carry gravity loads. This leaves us with the task of selecting appropriate members for the bracing and the beams.

In order to have a rational basis for the selection of these members, the long direction was designed for wind load. The wind pressure was assumed to be the same as in the short direction, that is 20 psf. Since the spacing of the bents in the long direction is not constant as opposed to the short direction, the nodal loads had to be calculated for each bent separately. A load factor of 1.3 was used (combination A4-4) and the design was made to accomodate wind from either side of the structure. The bracing size was specified to be kept the same in all stories of each bent. The bents were then designed separately, leading to different bracing sizes in almost all of them. Table 3.3 lists the sizes of the braces that were arrived at, and the final size that was used for all long direction bents.

III.2.4. Reselection of members for short direction

The columns in stories 9 and 10 on column line C (see figure 3.4), selected as W8x31 in the preliminary design, were found to be inadequate due to wind action in the long direction and had to be replaced by heavier sections with larger moment of inertia about their minor axis. These columns were repeatedly replaced and checked for their adequacy in the short direction until an acceptable section size was arrived at. Using SODA, a section size of W14x61 was found adequate. This concludes the design of the three-dimensional frame. A summary of all the final sections used, together with all section properties used in this report, is given in table 3.4. Printouts of the input files of the design runs performed with SODA are given in Fritz Laboratory Report No. 433.8 [20].

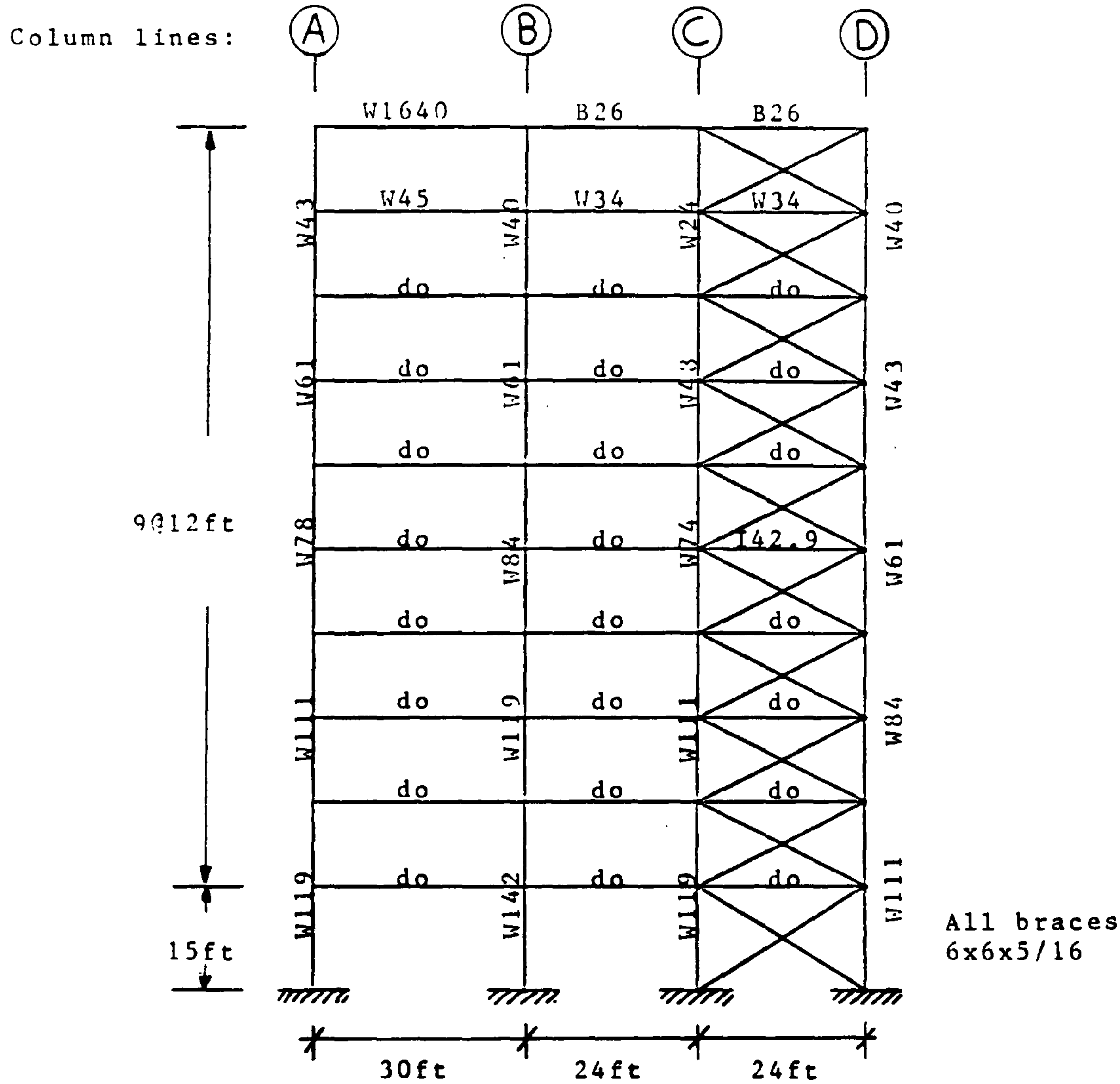
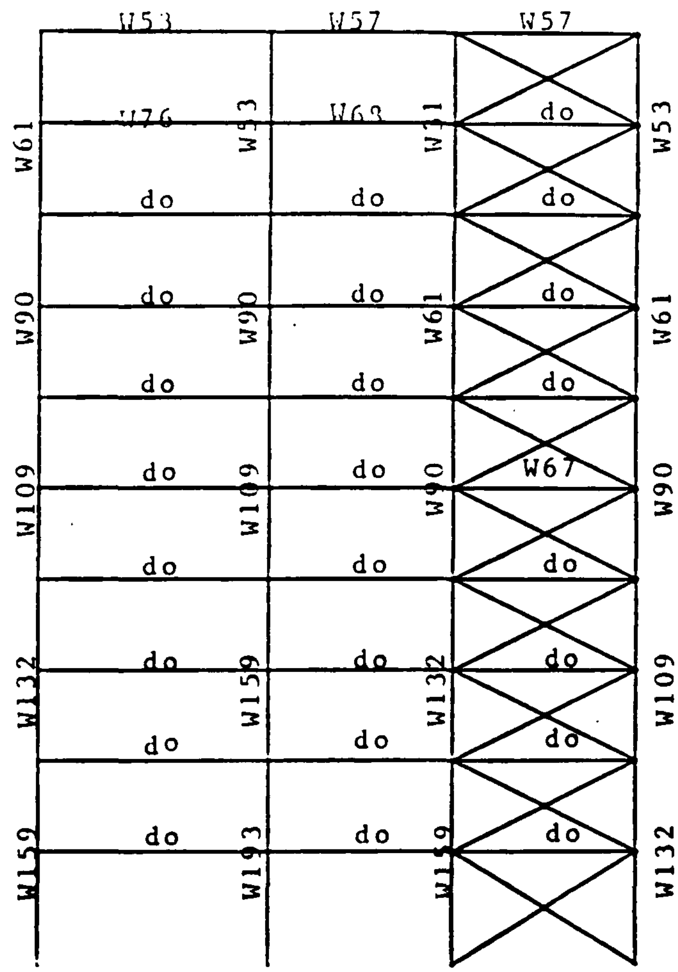


Fig.3.1: Dimensions of frame B, and rolled shapes used in the original design.



Braces in floors 2 to 10 are L8

First floor braces are 2L8

Fig.3.2: Preliminary design of plane frame in short direction (by using SODA [3]).

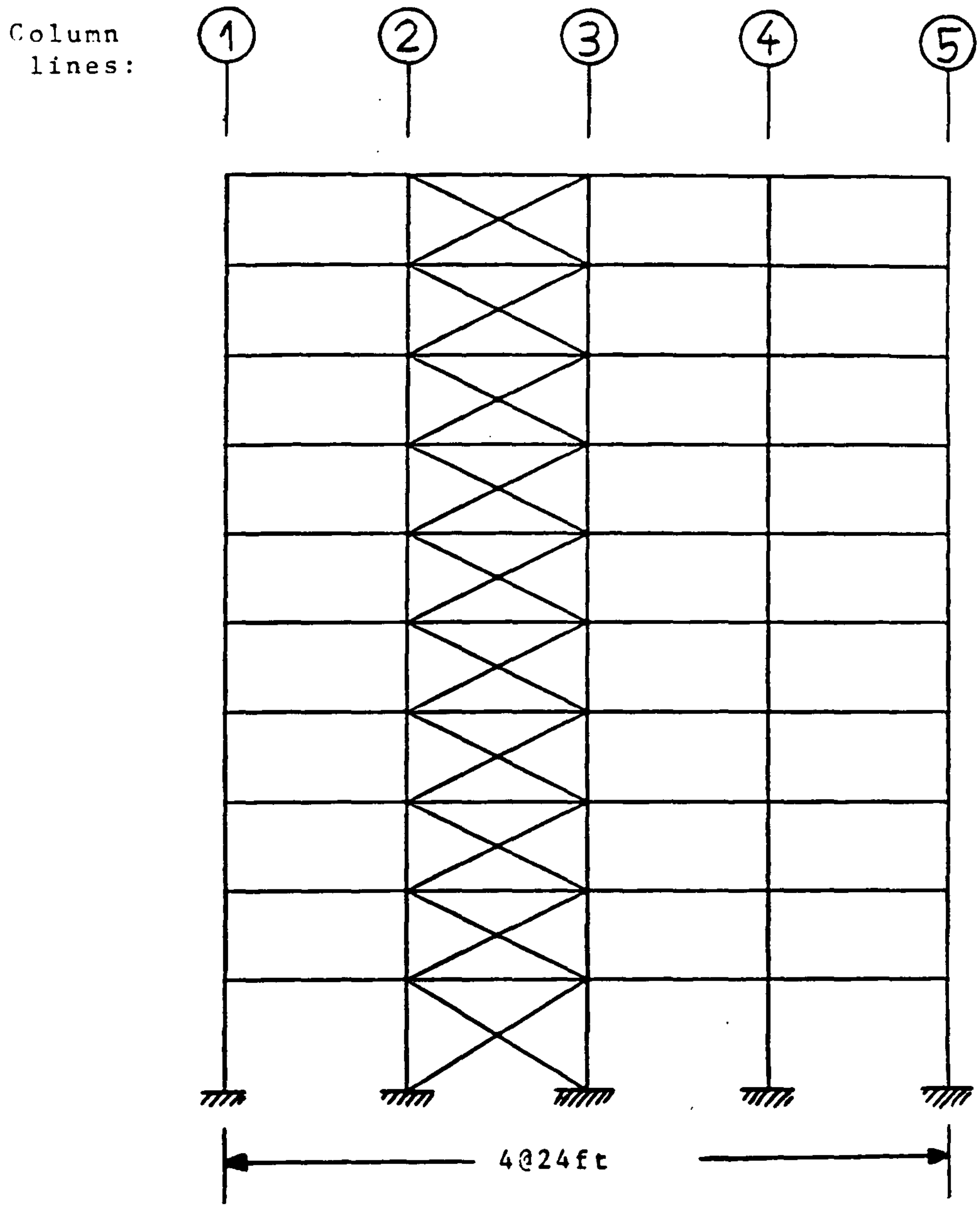


Fig.3.3: Long direction of building.

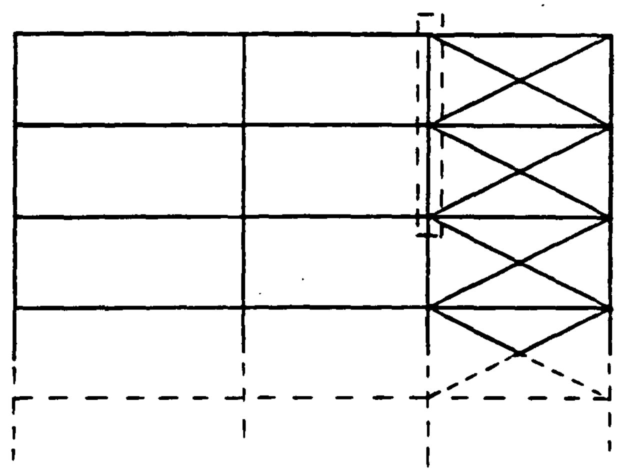


Fig.3.4: Columns enclosed by dashed box were replaced with heavier sections.

Section designation	Section size	Section designation	Section size
W142	14 WF 142	W43	14 WF 43
W119	14 WF 119	W40	12 WF 40
W111	14 WF 111	W1640	16 WF 40
W84	14 WF 84	W34	14 WF 34
W78	14 WF 78	W24	8 WF 24
W74	14 WF 74	I42.9	15 I 42.9
W61	14 WF 61	B26	16 B 26
W48	14 WF 48	L6	6x6x5/16
W45	18 WF 45		

Table 3.1: Rolled steel sections used in original "frame B" design.

Section designation	Section size	Section designation	Section size
W193	W 14 x 193	W67	W 16 x 67
W159	W 14 x 159	W61	W 14 x 61
W132	W 14 x 132	W57	W 16 x 57
W109	W 14 x 109	W53	W 12 x 53
W90	W 14 x 90	W31	W 8 x 31
W76	W 18 x 76	L 8	L 8 x 8 x 0.75
W68	W 14 x 68	2 L 8	L 8 x 8 x 0.625

Table 3.2: Rolled steel sections used in preliminary design of short direction frame.

Bent	Contributary width	Required bracing
A	15 ft	L 8 x 8 x 1.125
B	27 ft	L 8 x 8 x 1.125
C	24 ft	L 8 x 8 x 0.5
D	12 ft	L 8 x 8 x 0.5625
Final size: L 8 x 8 x 1.125 for all braces (L 8 p). W 8 x 24 for all beams (W 24).		

Table 3.3: Braces and beams for long direction.

Designation	Section	Ag in2.	d in.	tw in.	bf in.	tf in.	bf/(2*tf)	hc/tw	X1 ksi.	X2*e6 (1/ksi.)^2	J in.4	Cw in.6
Columns:												
W 193	W 14x193	56.8	15.48	0.890	15.710	1.440	5.5	12.8	5740	125	34.8	45900
W 159	W 14x159	46.7	14.98	0.745	15.565	1.190	6.5	15.3	4790	249	19.8	35600
W 132	W 14x132	38.8	14.66	0.645	14.725	1.030	7.1	17.7	4180	428	12.3	25500
W 109	W 14x109	32.0	14.32	0.525	14.605	0.860	8.5	21.7	3490	853	7.12	20200
W 90	W 14x90	26.5	14.02	0.440	14.520	0.710	10.2	25.9	2900	1750	4.06	16000
W 61	W 14x61	17.9	13.89	0.375	9.995	0.645	7.7	30.4	2720	2460	2.20	4710
W 53	W 12x53	15.6	12.06	0.345	9.995	0.575	8.7	28.1	2820	2100	1.58	3160
Beams:												
W 76	W 18x76	22.3	18.21	0.425	11.035	0.680	8.1	37.8	2180	6520	2.83	11700
W 68	W 14x68	20.0	14.04	0.415	10.035	0.720	7.0	27.5	3020	1650	3.02	5380
W 67	W 16x67	19.7	16.33	0.395	10.235	0.665	7.7	35.9	2350	4690	2.39	7300
W 57	W 16x57	16.8	16.43	0.430	7.120	0.715	5.0	33.0	2650	3400	2.22	2660
W 24	W 8x24	7.08	7.93	0.245	6.495	0.400	8.1	25.8	3020	1610	0.35	259
Braces:												
L 8	L 8x8x0.75	11.4										
L 8 p	L 8x8x1.125	16.7	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2 L 8	L 8x8x0.625	19.2										

Table 3.4: Cross-sectional properties of the rolled sections used for the collapse analyses.

Designation	Section	Ix in.4	Sx in.3	rx in.	Iy in.4	Sy in.3	ry in.	Zx in.3	Zy in.3	d*tw in.2	2*(bf*tf) in.2
Columns:											
W 193	**W 14x193**	2400	310	6.50	931	119	4.05	355	180	13.78	45.24
W 159	**W 14x159**	1900	254	6.38	748	96.2	4.00	287	146	11.16	37.04
W 132	**W 14x132**	1530	209	6.28	548	74.5	3.76	234	113	9.46	30.33
W 109	**W 14x109**	1240	173	6.22	447	61.2	3.73	192	92.7	7.52	25.12
W 90	**W 14x90**	999	143	6.14	362	49.9	3.70	157	75.6	6.17	20.62
W 61	**W 14x61**	640	92.2	5.98	107	21.5	2.45	102	32.8	5.21	12.89
W 53	**W 12x53**	425	70.6	5.23	95.8	19.2	2.48	77.9	29.1	4.16	11.49
Beams:											
W 76	**W 18x76**	1330	146	7.73	152	27.6	2.61	163	42.2	7.74	15.01
W 68	**W 14x68**	723	103	6.01	121	24.2	2.46	115	36.9	5.83	14.45
W 67	**W 16x67**	954	117	6.96	119	23.2	2.46	130	35.5	6.45	13.61
W 57	**W 16x57**	758	92.2	6.72	43.1	12.1	1.60	105	18.9	7.06	10.18
W 24	**W 8x24**	82.8	20.9	3.42	18.3	5.63	1.61	23.2	8.57	1.94	5.20
Braces:											
L 8	**L 8x8x0.75**			2.47			2.47		rz = 1.58		
L 8 p	**L 8x8x1.125**	n/a	n/a	2.42	n/a	n/a	2.42	n/a	rz = 1.56		
2 L 8	**L 8x8x0.625**			2.49			3.47		n/a		

Table 3.4 continued

IV. FORMULATIONS USED IN THE ANALYSIS OF THE BUILDING

In order to determine whether a certain structural element should be considered failed or not, it is necessary to know its limiting strength. The approach used in this study was to utilize the formulas given in the 1986 AISC-LRFD to compute the strength for the main structural action(s) (bending, shear, axial force, etc.) of columns, beams and braces. The reinforced concrete floor slabs were treated in a simplified manner using classical plate theory. The structural actions investigated in the different structural elements were:

- interaction between axial force and bending in columns,
- bending and shear force in beams,
- axial force in braces,
- bending in reinforced concrete slabs.

Four different computer programs were implemented in Fortran 77 for this purpose. They are called COLUMN, BEAM, BRACE and SLAB, respectively, and are listed in [20]. The strength of any given structural element is embedded in the corresponding programs, since it is needed to find the response ratio. The next four sections explain how the strengths of the different elements were calculated by using the 1986 AISC-LRFD specifications.

IV.1. Strength Criteria For Columns

IV.1.1. AISC-LRFD requirements

The 1986 AISC-LRFD specifications require columns to satisfy following formulas (equation numbers are from the AISC-LRFD specifications):

for $\frac{P_u}{\phi P_n} \geq 0.2$:

$$\frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \qquad \text{(H1-1a)}$$

for $\frac{P_u}{\phi P_n} < 0.2$:

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \qquad \text{(H1-1b)}$$

where ϕ_b = resistance factor for flexure = 0.90,

and $$\phi = \begin{cases} \phi_c = \text{resistance factor for axial compression} = 0.85 \\ \phi_t = \text{resistance factor for axial tension} = 0.90 \end{cases}$$

P_n is the nominal compressive axial strength when P_u is compressive, and the nominal tensile axial strength when P_u is tensile. M_n is the nominal flexural strength.

IV.1.2. Axial strength

When P_u is compressive, P_n is calculated as:

$$P_n = A_g \, F_{cr} \qquad \text{(E2-1)}$$

where, for $\lambda_c \leq 1.5$:

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_Y \qquad \text{(E2-2)}$$

and for $\lambda_c > 1.5$:

$$F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_Y \qquad \text{(E2-3)}$$

$$\text{where } \lambda_c = \frac{K \cdot l}{r \cdot \pi} \sqrt{\frac{F_Y}{E}} \qquad \text{(E2-4)}$$

and, A_g = gross area of member, in.^{2}

F_Y = specified yield stress, ksi

E = modulus of elasticity, ksi

K = effective length factor

l = unbraced length of member, in.

r = governing radius of gyration about plane of buckling, in.

Chapter C, section C.2.1 (p.6-35) of the AISC-LRFD requires that "in trusses and frames where lateral stability is provided by diagonal bracing, ..., the effective length factor K for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used." Thus, a value of 1 was used here.

When P_u is tensile, P_n is calculated as the lower value of yielding in the gross section and fracture in the net section (section D1, AISC-LRFD). In this study, it was assumed that adequate fracture strength would be provided and yielding would govern the strength. Thus,

$$P_n = F_Y A_g \qquad \text{(D1-1)}$$

Example 4.1 :

As an example, let us find the axial strength of the W14x193 section (designated as W193). Since this section is used on the first floor as well as on the second floor, two different compressive strength values are obtained, due to differences in height of the columns. The tensile strength is the same regardless of height.

Compressive strength:

Floor 1, l=180 in. :

$$\lambda_c = \frac{1 \cdot 180}{4.05 \cdot \pi} \cdot \sqrt{\frac{36}{29000}} = \frac{180}{4.05} \cdot 0.0112 = 0.50 \qquad \text{(E2-4)}$$

$$F_{cr} = 0.658^{(0.50)^2} \cdot 36 = 32.44 \text{ ksi} \qquad \text{(E2-2)}$$

$$\phi_c\ P_n = 0.85 \cdot 56.8 \cdot 32.44 = \underline{1566.4}\ ^k \qquad \text{(E2-1)}$$

Floor 2. l=144 in. :

$$\lambda_c = \frac{144}{4.05} \cdot 0.0112 = 0.40$$

$$F_{cr} = 0.658^{(0.40)^2} \cdot 36 = 33.68 \text{ ksi}$$

$$\phi_c\ P_n = 0.85 \cdot 56.8 \cdot 33.68 = \underline{1626.2}\ ^k$$

Tensile strength:

$$\begin{aligned} \phi_t\ P_n &= 0.9 \cdot F_Y \cdot A_g && \text{(D1-1)} \\ &= 0.9 \cdot 36 \cdot 56.8 \\ &= \underline{1840.3}\ ^k && \text{(for both floors)} \end{aligned}$$

The strengths of the other sections are found similarly. Table 4.1 lists the axial strength of all column sections.

IV.1.3. Flexural strength about major axis

The flexural strength of a column bent about its major axis is determined in accordance with section F1 of the AISC-LRFD. The nominal strength M_n for laterally unsupported compact section members is given in section F.1.3 as,

$$M_n = C_b\ [\ M_p - (\ M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)]\ \leq\ M_p \qquad \text{(F1-3)}$$

where $C_b = 1.75 + 1.05 \cdot \frac{M_1}{M_2} + 0.3 \cdot \left(\frac{M_1}{M_2}\right)^2 \leq 2.3,$

and M_1 and M_2 are the smaller and larger end moment, respectively, of the unbraced segment; $\frac{M_1}{M_2}$ is positive when the moments cause reverse curvature and negative when they cause single curvature;

$$L_p = \frac{300\ r_y}{\sqrt{F_Y}} \qquad \text{(F1-4)}$$

where r_y = radius of gyration about minor axis, in.,

M_p = plastic moment = $F_Y\ Z_x$

L_r = limiting laterally unbraced length

$$= \frac{r_y\ X_1}{(F_Y - F_r)} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot (F_Y - F_r)^2}} \qquad \text{(F1-6)}$$

M_r = buckling moment corresponding to L_r

$$= (F_Y - F_r) \cdot S_x \qquad \text{(F1-7)}$$

where F_r = compressive residual stress in flange

= 10 ksi for rolled shapes

S_x = major axis section modulus

X_1, X_2 = buckling factors depending on cross-sectional properties (see table 3.4)

Example 4.2 :

Let us find some of the key parameters for section W193 (W14x193):

$$L_p = \frac{300 \cdot 4.05}{\sqrt{36}} = \underline{202.5\ \text{in.}} \qquad \text{(F1-4)}$$

$$M_p = 36 \cdot 355 = \underline{12780\ \text{k-in.}}$$

$$L_r = \frac{4.05 \cdot 5740}{26} \cdot \sqrt{1 + \sqrt{1 + 125 \cdot 10^{-6} \cdot (26)^2}} = \underline{1277.5\ \text{in.}} \qquad \text{(F1-6)}$$

$$M_r = (36\text{-}10) \cdot 310 = \underline{8060\ \text{k-in.}} \qquad \text{(F1-7)}$$

Once the building is analysed, the end-moments are known. Then, C_b (which depends on the end-moments), and therefore M_n can be computed. Table 4.2 lists L_p, M_p, L_r and M_r for all column sections.

IV.1.4. Flexural strength about minor axis

For the determination of minor axis bending strength, section F.1.7 of the LRFD refers to its appendix F, where nominal strength parameters are listed in table A-F1.1 (pp. 6-94 to 6-97). The appendix states that the nominal flexural strength can be taken as the lowest value obtained according to the limit states of,

a) lateral-torsional buckling (LTB),
b) flange local buckling (FLB),
c) web local buckling (WLB).

For each limit state, M_n is determined as follows:

For $\lambda \leq \lambda_p$:

$$M_n = M_p \qquad \text{(A-F1-1)}$$

For $\lambda_p < \lambda \leq \lambda_r$:

for LTB:

$$M_n = C_b \cdot \left[M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \right] \leq M_p \qquad \text{(A-F1-2)}$$

for FLB and WLB:

$$M_n = M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{(A-F1-3)}$$

For $\lambda > \lambda_r$:

for LTB and FLB:

$$M_n = M_{cr} = S \cdot F_{cr} \qquad \text{(A-F1-4)}$$

where **M_p = minor axis plastic moment = $F_Y Z_y$**
M_r = minor axis limiting buckling moment = $F_Y S_y$
M_{cr} = buckling moment
F_{cr} = critical stress
S = section modulus

The controlling slenderness parameter λ is defined in table A-F1.1 in the LRFD as,

$$\lambda = \frac{L_b}{r_y} \text{ for LTB,}$$

$$\lambda = \frac{b}{t} = \text{flange width-thickness ratio for FLB,}$$

$$\lambda = \frac{h}{t_w} = \text{web depth-thickness ratio for WLB.}$$

The limiting values are:

$$\lambda_p = \frac{300}{\sqrt{F_y}} = \frac{300}{\sqrt{36}} = 50 \qquad \text{for LTB,}$$

$$\lambda_p = \frac{65}{\sqrt{F_y}} = \frac{65}{\sqrt{36}} = 10.8 \qquad \text{for FLB,}$$

$$\lambda_p = \frac{640}{\sqrt{F_y}} = \frac{640}{\sqrt{36}} = 106.7 \qquad \text{for WLB,}$$

and

$$\lambda_r = \frac{X_1}{(F_Y - F_r)} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot (F_Y - F_r)^2}} \qquad \text{for LTB,}$$

$$\lambda_r = \frac{141}{\sqrt{F_Y - 10}} = 27.7 \qquad \text{for FLB,}$$

$$\lambda_r = \frac{970}{\sqrt{F_Y}} = 161.7 \qquad \text{for WLB.}$$

The actual values of λ for the column sections used are listed in table 4.3. The following two examples will illustrate how the strength is computed.

Example 4.3 :

For section W193 (W14x193), since all λ's are less than the λ_p's,

$$M_n = M_p = F_Y Z_y = 36 \cdot 180 = \underline{6480.0 \text{ k-in.}} \qquad \text{(A-F1-1).}$$

Example 4.4 :

For section W61 (W14x61), since $\lambda > \lambda_p$ for the LTB limit state for this section, λ_r has to be computed. X_1 and X_2 are cross-sectional properties taken from the AISC-LRFD manual and listed in table 3.4. Thus, we have:

$$\lambda_r = \frac{2720}{(36\text{-}10)} \cdot \sqrt{1 + \sqrt{1 + 2460 \cdot 10^{-6} \cdot (36\text{-}10)^2}} = \underline{169.7}$$

Since $\lambda_p < \lambda \leq \lambda_r$, M_n is defined by equation A-F1-2 of the LRFD, and M_p and M_r have to be computed first:

$$M_p = F_Y Z_y = 36 \cdot 32.8 = \underline{1180.8 \text{ k-in.}}$$

$$M_r = F_Y S_y = 36 \cdot 21.5 = \underline{774.0 \text{ k-in.}}$$

and with C_b defined in a similar way as for the major axis, we have

$$\begin{aligned} \text{LTB-}M_n &= C_b \cdot [1180.8 - (1180.8 - 774.0) \cdot \left(\frac{58.78 - 50}{169.7 - 50}\right)] \\ &= \underline{C_b \cdot 1151.1 \text{ k-in.} \leq M_p} \end{aligned}$$

The strength of the other sections is computed in a similar way, and is listed in table 4.4.

After the axial strength and major and minor axis bending strength of the columns are established, the values are substituted in the left-hand side of the biaxial bending interaction formulas (eq. H1-1a or H1-1b). By turning these equations around and comparing to eq. 2.1 or eq. 2.2, the concept of a response ratio can easily be explained. For this purpose, let us use equations H1-1a and 2.1 (but the explanations are equally valid for equations H1-1b and 2.2).

$$1.0 \geq \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \tag{H1-1a}$$

$$\phi R' \geq \text{(forces induced by) } D + 0.5 \cdot L_{ANSI} + 0.2 \cdot W_{ANSI} \tag{2.1}$$

We see that eq. H1-1a represents a variation of eq. 2.1. Eq. 2.1 could be written as

$$1.0 \geq \frac{\text{(forces induced by) } D + 0.5 \cdot L_{ANSI} + 0.2 \cdot W_{ANSI}}{\phi R'}$$

In this case, eq. H1-1a sets the maximum tolerable response ratio automatically to 1.0. The same holds for eq. H1-1b. Since in this study an analysis rather than design is performed, however, the formulation is slightly modified to compute the response ratio for columns as follows:

for $\frac{P_u}{\phi P_n} \geq 0.2$:

$$\text{response ratio} = \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \tag{4.1}$$

for $\frac{P_u}{\phi P_n} < 0.2$:

$$\text{response ratio} = \frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \tag{4.2}$$

IV.2. Strength Criteria For Beams

IV.2.1. Bending strength

The flexural design strength of beams is defined in the AISC–LRFD as $\phi_b M_n$, where $\phi_b = 0.90$ is the resistance factor in bending and M_n is the nominal bending strength determined as shown in the following.

a) Beams with $L_b \leq L_r$

The nominal bending strength depends on the end-moments acting on the beam, and is given by the same formulas as for columns (F1-3 through F1-7) as explained in section IV.1.3.

b) Beams with $L_b > L_r$

For beams for which the unbraced length L_b exceeds the limiting laterally unbraced length L_r,

$$M_n = M_{cr} \leq C_b M_r \qquad \text{(F1-12)}$$

where M_{cr} is the critical elastic moment given by

$$M_{cr} = C_b \cdot \frac{\pi}{L_b} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi E}{L_b}\right)^2 \cdot I_y \cdot C_w} \qquad \text{(F1-13)}$$

All other parameters have the same meaning as in section IV.1.3.

Example 4.5 :

Beam section W57 (W16x57) is used in the braced bay of the short direction and has a length of 24 ft = 288 in. Since L_r = 273.7 (using formula F1-6), the critical elastic moment has to be computed.

$$\begin{aligned} M_{cr} &= C_b \cdot \frac{\pi}{288} \cdot \sqrt{29000 \cdot 43.1 \cdot 11200 \cdot 2.22 + \left(\frac{\pi \cdot 29000}{288}\right)^2 \cdot 43.1 \cdot 2660} \\ &= \underline{C_b \cdot 2250.1} \end{aligned}$$

and $\quad M_r = \underline{2397 \text{ k-in.}}$ $\quad$ (using equation F1-7)

Therefore, the requirement $M_{cr} \leq C_b \cdot M_r$ is always fulfilled in this case.

Table 4.5 lists L_p, M_p, L_r and M_r for the other beam sections.

IV.2.2. Shear strength

Shear strength of beams is computed using the formulas given in section F2 of the AISC–LRFD specifications. The design shear strength is $\phi_v V_n$, where V_n is the nominal shear strength and ϕ_v=0.90. Nominal shear strength is computed as:

$$V_n = 0.6 \cdot F_Y \cdot A_W \qquad \text{(F2-1)}$$

when $\frac{h}{t_w} \leq 187 \cdot \sqrt{\frac{k}{F_Y}}$

where F_Y = 36 ksi. = yield stress

$A_W = d \cdot t_W$ = web area (see section F2.1)

d = overall depth

t_W = web thickness

h = clear distance between flanges less the fillet radius

k = web buckling coefficient = 5 when stiffeners are not used.

For the frame analysed, the criterion above becomes:

$$187 \cdot \sqrt{\frac{k}{F_Y}} = 187 \cdot \sqrt{\frac{5}{36}} = 69.7$$

As can be verified from table 3.4, the five different beam sections used in the short direction of the frame all have $\frac{h}{t_w}$ ratios less than 69.7. Therefore, the nominal shear strength of these sections is defined by equation F2-1 and is as follows:

W 18x76: $V_n = 0.6 \cdot 36 \cdot 7.74 = 167.2^k$

W 14x68: $V_n = 0.6 \cdot 36 \cdot 5.83 = 125.9^k$

W 16x67: $V_n = 0.6 \cdot 36 \cdot 6.45 = 139.3^k$

W 16x57: $V_n = 0.6 \cdot 36 \cdot 7.06 = 152.5^k$

W 12x53: $V_n = 0.6 \cdot 36 \cdot 4.16 = 89.9^k$

IV.3. Strength Criteria For Braces

IV.3.1. Tensile strength

Strength of members subjected to axial tensile forces is to be taken as the lower value of yielding in the gross section and fracture in the net section (section D1, AISC-LRFD). It was assumed in this study that the braces would be designed so that they yield and no fracture occurs. Then, the design tensile strength is $\phi_t P_n$, according to the LRFD, where:

$$\phi_t = 0.90$$

$$P_n = F_Y \cdot A_g \qquad \text{(D1-1)}$$

F_Y is the yield stress and A_g is the gross area of the rolled section. The design strengths of the three sections used as bracing in the frame, are listed below.

2(L 8x8x0.625) : $\phi_t P_n = 0.9 \cdot 36 \cdot 19.2 = 622.1^k$
L 8x8x0.75 : $\phi_t P_n = 0.9 \cdot 36 \cdot 11.4 = 369.4^k$
L 8x8x1.125 : $\phi_t P_n = 0.9 \cdot 36 \cdot 16.7 = 541.1^k$

IV.3.2. Compressive strength

Compressive strength of the braces is defined by the same formulas as for the compressive strength for the columns (equations E2-1 through E2-4). Since braces are assumed to be pinned at both ends, the effective length factor becomes K=1. Table 4.6 lists the λ_c factors for the braces in the frame, together with the design compressive strengths.

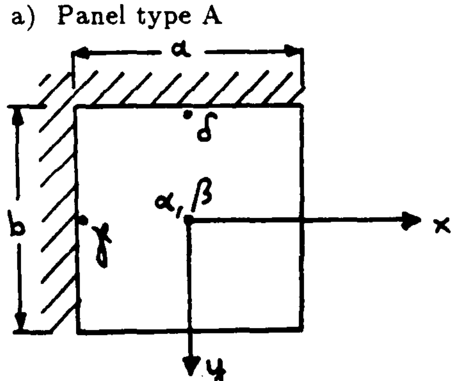
IV.4. Strength Criteria For Reinforced Concrete Slabs

The reinforced concrete slabs used for the floors and the roof were treated in a somewhat simplified manner using classical plate theory formulations [7,8]. This approach was used in order to avoid having to actually design steel reinforcement for the slabs, which is outside the scope of this study. Fixed or simple boundary conditions were assumed along the edges of each panel, depending on whether an edge is bordering on another panel or whether it is along the perimeter of the building. A typical floor plan is shown in figure 4.1, with the letters indicating different panel sizes and configurations. There are five types of panels:

Type A: 24x24 ft^2, two sides clamped, two sides simply supported;
Type B: 24x30 ft^2, two sides clamped, two sides simply supported;
Type C: 24x24 ft^2, three sides clamped, one side simply supported;
Type D: 24x30 ft^2, three sides clamped, one side simply supported;
Type E: 24x24 ft^2, all four sides clamped;

The positive directions of moments acting on an infinitesimal element of a slab are shown in figure 4.2. In the following, the formulas for the midspan and continuous edge moments will be given for each type of panel. At the simply supported edges along the perimeter, moments are assumed to be zero.

a) Panel type A



$\frac{b}{a} = 1.0$; l=a=b=24 ft

$\Rightarrow \alpha = \beta = 0.0281$

$\gamma = \delta = -0.0678$

Center of plate:

$$M_x = M_y = \alpha q l^2 = 0.0281 \cdot q \cdot (24)^2 = 16.1856 \cdot q \tag{4.3}$$

Middle of fixed edge:

$$\overline{M}_x = \overline{M}_y = \gamma q l^2 = -0.0678 \cdot q \cdot (24)^2 = -39.0528 \cdot q \tag{4.4}$$

b) Panel type B

$\frac{b}{a} = \frac{30}{24} = 1.25$: $l = a = 24$ ft

$\Rightarrow \alpha = 0.0396$, $\beta = 0.0275$, $\gamma = -0.0882$, $\delta = -0.0746$

Center of plate:

$$M_x = \alpha q a^2 = 0.0396 \cdot q \cdot (24)^2 = 22.8096 \cdot q \tag{4.5}$$

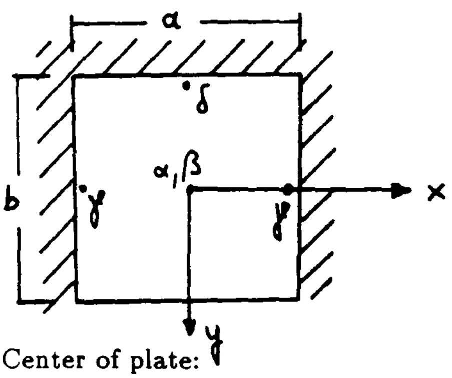
$$M_y = \beta q a^2 = 0.0275 \cdot q \cdot (24)^2 = 15.84 \cdot q \tag{4.6}$$

Middle of fixed edge:

$$\overline{M}_x = \gamma q a^2 = -0.0882 \cdot q \cdot (24)^2 = -50.688 \cdot q \tag{4.7}$$

$$\overline{M}_y = \delta q a^2 = -0.0746 \cdot q \cdot (24)^2 = -42.912 \cdot q \tag{4.8}$$

c) Panel type C



$\frac{b}{a} = 1.0$, $l = a = 24$ ft

$\Rightarrow \alpha = 0.0261$, $\beta = 0.0213$

$\gamma = -0.0600$, $\delta = -0.0547$

Center of plate:

$$M_x = \alpha q a^2 = 0.0261 \cdot q \cdot (24)^2 = 15.0336 \cdot q \tag{4.9}$$

$$M_y = \beta q a^2 = 0.0213 \cdot q \cdot (24)^2 = 12.2688 \cdot q \tag{4.10}$$

Middle of fixed edge:

$$\overline{M}_x = \gamma q a^2 = -0.0600 \cdot q \cdot (24)^2 = -34.56 \cdot q \tag{4.11}$$

$$\overline{M}_y = \delta q a^2 = -0.0547 \cdot q \cdot (24)^2 = -31.5072 \cdot q \tag{4.12}$$

b) Panel type D

$\frac{b}{a} = \frac{30}{24} = 1.25$; $l = a = 24$ ft

$\Rightarrow \alpha = 0.0335$, $\beta = 0.0186$, $\gamma = -0.0724$, $\delta = -0.0574$

Center of plate:

$M_x = \alpha q a^2 = 0.0335 \cdot q \cdot (24)^2 = 19.296 \cdot q$ (4.13)

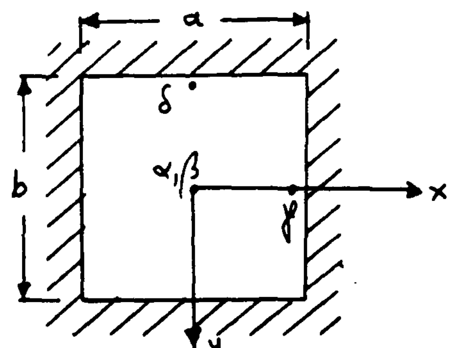
$M_y = \beta q a^2 = 0.0186 \cdot q \cdot (24)^2 = 10.6848 \cdot q$ (4.14)

Middle of fixed edge:

$\overline{M}_x = \gamma q a^2 = -0.0724 \cdot q \cdot (24)^2 = -41.7024 \cdot q$ (4.15)

$\overline{M}_y = \delta q a^2 = -0.0574 \cdot q \cdot (24)^2 = -33.0336 \cdot q$ (4.16)

a) Panel type E



$\frac{b}{a} = 1.0$; $l = a = b = 24$ ft

$\Rightarrow \alpha = \beta = 0.0213$

$\gamma = \delta = -0.0513$

Center of plate:

$M_x = M_y = \alpha q l^2 = 0.0213 \cdot q \cdot (24)^2 = 12.2688 \cdot q$ (4.17)

Middle of fixed edge:

$\overline{M}_x = \overline{M}_y = \gamma q l^2 = -0.0513 \cdot q \cdot (24)^2 = -29.5488 \cdot q$ (4.18)

The actual values of the moments are found by substituting the uniformly distributed design load for q. Since the dead and live loads are different for floors and for the roof (see section III.1), the design load (factored load) computed using formula A4-2 of the AISC–LRFD differs for floors and roof, and is:

for floors : design un. dist. load = 1.2·80 + 1.6·80 = 224 psf

for roof : design un. dist. load = 1.2·60 + 1.6·30 = 120 psf

The moments in the slabs, resulting from these design loads, are listed in table 4.7.

As stated at the beginning of this section, the design of the reinforced concrete slabs is not done rigorously, but is simplified. The simplification is that the slabs are assumed to be reinforced such that they have enough strength to carry exactly the moments listed in table 4.7. In other words, the design moments are used as design moment strengths, without explicitly specifying details of reinforcement. This is a conservative approach, since in actual design the slabs would be reinforced such that they would be able to carry slightly more moment than the design moment (through the application of a resistance factor).

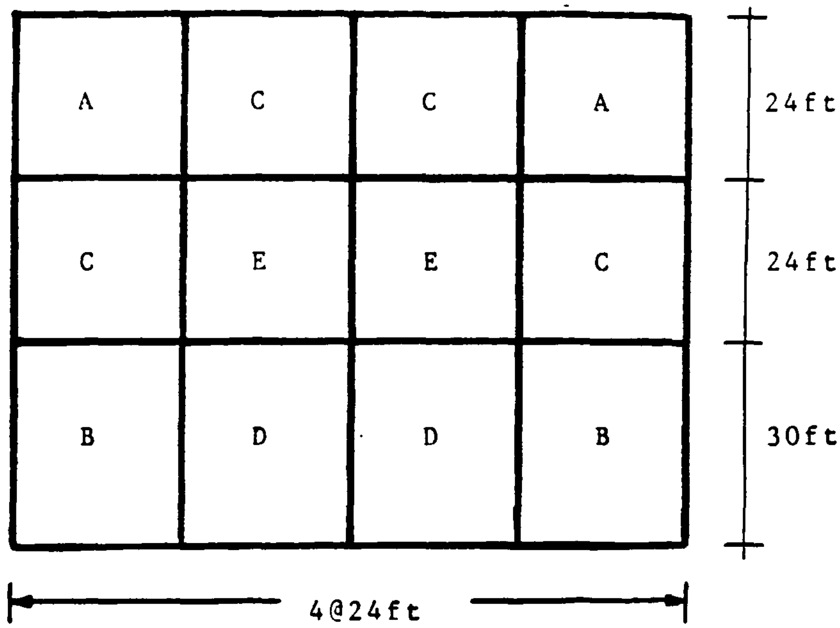


Fig.4.1: Typical floor plan with panel types A, B, C, D and E.

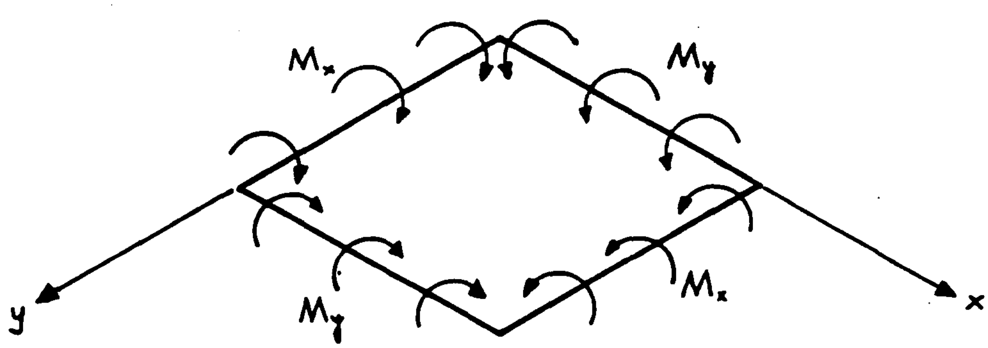


Fig.4.2: Positive directions of moments acting on infinitesimal slab element.

Section	lambda(c)	Compressive Strength (k)	Tensile Strength (k)
W 193, floor 1	0.50	1566.4	1840.3
floor 2	0.40	1626.2	1840.3
W 159, floor 1	0.50	1284.5	1513.1
other floors	0.40	1334.8	1513.1
W 132, floor 1	0.54	1052.3	1257.1
other floors	0.43	1099.1	1257.1
W 109	0.43	905.3	1036.8
W 90	0.44	748.8	858.6
W 61	0.66	456.7	580.0
W 53	0.65	399.7	505.4

Table 4.1: Axial strength of column sections.

Section	Lp in.	Mp k-in.	Lr in.	Mr k-in.
W 193	202.5	12780	1277.5	8060
W 159	200.0	10332	1063.0	6604
W 132	188.0	8424	883.4	5434
W 109	186.5	6912	752.0	4498
W 90	185.0	5652	649.6	3718
W 61	122.5	3672	415.8	2397
W 53	124.0	2804	430.0	1836

Table 4.2

	Sections						
lambda, mode	W 193	W 159	W 132	W 109	W 90	W 61	W 53
Lb/ry , LTB	Floor 1: 44.4 Floor 2: 35.6	Floor 1: 45.0 Floor 2: 36.0	Floor 1: 47.9 Floor 2: 38.3	38.6	38.9	58.8	58.1
b/t , FLB	5.5	6.5	7.1	8.5	10.2	7.7	8.7
h/tw , WLB	12.8	15.3	17.7	21.7	25.9	30.4	28.1

Table 4.3: The shaded values exceed their lower limiting values.

	Sections						
	W 193	W 159	W 132	W 109	W 90	W 61	W 53
Mp (k-in.)	6480.0	5256.0	4068.0	3337.2	2721.6	1180.8	1047.6
Mr (k-in.)	4284.0	3463.2	2682.0	2203.2	1796.4	774.0	691.2
Mn (k-in.)	6480.0	5256.0	4068.0	3337.2	2721.6	Cb*1151.1 <=Mp	Cb*1024.3 <=Mp

Table 4.4: Moment strengths of column sections used.

Section	Lp in.	Mp k-in.	Lr in.	Mr k-in.
W 76	130.5	5868	399.1	3796
W 67	123.0	4680	387.8	3042
W 57	80.0	3780	273.7	2397
W 68	123.0	4140	447.7	2678
W 53	124.0	2804	430.0	1836

Table 4.5

Angles	r (in.)	length (in.)	lambda(c)	øcPn (kips)
2(L 8x8x0.625)	2.49	339.6	1.53	220.2
L 8x8x0.75	1.58	322.0	2.29	58.6
L 8x8x1.125, first floor	1.56	339.6	2.44	75.2
upper floors	1.56	322.0	2.31	83.6

Table 4.6: Design compressive strength of braces.

Panel type	Equation	Slab location	
		floors (k-ft/ft)	roof (k-ft/ft)
A	4.3	3.626	1.942
	4.4	-8.748	-4.686
B	4.5	5.109	2.737
	4.6	3.548	1.901
	4.7	-11.354	-6.083
	4.8	-9.612	-5.149
C	4.9	3.368	1.804
	4.10	2.748	1.472
	4.11	-7.741	-4.147
	4.12	-7.058	-3.781
D	4.13	4.322	2.316
	4.14	2.393	1.282
	4.15	-9.341	-5.004
	4.16	-7.400	-3.964
E	4.17	2.748	1.472
	4.18	-6.619	-3.546

Table 4.7: Moments in slabs, resulting from design loads.

V. RESULTS OF THE ANALYSES

Six linear collapse analyses were performed using the finite element analysis package ADINA [19] on the Cyber mainframe of the Lehigh University Computing Center. The load case that was used in these analyses was found using SODA [3]. Use was made of SODA's capability to determine, from a number of different load cases, which load case governs the performance of the structure and which member is the critical member. The two different load combinations that were investigated using SODA were those proposed by Leyendecker and Ellingwood [4] for progressive collapse analysis. The two load combinations are:

$$D + 0.5 \cdot L_{ANSI} + 0.2 \cdot W_{ANSI} \tag{2.1}$$

$$D - 0.3 \cdot W_{ANSI} \tag{2.2}$$

Each load combination had to be used with wind from left and wind from right, making a total of four load cases for the analysis. The results of the analysis by SODA showed that equation 2.1 with wind acting from the right (from the braced side of the short direction) governed the response of the structure. The critical members for this load case are the columns in floors 1 and 2, with response ratios varying between 0.507 and 0.569, based on the combined stresses from the column interaction formula (code clause H1-1a).

V.1. Analysis Of The Intact Three-dimensional Structure

After the governing load combination was found using SODA on the two-dimensional model, an analysis of the intact three-dimensional structure was performed using ADINA with the same load combination (the input files for two ADINA runs, including one collapse case, are listed in [20]). This was done in order to determine which of the columns are most critical. It turned out that unlike in the analysis of the two-dimensional model, the most critical columns are not in floors 1 and 2, but in floors 9 and

10 on column line A, and mostly above the fifth floor in general on all column lines. Interestingly enough, the columns on floors 9 and 10 on column lines A2, A3 and A4 have response ratios over 1.0 (figure 5.1.a), although the total load is less than the design factored load. Also, the columns on floors 7 and 8 on column lines D2, D3 and D4 have response ratios between 0.8 and 1.0, indicating a low margin of safety (figure 5.1.c). This indicates that members designed properly as plane frame members — which is a common practice in regular building design — may not satisfy code criteria when investigated under three-dimensional action. Figure 5.1 also shows the deflected shape of the intact structure along different column lines. Table 5.1 summarizes the maximum and minimum response ratios for columns in the intact structure under the collapse loading (equation 2.1) and also lists the number of columns on each floor that fall into the two failure categories. Category (a) includes members with response ratios greater than or equal to 1.0, and category (b) those with response ratios between 0.8 and 1.0. The lower response ratio level of 0.8 was selected to account for the uncertainty in the analysis. Since a model of this size never reflects the actual behaviour of the structure very accurately, members with response ratios above 0.8 are considered near-critical (or potentially critical) in this study. The variation of maximum and minimum response ratios of columns on each floor is shown graphically in figure 5.2. The maximum vertical deflection for the intact structure was found to be 0.43 inches on column line A3 and 0.56 inches on column line B3.

Although the highest response ratios occurred in the columns of floors 9 and 10 on column lines A2, A3 and A4 (figure 5.1.a), the columns to be removed for the collapse analyses were selected from the third floor on column lines A and B. The reason for this is that if columns were to be removed from the upper floors, the resulting damage would probably be less than if columns on lower floors failed. In order to simulate cases that are more critical, it is necessary to remove columns on lower floors since they carry more load than columns on upper floors. The resulting damage should be expected to be larger. Thus, the third floor columns were selected. The first floor columns could also have been selected, but as table 5.1 shows, the response ratios on column line A are higher on the third floor, and on column line B they are very close to each other on both floors. Figure

5.3 shows a plan view of the third floor with the columns selected for the six analyses. The response ratios of these columns in the intact structure are:

Case	Response ratio of column	
	at top	at bottom
1	0.69	0.68
2	0.68	0.67
3	0.68	0.67
4	0.54	0.53
5	0.53	0.52
6	0.53	0.52

V.2. Analyses With Removal Of Columns

In the previous section it was explained how the intact structure was analysed and which columns were selected for removal in the collapse analyses. In each of the analyses one column was removed to simulate the loss of a primary structural member. The response ratios for the other elements of the structure were found using the Fortran programs COLUMN, BEAM, BRACE and SLAB (see listing in [20]). These programs use ADINA output to identify and list members which have response ratios in categories (a) and (b) (see section 5.1 for an explanation of these categories). In order to facilitate the explanation and interpretation of the results, the numbers of columns, beams and braces on each floor that have response ratios above 0.8 are listed in tables 5.2 through 5.7 for cases 1 through 6, respectively. In addition, figures 5.4 through 5.15 show the deflected shapes and the locations of the "failed" members and the initially removed column for each case, together with graphs similar to those in figure 5.2 that depict the variation of maximum and minimum response ratio with floor height. The numerical values for these graphs are taken from tables 5.8 through 5.13.

V.2.1. Case 1

The maximum vertical deflection for this case was found to be 4.80 inches on column line A4 (where the column was removed) and 0.80 inches on column line B4 (figure 5.4). A great number of columns on column line B4 have response ratios exceeding 1.0 (figure 5.4.b), most probably because of the large moments imposed on their ends by the failing beams on column line 4 (figure 5.4.f), and due partly to the incresed axial load. Since the columns directly above the removed column are left with no support, they can not carry any axial load. The load has to be carried by column line B. No significant part of the load is transferred to column lines A3 and A5 because the beams in the long direction are hinged and therefore rotate freely. The failing beams on column line 4 (figure 5.4.f) show response ratios for moment at their right end ranging from about 1.50 near the removed column to about 1.10 at the roof. The decreasing response ratios indicate that some load is carried by the floor slabs at each floor. Some floor slabs undergo reversal of moments at midspan and at the continous edges, especially the slabs adjacent to the column line where the column was removed (that is, failed). The bracing remains rather unaffected by the initial failure, except for the first floor bracing on column lines A and B, which fall into category (b).

The failing beams on column line 4 transfer most of their load to the columns on column line B4. Since the disturbance is dissipated to a large part as axial force, it does not reach the braced bay of the short direction. The braced bay of the long direction is also unaffected because the beams in that direction do not transmit shear forces from one column line to the other due to their hinge supports, therefore eliminating the possibility of a significant increase in axial load on neighboring column lines. As figure 5.5 shows, the effects of the loss of a column on column line A are mostly felt on column lines A and B, and are virtually absent on column lines C and D. While the difference between maximum and minimum response ratios ("bandwidth") is more or less constant on undisturbed column lines, it shows large variations on affected column lines. Also, the response ratios on undisturbed column lines are very close to their response ratios in the intact structure.

V.2.2. Case 2

Compared to case 1, the maximum vertical deflection in this case is only 0.83 inches on column line A3 and 0.63 inches on column line B3. Also, the number of failing columns is much less (figure 5.6). It seems as if the braces in the bay adjacent to the removed column acted as "buffers" and accomodated some of the disturbance. The number of braces with response ratios exceeding 1.0 (response ratios from 0.9 to about 3.0 on column line A) in the whole structure (table 5.3) is indicative of large forces being taken by the braces. Though braces are usually designed for and supposed to act in tension only, their compressive force capacity is "unintendedly" utilized. This is also in accord with the much lower vertical deflection in this case. Figure 5.7 and table 5.9 show that there is a slight increase in bandwidth generally across all column lines, and that the maximum response ratios in columns are also slightly higher than in the intact structure.

V.2.3. Case 3

The removal of the column for this case leads to similar observations as in case 2, except for the different deflected shape of the building (figure 5.8). As we see, the failure pattern and distribution of affected members is very similar to case 2, but with the difference that the pattern is like a mirror image of case 2 reflected about a vertical axis through the middle of the braced bay of the long direction. However, the number of columns, beams and braces failing at each floor level comes out to be exactly the same as in case 2 (see table 5.4). The maximum vertical deflections, 0.81 inches on A2 and 0.62 inches on B2, are also comparable to case 2, as are response ratios of the braces.

As figure 5.9 and table 5.10 show, the response ratios of columns exhibit a behaviour very similar to case 2. In general then, assumed that the corresponding member names and column lines are replaced properly in the wording, the discussion would be identical to that for case2.

V.2.4. Case 4

As in case 1, a column is removed here that is in unbraced bays in both directions. In addition, the column is located on B4, the column line with the heaviest dead and live loads acting on it. As should be expected, this is one of the most critical cases investigated in this study. The maximum vertical deflections are 0.60 inches on A4, 3.01 inches on B4 and 0.72 inches on C4. The deflected shapes are shown in figure 5.10, together with members falling in failure categories (a) or (b). As table 5.5 shows, there is a great number of columns and beams falling into the critical categories, with the number of columns in category (a) and the number of beams in both categories exceeding that in case 1. The beams indicated on figure 5.10 have response ratios (for moment at the circled end) ranging from approximately 1.25 on the third floor to about 0.90 on the upper floors. This decrease in response ratio with height is again indicative of the floor slabs' aid in redistributing some of the loads. The slabs undergoing the highest reversals of moments at midspan and/or continous edge are concentrated around the column being removed, on column line B4 (figure 5.10.g).

The redistribution of loads puts columns on A4 and C4 under higher axial load and also induces large end-moments on them, which causes the "bandwidth" in figures 5.11.a and 5.11.c to increase dramatically and display very irregular behaviour. A similar behaviour is noticed on figure 5.11.b, but with only a slight increase in maximum response ratios which is probably due to slight end-moments being induced in the columns on column lines 3 and 5 (caused by the inward "pull" of the beams in between). The minimum response ratios on column line B drop sharply to almost zero due to "unloading" on B2. Column line D seems to be rather unaffected which may be attributed to the "shielding" effect of the braced bay as discussed earlier.

V.2.5. Case 5

The maximum vertical deflection in this case is 0.50 inches on A3, 0.94 inches on B3 and 0.64 inches on C3 (figure 5.12). As in the other cases where a column is removed from a braced bay, the bracing effectively protects beams and columns in bays adjacent to the removed column by taking a significant amount of the redistributed load and failing themselves. The response ratios in the braces on column line B range from approximately 3.50 on the third floor to approximately 1.0 on the upper floors and on the floors below the removed column. On column lines A and C, the responseratios in the braces lie generally between 0.8 and 1.0. The bracing also seems to significantly prevent excessive vertical deflection as in earlier cases.

The total number of columns and beams in both failure categories is greatly reduced, accompanied by an increase in the number of failing braces (table 5.6). The plots of response ratios show generally a slight increase in the bandwidth across all column lines (figure 5.13), and there is a noticeable but insignificant increase in the maximum response ratios on column lines A, B and C. Since they are in an unbraced bay (in the short direction), the columns on column line A show a higher increase in maximum response ratios as compared to column line C, which is in a braced bay.

V.2.6. Case 6

The general performance of the structure is very similar to case 5, with the deflected shape (figure 5.14) and the locations of the failing members being the only difference. The maximum vertical deflection is 0.48 inches on A2, 0.93 inches on B2 and 0.63 inches on C2. The numbers of the various structural elements falling into failure categories (a) and (b) is very nearly the same as in case 5 as can be verified from table 5.7. The response ratios of the braces range from approximately 3.5 on floor 3 to approximately 1.2 on the floors far above and below on column line B. Again, these response ratios are between 0.8 and 1.0 on column lines A and C. The maximum and minimum response ratios of the columns (figure 5.15) show changes similar to case 5.

V.3. Conclusions

The observations made in the six cases investigated lead to following conclusions.

- When the maximum and minimum response ratios in columns on the same column line are plotted for the intact structure, they show a similar variation with the height of the floor, on all column lines. The plot of maximum response ratio is nearly parallel to the plot of minimum response ratio at every point, and the difference between the two plots (called "bandwidth") is almost uniform.

- Upon removal (failure) of a column, the bandwidth on the column line to which it belongs increases and becomes erratic. The maximum response ratios can increase significantly.

- Removal (failure) of a column adjacent to braced bays results in less damage to other columns and beams than the removal of a column in unbraced bays. However, the number of failing braces in the braced bay adjacent to the removed column will be higher. This is perhaps a desired situation since columns are usually more important primary structural elements than braces, and the replacement of a failed brace is generally easier than that of a failed column.

- Structural elements in bays or column lines not adjacent to removed columns seem to remain rather unaffected by the initial disturbance.

- Bracing, although designed for tension only, seems to be highly effective in shielding columns from moment disturbances. Braces seem to act as a "buffer" by dissipating a portion of the redistributed loads. Because of the stiffness they add to the bay they belong to, the deflections due to the disturbance are much less than in unbraced bays.
- Floor slabs also seem to provide a significant amount of stiffness and capacity to absorb some of the redistributed loads, as the decrease in response ratio in beams with height above the removed column indicates.

- Floor slabs show significant disturbances only in panels in the immediate vicinity of the removed column (figures 5.4.g and 5.10.g).
  This pattern repeats itself on all floors above the removed column.

V.4. Recommendations For Future Research

A research project is never complete, as there will always come new questions to the researcher's mind as the work progresses. New ideas will develop and the need for modifications and refinements will make itself felt as results are being obtained. In many instances there will not be enough time to follow through with the new ideas — as in this case — and recommendations are made for future investigators. The main points that emerged from this research as potential future work are summarized in the following paragraphs.

V.4.1. Nonlinear behaviour

The findings of this study are based on a linear finite-element model. Therefore, they can only serve as an indication of how the structure might actually behave. To model the behaviour of the actual structure more precisely, a finite-element model where material and geometric nonlinearity is considered, is absolutely necessary, since in an actual collapse case large displacements and plastification of certain regions in the members will occur. It is suggested that at least two nonlinear runs be made. One with the intact structure to determine the actual ultimate load the structure is capable of carrying, and one or more runs with critical members removed to determine the actual distribution of failure and how the initial failure will propagate after the failure of additional members.

V.4.2. Modeling of the load distribution

One difficulty commonly encountered is how to put the loads on the structure to achieve similarity between actual load distribution and load distribution on the model. This depends to some extent on how the structure itself is modeled. In this case, the loads were applied to the short direction bents only. For bents 2 to 4, a contributing width of 24 ft was used to compute the uniformly distributed loads on the beams. For bents 1 and 5 this contributary width was only 12 ft, since they are at the sides. The choice of such a load application causes the failure in the analyses to remain more or less confined to the bent in the short direction where the column is removed.

In the actual structure, the dead and live loads are not applied to the beams directly, and also not *only* to the beams. The dead and live loads are carried by the floor slabs and transferred to the beams and columns. The floor slabs as well as the beams are actually carrying the loads simultaneously. In the model however, the floor slabs only are acted upon by loads that are induced by deflections in the structure caused by the loading of the beams. In other words, there are no direct loads on the slabs, which results in much smaller bending moments in them, for example. If, however, the loads had been applied as uniformly distributed loads on the floor slabs, the situation would have been exactly reversed. The beams would not have been under direct load, but under *indirect* loading only. This is a result of the way the structure was modeled, which is discussed in the next section.

V.4.3. Refining the model

To keep the number of plate and beam elements as low as possible in order to save runtime and system resources, each panel of the floor slabs was modeled as four triangular plate elements with nodes at the columns and one node in the middle of the panel. This means that no direct load transfer was possible between floor slabs and beams. It also means that the stiffness of the floor slabs was artificially increased. A more accurate model should be one in which the beams are modeled by at least three beam elements. and the slabs should consist of a larger number of smaller elements with connections to beam nodes within the length of a beam. This would make it possible to apply the loads to the floor slabs which would then transfer them to the beams, which would more closely reflect the actual load transfer mechanism.

Another problem was that ADINA-PLOT, the post-processor of ADINA, does not show rotations at the nodes on the plots, although ADINA takes into account rotations and rotational degrees of freedom in the finite element analysis and also prints rotations at the nodes in the output. In other words, the stress resultants from the analysis are correct, but because ADINA-PLOT does not use a spline curve when plotting, it is necessary to use a number of finite elements to model one structural element in order to see the actual deflected shape. For example, each column should be modeled with at least three beam elements, and the same holds for beams.

V.4.4. Different configurations of the structure

The analysis in this study consisted of six cases where different columns were removed from the structure without any changes made to the structure itself. To get an idea of how the structure would behave if it were designed differently, the following configurations of the structure would be interesting to investigate, regardless of the way the structure is modeled:

- beams in the long direction connected to the columns via moment connections, instead of shear connections;
- slabs assumed to not fully interact with the beams and columns, but as simply resting on them (non-composite behaviour);
- bracing in one bent removed;
- more (or stronger) bracing used in the structure; it would be interesting to see if bracing at least every other bay would permit the use of lighter sections with the same performance;

The above suggestions should provide a sufficiently broad base for future research leading to interesting findings.

LEGEND FOR FIGURES :

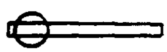
The meanings of the symbols, in the figures on the pages that follow, are explained below.

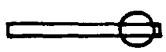
 column with RR $\geq$ 1.0 in interaction formula

 column with 0.8 $\leq$ RR $<$ 1.0 in interaction formula

 beam with RR $\geq$ 1.0 for moment at left end

 beam with RR $\geq$ 1.0 for moment at right end

 beam with 0.8 $\leq$ RR $<$ 1.0 for moment at left end

 beam with 0.8 $\leq$ RR $<$ 1.0 for moment at right end

 brace with RR $\geq$ 1.0 in compression

 brace with 0.8 $\leq$ RR $<$ 1.0 in compression

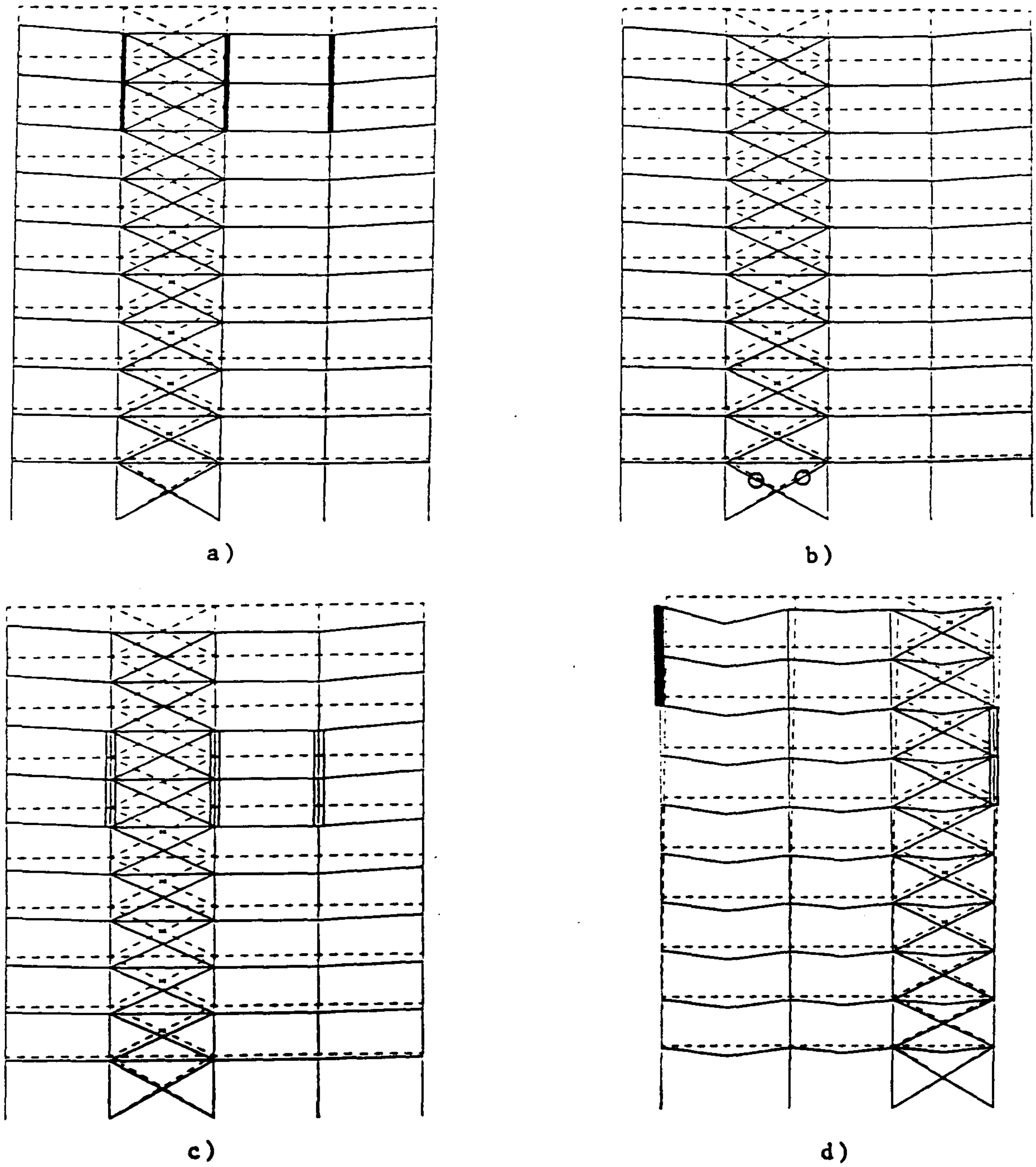


Fig.5.1: Deflected shapes, and members with RR≥0.8 in the intact structure. Column lines a)A b)B c)C d)D.

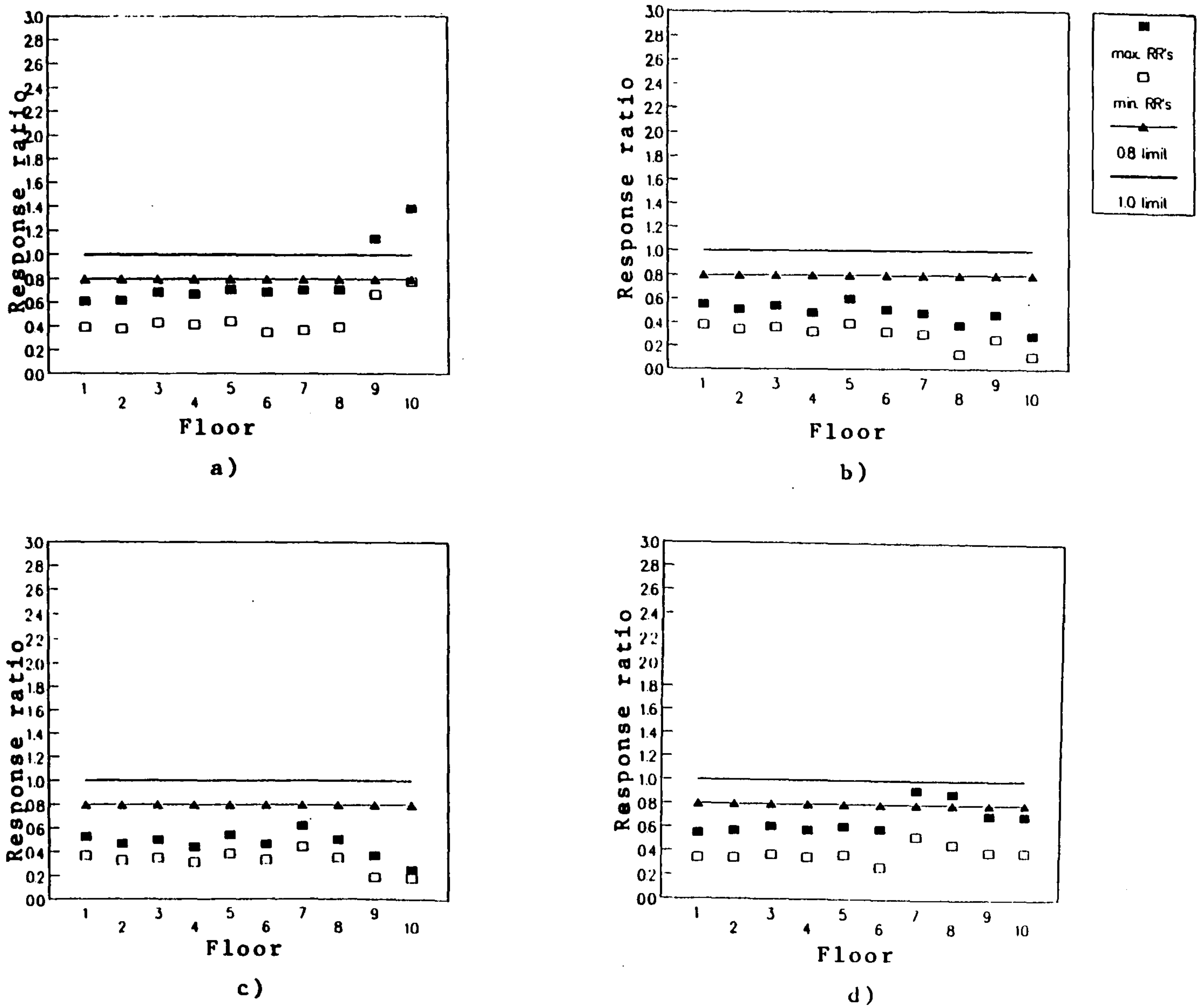


Fig.5.2: Maximum and minimum response ratios of columns of intact structure along column lines a)A b)B c)C d)D.

Floor	Column line								Total no. of columns with RR >= 1.0	Total no. of columns with 0.8 <= RR < 1.0
	A		B		C		D			
	max.	min.	max.	min.	max.	min.	max.	min.		
1	0.61	0.39	0.55	0.38	0.53	0.37	0.55	0.34	0	0
2	0.62	0.38	0.51	0.34	0.47	0.33	0.57	0.34	0	0
3	0.69	0.43	0.54	0.36	0.50	0.35	0.61	0.37	0	0
4	0.67	0.41	0.48	0.32	0.44	0.31	0.58	0.35	0	0
5	0.71	0.44	0.60	0.39	0.54	0.38	0.61	0.37	0	0
6	0.69	0.35	0.51	0.32	0.46	0.33	0.59	0.27	0	0
7	0.71	0.37	0.48	0.30	0.62	0.44	0.92	0.53	0	3
8	0.71	0.39	0.38	0.13	0.50	0.35	0.89	0.46	0	3
9	1.14	0.67	0.47	0.26	0.37	0.19	0.71	0.40	3	0
10	1.39	0.78	0.29	0.11	0.25	0.18	0.70	0.39	3	0

Table 5.1: Maximum and minimum response ratios on column lines A, B, C and D.

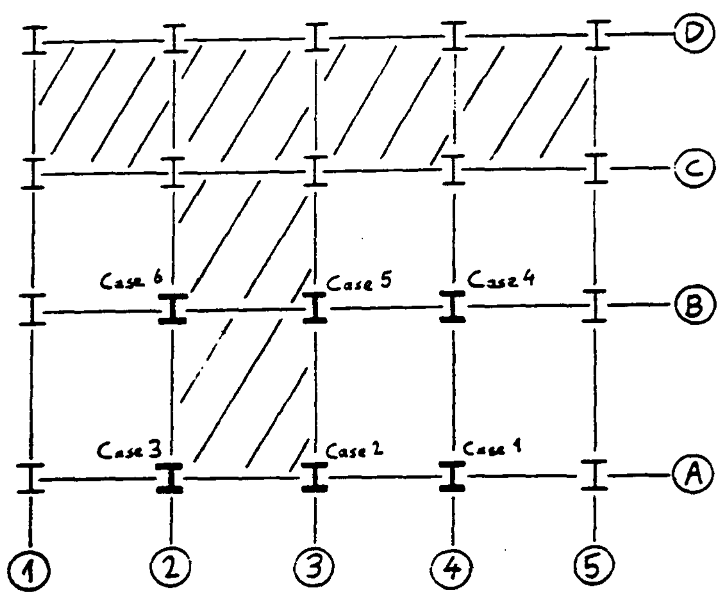


Fig.5.3: Third floor columns removed in the analyses. Shaded areas indicate braced bays.

Floor	Columns (a)	Columns (b)	Beams (a)	Beams (b)	Braces short direc. (a)	Braces short direc. (b)	Braces long direc. (a)	Braces long direc. (b)
1	0	0	0					4
2	0	0	0					0
3	1	0	1					0
4	2	1	1					0
5	1	1	1	none	none	none	none	0
6	1	2	1					0
7	1	5	1					0
8	1	5	1					0
9	4	1	1					0
10	5	1	1					0
Total	16	16	8	0	0	0	0	4

Categories: a) RR>=1.0 b) 0.8<=RR<1.0

Table 5.2: Structural elements with response ratios above 0.8 in case 1.

Floor	Columns (a)	Columns (b)	Beams (a)	Beams (b)	Braces short direc. (a)	Braces short direc. (b)	Braces long direc. (a)	Braces long direc. (b)
1	0	0					0	3
2	0	1					0	0
3	0	1					2	0
4	0	1					2	1
5	0	1	none	none	none	none	1	2
6	0	0					0	1
7	0	3					0	0
8	0	3					0	0
9	2	1					0	0
10	3	1					0	0
Total	5	12	0	0	0	0	5	7

Table 5.3: Structural elements with response ratios above 0.8 in case 2.

Floor	Columns (a)	Columns (b)	Beams (a)	Beams (b)	Braces short direc. (a)	Braces short direc. (b)	Braces long direc. (a)	Braces long direc. (b)
1	0	0					0	3
2	0	1					0	0
3	0	1					2	0
4	0	1					2	1
5	0	1	none	none	none	none	1	2
6	0	0					0	1
7	0	3					0	0
8	0	3					0	0
9	2	1					0	0
10	3	1					0	0
Total	5	12	0	0	0	0	5	7

Table 5.4: Structural elements with response ratios above 0.8 in case 3.

	Columns		Beams		Braces			
					short direc.		long direc.	
Floor	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
1	0	0	0	0		0		4
2	0	0	0	0		0		0
3	1	1	2	0		0		0
4	2	0	2	0		0		0
5	2	0	2	0	none	0	none	0
6	2	0	2	0		0		0
7	2	3	1	1		1		0
8	2	3	1	1		0		0
9	4	1	1	1		0		0
10	4	2	0	1		0		0
Total	19	10	11	4	0	1	0	4

Table 5.5: Structural elements with response ratios above 0.8 in case 4.

	Columns		Beams		Braces			
					short direc.		long direc.	
Floor	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
1	0	0					1	2
2	0	0					1	0
3	0	1					2	0
4	0	1					2	1
5	0	1	none	none	none	none	1	2
6	0	1					1	0
7	0	4					0	1
8	0	4					0	0
9	3	0					0	0
10	3	0					0	0
Total	6	12	0	0	0	0	8	6

Table 5.6: Structural elements with response ratios above 0.8 in case 5.

	Columns		Beams		Braces			
					short direc.		long direc.	
Floor	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
1	0	0					1	2
2	0	0					1	0
3	0	1					2	0
4	0	1					1	2
5	0	1	none	none	none	none	1	2
6	0	1					1	0
7	0	4					0	1
8	0	4					0	0
9	3	0					0	0
10	3	1					0	0
Total	6	13	0	0	0	0	7	7

Table 5.7: Structural elements with response ratios above 0.8 in case 6.

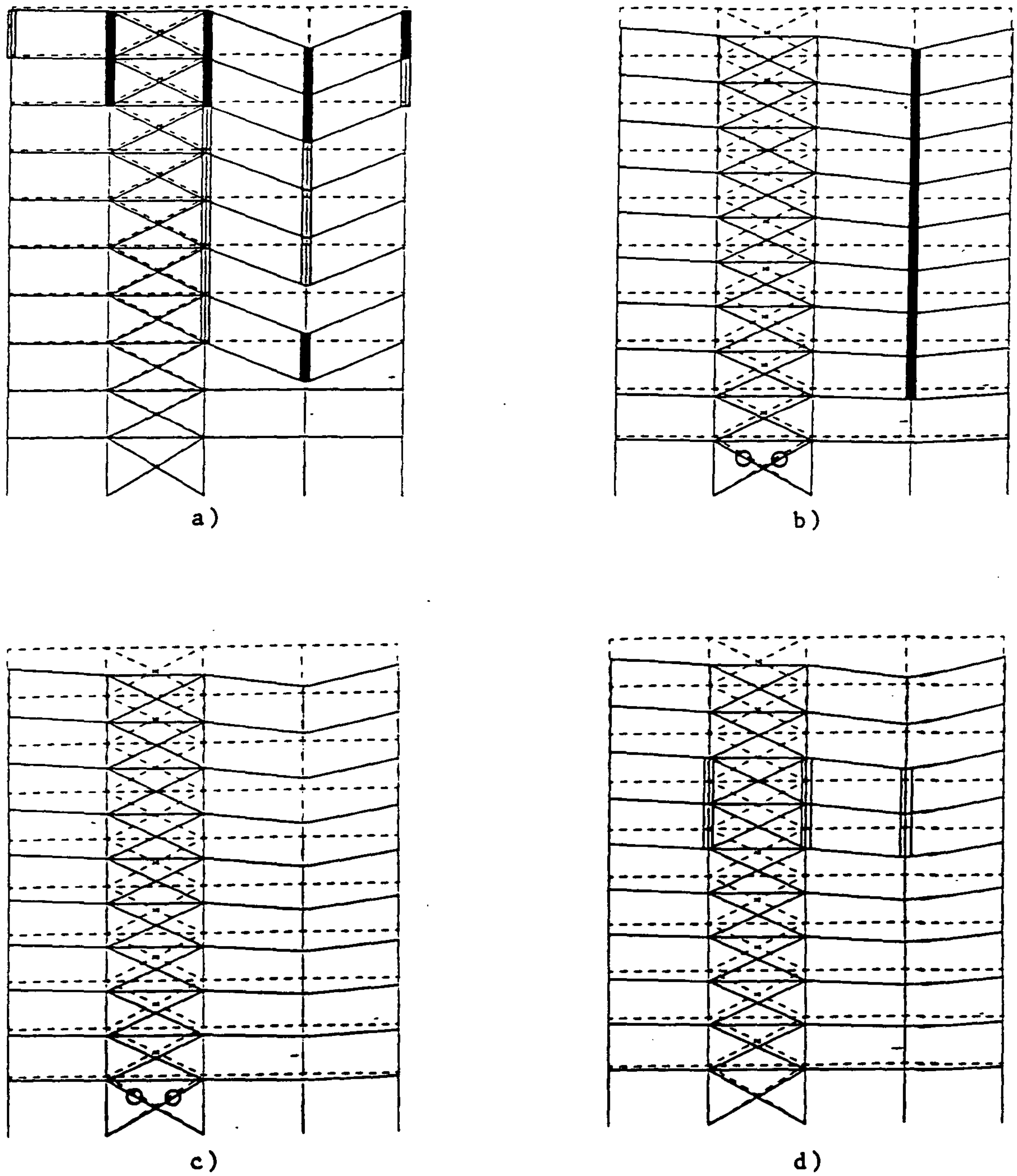
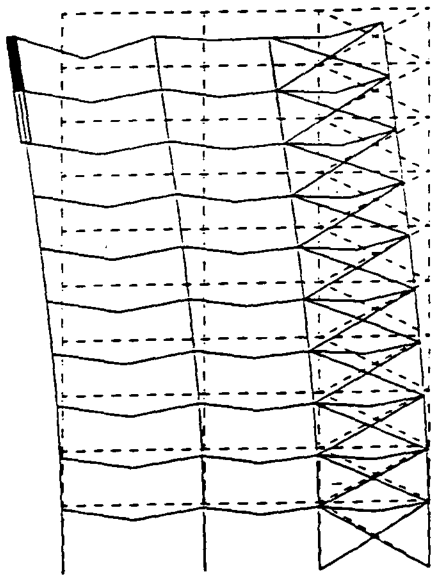
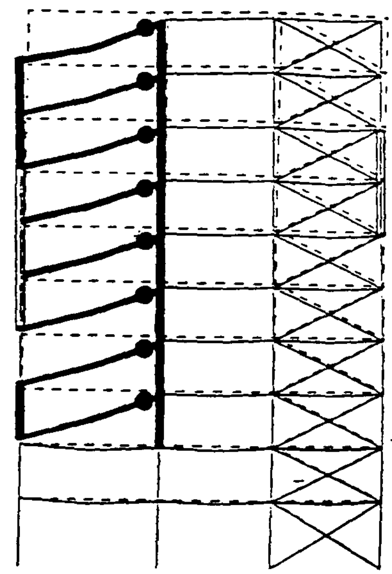


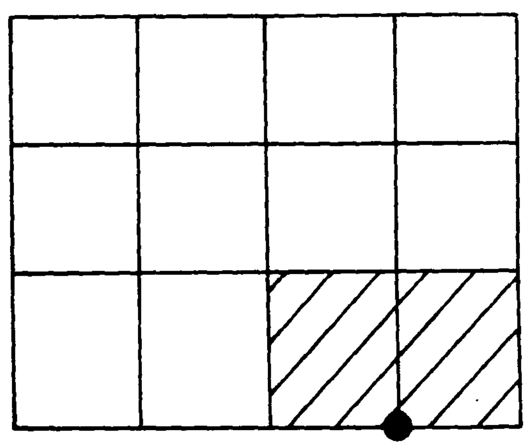
Fig.5.4: Deflected shapes and members with RR≥0.8 in case 1.
Column lines a)A b)B c)C d)D.



e)



f)



g)

Fig.5.4(cont.): e)column line 5, f)column line 4, g)floor slabs influenced by removed column.

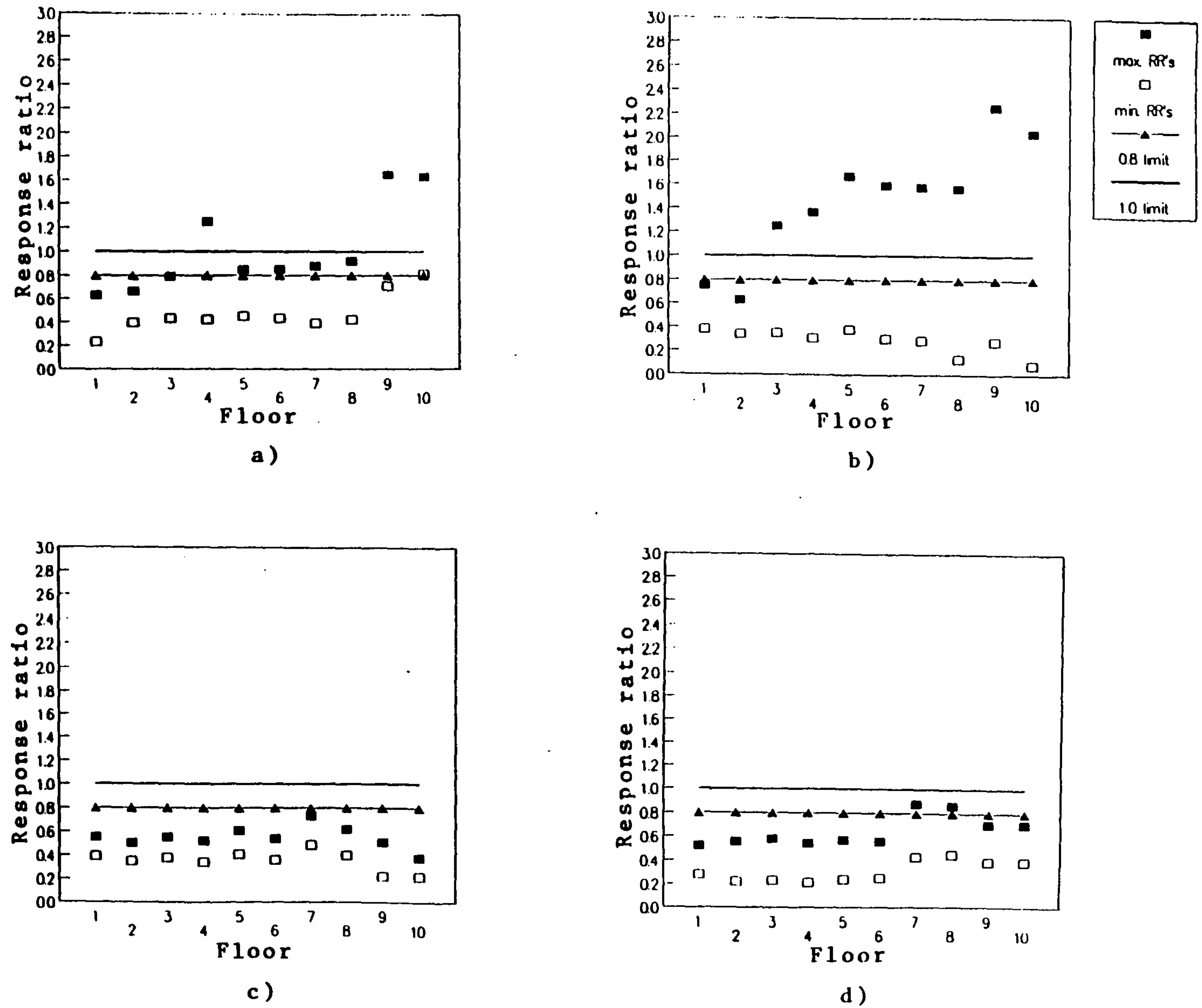


Fig.5.5: Max. and min. response ratios of columns in case 1, along column lines a)A b)B c)C d)D.

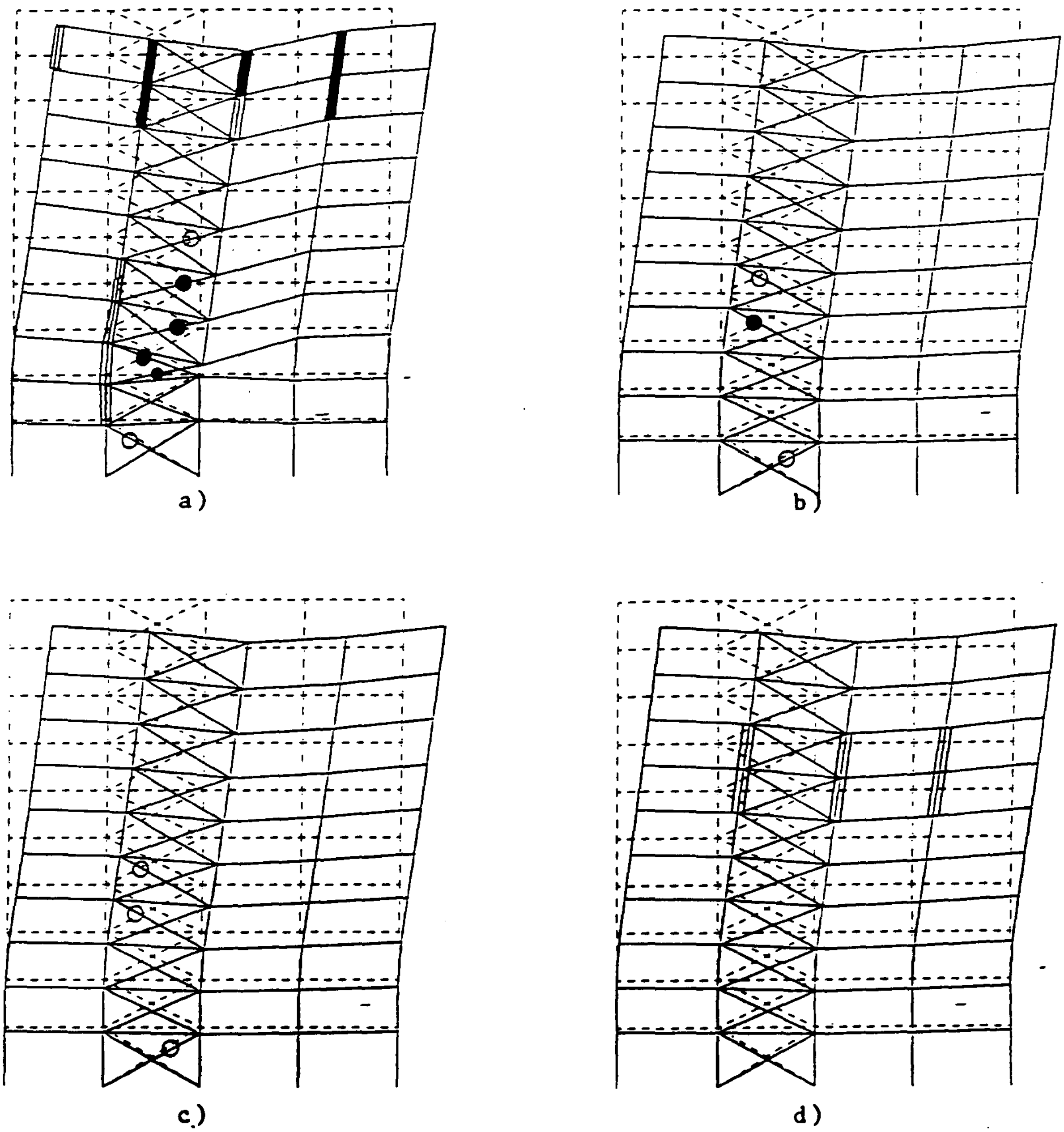
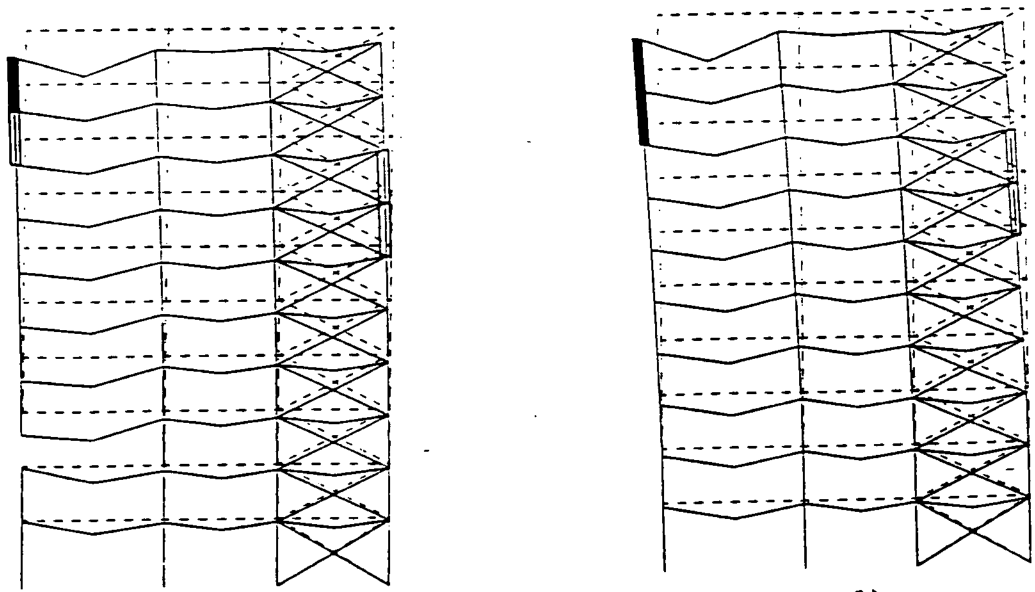


Fig.5.6: Deflected shapes and m$_e$mbers with RR≥0.8 in case 2, along column lines a)A b)B c)C d)D.



e) f)

Fig.5.6(cont.): Column lines e) 3 f)2

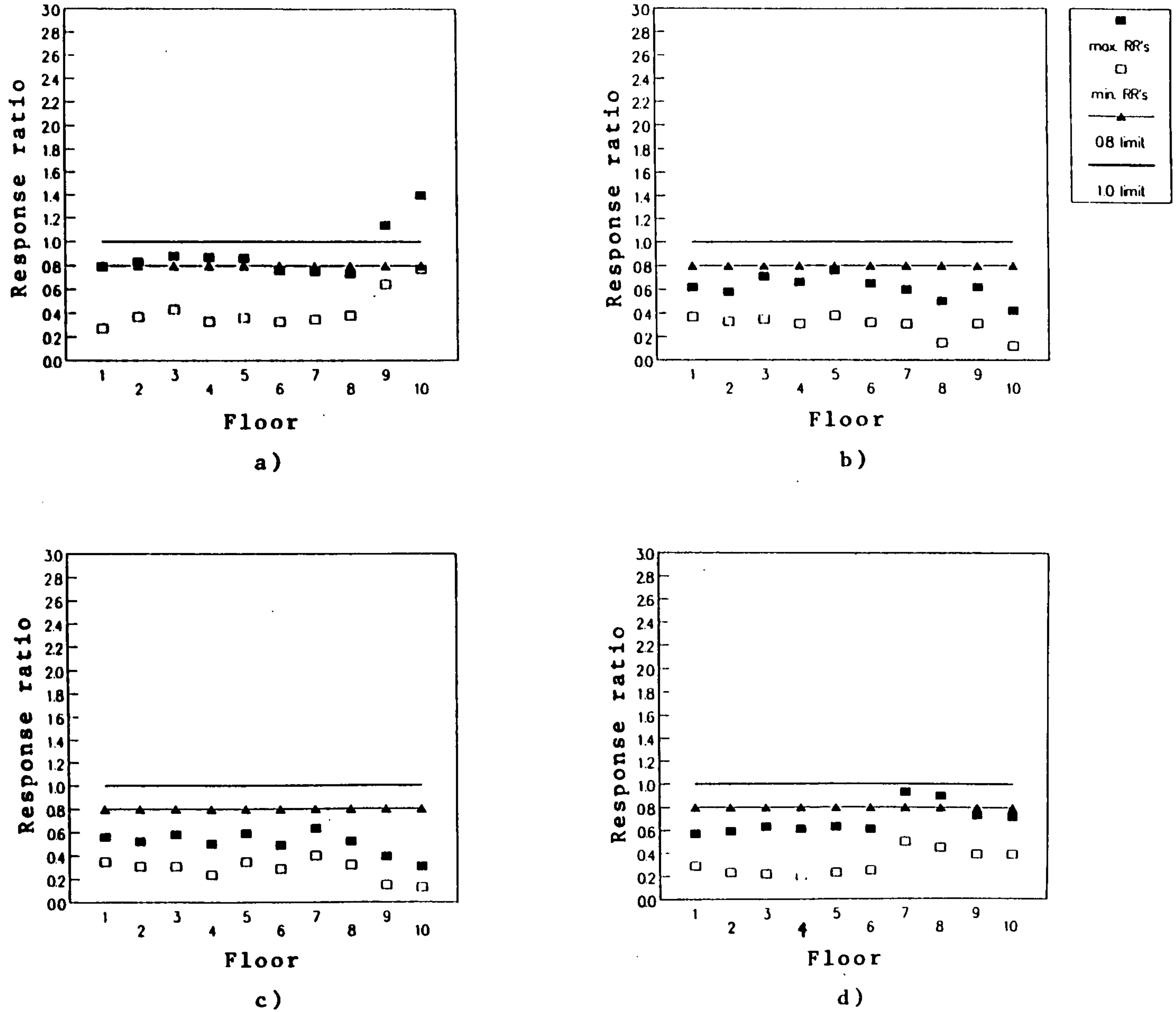


Fig.5.7: Max. and min. response ratios of columns in case 2, along column lines a)A b)B c)C d)D.

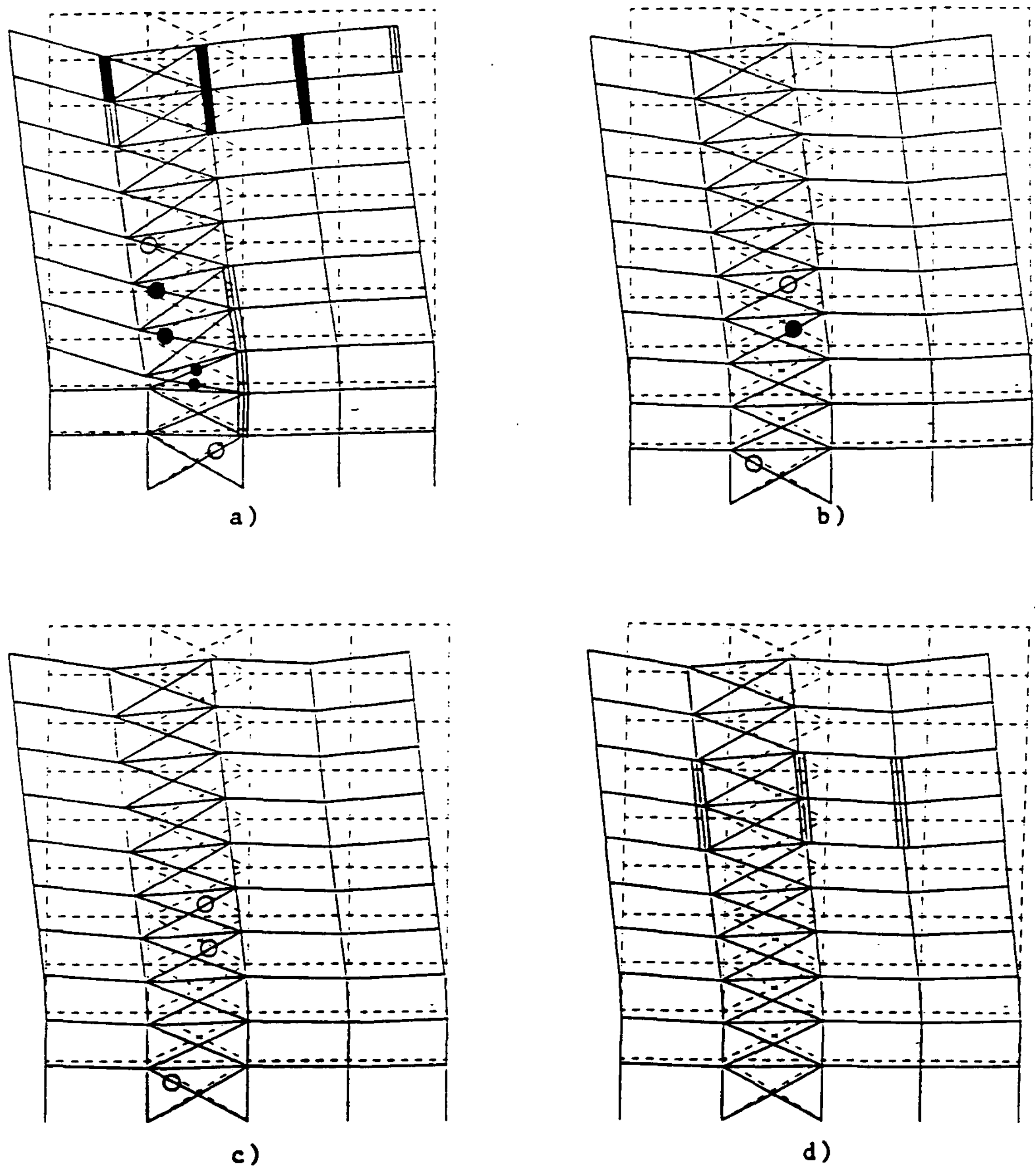
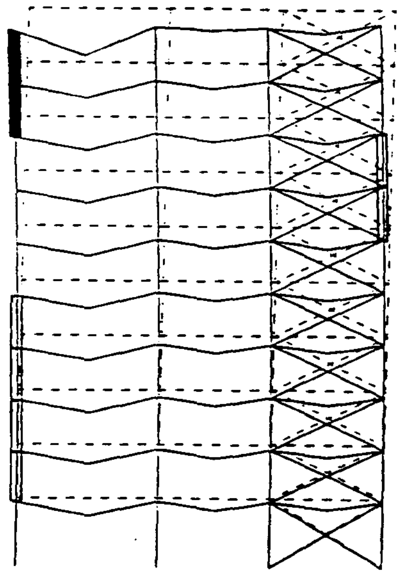
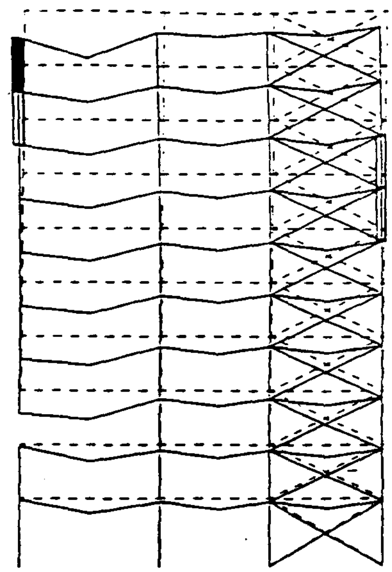


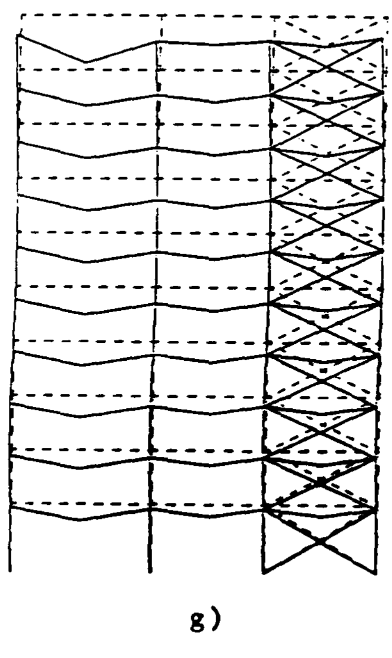
Fig.5.8: Deflected shapes, and members with RR≥0.8 in case 3, along column lines a)A b)B c)C d)D.



e)



f)



g)

Fig.5.8(cont.):
Column lines e)3 f)2 g)1

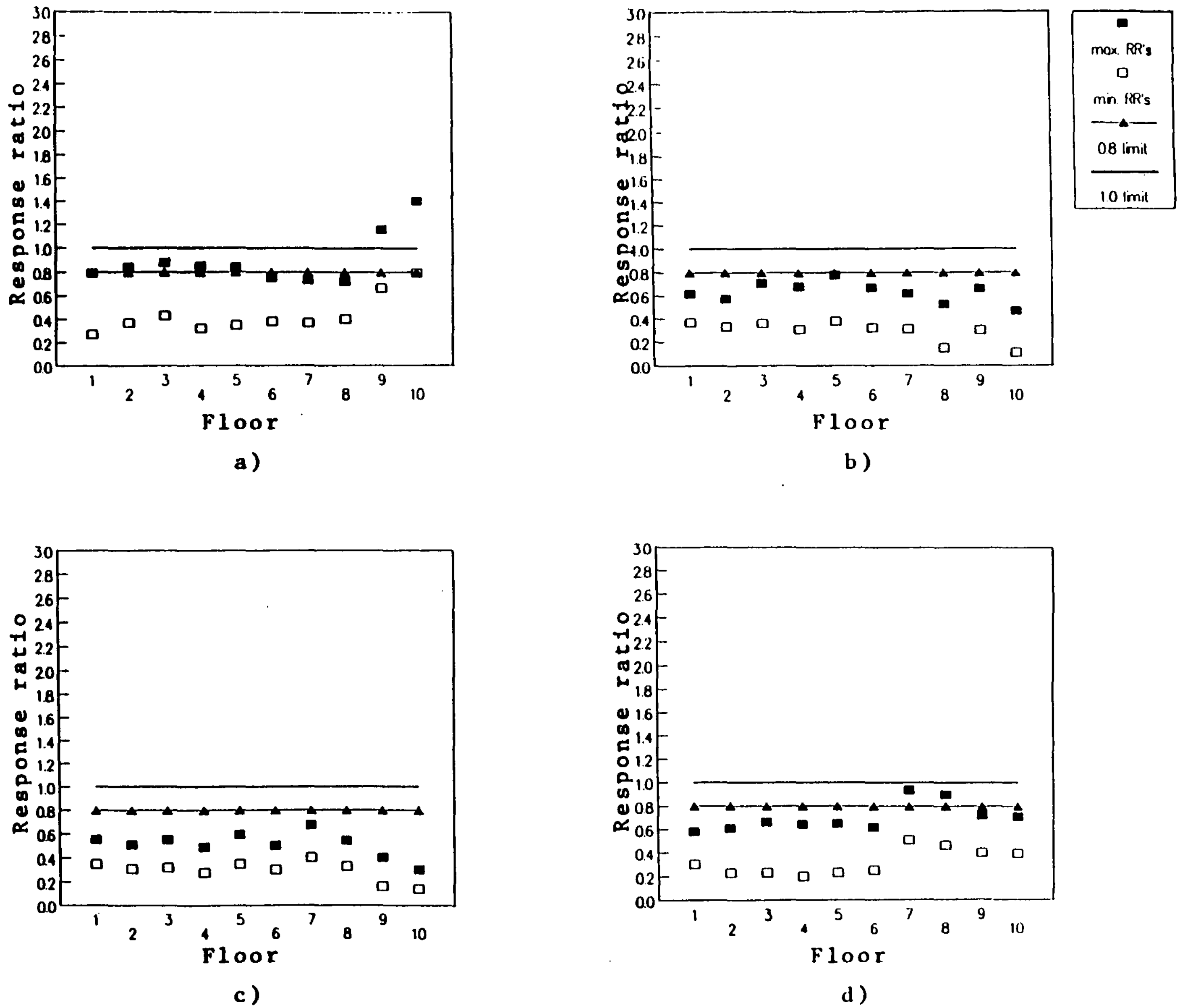


Fig.5.9: Max. and min. response ratios of columns in case 3, along column lines a)A b)B c)C d)D.

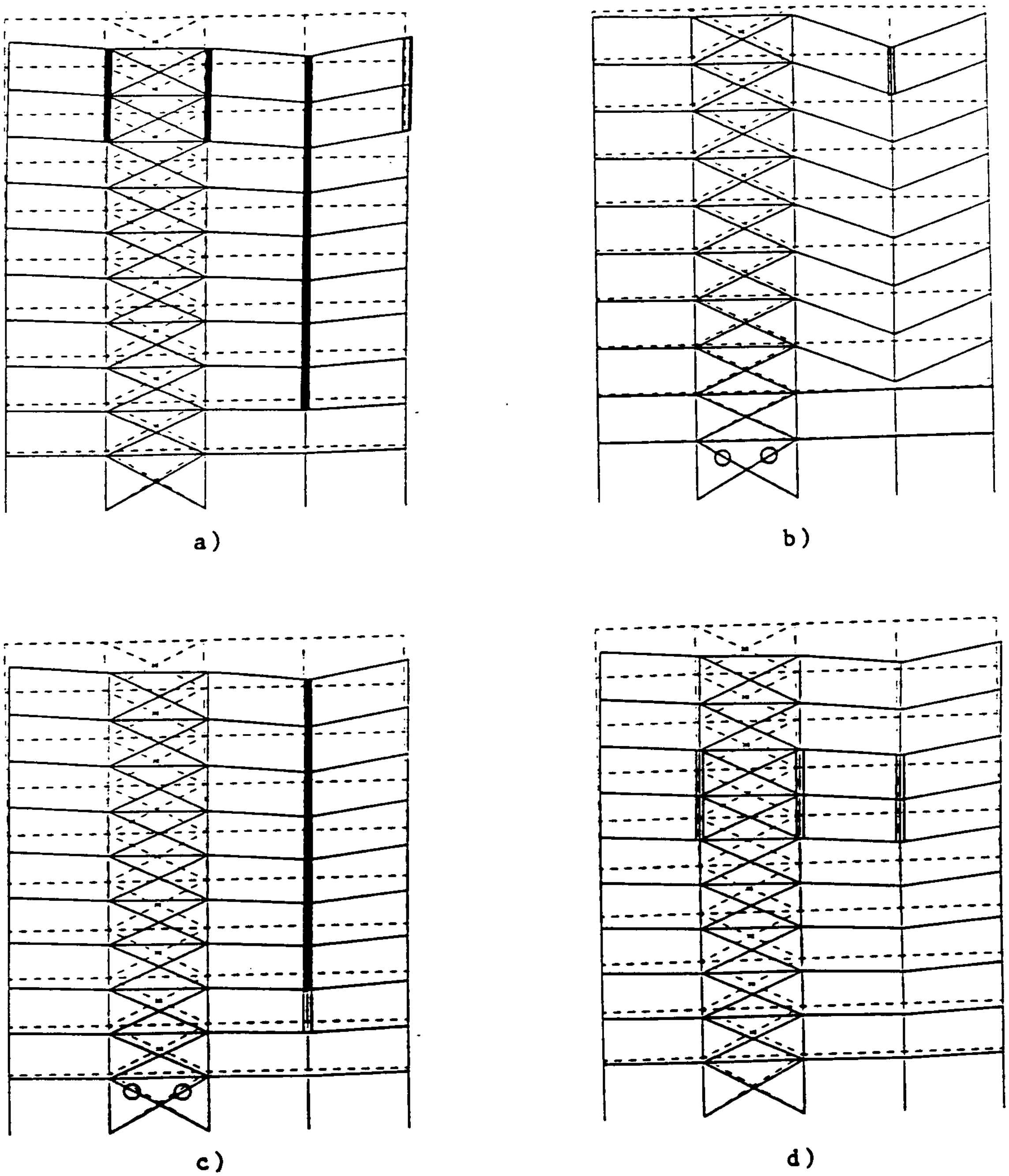
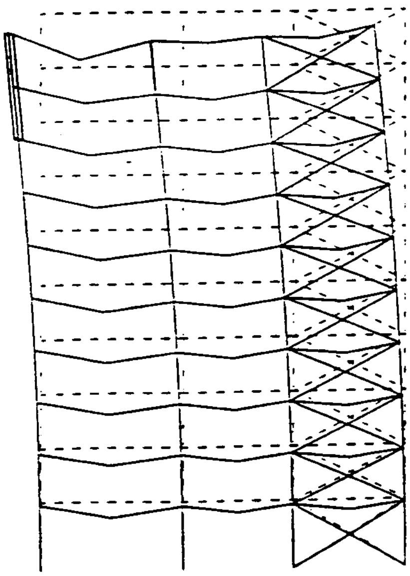
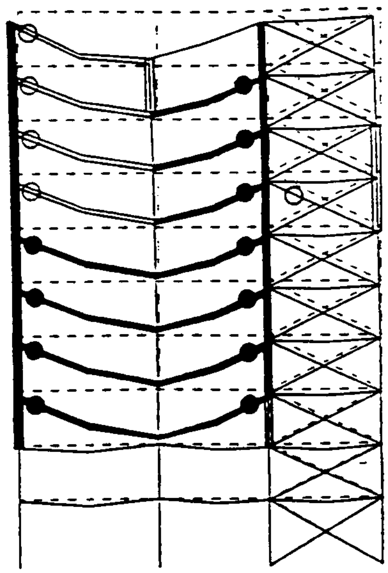


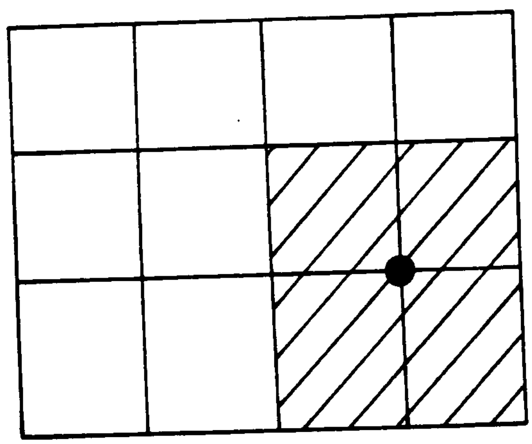
Fig.5.10: Deflected shapes, and members with RR≥0.8 in case 4, along column lines a)A b)B c)C d)D.



e)



f)



g)

Fig.5.10(cont.): e) column line 5 f)column line 4
g) floor slabs influenced by removed column.

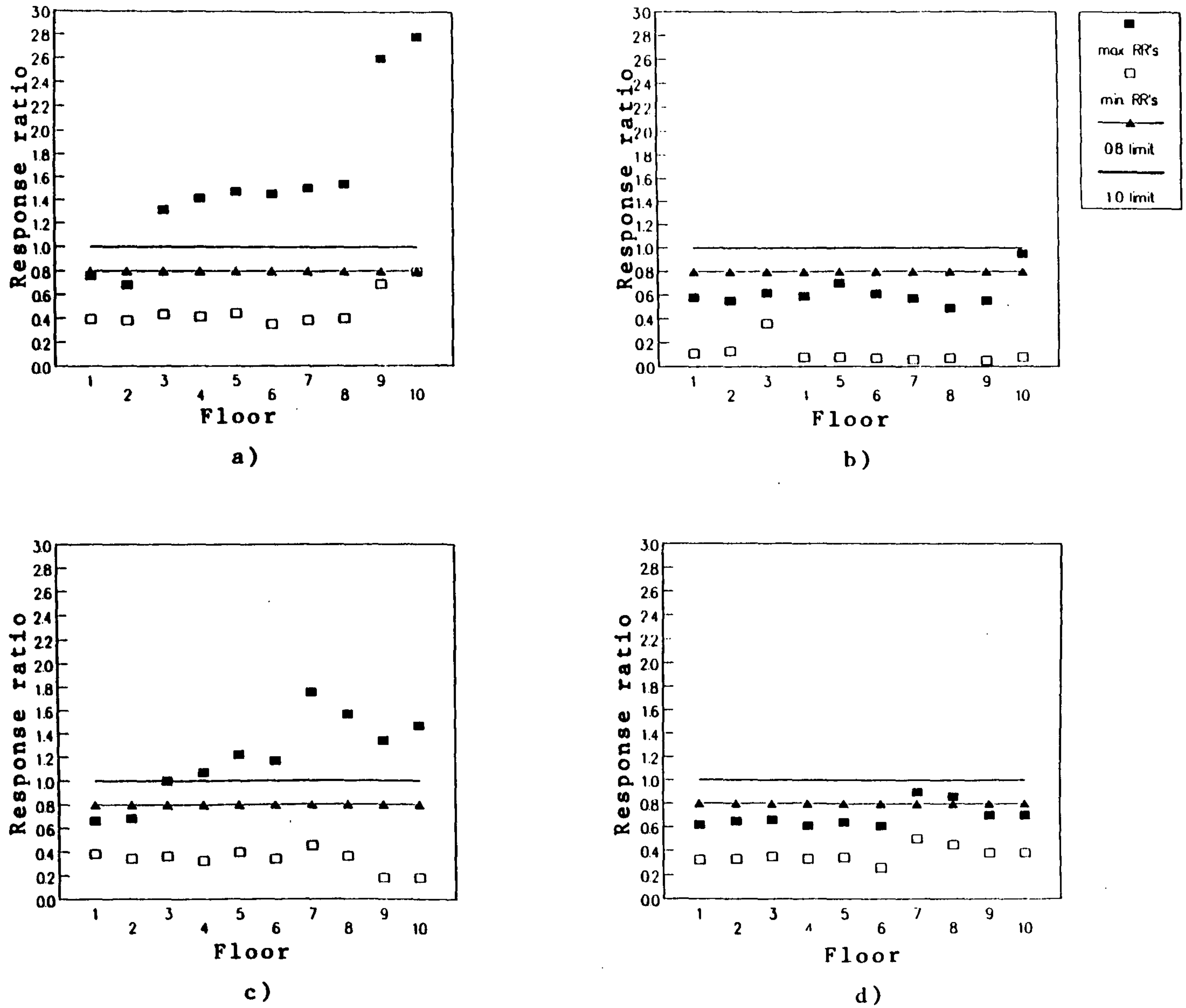
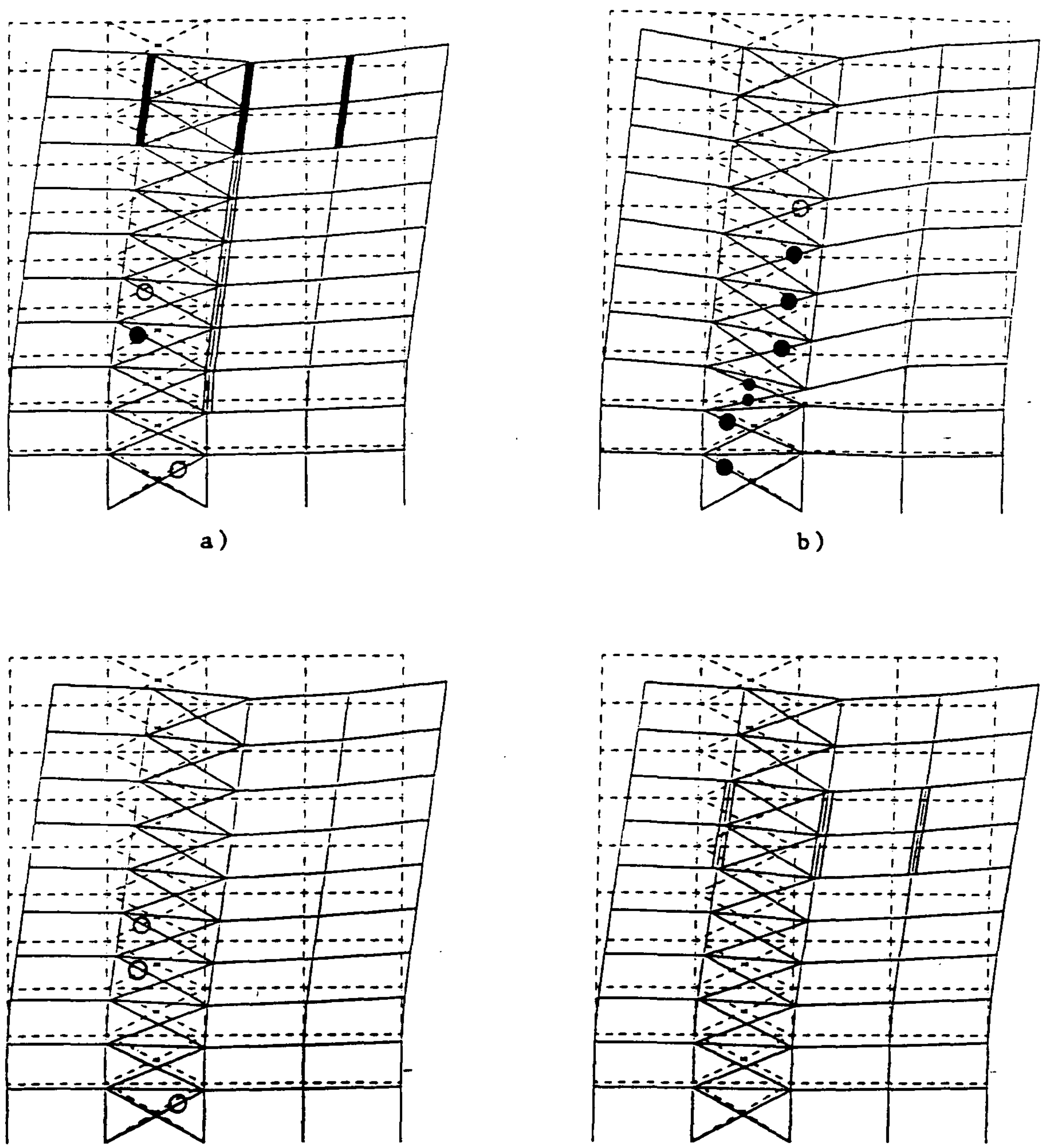


Fig.5.11: Max. and min. response ratios of columns in case 4, along column lines a)A b)B c)C d)D.



a) b) c) d)

Fig.5.12: Deflected shapes, and members with RR≥0.8 in case 5, along column lines a)A b)B c)C d)D.

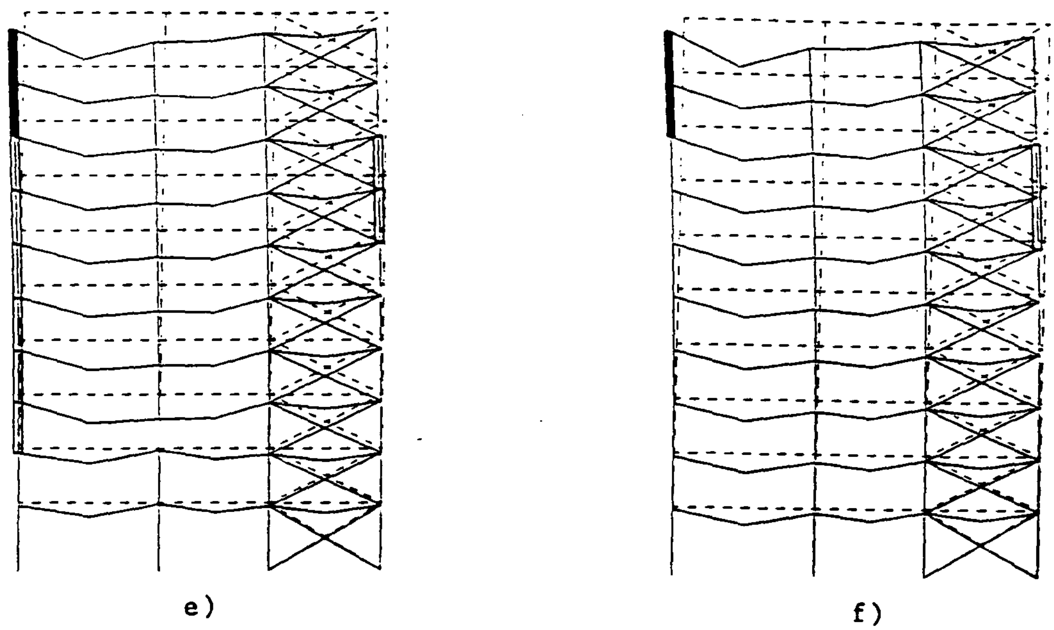


Fig.5.12(cont.): Column lines e)3 f)2.

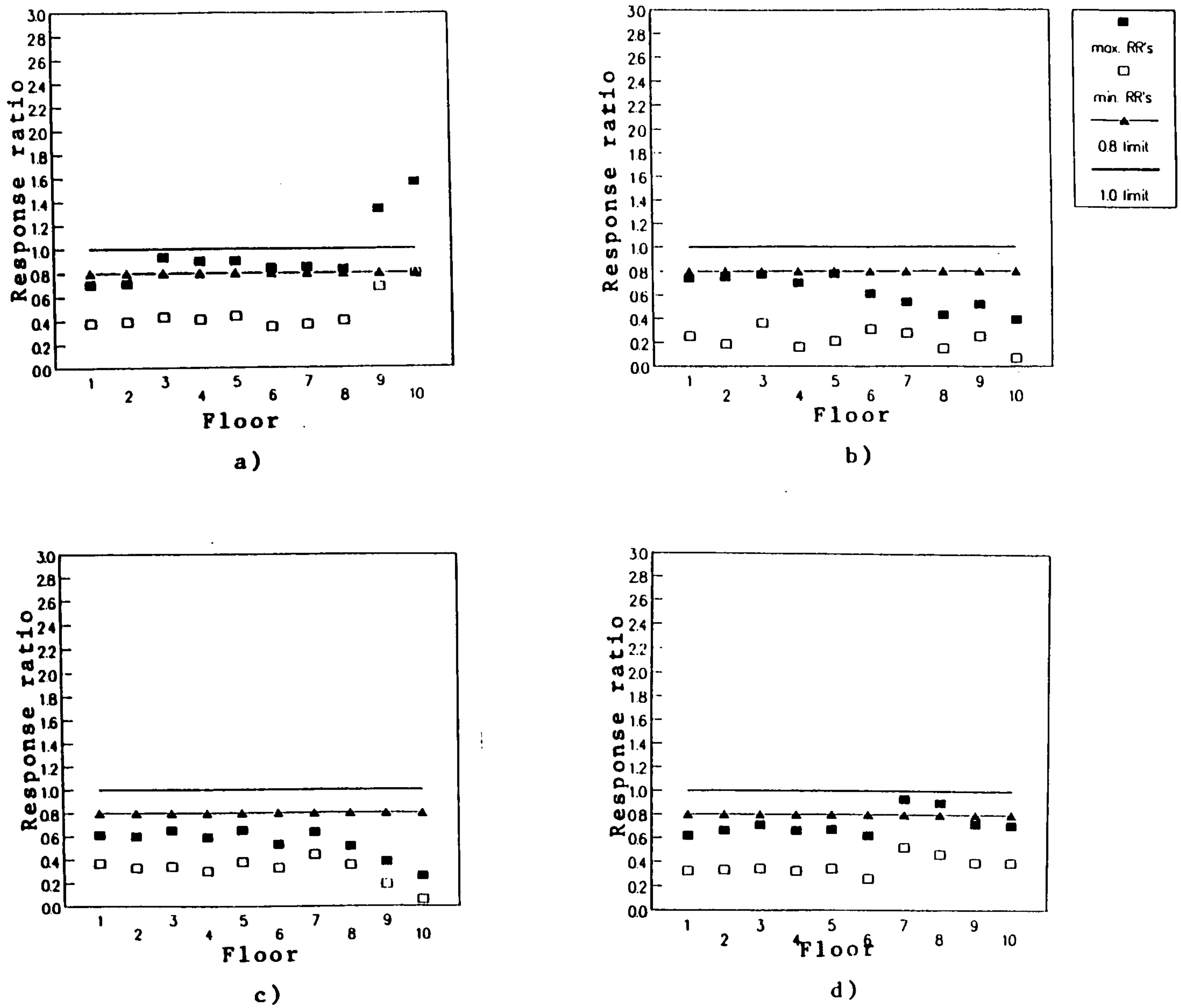


Fig.5.13: Max. and min. response ratios of columns in case 5, along column lines a)A b)B c)C d)D.

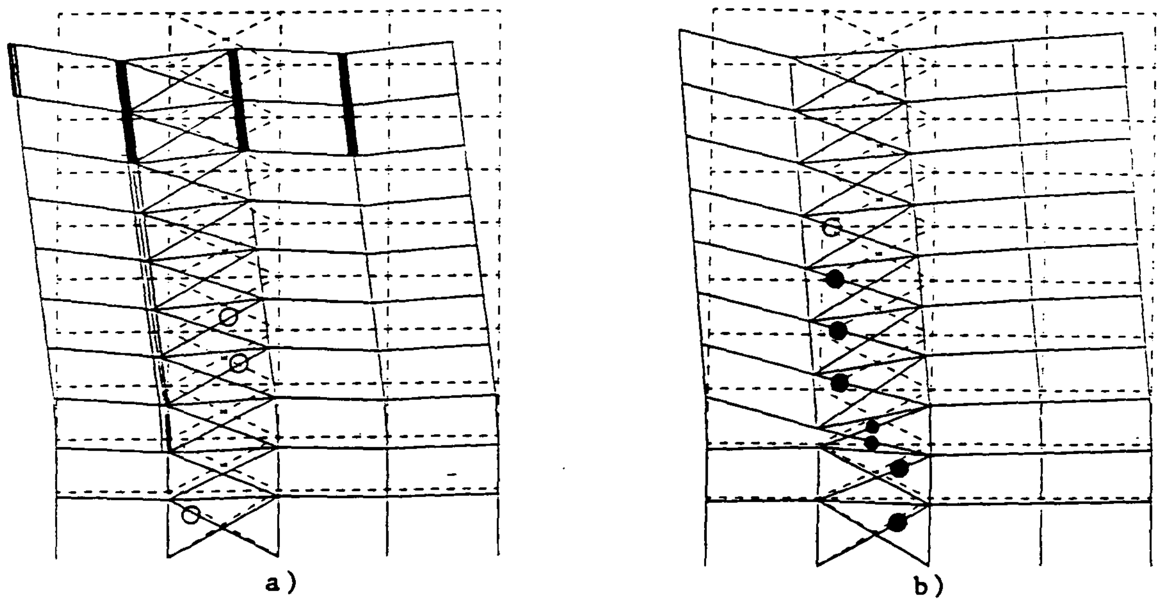
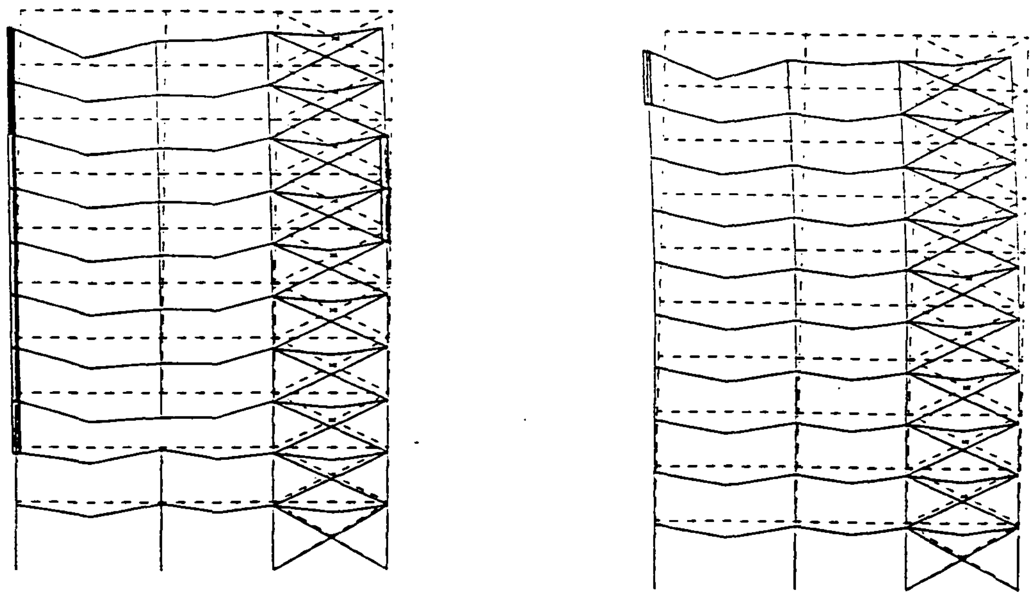


Fig.5.14: Deflected shapes, and members with RR≥0.8, in case 6, along column lines a)A b)B c)C d)D.



e) f)

Fig.5.14(cont.): Column lines e)2 f)1.

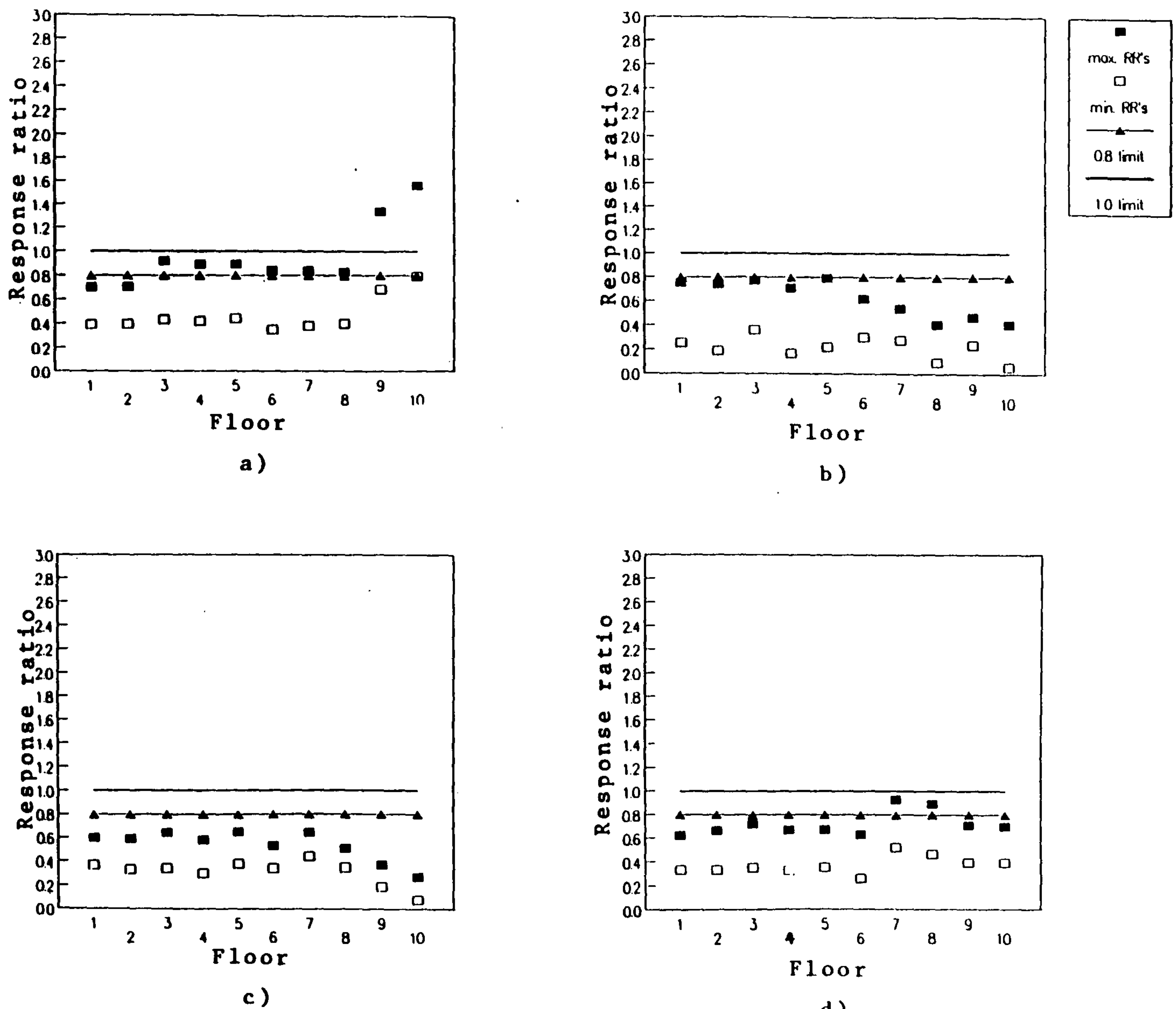


Fig.5.15: Max. and min. response ratios of columns in case 6, along column lines a)A b)B c)C d)D.

Floor	Column line							
	A		B		C		D	
	max.	min.	max.	min.	max.	min.	max.	min.
1	0.63	0.23	0.75	0.38	0.55	0.39	0.52	0.28
2	0.66	0.39	0.63	0.34	0.50	0.35	0.55	0.22
3	0.79	0.43	1.25	0.35	0.55	0.38	0.58	0.23
4	1.25	0.42	1.37	0.31	0.52	0.34	0.54	0.21
5	0.85	0.45	1.67	0.38	0.61	0.41	0.57	0.24
6	0.85	0.43	1.59	0.30	0.54	0.36	0.56	0.25
7	0.88	0.39	1.58	0.29	0.73	0.49	0.88	0.43
8	0.92	0.42	1.57	0.13	0.62	0.40	0.86	0.45
9	1.66	0.71	2.26	0.28	0.51	0.22	0.71	0.39
10	1.64	0.81	2.04	0.08	0.38	0.22	0.71	0.39

Table 5.8: Maximum and minimum response ratios for columns, in case 1.

Floor	Column line							
	A		B		C		D	
	max.	min.	max.	min.	max.	min.	max.	min.
1	0.79	0.27	0.62	0.37	0.56	0.35	0.57	0.29
2	0.83	0.37	0.58	0.33	0.52	0.31	0.59	0.23
3	0.88	0.43	0.71	0.35	0.58	0.31	0.63	0.22
4	0.87	0.33	0.66	0.31	0.50	0.24	0.61	0.20
5	0.86	0.36	0.76	0.38	0.59	0.35	0.63	0.23
6	0.76	0.33	0.65	0.32	0.49	0.29	0.61	0.25
7	0.75	0.35	0.60	0.31	0.63	0.40	0.93	0.50
8	0.73	0.38	0.50	0.15	0.52	0.32	0.90	0.45
9	1.14	0.64	0.62	0.31	0.39	0.15	0.73	0.39
10	1.40	0.77	0.42	0.12	0.31	0.13	0.72	0.39

Table 5.9: Maximum and minimum response ratios for columns, in case 2.

Floor	Column line							
	A		B		C		D	
	max.	min.	max.	min.	max.	min.	max.	min.
1	0.79	0.27	0.62	0.37	0.55	0.35	0.58	0.30
2	0.84	0.37	0.57	0.33	0.51	0.31	0.61	0.23
3	0.88	0.43	0.71	0.36	0.55	0.32	0.66	0.23
4	0.85	0.32	0.68	0.31	0.49	0.28	0.64	0.20
5	0.84	0.35	0.78	0.38	0.59	0.35	0.65	0.23
6	0.75	0.38	0.67	0.32	0.50	0.30	0.62	0.25
7	0.73	0.37	0.62	0.31	0.67	0.40	0.94	0.51
8	0.72	0.40	0.53	0.15	0.54	0.33	0.90	0.46
9	1.16	0.66	0.66	0.30	0.40	0.16	0.72	0.40
10	1.41	0.79	0.47	0.11	0.30	0.14	0.71	0.39

Table 5.10: Maximum and minimum response ratios for columns, in case 3.

Floor	Column line A max.	A min.	B max.	B min.	C max.	C min.	D max.	D min.
1	0.76	0.39	0.58	0.11	0.66	0.38	0.62	0.32
2	0.68	0.38	0.55	0.13	0.68	0.34	0.65	0.33
3	1.32	0.43	0.62	0.36	1.00	0.36	0.66	0.35
4	1.42	0.41	0.59	0.08	1.07	0.32	0.61	0.33
5	1.48	0.44	0.70	0.08	1.22	0.39	0.64	0.34
6	1.46	0.35	0.61	0.07	1.17	0.34	0.61	0.26
7	1.51	0.38	0.57	0.06	1.76	0.45	0.90	0.50
8	1.55	0.40	0.49	0.07	1.57	0.36	0.86	0.45
9	2.61	0.69	0.55	0.05	1.34	0.18	0.70	0.38
10	2.79	0.79	0.95	0.08	1.47	0.18	0.70	0.38

Table 5.11: Maximum and minimum response ratios for columns, in case 4.

Floor	Column line A max.	A min.	B max.	B min.	C max.	C min.	D max.	D min.
1	0.70	0.38	0.74	0.25	0.61	0.37	0.62	0.32
2	0.71	0.39	0.75	0.19	0.60	0.33	0.66	0.33
3	0.93	0.43	0.77	0.36	0.65	0.34	0.71	0.34
4	0.90	0.41	0.70	0.16	0.59	0.30	0.66	0.32
5	0.90	0.44	0.78	0.21	0.65	0.38	0.67	0.34
6	0.84	0.35	0.61	0.31	0.53	0.33	0.62	0.26
7	0.85	0.37	0.54	0.28	0.63	0.44	0.93	0.52
8	0.83	0.40	0.43	0.15	0.51	0.35	0.90	0.46
9	1.34	0.68	0.52	0.25	0.38	0.19	0.72	0.39
10	1.57	0.79	0.39	0.07	0.26	0.06	0.71	0.39

Table 5.12: Maximum and minimum response ratios for columns, in case 5.

Floor	Column line A max.	A min.	B max.	B min.	C max.	C min.	D max.	D min.
1	0.70	0.39	0.75	0.25	0.60	0.37	0.62	0.33
2	0.70	0.39	0.74	0.19	0.59	0.33	0.66	0.33
3	0.92	0.43	0.77	0.36	0.64	0.34	0.72	0.35
4	0.89	0.42	0.71	0.17	0.58	0.30	0.67	0.32
5	0.89	0.44	0.79	0.22	0.65	0.38	0.67	0.35
6	0.84	0.35	0.62	0.30	0.53	0.34	0.63	0.26
7	0.84	0.38	0.54	0.28	0.64	0.44	0.93	0.52
8	0.83	0.40	0.41	0.10	0.51	0.35	0.89	0.46
9	1.34	0.68	0.47	0.24	0.37	0.19	0.71	0.39
10	1.57	0.79	0.41	0.06	0.27	0.08	0.70	0.39

Table 5.13: Maximum and minimum response ratios for columns, in case 6.

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