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Design studies of the six-story steel test building, June 1983

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DESIGN STUDIES OF THE SIX STORY

STEEL TEST BUILDING

1. Introduction

The Working Group on Steel Structures of the Joint Technical Coordinating Committee for the U.S.-Japan Cooperative Research Program has recommended that a six-story, two-bay structure which would represent a portion of a complete building be adopted for the full-scale test. The floor plan of the test structure is shown in Fig. 1 and the elevations of the exterior and interior frames in Fig. 2. The two exterior frames are unbraced moment-resisting frames with one column in each oriented for weak-axis bending, and the interior frame is a braced frame with K bracing in one of the bays. The floor system consists of formed metal decking and cast-in-place lightweight concrete and acts compositely with the girders and the floor beams. Two types of K bracing system, the concentric K and the eccentric K, are to be installed in different stages of testing. Standard U.S. rolled structural shapes made of ASTM A36 Steel are to be used for the structural members (except the braces).

The test structure is designed to satisfy the requirements of both the 1979 Uniform Building Code (UBC) and the 1981 Japanese building code. In some respects, the design requirements in the two codes are significantly different. A major difference is in the magnitude of the base shear used in the static design procedure. However, it has been possible to reach a suitable compromise for the base shear to be used in the design of the test structure. Because of other differences in the codes, it is necessary to perform two structural designs, one in the U.S. based on the UBC and the other in Japan based on the Architectural Institute of Japan code. The two designs are compared and the final member sizes of the test structure are then selected. Static and dynamic analyses of the structure are carried out to study its behavior in the elastic and inelastic range. (This work is still in progress.) Presented in this report is a summary of the design based on the UBC and selected results of computer analyses already completed.

2. GENERAL DESIGN CRITERIA AND ASSUMPTIONS

The following design criteria and assumptions have been agreed upon by the U.S. and Japanese investigators.

2.1 Gravity loads

The gravity loads assumed in the U.S. and Japanese designs are shown in Table 1. The U.S. values are adopted in the UBC design.

2.2 Base shear coefficient and earthquake lateral forces

The UBC design base shear coefficient adopted for the design of the test structure is 0.113. Details of the derivation of this coefficient from the codes of both countries are shown in Table 2. In calculating the design earthquake lateral forces, the live loads and wall weights are not included.

2.3 Material

Girders, floor beams and columns: wide-flange shapes made of ASTM A36 steel

Braces: square or rectangular of ASTM A500 Grade B steel

Slabs: lightweight concrete, approximate dry weight = 105 pcf

f'_c = 4000 psi (changed to 3000 psi in final design)

Decking: 3" QL750-16 or equivalent (3" QL-99-16 is adopted in final design)

Studs: 3/4" (19ϕ)

2.4 Design assumptions

1. All the girder-to-column connections are designed as moment connections in the loading direction and shear connections in the transverse direction.

2. Member design is based on bare member strength only.

3. Girders in braced frame are designed without considering the supporting effect of the braces.

4. Braces are designed to resist both tension and compression. The braces in the eccentric bracing system are not to buckle before the formation of shear links in the girders.

5. Since concentric and eccentric K bracing systems are to be installed in the two bays at different stages of testing, the member sizes in both bays are to be symmetrical.

3. STRUCTURAL DESIGN OF THE TEST BUILDING

3.1 Design base shear

The UBC base shear assumed in the design is 0.113W, where W is the total dead load of the structure minus the weight of the walls. Fifty percent of the base shear is to be resisted by the moment frames (the interior braced frame without the braces is considered as a moment frame). Each of the exterior frames and the interior frame~~s~~ are designed for a base shear of 0.056W/3. The bracing system is designed to resist 1.25 x 0.113W. The interior frame with the bracing system can resist a combined base shear of $0.056W/3 + 1.25 \times 0.113W = 0.160W$. The total design base shear of the structure is 0.197W.

3.2 Earthquake lateral forces

The UBC vertical distribution is used to determine the equivalent static forces at each floor level. The calculation of the lateral forces and the profile of design shear coefficients along the height of the building are shown in Table 3. A comparison of the story shear coefficients of the U.S. and Japanese designs is shown in Fig. 3. The lateral forces at the various floor levels of the exterior and interior frames are given in Figs. 4 and 5. Also shown in the figures are the gravity loads acting on the girders and columns.

3.3 Member selection process

A set of preliminary member sizes was first selected based on an approximate structural analysis. With these member sizes, a conventional elastic indeterminate analysis was carried out and bending moments and axial forces in the various members were obtained. The adequacy of the member sizes was checked against the provisions contained in Part 1 (allowable - stress design) of the American Institute of Steel Construction (AISC) specification. For the girders, because of the restraining effect provided by the composite slab, no reduction in the allowable stress due to lateral-torsional buckling was considered. The preliminary member sizes were modified so that the maximum stresses in the members would be close to the specified allowable stresses. The member sizes thus obtained were compared with those of the Japanese design and when differences occurred, compromise sizes were adopted. The final member sizes selected are shown in Figs. 6, 7 and 8.

Few remarks about selection of the bracing sizes are in order here. For the case of concentric bracing, the decision to use square structural tubes made of A570, Grade B steel (yield stress = 46 ksi) was made following a Japanese recommendation. The braces are likely to buckle out of the plane of the frame. In the design of the braces, the critical buckling stresses of the tubes were determined using an effective column length factor of 1.0. The sizes of the eccentric braces were recommended by Messrs. H. J. Degenkolb and E. P. Popov. They were selected to ensure the development of shear links in the girders and are considerably larger than those selected for the concentric case. They are not expected to buckle during the test.

4. ELASTIC ANALYSES OF THE PLANAR FRAMES

Elastic analyses of the individual frames under the working dead and live loads and the design earthquake forces have been performed. For each frame, two separate analyses are made, one assuming no composite action between the girders and the slabs, and the other assuming full composite action. The properties of the composite girders are determined using a Japansese procedure.* Also, the earthquake forces are assumed to act either from the left or from the right. The resulting bending moment diagrams and axial force distributions are shown in Figs. 9 through 20.

Moment frame without composite action --- Figs. 9 and 10

Moment frame with composite action --- Figs. 11 and 12

Concentrically braced frame without composite action --- Figs. 13 and 14

Concentrically braced frame with composite action --- Figs. 15 and 16

Eccentrically braced frame without composite action --- Figs. 17 and 18

Eccentrically braced frame with composite action --- Figs. 19 and 20

The member sizes selected for the frames are checked again for the bending moment and axial force values shown in the figures. Many members have been found to be very conservatively designed.

*"Recommendations for the Design and Construction of Composite Beams", Architectural Institute of Japan, 1975.

5. ELASTIC ANALYSES OF THE BUILDING

The elastic response of the test building is studied analytically using a computer program developed previously for analyzing multistory building structures with flexible floors. The building is first analyzed statically for the design earthquake forces. Dynamic analyses are then performed using the El Centro (N.S.) and Miyagiken-oki ground acceleration records. The results of these studies are presented in this section.

5.1 Method of Analysis and Assumptions

The computer program used in the analyses is based on a substructuring technique and utilizes a continuous medium representation of the vertical structural elements, such as frames, walls and floor slabs. The 3-D structure is divided into vertical and horizontal substructures, and the analysis is carried out by treating the overall building as a planar intersecting-member system subjected to loads perpendicular to its plane. The displacement method is then used to solve the planar system.

The following assumptions are made in the analysis:

1. The in-plane deformation of the floor slab in included. For the test building this effect is found to rather small.
2. The rigidity variation along the height of the frames is considered.
3. In the braced frame, the deformations of the braces and columns are included.
4. The stability effects of the vertical loads on the lateral stiffness of the frames are considered.
5. The torsional rigidities of substructures are assumed to be small as compared to the in-plane rigidities and are neglected.

5.2 Static Analyses

The static behavior of the test structure subjected to the design earthquake forces is shown in Fig. 21. The lateral deflection at the roof level is 2.07 inches, corresponding to a drift index of 0.0024. The calculated distribution of the story shears between the moment frame and the braced frame are presented in Fig. 22 together with the distribution assumed in the design. The differences are noticeable, but not very large. A comparison of the calculated and the assumed distributions can also be found in Table 4.

5.3 Dynamic Analysis

In the dynamic analysis, the live loads and wall weights are not included in the calculation of the masses and a damping of 5% is assumed. For the El Centro earthquake record, the maximum base shear is found to be 975 kips, which is 3.65 times of the design base shear. The distribution of the story shears and the lateral displacements of the structure are shown in Fig. 23. The results of the Miyagiken-oki record are presented in Fig. 24. The calculated maximum base shear is 6.45 times of the design value. The distributions of the story shears between the moment frames and the braced frame of the test structure for the two earthquakes are summarized in Table 4. The story shear distributions from the dynamic analyses are the same as those of the static case.

6. SUMMARY

The design of the six-story steel test building, based on the Uniform Building Code, has been described in this report. A separate design has also been made in Japan to satisfy the Japanese code requirements. The total base shears assumed in the two designs are identical. The member sizes of the test structure have been selected by comparing the results of these designs.

Static and dynamic, elastic and inelastic analyses of the test structure with the selected member sizes have been carried out and the results of elastic analyses have been presented. A subsequent report will present the results of inelastic analyses including the effects of bracing buckling and panel zone deformation at the girder-to-column connections.

Table 1 Gravity Loads Assumed for Design

(1 psf = 4.88 kg/m^2)

DEAD LOADS	US (psf)	Japan (kg/m^2)
Floor		
Metal Deck	6	18
3-1/2" Lightweight Concrete	39	221
Ceiling and Floor Finishes	10	60
Partitions, etc.	20	50
	75 (366 kg/m^2)	349 (72 psf)
Structural Steel and Fireproofing	15	90
	90 (439 kg/m^2)	439 (90 psf)
Roof		
Metal Deck	6	18
Lightweight Concrete	39	221
Ceiling and Roofing	20	100
	65 (317 kg/m^2)	339 (69 psf)
Structural Steel and Fireproofing	10	70
	75 (366 kg/m^2)	409 (84 psf)
Exterior Wall Weight	30 psf wall surface	140 kg/m^2
LIVE LOADS		
Slabs and Beams	60	300
Girders	37	180

Table 2 Base Shear Coefficients

US (UBC)

T = 0.504, C = 0.094, K = 0.8, S = 1.5

V = 0.113 W $\quad V = ZKCISW = 0.1128\,W$

Braced bay 1.25 x 0.113 = 0.141

Moment frames 0.113 x 0.5 = 0.056

Total shear coefficient

0.141 + 0.056 = 0.197

Japan

Moment frames 34%

Each bay $\dfrac{0.197 \times 0.34}{6} = 0.0112$

Bracing 66%

0.197 x 0.66 = 0.130

Braced bay 0.130 + 0.0112 = 0.1412

Moment frames

0.0112 x 5 = 0.056

Table 3 Lateral Forces for Design

Story	W_i (kips)	ΣW_i (kips)	F_i (kips)	Q_i (kips)	Each Moment Frame (kips)	Braces (kips)	Braced Frame (kips)	$\overline{Q}_i$ (kips)	$C_i = \frac{\overline{Q}_i}{\Sigma W_i}$
R	193.73	193.73	36.90	36.90	6.11	46.04	52.15	64.37	0.332
6	232.47	426.20	37.27	74.17	12.27	92.55	104.82	129.36	0.304
5	232.47	658.67	30.27	104.44	17.28	130.32	147.60	182.16	0.277
4	232.47	891.14	23.27	127.71	21.13	159.36	180.49	222.75	0.227
3	232.47	1123.61	16.27	143.98	23.82	179.66	203.48	251.12	0.223
2	232.47	1356.08	9.26	153.24	25.35	191.22	216.57	267.27	0.197
				1356.08 × 0.113	1356.08 × $\frac{0.056}{3}$	153.24 × 1.25	191.22 + 25.35	216.57 + 2 × 25.35	

W_i = story weight

F_i = lateral force at floor level (base shear coefficient = 0.113)

Q_i = story shear (base shear coefficient = 0.113)

$\overline{Q}_i$ = total story shear of test structure

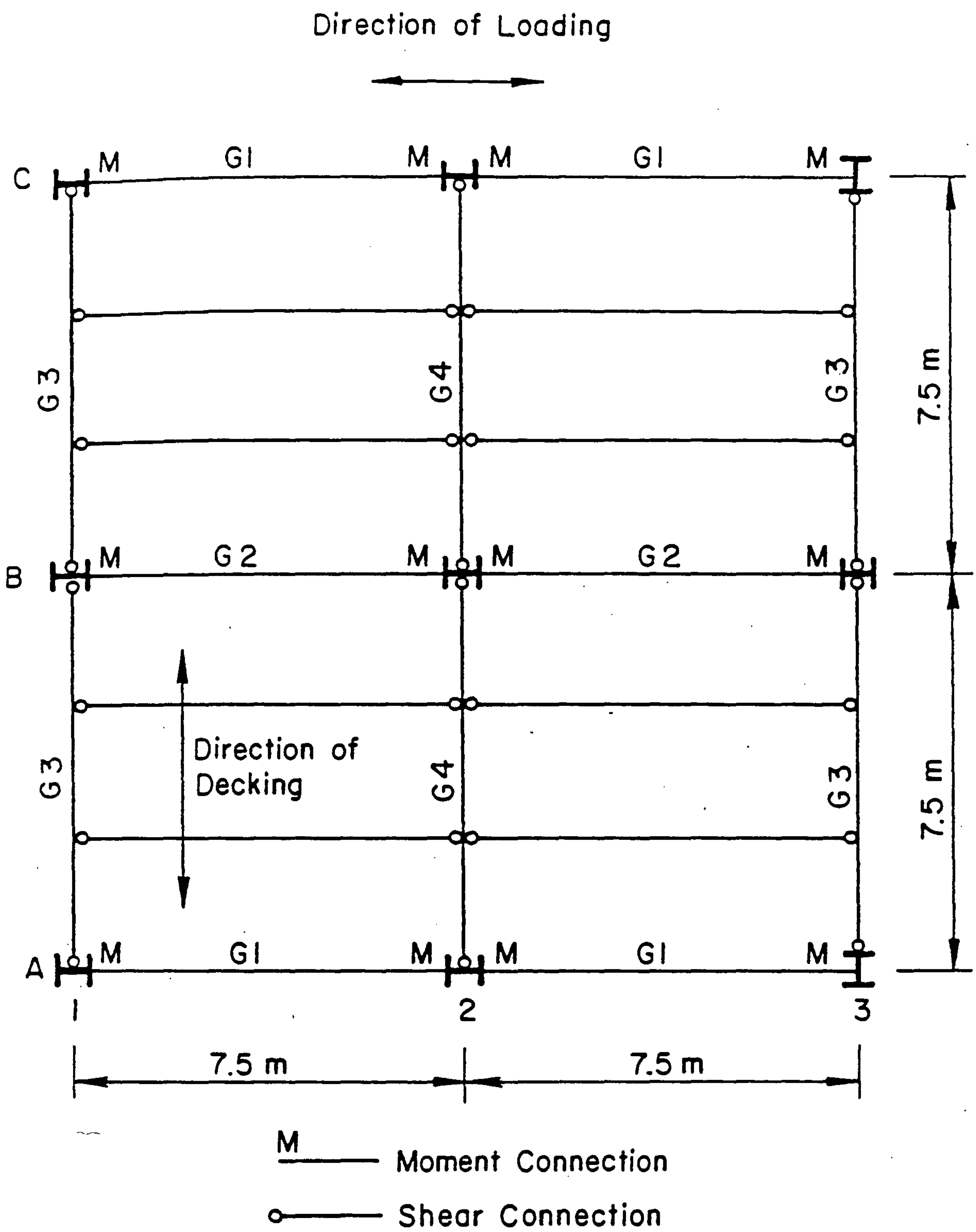
C_i = story shear coefficient

Table 4 Elastic Story Shear Distribution Between Moment Frames and Concentrically Braced Frame

Story	Design		Static Analysis		El Centro N.S.		Miyagiken-oki	
	Moment Frames	Braced Frame	Moment Frames	Braced Frame	Moment Frames	Braced Frame	Moment Frames	Braced Frame
R-6	19%	81%	17%	83%	17%	83%	17%	83%
6-5	19%	81%	14%	86%	14%	86%	14%	86%
5-4	19%	81%	14%	86%	14%	86%	14%	86%
4-3	19%	81%	15%	85%	15%	85%	15%	85%
3-2	19%	81%	14%	86%	14%	86%	14%	86%
2-1	19%	81%	24%	76%	24%	76%	24%	76%

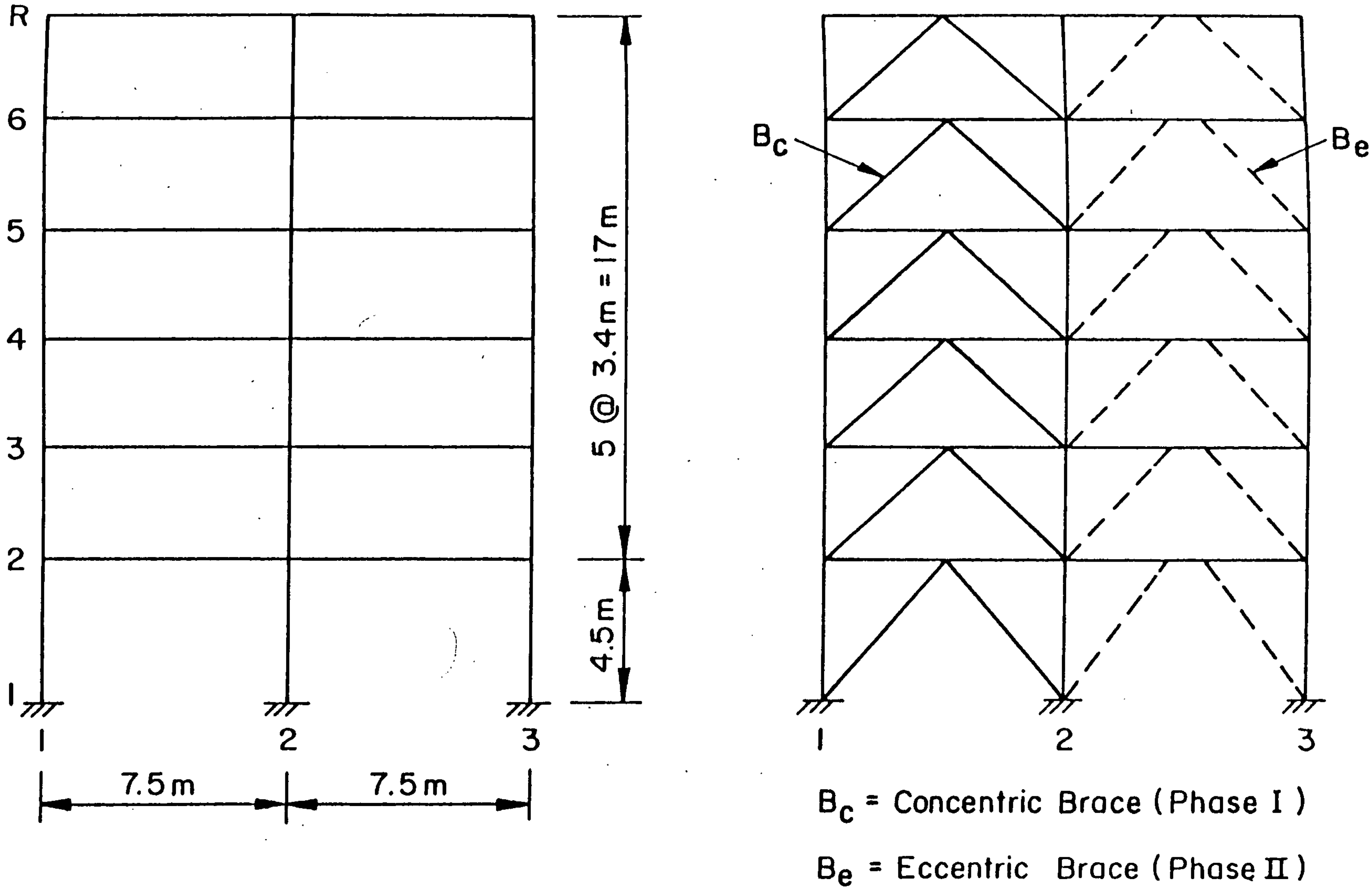
$0.141 + \frac{0.056}{6} \times 2$

0.197



Slab extends 0.5m beyond G1

Fig. 1 Plan of Test Structure



Exterior Frames A and C

Interior Frame B

Fig. 2 Elevation Views of Exterior and Interior Frames

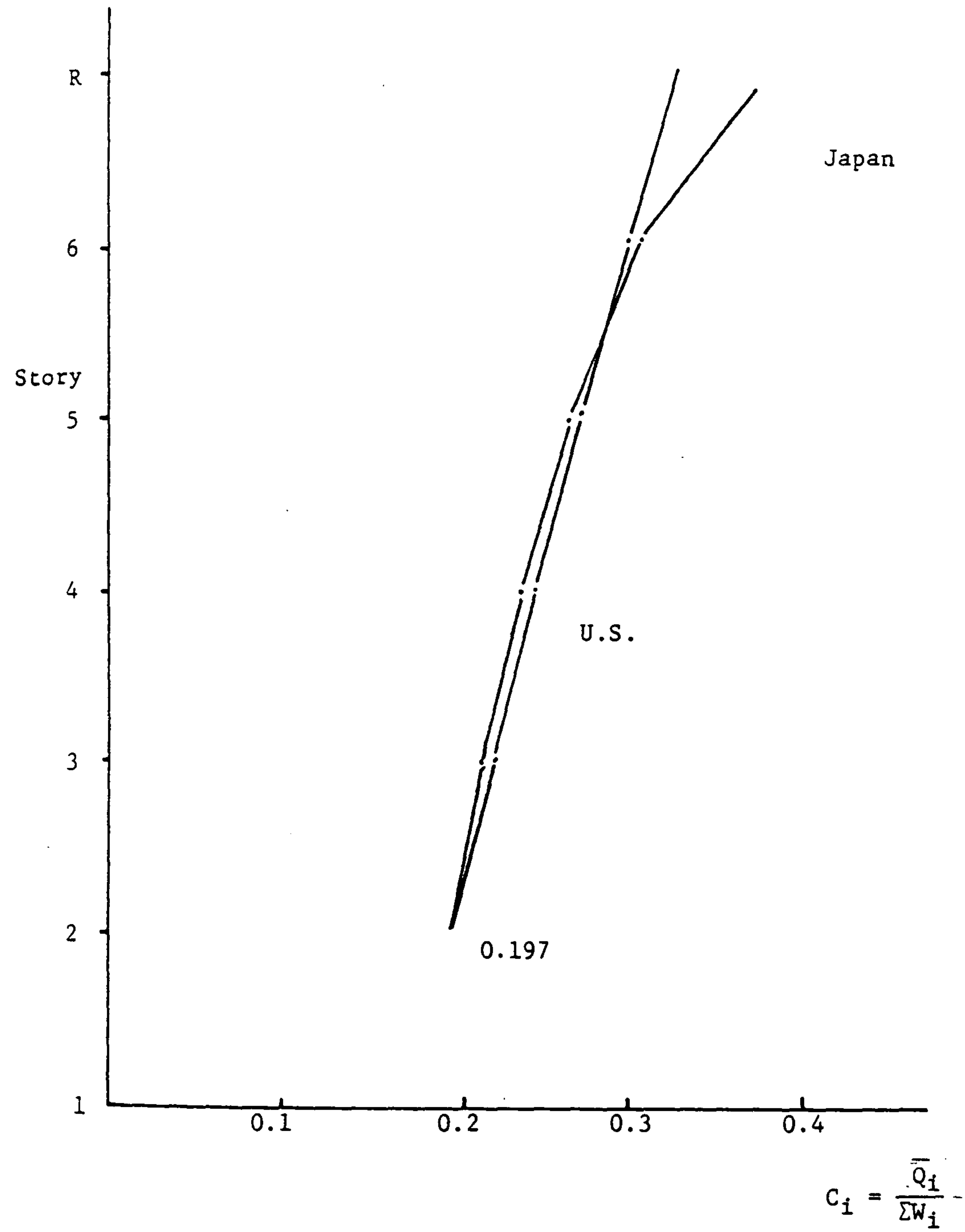


Fig. 3 Story Shear Distribution

FRAMES A AND C

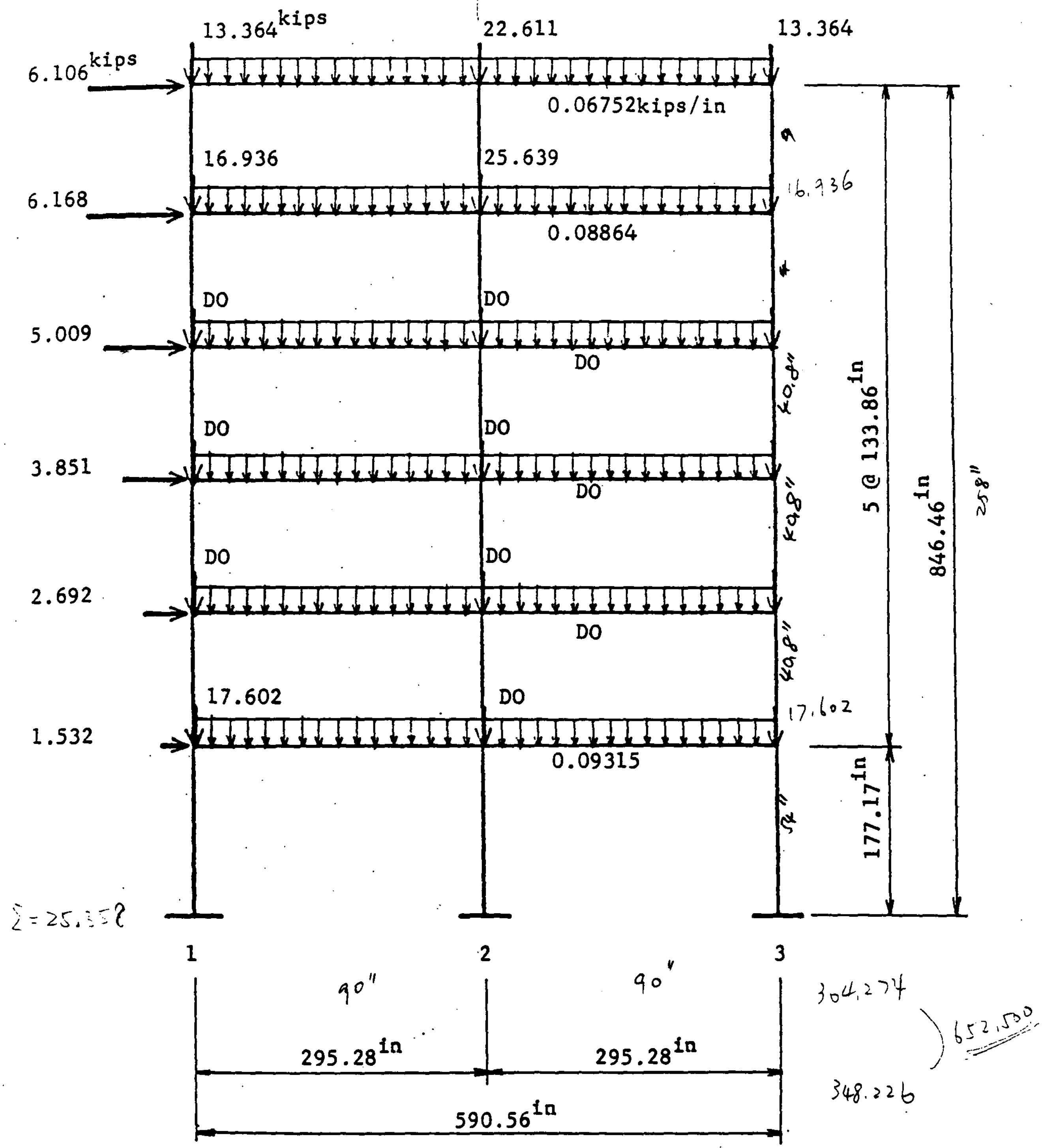


Fig. 4 Gravity and Earthquake Lateral Forces of Exterior Frames

FRAME B

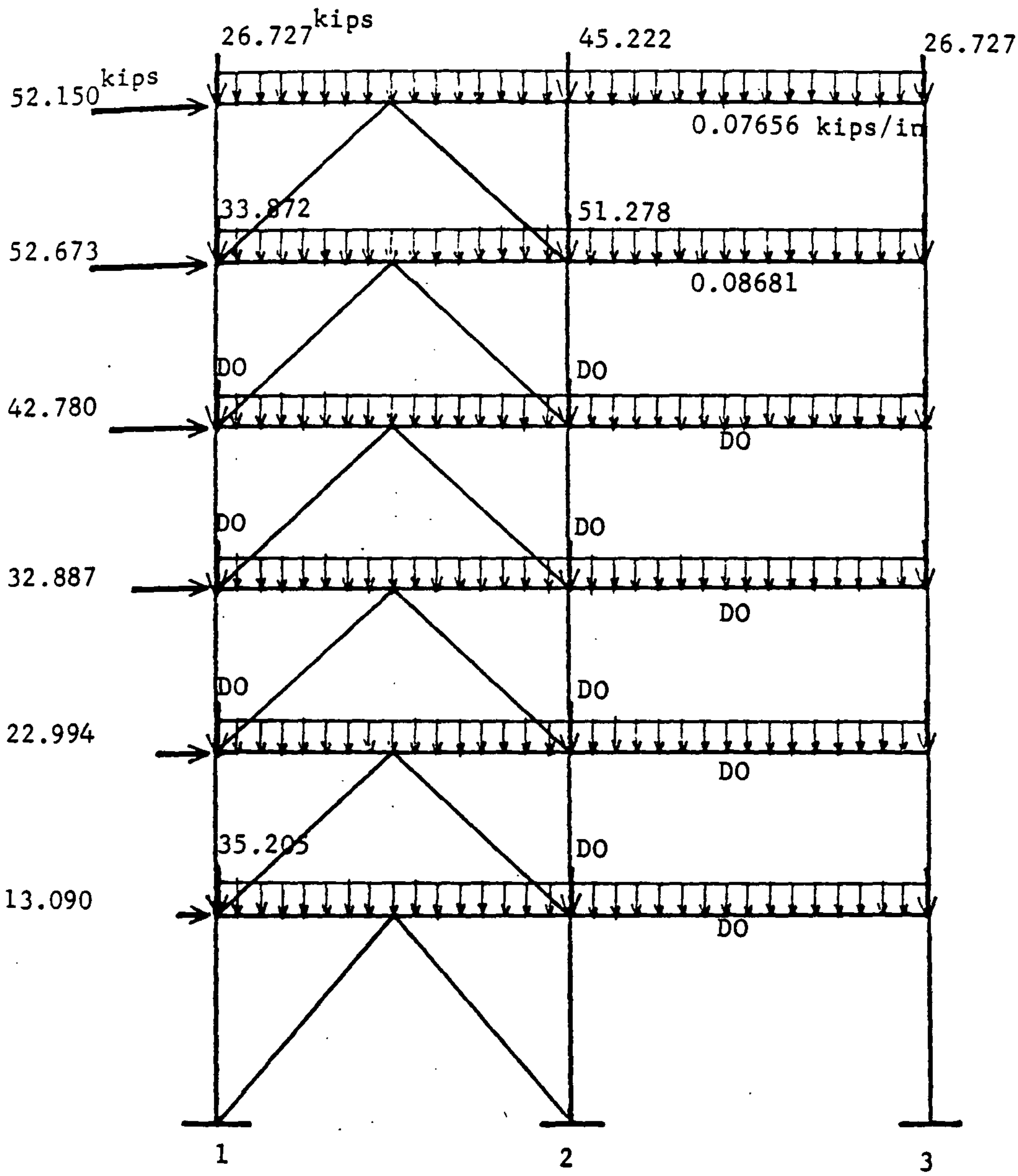
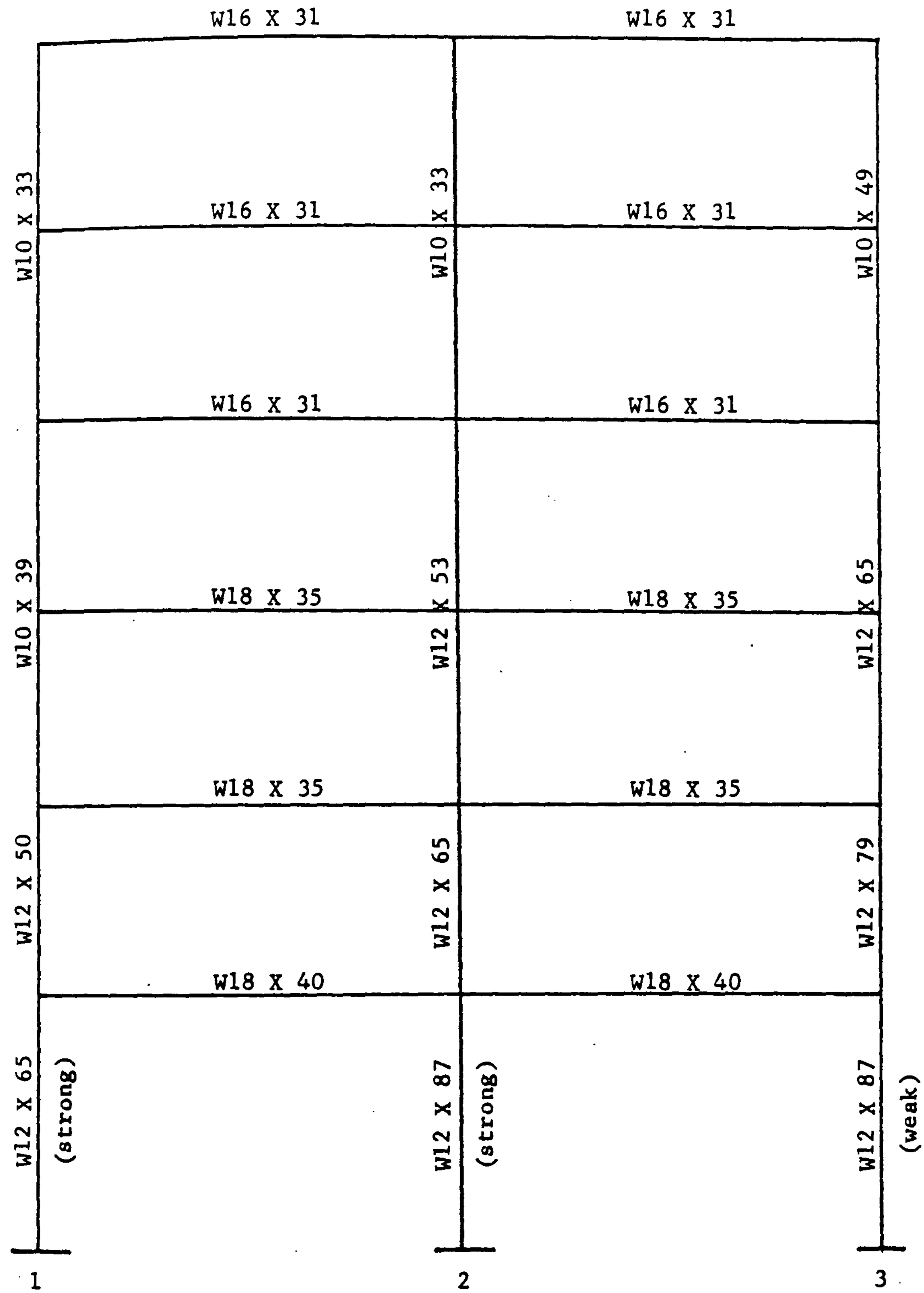


Fig. 5 Gravity and Earthquake Lateral Forces of Interior Frame

Frames A & C



Note: Girders for lines 1 & 3, G_3 (all levels) W18 X 35
Girders for line 2, G_4 (all levels) W21 X 50
Floor beams (all levels) W16 X 31

Fig. 6 Member Sizes of Moment Frames

Frame B

(Concentric Bracing)

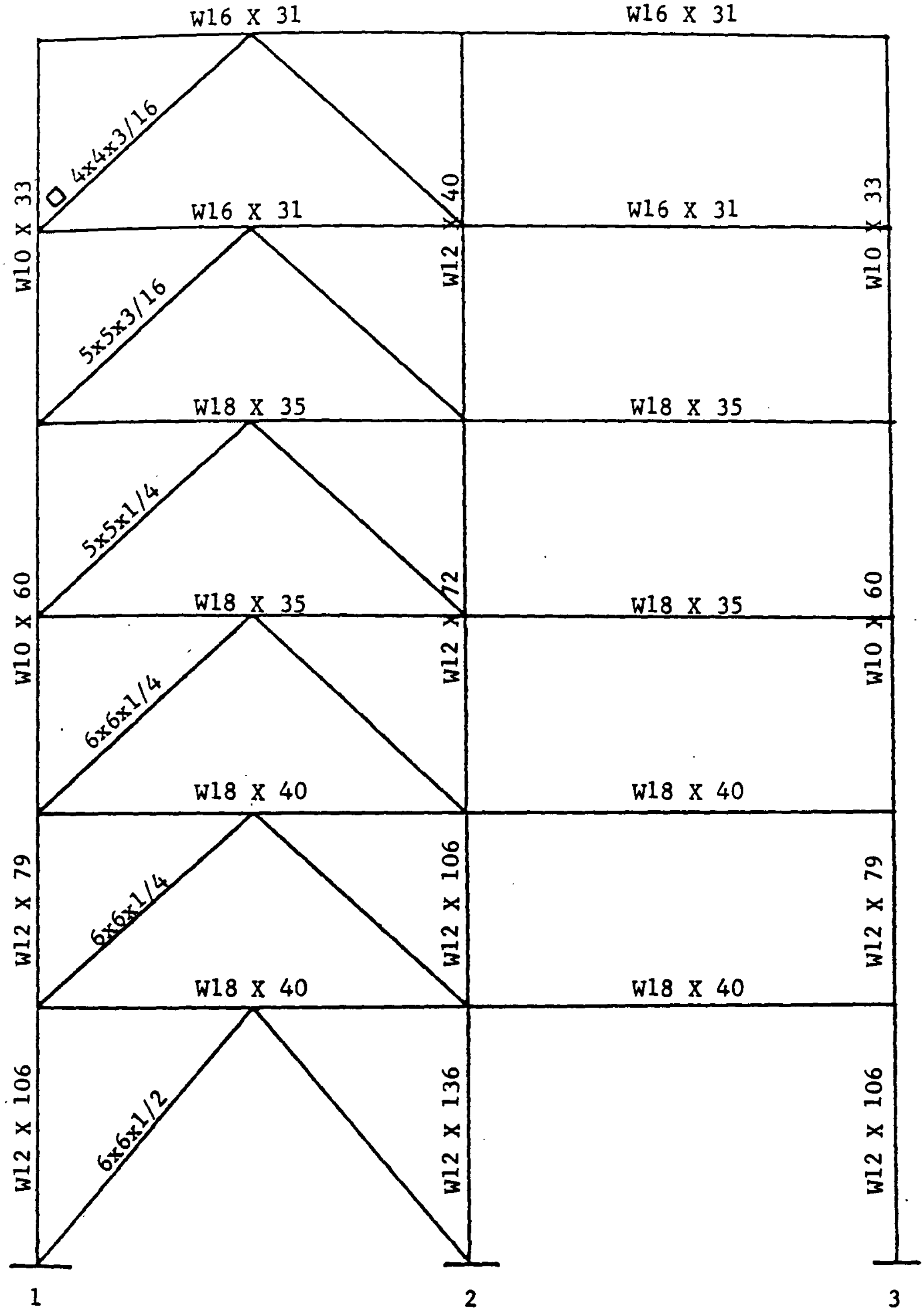


Fig. 7 Member Sizes of Braced Frame with Concentric Braces

Frame B

(Eccentric Bracing)

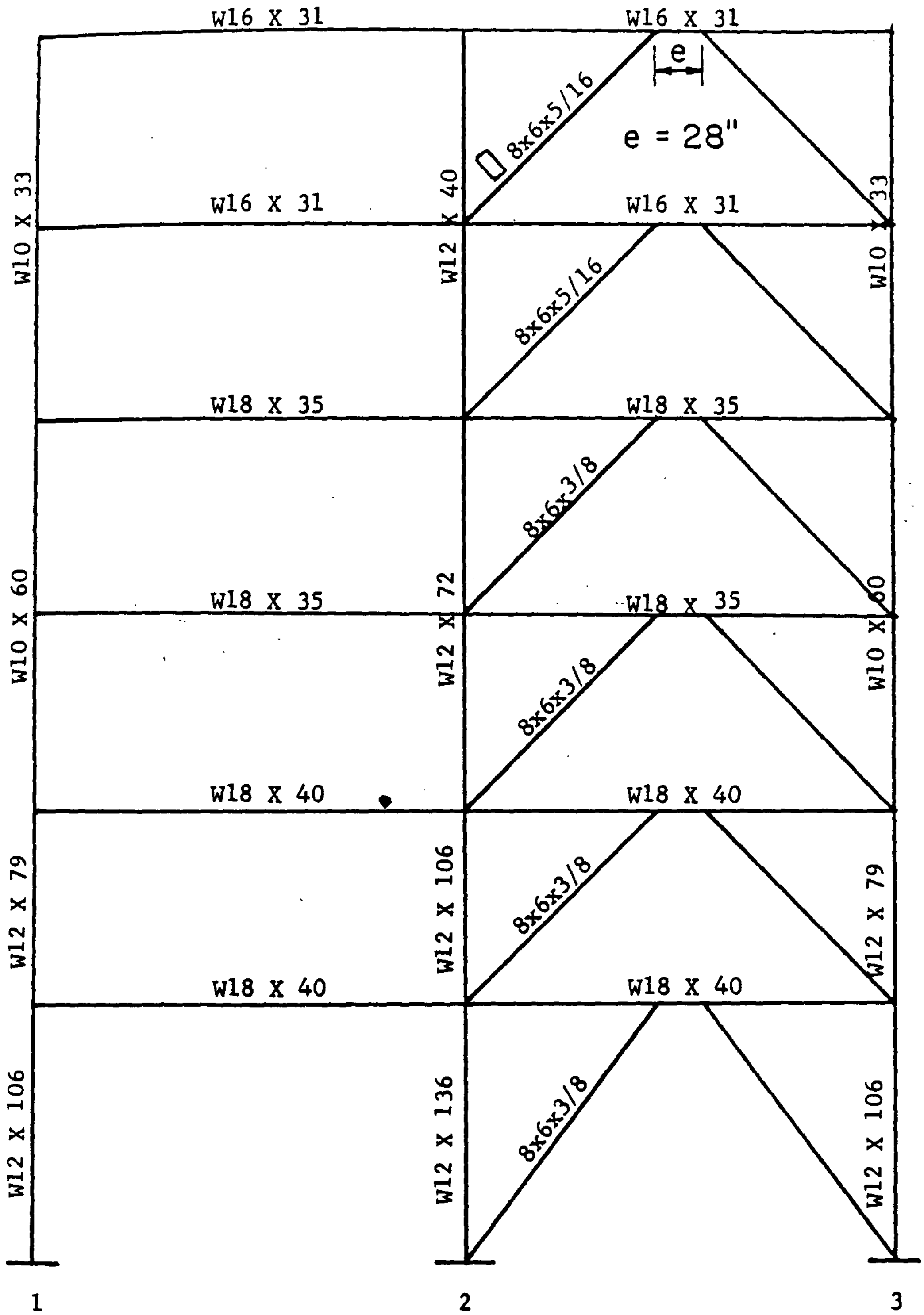


Fig. 8 Member Sizes of Braced Frame with Eccentric Braces

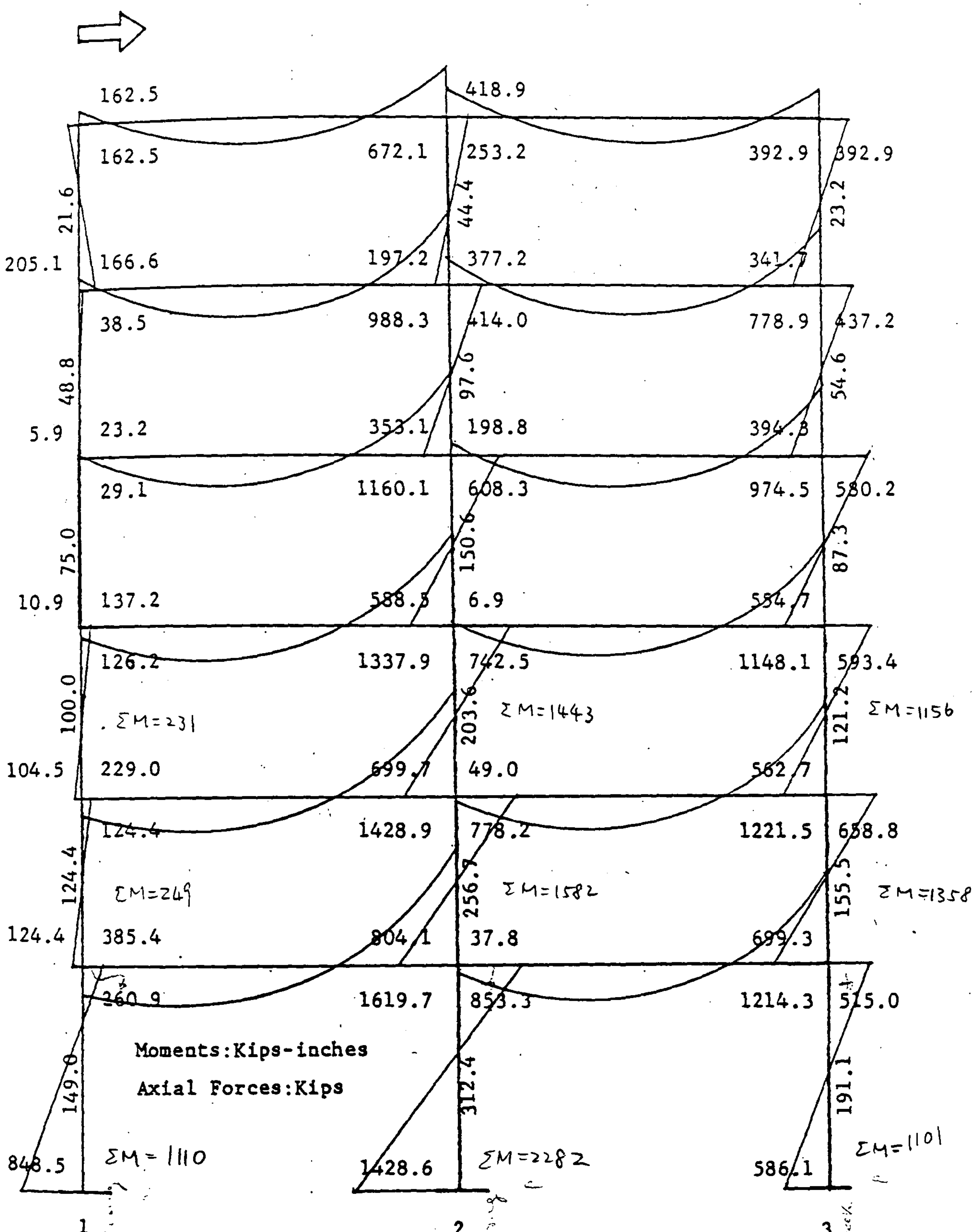


Fig. 9 Moment Diagram and Axial Forces of Moment Frame without Composite Action (Earthquake Froces from Left)

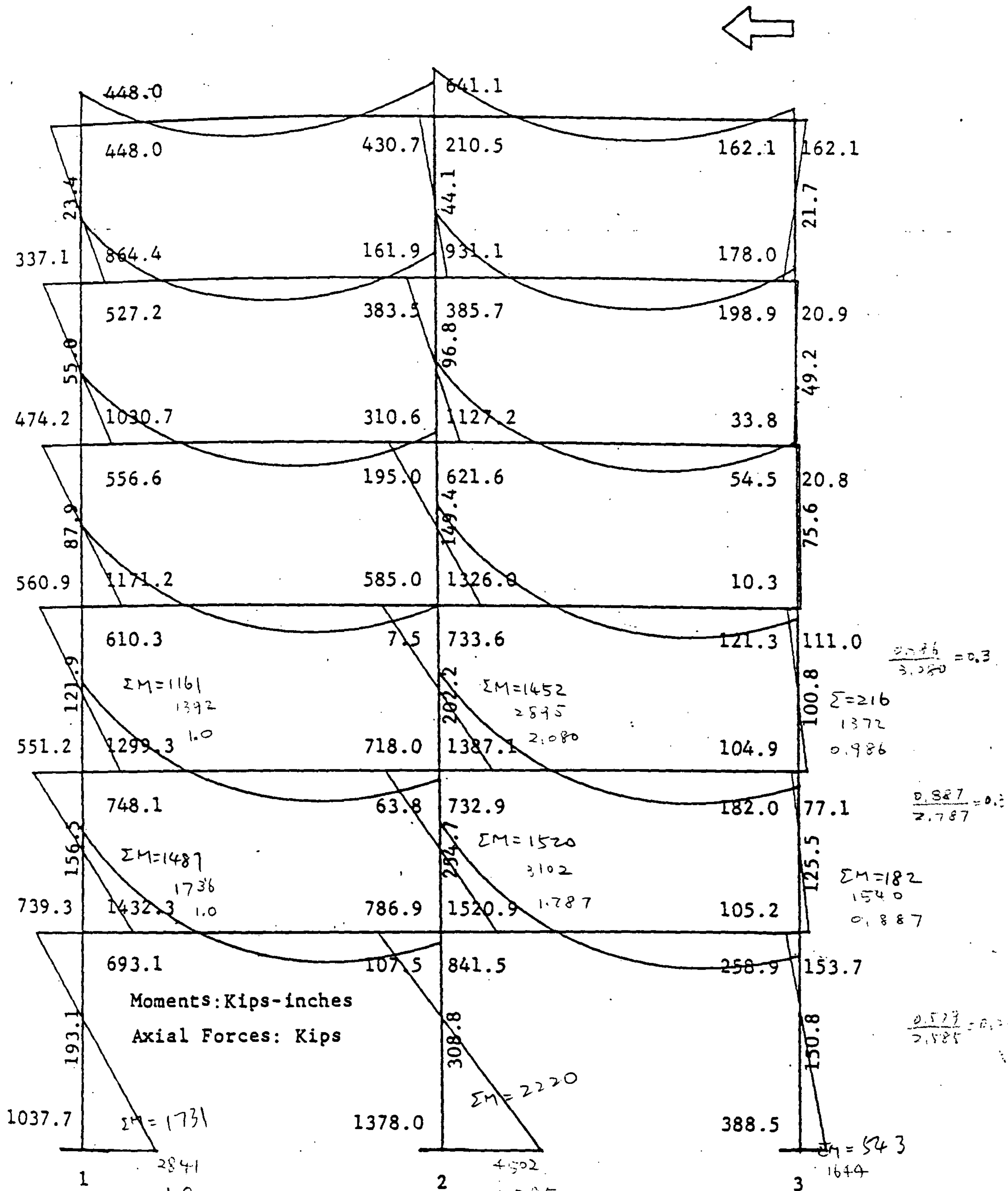


Fig. 10 Moment Diagram and Axial Forces of Moment Frame without Composite Action (Earthquake Forces from Right)

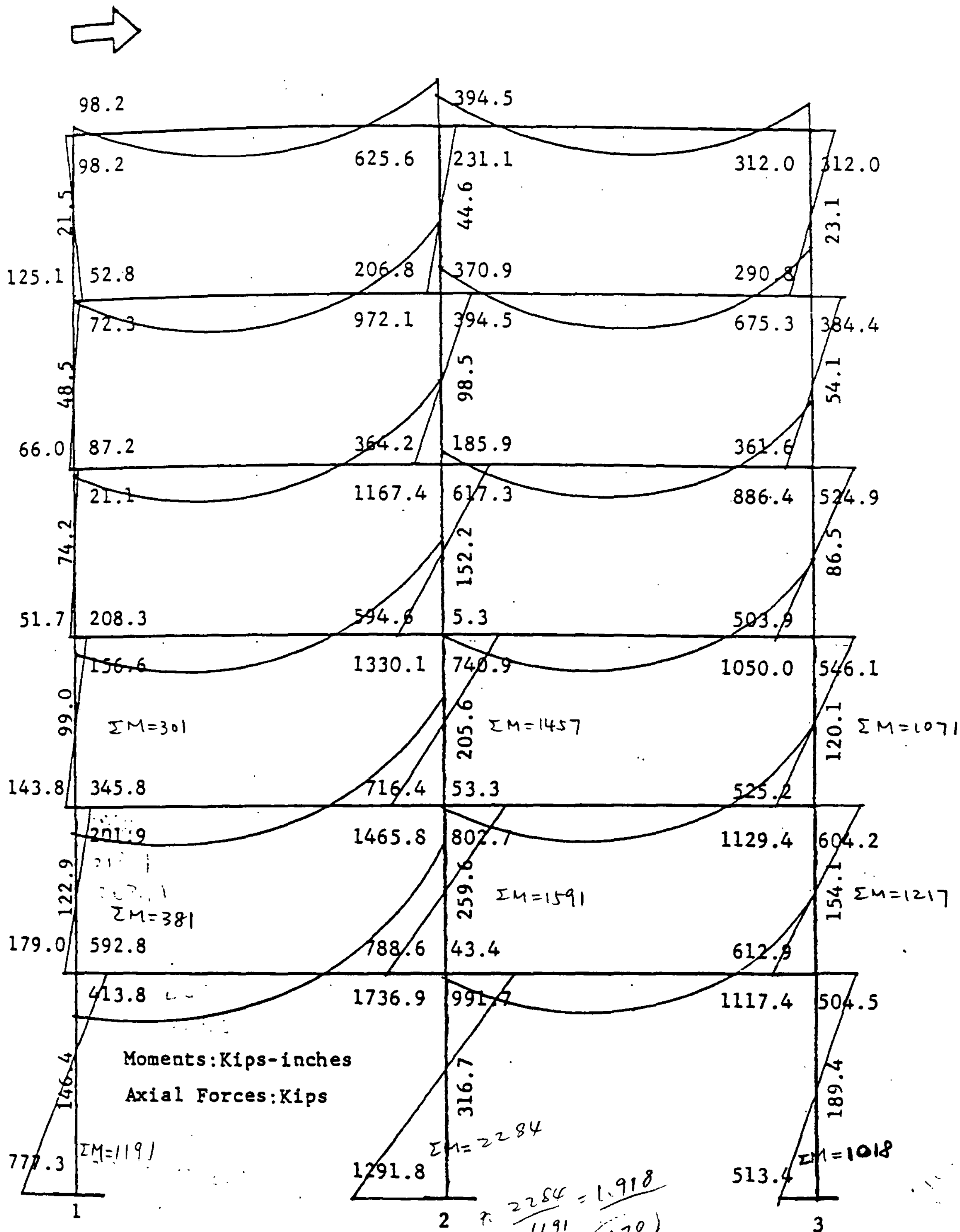


Fig. 11 Moment Diagram and Axial Forces of Moment Frame with Composite Action (Earthquake Forces from Left)

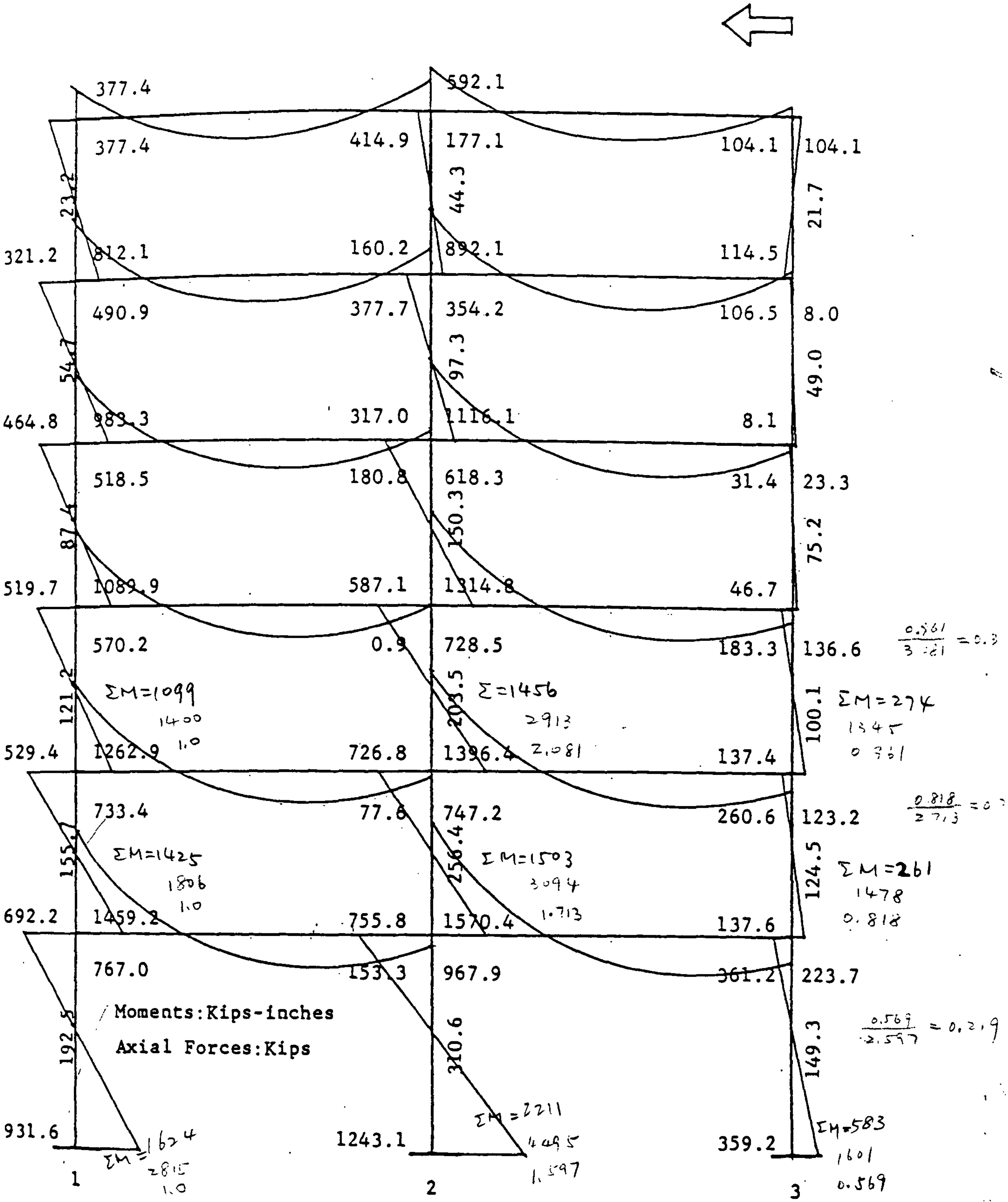


Fig. 12 Moment Diagram and Axial Forces of Moment Frame with Composite Action (Earthquake Forces from Right)

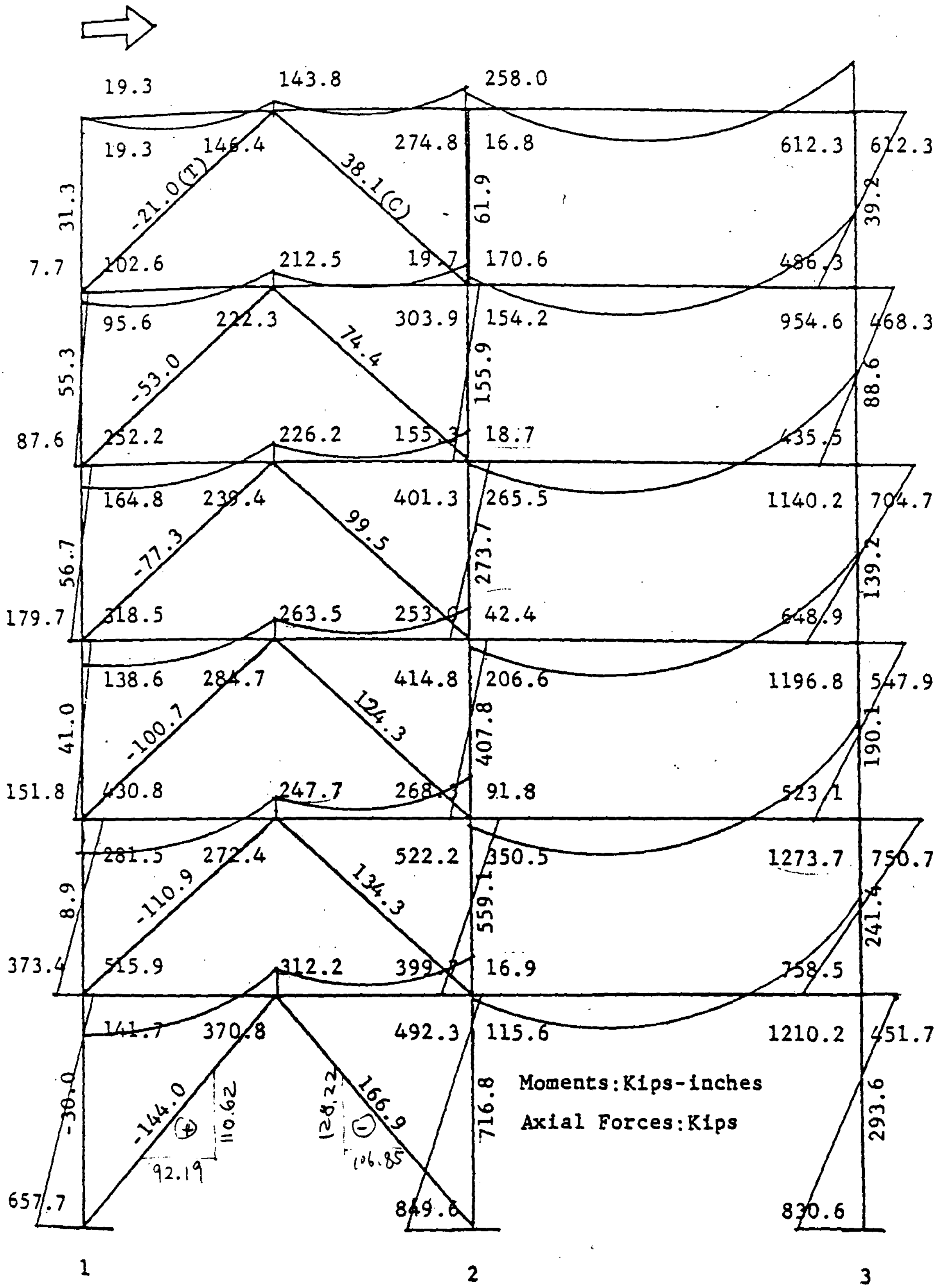


Fig. 13 Moment Diagram and Axial Forces of Concentrically Braced Frame without Composite Action (Earthquake Forces from Left)

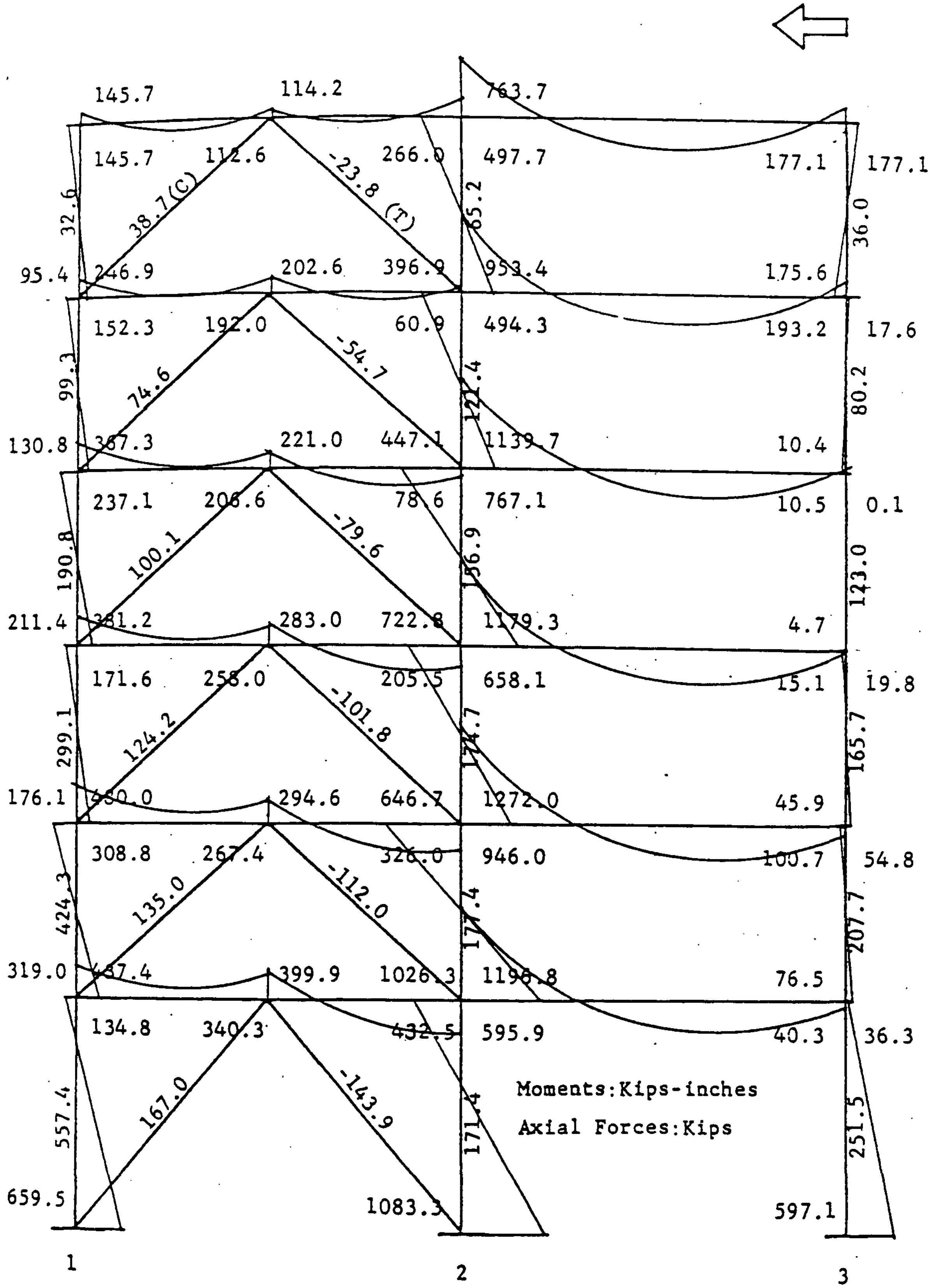


Fig. 14 Moment Diagram and Axial Forces of Concentrically Braced Frame without Composite Action (Earthquake Forces from Right)

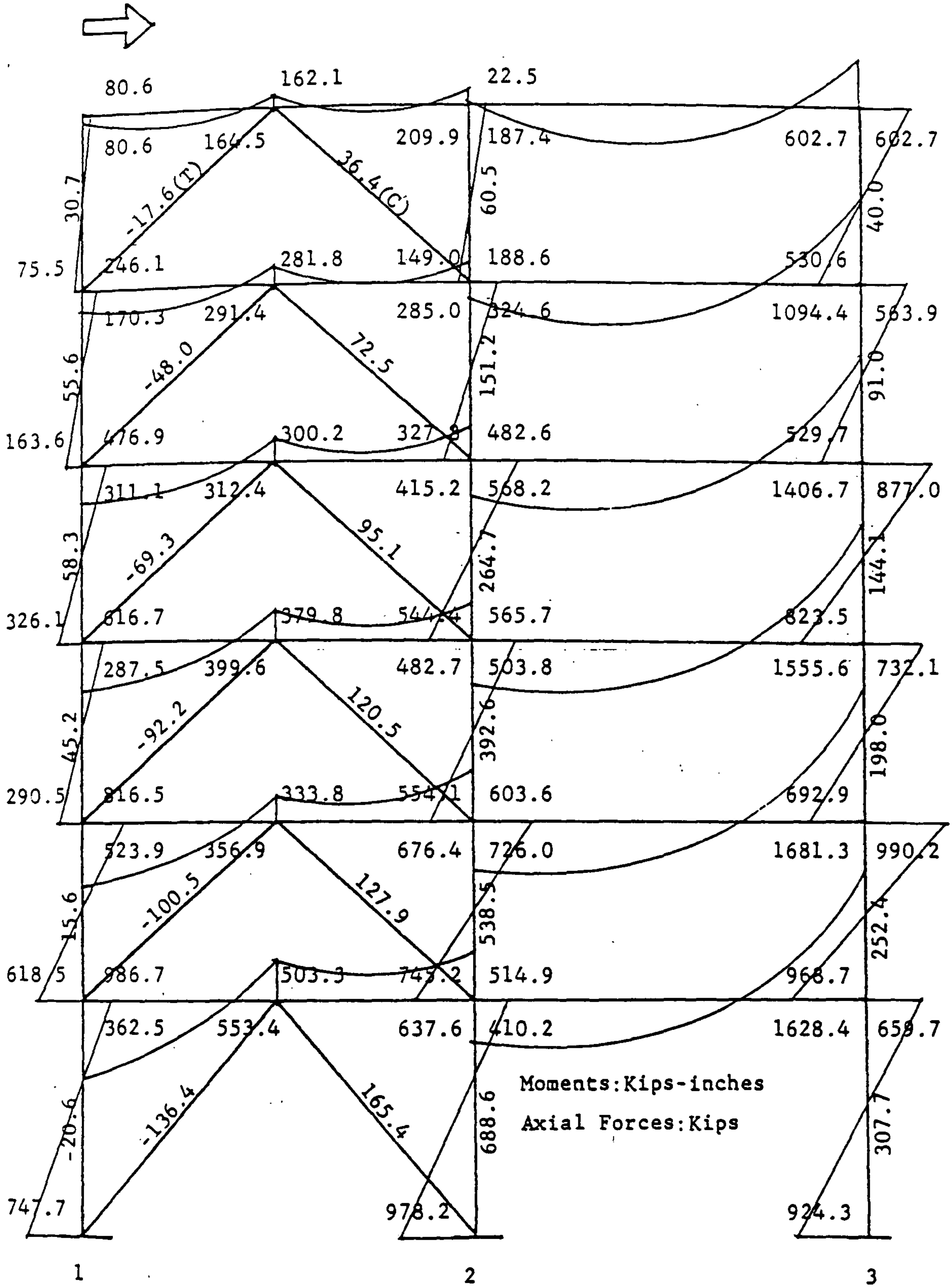


Fig. 15 Moment Diagram and Axial Forces of Concentrically Braced Frame with Composite Action (Earthquake Forces from Left)

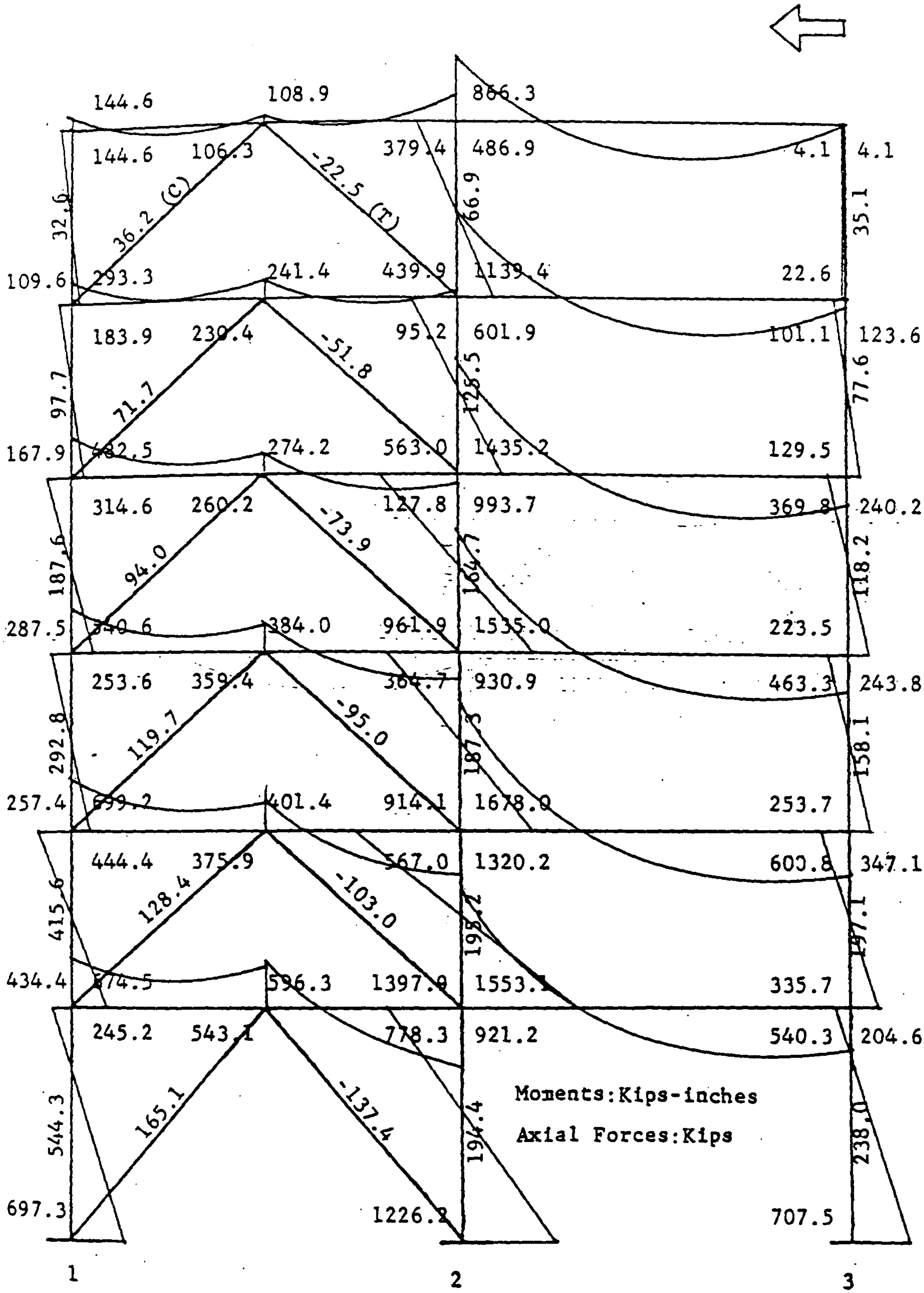
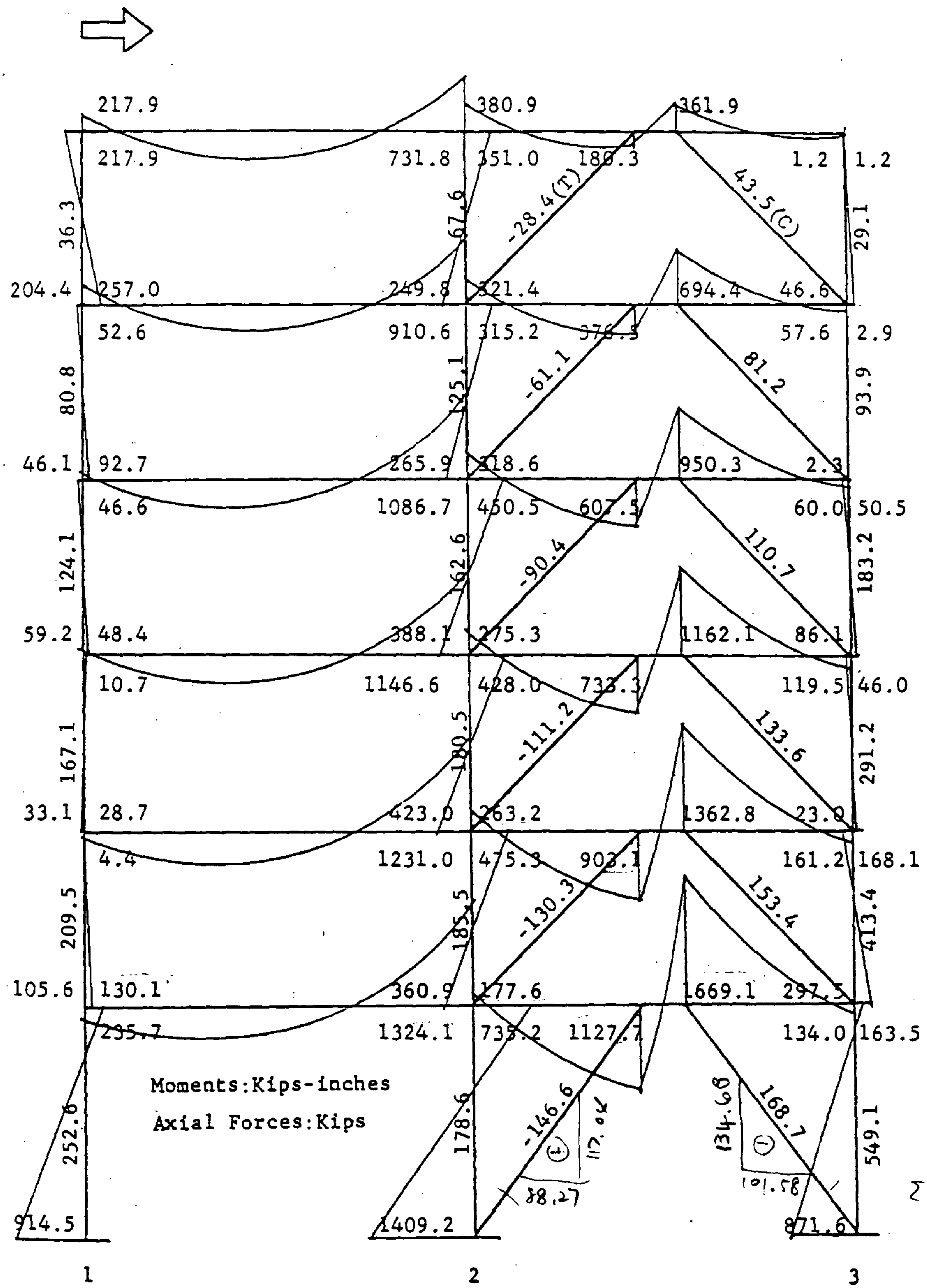


Fig. 16 Moment Diagram and Axial Forces of Concentrically Braced Frame with Composite Action (Earthquake Forces from Right)



Σ = 99?.?
≈ 998.

Fig. 17 Moment Diagram and Axial Forces of Eccentrically Braced Frame without Composite Action (Earthquake Forces from Left)

371
10.17
5.77
191.92
(88.6%)

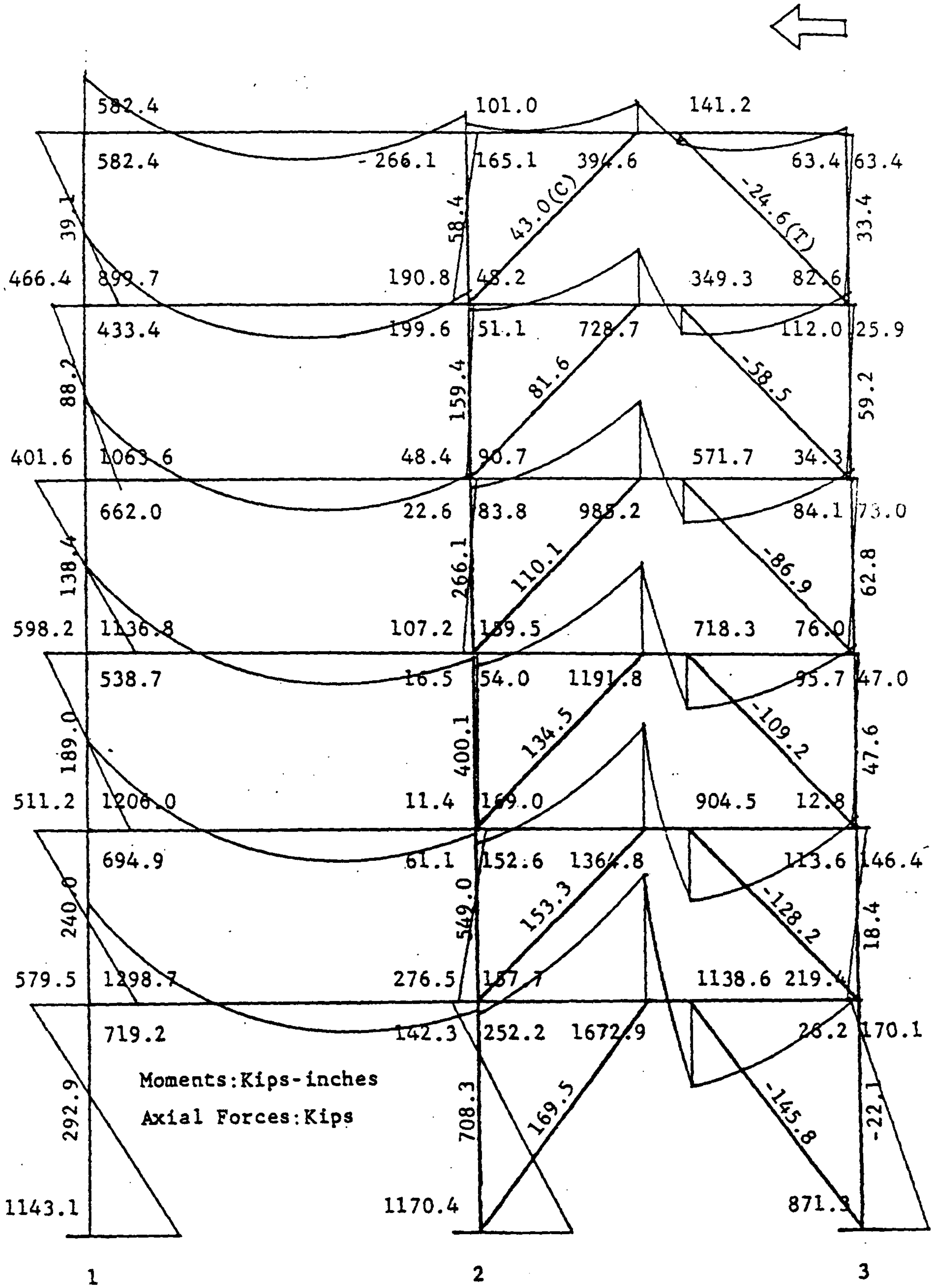


Fig. 18 Moment Diagram and Axial Forces of Eccentrically Braced Frame without Composite Action (Earthquake Forces from Right)

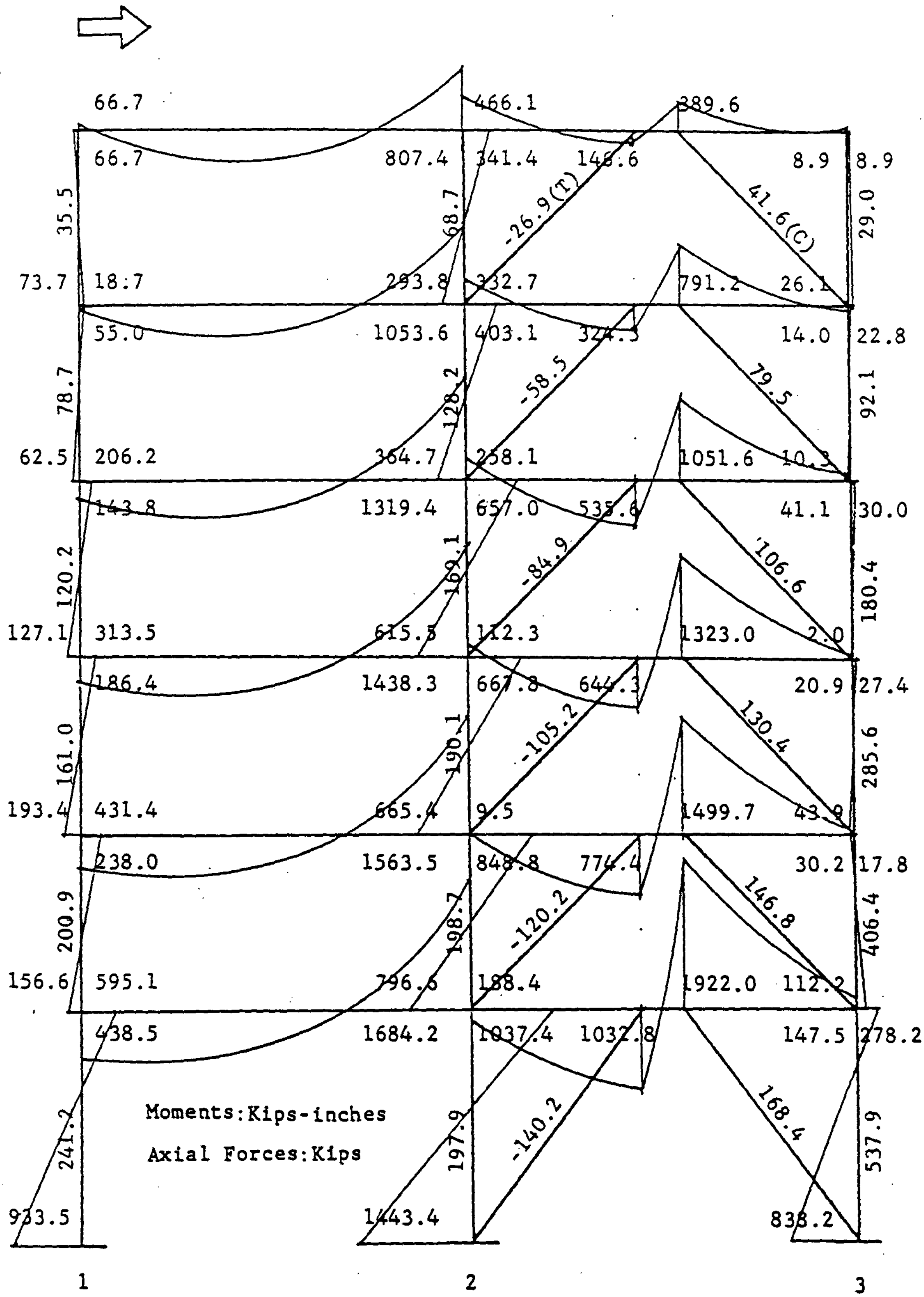


Fig. 19 Moment Diagram and Axial Forces of Eccentrically Braced Frame with Composite Action (Earthquake Forces from Left)

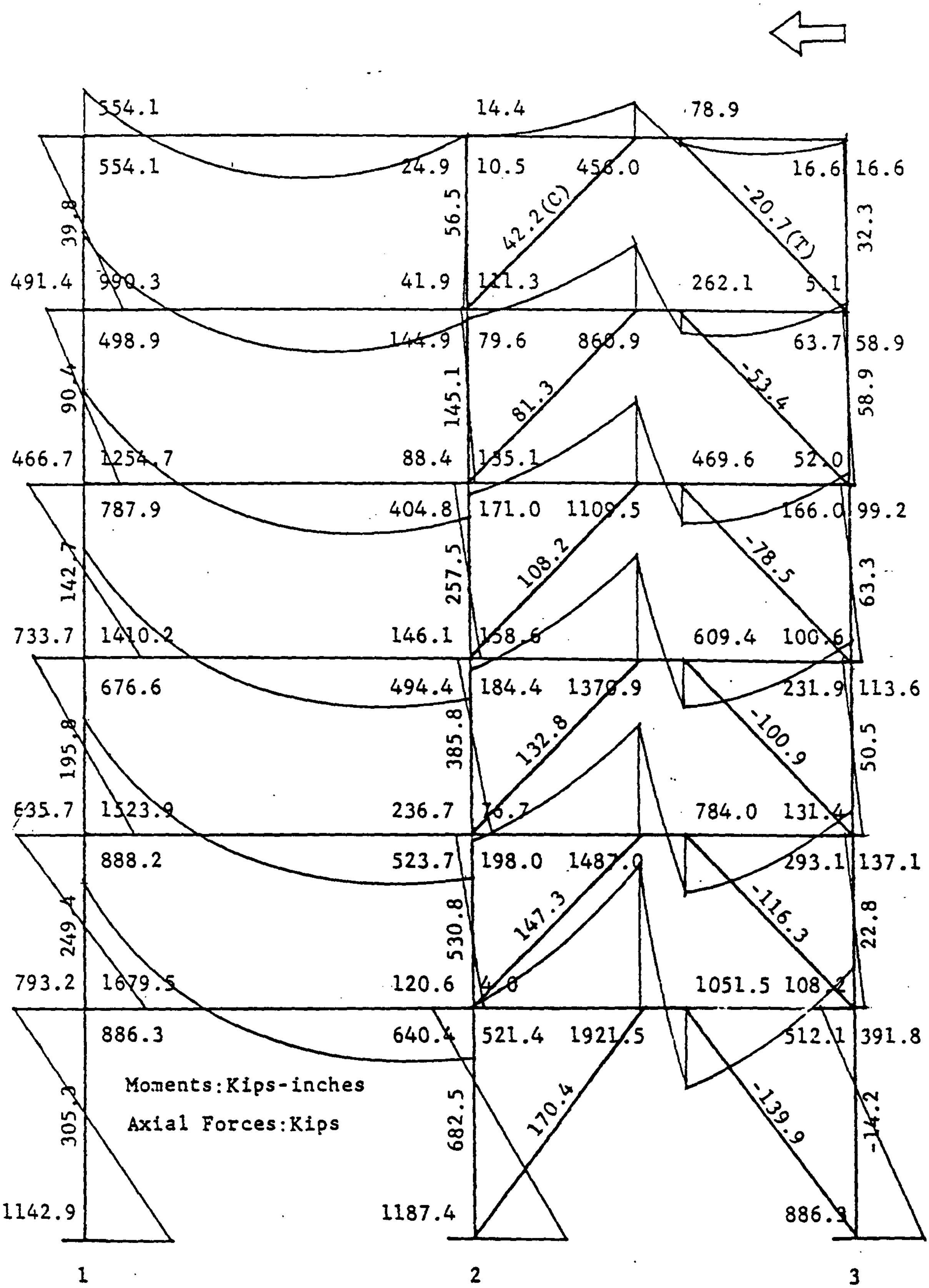


Fig. 20 Moment Diagram and Axial Forces of Eccentrically Braced Frame with Composite Action (Earthquake Forces from Right)

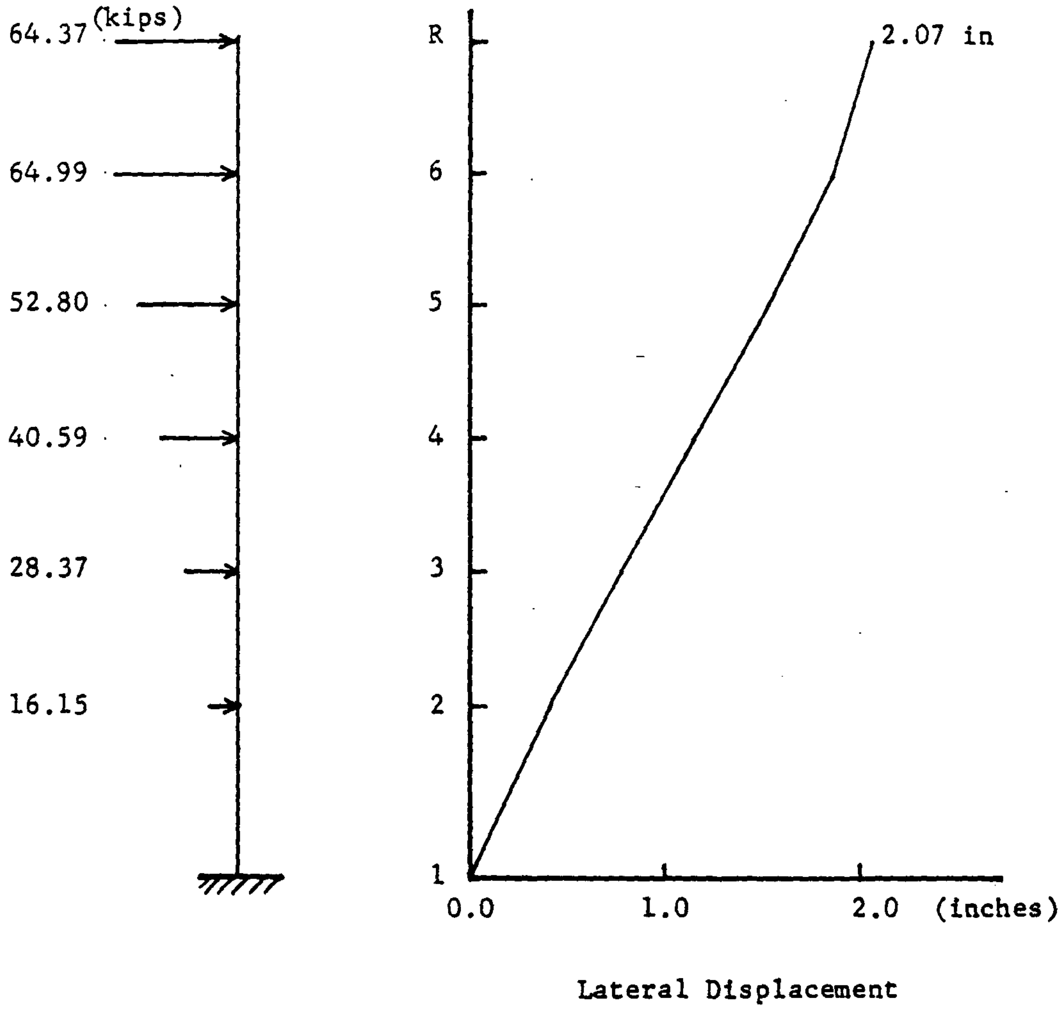


Fig. 21 Lateral Displacement of Test Building with concentric Bracing (Static Elastic Analysis)

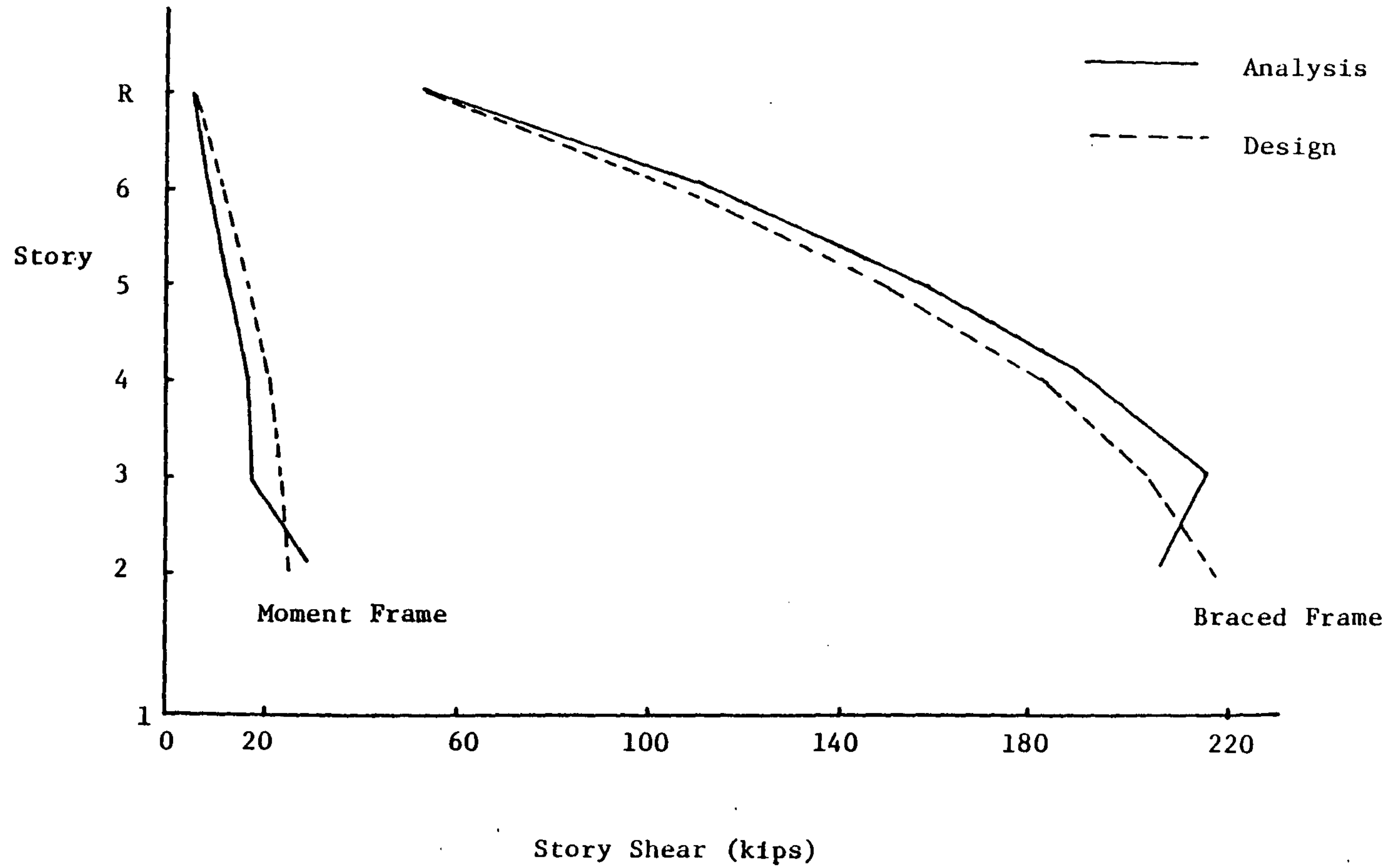
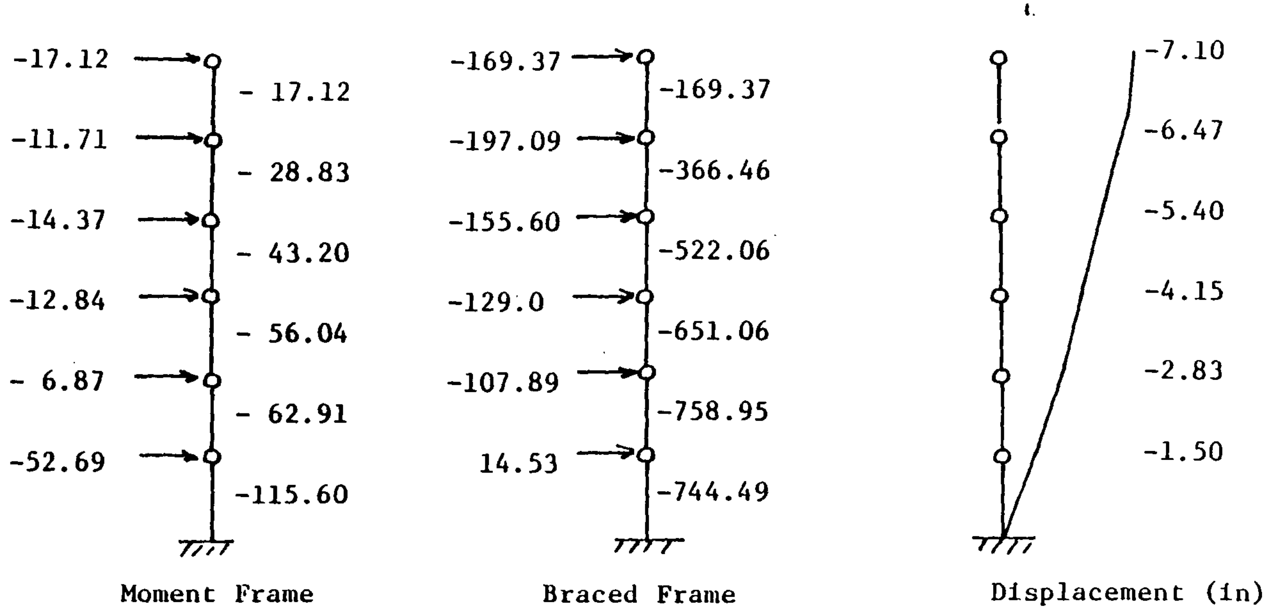


Fig. 22 Story Shears of Test Building with Concentric Bracing
(Static Elastic Analysis)

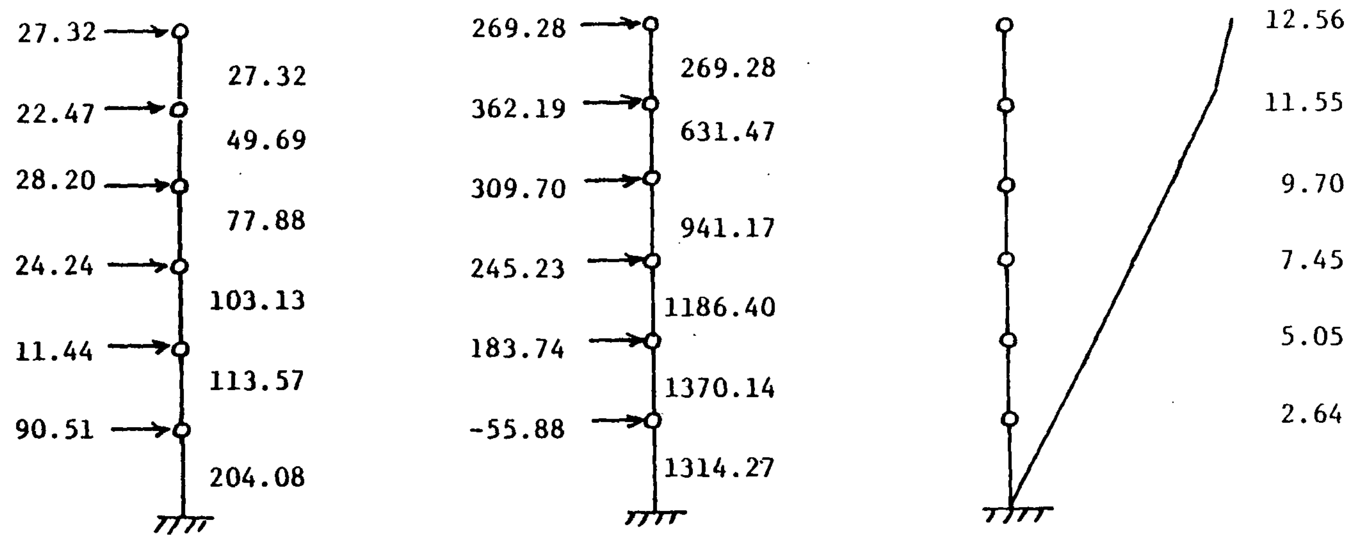
El Centro N.S (341.7 gals)



Base Shear:	Design	267.27 kips	(1)
	Analysis	975.61 kips	(3.652)

Fig. 23 Maximum Base Shears and Lateral Displacements of Test Building with Concentric Bracing (El Centro Earthquake)

Miyagiken-oki N.S (258.15 gals)



Base Shear:	Design	267.27	(1)
	Analysis	1722.46	(6.448)

Fig. 24 Maximum Base Shears and Lateral Displacements of Test Building with Concentric Bracing (Miyagiken-oki Earthquake)