Prepared jointly
by
University of California
and
University of Chile

FAILURE ANALYSIS OF A REINFORCED CONCRETE FRAME STRUCTURE

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UCLA • SCHOOL NATIONAL TECHNICAL D SCIENCE

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF-RA-E-74-498	2.	3. Recipient's Accession No. PB 293569
4. Title and Subtitle Failure Analysis of A Reinforced Concrete Frame Structure (UCLA-ENG-7482, EWE 74-05)			5. Report Date November 1974
			6.
7. Author(s) T. Guendelman, G.C. Hart, R. Husid			8. Performing Organization Rept. No. UCLA-ENG-7482
9. Performing Organization Name and Address University of California Mechanics and Structures Department School of Engineering and Applied Science Los Angeles, California 90024			10. Project/Task/Work Unit No. EWE 74-05
			11. Contract(C) or Grant(G) No. (C) (G)
12. Sponsoring Organization Name and Address Applied Science and Research Applications (ASRA) National Science Foundation 1800 G Street, N.W. Washington, D.C. 20550			13. Type of Report & Period Covered
			14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

This paper describes and explains the failure of a reinforced concrete frame building located in the town of Vina del Mar, approximately 50 kilometers southeast of the Chilean earthquake epicenter. The structure under consideration is of special interest because of the spectacular nature of its failure and also because of the ability of engineers to see its pre- and post-earthquake condition. This was possible because prior to the earthquake there existed two structurally identical reinforced concrete frame buildings at this site. The July 8, 1971 earthquake caused one of the buildings to completely collapse while the other, only a few meters away, remained standing. In addition to a description of the failure, a post-mortem analytical seismic analysis was performed on the structure: (1) A static deal load analysis of a typical interior frame was performed in order to estimate the frame's dead load bending moment diagram; (2) An eigenvalue analysis was performed on the same interior frame, assuming it to be linearly elastic, to determine its natural frequencies and normal mode shapes of vibration; and (3) A pseudo-dynamic failure analysis was carried out on the frame.

17. Document Analysis a. Descriptors

Failure
Collapse
Earthquakes

Seismology
Earthquake resistant structures
Dynamic structural analysis

Design
Buildings
Reinforced concrete

b. Identifiers/Open-Ended Terms
Vina del Mar
Santiago, Chile
Failure analysis

c. COSATI Field/Group

18. Availability Statement NTIS	19. Security Class (This Report)	21. No. of Pages 25
	20. Security Class (This Page)	22. Price A02

(See ANSI-Z39.18)   See Instructions on Reverse   OPTIONAL FORM 272 (4-77)
(Formerly NTIS-35)
Department of Commerce

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ABSTRACT

On 8 July 1971 at 23:09 hours local Santiago, Chile time an earthquake

of Richter magnitude 7.5 took place in central Chile. The epicenter of this

earthquake was located at latitude 32° 45' south and longitude 71° 58' west

and at a depth of approximately 60 kilometers.

This paper describes and explains the failure of a reinforced concrete

frame building located in the town of Viña del Mar, approximately 50 kilo-

meters southeast of the earthquake epicenter. While no recorded ground

motion exists at or near the building site, the estimated peak ground motion

in Viña del Mar was 0.15-0.20g and the modified Mercalli Intensity was VIII.

The structure under consideration is of special interest because of the spec-

tacular nature of its failure and also because of the ability of engineers to see,

in effect, its pre-and post-earthquake condition. This was possible because

prior to the earthquake there existed two structurally identical reinforced

concrete frame buildings at this site. The 8 July 1971 earthquake caused

one of the buildings to completely collapse while the other, only a few meters

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away, remained standing. Photographs of the two buildings are shown on the attached pages.

Following an inspection of the structural failure by the authors, a detailed analytical failure analysis was performed. A finite element model was created for a typical interior frame. The frame's span is 17.1 meters and the above grade height is 11.5 meters. Using this model a static dead load analysis was performed and the moments in the frame were calculated. It was found that these dead load moments were always less than one-half the cross-section's plastic moment capacity.

To study the frame's general dynamical character its first five natural frequencies and mode shapes of vibration were calculated. The fundamental structural mode shape was antisymmetric about the frame's center line and its undamped natural period of vibration was 0.52 second. The second mode shape was symmetrical about the frame's center line and its period was 0.25 second.

The frame's failure mode was studied by performing a pseudo-dynamic plastic response analysis. Using the frame's first mode shape we distributed the total lateral seismic load in proportion to the first antisymmetric modal displacements, the total lateral load being equal to a constant times the frame's tributary dead weight. This constant was increased until the first plastic hinge formed at 0.24 and then until collapse at 0.25. Similarly the total seismic vertical load was distributed in proportion to the first symmetric modal displacements. The first plastic hinge and structural collapse occurred at 0.15 and 0.52 times the structure's dead load, respectively.

An interpretation of this plastic seismic analysis in light of the observed structural failure is given in the text of the paper. Recommendations are made with respect to future design.

INTRODUCTION

On July 8, 1971 at 23:09 hours local Santiago, Chile time an earthquake of Richter magnitude 7.5 was detected at the University of California, Berkeley seismological station. The epicenter of this earthquake was located at latitude 32° 45' South and longitude 71° 58' West and at a depth of approximately 60 kilometers. The location of the epicenter is shown in Figure 1 and significant ground shaking was felt throughout central Chile.

A careful study of the damage experienced as a result of this South American earthquake is being made by the faculty of the University of Chile.[1]* The reader is referred to that report for a city-by-city description of damage. The authors have conducted a detailed study of the dramatic structural failure of a reinforced concrete frame building located in the city of Viña del Mar.

This paper describes the failure as well as a post-mortem analytical seismic analysis which was performed on the structure. First, a static dead load analysis of a typical interior frame was performed in order to estimate the frame's dead load bending moment diagram. Next, an eigenvalue analysis was performed on the same interior frame, assuming it to be linearly elastic, in order to determine its natural frequencies and normal mode shapes of vibration. Finally, a pseudo-dynamic failure analysis was carried out on the frame. In this latter analysis it was assumed that the seismic loading acted in a static manner and in proportion to the structure's generalized modal

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*Numbers in paranthesis refer to References Listed at End of Paper.

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displacement. This static loading was incrementally increased until structural collapse. Such a pseudodynamic analysis was necessary because no strong motion record existed at or near the structure's site.

This paper was encouraged and partially financed by the University of California-University of Chile Convenio Program. In August, 1971 as part of this program, Professors Gary C. Hart and C. Martin Duke of the University of California at Los Angeles, USA visited Chile and observed the damage experienced as a result of the July 8, 1971 earthquake.

DESCRIPTION OF THE STRUCTURE

The building under study is located in the city of Viña del Mar, see Figure 1, and approximately 50 kilometers from the epicenter of the July 8, 1971 earthquake. The city of Viña del Mar experienced significant ground shaking (peak acceleration estimated at approximately 20 percent gravity); and its Modified Mercalli Intensity was approximately VIII. Modern, reinforced concrete, high-rise buildings in the city experienced no major structural failure; however, most interior partitions had diagonal shear cracks. Also, significant damage was usually observed at the connection between two or more structural elements due to the differential movement. Major damage to exterior walls in the residential area and other smaller and older buildings was also observed.

The building discussed herein was constructed in 1946 and is one of two structurally identical reinforced concrete frame buildings. The structures are located within 20 meters of each other. One building experienced only minor visible damage during the earthquake; whereas the other building

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suffered complete structural collapse. Figure 2 shows plan and elevation views of these buildings and Figure 3 shows photographs of the building which remained standing after the earthquake.

Certain particularities are evident in the building design. First, the concrete vaporization vents located at the apex of the rigid frame provide a large dead load and only nominal reinforcing. Second, the rigid frame reinforcement is continuous throughout the frame's beam-column region; and tension spalling of the concrete on the interior of this interface seems highly probable. Third, all reinforcing bars in this structure are underformed.

The 8 July earthquake caused one of the above noted buildings to completely collapse. Figures 4 and 5 show the post-earthquake condition of the building.

ANALYTICAL MODEL OF STRUCTURE

The reinforced concrete frame shown in Figure 2 was analytically modeled by the authors as a two-dimensional planar structure. We utilized the finite element method of analysis to discretize a typical interior frame into a finite number of beam elements, each beam element being allowed to have constant structural properties. It was assumed that the internal deformation pattern of each finite beam element is completely defined in terms of the two perpendicular translational and one rotational degree of freedom at its ends.

Figure 6 shows an elevation of the structural frame and the 22 beam finite elements comprising the analytical model. All finite beam elements except numbers 11 and 12 were assigned equal structural properties, see Table 1.

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In all the analyses it was assumed that the base of the columns was rigid and that soil-structure interaction was not significant.

STATIC DEAD LOAD ANALYSIS

As a first step a static dead load analysis was performed on a typical interior frame. The tributary dead load acting on this frame was estimated to be 3.2 tons per horizontal meter. Figure 7 shows the frame's dead load bending moment diagram. Also shown in this figure is the plastic moment capacity of each structural element.[2] It is immediately apparent that the dead load structural moments never approach the structure's plastic moment capacity, and in fact, the dead load factor of structural safety was never less than 2.

MODAL CHARACTERISTICS OF THE STRUCTURE

The finite element model was then utilized to calculate the frame's natural periods and mode shapes of vibration. A stiffness matrix condensation procedure was used to reduce the order of the system analyzed from 63 static to 26 dynamic degrees of freedom. In so doing it was assumed that the inertial effect of the loading associated with the axial deformation and joint rotation was negligible.

Table 2 lists the frame's first six natural periods of vibration and Figure 8 shows its first antisymmetrical and symmetric mode shapes.

PLASTIC FAILURE ANALYSIS

A step-by-step dynamic inelastic analysis of this structure was not felt to be warranted because there existed no time history of seismic ground

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motion at or near the structure's site. Instead the authors decided to conduct a pseudo plastic dynamic analysis utilizing the structure's modal properties.

In the analysis performed, the static dead loads were first applied to the structure and the corresponding bending moments in all finite elements were calculated (Figure 7). Then, in addition to these dead loads a static loading equal in magnitude to a constant, α, times the frame's total tributary dead weight was added to the frame. In the first case studied, called Case A, the total dynamic load was distributed over the structure in proportion to the first antisymmetric modal displacements. The greatest modal displacement being set equal to unity. This case reflected a pseudo dynamic loading that was entirely lateral in direction and the constant α can be interpreted as the seismic lateral load coefficient. The second case studied, Case B, distributed the total pseudo dynamic load in proportion to the first symmetrical mode shape. This case resulted in a dynamic loading in the vertical direction and α can be interpreted as the seismic vertical load coefficient.

The analysis procedures followed in both cases were identical. First, the load coefficient, α, was increased in magnitude until a single plastic hinge was observed. Then the coefficient was increased even further in magnitude until a structural mechanism was formed and the frame collapsed.

Subjected to Case A type loading, the structure formed plastic hinges simultaneously at nodes 10 and 14 when $\alpha = 0.24$. The structure became a mechanism and hence collapsed at the slightly greater loading condition $\alpha = 0.25$. The mechanism of collapse was formed by the development of hinges at nodes 2, 10, 14, and 22. It is important that in this case where lateral

seismic loading was applied to the structure the increment of load necessary to go from first hinge to collapse was only one percent of the structure's weight.

The plastic response of the structure under vertical loading was studied in Case B. Here it was found that hinges were first formed at nodes 11 and 13 when $\alpha = 0.15$. Note that this seismic coefficient is significantly less than that of Case A. However, in Case B it was found that not until $\alpha = 0.52$ did a structural mechanism exist by virtue of hinges forming at nodes 2, 6, 11, 13, 18, and 22. A sketch of this failure mechanism under both of these loading cases is shown in Figure 10.

SUMMARY AND CONCLUSIONS

The preformance of the formulated analytical structural model and the actual building was quite similiar. Structural collapse was predictable from the structural model and hinge locations as postulated from the plastic analysis corresponded to observed failure mechanism.

It appears that the structural failure was a combination of vertical seismic forces which caused the formation of the first structural hinge and horizontal seismic forces which actually caused structural collapse.

This study recommends that seismic codes consider an extension of there static load concepts to include a seismic analysis of the type preformed here in. Such an analysis includes both vertical and lateral seismic forces and their influence upon structural behavior.

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REFERENCES

1. Universidad de Chile, Facultad de Ciencias Físicas y Matemáticas.
   "Informe Preliminar Sobre el Sismo del 8 de Julio de 1971" (1971).

2. ACI Special Publication No. 17 - Volume I.
   "Ultimate Strength Design Handbook".

TABLE 1

PROPERITIES OF STRUCTURAL ELEMENTS

Member Number	Bending Modulus of Elasticity	Shear Modulus of Elasticity	Moment of Inertia	Cross Sectional Area	Shape Factor
1-10, 13-22	2.1×10^6	0.84×10^6	1.82×10^{-2}	.27	1.2
11-12	2.1×10^6	0.84×10^6	14.00×10^{-2}	.69	1.2

Units: Tons - Meters

TABLE 2

NATURAL PERIODS OF VIBRATION

Mode Number	Period (sec)	Mode Type
1	0.52	1st Anti-Symmetric
2	0.25	1st Symmetric
3	0.12	2nd Antisymmetric
4	0.08	2nd Symmetric
5	0.04	3rd Antisymmetric
6	0.03	3rd Symmetric

First and second mode shapes are shown in figures 8 and 9.
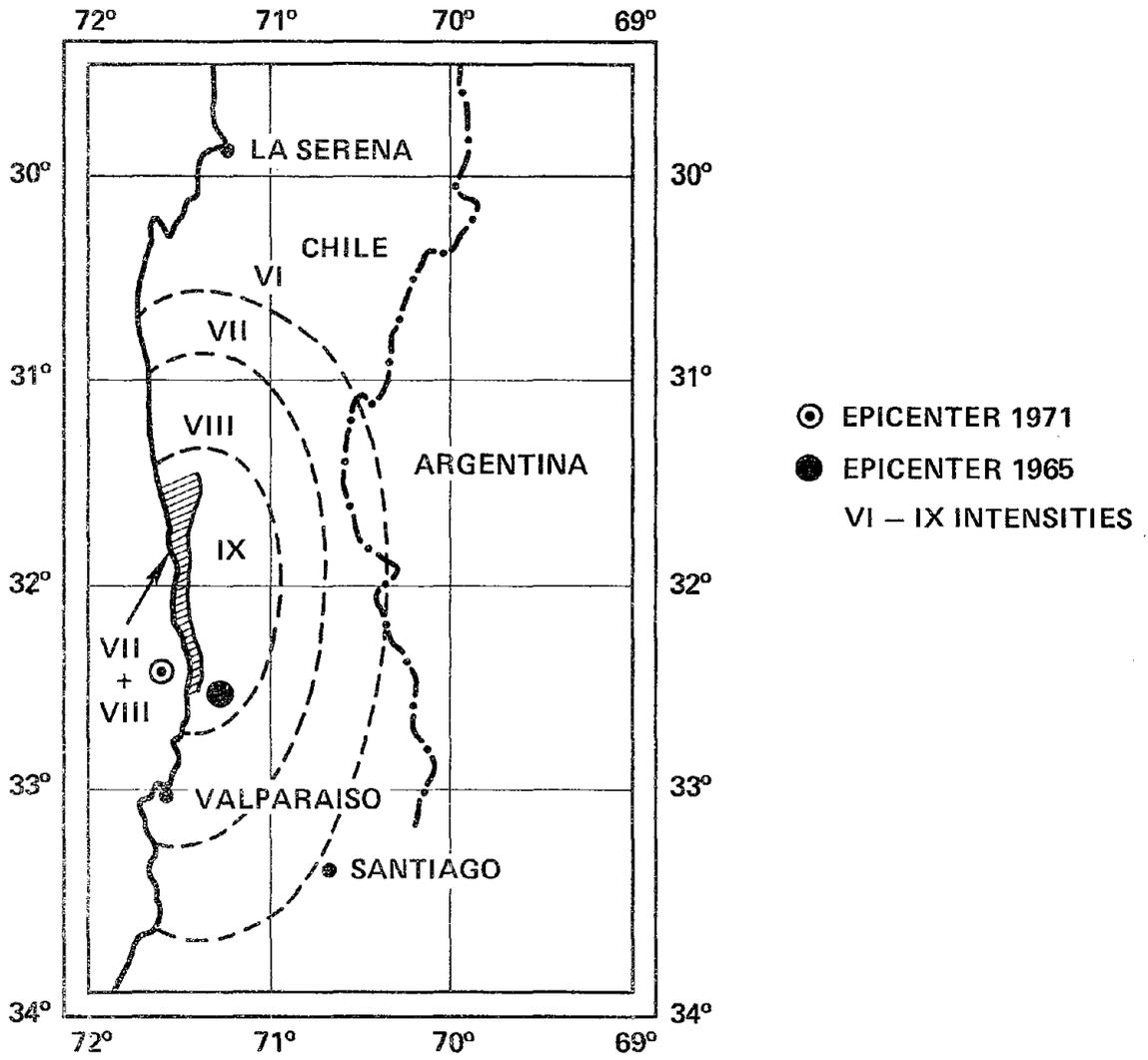
Figure 1.   Location Map.

Figure 2a. Plan View of Building.

Figure 2b.   Elevation of Building.

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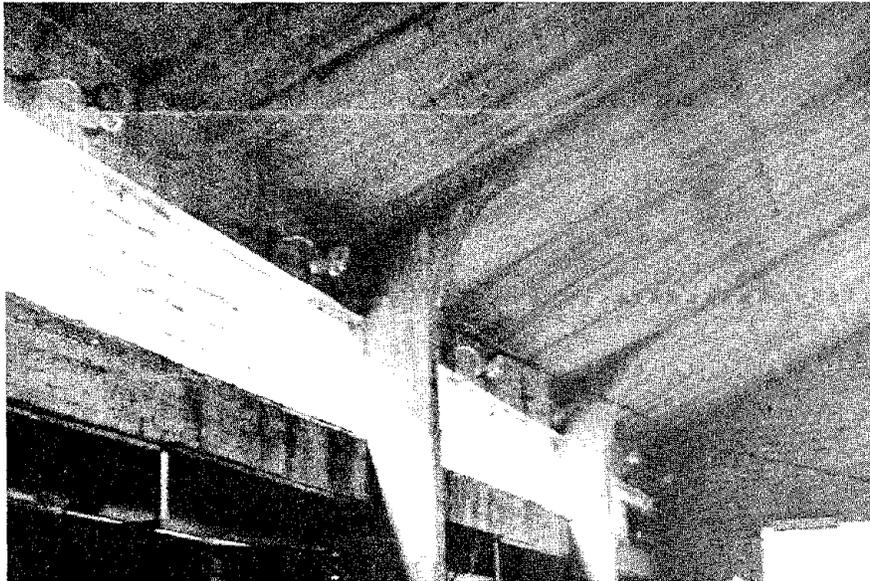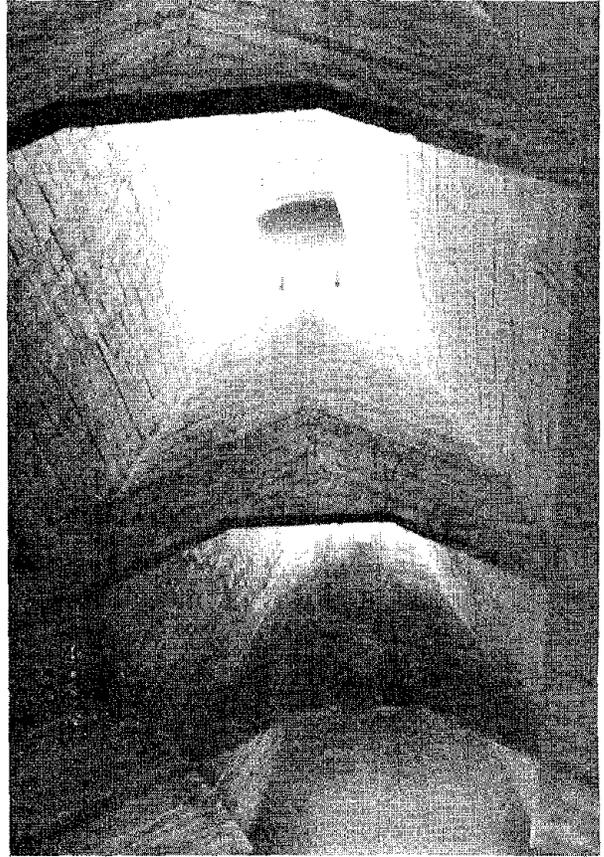
Figure 3.   Photographs of Building: Pre-Earthquake

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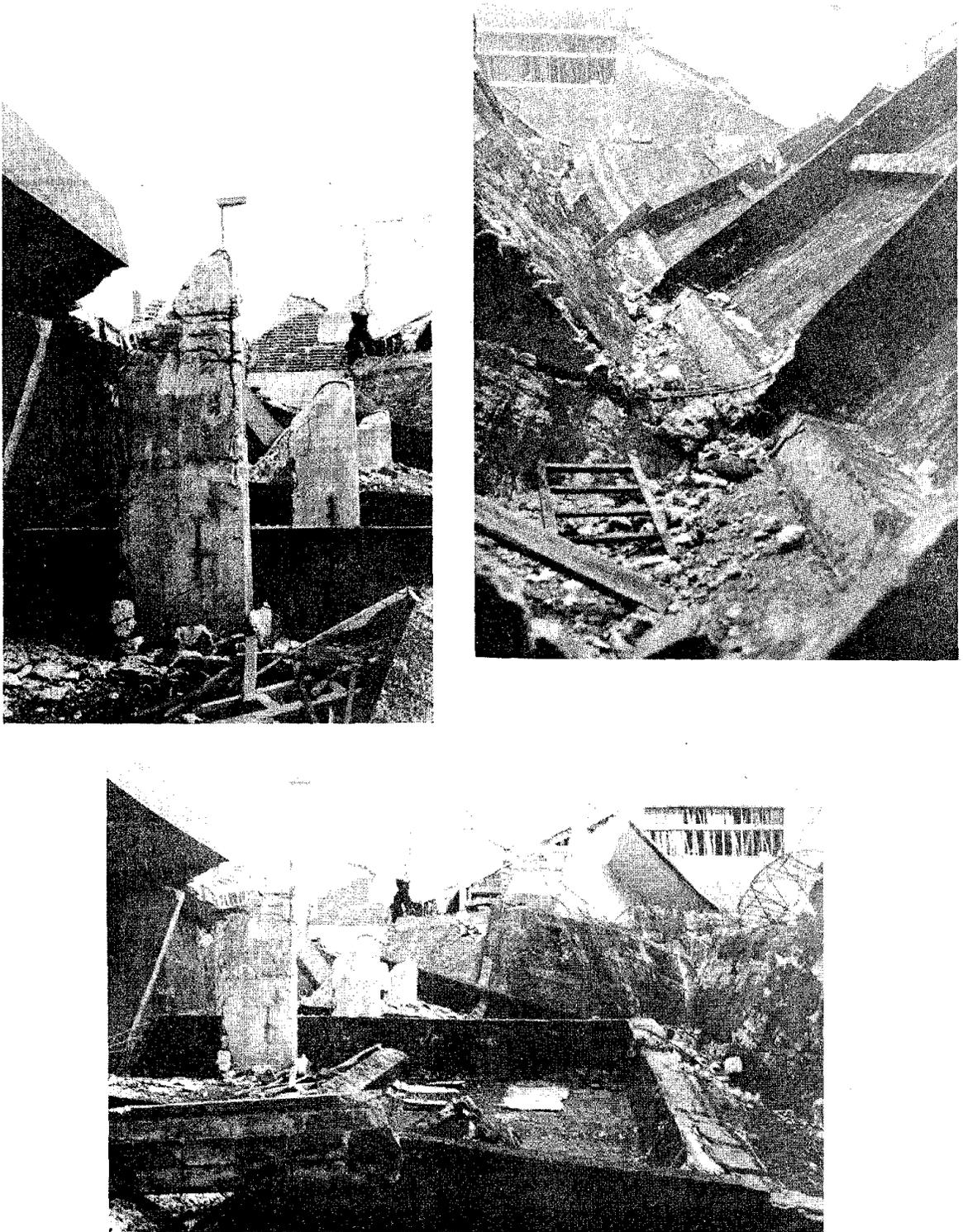
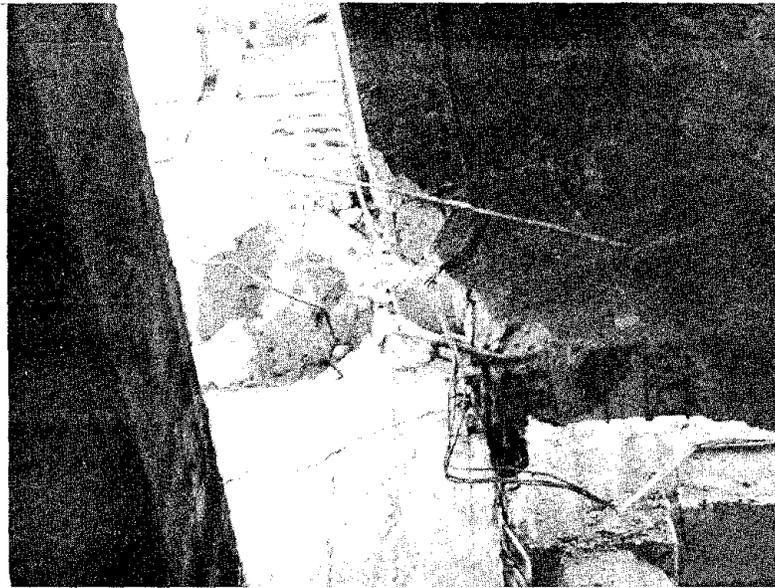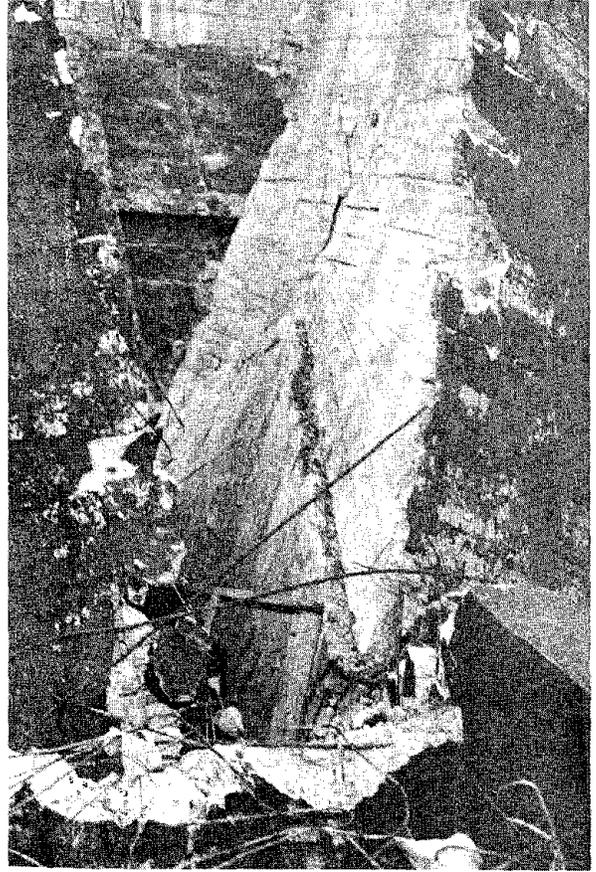Figure 4. Photographs of Building: Post-Earthquake

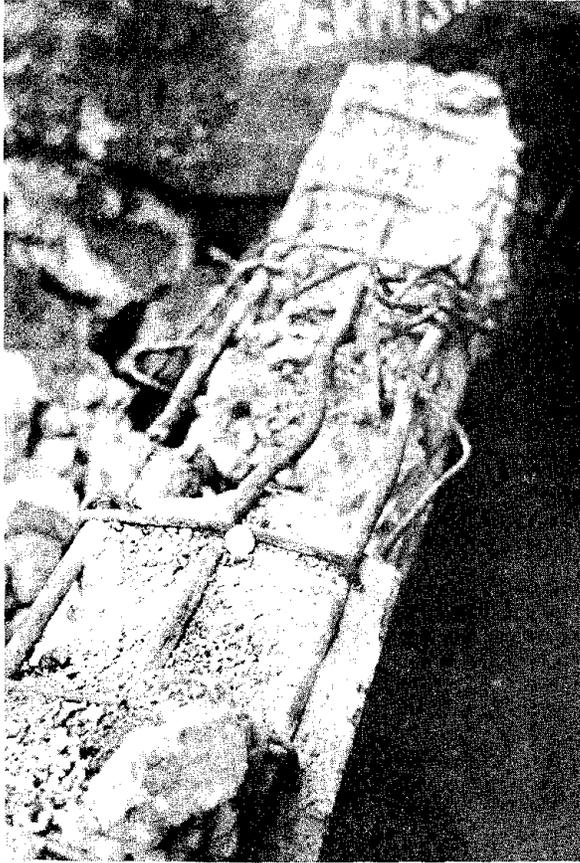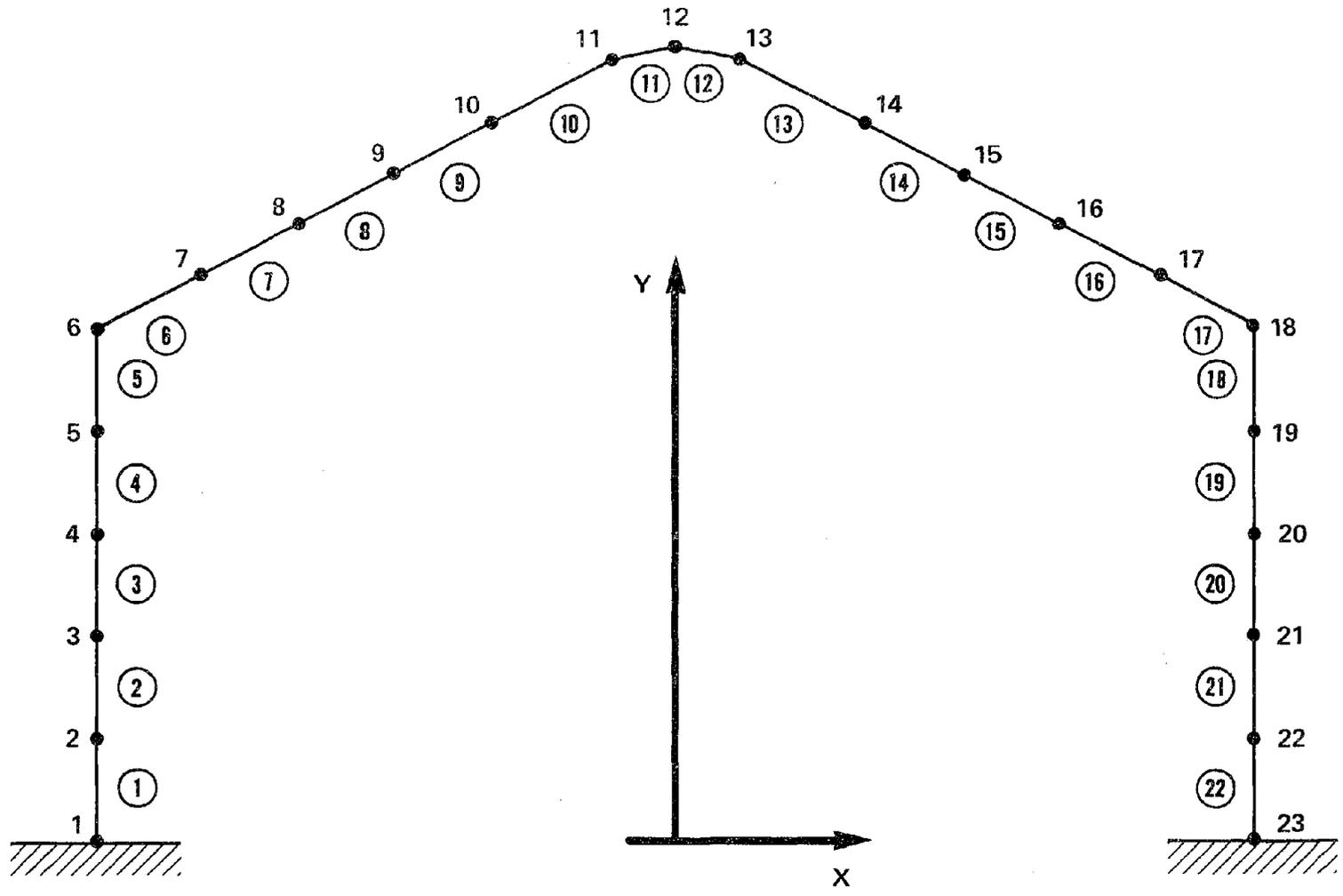Figure 5.   Photographs of Building: Post-Earthquake

Figure 6. Structure Definition. Joint, Members and Coordinates.

Figure 7. Bending Moment Diagram for Dead Load.
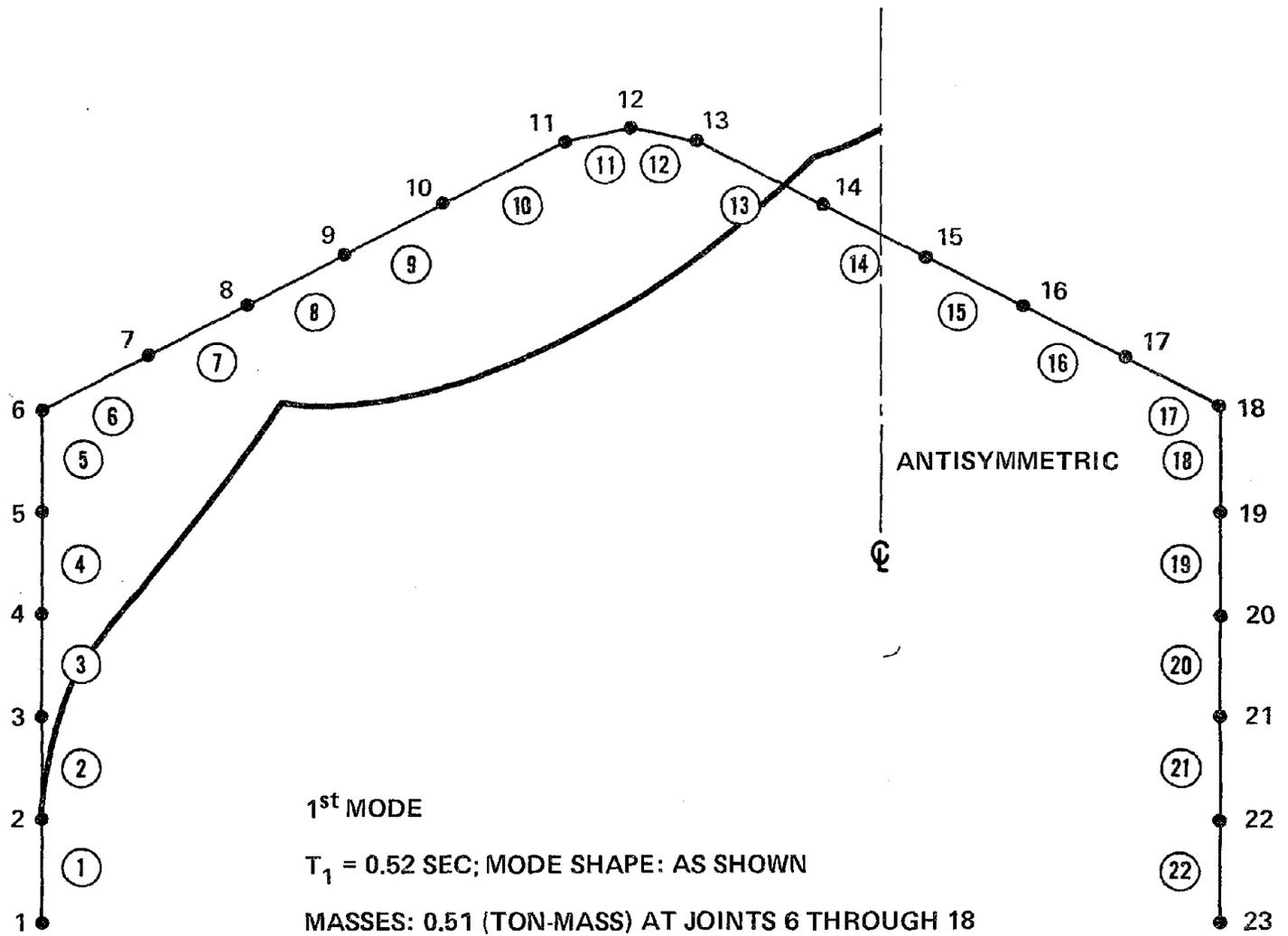
ANTISYMMETRIC

1st MODE

$T_1 = 0.52$ SEC; MODE SHAPE: AS SHOWN

MASSES: 0.51 (TON-MASS) AT JOINTS 6 THROUGH 18

Figure 8.   First Mode Shape.

2nd MODE

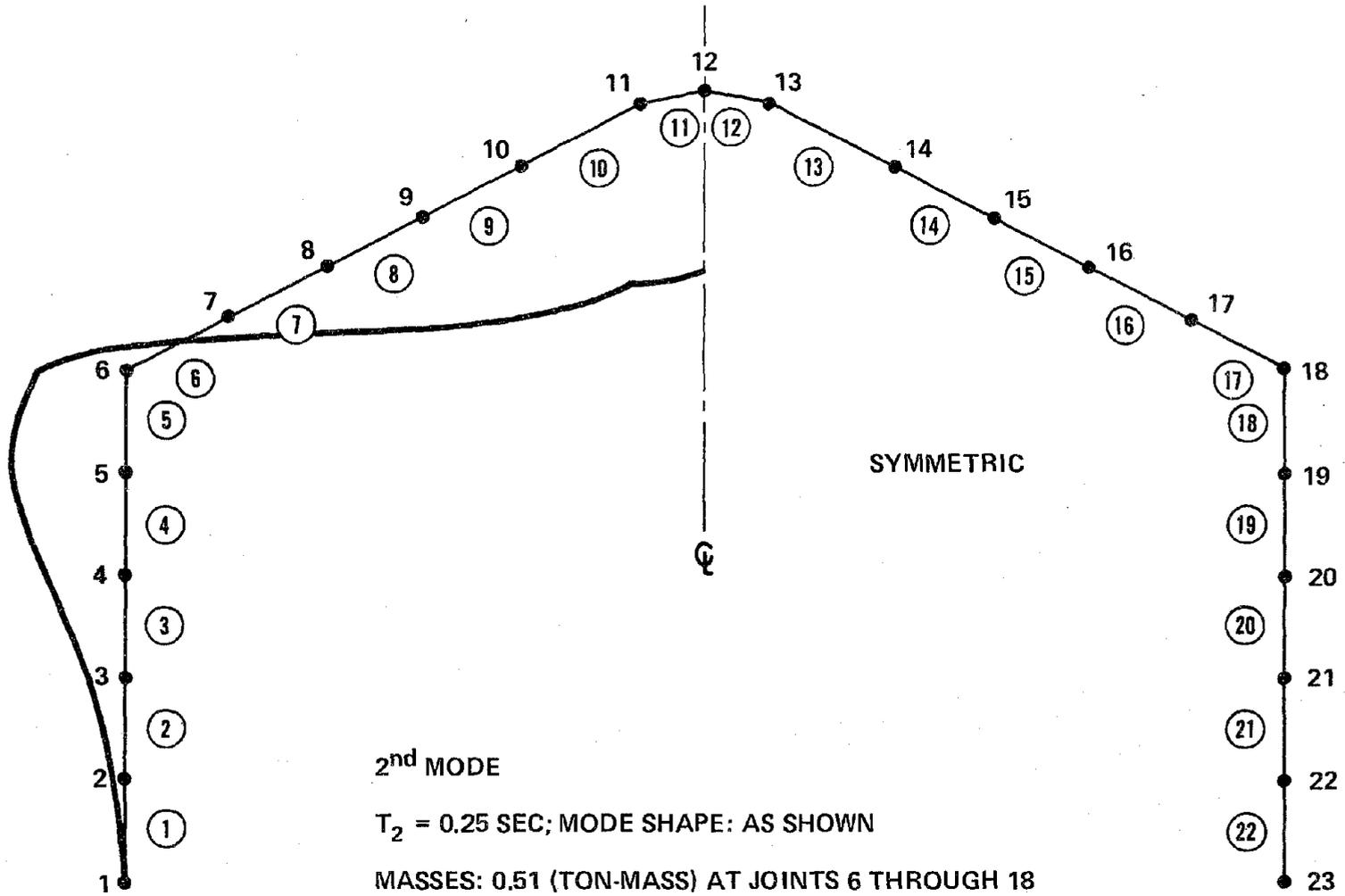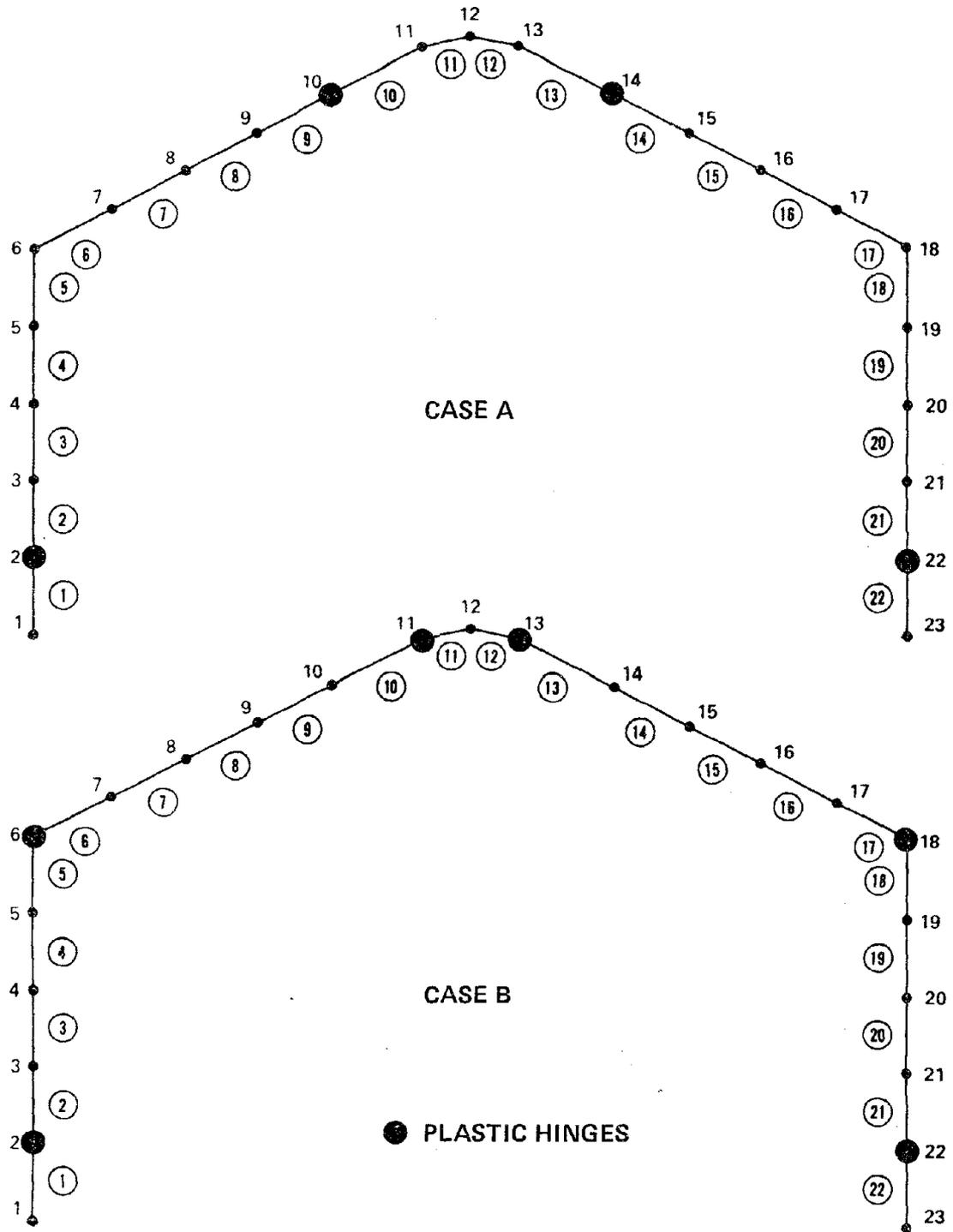$T_2 = 0.25$ SEC; MODE SHAPE: AS SHOWN

MASSES: 0.51 (TON-MASS) AT JOINTS 6 THROUGH 18

Figure 9. Second Mode Shape.

Figure 10. Failure Mechanisms.