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SHAKING TABLE STUDY OF A 1/5 SCALE STEEL FRAME COMPOSED OF TAPERED MEMBERS

by

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PREFACE

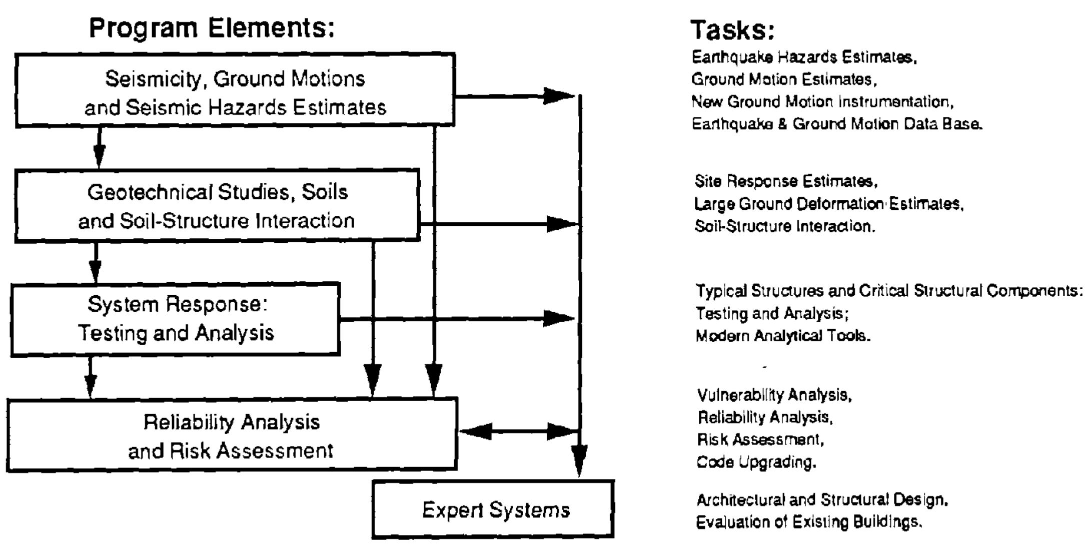
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to system response investigations.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. The work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



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System response investigations constitute one of the important areas of research in Existing and New Structures. Current research activities include the following:

1. Testing and analysis of lightly reinforced concrete structures, and other structural components common in the eastern United States such as semi-rigid connections and flexible diaphragms.
2. Development of modern, dynamic analysis tools.
3. Investigation of innovative computing techniques that include the use of interactive computer graphics, advanced engineering workstations and supercomputing.

The ultimate goal of projects in this area is to provide an estimate of the seismic hazard of existing buildings which were not designed for earthquakes and to provide information on typical weak structural systems, such as lightly reinforced concrete elements and steel frames with semi-rigid connections. An additional goal of these projects is the development of modern analytical tools for the nonlinear dynamic analysis of complex structures.

This report details the results of a shake table experiment of a steel gable frame consisting of tapered members. The testing was conducted at the University of Buffalo on a 1/5 scale model. The study objectives were threefold:

1. *To observe the seismic behavior of a structure of this type and to compare the results with similarly designed gable frames composed of prismatic members.*

2. *Experimentally determine the ultimate strength of the structure and compare it with predictions by several design provisions.*

3. *Compare results associated with the instability problems with those of other experimental results subjected to quasi-static loading conditions.*

ABSTRACT

Behavior of a 1/5 scale gable frame structure composed of tapered members subjected to the El Centro earthquake ground motion applied through a shaking table was observed. The test structure was designed according to the AISC working stress design method. The width-thickness ratio of the flange and the depth-thickness ratio of the web were selected to satisfy the requirements of the compact section. The unbraced length was also proportioned to meet the compact section criteria determined from the section dimensions of the small end. The structural failure was due to lateral buckling of rafters. No premature local buckling prior to lateral buckling was observed. In addition, the experimentally determined ultimate strength of the test structure was compared with those predicted by AISC LRFD, AS 1250, and BS 5950. The experimental results were also compared with those of a shaking table study of a similarly designed gable frame composed of prismatic members.

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SECTION 1

INTRODUCTION

The use of tapered members was first proposed by Amirikian [1]. Based on a series of analytical and experimental studies [2,3,4,5], basic working stress design guidelines for tapered members were established by AISC [6]. A more comprehensive summary on the design of frame structures composed of tapered members is given by Lee et al [7].

To establish the AISC working stress design of tapered members, an axial and a flexural equivalent length factors were introduced into the design formulas of prismatic members [2,6]. Using the formula of elastic lateral torsional buckling and considering the inelastic lateral buckling of I-shaped beam, the working stress design formulas for tapered beams were given in AISC Formulas D3-1 and D3-2 corresponding to inelastic and elastic lateral buckling [2,3,6]. The maximum allowable bending stress was limited to a maximum of 0.6 F_y for inelastic lateral torsional buckling case (F_y is the nominal yielding stress of steel).

Based on AISC Appendix D and Commentary, as well as the AISC working stress design formulas, the maximum flexural strength of a tapered member is M_y. The compact section requirements of AISC are optional for the design of tapered members. Therefore, uncertainties arise if tapered frame structures are subjected to the demand of inelastic deformation from extreme loading condition such as strong earthquake ground motions.

Experimental investigations of inelastic buckling behavior of tapered members have been conducted by several researchers [5,10,16]. Prawel et al [5] conducted a testing program to determine the bending and buckling strength of fifteen tapered steel members. In another study, the ultimate load capacities of eight tapered specimens with and without lateral support were experimentally determined by Salter et al [10]. The test results were compared with those predicted by the British draft limit state code for structural steelwork in buildings and BS 449. In addition, a total of twenty seven tapered elements were tested in Japan to validate a proposed strength formula for the design of tapered members [16]. All these studies were carried out by using monotonic, quasi-static loadings. Furthermore, Lateral and local buckling governed the strength of all test specimens.

In this report, the results of a shaking table study of a 1/5 scale tapered steel gable frame structure are presented. The N-S component of the 1940 El Centro earthquake with different intensities was used as the input. The main objectives of this study were (a) to observe the seismic behavior of the gable frame composed of tapered members and to compare the results with those of a similarly designed gable frame structure composed of prismatic members [13]; (b) to experimentally determine the ultimate strength of the test structure and compare it with those predicted by using several design provisions [8,11,12]; and (c) to compare the results associated with instability problem with those of other quasi-static member tests and to ascertain the research needs for the determination of inelastic deformation capability of tapered members.

SECTION 2

TEST STRUCTURE

As shown in Fig. 2.1, the span of the 1/5 scale test structure was 16′ – 0″ from center to center of the supporting pins. The total rise from the center of pin to the center of roof crown was 5′ – 8″ of which the column height was 2′ – 11″. The bay width between the two parallel test frames was 5′ – 0″. The structural dimensions and layout are given in Fig. 2.1. Connecting the frame are purlins and struts to which corrugated sheets (and seismic reactive weight) are attached. The inner and the lower column and rafter flanges were also laterally supported by small angle sections at a few locations (see Fig. 2.1). The sag rods were supplied at every 1/3 span of purlins and struts between two parallel test frames. The deflection requirement was the main consideration in the design of these purlins.

The design dead and live loads were assumed to be 8 psf and 25 psf, respectively. The wind load was presumed to be 20 psf. Since ATC [14] and UBC [15] both allow a maximum reduction of 75 % of the design live load for the determination of earthquake loading on storage and warehouse structures, the seismic reactive weight, W, imposed on the test structure was set to dead load plus 50 % of live load. The total seismic reactive weight of 3.70 kips was simulated by using lead blocks which were uniformly distributed on the roof of the test structure. These lead blocks were placed on corrugated sheets (Gage 26) and fastened to steel purlins. The seismic reactive masses were, therefore, assumed to be lumped at the purlin locations, and the seismic equivalent lateral force was then transmitted to the frame through steel purlins. In calculating the equivalent lateral force, the parameters specified

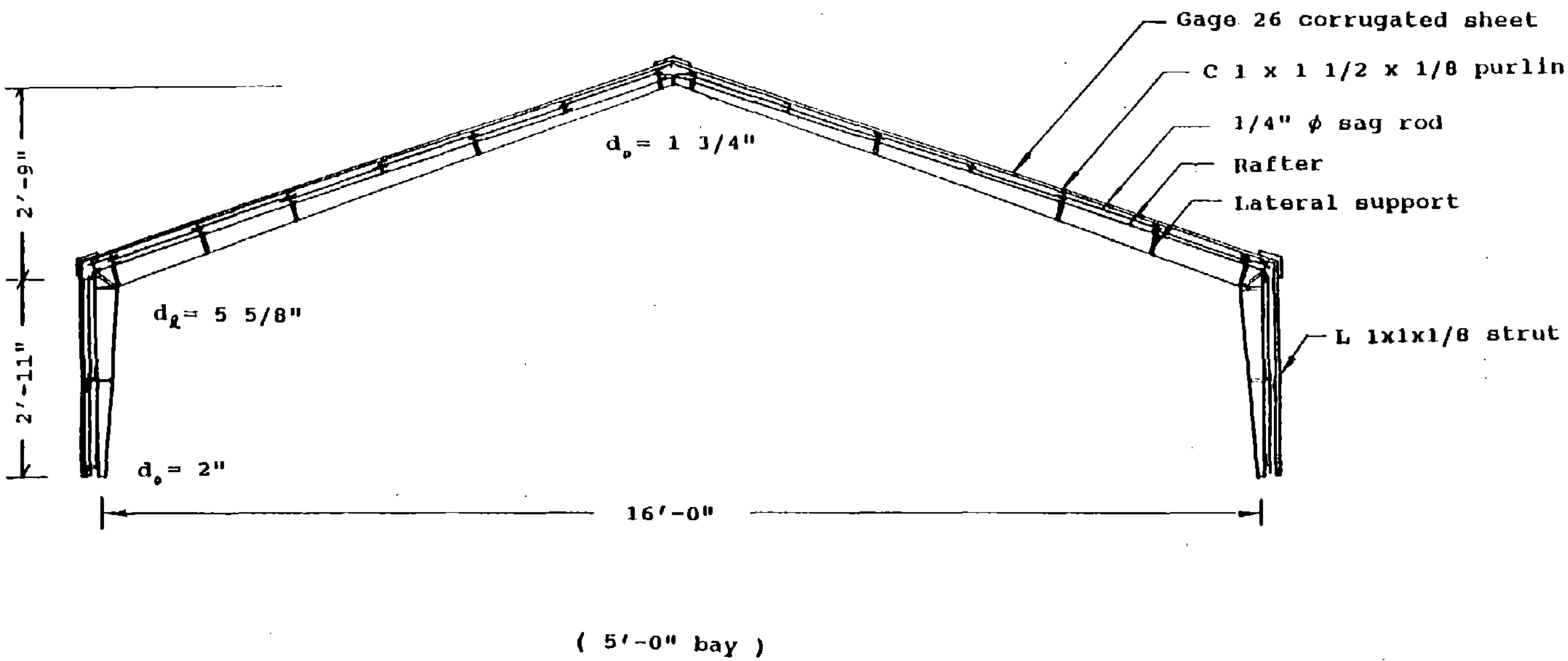


FIGURE 2.1 Structural Layouts

in ATC provisions were determined to be $A_a = A_v = 0.4$, S=1.5 (soil type S_3) and R=4.5 (ordinary steel moment frame). The base shear force was then equal to $V = C_s \times W = 2.0 \times A_a/R = 0.178\ W$. In order to obtain the total base shear force using UBC specifications, the natural frequency of the test structure was calculated to be 3.30 Hz. The natural frequency of the prototype then equals to 1.48 Hz ($3.30/\sqrt{5}$). Based on UBC, the base shear force was calculated to be $V = ZIKCSW = 0.112W$ with parameters Z=1.0 (seismic zone 4), I=1.0, K=1.0, C=0.081, S=1.5, and T=0.68 sec (natural period of the prototype). It should be noted that, in the seismic design of steel structures, UBC specifies the same working stress design as that of AISC. On the other hand, ATC suggests "significant yield" design by modifying the AISC working stress design.

The loading combinations considered in the design of the test structure were : (1) 1.0 DL + 1.0 LL; (2) 1.0 DL + 1.0 LL + 1.0 WL; (3) 1.0 DL + 1.0 LL + 1.0 EQ; and (4) 1.2 DL + 1.0 LL + 1.0 EQ, where DL, LL, WL and EQ are dead load, live load, wind load and earthquake load, respectively. The actual design of the test structure was governed by loading case 1. A 1/3 increase of the allowable stress was considered for loading cases 2 and 3. The loading case 4 was used for the significant yield design.

For the test structure, inelastic lateral buckling governed the flexural strength of structural members. The unbraced length of tapered members of the test structure satisfied the requirement [2]

$$\frac{h_w\ l}{r_{T_0}} < \sqrt{\frac{510 \times 10^3\ C_{br}}{F_y}} \tag{2.1}$$

where h_w is the equivalent length factor, l is the lateral unbraced length, C_{br}

is the moment gradient coefficient and r_{T_0} is the radius of gyration at the smaller end, taken about an axis in the plane of the web. Whereas, the slenderness ratio determined from the small end cross section of the unbraced segment exceeded the limit required by plastic design (AISC formulas 2.9-1a and 2.9-1b). Moreover, the width-thickness ratio of the flange and the depth-thickness ratio of the web satisfied the requirements of the compact section (AISC section 1.5.1.4) and the plastic design (AISC section 2.7).

SECTION 3

PREDICTION OF STRUCTURAL LATERAL STRENGTH

The ultimate strength of the test structure was determined from the flexural strength of tapered members specified by various provisions [8,11,12]. The internal forces of structural members were calculated using a mathematical model in which the masses were lumped at loading points (locations of purlins), and the pseudo-acceleration was uniformly distributed along the roof height. The calculated ultimate based shear force, V_{ult}, was normalized with respect to the base shear force at initial yielding, V_y. The ultimate and the initial yield base shear forces were determined, respectively, with and without the seismic reactive weight acting on the test structure. The strength formulas of tapered members given in several specifications are briefly summarized in the following :

(a) AISC LRFD : The AISC LRFD [8] specifies that the nominal flexural strength of tapered segments is equal to the allowable bending momemt multiplied by a constant of 5/3, as given in Eq. (3.1).

$$M_n = (5/3)\ S_x^{'} F_{b\gamma} \tag{3.1}$$

where $S_x^{'}$ is the section modulus of the critical section of a tapered member. $F_{b\gamma}$ is the allowable bending stress (Appendix D of Part 1 of AISC or Section F4 of Appendix F of AISC LRFD). Using this formula, the maximum flexural strength of tapered members predicted by AISC LRFD would be equal to My of the critical section.

Based on the nominal flexural strength provided in AISC LRFD, the predicted in-plane lateral capacity of the test structure is $V_{ult}/V_y = 0.86$. The maximum base shear coefficient is $(C_s)_{max} = 1.08$.

(b) AS 1250 : The Standards Association of Australia [11] specifies the elastic critical moment of a tapered member by

$$M_E = \alpha_{st} M_0 \tag{3.2}$$

and

$$\alpha_{st} = 1.0 - 0.6[1.0 - (0.6 + 0.4\frac{D_m}{D_c})\frac{A_m}{A_c}] \tag{3.3}$$

where D_m and D_c are, respectively, the depths of the small and the critical sections; A_m and A_c are their areas. The critical section is defined as the section where the ratio between the exerting and the plastic moments is the largest. M_0 in Eq. (3.2) is the elastic lateral buckling moment of a prismatic member with a section identical to the critical section of the tapered member.

The design flexural strength is determined by

$$M_b = \alpha_m \alpha_s M_p \tag{3.4}$$

and

$$\alpha_s = 0.6\{[(M_p/M_E)^2 + 3]^{1/2} - M_p/M_E\} \tag{3.5}$$

where α_m is a factor to consider the moment gradient and is essentially equal to C_b in AISC formulas (1.5-6a,b) and (1.5-7). M_p is the plastic moment of the critical section.

Based on the design flexural strength of AS 1250, the predicted in-plane lateral strength of the test structure is $V_{ult}/V_y = 1.21$. The maximum base shear force coefficient is $(C_s)_{max} = 1.53\ g$.

(c) BS 5950 : The British Standards Institution specifies that the elastic critical moment of a tapered member is expressed as

$$M_E = \frac{M_p \pi^2 E}{\lambda_{LT}^2 P_Y} \tag{3.6}$$

M_p is the plastic moment of a section where the applied moment is the largest. λ_{LT} is the equivalent slenderness factor (BS 5950 Sections B.2.5.1 and B.3) The buckling resistance moment is then obtained from

$$M_b = \frac{M_E M_p}{(\Phi_B + \Phi_B{}^2 - M_E M_p)^{1/2}} \tag{3.7}$$

and

$$\Phi_B = \frac{M_p + (\eta_{LT} + 1) M_E}{2} \tag{3.8}$$

η_{LT} is called the Perry coefficient (BS 5950 Sections B.2.3 and B.2.4).

Using the buckling resistance moment of BS 5950, the predicted strength is $V_{ult}/V_y = 0.61$. The maximum base shear coefficient is $(C_s)_{max} = 0.77$.

(d) ELASTIC-PLASTIC SOLUTION : Using the elastic-perfectly plastic model, the lateral capacity of the test structure was determined to be $V_{ult}/V_y = 1.23$. The maximum base shear coefficient is $C_s = 1.55$.

SECTION 4

EXPERIMENTAL RESULTS

(a) Test Program and Dynamic Characteristics

The experimental sequence for observing the elastic, inelastic and buckling behaviors is given in Table 4.1. A total of 80 channels of the data acquisition system were used to measure structural responses. Typical instrumentation layouts are shown in Fig. 4.1. Using the accelerations measured at steel purlin locations, the acceleration distribution along the roof height in each test was determined. Strain gages were used to identify the range of inelastic zones along rafters and columns, and to deduce the curvatures and the moments at plastic zones. The transfer functions and the dynamic structural characteristics after each test are shown in Fig. 4.2 and Table 4.2, respectively. From Table 4.2, it appears that the natural frequency after each test decreases gradually, and the damping factor becomes larger.

(b) Acceleration Distribution along Roof Height :

The distribution of maximum acceleration along the roof height of the test structure under different ground excitation intensities is shown in Fig. 4.3. These results suggest that assuming a constant pseudo-acceleration distribution over the steel purlin locations along the roof height is an appropriate approach to distribute the seismic equivalent lateral force. In each test, the small variation of peak acceleration responses (see Fig. 4.3) along the roof height is due to the local vibration of the rafter (higher mode).

(c) Strain Distribution :

During the elastic test, the strains measured along a large portion of the rafter

TABLE 4.1 Test Program

TEST SEQUENCE
White Noise Test
0.30 g ELC Test
White Noise Test
0.45 g ELC Test
White NOise Test
0.71 g ELC Test
White Noise Test
0.88 g ELC Test
White Noise Test
1.08 g ELC Test (i)
White Noise Test
1.08 g ELC Test (ii)
White Noise Test

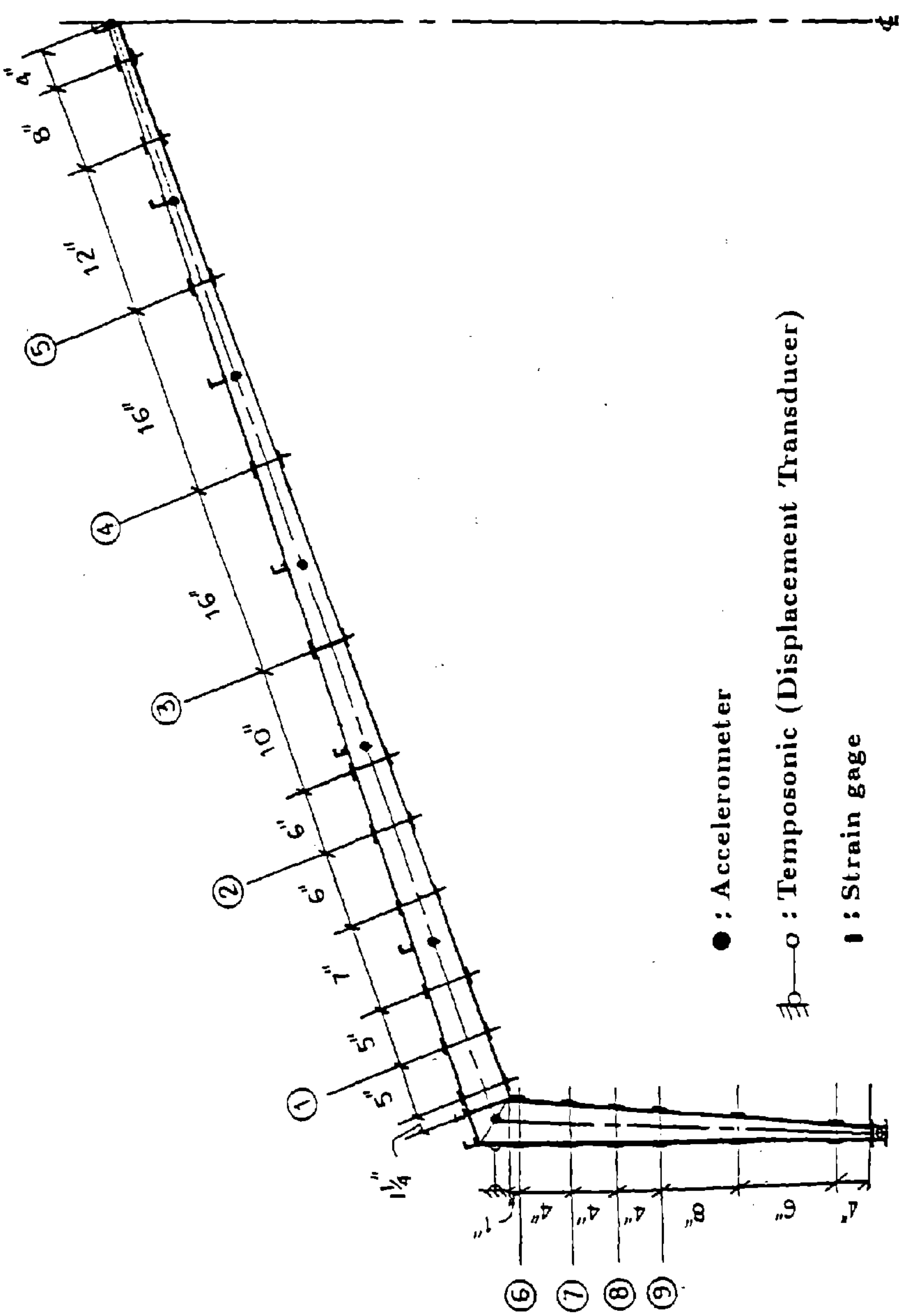
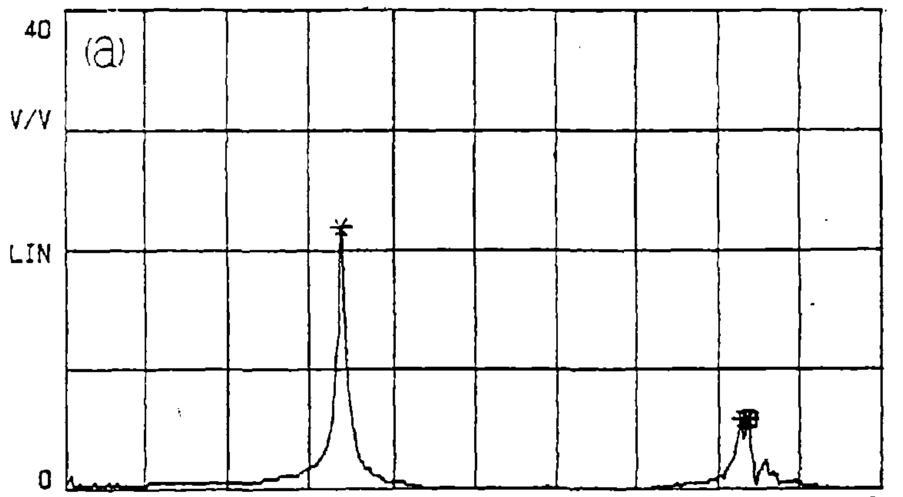
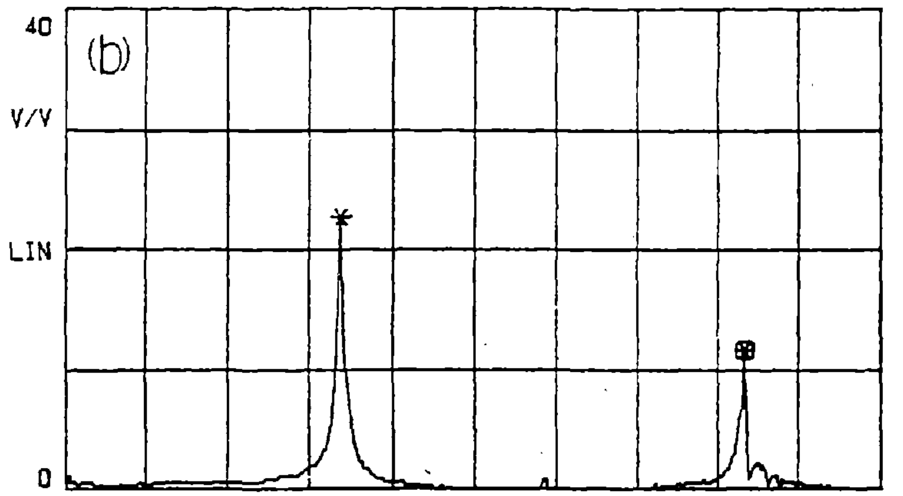


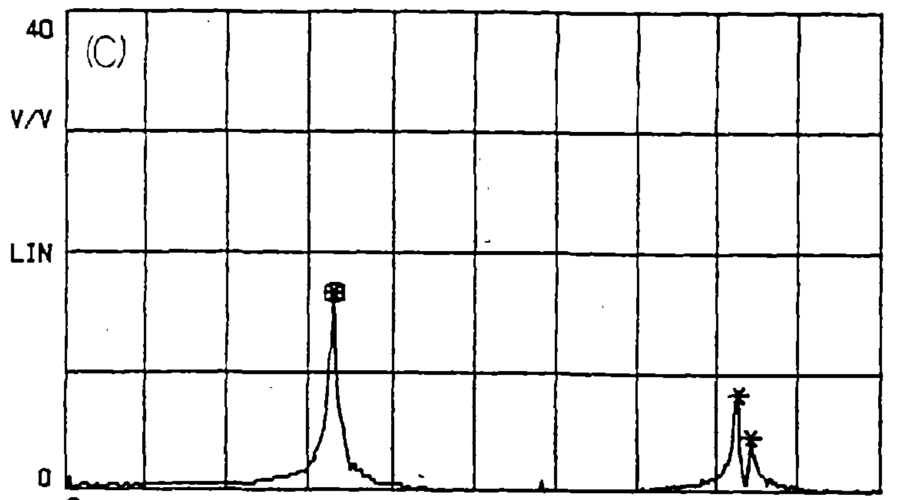
FIGURE 4.1 Typical Instrumentation



(a) Before 0.39 g ELC Test

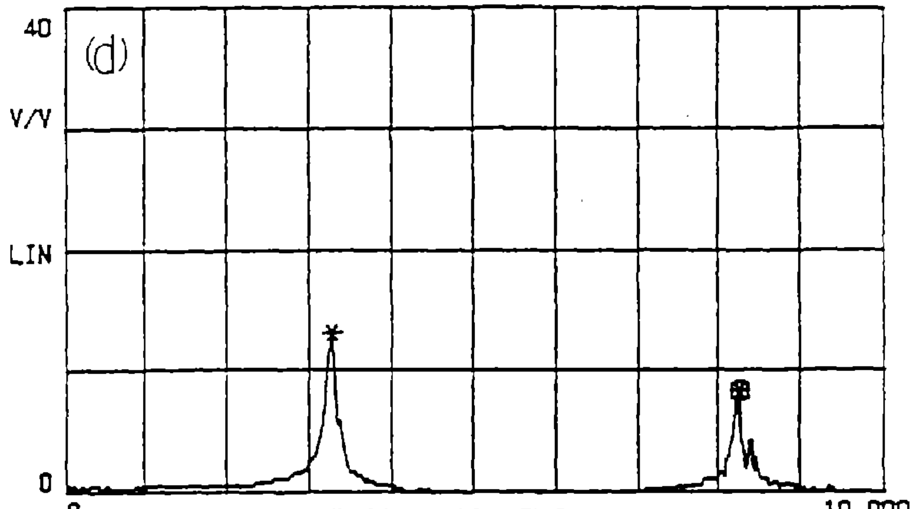


(b) After 0.45 g ELC Test

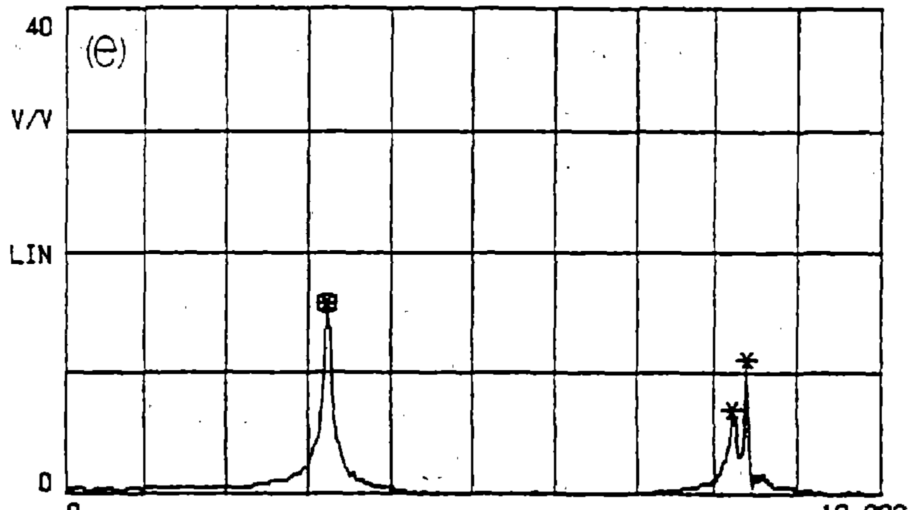


(c) After 0.71 g ELC Test

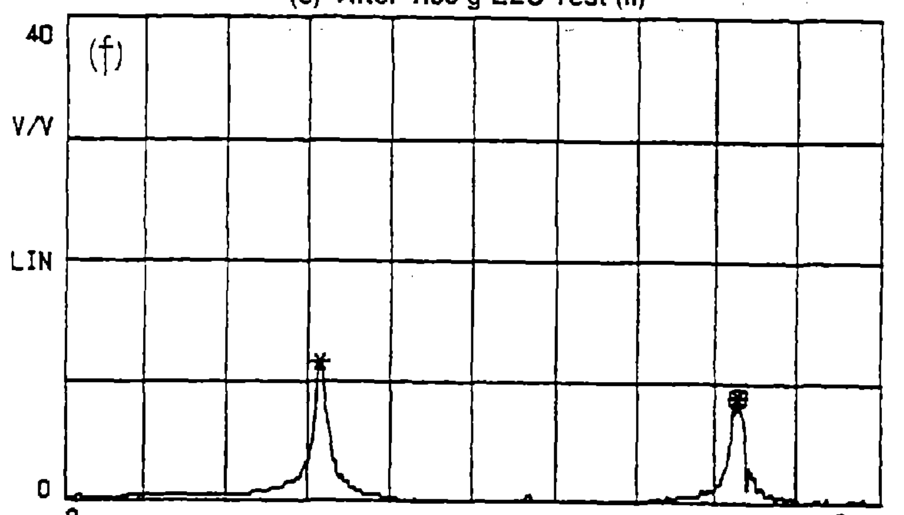
FIGURE 4.2 Transfer Functions



(d) After 0.88 g ELC Test



(e) After 1.08 g ELC Test (II)



(f) After 1.08 g ELC Test (II)

FIGURE 4.2 Transfer Functions (Cont'd)

TABLE 4.2 Dynamic Characteristics Resulting From Each Test

Test ID	Natural Frequency (Hz)	Damping (%)
0.30 g ELC	3.35	2.28
0.45 g ELC	3.35	2.22
0.71 g ELC	3.28	3.04
0.88 g ELC	3.25	3.71
1.08 g ELC (i)	3.20	3.18
1.08 g ELC (ii)	3.13	4.27

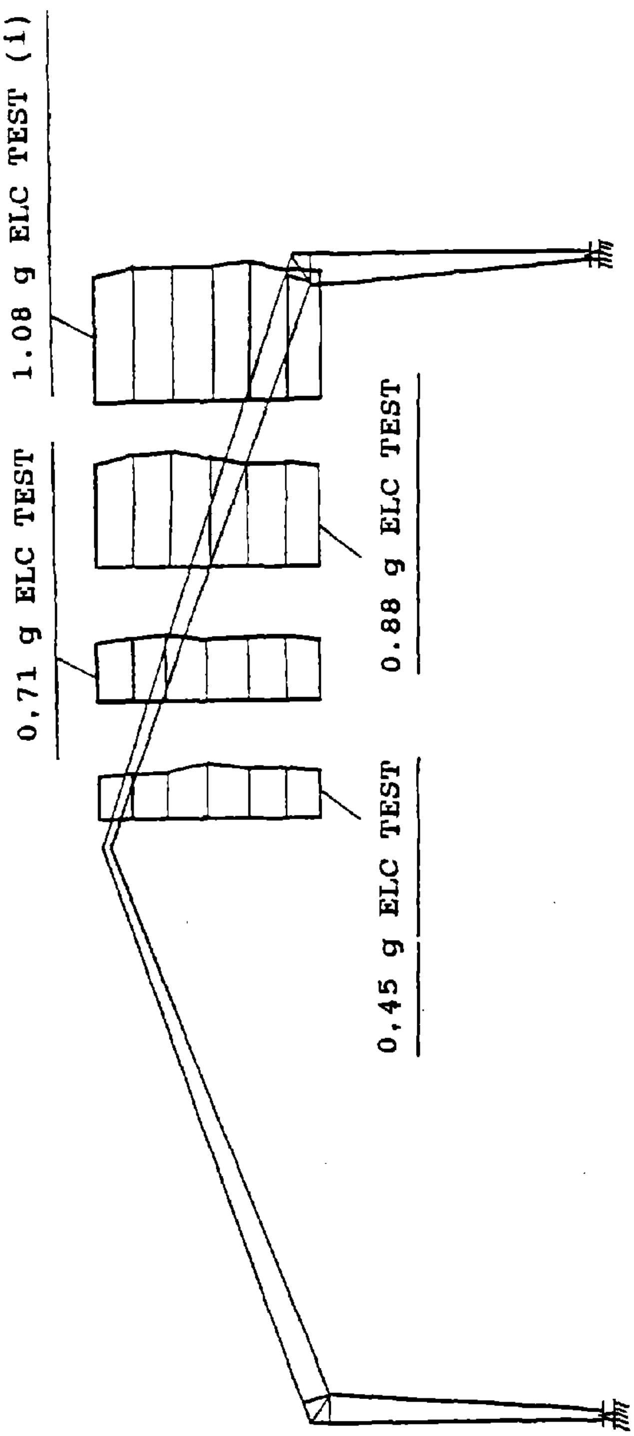


FIGURE 4.3 Acceleration Distribution of Each Test

and the column were basically identical. A typical result showing the elastic bending strains measured at rafter section 1 to 5 of Fig. 4.1 is given in Fig. 4.4. During inelastic tests, a large range of the rafter and the column yielded. During a severe inelastic test, 1.08 g ELC test (i), 75 % of the rafter length and 15 % of the column length become yielded. Time histories of flexural strains measured at rafter section 1 to 5 of Fig. 4.1 are shown in Fig. 4.5, and the moment-curvature hysteretic curves of column section 6 to 9 of Fig. 4.1 are shown in Fig. 4.6. The length of inelastic zones of the rafters and the columns during each of the tests is summarized in Fig. 4.7. Because the inelastic deformations were so widely spread over the member length, the local ductility demand would be smaller than that for a more concentrated plastic zone if the same amount of energy dissipation is expected. For example, the maximum flexural strain in the strong axis direction was only 0.28 % for the 1.08 g ELC test (i).

(d) Envelope Curve : The envelope of maximum base shear versus maximum story drift for each of the tests is shown in Fig. 4.8. The base shear force is normalized with respect to the predicted yielding base shear force. Based on this figure, the maximum lateral strength of the test structure is experimentally defined. The damage of the test structure was due to lateral buckling of the rafter (see Fig. 4.9) occurred at the final test, 1.08 g ELC (ii). It is seen that the maximum base shear force dropped suddenly because of lateral rafter buckling.

(e) Comparison with Predicted Ultimate Strength : As described earlier, the predicted structural strength based on various specifications varied considerably. As can be seen in Fig. 4.8, the experimental strength is 59 %, 62%, 128 % and 221 % more than those predicted by the elasto-plastic solution, AS 1250, AISC LRFD

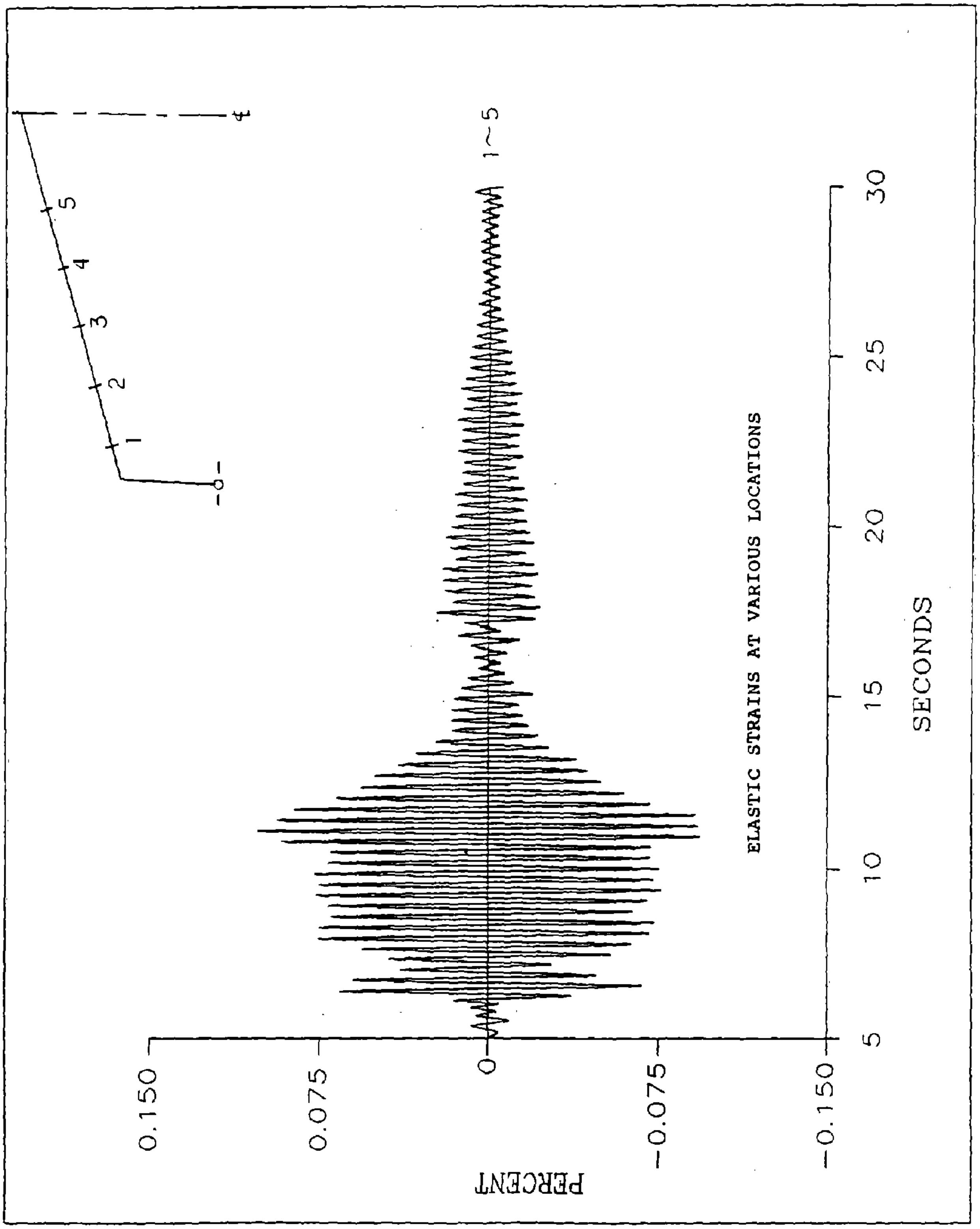


FIGURE 4.4 Elastic Bending Strains of Rafter Sections (0.30 g ELC Test)

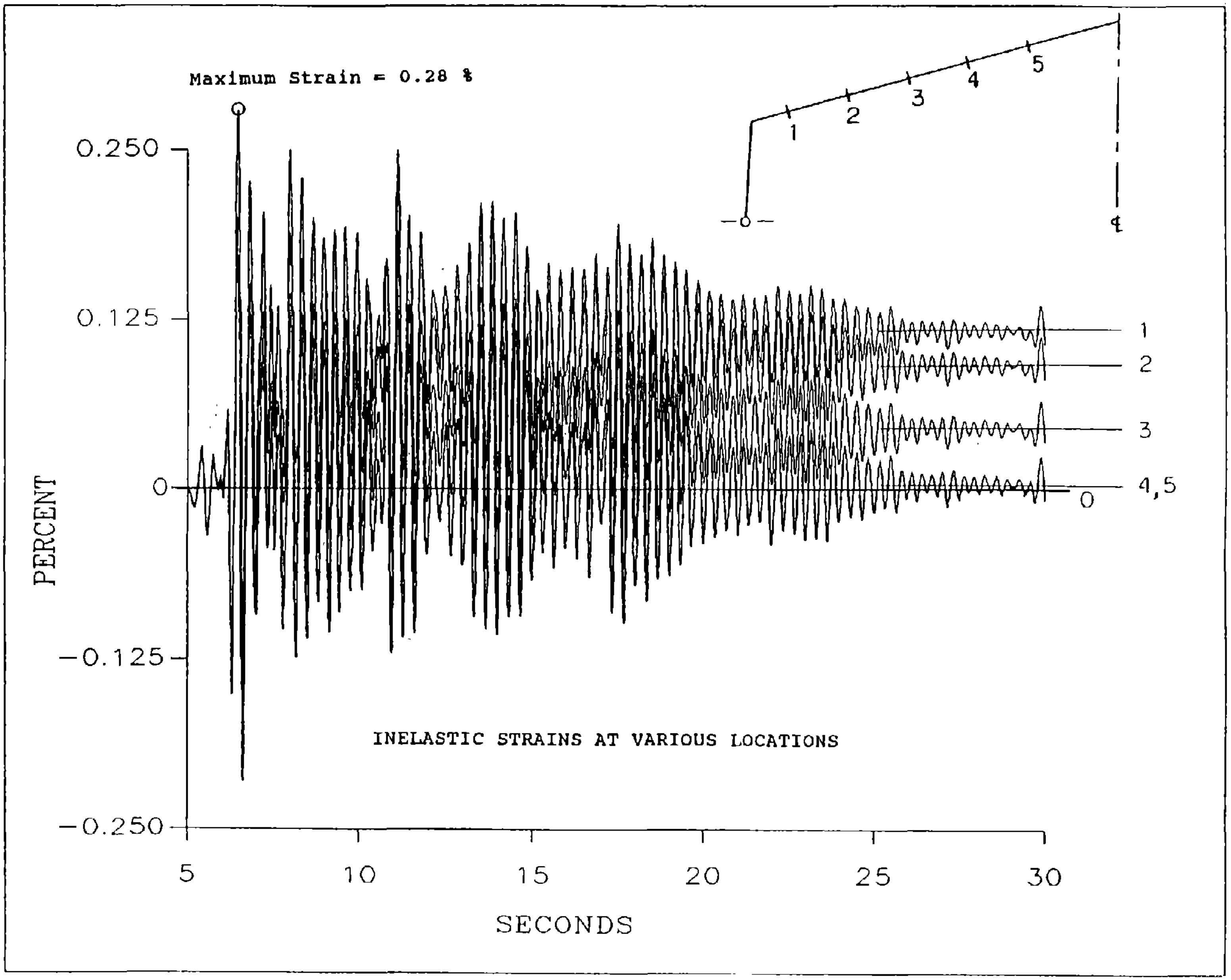
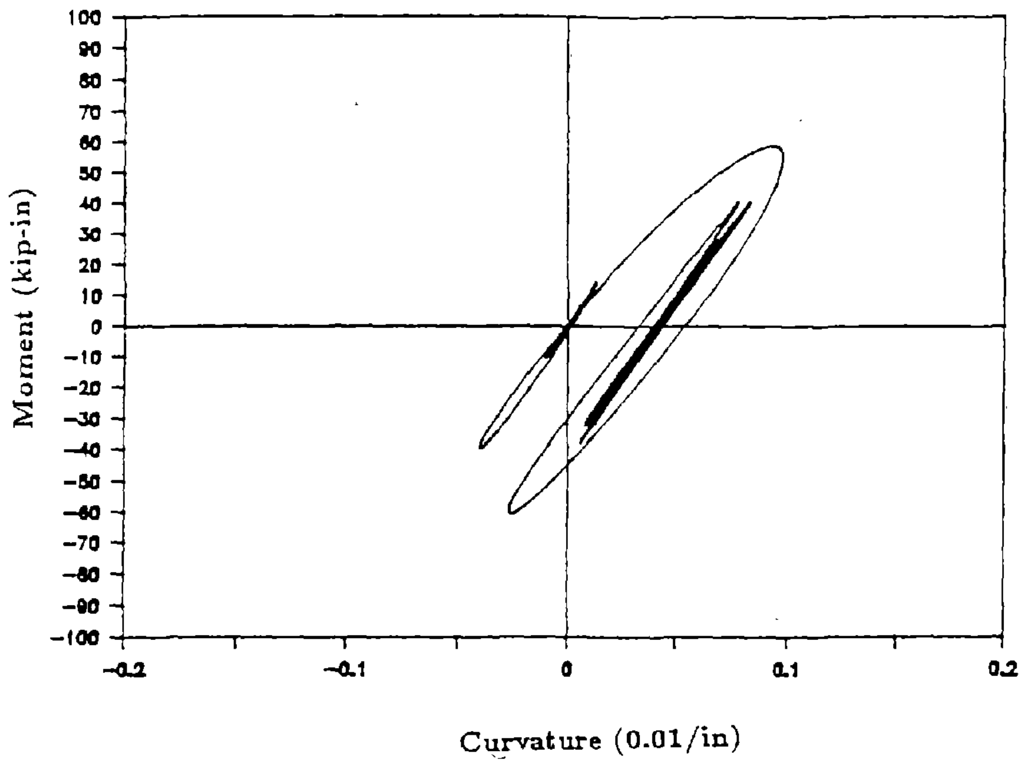
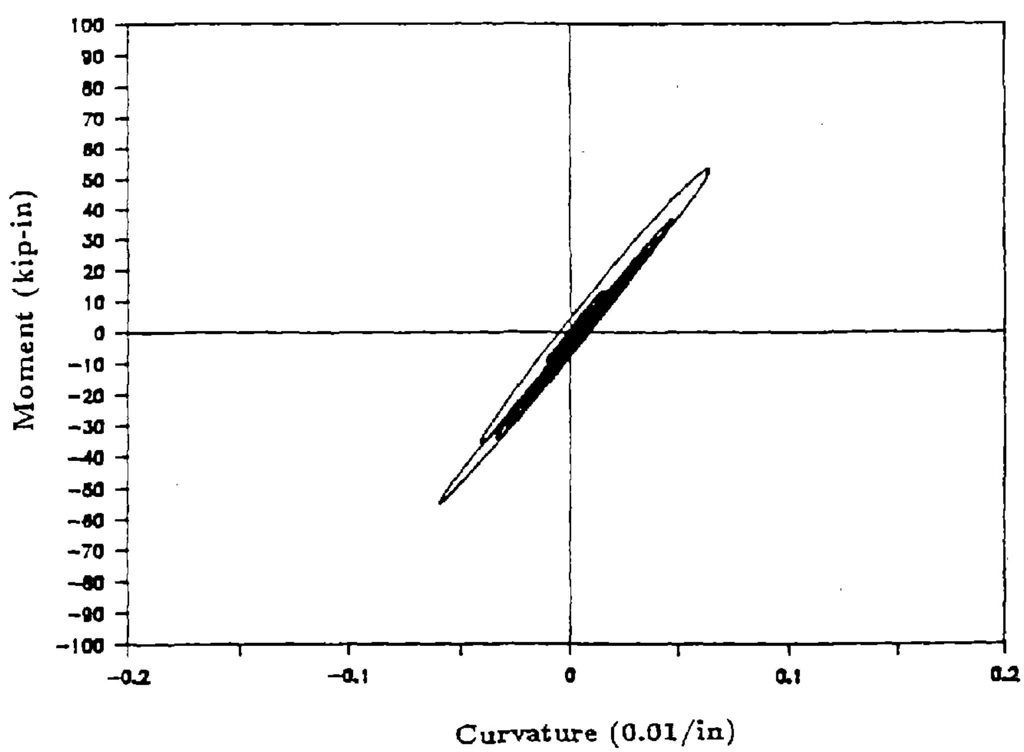


FIGURE 4.5 Inelastic Bending Strains of Rafter Sections (1.08 g ELC Test (I))

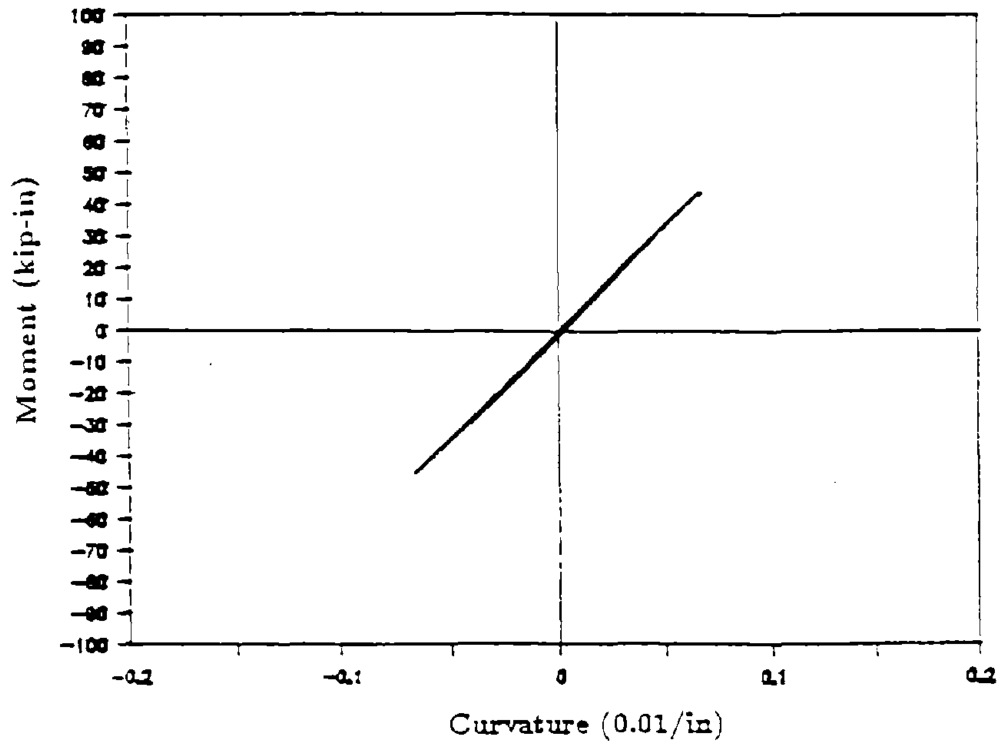


(a) Section 6

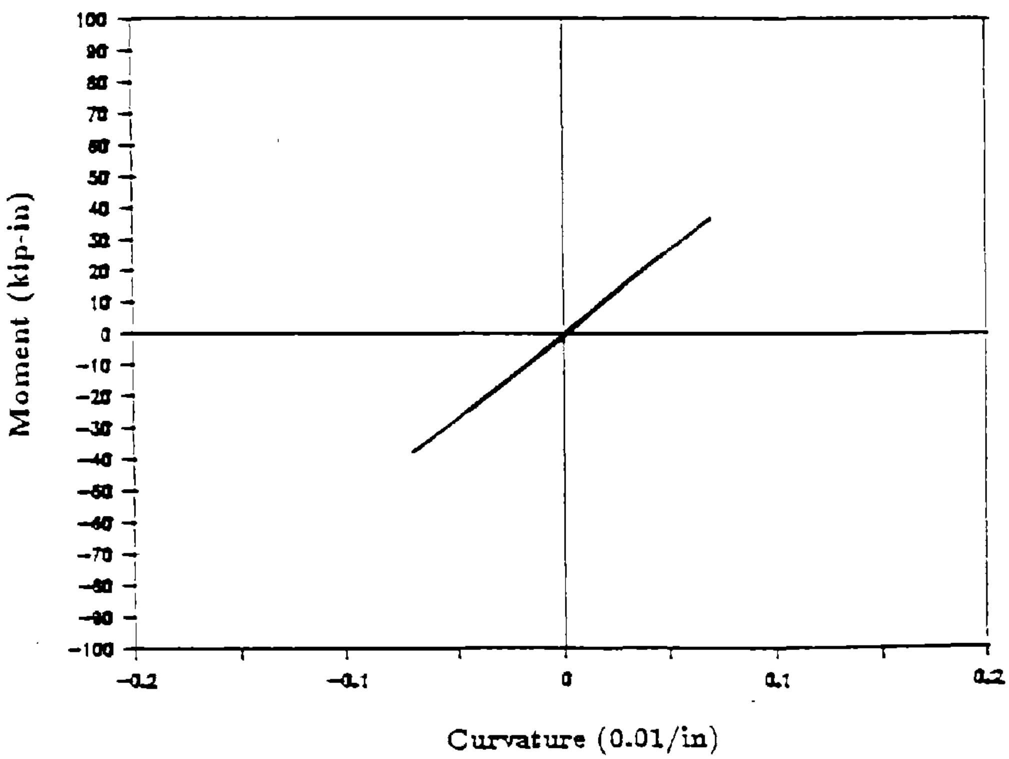


(b) Section 7

FIGURE 4.6 Moment-Curvature Hysteresis Curves of Column Sections During 1.08 g ELC Test (i)

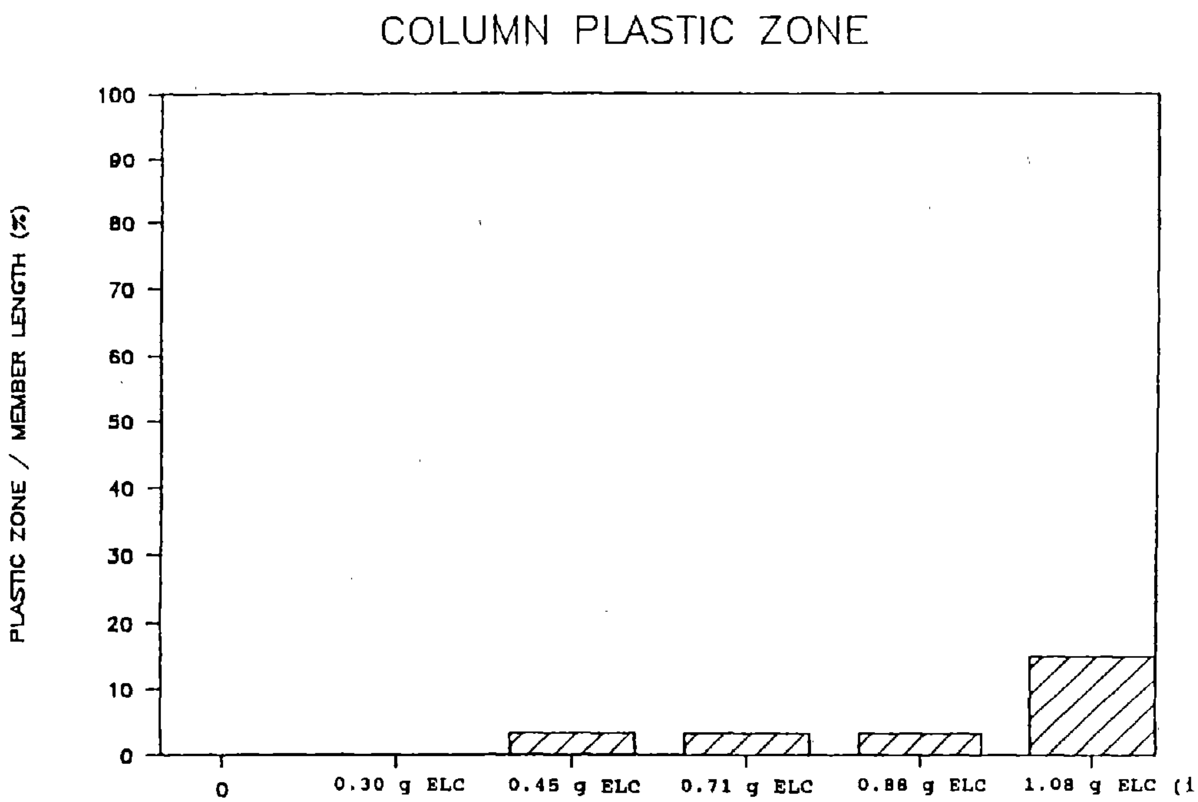


(c) Section 8

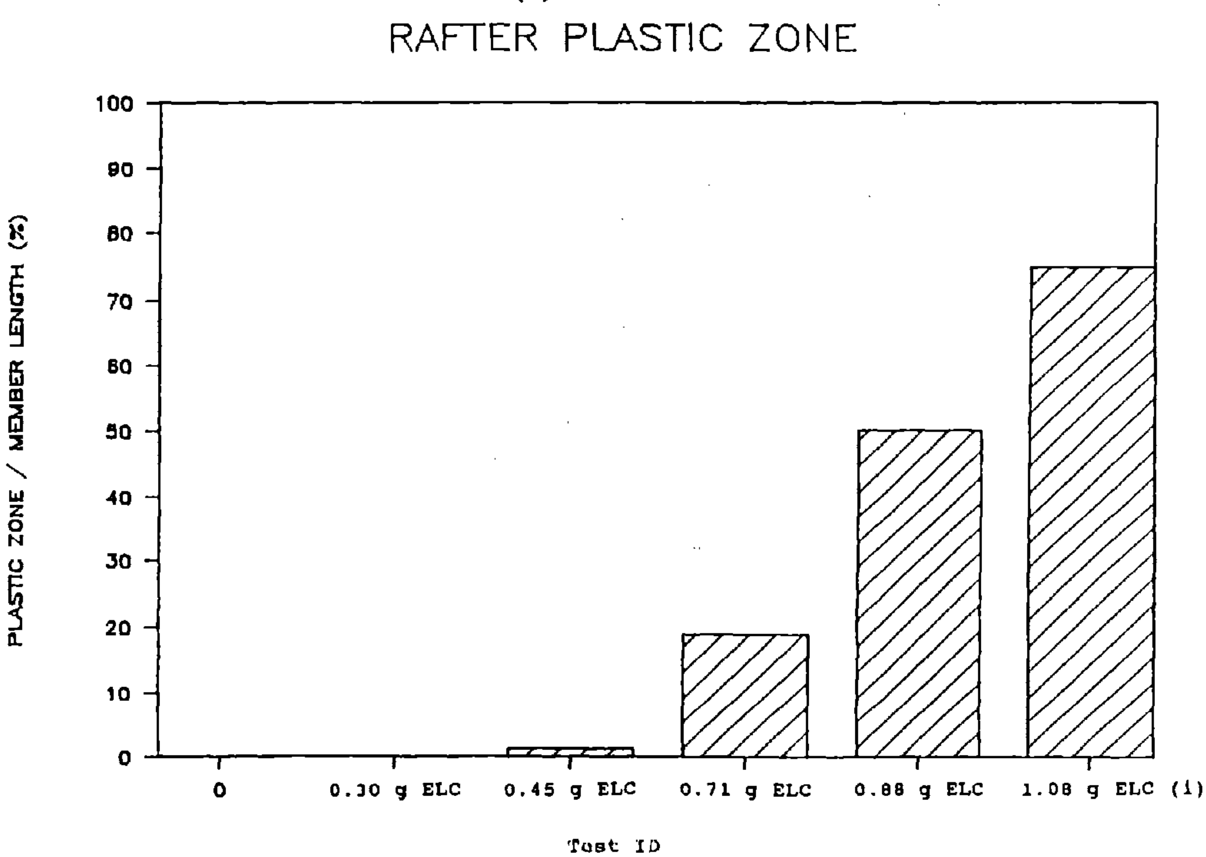


(d) Section 9

FIGURE 4.6 Moment-Curvature Hysteresis Curves of Column Sections During 1.08 g ELC Test (i) (Cont'd)



(a) Column



(b) Rafter

FIGURE 4.7 Length of Inelastic Zone During Each Test

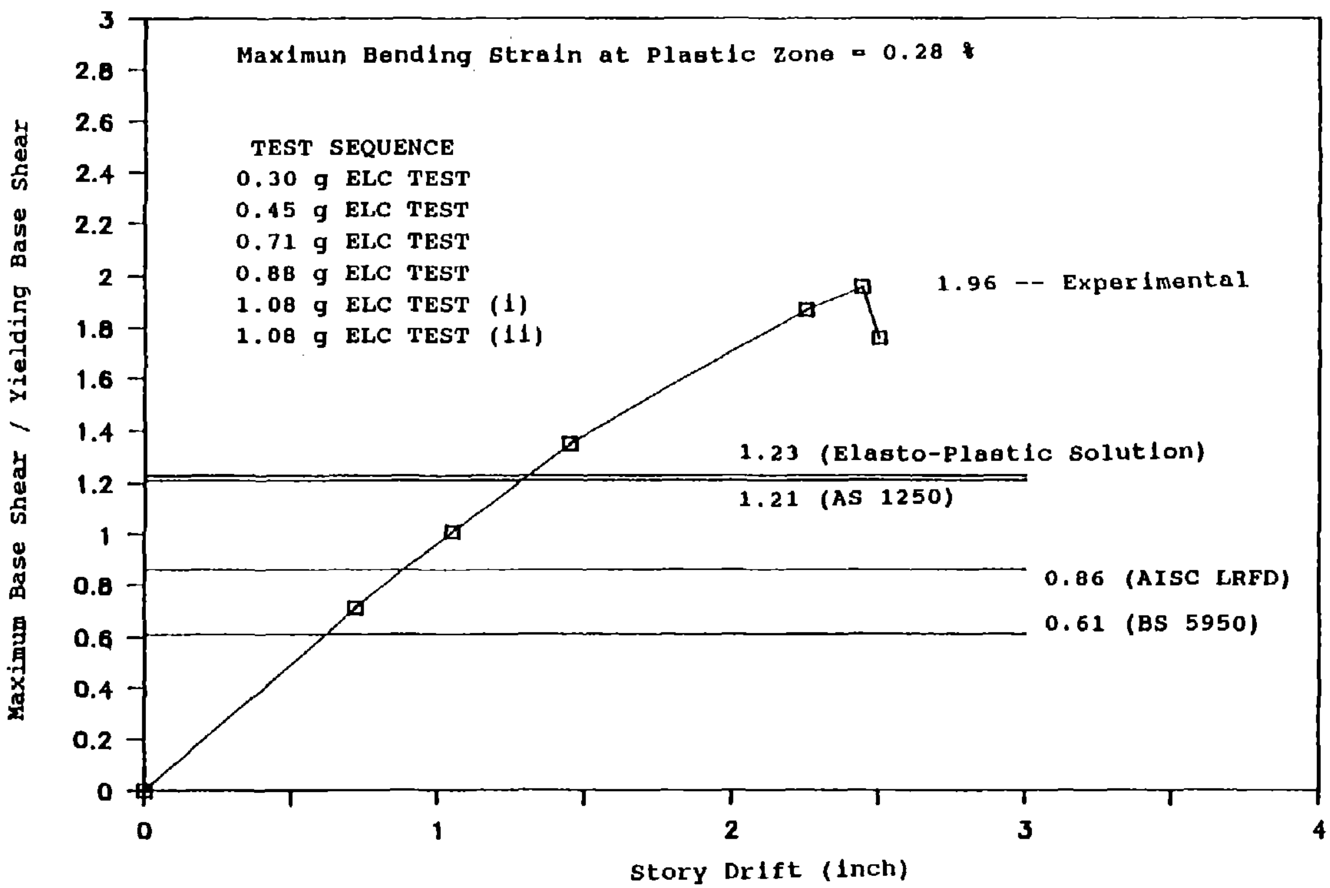


FIGURE 4.8 Envelope Curve of Tapered Gable Frame Tests

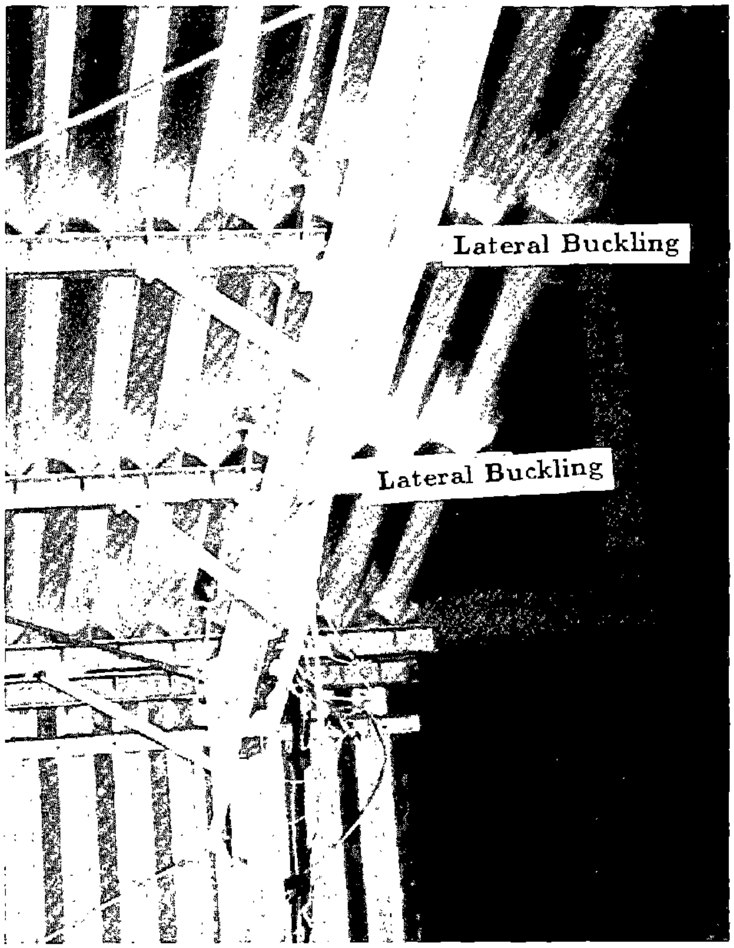


FIGURE 4.9 Photograph of Lateral Rafter Buckling

and BS 5950, respectively. Considerable discrepancies exits among the ultimate structural strength values predicted by using different specifications.

(f) Comparison with Other Quasi-Static Test Results : For the test structure, both the cross sections and the unbraced length meet the requirements of compact sections. It was observed from the experiemtal results that no premature local buckling occurred until after the lateral buckling of the rafters in the final test, 1.08 g ELC test (ii). These experimental results are compared in the following with those previously carried out using monotonic, quasi-static loadings by Salter et al [10] and Prawel et al [5].

As reported by Salter et al [10], The failure mode for all test specimens was lateral buckling. The width-thickness ratio of the flange and the depth-thickness ratio of the web were both smaller than the limits specified for the compact section, whereas the unbraced segments were longer than the requirement of the compact section.

According to the experimental results of Prawel et al [5], local flange buckling led directly to failure for most test specimens. The width-thickness ratio of the web and the depth-thickness ratio of the flange did not satisfied the requirements of the compact section. In addition, unbraced lenth exceeded the limit of the compact section.

From the above comparision, it may be concluded that premature locak buckling may occur prior to the lateral buckling if the cross section of tapered members does not meet the requirement of the compact section. Otherwise, lateral buckling may govern the flexural strength of tapered member, even though the unbraced length satisfies the requirement of the compact section.

(g) Comparison with Shaking Table Test of A Prismatic Gable Frame : The envelope curve obtained from a shaking table test of a similarly designed gable frame composed of prismatic members [13] is shown in Fig. 4.9. The damage of the prismatic gable frame was due to the local flange buckling occurred at column tops. In the final test, the 0.80 g ELC test (ii), the maximum base shear force remained the same as that of the 0.80 g ELC test (i), but the lateral relative displacement increased significantly. For the tapered gable frame, the rafters were subjected to lateral torsional buckling during the 1.08 g ELC test (ii), and the maximum total base shear force dropped suddenly. Therefore, the lateral buckling strength governed the capacity of the test structure composed of tapered members, on the other hand, the total base shear force required for the formation of structural failure mechanism determined the strength of the test structure composed of prismatic members.

The structural displacement ductility ratios , D_{ult}/D_y, of the tapered and the prismatic gable frames are approximately 2.3 and 3.0, respectively (see Figs. 4.7 and 4.9.) D_{ult} is the maximum lateral relative deflection and D_y is the lateral relative displacement at initial yielding. The hysteretic curves of normalized base shear versus normalized relative displacement of both test structures at their final tests are compared in Fig. 4.10. The hysteretic energy represented by the area enclosed in the hysteretic curve is larger for the prismatic gable frame than that for the tapered gable frame structure. From Fig. 4.11, the hysteretic energy time history compared with the input energy time history is larger for the prismatic test frame than that for the tapered test frame. This again confirms the fact that the failure mode is different for prismatic and tapered gable frames.

For both test structure, maximum story drifts were much larger than the limit specified in the seismic design provisions by UBC and ATC.

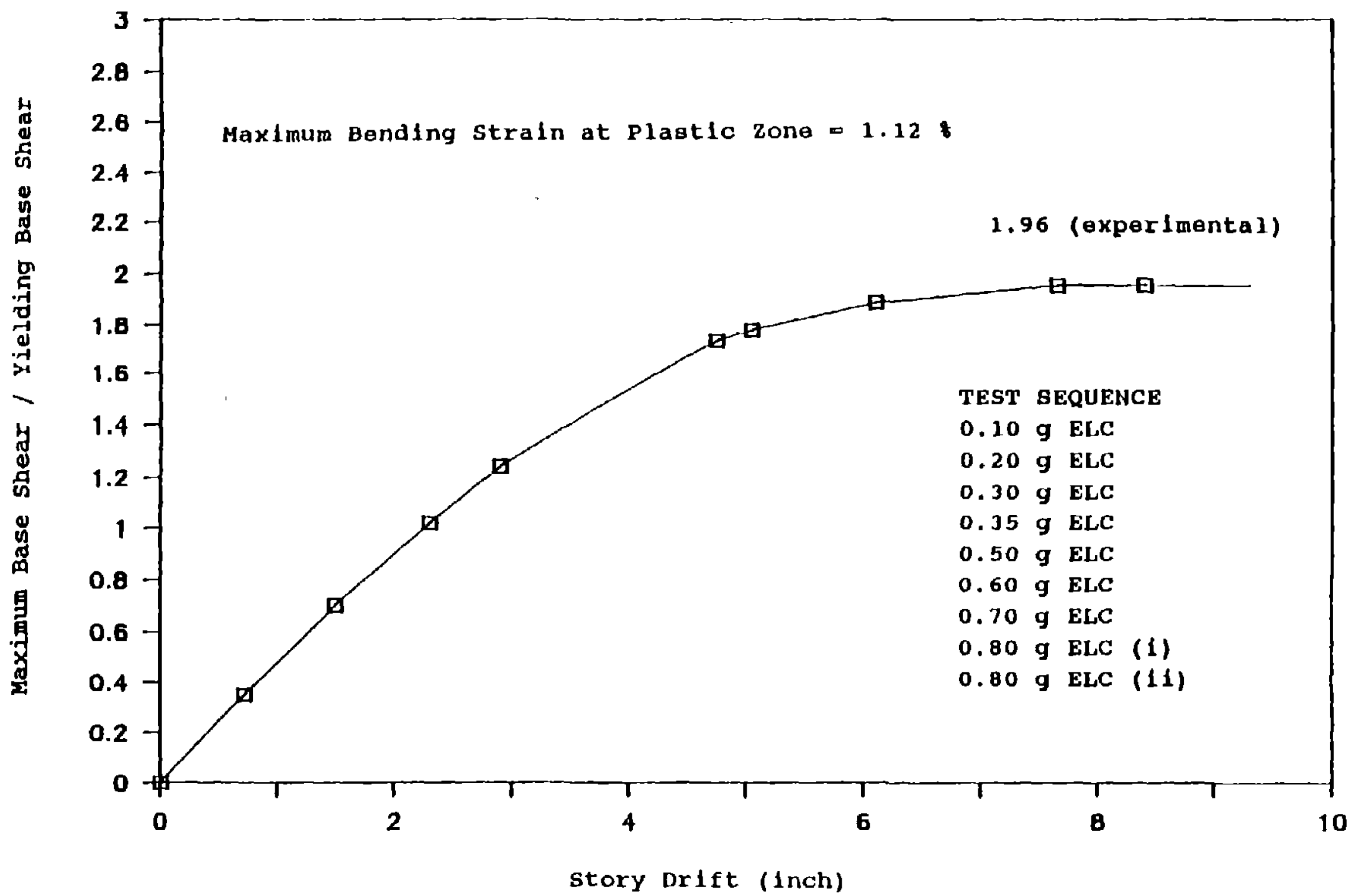
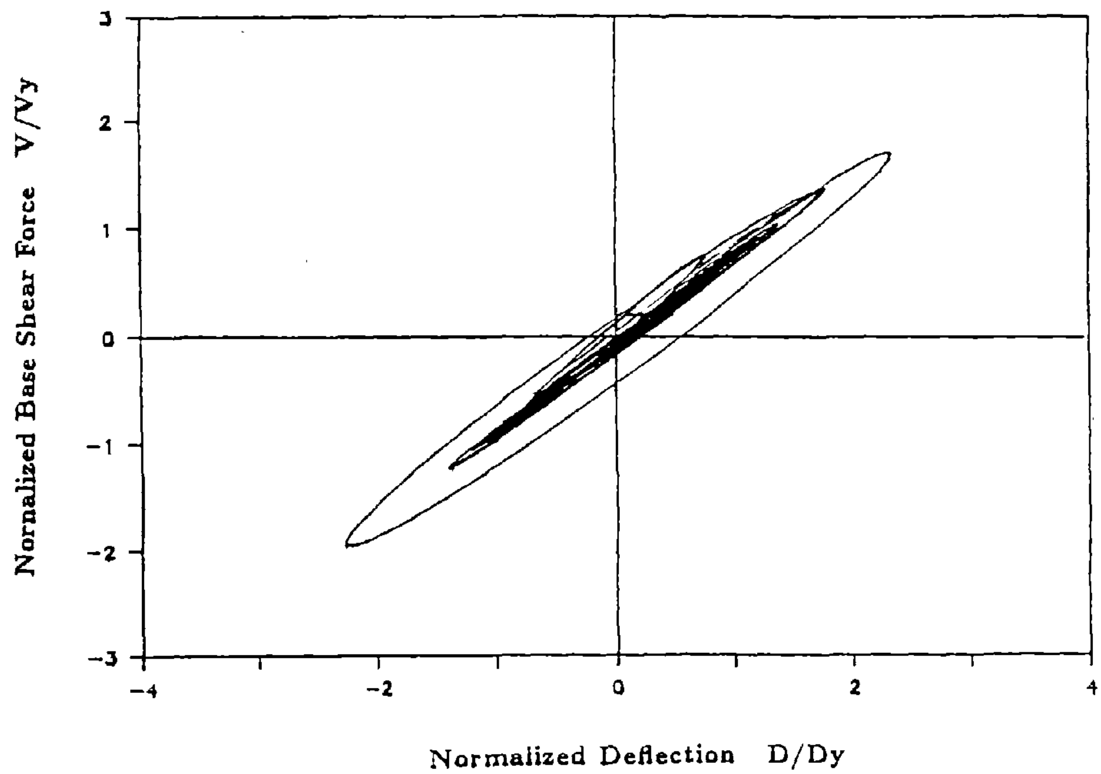
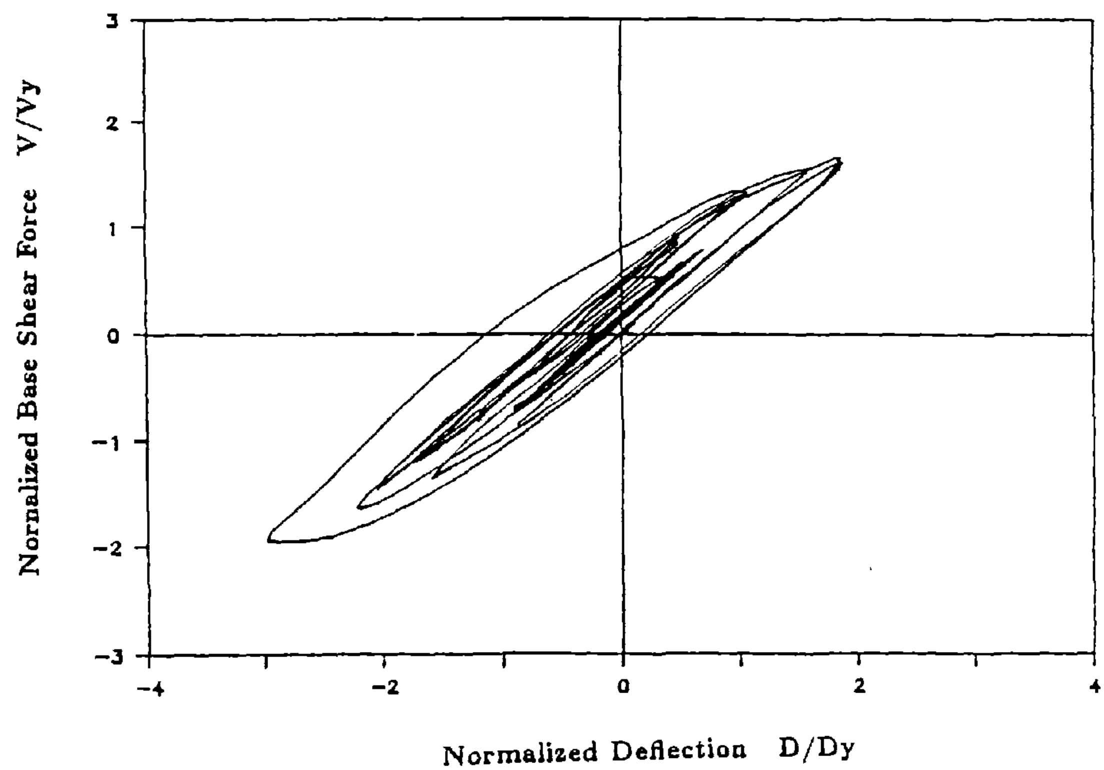


FIGURE 4.10 Envelope Curve of Prismatic Gable Frame Tests

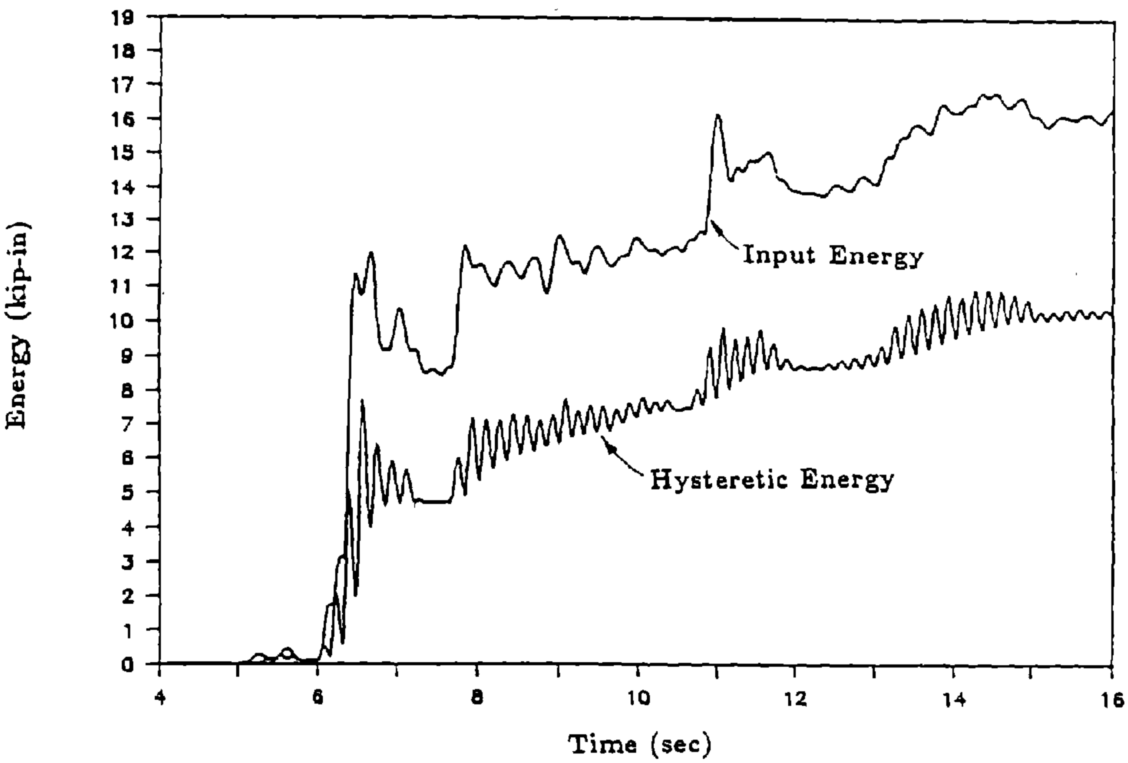


(a) Tapered Gable Frame

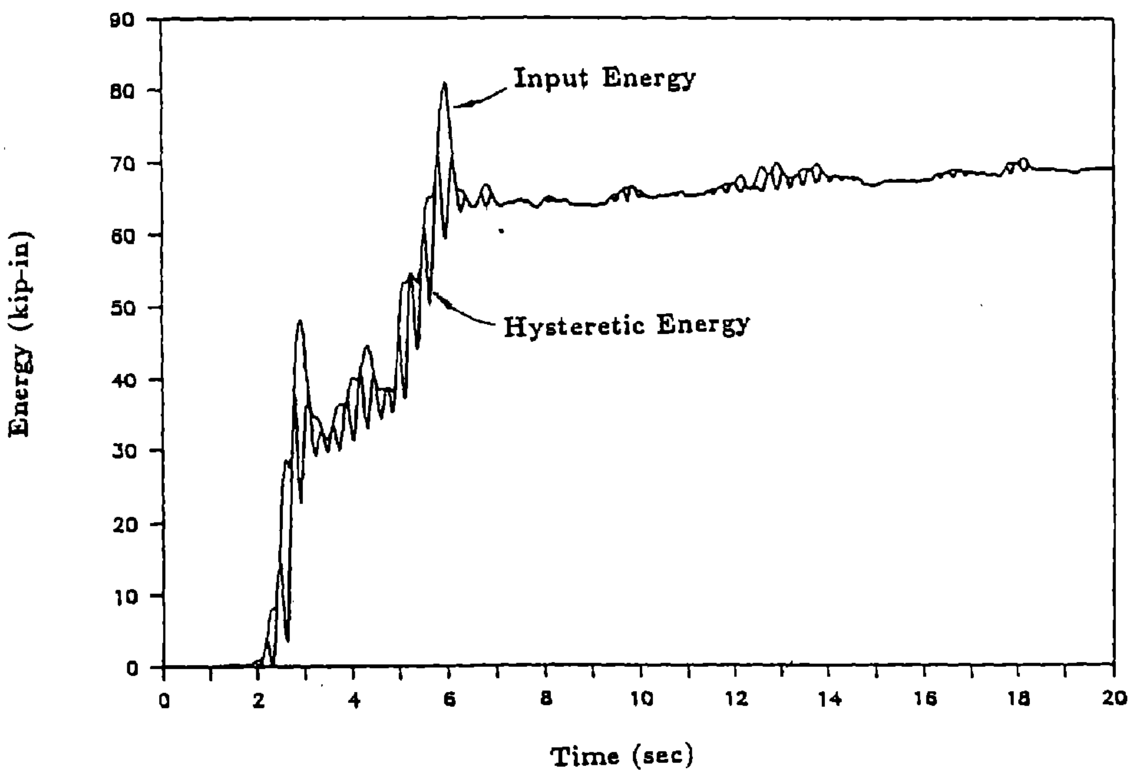


(b) Prismatic Gable Frame

FIGURE 4.11 Normalized Hysteresis Curve



(a) Tapered Gable Frame



(b) Prismatic Gable Frame

FIGURE 4.12 Hysteresis and Input Energy Time Histories

SECTION 5

SUMMARY AND CONCLUSIONS

The acceleration distribution at purlin locations along the roof girder was basically constant during each test. To distribute the equivalent lateral force, it is appropriate to assume an uniformly distributed pseudo-acceleration.

The predicted structural strength by using different specifications varied considerably. The experimental ultimate strength of the test structure was much higher than the predicted ultimate strength. There is a need to futher examine the methods of predicting the ultimate strength of tapered members.

The elastic flexural strains at various locations of the tapered members were basically identical, as expected. In the inelastic range, wide width "plastic hinges" were observed. Due to large inelastic zones, local ductility demand is smaller.

Comparing the experimental results with those of previous test under quasi-static loading condition, it may be concluded that if the width-thickness and the depth-thickness ratios are larger than the limits of the compact section, premature local buckling may occur prior to lateral buckling and lead directly to the failure of tapered member. If the cross section and the unbrace length meet the requirements of the compact section, lateral buckling strength may govern the flexural capacity of tapered member. Strict requirements on the unbraced length of tapered mnembers are necessary, particularly if the tapered members are subjected to inelastic deformations.

The dissipated energy compared with the input energy was larger for the prismatic gable frame than for the tapered gable frame, even though the tapered gable frame had much larger inelastic zones. Proper lateral supports are necessary to prevent the tapered members from lateral buckling so that high energy dissipation by large inelastic zone can be possibly obtained.

SECTION 6

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