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Seismic behavior of multistory braced steel frames

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SEISMIC BEHAVIOR OF MULTISTORY BRACED STEEL FRAMES

by S. C. Goel and R. D. Hanson

Committee of Structural Steel Producers • Committee of Steel Plate Producers

american iron and steel institute

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Seismic Behavior of Multistory Braced Steel Frames

Subhash C. Goel
Robert D. Hanson

Chapter 1
Introduction

By the mid 1950's, digital computer techniques for the elastic dynamic response of multistory building frames subjected to earthquake motion had been fairly well established. Three of the pioneers in the development of this field were Tung and Newmark (1) and Clough (2) whose studies indicated that the elastic dynamic analysis, with or without viscous damping, predicted story shears much greater than those specified by typical building codes. Yet the code specifications were favored by the fact that they led to economical designs which had successfully withstood severe earthquakes in the past, as for example, the Kern County earthquake of 1952 as reported by Degenkolb (3).

This discrepancy between the predicted linear elastic response and the observed behavior of similar structures was thought to lie in the ability of the structural frame to dissipate energy through inelastic deformation. In 1956, Housner (4) introduced the concept of inelastic design based on energy input to a structure during an earthquake and its capacity to dissipate energy through inelastic hysteretic deformation. It was pointed out that safe and economical structures could be designed to withstand a strong earthquake by permitting limited inelastic hysteretic activity in the structural frames.

A number of investigations were undertaken in the past decade to study the inelastic behavior of structures when subjected to strong earthquakes (5–20). Most of these studies have been performed on open, moment-resistant building frames.

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The use of a light bracing system in fully moment-resistant frames is quite common for the purpose of reducing relative horizontal deflections between floors, thereby reducing the amount of non-structural damage. It is also possible to design economical structures for a multistory building by using a combination of bracing and moment-resistant frame rather than just the frame alone. The use of bracing to resist nonseismic horizontal forces is common. A bracing system is permitted for the design of earthquake resistant buildings by the present Uniform Building Code upon extension of the definition of "shear wall."

An increasing number of braced frame buildings are being constructed in seismically active regions throughout the world. But the studies concerning their elastic or inelastic behavior during strong earthquakes have been very few and recent. Clough and Jenschke (9) studied the elastic behavior of a specific braced structure and Housner's (4) limit analysis was restricted to supporting structures of elevated water tanks or storage bins. More recently, Hanson and Fan (16) studied the inelastic earthquake response of a number of multistory braced frames with minimum cross bracing by assuming the columns to behave elastically as was done by Goel (14) in his study of multistory unbraced frames. The results showed that even a light cross bracing may cause large increases in the column axial forces and questioned using the assumption of elastic column behavior in braced frames.

Workman (17) followed the work of Hanson and Fan (16) and formulated a procedure of analyzing single-bay, multistory frames having light diagonal bracing members when subjected to an earthquake. The basic objective of his investigation was to study the significance of various assumptions used in the formulation of the inelastic dynamic analysis of such frames. He observed that an elastic analysis with or without viscous damping holds little value if a considerable yielding is expected to occur in the structure under a strong earthquake excitation.

Objectives and Scope of the Study

The present investigation reported herein is an attempt to study the inelastic earthquake behavior of a fairly wide variety of multistory frames of steel having different layouts of diagonal bracing and proportioned by two different design philosophies. The structures of the first series are designed by using current design procedures and the second series is a class of period-controlled structures. The method of analysis is a step-by-step numerical procedure and is essentially the one that was developed by Workman (17). Only those assumptions which were found to have significant influence upon the response results are retained.

The structures are modeled as single-bay frames having three distinct types of structural members: (1) girders which are ideal elasto-plastic bending members, (2) columns which are elasto-plastic beam-columns with axial force effect on the plastic moment and the P-Δ effect included, and (3) diagonal bracing members which act as elasto-plastic members in axial tension only. One of the reasons for using light diagonal bracing members is that a simple hysteresis behavior in axial tension can be used by ignoring their strength in compression. Secondly, comparison of the response of a lightly braced structural frame with that of an unbraced frame, when both are subjected to the same earthquake excitation, reveals the effect of the bracing more clearly than would be evident if heavier bracing were employed.

The assumptions and the method of analysis are briefly described in Chapter 2. This chapter also contains the definitions of the various output parameters that were used to characterize the dynamic response of the structures. Chapter 3 describes the layouts and the properties of the frames that were designed for this study, as well as the earthquake accelerogram used in the analyses. Chapter 4, which forms the core of the report, presents the results obtained from the scheduled analyses and their thorough discussion, highlighting the most significant features of the seismic behavior of these structures. The study of the results is aimed at developing an understanding of the inelastic behavior of such braced frame structures during a strong earthquake and deriving implications thay may have bearing on their seismic design. A summary of the study and the significant conclusions derived therefrom are given in the concluding Chapter 5.

Chapter 2
Method of Analysis

2.1 General

The structure is assumed to consist of a series of parallel, identical frames connected at the floor levels by rigid diaphragms. Thus, the lateral response of the entire structure can be obtained by considering one planar frame excited at its base by the horizontal component of the ground motion. The weight of the building is assumed to be concentrated at the floor levels, so that the structure can be treated as a lumped-mass system. The restoring forces are provided by the resistance of the frame members, and some form of damping if assumed for the system.

Once the mathematical model of the structure is formulated, the governing differential equations of motion in matrix form are integrated by a numerical technique using a step-by-step procedure in which the structure is assumed to respond linearly during each time step. However, the member properties may be changed from one time step to the next depending upon their stress condition. In order to perform the enormous volume of computations involved in such a procedure an electronic digital computer is required.

2.2 Method of Analysis

The method of analysis used in this investigation is essentially the same that was developed by Workman (17) for computing the response of a multistory braced frame when subjected to earthquake excitation. The procedure has been modified in this study to include viscous damping in the structures. For details of the procedure reference should be made to his original report.

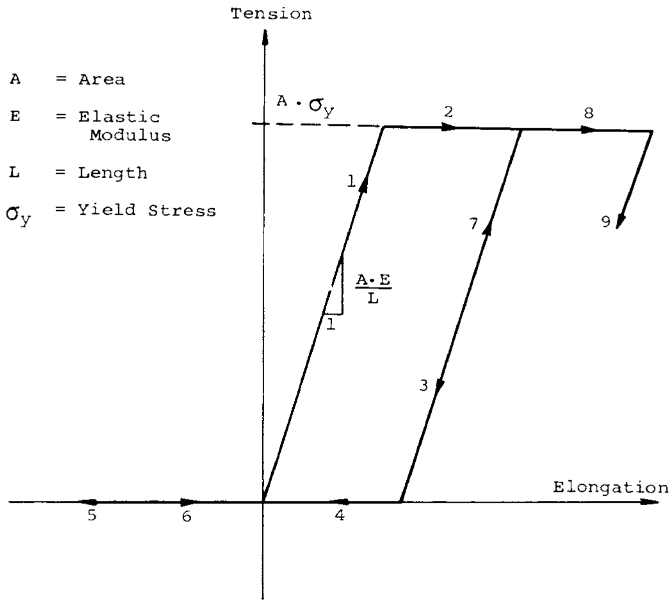


FIGURE 2.1. Force-deformation relation of diagonal cross bracing

However, for the sake of completeness the essential features of the method are described in the following:

To limit the amount of computational effort involved the analysis considers single-bay frames only. These frames are assumed to have three distinct types of structural elements:

(1) Diagonal Cross Bracing

The bracing is a tension carrying member with an elasto-plastic behavior as shown in Figure 2.1. The arrows and their corresponding numbers indicate a typical path that a member might follow while undergoing dynamic motion.

(2) Girders

The girders follow an elasto-plastic hysteresis behavior in moment-curvature terms as is shown in Figure 2.2. The axial forces and corresponding axial deformation are assumed to be negligible. This assumption acknowledges the contribution of the floor system in resisting axial deformation but ignores any contribution of the floor to the flexural stiffness and strength of the girders.

(3) Columns

The columns are also assumed to behave elasto-plastically with axial force effects included in the axial deformation, the lateral stiffness ("P-Δ Effect") and the plastic moment capacity. The moment-curvature relation for the columns is shown in Figure 2.3. The hysteresis behavior is nonlinear in the plastic range because of the modification of the plastic moment by the axial

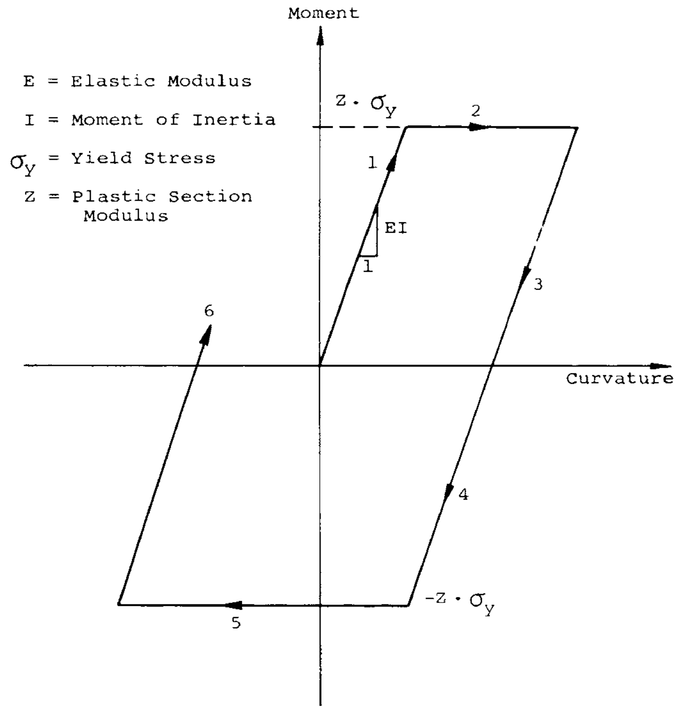


FIGURE 2.2. Girder elasto-plastic moment-curvature relation

force, which is constantly varying during the dynamic motion of the structure. The interaction relationship between the axial force and the plastic moment is the one given by Massonnet and Save (21) for bending of wide-flange sections

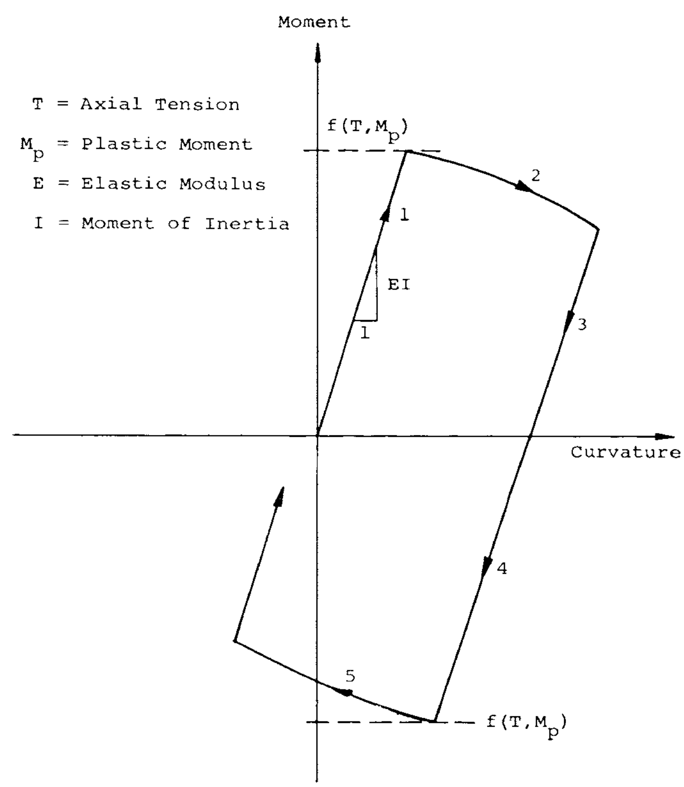


FIGURE 2.3. Column modified elasto-plastic moment-curvature relation

about their strong axis. This is expressed mathematically as:

$$M_{pt} = M_p$$

if

$$|T| < 0.15T_y,$$

and (2.1)

$$M_{pt} = 1.18(1.0 - |T|/T_y)M_p$$

if

$$|T| \geqslant 0.15T_y.$$

where

T = axial load
T_y = axial load at yield ($A\sigma_y$)
A = area of cross-section
σ_y = yield stress
M_p = plastic moment with no axial load
M_{pt} = plastic moment modified due to the axial load

The effect of initial stress condition due to gravity loads is not included in the analysis except for computing the axial forces in the columns. The effect of axial force on the rotational stiffness of the columns is also not considered. This is based on the earlier study by Workman (17).

The nonlinear dynamic response of the frame is computed by a step-by-step numerical technique. The response is assumed to be linear within each time step and the properties of the frame are modified after each step, if needed. Thus, the nonlinear response is obtained as a sequence of linear responses of successively differing systems.

For any time increment the individual member stiffness matrixes, based on the state of stress of the member at the beginning of the time step, is formulated into an overall incremental structural stiffness matrix by the well-known stiffness matrix methods. This structural stiffness matrix will have one lateral degree of freedom per floor with the two nodes at each floor level assumed to have no relative horizontal movement between them. Adding a vertical and a rotational degree of freedom for each node gives a matrix size which is five times the number of floors in the single-bay frame.

It is assumed that the vertical and rotatory inertia forces are zero for the dynamic motion of the frame due to earthquake excitation, leaving only the horizontal inertia forces. By this assumption the transformation of the overall incremental lateral stiffness matrix can be accomplished by standard procedures. This implies that there is static but not dynamic coupling between the horizontal deflections and the vertical deflections and rotations of the nodes.

It is assumed that the mass of the structure is lumped at the floor levels only and that no inertia coupling between the floors exists.

Once the lateral stiffness matrix of the structure is formulated at the beginning of each time step, the numerical integration of the resulting equations of motion within each time step is accomplished by a fourth-order Runge-Kutta procedure.

Viscous damping is incorporated into the solution as a full damping matrix whose values are computed to provide a prescribed percentage of critical damping in each initial elastic mode of vibration of the structure. Such a damping matrix can be obtained from the following matrix equation:

$$[C] = \lceil M \rfloor [\phi][2\beta\rfloor\lceil\omega\rfloor[\phi]^{-1} \qquad (2.2)$$

where

$[C]$ = damping matrix (full)
$\lceil M \rfloor$ = mass matrix (diagonal)
$[\phi]$ = modal matrix (mode shapes arranged column-wise)
$\lceil \beta \rfloor$ = diagonal matrix of percentage of critical damping
$\lceil \omega \rfloor$ = diagonal matrix of natural frequencies

The full damping matrix provides dynamic coupling between all floors of the building, and the damping forces and the dissipated energies through viscous damping are computed incrementally in the Runge-Kutta procedure. At the end of each time step the incremental horizontal displacements are known from the Runge-Kutta integration of the dynamic equations of motion. By using the appropriate matrix transformations, the vertical displacements and rotations of the frame associated with the horizontal displacements can be obtained.

With the total incremental displacement pattern known, the member incremental stress resultants can be computed. These incremental stress resultants are added to the total stress resultants and each member is checked to see if a change in its behavior pattern has occurred. If no changes have occurred, the solution moves ahead to the next time step. If a change is detected, then the process returns to the beginning of the time step, divides the time step by five and calculates the response at this reduced time size until this change is encountered again. Then the original time size is restored and the solution proceeds again with the changed incremental structure. However, a smaller time step had to be used whenever a peak on the accelerogram (which consisted of straight line segments) was encountered within the normal time step. The size of the normal time step in this study was taken equal to one-fifth of the smallest natural period of vibration of the elastic structure. For the cases reported in this study the normal time step ranged from 0.0102 sec to 0.0250 sec.

2.3 Output Parameters

The following parameters were monitored by the program and printed at each accelerogram time: the change in the strain energy, kinetic energy, dissipated energy (through inelastic action and viscous damping), and earthquake input energy from the onset of base motion, the total horizontal displacement of the floor levels, the horizontal restoring forces and the type of behavior experienced by all members. Also at a time interval of 0.5 sec the total maximum or minimum values of certain other parameters were printed. These parameters were: the maximum absolute value of the relative horizontal displacement between floor levels, the maximum absolute value of the floor level horizontal acceleration, the maximum ductility ratios, the maximum column axial force ratios, the energy dissipated by the members and the energy dissipated through viscous damping in each story of the structure.

The ductility ratio represents the amount of maximum fiber strain that a member undergoes during earthquake excitation. Each type of structural element utilizes a slightly different definition of ductility ratio. In the diagonal cross-bracing it is defined as the maximum elongation that the member undergoes during earthquake excitation divided by the yield elongation. This can be expressed mathematically as:

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \tag{2.3}$$

where $\Delta_{\max}$ = maximum elongation and Δ_y, yield elongation = $L\sigma_y/E$ (Fig. 2.1).

A concept derived for a symmetric displacement pattern which has been used earlier (10) has the advantage that the results can be compared to work previously reported (6, 14). For the girders this ductility ratio is defined at each end. When an end moment is in the elastic range the ductility ratio is defined as the ratio of the end moment to the plastic moment of that member. When the end moment is plastic the ductility ratio is defined as one plus the ratio of the maximum plastic hinge rotation for that excursion into the plastic range to the yield rotation for a beam with equal end rotations and no transverse end displacements. This can be expressed as:

For

$$|M| < M_p$$

$$\mu = \left|\frac{M}{M_p}\right|$$

For (2.4)

$$|M| = M_p$$

$$\mu = 1.0 + \frac{6\theta_{\max}EI}{LM_p}$$

where $\theta_{\max}$ = maximum absolute value of the plastic hinge rotation.

The ductility ratio of the columns is defined the same as the girders except the modified plastic moment, M_{pc}, is used in place of the plastic moment, M_p. The modified plastic moment, M_{pc}, is evaluated at each step of the computation and, accordingly, the column ductility ratio is recorded when $|\theta|/M_{pc}$ reaches a maximum value. This is an attempt to take into account the strain caused by the axial load.

The column force ratio is defined as the ratio of the column axial force to the column axial yield force. The column axial yield force is the product of the column area and its yield stress.

The energy dissipated by the members is the total amount of energy dissipated through plastic deformation of each member during the earthquake excitation.

It should be noted that the somewhat arbitrary basis for normalizing the output parameters has no effect on the dynamic response of the structure nor upon the other output parameters.

Chapter 3
Program of Investigation

3.1 General

The dual concepts of structural ductility and energy absorption capacity necessary for a structure to survive a severe earthquake are important to adequate, economical design. The ductility ratio is a measure of maximum inelastic deformation requirements in seismic design. However, seismic adequacy depends on more than satisfying ductility requirements. During a severe earthquake the structural members must be able to withstand numerous inelastic cycles of load reversal. That is to say, they must provide adequate energy absorption capacity. Consideration of only one of these concepts without the other represents an incomplete approach to seismic design.

This study provides information on both ductility and energy absorption requirements for lightly braced moment resisting frames.

Minimum cross bracing would normally be used to control lateral displacements of the building under wind and moderate earthquake excitation and to supply additional source of energy dissipation during a severe earthquake. With the use of minimum cross bracing their strength in compression can be ignored which results in a simpler hysteresis behavior in tension only for these members. A comparison of the response of lightly braced frames to the response of a comparable unbraced frame also provides a basis for expanding our design intuition.

This chapter describes the types of structural models and their variations, and the earthquake accelerogram used in this study. The structural models represent different design philosophies and different arrangements of diagonal bracing. A form of viscous damping was also introduced in some structural models to study its effect on their elastic and inelastic response to the same earthquake.

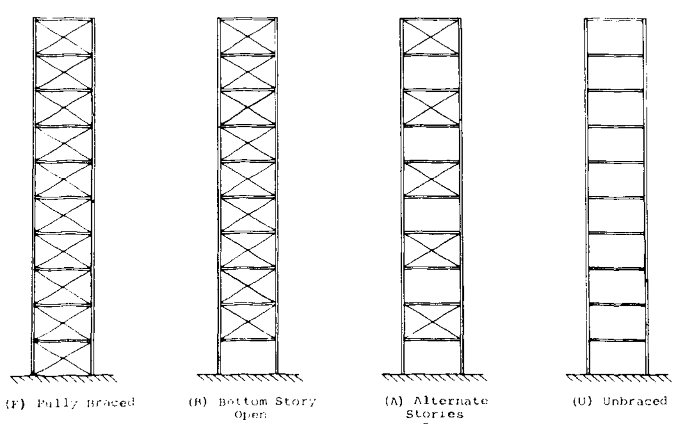


FIGURE 3.1. Types of structures

3.2 Structures Considered

This study deals with regular, rectangular buildings only. Any irregularities such as setbacks, eccentricities or appendages were not considered. The structures were 10 stories high with a uniform story height of 12 ft, bay width of 20 ft and fixed at the base. Only diagonal bracing was used with four different layouts:

(a)	Fully braced	(F)
(b)	Bottom story open	(B)
(c)	Alternate stories open	(A)
(d)	Completely unbraced	(U)

These patterns of bracing arrangement and their designations are shown in Figure 3.1.

Two different philosophies were used to design these frames. In Series One the structures were designed for minimum code lateral forces and using current allowable stress design procedures. The structures of Series Two were period controlled and were proportioned for a constant fundamental period of about 1.25 sec. The design procedures for the two series are further described in the following sections.

3.2.1 Series One

In the present state-of-the-art the primary design factors are the stresses or forces in the members and the lateral deflection of the structure under the combined dead, live and lateral earthquake or wind loads. The four 10-story structures of Series One were designed using typical dead and live loads for multistory office buildings and lateral earthquake forces similar to those specified by the Uniform Building Code 1970.

Two design criteria, member stress and a lateral deflection limit of 0.35% of the height at design earthquake loads were used. The lateral earthquake forces are based on a dead load per floor of 44 kips, $C = 0.05$ regardless of period, $K = 1.0$ regardless of the frame layout, and $J = 1.0$. This is an extremely crude but conservative estimate of the code lateral forces. The frames F1 and B1 were considered to carry three bays of lateral force and one bay of vertical load. The moment-resisting frame consisting of beams and columns for these two structures was designed for 25% of the total lateral load while the complete system carried 100% of the loads. Frames U1 and A1 carried one bay of lateral and vertical forces. The lateral forces which were used in the design of these frames are given in Table 3.1. Column stresses were calculated using AISC Formula (1.6-1a) with $C_m = 0.85$ for combined dead, live and lateral loads.* It should be noted that the use of $C_m = 0.85$ in the design of columns for frames F1 and B1 is on the conservative side. The live load was taken as 32 kips per floor with no reduction. In the design of beams and columns under combined gravity and earthquake forces allowable stresses were increased by 33% as permitted by the AISC Specifications. The first floor beam and first story columns of frame B1 were also selected so that the first story stiffness of B1 is approximately the same as in frame F1. The diagonal braces were designed as tension members using an allowable stress of 22 ksi without utilizing the 33% increase. A minimum area of 2.88 sq in. was used for the bracing members, which was determined from the maximum slenderness ratio requirement of 300. A36 steel was assumed for these frames.

The lateral deflections of the structures were found to be within the 0.35% limit without influencing the designs. The properties and design values are given in Tables 3.2–3.5. Even though the column stresses as calculated from the AISC Formula (1.6-1a) are somewhat larger than those which would perhaps be used in normal design practice, it is felt that these designs are reasonably conservative. No reduction in the live loads and the use of $C = 0.05$, $K = 1.0$ and $J = 1.0$ all lead to an over-estimation of the design forces. For example, the code values of C determined from the calculated periods of vibration for these frames would be at least 10% smaller than the value used.

In the structures F1, B1 and A1 of Series One the bracing was designed to provide strength against full lateral design forces. The lateral deflections of these structures at these design forces were found to be within permissible limits. But the completely unbraced frame U1 was found to be the most flexible structure in this series, showing the largest lateral drift at design forces. Minimum cross bracing (area of cross section = 2.88 sq in. and slenderness ratio of about 300) was added to this frame for the purpose of controlling lateral drift at design loads. This is quite often done in practice under similar circumstances. This frame is designated U1X and represents a different design concept than the previous four structures in this series. The structural properties of this frame are given in Table 3.6. In the frame U1X, therefore, the cross bracing provides extra strength than that required for the code lateral forces and an additional source of energy absorption during a severe earthquake. A comparison of the response of this structure with the other four models in Series One (U1 in particular) would be of interest to study the desirability of such a design procedure and the effect that the additional cross bracing would have on the response of the original unbraced structure.

The fundamental period of vibration of each of these frames is also listed in Tables 3.2–3.6. For computing the natural periods the frames were treated as elastic and the stiffness of only one diagonal member in each braced story was considered. The masses assigned to the floors were those corresponding to the weights used in calculating the design lateral forces. Thus, the floor masses in frames A1, U1 and U1X are for one bay of dead loads (44 kips per floor) and those in frames F1 and B1 for three bays, i.e., 132 kips per floor. The same floor masses were also used in computing the dynamic response of these structures.

TABLE 3.1 Distribution of Lateral Force for Series One Design (in Kips)

Floor	*U and A*	*F and B*
10	6.70	20.10
9	3.06	9.20
8	2.72	8.15
7	2.38	7.14
6	2.04	6.11
5	1.70	5.10
4	1.36	4.08
3	1.02	3.06
2	0.68	2.04
1	0.34	1.02
Total	22.00	66.00

These values are calculated using a value of $C = 0.05$. It should be noted that $K = 1.0$ was used for all cases although case U would have permitted a $K = 0.67$.

* AISC Formula (1.6-1a), 1969 Specifications:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e'}\right) F_b} \leqslant 1.0$$

TABLE 3.2 Structural Properties of Frame F1

Floor Level	Girders Section	Girders I_x	Girders Bracing A	Columns Section	Columns I_x	Columns A	Columns D.L. + L.L. + E.Q. Formula (1.6-1a)
10	W18×50	802	2.88	W14×34	340	10.0	1.41
9	W18×50	802	2.88	W14×53	542	15.6	1.08
8	W18×50	802	2.88	W14×53	542	15.6	1.28
7	W18×50	802	2.88	W14×78	851	22.9	1.03
6	W18×50	802	2.88	W14×78	851	22.9	1.23
5	W18×60	986	2.88	W14×103	1170	30.3	1.02
4	W18×60	986	2.94	W14×103	1170	30.3	1.18
3	W18×60	986	2.94	W14×119	1370	35.0	1.29
2	W18×60	986	3.38	W14×119	1370	35.0	1.30
1	W18×60	986	3.38	W14×136	1590	40.0	1.33
							Allowable = 1.33

Girders D.L. + L.L. + E.Q.: $f_b/22 = 0.982$ maximum.
The fundamental period of vibration, $T_1 = 1.58$ sec.

TABLE 3.3 Structural Properties of Frame B1

Floor Level	Girders Section	Girders I_x	Girders Bracing A	Columns Section	Columns I_x	Columns A	Columns D.L. + L.L. + E.Q. Formula (1.6-1a)
10	W18×50	802	2.88	W14×34	340	10.0	1.41
9	W18×50	802	2.88	W14×53	542	15.6	1.08
8	W18×50	802	2.88	W14×53	542	15.6	1.34
7	W18×50	802	2.88	W14×78	851	22.9	1.07
6	W18×50	802	2.88	W14×78	851	22.9	1.25
5	W18×60	986	2.88	W14×103	1170	30.3	1.06
4	W18×60	986	2.94	W14×103	1170	30.3	1.20
3	W18×60	986	2.94	W14×119	1370	35.0	1.22
2	W18×60	986	3.38	W14×119	1370	35.0	1.27
1	W18×77	1290	0.0	W14×184	2270	54.1	1.33
							Allowable = 1.33

Girders D.L. + L.L. + E.Q.: $f_b/22 = 1.03$ maximum.
The fundamental period of vibration, $T_1 = 1.84$ sec.

TABLE 3.4 Structural Properties of Frame A1

Floor Level	Girders Section	Girders I_x	Girders Bracing A	Columns Section	Columns I_x	Columns A	Columns D.L. + L.L. + E.Q. Formula (1.6-1a)
10	W18×45	706	2.88	W14×34	340	10.0	1.32
9	W18×45	706	0.0	W14×43	429	12.6	1.26
8	W18×45	706	2.88	W14×43	429	12.6	1.29
7	W18×45	706	0.0	W14×61	641	17.9	1.29
6	W18×45	706	2.88	W14×61	641	17.9	1.20
5	W18×50	802	0.0	W14×74	797	21.8	1.36
4	W18×50	802	2.88	W14×74	797	21.8	1.20
3	W18×50	802	0.0	W14×87	967	25.6	1.32
2	W18×50	802	2.88	W14×87	967	25.6	1.34
1	W18×50	802	0.0	W14×103	1170	30.3	1.36
							Allowable = 1.33

Girders D.L. + L.L. + E.Q.: $f_b/22 = 1.14$ maximum.
The fundamental period of vibration, $T_1 = 1.50$ sec.

TABLE 3.5 Structural Properties of Frame U1

Floor Level	Girders		Bracing A	Columns			
	Section	I_x		*Section*	I_x	A	*D.L. + L.L. + E.Q. Formula (1.6-1a)*
10	W18×50	802	0.0	W14×34	340	10.0	1.38
9	W18×50	802	0.0	W14×53	542	15.6	
8	W18×50	802	0.0	W14×53	542	15.6	1.28
7	W18×50	802	0.0	W14×68	724	20.0	
6	W18×50	802	0.0	W14×68	724	20.0	1.30
5	W18×60	986	0.0	W14×87	967	25.6	
4	W18×60	986	0.0	W14×87	967	25.6	1.23
3	W18×60	986	0.0	W14×103	1170	30.3	
2	W18×60	986	0.0	W14×103	1170	30.3	1.32
1	W18×60	986	0.0	W14×119	1370	35.0	1.24
							Allowable = 1.33

Girders D.L. + L.L. + E.Q.: $f_b/22$ = 1.20 maximum.
The fundamental period of vibration, T_1 = 2.03 sec.

TABLE 3.6 Structural Properties of Frame U1X

Floor Level	Girders		Bracing A	Columns		
	Section	I_x		*Section*	I_x	A
10	W18×50	802	2.88	W14×34	340	10.0
9	W18×50	802	2.88	W14×53	542	15.6
8	W18×50	802	2.88	W14×53	542	15.6
7	W18×50	802	2.88	W14×68	724	20.0
6	W18×50	802	2.88	W14×68	724	20.0
5	W18×60	986	2.88	W14×87	967	25.6
4	W18×60	986	2.88	W14×87	967	25.6
3	W18×60	986	2.88	W14×103	1170	30.3
2	W18×60	986	2.88	W14×103	1170	30.3
1	W18×60	986	2.88	W14×119	1370	35.0

The fundamental period of vibration, T_1 = 1.10 sec.

TABLE 3.7 Structural Properties of Frame U2

Floor Level	Columns			Girders		
	Section	I_x	A	*Section*	I_x	*Floor Weight*
10	W14×78	851	22.9	W18×96	1680	45.45
9	W14×78	851	22.9	W18×96	1680	45.45
8	W14×127	1480	37.3	W18×96	1680	45.45
7	W14×127	1480	37.3	W18×96	1680	45.45
6	W14×176	2150	51.7	W18×96	1680	45.45
5	W14×176	2150	51.7	W21×127	3020	45.45
4	W14×219	2800	64.4	W21×127	3020	45.45
3	W14×219	2800	64.4	W21×127	3020	45.45
2	W14×264	3530	77.6	W21×127	3020	45.45
1	W14×264	3530	77.6	W21×127	3020	45.45

The fundamental period of vibration, T_1 = 1.25 sec.

3.2.2 Series Two

The structures of Series One were designed according to the present state-of-the-art using the design forces and procedure close to those specified by typical codes in practice. It is noticed that this resulted in different periods of vibration for structures of similar dimensions but different layouts. It has been shown in some earlier investigations (14) that the period of vibration of a structure may significantly influence its response to a given earthquake. In fact, a comparison of the response of structures with different periods of vibration to the same earthquake is often complicated because of this reason. It was, therefore, decided for this investigation to design a second series of 10-story structures which would have a common fundamental period of vibration irrespective of their strength.

The basic structure in Series Two is a fully unbraced 10-story structure, with a fundamental period of vibration of 1.25 sec, which was used by Goel in an earlier study (14). This structure is designated as U2 and its structural properties are given in Table 3.7. The braced models in this series (designated as F2, B2 and A2) were obtained by adding minimum cross bracing to the frame U2 according to the layout which was shown in Figure 3.1. The slenderness ratio of these members was about 300 (the maximum permitted by AISC Specifications for a secondary tension member) and their area of cross section 2.88 sq in. The floor mass of these frames was then adjusted to maintain a fundamental period of about 1.25 sec for each. This could be considered the same as having different amounts of floor area contributing their inertia forces to one braced bay. That is, probably each bay would not be braced. The structural properties of the frames F2, B2 and A2 are given in Table 3.8.

Thus, the structures of Series Two have identical beam-column frames, identical bracing members but with different distribution, and different floor masses corresponding to a common fundamental period of vibration of 1.25 sec. This represents a rather unconventional design procedure. However, it would be interesting to study their behavior under the same input earthquake and cross-compare with the corresponding response of the structures of Series One.

3.3 Earthquake Accelerogram

The accelerogram used in this investigation is the N-S component of the May 18, 1940, El Centro earthquake with the ground acceleration values multiplied by a factor of 1.5, and subsequently referred to as the modified El Centro accelerogram. This modified accelerogram has been used by a number of investigators in the past (14, 15, 17). It is believed to be representative of a severe earthquake for structures founded on hard ground in the western part of the United States. The velocity response spectrum of this accelerogram is relatively flat over a wide range of the period of vibration of structures, i.e., approximately 0.8–3.0 sec (14). This has certain advantages when interpreting dynamic response.

Studies by Clough and Benuska (11) suggest that the structural response depends primarily on the peak acceleration impulse in the ground motion and that continuing motions of smaller amplitude have only a small effect on the maximum response. Therefore, in order to avoid excessive computational time, the duration of the earthquake used in most of their analyses was primarily limited to the first four or eight seconds of the El Centro earthquake.

Goel (14) and Workman (17) noted in their analyses that most of the maximum response

TABLE 3.8 Additional Structural Properties of Series Two Braced Frame Types F2, B2, A2

Floor Level	*Frame F2*		*Frame B2*		*Frame A2*	
	Bracing A	*Floor Weight*	*Bracing A*	*Floor Weight*	*Bracing A*	*Floor Weight*
10	2.88	93.82	2.88	90.00	2.88	67.40
9	2.88	93.82	2.88	90.00	0.0	67.40
8	2.88	93.82	2.88	90.00	2.88	67.40
7	2.88	93.82	2.88	90.00	0.0	67.40
6	2.88	93.82	2.88	90.00	2.88	67.40
5	2.88	93.82	2.88	90.00	0.0	67.40
4	2.88	93.82	2.88	90.00	2.88	67.40
3	2.88	93.82	2.88	90.00	0.0	67.40
2	2.88	93.82	2.88	90.00	2.88	67.40
1	2.88	93.82	0.0	90.00	0.0	67.40

The fundamental period of vibration, T_1 = 1.25 sec.

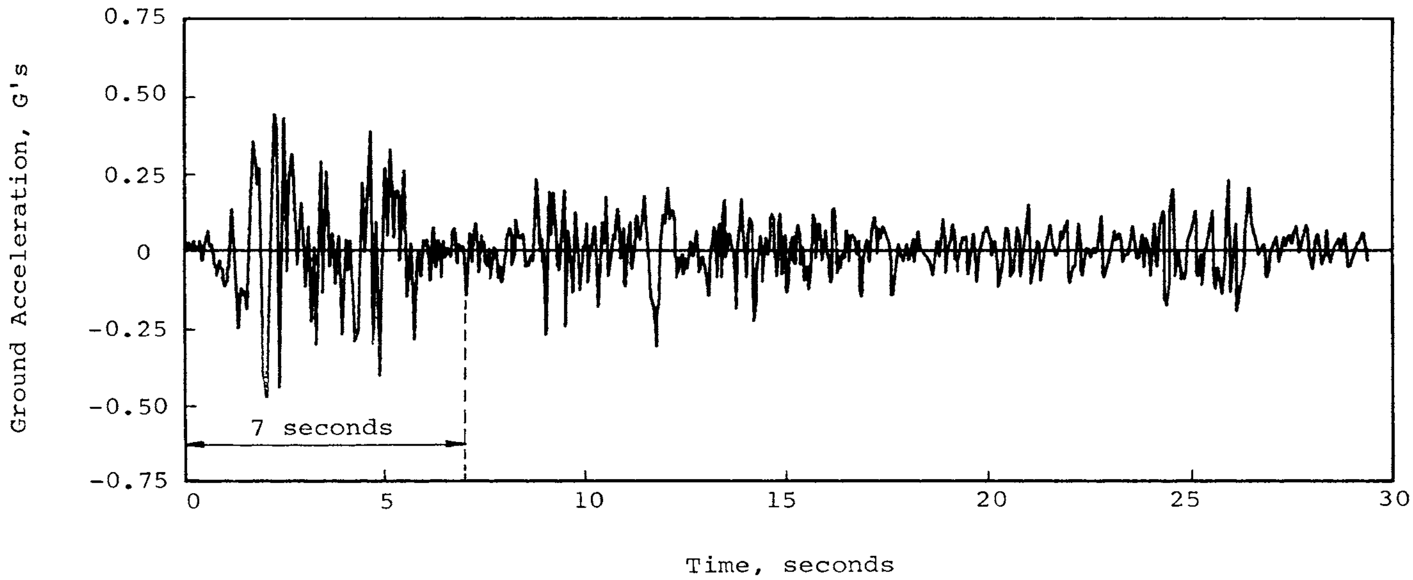


FIGURE 3.2. Modified El Centro accelerogram

parameters of their structures using the modified El Centro accelerogram occurred within the first seven seconds of the motion. The rate of input energy with time was noted to be very fast with this accelerogram so that more than 75% of the maximum input energy came from the first seven seconds of the accelerogram. It was, therefore, decided to use the first seven seconds of this modified El Centro accelerogram for most of the analyses performed for this investigation. In some cases, where the cost of computation was not excessive, analyses were run for a longer duration but the maximum response parameters were picked from the first seven seconds only. This modified accelerogram and its velocity response spectra computed for the first seven seconds of ground

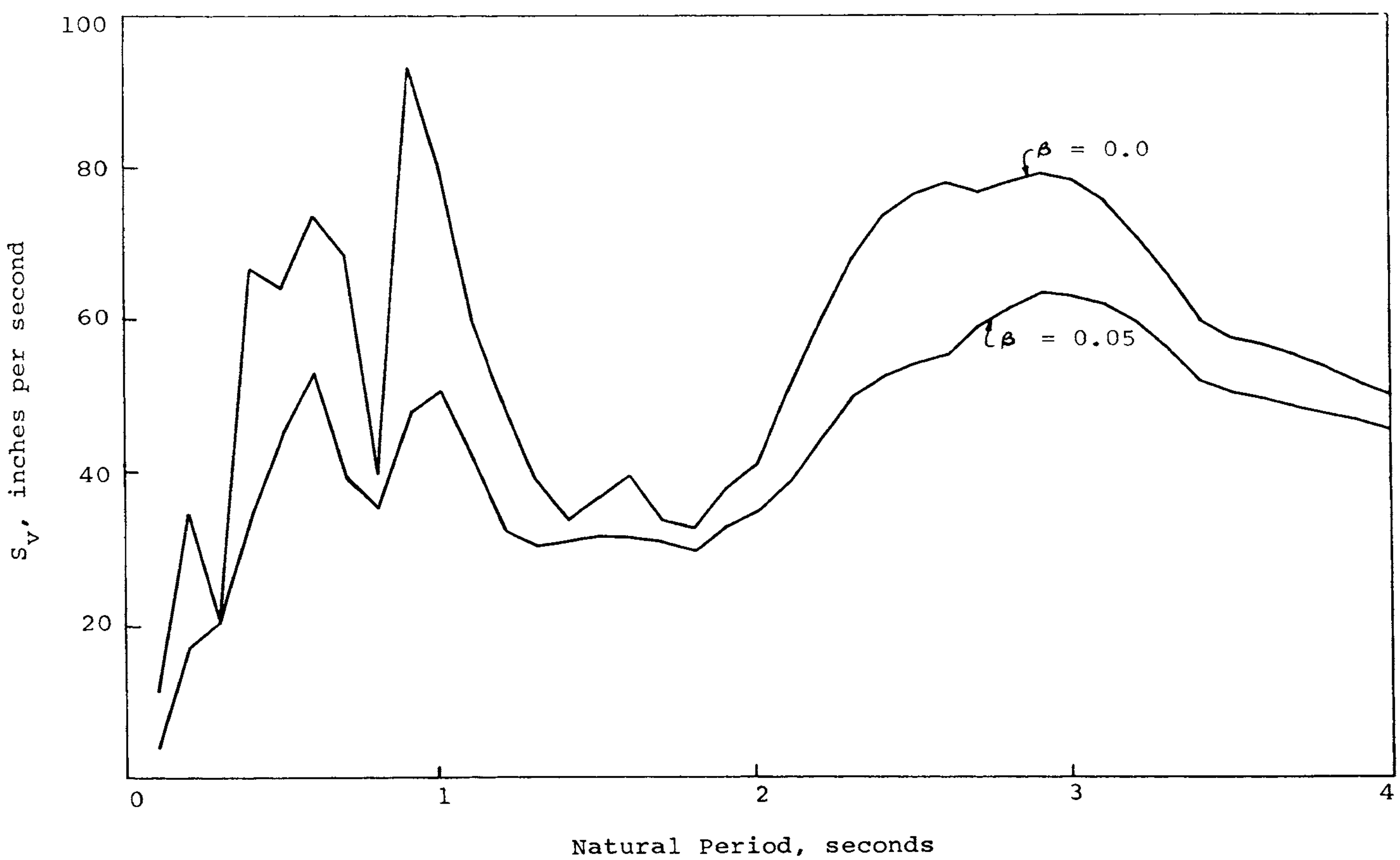


FIGURE 3.3. Velocity spectra for the first seven seconds of the modified El Centro accelerogram

motion are shown in Figures 3.2 and 3.3, respectively.

3.4 Structural Behavior

It is noted that structures of varying layouts of diagonal bracing, ranging from fully braced to fully unbraced, were designed by two different design concepts for this study. This was done to study the effect of light bracing on the inelastic behavior of these structures during a severe earthquake. Also, the characteristic features of the response of these structures proportioned by different procedures will be studied.

The structural models of the two series were also analyzed under varying assumptions of the analysis such as the inelastic or elastic behavior of the structural members, and the addition of viscous damping equivalent to 5% of critical in each elastic mode of the structure. The details of the various types of analysis thus formulated are described in Chapter 4. Comparisons of the response of these various structures under the varying assumptions when subjected to the same input earthquake will provide a basis for studying the influence of energy absorption through inelastic deformation, viscous damping and the combination of both types.

Chapter 4
Discussion of Results

4.1 General

The results obtained from the analyses described earlier are presented and discussed in this chapter. The schedule of analyses was planned to study the following effects:

1. **Type of Structure**

Different variations of the arrangement of diagonal braces were obtained both in Series One and Series Two frames. These structures represent different brace arrangements. Maximum response parameters of these structures with no damping as obtained from the inelastic analysis are compared.

2. **Type of Analysis**

The structural models were also varied depending upon whether the frame members were permitted to behave elastically or inelastically (elasto-plastic hysteresis behavior) and whether some viscous damping was included in the analysis or not. Each structure of Series One and Series Two was analyzed with these different assumptions of structural behavior. Comparison of the results is made with a view to evaluate the influence of hysteresis and viscous energy dissipation on the response of each structure studied. The comparison of the results of inelastic analysis without viscous damping and elastic analysis with viscous damping (5% of critical in each mode) would also indicate whether the inelastic behavior of these structures can be effectively modeled by an elastic structure with an equivalent viscous damping.

For the purpose of this study the comparison of dynamic response of the various cases is made in terms of maximum response parameters occurring in the first seven seconds of the earthquake. To avoid excessive repetition of the word maximum, the maximum response parameters will generally be called as simply response parameters in the discussion to follow.

4.2 Type of Structure

4.2.1 Series One

The frames of Series One, namely F1, B1, A1, U1 and U1X, were subjected to inelastic analysis without any viscous damping. Thus, the only source of energy dissipation in these models was the hysteresis energy through inelastic deformation of the structural members. The response parameters of these five frames from such an analysis are compared in this section. This comparison is intended to evaluate the influence of structural variations upon their inelastic response when the frames are subjected to the same earthquake. The results are presented in Figures 4.1(a)–(k) and discussed below.

The floor displacements of the five frames are plotted in Figure 4.1(a). It will be noticed that the floor displacements of A1 and U1X are the least while U1 shows the largest floor displacements. The displacements of F1 and B1 lie in between the above extremes. It is quite clear that the totally unbraced frame U1 is the most flexible one showing displacements which are largest of all the five frames in this series, including F1 and B1

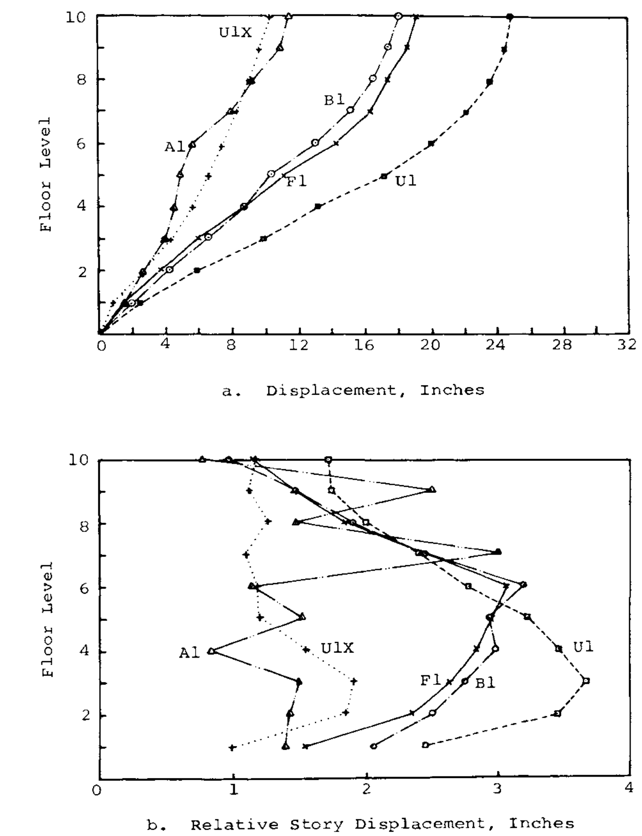


FIGURE 4.1. Series One—inelastic undamped analysis

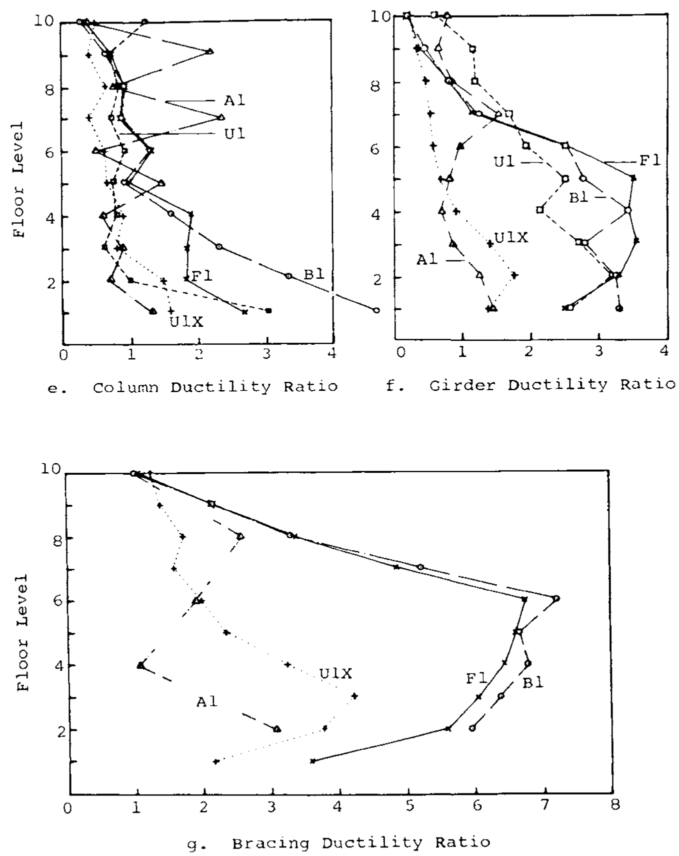


FIGURE 4.1. Series One—inelastic undamped analysis

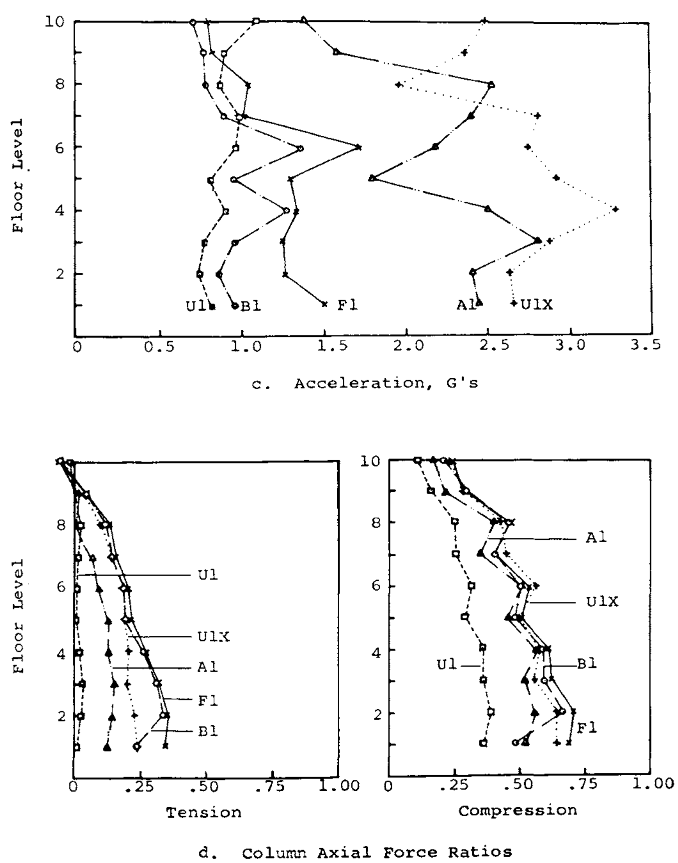


FIGURE 4.1. Series One—inelastic undamped analysis

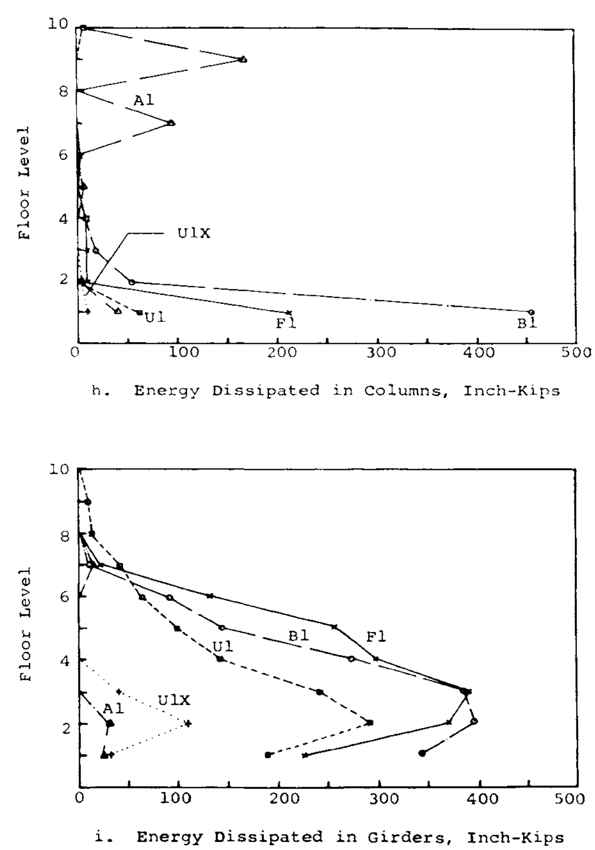
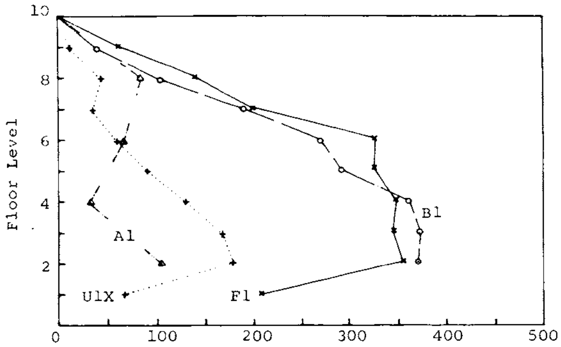
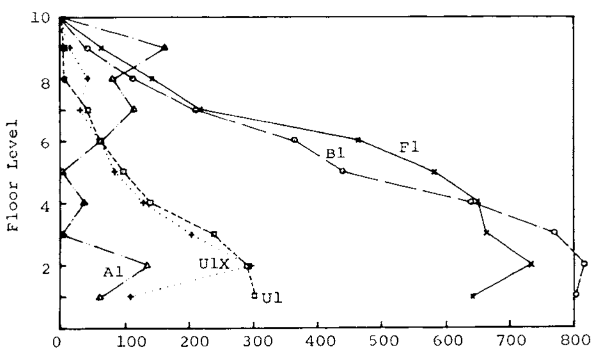


FIGURE 4.1. Series One—inelastic undamped analysis



j. Energy Dissipated in Bracing, Inch-Kips



k. Energy Dissipated at Each Floor, Inch-Kips

FIGURE 4.1. Series One—inelastic undamped analysis

which carried three times as much floor mass. Addition of minimum cross-bracing to the frame U1 for drift control proves very effective as can be noticed from a substantial reduction of floor displacements in the frame U1X.

Similar observations can be made about relative story displacements which are plotted in Figure 4.1(b). The unbraced frame U1 shows the largest relative story displacements which were reduced by more than 50% in the lower stories due to the addition of minimum cross bracing (Frame U1X). Frames F1 and B1 have relative story displacements comparable to those of U1 even though the former have three times the floor masses of the unbraced frame U1. The presence of diagonal bracing in these frames showed remarkable control over lateral displacements. For example, the alternate unbraced stories of frame A1 showed markedly larger relative story displacements as compared with the braced stories.

The normalized floor accelerations are plotted in Figure 4.1(c). The presence of cross bracing results in a large increase in the floor accelerations. It can be noticed that the frame U1 had least floor accelerations and the addition of minimum cross bracing to it resulted in a 3 to 4 times increase in the acceleration values. It is interesting to note that the accelerations in frames F1 and B1 are smaller than those in U1X or A1. A logical explanation for this would be that the frames F1 and B1 carry three times the floor mass compared with the other three frames of this series and since the diagonal braces yield at fairly small amplitude level this renders F1 and B1 as relatively more flexible structures for the most part of their dynamic response. Large floor accelerations in frame A1 may also be considered as partly due to a significant second mode contribution to the early response of A1 but not for F1 for instance (Figs. B.2 and C.2 in the Appendixes).

Column axial force ratios (maximum force of tension and compression divided by the yield force T_y) are shown in Figure 4.1(d). As could be expected, the effect of diagonal bracing in these frames resulted in a considerable increase in the column axial forces. The axial forces in the columns of the unbraced frame U1 are smaller than any of the braced frames in this series. It should be remembered that the combined dead load plus earthquake forces are used in computing the axial force ratio.

Column ductility ratios are shown in Figure 4.1(e). The results do not show a well-defined trend. However, it can be noticed that the columns of the unbraced frame U1 generally remained elastic except in the first and the top stories. Other braced frames show considerable inelastic activity in the columns, except U1X in which the presence of cross bracing resulted in reduced column ductility ratios as compared to those in U1.

Figure 4.1(f) shows a comparison of the girder ductility ratios for the five frames of Series One. Comparing the girder ductility ratios for frames U1 and U1X, it can be seen that the addition of minimum cross bracing to the unbraced frame U1 results in a substantial reduction in the inelastic activity in the girders. As a matter of fact, the girders of U1X generally remain elastic except for a few lower stories. The girder ductility ratios for frames F1 and B1 are about the same as in U1 even though the former have three times as much floor mass.

The ductility ratios for the braces in the braced frames are presented in Figure 4.1(g). The braces in F1 and B1 have ductility ratios as large as 7. These results indicate that the diagonal braces as used in these frames have to undergo quite large deformation beyond the yield point.

The next four figures, Figures 4.1(h)–(k), show energy dissipated by the columns, girders, braces and the total energy dissipated per story in these frames. It will be noticed that the energy demand on the columns is not as great as may have been suggested by quite substantial column ductility ratios in Figure 4.1(e). The girders and braces share most of the energy dissipation almost equally. Frames U1 and A1 are exceptions to the above observation. In U1 there are no braces so the girders dissipate most of the total dissipated energy in the frame. On the contrary to this the columns in the alternately braced frame A1 dissipate more energy than the girders and braces combined. The total energy dissipated in frame F1 or B1 is the largest of the other three perhaps because of larger floor mass of these two structures.

In the preceding paragraphs the dynamic response of the undamped models of the five structures of Series One when subjected to 1.5 times the N-S component of the El Centro 1940 accelerogram was presented and discussed. It could be noticed from the results that the frames F1, B1 and U1 are relatively flexible structures showing large lateral displacements, member ductility ratios and energy demands. The ductility ratios for the bracing members in frames F1 and B1, for example, are as large as 7. By comparing the results of the braced frames with the corresponding unbraced model the effect of bracing on the response parameters was also studied. It was noticed that one effect of bracing is to increase the axial forces in the columns (T/T_y values showing as high as 0.7). The addition of minimum cross-bracing for drift control (as in frame U1X) does serve the intended purpose but at the expense of substantial increase in column axial forces and the floor accelerations reaching as high as 3.5 g.

4.2.2 Series Two

Whereas the frames of Series One were designed using current design procedures and the minimum design lateral forces recommended by the Uniform Building Code 1967, the structural models of Series Two were obtained from a different design philosophy. The structures in this series were obtained by adding minimum cross bracing (slenderness ratio about 300) to an unbraced structure U2 having a fundamental period of 1.25 sec. The beams and columns were identical in all the frames in this series. To counteract the stiffening effect of the bracing members in frames F2, B2 and A2 their floor masses were adjusted to maintain a fundamental period of about 1.25 sec in each case. Thus, this series represents a class of structures which are period controlled. They are stiffer than those of Series One except for the special case U1X.

The maximum response of the four models of Series Two (F2, B2, A2 and U2) is presented in Figures 4.2(a)–(k). A striking feature of these results is that there is not as much variation in the response of different models in this series as was observed in the case of Series One. This may be partly due to the constant fundamental period of vibration of the Series Two structures and partly because of stronger beams and columns in these structures as compared to those in Series One.

It can be noticed, however, that the cross bracing does tend to limit the lateral floor displacements and the inelastic activity in the girders [Figs. 4.2(a), (b), (f) and (i)]. The floor accelerations and column axial forces are considerably larger in frames F2, B2 and A2 as compared with the unbraced model U2 [Figs. 4.2(c) and 4.2(d)]. The same effects were also observed to a relatively greater degree in the response of Series One structures. The columns of all these four frames show very little inelastic activity and generally remain elastic except in the top few unbraced stories of A2 and U2 [Figs. 4.2(e) and 4.2(h)]. The ductility ratio of the girders generally ranges between 1 and 2 and that for the bracing members between 1 and 4 [Figs. 4.2(f) and 4.2(g)]. These are about half of the corresponding values in Series One frames.

4.2.3 Series One vs. Series Two

In the preceding two sections the inelastic undamped response of the structural models of Series One and Series Two subjected to 1.5 times the El Centro 1940 accelerogram was presented. The structures in these two series represent two different design philosophies. The results as presented were aimed to study the seismic response of the braced frames having different arrangements of diagonal bracing. A cross comparison between the results of the two series would also indicate the basic similarities and differences between their response to the same input earthquake.

The frames U1, A1, B1 and F1 proved to be rather flexible structures showing large lateral displacements (up to 26 in. at the top 10th floor) and ductility requirements for the bracing members as high as about 7. These are braced or unbraced

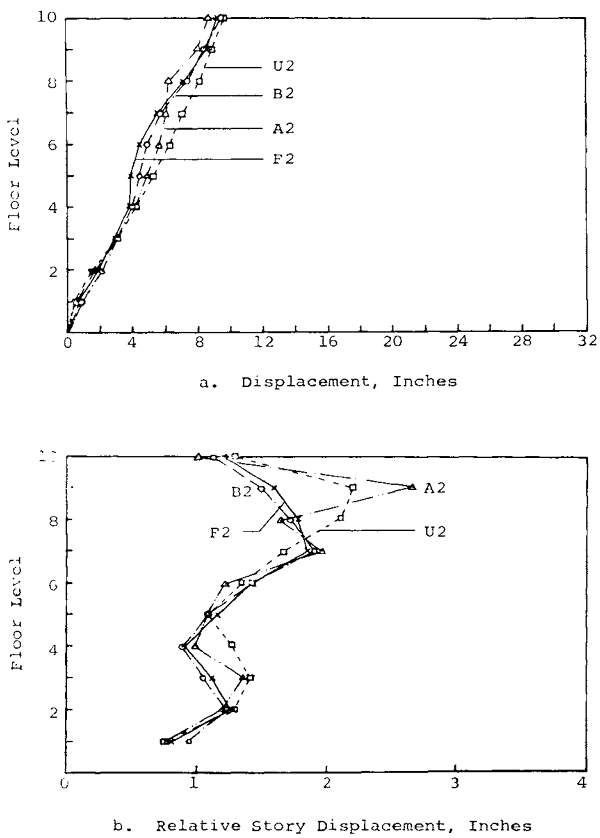


FIGURE 4.2. Series Two—inelastic undamped analysis

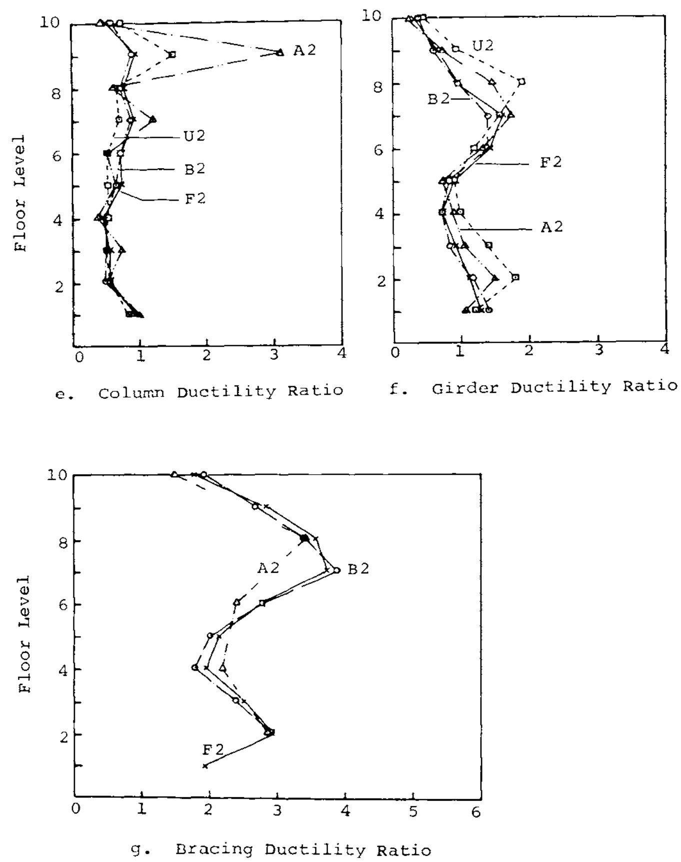


FIGURE 4.2. Series Two—inelastic undamped analysis

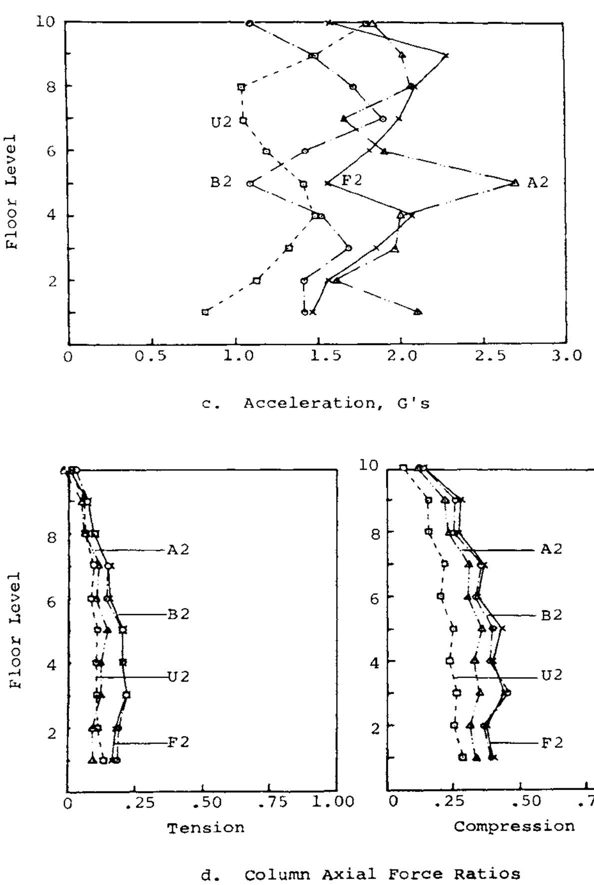


FIGURE 4.2. Series Two—inelastic undamped analysis

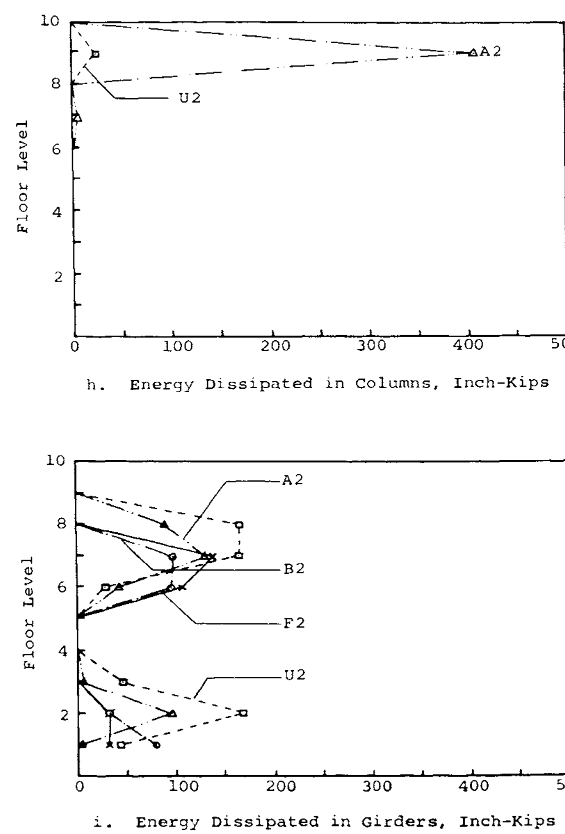
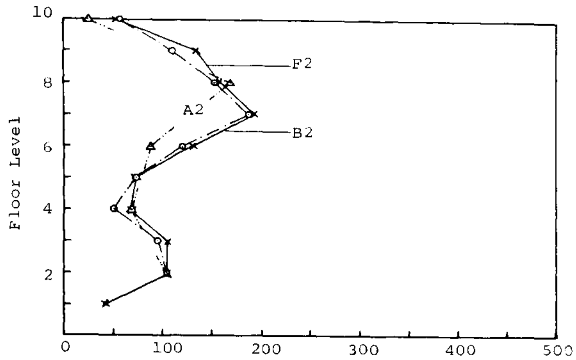
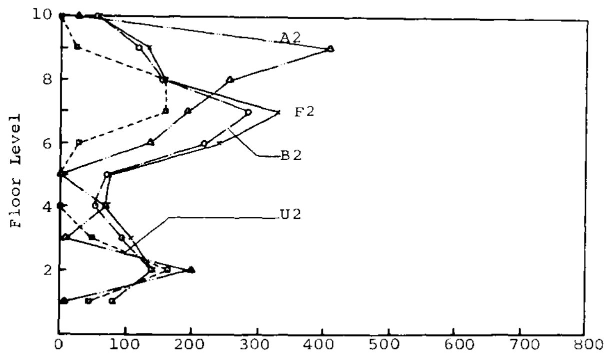


FIGURE 4.2. Series Two—inelastic undamped analysis



j. Energy Dissipated in Bracing, Inch-Kips



k. Energy Dissipated at Each Floor, Inch-Kips

FIGURE 4.2. Series Two—inelastic undamped analysis

structures designed for minimum code lateral forces. There is considerable inelastic activity in the structural members including the columns. The completely unbraced frame U1 showed the largest lateral displacements but least floor accelerations and column axial forces. The cross bracing in the other three braced models does tend to reduce the lateral displacements to a certain extent but the floor accelerations and axial forces in the columns were increased. An alternative design of adding minimum cross bracing to the unbraced frame U1 for drift control, the U1X frame, appears to behave better in that the lateral displacements and member ductilities were substantially reduced. But in this frame also the diagonal braces resulted in increased column axial forces and floor accelerations.

The four models in Series Two are controlled by their period rather than their strength and all have a common period of about 1.25 sec. These are stiffer structures than those of Series One and appear to perform better than the four basic structures of that series, i.e., U1, A1, B1 or F1. The lateral displacements and member ductilities are approximately half of those in the above four structures of Series One and the columns of Series Two frames generally remained elastic. It was noticed that in Series Two also the unbraced model U2 had least column axial forces and floor acceleration as compared to any of the other braced models in that series. It can almost be concluded that the diagonal braces do have a tendency to produce larger column axial forces and floor accelerations in such structures.

4.3 Type of Analysis

For the purpose of computing the dynamic response of a structure for a given ground motion, after a mathematical model of the structure is formulated, two important questions have to be resolved—representation of the hysteresis behavior of structural components and the nonstructural damping in the system. Both these factors have been known to have significant effect on the dynamic response of the structural systems but little experimental data is available regarding the actual inelastic hysteresis behavior of various types of structural members (20) and the magnitude and nature of damping present in real building structures.

A conventional elasto-plastic hysteresis model has been most commonly used by investigators to study the inelastic dynamic behavior of structures. This also has been used in the present investigation. To represent the damping used in most elastic response computations a viscous damping of 5% of critical in each elastic mode has been used in these analyses. In order to evaluate the effect of hysteresis energy dissipation through inelastic deformation of structural members and the energy dissipation through viscous damping on the response of structural models used in this study four different types of analyses were performed. These are given below along with the abbreviations which will be used in the discussion to follow:

1. Inelastic without viscous damping (IU)
2. Inelastic with 5% viscous damping (ID)
3. Elastic with 5% viscous damping (ED)
4. Elastic with no damping (EU)

In the current design practice an elastic analysis with some percentage of critical viscous damping in each mode (usually 5%) is generally performed to predict the anticipated response of a structure for a prescribed ground motion. A viscous damping such as above with an elastic analysis is considered to represent the effect of both hysteretic and nonhysteretic damping energy dissi-

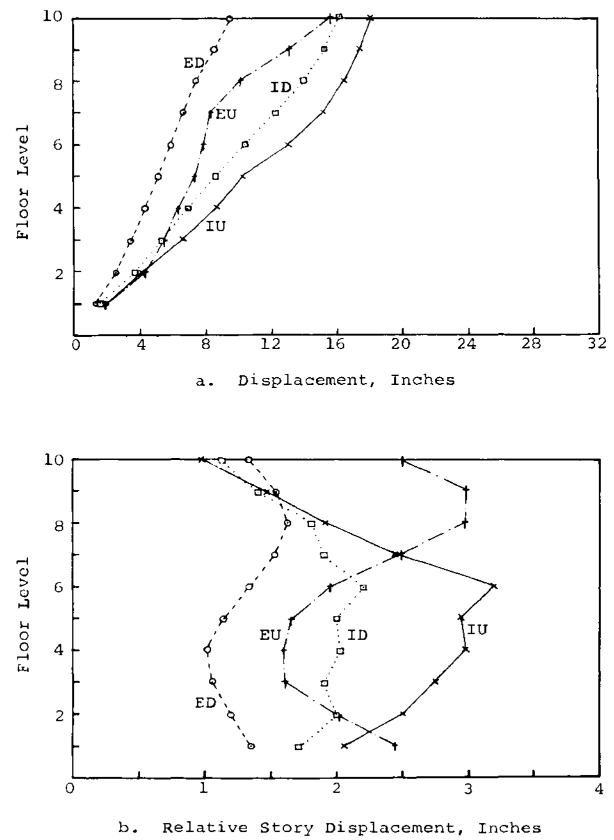


FIGURE 4.3. Structure B1 – IU, ID, ED and EU analyses

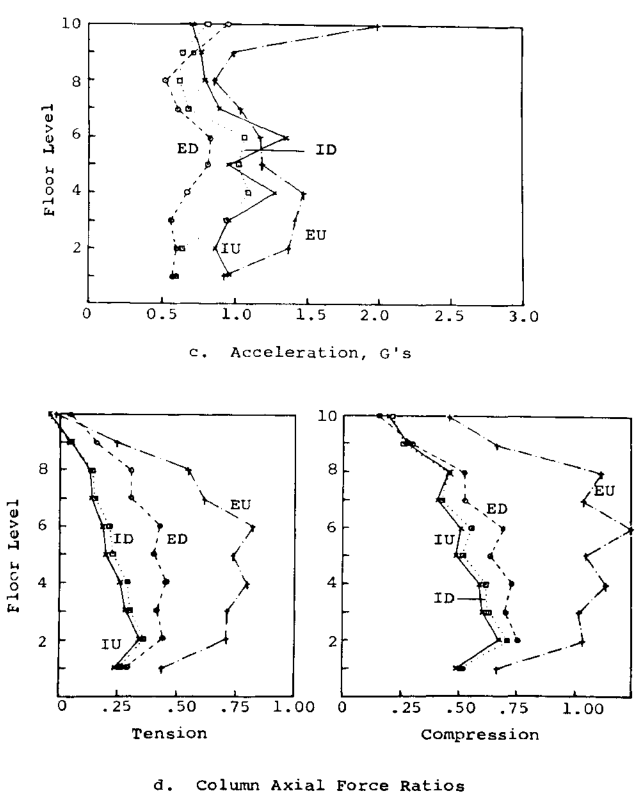


FIGURE 4.3. Structure B1—IU, ID, ED and EU analyses

pation in a real structure. A comparison of the results from the ED analysis with those of IU or ID analysis will be of particular interest to check the validity of such a procedure.

Analyses as mentioned above were performed on frames of Series One and Series Two. Same ground motion, i.e., 1.5 times the N-S component of the El Centro 1940 accelerogram, was used in each case. The response of each structure as computed from the different analyses is presented and discussed in the following.

4.3.1 Structure B1

The response of the structure B1 as computed from the four types of analyses is presented in Figures 4.3(a)–(k).

Figure 4.3(a) reveals that the floor displacements as computed from the IU analysis are the largest whereas those from the ED analysis form the lower bound. It is interesting to note that the displacements in the IU analysis are significantly greater than those in the EU case. There is a significant accentuation of displacements in the top stories (the so-called "whip-lash" effect) as found in the results of the EU analysis. This effect is damped out in the other three cases because of energy dissipation through inelastic action or viscous damping or both. It can also be noticed that the addition of viscous damping reduces the floor displacements in the elastic as well as inelastic case but the reduction is much more in the elastic results than the inelastic case. Similar observations are also applicable to relative story displacements which are shown in Figure 4.3(b).

Figure 4.3(c) shows that the EU analysis predicts the largest floor accelerations which are reduced by about 50% due to the addition of viscous damping. Same magnitude of viscous damping also decreases the floor accelerations in the inelastic analysis but the reduction is not as pronounced as in the elastic analysis.

The EU analysis grossly overestimates the column axial forces both in tension as well as in compression [Fig. 4.3(d)]. The addition of viscous damping decreases these forces by even more than 50% in some stories. Contrary to expectation the same source of viscous damping increased the column forces in the inelastic structure. The increase is not very significant but the trend is rather surprising.

The ductility ratios for the columns, girders and braces are shown in Figures 4.3(e), (f), and

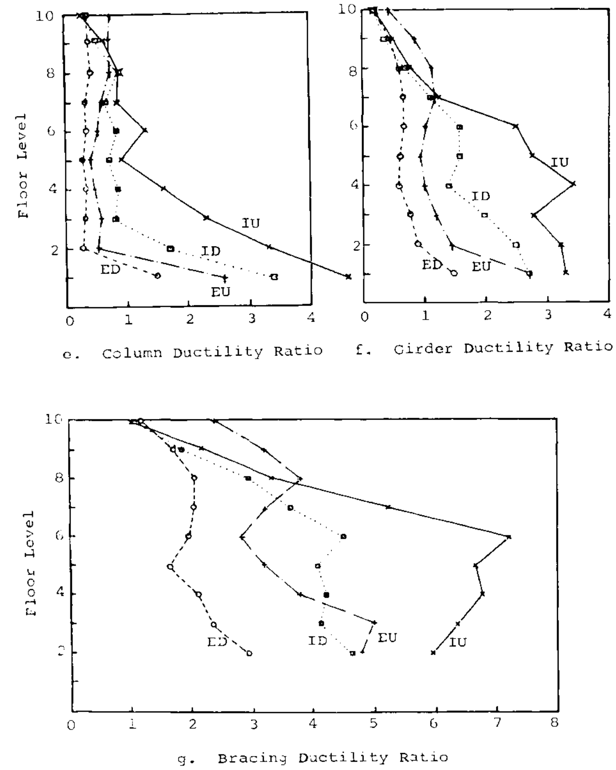


FIGURE 4.3. Structure B1—IU, ID, ED and EU analyses

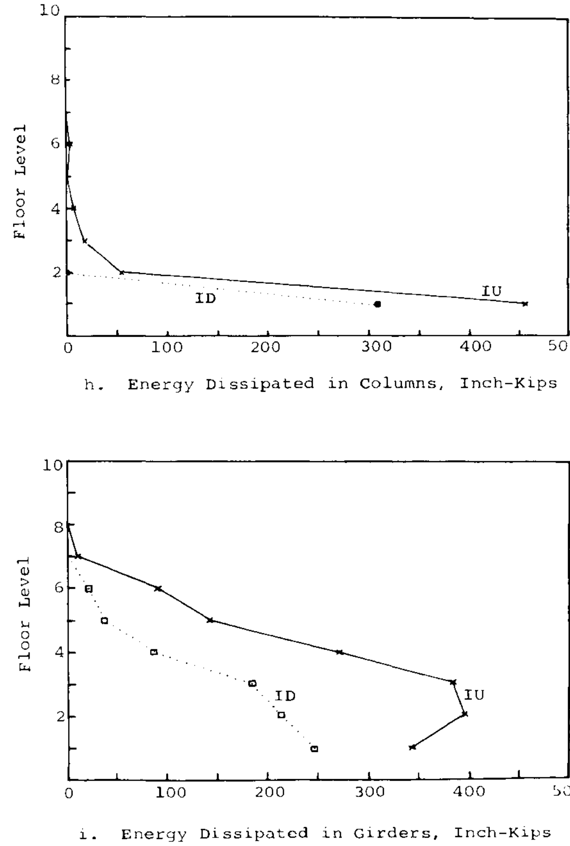


FIGURE 4.3. Structure B1—IU, ID, ED and EU analyses

(g), respectively. It will be noticed that except for a few stories near the top the member ductility ratios are very much underestimated by the elastic analysis both undamped as well as damped cases. The percentage reduction in the member ductility ratios caused by the viscous damping are about similar in the elastic and the inelastic analyses. This reduction is uniform for the elastic analysis whereas in the inelastic results the reduction is more pronounced in the lower stories where the ductilities are larger and tapers off towards the top where the ductilities are smaller.

The effect of viscous damping on the hysteresis energy dissipation of the structural members is shown in Figures 4.3(h), (i) and (j). The viscous damping decreases considerably the amount of energy dissipation required by inelastic action of the members. The decrease is so great that the columns in all the stories remain nearly elastic, with the exception of the bottom unbraced story. In either case the hysteresis energy dissipation comes from the girders and braces much more than from the columns.

The dissipated energy per floor as obtained from the IU and ID analyses is plotted in Figure 4.3(k). In the undamped case the energy dissipation through the inelastic deformation of structural members is more concentrated in the

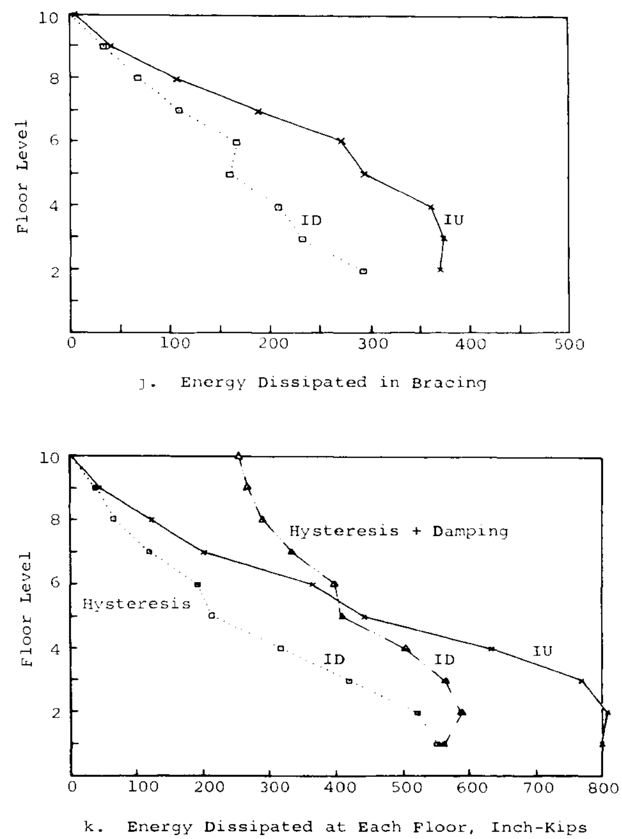
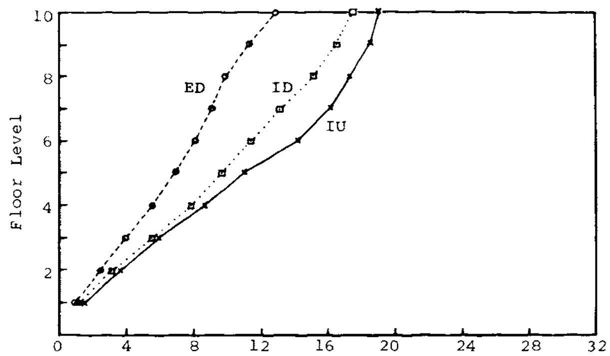
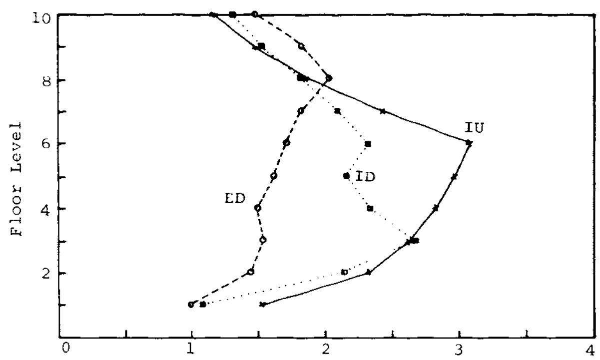


FIGURE 4.3. Structure B1—IU, ID, ED and EU analyses

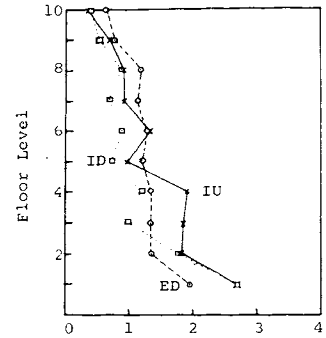


a. Displacement, Inches

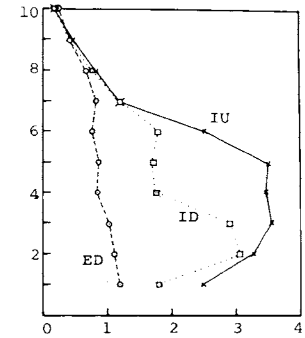


b. Relative Story Displacement, Inches

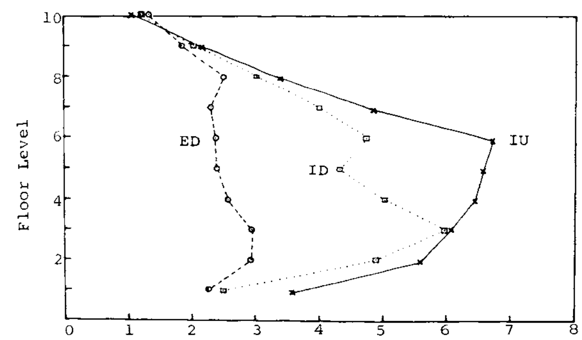
FIGURE 4.4. Structure F1—IU, ID and ED analyses



e. Column Ductility Ratio

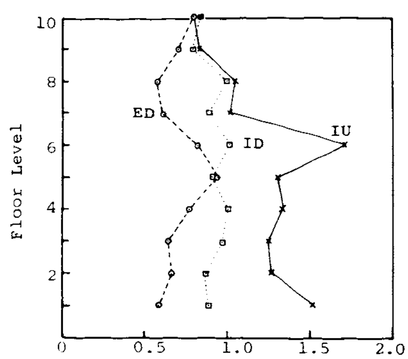


f. Girder Ductility Ratio

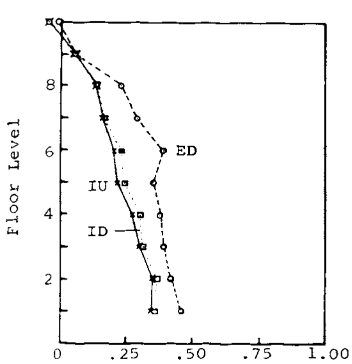


g. Bracing Ductility Ratio

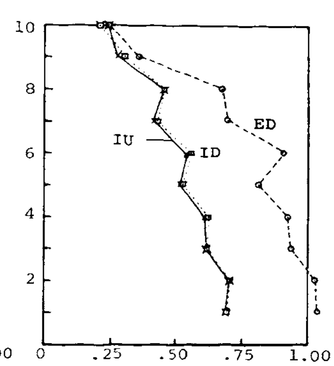
FIGURE 4.4. Structure F1—IU, ID and ED analyses



c. Acceleration, G's



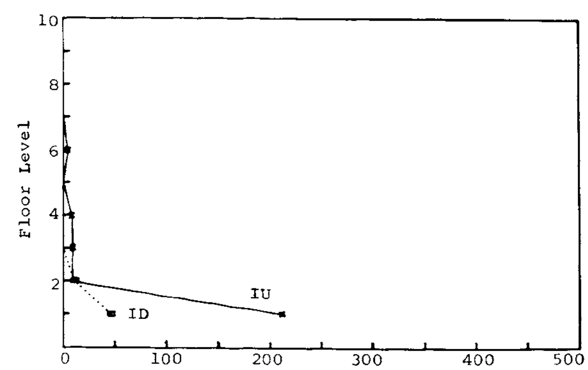
Tension



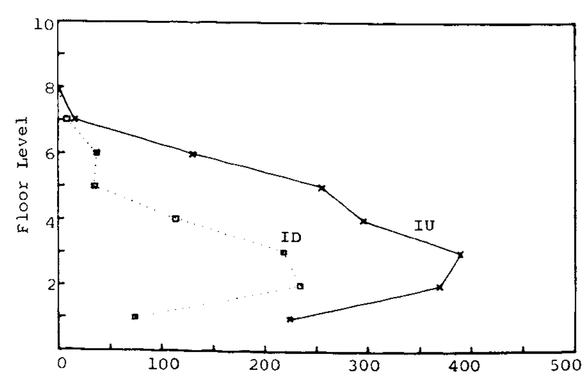
Compression

d. Column Axial Force Ratios

FIGURE 4.4. Structure F1—IU, ID and ED analyses

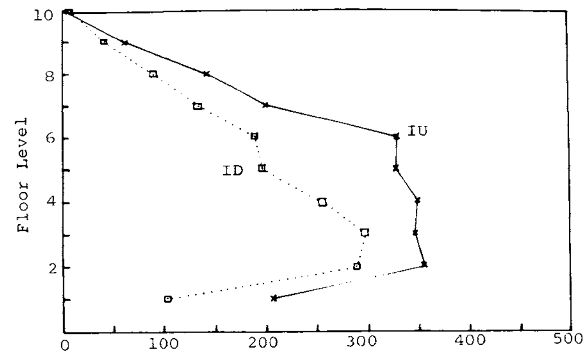


h. Energy Dissipated in Columns, Inch-Kips

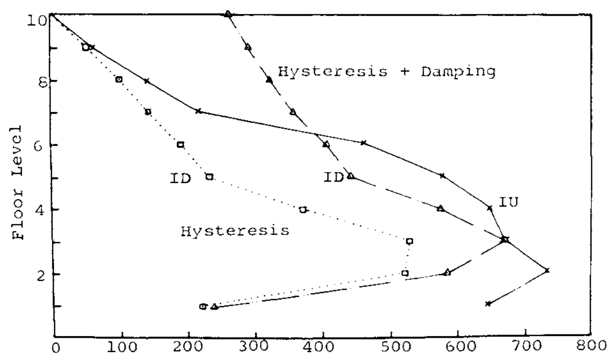


i. Energy Dissipated in Girders, Inch-Kips

FIGURE 4.4. Structure F1—IU, ID and ED analyses



j. Energy Dissipated in Bracing, Inch-Kips



k. Energy Dissipated at Each Floor, Inch-Kips

FIGURE 4.4. Structure F1 IU, ID and ED analyses

lower stories. It tapers off rapidly in the upper stories. Addition of 5% of critical damping in each elastic mode decreases the hysteresis energy dissipation in each story by about 30 to 50%. But when the total of hysteresis energy and damping energy is plotted against the story level it is noticed that the distribution along the height is more uniform than in the undamped case. This shows that the hysteresis energy dissipation is more in the lower stories while the energy dissipation through viscous damping is more concentrated in the upper stories of the structure.

4.3.2 Structure F1

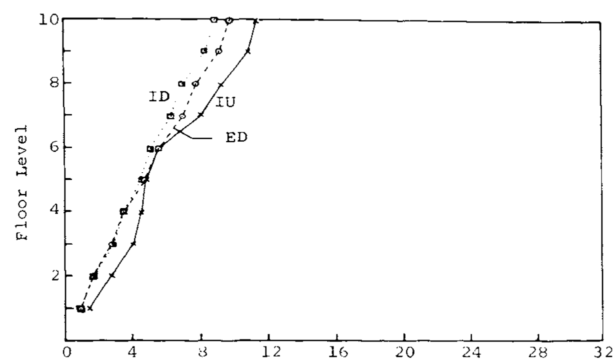
The frame F1 was subjected to three analyses—IU, ID and ED. The results are presented in Figures 4.4(a)-(k). Since the frames F1 and B1 are very similar in design it would be quite reasonable to expect similar behavior of these two frames when subjected to same earthquake and the type of analysis. The results presented in Figure 4.4 are in agreement with this expectation. The response of the frame F1 as obtained from IU, ID and ED analyses is similar to that of B1 from the corresponding analyses.

The same observations as were made about the frame B1 can also be made from these results about the frame F1. The ED analysis underestimates the response very significantly with the exception of column axial forces. It can be said about both the frames B1 and F1 that the response as computed from the elastic analysis with 5% of critical damping in each mode does not represent their inelastic behavior to any reasonable degree of accuracy.

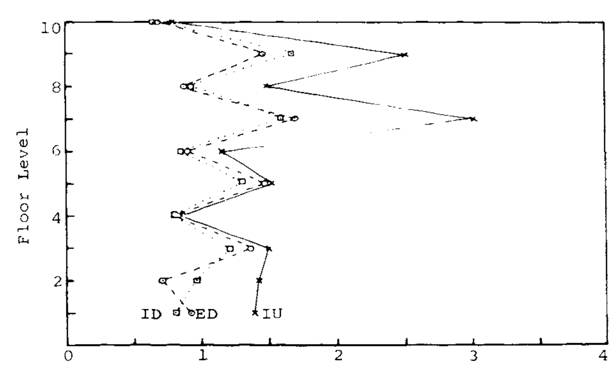
4.3.3 Structure A1

This is the alternately braced frame in Series One with diagonal braces located in the even-numbered stories. The response of this structure as computed from the IU, ID and ED analysis is presented in Figures 4.5(a–(k).

The frame A1 shows smaller floor displacements [Fig. 4.5(a)] than those in B1 or F1 possibly because the latter two frames have three times the floor masses than the former. But the omission of bracing in alternate stories caused substantially larger relative displacements than in the braced stories particularly in the upper portion of the frame [Fig. 4.5(b)]. The addition of viscous damping reduces the lateral displacements quite significantly but the most drastic effect is noticed on the floor accelerations [Fig. 4.5(c)]. The viscous



a. Displacement, Inches



b. Relative Story Displacement, Inches

FIGURE 4.5. Structure A1—IU, ID and ED analyses

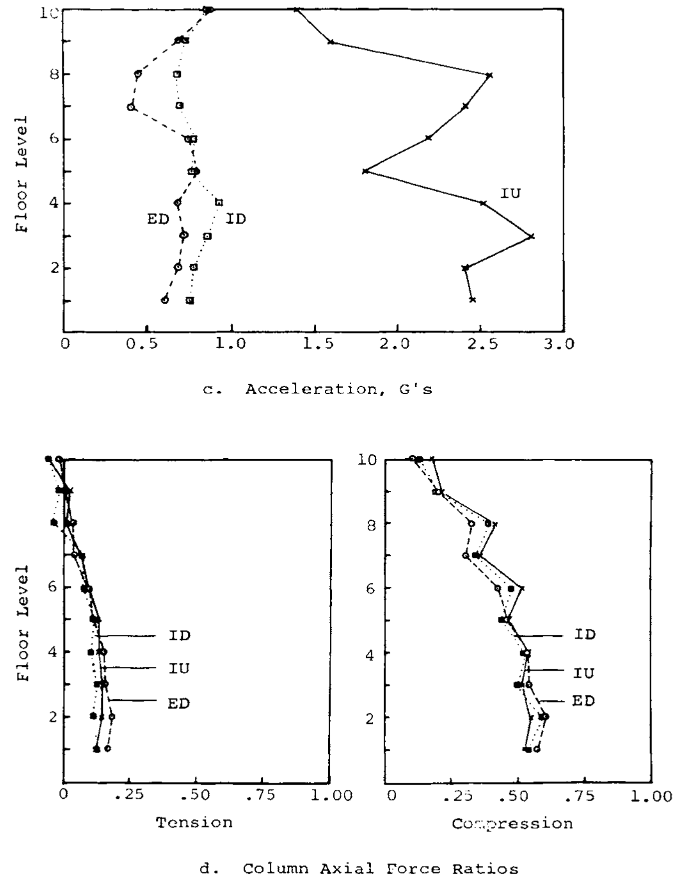


FIGURE 4.5. Structure A1—IU, ID and ED analyses

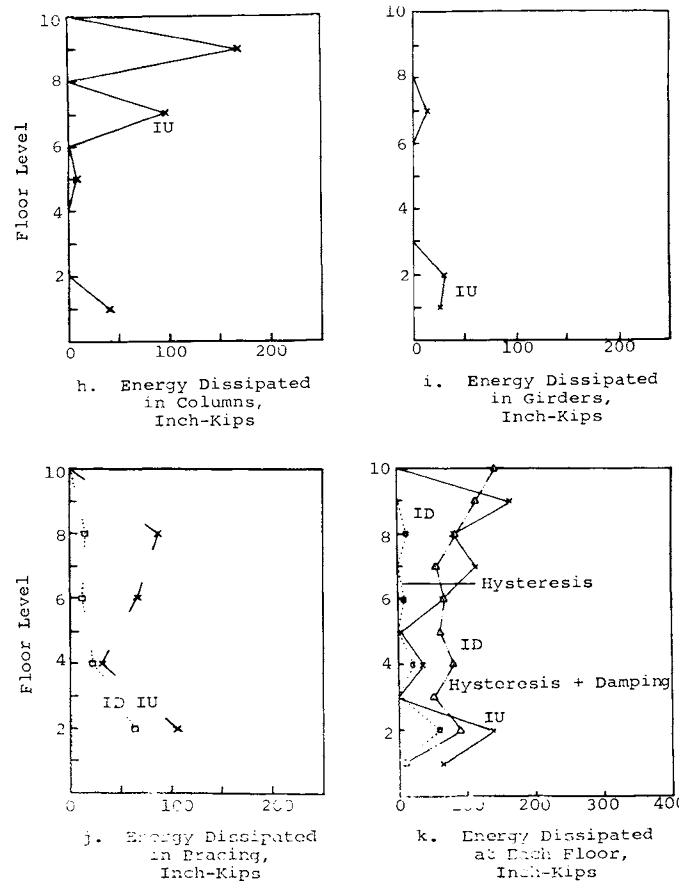


FIGURE 4.5. Structure A1—IU, ID and ED analyses

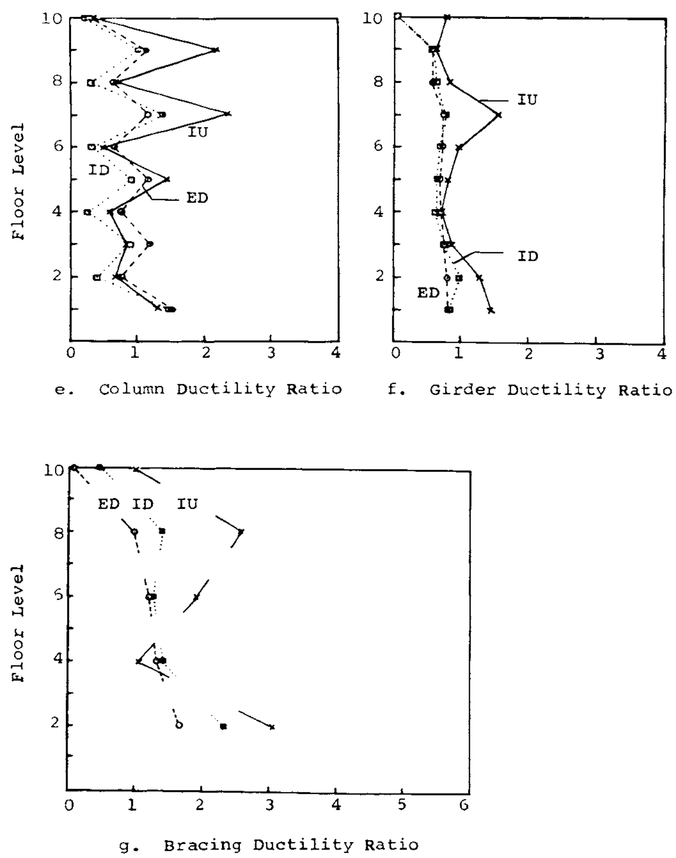


FIGURE 4.5. Structure A1—IU, ID and ED analyses

damping in the ID analysis causes a reduction up to even 80% in the floor accelerations at some floor levels. Like the relative story displacements the column ductility ratios are also larger in the unbraced stories than in the braced ones.

Addition of viscous damping causes a significant reduction in the ductility and hysteresis energy demands on the members of this structure. It is also noticed that the results of ED and ID analyses have better correspondence in this structure than in the case of frames B1 and F1.

4.3.4 Structure U1

U1 is the completely unbraced structure of Series One. Its response from the IU, ID and ED analyses is presented in Figures 4.6(a)–(i). As can be noticed from Figures 4.6(a) and (b) this frame turns out to be the most flexible structure in Series One showing large lateral displacements. Viscous damping reduces the displacements by about 40% in the inelastic analysis but these are still larger than those given by the ED analysis.

Unlike the frame A1 (which has the same floor mass as U1) the floor accelerations, and the column axial forces and ductility ratios in this

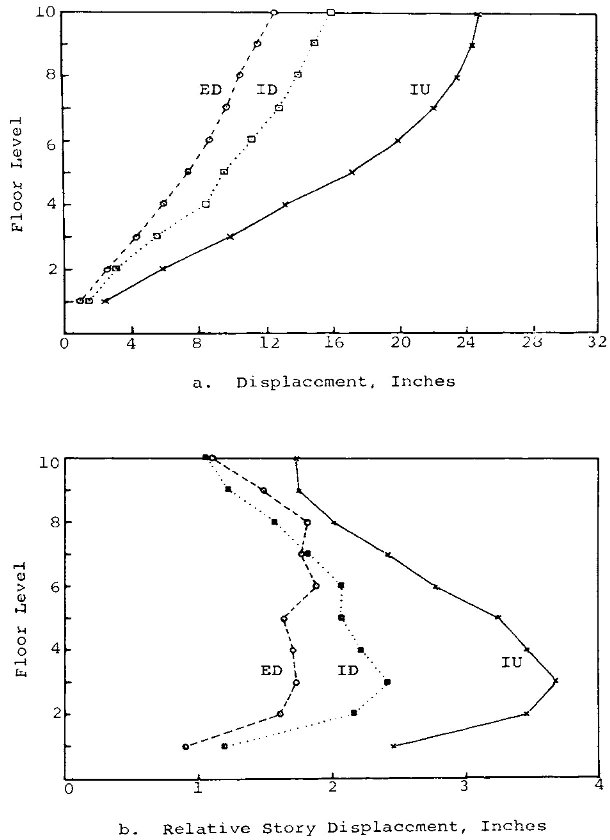


FIGURE 4.6. Structure U1—IU, ID and ED analyses

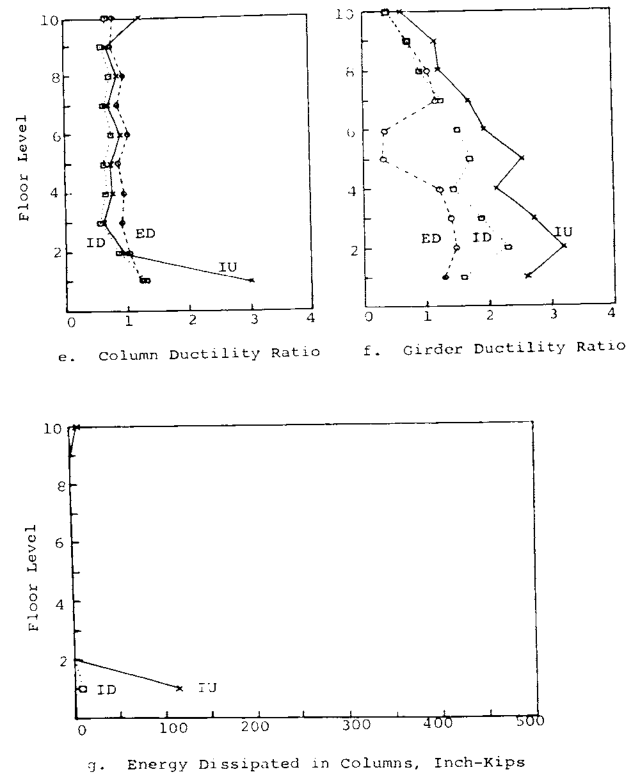


FIGURE 4.6. Structure U1—IU, ID and ED analyses

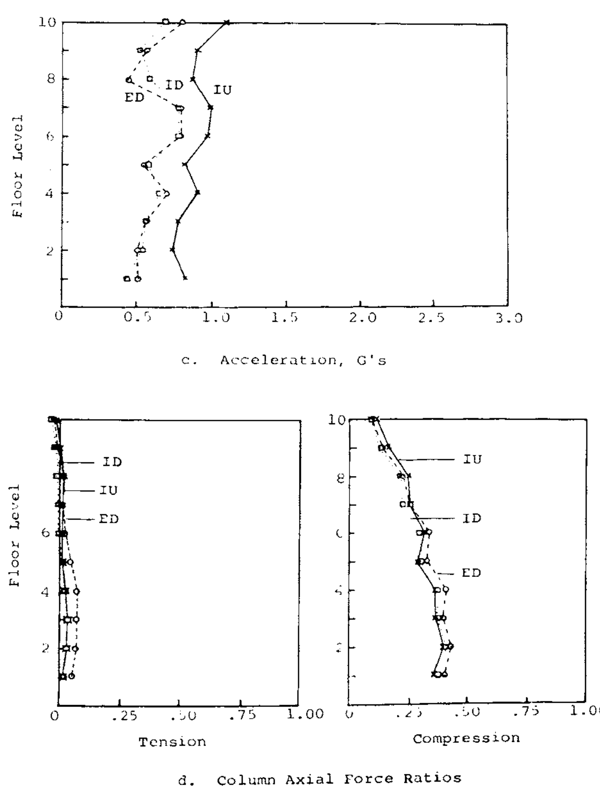


FIGURE 4.6. Structure U1—IU, ID and ED analyses

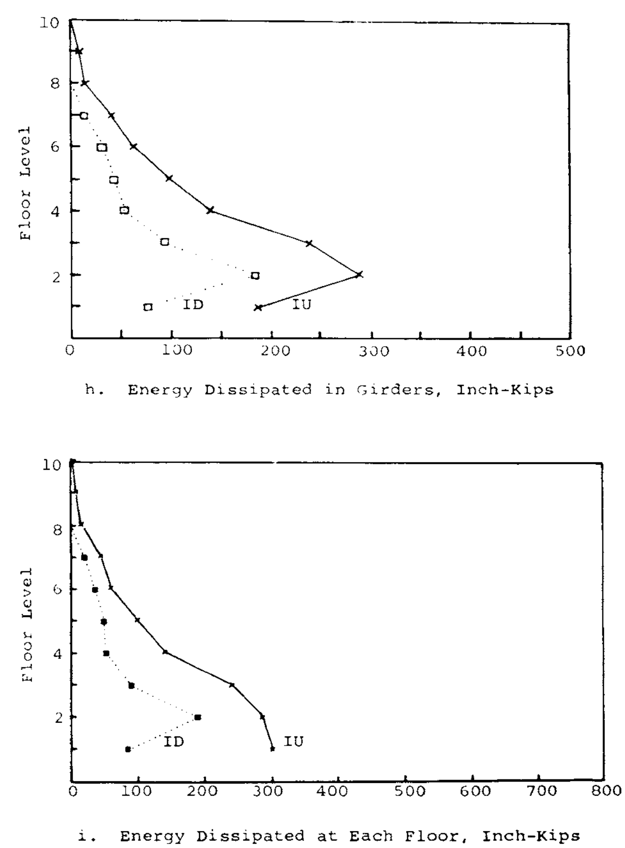


FIGURE 4.6. Structure U1—IU, ID and ED analyses

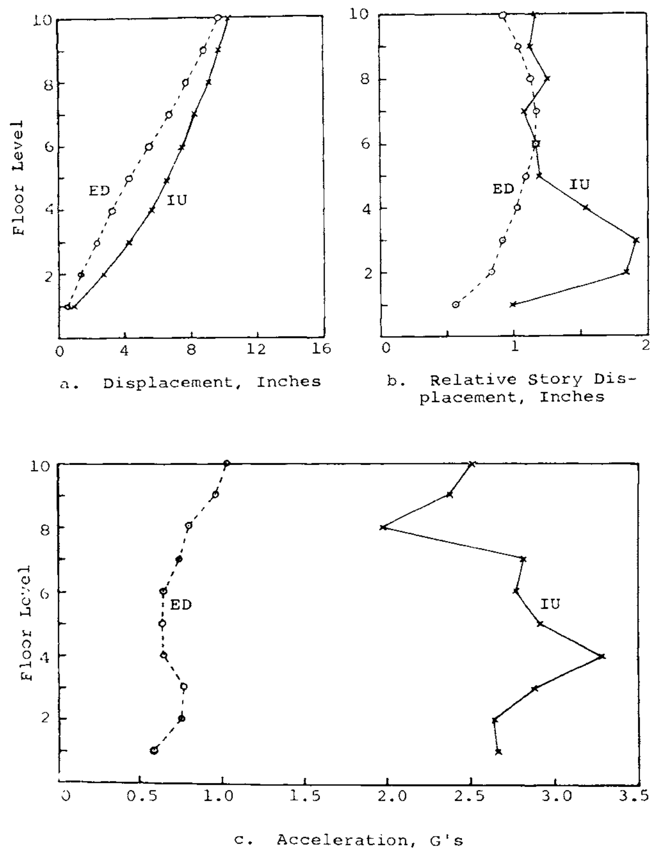


FIGURE 4.7. Structure U1X—IU and ED analyses

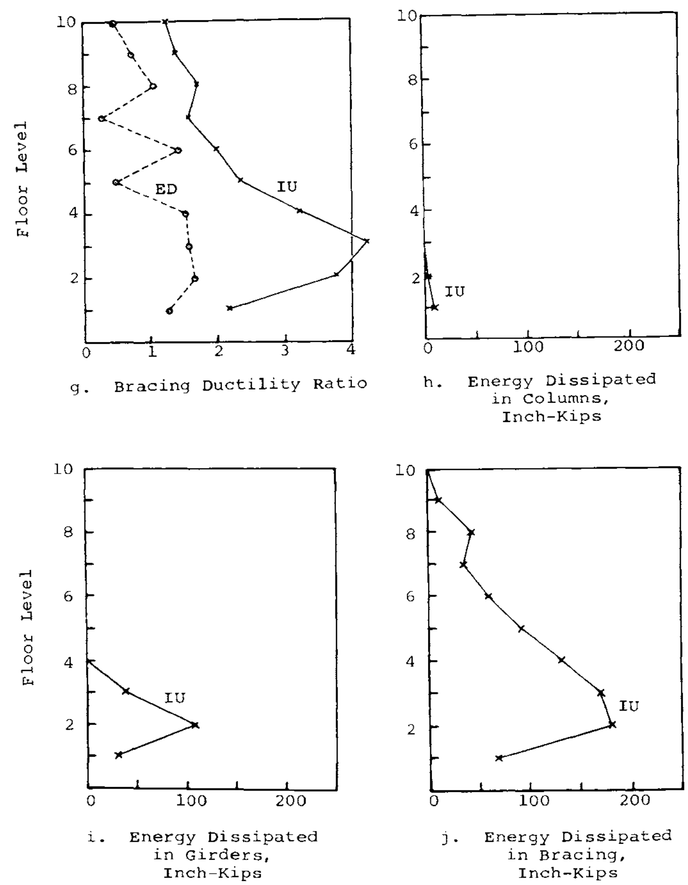


FIGURE 4.7. Structure U1X—IU and ED analyses

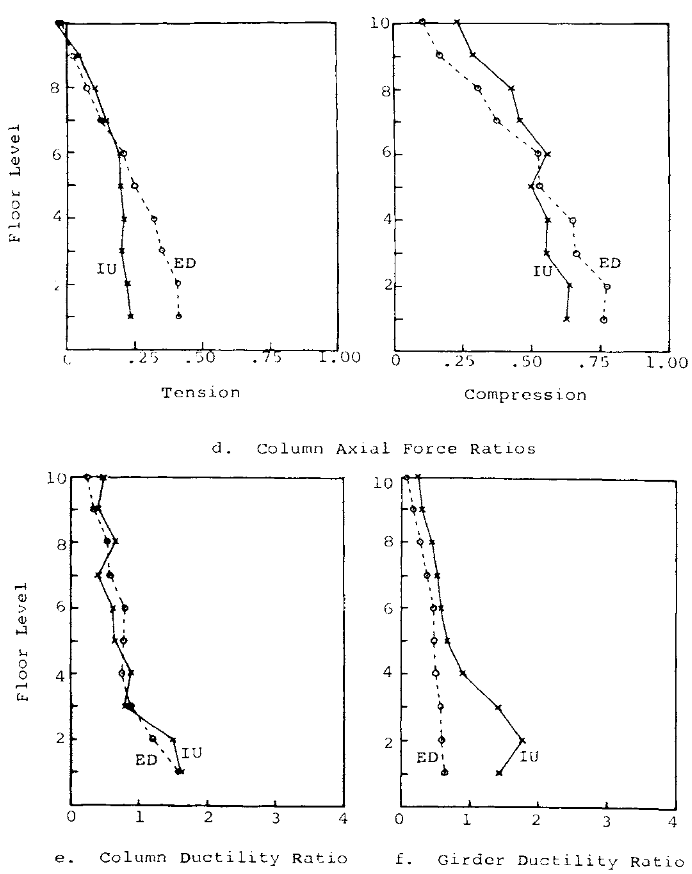


FIGURE 4.7. Structure U1X—IU and ED analyses

frame are not as high [Figs. 4.6(c), (d) and (e)]. In fact, there is minimal inelastic activity in the columns of this frame - the girders provide almost all of the hysteresis energy [Figs. 4.6(f), (g), (h) and (i)]. Viscous damping decreases the hysteresis energy demand on the girders by approximately 40%.

It is thus observed that the same magnitude of viscous damping is more effective in reducing the inelastic response of the frames A1 and U1 than it was for B1 and F1.

4.3.5 Structure U1X

It was noticed that the unbraced model U1 in Series One turned out to be a flexible structure showing large lateral displacements. The frame U1X was obtained by adding minimum cross bracing (slenderness ratio about 300) to the open frame U1 for the sole purpose of drift control as is quite often done in practice. The response of this structure from the IU and the ED analyses is compared in Figures 4.7(a)–(j).

It can be seen that the addition of minimum cross bracing does reduce the lateral displacements by even more than 50%. The results from the ED analysis are still under those from the IU

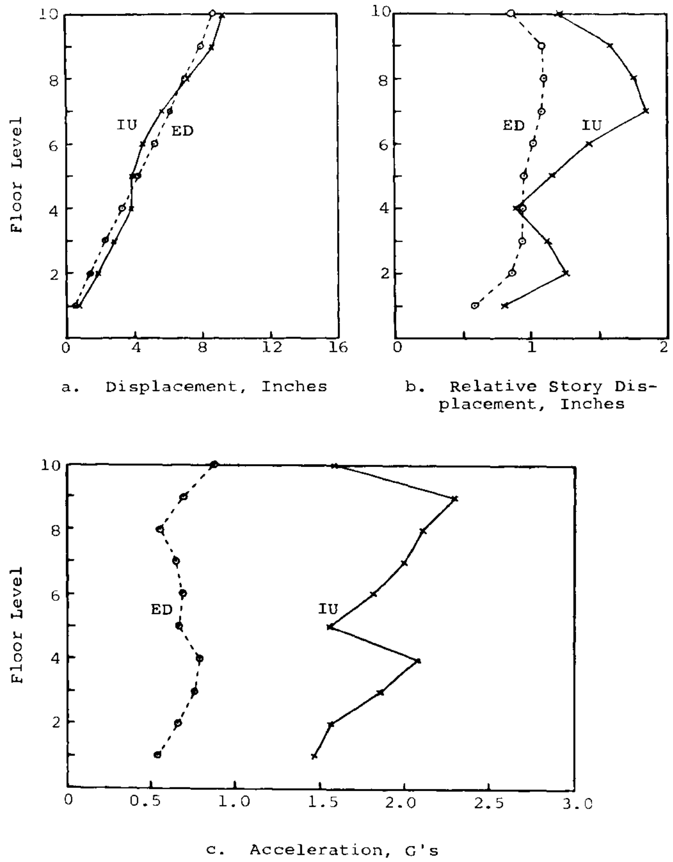


FIGURE 4.8. Structure F2—IU and ED analyses

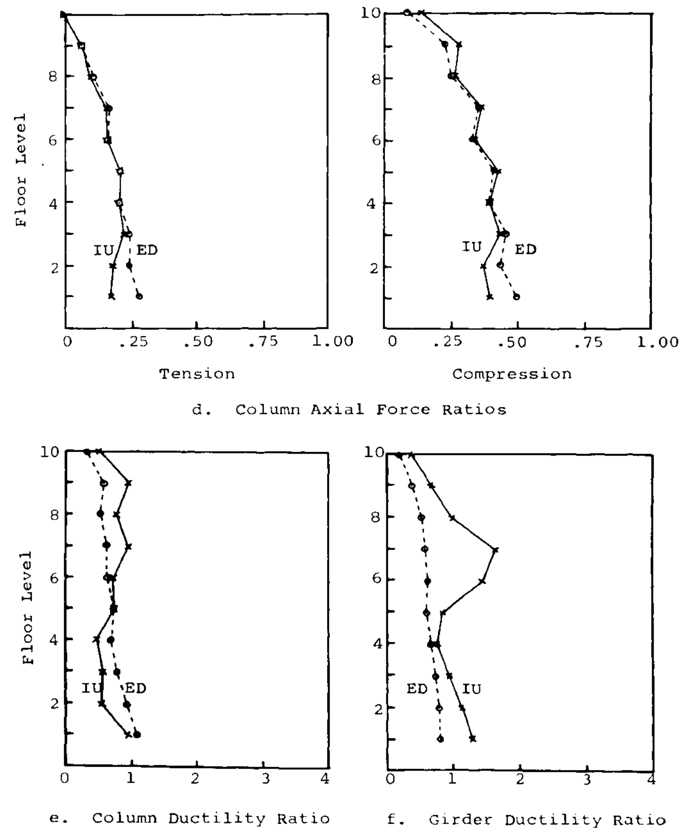


FIGURE 4.8. Structure F2—IU and ED analyses

analysis, but the difference becomes large in floor accelerations [Fig. 4.7(c)]. In the IU analysis the addition of minimum cross bracing to the open frame U1 results in a large increase in the floor accelerations and the column axial forces [Fig. 4.7(d)]. The IU and the ED analyses predict almost similar column ductilities but the girder and bracing ductilities are grossly underestimated by the ED analysis.

4.3.6 Structure F2

The response of the structure F2 as computed from the IU and ED analyses is plotted in Figures 4.8(a)–(i). It will be noticed that the floor displacements found in the two analyses are very nearly equal [Fig. 4.8(a)]. But the relative floor displacements in the IU analysis are generally greater than those of ED analysis, the latter being more uniform in each story than the former [Fig. 4.8(b)]. The IU analysis predicts floor accelerations which are about twice those from the ED analysis [Fig. 4.8(c)]. The column axial forces and ductility ratios are very similar in the two cases. But the girder and bracing ductilities are significantly underestimated in the ED analysis.

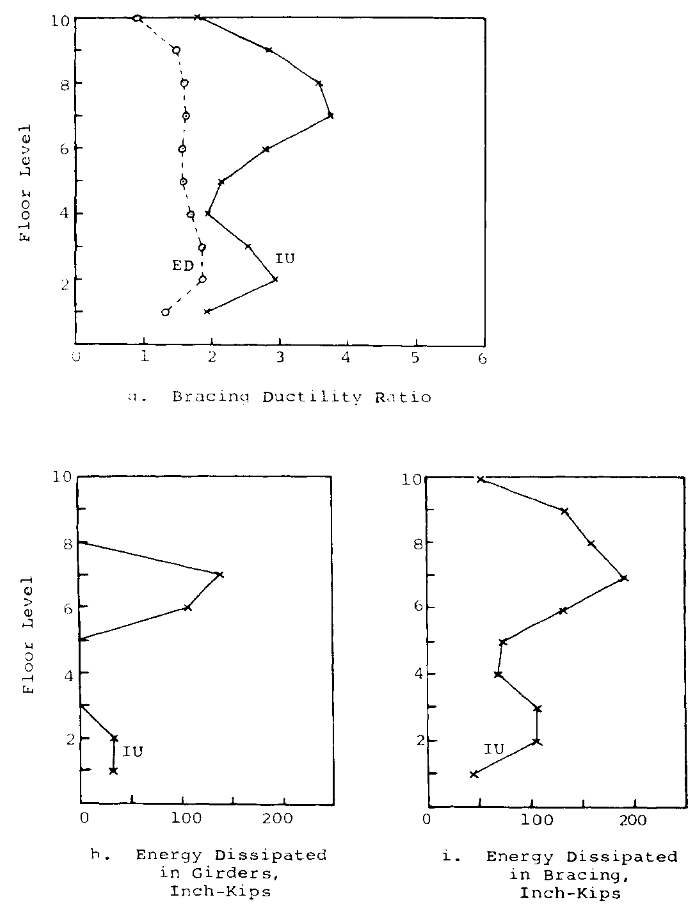


FIGURE 4.8. Structure F2—IU and ED analyses

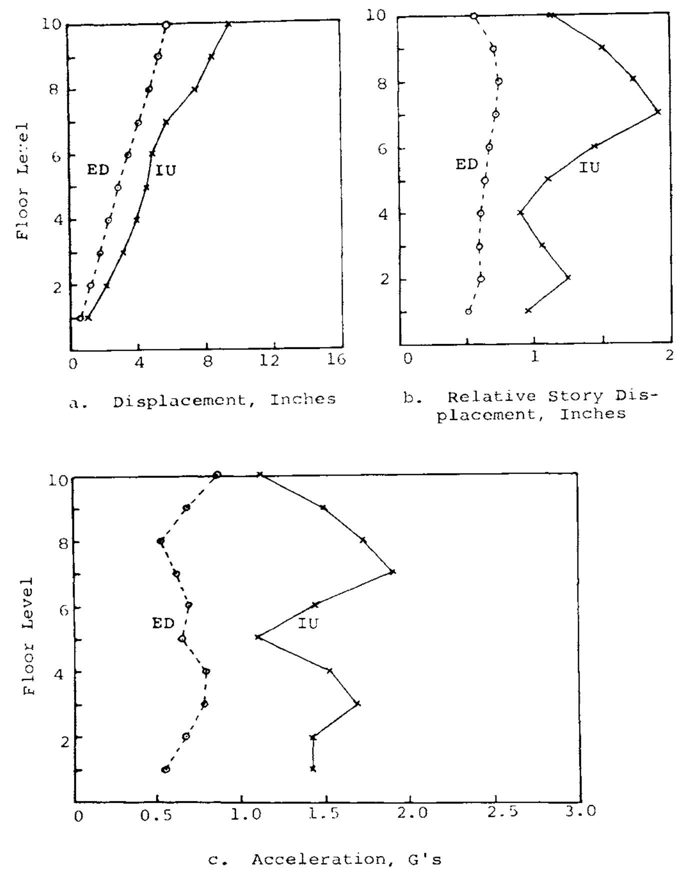


FIGURE 4.9. Structure B2—IU and ED analyses

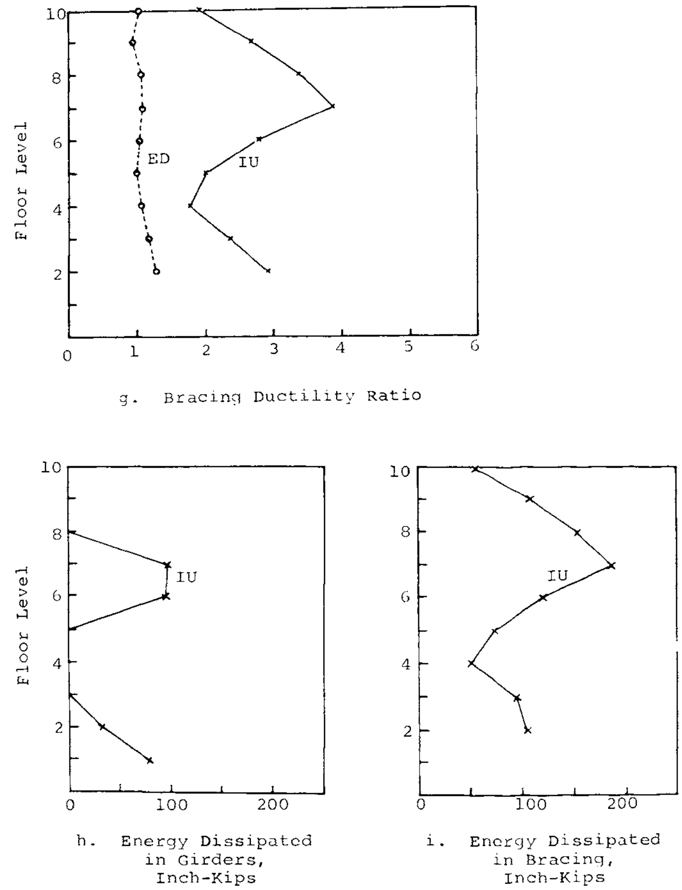


FIGURE 4.9. Structure B2—IU and ED analyses

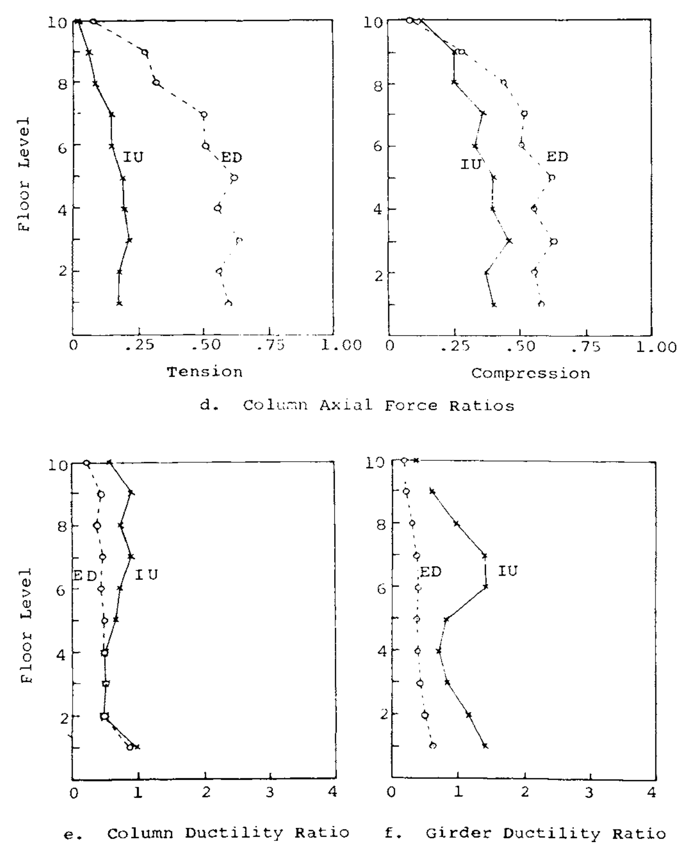


FIGURE 4.9. Structure B2—IU and ED analyses

4.3.7 Structure B2

The response of this structure as computed from the IU and ED analyses is shown in Figures 4.9(a)–(i). Once again the ED analysis underestimates the floor displacements, relative story displacements and floor accelerations—the discrepancy in the latter parameter being the largest. But the column axial forces turned out to be larger in the ED response than from the IU analysis. Column ductility ratios including the ductility ratios for girders and braces are significantly smaller in the ED analysis than in the IU case.

4.3.8 Structures A2 and U2

The frames A2 and U2 were also subjected to the IU and ED analyses and the results are shown in Figures 4.10(a)–(j) and Figures 4.11(a)–(h), respectively. A study of these figures will show that these two frames responded to the two analyses in the same manner as did the frame F2. Therefore, similar observations can also be made about the behavior of these two frames.

4.4 Energy- and Displacement-Time Histories

Time histories of certain response parameters, as obtained from the analyses discussed in this

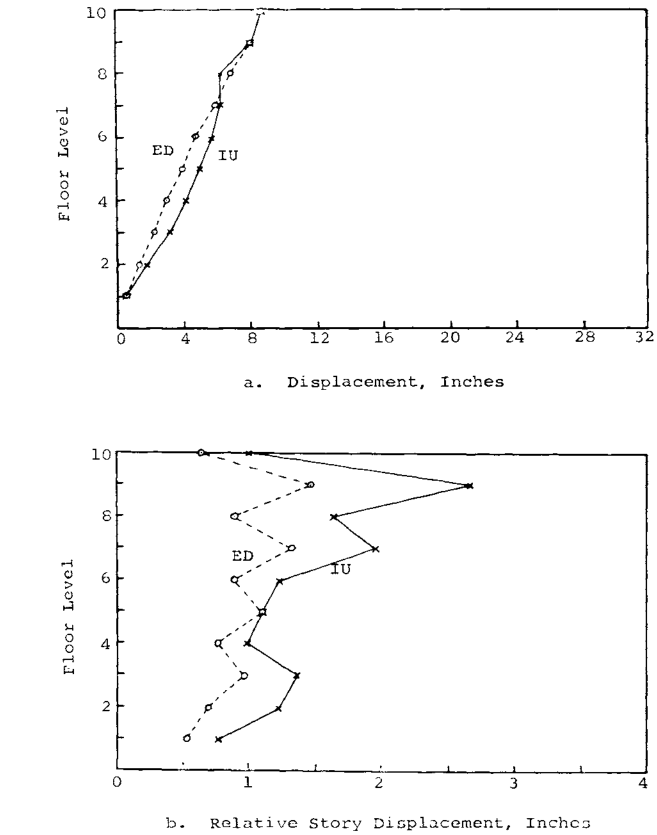


FIGURE 4.10. Structure A2—IU and ED analyses

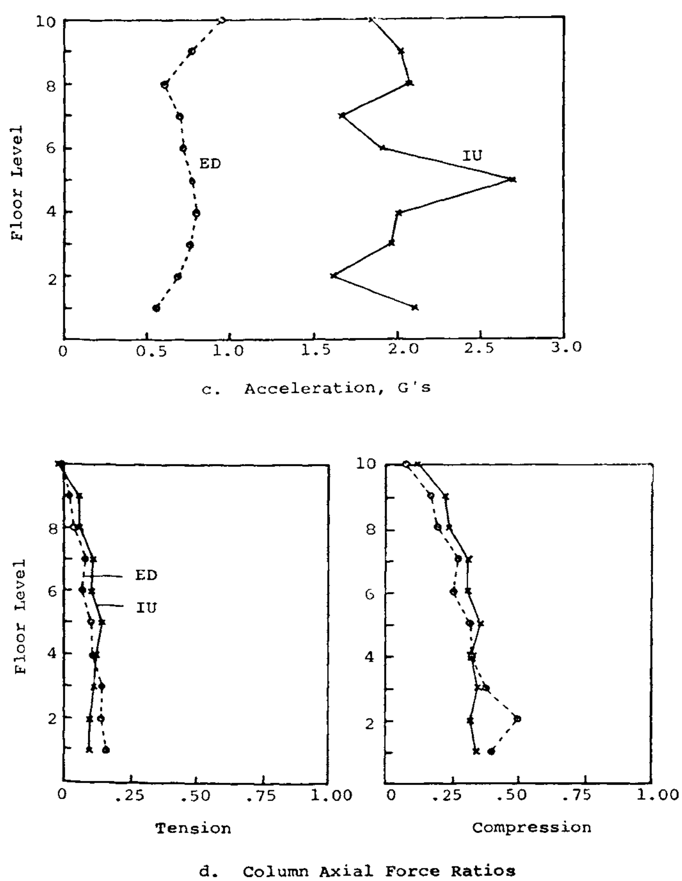


FIGURE 4.10. Structure A2—IU and ED analyses

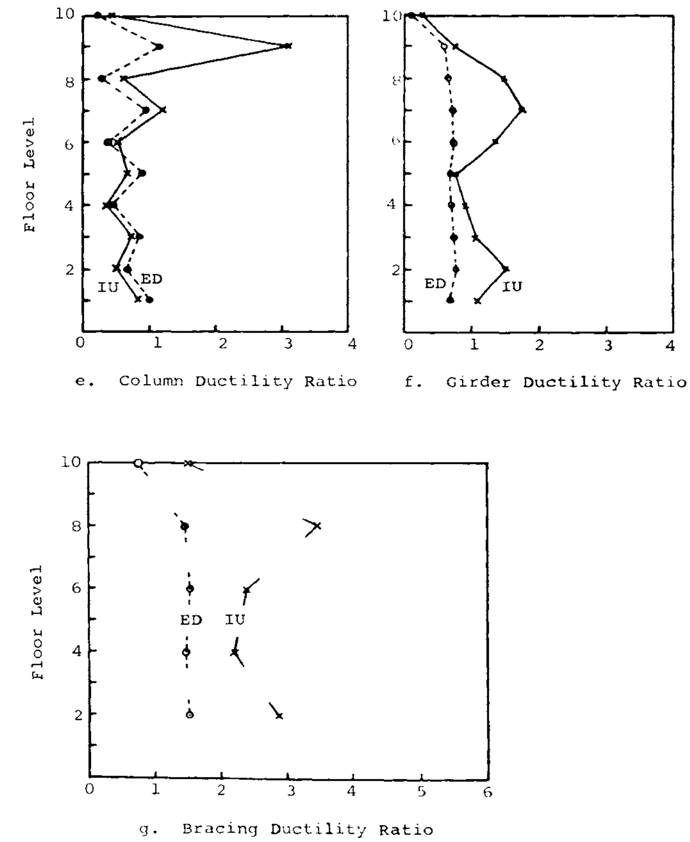


FIGURE 4.10. Structure A2—IU and ED analyses

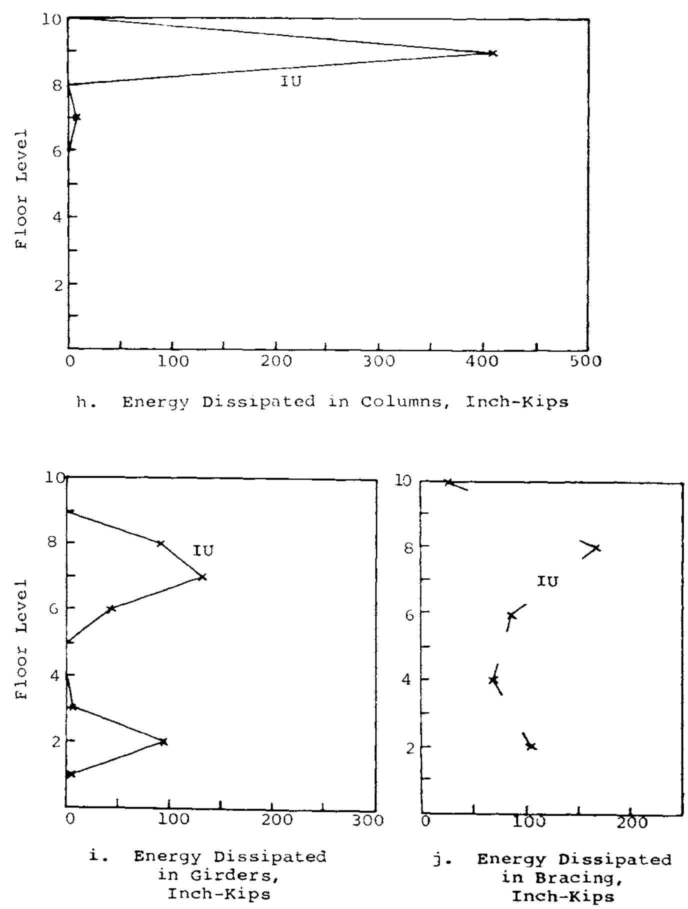
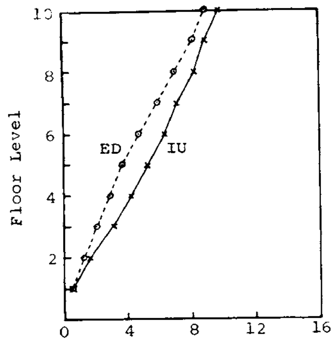
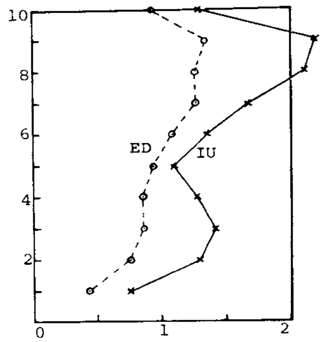


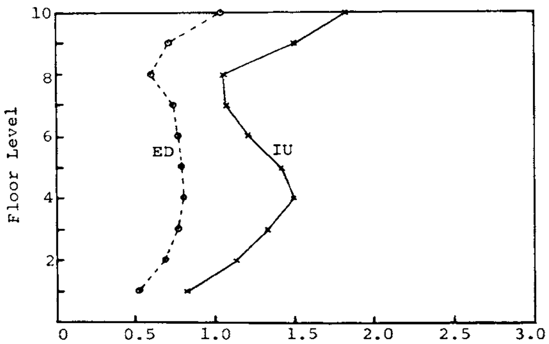
FIGURE 4.10. Structure A2—IU and ED analyses



a. Displacement, Inches

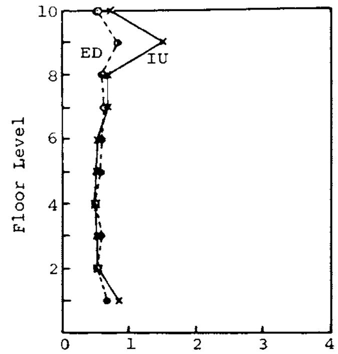


b. Relative Story Displacement, Inches

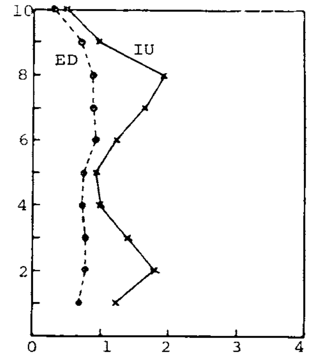


c. Acceleration, G's

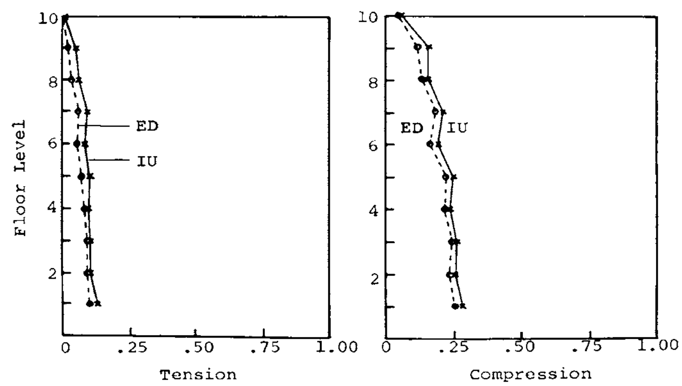
FIGURE 4.11. Structure U2—IU and ED analyses



d. Column Ductility Ratio

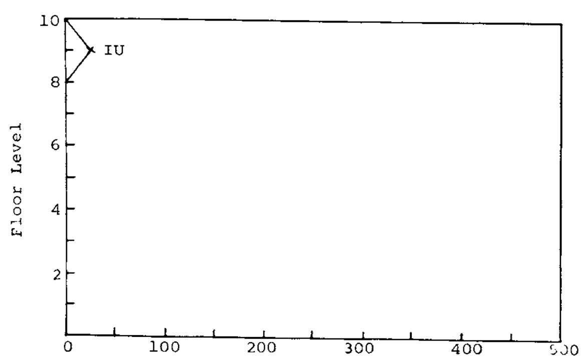


e. Girder Ductility Ratio

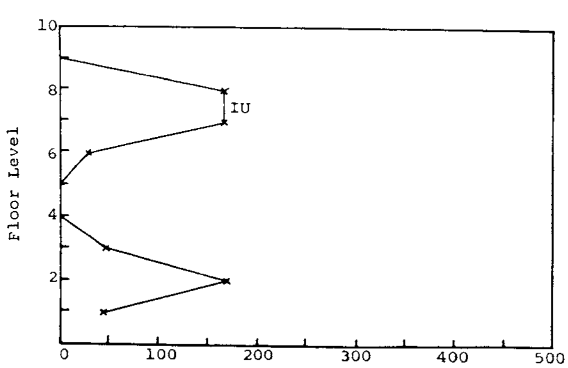


f. Column Axial Force Ratios

FIGURE 4.11. Structure U2—IU and ED analyses



g. Energy Dissipated in Columns, Inch-Kips



h. Energy Dissipated in Girders, Inch-Kips

FIGURE 4.11. Structure U2—IU and ED analyses

chapter, are presented in Appendixes A through I. These response parameters are: input energy, energies dissipated through inelastic deformation and viscous damping, and lateral displacements at selected floor levels. It should be noted that the energy dissipated through viscous damping is a significant part of the total dissipated energy. In the IU analysis the frame U1 shows a gradual buildup of permanent lateral drift, Figure D.2. This type of drift is not observed in any of the other cases.

Chapter 5
Summary and Conclusions

5.1 Summary

This report presents a study of the effect of minimum cross bracing upon the inelastic behavior of multistory building frames when subjected to a severe earthquake motion. The slenderness ratio of the diagonal bracing members was kept around 300—the maximum value that is generally permitted by codes for the secondary structural members. Because of their large slenderness the compression strength of the bracing members was neglected and they were assumed to behave elasto-plastically in tension only. The hysteresis behavior of the beams and columns in reversed bending was assumed to be elasto-plastic. The effect of axial forces on the plastic moment of the columns was included so that the column yield moments were varying during the lateral vibration of the structure. The effect of axial deformation of the columns was also included in the analysis. On the other hand the beams were assumed to be axially rigid and the effect of axial force on their yield moment was ignored. The P-Δ effect was included in the analysis.

The method of analysis, which was described in Chapter 2, was based upon the above assumptions regarding the hysteretic behavior of the structural members of the braced frames. The multistory frame was treated as a lumped-mass system and the resulting equations of lateral motion were solved by a Runge-Kutta fourth-order numerical procedure. The response of the frame to the in-plane horizontal component of ground motion was computed by an incremental technique. Viscous damping equivalent of 5% of critical in each elastic mode of the structure was also included in some cases.

The basic objective of this study was to evaluate the seismic behavior of the multistory braced frame structures of steel. The scope was limited to single bay, 10-story frames having slender diagonal bracing members which were designed as tension members only. Two different groups of structures were analyzed: Series One, which consisted of frames designed for the minimum lateral force requirement of a seismic code (similar to the Uniform Building Code 1967) and using current design procedures; and Series Two, in which the frames were designed for a common fundamental period of vibration of approximately 1.25 sec. In each series structural variations were obtained by changing the arrangement of the diagonal bracing in the stories. The base excitation was the N-S component of the El Centro 1940 earthquake with the acceleration ordinates multiplied by a factor of 1.5. In order to keep the computer time within reasonable limits, the dynamic response of the structures to the first seven seconds of this modified accelerogram was computed. The maximum response parameters which characterize the seismic behavior of the structures were recorded. A description of the program of investigation was given in Chapter 3.

The results of the various scheduled analyses were presented and discussed in Chapter 4. The influence of the varying arrangements of the minimum cross bracing upon the seismic behavior of the structural models in each series was studied by comparing their response for the same earthquake. A cross comparison between the two series also pointed out some significant similarities and differences in the seismic behavior of these structures, which were obtained by two different design philosophies. Comparisons were also made for these structures between the response results as computed from the inelastic undamped, inelastic damped, elastic damped and elastic undamped analyses. This was done to study how these structures would respond to the same earthquake under varying conditions of analysis.

5.2 Conclusions

Like any other single investigation the present study also has its limitations of scope and the assumptions made in the method of analysis.

Nevertheless, this initial study has provided some insight into the seismic behavior of multistory structures with minimum cross bracing. Some of the more significant aspects of the results are summarized in the following:

1. The frames of Series One were designed for minimum code lateral forces and using current design procedures. These frames proved to be rather flexible showing large lateral displacements, member ductility ratios and energy demands for the 1.5 times the N-S component of the El Centro 1940 earthquake that was used in the study. The ductility ratios for the bracing members were as large as 7 and those for the columns went up to about 5. These peak ductility ratios were obtained from the undamped analyses. While large ductility ratios for the bracing members do not appear to be serious for the overall structure, it would be desirable to use stronger columns for Series One structures in order to limit their inelastic activity to a minimum.

2. The effect of including 5% viscous damping was generally more pronounced on the elastic response than on the results of the inelastic analysis, with a few exceptions. The inelastic damped analyses generally show moderate axial force and ductility ratios for the columns of Series One. The largest column ductility ratio was 3.4 and ductility ratios for the bracing members were as large as 6. The floor accelerations were found to be most sensitive to viscous damping—a reduction of up to 80% was noticed at some floor levels in the inelastic response.

3. The frames of Series Two, on the other hand, were controlled by period rather than strength and were proportioned for a common fundamental period of 1.25 sec. These frames were undoubtedly stronger and stiffer than those of Series One and showed smaller lateral displacements and lesser inelastic activity in the members. The columns generally remained elastic except in a few instances.

4. The addition of minimum cross bracing to restrict the lateral drift of an unbraced frame was found to be effective in reducing the lateral displacements and the inelastic activity in the columns and girders. This was accompanied by substantially increased axial forces in the columns (T/T_y values reaching up to 0.7) and large floor accelerations of the order of 3.3 g.

5. The elastic analysis with or without viscous damping does not, to any degree of accuracy, represent the inelastic behavior of structures when considerable yielding occurs in most of the structural components. Of course, if the system has very small inelasticities, the elastic analysis will provide reasonable results. For the frames of Series One, the elastic analysis grossly underestimated the lateral displacements and the member ductility ratios, but overestimated the column axial forces.

Although both column axial forces and moments were used to define the column ductility ratio, this ratio and the column axial force have been discussed separately in the report and conclusions. It is recognized that the behavior of the column is influenced by a combination of these parameters. It should also be noted that 5% viscous damping was selected herein to study the effect of viscous damping on the response. In designing a structure the percentage of viscous damping which represents the energy dissipation by nonstructural elements must be selected, probably between 1 and 5%.

The behavior of minimum cross braced frames does not necessarily predict the behavior of other types of braced systems. A future study which incorporates bracing with compressive and flexural strength would be desirable to improve our understanding of the seismic behavior of braced frame structures.

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APPENDIX A

Energy- and Displacement-Time Histories

for

Structure B1

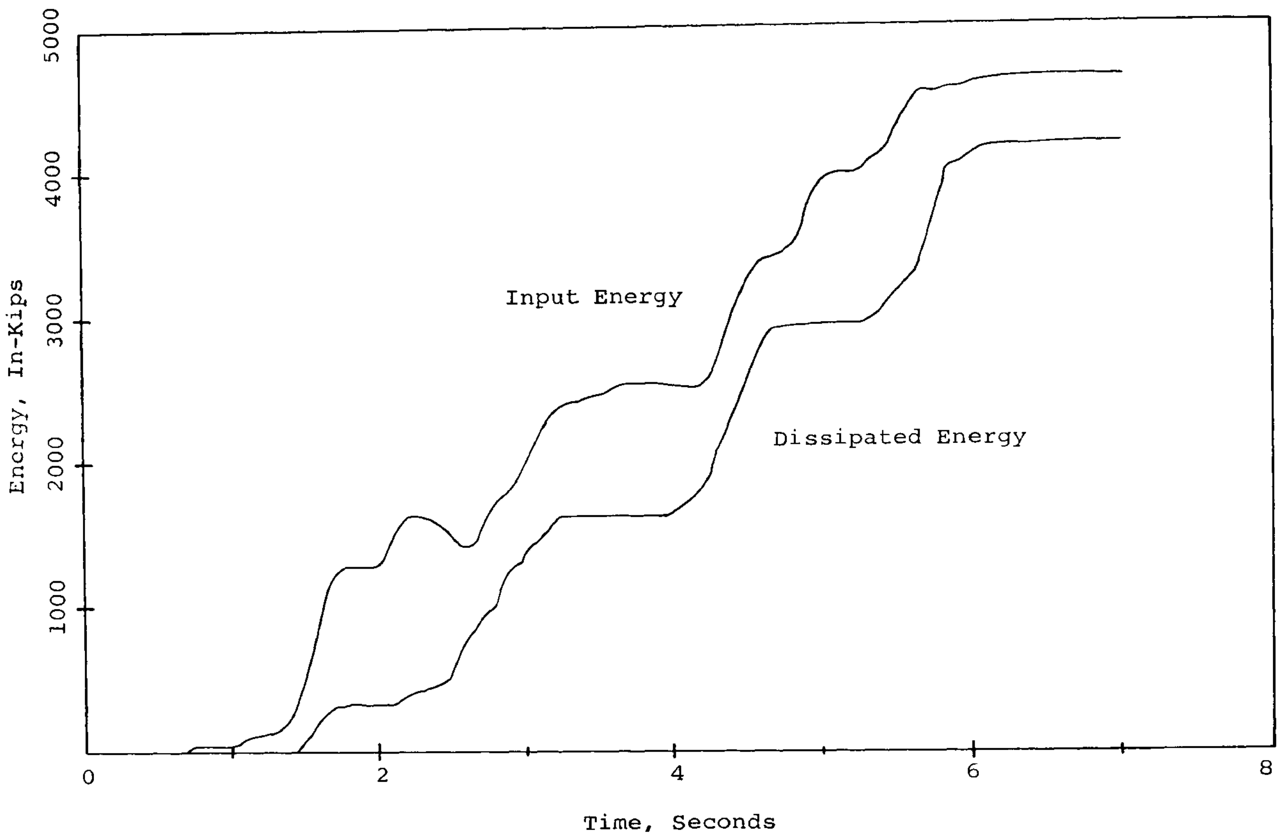


FIGURE A.1. Energy vs. time; structure B1; IU analysis

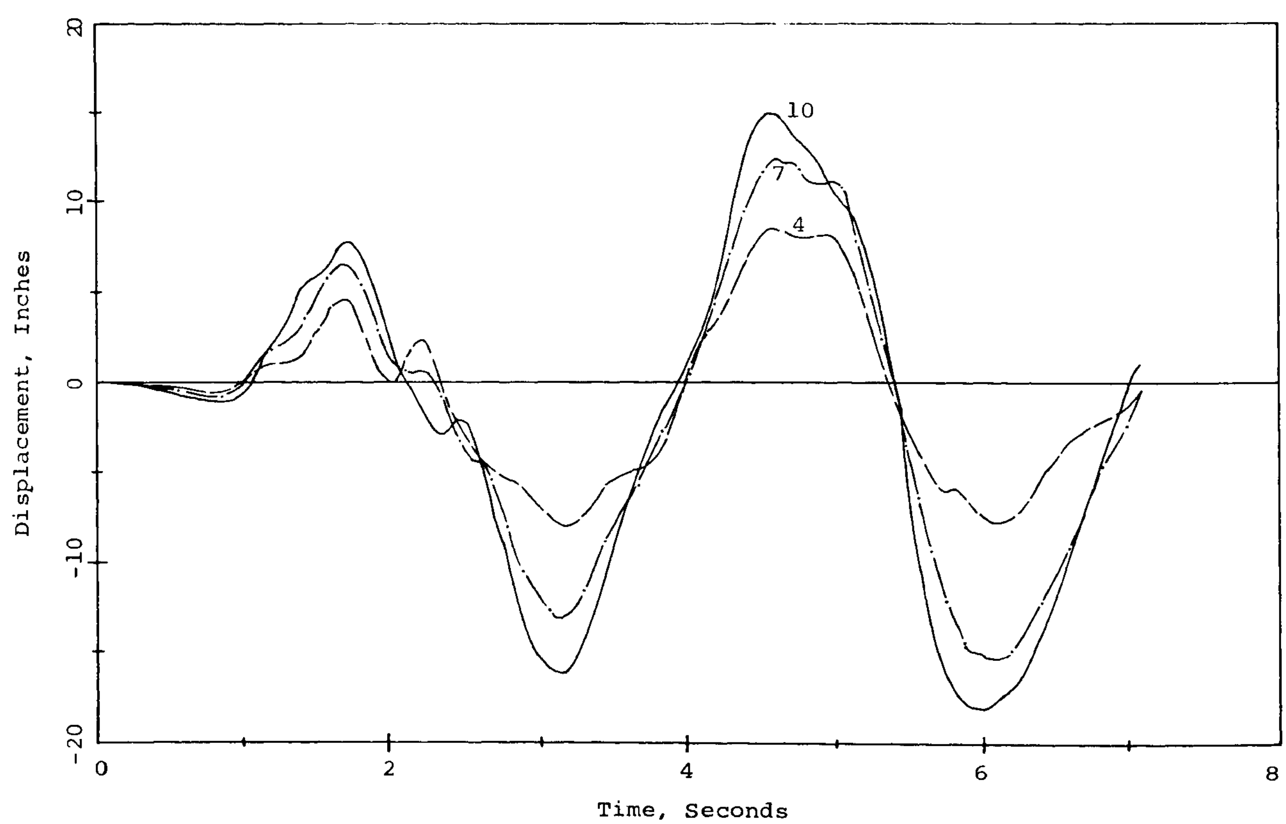


FIGURE A.2. Displacement vs. time; structure B1; IU analysis

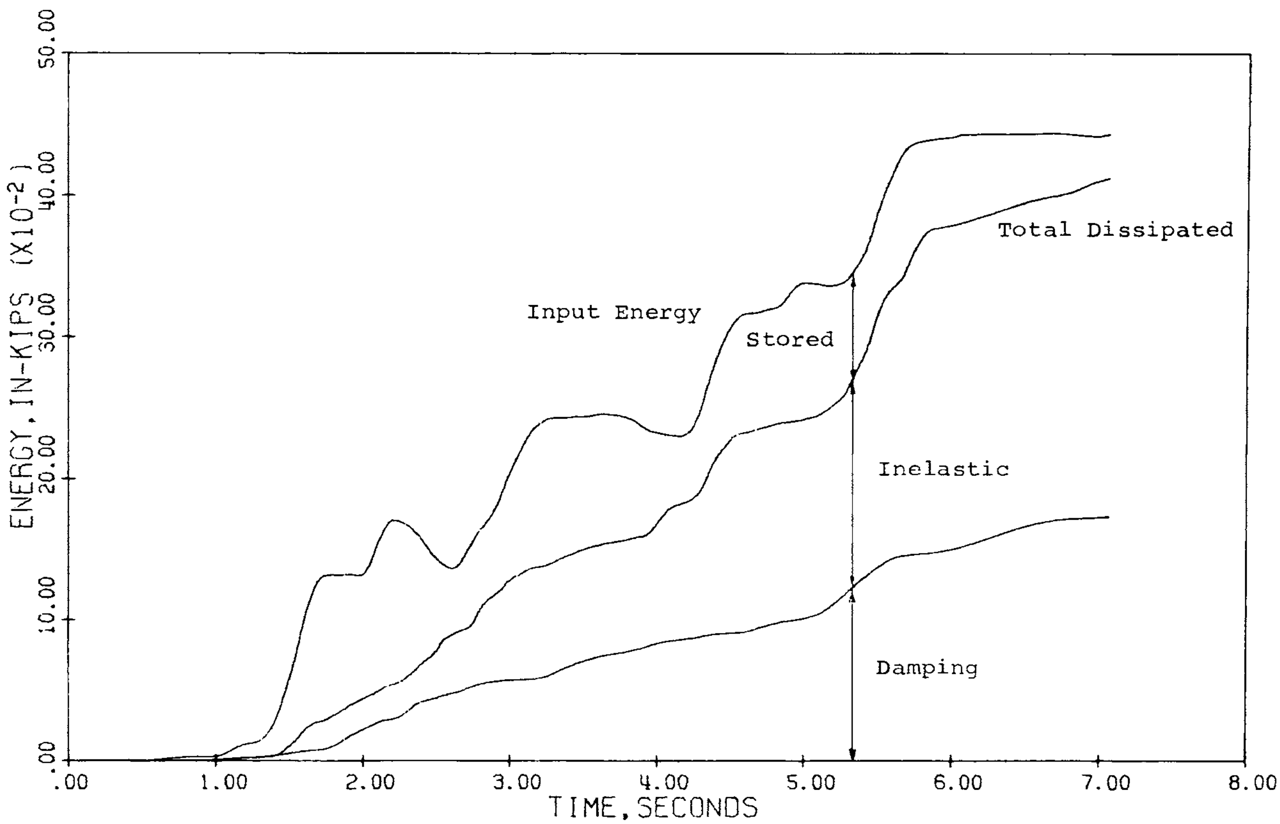


FIGURE A.3. Energy vs. time; structure B1; ID analysis

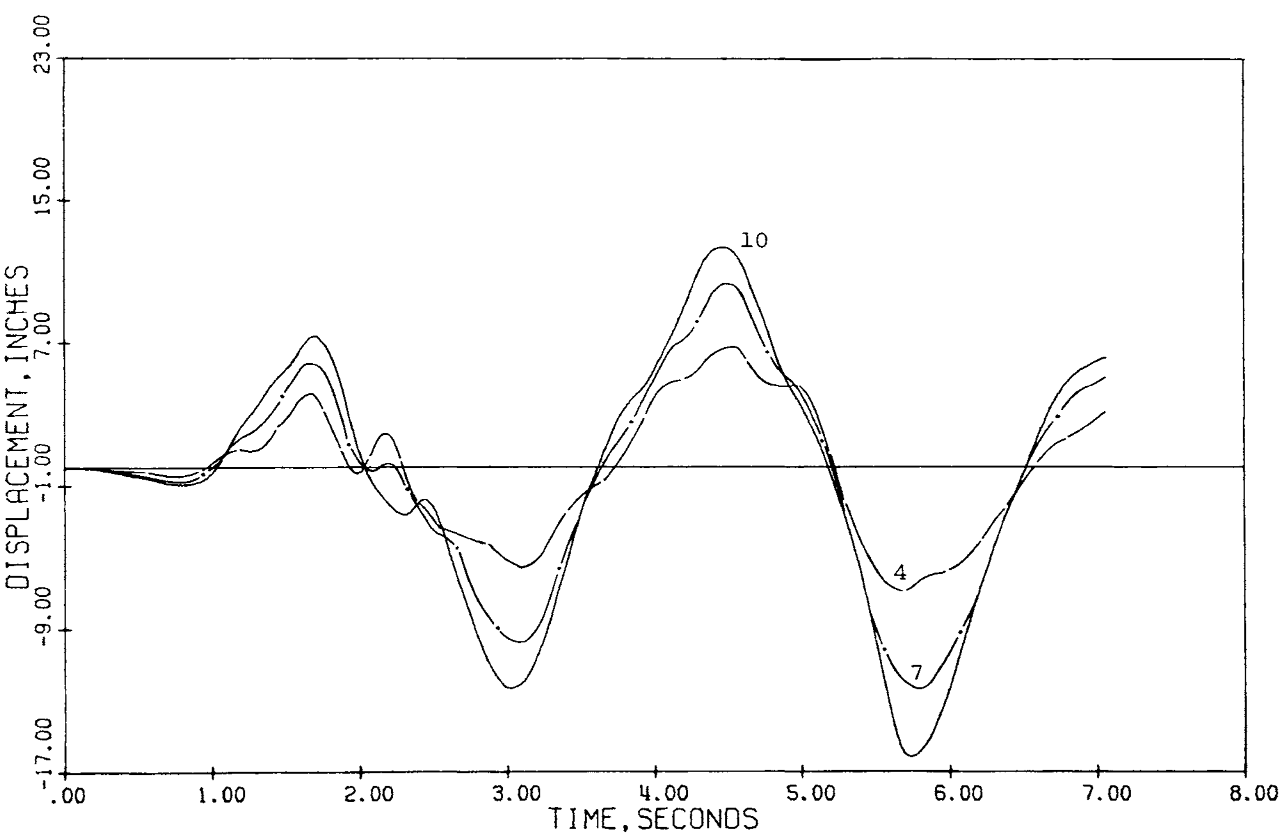


FIGURE A.4. Displacement vs. time; structure B1; ID analysis

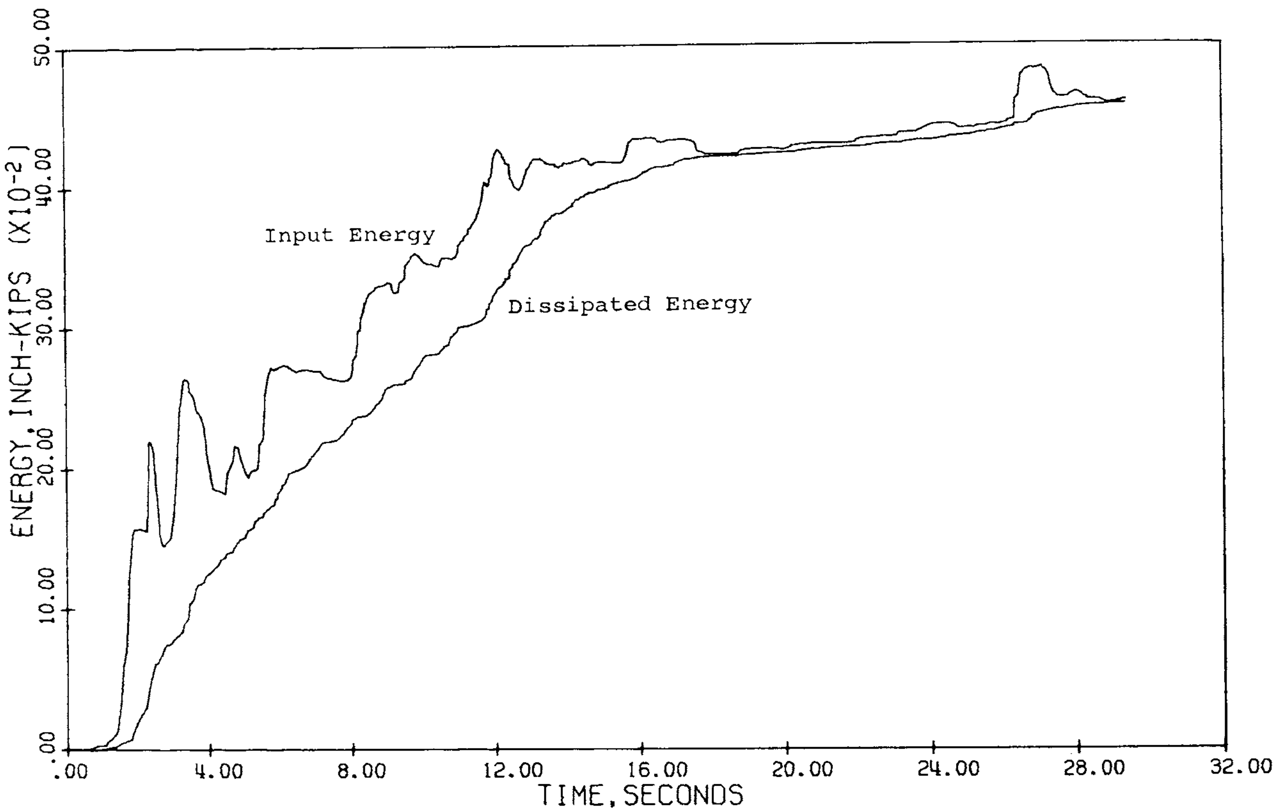


FIGURE A.5. Energy vs. time; structure B1; ED analysis

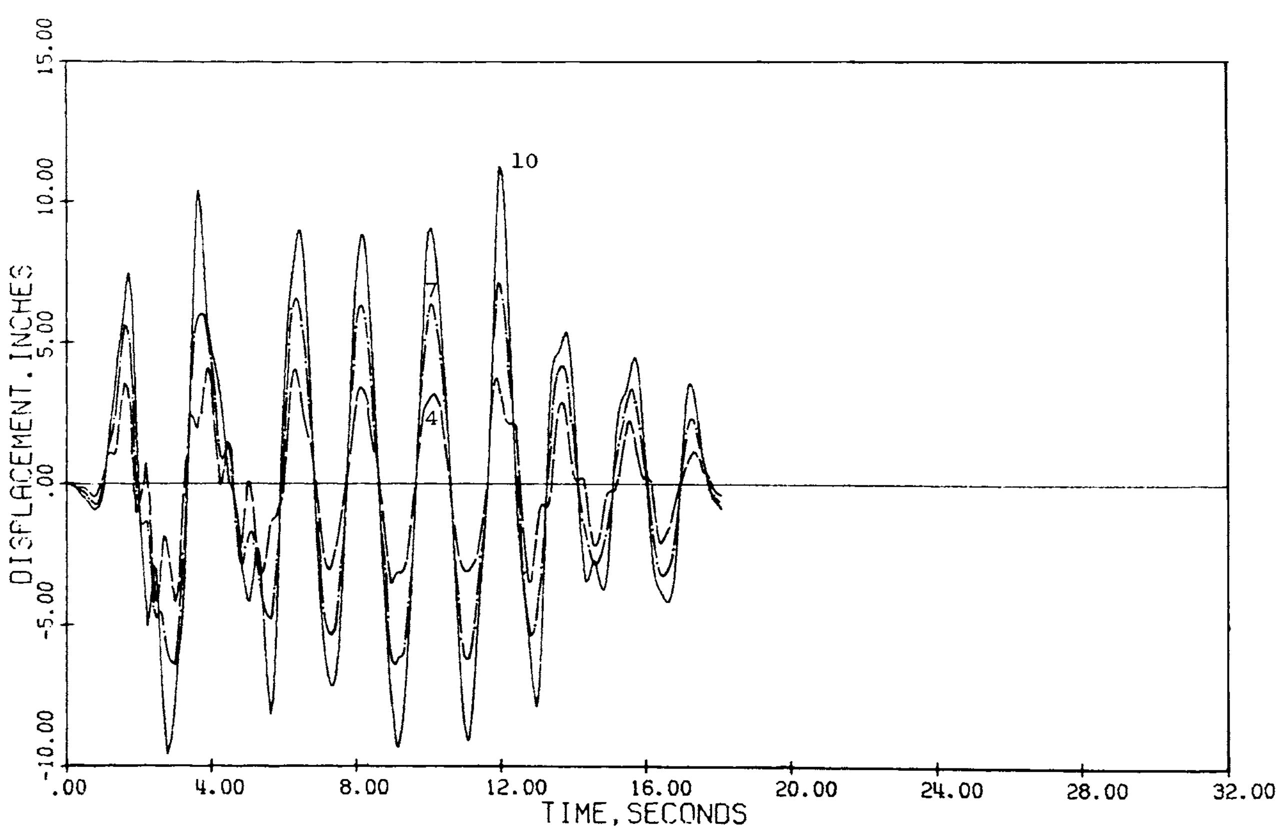


FIGURE A.6. Displacement vs. time; structure B1; ED analysis

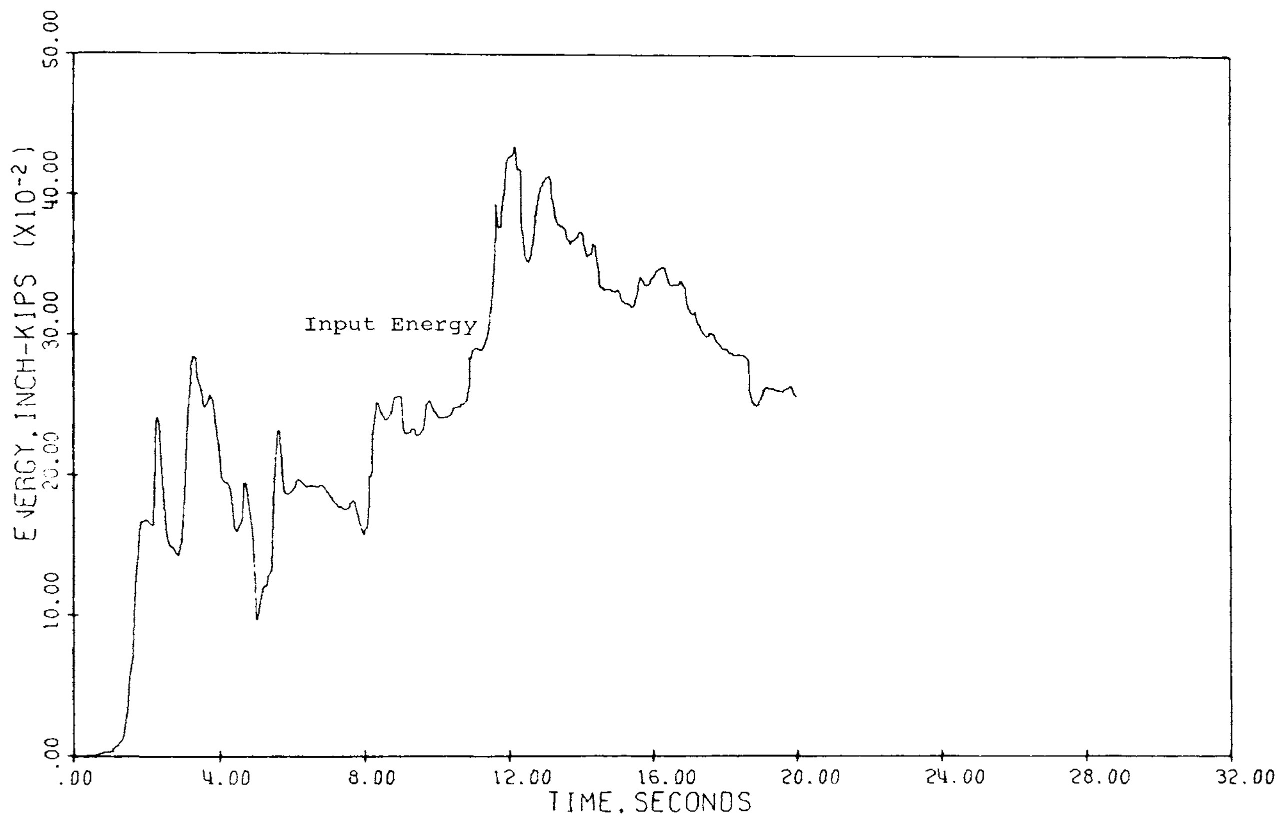


FIGURE A.7. Energy vs. time; structure B1; EU analysis

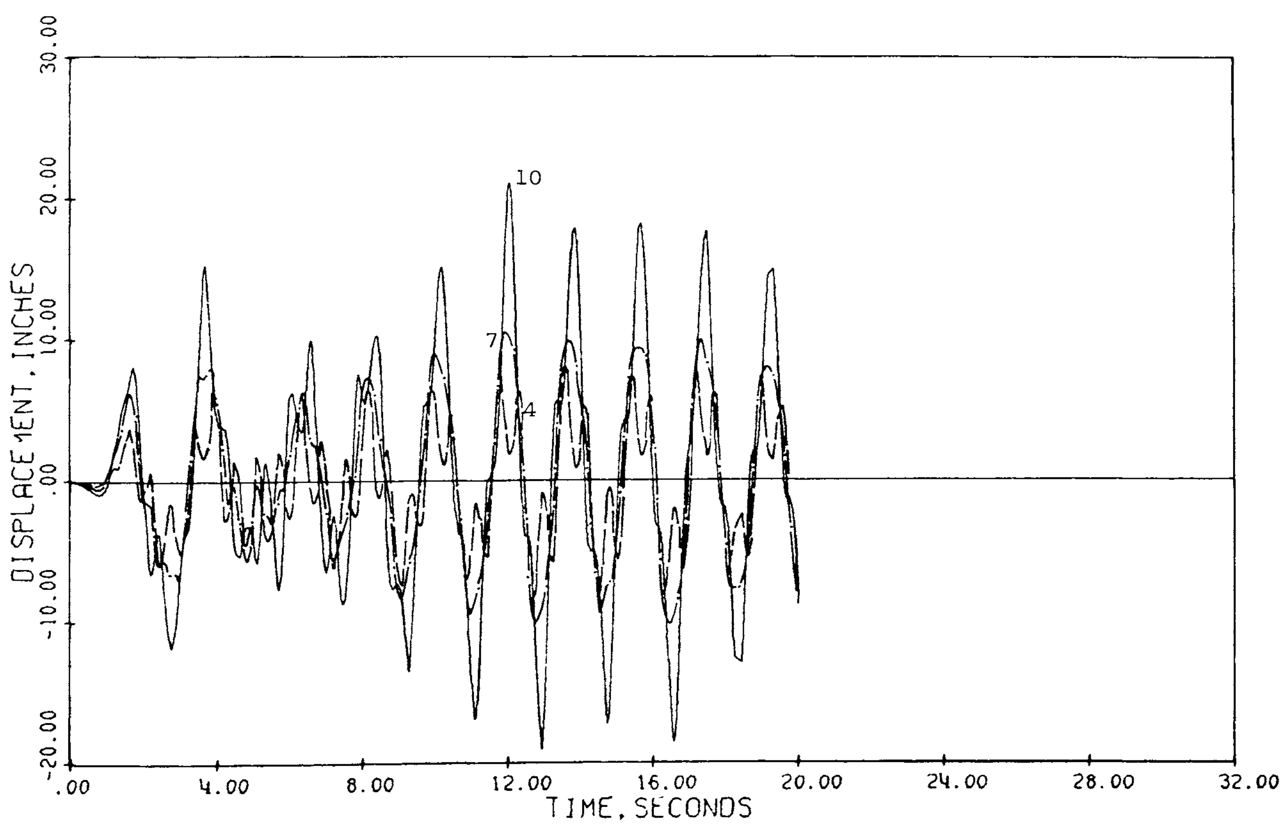


FIGURE A.8. Displacement vs. time; structure B1; EU analysis

APPENDIX B

Energy- and Displacement-Time Histories for Structure F1

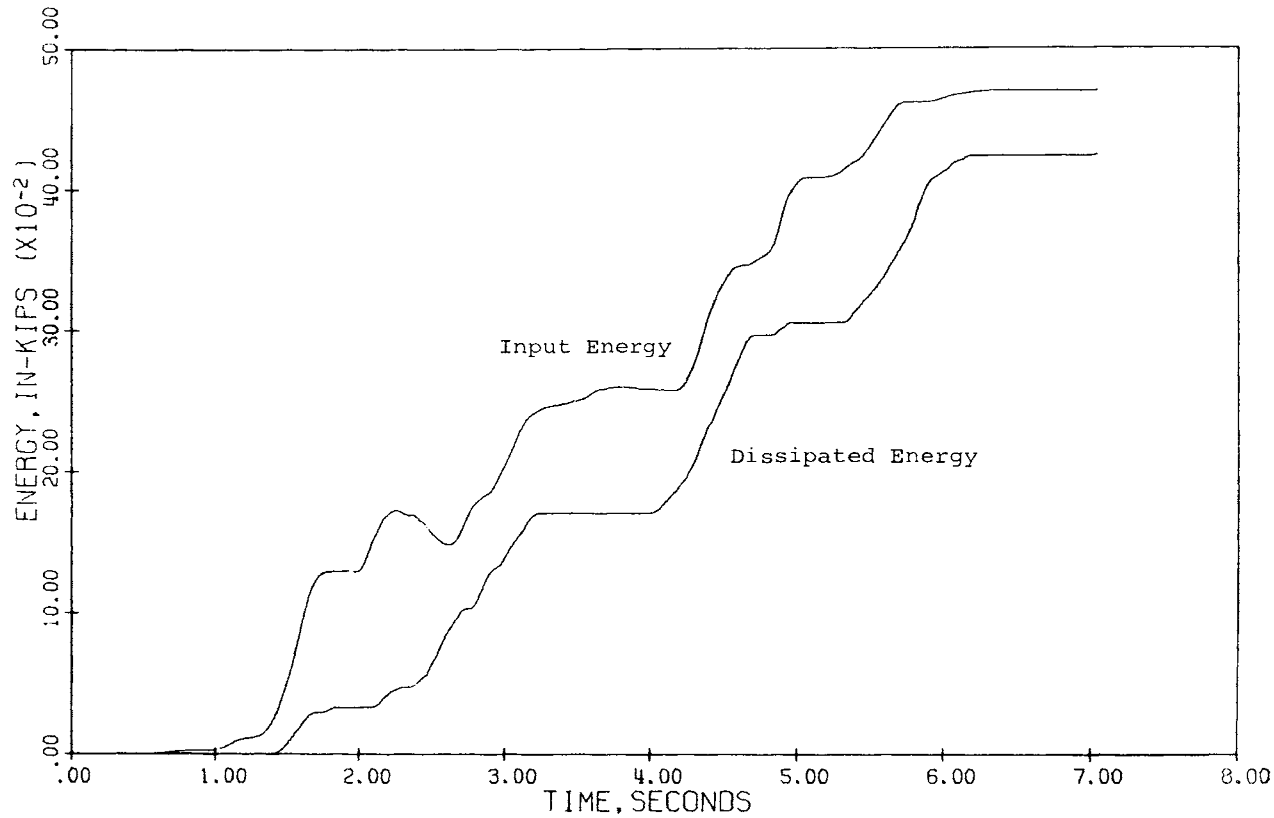


FIGURE B.1. Energy vs. time; structure F1; IU analysis

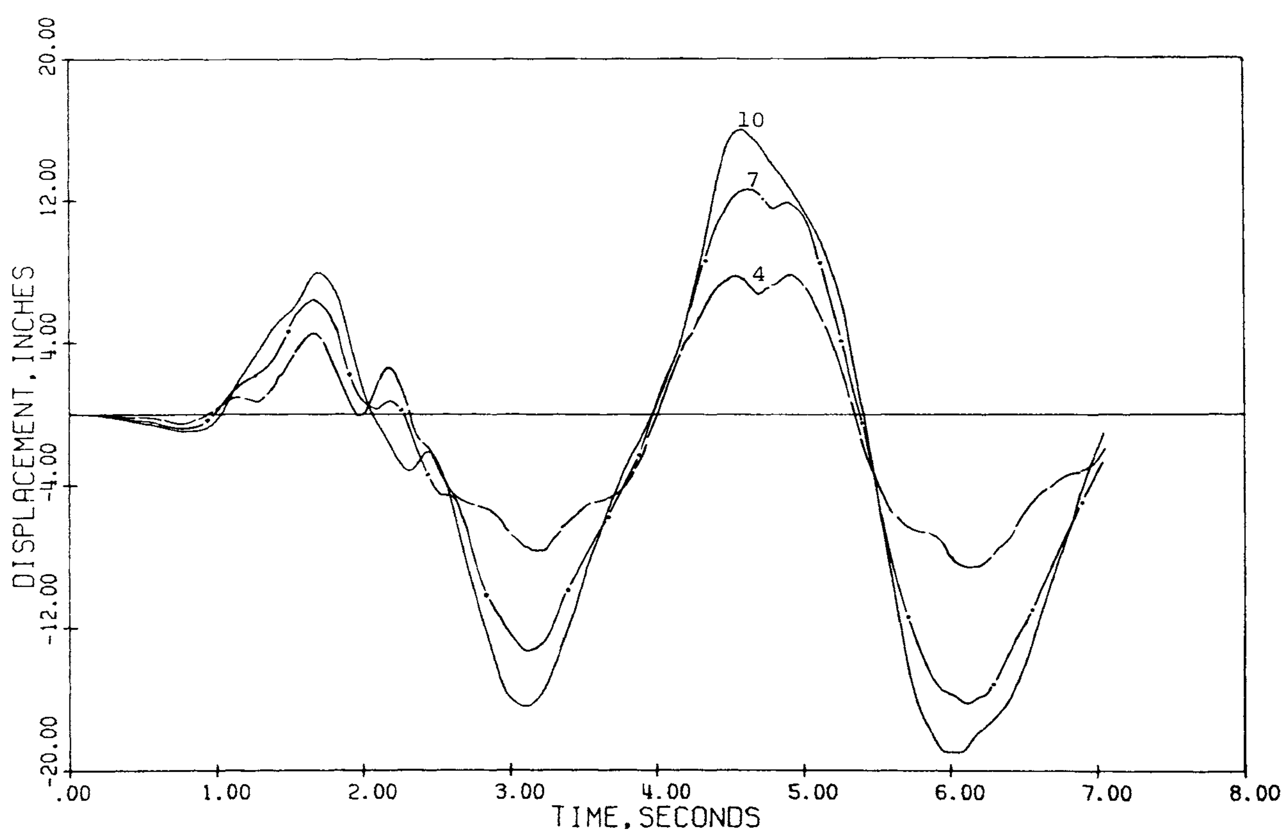


FIGURE B.2. Displacement vs. time; structure F1; IU analysis

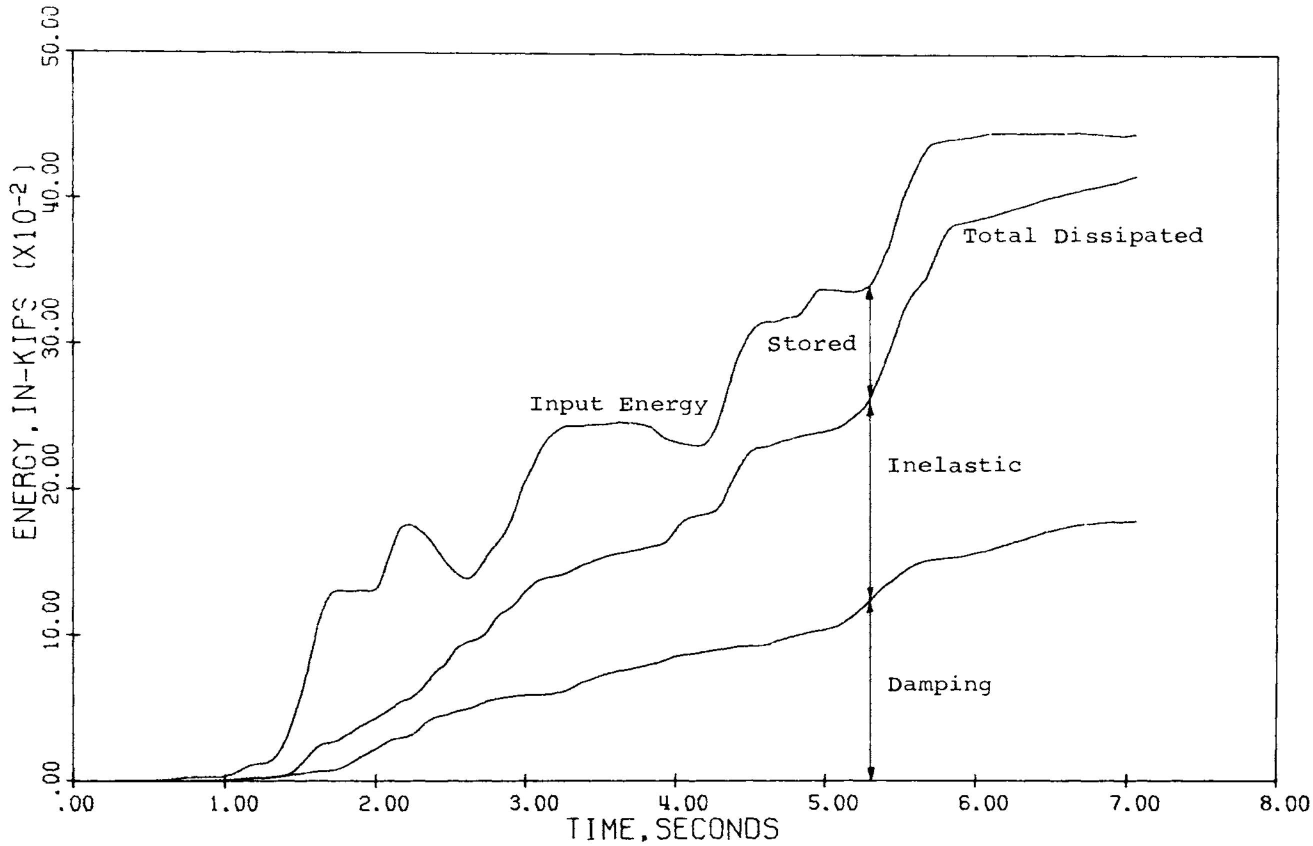


FIGURE B.3. Energy vs. time; structure F1; ID analysis

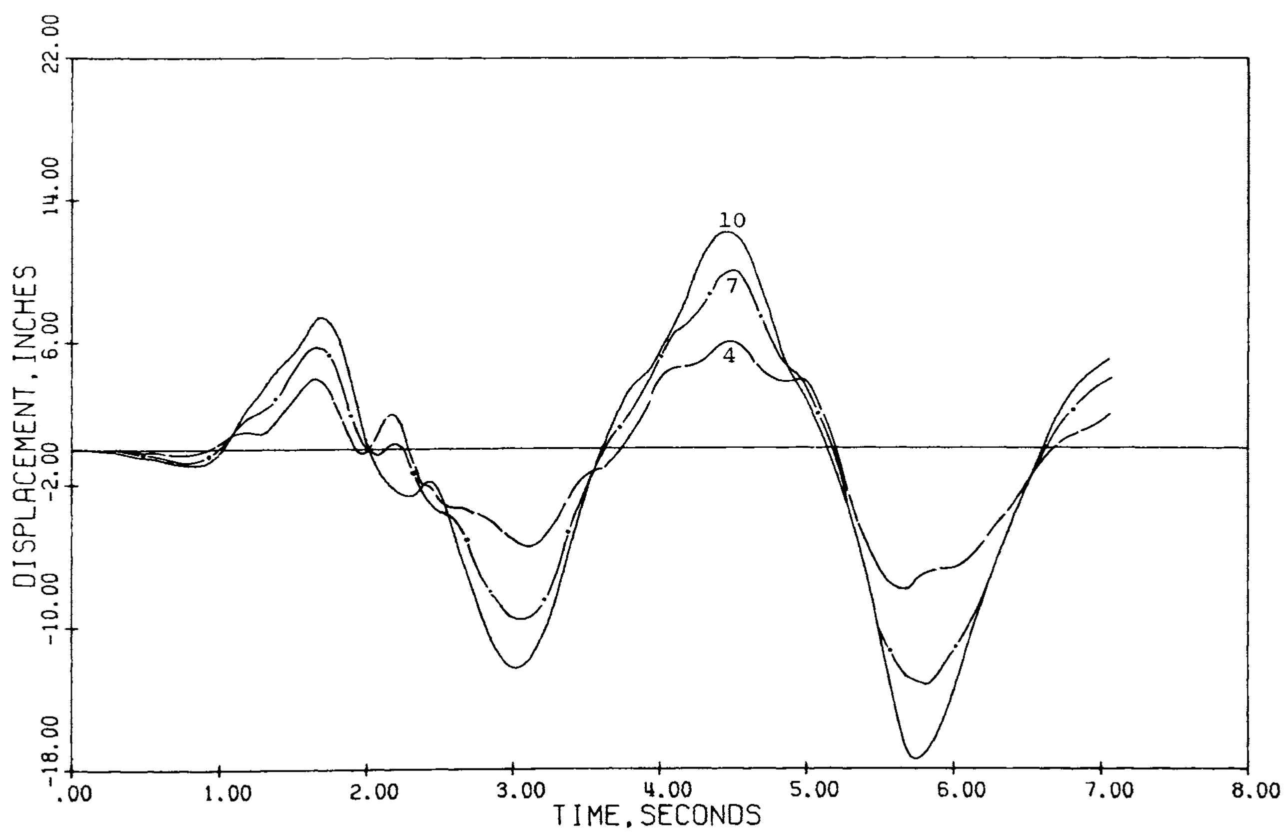


FIGURE B.4. Displacement vs. time; structure F1; ID analysis

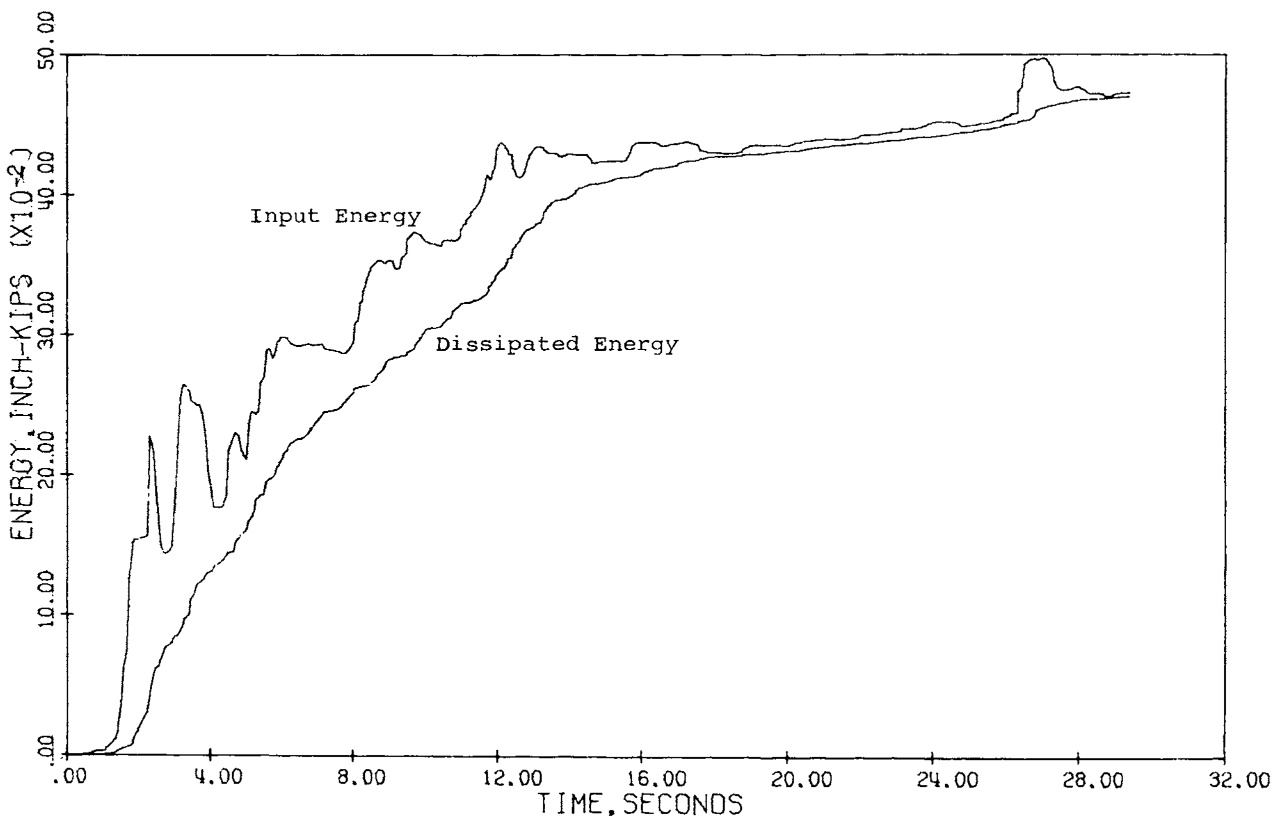


FIGURE B.5. Energy vs. time; structure F1; ED analysis

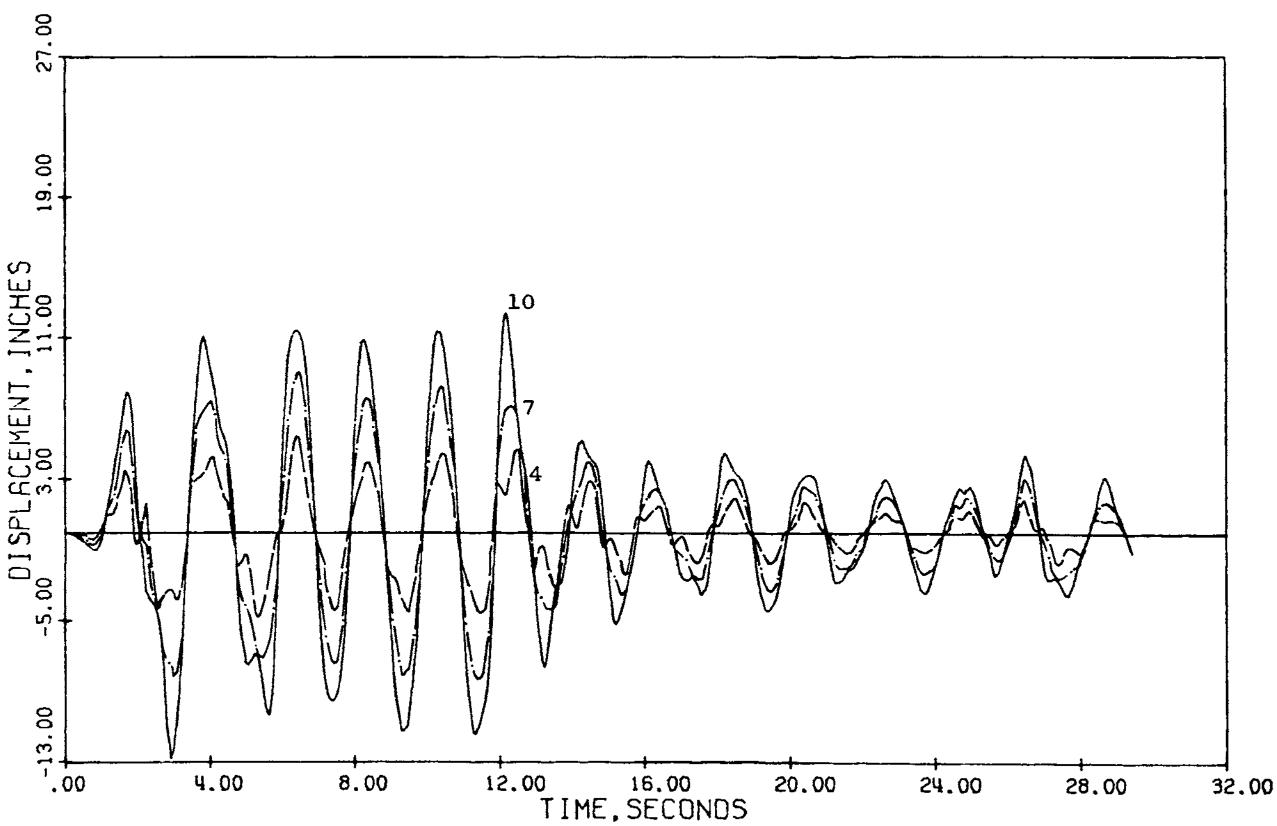


FIGURE B.6. Displacement vs. time; structure F1; ED analysis

APPENDIX C

Energy- and Displacement-Time Histories
for
Structure A1

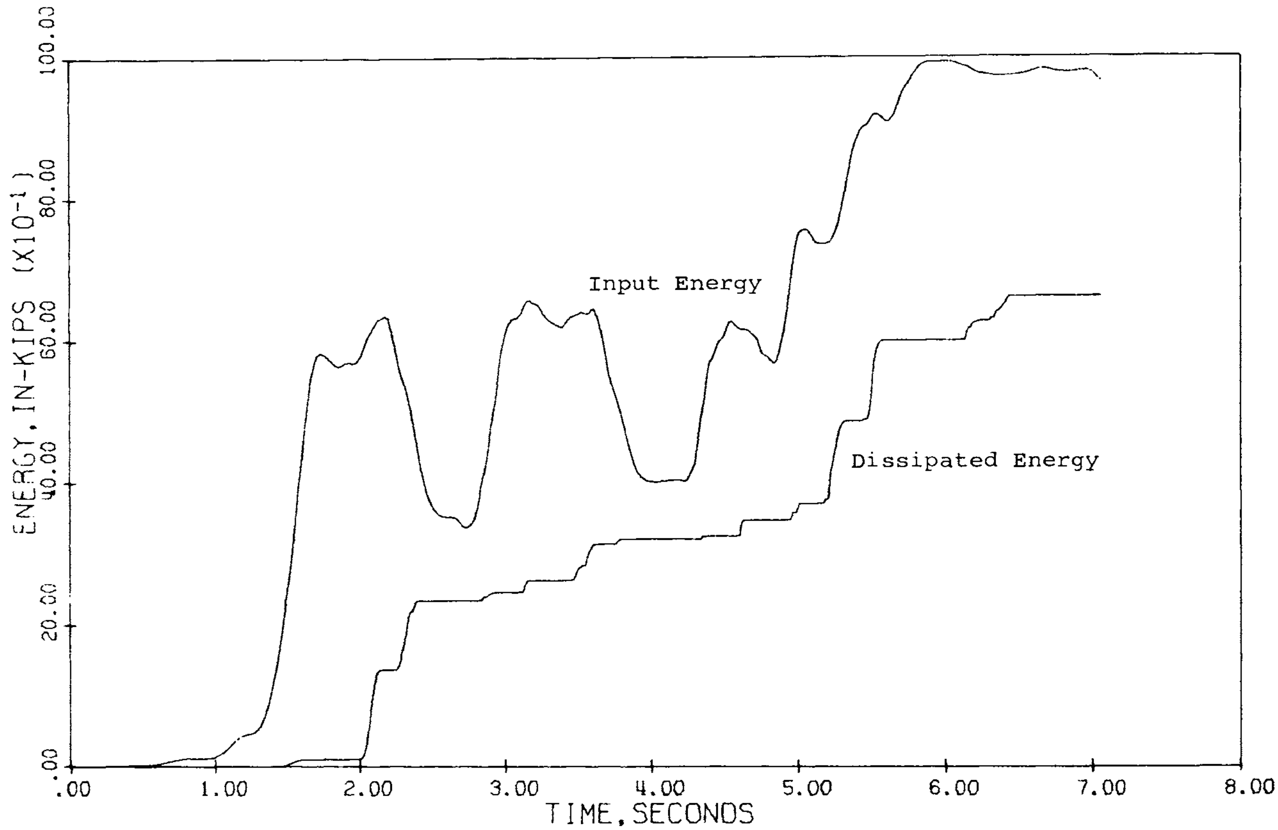


FIGURE C.1. Energy vs. time; structure A1; IU analysis

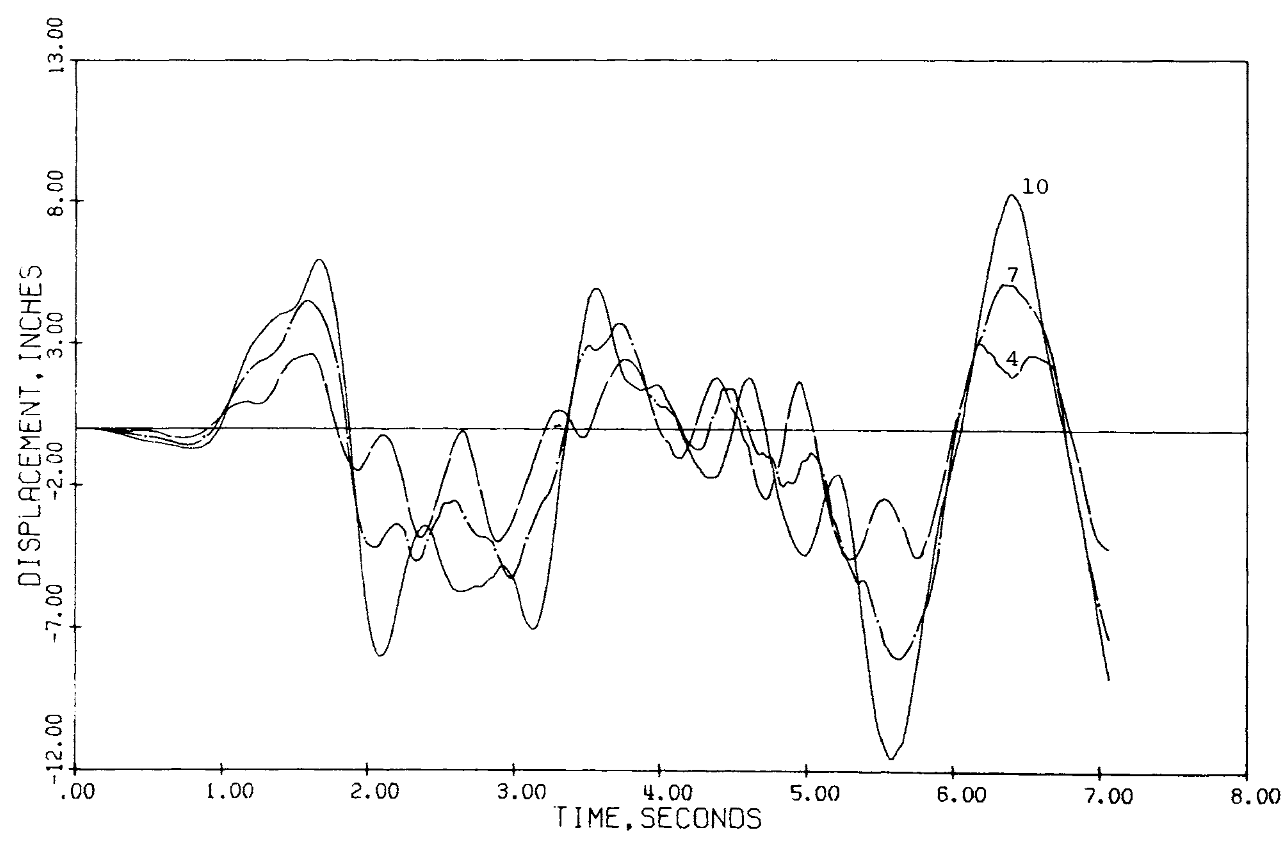


FIGURE C.2. Displacement vs. time; structure A1; IU analysis

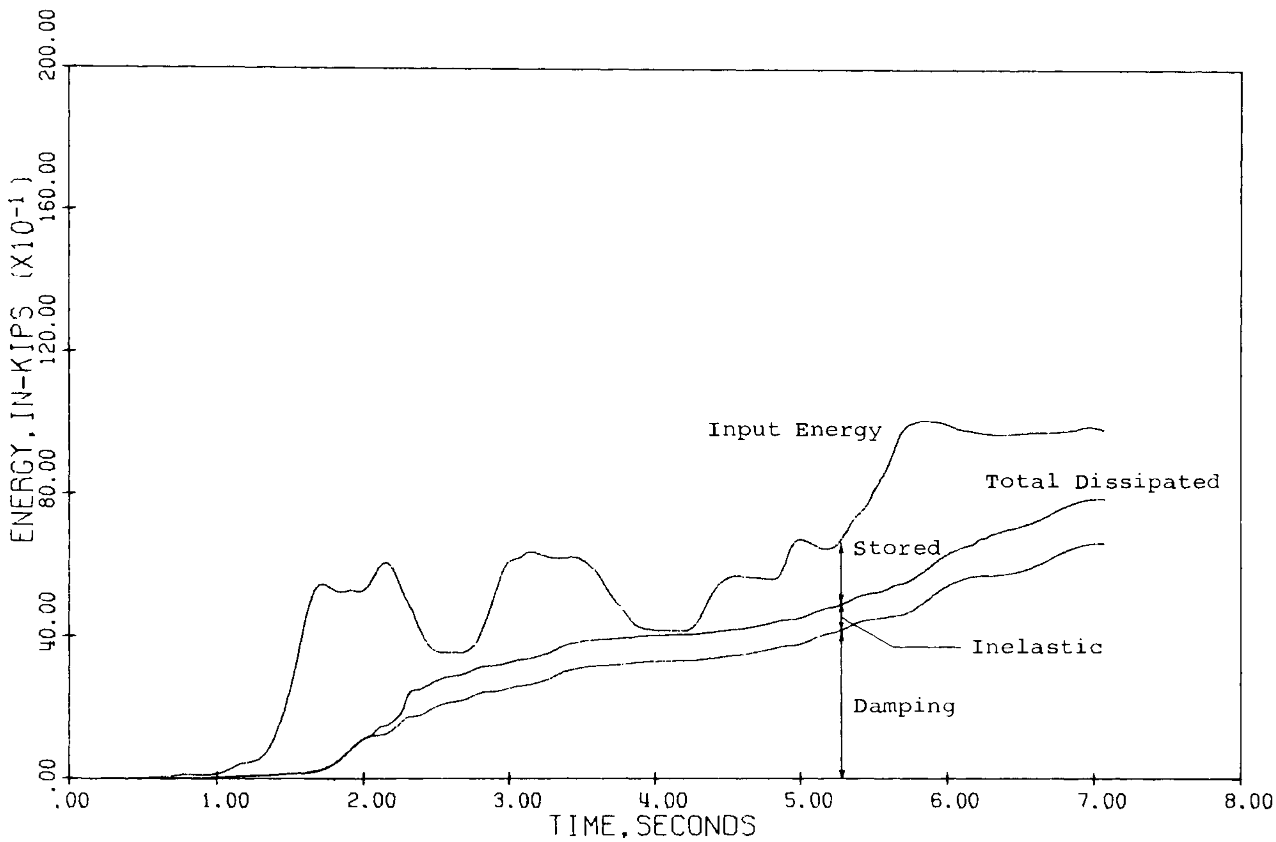


FIGURE C.3. Energy vs. time; structure A1; ID analysis

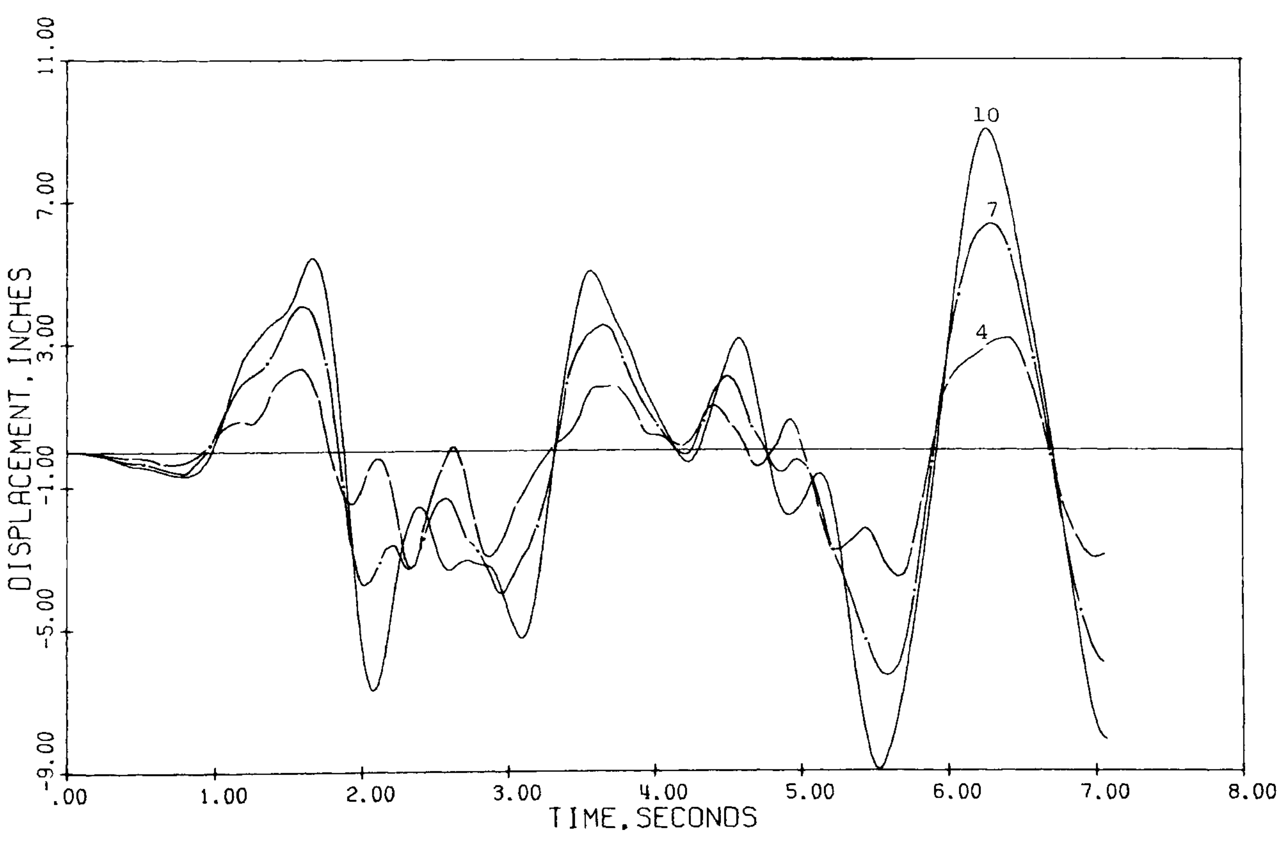


FIGURE C.4. Displacement vs. time; structure A1; ID analysis

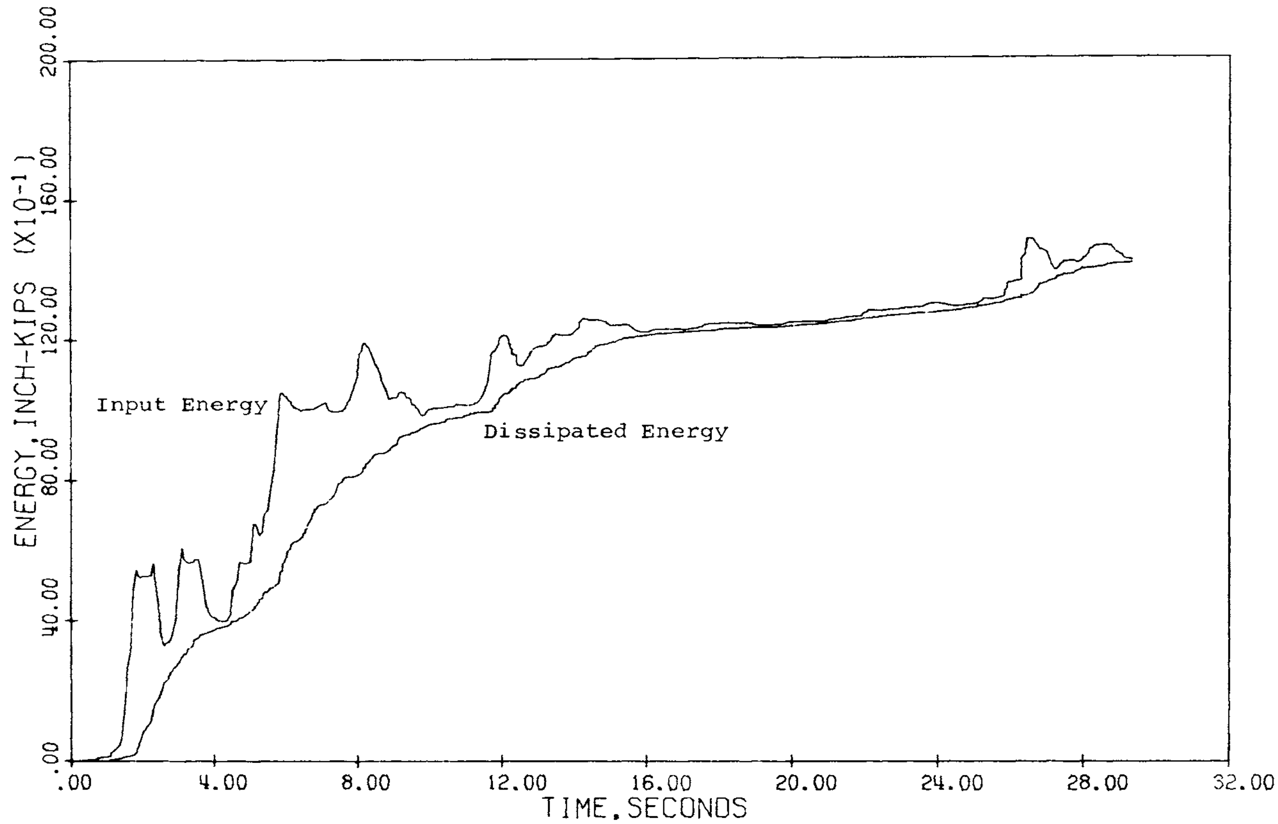


FIGURE C.5. Energy vs. time; structure A1; ED analysis

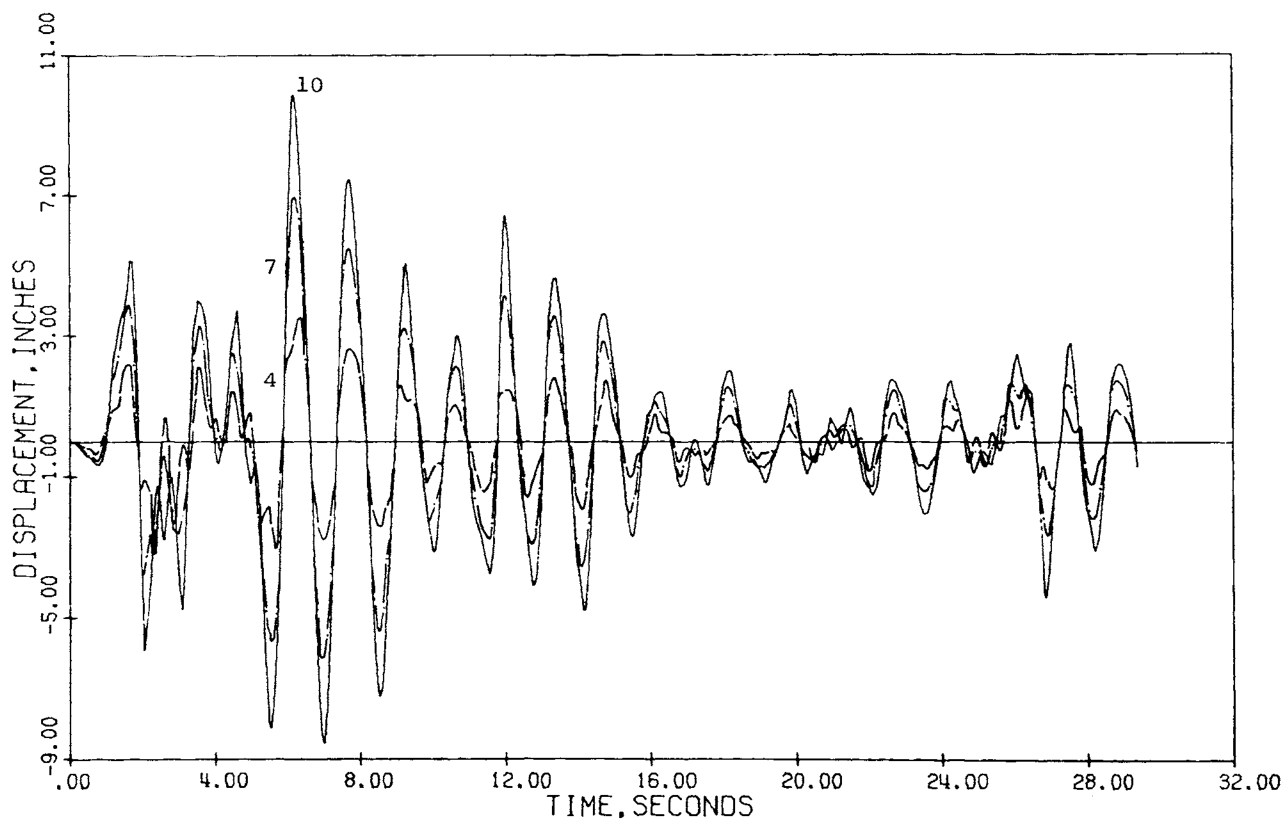


FIGURE C.6. Displacement vs. time; structure A1; ED analysis

APPENDIX D

Energy- and Displacement-Time Histories for Structure U1

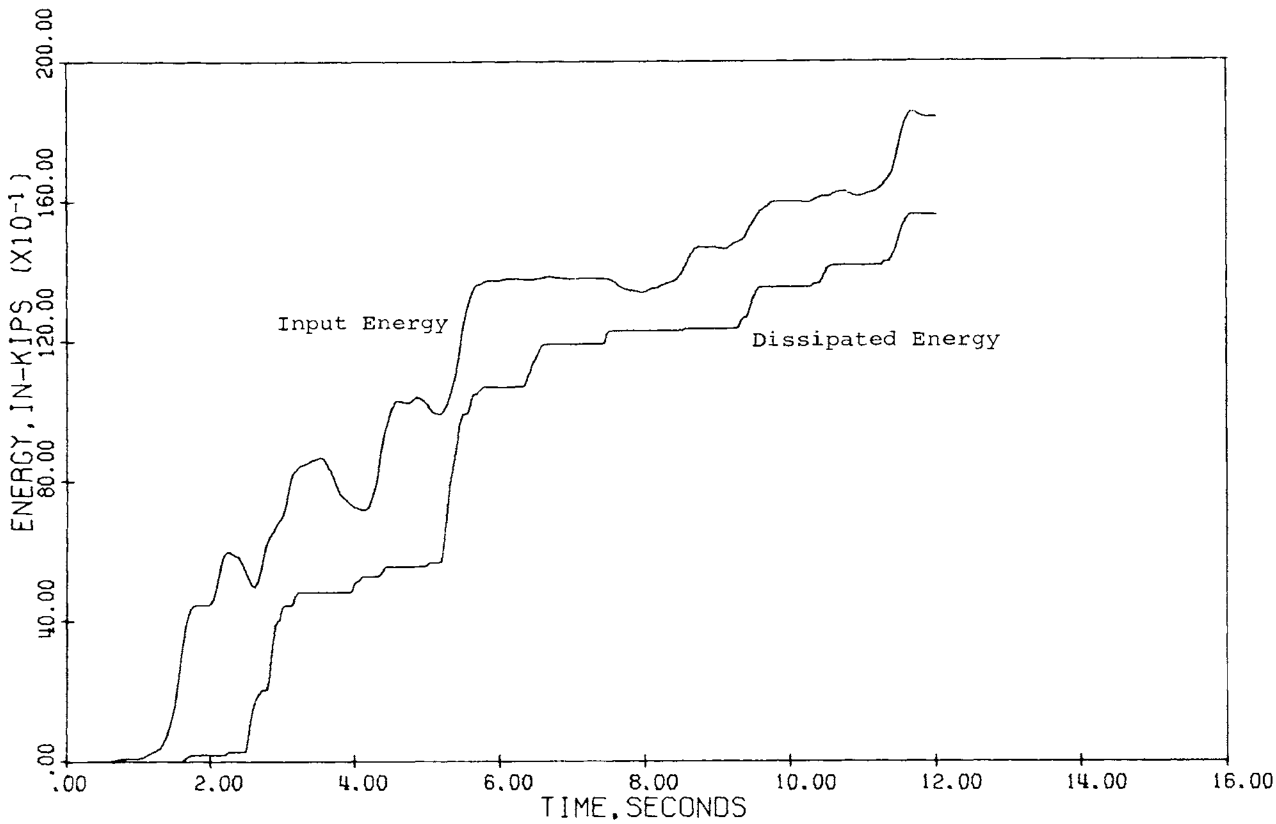


FIGURE D.1. Energy vs. time; structure U1; IU analysis

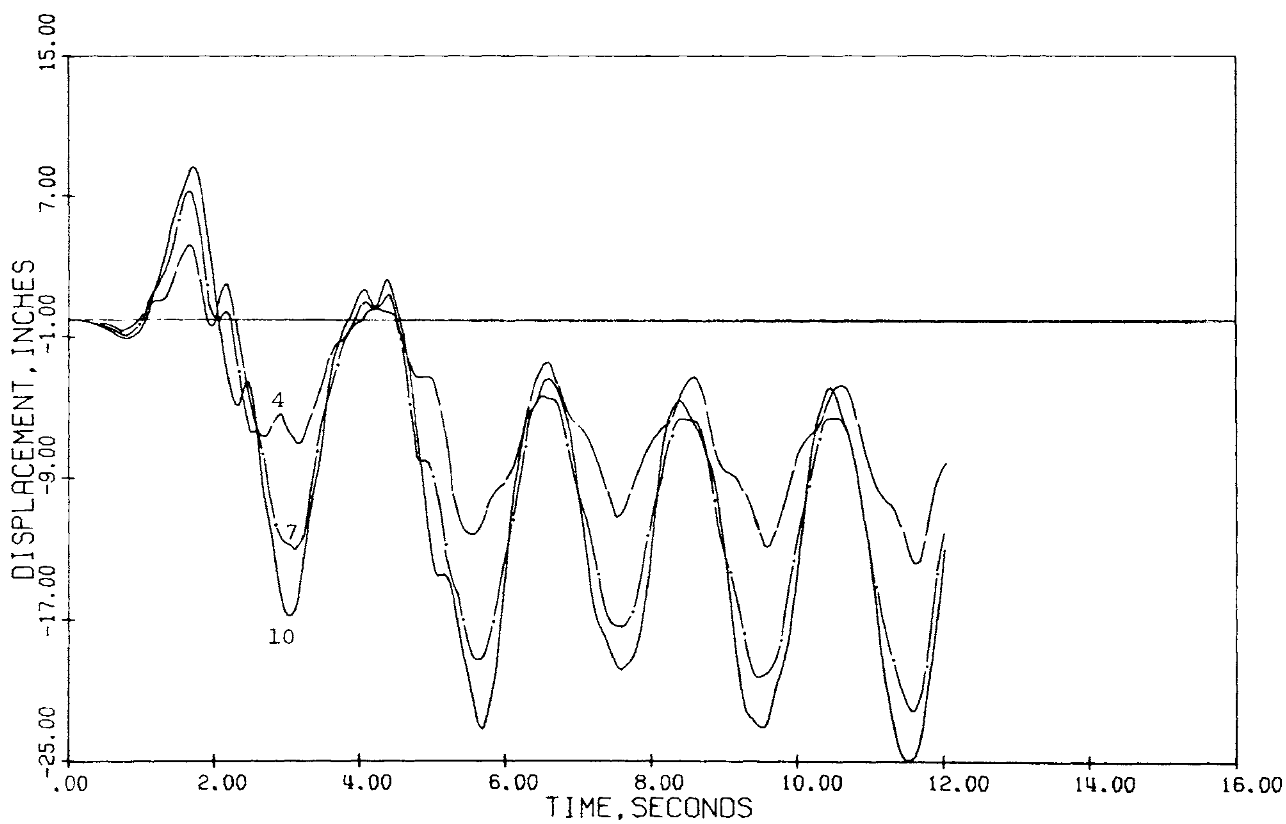


FIGURE D.2. Displacement vs. time; structure U1; IU analysis

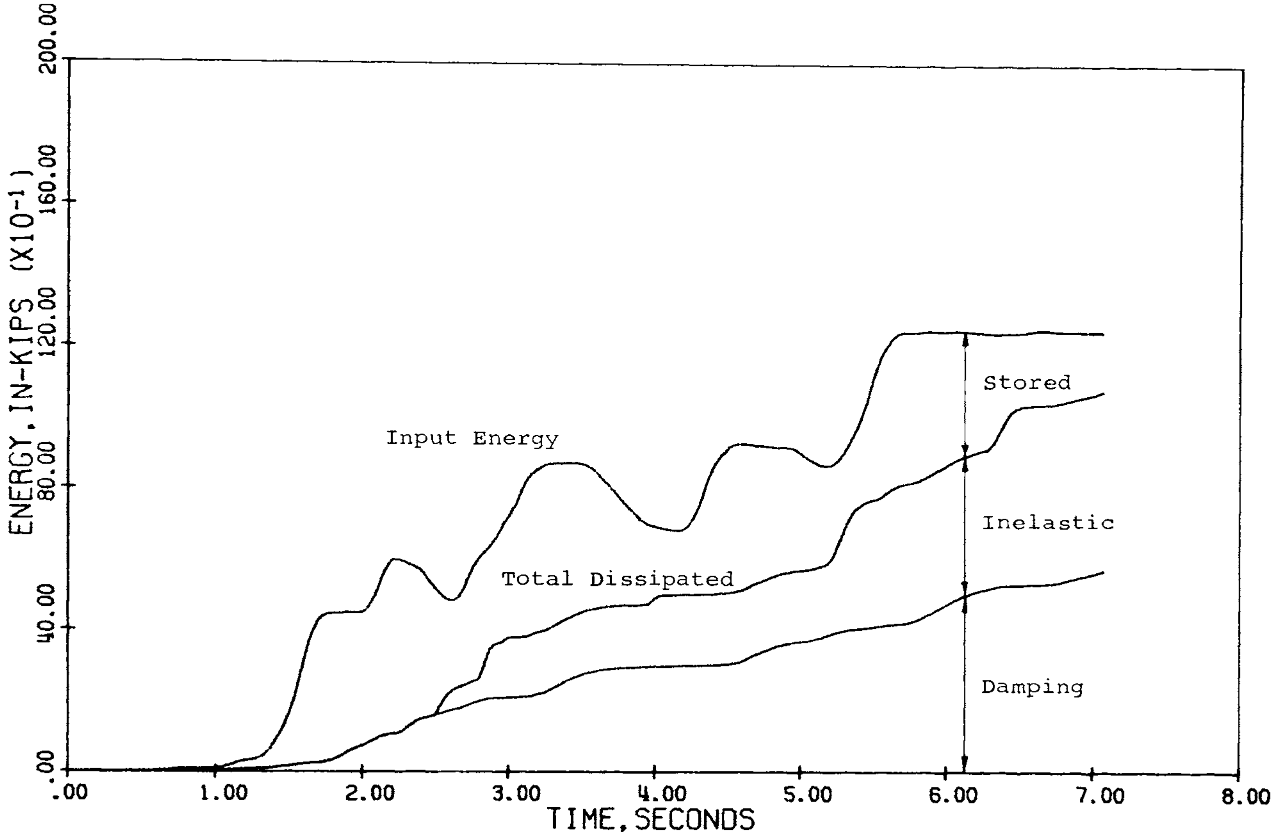


FIGURE D.3. Energy vs. time; structure U1; ID analysis

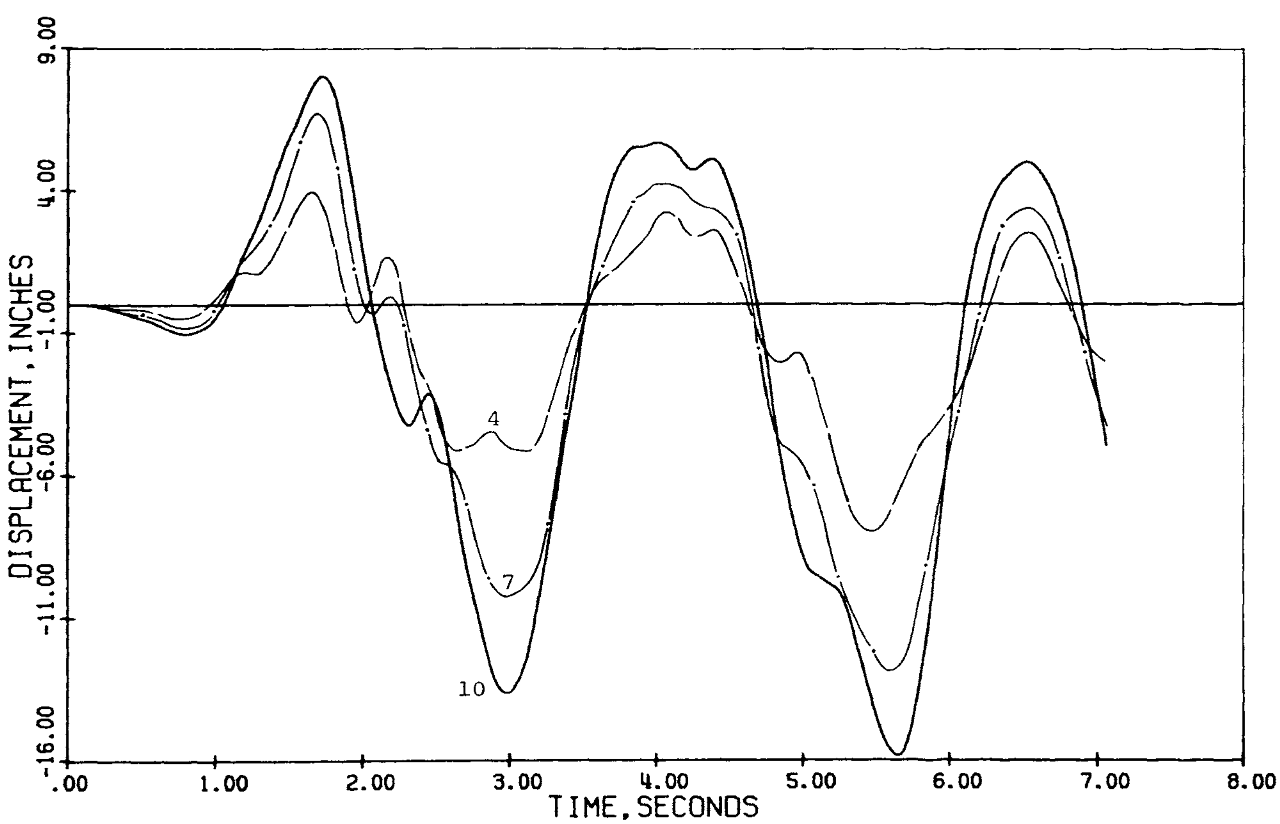


FIGURE D.4. Displacement vs. time; structure U1; ID analysis

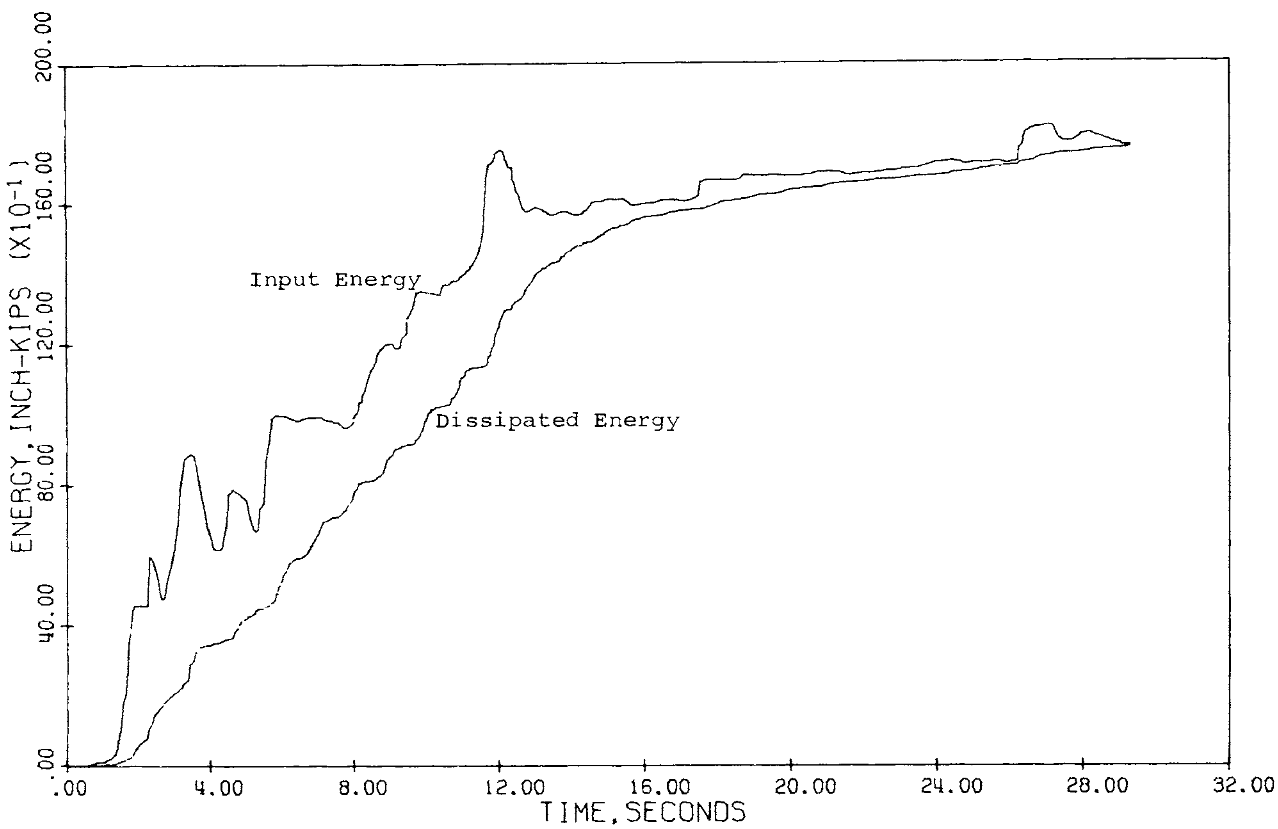


FIGURE D.5. Energy vs. time; structure U1; ED analysis

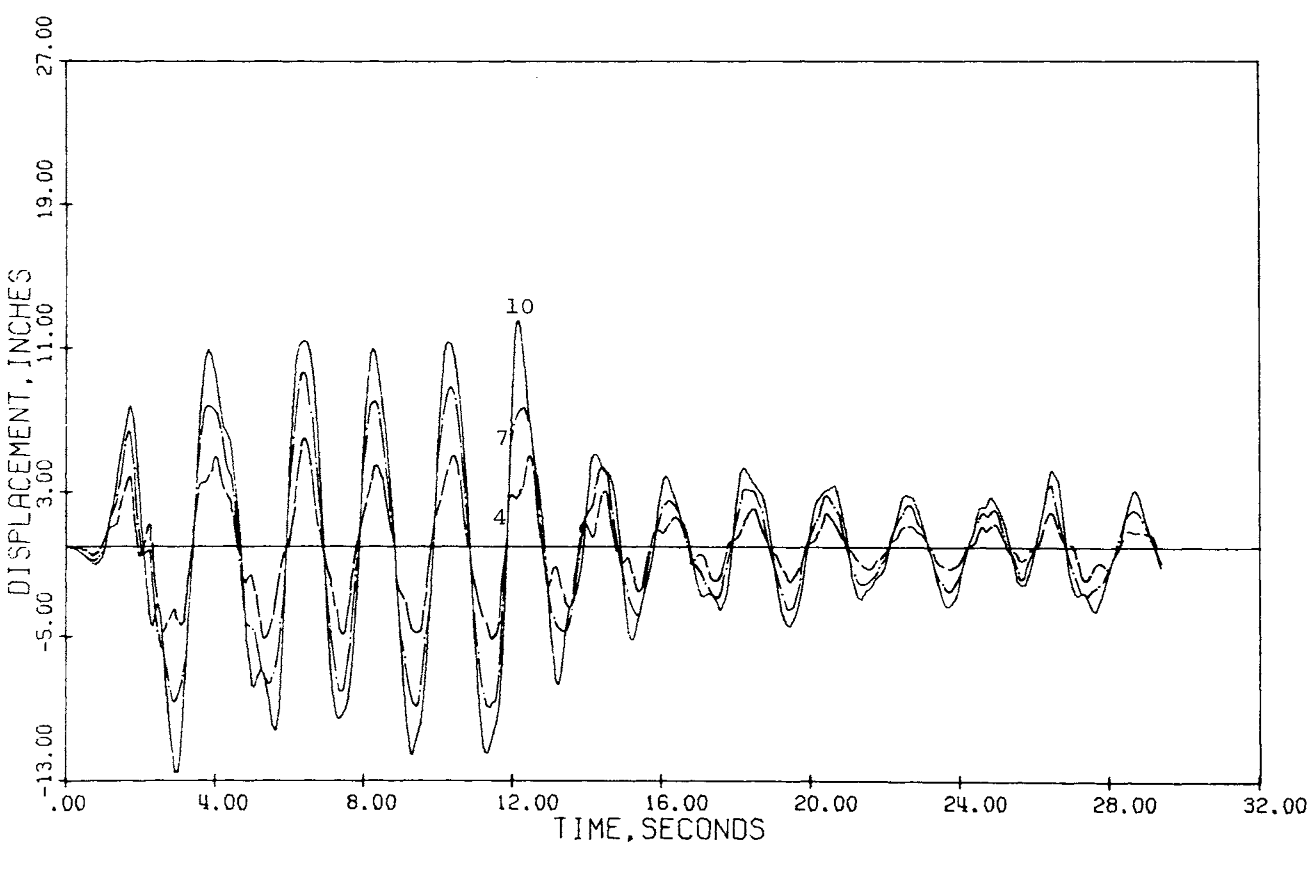


FIGURE D.6. Displacement vs. time; structure U1; ED analysis

APPENDIX E

Energy- and Displacement-Time Histories for Structure U1X

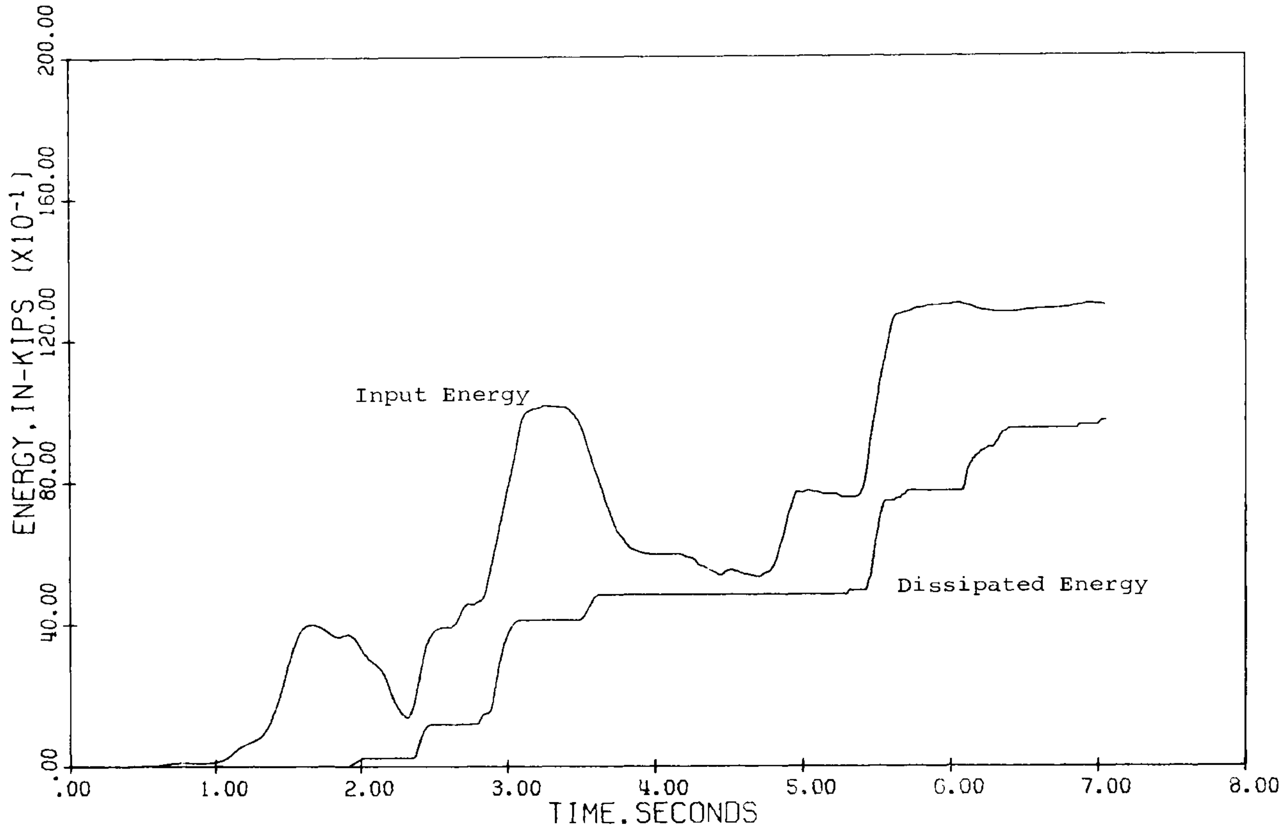


FIGURE E.1. Energy vs. time; structure U1X; IU analysis

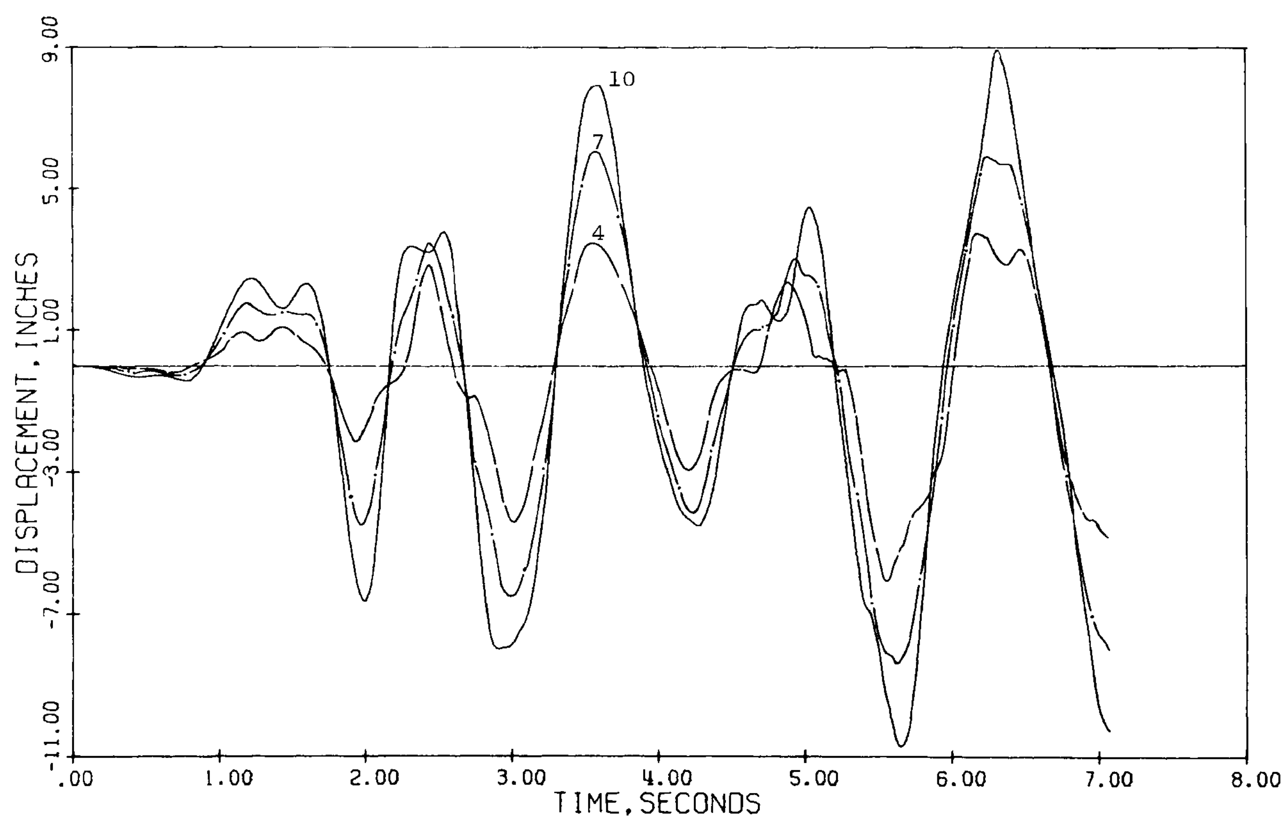


FIGURE E.2. Displacement vs. time; structure U1X; IU analysis

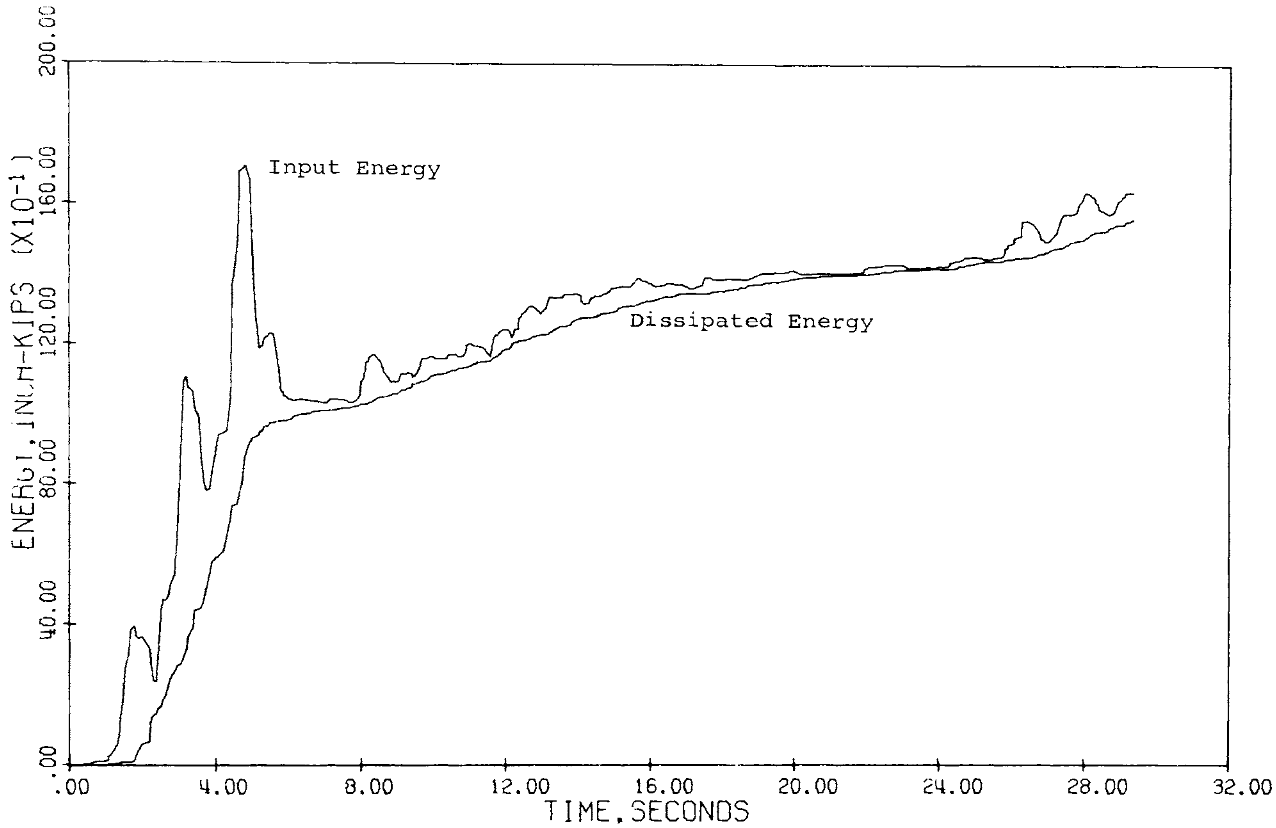


FIGURE E.3. Energy vs. time; structure U1X; ED analysis

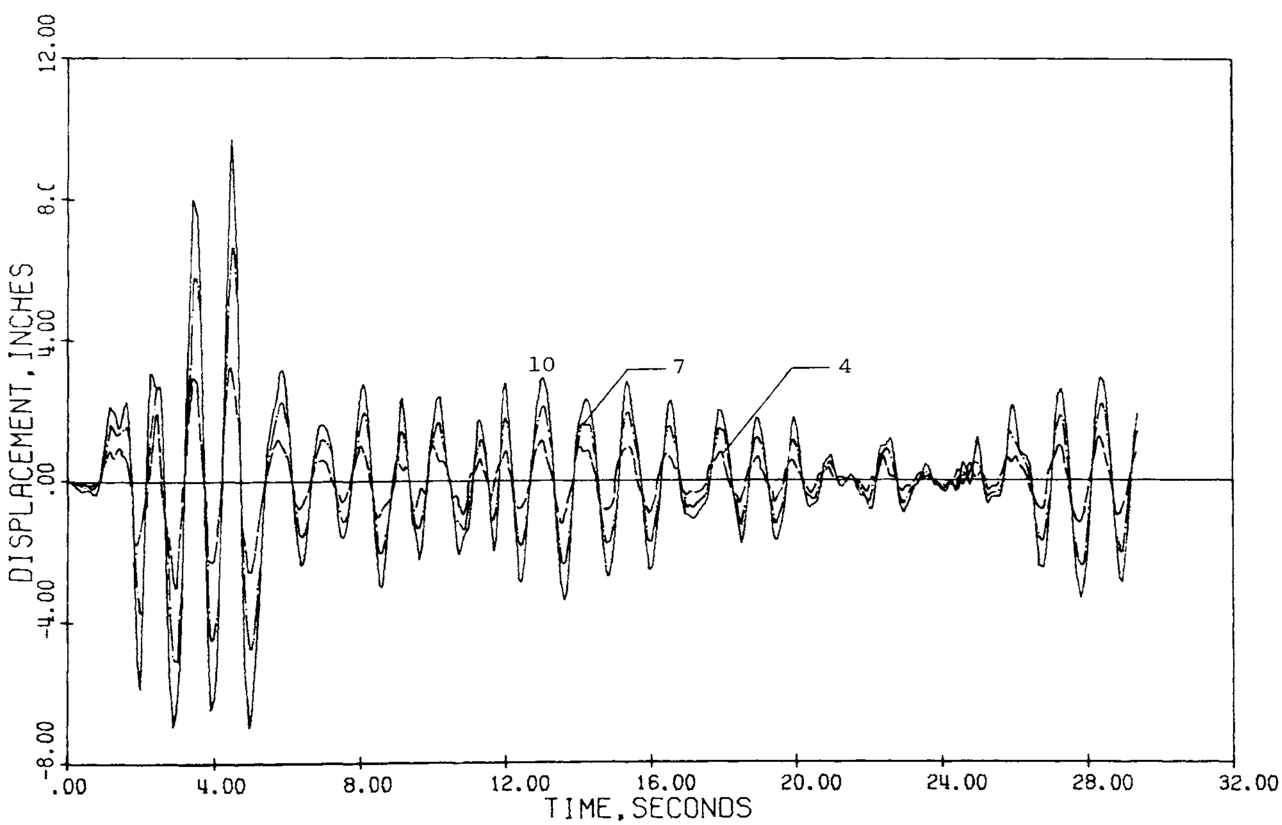


FIGURE E.4. Displacement vs. time; structure U1X; ED analysis

APPENDIX F

Energy- and Displacement-Time Histories
for
Structure F2

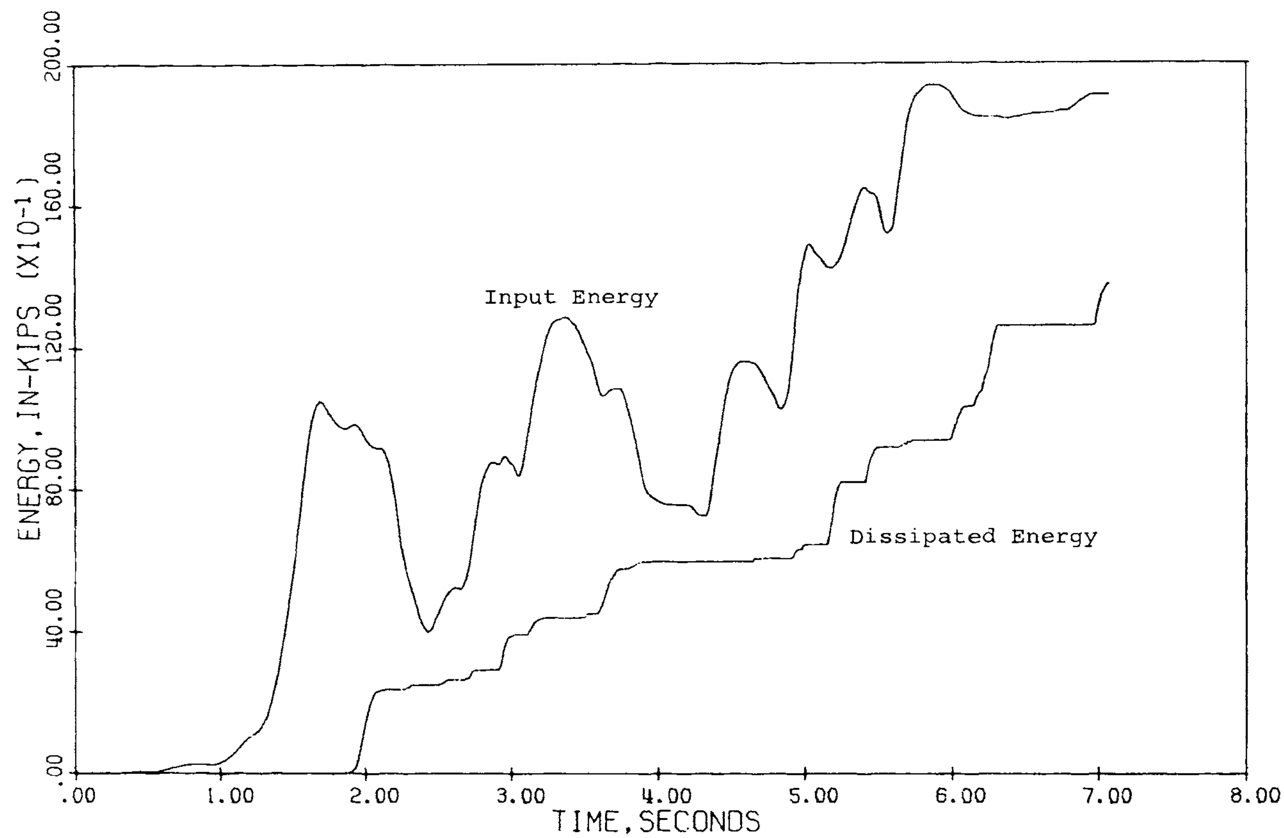


FIGURE F.1. Energy vs. time; structure F2; IU analysis

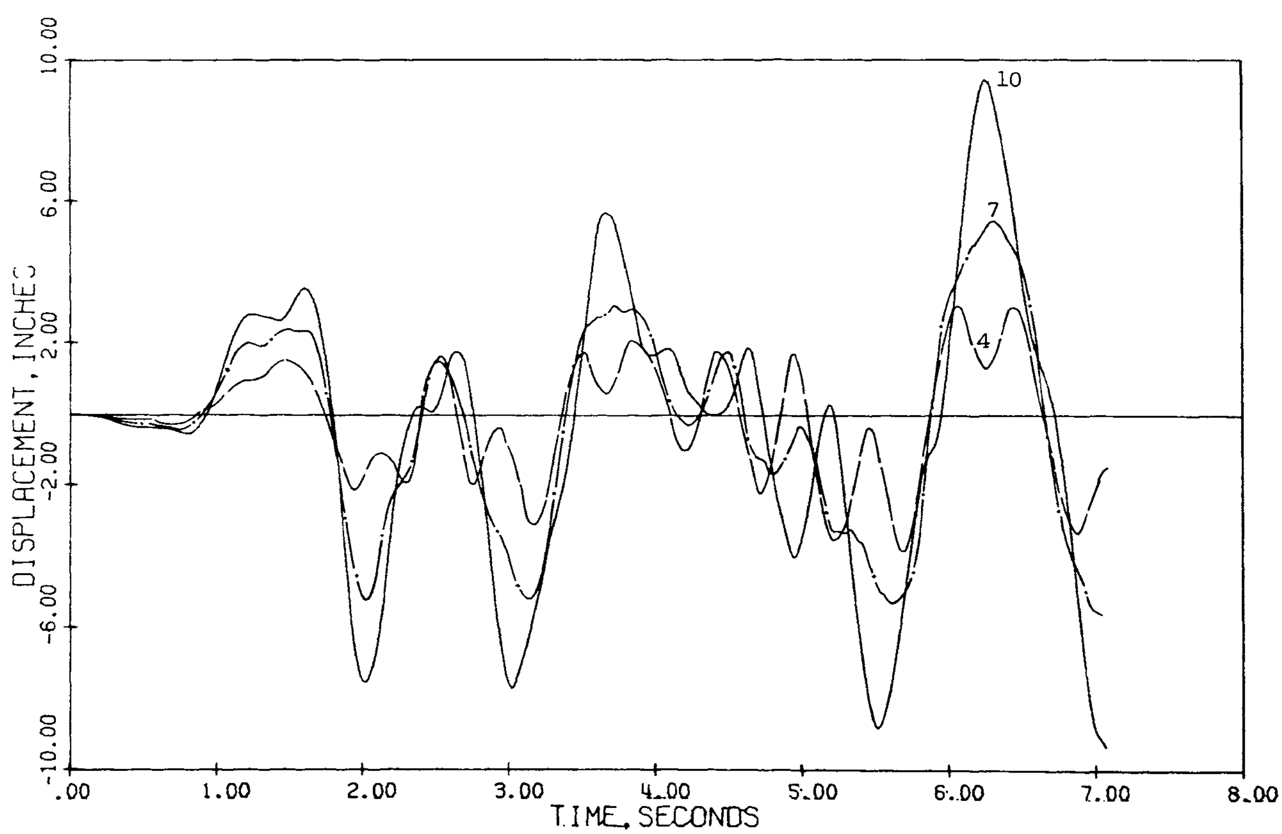


FIGURE F.2. Displacement vs, time; structure F2; IU analysis

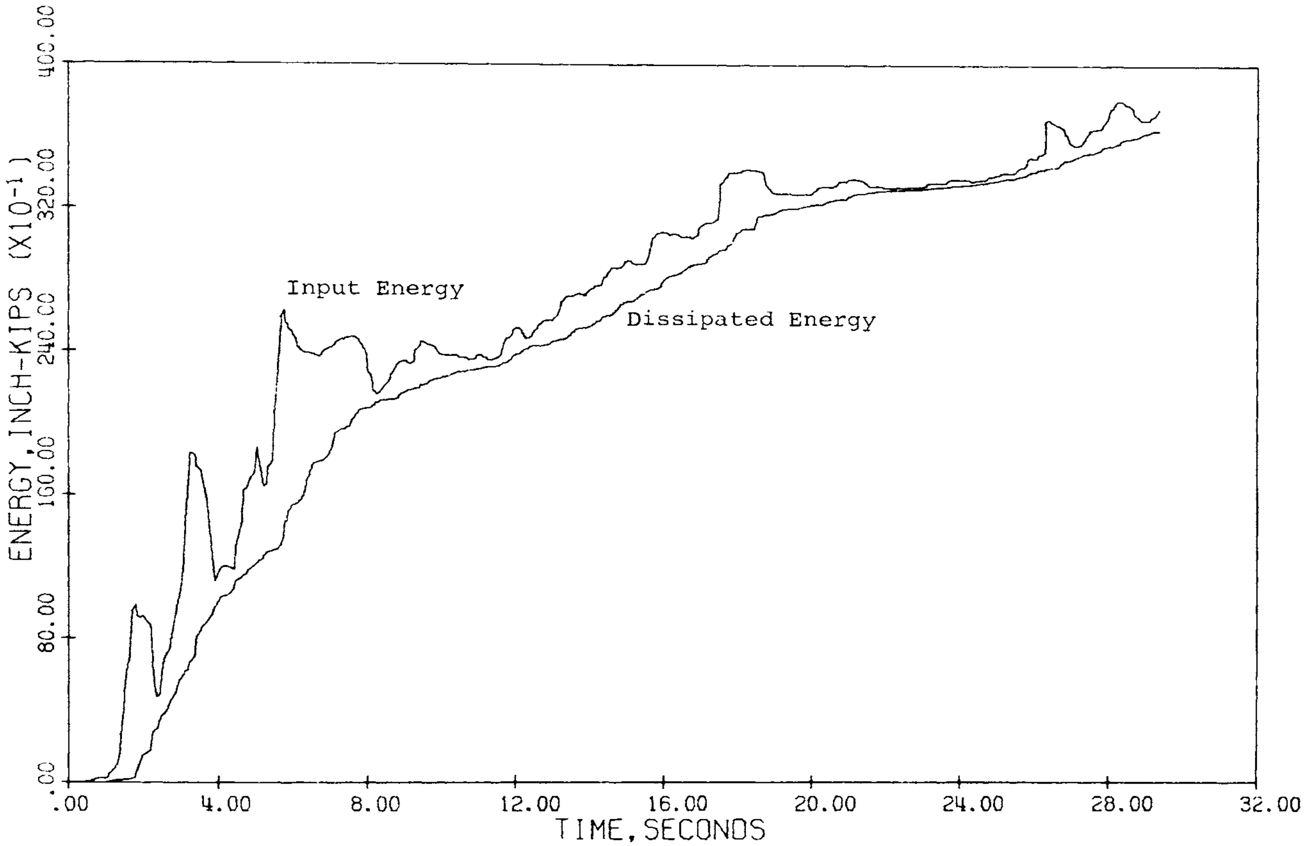


FIGURE F.3. Energy vs. time; structure F2; ED analysis

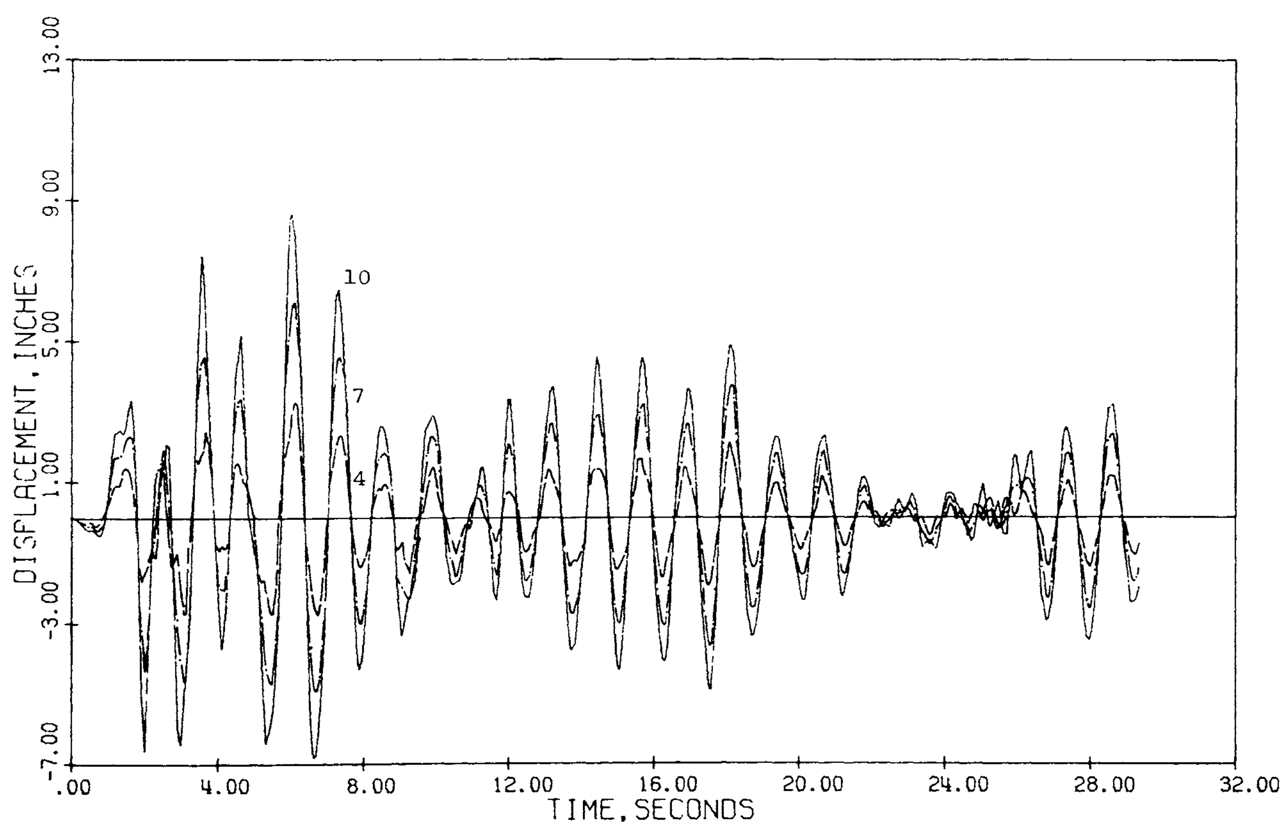


FIGURE F.4. Displacement vs. time; structure F2; ED analysis

APPENDIX G

Energy- and Displacement-Time Histories for Structure B2

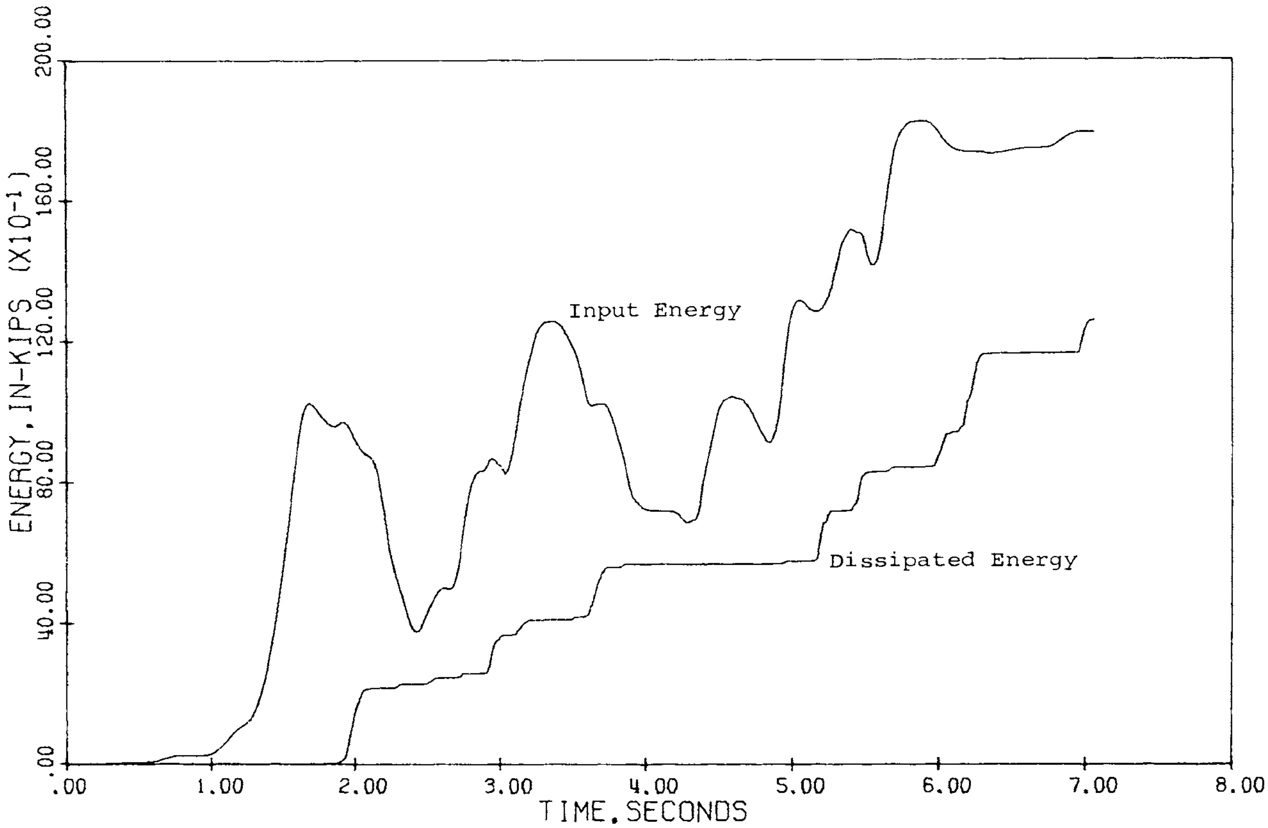


FIGURE G.1. Energy vs. time; structure B2; IU analysis

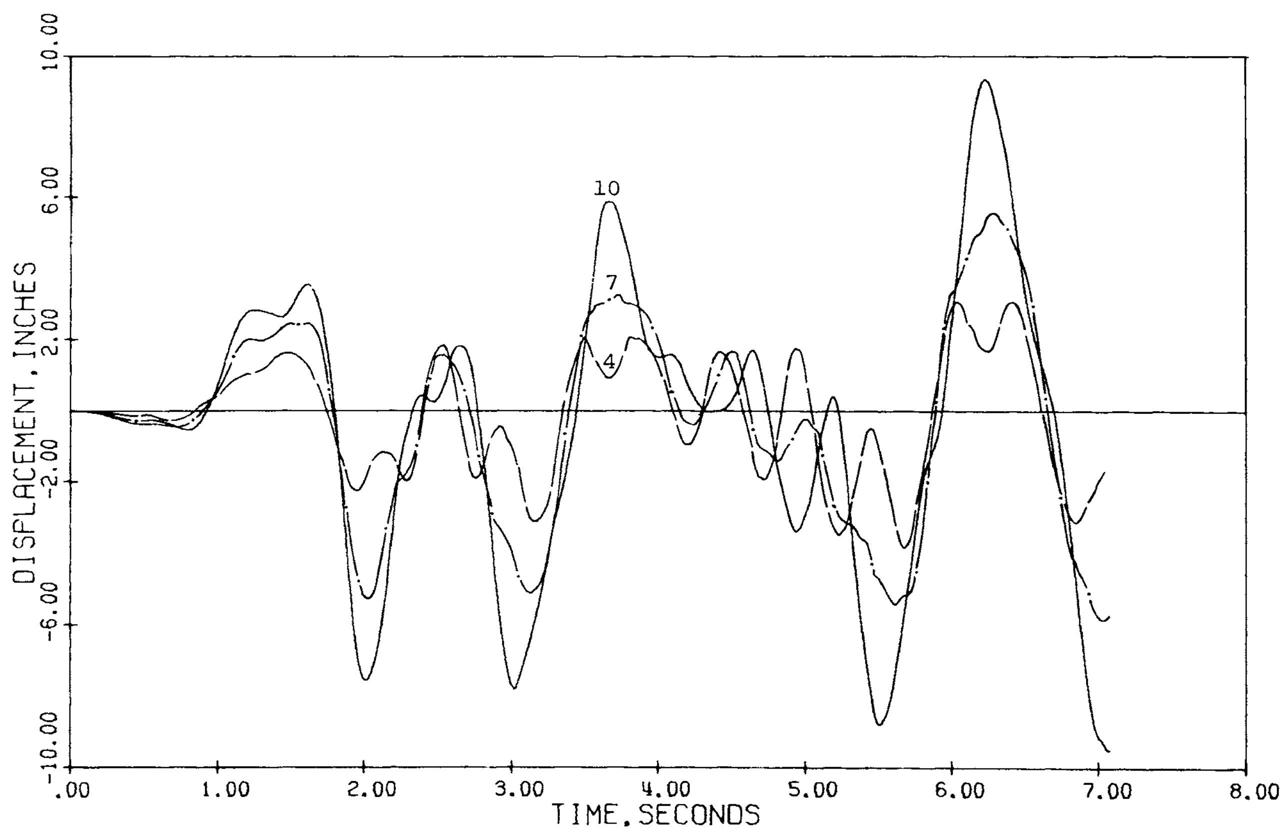


FIGURE G.2. Displacement vs. time; structure B2; IU analysis

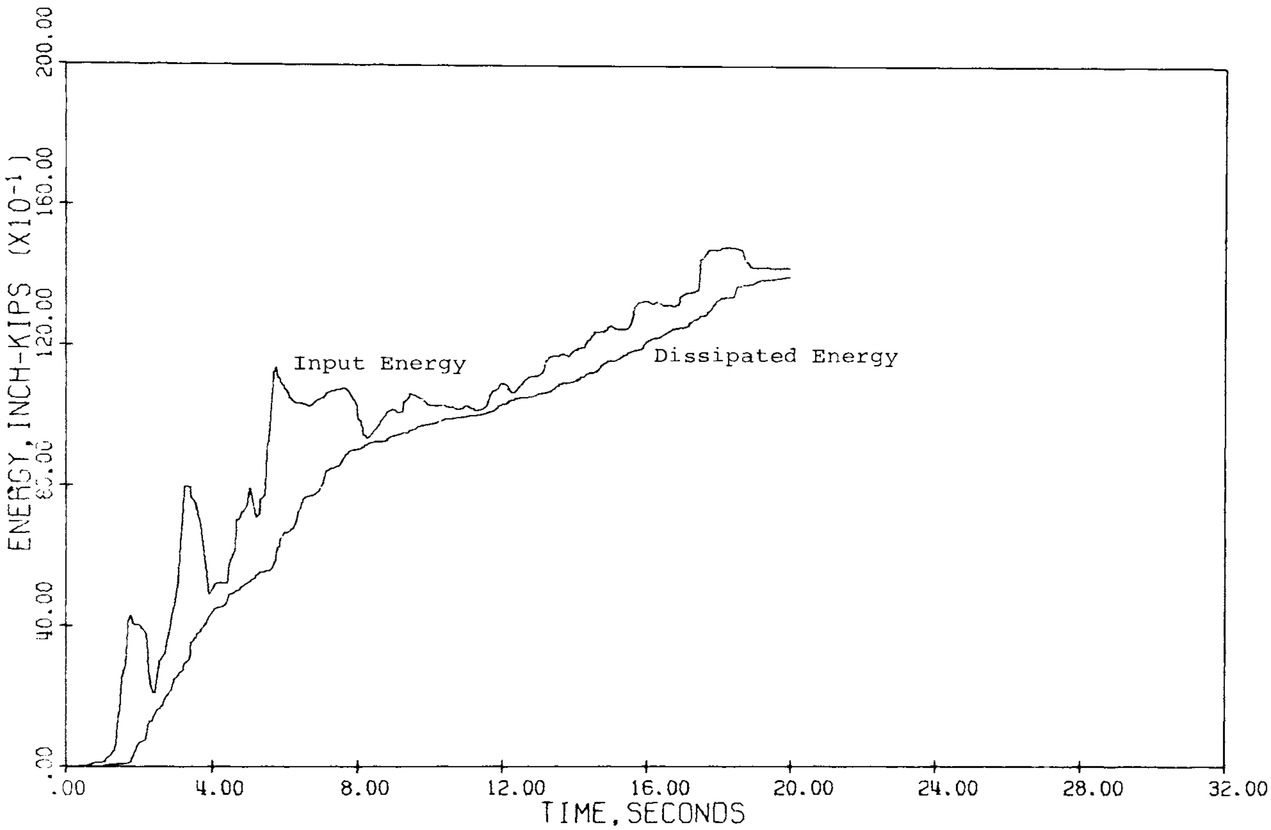


FIGURE G.3. Energy vs. time; structure B2; ED analysis

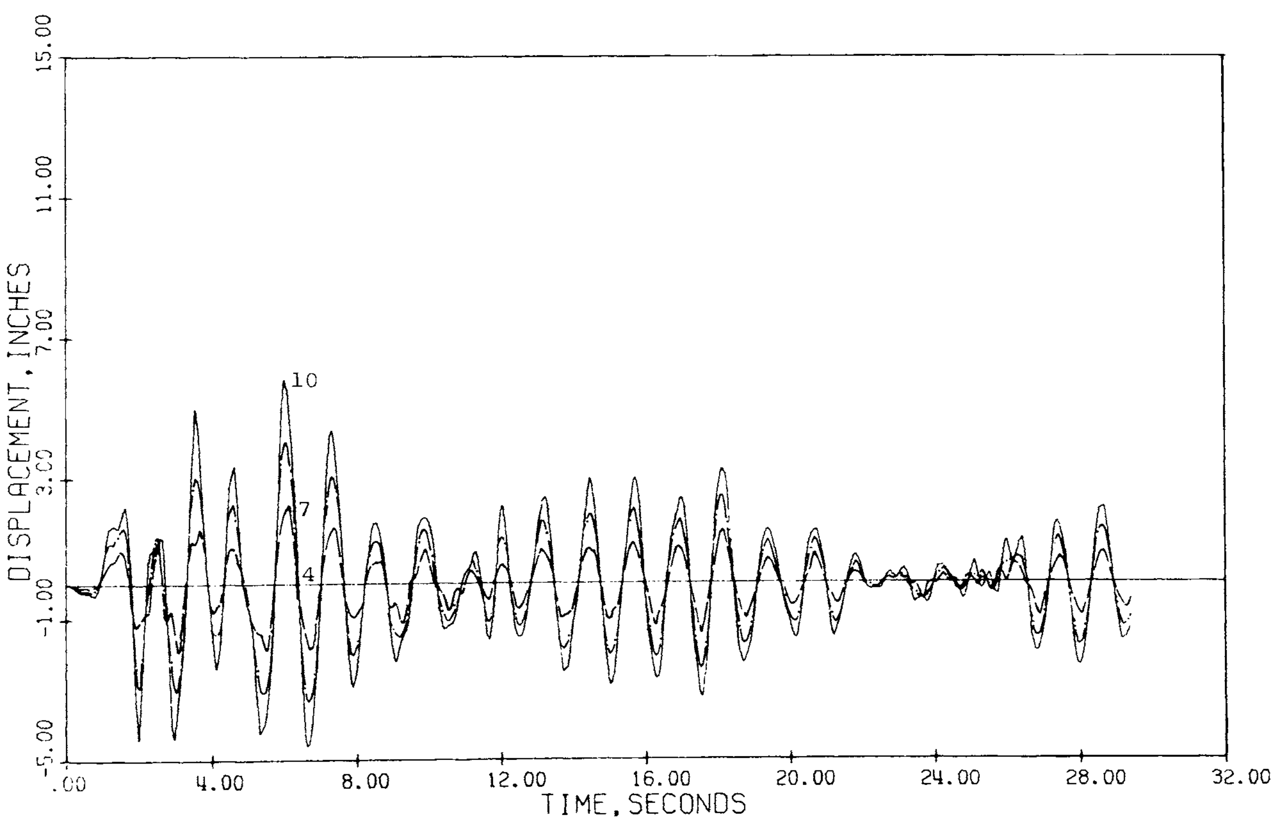


FIGURE G.4. Displacement vs. time; structure B2; ED analysis

APPENDIX H

Energy- and Displacement-Time Histories
for
Structure A2

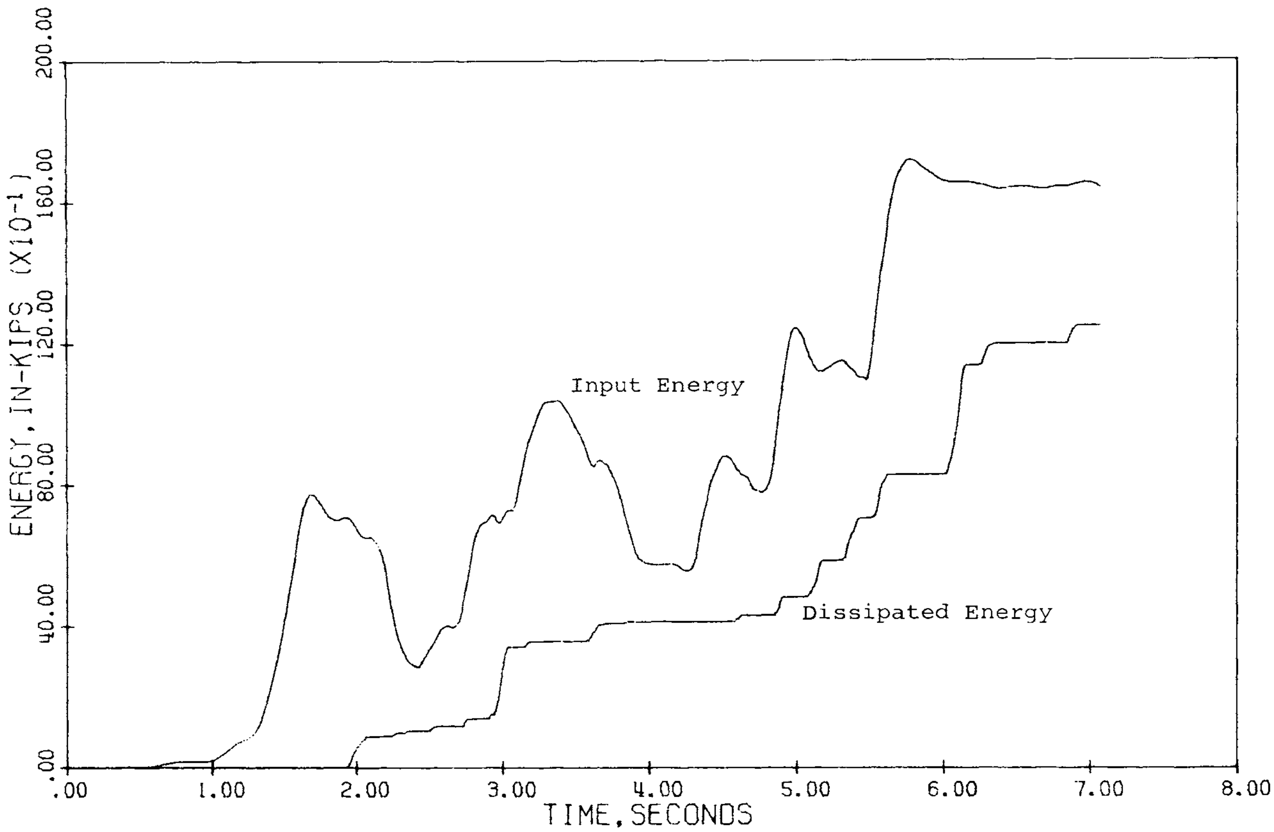


FIGURE H.1. Energy vs. time; structure A2; IU analysis

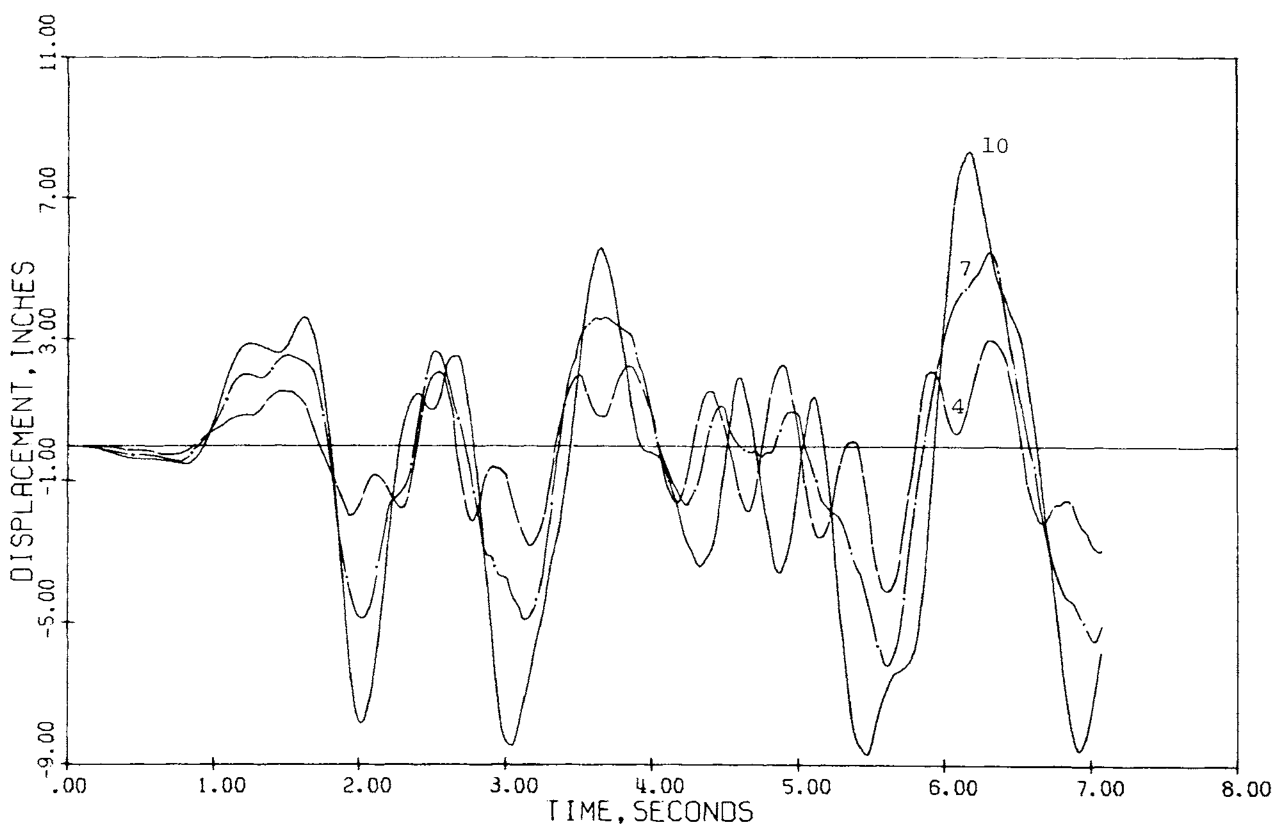


FIGURE H.2. Displacement vs. time; structure A2; IU analysis

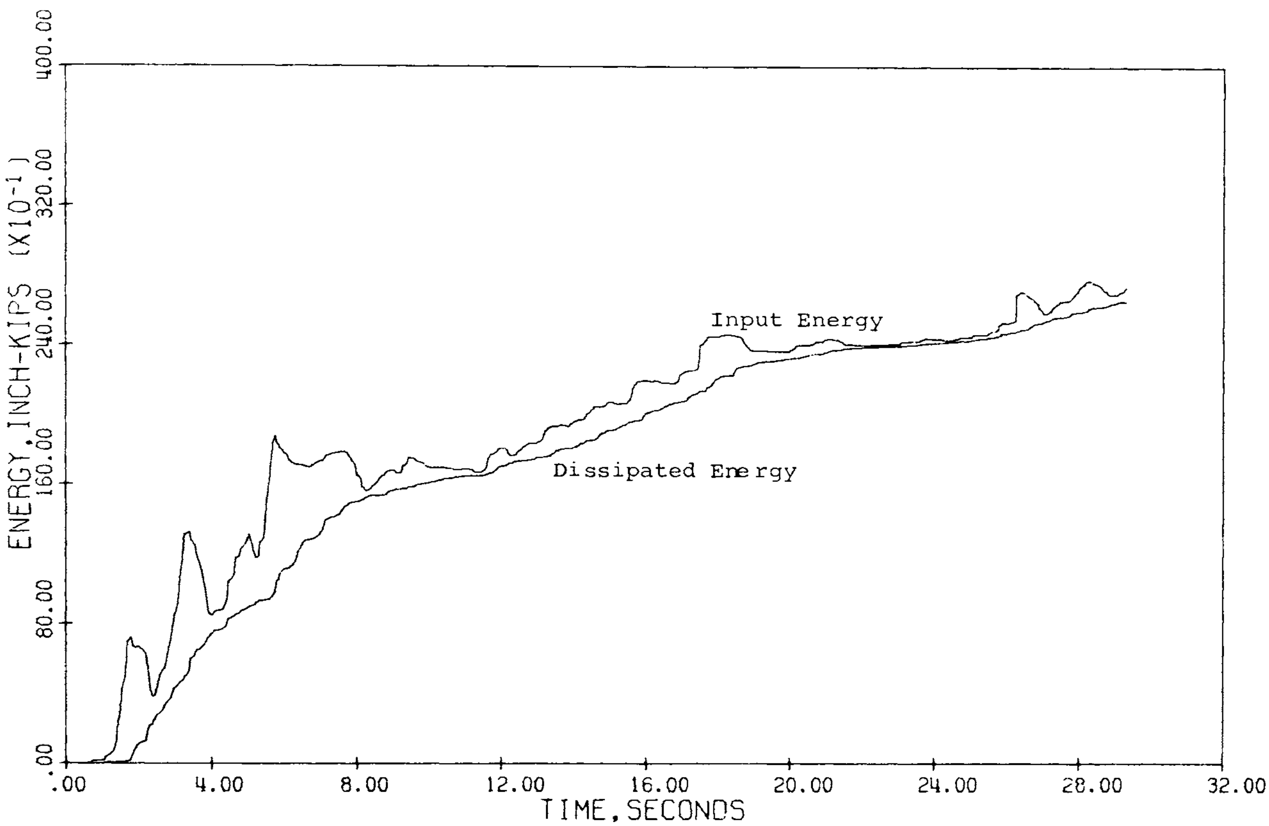


FIGURE H.3. Energy vs. time; structure A2; ED analysis

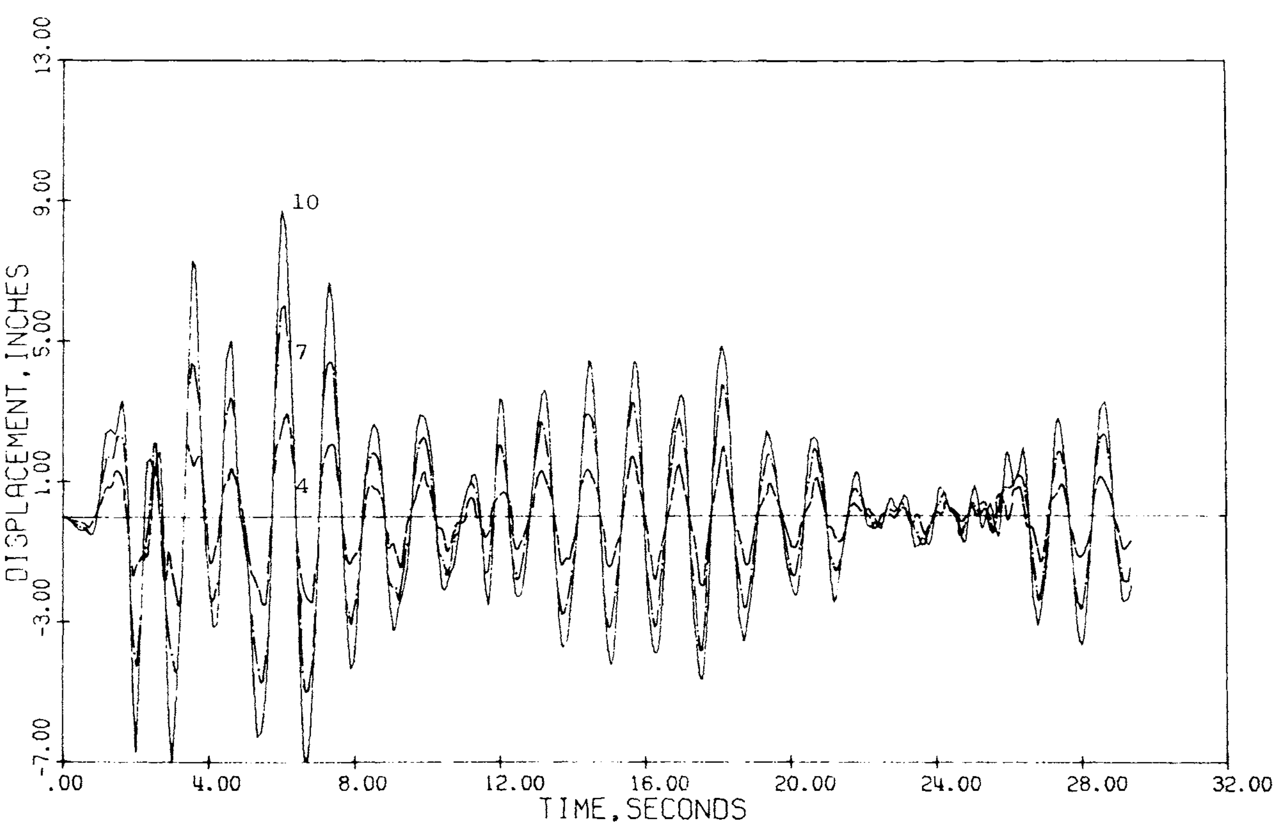
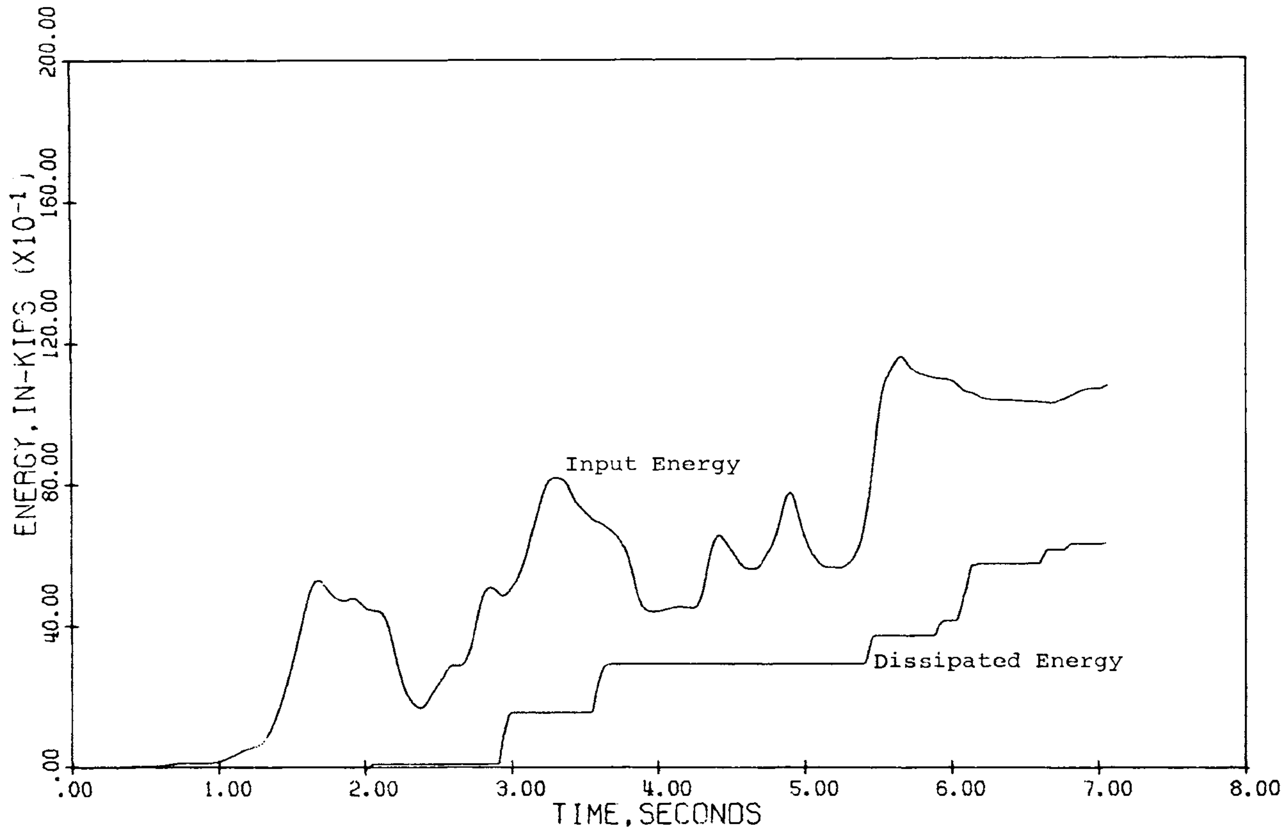


FIGURE H.4. Displacement vs. time; structure A2; ED analysis

APPENDIX I

Energy- and Displacement-Time Histories
for
Structure U2



IGURE I.1. Energy vs. time; structure U2; IU analysis

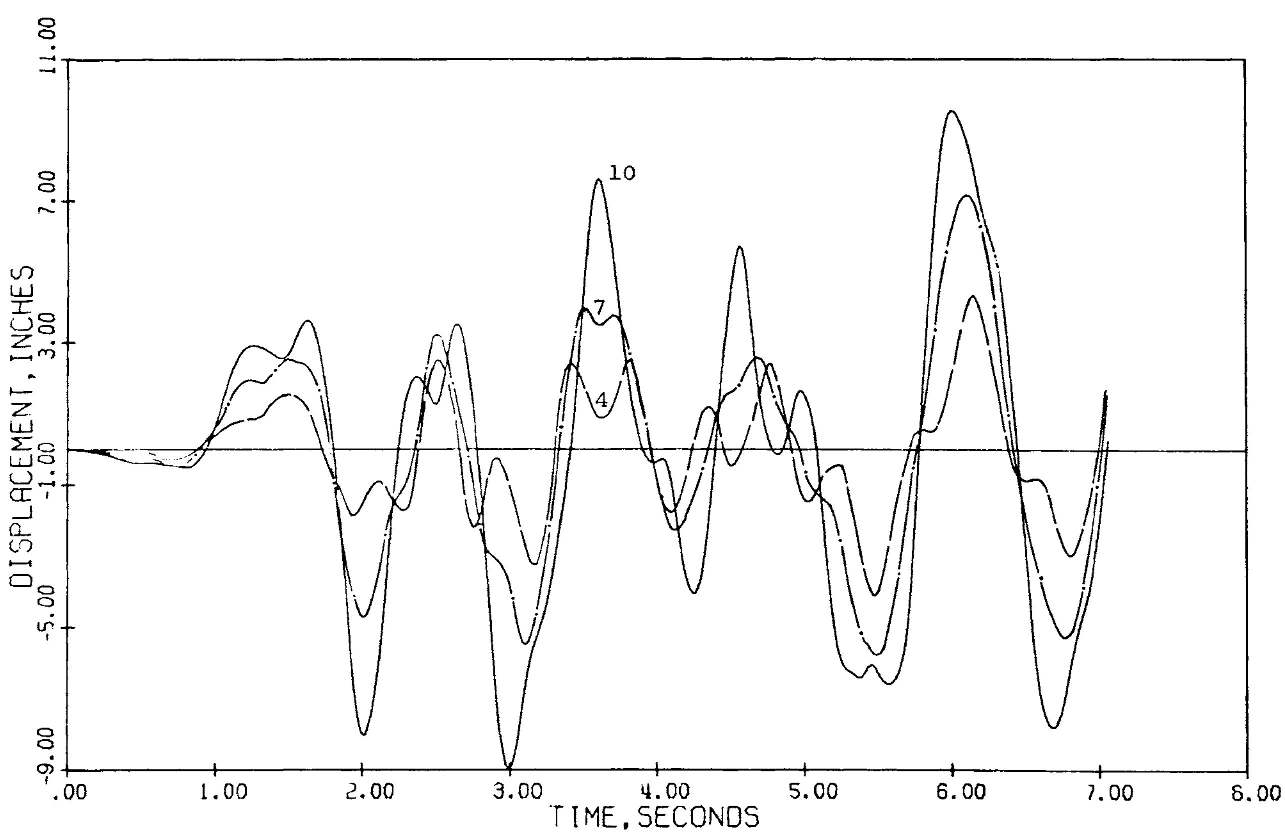


FIGURE I.2. Displacement vs. time; structure U2; IU analysis

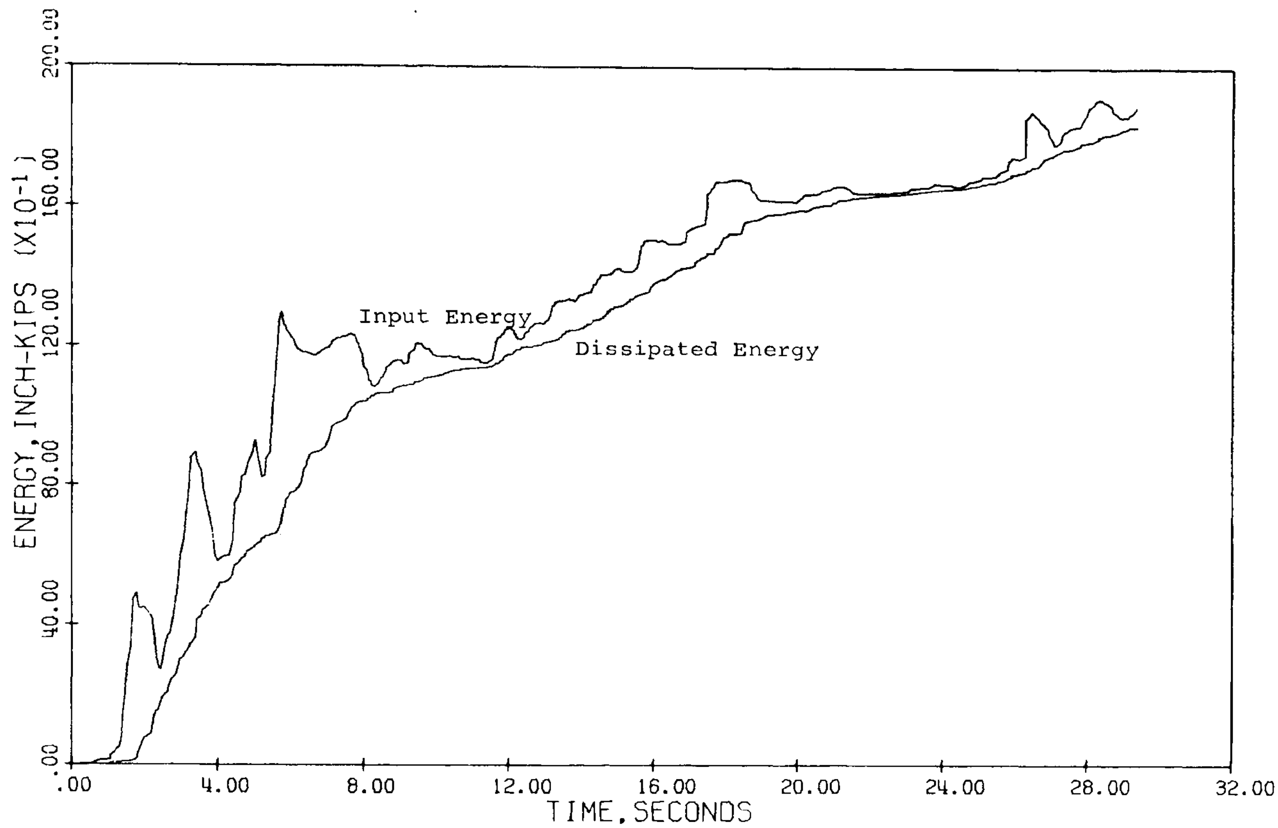


FIGURE I.3. Energy vs. time; structure U2; ED analysis

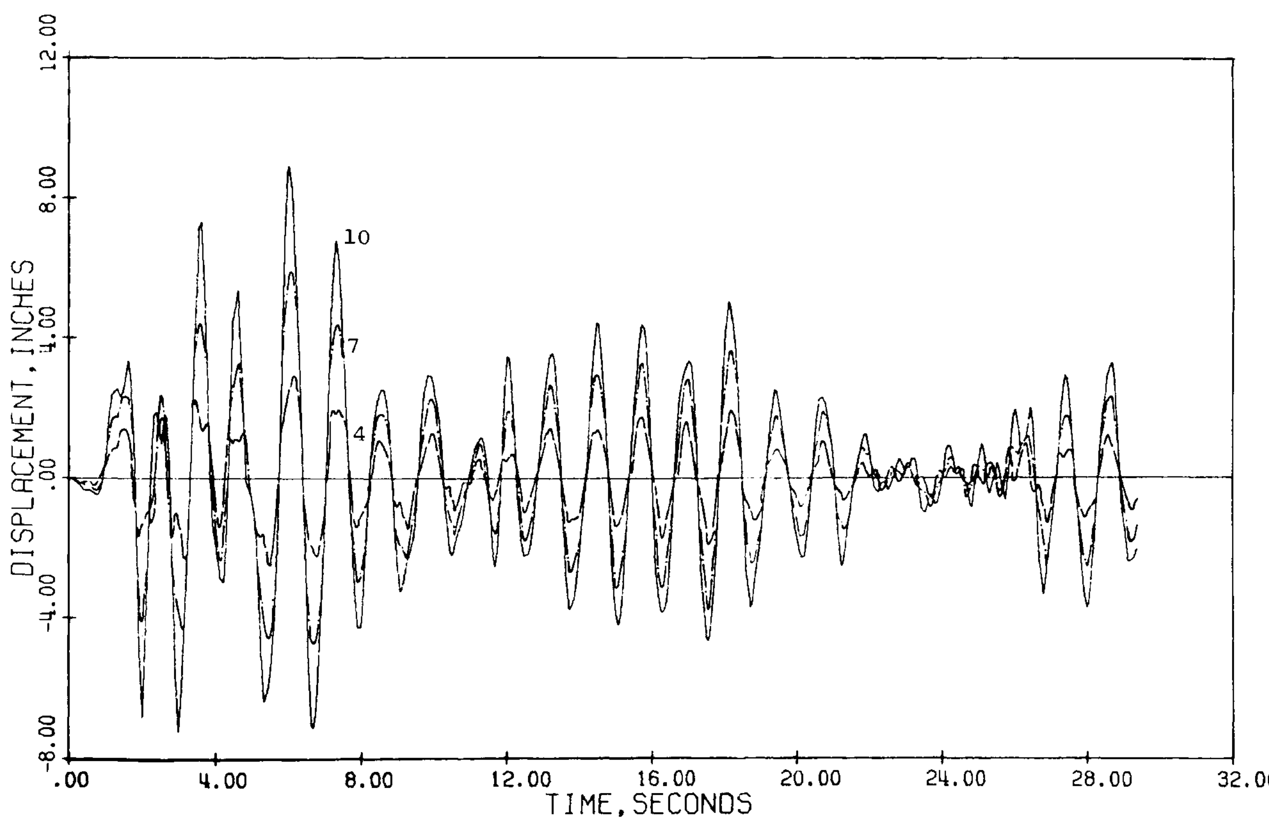


FIGURE I.4. Displacement vs. time; structure U2; ED analysis

BULLETINS

Steel Research for Construction

No. 1 Current Paving Practices on Orthotropic Bridge Decks—*Battelle Memorial Institute, October, 1965*

No. 2 Strength of Three New Types of Composite Beams—*A. A. Toprac, October, 1965*

No. 3 Research on and Paving Practices for Wearing Surfaces on Orthotropic Steel Bridge Decks, Supplement to Bulletin 1—*Battelle Memorial Institute, August, 1966*

No. 4 Protection of Steel Storage Tanks and Pipe Underground—*Battelle Memorial Institute, May, 1967*

No. 5 Fatigue Strength of Shear Connectors—*R. G. Slutter and J. W. Fisher, October, 1967*

No. 6 Paving Practices for Wearing Surfaces on Orthotropic Steel Bridge Decks, Supplement to Bulletins 1 and 3—*Battelle Memorial Institute, January, 1968*

No. 7 Report on Investigation of Orthotropic Plate Bridges—*D. Allan Firmage, February, 1968*

No. 8 Deformation and Energy Absorption Capacity of Steel Structures in the Inelastic Range—*T. V. Galambos, March, 1968*

No. 9 The Dynamic Behavior of Steel Frame and Truss Buildings—*Dixon Rea, J. G. Bouwkamp and R. W. Clough, April, 1968*

No. 10 Structural Behavior of Small-Scale Steel Models—*Massachusetts Institute of Technology, April, 1968*

No. 11 Response of Steel Frames to Earthquake Forces—Single Degree of Freedom Systems—*M. J. Kaldjian and W. R. S. Fan, November, 1968*

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