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EXPERIMENTAL STUDIES OF A SINGLE STORY STEEL STRUCTURE WITH FIXED, SEMI-RIGID AND FLEXIBLE CONNECTIONS

by

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The details of the structure are presented together with the instrumentation programs; the extent of the data collected in the tests is described. In addition, to the dynamic properties of the structure, moment-rotation, shear-rotation, and several other response parameters of the three different connections are presented.

The global responses of the structure with the three different connections under three types of excitations are examined. The behavior of the structure in these tests ranged from elastic to inelastic. Local responses of the structure such as force and deformation time histories, hysteresis diagrams and tabulated extreme values are shown. Important observations are made on the test results in each of the tests.

The behavior of flexible and semi-rigid structures under dynamic loading is studied, and their respective responses are compared to that of the fixed structure subjected to similar earthquakes. The use of flexible and semi-rigid structures in low to moderate earthquake zones is investigated and commented on.

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Table of Contents

List of Tables

List of Figures

CHAPTER ONE

INTRODUCTION

1.1 Introduction

Steel frames are constructed using three types of connections: fixed, semi-rigid, flexible. Flexible and semi-rigid steel structures are limited to areas of low seismicity by the Uniform Building Code (16). The reason for this limitation may be the belief that excessive deformations will occur in structures with semi-rigid and flexible connections, or that buckling under the structures own weight and the P-δ effect might take place during strong earthquakes. It is possible however, that in low-rise buildings (up to 5 stories), the flexibility provided by the connections might attract lesser inertia forces, and thus lesser deformations.

In order to investigate the effects of connection flexibility on the dynamic response of structures, the behavior of a one story steel structure with flexible, semi-rigid, and fixed connections was studied when subjected to three different base excitations. Figure 1.1 gives a schematic illustration of the test specimen. The three earthquakes are, the 1940 El-Centro, the 1952 Taft, and the 1985 Mexico-City earthquakes. The responses of flexible and semi-rigid structures were studied and compared with these of fixed structures subjected to similar earthquake loadings.

1.2 Literature Review

A survey of the literature on the behavior of flexible and semi-rigid structures, resulted in a number of relevant papers, some of which are summarized here.

Shing, Gerstle, and Harsoyo (12) studied the dynamic response of low-rise steel building frames designed by the " Type 2 Construction " method and subjected to moderate earthquakes. In this study, typical flexibly connected Type 2 frames were analyzed for seismic resistance using the 1988 Uniform Building Code (16). This study showed that these frames are adequate for seismic forces not greater than those of zone 2B, of the UBC, but that they may not have adequate lateral stiffness.

Hwang, Chang, Lee, and R. L. Ketter (11) investigated the seismic behavior of a pinned-base steel gable frame structure designed according to the AISC Manual (17). The inelastic lateral strength was evaluated and quantified. The story drifts were up to 7% at moderate to severe damage levels, and the observed experimental ultimate lateral strength was very close to the value of a 5% damping linear elastic response spectrum of the measured table acceleration.

Leon (13) conducted four full-scale tests on semi-rigid connections incorporating a composite floor slab at the University of Minnesota. These types of connections were found to exhibit bilinear moment-rotation curves with large initial stiffness, excellent ductility and predictable ultimate moment capacity.

A study of an eccentrically braced dual steel system (EBDS) subjected to severe earthquake ground motions was carried out by Whittaker, Uang, and Bertero on the earthquake simulator at the University of California at Berkeley. A six story EBDS system was analyzed according to current earthquake resistant regulations and codes. The results of these tests showed that these dual systems have a substantial overstrength when compared to its nominal yielding strength. The

UBC requirement that a ductile moment resisting frame be designed to resist 25% of the design base shear was found to be questionable, because such frames lack the strength and stiffness to be compatible with the braced frames.

1.3 Objectives and Scope of the Research

The investigation reported here had the following objectives:

(1) To compare the lateral deflections of flexible and semi-rigid frames with those of fixed frames when the structure is subjected to large base excitations (0.5g), similar to those that occur during severe earthquakes.

(2) To compare the base shear forces that prevail in the structure for different connections when subjected to similar earthquake loads.

(3) To investigate the moment-rotation hysteresis loops of the flexible, semi-rigid, and fixed connections.

(4) To study the effects of plastification in the connection on the overall behavior of the structure.

To achieve the above objectives, a single story steel frame was constructed. The beam to column connections were changed, from simple (flexible), to semi-rigid, to fixed. For each of the cases, three types of ground motions were used as input motions to the structure, each applied at progressively increasing intensities, making a total of forty four dynamic test runs. From this complete sequence of test data, twenty four were chosen for investigation. These tests represent the full range of performance of the structure in these experiments. The data collected is presented in this report.

CHAPTER TWO

TEST SET-UP

2.1 Shaking Table Facility and Data Acquisition System

The 20 feet by 20 feet shaking table is located at the Earthquake Engineering Research Center (EERC) of the University of California at Berkeley. The shaking table is capable of moving in the vertical direction and one horizontal direction in such a way that strong-motion earthquakes can be simulated accurately. The maximum displacement and velocity that can be achieved by the table are 5 inches and 25 in/sec., respectively. The shaking table may be used to subject a structure weighing up to 100 kips to a table acceleration of 1.0 g in the horizontal direction. The useful frequency range is from 0 to 20 Hz.

The earthquake motions, which are in the form of digitized acceleration time histories, cannot be used directly to excite the shaking table, since the input requires displacement time histories. Acceleration is converted to analog form using a digital to analog converter and then changed to displacement by integrating twice using an electronic analog integrator. The amplitude scaling of the displacement recording during a test is controlled using a "span" setting. A span of 1000 will give a displacement time history that has a nominal peak of 5 inches, the capacity of the table. The table facility is described in detail by Rea and Penzien (18); the data acquisition system is also described thoroughly in that report.

2.2 Test Structure

A one-story one-bay structure was used in the experiments. The following sections describe the properties of this test structure.

2.2.1 Geometry of the Structure

The test structure consisted of two parallel one bay single story frames. The frames were connected to each other by a horizontal channel at floor level, and by two sets of diagonal X bracing. The structure is shown in Figure 2.1 mounted on the shaking table. The dimensions of the test structure are given in Figure 2.2. The ground excitation was applied to the table along the unbraced frames, Figure 2.2. As mentioned earlier the beam-to-column joints were changed from simple, to semi-rigid, to fixed connections, thus providing three different test structures. The floor diaphragm, which was made of channels and W-shape beams, was connected to the girders by heavy structural tubes. Figure 2.3 gives a detail of the floor system and the floor-to-girder connections. As can be seen the inertia forces developed were transferred by six reinforced heavy 12x6x1/2 inch structural tubes to the girders of the two frames. It is evident that the concrete blocks could not interfere with the girder deformation.

Figures 2.4, 2.5, and 2.6 show details of the flexible, semi-rigid, and fixed beam-to-column connections, respectively. In the case of the flexible connection, two 2x2x3/16 inch angles were used with eight A325 1/2 inch diameter bolts bolted to the column flange and the beam web. The semi-rigid connection consisted of the same double-angle connection, with additional seated 2x3-1/2x3/16 inch angles which were added on the top and bottom flanges of the beam. In this case also 1/2 inch diameter A325 bolts were used. The fixed connection consisted of the same shear connection detail, but the double angles were welded to the beam to insure rigidity. Also, the flanges of the beams were welded to the columns by a full penetration field weld (E70XX) using backup plates, see Figure 2.6.

Base plates were 10x10x1 inch plates. Each base plate was bolted to another 10x10x1-3/4 inch plate by four 1 inch A490 bolts. The 1-3/4 inch base plate was in turn prestressed to the table by high strength steel rods. To insure fixity of columns to the base plates, a pair of triangular plates was used at the base of each column. Figure 2.7 shows a detail of column base connection.

2.2.2 Material and Section Properties of the Structure

The columns were fabricated from standard rolled shapes of steel having a yield strength of about 49 ksi according to the coupon tests. The beams were made of W10x15 section, while the columns were made of W4x13. Table 2.1 gives section properties of the beams and columns. The channels used to connect the two frames along the weak axis of the columns, were C-9x15, while the diagonal cross bracings were 2x2x1/4 angles. Table 2.1 gives the geometric properties of these sections.

2.2.3 Design Criteria

In order to develop a period of vibration in the range of periods of actual steel structures, six blocks of concrete were added to the structure. As shown in Figure 2.8 the concrete blocks were prestressed to the floor framing level so as to prevent any sliding, thus making the blocks a fully reactive mass. The concrete blocks were set in such a way so as to have the center of mass close to the center line of the floor framing level. The extent to which this vertical eccentricity may affect the earthquake response is thought to be small. Estimated weights of the steel components and the concrete blocks are listed in Table 2.3. The total weight of the structure was calculated to be 27,423 pounds.

The test structure was designed as an actual structural system and not as a scale model of a specific prototype. The relatively small dimensions of the members was due to the limitations of the shaking table dimensions. Member sections and, correspondingly, connection sizes were scaled down. In designing the structure the intent was to test and observe the effect of inelasticity in the connections on the overall response of the structure. Obviously, yielding was expected in the simple and semi-rigid connections only. Since the plastic moment of the semi-rigid connection is larger than that of the simple, the column and beam section moduli were controlled by the semi-rigid connection plastic moment.

2.2.4 Safety Considerations

The structures tested were planned to be subjected to severe ground motion accelerations. Since flexible and semi-rigid frames are rarely tested under these conditions, and their behavior is still unpredictable, two safety procedures were adopted:

(1) A block of timber beams was built under the structure so as to protect the table if the structure were to fail.

(2) The mass was attached loosely to a 40 kip capacity crane that would be able to hold up the mass.

Figure 2.9 shows a detail of the above two safety procedures.

CHAPTER THREE

INSTRUMENTATION

3.1 Introduction

To monitor the local as well as the global behavior of the structure four types of measuring devices were mounted on the structure. In principal, it should be sufficient to measure the behavior of one connection and to monitor the response of one frame. However, variability in the geometry and material properties is unavoidable; hence it was necessary to include sufficient instrumentation to verify the degree of symmetry in the structure. It was also necessary to include some degree of redundancy in the instrumentation, so as to have some backup in case one element of the instrumentation malfunctioned. In general, each quantity reported here was measured and checked by means of two independent instrumentation systems.

The following quantities were measured by direct instrumentation:

(1) Rotation response of three of the connections

(2) Shear displacement of the four connections

(3) Axial displacement of the four connections

(4) Shear forces in the columns in both directions

(5) Moment in each column at 24 inches from the bottom

(6) Axial forces in the columns

(7) Acceleration of the floor level in the vertical, and the two horizontal directions, and the rotational acceleration.

3.2 Types of Transducers

Four types of transducers were used to monitor the behavior of the structure. Accelerometers, potentiometers, DCDT's, and strain gages were the basic measuring devices used in this investigation. There were 88 data channels activated, 10 of which were allocated to monitor the performance of the shaking table, the other 78 were used to measure the behavior of the structure . There were 16 channels allocated for DCDT's, while 9 channels were allocated for potentiometers. Accelerometers were allocated 5 channels, and the rest of the data channels (48) were allocated for electric strain gages. Because various structural response quantities would eventually be determined from the data produced by these transducers, it was necessary to understand the physical performance characteristics of these transducers, as summarized here.

3.2.1 Accelerometers

The accelerometer was a model 141 made by Setre System, Inc. This model is a linear accelerometer that produces a high level instantaneous DC output signal proportional to sensed accelerations. The range of the accelerometer used is (-4g to +4g) with 0.1% nonlinearity error.

3.2.2 Potentiometers

The potentiometer used was a model PT-101 position or displacement transducer designed for measurement from 0 to 30 inches made by Celesco Transducer

Products. It provides an electrical signal proportional to the linear extension of the stainless steel cable. Nonlinearity errors are less than 0.1% of full range. Its frequency response is up to 60 Hz, and its sensitivity is 32 mV/V/inch.

3.2.3 DCDT's

DCDT stands for Direct Current Displacement Transducer. The DCDT used was a model 240 made by Trans Tek. Of the sixteen DCDT's used twelve had a range of (-1 inch to +1 inch), while the other four had a range of (-0.5 inch to +0.5 inch). Nonlinearity errors were less than 0.5%.

3.2.4 Electric Strain Gages

Two types of strain gages were used; shear strain gages and uniaxial strain gages. The shear strain gages were model EA-13250TD-120 made by Micro-Measurements, for which the resistance was 120 ohms and the nonlinearity error was 0.6%. Two brands of strain gages were used to measure axial strains. On the columns, model YFLA-2 made by Tokyo Sokki Kenkyuju Co.LTD, with a resistance of 120 ohms, and a nonlinearity error of 0.5% were used. On the diagonal bracings, CEA-06-125UW-120 model strain gages made by Micro-Measurements, with a resistance of 120 ohms, and a nonlinearity error of 0.6% were used.

All of these gages are capable of measuring strains up to 20% according to their respective manufacturers specifications. All were bonded to the test structure with fast setting adhesives. A synthetic rubber coating was applied on the top of the strain gages . This coating was intended to protect the gage from humidity, and chemical intrusions.

3.3 Structural Response Measurement

The position of the 78 transducers used to monitor the behavior of the structure is described in the following section.

3.3.1 Floor Acceleration and Displacement

The five accelerometers that were employed in this investigation were distributed as follows. Two accelerometers measured the horizontal floor response in the direction of excitation in each of the two frames. The third accelerometer was oriented upwards so as to detect any vertical component in the response of the structure, while the fourth was oriented in the transverse direction of excitation to detect any horizontal transverse component in the response. These four accelerometers were all attached to the structure at the floor level center line. The fifth accelerometer was attached to the concrete blocks at an elevation of 7 ft 5-1/2 inches, this accelerometer was used to measure the horizontal response of the mass in the direction of excitation, and to check for any difference between the mass and structural responses. Figure 3.1 shows the location of each of the accelerometers.

Potentiometers, attached to the independent reference frame erected outside the pit wall of the shaking table, were employed to measure the absolute displacement of the floor. Very light stainless steel cables were used to connect the potentiometers to their targets on the structure. Two potentiometers were used to measure the displacement of each of the frames at floor level. Another two potentiometers were used to measure the displacements of the concrete blocks. Five potentiometers were used to measure the deformation of the S-W column. Figure 3.2 shows the location of each of the potentiometers.

3.3.2 Connection Deformation Measurement

To measure the rotation of the connection four DCDT's were used as shown in Figure 3.3. The rotation is thus given by:

$$\Theta = (\ |\delta 1| + |\delta 2| \ ) \ / \ h \tag{3.1}$$

Since the vertical deformation of the connection was thought to be of importance, it was also measured. Figure 3.4 shows the location of the DCDT's. Because of the limited number of available DCDT's the rotation was only measured for three connections, while the vertical deformation was measured for all four connections.

3.3.3 Force Measurement

Two types of strain gages were used to measure moments, shears, and axial loads: shear strain gages, and uniaxial strain gages. Ten strain gages were attached on each column at 24 inches from the top of the base plate. Four of these strain gages were uniaxial resistors and were installed at the column flanges, two on each side. Two shear strain gages were mounted on the centerline of the column web. The other four shear strain gages were installed at the centerline of the column flange, two on either side. Figure 3.5 shows the location of the strain gages.

Also, uniaxial strain gages were installed on the diagonal bracings between the two frames. Each brace had two strain gages installed at 26 inches, from the edge of the beam, along its length. Figure 3.6 shows the location of the strain gages.

3.4 Instrumentation Channel Schedule

Table 3.1 lists the allocation of various transducers to specific channels of the data acquisition system. As can be seen in the table , the first 10 channels were reserved to monitor the performance of the shaking table, the next 5 channels were used for the accelerometers. The following 16 channels were allocated for DCDT's, the next 9 channels were allocated for potentiometers, after which the remaining 48 channels were allocated to strain gages.

CHAPTER FOUR

TEST PROGRAM

4.1 Introduction

In this chapter the testing sequence and the different ground motions that the structure was subjected to are summarized. In Chapter Five the experimental results are presented, and in Chapter Six conclusions and remarks about the investigation are given.

4.2 Test Sequence

As discussed earlier, the steel structure was tested with three different connections, simple (flexible), semi-rigid, and fixed. Details of the steel structure are given in Chapter Two. The intent in selecting a test sequence was to subject each of the three different structures to exactly the same sequence of earthquake loading. For each of the three structures a sequence of tests was selected. First a tie-back test was conducted to measure the elastic stiffness, as well as to compute the dynamic properties of the structure. Then the structure was subjected to white noise of intensity 0.05g, to find its natural period of vibration more precisely. Then, a series of forced ground motions was applied. Three different historical earthquake ground motions were used to observe the behavior of the structure under different ground motions, and thus a variety of frequency ranges. The 1940 El Centro S00E earthquake, the 1952 Taft N21E earthquake, and the 1985 Mexico

City (Sct.) S60E earthquake were chosen. Each of the earthquakes was increased in intensity from 0.05g to 0.5g. This resulted in a total of 44 shaking table tests. Table 4.1 lists the sequence which was followed in testing each structure. Usually, the data for an earthquake is collected every 0.02 seconds. Looking at column five in Table 4.1, one can see that all of the signals were inputed at a full time scale, except for the Mexico earthquake, which was squeezed so that the earthquake response spectra would have a peak close to the natural period of the structure. In column six of this table, the span of each signal is listed. A span of 5 seconds corresponds to a horizontal table displacement of 5 inches. The experimental data was recorded on two tapes. The contents of these two tapes with the file name for each run are listed in Table 4.2. Figures 4.1, 4.2, and 4.3 show different acceleration time histories of the ground motions that were used.

4.3 Runs Selected for Presentation

As stated earlier, one of the major purposes of this report is to present detailed data on the structural response for a variety of test conditions. Out of the 44 shaking table tests, 24 test runs were selected for detailed evaluation. These 24 tests give a full representation of all the tests performed. These 24 tests include the three earthquake loadings with three different intensities for which the behavior of the structure ranged from elastic to largely inelastic.

CHAPTER FIVE

DATA REDUCTION

5.1 Introduction

For each shaking table test, the data collected was stored on a magnetic tape. The data recorded was expressed in units of g's for accelerometers, in units of inches for potentiometers and DCDT's, and in units of micro strains for the strain gages.

Before major data reduction was performed, two steps were taken. The first was the zero correction, which was accomplished by subtracting the initial "zero reading". This zero reading was taken just before the shaking table test was started. The second was a check on the validity of the reading taken by each channel in that test. When a channel was found to have malfunctioned, it was excluded in the data reduction process, and was replaced by other relevant back-up channels in the reduction formulas.

An interactive environment for data analysis and graphics was used to reduce the data collected. The "S" program was used in this case (19). In the following section, a detailed description of the data reduction process is presented.

5.2 Data Noise

Recorded data containing noise is an inevitable phenomenon when using an electronic data acquisition system. Significant noise was mixed with the true signal especially in the data collected by the accelerometers. Two kinds of undesirable noise were encountered during the tests.

(1) **High Frequency Noise:** The high frequency noise could easily be identified. A numerical filtering technique was used throughout the data reduction procedure. The Ormsby low-pass filter, was adopted and implemented into the S computer program to remove the high frequency noise. A cut-off frequency of 20 Hz was used for the following reasons: (i) the frequency of vibration of the structure was far lower than 20 Hz; (ii) the Fourier Amplitude of the input excitation showed very little energy associated with frequencies higher than 20 Hz as shown in Figure 5.1; (iii) the oil column of the shaking table changes the input frequency content at frequencies higher than 20 Hz, thus introducing significant errors. A typical example of the time history response before and after applying the Ormsby low-pass filter is shown in Figure 5.2.

(2) **Low Frequency Noise:** The low frequency noise was not filtered in general. Filtering low frequency noise would have been wrong because of permanent deformations or plastic strains and therefore, strain gages and DCDT's as well as linear potentiometers were not filtered. On the other hand, accelerometers were not allowed to show any permanent non-zero readings at the end of each test run. The data recorded by the accelerometers were of very good quality.

5.3 Sign convention

The following sign convention is used throughout the report:

(1) The lateral drift is positive to the right (south). Refer to Figure 5.3.

(2) The rotation of the connections corresponding to a positive lateral drift is positive. Refer to Figure 5.4.

(3) Axial strains for elongation of the brace member are positive.

(4) Positive column shear and axial forces are induced by positive floor drift, refer to Figure 5.3.

(5) The moment sign convention for the section at which the strain gages were mounted on the column is shown in Figure 5.3.

(6) The sign convention for shear and moment for the connections is shown in Figure 5.4.

5.4 Data Reduction

5.4.1 Table Motion Data

The table motion records are an important record, for they represent the excitations the structure was subjected to. This information will be needed for making analytical predictions. The acceleration time histories are thought to be the important parameter. The basic table motion for each of eight different shaking table tests is presented in the form of a time history plot of acceleration, see Figures 4.1, 4.2, and 4.3. These plots were directly obtained by plotting the readout of the respective channels versus time, specifically, Channel 3.

5.4.2 Structural Response

The global response of the structure is indicated basically by the accelerometer, potentiometer, and the shear strain gage measurements. The member designation for the subsequent discussion is shown in Figure 5.5. In the following the data reduction process for the main parameters is described.

(1) **Base Shear:** Since the structure under investigation had a single degree of freedom, the shear force obtained by using the shear strain gages on each column should agree with the shear force obtained from the accelerometers attached to the structure. The formula by which the shear value was obtained using the accelerometers is:

$$S_{base} = W\ (ac1 + ac2)/2 \tag{5.1}$$

where,

S_{base} = base shear in kips

W = weight of the structure in kips

ac1 = accelerometer 1 in units of g's

ac2 = accelerometer 2 in units of g's.

The formula for shear using the shear strain gages is:

$$S_{base} = \text{Shear}\,1 + \text{Shear}\,2 + \text{Shear}\,3 + \text{Shear}\,4 \tag{5.2}$$

where,

Shear(i) = shear force in column " i " in kips.

The shear force in column " i " can be evaluated as follows:

$$\text{Shear}(i) = (I/Q)\ t_w\ G\ (\text{sgi}5 - \text{sgi}4)\ 10^{-6} \qquad (5.3)$$

where,

I = moment of inertia of the column (in^4)

Q = moment area of the column about the center of gravity (in^3)

t_w = web thickness in inches

G = shear modulus of elasticity (11200 ksi)

sgi5 = strain gage i5 for column " i " in micro-strains

sgi4 = strain gage i4 for column " i " in micro-strains.

A comparison of these two shear forces, shows very good agreement between the two independently measured values. Figures 5.6(a) and 5.6(b) show a comparison between shear force measured by these two methods. In the data presented, the shear forces that are used are the ones obtained from the shear strain gages attached to the columns. The reason for using the shear strain gages was that no low-pass filter was needed for the strain gages, besides, the shear values obtained by both methods were the same.

In all of the plots that contain the base shear, the absolute accelerations (in g's) of the floor can be obtained by dividing the base shear values by the weight of the floor which was 27.42 kips.

(2) **Relative Lateral Floor Displacement:** The relative floor displacement was obtained by subtracting the table motions from the absolute floor level motions. The displacements were calculated using the following formula:

$$\text{Disp} = ((\text{pot}8 + \text{pot}5)/2) - ((h1 + h2)/2) \tag{5.4}$$

where,

Disp = lateral displacement of the floor in inches

pot8 = potentiometer 8 in inches

pot5 = potentiometer 5 in inches

h1 = horizontal displacement of shaking table (inches)

h2 = horizontal displacement of shaking table (inches)

(3) **Axial Force in the columns:** The axial force in the columns consisted of two parts, static, which was equal to the weight of the structure divided by four, and a dynamic part which was calculated as follows:

$$\text{Axial}(i) = A\ E\ ((sgi0 + sgi3 + sgi6 + sgi9)/4)\ 10^{-6} \tag{5.5}$$

where,

Axial(i) = Axial force in column "i" in kips

A = area of column cross section (in^2)

E = modulus of elasticity (29000 ksi)

sg = strain gage in micro-strains

(4) **Local moment in the columns:** The moment in the columns at the location where the strain gages were attached, was calculated using the following formula:

$$\text{Moment}(i) = S\ E\ ((sgi\,0 + sgi\,3 - sgi\,6 - sgi\,9)/4)\ 10^{-6} \qquad (5.6)$$

where,

Moment(i) = moment in column " i " at 24 inches from the bottom (k –in)

S = column section modulus (in^3)

E = modulus of elasticity (29000 ksi)

sg = strain gage in micro-strains

(5) **Moment at the connection:** The moment at the connection was easily derived from the local moment and shear in each column. The following formula describes the statics equation:

$$\text{Moment}(\text{connection } 1) = (48.75)\ \text{Shear } 1 - \text{Moment } 1 \qquad (5.7)$$

(6) **Connection Rotation:** The method by which the connection rotation was calculated was described earlier in section 3.3.2. The following formula shows how the rotation was calculated for connection 1:

$$\Theta = ((dc\,5 + dc\,4 - dc\,3 - dc\,2)/2)/16.87 \qquad (5.8)$$

CHAPTER SIX

TEST RESULTS

6.1 Introduction

In this chapter, the results obtained from the 24 selected shaking table tests are presented. The chapter is divided into two parts. In the first part, an investigation of the three structures (flexible, semi-rigid, and fixed) when subjected to a simulated 0.35g Taft ground motion is presented. The second part considers the major response parameters of the structures. In this part, a description of the behavior of each structure in each of the 24 tests is summarized, then extreme values of various response variables are tabulated, after which time history plots, as well as hysteresis plots, are presented.

6.2 0.35 g Taft Earthquake

A complete investigation of the three structures when subjected to a simulated 0.35g Taft Earthquake is presented in this section. This part of the investigation had three objectives:

(1) To study the complete response of the structures, globally as well as locally, in order to identify the parameters that affect the response significantly. The behavior of these parameters in the 24 shaking table tests was later investigated.

(2) For certain response parameters, one series of tests (for the three different structures: flexible, semi-rigid, and fixed) was deemed to be sufficient for comparative studies of the behavior. The data collected from the 0.35g Taft test provided some of the the most dependable and significant data.

(3) Comparisons of data collected from different channels were important in checking the validity of such assumptions as (i) the mass does not slip during testing, and (ii) the structures tested have one major degree of freedom.

6.2.1 Checking Performance of the Structure during Tests

Using the experimental data an investigation was conducted to study the adequacy of the structure tested. The reason for this investigation was that in designing and constructing the test structure certain objectives were set and assumptions were made. Therefore, it was necessary to investigate the validity of the assumptions, and how well the structure fulfilled its function. Some of the objectives were:

(i) to rigidly connect the mass to the floor system;

(ii) to build the structure to have one major dynamic degree of freedom;

(iii) to add some stiffness to the structure in the direction of the weak axis of bending of the columns.

The following three plots were generated for this purpose, these are:

(a) A comparison of the time histories of accelerometers 1 and 4, for the semi-rigid frame, see Figure 6.1. Accelerometer 1 was attached to the structure, while accelerometer 4 was attached to the concrete blocks, refer to Figure 3.1. This comparison was essential, because a difference between the two readings would mean that the mass was not rigidly connected to the floor diaphragm, and probably the mass was slipping. Figure 6.1 shows almost exact replicas, which indicates that the mass was a dynamic reactive mass, and was almost completely in tune with the rest of the structure.

(b) Although the response of the structure to the ground excitation would have three components, one vertical, and two horizontal, the horizontal component perpendicular to excitation and the vertical component were expected to be very small as compared to the horizontal component parallel to excitation. Figures 6.2, 6.3 and 6.4 show that, indeed, the response of the structure was mainly in the direction of excitation, and the other two components are relatively very small. Before testing, there was a concern about development of a significant second torsional mode. To control torsion, diagonal braces were added to the structure in the direction perpendicular to the direction of excitation.

(c) During testing, it was noticed that the response of the structure included a minor torsional component. Figure 6.5 shows a comparison of the responses of the two parallel semi-rigid frames. It can be seen that the two time histories of the two frames are not similar, and that a torsional component is present. Figure 6.6 shows time histories of the rotational acceleration of the response of the flexible, semi-rigid, and fixed structures.

6.2.2 Global Response of the Structure

The global response of the structure is presented by the following three plots.

(a) A time history plot of the lateral horizontal drift of the floor in the direction of excitation. The time history plots in Figure 6.7 indicate that for this case, the drift response of the three structures; flexible, semi-rigid, and rigid did not have large variations in amplitude. However, the period of vibration were dominated by the natural periods of the structures.

(b) A time history plot of the base shear of the structure. Figure 6.8 shows these plots. Unlike drift response, the three structures with increasing level of connection stiffness, showed distinctly different base shear responses. As Figure 6.8 indicates, as stiffness of connection increased the base shear value also increased. Again, because of the dominant first mode of vibration, the frequency of the base shear response was governed by the natural frequency of the structure.

(c) A plot of base shear versus lateral drift, refer to Figure 6.9. This plot is an indicator of structural stiffness, strength, and energy dissipation characteristics. The following observations could be made

(i) The response of the rigid frame was almost elastic with very small hysteresis loops. In addition, the lateral stiffness of the structure was relatively stable and equal to about 22.5 k/in. Maximum values of shear and drift were 25.88 kips, and 1.22 inches, respectively.

(ii) The semi-rigid frame showed more inelastic hysteresis response than the rigid structure. However, the stiffness did not show significant deterioration. The stiffness was about 13.5 k/in. Maximum values of shear and drift were 20.00 kips, and 1.41 inches, respectively.

(iii) The response of the flexible frame was significantly nonlinear. The initial stiffness was about 10.14 k/in., while the stiffness at later cycles was close to 5.27 k/in.. Maximum values of shear and drift were 9.52 kips and 1.57 inches, respectively.

(d) A fast Fourier transform was performed on the horizontal response of the structure, to find its natural frequency of vibrations. Figure 6.10 shows the FFT for the three structures. The fundamental frequencies of vibration for the three structures were established at 2 Hz, 2.5 Hz, and 2.87 Hz, respectively. As Figure 6.10 (b) indicates, the semi-rigid frame had a pronounced second mode of vibration at 4.7 Hz. This mode was torsional and is also shown in Figure 6.6.

6.2.3 Connection Response

Connection forces and deformations, are plotted against each other as listed below. In the following, connection 4 (N-W) is investigated for the three different structures.

(a) Axial Force versus Axial Displacement. Figure 6.11 shows typical plots of axial force versus axial deformation, for the three connections. The following observations can be made from the experimental data:

  (i) In the rigid structure, the connection showed a symmetric axial response with initial stiffness of about 540 k/in. During large deformations, nonlinearities were observed that are related to cyclic yielding of steel in the connection area.

  (ii) In the semi-rigid connections, Figure 6.11 (b), the response was significantly unsymmetric. The compressive side response (left half of the plot) was almost elastic with some nonlinearity for cycles that were preceded by large tension cycles. However, the tension side of the hysteresis loops (right half of the plot) showed significant nonlinearity. The nonlinearity is mainly attributed to gap openings due to inelastic deformation of the connection angles, as shown in Figure 6.12.

  (iii) The response of the flexible connection was completely unsymmetric and nonlinear. The nonlinearity is attributed mainly to two sources: (a) gap openings due to bending of the outstanding legs of the web angles; and (b) slip in the bolt holes. It should be mentioned that observation of connection angles during and after the tests of the flexible frames, clearly indicated that considerable slip was taking place in bolt holes of back-to-back legs. An examination of web angles after the test indicated minor hole elongation as well, that might have contributed to development of larger slips. The response of the flexible connections showed certain hardening and increase of axial force, when deformations were large. The main component of hardening is believed to be kinematic and due to formation of catenary forces when angle legs undergo large deformations.

(b) Shear Force versus Shear Deformation. Figure 6.13 shows typical plots of shear force versus shear deformation, for the three connections. These plots indicate that the energy dissipation due to shear was more pronounced in the semi-rigid connections, than in either the flexible or rigid. The approximate shear stiffness of the connections could be established as 80, 390, and 392 k/in. for flexible, semi-rigid, and fixed connections, respectively.

(c) Moment versus Rotation. Figure 6.14 shows typical plots of moment versus rotation, for the three connections. In these plots the response resembles very closely, the lateral load versus lateral displacement response of the three structures shown in Figure 6.9. The moment-rotation response of the rigid connection was almost elastic whereas a pronounced " pinching " effect could be observed in the semi-rigid connection response. In the case of the flexible connection, large rotations imposed by the column deformation resulted in a rotation in the opposite direction of the connection moment. Figure 6.15 gives a schematic explanation.

(d) Moment versus Shear. Figure 6.16 shows typical plots of moment versus shear, for the three connections. In this plot, the intent is to demonstrate the relation between moment and shear, and to see how connection yielding affects this relationship. The slope in this plot, represents the distance of the point of inflection of the beam from the center line of the column, in inches.

Refer to Figure 6.17 for a schematic explanation. The plot of the flexible connection shows that there was a lot of slippage, and that the point of inflection moved back and forth in the vicinity of the connection center line. The plot in the semi-rigid case shows some slippage in the connection, but an approximate slope of 26 inches is dominant. In the plot for the fixed connection, the relation between shear and moment is very consistent, and shows a slope of 27 inches. The irregular behavior that prevails in this plot is due to the mass slipping at the end of the test. The slope in these plots also represents the path of the load on the column in the interaction diagram.

(e) Shear versus Rotation. Figure 6.18 shows typical plots of shear versus rotation. For the flexible and most likely in the semi-rigid connections, the shear force is the major force affecting the behavior of the connection, and not the moment as in the case of the fixed connection. This type of relation has been investigated by Astaneh and Nader (15). The slope in the shear-rotation plots represents the off-diagonal term relating shear to rotation in the 3x3 stiffness matrix of moment, shear, and axial forces.

$$\begin{bmatrix} M \\ V \\ N \end{bmatrix} = \begin{bmatrix} X & V & W \\ V & Y & U \\ W & U & Z \end{bmatrix} \begin{bmatrix} \Theta \\ \Delta \\ \dot{\alpha} \end{bmatrix} \tag{6.1}$$

where,

M = moment on the connection

V = shear on the connection

N= axial force on the connection

Θ = rotation of the connection

Δ = shear displacement of the connection

α = axial displacement of the connection

To properly understand and represent the behavior of a connection, one should treat it as a structure and develop the complete 3x3 stiffness matrix as given by equation (6.1).

6.2.4 Other Significant Responses

Other structural and local parameters that were studied are summarized in the following section:

(a) The forces in the braces were of interest, because they represent the torsional effect on the structure. Figures 6.19, 6.20, and 6.21, clearly indicate that considerable torsional effects were present in the semi-rigid frame. The torsional mode was also observed in other tests of semi-rigid structure. As noted in the case of the fixed structure the braces show permanent deformation, also the time at which the mass slipped can be easily noted.

(b) The deformation of the column during severe dynamic loading was of interest, so a plot of extreme column deflection with the corresponding forces, for each of the three structures, is shown in Figure 6.22.

(c) The time history of the moment at the base plate of column 4 for each of the structures is plotted in Figure 6.23. The base plate moment response was the same for the three structures. Again the period of vibrations were dominated by the natural period of the structure. The maximum values of the base plate moment for the flexible, semi-rigid, and fixed frames were 200.8, 165.9, and 254.9 k-in. respectively. It is believed that the base plate moment of the semi-rigid structure was less than that of the flexible structure, because of the slight rocking that was observed in the base plates when the semi-rigid structure was being tested.

6.3 Test Data of the Selected Test Runs

6.3.1 Introduction

In this section, results from representative tests are presented. Table 4.1 provides information on shaking table tests that were conducted. For each of the three structures, a free vibration tie-back test was conducted to obtain the dynamic properties of the structures. The dynamic test runs selected, present the full range of the experiments. Of the different plots that were presented in previous section (6.2), a number of plots were found to be necessary to generate for all the selected tests, mainly:

(1) Connection 4: Moment versus Rotation

(2) Connection 4: Shear versus Rotation

(3) Connection 4: Moment versus Shear

(4) Time history of moment of base plate 4

(5) Plot of the fast Fourier amplitude versus frequency

(6) Time history of the base shear

(7) Time history of the lateral drift

(8) Plot of stiffness

Locations of connection 4 and base plate 4 are shown in Figure 5.5.

6.3.2 Important Observations During Testing

In this part a descriptive behavior of the structure response is given. Each of the structures was subjected to similar sequences of dynamic loading. The following sections give important observations that were noted during the testing of each of the structures. As the discussion proceeds, the reader is advised to refer to Tables 4.1, and 4.2.

(i) Semi-Rigid Frame

The semi-rigid frame was the first structure tested. The initial test was a **Pull-Back** test, in which the structure was displaced 0.1 inch, followed by **White Noise Shaking** of intensity of 0.05g. The structure response was elastic as expected, and very small drifts were observed when the White Noise was applied. White Noise was followed by El Centro, Taft, and Mexico-City earthquakes, respectively.

The **El Centro Earthquake** was applied with different intensities (0.05g, 0.15g, 0.20g, 0.25g, 0.35g peak accelerations). In the El Centro Earthquake series, the structure behaved almost elastically, and no significant yielding could be observed, until an intensity of 0.35g was applied. The semi-rigid connections did not experience significant rotation, and behaved almost as fixed connections. When the 0.35g intensity was applied, yielding was observed in all the columns at the areas immediately beneath the connections. Also, the seated bottom angle in both south columns showed a slip of about 1/8 inch. Figure 6.24 shows the observed yielding.

The **Taft Earthquake** was applied after the El Centro Earthquake. The Taft earthquake was also applied with increasing intensities (0.05g, 0.15g, 0.25g, 0.35g, 0.50g). At 0.25g peak acceleration very thin yield lines were observed at the N-W column at the same location as shown in Figure 6.24. At 0.35g peak base acceleration, the base plates were observed to be slightly rocking, which meant that the prestressing of the plates to the shaking table was not enough. Also, at this intensity the torsional mode was clearly apparent. The behavior of the structure was inelastic at 0.5g, and significant yielding and plastic deformations could be observed in the same locations as in Figure 6.24.

The **Mexico Earthquake** was applied with three intensities, 0.05g, 0.35g, 0.50g. At 0.35g intensity the S-W connection showed clear signs of plastic deformations. The shear connection slipped 3/16 inch, and the top and bottom seated angles were bent about 1/8 inch in the out-of-plane direction. Figure 6.25 shows the connection. At 0.50g the N-W and N-E connections also experienced plastic deformations similar to the S-W connection, but more pronounced.

(ii) Flexible Frame

Shaking table tests of the flexible frame were conducted after the tests of the semi-rigid frame. As mentioned in section 4.2 each of the three structures was subjected to the same sequence of dynamic loading.

Tie-back and White Noise: In the pull-back test, higher damping was observed for the flexible frame compared with that of the semi-rigid frame. The response of the structure was very flexible when the White Noise was applied.

El Centro Earthquake: The column deflection under lateral loads was a single curvature deflection, and not a double curvature as in the case of the semi-rigid connection. At 0.35g intensity significant lateral drift was taking place. Also the simple connections showed large slippage and almost no yielding, see to Figure 6.26.

Taft Earthquake: No yielding was observed in the structure during this test even under 0.5g intensity. However, ever-increasing slippage took place in the connections, as the intensity was increased from 0.05g to 0.5g. The slip in shear was about 3/16 inch, while the slip in rotation was about 3/32 inch. The bolt holes on the web of the beam were suspected to be undergoing bearing deformations.

Mexico Earthquake: The behavior of the structure under the Mexico Earthquake was very similar to its behavior under the Taft Earthquake.

(iii) Fixed Frame

The flexible structure tests were followed by the same sequence of tests of the fixed frame.

Tie-back and White Noise: In the pull-back test, the structure was noted to have lesser damping and higher natural frequency of vibration.

El Centro: At 0.25g intensity, some yielding was observed at the weld of the stiffeners in the N-W column at the connection location. At 0.35g yielding was observed in the N-W column, as shown in Figure 6.27.

Taft Earthquake: At 0.25g intensity more yielding in all the columns was noted (Figure 6.27). At 0.35g a very loud noise occurred due to slippage of the concrete blocks acting as the mass. This slip resulted from the very violent response of the structure. At this point, it was decided to stop the test for the following reasons:

(1) There was concern for the safety of the personnel and instrumentation.

(2) At this point enough data had been collected.

(3) The slip of the mass meant that it was no longer a fully reactive mass.

6.3.3 Tables of Extreme Response Values

The data collected is presented in two forms; first, in the form of tables of extreme values of various response parameters, and second, in the form of plots showing variation of important variables.

In Table 6.1, the dynamic properties of the three structures are presented. These values are determined from the **tie-back** tests, and thus represent the elastic dynamic properties of the structures.

In Table 6.2 a comparison between the maximum values of the base shear and lateral drift for flexible, semi-rigid, and fixed structures for different tests, is given.

Tables 6.3 through 6.10 list the maximum values of moment, shear, and rotation that occurred in connection 4, as well as, the moment of base plate 4.

In Table 6.11 a comparison of stiffnesses of the three structures, subjected to different earthquakes, is given. Since the behavior of a structure ranges from elastic to inelastic, the stiffnesses recorded vary. To show the range of stiffness, an elastic stiffness is given, which represents the unloading cycle; followed by an inelastic stiffness, which is the slope of the inelastic portion of the loading branch.

Table 6.12 gives the Fast Fourier Transform reading for the lateral and torsional harmonic periods of vibration of the three structures under different earthquake loads.

6.3.4 Description of the Schedule of Plots

As mentioned earlier, only 8 plots were developed for the 24 selected shaking table tests. These plots are presented in Appendix A. For each of the test runs the the following plots are presented:

(1) Connection 4: Moment versus Rotation

(2) Connection 4: Shear versus Rotation

(3) Connection 4: Moment versus Shear

(4) Time history of the moment of base plate 4

(5) Time history of the base shear

(6) Time history of the lateral drift

(7) Plot of stiffness

(8) Plot of the fast Fourier amplitude versus frequency

CHAPTER SEVEN

SUMMARY AND CONCLUSIONS

The purpose of this investigation was to study and explore the behavior of flexible and semi-rigid structures under dynamic loading. The potential for using semi-rigid structures in low seismic zones was the main drive behind this experimental research. Also, the effect of yielding in the connection zones on the response of different structures was of great interest.

From the data collected and presented in Chapter Six and Appendix A, the following remarks can be made.

(1) As the stiffness of the connection increased, the base shear resulting from the same ground motion increased, while the corresponding lateral drift did not decrease in a similar manner. This type of structural behavior leads to the idea of optimal design and how it can be approached. To design a structure to resist a certain dynamic load, one should search for the optimum system of beam-to-column connections so that the structure would develop the least possible amount of base shear, and yet not have large lateral deformations. In this case of a single story structure, having a fixed connection is not the optimal solution.

(2) To understand and incorporate the real behavior of a connection, a 3x3 stiffness matrix should be established for the connection. This matrix would relate moment, shear, and axial force to rotation, displacement, and elongation.

(3) Energy can be dissipated in a connection in different ways and not only by moment-rotation hysteresis loops. The energy can also be dissipated by axial force - axial displacement hysteresis loops, and shear force - shear displacement hysteresis loops.

(4) Changing the connection type in a structure can drastically change the response characteristics of the structure. This was demonstrated when the torsional mode was actually excited in the case of the semi-rigid structure, although it was not as apparent in the other two types of structures.

(5) The semi-rigid connections behaved almost as a rigid connection in most of the dynamic tests. The moment capacity of the semi-rigid connections turned out to be higher than expected. The catenary forces that were developed in the seated connections, could double the expected plastic moment of such connections. Semi-rigid connections have considerable potential for resisting earthquake loading, and need further study.

Need for Further Research

As can be noted from the experimental results presented in this report, flexible and semi-rigid connections have considerable potential for resisting dynamic loading in low to medium earthquake zones. The behavior of such connections can drastically change the response of a structure. To understand how such connections can be used in optimal design of structural systems, more research is needed. The research should include testing structures with more degrees of freedom and various configurations of connections. It is also necessary to include non-structural elements in the structure under investigation.

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TABLE 2.1. Section Properties

SECTION PROPERTIES				
PROPERTIES	FRAME MEMBERS		TRANSVERSE MEMBERS	
	Column	Girder	Channel	Bracing
	W4x13	W10x15	C9x15	L2x2x1/4
A (in^2)	3.83	4.41	4.41	0.94
d (in)	4.16	9.99	9.00	2.00
t_w (in)	0.28	0.23	0.29	0.25
b_f (in)	4.06	4.00	2.49	2.00
t_f (in)	0.35	0.27	0.41	0.25
wt/ft (#/ft)	13.00	15.00	15.00	3.19
I_{xx} (in^4)	11.30	68.90	51.00	0.35
S_x (in^3)	5.46	13.80	11.30	0.25
I_{yy} (in^4)	3.86	2.89	1.93	0.35
S_x (in^3)	1.90	1.45	1.01	0.25
J (in^4)	0.15	0.10	...	...
Z_x (in^3)	6.28	16.00	...	...
Z_y (in^3)	2.92	2.30	...	...

TABLE 2.2. **Weight of the Structure**

WEIGHT OF STRUCTURE			
Member	Weight/foot (lbs/ft)	Length * (feet)	Weight (lbs)
Column	13.0	13.33	173.3
Beam	15.0	11.20	168.0
Channel	15.0	18.70	280.5
Bracing	3.2	22.40	71.7
Connections	...	...	30.0
Floor Diaphragm	...	...	2300.0
Added Mass	...	...	24400.0
TOTAL WEIGHT =			27423.5

* length included for columns and diagonal bracings are from mid height of structure to floor level

TABLE 3.1. Allocation of Various Transducers to Specific Channels

CHANNEL ALLOCATION		
CHANNEL #	NAME	UNITS
1	Horizontal Disp.1	inches
2	Horizontal Disp.2	inches
3	Av. Horizontal Acc.	g's
4	Av. Vertical Acc.	g's
5	Pitch Acceleration	rad/sec^2
6	Roll Acceleration	rad/sec^2
7	not activated	...
8	Vertical Disp.1	inches
9	Vertical Disp.2	inches
10	Vertical Disp.3	inches
11	Horizantal Velocity	inches/sec
Channels 12 through 25 were not activated		
26	Accelerometer 1	g's
27	Accelerometer 2	g's
28	Accelerometer 3	g's
29	Accelerometer 4	g's
30	Accelerometer 5	g's
Channels 31 through 33 were not activated		
34	DCDT 1	inches
35	DCDT 2	inches
36	DCDT 3	inches
37	DCDT 4	inches
38	DCDT 5	inches
39	DCDT 6	inches
40	DCDT 7	inches
41	DCDT 8	inches
42	DCDT 9	inches
43	DCDT 10	inches
44	DCDT 11	inches
45	DCDT 12	inches
46	DCDT 13	inches
47	DCDT 14	inches
48	DCDT 15	inches
49	DSDT 16	inches
50	Potentiometer 1	inches
51	Potentiometer 2	inches
52	Potentiometer 3	inches
53	Potentiometer 4	inches
54	Potentiometer 5	inches
55	Potentiometer 6	inches
56	Potentiometer 7	inches
57	Potentiometer 8	inches
58	Potentiometer 9	inches
59	Strain Gage 10	mstrain
60	Strain Gage 11	mstrain

CHANNEL ALLOCATION		
CHANNEL #	NAME	UNITS
61	Strain Gage 12	mstrain
62	Strain Gage 13	mstrain
63	Strain Gage 14	mstrain
64	Strain Gage 15	mstrain
65	Strain Gage 16	mstrain
66	Strain Gage 17	mstrain
67	Strain Gage 18	mstrain
68	Strain Gage 19	mstrain
69	Strain Gage 20	mstrain
70	Strain Gage 21	mstrain
71	Strain Gage 22	mstrain
72	Strain Gage 23	mstrain
73	Strain Gage 24	mstrain
74	Strain Gage 25	mstrain
75	Strain Gage 26	mstrain
76	Strain Gage 27	mstrain
77	Strain Gage 28	mstrain
78	Strain Gage 29	mstrain
79	Strain Gage 30	mstrain
80	Strain Gage 31	mstrain
81	Strain Gage 32	mstrain
82	Strain Gage 33	mstrain
83	Strain Gage 34	mstrain
84	Strain Gage 35	mstrain
85	Strain Gage 36	mstrain
86	Strain Gage 37	mstrain
87	Strain Gage 38	mstrain
88	Strain Gage 39	mstrain
89	Strain Gage 40	mstrain
90	Strain Gage 41	mstrain
91	Strain Gage 42	mstrain
92	Strain Gage 43	mstrain
93	Strain Gage 44	mstrain
94	Strain Gage 45	mstrain
95	Strain Gage 46	mstrain
96	Strain Gage 47	mstrain
97	Strain Gage 48	mstrain
98	Strain Gage 49	mstrain
99	Strain Gage 51	mstrain
100	Strain Gage 52	mstrain
101	Strain Gage 53	mstrain
102	Strain Gage 54	mstrain
103	Strain Gage 55	mstrain
104	Strain Gage 56	mstrain
105	Strain Gage 57	mstrain
106	Strain Gage 58	mstrain

Table 4.1.a. Sequence of Testing

SEQUENCE OF TESTING					
Semi-Rigid Connection					
FILENAME	EXCITATION SIGNAL **	DURATION of SIGNAL (sec)	RATE (sec)	REMARKS	
				INTERVAL (sec)	SPAN ***
880705.01 *	f.v.	20	.005	...	...
880705.02	r.30.d	35	.005	...	250
880705.03	ec2	35	.005	0.02	55
880705.04 *	ec2	35	.005	0.02	110
880705.05	ec2	35	.005	0.02	275
880705.06 *	ec2	35	.005	0.02	362
880705.07 *	ec2	35	.005	0.02	551
880705.08	r.30.d	35	.005	...	110
880705.09	taft2	35	.005	0.02	104
880705.10 *	taft2	35	.005	0.02	312
880705.11	taft2	35	.005	0.02	520
880705.12 *	taft2	35	.005	0.02	758
880705.13 *	taft2	35	.005	0.02	1000
880705.14	sct.o	13	.005	0.0035	50
880705.15 *	sct.o	13	.005	0.0035	116
880705.16 *	sct.o	13	.005	0.0035	166

* selected tests reported here

** f.v. = Free Vibration

r.30.d = Random White Noise

ec2 = El-Centro S00E Earthquake

taft2 = Taft N21E Earthquake

sct.0 = SCT S60E Mexico City Earthquake

*** a span of 1000 corresponds to 5 inches of shaking table displacement

TABLE 4.1.b. Sequence of Testing

SEQUENCE OF TESTING					
Flexible Connection					
FILENAME	EXCITATION SIGNAL **	DURATION of SIGNAL (sec)	RATE (sec)	REMARKS	
				INTERVAL (sec)	SPAN ***
880706.01	f.v.	20	.005	...	...
880706.02	r.30.d	35	.005	...	250
880706.03	ec2	35	.005	0.02	55
880706.04	ec2	35	.005	0.02	110
Retightened the prestress rods attached to the base plates to 60 ksi.					
880706.05 *	f.v.	20	.005	...	...
880706.06	r.30.d	35	.005	...	110
880706.07	ec2	35	.005	0.02	55
880706.08 *	ec2	35	.005	0.02	110
880706.09	ec2	35	.005	0.02	275
880706.10 *	ec2	35	.005	0.02	362
880706.11 *	ec2	35	.005	0.02	551
880706.12	r.30.d	35	.005	...	110
880706.13	taft2	35	.005	0.02	104
880706.14 *	taft2	35	.005	0.02	312
880706.15	taft2	35	.005	0.02	520
880706.16 *	taft2	35	.005	0.02	758
880706.17 *	taft2	35	.005	0.02	1000
880706.18	sct.o	13	.005	0.0035	50
880706.19 *	sct.o	13	.005	0.0035	116
880706.20 *	sct.o	13	.005	0.0035	166

* selected tests reported here

** same notation are used as in Table 4.1.a

*** a span of 1000 corresponds to 5 inches of shaking table displacement

TABLE 4.1.c. Sequence of Testing

SEQUENCE OF TESTING					
Fixed Connection					
FILENAME	EXCITATION SIGNAL **	DURATION of SIGNAL (sec)	RATE (sec)	REMARKS	
				INTERVAL (sec)	SPAN ***
880707.01 *	f.v.	20	.005	...	...
880707.02	r.30.d	35	.005	...	250
880707.03	ec2	35	.005	0.02	55
880707.04 *	ec2	35	.005	0.02	110
880707.05	ec2	35	.005	0.02	275
880707.06 *	ec2	35	.005	0.02	362
880707.07 *	ec2	35	.005	0.02	551
880707.08	r.30.d	35	.005	...	250
880707.09	taft2	35	.005	0.02	104
880707.10 *	taft2	35	.005	0.02	312
880707.11	taft2	35	.005	0.02	520
880707.12 *	taft2	35	.005	0.02	758

* selected tests reported here
** f.v. = Free Vibration
r.30.d = Random White Noise
ec2 = El-Centro S00E Earthquake
taft2 = Taft N21E Earthquake
sct.o = SCT S60E Mexico City Earthquake
*** a span of 1000 corresponds to 5 inches of shaking table displacemrnt

Table 4.2. Contents of Magnetic Tapes
Which Save the Data

CONTENTS OF TAPES		
Tape 1	Tape 2	
Semi-rigid	Flexible	Fixed
880701.01 *	880706.01	880707.01
880701.02 *	880706.02	880707.02
880701.03 *	880706.03	880707.03
880701.04 *	880706.04	880707.04
880701.05 *	880706.05	880707.05
880705.01	880706.06	880707.06
880705.02	880706.07	880707.07
880705.03	880706.08	880707.08
880705.04	880706.09	880707.09
880705.05	880706.10	880707.10
880705.06	880706.11	880707.11
880705.07	880706.12	880707.12
880705.08	880706.13	
880705.09	880706.14	
880705.10	880706.15	
880705.11	880706.16	
880705.12	880706.17	
880705.13	880706.18	
880705.14	880706.19	
880705.15	880706.20	
880705.16		

* preliminary tests with load cells

TABLE 6.1. Elastic Dynamic Properties

FREE VIBRATION			
Structure	Damping (%)	Period of Vibration (sec)	
		FFT	Cycles/time
Flexible	1.87	0.44	0.44
Semi-rigid	0.50	0.33	0.31
Fixed	0.67	0.30	0.31

TABLE 6.2. Base Shear and Lateral Drift

MAXIMUM VALUES OF BASE SHEAR AND LATERAL DRIFT						
Earthquake Signal and Intensity	Flexible Structure		Semi-rigid Structure		Fixed Structure	
	Shear (kips)	Drift (in.)	Shear (kips)	Drift (in.)	Shear (kips)	Drift (in.)
El-Centro 0.15g	4.14	0.42	4.62	0.20	5.40	0.23
El-Centro 0.25g	8.10	1.09	11.76	0.56	14.95	0.61
El-Centro 0.35g	10.00	2.08	18.81	1.15	18.12	0.82
Taft 0.15g	5.35	0.61	9.14	0.55	16.82	0.60
Taft 0.35g	9.52	1.57	20.00	1.41	25.88	1.22
Taft 0.50g	11.49	2.00	24.28	2.35	N.C	N.C
Mexico 0.35g	9.22	1.45	21.2	2.00	N.C	N.C
Mexico 0.50g	12.26	2.05	22.26	2.30	N.C	N.C

N.C = test was not conducted

TABLE 6.3. Extreme Response Values For 0.15g El Centro

0.15g El-Centro Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	14.0	44.1	48.5
Connection Shear * (kips)	1.08	1.94	1.90
Connection Rotation * (radians)	0.0071	0.0007	0.0006
Base Moment (k-in) **	77.3	51.2	57.8

* Moment, shear, and rotation for connection # 1

** Moment for base plate # 1

TABLE 6.4. Extreme Response Values For 0.25g El-Centro

0.25g El Centro Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	33.9	123.2	146.0
Connection Shear * (kips)	2.16	4.61	4.59
Connection Rotation * (radians)	0.022	0.0020	0.0016
Base Moment (k-in) **	166.9	123.6	151.4

* Moment, shear, and rotation for connection # 1

** Moment for base plate # 1

TABLE 6.5. Extreme Response Values For 0.35g El Centro

0.35g El Centro Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	112.5	183.8	175.9
Connection Shear * (kips)	2.11	7.34	5.62
Connection Rotation * (radians)	0.040	0.008	0.0018
Base Moment (k-in) **	247.6	214.3	167.7

* Moment, shear, and rotation for connection # 1
** Moment for base plate # 1

TABLE 6.6. Extreme Response Values For 0.15g Taft

0.15g Taft Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	27.4	107.5	161.2
Connection Shear * (kips)	1.34	4.25	5.50
Connection Rotation * (radians)	0.015	0.003	0.0018
Base Moment (k-in) **	111.9	101.4	152.9

* Moment, shear, and rotation for connection # 1
** Moment for base plate # 1

TABLE 6.7. Extreme Response Values For 0.35g Taft

0.35g Taft Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	44.6	200.7	251.2
Connection Shear * (kips)	1.93	7.86	11.97
Connection Rotation * (radians)	0.026	0.008	0.0033
Base Moment (k-in) **	200.8	165.9	254.9

* Moment, shear, and rotation for connection # 1

** Moment for base plate # 1

TABLE 6.8. Extreme Response Values For 0.50g Taft

0.50g Taft Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	59.5	228.0	N.C
Connection Shear * (kips)	2.47	8.70	N.C
Connection Rotation * (radians)	0.037	0.031	N.C
Base Moment (k-in) **	239.9	292.0	N.C

* Moment, shear, and rotation for connection # 1

** Moment for base plate # 1

N.C = test was not conducted

TABLE 6.9. Extreme Response Values For 0.35g Mexico

0.35g Mexico Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	40.8	203.2	N.C
Connection Shear * (kips)	1.58	7.67	N.C
Connection Rotation * (radians)	0.027	0.024	N.C
Base Moment (k-in) **	196.4	238.7	N.C

* Moment, shear, and rotation for connection # 1
** Moment for base plate # 1
N.C = test was not conducted

TABLE 6.10. Extreme Response Values For 0.50g Mexico

0.50g Mexico Earthquake			
Variable	Flexible	Semi-rigid	Fixed
Connection Moment * (k-in.)	59.5	225.6	N.C
Connection Shear * (kips)	2.26	9.46	N.C
Connection Rotation * (radians)	0.033	0.030	N.C
Base Moment (k-in) **	253.6	281.0	N.C

* Moment, shear, and rotation for connection # 1
** Moment for base plate # 1
N.C = test was not conducted

TABLE 6.11. Structure Stiffnesses

STIFFNESS OF THE THREE STRUCTURES						
Earthquake Signal and Intensity	Flexible Structure		Semi-rigid Structure		Fixed Structure	
	Elastic "Unloading" (k/in)	Inelastic "Loading" (k/in)	Elastic "Unloading" (k/in)	Inelastic "Loading" (k/in)	Elastic "Unloading" (k/in)	Inelastic "Loading" (k/in)
El-Centro 0.15g	10.33	10.33	22.57	22.57	25.2	25.2
El-Centro 0.25g	10.33	6.49	23.14	19.06	24.0	24.0
El-Centro 0.35g	7.77	4.30	19.81	16.1	23.5	23.5
Taft 0.15g	9.88	7.43	17.46	17.46	24.05	24.05
Taft 0.35g	10.14	5.27	14.58	12.4	23.65	21.38
Taft 0.50g	8.16	5.21	18.00	13.20	N.C	N.C
Mexico 0.35g	8.47	5.42	18.38	8.38	N.C	N.C
Mexico 0.50g	9.62	5.39	16.81	7.61	N.C	N.C

N.C = test was not conducted

TABLE 6.12. Natural Periods of Vibration

PERIODS OF VIBRATION USING FFT *						
Earthquake Signal and Intensity	Flexible Structure		Semi-rigid Structure		Fixed Structure	
	Lateral (sec)	Torsional (sec)	Lateral (sec)	Torsional (sec)	Lateral (sec)	Torsional (sec)
El-Centro 0.15g	0.47	0.23	0.33	0.17	0.36	0.17
El-Centro 0.25g	0.48	0.24	0.35	0.19	0.38	0.18
El-Centro 0.35g	0.52	0.23	0.35	0.19	0.38	0.17
Taft 0.15g	0.48	0.22	0.40	0.20	0.32	0.17
Taft 0.35g	0.50	0.25	0.40	0.21	0.36	0.18
Taft 0.50g	0.53	0.25	0.46	0.23	N.C	N.C
Mexico 0.35g	0.49	0.25	0.48	0.22	N.C	N.C
Mexico 0.50g	0.53	0.25	0.44	0.25	N.C	N.C

N.C = test was not conducted

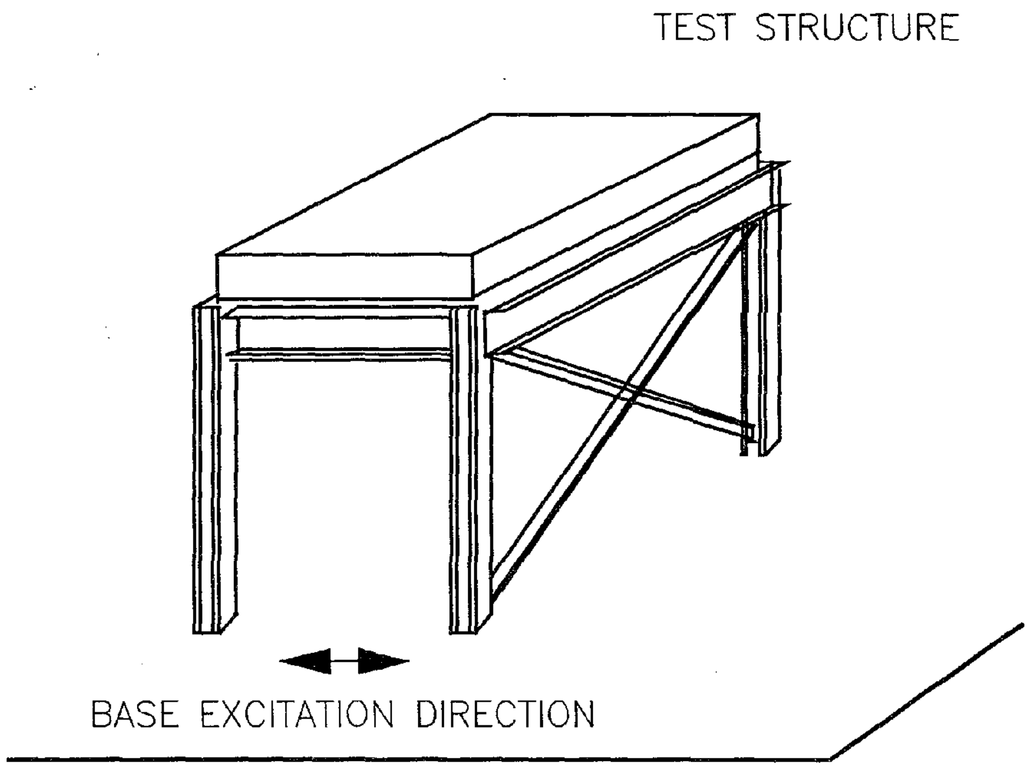


Figure 1.1 Test Structure

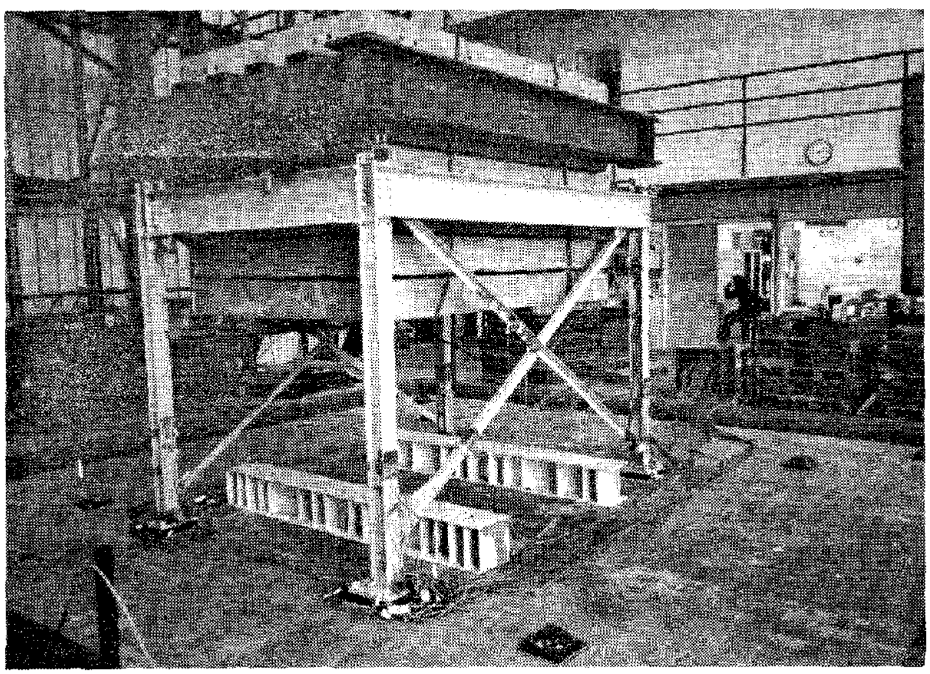
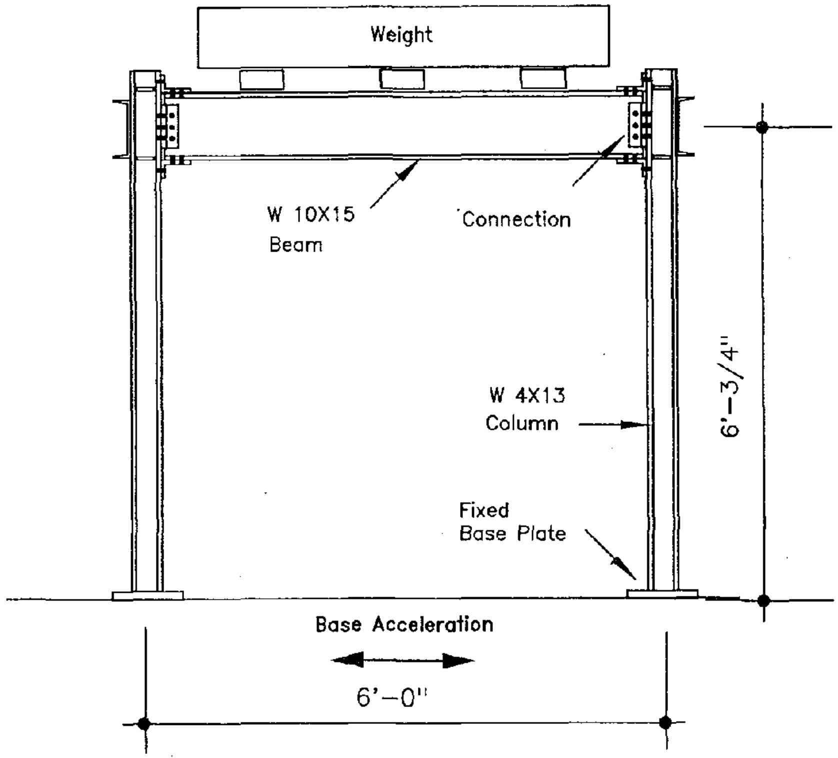


Figure 2.1 Test Structure Mounted on the Shaking Table



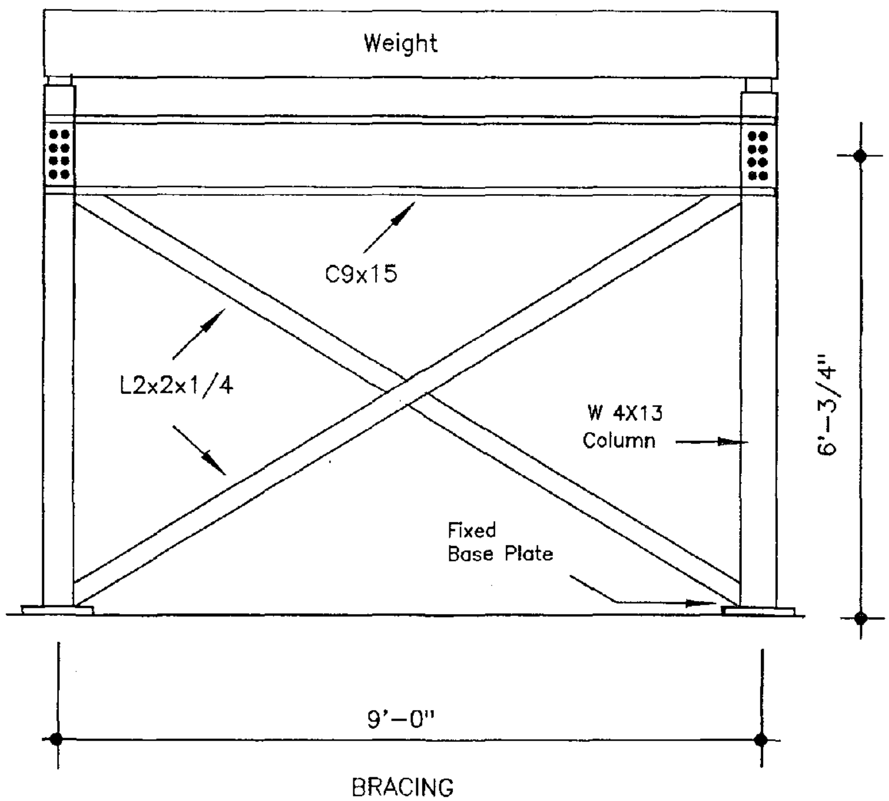


Figure 2.2 Dimensions of the Test Structure

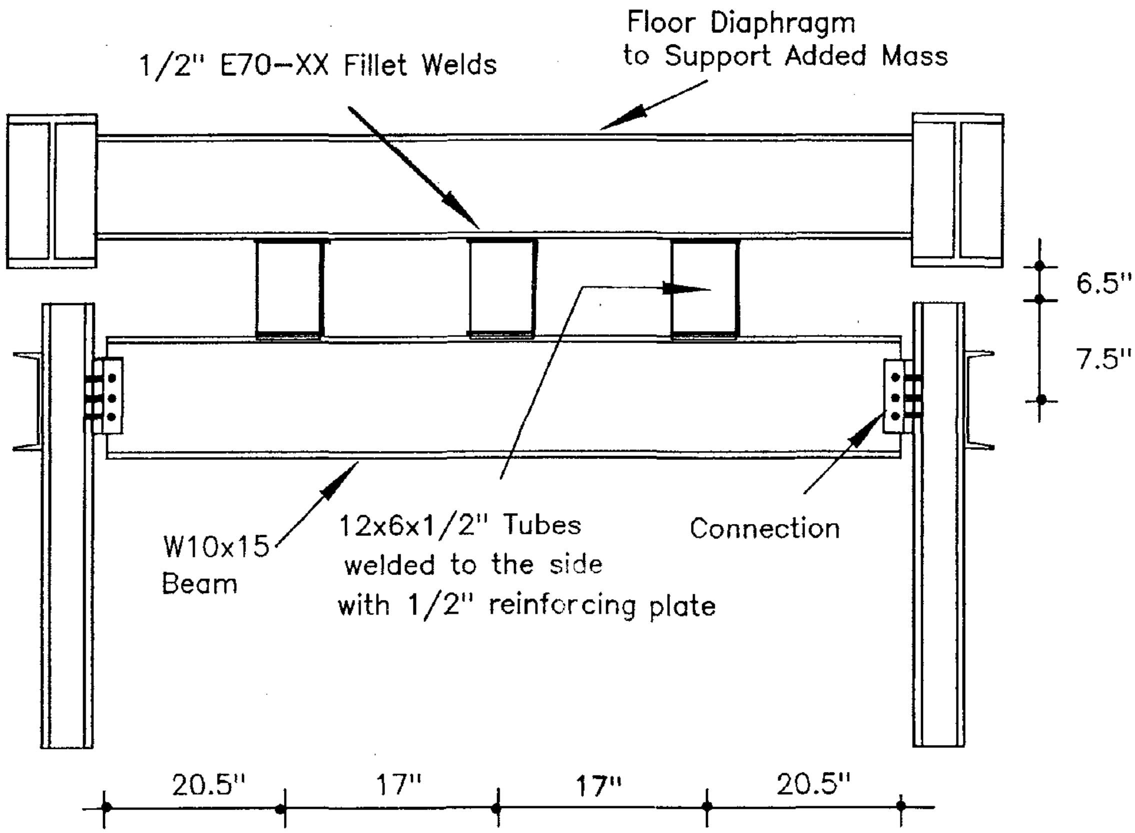


Figure 2.3 Floor System and Floor-to-Beam

Connection Detail

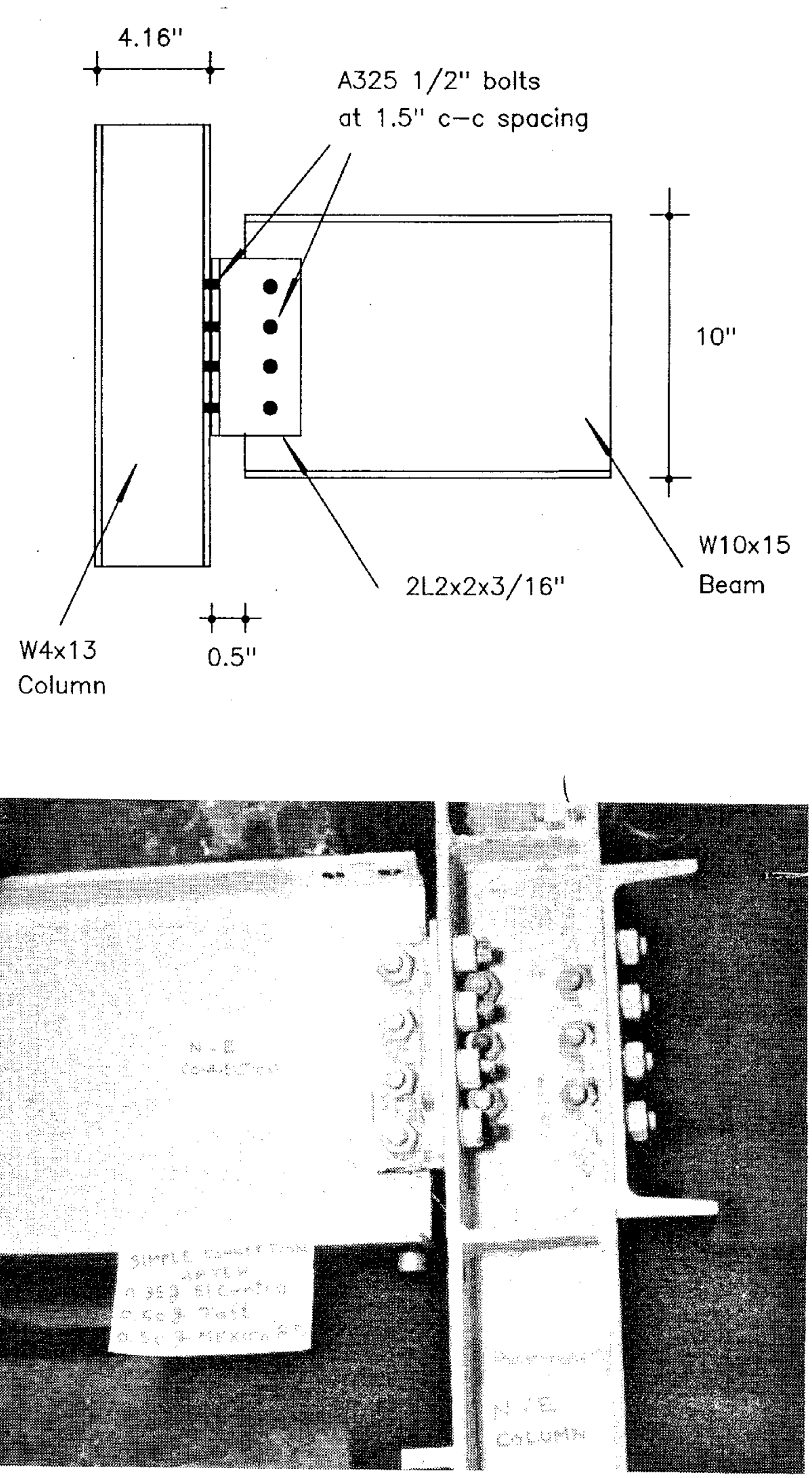
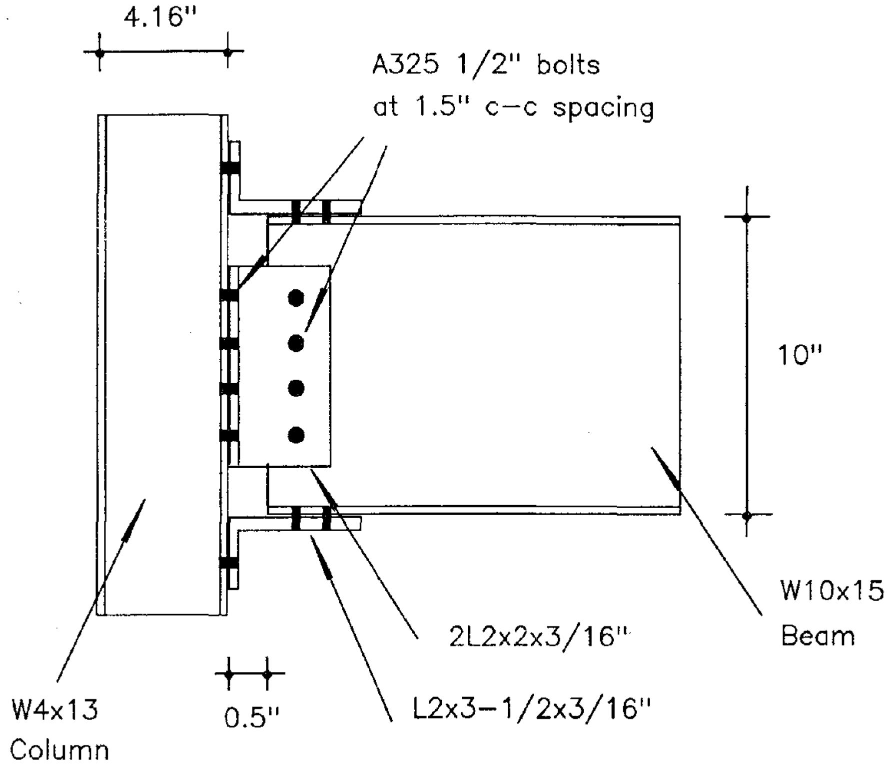


Figure 2.4 Flexible Connection Detail



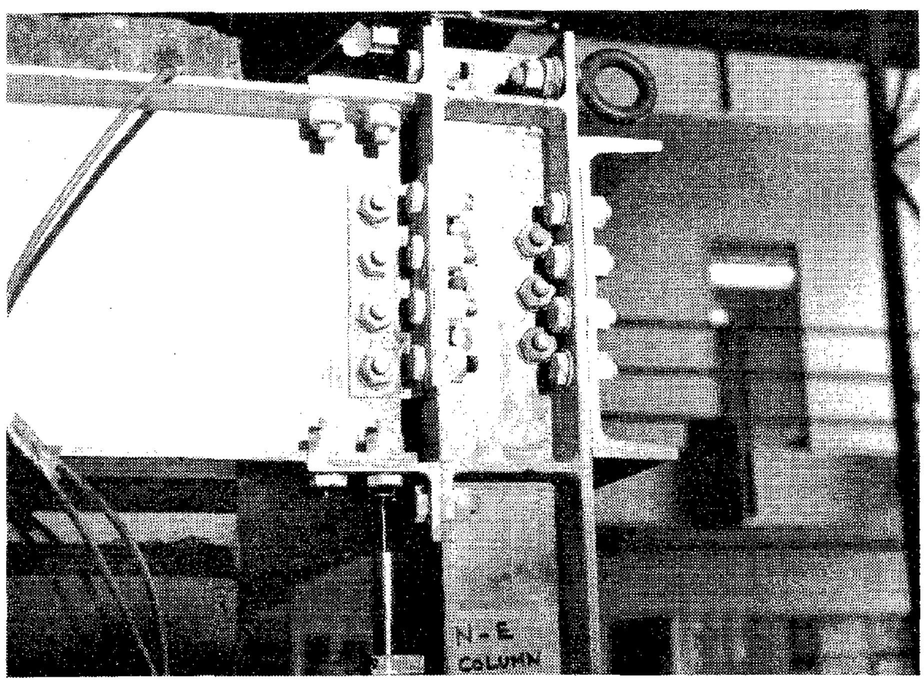
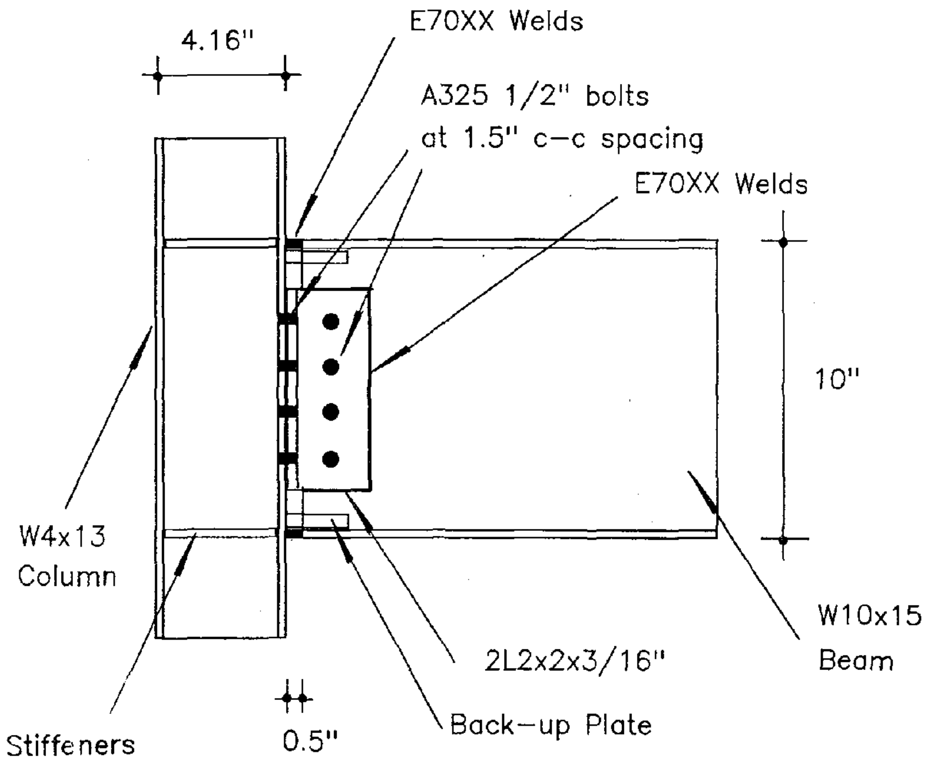


Figure 2.5 Semi-Rigid Connection Detail



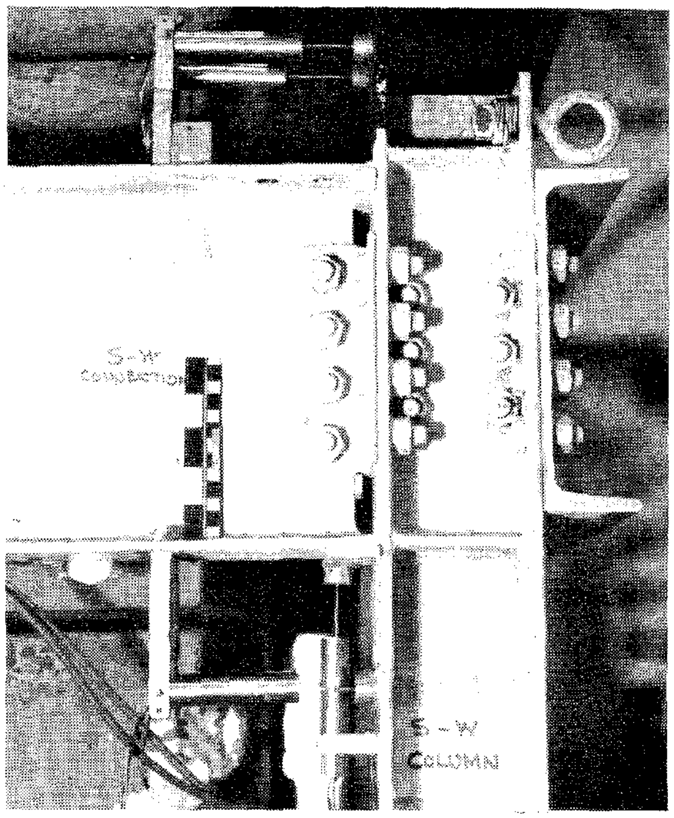


Figure 2.6 Fixed Connection Detail

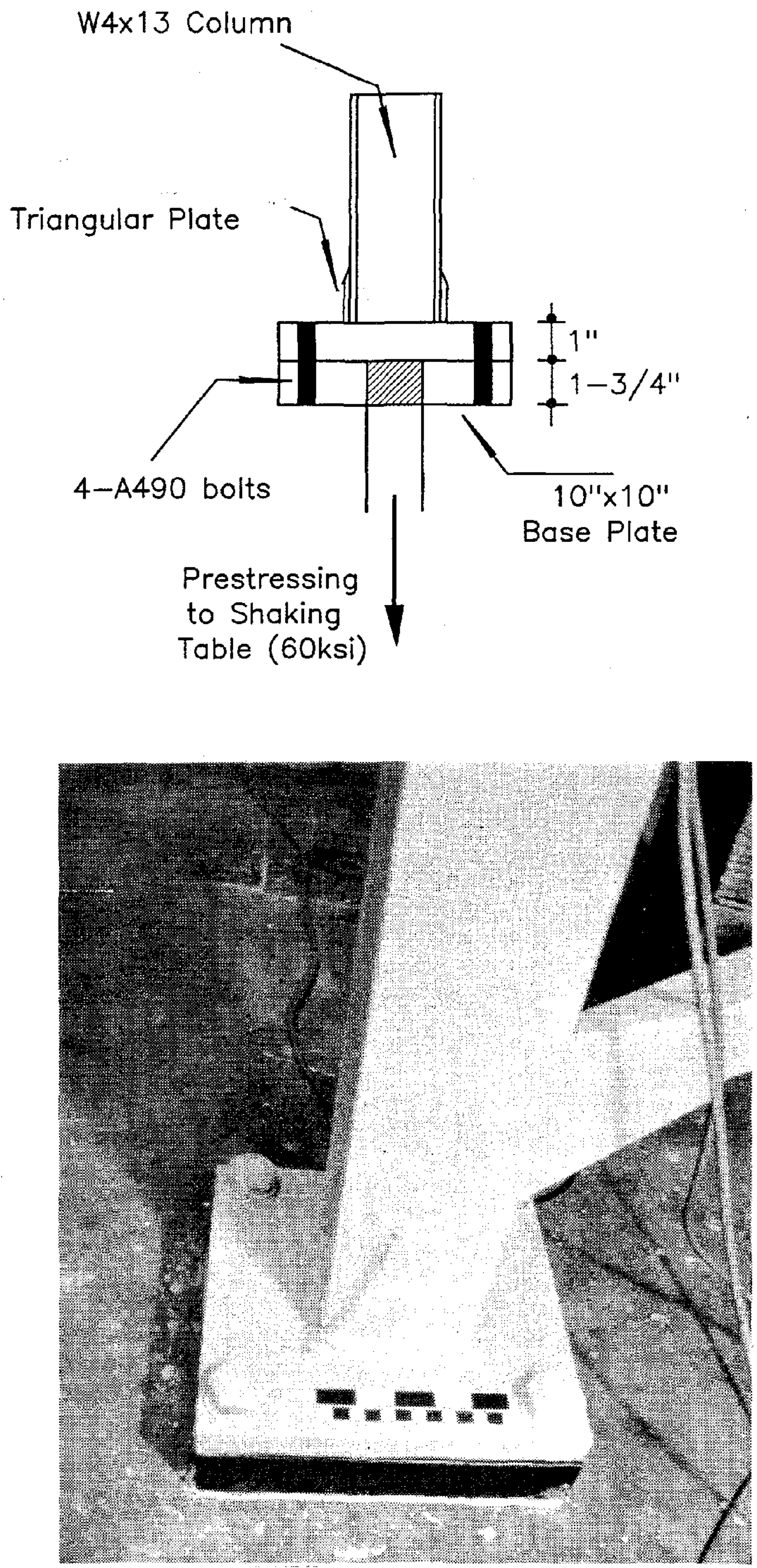


Figure 2.7 Base Plate Detail

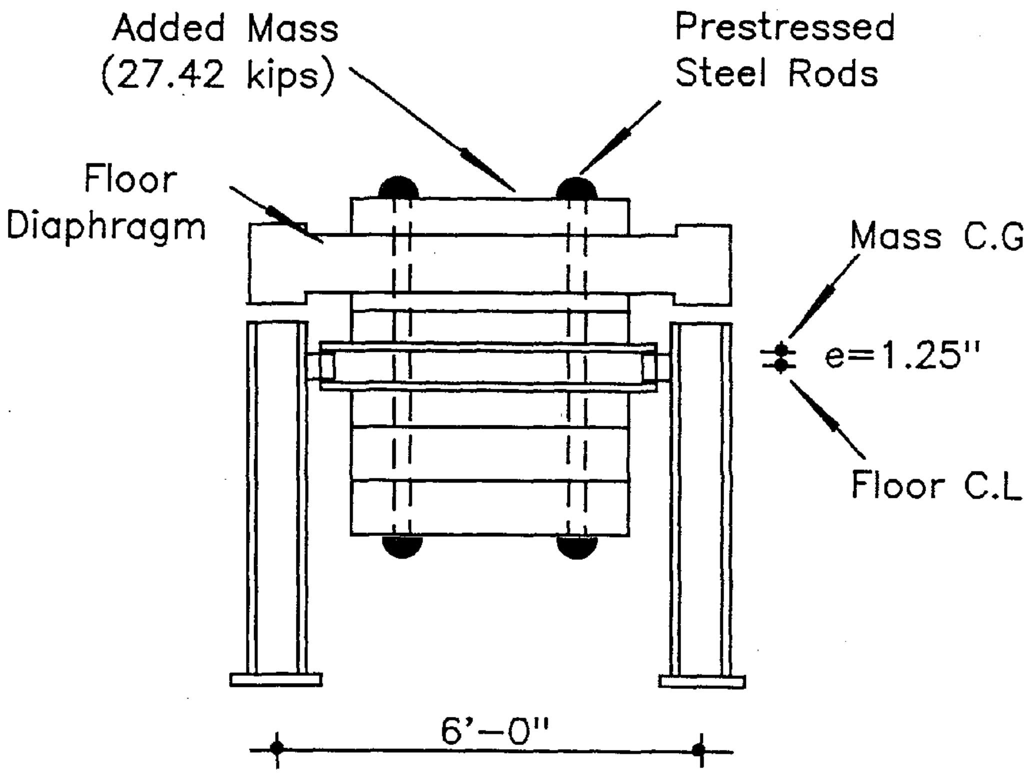


Figure 2.8 Added Mass on Structure

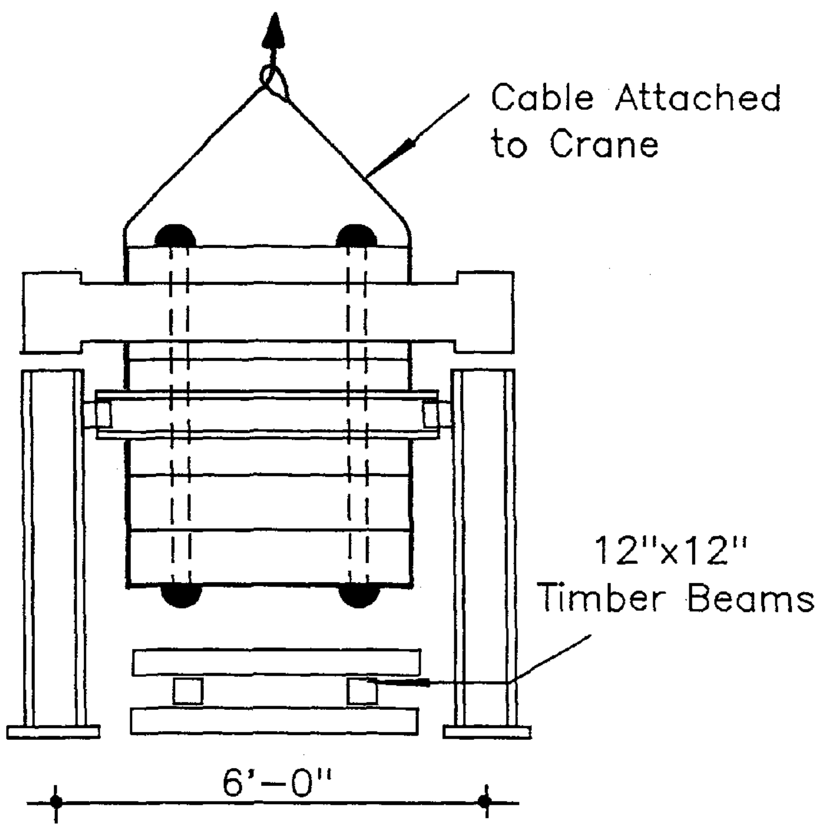
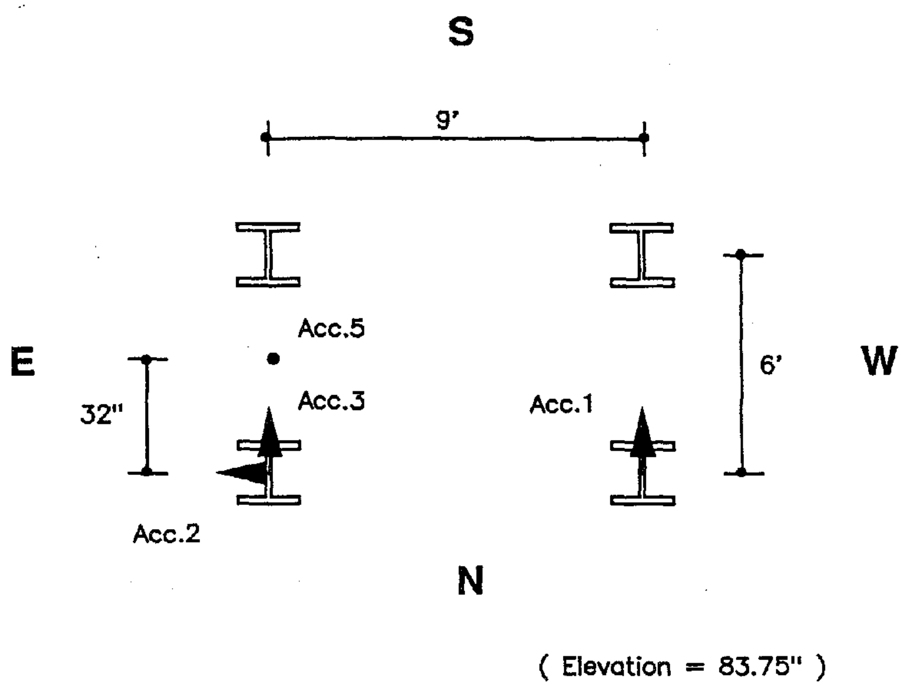


Figure 2.9 Safety Considerations



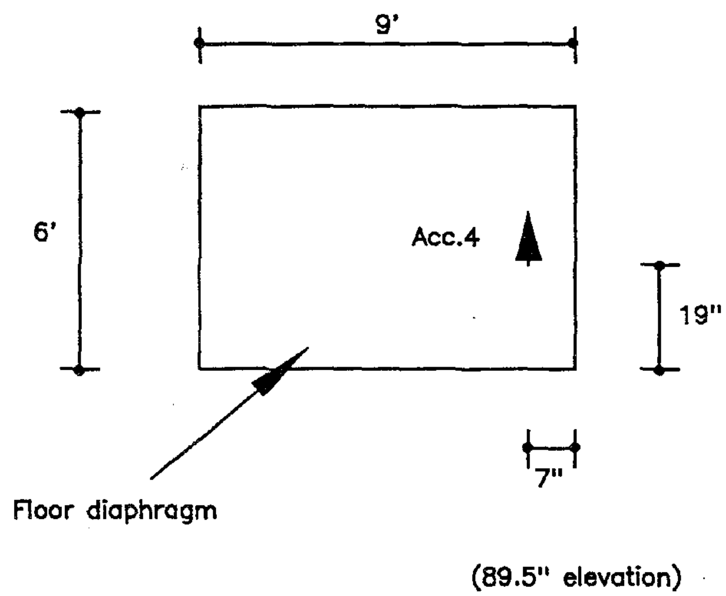


Figure 3.1 Location of Accelerometers

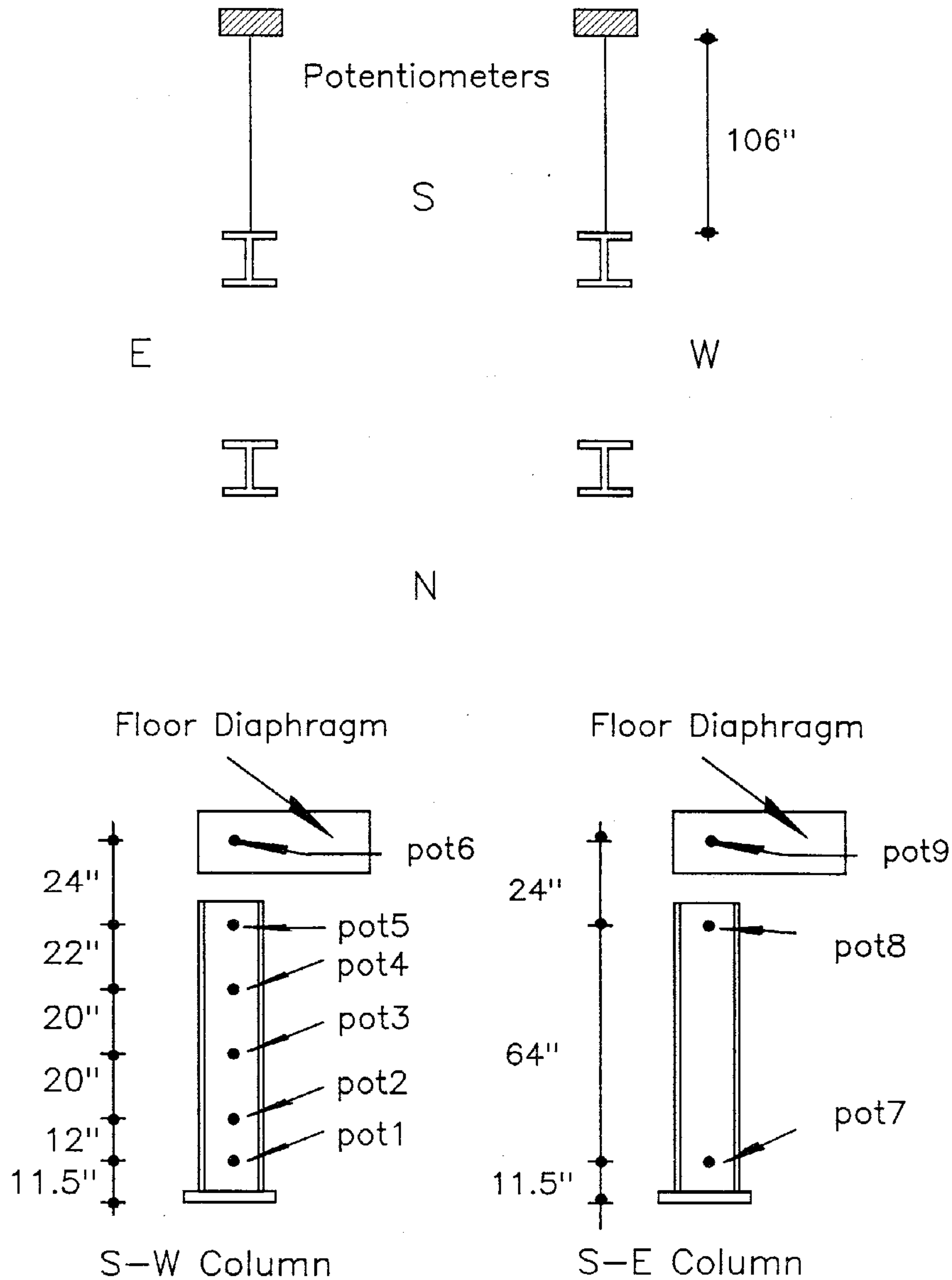


Figure 3.2 Location of Potentiometers

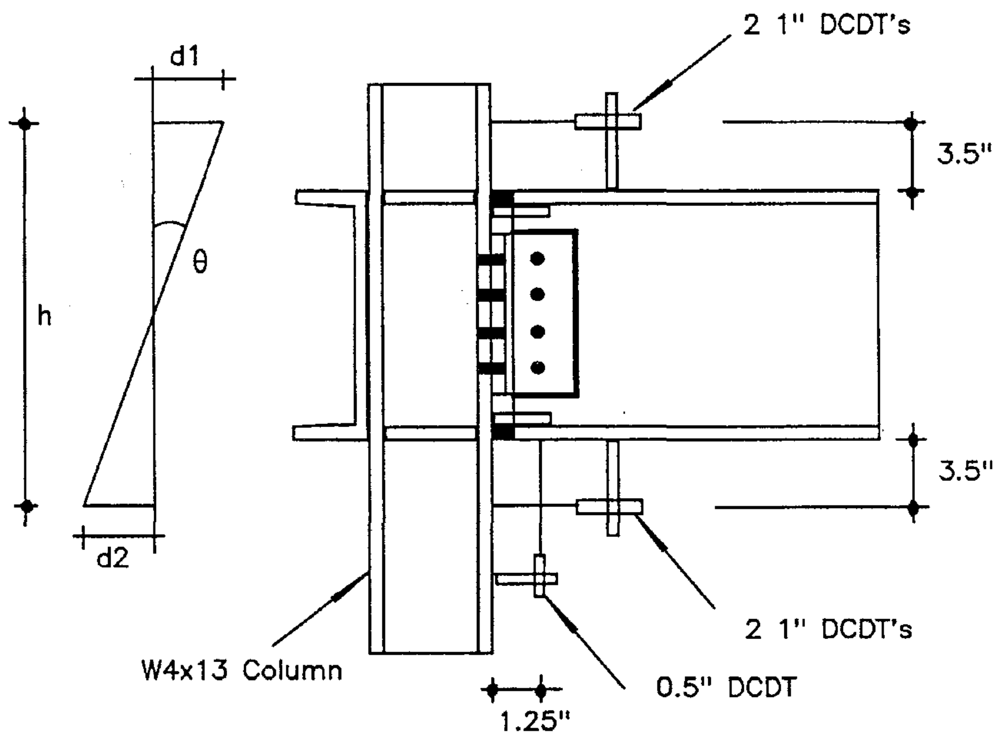


Figure 3.3 Method of Measuring Connection Rotation

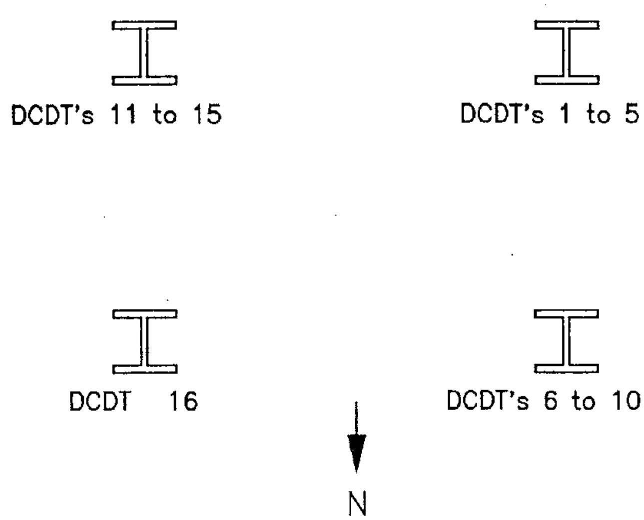
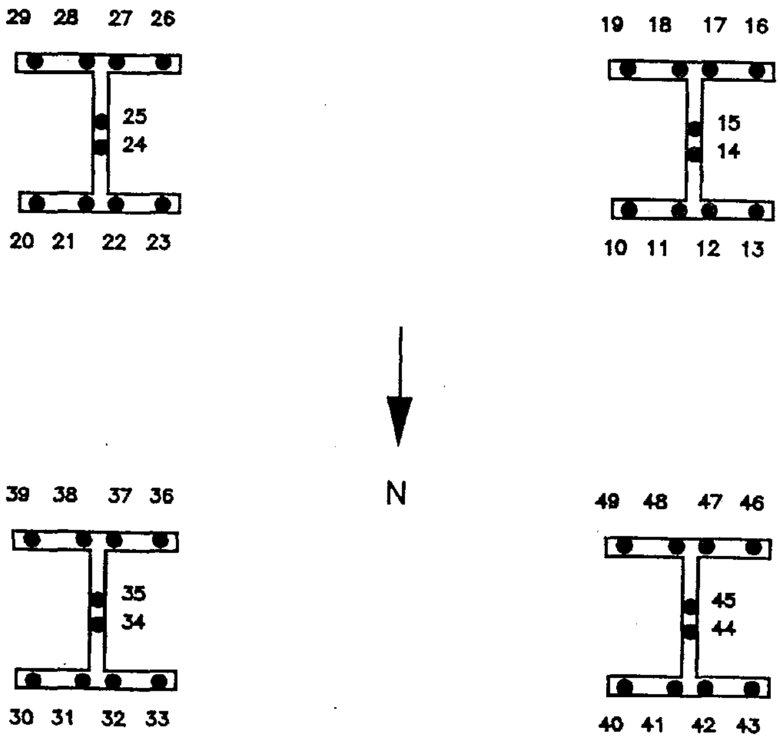


Figure 3.4 Location of DCDT's



The Strain Gages are at 24" from the Top of the Base Plate

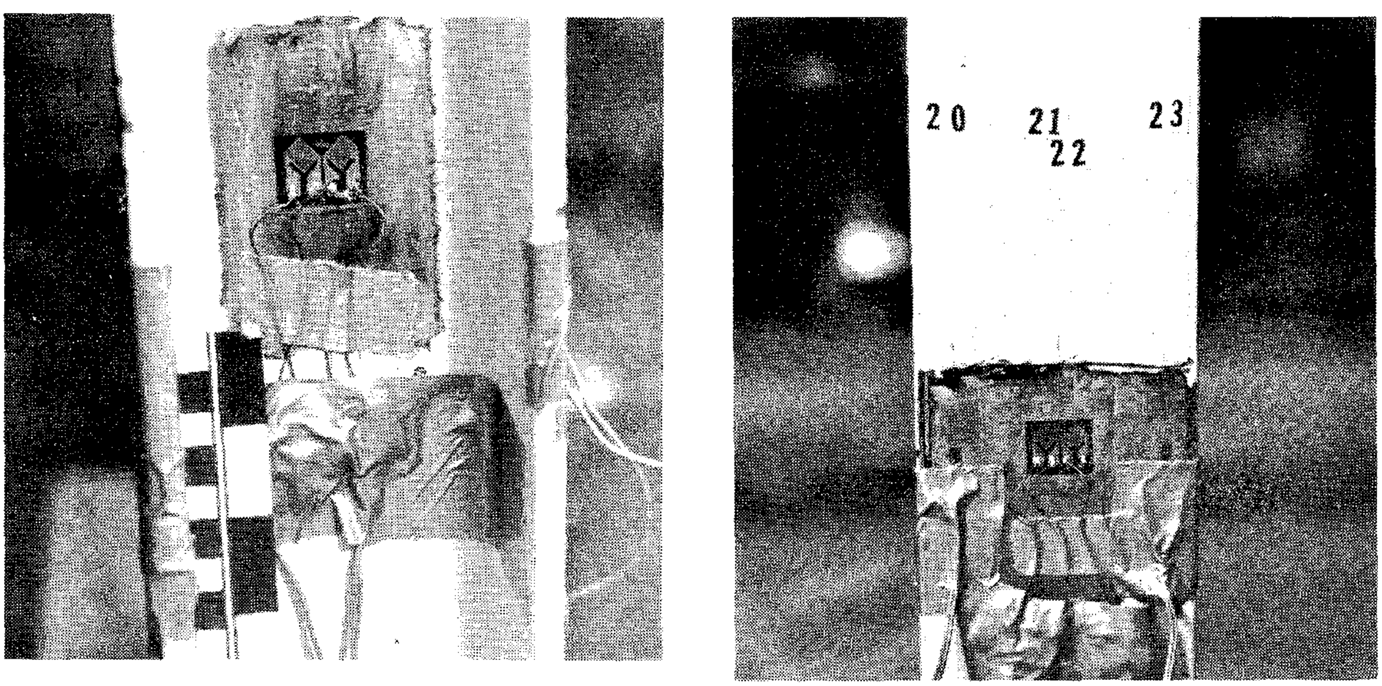
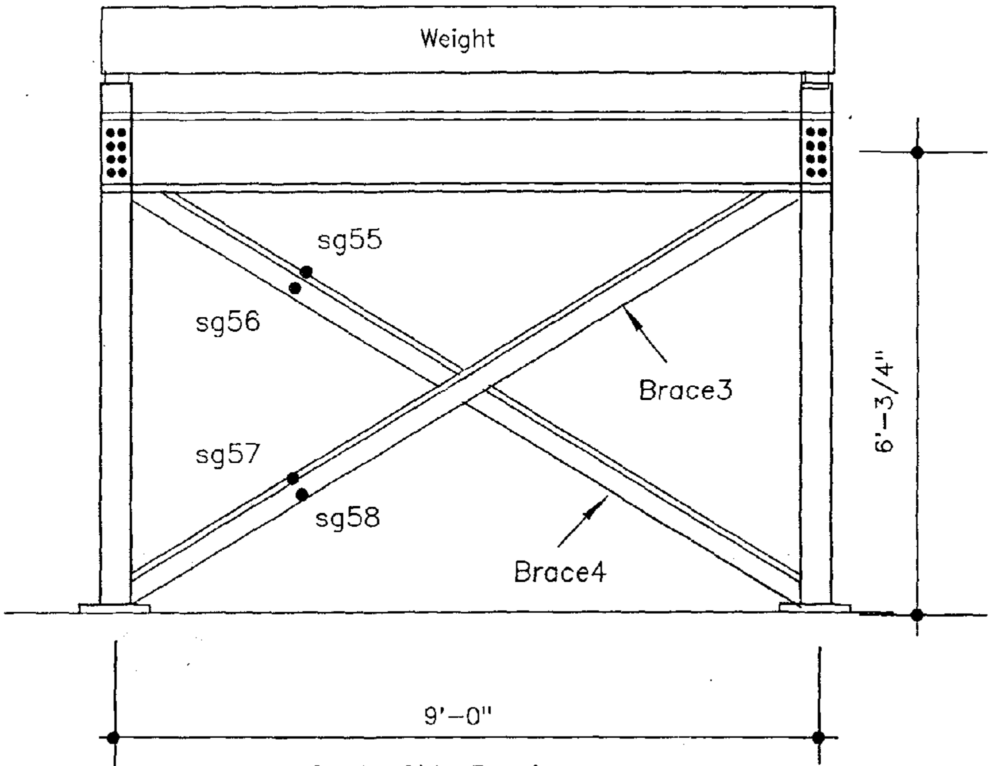
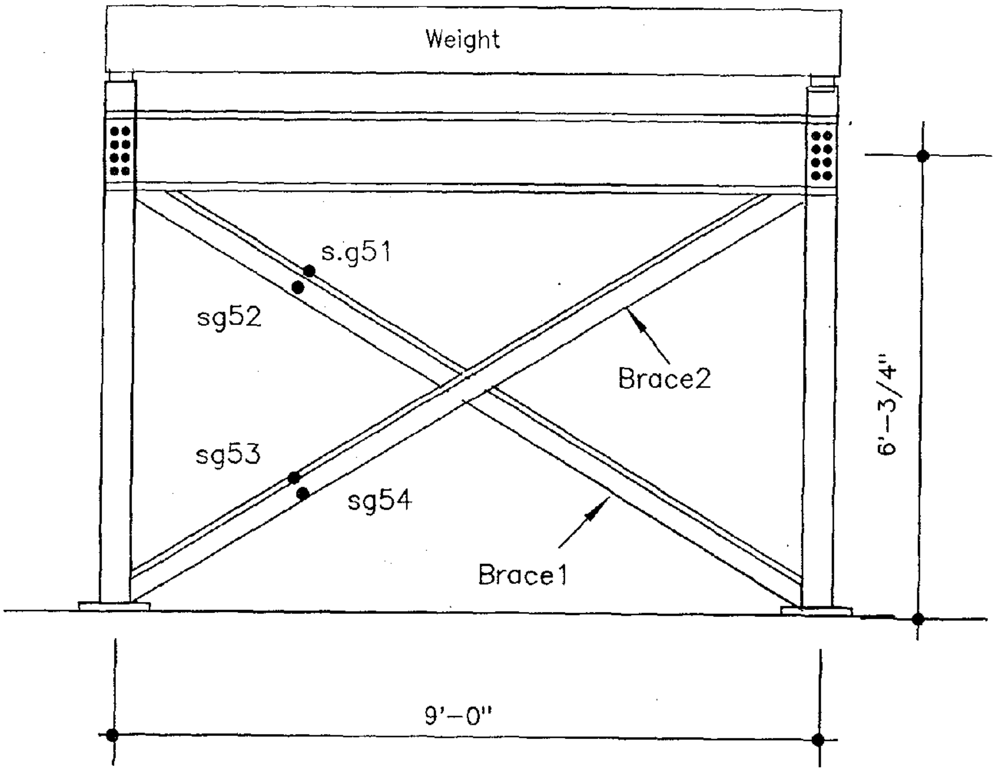


Figure 3.5 Location of Strain Gages on Columns



South Side Bracing

note: Strain Gages are at 26" from edge of column



North Side Bracing

note: Strain Gages are at 26" from edge of column

Figure 3.6 Location of Strain Gages on Diagonal Bracings

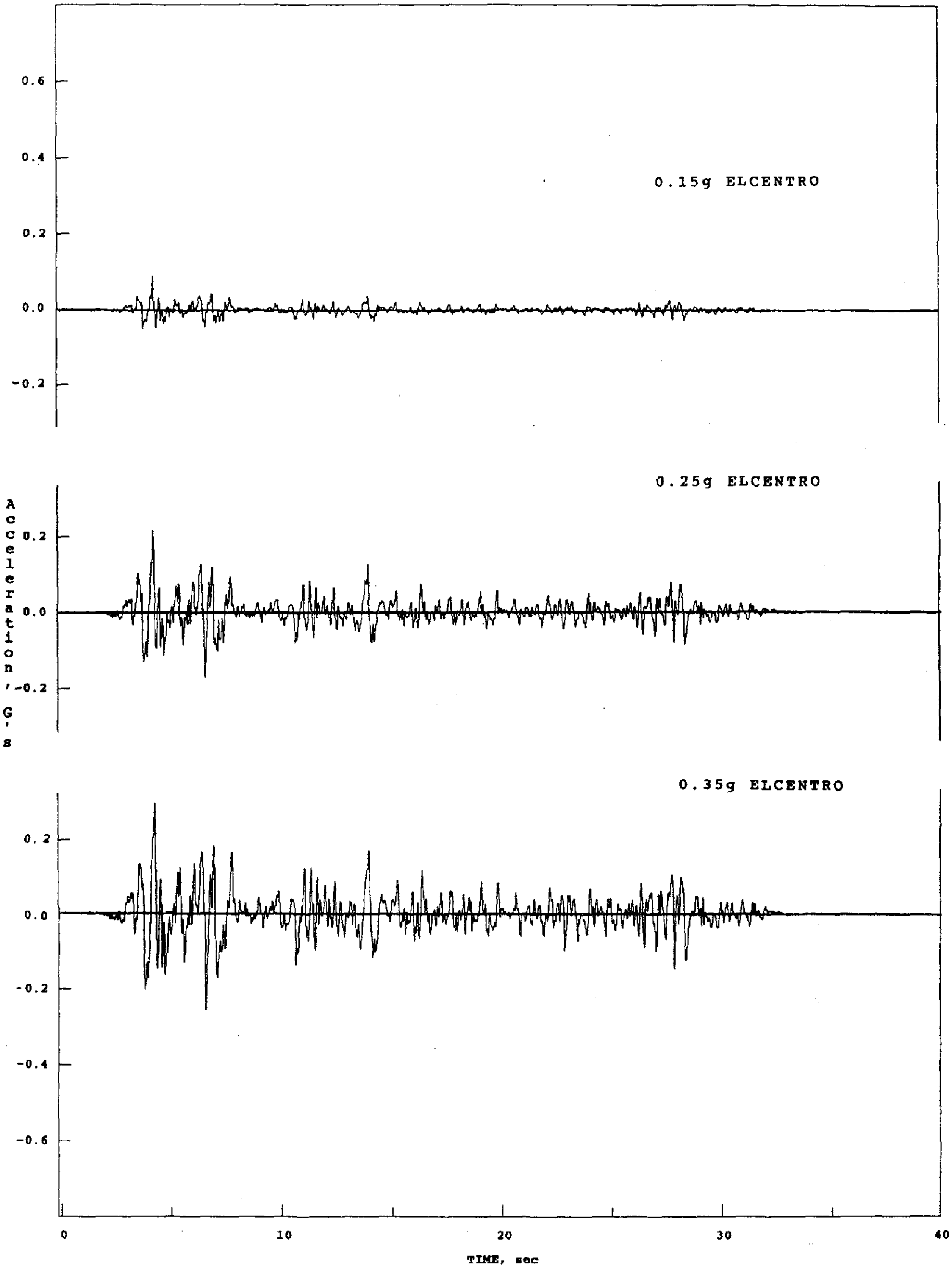


Figure 4.1 Acceleration Time History of El Centro Earthquake

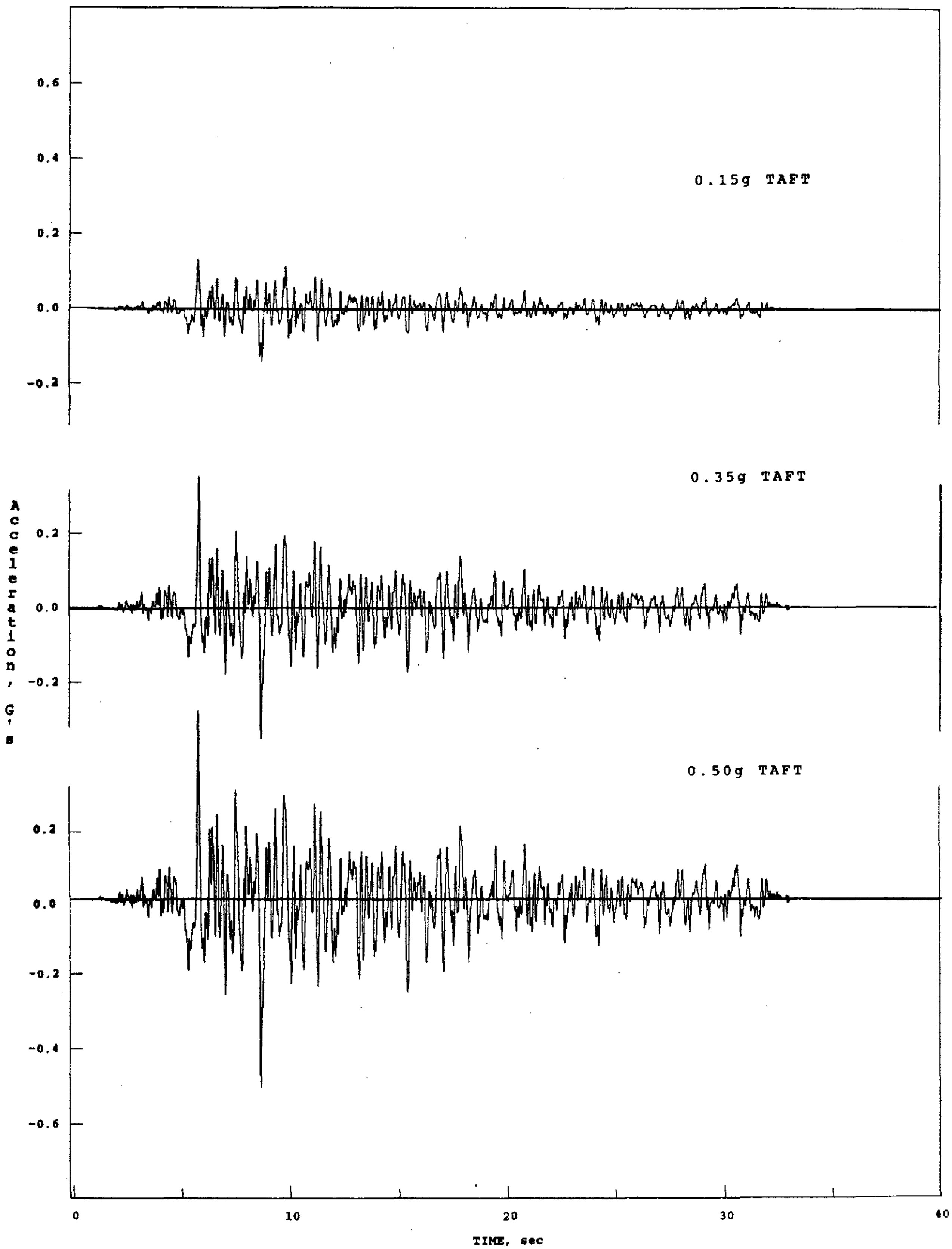


Figure 4.2 Acceleration Time History of Taft Earthquake

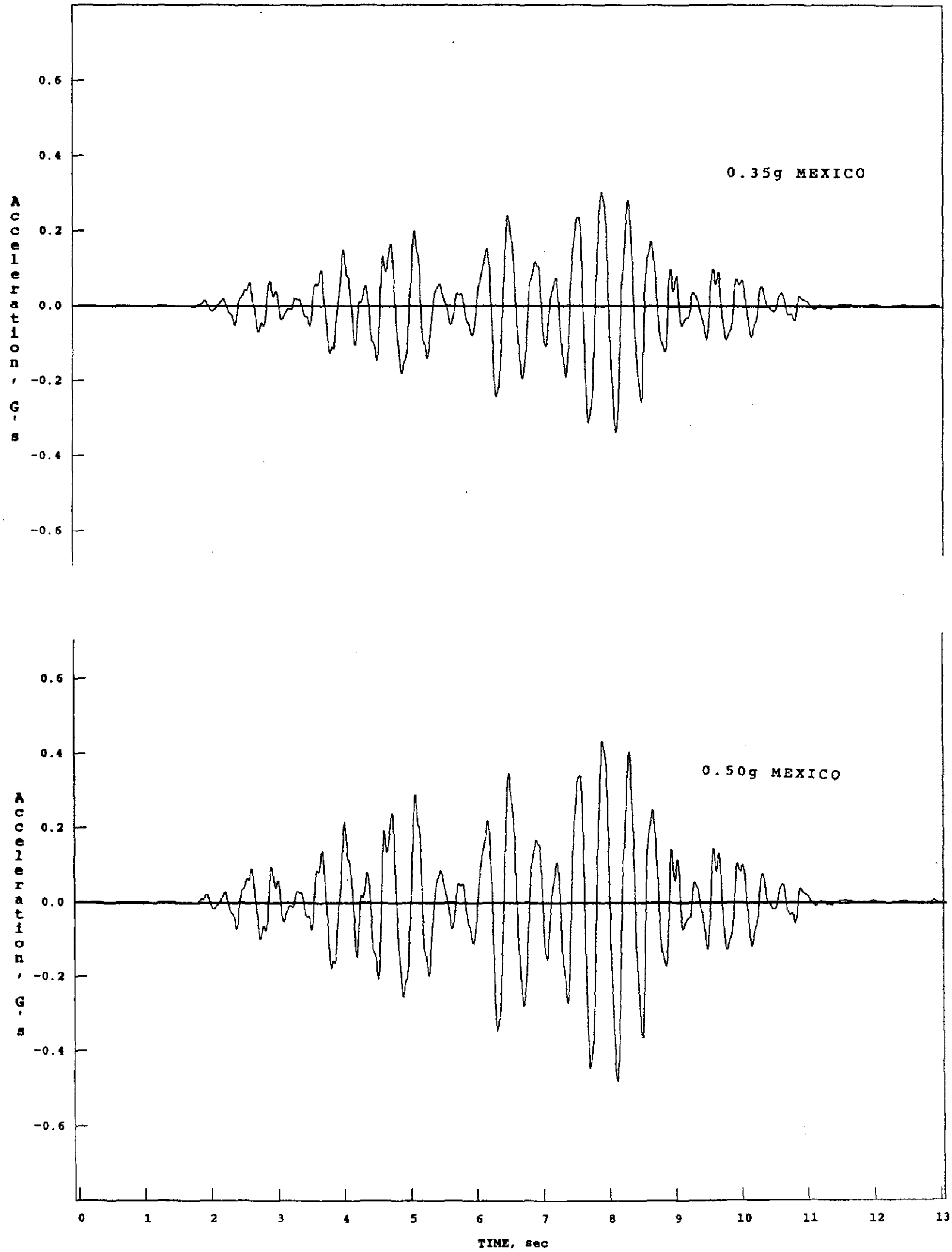


Figure 4.3 Acceleration Time History of Mexico Earthquake

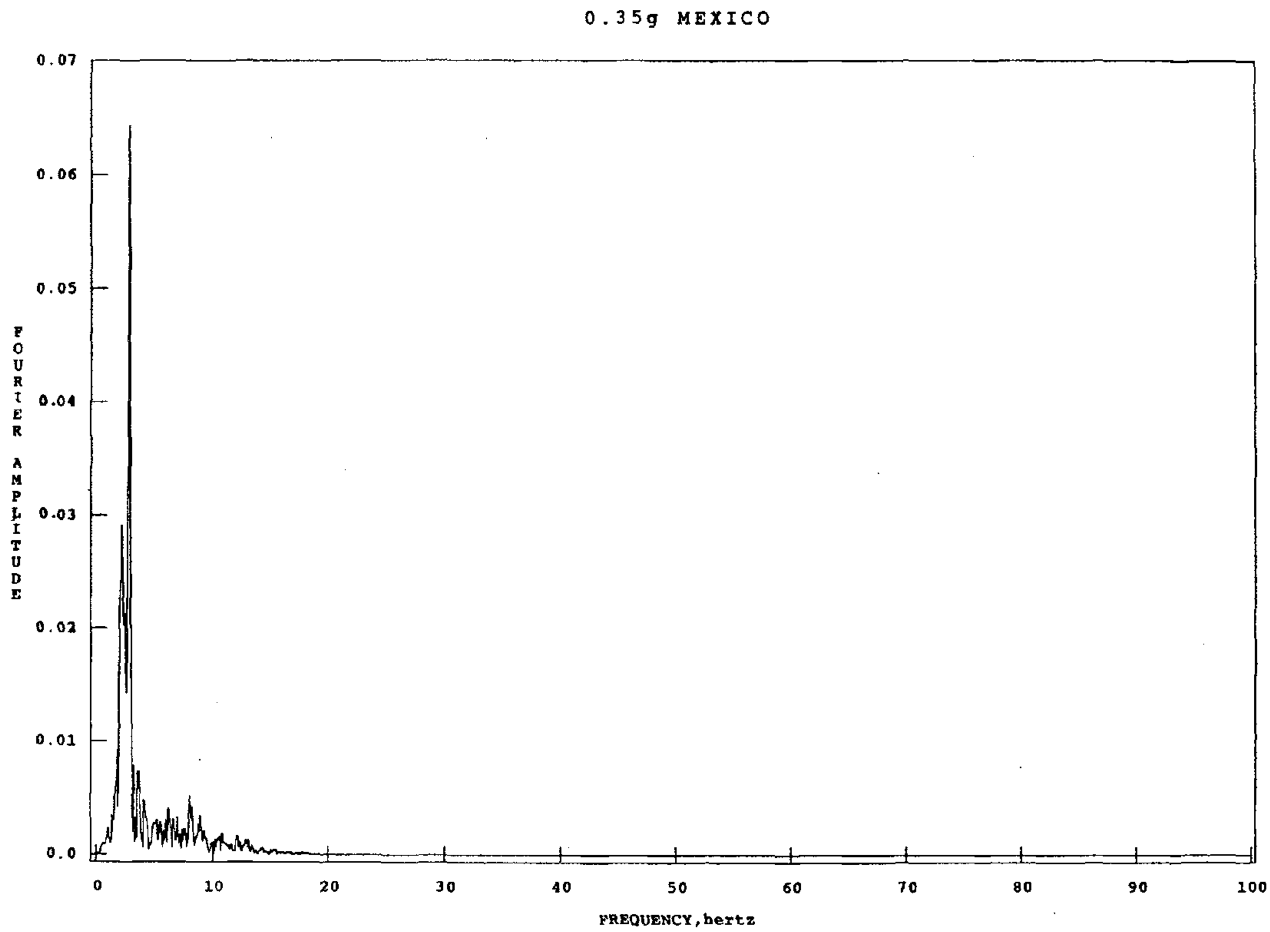


Figure 5.1 Fast Fourier Transform of a Typical Shaking Table

Ground Motion Input

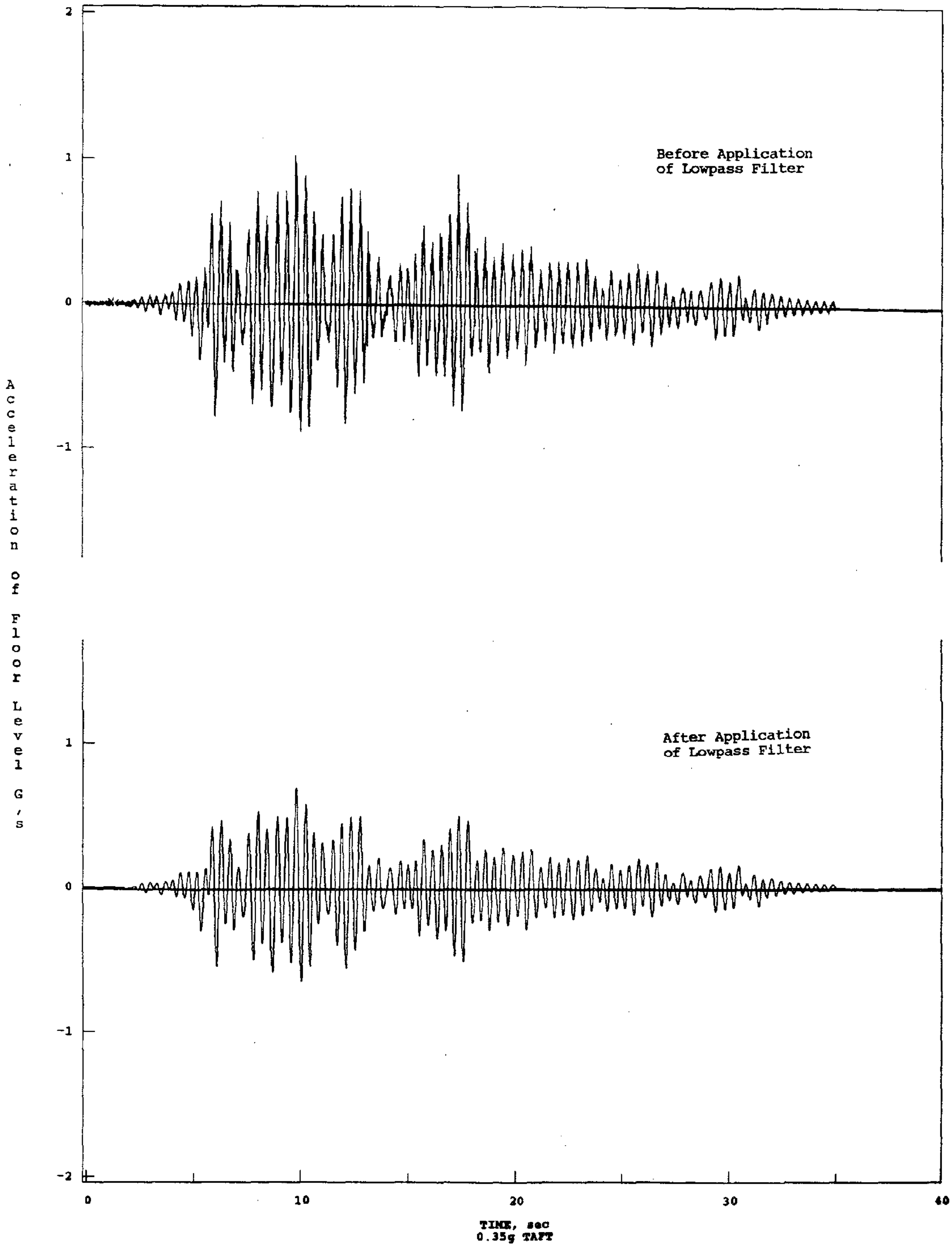


Figure 5.2 Typical Time History of Floor Acceleration Response before and after the Ormsby Lowpass Filter was applied

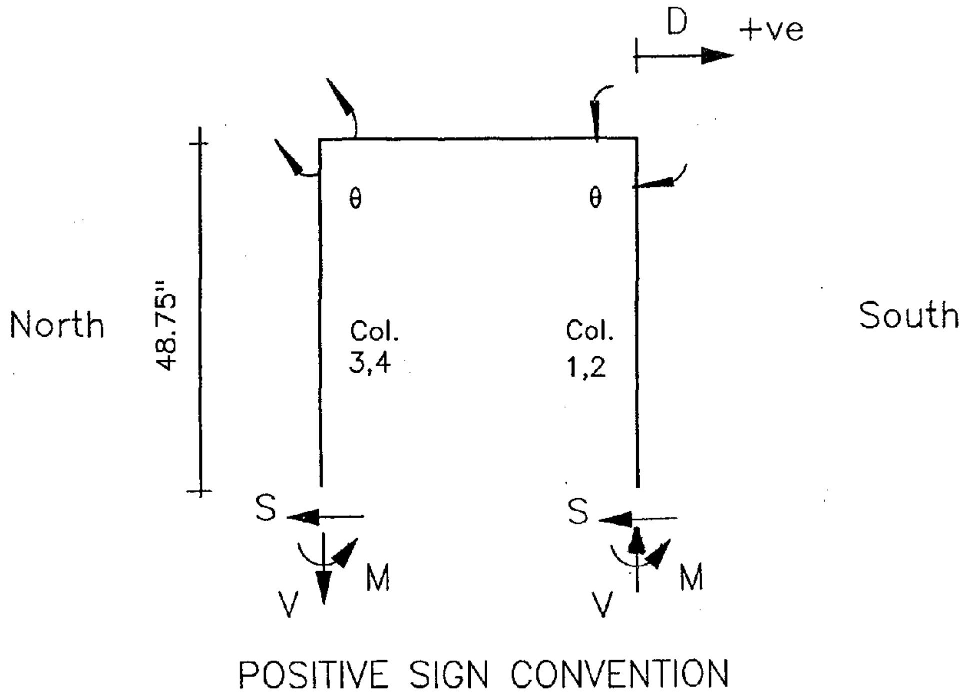


Figure 5.3 Global Sign Convention

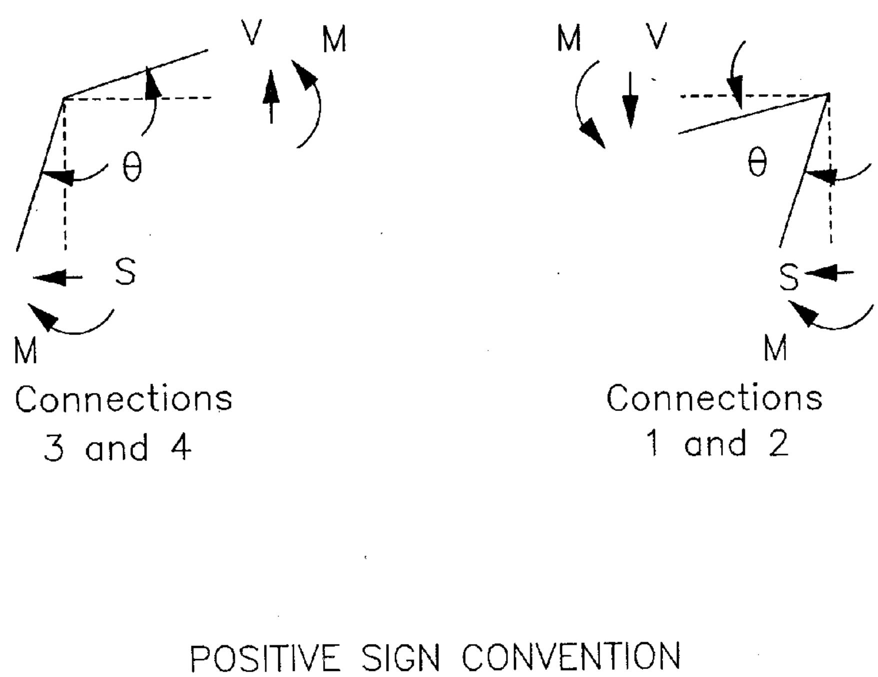


Figure 5.4 Connection Sign Convention

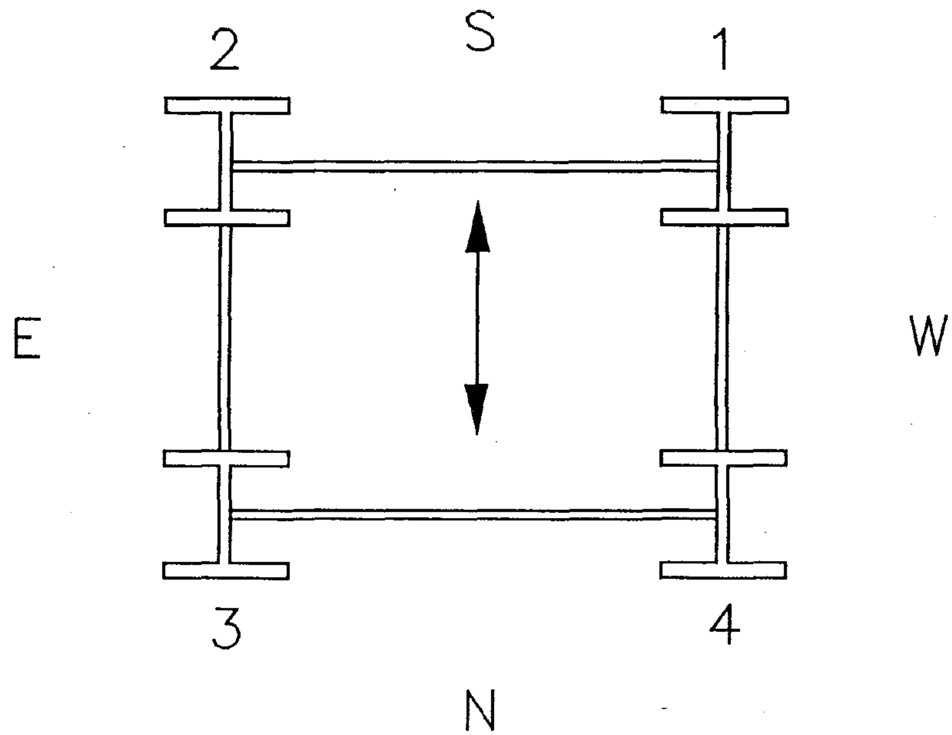


Figure 5.5 Member Designations

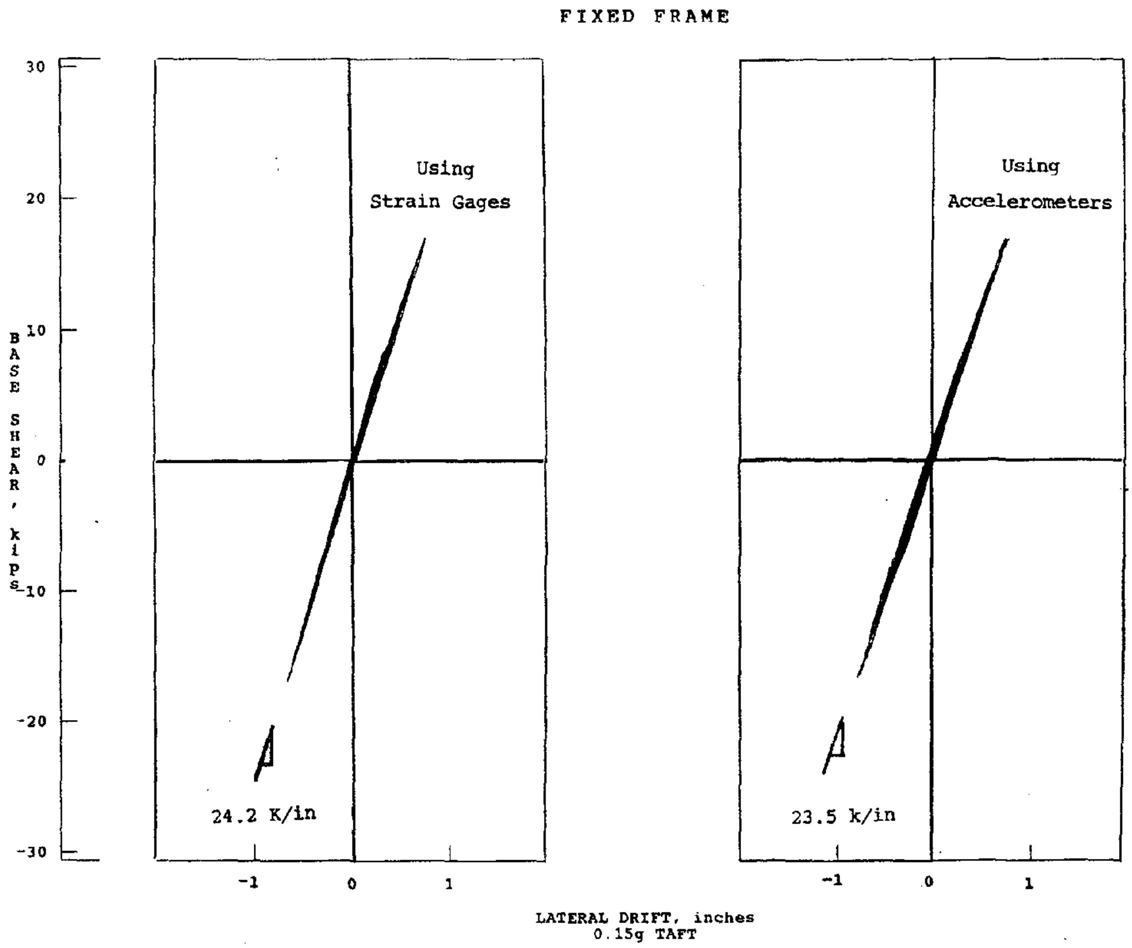


Figure 5.6(a) Comparison of Stiffness Calculated by Strain Gages and Accelerometers

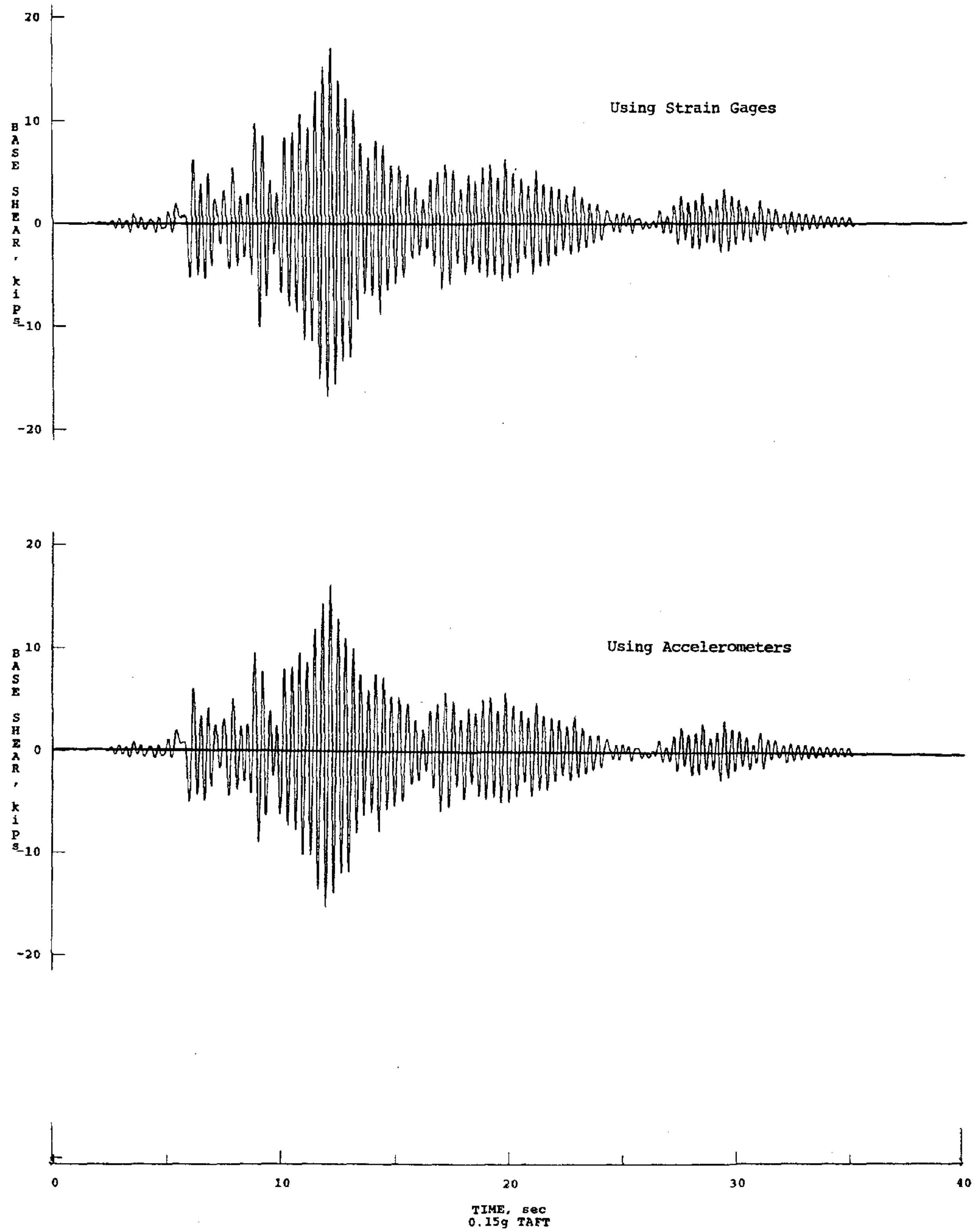


Figure 5.6(b) Time History Comparison of Base Shear Calculated

by Strain Gages and Accelerometers

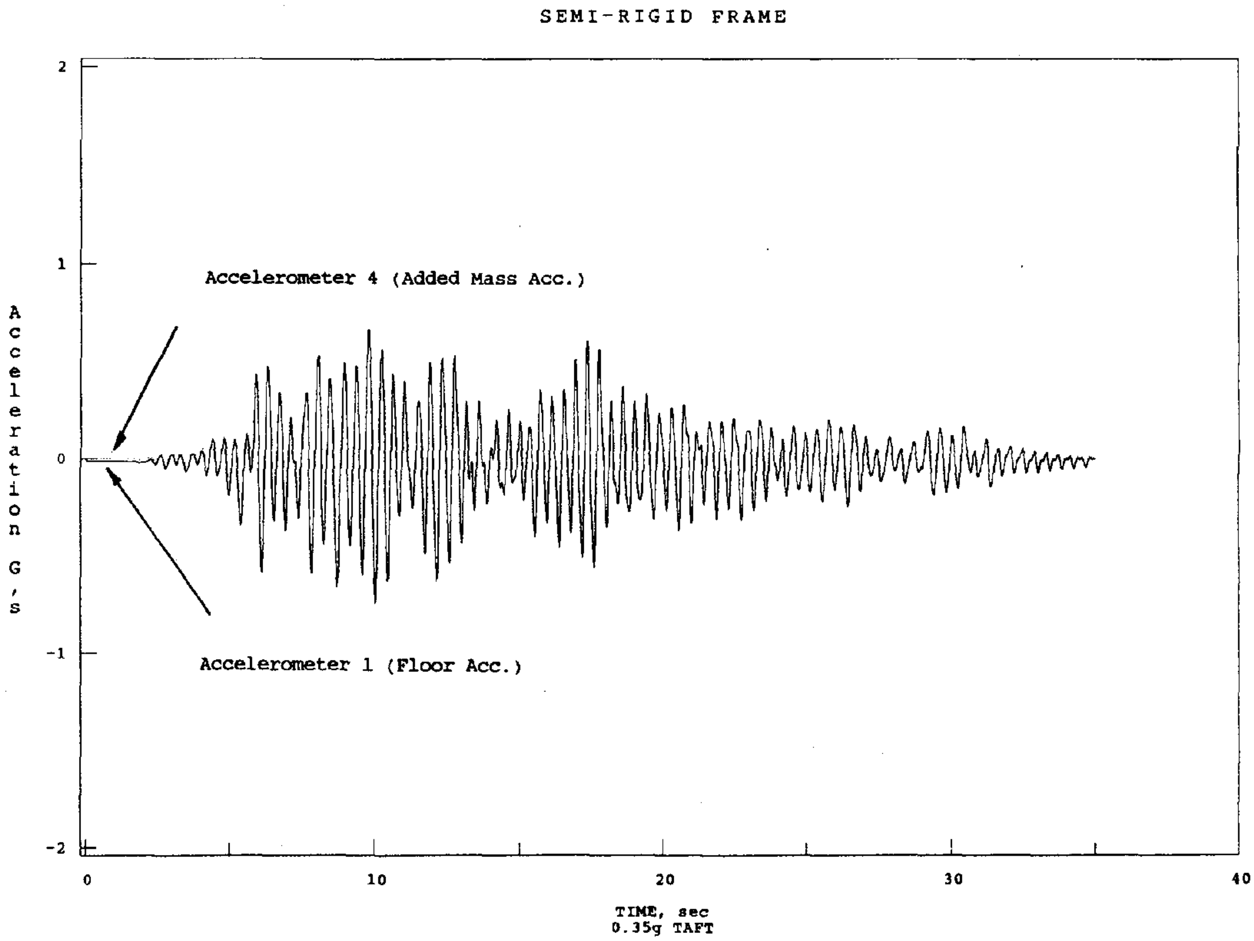


Figure 6.1 Time Histories of Floor Acceleration and

Added Mass acceleration

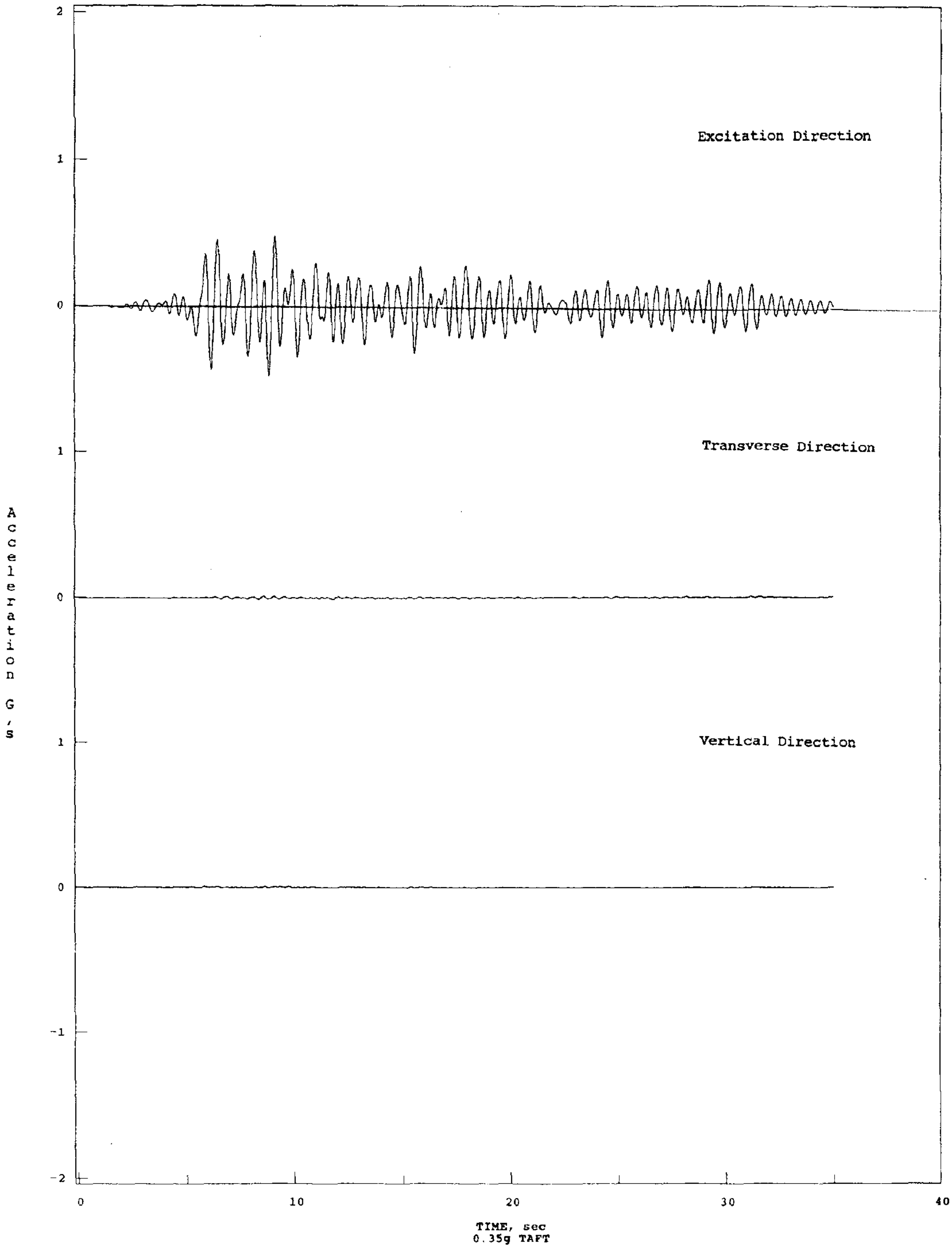


Figure 6.2 Acceleration Time History of Different Components of the Response of the Flexible Structure

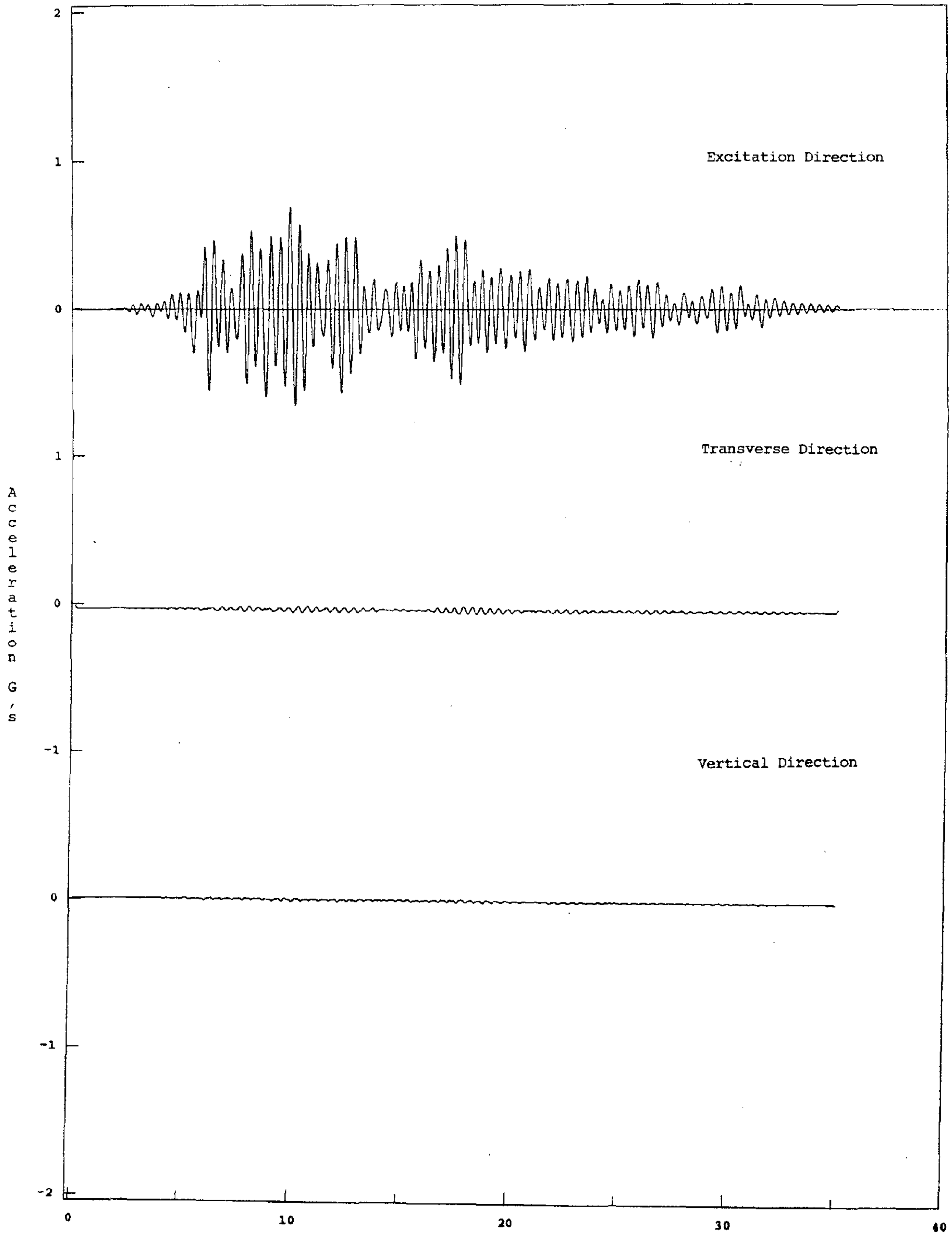


Figure 6.3 Acceleration Time History of Different Components of the Response of the Semi-Rigid Structure

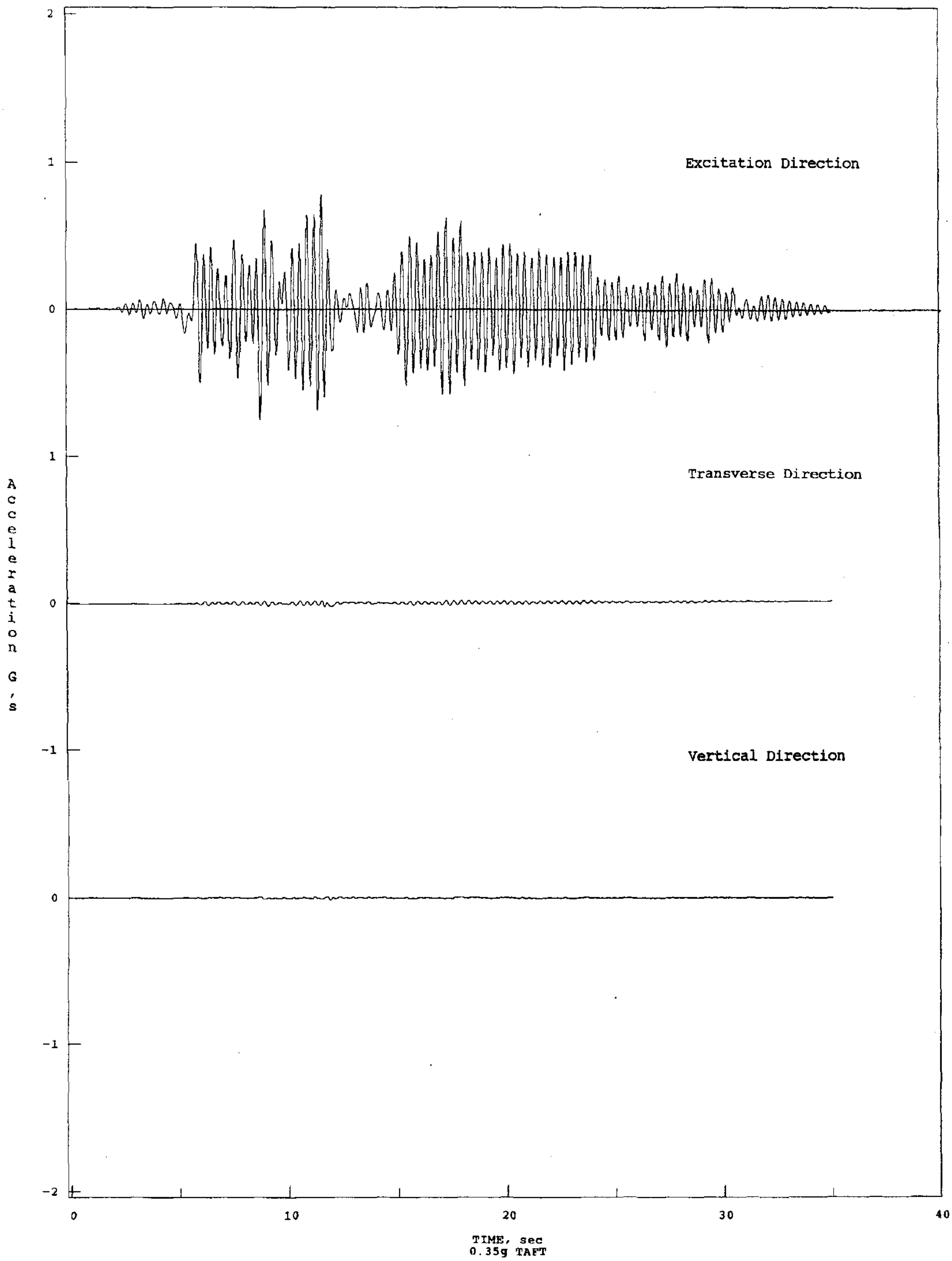


Figure 6.4 Acceleration Time History of Different Components of the Response of the Fixed Structure

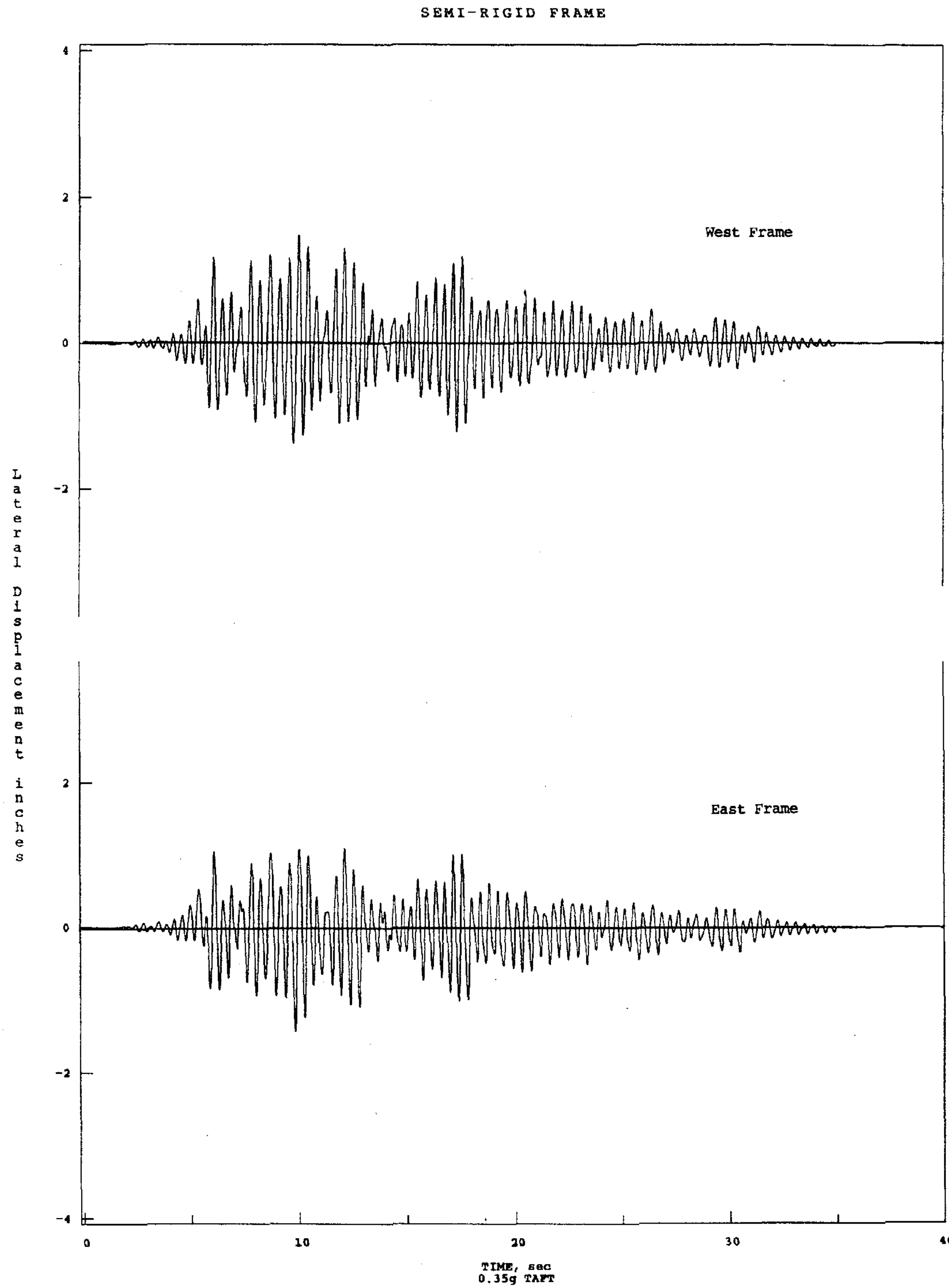


Figure 6.5 Comparison of Displacement Time History of the Two Parallel Frames

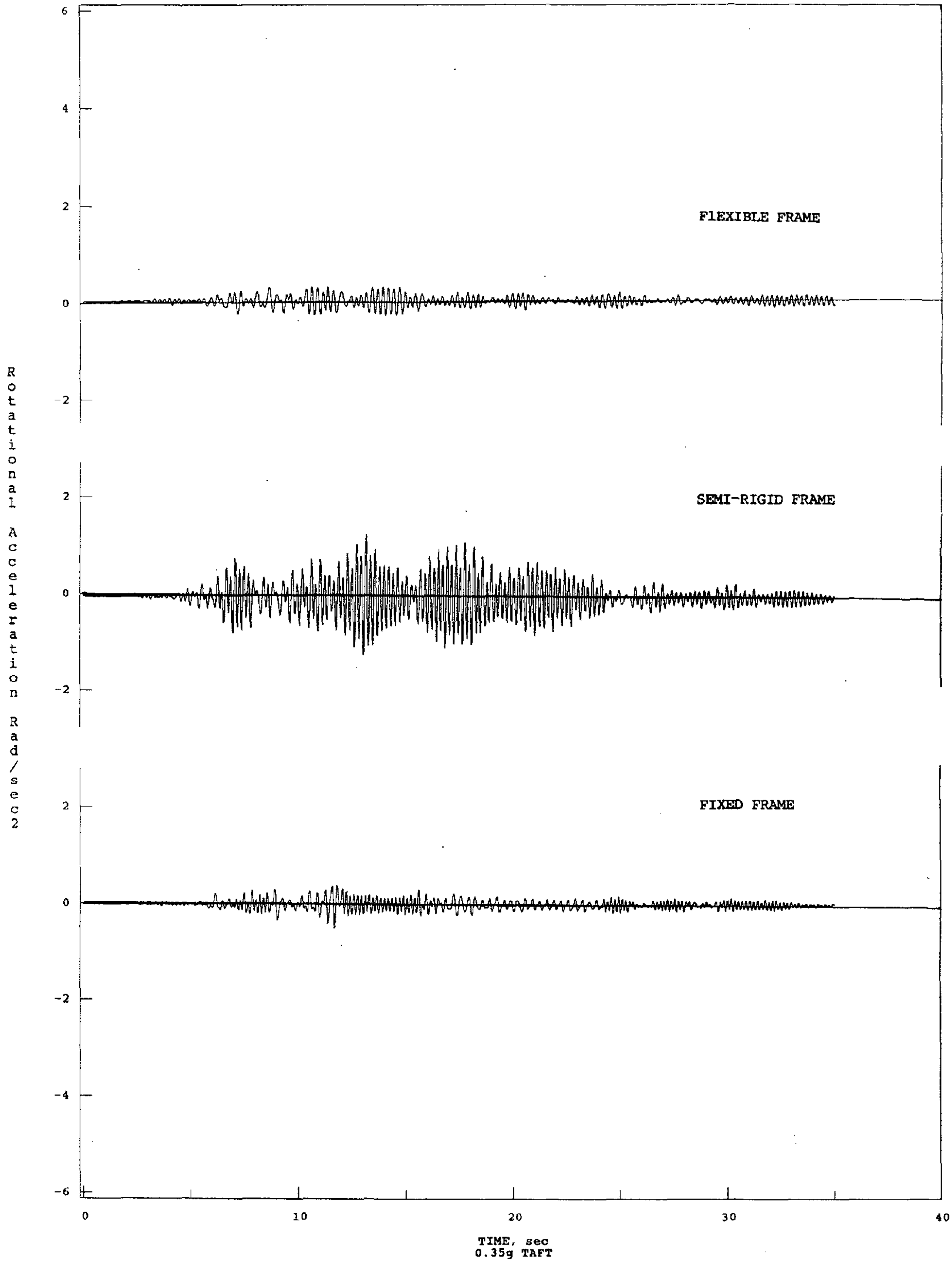


Figure 6.6 Rotational Acceleration Time History for Flexible, Semi-Rigid, and Fixed Structures

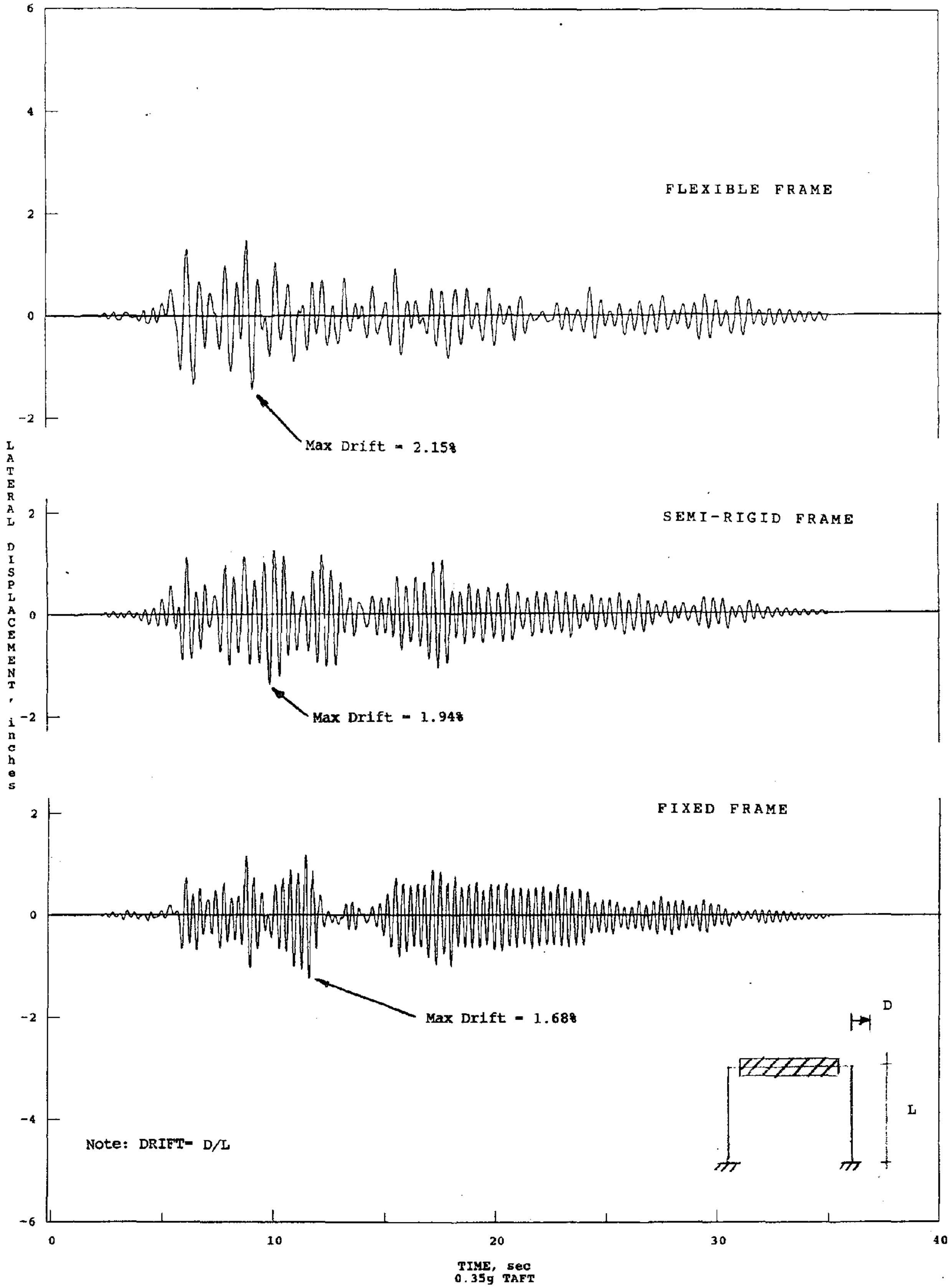


Figure 6.7 Time History of Floor Lateral Drift of Flexible, Semi-Rigid, and Fixed Structures

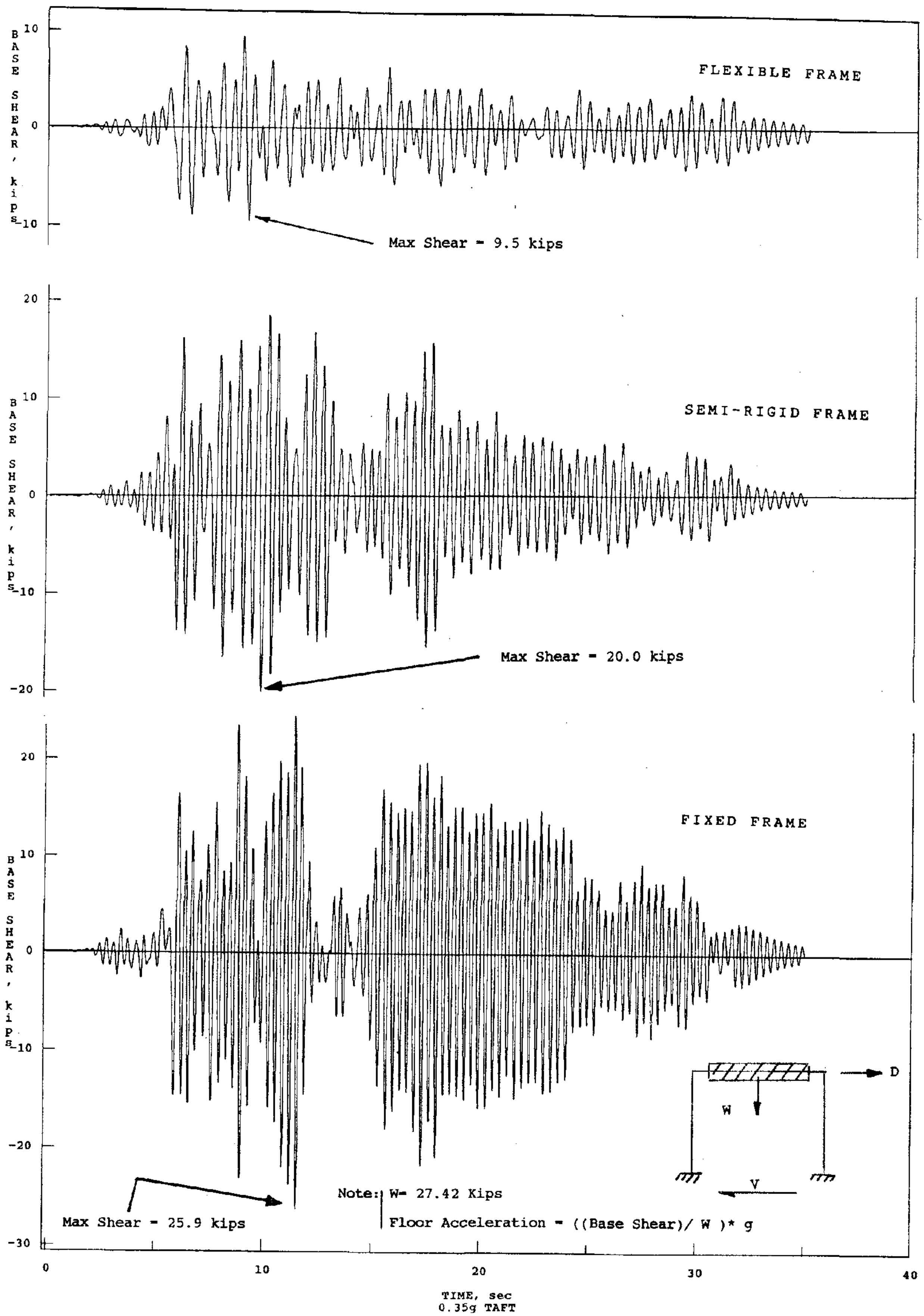


Figure 6.8 Time History of Base Shear for Flexible, Semi-Rigid, and Fixed Structures

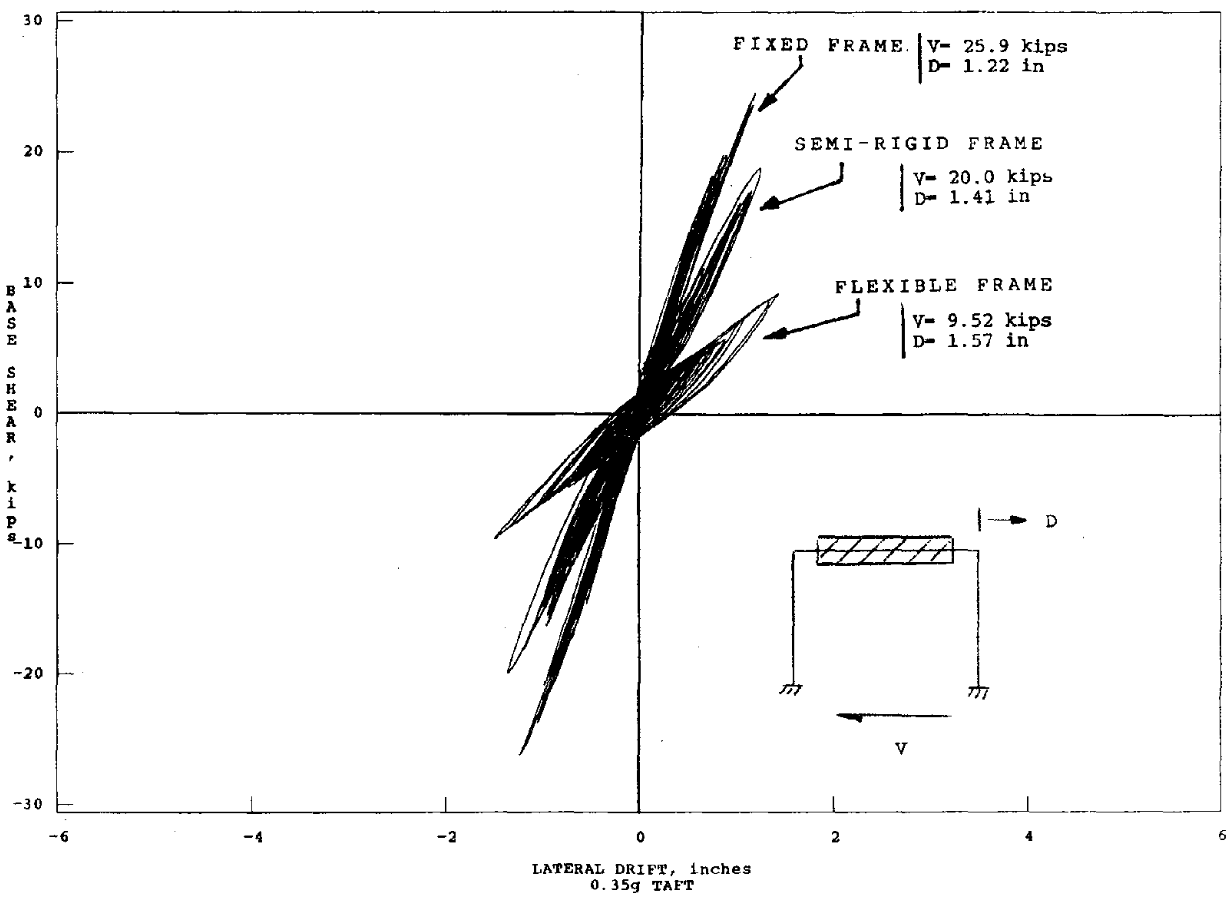


Figure 6.9 Comparison of Stiffness of Flexible, Semi-Rigid,

and Fixed Structures

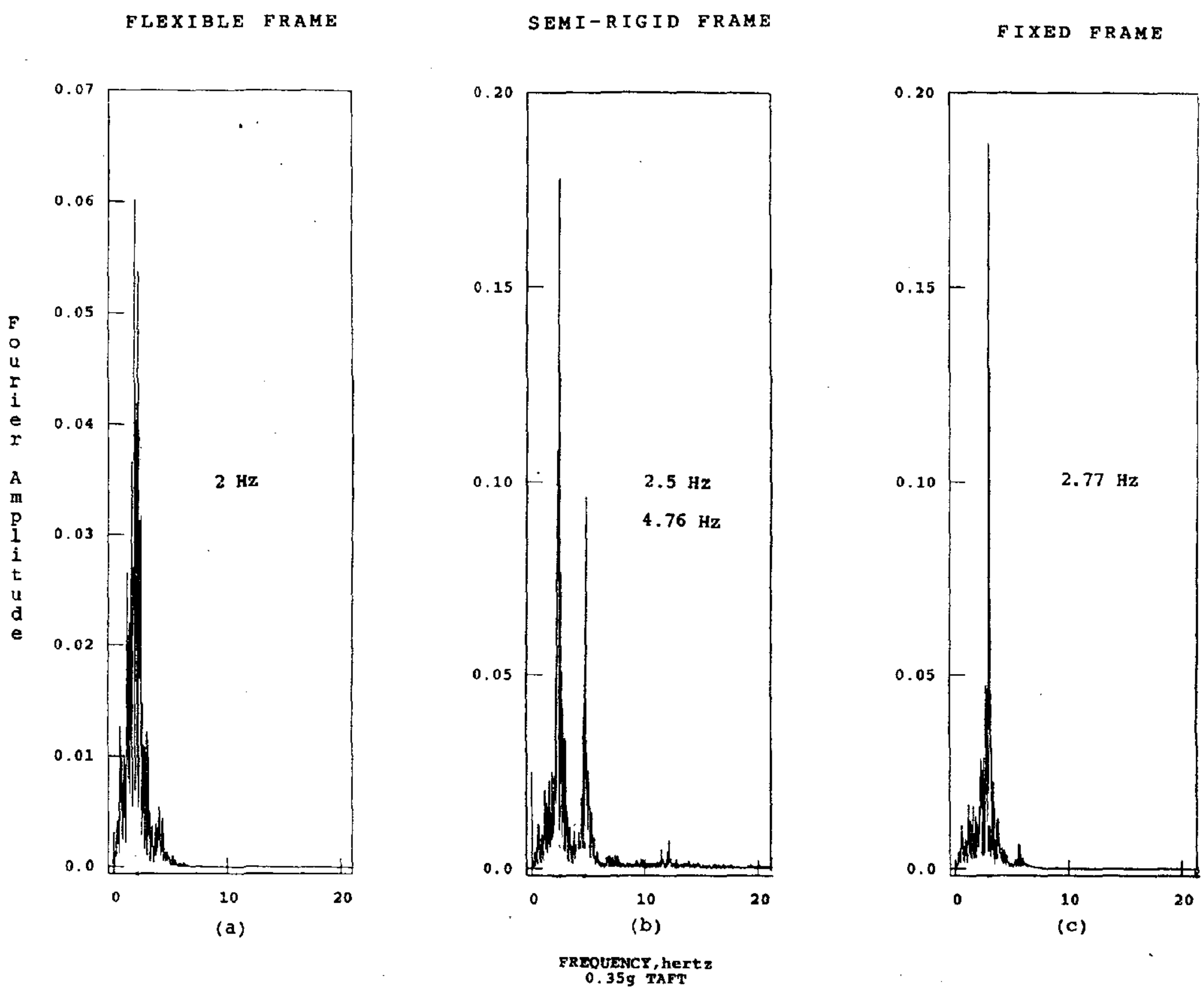


Figure 6.10 Typical Fast Fourier Transform of The Response

of Flexible, Semi-Rigid, and Fixed Structures.

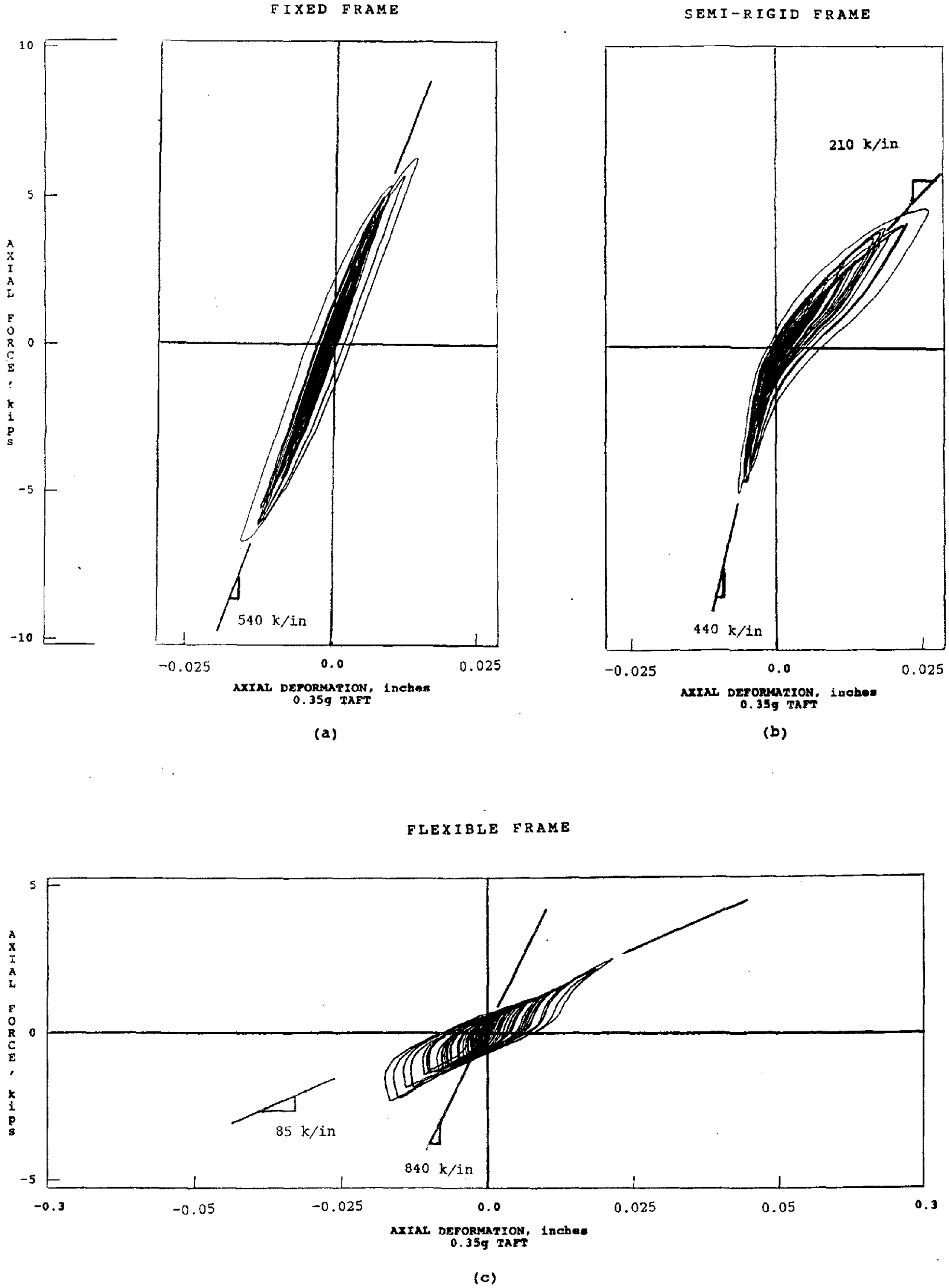


Figure 6.11 Axial Force versus Axial Deformation of Connection # 4

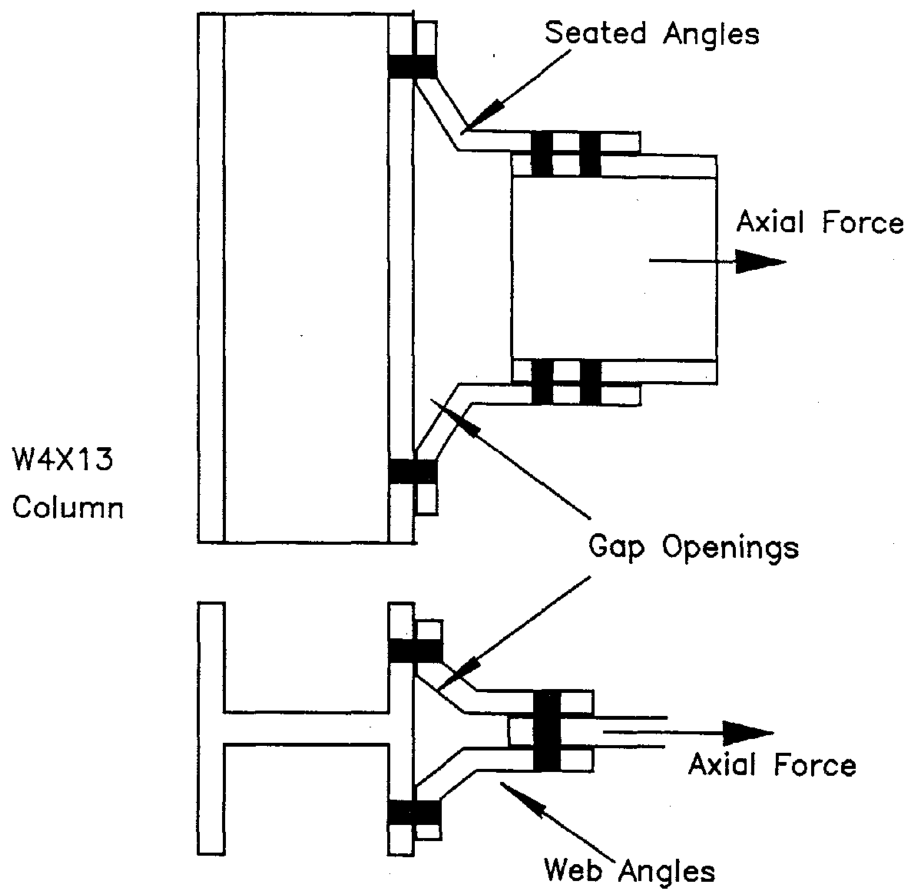


Figure 6.12 Inelastic Deformation of the Connection Angles

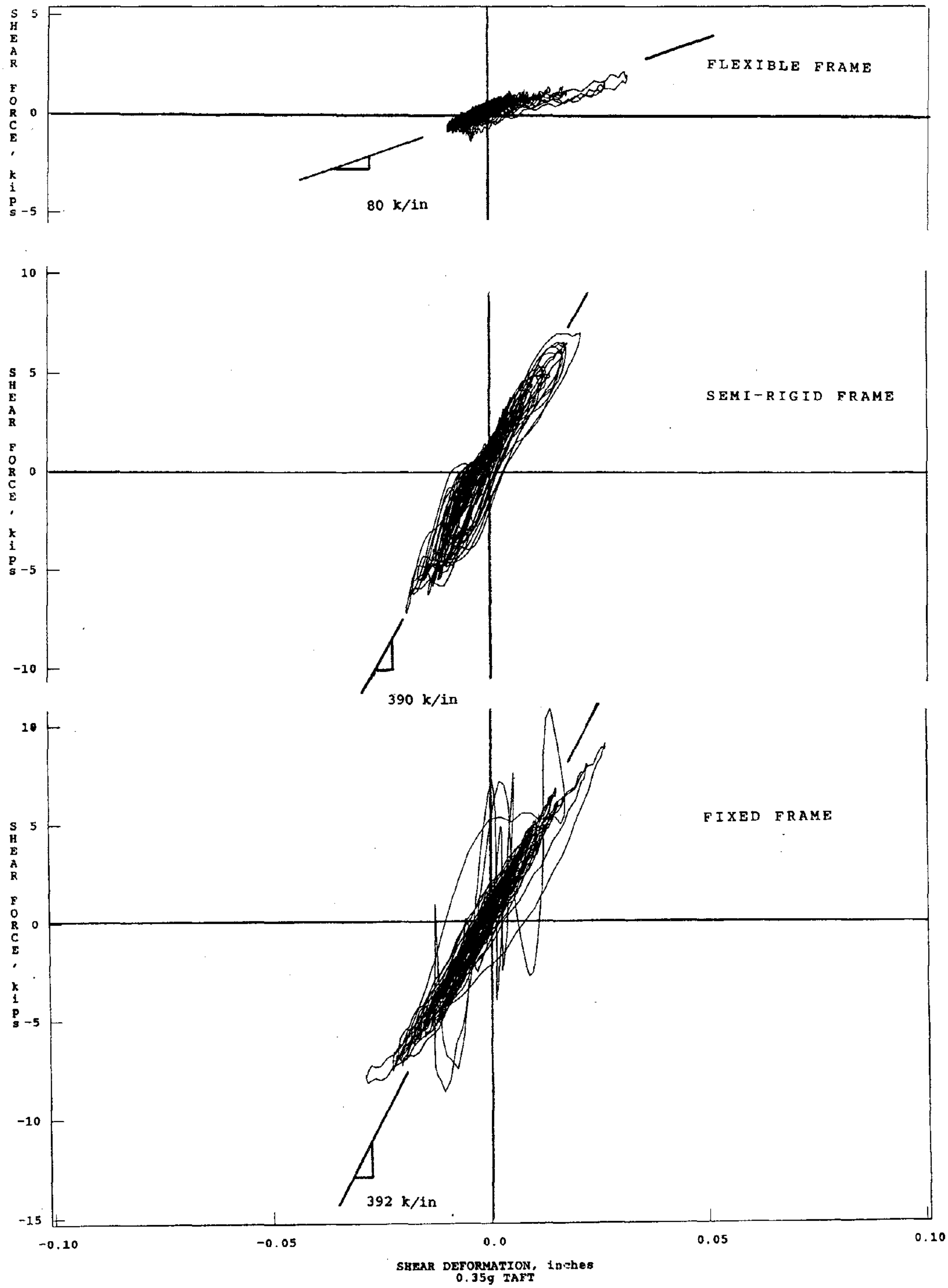


Figure 6.13 Shear Force versus Shear Deformation of Connection # 4

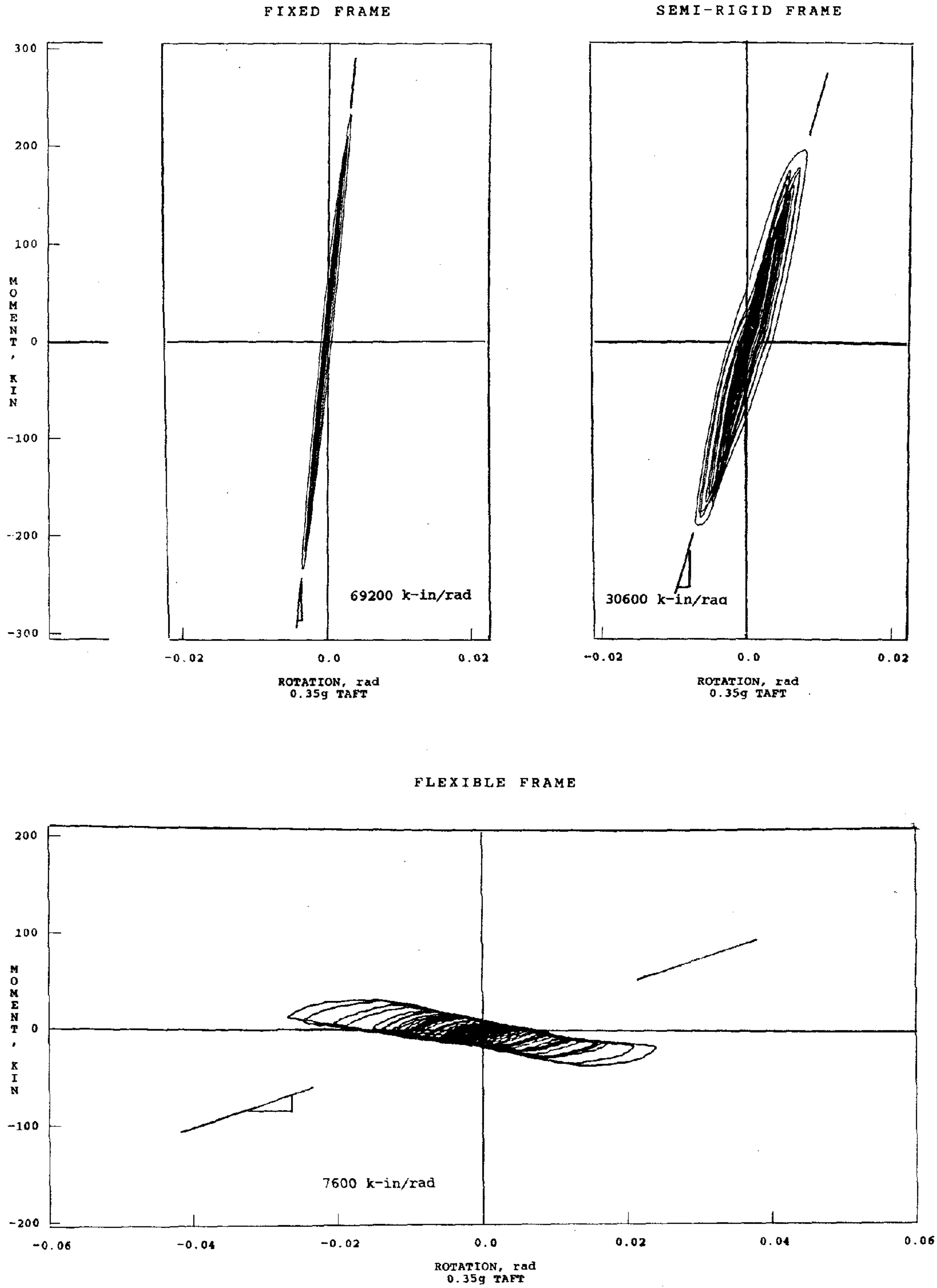
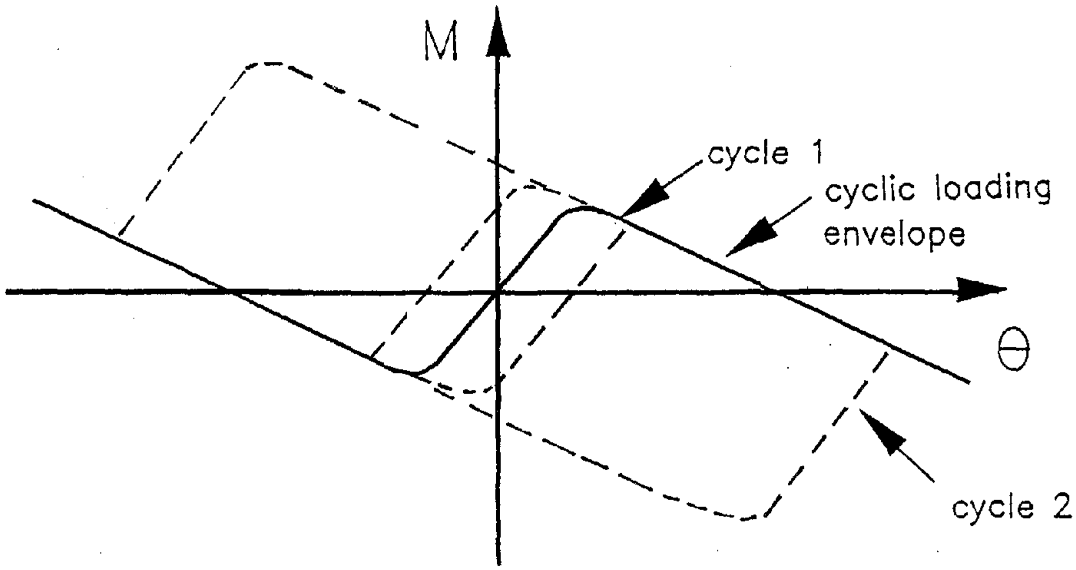


Figure 6.14 Moment versus Rotation of Connection # 4



The plastic moment decreases as the rotation increases due to loss of stiffness

Figure 6.15. Degradation of M–Θ in Flexible Connection

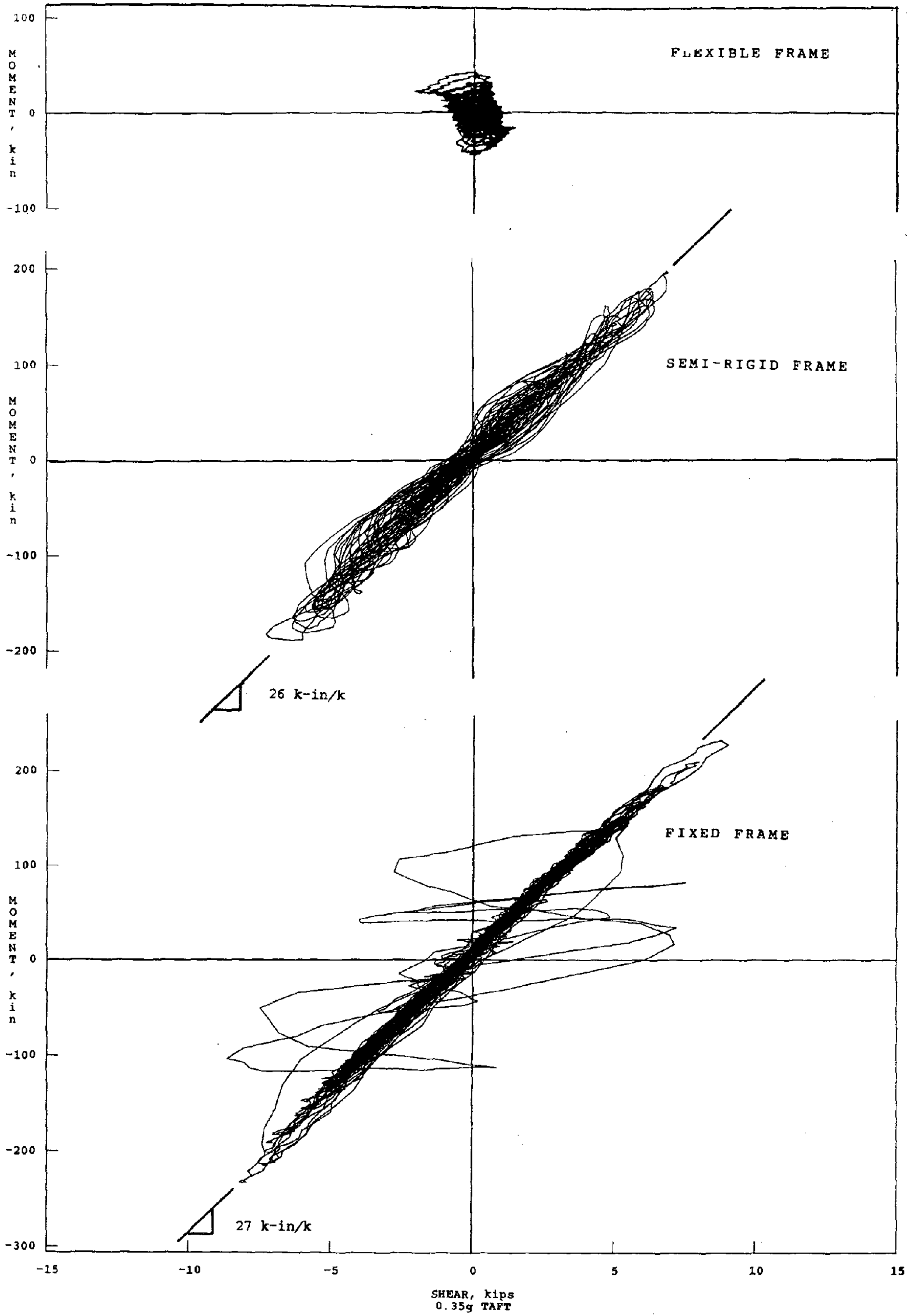


Figure 6.16 Moment versus Shear of Connection # 4

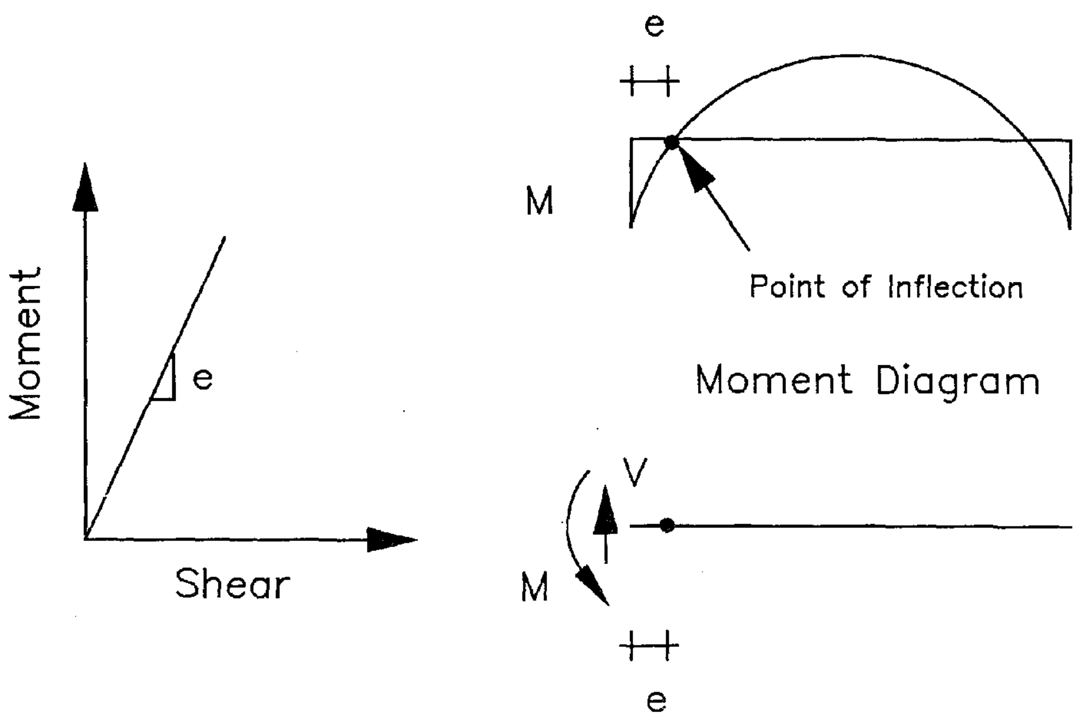


Figure 6.17 Relation Between the Slope in Moment vs Shear Plot and the Location of the point of Inflection in the Moment Diagram

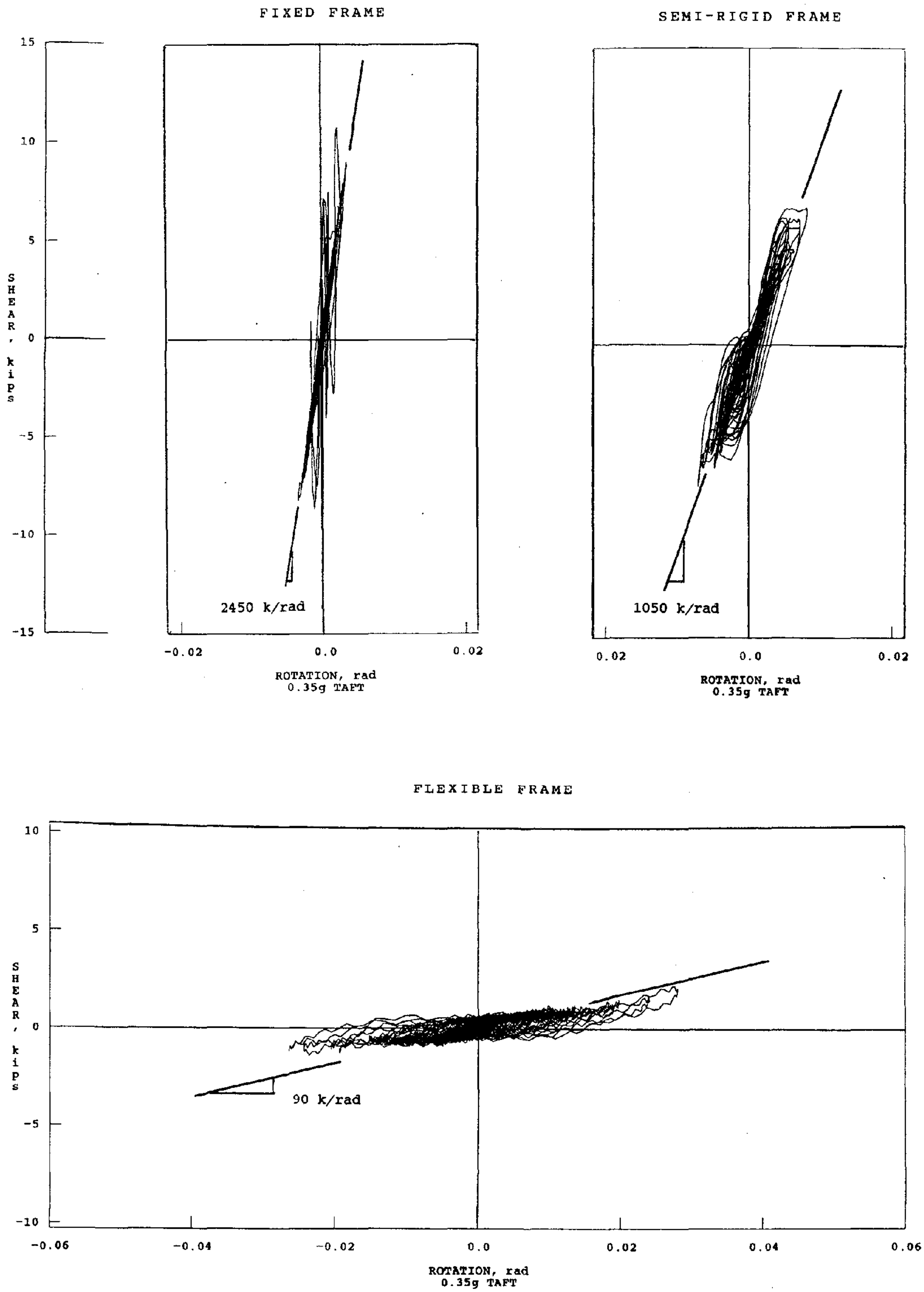


Figure 6.18 Shear Force versus Rotation of Connection # 4

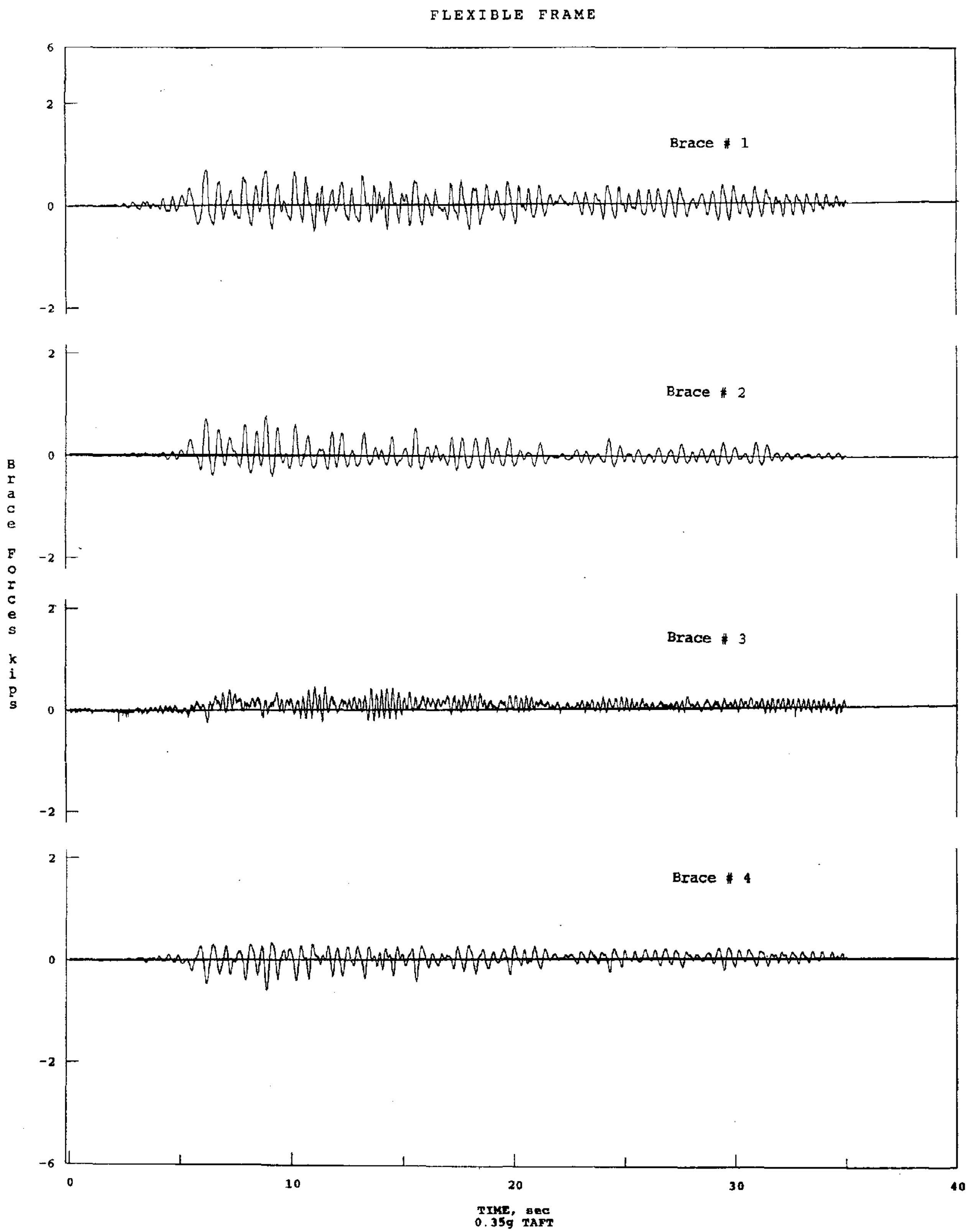


Figure 6.19 Time History of the Axial Force in the Diagonal Braces of the Flexible Structure

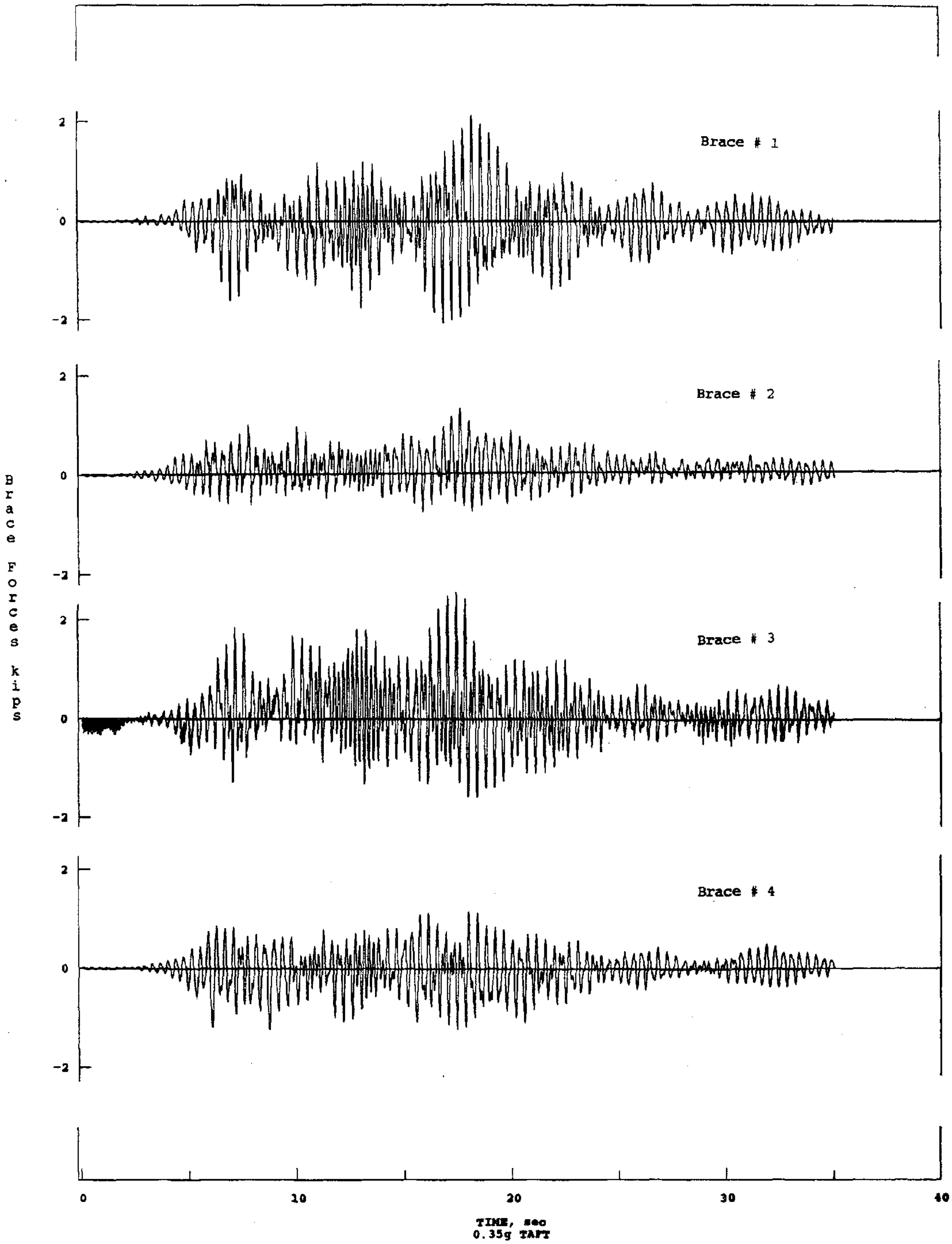


Figure 6.20 Time History of the Axial Force in the Diagonal Braces of the Semi-Rigid Structure

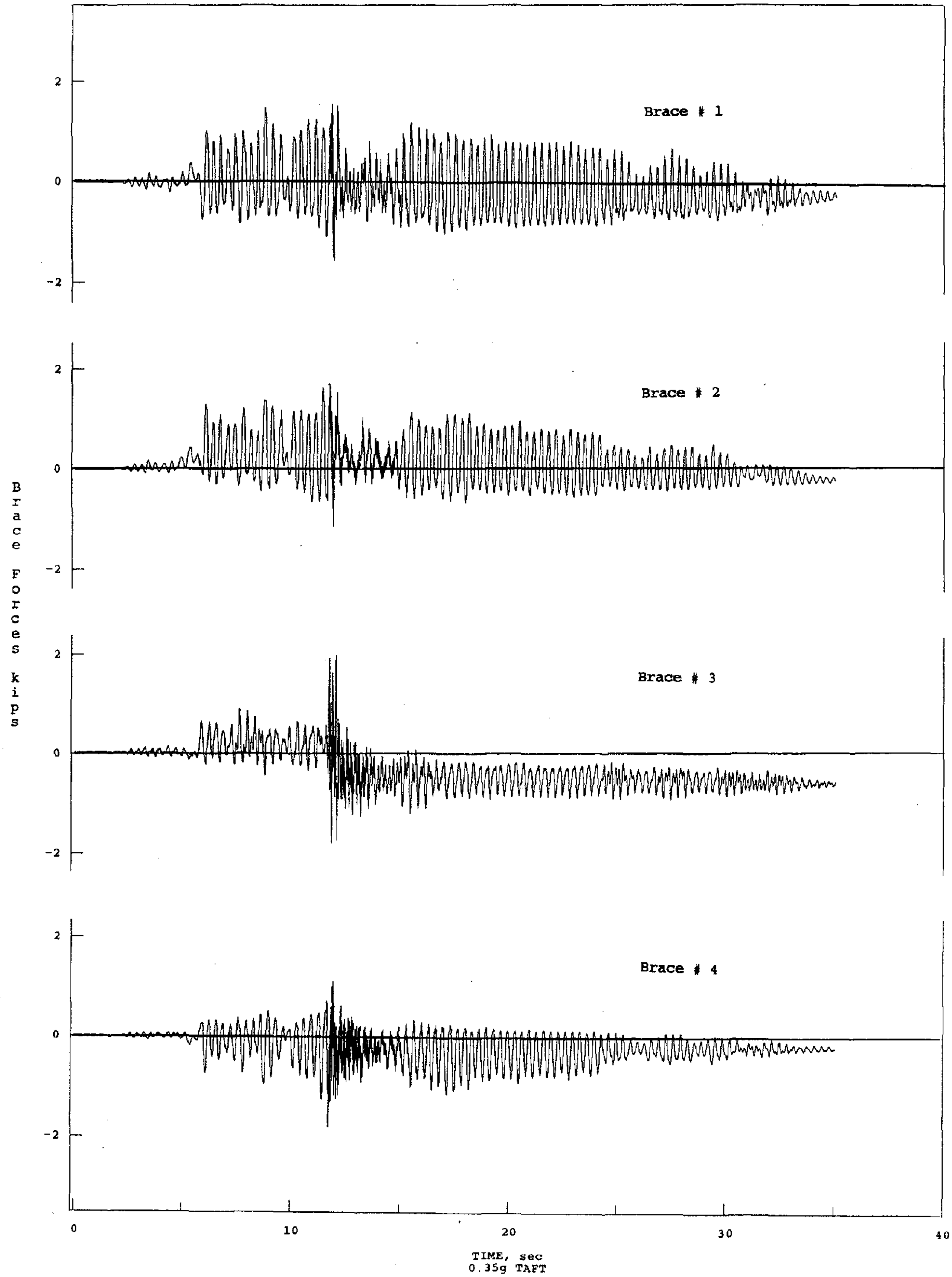


Figure 6.21 Time History of the Axial Force in the Diagonal Braces of the Fixed Structure

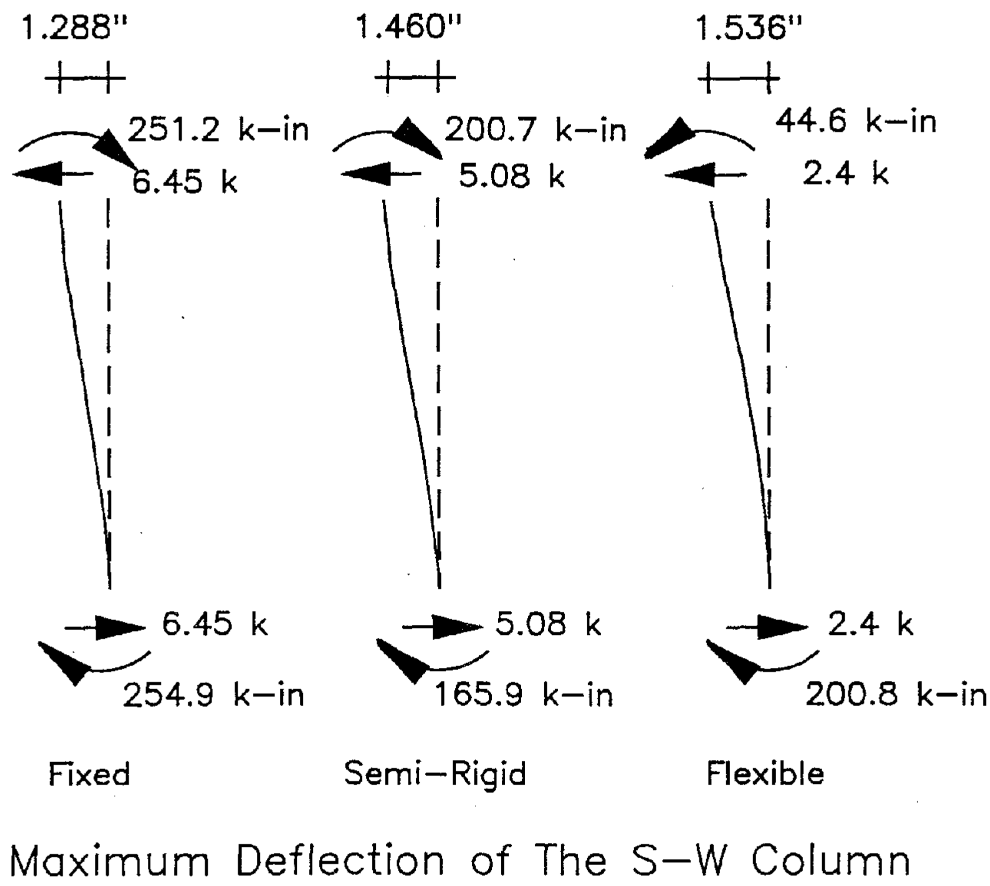


Figure 6.22 Maximum Deflection of S-W Column For the Three Structures Under Dynamic Loading

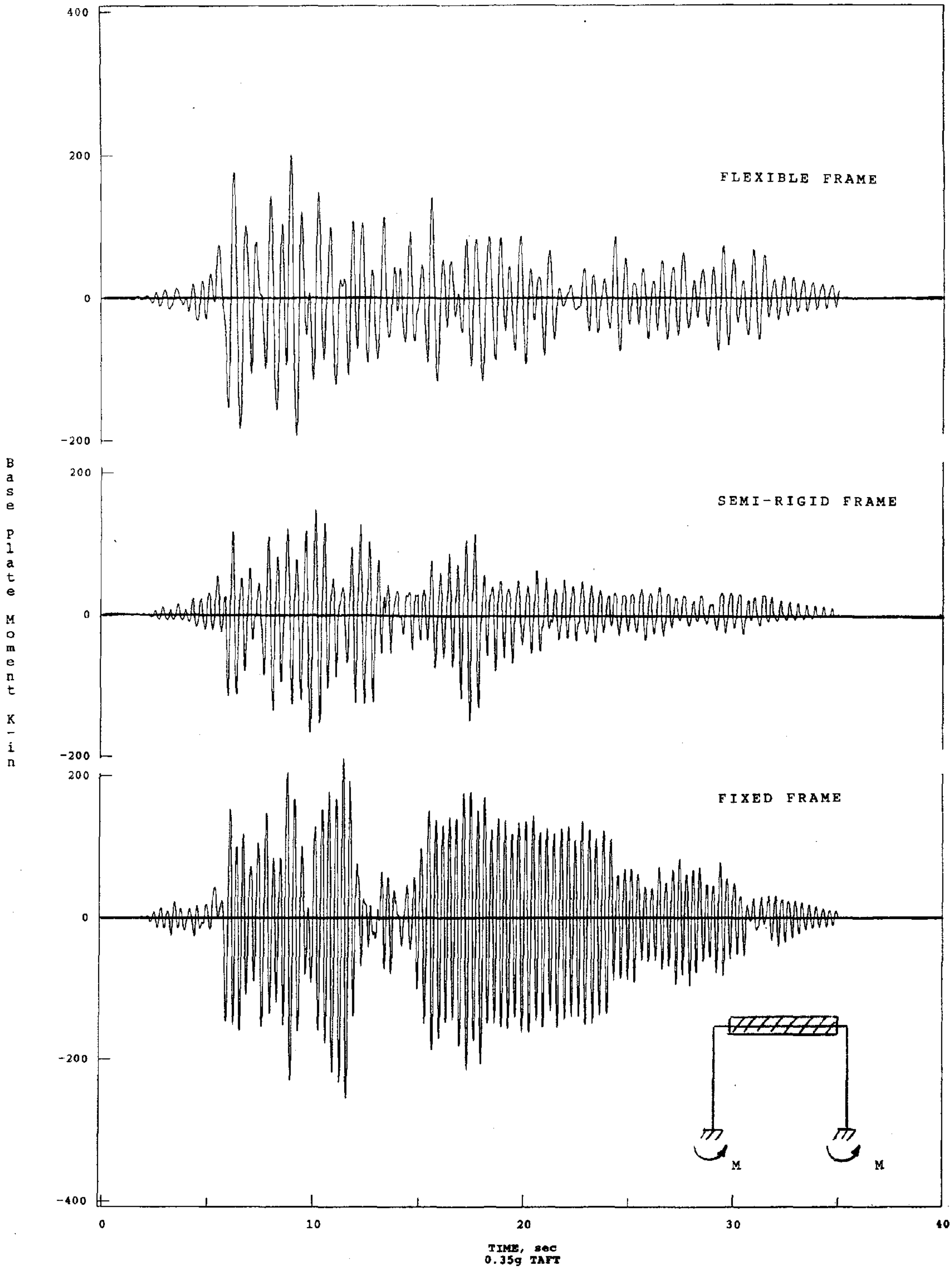


Figure 6.23 Time History of the Moment in Base Plate # 4

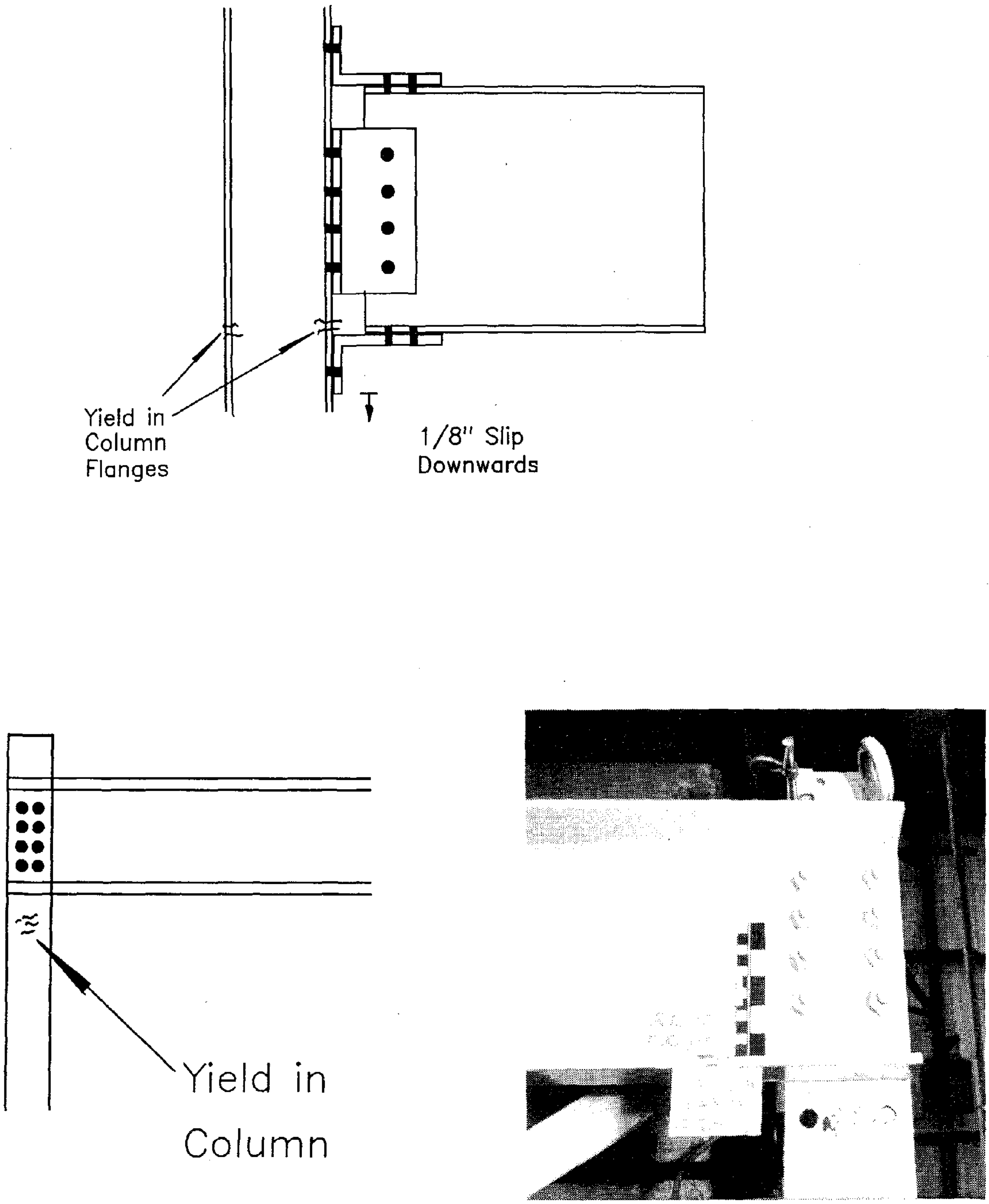


Figure 6.24 Yielding in the Columns of Semi-Rigid Structure

After 0.35g El-Centro

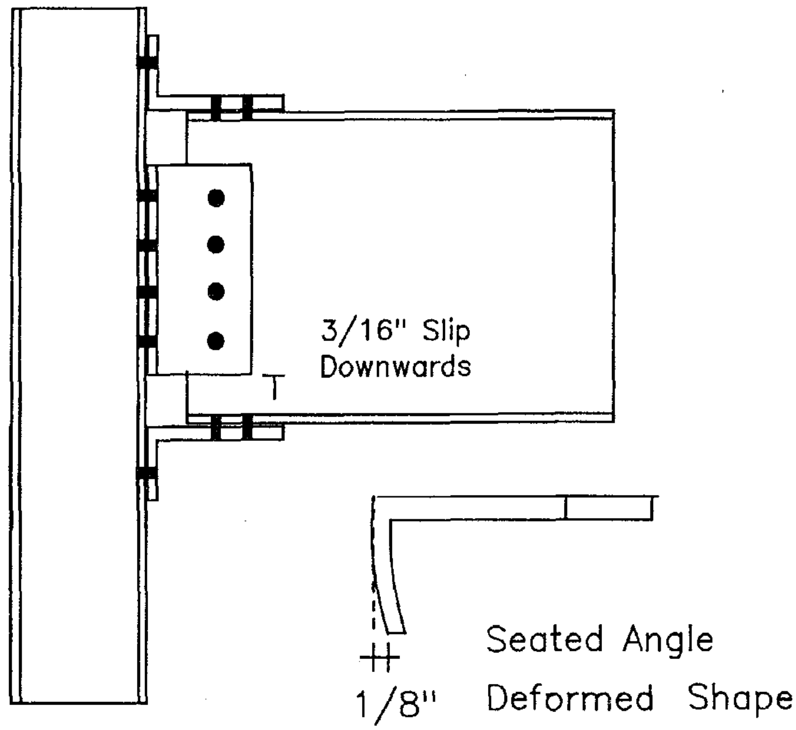
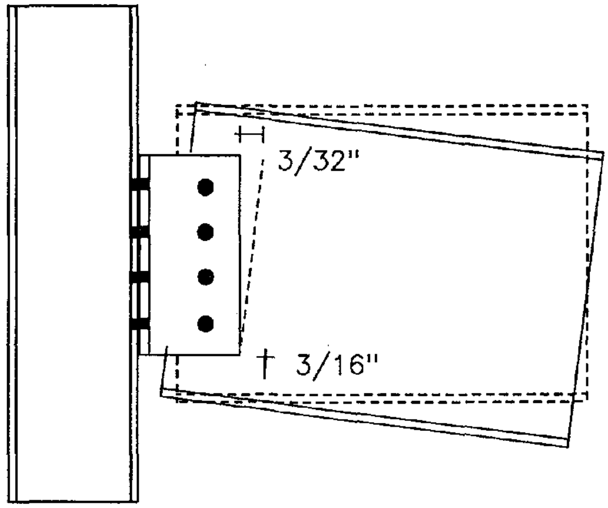


Figure 6.25 Plastic Deformation in Semi-Rigid

Connections after 0.5g Taft



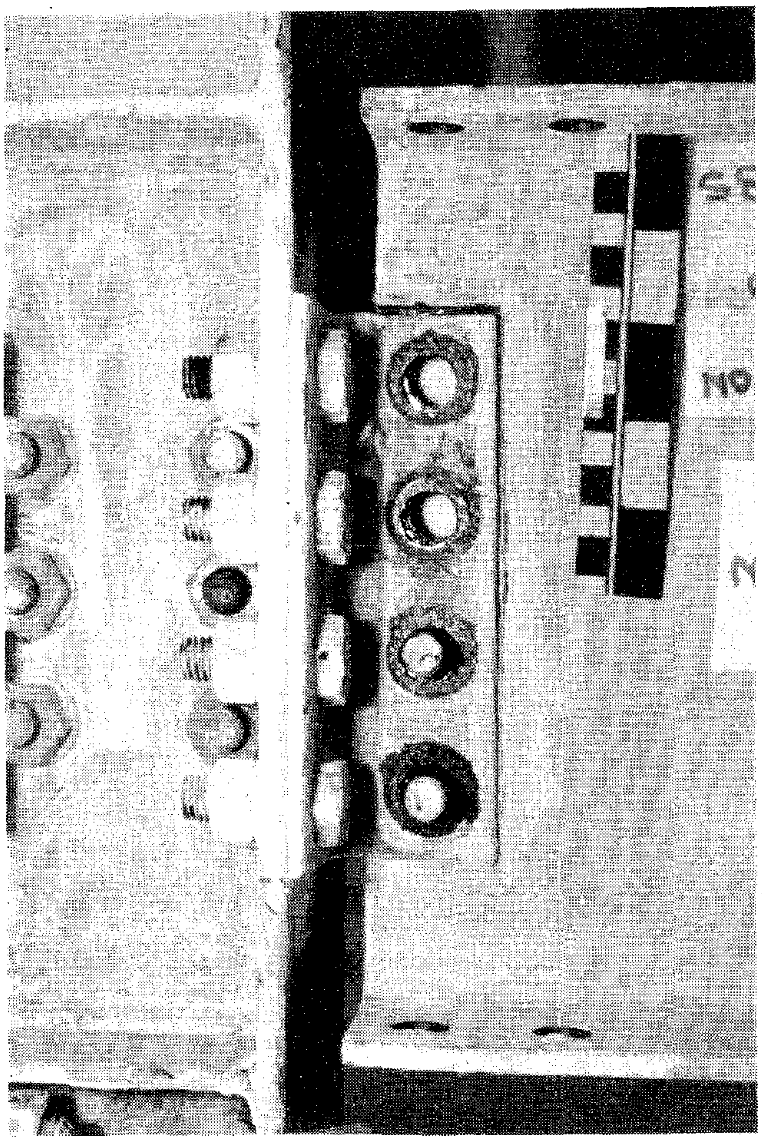


Figure 6.26 Slippage in Flexible Connection

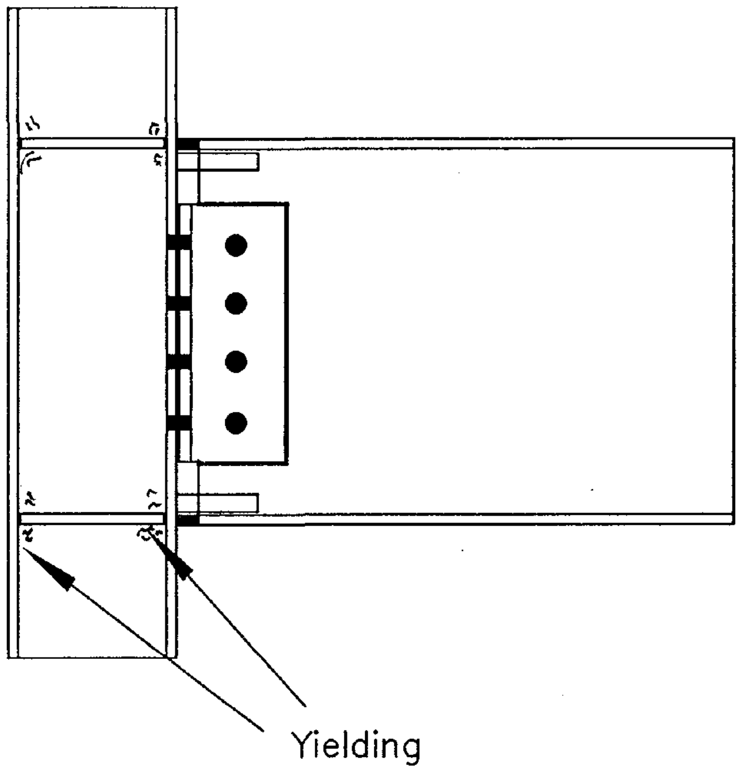


Figure 6.27 Yielding in the N-W Column of Fixed Structure after

0.35g Taft

APPENDIX A

As mentioned earlier, only 8 plots were developed for the 24 selected shaking table tests. The plots are presented as follows:

I. FLEXIBLE FRAME

FREE VIBRATION

EL CENTRO E.Q.

- 0.15g intensity
- 0.25g intensity
- o.35g intensity

TAFT E.Q.

- 0.15g intensity
- 0.35g intensity
- 0.50g intensity

MEXICO E.Q.

- 0.35g intensity
- 0.50g intensity

II. SEMI-RIGID FRAME

FREE VIBRATION

EL CENTRO E.Q.

- 0.15g intensity
- 0.25g intensity
- o.35g intensity

TAFT E.Q.

- 0.15g intensity
- 0.35g intensity
- 0.50g intensity

MEXICO E.Q.

- 0.35g intensity
- 0.50g intensity

III. FIXED FRAME

FREE VIBRATION

EL CENTRO E.Q.

0.15g intensity

0.25g intensity

o.35g intensity

TAFT E.Q.

0.15g intensity

0.35g intensity

NOT AVAILABLE

MEXICO E.Q.

NOT AVAILABLE

NOT AVAILABLE

For each of the test runs the the following plots are presented:

(1) Connection 4: Moment versus Rotation

(2) Connection 4: Shear versus Rotation

(3) Connection 4: Moment versus Shear

(4) Time history of the moment of base plate 4

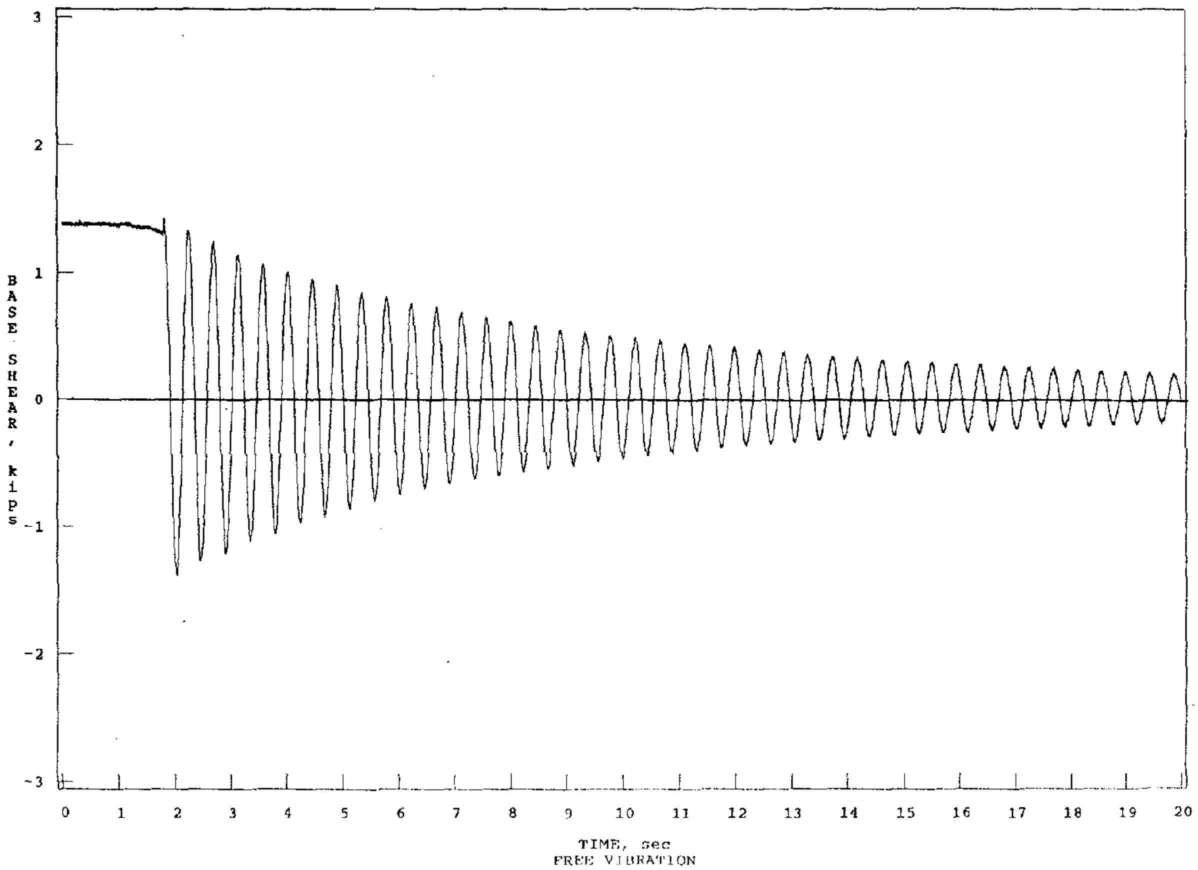
(5) Time history of the base shear

(6) Time history of the lateral drift

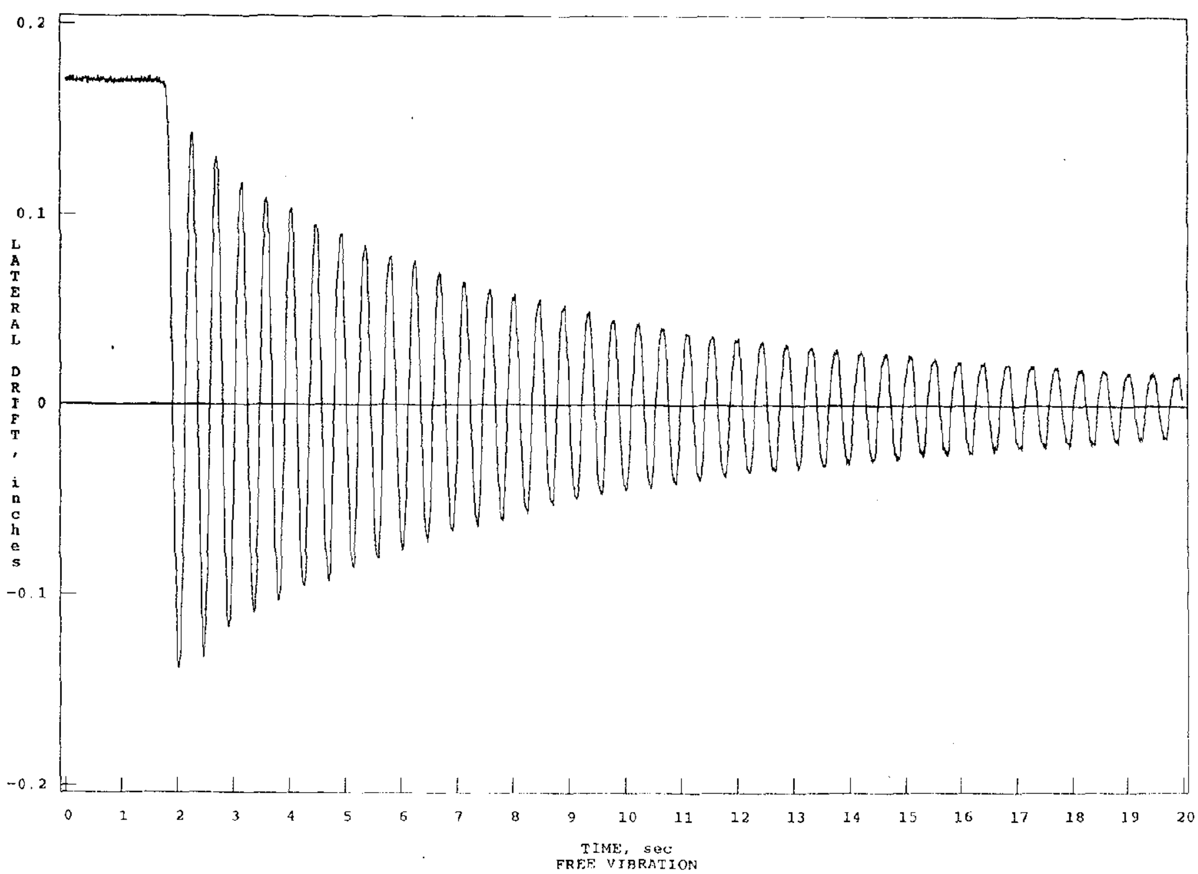
(7) Plot of stiffness

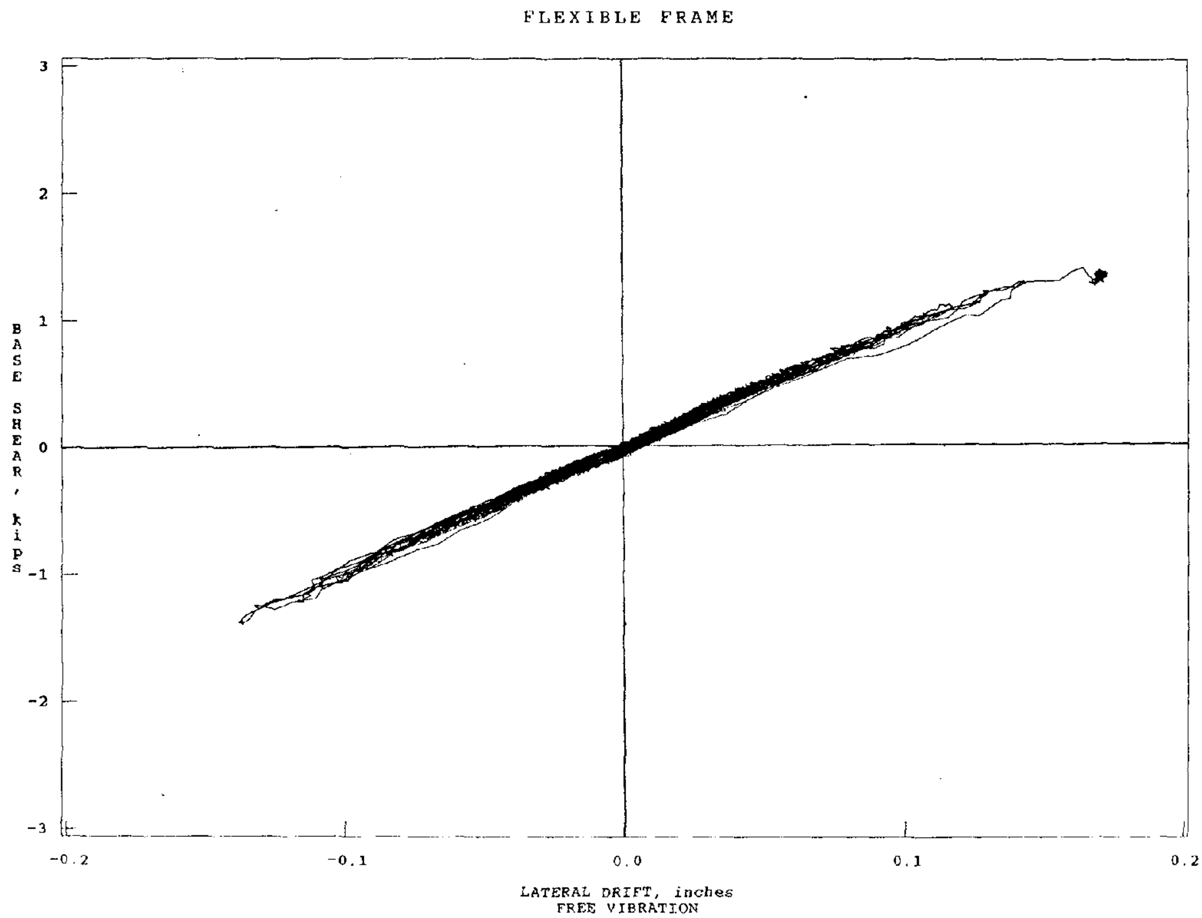
(8) Plot of the fast Fourier amplitude versus frequency

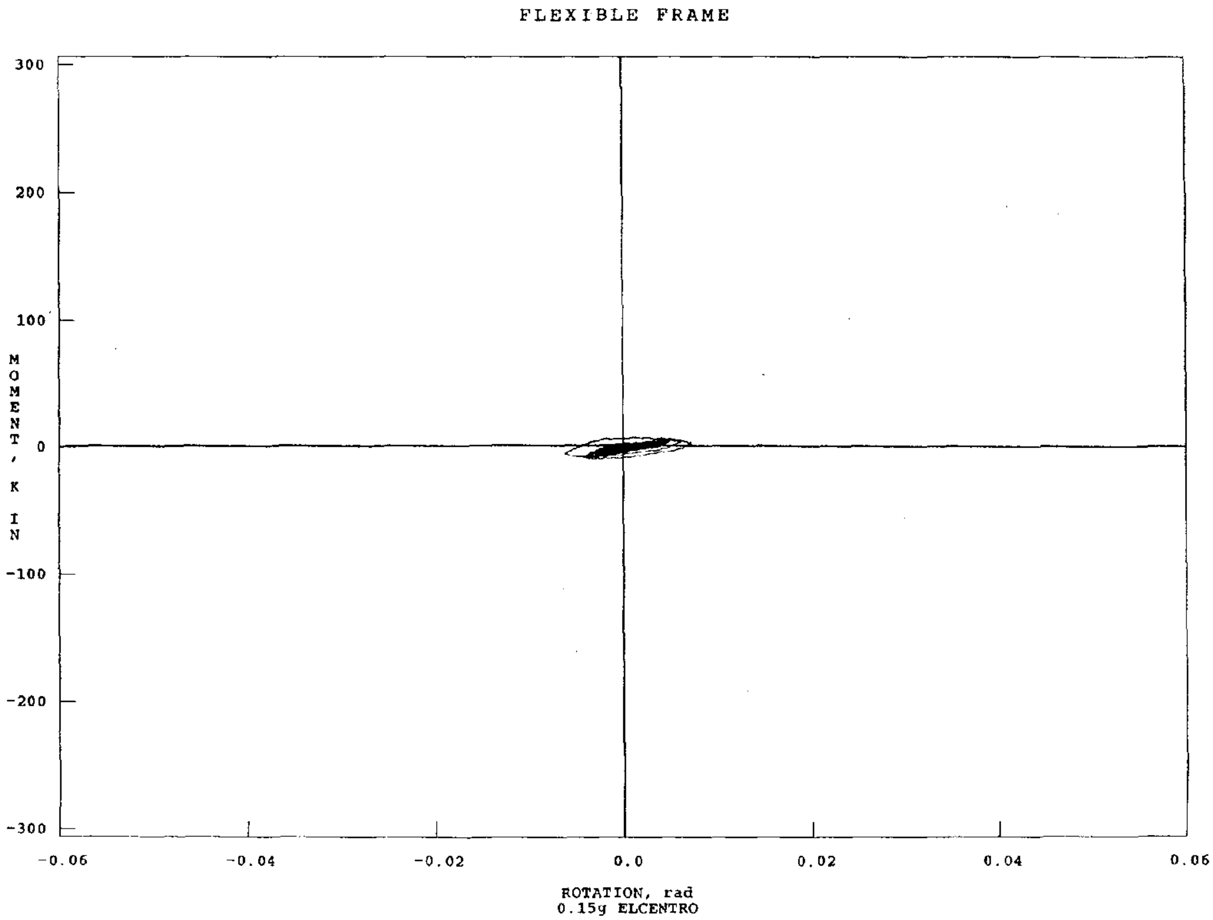
FLEXIBLE FRAME

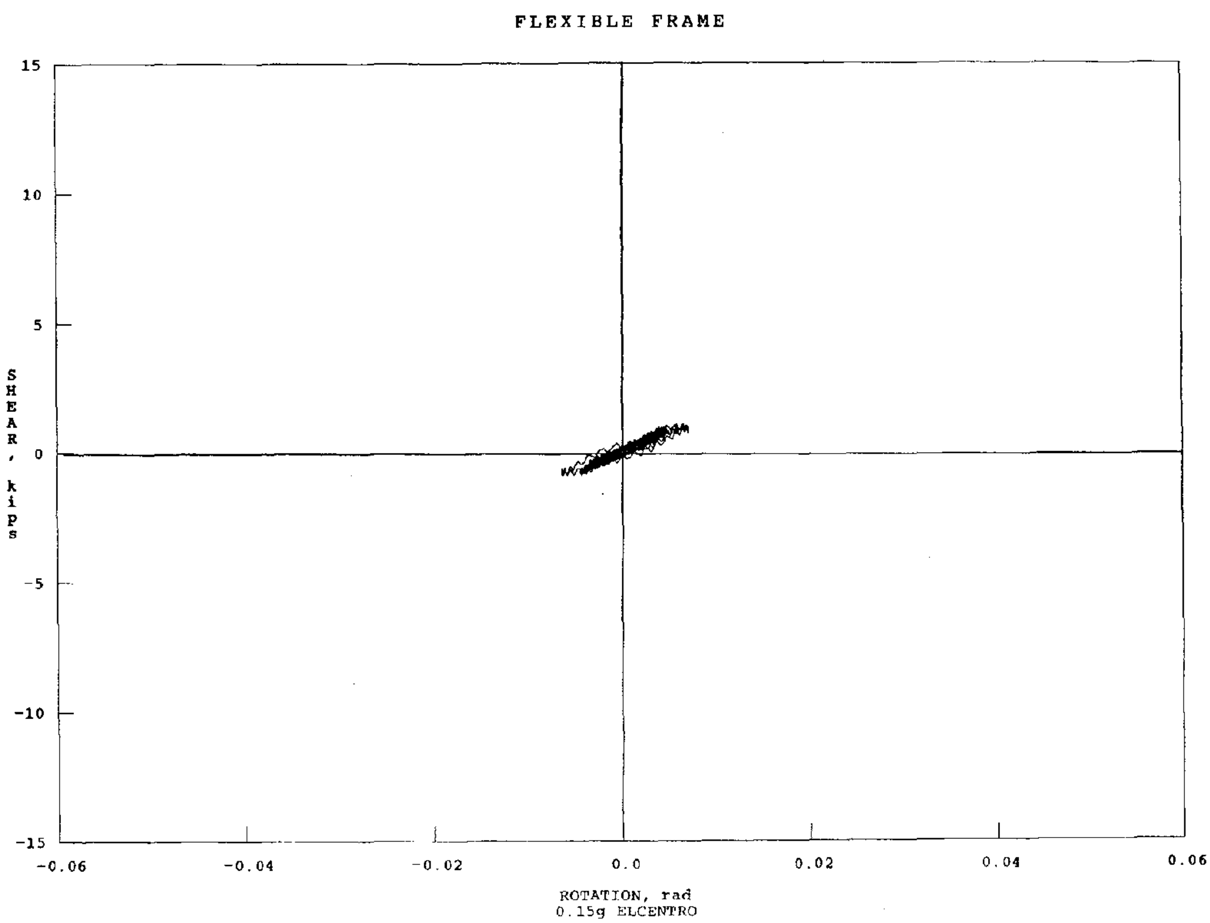


FLEXIBLE FRAME

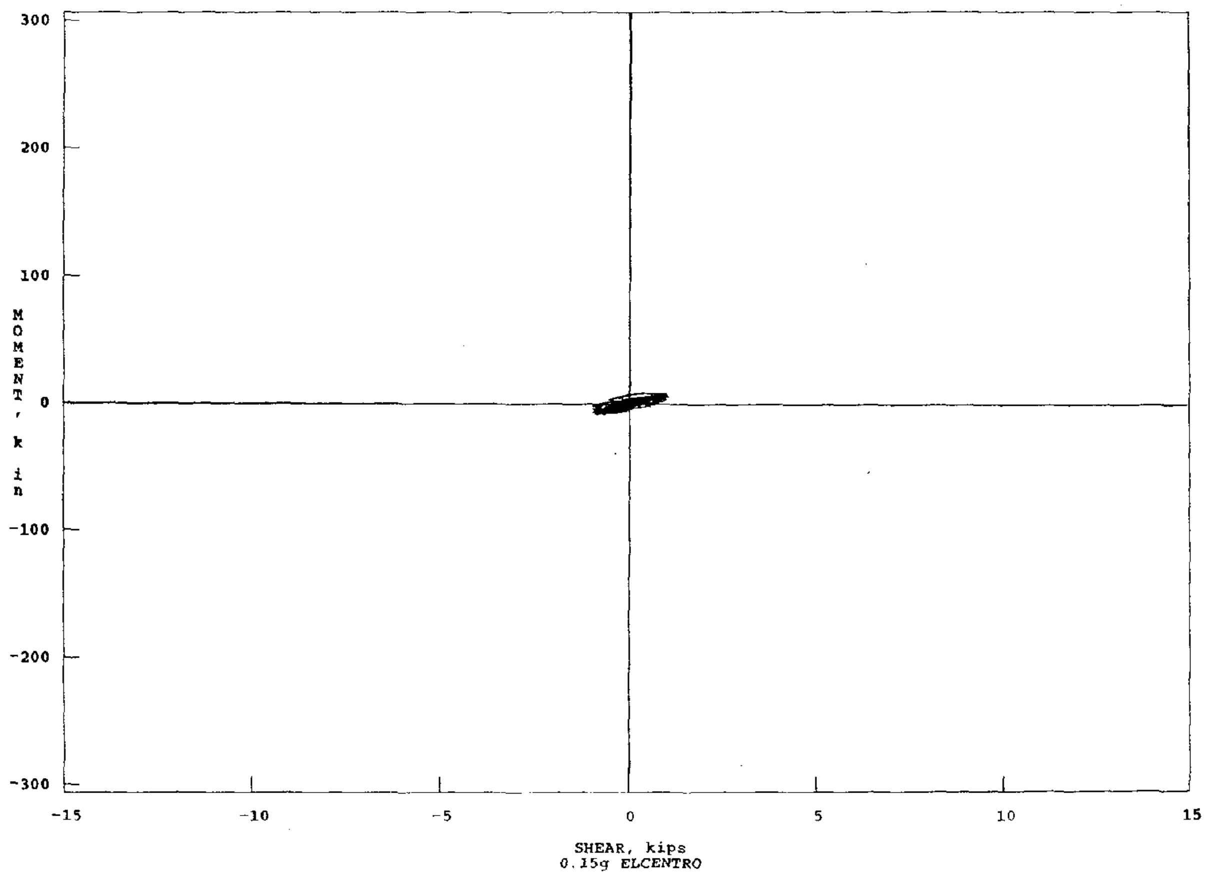



FLEXIBLE FRAME
BASE SHEAR, kips
3
2
1
0
-1
-2
-3
-0.2
-0.1
0.0
0.1
0.2
LATERAL DRIFT, inches
FREE VIBRATION

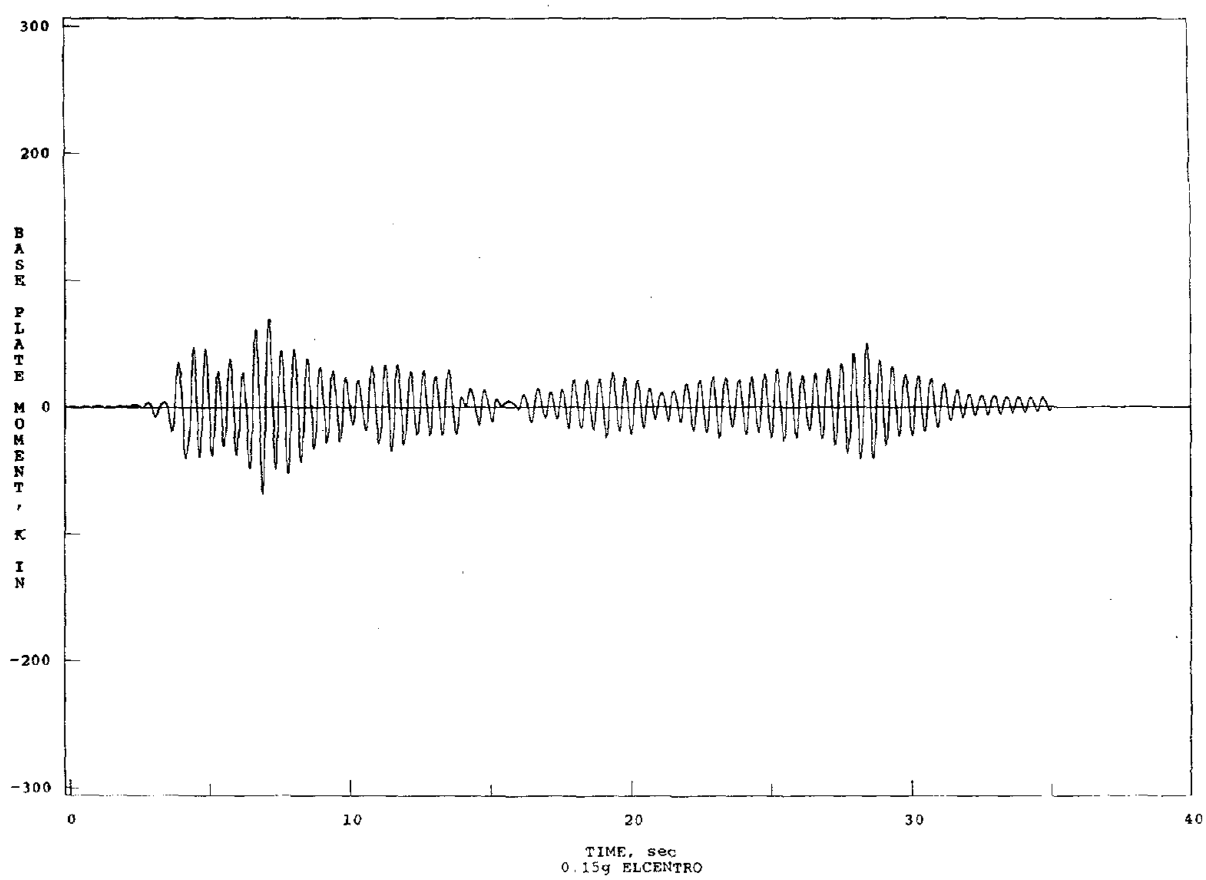

FLEXIBLE FRAME
300
200
100
0
-100
-200
-300
MOMENT, K IN
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.15g ELCENTRO


FLEXIBLE FRAME
15
10
5
0
-5
-10
-15
SHEAR, kips
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.15g ELCENTRO

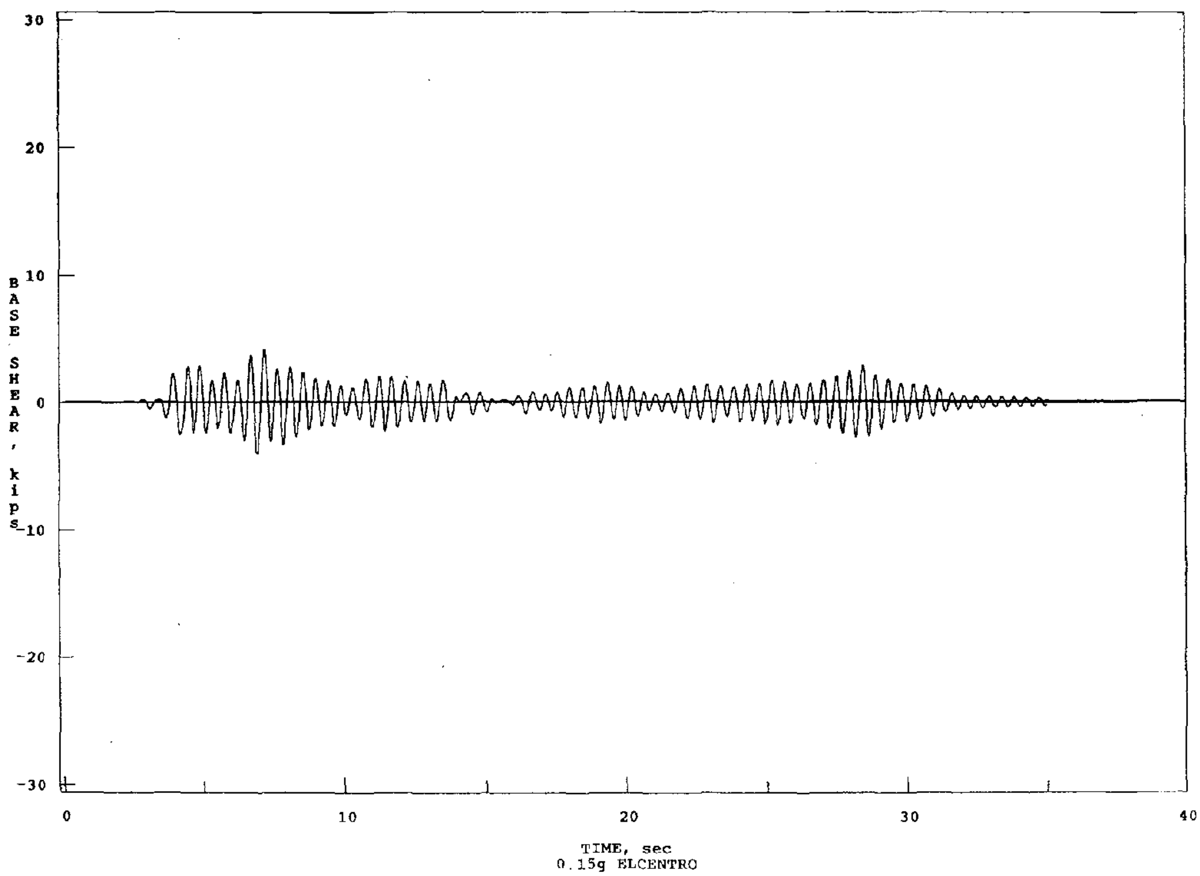
FLEXIBLE FRAME



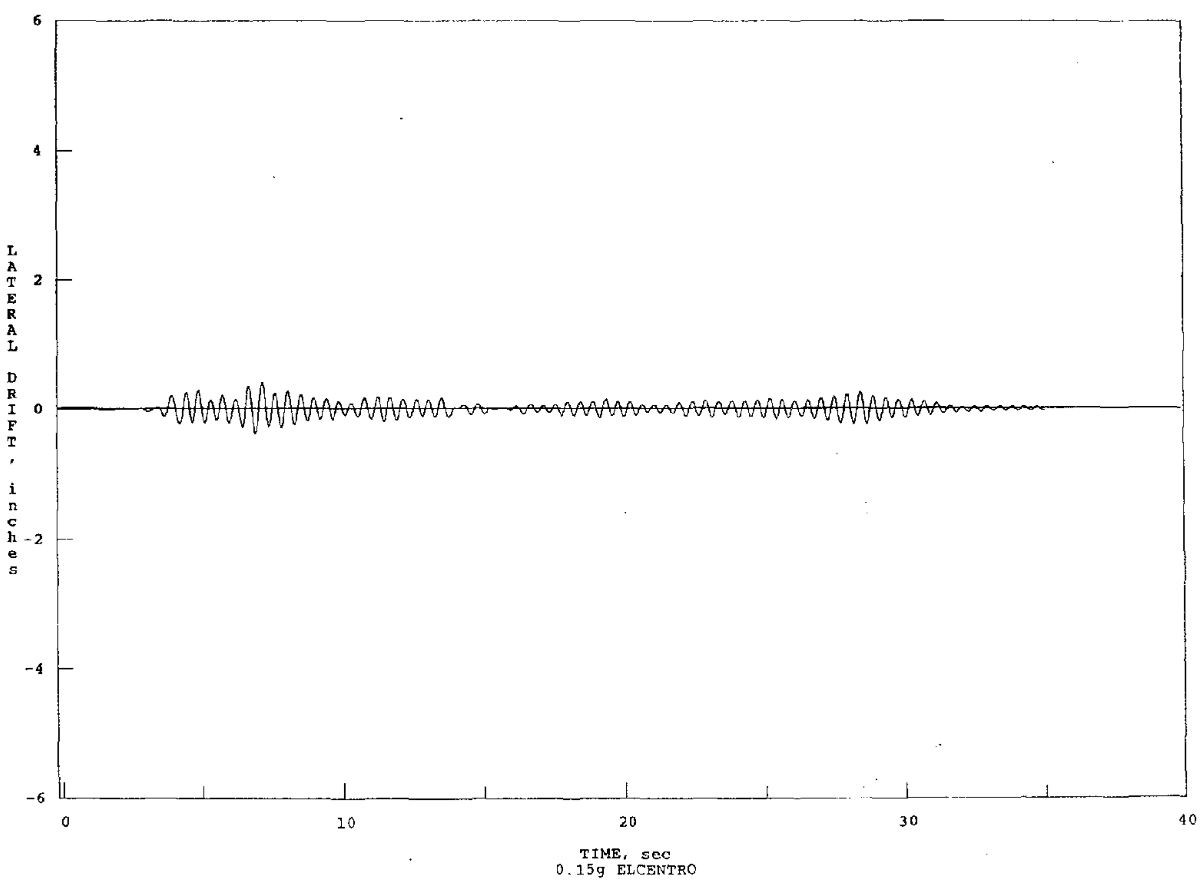
FLEXIBLE FRAME

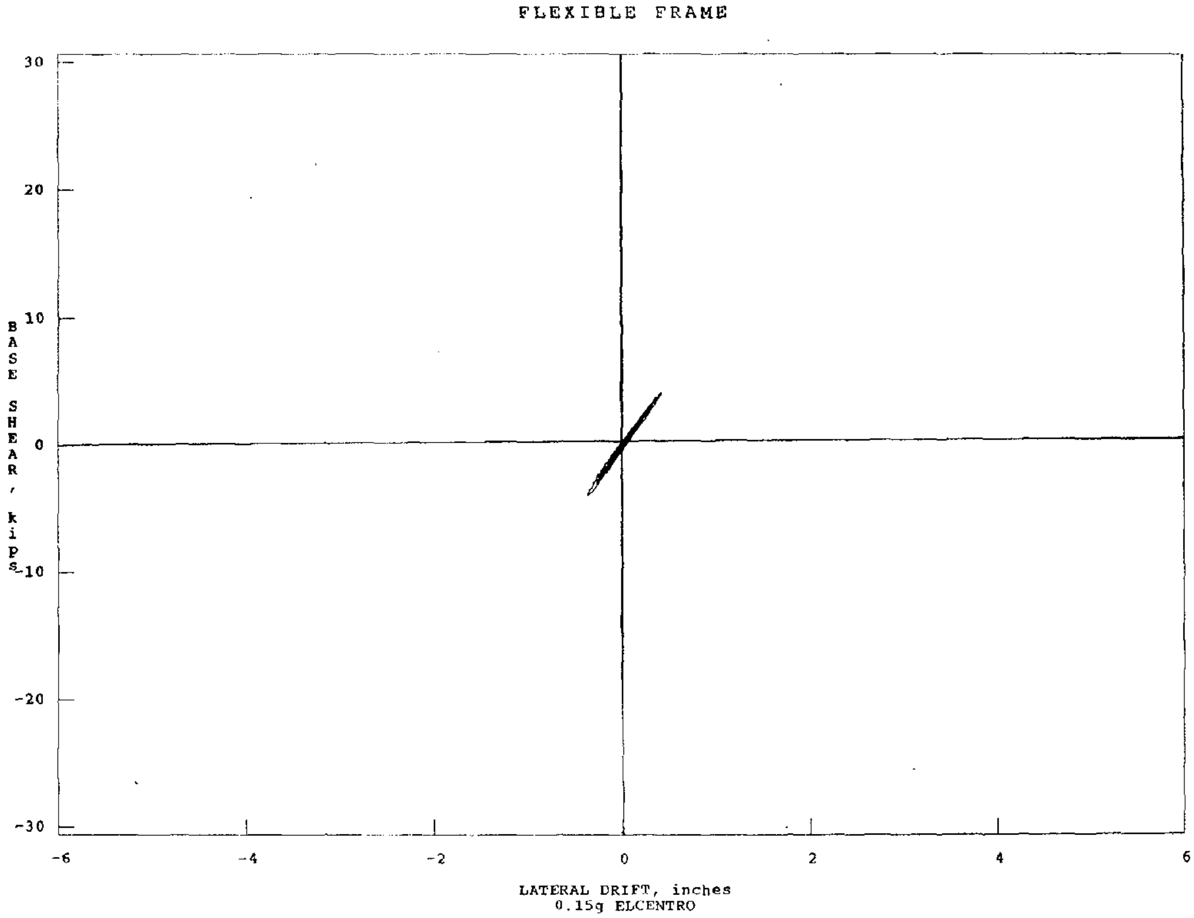


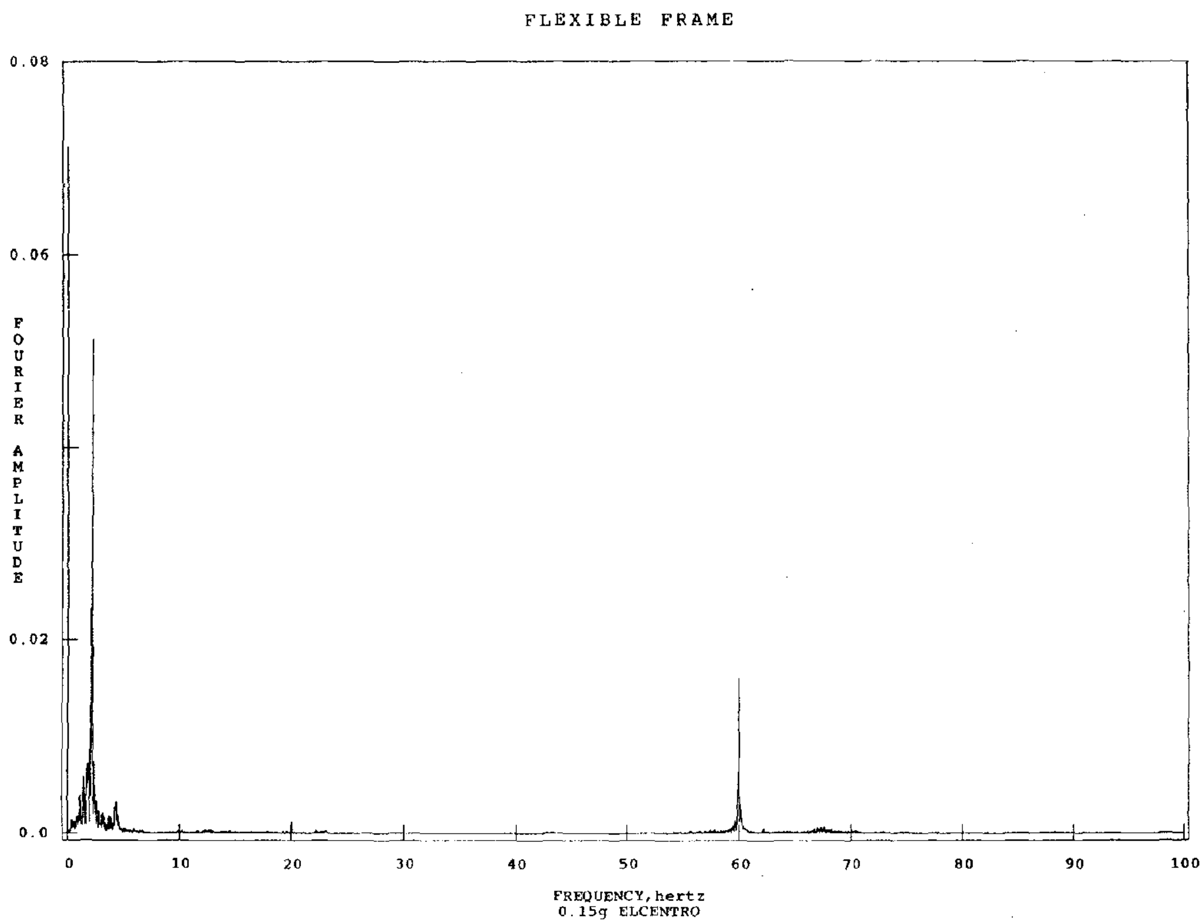
FLEXIBLE FRAME



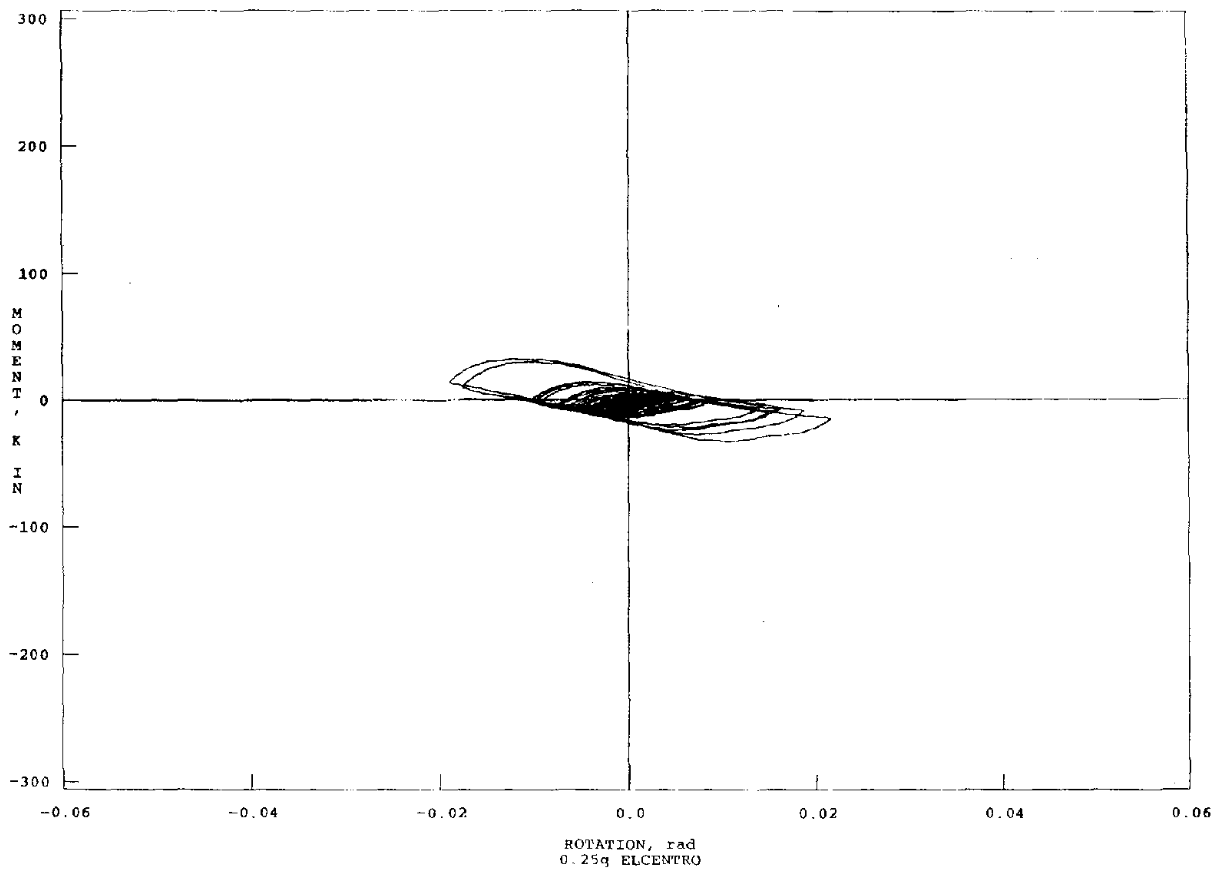
FLEXIBLE FRAME



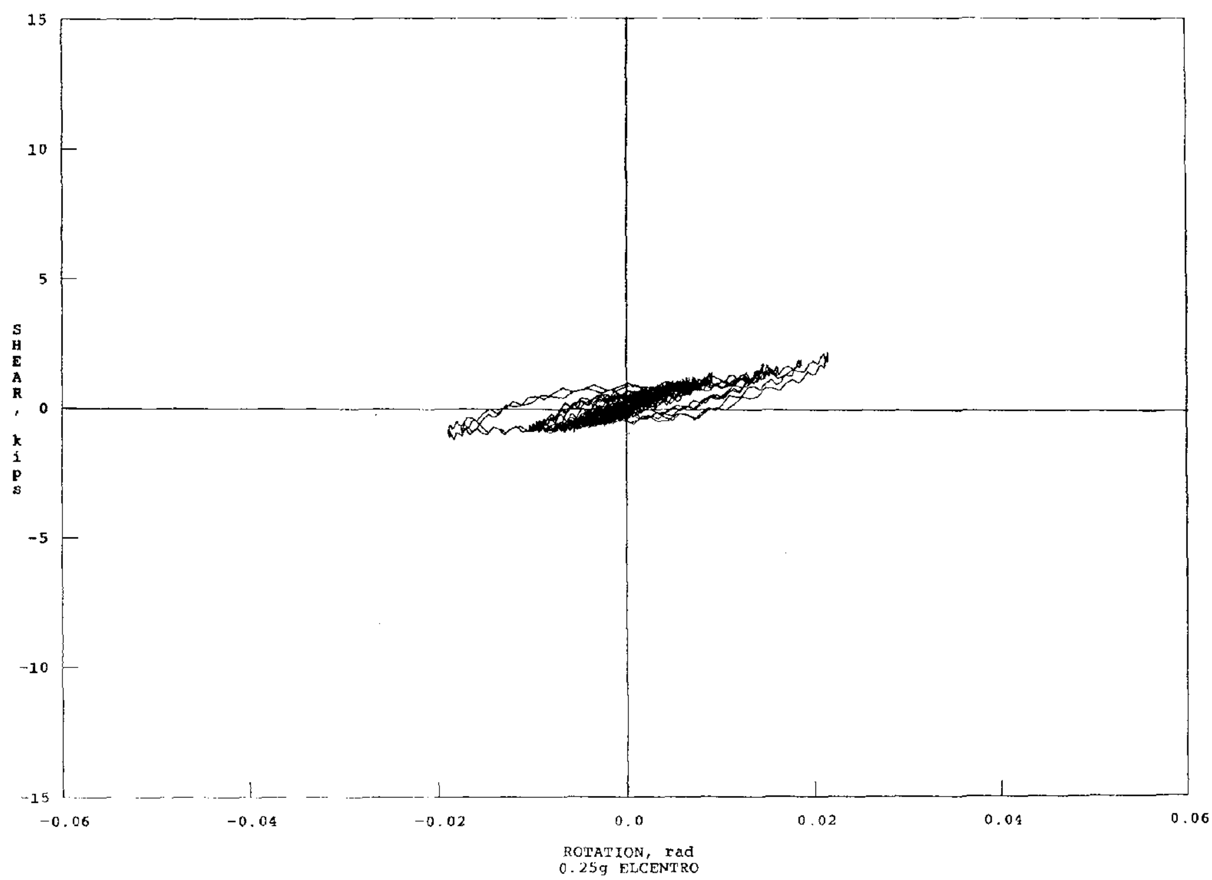

FLEXIBLE FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.15g ELCENTRO


FLEXIBLE FRAME
0.08
0.06
0.02
0.0
FOURIER AMPLITUDE
0
10
20
30
40
50
60
70
80
90
100
FREQUENCY, hertz
0.15g ELCENTRO

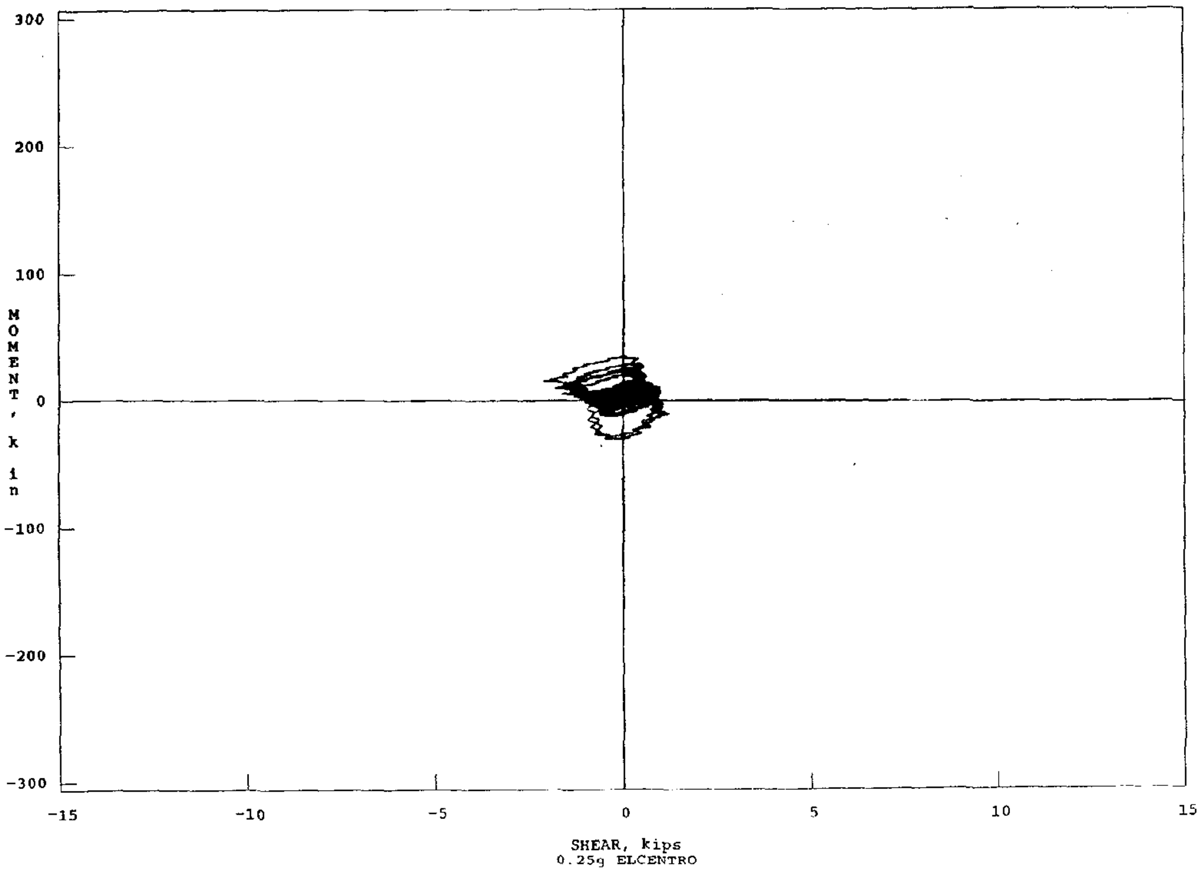
FLEXIBLE FRAME



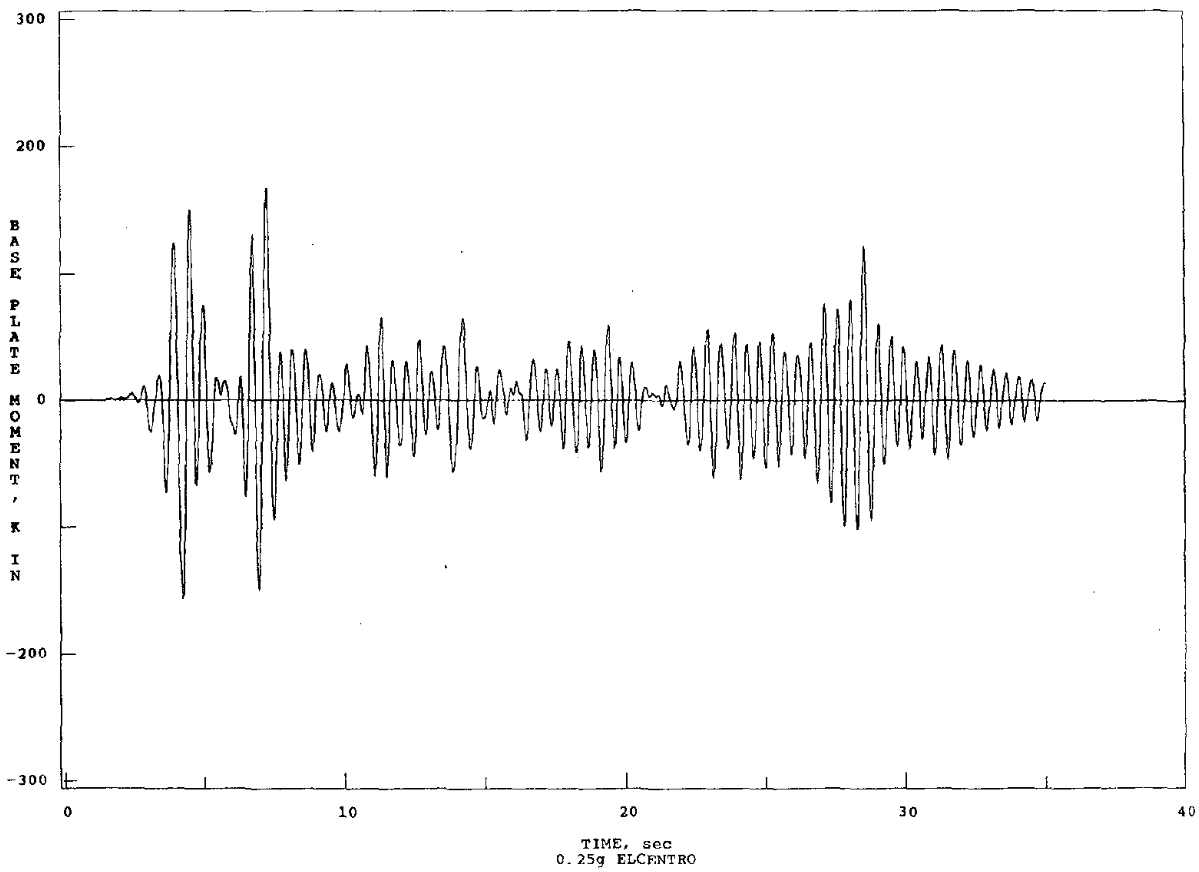
FLEXIBLE FRAME



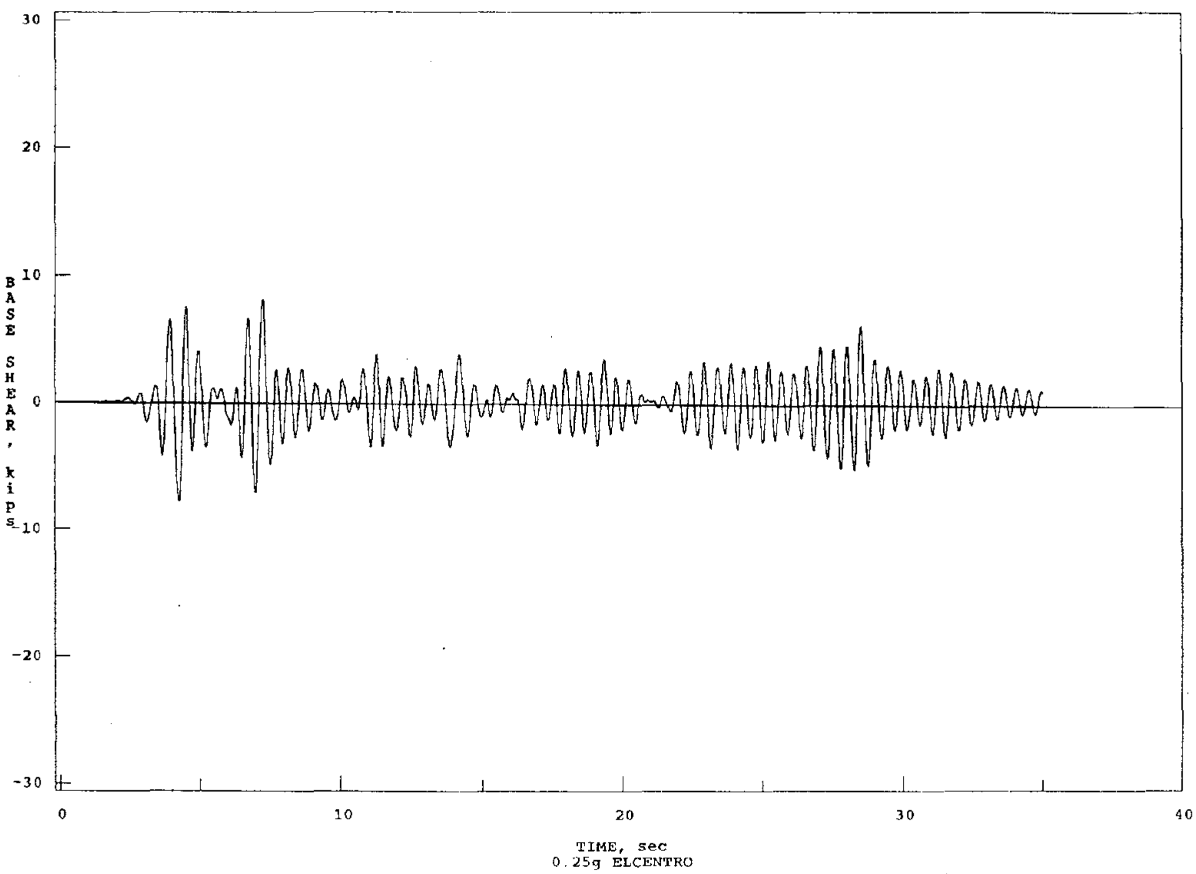
FLEXIBLE FRAME



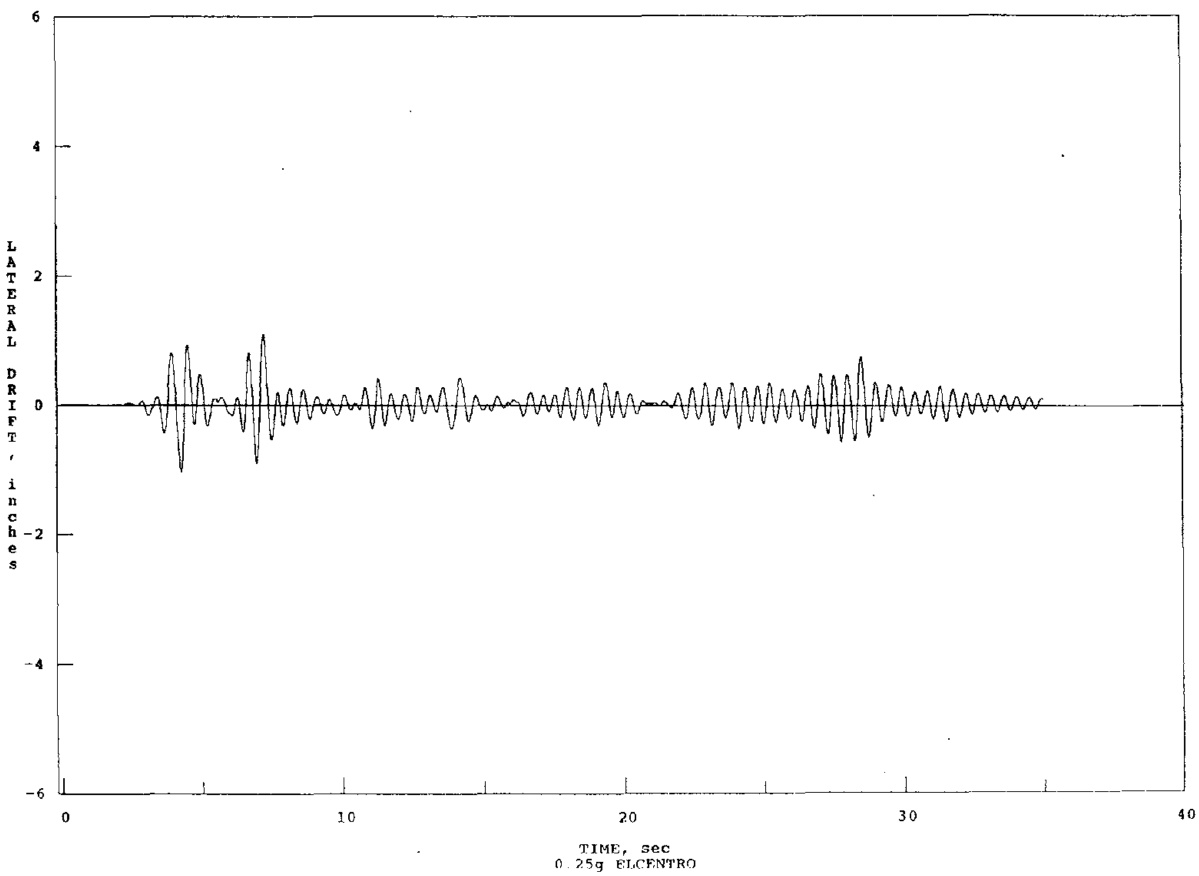
FLEXIBLE FRAME



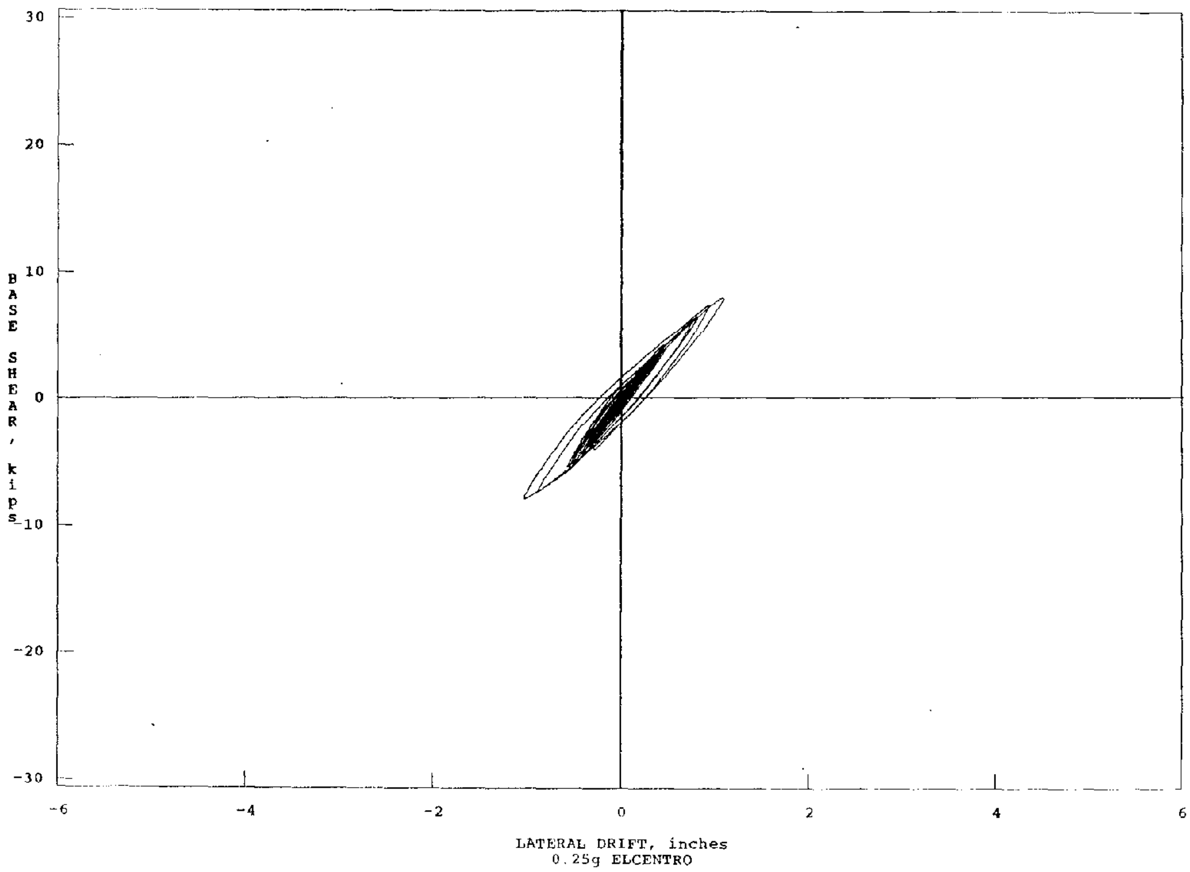
FLEXIBLE FRAME



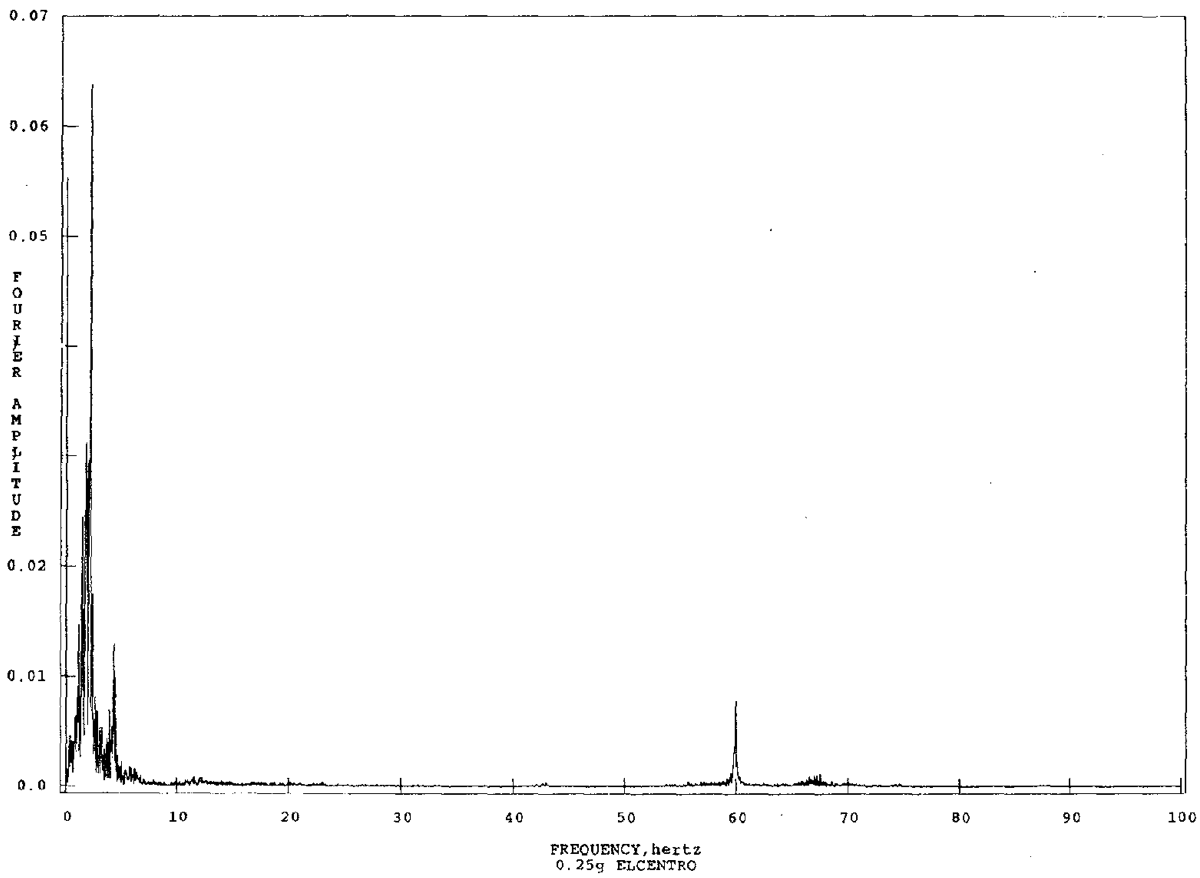
FLEXIBLE FRAME

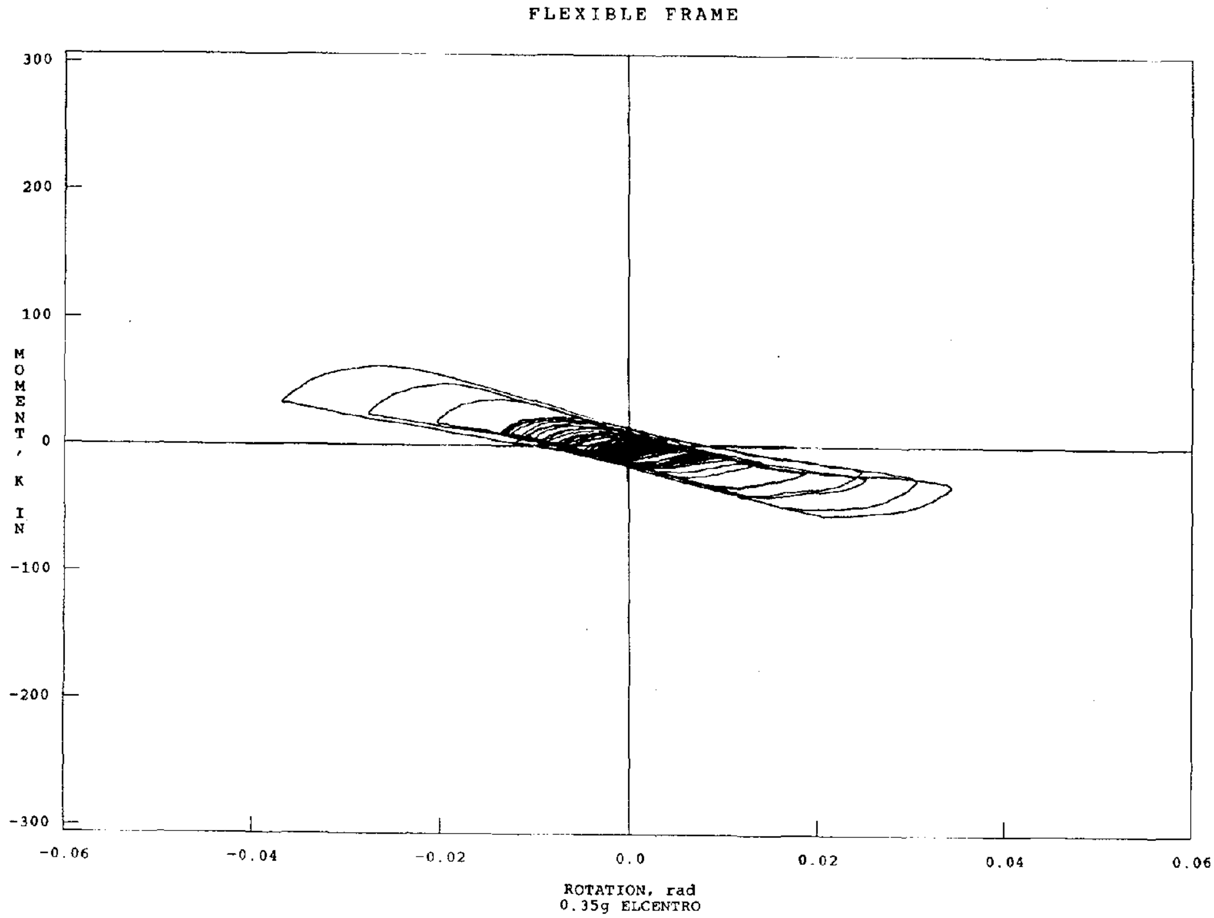


FLEXIBLE FRAME

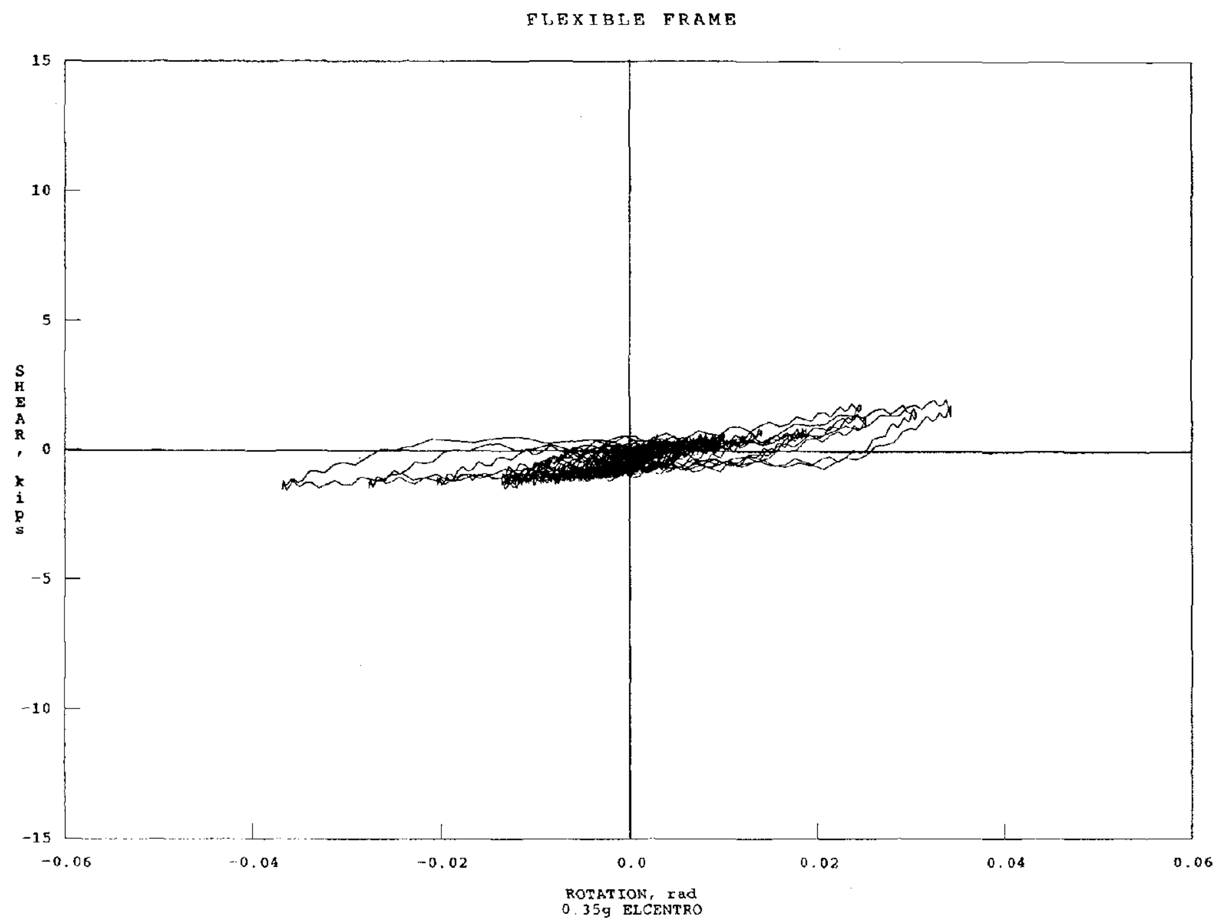


FLEXIBLE FRAME



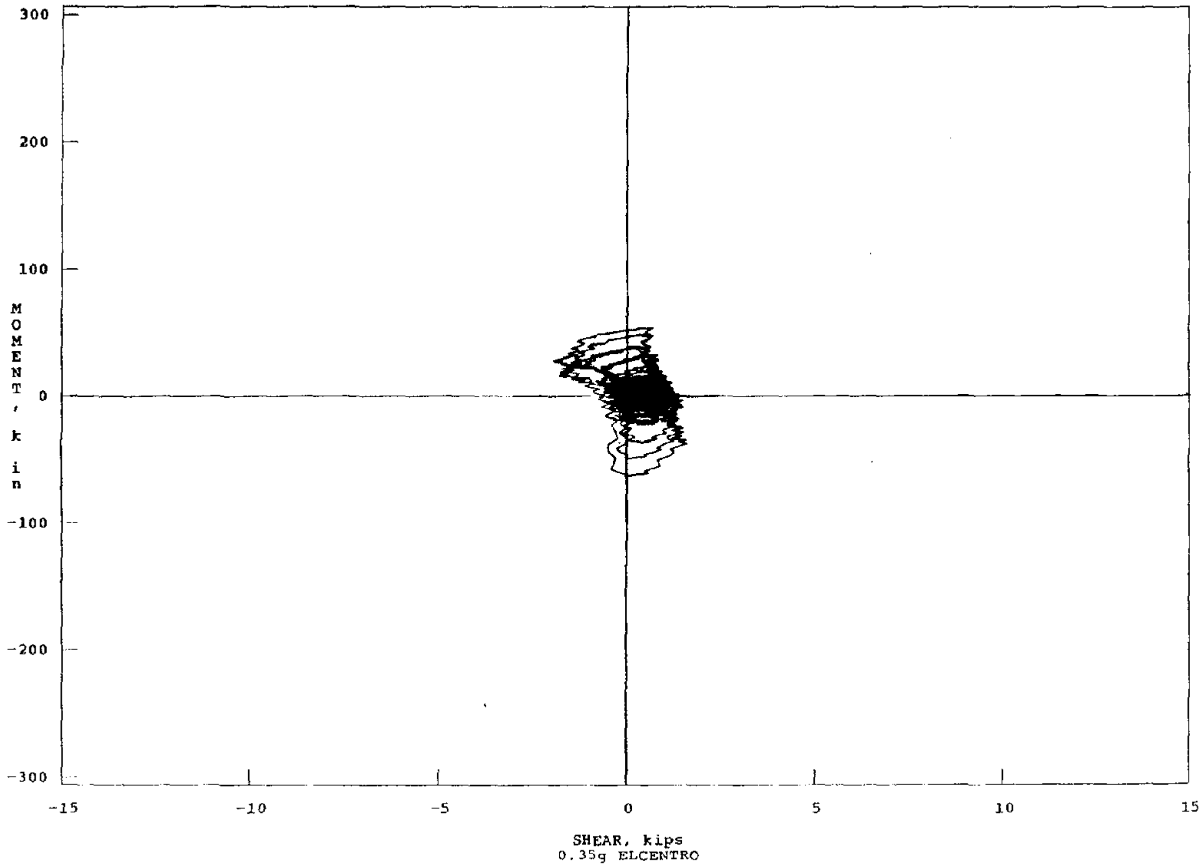


FLEXIBLE FRAME
300
200
100
0
-100
-200
-300
MOMENT, K IN
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.35g ELCENTRO

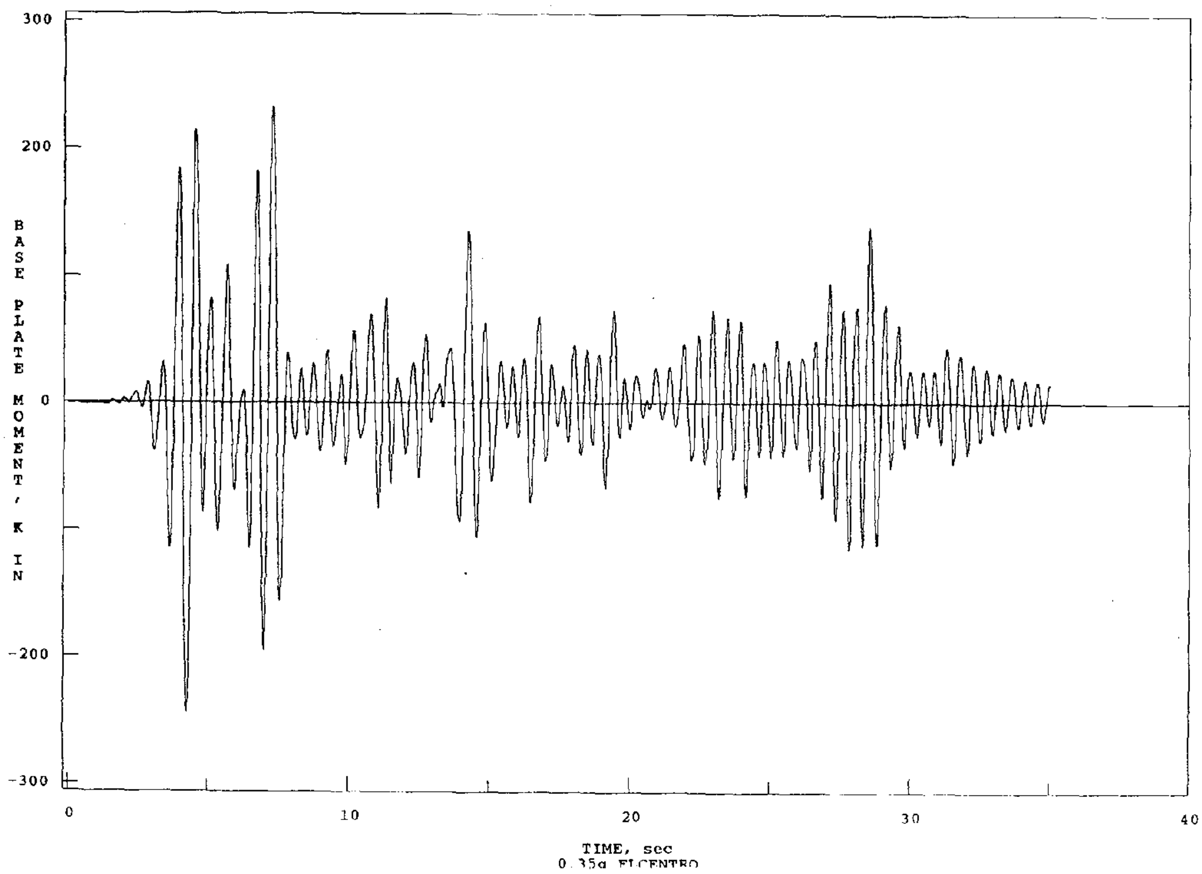


FLEXIBLE FRAME
15
10
5
0
-5
-10
-15
SHEAR, kips
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.35g ELCENTRO

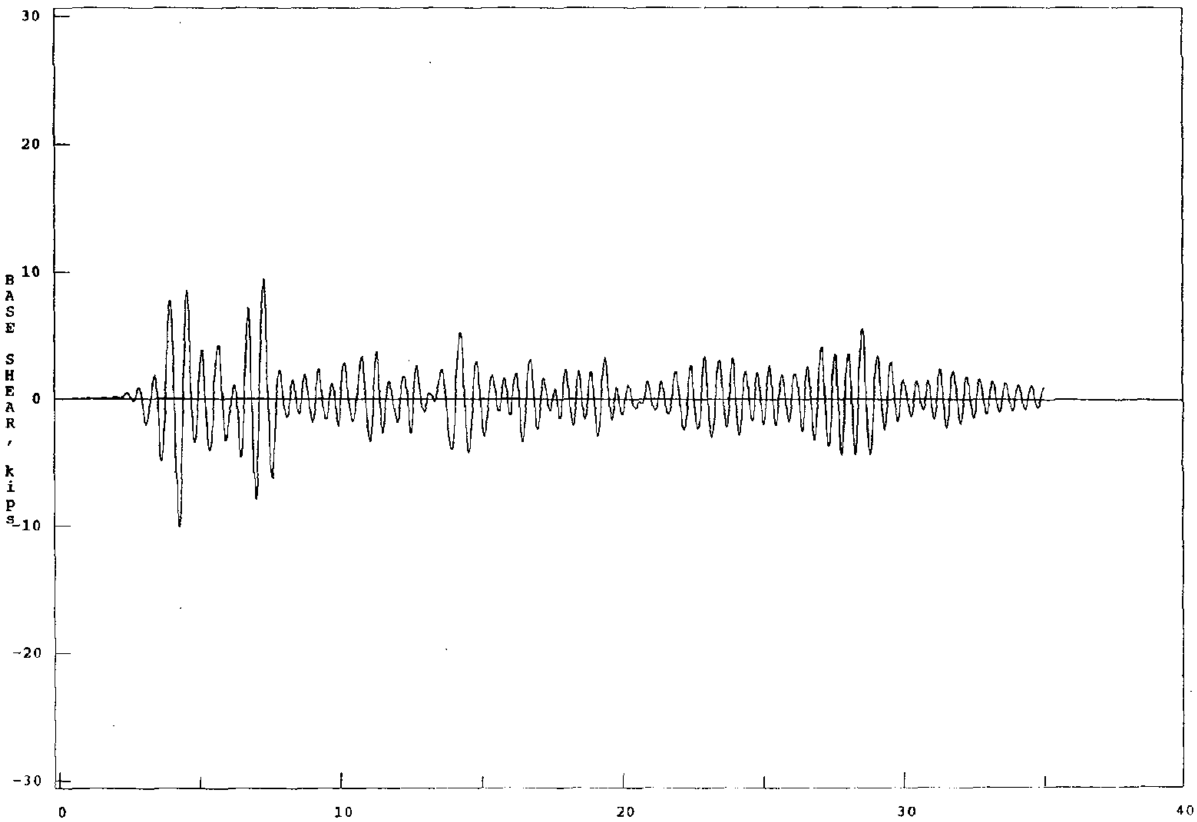
FLEXIBLE FRAME



FLEXIBLE FRAME

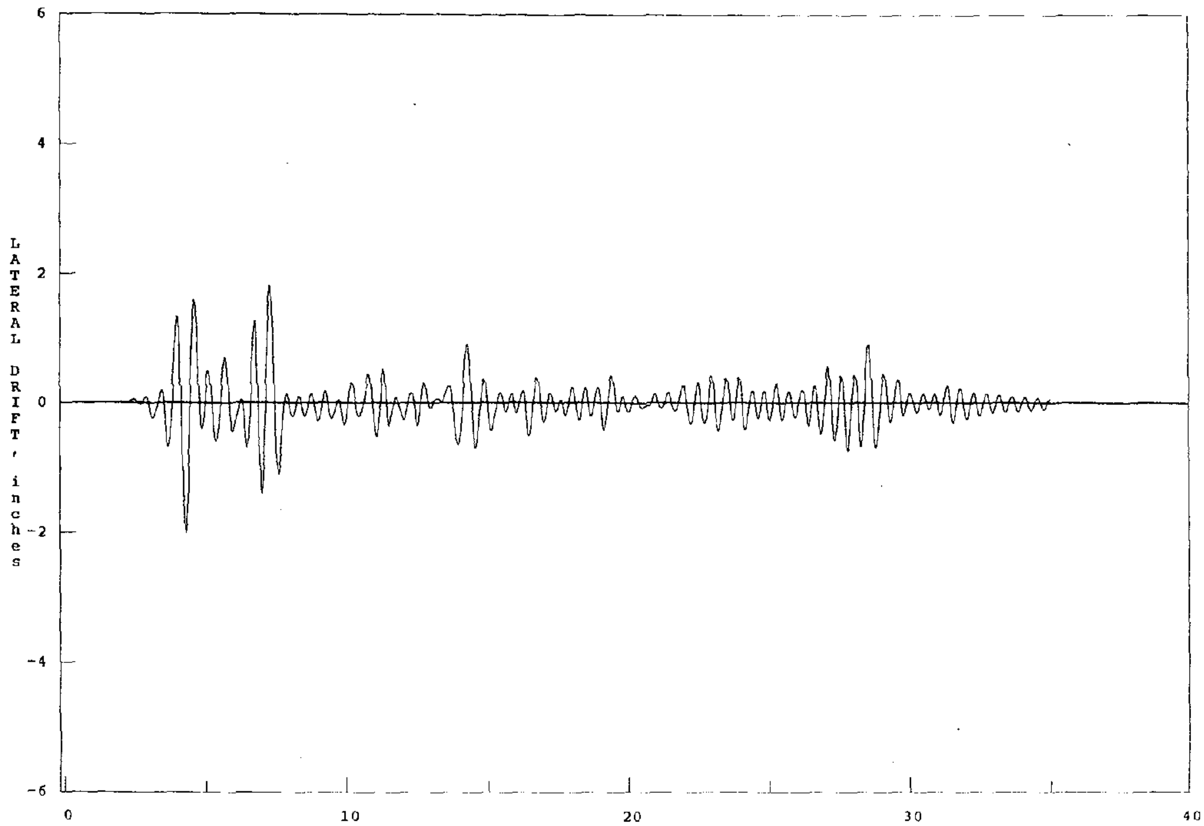


FLEXIBLE FRAME



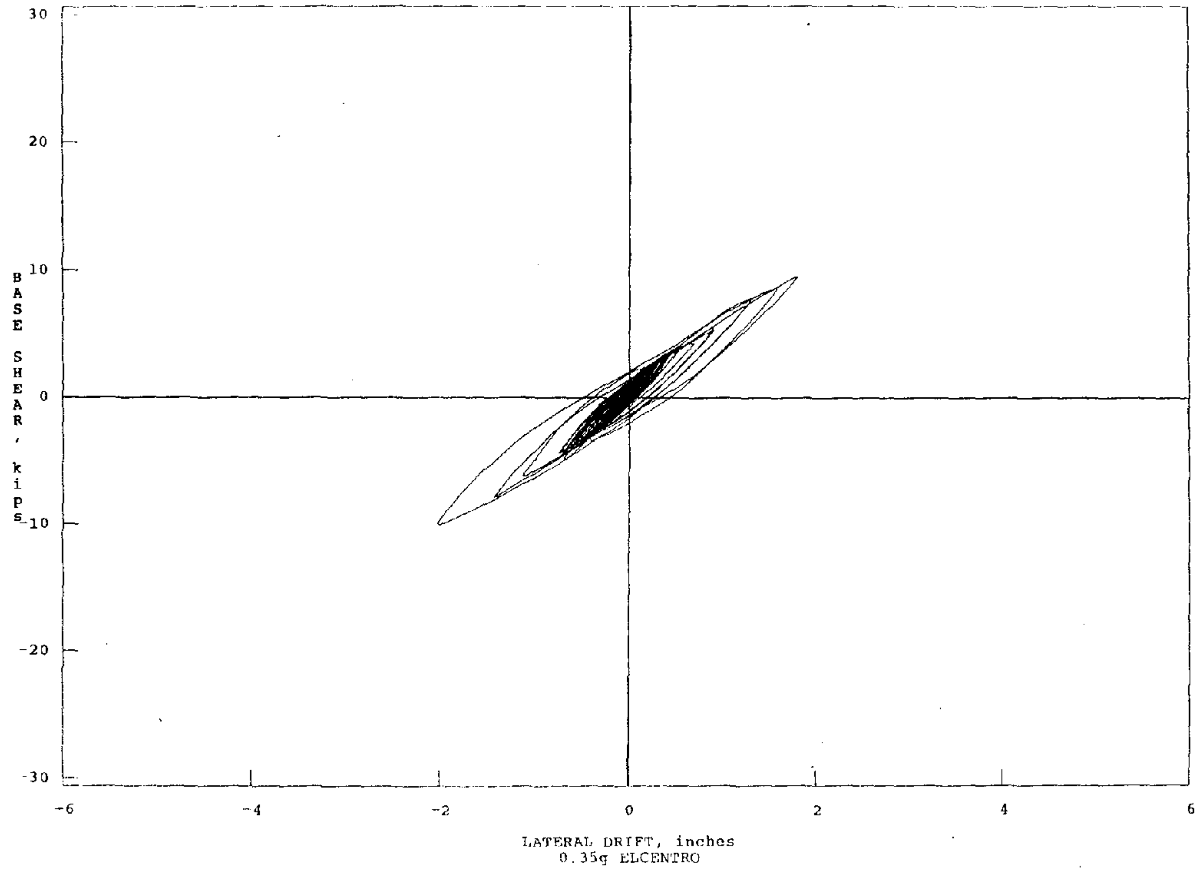
TIME, sec
0.35g ELCENTRO

FLEXIBLE FRAME

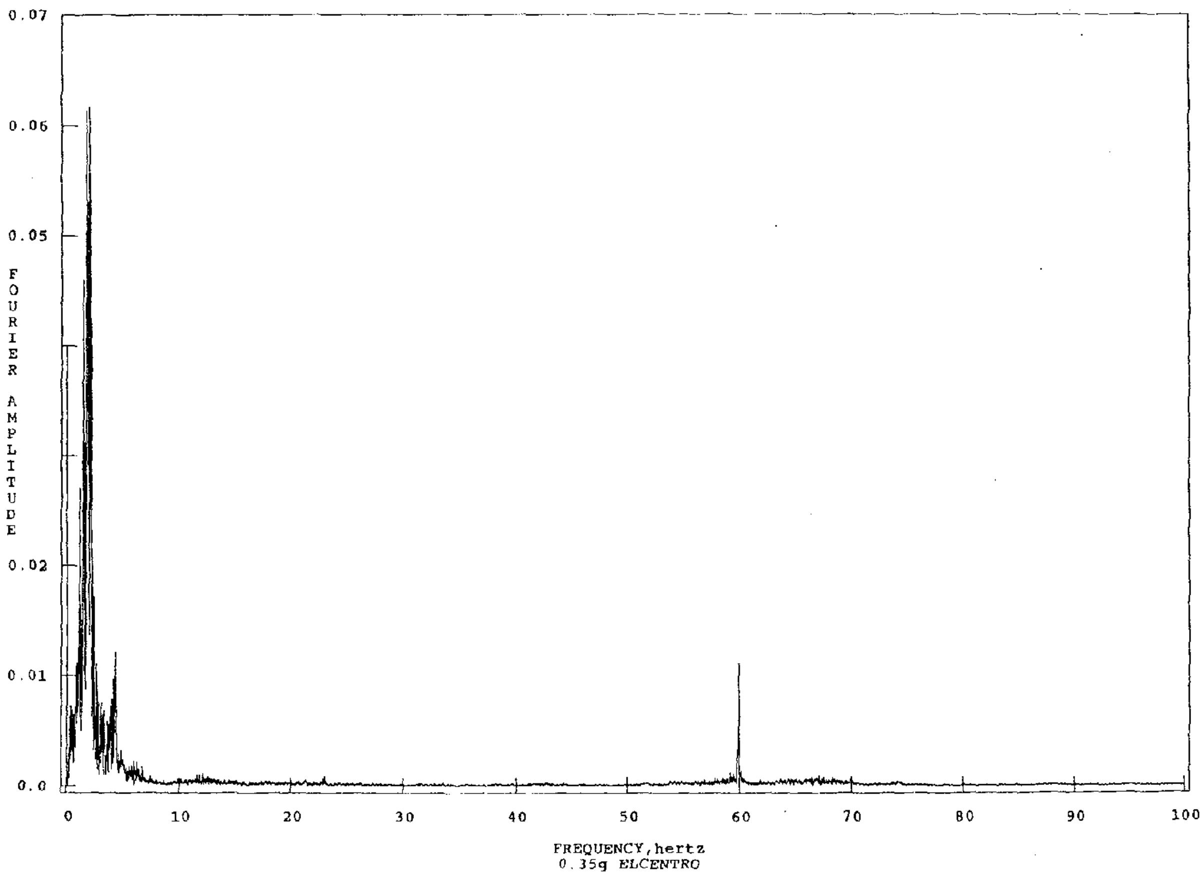


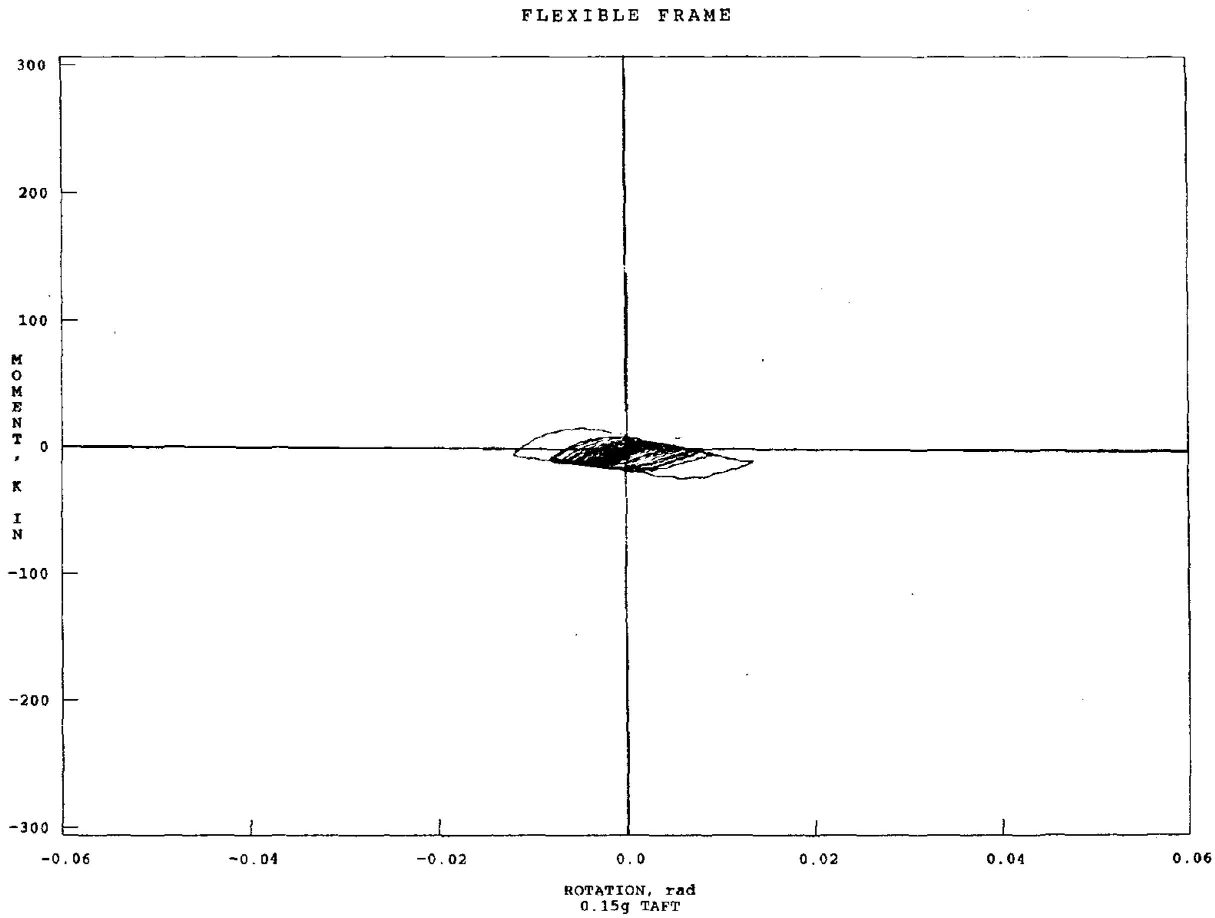
TIME, sec
0.35g ELCENTRO

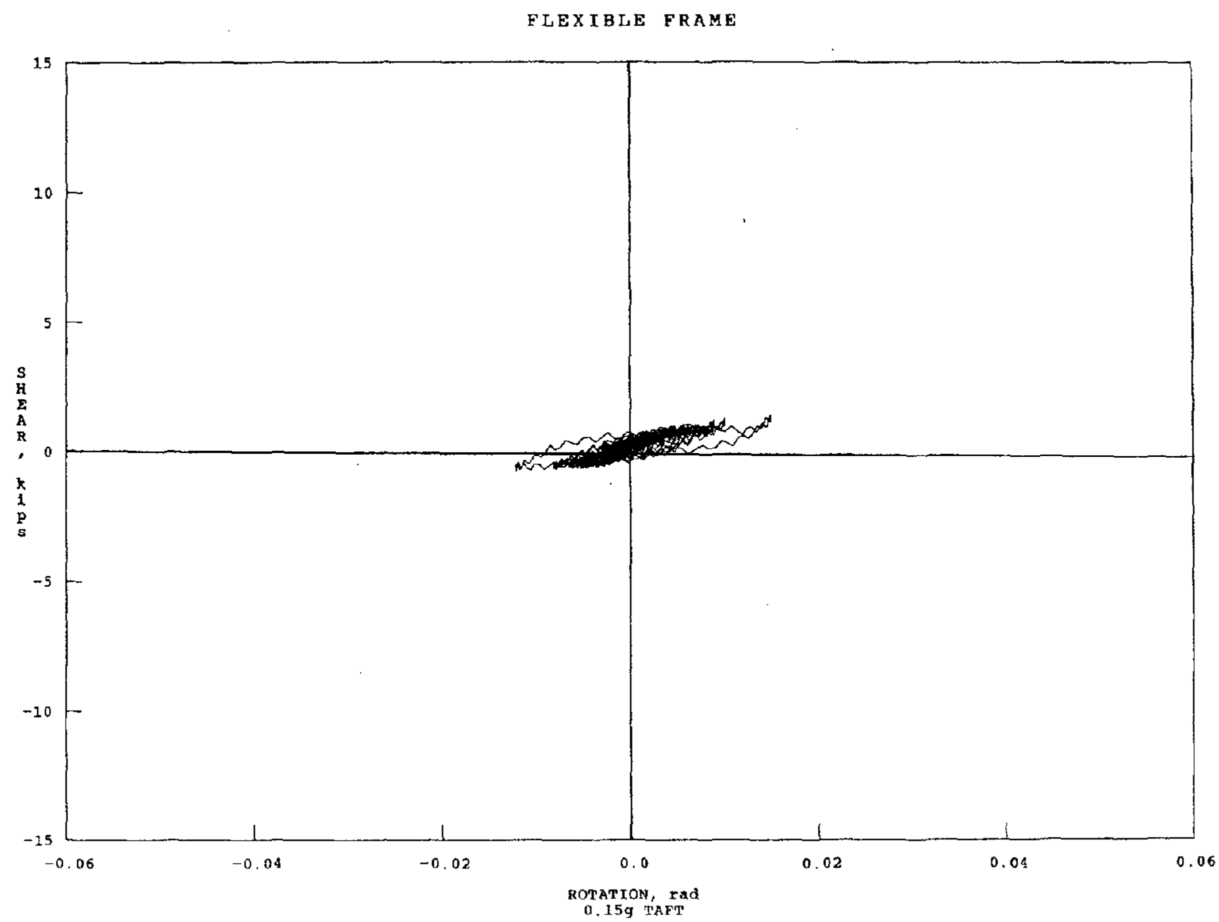
FLEXIBLE FRAME



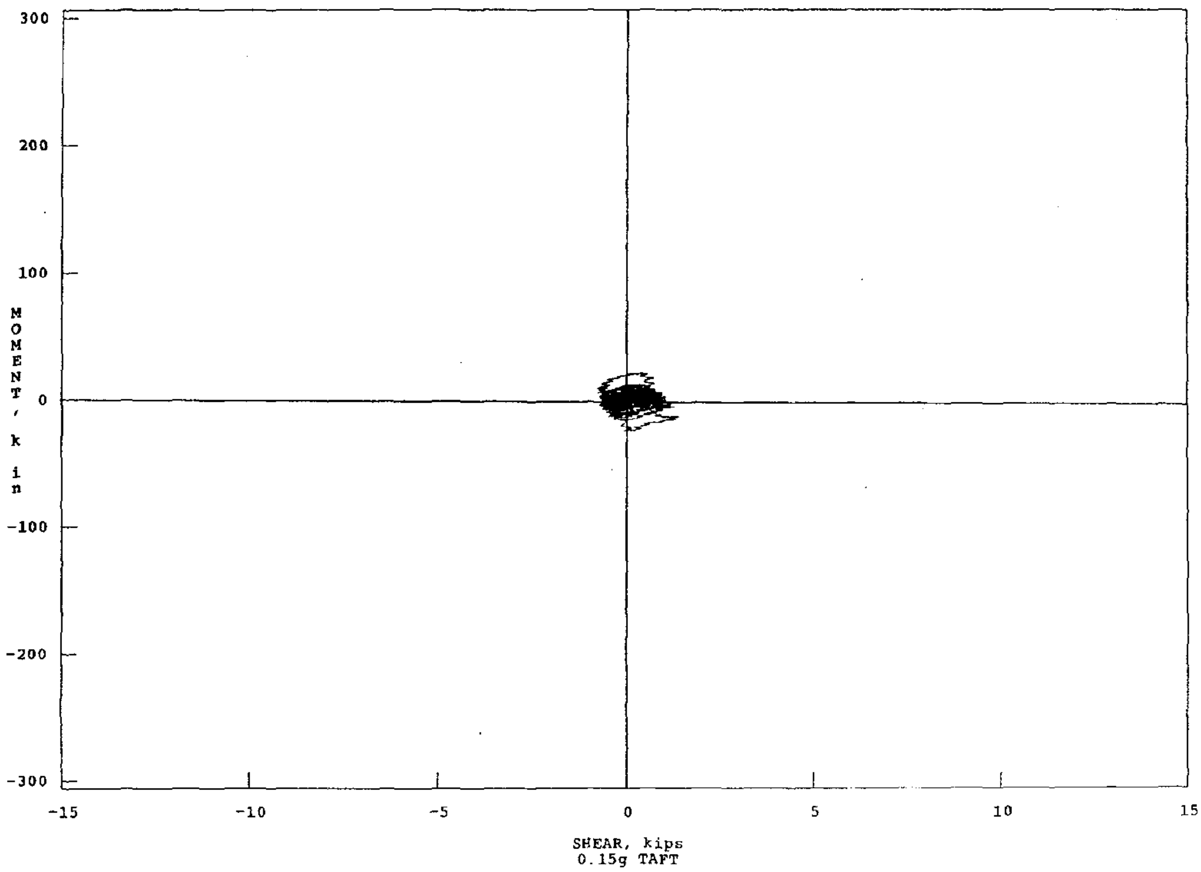
FLEXIBLE FRAME



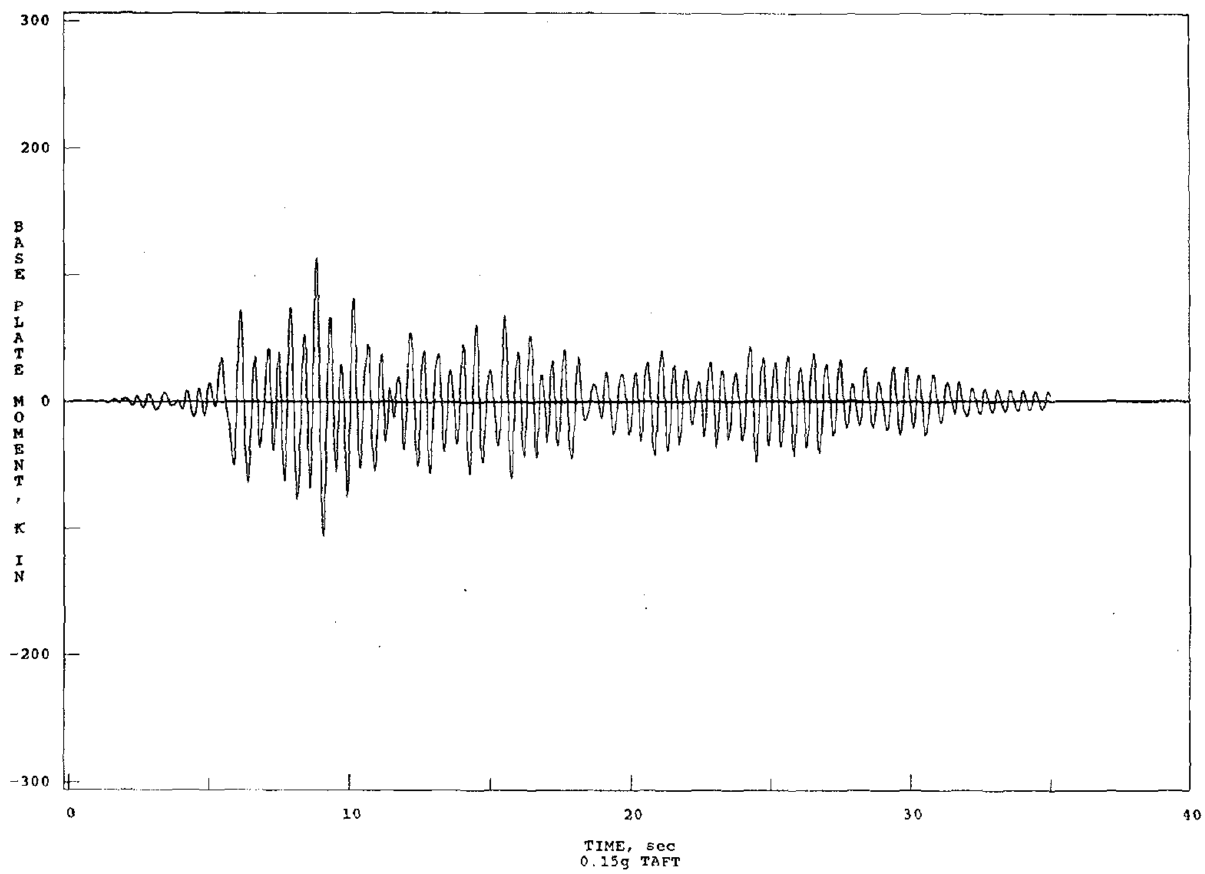

FLEXIBLE FRAME
300
200
100
0
-100
-200
-300
MOMENT, K IN
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.15g TAFT

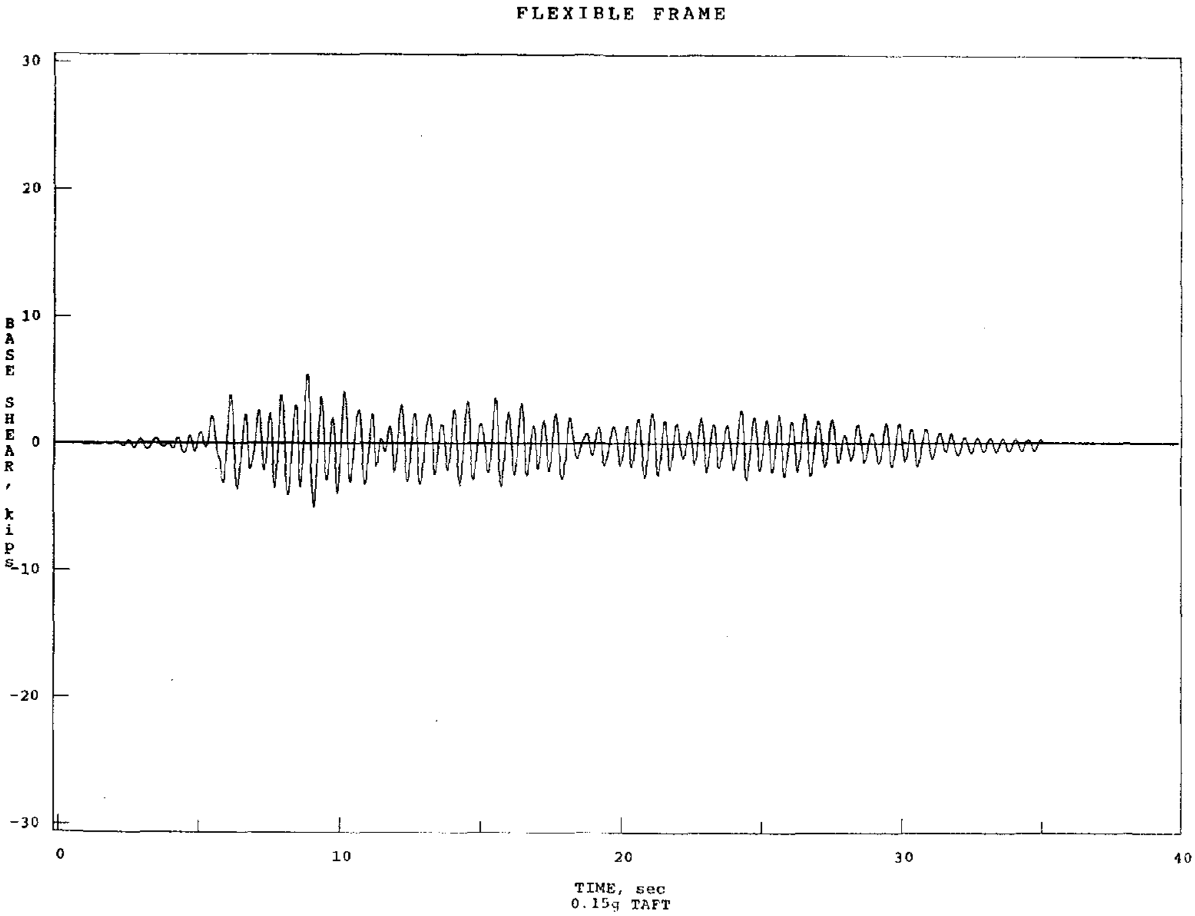

FLEXIBLE FRAME
15
10
5
0
-5
-10
-15
SHEAR, kips
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.15g TAFT

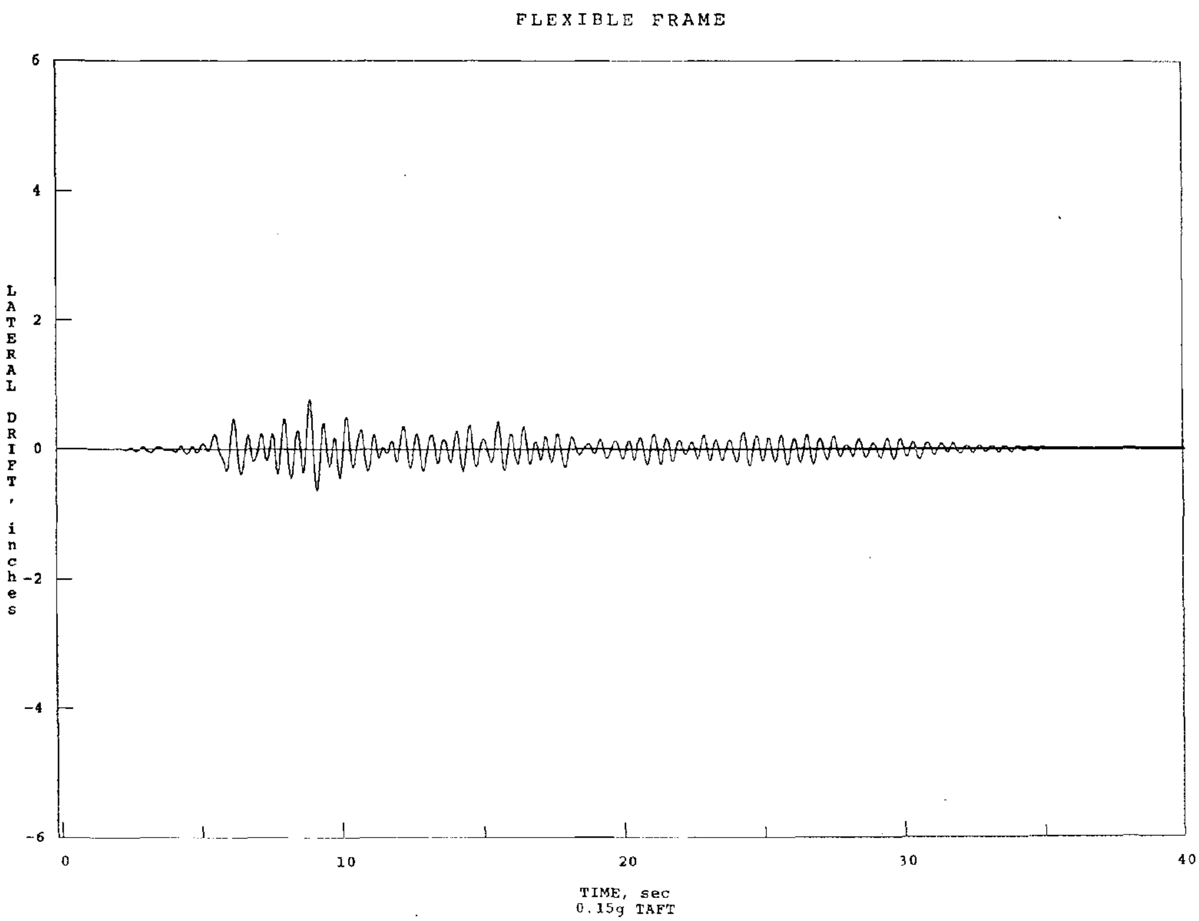
FLEXIBLE FRAME

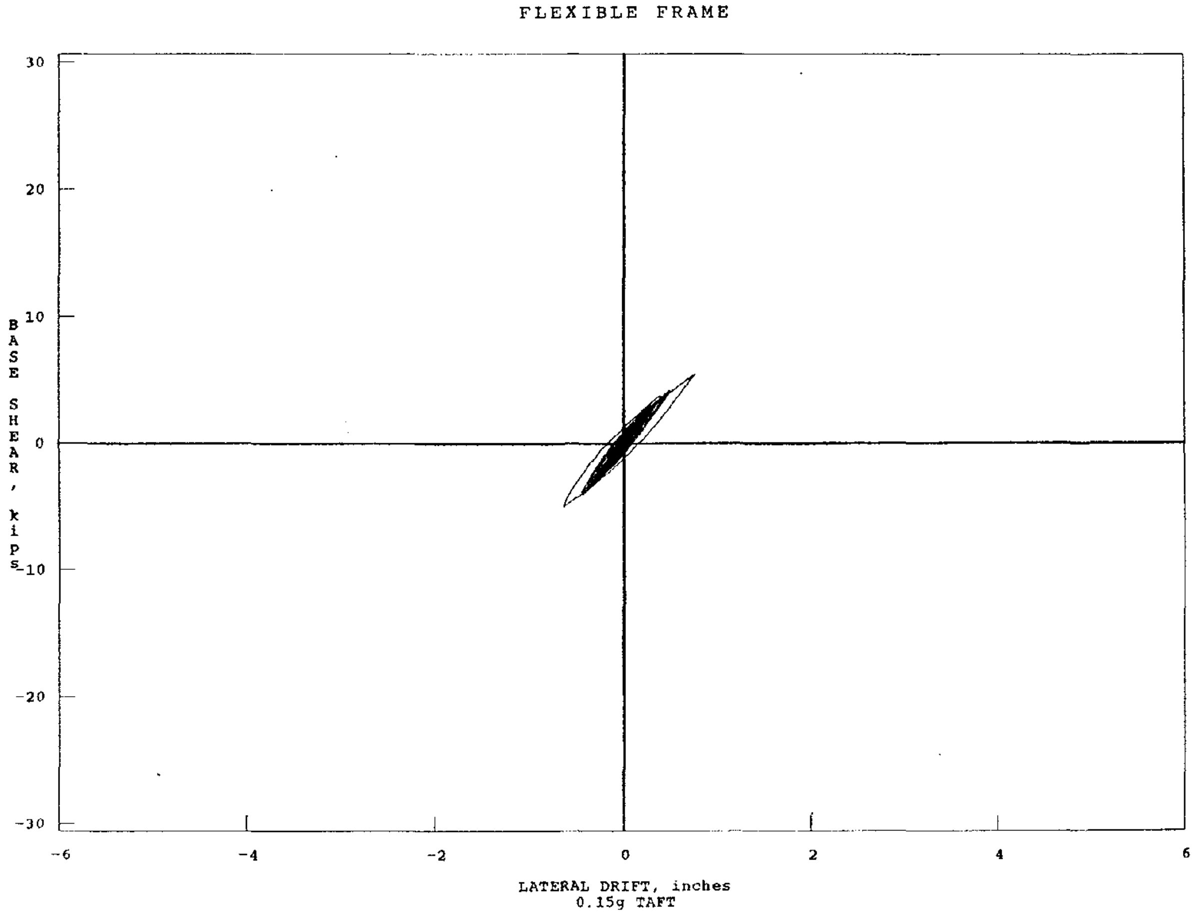


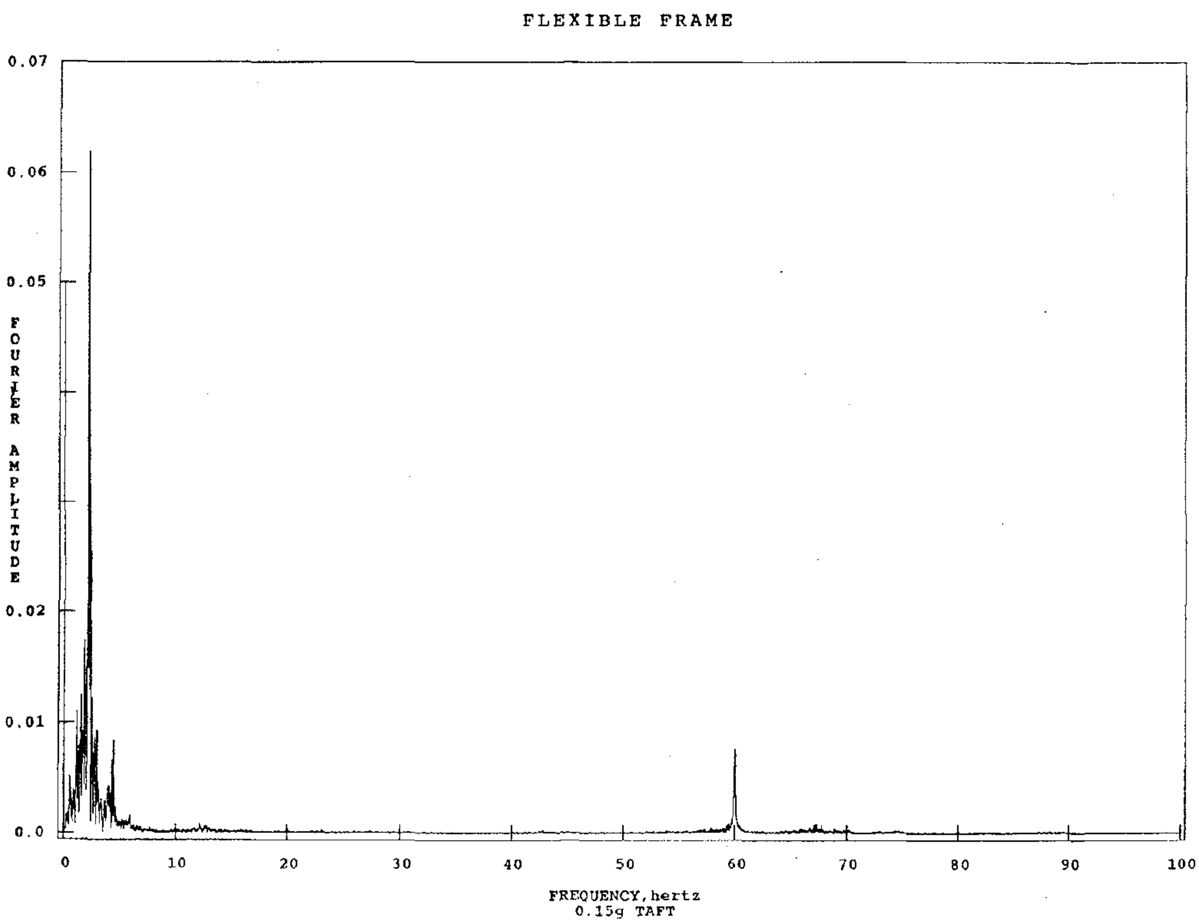
FLEXIBLE FRAME



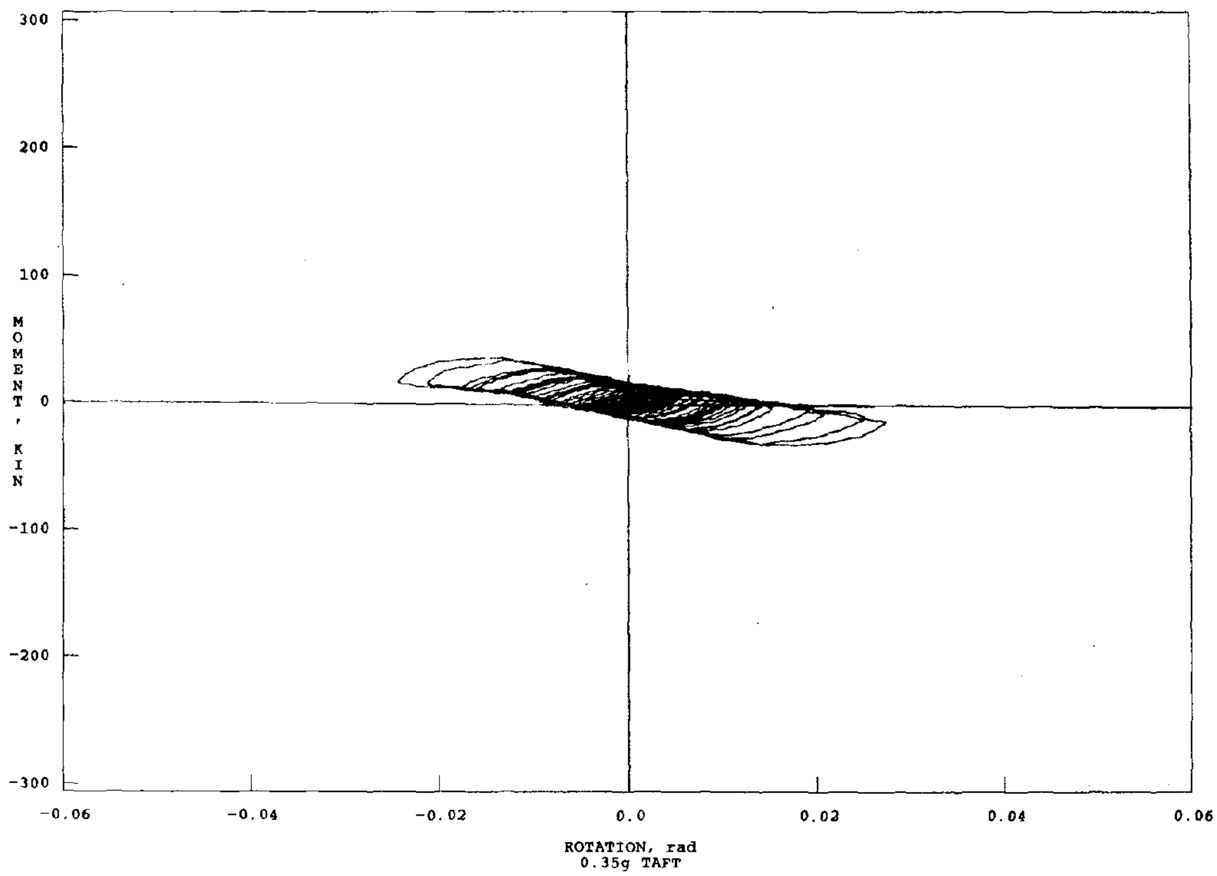

FLEXIBLE FRAME
BASE SHEAR, kips
30
20
10
0
-10
-20
-30
0
10
20
30
40
TIME, sec
0.15g TAFT


FLEXIBLE FRAME
LATERAL DRIFT, inches
6
4
2
0
-2
-4
-6
0
10
20
30
40
TIME, sec
0.15g TAFT

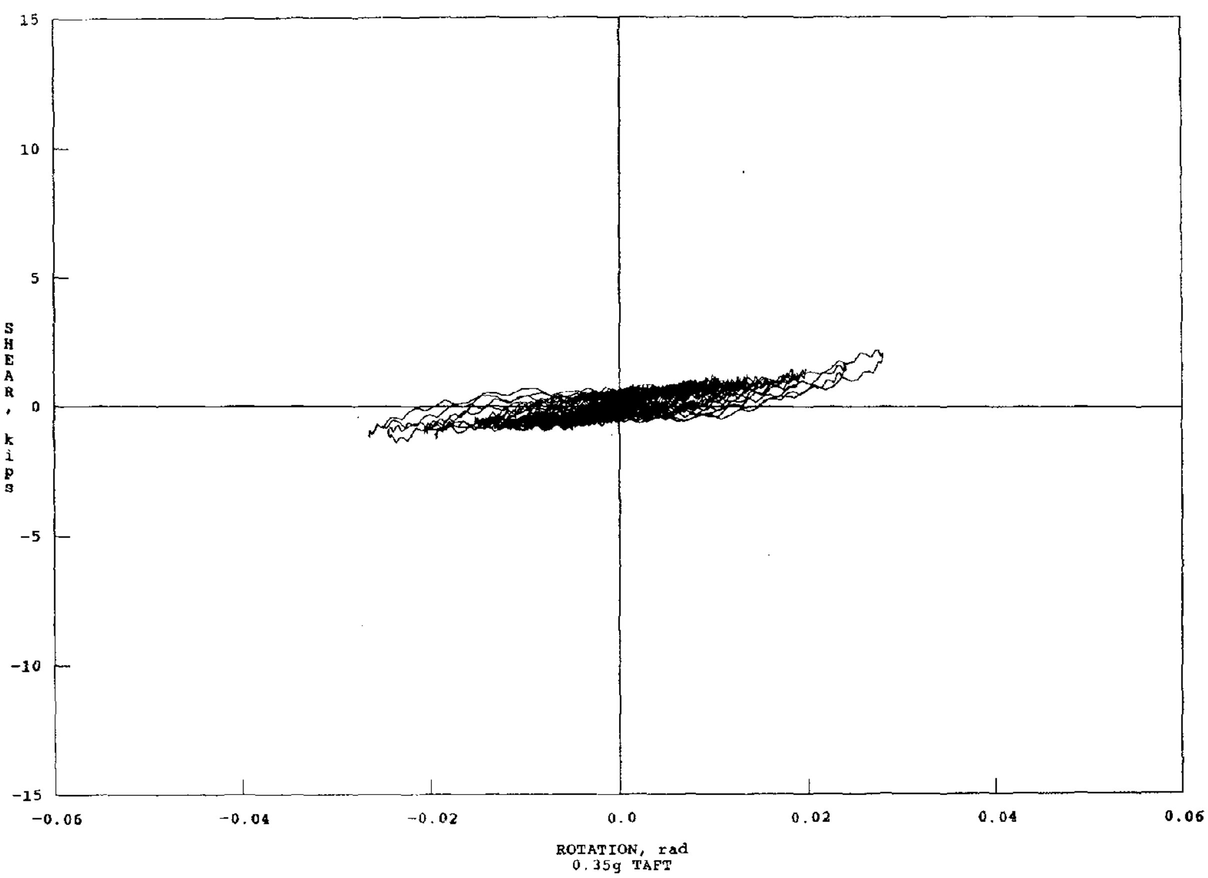

FLEXIBLE FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.15g TAFT


FLEXIBLE FRAME
0.07
0.06
0.05
0.02
0.01
0.0
FOURIER AMPLITUDE
0
10
20
30
40
50
60
70
80
90
100
FREQUENCY, hertz
0.15g TAFT

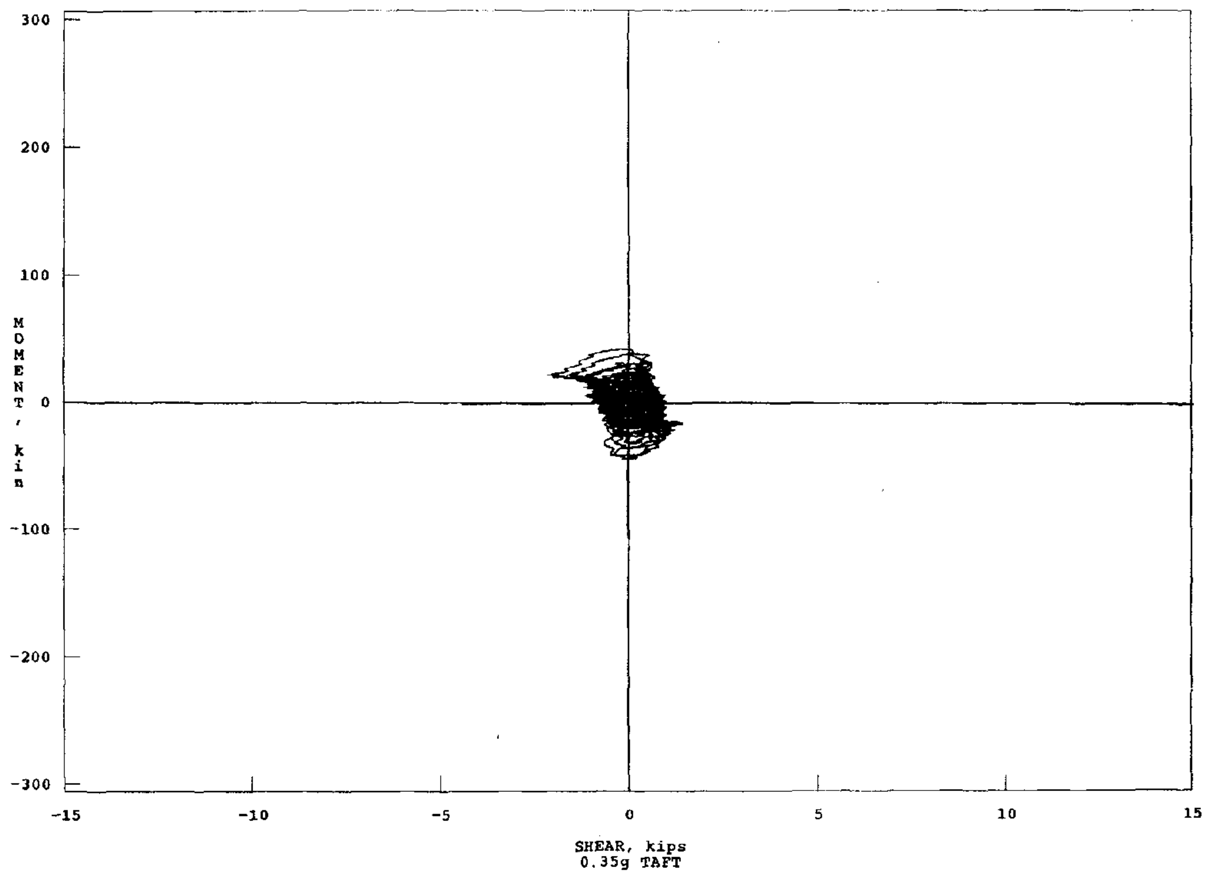
FLEXIBLE FRAME



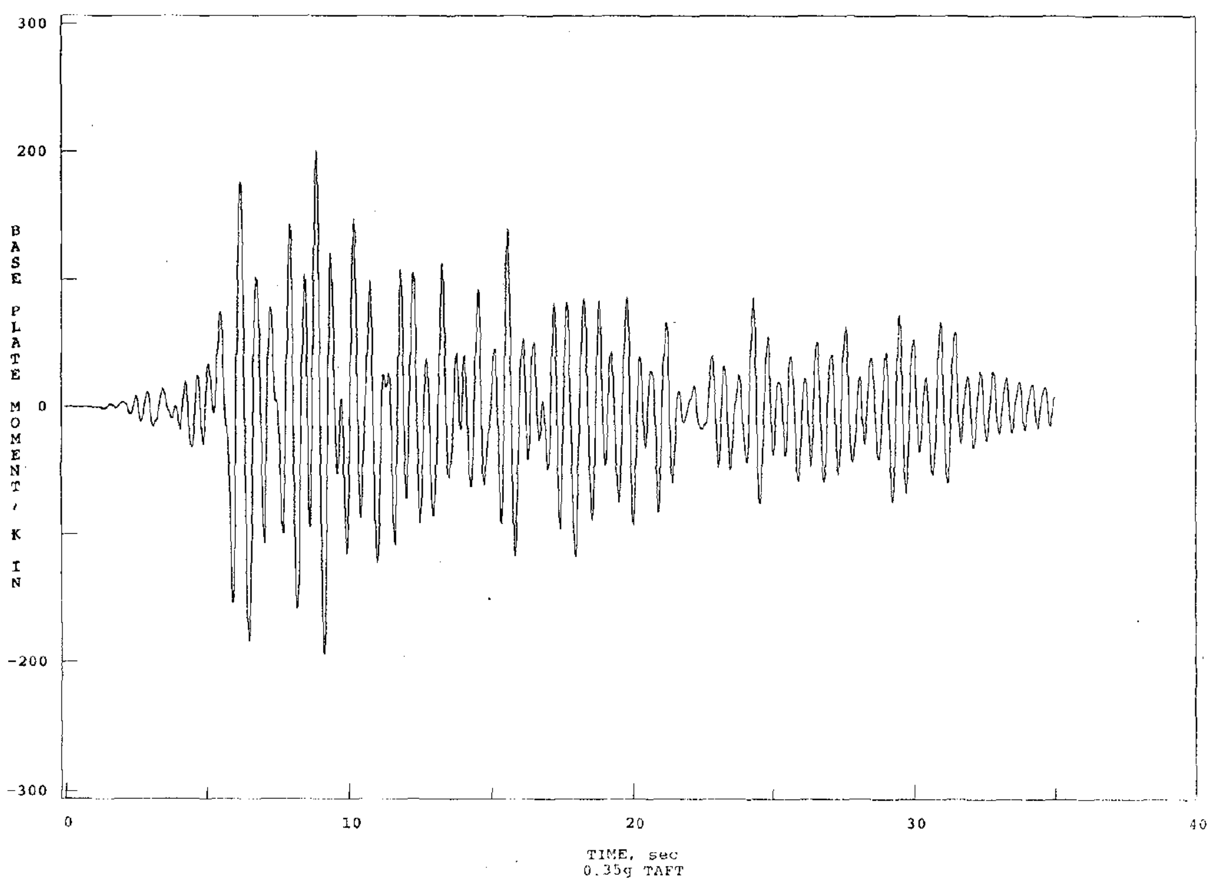
FLEXIBLE FRAME



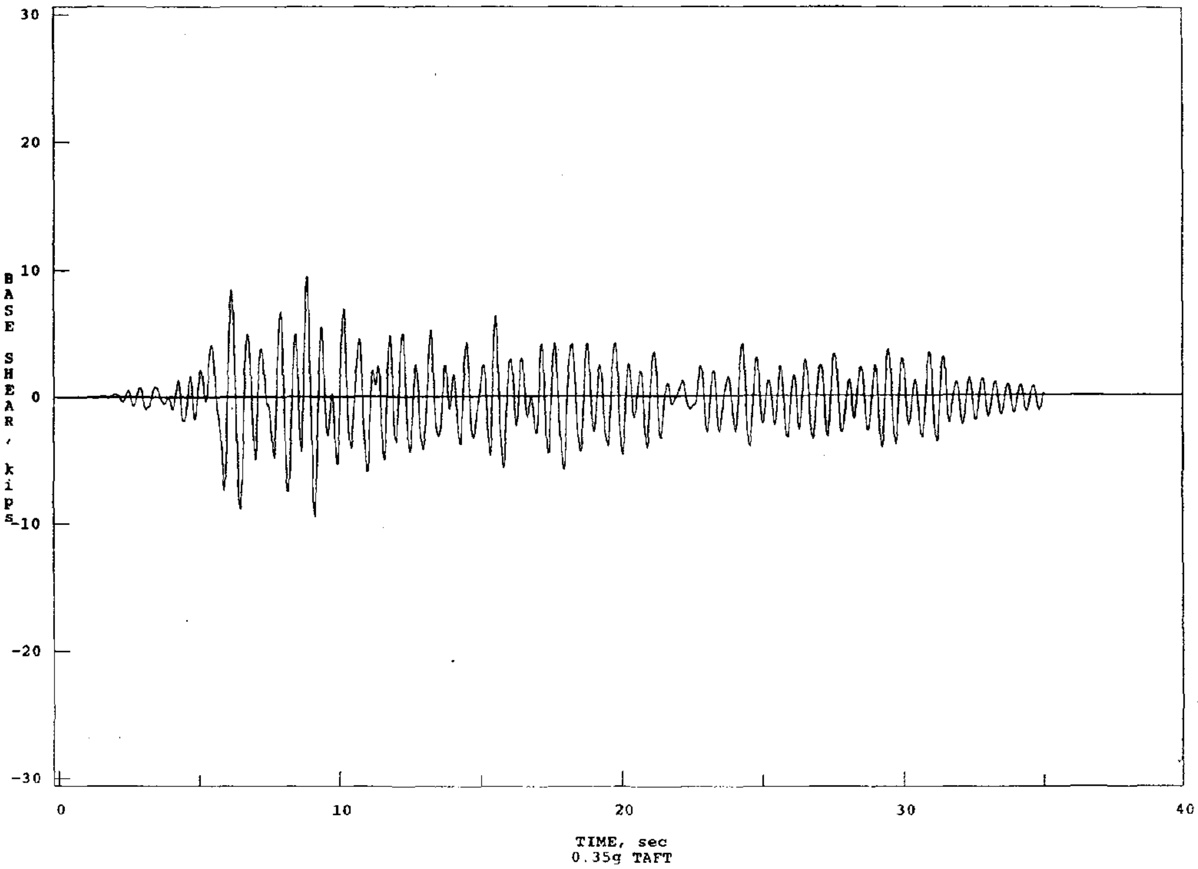
FLEXIBLE FRAME



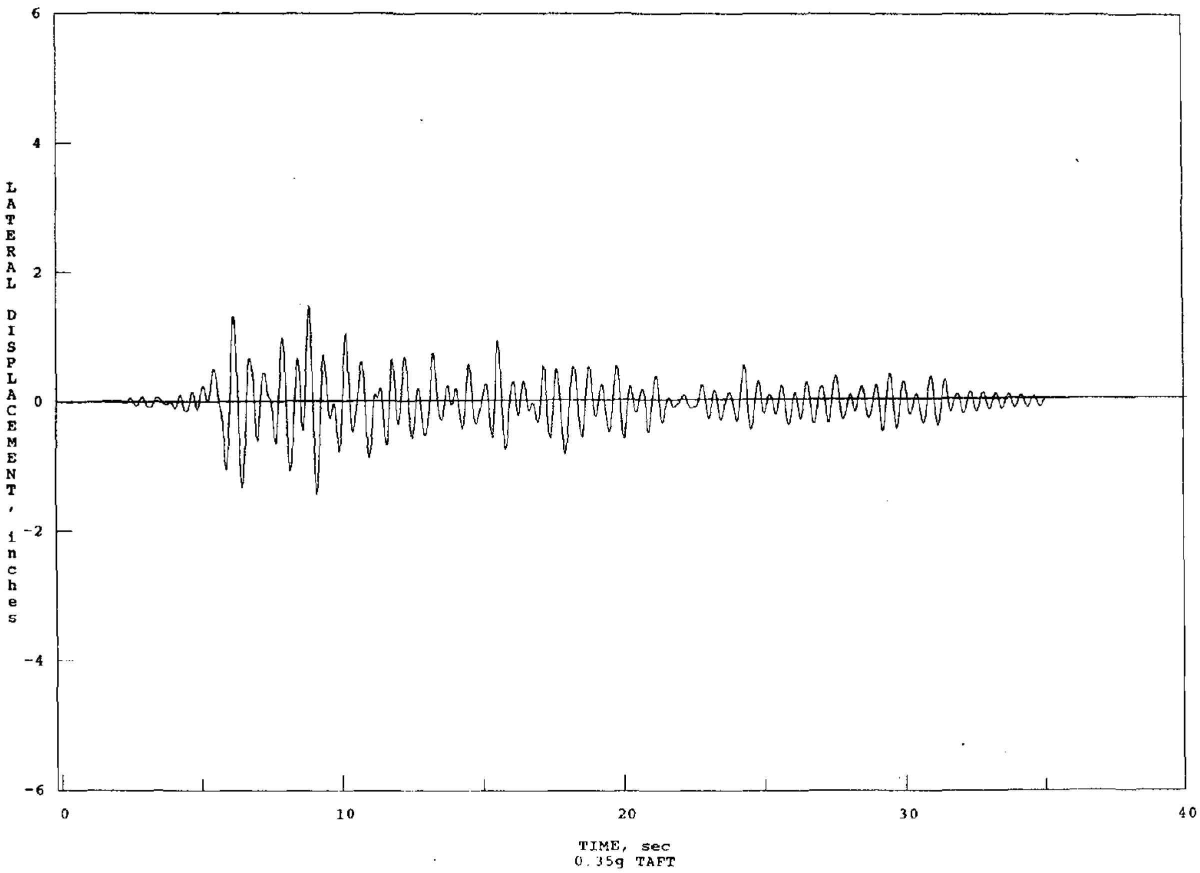
FLEXIBLE FRAME

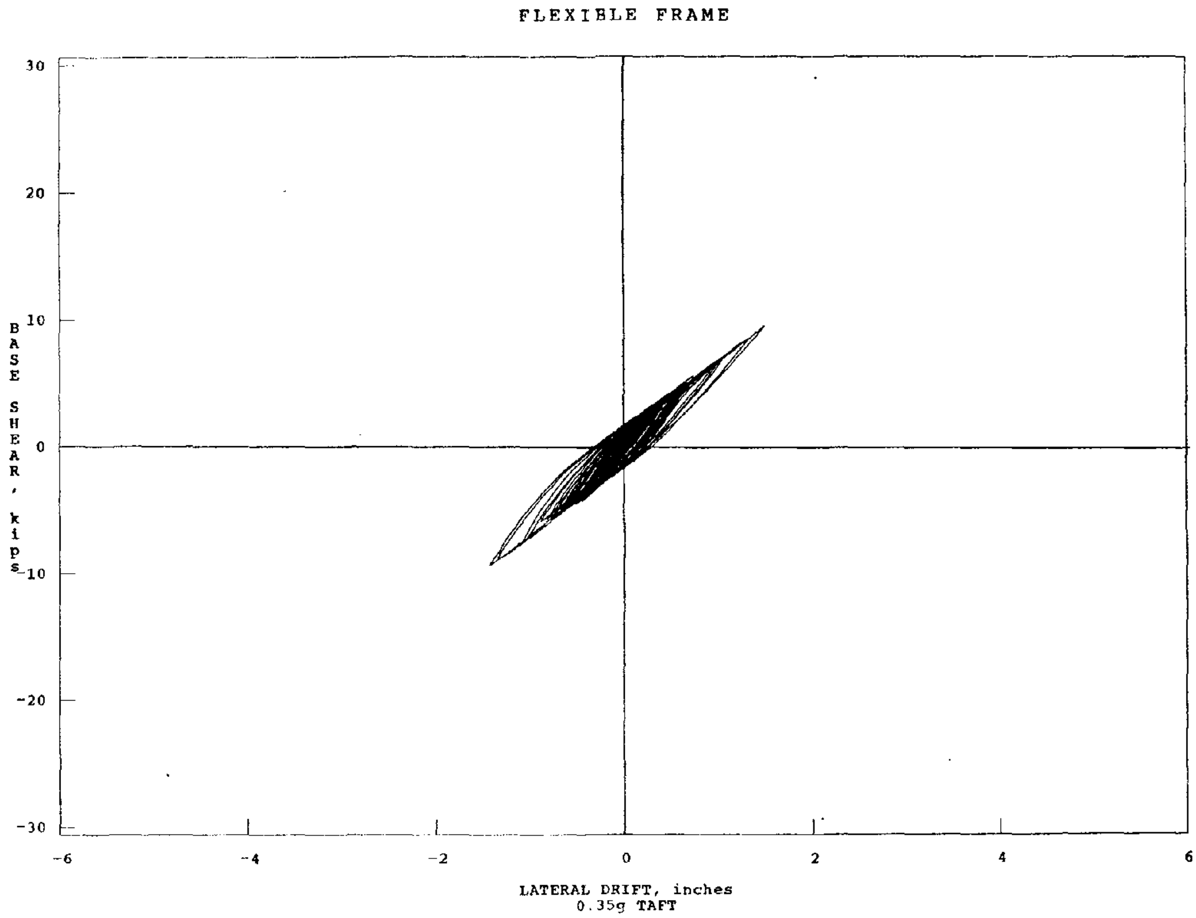


FLEXIBLE FRAME

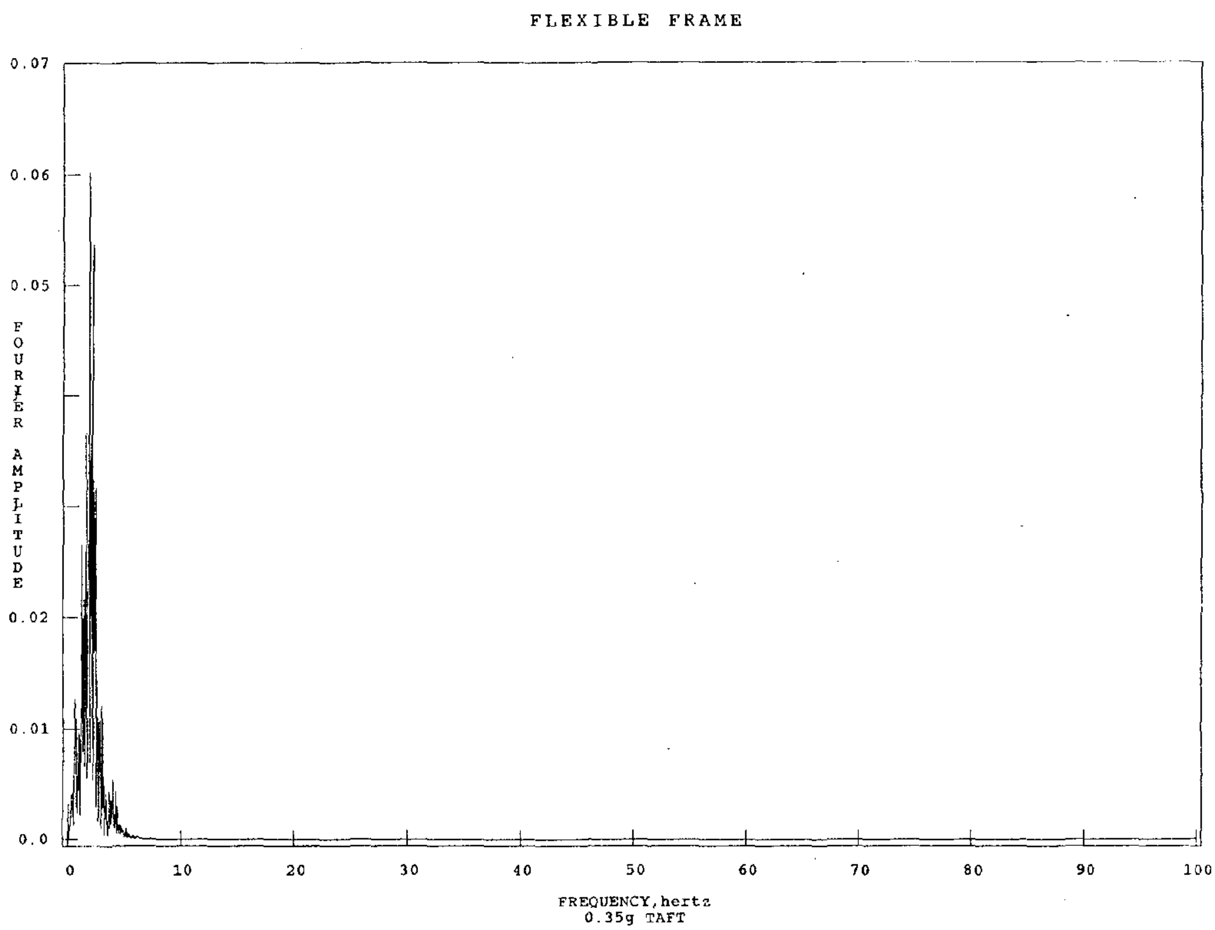


FLEXIBLE FRAME



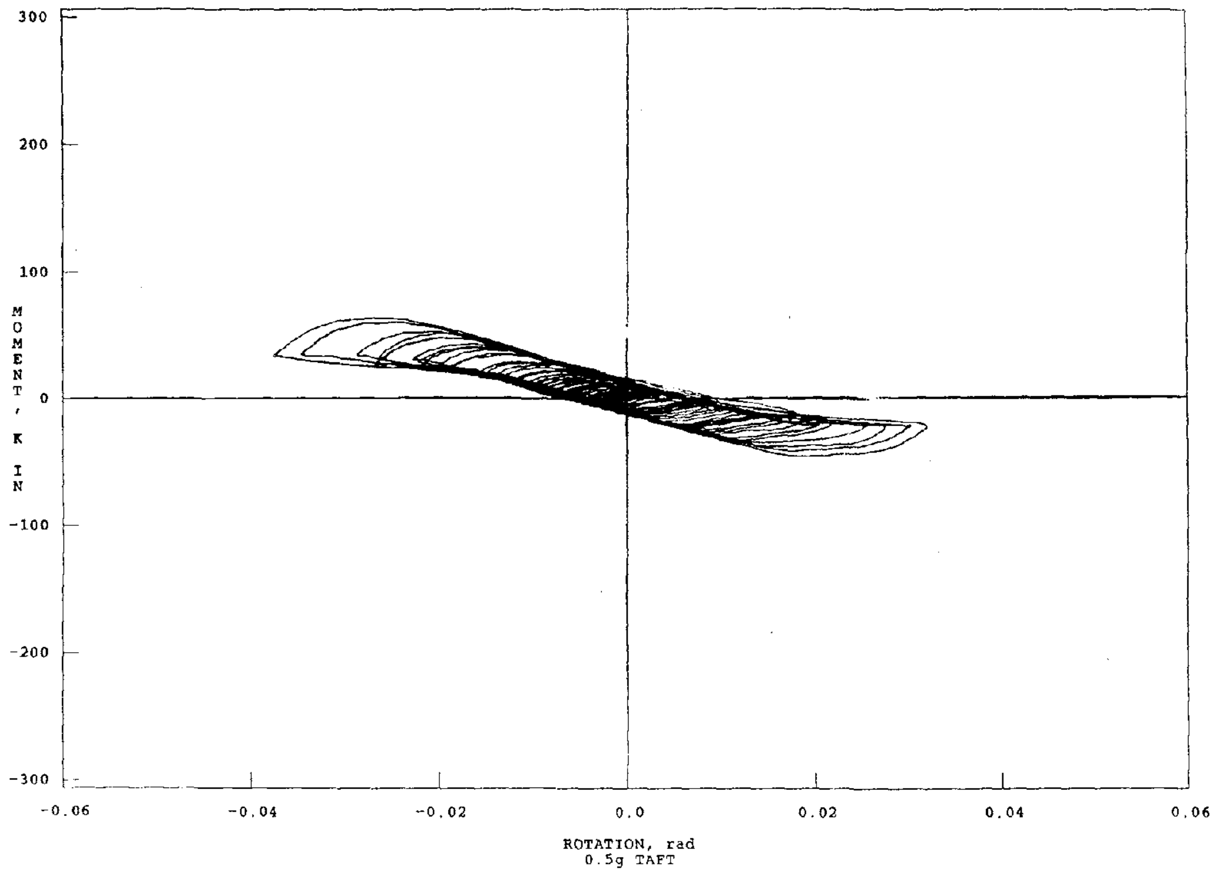


FLEXIBLE FRAME
BASE SHEAR, kips
30
20
10
0
-10
-20
-30
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.35g TAFT

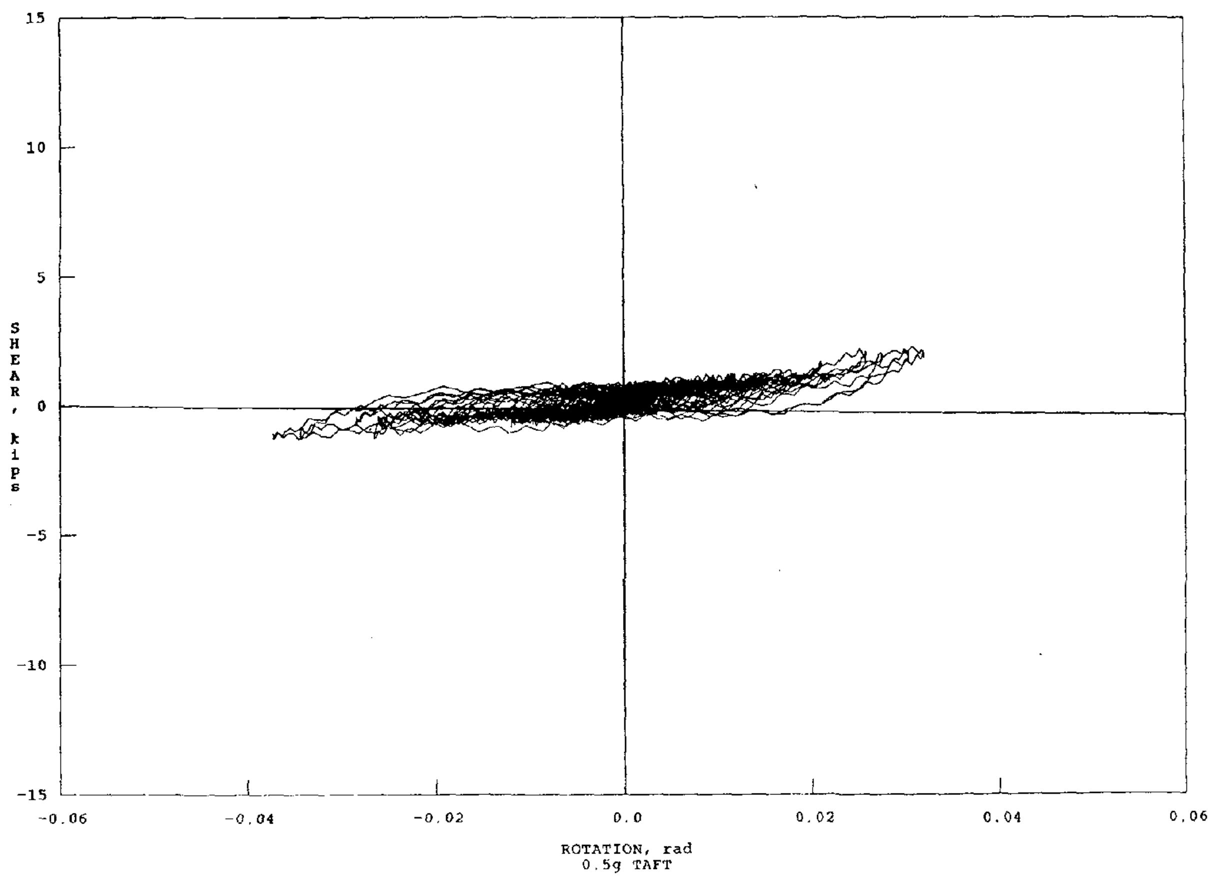


FLEXIBLE FRAME
FOURIER AMPLITUDE
0.07
0.06
0.05
0.02
0.01
0.0
0
10
20
30
40
50
60
70
80
90
100
FREQUENCY, hertz
0.35g TAFT

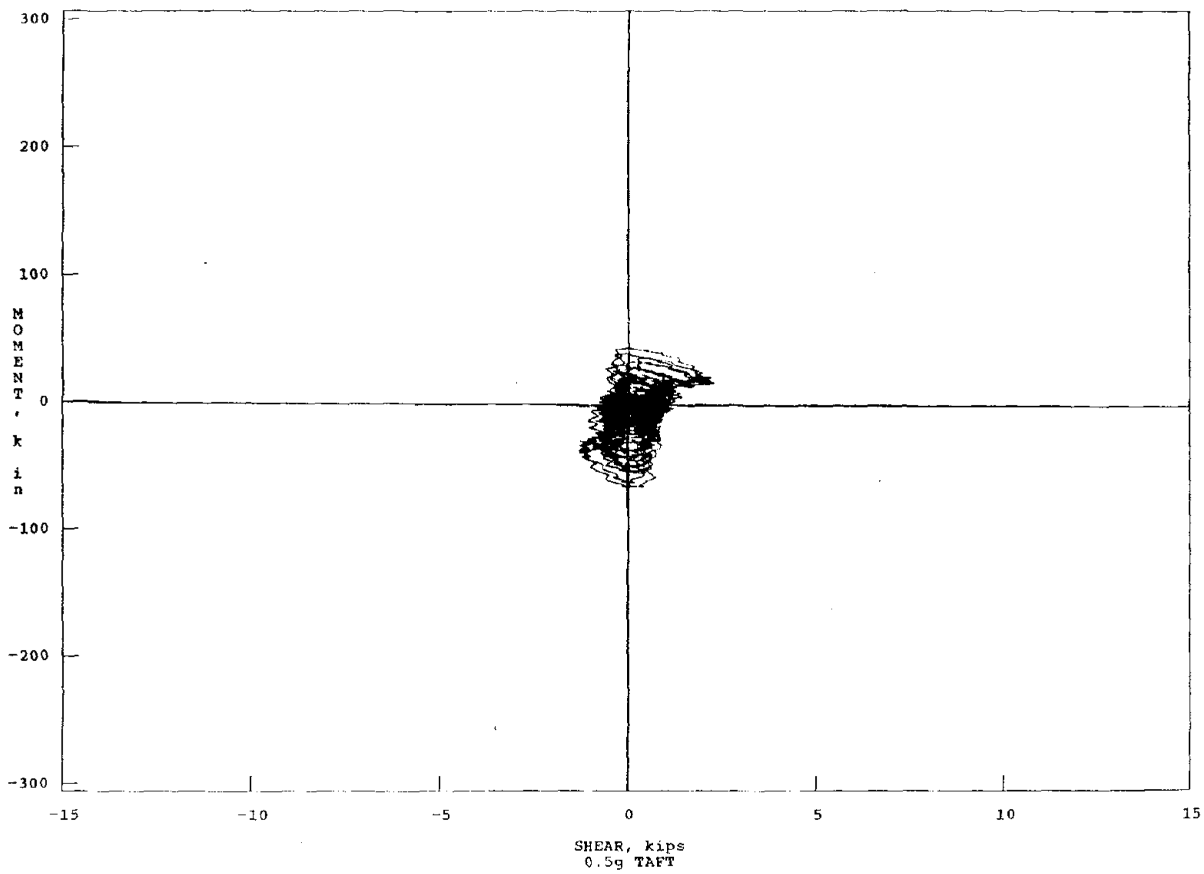
FLEXIBLE FRAME



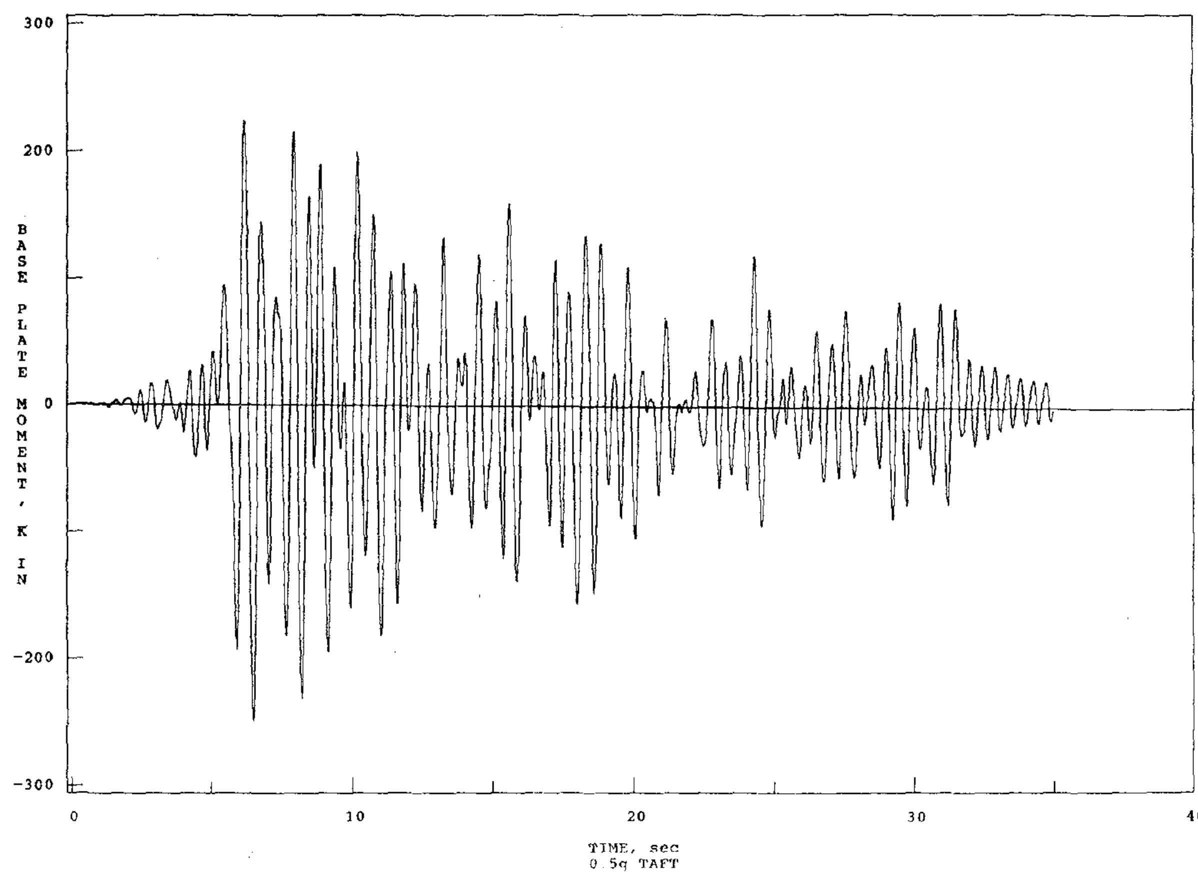
FLEXIBLE FRAME

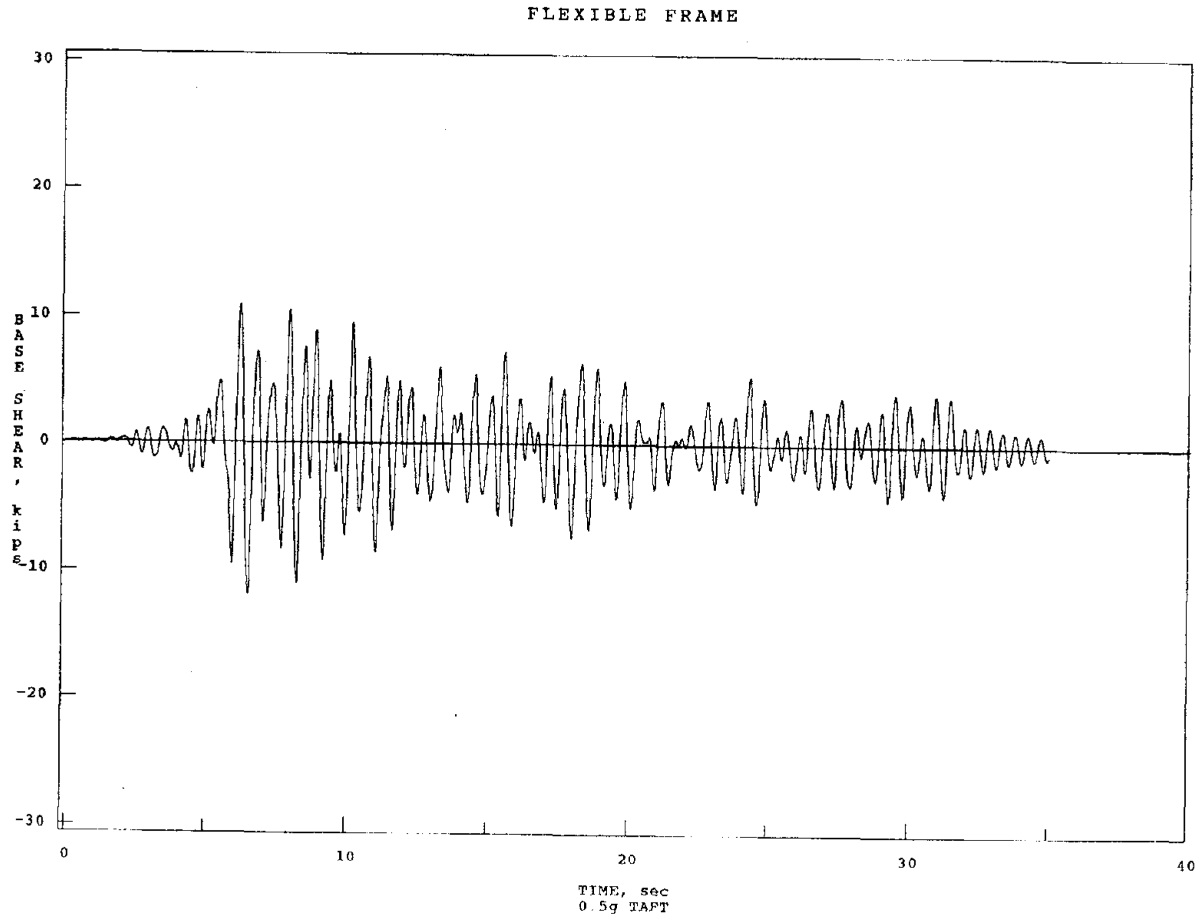


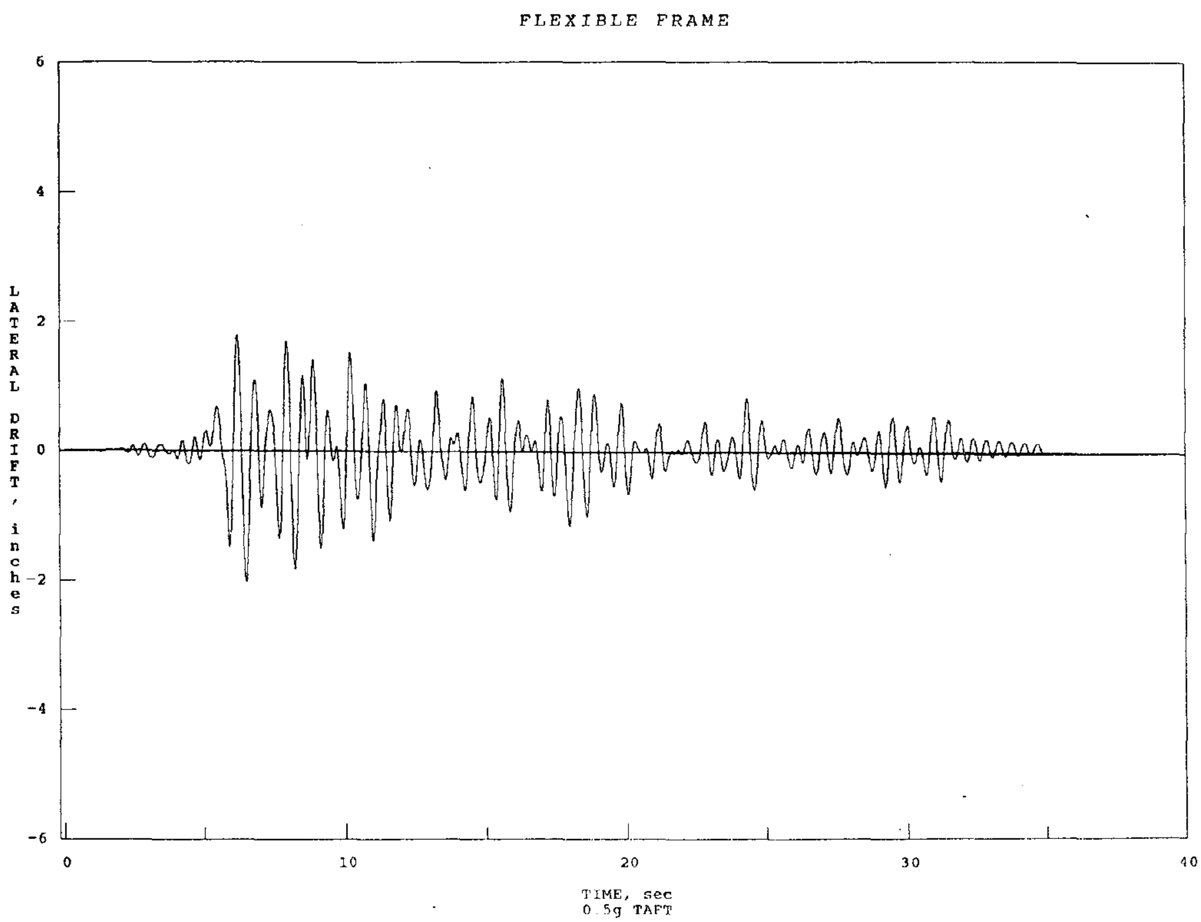
FLEXIBLE FRAME



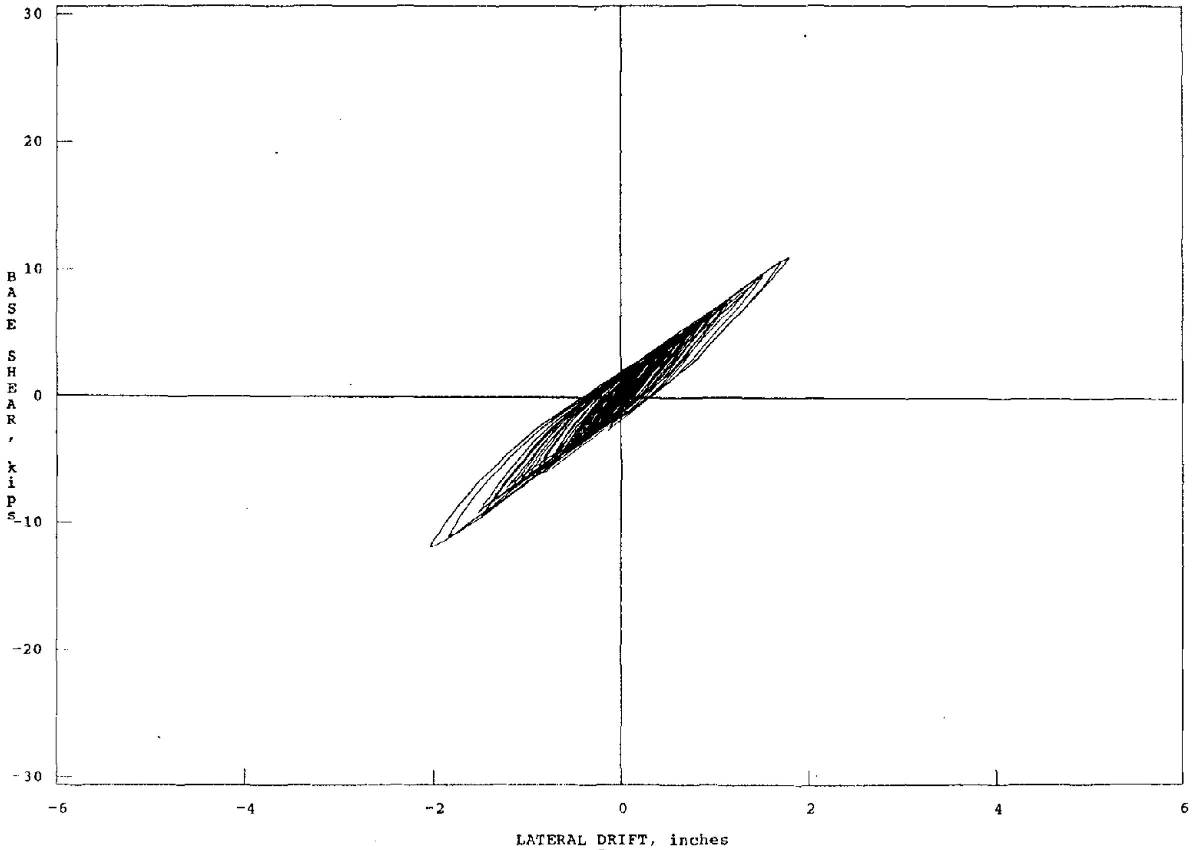
FLEXIBLE FRAME




FLEXIBLE FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
0
10
20
30
40
TIME, sec
0.5g TAFT

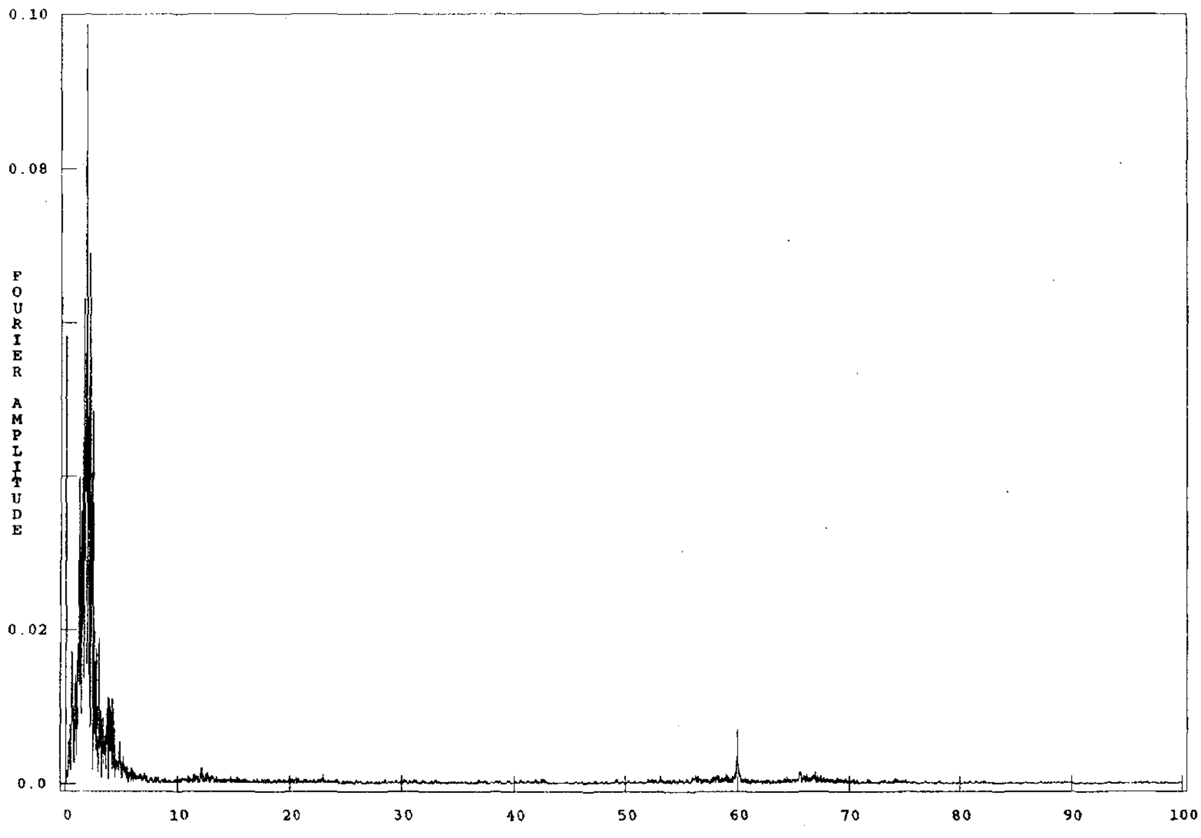

FLEXIBLE FRAME
6
4
2
0
-2
-4
-6
LATERAL DRIFT, inches
0
10
20
30
40
TIME, sec
0.5g TAFT

FLEXIBLE FRAME



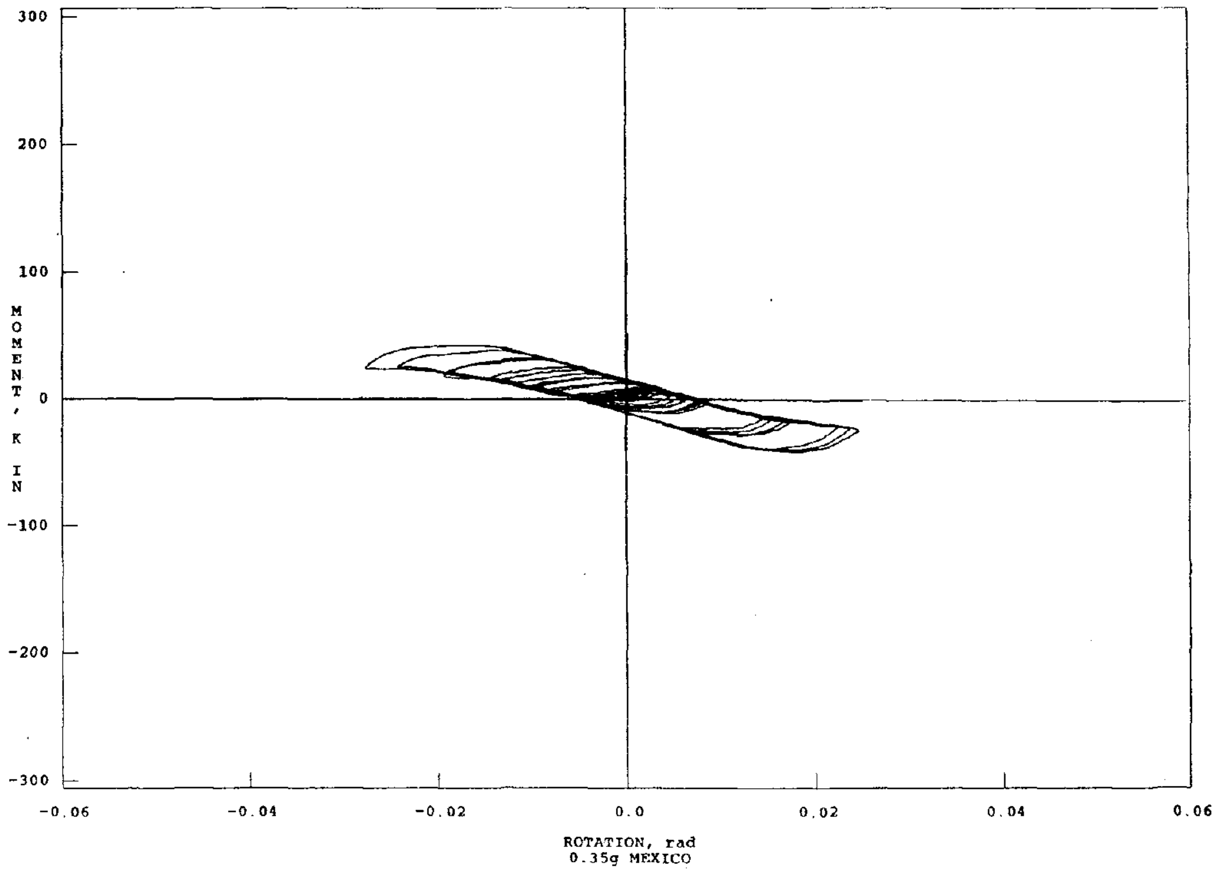
0.5g TAFT

FLEXIBLE FRAME



0.5g TAFT

FLEXIBLE FRAME



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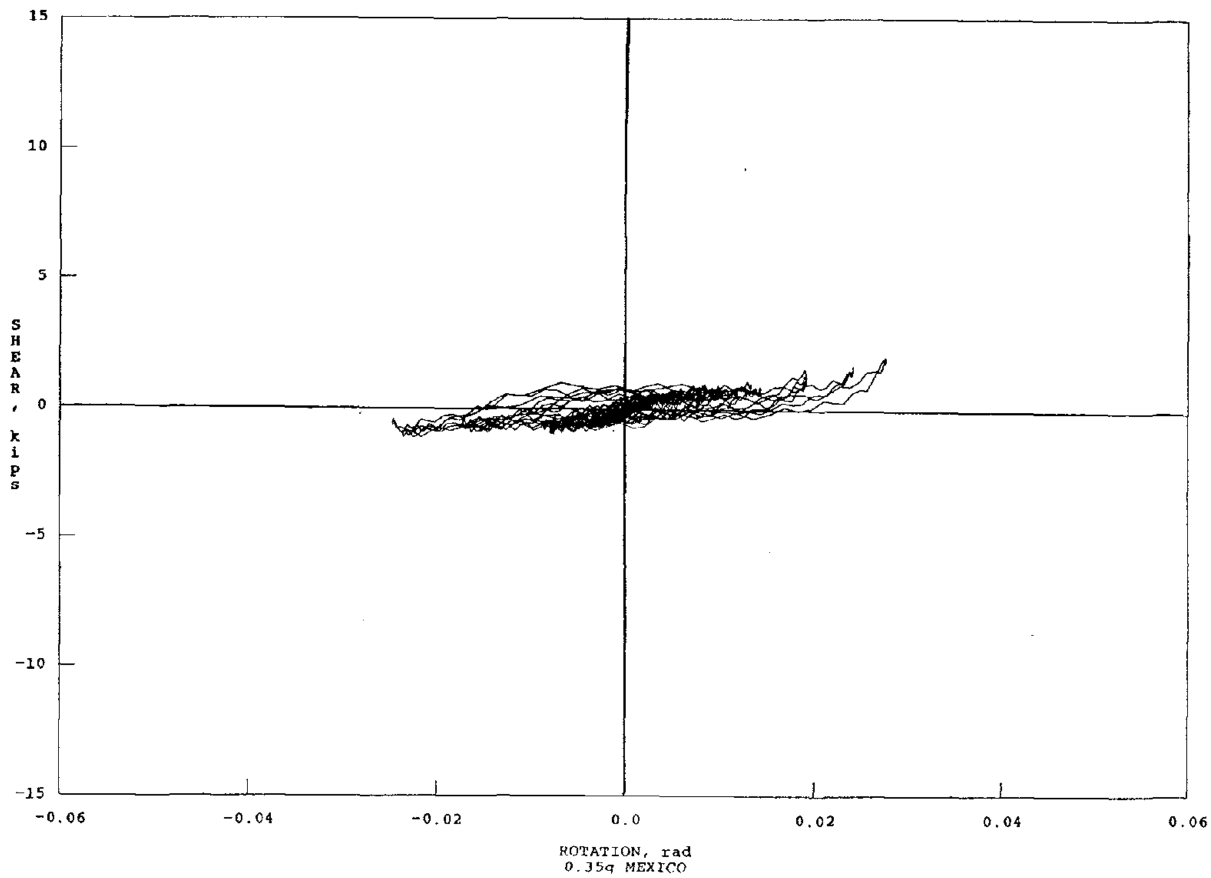
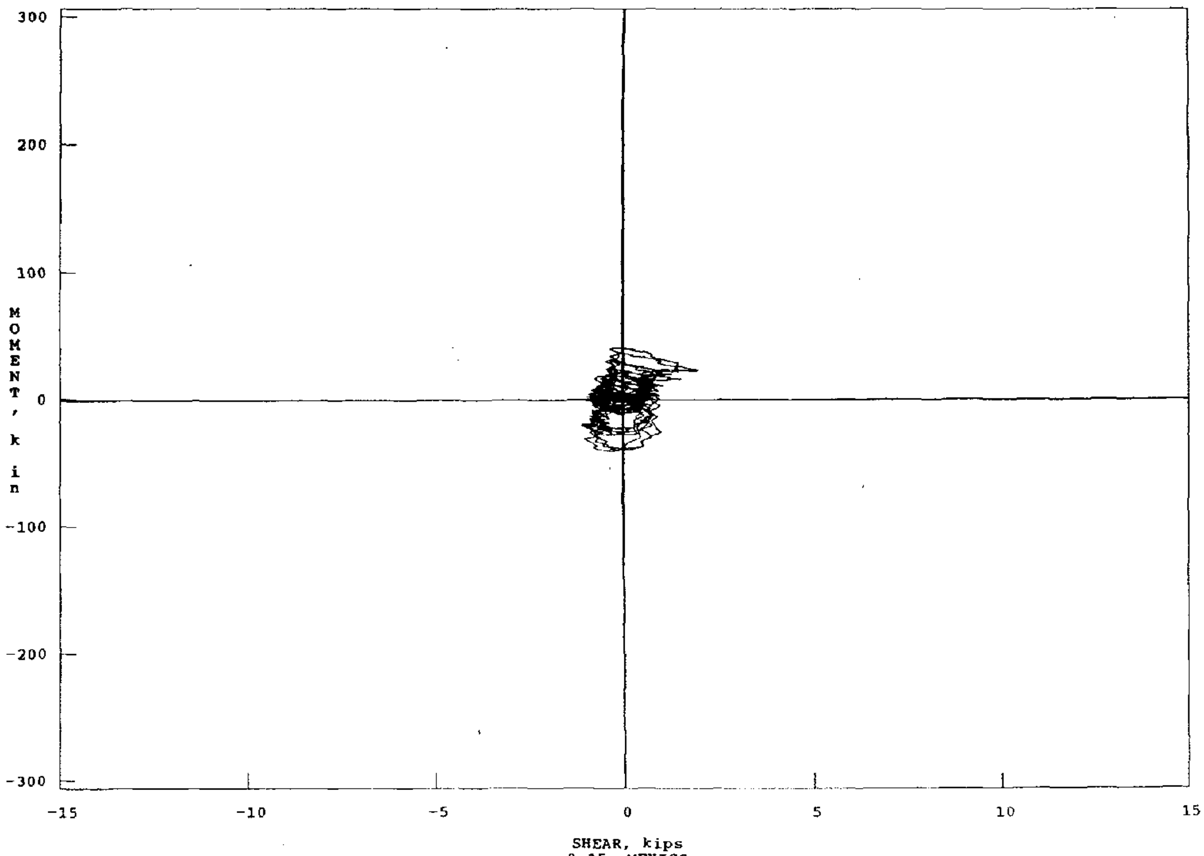
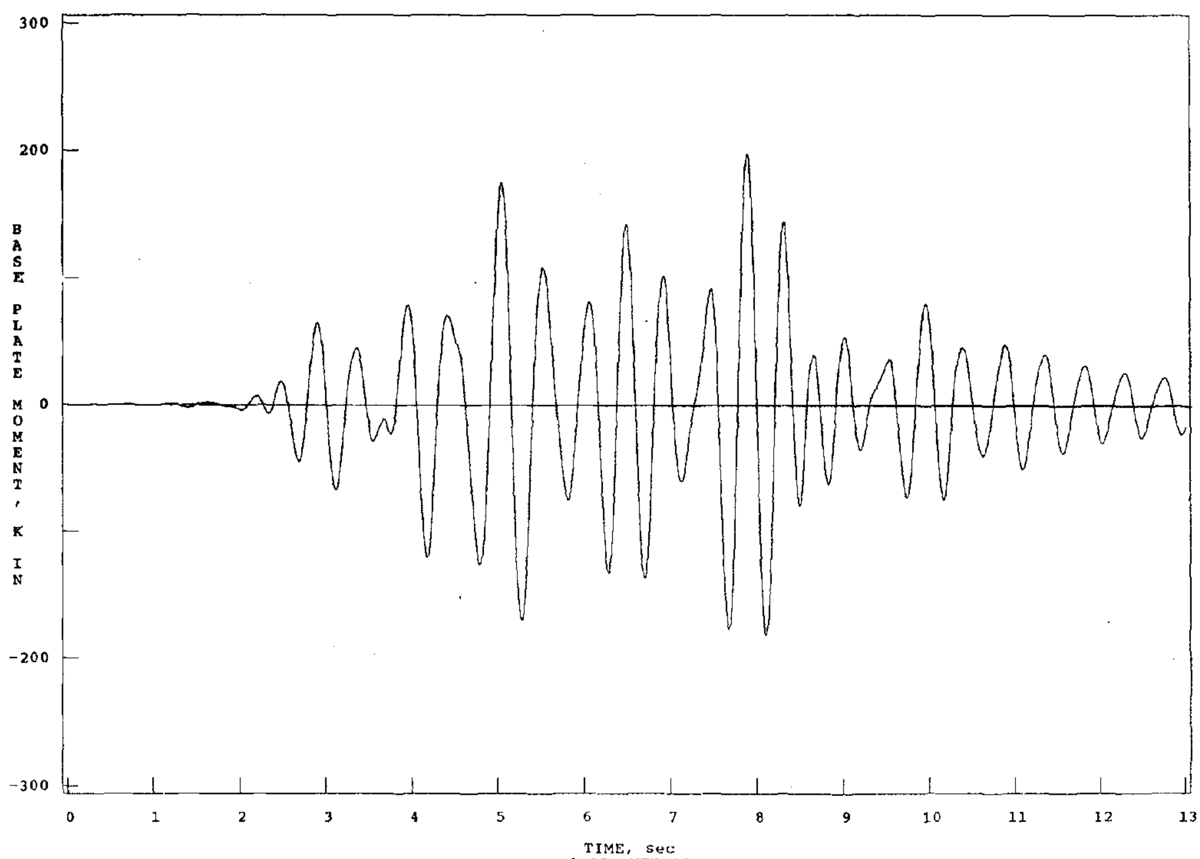


FLEXIBLE FRAME



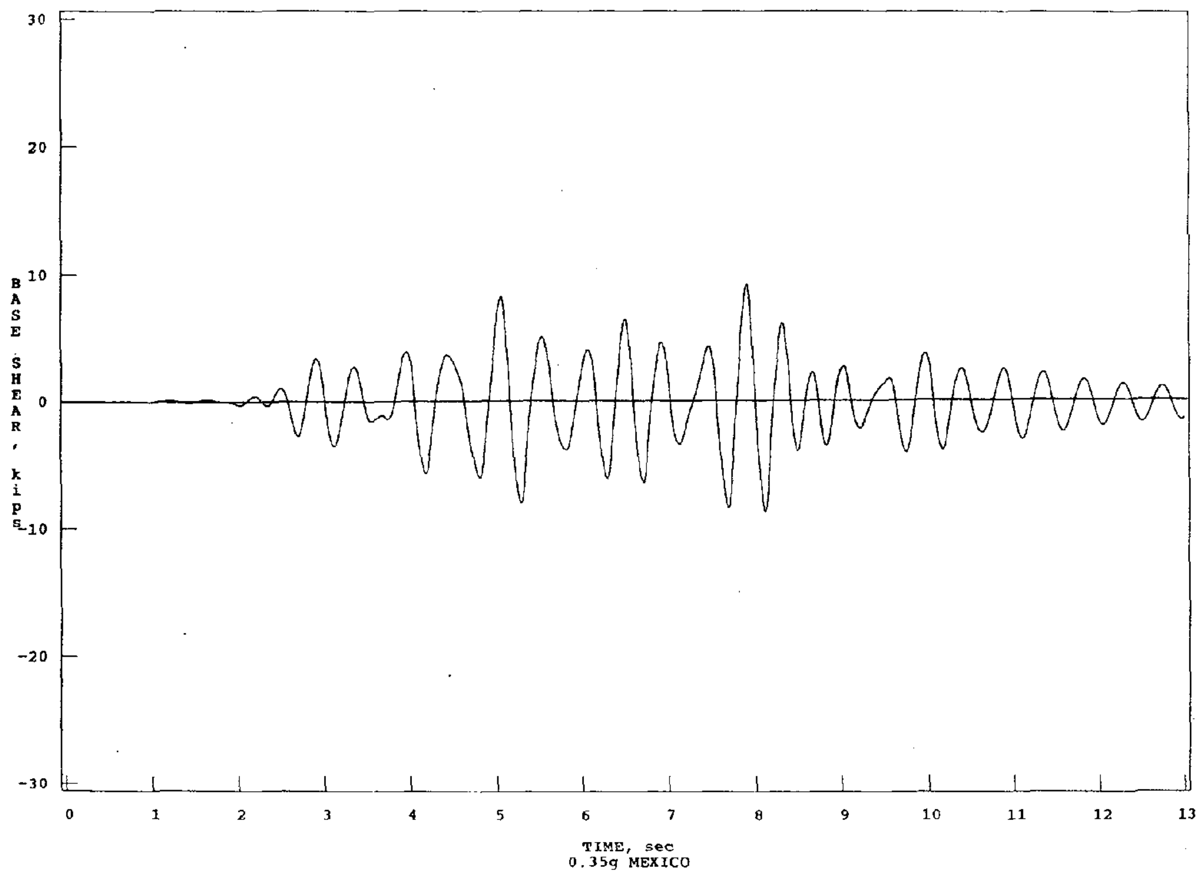
0.35g MEXICO

FLEXIBLE FRAME

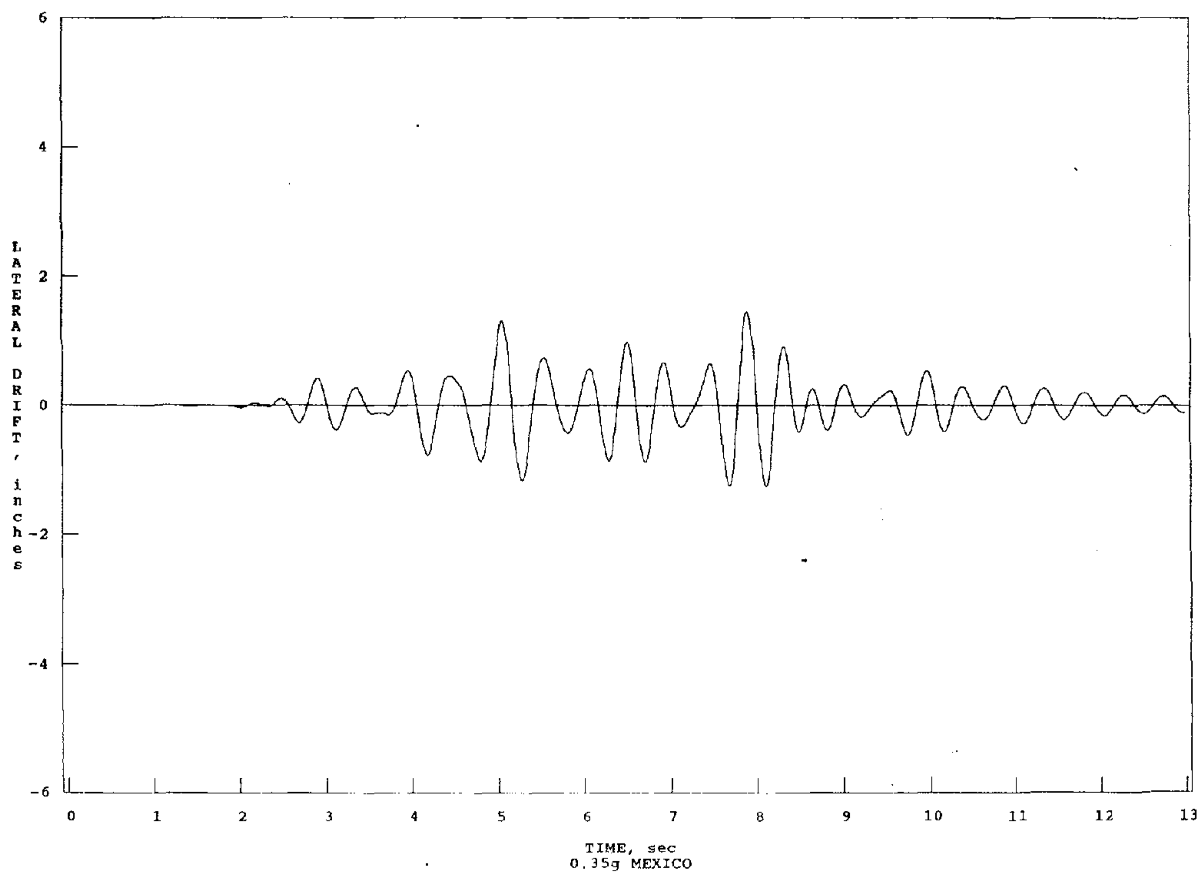


0.35g MEXICO

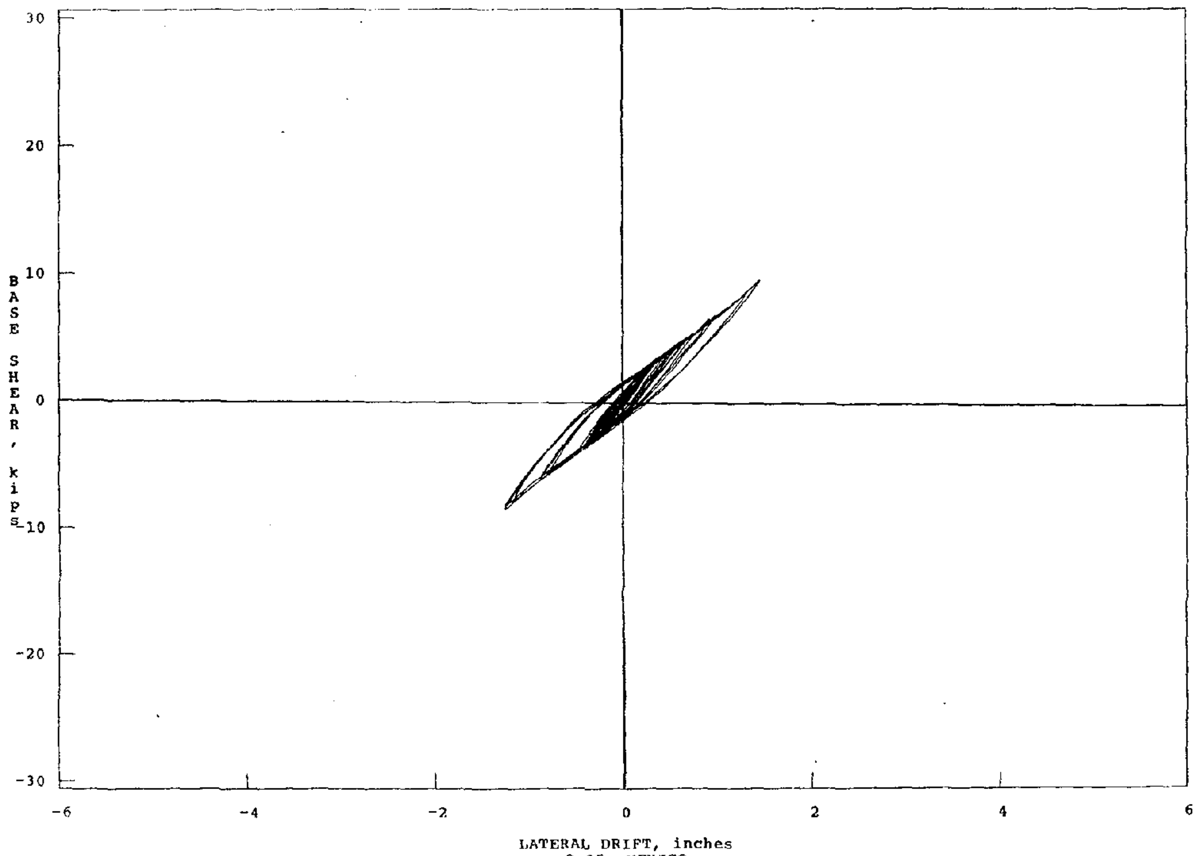
FLEXIBLE FRAME



FLEXIBLE FRAME

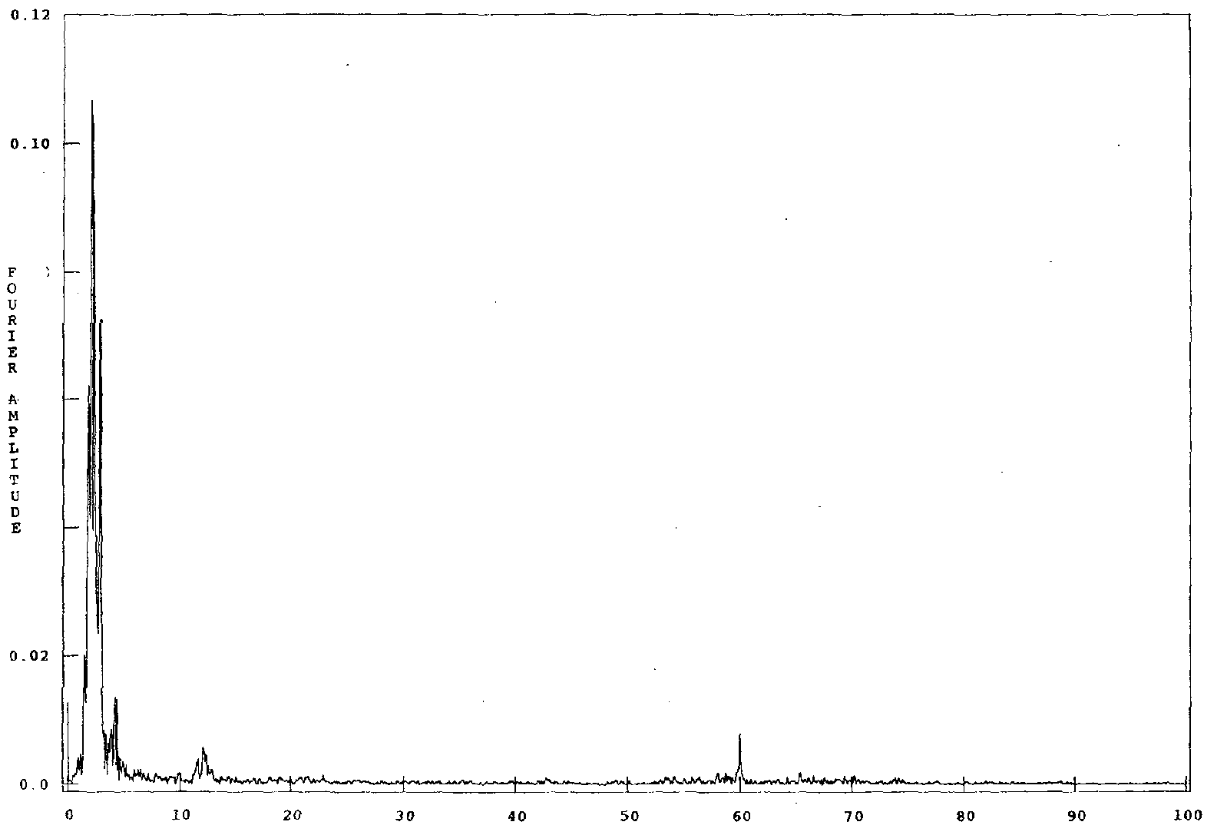


FLEXIBLE FRAME



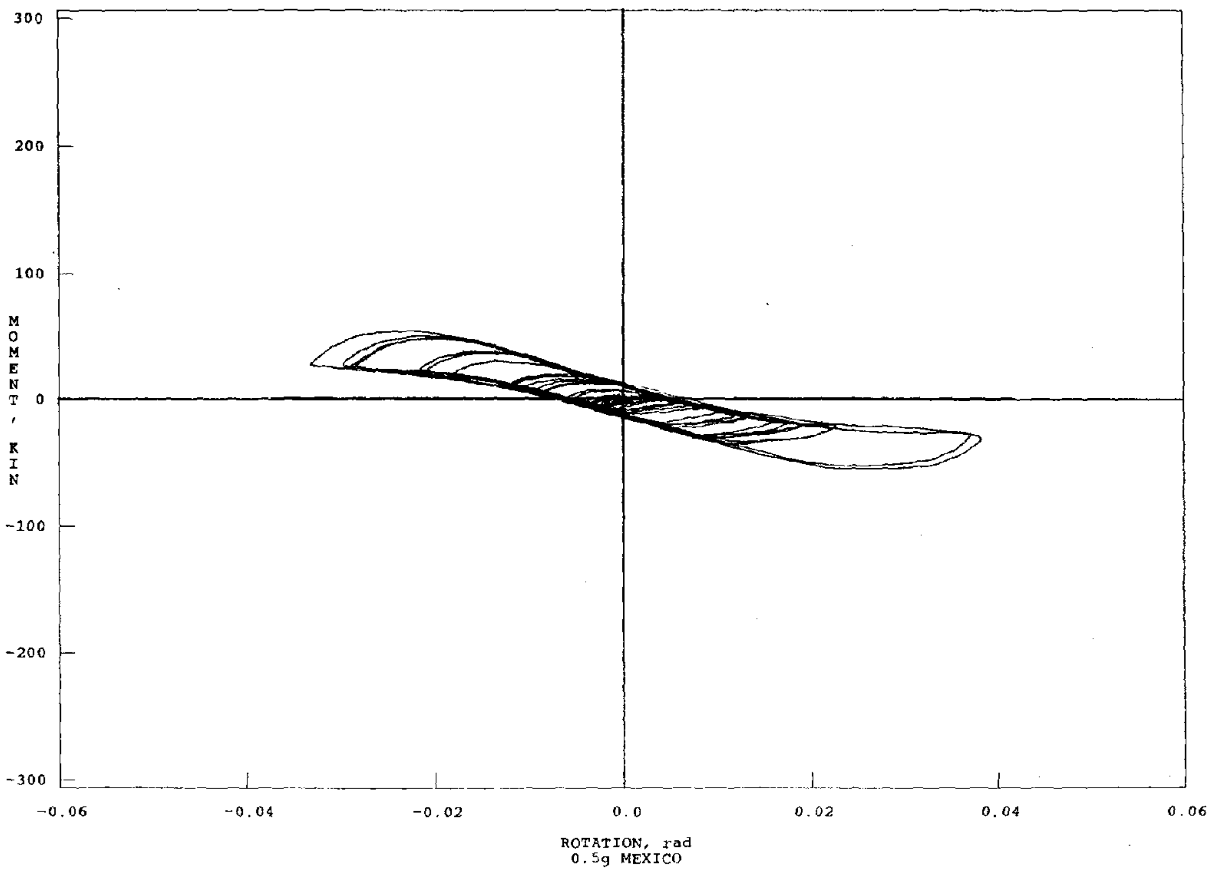
0.35g MEXICO

FLEXIBLE FRAME

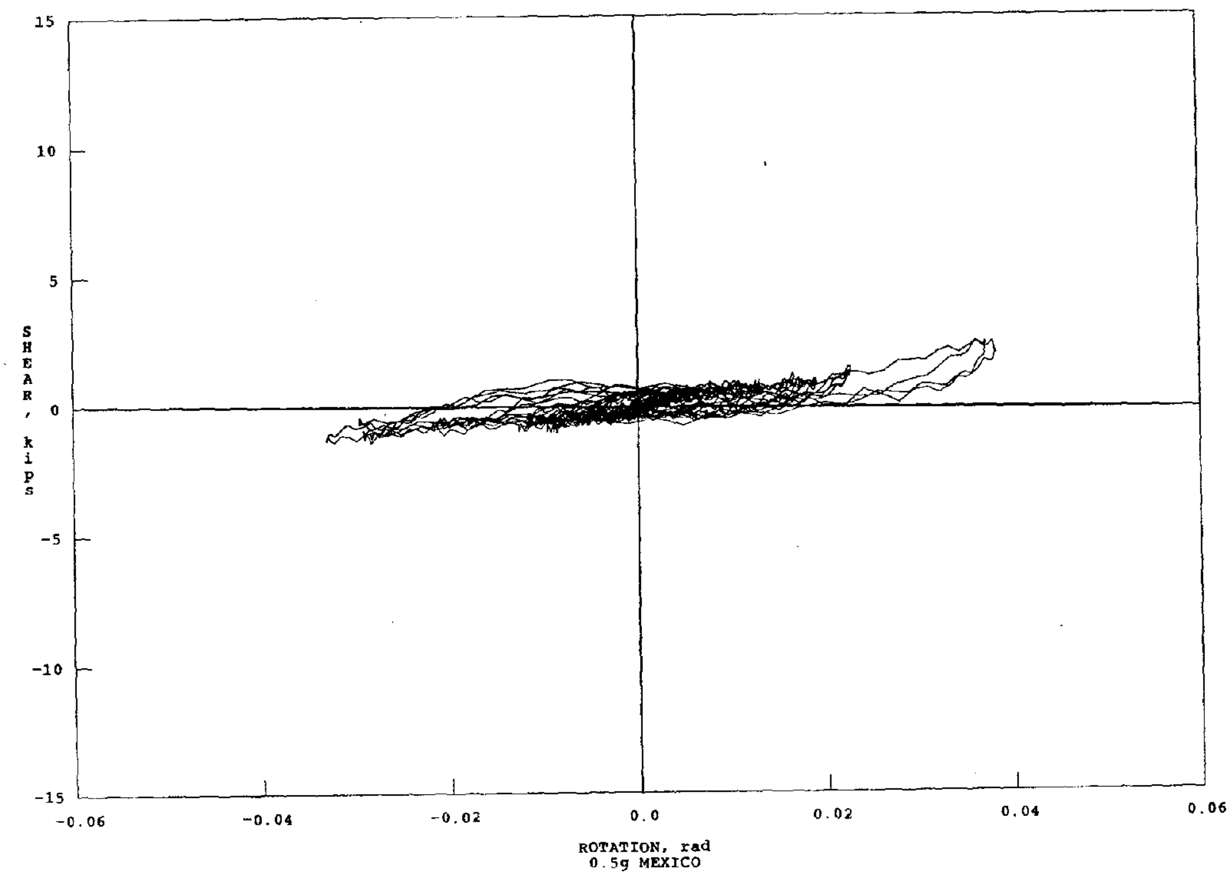


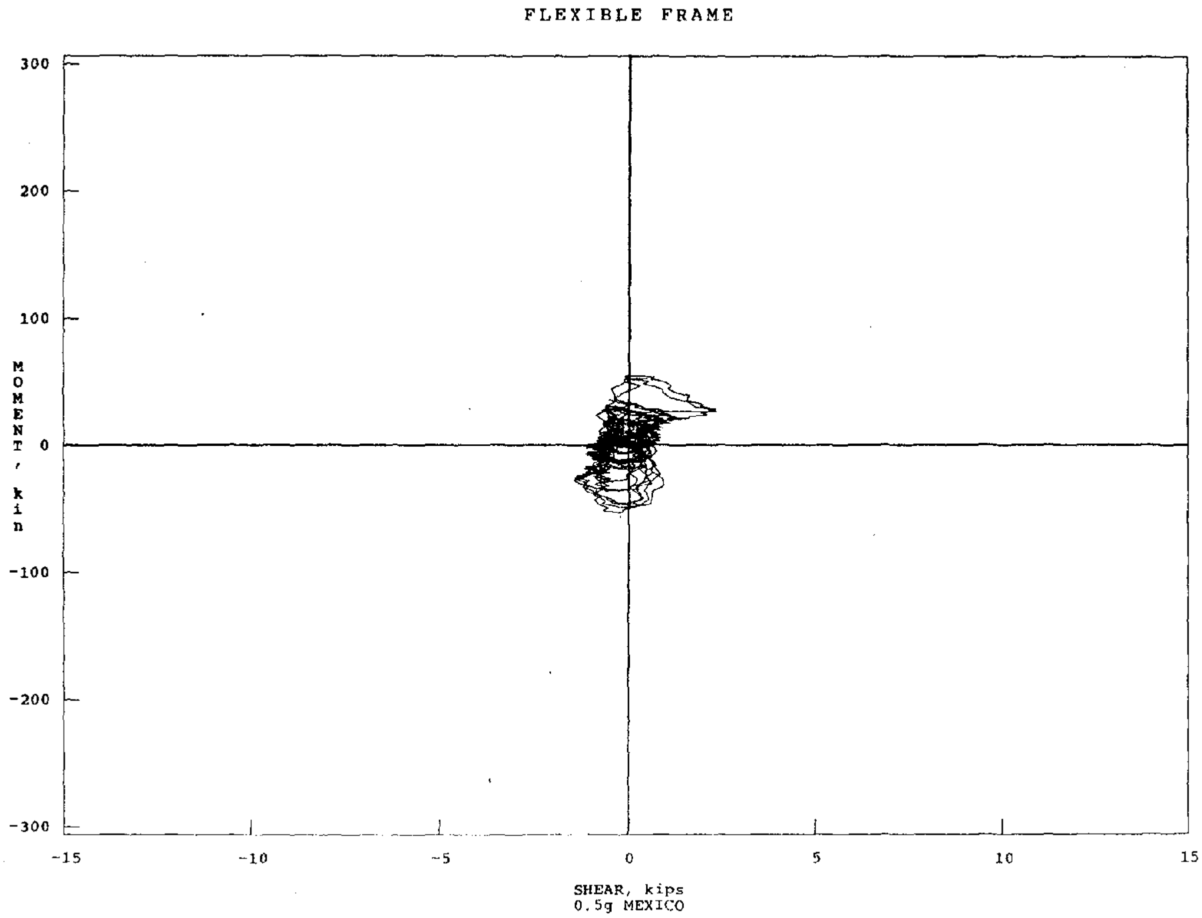
0.35g MEXICO

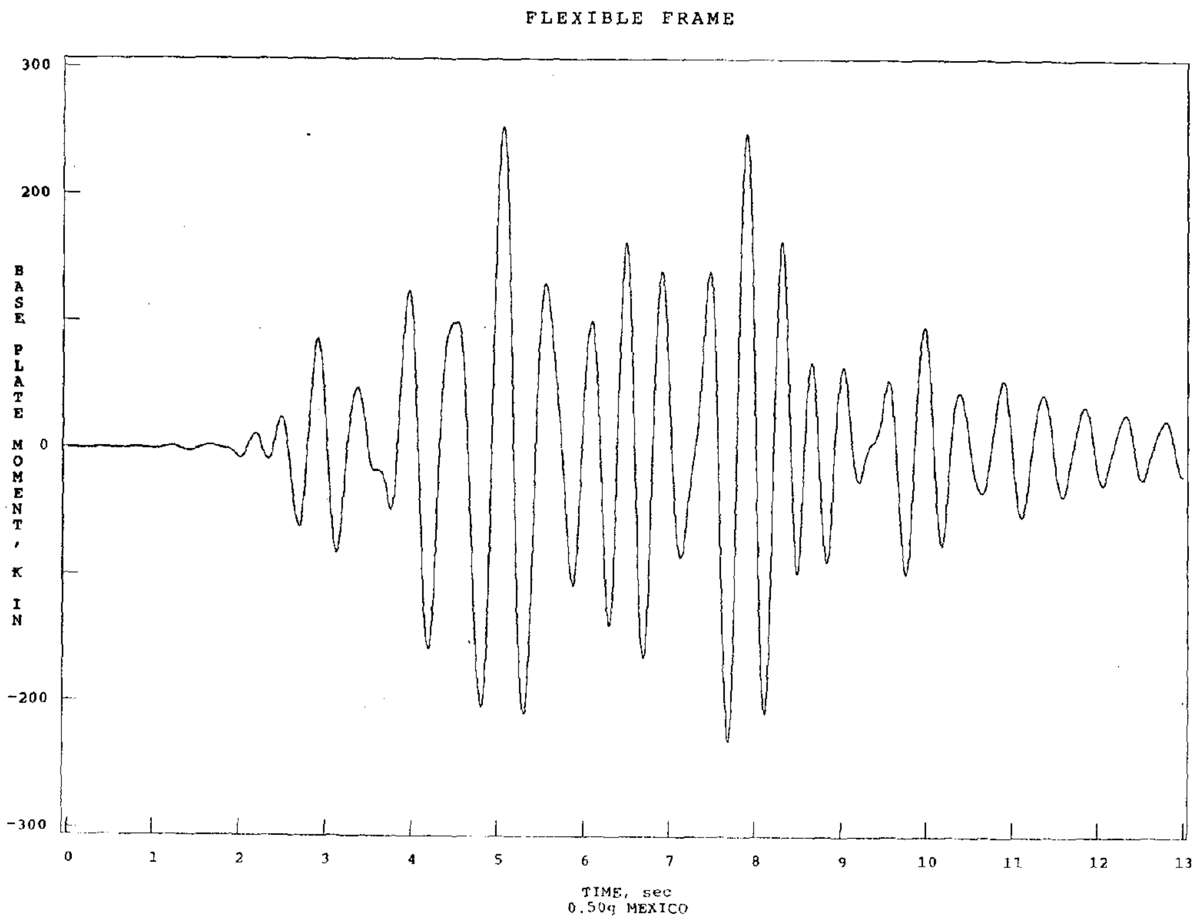
FLEXIBLE FRAME



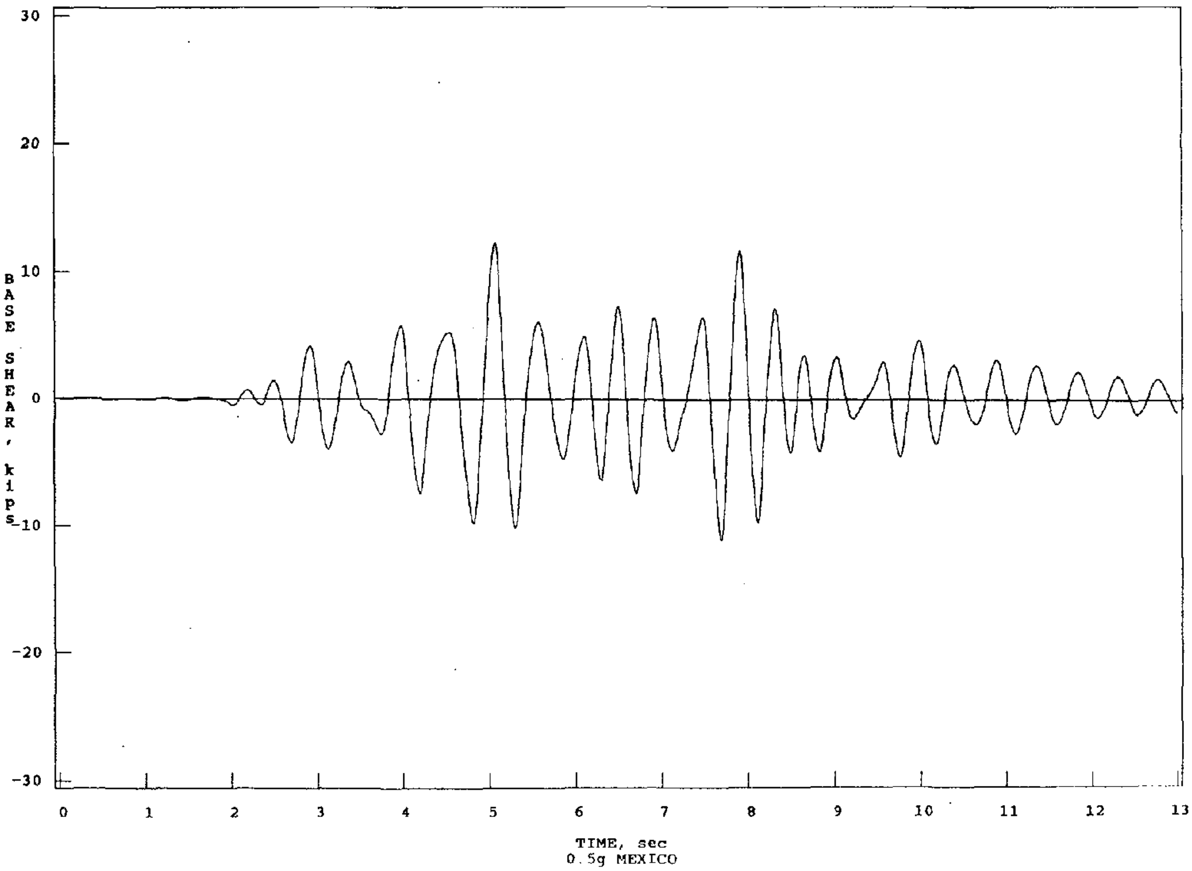
FLEXIBLE FRAME



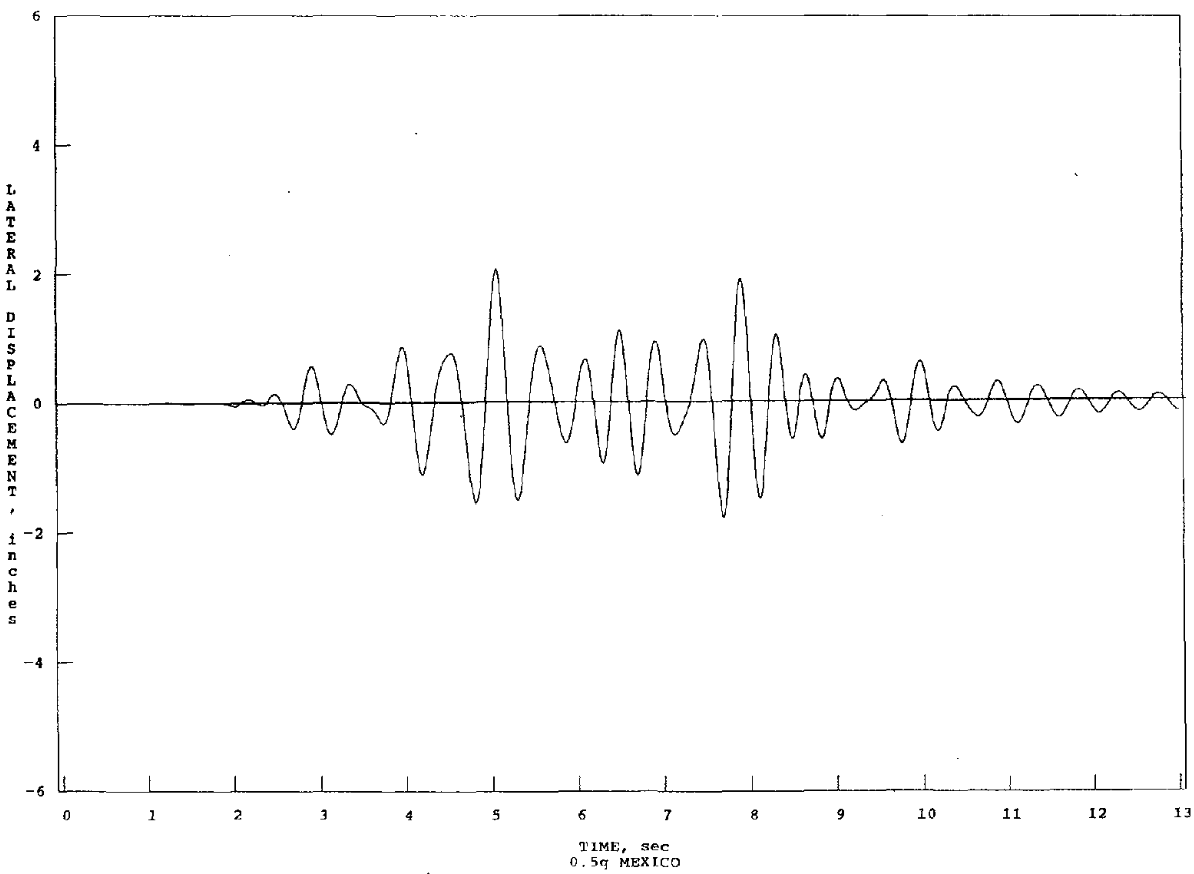

FLEXIBLE FRAME
300
200
100
0
-100
-200
-300
MOMENT, kin
-15
-10
-5
0
5
10
15
SHEAR, kips
0.5g MEXICO


FLEXIBLE FRAME
300
200
0
-200
-300
BASE PLATE MOMENT, K IN
0
1
2
3
4
5
6
7
8
9
10
11
12
13
TIME, sec
0.50g MEXICO

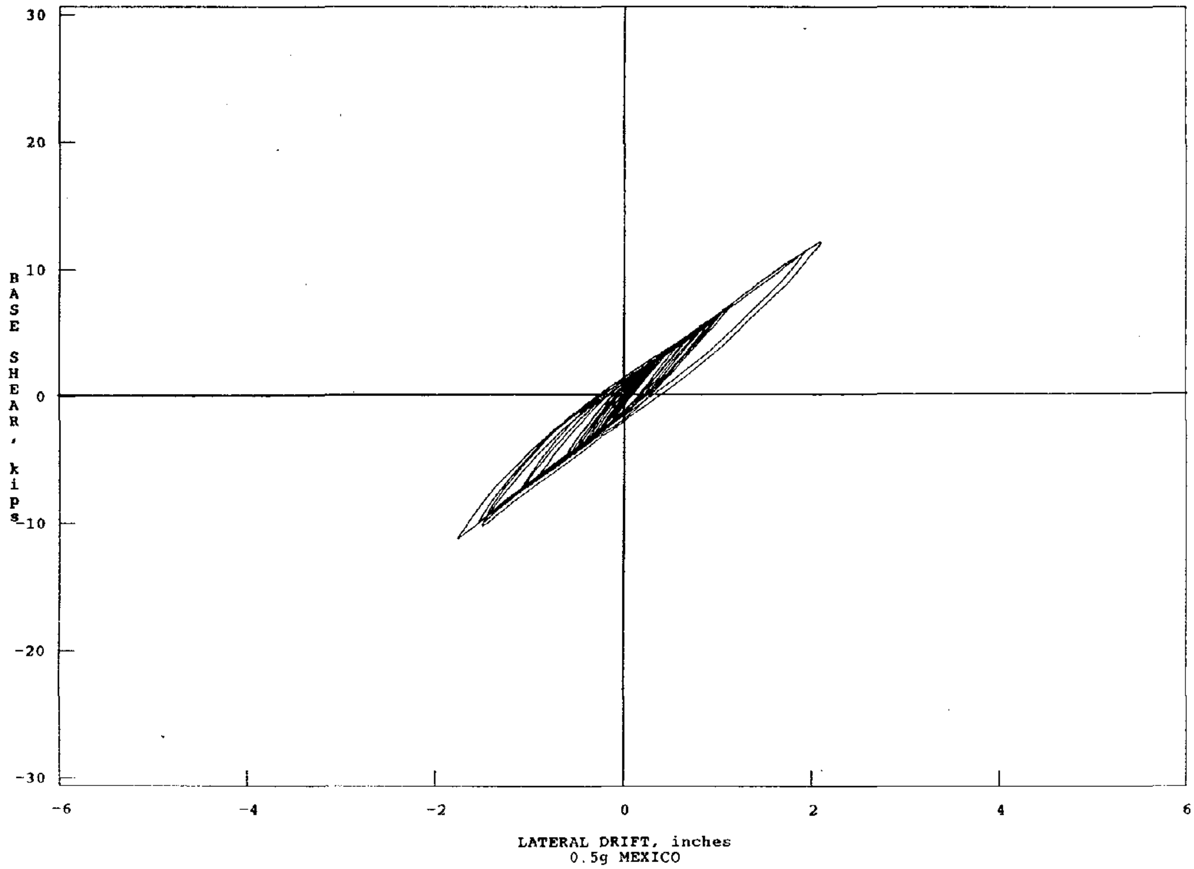
FLEXIBLE FRAME



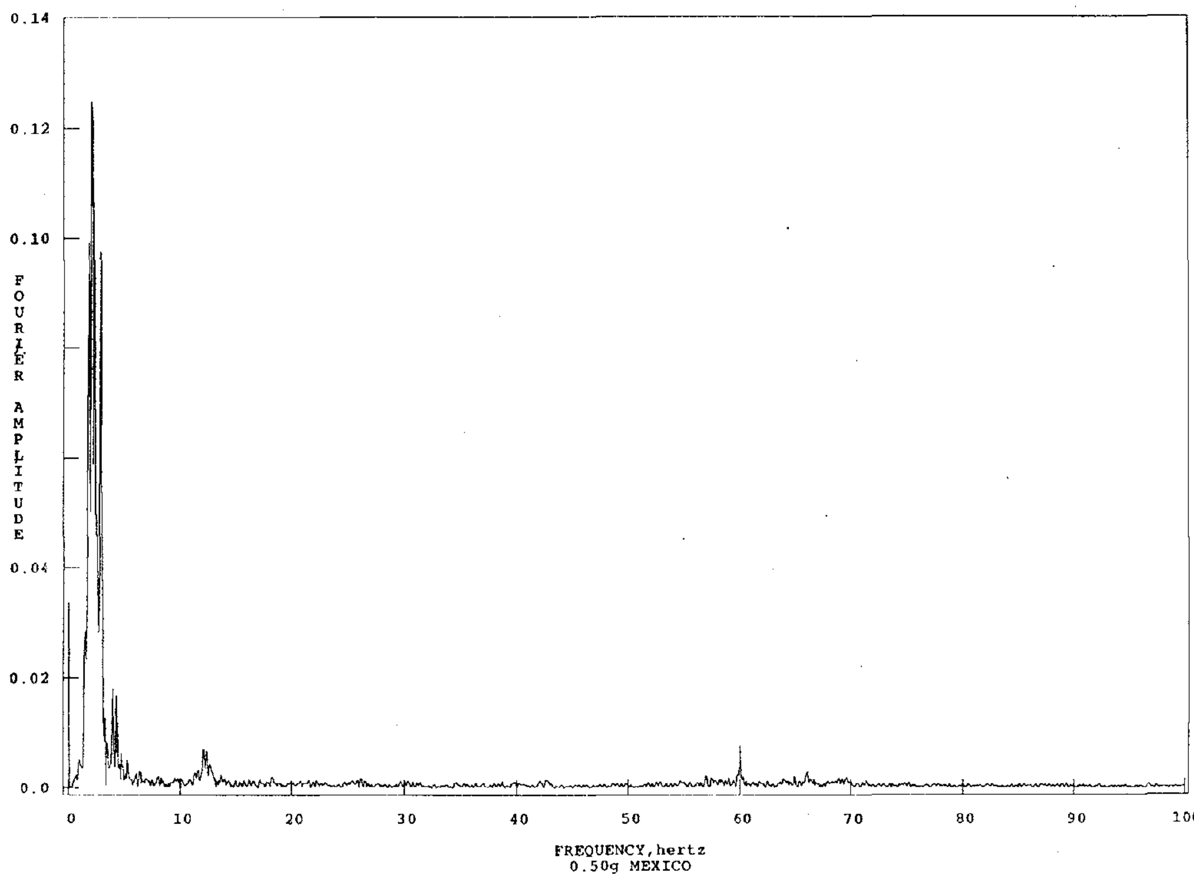
FLEXIBLE FRAME



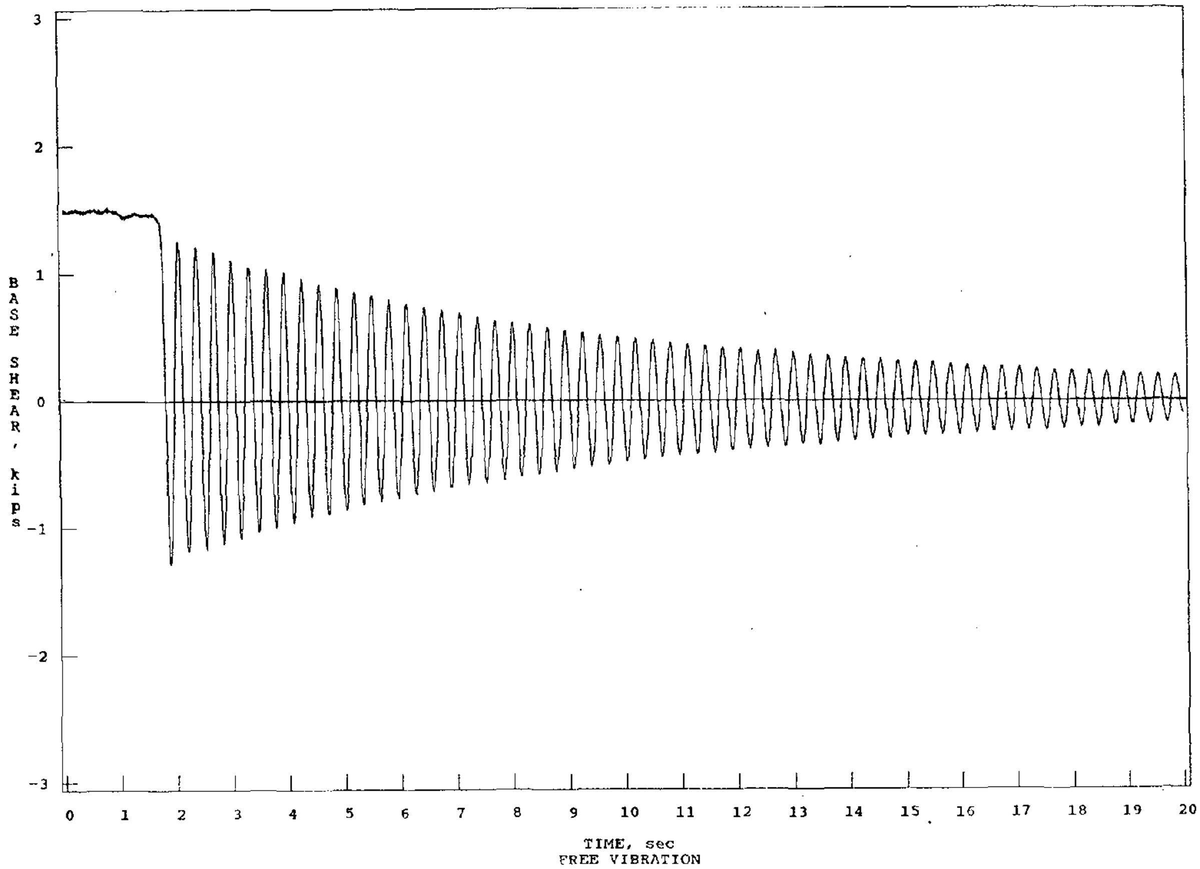
FLEXIBLE FRAME



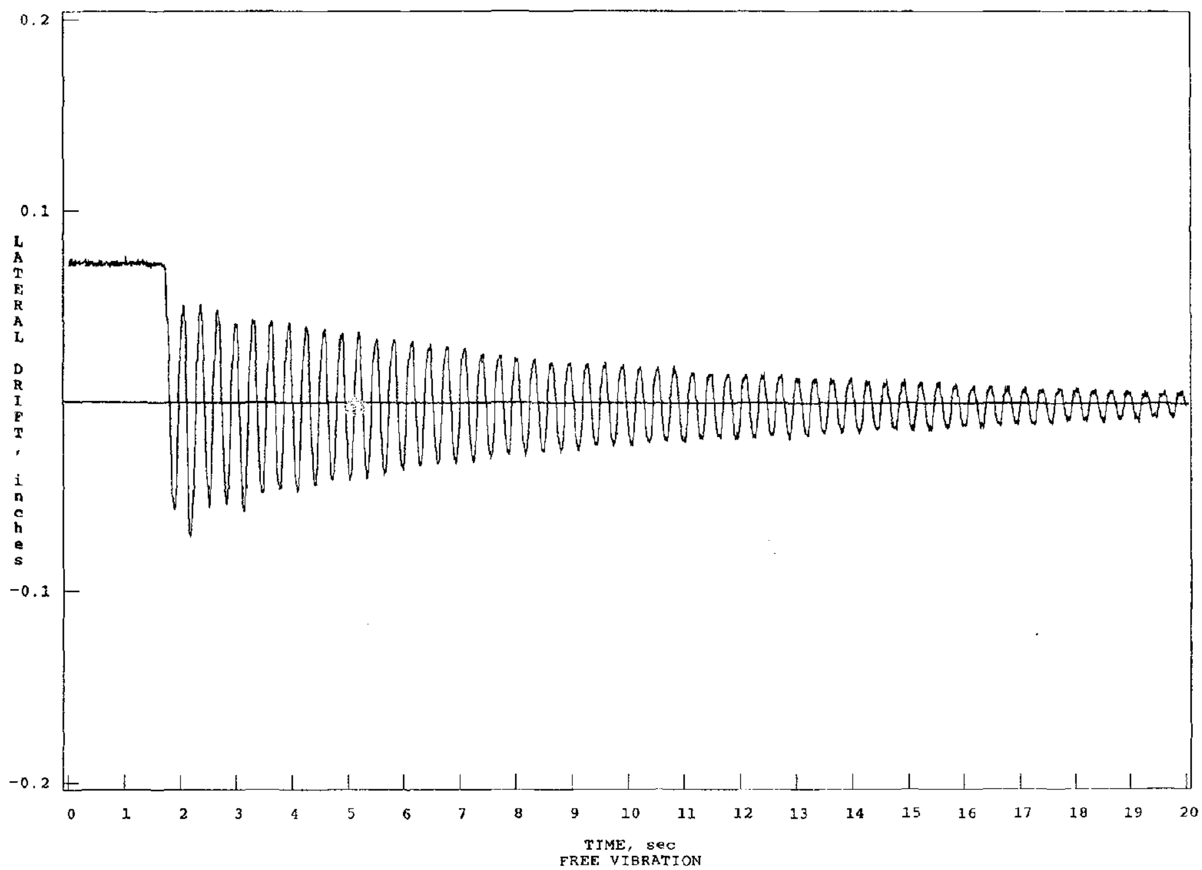
FLEXIBLE FRAME



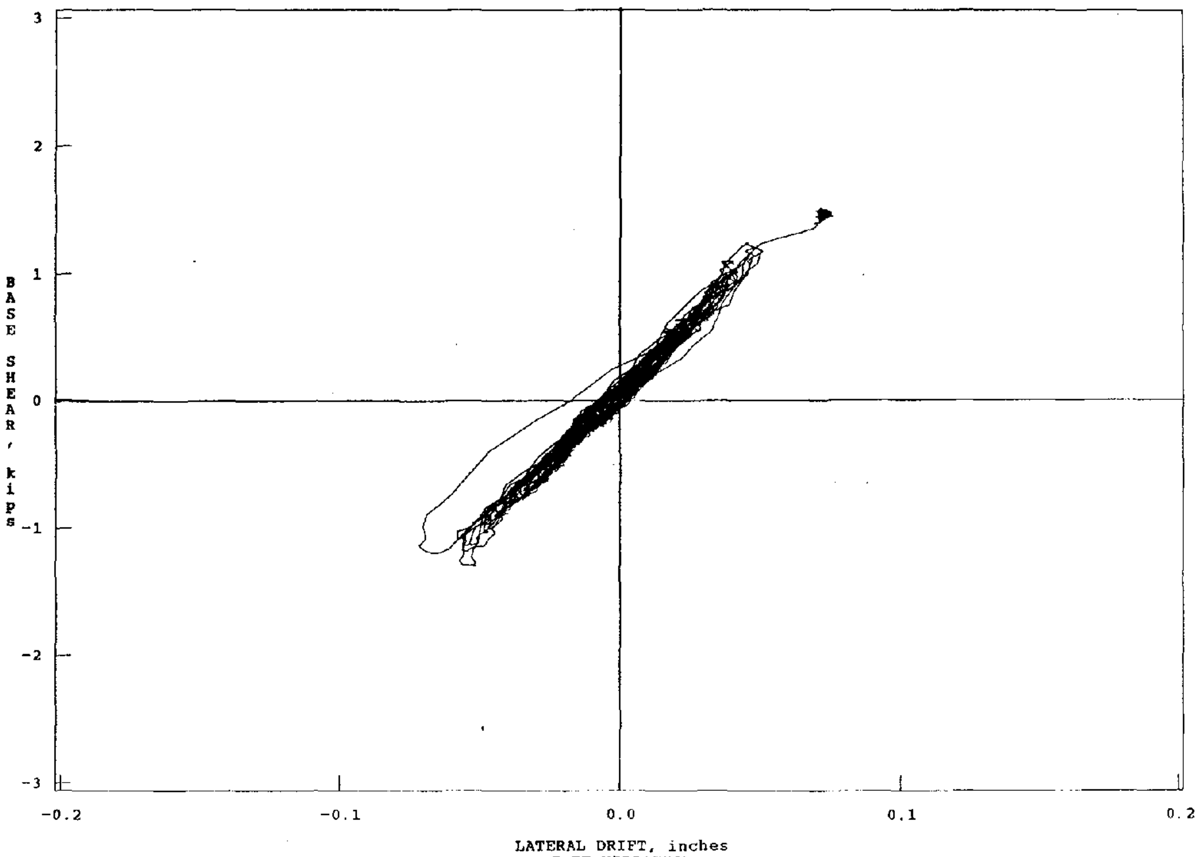
SEMI-RIGID FRAME



SEMI-RIGID FRAME

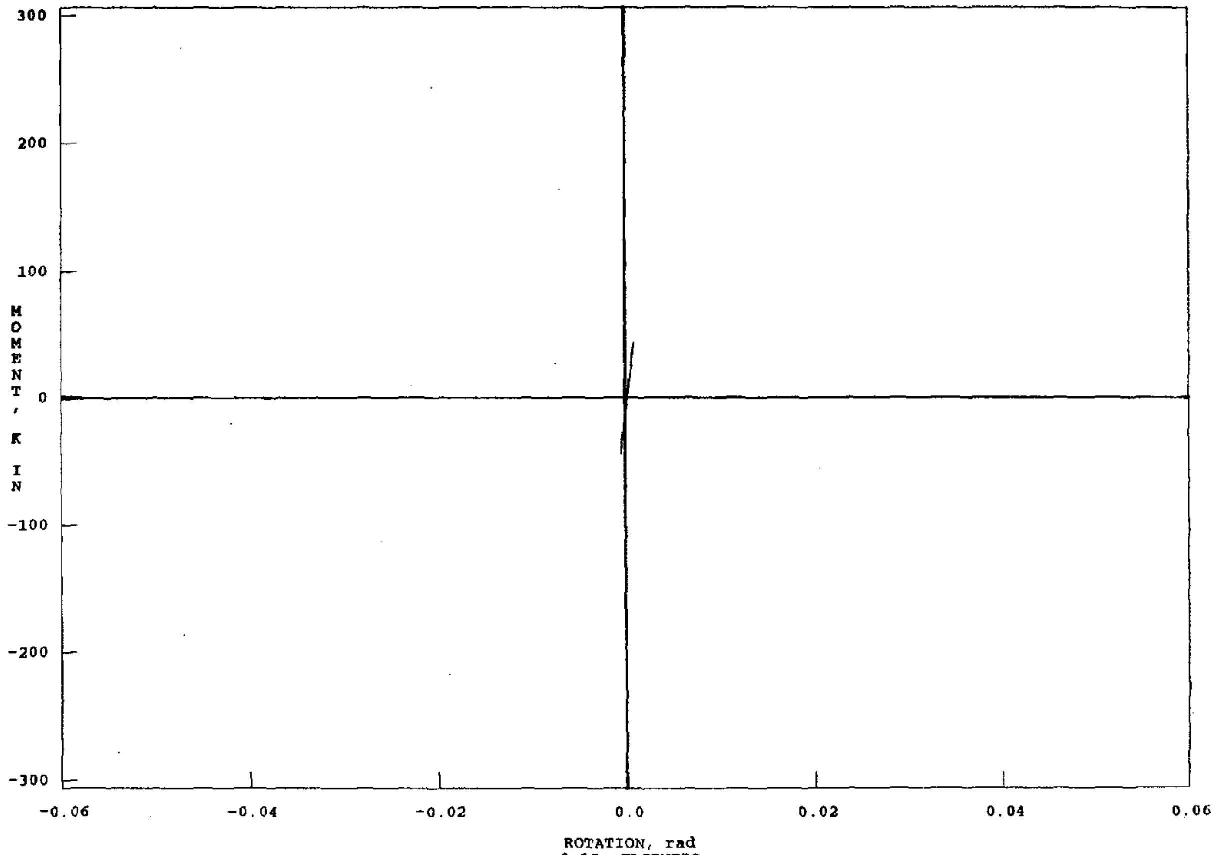


SEMI-RIGID FRAME



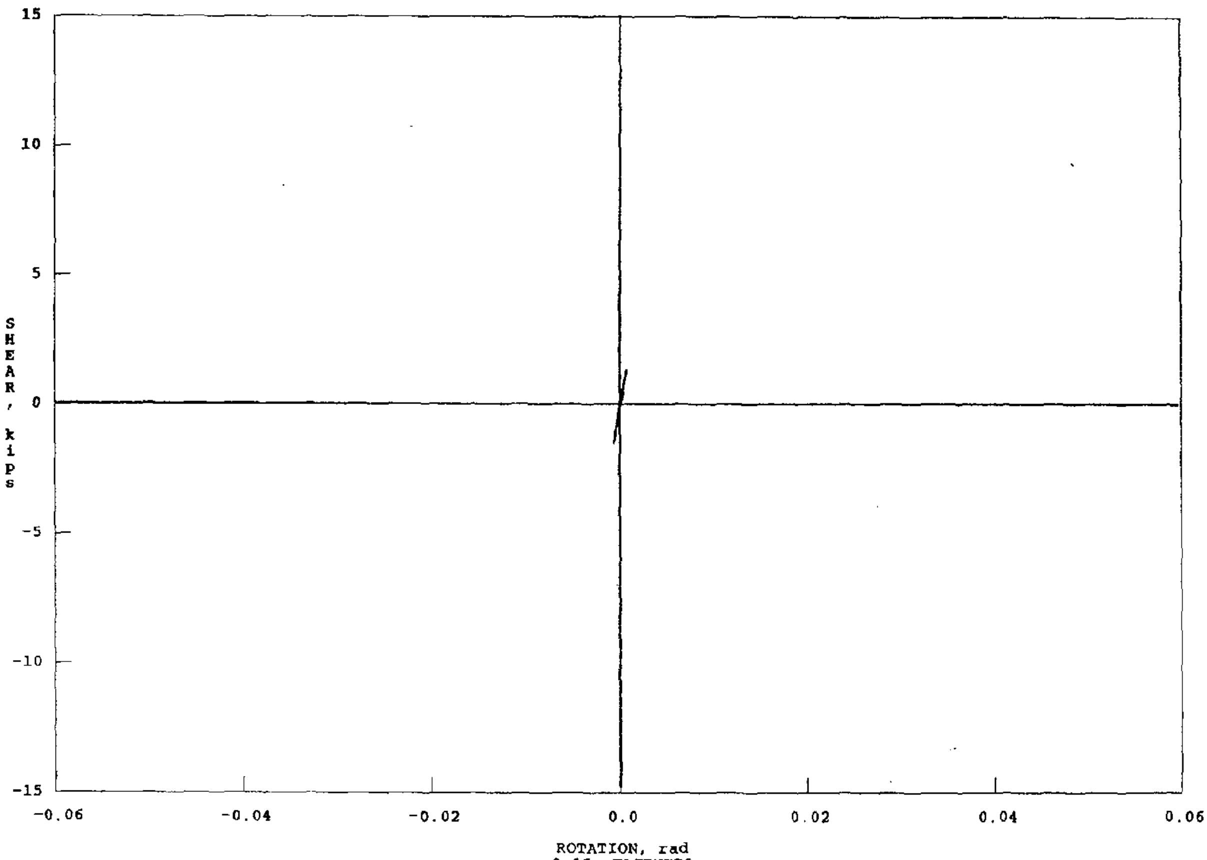
FREE VIBRATION

SEMI-RIGID FRAME



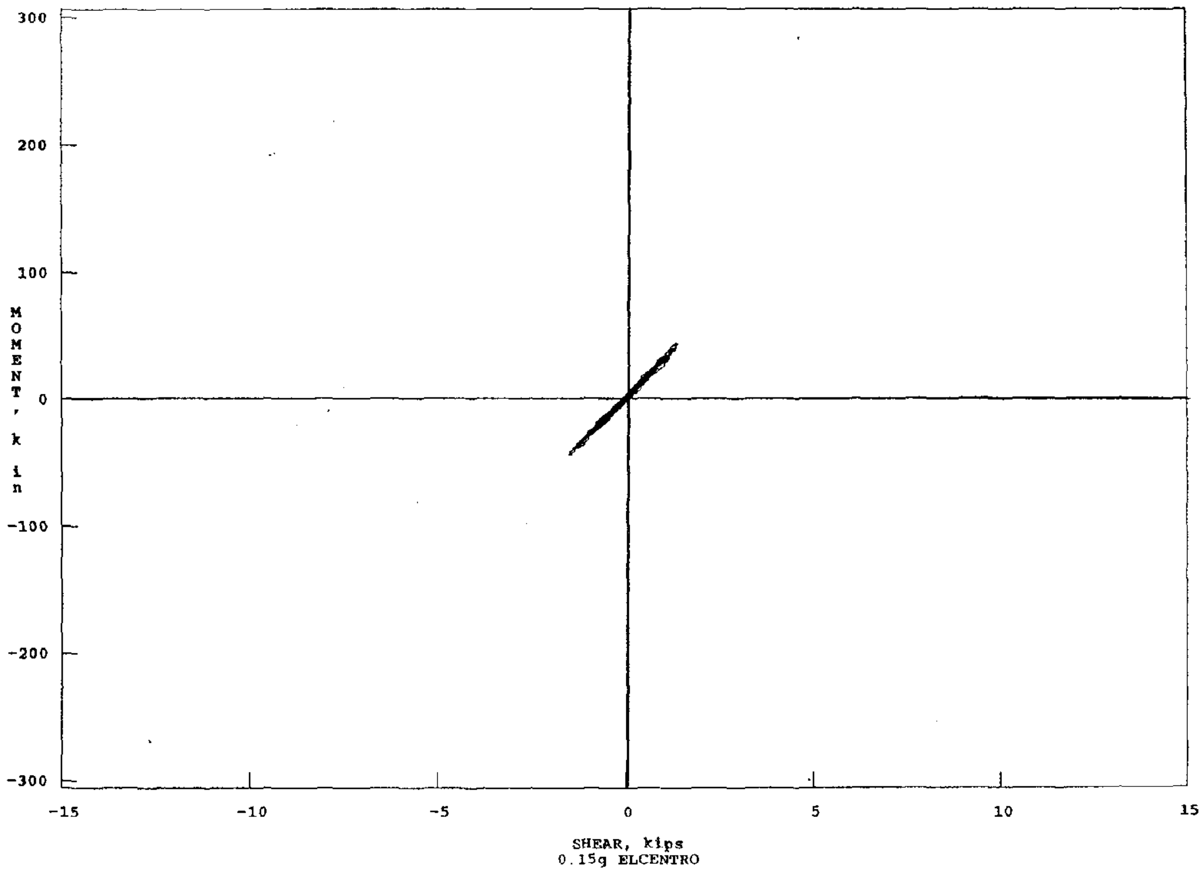
0.15g ELCENTRO

SEMI-RIGID FRAME



0.15g ELCENTRO

SEMI-RIGID FRAME



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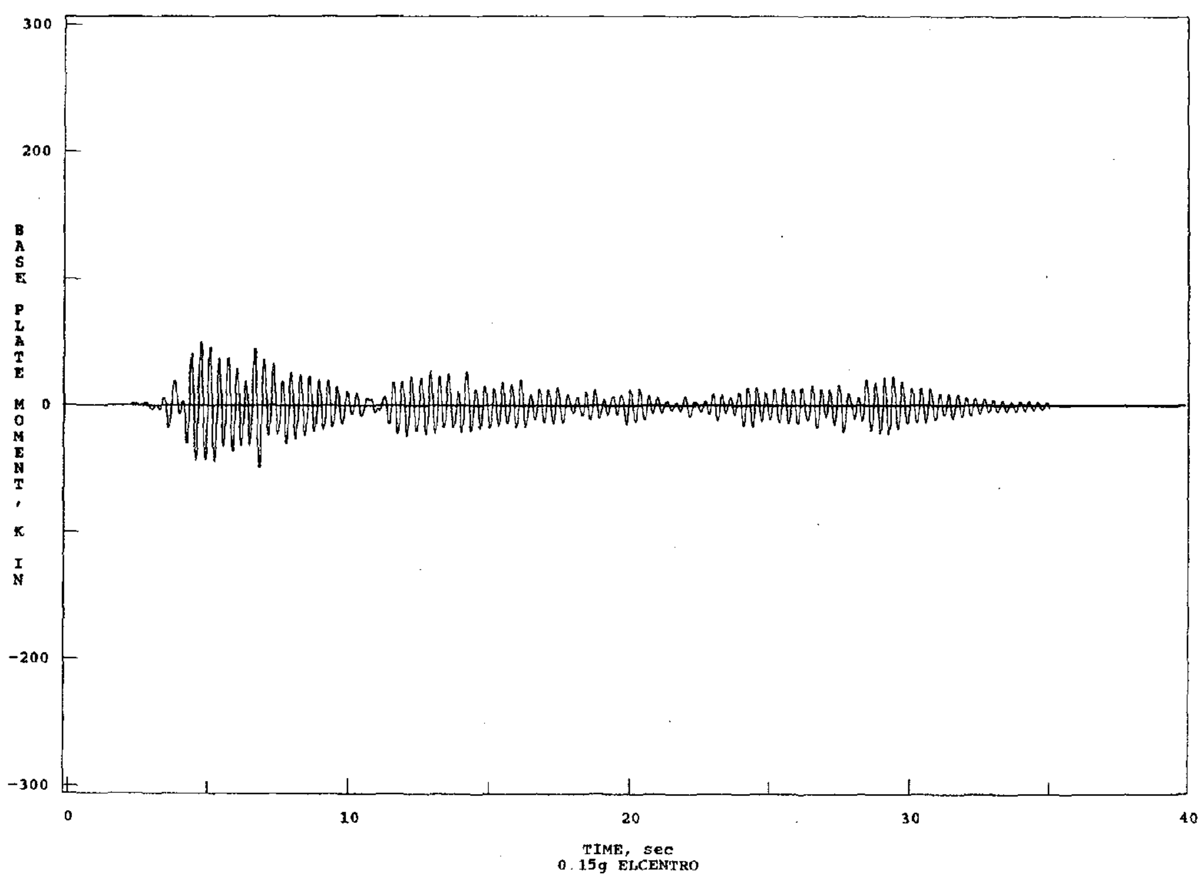

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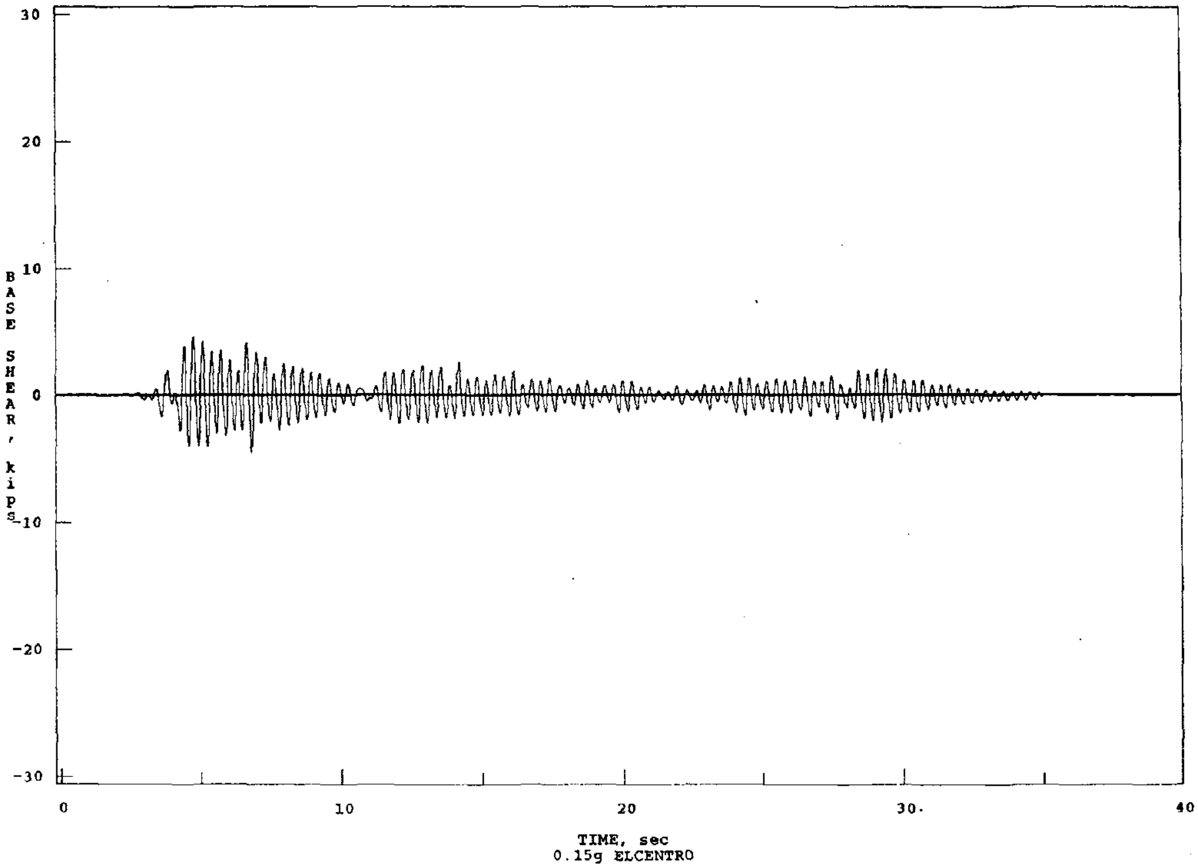

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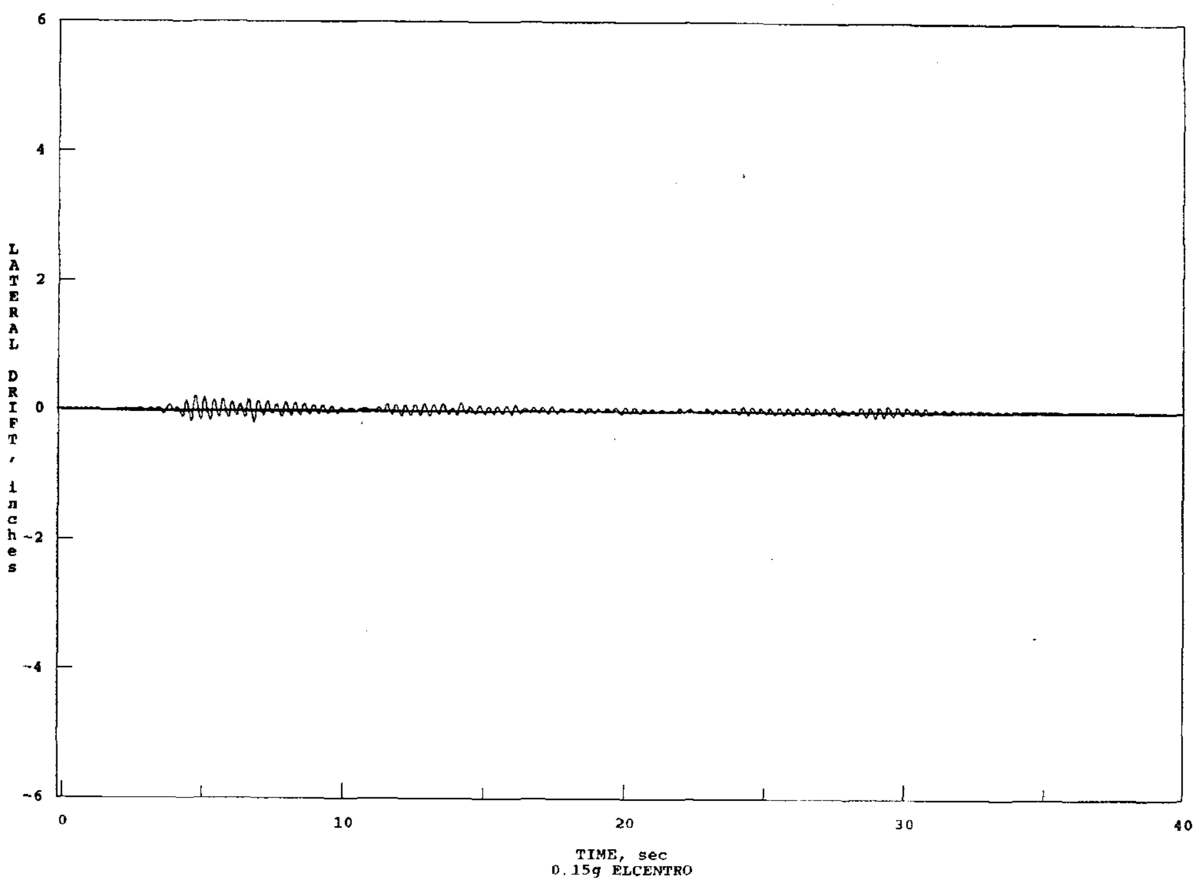
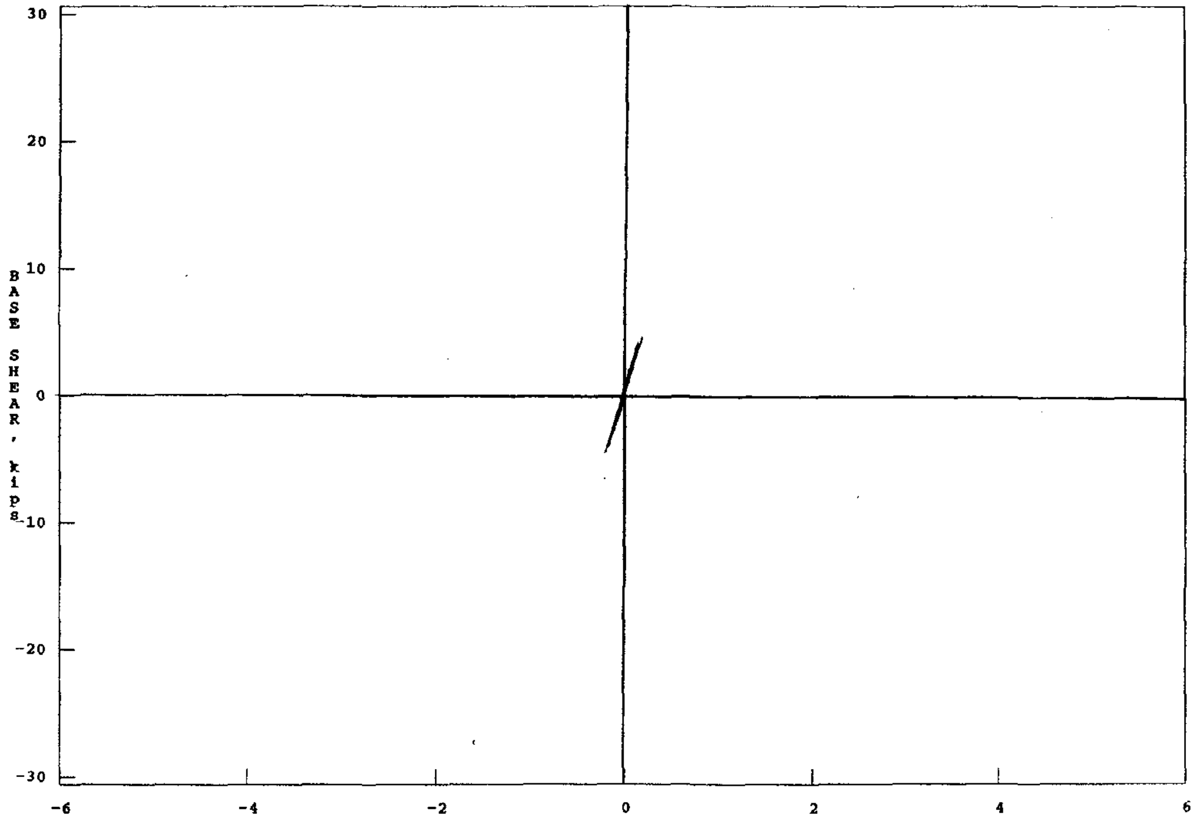
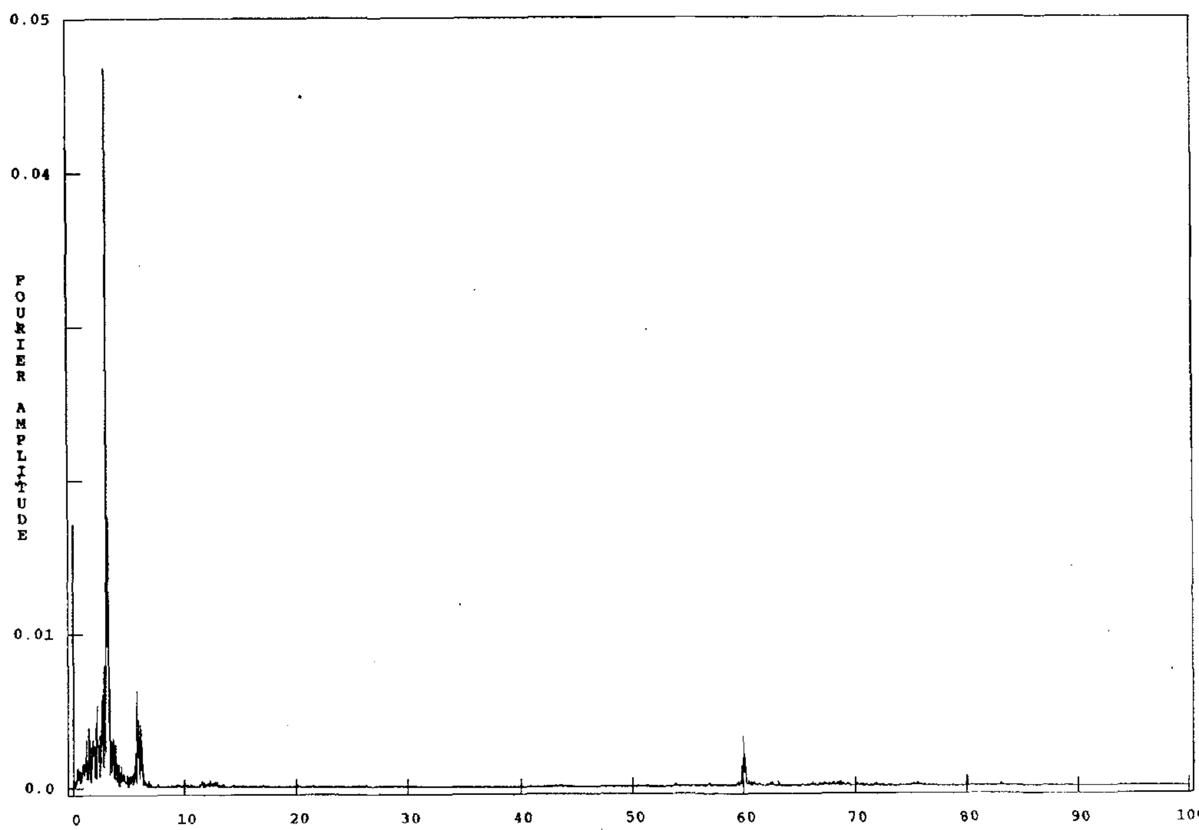


SEMI-RIGID FRAME

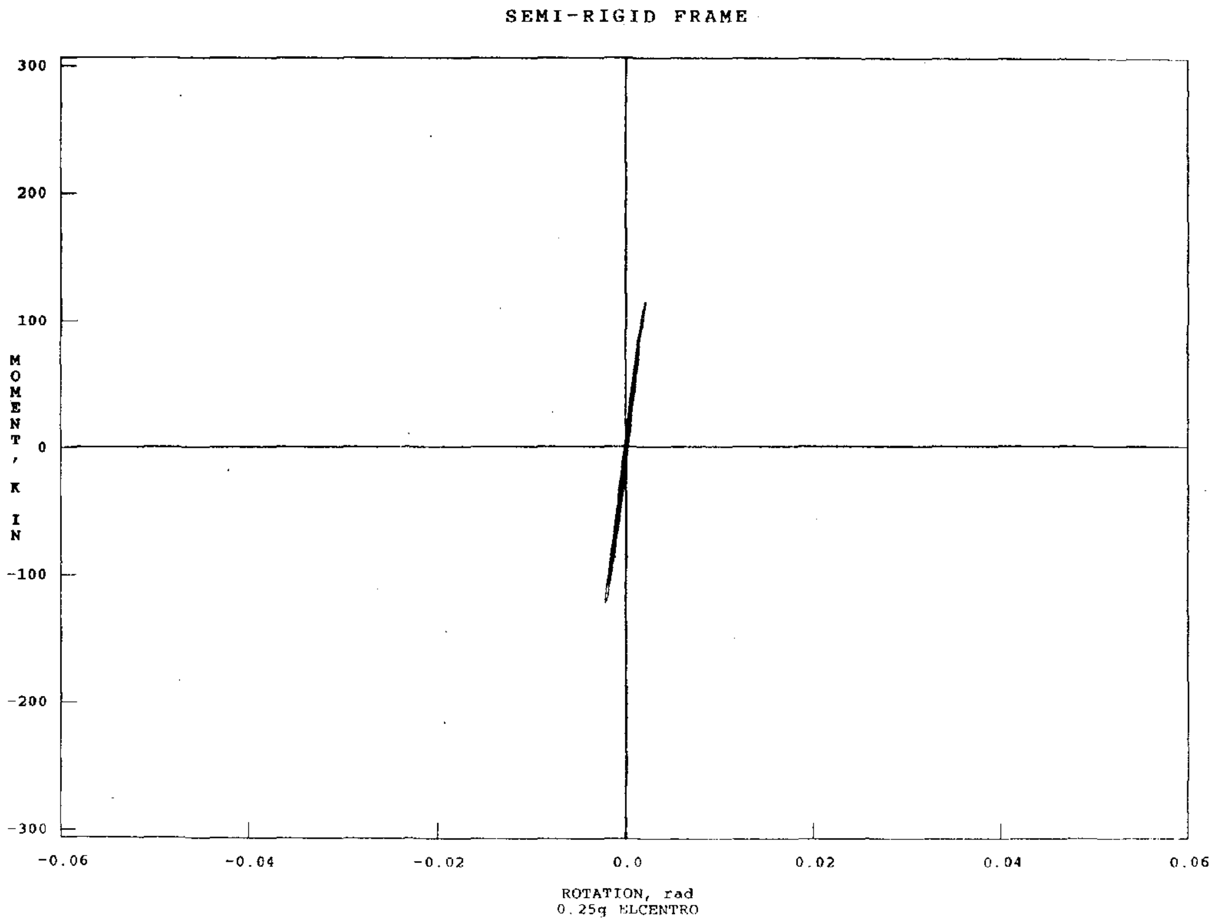


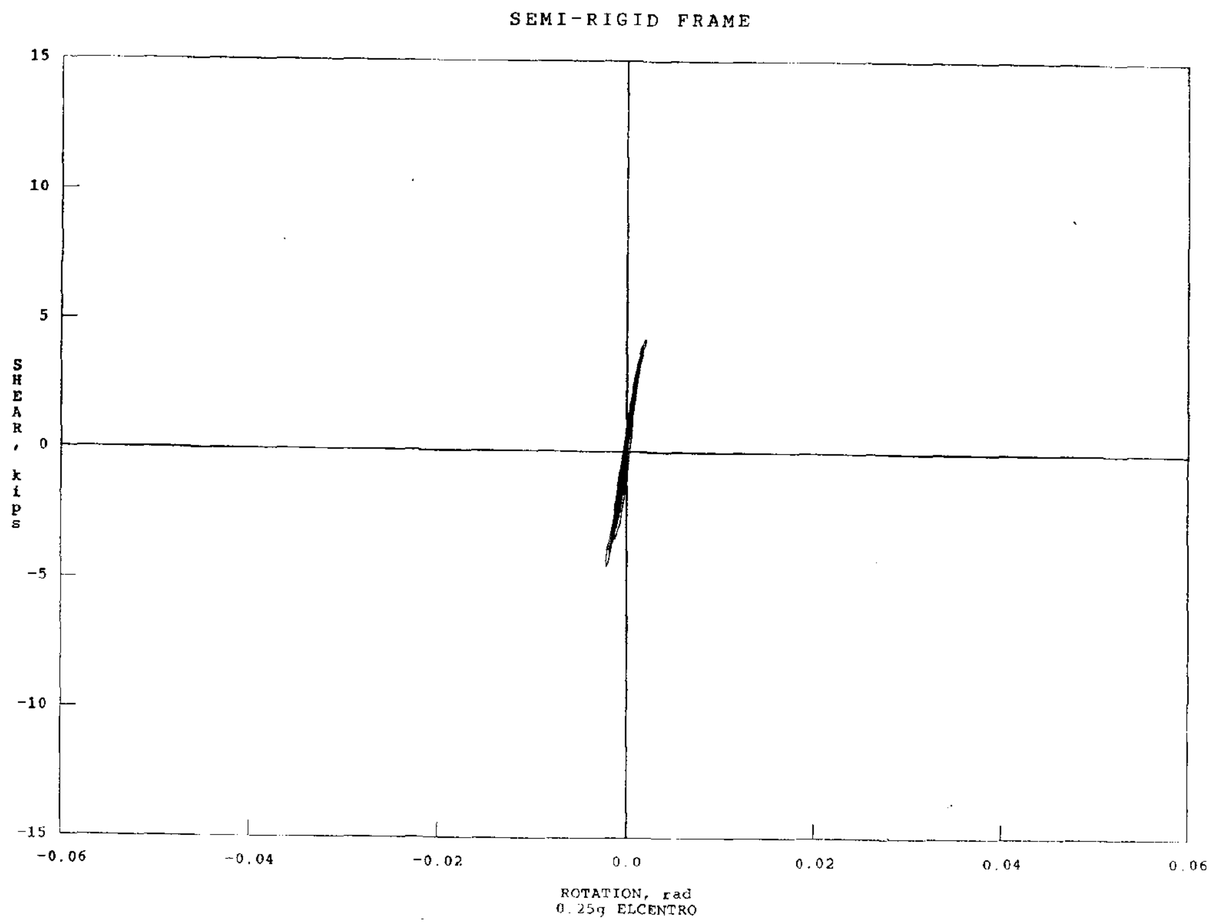
LATERAL DRIFT, inches
0.15g ELCENTRO

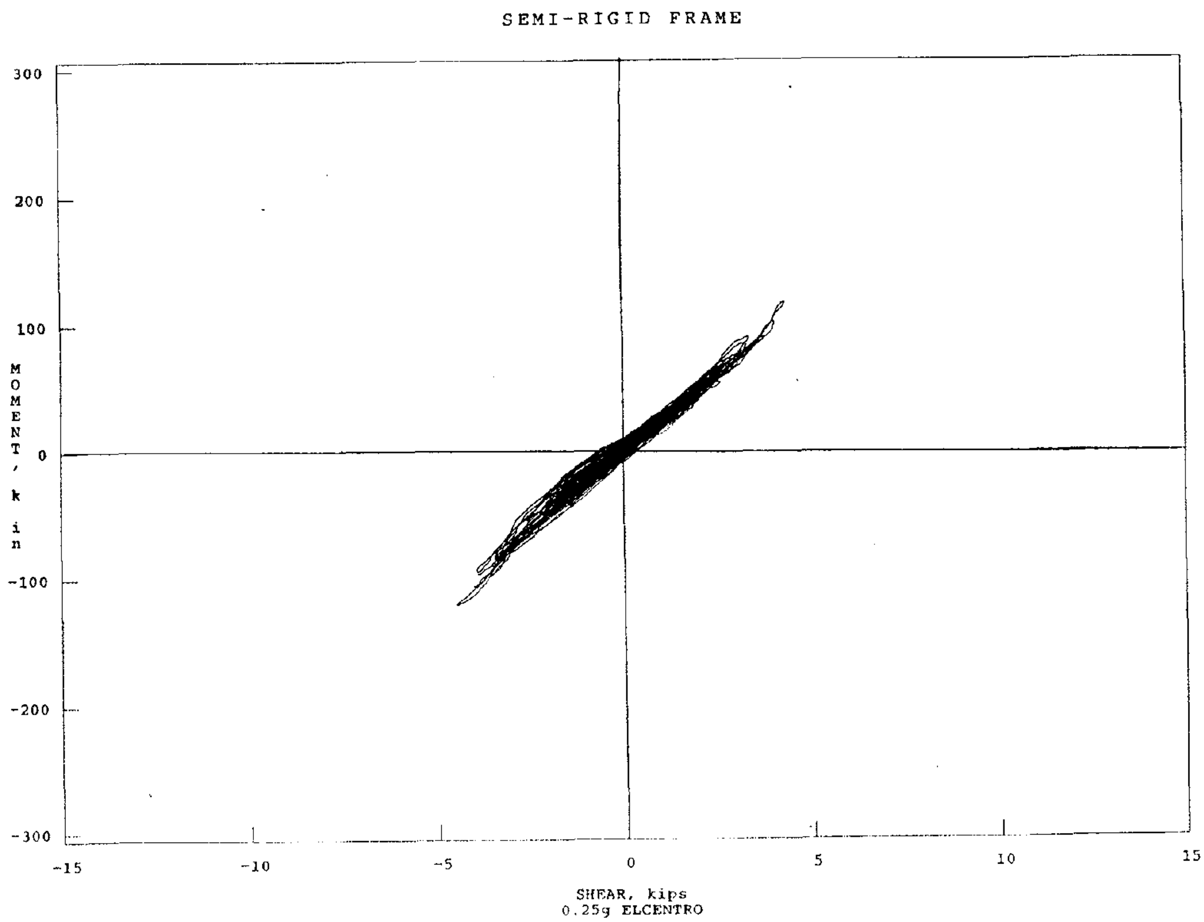
SEMI-RIGID FRAME

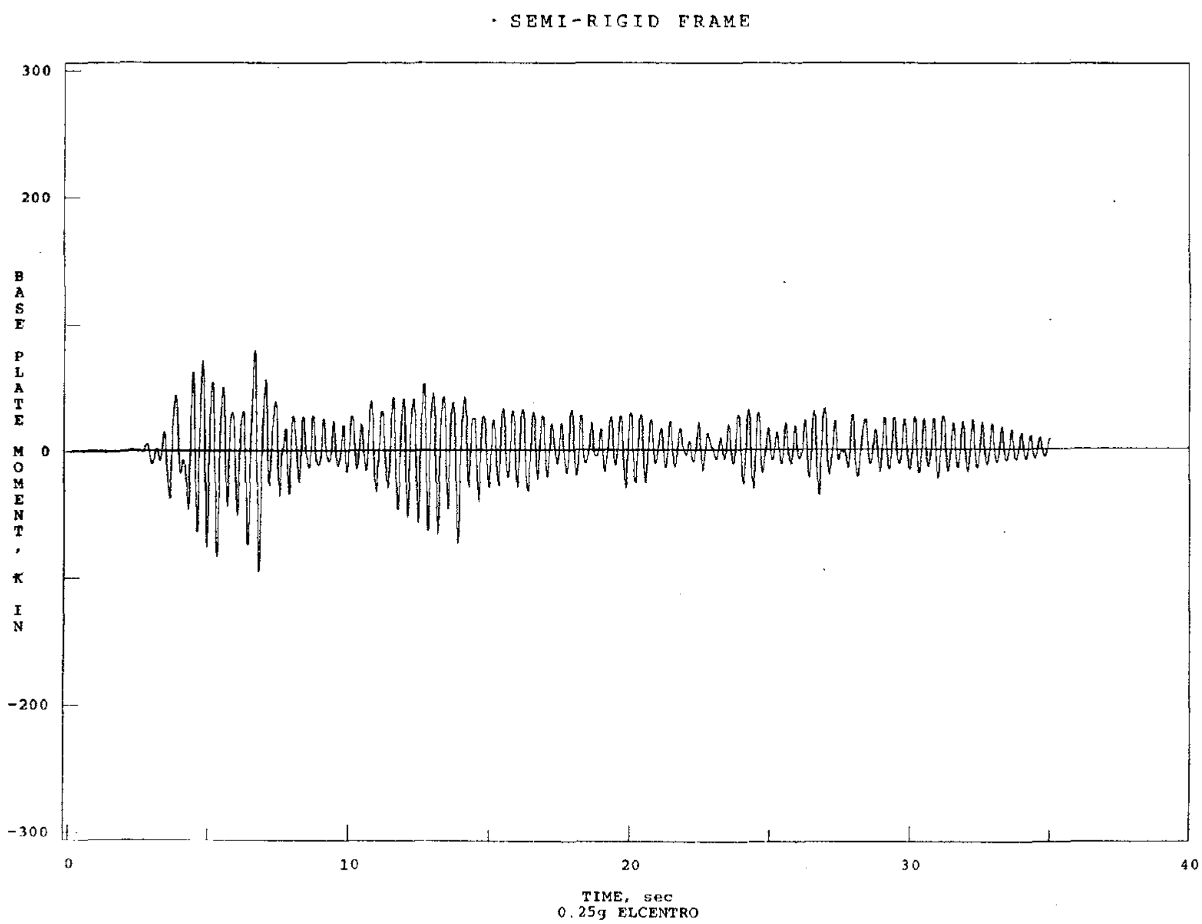


FREQUENCY, hertz
0.15g ELCENTRO

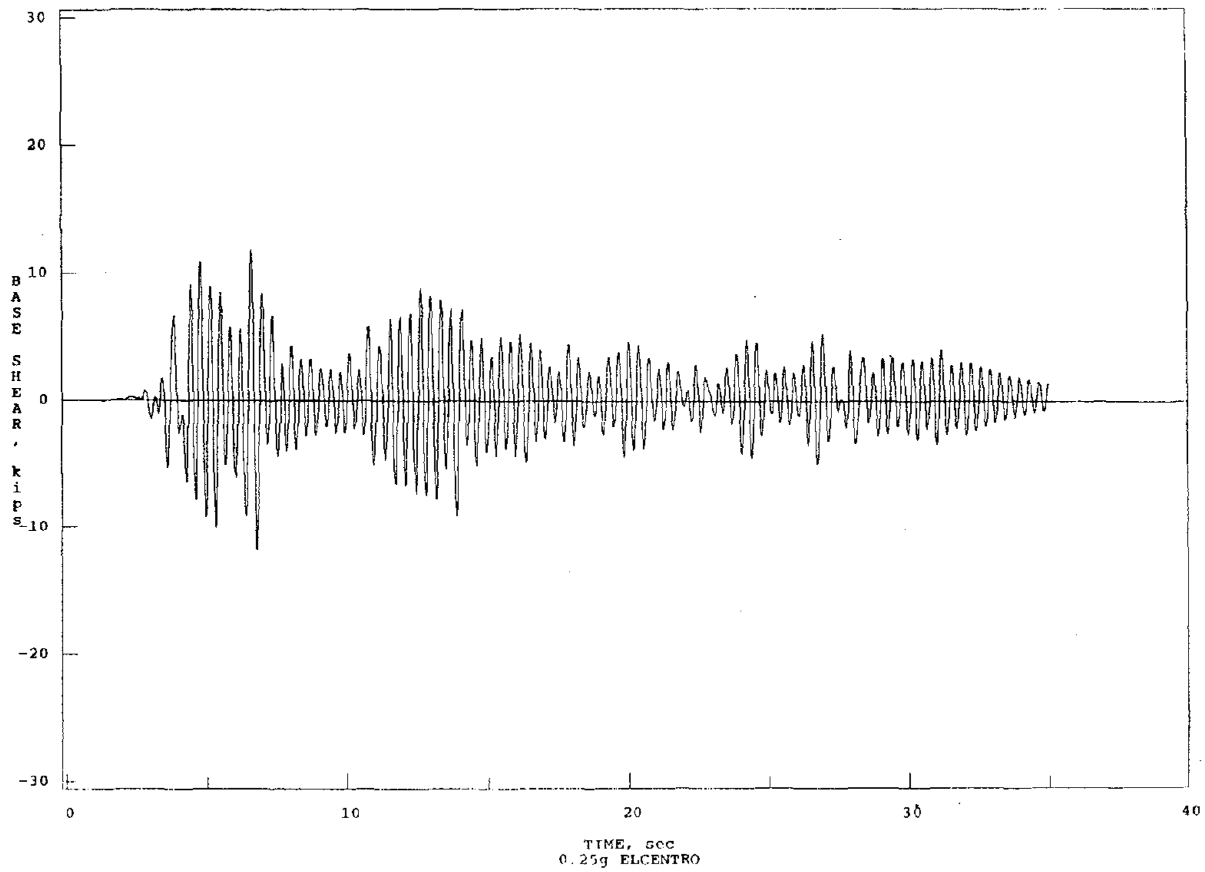

SEMI-RIGID FRAME
300
200
100
0
-100
-200
-300
MOMENT, K IN
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.25g ELCENTRO


SEMI-RIGID FRAME
15
10
5
0
-5
-10
-15
SHEAR, kips
-0.06
-0.04
-0.02
0.0
0.02
0.04
0.06
ROTATION, rad
0.25g ELCENTRO

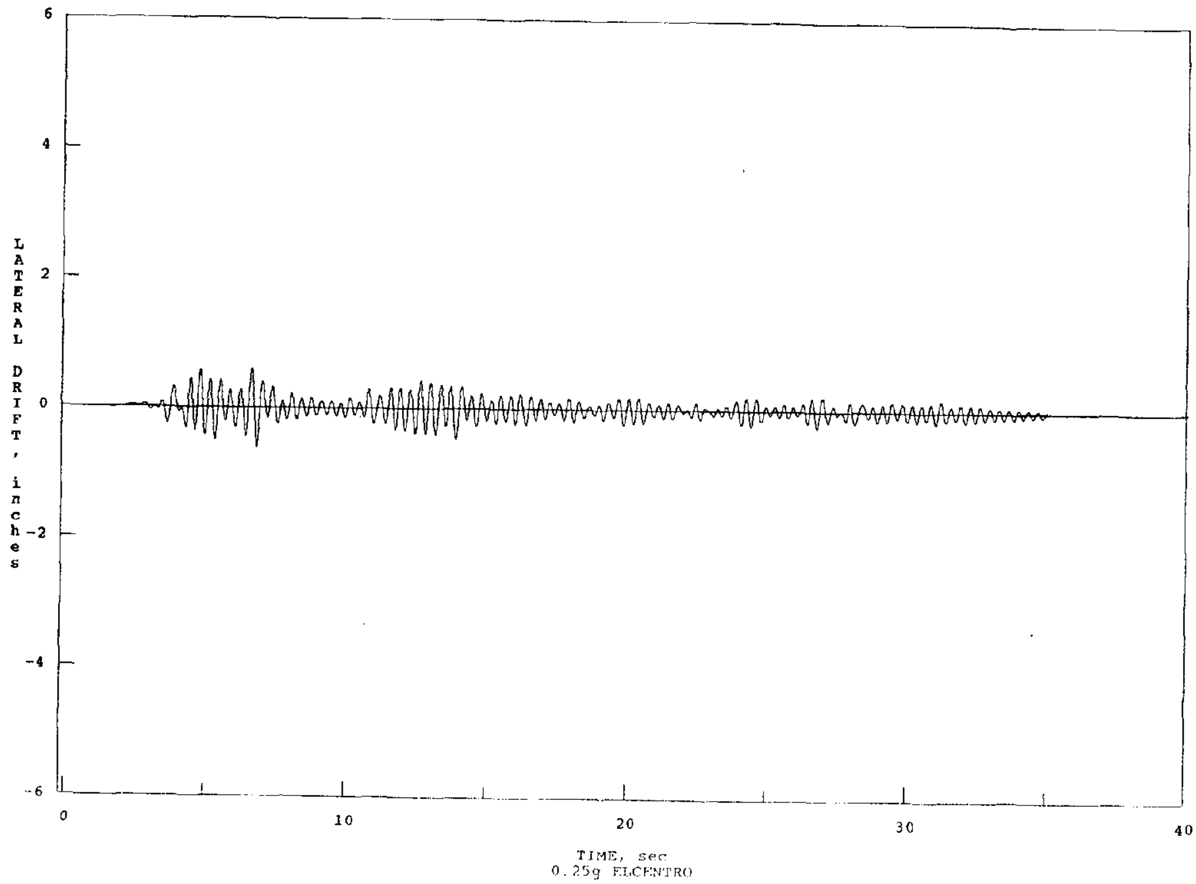

SEMI-RIGID FRAME
300
200
100
0
-100
-200
-300
MOMENT, k in
-15
-10
-5
0
5
10
15
SHEAR, kips
0.25g ELCENTRO


SEMI-RIGID FRAME
300
200
0
-200
-300
BASE PLATE MOMENT, K IN
0
10
20
30
40
TIME, sec
0.25g ELCENTRO

SEMI-RIGID FRAME



SEMI-RIGID FRAME



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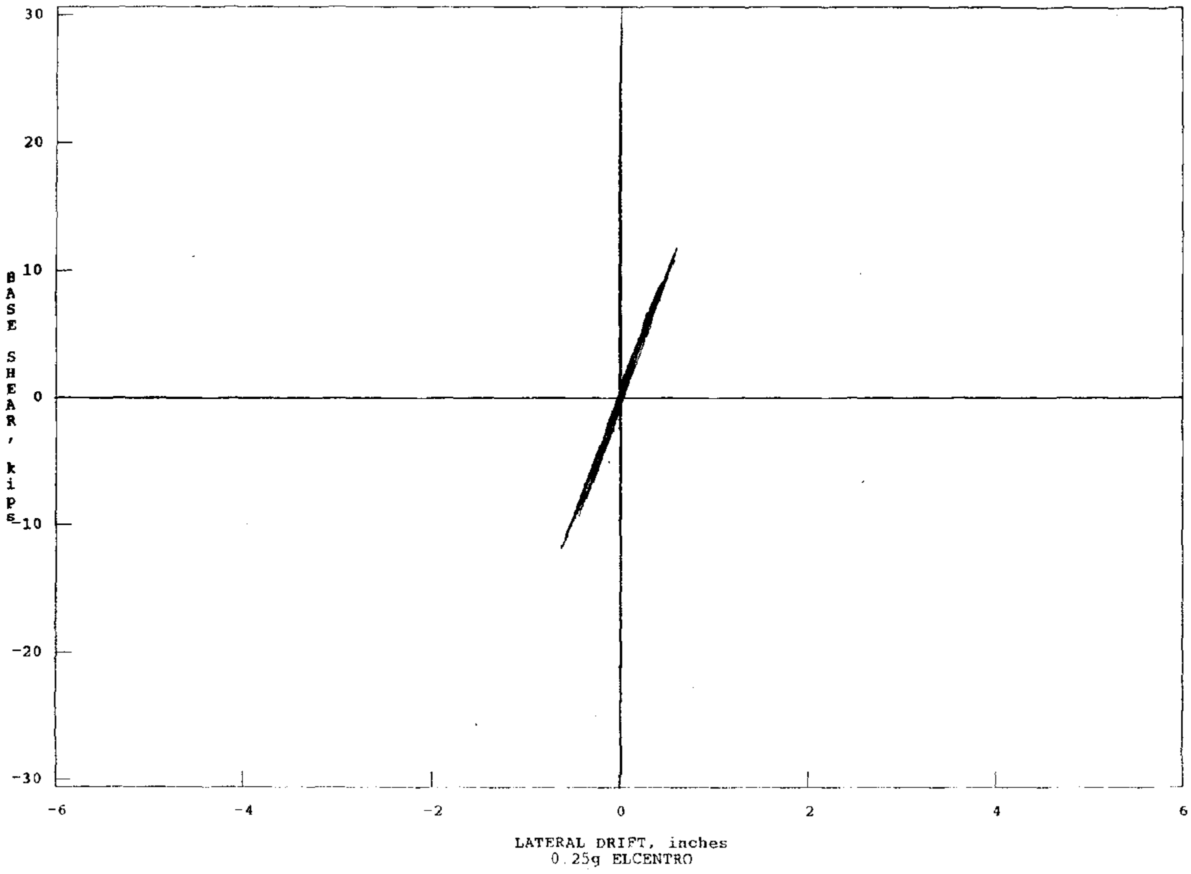

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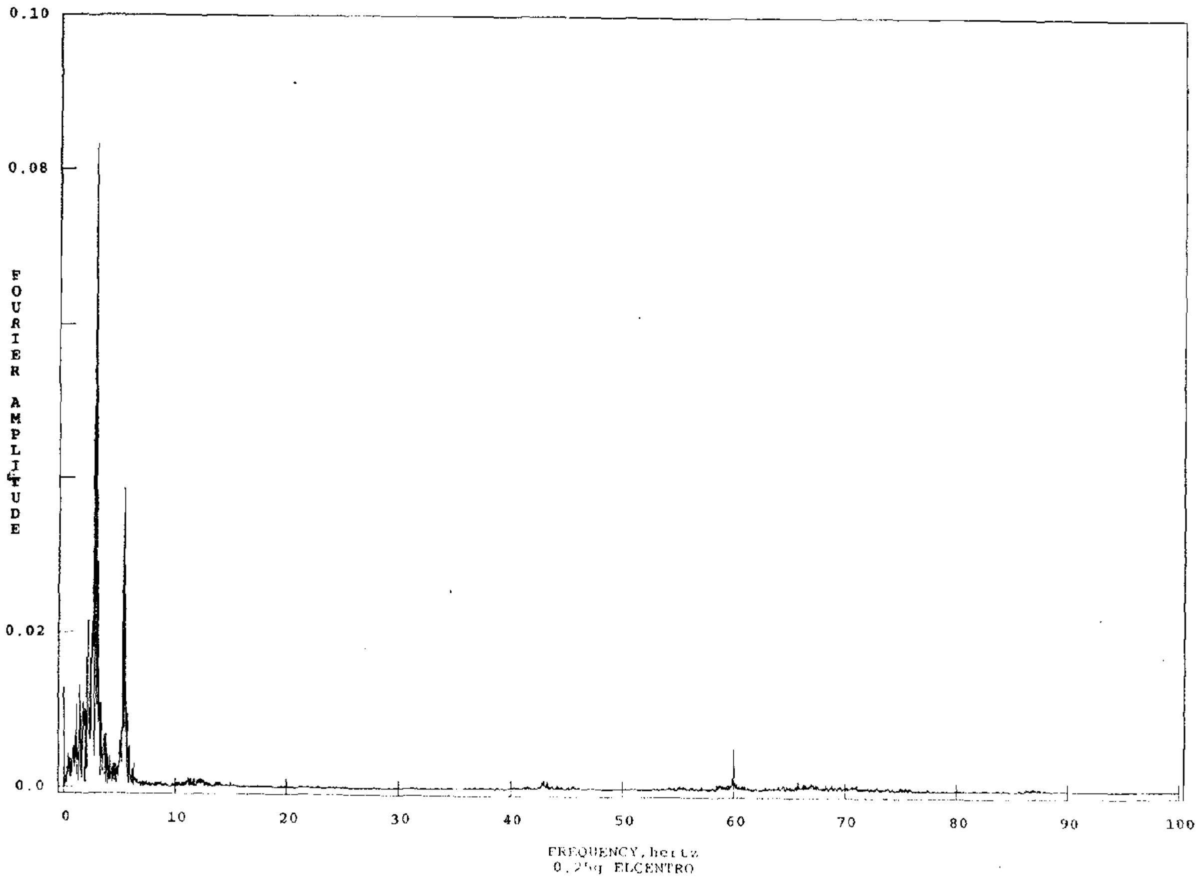
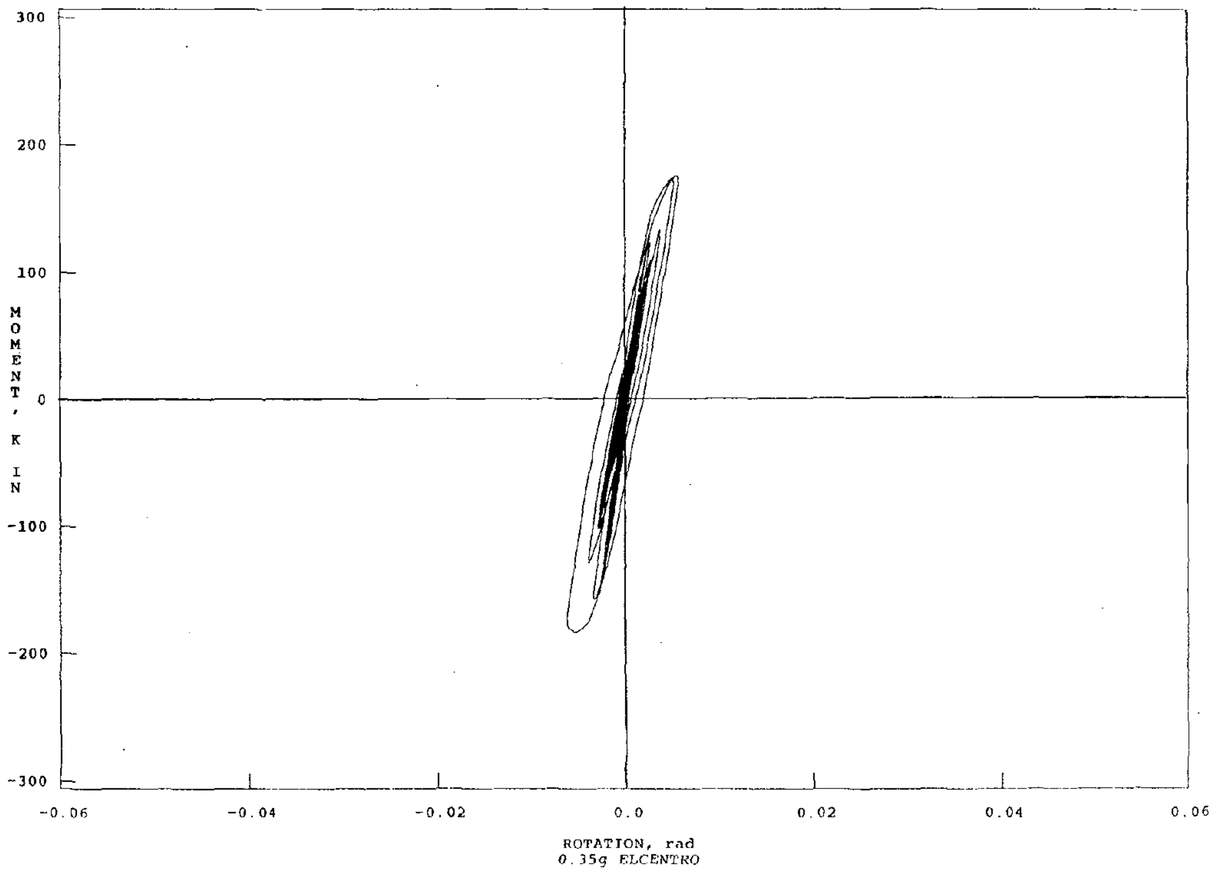


SEMI-RIGID FRAME



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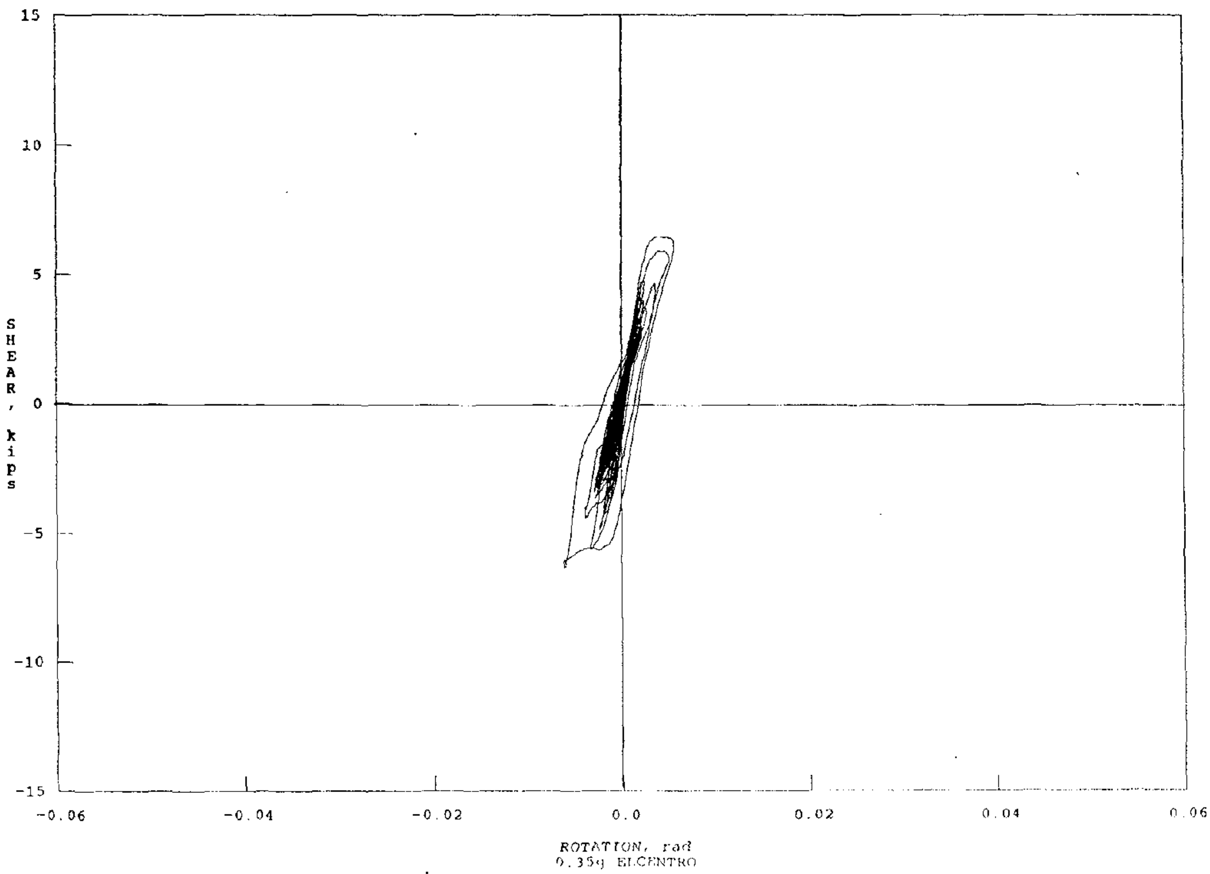
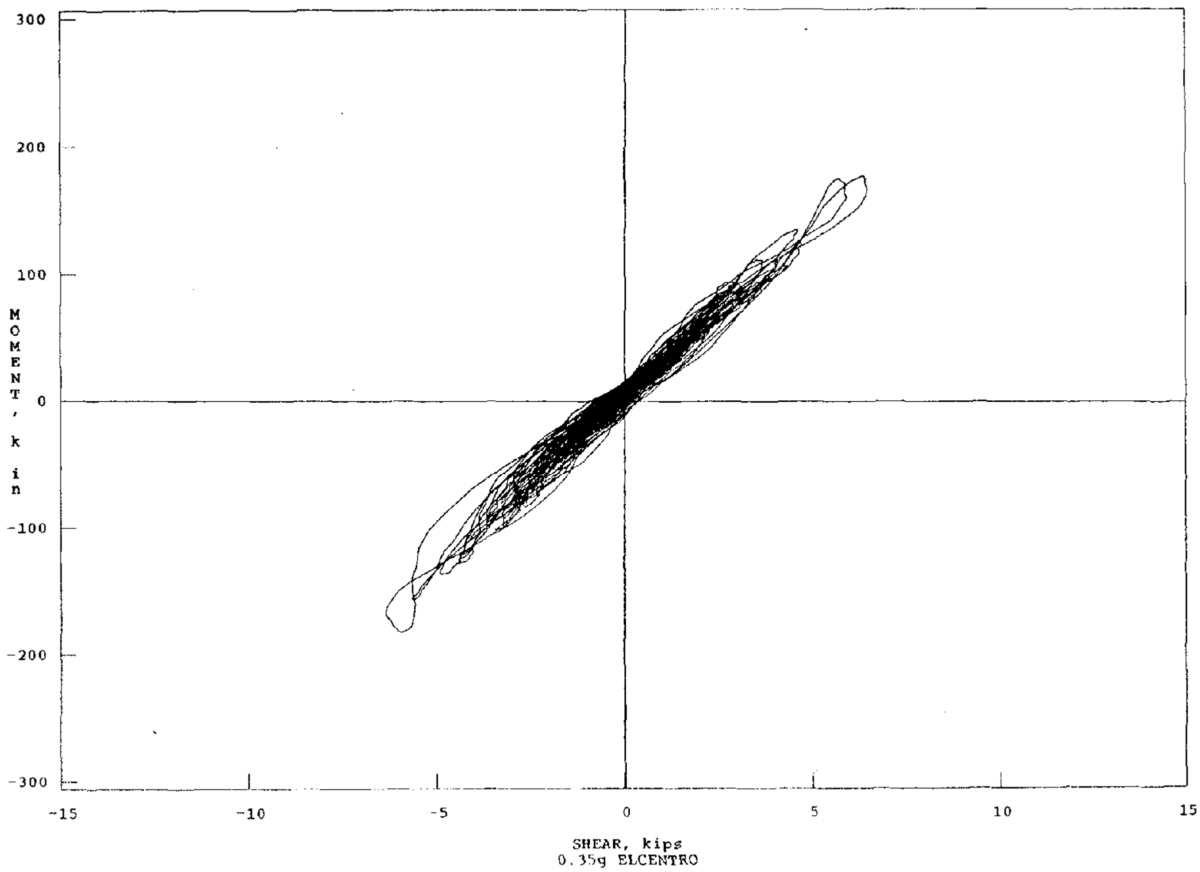


SEMI-RIGID FRAME



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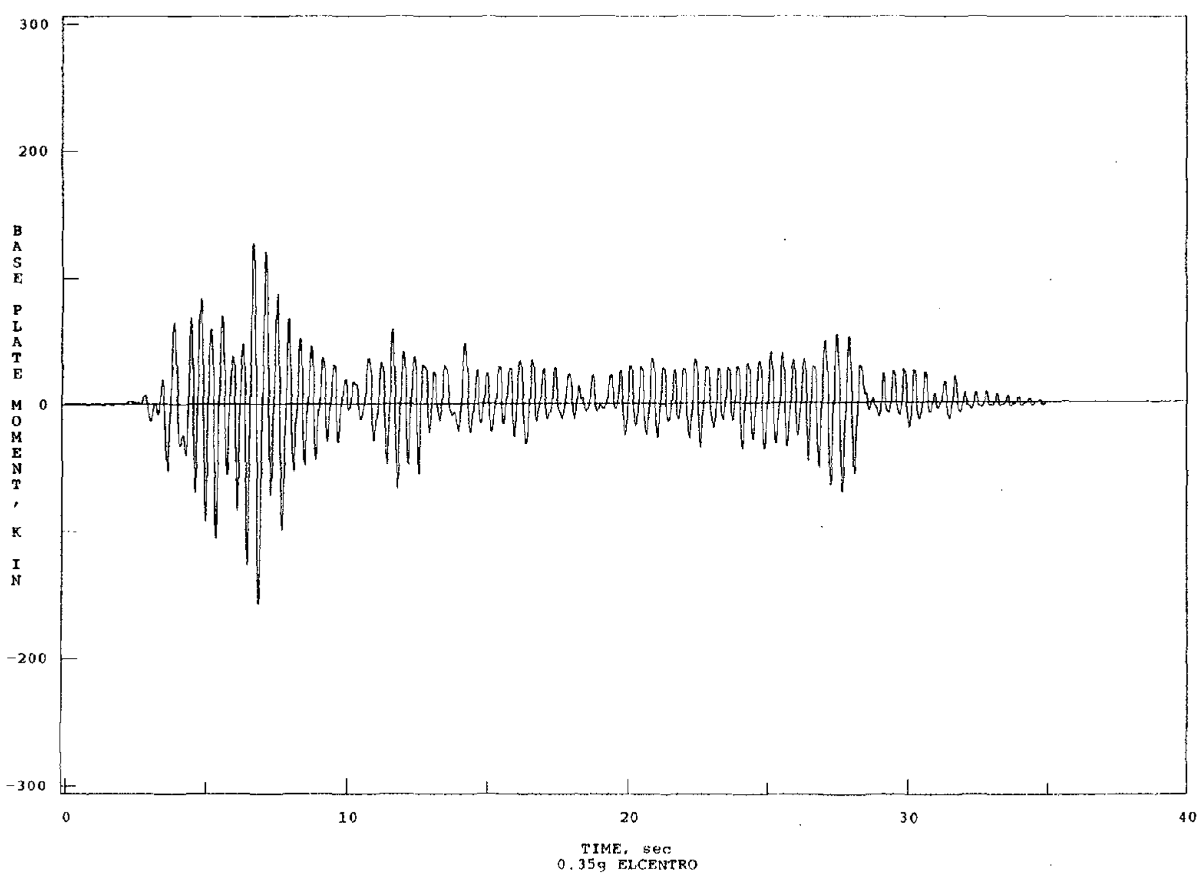

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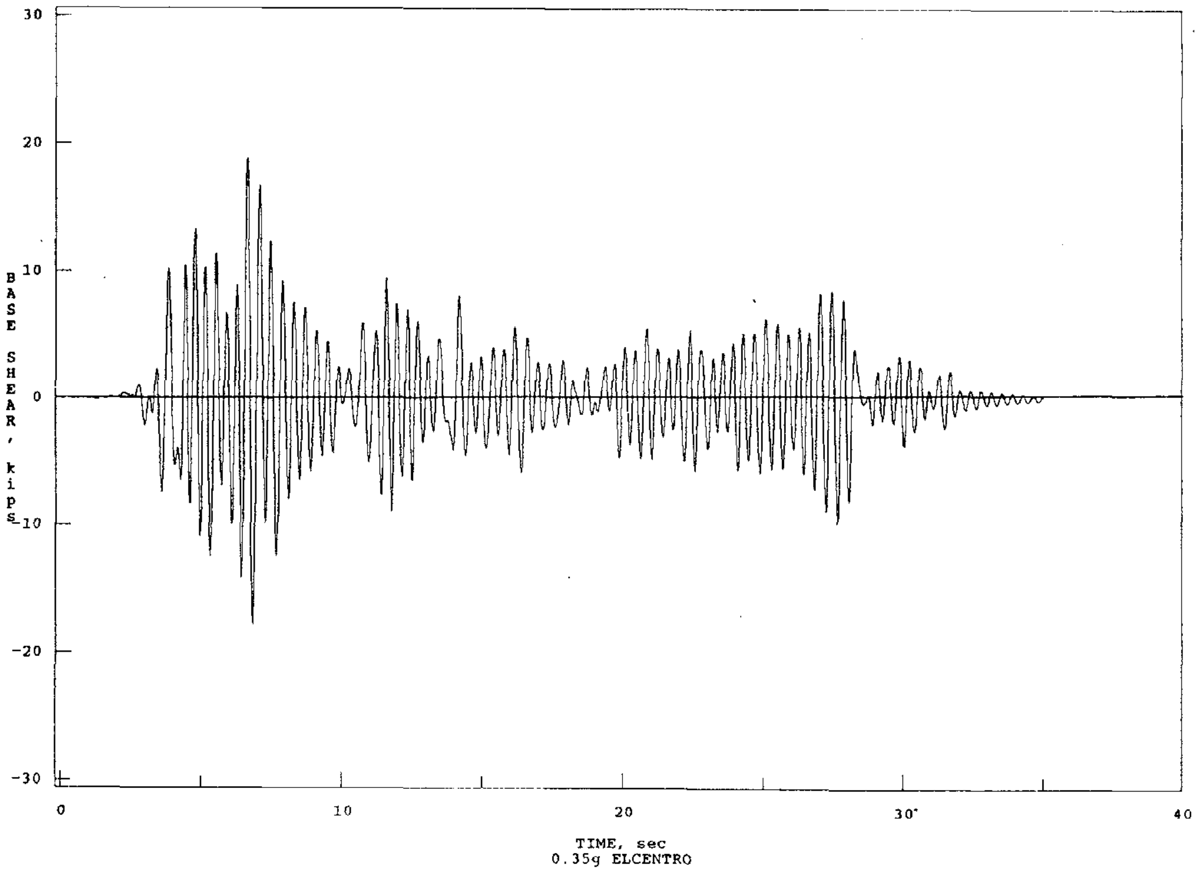
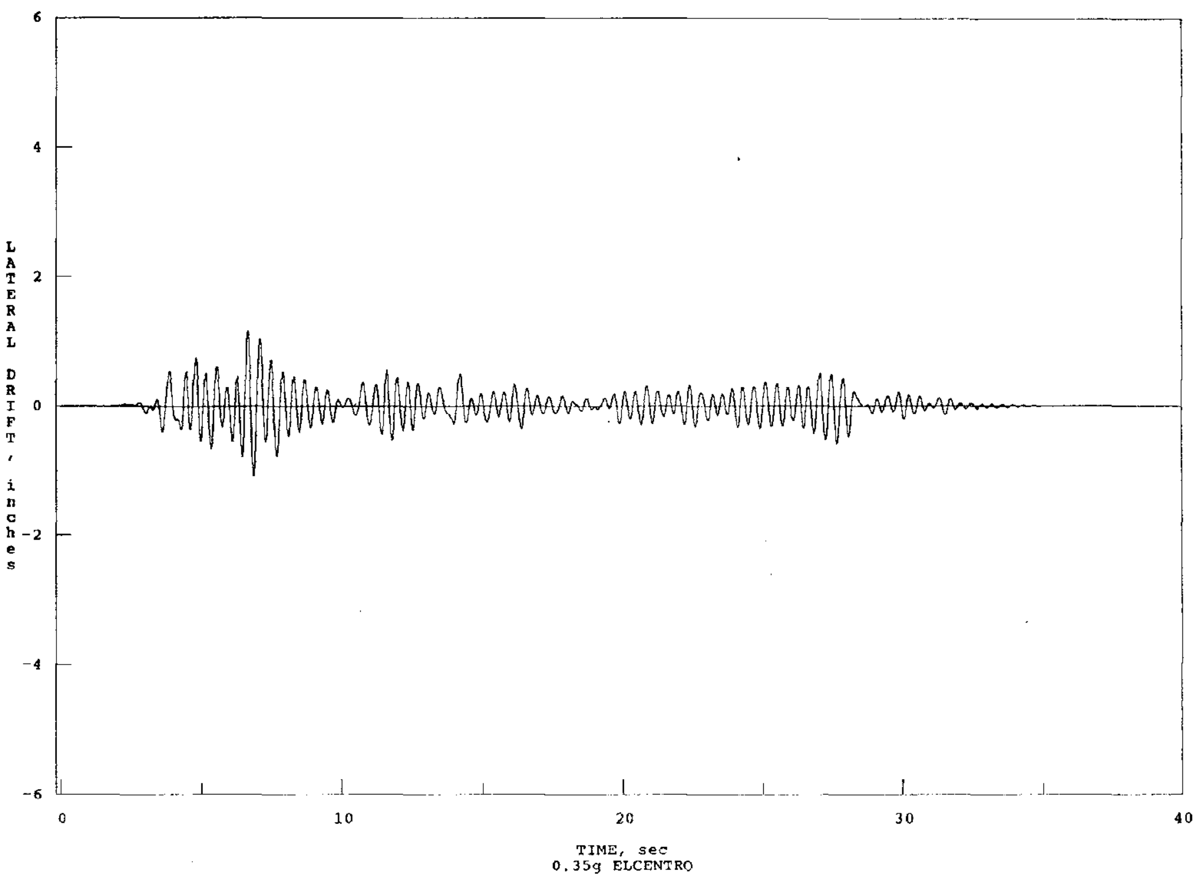
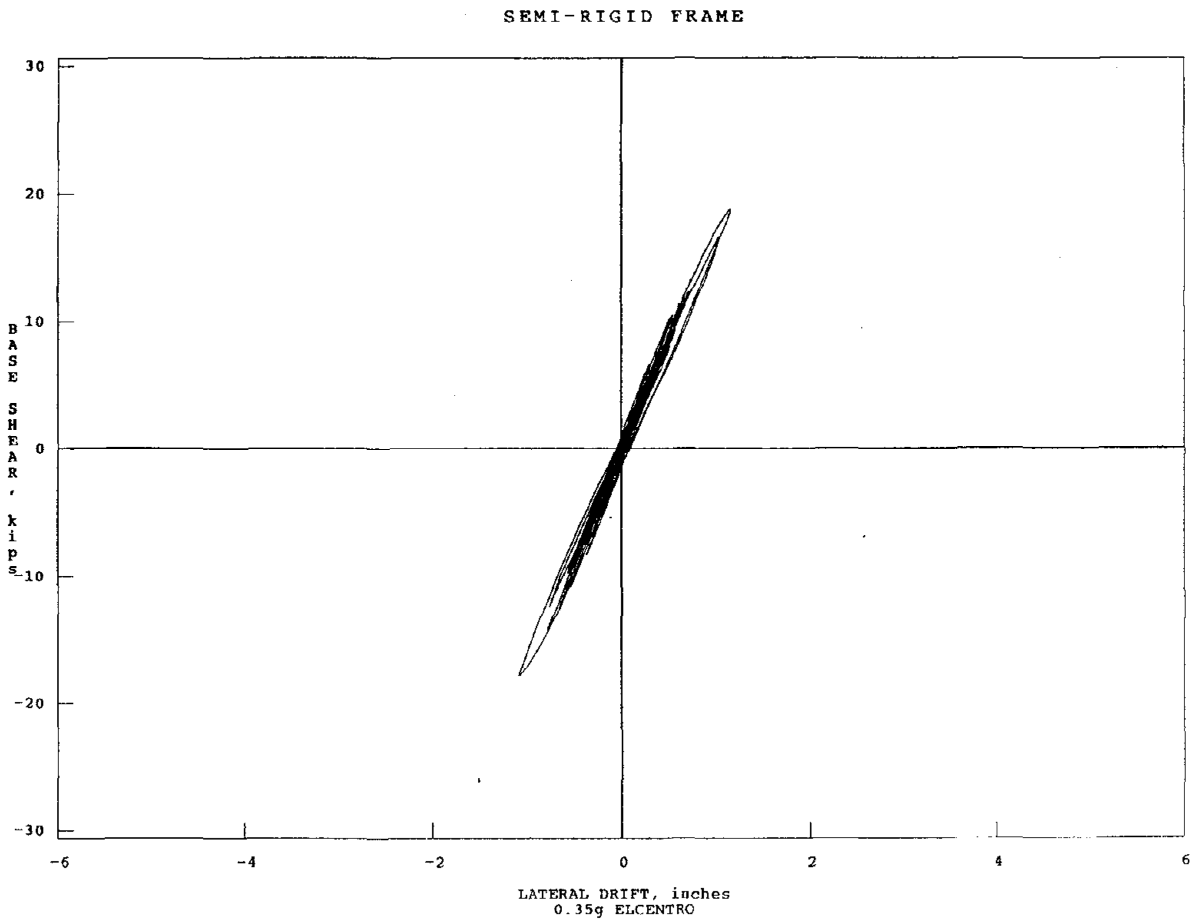
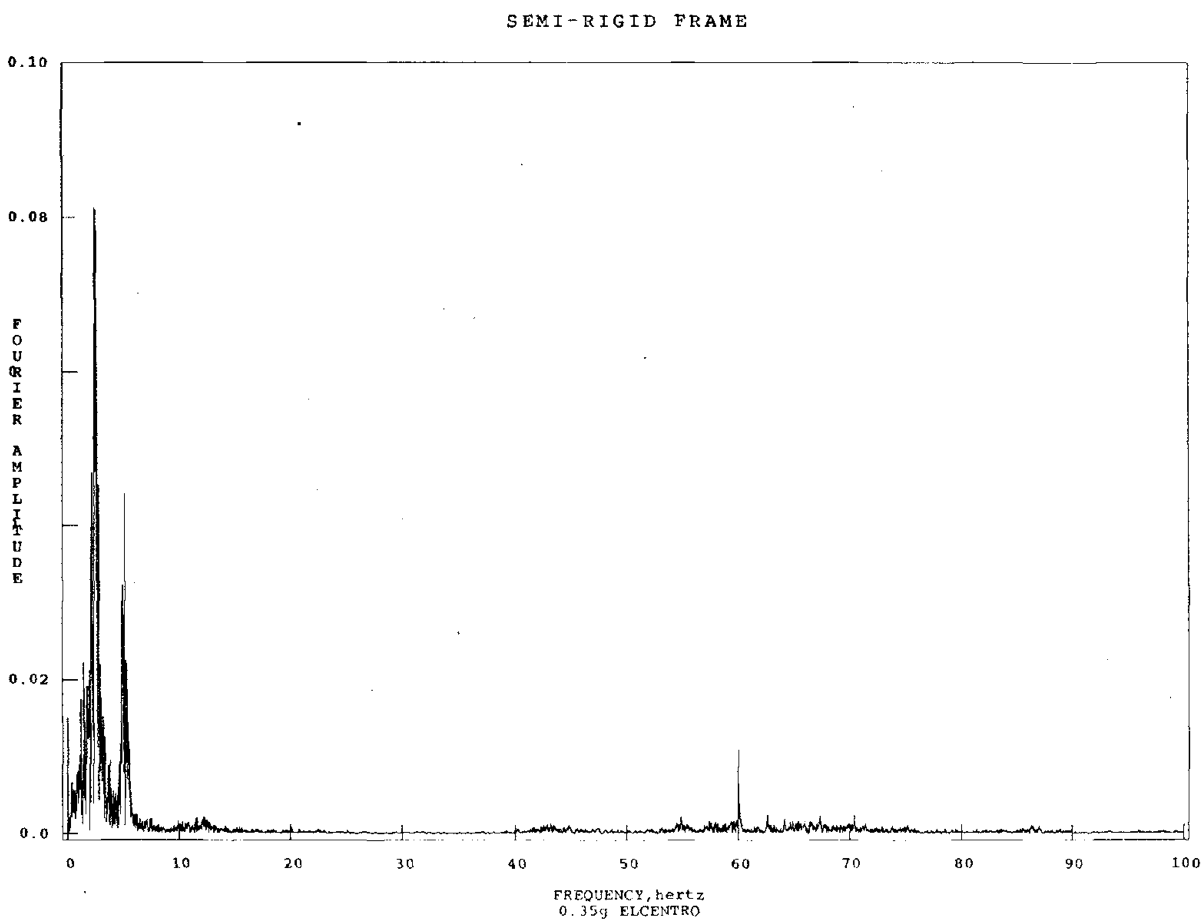


SEMI-RIGID FRAME



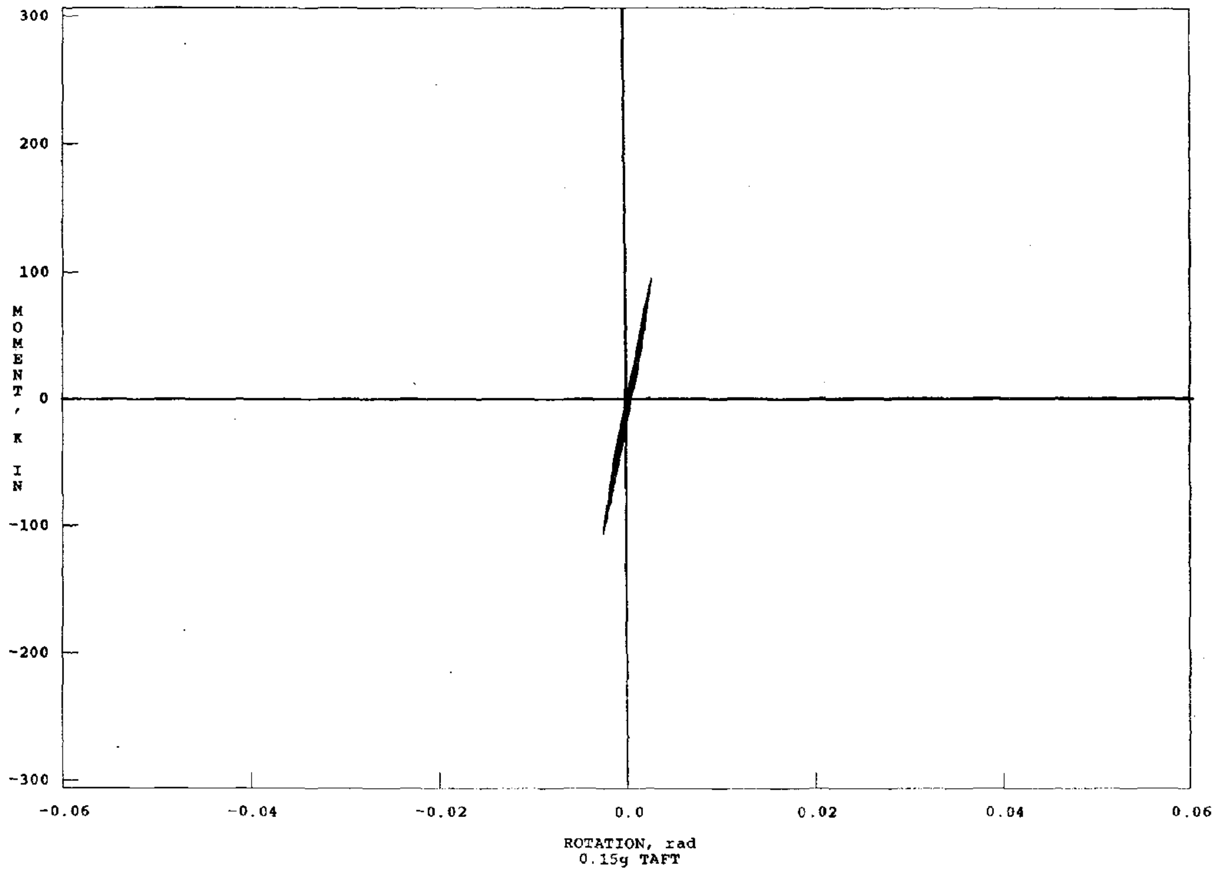


SEMI-RIGID FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.35g ELCENTRO

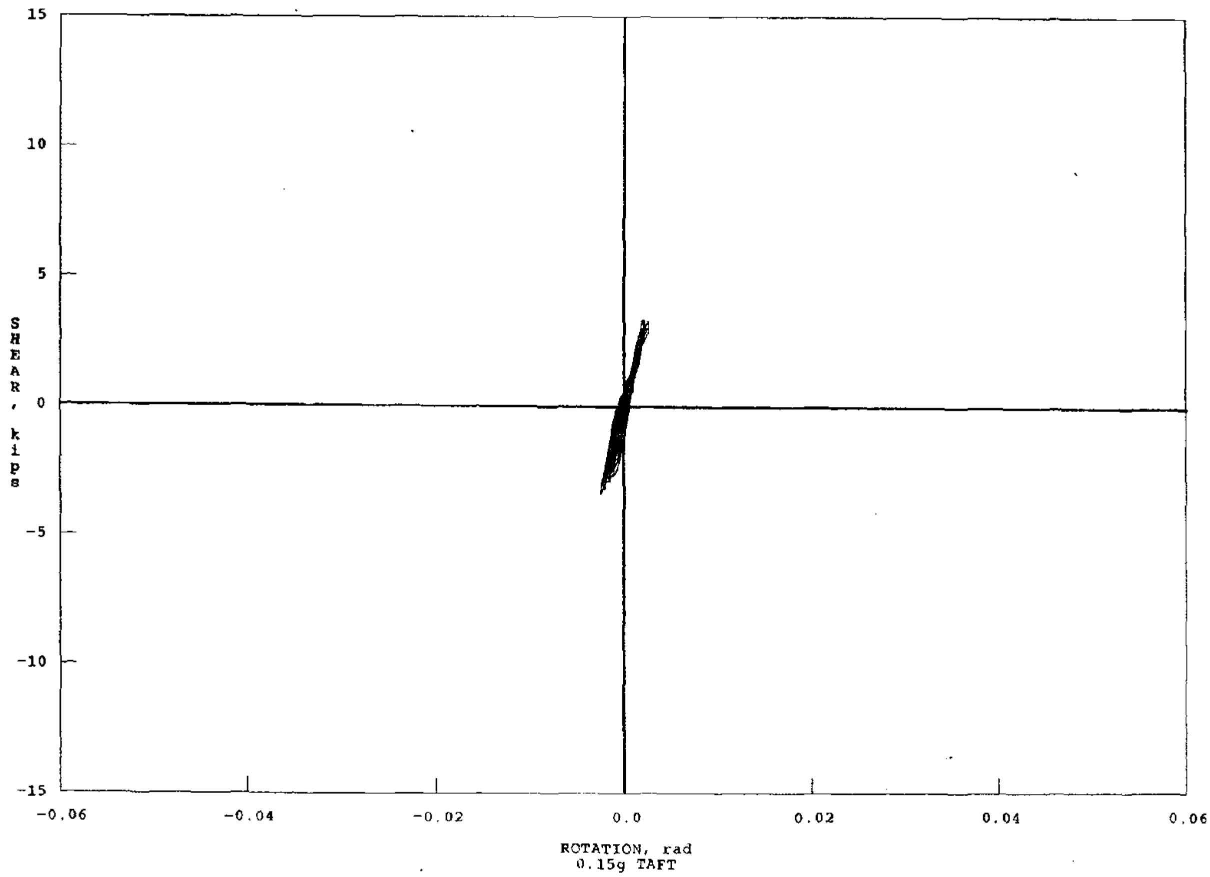


SEMI-RIGID FRAME
0.10
0.08
0.02
0.0
FOURIER AMPLITUDE
0
10
20
30
40
50
60
70
80
90
100
FREQUENCY, hertz
0.35g ELCENTRO

SEMI-RIGID FRAME

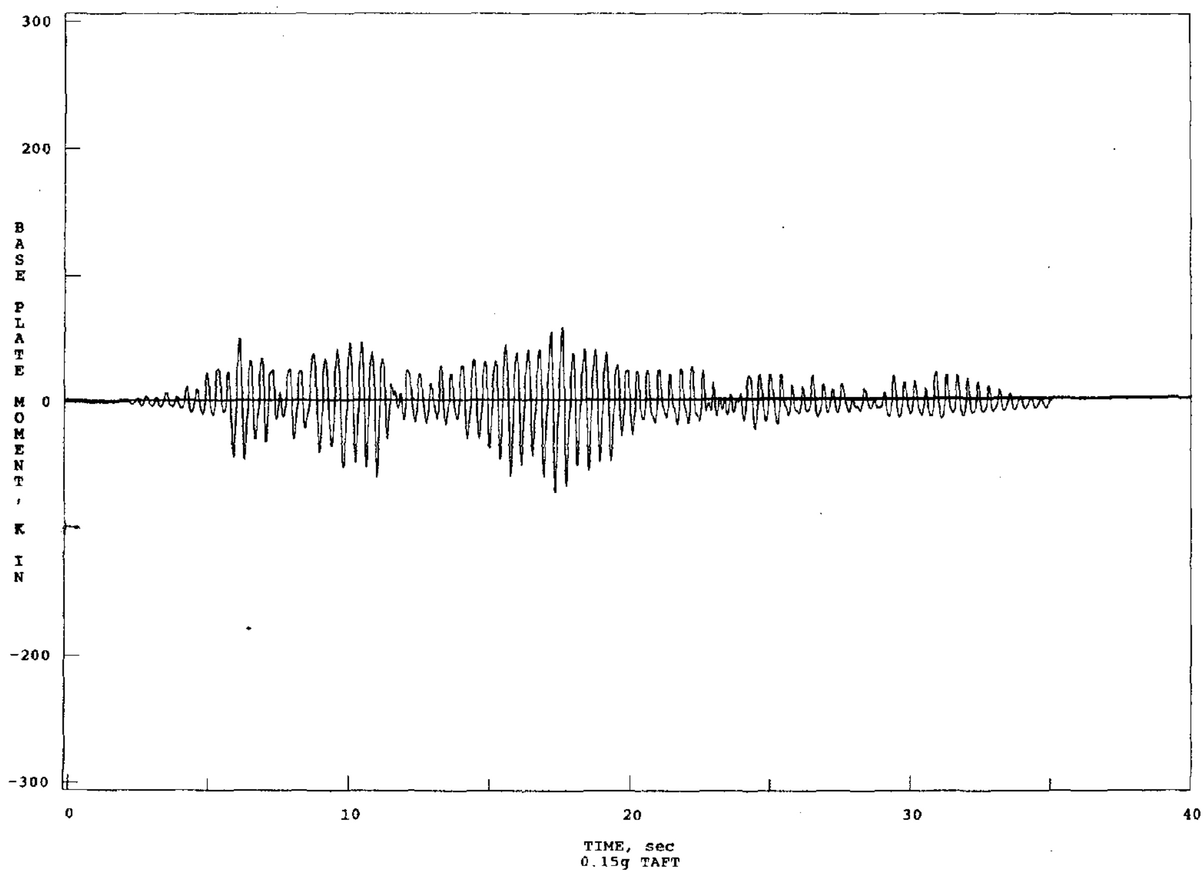


SEMI-RIGID FRAME



SEMI-RIGID FRAME

SEMI-RIGID FRAME



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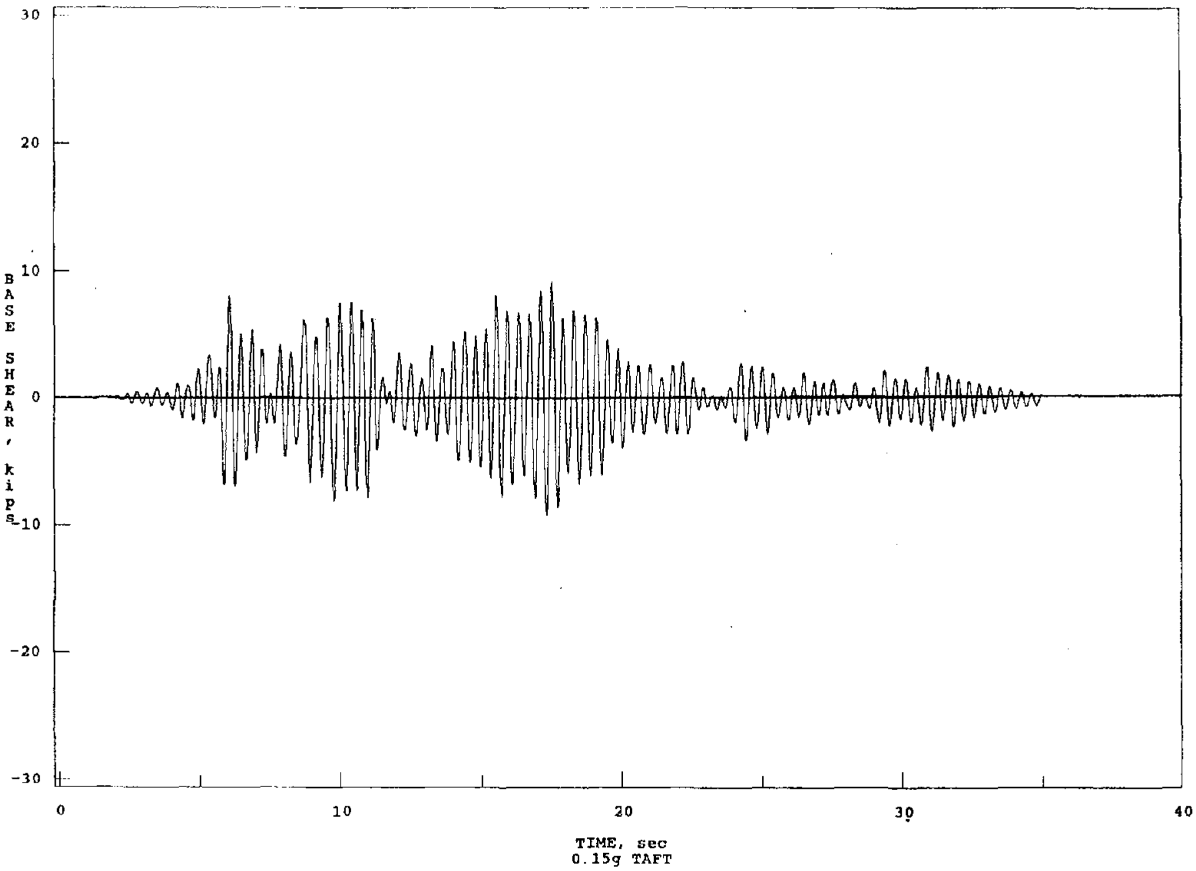

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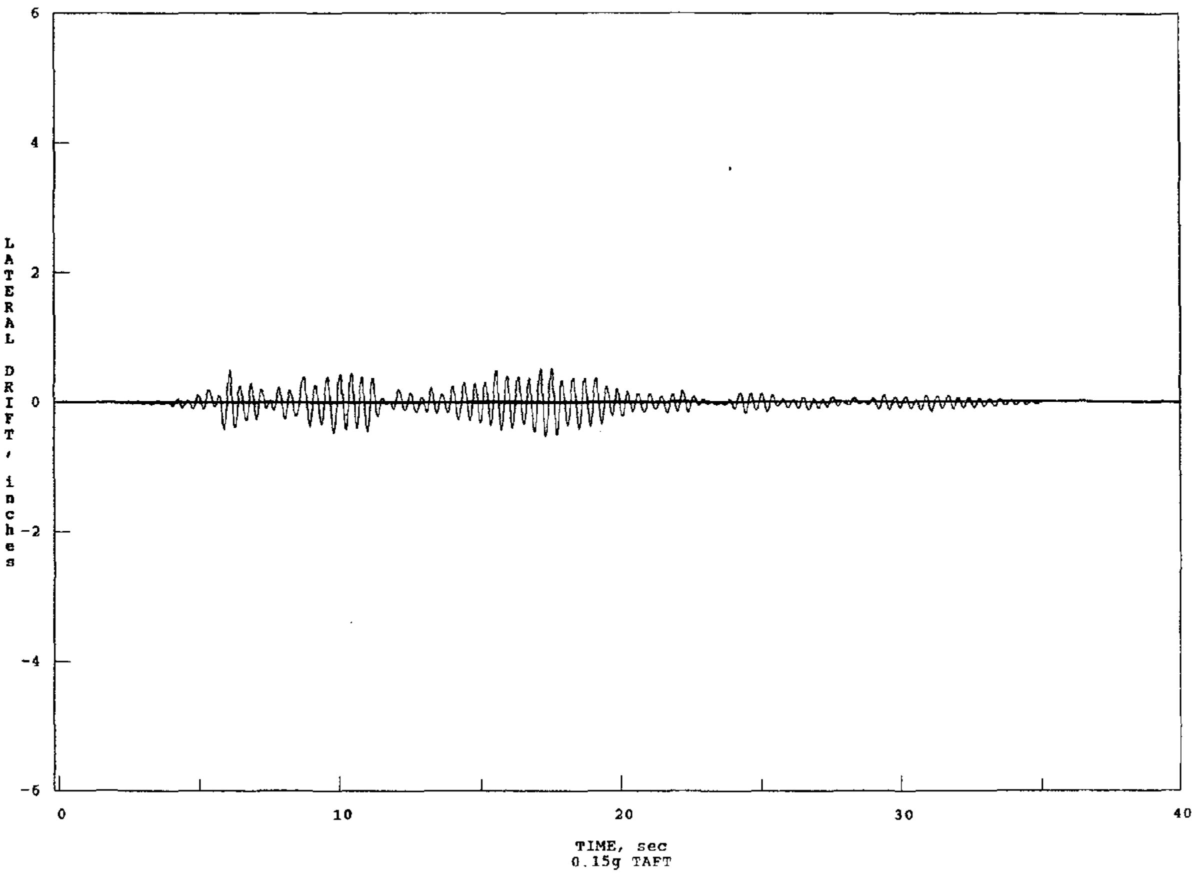
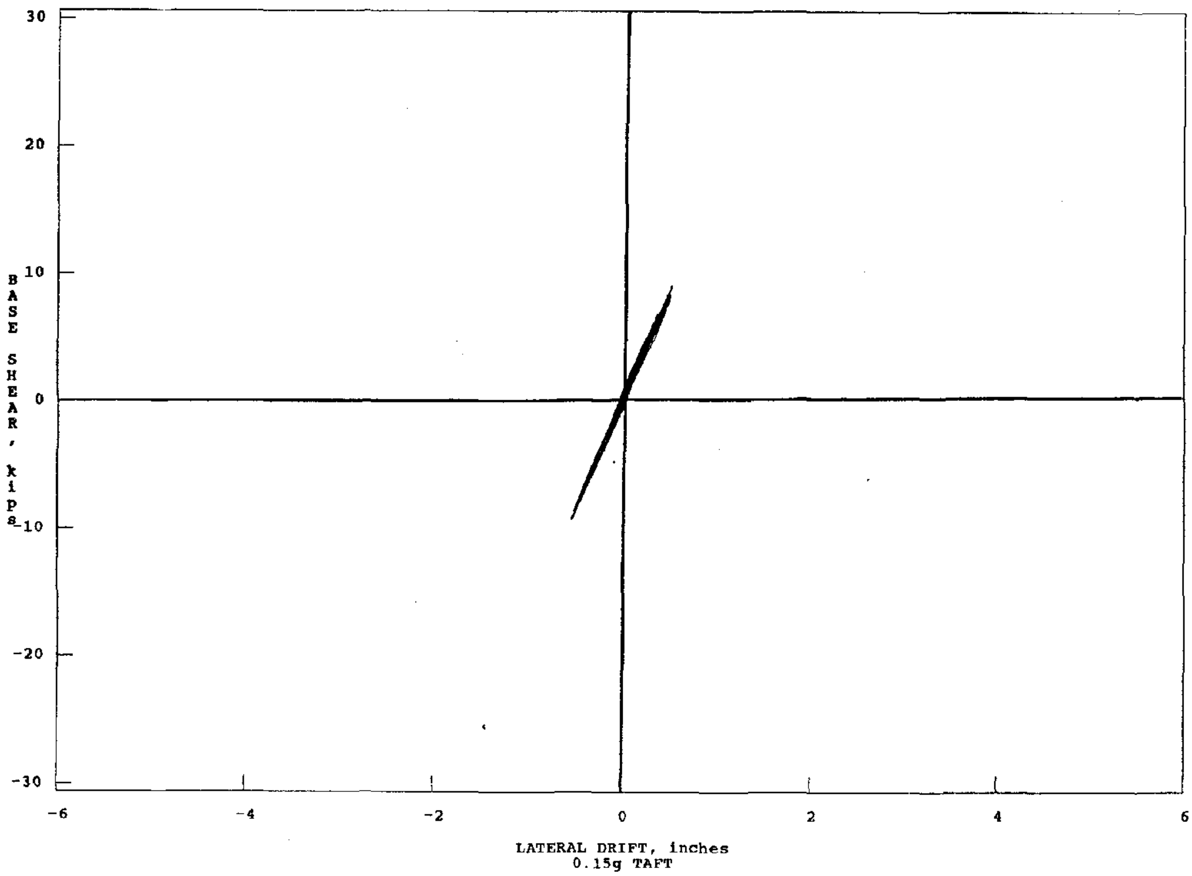
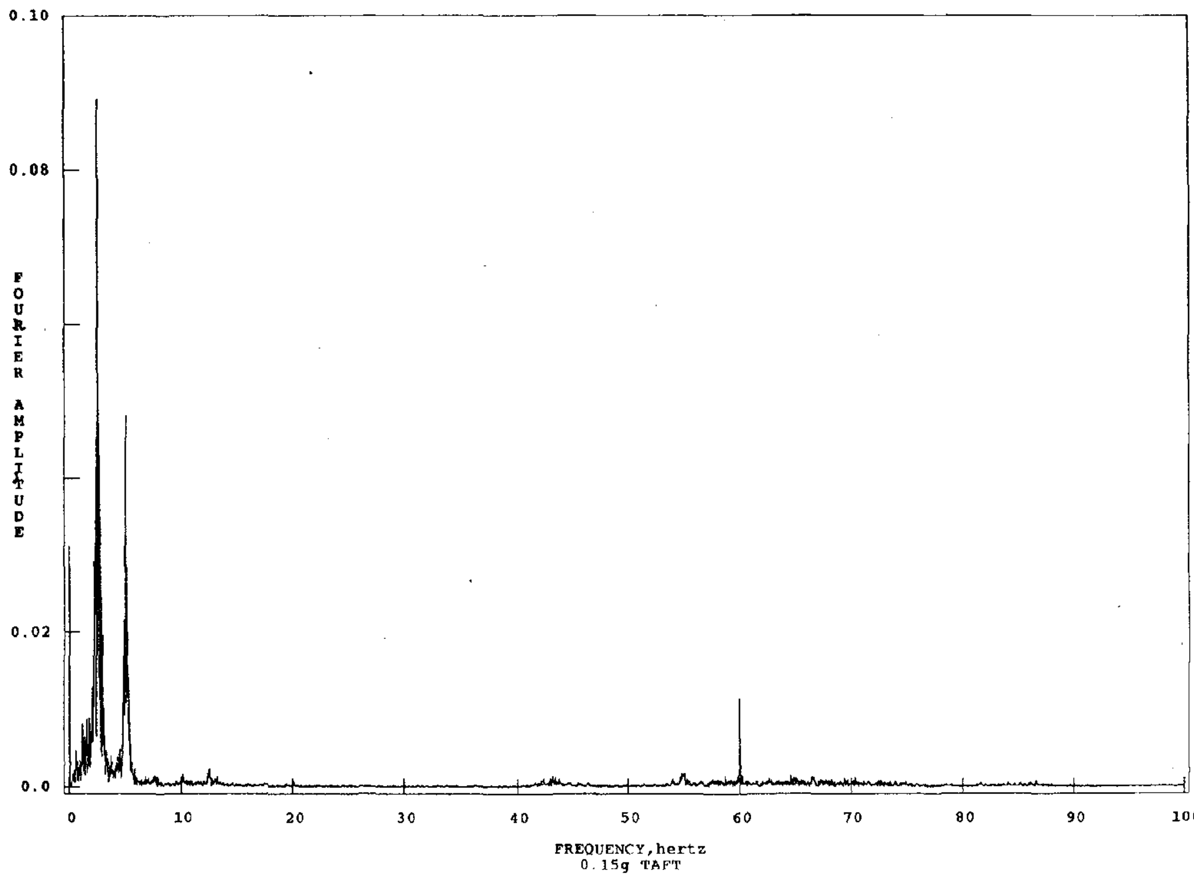


SEMI-RIGID FRAME



SEMI-RIGID FRAME



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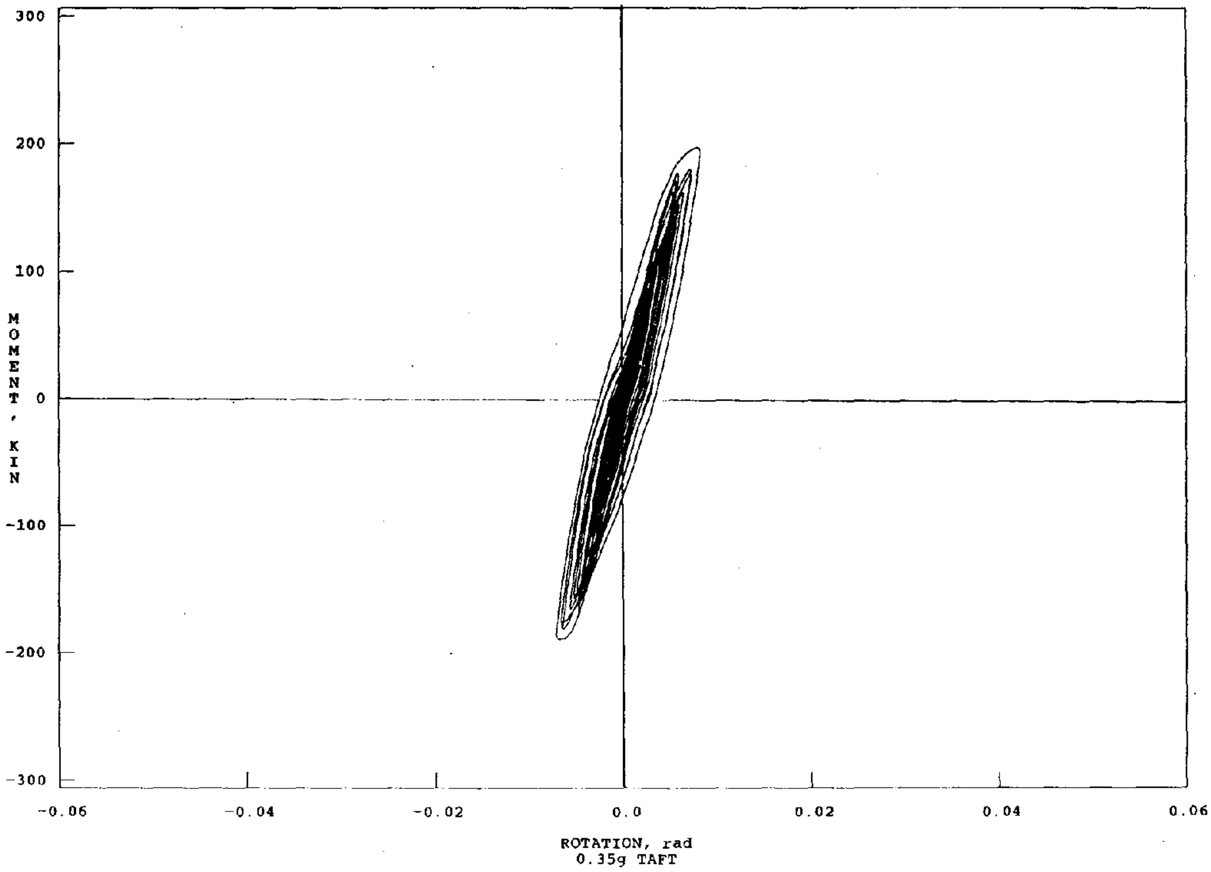

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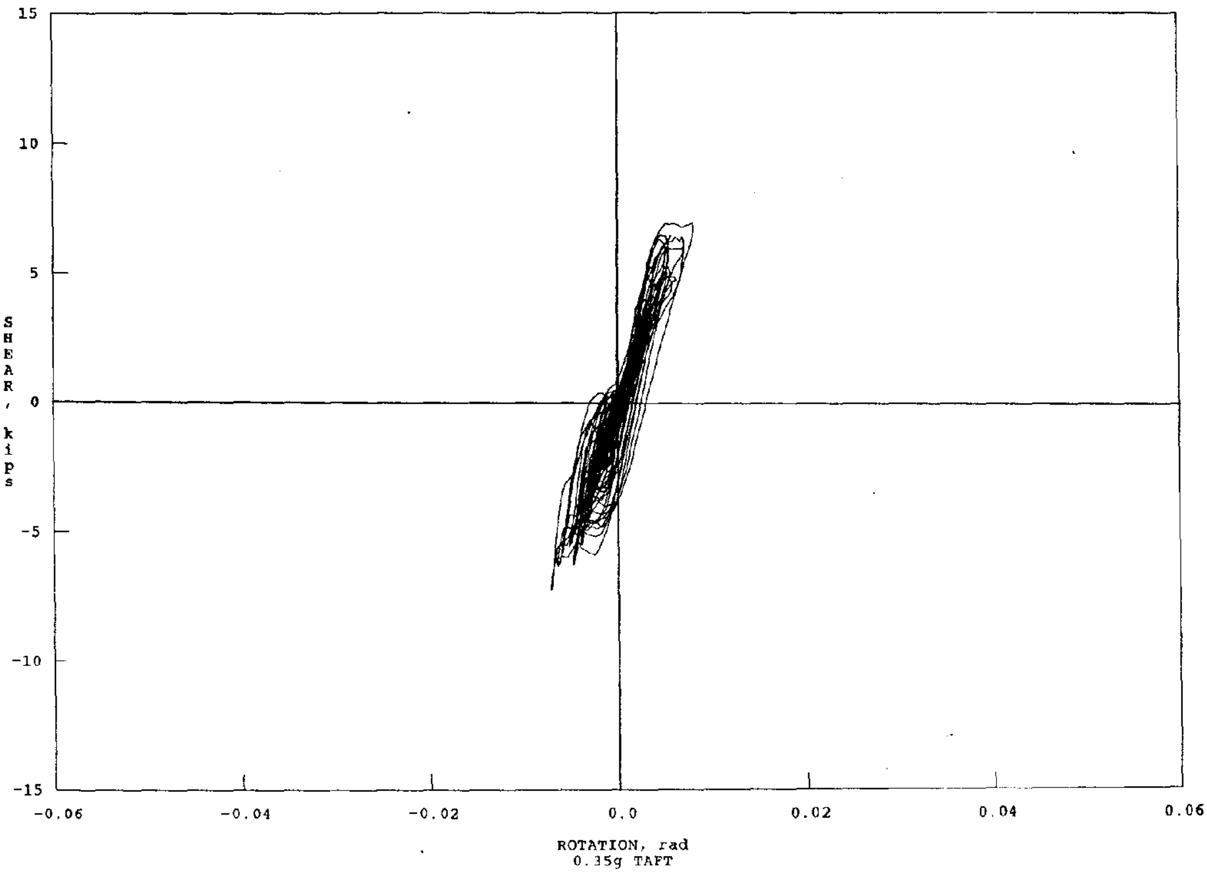

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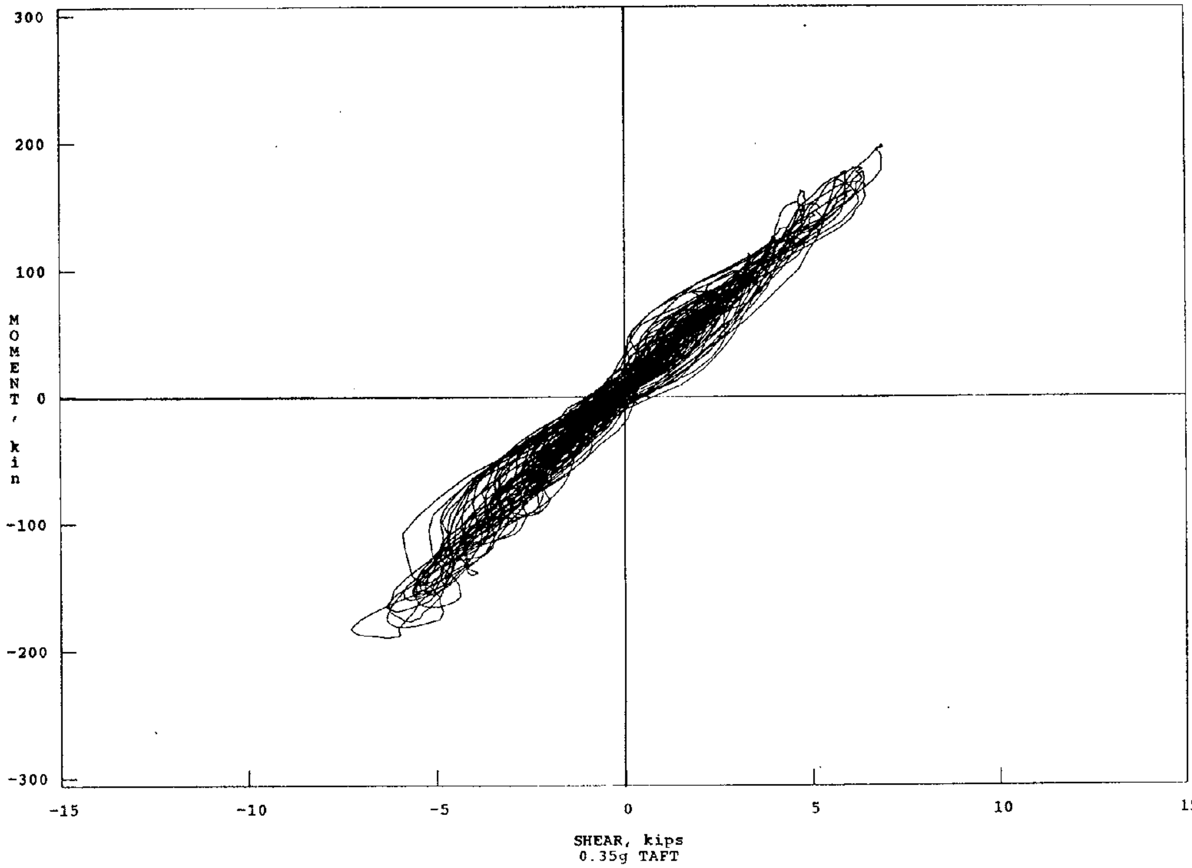

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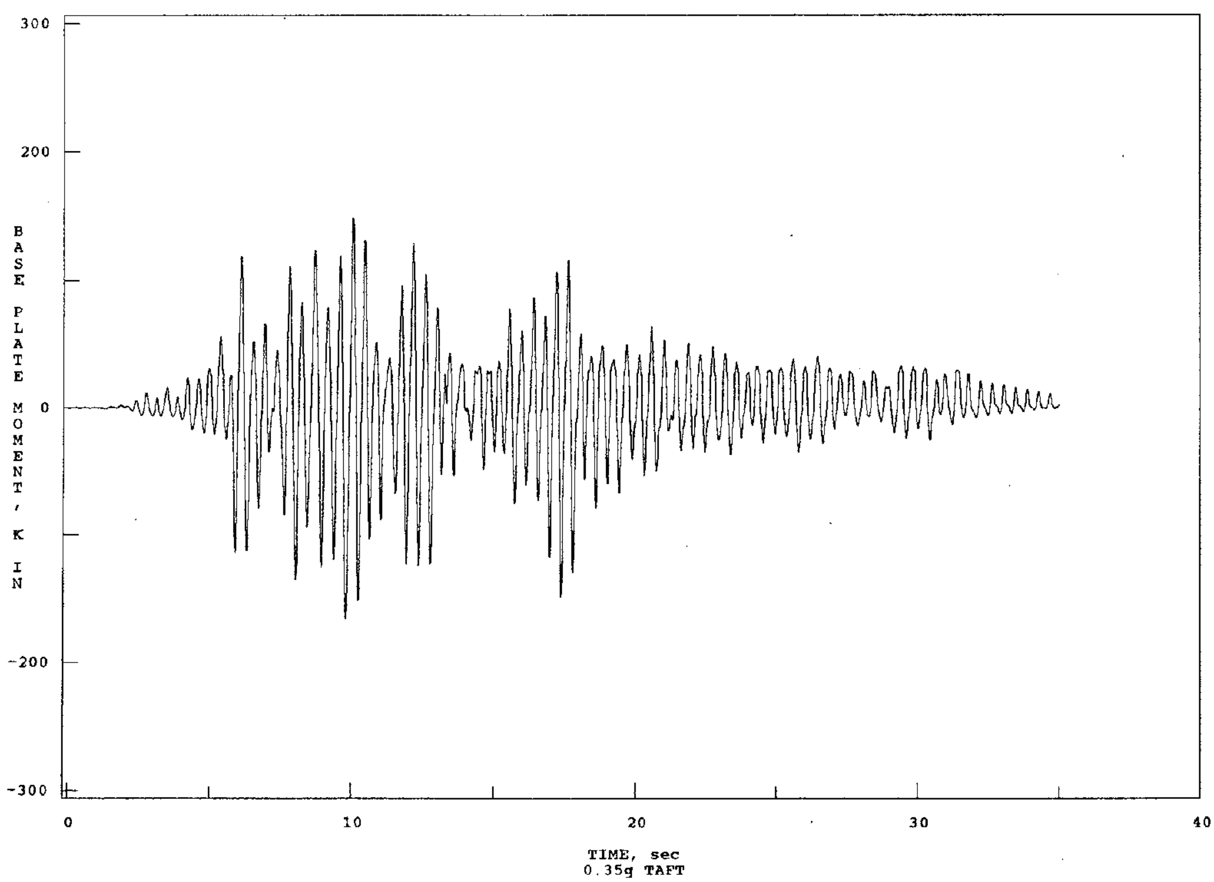

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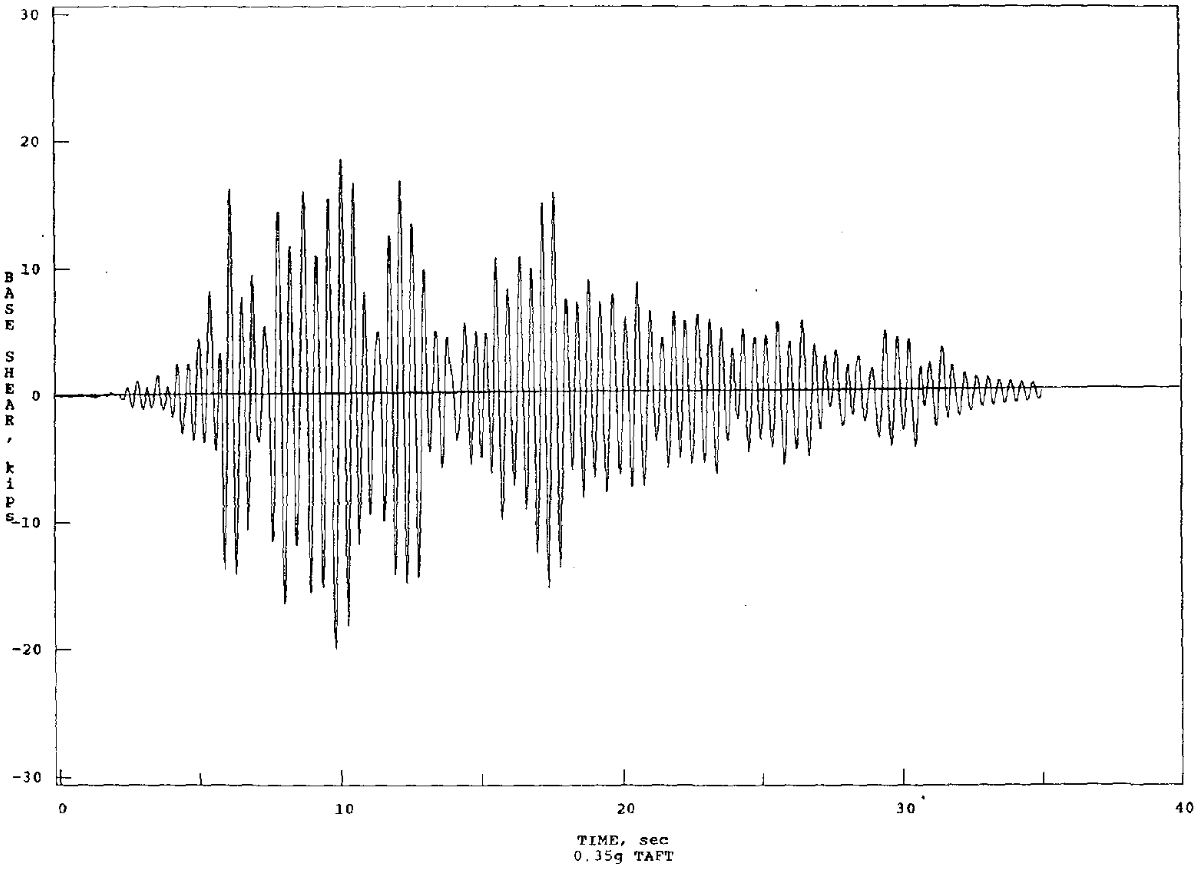
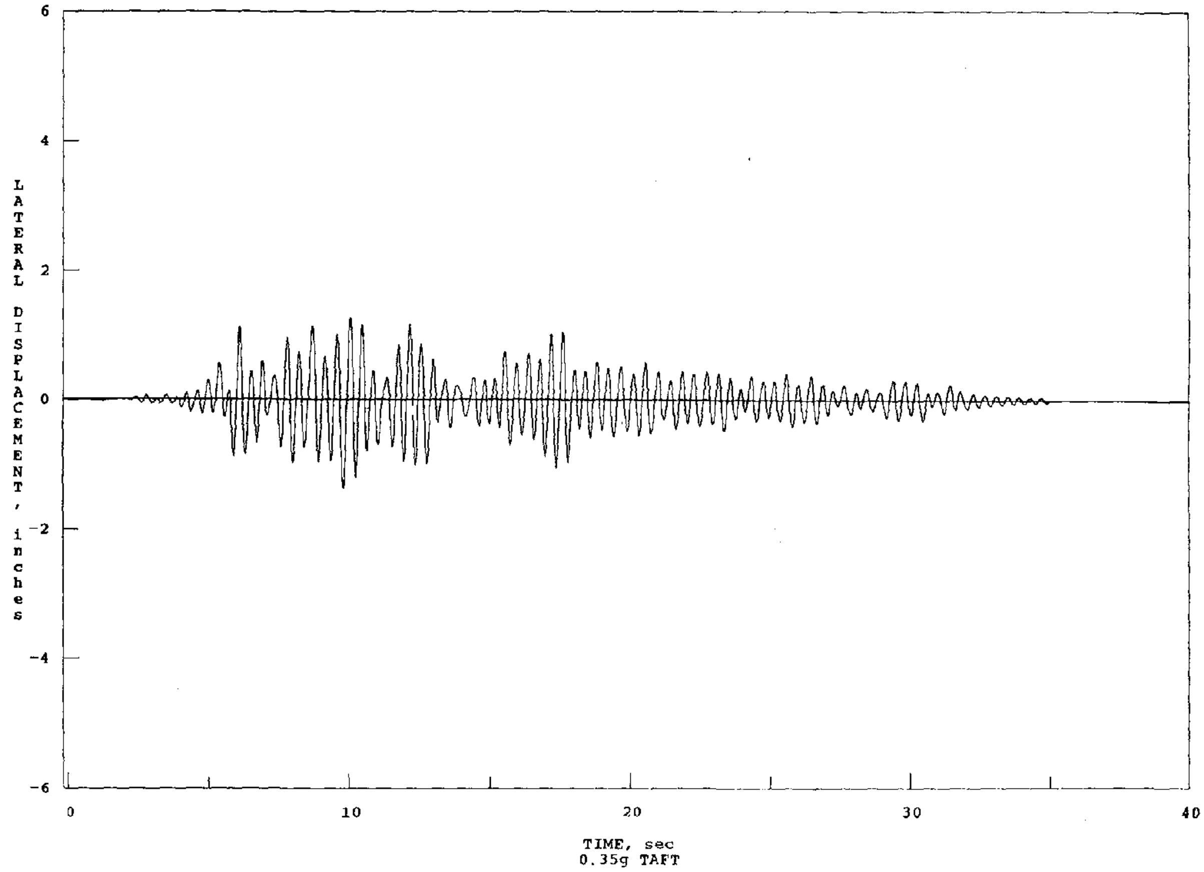


SEMI-RIGID FRAME



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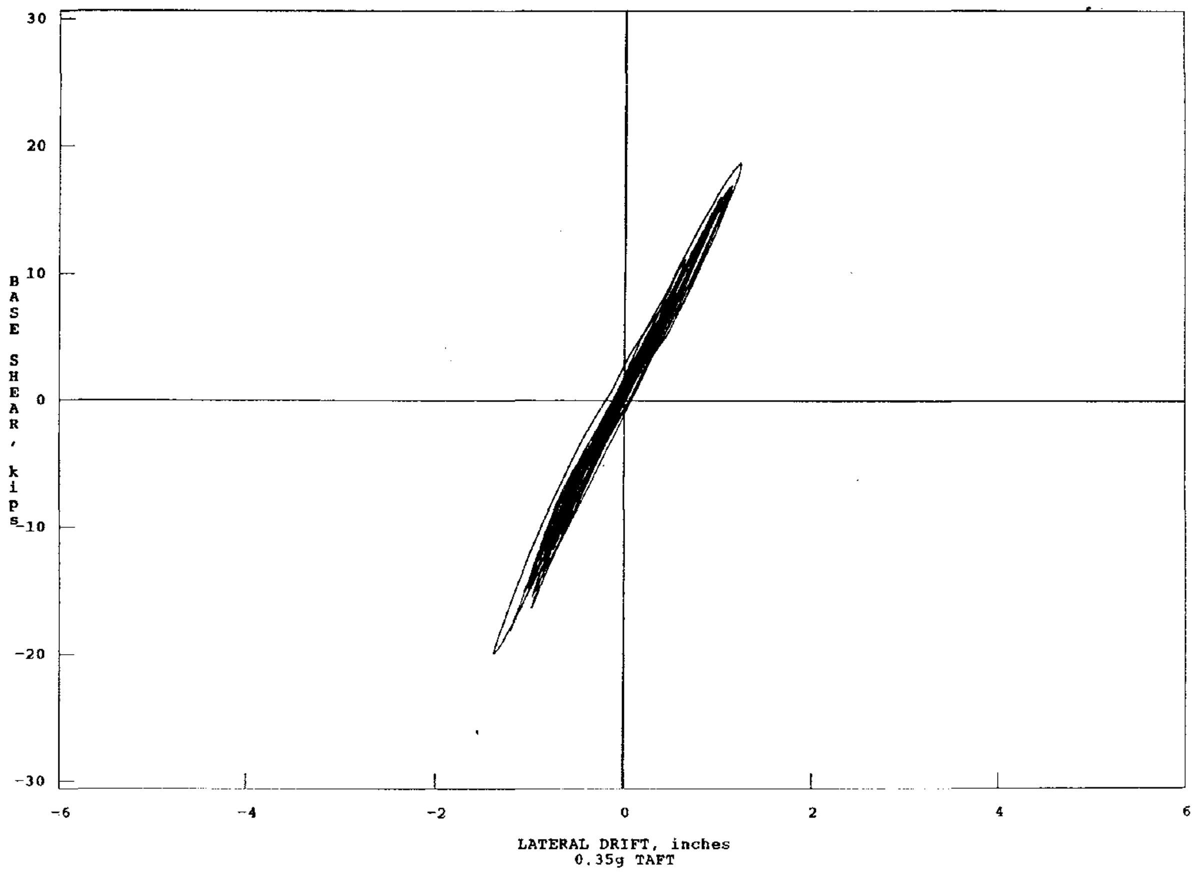

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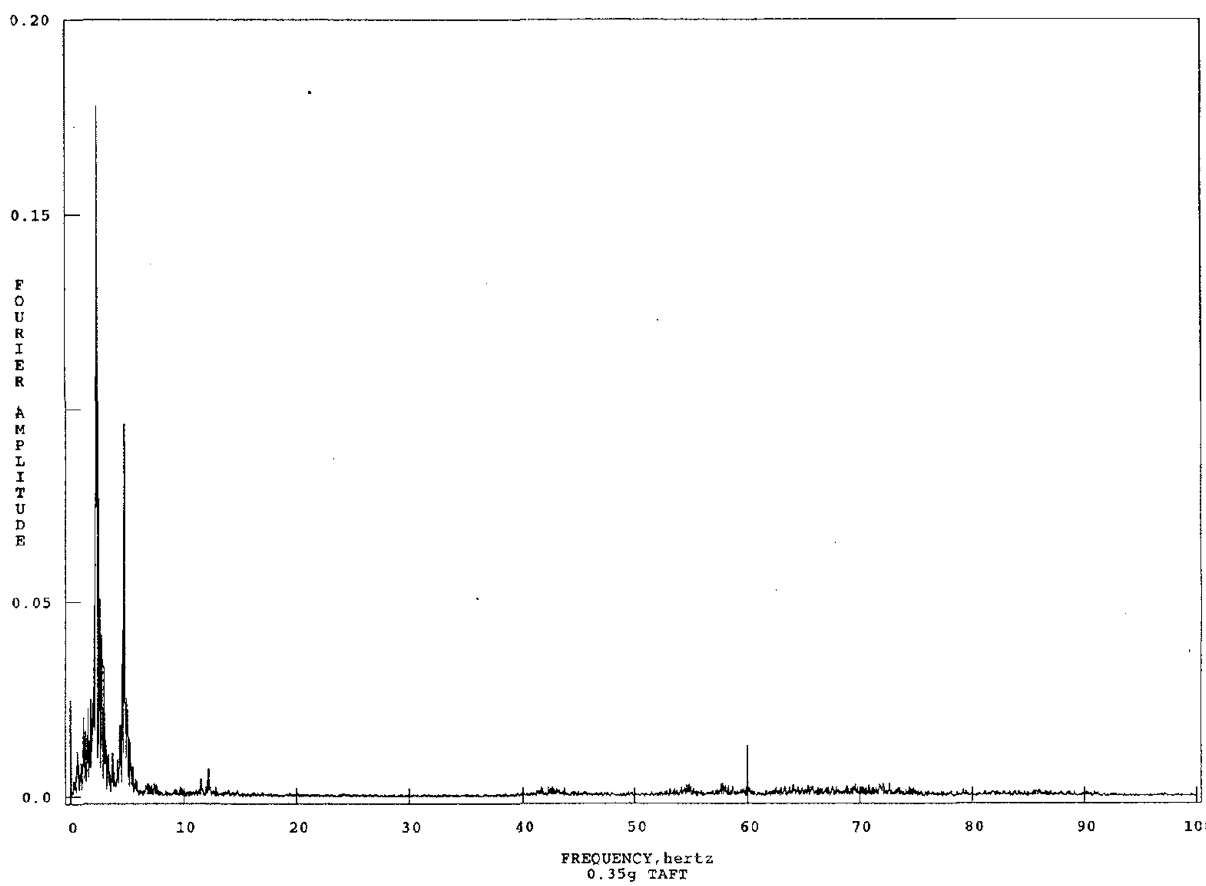

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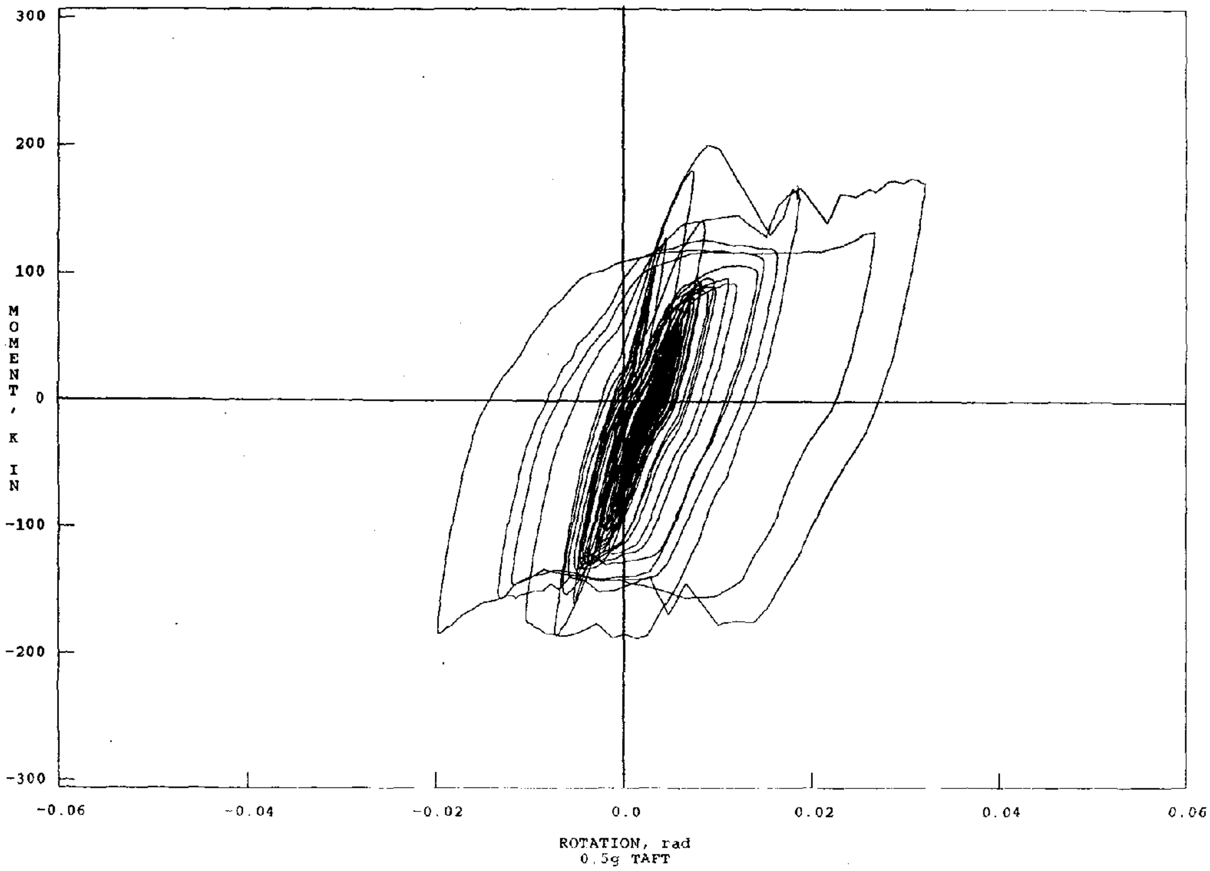

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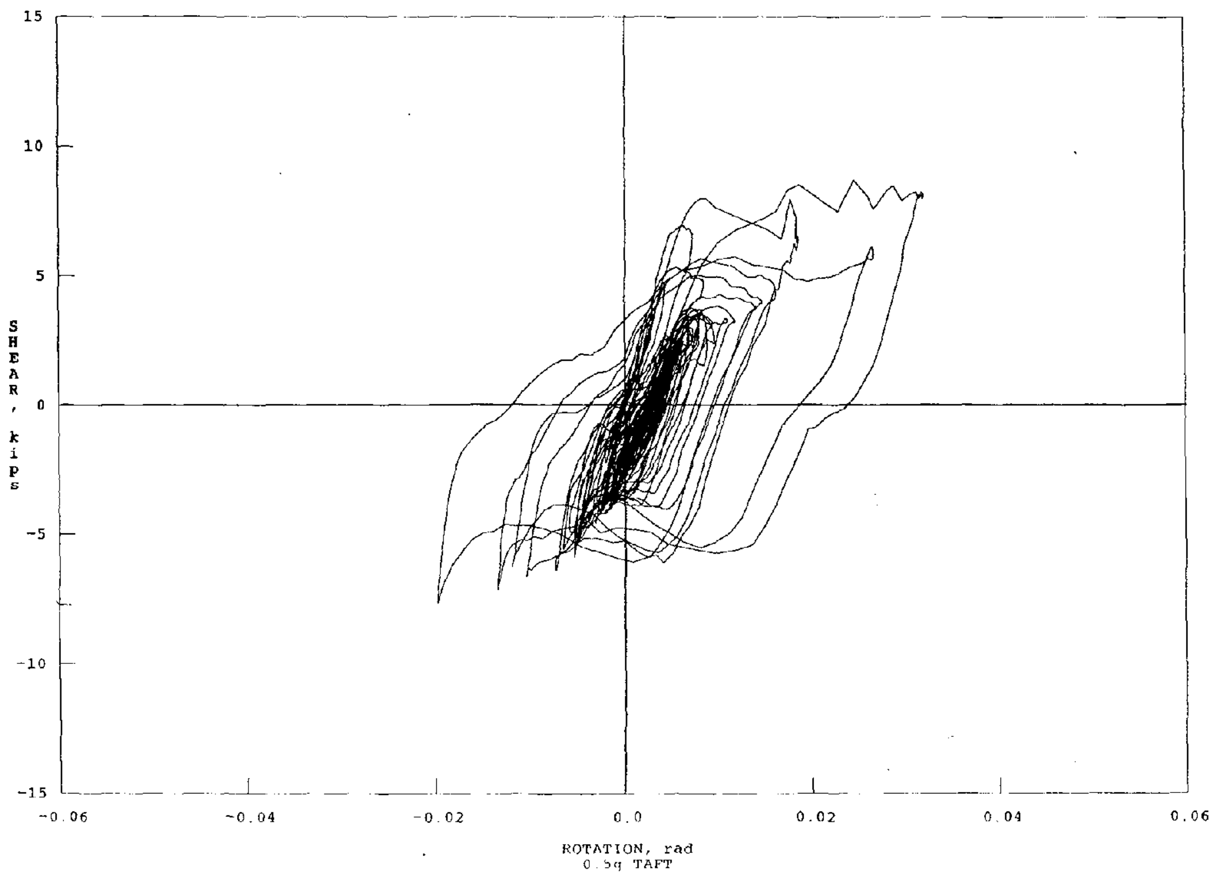

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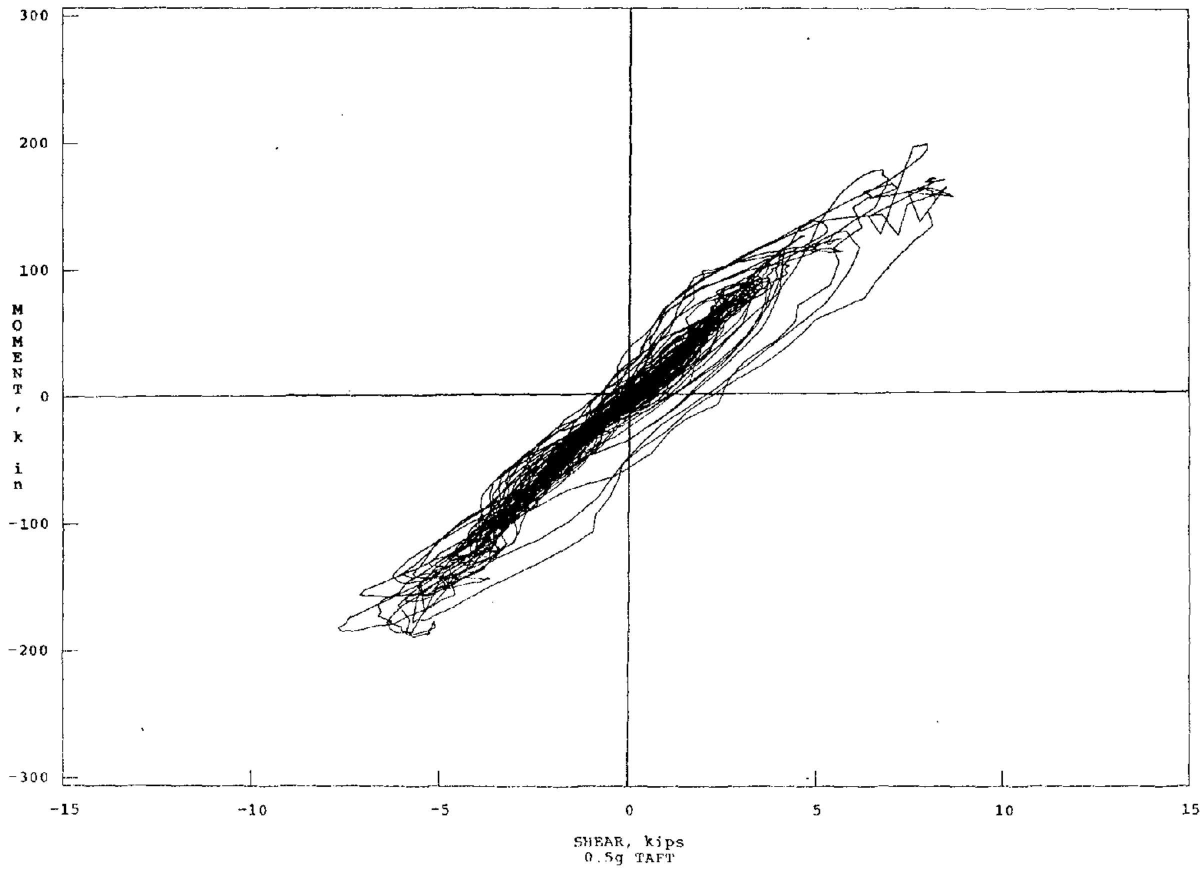

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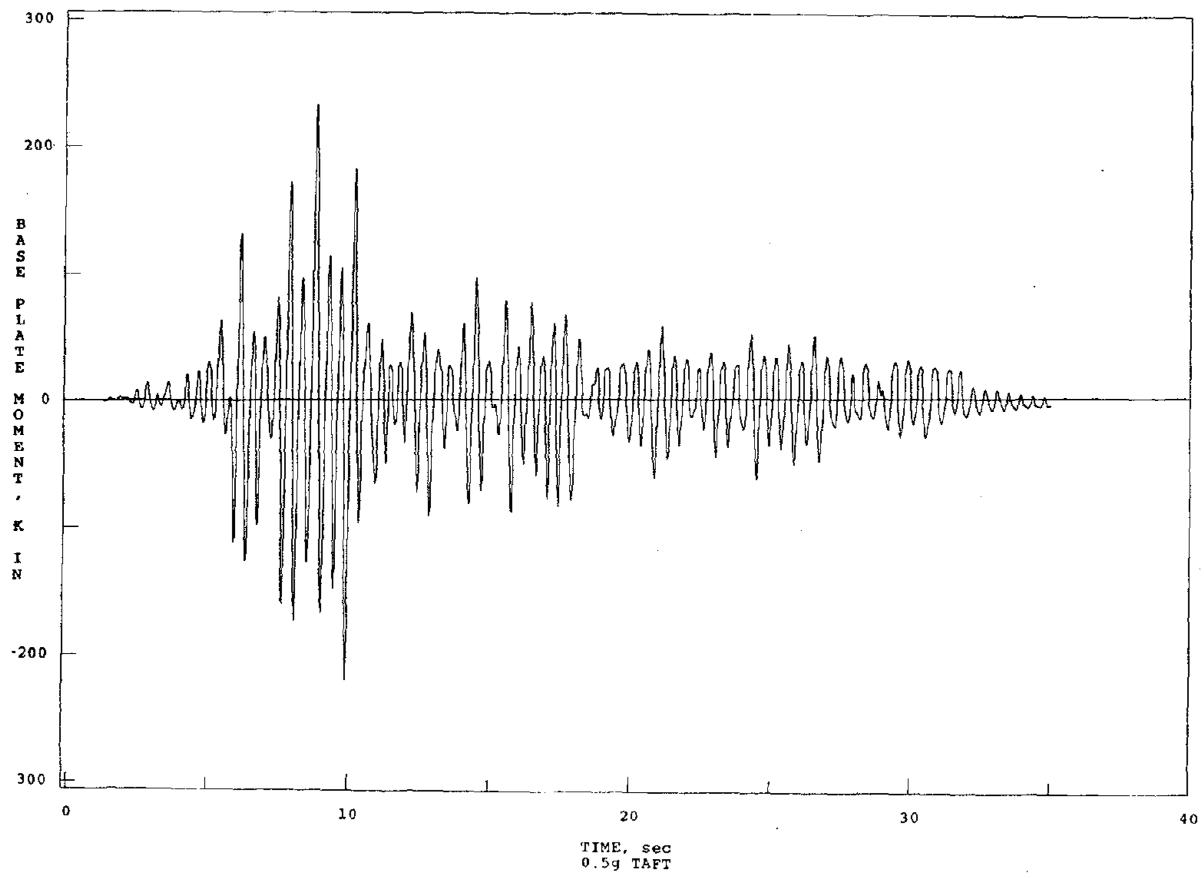

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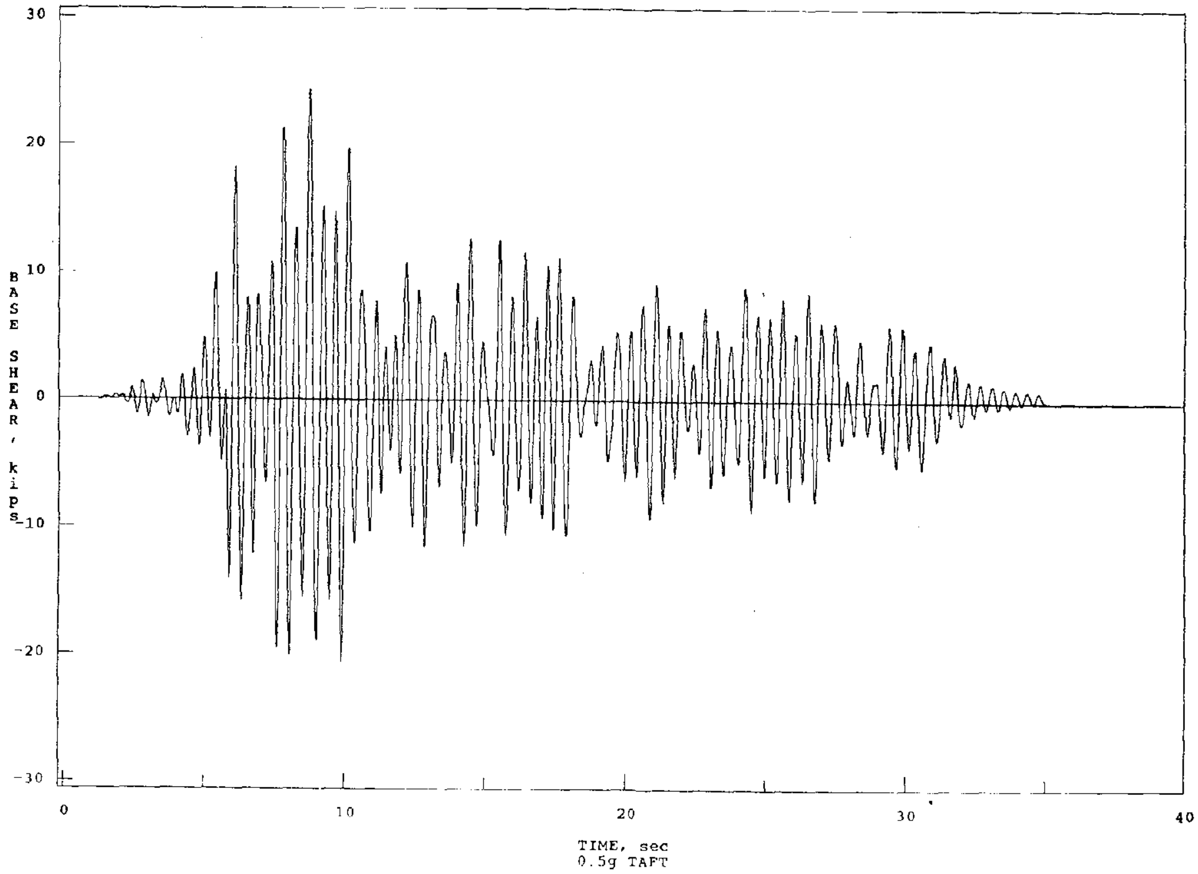

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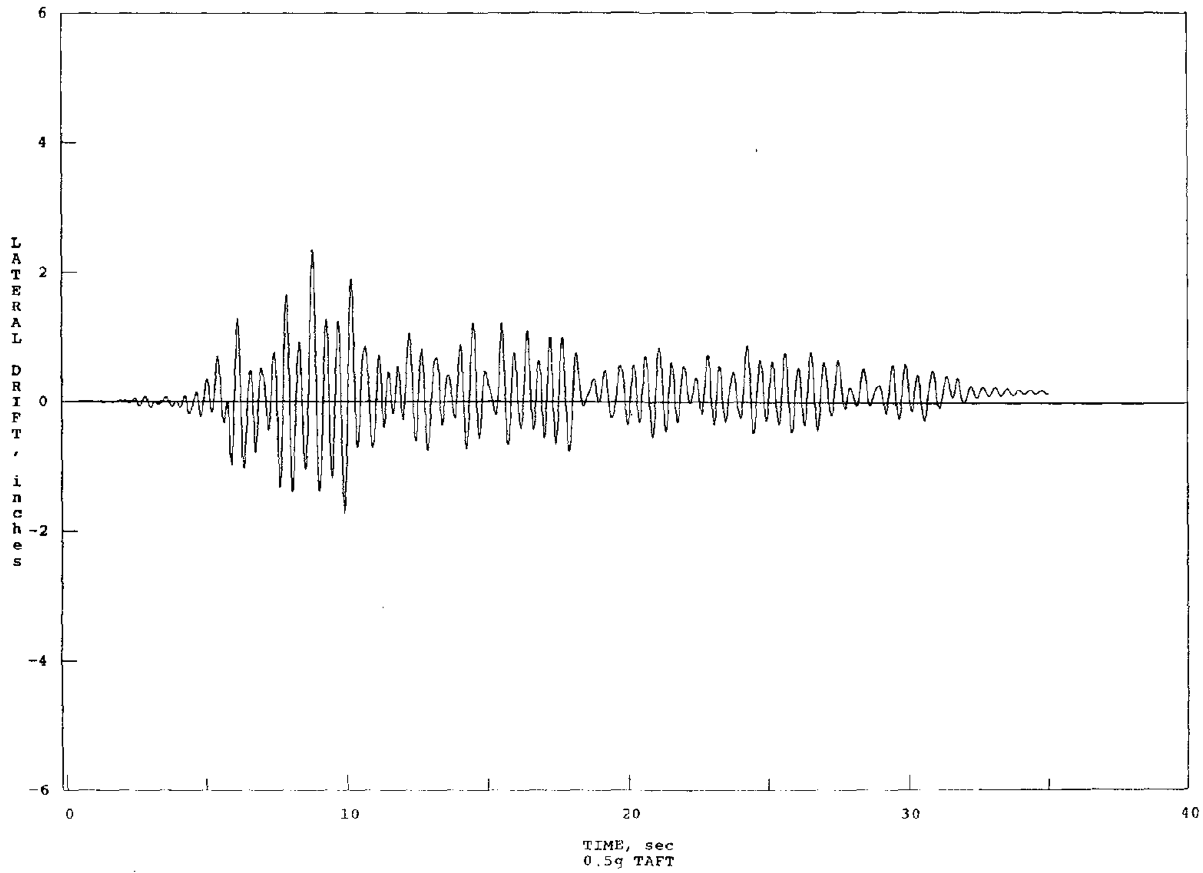
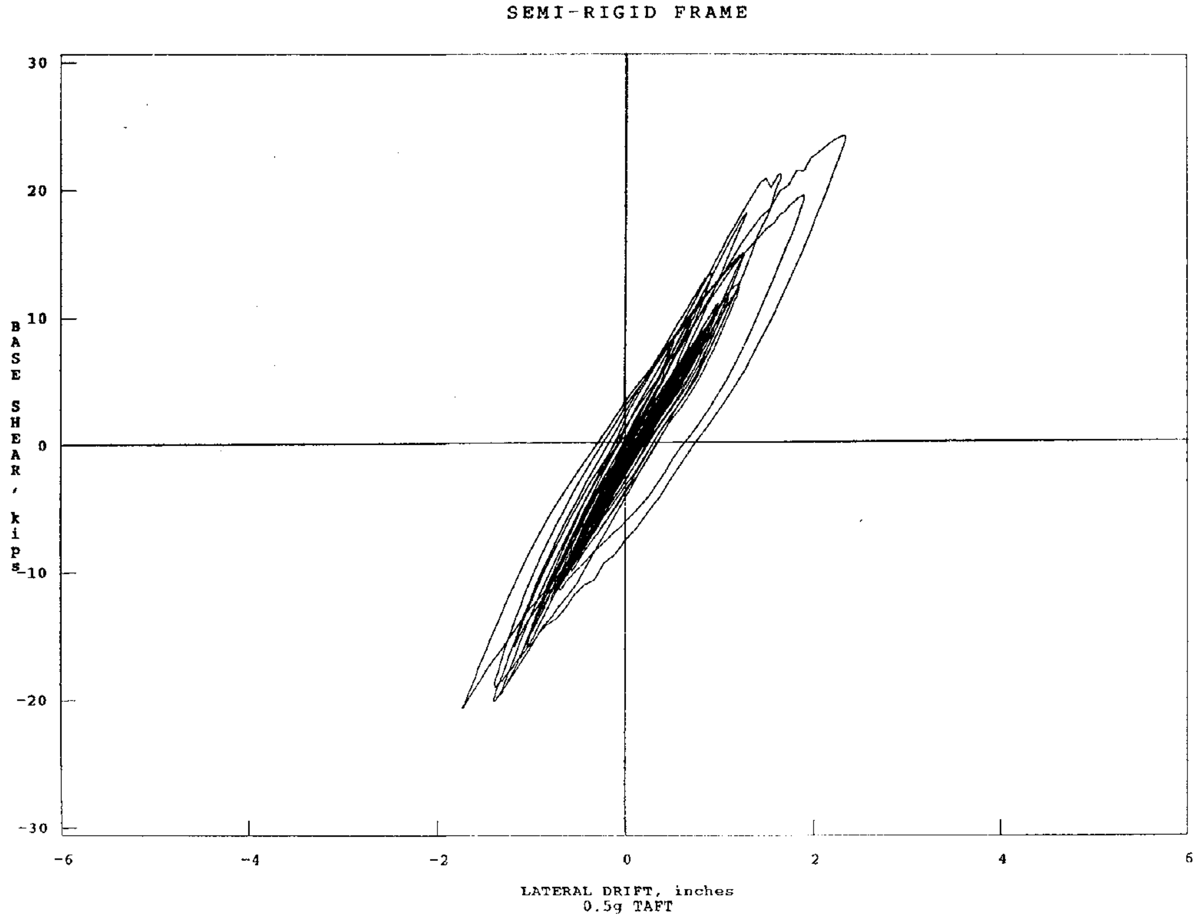
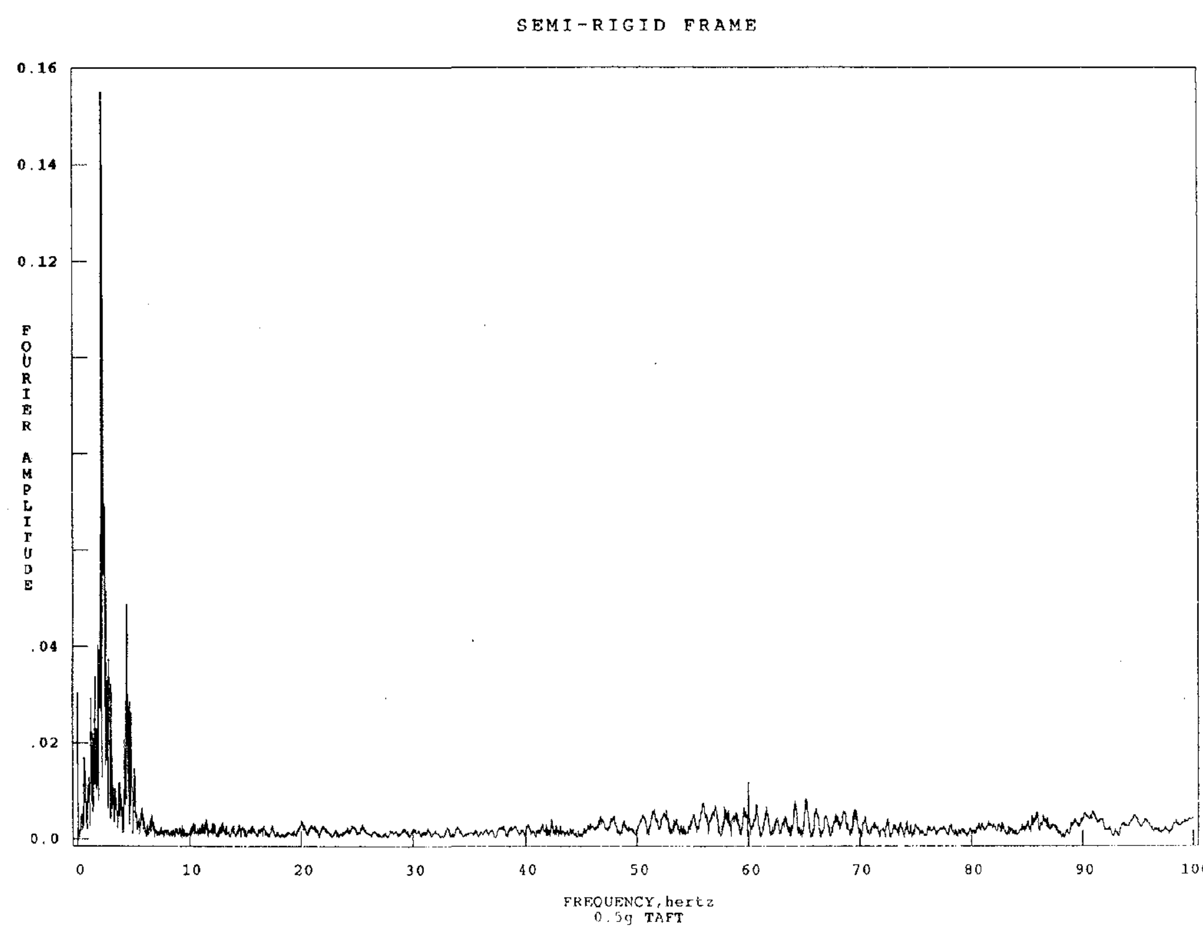
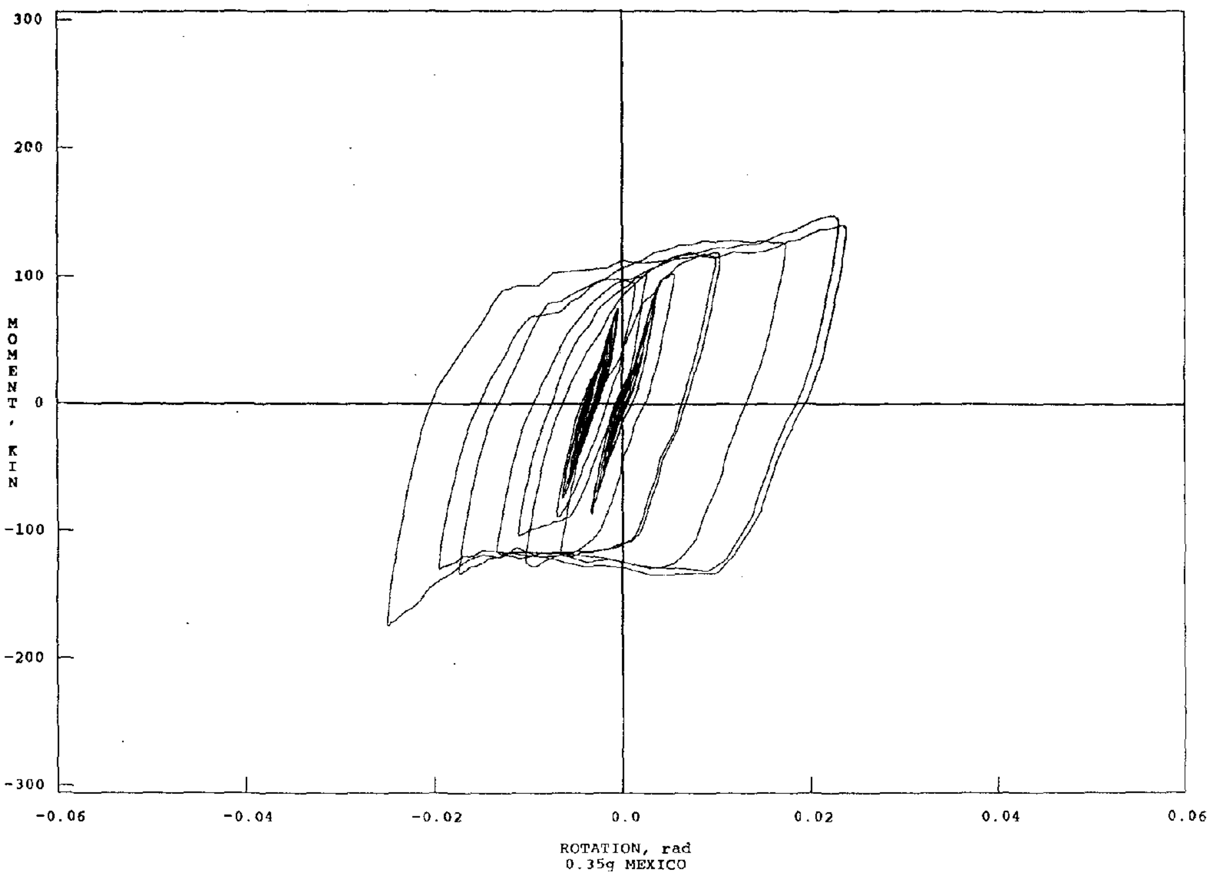



SEMI-RIGID FRAME
BASE SHEAR, kips
30
20
10
0
-10
-20
-30
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.5g TAFT


SEMI-RIGID FRAME
FOURIER AMPLITUDE
0.16
0.14
0.12
.04
.02
0.0
0
10
20
30
40
50
60
70
80
90
100
FREQUENCY, hertz
0.5g TAFT

SEMI-RIGID FRAME



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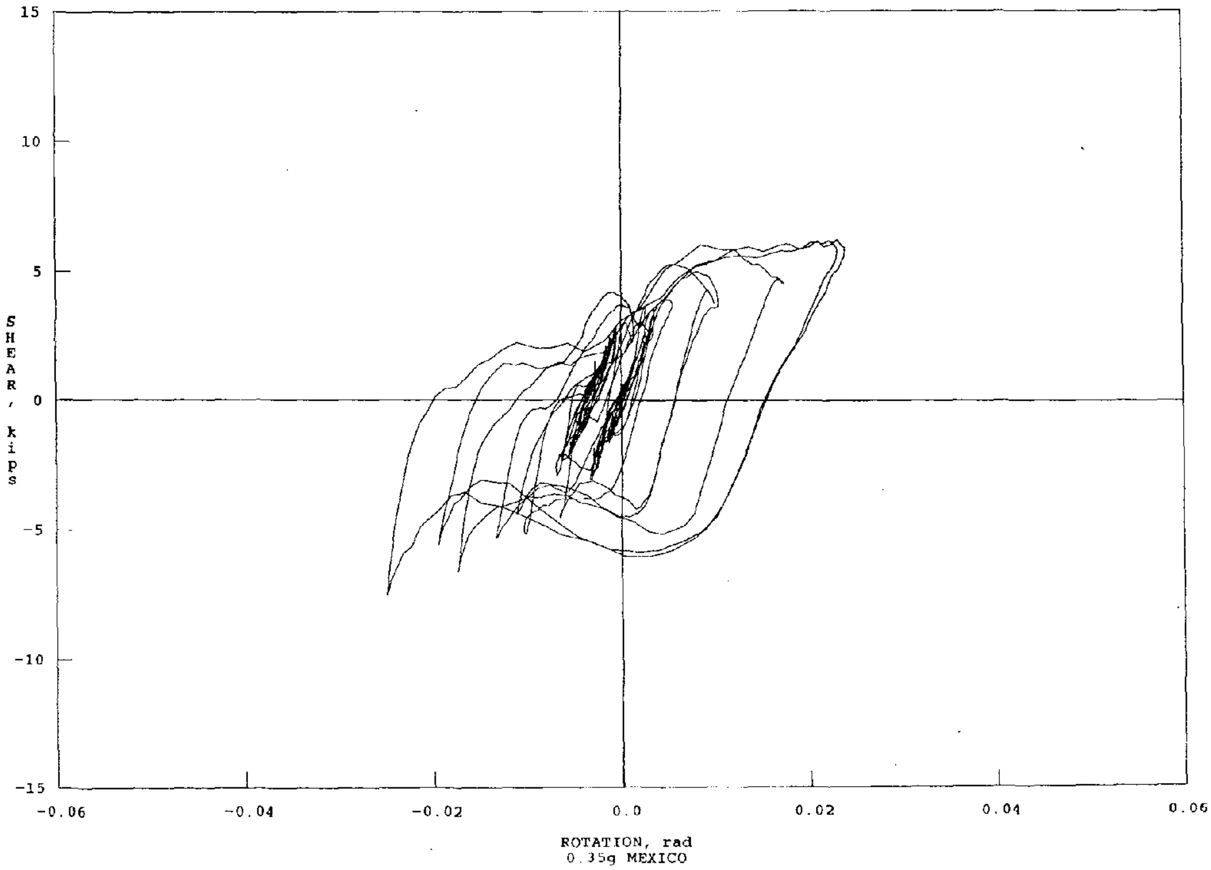

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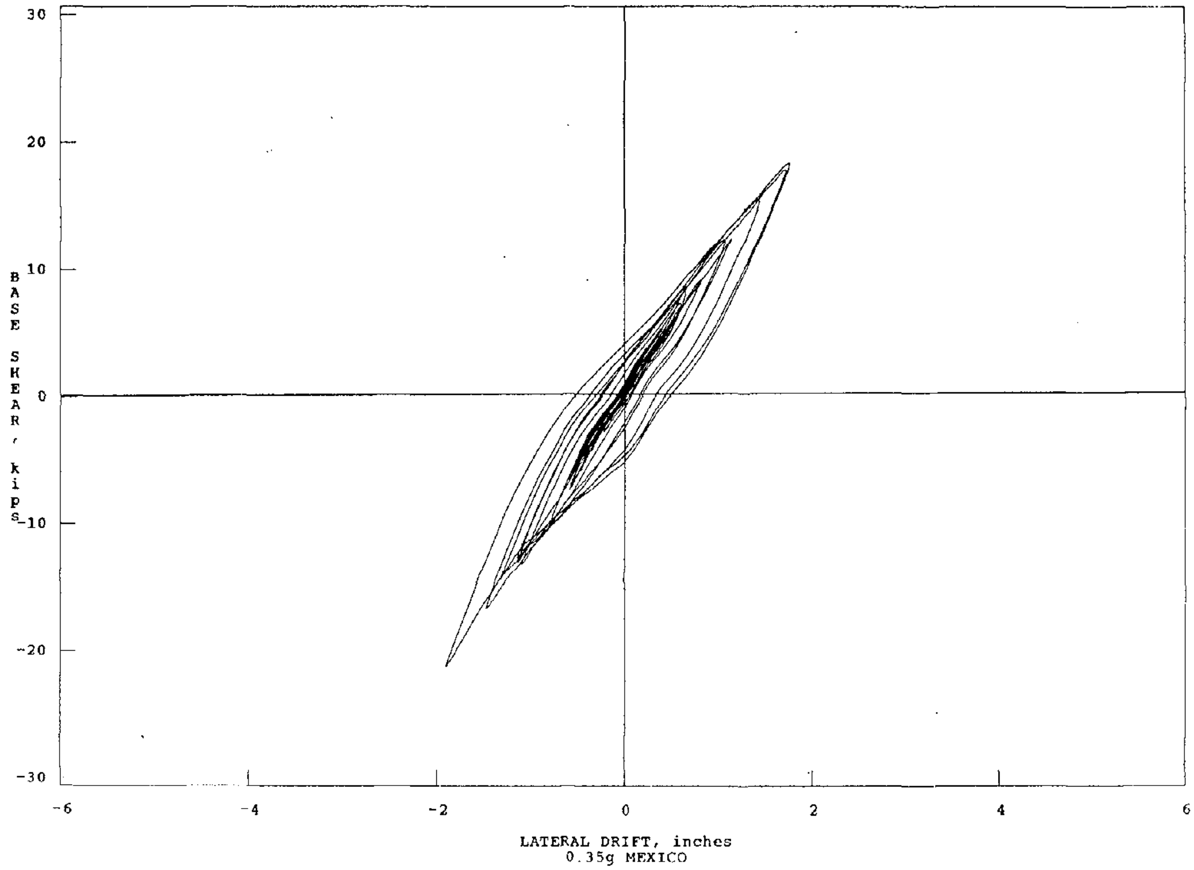

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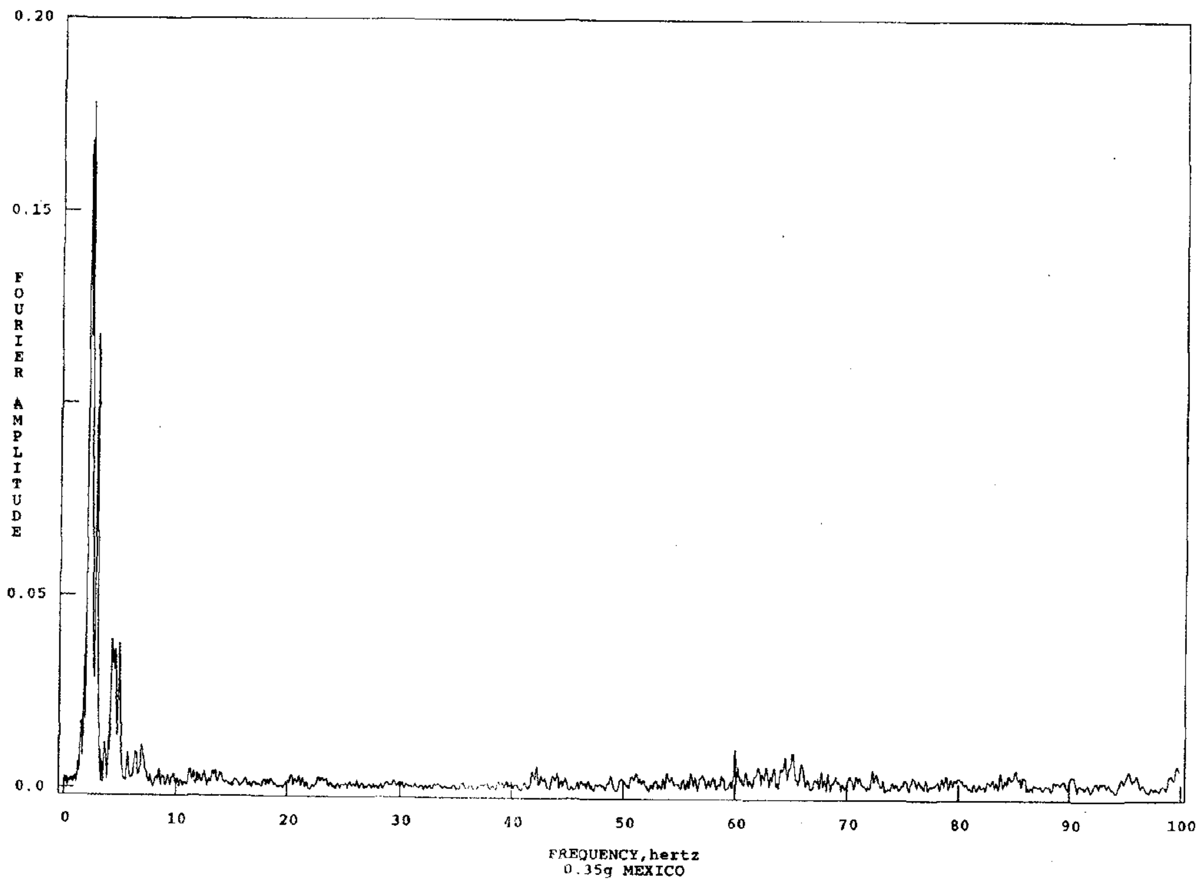
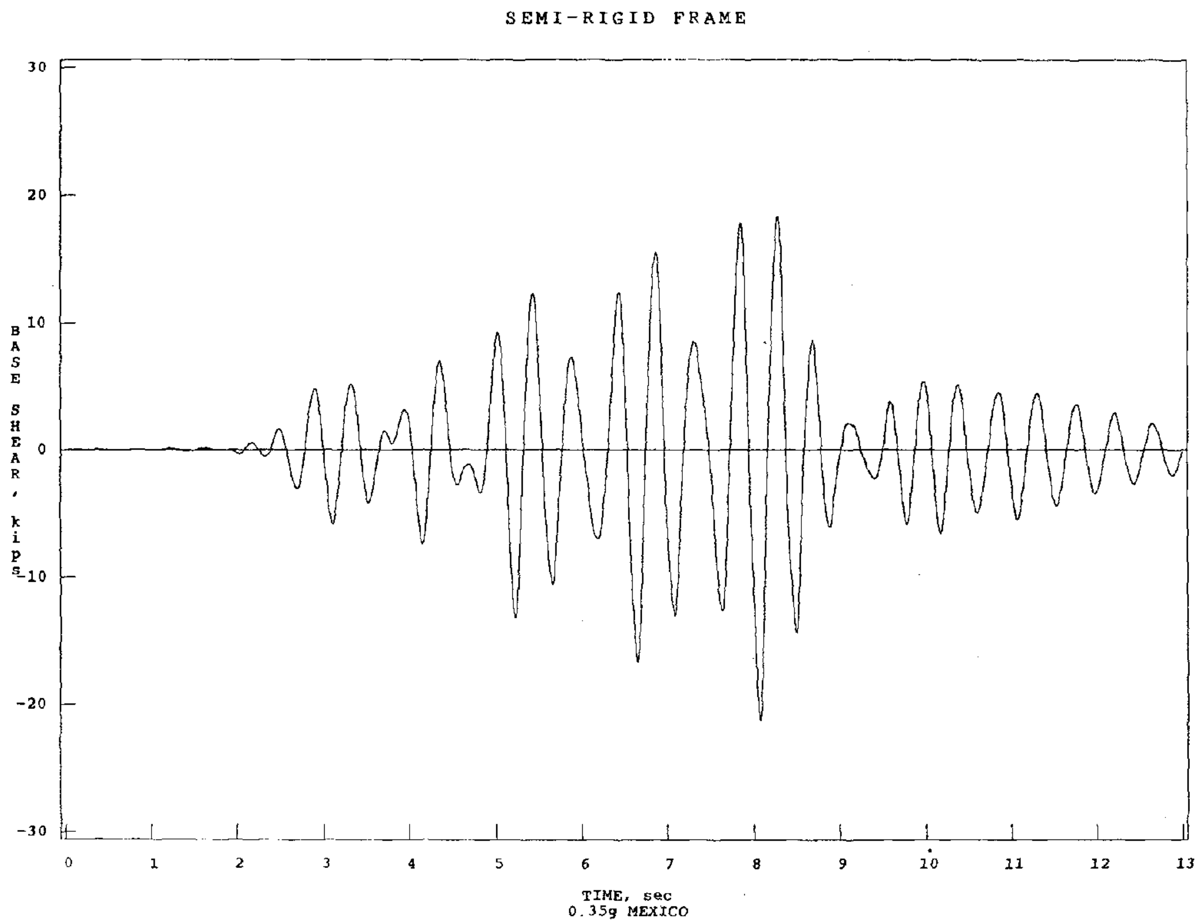
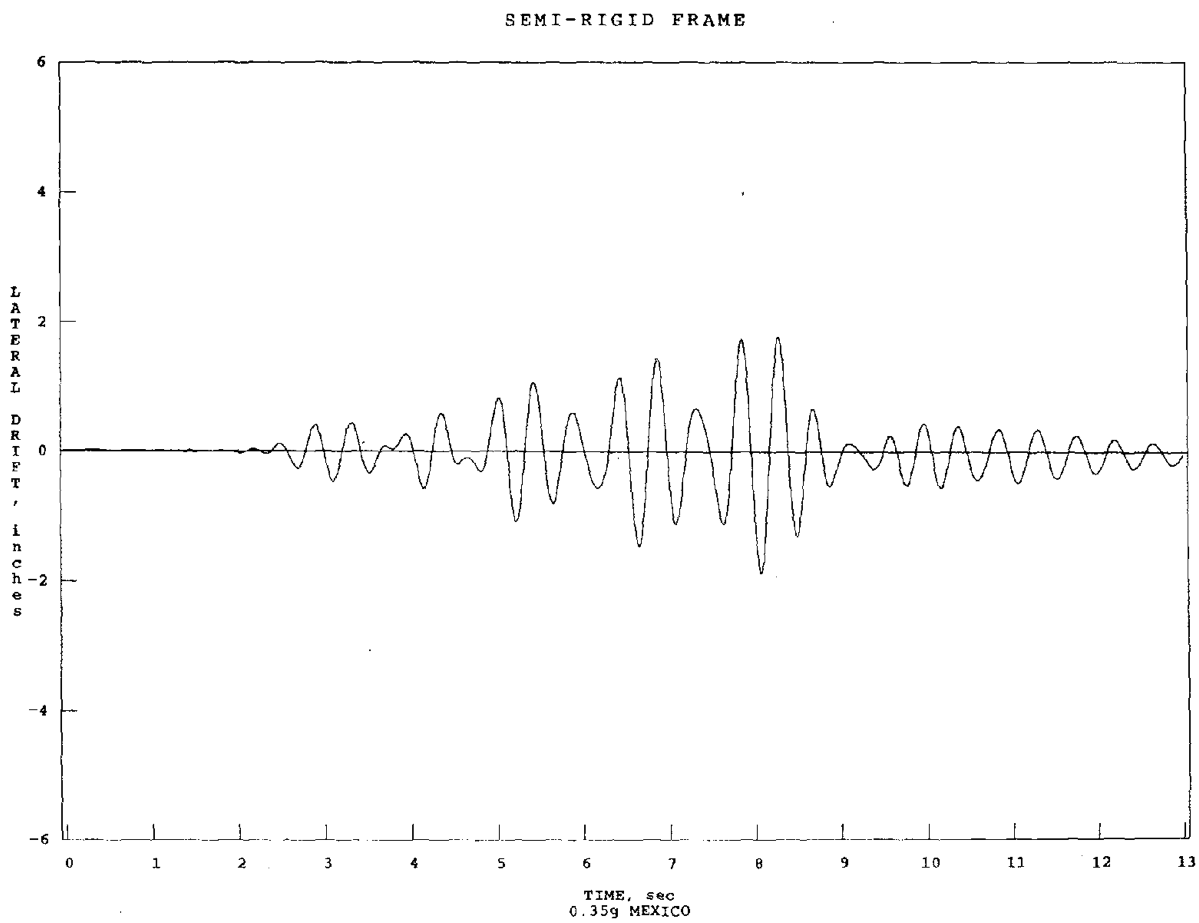
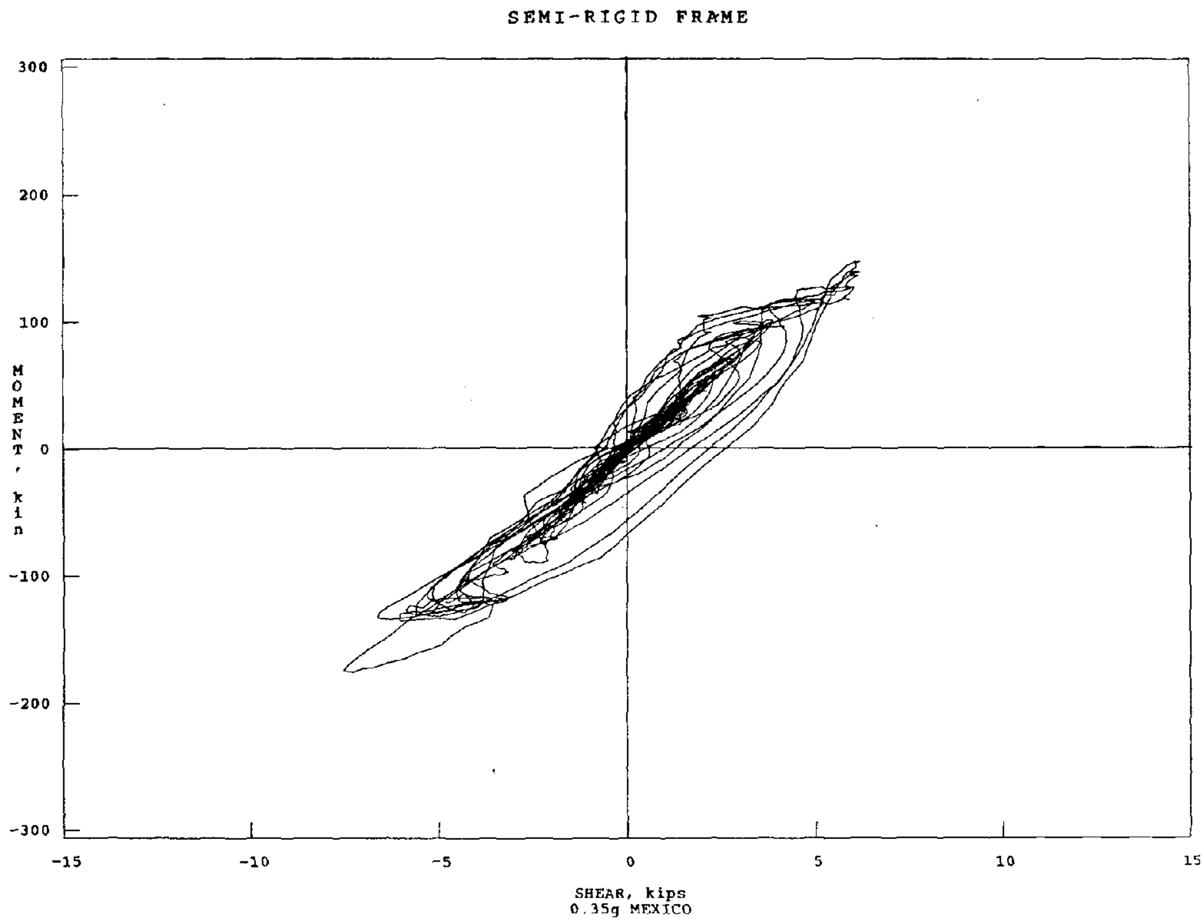


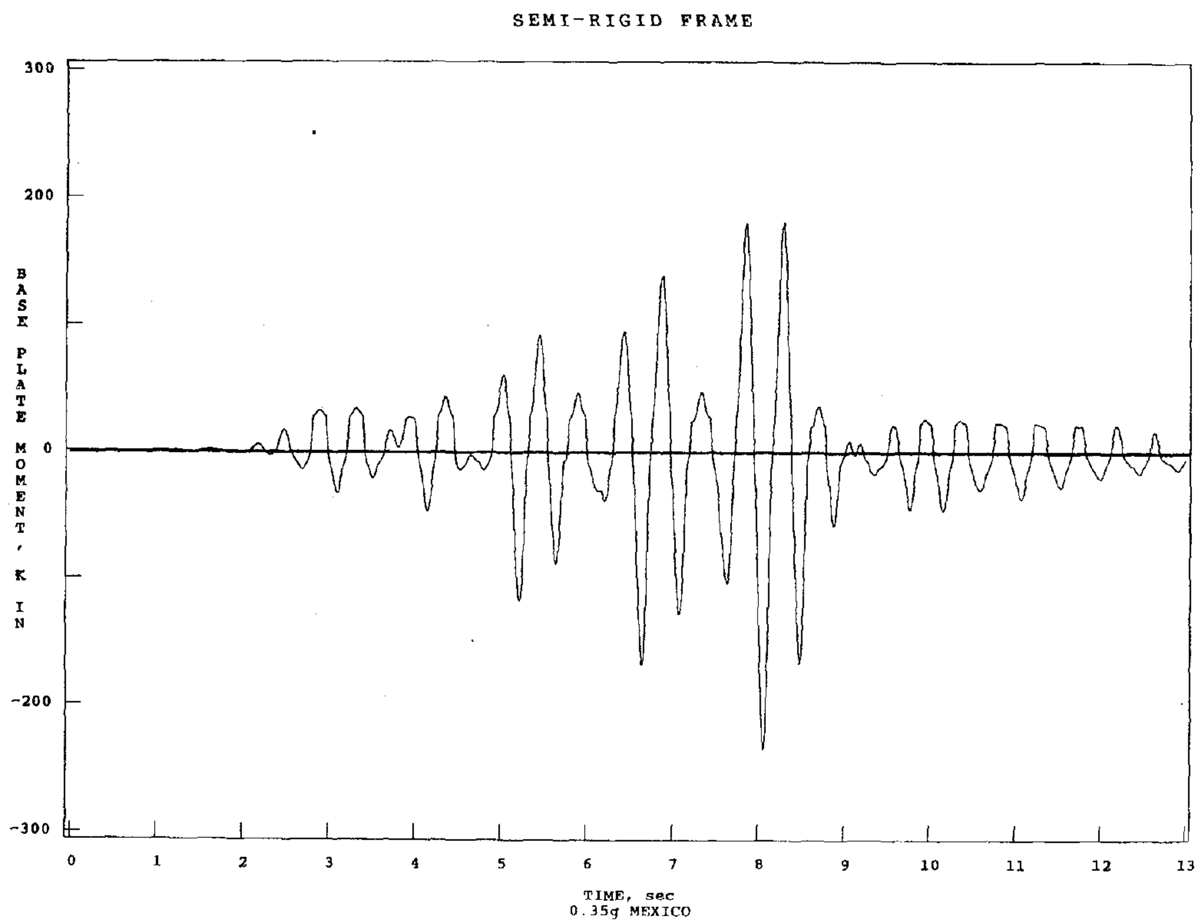


SEMI-RIGID FRAME
BASE SHEAR, kips
30
20
10
0
-10
-20
-30
0 1 2 3 4 5 6 7 8 9 10 11 12 13
TIME, sec
0.35g MEXICO

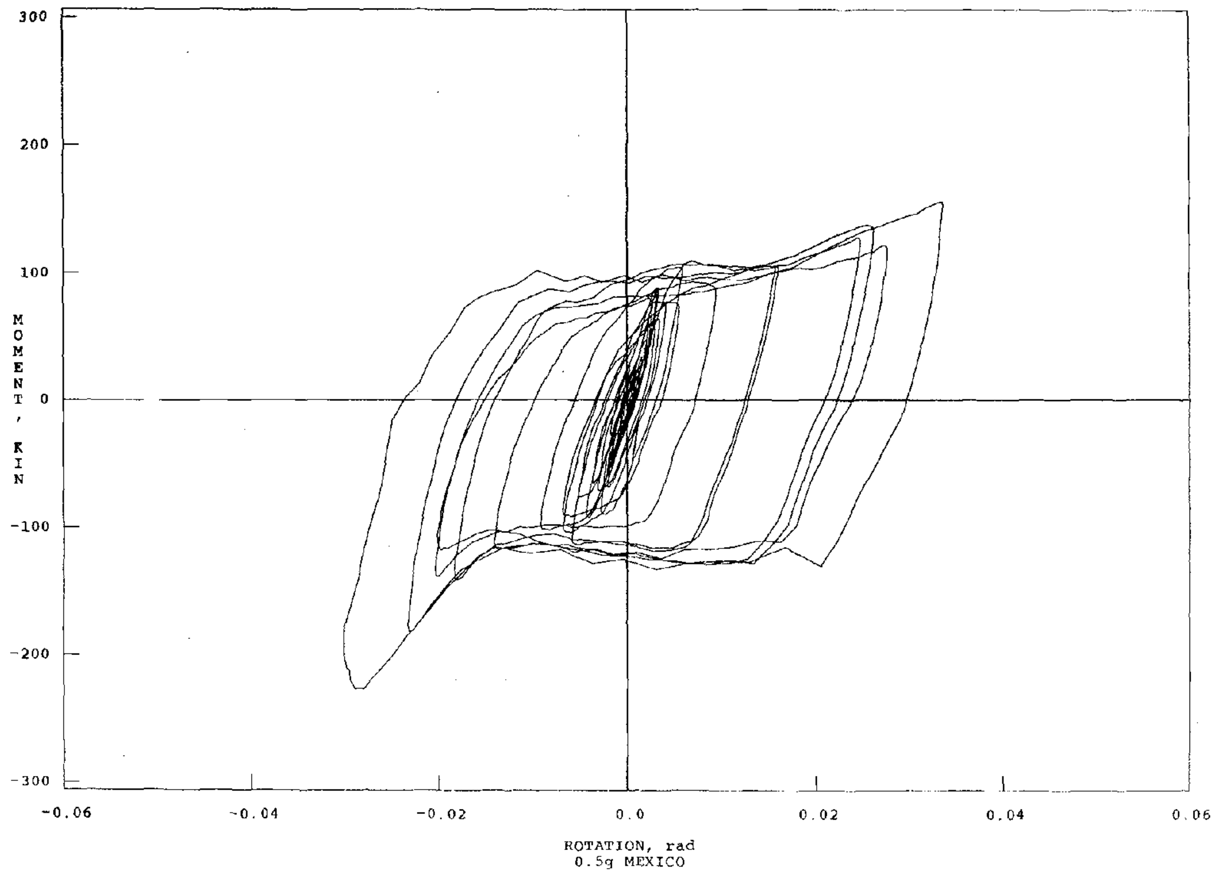


SEMI-RIGID FRAME
LATERAL DRIFT, inches
6
4
2
0
-2
-4
-6
0 1 2 3 4 5 6 7 8 9 10 11 12 13
TIME, sec
0.35g MEXICO

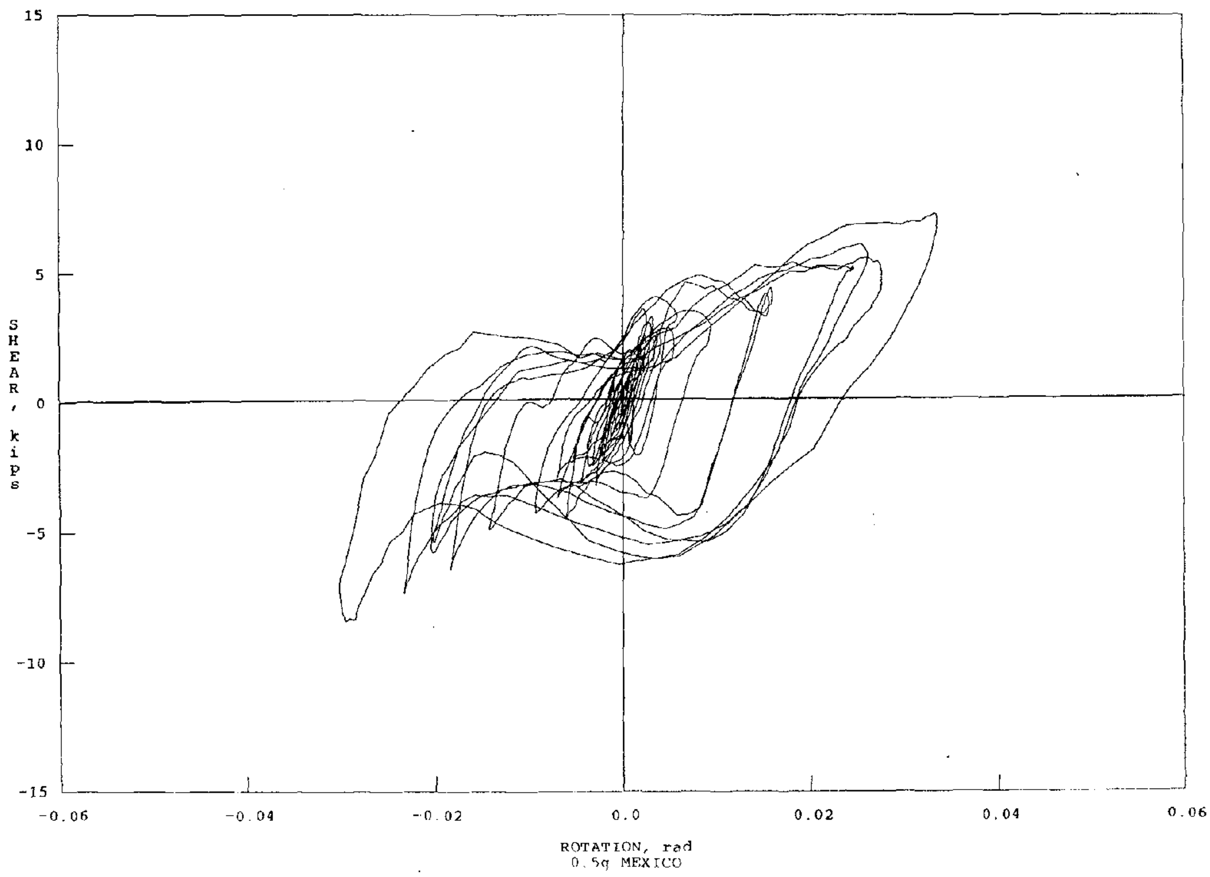

SEMI-RIGID FRAME
300
200
100
0
-100
-200
-300
MOMENT, kin
-15
-10
-5
0
5
10
15
SHEAR, kips
0.35g MEXICO


SEMI-RIGID FRAME
300
200
0
-200
-300
BASE PLATE MOMENT, K IN
0
1
2
3
4
5
6
7
8
9
10
11
12
13
TIME, sec
0.35g MEXICO

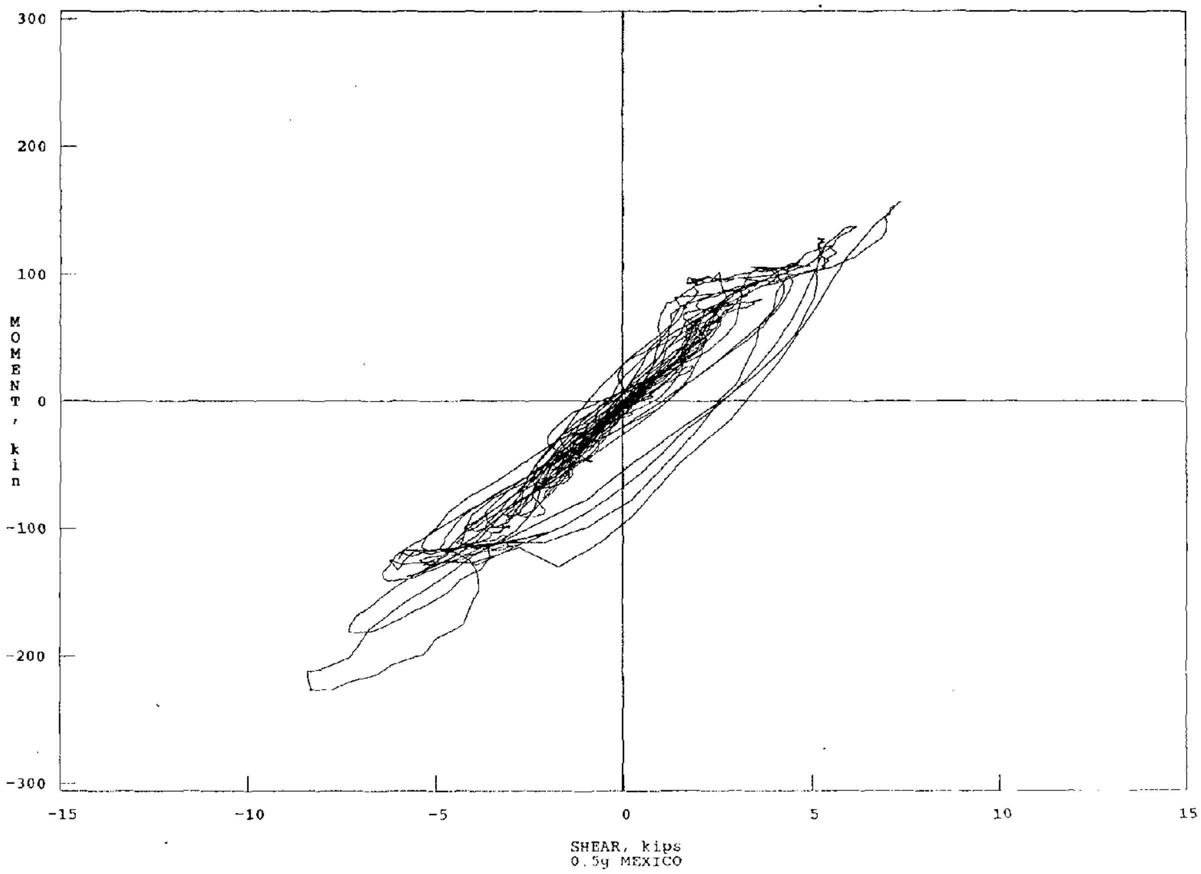
SEMI-RIGID FRAME



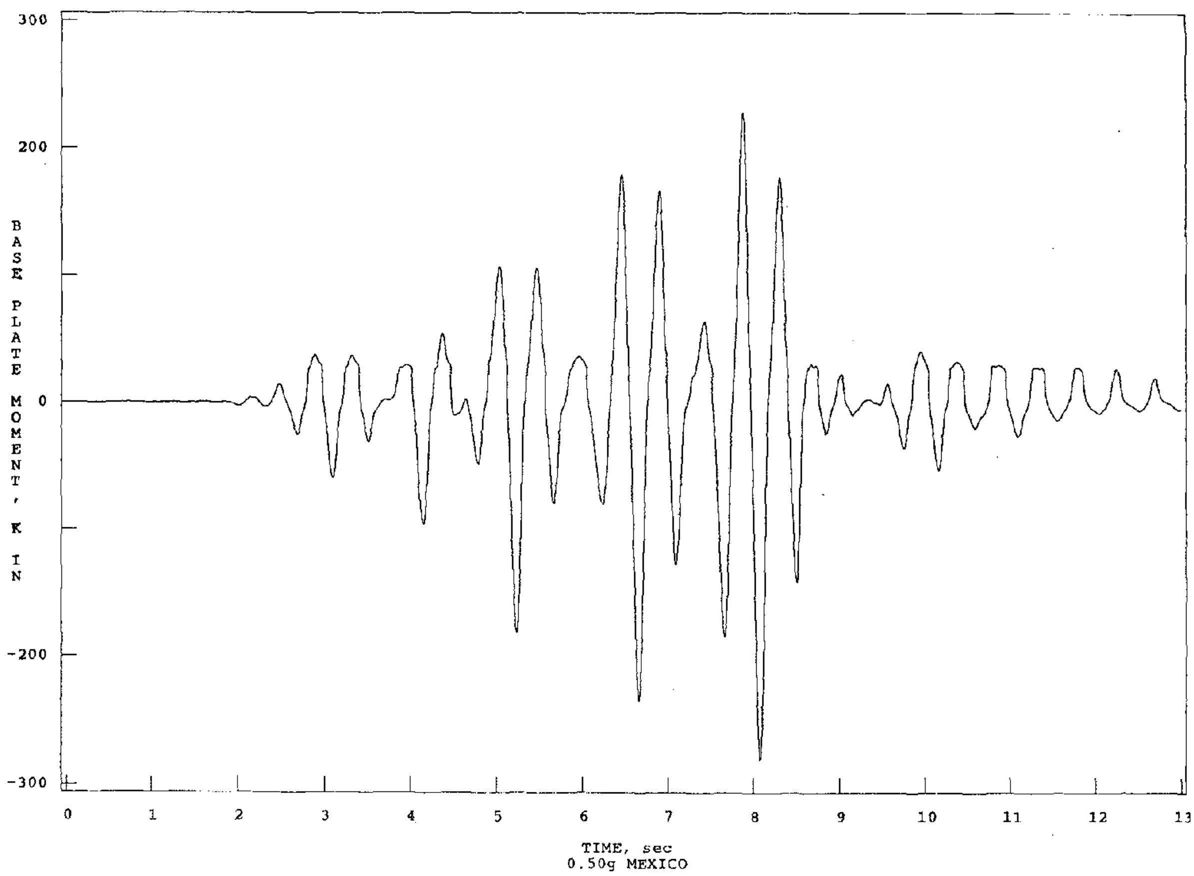
SEMI-RIGID FRAME

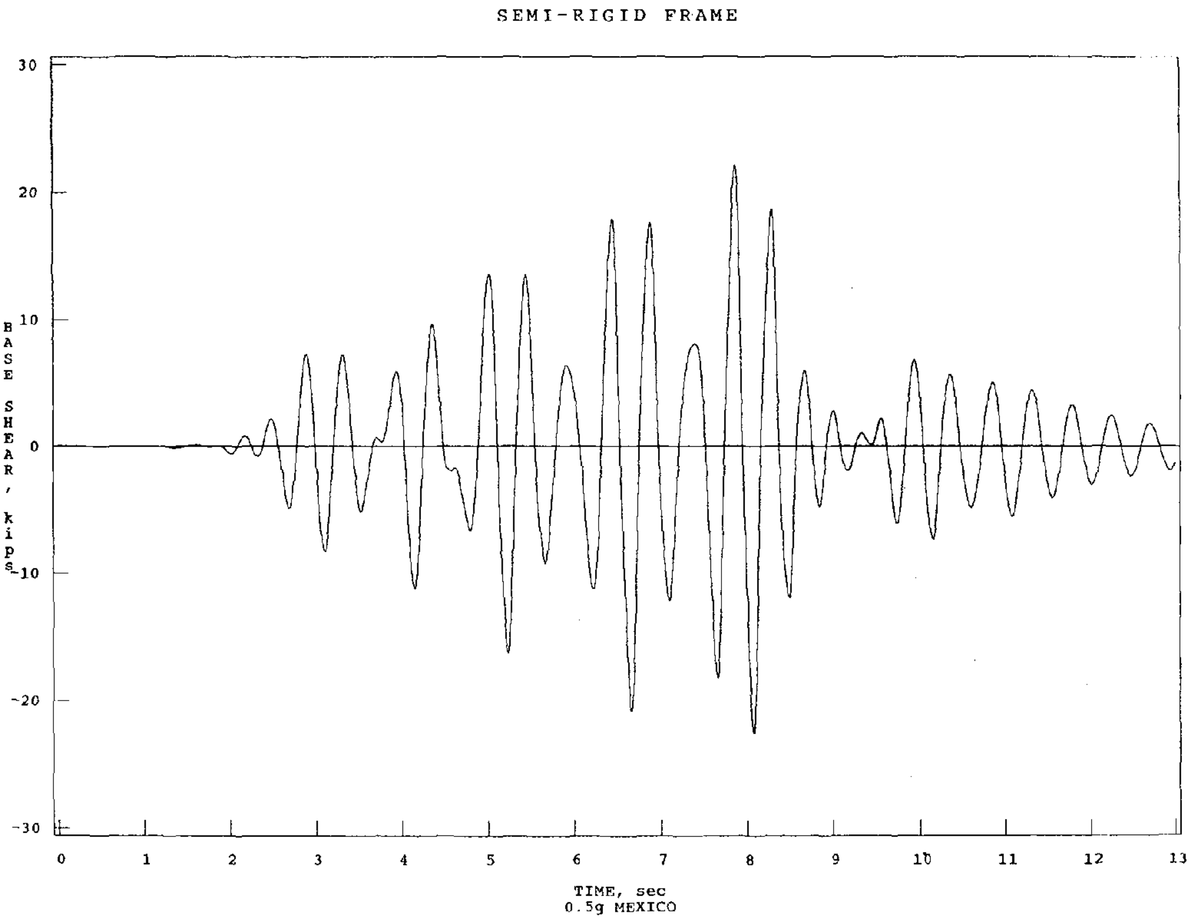


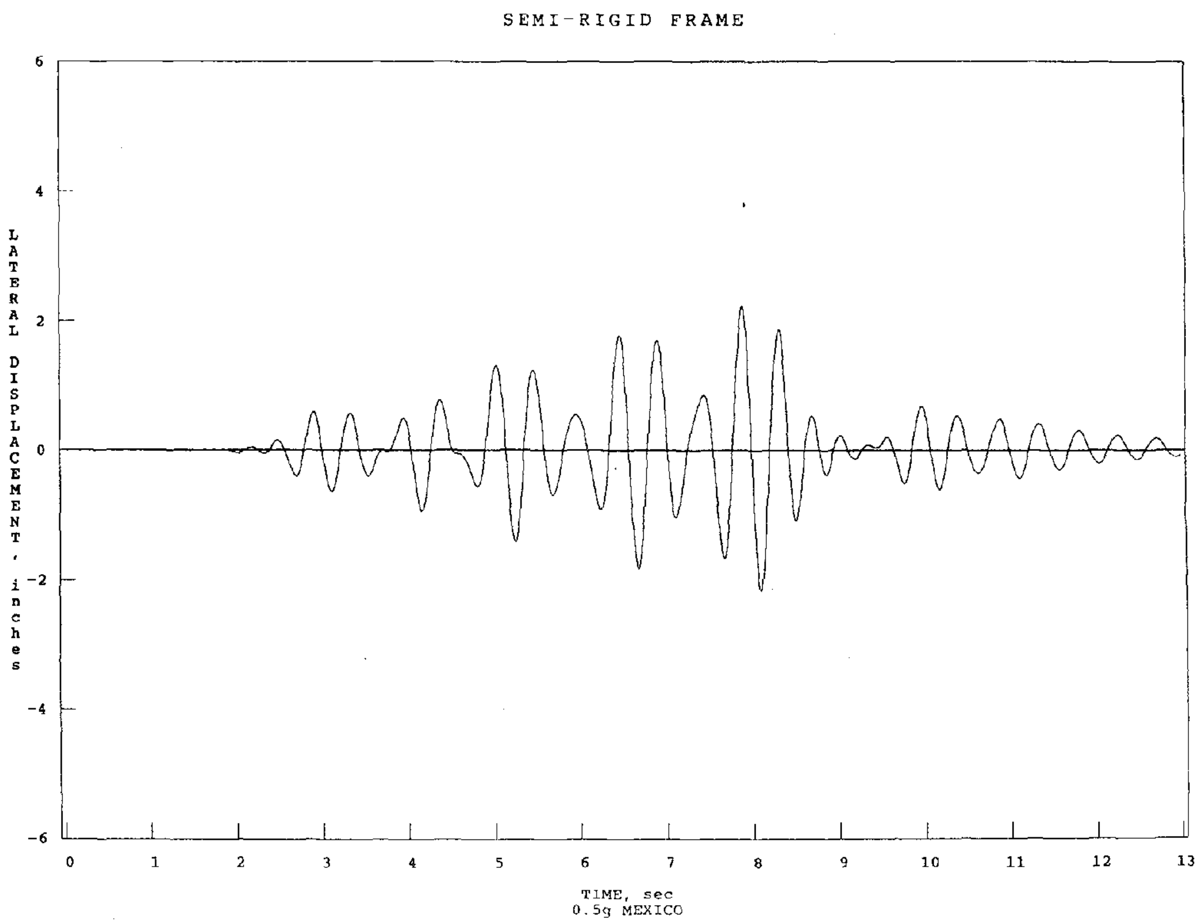
SEMI-RIGID FRAME



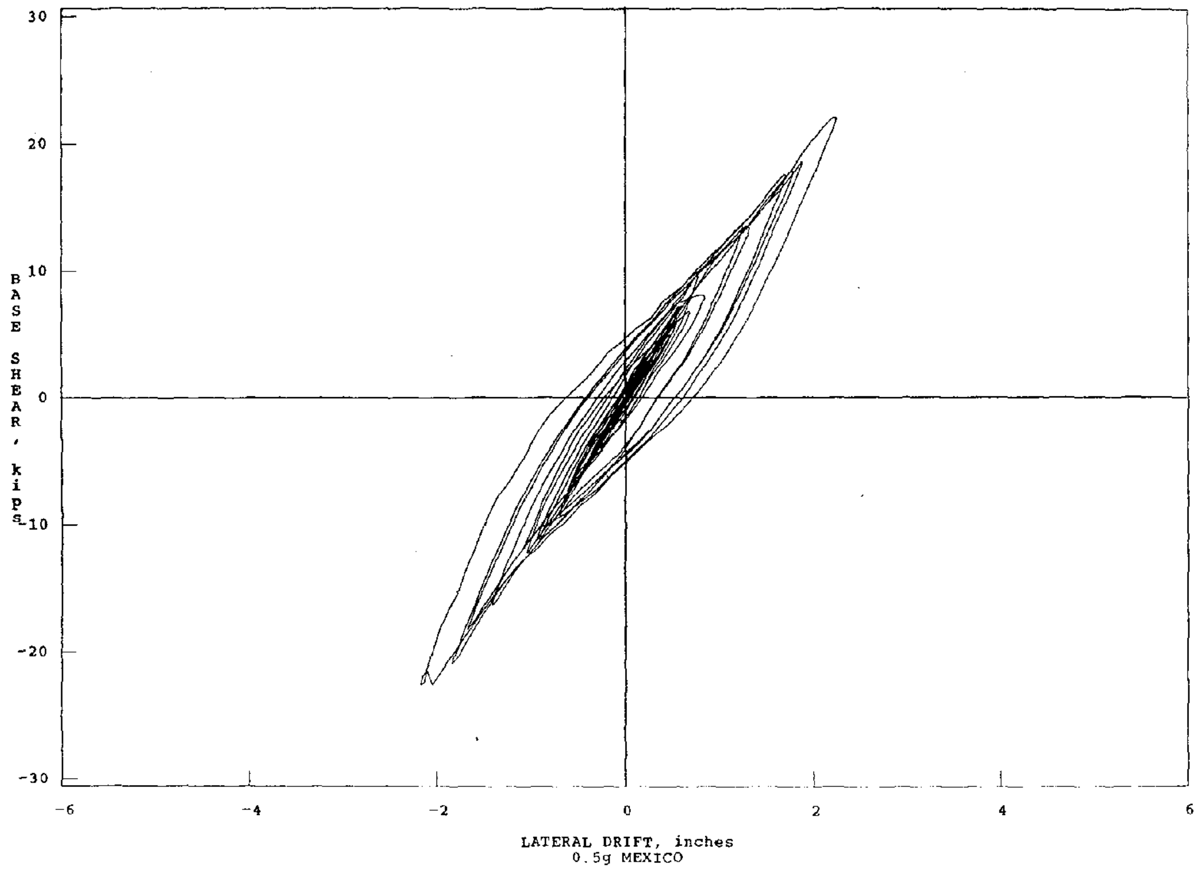
SEMI-RIGID FRAME




SEMI-RIGID FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
0
1
2
3
4
5
6
7
8
9
10
11
12
13
TIME, sec
0.5g MEXICO


SEMI-RIGID FRAME
6
4
2
0
-2
-4
-6
LATERAL DISPLACEMENT, inches
0
1
2
3
4
5
6
7
8
9
10
11
12
13
TIME, sec
0.5g MEXICO

SEMI-RIGID FRAME



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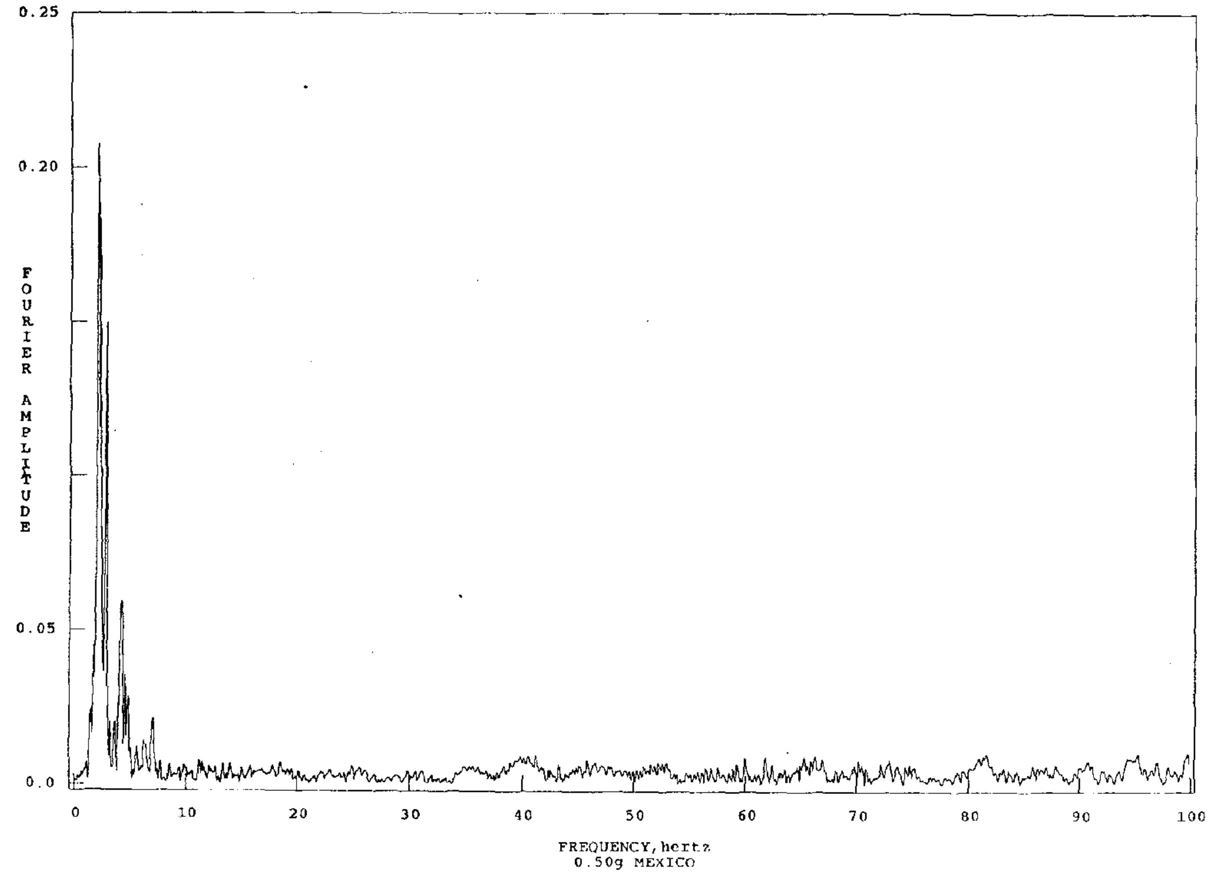
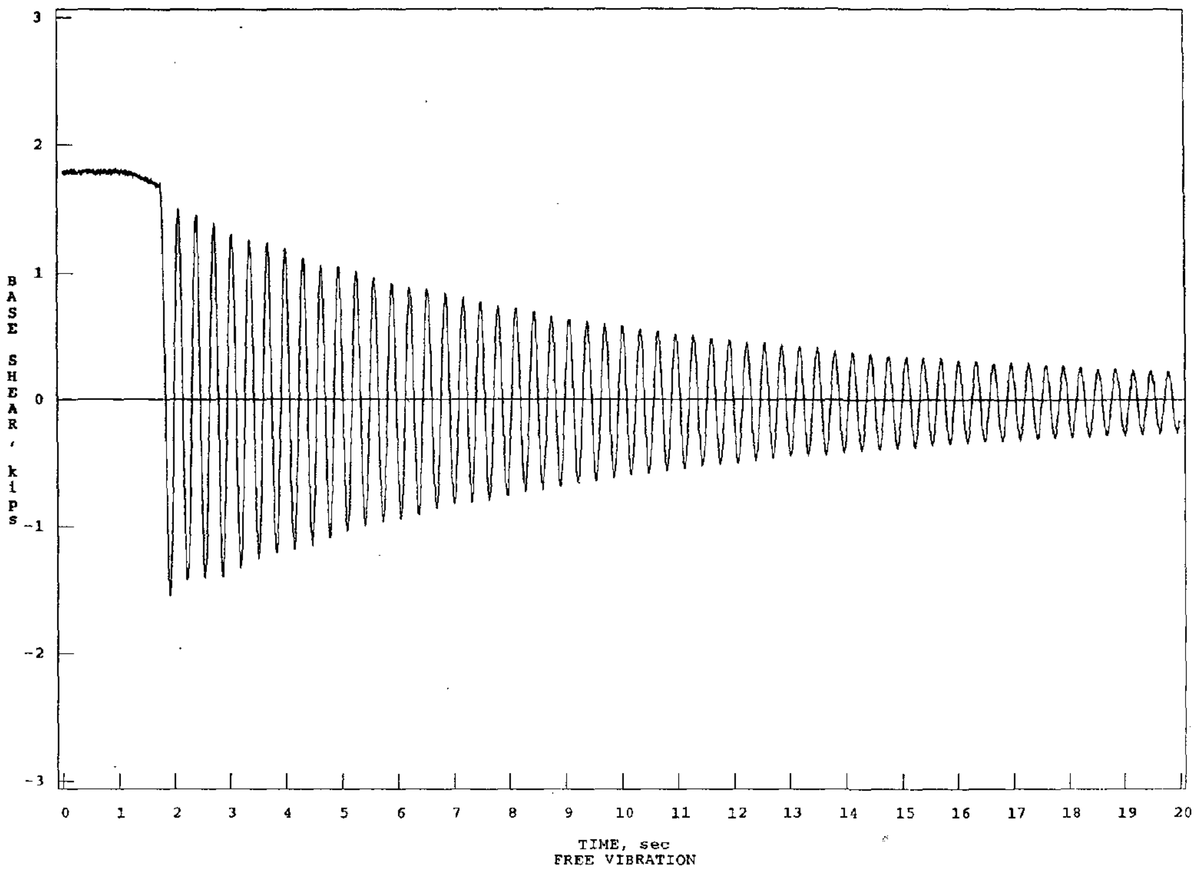
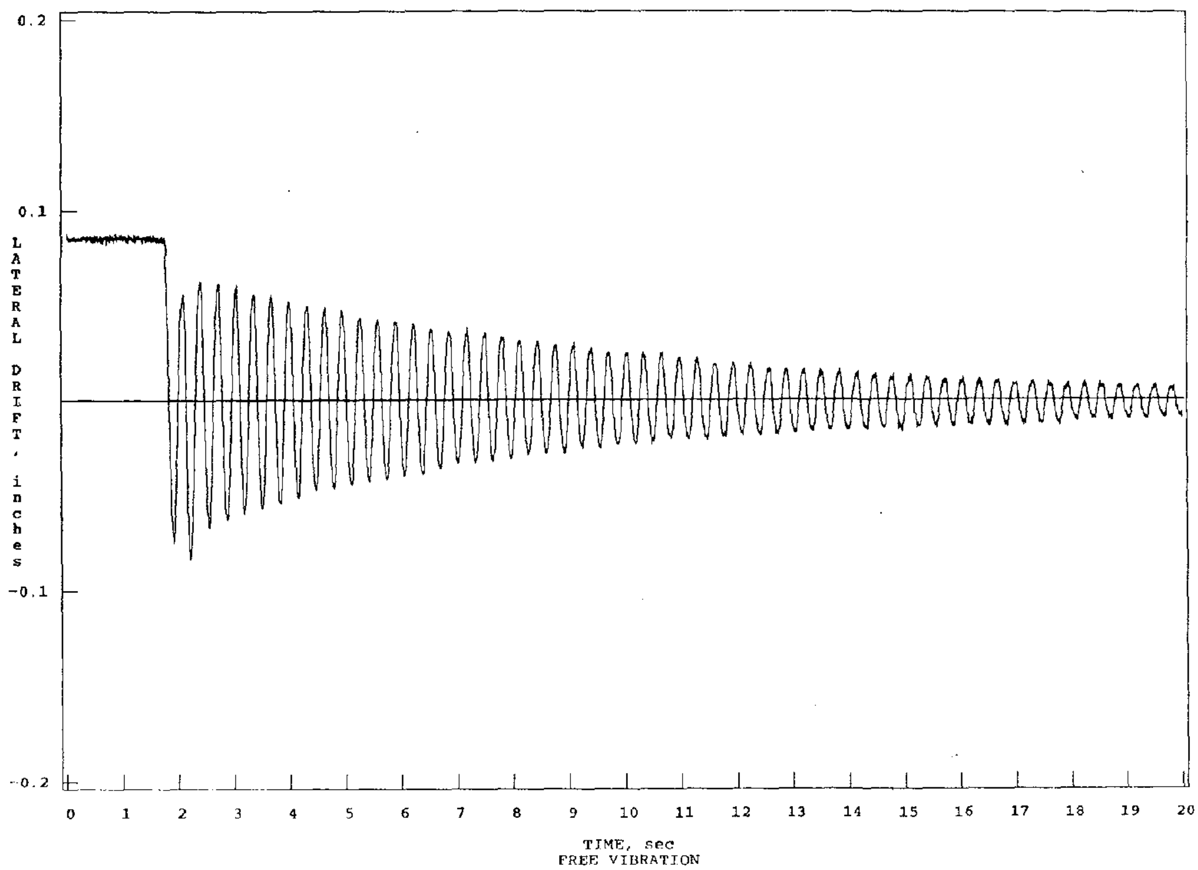


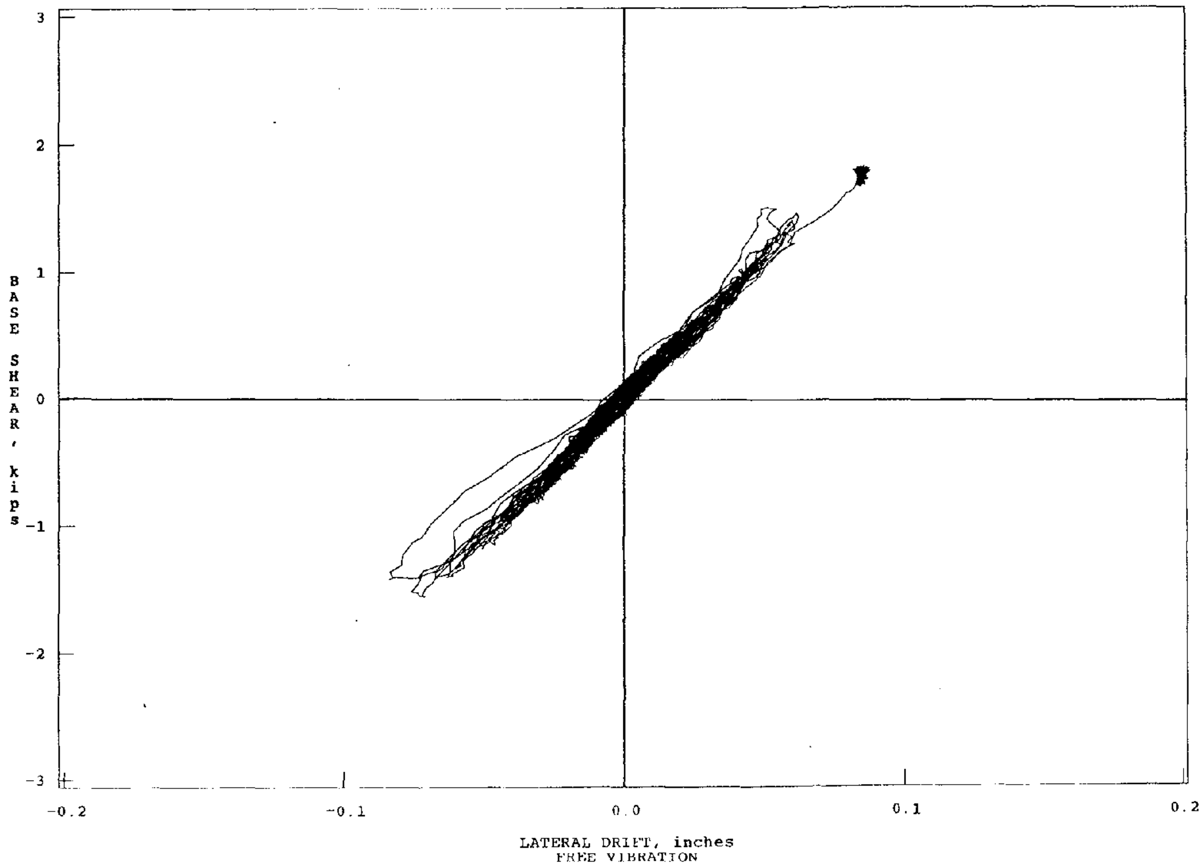
FIXED FRAME



FIXED FRAME

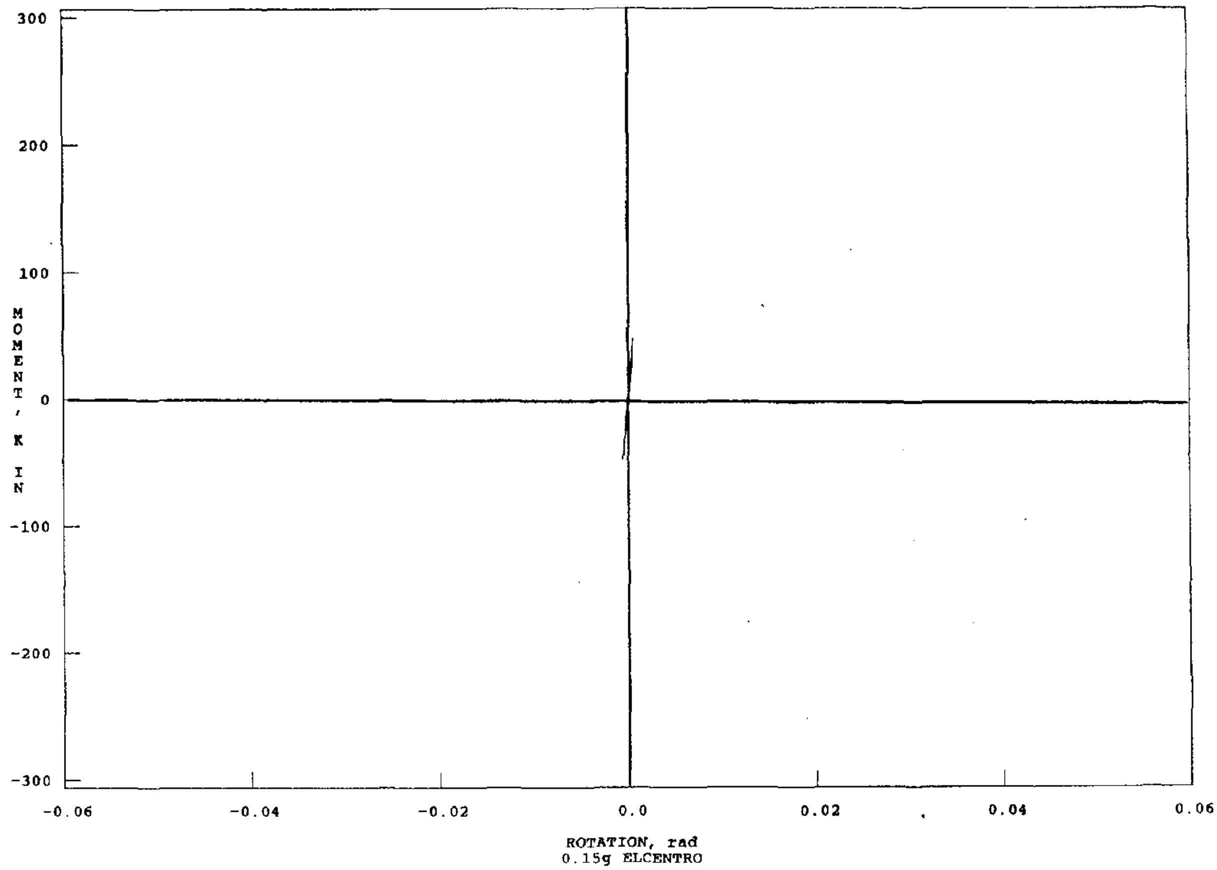


FIXED FRAME

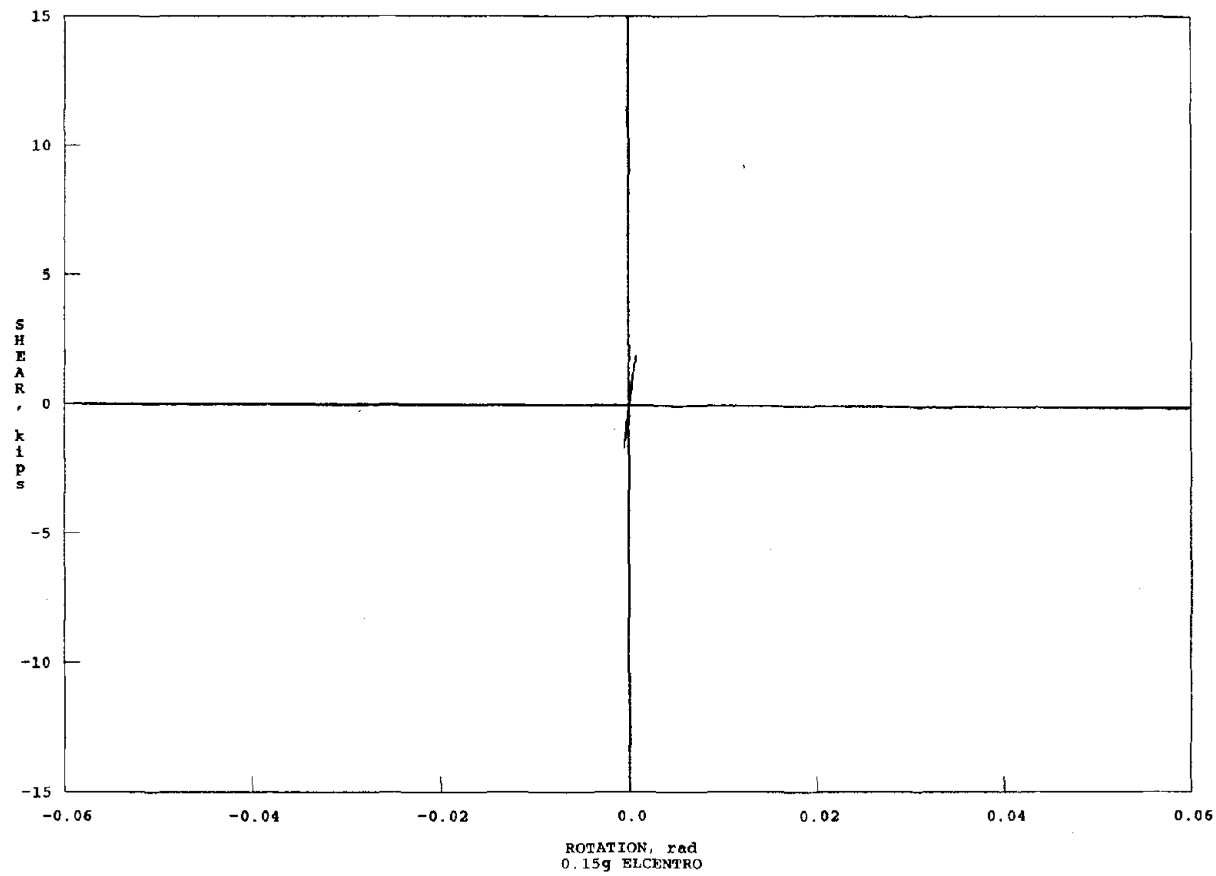


3
2
1
0
-1
-2
-3
BASE SHEAR, kips
-0.2
-0.1
0.0
0.1
0.2
LATERAL DRIFT, inches
FREE VIBRATION

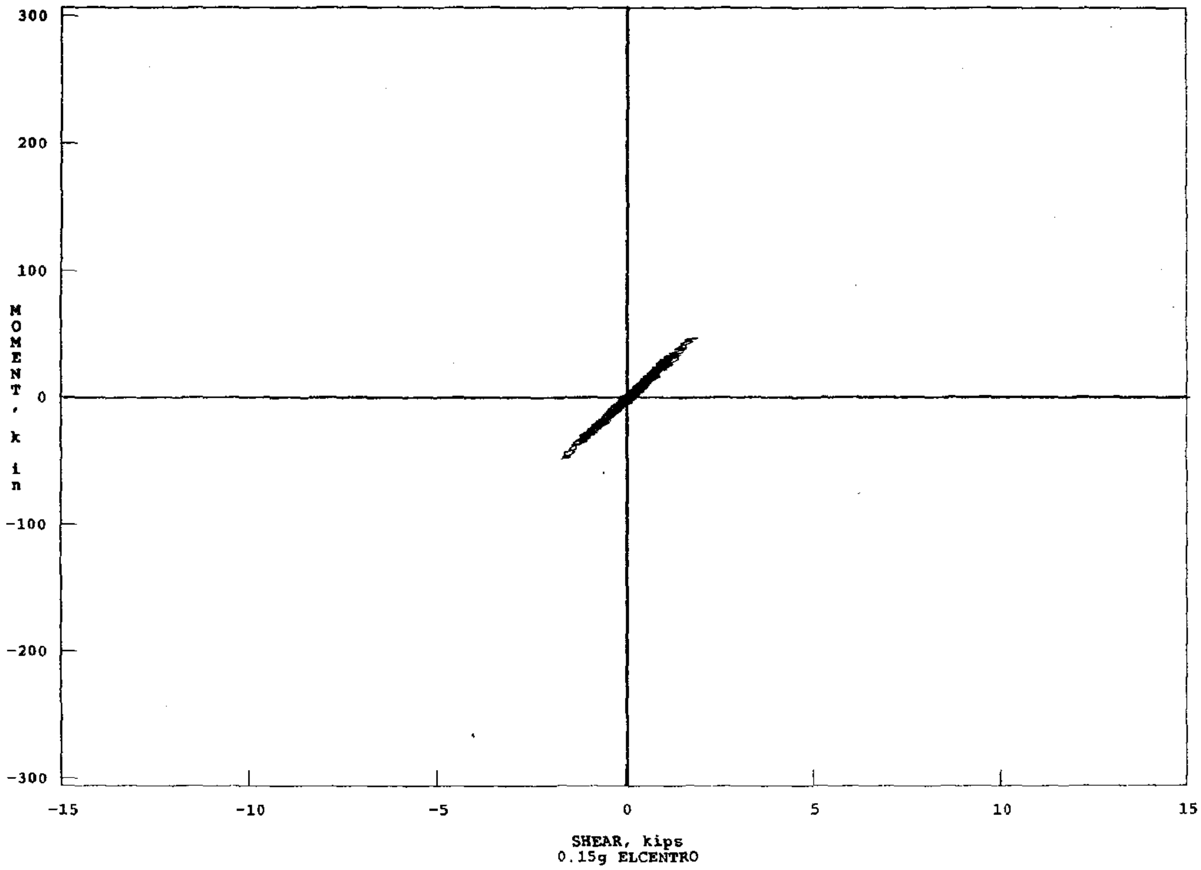
FIXED FRAME



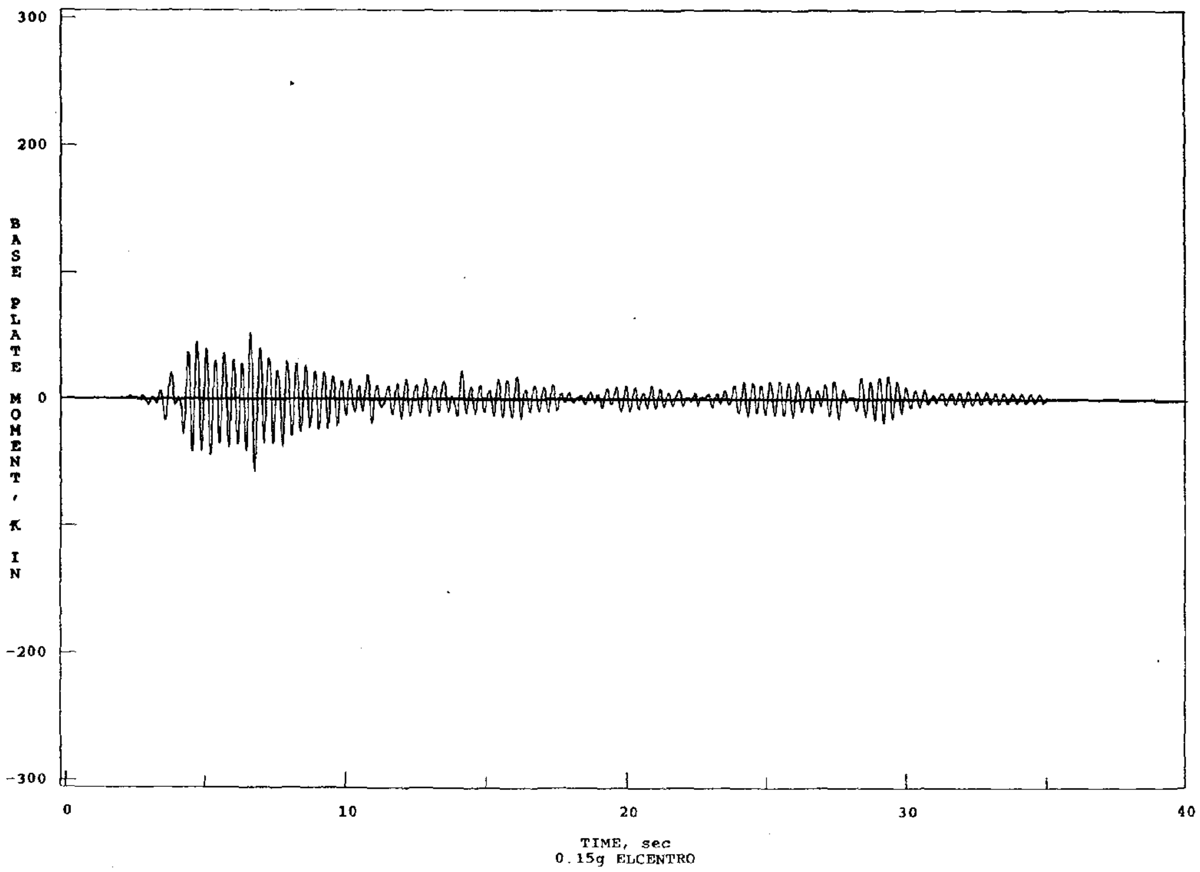
FIXED FRAME



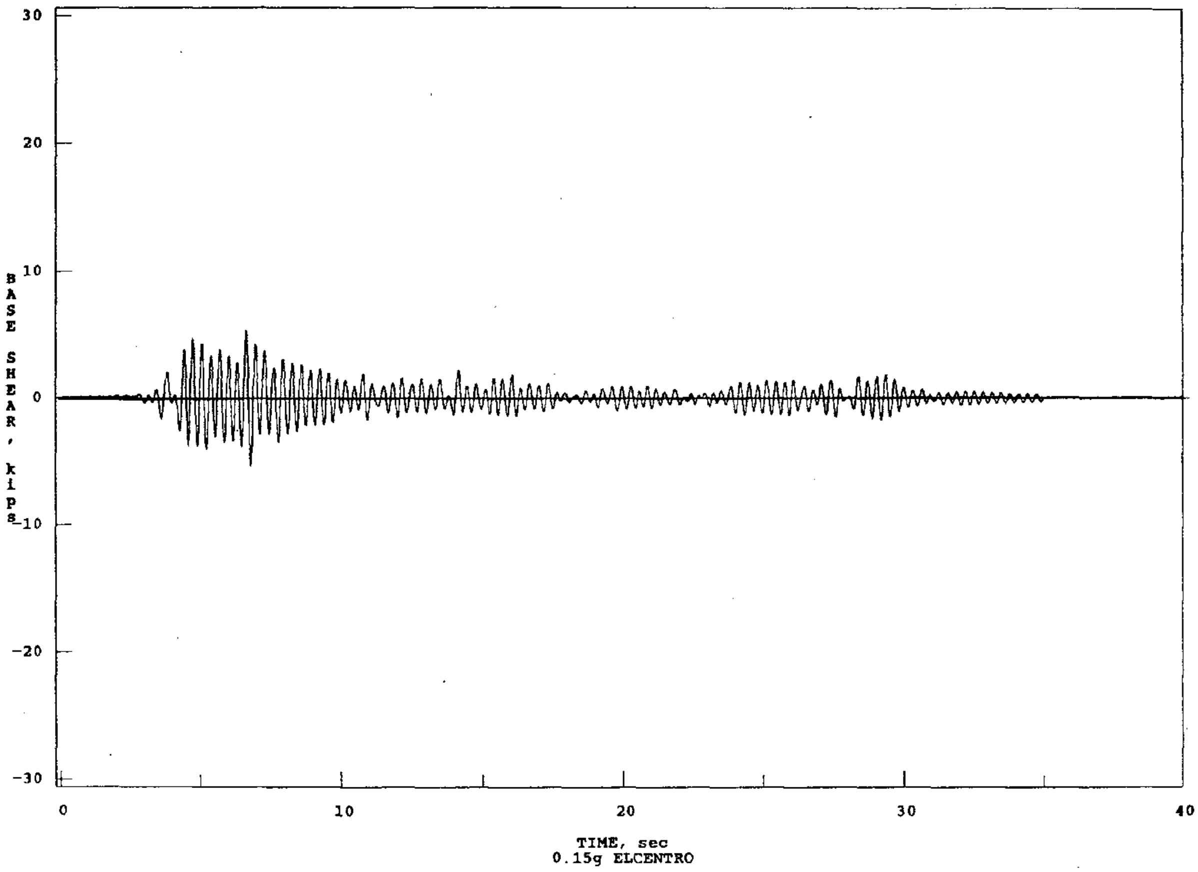
FIXED FRAME



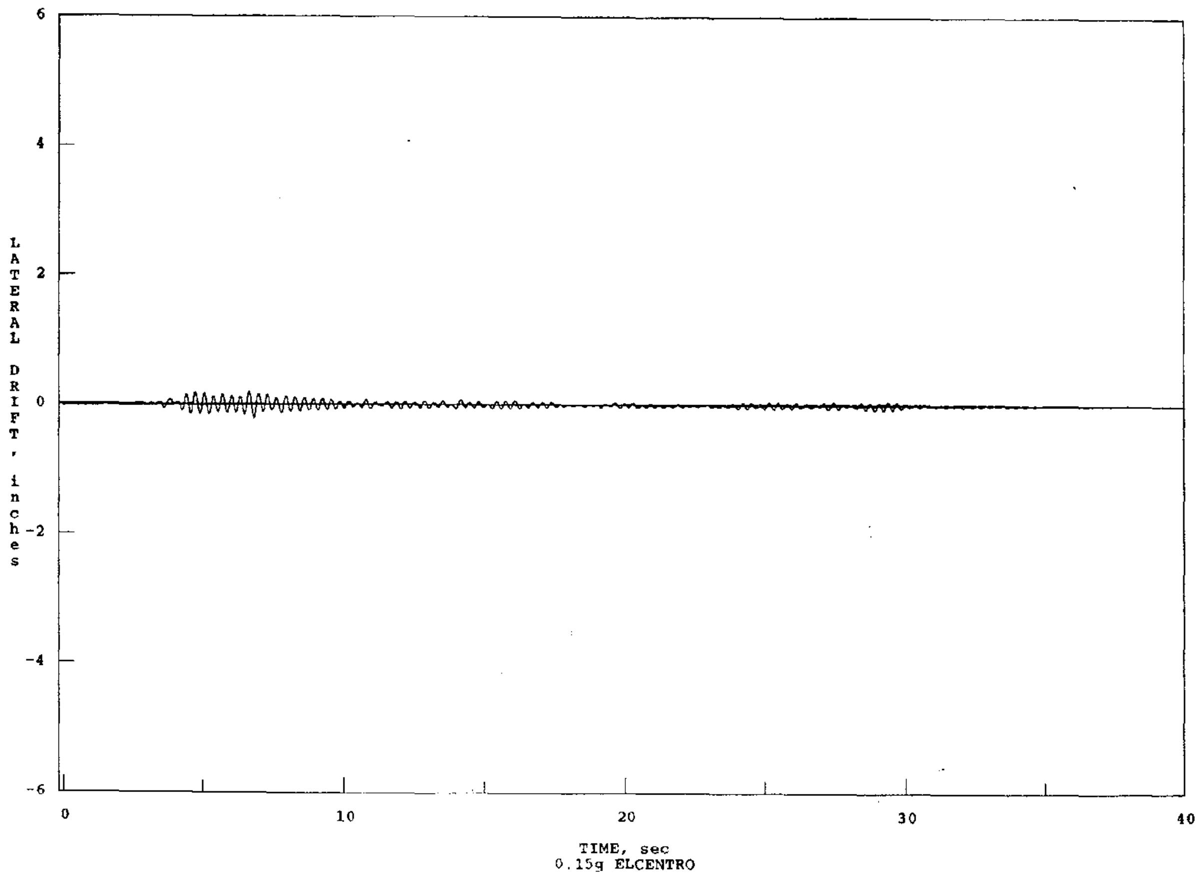
FIXED FRAME



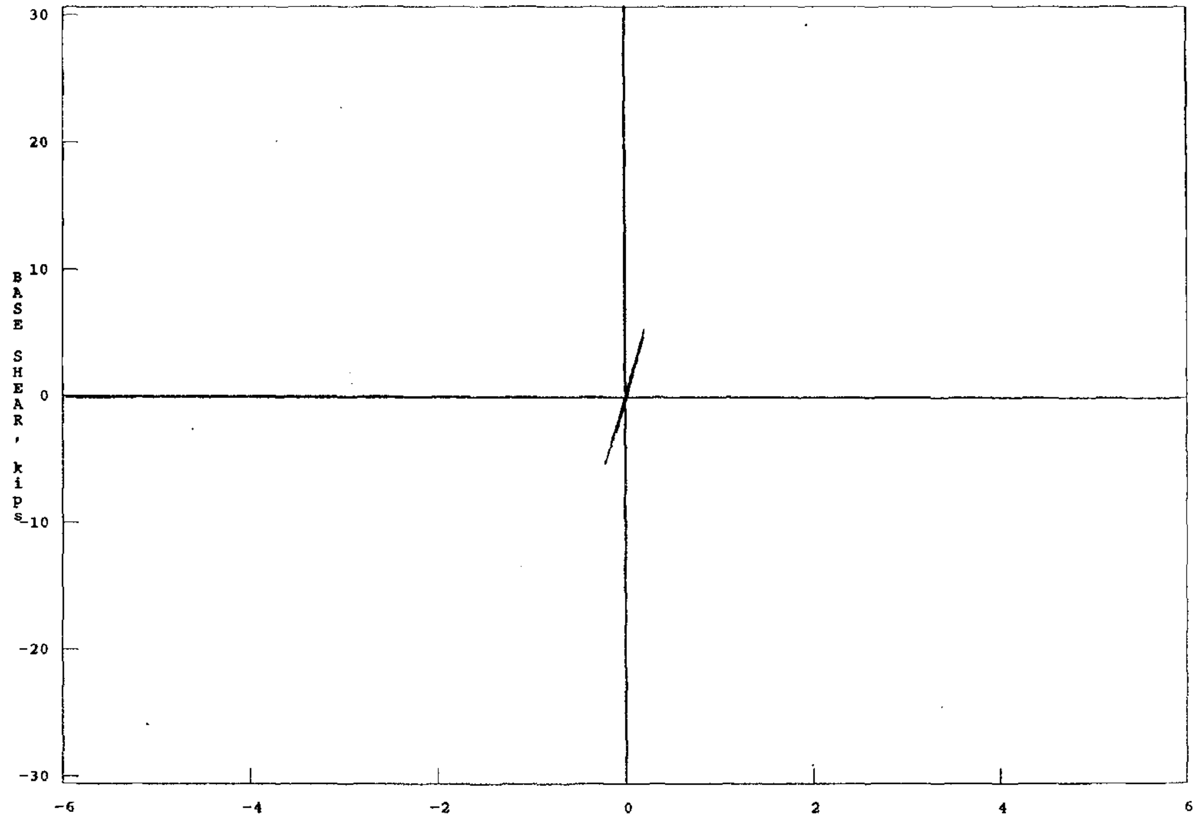
FIXED FRAME



FIXED FRAME

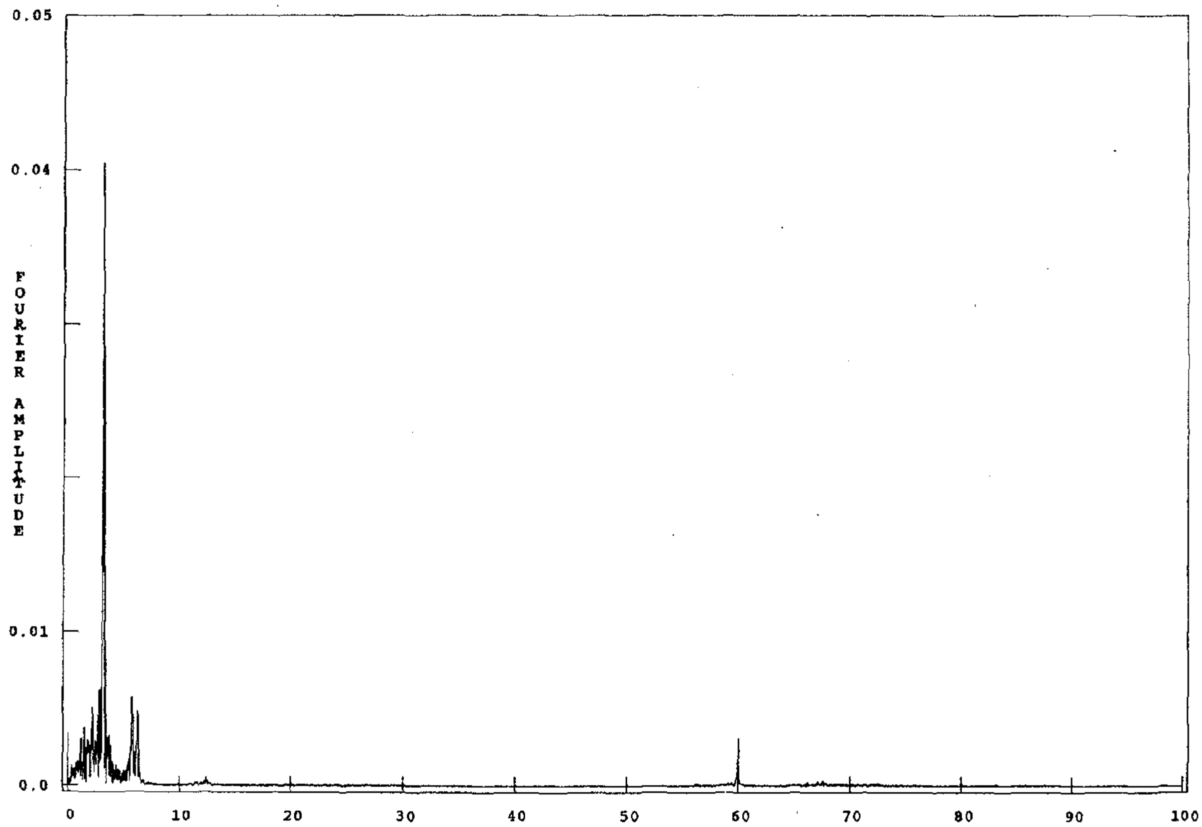


FIXED FRAME



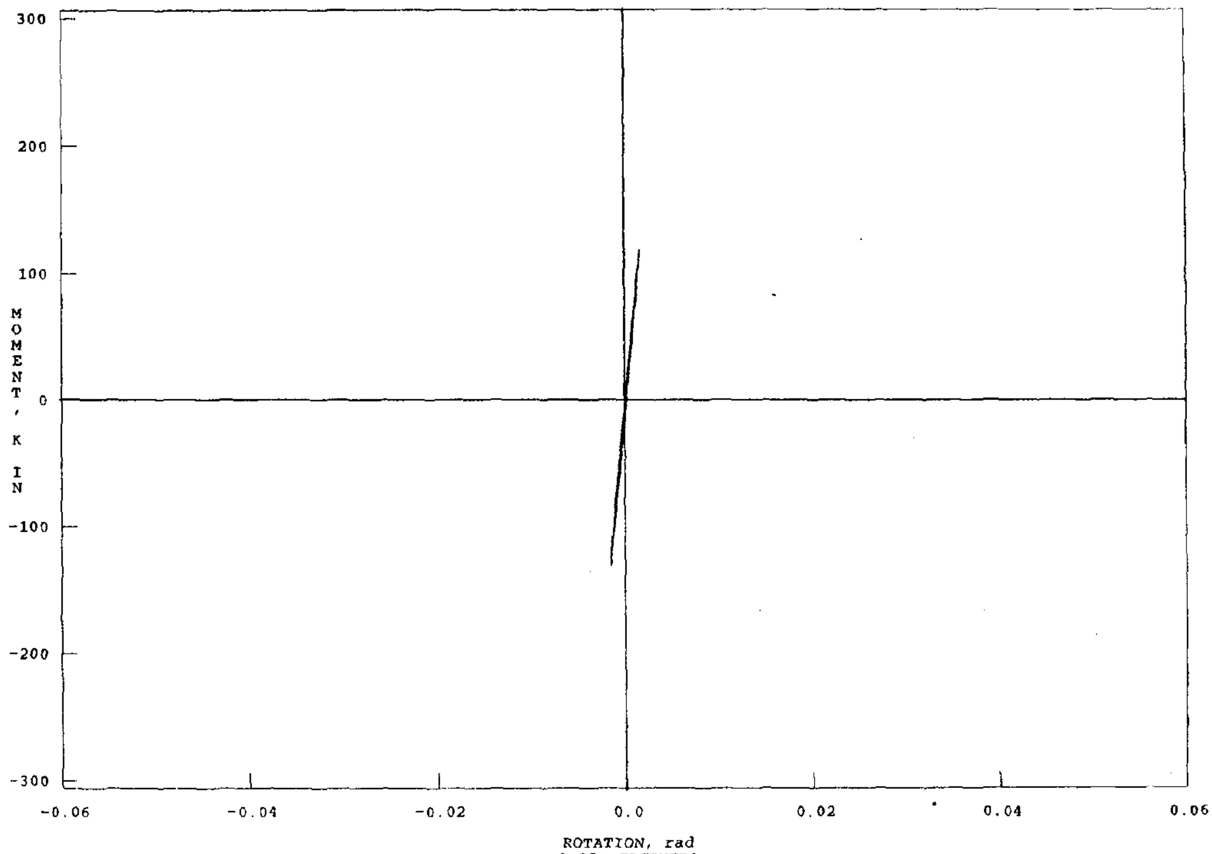
LATERAL DRIFT, inches
0.15g ELCENTRO

FIXED FRAME



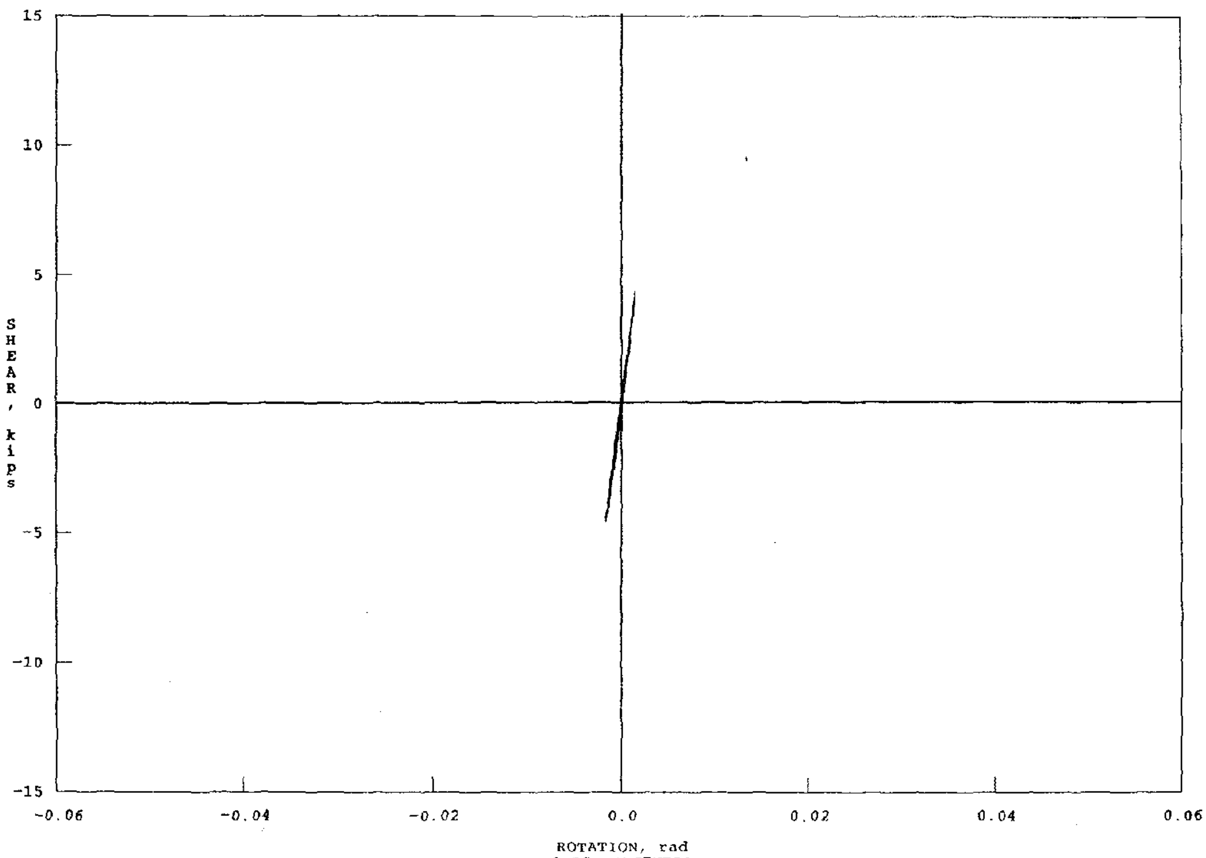
FREQUENCY, hertz
0.15g ELCENTRO

FIXED FRAME



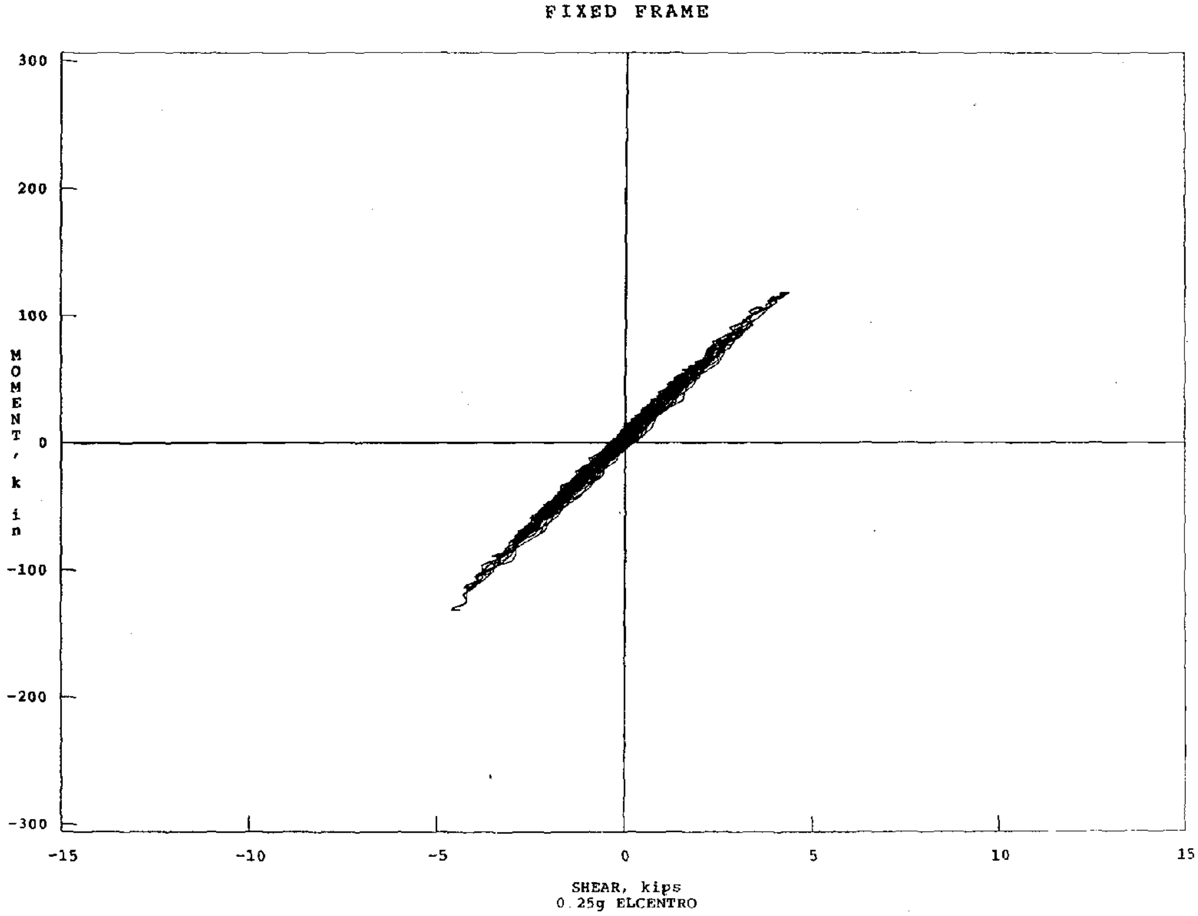
0.25g ELCENTRO

FIXED FRAME

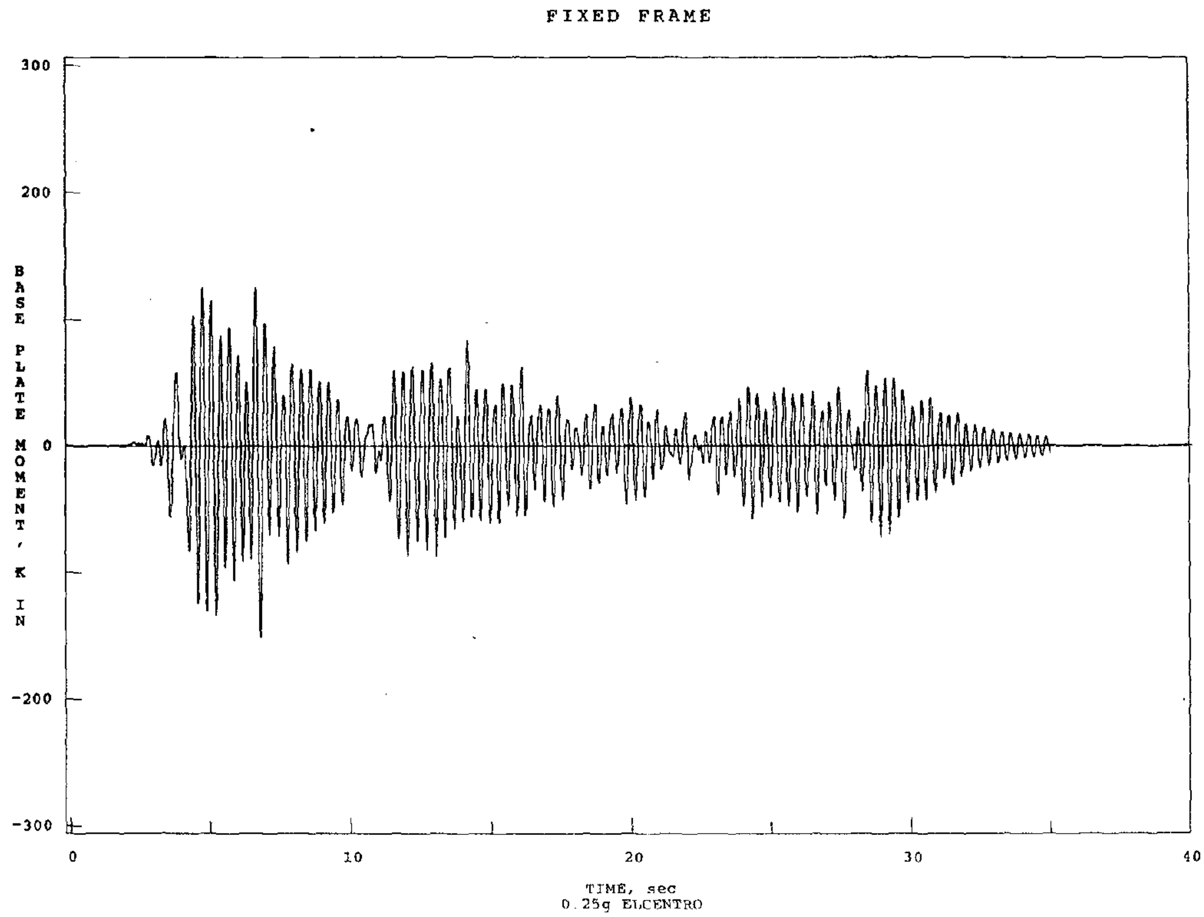


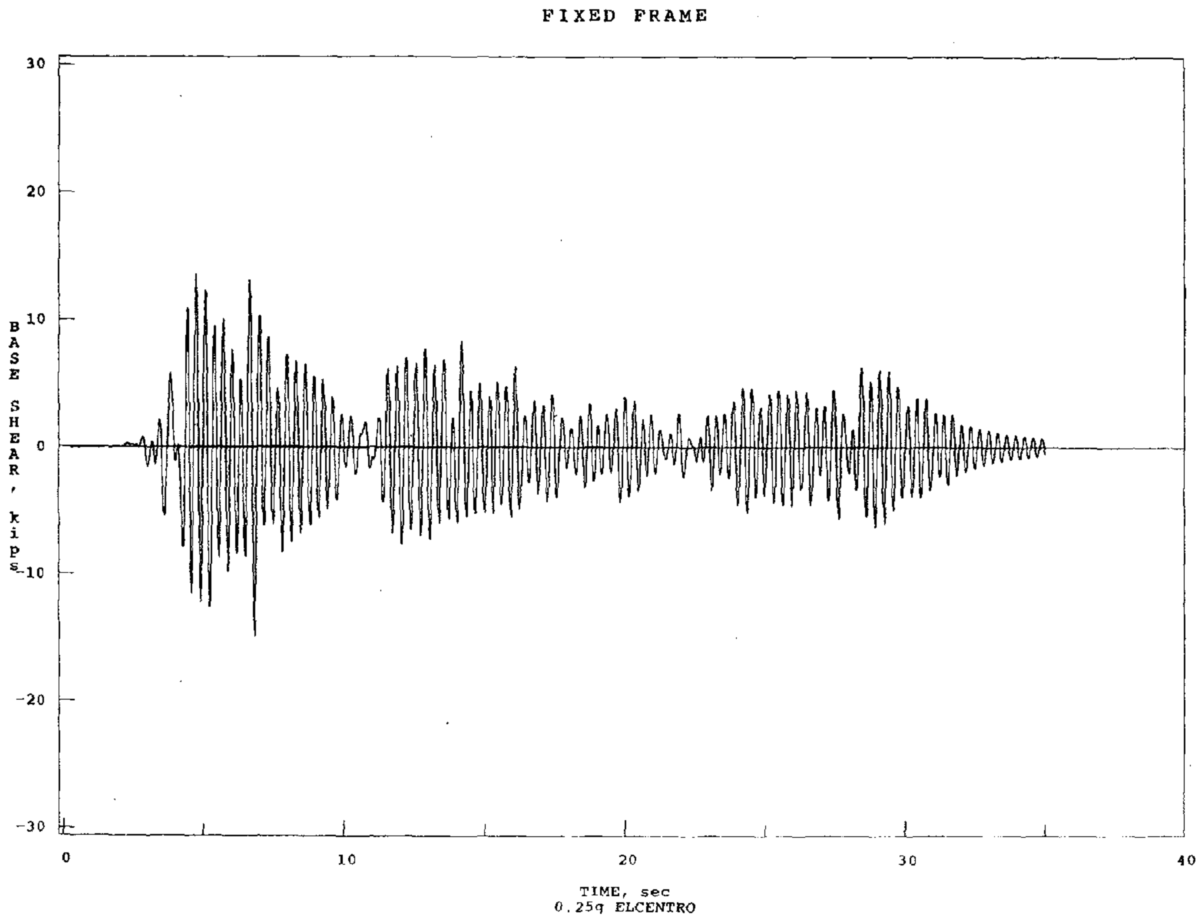
0.25g ELCENTRO

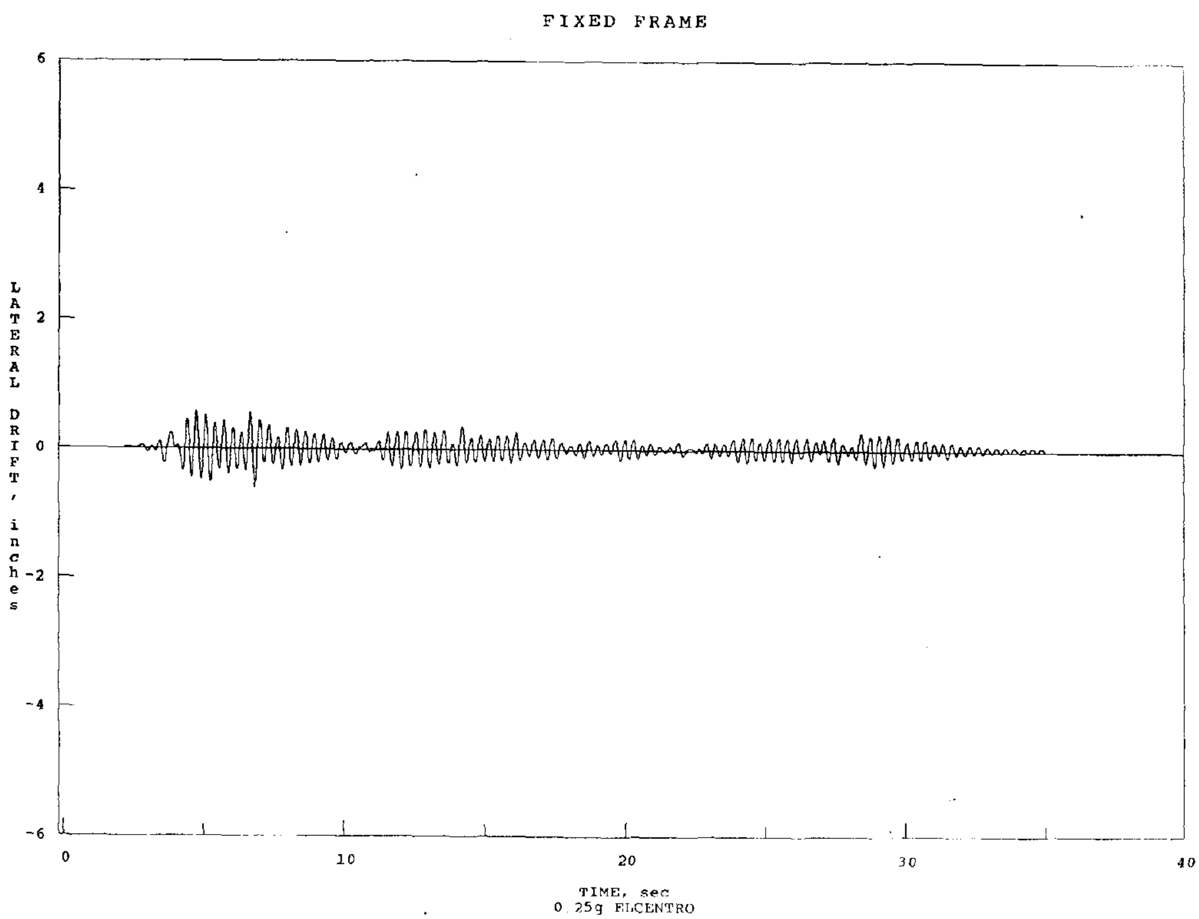
FIXED FRAME

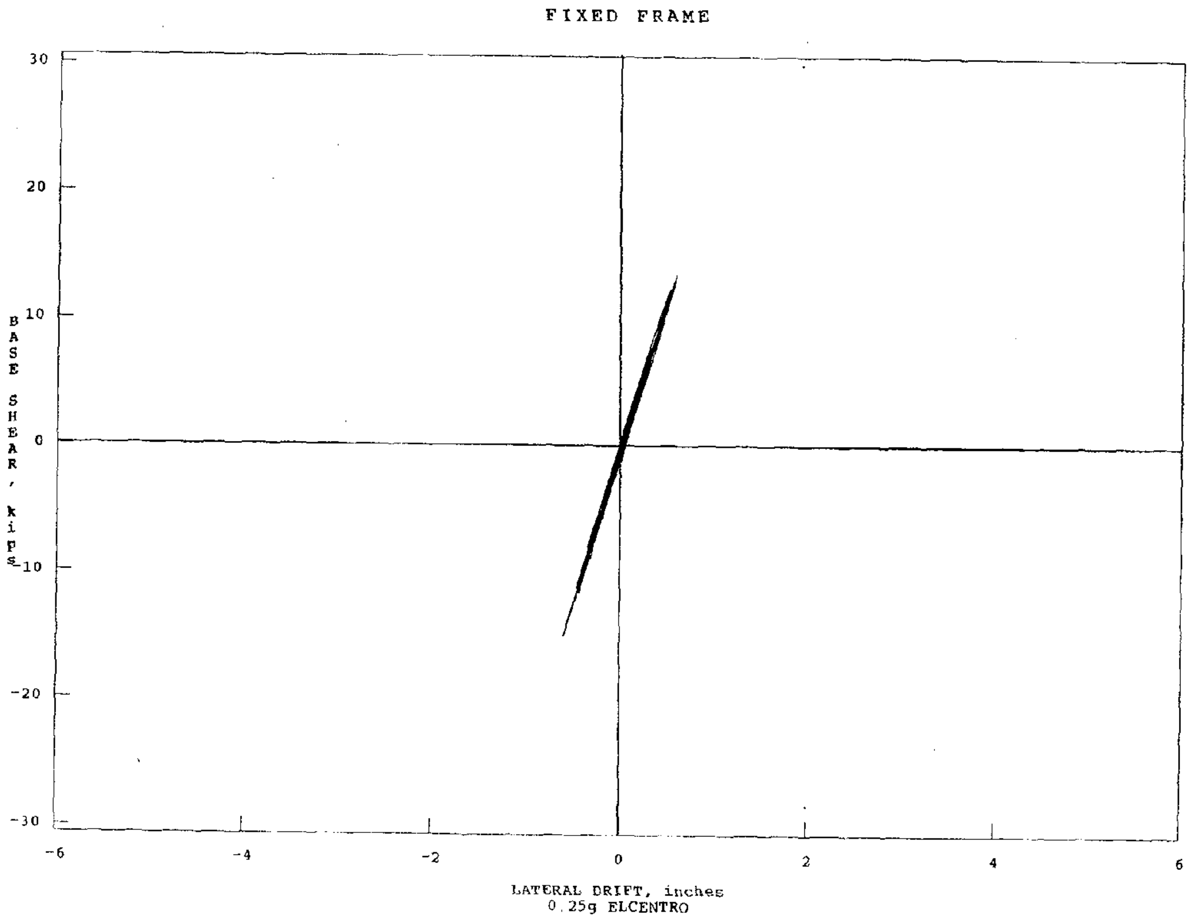


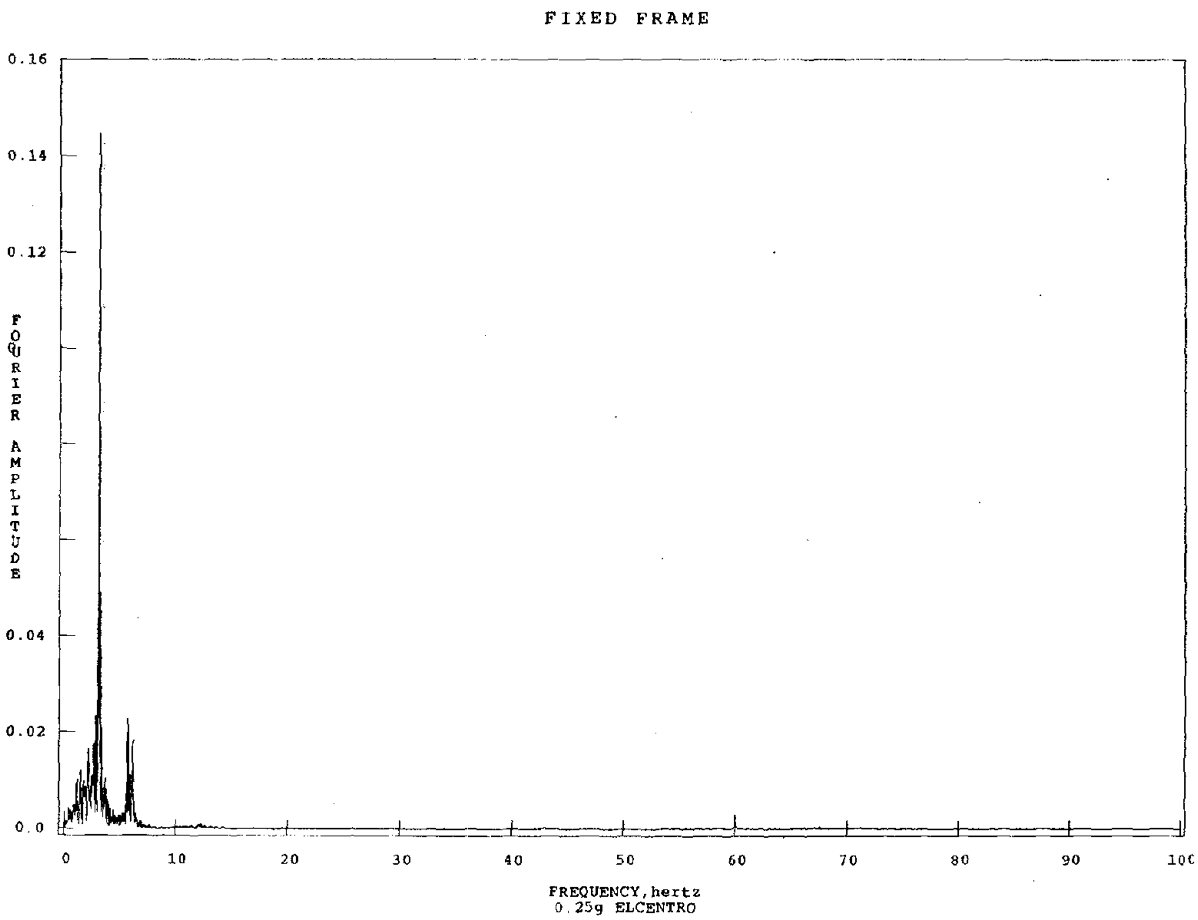
FIXED FRAME




FIXED FRAME
BASE SHEAR, kips
30
20
10
0
-10
-20
-30
0
10
20
30
40
TIME, sec
0.25g ELCENTRO


FIXED FRAME
LATERAL DRIFT, inches
6
4
2
0
-2
-4
-6
0
10
20
30
40
TIME, sec
0.25g ELCENTRO


FIXED FRAME
30
20
10
0
-10
-20
-30
BASE SHEAR, kips
-6
-4
-2
0
2
4
6
LATERAL DRIFT, inches
0.25g ELCENTRO


FIXED FRAME
0.16
0.14
0.12
0.04
0.02
0.0
FOURIER AMPLITUDE
0
10
20
30
40
50
60
70
80
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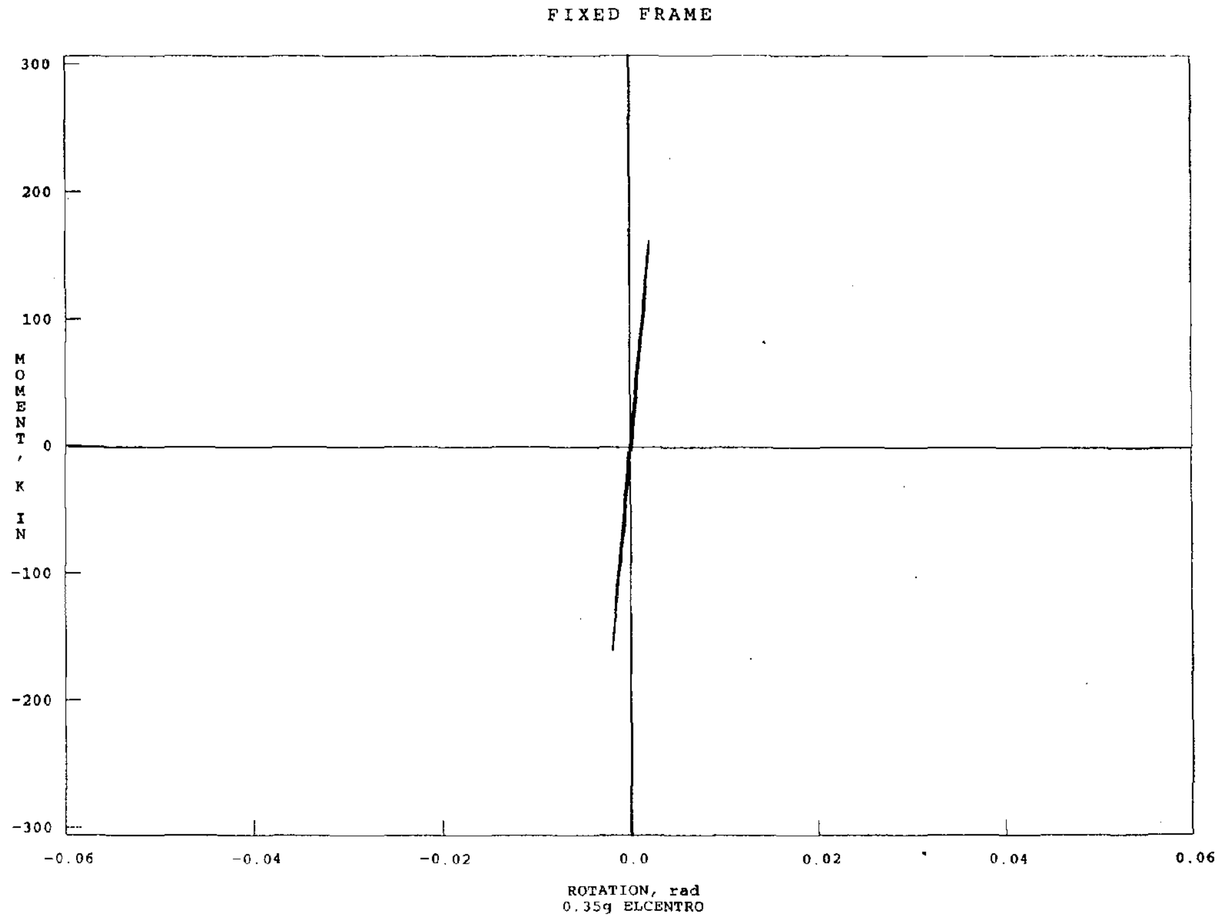
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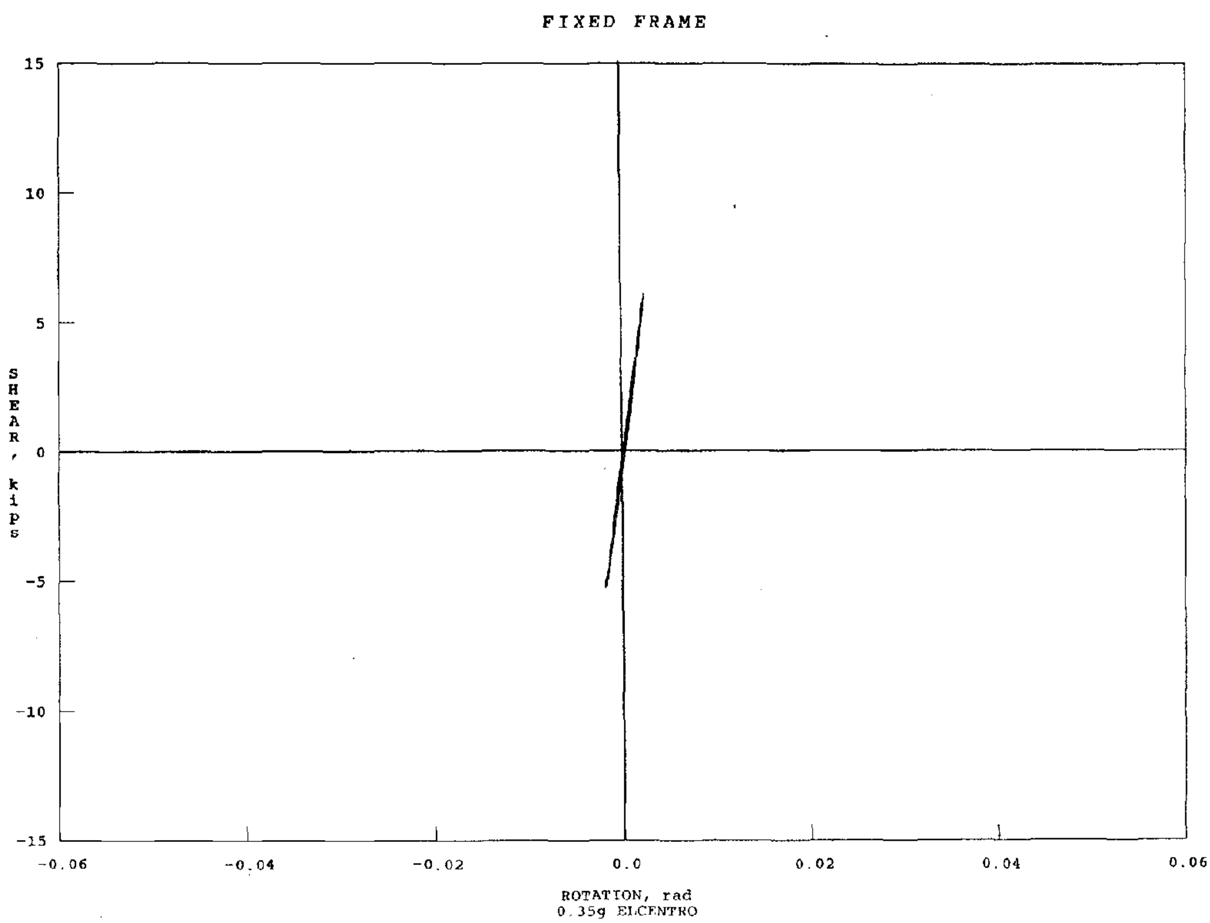
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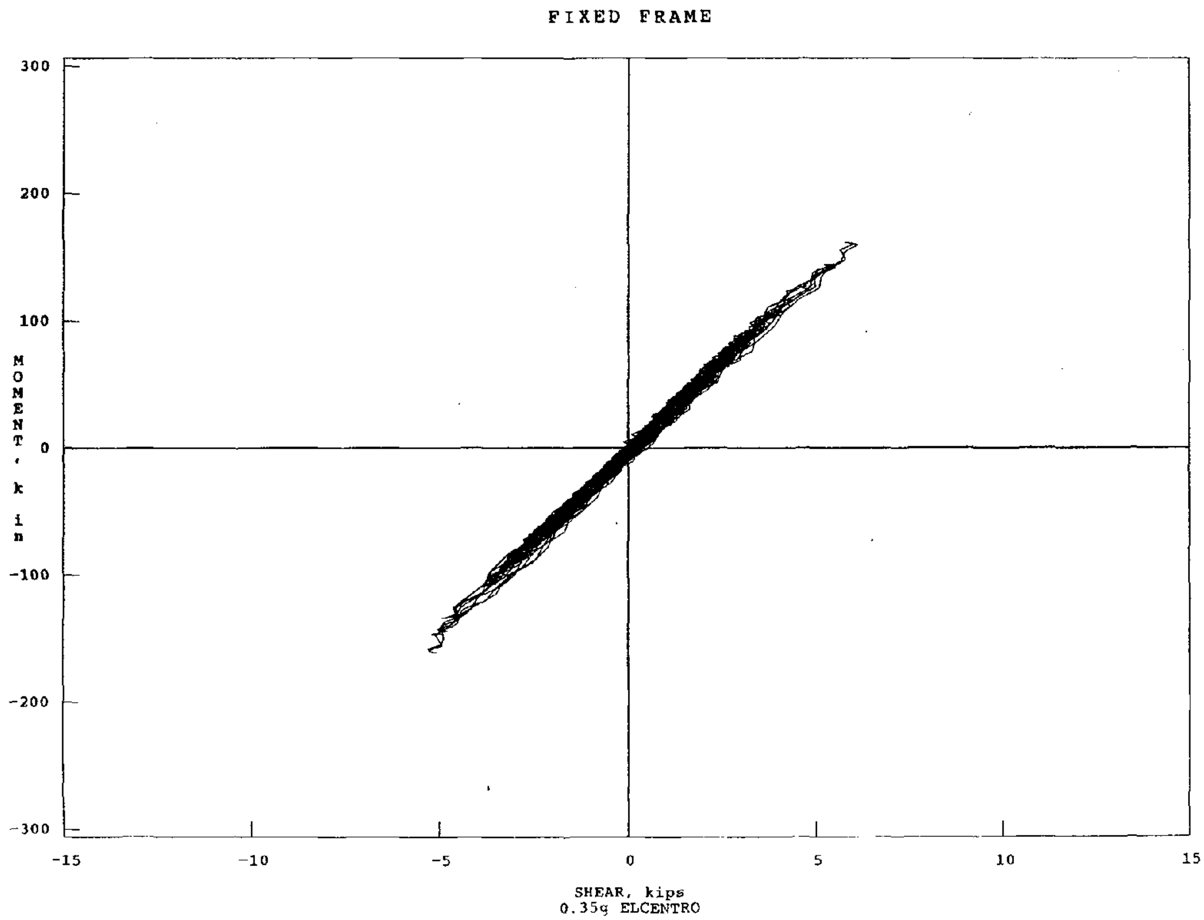
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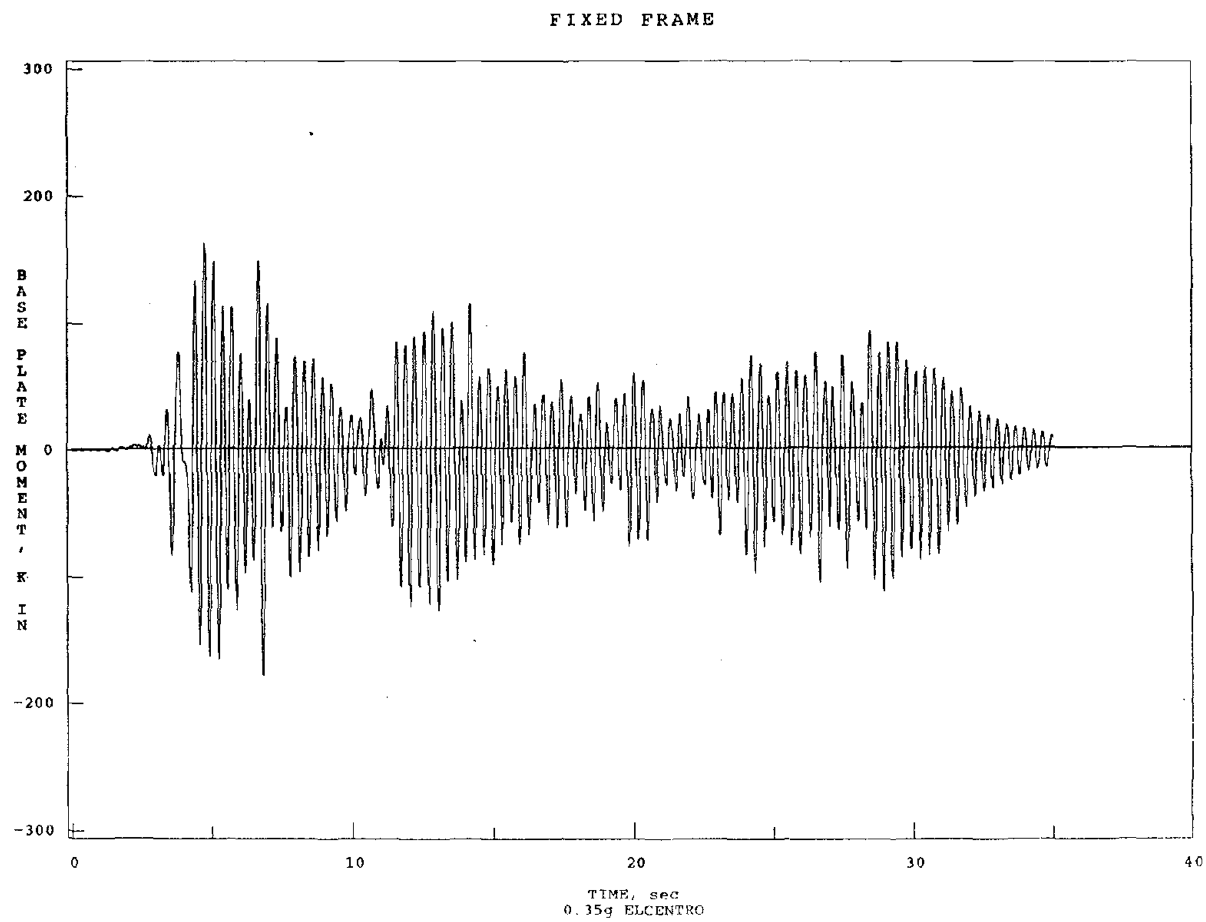
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-300
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10
20
30
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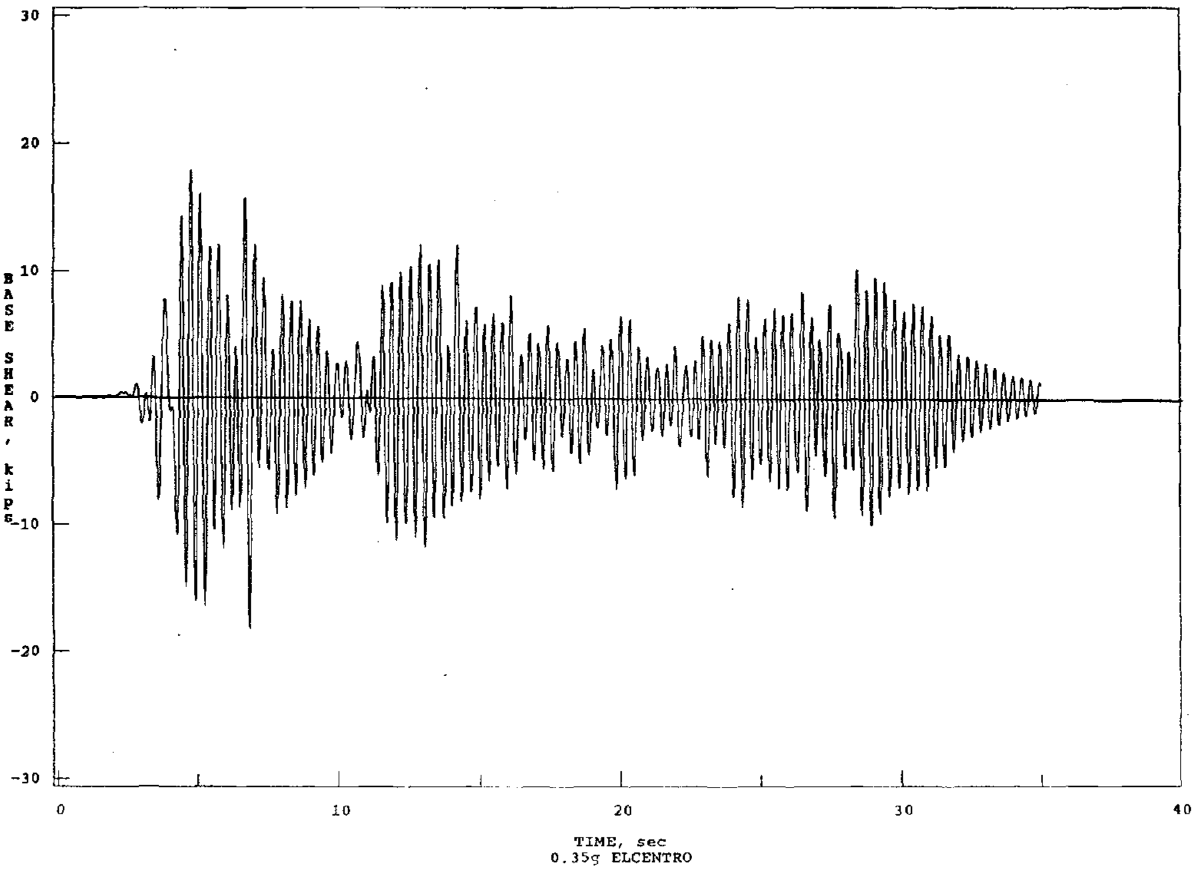

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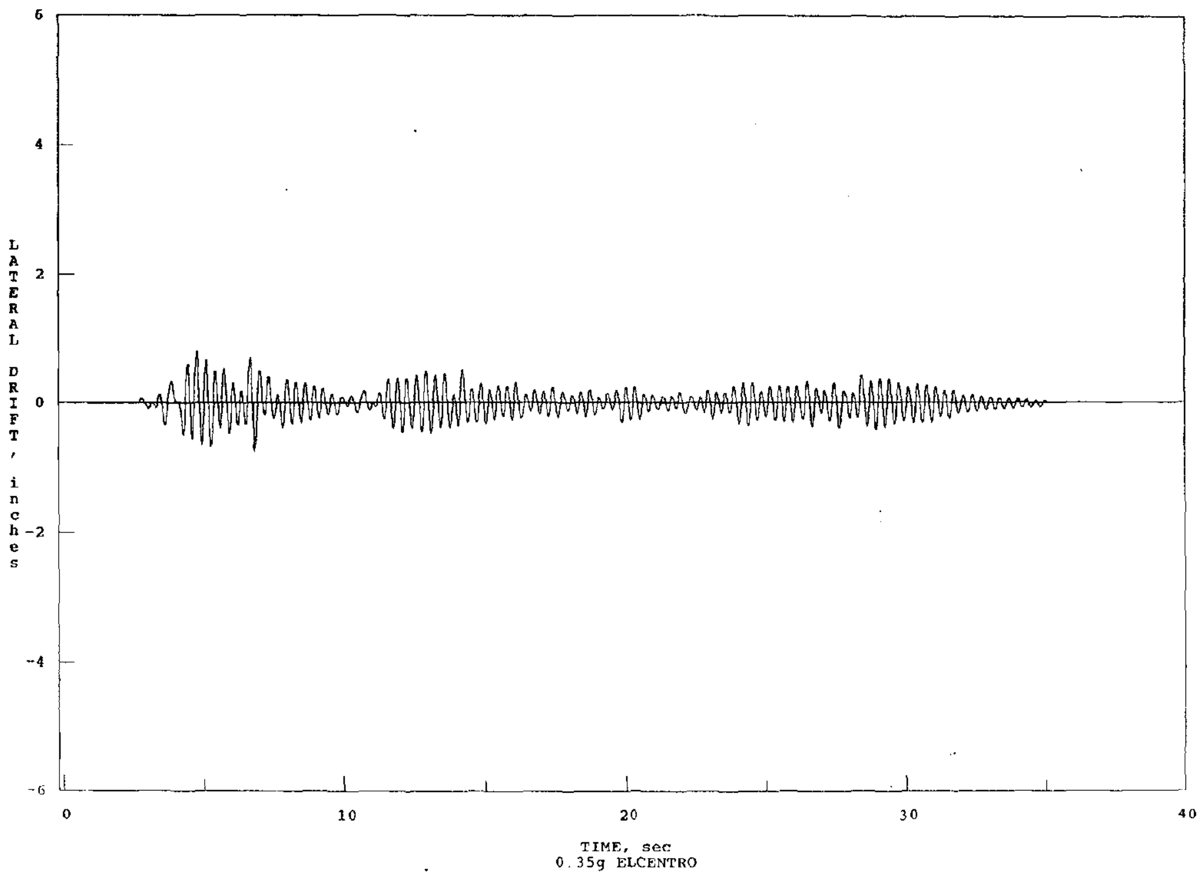

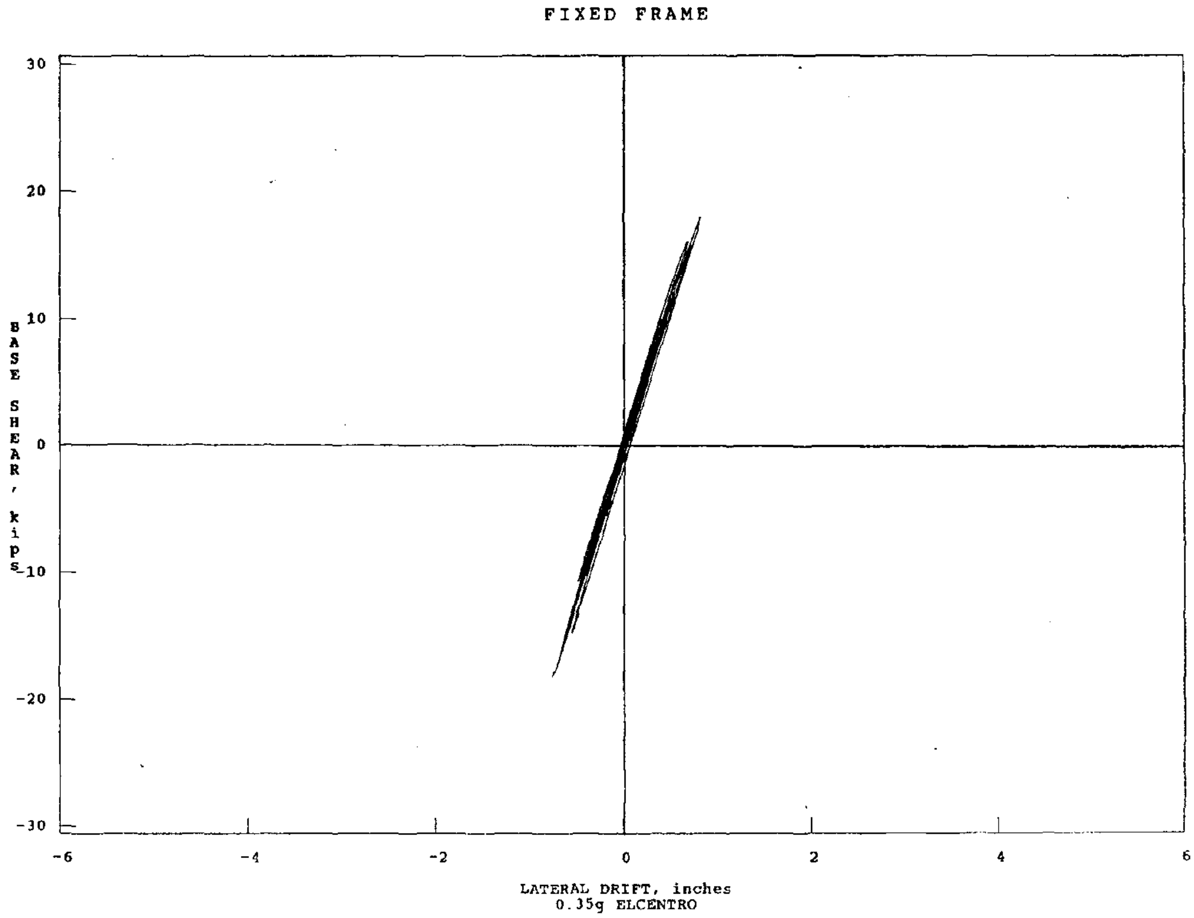
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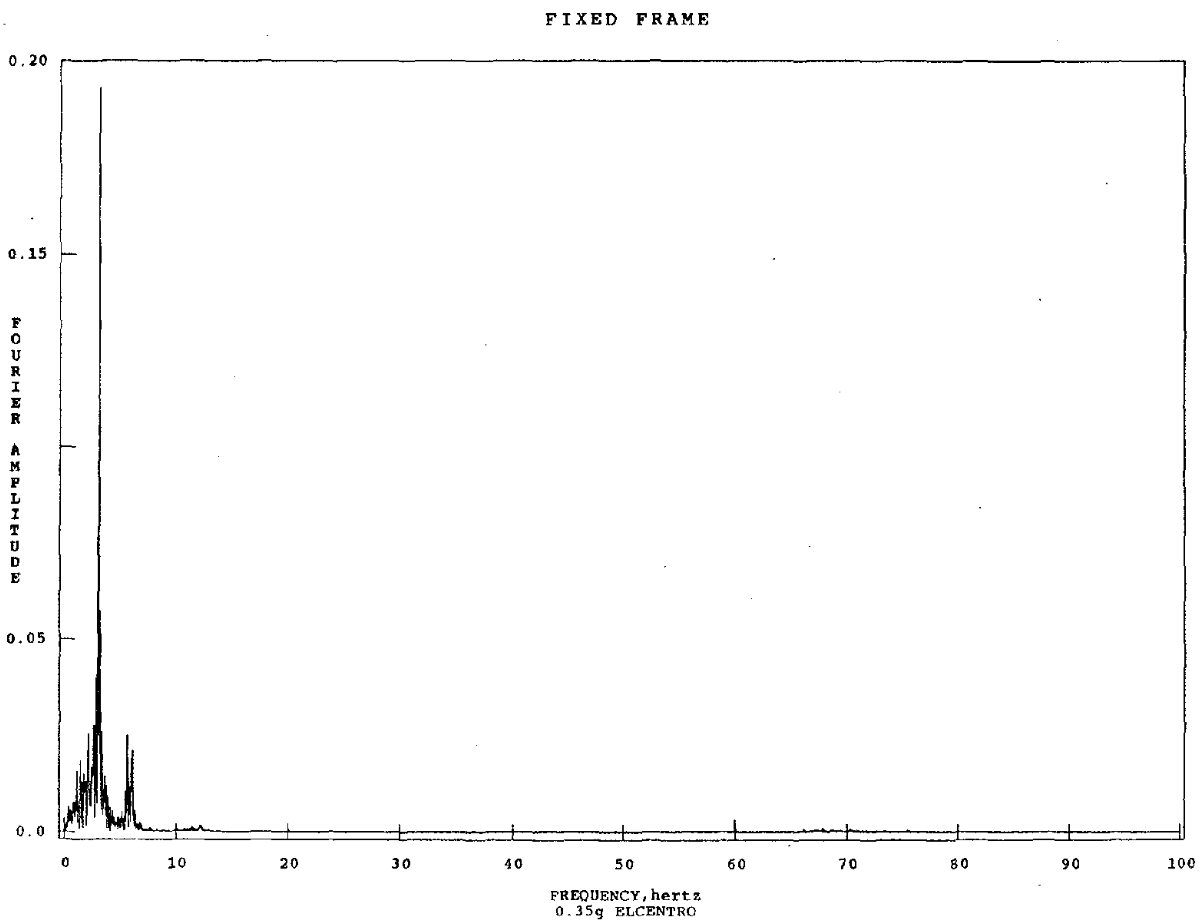
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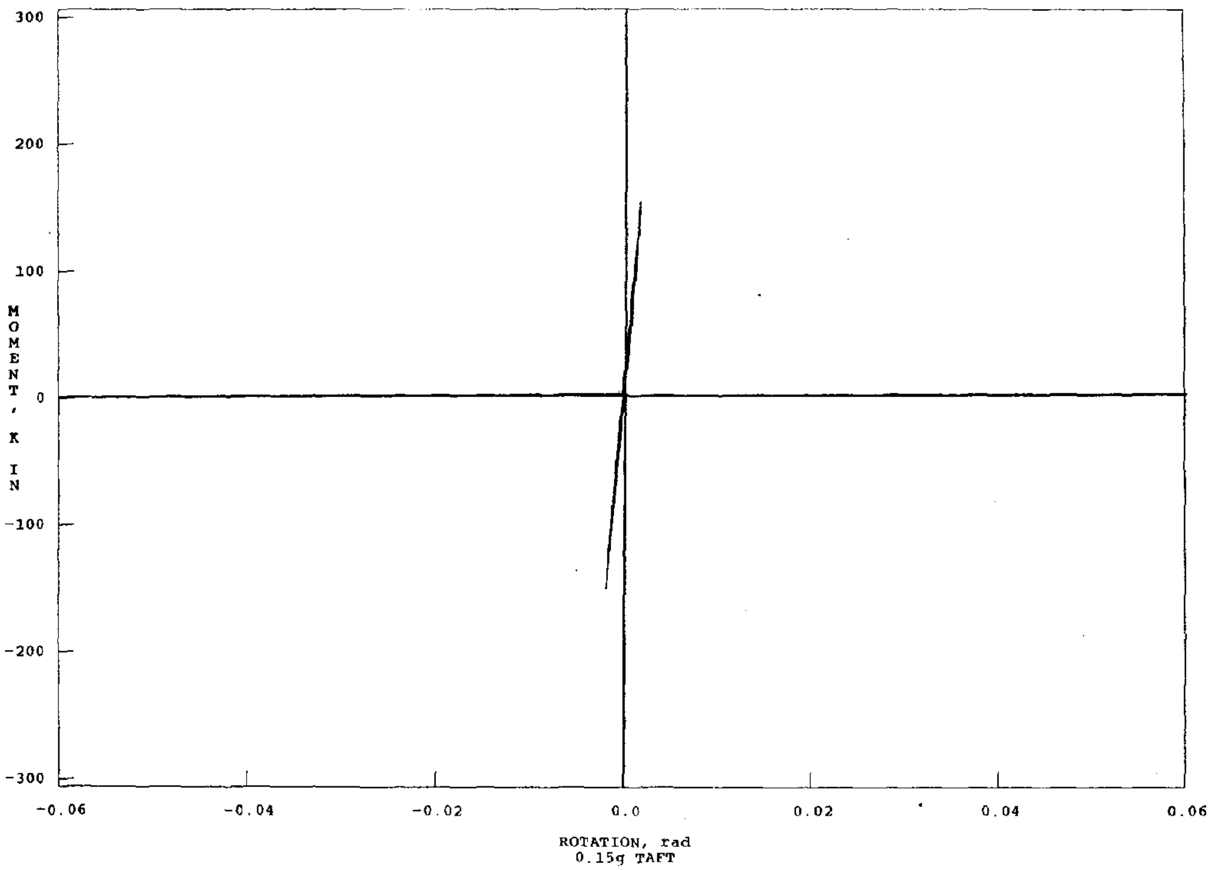

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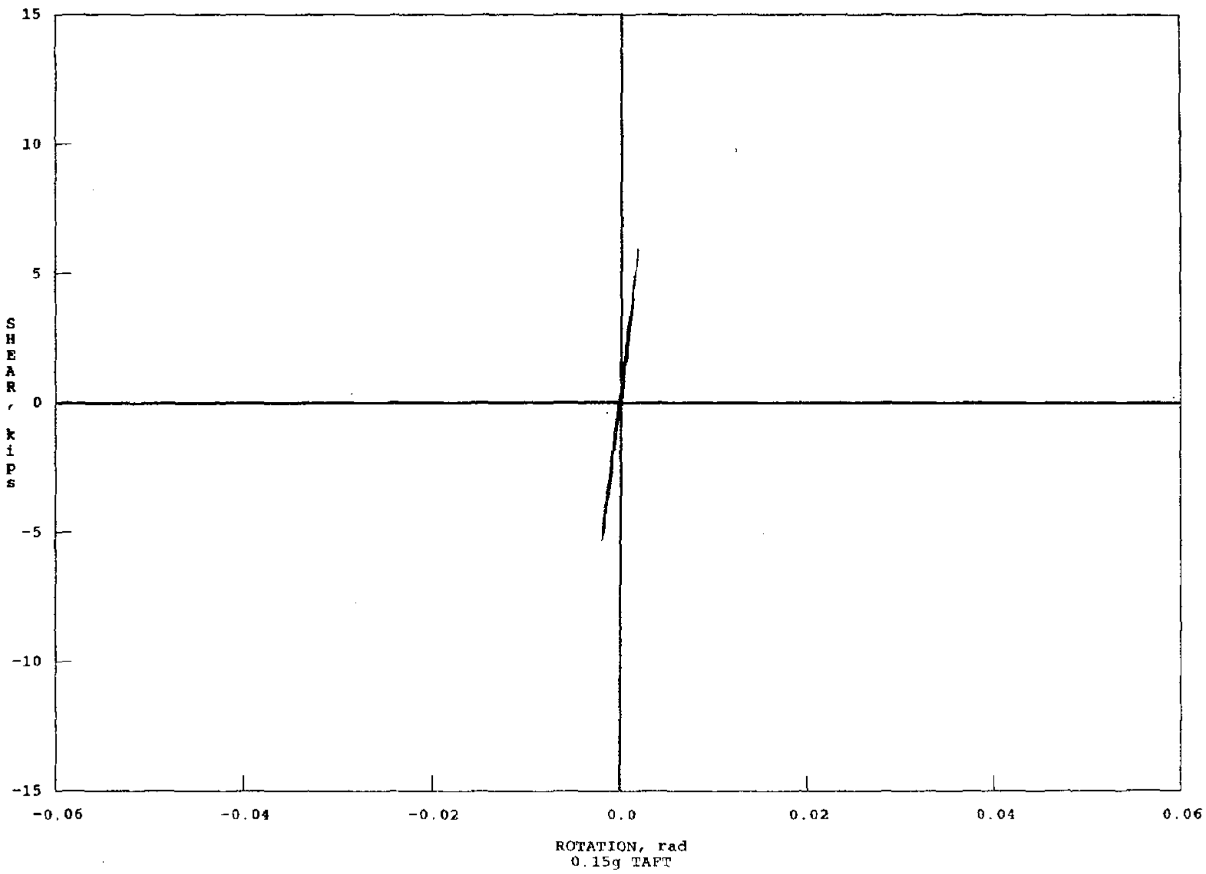

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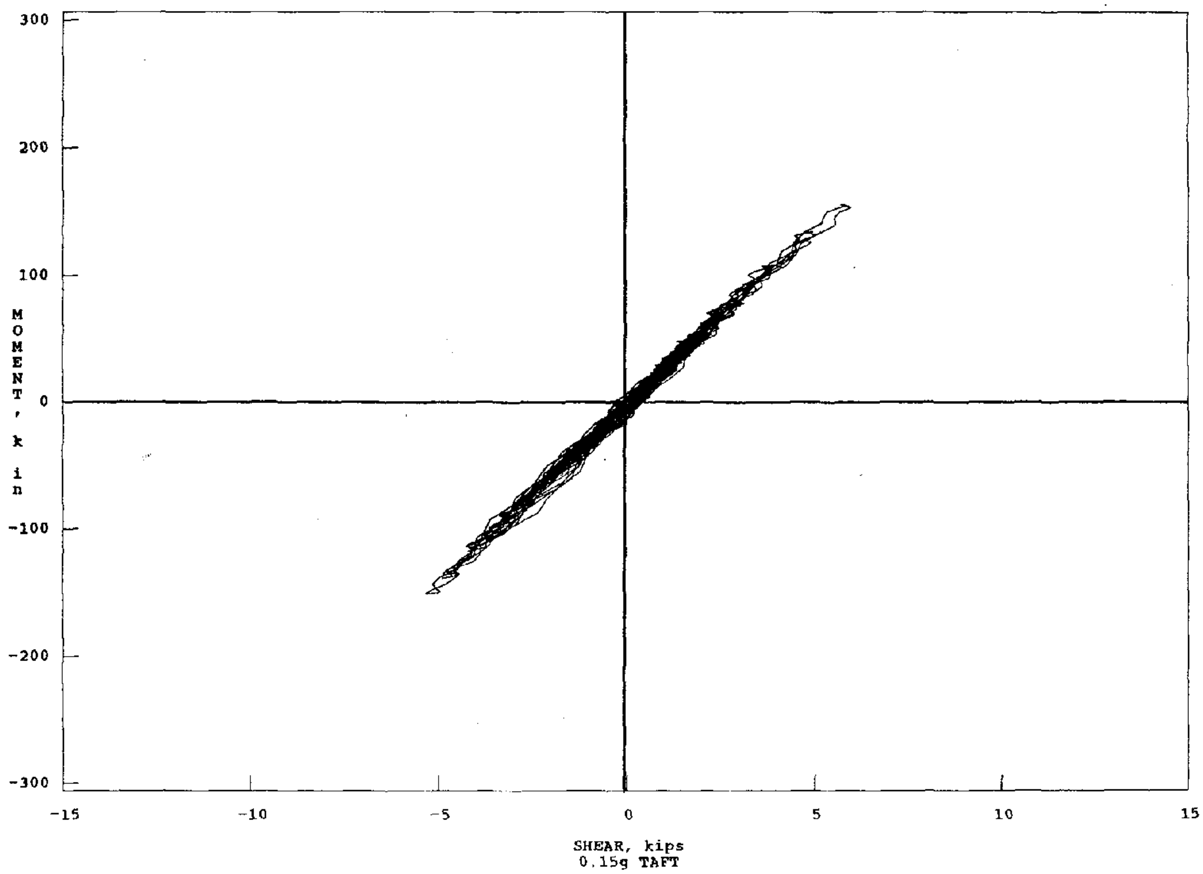

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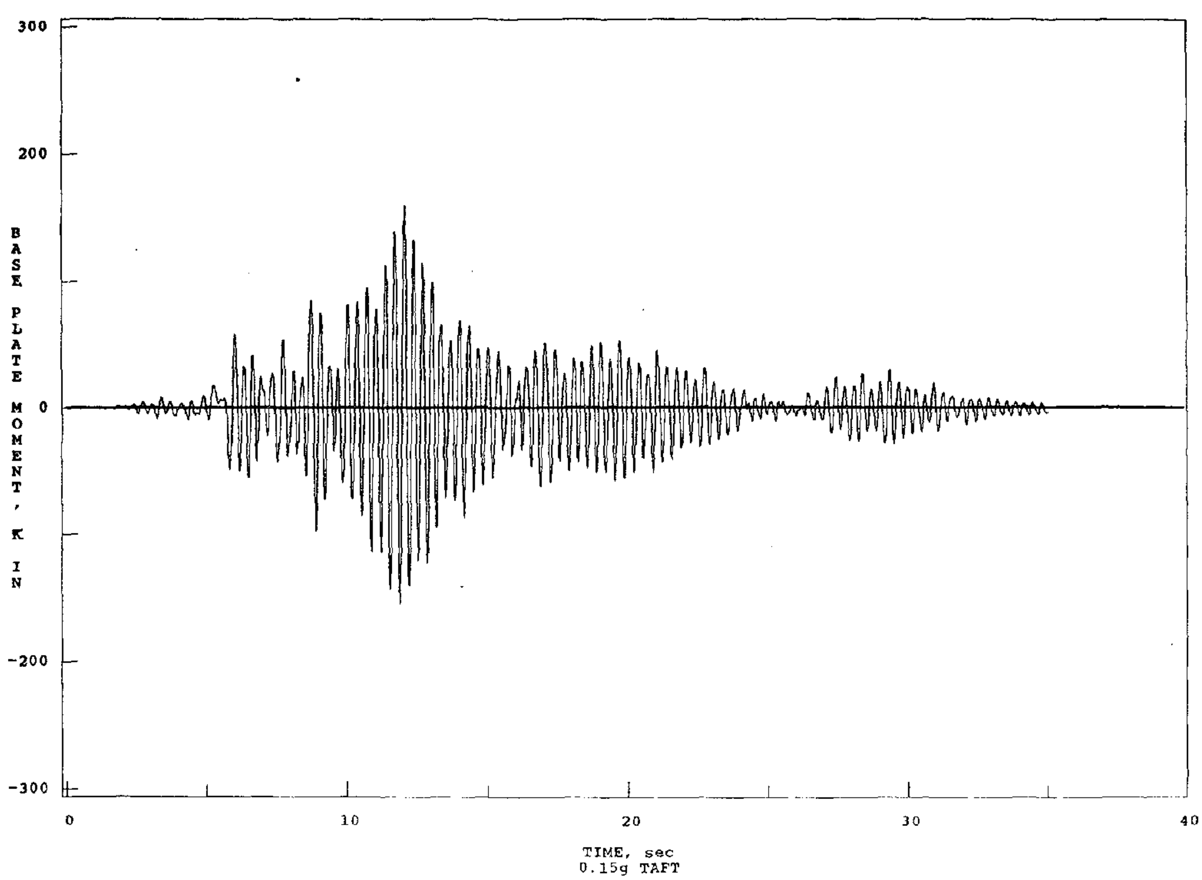

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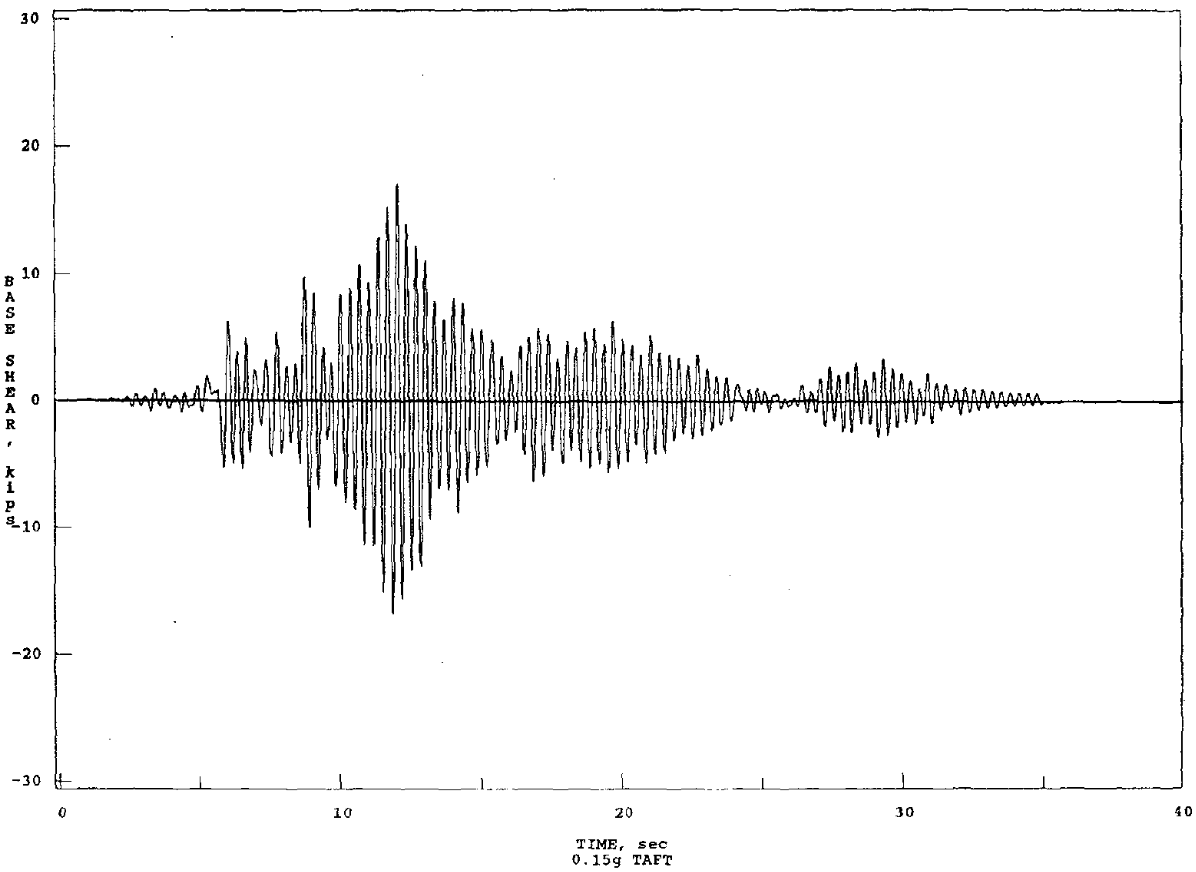

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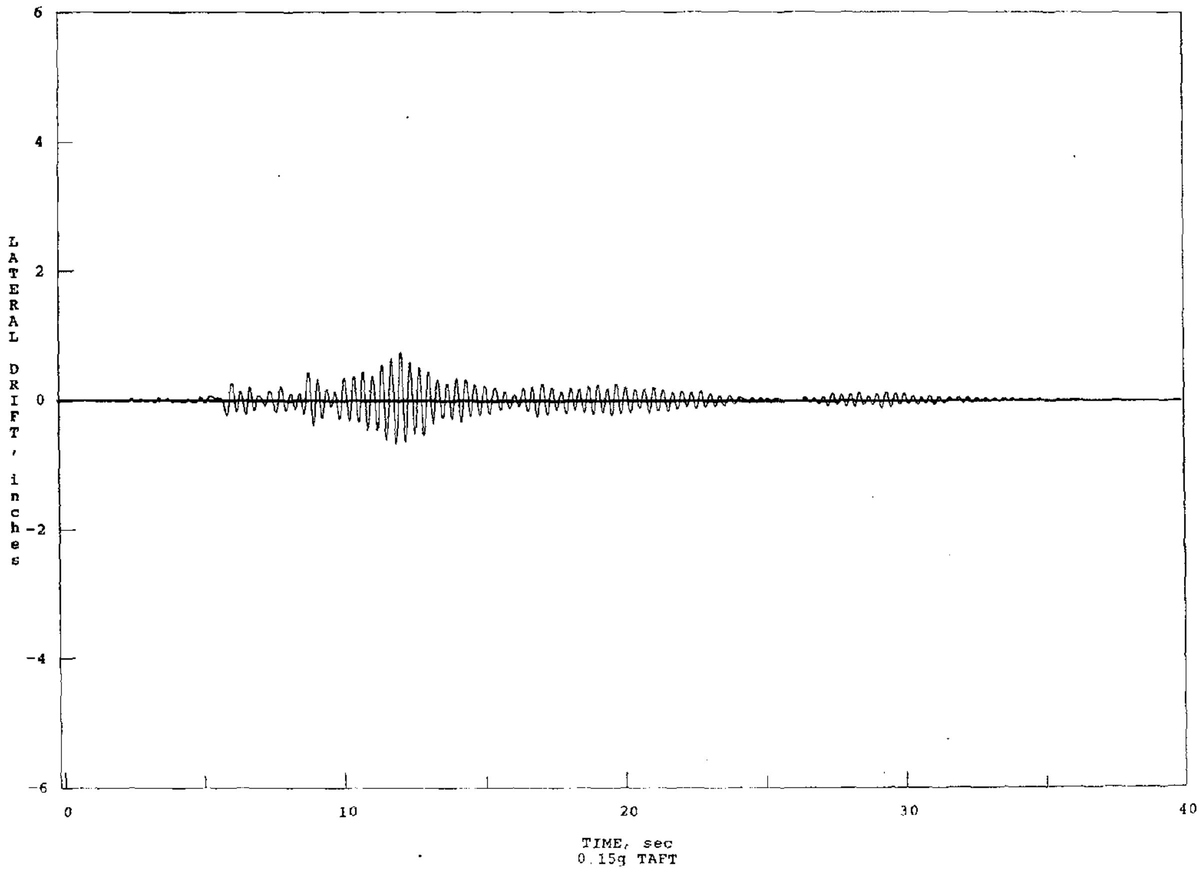

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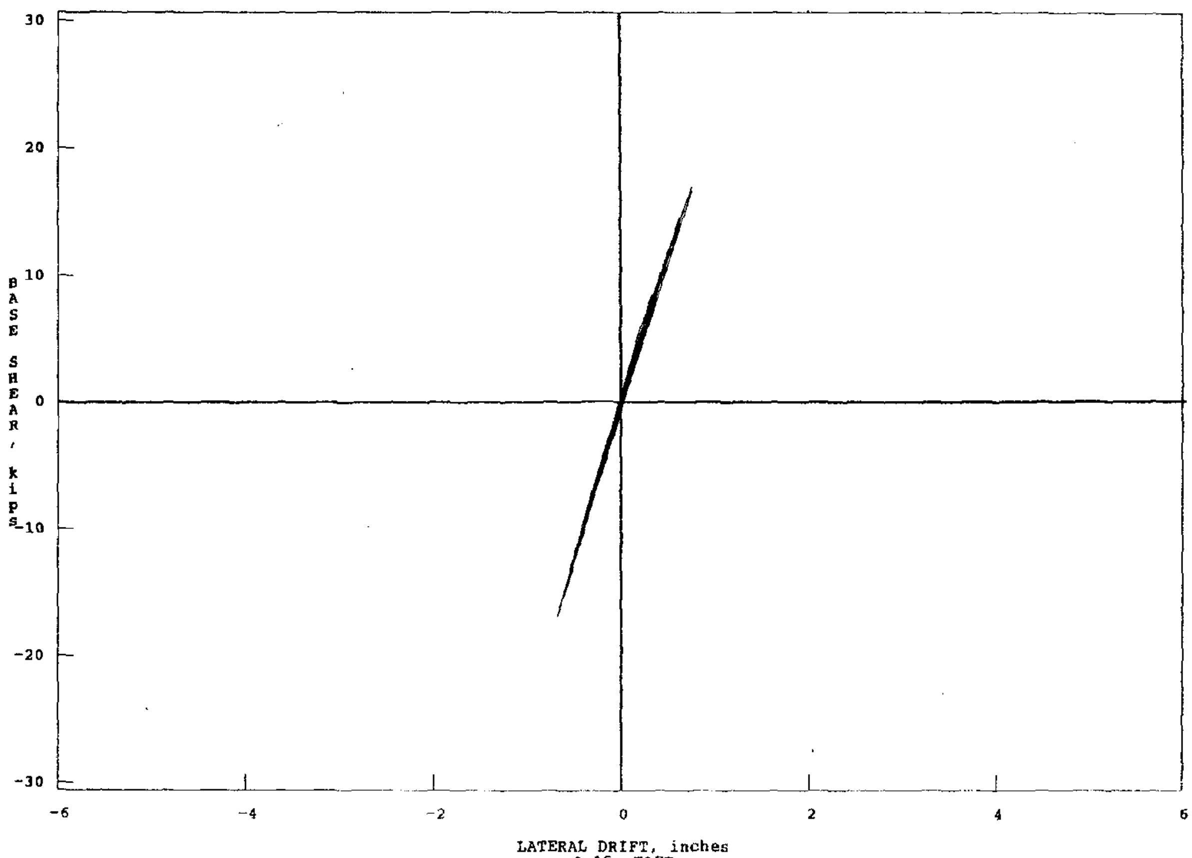

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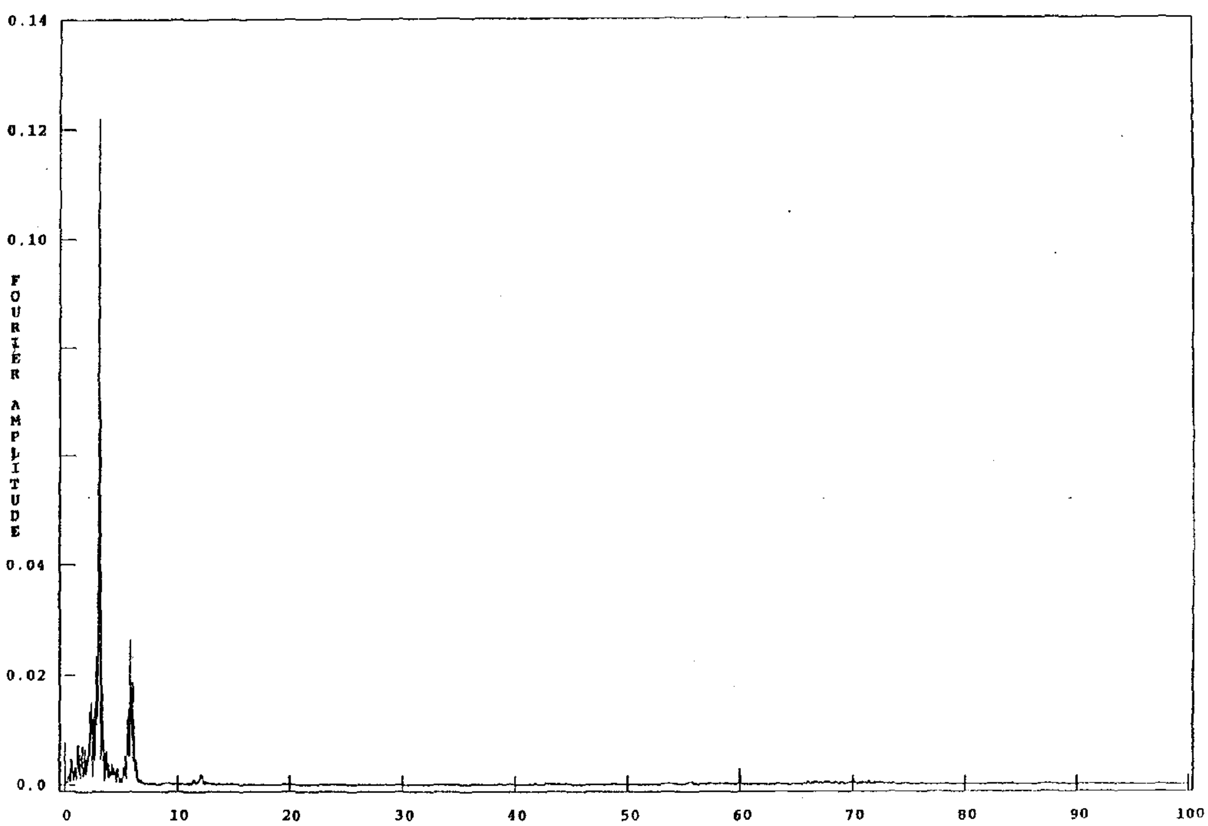

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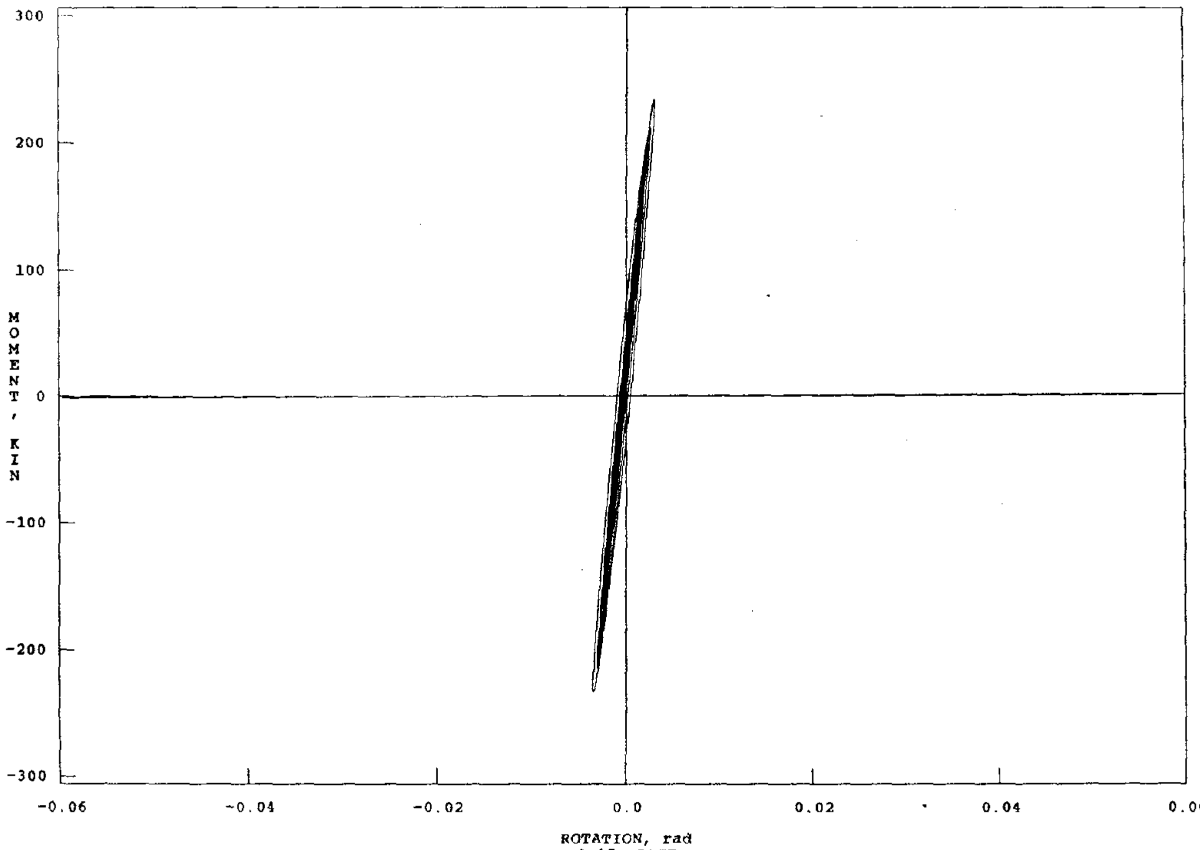

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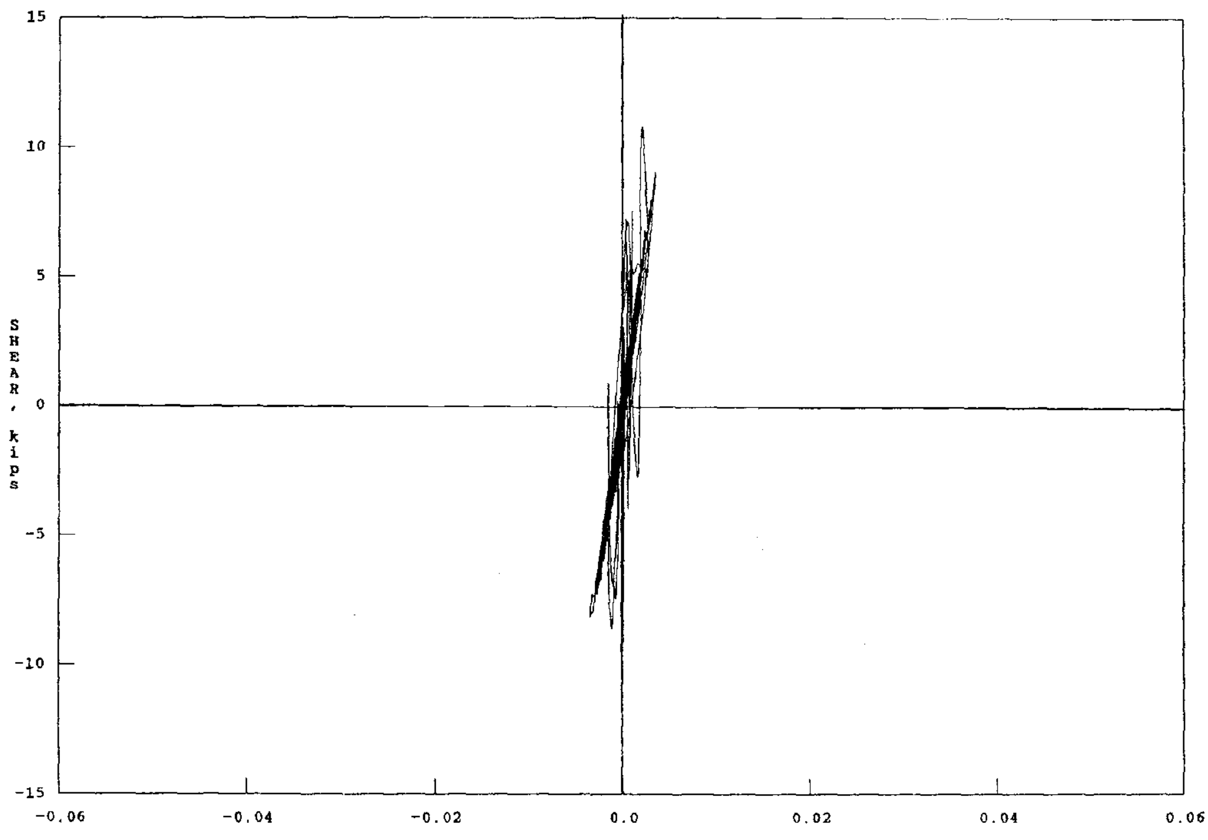

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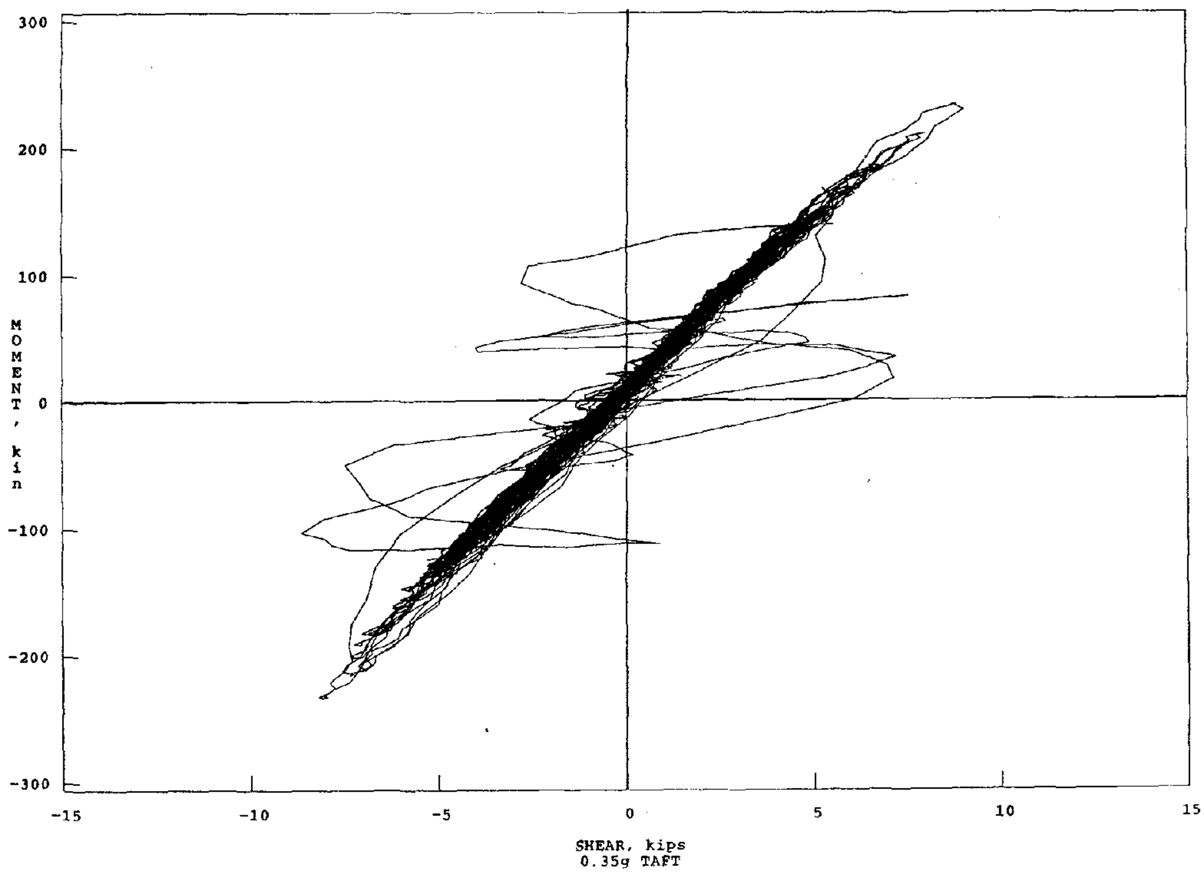

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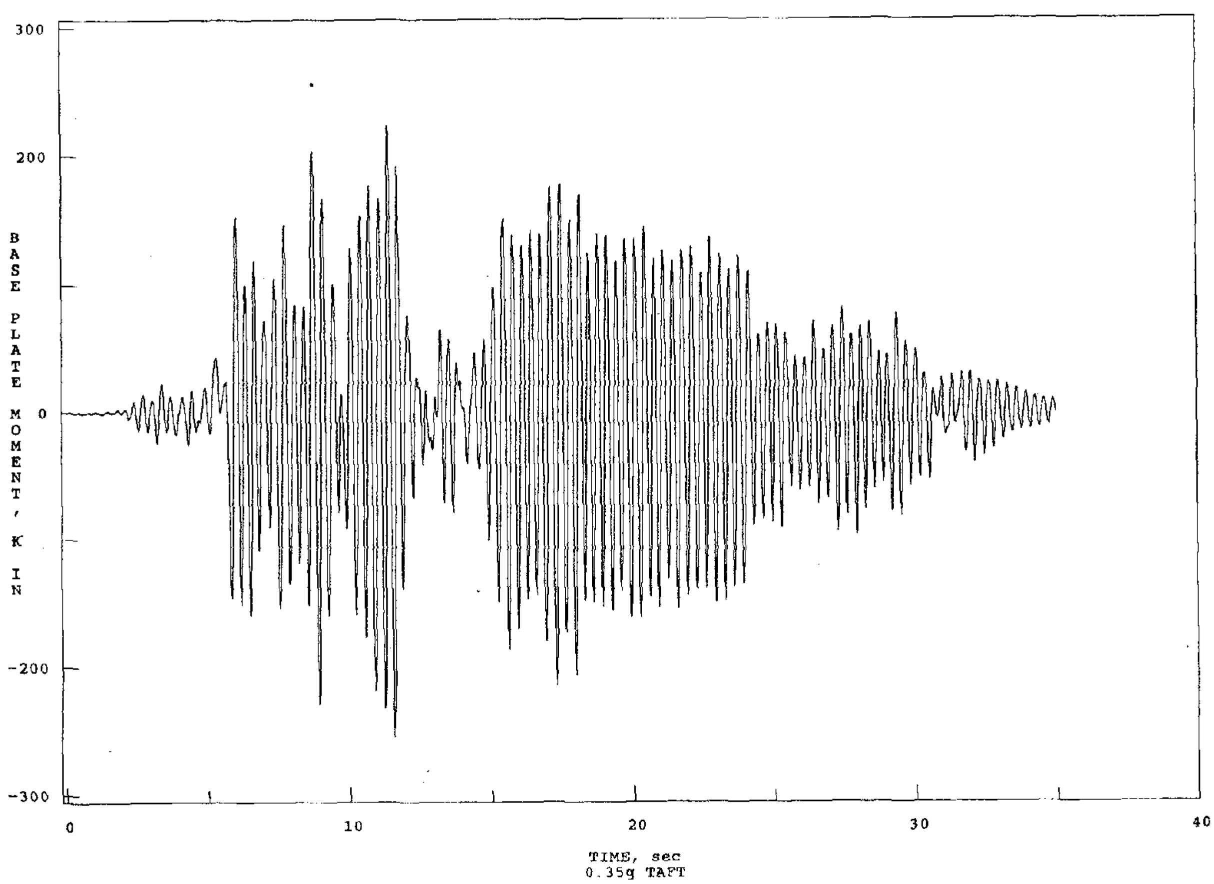

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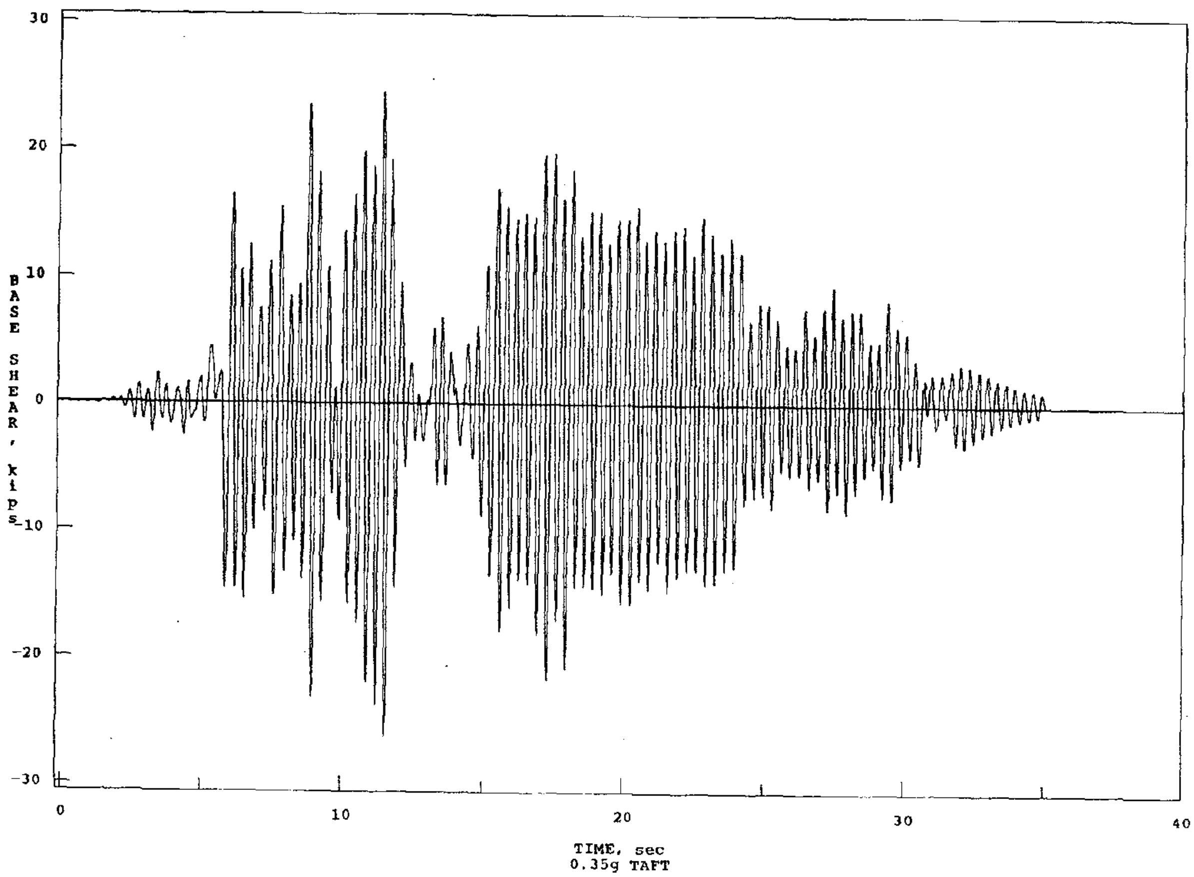

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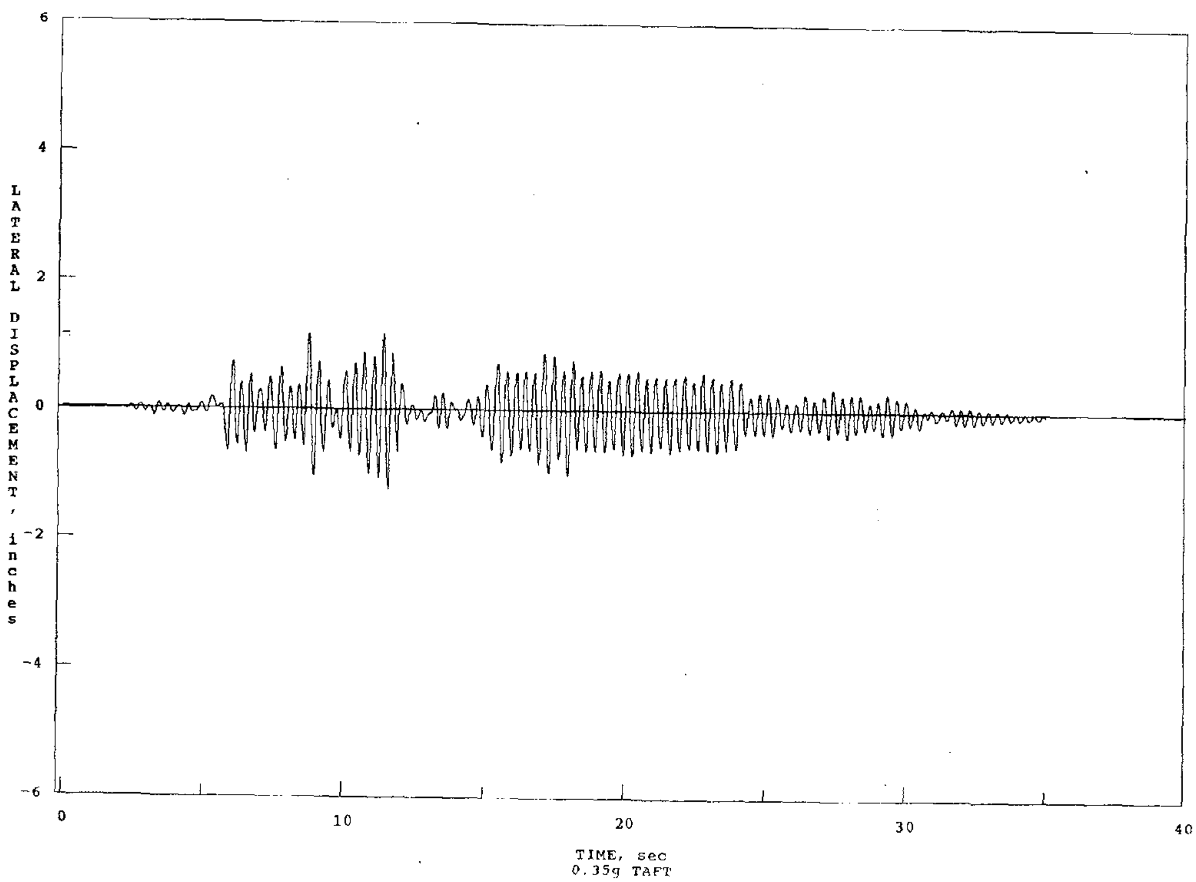

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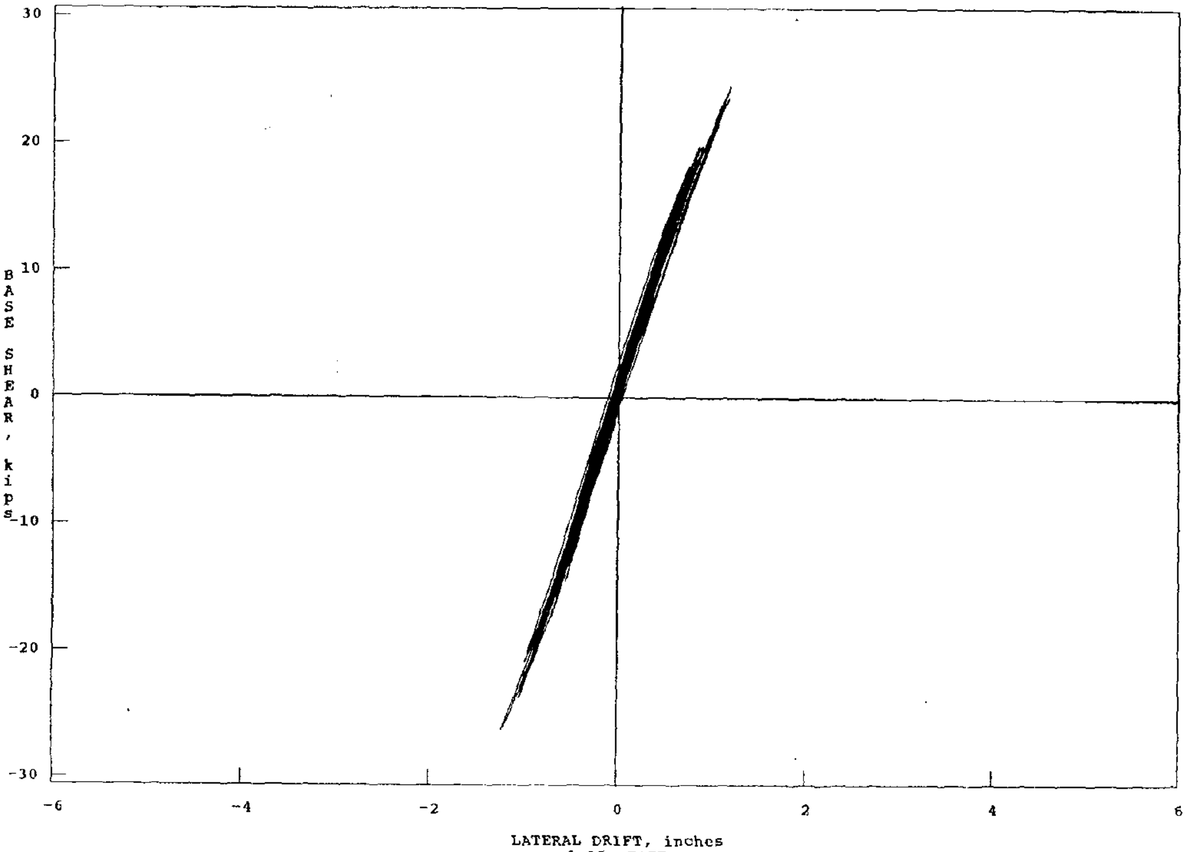

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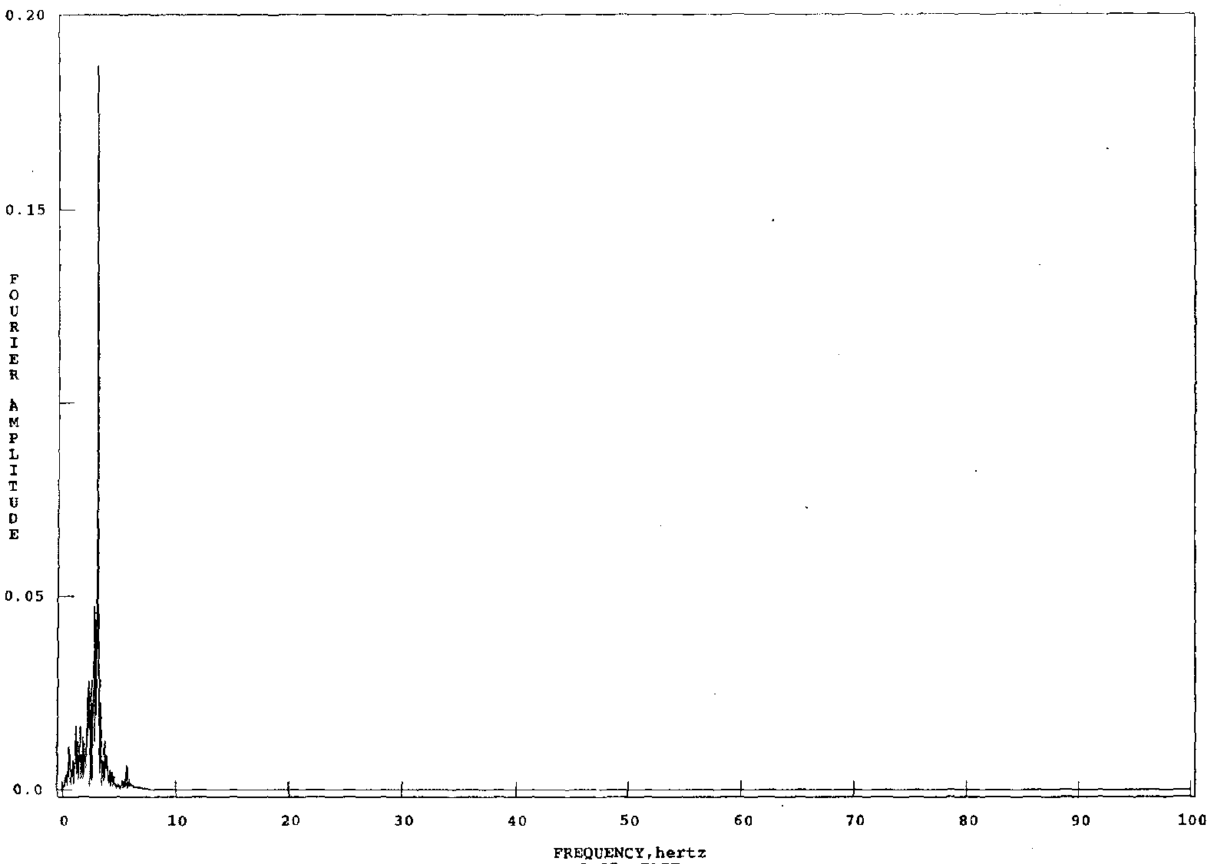


0.35g TAFT

APPENDIX B

COUPON TESTS

The stress-strain relation of different parts of the structure were developed, using standard ASTM coupon specimens. The basic interest was to find the stress-strain curve of the connection angles, since most of the yielding was in that area. The behavior of the columns, in the areas near the connection, was also inelastic. This inelastic behavior, made it necessary to find the stress-strain curve of the column so as to understand the behavior of the structure.

Since it had been anticipated that the yield point for material in different parts of the rolled sections would be different due to variation in rolling of web and flanges, it was decided that the mechanical properties should be identified for each pert of the sections.

As indicated in Figure B.1, a total of six coupon specimens, three for W4x13, two for L2x3-1/2x3/16, and one for L2x2x3/16 were prepared for the uniaxial monotonic axial tests. The tests were run by a 120 kip Baldwin machine, the output of which was fed to the vertical input of an XY recorder. The strains were monitored by extentiometers consisting of two LVDT's with a gage length of 1 inches, whose output served as horizantal input of the XY plot. The load rate used was 20,000 #/min.

Typical Stress-strain curves are shown in Figure B.2. Table B.1 summarizes the findings.

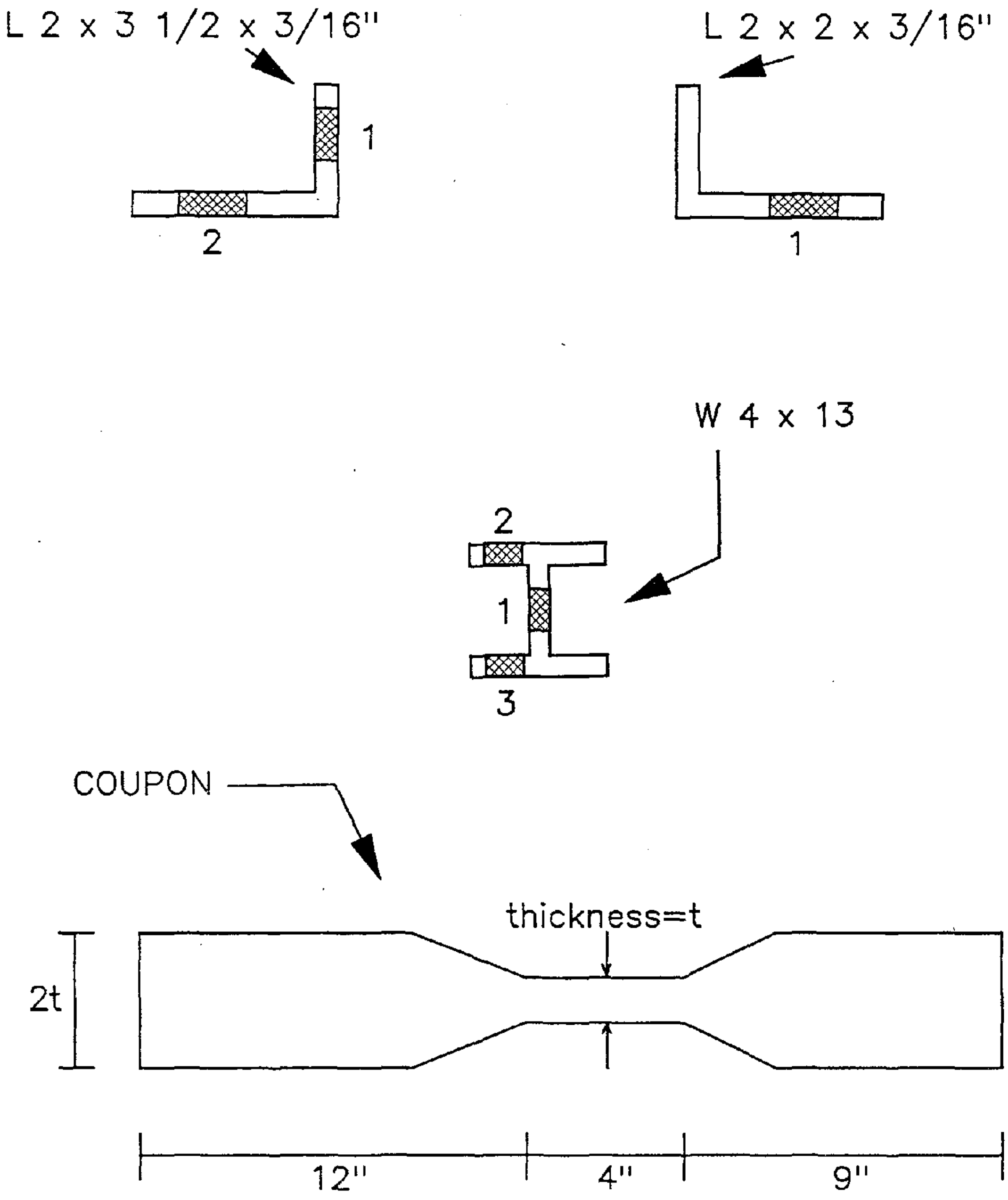
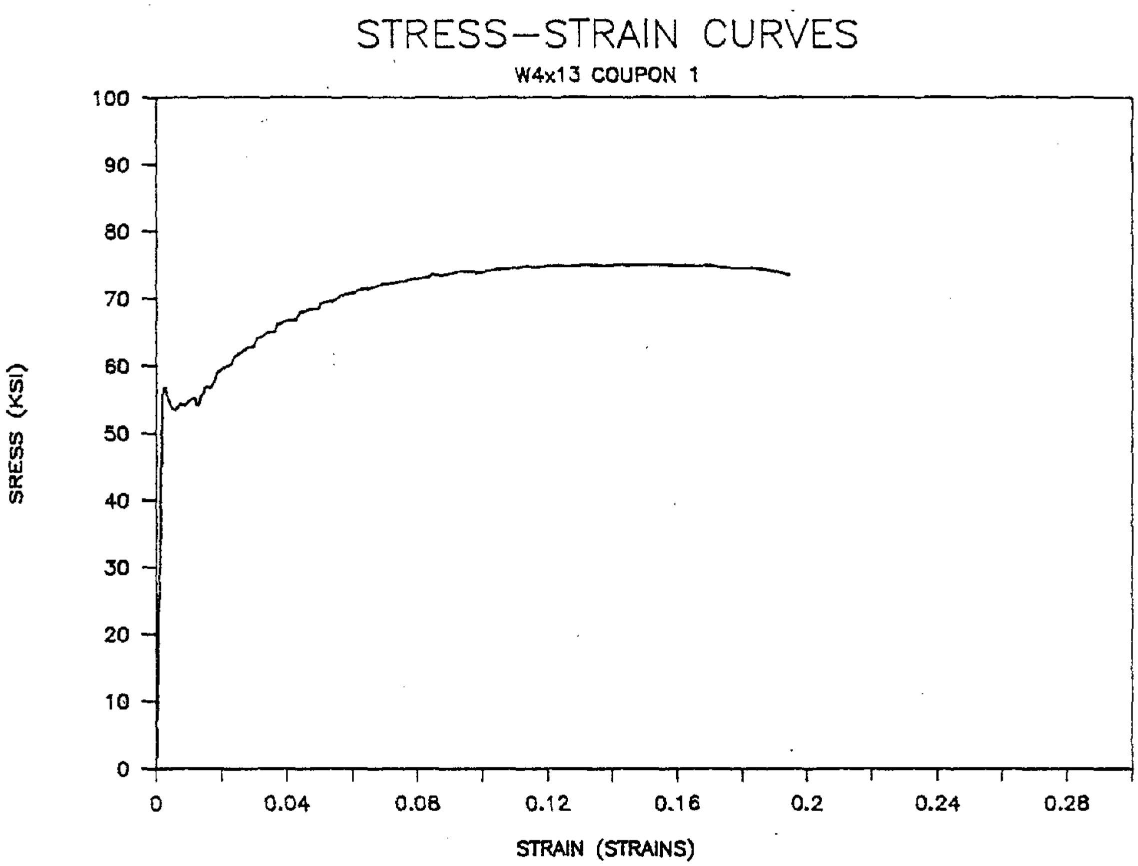


Figure B.1. Coupon Specimens



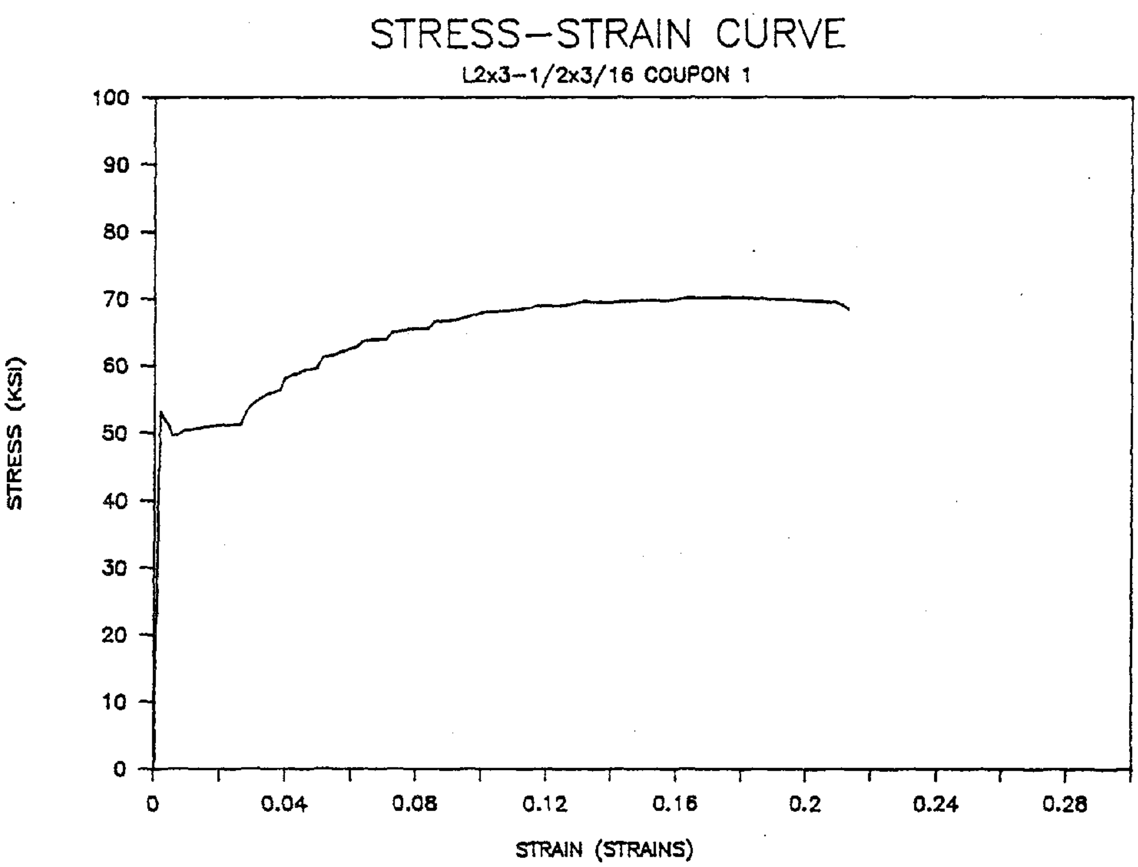


Figure B.2. Stress-Strain Relationship

TABLE B.1. Coupon Test Results

STRESS-STRAIN RELATIONSHIP						
Variable	Column W4x13			L2x3-1/2x3/16		L2x2x3/16
Specimen #	1	2	3	1	2	1
Avg. Thickness (in)	0.312	0.329	0.338	0.190	0.191	0.188
Avg. Area (in^2)	0.232	0.244	0.250	0.141	0.141	0.069
Mod. of Elasticity (10^3 ksi)	29.03	30.49	29.57	29.20	28.70	29.04
F_y upper (ksi)	56.55	50.08	48.00	61.90	52.70	52.03
F_y lower (ksi)	53.86	47.20	47.00	59.70	49.80	52.03
ϵ_y (strain)	0.022	0.016	0.018	0.018	0.018	0.017
$E_{str.hard}$ (ksi)	48.00	45.00	41.00	37.00	44.00	50.00
$F_{ult.}$ (ksi)	73.52	70.00	68.40	74.02	69.40	NA
ϵ_{ult} (strain)	0.196	0.210	0.225	0.200	0.210	NA

NA = test was stopped before reaching ultimate strength due experimental problems

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