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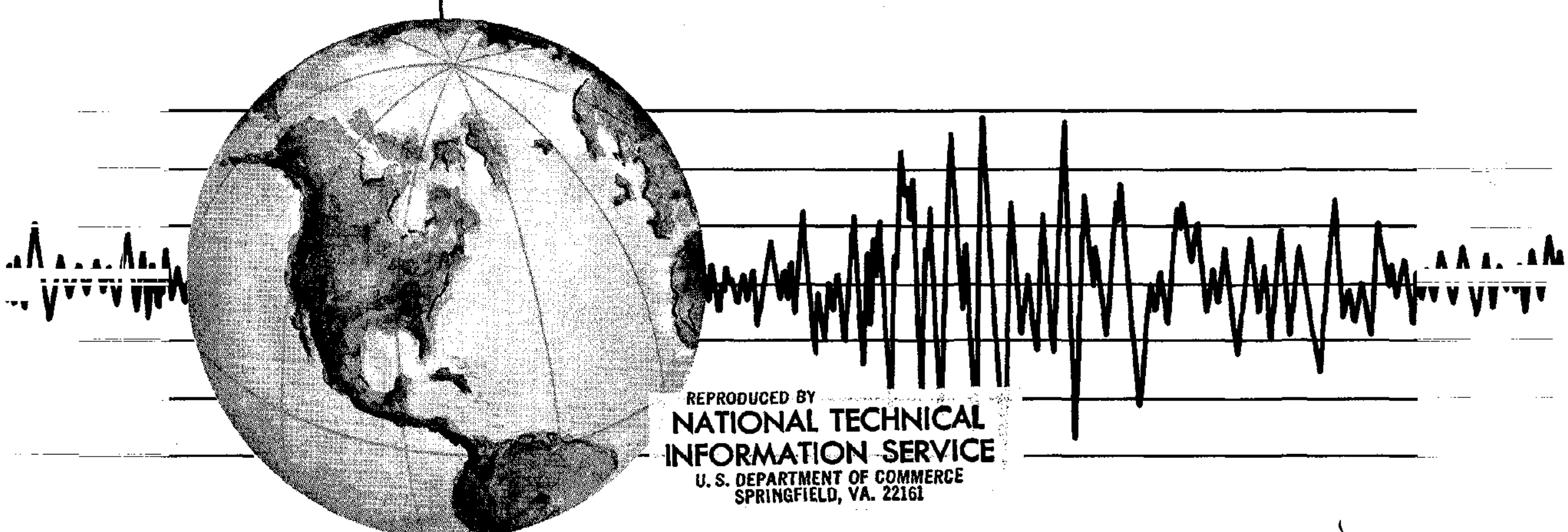
TESTING OF A WIND RESTRAINT FOR ASEISMIC BASE ISOLATION

by

J. M. Kelly
and
D. E. Chitty

Report to:
National Science Foundation
and the
Malaysian Rubber Producers' Research Association



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Base isolation has been proposed as an economical approach to aseismic design for many types of buildings and structural systems. In general, the base of a structure is isolated when a support system with extremely low horizontal stiffness limits the transmittal of horizontal shear from the ground to the structure. The displacement of such a structure under wind loading will, however, be excessive since the horizontal stiffness of the support system is so low. The horizontal deflection of a building with an isolated natural frequency of 0.5 Hz would, for example, be four inches for a wind load of one-tenth the weight of the building. While such a horizontal deflection would not be acceptable, a base isolation system could be equipped with a mechanical fuse which would be sufficiently strong to resist wind loading, but which would fracture during an earthquake, leaving the building free on the isolation system.

In this report we describe an extensive series of shaking table tests of such a device. The wind restraint consisted of a notched shear pin. Several shear pins were tested in conjunction with a natural rubber isolation system placed beneath a three-story 40,000 lb. steel frame model structure. Pins with breaking forces ranging from 3% to 20% of the weight of the model were tested. The model structure was subjected to various peak accelerations of three earthquake inputs.

The shear pins fractured rapidly and cleanly. The breaking force for each pin was reasonably predictable. Although higher mode accelerations were induced in the model structure when the pins broke, these disappeared rapidly. The design of a shear pin mechanical fuse system for a full-scale structure is discussed at the end of this report in view of the experimental results described.

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Earthquake Engineering Research Center
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September 1978

ABSTRACT

Base isolation has been proposed as an economical approach to aseismic design for many types of buildings and structural systems. In general, the base of a structure is isolated when a support system with extremely low horizontal stiffness limits the transmittal of horizontal shear from the ground to the structure. The displacement of such a structure under wind loading will, however, be excessive since the horizontal stiffness of the support system is so low. The horizontal deflection of a building with an isolated natural frequency of 0.5 Hz would, for example, be four inches for a wind load of one-tenth the weight of the building. While such a horizontal deflection would not be acceptable, a base isolation system could be equipped with a mechanical fuse which would be sufficiently strong to resist wind loading, but which would fracture during an earthquake, leaving the building free on the isolation system.

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1. INTRODUCTION

Results of tests of a simple mechanical fuse designed for use in conjunction with an earthquake-resistant base isolation system that incorporates natural rubber bearings are reported. The base isolation system was suggested by Derham, Wootton, and Learoyd [1]; a practical system has been developed and tested [2, 3]. The lateral stiffness of the natural rubber foundation bearings is very low; structures and their contents are protected from the effects of earthquakes because buildings on these bearings are effectively uncoupled from ground motion. The bearings are able to undergo relatively large (on the order of several inches) horizontal displacement. When a structure rests on such a system, the first mode period of the building is increased to a range above that of most earthquake energy and the lateral earthquake load on the structure is thereby reduced. The acceleration of the base-isolated structural model during testing was reduced by as much as 90% when compared to that induced in the model structure on a conventional rigid foundation. Such reductions in response were, however, accompanied by substantial lateral displacement. Furthermore, the extremely low transverse stiffness of the isolation system would, in a full-scale structure, lead to a low-frequency motion under wind loading that would be uncomfortable for occupants and could induce fatigue in utility connections. It was therefore necessary to develop a form of restraint against loading less severe than damaging earthquake ground motion.

Base isolation systems that incorporate wind restraints have been proposed before. Caspe [4] published a conceptual study of an earthquake isolation system in which ball bearings were proposed as the basic mechanism of isolation. Wind restraint in this system was to be provided by control rods that were to remain elastic under wind loading, but would yield under earthquake loading. An isolation system developed at the Centre National de la Recherche Scientifique de France and described by Delfosse in [5] incorporated wind restraints although the precise form was not specified. An isolation system suggested by Ikonomu [6] included wind restraint in the form of steel bars in tension which were intended to break at a specified force. The use of an energy-absorbing device as a mechanical fuse has been studied experimentally [3, 7] for the present natural rubber base isolation system. The elastic stiffness of the energy-absorbing device was very high relative to that of the bearings. The device was effectively rigid under small loads, but yielded under more intense seismic loading, providing a much reduced stiffness and damping in the isolation system. A detailed description of the response of the energy-absorbing devices when subjected to dynamic loading on a shaking table is given in [3].

An alternative method of restraining structures from movement under wind and other nondamaging loading is proposed here. A type of wind restraint has been developed and tested on the shaking table at the Earthquake Simulator Laboratory at the Earthquake Engineering Research Center, University of California, Berkeley. The wind restraint is a simple mechanical fuse system which prevents lateral movement under small horizontal loads, but which breaks at a specified force, allowing the structure to move freely on the rubber bearings. The mechanical fuse comprises a small steel pin with a notch machined around its circumference. The pin is loaded in single shear at the notch and when subjected to a dynamic load breaks suddenly at a reasonably well-defined shear load.

In this test series, pins with breaking forces ranging from 2-1/2% to 20% of the weight of the model structure in which they were incorporated were tested. Each of three simulated earthquakes -- the El Centro NS 1940, Pacoima Dam S16E 1971, and Parkfield N65E 1966 records -- were scaled to several peak accelerations and used as input to the shaking table. The results of these tests indicate that the shear pin mechanical fuse is a simple, effective, and practicable wind restraint. The influence of the sudden breaking of the pin on the response of the isolated structure was predictable within the limits possible for brittle fracture, which data is always subject to some scatter. A design technique for a full-scale system can be developed from the results of the tests reported herein.

The most interesting physical result of the tests was that the sudden breaking of the pin induced an acceleration that was predominately in the second mode of the isolated structure. The pin can be so designed that this acceleration will be no greater than the peak acceleration induced predominately in the first mode by earthquake ground motion, which acceleration typically occurs very late during an earthquake. A parallel analytical study of this phenomenon was carried out and the implications of this study on design are described in a later section of this report.

2. TEST DESCRIPTION

The tests were conducted at the Earthquake Simulator Laboratory of the Earthquake Engineering Research Center of the University of California, Berkeley. The experimental model was a three-story steel frame with natural rubber bearings under each of four columns. The frame was loaded by concrete weights. The total weight of the model was 39,500 lbs. and its height was twenty feet.

Several tests on pins with breaking forces in the range 4.5 to 8.2 kips were performed in May 1977. A further series was carried out in April 1978 on pins with lower breaking forces, in the range 1.0 to 4.5 kips. In the second series of tests, diagonal bracing was added to the frame in the first floor as shown in Fig. 1. The long arm connecting the pin mounting to the frame would not be present in a similar full-scale structure, but was used here to facilitate mounting of the shear pin on a load cell as shown in Fig. 2.

The test structure was extensively instrumented to measure displacement, acceleration, and shear force under each rubber bearing and in the shear pin. Data were taken from each measuring device at a rate of approximately 50 samples per second. The experimental model and isolation system were as described in [3] with the exception that shear pins rather than energy-absorbing devices were used.

2.1 Design and Development of the Shear Pin

A base isolation system must include a mechanism that will restrain a structure from swaying or undergoing large deflection under wind loading. This mechanism by which the base is kept rigid must act as a mechanical fuse that quickly deactivates when lateral shear force reaches a certain level, e.g. during a strong motion earthquake, thus allowing the base isolation system to protect the building from structural damage. A notched shear pin is ideal as mechanical fuse since it is rigid, yet rapidly deactivates, and is easily fabricated and adapted to a structure. In this section the design and development of the shear pin are described and results of static tests on the device are reported.

The base shear force that will develop from wind loading depends on building type and geometry, but is unlikely to exceed 5% of structural weight.

A base shear force induced by a strong motion earthquake of 10% or more of structural weight would result in severe structural damage unless the building were extremely ductile. Shear pins with breaking forces between 10% and 20% of structural weight were tested in order to study the effect of pin rupture on the response of the model structure. The shear pins were fabricated from cold-finished C1018 carbon steel rod since this material was not only suitable, but could be obtained readily and in several sizes.

The shear pins were tested in single shear in a standard shear box tester and a Baldwin compression testing machine. From a preliminary analysis, four sizes of steel rod were selected from which shear pins were fabricated: 1/4", 5/16", 3/8", and 1/2" in diameter. Notches of various depths were machined around the circumference of the shear pins where shearing action would occur. Load versus ram displacement was plotted for each test. The peak load in each case was read from the load dial of the Baldwin testing machine and designated the breaking force of the shear pin. The static breaking forces are tabulated in Table 1.

In each instance, the pins sheared cleanly, indicating that the design was adequate. Although some plastic flow occurred in the shear area prior to failure, this was not considered a problem and was not observed during dynamic testing on the shaking table. The breaking force of the shear pins under static loading was consistent and predictable given pin diameter and notch depth. Breaking forces of the shear pins under dynamic loading were more widely scattered and consistently higher. Breaking forces of the pins under both static and dynamic loading are given in Table 1. The numbering system used in this table to distinguish pin type is as follows: the first number indicates pin diameter in 1/16" units and the second notch depth in thousandths of an inch, e.g., 4/07 designates a shear pin with a diameter of 1/4" and a notch of .070-inch depth.

2.2 Incorporation of Shear Pins in Isolation System

A 2" x 2" x 1/4" tubular steel member was joined by a 3/4" pin to the base floor transverse girder of the steel frame for the shear pin tests on the shaking table. The other end of this steel member was fastened by a notched shear pin to the top of a load cell, which was in turn fixed to the shaking table. This connection is illustrated in Fig. 2. The connection was designed and the pin so positioned that all force was transferred at the notched cross section of the shear pin. Nearly all the shear force at the base level of the frame was transferred to the weakened section of the shear pin since the horizontal stiffness of the pin assembly was approximately twenty-five times greater than that of the four rubber bearings combined. The base of the frame remained fixed to the shaking table until the base shear was sufficiently great to cause the pin to break. The frame was then attached to the shaking table only by the rubber bearings which functioned as an isolation system.

2.3 Earthquake Input

The earthquake signals used in this series of tests were the El Centro N-S (1940), the Parkfield N65E (1966), and the Pacoima Dam S16E (1971). The span number associated with a given earthquake signal is a scaling factor. Shaking table displacement is linearly proportional to span number. A span of 1000 corresponds to a maximum table displacement of ± 5 inches. Time histories of shaking table displacement and acceleration for the three earthquake signals are shown in Fig. 3; the peak acceleration and displacement for the span levels used in the tests are given in Table 2.

3. TEST RESULTS

More than fifty tests were performed on the notched shear pins (Table 1) at various intensities of the three earthquake signals (Table 2). Additional tests were conducted on pins with small diameter holes rather than notches, but since the results were virtually identical to those for notched pins they are not described here. Results of tests on pins with very shallow notches, e.g. less than .040 inch deep, or no notch are also not reported here since the pins did not break cleanly in these tests, but bent and pulled out of the connectors. This last result indicates that the notch on a shear pin must be of at least a minimum depth.

Typical time histories of shear pin force to the breaking point are shown in Fig. 4a for the shear pin with the lowest value of breaking force and in Fig. 4b for a shear pin with an intermediate value of breaking force; both pins were subjected to the El Centro 300 input. Displacement and acceleration for all four floors of the model structure are shown in Figs. 5 through 8 for a single earthquake input, El Centro 300, with a peak table acceleration of 0.200g. A peak third floor acceleration of .093g was induced in the isolated system when no shear pin was used. The displacement patterns for pins with small, intermediate, and large breaking forces were nearly identical. After the pins broke, the response pattern was a rigid body motion with a frequency of approximately 0.55 Hz. A higher mode acceleration at 4.85 Hz was induced in the frame when pins with a higher breaking force were tested. While the third and second floors moved in one direction, the first and base floors moved in the opposite direction, a pattern of motion and frequency characteristic of second mode response on the rubber bearings alone. The rigid body motion clearly visible in the displacement traces is the first mode response of the isolated system. Second mode response was negligible in tests where shear pins were not incorporated into the system, but significantly affected acceleration of the frame when shear pins were used.

Peak third floor acceleration increased linearly with breaking force as shown in Fig. 9 where peak table acceleration and acceleration induced when no shear pin was used are also plotted. When the results are extrapolated to zero pin force, the resulting value of peak acceleration does not correlate with that observed during tests of the system with no shear pin. While peak acceleration induced in the system with the shear pin incorporated occurs when the pin breaks, i.e. early in the input motion, it occurs late during the ground motion when no pin is present. The two peaks are thus well separated in time. Time history plots for the Pacoima Dam 220 and Parkfield 300 earthquake signals (Figs. 10 through 15) indicate that the response described above for the El Centro 300 signal is characteristic.

4. INTERPRETATION OF RESULTS

An analytical model of the system was developed to facilitate interpretation of the experimental results described above in a form useful for design purposes. The structure was treated as two linear elastic systems, one that characterized the response of the system before and the other after the shear pin broke. The system could be so characterized because stresses in frame members, except in the shear pin, never approach the yield level. The frame model was idealized as a plane frame. A stiffness matrix was formed for the system idealized by twenty-three degrees of freedom. Since the mass of the structure was concentrated at the floor levels, the horizontal displacement at each of the four floors was designated a dynamic degree of freedom and all others were eliminated by static condensation.

The coupling of the shear pin to the model structure was much more flexible than it would be in a similar full-scale structure since the pin was coupled to the frame by a number of flexible elements, such as the connecting arm, necessary to incorporate the load cell into the system. The stiffness between the base floor and the table was estimated from a number of curves that related load cell output to base floor displacement for several of the pins tested. A nominal value of 30 kips per inch was used in these calculations. A first mode natural frequency of 2.31 Hz and a mode shape of (1.0, 0.79, 0.58, 0.48) from the top of the structure down were predicted. After the shear pin broke, the horizontal stiffness between the base floor and the table was that of the bearings alone, 1.2 kips per inch. First and second mode frequencies of .55 Hz and 4.85 Hz, respectively, were predicted for this modified configuration, corresponding to mode shapes of (1.0, .99, .98, .97) and (1.0, .20, -.51, -.62), respectively.

In order to illustrate the effect of the pin breaking on acceleration in the second mode, the time history of acceleration of each of the four floors was reduced to modal coordinates based on the modes of the isolated, pin-free system and the maximum modal acceleration was determined. The contribution from each modal maximum to third floor acceleration for various pins and earthquake intensities is plotted in Fig. 16 for the first mode and in Fig. 17 for the second mode. Third and fourth mode contributions were negligible. The second mode contribution was linear with breaking force to a level of pin force that initially depended on earthquake intensity and then increased slowly with pin-breaking force. The pin-independent maxima occurred late during the earthquake ground motion inputs and the pin-dependent maxima close to the time when the pins broke. These two peaks were well separated in time and should not be considered additive since they represent different maxima.

A design technique for a wind restraint system based on the shear pin described herein can be developed from the above experimental and analytical results. Peak acceleration due to pin breakage can be estimated if the force at which a pin will break is specified and the shape of the structure is evaluated, where the shape is assumed to be the first mode of the pin-restrained system with an amplitude sufficiently large to induce a base shear equal to the pin-breaking force. This displacement would then be used to establish an initial condition for the second mode of a pin-free system, from which peak modal acceleration could be easily calculated if the system were assumed to be in free vibration. This latter assumption is justified since the contribution of earthquake ground motion to second mode response of an isolated structure is negligible. For the model structure studied here, results calculated as described above were plotted as lines and correlated with experimental results in Figs. 17a through 17c. A pin-breaking force that will produce accelerations in the second mode not greater than the acceleration produced by an earthquake in the first mode can thus be specified. In most cases this pin force will be greater than that necessary to restrain structures from the undesirable effects of wind loading.

5. CONCLUSIONS

The notched shear pin functioned effectively as a mechanical fuse during tests of the rubber bearings base isolation system. When subjected to dynamic loading, the pins broke completely and rapidly, and underwent little plastic deformation. The dynamic breaking force of the shear pins can be predicted reliably from the results of static tests. The mode shapes of the model with and without the shear pin in place differed greatly. When the shear pins broke, frame displacement was approximately in the first mode. The isolated system then responded in both the first and second modes. The magnitude of second mode response was approximately proportional to shear pin breaking force. In tests of shear pins with the highest breaking force, second mode acceleration in excess of 0.5g was recorded. At the 4.85 Hz second mode frequency, this acceleration induced a story drift on the order of .1 inch, negligible in comparison with earthquake-induced displacements which may be on the order of several inches.

Although accelerations measured during tests of the isolation system with shear pins with large breaking forces in place exceeded values acceptable in an earthquake-isolated structure, these tests were performed only to clarify the structural behavior of the model at the instant that the pin breaks, the breaking forces of these pins being much in excess of that necessary to restrain a structure under wind loading. The experimental results confirm that the acceleration induced when the shear pin breaks does not increase with an increase in earthquake intensity. Second mode acceleration will therefore be less significant for more intense earthquakes and smaller pin loads. The shear pins can be used to limit deflection due to wind loads and can be designed so as not to induce excessive acceleration when they break.

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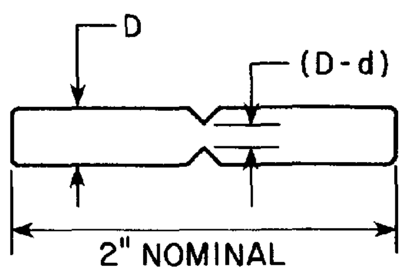
TABLE 1 SHEAR PIN DIMENSIONS AND BREAKING FORCES

SHEAR PIN DESIGNATION	DIAMETER (D)(INCHES)	NOTCH DEPTH (d)(INCHES)	NOMINAL STATIC BREAKING FORCE (KIPS)	AVERAGE DYNAMIC BREAKING FORCE (KIPS)
4/07	1/4	.070	.93	1.48
5/10	5/16	.100	1.58	2.25
5/055	5/16	.055	2.70	3.40
6/09	3/8	.090	3.16	4.09
6/06	3/8	.060	4.30	5.35
8/12	1/2	.120	5.76	6.77
8/08	1/2	.080	7.22	8.22



SHEAR PIN DIMENSIONS

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TABLE 2 MAXIMUM VALUES OF EARTHQUAKE INPUT SIGNALS

EARTHQUAKE SIGNAL	TABLE DISPLACEMENT		TABLE ACCELERATION	
	MAXIMUM (INCHES)	MINIMUM (INCHES)	MAXIMUM (G)	MINIMUM (G)
EL CENTRO 200	1.059	.828	.114	.129
EL CENTRO 300	1.587	1.241	.176	.200
EL CENTRO 400	2.118	1.663	.240	.280
PACOIMA DAM 100	.534	.454	.115	.127
PACOIMA DAM 150	.803	.684	.181	.192
PACOIMA DAM 220	1.180	1.000	.269	.272
PARKFIELD 300	1.476	.826	.149	.133
PARKFIELD 400	1.971	1.102	.191	.184

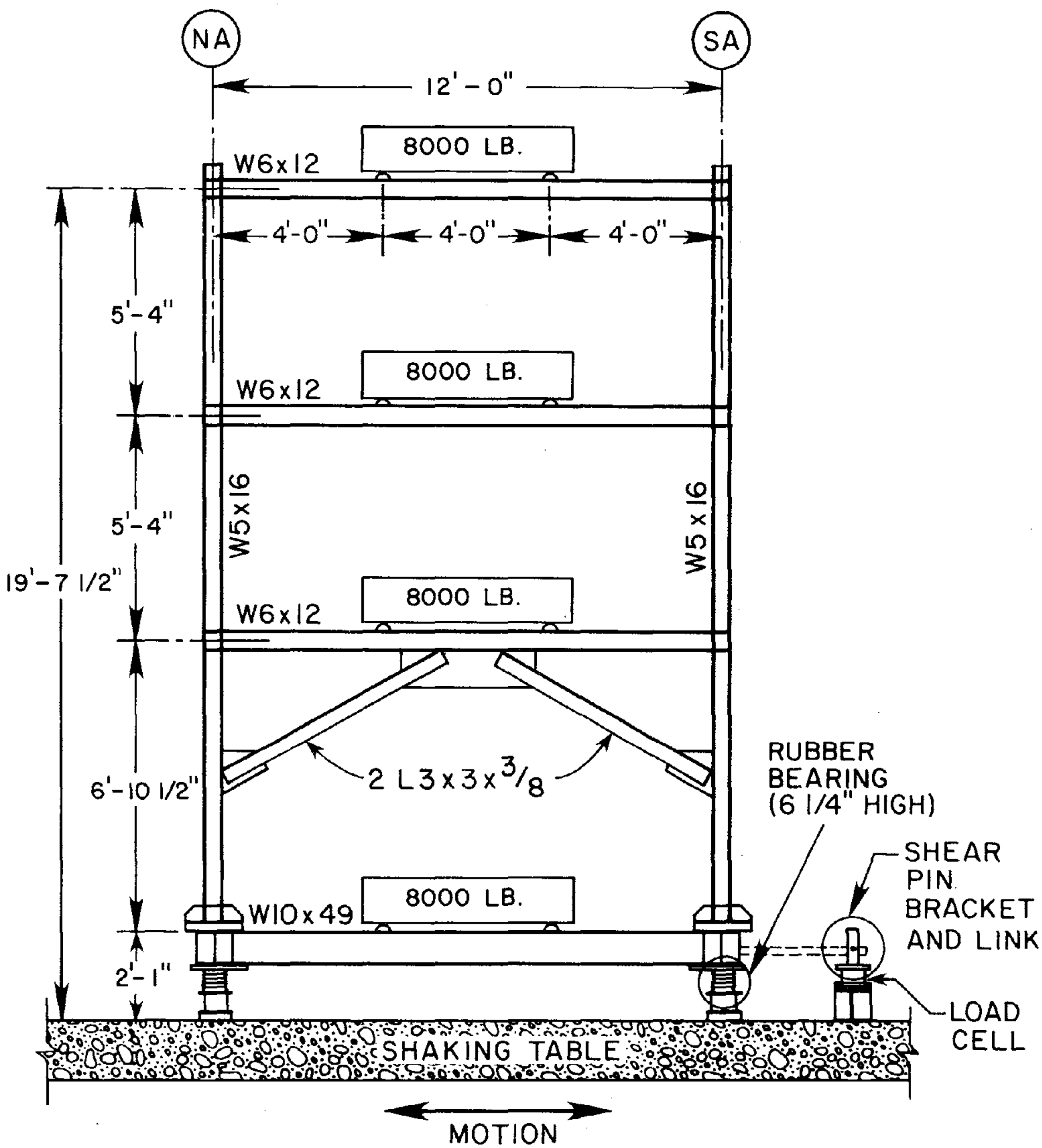


FIGURE 1 DIMENSIONS OF TEST MODEL

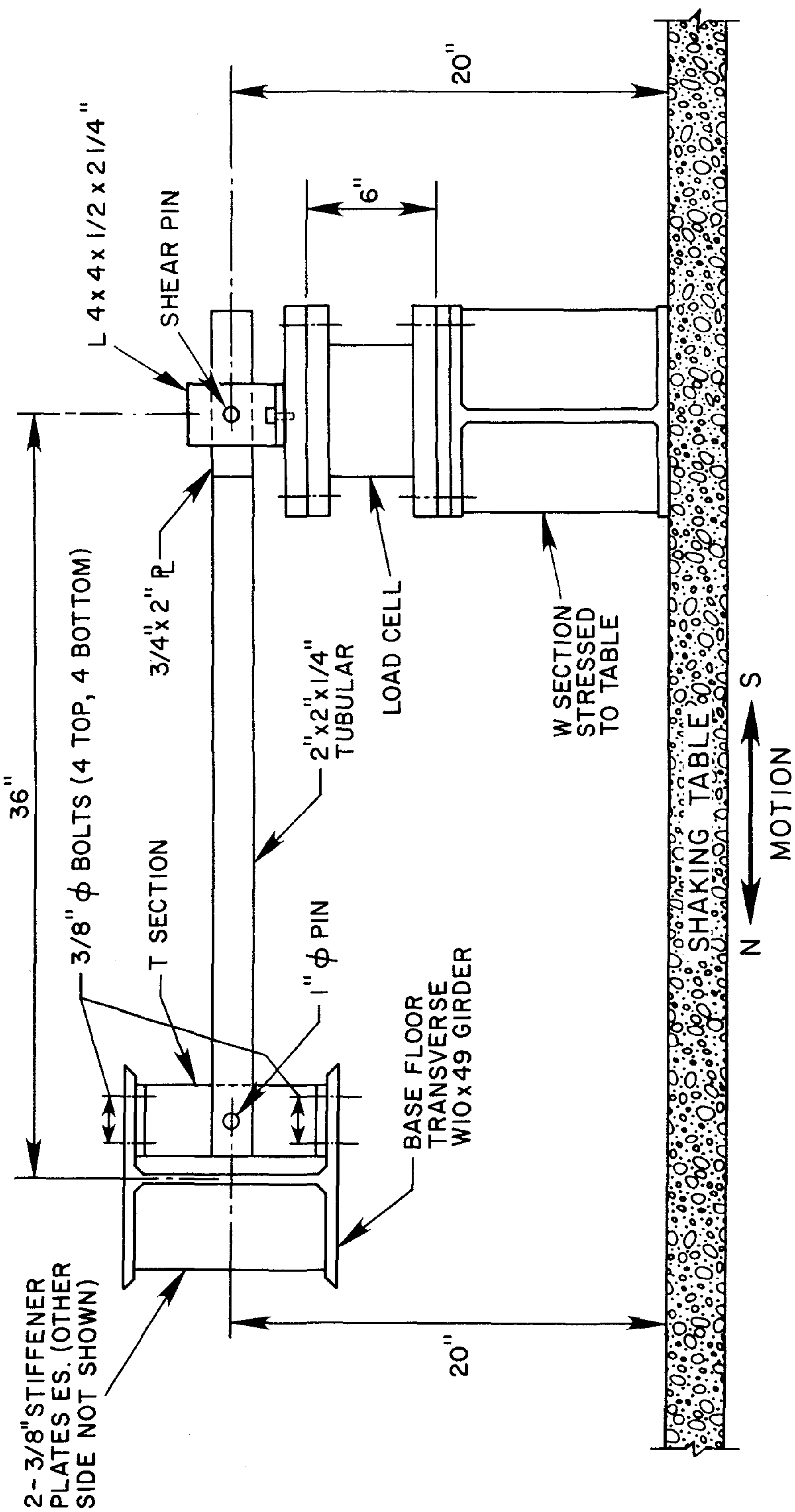


FIGURE 2 DETAILS OF SHEAR PIN CONNECTION

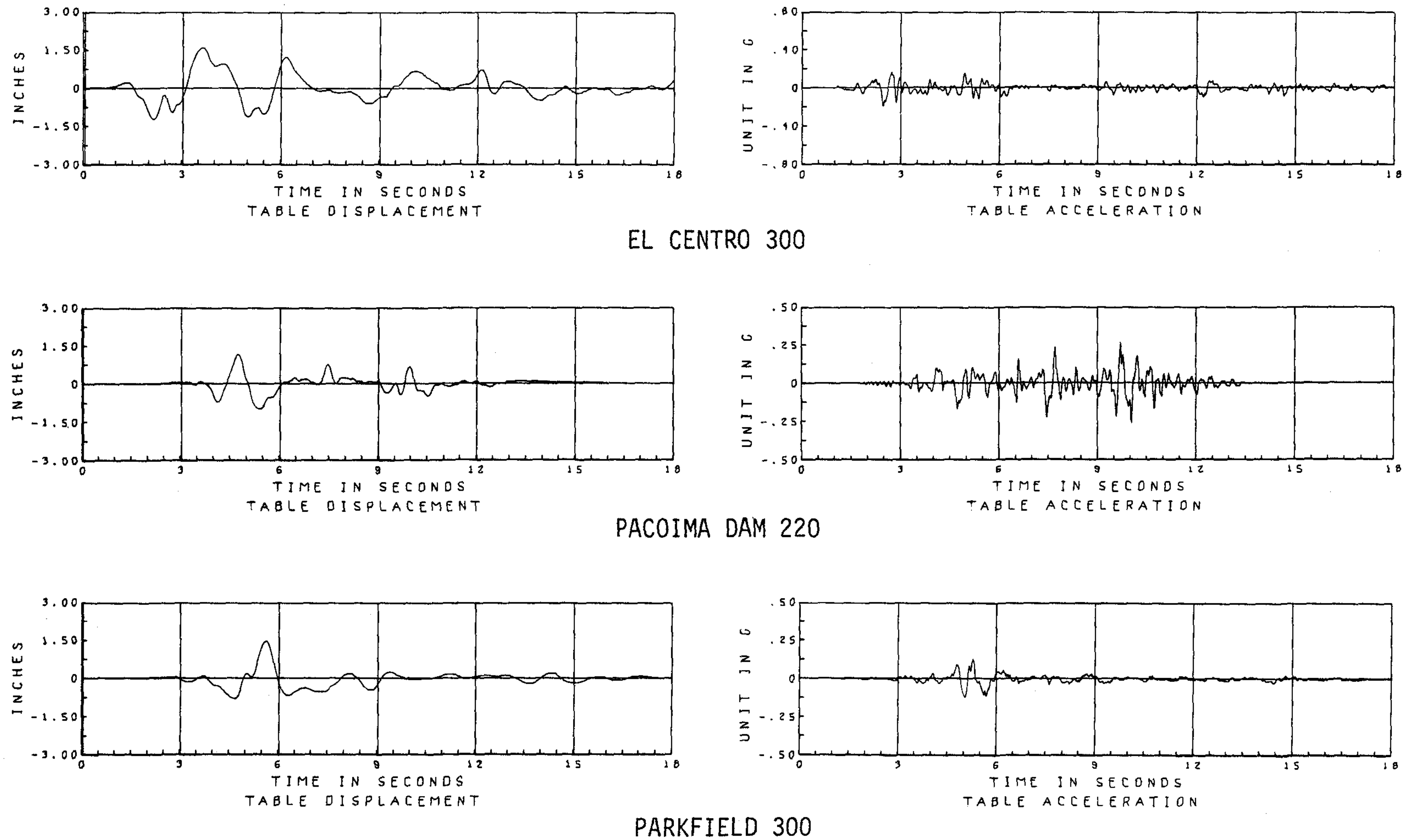
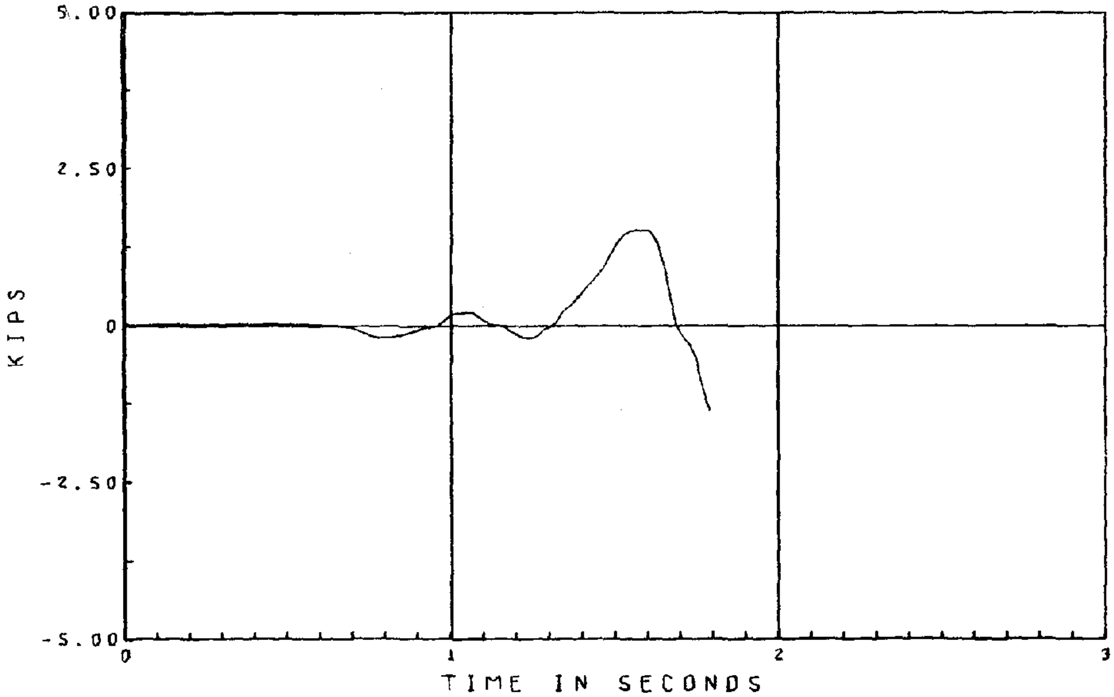
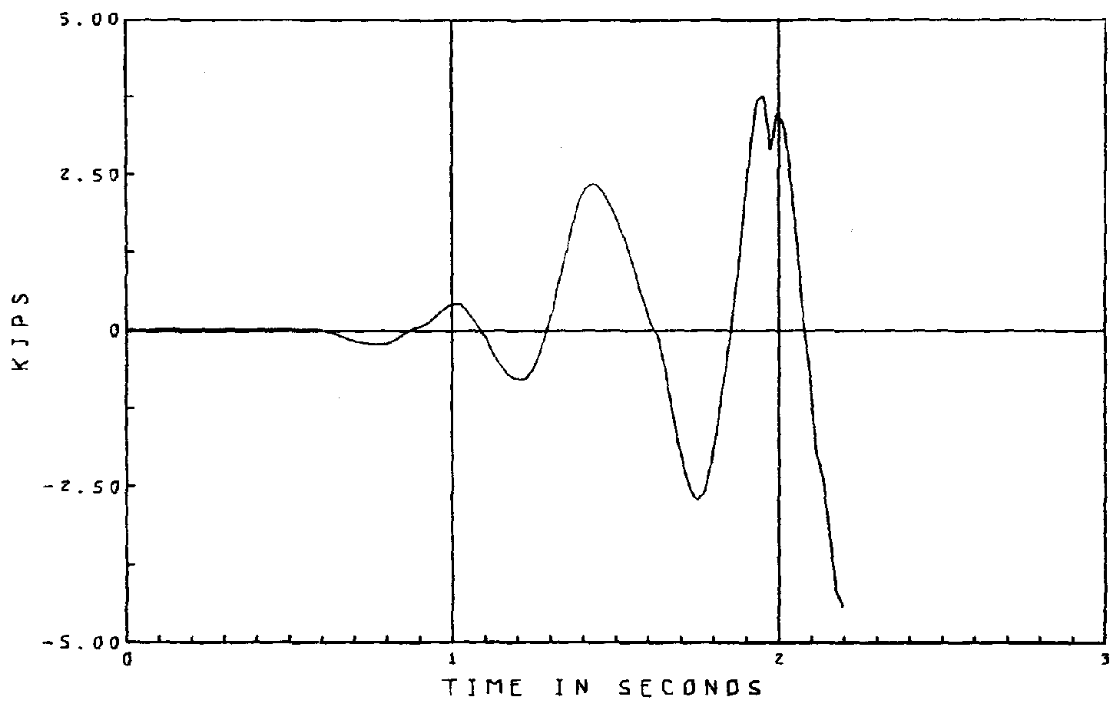


FIGURE 3 TABLE DISPLACEMENT & ACCELERATION: EL CENTRO 300, PACOIMA DAM 220, PARKFIELD 300



SHEAR PIN FORCE BREAK--1.364 KIPS



SHEAR PIN FORCE BREAK--4.429 KIPS

FIGURE 4 SHEAR PIN FORCE, EL CENTRO 300

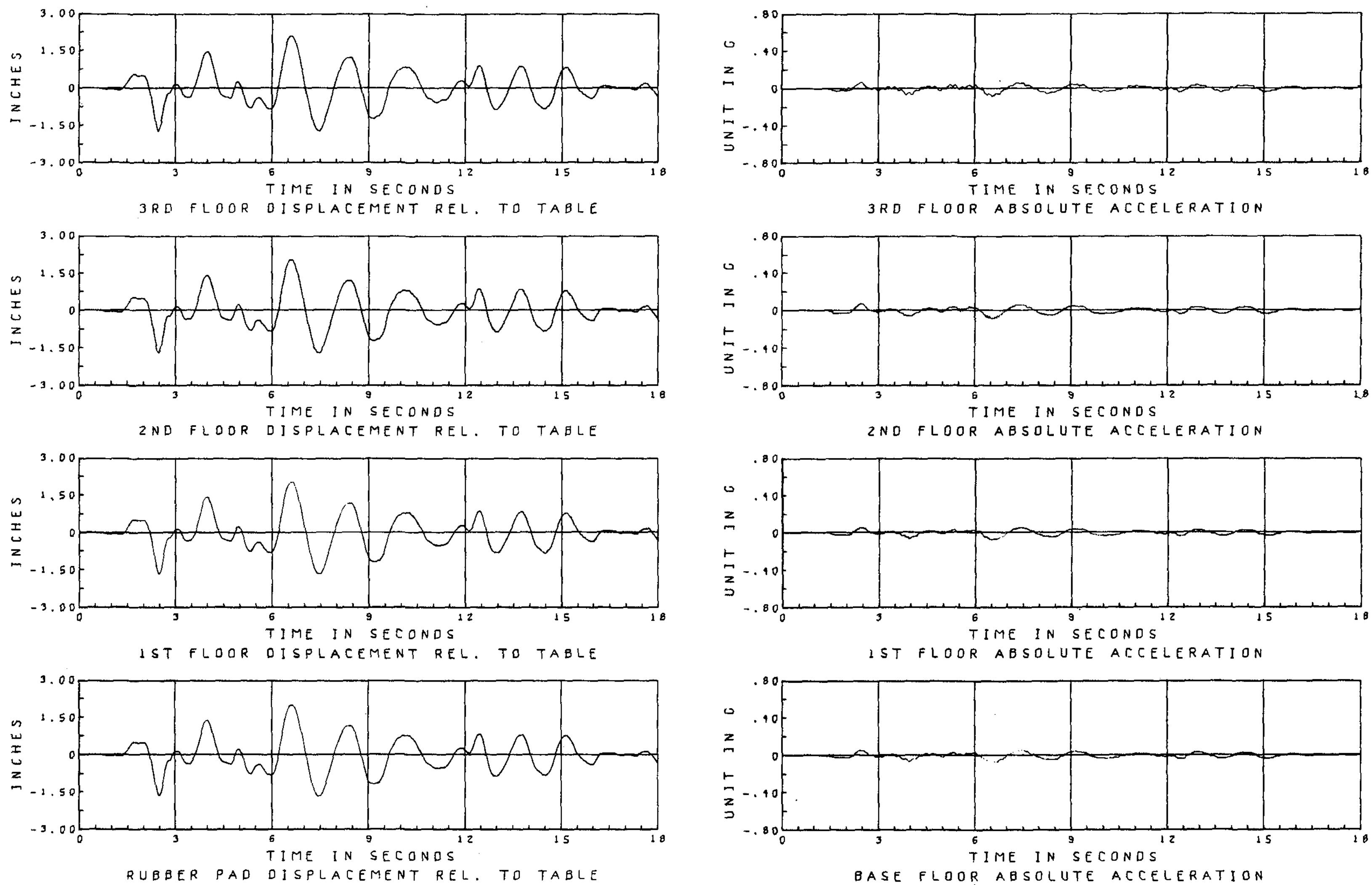


FIGURE 5 EL CENTRO 300, NO PIN

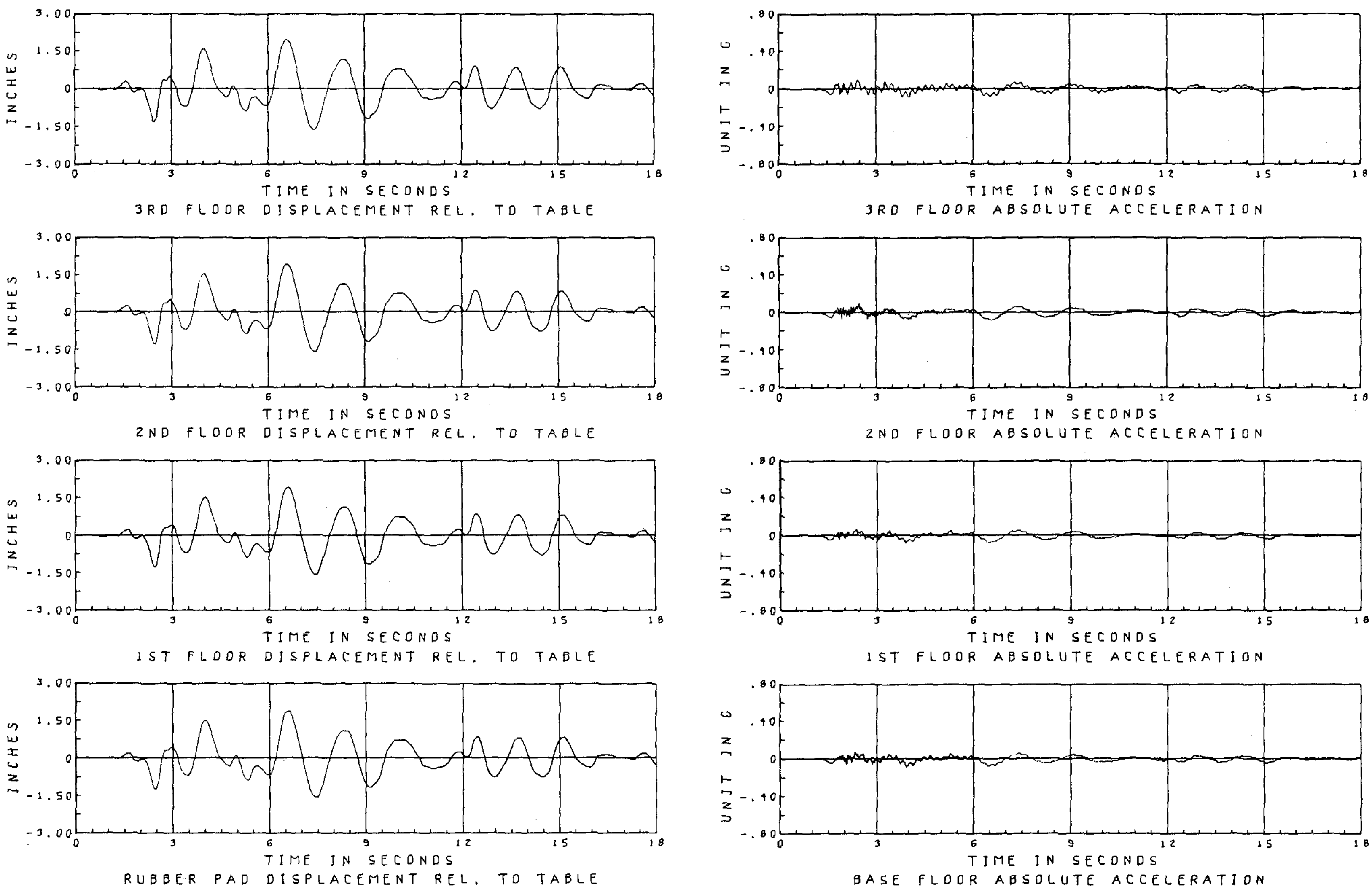


FIGURE 6 EL CENTRO 300, PIN 4/07, BREAKING FORCE OF 1.36 kips

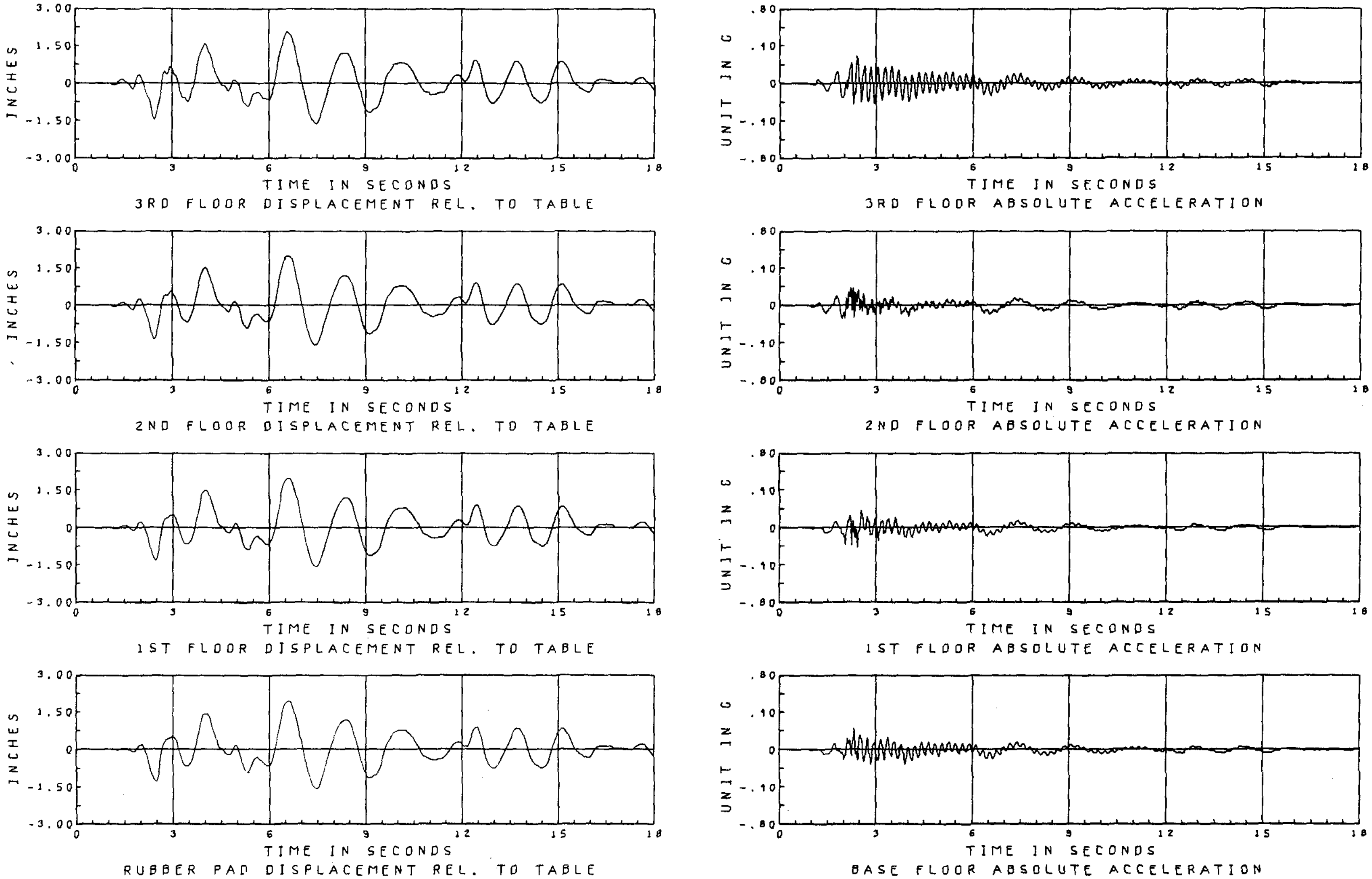


FIGURE 7 EL CENTRO 300, PIN 6/09, BREAKING FORCE OF 4.43 kips

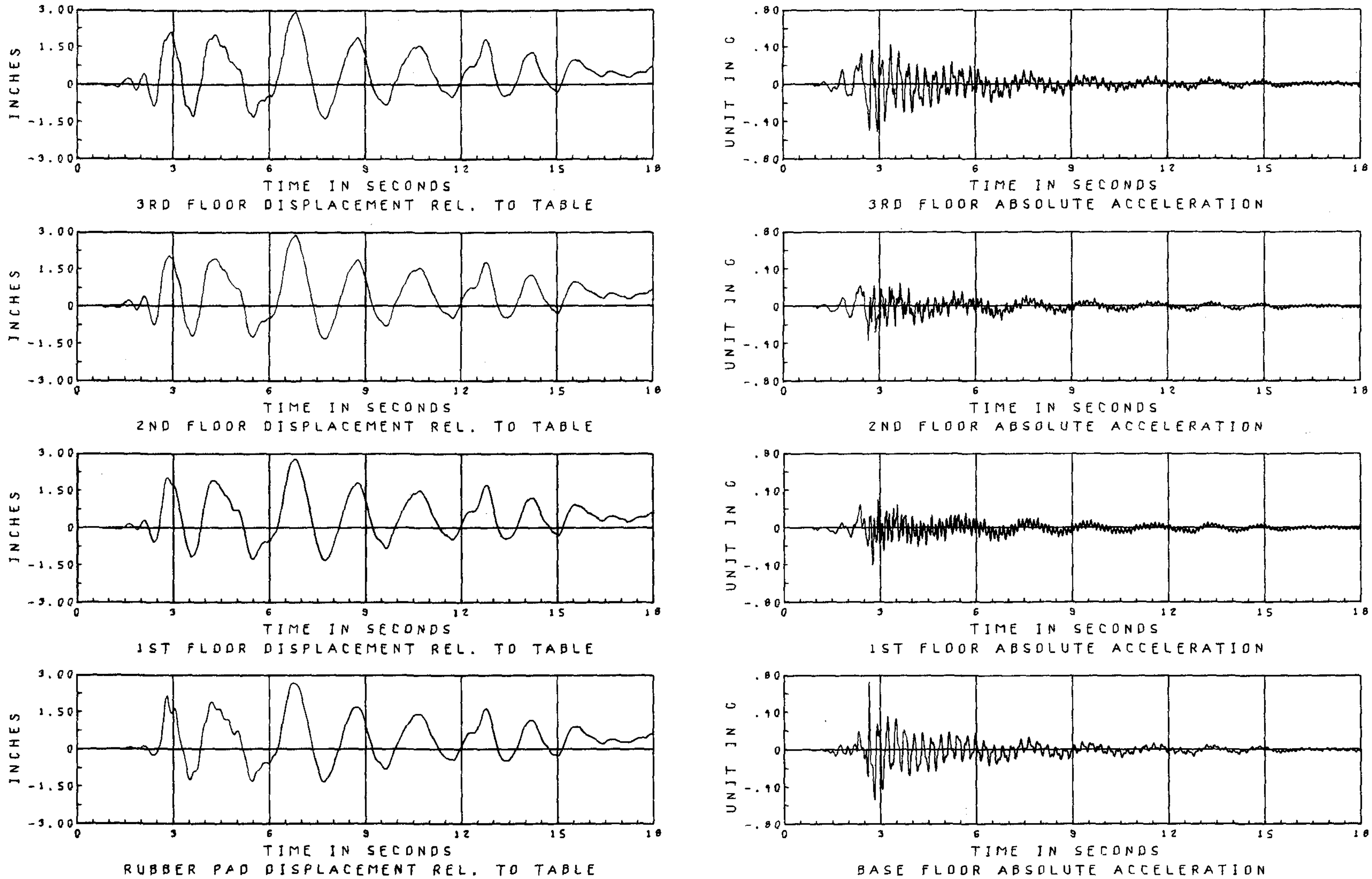


FIGURE 8 EL CENTRO 300, PIN 8/08, BREAKING FORCE OF 8.22 kips

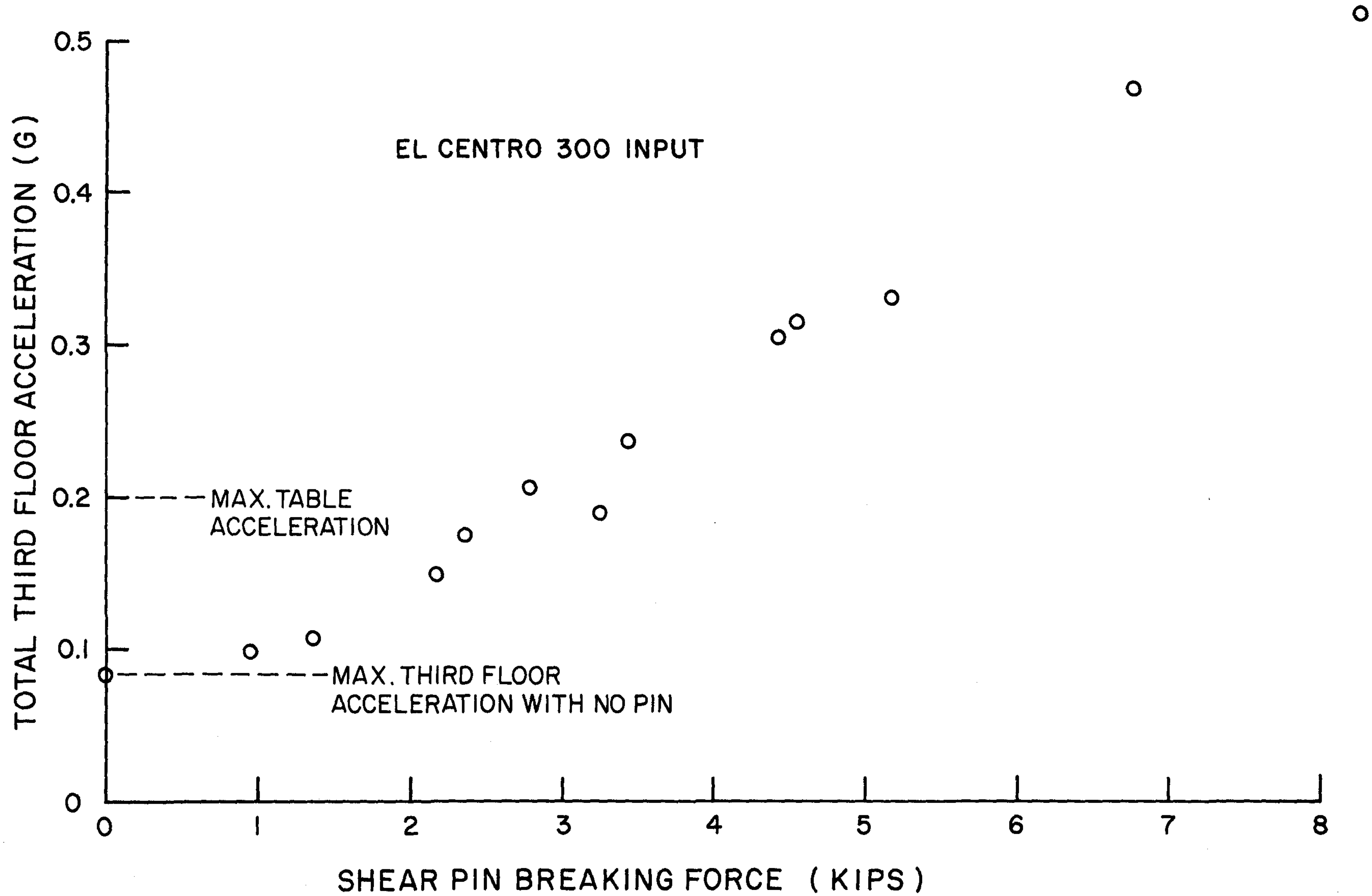


FIGURE 9 THIRD FLOOR PEAK ACCELERATION VERSUS SHEAR PIN FORCE

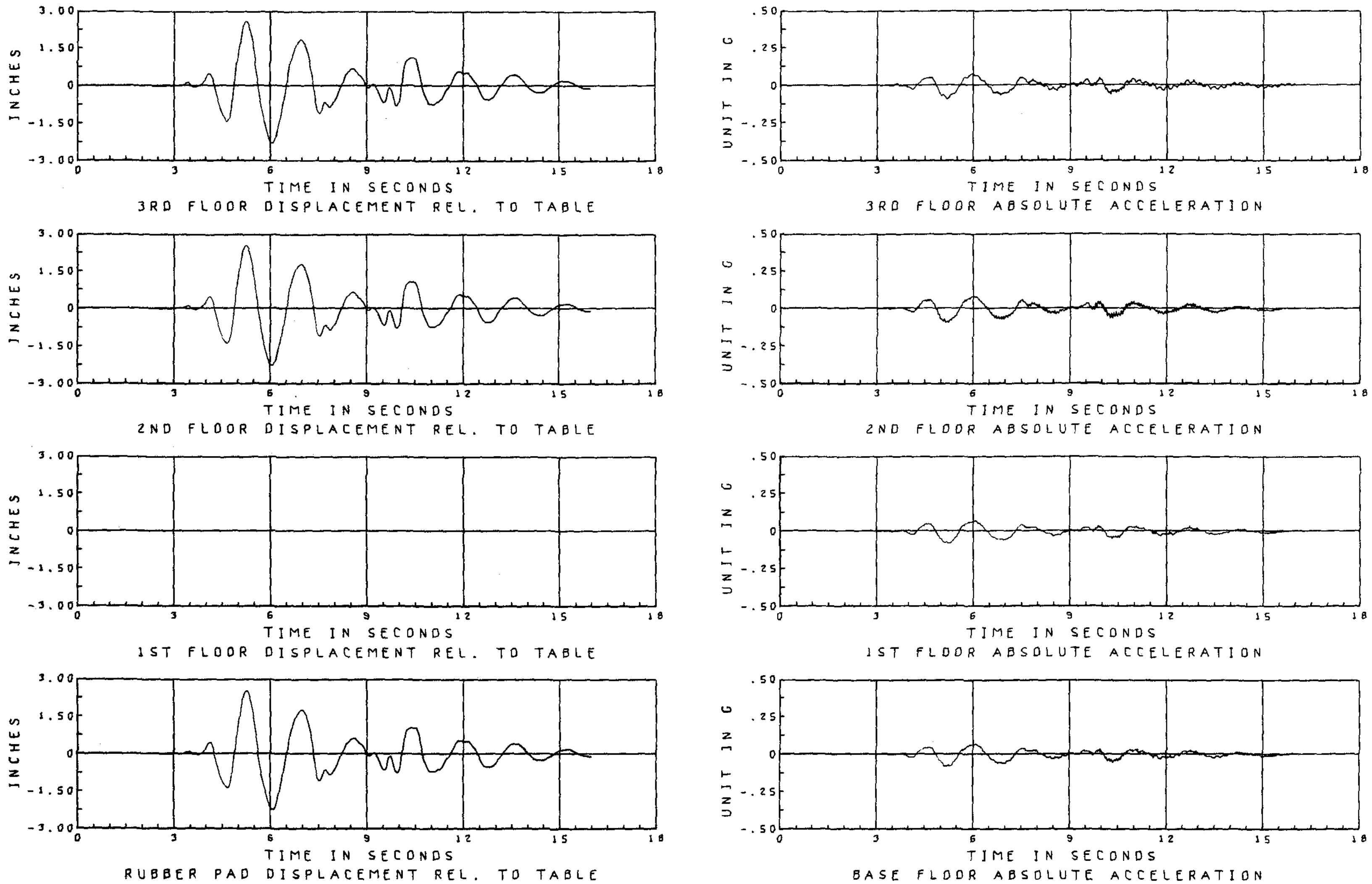


FIGURE 10 PACOIMA DAM 220, NO PIN

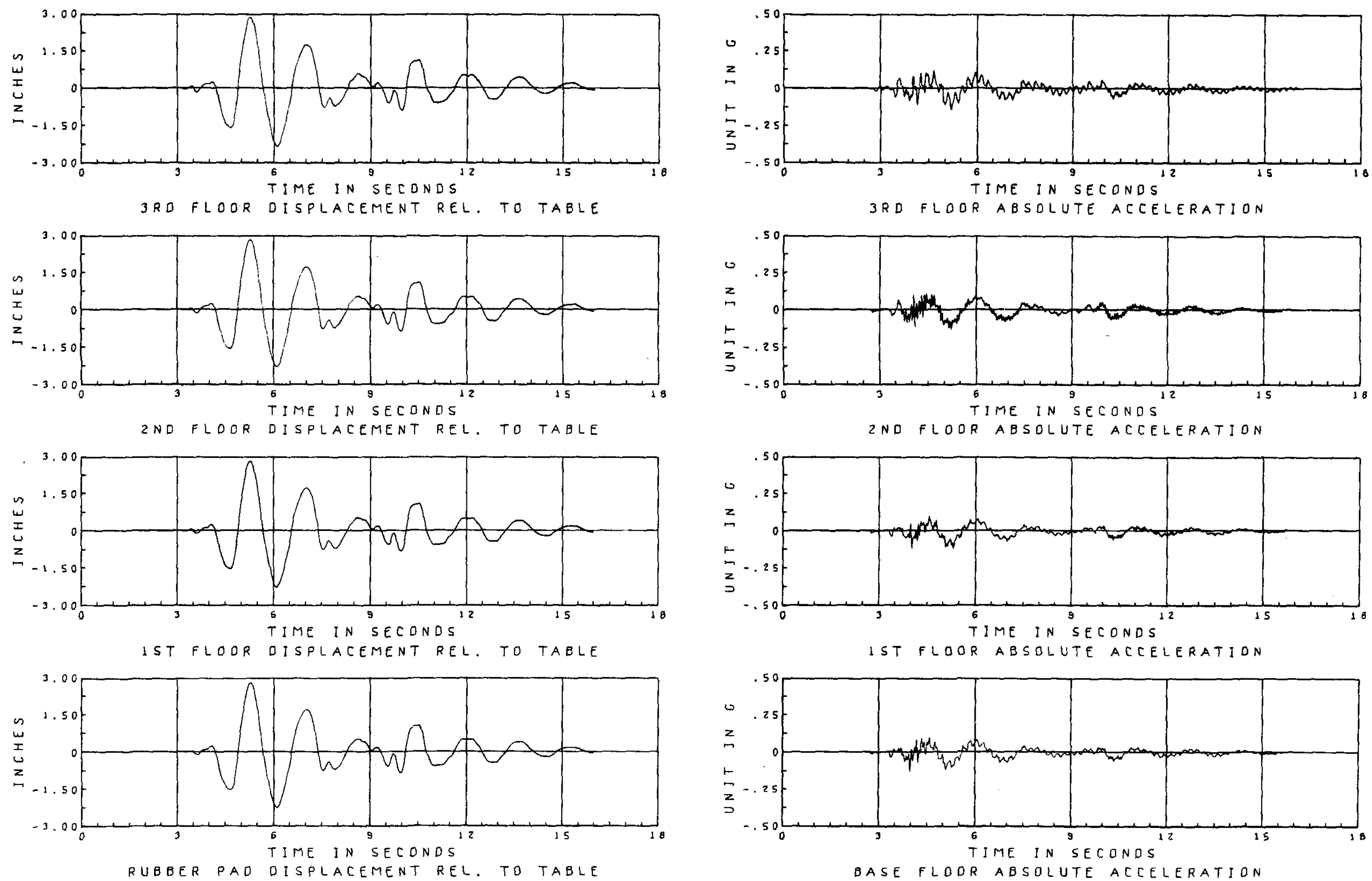


FIGURE 11 PACOIMA DAM 220, PIN 4/07, BREAKING FORCE OF 1.51 kips

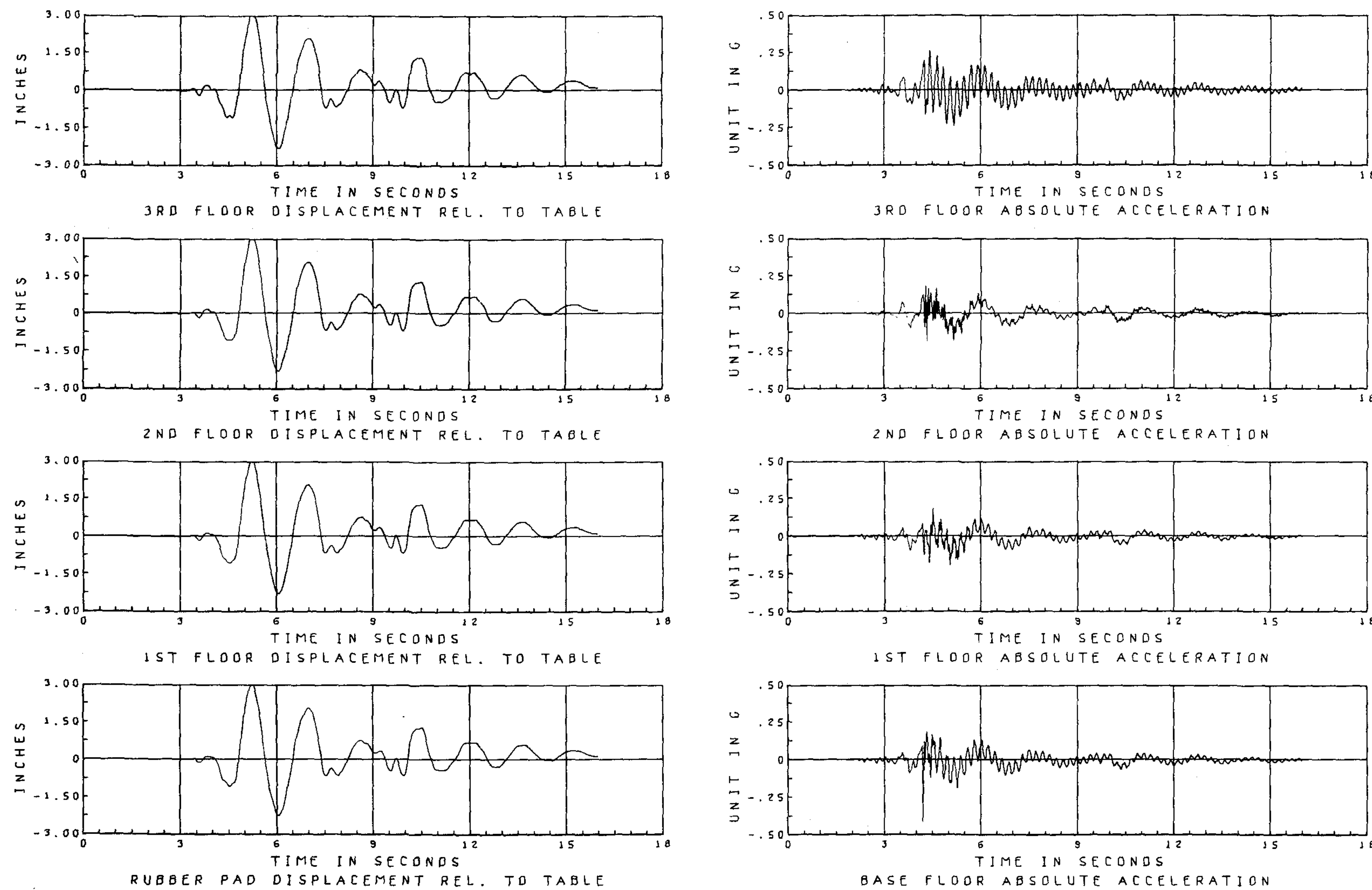


FIGURE 12 PACOIMA DAM 220, PIN 6/09, BREAKING FORCE OF 3.86 kips

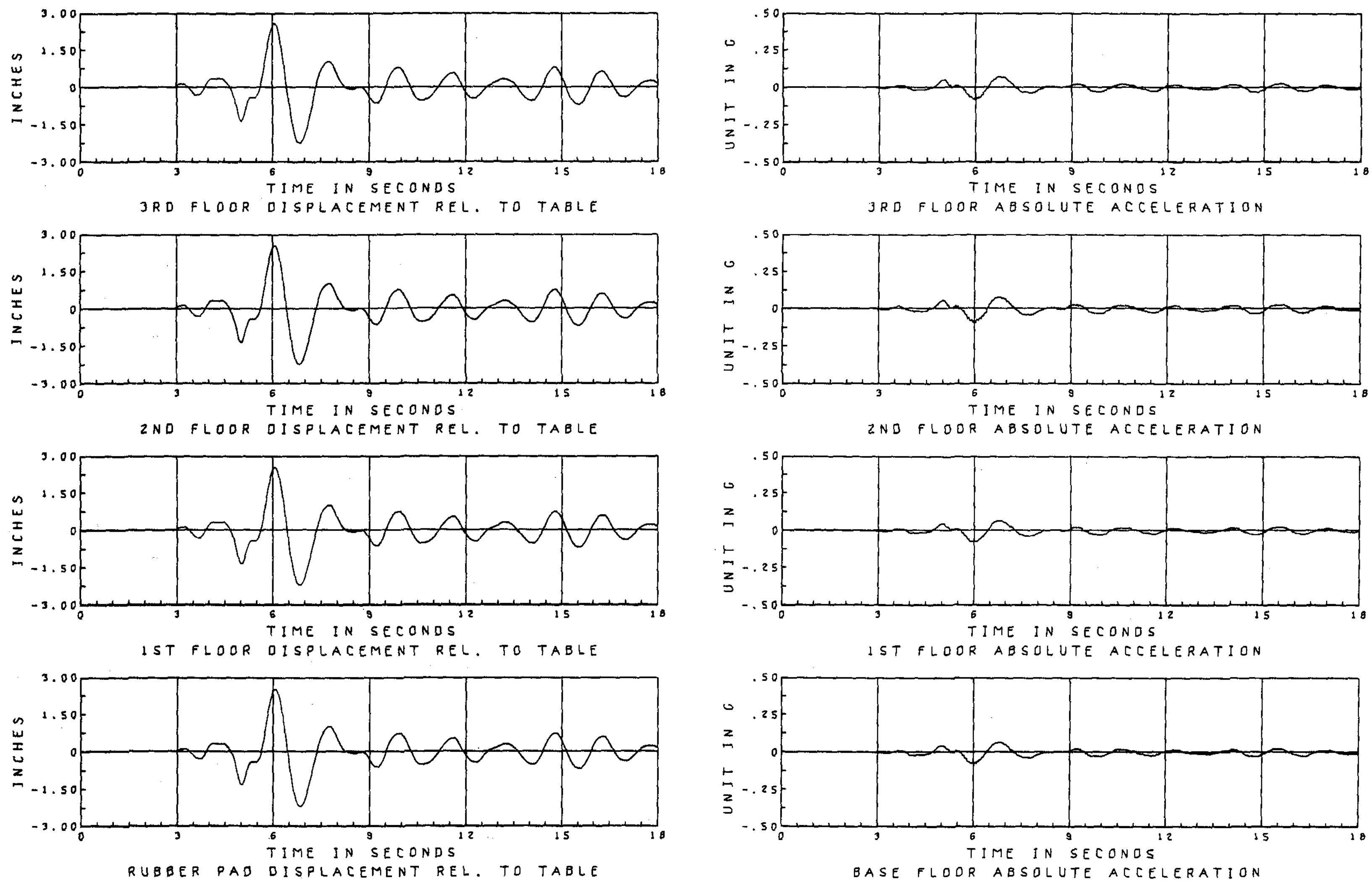


FIGURE 13 PARKFIELD 300, NO PIN

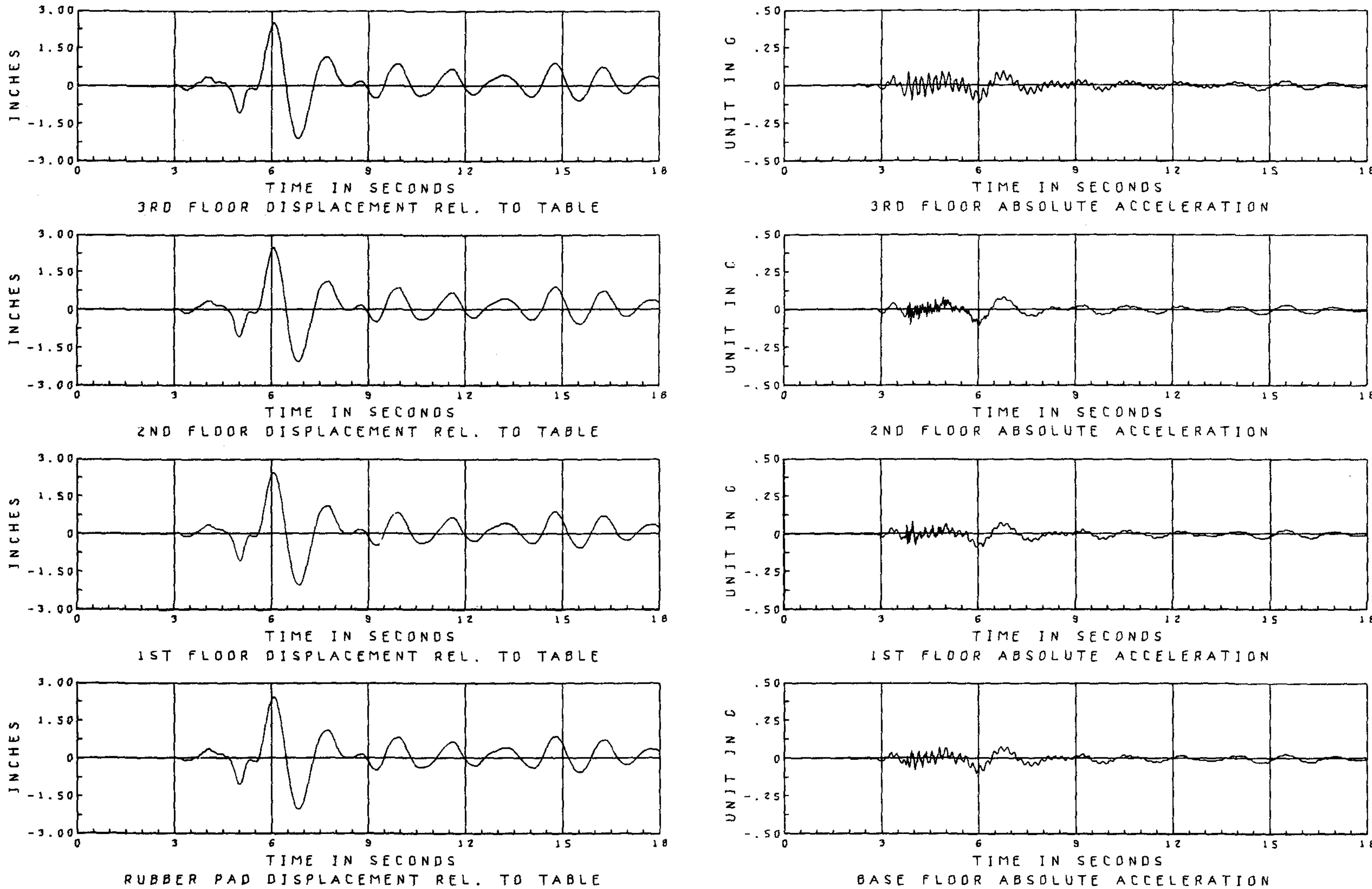


FIGURE 14 PARKFIELD 300, PIN 4/07, BREAKING FORCE OF 1.49 kips

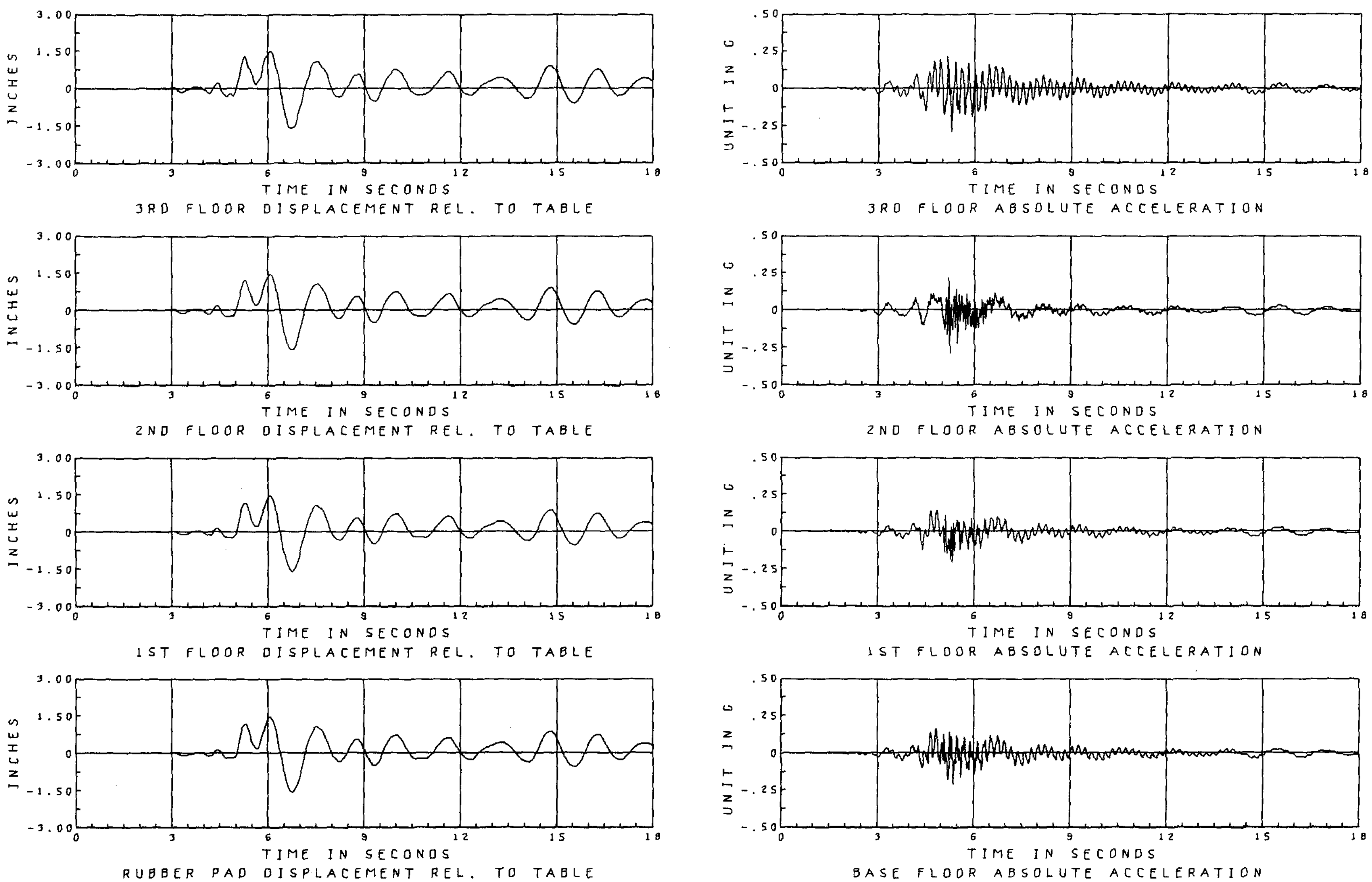


FIGURE 15 PARKFIELD 300, PIN 6/09, BREAKING FORCE OF 3.57 kips

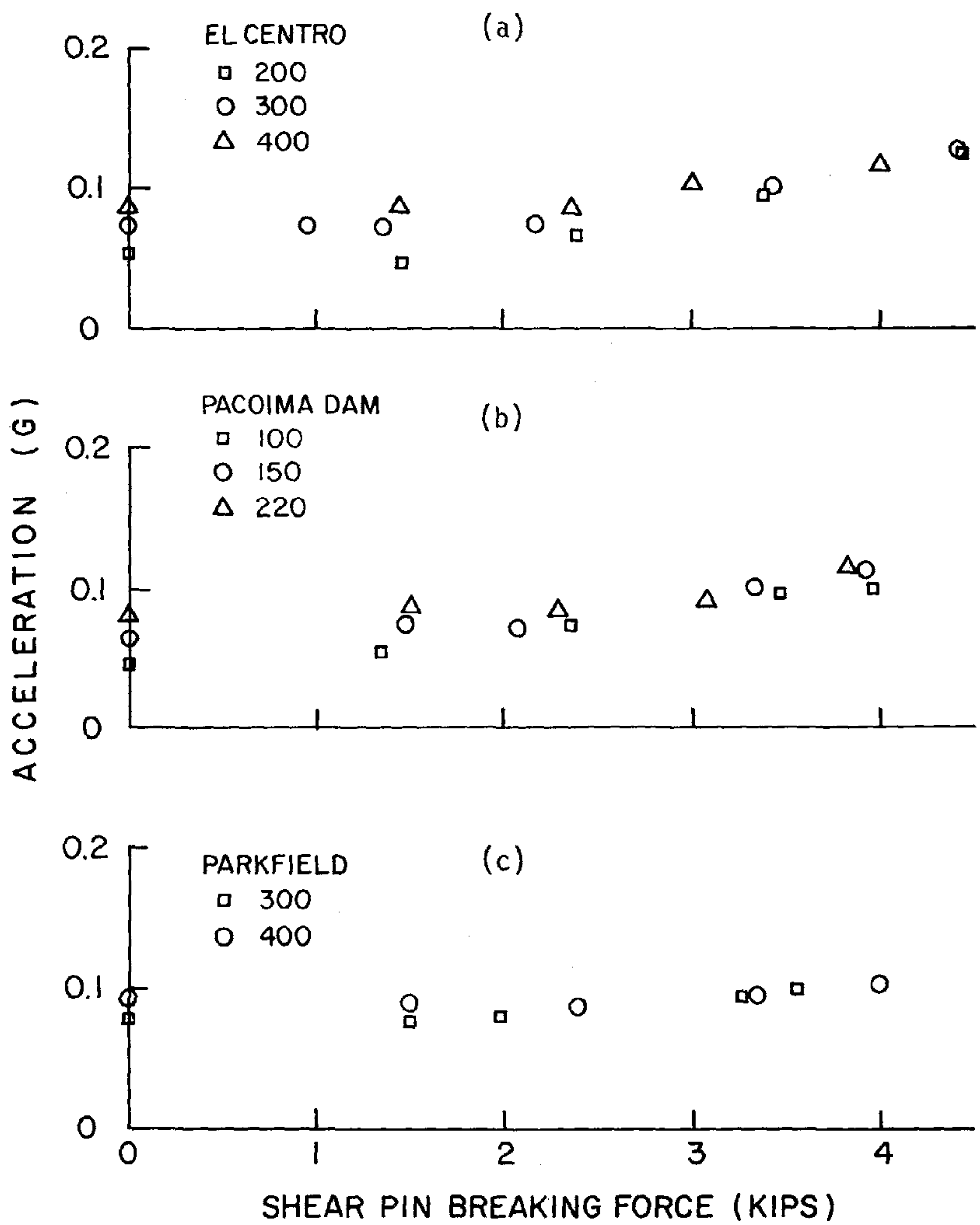


FIGURE 16 FIRST MODE ACCELERATION IN THE THIRD FLOOR VERSUS SHEAR PIN FORCE

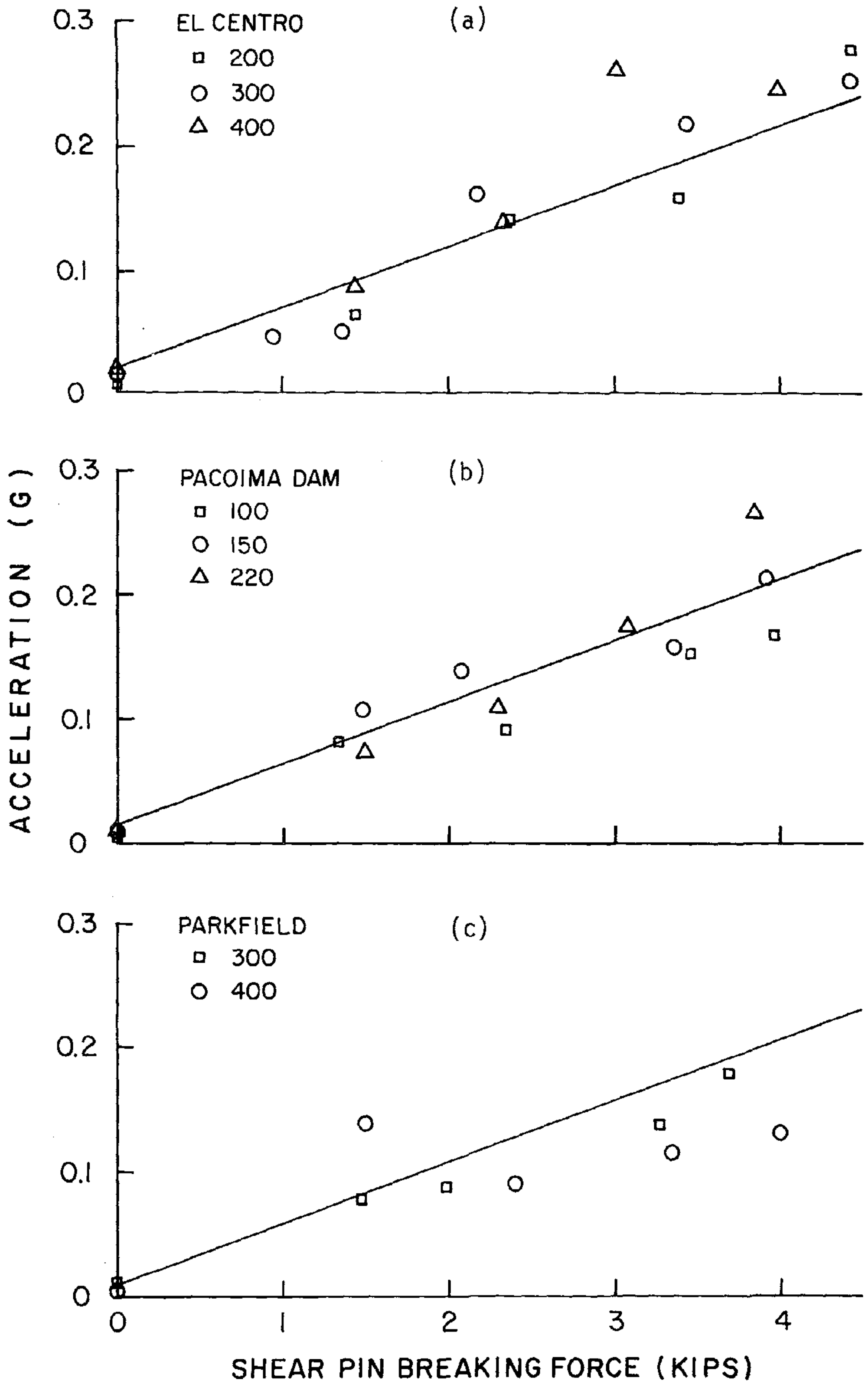


FIGURE 17 SECOND MODE ACCELERATION IN THE THIRD FLOOR VERSUS SHEAR PIN FORCE

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