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LINEAR MODELS TO PREDICT THE NONLINEAR SEISMIC BEHAVIOR OF A ONE-STORY STEEL FRAME

by

HELGI VALDIMARSSON
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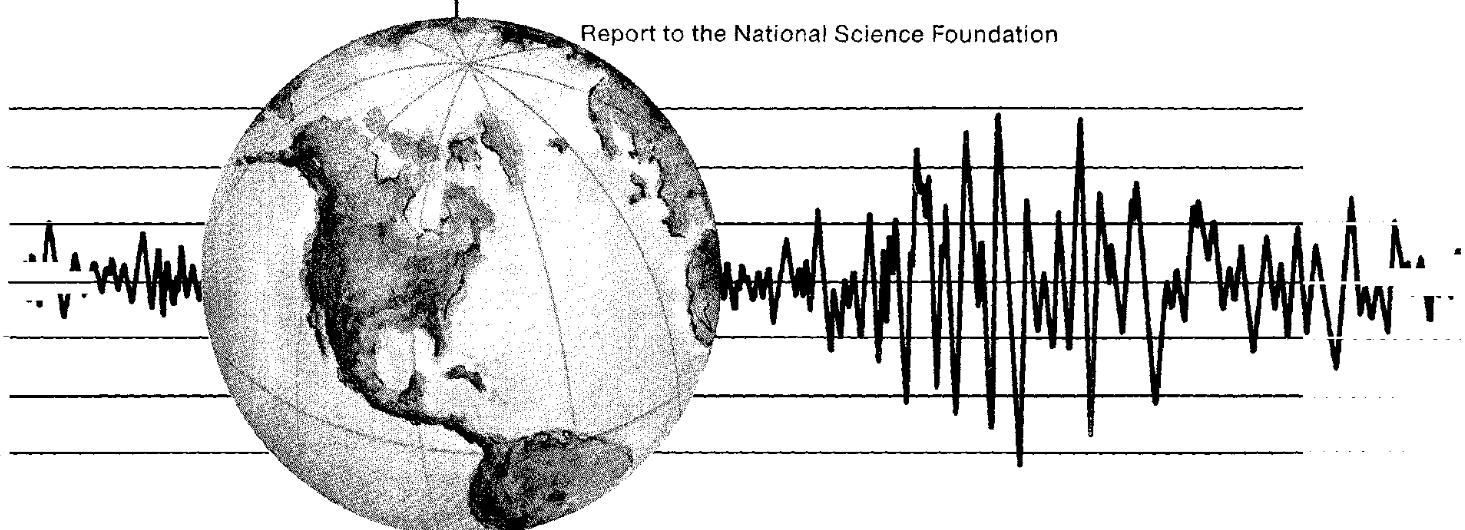
Report to the National Science Foundation



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In this report, six methods of linearization are used to construct various equivalent linear models to predict the nonlinear seismic behavior of a one-story steel frame which was constructed and tested by Sveinsson and McNiven at the Earthquake Engineering Research Center of the University of California, Berkeley.

Four of the methods of linearization depend on the restoring force-displacement relation of the frame. Two bilinear models are constructed; one to represent the elastic-plastic nature of the structural steel, the other to represent the work hardening nature. Both bilinear models reproduce the response time histories quite accurately in the domain appropriate to each.

The models are assessed by comparing their response predictions with the measured response for the El Centro, Pacoima, Taft and Parkfield excitations.

The study indicates that the dependence of the nonlinear response of a structure on the characteristics of the earthquake excitation is so complex that there is no way that the linearization schemes considered can have the required generality to limit the maximum displacement response to a specified value. Nonetheless, these methods can provide very valuable guidelines for design, if their limitations and relationship to the overall design process is fully recognized.

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ABSTRACT

In this report, six methods of linearization are used to construct various equivalent linear models to predict the nonlinear seismic behavior of a one-story steel frame which was constructed and tested by Sveinsson and McNiven at the Earthquake Engineering Research Center of the University of California, Berkeley.

Four of the methods of linearization depend on the restoring force-displacement relation of the frame. Since explicit expressions for the linear model parameters, based on a bilinear hysteretic model, are readily available in the literature and it is evident from the test results that the hysteretic behavior of the frame can be approximated by such a model, two bilinear models are constructed; one to represent the elastic-plastic nature of the structural steel, the other to represent the work hardening nature. Both bilinear models reproduce the response time histories quite accurately in the domain appropriate to each.

The construction of all the equivalent linear models is based on the measured nonlinear response of the frame to El Centro excitation, and the objective for their construction is the ability to predict the maximum response values, with precedence being given to the maximum displacement response. The assessment of the models is made by comparing their response predictions with the measured response for El Centro and also three other excitations, i.e., Pacoima, Taft and Parkfield excitations.

The results of this study indicate that the dependence of the nonlinear response of a structure on the characteristics of the earthquake excitation is so complex that there is no way that the

linearization schemes considered can have the required generality to limit the maximum displacement response to specified value. Nonetheless, these methods can provide very valuable guidelines for design, if their limitations and relationship to the overall design process is fully recognized.

ACKNOWLEDGEMENTS

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Toni Avery and Shirley Edwards typed the manuscript. Gail Feazell and her associates are responsible for the drafting.

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In all of the Figures in which comparison is made between measured and computed responses, the solid line represents the measured response, the dotted line the computed.

1. INTRODUCTION

Because of economic considerations, structures are usually not designed to remain elastic during severe earthquake excitations. This design philosophy may be acceptable if it is possible to take advantage of a structure's ability to dissipate energy through inelastic deformations. These deformations, however, must be controlled in order to prevent collapse due to exhaustion of the structure's energy dissipation capacity or due to excessive lateral displacements. The ability to predict the ductility demand of a future earthquake excitation for a structure designed with a specified yield strength is therefore of great interest in earthquake resistant design.

Representing the future earthquake excitation by a recorded historical ground motion and knowing the hysteretic material behavior of a structure, we can predict accurately the ductility demand. Using the actual hysteretic behavior introduces on the other hand both technical and practical difficulties for the computations and it would be very helpful, if the ductility demand could be predicted by a simpler method of approach. The most practical approach is to replace the actual hysteretic system by an equivalent linear system and use it's maximum deformation as a basis for approximating the ductility demand for the structure. This kind of an approach would be very useful, if it could give reasonably accurate predictions of the actual response.

The linearization of nonlinear systems is not new and it is not a recent development in earthquake engineering. One of the problems of this study was that of selecting from the large array of linearization schemes a limited number that we could examine. Many of the schemes are very ingenious, and we hope that we have selected a representative group.

Whenever a nonlinear system is replaced by a linear one, it is critical to ascertain the limits of the linear system and to appraise the system in the context what it is that it must predict. To our knowledge none of the methods of linearization presented in this report has been appraised against the only real test of its value, that of predicting physical response to an earthquake excitation. This then is the purpose of this work.

To construct and later assess equivalent linear models, we use test results from an experimental program on a one-story steel frame performed by Sveinsson and McNiven [1] presented in Chapter 2. The frame was subjected to four historical ground motions - El Centro, Pacoima, Taft and Parkfield - causing inelastic deformations of the structure in all cases.

Some of the methods of equivalent linearization used in this investigation depend on the restoring force-displacement relation. Since explicit expressions for the equivalent linear parameters based on a bilinear hysteretic model are readily available in the literature [2,3,4,5], and it is evident from the test results of Chapter 2 that the hysteretic behavior of the structure can be approximated by such a model, we construct two bilinear models in Chapter 3. At this point we must be cognizant of what it is that the linear systems are constructed to predict. They are not attempting to predict the complete time histories of the acceleration and displacement response, but only the maximum values of these, with precedence being given to the maximum displacement response.

Herein lies the reason that we have to construct two bilinear models. Structural steel behaves beyond yield as if it were two different materials. When the strain imposed forces the stress beyond yield, the first excursion into the plastic zone is elastic-plastic, but further hysteretic behavior reveals that the steel is work hardened. One bilinear model is needed for each behavior. The elastic-plastic bilinear model is appropriate when the maximum displacement response occurs in that domain, the work hardening bilinear model when it occurs later in the response.

Six methods of equivalent linearization for SDOF systems subjected to earthquake excitation are described in Chapter 4. Two of these methods are independent of the restoring force-displacement relation but depend instead on the response time histories of the system to the given excitation. The remaining four methods depend on the restoring force-displacement relation in addition to the maximum displacement of the system to the given excitation.

In Chapter 5 we use the methods of equivalent linearization described in Chapter 4 and the structural response to El Centro to construct various equivalent linear models. For the methods requiring restoring force-displacement relation, bilinear models of Chapter 3 are used. We then make an assessment of the models by comparing their response predictions with the measured nonlinear responses for El Centro and the other three excitations of Chapter 2.

Concluding remarks on the applicability of the method of equivalent linearization for SDOF systems subjected to earthquake excitations are given in Chapter 6.

2. THE EXPERIMENTAL PROGRAM

The tests performed on the one-story steel frame, the results of which are used in this research, are discussed briefly in this chapter. A more detailed description of the program is contained in a report by Sveinsson and McNiven [1].

2.1 Test Structure

The primary requirements for the design of the test structure were that it have essentially a single-degree-of-freedom and that it exhibit a very simple hysteretic energy dissipating behavior. Fortunately such a structure had been designed, built and tested at EERC by Rea, Clough and Bouwkamp in 1969 and Reference [6] gives a complete description their structure.

The structure tested by Sveinsson and McNiven is shown in Figs. 2.1 and 2.2. Briefly the structure consists of a heavy steel platform supported by four columns; two fixed to the table and pinned at the top, and two pinned at both the top and bottom. The platform, which is rigid compared to the columns, has overall plan dimensions of 10 ft by 7 ft. The fixed-end columns, fabricated from WF 4 x 13 lb. mild steel, are 66.5 in. in overall length and are installed so that they bend about their weak axes. Parabolic straps are added to strengthen the base of the fixed-end columns.

Two identical pairs of fixed-end columns were used. Each pair was used twice as virgin columns by rotating them top to bottom after the completion of a test causing a nonlinear response of the structure. All four of these columns were fabricated from the same piece of steel.

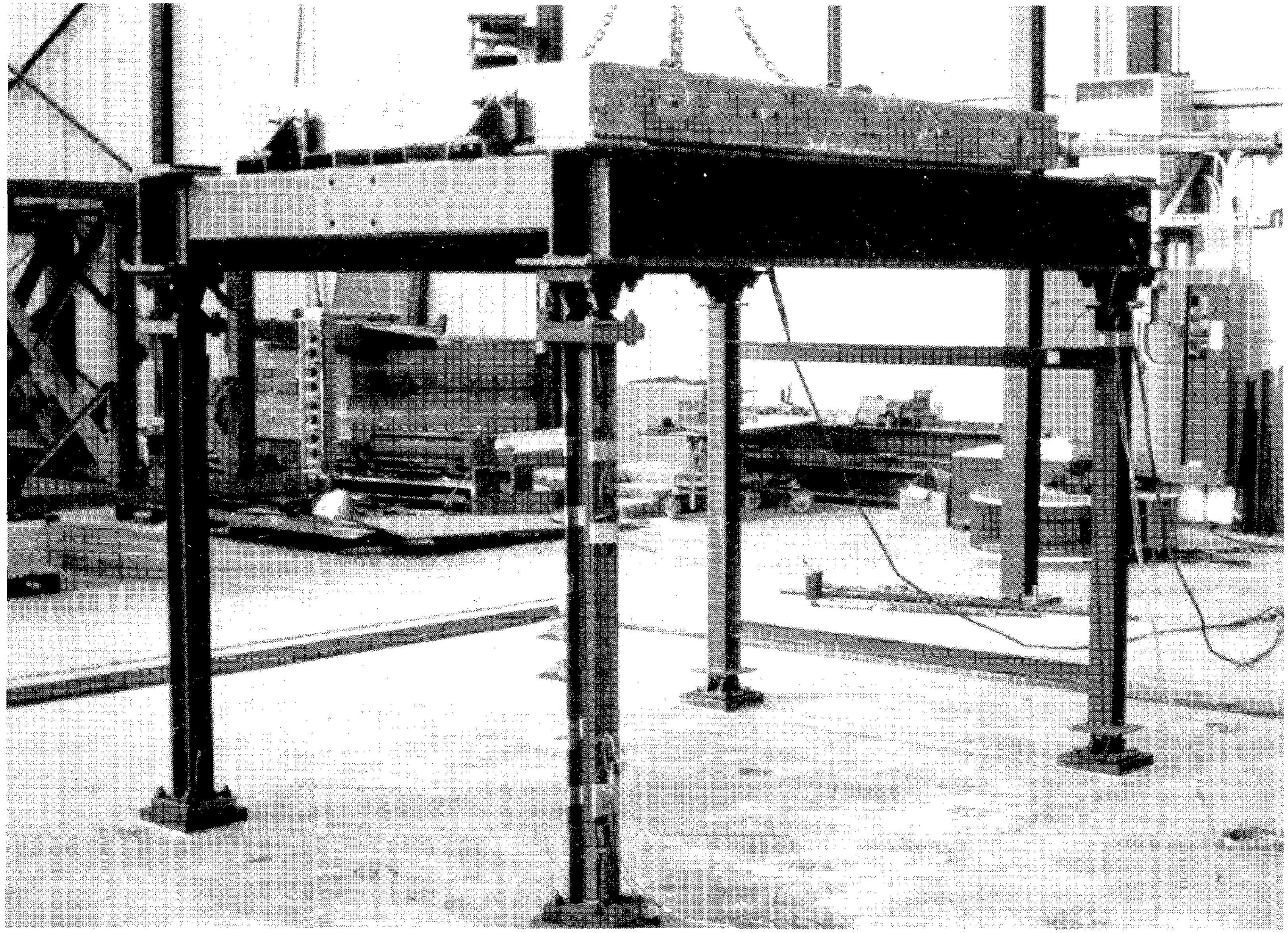

FIG. 2.1 TEST STRUCTURE ON SHAKING TABLE

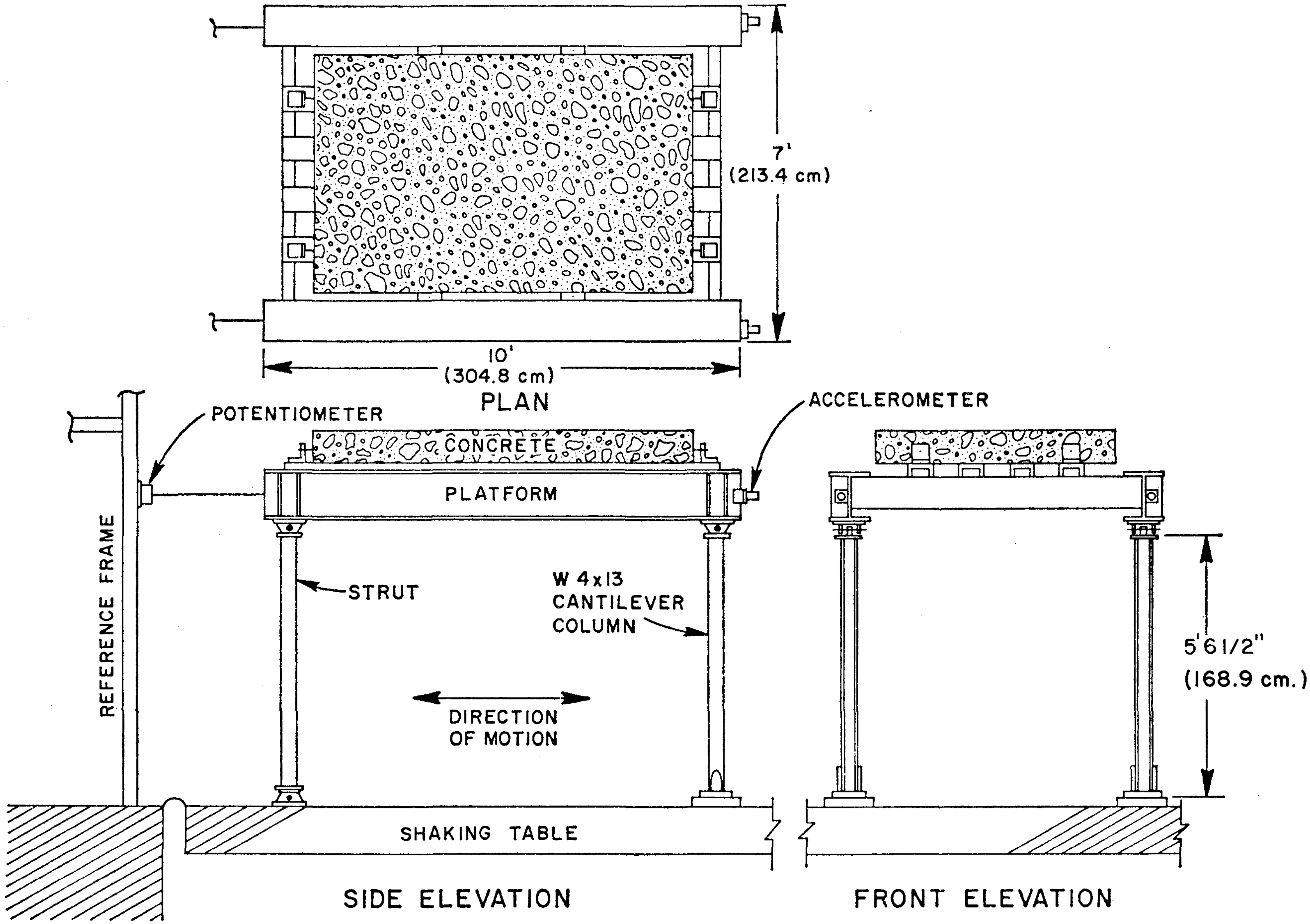

FIG. 2.2 PLAN AND ELEVATION VIEWS OF TEST STRUCTURE

The generalized weight of the structure, defined as the total weight of the platform plus 1/3 of the total weight of the columns, is 5978 lb.

2.2 Instrumentation

Accelerometers were mounted on both sides of the platform to record the absolute accelerations of each side. The absolute displacements of each side of the structure relative to a reference frame remote from the shaking table was measured by potentiometers. In all subsequent computations the absolute accelerations (or displacements) are taken as the average of the measured accelerations (or displacements) of each side. The accelerations of the shaking table were recorded by three built-in accelerometers; one in the middle of the table and one on each side. The table accelerations are taken as the average of the three. The table displacements were measured in the same manner as the table accelerations.

The accuracy of the recorded data cannot, of course, be precisely determined since it depends on the accuracy of calibration for each test, among other things. However, the overall accuracy of the data acquisition system is thought to be within about 0.1%.

2.3 Test Results

To accomplish the objectives of this research we need records of the nonlinear response of the structure due to a variety of excitations. Sveinsson and McNiven subjected the structure to four historical earthquake excitations, each severe enough to cause significant inelastic deformations.

2.3.1 El Centro, 1940

The 1940 El Centro, N-S component was used, and the measured table acceleration time history is shown in Fig. 2.3. The measured relative acceleration and displacement time histories of the structure are shown in Figs. 2.4 and 2.5 respectively.

The equation of motion for a SDOF system with viscous damping and subjected to support excitation may be written as:

$$m_o \ddot{x}_{abs}(t) + c_o \dot{x}(t) + h(x,t) = 0 \; ; \; \dot{x}(0) = x(0) = 0 \tag{2.1}$$

or

$$h(x,t) + c_o \dot{x}(t) = - m_o \ddot{x}_{abs}(t) \; ; \; \dot{x}(0) = x(0) = 0 \tag{2.2}$$

where

- m_o is the mass,
- c_o is the viscous damping coefficient,
- $\ddot{x}_{abs}(t)$ is the absolute acceleration,
- $x(t)$ is the relative displacement,
- $h(x,t)$ is the restoring force.

We can therefore obtain an approximate restoring force time history of the structure by multiplying the measured absolute acceleration of the platform by the generalized mass of the structure and changing the sign. This relation is obviously only absolutely true at the peak values of the displacement, but plotting this force against the displacement gives an idea of the shape of the dynamic hysteretic loops. Fig. 2.6 shows this relation, which will be referred to as pseudo-hysteretic behavior.

An important characteristic of the cyclic inelastic behavior of mild steel is evident from the shape of the pseudo-hysteretic loops.

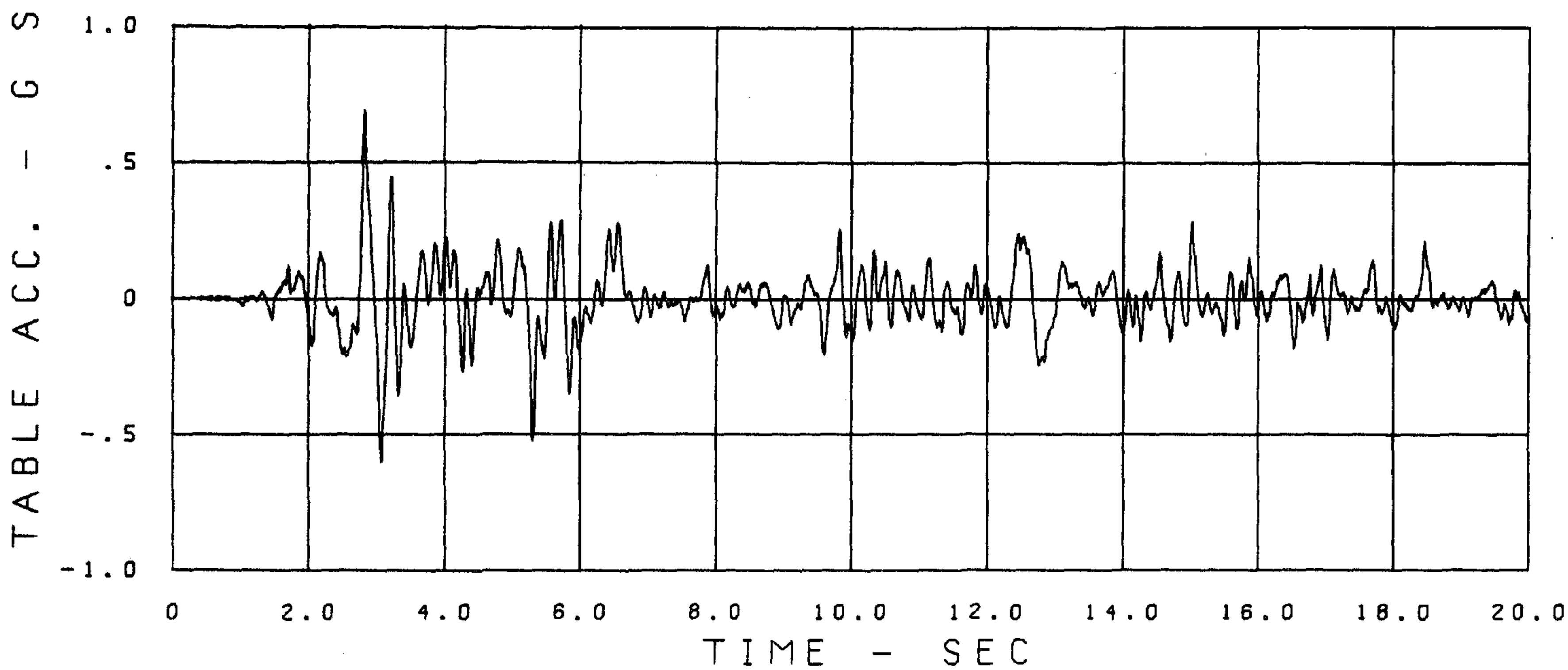

FIG. 2.3 MEASURED TABLE ACCELERATION, EL CENTRO

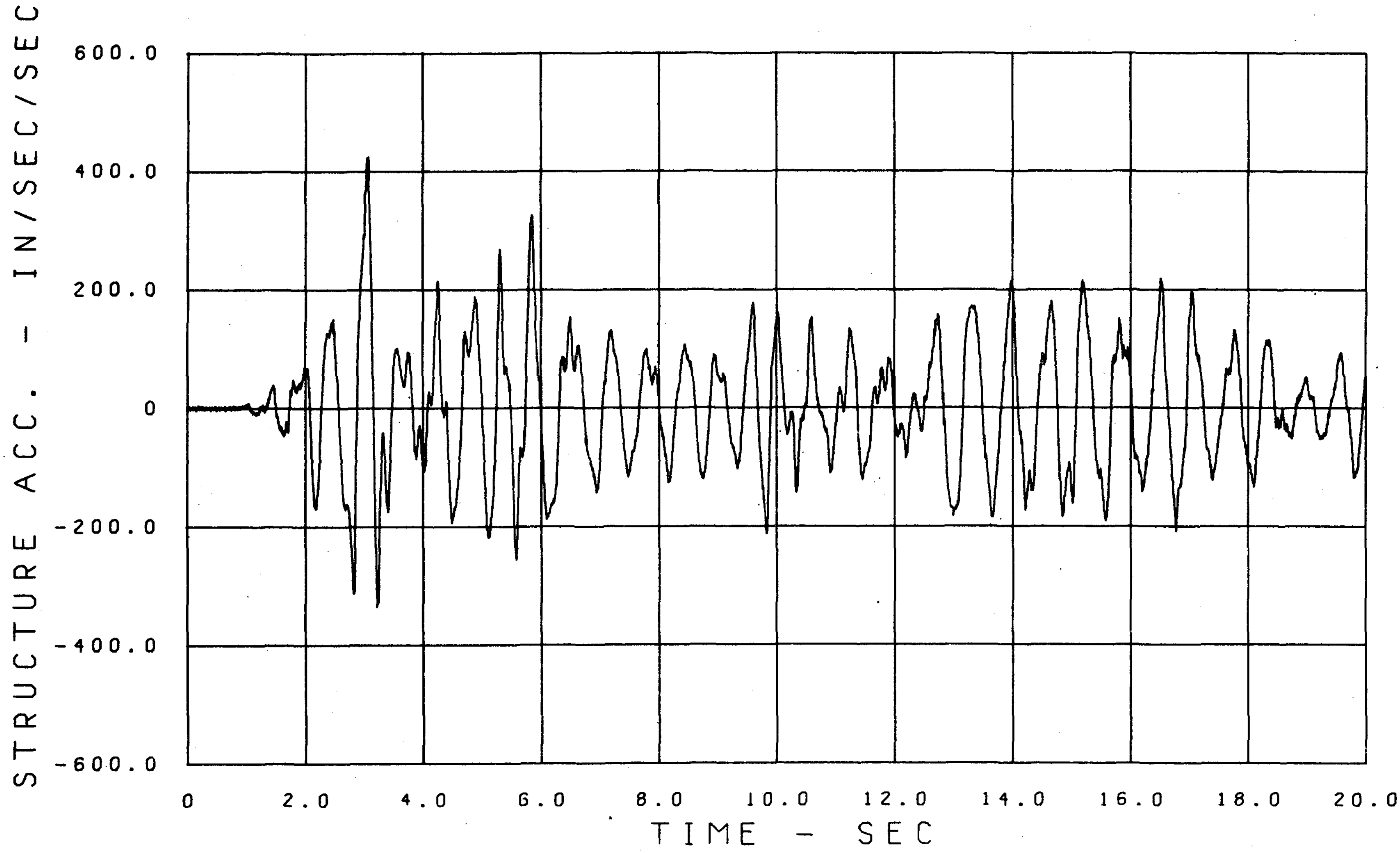

FIG. 2.4 MEASURED RELATIVE ACCELERATION RESPONSE TIME HISTORY, EL CENTRO

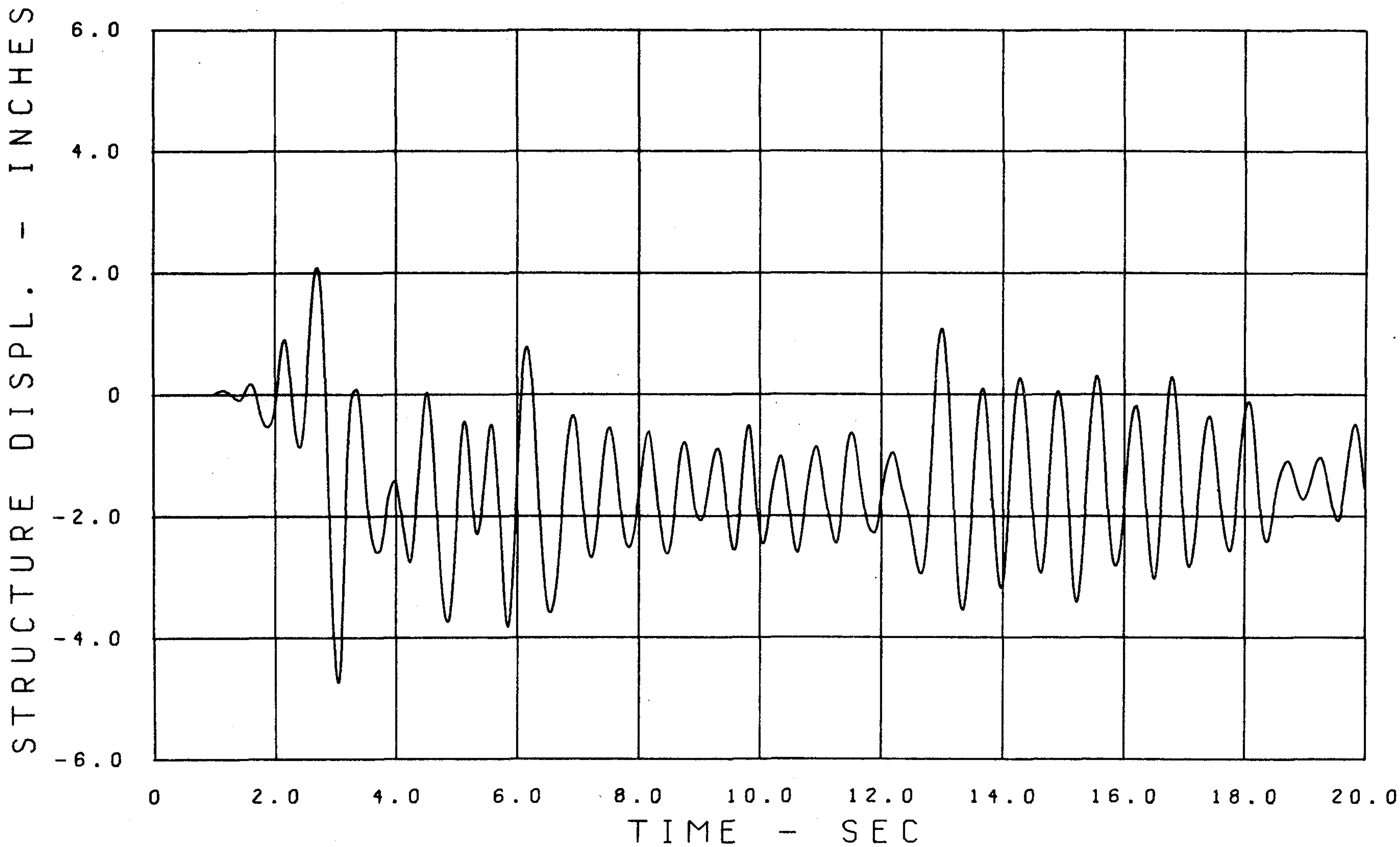

FIG. 2.5 MEASURED RELATIVE DISPLACEMENT RESPONSE TIME HISTORY, EL CENTRO

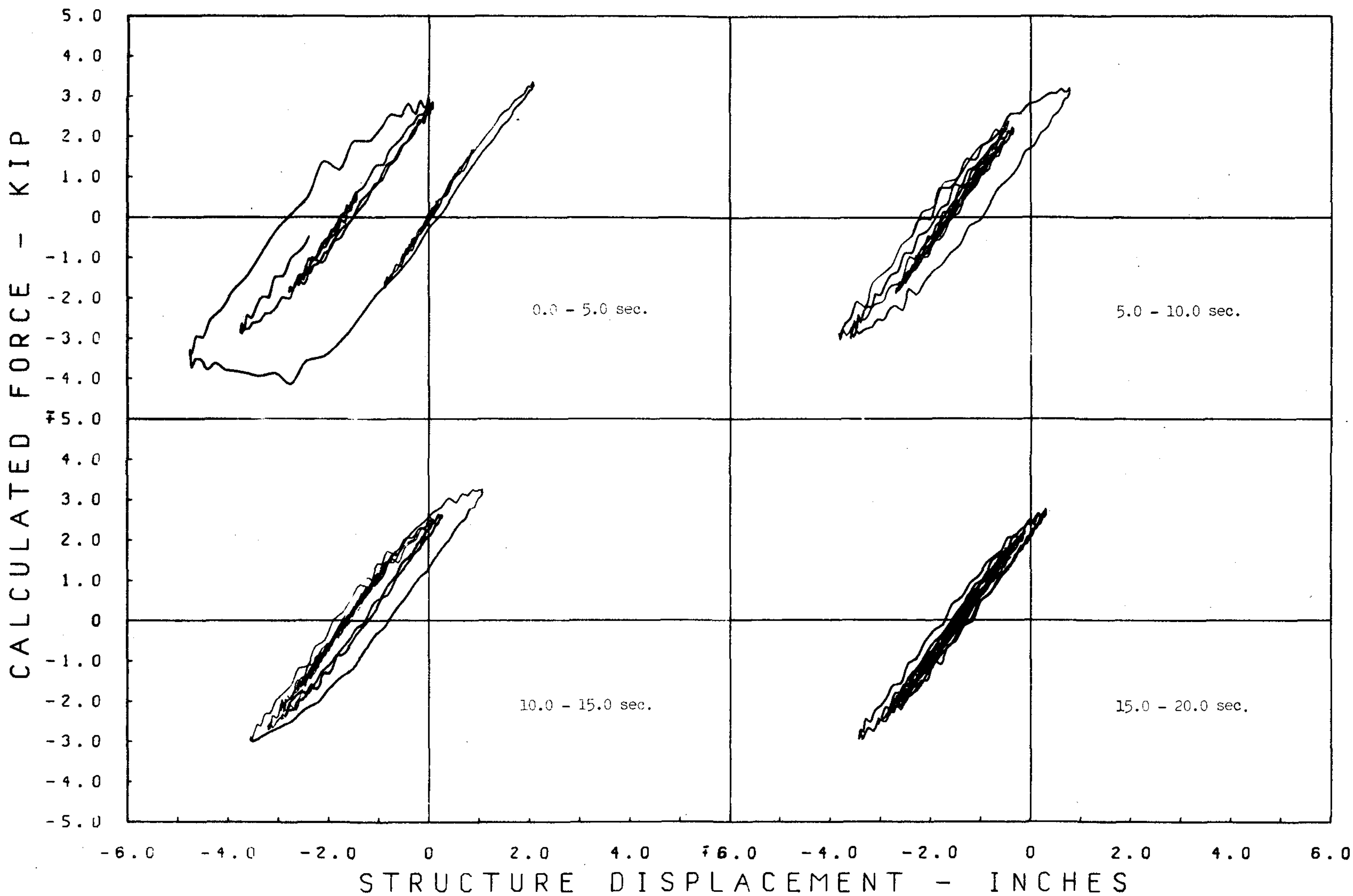

FIG. 2.6 MEASURED PSEUDO-HYSTERETIC LOOPS, EL CENTRO

The first major excursion into the inelastic region is essentially elastic-plastic (phase I), whereas all subsequent loops have a smooth transition from elastic to inelastic response indicating work hardening behavior (phase II). This two-phase character is central to the problem of modeling the behavior of mild steel structures.

2.3.2 Pacoima, 1971

The 1971 Pacoima, S16E component was used, and the measured table acceleration time history is shown in Fig. 2.7. The measured relative acceleration and displacement time histories of the structure are shown in Figs. 2.8 and 2.9 respectively. Fig. 2.10. shows the pseudo-hysteretic loops.

2.3.3 Taft, 1952

The 1952 Taft, N69W component was used, and the measured table acceleration time history is shown in Fig. 2.11. The measured relative acceleration and displacement time histories of the structure are shown in Figs. 2.12 and 2.13 respectively. Fig. 2.14 shows the pseudo-hysteretic loops.

2.3.4 Parkfield, 1966

The 1966 Parkfield, N65E component was used, and the measured table acceleration time history is shown in Fig. 2.15. The measured relative acceleration and displacement time histories of the structure are shown in Figs. 2.16 and 2.17 respectively. Fig. 2.18 shows the pseudo-hysteretic loops.

2.3.5 Maximum Response Values

Table 2.1 gives the maximum relative displacement and the maximum absolute acceleration of the structure for each earthquake excitation.

TABLE 2.1

MEASURED MAXIMUM RESPONSE VALUES

Earthquake Excitation	$\ddot{x}_{abs}$ (in./sec^2)	x (in.)
EL CENTRO	267.9	4.75
PACOIMA	261.5	4.85
TAFT	258.1	3.47
PARKFIELD	248.8	4.18

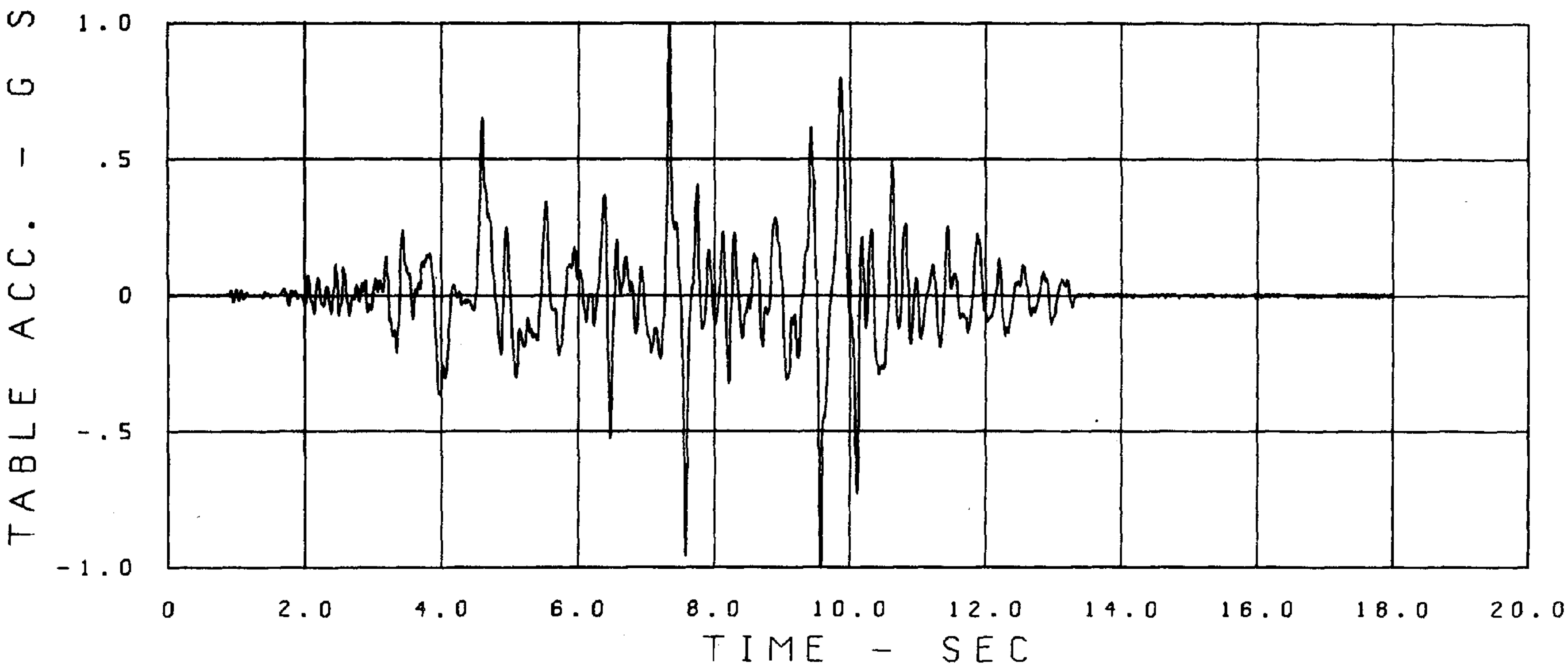

FIG. 2.7 MEASURED TABLE ACCELERATION, PACOIMA

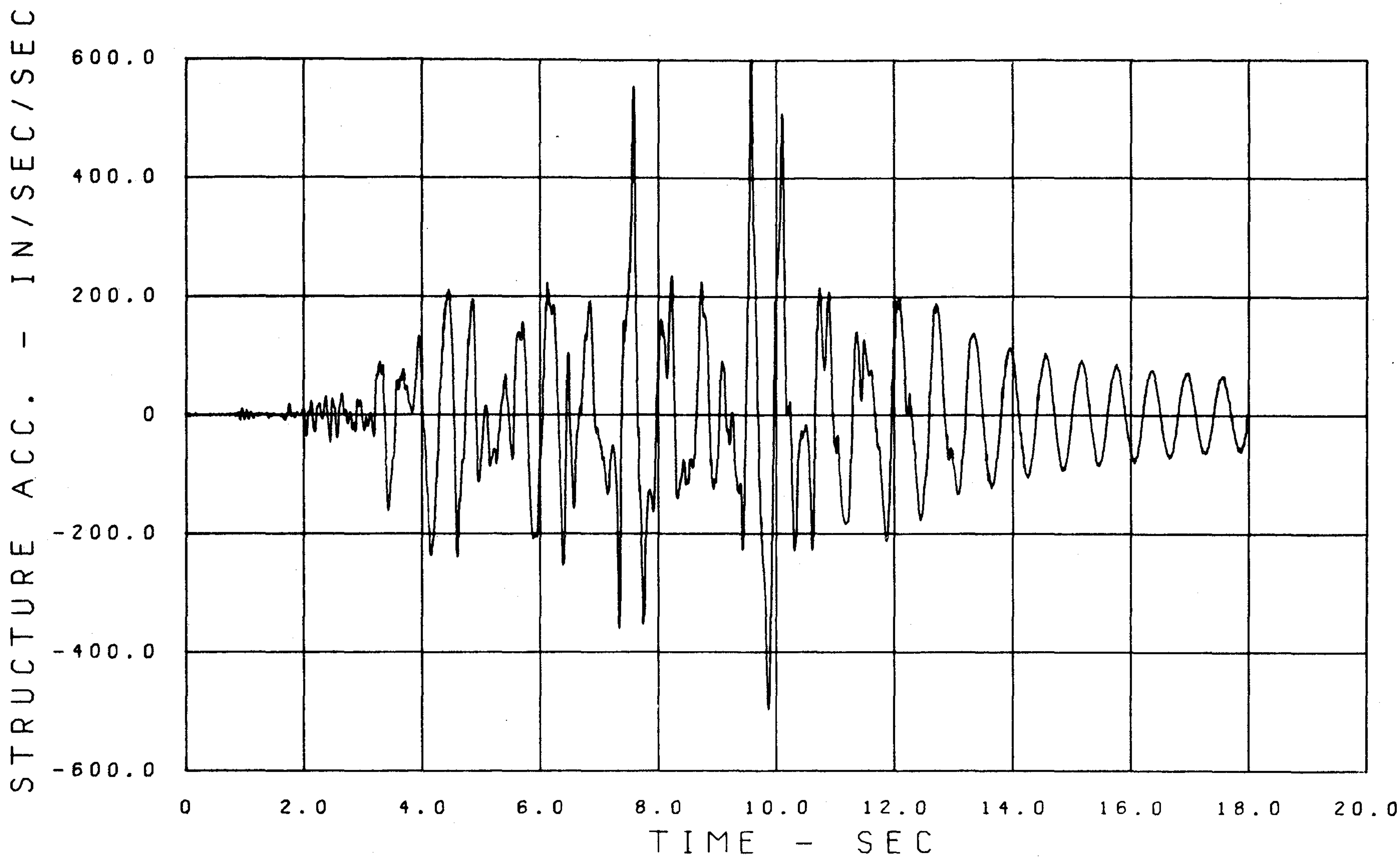

FIG. 2.8 MEASURED RELATIVE ACCELERATION RESPONSE TIME HISTORY, PACOIMA

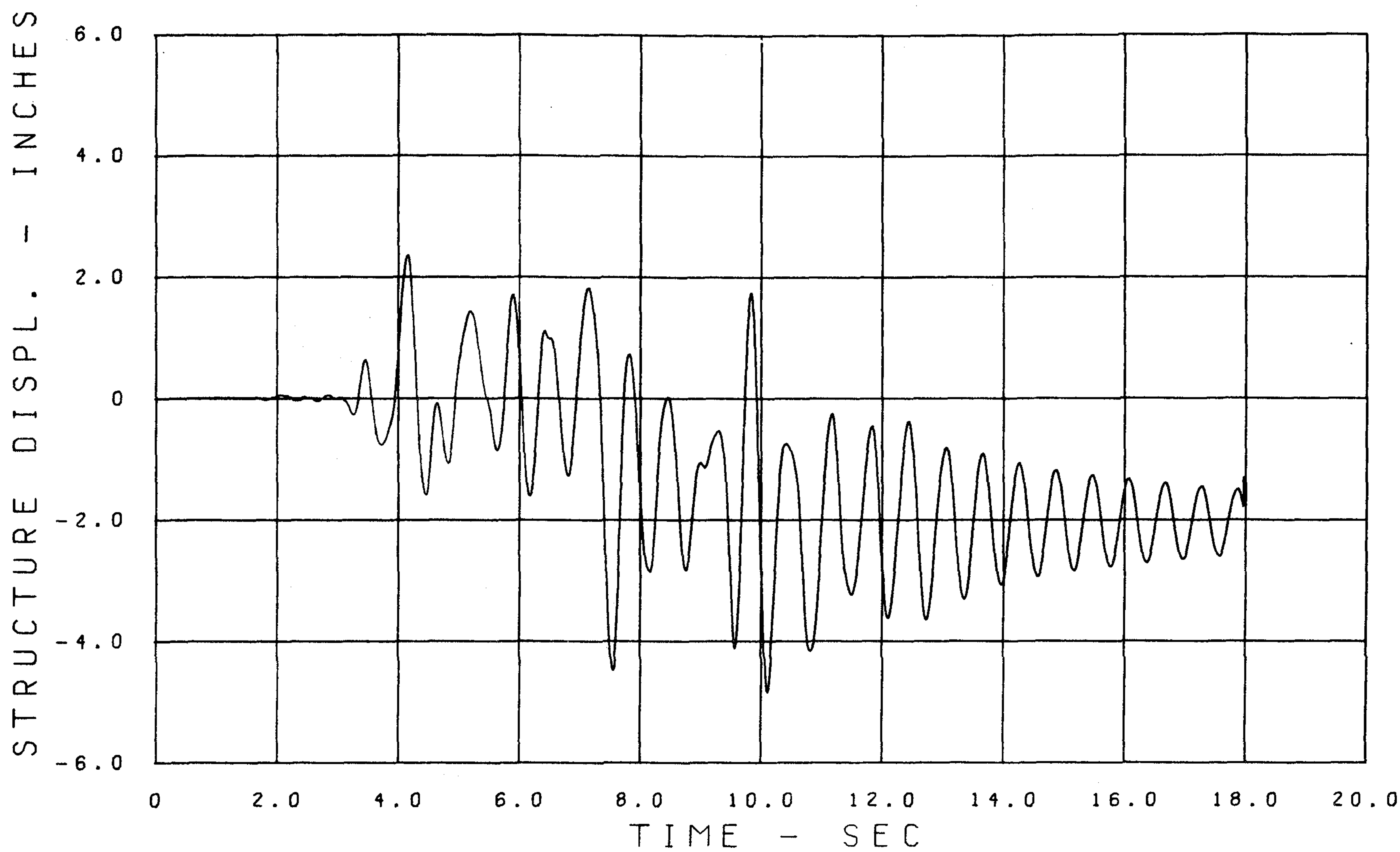

FIG. 2.9 MEASURED RELATIVE DISPLACEMENT RESPONSE TIME HISTORY, PACOIMA

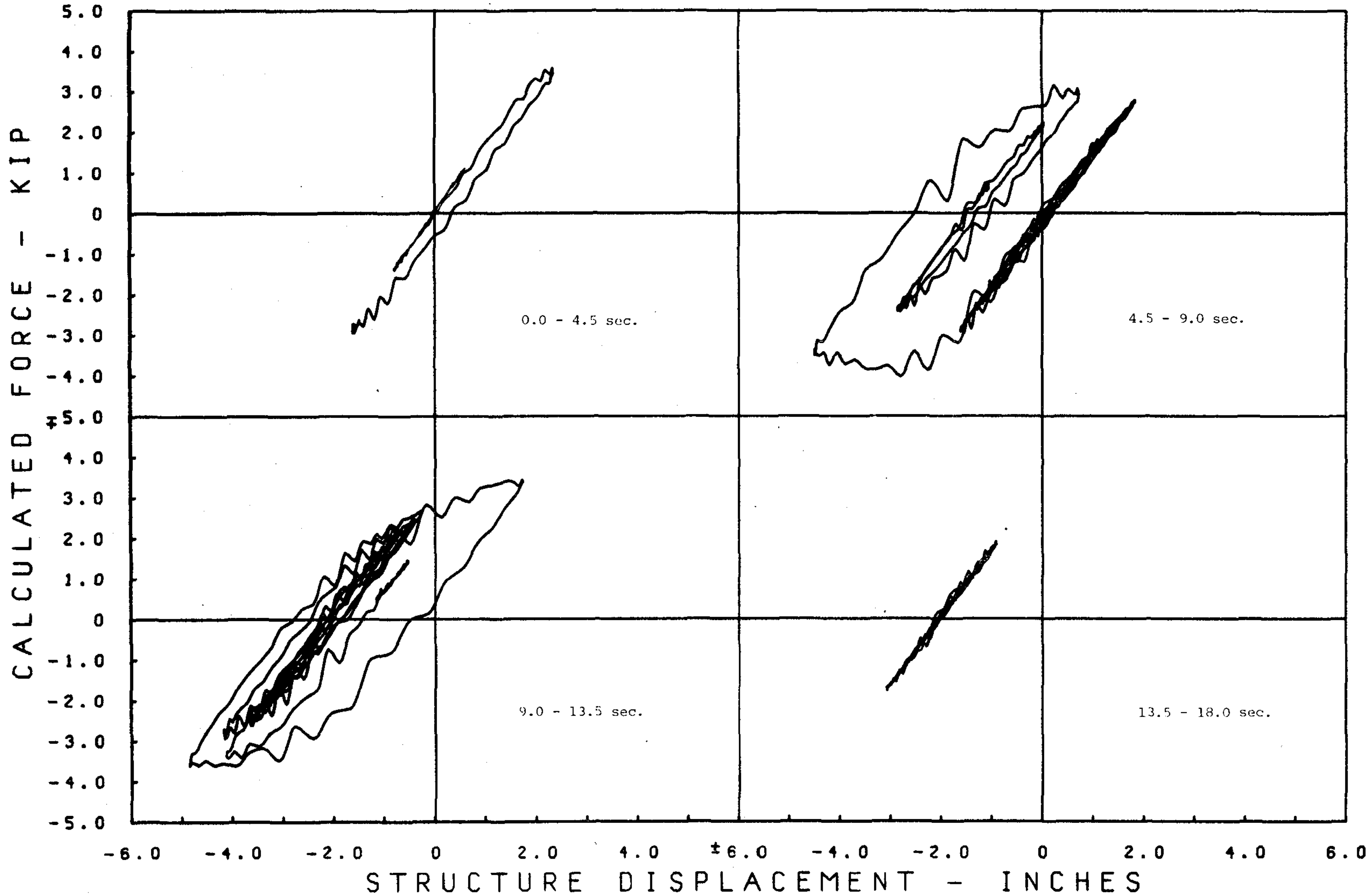

FIG. 2.10 MEASURED PSEUDO-HYSTERETIC LOOPS, PACOIMA

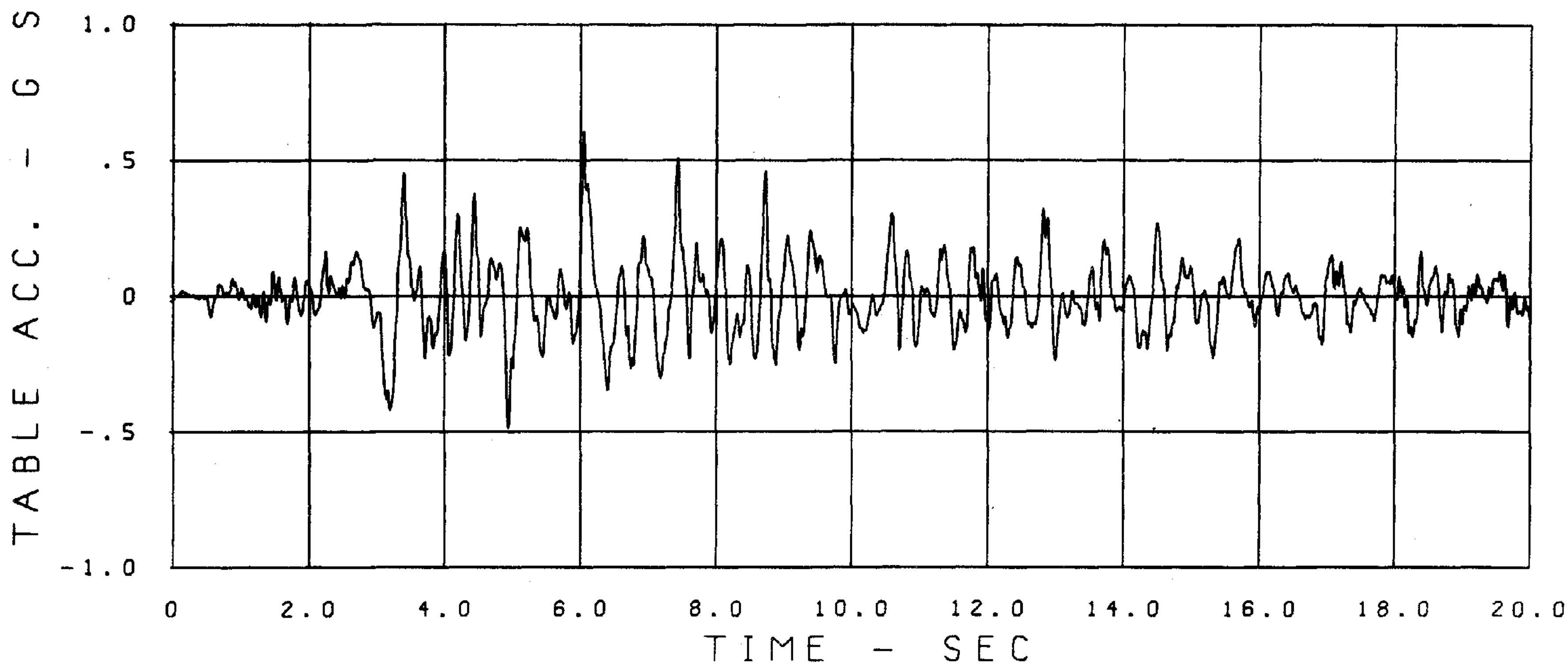

FIG. 2.11 MEASURED TABLE ACCELERATION, TAFT

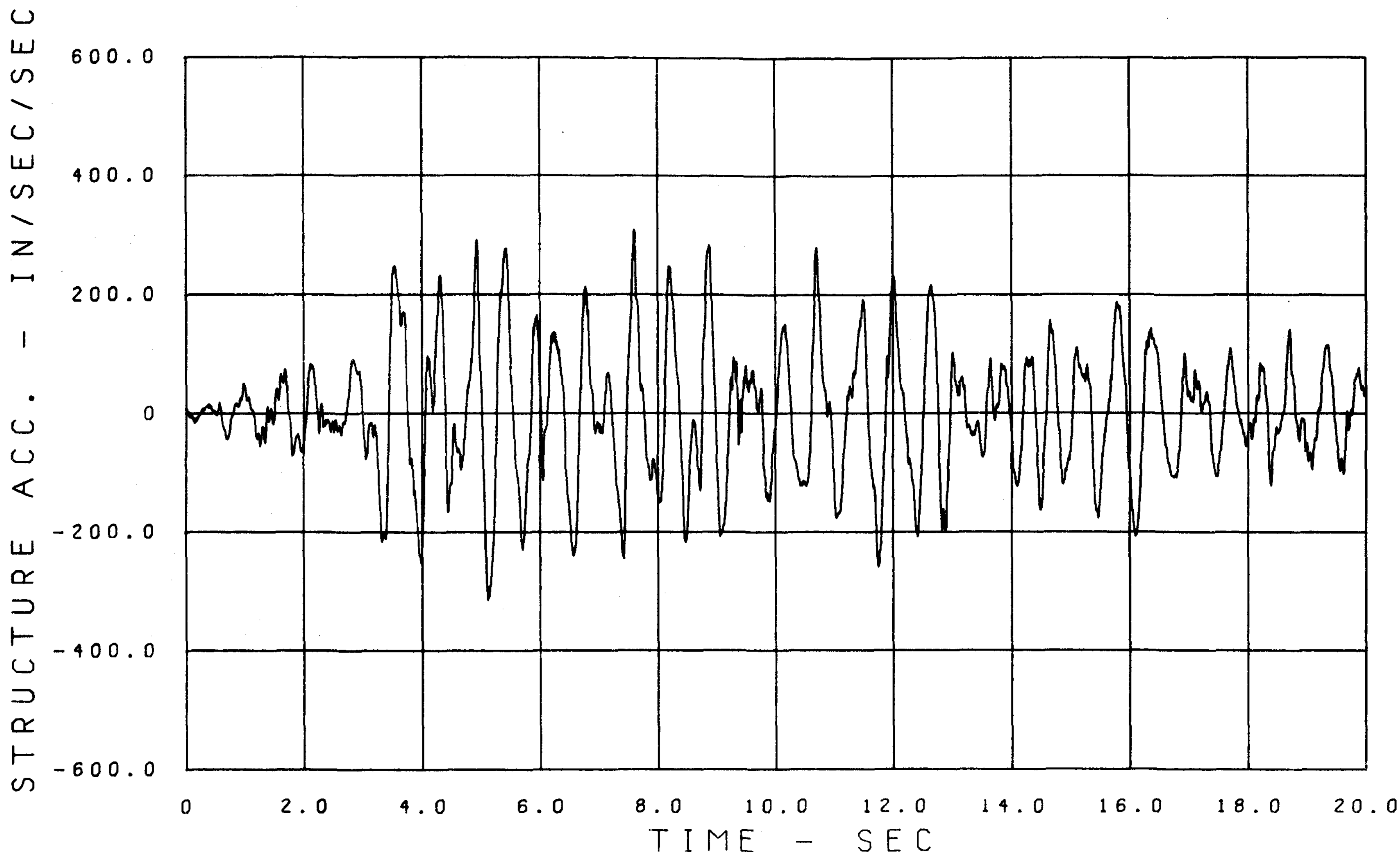

FIG. 2.12 MEASURED RELATIVE ACCELERATION RESPONSE TIME HISTORY, TAFT

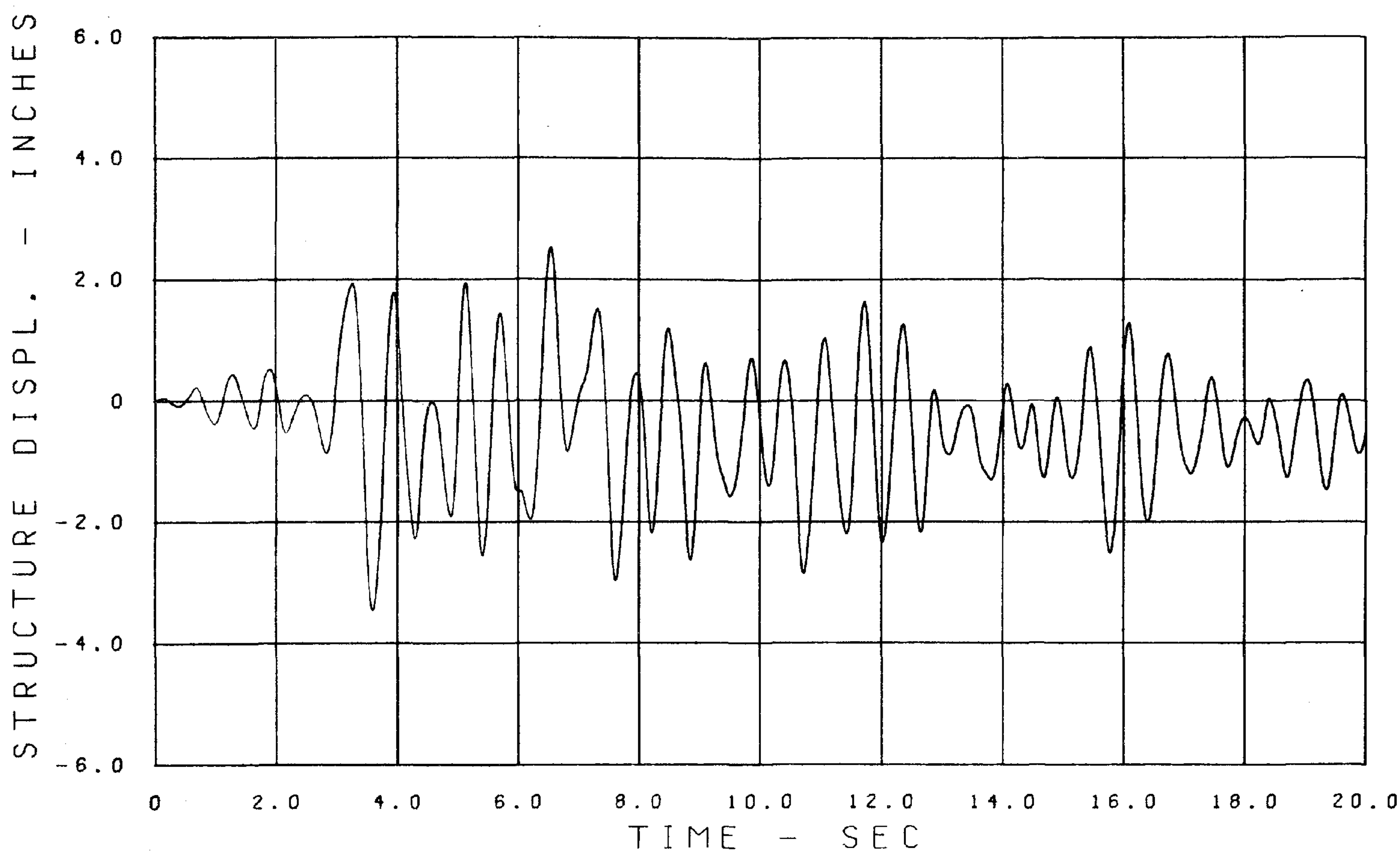

FIG. 2.13 MEASURED RELATIVE DISPLACEMENT RESPONSE TIME HISTORY, TAFT

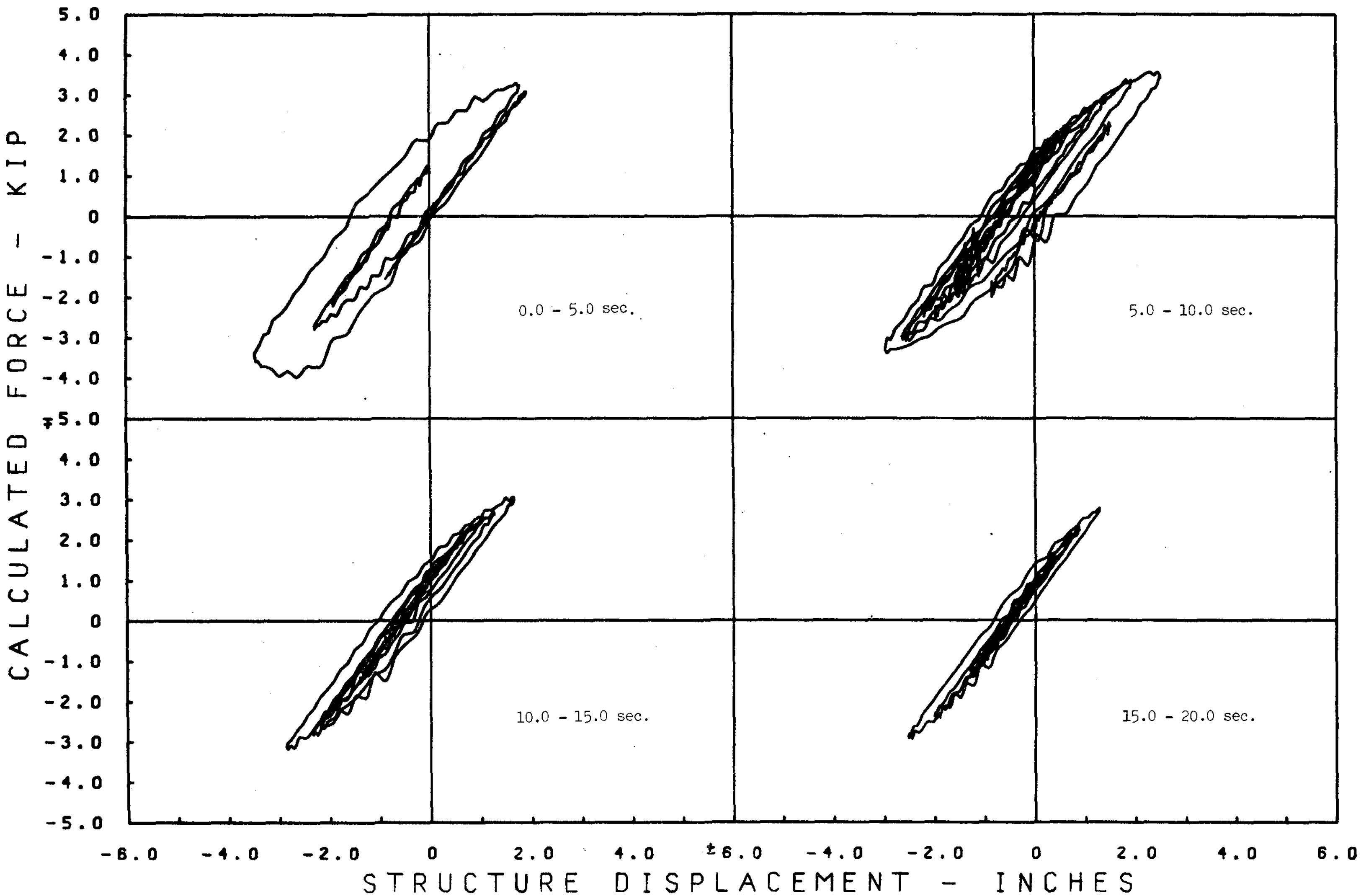

FIG. 2.14 MEASURED PSEUDO-HYSTERETIC LOOPS, TAFT

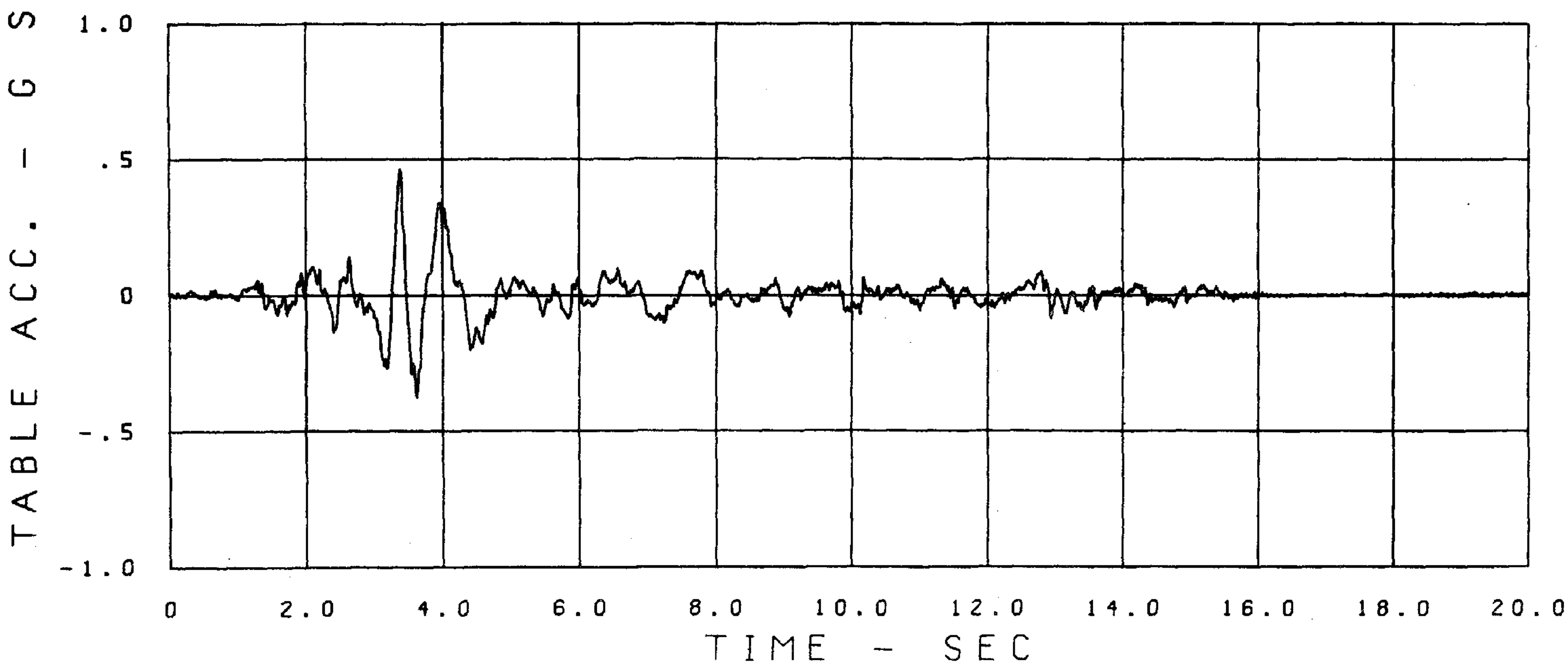

FIG. 2.15 MEASURED TABLE ACCELERATION, PARKFIELD

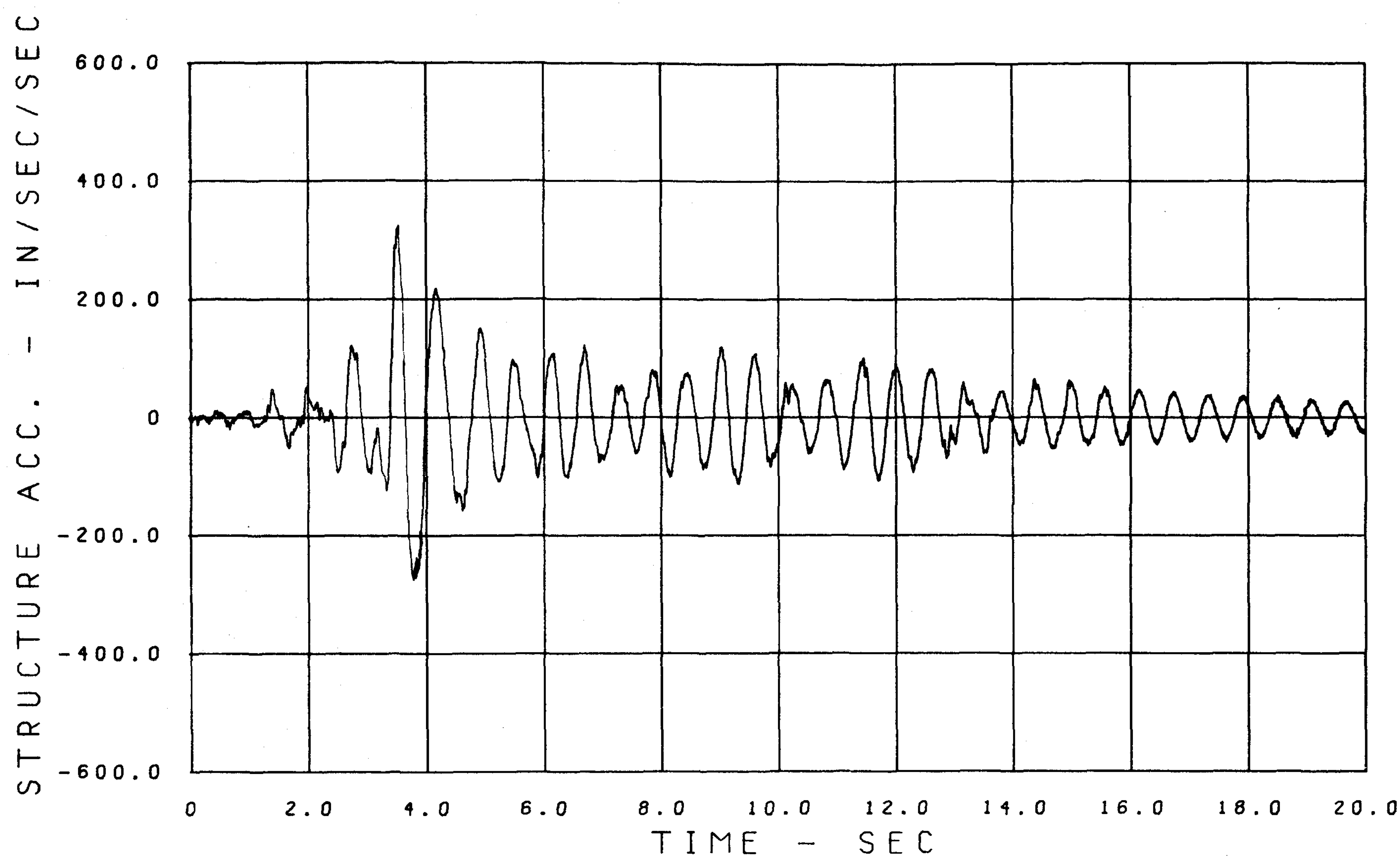

FIG. 2.16 MEASURED RELATIVE ACCELERATION RESPONSE TIME HISTORY, PARKFIELD

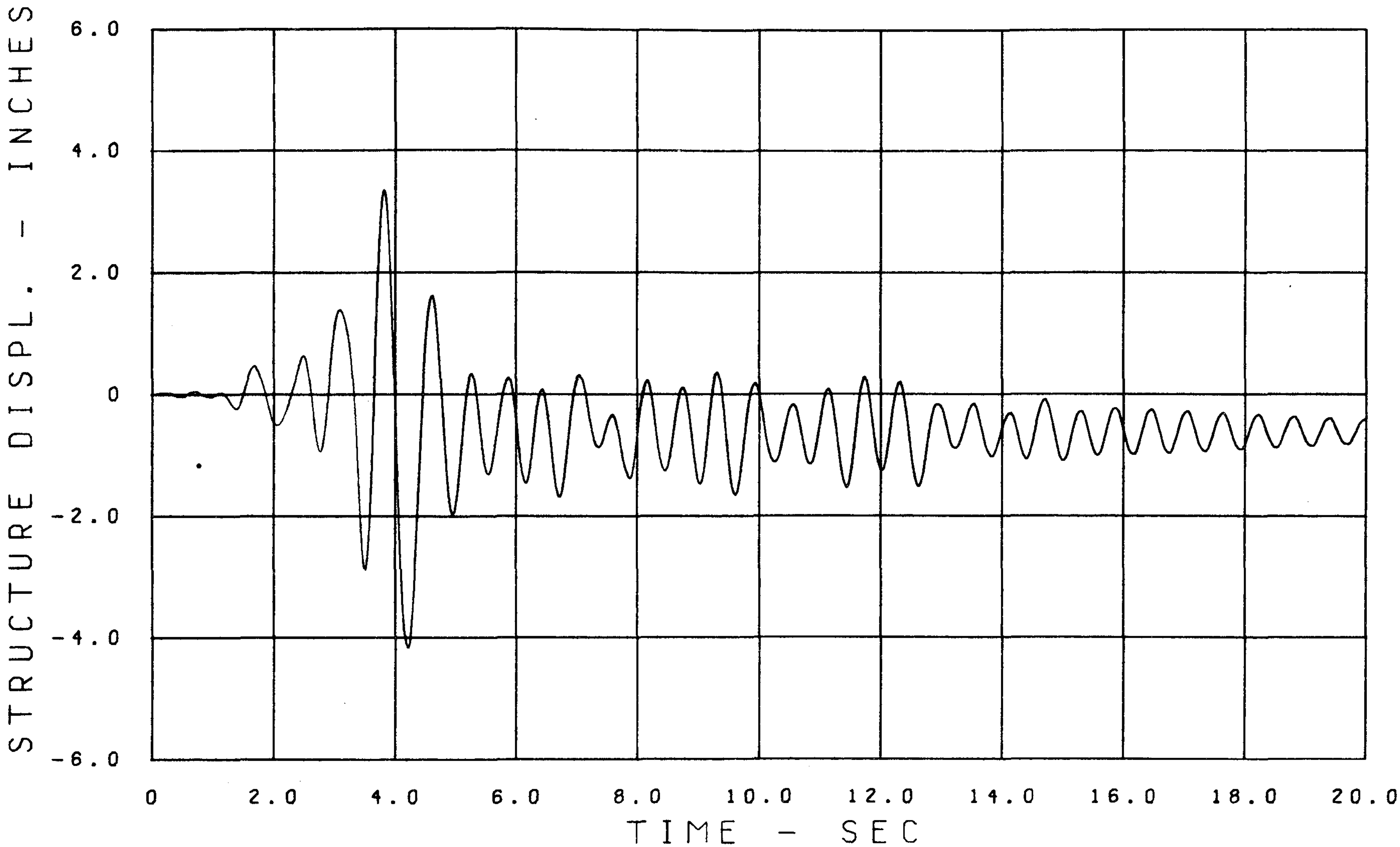

FIG. 2.17 MEASURED RELATIVE DISPLACEMENT RESPONSE TIME HISTORY, PARKFIELD

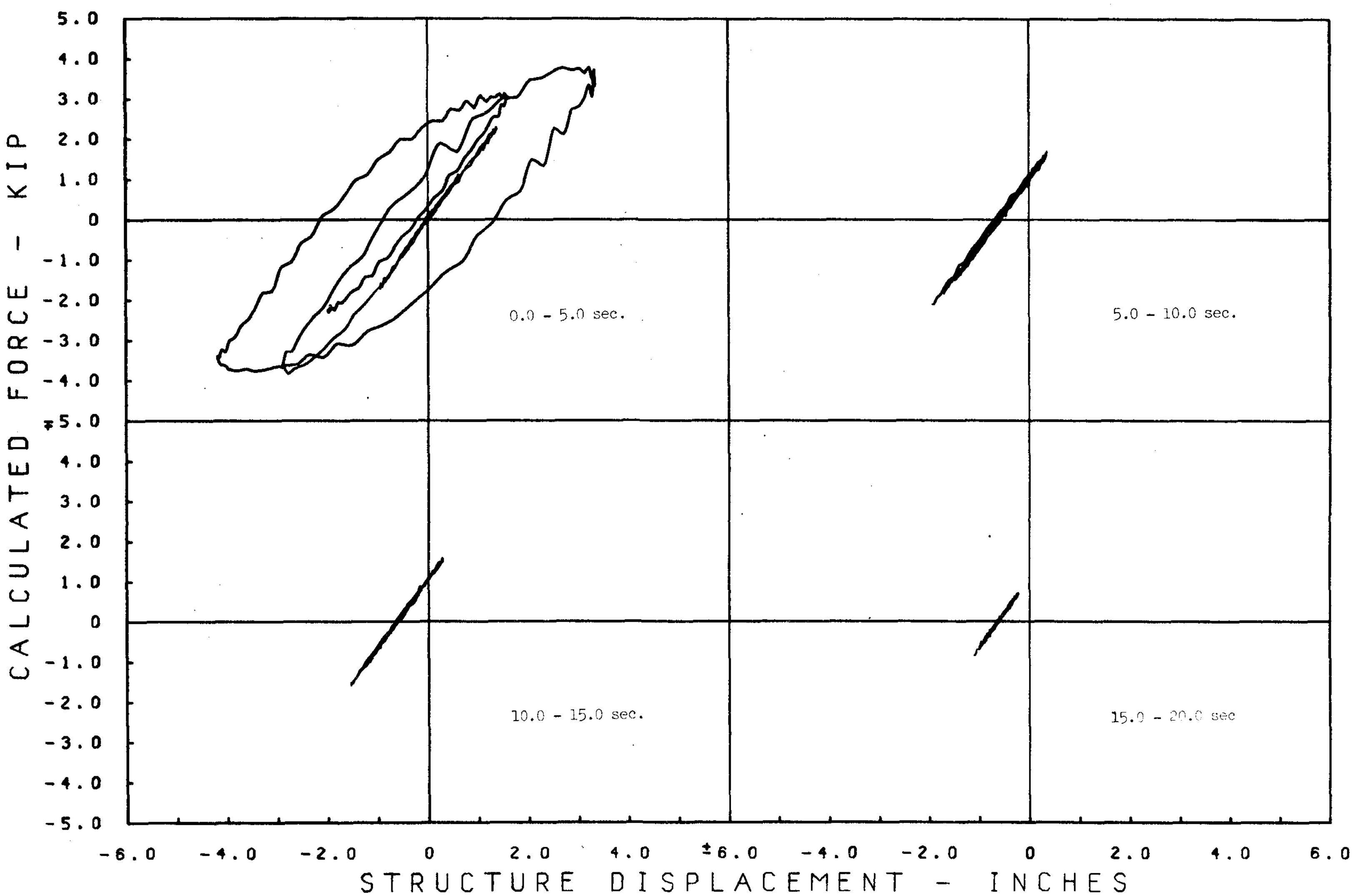

FIG. 2.18 MEASURED PSEUDO-HYSTERETIC LOOPS, PARKFIELD

3. CONSTRUCTION OF BILINEAR HYSTERETIC MODELS

As mentioned in Chapter 1, some of the methods of linearization presented in this report depend on the restoring force-displacement relation. Since explicit expressions for the linear model parameters, based on a bilinear hysteretic model, are readily available in the literature [2,3,4,5], and it is evident from the test results that the hysteretic behavior of the structure can be approximated by such a model, two bilinear models are constructed in this chapter based on the experimental results in Chapter 2.

The equation of motion for a SDOF bilinear hysteretic system with viscous damping and subjected to support excitation may be written as:

$$m_o \ddot{x}(t) + c_o \dot{x}(t) + h(x,t) = -m_o \ddot{x}_g(t); \; \dot{x}(0) = x(0) = 0 \qquad (3.1)$$

where

m_o	is the mass,
c_o	is the viscous damping coefficient,
$\ddot{x}_g(t)$	is the support acceleration,
$x(t)$	is the relative displacement,
$h(x,t)$	is the restoring force.

Figure 3.1 shows the restoring force-displacement relation for a bilinear hysteretic system. This system has initial stiffness k_o, post yield stiffness αk_o and yield displacement x_y. The maximum response displacement is shown as $x_m = \mu x_y$, where μ is the displacement ductility ratio.

To represent the measured structural behavior by differential equation (3.1), we need to establish the appropriate values of m_o, c_o, k_o, α, x_y using the measured responses and some parameter adjustment

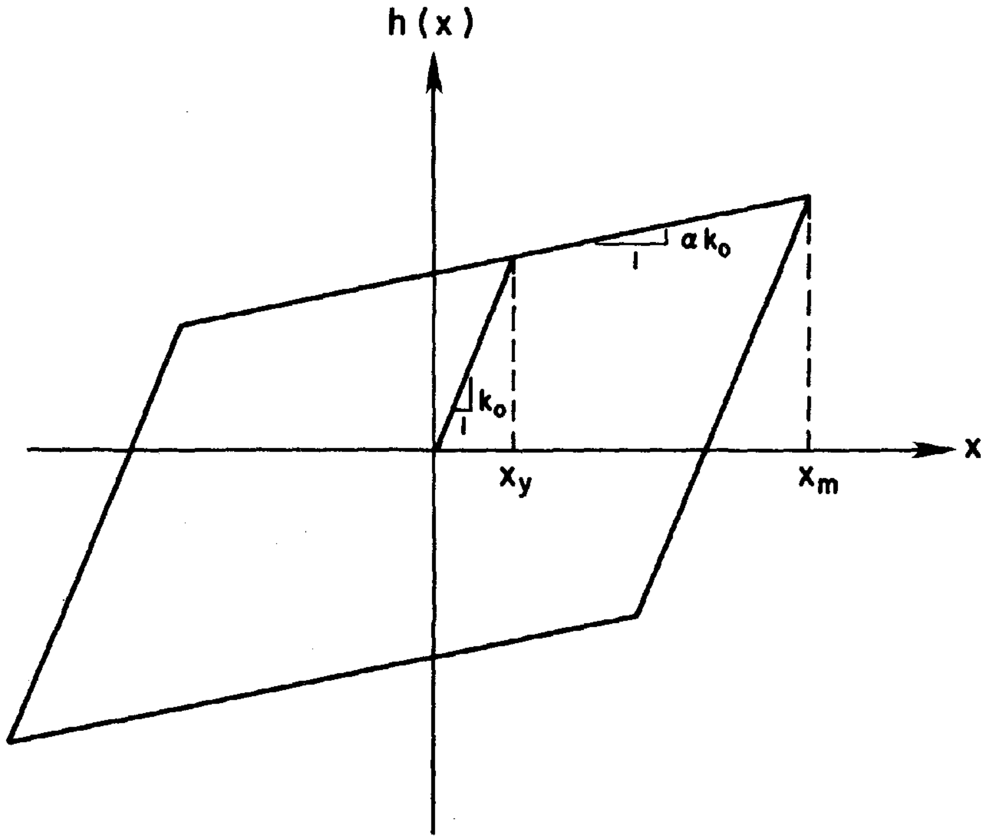

FIG. 3.1 RESTORING FORCE - DISPLACEMENT RELATION FOR A BILINEAR HYSTERETIC SYSTEM

algorithm. It is obvious from the pseudo-hysteretic loops in Fig. 2.6 that one bilinear model cannot completely describe the two-phase behavior of the structural material, and therefore it is essential to clearly define what response character we want our bilinear model to approximate.

As described in the introduction we need two bilinear models: a work hardening model and an elastic-plastic model. For both models we fixed the values of m_o and c_o as

$$m_o = 15.47 \frac{\#\ \sec^2}{\text{in.}}$$

$$c_o = 3.671 \frac{\#\ \sec}{\text{in.}}$$

based on results in Reference [1].

3.1 Work Hardening Model

In their work using a Ramberg-Osgood model and System Identification Sveinsson and McNiven [1] noted that a work hardening model resulted when the full duration of the relative acceleration response was used in the criterion function. Accordingly, here we use the full duration of the relative acceleration response to El Centro and derive the three bilinear parameters by trial and error, so that the model matches that behavior. Accurate matching was achieved when the bilinear parameters had the values:

$$k_o = 1745.5 \frac{\#}{\text{in.}}$$

$$\alpha = 0.4196$$

$$x_y = 1.375 \text{ in.}$$

To ascertain the predictive ability of the above model when subjected to El Centro excitation the calculated and measured (dashed line and solid line, respectively) relative acceleration and displacement time histories are compared in Figs. 3.2 and 3.3. It is evident that the acceleration time history is very well predicted and the displacement time histories have the same general character, although the model cannot predict the inelastic shift in the displacement due to the almost elastic-plastic behavior of the structure in the first-phase of the response.

To assess the general applicability of the model, we subject it to other support excitations and compare the relative acceleration and displacement time histories predicted by the model to the measured responses; Figs. 3.4 and 3.5 show the comparison for Pacoima excitation, Figs. 3.6 and 3.7 for Taft excitation and Figs. 3.8 and 3.9 for Parkfield excitation. From the Figs. 3.2 - 3.9 we observe that the bilinear mathematical model, constructed using response data from the El Centro excitation, predicts responses to the other excitations as accurately as it does to the El Centro. Furthermore, we observe that the responses predicted using the bilinear model are as accurate as the ones predicted using Ramberg-Osgood model [1].

Table 3.1 gives the maximum relative displacement and maximum absolute acceleration predicted by the model for each earthquake excitation.

3.2 Elastic-Plastic Model

To construct this bilinear model, we again borrow from Sveinsson and McNiven [1]. From their pseudo-hysteretic loops for El Centro (Fig. 2.6 here), we ascertain that the structural behavior is elastic-plastic up to the maximum displacement which occurs after approximately

TABLE 3.1

MAXIMUM RESPONSE VALUES

Measured/Predicted	MAXIMUM RESPONSE VALUES							
	EL CENTRO		PACOIMA		TAFT		PARKFIELD	
	$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x
Measured Nonlinear Response	267.9	4.75	261.5	4.85	258.1	3.47	248.8	4.18
Work Hardening Bilinear Model	285.8	4.13	288.4	4.18	248.9	3.35	310.9	4.66
Elastic-Plastic Bilinear Model	267.2	4.75	251.2	3.58	248.2	3.39	251.4	3.62

NOTE: $\ddot{x}_{abs}$ (in./sec^2)

x (in.)

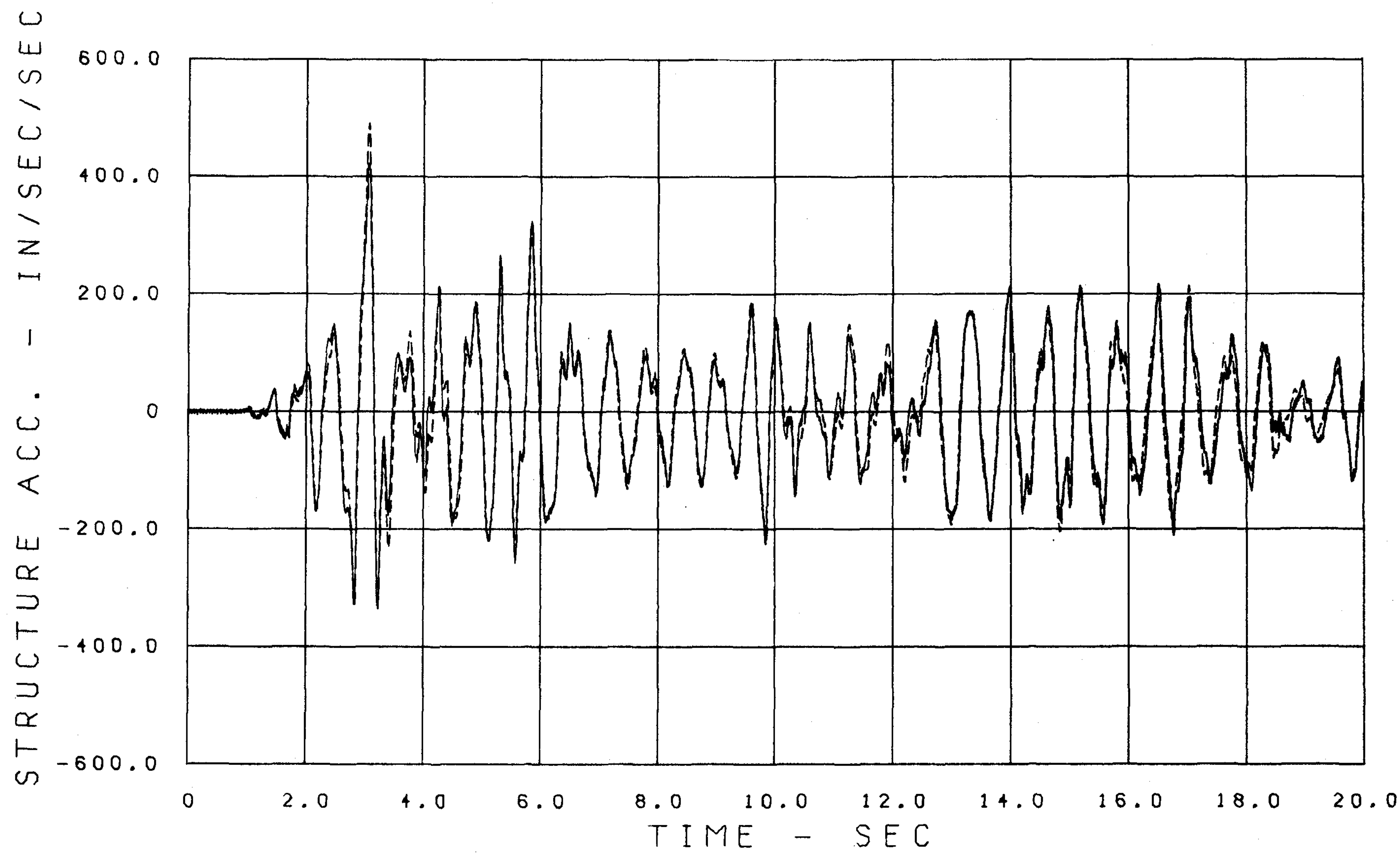

FIG. 3.2 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, EL CENTRO

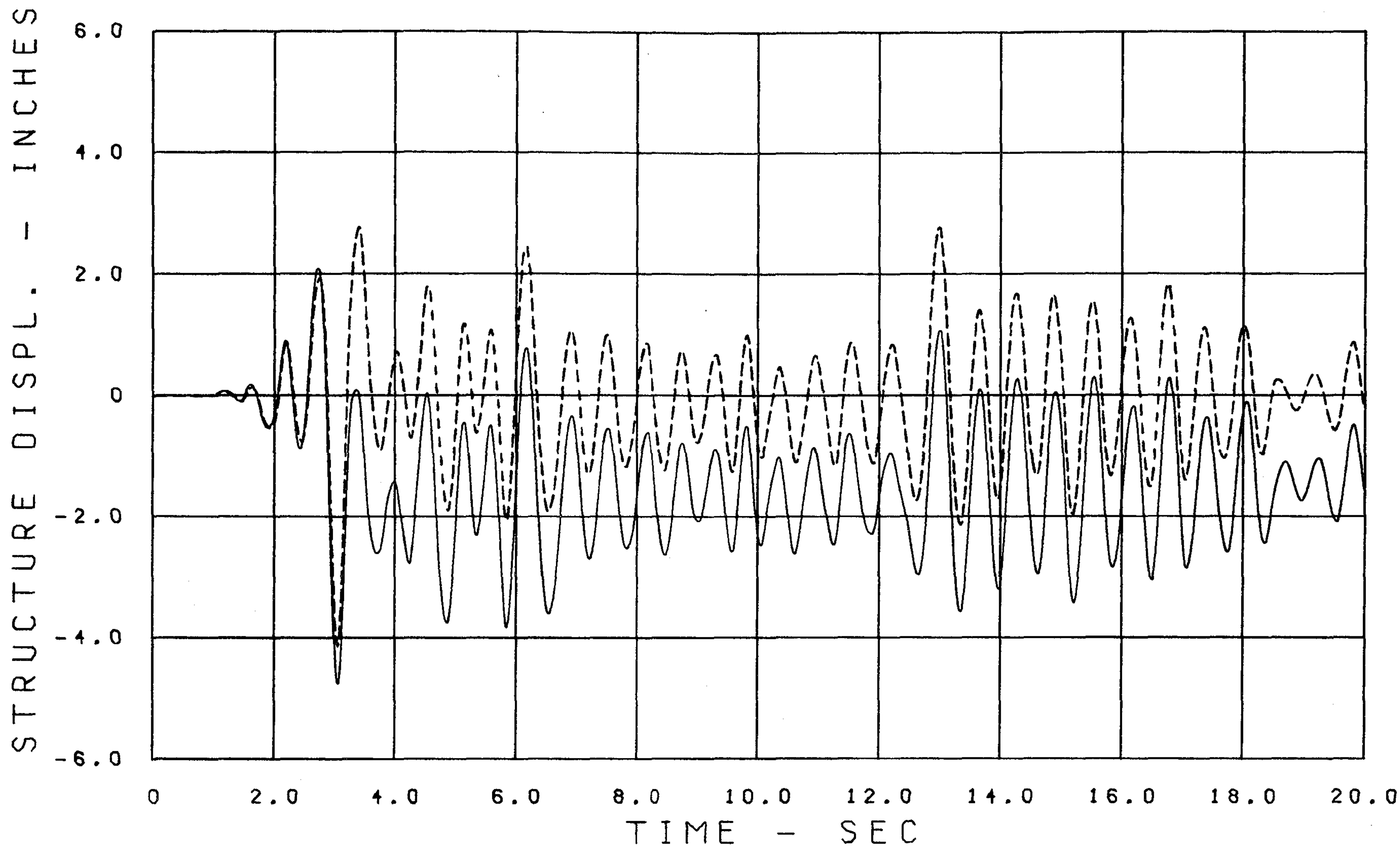

FIG. 3.3 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, EL CENTRO

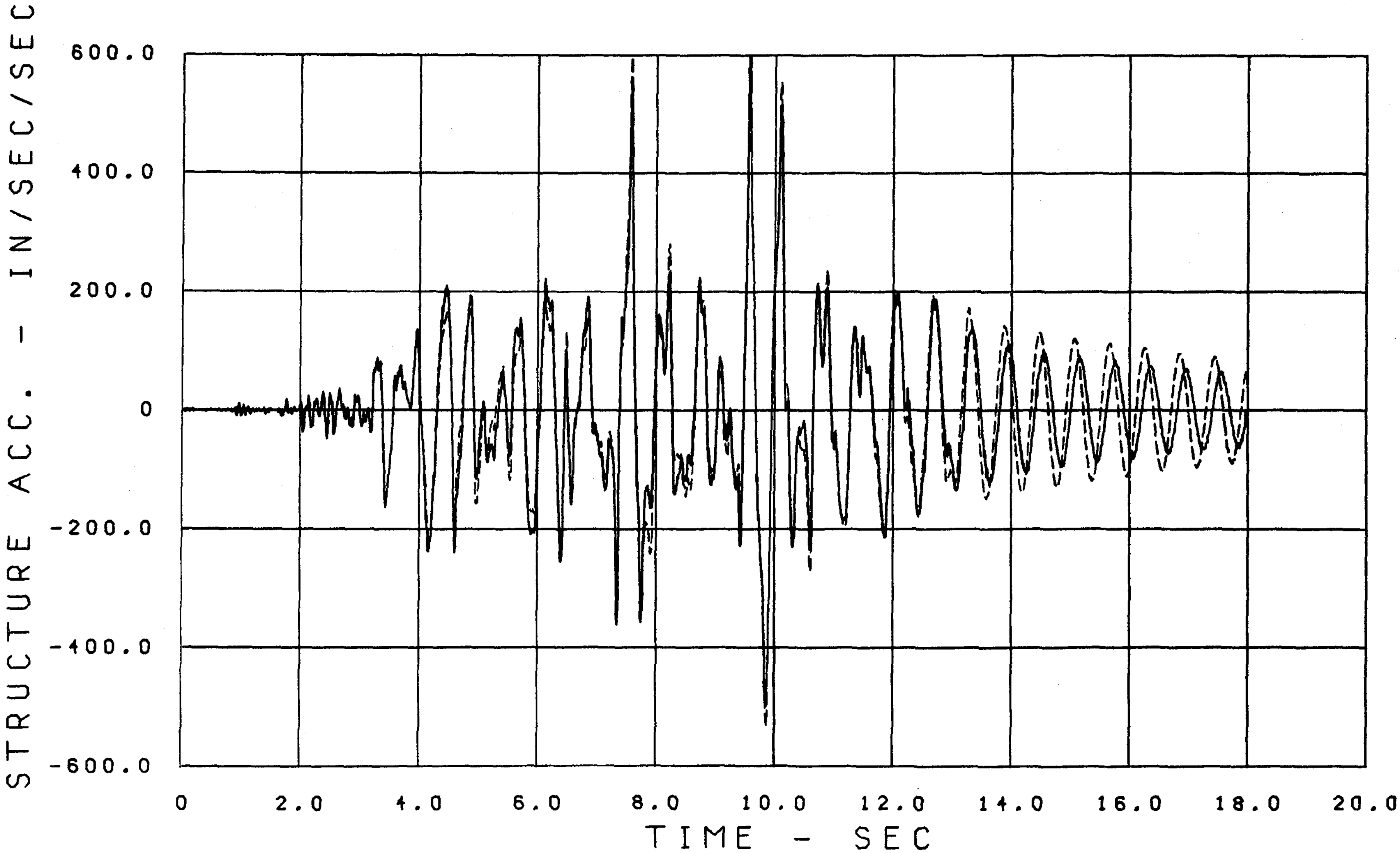

FIG. 3.4 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, PACOIMA

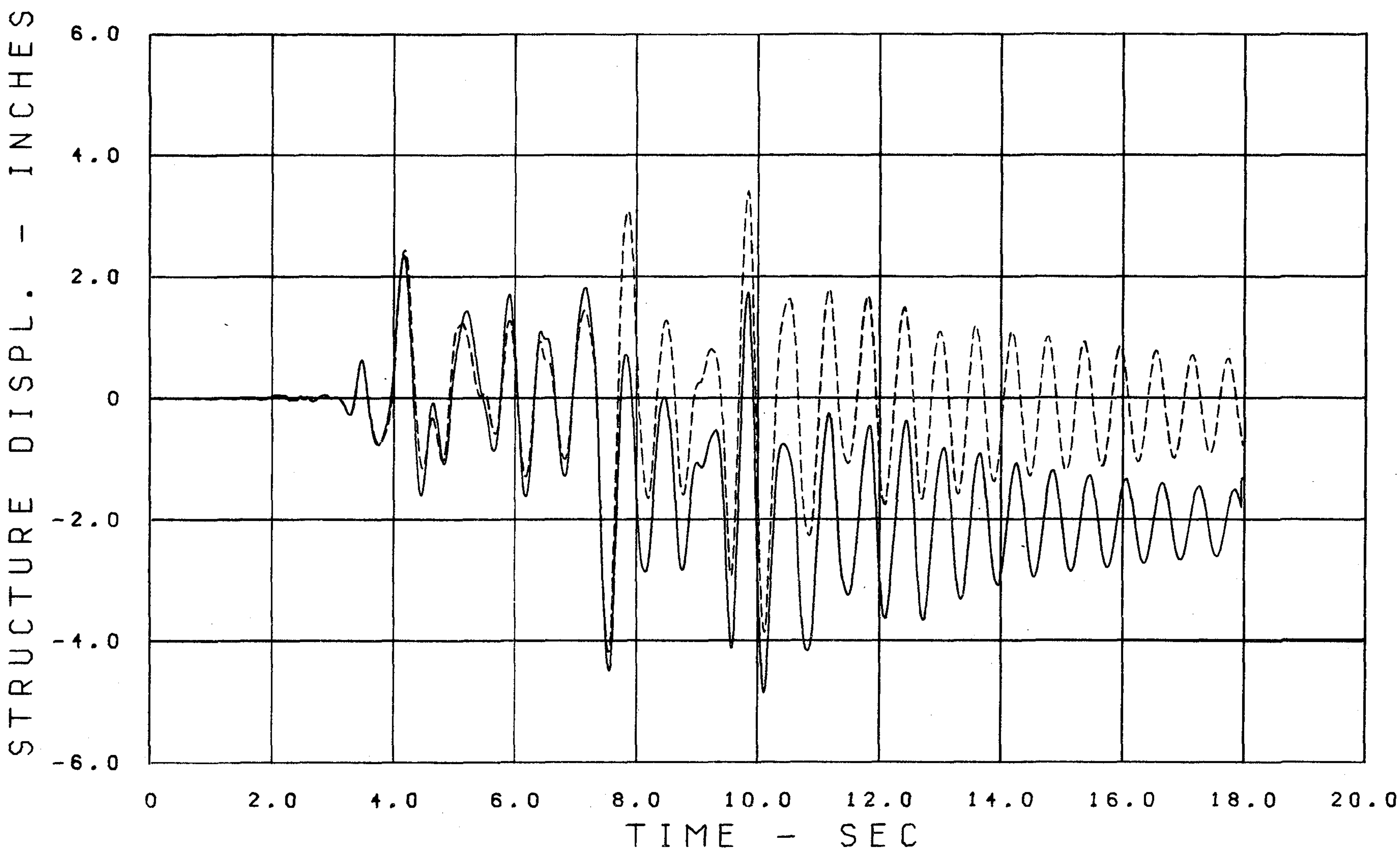

FIG. 3.5 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, PACOIMA

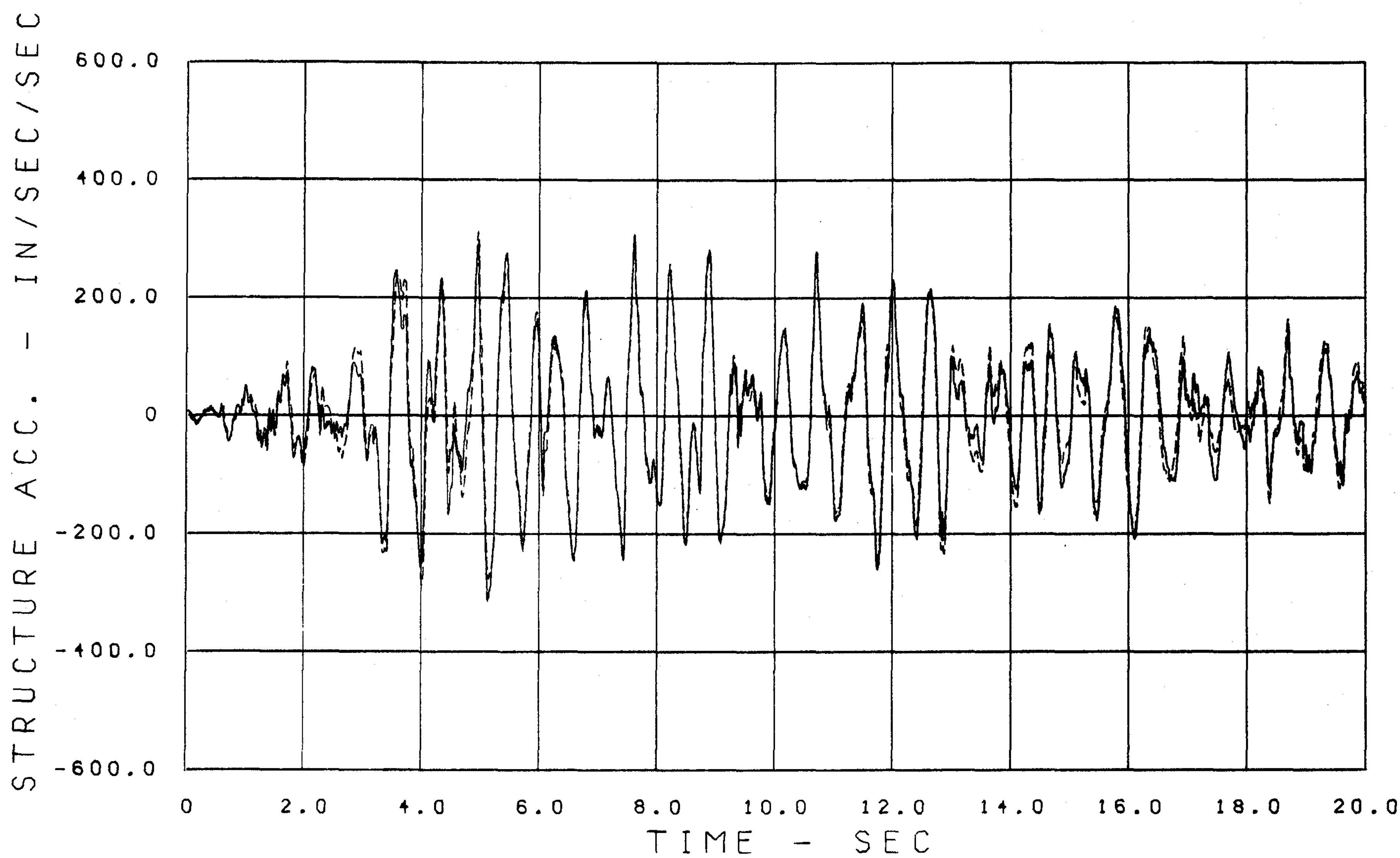

FIG. 3.6 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, TAFT

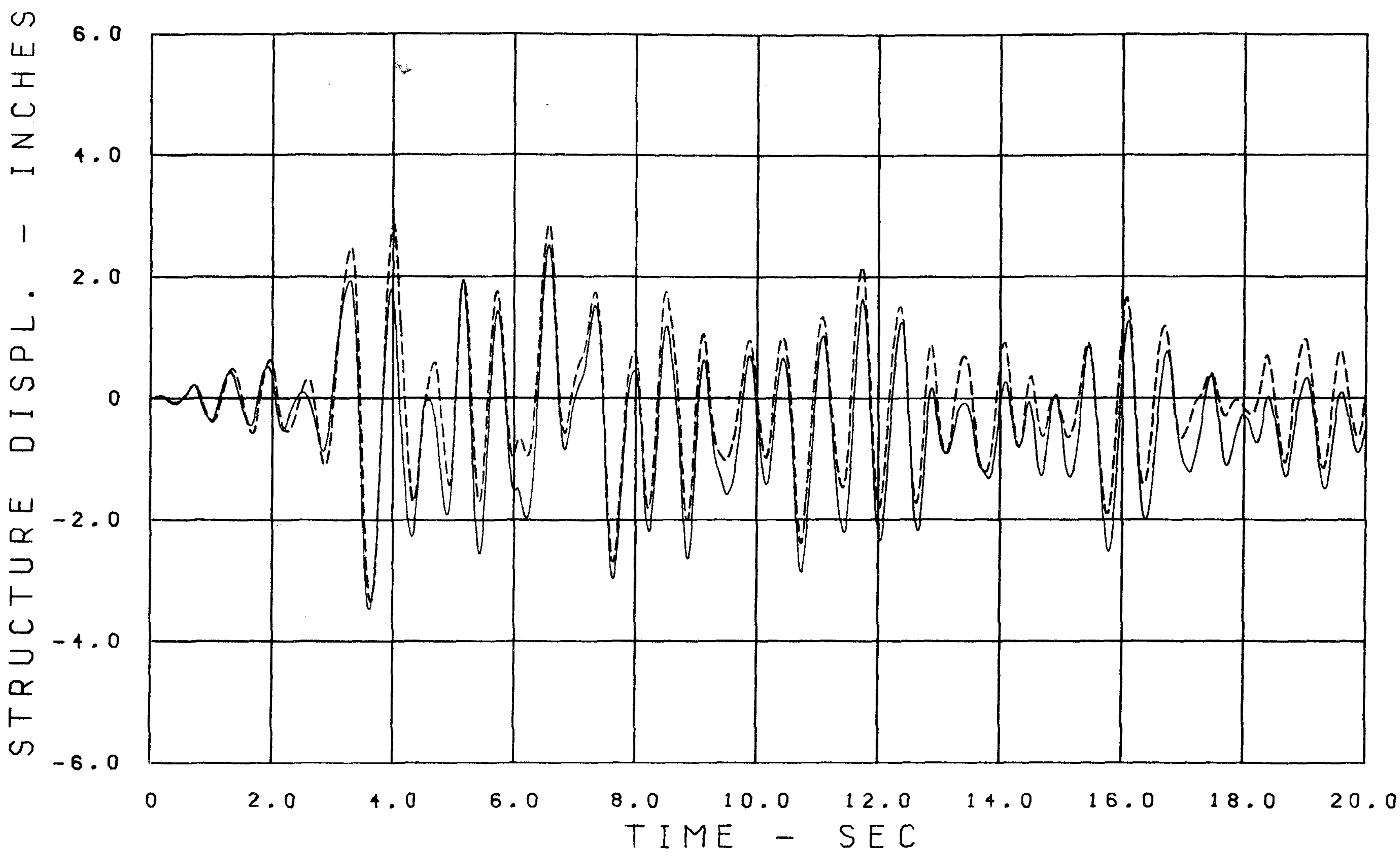

FIG. 3.7 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, TAFT

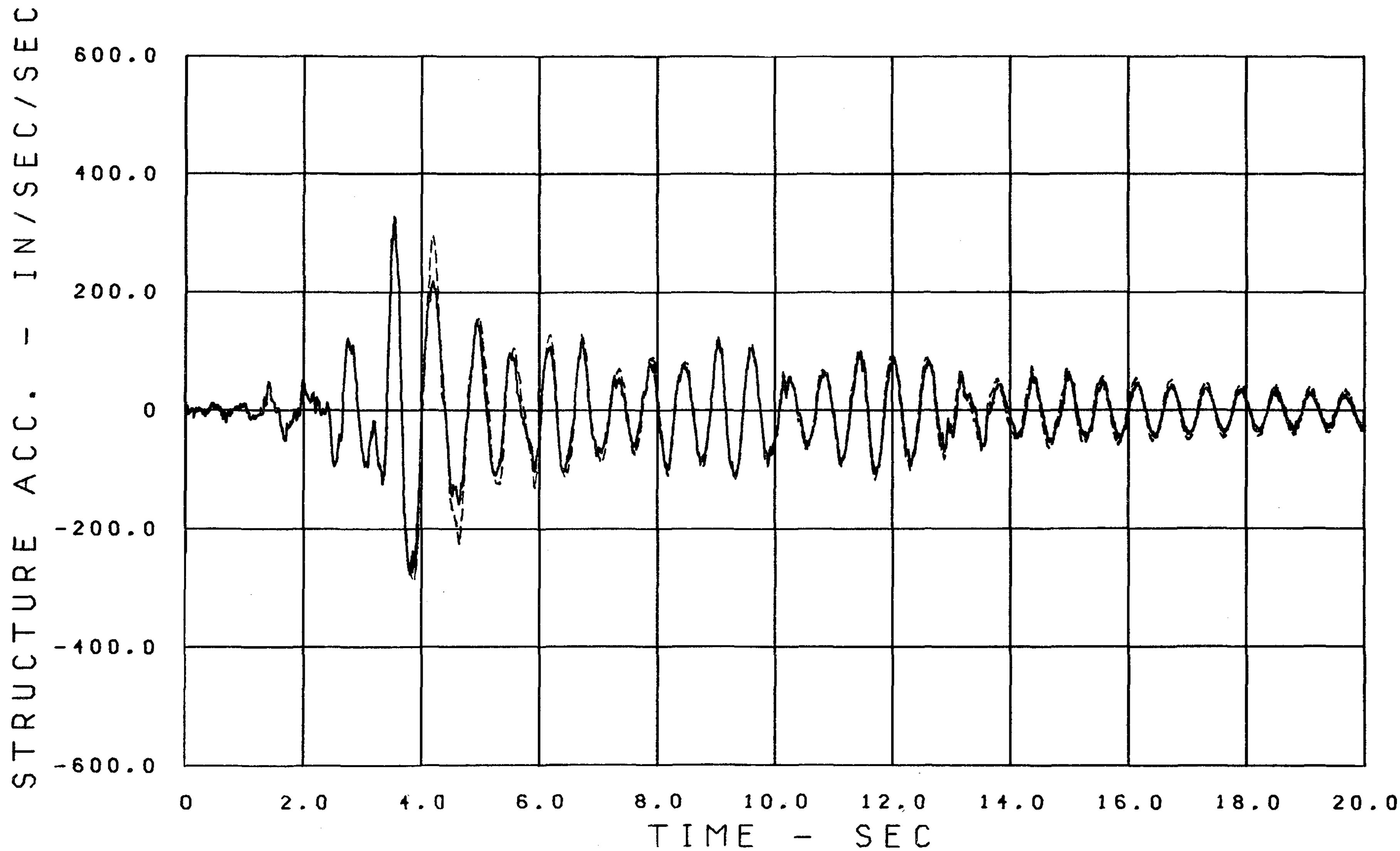

FIG. 3.8 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, PARKFIELD

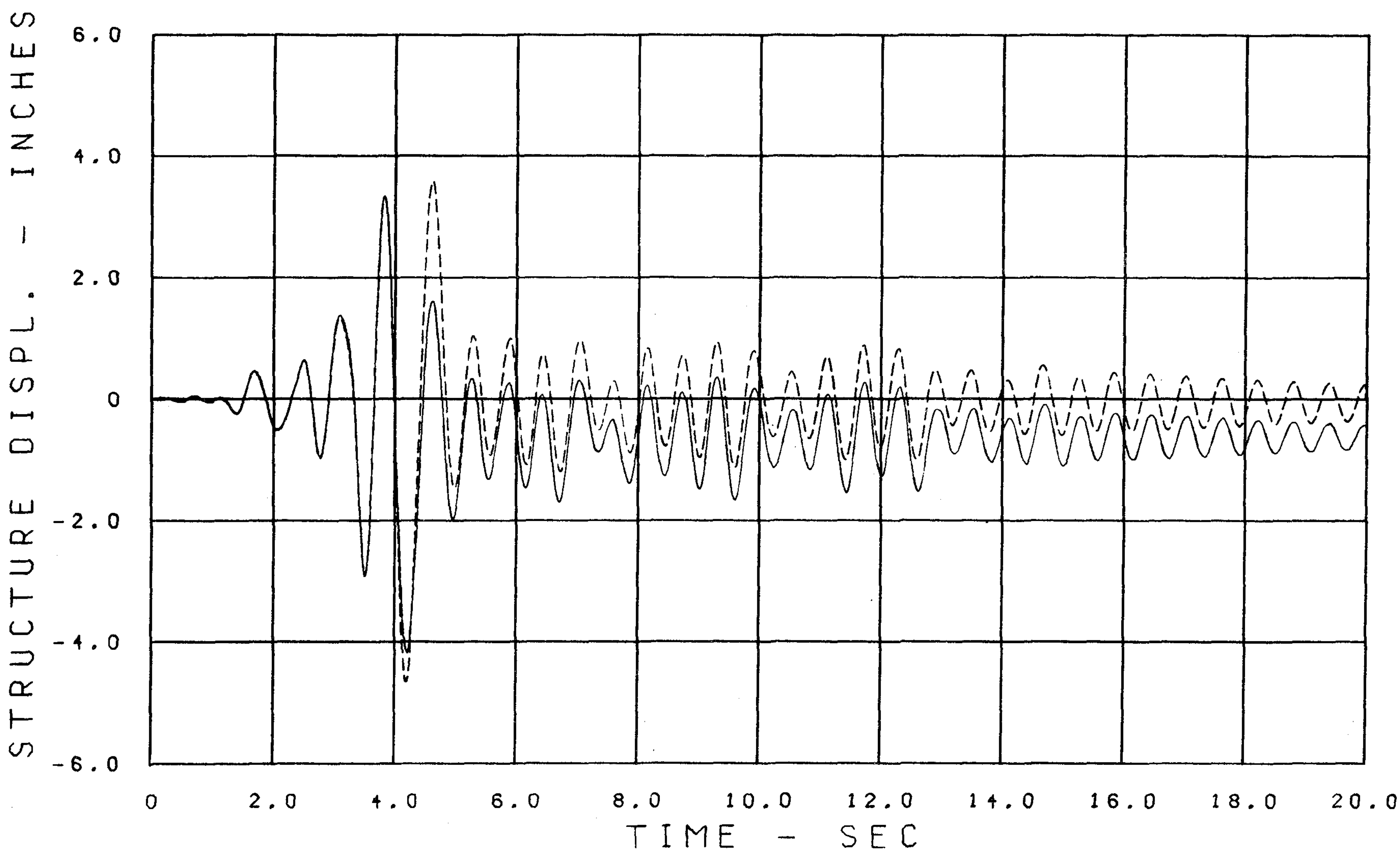

FIG. 3.9 WORK HARDENING MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, PARKFIELD

3 sec. Here, therefore, we only try to match response for the first 3 sec. By matching the maximum values of acceleration and displacement only, these first 3 sec. of both acceleration and displacement time histories were well matched. The parameters capable of achieving this match are:

$$k_o = 1850 \frac{\#}{\text{in.}}$$

$$\alpha = 0.1150$$

$$x_y = 1.90 \text{ in.}$$

To observe the performance of the above model in the first 3 sec. of the response the calculated and measured (dashed line and solid line, respectively) relative acceleration and displacement time histories to El Centro are compared in Figs. 3.10 and 3.11. Both response time histories are very well matched by the model for the first 3 sec., predicting the maximum response values exactly as shown in Table 3.1. From 3 sec. on the model is not able to match the measured time histories to an acceptable level of accuracy, which is immaterial.

It is of great interest to observe how well the elastic-plastic model can predict maximum response values for the other excitations. It is clear from Table 3.1, that the elastic-plastic model can predict the maximum absolute acceleration to all excitations very accurately and also the maximum relative displacements to El Centro and Taft, but the relative displacements to Pacoima and Parkfield are somewhat underestimated (26% and 13.5%, respectively). By looking at the measured response time histories we can explain this. For all excitations the maximum absolute acceleration occurs in the elastic-plastic phase of the response and the same is true for the maximum relative displacements

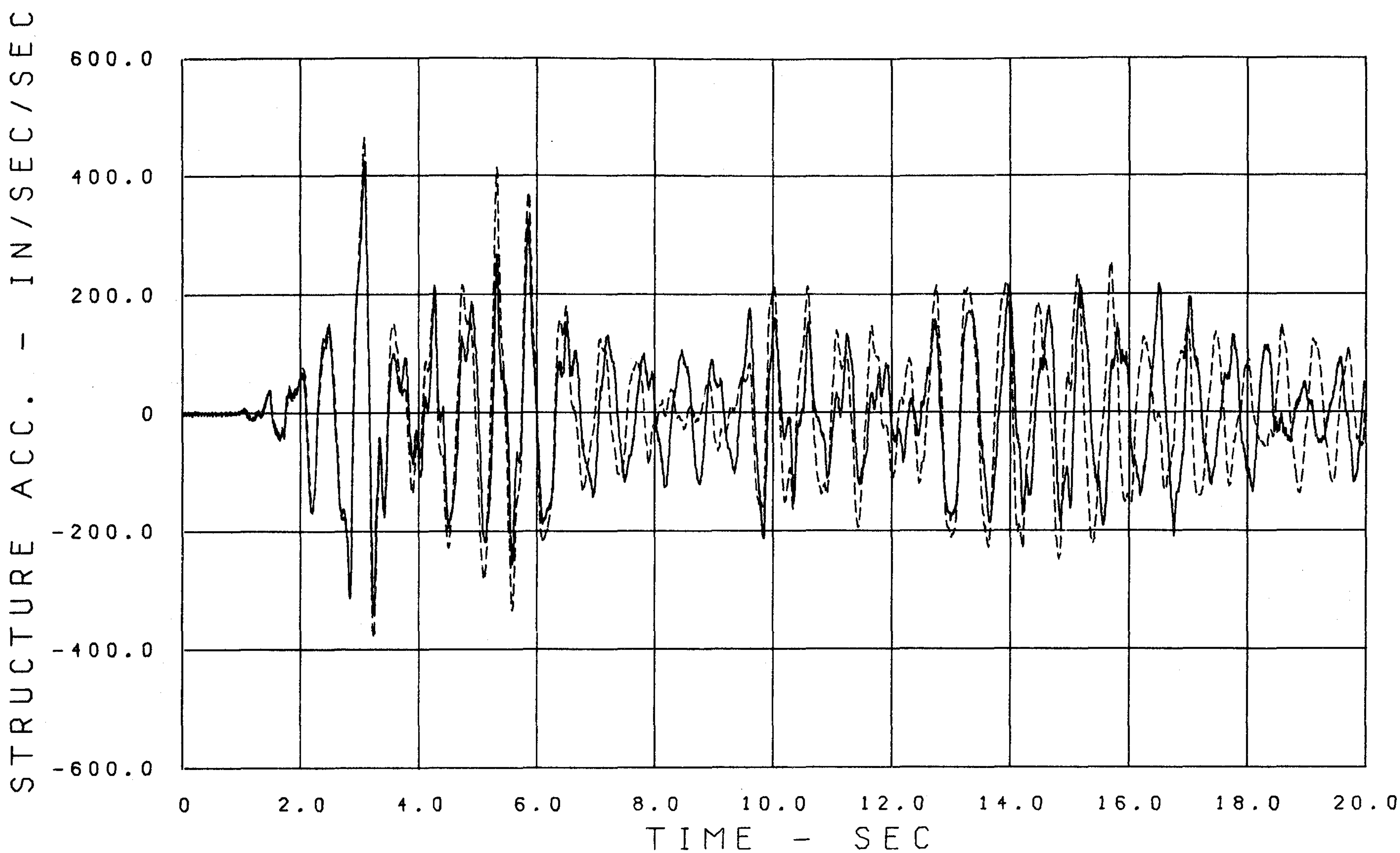

FIG. 3.10 ELASTIC-PLASTIC MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, EL CENTRO

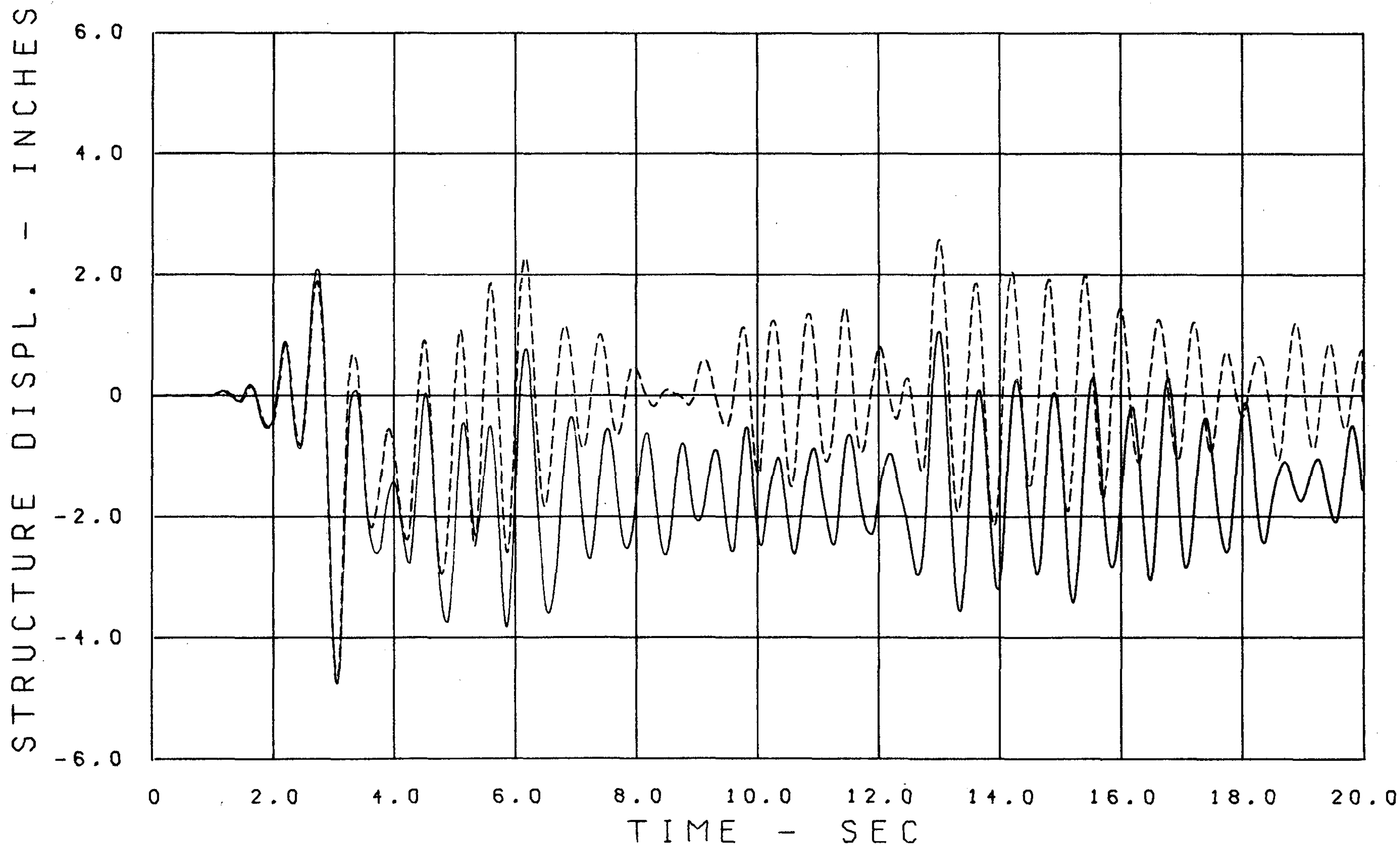

FIG. 3.11 ELASTIC-PLASTIC MODEL;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, EL CENTRO

to El Centro and Taft, while the maximum relative displacements to Pacoima and Parkfield occur in the work hardening phase. With these facts in mind the general applicability of the model to represent the elastic-plastic phase of the structural response is evident.

3.3 Comments

We could perhaps be critized for formulating these bilinear models somewhat crudely but they do reproduce time histories quite accurately in the domain appropriate to each. Without prejudging the linearization schemes that depend on such a model we cannot avoid pointing out two things. Those people that have constructed equivalent linear systems using a bilinear model have to our knowledge not recognized the two-phase nature of structural steel and when they choose a single bilinear model give no rational reasoning for the model they do assume.

4. METHODS OF EQUIVALENT LINEARIZATION FOR EARTHQUAKE EXCITATION

After appearances of a series of papers on methods of equivalent linearization by Caughey [2,7,8] of the nonlinear dynamic equation of motion, these methods have gained a wide application in engineering. Based on different modeling approximations numerous equivalent linearization models were formulated by Jennings [9] for harmonic excitation and by Lutes [10] for stationary random excitation. For harmonic and stationary random excitations, methods of equivalent linearization and their applications are summarized in review articles by Iwan [11] and Spanos [12].

Since the earthquake excitation is neither harmonic nor stationary random, very few methods of equivalent linearization have been proposed for systems subjected to that type of excitation [3,4].

In this chapter we present six methods of equivalent linearization for SDOF systems subjected to earthquake excitation. The first two of these methods do not depend on the restoring force-displacement relation of the system but are instead dependent on the time histories of the response of the system to the given excitation. The remaining four methods depend on the restoring force-displacement relation in addition to the maximum relative displacement of the system to the given excitation. For these methods the restoring force-displacement relation of our structure is approximated as bilinear hysteretic, as mentioned in Chapter 3.

4.1 Formulation

The equation of motion for a general nonlinear hysteretic SDOF system may be written as:

$$m_o\, \ddot{x}(t) + c_o\, \dot{x}(t) + h(x,t) = -m_o\, \ddot{x}_g(t);\ \dot{x}(0) = x(0) = 0 \tag{4.1}$$

where

m_o	is the nominal mass,
c_o	is the nominal viscous damping coefficient,
$h(x,t)$	is the restoring force function,
$x(t)$	is the displacement of the system relative to the ground,
$\ddot{x}_g(t)$	is the ground acceleration.

A system represented by Eq. (4.1) dissipates the supplied energy in two different ways; by viscous damping and by hysteretic behavior of the material.

The restoring force-displacement relation for a bilinear hysteretic system, shown in Fig. 3.1, is characterized by the nominal stiffness k_o, the nominal post yield stiffness $\alpha\, k_o$ and the yield displacement x_y. If the maximum relative displacement of the system to a given excitation is x_m, the displacement ductility ratio of the response is defined as $\mu = x_m/x_y$.

In the methods of equivalent linearization, the nonlinear hysteretic system, Eq. (4.1), is replaced by an "equivalent" linear system. The peak earthquake response of the nonlinear system is then obtained by calculating the peak response of the linear system specified by it's equivalent linear parameters.

The equation of motion for the equivalent linear SDOF system may be written as:

$$m_e\, \ddot{x}(t) + c_e\, \dot{x}(t) + k_e\, x(t) = -\, m_e\, \ddot{x}_g(t);\ \dot{x}(0) = x(0) = 0 \tag{4.2}$$

where

m_e is the equivalent mass,

c_e is the equivalent viscous damping coefficient,

k_e is the equivalent stiffness.

An equivalent linear system dissipates the supplied energy only by viscous damping.

For all the methods of equivalent linearization presented in this chapter we take

$$m_e = m_o. \tag{4.3}$$

In determining the values of c_e and k_e, Eq. (4.2) is made equivalent in some sense to Eq. (4.1), each method using a different criterion.

4.2 Methods of Equivalent Linearization

4.2.1 Methods Independent of Restoring Force-Displacement Relation

For these methods it is assumed that the response time histories for the full duration of a given earthquake excitation are known.

Method 1: System Identification (SI)

The method of System Identification has been extensively used by McNiven and his coworkers [1,13,14,15] to identify the system parameters for linear as well as nonlinear systems under earthquake excitation. Recently Beck and Jennings [16] used this method to identify linear models from earthquake records.

The method has been very well documented in Reference [13]. The criterion function used in this investigation is an integral squared error function that includes error in acceleration and can be written as:

$$J(\overline{\beta}, T_d) = \int_0^{T_d} [\ddot{x}(\overline{\beta}, t) - \ddot{y}(t)]^2 \, dt \tag{4.4}$$

where

$\overline{\beta}$ is a vector of the parameters c_e and k_e,

T_d is the full duration of the excitation or any portion of it,

$\ddot{x}(\overline{\beta},t)$ is the relative acceleration of the equivalent linear model using parameters $\overline{\beta}$ and excitation $\ddot{x}_g(t)$,

$\ddot{y}(t)$ is the relative acceleration of the nonlinear hysteretic system when it is subjected to the same excitation.

To establish the values of the equivalent linear parameters the criterion function is minimized using modified Gauss-Newton algorithm.

Method 2: Gülkan-Sozen Procedure (GSP)

The method was proposed by Gülkan and Sozen [17] for reinforced concrete structures subjected to earthquake excitation, but is applicable to all types of structures for which the response time histories are known.

In this method the equivalent stiffness is defined as

$$k_e = \frac{m_o|\ddot{y}+\ddot{x}_g|_{max}}{|y|_{max}} \tag{4.5}$$

where $\ddot{y}(t)$ and $y(t)$ are the measured response quantities of the structure when it is subjected to the excitation $\ddot{x}_g(t)$. It may be noted that this definition of equivalent stiffness actually defines equivalent secant stiffness, where secant stiffness is defined as the slope of the line from the origin of the restoring force-displacement diagram to that point on the primary curve where the displacement is $|y|_{max}$, if $|\ddot{y} + \ddot{x}_g|_{max}$ and $|y|_{max}$ occur at the same time. For a nonlinear hysteretic structure, in general, they do not occur simultaneously.

The equivalent viscous damping coefficient is calculated by assuming that all the energy supplied to the structure over the full duration, T_d, of the excitation is dissipated by an equivalent viscous dashpot. Thus,

$$c_e = \frac{-m_o \int_0^{T_d} \ddot{x}_g \, \dot{y} \, dt}{\int_0^{T_d} \dot{y}^2 \, dt} \tag{4.6}$$

where $\dot{y}(t)$ is the derived relative velocity of the structure when it is subjected to the excitation $\ddot{x}_g(t)$.

4.2.2 Methods Dependent on Restoring Force-Displacement Relation

For these methods it is assumed that a bilinear hysteretic model is known, i.e., the parameters k_o, α and x_y are known. Furthermore, it is assumed that the maximum relative displacement, x_{max}, of the model to a given earthquake excitation is known, i.e., μ is known.

Method 3: Modified Dynamic Stiffness (MDS)

The method of Dynamic Stiffness or Harmonic Equivalent Linearization was modified by Tansirikongkol and Pecknold [4] for earthquake excitation.

Using the method of Dynamic Stiffness for a bilinear hysteretic system under harmonic excitation of amplitude A and circular frequency ω_e, Caughey [7] has derived the following expressions:

$$k_e = m_o \, \omega_e^2 = C(\mu), \tag{4.7a}$$

$$c_e = c_o - \frac{S(\mu)}{\omega_e} \tag{4.7b}$$

where

$$C(\mu) = \begin{cases} k_o; & \mu \leq 1 \\ \frac{k_o}{\pi}\,[(1-\alpha)\theta + \alpha\pi - (\frac{1-\alpha}{2})\,\sin\theta]; & \mu > 1 \end{cases} \qquad (4.8a)$$

$$S(\mu) = \begin{cases} 0; & \mu \leq 1 \\ -\frac{(1-\alpha)}{\pi}\,k_o\,\sin^2\theta; & \mu > 1 \end{cases} \qquad (4.8b)$$

$$\theta = \begin{cases} \pi; & \mu \leq 1 \\ \cos^{-1}\,(\frac{\mu-2}{\mu}); & \mu > 1 \end{cases} \qquad (4.8c)$$

In Eq. (4.8) μ is the displacement ductility ratio of the response defined as $\mu = A/x_y$.

For earthquake excitation of total duration T_d, Eq. (4.7) was modified to give

$$k_e = C(\mu_{AF}), \qquad (4.9a)$$

$$c_e = c_o - \frac{S(\mu_{AD})}{\sqrt{2}\,\omega_e} \qquad (4.9b)$$

where

$$\mu_{AF} = \frac{\mu_Z}{\alpha^{0.2}[\ell n(f_oT_d)]^{1/2}}\,, \qquad (4.10a)$$

$$\mu_{AD} = \frac{\mu_Z}{[\ell n(f_oT_d)]^{1/2}}\,, \qquad (4.10b)$$

$$\mu_Z = \frac{x_{max}}{x_y} \qquad (4.10c)$$

and $f_o = \frac{\omega_o}{2\pi}$, where $\omega_o{}^2 = \frac{k_o}{m_o}$.

Method 4: Average Period and Damping (APD)

For a hysteretic system under harmonic excitation of amplitude A and circular frequency ω_e, in the Geometric Stiffness method (GS), the equivalent stiffness, k_e', is taken as the secant stiffness while energy balance per cycle is used to evaluate the equivalent viscous damping [3,9].

Thus, for a bilinear hysteretic system equivalent stiffness is given by

$$k_e'(A) = \begin{cases} k_o; & A \le x_y \\ k_o[\frac{(1-\alpha)}{A} x_y + \alpha]; & A > x_y \end{cases} \tag{4.11}$$

Since

$$m_e = m_o = \frac{k_o}{\omega_o^2} = \frac{k_e'}{\omega_e^2} \tag{4.12}$$

Eq. (4.11) can be written in terms of periods as

$$\frac{T_e'(A)}{T_o} = \begin{cases} 1 ; & A \le x_y \\ [\frac{(1-\alpha)}{A} x_y + \alpha]^{-1/2}; & A > x_y \end{cases} \tag{4.13}$$

where $T_e'(A) = 2\pi\sqrt{\frac{m_o}{k_e'(A)}}$ and $T_o = 2\pi\sqrt{\frac{m_o}{k_o}}$.

From Eq. (4.7b), after some manipulation, the equivalent viscous damping ratio, $\xi_e'(A)$, can be written as:

$$\xi_e'(A) = \begin{cases} \xi_o \frac{T_e'(A)}{T_o} ; & A \le x_y \\ \xi_o \frac{T_e'(A)}{T_o} + \frac{2}{\pi} (1-\alpha) \frac{(A-x_y)}{A^2} x_y \left(\frac{T_e'(A)}{T_o}\right)^2 ; & A > x_y \end{cases} \tag{4.14}$$

Newmark and Rosenblueth [5] extended the GS method to earthquake excitation by defining the equivalent linear system to be an average of all the linear systems corresponding to amplitudes less than or equal to

$x_m = x_{max}$. For a bilinear hysteretic system the average period and damping are given by [3]

$$\frac{T_e}{T_o} = \frac{1}{x_m} \int_0^{x_m} \frac{T_e'(A)}{T_o} \, dA$$

$$= \frac{1}{\mu} \left[1 + \frac{\sqrt{\alpha\mu^2 + (1-\alpha)\mu} - 1}{\alpha} - \frac{(1-\alpha)}{2\alpha^{3/2}} \ln \zeta ; \; \mu > 1 \right. \tag{4.15a}$$

where

$$\zeta = \frac{2\sqrt{\alpha[\alpha\mu^2 + (1-\alpha)\mu]} + 2\alpha\mu + (1-\alpha)}{\alpha + 2\sqrt{\alpha} + 1} \tag{4.15b}$$

and

$$\xi_e = \frac{1}{x_m} \int_0^{x_m} \xi_e'(A) \, dA$$

$$= \xi_o \frac{T_e}{T_o} + \frac{2}{\pi} \frac{(1-\alpha)}{\mu} \left[\frac{1}{\alpha} \ln(1-\alpha + \alpha\mu) + \right.$$

$$\left. \frac{1}{(1-\alpha)} \ln \left(\frac{(1-\alpha)}{\mu} + \alpha \right) \right]; \; \mu > 1 \tag{4.16}$$

Thus,

$$k_e = \frac{k_o}{\left(\frac{T_e}{T_o}\right)^2} \tag{4.17a}$$

$$c_e = 2 \, m_o \, \omega_e \, \xi_e \tag{4.17b}$$

<u>Method 5: Average Stiffness and Energy (ASE)</u>.

For a bilinear hysteretic system under harmonic excitation of amplitude A, the secant stiffness, $k_e'(A)$, is given by Eq. (4.11), while the energy dissipated per cycle, $\Delta W_o'(A)$, is given by

$$\Delta W_o'(A) = V_o'(A) + H_o'(A) \tag{4.18a}$$

where

$$V_o'(A) = \begin{cases} 2\pi\ \xi_o A^2 k_o\ ; & A \leq x_y \\ 2\pi\ \xi_o A^2 k_o\ [\frac{(1-\alpha)}{A}\ x_y + \alpha]\ ; & A > x_y \end{cases} \tag{4.18b}$$

$$H_o'(A) = \begin{cases} 0\ ; & A \leq x_y \\ 4\ k_o \frac{(1-\alpha)}{x_y}\ (A-x_y); & A > x_y \end{cases} \tag{4.18c}$$

Similarly for a linear system under harmonic excitation of amplitude A the energy dissipated per cycle, $\Delta W_e'(A)$, is given by

$$\Delta\ W_e'(A) = V_e'(A) \tag{4.19a}$$

where

$$V_e'(A) = 2\pi\ \xi_e\ A^2 k_e. \tag{4.19b}$$

As in the APD method, in the ASE method the equivalent linear system is defined in terms of the average values of the fundamental parameters [9]. In this method the fundamental system parameters are taken as the stiffness and the energy dissipated per cycle. Thus, the equivalent stiffness, k_e, is given by

$$k_e = \frac{1}{x_m} \int_0^{x_m} k_e'(A)\ dA,$$

thus,

$$k_e = \begin{cases} k_o; & \mu \leq 1 \\ k_o[\frac{(1-\alpha)}{\mu}\ (1 + \ell n\ \mu) + \alpha];\ & \mu > 1 \end{cases} \tag{4.20}$$

By equating

$$\Delta\ W_o = \Delta\ W_e \tag{4.21a}$$

where

$$\Delta W_o = \frac{1}{x_m} \int_0^{x_m} \Delta W_o'(A)\, dA, \tag{4.21b}$$

$$\Delta W_e = \frac{1}{x_m} \int_0^{x_m} \Delta W_e'(A)\, dA, \tag{4.21c}$$

the equivalent viscous damping ratio is given by

$$\xi_e = \frac{H_o(\mu) + V_o(\mu)}{V_e(\mu)} \tag{4.22a}$$

where

$$H_o(\mu) = \begin{cases} 0; & \mu \leq 1 \\ 2\ k_o(1-\alpha)\ \dfrac{(\mu-1)^2}{\mu}\ ; & \mu > 1 \end{cases} \tag{4.22b}$$

$$V_o(\mu) = \begin{cases} \dfrac{2}{3}\pi\ \xi_o\ k_o\ \mu^2; & \mu \leq 1 \\ \dfrac{\pi\ \xi_o\ k_o}{\mu}\ [(1-\alpha)\ (\mu^2 - \dfrac{1}{3}) + \dfrac{2}{3}\,\alpha\ \mu^3]; & \mu > 1 \end{cases} \tag{4.22c}$$

$$V_e(\mu) = \frac{2}{3}\,\pi\ k_e\ \mu^2 \tag{4.22d}$$

Method 6: Stationary Random Equivalent Linearization (SREL)

In this method the earthquake excitation is assumed to be stationary random Gaussian, and the response is assumed to be stationary, ergodic, Gaussian and narrow band process. It should be noted that these assumptions for the response will only be satisfied by a weakly nonlinear system.

For a bilinear hysteretic system, Caughey [2], has derived the following expressions for the equivalent linear parameters based on the above assumptions,

$$\frac{k_e}{k_o} = 1 - \frac{8(1-\alpha)}{\pi} \int_1^\infty \sqrt{A-1}\, [\frac{1}{2A\sigma_\mu^2} + \frac{1}{A^3}] \exp (\frac{-A^2}{2\sigma_\mu{}^2})\, dA \tag{4.23}$$

and

$$\xi_e = \xi_o \sqrt{\frac{k_o}{k_e}} + \frac{k_o}{k_e} \frac{(1-\alpha)}{\sqrt{2\pi}\, \sigma_\mu} \text{ erfc } (\frac{1}{\sqrt{2}\, \sigma_\mu}) \tag{4.24}$$

and $\sigma_\mu = \frac{\sigma_x}{x_y}$, where σ_x is the root mean square value of the response, and erfc is the complimentary error function.

5. CONSTRUCTION AND ASSESSMENT OF THE EQUIVALENT LINEAR MATHEMATICAL MODELS

Using the structural response to El Centro discussed in Chapter 2, we will construct in this chapter various equivalent linear models based on the methods of equivalent linearization mentioned in Chapter 4. For the methods requiring hysteretic model, bilinear models of Chapter 3 are used. It may be noted that since two bilinear hysteretic models were constructed, two sets of equivalent linear parameters will be obtained for these methods.

The assessment of the equivalent linear models is made by comparing their response predictions with the measured response both for El Centro and the other three excitations of Chapter 2.

It should be noted that the objective of all the equivalent linear methods except the method of System Identification is to predict the maximum relative displacement and not the time histories of the response.

5.1 Construction of the Equivalent Linear Models

5.1.1 System Identification (SI)

To accommodate the iterative schemes and to solve the equations involved in the System Identification procedure, a computer program was developed, the details of which are given in Appendix A. Before subjecting the identification program to actual test data it was tested with simulated data to ensure that the algorithms it contains are correct and also to get a feel for the process.

As mentioned in Chapter 4, System Identification can be used to establish the parameters of an equivalent linear model by matching a selected response time history either over the full duration of the excitation or any portion of it.

First we use the method over the full duration of the measured relative acceleration response. Using the computer program we obtain the following parameter set:

$$\left.\begin{aligned} c_e &= 23.60 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1566.5 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{matrix}\text{Parameters}\\ \text{for}\\ \text{ELMSI1}\end{matrix}$$

As the response time history match does not pay any special attention to the maximum response values, it can not be expected that the model, ELMSI1, constructed above will be able to predict the maximum response values very accurately. An attempt is therefore made to match the maximum relative displacement occuring at 3.06 sec. by only including that instant of time in the criterion functions (which now includes error in displacement instead of acceleration). Using various sets of initial values of the parameters the program always converged to the same final parameter set, including a negative value for the viscous damping coefficient. This indicates the fact that the El Centro excitation does not have enough input energy in the first 3.06 sec. to be able to excite a physically acceptable linear system to the relative displacement of 4.75 in. in the end of that duration. Although we are unsuccessful in matching the maximum displacement of the response we will try to match the maximum acceleration. Using reasonable guess of the initial values of the parameters and the computer program we obtain:

$$\left.\begin{aligned} c_e &= 32.68 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1344.3 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{matrix}\text{Parameters}\\ \text{for}\\ \text{ELMSI2}\end{matrix}$$

5.1.2 Gülkan - Sozen Procedure (GSP)

As mentioned in Section 4.2.1, the establishment of the equivalent linear parameters by this method requires the derivation of the relative velocity time history of the structure. Using a still unpublished computer program, DIPS, written by Marcial Blondet, a graduate student in SESM at the University of California, Berkeley, we obtain the relative velocity time history both by differentiating the measured displacement, Fig. 5.1, and by integrating the measured acceleration, Fig. 5.2. This is done to check the measured data and also to secure the correct selection of filters in the derivation of the relative velocity.

Looking at the time histories of absolute acceleration and relative displacement, respectively, we obtain:

$$|\ddot{y} + \ddot{x}_g|_{max} = 267.7 \ \frac{\text{in}}{\text{sec}^2} \quad \text{at } t = 2.95 \text{ sec.}$$

$$|y|_{max} = 4.75 \text{ in. at } t = 3.06 \text{ sec.}$$

i.e., the maximum response values occur almost simultaneously.

From the information derived above and with the use of Eqs. (4.5) and (4.6) we obtain:

$$\left.\begin{aligned} c_e &= 20.69 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 872.4 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{array}{c}\text{Parameters}\\ \text{for}\\ \text{ELMGSP}\end{array}$$

5.1.3 Modified Dynamic Stiffness (MDS)

Using the bilinear hysteretic models constructed in Chapter 3 and Eqs. (4.9) we obtain:

$$\left.\begin{aligned} c_e &= 24.01 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1305.5 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{array}{c}\text{Parameters}\\ \text{for}\\ \text{ELMMDS1}\end{array}$$

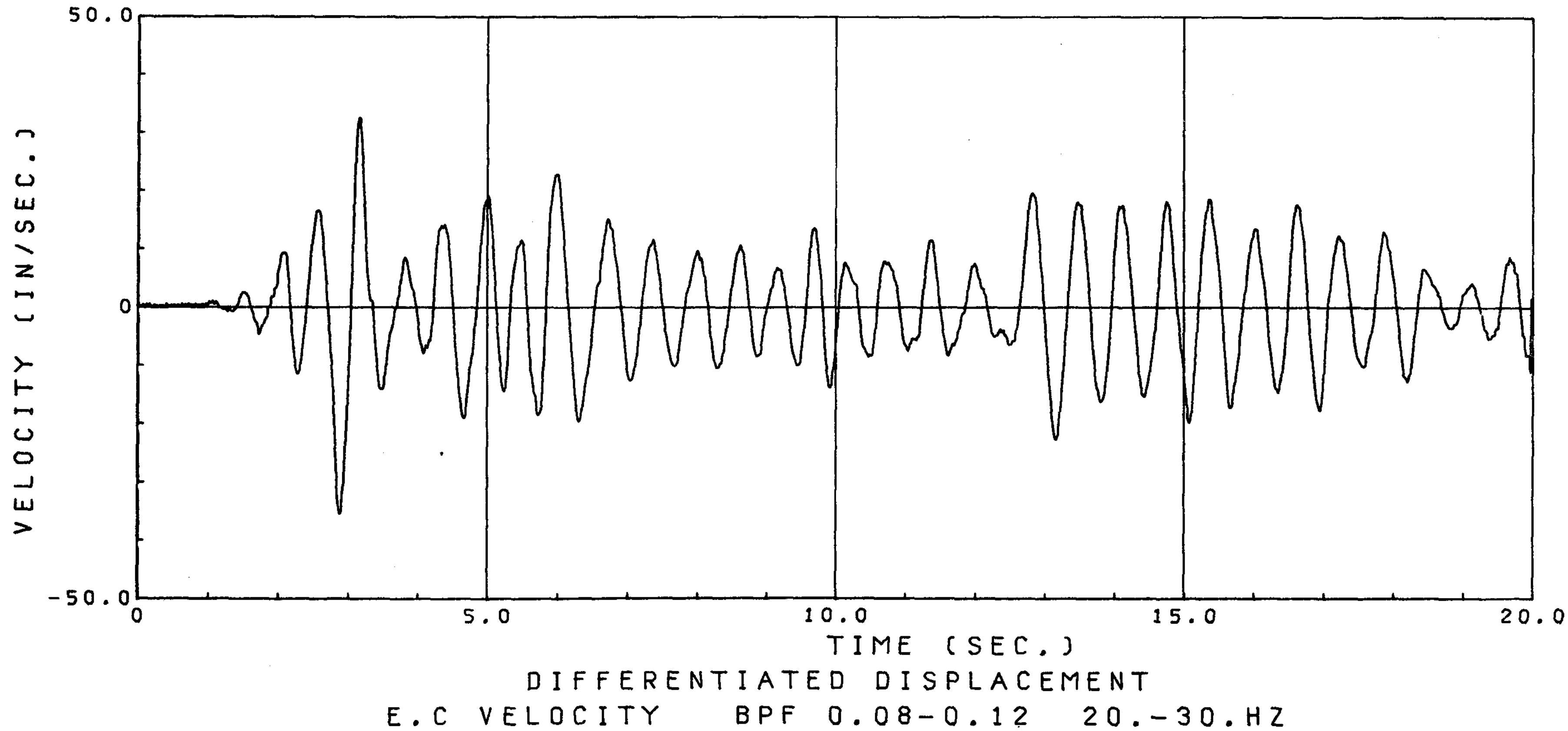

FIG. 5.1 RELATIVE VELOCITY RESPONSE TIME HISTORY FROM DIFFERENTIATED DISPLACEMENTS, EL CENTRO

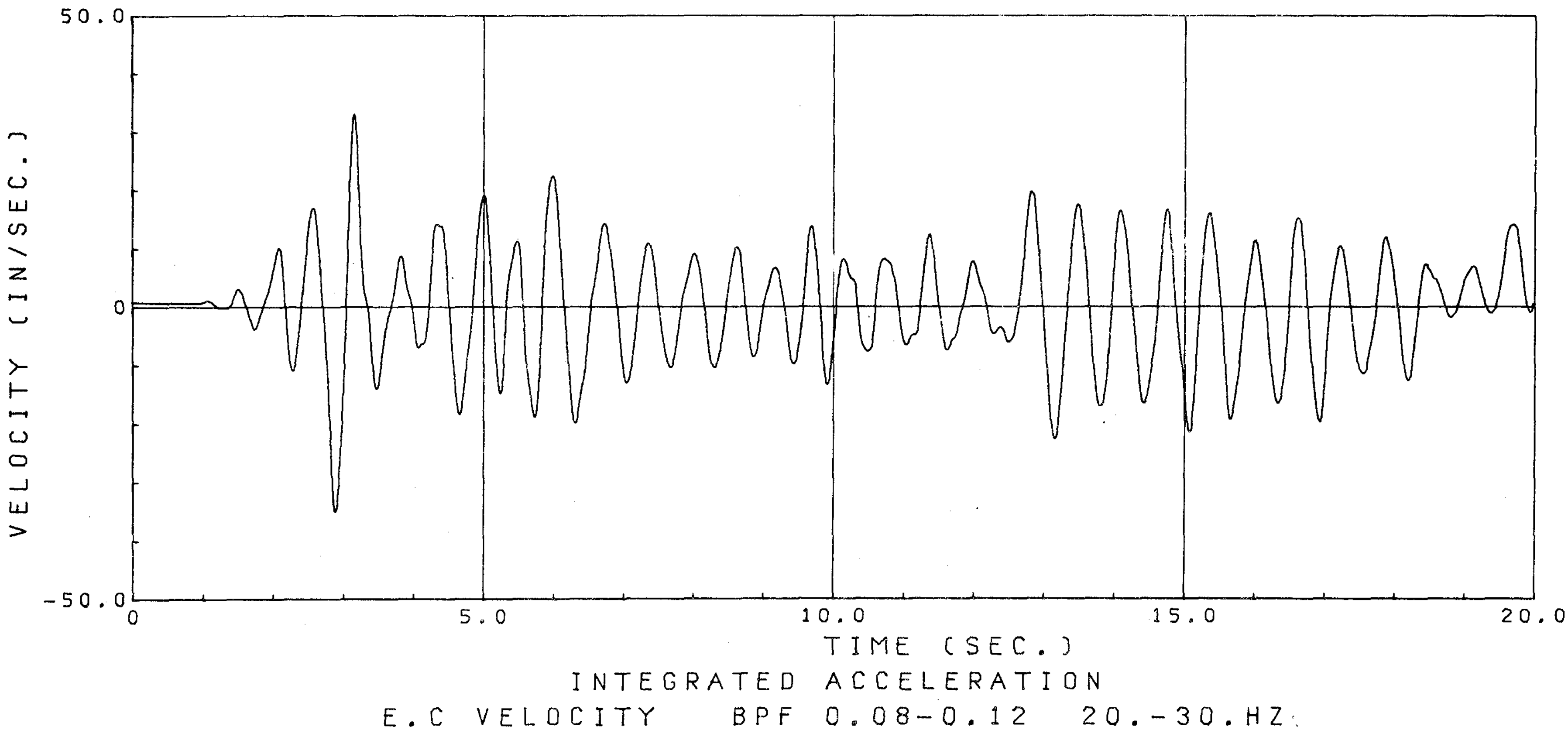

FIG. 5.2 RELATIVE VELOCITY RESPONSE TIME HISTORY FROM INTEGRATED ACCELERATIONS, EL CENTRO

and

$$\left.\begin{aligned} c_e &= 23.06 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1060.5 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \text{Parameters for ELMMDS2}$$

where ELMMDS1 and ELMMDS2 indicates an equivalent linear model constructed with Modified Dynamic Stiffness method using work hardening model and elastic-plastic model, respectively.

5.1.4 Average Period and Damping (APD)

Using the bilinear hysteretic models constructed in Chapter 3 and Eqs. (4.15) and (4.16) we obtain:

$$\left.\begin{aligned} c_e &= 25.82 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1404.5 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \text{Parameters for ELMAPD1}$$

and

$$\left.\begin{aligned} c_e &= 38.58 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1389.4 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \text{Parameters for ELMAPD2}$$

5.1.5 Average Stiffness and Energy (ASE)

Using the bilinear hysteretic models constructed in Chapter 3 and Eqs. (4.20) and (4.22) we obtain:

$$\left.\begin{aligned} c_e &= 32.53 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1440.4 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \text{Parameters for ELMASE1}$$

and

$$\left.\begin{aligned} c_e &= 48.77 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1467.7 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \text{Parameters for ELMASE2}$$

5.1.6 Stationary Random Equivalent Linearization (SREL)

Using the bilinear hysteretic models constructed in Chapter 3 and Eqs. (4.23) and (4.24) we obtain:

$$\left.\begin{aligned} c_e &= 21.24 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1545.1 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{array}{c}\text{Parameters}\\ \text{for}\\ \text{ELMSREL1}\end{array}$$

and

$$\left.\begin{aligned} c_e &= 15.23 \ \frac{\#\ \text{sec}}{\text{in}} \\ k_e &= 1746.0 \ \frac{\#}{\text{in}} \end{aligned}\right\} \quad \begin{array}{c}\text{Parameters}\\ \text{for}\\ \text{ELMSREL2}\end{array}$$

5.2 Assessment of the Equivalent Linear Models

All the equivalent linear models were constructed directly or indirectly from the measured nonlinear response of the structure to El Centro excitation. It is therefore of main interest to observe how accurately these models can predict the nonlinear response of the structure to that excitation. The models are also subjected to the Pacoima, Taft and Parkfield excitations and their response predictions compared with the measured nonlinear response of the structure to those same excitations to observe the general applicability of the models.

It should be noted that only the construction of ELMSI1 aimed at a model to predict the response time histories; the objective for the construction of all the other models was the ability to predict the maximum response values, especially the maximum relative displacement.

5.2.1 Prediction of the Response Time Histories

The construction of ELMSI1 is based on the full duration of the measured relative acceleration to El Centro and it is therefore of great interest to observe how well that model can predict the response time histories to that excitation. The calculated and measured (dashed line and solid line, respectively) relative acceleration and displacement time histories for El Centro are compared in Figs. 5.3 and 5.4. The relative acceleration time history is fairly well reproduced by the model and the relative displacement time history predicted is in phase with the measured one, although the yielding shift is of course not predicted. The greater contribution of the work hardening phase of the measured response in the equivalent linear parameters is evident in the first 4 sec. of the predicted response, leading to overestimation of the maximum absolute acceleration and underestimation of the maximum relative displacement.

It is also a matter of interest to examine whether an equivalent linear model constructed in this way using measured response to El Centro can predict the response time histories to other excitations as accurately as it does to El Centro. With the criterion for the construction of ELMSI1 in mind we would expect the same order of prediction accuracy, if the other excitation causes a similiar relative acceleration response of the structure, but using ELMSI1 to predict the response of the structure to an excitation causing very different relative acceleration response can not be expected to be that accurate. To observe the general applicability of ELMSI1 we therefore subject it to the remaining three excitations of Chapter 2 and compare the relative acceleration and displacement time histories predicted by the model with the measured nonlinear responses; Figs. 5.5 and 5.6 show the comparison for Pacoima excitation, Figs. 5.7 and 5.8 for Taft excitation and Figs. 5.9 and 5.10 for Parkfield excitation. Having just described the prediction characteristics of ELMSI1 it is not surprising that the response time

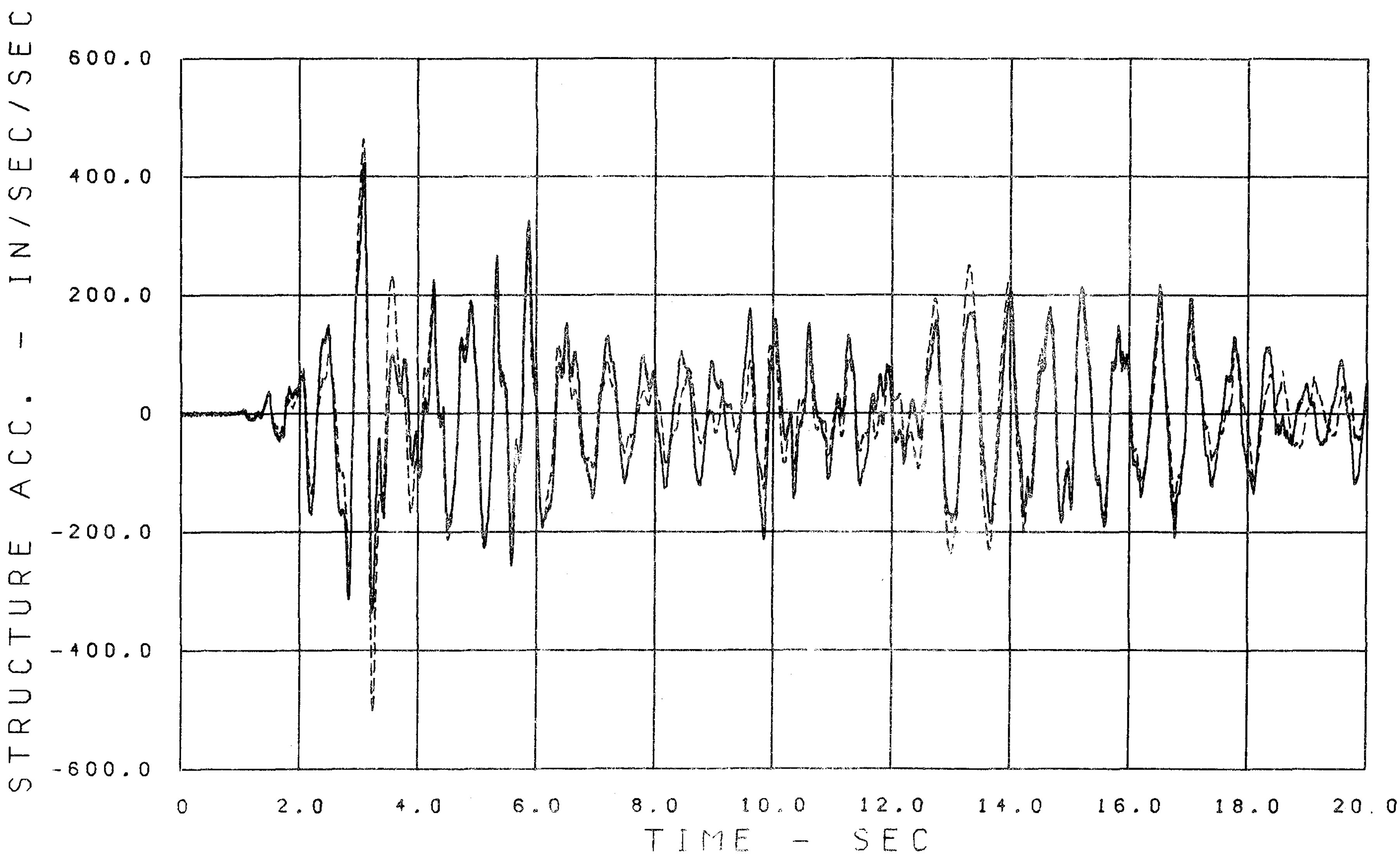

FIG. 5.3 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, EL CENTRO

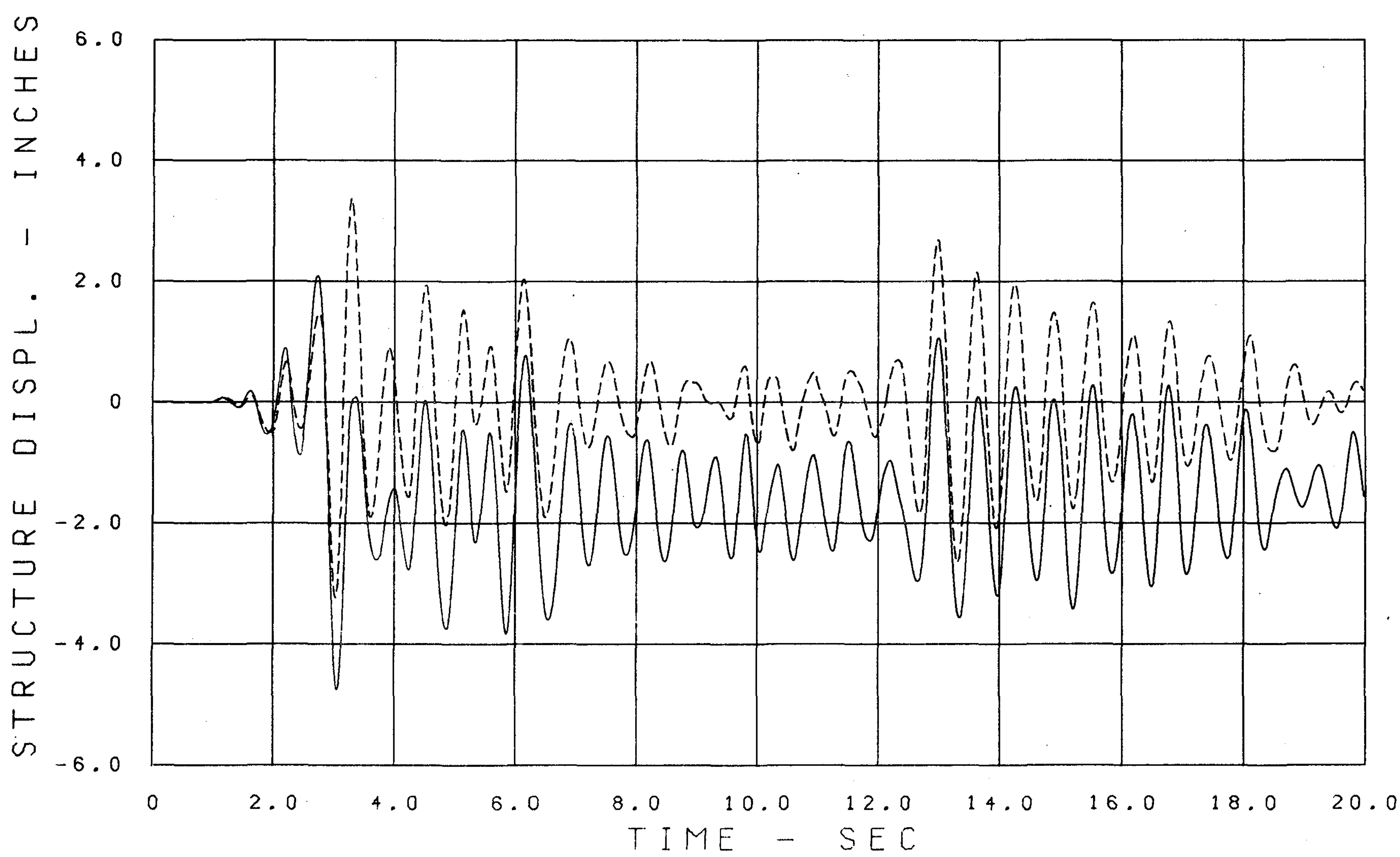

FIG. 5.4 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, EL CENTRO

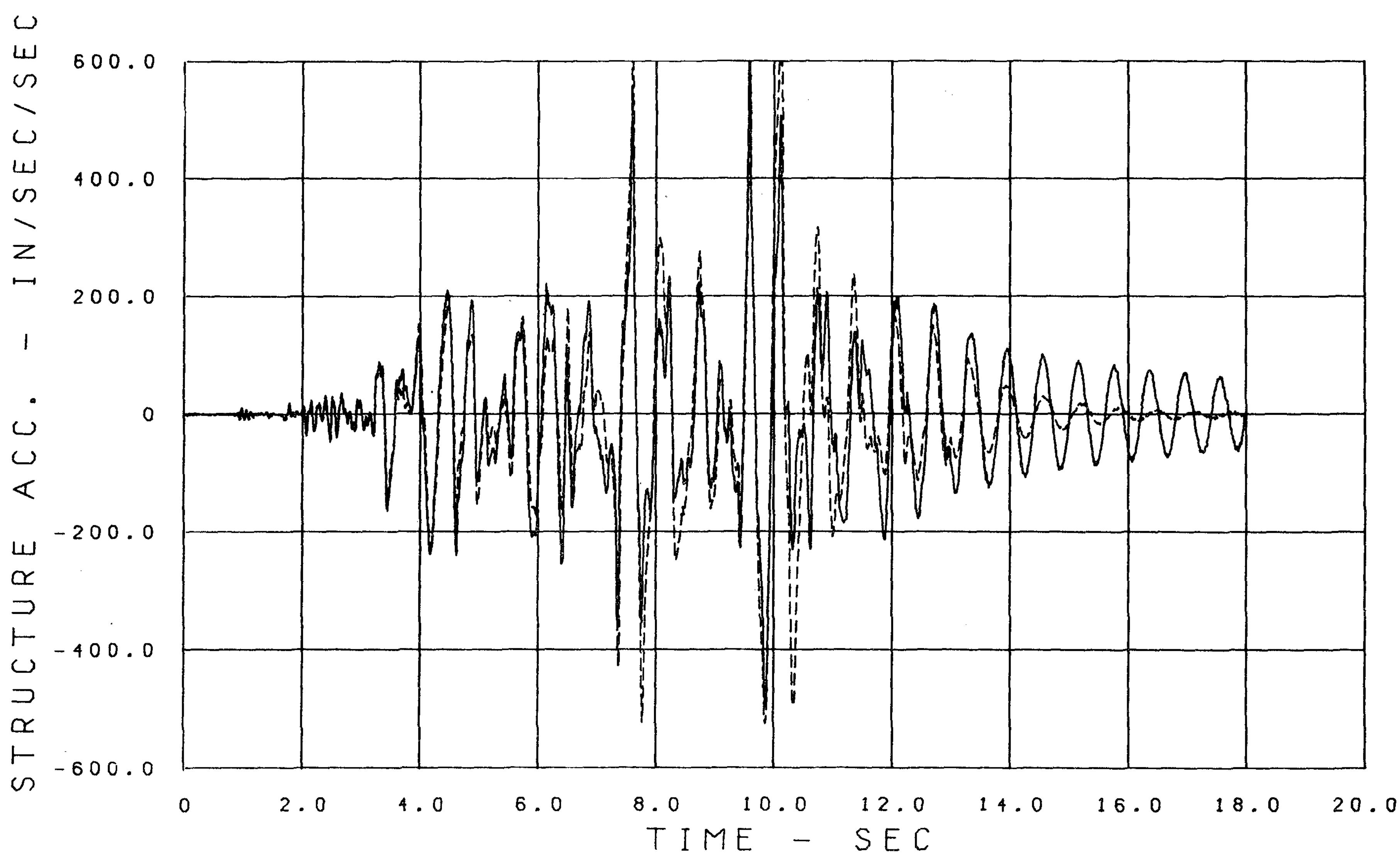

FIG. 5.5 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, PACOIMA

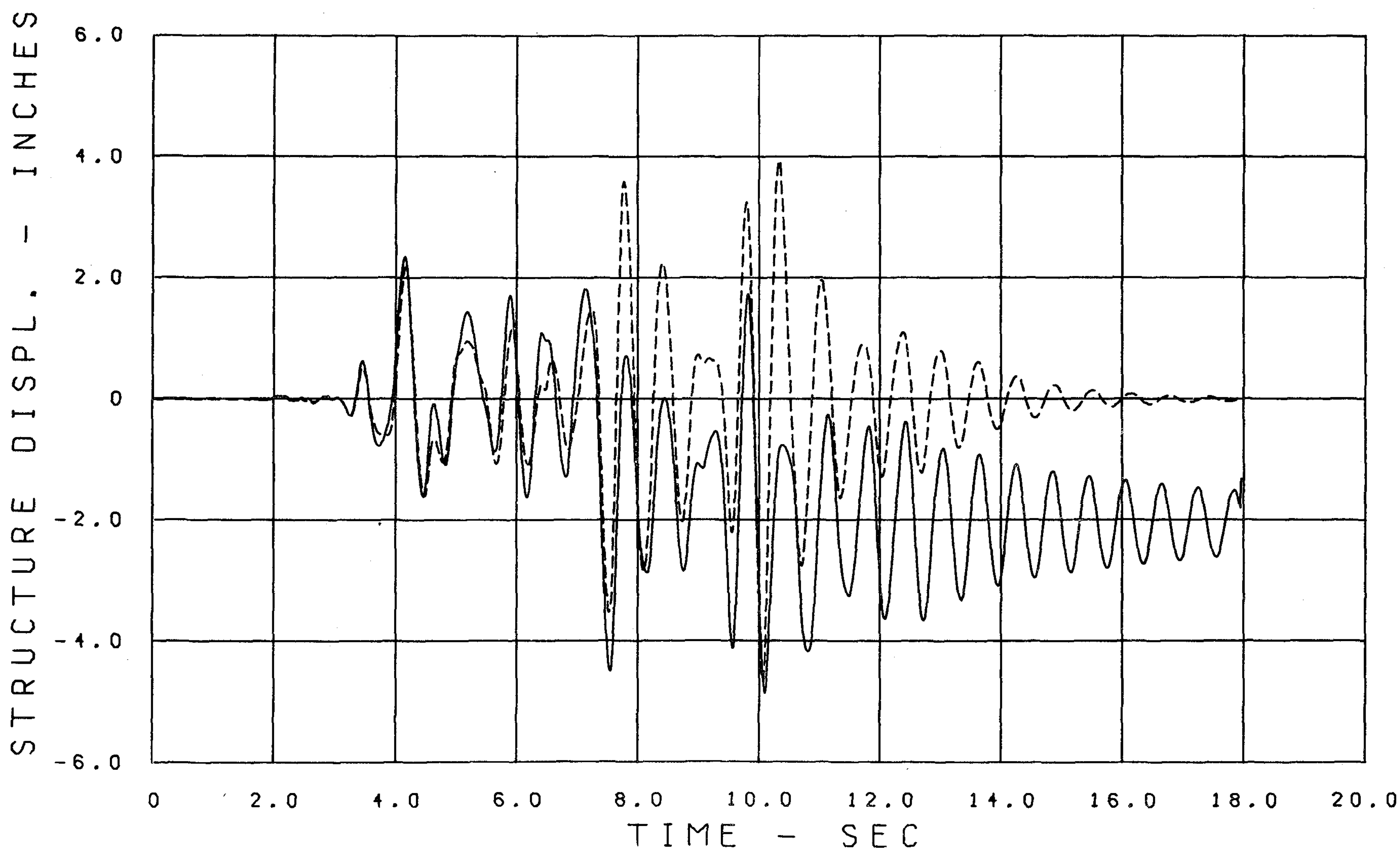

FIG. 5.6 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, PACOIMA

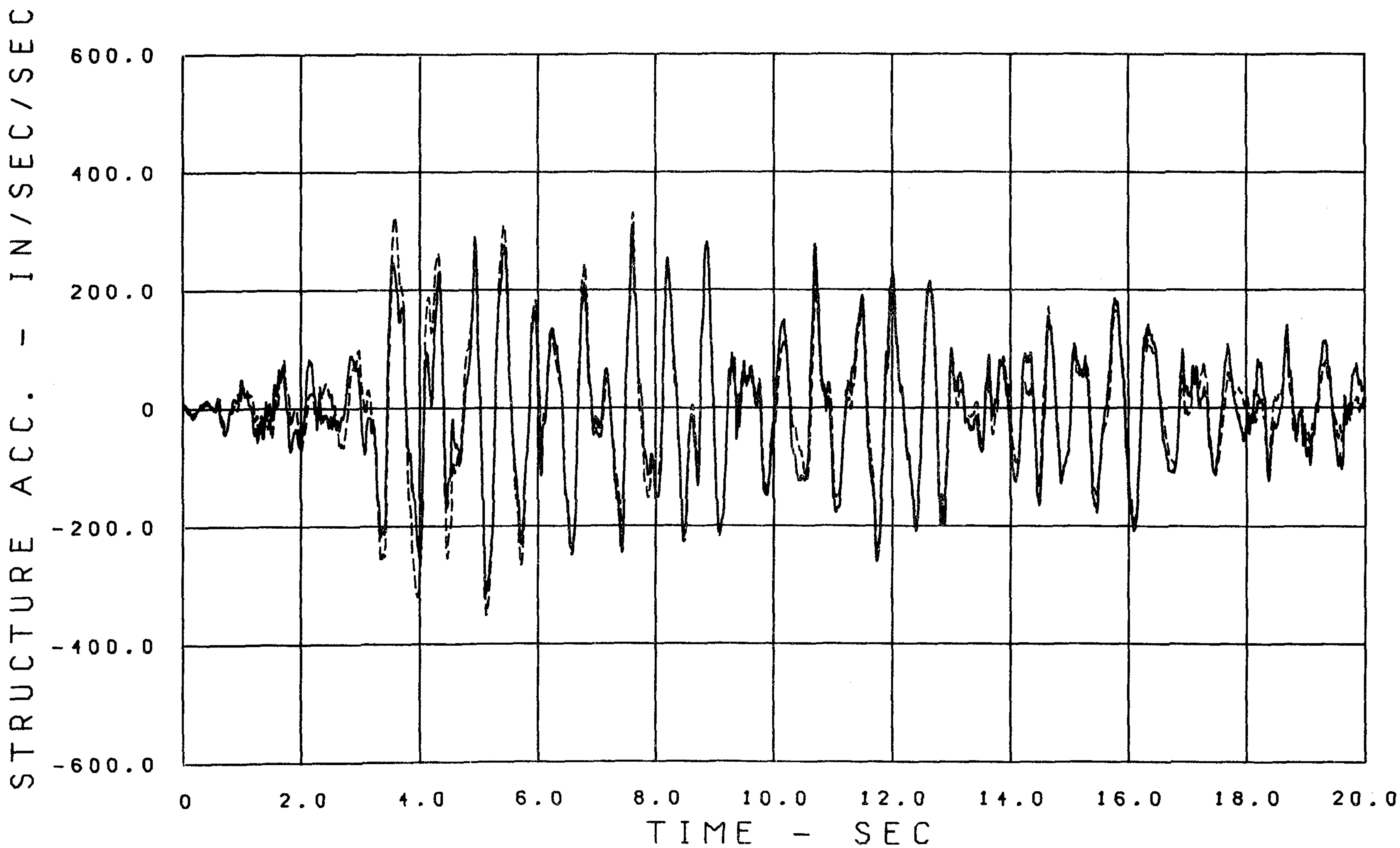

FIG. 5.7 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, TAFT

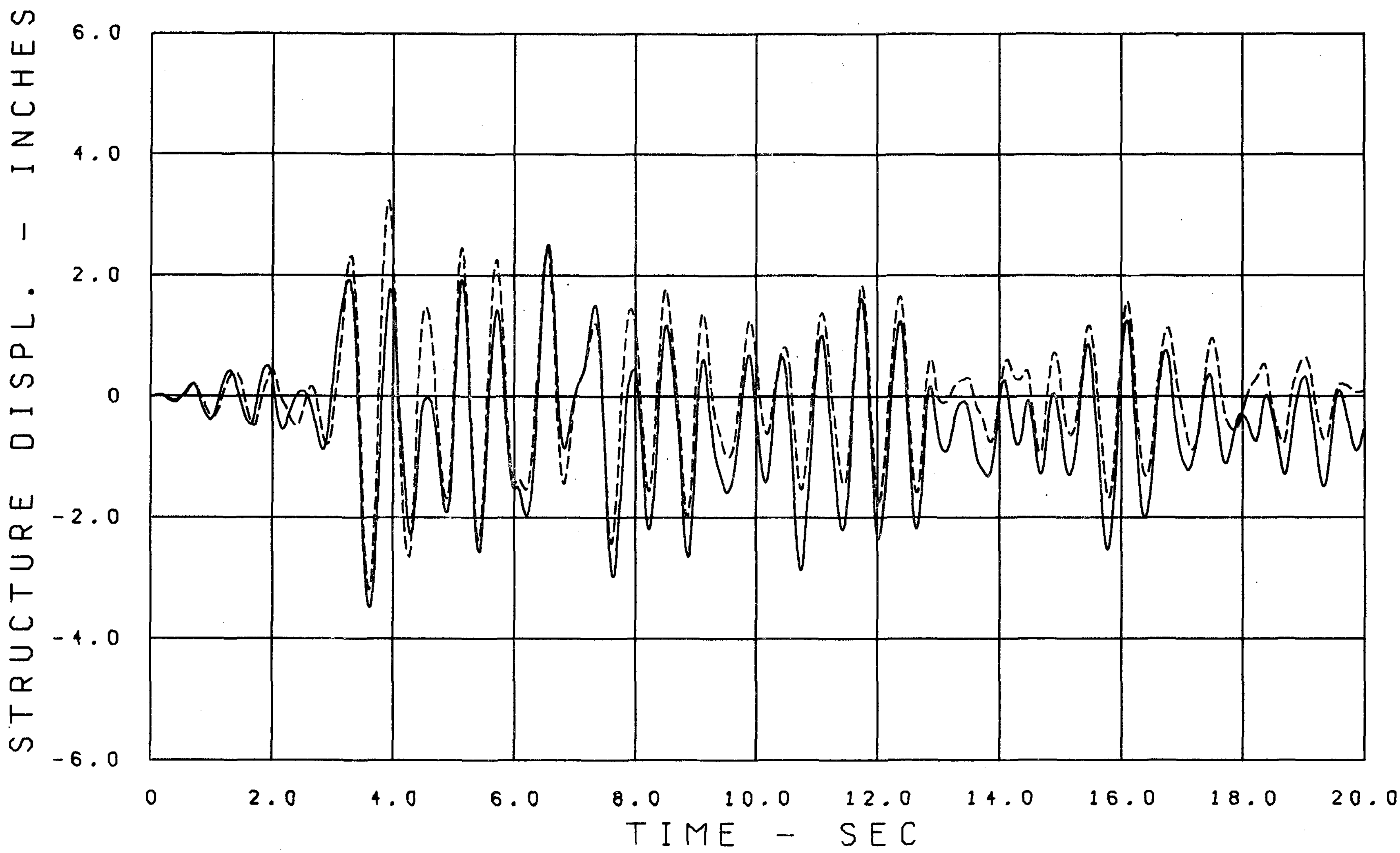

FIG. 5.8 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, TAFT

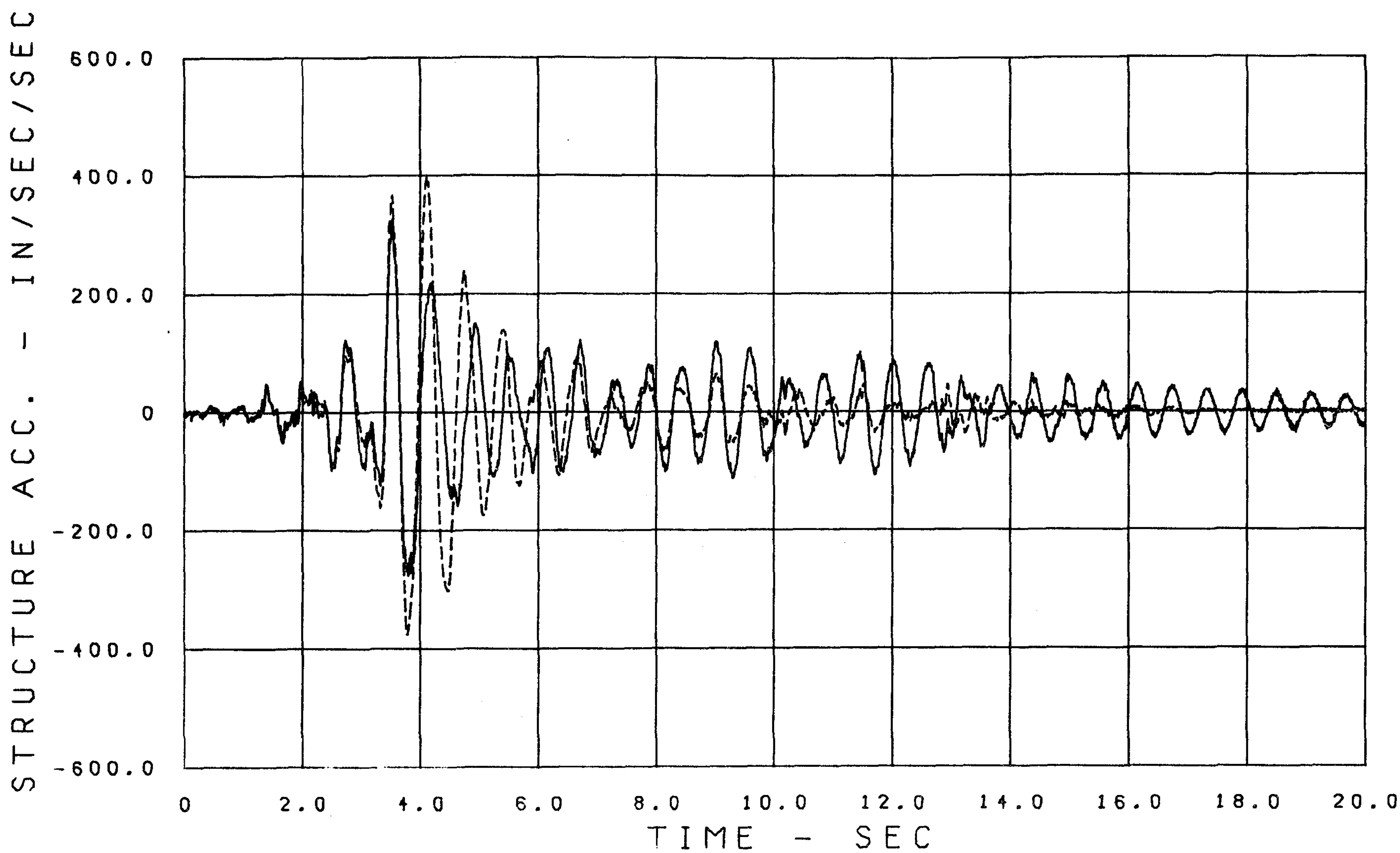

FIG. 5.9 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE ACCELERATION
RESPONSE TIME HISTORIES, PARKFIELD

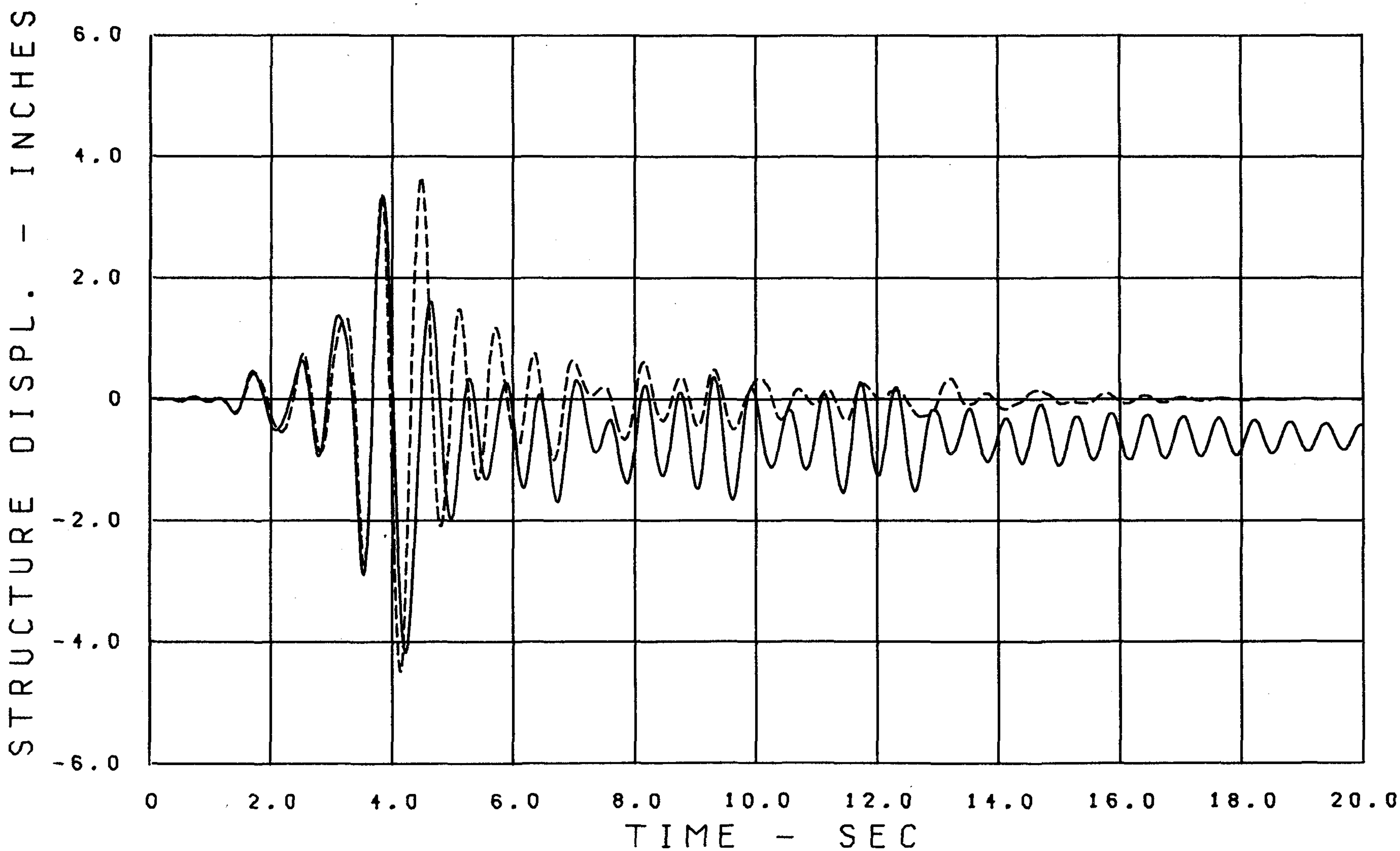

FIG. 5.10 ELMSI1;
COMPARISON OF MEASURED AND COMPUTED RELATIVE DISPLACEMENT
RESPONSE TIME HISTORIES, PARKFIELD

histories to Taft are most accurately predicted and the response time histories to Pacoima and Parkfield are not as well obtained. As in general the nonlinear response of a structure to different earthquake excitations can have very different characteristics, the general applicability of an equivalent linear model to predict the response time histories is very limited.

5.2.2 Prediction of the Maximum Response Values

The maximum response values to El Centro and the other three excitations as predicted by all the equivalent linear models are summarized in Table 5.1.

All of the methods of equivalent linearization except GSP and SREL underestimate the maximum displacement for El Centro more than 30% but ELMGSP is able to predict it within 10% accuracy. It is interesting to note that all methods except SREL predict the maximum acceleration for El Centro with more accuracy than the maximum displacement.

As the nonlinear response of a structure to an earthquake excitation is very dependent on the overall characteristics of the excitation and can therefore be very different for different excitations, we would not expect that an equivalent linear model constructed from the nonlinear response to one excitation will in general be able to predict the nonlinear maximum response values for other excitations very accurately. It is therefore surprising to observe that the maximum displacements for the other three excitations of Chapter 2 are predicted with more accuracy than the maximum displacement for El Centro by most of the models. In fact some of the models predict those maximum displacements with very high level of accuracy, e.g., ELMSI1 which predicts all

TABLE 5.1

NUMERICAL RESULTS FOR THE EQUIVALENT LINEAR MODELS

Method of Equivalent Linearization	Equivalent Linear Model	Eq. Lin. Parameters		Maximum Response Values for							
		c_e (#sec/in.)	k_e (#/in.)	EL CENTRO		PACOIMA		TAFT		PARKFIELD	
				$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x	$\ddot{x}_{abs}$	x
System Identification	ELMSI1	23.60	1566.5	344.8	3.37	477.7	4.67	332.2	3.26	458.2	4.49
	ELMSI2	32.68	1344.3	268.2	3.00	315.5	3.51	265.2	2.98	355.1	4.00
Gulkan-Sozen Procedure	ELMGSP	20.69	872.4	252.1	4.41	250.9	4.39	294.9	5.16	253.0	4.43
Modified Dynamic Stiffness	ELMMDS1	24.01	1305.5	274.5	3.20	307.0	3.58	299.2	3.50	394.0	4.62
	ELMMDS2	23.06	1060.5	233.3	3.37	218.7	3.16	268.6	3.86	305.4	4.40
Average Period and Damping	ELMAPD1	25.82	1404.5	292.5	3.17	381.6	4.13	307.1	3.34	412.7	4.49
	ELMAPD2	38.58	1389.4	267.0	2.86	325.1	3.47	254.7	2.75	337.9	3.67
Average Stiffness and Energy	ELMASE1	32.53	1440.4	286.5	3.00	370.7	3.87	278.9	2.93	379.2	4.00
	ELMASE2	48.77	1467.7	264.4	2.63	333.4	3.32	240.7	2.42	311.9	3.16
Stationary Random Equivalent Linearization	ELMSREL1	21.24	1545.1	345.9	3.44	492.9	4.90	346.9	3.45	473.2	4.72
	ELMSREL2	15.23	1746.0	464.2	4.11	477.9	4.23	405.2	3.59	528.8	4.68
MEASURED NONLINEAR RESPONSE				267.9	4.75	261.5	4.85	258.1	3.47	248.8	4.18

NOTE: $\ddot{x}_{abs}$ (in./sec^2)

x (in.)

of them within 10% accuracy, ELMAPD1 and ELMSREL1 all within 15% and ELMGSP, all except Taft's, within 10%. The maximum accelerations for the other three excitations of Chapter 2 are predicted with less accuracy than the maximum acceleration for El Centro by all the models except ELMGSP.

ELMGSP predicts very accurately the maximum response values for El Centro and also for excitations causing similiar degree of inelastic deformations of the structure as El Centro does but tends to heavily overestimate the maximum displacement for excitation causing very mild inelastic deformations.

For the methods requiring a bilinear hysteretic approximation of the structural behavior, the equivalent linear models based on the work hardening mechanism predict the maximum displacement in general more accurately than the models based on the elastic-plastic model, result a kind of unexpected. SREL gives fairly accurate predictions of the maximum displacements while ASE is always too conservative.

6. CONCLUSIONS

Six methods of linearization have been used to construct various equivalent linear models to predict the nonlinear seismic behavior of a one-story steel frame which was constructed and tested by Sveinsson and McNiven [1] at the Earthquake Engineering Research Center of the University of California, Berkeley.

Four of the methods of linearization depend on the restoring force-displacement relation of the frame. We have, therefore, constructed two bilinear models to approximate the actual hysteretic behavior, one to represent the elastic-plastic nature of the structural steel, the other to represent the work hardening nature. Both bilinear models reproduce the response time histories quite accurately in the domain appropriate to each. The bilinear material models were constructed only for these four schemes, imposing on them the complications that the dual material models introduce. The other two schemes are independent of these material considerations.

The construction of all the equivalent linear models was based on the measured nonlinear response of the structure to El Centro excitation, and the objective for their construction was the ability to predict the maximum response values, with precedence being given to the maximum displacement response. This is the basis of assessment that authors of the schemes set for themselves.

All six schemes of linearization except Gülkan-Sozen Procedure underestimate the maximum displacement for El Centro more than 30%, but are in general able to predict the maximum acceleration for El Centro with more accuracy. An underestimation of the maximum displacement of 30% can be very dangerous, so when the ductility demand for a structure is estimated using these linear models a detailing of the critical regions in the structure that insures ductility capacities far in excess of the values computed is recommended.

As might be expected, none of the methods of linearization has the necessary generality to be able to predict the maximum response values for other excitations with acceptable level of accuracy, even though we have considered the two-phase nature of the structural steel for the material dependent schemes. We note, without comment, that the two methods that are independent of the restoring force-displacement relation predict the maximum displacements for other excitations with more accuracy than the methods dependent on the material properties.

The results of the study indicate that the dependence of the nonlinear response of a structure on the characteristics of the earthquake excitation is so complex that there is no way that the linearization schemes considered can have the required generality to limit the ductility demands to specified values. Nonetheless, these methods can provide very valuable guidelines for design, if their limitations and relationship to the overall design process is fully recognized.

Finally, it is worth noting that the four material dependent schemes are modifications of schemes for harmonic or random inputs and they are probably able to predict the maximum displacements more accurately for those excitations than for earthquake excitation.

APPENDIX A: COMPUTER PROGRAM

Program Description

The computer program presented in this appendix, has been developed to establish the characteristic parameters of a linear SDOF system subjected to earthquake excitation. Assuming the mass of the system, m, to be known, only the viscous damping coefficient, c, and the stiffness, k, are open for establishment.

These two parameters are established by the method of System Identification, which is a process for constructing a mathematical description or model of a physical system when both the input to the system and the corresponding output are known. The resulting model, when it is subjected to the same input should produce a response that matches in some sense the system's output. The exactness of the match is measured by a criterion function, which here is taken as an integral squared error function in the relative acceleration, thus

$$J(\overline{\beta},T_1,T_2) = \int_{T_1}^{T_2} \{\ddot{x}(\overline{\beta},t) - \ddot{y}(t)\}^2 \, dt$$

where

- $\overline{\beta}$ is a vector of the parameters c and k,
- T_1 is the lower limit of integration,
- T_2 is the upper limit of integration,
- $\ddot{x}(\overline{\beta},t)$ is the relative acceleration of the model using parameters $\overline{\beta}$ and excitation $\ddot{X}_g(t)$,
- $\ddot{y}(t)$ is the relative acceleration of the physical system when it is subjected to the same excitation.

The response of the mathematical model to a specified ground acceleration is computed by Wilson-θ-Method of numerical integration. Finally, to establish the optimum value of the parameters the criterion function is minimized by Modified Gauss-Newton algorithm. A more detailed description of the method of System Identification as used here can be found in References [13,14].

This process of System Identification is incorporated by the program IDEN, listing of which is given in Appendix B. It should be noted that the program was written as a special purpose program and the authors are fully aware of the fact, that various refinements could be made to increase it's clarity and generality of application. With some modifications the program can be extended to deal with the identification of a linear MDOF system subjected to earthquake excitation.

The computer program is written in FORTRAN IV and was developed on the CDC 6400 computer at the University of California, Berkeley.

Input Data

The following sequence of punched cards and data on a tape are required for an identification run using the program IDEN.

Data Cards

1st Card (2I10)

Cols. 1-10 NP: Number of parameters
NP=2, for linear SDOF system

11-20 NDOF: Number of degrees of freedom
NDOF=1, for linear SDOF system

2nd Card (F10.0, I10)

Cols. 1-10 R : Pseudostatic influence coefficient

$R = 1$, for rigid base translation

11-20 IR : Select response quantity in the criterion function

$IR = 3$, for relative acceleration

3rd Card (4I10, 3F10.0)

Cols. 1-10 NES: $= \frac{T_1}{\Delta t}$,

T_1: lower limit in the integration of the criterion function (sec)

Δt: time interval between digitized values of the measured response of the physical system (sec) ($\Delta t = 0.01$ sec in our case)

Cols. 11-20 NPTS: $= \frac{T_2}{\Delta t}$,

T_2: upper limit in the integration of the criterion function (sec)

Cols. 21-30 ITLS: Maximum no. of iterations allowed in each line search

Cols. 31-40 IT: Maximum no. of iterations allowed for minimization of the criterion function

Cols. 41-50 SLMIN: Line search tolerance, i.e., stop the line search if the slope of the error surface is less than SLMIN

Cols. 51-60 ENDTOL: Program stopping tolerance, i.e., stop the program execution if

$|ERROR(I-1) - ERROR(I)| \leq ENDTOL$

Cols. 61-70 DDF: Finite difference control parameter

DDF = 0.0, if finite difference is not used

4th Card (4F10.0)

Cols. 1-10 DELTA: = 0.5

11-20 ALPHA: = 1/6

21-30 THETA: > 1.37, to secure unconditional stability in the numerical integration

31-40 DELT: Time interval between digitized values of the measured response of the physical system (sec)

NOTE: These four parameters are for the numerical integration by Wilson-θ-Method.

5th Card (F10.0)

Cols. 1-10 SM: The mass of the system

6th Card (2F10.0)

Cols. 1-10 B(1): Initial guess of the viscous damping coefficients.

11-20 B(2): Initial guess of the stiffness

Tape

For the identification, the number of digitized points in the measured response time histories of the physical system, along with the ground (table) acceleration and the relative acceleration of the system, should be available on a tape.

Example

Using the measured nonlinear response to El Centro of the structure tested by Sveinsson and McNiven [1] we will use the computer program to establish an equivalent linear model of the structure based on the first 4 sec. of the measured relative acceleration time history.

The necessary input data from cards is shown on page 81. The computer output is shown on pages 82-85.

EXAMPLE - INPUT

```
                2            1
1.Ø                         3
               Ø          4ØØ              5                 5.ØØØØØ1     .ØØ1        Ø.Ø
.5             .1666666671.4              .Ø1
15.47
25.            15ØØ.
```

EXAMPLE - OUTPUT

```
SYSTEM IDENTIFICATION FOR A LINEAR SINGLE-DEGREE-OF-FREEDOM
SYSTEM SUBJECTED TO EARTHQUAKE EXCITATION.

I N P U T

NP=     2  NDOF=     1

R=1.000  IR=     3

NES=     0  NPTS=  400  ITLS=    5  IT=    5

SLMIN=  .0000010  ENDTOL=  .0010000  CDF=0.

DELTA=     .50000  ALPHA=    .16667

THETA=   1.40000  DELT=     .01000

M=  15.47000  C=  25.00000  K=1500.00000
```

I T E R A T I O N

STEP 1

NP	PARAMETER	GRADIENT	DIRECTION
1	.25ØØØØE+Ø2	-.268778E+Ø3	-.767418E+Ø1
2	.15ØØØØE+Ø4	.369889E+Ø2	.123913E+Ø3

INTERPOLATION:

FIRST POINT			SECOND POINT				
ALPHA	ERROR	SLOPE	ALPHA	ERROR	SLOPE	INTER-POLATED ALPHA	BOUNDARY REACHED
Ø.	.765E+Ø4	-.665E+Ø4	.1ØØE+Ø1	.437E+Ø4	-.682E+Ø3	.113E+Ø1	NO
Ø.	.765E+Ø4	-.665E+Ø4	.113E+Ø1	.431E+Ø4	-.193E+Ø3	.117E+Ø1	NO
Ø.	.765E+Ø4	-.665E+Ø4	.117E+Ø1	.431E+Ø4	-.557E+Ø2	.118E+Ø1	NO
Ø.	.765E+Ø4	-.665E+Ø4	.118E+Ø1	.431E+Ø4	-.162E+Ø2	.119E+Ø1	NO
Ø.	.765E+Ø4	-.665E+Ø4	.119E+Ø1	.431E+Ø4	-.472E+Ø1	.119E+Ø1	NO

STEP 2

NP	PARAMETER	GRADIENT	DIRECTION
1	.34108ØE+Ø2	.24Ø538E+Ø2	.1482Ø1E+Ø1
2	.135294E+Ø4	.15ØØ79E+Ø1	.93Ø692E+Ø1

INTERPOLATION:

FIRST POINT			SECOND POINT				
ALPHA	ERROR	SLOPE	ALPHA	ERROR	SLOPE	INTER-POLATED ALPHA	BOUNDARY REACHED
Ø.	.431E+Ø4	-.496E+Ø2	.1ØØE+Ø1	.428E+Ø4	.161E+Ø1	.97ØE+ØØ	NO
Ø.	.431E+Ø4	-.496E+Ø2	.97ØE+ØØ	.428E+Ø4	.667E-Ø2	.97ØE+ØØ	NO
Ø.	.431E+Ø4	-.496E+Ø2	.97ØE+ØØ	.428E+Ø4	.276E-Ø4	.97ØE+ØØ	NO

STEP 3

NP	PARAMETER	GRADIENT	DIRECTION
1	.326700E+02	.642082E+00	.385775E-01
2	.134391E+04	-.102243E+00	-.605929E+00

INTERPOLATION:

FIRST POINT ALPHA	FIRST POINT ERROR	FIRST POINT SLOPE	SECOND POINT ALPHA	SECOND POINT ERROR	SECOND POINT SLOPE	INTER-POLATED ALPHA	BOUNDARY REACHED
0.	.428E+04	-.867E-01	.100E+01	.428E+04	-.254E-02	.103E+01	NO
0.	.428E+04	-.867E-01	.103E+01	.428E+04	-.898E-04	.103E+01	NO
0.	.428E+04	-.867E-01	.103E+01	.428E+04	-.310E-05	.103E+01	NO

STEP 4

NP	PARAMETER	GRADIENT	DIRECTION
1	.326311E+02	.227356E-01	.133519E-02
2	.134453E+04	.144729E-02	.836931E-02

INTERPOLATION:

FIRST POINT ALPHA	FIRST POINT ERROR	FIRST POINT SLOPE	SECOND POINT ALPHA	SECOND POINT ERROR	SECOND POINT SLOPE	INTER-POLATED ALPHA	BOUNDARY REACHED
0.	.428E+04	-.425E-04	.100E+01	.428E+04	.745E-06	.100E+01	NO

F I N A L   R E S U L T S

NP	PARAMETER
1	.32630E+02
2	.13445E+04

ERROR= .42807E+04

APPENDIX B

LISTING OF THE COMPUTER PROGRAM IDEN

```
      PROGRAM IDEN(INPUT,TAPE5=INPUT,OUTPUT,TAPE99=OUTPUT,TAPE1,
     *   TEMP,TAPE2=TEMP)
C
C*************************************************************************
C                                                                        *
C     SYSTEM IDENTIFICATION PROGRAM FOR A LINEAR                         *
C     SINGLE-DEGREE-OF-FREEDOM SYSTEM SUBJECTED TO                       *
C     EARTHQUAKE EXCITATION                                              *
C                                                                        *
C     WRITTEN BY            - JERRY DIMSDALE                             *
C     MODIFICATIONS BY      - HELGI VALDIMARSSON                         *
C                           - UNIVERSITY OF CALIFORNIA                   *
C                             BERKELEY                                   *
C                                                                        *
C*************************************************************************
C
      COMMON /KDYN/ A0,A1,A2,A3,A4,A5,A6,A7,A8,A9,DELTA,THETA,DELT,RINT
     *    ,RINTT,NDOF,NP,DELT2,ALPHA
      COMMON/INPT/VG(4000),V(1,4000),IR(1),R(1),NPTS,NES
      COMMON /FFERR/ FERR(2)
      COMMON /FDIFF/ DF,DDF
      DIMENSION B(2),K(2,2),M(2,2),C(2,2),DC(2,2,2),DK(2,2,2),
     *   KS(2,2),GRAD(2),AH(2,2),D(6,2),DLDB(6,2,2),F(2),
     *   DB(2),IB3(2),BN(2)
      REAL K,M,KS
      REWIND 1
      REWIND 2
C
C     READ INFORMATIONS FROM CARDS
C
      READ 1000,NP,NDOF
 1000 FORMAT (2I10)
      READ 1002,((R(I),IR(I)),I=1,NDOF)
 1002 FORMAT (F10.0,I10)
      READ 10,NES,NPTS,ITLS,IT,SLMIN,ENDTOL,DDF
10    FORMAT(4I10,3F10.0)
      READ 1111,DELTA,ALPHA,THETA,DELT
1111  FORMAT(4F10.0)
      READ 4,SM
4     FORMAT(F10.0)
      READ 1001,(B(I),I=1,NP)
 1001 FORMAT (8F10.0)
      IF(EOF,5) 173,183
  173 READ (2) (B(I),I=1,NP)
C
C     INITIALIZE VARIOUS PARAMETERS
C
183   IUSEFD=0
      NCALL=2
      NUMIT=0
      BOUND=2HNO
      PRELRR=1E10
      IF (ABS(DDF).GT.1.E-10) IUSEFD=1
      DF=1.0+DDF
      CALL INIT
```

```
C
C     READ INFORMATIONS FROM TAPE
C
      READ(1,111) NPT
111   FORMAT(I5)
      READ(1,222)  (VO(I),I=1,NPT)
222   FORMAT(6E15.9)
      READ(1,222)  (V(1,I),I=1,NPT)
C
C     PRINT INPUT INFORMATIONS
C
      PRINT 1004
1004  FORMAT(////,5X,*SYSTEM IDENTIFICATION FOR A LINEAR SINGLE-DEGREE-O
     1F-FREEDOM*,/,5X,*SYSTEM SUBJECTED TO EARTHQUAKE EXCITATION.*,///,5
     2X,*I N P U T*)
      PRINT 5000,NP,NDOF
5000  FORMAT(///,5X,*NP=*,I5,2X,*NDOF=*,I5)
      PRINT 5002,R(1),IR(1)
5002  FORMAT(//,5X,*R=*,F5.3,2X,*IR=*,I5)
      PRINT 5003,NES,NPTS,ITLS,IT,SLMIN,ENDTOL,DDF
5003  FORMAT(//,5X,*NES=*,I5,2X,*NPTS=*,I5,2X,*ITLS=*,I5,2X,*IT=*,I5,//,
     15X,*SLMIN=*,F10.7,2X,*ENDTOL=*,F10.7,2X,*DDF=*,F5.3)
      PRINT 2113,DELTA,ALPHA,THETA,DELT
2113  FORMAT(//,5X,*DELTA=*,F10.5,2X,*ALPHA=*,F10.5,//,5X,*THETA=*,F10.5
     1,2X,*DELT=*,F10.5)
      PRINT 5001,SM,B(1),B(2)
5001  FORMAT(//,5X,*M=*,F10.5,2X,*C=*,F10.5,2X,*K=*,F10.5,///,5X,*I T E
     1R A T I O N*,///)
C
C
C     MINIMIZATION OF THE CRITERION FUNCTION
C
  190 NUMIT=NUMIT+1
      LEVEL=3
C
C     ESTABLISHMENT OF THE SEARCH DIRECTION BY GAUSS-NEWTON METOD
C
      CALL DIR(B ,R,M,C,LEVEL,DK,DC,KS,ERR,GRAD,AH,U,DUDB,F,NCALL
     * ,DB,E,D,DJ,SM)
      IF(IUSED.EQ.1)          LEVEL=2
  945 FORMAT (1X,*STEP *,I2//6X,*NP*,5X,*PARAMETER*,5X,
     *    *GRADIENT*,8X,*DIRECTION*)
  946 FORMAT (6X,I2,4X,E12.5,4X,E12.6,4X,E12.6)
 1005 FORMAT (9X,*INTERPOLATION:*//20X,*FIRST POINT*,
     +    19X,*SECOND POINT*/14X,24(1H-),6X,24(1H-),7X,*INTER-*/
     +    74X,*POLATED  BOUNDARY*/11X,2(5X,*ALPHA*,5X,*ERROR*,5X,*SLOPE*)
     +    ,5X,*ALPHA   REACHED*/)
      INTR=0
      IF(ERR.GT.PREERR) GO TO 2006
      IF (ABS(ERR-PREERR).LE.ENDTOL) GO TO 600
      ERRA=ERR
      PREERR=ERR
  200 DO 210 L=1,NP
  210 DB(L)=GRAD(L)
      CALL SYMSOL(AH,DB,NP   ,1,0,NCALL)
      PRINT 945,NUMIT
```

```
      PRINT 946,((I,B(I),GRAD(I),DB(I)),I=1,NP   )
C
C     ESTABLISHMENT OF THE STEP SIZE BY LINE SEARCH
C
      ALPHAA=0.0
      DJA=0.0
      DO 220 L=1,NP
      BN(L)=B(L)
      DJA=DJA-DB(L)*GRAD(L)
  220 CONTINUE
      IF(NUMIT.GT.IT) GO TO 700
      PRINT 1005
      ALPHAB=1.0
      CALL CHECK(B,BN,DB,ALPHAB,OKB)
      CALL DIR(BN,K,M,C,LEVEL,DK,DC,KS,ERRB,GRAD,AH,U,DUDB,F,NCALL
     * ,DB,ALPHAB,DJB,SM)
  226 IF (ABS(DJB).LT.SLMIN) GO TO 450
      INTR=INTR+1
      IF (DJB) 300,450,227
  227 ALPHAN=CUBIC(ERRA,DJA,ERRB,DJB,ALPHAA,ALPHAB)
      IF ((OKB.LT.1.0).AND.(INTR.EQ.1)) BOUND=3HYES
      PRINT 6,ALPHAA,ERRA,DJA,ALPHAB,ERRB,DJB,ALPHAN,BOUND
      BOUND=2HNO
    6 FORMAT (11X,7E10.3,7X,A3)
      DO 230 L=1,NP
  230 BN(L)=B(L)-ALPHAN*DB(L)
      CALL DIR(BN,K,M,C,LEVEL,DK,DC,KS,ERRN,GRAD,AH,U,DUDB,F,NCALL
     * ,DB,ALPHAN,DJN,SM)
      IF (ABS(DJN).LT.SLMIN) GO TO 455
      IF (DJN) 240,455,245
  240 ERRA=ERRN
      DJA=DJN
      ALPHAA=ALPHAN
      GO TO 400
  245 ERRB=ERRN
      DJB=DJN
      ALPHAB=ALPHAN
      GO TO 400
  300 IF (ERRB.GT.ERRA) GO TO 227
      ALPHAN=SQ(ERRA,DJA,ERRB,DJB,ALPHAA,ALPHAB)
      CALL CHECK(B,BN,DB,ALPHAN,OKN)
      IF (OKN.LT.1.0) BOUND=3HYES
      PRINT 6 ,ALPHAA,ERRA,DJA,ALPHAB,ERRB,DJB,ALPHAN,BOUND
      BOUND=2HNO
      CALL DIR(BN,K,M,C,LEVEL,DK,DC,KS,ERRN,GRAD,AH,U,DUDB,F,NCALL
     * ,DB,ALPHAN,DJN,SM)
      IF (ABS(DJN).LT.SLMIN) GO TO 455
      IF (DJN) 320,455,245
  320 IF (ERRN.LE.ERRB) GO TO 245
      ERRA=ERRB
      ALPHAA=ALPHAB
      DJA=DJB
      GO TO 245
400   IF(INTR.LT.ITLS) GO TO 226
      GO TO 455
  450 CONTINUE
```

```
      IF (ERRB.GT.ERRA) GO TO 227
      ERR=ERRB
      ALPHAN=ALPHAB
      PRINT 6,ALPHAA,ERRA,DJA,ALPHAB,ERRB,DJB,ALPHAN,BOUND
      GO TO 460
  455 ERR=ERRN
  460 DO 465 L=1,NP
  465 B(L)=BN(L)
      BOUND=2HNO
      GO TO 190
 2006 PRINT 2007
 2007 FORMAT (* INCREASING ERROR*)
  700 PRINT 3000,DJA,ERRA
 3000 FORMAT (1H0,*FINAL SLOPE:*,G12.6/* FINAL ERROR:*,G12.6)
  600 WRITE (2) (B(I),I=1,NP)
      PRINT 5973
5973  FORMAT(///,5X,*F I N A L  R E S U L T S*,//,19X,*NP*,9X,*PARAMETER
     1*)
      PRINT 9602,((I,B(I)),I=1,NP)
 9602 FORMAT (1H0,10X,110,5X,E15.5)
      PRINT 9603,(PERR(I),I=1,NDOF)
9603  FORMAT(//,20X,*ERROR=*,E15.5)
C
      STOP
      END
```

```
      SUBROUTINE DIR(B,K,M,C,LEVEL,DK,DC,KS,ERR,GRAD,AH,U,DUDB,F,NCALL
     * ,DB,ALPHA,DJ,SM)
C
C
C     ROUTINE TO CALCULATE THE ERROR, GRADIENT AND HESSIAN OF
C     THE CRITERION FUNCTION USING WILSON-THETA-METHOD
C     FOR THE NUMERICAL INTEGRATION
C
      COMMON /MLYN/ A0,A1,A2,A3,A4,A5,A6,A7,A8,A9,DELTA,THETA,DELT,RINT
     *     ,RINTT,NDOF,NP,DELT2
      COMMON/INPT/VG(4000),V(1,4000),IR(1),R(1),NPTS,NES
      COMMON /FPERR/ PERR(2)
      COMMON /FDIFF/ DF,DDF
      COMMON /INDEX/ I11,I21,I31,I12,I22,I32
      DIMENSION B(1),K(NCALL,1),M(NCALL,1),C(NCALL,1),DK(NCALL,NCALL,1),
     *   GRAD(1),AH(NCALL,NCALL),VGC(2),
     * DUDB(6,NCALL,1),U(6,1),F(1)
     * ,DC(NCALL,NCALL,1),KS(NCALL,1)
     *   ,DB(1),BT(2)
      REAL K,M,KS
C
      IFD=0
      DA=ALPHA*DDF
      DO 70 I=1,NP
      GRAD(I)=0.0
      DO 60 L=1,3
      DO 60 LL=1,NDOF
      DUDB(L,LL,I)=0.0
   60 CONTINUE
      DO 70 J=1,NP
      AH(I,J)=0.0
   70 CONTINUE
C     GET THE INFLUENCE COEFFICIENTS
90    CALL MFIND (K,M,C,DK,DC,NCALL,B,SM)
      ERR=0.0
      IC=0
      VGC(1)=VG(1)
      DO 105 J=1,NDOF
      PERR(J)=0.
C     GET THE EFFECTIVE STIFFNESS
      DO 100 I=1,NDOF
      KS(I,J)=K(I,J)+A0*M(I,J)+A1*C(I,J)
  100 CONTINUE
      DO 102 I=1,3
      U(I,J)=0.0
  102 CONTINUE
  105 CONTINUE
      CALL SYMSOL(KS,K,NDOF,1,1,NCALL)
C
C     THE NUMERICAL INTEGRATION
C
  110 IC=IC+1
      IF (IC.GT.NPTS) GO TO 205
      K1=1
      K2=MOD(IC,2)+1
```

```
      IF (K2.EQ.1) K1=2
      ITK1=3*(K1-1)
      ITK2=3*(K2-1)
      I11=1+ITK1
      I21=I11+1
      I31=I21+1
      I12=1+ITK2
      I22=I12+1
      I32=I22+1
      VGC(K2)=VG(IC+1)
C     GET THE EFFECTIVE LOAD FOR CALCULATION OF THE ERROR
      DO 120 I=1,NDOF
      F(I)=-M(I,I)*R(I)*(VGC(K1)+THETA*(VGC(K2)-VGC(K1)))*386.4
  120 CONTINUE
      CALL XNT (K1,K2,F,U,KS,NCALL,          1,M,C)
      IF(IC.LT.NES) GO TO 2212
      DO 130 I=1,NDOF
      IRT=IR(I)
      IF (IRT.EQ.0) GO TO 130
      IS=IRT+ITK1
      T=U(IS,1)-V(I,IC)
      T2=T*T
      ERR=ERR+T2
      PERR(I)=PERR(I)+T2
  130 CONTINUE
2212  CONTINUE
      IF (LEVEL.LE.2) GO TO 110
C     GET THE EFFECTIVE LOAD FOR CALCULATIONS
C     OF THE GRADIENT AND THE HESSIAN
      DO 150 L=1,NP
      DO 140 I=1,NDOF
      F(I)=0.0
      DO 140 J=1,NDOF
      F(I)=F(I)-DC(I,J,L)*(U(I21,J)+THETA*(U(I22,J)-U(I21,J)))-
     *  DK(I,J,L)*(U(I11,J)+THETA*(U(I12,J)-U(I11,J)))
  140 CONTINUE
      CALL XNT(K1,K2,F,DUDB,KS,NCALL,          L,M,C)
  150 CONTINUE
      IF(IC.LT.NES) GO TO 2112
      DO 203 J=1,NDOF
      IRT=IR(J)
      IF (IRT.EQ.0) GO TO 203
      I1=IRT+ITK1
      I2=IRT+ITK2
      DO 202 IP=1,NP
      GRAD(IP)=GRAD(IP)+(U(I1,J)-V(J,IC))*DUDB(I1,J,IP)
      DO 200 IS=1,IP
      AH(IP,IS)=AH(IP,IS)+DUDB(I1,J,IP)*DUDB(I1,J,IS)
  200 CONTINUE
  202 CONTINUE
  203 CONTINUE
2112  CONTINUE
      GO TO 110
C
C     THE ERROR VALUE
  205 ERR=ERR*DELT
```

```
      DO 206 I=1,NDCF
  206 PERR(I)=PERR(I)*DELT
      IF (LEVEL.EQ.2) GO TO 5000
      IF (LEVEL.EQ.1) RETURN
C     THE GRADIENT VECTOR
      DO 210 I=1,NP
      GRAD(I)=DELT2*GRAD(I)
C     THE HESSIAN MATRIX
      DO 210 J=1,I
      AH(J,I)=DELT2*AH(I,J)
      AH(I,J)=AH(J,I)
  210 CONTINUE
      IF (ALPHA.EQ.0.0) RETURN
      DJ=0.0
      DO 300 L=1,NP
  300 DJ=DJ-DB(L)*GRAD(L)
      RETURN
 5000 CONTINUE
      IF (IFD.EQ.1) GO TO 5200
      TERR=ERR
      IFD=1
      DO 5100 L=1,NP
      BT(L)=B(L)
      B(L)=B(L)-DA*DB(L)
 5100 CONTINUE
      GO TO 90
 5200 DJ=(ERR-TERR)/DA
      DO 5250 L=1,NP
 5250 B(L)=BT(L)
      ERR=TERR
      RETURN
      END
```

```
      SUBROUTINE INIT
C
C
C     ROUTINE TO INITIALIZE THE NUMERICAL INTEGRATION PARAMETERS
C
      COMMON /KEYN/ A0,A1,A2,A3,A4,A5,A6,A7,A8,A9,DELTA,THETA,DELT,RINT
     *      ,RINTT,NOUF,NF,DELT2,ALPHA
      TAU=THETA*DELT
      A2=1.0/(ALPHA*TAU)
      A0=A2/TAU
      A1=A2*DELTA
      A3=.5/ALPHA-1.0
      A4=DELTA/ALPHA-1.0
      A5=.5*TAU*(A4-1.0)
      A6=DELT*(1.0-DELTA)
      DELT2=2.0*DELT
      A7=DELT*DELTA
      DTT=DELT*DELT
      A9=ALPHA*DTT
      A8=.5*DTT-A9
      RINT=THETA-FLOAT(IFIX(THETA))
      RINTT=1.0-RINT
      RETURN
      END
```

```
      SUBROUTINE MFIND(K,M,C,DK,DC,NCALL,B,SM)
C
C
C     ROUTINE TO SET THE VALUES OF THE INFLUENCE COEFFICIENTS
C     IN THE DIFFERENTIAL EQUATIONS
C
      DIMENSION K(NCALL,1),M(NCALL,1),C(NCALL,1),DK(NCALL,NCALL,1),
     *   DC(NCALL,NCALL,1)
     * ,B(1)
      REAL K,M
      M(1,1)=SM
      C(1,1)=B(1)
      K(1,1)=B(2)
      DC(1,1,1)=1.
      DC(1,1,2)=0.
      DK(1,1,1)=0.
      DK(1,1,2)=1.
      RETURN
      END
```

```
      SUBROUTINE XNT(K1,K2,FS,U,KS,NCALL,IDP,M,C)
C
C
C     ROUTINE TO SOLVE THE EFFECTIVE EQUILIBRIUM EQUATION
C     IN THE NUMERICAL INTEGRATION
C
      COMMON /KDYN/ A0,A1,A2,A3,A4,A5,A6,A7,A8,A9,DELTA,THETA,DELT,RINT
     *     ,RINIT,NDOF,NP,DELT2
      COMMON /INDEX/ I11,I21,I31,I12,I22,I32
      DIMENSION U(6,NCALL,1  ),KS(NCALL,1),M(NCALL,1),C(NCALL,1)
     * ,FS(1)
      REAL M,KS
      DO 120 J=1,NDOF
      TU=U(I11,J,IDP)
      TUD=U(I21,J,IDP)
      TUDD=U(I31,J,IDP)
      DO 120 I=1,NDOF
      FS(I)=FS(I)+M(I,J)*(A0*TU+A2*TUD+A3*TUDD)+C(I,J)*(A1*TU+A4*TUD+
     *   A5*TUDD)
  120 CONTINUE
      CALL SYMSOL(KS,FS,NDOF,1,2,NCALL)
      DO 150 I=1,NDOF
      U11=U(I11,I,IDP)
      U21=U(I21,I,IDP)
      U31=U(I31,I,IDP)
      T=A0*(FS(I)-U11)-A2*U21-A3*U31
      U32=U31+(T-U31)/THETA
      U(I22,I,IDP)=U21+A6*U31+A7*U32
      U(I12,I,IDP)=U11+DELT*U21+A8*U31+A9*U32
      U(I32,I,IDP)=U32
  150 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE CHECK(B,BT,DB,ALPHA,OK,NCALL)
C
C
C     ROUTINE TO CHECK WHETHER A CALCULATED PARAMETER SET
C     IS WITHIN THE FEASIBLE REGION
C
      COMMON /KLYN/ A0,A1,A2,A3,A4,A5,A6,A7,A8,A9,DELTA,THETA,DELT,RINT
     *     ,RINTT,NOOF,NP,DELT2
      DIMENSION TS(20),T(20),B(NCALL),DB(NCALL),BN(NCALL)
      OK=1.0
      DO 100 L=1,NP
  100 BN(L)=B(L)            -ALPHA*DB(L)
      RETURN
      END
```

```
      FUNCTION CUBIC(Y1,D1,Y2,D2,X1,X2)
C
C
C     ROUTINE TO PERFORM A CUBIC INTERPOLATION FOR THE LINE SEARCH
C
      EQUIVALENCE (A,COEFF(1)),(B,COEFF(2)),(C,COEFF(3))
      DIMENSION T(4,4),COEFF(4),IC4(4)
      T(1,4)=1.0
      T(2,3)=1.0
      T(2,4)=0.0
      T(3,4)=1.0
      T(4,3)=1.0
      T(4,4)=0.0
      DO 100 K=2,4
      KM=5-K
      T(1,KM)=X1*T(1,KM+1)
      T(3,KM)=X2*T(3,KM+1)
  100 CONTINUE
      COEFF(1)=Y1
      COEFF(2)=D1
      COEFF(3)=Y2
      COEFF(4)=D2
      T(2,1)=3.0*X1*X1
      T(2,2)=2.0*X1
      T(4,1)=3.0*X2*X2
      T(4,2)=2.0*X2
      CALL SOLVEQ(T,COEFF,4,1,4,IC4)
      IF (ABS(A).LT.1.0E-10) GO TO 1000
      DISC=B*B-3.0*A*C
      IF (DISC.LT.0.0) GO TO 1060
      DISC=SQRT(DISC)
      TA=3.0*A
      AMIN1=(DISC-B)/TA
      AMIN2=(-DISC-B)/TA
      CUBIC=AMIN1
      TEST=6.0*A*AMIN2+2.0*B
  150 IF(TEST.GT.0.0) CUBIC=AMIN2
      RETURN
 1000 IF (ABS(B).LT.1.0E-10) GO TO 1050
      AMIN2=-C/(2.0*B)
      CUBIC=X2
      TEST=2.0*B
      GO TO 150
 1050 PRINT 5
    5 FORMAT (* CUBIC INTERPOLATION APPEARS CONSTANT*)
 1060 CUBIC=X2
      RETURN
      END
```

```
      FUNCTION SQ (Y1,D1,Y2,D2,X1,X2)
C
C
C     ROUTINE TO PERFORM A QUADRATIC EXTRAPOLATION FOR THE LINE SEARCH
C
      EQUIVALENCE (A,COEFF(1)),(B,COEFF(2))
      DIMENSION T(3,3),COEFF(3),ID3(3)
      T(1,3)=1.0
      T(2,3)=1.0
      T(3,2)=1.0
      T(3,3)=0.0
      SQMAX=5.0*X2
      DO 150 K=2,3
      KM=4-K
      T(1,KM)=X1*T(1,KM+1)
      T(2,KM)=X2*T(2,KM+1)
  150 CONTINUE
      T(3,1)=2.0*X2
      COEFF(1)=Y1
      COEFF(2)=Y2
      COEFF(3)=D2
      CALL SOLVEQ(T,COEFF,3,1,3,ID3)
      IF (A.LT.1.0E-10) GO TO 200
      SQ=-B/(2.0*A)
      IF(SQ.LT.SQMAX) RETURN
  200 SQ=X2-Y2/D2
      SQ=AMIN1(SQ,SQMAX)
      RETURN
      END
```

```
      SUBROUTINE SYMSOL (A,B,NN,LL,M,NCALL)
C
C
C     M=0 TRIANGULARIZE AND SOLVE
C     M=1 TRIANGULIZE ONLY
C     M=2 FORWARD REDUCTION AND BACK SUBSTITUTION ONLY
C
      DIMENSION A(NCALL,NCALL),B(NCALL,LL)
      IF (M.EQ.2) GO TO 500
      DO 400 N=1,NN
      IF (N.EQ.NN) GO TO 500
      D=A(N,N)
      IF (D.EQ.0.0) PRINT 2000,N
      N1=N+1
      DO 300 J=N1,NN
      IF (A(N,J).EQ.0.0) GO TO 300
      A(N,J)=A(N,J)/D
      DO 200 I=J,NN
      A(I,J)=A(I,J)-A(I,N)*A(N,J)
  200 A(J,I)=A(I,J)
  300 CONTINUE
  400 CONTINUE
C     FORWARD REDUCTION AND BACKSUBSTITUTION
  500 IF (M.EQ.1) RETURN
      DO 700 N=1,NN
      DO 600 L=1,LL
  600 B(N,L)=B(N,L)/A(N,N)
      IF (N.EQ.NN) GO TO 800
      N1=N+1
      DO 700 L=1,LL
      DO 700 I=N1,NN
  700 B(I,L)=B(I,L)-A(I,N)*B(N,L)
C
  800 N1=N
      N=N-1
      IF (N.EQ.0) RETURN
      DO 900 L=1,LL
      DO 900 J=N1,NN
  900 B(N,L)=B(N,L)-A(N,J)*B(J,L)
      GO TO 800
C
 2000 FORMAT (39H0***ZERO DIAGONAL TERM EQUATION NUMBER        ,I4)
      END
```

```
      SUBROUTINE SOLVEQ(A,B,NN,LL,MAX,ID)
C
C
C     GENERAL EQUATION SOLVER
C
      DIMENSION A(MAX,MAX),B(MAX,1),ID(1)
C
C     SET I.D. ARRAY
C
      DO 50 N=1,NN
   50 ID(N)=N
C
      DO 475 N=1,NN
      N1=N+1
C
C     LOCATE LARGEST ELEMENT
C
      D=0.0
      DO 100 I=N,NN
      DO 100 J=N,NN .
      IF (ABS(A(I,J))-D) 100,90,90
   90 D=ABS(A(I,J))
      II=I
      JJ=J
  100 CONTINUE
C
C     INTERCHANGE COLUMNS
C
      DO 110 I=1,NN
      D=A(I,N)
      A(I,N)=A(I,JJ)
  110 A(I,JJ)=D
C
C     RECORD COLUMN INTERCHANGE
C
      I=ID(N)
      ID(N)=ID(JJ)
      ID(JJ)=I
C
C     INTERCHANGE ROWS
C
      DO 120 J=N,NN
      D=A(N,J)
      A(N,J)=A(II,J)
  120 A(II,J)=D
C
      DO 130 L=1,LL
      D=B(N,L)
      B(N,L)=B(II,L)
  130 B(II,L)=D
C
C     FORM B(N,L)
C
      DO 150 L=1,LL
  150 B(N,L)=B(N,L)/A(N,N)
```

```
C
C      CHECK FOR LAST EQUATION
C
       IF (N-NN) 200,500,200
C
  200  DO 450 J=N1,NN
C
C      FORM F(N,J)
C
       IF (A(N,J)) 250,350,250
  250  A(N,J)=A(N,J)/A(N,N)
C
C      MODIFY A(I,J)
C
       DO 300 I=N1,NN
  300  A(I,J)=A(I,J)-A(I,N)*A(N,J)
C
C      MODIFY B(I,L)
C
  350  DO 400 L=1,LL
  400  B(J,L)=B(J,L)-A(J,N)*B(N,L)
  450  CONTINUE
  475  CONTINUE
C
C      BACK-SUBSTITUTION
C
  500  N1=N
       N=N-1
       IF (N) 700,700,550
C
  550  DO 600 L=1,LL
       DO 600 J=N1,NN
  600  B(N,L)=B(N,L)-A(N,J)*B(J,L)
C
       GO TO 500
C
C      REORDER UNKNOWNS
C
  700  DO 950 N=1,NN
       DO 900 I=N,NN
       IF (ID(I)-N) 900,750,900
  750  DO 800 L=1,LL
       D=B(N,L)
       B(N,L)=B(I,L)
  800  B(I,L)=D
       GO TO 950
  900  CONTINUE
  950  ID(I)=ID(N)
C
       RETURN
C
       END
```

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