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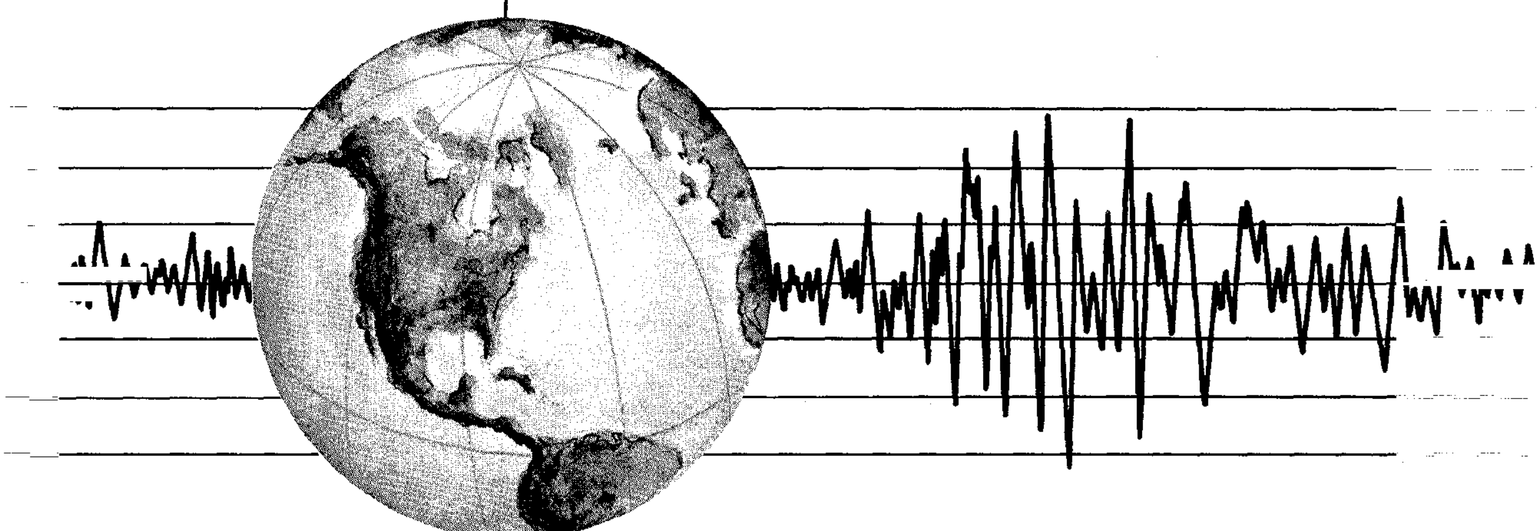
INPUT IDENTIFICATION FROM STRUCTURAL VIBRATIONAL RESPONSE

by
YUXIAN HU

Report to the National Science Foundation



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Shaking table tests of two three-story braced steel frames are used as examples to check the accuracy of the suggested input identification procedure. The bracing systems used show strong nonlinearities due to buckling and material yielding.

Sources of error in the analysis are discussed and possible improvements of the input identification procedure are suggested.

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ABSTRACT

The equivalent linearization method is applied to input identification from known structural response and system parameters, working in frequency domain. In addition to the commonly used one-model procedure, a two-model input identification procedure is proposed. Numerical examples of structural systems having elements of bilinear force-deformation relations are presented for different cases of stiffness, input motion and level of nonlinearity. Structural response obtained from true hysteretic loop analyses are used as the known responses with the unknown input motions to be identified. These identified motions are then compared with the real motions to check agreement with their maximum accelerations, response spectra, Fourier amplitude spectra, and acceleration time-histories. Comparisons of accelerations and spectra show differences generally less than 10% and the comparisons of time-histories show surprisingly good agreement even for cases involving strong nonlinearities having ductility factors in the range $6<\mu<8$.

Shaking table tests of two three-story braced steel frames are used as examples to check the accuracy of the suggested input identification procedure. The bracing systems used show strong nonlinearities due to buckling and material yielding. Comparisons of the identified and real input motions reveal differences that are in general small except for the large pulses which produce high levels of nonlinear displacement.

Sources of error in the analysis are discussed and possible improvements of the input identification procedure are suggested.

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1. INTRODUCTION

While the analysis of response of a known structure, linear or nonlinear, subjected to known dynamic excitations, may be referred to as the normal problem, the reversed procedure determining structural parameters or excitations from known structural response may be referred to as the inverse problem.

The inverse problem of determining structural parameters from given input and responses of the structure is known as structural identification. Structural identification was originally the application of system identification methods used successfully in electronic engineering field to structural engineering problems. It has been applied to the deduction, justification and improvement of analytical modelling of structures in the laboratory and in the field subjected to a vast variety of dynamical excitations, artificial or natural, stationary or transient. Many well-written reviews have already been published for structural dynamical problems in the last decade[5,7,10,15,18]. The fundamental idea of structural identification is to minimize, in the least squares sense, errors between the observed and computed structural dynamic responses by selecting the structural parameter in a systematic manner, i.e. the so-called output error approach.

The other inverse problem of identifying the input or excitation to a given structure from given structural responses is relatively slightly touched so far by Ibanez[11] on dynamic forces applied to structures, by Reimer et al[16] on Pacoima dam analysis, by Seed and Idriss[18] to obtain the bedrock motion in the case of horizontal soil layers and by the author[9] in 1978 in the analysis of strong motion records obtained on an earth dam. The input identification is useful either in finding the unknown excitation such as in the case of strong soil-structure interaction where the record obtained at the basement of a building cannot be considered as input to the building, or in finding the unknown excitation at bedrock where no records are obtained during a strong earthquake. The present report deals with this inverse problem of input identification.

In most earthquake engineering problems, nonlinear behavior of structures is inevitably induced; this is especially true when soils are considered as in the case of soil-structure interaction. The present report will be devoted primarily to nonlinear problems with linear problems treated as a special case.

Although methods of direct integration in the time domain and modal superposition[16] have been widely used in system identification problems, the method of superposition in the frequency domain will be used in the present report for ease and simplicity. This method has been used in the input identification of horizontal layers of soil deposits with success[18] but the identified input motions have not been compared with the real ones. The present paper tries to apply it to a more general problem including structures and to investigate the accuracy of identification by comparing the results with known input motions.

2. METHOD OF INPUT IDENTIFICATION

For linear cases where the principle of superposition may be applied, the responses and the input of a structure are related in the frequency domain by the simple relation

$$U(i\omega) \quad = \quad H(i\omega)\ V(i\omega) \qquad (2.1)$$

where U and V are complex Fourier spectra of the response u(t) at some location of the structure and the input motion v(t) applied at some location of the structure, respectively, and H is the transfer function relating response to input. Here, ω is frequency and $i = \sqrt{-1}$.

Evaluating terms in Eq. (2.1) by the Fast Fourier Transform (FFT), transient vibration of finite duration S_1 is considered to repeat together with a quiescent period S_2 following it as shown in Fig. 2.1. The quiescent part of the input motion is added to guarantee that the response u(t) of the structure repeatedly starts from the correct given initial conditions. Although zero initial conditions u(t=0) = v(t=0)=0 are generally used, other given initial conditions can also be considered by some specially selected motions within duration S_2. The motions of both response u(t) and input v(t) within the duration

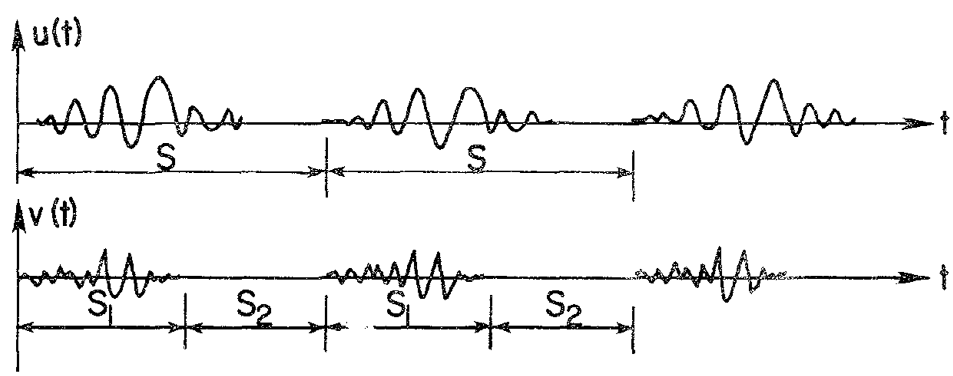


FIGURE 2.1 PERIODICITY OF TIME HISTORIES u(t) and v(t)

$0<t<s$ will be transient as shown in Fig. 2.1; however over the infinite duration they repeat with period s. This periodic condition could be thought of as steady state in a general sense.

Since there are only three complex frequency-dependent functions in Eq. (2.1), it may easily be used to determine any one of them when the other two are known. In the normal problem, $U(i\omega)$ is found given $V(i\omega)$ and $H(i\omega)$, in the structural or system identification problem, $H(i\omega)$ is found given $U(i\omega)$ and $V(i\omega)$ and in the input identification problem treated here, $V(i\omega)$ is found given $H(i\omega)$ and $U(i\omega)$; thus,

$$V(i\omega) = U(i\omega)/H(i\omega) \tag{2.2}$$

The computational work required to Fourier transform $U(t)$ to obtain $U(i\omega)$ and to inverse Fourier transform $V(i\omega)$ obtained from Eq.(2.2) would have been prohibitive 30 years ago, but the computational effort today is quite acceptable due to the availability of digital computers and the FFT algorithm.

Nonlinearity of a problem may be taken into consideration by the equivalent linearization method. In this case, the nonlinear force-displacement relationship is replaced by single-valued relations involving

$$\begin{aligned} K &= K(\gamma) \\ \xi &= \xi(\gamma) \end{aligned} \tag{2.3}$$

where γ is the strain or deformation of the element of the system and K and ξ are, respectively, the equivalent stiffness and damping ratio of that element. For many nonlinear vibrational problems, these equivalent parameters corresponding to a stress-strain loop of strain amplitude γ may be defined as shown in Fig. 2.2.

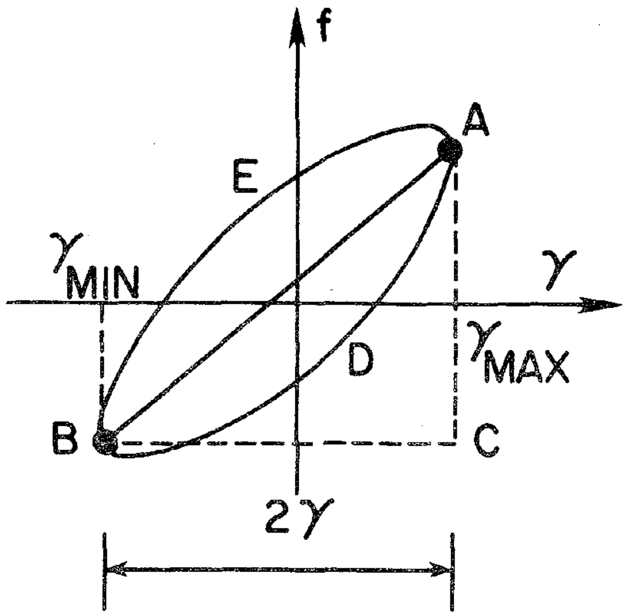


FIGURE 2.2 NONLINEAR STRESS-STRAIN RELATION

$$K = \text{slope of straight line AB} = \frac{AC}{BC}$$

$$\xi = \frac{2}{\pi}\ \text{loop area ADBE/triangular area ABC} \qquad (2.4)$$

$$\gamma = \frac{1}{2}\ (\gamma_{max} - \gamma_{min})$$

where points A and B are, respectively, points of maximum and minimum strain. In the case of single-valued functional relationships given by Eq. (2.3), the nonlinear properties of the problem may be considered by a standard iteration procedure which will be explained later.

Two procedures will be considered in this report, namely one-model and two-model input identification. The one-model procedure is the standard one that has been used in equivalent linearization in both normal and inverse problems while the two-model procedure is one suggested by the author in 1978 to be used in normal problems and input identifications. These two procedures will be discussed in detail in the following two sections.

3. ONE-MODEL PROCEDURE

In the one-model procedure of input identification, the whole process of real structure behavior is simulated by means of a single equivalent linear model. The entire procedure as represented in Fig. 3.1 can be explained as follows:

(1) The input identification starts with some given structural response $u_2(t)$ and given structural properties, including the equivalent stiffness $K(\gamma)$ and equivalent damping ratio $\xi(\gamma)$ of each element of the structure.

(2) A standard Fast Fourier Transform (FFT) is applied to obtain the Fourier spectrum $U_a(i\omega)$ of the given response history $u_a(t)$.

(3) With the given structural parameters and the given response, an estimate of strain level for each element will give a set of stiffnesses K and damping ratios ξ to start the iteration. Although the accuracies of the estimates of K and ξ are not critical, good estimates will give fast convergence of the iterations. Initial values corresponding to zero strain are used herein for reasons of simplicity.

(4) The transfer function may then be computed by solving a set of n simultaneous equations of the type

$$(-\omega^2[M] + i\omega[C] + [K])\{u_o\} = - [M]\{v_o\} \tag{3.1}$$

where $[M]$, $[C] = \alpha[M] + \beta[K]$, and $[K]$ are, respectively, the mass, damping and stiffness matrices of size nxn; $\{v\} = \{v_o\}e^{i\omega t}$

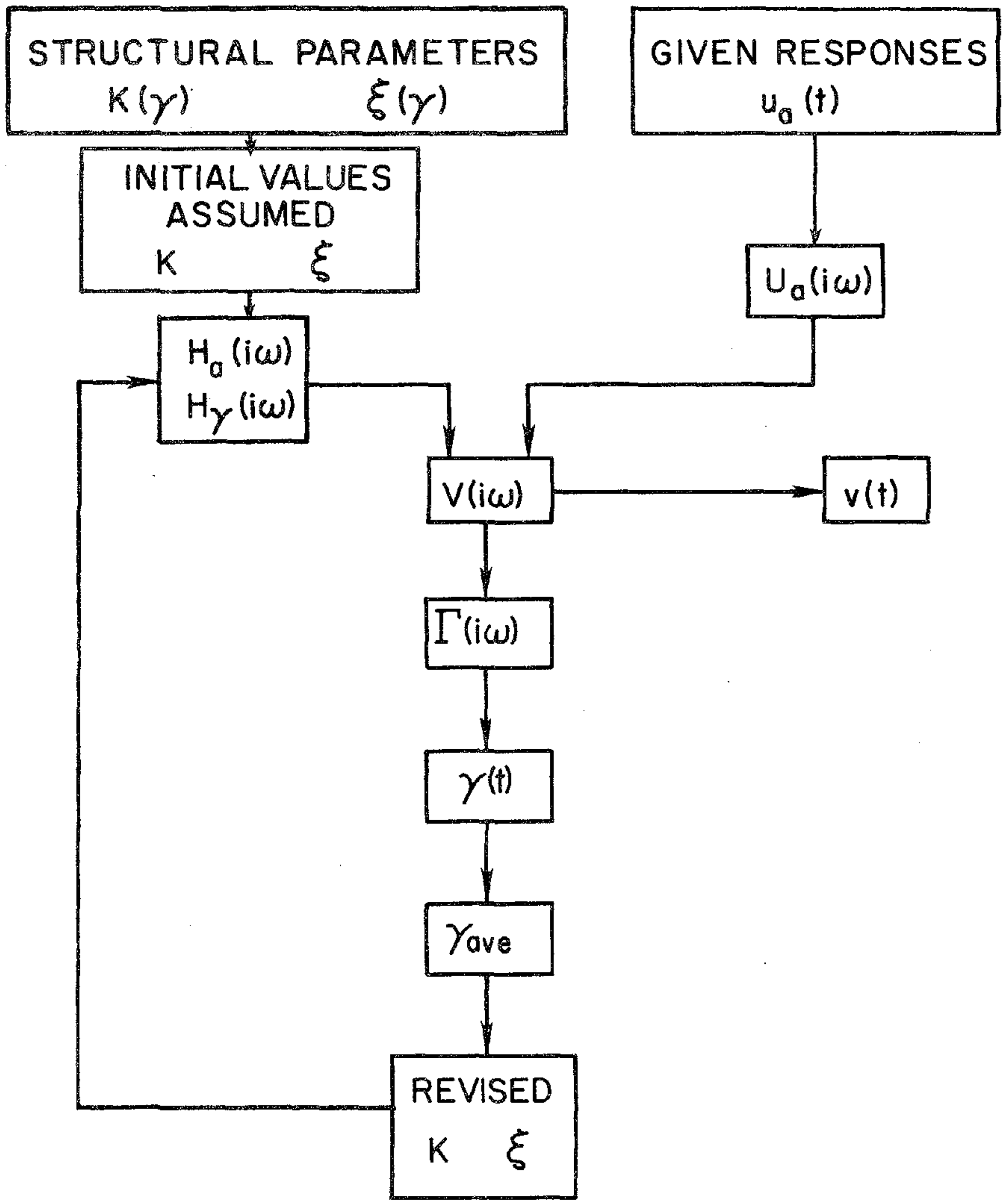


FIGURE 3.1 ONE-MODEL INPUT IDENTIFICATION PROCEDURE

and $\{u\} = \{u_o\}e^{i\omega t}$ are respectively input and response vectors. The n component vectors $\{v_o\}$ and $\{u_o\}$ are time-independent. Solution of Eq. (3.1) gives the displacement responses $\{u\} = \{u_o\}e^{i\omega t}$ of the structure subjected to harmonic inputs $\{v\} = \{v_o\}e^{i\omega t}$. The transfer function of displacement responses will, by definition, be as follows

$$H_{ab}(i\omega) = u_{oab}/v_{ob} \tag{3.2}$$

where v_{ob} is that part of $\{v_o\}$ representing the vector of input at b and u_{oab} is that part of $\{u_o\}$ representing the displacement at a due to v_{ob}. Transfer function $H_{ab}(i\omega)$ is complex since both u_{oab} and v_{ob} may be complex. It contains both the amplitude and phase shift of response resulting from the harmonic input, and it is, of course, a function of the forcing frequency ω.

Other transfer functions can easily be found from the solution $\{u_o\}$ for different response quantities such as accelerations or strains. The number of transfer functions may be more than the number of DOF of the system, but only n such functions are independent. Two sets of transfer functions are used here, namely $H_a(i\omega)$ for given responses $u_a(t)$ and $H_\Gamma(i\omega)$ for element strains.

(5) The input motion spectrum is calculated from the relation

$$V_b(i\omega) = U_a(i\omega) \;/\; H_{ab}(i\omega)$$

where $H_{ab}(i\omega)$ is the transfer function of response $u_a(t)$ to a unit harmonic input at b.

(6) The Fourier spectrum $\Gamma(i\omega)$ of the element strain history

$\gamma(t)$ is computed from the relation

$$\Gamma(i\omega) = H_\Gamma(i\omega)V(i\omega) \tag{3.3}$$

where $H_\Gamma(i\omega)$ is the transfer function of the element strain $\gamma(t)$ defined in (4) above.

(7) Element strain history $\gamma(t)$ is then obtained from $\Gamma(i\omega)$ by an inverse FFT for each element.

(8) Some average strain γ_{ave} is obtained from $\gamma(t)$ by first finding the maximum absolute value of γ_i for the segment of $\gamma(t)$ between the i^{th} and $(i+1)^{th}$ zero-crossing as given by

$$\gamma_i = \max |\gamma(t)| \qquad t_i \le t < t_{i+1} \tag{3.4}$$

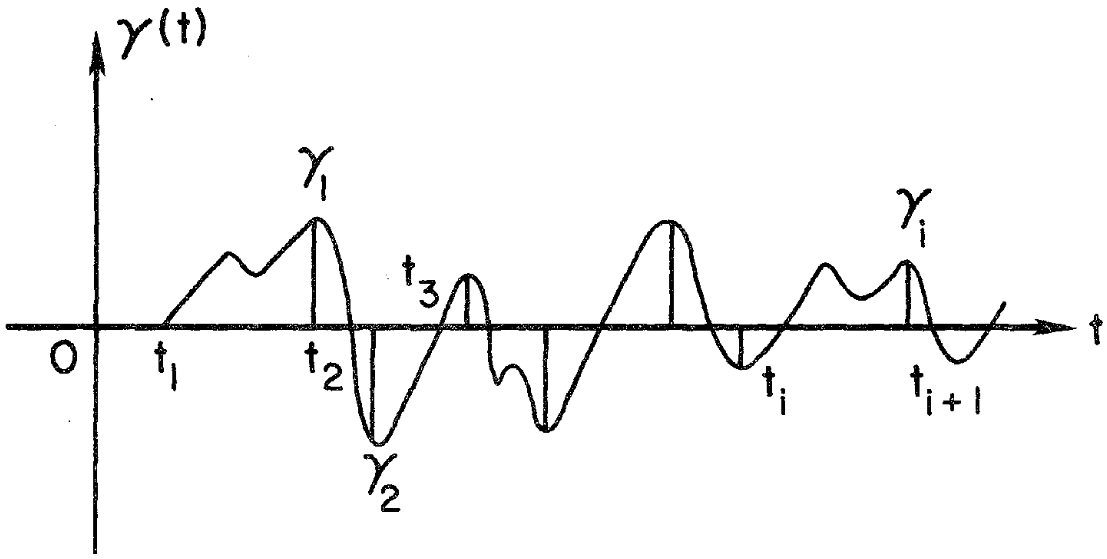


FIGURE 3.2 STRAIN AMPLITUDE

and then by averaging the top m values of γ_i to obtain

$$\gamma_{ave} = \frac{1}{m}\sum_{i=1}^{m}\gamma_i \tag{3.5}$$

where $\sum\gamma_i$ is a descending array rearranged from the γ_i's obtained above. Professor Seed and his colleagues use the fixed value $\gamma_{ave} = 0.65$.

(9) Since the equivalent element parameters K and ξ are single-valued functions of element strain ξ, a new set of K and ξ are obtained for each element as

$$K = K(\gamma_{ave})$$

$$\xi = \xi(\gamma_{ave})$$

which will then be used to start the next iteration.

(10) Repeat the foregoing steps (3) - (9) until two sets of consecutive values of γ_{ave}, or K and ξ, are considered sufficiently close. In this report, a maximum difference of 2% is used.

(11) After the completion of iteration, the identified input history v(t) can be obtained from V(iω) by an inverse FFT and any response of the structural system can then be obtained therefrom.

Since the problem of equivalent linearization analysis applied to nonlinear problems is well known, only certain unique features of the present analysis will be described in the subsequent sections.

3.1 Cut-Off Frequencies

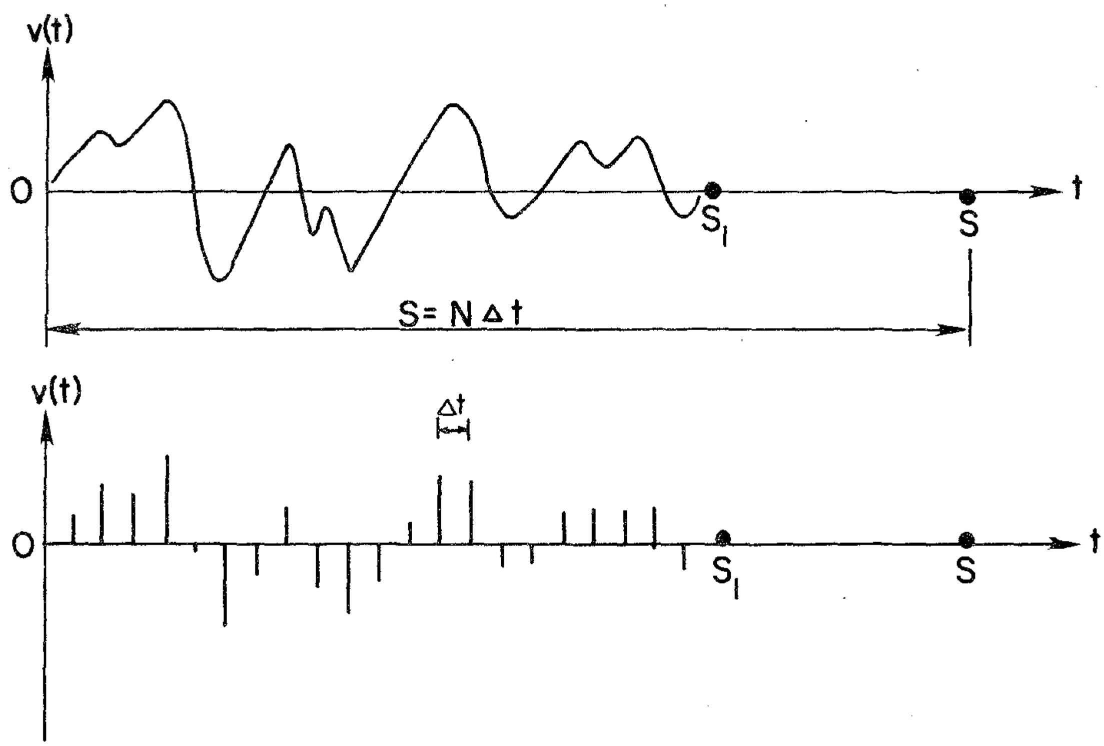


FIGURE 3.3 DISCRETIZATION OF PROCESS v(t)

In the numerical analysis procedure, we have to deal with a digital discretization of a continuous function v(t) of finite duration s as shown in Fig. 3.3, where the time step Δt is very small but finite. Inherent from the nature of this discretization and finite duration, only components of frequencies within the range $f_u = \frac{1}{2\Delta t}$ (the Nyquist, folding or aliasing frequency) and $f_\ell = \frac{1}{2s}$ can be found in the Fourier transformation $V(i\omega)$ of v(t). Components outside this range will not appear.

In addition to the limitations introduced from the cut-off frequencies f_u and f_ℓ, it is necessary to introduce, in the present analysis, further limitations to the cut-off frequencies.

A necessary high cut-off frequency f_{ui} is introduced to prevent undesirable amplification of the high-frequency responses introduced through inaccuracies of the numerical procedures used. Errors generally exist in test data due to noise and instrumental limitations and discrepancies certainly exist in numerical analysis between the actual structural system and its mathematical model used in identification, especially in equivalent linearization. Although the spectral distribution of these noises or discrepancies in $U_a(i\omega)$ is in general not known, it is certain that they are to be amplified in $V(i\omega)$ by a factor of $\frac{1}{H(i\omega)}$ as shown in Eq. (2.2). Since transform function $H(i\omega)$ is simply a resonance curve of the structural system, it approaches zero on the high frequency tail away from the natural frequencies of the system; thus, the factor $1/H(i\omega)$ becomes very large. A cut-off frequency f_{ui} is therefore required to limit this kind of undesirable amplification of errors at a sacrifice of losing the information of frequencies beyond it in the input motion. This cut-off frequency should be different for different problems, but for all the numerical cases given in this report, it is taken as $f_{ui} = \frac{130}{20.48} = 6.5$ Hz for simplicity.

A low cut-off frequency $f_{\ell i}$ is introduced for nearly the same reason, except that the discrepancy between the actual structural system and its equivalent-linearized model is much more pronounced in low-frequency components as shown in Fig. 3.4. In general, the strong nonlinearity in responses $u(t)$ shows itself as a low frequency character, since nonlinear displacement generated in the process of nonlinear response tends to remain there for some time, such as shown by the dotted line in Fig. 3.4 and appears obviously as permanent displacement on later part of the vibration. This nonlinear drift shown by the dotted line in

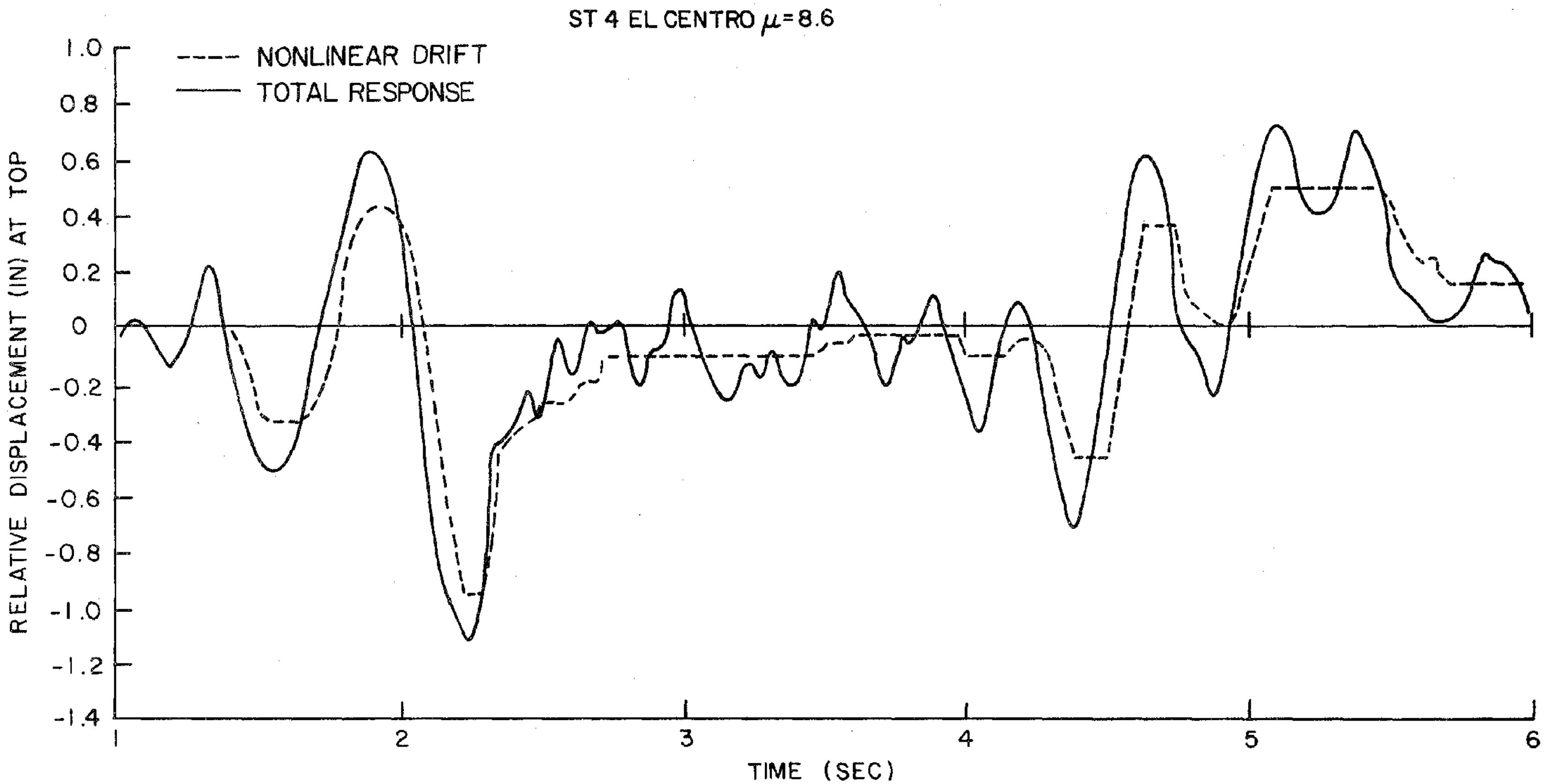


FIGURE 3.4 RESPONSE WITH NONLINEAR DRIFT

Fig. 3.4 will result in large Fourier amplitudes in the low frequency range. Two measures are taken to reduce errors due to this nonlinear drift in the low frequency range, namely a low frequency cut-off and a shift of base line in the given response $u_a(t)$. The low cut-off frequency is taken as $f_{\ell i} = \frac{15}{20.48} = 0.75$ Hz based on experience. The base line change in given response $u_a(t)$ representing permanent drift is selected in the form

$$\begin{aligned} u(t) &= u_a(t) & o \leq t < S_3 \\ &= u_a(t) - \frac{t-S_3}{S-S_3} u_p & S_3 \leq t \leq S \end{aligned}$$

where $S_3 < S$ is chosen arbitrarily and u_p is estimated from the given response curve. This base line change should not include large high frequency components unless there is definite evidence to justify doing so.

As a consequence of the necessity of introducing cut-off frequencies f_{ui} and $f_{\ell i}$, the input motion can only be identified within the range between $f_{\ell i}$ and f_{ui}, which is of course a serious limitation of the procedure suggested here. However, the effect of frequency cut-off on the original input history and response spectrum is in general not very large as shown in the figures given in Appendix A; however, it can seriously affect the peak values in some input time histories where high frequency components are pronounced. For example, the Pacoima records in Appendix A may have peak values reduced to a half. In this case, the response spectrum is almost unchanged in the period range 0.2 - 1.2 sec.

3.2 Averaging Strain or Averaging Stiffness

During earthquake-type strong nonlinear vibrations, the hysteresis loop diagram of the vibration history may be very complex as shown in

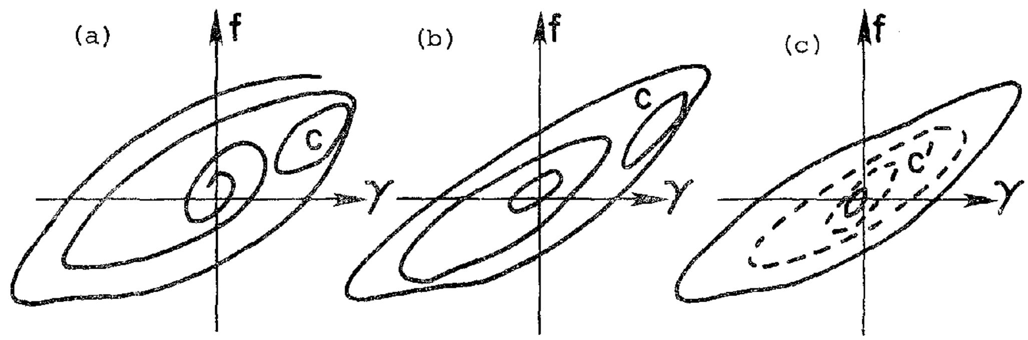


FIGURE 3.5 SIMPLIFICATION OF HYSTERETIC LOOPS

Fig. 3.5(a). The only pattern recognizable in this figure may be its loops, although they are rather irregular. These irregular loops may be smoothed at first into those as shown in Fig. 3.5(b), and any loop c that is not centered at the origin can be shifted to the origin as indicated by c'. This smoothing process will affect only certain details of the vibration while the shifting corresponds to ignoring nonlinear drift from the origin to the center of loop c; thus, affecting only the low frequencies in the input motion. The resulting simplification will then be a series of smooth loops as shown in Fig. 3.5(c). For elements where these simplified hysteresis loops apply, the equivalent stiffness and damping are considered as single-valued functions of strain amplitude.

During a whole process of vibration, each element will usually undergo many loops of various strain amplitudes in a random fashion. Since each loop of different strain amplitude gives a definite pair of values for equivalent stiffness K and damping ξ, there will be many different pairs of stiffness and damping at different strain amplitudes. However, for the usual one-model equivalent linearization procedure, only one equivalent linear pair is used for a single element in representing overall behavior. This points to the serious problem, "What equivalent pair should be used?".

First of all, this equivalent pair of stiffness and damping should be some kind of average of all actual pairs corresponding to some average value of strain amplitudes experienced in the process. Prof. Seed[17] and others suggested using values corresponding to an average strain amplitude γ_{ave}

$$\gamma_{ave} = C\,\gamma_{max} \tag{3.6}$$

where γ_{max} is the maximum, absolute value of the strain history $\gamma(t)$ and the coefficient c is taken as 0.65 or $\frac{2}{3}$ for all input motions and structures.

This coefficient c or the relation of γ_{ave} with respect to the actual strain amplitudes is supposed to reflect the transient nature of vibration. As shown in Fig. 3.6, $\gamma_1(t)$ shows a rather stationary

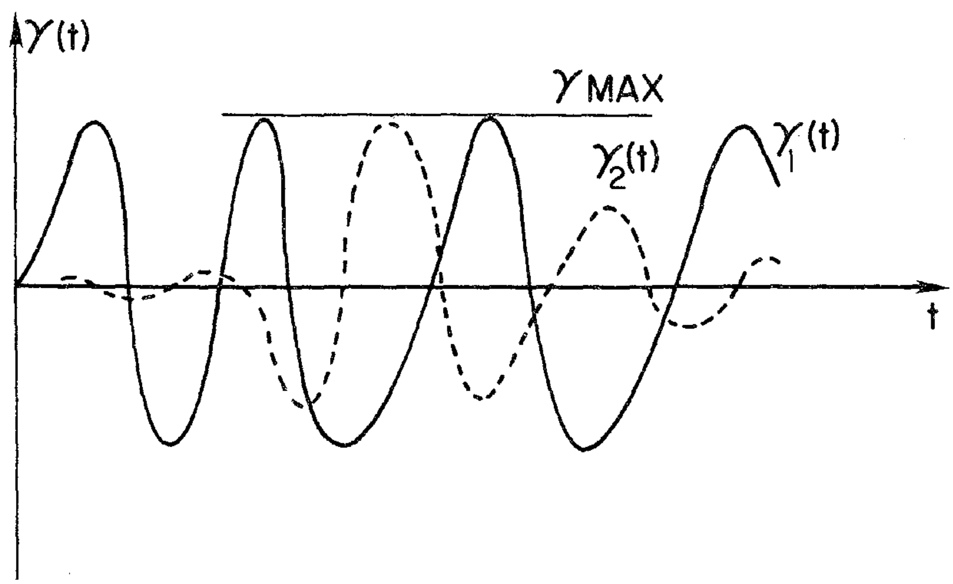


FIGURE 3.6 STATIONARITY OF AMPLITUDE OF VIBRATION

or steady-state vibration where the strain amplitudes of all loops are about equal, γ_{ave} should be near or equal to γ_{max} and $C \cong 1$; but $\gamma_2(t)$ shows a nonstationary or transient vibration where strain amplitudes of all loops are quite different and the maximum amplitude appears only once. In this case, γ_{ave} should be much smaller than γ_{max}. It is for this reason that Eq. (3.5) is suggested in defining γ_{ave} rather than Eq. (3.6). If m in Eq. (3.5) is kept constant, coefficient c will equal 1 for $\gamma_1(t)$ and a much smaller value for $\gamma_2(t)$. After some numerical trials, a numerical value of 10 for m has been found most suitable in the one-model procedure. The corresponding values for coefficient c in Eq. (3.6) for the numerical examples used here are given in Table 5.2. They range from 0.55 to 0.86 for m=10 and from 0.23 to 0.46 for m=60.

4. TWO-MODEL IDENTIFICATION

Although the one model identification procedure mentioned above works for many cases, it fails sometimes for high levels of nonlinearity as represented by ductility factors in the range 6–8. Figure 4.1 shows such a case for ST4 (See Tables 5.1 and 5.2). Curves in (a) are the Fourier amplitude spectra of responses at the top of a three-story frame subjected to the El Centro input for ductility factors μ =1.8 and 8.6, respectively, while those in (b) are the original input and the inputs identified from the responses, with the abscissa given by a frequency sequential number K of the frequency expression $\omega = K \frac{2\pi}{S}$ with S = 20.48 sec. At K = 68 which corresponds to the linear fundamental period T_1 = 0.30 sec, there is a very prominent peak in the Fourier amplitude spectrum for μ = 1.8 but only a much smaller peak for μ = 8.6. In Fig. 4.1(b), it can be seen that the identified input spectrum for μ = 1.8 is very close to the real one in the whole range of K = 10 to 140 (corresponding to 2.0 to 0.16 sec) and the corresponding equivalent linear fundamental period T_1' is 0.32 sec. This close match of spectra suggests a close match of time histories. But on the other hand, the identified input spectrum for μ = 8.6 is rather different from the real one in the following manner, higher in both lower ($K < 15$) and higher ($K > 90$) frequency regions and also near the linear fundamental frequency.

The identifications were made by using one-model procedure as outlined above with m = 10. Different values of m will give different equivalent linear structures with different identified input spectra, but with no better results. A smaller value of m will give a more flexible equivalent structure with its fundamental period $T_1' > 0.45$ sec,

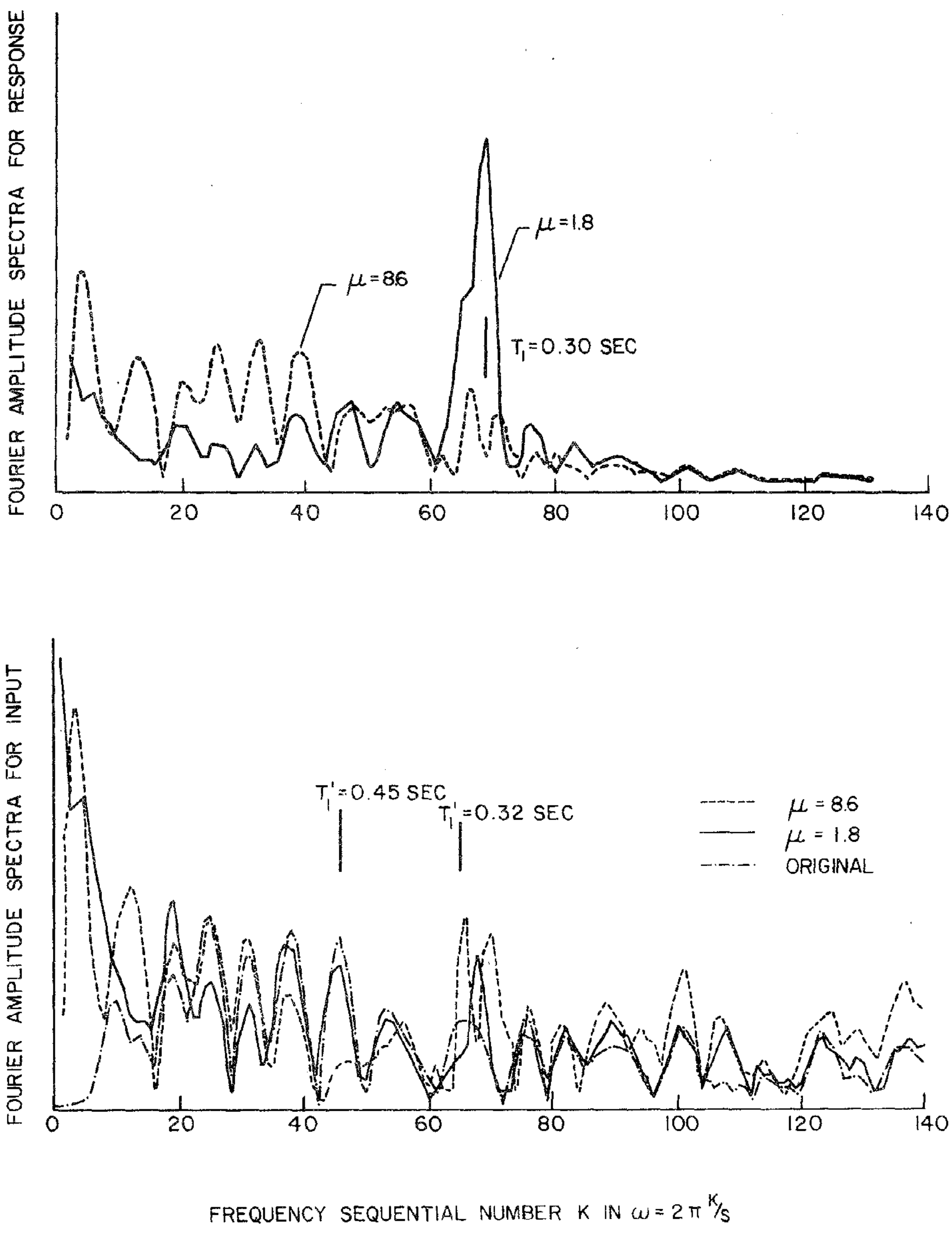


FIGURE 4.1 FOURIER SPECTRA OF INPUT AND RESPONSE

and the amplitudes for $T > 0.45$ sec or $K < 46$ will be smaller than those for $\mu = 8.6$ in Fig. 4.1(b). However, the amplitudes for $T < 0.45$ or $K > 46$ will be greater which means consistently greater discrepancies near T_1 and within high frequency portion. A greater value of m will reduce the discrepancies near T_1 and within the high frequency portion, but it will increase the discrepancies in the low frequency portion.

These discrepancies are believed to come from the over-simplification of using one linear system as the equivalent of the actual non-linear structure. A two-model identification procedure is therefore introduced here to reduce the discrepancy.

The sketch in Fig. 4.2 shows the procedure of the two-model input identification suggested. It starts the same as the one-model procedure from given structural parameters $K(\gamma)$ and $\xi(\gamma)$ for each element and some given structural response $u_a(t)$; however, two equivalent linearization models are worked out separately for $m = m_1$ and $m = m_2$ resulting in two identified input Fourier spectra $V_1(i\omega)$ and $V_2(i\omega)$, respectively. In this report, $m_1 = 10$ and $m_2 = 60$ are used temporarily. The second step is to combine these two models into one $V(i\omega)$ by adopting the low frequency part NL - NM from $V_1(i\omega)$ of the m_1 model and the high frequency part NM - NH from $V_2(i\omega)$ of the m_2 model, where NL, NM and NH are respectively sequential numbers K of the low cut-off, some medium, and the high cut-off frequencies as in frequency expression $f = 2\pi k/S$ where S is duration of vibration. In the present report, NL, NM and NH are temporarily taken as 15, 35, and 130, respectively, for $S = 20.48$ sec. The final step is then to have the identified input history $v(t)$ by an inverse FFT from the combined $V(i\omega)$.

The idea of the two-model identification is to use one flexible

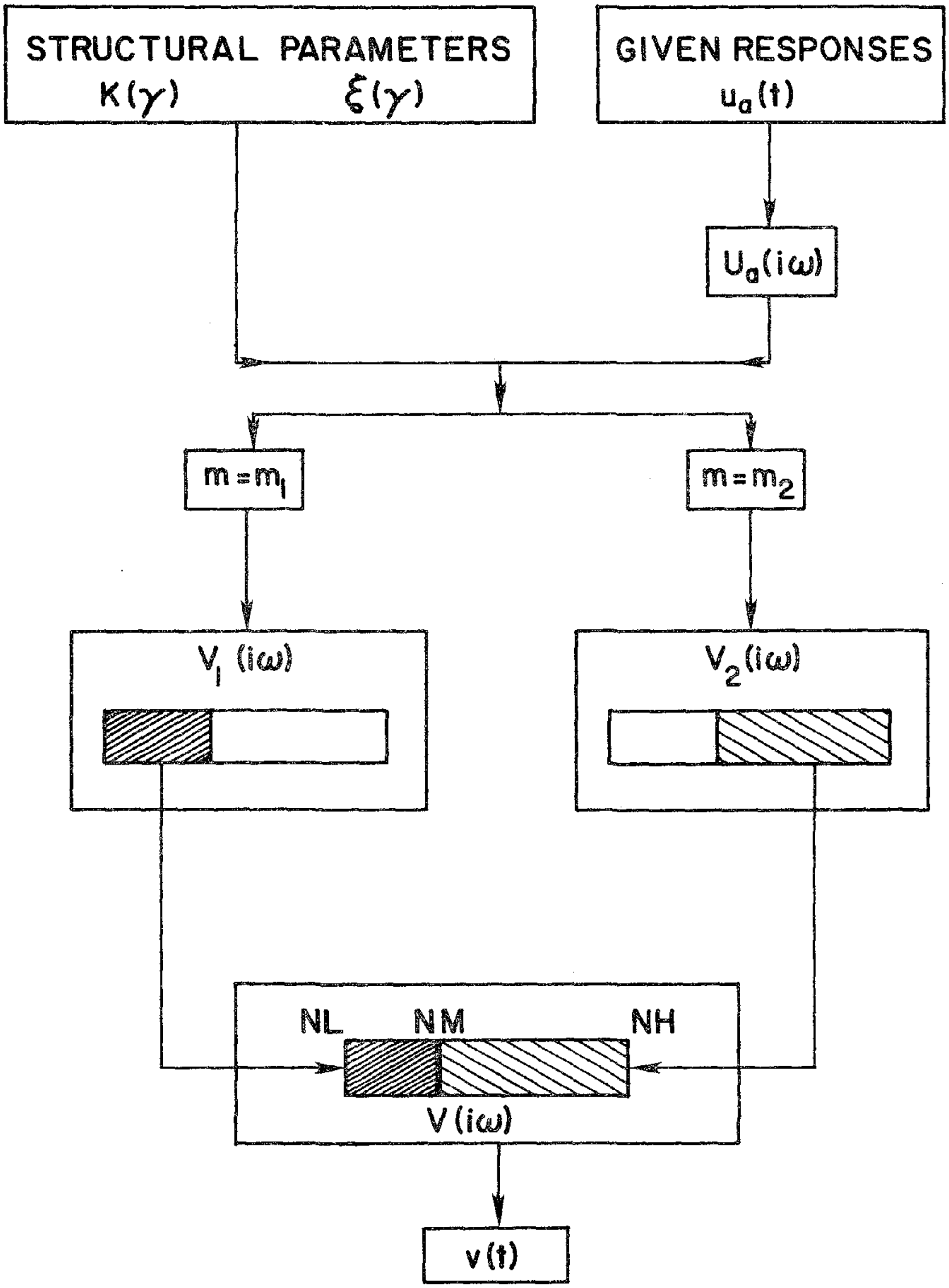


FIGURE 4.2 TWO-MODEL INPUT IDENTIFICATION PROCEDURE

equivalent model structure to identify the low-frequency portion of the input and another stiffer equivalent model structure to identify the high-frequency portion of the input. This idea is founded on the following two considerations.

Firstly, judging from Fig. 4.1 and many others, as explained at the beginning of this section, there is a consistent and strong general trend in the structural responses that the higher the level of nonlinearity, the larger the amplitudes of the lower frequency components, and the smaller the amplitudes of the higher-frequency components but always with some peaks near the linear fundamental frequency although much smaller in the amplitudes. This phenomenon seems to suggest that there is a qualitative connection between low-frequency vibration and non-linear deformation in structures of softening nonlinearity.

Secondly, as shown in Fig. 3.5, a larger strain amplitude in the free vibration does show a smaller equivalent stiffness of the structure and hence a lower frequency of vibration. Although vibrations due to strong earthquake motions are in general not free vibrations, but vibrations of the forced transient type, we may interpret them from the point of view of equivalent linear systems, i.e. the equivalent structural stiffness will have a strong influence on the transient vibration. Strong motion observation data reveal that the vibration of structures under even minor nonlinear deformations show response time histories containing longer predominant periods.

Based on these two considerations, it is believed that there is at least a general effect of the nonlinear deformation to shift the vibration to lower frequencies, although equivalent linearization is only a rough overall approximation of the complex structural vibrations during strong earthquakes. The two-model identification idea suggested here is a first step improvement over the one-model identification by taking into consideration this relation between nonlinearity and frequency content of vibrations.

5. NUMERICAL EXAMPLES

Some numerical examples and experimental data are presented here to show how the proposed input identification procedure works.

In all the numerical examples, a three degree-of-freedom (3 DOF) structure as shown in Fig. 5.1 is used to approximate, but not to simulate a 3-story frame tested on the shaking table at the Earthquake Engineering Research Center of the University of California at Berkeley [6, 12, 13]. This structure is subjected to horizontal ground motion at node 4, and all elements have deformation only in shear. Structural properties of the elements of the mathematic model in Fig. 5.1 and the corresponding ones of the test frame are given in Table 5.1.

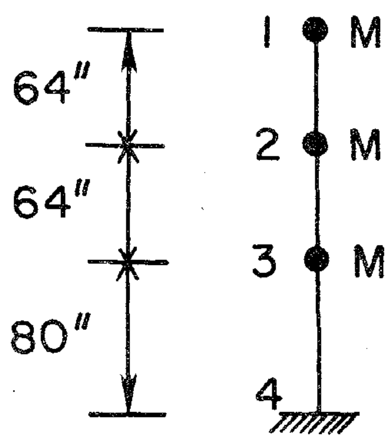


FIGURE 5.1 MATHEMATIC MODEL USED IN NUMERICAL EXAMPLES

The stiffness values given in Table 5.1 correspond to very small strains. Masses lumped at all nodes are $M = 0.01024$ kips-sec^2/in. The linear natural frequencies are 2.8, 8.4 and 17.8 Hz for the test frame, and 3.45, 10.59 and 18.0 Hz for the corresponding numerical examples for the case of $K_{23} = 31.7$.

TABLE 5.1 STRUCTURAL PARAMETERS OF MATHEMATIC MODELS

Structure	Stiffness (kips/in.)			Linear Fundamental Period (Sec)	p	α	β
	Element 1-2	2-3	3-4				
Test Frame Calculated	57.7	57.7	29.6	.4		.234	.0003
Test Frame* Identified	24.33 -37.40	58.16 -59.01	19.18	.4		.308	.00015
Mathematic Model ST1	31.7	31.7	18.1	.30	.3	1.0	.0002
ST2	7.66	12.0	4.38	.59	.1	1.0	.0002
ST3	2.19	3.44	1.25	1.14	.1	1.0	.0002
ST4	31.7	31.7	18.1	.30	.1	.3	.0002
ST5	31.7	31.7	18.1	.30	.1	1.0	.0002
ST6	31.7	31.7	18.1	.30	.25	1.0	.0002

* Full stiffness matrix is identified

TABLE 5.2 CASES OF NUMERICAL EXAMPLES IDENTIFIED

Example	Linear T_1 (sec)	σ_y (kips/in.^{2})			Input	μ_{12}	μ_{23}	μ_{34}	m	c	Id. T_1 (sec)	RK_{34}	ξ_{34}
		σ_{y12}	σ_{y23}	σ_{y34}									
ST1	.30	270	47	55	El Centro	--	7.1	3.5	10	.76	.36	.61	.18
									60	.36	.30	1.00	0
ST2	.59	55	40	20	"	--	--	7.8	10	.68	1.02	.33	.33
									60	.34	.73	.59	.24
ST3	1.14	55	50	10	"	--	--	7.2	10	.61-.81	1.46	.53	.27
									60	.46	1.20	.79	.13
ST4	.30	90	90	120	"	--	--	1.8	10	.86	.32	.87	.08
									60	.43	.30	1.00	0
ST4	.30	55	55	55	"	--	--	4.5	10	.67	.39	.51	.28
									60	.30	.30	1.00	0
ST4	.30	60	60	45	"	--	--	8.6	10	.55	.45	.39	.32
									60	.23	.32	.93	.05
ST4	.30	90	90	120	2X El Centro	--	--	3.5	10	.70	.38	.57	.25
									60	.36	.30	1.00	0
ST5	.30	40	40	25	Taft	--	--	8.0	10	.63	.46	.38	.32
									60	.31	.33	.83	.10
ST5	.30	150	150	110	Pacoima	--	--	6.3	10	.72	.42	.46	.30
									60	.39	.32	.88	.08
ST6	.30	30	30	40	Taft	7.1	6.4	3.3	10	.72	.35	.65	.18
									60	.32	.30	1.00	0

The nonlinearity of elements of the mathematical models is taken as bilinear, with the slope of the second straight portion equal to p-times that of the first portion and with symmetry assumed about the two directions of deformation.

Table 5.2 summarizes all of the cases studied for the 3 DOF system.

The procedure used for the numerical examples of input identification is as follows:

(1) Compute the relative displacement u(t) of the top mass of the given structure due to a given input time-history using program DRAIN-2D with a time step DT = 0.02 sec. For some examples, DT = 0.005 sec has also been used to check the accuracy of the results using DT = 0.02 sec. Figure 5.2 shows a comparison for ST4 with $\mu = 8.6$ which gives the largest difference among the cases compared. This difference is not large; therefore a time step DT = 0.02 sec is used for all examples.

Some important parameters of the four input motions used here are listed in Table 5.3.

TABLE 5.3 PARAMETERS OF INPUT MOTIONS

Earthquake	Duration (sec)		A_{max}/g	Predominate Period (sec)
	S_1	S		
El Centro	6.0	20.48	.34	.25, .45 - .6
2X El Centro	6.0	20.48	.69	.25, .45 - .6
Taft	12.6	20.48	.18	.45
Pacoima	13.0	20.48	1.15	.2, .4

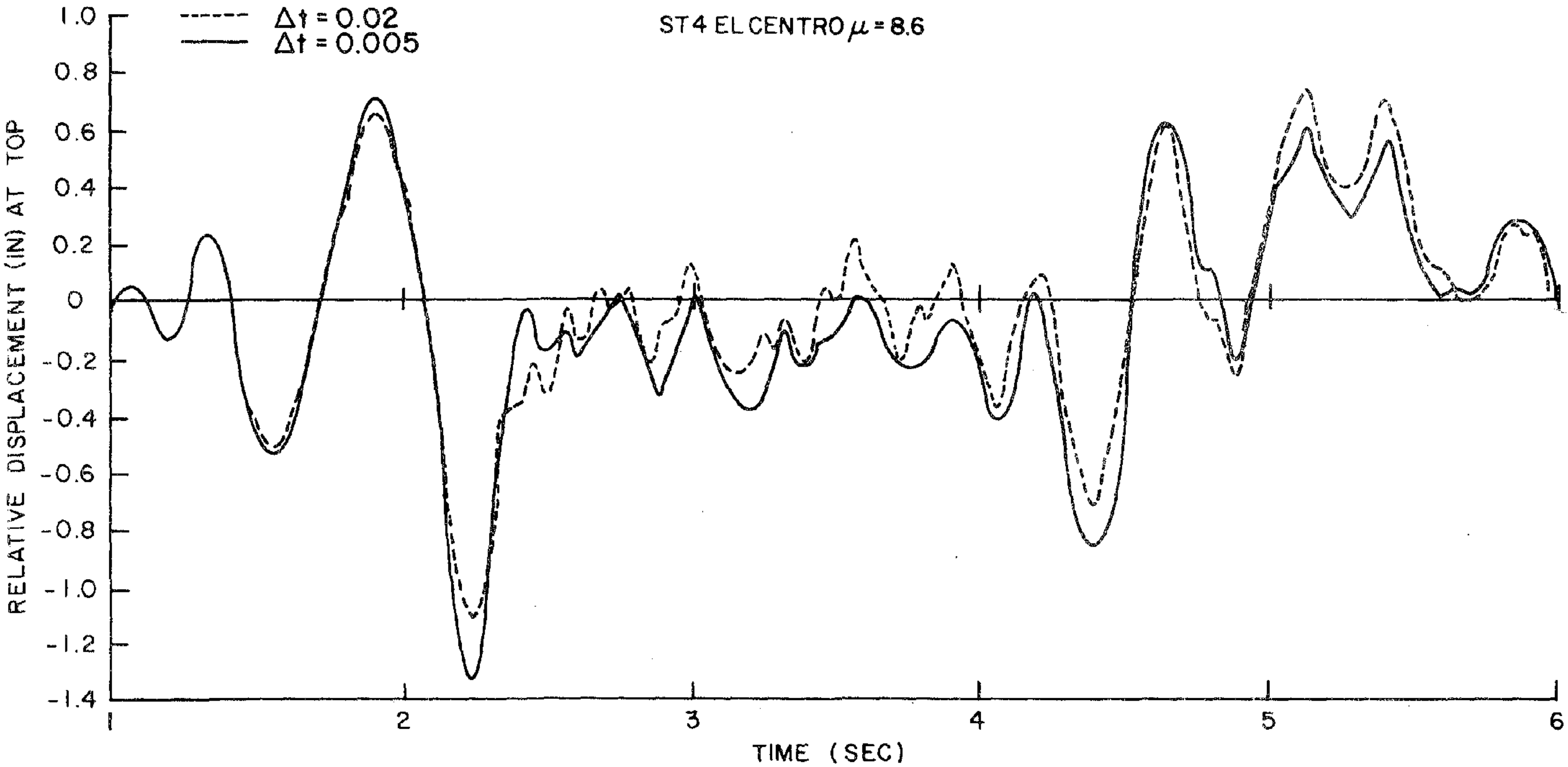


FIGURE 5.2 COMPARISON OF NONLINEAR RESPONSES FOR Δt = 0.02 and 0.005 SEC

(2) Relations for the equivalent stiffness K_{eqv} and damping ratio ξ_{eqv} as functions of element strain γ have been reduced from the bilinear stress-strain curve to give

$$c = K_{eqv}/K = p + (1-p)X$$

$$\xi_{eqv} = \frac{2}{\pi} \circ \frac{(1-p)X(1-X)}{(1-p)X + p}$$

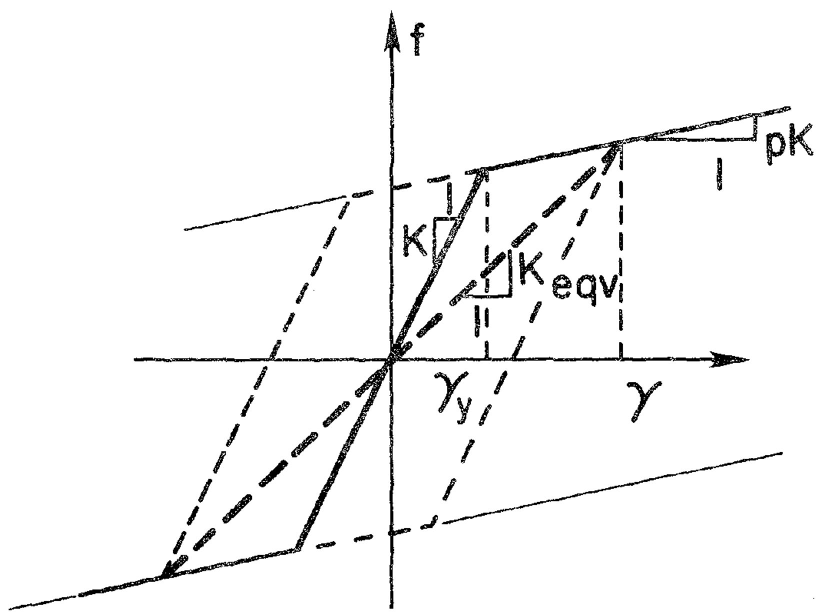


FIGURE 5.3 EQUIVALENT STIFFNESS FOR BILINEAR NONLINEARITY

where $X = \gamma_y/\gamma$ is the ratio of strain at the elastic limit to that at maximum and where p is the ratio of slopes of the 2nd to 1st straight lines of the bilinear property of the material, as shown in Fig. 5.3.

(3) Input identification is then performed from the computed responses u(t) at the top mass and the given structural properties as outlined in Fig. 4.2 by the two-model procedure, with some cases also by the one-model procedure. The one-model identification works

TABLE 5.4 PARAMETERS OF TEST EXAMPLES

Test	Bracings	M kips-sec^2/in.	a_{max}/g	m	c	Ident. T_1	RK Element 1-2	RK Element 2-3	RK Element 3-4	ξ_{eqv} Element 1-2	ξ_{eqv} Element 2-3	ξ_{eqv} Element 3-4
P400	Pipe	.024	.48	10	.56	.41	.69	.62	.66	.10	.08	.13
				60	.19	.37	.92	.84	.78	.25	.08	.14
P800	Angle	.024	1.31	10	.55	.60	1.00	.81	.22	.10	.10	.17
				60	.19	.36	1.00	.84	.81	.10	.10	.10

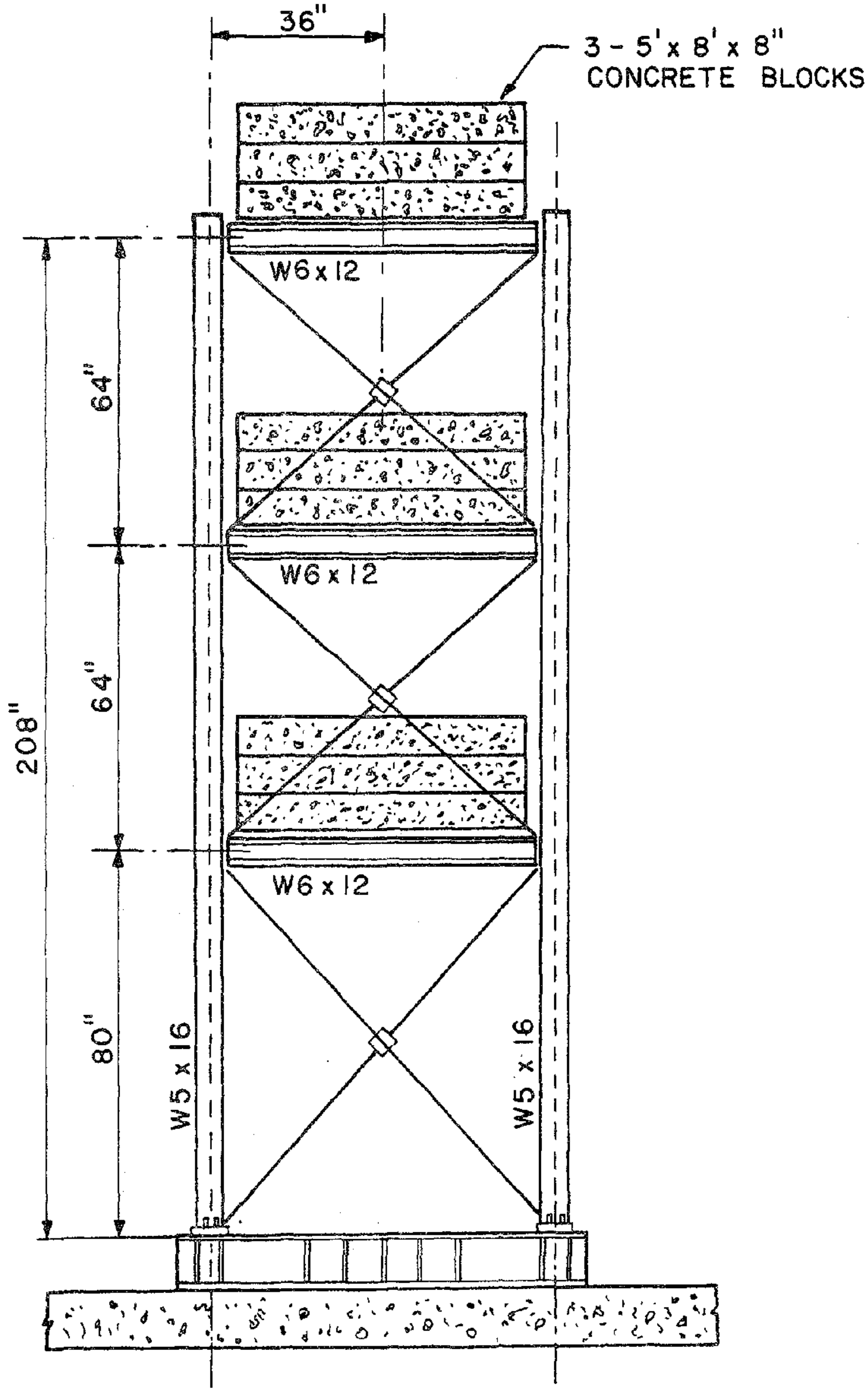
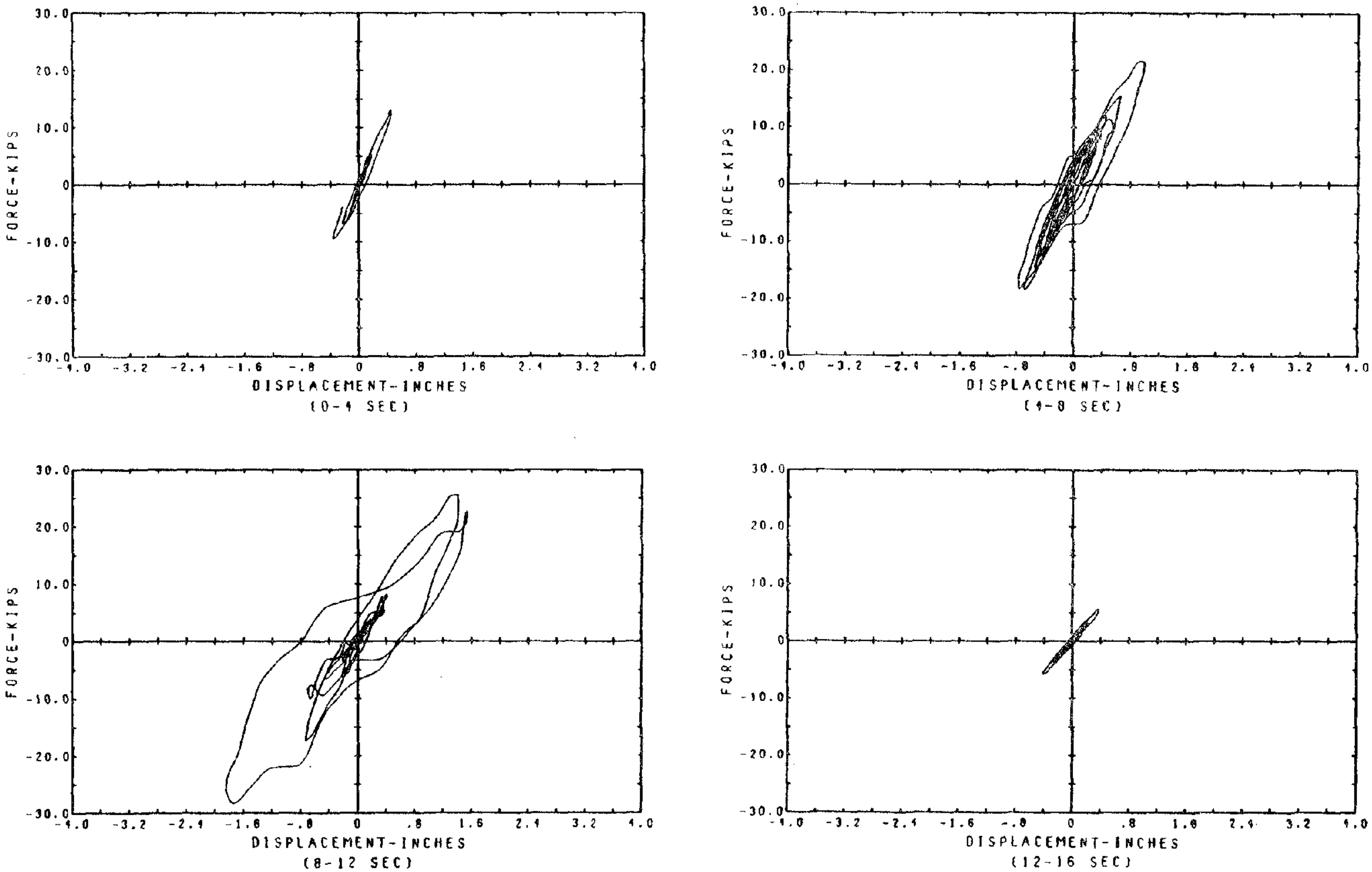
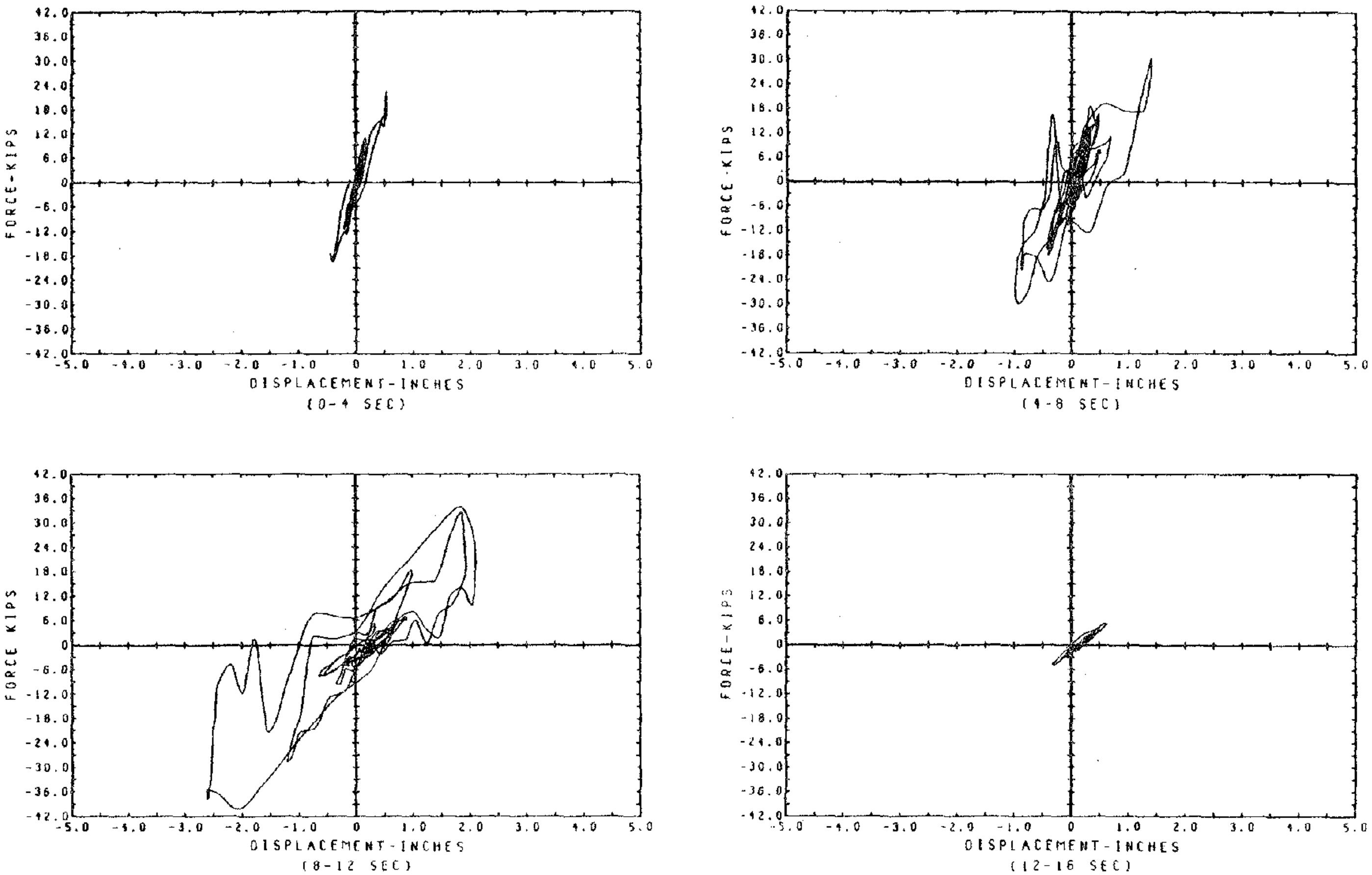


FIGURE 5.4 TESTED FRAME (AFTER Y. GHANAAT)



1ST FLR SHEAR VS. DISPLACEMENT
PIPE DIAGONALLY BRACED STRUCTURE
PACOIMA SPAN 400

FIGURE 5.5 FIRST-FLOOR SHEAR-DISPLACEMENT LOOP OF P400 (AFTER Y. GHANAAT)



1ST FLR SHEAR VS. DISPLACEMENT
REFERENCE FRAME N
DOUBLE ANGLE DIAGONALLY BRACED STRUCTURE
PACOIMA RECORD-1.314 G'S

FIGURE 5.6 FIRST-FLOOR SHEAR-DISPLACEMENT LOOP OF P800 (AFTER Y. GHANAAT) only for certain cases while the two-model identification works for all cases; therefore, the two-model procedure is recommended.

(4) Comparison of the identified input to the real input is made for three quantities, namely acceleration time history $a_g(t)$, absolute acceleration response spectrum SA(T) for damping ratio 0.05, and Fourier amplitude spectrum FAG(ω). Results are given in Figs. A1-A11 at the end of this report. Discussions and conclusions from these results will be given in the next section.

Several steel frames, with and without bracings, were tested on the shaking table at the Earthquake Engineering Research Center of the University of California, Berkeley, in the last few years[2,3,6]. A series of early tests[3] was carried out to rather small nonlinear levels. For these tests, a linear model with equivalent damping can be used as a good mathematical model to predict the responses. A series of recent tests[2,6] of frames (Fig. 5.4) with various types of bracings, however, produced rather strong nonlinear deformation due to the alternating buckling and yielding of the bracings. Two of them with strong nonlinear deformations (Figs. 5.5 and 5.6) are chosen here to test the input identification procedure suggested, with related parameters given in Table 5.4 and Figs. 5.4-5.6. All data presented here have been obtained from Mr. Y. Ghanaat. For further details, the reader is referred to his original report[6].

The identification procedure used is the same as before except for the reductions of the functional relations of equivalent stiffness and damping ratios. These reductions were carried out as follows:

The functional relations of equivalent stiffness K_{eqv} and damping ratio ξ_{eqv} are reduced from the given test results of time histories of story shear V(t) and story relative displacement d(t) for each story of each test structure. Shown in Fig. 5.7 is one complete loop of the V-d diagram plotted from V(t) and d(t) provided by Mr. Y. Ghanaat.

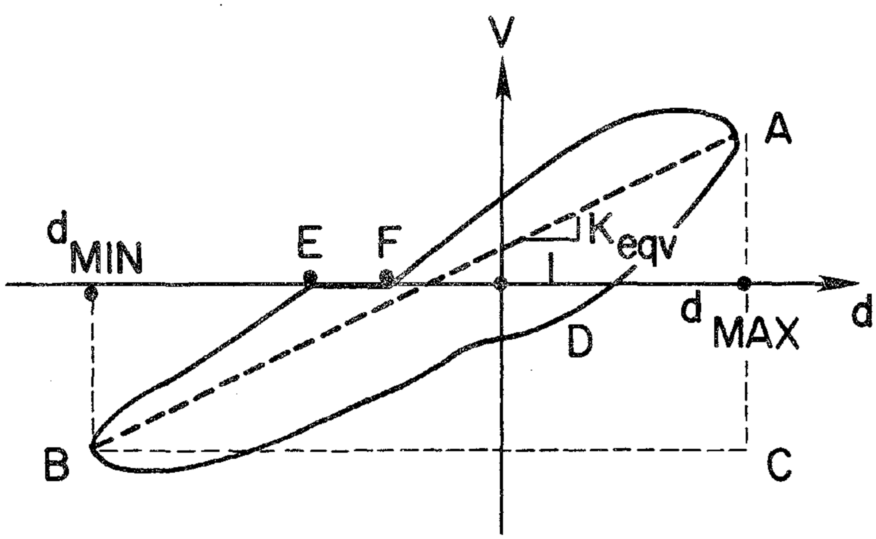


FIGURE 5.7 DEFINITION OF EQUIVALENT PARAMETERS FROM TEST DATA

With points A and B representing, respectively, points of maximum and minimum story displacement of that loop, the equivalent stiffness K_{eqv} is taken as the slope of straight line AB, the equivalent damping ratio ξ_{eqv} as the ratio of loop area ADBEF/($\frac{\pi}{2}$ x triangular area ABC), and the strain amplitude d as $(d_{max} - d_{min})/2$. For each loop of the main portion of the test data, a set of $K_{eqv}(d)$ and $\xi_{eqv}(d)$ may be obtained and plotted, with one of them shown in Fig. 5.8. It is obvious from this plot, and from other similar plots, that the functional relations of K_{eqv} and ξ_{eqv} to d are very complicated. First of all, they are not single-valued functions which is a result of the strong nonlinear behavior of the bracings produced by the complex combinations of yielding,

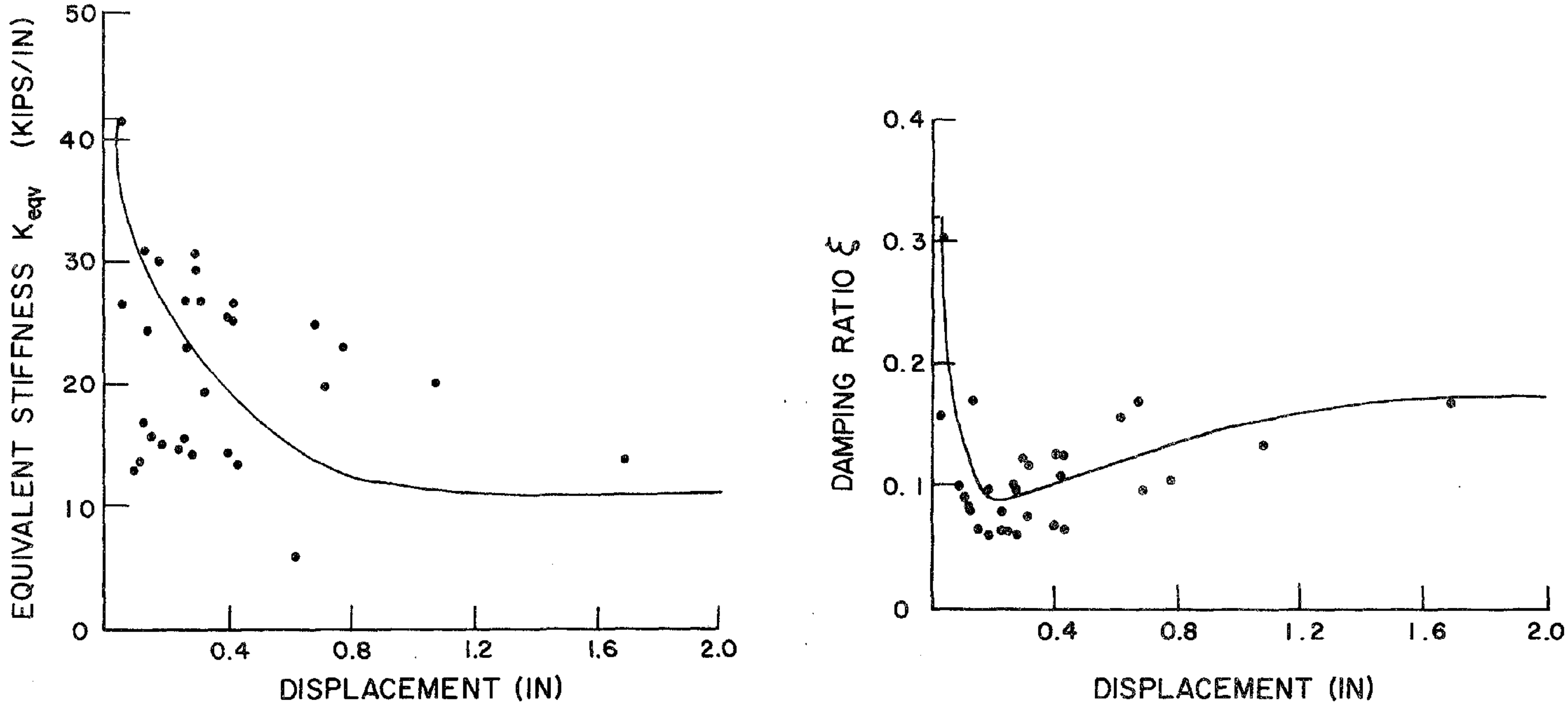


FIGURE 5.8 EQUIVALENT PARAMETERS FROM TEST DATA

buckling and pinching. The present equivalent linear representation cannot adequately simulate this complicated case. A rough approximation can be obtained, however, by using averaged single-valued functional relations as shown in Fig. 5.8. Results obtained this way are listed in Table 5.5. A discussion of the results will be given in the next section.

TABLE 5.5 RATIOS OF STIFFNESS RK = K_d/K_o AND DAMPING ξ FOR TEST EXAMPLES

Test	Story	K_o*		Story Displacement d (in.)										
				0	.02	.1	.2	.3	.4	.6	.8	1.0	1.4	2.0
P400	3	60	RK	1.0	1.0	.78	.634	.537	.463	.354	.305		.268	.268
			ξ	.32	.32	.13	.09	.09	.10	.12	.135		.17	.17
	2	45	RK	1.0	1.0	.867	.800	.689	.633	.567	.522		.467	.467
			ξ	.35	.35	.09	.06	.06	.07	.10	.13		.17	.17
	1	41	RK	1.0	1.0	.850	.800	.750	.717	.658	.617		.550	.550
			ξ	.30	.30	.22	.16	.12	.12	.13	.14		.17	.17
P800	3	60	RK	1.0		1.0	.857	.732	.643	.482	.357	.250	.214	.214
			ξ	.1		.1	.1	.1	.1	.12	.15	.16	.17	.17
	2	45	RK	1.0		1.0	.820	.803	.656	.475	.361	.246	.197	.197
			ξ	.1		.1	.1	.1	.1	.12	.15	.16	.17	.17
	1	41	RK	1.0		1.0	.820	.803	.656	.475	.361	.246	.197	.197
			ξ	.1		.1	.1	.1	.1	.12	.15	.16	.17	.17

* kips/in.

6. DISCUSSION

6.1 Coverage of Numerical Examples

Numerical examples given in Table 5.2 are selected to have an adequate coverage of practical conditions in type of input ground motion, parameters of structure, level of nonlinearity, and distribution of nonlinear deformation.

As for type of input ground motion, the El Centro record of the NS component of May 18, 1940, is chosen for its popularity in response analysis, the Pacoima record of the S16E component of February 9, 1971, for its high peak acceleration and as a representative of the single-predominant pulse type of ground motion, and the Taft record of the S69E component of July 21, 1952, as a representative of the more random type of ground motion. Only the strong phase of motion is used.

As for parameters of the structure, the variation of stiffness of whole structure is allowed by changing the stiffness matrix in proportion in models ST1, ST2 and ST3 to give fundamental linear periods 0.30, 0.59 and 1.14 sec, respectively. The mass-proportional damping α varies in model ST4 and ST5 and the damping ratios in ST1, ST2 and ST3 also change due to change of natural frequencies. The corresponding damping ratios for the fundamental mode are given in Table 6.1.

TABLE 6.1 DAMPING RATIO ξ OF FUNDAMENTAL MODE OF STRUCTURES IN NUMERICAL EXAMPLES

Structure	ST1, ST5, ST6	ST2	ST3	ST4
ξ	.02	.05	.09	.01

Level of nonlinearity expressed by element ductility factor μ and distribution of nonlinear deformation are controlled and designed to vary by adjusting the elastic limit σ_y of each element. As shown in Table 5.2, the ductility factor μ varies over a wide range from 1.8 to 8.6. In general, nonlinear deformation occurs at the bottom story as indicated by $\mu_{34}>1.0$ in Table 5.2; however, in two cases nonlinear deformations also occur at other stories. In model ST6, nonlinear deformations occur almost simultaneously at all three stories as in the case of balanced design. In model ST1, nonlinear deformations occur at the bottom two stories representing a special case.

The slope of the second straight line of the bilinear force-deformation relation (Fig. 5.3) is also allowed to vary from $p=0.1$ to $p=0.3$ causing a strong effect on the nonlinear levels of responses.

6.2 Sources of Error

For linear behavior of structures, the errors in the input identification procedure come only from the following sources. One source is the frequency cut-off which is a necessary step in the discrete analysis of a finite length of data. This error can be reduced by increasing the data samples and decreasing the time interval between samples. Another source is noise which inevitably is introduced into the data when estimating structural parameters and when modelling the structure. This is common to all identification problems.

For nonlinear behavior of structures, other errors may be introduced by the equivalent linearization method in the frequency domain. The most serious error is from the differences in mechanism of the nonlinear deformation represented by the equivalent linear structural model and the real nonlinear structure shown in Fig. 3.4. If a transfer function may be defined for the nonlinear behavior of the structure from the Fourier spectra of its nonlinear response u(t) and the input v(t) as given by,

$$H(i\omega) = U(i\omega)/V(i\omega) \tag{6.1}$$

this relation may give quite different values of transfer functions at those frequencies far from the main peaks. Figure 6.1 provides a comparison for case ST1 where the transfer functions $H(i\omega)$ of the equivalent linear structures are obtained from $m_1 = 10$ and $m_2 = 60$ for the two-model identification and from the ratio $U(i\omega)/V(i\omega)$ obtained directly from the Fourier spectra of the nonlinear response u(t) and input v(t). The high fluctuations present in the ratio $U(i\omega)/V(i\omega)$ seem to show the invalidity of processing nonlinear responses in the frequency domain. It is realized that nonlinear behavior cannot be viewed in frequency domain since the law of superposition is no longer valid; therefore, this comparison is used only to show that, at both ends of the spectrum, the equivalent linear models consistently give too small a value for the transfer function.

Because there is a significant discrepancy between the actual nonlinear behavior and that of the equivalent linearized structure due to the inability of the equivalent linearized structure to show permanent deformation, as explained in Sec. 3.1, the actual response will be different in the time history u(t) or in its Fourier spectrum, mainly at both ends of the frequency range where the spectrum has small ordinates. The differences at both ends of the frequency range will be very much amplified after dividing by the transfer function $H(i\omega)$ of the equivalent structure which has small ordinates, as shown in Fig. 4.1 or Fig. 6.1. A further frequency cut-off at both ends is needed to reduce this error.

6.3 Fitness of the Identified Input

Since there are frequency cut-offs necessary in the input identification procedure, the identified input can never equal the original input, but it can approach the original input filtered between the cut-off frequencies. The effect of this filtering process for all cases investigated is, in general, not large on both the time history and the reponse spectrum in the medium frequency range, as may be seen from the figures in Appendix A; however, it may be considerable for those cases where high frequency components are important for defining peak values in the time history, and such as in the Pacoima motion shown in Fig. A.8.

The fitness of the results is tested in three comparisons of time history, Fourier spectrum, and response spectrum.

As may be seen from a comparison of all numerical examples, the response spectra (damping ratio 0.05) of the identified inputs are quite close to the original in the range of periods $T = 0.02 - 1.4$ sec. For longer periods, the identified response spectra are considerably in error due primarily to the frequency cut-off used in the analysis.

For spectra in the high frequency portion, mismatch shows two errors, one from the high frequency cut-off and another from the fluctuation of the spectrum ordinate. Since smoothing is usually used in obtaining a "standard" response spectrum, the averages of the high frequency portion from $T \leq 0.5$ sec will be used for comparison in Table 6.2. The identified response spectra are quite close to the filtered ones and differ from the original ones by errors within 10% for almost all numerical examples.

TABLE 6.2 COMPARISON OF RESPONSE SPECTRA IN $T \leq 0.5$ SEC and a_{max}

Case	μ_{34}	AVERAGE ORDINATE			a_{max}/g		
		Original	Filtered	Identified	Original	Filtered	Identified
ST1	3.5	287	271	286	0.34	0.35	0.37
ST2	7.8	287	271	263	0.34	0.35	0.40
ST3	7.2	287	271	242	0.34	0.35	0.26
ST4	1.8	287	271	254	0.34	0.35	0.32
	4.5	287	271	317	0.34	0.35	0.37
	8.6	287	271	300	0.34	0.35	0.34
	3.5	574	542	567	0.69	0.68	0.80
ST5	8.0	146	142	147	0.18	0.16	0.24
	6.3	725	669	627	1.15	0.66	0.60
ST6	3.3	146	142	154	0.18	0.16	0.20
P400		594	404	416	0.48	0.43	0.64
P800		791	761	659	1.31	0.86	0.62

If individual ordinates of the response spectra are compared, although errors may be as high as 50% at some small portion for individual cases and in the short and long period ends for many cases due to small ordinates, the fitness of the spectrum is in general within errors less than 20% for most numerical examples.

Comparison of the time-histories and Fourier amplitude spectra can only be made by a general overall visual view since both of them are highly oscillating in nature. For nonlinearities in the range $\mu \doteq 6-8$, the results of input identification using the equivalent linearized model may be considered good for all numerical examples. The identified input time-histories always follow the corresponding original ones for the important wave pulses such as those near $t = 2.1$, 4.5 and 4.8 sec of the El Centro record; $t = 3.7$, 6.6, 7.7 sec of the Taft record. For the Pacoima record, the amplitudes of filtered acceleration time-history are much smaller than those of the original because of the high frequency components in the original record and Fig. A8 shows that the identified input time-history follows the filtered one for the important wave pulses such as those near $t = 3.3$, 6.2, 7.3, 7.5, 7.6 and 8.2 sec. Table 6.2 gives a comparison of the maximum accelerations, showing the errors to be in general about 10%.

Rather similar comments may be made for the identifications using test results with strong nonlinearities, except the following should be mentioned. Firstly, two large consecutive cycles of vibration appeared in the identified time history from $t = 9.2$ to 10.4 sec with a period about $T = 0.6$ sec (apparently due to a similar wave train in the input motion) for the case P400 which makes the identified response spectrum near that period much higher than the original. Secondly, the input motion for case P800 contains so many high frequency components that the amplitudes of the identified motion are consistently lower than the original, although wave pulses are generally close and the ordinates in high frequency range of the response spectrum are consistently too low. Thirdly, the greater differences of the identified input motions are believed to come from the large scattering and multiple-valued nature of stiffness as a function of deformation amplitude as shown in Fig. 5.5.

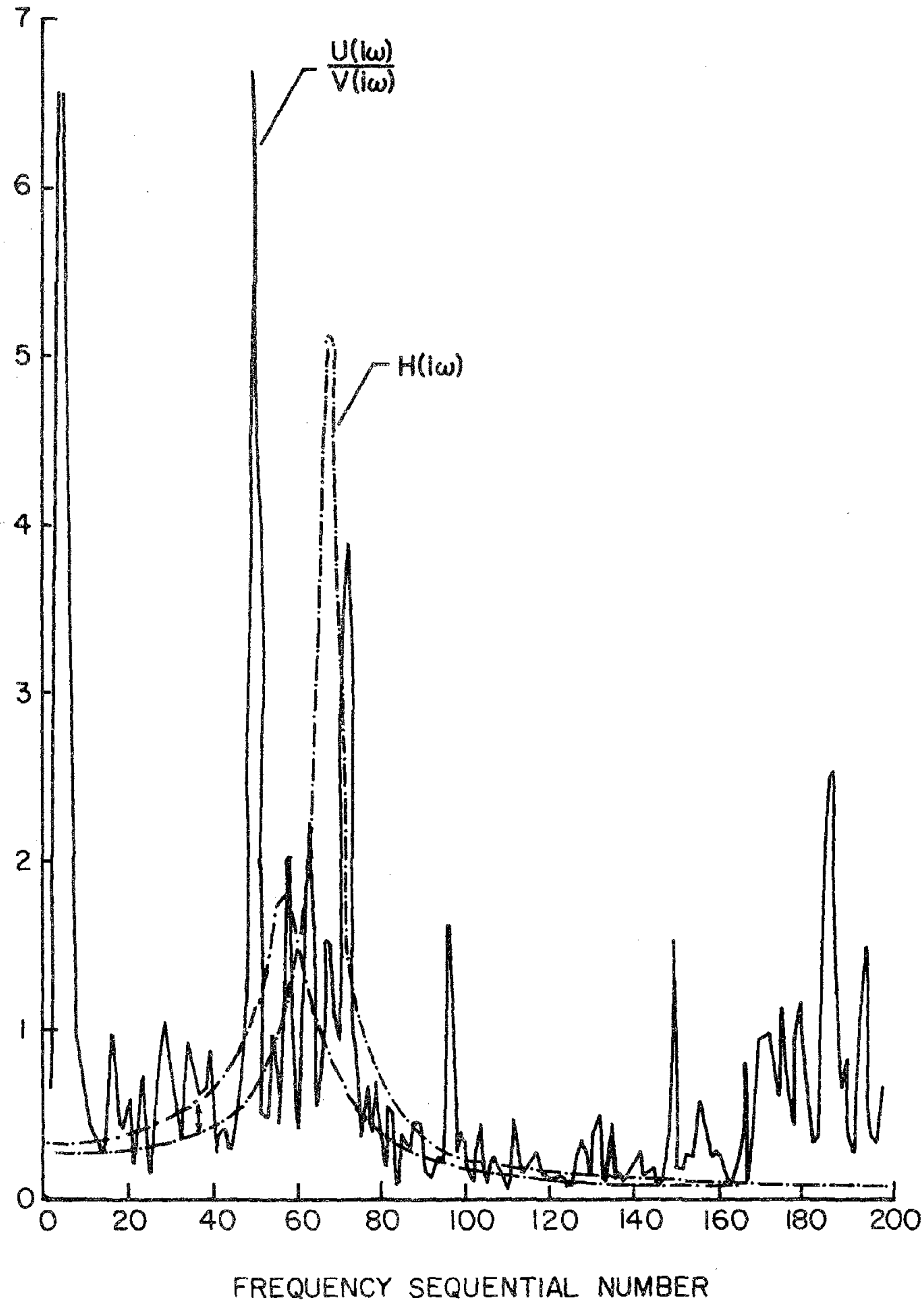


FIGURE 6.1 AMPLITUDE SPECTRA OF TRANSFER FUNCTION H(iω) AND u(iω)/V(iω) FOR ST1

7. SUMMARY AND POSSIBLE IMPROVEMENTS

A series of structures with bilinear element stiffnesses subjected to several earthquake excitations were used as a means to test the feasibility of the input identification procedure presented. The significant findings of the investigation were the following:

(a) It is possible to identify input peak acceleration values, response spectra and time-history from structural responses by the equivalent linearization procedure with acceptable results.

(b) There are two sources of error in the proposed procedure for input identification. One is inherent in discrete analysis of finite data in the form of frequency cut-off which makes the components of motion identifiable only within certain frequency limits. For linear or weak-nonlinear cases, this error can be reduced to negligible magnitude by increasing the duration of analysis and by decreasing the time steps of sampling. The second error, which becomes apparent with highly nonlinear cases, is caused by nonlinear drift or by the difference in responses calculated from the true nonlinear model and the equivalent linear model. This error is more pronounced for the low-frequency components. This suggests using a cut-off preventing the drift associated with permanent deformations.

(c) For linear or weak-nonlinear cases, one-model identification procedure gives good results; however, for strong nonlinear cases, a two-model identification procedure is required to yield good results.

Further improvements of the identification procedure are believed possible by taking the following steps:

(a) As shown in Fig. 4.2, the combination of two models obtained by taking the lower-frequency components (frequencies $\le f_{NM}$) from one model and the higher-frequency components (frequencies $>f_{NM}$) from another. The shift from one model to another is suddenly taken at some arbitrarily chosen frequency f_{NM}; therefore, a suitable gradual shift should give better results. Although several values of f_{NM} as well as m_1 and m_2 have been tried, it is not claimed here that they are the best.

(b) Accepting the belief that a better two-model identification procedure can be developed, it is reasonable to believe also that even better results might be obtained with higher order models with each model being responsible primarily for a specified frequency band of limited width.

(c) Although the strain-averaging method used herein in the equivalent linearization procedure is believed better than using some predetermined fraction of the maximum strain, it could possibly be improved by recognizing the influence of other factors, such as the variation of intensity of vibration with time.

(d) In many practical cases, there are several responses known at the same time. Undoubtedly, the input identification procedure could be improved by developing effective means of using the additional response data available. One approach which appears promising would be to weight the inputs identified from individual responses in a manner which would minimize in a least squares sense the differences between the given and computed responses from identified input.

APPENDIX A: COLLECTION OF IDENTIFIED INPUTS

This appendix contains a set of figures which show comparisons in time-history, response spectrum and Fourier spectrum of the identified and the real input. The identified inputs are those obtained from two-model identification procedure. Results of all numerical cases in Table 5.2 are included except one for ST4, El Centro, $\mu = 1.8$ which shows very good coincidence between the identified and the real input, but is not included because it is almost a linear case.

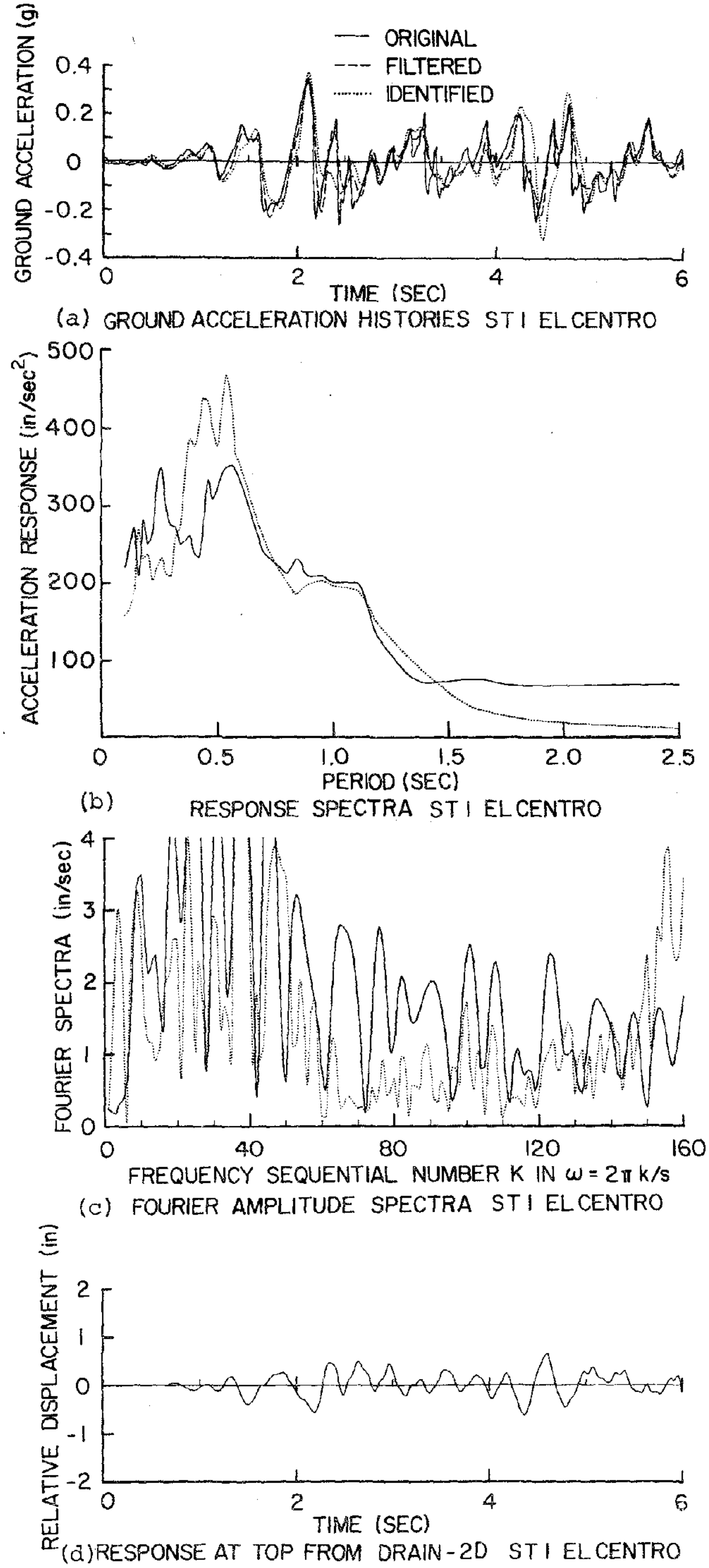


FIGURE A1 RESULTS FOR ST1, EL CENTRO, μ = 7.1 & 3.5

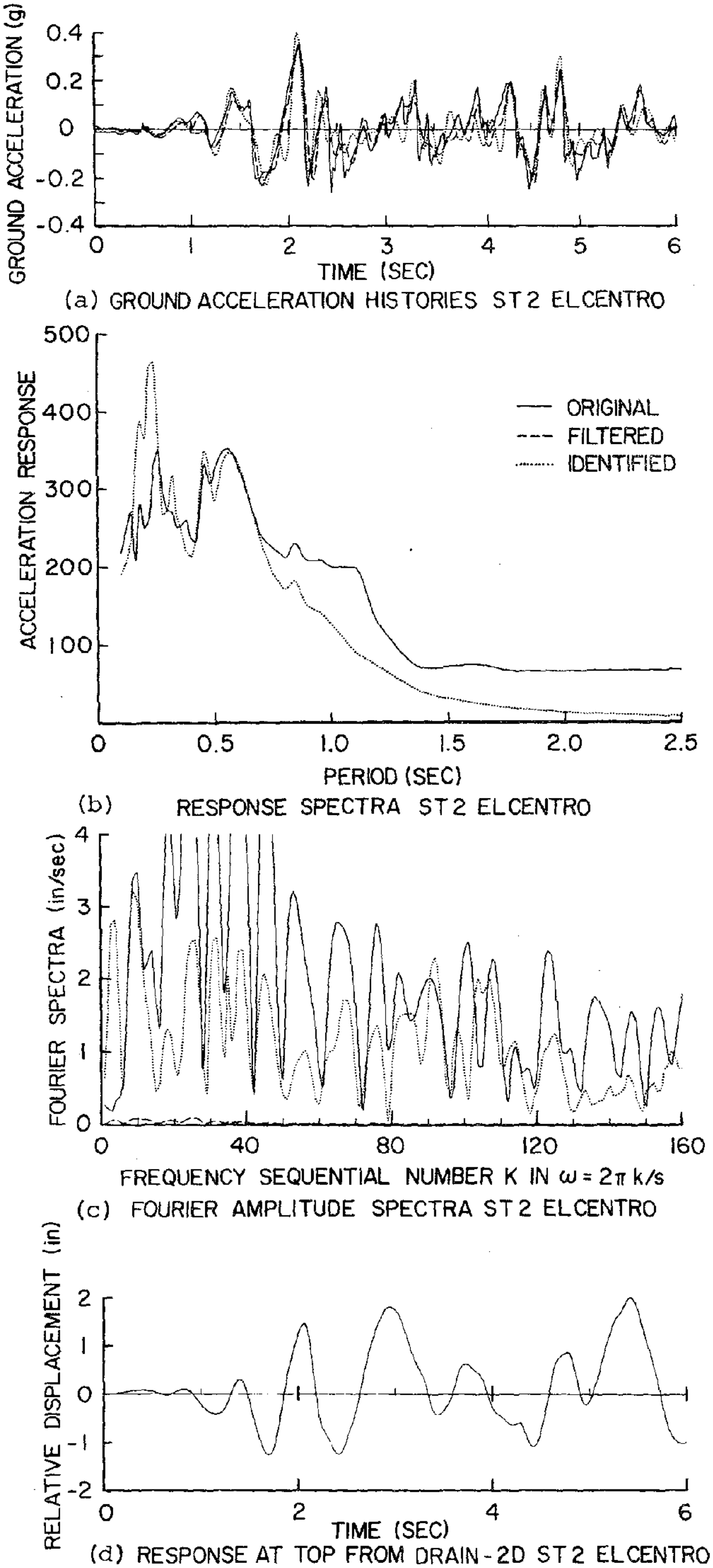


FIGURE A2 RESULTS FOR ST2, EL CENTRO, $\mu = 7.8$

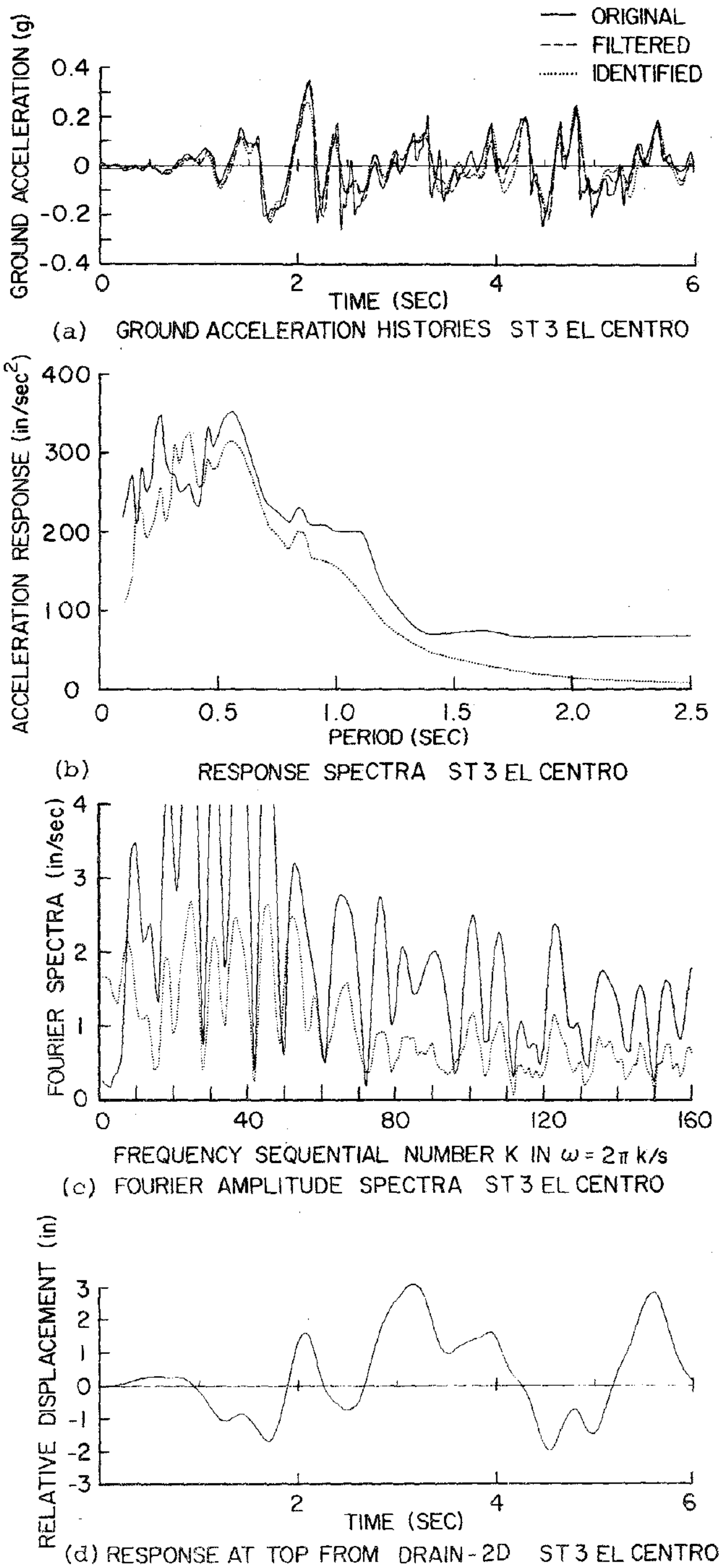
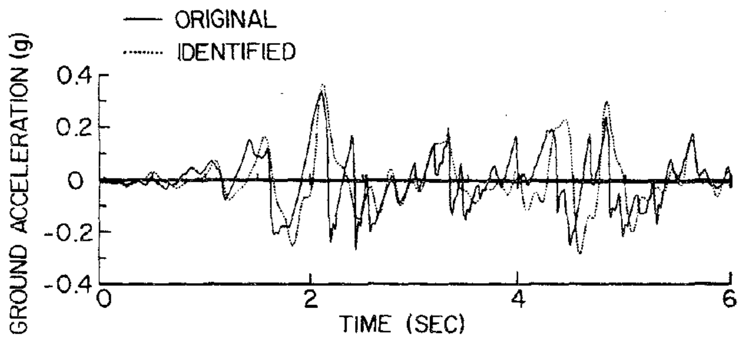
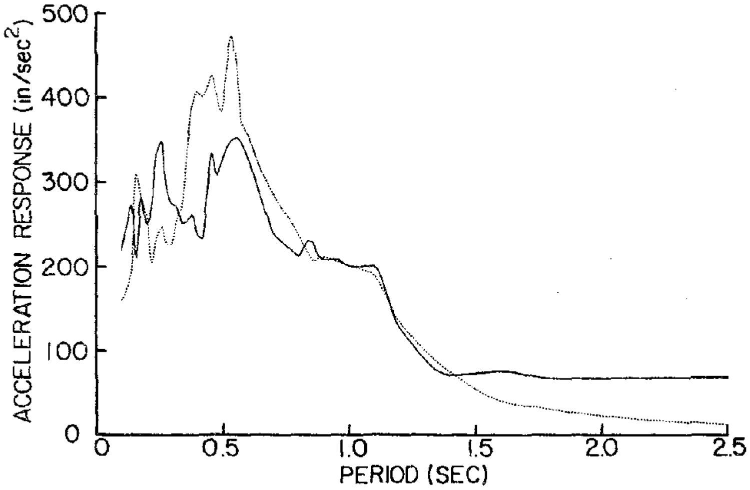


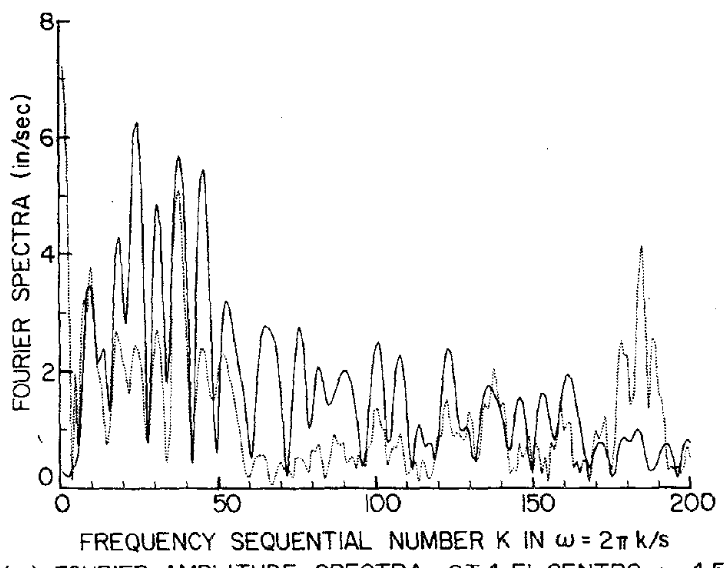
FIGURE A3 RESULTS FOR ST2, EL CENTRO, μ = 7.2



(a) GROUND ACCELERATION HISTORIES ST4 EL CENTRO $\mu = 4.5$

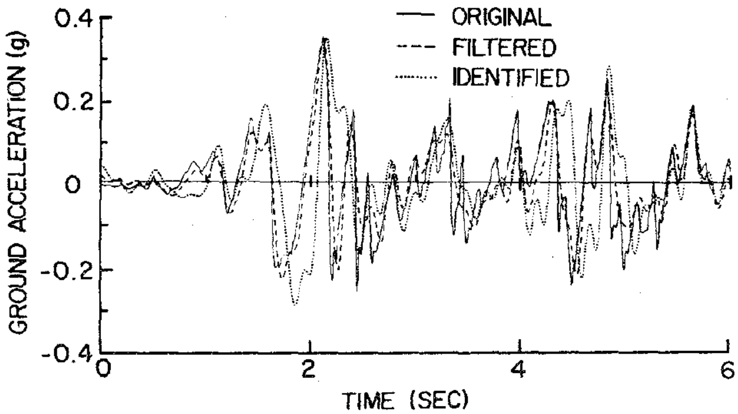


(b) RESPONSE SPECTRA ST4 EL CENTRO $\mu = 4.5$

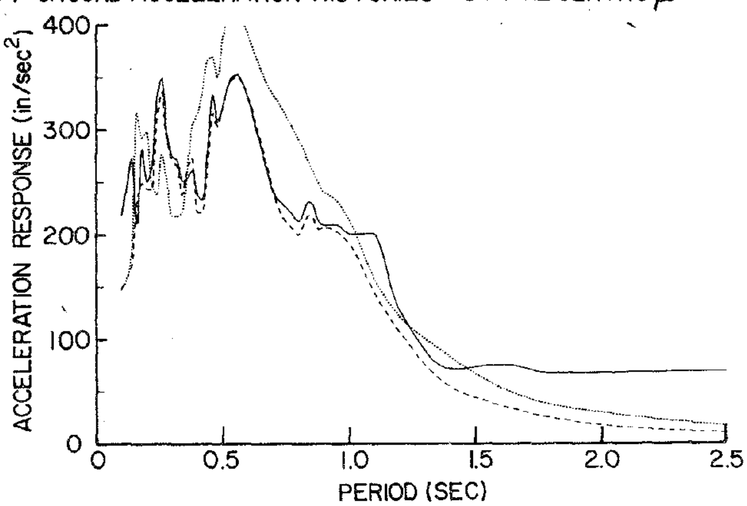


(c) FOURIER AMPLITUDE SPECTRA ST4 EL CENTRO $\mu = 4.5$

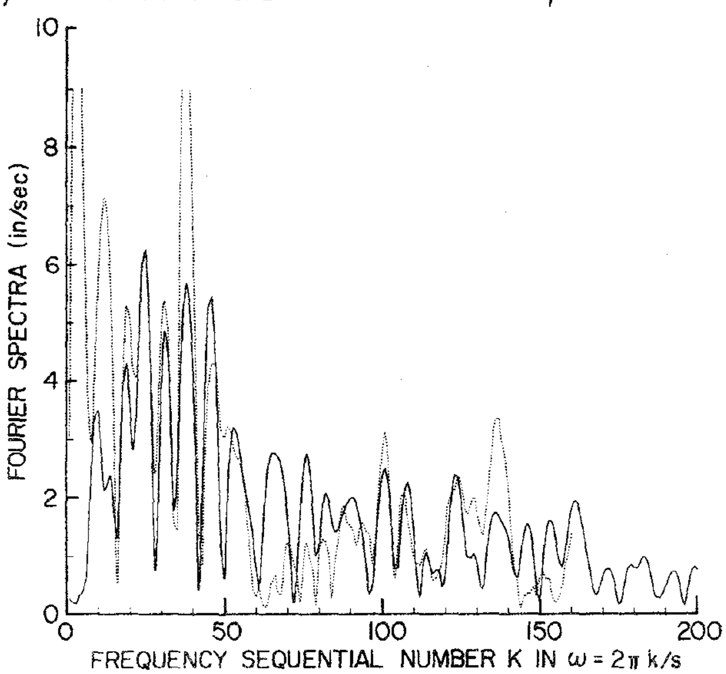
FIGURE A4 RESULTS FOR ST4, EL CENTRO, $\mu = 4.5$



(a) GROUND ACCELERATION HISTORIES ST4 EL CENTRO $\mu = 8.6$



(b) RESPONSE SPECTRA ST4 EL CENTRO $\mu = 8.6$



(c) FOURIER AMPLITUDE SPECTRA ST4 EL CENTRO $\mu = 8.6$

FIGURE A5 RESULTS FOR ST4, EL CENTRO, $\mu = 8.6$

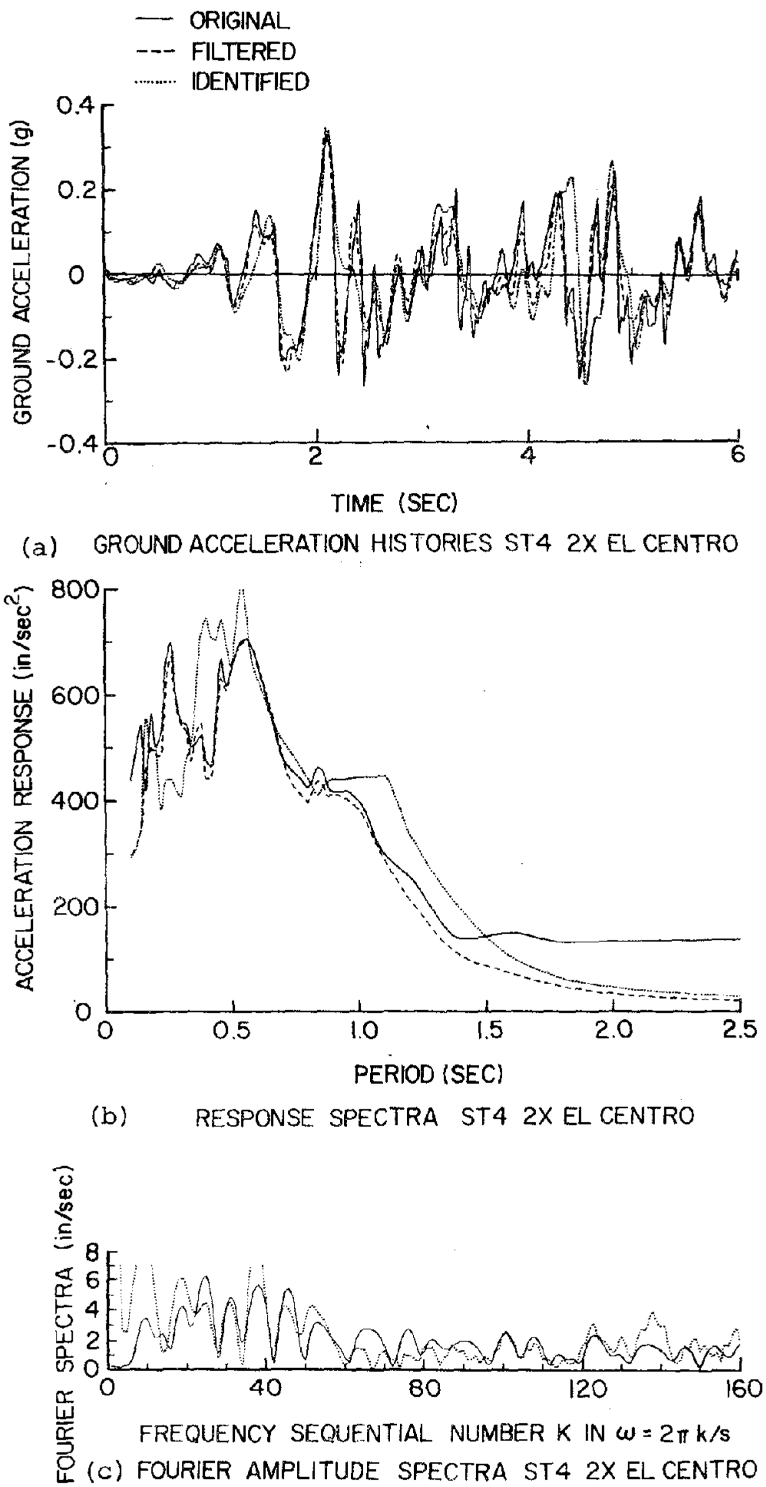


FIGURE A6 RESULTS FOR ST4, 2 X EL CENTRO, μ = 8.6

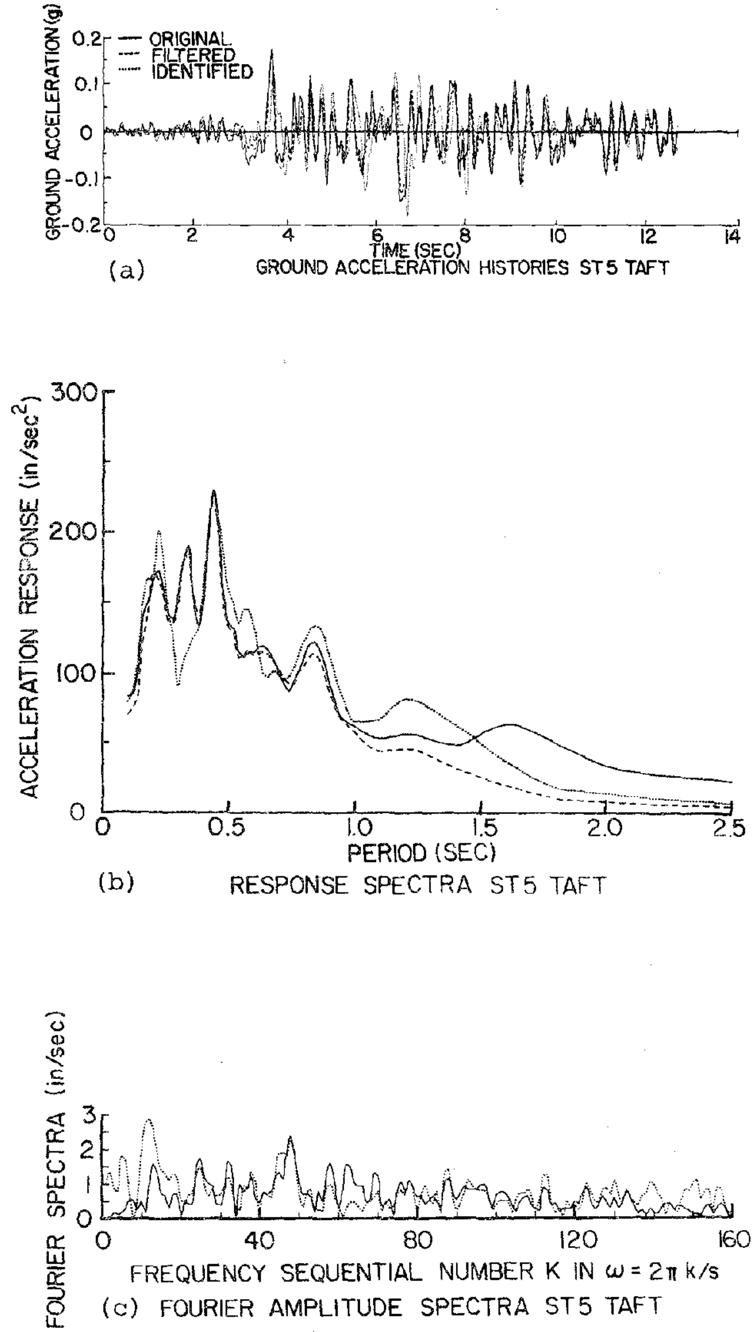
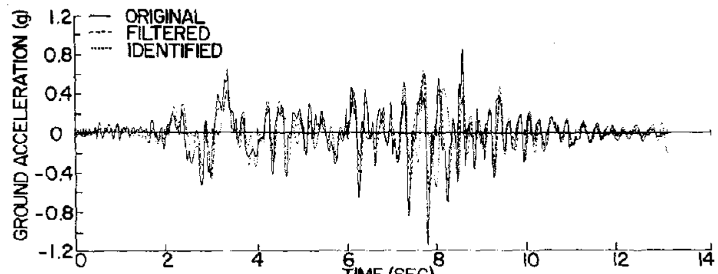
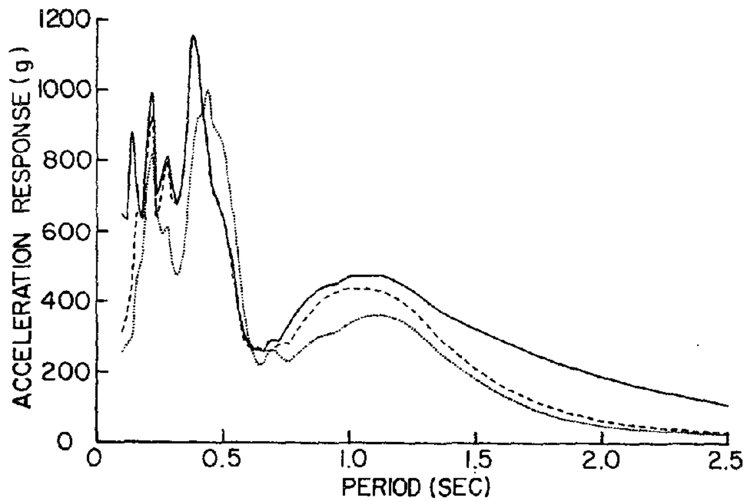


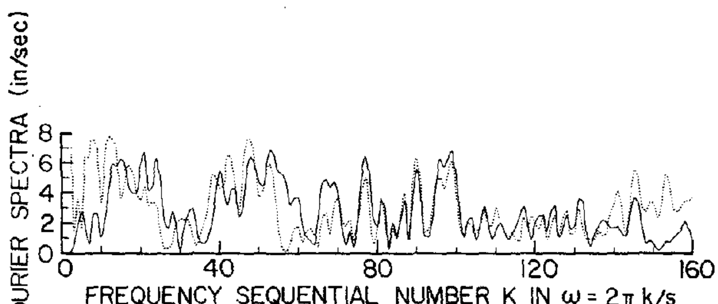
FIGURE A7 RESULTS FOR ST5, TAFT, μ = 3.5



(a) GROUND ACCELERATION HISTORIES ST 5 PACOIMA



(b) RESPONSE SPECTRA ST 5 PACOIMA



(c) FOURIER AMPLITUDE SPECTRA ST 5 PACOIMA

FIGURE A8 RESULTS FOR ST5, PACOIMA, μ = 6.3

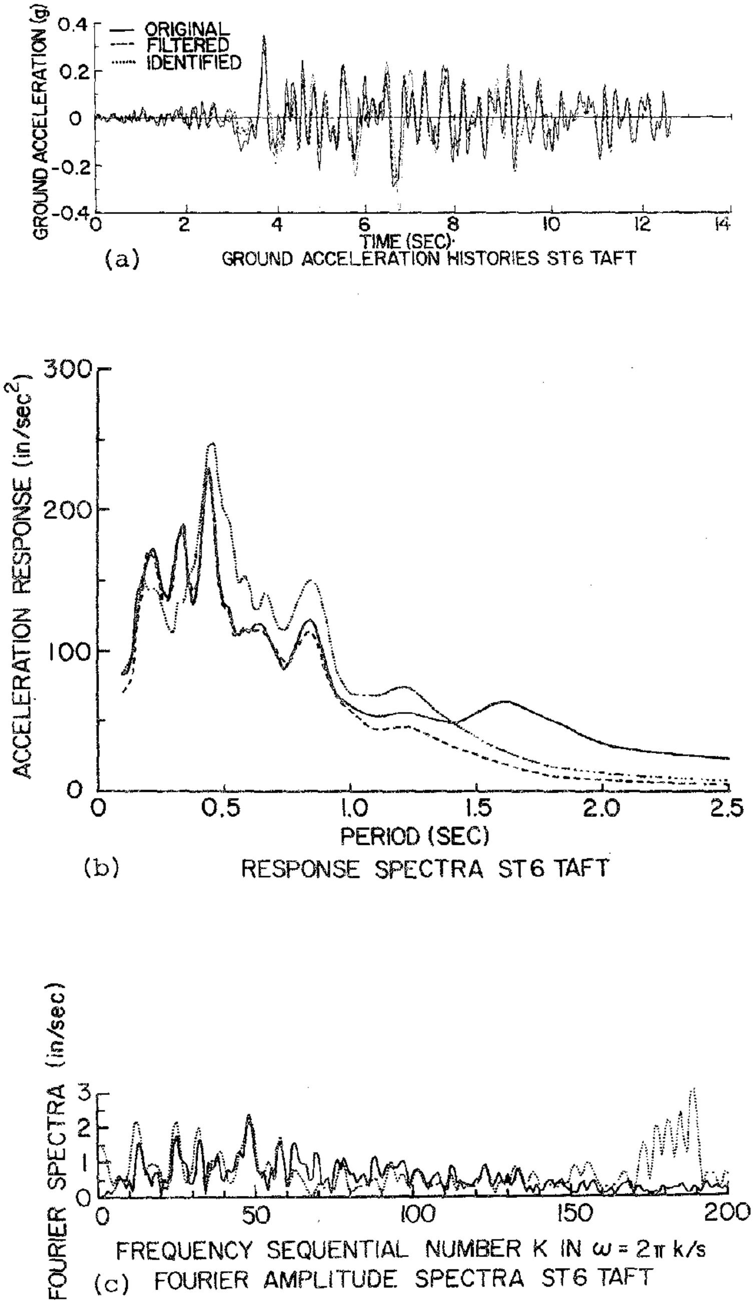


FIGURE A9 RESULTS FOR ST6, TAFT, μ = 7.1, 6.4, 3.3

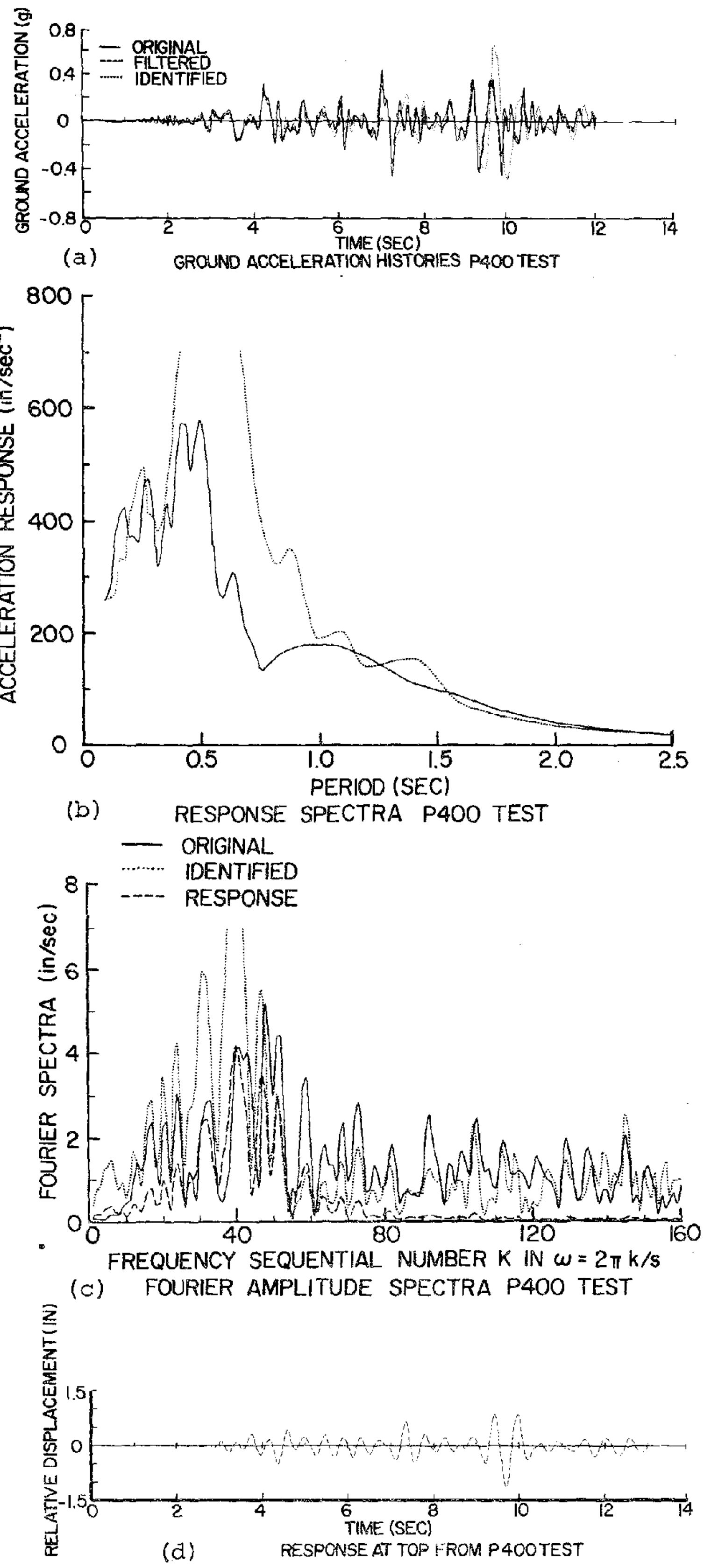


FIGURE A10 RESULTS FOR TEST FRAME, P400

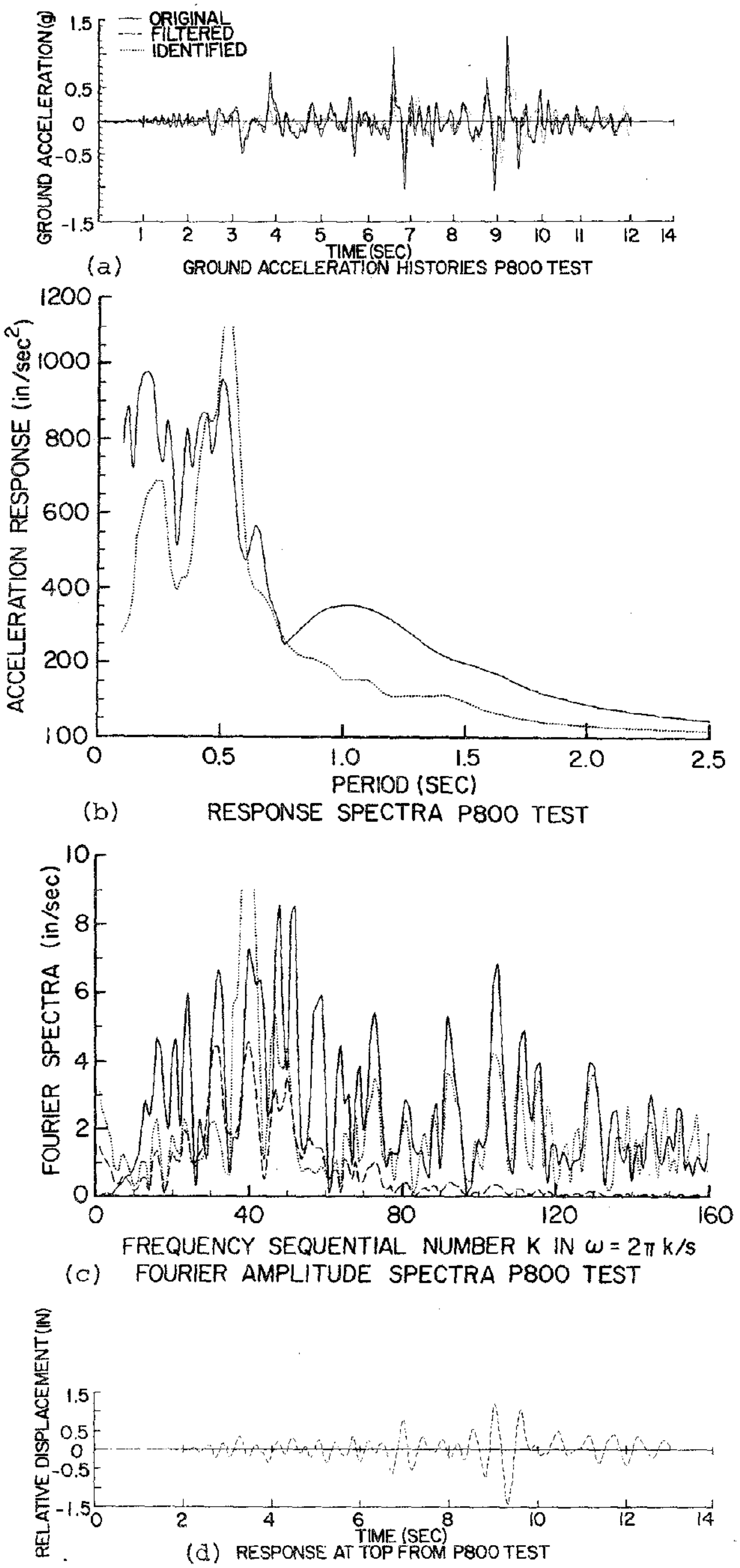


FIGURE A11 RESULTS FOR TEST FRAME, P800

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