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IDENTIFICATION OF THE SERVICEABILITY LIMIT STATE AND DETECTION OF SEISMIC STRUCTURAL DAMAGE

by

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Damage analysis models based on equivalent modal parameters are used to identify the serviceability limit state for structures subjected to earthquake loads. A model for the analysis of seismic structural damage consists of a finite number of numerical indicators (damage indices) and of a procedure for their computation. A structure exhibits a nonlinear behavior when it experiences an earthquake. Nonetheless an equivalent linear structure can be defined. Damage indices can be defined from the vibrational parameters of the equivalent linear structure considered. In the first place, it is shown that the problem of identification of the serviceability limit state is equivalent to the problem of detection of seismic structural damage. A limit state can be defined as a surface in the space of the damage indices relative to the model considered. Experiments on small-scale models are analyzed to obtain an analytical definition of the serviceability limit state. Since many uncertainties are present in any damage analysis problem, a probabilistic definition is obtained, in terms of the probability that the structure considered is damaged after the seismic event. Finally, records from large-scale models and full-scale structures are analyzed. The model proposed performs very well in detecting seismic structural damage.

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by

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PREFACE

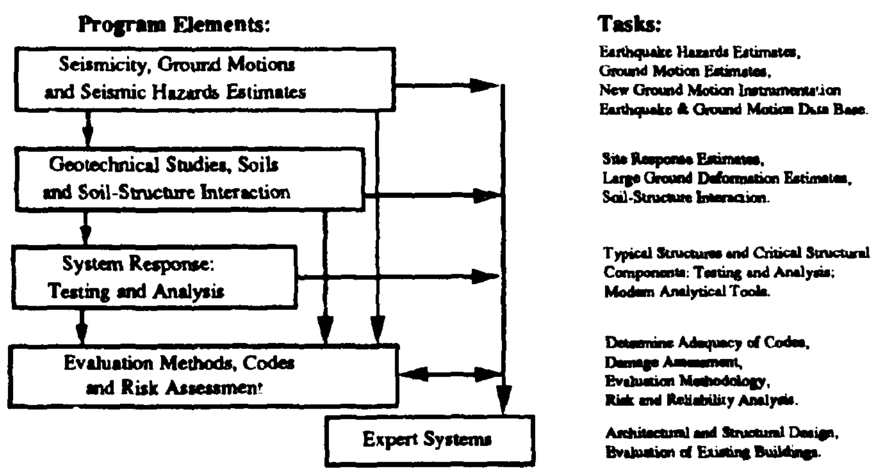
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. Initially, the emphasis is on structures and lifelines that are found in zones of moderate seismicity, such as the eastern and central United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to Evaluation Methods and Risk Assessment.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. The work will rely on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



Evaluation Methods and Risk Assessment Studies constitute one of the important areas of research in Existing and New Structures. Current research addresses, among others, the following issues:

1. Code issues: such as probabilistic Load Resistance Factor Design (LRFD) which includes the investigation of wind vs. seismic issues, and of design seismic loads for areas of moderate to high seismicity.
2. Response modification factors: which combine the effect of shear and bending for various types of buildings.
3. Seismic damage: in terms of global and local damage index, and damage control by design.
4. Seismic reliability analysis of building structures: including limit states which correspond to serviceability and collapse.
5. Risk and societal impact: such as retrofit procedures, restoration strategies and damage estimates.

The ultimate goal of projects concerned with Evaluation Methods and Risk Assessment is to provide practical tools for engineers to assess seismic risk to individual structures and thus, to estimate social impact.

This study addresses the issue of defining and identifying seismic damage sustained by building structures. The authors have correlated the degree of damage thus identified and have summarized it in terms of damage indices with serviceability limit state probability. Results of existing laboratory experiments are also utilized in this study.

ABSTRACT

Damage analysis models based on equivalent modal parameters are used to identify the serviceability limit state for structures subjected to earthquake loads. A model for the analysis of seismic structural damage consists of a finite number of numerical indicators (damage indices) and of a procedure for their computation. A structure exhibits a nonlinear behavior when it experiences an earthquake. Nonetheless an equivalent linear structure can be defined. Damage indices can be defined from the vibrational parameters of the equivalent linear structure considered. In the first place, it is shown that the problem of identification of the serviceability limit state is equivalent to the problem of detection of seismic structural damage. A limit state can be defined as a surface in the space of the damage indices relative to the model considered. Experiments on small-scale models are analyzed to obtain an analytical definition of the serviceability limit state. Since many uncertainties are present in any damage analysis problem, a probabilistic definition is obtained, in terms of the probability that the structure considered is damaged after the seismic event. Finally, records from large-scale models and full-scale structures are analyzed. The model proposed performs very well in detecting seismic structural damage.

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SECTION 1: INTRODUCTION

1.1. Statement of the Problem

In a preceding report (DiPasquale and Cakmak, 1987) it has been shown that strong motion records contain information about the history of strain that a structure experiences during an earthquake. A definition of a model for the analysis of seismic damage has been given. A model for the analysis of seismic damage consists of a finite number of damage indices and of a procedure for their computation. In a parameter-based model, the damage indices are obtained from the equivalent linear parameter of the structure. These parameters are computed using Maximum Likelihood techniques. In general, at least two indices are to be computed. It was proposed that the maximum and the cumulative softening be used for the detection and the assessment of seismic damage. The softening of a structure was defined as:

$$1 - \frac{(T_0)_{initial}}{(T_0)_{equivalent}} \tag{1.1.1}$$

The structure's softening is thus computed from the ratio between the equivalent fundamental period, estimated from the strong motion records using a time variant linear model, and the fundamental period of the (undamaged) structure before it experiences the earthquake. The rationale for proposing parameter based indices is the intuitive consideration that both damage and plastic deformation in a structure would result in a shift of its equivalent natural frequencies toward lower values. In the above mentioned report the authors presented a review of the procedures for damage assessment proposed in the literature both for simple elements and for complex structures. A parameter-based model for damage analysis was introduced. The model consisted of a family of damage indices, computed from the vibrational parameters of a linear structure equivalent to the structure analyzed, which in general presents a nonlinear behavior. A procedure for the estimation of the equivalent linear parameters was also developed, implemented on the Microvax II of the Department of Civil Engineering of Princeton University and applied to strong motion accelerograms. The model proposed was consistent with actual levels of damage and showed sensitivity to very low levels of damage.

In this report some methods for detection and assessment of seismic damage based on strong motion records are proposed. The emphasis is placed on the identification of the serviceability limit state, i.e., on the detection of the onset of seismic structural damage.

The procedures described in this report can be applied to any instrumented structure for the purpose of post earthquake reliability analysis. Some buildings in seismic areas such as California are equipped with traditional strong motion accelerometers that record the measured waveform on rolls of paper. The digitization of such waveforms being a lengthy process, damage analysis based on strong motion records is not yet practical. In fact, it would take more to analyze the strong motion records than to inspect the structure. However, the state of the art in transducers, A/D converters and data network technology is such that the motion could be recorded digitally and transmitted to a central computer after the occurrence of an earthquake. The creation of such a network would make the procedures described in this report immediately applicable to practical post-earthquake analysis of existing structures.

1.2. Organization of the Work

In the first part of this report, the authors give a definition of the serviceability limit state. It is shown that the serviceability limit state can be defined as a surface in the space of the damage indices and that this definition is equivalent to a damage detection rule. In order to define a criterion for damage detection in practical applications, seismic simulation tests of small scale reinforced concrete structural models are analyzed.

In the second part, the criterion derived from the analysis of a series of shaking table experiments is applied to practical cases of damage detection. Both shaking table experiments on larger scale models and strong motion records from full scale structures are analyzed.

SECTION 2: DEFINITION OF THE SERVICEABILITY LIMIT STATE

2.1. The Serviceability Limit State and Damage Detection

In a post earthquake investigation, the engineer must decide whether a structure that has experienced a strong motion event can still be considered safe or whether some service is required. If such a decision must be taken within the framework of a damage assessment model, it is necessary to define a function of the damage indices $f(\delta_1,\ldots,\delta_m)$, whose value can be related both to a judgment on the post earthquake reliability of the structure and to a decision to be made regarding possible repairs.

damage space	reliability assessment	field decision
$f \leq 0$	structure safe	no repair
$f > 0$	structure unsafe	repair necessary

A limit state would thus be defined as the boundary between the set of safe and the set of unsafe structures in the space of the damage indices, otherwise called the damage space. This limit state will be called serviceability limit state. Defining a limit state implicitly provides a detection rule for seismic damage assessment. Detection of seismic damage can be defined as the process of deciding whether a structure is safe after being hit by an earthquake. This is probably the most important information that a real time damage analysis system can provide. Sorting safe structures from unsafe can help set a rational schedule for building inspection, which is one of the major concerns after a major seismic event.

Due to the high level of uncertainty present in any problem of damage analysis, it would not be possible to propose a deterministic criterion for the detection of seismic damage. The objective of this work will therefore be the determination of the probability that a given structure has been damaged after an earthquake. Furthermore, the criterion sought will depend upon the kind of structure considered, whether it is made of steel or of reinforced concrete, on its size and morphology.

In this report, the case of reinforced concrete structures is considered. The serviceability limit state is identified from the analysis of experiments performed on small scale structures, but the damage detection criterion proposed performs very well in the case of larger scale models and of full scale structures.

2.2. Choice of the Appropriate Damage Indices

Structural damage presents features of both brittle, ductile and fatigue damage. Each structural component maintains a memory of the past load history that must be described using at least two parameters. Maximum strain and dissipated energy have been proposed to measure damage of reinforced concrete members (Park and Ang, 1985). In a preceding report (DiPasquale and Cakmak, 1987) the authors have proposed that two indices, the maximum softening and the cumulative softening, be used to measure global structural damage.

In general, therefore, fatigue plays an important role in structural damage phenomena. However, the structures considered in this report, as well as a vast class of existing buildings, are built in standard reinforced concrete, which has very poor ability to endure cyclic loads. The expression "standard reinforced concrete" is used here in contrast to "heavily reinforced concrete", as it is used in power (especially nuclear) facilities and long-spanned bridges. In the first part of this report, a definition of the serviceability limit state is given, based on the analysis of some of the seismic simulation experiments conducted at the University of Illinois at Urbana-Champaign. For this set of data, some experimental evidence (Cecen, 1979, Sozen, 1981) suggests that, for this kind of structure, damage is path-independent. Thus, only one parameter, namely the maximum softening, has been considered in the following analysis. Thus, it will be assumed that the damage state of a structure can be characterized by one variable only, namely the maximum softening δ_M. The maximum softening is defined as:

$$\delta_M = 1 - \frac{(T_0)_{initial}}{(T_0)_{\max}} \tag{2.2.1}$$

With respect to an earlier definition (DiPasquale and Cakmak, 1987) expression (2.2.1) has the advantage of yielding a value for δ_M that is always between 0 and 1, as it is customary for damage indices.

SECTION 3: IDENTIFICATION OF THE SERVICEABILITY LIMIT STATE

3.1. Analysis of Seismic Structural Response

A validation of the proposed method must come from the analysis of damaged structures. Unfortunately, there are very few records from buildings that have been damaged during an earthquake. In order to find a sufficient number of structures, it is necessary to resort to seismic simulations on shaking tables. Such experiments are particularly useful for model validation, because of the following reasons:

(a) The structural models tested are usually a physical realization of an engineer's concept. Most structures analyzed in the following are designed and built so that their behavior is as close as possible to that of a moment resisting frame, sometimes in parallel with other structural elements such as a shear wall. Their response is thus very close to the analytical predictions. Furthermore, in most cases lateral and torsional motion are prevented. The only motion possible is then parallel to the direction of the excitation. This reduces the number of degrees of freedom and simplifies the damage analysis.

(b) The structural models are extensively instrumented. Consequently it is possible to compare the performances of the method to be validated with the performances of other methods.

(c) Each structural model is used for several simulations with different levels of earthquake intensity, thus providing a large amount of data from a single structure.

3.2. Description of the Test Structures

A particularly interesting program of shaking table experiments has been realized at the University of Illinois at Urbana-Champagne (UIUC) by Sozen and his associates (Healey and Sozen, 1978, Abrams and Sozen, 1979, Cecen, 1979, Sozen, 1981). The experiments analyzed here come from a population of seven structures (Table 3-1), which may be considered grouped into two series. Each one of the first group of three test structures was made up of two ten-storied frames working in parallel, with the

TABLE 3-1: SHAKING TABLE EXPERIMENTS AT UIUC					
model	authors	type of structure	earthquake excitation	undamaged first mode frequency (Hz)	total number of runs
MF1	Healey and Sozen (1978)	10-story, 3-bay double frame tall first story	El Centro (1940)	3.7	3
FW1	Abrams and Sozen (1979)	10-story, 3-bay double frame + wall heavily reinforced wall	El Centro (1940)	4.3	3
FW2	"	10-story, 3-bay double frame + wall lightly reinforced wall	El Centro (1940)	4.5	3
FW3	"	10-story, 3-bay double frame + wall lightly reinforced wall	Taft (1952)	4.0	3
FW4	"	10-story, 3-bay double frame + wall heavily reinforced wall	Taft (1952)	5.2	3
H1	Cecen and Sozen (1979)	10-story, 3-bay double frame weak beams	El Centro (1940)	2.2	3
H2	Cecen and Sozen (1979)	10-story, 3-bay double frame weak beam design	El Centro (1940)	2.7	7

story weights positioned between. Test structures H1 and H2 had a uniform d: tribution of story heights. The first and top story for the structure MF1 were taller than the other stories. Cross-sectional dimensions of the frame elements were the same for all four test structures. The second group of test structures (FW1, FW2, FW3 and FW4) comprised three elements, two ten-storied frames working in parallel with a wall. For all test structures, the story mass weighed approximately 4.5 kN.

Each structure was tested at the University of Illinois Earthquake Simulator. Test runs of a given structure included repetitions of the following sequence:

(1) Free vibration test to determine low-amplitude natural frequencies.

(2) Earthquake simulation.

(3) Recording of any observable signs of damage such as cracking and spalling of concrete.

This sequence was repeated with the intensity of the earthquake simulation being increased in successive sequences. For all but one (H2) structure, the first earthquake simulation represented the "design" level.

The natural frequency of a structure's linear oscillations can be estimated from the strong motion records. In the case of real-world buildings it must be assumed that no prior information about the structure is available and that the undamaged natural frequency must be estimated from the strong motion records. In Table 3-2 the fundamental frequencies estimated by the experimenters from free vibration tests are compared with those estimated from strong motion records. The general trend in both cases is toward a decrease in the fundamental frequency after earthquake simulation tests and subsequent damage. The frequencies estimated from strong motion records are lower than those estimated from free vibration tests, with the exception of the FW structures. This is due to the different amplitude of the oscillations of the structure in the two cases. Nonlinearities do not appear abruptly in structural behavior. A rather smooth change in the response can be observed as the amplitude of the oscillation increases. This accounts, for instance, for the difference that is found between the frequencies estimated from forced and from ambient vibration measurements. The values of the initial fundamental frequencies estimated for the FW structures are close to the values that the experimenters had computed analytically (Sozen,1981).

TABLE 3-2: FUNDAMENTAL FREQUENCY (HZ) AFTER TEST RUNS			
structure	run	frequency (free vibrations)	frequency (earthquake records)
MF1	initial	3.2	3.7
MF1	1	2.9	2.2
MF1	2	2.3	1.9
MF1	3	1.9	1.2
FW1	initial	3.4	4.3
FW1	1	3.7	2.7
FW1	2	3.1	1.9
FW2	3	2.9	1.9
FW2	initial	3.3	4.5
FW2	1	3.2	3.0
FW2	2	3.1	2.0
FW2	3	nr[1]	1.8
FW3	initial	3.2	4.0
FW3	1	3.3	2.7
FW3	2	2.7	2.4
FW3	3	2.7	2.4
FW4	initial	3.2	5.2
FW4	1	3.0	3.1
FW4	2	2.8	2.1
FW4	3	2.2	1.9
H1	initial	3.0	2.2
H1	1	2.2	1.3
H1	2	1.8	0.91
H1	3	nr[1]	0.61
H2	initial	4.4	2.7
H2	1	3.3	2.2
H2	2	2.6	1.8
H2	3	2.3	1.8
H2	4	2.3	1.6
H2	5	2.0	1.4
H2	6	1.9	1.3
H2	7	nr[1]	0.9

[1] not reported.

3.3. Qualitative Definition of Damage for the UIUC Database

The seismic simulation experiments performed at the University of Illinois at Urbana-Champaign constitute a particularly useful database for damage analysis because of the information about the damage state of the structures after each test provided by the experimenters. In particular, information about crack pattern and width as well as about permanent interstory displacement is available.

On the basis of the experimenters' report, the response records and engineering experience, Yao and his associates (Toussi and Yao, 1983, Stephens and Yao, 1987) introduced a qualitative classification of damage. Structural damage would be divided in four classes:

"Safe"	S
"Lightly Damaged"	L
"Damaged"	D
"Critically Damaged"	C

The features of the four classes can be summarized as follows:

S Cracks within 0.05 mm. No new cracks opening.
No macroscopic or global nonlinearities. The structure "stays stiff", the vibration of the upper floors does not reflect the irregularities of the base motion, and the top displacement vs. base shear curve is linear.
The story drift does not exceed 1%.
Some local yieldings and permanent displacements are tolerated.

L Flexural cracks open, width 0.1 mm.
The motion of the upper floors reflects the irregularities in the base motion.
Permanent displacements are measured (≈0.5% of the story height).
Very short intervals of flat top displacement are observed.
The maximum base shear is close to the level of collapse, computed using limit analysis design techniques.

D Cracking is extensive, crack width 0.3mm. Possible local spalling.
Permanent displacements around 1% of the story height.
Several flat top displacement intervals in the response.
The base shear and the top displacement waveforms exhibit a qualitative difference.

C Crack width ≈ 0.4 mm and beyond, crushing, spalling of several elements.
Top story displacement shows some aperiodicity at the end of the record.
Poor correlation between base shear and top level displacement.

On the basis of the experience acquired from these and other experiments, Sozen (1981) concluded that the story drift γ (ratio of the inter-story displacement to the story height) was the controlling parameter in the seismic damage of such structures. The inherent randomness of the phenomenon led him to suggest an acceptability quotient A based on γ:

$$A = \frac{(5 - 2\gamma)}{4} \tag{3.3.1}$$

In the equation above γ is the maximum story drift in percentage and A $(0 \leq A \leq 1)$ represents the fraction of structures that are expected to remain safe after experiencing a story drift equal to γ. For example, a story drift of 0.5% will be always acceptable, whereas in as many as three-fourths of the cases a story drift of 2% will provoke major damage.

In the experimenter's words, the "effective" or equivalent fundamental period is said to be "the most informing property of a multi-story slender structure with respect to drift control".

3.4. Practical Analysis of Strong Motion Records

In order to estimate the equivalent linear parameters of a structure, records from the basement and from some upper level are needed. The basement record is used as input to a numerical model that is equivalent to the original structure in the Maximum Likelihood sense (DiPasquale and Cakmak, 1987). As the actual structure is nonlinear, the equivalent linear model must be time variant. Therefore, the interval of duration of the earthquake is divided into segments (time windows) of appropriate length, and the parameter estimation is performed separately for each of these windows. Using the basement record as input is equivalent to neglecting the interaction between the soil and the structure. Figures 3.1 through 3.25 show the acceleration records analyzed and the corresponding evolution of the softening of the structure, defined as

$$1 - \frac{(T_0)_{initial}}{(T_0)_i} \tag{3.4.1}$$

where the subscript i indicates the ith window considered, and $(T_0)_i$ the respective equivalent fundamental period estimated.

The analysis of each window can be divided into two phases. First the order of the model, i.e., the number of modes whose parameters are estimated, is determined (model identification). Then the parameters are estimated using maximum likelihood techniques. If the effect of the input noise is neglected, the estimation of the modal parameters using the Maximum Likelihood criterion reduces to matching the recorded acceleration of the upper floor with the output of the modal model considered. Experience shows that one-mode models may sometimes match the observed motion very poorly, although the estimates of the fundamental frequency are very close to those obtained using higher order models. Two-mode models usually can be fit to the output with very good results, while three-mode models are very difficult to treat, due to the large numbers of parameters involved. Figures 3.26a and 3.26b show the graphic output of the estimation program (MUMOID) (see DiPasquale and Cakmak, 1987) for the structure FW4, run 2, time window between 1 sec and 2 sec. Four plots are provided in the standard graphic output of MUMOID: basement (input) motion, observed upper level motion (continuous line) and predicted upper level motion (dotted line), prediction error and autocorrelation function of the prediction error. In figure 3.26a, only one mode is used to fit the data, resulting in a poor match. In figure 3.26b a two mode model is used with an apparent improvement.

In the computation of the maximum softening there are two sources of error. The first source of error is the uncertainties involved in the computation of $(T_0)_{max}$. When the structure is in the nonlinear regime, the equivalent linear parameters vary with time. The values estimated depend upon the particular time window selected. In addition, some statistical errors would be present even in the case of purely linear structures. When the structure is in the linear regime it is possible to obtain approximate expressions for the covariance matrix of the parameter estimates (DiPasquale and Cakmak, 1987). When nonlinearities are present, these same expressions will give information on the accuracy of the estimates computed.

The second source of error lies in the uncertainties in the values of $(T_0)_{initial}$ selected. In the analysis presented here, $(T_0)_{initial}$ has been estimated from the earthquake records relative to the first run, as would be the case if a real structure was analyzed and no prior information was available. In order to estimate $(T_0)_{initial}$, it is important that the structure's behavior is linear in the interval analyzed. Therefore, very short intervals close to the beginning of the record have been selected. For the structure H2, for which the first earthquake had a much smaller intensity than for the other structures, the "initial" time window was chosen so that the intensity of the input motion considered was the same as the intensity of the corresponding windows for the other structures. In the analysis of real world buildings the problem may be even more complicated. The history of the structure would not in general be known and the estimate of $(T_0)_{initial}$ would in general be different from the fundamental period of the undamaged building. In this case it would probably be convenient to use an approximate expression for $(T_0)_{initial}$, such as:

$$(T_0)_{initial} = 0.1\,N \tag{3.4.2}$$

where the fundamental period is expressed in seconds and N is the number of stories of the structure.

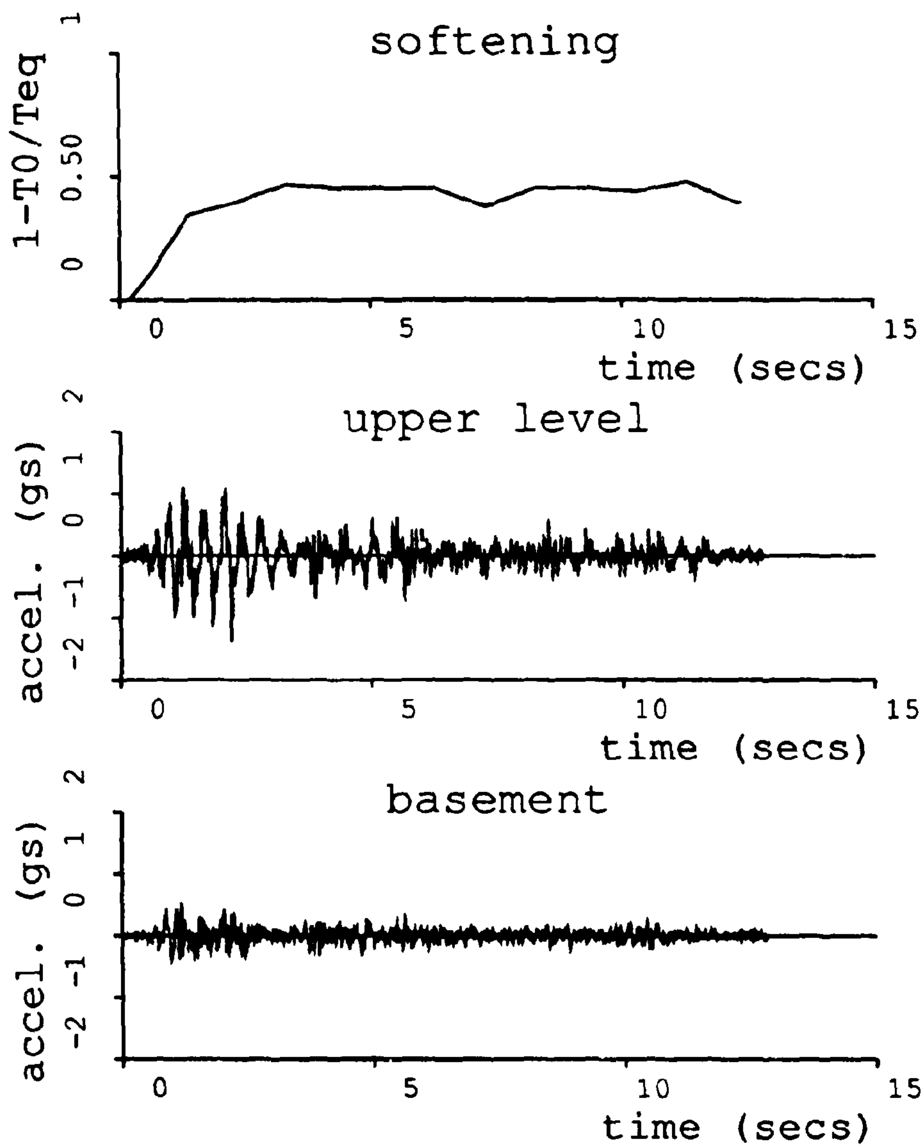

FIGURE 3-1 Recorded Acceleration and Estimated Softening for UIUC, Model FW1, Run 1

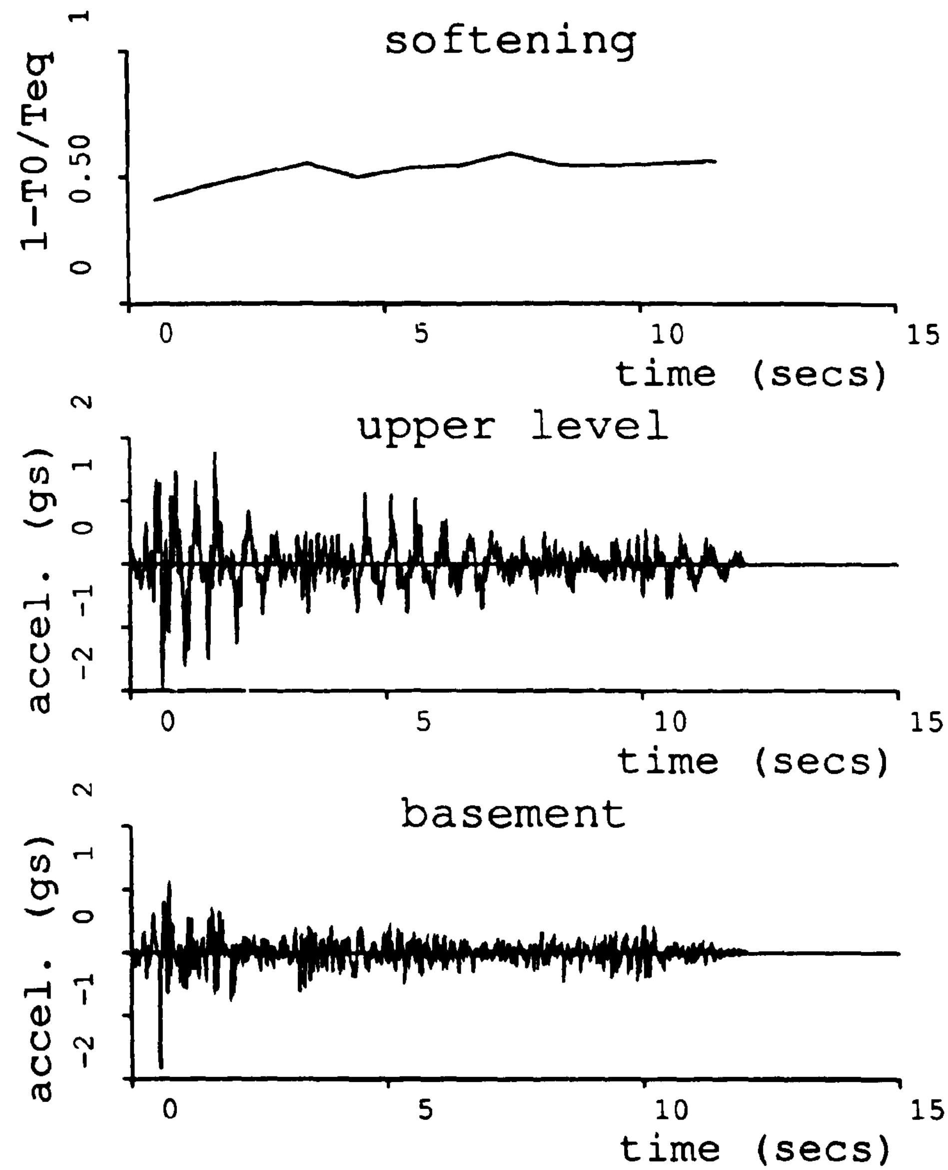

FIGURE 3-2 Recorded Acceleration and Estimated Softening for UIUC, Model FW1, Run 2

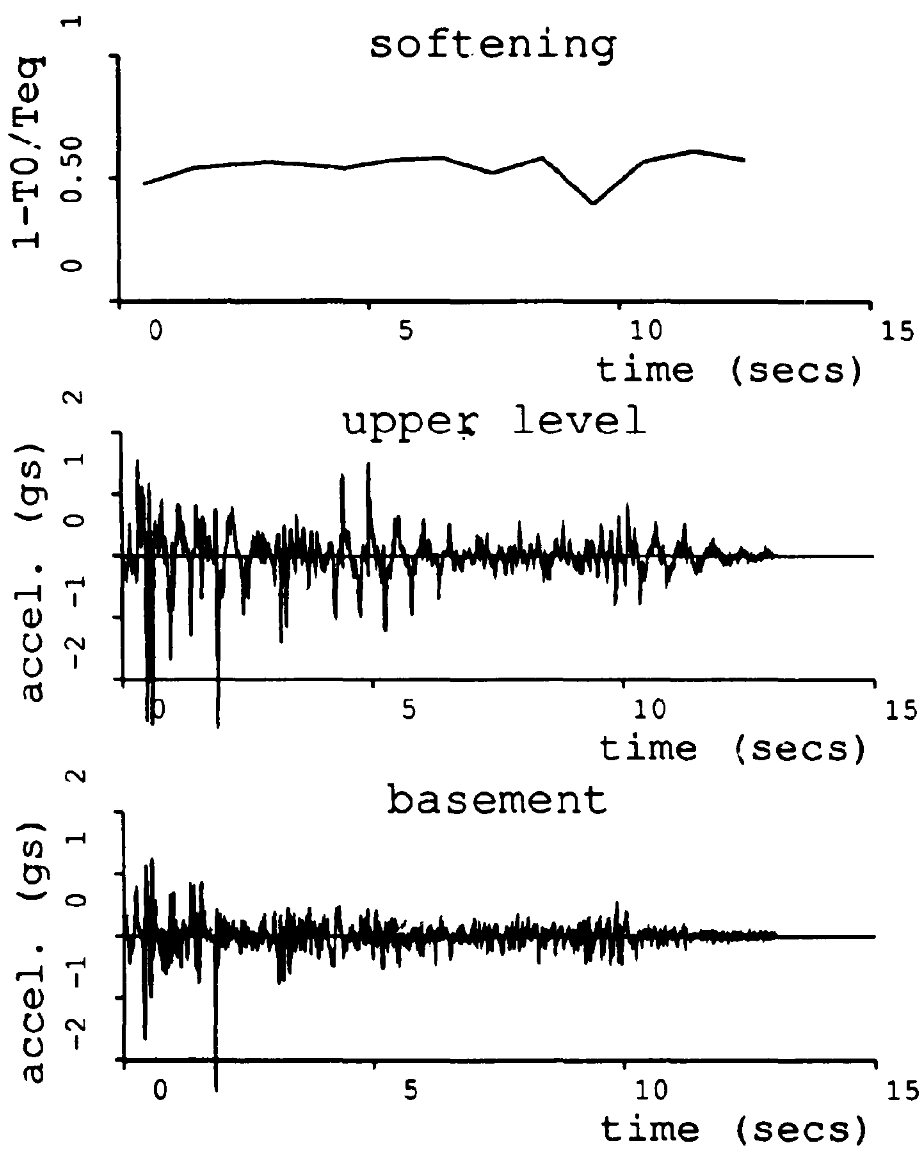

FIGURE 3-3 Recorded Acceleration and Estimated Softening for UIUC, Model FW1, Run 3

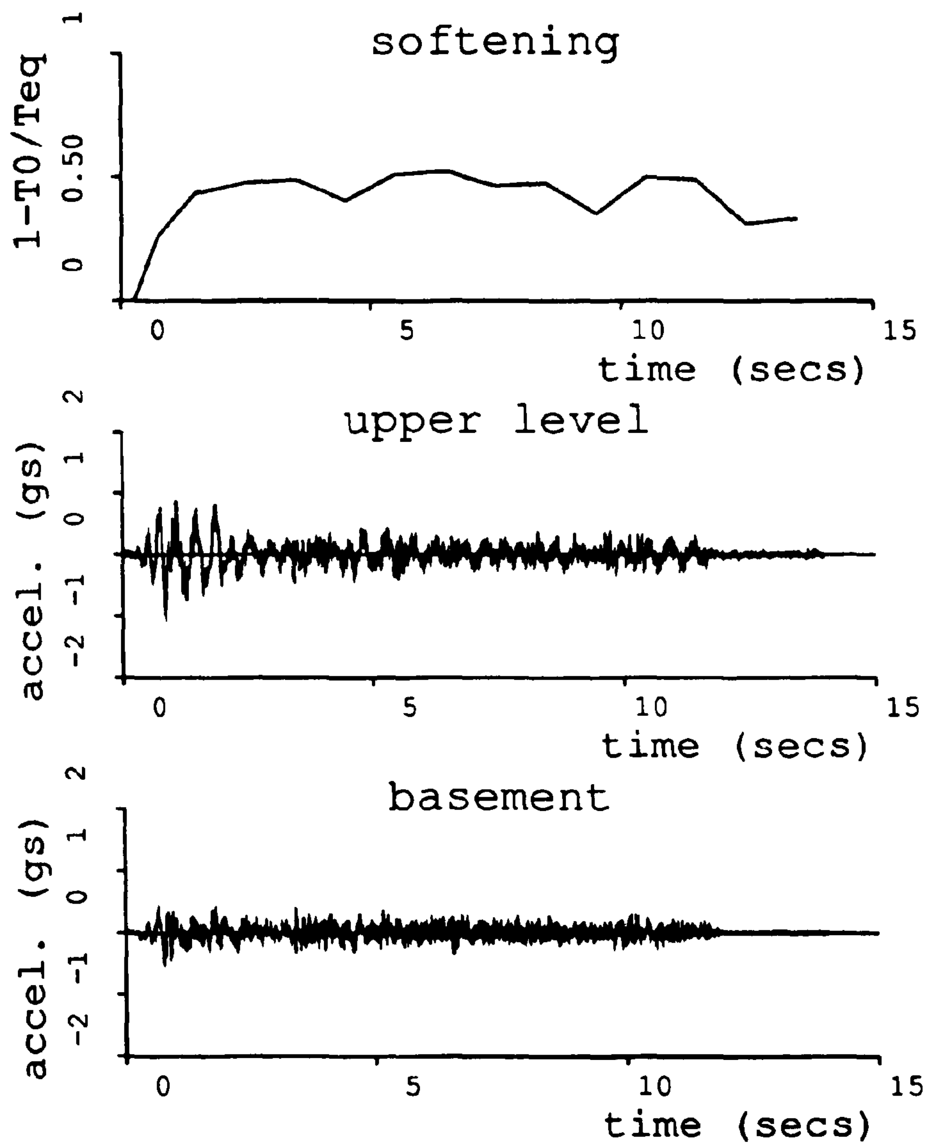

FIGURE 3-4 Recorded Acceleration and Estimated Softening for UIUC, Model FW2, Run 1

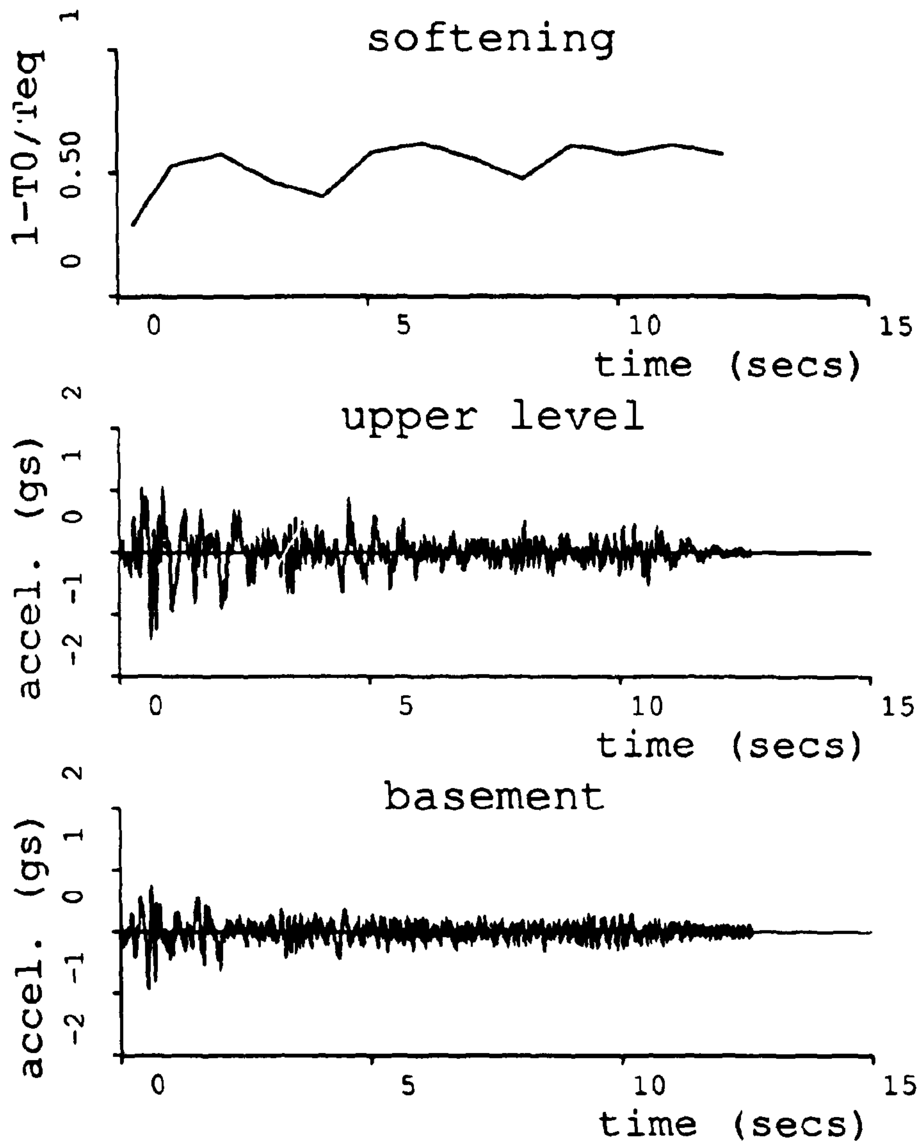

FIGURE 3-5 Recorded Acceleration and Estimated Softening for UIUC, Model FW2, Run 2

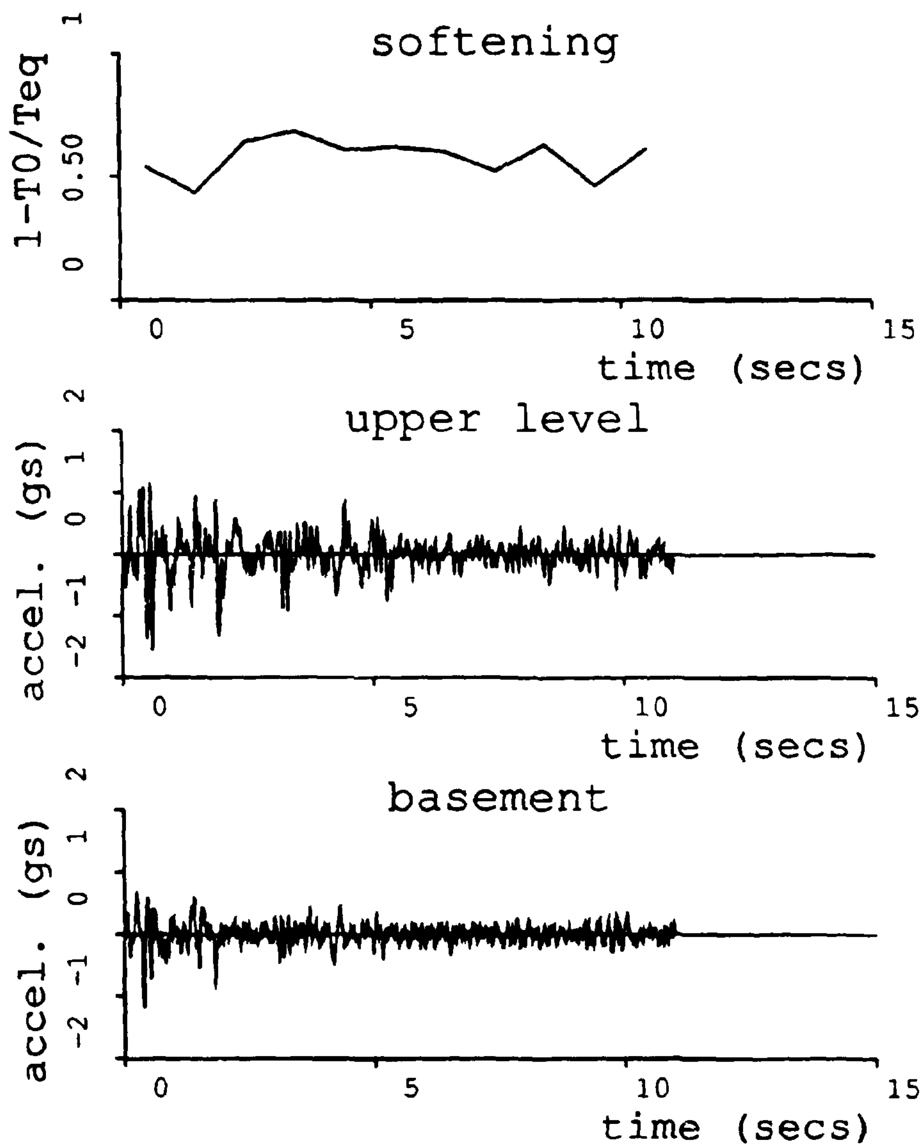

FIGURE 3-6 Recorded Acceleration and Estimated Softening for UIUC, Model FW2, Run 3

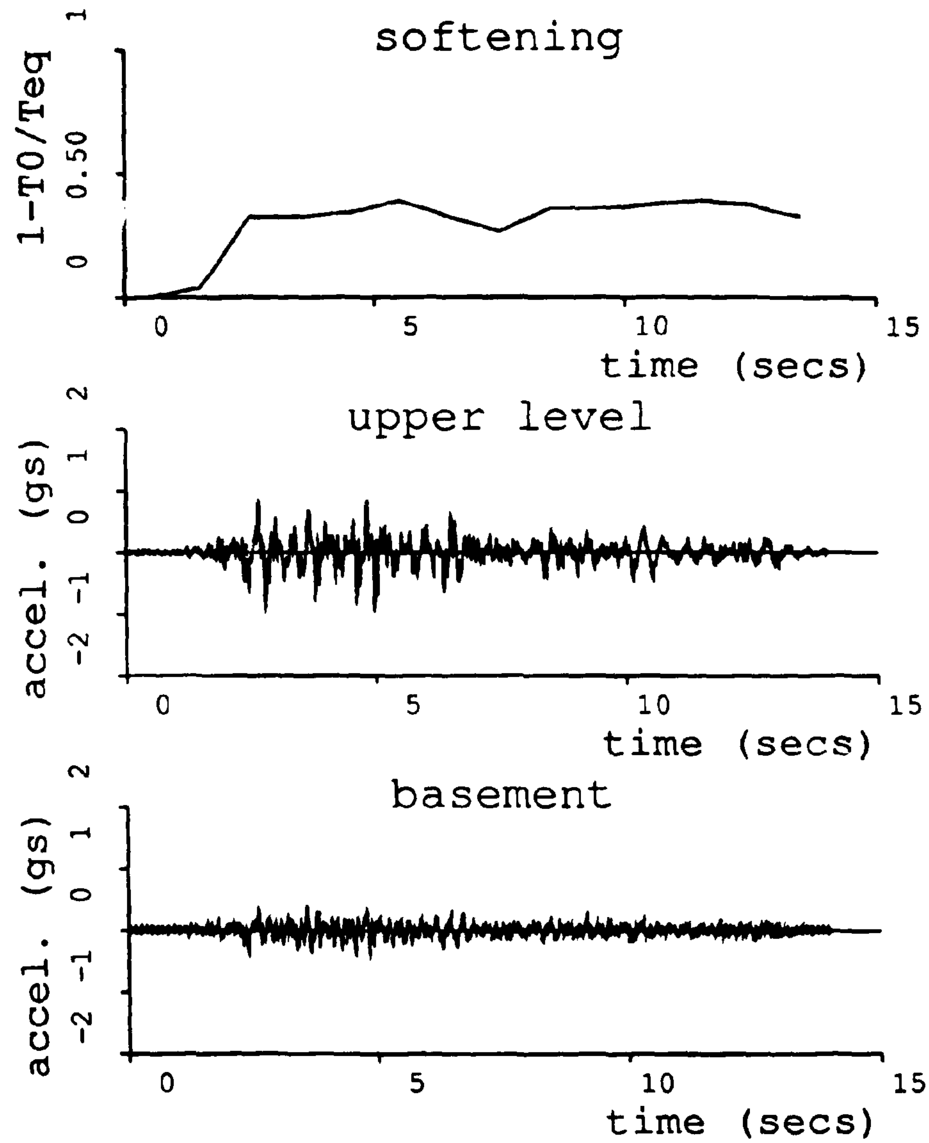

FIGURE 3-7 Recorded Acceleration and Estimated Softening for UIUC, Model FW3, Run 1

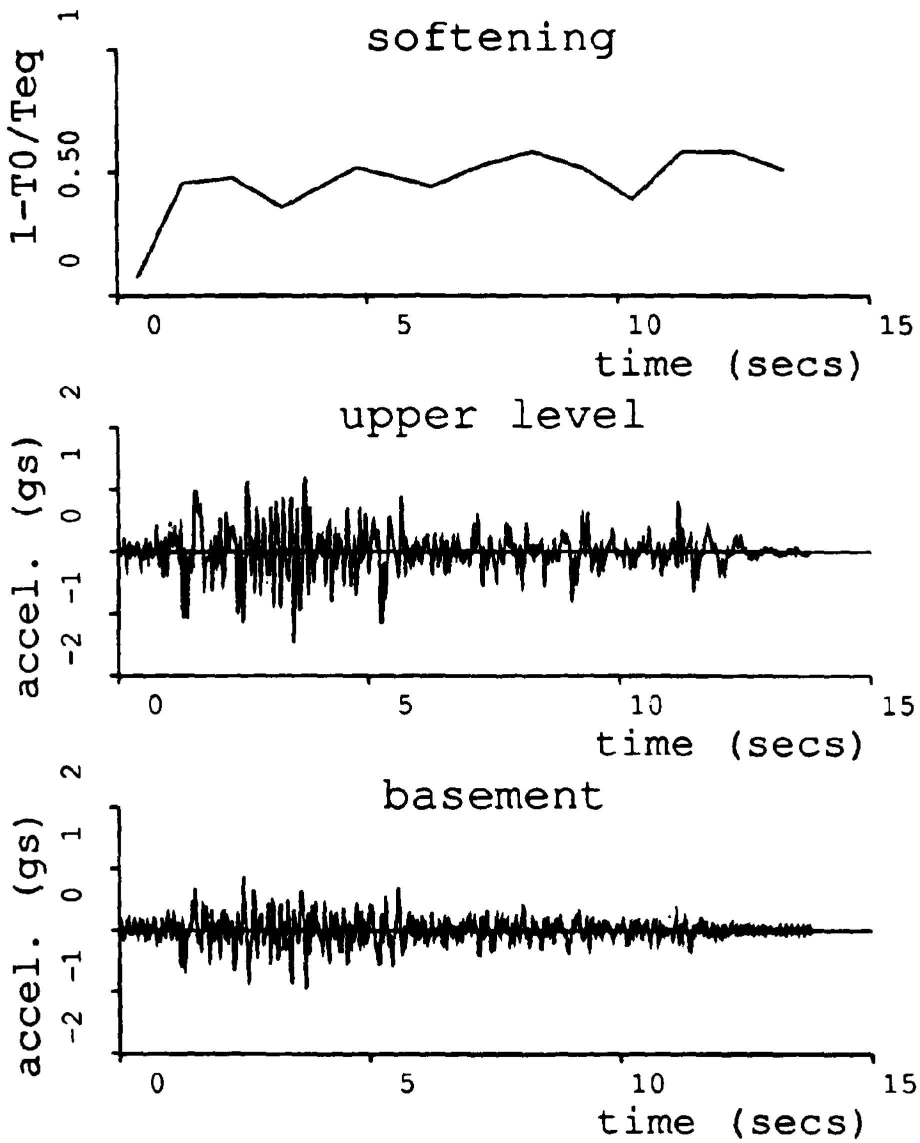

FIGURE 3-8 Recorded Acceleration and Estimated Softening for UIUC, Model FW3, Run 2

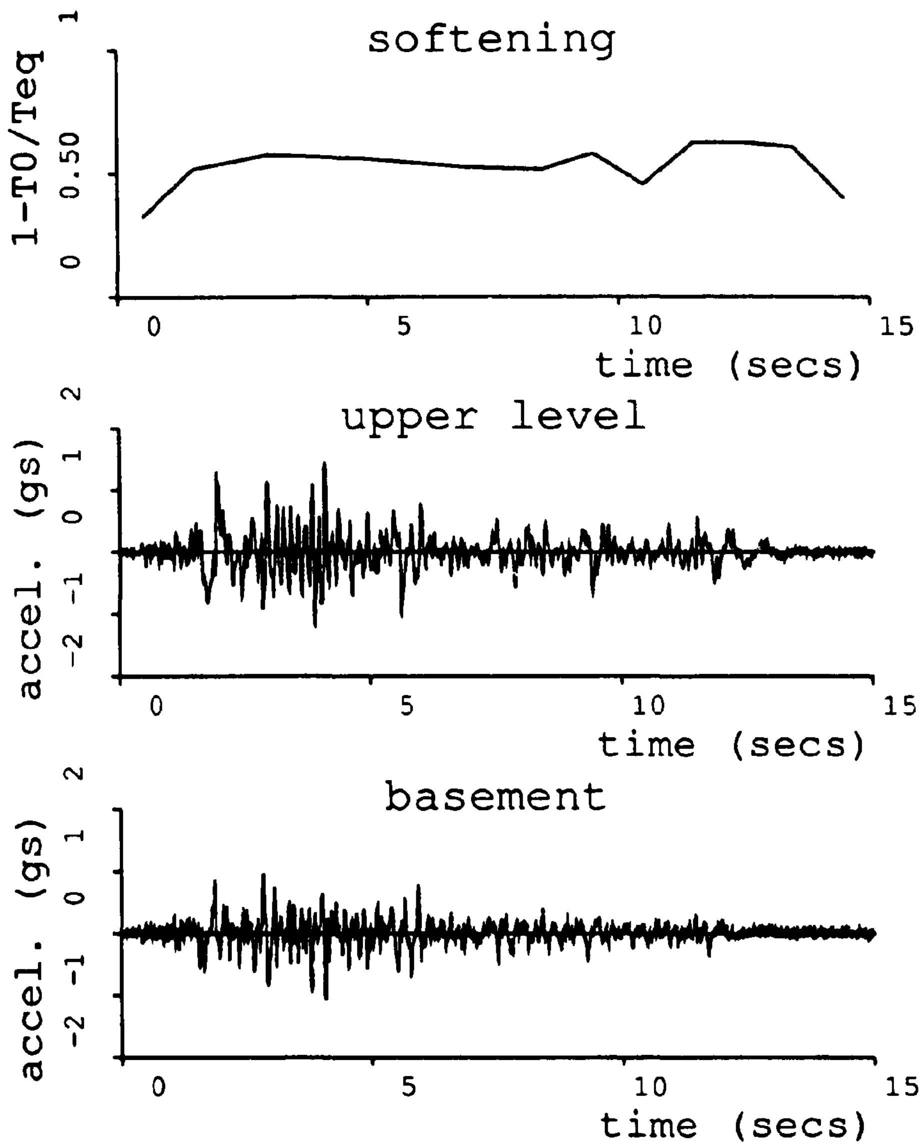

FIGURE 3-9 Recorded Acceleration and Estimated Softening for UIUC, Model FW3, Run 3

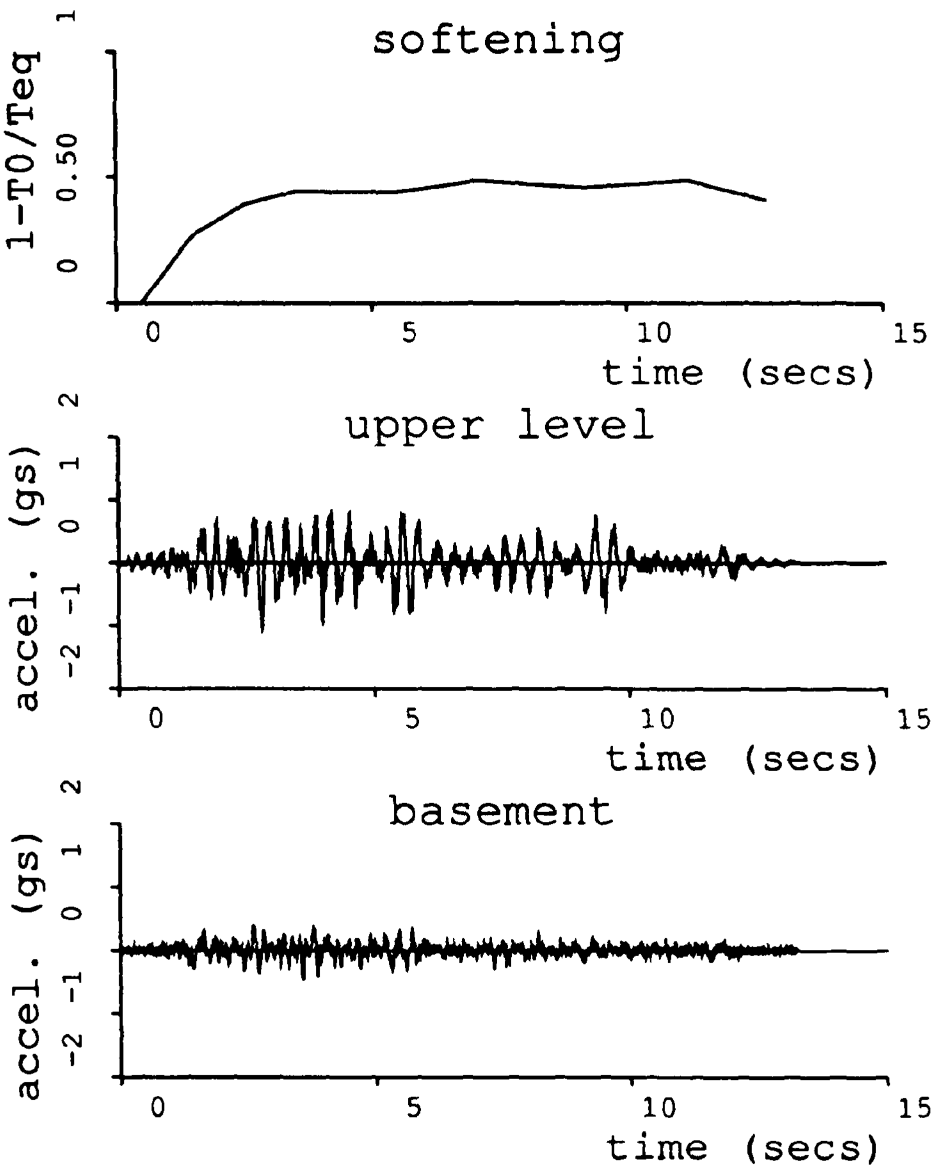

FIGURE 3-10 Recorded Acceleration and Estimated Softening for UIUC, Model FW4, Run 1

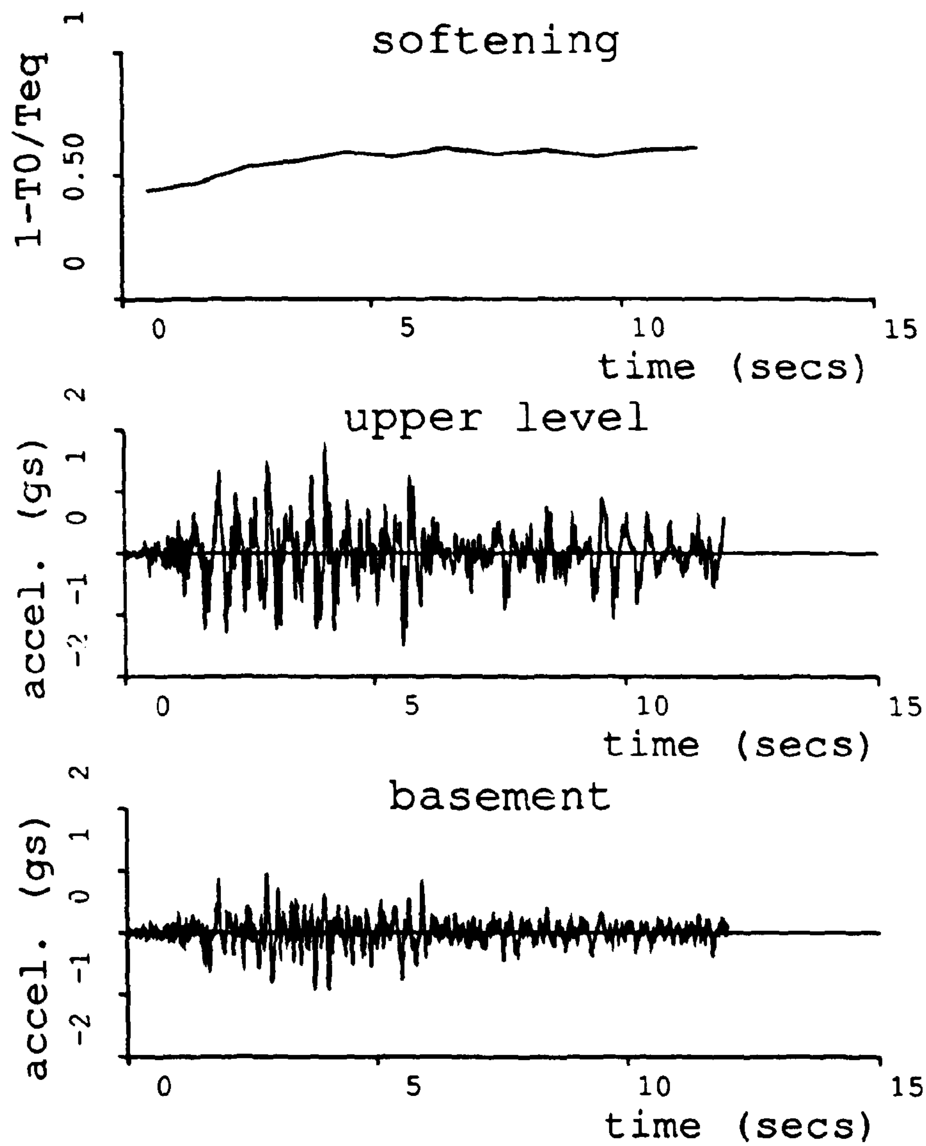

FIGURE 3-11 Recorded Acceleration and Estimated Softening for UIUC, Model FW4, Run 2

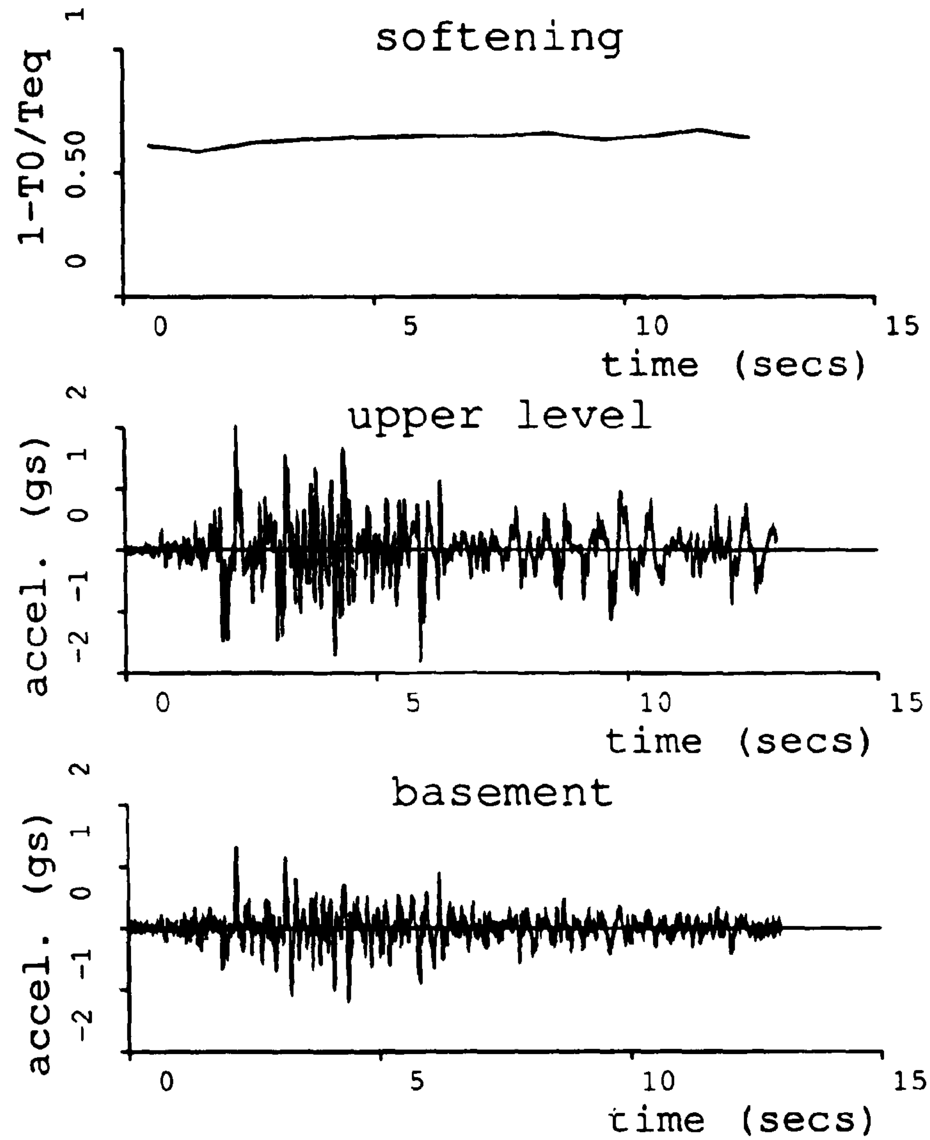

FIGURE 3-12 Recorded Acceleration and Estimated Softening for UIUC, Model FW4, Run 3

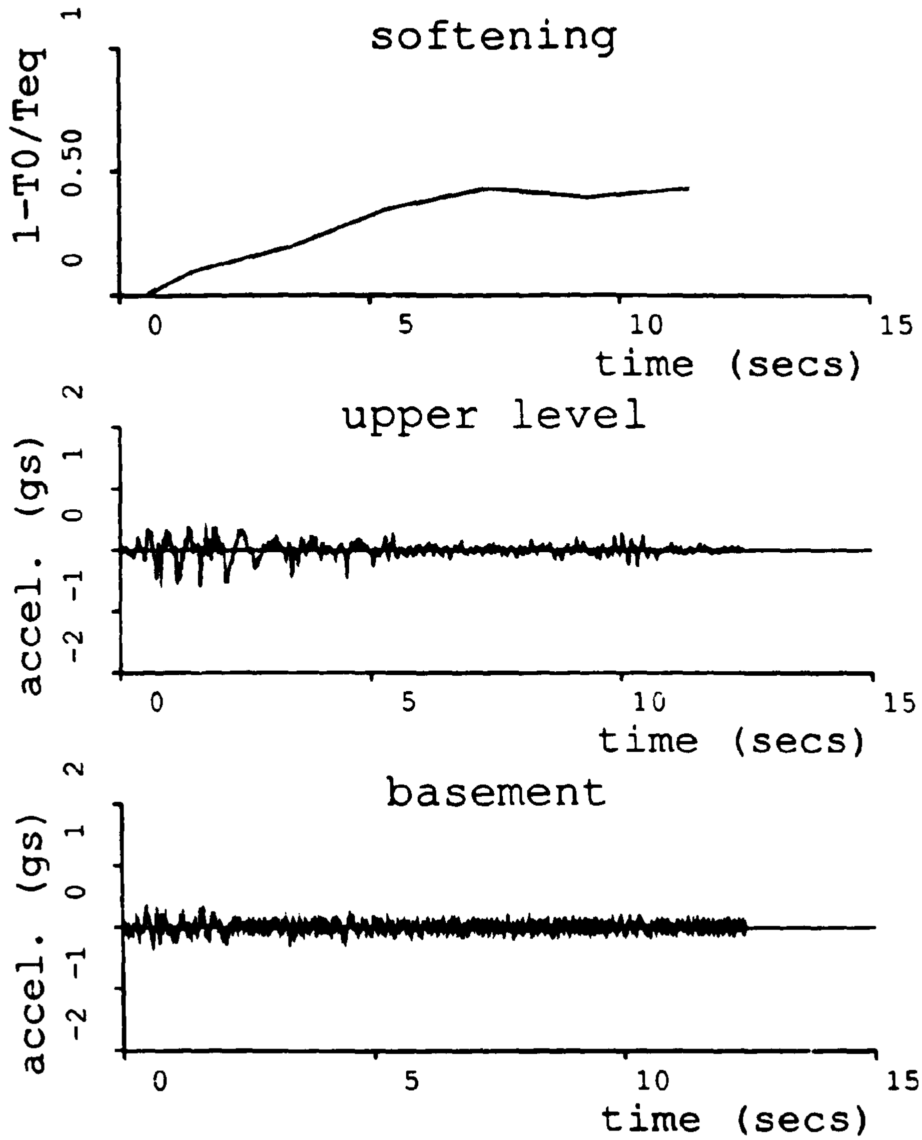

FIGURE 3-13 Recorded Acceleration and Estimated Softening for UIUC, Model H1, Run 1

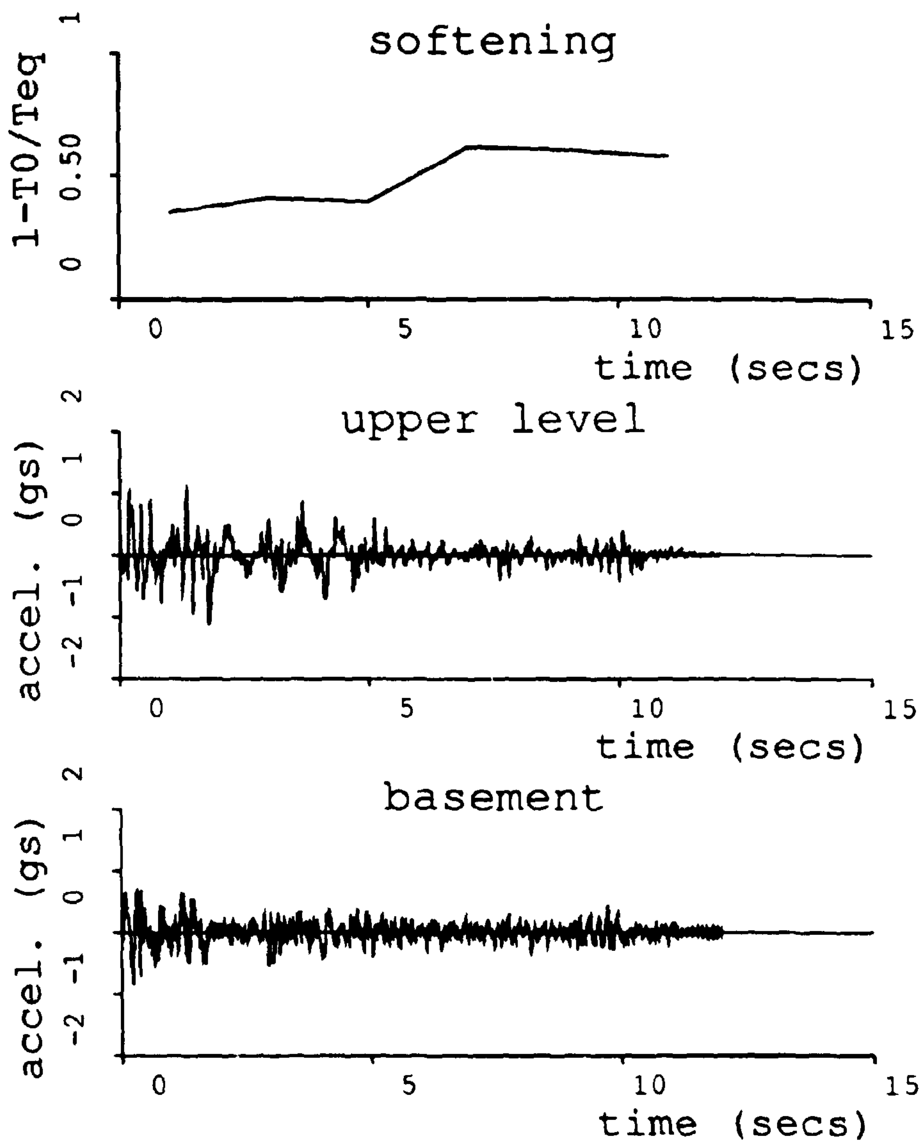

FIGURE 3-14 Recorded Acceleration and Estimated Softening for UIUC, Model H1, Run 2

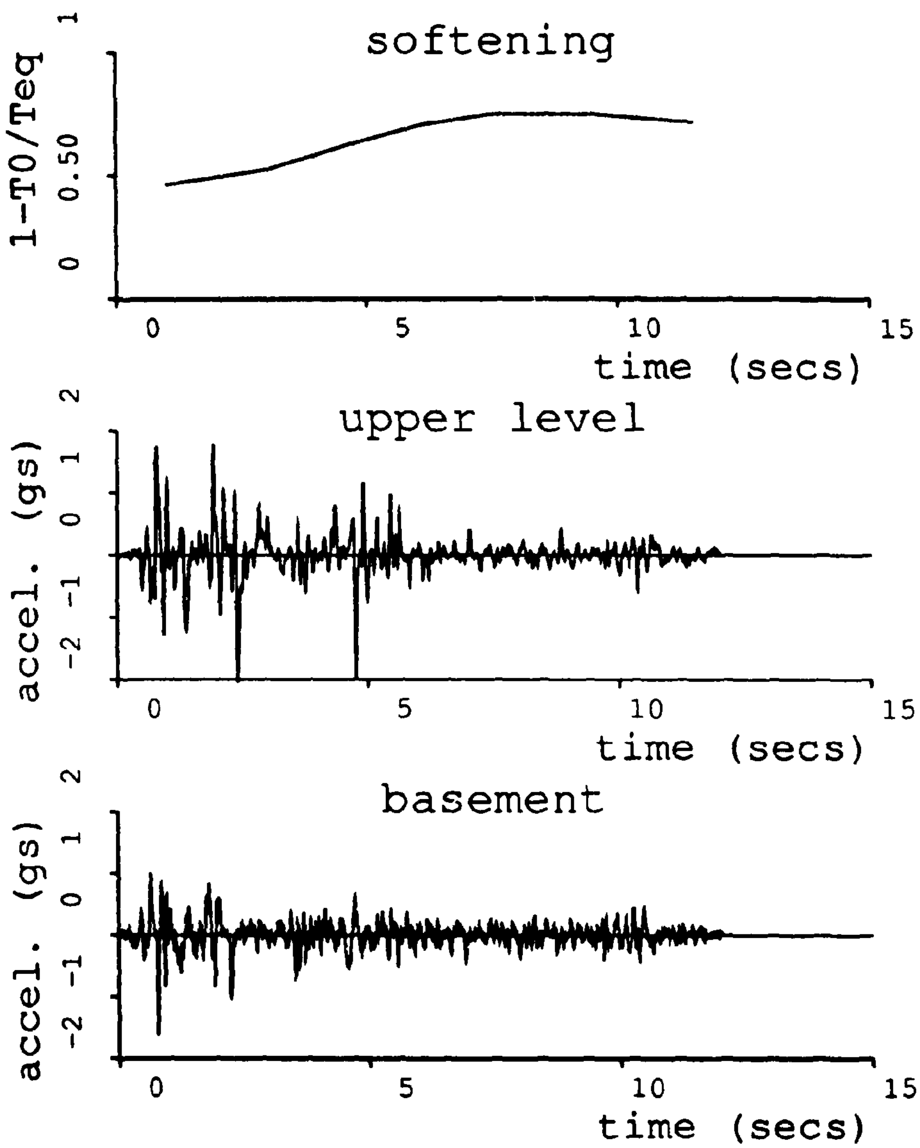

FIGURE 3-15 Recorded Acceleration and Estimated Softening for UIUC, Model H1, Run 3

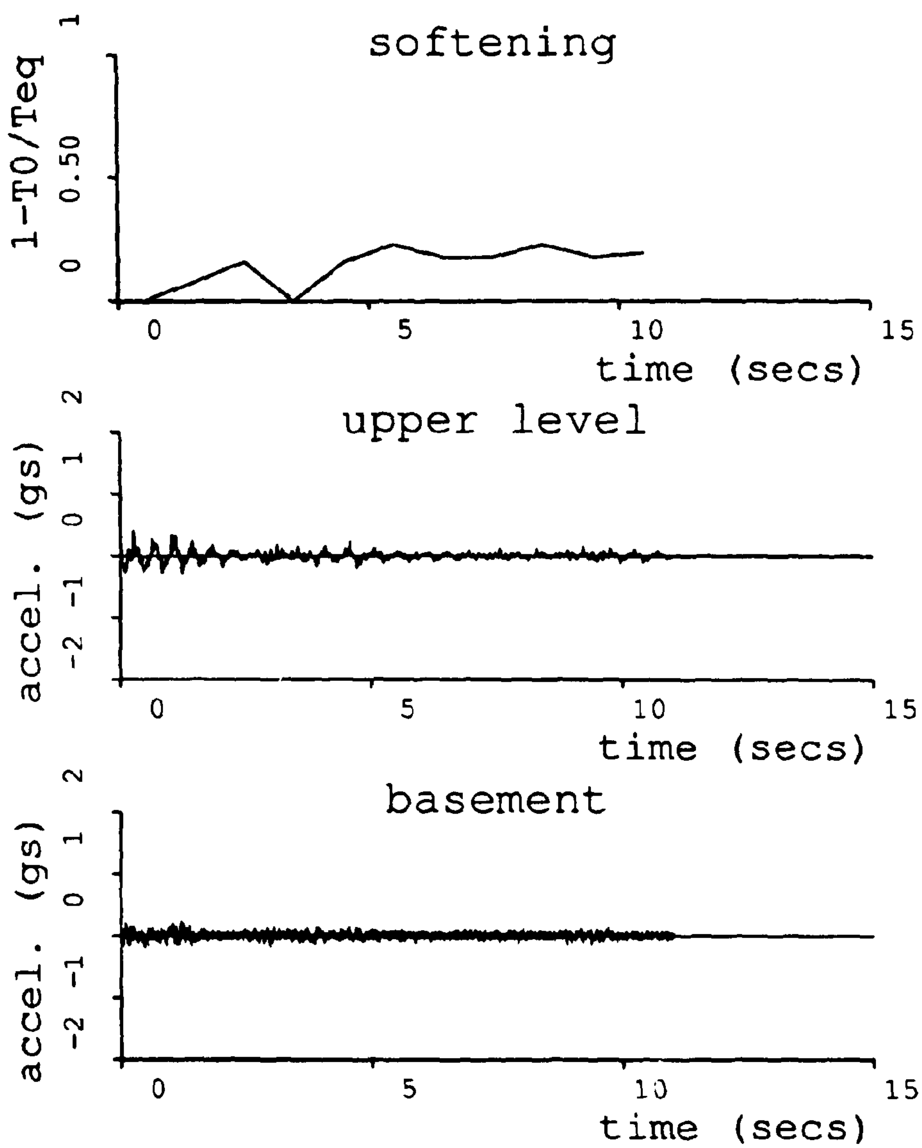

FIGURE 3-16 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 1

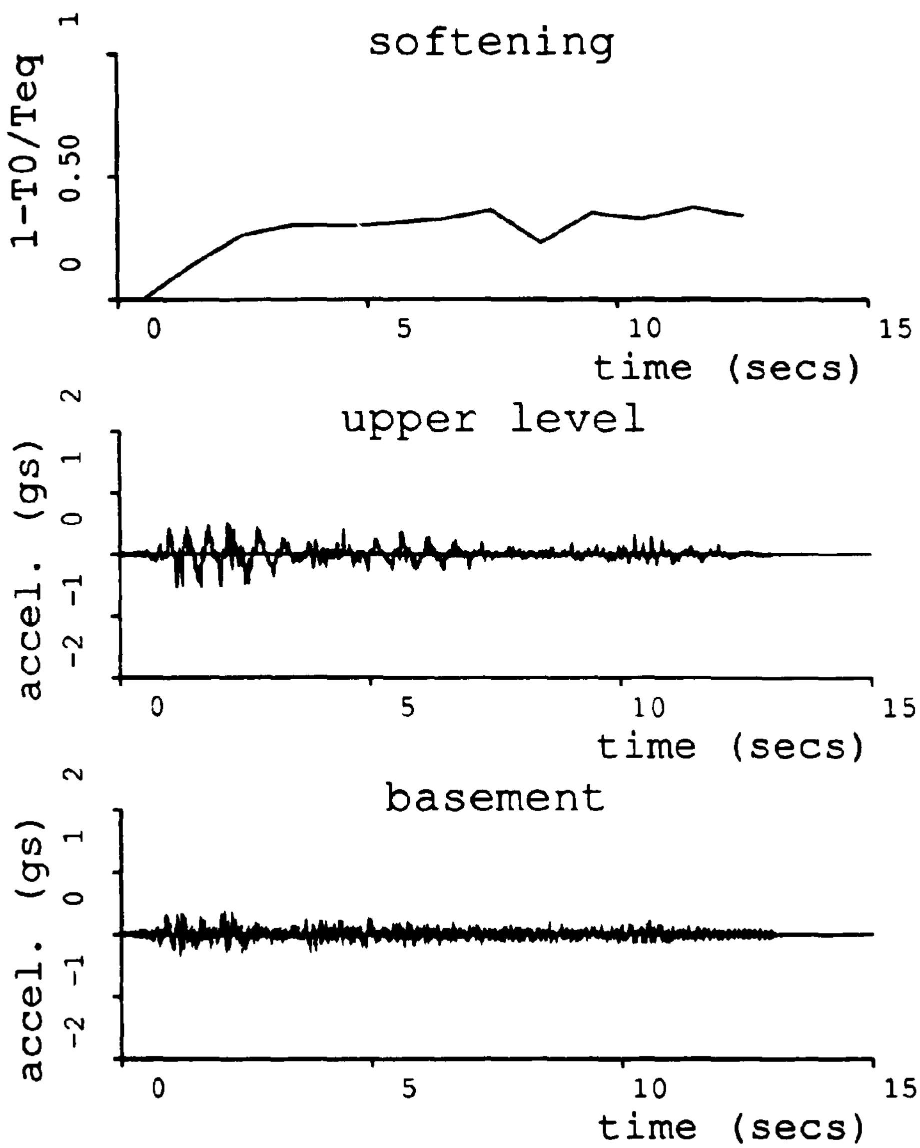

FIGURE 3-17 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 2

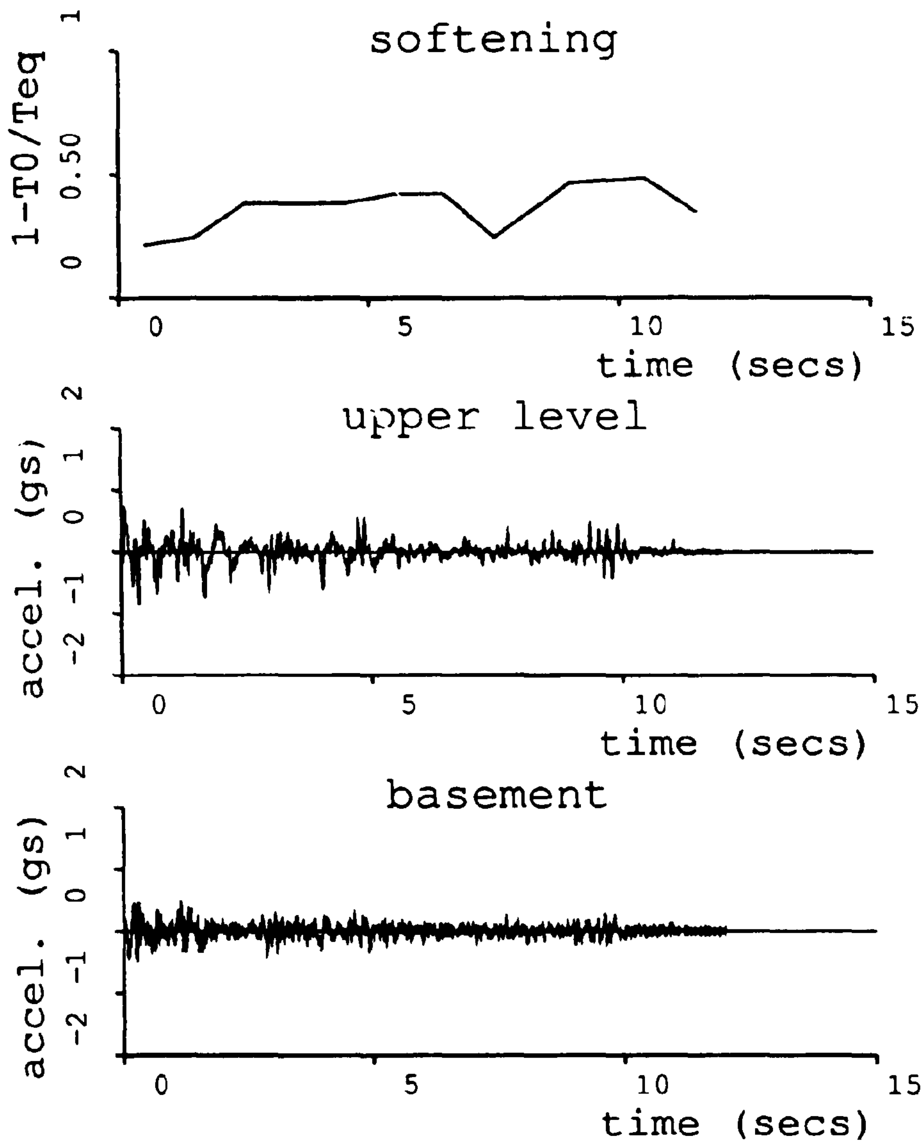

FIGURE 3-18 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 3

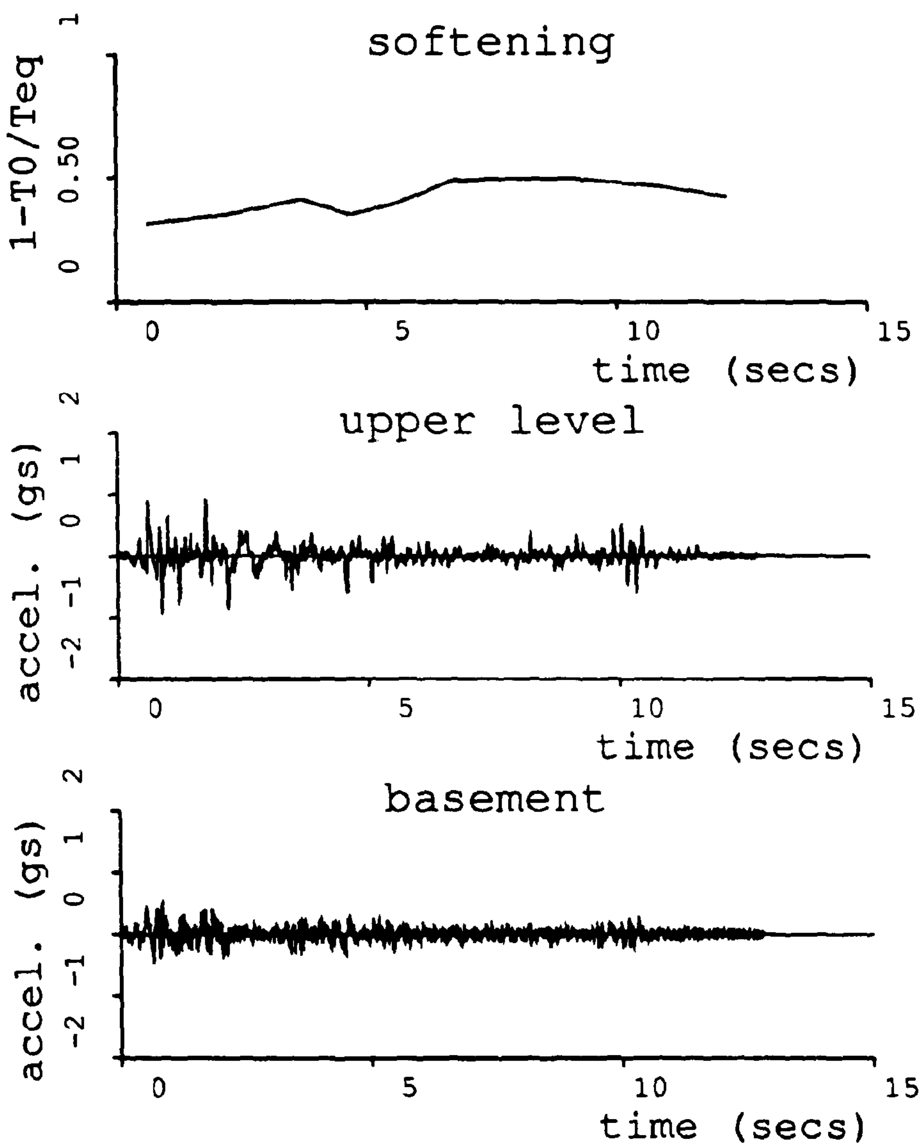

FIGURE 3-19 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 4

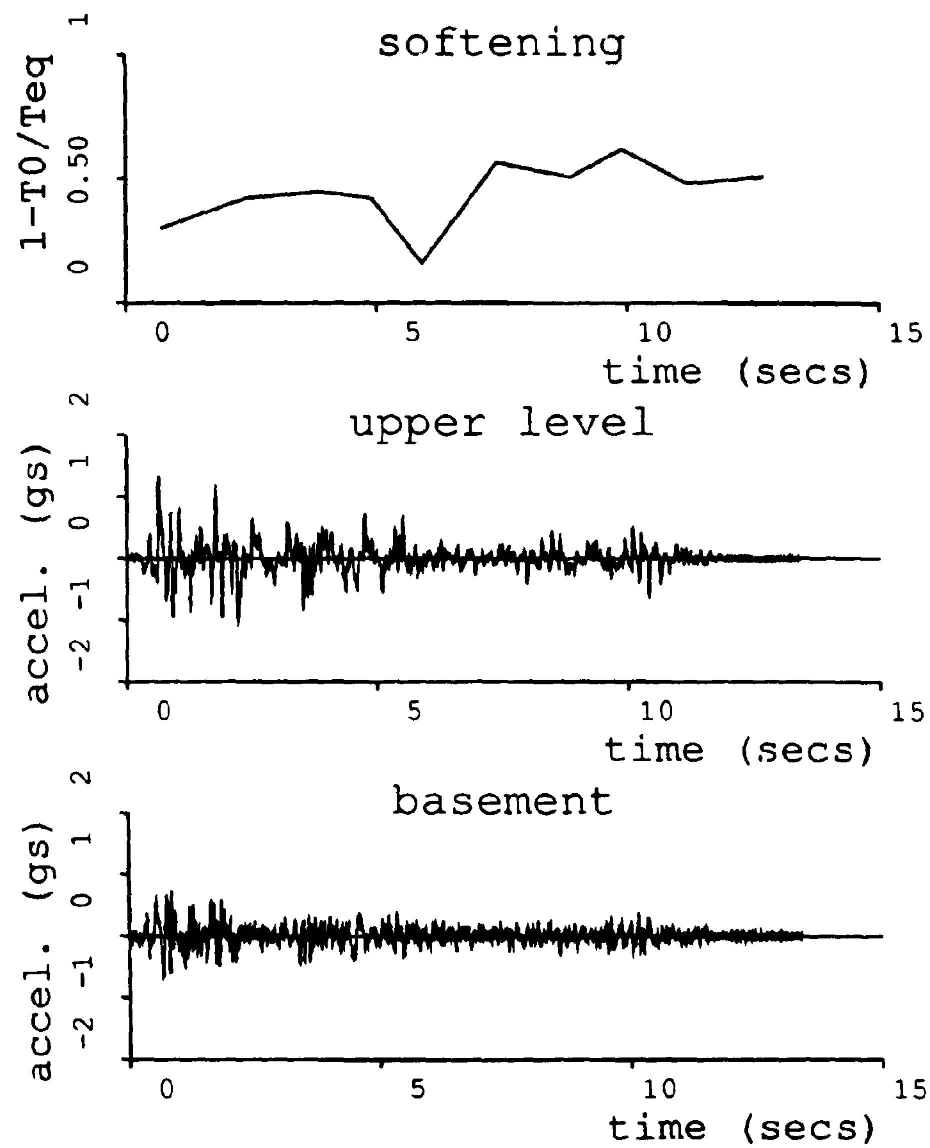

FIGURE 3-20 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 5

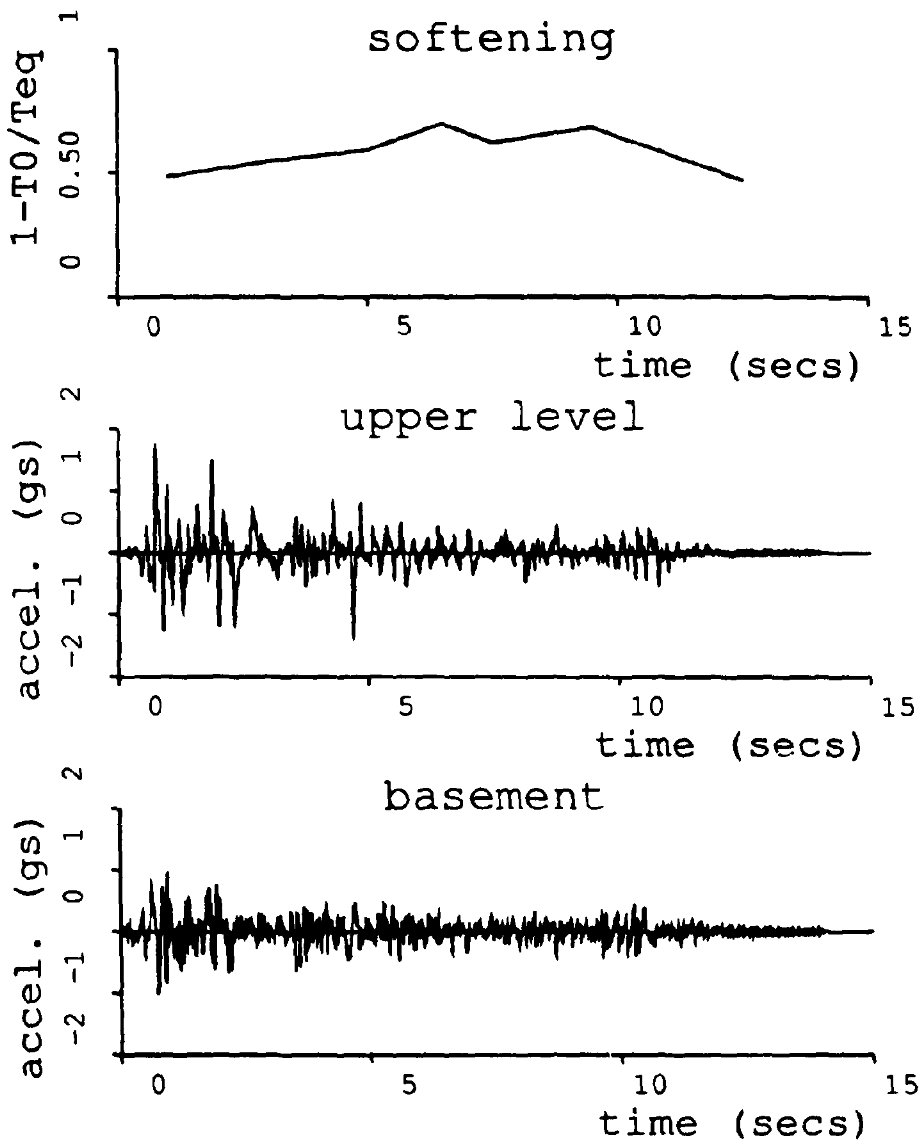

FIGURE 3-21 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 6

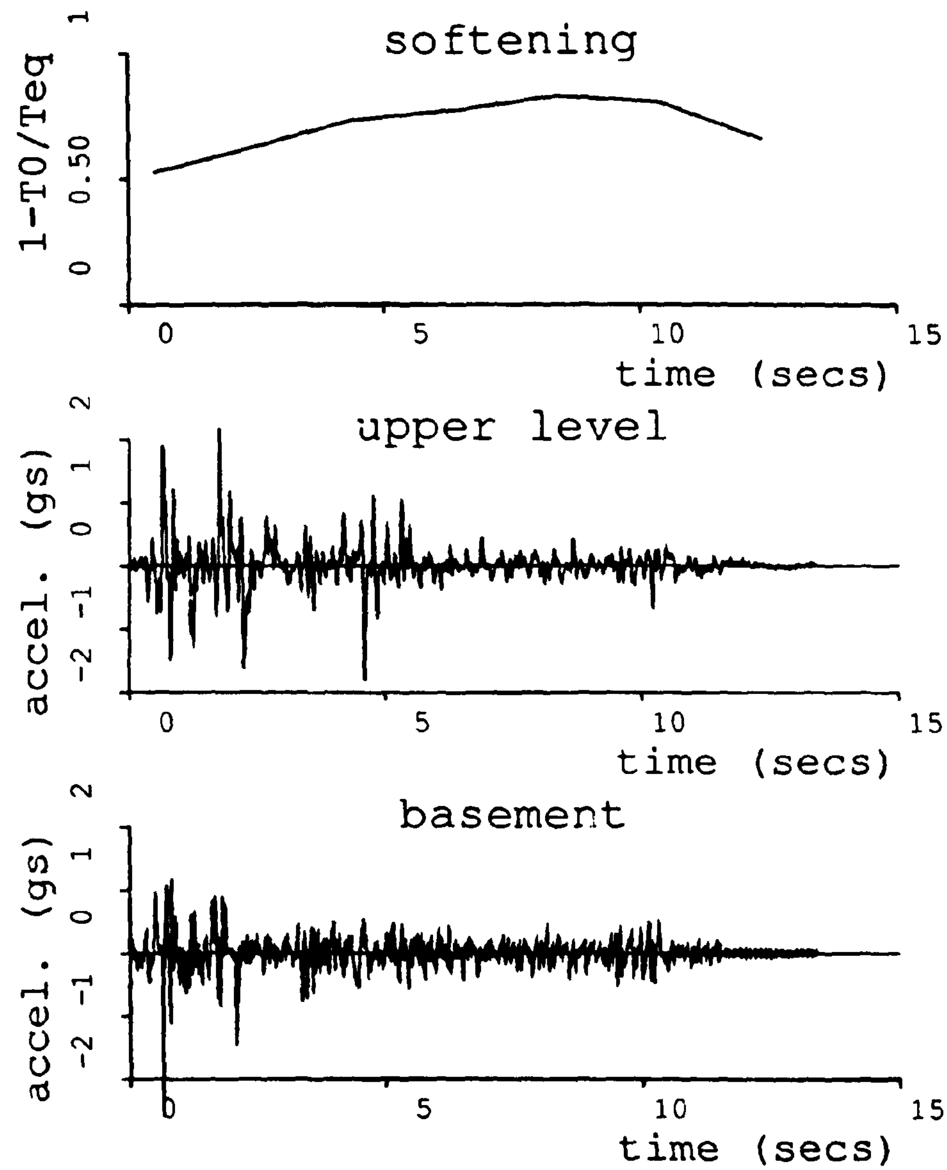

FIGURE 3-22 Recorded Acceleration and Estimated Softening for UIUC, Model H2, Run 7

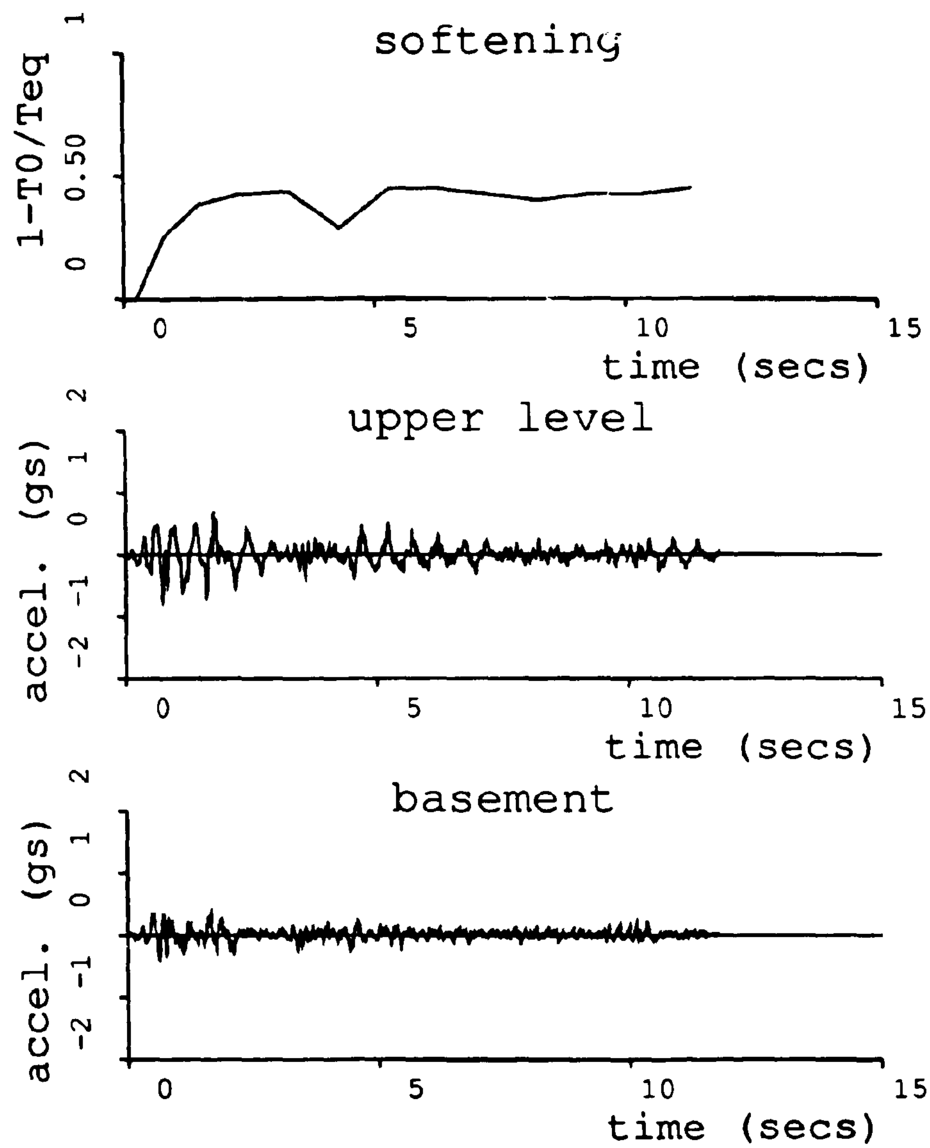

FIGURE 3-23 Recorded Acceleration and Estimated Softening for UIUC, Model MF1, Run 1

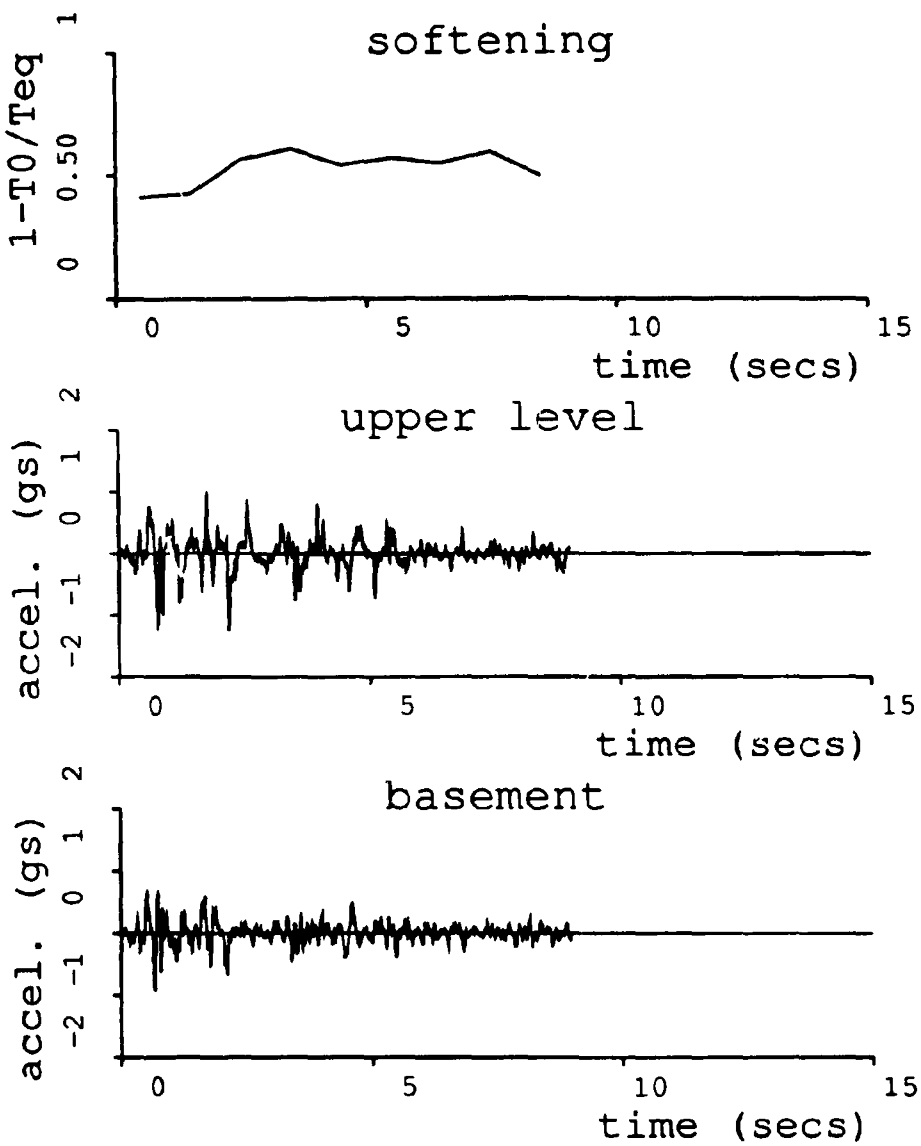

FIGURE 3-24 Recorded Acceleration and Estimated Softening for UIUC, Model MF1, Run 2

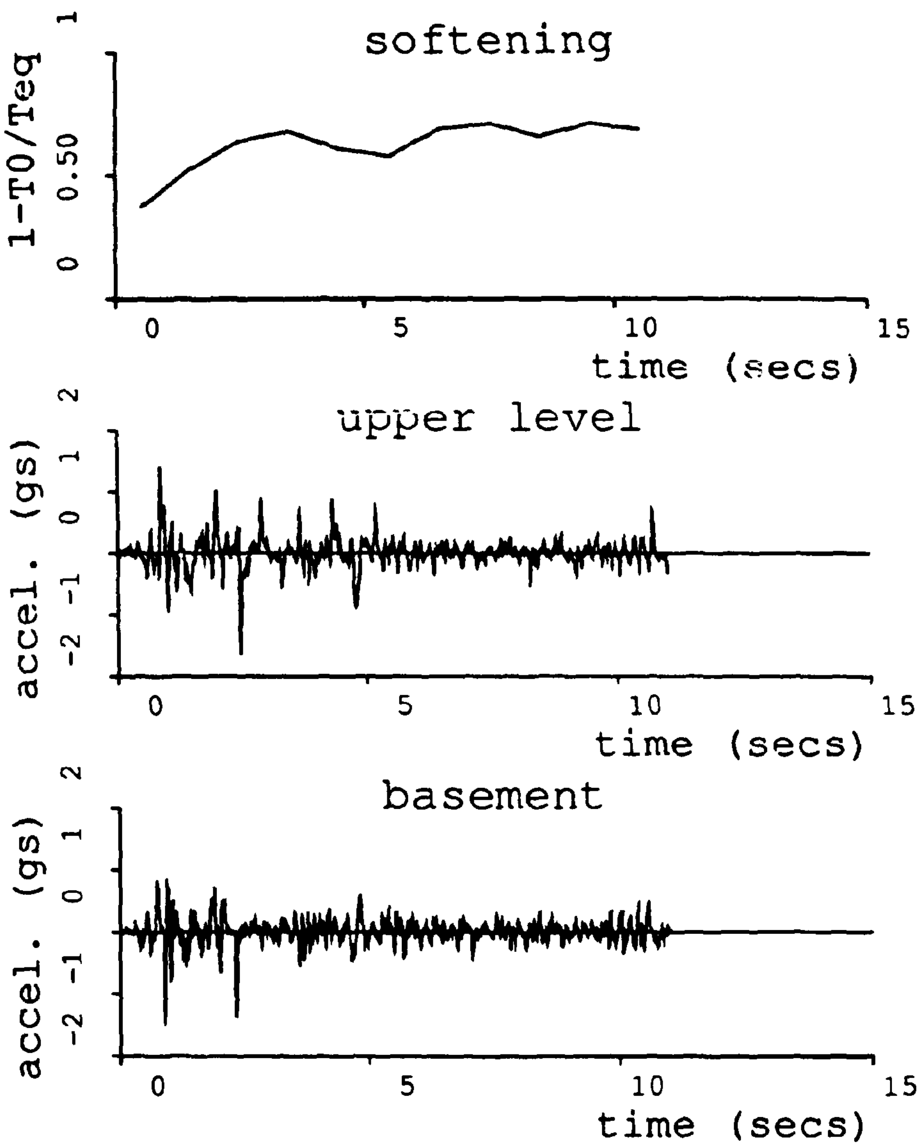

FIGURE 3-25 Recorded Acceleration and Estimated Softening for UIUC, Model MF1, Run 3

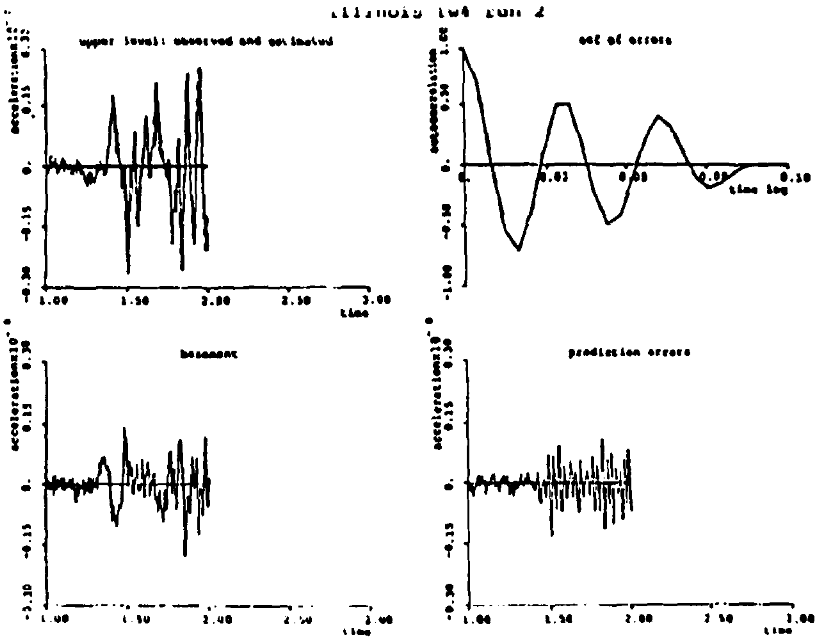

FIGURE 3-26b Example of Analysis Using a Two-Mode Model

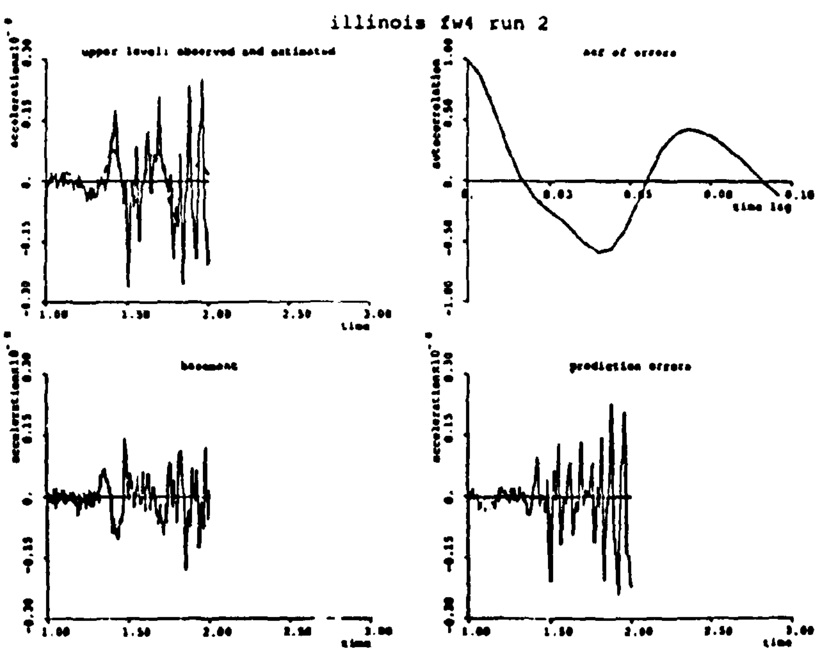

FIGURE 3-26a Example of Analysis Using a One-Mode Model

3.5. Identification of the Serviceability Limit State

Figure 3.27 shows a plot of the maximum softening versus the normalized intensity of the earthquake, i.e., the ratio of the intensity of the earthquake to which the structure is subjected to the intensity of the design earthquake. As it is customary, the intensity of an earthquake is defined as the peak acceleration recorded during the strong motion. It can be observed that the maximum softening consistently increases as the intensity increases, and that the curves relative to different structures are close to one another and similar in pattern. The values of the maximum softening computed are also consistent with the qualitative classification of damage described earlier. In figure 3.27, the data are divided into four horizontal bands, corresponding to different degrees of damage.

In practical damage detection the maximum softening is measured and the engineer wants to know whether the intensity of the earthquake was high enough to damage the structure. The lowest damaging intensity I_{damage} will not, in general, be known. Models of structures for shaking table tests are designed and built with much greater care than standard buildings. In particular their actual behavior is very close to analytical or numerical predictions. Furthermore, they are stiffened against out-of-plane (lateral and torsional) motion. For these reasons the "design" earthquake intensity relative to the models considered is a very meaningful quantity. The structures were expected to undergo moderate damage when subjected to an earthquake of the design intensity, while no damage was expected for weaker earthquakes. For the structures considered here, therefore, the design intensity I_{design} and the lowest damaging intensity I_{damage} coincide. An intensity ratio can be defined as the ratio of the earthquake intensity to the design intensity:

$$I = \frac{I_{earthquake}}{I_{design}} = \frac{I_{earthquake}}{I_{damage}} \tag{3.5.1}$$

The intensity ratio can be used to detect the insurgence of structural damage. When this ratio is less than one, the structure may be considered safe, otherwise some damage must be expected. In figure 3.28 the intensity ratio is plotted versus the linear maximum softening. The functional form of the relationship between the two quantities is certainly complicated. Nonetheless, as most data lie in the intermediate damage classes, it can be expected that a linear regression would yield results valid in

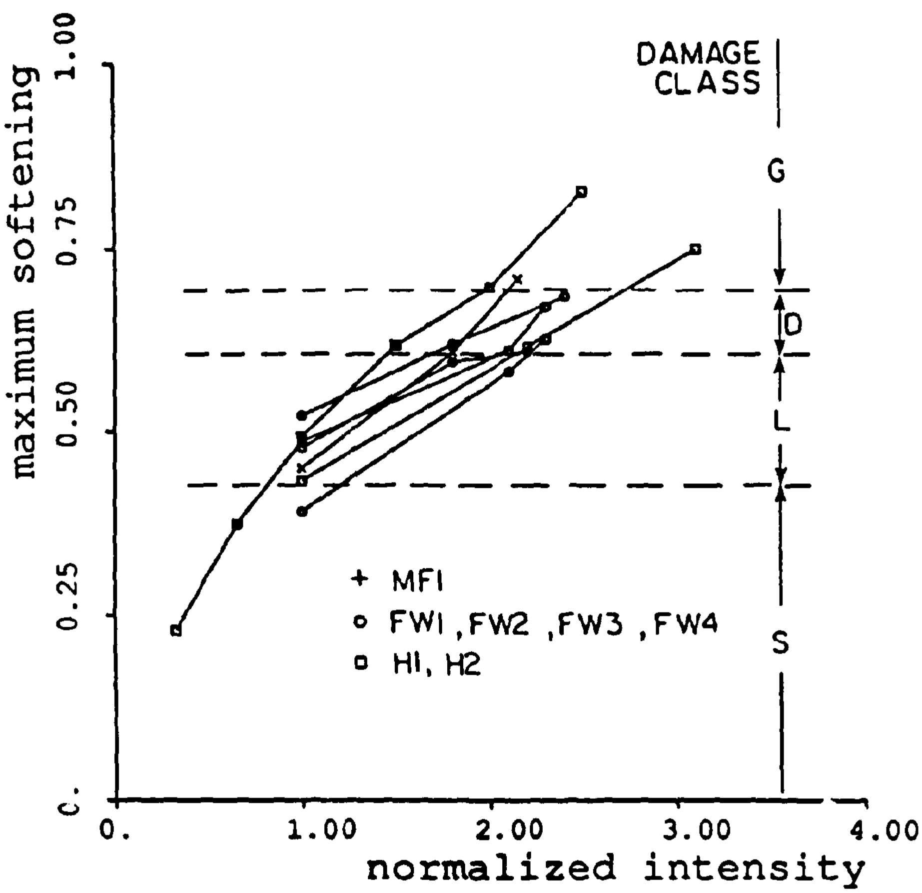

FIGURE 3-27 Maximum Softening vs. Normalized Intensity (Damage Classification After Toussi and Yao (1983))

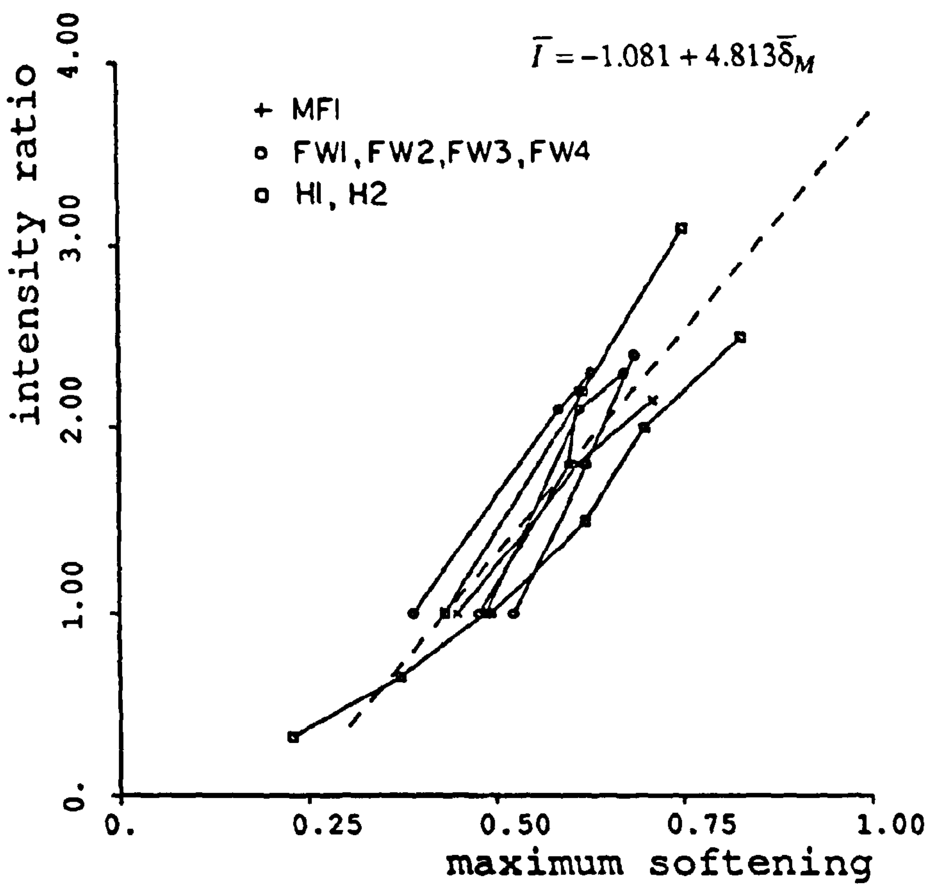

FIGURE 3-28 Intensity Ratio vs. Maximum Softening (Dotted Line: Linear Regression Line)

the neighborhood of $I = 1$, that is, the damage threshold. A linear model:

$$I = \alpha + \beta\delta_M \qquad (3.5.2)$$

has been fitted to the data of figure 3.28. The least square estimates of the regression parameters are:

$$\bar{\alpha} = -1.081 \qquad (3.5.3a)$$

$$\bar{\beta} = 4.813 \qquad (3.5.3b)$$

with standard deviations:

$$\sigma_\alpha = 0.260 \qquad (3.5.4a)$$

$$\sigma_\beta = 0.446 \qquad (3.5.4b)$$

For simplicity the cross covariance $\sigma_{\alpha\beta}$ of the regression parameters has been neglected. α and β are therefore treated as independent random variables. Equation (3.3) can be used for damage detection. The maximum softening $\bar{\delta}_M$ is estimated from the analysis of strong motion records. An expected value $\bar{I}$ for the intensity ratio is easily computed:

$$\bar{I} = \bar{\alpha} + \bar{\beta}\bar{\delta}_M \qquad (3.5.5)$$

The uncertainties present in the problem can be easily taken into account if it is asssumed that α, β and δ_M are independent gaussian random variables with means $\bar{\alpha}$, $\bar{\beta}$ and $\bar{\delta}_M$ and standard deviation σ_α, σ_β and σ_δ. Then the intensity ratio I will be gaussian as well, with mean $\bar{I}$ and variance σ_I.

$$\sigma_I = \sigma_\alpha^2 + \sigma_\beta^2\sigma_\delta^2 + \bar{\beta}^2\sigma_\delta^2 + \bar{\delta}_M^2\sigma_\beta^2 \qquad (3.5.6)$$

It is now an easy exercise to compute the probability that the damaging earthquake intensity has been exceeded. The maximum softening δ_M has already been estimated. Therefore the probability of damage computed takes into account the previous knowledge of $\bar{I}$.

$$P\,[\text{structure is damaged} \mid \bar{I}] = P\,[I > 1 \mid \bar{I}] = P\,[I - \bar{I} > 1 - \bar{I}] =$$

$$= P\left[\frac{I-\bar{I}}{\sigma_I} > \frac{1-\bar{I}}{\sigma_I}\right] = \int_{\frac{1-\bar{I}}{\sigma_I}}^{\infty} \frac{1}{\sqrt{2\pi}} e^{-\frac{x^2}{2}} dx = erf\left(\frac{1-\bar{I}}{\sigma_I}\right) \tag{3.5.8}$$

A conservative estimate of the standard deviation σ_δ of the maximum softening is:

$$\sigma_\delta = \delta_M \left(\frac{\sigma_{T_0}}{T_0} + \frac{\sigma_{T_{max}}}{T_{max}}\right) \tag{3.5.9}$$

where $T_0 = (T_0)_{initial}$ and $T_{max} = (T_0)_{max}$.

From the analysis of strong motion records it is therefore possible to derive the probability that the structure has undergone some damage and therefore needs to be serviced. Strictly speaking, these results are valid only for moderately slender reinforced concrete structures, whose resistance to horizontal load is provided by moment resistant frames or shear walls. On the other hand, most buildings found in highly seismic areas of the U.S. belong to this category. It could also be argued that the definition of serviceability limit state based on the maximum softening depends upon the scale of the structure considered, and that the criterion (3.5.8) may not be valid for large scale structures. The applications described in the next chapter, however, suggest that this method for the detection of seismic structural damage can be applied to larger scale models as well as to full-scale structures.

SECTION 4: DETECTION OF SEISMIC STRUCTURAL DAMAGE

4.1. Description of the Data Analyzed

In this chapter, the criterion (3.5.8) is applied to cases of buildings that sustained different levels of structural damage and to a 1/5th scale, seven-storied building that was tested at the University of California at Berkeley (UCB) within the U.S.-Japan joint research project (Bertero et al., 1984). The analysis of the shaking table experiments presented here is limited to the five tests for which some information regarding structural damage is available. The strong motion data analyzed include five records from the San Fernando earthquake (1971) and two records from the Imperial Valley earthquake (1979), relative to the Imperial County Service building.

4.2. Analysis of shaking table experiments

Series of seismic simulations on shaking tables constitute an excellent test for damage assessment procedures. The experimental program considered here was carried out at UCB by Bertero and his associates (1984) within the U.S.-Japan joint research project. The structure tested was a 1/5th scale model of a seven-storied reinforced concrete building designed according to the Uniform Building Code. Resistance to lateral loads was provided by a moment resistant frame acting in parallel with a shear wall. After a series of low amplitude tests, the structure was subjected to simulated seismic excitation of increasing intensity until collapse occurred at the first floor. The structure was then repaired and tested again, until a second collapse occurred.

The experimenters have reported information on the damage state subsequent to seven tests. Records from six of these tests were available and have been analyzed. The acceleration records, along with the evolution of the apparent softening, are shown in figures 4.1 through 4.6. The procedure developed in the last chapter has been used for the detection of seismic damage. From the analysis of the acceleration records relative to each test, the maximum softening and its standard deviation were computed; hence the expected intensity ratio and its standard deviation were obtained.

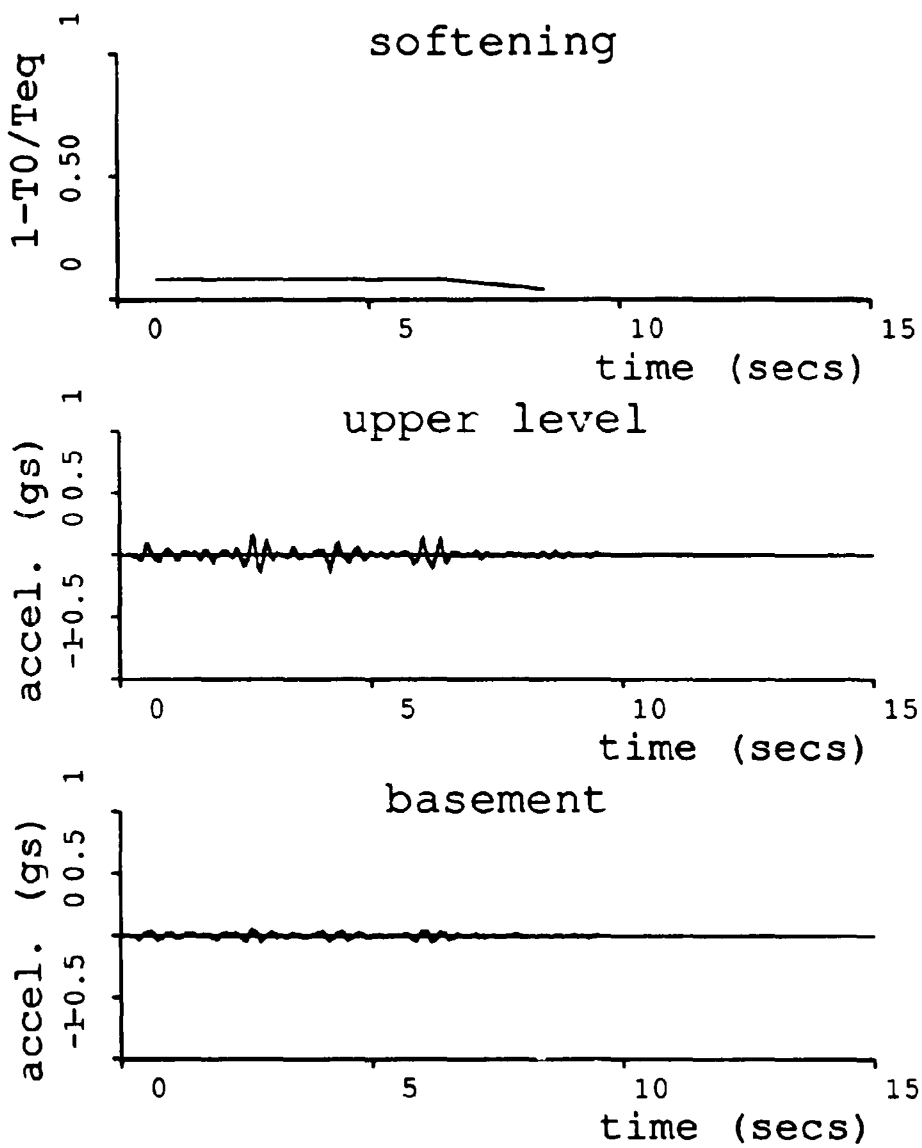

FIGURE 4-1 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.056gs

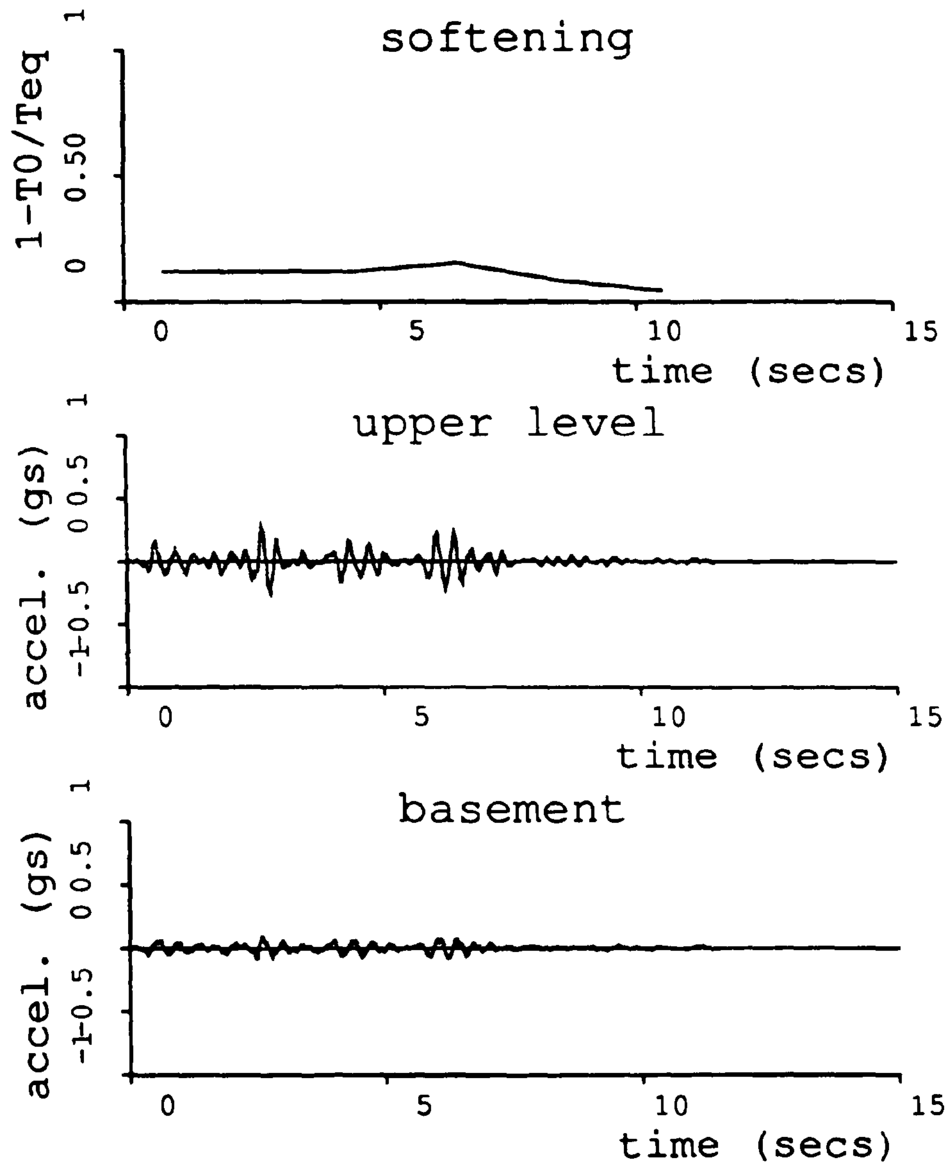

FIGURE 4-2 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.097gs

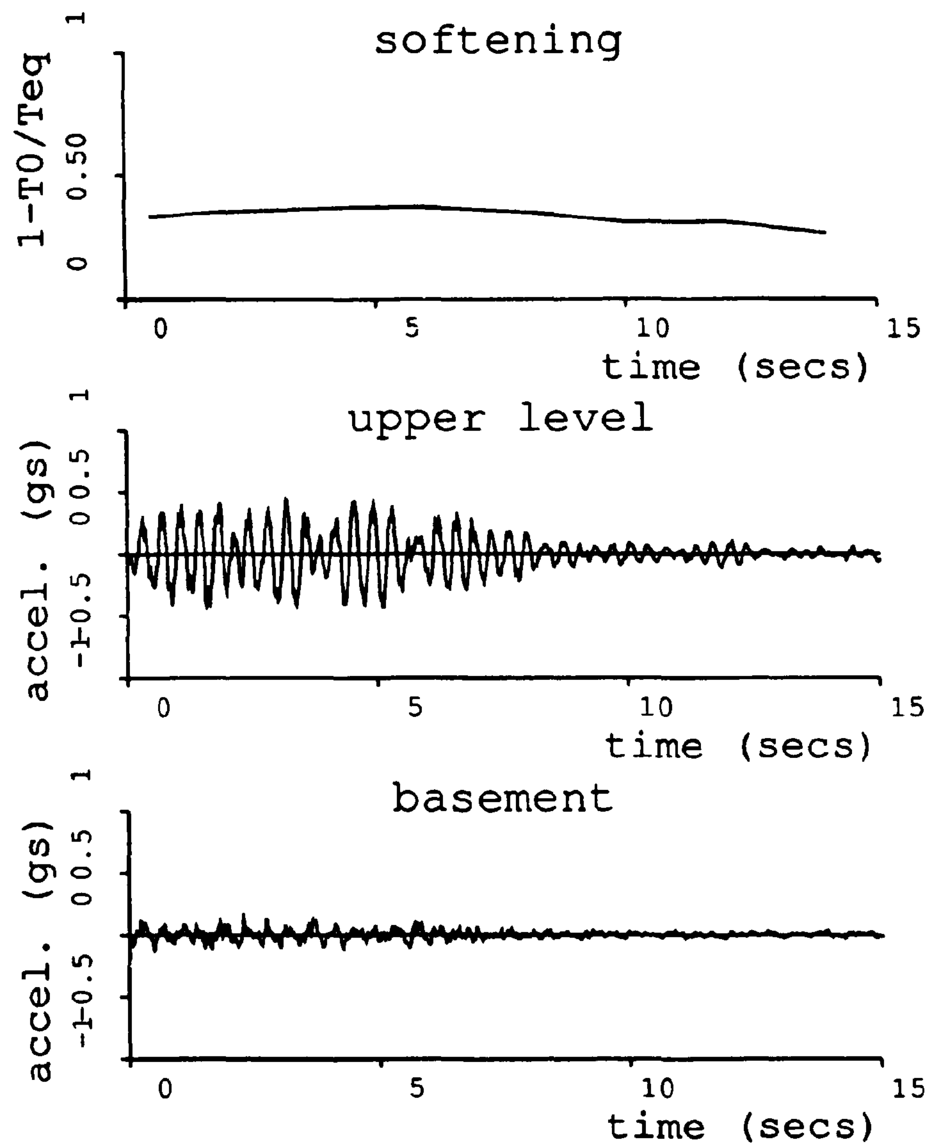

FIGURE 4-3 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.147gs

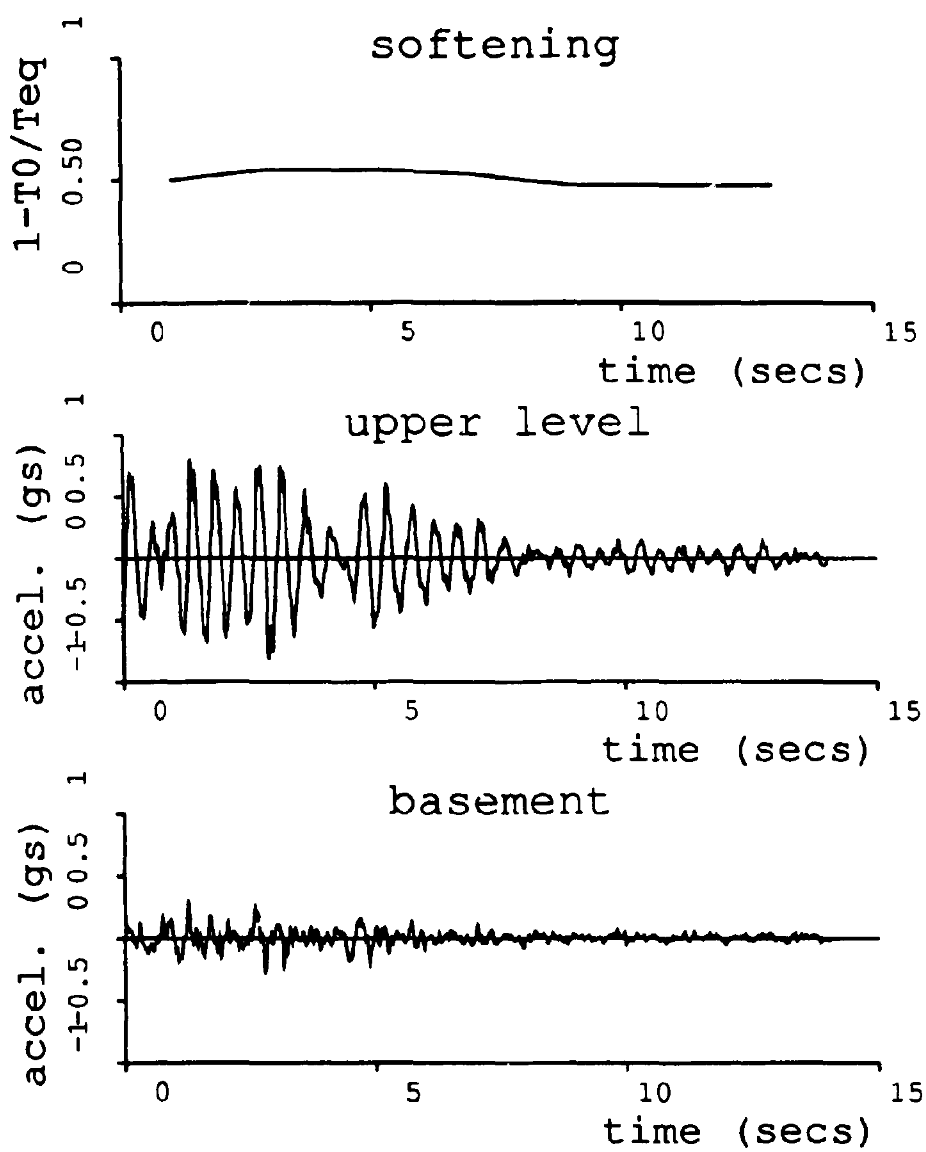

FIGURE 4-4 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.280gs

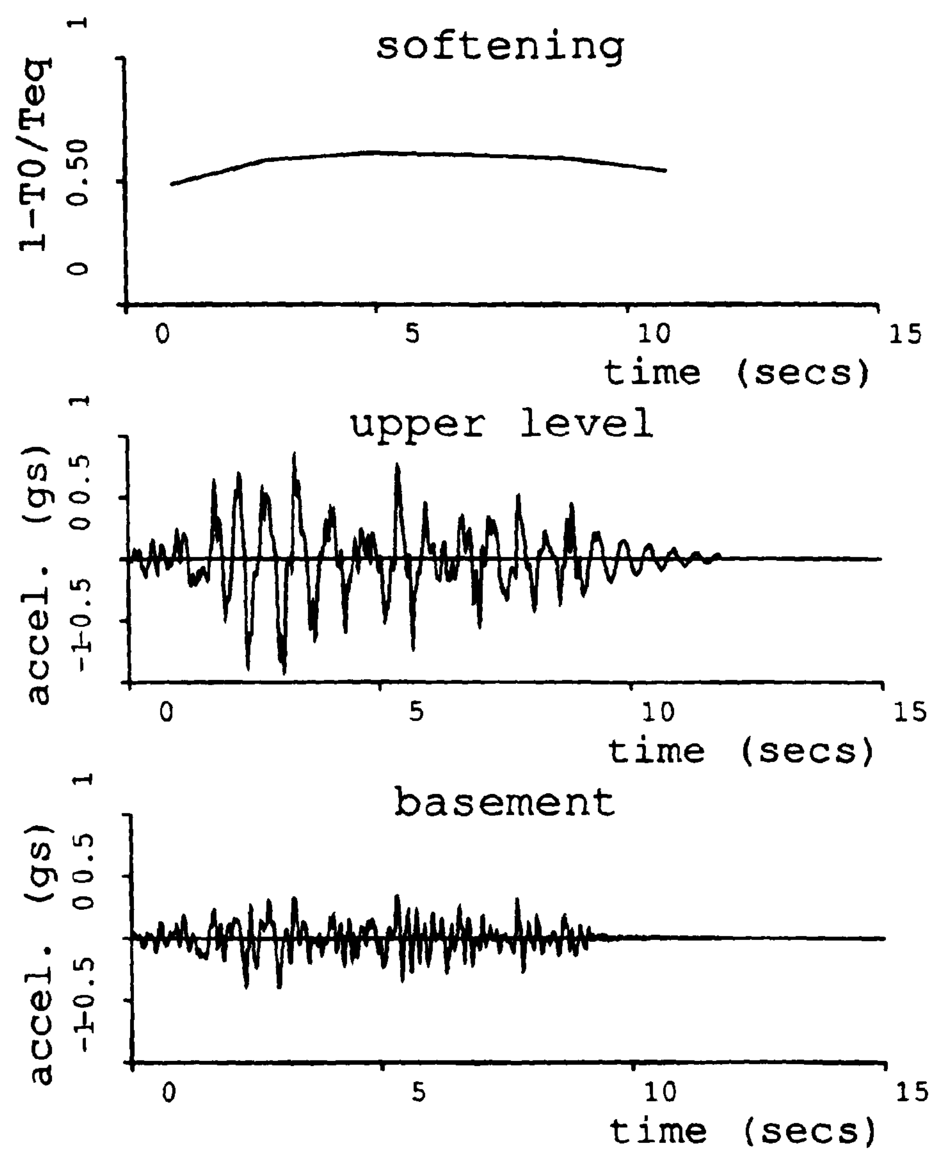

FIGURE 4-5 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.400gs

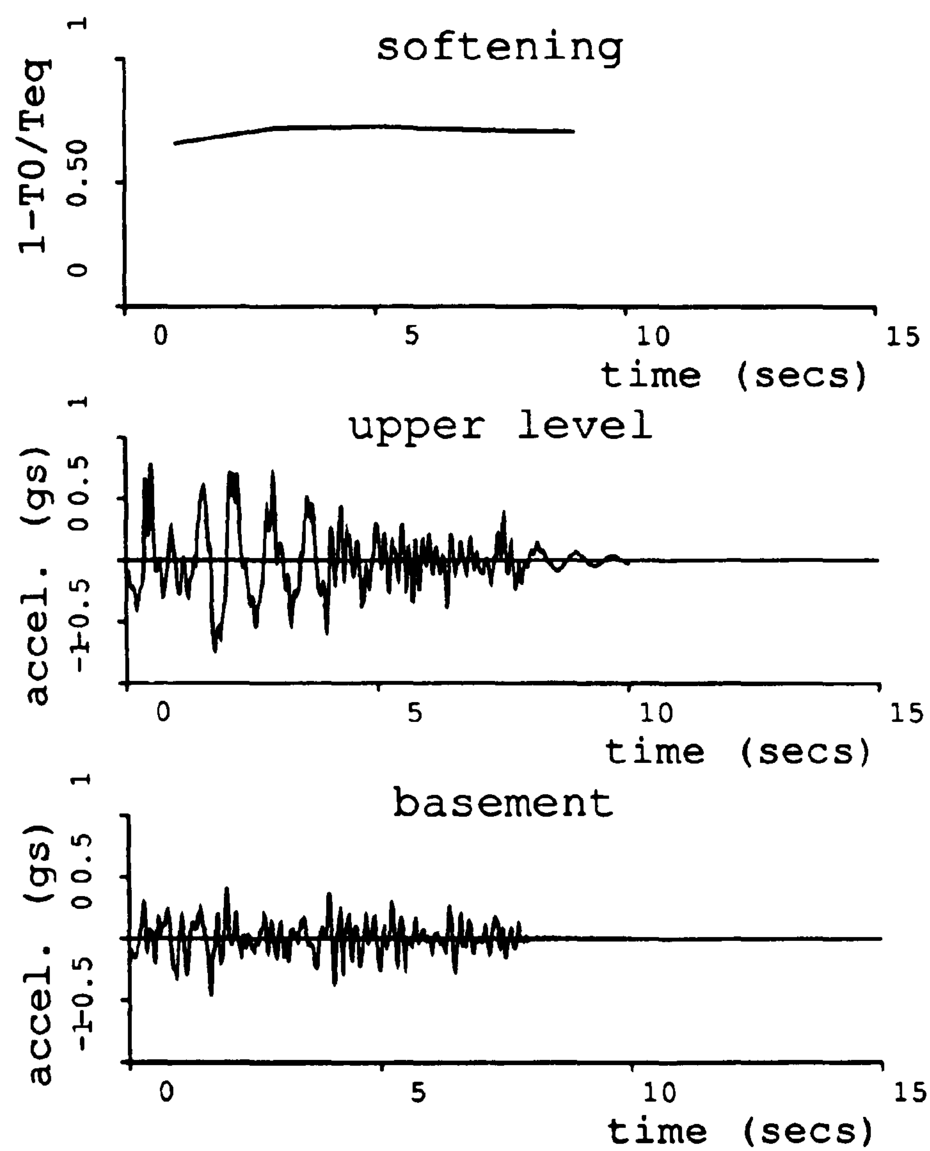

FIGURE 4-6 Recorded Acceleration and Estimated Softening for UCB, RC Model, AMAX=0.460gs

TABLE 4-1: DAMAGE DETECTION FOR UCB EXPERIMENTS					
input motion	repair action	δ_M	$\bar{I}$	P [no damage]	P [damage]
MO 5.0	none	0.043	-0.87	~ 1	~ 0
MO 9.7	none	0.12	-0.50	~ 1	~ 0
MO 14.7	none	0.37	0.70	0.17	0.83
MO 28.3	none	0.54	1.52	0.07	0.93
T 40.3	base of structure was repaired	0.61	1.87	0.03	0.97
T 46.3	none	0.73	2.41	0.02	0.98

MO Miyagi-Oki

T Taft

δ_M $1 - \frac{(T)_{initial}}{(T_0)_{max}}$

I $\frac{I_{earthquake}}{I_{damage}}$

peak acceleration in % g

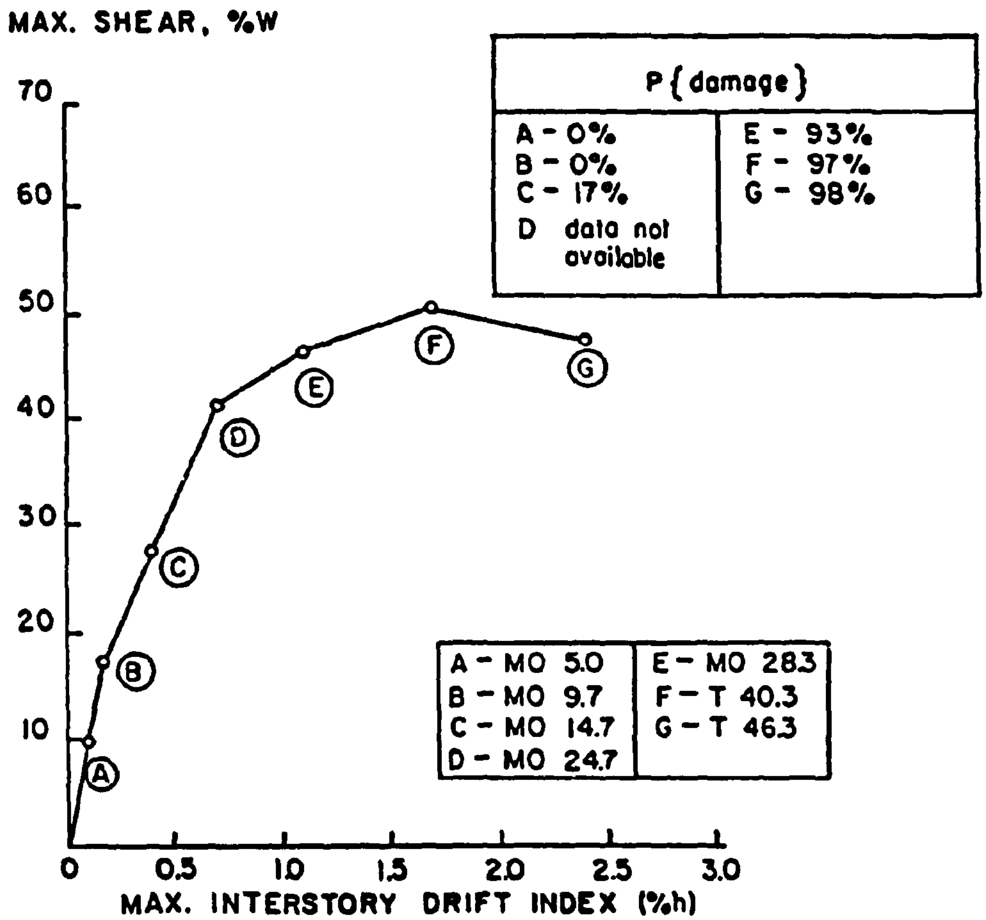

FIGURE 4-7 Prediction of Damage for the UCB Model

Finally, the probability that the the structure underwent some damage was computed. Table 4-1 summarizes the results of such analysis. The table reports the expected value of the maximum softening δ_M, of the intensity ratio $\overline{I}$, and the probability that the structure is damaged as well as its complement (probability of safety). Figure 4.7 has been adapted from the experimenters' report. The points marked identify the test runs in the maximum-story-drift vs. maximum-base-shear plane. The connecting line represents an envelope of the hysteretic cycles of the structure, and provides information on the progressive onset of nonlinearities and the eventual collapse. Correspondingly to each test run, the probability of damage computed using parameter-based indices is reported. It can be noted that the method is successful in identifying the onset of damage. The class of tests, after which the structure is damaged, are clearly identified by very large probability of damage. Very low probability of damage correspond to those tests, after which the structure is safe.

4.3. Analysis of Strong Motion Records

Figures 4.8 through 4.12 show the strong motion records from the San Fernando earthquake analyzed for this study. The structures considered sustained little or no damage (Jennings et al.,1971). They are therefore a very good test for the sensitivity of the damage detection procedure.

Figures 4.13 and 4.14 show the strong motion records from the Imperial County Service building, Imperial Valley earthquake, 1979. In the North-South direction the lateral resistance was provided by shear walls in parallel with a moment-resisting frame, while a moment-resisting frame acted along the East-West direction. In this simplified analysis, the coupling between the two translational degrees of freedom, as well as the torsional motion, is neglected. This building was severely damaged during the earthquake. Its repair was considered not convenient and it was decided to demolish it (Wosser et al., 1982). The results of the serviceability analysis are shown in Table 4.2. It can be seen that the model performs very well in detecting damage even at very low levels.

The damage detection method proposed does not take into account the interaction between soil and structure. Consequently the apparent softening of the building may be partly due to nonlinearities in the soil behavior. This potential error does not seem to pose any problem in the practical cases analyzed.

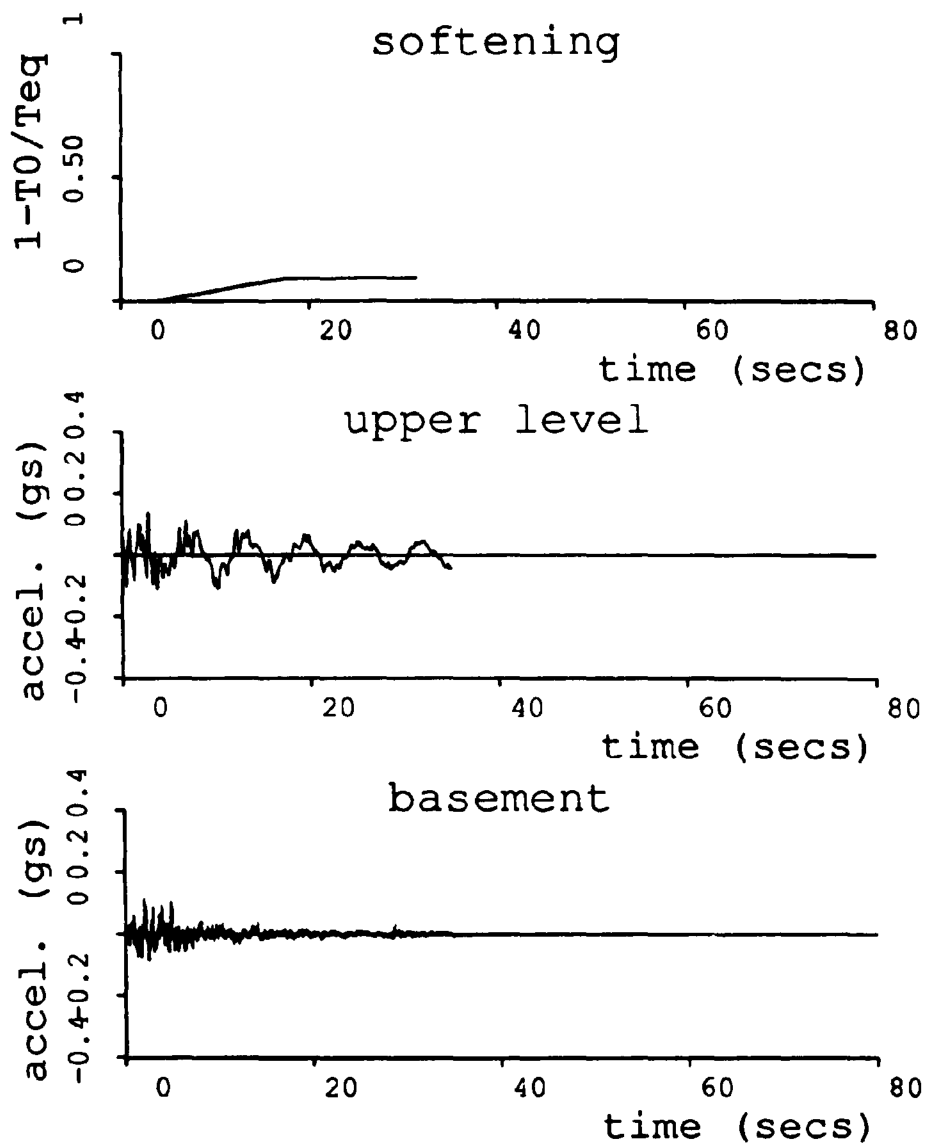

FIGURE 4-8 Recorded Acceleration and Estimated Softening for 611 West 6th St. (San Fernando, 1971)

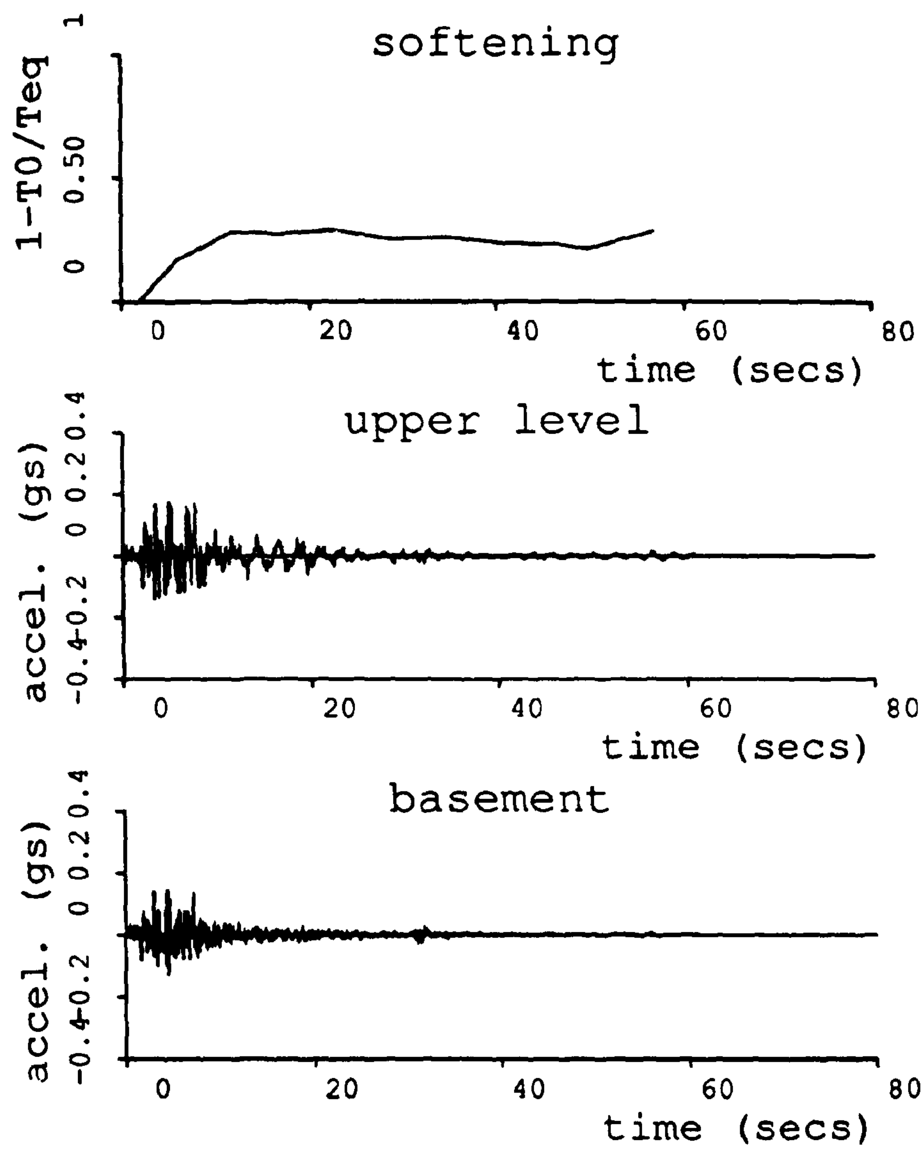

FIGURE 4-9 Recorded Acceleration and Estimated Softening for Sheraton Hotel (San Fernando, 1971)

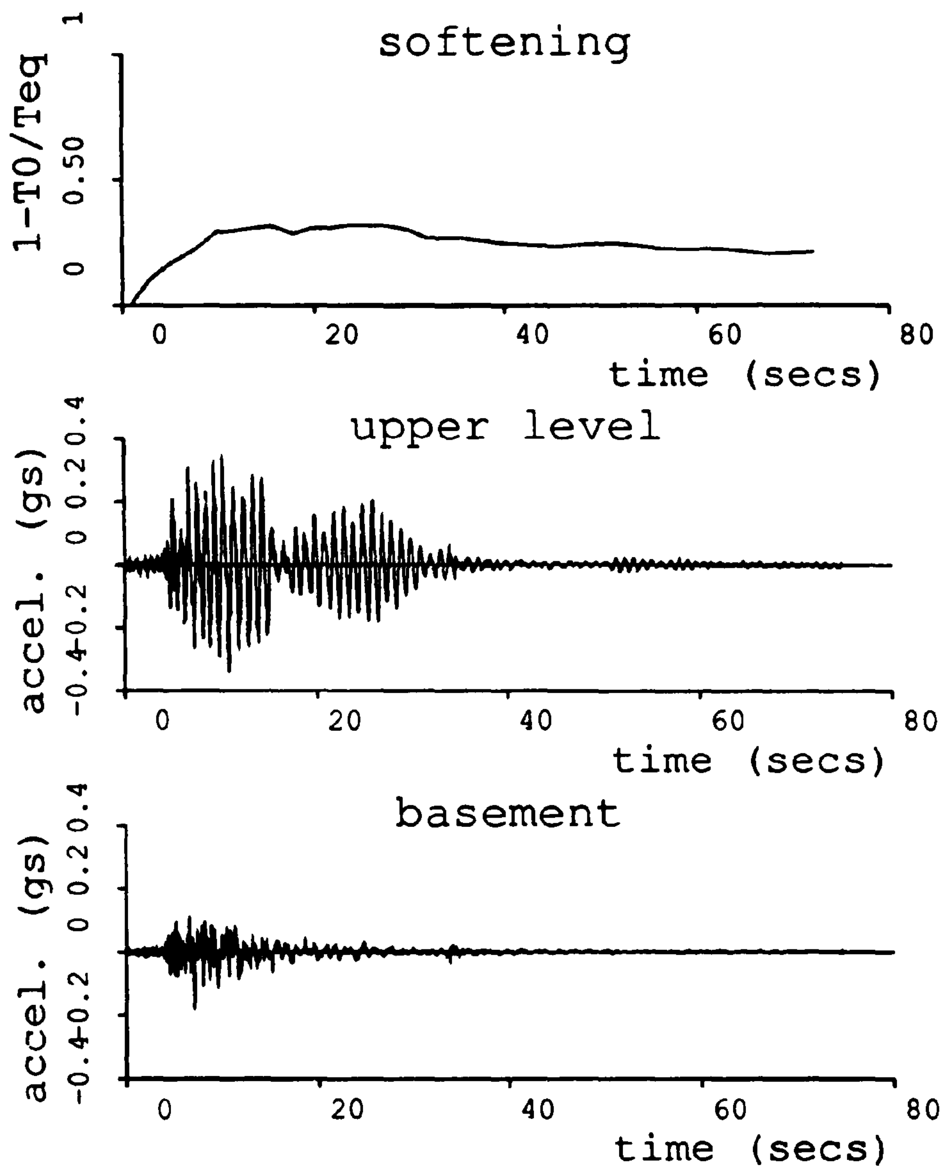

FIGURE 4-10 Recorded Acceleration and Estimated Softening for Millikan Library (San Fernando, 1971)

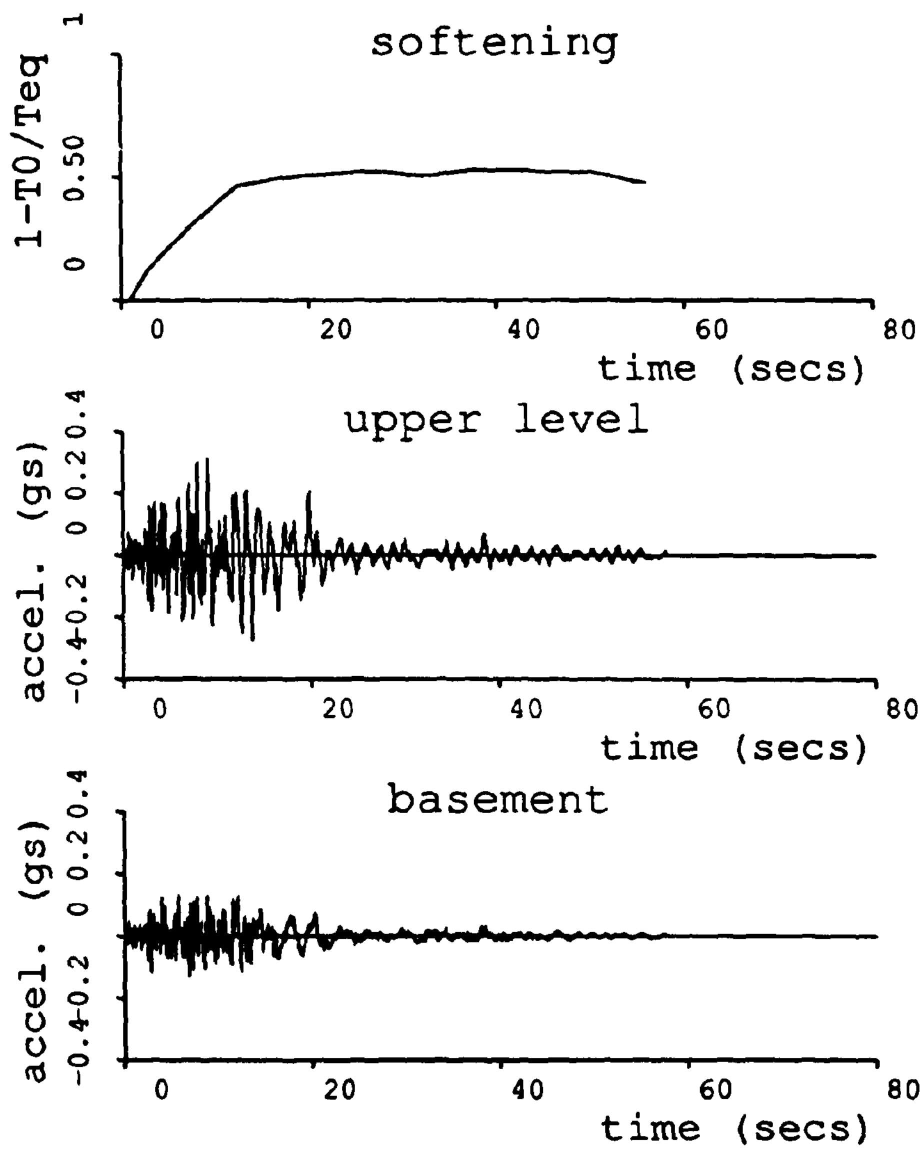

FIGURE 4-11 Recorded Acceleration and Estimated Softening for Holiday Inn Orion (San Fernando, 1971)

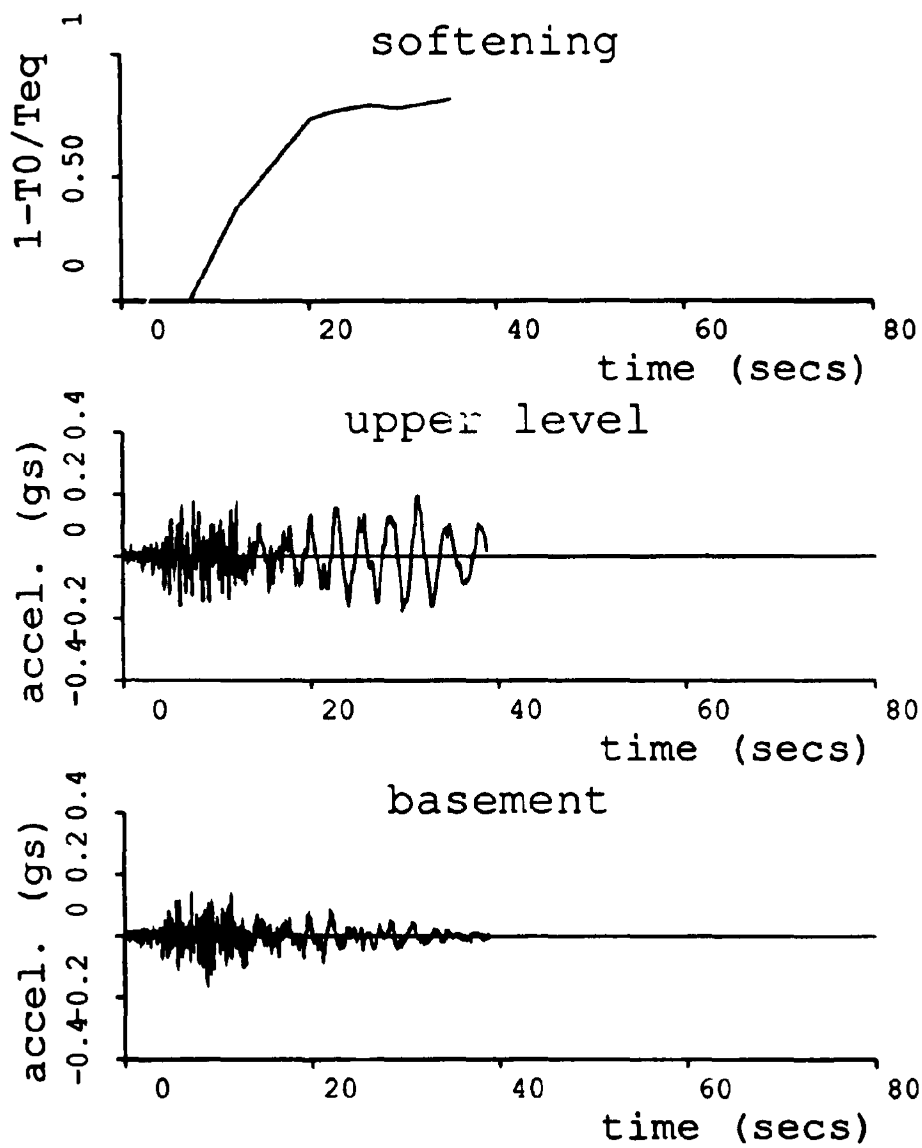

FIGURE 4-12 Recorded Acceleration and Estimated Softening for Bank of California (San Fernando, 1971)

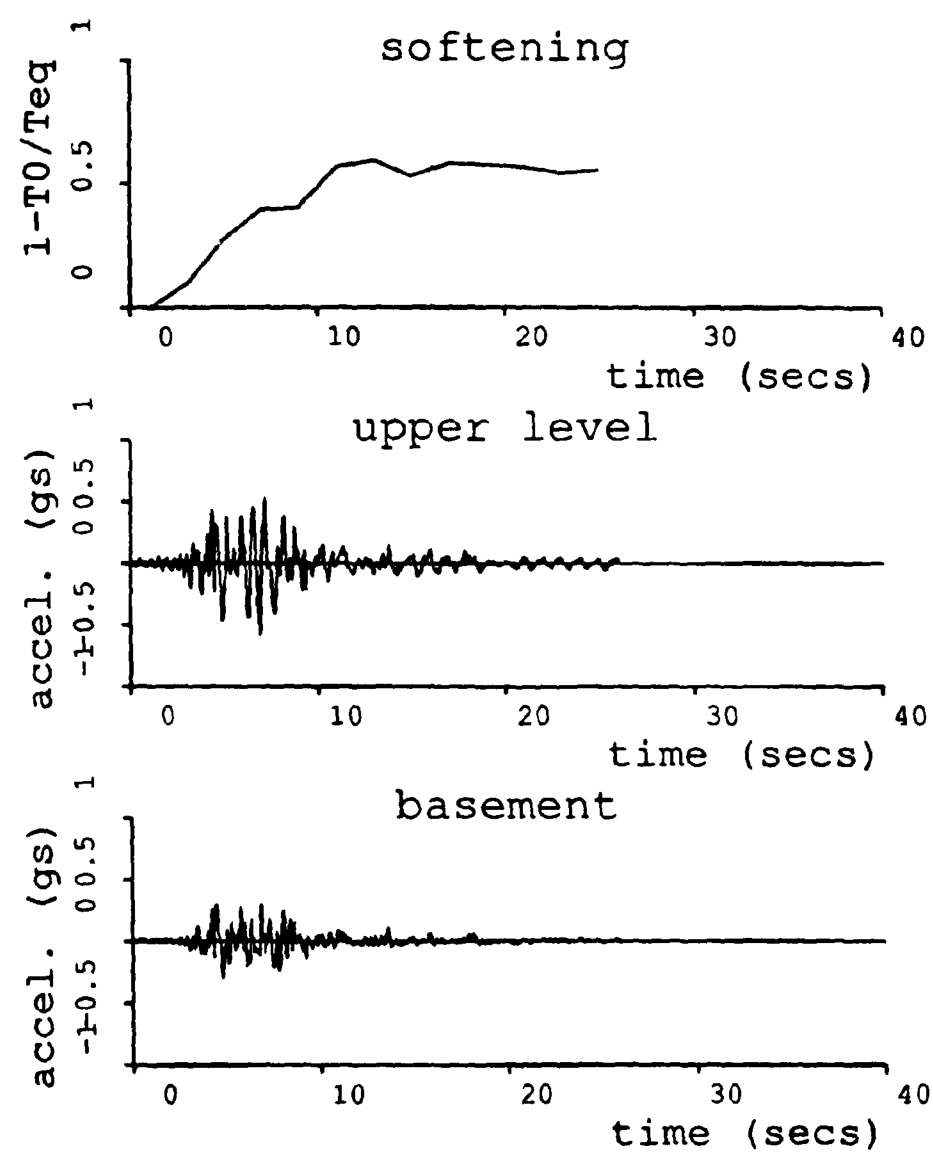

FIGURE 4-13 Recorded Acceleration and Estimated Softening for Imperial County Service Building, NS (Imperial Valley, 1979)

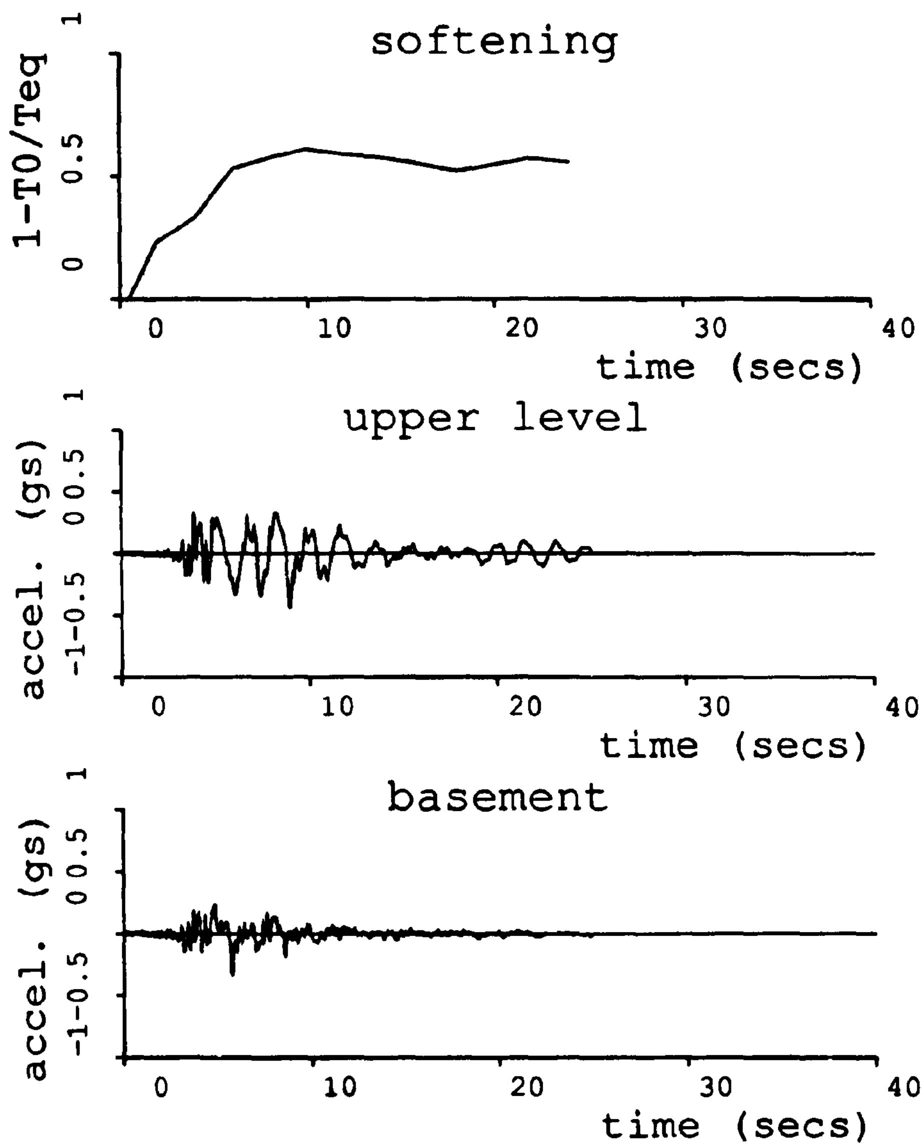

FIGURE 4-14 Recorded Acceleration and Estimated Softening for Imperial County Service Building, EW (Imperial Valley, 1979)

TABLE 4-2: DAMAGE DETECTION FOR STRONG MOTION RECORDS					
San Fernando Earthquake (1971)					
structure	**actual damage (repair action)**	δ_M	I	**P [no damage]**	**P [damage]**
611 6th St.	none	0.13	-0.45	~1	~0
Sheraton Hotel	none	0.29	0.31	~1	~0
Millikan Library	none	0.32	0.46	0.97	0.03
Holiday Inn	cracks filled with epoxy cement	0.52	1.42	0.12	0.88
Bank of California	cracks filled with epoxy cement	0.81	2.81	~0	~1
Imperial Valley Earthquake (1979)					
structur.	**actual damage (repair action)**	δ_M	I	**P [no damage]**	**P [damage]**
Imperial County (EW)	demolition	0.61	1.86	0.03	0.97
Imperial County (NS)	demolition	0.59	1.76	0.03	0.97

$$\delta_M = 1 - \frac{(T)_{initial}}{(T_0)_{max}}$$

$$I = \frac{a_{earthquake}}{a_{damage}}$$

SECTION 5: CONCLUSIONS

The serviceability limit state for reinforced concrete structures has been defined as a surface in the space of the damage indices. The model for the assessment of seismic damage proposed by DiPasquale and Cakmak (1987) has been simplified for application to the case of reinforced concrete structures. The simplified version of this model, based on equivalent linear parameters, takes into account only the maximum softening. Consequently the serviceability limit state reduces to one point, which lies in the interval (0,1). It has also been shown that the definition of a serviceability limit state is equivalent to the definition of a damage detection rule.

In order to identify the serviceability limit state for reinforced concrete structures, acceleration records from seismic simulation experiments performed at the University of Illinois at Urbana-Champaign have been considered. The seismic behavior of seven structures has been analyzed, for a total of 25 tests. The damage detection criterion proposed yields the probability that the structure has undergone damage after the seismic event and must therefore be serviced.

The criterion has been used for the damage analysis of shaking table experiments performed at the University of California at Berkeley, and of strong motion records from the San Fernando (1971) and the Imperial Valley (1979) earthquakes. In all the cases considered the damage detection criterion has given results consistent with the actual damage observed. This shows that damage analysis models based on equivalent linear parameters can identify the serviceability limit state for reinforced concrete structures.

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