NONLINEAR RESPONSE SPECTRA FOR PROBABILISTIC SEISMIC DESIGN AND DAMAGE ASSESSMENT OF REINFORCED CONCRETE STRUCTURES

by

MASAYA MURAKAMI
JOSEPH PENZIEN

Report to the National Science Foundation

COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA • Berkeley, California
In the investigation reported herein, twenty each of five different types of artificial earthquake accelerograms were generated for computing nonlinear response spectra of five structural models representing reinforced concrete buildings. To serve as a basis for probabilistic design and damage assessment, mean values and standard deviations of ductility factors were determined for each model having a range of prescribed strength values and having a range of natural periods. Adopting the standard philosophy, i.e. only minor damage is acceptable under moderate earthquake conditions and total damage or complete failure should be avoided under severe earthquake conditions, required strength levels were investigated for each model. Selected results obtained in the overall investigation are presented and interpreted in terms of prototype behavior.
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Masaya Murakami
Visiting Assistant Research Engineer, University of California, Berkeley
and Associate Professor of Architectural Engineering, Chiba University, Japan.

Joseph Penzien
Professor of Structural Engineering,
University of California, Berkeley

Prepared under the sponsorship of the
National Science Foundation
Grant NSF-GI-38463

Report No. EERC 75-38
Earthquake Engineering Research Center
College of Engineering
University of California
Berkeley, California

November 1975
ACKNOWLEDGMENT

This report covers one phase of the research program "Seismic Safety of Existing School Buildings" sponsored by the National Science Foundation under grant NSF-GI-38463 with Professor Boris Bresler serving as principal faculty investigator.

Dr. Masaya Murakami's participation was under the program "U.S.-Japan Cooperative Research in Earthquake Engineering with Emphasis on the Safety of School Building" sponsored by the U.S.-Japan Cooperative Science Program administered jointly by the National Science Foundation and the Japan Society for the Promotion of Science.

A portion of this report was presented at the review meeting of the cooperative program "U.S.-Japan Cooperative Research in Earthquake Engineering with Emphasis on the Safety of School Buildings," East-West Center, University of Hawaii, Honolulu, Hawaii, August 18-20, 1975.
ABSTRACT

In the investigation reported herein, twenty each of five different types of artificial earthquake accelerograms were generated for computing nonlinear response spectra of five structural models representing reinforced concrete buildings. To serve as a basis for probabilistic design and damage assessment, mean values and standard deviations of ductility factors were determined for each model having a range of prescribed strength values and having a range of natural periods. Adopting the standard philosophy, i.e. only minor damage is acceptable under moderate earthquake conditions and total damage or complete failure should be avoided under severe earthquake conditions, required strength levels were investigated for each model. Selected results obtained in the overall investigation are presented and interpreted in terms of prototype behavior.
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I. INTRODUCTION

The general philosophy of seismic resistant design in most countries of the world, including Japan and the United States, is that only minor damage is acceptable in buildings subjected to moderate earthquake conditions and that total damage or complete failure should be prevented under severe earthquake conditions. This philosophy serves as the basic criterion for assessing the potential seismic performance of existing buildings and for defining design criteria for new buildings.

Usually, the above philosophy is applied to performance assessments and to design in a deterministic manner. In this case, seismic response analyses are carried out for fixed mathematical models using fully prescribed ground motion excitations. It should be realized however that many uncertainties exist in this method. The highly variable characteristics of ground motions, even for a given site, is the major cause of these uncertainties. However, other causes also exist such as the variability of structural properties. For this reason, nondeterministic methods which formally recognize uncertainties and which predict response in probabilistic terms should be encouraged. Meanwhile every effort should be made to reduce the uncertainties through experimental and analytical research and through improved design and construction methods.

To carry out nondeterministic seismic analyses, an appropriate stochastic model must be established for the expected ground motions. If sufficient strong ground motion data were available this model could be obtained by direct statistical analyses. However, due to the limited data available, one is forced to hypothesize model forms and to use the existing data primarily in checking the appropriateness of these forms. The particular model used in this investigation is essentially
nonstationary filtered white noise as commonly used by many investigators [1,2]. While this model is admittedly not perfect, it does reflect the main statistical features of real ground motions; therefore, its use in seismic response analyses leads to more realistic predictions than does a single fully prescribed accelerogram.

Since it was the intent of this investigation to concentrate on low-rise reinforced concrete buildings, two basic single degree of freedom structural models were selected for dynamic analysis purposes, namely, the so called "Origin-Oriented Model" and the "Trilinear Stiffness Degrading Model" [3]. These models were selected to represent structures which fail primarily in shear and flexure, respectively. Various strength values were prescribed for these models and their initial stiffnesses were varied to produce a wide range of fundamental periods.

Mean values and standard deviations of ductility factor were generated using the five different classes of earthquake accelerograms for each structural model having a prescribed period and assigned strength values. These statistical quantities can be used as the basis for probabilistic design and damage assessment.

Accepting the basic philosophy previously mentioned, namely that only minor damage is acceptable under moderate earthquake conditions and that total damage or complete failure should be avoided under severe conditions, Umemura has proposed a basic criterion for seismic design which has been adopted herein [3]. This criterion has been used in probabilistic terms to establish appropriate strength levels for each model consistent with the basic design philosophy.

A computer program which generates artificial earthquake accelerograms and nonlinear response spectra is presented in Appendix A.
II. GENERATION OF ARTIFICIAL EARTHQUAKE ACCELEROGrams

2.1 STOCHASTIC MODELS

Two basic types of nonstationary processes are commonly used to represent earthquake ground motions, namely, nonstationary filtered white noise and filtered shot noise [1,4,5]. Shinozuka and Sato suggest that under similar conditions both types lead to essentially the same response characteristics of linear systems [6].

In the present investigation, five specific types (Types A, B, B₀₂', C, and D) of artificial accelerograms were generated using the second of the above mentioned basic types. The computer program used for this purpose was a modified version of the program (PSEQGN) developed by Ruiz. It follows a procedure consisting of five phases, (1) stationary wave forms are generated having a constant power spectral density function (white noise) of intensity \( S_o \) over a wide range of frequencies starting at zero frequency, (2) nonstationary shot noise is next obtained by multiplying each stationary wave form by a prescribed time intensity function, (3) each of the resulting wave forms of shot noise is then passed through a second-order filter which amplifies the frequency content in the neighborhood of a characteristic frequency and attenuates the higher frequencies, (4) next each of these filtered wave forms is passed through a second second-order filter which eliminates the very low frequency content, and finally (5) a baseline correction is applied to the double filtered accelerograms in accordance with the procedure of Berg and Housner [7]. Both second-order filterings are accomplished digitally by solving numerically the second-order differential equations relating filter outputs to their corresponding inputs [8]. These
solutions are obtained numerically by the standard linear acceleration method using constant integration time intervals of 0.01 seconds. By this procedure, final accelerograms are obtained in digitized form with each having similar 0.01 second time intervals.

2.2 TIME INTENSITY FUNCTIONS

Five classes of earthquake accelerograms (Types A, B, B02, C and D) were generated using four different time intensity functions as shown in Fig. 1. These intensity functions are the same as those used previously by Jennings, et al [2]. Note that accelerograms of Types B and B02 were generated using the same intensity function. All four intensity functions consist of three phases (1) a parabolic or cubic build-up phase, (2) a constant intensity phase, and (3) an exponential decay phase. The total durations of these particular functions are 120, 50, 12 and 10 seconds, respectively; however, since the ends of the decay phase do not affect maximum response of damped structural systems, they were cut off at 75, 30, 10 and 5 seconds for Types A, B, C and D, respectively.

2.3 HIGH FREQUENCY FILTER CHARACTERISTICS

As previously stated, the nonstationary shot noise wave forms were obtained by multiplying each stationary wave form having a power spectral intensity $S_o$ by a prescribed time intensity function.

The high frequency filtering procedure was then used to shape the frequency content of the shot noise wave forms using the transfer function (complex frequency response function [8])

$$H_1(i\omega) = \frac{[1 + (4 \xi^2 - 1)(\omega/\omega_o)^2] - 2i \xi\omega (\omega/\omega_o)^3}{[1 - (\omega/\omega_o)^2]^2 + 4 \xi^2(\omega/\omega_o)^2}$$  \(1\)
This transfer function, previously suggested by Kanai and Tajimi for this purpose [9,10], is usually written in the more familiar form

\[ |H_1(i\omega)|^2 = \frac{1 + 4 \xi_o^2(\omega/\omega_o)^2}{[1 - (\omega/\omega_o)^2]^2 + 4 \xi_o^2(\omega/\omega_o)^2} \]  \( (2) \)

Jennings, Ruiz, and other investigators have also used this same transfer function.

Parameters \( \omega_o \) and \( \xi_o \) appearing in the above filter function may be thought of as some characteristic ground frequency and damping ratio, respectively. Kanai has suggested 15.6 rad/sec for \( \omega_o \) and 0.6 for \( \xi_o \) as representative values for firm soil conditions. The frequency transfer function in the form of Eq. (2) is plotted in Fig. 2a for \( \xi_o = 0.6 \). These same values of \( \omega_o \) and \( \xi_o \) were used in the present investigation for four of the five classes of accelerograms, namely, Types A, B, C, and D. Accelerograms of Type B\(_{02}\) used the same value for \( \omega_o \), i.e. 15.6 rad/sec., but a different value for \( \xi_o \), namely, 0.2. This damping value was selected for Type B\(_{02}\) accelerograms to study the influence of a relatively narrow band excitation on structural response.

2.4 LOW FREQUENCY FILTER CHARACTERISTICS

The low frequency filter used in this investigation had the transfer function [2,8]

\[ H_2(i\omega) = \frac{(\omega/\omega_f)^2[1 - (\omega/\omega_f)^2] - 2i \xi_f(\omega/\omega_f)^3}{[1 - (\omega/\omega_f)^2]^2 + 4 \xi_f^2(\omega/\omega_f)^2} \]  \( (3) \)

or

\[ |H_2(i\omega)|^2 = \frac{(\omega/\omega_f)^4}{[1 - (\omega/\omega_f)^2]^2 + 4 \xi_f^2(\omega/\omega_f)^2} \]  \( (4) \)
where \( \omega_f \) and \( \xi_f \) are the characteristic frequency and characteristic damping ratio, respectively, for the filter. The damping ratio term \( \xi_f \) was assigned the numerical value \( 1/\sqrt{2} \) which reduces Eq. (4) to

\[
|H_2(i\omega)|^2 = \frac{(\omega/\omega_f)^4}{1 + (\omega/\omega_f)^4}
\]

Introducing the period ratio \( T/T_f \), where \( T = 2\pi/\omega \) and \( T_f = 2\pi/\omega_f \), Eq. (5) becomes

\[
|H_2(iT)|^2 = \frac{1}{1 + (T/T_f)^4}
\]

In this investigation, \( T_f \) equals 7 and 2 seconds for Types A, B and \( B_02 \) and for Types C and D, respectively. The square root of the function given by Eq. (6) is shown in Fig. 2b.

2.5 DISCUSSION ON ARTIFICIAL ACCELEROMETERS

The constant power spectral intensity \( S_0 \), used in generating the stationary wave forms, was assigned the value 0.08952 \( \text{ft}^2/\text{sec}^3 \). Using a family of 20 Type B accelerograms, this intensity resulted in a mean peak acceleration of 0.300g with a standard deviation of 0.032g. Increasing the number of accelerograms to 40 gave a mean peak acceleration of 0.308g and a standard deviation of 0.037g. Following the method of Gumbel [11], it is estimated that for an infinite number of similar accelerograms, the mean peak acceleration would be 0.309g and the standard deviation would be 0.041g. Therefore, in view of this mean peak acceleration and the time intensity function used, the Type B accelerograms closely represent that class of motions containing the N-S component of acceleration recorded during the 1940 El Centro, California, earthquake [2,4].
Table 1 lists the mean values and standard deviations for the peak accelerations in all 5 classes of accelerograms, i.e. for Types A, B, C, D, and B\textsubscript{02}. In obtaining these results, \( S_0 \) was assigned the same value 0.08952 \( \text{ft}^2/\text{sec}^3 \) in each case. Notice that the mean peak acceleration decreases as the duration of the constant intensity phase in the motion decreases. This observation is, of course, consistent with the theory of extreme values. Notice also that the standard deviations are relatively small in each case.

Following the suggestion of Jennings, et. al. [2], the Type A accelerograms are intended to represent the upper bound ground motions expected in the vicinity of the causative fault during an earthquake having a Richter Magnitude 8 or greater. The Type B accelerograms are intended to represent the motions close to the fault in a Magnitude 7 earthquake, such as the 1940 El Centro, California, earthquake and the 1952 Taft, California, earthquake. The Type C accelerograms are intended to represent the ground motions in the epicentral region of a Magnitude 5.5 shock, such as occurred during the 1957 San Francisco earthquake, and the Type D accelerograms are intended to represent the motions present in the immediate vicinity of the fault of a 4.5 to 5.5 Magnitude earthquake having a small focal depth, such as the 1966 Parkfield, California, earthquake. If the artificial accelerograms generated as Types A, B, C, and D are indeed to be representative of these conditions, then each class of motions should be normalized by the appropriate factors to raise the mean peak acceleration levels from 0.332g, 0.309g, 0.244g, and 0.189g, respectively, to approximately 0.45g, 0.33g, 0.10g, and 0.50g.

Since the extreme values of response for all 5 structural models used in this investigation were measured in terms of ductility factors, the above mentioned normalization of accelerograms is not required. These
ductility factors are controlled by a structural model strength to ground motion intensity ratio; i.e. $p_s/m \frac{\bar{v}}{\bar{v}_{go}}$ where $p_s$ is the significant structural strength parameter, $m$ is the mass of the single degree of freedom system, and $\bar{v}_{go}$ is the mean peak acceleration. Allowing this ratio to vary over a prescribed range of values is equivalent to allowing $p_s$ and/or $\bar{v}_{go}$ to vary independently over restricted ranges.

Further, it should be recognized that the structural response data generated for ground motions of Types A, B, C, D and B02 can be interpreted in terms of structural response to other classes of motions. For example, suppose one wished to interpret these response data for similar classes of earthquake motions but for a change in the characteristic ground frequency $\omega_0$ to reflect a change in soil conditions. This interpretation can be accomplished by considering a change in the time scale of the accelerograms; thus, forcing corresponding changes in the time intensity functions, the value of $T_f$, the value of $\omega_0$, and the mean peak acceleration. Since the value of $S_o$ representing the new classes of accelerograms is to remain unchanged, the mean peak accelerations of the new motions will be changed exactly in proportion to the square root of the ratio of the original time interval to the new time interval. Specifically, suppose the time interval is considered to be changed from 0.01 sec. to 0.005 sec. for the Type A accelerograms. In this case, the total duration (as represented by OC, Fig. 1) is reduced from 75 sec. to 37.5 sec., $\omega_0$ is increased from 15.6 rad/sec. to 31.2 rad/sec., $T_f$ is reduced from 7 sec. to 3.5 sec., and the mean peak acceleration is increased from 0.33g to 0.46g ($\sqrt{2} \cdot 0.33 = 0.46$).
III. STRUCTURAL HYSTERETIC MODELS

3.1 BASIC PARAMETERS OF MODELS

The single degree of freedom system shown in Fig. 3a was used as the basic form for all structural models investigated. This model has a linear viscous dashpot but a nonlinear hysteretic spring. The restoring spring force is therefore some prescribed nonlinear function $F(v)$ of the relative displacement $v(t)$. The principal quantities used to characterize this function are $P_c$, $p_y$, $v_c$, and $v_y$ as shown in Fig. 3b. Loads $p_c$ and $p_y$ represent the spring restoring forces corresponding to the concrete cracking strength and the ultimate strength, respectively. Displacements $v_c$ and $v_y$ are the corresponding relative displacements.

3.2 ORIGIN-ORIENTED SHEAR MODEL

One of the five structural models used in this investigation was the so-called "Origin-Oriented" hysteretic model proposed by Umemura, et. al. [3]. This model is shown in Fig. 4 where it is characterized by $P_{sc}$, $p_{sy}$, $v_{sc}$, and $v_{sy}$ which represent the concrete shear cracking strength, the ultimate shear strength, the relative displacement produced by $P_{sc}$, and the relative displacement produced by $p_{sy}$, respectively. Application of this model is restricted to those structural types where the nonlinear deformations and failure characteristics are controlled primarily by shear.

This model is defined such that the hysteretic behavior takes place with increasing relative displacements greater than $v_{sc}$ or decreasing displacements less than $-v_{sc}$. Reduction of loads from values greater than $P_{sc}$ or less than $-P_{sc}$ follow linear paths always directed through the origin, e.g. paths A'O and A"O in Fig. 4. Oscillatory motions can, of course, take place along the linear paths such as A'OA'.
and A"OA" without developing hysteretic loops provided the maximum displacements do not exceed the maximum displacement previously developed. The particular model plotted in Fig. 4 is for the case where $p_{sy} = 1.9 p_{sc}$, $v_{sy} = 10.0 v_{sc}$, $k_2 = 0.1 k_1$, and $k_3 = 0.19 k_1$.

3.3 TRILINEAR STIFFNESS DEGRADING FLEXURE MODEL

Four of the five structural models used in this investigation were the so-called "Trilinear Stiffness Degrading" hysteretic model [3]. This model is shown in Fig. 5 where it is characterized by $p_{Bc}$, $p_{By}$, $v_{Bc}$, and $v_{By}$ which represent the load at which the concrete cracks due to flexure, the load at which the main reinforcing steel starts yielding due to flexure, the relative displacement produced by $p_{Bc}$, and the relative displacement produced by $p_{By}$, respectively. Application of this model is restricted to those structural types where the nonlinear deformations and failure characteristics are primarily controlled by flexure.

The trilinear model is defined such that linear elastic behavior (without hysteretic loops) always takes place for oscillatory displacements where the corresponding oscillator loads are in the range $-p_{Bc} < p < p_{Bc}$; however, hysteretic behavior occurs with every cycle of deformation which has load levels above $p_{Bc}$ or below $-p_{Bc}$. During that period of time between the initiation of loading and that instant at which the relative displacement first increases above $v_{By}$ or decreases below $-v_{By}$, the trilinear model behaves exactly like the standard bilinear hysteretic model having stiffnesses $k_1$ and $k_2$ (QPOAB; Fig. 5a). However, as soon as the relative displacement increases above $v_{By}$ or decreases below $-v_{By}$, a new bilinear hysteretic relation controls the response. For example, suppose the relative displacement for the first time increases above $v_{By}$ to level $v_{\text{max}}$ as represented by C in Fig. 5a. Upon decreasing the displacement from this level, the corresponding load decreases along path CD which has a slope equal to $\alpha k_1$, where
As soon as the load drops by the amount $2p_{BC}$ reaching point D in Fig. 5a, any further drop in load will follow the continuing path shown having a slope $a_k_2$. It should be noted that point D is located at load level $p_{BC}$ in Fig. 5a but only because the particular trilinear model represented in that figure is for $p_{By}/p_{BC} = 3.0$. If this ratio had been assigned a different numerical value, the load level at point D would be different from $p_{BC}$.

The new bilinear hysteretic model controlling the continuing motion is shown in Fig. 5b. Note that the origin of the skelton curve is shifted from point 0, the origin of the original bilinear hysteretic model, to point $0'$. This point is the intersection point of line QC and the abscissa axis in Fig. 5a; therefore $00'$ is equal to $BC/2$. The stiffnesses of the new bilinear model are $a_k_1$ and $a_k_2$.

If during the period of response controlled by the second bilinear model (Fig. 5b) the relative displacement should increase beyond $v_{max}$ ($v_{max} = v_{By}$) as represented by point B' to a new level as represented by C', the continuing response would be controlled by a third bilinear hysteretic model whose characteristics could be obtained in exactly the same manner as the characteristics of the second model. Also, if yielding of the trilinear model had taken place at load level $-p_{By}$ rather than load level $p_{By}$, the new bilinear model controlling the continuing motion would be obtained by a similar procedure.

One characteristic feature of the trilinear stiffness degrading model worth noting is that when subjected to full-reversal cyclic displacements at a constant amplitude the bilinear hysteretic loops are
perfectly stable, i.e. each loop retraces the preceding one. The energy absorbed during each successive cycle must therefore be equal. Using a period \( T_2 = \frac{2\pi}{\sqrt{m/k_y}} \), where \( k_y \) is an average stiffness as shown in Fig. 6, one can calculate the equivalent damping ratio \( \xi \) for a linear viscously-damped single degree system which represents the same energy absorption per cycle of oscillation. This damping ratio is shown in Fig. 6 for each of four different bilinear models.
IV. SELECTION OF MODEL PARAMETERS

4.1 ORIGIN-ORIENTED SHEAR MODEL

As previously defined, the origin-oriented hysteretic model shown in Fig. 4 is completely characterized by any four of the seven parameters $k_1, k_2, k_3, p_s, p_y, v_s, v_y$. Based on experimental data [3], it has been determined that

$$p_y = 1.9 p_s \quad (8)$$

$$v_y = 10 v_s \quad (9)$$

which reduces the number of independent parameters to two. It is most meaningful to let one of these two parameters be a stiffness parameter and the other be a strength parameter. For this purpose, it is convenient to use period $T_l = 2\pi \sqrt{m/k_1}$ and the concrete cracking force $p_{sc}$. As shown later, $p_{sc}$ is normalized by the force $\bar{v}_{go}$, where $\bar{v}_{go}$ is the mean peak ground acceleration.

4.2 TRILINEAR STIFFNESS DEGRADING FLEXURE MODEL

The general trilinear stiffness degrading hysteretic model shown in Fig. 5 is completely characterized by any four of the seven parameters $k_1, k_2, k_y, p_{Bc}, p_{By}, v_{Bc}, v_{By}$. Four specific models, which were previously studied by other investigators [3], were selected for this investigation,

1. $k_1 = 2k_y, p_{By} = 3p_{Bc}$
2. $k_1 = 2k_y, p_{By} = 2p_{Bc}$
3. $k_1 = 4k_y, p_{By} = 3p_{Bc}$
4. $k_1 = 4k_y, p_{By} = 2p_{Bc}$
These four models were chosen because $k_y$ and $p_{By}$ are often found in the ranges $2k_y < k_1 < 4k_y$ and $2p_{Bc} < p_{By} < 3p_{Bc}$, respectively, for reinforced concrete members. For frame structures, these ranges are not so well defined so that engineering judgment must be relied upon in assigning values consistent with their overall nonlinear behaviors.

Having assigned numerical values to the ratios $k_y/k_1$ and $p_{By}/p_{Bc}$, only two independent model parameters remain. In this case it is most convenient to select a stiffness parameter measured in terms of $T_1 = 2\pi \sqrt{m/k_1}$ and a strength parameter measured in terms of $p_{By}$. Again, the strength parameter selected ($p_{By}$) is normalized by the force $m v_{go}$.

4.3 VISCOS DAMPING MODEL

As shown in Fig. 3a, the single degree of freedom model used in this investigation included a linear viscous dashpot having a variable coefficient $c$. The coefficient used with the origin-oriented shear model is defined by the relation $c(t) = 2 \xi_1 k(t)/\omega(t)$ where $\xi_1$ is a constant damping ratio, and $k(t)$ and $\omega(t)$ are variable stiffness and natural circular frequency in accordance with the stiffness at time $t$, respectively. The coefficient used with the trilinear stiffness degrading flexure model is defined by the relation $c(t) = 2 \xi_1 k(t)/\omega_1$ where $\omega_1$ is a initial natural circular frequency. This coefficient becomes smaller with a reduction or degradation of stiffness.
V. DYNAMIC RESPONSE ANALYSIS

The complete time history of dynamic response was generated for the single degree of freedom system using the five different structural models subjected separately to the twenty artificially generated earthquake ground motions. The equation of motion governing this response is the well known relation

\[ m \ddot{v}(t) + c(t) \dot{v}(t) + F(v) = - m \ddot{g}(t) \]  

(11)

where \( F(v) \) is the nonlinear spring force defined by the hysteretic model being considered; i.e. the spring force defined by either Fig. 4 or Fig. 5. Dividing through by \( \frac{\pi}{v_{go}} \) (a constant) gives

\[
\left[ \frac{1}{\frac{\pi}{v_{go}}} \right] \ddot{v}(t) + \left[ \frac{c(t)}{m \frac{\pi}{v_{go}}} \right] \dot{v}(t) + \frac{F(v)}{m \frac{\pi}{v_{go}}} = - \frac{\ddot{g}(t)}{\frac{\pi}{v_{go}}} 
\]

(12)

Note that the third term on the left hand side of this equation is the same force-displacement relation defined by the hysteretic model but with the force normalized (as previously mentioned) by the constant \( m \frac{\pi}{v_{go}} \). Knowing the numerical values assigned to constants \( \frac{\pi}{v_{go}} \), \( \xi_1 \), and \( T_1 \), as well as the prescribed value of \( p_{sc}/m \frac{\pi}{v_{go}} \) (or \( p_{By}/m \frac{\pi}{v_{go}} \)), one can solve Eq. (12) for the complete time history of response \( v(t) \). This solution is obtained numerically using the standard "linear acceleration" method. The time interval \( \Delta t \) generally used in the integration was shortened to a subdivided value \( \Delta t' \) during short periods of time in which the model stiffness changed value. The numerical values of \( \Delta t \) and \( \Delta t' \) used for four different ranges of period \( T_1 \), are shown in Table 2.
The response quantity of primary interest is the ductility factor $\mu$ which is defined as $v(t)_{\text{max}}/v_{sc}$ for the origin-oriented model and as $v(t)_{\text{max}}/v_{By}$ for the trilinear stiffness degrading model. This factor was obtained for each of the five structural models when subjected separately to each of the 20 ground motions generated for Types A, B, C, D, and B02. The damping ratio $\xi_1$ was assigned the value 0.05 for the origin-oriented model and 0.02 for the trilinear stiffness degrading model. Since the ductility factor was desired for a range of stiffnesses, period $T_1$ was assigned 10 different numerical values as given by

$$T_1 = 0.1(2)^{n/2} \ (n = 0,1,2,...,9) \ (13)$$

Using the origin-oriented model, ductility factors were obtained for a range of values of $p_{sc}/m \bar{v}_{go}$, namely 0.50, 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, 2.50, and 3.00. Using the trilinear stiffness degrading model, these values were obtained for $p_{By}/m \bar{v}_{go}$ equal to 0.50, 0.75, 1.00, 1.125, 1.25, 1.50, and 1.75.
VI. DUCTILITY RESPONSE SPECTRA

6.1 LINEAR ELASTIC MODEL

To characterize the five classes of earthquake motions (Types A, B, C, D, and \( B_{02} \)) in most familiar terms, all 20 accelerograms of each type were separately used as the excitation applied to a linear, viscously damped (\( \xi = 0.05 \)) single degree of freedom system. Mean absolute acceleration response ratios \( \alpha \) as defined by

\[
\alpha = \frac{\ddot{v}(t)_{\text{max}}}{\ddot{v}_{go}}
\]

where \( \ddot{v}(t)_{\text{max}} \) is the mean value of 20 maximum absolute accelerations \( [\ddot{v}(t)_{\text{max}}] \) and where \( \ddot{v}_{go} \) is the peak mean value of ground accelerations, were determined for each excitation over a range of periods T. The coefficients of variation (ratio of standard deviation to mean value) of \( \ddot{v}(t)_{\text{max}} \) were also determined for the 20 accelerograms in each type of excitation.

The results of the analyses for all five classes of earthquake are shown in Fig. 7 where the mean absolute acceleration response ratios \( \alpha \) and the coefficients of variation of \( \ddot{v}(t)_{\text{max}} \) are plotted as functions of period T. As would be expected, the values of \( \alpha \) for the five classes of earthquakes are widely separated at the long period end of the abscissa scale but converge together towards the low period end of the scale. As the period goes to zero, \( \alpha \) must of course, approach unity. It is seen in Fig. 7 (excluding Type \( B_{02} \)) that \( \alpha \) increases with duration of the earthquake excitation. The very high peak shown in the function of \( \alpha \) for Type \( B_{02} \) is caused by the narrow band excitation in the ground motion in the neighborhood of \( T = 0.4 \) sec.
The coefficients of variation of $\ddot{v}_t(t)$ decrease with duration of excitation and increase generally with period $T$. It should be recognized that as $T$ approaches zero the coefficients of variation of $\ddot{v}_t(t)$ approach the corresponding coefficients of variation of $\ddot{v}_{go}(t)$, as given in Table 1.

6.2 ORIGIN-ORIENTED SHEAR MODEL

Mean ductility factors $\bar{\mu}$ and their corresponding coefficients of variation were generated for the origin-oriented shear model using the 20 response time histories for each class of earthquake ground motions. Values, as obtained over the period range $0.1 < T_1 < 1.6 \sqrt{2}$ and over the normalized load range $0.50 < \beta_s < 3.00$ (where $\beta_s$ is defined as the ratio $P_c/m \bar{\ddot{v}}_{go}$), are shown in Figs. 8a-8e. For each type of earthquake, these ductility factors generally increase with decreasing period and the spread of ductility factors over the full strength range increases with decreasing period. Also the ductility factors for a fixed period increases with decreasing structural strength.

The trends of the coefficients of variation with period are similar to those previously described for mean ductility factor, particularly regarding strength level and strength variation. It is most significant to note that the coefficients of variation are low when the response is essentially elastic ($\mu < 1$) but they can become very large with increasing inelastic deformations.

When interpreting the results in Figs. 8a-8e, it should be noted that the strength ratio $\beta_s = P_c/m \bar{\ddot{v}}_{go}$ can be expressed in the form

$$\beta_s = (P_c/W)/(\bar{\ddot{v}}_{go}/g)$$

(15)
where \( W \) is the weight of the single degree of freedom mass and \( g \) is the acceleration of gravity. Therefore, this parameter can be considered as the ratio of base shear to coefficient of mean peak ground acceleration.

If for any particular case one wishes to determine the mean maximum relative displacement \( \bar{v}(t)_{\text{max}} \), this can be accomplished by using the appropriate mean ductility factor \( \bar{\mu} \) taken from Figs. 8a-8e. By definition of ductility factor, one can state

\[
\bar{v}(t)_{\text{max}} = v_c \bar{\mu} = (p_c/k_1) \bar{\mu}
\]  

Making use of the definition of \( \beta_s \) given above, this equation can be written in the form

\[
\bar{v}(t)_{\text{max}} = \frac{m}{k_1} \beta_s \bar{\mu} \frac{\pi}{\nu \, g_0}
\]  

or

\[
\bar{v}(t)_{\text{max}} = T_1^2 \beta_s \bar{\mu} \left[ \frac{g}{4\pi^2} \right] \left[ \frac{\nu \, g_0}{g} \right]
\]  

Equation (18) is the most convenient form for calculating \( \bar{v}(t)_{\text{max}} \).

6.3 TRILINEAR STIFFNESS DEGRADING FLEXURE MODEL

Mean ductility factors and their corresponding coefficients of variation were generated for the four trilinear stiffness degrading flexure models using the 20 response time-histories for each class of earthquake ground motions. Values, as obtained over the period range \( 0.1 < T_1 < 1.6 \sqrt{2} \) and over the normalized load range \( 0.50 < \beta_f < 1.75 \) (where \( \beta_f \) is defined as the ratio \( p_y/m \nu \, g_0 \)), are shown in Figs. 9a-9d, 10a-10d, 11a-11d, 12a-12d, and 13a-13d for earthquake Types A, B, C, D, and B'02', respectively.
The general trends of these results are very similar to those previously described for the origin-oriented shear model. It is worth pointing out again that the coefficients of variation of maximum response are relatively low for cases of essentially elastic behavior but can become very large for cases involving inelastic deformations.

As in the case of the origin-oriented model, mean maximum response can be calculated using the relation

\[ \bar{v}(t)_{\text{max}} = T^2 \cdot \mathbf{g} \cdot \bar{\mu} \left[ \frac{\bar{g}}{4\pi} \right] \left[ \frac{\bar{g}_0}{g} \right] \] (19)
VII. USE OF DUCTILITY RESPONSE SPECTRA FOR PROBABILISTIC SEISMIC DESIGN

7.1 SELECTION OF REQUIRED DUCTILITY LEVELS

It is implied in the basic philosophy of design previously stated that economical considerations do not permit the design of structures for zero risk of damage in high seismic regions. To minimize total costs (initial costs, repair costs after earthquakes, etc.), damage is often permitted to limited degrees under moderate to severe earthquake conditions. It should be understood that permitting some damage to occur in a well designed structure has the beneficial effect of limiting damage to that same structure. This is due to the fact that the energy absorption associated with damage is effective in limiting the maximum levels of oscillatory motion in the structure. Therefore, a good seismic resistant structure should be designed for high energy absorption capacity assuming it will experience controlled damage under severe to moderate earthquake conditions. In terms of the hysteretic structural models presented herein, this concept means that the ductility factor should be limited to certain values consistent with the basic design philosophy.

Assume for the moment that one prescribes two numerical values of ductility factor for a given structural model. The smaller value was chosen to be consistent with light damage under moderate earthquake conditions and the large value was chosen to be consistent with heavy damage (but not complete failure) under severe conditions. Two questions come to mind (1) "What is the probability of these ductility factors being exceeded during a single earthquake of Types A, B, C, D, or B_02?" and (2) "What ductility factors are required, consistent with the design
philosophy?". To answer these questions, one must establish the appropriate probability density or distribution functions.

Previous investigations have shown that the probability distribution function for extreme value of structural response for a single earthquake follows closely the Gumbel Type I distribution [1,4]

\[
P(\mu) = \exp \left\{ - \exp \left[ - \alpha (\mu - u) \right] \right\}
\]

(20)

where \( \mu \) is the maximum response measured in terms of ductility factor, and \( \alpha \) and \( u \) are parameters which depend on the average and standard deviation of \( \mu \). If only 20 sample values of \( \mu \) are available as in this investigation, \( \alpha \) and \( u \) can be obtained using the relations [11]

\[
\alpha = 1.063/\sigma_{\mu}
\]

(21)

and

\[
u = \bar{\mu} - 0.493 \sigma_{\mu}
\]

(22)

where \( \bar{\mu} \) and \( \sigma_{\mu} \) are the mean and standard deviation of the 20 sample values of \( \mu \). Using these equations and expressing the standard deviation of \( \mu \) in terms of its coefficient of variation \( (\sigma_{\mu} = c \bar{\mu}) \), Eq. (20) can be written in the nondimensional form

\[
P(q) = \exp \left\{ - \exp \left[ - \frac{1.063}{c} (q - 1 + 0.493c) \right] \right\}
\]

(23)

where

\[
q \equiv (\mu/\bar{\mu})
\]

(24)

This probability distribution function is plotted in Fig. 14 over a range of values of \( c \), i.e. over the range \( 0 < c < 1.5 \). Since the probability distribution function is defined such that
The first question previously raised, namely, "What is the probability of these ductility factors being exceeded during a single earthquake of Types A, B, C, D, or B\textsubscript{02}?", can be easily answered using Eq. (26), Fig. 14 and the data provided in Figs. 8a-13d. The second question raised, i.e. "What ductility factors are required consistent with the design philosophy?", is more difficult to answer. Before attempting to answer this question, one must realize that the basic design criteria cannot be met in absolute terms, i.e. with 100% confidence. This complication is due to the scatter of coefficient of variation of ductility factor present for each family of earthquake excitations. The best one can do is reduce the probability of exceedance associated with each of the two ductility factors to an acceptable level. Deciding on an acceptable level is complex as it involves economic, social, and political considerations.

Suppose for example, it was decided that a 15 percent probability of exceedance was acceptable, i.e. $Q(\mu) = 0.15$ and $P(\mu) = 0.85$. Using Fig. 14 and the data provided in Figs. 8a-13d, one can easily establish that ductility factor $\mu_{85}$ associated with $P(\mu) = 0.85$. This has been done for all four trilinear stiffness degrading models subjected to Type A ground motions giving the results shown in Fig. 15.

7.2 SELECTION OF REQUIRED STRENGTH LEVELS

To establish the required strength levels of the various structural models for each class of earthquake motions, one must first prescribe
basic criteria consistent with the basic design philosophy. In the following discussion, 20, 15, 10 and 5 percent probabilities of exceedance were selected as examples of acceptable risk and it was assumed that moderate and severe earthquake conditions are represented by 0.30g and 0.45g, respectively, for the mean peak acceleration of ground motions. Finally, the two ductility factors, consistent with light and heavy (but controlled) damage, are chosen as 2 and 10 for the origin-oriented shear model and 2 and 4 for the trilinear stiffness degrading model. The values of peak accelerations and ductility factors selected above follow the suggestions of Umemura, et al. [3].

Using data such as shown in Fig. 15 for each structural model and for each type of earthquake motions, i.e. using curves of $\mu_{85}$ vs. $T_1$, one can easily obtain the required strength ratios ($\beta = p/m \sqrt{g_0}$) for discrete values of $T_1$. Linear interpolation between the curves ($\mu_{85}$ vs. $T_1$) for a fixed value of $T_1$ can be used for this evaluation. The resulting required strength ratios for each prescribed risk level can then be plotted as functions of period $T_1$ as shown in Figs. 16-19 for the trilinear stiffness degrading flexure model subjected to earthquake Types A, B, C and D. Figures 20 and 21 show the required strength ratios corresponding to ductility ratios $\mu_{80}$, $\mu_{85}$, $\mu_{90}$ and $\mu_{95}$ equal to 6. Figs. 22 and 23 show similar results but with the ductility ratios equal to 8. These results have no direct relation to the basic design criteria but are of interest in showing the influence of high ductility on the required strength level. One characteristic feature of all sets of curves in these figures is that the four curves representing earthquake Types A, B, C and D are quite close to each other, except for the case of Type D earthquakes when 10 and 5 percent probability of exceedance is prescribed. One finds the required strength ratios are significantly
influenced by the area of the hysteretic loop; see Fig. 6. The required strength ratios for the trilinear model subjected to earthquake Type B_{02} are shown in Figs. 26 and 27.

One very significant feature to notice in these figures for all five types of earthquakes is that generally, the required strength ratios for the trilinear model in the range \( T_1 > 0.2 \) sec. vary in a linear manner with negative slopes along the log scale for \( T_1 \). Converting to a linear scale, the required strength ratios would vary in inverse proportion to the square root of \( T_1 \), i.e. \( \beta \sim (T_1)^{-1/2} \) for \( T_1 > 0.2 \) sec. This implies that buildings represented by the trilinear model which have a shorter natural period than the predominant period in the ground motions are likely to suffer excessive deformation, especially in a case of earthquake Type B_{02}, because a lengthening of the period caused by reduction and degradation of the stiffness brings the characteristic period more in line with the predominant period in the ground motions.

The required strength ratios (\( \beta_s = \frac{p_c}{m \bar{v}_g} \)) are shown in Figs. 24 and 25 for the origin-oriented shear model subjected to earthquake Types A, B, C and D. The results in Fig. 24 are obtained for the basic criteria previously established; however, the results in Fig. 25 are for ductility factors \( \mu \) set equal to 1.5 and 5 which represent brittle structures. These latter results show the influence of brittleness on the required strength level. The four curves representing earthquake Type A, B, C and D are quite close to each other similar to the corresponding curves for the trilinear model. The required strength ratios for the shear model subjected to earthquake Type B_{02} are shown in Fig. 28. The results in Figs. 24, 25 and especially, in Fig. 28, show a high tendency to peak at \( T_1 = 0.4 \) sec. which corresponds with the predominant period of the input ground motions. This tendency is most
significant in the case of lower ductility factors which correspond to small degradations in stiffness. The required strength ratios vary in inverse proportion to the square root of $T_l$ for $T_l > 0.4$. As for the shear model, the period at the peak of these curves becomes smaller with the higher ductility factors which accompany the larger degradations of the stiffness.

When judging which of the two prescribed ductility factors control a particular design or damage assessment, one should be careful not to base the decision on a direct comparison of the required strength ratios as shown in Figs. 16-19, 24, 26 and 28 since these ratios have different normalization factors. For example, consider a shear model with $T_l = 0.4$ sec. as shown in Figs. 24c and 24d. Using the light damage criteria, i.e., $\bar{\gamma}_{go} = 0.30$ g and $\mu_{85} = 2$, gives $\beta = 2.2$ and $p_c = 0.66$ mg. Using the heavy damage criteria, i.e. $\bar{\gamma}_{go} = 0.45$ g and $\mu_{85} = 10$, gives $\beta = 0.8$ and $p_c = 0.36$ mg. Note that for these two different levels of damage, the resulting values for $\beta$ have a different ratio to each other than do the two values for $p_c$. Obviously in this case, the light damage criteria requiring $p_c = 0.66$ mg control the design or damage assessment. Let us consider a second example of the origin-oriented model with $T_l = 0.15$ sec. In this case the light damage criteria give $\beta = 1.7$ and $p_c = 0.51$ mg and the heavy damage criteria give $\beta = 1.3$ and $p_c = 0.58$ mg. For this particular structural model, the heavy damage criteria requiring $p_c = 0.58$ mg control the design or damage assessment. Making similar comparisons for the various trilinear stiffness degrading models represented in Figs. 16-19 and 26, one finds that the heavy damage criteria ($\bar{\gamma}_{go} = 0.45$ g and $\mu_{85} = 4$) always control the design or damage assessment.
When using the results in Fig. 16-28 in accordance with the above example calculations, one should remember that they are based on the ground motion parameters $\omega_0 = 15.6 \text{ rad/sec} \ (T_0 = 0.4 \text{ sec})$ and $\xi_0 = 0.6$ which represent firm ground conditions. If one should have quite different ground conditions, these parameters should be adjusted appropriately. These adjustments shift the level of the predominant frequencies in the ground motions and also change the mean intensity level $\bar{v}_{go}$. With considerable experience and using engineering judgment, certain modifications to the data in Figs. 16-28 can be made to reflect these new conditions.

One should also keep in mind that these results do not include the influence of soil-structure interaction which lengthens the natural period and often increases damping in the overall system.
The response ductility factors and coefficients of variation presented herein provide the necessary data for carrying out probabilistic seismic resistant designs and for conducting damage assessments consistent with basic design criteria and the statistical nature of earthquake ground motions.
IX. BIBLIOGRAPHY


### TABLE 1
MEAN VALUES AND STANDARD DEVIATIONS OF PEAK GROUND ACCELERATIONS

<table>
<thead>
<tr>
<th>Type of Earthquake</th>
<th>Statistical Quantity</th>
<th>Number of Earthquakes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>A</td>
<td>Mean</td>
<td>0.327</td>
</tr>
<tr>
<td></td>
<td>Std. Deviation</td>
<td>0.023</td>
</tr>
<tr>
<td>B</td>
<td>Mean</td>
<td>0.300</td>
</tr>
<tr>
<td></td>
<td>Std. Deviation</td>
<td>0.032</td>
</tr>
<tr>
<td>C</td>
<td>Mean</td>
<td>0.240</td>
</tr>
<tr>
<td></td>
<td>Std. Deviation</td>
<td>0.022</td>
</tr>
<tr>
<td>D</td>
<td>Mean</td>
<td>0.191</td>
</tr>
<tr>
<td></td>
<td>Std. Deviation</td>
<td>0.041</td>
</tr>
<tr>
<td>B02</td>
<td>Mean</td>
<td>0.346</td>
</tr>
<tr>
<td></td>
<td>Std. Deviation</td>
<td>0.048</td>
</tr>
</tbody>
</table>

### TABLE 2
STANDARD TIME INTERVAL AND SUBDIVIDED TIME INTERVAL

<table>
<thead>
<tr>
<th>Interval Type</th>
<th>Natural Period $T_1$, sec.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1 and 0.14</td>
</tr>
<tr>
<td>Standard</td>
<td>0.005</td>
</tr>
<tr>
<td>Subdivided</td>
<td>0.000625</td>
</tr>
</tbody>
</table>
FIG. 1 TIME INTENSITY FUNCTIONS
\[
|H(i\omega)|^2 = \frac{1 + 4\xi_0 \left( \frac{\omega}{\omega_0} \right)^2}{\left[ 1 + \left( \frac{\omega}{\omega_0} \right)^2 \right]^2 + 4\xi_0 \left( \frac{\omega}{\omega_0} \right)^2} \\
\xi_0 = 0.6
\]

FIG. 2 FILTER TRANSFER FUNCTIONS

\[
T_r = \frac{1}{\sqrt{1 + \left( \frac{T}{T_f} \right)^4}}^{1/2} \\
T_f \equiv \sqrt{|H_2(i\omega)|^2}
\]

FIG. 3 SINGLE DEGREE OF FREEDOM MODEL WITH FORCE-DISPLACEMENT RELATIONSHIP
FIG. 4 ORIGIN-ORIENTED HYSYTERETIC MODEL

$P_{sy} = 1.9 \ P_{sc}$
$v_{sy} = 10.0 \ v_{sc}$
$k_2 = 0.1 \ k_1$
$k_3 = 0.19 \ k_1$

FIG. 5 TRI-LINEAR HYSYTERETIC MODEL
FIG. 6 STABLE BILINEAR HYSTERETIC LOOP FOR TRILINEAR STIFFNESS DEGRADING MODEL
FIG. 7 RESPONSE ACCELERATION RATIOS FOR LINEAR MODEL

FIG. 8a MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR ORIGIN-ORIENTED MODEL HAVING DIFFERENT STRENGTH LEVELS
FIG. 8b MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR ORIGIN-ORIENTED MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 8c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR ORIGIN-ORIENTED MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE D EARTHQUAKE

\[ \xi = 0.05 \quad \rho = 1.9 \rho_c \quad v_y = 10.0 \nu_c \]

\[ \mu = \frac{\nu(t)_{\text{max}}}{\nu_c} \]

\[ T_1 = 2 \pi \sqrt{\frac{m}{k_1}} \]

\[ k_2 = 0.1 k_1 \]

FIG. 8d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR ORIGIN-ORIENTED MODEL HAVING DIFFERENT STRENGTH LEVELS

TYPE B02 EARTHQUAKE

\[ \xi = 0.05 \quad \rho = 1.9 \rho_c \quad v_y = 10.0 \nu_c \]

\[ \mu = \frac{\nu(t)_{\text{max}}}{\nu_c} \]

\[ T_1 = 2 \pi \sqrt{\frac{m}{k_1}} \]

\[ k_2 = 0.1 k_1 \]

FIG. 8e MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR ORIGIN-ORIENTED MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE A EARTHQUAKE

\[ \xi_1 = 0.02 \quad T_2 = \sqrt{2} T_1 \quad \beta_y = 2 \beta_c \]

\[ T_2 = 2 \pi \sqrt{m/k_2} \quad T_1 = 2 \pi \sqrt{m/k_1} \quad \xi_1 = c/2 \sqrt{m/k_1} \]

\[ \gamma \equiv \frac{\nu(t)}{\nu_y} \]

\[ p \equiv 0.50 m \nu_{q_0} \]

FIG. 9a MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

\[ T_2 = 2 \pi \sqrt{m/k_2} \quad T_1 = 2 \pi \sqrt{m/k_1} \quad \xi_1 = c/2 \sqrt{m/k_1} \]

\[ q = 0.50 m \nu_{q_0} \]

\[ \gamma \equiv \frac{\nu(t)}{\nu_y} \]

FIG. 9b MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
FIG. 9c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TROTLINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS.

FIG. 9d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TROTLINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS.
FIG. 10a MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 10b MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE B EARTHQUAKE
\[ \xi_1 = 0.02 \quad T_2 = 2T_1 \quad P_y = 2P_c \]

FIG. 10c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 10d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE C EARTHQUAKE

\( \xi_1 = 0.02 \quad T_2 = \sqrt{2} T_1 \quad p_y = 2 p_c \)

\[ T = 2 \pi \sqrt{m/k_1} \]

\[ T = 2 \pi \sqrt{m/k_1} \]

\[ T = c/2 \sqrt{m/k_1} \]

\[ \xi = c/2 \sqrt{m/k_1} \]

\[ \mu = \xi \left( \frac{T_{\text{max}}}{y} \right) \]

FIG. 11a MEAN DUCTILITY FACTORS AND CORRESPONDING
COEFFICIENTS OF VARIATION FOR TRILINEAR
STIFFNESS DEGRADING MODEL HAVING DIFFERENT
STRENGTH LEVELS

FIG. 11b MEAN DUCTILITY FACTORS AND CORRESPONDING
COEFFICIENTS OF VARIATION FOR TRILINEAR
STIFFNESS DEGRADING MODEL HAVING DIFFERENT
STRENGTH LEVELS
FIG. 11c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 11d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE D EARTHQUAKE

\[ \xi_1 = 0.02 \quad T_2 = \sqrt{2} T_1 \quad \rho_y = 2 \rho_c \]

FIG. 12a MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 12b MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
FIG. 12c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

FIG. 12d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
TYPE B02 EARTHQUAKE

\[ \xi_1 = 0.02 \quad T_2 = \sqrt{2} T_1 \quad p_y = 2 p_c \]

\[ T_2 = 2 \pi \sqrt{m/k_2} \]
\[ T_1 = 2 \pi \sqrt{m/k_1} \]
\[ \xi_1 = \xi_2 = \sqrt{2} \sqrt{m/k_1} \]
\[ p_y = \frac{w_{max}}{v_y} \]
\[ y_c \]

FIG. 13a MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

TYPE B02 EARTHQUAKE

\[ \xi_1 = 0.02 \quad T_2 = \sqrt{2} T_1 \quad p_y = 3 p_c \]

\[ T_2 = 2 \pi \sqrt{m/k_2} \]
\[ T_1 = 2 \pi \sqrt{m/k_1} \]
\[ \xi_1 = \xi_2 = \sqrt{2} \sqrt{m/k_1} \]
\[ p_y = \frac{w_{max}}{v_y} \]
\[ y_c \]

FIG. 13b MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRILINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS
FIG. 13c MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRI_LINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

TYPE B02 EARTHQUAKE
\[ \varepsilon = 0.02 \]
\[ T_2 = 2T_1 \]
\[ p = 3p_c \]

FIG. 13d MEAN DUCTILITY FACTORS AND CORRESPONDING COEFFICIENTS OF VARIATION FOR TRI_LINEAR STIFFNESS DEGRADING MODEL HAVING DIFFERENT STRENGTH LEVELS

TYPE B02 EARTHQUAKE
\[ \varepsilon = 0.02 \]
\[ T_2 = 2T_1 \]
\[ p = 3p_c \]
FIG. 14 PROBABILITY DISTRIBUTION FUNCTIONS FOR DUCTILITY FACTOR RATIOS ON GUMBEL PLOTS
FIG. 15a RESPONSE DUCTILITY FACTORS FOR 85% LEVEL ON PROBABILITY DISTRIBUTION FUNCTIONS

FIG. 15b RESPONSE DUCTILITY FACTORS FOR 85% LEVEL ON PROBABILITY DISTRIBUTION FUNCTIONS
FIG. 16 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 17 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 18 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 19 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 20  REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED.
FIG. 21 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 22 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 23 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 24 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 25 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 26 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 27 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
FIG. 28 REQUIRED STRENGTH RATIOS FOR STRUCTURAL MODELS, DUCTILITY FACTORS, PROBABILITY DISTRIBUTION LEVELS AND EARTHQUAKE TYPES INDICATED
APPENDIX A

COMPUTER PROGRAM "SHOCU"
PROGRAM SHOCU (INPUT, OUTPUT, PUNCH)

INPUT DATA

MCNTRL INDEX FOR KINDS OF JOB

MCNTRL = 0 STOP
= 1 PSEUDO EARTHQUAKE GENERATION
= 2 EARTHQUAKE GENERATION AND RESPONSE ANALYSIS OF ORIGIN-ORIENTED SHEAR MODEL
= 3 EARTHQUAKE GENERATION AND RESPONSE ANALYSIS OF TRI-LINEAR STIFFNESS DEGRADING FLEXURE MODEL
= 4 EARTHQUAKE GENERATION AND RESPONSE ANALYSIS OF BOTH MODELS

DATA ARE READ IN THE CORRESPONDING SUBROUTINES
(INSEQN, ORIGIN, OTRIL1)

NSTEP = 2 NUMBER OF INTERPOLATION OF ACCELERATION

COMMON AA(32000)
COMMON /CNTL/, NEQREC, DT, EAMXX, DAMP, FO, TI, TO, EL2, NUUT(0), HED(0)
1, NEQ, ISHAPE, ACCMAX(401), VELMAX(401), DISMAX(401)
COMMON /GENII/, P12, NU, NDEC, NTOT, AMPL, DI, D2, D3, WJ, WJG, J, C
1, A1, A2, A3, A4, A5, A6, A7, CL, OL, NACC, NVEL

READ 100, MCNTRL

IF ( MCNTRL.EQ.0 ) GO TO 2

PSEUDO EARTHQUAKE GENERATION

CALL PSEQN

IF ( MCNTRL.EQ.1 ) GO TO 3

LINEAR INTERPOLATION OF EARTHQUAKE ACCELERATION

NSTEP=2

NTOT2=NSTEP*NTOT
TH=DT/FLUAT(NSTEP)

CALL ERIKO (AA(NACC), AA(NVEL), NTOT2, NSTEP)

IF ( MCNTRL.EQ.3 ) GO TO 4

RESPONSE ANALYSIS OF ORIGIN ORIENTED SHEAR MODEL

CALL ORIGIN (AA(NVEL), NTOT2, TH, ACCMAX(1))

IF ( MCNTRL.EQ.2 ) GO TO 3

RESPONSE ANALYSIS OF TRI-LINEAR STIFFNESS DEGRADING FLEXURE MODEL

CALL OTRIL1 (AA(NVEL), NTOT2, TH, ACCMAX(1))
SUBROUTINE PSEQGN

INPUT DATA

VEQREL = NUMBER OF EARTHQUAKE RECORDS
T = DURATION OF RECORDS
TH = DT = TIME INCREMENT
DENST = POWER SPECTRAL DENSITY (CM**2/SEC**3)
NOUT=OUTPUT CONTROL ARRAY

HIGH FREQUENCY FILTER PARAMETERS

DAMP = DAMPING RATIO
F0 = UNDAMPED NATURAL FREQUENCY (CPS)
DENST = POWER SPECTRAL INTENSITY

LOW FREQUENCY FILTER PARAMETERS

DAMPL = DAMPING RATIO
FOL = UNDAMPED NATURAL FREQUENCY (CPS)

SHAPING FUNCTION PARAMETERS

TI = DURATION OF INITIAL PARABOLIC BUILDUP
TO = TIME AT END OF STATIONARY PORTION
CE2 = EXPONENTIAL DECAY CONSTANT
ISHAPE = 2 (PARABOLIC BUILD-UP PHASE)
= 3 (CUBIC BUILD-UP)

BLANK COMMON STORAGE ALLOCATION
SET COMM AAINNN AND NAA=NNN
WHERE NNN EXCEEDS 3*NTOT+NI+NDEC

WHERE NTOT=T/DT+1
NI=T/DT+1
NDEC=NTOT-TO/DT

**NEQREC** = 1 IS REQUIRED FOR RESPONSE ANALYSIS.
IF LAST TWO CARDS ARE "BLANK", SUBROUTINE RETURNS TO MAIN.

COMMON AA(32000)
NAA=32000

READ SPECIFICATIONS
1 READ 6, HED,NEQREC,T,TH,DENST
IF (NEQREC.EQ.0) GO TO 2
DT=1.0
READ 8, DAMP,FO
READ 9, ISHAPE
READ 7, TI,TO,CE2
READ 8, DAMPL,FUL
READ 9, NOUT
PRINT 10, HED,NEQREC,DENST,T,DT,NOUT,FO,DAMP,FUL,DAMPL,II,ISHAPE,
ITO,CE2

NTUT=T/DT+1.0001
N0=T0/DT+1.0001
NI=T/DT+1.0001
NDEC=NTOT-NO+1
MAA=3*NTOT+NDEC+NI+1
IF (NAA-MAA) 2,3,3
2 PRINT 7, MAA
STOP
3 CONTINUE

STORAGE ALLOCATION
NP=1
NP1=NP+NDEC
NACC=NPI+NI
NVEL=NACC+NTUT
NDISP=NVEL+NTUT

GENERATE INTENSITY-TIME FUNCTION
CALL SHAPE (AA(NP),AA(NP1),NI,NDEC)

GENERAL CONSTANTS
PI2=6.2831853
W0=P12*FO
D=2.*DAMP*W0
A2+6./DT
A1+1/D
A3=3./DT
A4=6./DT/2.
A7=A4
A5=A7*DT/3.0
A6=2.0*A5
C1=A1*D+A3*W0
W0=PI2*F0L
D1=2.0*AMPL*W0
CL=A1*DL*A3+W0
AMPL=PI2*DENST/Ur
AMPL=SQRT(AMPL)
D1=DT/T
D2=DT*DI/T
D3=DT*DI
D4=DT*DI
C
C GENERATE, PRINT AND PUNCH RECORDS
C
DO 4 NEQ=1,NEQMAX
CALL GEN (AA(NP),AA(NPL),AA(NACC),AA(NVEL),AA(NDSP))
CALL OUT (AA(NACC),AA(NVEL),AA(NDSP),NDT)
4 CONTINUE
GO TO 1
5 CONTINUE
RETURN
C
C 6 FORMAT (8A4/15,F10.0)
7 FORMAT (1//21H EXECUTION TERMINATED/)
1 27H ......SEE USER GUIDE TO SET /
2 35H NAA AND DIMENSION OF ARRAY AA )
8 FORMAT (8F10.4)
9 FORMAT (611)
10 FORMAT (8A4//)
1 32H NUMBER OF EARTHQUAKE RECORDS = ,I5/
2 32H INTENSITY (ICM**2/SEC**3) = ,F10.5/
3 32H DURATION OF RECORDS (SECS) = ,F10.5/
4 32H TIME INCREMENT (SECS) = ,F10.5/
5 32H OUTPUT CONTROL ARRAY = ,A11/!!!
6 33H HIGH FREQUENCY FILTER PROPERTIES //
7 20H NATURAL FREQUENCY = ,F10.5//
8 20H DAMPING RATIO = ,F10.5//
9 33H LOW FREQUENCY FILTER PROPERTIES //
$ 20H NATURAL FREQUENCY = ,F10.5/
$ 20H DAMPING RATIO = ,F10.5////
$ 28H SHAPING FUNCTION PARAMETERS //
$ 32H DURATION OF BUILD-UP = ,F10.5/
$ 32H BUILD-UP CURVE = ,A12/
$ 32H TIME AT BEGINNING OF DECAY = ,F10.5/
$ 32H EXPONENTIAL DECAY CONSTANT = ,F10.5////
END

SUBROUTINE SHAPE (P,PL,NI,NDEC)
C
C SHAPING FUNCTION FOR INITIAL PARABOLIC OR CUBIC BUILD-UP
C
C
COMMON /CTRL/, NEQREC, DT, EAMAX, DAMP, FU, TI(10), LE2, NGUT(3), MED(8)
1, NEL, ISHAPE, ACCMAX(40), VELMAX(40), DISMAX(40)
DIMENSION P(11), PL(1)
IF (N1) 3, 3, 1
1 CEFF = (DT/TI)**ISHAPE
DO 2 N = 1, N1
F = (**ISHAPE
2 PL(N) = F*CEFF
3 CONTINUE
C SHAPING FUNCTION FOR EXPONENTIAL DECAY
C
IF (NOEC) 0, 0, 4
4 CE2 = CE2*DT
DO 5 N = 1, NOEC
F = E
5 P(N) = EXP(CE2*F)
6 RETURN
END

SUBROUTINE GEN (P, PL, ACC, VEL, DISP)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C C PSEUOD EARTHQUAKE GENERATION
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
DIMENSION P(1), PL(1), ACC(1), VEL(1), DISP(1)
COMMON /GENG/, PL, N0, V, NREC, NUVT, AMPL, DI, D2, D3, D4, D5, D6, D7
COMMON /CTRL/, LE2, NGUT(3), MED(8)
1 NEL, ISHAPE, ACCMAX(40), VELMAX(40), DISMAX(40)
C INITIAL CONDITIONS
C
J = 1
Z = 0.0
ZD = 0.0
ZO = 0.0
ZL = 0.0
ZDL = 0.0
ZODL = 0.0
F = 0.0
AG = 0.0
V = 0.0
S1 = 0.0
S2 = 0.0
S3 = 0.0
DISP(1) = 0.0
VEL(1) = 0.0
ACC(1) = 0.0
C DO 8 N = 2, NTO
C C WHITE NOISE

SCH 233
SCH 234
SCH 235
SCH 236
SCH 237
SCH 238
SCH 239
SCH 240
SCH 241
SCH 242
SCH 243
SCH 244
SCH 245
SCH 246
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SCH 272
SCH 273
SCH 274
SCH 275
SCH 276
SCH 277
SCH 278
SCH 279
SCH 280
SCH 281
SCH 282
SCH 283
SCH 284
SCH 285
SCH 286
SCH 287
SCH 288
GO TO 1, 2

1) RANF IS THE CDC RANDOM NUMBER GENERATION FUNCTION

AT EACH CALL IT RETURNS A RANDOM NUMBER BETWEEN 0.0 AND 1.0

1) X1 = RANF(0,0)

X2 = PI2 + RANF(0,0)

X1 = SQRT(X1 * X1)

W = X1 * COS(X2)

J = 2

GO TO 3

2) W = X1 * SIN(X2)

J = 1

3 CONTINUE

SHAPE THE WHITE NOISE

I = N - 1

IF (I) 4, 5, 5

4) W = W * Pi(I)

5 CONTINUE

I = N - NO

IF (I) 7, 7, 6

6) W = W * Pi(I)

FILTER THE SHOT NOISE

7) A = A1 * Z + A2 * ZD + ZDO + ZD

B = A3 * Z + ZD + A4 * ZDO

Z = (A - W) / C

ZD = A3 * Z - B

ZDD = A1 * Z - A

W = W * ZDD

LO- FREQUENCY FILTER

A = A1 * ZL + A2 * ZDL + ZDOL + ZDOLL

B = A3 * ZL + ZDL + A4 * ZDOL

ZL = (A - W) / CL

ZDL = A3 * ZL - B

ZDO = A1 * ZL - A

AG = ZDOL

BASELINE CORRECTION FACTORS

C1 = F + 5

C2 = F / 0.125

C3 = F / 3 + 5 / 24

S1 = S1 + C1 * V + (C2 * AG + C3 * AG1) * DT

F2 = F + F

C1 = F2 + F / 1.3

C2 = F2 / 6 + F / 4 + 0.1

C3 = F2 / 3 + F / 2 + 0.15

S2 = S2 + C1 * V + (C2 * AG + C3 * AG1) * DT

F3 = F + F

C1 = F3 + 1.5 * F2 + F / 25

C2 = F3 / 6 + F / 32 + 0.1 + 1.2

C3 = F3 / 3 + 0.25 + F / 45 + F / 7 + 0.4
S3=S3*Cl*V+(C2*AG+G3*AG1)*DT  
F=1.  
V=V+(AG*AG1)*A+  
AG1=AG  
ACC(N+1)=AG  
8 CONTINUE

C BASELINE CORRECTION COEFFICIENTS
B1=02*51  
B2=03*52  
B3=04*53  
Cl=-300.*B1+900.*B2-630.*B3  
C2=1800.*B1-5760.*B2+4200.*B3  
C3=-1890.*B1+6300.*B2-4725.*B3

C BASELINE CORRECTIONS AND MAXIMUM VALUES
ACC(MAX(N))=0.0  
VEL(MAX(N))=0.0  
DIS(MAX(N))=0.0  
X1=0.0  
DJ=9.0  
ACC(N)=(ACC(N)+C1*X1+C2*X1+C3*X1*X1)*AMPL  
CALL AVOMAX(ACC(N),ACC(MAX(N)))  
VEL(N)=VEL(N-1)+(ACC(N)+ACC(N-1))*AT  
CALL AVOMAX(VEL(N),VEL(MAX(N)))  
DISP(N)=DISP(N-1)+VEL(N-1)*DT+ACC(N-1)*Ab+ACC(N)*A5  
CALL AVOMAX(DISP(N),DIS(MAX(N)))

9 CONTINUE

PRINT 10, NEQ
10 FORMAT(1H,///)  
   1  ZIN accelerate RECORD NUMBER, z12)
   PRINT 11,ACC(MAX(N)),VEL(MAX(N)),DIS(MAX(N))
11 FORMAT(4H,/)  
   1  36H MAXIMUM ACCELERATION(CM/SEC**2) = ,F7.2/  
   1  36H MAXIMUM VELOCITY(CM/SEC) = ,F7.2/  
   1  36H MAXIMUM DISPLACEMENT(CM) = ,F7.2/  
RETURN
END

SUBROUTINE OUT (ACC,VEL,DISP,NTOT)

PRINT AND PUNCH RECORDS

COMMON /CTRL/ NEQREC,T,T,DT,EMAX,DAMP,F0,TI,T0,CE2,NGUT(6),HED(10)
   1,NEQ,ISHAPE,ACCMAX(40),VELMAX(40),DISMAX(40)
DIMENSION ACC(TOT),VEL(TOT),DISP(TOT)
   DT=5.*DT  
IF (NOUT(1)).NE.0) GO TO 1
   PRINT 7, HED,NEQ  
   CALL PRIN (ACC,NTOT,DT)
   RETURN

END
1 IF (OUT(2).NE.0) GO TO 2
PRINT 8, HED,NE
CALL PRIN (VEL,NTOT,DT5)
2 IF (OUT(3).NE.0) GO TO 3
PRINT 9, HED,NE
CALL PRIN (DISP,NTOT,DT5)
3 IF (OUT(4).NE.0) GO TO 4
PUNCH 11, HED,NE,NTOT,DT
PUNCH 10, ACC(1),=1,NTOT)
4 IF (OUT(5).NE.0) GO TO 5
PUNCH 12, HED,NE,NTOT,DT
PUNCH 10, (VEL(1),=1,NTOT)
5 IF (OUT(6).NE.0) GO TO 6
PUNCH 13, HED,NE,NTOT,DT
PUNCH 10, (DISP(1),=1,NTOT)
6 CONTINUE
RETURN
C
7 FORMAT (1,8A4,5X,26HACCELERATION RECORD NUMBER,13//
1 6X,4HTIME,5F4,10ACCN (CM/SEC**2))
8 FORMAT (1,8A4,5X,22HVELLOCITY RECORD NUMBER,13//
1 6X,4HTIME,5F8,12VEL (CM/SEC))
9 FORMAT (1,8A4,5X,24HDISPLACEMENT RECORD NUMBER,13//
1 6X,4HTIME,5F11X,9HDISP (CM))
10 FORMAT (8F10.4)
11 FORMAT (8A4,12HACCN RECORD,13,7H NPTS,=15,5H DT,=F5.3)
12 FORMAT (8A4,12HVEL RECORD,13,7H NPTS,=15,5H DT,=F5.3)
13 FORMAT (8A4,12HDISP RECORD,13,7H NPTS,=15,5H DT,=F5.3)
END

SUBROUTINE PRIN (A,NTOT,DT5)
C
** End of Records **
C
C
DIMENSION A(NTOT)
N1=1
N2=5
TT=0.
1 PRINT 2, TT,(A(I),=1,N1,N2)
 IF (N2.EQ.NTOT) RETURN
 N1=N1+5
 N2=N2+5
 TT=TT+DT5
 IF (N2.GT.NTOT) N2=NTOT
 GO TO 1
C
2 FORMAT (F10.3,1P5E20.3)
END

SUBROUTINE AVDMAX(A,B)
DETERMINATION OF MAXIMUM VALUE

X = ABS(A)
IF(X.GT.B) B = X
RETURN
END

SUBROUTINE ORIGINIZ,E,MJSKE,TH,ZMAX)

INPUT DATA

NAME = NAME OF MODEL
T1 = INITIAL NATURAL PERIOD (SEC)
X1STR = CRACKING STRENGTH IN TERMS OF BASE SHEAR COEFFICIENT
X2STR = ULTIMATE STRENGTH IN TERMS OF BASE SHEAR COEFFICIENT
DAMP = DAMPING RATIO
AMAX = AVERAGE OF MAXIMUM ACCELERATION IN TERMS OF GRAVITY
ID = NUMBER OF T1 INCREASED IN GEOMETRICAL RATIO (SQRT(2.0))

Z = EARTHQUAKE ACCELERATION
MJSKE = NUMBER OF DATA OF EARTHQUAKE ACCELERATION
TH = TIME INCREMENT
ZMAX = MAXIMUM ACCELERATION
B1 = STIFFNESS OF EACH REGION
X1(1) = CRACKING DISPLACEMENT
X1(2) = YIELDING DISPLACEMENT

IF LAST CARD IS "STOP":COLUMNS 1-41, SUBROUTINE RETURNS TO MAIN.

DIMENSION NAME(81,B1(3),X1(2),Z,MJSKE),DAMX(10)
DATA KAERE/=1,HSTOP/
1000 READ 100, NAME
IF (NAME.EQ.KAERE) GO TO 9999
READ 102, T1,X1STR,DAMP,AMAX,ID
X2STR=1.9*X1STR
RESU = 0.19
DO 1 11 = 1, 10
II = II - 1
T1 = T1 * SQRT(2.0)**II
PRINT 104, NAME, AMAX, XLSTR, ZZSTR, REDUCT, DAMP
B1(1) = (6.283185/T1)**2
B1(2) = REDUCT*B1(1)
B1(3) = 0.001*B1(2)
NSTEP = 1
IF(III.GT.2) NSTEP = 2
DELTAT = TH*FLOAT(NSTEP)
PRINT 106, T1, DELTAT, A1
A2 = B1(1)
A2 = 2.*A2*SQR(A1)
LLL = 1
IOA = 0
IF(III.GT.5) IOA = 4
IF(III.GT.7) IOA = 2
A2 = 0.0
V2 = 0.0
D2 = 0.0
V2 = 0.0
DOMAX = 0.0
DXE(1) = XL(1)
DXE(2) = XL(2)
K2 = 1
GG = IOA
DT = DELTAT/CG
C RESPONSE ANALYSIS
DO 2 I = 1, JSKE, NSTEP
A1 = A2
V1 = V2
D1 = D2
V1 = V2
L1 = K2
IF(I.EQ.1) GO TO 10
Z11111 = ZZ111
GO TO 15
10 III = I - NSTEP
Z11111 = ZZ11111
GO TO 15
15 CONTINUE
ZZ11111 = 980.*ZZ11111*AMAX/IMAX
CALL RESP1 (ZZ11111, A2, A1, V1, DXE, K1, L1, DELTAT, V2, D2, VV2, LLL)
LLL = 0
IF(I.EQ.1) 20, 25, 2
20 CONTINUE
C JUDGE FOR CHANGE OF POSITION
CALL MASA(VV1, VV2, D2, DXE, K1, LLL)
IF(LLLL) 30, 25, 30
25 CONTINUE
CALL AVOMAX(D2, DOMAX)
GO TO 2
30 ZZZZZZ=ZZZZZZ/GO
C RESPONSE ANALYSIS FOR SUBDIVIDED INTERVAL
C DO 3 IA=1,10A
I (FIA-1) 40,43,40
40 CONTINUE
A1=A2
V1=V2
D1=D2
V1=V2
K1=K2
45 CONTINUE
CALL RESP1 (ZZZZZ,A2,AR,A1,V1,D1,DT,R2,D2,VV2,LLL)
LLL=0
C JUDGE FOR CHANGE OF POSITION AND DETERMINATION OF STIFFNESS
C CALL JUNK1(01,AB,A2STR,Vv1,Vv2,D2,DXE,K1,K2,LLL)
IF (LLL) 50,55,50
50 CONTINUE
AR=2.*AMP*SQRT(Adl)
CALL AVDMAX(ID2,DOMAX)
GO TO 3
55 CONTINUE
CALL AVDMAX(D2,DOMAX)
3 CONTINUE
LLL=1
2 CONTINUE
DUCT=DOMAX/Xlll
PRINT 18, OQMAX, UQCT
1 CONTINUE
GO TO 1000
100 FORMAT(8A4)
102 FORMAT(8A4)
104 FORMAT(4F8.0)
9999 RETURN
END

SUBROUTINE JUNK1 (BL,AB,A2STR,V1,V2,D2,DXE,K1,K2,K)
C
C SELECTION OF STIFFNESS AT NEXT STEP
SUBROUTINE MASA (V1,V2,D2,DXE,K1,K)

C ** ** ** ** ** ** ** ** ** ** ** ** ** ** ** ** ** ** 

C JUDGE FOR CHANGE OF POSITION
C
C INPUT DATA
C V1  INCREMENTAL DISPLACEMENT AT LAST STEP
C V2  INCREMENTAL DISPLACEMENT
C D2  RELATIVE DISPLACEMENT
C DXE ORIGINAL OR MODIFIED DISPLACEMENT CORRESPONDING TO
C BREAK-POINT
C
C OUTPUT DATA
C K1  INDEX FOR POSITION
C K  INDEX FOR CHANGE OF POSITION  K=0 NON
C
C DIMENSION DXE(2)
C IF(V1=V2) 10,10.20
C 10 [FI(K1.EQ.1)] GO TO 30
C K=1
C GO TO 30
C 20 [FI(K1.EQ.3)] GO TO 30
C IF(Z.LT.DXE(K1)]) GO TO 30
C K=2
C 30 RETURN
C END

SUBROUTINE DTKII(ZZ,MJSKE,TH,UZMA)
C
C NONLINEAR RESPONSE ANALYSIS FOR DEGRADING TRI-LINEAR MODEL
C ORIGINALLY PROGRAMMED BY UMEMURA LABORATORY (UNIVERSITY OF TOKYO)
C MODIFICATIONS : M. MURAKAMI 1975

C INPUT DATA
C NAME1 NAME OF MODEL
C T1 INITIAL NATURAL PERIOD (SEC)
C REDUCT RATIO OF T1 TO NATURAL PERIOD CORRESPONDING TO
C YIELDING STIFFNESS
C XISTR CRACKING STRENGTH IN TERMS OF BASE SHEAR COEFFICIENT
C ZISTR YIELDING STRENGTH IN TERMS OF BASE SHEAR COEFFICIENT
C DAMP DAMPING RATIO
C AMAX AVERAGE OF MAXIMUM ACCELERATION IN TERMS OF GRAVITY
C ID NUMBER OF T1 INCREASED IN GEOMETRICAL RATIO
C (SQRT(2.0))
C ZZ EARTHQUAKE ACCELEROGRAM
C MJSKE NUMBER OF DATA OF EARTHQUAKE ACCELEROGRAM
DIMENSION NAME(10), BL(3), XI(2), ZZ(MJSKE)
DATA KAERE/A STOP/

READ 100, NAME
IF (NAME(1).EQ.KAERE) GO TO 9999
READ 102, TI, XI1ST, X2ST, REDUCT, DAMP, AMAX, IU
DO 1 11 = 1, IU
[Details of the code are not fully readable in this image]

IF (11.LT.1) RETURN

PRINT 105, NAME1, AMAX, XI1ST, X2ST, REDUCT, DAMP

IF (11.GT.2) NSTEP=2

DO Z I = 1, MJSKE, NSTEP
[Details of the code are not fully readable in this image]
\begin{verbatim}
A1=A2
V1=V2
D1=D2
VV1=VV2
K1=K2
IF(I1*NE.1) GO TO 10
ZZZZZ=ZZZ(1)
GO TO 15
10 II=I-NSTEP
ZZZZZ=ZZZ11-ZZZ111
15 CONTINUE
ZZZZZ=990.*ZZZZZ*AMAX/ZMAX
CALL RESPI(ZZZZZZ,A2,AR,AB,A1,V1,D1,DELTA,T,2,2,2,2,2,LLL)
IF(I1=1) 20,25,20
20 CONTINUE
DE=D2-001-002
CONTINUE
IF(I1=1) 1-NSTEP
25 CONTINUE
CALL AVOMAX(D2,DMAX)
GO TO 2
30 ZZZZZ=ZZZZZ/ZZ
3 CONTINUE
DE=02-001-002
CONTINUE
CALL RESPI(ZZZZZZ,A2,AR,AB,A1,V1,D1,DT VV2,D2,2,2,2,2,LL)
LL=0
DE=02-001-002
CONTINUE
CALL AVOMAX(D2,DMAX)
GO TO 3
50 CALL SUBMODD(81,AB,LLL,JJJ,DMAX,DEMN,001,002,DGE,ALPH,01,02,2)
CONTINUE
CALL AVOMAX(D2,DMAX)
30 CONTINUE
CONTINUE
PRINT 108, DMAX,DUCT
1 CONTINUE
\end{verbatim}
GO TO 1000
100 FORMAT(3A4)
102 FORMAT(6F8.0,1X)
104 FORMAT(1H1,3A4/)
   1 35H AVERAGE OF MAXIMUM ACCELERATION  =  ,F0.3/  
   2 35H CRACKING STRENGTH               =  ,F0.3/  
   3 35H YIELDING STRENGTH               =  ,F0.3/  
   4 35H RATIO OF K2 TO K1 (REDUCTION)   =  ,F0.3/1  
   5 35H DAMPING RATIO                   =  ,F0.3///  
106 FORMAT(1H1,/
   1 25H NATURAL PERIOD                =  ,F0.3/  
   2 25H TIME INCREMENT                =  ,F0.3/  
   3 25H CRACKING DISPLACEMENT        =  ,F0.3,2X,3HCM /  
   4 25H YIELDING DISPLACEMENT        =  ,F0.3,2X,3HCM ///  
108 FORMAT(1H1,/
   1 24H MAXIMUM DISPLACEMENT         =  ,F10.3,2X,3HCM /  
   2 24H DUCTILITY FACTOR NT          =  ,F10.3///  
9999 RETURN
END

SUBROUTINE ERIKOIZ,ZZ,N,NSTEP)
  *********************  
LINEAR INTERPOLATION FOR EARTHQUAKE MOTION  
  *********************  
DIMENSION Z(II),ZZ(II)
  *********************  
FN=NSTEP
DO 1 I=1,NSTEP
   ZZ(IJ)=Z(IJ)*FLOAT(IJ)/FN
1 CONTINUE
DO 2 I=2,N
   II=NSTEP*(I-1)
   DO 3 J=1,NSTEP
      IIJ=II+J
      ZZ[IIJJ]=(Z(II)-Z(II))*FLOAT(JJ)/FN+Z(II)
3 CONTINUE
2 CONTINUE
RETURN
END

SUBROUTINE RESPIIZZ,A2,A3,A4,A5,A1,V1,O1,O2,V2,V2,VVZ,KKK)
  *********************  
LINEAR ACCELERATION METHOD  
  *********************  
IF(KKK.EQ.0) GO TO 100
DT2=DT/2.0
**SUBROUTINE SHUSO0(S,B,~,~,J,DMAX,OMIN,001,DOZ,OCE,ALFA,DY,OC,O2)**

* * * * • * * * * * * • * * * • • * * * * * * • • • • • • • • •

**SELECTION OF STIFFNESS AT NEXT STEP**

**INPUT DATA**

B    ORIGINAL STIFFNESS OF EACH REGION
C    DC    ORIGINAL CRACKING DISPLACEMENT
C    DY    ORIGINAL YIELDING DISPLACEMENT
C    K    INDEX FOR CHANGE OF POSITION  K#0 NON
C    J    INDEX FOR CHANGE OF SIGN OF VELOCITY  J#0 NON
C    DZ    RELATIVE DISPLACEMENT
C

**OUTPUT DATA**

BK    STIFFNESS AT NEXT STEP
C    DMAX    YIELDING OR MAXIMUM DISPLACEMENT
C    DMIN    NEGATIVE YIELDING OR MINIMUM DISPLACEMENT
C    D01    CENTER OF FIRST REGION
C    D02    CENTER OF SECOND REGION
C    OCE    ORIGINAL OR MODIFIED CRACKING DISPLACEMENT
C    ALFA    RATIO OF REDUCTION IN RIGIDITY
C

**DIMENSION 8(3)**

GO TO (1000,2000),J

1000 GO TO (1100,1200,1300),K
1100 BK=ALFA*B(1)
GO TO 9000
1200 BK=ALFA*B(2)
GO TO 9000
1300 BK=ALFA*B(3)
GO TO 9000
C
2000 IF(K) 3000,3000,4000
C
3000 K=1ANB(K)
GO TO (6000,3200,3300),K
3200 D01=DO2-DO2*OCE
GO TO 6000
3300 DMIN=02
C

4000 GO TO (6000,+100,+200),N
4100 001=02-002-DCE
   GO TO 6000

4200 0MAX=0Z
   Z1=OMAX-OMIN
   Z2=0Y-DC
   D01=Z1*Z2/(2.0*0Y)
   GO TO 5000

5000 Z3=(Z1-2.0*0Y)*Z2(3)
   Z4=(B1)*DC+B(2)*Z2
   Z5=2.0*Z3/Z4
   ALFA=0Y#Z5/Z1
   D02=(OMAX+OMIN)/2.0
   DCE=Z1*0C/2.0/0Y
   6000 8K=ALFA*B(1)
   9000 CONTINUE
   RETURN
   END

SUBROUTINE KAZU(V1,V2,X,XMAX,XMIN,XE,XCE,K1,K2,K,J)

C

JUDGE FOR CHANGE OF POSITION AND DETERMINATION OF POSITION

C

INPUT DATA

V1  INCREMENTAL DISPLACEMENT
V2  INCREMENTAL DISPLACEMENT OF LAST STEP
X   RELATIVE DISPLACEMENT
XMAX YIELDING OR MAXIMUM DISPLACEMENT
XMIN NEGATIVE YIELDING OR MINIMUM DISPLACEMENT
XE  RELATIVE DISPLACEMENT SHOVED IN SKELETON CURVE
XCE ORIGINAL OR MODIFIED CRACKING DISPLACEMENT
K1  INDEX FOR POSITION

C

OUTPUT DATA

K2  INDEX FOR POSITION AT NEXT STEP
K   INDEX FOR CHANGE OF POSITION K=0 NON
J   INDEX FOR CHANGE OF DIRECTION

C

IF(V1*V2) 2000,2000,100

C

100 J=1
   IF(V2) 1000,110,110
   110 IF(X-XMAX) 200,150,150
   150 K2=3
   GO TO 1600
   200 IF(XE-XCE) 300,250,250
   250 K2=2
   GO TO 1600
   300 IF(XE-XCE) 9999,350,350
350  K2=1
   GO TO 1600

C
1000 IF(X-XMIN) 1150,1150,1200
   1150  K2=3
   GO TO 1600
   1200 IF(XE+XCE) 1250,1250,1300
   1250  K2=2
   GO TO 1600
   1300 IF(XE-XCE) 1350,1350,9999
   1350  K2=1
   GO TO 1600

C
1600 IF(K2-KL) 1620,1630,1610
   1620  K=K2
   GO TO 5000
   1630  K=O
   GO TO 5000

C
2000 J=2
   IF(V2) 3100,3100,2100
2100 IF(X-XMIN) 2150,2150,2200
   2150  K2=-3
   GO TO 3500
   2200 IF(XE+XCE) 2250,2250,4000
   2250  K2=-2
   GO TO 3500
   3100 IF(X-XMAX) 3200,3150,3150
   3150  K2=3
   GO TO 3500
   3200 IF(XE-XCE) 4000,3250,3250
   3250  K2=2
   GO TO 3500
   4000  K2=1
   IF(K2-KL) 3500,5500,3500
   5500  K=O
   GO TO 5000
   3500  K=K2
   3550  K2=1
   GO TO 5000

C
9999 WRITE(16,8000)
3000 FORMAT(11X,5X,28H**** LOGICAL MISTAKES ****/)
5000 CONTINUE
   RETURN
END

C
DATA EXAMPLE
C
4
D TYPE
15.0   0.01   8310.792
0.6    2.5
3
2.0    2.5   1.606
ORIGIN ORIENTED MODEL
0.1 0.50 0.05 1.0 2
STOP
DEGRADING TRI-LINEAR MODEL
0.1 0.25 0.50 0.25 0.02 1.0 2
STOP

SMCH 1081
SMCH 1082
SMCH 1083
SMCH 1084
SMCH 1085
SMCH 1086
SMCH 1087
SMCH 1088
SMCH 1089
SMCH 1090
SMCH 1091
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