EARTHQUAKE FAILURE CRITERIA OF DETERIORATING HYSTERETIC STRUCTURES

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SUMMARY

A proposed simple technique to represent deteriorating force-displacement relations is verified through dynamic bending tests of reinforced concrete specimens. Deterioration functions of stiffness and damping adopting a measure of accumulated damage, defined from the test results, are used to calculate earthquake response with a step-wise linearization technique. Deterioration effects are found predominant for short-period structures relative to peak frequency of excitations. Use of accumulated damage or total dissipated energy instead of conventional maximum ductility factor is recommended as an earthquake failure criteria of deteriorating hysteretic structures.

INTRODUCTION

Recently, increasing emphasis has been given to deterioration effects of reinforced concrete structures during strong earthquakes, on the basis of recorded seismograms[1)] and loading tests of structural elements[2)]. The purposes of this paper are (1) to verify a previously proposed new deteriorating hysteretic model[3)] through several types of bending tests of reinforced concrete specimens, (2) to clarify the effects of deterioration of structural stiffness and energy absorbing capacity on earthquake response, (3) to find the best measure of structural failure from comparison of earthquake response of linear, conventional bilinear hysteretic and the proposed deteriorating hysteretic structures.

AN EQUIVALENT LINEAR MODEL OF DETERIORATING HYSTERETIC STRUCTURES

Equation of motion of single-degree-of-freedom structures with equivalent linear parameters is written as,

$$\ddot{\mu}+\beta_{eq}(D,\mu)\dot{\mu}+\omega_{eq}^2(D,\mu)\mu = -r_s\omega_o^2 g(t)/g_{max} \tag{1}$$

where, μ:ductility factor displacement, β_{eq} and ω_{eq}^2:equivalent damping coefficient and stiffness of deteriorating structures, respectively, D:accumulated damage, r_s:a parameter which sets ratio of maximum acceleration of excitation to yielding acceleration ω_o^2, g_{max}:maximum value of excitation g(t).

The author proposed a new and relatively simple technique[3)] which represents deterioration effects by a measure of accumulated damage, i.e.,

$$\omega_{eq}^2(D,\mu) = f_s(D)\omega_{eq}^2(\mu), \qquad \beta_{eq}(D,\mu) = f_d(D)\beta_{eq}(\mu) \tag{2}$$

in which, $\omega_{eq}^2(\mu)$ and $\beta_{eq}(\mu)$ are equivalent structural parameters depending on amplitude μ without deterioration effects, $f_s(D)$ and $f_d(D)$ are proposed

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deterioration functions which define degree of deterioration by a measure of D. A significant merit of this model is the capability of simple representation of complex deterioration processes by employing new deterioration functions. Hence, even nondeterministic response analysis of deteriorating structures has become possible[3].

CYCLIC BENDING TESTS OF REIFORCED CONCRETE SPECIMENS

Testing Apparatus

Reinforced concrete specimens were subjected to cyclic bending tests to verify the proposed technique and also to determine parameters of deterioration functions. General view of experiment is shown in Photo-1. Specimens shown in Fig.1 have cross section of 150X100mm and length of 1200mm. Four reinforcing bars (9.5mmϕ) are used, equivalent to a reinforcement ratio of 1.1%. Lateral tie hoops (6. mmϕ)are placed at every 80mm to prevent shear failure.

The deteriorating force P and displacement δ relation at the center of the simply supported specimen with a span L (=1000mm) shown in Fig.2 is measured for further analyses in this study. It is noted that bending moment M at the center and rotation θ at the supports are proportional to P and δ, respectively, for an elastic members as,

$$M = (L/4)P, \qquad \theta = 3L\delta \qquad (3)$$

Loading Test with Constant Displacement

Fig.3 is an example of the deteriorating restoring force-displacement relation of a specimen under cyclic bending with constant ductilty factor (D.F.) amplitude of 4.4. D.F. is normalized displacement by a measure of yielding displacement of longitudinal tensile bars. Hysteresis loops of 1st, 10th and 20th loading cycle exhibit significant deterioration both in stiffness and area of loops. Shapes of hysteresis loops of most specimens were found to change from a foot-ball type to inverted S type with narrow area, which are mainly due to yielding of reinforcing bars, cracking and crashing of concrete.

Deterioration of maximum reaction of hysteresis loops as compared to that of the 1st loop is plotted in Fig.4 against number of loading cycles N for various values of D.F.. Since loading amplitude is kept constant at each test, deterioration of average stiffness of each loop can be esitmated approximately from that of maximum reaction.

In calculating accumulated damage, failure is tentatively defined as the point where the maximum reaction is reduced to 60% of the initial value. Relation between log N and log μ at the defined failure plotted in Fig.5 results in

$$N\mu^{4.1} = 9.X10^{3} \qquad (5)$$

Thus, the accumulated damage D(n) due to n_i times of loading with μ_i amplitude (i=1,·····,m) is calculated as follows[4],

$$D(n) = \sum_{i=1}^{m} \Delta D_i = \sum_{i=1}^{m} n_i(\mu_i/9.4)^{4.1} \qquad (6)$$

In Fig.6, horizontal axis of Fig.4 is changed into accumulated damage D. Even though some variation is found among different loading levels, it seems that the maximum reaction which is almost equivalent to stiffness deteriorates linearly with accumulated damage. In Fig.7, deterioration of area of hysteresis loops showing capacity of energy absorption is also plotted to decrease exponentially with accumulated damage.

These results appear to verify the proposed technique, which measures effects of structural deterioration by accumulated damage. Thus, the next two deterioration functions $f_s(D)$ and $f_d(D)$ are obtained by least square fit of experimental data.

$$f_s(D) = 1.-.4D(n), \qquad f_d(D) = 1.-.8D^{.2}(n) \tag{6}$$

These functions are shown in Fig.8.

Loading Test with 3 Stages of Displacement

In order to check the effects of the sequence of loading with different displacement, specimens were subjected to 3 stages of D.F. amplitude of 2, 3 and 4 in which the number of loading cycles were controlled to result in the same accumulated damage.

Sections of deteriorating stiffness with D.F. amplitude of 3 are excerpted from the test results of 3 different sequences of loading and plotted in Fig.9. Deterioration of area is also plotted in Fig.10 for the same tests as in Fig.9. In these two figures, the larger deterioration is found after the larger amplitude of loading, even though accumulated damage has the same value. The reason is due to different range and consequently different mechanisms of damage. However, disprepancies from the results with constant amplitude loading are such that the deterioration process can also be described by Eq.(6).

Loading Test with Constant Reaction

In addition to displacement controlled loading tests, reaction controlled loading tests were conducted to check the effects of types of loading on deterioration process. After a gradual increase of displacement and accumulated damage due to reaction controlled tests, the specimens suddenly collapsed showing a clear point of failure.

Deterioration of equivalent stiffness of hysteresis loops is plotted in Fig.11 against accumulated damage. Although specimens do not survive until accumulated damage reaches 1.0, deterioration processes for different loading levels can also be explained well by Eq.(6), which again verifies the effectiveness of the proposed technique to be used for earthquake response analyses in the following sections of this study.

EARTHQUAKE RESPONSE AND FAILURE CRITERIA

Calculation of earthquake Response

Time history of earthquake response of deteriorating, hysteretic structures is calculated using a step-wise linearization technique[5)], as represented in Eq.(1). After every half cycle of structural response, accumulated damage D is calculated from Eq.(5) and their deterioration functions $f_s(D)$ and $f_d(D)$ are determined from Eq.(6). Amplitude dependent equivalent structural parameters $\beta_{eq}(\mu)$ and $\omega^2_{eq}(\mu)$ in Eq.(2) are calculated from W.D.Iwan's hysteresis loops of virginal loading[6] shown in Fig.12. Iterative process of calculation is taken so as to let $\beta_{eq}(\mu)$ and $\omega^2_{eq}(\mu)$ match the corresponsding response amplitude μ.

An example of calculated earthquake response is shown in Fig.13, in which the following sets of parameters are used. T_o=0.4sec., h_o=0.02 for a structure, α=0.05, μ_c=1.0, γ=0.375 for Iwan's model, r_s=1.5 for acceleration of Millikan base EW component recorded at San Fernendo earthquake (1971).

A gradual decrease of natural frequency is found, depending both on increasing response amplitude and accumulated damage. It is noted that response amplitude does not decrease even after 12 seconds where the level of input acceleration is decreasing. This is due to the matching of equivalent natural frequency and predominant frequency of input acceleration, which can easily be understood from Fig.14 where transition of ω_{eq} is plotted on a nonstationary envelope spectrum[7] of the excitation.

Earhquake Failure Criteria

Maximum value of ductility factor response has been widely used both in research and practice for a measure of structural safetey when nonlinear earthquake response analyses were conducted[8]. As discussed in previous sections, howevèr, number of loaded cycles is also an important parameter in judging structural safety, especially when deterioration effects can not be neglected.

In this section, relations among maximum ductility factor μ_{max}, accumlated damage D and total disspated energy E are investigated to furnish a *best parameter for earthquake failure criteria of deteriorating* hysteretic structures.

Three values of μ_{max}, D and E of linear, conventional bilinear hysteretic and the proposed deteriorating hysteretic structural models subjected to earthquake acceleration of ElCentro S00°E component recorded on 19-5-1940 are plotted on Figs.15.16 and 17 against intensity parameter r_s. Structural parameters of T_o=0.65sec. and h_o=0.02 are used. Other parameters are same as in Fig.13.

In the range where maximum input acceleration is less than yielding acceleration (r_s<1.0), maximum ductility factor response μ_{max} of each model is less that 3.0. Hence, little difference is found in values of μ_{max}, D and E depending on types of structural models. This result suggests that, in this range, any values of μ_{max}, D and E of any model can be a measure of structural safetey.

In the range of 1.0<r_s<1.5, μ_{max} of both stationary and deteriorating hysteretic response becomes somewhat larger than that of the linear model. Effects of deterioration in this range is found only in accumulated damage D. Hence,D of deteriorating model is preferable as a measure of structural safety.

When intensity of maximum acceleration becomes very large (r_s>1.5), μ_{max} and D of the deteriorating model grow significantly and E of the same model shows its upper limit to indicate structural failure clearly. Thus, in this range, any values of μ_{max}, D and E of linear and stationary hysteretic models can not be, but any values of a deteriorating hysteretic model can be, a dynamic failure criteria of the structure.

To examine the effects of natural period T_o of the structure in small elastic vibration on earthquake response of the 3 models, response spectrum of μ_{max} and D are calculated. Maximum values of 8 acceleration records in Table-1 are scaled to be 300cm/sec^2 for the excitation. The yielding acceleration of structures is set following the design spectrum shown in Fig.18.

Average ductility factor of 3 models are plotted in Fig.19 to show the significant difference depending on a model, where T_o<0.6sec.. The reason that deterioration effects are predominant for short-period structures relative to peak frequency of the excitation can also be understood from the transition of equivalent frequency as discussed in Fig.14. However, it is still difficult to judge structural failure only from μ_{max}.

The accumulated damage D of the deteriorating models at the end of the earthquake response is shown in Table-1 for different T_o and different excitations. The sign "F" in the table means complete failure of structures, i.e., D≥2.0. Finding just a few values of D between 0.5 and 2.0, it can be concluded that failure of deteriorating structures occurs not gradually but rapidly when the value of D becomes larger than 0.5. The same conclusion can be drawn from Fig.16. Relatively sudden collapse of tested specimens due to constant reaction controlled tests in Fig.11 at the point where D is about 0.5 also agrees well with the accumulated earthquake response of this section.

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REFERENCES

1) Iemura,H. and Jennings,P.C., "Hysteretic Response of a Nine-Story Reinforced concrete Buildings", Earthquake Engineering and Structural Dynamics, Vol.3, pp.183~201, 1974.
2) Shiga,T., et al, "Experimental Study on Dynamic Properties of Reinforced Concrete Shear Walls", Proc. of 5th WCEE, Vol.I, pp.1157~1166, 1974.
3) Goto,H., and Iemura,H., "Earthquake Response Characteristics of Deteriorating Hysteretic Structures", Proc. of 6th WCEE, Preprint 3,pp. 133~138, 1977.
4) Miner,M.A., "Cumulative Damage in Fatigue", Journal of Applied Mechanics, Vol.12, pp.A159~A164, 1945.
5) Goto,H. and Iemura,H., "Linearization Techniques for Earthquake Response of Simple Hysteretic Structures", Proc.of JSCE, No.212,pp.109~119,1973.
6) Iwn,W.D., "The Response of Simple Stiffness Degrading Structures", Proc. of 6th WCEE, Preprint 3, pp.121~126, 1977.
7) Trifunac,M.D., "Response Envelope Spectrum and Interpretation of Strong Earthquake Ground Motions", BSSA, Vol.61.No.21, pp.346~356, 1971.
8) Otani,S., "SAKE (A Computer Program for Inelestic Analysis of R/C Frames to Earthquake)", Dept. of Civil Engrg., Univ. of Illinois, 1974.

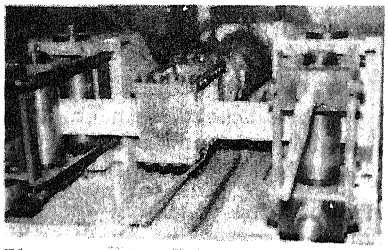


Photo-1 Dynamic Bending Test

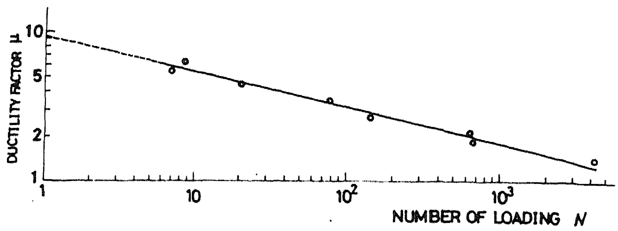


Fig.5 μ-N Curve at Defined Failure

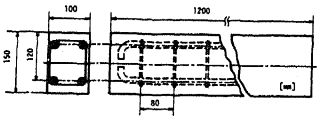


Fig.1 Dimension of Specimen

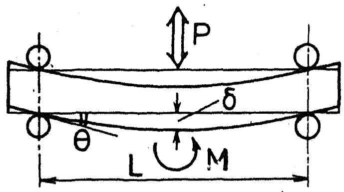


Fig.2 Bending System

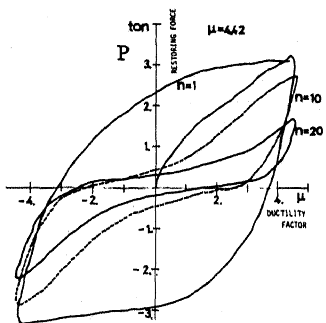


Fig.3 Deteriorating Hysteresis Loops

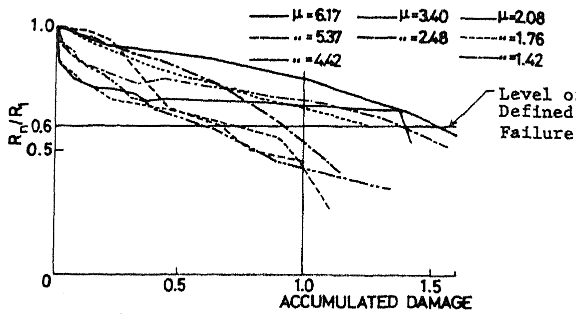


Fig.6 Deterioration of Peak of the Loops vs Accumulated Damage

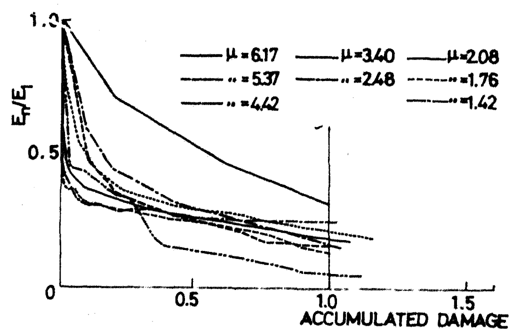


Fig.7 Deterioration of Area of the Loops vs Accumulated Damage

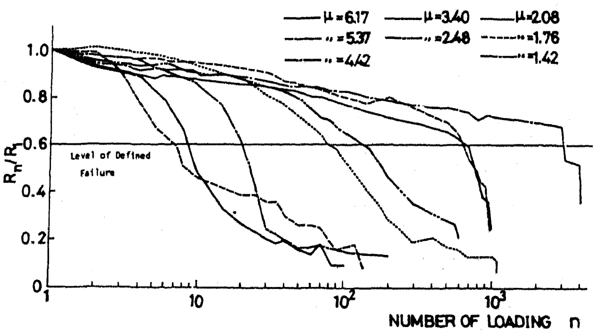


Fig.4 Peak of Hysteresis Loops against Number of Loading

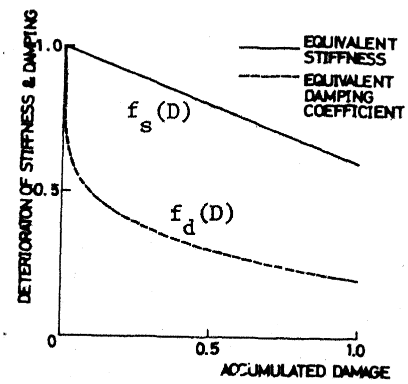


Fig.8 Deterioration Functions

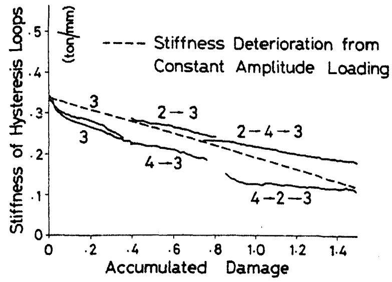


Fig.9 Effects of Sequence of 3 Stages of Loading

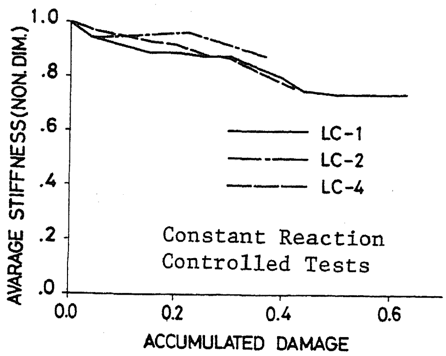


Fig.11 Deterioration of Stiffness

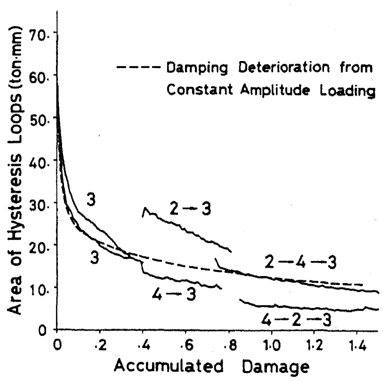


Fig.10 Effects of Sequence of 3 Stages of Loading

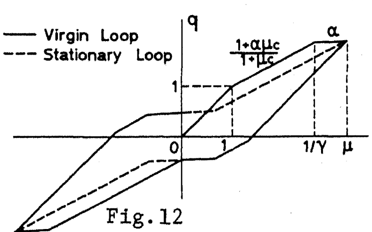


Fig.12 Iwan's Hysteresis Loop

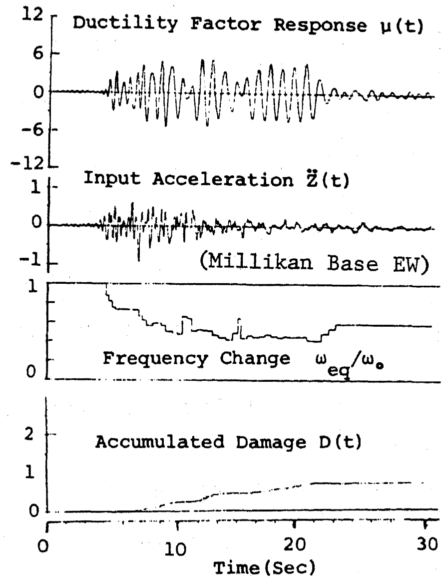
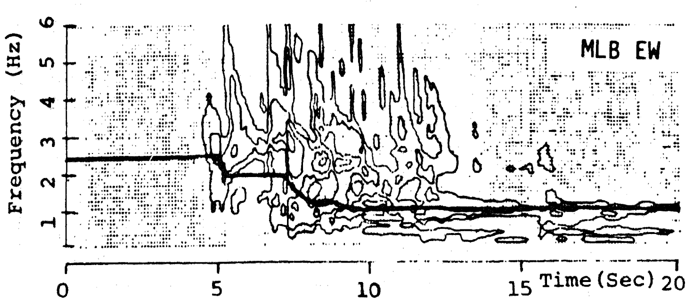


Fig.13 Response of a Deteriorating Hysteretic Structure.

Fig.14 Transition of ω_{eq} and Non-statinary Envelope Spectrum



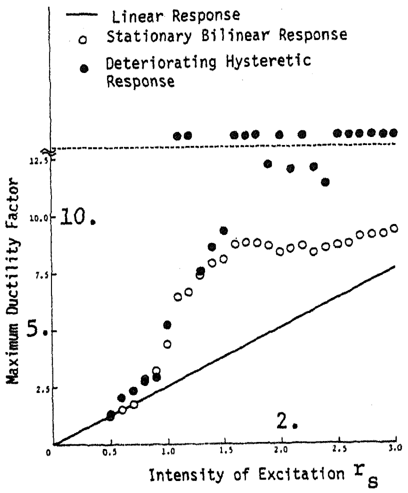


Fig.15 Maximum Ductility Factor of 3 Models

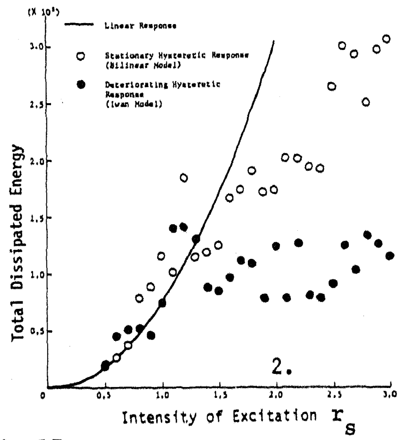


Fig.17 Total Dissipated Energy

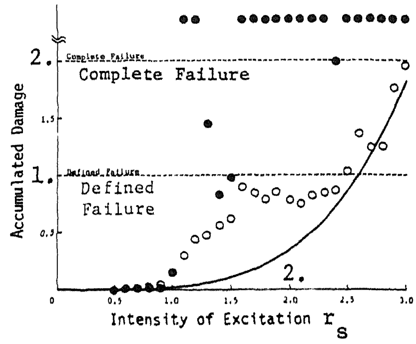


Fig.16 Accumulated Damage

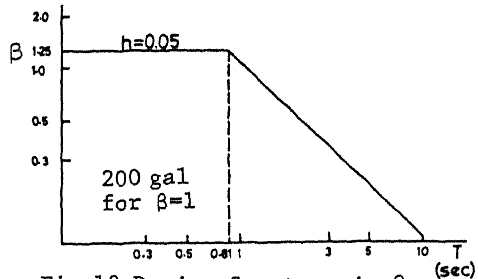


Fig.18 Design Spectrum in β

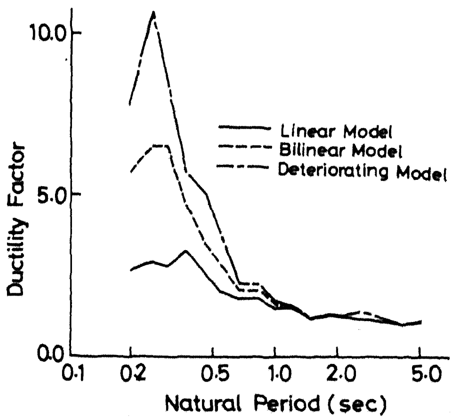


Fig.19 Average of Maximum Ductility Factor Response

Table-1 Accumulated Damage

Input / T_0(Sec)	ElCentro (NS)	ElCentro (EW)	Taft (NS)	Taft (EW)	MLB (NS)	MLB (EW)	JPL (NS)	JPL (EW)
0.58	F	.34	.31	.02	.00	.00	.00	.00
0.45	.98	F	F	.21	.00	.04	.01	.00
0.37	.02	F	F	F	.55	F	.05	.02
0.30	F	F	F	F	.21	F	F	.33
0.25	F	F	F	F	F	F	F	.49
0.20	F	F	F	F	F	.02	F	F