AN EVALUATION METHOD FOR THE EARTHQUAKE RESISTANT CAPACITY OF REINFORCED CONCRETE AND STEEL REINFORCED CONCRETE COLUMNS

by

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SYNOPSIS

An evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns by utilizing the force deflection relationship of column specimens subjected to axial force and repeated and reversed lateral loading of considerable intensity is proposed. An approximate response analysis for a non-linear structural system was developed based on random vibration theory and was applied to models represented by a single-degree-of-freedom system subjected to a constant white noise acceleration. It is confirmed that yield point level, hysteretic envelope slope, hysteretic damping ratio and ductility factor are the most important components of earthquake resistant capacity.

INTRODUCTION

A number of modern reinforced concrete buildings had severe damage caused by recent strong earthquakes. Some reinforced concrete buildings suffered from big damage in the columns due to the shearing forces of the earthquakes resulting in collapse of the whole buildings.

Large scale model tests of reinforced concrete and steel reinforced concrete column specimens have been carried out to investigate the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns. A lot of force deflection relationship were obtained from the testing results and were reported in the recent technical papers. There is, however, no reasonable evaluation method for the testing results - particularly for the testing results of column specimens with ductility after yielding.

This paper deals with an evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns with ductility after yielding utilizing the force deflection relationship of column specimens subjected to axial force and repeated and reversed lateral loading of considerable intensity. It is considered that the same method is applicable to the evaluation for the earthquake resistant capacity of girders, joints of girders and columns, shearing walls and any building structures constructed of reinforced concrete and steel reinforced concrete.

ASSUMPTIONS

a) Stationary acceleration spectrum of duration 30 sec. without any predominant frequency (white noise) was utilized to represent strong motion earthquake excitation.

One of the most significant contribution to the field of earthquake engineering was the introduction of the idea of the earthquake response spectrum which

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was first introduced by M.A. Biot, E.C. Robinson, G. Housner and others. Using the eight components of the four strongest ground motions recorded to date, Housner normalized each accelerogram to a common intensity level and by averaging the velocity spectra resulting therefrom obtained what is now commonly known as Housner's standard velocity spectra.

Using an analog computer, Bycroft studied the possibility of using a white noise acceleration process to represent earthquake ground motion of a given intensity level. Bycroft's results would seem to indicate that white noise is a reasonable simulation of earthquake excitation.

A more accurate method of predicting the maximum or peak response when concerned with long period and low damping ratio and when assuming a white noise acceleration process of finite duration is the method presented by Rosenblueth and Bustamente. The results were compared with Housner's standard velocity spectra and quite good agreement between them over the entire ranges of damping and period was reported in Reference (5). Therefore, a stationary acceleration of duration 30 sec. represented by a white noise was utilized for this evaluation.

b) Non-linear structural models with a degrading stiffness system represented by a single-degree-of-freedom system were used as shown in Fig.1. A non-linear dynamic response analysis for the models subjected to a stationary white noise acceleration was carried out by utilizing the force deflection curves obtained by large model tests of column specimens. An idealized force deflection curve with a degrading stiffness system is shown in Fig. 2.

In general, reinforced concrete and steel reinforced concrete column specimens show a tendency that the maximum strength at each step of deflection after yielding reduces due to repeated and reversed loading. An envelope line which plots the reduced maximum strength corresponding to each step of deflection can be approximated by a straight line, as shown in Fig. 2. The slope of the straight line is defined as the hysteretic envelope slope, and the ratio of the hysteretic envelope slope tangent to the initial stiffness slope tangent is named the slope tangent ratio of ductility. The damping factor induced by viscous damping (viscous damping ratio ζ_v) is considered to be 0.05 of the critical damping for the entire processes of the models. The damping factor induced by hysteretic damping (hysteretic damping ratio ζ_H) can be calculated by measuring the areas of hysteretic loops corresponding to each step of deflection of specimens. For example, the hysteretic damping ratio at the deflection $_2\delta_y$ is simply obtained, if the hysteretic loops are symmetrical with respect to the original point, as follows;

$$\zeta_{H(\delta = {}_2\delta_y)} = \frac{1}{\pi} \frac{\text{Area } 2,\ 3,\ 5}{\text{Area } 2,\ ②,\ 5 + \text{Area } 2,\ ②,\ 3} \tag{1}$$

where δ_y is the deflection at the yielding point of specimens.

In the case of reinforced concrete and steel reinforced concrete specimen tests, the hysteretic damping ratio ζ_H can be assumed to increase proportionally to the increase of $\sqrt{(\delta-\delta_y)/\delta_y}$ after yielding. Therefore, the hysteretic damping ratio at the deflection δ is approximated by

$$\zeta_H = {}_c\zeta_H \sqrt{\frac{\delta-\delta_y}{\delta_y}} \qquad (\delta \geq \delta_y) \tag{2}$$

where ${}_c\zeta_H$ is defined as the coefficient of hysteretic damping ratio. By using the method of least squares, the coefficient of hysteretic damping ratio can be obtained from the hysteretic damping ratios ζ_{Hn} at the deflection $\delta n (n = 1, 2, \ldots\ldots\ldots n)$ as

$$ {}_c\zeta_H = \sqrt{\delta y}\,\frac{\sum_{1}^{n} \zeta_{Hn}}{\sum_{1}^{n} \sqrt{(\delta n - \delta y)}} \tag{3}$$

In calculation, the hysteretic damping with a loop was transformed to the equivalent viscous damping for a linear system. Therefore, the equivalent viscous damping ratio ζ was obtained by adding the viscous damping ratio ζ_V to the hysteretic damping ratio ζ_H.

NUMERICAL CALCULATION

The mean expected maximum response values of the structural models having a degrading stiffness system shown in Fig. 2 were calculated by a non-linear response analysis when they were subjected to a stationary white noise acceleration.

The models with natural period Tn=0.1 sec.(m=1.0 ton sec.^{2}/cm, k=4940.0 ton/cm) were divided into four types according to the four different yield point levels of Model 1, 2, 3, and 4 with 1/2, 1/3, 1/4 and 1/5 of the mean expected maximum response values in linear system, respectively, where the mean expected maximum response value in linear system is defined as the mean of the maximum response values, if the structural model has a linear system without the yield point. In other words, the mean maximum response value in the degrading stiffness system can be replaced by the strength of the linear model with the equivalent earthquake resistant capacity.

In each model, the slope tangent ratio of ductility varies as 0.8, 0.6, 0.4, 0.2, 0.1, 0.0, ... -0.1 after yielding. The ductility factors of the models were calculated by a non-linear response analysis, when the coefficient ${}_c\zeta_H$ of hysteretic damping ratio varies from 0.03 to 0.12. For example, if the maximum response value of a column having a linear system is three times the yield point level of the column having a non-linear system, the ductility indicated in Fig. 3b will be required to prevent the complete collapse from the structure, when the structure is subjected to a strong earthquake. In other words, if the yield point, hysteretic envelope slope tangent, coefficient of hysteretic damping ratio and ductility factor of a column are measured by testing, the maximum strength of the linear model having the equivalent earthquake resistant capacity of the column can be presumed. The models with natural period Tn= 0.5 sec were also calculated by the same manner. There is, however, little difference between the models with the different natural periods in the calculated response values.

In this calculation, an approximate non-linear response analysi was developed based on random vibration theory and was applied to models with a single-degree-of-freedom system subjected to a constant white noise acceleration. The non-linear response analysis based on random vibration theory is introduced in Reference (4) and examined by the method of simulation in APPENDIX I.

CONCLUSIONS

It is confirmed that yield point level, hysteretic envelope slope, hysteretic damping ratio and ductility factor are the most important components of earthquake resistant capacity, when reinforced concrete and steel reinforced concrete columns are subjected to a white noise ground motion.

In order to maintain desirable hysteretic envelope slope after yielding, it will be recommended to avoid the shear-type failure of columns. It is also important to use sufficient ductile materials such as steel and reinforcing bars in columns to increase hysteretic damping ratio and ductility factor of columns. When a column without sufficient ductility is used, the allowable strength level should be estimated lower than the conventional one in the case of strong earthquakes.

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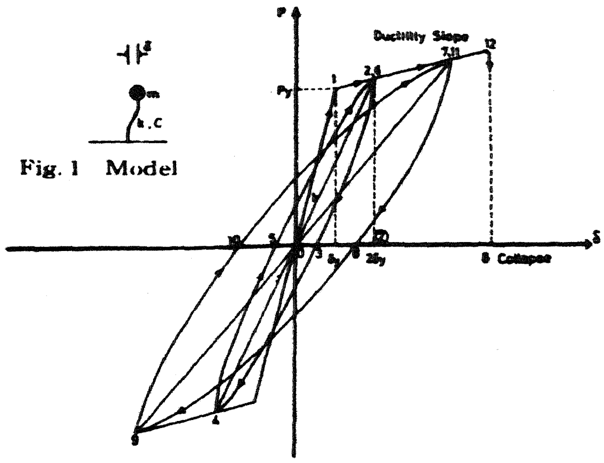

Fig. 1 Model

Fig. 2 Hysteretic Curve

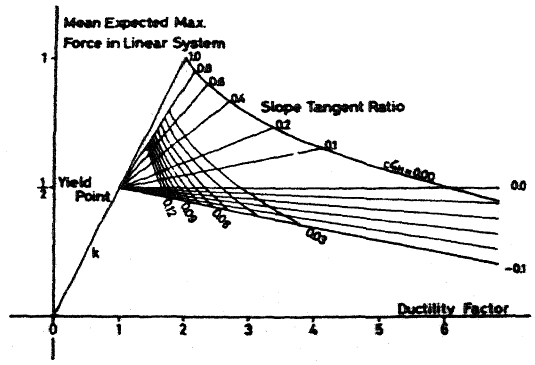

Fig. 3a Ductility Requirement
(Model 1)

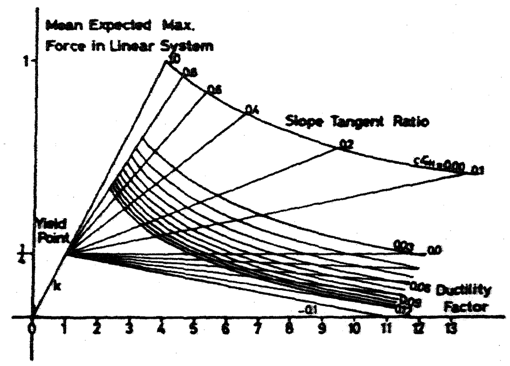

Fig. 3c Ductility Requirements
(Model 3)

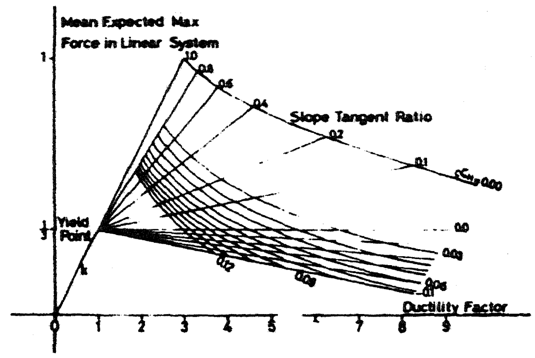

Fig 3b Ductility Requirement
(Model 2)

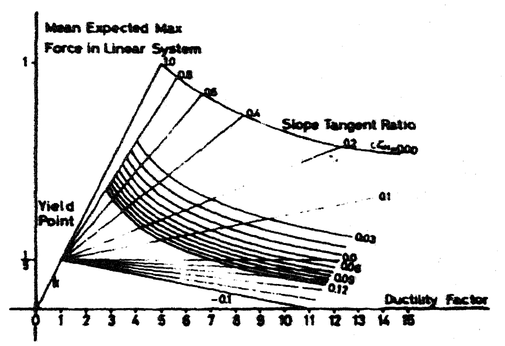

Fig. 3d Ductility Requirements
(Model 4)

APPENDIX I : Non Linear Response by Random Vibration Theory and Simulated Deterministic Analysis

Earthquake : Stationary

$S = 70.0\ gal^2 \cdot sec./rad.$ (Ave. Max. Acc. 300gal.)

$T = 20.48$ sec.

White Noise

Building : One degree of freedom

$m = 0.01\ t.sec^2/cm$ $P_y = 19.6$ ton.

$k = 15.8$ t./cm ($T = 0.5$ sec.)

$\zeta_v = 0.05$

$c^{\zeta}H = 0.08$

ky / k	Max. D.F. Response by 10 Simulated Earthquakes												Random Response			Comparison Ratio		
	1	2	3	4	5	6	7	8	9	10	Ave.	S.D.	(1)	(2)	(3)	(4)	(5)	(6)
1.0	1.60	1.93	1.53	2.02	1.74	1.62	1.82	2.15	1.68	1.59	1.77	0.21	1.66	1.78	1.93	0.94	0.99	1.03
0.6	1.65	1.95	1.80	1.97	1.85	1.93	1.95	2.08	1.75	1.69	1.86	0.14	1.80	1.96	2.18	0.96	0.95	0.92
0.4	1.77	2.05	2.06	2.15	1.93	1.77	2.03	1.95	1.86	1.95	1.95	0.13	1.90	2.11	2.38	0.96	0.92	0.87
0.2	2.07	2.55	2.14	2.17	2.15	2.18	2.72	2.16	2.02	2.59	2.28	0.24	2.06	2.34	2.75	0.99	0.97	0.92
0.1	2.14	2.85	2.32	2.19	2.24	2.58	3.35	2.34	2.11	2.87	2.50	0.41	2.17	2.54	3.07	0.96	0.98	0.95
0.0	2.33	3.20	3.05	2.77	2.22	3.05	4.08	3.04	2.69	2.92	2.94	0.51	2.33	2.84	3.68	1.04	1.04	0.94
-0.02	2.47	3.27	3.26	2.86	2.25	3.11	4.26	3.24	2.79	2.94	3.05	0.55	2.38	2.93	3.90	1.05	1.04	0.92
-0.04	2.60	3.39	3.59	2.93	2.25	3.18	4.40	3.44	2.95	3.06	3.18	0.59	2.43	3.05	4.18	1.07	1.04	0.90
-0.06	2.82	3.45	4.25	2.97	2.25	3.22	4.64	3.58	3.29	3.10	3.36	0.69	2.49	3.18	4.60	1.07	1.06	0.88
-0.08	3.14	3.50	5.94	2.90	2.29	3.29	4.84	3.61	3.61	3.35	3.65	1.03	2.55	3.34	5.32	1.03	1.09	0.88
-0.1	3.45	3.56	9.26	2.77	2.32	3.33	5.02	3.72	4.05	4.24	4.17	1.94	2.62	3.54	8.35	0.85	1.18	0.73

Ave. : Average , S.D. : Standerd Deviation , - : Collapse happens before the end of Earthquake

(1),(2),(3) : Max. D.F. response by Random Vibration Theory with Reliability 0.159 , 0.5 , 0.841 respectively

(4) : (Ave.-S.D.)/(1) , (5) : Ave./(2) , (6) : (Ave.+S.D.)/(3)

Table 1 Comparison of Response Value