RELATION BETWEEN YIELD STRENGTHS AND RESPONSE DISPLACEMENTS OF STRUCTURES

by

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SYNOPSIS

To judge the safety of the low and medium height framed typed reinforced concrete structures during strong earthquakes, the ductility evaluation concept is most commonly used. The relation between the amount and distribution mode of strength for the weak column and strong girder type structures and the response ductility factors of each story level were presensed and the required strength ratio and its distribution mode for the seismic resistant design of such structures were discussed based on the response results for the artificial earthquakes.

INTRODUCTION

Two evaluation concepts are most commonly employed to the seismic resistant design of the low rize reinforced concrete structures, also to the safety judgement of the existing buildings.

One is the strength evaluation and the other is the ductility evaluation. Walled type structures (non-ductile structures) are usually evaluated their safety by the amount of strength which ensures that response displacements do not exceed the cracking or the yielding displacements. On the other hand, moment resisting frame type structures (ductile structures) are evaluated by the amount of ductility after yielding, because those structures are very hard to have enough strength like walled type structures such as response displacements stop before yield displacements. Studies on single-degree-of-freedom system have suggested that there are close relationships between the yield strength values and the response ductility factors. But same kind of studies on multi-degree-of-freedom system are very few.

This report has discussed the relation between the yield strength and the response ductility factor of multi-degree-of-freedom system by the statistical procedure.

The weak column and strong girder type reinforced concrete structures of low and medium height in which yielding of the column ends occures prior to that of girder ends were taken as the object of analysis and were modeled into the pure shear deformation models which were three-degrees-of-freedom systems. The hysteresis curve of each spring was assumed to be stiffness degrading tri-linear (D-TRI). Fundamental periods of models, strength distribution modes to each story and the amounts of their story's strength were varied as parameters of models. For the input accelerograms, artificial earthquakes were used to eliminate the soil conditions or the peculiar characteristics of real earthquakes. Ten each of two types of earthquakes which have different peak frequencies were generated in this analysis.

GENERATION OF ARTIFICIAL EARTHQUAKE

Many methods to generate the artificial earthquake have been reported until now. In this report, almost the same procedure of Dr. Toki's method

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has been applied. The ground acceleration $X(t)$ during earthquake is reported as a product of a deterministic function $\psi(t)$ and a stationary random process $g(t)$ as follows;

$$X(t) = \psi(t) \cdot g(t) \qquad \text{———} \quad (1)$$

where $\psi(t)$ is a slowly varing function relative to the fluctuation of $g(t)$.

Herein the following representation of $g(t)$ is used

$$g(t) = \frac{1}{\sqrt{N}} \sum_{n=1}^{N} \cos(\omega_n t + \phi_n) \qquad \text{———} \quad (2)$$

in which W_n is a random variable with probablistic density $p(\omega_n)$ and ϕ_n is a random phase angle uniformly distributed in $(0, 2\pi)$ and N is a large possitive integer. The stationary random process expressed in Eq. (2) has been generated on a digital computer. Numerical computation is carried out by the Monte Carlo method. A random number W_n with density $p(\omega_n)$ and ϕ_n with uniform distribution in $(0, 2\pi)$ are computed from the pseudo-random numbers which are generated successively from a pair of preceding ones.

A set of twenty stationary random process $g(t)$ were generated by this method. In these computations, N was fixed to be 200 and time interval was 0.01 sec.

For the purpose of generation of random nonstationary process, the deterministic function $\psi(t)$ and probablistic function $p(\omega_n)$ are assumed to be

$$\psi(t) = a\,(t/t_p) \cdot \exp(1 - t/t_p) \cdot U(t) \qquad \text{———} \quad (3)$$

$$p(\omega_n) = \frac{1 + 4h_g^2\,(\omega/\omega_g)^2}{\{1 - (\omega/\omega_g)^2\}^2 + 4h_g^2\,(\omega/\omega_g)^2} \qquad \text{———} \quad (4)$$

where t_p, ω_g, h_g, $U(t)$ and a, are respectively, the peak time of $\psi(t)$, some characteristic ground frequency, damping ratio, unit step function and constant with the dimension of acceleration. The parameters of t_p, h_g were choiced to be 3.0 sec. and 0.6, respectively, for all nonstationary random process. But supposing two different kind of soil conditions, $\omega_g = \pi/0.3$ ($T_g = 0.3$ sec.) and $\omega_g = \pi/1.0$ ($T_g = 1.0$ sec.) were assumed. The ten artificial earthquakes with $\omega_g = \pi/0.3$ were indicated as TYPE S earthquake, and the other ten earthquakes as TYPE L earthquake.

In Fig. 3, each one of two types of sample records were shown and in Fig. 4, the average velocity spectrum were also shown.

MATHEMATICAL RESPONSE MODELS

In this report, the strength ratio was used instead of strength itself. The strength ratio α_i was defined by $\alpha_i = k_{yi}/k_g$, where k_{yi} is a shear coefficient as $k_{yi} = Q_{yi}/\Sigma W_i$ and k_g is a intensity of ground acceleration as $k_g = \alpha_{max}/g$. Q_{yi} and W_i are yield shear force and story weight of i-th story, respectively, α_{max} is a maximum ground acceleration and g is the acceleration of gravity.

The structures which yield in column ends prior to girder ends were replaced to the shear deformation models. Those models are the three-degrees-of-freedom system which have the equal lumped mass at each floor level. Four types of structural models which have different amounts of strength and its distribution mode as shown in Fig. 6 were analyzed. Case A ~ C have equal spring constant in each story and case D has a spring constant distribution mode such that the fundamental mode of elastic vibration is triangular in shape. Case A has a uniform strength distribution mode in each story, case B has one low strength story compared to the strengths of the other stories and case C, D have strength distribution mode which was determined as if all stories yielded simultaneously in the foundamental mode shape.

The skeleton curve of D-TRI model (Fig. 5) was determined by the assumption that $Q_y = 2Q_c$, $K_y = 1/4\ K_1$, where Q_c is the cracking shear force, K_1 is the initial stiffness and K_y is the equivalent yield stiffness.

RESPONSE ANALYSES

The relations between mean ductility factors μ and the fundamental period T_1 of structure were plotted in Fig. 7. In the case of the structure which has the uniform strength mode, the top story's ductility factor became larger than the lower story's ductility factor, especially for TYPE S earthquake in the shorter period range. If $\alpha_i = 1.0$ was choiced, the ductility factors of all stories were smaller than 2 for the range of $T_1/T_g > 1.0$, and if $\alpha_i = 1.5$, they did not exceed 1 for all period range. In case that the structure has a story of lower strength compared to the strengths of the other stories, the response displacement of that story is expected to be extremely large. This tendency was evident for the TYPE L earthquake, but when $T_1/T_g > 1.0$, the ductility factors were not so large. Case C, D whose yield strengths were determined by the assumption that the simultaneous yielding will occur in the first mode shape, represented the same tendency. If the first story's strength ratio α_1 was settled to be 1.0, the probability of exceeding the ductility factor 4 looks like very few, and in the zone of $T_1/T_g > 0.2$, ductility factors of each story were smaller than 2 and when $T_1/T_g > 0.5$, they were almost smaller than 1.0. If α_1 was settled to be 0.8, in the zone of $T_1/T_g > 0.3$, ductility factors did not exceed 4, when $T_1/T_g > 0.5$, smaller than 2 and when $T_1/T_g > 1.0$, they were smaller than 1.0.

CONCLUDING REMARKS

Although it is a big and difficult problem to determine the allowable ductility of structure and further experimental and analytical studies are necessary to be done, the relations between strength values or its distribution mode and response ductility factors were presented through the above mentioned analyses. They indicate that some preferable combination of strength ratio and strength distribution mode can control the response ductility factors, and also provide the following suggestions for the seismic resistant design of the weak column and strong girder type structures.

1. It is desirable to determine the strength distribution mode such that the each story yield simultaneously in the fundamental mode shape, because it is easy to control the each story's ductility factors almost equal. The strength ratio α_1 of first story is better to be taken as 1.0.

2. If α_1 is taken as 0.8, smaller than 1.0, the natural period T_1 is better to be settled longer than one half of predominant period of earthquake.

3. If the structure has the uniform distribution mode of strength, the top story's ductility factor will be expected to become fairly large, so $\alpha_i = 1.5$ is better to be choiced.

4. It is not preferable to design the structure which has a fairly low strength in some one story only, because the response displacement of that special story becomes very large.

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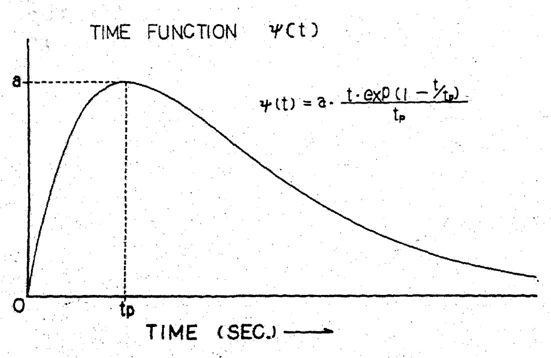

Fig-1 Time Function ψ (t)

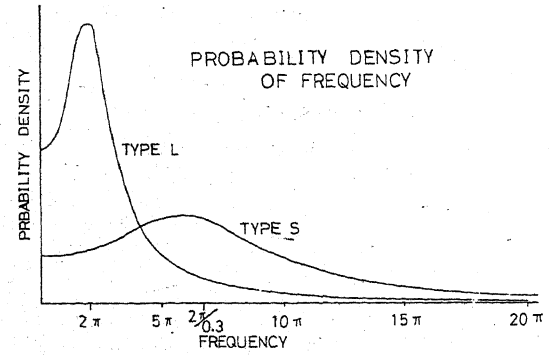

Fig-2 Probablistic Density of Frequency

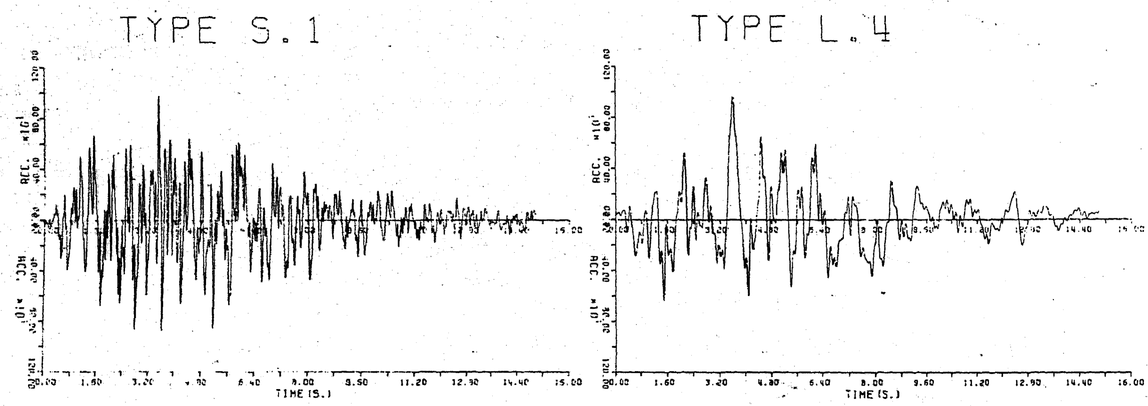

Fig-3 Wave Shape of Artificial Earthquake

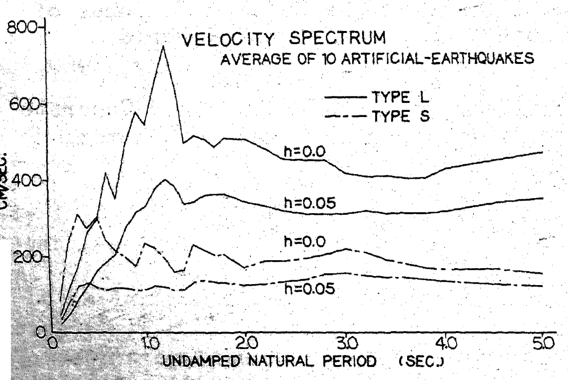

Fig-4 Average Velocity Spectrum

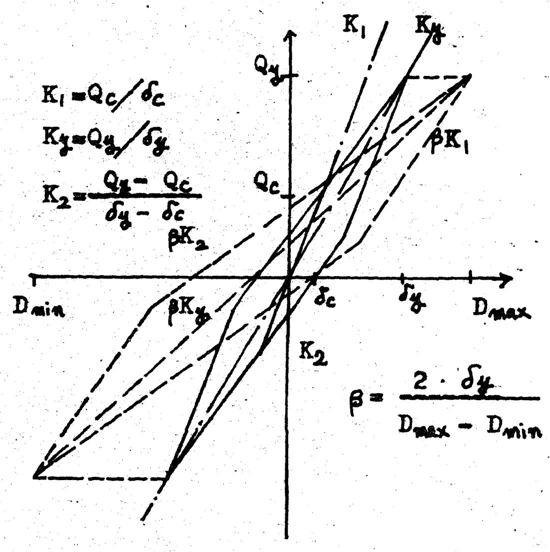

Fig-5 Hysteresis Rule of D-TRI Model

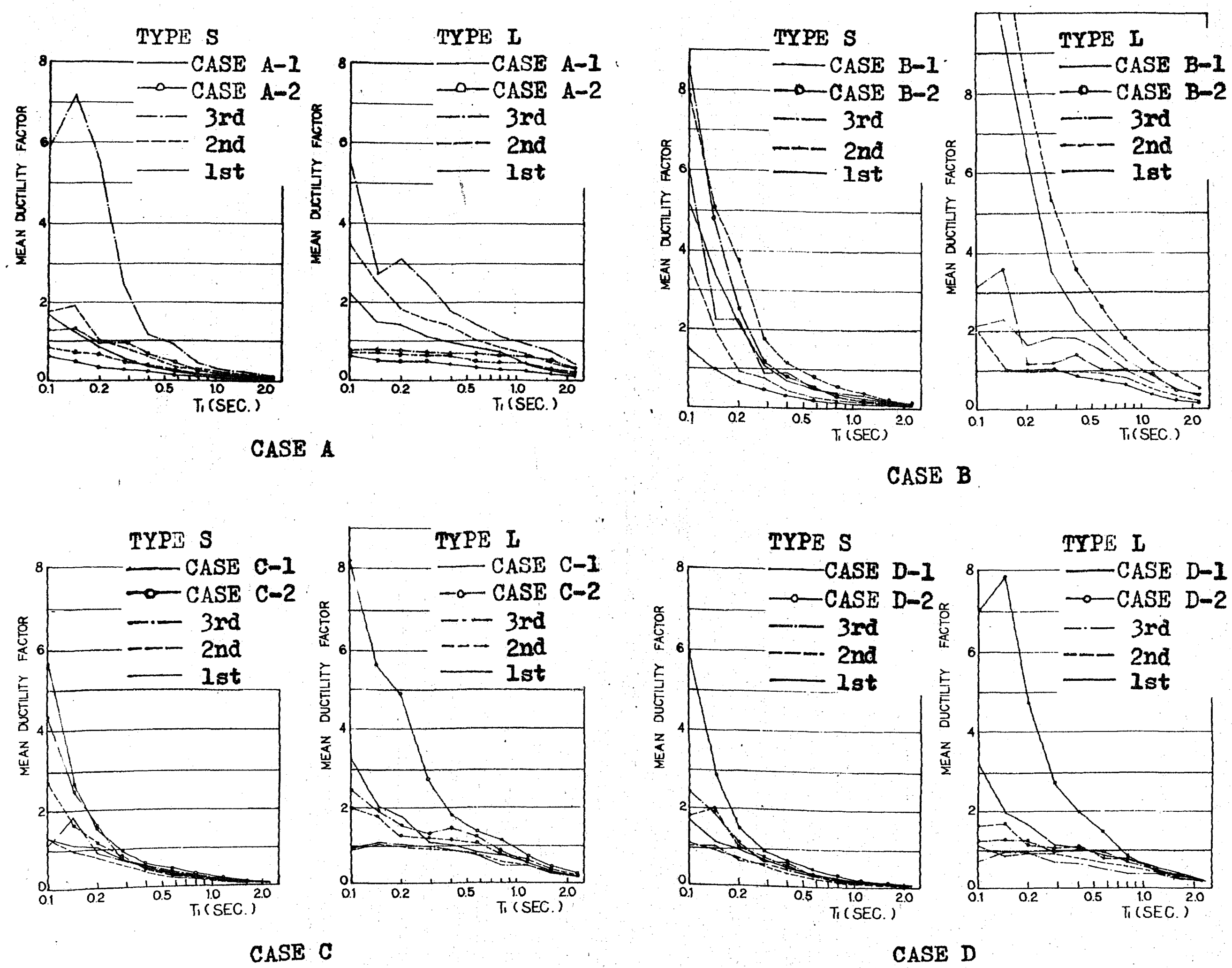

Fig-7 Relation Between Mean Ductility Factor and Fundamental Period of Structure

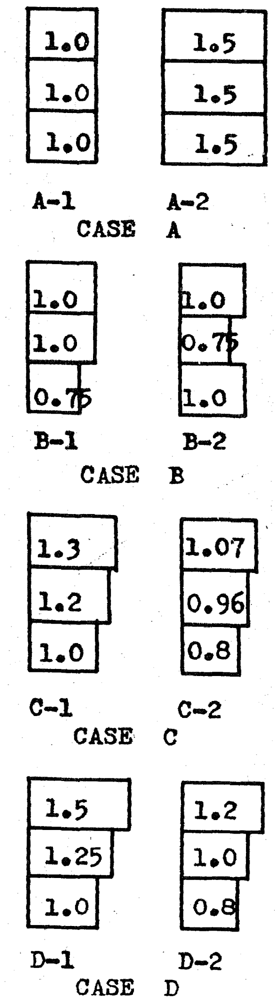

Fig-6 Mathematical Model