

Automated Seismic Design Method for Reinforced Concrete Structures

철근 콘크리트 구조물의 전산에 의한 내진설계법

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요 약

작업에 사용되고 있는 RC 구조물의 대개의 내진설계기법은 지진시 RC 구조물에 발생하는 손상의 분포상태를 고려치 않고 있다. 본 논문은 철근 콘크리트 구조물의 새로운 내진설계기법 즉 Miner's 법칙을 수정한 지진 발생시의 흡수에너지(Dissipated Energy)를 변수로 하는 손상모델[3]을 사용하여 RC 프레임의 각각의 Node에서의 손상정도를 수치적으로 나타내고 이들 손상값의 크기가 전 부재에 고르게 분포토록 하기 위하여 각 부재의 주철근량을 설계 변수로 채택한 설계기법을 소개하였다. 사용된 이력모델 및 손상모델의 정확성을 평가하기 위하여 해석적인 하중-변형곡선을 재생하여 실험곡선과 비교분석하였으며 제안된 내진설계법의 유용성은 3-bay 4-story 프레임 모델을 사용하여 입증하였다.

Abstract

Most of the conventional aseismic design methods for reinforced concrete structures, based on the strong-column weak-beam design concept, do not necessarily the state of damage distribution over the entire frame. This paper introduces a seismic damage-controlled design method for RC frames which aim at individual member damage indices. Three design parameters, namely the longitudinal steel ratio, the confinement steel ratio and the frame member depth, were studied for their influence on the frame response to an earthquake. The usefulness of this design method will be demonstrated with a three-bay four-story building frame so that, on the one hand, the method will reduce the damage as measured by the global damage index under the same earthquake and, on the other hand, will lead to a larger capacity enabling stronger earthquakes to be accommodated.

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1. Introduction

Current aseismic design philosophy relies strongly on the energy dissipation in structural components undergoing large inelastic deformations. Thus, it is commonly recognized that the seismic safety of RC framed structures depends on their total energy dissipation capacity. To utilize the full energy dissipation capacity of a RC frame, all the elements should be designed to absorb the equal energy dissipation level, so that the resulting damage is uniformly distributed over all elements of the frame.

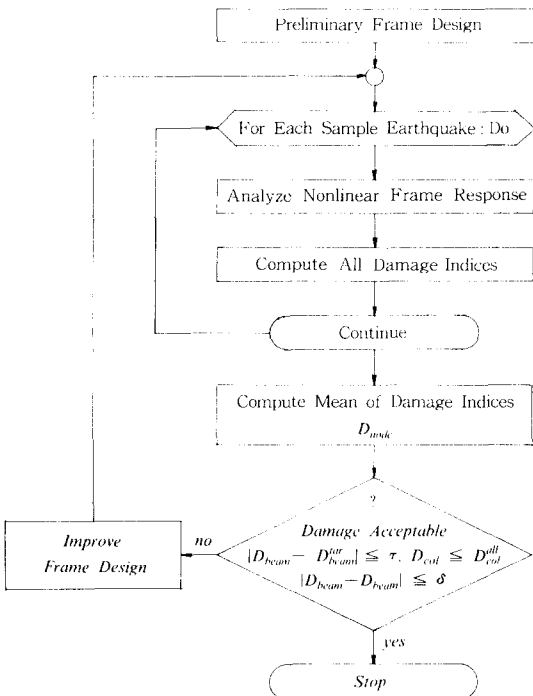
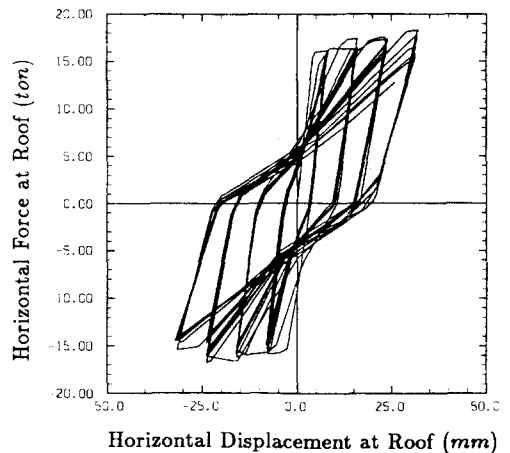


Fig. 1. Automated Damage-Control Design Method

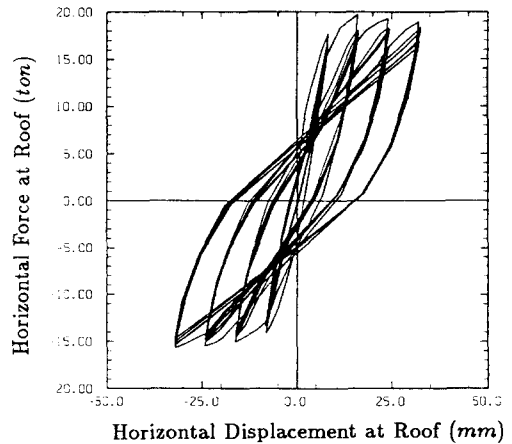
As can be seen in Fig 1, the general design procedure consists of the following steps: 1) perform a preliminary design of a frame, for example to satisfy the equivalent static lateral load requirements of the Uniform Building Code[11]; 2) perform a nonlinear dynamic analysis of the frame for seismic ground shaking of specified intensity, duration and spectral content, using program SARCF[6]; 3) compute the

mean damage index for both ends of each frame member; 4) evaluate the damage distribution using acceptance criteria specified by the engineer; 5) if the damage is unacceptable, automatically introduce certain design changes on the basis of design rules incorporated in the program; and 6) repeat steps 2 through 5 until the level of the frame damage is acceptable for the specified intensity of ground motion.

A basic hysteretic model [1] a damage model[2, 3, 4] for RC members, which play an important role in this study, are further verified by reproducing



Analysis



Experiment

Fig 2. Experimental and Analytic Load-Deformation Curves for Ohno's Frame[8]

experimental load–deformation curves for Ohno’s one-bay one–story frame[9], Fig 2. The main emphasis of this article is placed on numerical parameter studies that were conducted to gain a better understanding of some important design parameters and how they affect the seismic performance of framed concrete structures. From these studies a few design rules were derived and incorporated into a nonlinear frame analysis program to assure convergence towards an acceptable damage distribution.

2. Nodal Damage Model

In an earlier study[4], numerous damage models were evaluated critically, which had been proposed to represent damage of concrete members. It was concluded that none of these prior models is well suited to measure the residual strength and stiffness of damaged structural members and thus permit an acceptably accurate prediction of response to subsequent cyclic loading. The damage index D_c of Eq (1) quantifies the damage of a member section in a plastic hinge. Expressed in the form of a modified Miner’s rule, it takes into consideration the nonlinear relationship between moment and curvature, the strength deterioration rate and the number of load cycles to failure[4]. It contains damage modifiers, which reflect the effect of the loading history, and it considers the fact that RC members typically to positive and negative moments:

$$D_c = \sum_i (\alpha_i^+ \frac{n_i^+}{N_i^+} + \alpha_i^- \frac{n_i^-}{N_i^-}) \quad (1)$$

where i =indicator of curvature level, $N_i=(M_i-M_{fi})/\Delta M_i$ =number of cycles up to curvature level i to cause failure, n_i =number of cycles at curvature level i actually applied, α_i =damage modifier. The + and – superscripts indicate the direction of loading. $(M-M_{fi})$ and ΔM_i denote the strength drops at curvature level i , up to the failure moment and in a single load cycle, respectively[4].

The loading history effect is captured by including the

damage modifier α_i , which, for positive moment loading,

$$\text{is defined as } \alpha_i^+ = \frac{\frac{1}{n_i^+} \sum_{j=1}^{n_i^+} k_{ij}^+}{\bar{k}_i^+} \cdot \frac{\phi_i^+ + \phi_{i-1}^+}{2\phi_i^+},$$

where $k_{ij}^+ = M_{ij}^+/\phi_i^+$ is the stiffness during the j -th cycle up to curvature level i and $\bar{k}_i^+ = \frac{1}{N_i^+} \sum_{j=1}^{N_i^+} k_{ij}^+$ is the average stiffness during N_i^+ cycles up to curvature level i . This damage modifier is introduced to take into account that 1) the damage increment in the first cycle is larger than that of successive cycles under constant amplitude loadings, and 2) the damage caused by the cycle which follows a larger amplitude loading is larger than that which follows a smaller amplitude loading.

3. Generation of Artificial Earthquakes

As a nonstationary random process, artificial earthquake ground motions, $x(t)$, are generated by multiplying an envelope function, $s(t)$, and a stationary Gaussian process, $g(t)$. The envelope is here assumed to have a trapezoidal shape.

A Gaussian process, $g(t)$, can be obtained by using the well-known Kanai-Tajimi spectrum as the power spectral density

$$\text{function, } S(\omega) = S_n \cdot \frac{1 + 4\xi_g^2 (\frac{\omega}{\omega_g})^2}{(1 - (\frac{\omega}{\omega_g})^2)^2 + 4\xi_g^2 (\frac{\omega}{\omega_g})^2},$$

where ω_g is the characteristic ground frequency, ξ_g is the predominant damping coefficient, S_n is the intensity of Gaussian white noise over the range $-\infty < \omega < \infty$. The Gaussian process, $g(t)$, can be generated by using Monte Carlo technique[9],

$$g(t) = \sqrt{2} \sum_k \sqrt{G(\omega_k)\Delta\omega} \cdot \cos(\omega_k t - \phi_k) \quad (2)$$

where ϕ_k is the random phase angle, uniformly distributed between 0 and 2π , $\omega_k = \pi\Delta\omega$, and $\omega_u = N\Delta\omega$ is the upper cut-off frequency. $G(\omega_k) = 2S(\omega_k)$ is the one-sided power spectrum. To generate an artificial earthquake, Shinozuka[10] suggested the following relationship between the intensity, S_n , and the peak ground acceleration, PGA . With $\sigma_g^2 = E[\ddot{x}_g^2] = \int S(\omega)d\omega =$

$S_o \pi \omega_g (1 + 4\xi_g^2) / 2\xi_g$, the peak ground acceleration can be written as $PGA = \alpha_g S_o^{0.2}$, where

$\alpha_g = p_g [\pi \omega_g (\frac{1}{2\xi_g} + 2\xi_g)]^{0.5}$, and p_g denotes the peak factor, empirically assumed to be 3.0 in this study.

Because of the random nature of earthquake acceleration histories, structure response quantities (such as damage indices) are more meaningful if formed as averages for an ensemble of responses, rather than responses to individual input functions. In a separate study, it was determined that at least ten sample functions are necessary to give useful mean responses [5].

4. Numerical Experiments

Three design parameters were singled out for their impact on frame response to strong ground motions : 1) the longitudinal steel ratio, 2) the confinement steel ratio, and 3) the depth of severely stressed frame members. This selection implies that sufficient shear reinforcement is provided to preclude shear failures. Also, bond failures due to cyclic loadings are not considered herein. Since the damage indices were of prime interest, the effect of a single design parameter was studied by changing only this one parameter in one member by a small amount and then plotting the resulting changes of all member damage indices. Such a plot can be interpreted as an influence surface.

4.1 Experimental Frame

A four-story three-bay concrete frame for a typical office building has been designed to serve as a model numerical experiments, Fig 3. This building has been designed according to the ACI 318-89 Code[1] to resist the equivalent static lateral loads specified in the Uniform Building Code[11]. The design base shear is given as: $V = \frac{ZIC}{R_w} W$, where in our case, $Z=0.4$ for seismic zone 4, $I=1.0$ for occupancy importance factor, $R_w=12$ for special moment-resisting space

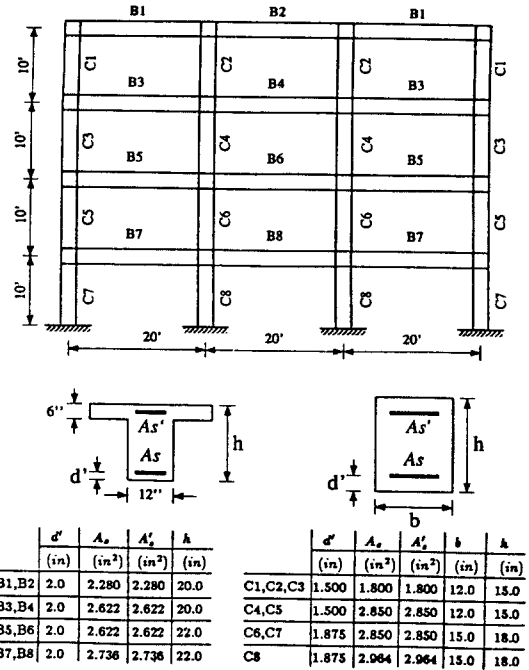


Fig 3. Details of Experimental Frame
(1 in = 2.54cm ; 1 kip = 4.448 kN)

frame, $C = 1.25S/T^{0.3} = 2.75$ for site coefficient and period, and $W = 656$ kips is the dead weight. The fundamental natural frequency and the natural period of this frame have been computed to be $f_1 = 1.15$ Hz and $T = 0.87$ sec respectively, using the moments of inertia of the cracked sections. The maximum roof displacement for the static code lateral loads is $\Delta = 2.23$ inch.

A mathematical model of this example frame was analyzed by program SARCF[6] for artificial ground acceleration histories, with peak values of 1.0g. Use of symmetry was made throughout by analyzing only one-half of the frame, in order to reduce the computational effort. Even though the axial forces in the columns vary considerably as functions of time, thereby affecting their yield moments, it was felt that the strong-column weak-beam design concept justifies this assumption of symmetry, since plastic hinges in columns were rare occurrences, except at the foundations[5]. The mean damage indices obtained for ten

sample input functions are summarized in Fig 4(a). Even for the earthquake with 1.0g peak acceleration, the damage indices are moderately small(maximum 0.281). This indicates a well-designed strong frame. Damage appears to be heavily concentrated in the beams of the lower stories, indicating that these contribute an overproportional share to the energy dissipation of the frame. It would be desirable to distribute this energy dissipation and resulting damage more evenly over the entire frame.

4.2 Design Parameter Study

All parameter studies reported below were carried out on the experimental frame of Fig 3. It should be recalled that we are concerned only with mean values of damage indices, so that each case implies ten nonlinear dynamic time history analyses. In each case a design parameter was increased by a certain amount and in a separate case decreased by an equal amount. Thus the longitudinal steel ratios were varied by $\pm 5\%$, the confinement steel ratios by $\pm 30\%$, and the member depths by $\pm 5\%$.

Even though the principle of linear superposition does not apply to the case of highly nonlinear frame response, the necessity for accelerating design convergence called for a separate investigation of this issue. This was accomplished by changing a design parameter, 1) in only the most heavily damaged member ; 2) in only the second most heavily damaged member ; 3) in both members together, to determine to what extent the influence surface can be superimposed. The degree of linearity of the cause-and-effect relationship was further studied by changing the longitudinal steel ratios by $\pm 10\%$, in addition to the earlier $\pm 5\%$.

All in all, well over one thousand analyses have been performed[5]. Herein, only the results of the study of the longitudinal steel ratio are given. Table 1 summarizes the complete matrix of influence coefficients, i.e. element a_{ij} is the change in damage index of element node i due to 5% increase of the flexural

steel in frame element j . These results permit the following observations: 1) Increasing the steel in any member consistently reduces its damage, with one exception. As a result, there are generally no positive numbers on the diagonal of the matrix. (As shown in Refs[5], this effect is consistently reversible, i.e. a reduction of member reinforcement almost always increases the damage in the modified member.) Since some members are not damaged, any additional reinforcement has no effect on their damage, i.e. the corresponding diagonal elements are zero. 2) Increasing the steel in any member has led by and large a beneficial effect on the other elements in the same story, as can be seen in the 4 by 4 diagonal submatrices. Small positive off-diagonal entries can be discounted as the effect of randomness and due to sensitivity to the time step size, Δt . There are some notable exceptions. For example, a steel increase in beam B3 increases the damage of columns C3 and C4. These results are less consistent in this regard than was observed in a similar earlier study[5]. 3) The net effect of adding reinforcement to a member is that the damage improvements exceed the damage increases. The one exception in beam B7, for which additional reinforcement increases the damage in most other members of the frame. It should be noted that beam B7 is one of the most heavily damaged members.

4.3 Conclusions from Numerical Experiments

The observations made in the numerical experiments lead to the following conclusions: 1) The amount of flexural reinforcement appears to be the most effective tool for influencing the damage distribution in a frame. The effect is almost consistent and diagonal-dominant (See Table 1). 2) The amount of confinement steel in zones of plastic deformations does not appear to influence the damage distribution to any useful extent. This finding is not completely as expected, and further studies should be undertaken before generalizing

it. 3) The effects of changes of member depths are often inconclusive. Thus, the member depths are not a very effective tool for a seismic damage-controlled design method and are typically subject to architectural constraints. 4) As far as the longitudinal steel is concerned, the principle of linear superposition holds in an overall sense. This applies to the magnitude of a particular change, the sign of the change, and the superposition of effects caused by changes in more than one frame element. 5) Although all of these conclusions are based on average results for 10 sample ground acceleration histories, some of the results are influenced by the randomness of the input, especially if the absolute values of the damage changes are small.

5. Seismic Damage-controlled Design Method

The key components of the procedure are an algorithm to evaluate the computed damage distribution by comparing it with user-specified acceptance criteria, and a set of design rules which permit the automatic modification of the structure such that improved performance is guaranteed[5].

5.1 Design Procedure

The Damage acceptance algorithm contains the following components: 1) damage in columns (D_{col} is unacceptable, as required by the strong-column weak-beam concept (except at the foundation); 2) mean value of all beam damage indices (\bar{D}_{beam}) shall not exceed a user-specified acceptance level (D_{beam}^{tar} , such as 0.3, with a small allowable tolerance (τ), such as ± 0.03); 3) damage index of any individual beam element (D_{beam}) shall not deviate from the mean value computed for all beams by more than a user-specified allowance (δ), such as 0.05.

$$\begin{aligned} D_{beam}^{tar} - \tau &\leq \bar{D}_{beam} \leq D_{beam}^{tar} + \tau \\ \left| D_{beam} - \bar{D}_{beam} \right| &\leq \delta \\ D_{col} &\leq D_{col}^{all} \end{aligned} \quad (3)$$

If the damage index of at least one frame member is unacceptable, corrective action has to be taken, i.e. the design will have to be modified such that an improved performance in a reanalysis is guaranteed and convergence towards an acceptable design is assured. The rules used in the automated design procedure are[5]:

1. For any beam element which showed an unacceptable level of damage in the preliminary analysis, the longitudinal steel will be increased (or decreased) by 5%, $\Delta A_s^i = 0.05 A_s \text{SIGN} [D_{beam} - D_{beam}^{tar}]$, where A_s is the original amount of steel, D_{beam} is the amount of damage determined in the preliminary analysis, and D_{beam}^{tar} is the target damage index of the beam element.

2. In a subsequent design iteration "i", the amount of steel in any beam with unacceptable damage is changed according to, $\Delta A_s^i = \Delta A_s^{i-1} \cdot \frac{D^i - D_{beam}^{tar}}{D^{i-1} - D^i}$, where ΔA_s^i denotes the increase (or decrease) of longitudinal steel for the element in question, ΔA_s^{i-1} denotes the steel added (or reduced) in the previous iteration, D^i and D^{i-1} represent the damage values in the (i)th and (i-1)th iteration, respectively.

3. To adhere the strong-column weak-beam concept, any column with unacceptable damage has to satisfy the requirement, $M_v^{col} \geq 1.25 M_v^{beam}$, where M_v^{col} is the yield moment of the column considered, and M_v^{beam} is the yield moment of the beam framing into the same joint[5]. Then, the reinforcing steel of each column will be linearly increased (or decreased) by the amount, $\Delta A_s^i = \Delta A_s^{col} \cdot \frac{M_v^i - M_v^{i-1}}{\Delta M_v^{col}}$, where the superscript indicates the iteration number, ΔM_v^{col} denotes the increment of the yield moment of the column when the longitudinal steel of the corresponding column is increased by ΔA_s^{col} .

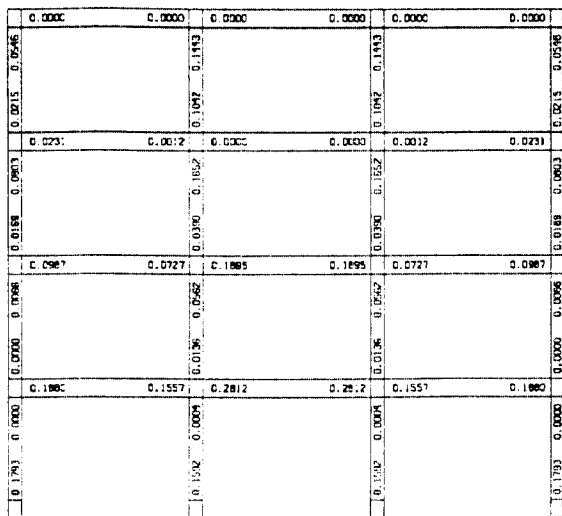
4. At any section of an element, the longitudinal steel ratio ρ shall satisfy the maximum and minimum limits specified by the ACI 318-89 Code.

5.2 Demonstration Example

Providing program SARCF with a target mean damage value of 0.1 together with a tolerance allowance of 0.05 and a maximum deviation of 0.05, the example office building of Fig 3 was subjected to 12 automatic design iterations. According to the results summarized in Fig 4(b), all damage acceptance criteria for the modified frame are satisfied.

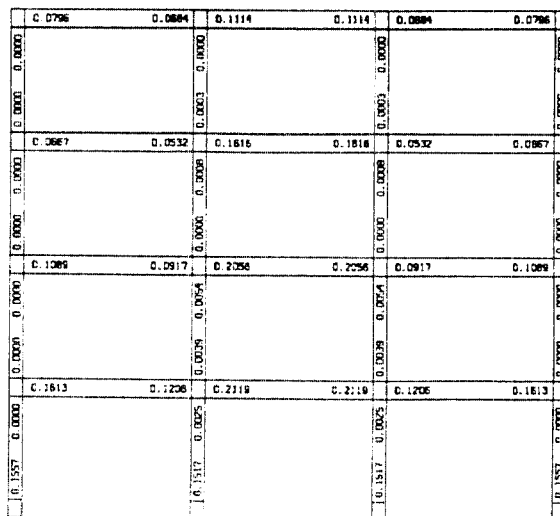
The usefulness of this design method will be illustrated by demonstrating how the seismic capacity

is enhanced when the frame is redesigned. The sensitivity of this enhancement to structural properties and input earthquake characteristics will be discussed using 3-bay 4-story example frame subjected to three different kinds of earthquake waves. Both the original and the revised frames were analyzed for their responses to the north-south component of the E1Centro earthquake, the Taft earthquake and the Hachinohe earthquake, with varying peak ground accelerations. The structural damage index proposed by DiPasquale and Cakmak[7], $D_k = 1 - \frac{T_d}{T_d}$, was computed and plotted



Mean Damage of All Beams : $\bar{D} = 0.084$
 Standard Deviation of All Beams : $\sigma_D = 0.094$

(a) Preliminary Design Frame



Mean Damage of All Beams : $\bar{D} = 0.120$
 Standard Deviation of All Beams : $\sigma_D = 0.051$

(b) Modified Design Frame

Fig 4. Mean Damage Indices for Demonstration Frame

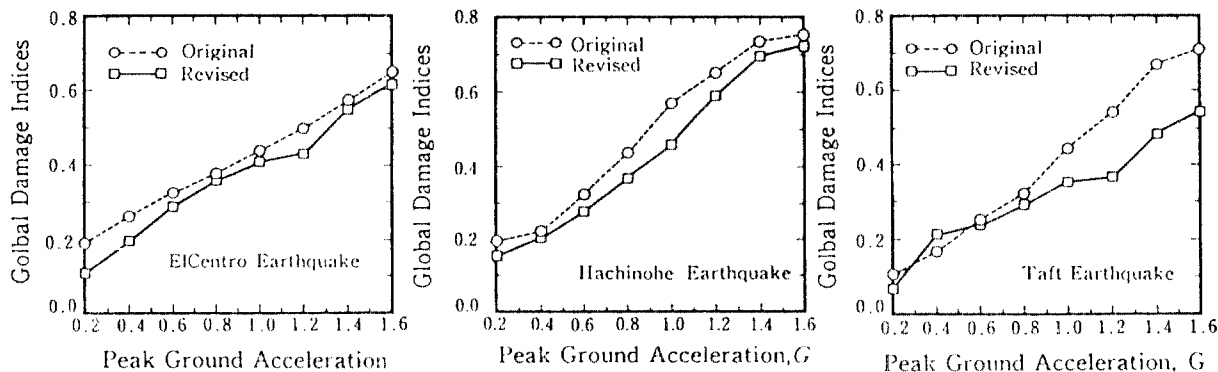


Fig 5. Comparison of Structural Damage Indices

Table 1. Changes of D_e Due to 5% Increase of Reinforcement, (1.0g Peak Acceleration)

unit: $\times 10^4$

Cause Effect	Reference D_e (See Fig 4(a))	Top Story				3rd Story				2nd Story				1st Story			
		B1	B2	C1	C2	B3	B4	C3	C4	B5	B6	C5	C6	B7	B8	C7	C8
B1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
B2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
C1	546	27	-37	-143	-71	-32	10	-24	30	-15	9	-10	-4	84	24	9	30
	215	8	1	-21	4	-20	-20	-29	-49	-13	123	-4	-4	-2	2	-9	-9
C2	1443	-83	-130	-44	-185	-57	15	-73	-59	-117	45	-32	-100	21	-94	-38	-54
	1042	-60	-149	-129	-148	-7	49	-140	-22	-98	-22	-12	-83	52	-86	-80	-118
B3	231	-32	-23	-35	-68	-64	0	-23	-14	-19	-1	-1	-17	1	-32	-21	-31
	12	-4	3	4	-7	-10	-7	3	5	1	-4	-6	2	9	1	-1	1
B4	0	0	0	0	2	0	0	3	22	0	0	1	0	9	1	0	2
C3	803	-11	-37	-34	108	217	-41	5	32	-31	37	109	91	65	-58	73	-19
	169	-15	29	-13	41	41	49	25	4	36	44	30	43	61	-10	-10	-29
C4	1652	-113	-143	-131	-53	16	-39	-151	-148	-263	-181	-24	54	192	-109	-41	-189
	390	-40	-68	-28	-6	23	-19	-78	-65	-36	21	5	-34	23	-42	-56	-70
B5	987	-14	3	-35	-8	-25	-19	-72	-58	-221	-109	4	-31	108	11	2	37
	727	26	41	19	70	51	7	3	16	-121	-63	4	89	112	59	66	59
B6	1895	60	71	32	41	-14	36	36	36	-336	-300	5	127	137	44	103	98
C5	66	-7	-12	-8	-8	-7	-10	9	18	37	5	-45	-13	-22	-16	-5	-16
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
C6	562	-24	-55	-23	-13	-70	-42	-19	-28	62	63	-55	-195	-95	-79	-92	-93
	136	-4	19	-20	-3	-12	-22	-31	-22	-35	-32	11	-30	19	-1	18	-8
B7	1880	25	-53	-57	-21	-44	-27	-5	-11	30	8	25	-89	-136	-46	-9	-118
	1557	-62	37	0	-27	-54	-23	46	15	39	77	-24	-69	-44	25	-87	-41
B8	2812	-29	-3	-76	-4	-33	-16	46	22	50	25	-30	-114	-111	-105	-37	-63
C7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	1793	-21	-35	41	1	-24	5	15	32	15	45	-9	-54	-117	-73	-105	-70
C8	4	1	-1	-3	-3	-3	-3	-2	-2	4	1	-3	4	5	0	0	-4
	1592	-18	79	18	36	-21	-24	122	53	119	52	-23	82	59	80	66	-19

* For member designations, see Fig 3.

Table 2. Acceptable Earthquake Intensity For 3-Bay 4-Story Frame when $D_e^{max}=0.5$

EL Centro Earthquake		Taft Earthquake		Hachinohe Earthquake	
Original	Revised	Original	Revised	Original	Revised
0.86 g	1.04 g	0.93 g	1.06 g	0.77 g	1.03 g

in Fig 5. Herein, T_1 is the fundamental period of the undamaged structure and T_d is the maximum value of the fundamental period during the nonlinear behavior of the structure. As can be seen, the seismic damage-controlled design method reduces damage as measured by this structural damage index, even though the mean beam damage value was increased from 0.084 to 0.120.

As shown in Table 2, meanwhile, the acceptable earthquake intensity of the original 3-bay 4-story frame was 0.86g and that of the revised frame was 1.04g under El Centro N-S earthquake when the damage value in one member reached a specified maximum value, $D_e^{max}=0.5$ for this study. This result, 14% stronger ground motion was acceptable for the revised frame, shows that this damage-controlled design method is an effective tool for improving the seismic safety of RC frame.

6. Conclusions

The main objective of this study was to propose a seismic damage-controlled design method for reinforced concrete frame buildings, which aim at an uniform damage distribution throughout the building frame, as measured by a member damage indices. The uniformity of damage distribution is thought to be a desirable goal of earthquake resistant design. Concentration of heavy damage in some vulnerable structural members, which has led to many collapses in recent earthquakes, are avoided. It is also felt that by keeping the damage in a frame uniform, an optimum response to an earthquake of given intensity is achieved.

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