# Seismic Safety Enhancement of Damage-Controlled Reinforced Concrete Frames

## 손상제어 설계된 철근 콘크리트 프레임의 내진력 향상

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## Abstract

Conventional aseismic design methods of R/C frame all but disregard the state of damage over the entire building frame. This paper presents an automated damage-controlled design method for R/C frames which aims at an uniform energy dissipation rate throughout the building frame, so that the resulting damage is uniformly distributed as much as possible over all element. The accuracy of the basic hystertic model and the damage model for R/C members is verified by reproducing the experimental load-deformation curves of one-bay one-story frames. Application of this design method to various frame structures indicate that 1) regardless of the structural properties or input earthquake characteristics, damage-controlled frames generally survive more severe earthquake excitations and suffer less damage

## 요 약

본 논문은 지진발생시 빌딩프레임상의 각 부재의 흡수에너지율이 일정하도록 유도하는 다시말하면 각각의 부재의 손상지수값이 고루게 분포토록 하는 새로운 설계법인 손상제어 전산설계법의 유용성을 입증코져 하는 것이다. 이를 위하여 우선 사용된 기본적인 이력모델 및 손상모델의 정확성을 평가하기 위하여 one-bay one-story 프레임의 실험적인 하중－변위곡선을 해석적으로 재생하여 비교분석하였다. 그리고 본 설계법을 각종 프레임에 적용한 결과 1) 구조물의 특징 및 사용된 지진 형상에 관계 없이 손상제어 설계된 프레임은 일반적으로 종래의 방법으로 설계된 프레임보다 같은 지진하중하에서 더 작은 손상값으로 저항하였으며, 2) 손상제어 설계된 프레임의 하층부 부재들은 더 큰 항복강도를 나타내는 현상을 보였으며 상층부 부재들은 반대 현상을 보였다.

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than conventionally designed frames, and 2) member yielding strength in the lower stories of damage-controlled frames is larger than that for conventionally designed frames, while the trend is opposite in the upper stories.

## 1. INTRODUCTION

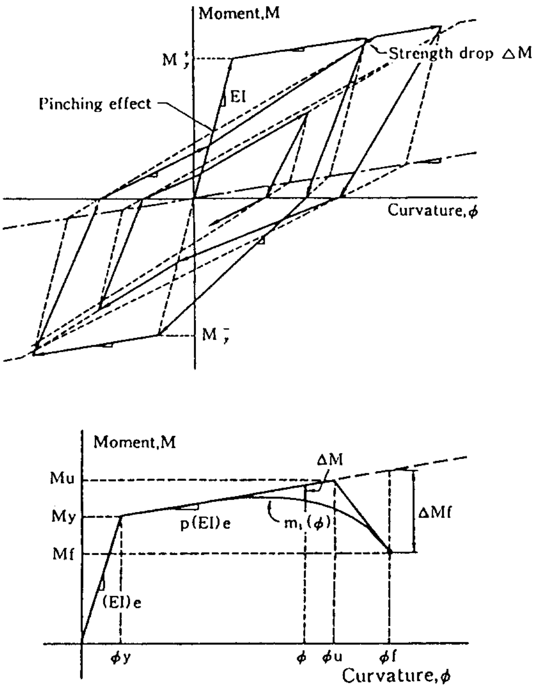
Current aseismic design philosophy relies strongly on the energy dissipation in structural components undergoing large inelastic deformation. Thus, it is commonly recognized that the seismic safety of R/C frame structures depend on their total energy dissipation capacity. To utilize the full energy dissipation capacity of a R/C frame structure, 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. However, most of the conventional building design codes, based on the weak-beam strong-column concept, do conventional building design codes, based on the weak-beam strong-column concept, do not necessarily ensue such an equal energy distribution level. In this respect, energy dissipation level is interpreted as synonymous with damage level.

A computer code, SARCF[3]. which automatically redesigns a R/C frame designed initially in accordance with a standard design code, has been developed by Chung, Meyer and Shinozuka [1]. A basic hysteretic model and a damage model for R/C members which play an important role in this study, will be further verified by reproducing experimental load-deformation curves of one-bay one-story frames.

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 three different frames subjected to three different kinds of earthquake waves.

## 2. HYSTERETIC AND DAMAGE MODEL

An earlier study [2] reviewed part models and proposed a new hysteretic model and a damage model for R/C flexural members. The hysteretic model proposed by Roufaiel and Meyer[8], which has a simple bilinear skeleton, has been refined to take into account stiffness degradation under cyclic loadings, strength deterioraton after yielding and the pinching effect. These factors are unique to the R/C members, Fig. 1.



$\Delta M$ =Strength drop in one cycle at curvature $\phi$

$\phi_y$ =Yield curvature

$\phi_f$ =Failure curvature

Fig. 1 Hysteretic Model for R/C Flexural Members

Many researchers express eseismic damage of R/C membersin terms of a damage index, $D_e$ which remains zero as long as the response remains in the elastic zone and indicates the "failure" when it reacher 1.0[7]. The damage index is expressed in the form of a modidied Miner's Rule, Eq(1). It contains damage modifiers, which reflect the effect of the loading history, and it considers the fact that reinforced concrete members typically respond differently to positive and negative loadings:

$$D_e=\sum_i\left( \alpha_i^+ \frac{n_i^+}{N_i^+} + \alpha_i^+ \frac{n_i^+}{N_i^+} \right) \tag{1}$$

where $N_i$ is the number of cycles to cause failure at curvature $\phi_i$, $n_i$ is th number of actually applied loading cycles at curvature $\phi_i$, $\alpha_i$ is the damage modifier and $+/-$ shows loading direction.

The damage mdifier, $\alpha$, is defined as a function of the number of loading cycles and the previous loading history, Fig. 2. 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.

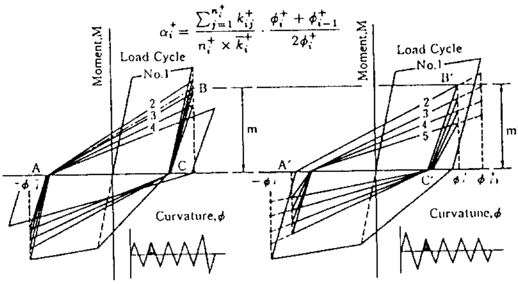


Fig. 2 Damage Modifier, $\alpha$

## 3. VERIFICATION OF HYSTERETIC AND DAMAGE MODEL

The verify the accuracy of the hysteretic model and the damage model, experimental load-deformation curves of one-bay one-story R/C frames performed by Ohno, Fujikake and Nishioka[5] have been simulted. The results show that the envelope curves of story shear forces, Fig. 3 and the cumulative dissipated energy, Fig. 4, which plays the most important role in this study, have generally been in good agreement.

When this seismic design method is utilized, the acceptable damage values must be specified based only the $D_e$ values. To relate the $D_e$ values and physical damage of R/C members,

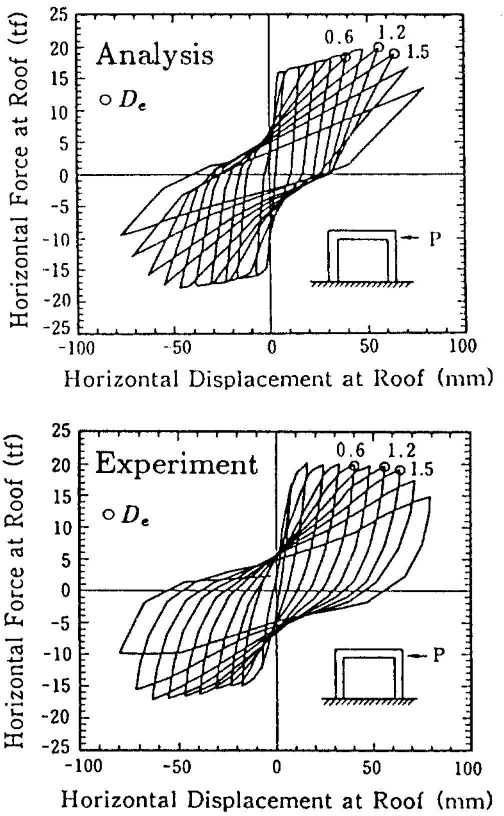


Fig. 3 Experimental and Analytic Load-Deformation Curves

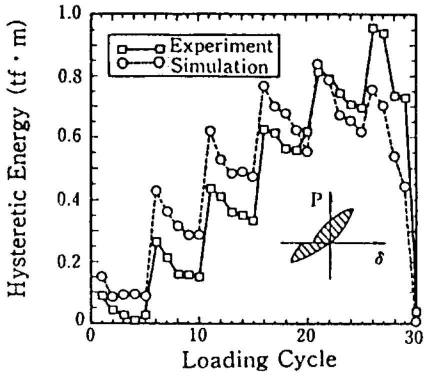


Fig. 4 Cumulative Hysteretic Energy for R/C Flexural Members

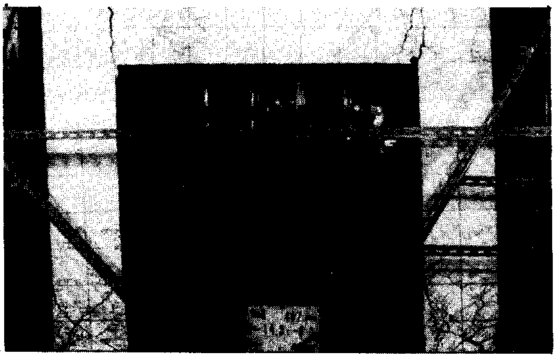


Fig. 5 Physical Damage($D_e^{max}$=(1.2)

Table 1. Damage Index and Physical Damage

| $D_e$ | Physical Damage | Hysteresis | Description |
|---|---|---|---|
| 0.02-0.2 | Invisible Cracking | Stable Hysteresis | Minor Damage |
| 0.2-0.5 | Visible Cracking | Small Strength Drop | Repairable |
| 0.5-1.0 | Concrete Spalling | Progressive Strength Drop | Inrrepairable |
| 1.0 ≤ | Concrete Crushing | Significant Strength Drop | Structure Unsafe |

a numerical calibration was made by comparing the visible damage of one-bay one-story frames and simulated $D_e$ values at the same loadign step, Fig. 5. The result of this calibration is summarized in Table 1. From the engineering point of view, to prevent a total collapse, the $D_e$ in all members should be limited to a maximum value of 0.5. For a credible earthquake, which is expected to strike the structure several times in its service life, $D_e=0.2$ is the maximum acceptable damage to maintain the structure's serviceability with only monir repair.

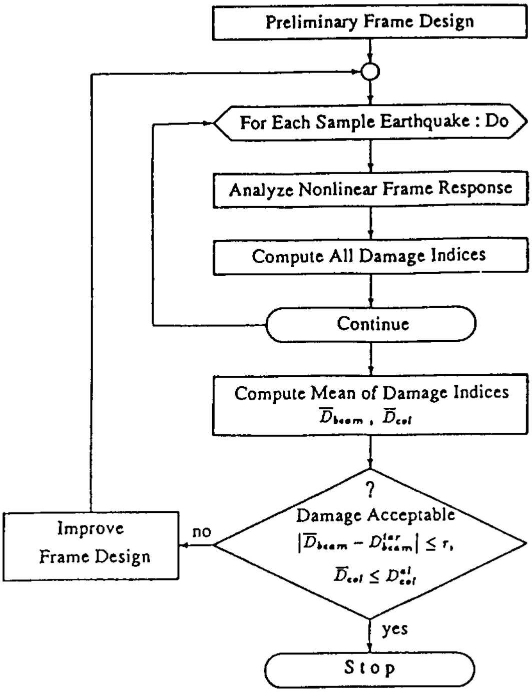


Fig. 6 Damage-Controlled Design Method

## 4. DAMAGE-CONTROLLED DESIGN METHOD

This design procedure which aims at an uniform damage distribution over a structure is summarize in Fig. 6. First, an original design based on appropriate building code, is performed by the structural engineer. The remaining processes are then performed automatically by the computer.

The dynamic analysis part of this program is based on DRAIN-2D[6] with the proposed hysteresis model. Ten artificial earthquake waves with a peak ground acceleration of 1.0 g were generated. The well known Kanai-Tajumi spectral and trapezoidal envelope function were employed, Fig. 7. An early study [1] has shown that at least ten sample functions are required to get an useful mean response.

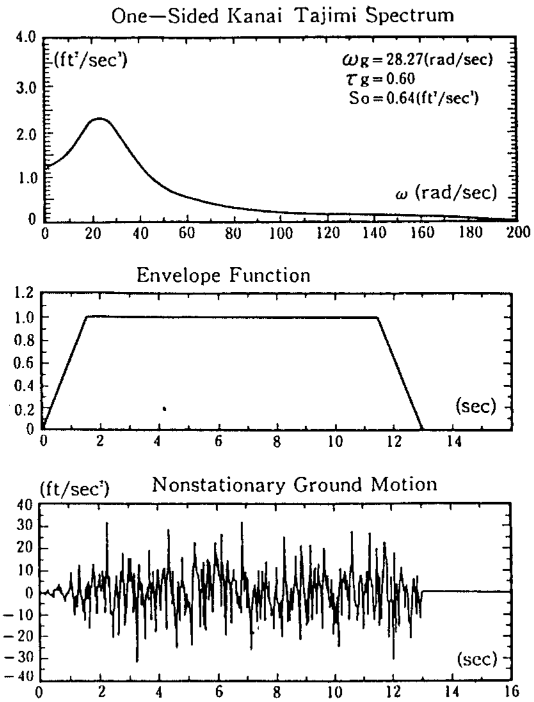


Fig. 7 Generation of Ground Acceleration

### 4.1 Design Criteria

The design critera employed in this method are summarized as follows:

1. Large damage(or plastic hinges) in columns is unacceptable, as required by the strong-column weak-beam concept. Only slight damage, for example $D_{al}^{col}=0.01$ for this study, is allowed in all columns, except at the base of the first story and at the top of the top story. The reasons for these exceptions ase that a) it is difficult to arrive at reasonable designs without allowing some degres of damage at the base of first story columns and b) the overall safety of buildings is rather insensitive to the degree of damage at th top of top story columns.
2. The mean value of any beam damage index $\overline{D}_{beam}$ shall not deviate from a user specified target value $D_{tar}^{beam}$ by a user specified tolerance $\tau$.

### 4.2 Damage-Controlled Design Procedure

If the damage index of at least one frame member is unacceptable, corrective action is taken, i.e. the design is modified toward the user specified damage distribution. From the results of a large number of numerical studies, it is found that the longitudinal steel ratio is the dominant parameter for damage of R/C members, and that other parameters such as the confinement steel ratio, concrete strength and member dimensions relatively have no significant effect on member damage[1]. Herein, only the longitudinal steel amount of members with unacceptable damage is changed in this design method as follows:

1. For any frame element which exhibits a smaller(or larger) level of damage than specified, the longitudinal steel is decreased(or increased).
2. To adhere to the strong-column weak-beam concept, all columns must satisfy the requirement, $M_y^{col} \geq 1.25 \times M_y^{beam}$, where $M_{col}$ is the sum of the absolute yield moments of the columns considered, and $M_{beam}$is the sum of the absolute yield mements of all beams framing into the same joint.
3. At any section of an element, the longitudinal steel ratio $\rho$ shall meet the provision in the ACI 318−89 Code[9] to provide sufficient ductility.
4. The amount of steel of an element exhibiting an unacceptable level of damage is increased (or decreased) without taking into account the effect of such an increase (or decrease) on the state of damage in other element.

### 4.3 Result of Damage-Controlled Design Method

Three R/C frames, a two-bay three-story frame, a three-bay four-story frame and a three-bay ten-story frame, were originally designed based on the static lateral load requirement for UBC Zone 4 (Table 2). These frames were automatically redesigned until the userspecified damage distribution was satisfied. In this study $D_{tal}^{beam}=0.1$ and $\tau=0.05$ were adopted for the two-bay three-story and the three-bay four-story frames. For the three-bay ten-story frame, $D_{tar}^{beam}=0.05$ $\tau=0.02$ were adopted. The acceptable damage for the columns in all three frames, $D_{al}^{col}$, was 0.01.

Table 2. Prperties of Demonstration Frames

| Frame | Height (m) | Bay Width (m) | Fundamental Freq.(Hz) |
|---|---|---|---|
| 2-bay 3-story | 8.2 | 6.1 | 1.38 |
| 3-bay 4-story | 12.2 | 6.1 | 1.14 |
| 3-bay 10-story | 30.5 | 6.1 | 0.48 |

After 12 design iterations for three-bay four-story R/C frame, the user-specified damage distribution is obtained. Some satistical damage values for both the original and revised fromes of three cases are shown in Table 3. In Table 3, $D_{max}^{beam}$, $D_{min}^{beam}$, $\overline{D}_{beam}$ and $\delta_{beam}$ denote the maiximum, minimum, mean and standard deviation of beam damage index, respectively, and $D_{max}^{col}$ indicaes the maximum value of column damage index. Fig. 8 shows the damage variance in a typical member under ten artificial earthquakes. The damage-controlled design method has also reduced the damage variance in each member for the ten artificial earthquakes, as well as the damage variance over the entire frame.

Table 3. Damage Comparision of Both Original and Revised Frames

| Frame | $D_{beam}^{max}$ |  | $D_{beam}$ |  | $\delta_{beam}$ |  | $D_{col}^{max}$ |  | Iteration |
|---|---|---|---|---|---|---|---|---|---|
|  | Orig. | Rev. | Orig. | Rev. | Orig. | Rev. | Orig. | Rev. | Numbers |
| 2-Bay | 0.3047 | 0.1230 | 0.121 | 0.110 | 0.121 | 0.014 | 0.141 | 0.009 | 8 |
| 3-Story | 0.0 | 0.0815 | 0.121 | 0.110 | 0.121 | 0.014 | 0.141 | 0.009 | 8 |
| 3-Bay | 0.2577 | 0.1396 | 0.079 | 0.089 | 0.084 | 0.032 | 0.160 | 0.005 | 12 |
| 4-Story | 0.0 | 0.0543 | 0.079 | 0.089 | 0.084 | 0.032 | 0.160 | 0.005 | 12 |
| 3-Bay | 0.0911 | 0.0796 | 0.022 | 0.055 | 0.031 | 0.018 | 0.139 | 0.008 | 24 |
| 10-Story | 0.0 | 0.0231 | 0.022 | 0.055 | 0.031 | 0.018 | 0.139 | 0.008 | 24 |

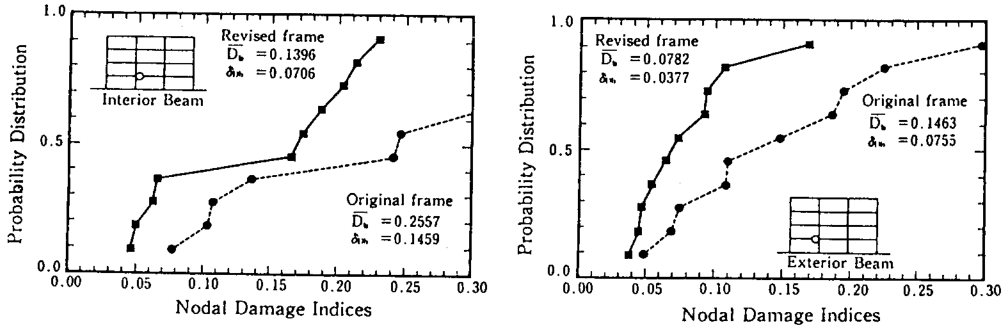


Fig. 8 Member Damage Variance

Fig. 9 shows the member yielding strength distribution (sum of the beam yielding moment in one story) of both the original and revised frames. The required member yielding strength in the lower stories of the improved frame is larger than that for the original frame, while the trend is opposite in the upper stories. Table 4 lists the amount of reinforcing steel, comparing the values required for both the original and the revised frames. To assure the strong-column weak-beam failure mode, at large increase inc-

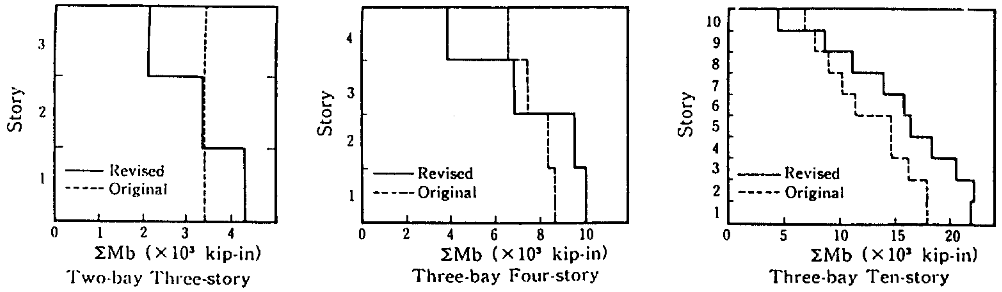


Fig. 9 Member Yielding Strength Distribution

Table 4. Amount of Reinforcing Steel ㎠

| Frame | 2-Bay3-Story | | 3-Bay4-Story | | 3-Bay10-Story | |
|---|---|---|---|---|---|---|
| Member | Original | Revised | Original | Revised | Original | Reivised |
| Beam | 83066 | 80116 | 242102 | 245970 | 949565 | 1162302 |
| Column | 80919 | 90276 | 155480 | 230533 | 519470 | 884869 |
| Total | 163985 | 170392 | 397582 | 476503 | 1469035 | 2047170 |

lumn reinforcement was necessary. However, the total increase in reinforcement was relatively small.

## 5. SEISMIC SAFETY IMPROVEMENT OF DAMAGE-CONTROLLED FRAMES

The usefulness of this design method will be shown by demonstrating 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 larger capacity enabling stronger earthquake to be accommodated.

### 5.1 Global Damage Index

Both the original and the revised frames were analyzed for their reponses to the northsouth component of the El Centro earthquake, the Taft earthquake and the Hachinohe earthquake, with varying peak ground accelerations. The global damage in index proposed by DiPasquale and Cakmak[4], which is a measure of the softening of the structure in terms of first natural period, was used to estimate the structural damage.

$$D_g = 1 - \frac{T_i}{T_d} \qquad (2)$$

where $T_g$ is the natural period of the undamaged frame and $T_d$ is the natural period of the damaged frame.

Fig. 10 shows the global damage indices of three-bay four-story frame to the El Centro, Taft and Hachinohe earthquakes, respectively. As can be seen, this automated damage-controlled design method reduced damage as measured by the global damage index, even though the mean beam damage value was increased from 0.079 t 0.089, as shown in Table 3.

### 5.2 Acceptable Earthquake Intensity

Earthquake intensities applied to all the frames were progressively scaled up until the damage value in one member reached a specified miximum value (in this study $D_e^{max} = 0.5$). The peak ground acceleration at this intensity is defined as a measure of acceptable earthquake intensity.

As shown in Table 5, the acceptable earthquake intensity of the original three-bay four-story frame was 0.86 g and that of the revised frame was 1.04 g under El Centro Earthquake. 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 R/C

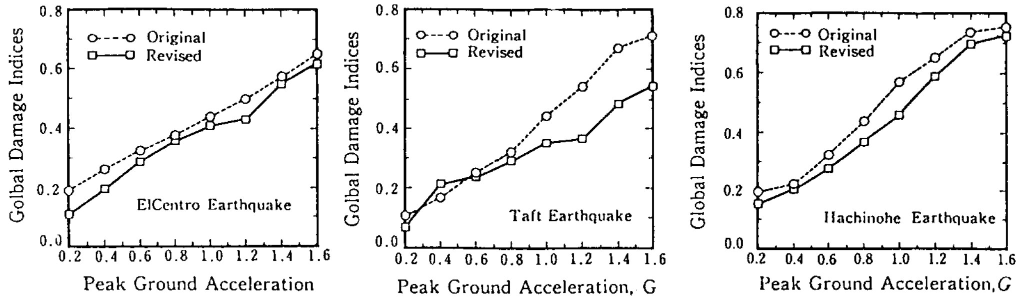


Fig. 10 Global Damage Index vs Peak Ground Acceleration for Three-Bay Four-Story Frame

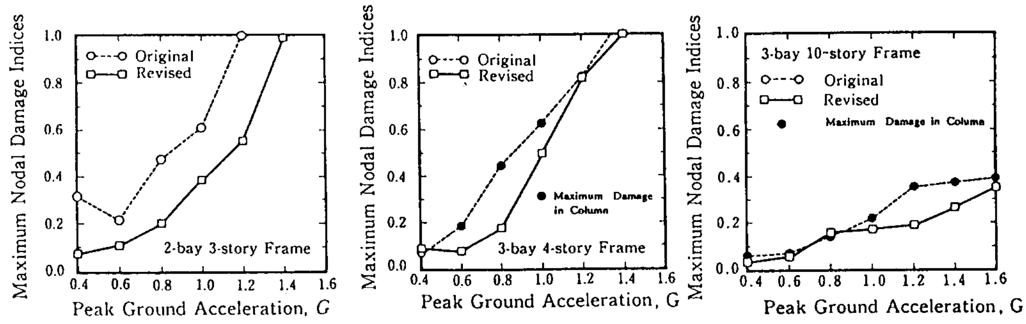


Fig. 11 Maximum Member Damage Index vs Peak Ground Acceleration

Table 5. Acceptable Earthquake Intensity

unit : g

| Frame | EL Centro Earthq. | | Taft Earthq. | | Hachinohe Earthq. | |
|---|---|---|---|---|---|---|
| | Original | Revised | Original | Revised | Original | Revised |
| 2-Bay 3-Story | 0.84 | 1.13 | 1.00 | 1.21 | 0.76 | 0.96 |
| 3-Bay 4-Story | 0.86 | 1.04 | 0.93 | 1.06 | 0.77 | 1.03 |
| 3-Bay 10-Story | 1.75 | 1.94 | 0.93 | 1.27 | 0.80 | 0.83 |

frames. Application of this design method to various structures confirms that the damage-controlled design method is effective regardless of the structural properties and earthquake characteristics.

### 5.3 Maximum Member Damage

Firure 11 shows the relation between the maximum member damage index and the peak ground acceleration for the three different frames under the El Centro earthquake. The maximum damage values of the revised frames ar generally smaller than those of the original frames and are observed in a beam. This damage-controlled design method not only reduces the maximum damage, but also prevents the undesirable column failure mode.

## 6. CONCLUSION

After certain numbers of design iteration, the damage-controlled design method has reduced the damage variance in each member as well as the damage variance over the entire frame, as expected. The effectiveness of this design method was verified by demonstrating with various frame structures that, on the one hand, the method reduced the damage as measured by the global damage index under the same earthquake and, on the other hand, led to a larger capacity enabling stronger earthquakes to be accomodated. Thus, a newly proposed design method which aims at an uniform damage distribution under severe earthquake, has been generally found to improve the seismic safety of R/C frame structures regardless of structural properties and earthquake characteristics.

## 7. ACKNOWLEDGEMENTS

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