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Automated Damage-Controlled Desingn Method of Reinforced Concrete Frames
철근 콘크리트 프레임의 손상제어 전산설계법

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Abstract

Conventional aseismic design methods of reinforced concrete frame all but disregard the state of damage over the entire building frame. This paper presents an automated damage-contorlled design method, which aims for uniform damage distribution throughout the entire building frame, as measured by the individual member damage indexes. 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 responce to an earthquake. The usefulness of this design method is demonstrated with a four story example office building predicting the extent of structural damage.

1. Introduction

Current aseismic design philosophy relies strongly on energy dissipation in structural components undergoing large inelastic deformations. There are basically two approaches to achieve this goal: 1) to introduce deliberate weak spots in the structure assigned to develop plastic hinges and to dissipate energy under tightly controlled conditions, and 2) uniformly distribute the resulting damage over the entire frame, thereby keeping the damage down to a minimum average value. It is the second design approach which is followed in the automated damage-controlled design procedure for reinforced concrete frames described herein. A preliminary design is modified iteratively until the damage has reached a preselected uniform distribution. As can be seen Fig 1, the general 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 [10]; 2) perform a nonlinear dynamic analysis of the frame for seismic ground shaking of specified intensity, duration and spectral content, using program SARCF[5]; 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.

The member model and damage index used in this study were described in detail elsewhere [2,3]. 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 [2], 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

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and stiffness of damaged structural members and thus permit an acceptably accurate prediction of response to subsequent cyclic loading. The damage index D_e 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 [3]. 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:

$$D_e = \Sigma_i \left(\alpha_i^+ \frac{n_i^+}{N_i^+} + \alpha_i^- \frac{n_i^-}{N_i^-} \right) \qquad (1)$$

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. $(M_i - M_{fi})$ and ΔM_i denote the strength drops at curvature level i, up to the failure moment and in a single load cycle, respectively [3].

The loading history effect is captured by including the damage modifier α_i, which, for positive moment loading, is defined as

$$\begin{aligned} \alpha_i^+ &= \frac{\frac{1}{n_i^+}\sum_{j=1}^{n_i^+} k_{ij}^+}{\overline{k_i}^+} \cdot \frac{\phi_i^+ + \phi_{i-1}^+}{2\phi_i^+} \\ &= \frac{M_{i1}^+ - \frac{1}{2}(n_i^+ - 1)\Delta M_i^+}{M_{i1}^+ - \frac{1}{2}(N_i^+ - 1)\Delta M_i^+} \cdot \frac{\phi_i^+ + \phi_{i-1}^+}{2\phi_i^+} \end{aligned} \qquad (2)$$

where $k_{ij}^+ = M_{ij}^+/\phi_i^+$ is the stiffness during the j-th cycle up to curvature level i, $\overline{k_i}^+ = \frac{1}{N_i^+}\sum_{j=1}^{N_i^+} k_{ij}^+$ is the average stiffness during N_i^+ cycles up to curvature level i, and $M_{ij}^+ = M_{i1}^+ - (j-1)\Delta M_i^+$ is the moment reached after j cycles up to curvature level i [3].

3. Generation of Artificial Earthquakes

As a nonstationary random process, artificial earthquake ground motions, $\ddot{x}(t)$, 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 function, $S(\omega) = S_o \cdot \frac{1+4\zeta_g^2\left(\frac{\omega}{\omega_g}\right)^2}{\left[1-\left(\frac{\omega}{\omega_g}\right)^2\right]^2 + 4\zeta_g^2\left(\frac{\omega}{\omega_g}\right)^2}$, where ω_g is the characteristic ground frequency, ζ_g is the predominant damping coefficient, S_o 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 [8],

$$g(t) = \sqrt{2}\sum_{k=1}^{N} \sqrt{G(\omega_k)\Delta\omega} \cdot cos(\omega_k t - \phi_k) \qquad (3)$$

where ϕ_k is the random phase angle, uniformly distributed between 0 and 2π, $\omega_k = k\Delta\omega$, 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 [9] suggested the following relationship between the intensity, S_o, and the peak ground acceleration, PGA. With $\sigma_g^2 = E\left[\ddot{x}_g^2\right] = \int S(\omega)d\omega = S_o\pi\omega_g\left(1+4\zeta_g^2\right)/2\zeta_g$, the peak ground acceleration can be written as $PGA = \alpha_g S_o^{\frac{1}{2}}$, where $\alpha_g = p_g\left[\pi\omega_g\left(\frac{1}{2\zeta_g} + 2\zeta_g\right)\right]^{0.5}$, 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 reponses to individual input functions. In a seperate study, it was determined that at least ten sample functions are necessary to give useful mean responses [4].

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 Example Office Building

A four-story three-bay concrete frame for a typical office building has been designed to serve as a model for numerical experiments, Fig 2. 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 [10]. 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 frame, $C = 1.25S/T^{2/3} = 2.75$ 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 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 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 [4]. The mean damage indices obtained for ten sample input functions are summarized in Fig 3(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 example building frame of Fig 2. 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 ± 5%, the confinement steel ratios by ± 30%, and the member depths by ± 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 surfaces can be superimposed. The degree of linearity of the cause-and-effect relationship was further studied by changing the longitudinal steel ratios by ± 10%, in addition to the earlier ± 5%.

All in all, well over one thousand analyses have been performed [4]. 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 [4], 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 [4]. 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. 3) The effects of changes of member depths are often inconclusive. Thus, the member depths are not a very effective tool for an automated 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. Automated Damage-controlled Design

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 [4].

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 ($\overline{D}_{beam}$) shall not exceed a user-specified acceptance level (D^{tar}_{beam}), 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.

$$\begin{gathered} D^{tar}_{beam} - \tau \leq \overline{D}_{beam} \leq D^{tar}_{beam} + \tau \\ \left| D_{beam} - \overline{D}_{beam} \right| \leq \delta \\ D_{col} \leq D^{all}_{col} \end{gathered} \tag{4}$$

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 [4]:

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%,

$$\Delta A^1_s = 0.05 A_s SIGN \left[D_{beam} - D^{tar}_{beam} \right] \tag{5}$$

where A_s is the original amount of steel, D_{beam} is the amount of damage determined in the preliminary analysis, and D^{tar}_{beam} is the target damage index of the beam element. The steel increments (or reductions) of Eq (5) are only trial amounts to determine in a first design iteration the influence of these changes.

2) In a subsequent design iteration "i", the amount of steel in any beam with unacceptable damage is changed according to,

$$\Delta A^i_s = \Delta A^{i-1}_s \cdot \frac{D^i - D^{tar}_{beam}}{D^{i-1} - D^i} \tag{6}$$

where ΔA^i_s denotes the increase (or decrease) of longitudinal steel for the element in question, ΔA^{i-1}_s 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, $M^{col}_y \geq 1.25 M^{beam}_y$, where M^{col}_y is the yield moment of the column considered, and M^{beam}_y is the yield moment of the beam framing into the same joint [4], Then, the reinforcing steel of each column will be linearly increased (or decreased) by the amount,

$$\Delta A^i_s = \Delta A^{col}_s \cdot \frac{M^i_y - M^{i-1}_y}{\Delta M^{col}_y} \tag{7}$$

where the superscript indicates the iteration number. ΔM^{col}_y denotes the increment of the yield moment of the column when the longitudinal steel of the corresponding column is increased by ΔA^{col}_s.

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 2 was subjected to 12 automatic design iterations. According to the results summarized in Fig 3(b), all damage acceptance criteria for the modified frame are satisfied.

Analyzing both the original and improved frames for the North-South component of the 1940 El Centro Earthquake with varying scale factors, the structural damage index proposed by DiPasquale and Cakmak [6], $D_g = 1 - \frac{T_o^{initial}}{T_o^{max}}$, was computed and summarized in Table 2 and plotted in Fig 4. Herein, $T_o^{initial}$ is the fundamental period of the undamaged structure and T_o^{max} is the maximum value of the fundamantal period during the nonlinear behavior of the structure. As can be seen, the automated 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. This example demonstrates that this automated damage-controlled design method is effective in achieving not only a relatively uniform distribution of damage over the entire frame, but also an increase of frame reliability, while only moderately changing the main member reinforcement [4].

6. Conclusions

The main objective of this study was to propose an automated damage-controlled design method for reinforced concrete frame buildings subjected to strong earthquake ground motions. The design methodology consists of six main components: 1) an accurate mathematical model of RC frame elements; 2) an objective measure of damage of RC members; 3) a nonlinear frame analysis program; 4) a mathematical model of the earthquake ground motion as a random process; 5) an algorithm to evaluate the damage predicted for a given frame design and earthquake intensity; 6) a set of design rules for the automatic modification of the structure.

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.

References

1) ACI Committee 318-89, "Building Code Re-

2) Chung, Y. S., Meyer, C. and Shinozuka, M., "Seismic Damage Assessment of Reinforced Concrete Members," Report No. NCEER-87-22, NCEER, Buffalo, NY, Oct. 1987.

3) Chung, Y. S., Meyer, C. and Shinozuka, M., "Modelling of Concrete Damage," ACI Struct. Jou., Vol. 86, No. 3, May-Jun. 1989.

4) Chung, Y. S., Meyer, C. and Shinozuka, M., "Automated Seismic Design of RC Building Frames," ACI Struct. Jou., Vol. 87, No. 3, May-Jun. 1990.

5) Chung, Y. S., Meyer, C. and Shinozuka, M., "SARCF User's Guide(Seismic Analysis of Reinforced Concrete Frames)," Report No. NCEER-88-44, NCEER, Buffalo, NY, Nov. 1988.

6) DiPasquale, E. and Cakmak, A.S., "Detection and Assessment of Seismic Structural Damage," Report No. NCEER-87-15, NCEER, Buffalo, NY, Aug., 1987.

7) Kanaan, A.E. and Powell, G.H., "General Purpose Computer Program for Inelastic Dynamic Response of Plane Structures," Report No. EERC-73-6, Univ. of California at Berkeley, CA, 1973.

8) Shinozuka, M. and Wai, P., "Digital Simulation Short-Crested Sea Surface Elevations," Jou. of Ship Resr., Vol. 23, No. 1, Mar., 1979.

9) Shinozuka, M., Hwang, H. and Reich, M., "Reliability Assessment of Reinforced Concrete Containment Structures," Nuclear Engr. and Design, Vol. 80, 1984.

10) "Uniform Building Code," International Conference of Building Officials, Whittier, California, 1988.

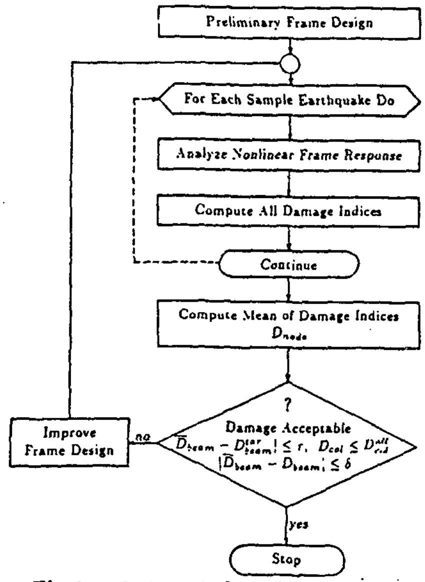
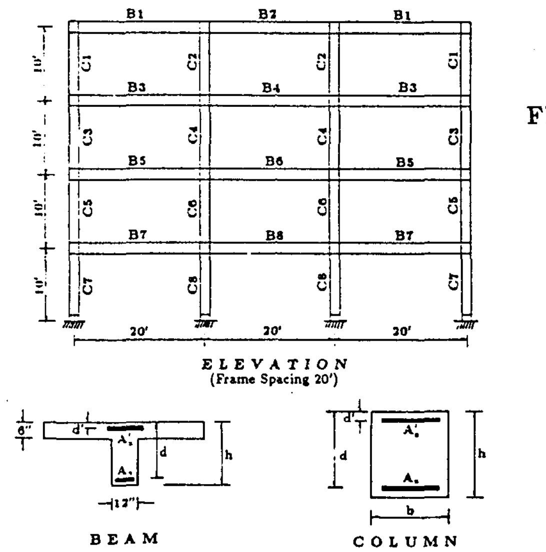


Fig 1 – Automated Design Method

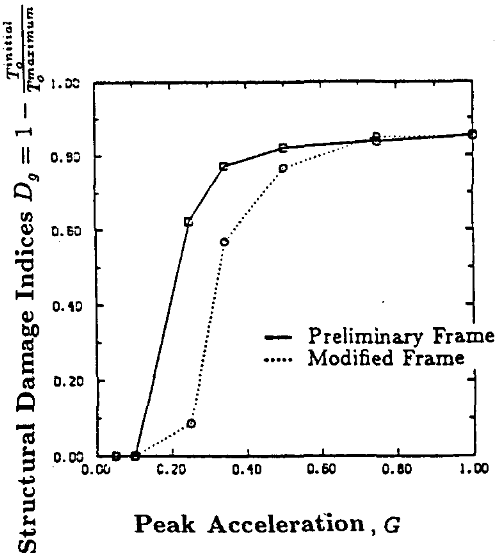


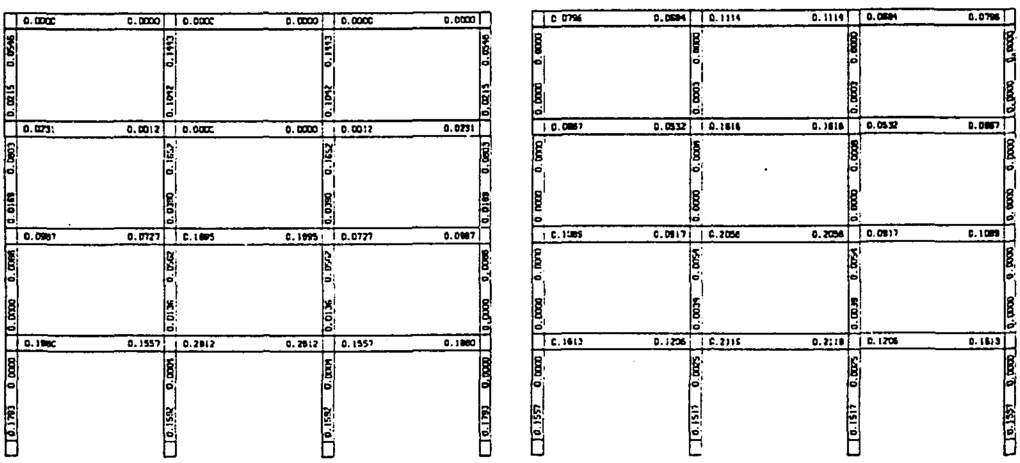
	d'	A_s	A'_s	h
	(in)	(in^2)	(in^2)	(in)
B1,B2	2.0	2.280	2.280	20.0
B3,B4	2.0	2.622	2.622	20.0
B5,B6	2.0	2.622	2.622	22.0
B7,B8	2.0	2.736	2.736	22.0

	d'	A_s	A'_s	b	h
	(in)	(in^2)	(in^2)	(in)	(in)
C1,C2,C3	1.500	1.800	1.800	12.0	15.0
C4,C5	1.500	2.850	2.850	12.0	15.0
C6,C7	1.875	2.850	2.850	15.0	18.0
C8	1.875	2.964	2.964	15.0	18.0

Fig 2 – Details of Example Office Building (1 in = 2.54 cm; 1 kip = 4.448 kN)

Fig 4 – Comparison of Structural Damage Indices With Scaled El Centro North-South Earthquake





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

(a) Preliminary Design Frame

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

(b) Modified Design Frame

Fig 3 – Mean Damage Indices for Example Office Building

Table 1 Changes in Member Damage Indices Due to 5% Increase of Reinforcement, (1.0 g Peak Acceleration) $\times 10^{-4}$

Cause / Effect	Reference Damage Index (See Fig 3(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 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	546 215	27 8	−37 1	−143 −21	−71 4	−32 −20	10 −20	−24 −29	30 −49	−15 −13	9 123	−10 −4	−4 −4	84 −2	24 2	9 −9	30 −9
C2	1443 1042	−83 −60	−130 −149	−44 −129	−185 −148	−57 −7	15 49	−73 −140	−59 −22	−117 −98	45 −22	−32 −12	−100 −83	21 52	−94 −88	−38 −80	−54 −118
B3	231 12	−32 −4	−23 3	−35 4	−66 −7	−64 −10	0 −7	−23 3	−14 5	−19 1	−1 −4	−1 −6	−17 2	1 9	−32 1	−21 −1	−31 1
B4	0	0	0	0	2	0	0	3	22	0	0	1	0	9	1	0	2
C3	803 169	−11 −15	−37 29	−34 −13	108 41	217 41	−41 49	5 25	32 −4	−31 36	37 44	109 30	91 43	65 61	−58 −10	73 −10	−19 −29
C4	1652 390	−113 −40	−143 −68	−131 −28	−53 −6	16 23	−39 −19	−151 −78	−148 −65	−263 −36	−181 21	−24 5	54 −34	192 23	−109 −42	−41 −56	−189 −70
B5	987 727	−14 26	3 41	−35 19	−8 70	−25 51	−19 7	−72 3	−58 16	−221 −121	−109 −63	4 4	−31 89	105 112	11 59	2 66	37 59
B6	1895	60	71	32	41	−14	36	36	36	−336	−300	5	127	137	44	103	98
C5	66 0	−7 0	−12 0	−8 0	−8 0	−7 0	−10 0	9 0	18 0	37 0	5 0	−45 0	−13 0	−22 0	−16 0	−5 0	−16 0
C6	562 136	−24 −4	−55 19	−23 −20	−13 −3	−70 −12	−42 −22	−19 −31	−28 −22	62 −35	63 −32	−55 11	−195 −30	−95 19	−79 −1	−92 18	−93 −8
B7	1880 1557	25 −62	−53 37	−57 0	−21 −27	−44 −54	−27 −23	−5 46	−11 15	30 39	8 77	25 −24	−89 −69	−136 −44	−46 25	−9 −87	−118 −41
B8	2812	−29	−3	−76	−4	−33	−16	46	22	50	26	−30	−114	−111	−105	−37	−63
C7	0 1793	0 −21	0 −35	0 41	0 1	0 −24	0 5	0 15	0 32	0 15	0 45	0 −9	0 −54	0 −117	0 −73	0 −105	0 −70
C8	4 1592	1 −18	−1 79	−3 18	−3 36	−3 −21	−3 −24	−2 122	−2 53	4 119	1 52	−3 −23	4 82	5 59	0 80	0 66	−4 −19

Note :

1. **Two Numbers in one box signify separate indices at both ends of a member.**
2. **For comparison, teh reference damage indices of Fig 3(a) are given in Column 2.**

Peak Acceleration	Structural Damage Index		Difference
	Original	Improved	
0.05 G	0.00	0.00	0.00
0.1 G	0.00	0.00	0.00
0.25 G	0.623	0.086	-0.577
0.342 G	0.772	0.568	-0.204
0.5 G	0.821	0.766	-0.055
0.75 G	0.837	0.849	+0.012
1.0 G	0.853	0.850	-0.003

Table 2 Structural Damage Indices With Scaling the El Centro North-South Earthquake

$$D_g = 1 - \frac{T_o^{initial}}{T_o^{maximum}}$$