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SEISMIC DAMAGE ANALYSIS OF REINFORCED CONCRETE BUILDGINGS

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SUMMARY

One of the objectives of a recent study of reinforced concrete structures was the development of a reliable analytical tool to asses the seismic damage based on post-earthquake damage inspection and/or laboratory testing of components. A combination of in depth structural modeling of components and of overall structural behavior was included in a new computer code, IDARC (Ref.1) which performs a damage assessment of individual structural components (beams, columns, shear-walls, etc.) and of entire system. The damage model and the structural modeling were examined based on the past records of damaged buildings and full scale test results. The paper presents the damage model used in this study and several examples which illustrate the analysis of complex structures.

INTRODUCTION

The damage of reinforced concrete structures during earthquakes is a major engineering problem. Better understanding of factors which contribute to such damage is absolutely necessary. Tests performed in laboratories are an invaluable source of information. However due to the size of real structures, the tests are limited to components only or to single full size specimens. An analytical structural model which integrates the information obtained for individual components to determine the expected damage of the entire system was developed. A combination of a damage model with an analysis of structures with components for which the properties were determined from testing was included in a computer package for Inelastic Damage Analysis of Reinforced Concrete frame-shear-wall structures, IDARC (Ref.1).

A large number of damage indexes were suggested to quantify the damage which occur in components as well as in whole systems (Ref.2), however, the majority refer to either the deformation or the energy absorbed during damage. The component studies showed that the damage is dependent on both permanent deformations and the energy absorbed in hysteretic cycles leading often to low cycle fatigue. (Ref.3). For the evaluation of damage it is necessary to determine the structural state variables. Most models available today (see Ref.1) are based on linearization of component properties with little or no attention to cracking, to nonsymmetric behavior of elements, to separate modeling of shear and flexure, to distribution of inelastic flexibility along line elements and to 3D contribution of transverse elements to planar systems. Such effects were included in the damage analysis presented herein. The analysis includes modeling of individual elements, such as beams, columns, shear-walls with edge columns and transverse beams as hysteretic springs. A static collapse analysis followed by a comprehensive dynamic analysis with earthquake ground motion is performed to determine the actual response and to quantify the accumulated damage. The damage model used in this study is presented along with several examples which illustrate it's efficiency.

DAMAGE MODEL

The seismic damage of each component is measured using a damage index, D, based on a linear combination of maximum deformation ratio and energy dissipation during cyclic loading (see also Ref.3):

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y \delta_u}\int dE \qquad .....(1)$$

where δ_m, δ_u are the maximum deflection experienced by the R/C component during a seismic event and the ultimate deformation capacity of same component, respectively. Q_y is the yield capacity and dE is the dissipated energy increment. β is a constant which emphasizes the strength deterioration per cycle. The damage index was normalized to produce values between 0 and 1, where 1 signifies collapse.

Only two of the five parameters necessary to evaluate the damage index (see Eq.(1)) are component characteristics which need to be identified: (a) the ultimate deformation capacity, δ_u, and (b) the strength deterioration parameter, β. The three remaining parameters , δ_u, Q_y and dE, are response quantities that are determined during the comprehensive dynamic analysis of the entire system. To establish the ultimate deformation capacity, δ_u, two empirical formulations have been derived:

(a) For ***beams and columns***, a regression analysis was performed on 402 R/C components which gave the following minimum-variance solution:

$$\delta_u(\%) = 0.52\, l_s^{1.93} d^{-0.93} \rho^{-0.27} p_w^{0.48} n_o^{-0.48} (f_c')^{-0.15} \qquad .....(2)$$

in which: δ_u is the ultimate deformation capacity, l_s is the shear span, $\rho = p_t f_y / f_c'$ is the normalized steel ratio and no is the axial stress ratio (> 0.05).

(b) For ***shear walls***, a similar analysis was performed on results reported by Hirosawa (Ref.6) on the cyclic testing of 67 specimens:

$$\delta_u(\%) = 0.53\, l_s^{1.23} d^{-0.23} \rho_h^{0.56} \rho_c^{-0.05} \rho_v^{-0.3} n_o^{0.09} (f_c')^{0.85} \qquad .....(3)$$

where: ρ_h is the horizontal reinforcement ratio (> 0.4%), ρ_c is the edge column reinforcement ratio (> 0.2%) and ρ_v is the vertical reinforcement ratio.

The parameter β specifies the rate of strength degradation and gives the ratio of the incremental damage caused by the increase of the maximum response to the normalized incremental hysteretic energy (Ref.4). The strength deterioration parameter has been empirically formulated from analysis of 260 beam and column test data:

$$\beta = \left[0.37 n_o + 0.36(k_p - 0.2)^2\right] 0.9^{p_w} \qquad .....(4)$$

where n_0 is the normalized axial stress, k_p is the normalized steel ratio ($=p_t f_y / 0.85\ f_c'$), f_y is the yield stress of reinforcement and p_w is the confinement ratio (> 0.4%). The equation shows a negative correlation between the deterioration parameter and the confinement ratio and weak positive correlations between all the other parameters.

The quantification of the local damage is not always sufficient for the description of the entire structure or parts of it. A ***story level damage index*** was therefore defined. Such an index is useful when analyzing weak-column strong-beam type buildings where sudden shear drifts may trigger progressive collapse of the total structure. For the purpose of establishing the story-level damage index, the ***local damage indexes***, D_i, are averaged using a weighting factor based on the energy-absorbing capacity of elements:

$$D = \Sigma \lambda_i D_i \ ; \ \lambda_i = E_i \,/\, \Sigma E_i \qquad .....(5)$$

where λ_i is the energy weighting factor and E_i is the total energy absorbed by the component. In the case of strong-column weak-beam type buildings, it is necessary to extend the above concept to the entire structure. An ***overall damage index*** is obtained using Eq.(5) for all the elements in the building. The overall damage index used has been calibrated (Ref.4) with observed damage data of nine reinforced concrete buildings that were moderately or severely damaged during the 1971 San Fernando earthquake and the 1978 Miyagiken-Oki earthquake in Japan.

HYSTERETIC COMPONENT MODELING

All the components are assumed to behave as hysteretic springs. The hysteretic model that was developed for the analysis uses three parameters in conjunction with a non-symmetric trilinear curve (Fig.1) to establish the rules under which inelastic loading reversals take place. A variety of hysteretic properties can be achieved through the combination of the trilinear envelope and the three parameters, henceforth to be referred to as α, β and γ . The values of these parameters determine the properties of stiffness degradation, strength deterioration and pinching, respectively. ***Stiffness degradation*** represented by α is introduced by setting a common point on the extrapolated initial stiffness line and assumes that unloading lines target this point until they reach the x-axis (Fig.1a) after which they aim the previous maximum or minimum points. ***Strength degradation*** is represented by the parameter β, the same parameter as used in the definition of the damage index. ***Pinching behavior*** is introduced by lowering the target maximum or minimum point to a straight level of γP_y along the previous unloading line (as shown in Fig.1c), until the crack-closing point is reached after which the target is the previous maximum or minimum point.

The versatility of the 3-parameter model was shown qualitatively in Ref.1 which demonstrates how the variation of the parameters enables the simulation of different existing models. For α=2.0, β=0.1 and $\gamma=\infty$, the model impalements the modified Takeda model. Or, if α, γ are set to ∞ and β assumes a null value, then the 3-parameter model reduces to a non-degrading Clough-type model. For α=0.0, β=0.0 and $\gamma=\infty$, the 3-parameter model reproduces the origin-oriented model. A variety of simulations is thus possible.

When component test data is available, the determination of the three parameters is easily achieved through identification procedures. One such identification technique has been developed by Yeh (Ref.5) where nonlinear search algorithms in conjunction with error-optimization functions are employed. While such information is not available, the following schemes are suggested (based primarily on generalized component behavior): (a) An origin-oriented model for the shear spring in shear walls achieved by setting $\alpha=0$ and consequent pinching behavior due to inelastic shear is obtained by using a value of γ between 0.01 and 0.1 depending upon the degree of pinching expected; (b) Nominal degradation for moderate to heavily reinforced elements is achieved by setting α=2 - 3; (c) No slip is suggested for beams and columns (though the biased loop behavior in T-Beams is automatically simulated) unless poor confinement or other factors indicate high shear stresses leading to bond-slip.

IDENTIFICATION OF COMPONENT PROPERTIES

The identification tasks can be divided into two main elements: the first consists of establishing the trilinear envelops (generally symmetric in compression and tension for columns, but nonsymmetric for T-beams, coupled walls or other cross-sections lacking symmetry about the bending axis); the second is to identify the parameters which define the hysteretic rules related to stiffness degradation, bond-slip, etc., as mentioned above. An envelope curve is determined for all elements and it is characterized by two turning points: cracking and yielding. All formulations are empirical and based primarily on regression analysis of extensive experimental data (details of the formulation may be found in Ref.1)

NUMERICAL EXAMPLES

A set of columns tested by Wight and Sozen (Ref.7) under cyclic loading was analyzed to demonstrate the effectiveness of the three-parameter model. The curves produced by the proposed model are shown alongside the experimental data in Fig.2.

The seven story building (see Fig.3) tested using the full-scale pseudo-dynamic facility at Tsukuba, Japan under the U.S.-Japan Cooperative Research Program (Ref.8) is analyzed. The recorded accelerogram of the 1968 Tockachi-Oki earthquake with a scaled peak horizontal acceleration of 0.357g was used as input base motion. A comparative response of the top story displacement history for the first 5 seconds of the analysis is shown in Fig.4. In comparing analytical predictions, it must be remembered that the test results were produced by imposing a load proportional to the fundamental mode which does not represent a general MDOF response. Fig.5 compares the quantified damage based on the proposed analysis scheme with the actual damage sustained after testing. It can be seen that the indices generated by the program reflect the degree of damage experienced by the components, especially the severe damage suffered by the bottom story wall (and decreasing rapidly to virtually no damage at the top) and the almost even damage sustained by the beams throughout the height of the building.

Another example demonstrates the efficiency of the analytical model for parametric studies. The results of a study (Ref.9) to classify damageability of typical buildings in the eastern U.S. are presented. A six story frame building was designed only for gravity loads according to ACI318/83 and verified to be suitable to Zone 2 of UBC 1983. Building was composed of four frames. (Fig.6). Ground motions were generated artificially using spectral characteristics typical to earthquakes in the region. Monte Carlo simulations were performed to determine the mean and expected maximum damage to the typical building under study. Fig.7 is a summary of the results following simulation and analysis. The importance of story-level damage indices is clearly emphasized in Fig.7b. For weak-column type structures as in this study, this index reflects the possibility of a progressive structural collapse though the overall damage index may indicate adequate energy and strength reserves.

CONCLUSIONS

A rational analytical modeling scheme is developed to assess damageability of R/C building structures based on identified component behavior. The proposed technique implies: (a) identification of component properties by micro-modeling or testing; (b) integration of component behavior into macro-models with dynamic characteristics; (c) step-by-step inelastic response analysis of the macro-models and (d) quantitative assessment of damage using a combined energy and ductility based index. The proposed model integrates and generalizes several schemes currently in use for micro-modeling of R/C elements. New approaches suggested and implemented include: the use of a non-symmetric trilinear envelop curve; the development of an experimental-based identification module, the integration of shear-wall and out-of-plane element models and the new generalized 3-parameter hysteretic model.

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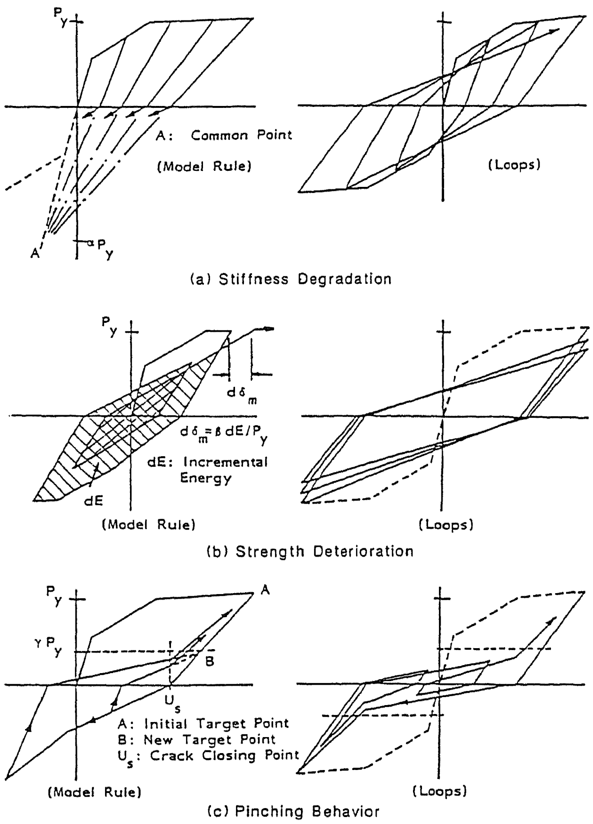

Fig.1 - Three Parameter Model

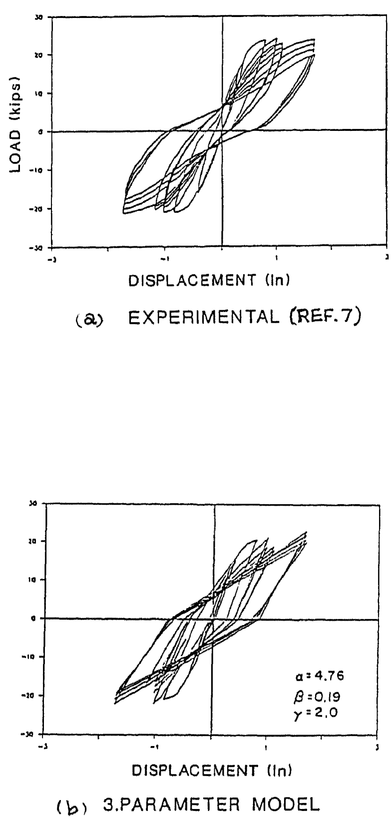

Fig.2 - Reproduction of Experimental Data (Ref.7)

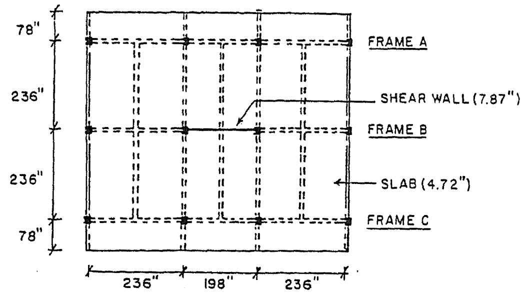

Fig.3 7 Story Frame-Wall Building (Ref.8)

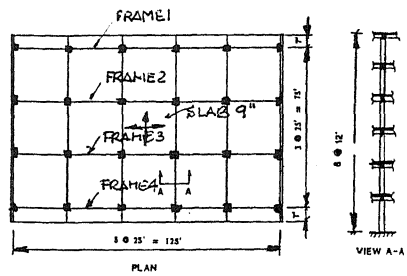

Fig.6 6 Story Building - Floor Plan

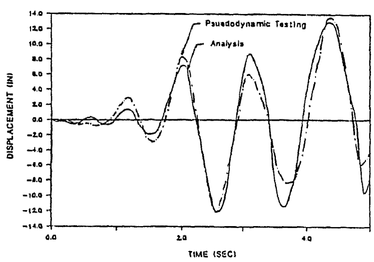

Fig.4 Top Story Displacement of 7-Story Building (Ref.8)

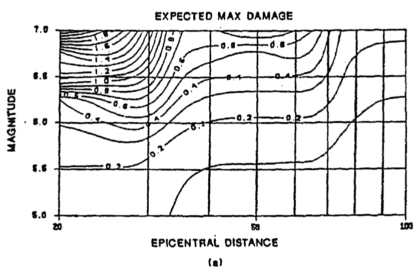

Fig.7a Expected Maximum Damage in 6-Story Building in Eastern U.S.A.

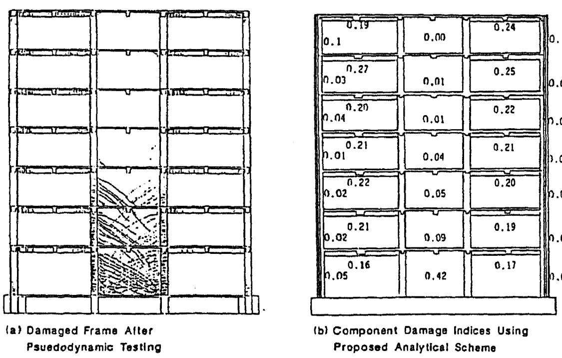

Fig.5 Damage of Mid Frame of 7-Story Building (Ref.8)

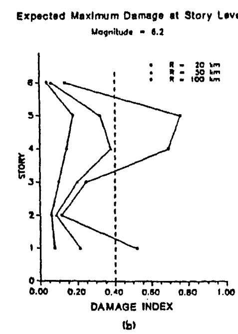

Fig.7b Expected Maximum Story Level Damage in 6-Story Building