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Use of Push-Over Tests to Evaluate Damage of Reinforced Concrete Frame Structures Subjected to Strong Seismic Ground Motions

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ABSTRACT

This paper presents a simplified method for evaluating seismic damage of reinforced concrete frame structures. The method is based on the use of the so-called "push-over test" and takes into account inelastic deformations, energy absorbed by the structure, and energy dissipated by each structural component. The method is applied to three frame structures, i.e. 4, 10, and 18 storey frames, representing low, medium, and tall structures. Three groups of earthquake records having different frequency contents are used as input ground motions. This simplified method allows the designer to determine the performance of a designed structure when subjected to a specified seismic motion without having to perform an inelastic dynamic analysis of the complete structure. Results indicate that the method provides a very good estimate of damage for the 4 and 10 storey frames but some adjustments are required for the 18 storey frame to take into account higher mode effects.

INTRODUCTION

In practice, seismic design for most buildings is carried out by determining loads according to code provisions, e.g. as specified in section 4.1.9. of the 1990 edition of the National Building Code of Canada (NBCC 1990), and applying these loads to a structure which has been designed in a preliminary manner. The properties of the structure are then modified so that the stress resultants in the members of the structure don't exceed specified capacities. Such designs, based on loads and forces, meet code requirements but the structure may not respond appropriately, and it may be excessively damaged when subjected to earthquake ground motion. It is the deformations as well as the inelastic energy dissipation which govern the damage, and which are of concern when buildings respond to earthquakes. While sophisticated methods for inelastic dynamic analysis are available to determine the behaviour and the damage state of buildings subjected to earthquakes, the time and expense to perform and interpret such analyses make their use unsuitable for design of most buildings. Taking this into account, a simplified method is developed for evaluating the damage state of reinforced concrete (RC) structures when subjected to strong seismic motions, which is described in this paper. A detailed explanation and applications of the method are given by Biddah (1994).

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DAMAGE INDICES AS A TOOL FOR EVALUATING DAMAGE

The amount of damage to individual members, storey levels, and the overall structure from seismic excitations can be described analytically in terms of damage indices (ranging from 0.0 for no damage to 1.0 for total damage or collapse). In recent years, a large number of indices have been proposed which recognize the importance of the dissipated energy. Chung et al. (1987), and Reinhorn et al. (1992) have summarized the state-of-the-art for proposed damage models. According to assessment of damage models done by Fardis (1994), it was concluded that among the available damage models, the model developed by Park and Ang (1985) is the most reliable for determining the degree of damage for RC structures. This model, with a modification for use of moment-curvature relationship to determine the damage index is used in this study. The resulting damage index for an individual member is given as:

$$D.I. = \frac{\phi_m}{\phi_u} + \frac{\beta}{\phi_u M_y} \int dE^* \qquad (1)$$

where ϕ_m = maximum observed curvature, ϕ_u = ultimate monotonic curvature, β= strength deterioration factor, $\int dE^*$ = area of hysteretic loops of M-ϕ diagram, and M_y = yield moment.

The damage index is formulated to vary between 0.0 and 1.0; the relationship to member damage states is given below according to Bracci et al. (1989):

	D.I.	= 1.00	Collapse
0.66 <	D.I.	< 1.00	Severe (irreparable) damage
0.33 <	D.I.	< 0.66	Moderate (repairable) damage
0.00 <	D.I.	< 0.33	Minor (serviceable) damage

The global damage index for the complete structure is given by:

$$D.I._{structure} = \sum \lambda_i \, D.I._i \, , \qquad \lambda_i = \frac{E_i}{\sum E_i} \qquad (2)$$

where λ_i = energy weighting factor, and E_i = energy absorbed by each member.

The relationship of the overall damage index to damage state is given by:

	$D.I._{structure}$	= 1.00	Collapse
0.40 <	$D.I._{structure}$	< 1.00	Moderate/severe damage
0.00 <	$D.I._{structure}$	< 0.40	Minor/moderate damage

DESCRIPTION OF THE PROPOSED SIMPLIFIED METHOD

The suggested method involves two push-over analyses (inelastic static analyses) of the actual frame structure and one inelastic dynamic analysis of a transformed equivalent single-degree-of-freedom (SDOF) system. The main steps of the method are as follows:

1. Determine a normalized static deflected mode shape for the actual frame structure as a result of the application of loading specified by the code provisions (e.g. NBCC 1990).

2. Calculate an equivalent mass M_e, and equivalent height L_e for the frame based on the approach proposed by Saiidi (1981), using the assumed static mode shape (from step 1), concentrated mass at each floor level, and height of each storey.

3. Perform a push-over (inelastic static) analysis of the actual frame by applying monotonically increased lateral loading. Results from this analysis are used to determine force-deformation relationship for the equivalent SDOF system. The force is represented by the overturning moment divided by L_e, and the deformation is the lateral displacement at L_e. The required equivalent SDOF system is determined by the equivalent mass M_e, the equivalent height L_e, and the force-deformation relationship.

4. Perform an inelastic dynamic analysis of the equivalent SDOF system subjected to a specified seismic ground motion and determine its maximum displacement response as well as the absorbed energy per unit mass. Based on these quantities, the following can be estimated:
   (i) the energy absorbed by the actual frame structure (using the absorbed energy per unit mass and the total mass of the frame structure).
   (ii) the corresponding roof displacement of the actual frame structure (using the normalized deflected mode shape).

5. Perform a push-over analysis of the actual frame by pushing the structure laterally by a loading distributed as specified by the code provisions until its roof displacement reaches the value estimated in step 4; at this stage, the estimated amount (in step 4) of energy absorbed by the structure is distributed among all the structural elements in proportion to the energy dissipated under monotonic static loading. The local damage index for each element is calculated and the weighted summation of these indices leads to the global damage index for the frame.

APPLICATION OF THE METHOD

Three reinforced concrete ductile moment-resisting frame buildings having 4, 10, and 18 storeys are analyzed. The three buildings have the same floor plan but with different elevations as shown in Fig. 1. The effects of seismic action are considered in the N-S direction and an interior frame is analyzed. The buildings were designed for combined gravity and seismic loads in accordance with the NBCC 1990 and the Canadian concrete code (1984), by Zhu (1992). The fundamental periods of the 4, 10 and 18 storey frames are 0.53, 1.14, and 1.92 s respectively. The buildings are subjected to a total of 45 strong motion records recorded on rock or stiff sites; these records are subdivided into three groups (15 records each) having high (NH), intermediate (NI), and low (NL) peak Acceleration/Velocity (A/V) ratios (note that "N" indicates that these are new sets, i.e. different from the previous McMaster sets). The A/V ratios of the NH, NI, and NL sets are approximately 2, 1, and 0.7 respectively. All records are scaled to peak ground velocity of 0.4 m/s which is the same value used in the design of the frames. A modified version of the IDARC computer program is used for the analysis of these buildings; the original version was developed by Park et al. (1987), and it was later modified by Gaspersic et al. (1992).

For purposes of validation of the proposed method, damage analysis for each frame subjected to each record is done twice, first using inelastic dynamic analysis, and second using the proposed push-over analyses. The comparison of the results is done on a statistical basis. Figures 2, 3, and 4 show the results from statistical analyses of the global damage indices for the 4, 10, and 18 storey frames respectively; each figure shows the maximum, mean+sigma, mean, mean-sigma, and minimum values of the global damage index for each set of records, where "sigma" represents standard deviation.

It can be seen from these figures that the simplified procedure provides a reasonably good estimate of the global damage index for the frames. In case of the 18 storey frame, the procedure under-estimates the global damage index due to the contribution of higher modes. The NI set of records gives the highest global damage index for the 4 storey building (with a mean of 0.29), while the NL set gives the highest global damage indices for both the 10 and the 18 storey buildings (with a mean of 0.21 and 0.18 respectively). This is because the fundamental period of the 4 storey frame is in the range of the predominant periods of the NI records (i.e. 0.3 to 1.0 s), while the fundamental periods of the 10 and 18 storey frames are in the range of the predominant periods of the NL records (i.e. greater than 1.0 s).

It can be noticed also that the damage indices for the 4 storey building are higher than those for the 10 storey building whose damage indices are in turn higher than those for the 18 storey building. Thus, the global damage index decreases with increasing the number of storeys (i.e. increasing natural period of the structure). This is because the NBCC 1990 seismic response factor S (used in the design of the frames) is close to the mean spectrum of the records used in this study in the short period region (i.e. 4 storey frame), and is somewhat higher than the mean spectrum of the records in the long period region (i.e. 10 and 18 storey frames).

As examples of the detailed results, Figures 5 and 6 show the local damage indices, and the state of damage of each structural element for the 4 and 10 storey frame respectively. Figure 5 shows the results for the 4 storey frame from 3 seismic records representative for the mean+sigma, mean, and mean-sigma global damage index from the NI set of records, while Figure 6 shows the results for the 10 storey frame using the same kind of illustration but considering the NL set of records. It can be noticed that the damage mechanisms follow the philosophy of capacity design of structures, or in other words, the plastic hinges take place in beams rather than in columns. Also, as the beams have high ductility capacities (mainly under flexural bending), their local damage indices are relatively low (D.I. = 0.10 to 0.25) even though the ends of the beams have already yielded in most cases. On the other hand, as the columns have lower ductility capacities (mainly under axial compression in addition to flexural bending), they show higher local damage indices (especially the first storey columns, D.I. = 0.40 to 0.60) even though the ends of the columns may only be in stage of cracking.

The application of the simplified push-over analyses to evaluate seismic damage of RC frame structures is limited to frames which oscillate predominantly in the first mode. This is because the parameters used in the push-over analyses may be underestimated for structures for which higher modes have significant effects. A particular solution for taking into account the higher mode effects might be the use of an appropriate dynamic magnification factor. A simplified formula for a magnification factor is derived by Biddah (1994). Results show that the use of such magnification factors give good results in most of the cases except for the case when the 18 storey frame is subjected to the NH set of records.

CONCLUSIONS

A simplified method is proposed for determining damage state of reinforced concrete structures subjected to seismic motions. For purposes of validation of the method, it is applied to three frame structures (4, 10, and 18 storeys) designed according to the NBCC 1990. The following are the main conclusions and observations which arise from this study:

1. The simplified method yields results of reasonable accuracy provided that the structure oscillates predominantly in the first mode.
2. The mean global damage indices for the 4, 10, and 18 storey frames, analyzed in this study, are 0.29, 0.21, and 0.18 respectively, which can be considered as acceptable level of damage for structures subjected to strong seismic motions.
3. All the damage mechanisms follow the philosophy of capacity design of structures.
4. The use of dynamic magnification factor to take into account the higher mode effects gives good results except for the case when the 18 storey frame is subjected to high frequency motions.

REFERENCES

Associate Committee on National Building Code. 1990. National Building Code of Canada, National Research Council of Canada, Ottawa, Canada.

Biddah, A. 1994. Use of push-over tests to evaluate seismic damage of reinforced concrete frame structures. M. Eng. Thesis, Dept. of Civil Engineering, McMaster University, Hamilton, Canada, 192 pages.

Bracci, J.M., Reinhorn, A.M., Mander, J.B., and Kunnath, S.K. 1989. Deterministic model for seismic damage evaluation of reinforced concrete structures. Technical Report No. NCEER-89-0033, National Center for Earthquake Engineering Research., Buffalo, N.Y.

Canadian Standard Association (CSA). 1984. Design of Concrete Structures for Buildings, CAN3-A23.3-M84, Rexdale, Ontario, Canada.

Chung, Y.S., Meyer, C., and Shinozuka, M. 1987. Seismic damage assessment of reinforced concrete members. Technical Report No. NCEER-87-0022, National Center for Earthquake Engineering Research, Buffalo, N.Y.

Fardis, M.N. 1994. "Damage measures and failure criteria for reinforced concrete members. Proceedings of the Tenth European Conference on Earthquake Engineering, Vienna, Austria.

Gaspersic, P., Fajfar, P., and Fishinger, M. 1992. An approximate method for seismic damage analysis of buildings. Proceedings of the Tenth World Conference on Earthquake Engineering, Madrid, Spain, pp. 3921-3926.

Park, Y.J., Ang, A.H.-S. 1985. Mechanistic seismic damage model for reinforced concrete. Journal of Structural Engineering, ASCE, Vol. 111, No.4, pp. 722-739.

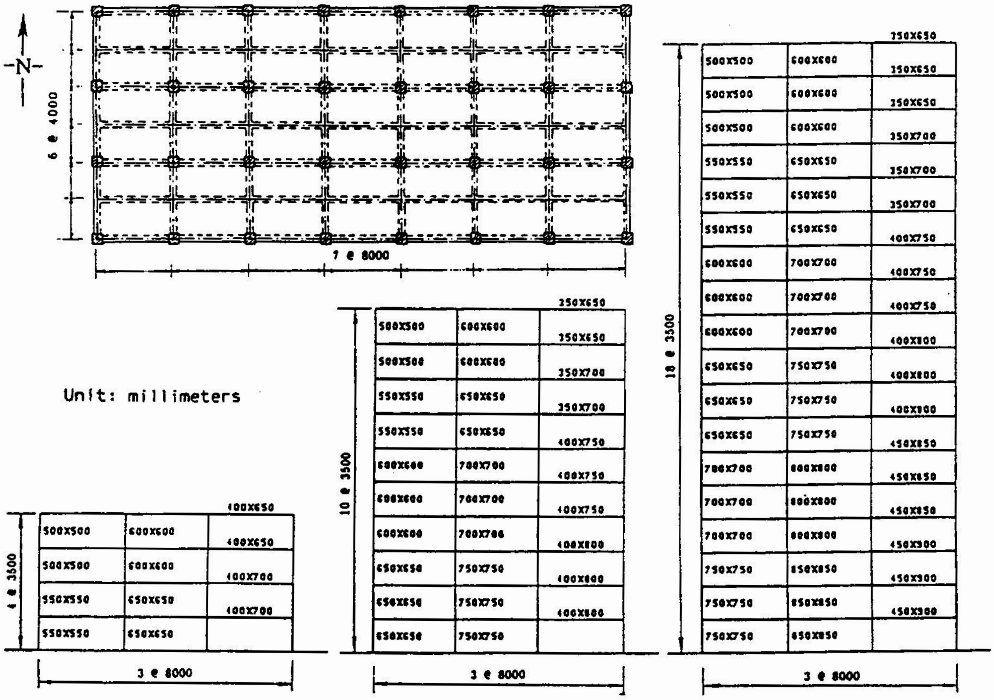
Park, Y.J., Reinhorn, A.M., and Kunnath, S.K. 1987. IDARC: Inelastic Damage Analysis of Frame

Shear-Wall Structures. Technical Report NCEER 87-0008, National Center for Earthquake Engineering Research, Buffalo, N.Y.

Reinhorn, A.M., Kunnath S.K., and Mander J.B. 1992. Seismic design of structures for damage control. In Nonlinear Seismic Analysis and Design of RC Buildings (Eds. P. Fajfar and H. Krawinkler), Elsevier, Amsterdam, pp. 63-76.

Saiidi, M., and Sozen, M.A. 1981. Simple non-linear seismic analysis of RC structures. Journal of Structural Engineering, ASCE, Vol. 107, pp. 937-952.

Zhu, T.J., Tso, W.K, and Heidebrecht, A.C. 1992. Seismic performance of reinforced concrete ductile moment-resisting frame buildings located in different seismic regions. Canadian Journal of Civil Engineering, Vol . 19, pp. 688-710.



4 storey frame

10 storey frame

18 storey frame

Figure 1 Floor plan and frame elevation.

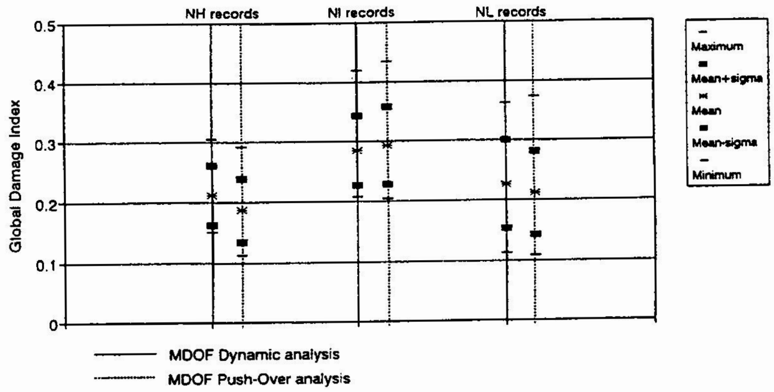


Figure 2 Global damage indices for the 4 storey frame.

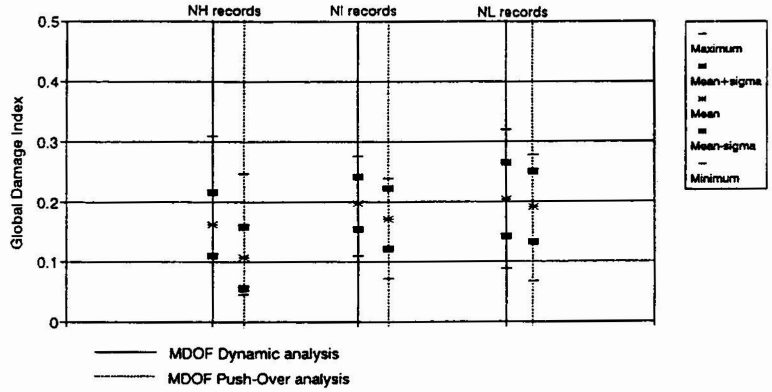


Figure 3 Global damage indices for the 10 storey frame.

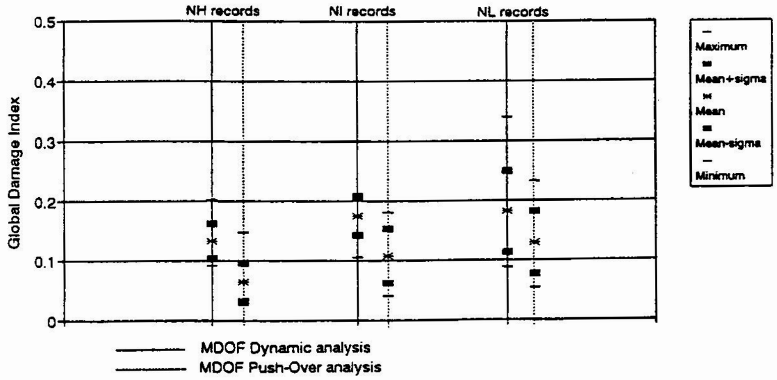
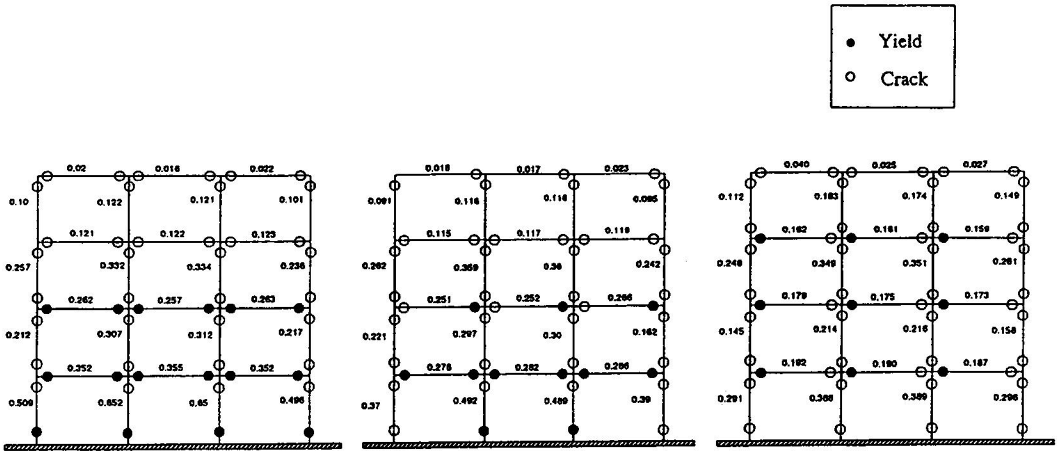
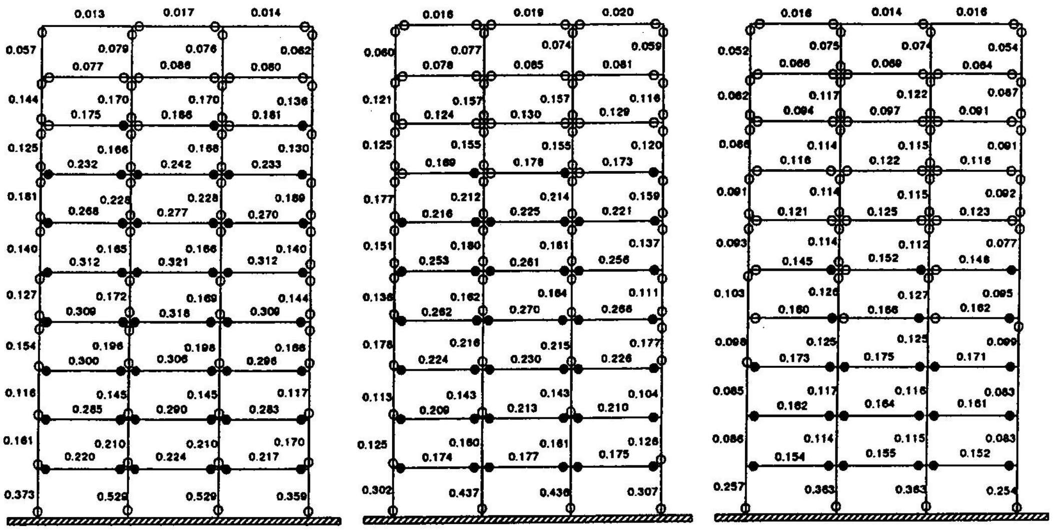


Figure 4 Global damage indices for the 18 storey frame.



NI4 record NI3 record NI6 record

Figure 5 Local damage indices and states of damage for the structural elements of the 4 storey frame, when subjected to three records from the NI set.



NL15 record NL12 record NL1 record

Figure 6 Local damage indices and states of damage for the structural elements of the 10 storey frame, when subjected to three records from the NL set.