Seismic damage assessment of old designed concrete buildings

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This paper presents a simplified approach for evaluating the seismic damage of old designed reinforced concrete (RC) frame buildings. The approach relies on calculating the roof displacement response of the RC building approximately from the spectral data and the free vibration characteristics of the building. Then, a static cyclic analysis of the building is performed up to the calculated roof displacement level. The seismic performance of the structure is estimated based on its static cyclic response. The procedure is examined in cases of a 3-story and a 9-story old designed RC buildings. It is concluded that the proposed approach is an efficient tool for seismic performance evaluation of old designed RC buildings.

في هذا البحث تم تقديم طريقة مبسطة لتقييم التلف الناتج من أحمال الزلازل في المباني الخرسانية القديمة. وتعتمد هذه الطريقة على حساب إزاحة سطح المبنى الخرساني تقريبيا" باستخدام خواص المبنى الخرساني وكذلك طيف الزلازل فى الكود المصري. ويتم تقدير التلف لهذه المباني عن طريق التأثير عليها بأحمال استاتيكية جانبية دورية متزايدة حتى وصول إزاحة سطح المبنى للمستوى الذي سبق حسابه. وقد تم اختبار هذه الطريقة فى حالة مبنى مكون من ثلاثة أدوار وآخر مكون من تسعة أدوار. وقد أثبتت نتائج البحث فاعلية هذه الطريقة لتقييم تلف المباني الخرسانية القديمة الناتج من أحمال الزلازل.

Keywords: Earthquakes, Damage, Reinforced Concrete Buildings, Simplified Analysis

1. Introduction

In recent years, seismic hazard as well as seismic design has become an important research topic in Egypt because of the implications of the 1992 Cairo earthquake. The amount of structural damage caused by this earthquake has drawn the attention to the importance of employing seismic design in newly designed building structures. For old designed buildings, a great effort has yet to be done for developing reliable procedures to assess the performance of these buildings under the effect of future earthquakes.

Simplified evaluation of seismic performance of building structures can be performed using a time history analysis approach [1, 2]. This approach requires the seismic input to be represented by time history records. However, in building codes, seismic input is usually represented by a smoothed elastic response spectrum.

Krawinkler [3] evaluated the seismic performance of ductile buildings without the need for time history records. He calculated the structure roof displacement demand approximately based on the spectral data and the structure fundamental period, then applying a pushover analysis to the structure up to the calculated roof displacement. The structure seismic response is determined from the pushover final response after using some amplification factors to account for the duration effect of the earthquakes.

Evaluation of the seismic performance of structures based on the structure static response and the elastic spectral data is approximate in nature. The spectral information is based on the response of elastic Single-Degree-Of-Freedom (SDOF) system. It is assumed that the structure response is controlled by the first mode of vibration and that the contribution of higher modes is negligible. Moreover, the structure has to enter the inelastic range of deformation in order to absorb the seismic energy and, in this situation, the inelastic displacement demands of the structure has to be computed approximately from the elastic displacement demands. Finally, the structure damage under the effect of the earthquake loading is expected to be different from the structure damage due to the static loading even if the roof displacement response is constant in both loading cases. This can be attributed to the duration effect of the earthquakes as well as the contributions of higher mode of vibrations.

Buildings designed before the advent of

earthquake regulations using gravity load design procedures (referred here as old designed buildings) have several of construction details that do not conform to current code requirements for seismic design and detailing. These buildings are characterized by poor behavior under the effect of seismic loading. The structural members are expected to experience severe strength deterioration and stiffness degradation under the effect of a cyclic lateral loading [4].

The objective of this research is to present a procedure for seismic damage assessment of old designed reinforced concrete (RC) buildings. This has been achieved by enhancing the Krawinkler approach, which relies on estimating the structure seismic performance approximately from the structure static response. The proposed procedure is applied for evaluating the seismic damage of two old designed RC buildings with different heights.

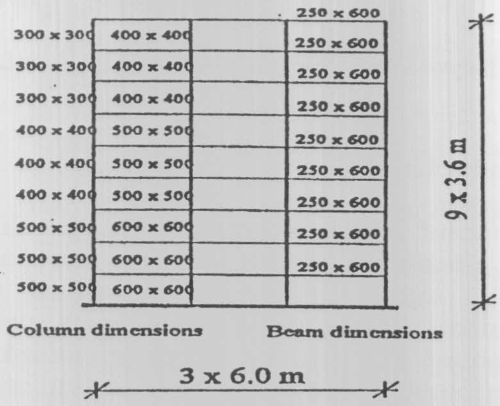
2. Old designed RC buildings

The selected old designed RC buildings are 3 and 9-story buildings that are designed for gravity loads by Biddah [5] according to the 1963 ACI code [6]. The 9-story building details are shown in fig. 1. The 3-story building is considered as the upper three stories of the 9-story building. The concrete strength is 21 MPa and the steel yield strength is 300 MPa. The design live load for the buildings is taken as 2.4 kN/m^2.

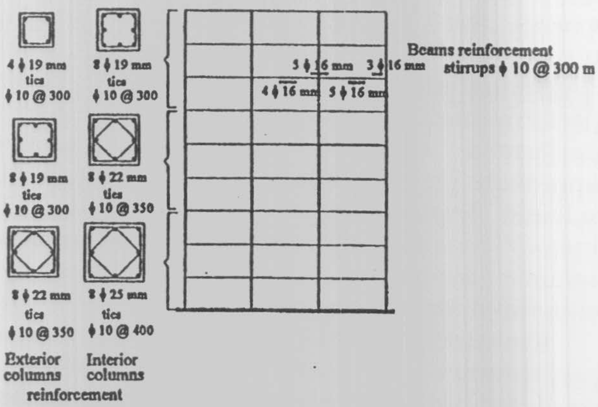
The buildings were modeled as a series of planar frames connected at each floor level by rigid diaphragms, therefore, only 2D analysis is performed. The beams and columns were represented using a beam-column model that is capable of representing the behavior characteristics of beams and columns of old designed RC frames. Stiffness degradation, pinching and strength deterioration, were explicitly taken into account in the model. The hysteretic rules of the model are shown in fig. 2. Detailed description of the model can be found in Ghobarah et al. [7]. The seismic performance of the old designed RC buildings is evaluated in terms of deformations and damage indices. The damage index used in this study is based on measuring the change in the building stiffness due to the application of earthquake loading. The approach requires performing a pushover analysis for the building twice; once before subjecting the building to the earthquake and once after subjecting the building to the earthquake loading, as shown in fig. 3. The damage index is defined as;

$$D_g = 1 - (K_{final} / K_{initial}) \ . \tag{1}$$

Where $K_{initial}$ is the initial building stiffness before applying the earthquake loading and K_{final} is the final building stiffness after applying the ground motion. More details of the damage calculation procedure are presented by Ghobarah et al. [8].



a) The dimensions of columns and beams.



b) The reinforcements of beams and columns.

Fig. 1. The 9-story building details [5].

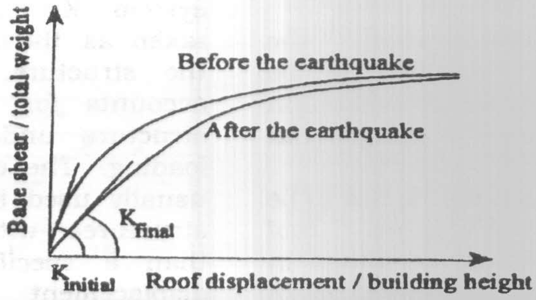


Fig. 2. Pushover analysis: Change of stiffness for calculating the damage index of frame.

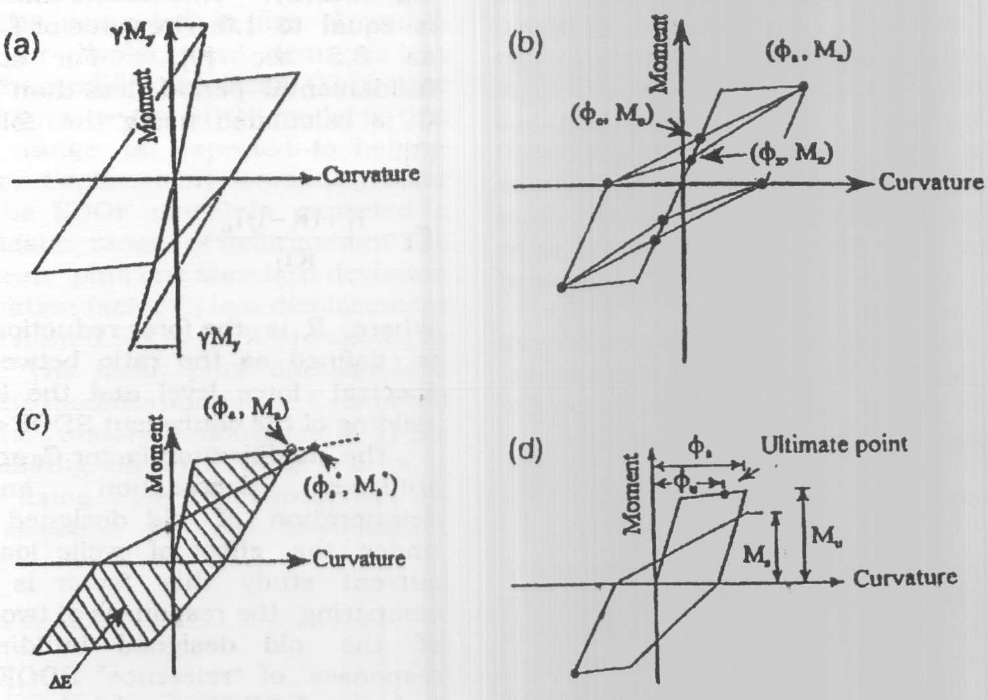


Fig. 3. Hysteretic modeling of the moment-curvature relationship: (a) stiffness degradation; (b) pinching; (c) strength deterioration; (d) softening.

3. Simplified performance evaluation approach

Despite of the approximate nature of the seismic performance evaluation procedure that is based on the code response spectrum, it represents an appropriate evaluation tool for practical applications. Krawinkler [3] presented a description of this procedure and applied it for seismic damage evaluation of some ductile buildings. The procedure can be summarized in the following steps:

1- The structure target roof displacement response is calculated assuming that the period of the structure, the elastic spectral force level and the lateral load capacity of the structure are determined.

2- A static pushover analysis is performed up to the target roof displacement level using the code lateral load distribution.

3- The structure seismic response is determined by amplifying the static response to account for the duration effect of the earthquakes.

In the current investigation, the Krawinkler approach is modified to be applicable for seismic performance evaluation of old designed RC buildings. Changes that are made include:

1- Determining an amplification factor to be used in estimating the target roof displacement of old designed buildings in order to account for the deterioration of strength and the degradation of stiffness under the effect of cyclic lateral loading.

2- A static cyclic loading is used instead of the monotonic pushover loading. Old designed buildings exhibit significant deterioration in stiffness and strength under the effect of seismic loading. Cyclic loading causes deterioration and damage to the structural members similar to that of the earthquake loading. The structure damage at the final stage of the cyclic loading is expected to be more close to seismic damage than in case of using monotonic static loading.

3- Calculating response amplification factors for amplifying the structure static response. These amplification factors are based on the real response of old designed RC buildings.

4. Target roof displacement

The target displacement of the structure due to seismic loading is estimated by assuming that the response of the structure is controlled by the first mode of vibration. Krawinkler [3] expressed the structure target displacement demands "u" as follows:

$$u = (\Pi C_i)\frac{T_1^2}{4\pi^2} S_a . \tag{2}$$

Where T_1 is the structure fundamental period, S_a is the elastic spectral acceleration and $\Pi\ C_i$ is the product of modification factors that account for the difference in response between the elastic SDOF model and the inelastic Multi-Degree-Of-Freedom (MDOF) structure. In the current study, three different modification factors, C_o, C_1 and C_2 were considered. The modification factor, C_o, transforms the displacement of the SDOF model to the roof displacement of the MDOF system. Krawinkler [3] suggested that C_o to be taken as the first mode participation factor of the structure. The modification factor, C_1, accounts for the inelastic behavior of the structure under the effect of the seismic loading. The equal displacement concept is usually used to estimate the value of C_1 for structures with fundamental period greater than a specified value T_o. In the equal displacement concept, the displacement response of the structure is the same whether the structure responds elastically or yields significantly. This means that the value of C_1 is equal to 1.0. The value of T_o was estimated as 0.5 sec [9]. For structures with fundamental period less than T_o, the value of C_1 is calculated using the following formula [3]:

$$C_1 = \frac{T_1 + (R-1)T_o}{RT_1} . \tag{3}$$

Where, R is the force reduction factor, which is defined as the ratio between the elastic spectral force level and the lateral load at yielding of the equivalent SDOF model.

The modification factor C_2 accounts for the stiffness degradation and strength deterioration of old designed RC buildings under the effect of cyclic loading. In the current study this factor is calculated by comparing the responses of two SDOF models of the old designed buildings with the responses of "reference" SDOF models. The "reference" SDOF models have bilinear force-displacement relationships that do not exhibit any strength deterioration or stiffness degradation under the effect of cyclic loading. The analysis is performed using scaled versions of twelve selected ground motion records (table 1). The results of the comparison are presented separately in figs. 4 and 5. These figures represent the relationship between the displacement demands obtained from the reference model and the modification factor C_2, which is defined as:

$$C_2 = \frac{\text{displacement of the old designed model}}{\text{displacement of the reference model}} . \tag{4}$$

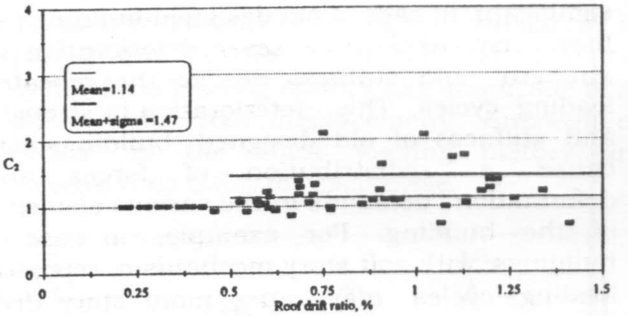


Fig. 4. Effect of non-ductile response on the displacement demands of the 3-story building.

The results presented in figs. 4 and 5 indicate that for displacement demands less than 0.5%, the modification factor C2 is close to 1.0. This is because the SDOF model in this displacement range is expected to behave elastically. For displacement demands greater than 0.5%, the SDOF model is expected to inter the inelastic range of deformation. The mean and mean plus one standard deviation of the modification factor C_2 in a displacement range varies from 0.5% to 1.5% are shown in figs 4 and 5. The mean plus one standard deviation of the modification factor C_2 is equal to 1.47 for the 3-story building and 1.51 for the 9-story building. Based on these results, it appears that using C_2 =1.51 represents an appropriate selection for the modification factor C_2.

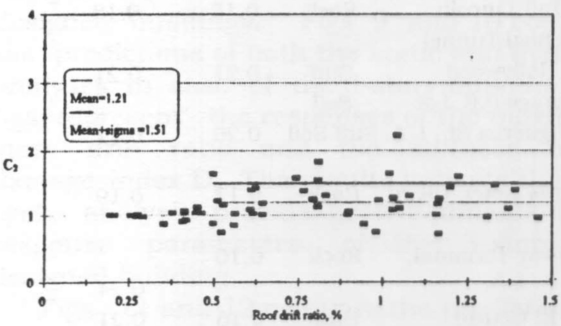


Fig. 5. Effect of non-ductile response on the displacement demands of the 9-story building.

5. Verification of the roof displacement predictions

The inelastic displacement demands calculated from equation 2 are compared with those obtained from the inelastic dynamic analysis of the MDOF models of the RC buildings. The analysis is performed using scaled versions of the selected twelve ground motion records. The analysis results are shown in figs 6 to 7. The figs represent the relationship between the roof displacement demands and a drift factor DF. The drift factor DF is defined as:

$$DF = \frac{\text{displacement of the MDOF model}}{\text{displacement of Eq. 2}} . \quad (5)$$

The mean and mean plus one standard deviation of the drift factor DF are shown in fig. 6 for the 3-story building and in figs. 7 for the 9-story building. The analysis results presented in fig. 6 indicate that on average, the approximate roof displacement prediction of the 3-story building is in the conservative side (Mean of DF=0.80). The standard deviation of DF is equal to 0.16 for the case of the 3-story building.

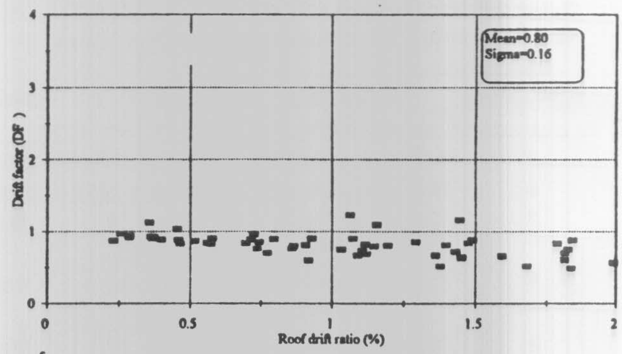


Fig. 6. Comparison between the roof displacement prediction of the simplified approach and the dynamic analysis for the case of the 3-story building.

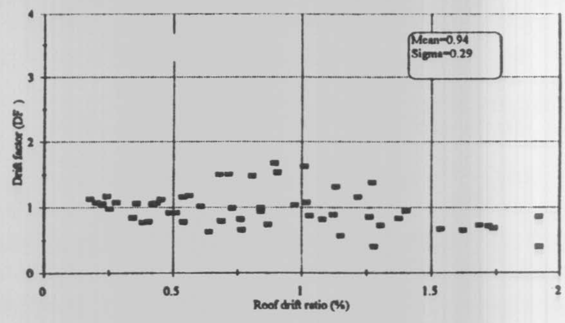


Fig. 7. Comparison between the roof displacement prediction of the simplified approach and the dynamic analysis for the case of the 9-story building.

Fig. 7 shows the results of the 9-story building which indicates that, on average, the approximate prediction of the 9-story building is also in the conservative side (Mean of DF =0.94). However, the standard deviation is significantly higher than the level reached in case of the 3-story building. This can be attributed to the effects of higher modes in case of the 9-story building.

In general, it can be concluded that the accuracy of the approximate approach is reasonable taking into consideration the approximate nature of the simplified approach that is discussed in the introduction section of this paper.

6. The structure static response

During an earthquake loading, the structure experiences repeated loading cycles with different amplitudes. Consequently, the seismic damage will be dependent on the duration of the earthquake. The effect of the ground motion duration is especially significant in case of old designed buildings as they may experience sever deterioration in strength and stiffness due to the repeated loading cycles. This deterioration in strength and stiffness of old designed buildings may cause a redistribution of forces and deformation demands in the various elements of the building. For example, in case of buildings with soft story mechanism, repeated loading cycles may cause more story drift demand in the soft story than in the case of a monotonic static loading.

In the current study, a static cyclic analysis is used for determining the performance parameters (story drifts and damage indices) of old designed buildings at specific target roof displacements. The analysis is performed using the code lateral load distribution. The choice of the cyclic history of the static cyclic loading is arbitrary. The time history of a ground motion has random characteristics and can not be represented accurately by a single static loading history. In the current study, the

Table 1
Site information for the selected ground motions

Rec. No.	Earthquake	Date	Comp.	Site	Soil type	A (g)	V (m/s)
1	Parkfield, California	June 27, 1966	N65W	Temblor, No.2	Rock	0.27	0.15
2	Nahanni, Canada	Dec. 23,1985	LONG	Site 1, Iverson	Rock	1.10	0.46
3	Imperial Valley, California	May 18, 1940	S00E	El Centro	Stiff Soil	0.35	0.33
4	Kern County, California	July 21, 1952	S69E	Taft Lincoln School Tunnel	Rock	0.18	0.18
5	San Fernando, California	Feb. 9, 1971	N90E	Hollywood Storage P.E. Lot	Stiff Soil	0.21	0.21
6	San Fernando, California	Feb. 9, 1971	N37E	234 Figueroa St., L.A.	Stiff Soil	0.20	0.17
7	Monte Negro, Yugoslavia	Apr. 15, 1979	N00E	Albatros Hotel, Ulcinj	Rock	0.17	0.19
8	Long Beach, California	Mar. 10, 1933	N51W	Subway Terminal, L.A.	Rock	0.10	0.24
9	Lower California	Dec. 30, 1934	S00W	El Centro	Stiff Soil	0.16	0.21
10	San Fernando, California	Feb. 9, 1971	N61W	2500 Wilshire Blvd., L.A.	Stiff Soil	0.10	0.19
11	Near E. Coast of Honshu, Japan	May 16, 1968	N00E	Muroran Harbor	Stiff Soil	0.23	0.33
12	Mexico	Sep. 19, 1985	S00E	Zihuatenejo, Guerrero Array	Rock	0.10	0.16

A= Peak ground acceleration, V= Peak ground velocity.

cyclic history shown in fig 8 is considered in performing the static cyclic analysis. The first cycle of the loading history has amplitude of Δ_y (Δ_y = yield roof displacement). Subsequent cycles of the static loading history are increased by $0.5\Delta_y$ for each cycle.

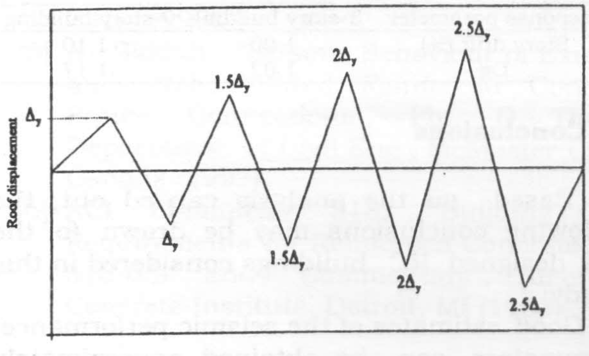


Fig. 8. Roof displacement history of the static analysis.

7. Amplification factors for the static response

Amplification factors are needed to amplify the response parameters obtained from the static cyclic analysis in order to account for the response differences between the static loading and the seismic loading. Amplification factors are estimated by comparing the response parameters obtained from the static analysis with those obtained from the seismic analysis of the MDOF models of the old designed buildings. Figs 9 and 10 compare the predictions of both the static and dynamic analyses in case of the 3-story building. The figs represent the responses of the maximum story drift ratio and the stiffness based damage index D_g. The results indicate that the cyclic analysis provide good estimates to the response parameters of the 3-story old designed building.

Figs 11 and 12 compare the predictions of the performance parameters in case of the 9-story building. The results indicate that the cyclic analysis provide good estimates to the maximum story drift ratio of the 9-story building, while it under predicts the damage index Dg. This may be attributed to different reasons:

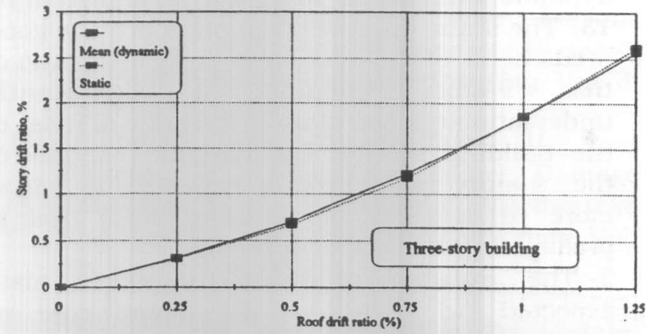


Fig. 9. Comparison between the story drift predictions of the static and dynamic approaches for case of the 3-story building.

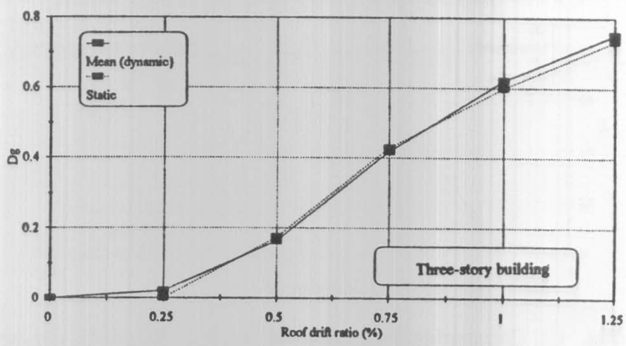


Fig. 10. Comparison between the damage index D_g predictions of the static and dynamic approaches for the case of the 3-story building.

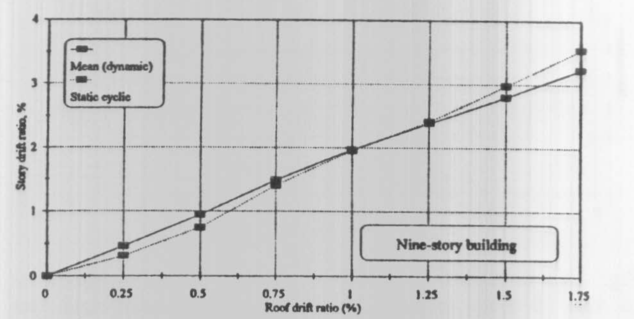


Fig. 11. Comparison between the story drift predictions of the dynamic approaches for the case of the 9-story building.

1- In the dynamic analysis, the 9-story building experienced different deflection profiles during response due to the effects of higher modes while, the static response is controlled by single mode. This is apparent when comparing the distribution of the story drift ratios of the various stories in case of the dynamic and static analyses as shown in fig 13. The static approach although provide good predictions of the maximum story drift ratio of the 9-story building, it significantly underestimates the story drift ratios of most of the building stories. The dynamic response of the 9-story structure is expected to cause more damage to the building than that is predicted by the static approach.

2- The duration of the earthquake is also expected to play a significant rule in increasing the level of the damage indices in the dynamic response as compared to the levels predicted from the static approach.

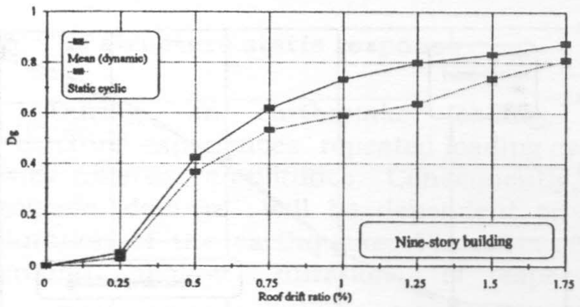


Fig. 12. Comparison between the damage index D_g predictions of the dynamic approaches for the case of the 9-story building.

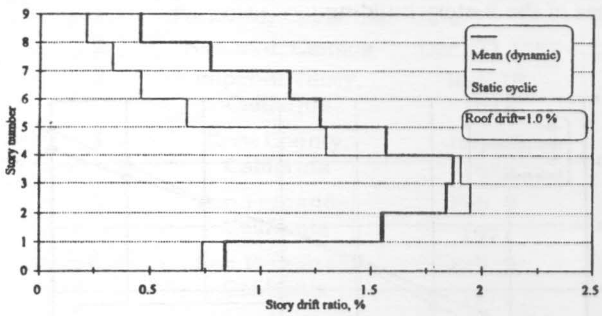


Fig. 13. Comparison between the story drift distributions of the 9-story building in case of the static and the dynamic analyses.

The response amplification factors recommended for the 3-story and the 9-story buildings are summarized in table 2. For each of figs 9 to 12, an amplification factor is calculated as the average of the ratios between the dynamic and static responses calculated at the points of black square dots. Only the ratios greater than 1.0 are considered (these values represent static results lower than the dynamic ones). The maximum level of the amplification factors is 1.17 and it corresponds to the damage index of the 9-story building.

Table 2
Amplification factors of the response parameters

Response parameter	3-story building	9-story building
Story drift (%)	1.00	1.10
Dg	1.02	1.17

8. Conclusions

Based on the analysis carried out, the following conclusions may be drawn for the old designed RC buildings considered in this study:

1- Good estimates of the seismic performance parameters can be obtained approximately using the proposed simplified approach.

2- An amplification factor of 1.51 is an appropriate selection for magnifying the displacement demands in order to account for strength deterioration and stiffness degradation.

3- Static cyclic analysis is a suitable tool for predicting the seismic performance parameters.

4- The accuracy of the static analysis in predicting the building response parameters can be improved by using response amplification factors. The amplification factors of the 3-story and the 9-story buildings are calculated. The maximum level of the amplification factors is 1.17 which corresponds to the damage index of the 9-story building.

References

[1] K. R. Collins, and D. A., Foutch, "Investigation of Alternative Seismic Design Procedure for Standard Buildings", Structural Research Series (600), Department of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana, Illinois (1995).

[2] S.W. Han, and Y.K., Wen, "Method of Reliability-Based Seismic Design", Journal of Structural Engineering, ASCE, Vol. 123, (3), pp. 256-270 (1997).

[3] H. Krawinkler, "Pushover Analysis: Why,

How, When, and When Not to Use It", Proceedings of the 65 Annual Convention of the Structural Engineers Association of California, Maui, Hawaii, October 1-6, 1996, pp.17-36 (1996).

[4] A. Ghobarah, "Seismic Assessment of Existing RC Structures", Progress in Structural Engineering and Materials, Vol. 2 (1) (1998).

[5] A. Biddah, "Seismic Behaviour of Existing and Rehabilitated Reinforced Concrete Frame Connections", Ph. D. Thesis, Department of Civil Eng., McMaster Univ., Canada (1997).

[6] ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-63) and Commentary", American Concrete Institute, Detroit, MI (1963).

[7] A. Ghobarah, T. Aziz, and H. Abou-Elfath, "Softening Effects on the Seismic Response of Nonductile Concrete Frames", Journal of Earthquake Engineering, Vol. 3, pp.59-81 (1999).

[8] Ghobarah, A., Abou-Elfath, H. and Biddah, A., "Response-Based Damage Assessment of Structures", Earthquake Engineering and Structural Dynamics, Vol. 28, pp.79-104 (1999).

[9] A. K. Chopra, "Dynamics of Structures, Theory and Applications to Earthquake Engineering", Prentice Hall, Englewood Cliffs, New Jersey (1995).

Received November 29, 2000
Accepted July 31, 2001