Assessment of the Relevance of Displacement-Based Design Methods/Criteria to Nuclear Plant Structures

C. Hofmayer[1], C. Miller[1], Y. Wang[1], J. Costello[2]

1) Brookhaven National Laboratory, Upton, NY
2) U.S. Nuclear Regulatory Commission, Washington D.C.

ABSTRACT

Revisions to the USNRC Regulatory Guides and Standard Review Plan Sections devoted to earthquake engineering practice are currently in process. The intent is to reflect changes in engineering practice that have evolved in the twenty years that have passed since those criteria were originally published. Additionally, field observations of the effects of the Northridge (1994) and Kobe (1995) earthquakes have inspired some reassessment in the technical community about certain aspects of design practice. In particular, questions have arisen about the effectiveness of basing earthquake resistant designs on resistance to seismic forces and, then evaluating tolerability of the expected displacements. Therefore, a research effort was undertaken to examine the implications for NRC's seismic practice of the move, in the earthquake engineering community, toward using expected displacement rather than force (or stress) as the basis for assessing design adequacy.

The results of the NRC sponsored research on this subject are reported in this paper. A slow trend toward the utilization of displacement based methods for design was noted. However, there is a more rapid trend toward the use of displacement based methods for seismic evaluation of existing facilities. A document known as FEMA 273, has been developed and is being used as the basis for the design of modifications to enhance the seismic capability of existing non-nuclear facilities. The research concluded that displacement based methods, such as given in FEMA 273, may be useful for seismic margin studies of existing nuclear power stations. They are unlikely to be useful for the basic design of new stations since nuclear power stations are designed to remain elastic during a seismic event. They could, however, be useful for estimating the margins associated with that design.

INTRODUCTION

The NRC is in the process of updating its requirements for earthquake engineering design of nuclear power plants. The regulation governing seismic criteria and design, Appendix A to 10 CFR Part 100, was revised in December 1996. Subsequently, field observations of the effects of the Northridge (1994) and Kobe (1995) earthquakes have inspired some reassessment in the technical community about certain aspects of design practice. In particular, questions have arisen about the effectiveness of basing earthquake resistant designs on resistance to seismic forces and, then evaluating tolerability of the expected displacements. Therefore, a research effort was undertaken to examine the implications for NRC's seismic practice of the move, in the earthquake engineering community, toward using expected displacement rather than force (or stress) as the basis for assessing design adequacy.

A literature survey was conducted on the recent changes in seismic design codes and standards, on-going activities of code-writing organizations and published documents by researchers on the displacement-based design methods. The detailed results of the literature survey are reported in Appendix A to NUREG/CR-6719 [1]. A summary of this survey was presented in SMiRT-15 [2]. Based on the survey, it was observed that the transition to displacement based seismic design is a rather slow process due to inertia invariably encountered in the engineering community. Changes in one element of a design tend to be counterbalanced by changes in another element. Uniform nationwide acceptance is expected to come slowly. Thus, it did not appear that there would be a major "ground swell" of demand to change NRC criteria for new plants.

In the area of rehabilitation of existing buildings, however, it was noted that a need for change has been accepted. Researchers and practitioners tend to experiment with their new ideas in the areas of repair or rehabilitation. Thus, it was concluded that if the nuclear industry proposed to utilize some of the recent developments, it would at first be most likely applied to issues related to seismic reevaluation or seismic margin/PRA studies.

NONLINEAR ANALYSIS OF NUCLEAR PLANT STRUCTURES

The response of structures to seismic induced loadings has been traditionally performed using elastic methods. This approach was a natural outgrowth of the use of elastic analysis methods to evaluate structural performance under working loads. The acceptance criteria for load combinations on structures, including seismic effects, have been based on ultimate strength provisions. Seismic loads have often been reduced in this process by dividing the loads by ductility factors to account for the fact that ductile structures can withstand dynamic loads larger than the elastic limit load.

This reliance on elastic analytical methods has been changing over the past few years as a result of the growing interest in reducing the potential effects of earthquakes on the nation's building inventory. Under the National Earthquake Hazards Reduction Program (NEHRP), all federal agencies are required to evaluate the seismic capacities of their building inventory, to develop retrofits that reduce the seismic risk, and to prioritize the repairs based on cost benefit criteria. As agencies began to implement this requirement, it soon became apparent that budgetary constraints place great importance on the last of these tasks. Useful cost benefit criteria require that the seismic response used to evaluate the buildings be as realistic as possible. Elastic analysis methods (even with the use of ductility factors) are not adequate for this purpose. Rather the analytical methods must focus on inelastic methods which rationally account for the effect of ductile behavior on the seismic capability of the building. FEMA 273 [3] sets the basic criteria to be used in implementing NEHRP. Inelastic analysis methods are proposed which focus on predicting the maximum seismic displacement rather than the seismic load that a structure can withstand. It is expected that these requirements will transform the profession so that inelastic deformation seismic analyses are used rather than elastic load based methods.

Nonlinear analyses of nuclear power station structures have been used for margin studies where it is desired to account for ductility effects in a rigorous manner. Seismic margin studies relate demand loads to a prediction of ultimate capacity. The ultimate capacity for ductile structures subjected to dynamic loading is tied to a deformation criteria, such as a number of yield deflections, for estimating failure. Elastic analysis is not suited to this task as it focuses on load and says nothing about structural behavior post yield. A nonlinear dynamic analysis is required, but is difficult and time consuming to perform. Hence attempts have been made to apply factors (ductility) to elastic analysis to account for acceptable structural response into the post yield range.

The FEMA 273 methodology is an alternate approach that accounts for performance into the post yield range. It requires the performance of a nonlinear static analysis of the structure with the loading monotonically increased (pushover analysis). Criteria are then given for the maximum displacement that the structure must withstand; this displacement is related to the level of the earthquake and the dynamic characteristics of the structure. The distribution of loads and displacements throughout the elements of the structure at this displacement are then investigated by comparing the element deformations with acceptance limits. The acceptance limits are set to values typically suitable for margin studies.

This paper reports the results of a study [1] to explore the extent to which FEMA 273 methodology could be useful for reviewing nuclear power stations. The FEMA 273 methodology has the very desirable characteristic in that the same analysis can be used for evaluating the facility at the design level earthquake and at larger magnitude earthquakes associated with margin studies. It is also directly applicable to a graded criteria where more important facilities would be subjected to more stringent acceptance limits than less important facilities.

Two structures have been chosen as the analysis models used for nonlinear analysis. The first structure is a four story frame structure with shear walls providing the primary lateral load system, referred herein as the shear wall model. The second structure is the turbine building of the Diablo Canyon nuclear power plant. Both models are analyzed using the displacement based (pushover) analysis and nonlinear dynamic analysis. In addition, for the shear wall model an elastic analysis with ductility factors applied was also performed. The objective of this work is to compare the results between the analyses, and to develop insights regarding the work that would be needed before the displacement based analysis methodology could be considered applicable to facilities licensed by the NRC.

SHEAR WALL MODEL

Description of the Model and Loading

The shear wall model is a four story reinforced concrete building with shear walls. The typical floor framing plan of the building is shown in Fig. 1. The building is 197 feet (60 m) long in the North-South direction and 95.75 feet (29.18 m) wide in the East-West direction, and it is symmetric in both directions. Since the building is

symmetric and the input loading is applied in the North-South direction, a simplified 2D model which represents half of the building in the East-West direction has been generated and used in the analyses. This building was previously used as a sample problem for the IDARC program [4].
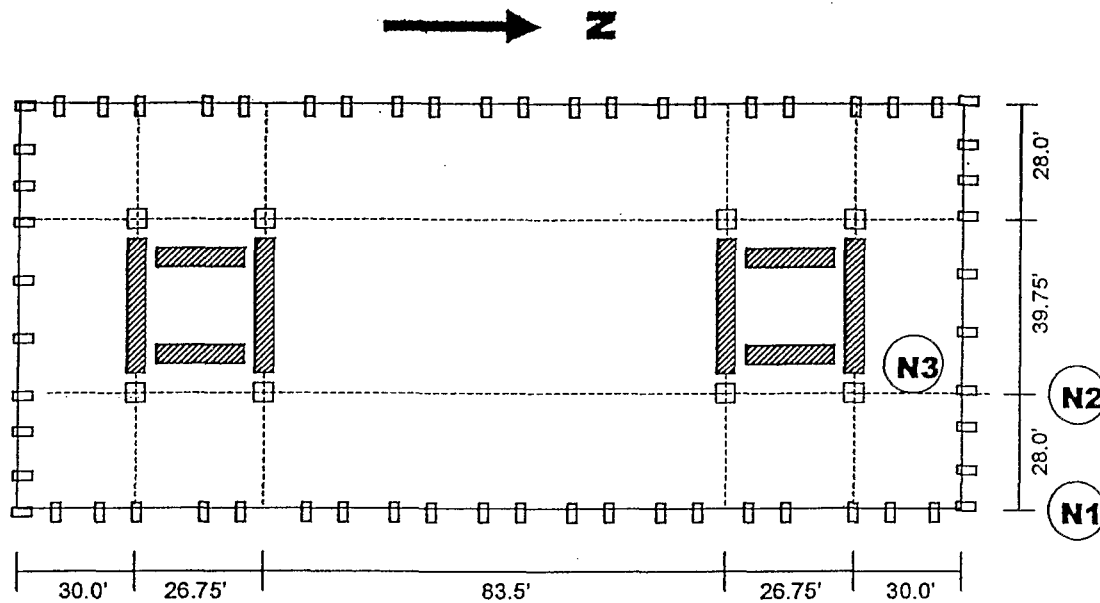

Figure 1 Plan View of the Building for the Shear Wall Model [4]
(1 ft. = .3048 m)

IDARC is a Fortran program developed and maintained by the National Center for Earthquake Engineering Research (NCEER) at the State University of New York at Buffalo. The program was designed to perform Inelastic Damage Analysis for Reinforced Concrete structures; thus it was named IDARC. Since the code has been used to perform nonlinear static (pushover) analysis for commercial buildings, it was selected for this study to perform both the time history analyses and the FEMA analyses.

The 2D model is based on the combined stiffness of the three frames marked as N1, N2, and N3 in Fig. 1. Frame N1 contains 22 columns, frame N2 contains 6 columns and frame N3 consists of 2 shear walls. The lateral load resisting capacity of the building in the North-South direction comes mainly from the shear walls. The total height of the building is 48 feet (14.6 m) as each floor has the same height of 12 feet (3.66 m).

All of the components of the building; columns, beams, and shear walls are modeled as reinforced concrete elements in the IDARC model. The bases of all of the columns and shear walls are assumed fixed in all degrees of freedom. The weight of the building is assumed evenly distributed to the joints of the beams and columns as nodal weights. A stick model with four nodal masses was generated to represent the mathematical model of the building. The mass of one half of the building is lumped at these four nodes with each node representing one floor of the building.

Nonlinear Time History Analysis
In order to evaluate the efficiency and accuracy of the FEMA process, a nonlinear time history analysis was performed on the shear wall model to provide a comparison basis. The ground excitation input used in the nonlinear time history analysis was the El Centro 1940 NS earthquake, a record of 20 seconds with an interval of 0.02 seconds. The peak acceleration of the ground motion is 0.348g. A response spectrum of 5% damping has been generated from this time history record and used in the response spectrum analysis. The viscous damping of 5% used in the response spectrum analysis was modeled as mass proportional damping in the time history analysis. An integration time interval of 0.005 seconds was used to ensure that the responses of high frequency modes were not missed from the result. The result shows that the maximum displacement at the roof is 4.75 inches (12.1 cm). A comparison of the results of the time history analysis with the results from the FEMA process is discussed below.

A series of runs were executed to calculate the magnitude of the El Centro Earthquake that would cause the maximum floor drift ratio to reach 0.75%, the FEMA 273 allowable drift ratio. This is because the time history

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analysis is nonlinear; thus interpolation is not applicable. After seven tries, the closest answer to the target is 71.55% (0.249g), at which the maximum floor drift ratio is 0.69%. With a slight change of the magnitude of the earthquake (i.e., 0.0005g, from 71.55% to 71.69%), it was observed that the floor drift ratio jumps up from 0.69% to 0.83%.

Analysis of the Shear Wall Model by FEMA 273

To demonstrate the FEMA 273 procedure, two analyses based on different input loading were completed. One loading was with the uniform load pattern and the other was with the modal load pattern. In the Uniform Loading Case the distribution of the lateral input loading applied to each floor of the model is proportional to the mass of that floor divided by the total mass of the structure. In the modal loading case, the distribution of the lateral loading at each floor level is consistent with the distribution of the inertia force of that floor obtained from a response spectrum analysis of the building. This analysis results in a predicted roof displacement equal to 4.36 inches (11.1 cm). It is also found that 92 % of the El Centro earthquake results in the FEMA allowable drift ratio (0.75 %).

Response Spectrum Analysis of Shear Wall Model

A response spectrum analysis was performed for the shear wall model. This is representative of the type of analysis that is performed using force based methods. The base shear predicted for the El Centro input motion is 6,301 k (28,028 kN).

Comparison Between Methods

Table 1 compares the time history analysis results to those obtained using the pushover analyses. Since the modal pattern results in the larger maximum floor drift, it is controlling and used to compare with the time history results. The displacement based method predicts a roof displacement of 4.36" (11.1 cm) or 8 % lower than the time history analysis. This comparison is quite good. For the floor drifts, the Modal Pattern loading case shows the same trend as the time history analysis; the floor drift gets larger as the height increases, and the third floor has the largest drift. It is also interesting to compare the predicted seismic capacity of the building using both the time history and displacement based methods. The capacity is based on an allowable drift of 0.75% as specified in FEMA 273. The seismic capacity of the building was found from the time history analysis to be defined with an El Centro response spectra anchored at 0.25 g. This compares with a displacement based predicted seismic capacity of 0.32g ZPA .

The pushover analysis indicated that the building could withstand 0.92 times El Centro. If earthquakes of this size were used in the response spectrum analysis, the base shear would be 0.92 * 6301 = 5,797 kips (25,777 kN). The capacity of the walls is set at V_y = 1,310 kips (5,827 kN). The response spectrum would predict the same capacity as the pushover analysis if the ductility factor of 5797 / 1310 = 4.4 were used. The Uniform Building Code allows an R factor (accounting for ductility, overstrength, and load redistribution effects) equal to 5 for a shear wall structure so that the pushover analysis gives slightly more conservative results for this case.

Table 1 Comparison of Nonlinear Time History Analysis with Pushover Analyses
(1in. = 25.4 mm)

	Nonlinear T.H.		Uniform Pattern		Modal Pattern	
Roof Disp.(in)	4.75		4.38		4.36	
Roof Drift (%)	0.82		0.76		0.76	
	Floor Drift (inches)	Drift Ratio (%)	Floor Drift (inches)	Drift Ratio (%)	Floor Drift (inches)	Drift Ratio (%)
Fourth Floor	1.40	0.97	1.05	0.73	1.15	0.81
Third Floor	1.41	0.98	1.08	0.75	1.18	0.82
Second Floor	1.41	0.98	1.08	0.75	1.14	0.79
First Floor	0.77	0.54	1.17	0.81	0.88	0.61

DIABLO CANYON TURBINE BUILDING

The Diablo Canyon turbine building is selected for the second case study comparing results obtained using the nonlinear time history and displacement based methods. This is selected because it is a nuclear power plant structure for which complete nonlinear time history analyses are available. These analyses are available for two different seismic input levels such as would be required for a seismic margin study. It is also of interest since the nonlinear effects include both material nonlinearity and geometric nonlinearity (gaps).

A probabilistic evaluation of the Diablo Canyon turbine building was performed [5] during the plant licensing reviews. The objective of that evaluation was to determine the probability of failure for several levels of severe earthquake inputs. The study developed a simple model of the building that characterized its performance through displacements that were likely to cause collapse. Nonlinear load – deflection curves were defined for each element of the model. A suite of 25 seismic motions, defined with response spectra, was then selected from actual earthquake records recorded at sites that have similar geologic formations as found at the Diablo Canyon site. These records were scaled to obtain any required magnitude of input motions. The dynamic analyses were performed using 25 time histories scaled so that the average (over the 3 cps to 8.5 cps frequency range) spectral accelerations were 3 g's and 6 g's.

Nonlinear dynamic response analyses were then performed to evaluate the peak model displacements for each of the 25 seismic input motions scaled to a common average spectral acceleration (averaged over the 3 cps to 8.5 cps frequency range). A statistical analysis was performed on the 25 predicted displacements to obtain median and standard deviation estimates of the displacements. A comparison of this displacement data with likely element failure displacements resulted in a prediction of the probability of failure for each earthquake level.

Two models, designated A and B, are used for the displacement based analyses. Model A is identical to the one used in the original Diablo Canyon study [5]. Model B is shown in Fig. 2. The two elements of the operating floor diaphragm for Model A are combined into a single element for Model B with two rigid links used to connect the center of the operating floor to the gap elements around the turbine.
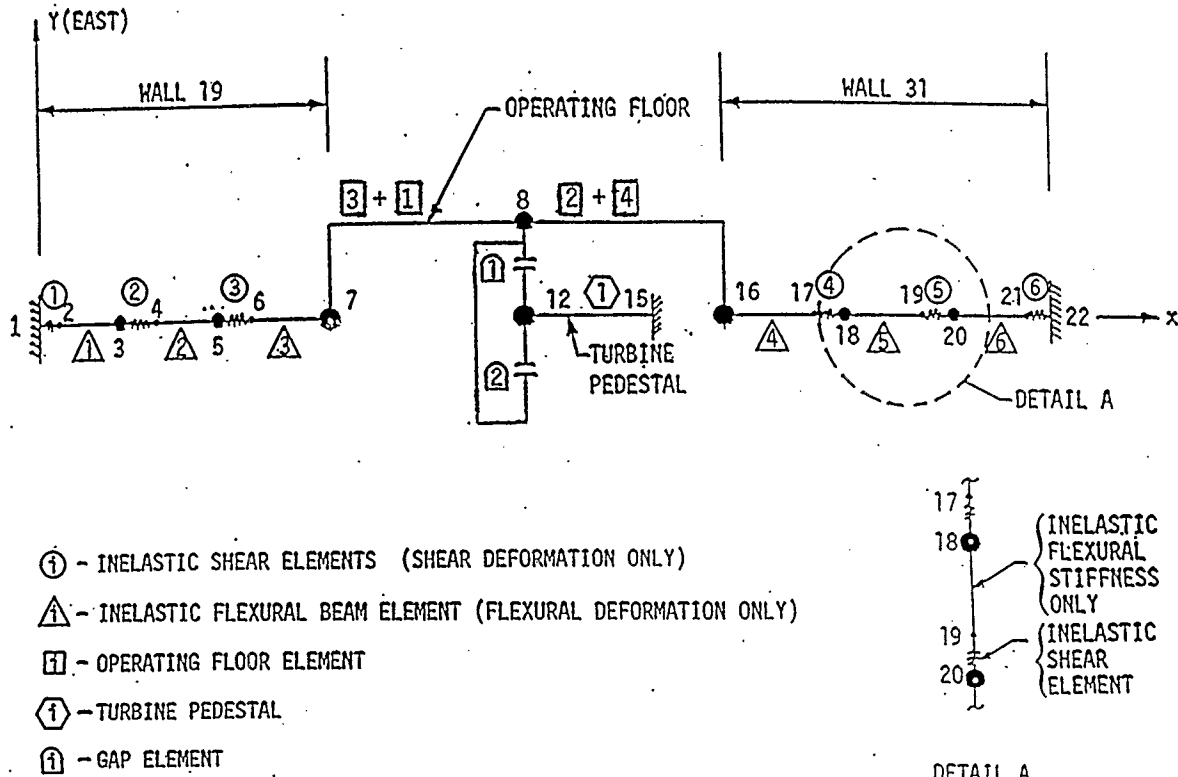

Figure 2 Diablo Canyon Turbine Building Model B

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A displacement-based analysis (FEMA 273) is performed for this structure and the results compared with those obtained from the time history methodology used in Ref. 5. Median model characteristics are used and the input seismic motion is defined with the median response spectra for the 25 input motions used in the Ref. 5 study. These predictions are then compared with the median results obtained from the force based probabilistic analyses.

Comparison of Time History and Displacement Based Results

The displacement results obtained with the displacement-based method and the time history methods are compared in this section. The time history methods developed log normal distributions for the displacements. The error between the two is normalized with respect to the log standard deviation and is defined as:

$$E = ABS \, [\ln (D_{fema} / D_m)] / \beta_D$$

Where,

D_{fema} = displacement based prediction
D_m = median of time history prediction
β_D = log standard deviation for time history analysis

The results of the time history analyses are combined with the results of the displacement based analyses to show the differences between the two sets of results, with summaries given in Tables 2 and 3 for the 3 g and 6 g cases respectively.

Table 2 Differences Between Forced Based and Displacement Based Analyses for 3g Input
(1 in. = 25.4 mm)

Location	Top of Wall 19	Top of Wall 31	Operating Floor	Turbine
Dm	0.537"	0.704"	3.252"	2.579"
Bd	0.662	0.624	0.417	0.3
Model A Dfema	0.44"	0.432"	7.08"	1.92"
Model A - E	0.3	0.78	1.87	0.98
Model B Dfema	0.90"	1.39"	6.60"	2.77"
Model B - E	0.78	1.09	1.7	0.24

Table 3 Differences Between Forced Based and Displacement Based Analyses for 6g Input
(1 in. = 25.4 mm)

Location	Top of Wall 19	Top of Wall 31	Operating Floor	Turbine
Dm	3.522"	4.922"	8.574"	6.227"
Bd	0.587	0.541	0.412	0.415
Model A Dfema	2.88"	3.08"	14.16"	6.77"
Model A - E	0.34	0.87	1.28	0.2
Model B Dfema	3.26"	3.46"	13.20"	11.91"
Model B - E	0.13	0.65	1.05	1.56

It can be seen that the agreements between the time history and displacement results are not very good and that the displacement-based method generally over-predicts the response. The predictions between the two methods are closer for the response at the top of the shear walls (Wall 19 and 31) than for the operating floor diaphragms or for the turbine pedestal. For the 3 g input motion, the Model A predictions of the shear wall displacements are better than the Model B predictions, but the reverse is true for the operating floor diaphragm and turbine pedestal displacements. The Model B predictions are better than the Model A predictions for the 6g input except for the turbine pedestal deflection.

There are probably four reasons for the large differences:

- The dynamic characteristics of the building change dramatically when the gaps close since the turbine is so massive. The basic idea behind the displacement-based approach is that an "equivalent" static analysis can be performed to represent the dynamic response. It is unlikely that a single static model could adequately model the response of a system that changes so dramatically as the gaps close and open.

- The load path changes from the turbine pedestal supporting the building to the building supporting the turbine pedestal as the operating floor diaphragm and then the turbine pedestal reach their respective yield loads. It is also unlikely that this could be modeled with a single equivalent static model.

- The displacement-based methodology was developed for cases where the building has softening stiffness characteristics. Some aspects of the turbine building problem have the opposite characteristic. After the operating floor diaphragm yields it is partially supported from the turbine pedestal. This support results in a nonlinear increase in building stiffness.

- The turbine pedestal and shear wall structure behave as uncoupled systems during a large part of the response. The displacement based method attempts to model this with a single degree of freedom system which cannot capture the dynamic characteristics of both in a single model.

SUMMARY

Current seismic analysis methods are force based. Member forces rather than displacements are compared with acceptance limits. The specific methods used to perform these analyses for those problems where inelastic action is anticipated, and acceptable, are either a nonlinear time history analysis or an elastic analysis with member seismic loads reduced by ductility factors to account for inelastic action. Analyses presented in this report compare results from displacement based analyses with those obtained from each of the other methods.

The following conclusions are found from these comparisons:

1. The displacement based method gives results comparable to the nonlinear time history analysis for the shear wall building where there are only material nonlinearities.

2. The use of ductility factors with a linear response spectrum analysis gives results which are comparable to those obtained from either the nonlinear time history analysis or the displacement based method.

3. The displacement based method does not give results which are comparable to the nonlinear time history analysis for the Diablo Canyon turbine building where both material and geometric nonlinearities (gaps) are included. This conclusion is probably due to the strong effect of the gaps on the system response.

Based on these conclusions, the following recommendations are given:

1. The displacement based approach is not likely to be used for the design of nuclear power facilities since the current acceptance criteria are force based and all responses are required to remain in the linear elastic range. While the displacement based approach could be used in this area, it offers no advantages over the force based methodologies currently in use for evaluating design adequacy. Therefore, it was concluded that there is no need to revise nuclear plant acceptance criteria for seismic design to address displacement based methods.

2. Seismic margin studies for nuclear facilities are based on displacement acceptance criteria (usually inelastic deformation limits corresponding to a given probability of failure). The displacement based analysis is directly applicable to problems where only material nonlinearities occur. The displacement based methods offer two advantages over the nonlinear time history analysis. First, the displacement based approach (or pushover analysis) is much simpler and less time consuming to use than the time history analysis. This simplification is likely to reduce the potential for erroneous results and to increase the number of engineers that have the background required to perform the analysis. Second, the method greatly reduces the effort required to produce structural fragility curves from that which is required using time history analyses. A single static nonlinear analysis is required to produce the pushover curve. Solutions for different probabilities of failure are then obtained by evaluating the criteria earthquake required for the structural displacement to reach the acceptance criteria associated with the probability of failure. Since many nonlinear time history analyses would be required to generate the fragility curve, a displacement based approach has potential for cost savings and is likely to become popular. It is recommended that guidance be developed for the use of the displacement based approach for seismic margin/fragility analysis.

3. Additional studies need to be performed for nuclear power plant structures with both material and geometric nonlinearities to further define the scope of the problems that can be treated with the displacement based methods before it would be possible to establish sufficient guidance for their use.

4. If displacement based methods of FEMA 273 are to be applied on a wide scale to nuclear facilities, efforts must be undertaken to develop appropriate "C" coefficients and drift limits that are consistent with the importance of the structure. Alternative forms of displacement based methods are also possible. The primary steps in any displacement based method are to predict the expected displacement of the structure to earthquakes of interest accounting for nonlinear characteristics of the structure, and to evaluate the details of the structure to determine whether sufficient ductility is available to accommodate the displacement pattern with adequate margin. A method, similar to FEMA 273, could be developed specifically for nuclear structures.

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