Pushover Analysis: Why, How, When, and When Not to Use It

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Abstract

The static pushover analysis is becoming a popular method of predicting seismic force and deformation demands for the purpose of performance evaluation of existing and new structures. If applied thoughtfully and with good judgment, it will provide much useful information that cannot be obtained from elastic static or dynamic analysis procedures. However, the method is based on many assumptions that may provide misleading results in some cases. Thus, the pushover needs to be viewed as an engineering tool with much potential, but with pitfalls that have to be recognized in order to avoid gross misinterpretations. This paper points out some of these pitfalls, but its main focus is a general discussion on the pushover's advantages (why to use it), background and implementation (how to use it), and applications (when to use it).

Introduction

In most circles concerned with the development of seismic design procedures there is general consensus that presently employed elastic design and analysis methods cannot capture many important phenomena that control seismic performance of structures in severe earthquakes. The search for a more rational and transparent design process has been with us for some time and will remain to be an issue of much debate and controversy for years to come. Design will always be a compromise between simplicity and reality; with the recognition that reality is very complex and uncertain in imposed demands and available capacities, and simplicity is a necessity driven by limited fees and the limited ability to implement complexity with commonly available knowledge and tools. The pushover analysis is by no means a final answer to our design/analysis problems, but it is a significant step forward by giving consideration to those inelastic response characteristics that will distinguish between good and bad performance in severe earthquakes.

The static pushover analysis is a partial and relatively simple intermediate solution to the complex problem of predicting force and deformation demands imposed on structures and their elements by severe ground motion. The important terms are **static** and **analysis**. *Static* implies that a static method is being employed to represent a dynamic phenomenon; a representation that may be adequate in many cases but is doomed to failure sometimes. *Analysis* implies that a system solution has been created already and the pushover is employed to evaluate the solution and modify it as needed. In the writer's opinion this is an important limitation. The pushover does not create good solutions, it only evaluates solutions. If the engineer starts with a poor lateral system, the pushover analysis may render the system acceptable through system modifications, or prove it to be unacceptable, but it will not provide a safe path to a good structural system.

The pushover is part of an evaluation process and provides estimates of demands imposed on structures and elements. Evaluation implies that imposed demands have to be compared to available capacities in order to assess acceptability of the design. It is fair to say that at this time deformation capacities cannot be estimated with great confidence, not for new elements and less so for elements of existing structures. Recognizing this limitation, the task is to perform an evaluation process that is relatively simple but captures the essential features that significantly affect the performance goal. In this context, accuracy of demand prediction is desirable, but it may not be essential since neither seismic input nor capacities are known with accuracy. The inelastic pushover analysis, which is the subject of this paper, serves this purpose provided its limitations and pitfalls are fully recognized.

Pushover Analysis Procedure

The process is to represent the structure in a two- or three-dimensional analytical model that accounts for all important linear and nonlinear response characteristics, apply lateral loads in predetermined patterns that represent approximately the relative inertia forces generated at locations of substantial masses, and push the structure under these load patterns to specific target displacement levels. The internal forces and deformations computed at the target displacement levels are estimates of the strength and deformation demands, which need to be compared to available capacities.

A target displacement is a characteristic displacement in the structure that serves as an estimate of the global displacement experienced by the structure in a design earthquake associated with a specified performance level. A convenient definition of target displacement is the roof displacement at the center of mass of the structure

The evaluation of a lateral load resisting system is based on an assessment of capacities and demands of important performance parameters. Such parameters include global drift, interstory drift, inelastic element deformations (either absolute or normalized w.r.t. a yield value), deformations between elements or components, and component and connection forces (for components and connections that cannot sustain inelastic deformations). The types of parameters and the acceptance criteria depend on the performance level to be evaluated.

A simple example of a pushover analysis is illustrated in Figure 1. The shown 2-dimensional frame could represent the lateral load resisting system for a steel perimeter frame structure. Gravity loads are applied to the structure first, and lateral loads are then applied in an incremental fashion. In simple cases the analysis can be performed as a series of elastic analyses in which, for instance, points where the bending strength has been reached are treated as hinges in the application of additional lateral loads. Bilinear or multi-linear load-deformation diagrams need to be incorporated if strain hardening or softening are important characteristics of element response. The analysis continues beyond the target displacement, $\delta_{t,t}$, resulting in a base shear versus roof displacement response of the type shown in the figure. In most cases it will be necessary to perform the analysis with displacement rather than load control since the target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. At the target displacement level the member forces and deformations (e.g., the plastic hinge rotation at point 1) are compared to available capacities for performance evaluation.

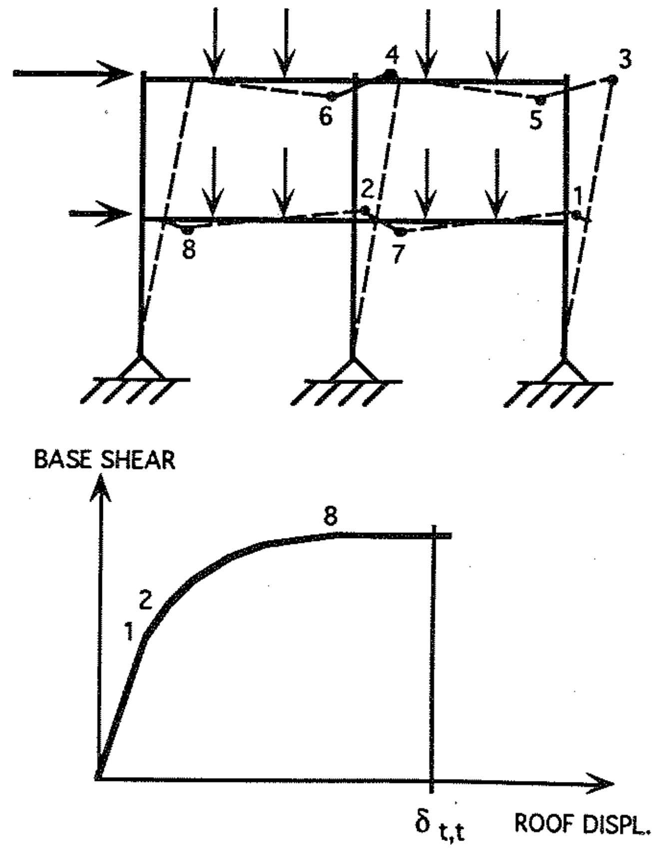


Fig. 1. Illustration of a Pushover Analysis

Advantages of Pushover Analysis

The advantage of the pushover is that it applies equally to the evaluation and retrofit of existing structures as to the design of new ones. A comprehensive evaluation of a lateral system would require the execution of a series of nonlinear time history analyses of the structure subjected to a representative suite of earthquake ground motions. This should be the emphasis for evaluation procedures of the future. At this time this approach is deemed to be unfeasible except for a few special cases. Recognizing this limitation, the task is to perform an evaluation process that is relatively simple but more realistic and more comprehensive than one based on a linear elastic analysis of the structure. The static pushover analysis fulfills this objective. The method can be viewed as an analysis process that accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that can no longer be resisted within the elastic range of structural behavior.

The pushover is expected to provide information on many response characteristics that cannot be obtained from a linear elastic static or dynamic analysis. The following are examples of such response characteristics:

- Force demands on potentially brittle elements, such as axial force demands in columns, force demands on brace connections, moment demands on beam-to-column connections, shear force demands in deep reinforced concrete spandrel beams and in unreinforced masonry wall piers, etc.
- Estimates of deformation demands for elements that have to deform inelastically in order to dissipate the energy imparted to the structure by ground motions.
- Consequence of strength deterioration of individual elements on the behavior of the structural system.
- Identification of critical regions in which the deformation demands are expected to be high and that have to become the focus of thorough detailing.
- Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range.
- Estimates of interstory drifts, which account for strength or stiffness discontinuities and may be used to control damage and evaluate P-delta effects.
- Estimates of global drift, which may be used to assess the potential for pounding.
- Verification of completeness and adequacy of load path, considering all elements of the structural system, all connections, stiff nonstructural elements of significant strength, and the foundation system.

The last item is perhaps the most important one, provided the pushover analysis incorporates all elements, whether structural or nonstructural, that contribute significantly to lateral load distribution. For instance, load transfer across connections between ductile elements can be checked with realistic forces; the effects of stiff partial-height infill walls on shear forces in columns (short columns) can be evaluated; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements, can be estimated.

Clearly, these benefits come at the cost of additional analysis effort, associated with incorporating all important elements, modeling their inelastic load-deformation characteristics, and executing incremental inelastic analysis, preferably with a three-dimensional analytical model. At this time, with few exceptions, adequate analytical tools for this purpose are either very cumbersome or not available. But several good tools are under development since the demand for the pushover analysis has been established, primarily through the recent pre-publication of the FEMA documents on "NEHRP Guidelines for the Seismic Rehabilitation of Buildings" (FEMA 273 and FEMA 274, 1996). These documents, which are the result of a 5-year development effort, include also extensive recommendations for load-deformation modeling of individual elements and for acceptable values of force and deformation parameters for performance evaluation.

The pushover is most useful for the evaluation at performance levels that are associated with large inelastic deformations (e.g., collapse prevention level). The method is applicable and useful, however, for evaluation at any performance level at which inelastic deformations will occur.

Background to Pushover Analysis

The static pushover analysis has no rigorous theoretical foundation. It is based on the assumption that the response of the structure can be related to the response of an equivalent single degree of freedom (SDOF) system. This implies that the response is controlled by a single mode, and that the shape of this mode remains constant throughout the time history response. Clearly, both assumptions are incorrect, but pilot studies carried out by several investigators (e.g., Saiidi and Sozen, 1981, Fajfar and Fischinger, 1988, Miranda 1991, Lawson et al., 1994) have indicated that these assumptions lead to rather good predictions of the maximum seismic response of multi degree of freedom (MDOF) structures, provided their response is dominated by a single mode.

The formulation of the equivalent SDOF system is not unique, but the basic assumption common to all approaches is that the deflected shape of the MDOF system can be represented by a shape vector $\{\Phi\}$ that characterizes the elastic and inelastic response of the structure. Accepting this assumption and defining the relative displacement vector $\mathbf{X}$ of an MDOF system as $\mathbf{X} = \{\Phi\}x_t$ (x_t = roof displacement), the governing differential equation of an MDOF system can be written as

$$\mathbf{M}\{\Phi\}\ddot{x}_t + \mathbf{C}\{\Phi\}\dot{x}_t + \mathbf{Q} = -\mathbf{M}\{1\}\ddot{x}_g \qquad (1)$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices, $\mathbf{Q}$ denotes the story force vector, and $\ddot{x}_g$ is the ground acceleration.

If we define the equivalent SDOF displacement x^* as

$$x^* = \frac{\{\Phi\}^T \mathbf{M}\{\Phi\}}{\{\Phi\}^T \mathbf{M}\{1\}} x_t \qquad (2)$$

and pre-multiply Eq. 1 by $\{\Phi\}^T$, and substitute for x_t using Eq. 2, we obtain the following differential equation for the response of the equivalent SDOF system:

$$M^*\ddot{x}^* + C^*\dot{x}^* + Q^* = -M^*\ddot{x}_g \qquad (3)$$

where M^*, C^*, and Q^* denote the properties of the equivalent SDOF system and are given by

$$M^* = \{\Phi\}^T \mathbf{M}\{1\} \qquad (4)$$

$$Q^* = \{\Phi\}^T \mathbf{Q} \qquad (5)$$

$$C^* = \{\Phi\}^T \mathbf{C}\{\Phi\} \frac{\{\Phi\}^T \mathbf{M}\{1\}}{\{\Phi\}^T \mathbf{M}\{\Phi\}} \qquad (6)$$

Presuming that the shape vector $\{\Phi\}$ is known, the force-deformation characteristics of the equivalent SDOF system can be estimated from the results of a nonlinear incremental static analysis of the MDOF structure, which usually produces a base shear (V) - roof displacement (x_t or δ_t) diagram of the type shown with solid lines in Fig. 2.

In the capacity spectrum method, which is not discussed here in detail, the properties of the equivalent SDOF system (the "capacity spectrum") are determined at this stage, and the target displacement is estimated by intersecting this capacity spectrum with a modified demand spectrum that is intended to account for inelastic response through a period shift and increased viscous damping. The capacity spectrum properties (spectral acceleration S_a and spectral displacement S_d) are obtained from the V-δ_t diagram of the structure through the transformations

$$S_a = \frac{V / W}{\alpha_1} \qquad (7)$$

$$S_d = \frac{\delta_t}{(PF_1)\phi_{1,t}} \qquad (8)$$

where α_1 and PF_1 are, respectively, the modal mass coefficient and participation factor for the first natural mode of the structure, and $\phi_{1,t}$ is the roof level amplitude of the first mode. Discrete values for S_a and S_d are determined at points at which the V-δ_t diagram shows evident kinks, with the shape vector $\{\Phi\}$ being updated to correspond to the first mode shape at the displacement level δ_t. The reader is referred to the literature (e.g., Mahaney, et al., 1993) for more details on this method.

The subsequent discussion focuses on the procedure implemented in FEMA 273, which utilizes spectral information for elastic and inelastic SDOF systems (see list of references) to estimate the target displacement without requiring the computation of updated shape vectors and modal properties. The procedure requires determination of global strength and stiffness quantities of the equivalent SDOF system, and accounts for dynamic and hysteretic response characteristics through inelastic spectra and a series of modification factors.

In order to identify nominal global strength and displacement quantities, the multi-linear V-δ_t diagram shown in Fig. 2 needs to be represented by a bilinear relationship that defines a "yield" strength, V_y, an effective "elastic" stiffness, $K_e = V_y/\delta_{t,y}$, and a hardening (or softening) stiffness, $K_s = \alpha K_e$ for the structure. Some judgment may be needed to define these properties. The simplified bilinear base shear - roof displacement response curve is needed to define the properties of the equivalent SDOF system.

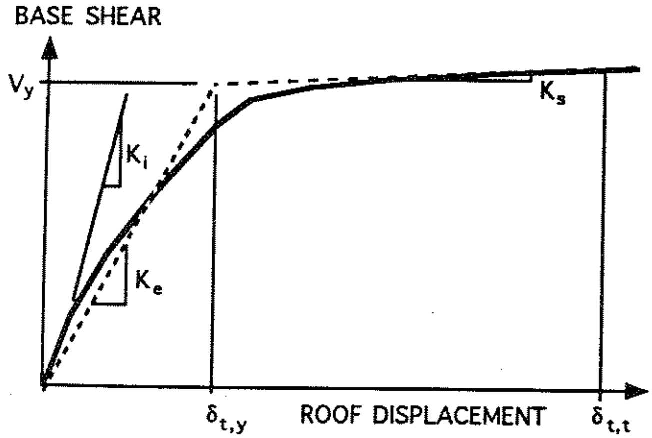


Fig. 2. Base Shear - Roof Displacement Response of MDOF Structure

The yield value of the base shear, V_y, and the corresponding roof displacement, $x_{t,y}$ ($\delta_{t,y}$ in Fig. 2), are used together with Eqs. 2 and 5 to compute the force - displacement relationship for the equivalent SDOF system as follows:

$$x_y^* = \frac{\{\Phi\}^T M\{\Phi\}}{\{\Phi\}^T M\{1\}} x_{t,y} \tag{9}$$

and

$$Q_y^* = \{\Phi\}^T Q_y \tag{10}$$

where Q_y is the story force vector at yield; i. e.,

$$V_y = \{1\}^T Q_y \tag{11}$$

The initial period of the equivalent SDOF system, T_{eq}, can be computed as

$$T_{eq} = 2\pi \left[\frac{x_y^* M^*}{Q_y^*} \right]^{1/2} \tag{12}$$

The strain hardening ratio α of the V-δ_t relationship of the MDOF structure defines the strain hardening ratio of the equivalent SDOF system.

The basic properties of the equivalent SDOF system are now known. The roof displacement of the structure, x_t, is related to the equivalent SDOF displacement, x^*, by means of Eq. 2. Thus, the target displacement can be found if the displacement demand of the equivalent SDOF system can be estimated for the design earthquake. For inelastic systems the SDOF displacement demand needs to be obtained from inelastic spectra as is discussed later. The utilization of inelastic spectral demand information requires the estimation of the ratio of elastic strength demand to yield strength of the equivalent SDOF system, usually referred to as the R-factor. Since inelastic spectra are usually obtained for unit mass systems, it is convenient to divide Eq. 3 by M^* to obtain the differential equation of the unit mass equivalent SDOF system:

$$\ddot{x}^* + \frac{C^*}{M^*}\dot{x}^* + \frac{Q^*}{M^*} = -\ddot{x}_g \tag{13}$$

Equation 13 describes the response of a unit mass SDOF system with period T_{eq} and yield strength $F_{y,eq}$ given as

$$F_{y,eq} = Q_y^*/M^* \tag{14}$$

[If the first mode shape is used for $\{\Phi\}$, and if the story force vector Q is based on the first mode shape, then Eq. 14 and Eq. 7 become equivalent.]

If an elastic design spectrum is given, the elastic strength demand of the unit mass equivalent SDOF system can be computed as

$$F_{e,eq} = S_a(T_{eq}) \tag{15}$$

where $S_a(T_{eq})$ is the spectral ordinate of the elastic acceleration spectrum. The strength reduction factor R can then be obtained from the relationship

$$R = \frac{F_{e,eq}}{F_{y,eq}} = \frac{S_a(T_{eq})M^*}{Q_y^*} \tag{16}$$

The utilization of the R-factor for estimating inelastic displacement demands is discussed later.

Both the R-factor and the target displacement depend on the choice of the shape vector $\{\Phi\}$. Most of the investigators who have utilized the pushover in pilot studies have recommended the use of the normalized displacement profile at the target displacement level as shape vector. Since this displacement is not known a priori, an iteration process will have to be performed if this shape vector is selected.

The use of T_{eq} and of the aforementioned shape vector for estimating the properties of the equivalent SDOF system and the target displacement requires elaborate computations and time consuming iterations. Recognizing all the assumptions and approximations inherent in the pushover procedure, there is no good justification to be rigorous in the computations leading to the estimate of the target displacement, and accuracy can often be sacrificed for the sake of simplicity.

Sensitivity studies have shown that the difference between T_1 (first mode structure period) and T_{eq} is usually small and its effect on the target displacement can be neglected unless the design spectrum is very sensitive to small variations in period. Simplifications in the shape vector $\{\Phi\}$ should also be acceptable. The use of a shape vector corresponding to the deflected shape at the target displacement is only a recommendation and has no theoretical foundation. In all cases studied by the writer the use of a simple predetermined shape vector (such as the elastic first mode shape vector) resulted in good predictions of the target displacement. In fact, in extensive studies of soft story structures with uniform mass distribution over the height it was found that the use of a straight line shape vector gives better predictions of the roof displacement than the use of the shape vector corresponding to the deflected shape at the maximum displacement (Seneviratna, 1995). Employment of the aforementioned simplifications facilitates the computation of the equivalent SDOF properties and eliminates the need for iterations.

There are many additional considerations that will affect the accuracy of seismic demand predictions by means of a pushover analysis. These considerations have to do primarily with the estimate of the target displacement and the selection of load patterns that are supposed to deform the structure in a manner similar to that experienced in a design earthquake. Some of the important issues are discussed next.

Target Displacement

In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the MDOF domain through the use of a shape vector and Eq. 2. This assumption, which is always an approximation, can only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure.

Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape vector without regards to higher mode effects. Parameter studies (Seneviratna, 1995) have shown that for frame and wall structures with a first mode period of less than 2 seconds this assumption is rather accurate for elastic system and conservative (overestimates the MDOF displacement) for inelastic systems. The assumption was not checked for systems with longer periods since in the writers' opinion the pushover should not be employed in its present form for long period structures.

Incorporation of all important structural response characteristics in the prediction of the SDOF displacement demand implies the ability to represent the global load - deformation response of the structure by an equivalent SDOF system with appropriate hysteretic characteristics. For this purpose the simplified bilinear base shear - roof displacement diagram shown in Fig. 2 may serve as a skeleton, defining a yield level and an effective elastic and post-elastic stiffness. The skeleton alone does not necessarily define the hysteretic characteristics of the SDOF system. Depending on the structural system and material, the restoring force characteristics may be bilinear, or may exhibit stiffness degradation or pinching, or may even exhibit strength deterioration. If the displacement demand depends strongly on these characteristics, their incorporation in the equivalent SDOF model will be necessary.

In general, there are several steps involved in deriving the inelastic displacement demand for the equivalent SDOF model. The conversion of the structure's base shear - roof displacement response to the equivalent SDOF skeleton curve and the estimation of the associated R-factor (Eq. 16) have been discussed previously. Once the R-factor is known, the SDOF displacement demand can be computed, with due regard given to the hysteretic characteristics of the equivalent SDOF system. If the seismic input is represented by a time history record, the inelastic displacement demand can be computed directly through a time history analysis using the equivalent SDOF system with properly modeled hysteretic characteristics.

The purpose of a pushover analysis is rarely the prediction of demands for a specific ground motion. It is employed mostly as a design evaluation tool. For this purpose, the seismic input is usually represented by a smoothed elastic response spectrum rather than individual ground motion records. Thus, the inelastic displacement demand cannot be computed directly but needs to be deduced from spectral data and auxiliary information that accounts for differences between the elastic and inelastic displacement demand. There is no unique way to achieve this objective. The following approach is only one of several feasible ones; but it has the advantages of being based on transparent physical concepts and being able to take advantage of seismic demand information for inelastic SDOF systems generated by the writer and many others.

Given the design spectral acceleration S_a, the elastic SDOF displacement demand can be computed as $(T^2/4\pi^2)S_a$. This displacement becomes the base line for predicting the inelastic displacement demand, which needs to be accomplished with due consideration given to the yield strength and hysteretic characteristics of the equivalent SDOF system. Both the effects of yield strength and hysteretic characteristics can be accounted for through cumulative modification factors applied to the elastic displacement demand. Thus, the target displacement can be expressed by the equation

$$\delta_{t,t} = (\prod C_i)\frac{T_1^2}{4\pi^2}S_a \tag{17}$$

where $\prod C_i$ is the product of modification factors. Modifications presently implemented in FEMA 273 are discussed next.

Modification factor for MDOF effects, C_0. This factor transforms the equivalent SDOF displacement to the building roof displacement. It is defined by Eq. 2, i.e.,

$$C_0 = \frac{\{\Phi\}^T M\{1\}}{\{\Phi\}^T M\{\Phi\}} \tag{18}$$

Thus, C_0 depends on the selected shape vector $\{\Phi\}$. If the elastic first mode shape is used for $\{\Phi\}$, then C_0 becomes the first mode participation factor PF_1. FEMA 273 permits the use of values for C_0 taken from a table. These values are based on a straight line mode shape and uniform mass distribution over the height. The tabulated values should not be used if masses vary considerably over the height.

Modification factor for yield strength, C_1. In general, the inelastic displacement of the SDOF system will differ from the elastic one, dependent on the extend of yielding and the period of the system. The SDOF yield strength is related to the elastic strength demand by the previously discussed R-factor (see Eq. 16). Many studies have been performed that relate this R-factor to the ductility demand μ, resulting in R-μ-T relationships for different hysteresis systems and site soil conditions (see Miranda & Bertero, 1994, for a summary of various results). As a representative example, Fig. 3 shows R-μ-T relationships obtained from a statistical study of bilinear SDOF systems subjected to a set of 15 rock and stiff soil (soil type

S_1) ground motion records (Nassar & Krawinkler, 1991). The results of most studies reported in the literature are based on bilinear nondegrading SDOF systems with positive or zero strain hardening. In general, using zero strain hardening ($\alpha = 0$) will result in the smallest R-factor (or largest displacement demand).

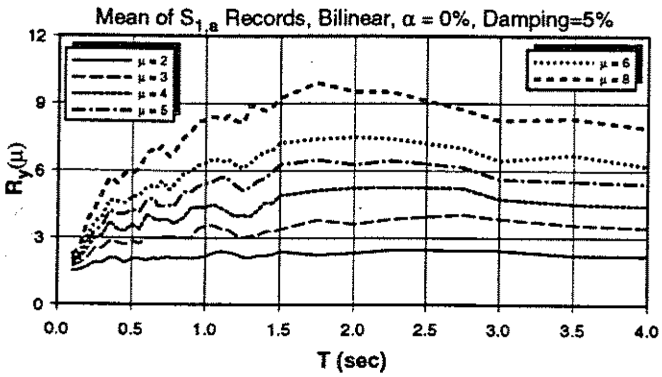


Fig. 3. R-μ-T Relationships for Bilinear SDOF Systems on Soil Type S_1

For the base case, i.e., elastic-plastic SDOF systems, FEMA 273 assumes a simplified R-μ-T relationship of the type shown in Fig. 4. T_o is the period associated with the transition from the constant acceleration region to the constant velocity region of the design spectrum. Since $C_1 = \delta_{in}/\delta_{el} = \mu/R$, the following values for C_1 are obtained

$$C_1 = 1.0 \qquad \text{for } T_1 \geq T_o \qquad (19\text{-a})$$

$$C_1 = [1.0 + (R-1)T_o/T_1]/R \qquad \text{for } T_1 < T_o \qquad (19\text{-b})$$

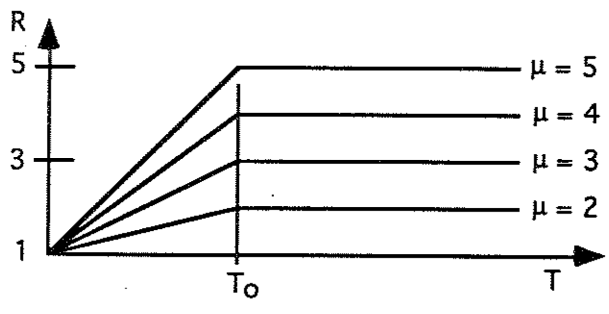


Fig. 4. Simplified R-μ-T Relationships

To simplify matters further, FEMA 273 permits also the use of the following approximation for R:

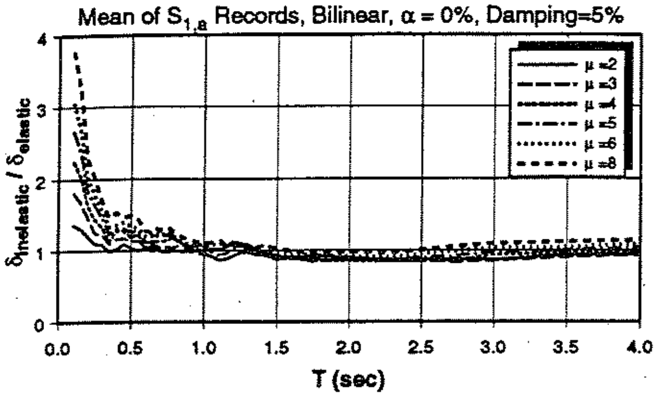
$$R = \frac{S_a}{V_y/W}\frac{1}{C_0} \qquad (20)$$

The quantity R, as defined in Eq. 16, differs from this approximation. If the first mode shape is used for $\{\Phi\}$, and if the story force vector Q is based on the first mode shape, then Eq.16 will give the following result:

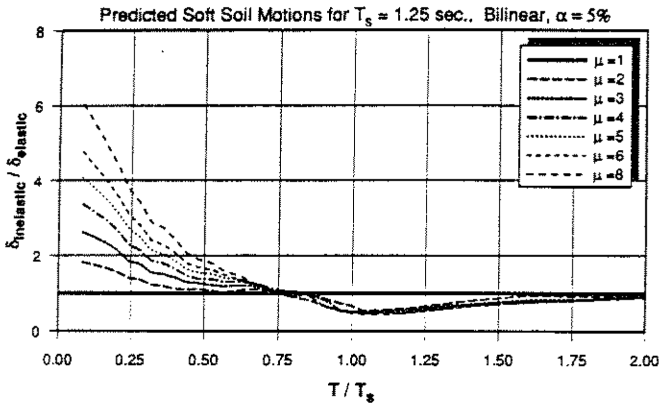
$$R = \frac{S_a}{V_y/W}\alpha_1 \qquad (21)$$

where α_1 is the modal mass coefficient as used in Eq. 7. There are good arguments to use, for simplicity, the already computed factor C_0 instead. The above "ifs" do not apply in general and one can argue at length about the appropriateness of the use of α_1. A sensitivity study performed by the writer has shown that the use of $1/C_0$ is reasonable for a great variety of shape vectors. Moreover, a reasonable approximation in the estimate of R has little impact on the final target displacement, particularly since FEMA 273 caps C_1 at a value of 2.0.

This cap, and the use of the approximate R-μ-T relationships shown in Fig. 4 for soft soil sites may be of larger impact on the accuracy of the target displacement prediction than the approximation in Eq. 20. Capping C_1 at 2.0 is purely judgmental. R-μ-T relationships for soft soils differ from those for stiff soils, resulting in ratios of δ_{in}/δ_{el} that are very sensitive to the predominant soil period. Typical ratios obtained from numerical simulations are shown in Fig. 5. Figure 5(a) shows the mean of the ratio δ_{in}/δ_{el} for the 15 soil type S_1 records on which Fig. 3 is based. These ratios are represented rather accurately by the C_1 values given in Eqs. 19 (except for $\delta_{in}/\delta_{el} > 2.0$). Figure 5(b) shows the mean ratio for 10 generated records in a soft soil with a predominant period T_s = 1.25 sec. (Rahnama & Krawinkler, 1993). The ratio δ_{in}/δ_{el} for soft soil records follows a different pattern than for rock and firm soil records. It was found that this ratio is very sensitive to the soil period T_s, but, once normalized to this period, the period dependence of δ_{in}/δ_{el} follows closely the pattern shown in Fig. 5(b) regardless of the value of T_s.



(a) Rock and Stiff Soil Records



(b) Soft Soil Records, T_S = 1.25 sec.

Fig. 5. Ratio of Inelastic to Elastic SDOF Displacement Demands

<u>Modification factor for stiffness degradation and strength deterioration, C_2</u>. Degradation of the unloading or reloading stiffness may have an effect on the inelastic displacement demand. The displacement amplification, compared to a bilinear nondegrading SDOF system, has been studied with the same 15 S_1 records referenced previously. It was found that stiffness degradation or pinching does not have a significant effect on the inelastic displacement demand except for very short period systems. This is illustrated in Fig. 6, which shows the displacement amplification for a severely pinched hysteresis model (reloading target strength is 25% of yield strength). This conclusion has to be viewed with caution since it is based only on SDOF studies and needs to be verified on MDOF structural systems.

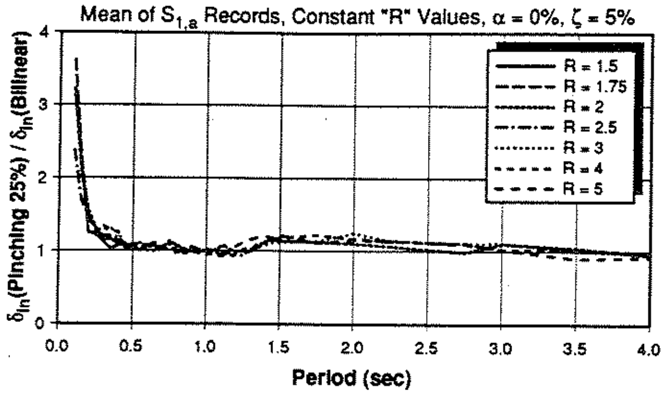
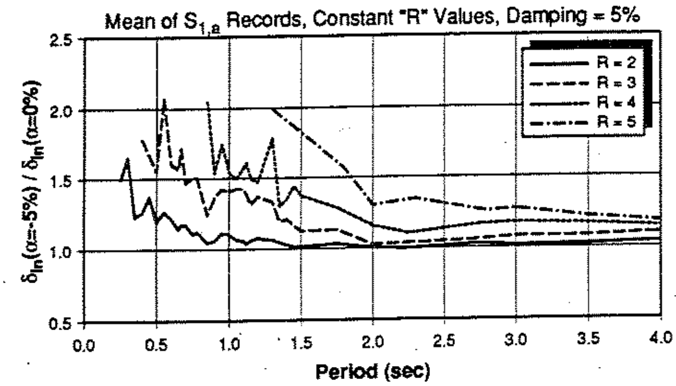


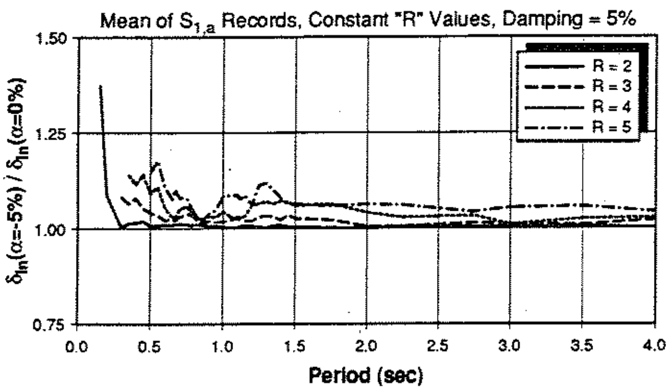
Fig. 6. Effect of Stiffness Degradation on Inelastic Displacement Demand

Strength deterioration, however, may have a significant effect on the inelastic displacement demand. Unfortunately there is no simple answer to the magnitude of its effect, which will depend strongly on the rate of deterioration and the strong motion duration of the ground motion. If a structure has the potential for significant strength deterioration, its effect on displacement demand may overpower most of the other modifications discussed here. More research on this subject is urgently needed. For the time being, FEMA 273 is accounting for stiffness degradation and strength deterioration by means of a table of C_2 factors, with values based on judgment and ranging from 1.0 to 1.5 dependent on the type of structural system and the performance level.

<u>Modification factor for P-delta effect, C_3</u>. Structure P-delta effect (caused by gravity loads acting on the deformed configuration of the structure) will always lead to an increase in lateral displacements. If P-delta effect causes a negative post-yield stiffness in any one story, it may affect significantly the interstory drift and the target displacement. Such a negative stiffness will lead to drifting of the displacement response (increase in displacements in one direction). Results of mean displacement amplifications for SDOF systems with 5% negative stiffness ($\alpha = K_s/K_e = -0.05$), obtained from the previously mentioned 15 S_1 records, are presented in Fig. 7. It can be seen that the amplification is large for bilinear systems with a short period and low strength (large R-factor). The amplification is much smaller for peak-oriented stiffness degrading systems (Fig. 7(b)).



(a) Bilinear Systems with 5% Negative Stiffness



(b) Peak-Oriented Stiffness Degrading Systems with 5% Negative Stiffness

Fig. 7. Effect of Negative Post-Yield Stiffness on Inelastic Displacement Demand

In a MDOF structure the extent of displacement amplification will depend on the ratio of the negative post-yield stiffness to the effective elastic stiffness, the fundamental period of the structure, the strength reduction factor R, the hysteretic load-deformation characteristics of each story, the frequency characteristics of the ground motion, and the duration of the strong motion portion of the ground motion. Because of the many parameters involved, the effect of P-delta on drifting of the seismic response (increase in target displacement) is difficult to describe with a single modification factor.

At this time it is not clear whether the SDOF displacement amplifications, as illustrated in Fig. 7, should be applied directly to the prediction of the MDOF target displacement. In most structures the negative stiffness is not mobilized before significant inelastic deformations have occurred. This decreases the displacement drifting compared to systems with bilinear response characteristics. On the other hand, it must be recognized that the negative stiffness represented in the global base shear vs. roof displacement response may not be representative of the negative stiffness existing in the critical story, which likely is at the bottom of the structure. P-delta is a story drift amplification problem and not a global drift amplification problem, and the P-delta effects are usually highest in the lowest stories in which the gravity loads are largest. Again, more research is needed to develop a reliable procedure that permits the incorporation of the MDOF story P-delta effect in the target displacement estimate.

Since dynamic P-delta effects may lead to a significant amplification of displacements, and may even lead to incremental collapse, they cannot be ignored even though inadequate knowledge exists to incorporate them accurately in the prediction of the target displacement. The following estimate of the displacement amplification, expressed by the modification factor C_3, is presently incorporated in FEMA 273:

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{2/3}}{T_1} \qquad (22)$$

where α is the ratio of post-yield stiffness to effective elastic stiffness. C_3 needs to be considered only if α is negative. This equation was obtained by expressing the displacement amplification for bilinear systems by an approximate equation and using half this value for C_3. The one-half compromise is rationalized as follows. First, most buildings behave more like stiffness degrading models than bilinear models, and second, in most buildings the negative stiffness is not developed until after significant inelastic deformations have occurred.

Other modification factors. There are several other effects that could be included as modifications to the target displacement. For instance, all previous modifications apply for systems with 5% viscous damping. The inelastic displacement demand needs to be modified if the effective viscous damping is judged to be significantly different from 5%. Figure 8 presents mean data (using the same 15 S_1 records) on the dependence of inelastic displacement demands on the percentage of critical damping, ξ, for bilinear systems with an R-factor of 3.0. The results are normalized with respect to systems with 5% damping. Results for other R-factors lead to the conclusion that the displacement ratios are rather insensitive to the R-factor and that the presented results can be used with reasonable accuracy for all R-factors.

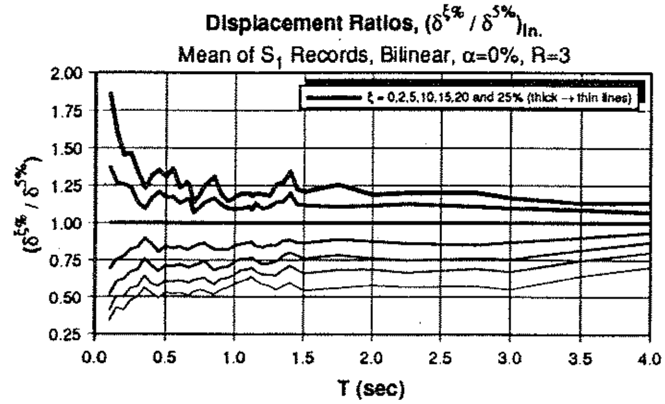


Fig. 8. Effect of Viscous Damping on Inelastic Displacement Demands

Nonlinear time history analysis of MDOF frame structures has shown that the sum of maximum interstory drifts is about 10% to 30% larger than the maximum roof displacement (Seneviratna, 1995). Since the purpose of a pushover analysis is a performance evaluation at the element level, it would be appropriate to increase the target roof displacement accordingly. Also, foundation uplift, torsional effects, and semi-rigid floor diaphragms are expected to affect the target displacement. At this time inadequate information exists to incorporate these effects in the prediction.

The following summary observations can be made with respect to the prediction of the target displacement for inelastic MDOF structures. The equivalent SDOF displacement demand can be estimated with good accuracy for standard conditions of ground motion spectra and restoring force characteristics. Greater approximations are involved if the design spectrum represents soft soil conditions or if the structural system experiences strength deterioration or significant P-delta effect. A good choice for the shape vector $\{\Phi\}$, which is used to convert the SDOF displacement demand into an MDOF target displacement, is the elastic first mode shape.

The prediction of the target displacement will usually be rather accurate for structures without significant strength or stiffness irregularities and without the formation of weak story mechanisms. Even in the latter cases the predictions will be good in most cases. But it should not be expected that the target displacement can be predicted with great accuracy for all possible cases. The writer must confess that he does not consider this a major drawback of the pushover analysis. A reasonable prediction of the displacement, which can be achieved in most practical cases, is believed to be adequate in the context of all the other approximations involved in this evaluation procedure. What is believed to be more important than accuracy is the explicit consideration of the different phenomena that may affect the displacement response, and an approximate evaluation of their importance. The data presented in Figs. 3 to 8 serves this purpose.

Lateral Load Patterns

For a realistic performance evaluation the load pattern selection is likely more critical than the accurate determination of the target displacement. The load patterns are intended to represent and bound the distribution of inertia forces in a design earthquake. It is clear that the distribution of inertia forces will vary with the severity of the earthquake (extent of inelastic deformations) and with time within an earthquake. If an invariant load pattern is used, the basic assumptions are that the distribution of inertia forces will be reasonably constant throughout the earthquake and that the maximum deformations obtained from this invariant load pattern will be comparable to those expected in the design earthquake. These assumptions may be close to the truth in some cases but not in others. They likely are reasonable if (a) the structure response is not severely affected by higher mode effects, and (b) the structure has only a single local yielding mechanism that can be detected by an invariant load pattern.

In such cases carefully selected invariant load patterns may provide adequate predictions of element deformation demands. Since no single load pattern can capture the variations in local demands expected in a design earthquake, the use of at least two load patterns that are expected to bound inertia force distributions is recommended. One should be a "uniform" load pattern (story forces proportional to story masses) which emphasizes demands in lower stories compared to demands in upper stories and magnifies the relative importance of story shear forces compared to overturning moments. The other could be the design load pattern used in present codes or, preferably, a load pattern that accounts for elastic higher mode effects, such as a load pattern derived from SRSS story shears.

Clearly, none of these invariant load patterns can account for a redistribution of inertia forces, which may occur when a local mechanism forms and the dynamic properties of the structure change accordingly. Thus, it is attractive to utilize adaptive load patterns that follow more closely the time variant distribution of inertia forces. Different suggestions have been made in this regard, including the use of story loads that are proportional to the deflected shape of the structure (Fajfar and Fischinger 1988), the use of SRSS load patterns based on mode shapes derived from secant stiffnesses at each load step (Tri-Service Manual), and the use of patterns in which the applied story loads are proportional to story shear resistances at the previous step (Bracci et al., 1995). At this time there is no consensus on the advantages of these adaptive load patterns, but there is no doubt that improved load patterns need to be developed in order to make a demand prediction by means of a static pushover analysis a more reliable process.

The writer believes that the load pattern issue is at this time the weak point of the pushover analysis procedure. The use of invariant patterns may lead to misleading predictions, particularly for long period structures with localized yielding mechanisms. These problems will be discussed later in the section on limitations of the pushover analysis. The suggested adaptive patterns may improve the prediction in some cases, but none have proven to be universally applicable. Further improvements of the pushover procedure need to focus on the load pattern issue.

Implementation Issues

In the implementation of the pushover analysis the questions are: how to model the structure and its elements, how to execute the analysis, what load pattern(s) to apply, how far to push, and what is to be evaluated. Load patterns and target displacement issues (how far to push) have been discussed. This section provides a discussion on other implementation issues.

Modeling of Structure and Elements. In general, a building needs to be modeled and analyzed as a three-dimensional assembly of elements and components. Two-dimensional modeling is acceptable only if torsional effects are small or 3-D effects can be accounted for separately. All elements that are part of the lateral or gravity load system and have significant rigidity or limited deformation capacity need to be represented in the analytical model. Elements of the foundation system need to be included in the model if they are expected to reach their strength value, or contribute significantly to the flexibility of the foundation-structure system, or limit the force transfer to the soil at or below the target displacement level. Uplift and other nonlinear response characteristics of foundation elements need to be represented in the analytical model.

The elastic and inelastic strength and stiffness characteristics of each element need to be modeled to the extent that their important effects on the response of the building are reasonably represented. Experimental and analytical information as well as sound judgment are needed in the selection of suitable element force-deformation models. In general, simple models are preferred. The simplest element model is a bilinear model that represents an effective elastic stiffness, the element strength, and a "hardening" stiffness that may be positive (for strain hardening elements), zero, or negative (for elements that deteriorate in strength). The term effective stiffness implies that cracking is accounted for in the elastic stiffness.

If elements have limited (or no) inelastic deformation capacity but their deterioration in strength does not necessarily lead to unacceptable performance (e.g., spandrel beams in a reinforced concrete coupled shear wall), then their force-deformation model should incorporate the post-deterioration range, as is illustrated in Fig. 9. If deterioration in strength leads to unacceptable performance, the need for refined modeling of the type shown in Fig. 9 does not exist because performance evaluation at the target displacement level will disclose whether or not the acceptable deformation (δ_1) has been exceeded.

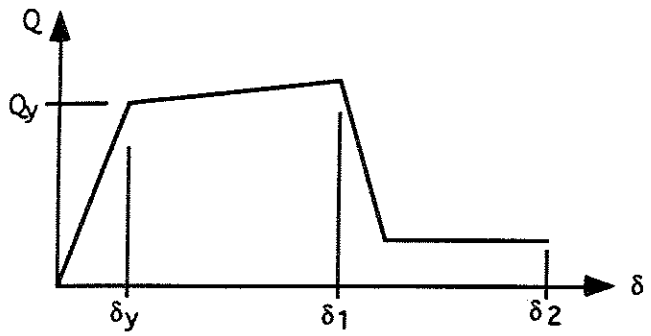


Fig. 9. Refined Element Model

Analysis Procedure. The analysis consists of the application of gravity loads and a representative lateral load pattern, and an incremental event-by-event analysis in which the load pattern is applied in increments corresponding to stiffness changes in each structural component. Thus, the first load step consists of an elastic analysis of the structure and scaling of the loads to a level that corresponds to the attainment of the first discontinuity in the force-deformation responses of all the elements (first "event"). For the next load increment the stiffness of that particular component is modified and another elastic analysis is performed, with the incremental loads again scaled to a level that corresponds to the attainment of the next discontinuity in the force-deformation response in any of the elements (second "event"). This process is continued until either unacceptable behavior is detected anywhere in the structure (e.g., column buckling, excessive strength deterioration in an important element) or the target displacement level is exceeded.

In a complex structure many "events" may have to be traced and the event-by-event strategy may become cumbersome and unnecessarily lengthy. A standard inelastic analysis with predetermined displacement increments, in which iterations are performed to balance internal forces, will achieve the same objective. The important issue is to determine the forces and deformations in all elements at the target displacement level for a capacity/demand evaluation.

Geometric nonlinearities (static P-delta effects) should be considered in the analysis, particularly if the chance exists that a mechanism develops and a negative post-mechanism stiffness may occur.

Determination of Component/Structure Acceptability. The pushover analysis provides information on force and deformation demands at target displacements that are associated with specified levels of performance. Performance evaluation consists of a capacity/demand evaluation of relevant parameters, such as the roof displacement (for pounding), interstory drifts (for damage control and P-delta control), inelastic deformations in elements and connections (for damage control and prevention of unacceptable deterioration), and internal forces (for prevention of overloads in potentially brittle elements and connections).

Acceptability needs to be based on capacity information. FEMA 273 provides extensive data on capacities in the material chapters. It is clear that at this time much judgment has to be used in establishing acceptability criteria, particularly if they are associated with deformation quantities. This may not be a major drawback since accurate predictions are desirable but not critical, particularly for elements that deteriorate in a gradual manner. What is more important is the realization that life safety hazards are caused primarily by brittle failure modes in components and connections that are important parts of the gravity and lateral load paths. Thus, the emphasis in performance prediction needs to be on

- verification that an adequate load path exists,
- verification that the load path remains sound at the deformations associated with the target displacement level,
- verification that critical connections remain capable of transferring loads between the elements that form part of the load path,
- verification that individual elements that may fail in a brittle mode and that are important parts of the load path are not overloaded, and
- verification that localized failures (should they occur) do not pose a collapse or life safety hazard, i.e., that the loads tributary to the failed element(s) can be transferred safely to other elements and that the failed element itself does not pose a falling hazard.

A thoughtful application of the pushover analysis will provide adequate answers in many cases. Exceptions will be discussed later. However, there are many unresolved issues that need to be addressed through research and development. Examples of the important issues that need to be investigated are:

- Incorporation of torsional effects (due to mass, stiffness, and strength irregularities)
- 3-D problems (orthogonality effects, direction of loading, semi-rigid diaphragms, etc.)
- Use of site specific spectra
- Cumulative damage issues, and, perhaps most important,
- Consideration of higher mode effects once a local mechanism has formed.

The simple conclusion is that much more work needs to be done to make the static pushover analysis a general tool applicable to all structures, which can be employed with confidence by engineers who are not necessarily experts in the evaluation of demands and capacities of structures that respond inelastically to earthquake ground motion.

A Pushover Case Study

Many papers are available in the literature that summarize pilot studies on the pushover analysis, as for instance, Saiidi and Sozen, 1981, Fajfar and Fischinger, 1988, Qi & Moehle, 1991, Miranda 1991, Lawson et al., 1994. The general conclusion of these studies is that the static pushover provides reasonable estimates of strength and deformation demands if the dynamic response is governed by the first mode. This appears to hold true even if severe strength or stiffness discontinuities in elevation exist.

The following discussion of a pushover case study is extracted from Lawson, Vance, and Krawinkler, 1994.

Four moment resisting steel frame structures, which vary in height from 2 to 15 stories and are designed according to minimum code standards, are employed in the referenced study. Results from the pushover analysis are compared with results obtained from inelastic time history analyses performed with seven ground motion records. In addition, equivalent SDOF systems are developed for the structural frames in order to evaluate methods of determining the target displacement for the pushover analysis.

The moment resisting frames used in this study are designed as the lateral load resisting elements of the 3 bay by 7 bay perimeter frame structure shown in Figure 10. The moment resisting frames have three bays, with all bays 30 feet wide and all stories 12 feet high. Frames with 2, 5, 10, and 15 stories are designed. The columns are assumed to be fixed at the base and spliced every two stories. All girders in each floor are constrained to be identical. The gravity loads include a dead load of 85 psf and a live load of 20 psf for the roof, and a dead load of 100 psf and live load of 50 psf for all floors. Exterior wall panels are assumed to have a weight of 25 psf. Based on tributary area, each frame is designed for a seismically effective dead weight of 850 kips at the roof and 1035 kips at all floors.

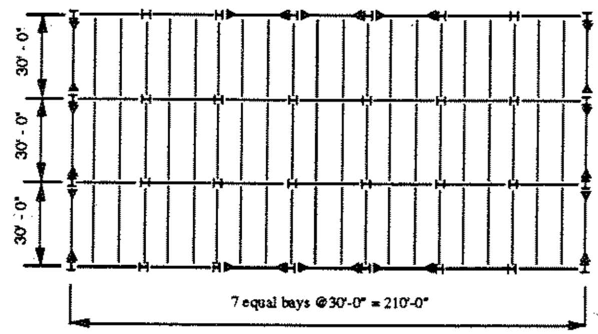


Fig. 10. Plan View of Frame Structure

Figure 11 shows normalized base shear - roof displacement diagrams for the four frames. These results are obtained from a pushover analysis using a load pattern based on the UBC vertical distribution of lateral forces. Significant column hinging occurs in all of the structures, and significant beam hinging occurs in all but the 2-story structures.

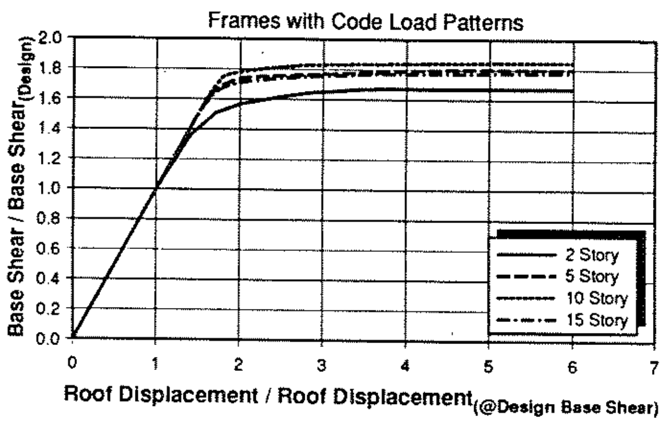


Fig. 11. Base Shear - Roof Displ. Diagrams

The diagrams are close to bilinear elastic-plastic since gravity load effects in these perimeter frames are small and zero strain hardening is assumed in the moment-rotation relationships. The overstrength is rather small in all cases because of the small gravity load effect and the tuning of stiffnesses to minimum code requirements.

<u>Analysis and Modeling Assumptions.</u> The lateral stiffness of the bare frames is very low and the first mode period is very long. In order to attract seismic forces that correspond to the design assumptions, the moment of inertia of all members is increased uniformly throughout the structure so that the fundamental period is equal to the original code design period ($T = 0.035h_n^{3/4}$).

The structural frames are analyzed statically and dynamically using a modified version of the DRAIN-2DX analysis program. Three different load patterns are applied in this study to assess the sensitivity of the results to variations in the lateral load distribution. The patterns include the UBC lateral load pattern (CODE), a uniform load pattern (UNFM), and a load pattern based on an SRSS combination of elastic modal properties of the frames assuming an ATC-3 ground motion spectrum for soil type S_1 (SRSS).

The dynamic analyses for each of the frames are performed using seven ground motion records, which are selected such that they represent expected variations of motions within the constraint that they should resemble, in average, motions whose elastic response spectrum is similar to the ATC-S_1 ground motion spectrum. All records are scaled to a peak acceleration value of 0.4 g.

<u>Deflection Profiles - Dynamic vs. Pushover.</u> Deflection profiles for the 2-story and 15-story frames are presented in Figures 12 and 13. Pushover results are plotted for each load pattern to assess the sensitivity of the analysis results to variations in the load distribution. The dynamic results are obtained at maximum roof displacement for two of the seven ground motions.

A comparison of the deflection profiles indicates good correlation for low-rise structures and poor correlation for tall structures in which higher mode effects are important. In addition, the static results for the 15-story frame are very sensitive to changes in the load pattern. The SRSS pattern provides a drastically different deflection profile that is governed by a two-story mechanism in stories 13 and 14. Neither one of the three patterns provide a good estimate of dynamic story drifts for the full height. The primary problems are higher mode effects, but it is doubtful that these effects can be blamed in all cases. Different load patterns may find different localized mechanisms in a structure, and it will depend on the frequency characteristics of the ground motion which, if any, of these localized mechanisms will be activated and amplified in an earthquake.

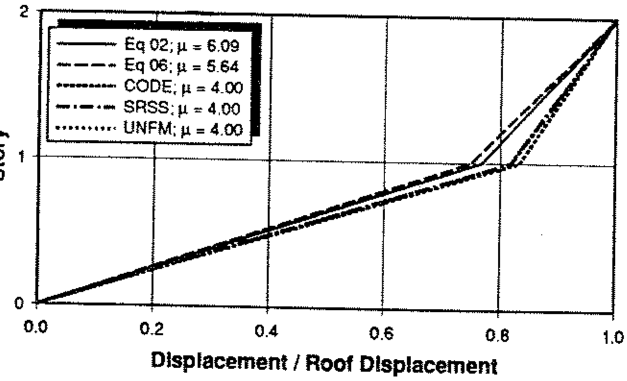


Fig. 12. Deflection Profiles, 2-Story Frame

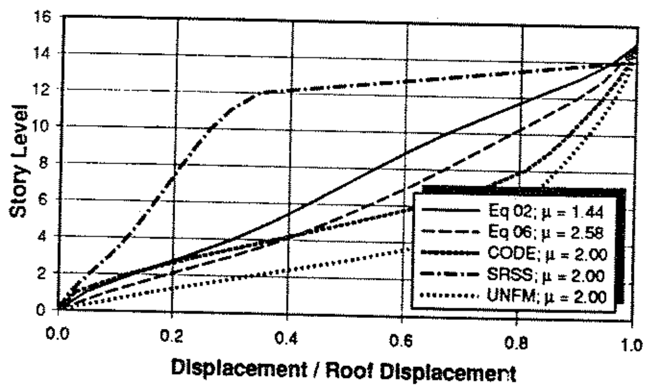


Fig. 13. Deflection Profiles, 15-Story Frame

<u>Interstory Drift - Dynamic vs Pushover.</u> Mean values of story ductility ratios obtained from seven dynamic analyses and from the corresponding pushover tests using the Code load pattern are presented in Fig. 14. The results show a good correlation between dynamic and pushover results for the 2 and 5 story frames, but poor correlation in the upper stories of the 10 and 15 story frames. Again, much blame can be laid on higher mode effects. It needs to be observed, however, that the inelastic components of story drift are relatively small in the

tall structures, and the correlation might improve under earthquakes that cause larger story ductilities.

Plastic Hinge Rotations - Dynamic vs Pushover. The maximum column plastic hinge rotation demands for the four frames are presented in Figure 15. The figure compares the mean of the maximum dynamic demand to the static demand (for the same roof displacement) using the code load pattern. The results are very similar to those presented in Fig. 14 for story ductility ratios. In the 2-story frame the dynamic as well as the pushover response are controlled by a first story mechanism caused by column hinging. Thus, the pushover analysis would clearly identify this mechanism and would provide a good estimate of dynamic deformation demands in the critical elements. This does not hold true for the demands on column (and beam) plastic rotations in the upper stories of the 10 and 15 story structures.

Target Displacement for Pushover Analysis. Target displacements are determined from the equivalent SDOF system discussed previously. Two approaches are used, one utilizing the shape vector associated with the target displacement (Method 1), the other using the approximate formulation given by Equation (17) (Method 2). In the latter case, C_0 is taken as $(PF)_1$, and all other C_i values are taken as 1.0. Estimates of roof displacements based on the two methods are computed for the seven ground motions, and the mean and COV of the ratio of roof displacements from the MDOF inelastic dynamic analyses and the SDOF predictions are presented in Table 1. Both methods provide good estimates for all four structures. Considering the significant discrepancies in local deformation demands obtained from the pushover analysis compared to dynamic analyses, the second, much simpler method is clearly adequate for the cases studied here.

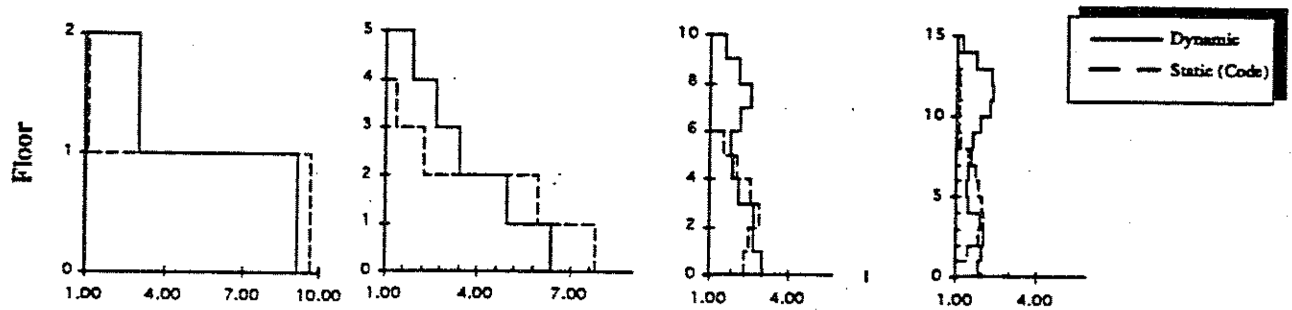


Fig. 14. Story Ductility Ratios Obtained from Pushover and Time History Analysis (Means)

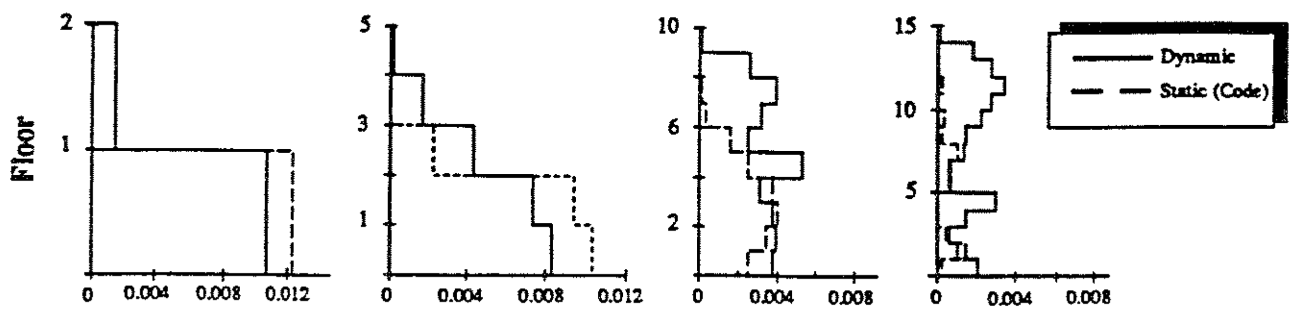


Fig. 15. Column Plastic Hinge Rotations Obtained from Pushover and Time History Analysis

Table 1. Ratio of MDOF/SDOF Roof Displacements

Method	2-story		5-story		10-story		15-story	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV
1	0.92	0.12	0.93	0.22	0.96	0.15	1.00	0.17
2	1.12	0.20	0.98	0.33	0.91	0.20	0.94	0.22

Limitations of Pushover Analysis

It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others. Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

Limitations are imposed also by the load pattern choices. Whatever load pattern is chosen, it is likely to favor certain deformation modes that are triggered by the load pattern and miss others that are initiated by the ground motion and inelastic dynamic response characteristics of the structure. A simple example is a structure with a weak top story. Any invariant load pattern will lead to a concentration of inelastic deformations in the top story and may never initiate inelastic deformations in any of the other stories. Thus, good judgment needs to be employed in selecting load patterns and in interpreting the results obtained from selected load patterns.

A comprehensive assessment of the accuracy of pushover demand predictions will require the execution of a great number of case studies for many different configurations. The writer urges the profession to perform a well planned series of case studies before the pushover analysis is made a general engineering tool. Until this is done, a preliminary assessment needs to be based on studies whose primary objective may not have been an evaluation of the pushover. A study that was concerned with a general evaluation of inelastic MDOF effects in frame and wall structures (Seneviratna, 1995) provides extensive data which can be used to draw conclusions on specific aspects of pushover demand predictions. Some of the conclusions drawn from this study are discussed next.

An evaluation of the analysis results from Seneviratna's study leads to the conclusion that the prediction of the target roof displacement by means of the previously discussed procedure is very good in all cases, whether the structures are frames with beam hinge mechanisms, column hinge mechanisms, or weak story mechanisms, or walls with flexural hinging at the base. But there are many other aspects which show clear limitations of the pushover analysis predictions. In general, these limitations do not apply to low-rise structures, but show up in increasing magnitude as the structures become taller, i.e., as the higher mode effects become more important.

An example of the expected accuracy of story drift predictions for a relatively tall frame structure (T = 1.22 sec.) is illustrated in Fig. 16. This structure is designed to have a straight line deflected shape and to yield simultaneously in every story under a static code load pattern. If a pushover analysis with the same load pattern is applied, the deflected shape will remain linear and the same story drift will be predicted in each story. Thus, the predicted story drift index δ_i/h_i will be the same in each story and will be equal to the global drift index δ_t/h_t. The latter value, obtained from dynamic analysis with a particular record, is indicated with a solid vertical line in Fig. 16. The individual story drift indices obtained from dynamic analysis with this record are shown with a stepped line. They vary significantly over the height and have a maximum value in the first story of more than twice the value predicted by the pushover analysis. This large discrepancy in prediction is caused by higher mode effects. It clearly demonstrates a basic limitation of the pushover analysis for taller structures.

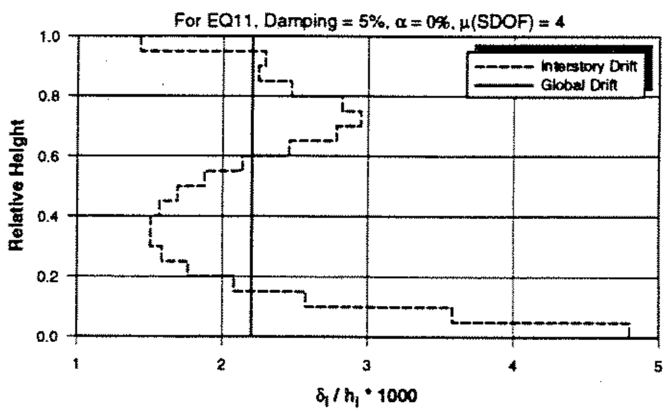


Fig. 16. Variation of Story Drift Over Height

The discrepancy in maximum story drift predictions between pushover analysis and dynamic analysis is a function of the period and yield strength of the structure (the latter is characterized in this study by μ(SDOF)). For the type of frame structures discussed in the previous paragraph this discrepancy is illustrated in Fig. 17, which shows mean values of the ratio of maximum story drift obtained from dynamic analysis (using the 15 S_1 records) to story drift predicted from a pushover analysis. For low-rise structures this ratio is close to 1.0 regardless of the strength of the structure, but for tall structures it increases up to a value of 3. This result reinforces the previously made statement that the pushover analysis will not provide good predictions for tall structures in which higher mode effects are important. This statement needs to be qualified since the pushover prediction accuracy depends on the selected load pattern. If an SRSS load pattern is used, the quality of prediction improves somewhat but not by a drastic amount. Adaptive load patterns have not been investigated in this study. On the positive side it can be said that the pushover prediction of maximum interstory drift was found to be reasonably accurate for structures of all heights if the deformation mode was a weak story mechanism in the first story.

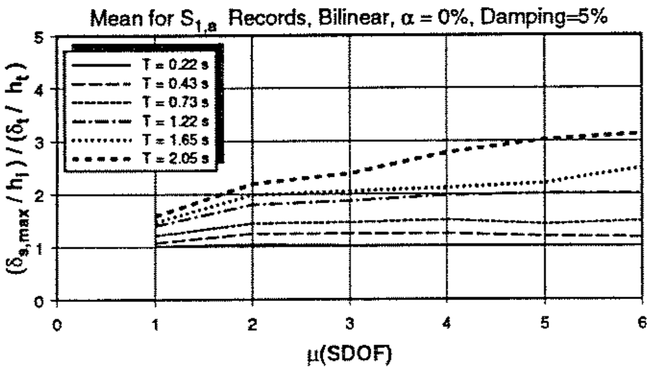


Fig. 17. Ratio of Max. Story Drift to Global Drift

An example that demonstrates other potential problems with the pushover analysis is that of multi-story wall structures modeled by a single shear wall. In these wall structures it is assumed that the bending strength of the wall is constant over the height, and that the shear strength and stiffness are large so that the behavior of the wall is controlled by bending. It is also assumed that no strain hardening exists once a plastic hinge has formed in the wall. A pushover analysis will predict hinging at the base of the wall for all rational load patterns. A mechanism exists once this single plastic hinge has formed, the wall will rotate around its base, and the lateral loads can no longer be increased. Thus, a pushover analysis will not permit propagation of plastic hinging to other stories and will predict a base shear demand that corresponds to the sum of lateral loads needed to create the plastic hinge at the base.

Nonlinear dynamic time history analysis gives very different results. For taller wall structures higher mode effects significantly amplify the story shear forces that can be generated in the wall once a plastic hinge has formed at the base. This is illustrated in Fig. 18, which shows mean values of base shear amplification, defined as the maximum base shear from dynamic analysis over pushover base shear causing plastic hinging at the base. The amplification depends on the period (number of stories) of the wall structure and on the wall bending strength (represented by μ(SDOF)). The diagram shows that the amplification of base shear demands may be as high as 5 for tall wall structures with reasonable bending strength (μ(SDOF) $\leq$ 4). This amplification implies that the base shear demand may be much higher than the base shear obtained from the lateral loads that cause flexural hinging at the base of the structure. Thus, wall shear failure may occur even though the pushover analysis indicates flexural hinging at the base.

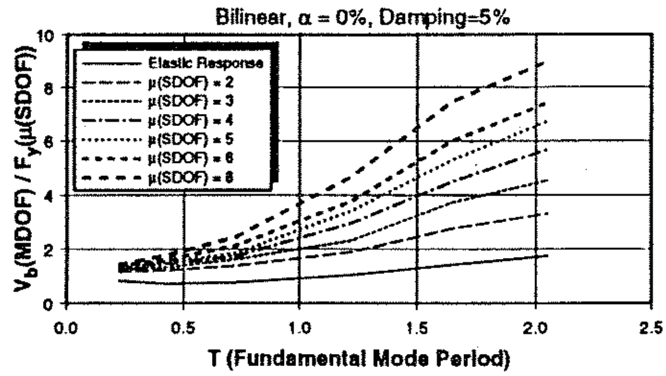


Fig. 18. Amplification of Base Shear Demand for Wall Structures

Nonlinear dynamic time history analysis also shows that flexural hinging in tall structures is not necessarily limited to the first story. It may propagate into other stories to an extent that depends on the period and flexural strength of the structure. This is illustrated in the story overturning moment envelopes presented in Fig. 19 for a wall structure with a period of 2.05 seconds. The moment envelope obtained from dynamic analyses is very different from that obtained from a code type load pattern (solid line). Thus, if such a code load pattern is used in the pushover analysis, a very misleading picture of the story overturning moment demand is obtained. The use of an SRSS load pattern does not change the behavior by much.

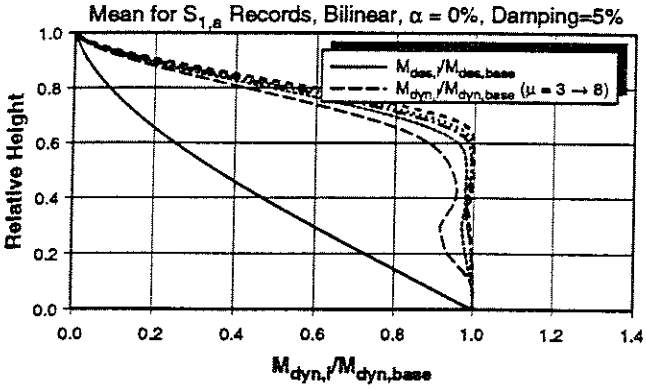


Fig. 19. Variation of Overturning Moment over Height (Wall Structure with T = 2.05 sec.)

No static pushover analysis could have predicted this behavior. This example shows that additional measures need to be taken in some cases to allow a realistic performance assessment. Such measures need to be derived from nonlinear dynamic analyses and need to be formalized to the extent that they can be incorporated systematically in a pushover analysis procedures.

Conclusions

There are good reasons to advocate the use of the inelastic pushover analysis for demand prediction, since in many cases it will provide much more relevant information than an elastic static or even dynamic analysis. But it would be counterproductive to advocate this method as a general solution technique for all cases. The pushover analysis is a useful but not infallible tool for assessing inelastic strength and deformation demands and for exposing design weaknesses. Its foremost advantage is that it encourages the design engineer to recognize important seismic response quantities and to use sound judgment concerning the force and deformation demands and capacities that control the seismic response close to failure. But it needs to be recognized that in some cases it may provide a false feeling of security if its shortcomings and pitfalls are not recognized.

On the positive side, a carefully performed pushover analysis will provide insight into structural aspects that control performance during severe earthquakes. For structures that vibrate primarily in the fundamental mode, the pushover analysis will very likely provide good estimates of global as well as local inelastic deformation demands. This analysis will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities, and overloads on potentially brittle elements such as columns and connections.

On the negative side, deformation estimates obtained from a pushover analysis may be very inaccurate (on the high or low side) for structures in which higher mode effects are significant and in which the story shear force versus story drift relationships are sensitive to the applied load pattern. This problem can be mitigated, but usually not eliminated, by applying more than one load pattern, including load patterns that account for elastic higher mode effects (e.g., SRSS load patterns). Perhaps most critical is the concern that the pushover analysis may detect only the first local mechanism that will form in an earthquake and may not expose other weaknesses that will be generated when the structure's dynamic characteristics change after formation of the first local mechanism.

In the writer's opinion the pushover analysis can be implemented for all structures, but it should be complemented with other evaluation procedures if higher mode effects are judged to be important. No single criterion can be established for this condition since the importance of higher mode effects depends on the number of stories as well as on the relative position of the modal periods with respect to the peak(s) and plateau(s) of the design spectrum. Candidates for additional evaluation procedures are, in order of preference, inelastic dynamic analysis with a representative suite of ground motions (probably unfeasible in most practical cases), and elastic dynamic (modal) analysis using the unreduced design spectrum and a suitable modal combination procedure (SRSS, CQC). The latter procedure will provide estimates of elastic demand/capacity ratios which need to be compared to acceptable values. Again, FEMA 273 provides many recommendations for this procedure.

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