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# Methods to Assess the Seismic Collapse Capacity of Building Structures: State of the Art

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**Abstract:** The collapses of building structures during recent earthquakes have raised many questions regarding the adequacy of current seismic provisions to prevent a partial or total collapse. They have also brought up questions as to how to determine the collapse safety margin of structures, what is the inherent collapse safety margin in code-designed structures, and how to strengthen structures to effectively augment such margin. The purpose of this paper is to present a comprehensive review of the analytical methods that are currently available to assess the capacity of building structures to resist an earthquake collapse, point out the limitations of these methods, describe past experimental work in which specimens are tested to collapse, and identify what is required for an accurate evaluation of the seismic collapse capacity of a structure and the safety margin against such a collapse. It is contended that further research is needed before the collapse capacities of structures and their safety margin against collapse may be evaluated with confidence.

**CE Database subject headings**: Structural analysis; Structural failures; Earthquake engineering; Seismic analysis; Structural safety; Structural stability; Collapse.

## Introduction

Buildings have partially or totally collapsed during the earthquakes in Valparaiso, Chile in 1985 (Wyllie et al. 1986; Leiva and Wiegand 1996); Mexico City in 1985 (Osteraas and Krawinkler 1988; Villaverde 1991); Armenia in 1988 (Wyllie and Filson 1989); Luzon, Philippines in 1990 (Schiff 1991); Guam in 1993 (Comartin 1995); Northridge, Calif. in 1994 (Hall 1994); Kobe, Japan in 1985 (Comartin et al. 1995; Nakashima et al. 1998); Kocaeli, Turkey in 1999 (Youd et al. 2000); Chi-Chi, Taiwan in 1999 (Uzarski and Arnold 2001; Huang and Skokan 2002); and Bhuj, India in 2001 (Jain et al. 2002). Many of these collapses occurred in older buildings designed with what is nowadays considered inadequate design standards. Others have been attributed to shoddy design and construction practices, which undoubtedly was the case in many of them. Several of the collapses, however, took place in buildings that were designed and constructed in accordance with modern seismic design principles (see Fig. 1.). An example is the 22-story steel frame tower of the Pino Suarez complex in Mexico City, which totally collapsed during the 1985 earthquake there (Osteraas and Krawinkler 1988; Ger et al. 1993). Therefore, as demonstrated by the fractures observed in the welded connections of modern steel buildings during the 1994 Northridge earthquake (Bertero et al. 1994) and the 1995 Hyogoken-Nanbu earthquake (Nakashima et al. 1998), it is possible that many of the observed collapses have been the result of deficiencies in our knowledge of the regional seismic hazard, the behavior of structural materials under dynamic loads, and the postelastic behavior of structural systems.

(a)

(b)

(c)

Fig. 1. Collapse of modern multistory buildings during the (a) 1985 Mexico City earthquake (National Geophysical Data Center, NOAA); (b) 1995 Kobe, Japan earthquake; and © 1999 Chi-Chi, Taiwan earthquake (National Information Service for Earthquake Engineering, University of California, Berkeley)

The aforementioned collapses raise many questions regarding the adequacy of current seismic provisions to prevent a partial or total collapse. As is well known, modern seismic provisions rely on a philosophy based on strong column-weak beam designs, story drift limits, and postelastic energy dissipation to guaranty the survivability of building structures in the event of a severe earthquake. However, this survivability has never been demonstrated analytically, experimentally, or by field studies. In fact, some investigators have put into question the validity of such an assumption; that is, the assumption that current code provisions are sufficient to avert the collapse of a structure if subjected to the extreme earthquake considered in its design. For example, Jennings and Husid (1968) note that if repeated excursions into the inelastic range of deformation of a structure occur in response to ground shaking, then the accumulated permanent deformations in the structure may render gravity forces the dominant forces and make the structure collapse by lateral instability. This effect, however, is not properly considered in modern design provisions. As noted by Bernal (1987), code provisions account for the P-$\Delta$ effect of gravity loads in terms of an inadequate extrapolation of results pertaining to static elastic behavior. In his study of the instability of buildings subjected to earthquakes, Bernal (1992) also notes that the safety of a structure against inelastic dynamic instability cannot be ensured by simply limiting the maximum elastic story drifts of the structure, a finding that has been recently confirmed by Williamson (2003). Similarly, Challa and Hall (1994), in their investigation of the collapse capacity of a 20-story steel frame, observe significant plastic hinging in the structure's columns and a likely collapse of the structure when subjected to a severe earthquake ground motion. This despite the fact that, as required by current code provisions, the flexural strength of its columns exceeds that of its beams at all joints. Interestingly enough, that observation has been recently corroborated by Medina and Krawinkler (2005). For, in a study to evaluate the strength demands of a large number of regular moment-resistant frames under different ground motions, they find that the potential for the formation of plastic hinges in columns is high for regular frames designed according to the strong column-weak beam requirements of recent code provisions. In a study similar to that of Challa and Hall, Martin and Villaverde (1996) also find that a two-story, two-bay frame structure collapses under a moderately strong ground motion even when the structure meets all the requirements from the 1992 AISC seismic provisions (AISC 1992). Likewise, Roeder et al. (1993) and Schneider (1993) observed in a study performed with an eight-story steel frame that the minimum design criteria required by the 1988 Uniform Building Code [International Conference of Building Officials (ICBO) 1988] are not enough to ensure that the inelastic story drifts of the structure are always below the maximum values considered in its design.

The collapses of modern structures during past earthquakes and the unproven adequacy of current design standards to prevent these collapses thus bring the question as to what is the actual safety margin of structures against an earthquake-induced collapse, a question that has acquired renewed importance as a result of the desire of the profession to move toward performance-based designs. As is well known, collapse prevention is one of the objectives of a performance-based design, and one of its promises is the assurance of an adequate safety margin against collapse under the expected maximum seismic load. Currently, however, as pointed out by several investigators (Hamburger 1997; Astaneh-Asl et al. 1998; Li and Jirsa 1998; Bernal 1998; Griffith et al. 2002; Esteva 2002), there are no established methods other than the collective judgment of code writers to calculate such a safety margin. Even more, it is not known if the available analytical tools are adequate enough to evaluate it in a reliable way since the process of collapse involves large deformations, significant second-order effects, and a complex material degradation due to localized phenomena such as cracking, local buckling, and yielding. What is worse, it appears that not even an established criterion exists to identify when and how a structure collapses under the effect of dynamic loads. The reason is that reaching an unstable condition a singular effective stiffness matrix, for example is not sufficient to infer the collapse of a structure under dynamic loads as unloading right after the structure reaches such an unstable condition may restore its stability (Araki and Hjelmstad 2000).

The objective of this paper is to review the methods that are currently available to assess the collapse capacities of structures under earthquake ground motions, point out the limitations of these methods, describe past experimental work in which specimens are tested to collapse, and identify what are the needs and challenges for an accurate evaluation of the aforementioned collapse capacities and the safety margin against such a collapse. For the purpose of this review, collapse is defined as the condition at which a structure, or a significant portion of it, is unable to support its gravity loads during a seismic excitation. It may be caused by the gradual deterioration in the stiffness and strength of some structural members when subjected to repeated reversed inelastic deformations (low-cycle fatigue) or the progressive accumulation of lateral drifts owing to the application of a series of large inelastic deformation cycles and significant P-Δ effects (incremental collapse).

## Collapse Assessment Methods

### Single-Degree-of-Freedom Models

Several methods have been suggested or used in research studies in which structures are modeled as single-degree-of-freedom systems to assess their collapse capacity. Takizawa and Jennings (1980), for example, employ an equivalent single-degree-of-freedom model to examine the ultimate capacity of ductile reinforced concrete frame structures under the combined action of strong ground shaking and gravity loads. The model exhibits a nondegrading trilinear force-deformation behavior and accounts for the destabilizing effect of gravity forces. The single-degree-of-freedom idealization results from linking the actual structure to an auxiliary rigid frame that makes the structure follow a specified deformation pattern. The actual structure is considered composed of structural elements formed with a rigid link and rotational springs at the two ends of the link. For the purpose of their study, they analyze a series of rigid and flexible structures under two sets of ground motions. One set corresponds to seven records from earthquakes with a 6.0–7.0 magnitude and a strong shaking duration ranging from 10 to 25 s. The other set consists of four short-duration, pulselike motions with high peak accelerations. They were recorded in the epicentral areas of magnitude 4.7–5.5 earthquakes epicentral distances ranging from 0.2 to 8.5 km, have a duration of 1 – 2 s, and have peak ground accelerations between 0.40 and 0.62 g. It is considered that a ground motion leads to a collapse if the gravity load and the induced displacements reduce to zero the restoring force in the system. For the class of structures and ground motions considered in their investigation, Takizawa and Jennings find that: (1) serious damage and collapse are significantly influenced by ground motion duration and that, in particular, short-duration ground motions have a low destructive capability, even when they have a high peak ground acceleration; (2) the stiff structures have a much wider margin between damage (ductility ratios greater than 3) and collapse than the flexible ones; and (3) the conventional single-parameter measures used to characterize ground motions are unsatisfactory to describe their destructive potential. It should be pointed out that the conclusion reached by Takizawa and Jennings in regard to the influence of ground motion duration in a structural collapse contradicts, as indicated later on, the findings reported by Ibarra and Krawinkler in their 2004 study. It is also opposite to the observations from other investigators (e.g., Hall et al. 1995; Krawinkler et al. 2003) that near-field ground motions, which can be characterized as short-duration, pulse-like motions, are capable of causing severe distress in long-period structures.

In a similar study, Bernal (1987) proposes an empirical formula in terms of ductility factor and a stability coefficient that characterizes the influence of gravity loads to compute an amplification factor that accounts for P-Δ effects in inelastic structures subjected to earthquakes. The formula is derived from amplification spectra generated by dividing the inelastic acceleration response spectrum ordinates obtained when gravity effects are included by those determined when these effects are not considered. A nondegrading elastoplastic single-degree-of-freedom model and an ensemble of four recorded earthquake ground motions are used in the generation of these amplification spectra. Based on the results from his study, he shows that the amplification formulas recommended by seismic codes are unconservative and that this unconservatism increases in direct proportion to the ductility factor considered in the design of the structure. Bernal (1992) also develops a simplified method to check the safety against dynamic instability of two-dimensional buildings. The method is based on the reduction of a multistory structure to an equivalent single-degree-of-freedom system and the derivation of statistical expressions to correlate the minimum base shear needed to prevent instability with some key structural and ground motion parameters. The statistical expressions are derived considering elastoplastic and stiffness-degrading systems, second-order effects, and an ensemble of 24 earthquake records from firm ground sites. The minimum base shear is defined as the yield lateral strength at which an out of bound structural response is imminent and is determined from the inspection of maximum response versus yield lateral strength plots. With the proposed method, the safety margin of a structure is estimated by dividing the actual base shear capacity of the structure by such a statistically determined minimum base shear. By comparing the results obtained using his method for a series of multistory structures against those attained considering the actual structures, Bernal finds that the safety against dynamic instability strongly depends on the assumed shape of the failure mechanism, i.e., the configuration of the deformed structure just before instability occurs. For buildings with a regular configuration, he identifies this failure mechanism using a static limit analysis with lateral forces proportional to the structure's story weights. Using an approach similar to the one employed in Bernal's 1987 study and based on the analysis of bilinear oscillators with different post-elastic to elastic stiffness ratios, MacRae (1994) proposes a method for considering P-Δ effects on oscillators with different hysteretic characteristics. From the results of his investigation, MacRae observes that the post-elastic to elastic stiffness ratio is a major parameter that affects the unidirectional accumulation of inelastic deformation in a system and, hence, the system's stability.

More recently, Williamson (2003) uses a simple model that explicitly considers the state of damage in a system to determine the response of a number of single-degree-of-freedom systems under various earthquake ground motions. The purpose is to study the role of damage accumulation and P-Δ effects on the response of inelastic systems. The model considered is a rigid column with a concentrated mass at the top and a rotational spring with a bilinear, damage-degrading moment-rotation relationship at the base. The earthquake vertical acceleration is considered explicitly in the analysis of the studied systems. Williamson finds that damage accumulation has a significant effect on the behavior of the analyzed systems. He also finds that P-Δ effects are important even for small values of the axial force and may lead to responses that are as much as five times greater than in the case when P-Δ effects are ignored. In agreement with the findings from other investigators (Jennings and Husid 1968; Takizawa and Jennings 1980), he additionally observes that a system's response is not affected significantly by the earthquake vertical acceleration. In a similar study, Miranda and Akkar (2003) propose an empirical equation to estimate the minimum lateral strength that is required to prevent the collapse by dynamic instability of single-degree-of-freedom systems. The systems considered exhibit a bilinear force-deformation behavior characterized by a post-yield branch with a negative slope. This force-deformation behavior is intended to represent the effect of geometric nonlinearities and strength degradation and leads to collapse when the restoring force in the system is reduced to zero. The empirical equation is derived through a nonlinear regression using the results from a statistical analysis with a large number of systems and an ensemble of 72 ground motions recorded on firm soils from California earthquakes. It is expressed as a function of natural period and post-yield stiffness as they find that the collapse strength of the systems considered varies significantly with these two parameters. Its application for any given system requires a reliable estimate of the post-yield stiffness of the system.

Finally, Adam et al. (2004) propose a procedure to consider P-Δ effects in multidegree-of-freedom structures through the use of an equivalent single-degree-of-freedom system with properties defined based on the results from a pushover analysis see the following subsection. The underlying assumption in this procedure is that the postyielding global stiffness obtained from a pushover analysis characterizes the global or local mechanism involved when the actual structure approaches dynamic instability. With it, the collapse capacity of a structure is determined through a series of dynamic analyses of the equivalent single-degree-of-freedom system under progressively increasing ground motion intensities. Collapse is assumed to occur when a small increment in the ground motion intensity produces a large increase in the structural response. The study is limited to nondegrading systems. To assess the accuracy of the procedure, Adam and coworkers compare the collapse capacities of two single-bay, nondeteriorating, multi-degree-of-freedom frame structures obtained using the proposed procedure to those determined analyzing the actual structures. An ensemble of 40 ground motions is used in this comparative analysis. They conclude that the global P-Δ effects of nondeteriorating structures may be predicted with good accuracy with the proposed procedure and that in most cases the predictions err on the conservative side.

## Nonlinear Static Procedure

The nonlinear static procedure, colloquially known as "pushover analysis," has become a standard method for estimating seismic deformation demands in building structures as well as their local and global capacities. As such, it has become a popular tool among practicing engineers for the evaluation of the safety of structures against an earthquake-induced collapse. It is introduced in FEMA Publication No. 273 (1997) and has been updated in FEMA Publication No. 356 (2000b). Its use is recommended for structures in which "higher mode effects" are not significant. If these higher mode effects are significant, then the procedure needs to be supplemented with a linear dynamic analysis.

In FEMA-356's nonlinear static procedure, a model of the structure is constructed considering explicitly the nonlinear force deformation behavior of its elements. Then, a base shear-lateral displacement relationship is established by subjecting this model to monotonically increasing lateral forces with a prescribed heightwise distribution until the displacement of a control node the center of mass of the building's roof exceeds a target displacement or the structure collapses. The target displacement is intended to represent the maximum displacement likely to be experienced by the structure under a selected seismic hazard level. The demands at this target displacement element forces, story drifts, or plastic hinge rotations are then compared against a series of prescribed acceptability criteria. These acceptability criteria depend on the construction material steel, reinforced concrete, etc., member type beam, column, etc., member importance primary or secondary, and a preselected performance level operational, immediate occupancy, life safety, or collapse prevention. A global collapse is assumed to occur whenever the base shear-lateral displacement curve attains a negative slope due to P-Δ effects and reaches afterward a point of zero base shear. Such point implies no lateral resistance and the inability of the structure to resist gravity loads.

The procedure lacks a rigorous theoretical foundation. It is based on the incorrect assumptions that the nonlinear response of a structure can be related to the response of an equivalent single degree-of-freedom system and that the distribution over the height of the structure of the equivalent lateral forces remains constant during the entire duration of the structural response. It neglects duration and cyclic effects, the progressive changes in the dynamic properties that take place in a structure as it experiences yielding and unloading during an earthquake, the fact that nonlinear structural behavior is load-path dependent, and the fact that the deformation demands depend on ground motion characteristics. In fact, in correlations with observed damage in several of the buildings damaged during the 1994 Northridge earthquake and comparisons with results from nonlinear time-history analyses, several investigators (Krawinkler and Seneviratna 1998; Chi et al. 1998; Kim and D'Amore 1999; Gupta and Kunnath 2000; Goel and Chopra 2004; Chopra and Goel 2004; Maison and Hale 2004) have found that the procedure does not provide an accurate assessment of building behavior. It may lead to gross underestimation of story drifts and may fail to identify correctly the location of formed plastic hinges. This is particularly true for structures that deform far into their inelastic range of behavior and undergo a significant degradation in lateral capacity. In recognition of these deficiencies, improved nonlinear static procedures have been proposed. Among these improved procedures are those that use a time-variant distribution of the equivalent lateral forces (Bracci et al. 1997; Gupta and Kunnath 2000) and those that consider the contribution of higher modes (Sasaki et al. 1998; Chopra and Goel 2002; Goel and Chopra 2005). These improved procedures lead to better prediction in some cases, but none has been proven to be universally applicable. It is doubtful, thus, that nonlinear static methods may be used reliably to predict the collapse capacity of structures and estimate their margin of safety against a global collapse.

## Step-by-Step Finite-Element Analyses and Detection of Abrupt Response Increase

Several investigators have used a step-by-step finite-element analysis and the detection of a sudden increase in lateral displacements to assess the collapse capacities of building structures. Ger et al. (1993) investigate the factors that led to the collapse of a 22-story steel building during the 1985 earthquake in Mexico City. To this end, they establish first realistic nonlinear hysteretic constitutive relationships for the individual members of the structure, which consisted of open-web girders, welded box columns, and H-shape diagonal braces. Then, using a three-dimensional finite-element model that includes the geometric stiffness of its structural elements, they analyze the building under the three components of the ground motion recorded at a station near the building's site during the earthquake. From this analysis, they observe that most of the building's longitudinal girders experience severe inelastic behavior, leading to ductility demands that exceeded the ductility capacities considered in their design. Because of the failed girders, a redistribution of forces takes place and local buckling occurs in the columns of the second to fourth floors. This local buckling, in turn, makes the affected columns lose their load-carrying capacity. In consequence, the building tilts and rotates, experiences significant drifts and P-Δ effects, and eventually collapses. Their results are in agreement with field observations of an almost identical building adjacent to the collapsed one, which was heavily damaged and was at the verge of collapse during the same earthquake. Perhaps a significant conclusion from this study is that it is possible to predict the collapse of a building using simplified models, provided realistic force-deformation relationships are adopted.

In like fashion, Challa and Hall (1994) investigate the nonlinear response and collapse of moment-resisting plane steel frames under the effect of severe earthquake ground motions. For this purpose, they analyze a 20-story frame designed to meet the Zone 4 requirements of the 1991 Uniform Building Code (ICBO 1991) under ground motions of two types. One is of the oscillatory type with long period components a scaled-up version of a record from the 1971 San Fernando earthquake, and the other is of the impulsive type (simulated displacement pulses intended to represent a near-field ground motion). Fiber elements are used to model the members of the frame and shear panel elements to model its beam-column joints. In addition, they consider the geometric stiffnesses of the elements and realistic stress-strain relationships for both the element fibers and the panel zones. They also update the deformed configuration of the structure at each time step in the step-by-step analysis of the structure. As such, strain-hardening, axial-flexural yield interaction, residual stresses, spread of yielding, P-Δ effects, and column buckling are accounted for in the analysis. However, stiffness and strength degradation is neglected, as is the effect of torsion and multicomponent ground motions. They find that the frame collapses under the oscillatory ground motion. That is, the frame reaches a state in which its lateral displacements grow quite large with each cycle of response and then a P-Δ instability unbound growth of lateral displacements occurs. The collapse takes place despite the fact that structural degradation is not accounted for, which, as Challa and Hall recognize, leads to unconservative results. Although stiffness and strength degradation is an important factor in the analysis of a structure when it is near collapse (Ibarra et al. 2005), it is likely that the consideration of this degradation in Challa and Hall's study would have simply led to an earlier occurrence of the collapse or a collapse under a less severe ground motion.

In a somewhat different study, Martin and Villaverde (1996) develop a methodology to (1) determine analytically if a structure excited by a given earthquake ground motion will experience a partial or total collapse, and (2) identify the structural elements that fail first and lead in consequence to the collapse of the structure. The method is formulated on the basis of a step-by-step, nonlinear, finite-element analysis and the examination at each integration step of the updated effective stiffness matrix of the structure. A partial or total collapse is detected when one or more of the pivots of the triangularized effective stiffness matrix become zero or negative. The part of the structure where the collapse is initiated i.e., region of local instabilities is located by identifying the structural nodes that correspond to the found zero or negative pivots. Martin and Villaverde apply their methodology to a steel cantilever beam and a two-story, three-dimensional steel moment-resisting frame to validate the adequacy of the method.

More recently, Mehanny and Deierlein (2001) propose a methodology to evaluate the likelihood of a structural collapse under earthquake effects using a component damage index developed by them. The methodology involves an inelastic time-history analysis with a finite-element model, the calculation of the aforementioned damage index for each of the components of the structure in accordance with the results of the analysis, the modification of the analytical model according to the calculated damage indices and the corresponding local damage effects, and a second-order analysis of this modified model under gravity loads alone. The ratio of the gravity load under which the structure reaches its global stability limit to the actual gravity load defines a global stability index that correlates collapse capacity to ground motion intensity.

## Incremental Dynamic Analyses

Incremental dynamic analyses have recently emerged as a powerful means to study the overall behavior of structures, from their elastic response through yielding and nonlinear response and all the way to global dynamic instability (FEMA 2000a). An incremental dynamic analysis involves performing a series of nonlinear dynamic analyses in which the intensity of the ground motion selected for the collapse investigation is incrementally increased until the global collapse capacity of the structure is reached. It also involves plotting a measure of the ground motion intensity e.g., spectral acceleration at the fundamental natural period of the structure against a response parameter demand measure such as peak story drift ratio. The global collapse capacity is considered reached when the curve in this plot becomes flat. That is, when a small increase in the ground motion intensity generates a large increase in the structural response. As different ground motions i.e., ground motions with different frequency content and different durations lead to different intensity versus response plots, the analysis is repeated under different ground motions to obtain meaningful statistical averages.

Incremental dynamic analyses have been proposed as early as 1977 (Bertero 1980) and have been studied extensively lately by several investigators. Vamvatsikos and Cornell (2002), for example, describe the method in detail, determine intensity-response curves for several structures (a 20-story moment-resisting steel frame, a 5-story braced steel frame, and a 3-story moment-resisting frame with fracturing connections), examine the properties of these response-intensity curves, and propose techniques to perform efficiently an incremental dynamic analysis and summarize the information obtained from the different curves that different ground motions produce. They observe that incremental dynamic analyses are a valuable tool that simultaneously addresses the seismic demands on structures and their global capacities. They also call attention to some unusual properties of the response-intensity curves such as nonmonotonic behavior, discontinuities, multiple collapse capacities, and their extreme variability from ground motion to ground motion. Recognizing that a complete incremental dynamic analysis requires an intense computational effort, Vamvatsikos and Cornell (2004) propose a practical method to perform it efficiently, and show, through a detailed example with a 9-story steel moment-resisting frame, how to apply it, how to interpret the results, and how to use these results within the framework of performance-based earthquake engineering. By exploiting the relationship between an incremental dynamic analysis and a static pushover analysis, they also develop (Vamvatsikos and Cornell 2005) a simplified method to estimate the seismic demands and collapse capacities of multi-degree-of-freedom structures through the use of an equivalent single-degree-of-freedom system.

Along the same lines, Ibarra and Krawinkler (2004) propose a methodology to evaluate the global collapse capacity of deteriorating frame structures under earthquake ground motions. The methodology is based on the use of a relative intensity measure, which they define as $S_aT_1/g/\gamma$, where $S_aT_1$ denotes the 5%-damping spectral acceleration at $T_1$, the fundamental period of the structure; $g$ is the acceleration due to gravity; and $\gamma$ is a base shear coefficient equal to $V_y$ /W, where $V_y$= yield base shear without P-$\Delta$ effects and W=weight of the structure. For structures with no overstrength, this intensity measure is equivalent to the reduction factor used in building codes for the analysis of yielding structures. The methodology is also based on the use of deteriorating hysteretic models to represent the cyclic behavior of the structural components under large inelastic deformations. These deteriorating models reproduce the important modes of deterioration observed in experimental tests. To evaluate collapse capacity, they increase the intensity measure until the intensity measure versus normalized maximum roof drift curve becomes flat. The relative intensity at which this curve becomes flat is then considered to be the collapse capacity of the structure. The evaluation is made using a probabilistic format to consider the uncertainties in the frequency content of ground motions and the deterioration characteristics of the structural elements. After implementing the deteriorating hysteretic models in the computer program Drain2DX (Prakash et al. 1993), Ibarra and Krawinkler use the proposed methodology to (1) conduct a parametric study with typical frame structures and study the influence of several parameters on the collapse capacity of structures; (2) develop collapse fragility curves, and (3) determine mean annual frequency curves. The structures considered in the parametric study are stiff and flexible single-bay frames with 3, 6, 9,12,15, and 18 stories with plasticity concentrated at the beam ends and the base of the columns, i.e., plastic hinges are permitted to form only at the beam ends and at the base of the columns. From the parametric study, they find that the two parameters that most influence the collapse of a structure are the slope of the postyield softening branch in the moment-rotation relationship of the yielding members and the displacement at which this softening begins. They also find that cyclic deterioration and hence ground motion duration is an important but not a dominant factor in the collapse of structures. This latter finding is in contrast with the findings from Takizawa and Jennings (1980), who, as described earlier, conclude that collapse is significantly influenced by ground motion duration.

In a similar study, Ayoub et al. (2004) investigate the effect of stiffness and strength degradation on the seismic collapse capacity of structures. To this end, they perform an incremental dynamic analysis of a single-degree-of-freedom structure with a natural period of 1.0 s considering three degrading constitutive models that explicitly account for the possibility of collapse. They also develop collapse fragility curves for such a system. The constitutive models consider strength softening branch with negative stiffness and degradation of strength and stiffness under cyclic loading. It is assumed that collapse occurs when the system's strength is reduced to zero. The three constitutive models considered are: (1) a bilinear model, (2) a modified Clough model, and (3) a pinching model. An energy-based criterion is used to define strength softening and stiffness and strength degradation. The study is carried out employing an ensemble of 80 ground motions and different levels of degradation. They find that for a given ground motion intensity the probability of collapse of systems with low degradation is similar to the one for systems with moderate degradation. In contrast, that probability is much higher for systems with severe degradation. Observing that collapse assessment requires hysteretic models capable of reproducing all the important modes of deterioration detected in experimental tests, Ibarra et al. (2005) similarly investigate the effect of stiffness and strength degradation on the seismic demands on structures as they approach collapse. For this purpose, they develop simple hysteretic models that include stiffness and strength deterioration properties; calibrate them utilizing experimental data from tests of steel, plywood, and reinforced concrete components; and determine employing alternatively some of the proposed models the response of a single-degree-of-freedom system with a natural period of 0.9 s and a damping ratio of 5% under an ensemble of 40 ground motions. They also scale the considered ground motions to various intensity levels and develop demand versus intensity curves to evaluate in each case the collapse capacity of the system. They conclude that deterioration is an overriding consideration in the seismic response analysis of a structure when the structure is near the limit state of collapse.

Finally, Lee and Foutch (2002) evaluate the performance of 20 steel frame buildings designed according to the recommendations of the 1997 NEHRP provisions (FEMA 1998) and considering prequalified post-Northridge beam-column connections. To this end, the buildings are subjected to a nonlinear time-history analysis under an ensemble of 20 earthquake ground motions. For a performance objective of collapse prevention, they scale the ground motions so as to have spectral accelerations that have a 2% probability of being exceeded in 50 years. The analytical models used in the analysis account for the ductility of beam-column joints, the panel zone deformations, and the influence of interior gravity frames. The behavior of the beam-column joints is characterized by a gradual strength degradation after a rotation of 0.03 rad. To evaluate the buildings' performance for the afore mentioned collapse prevention objective, they compare the maximum story drift demands against the buildings' drift capacities. Local and global drift capacities are considered in this comparison. The local drift capacities are the maximum drift angles the beam-column joints can sustain before losing their gravity load carrying ability. Based on the results from full-scale experiments, they consider a local drift capacity of 0.07 rad. The global drift capacities are determined by performing an incremental dynamic analysis for each of the buildings and plotting the corresponding maximum story drift ratio versus spectral acceleration curves. A building's global drift capacity is considered to be the maximum story drift ratio at which the maximum story drift ratio versus spectral acceleration curve becomes flat, or, alternatively, the maximum story drift ratio at which this curve reaches a slope equal to 20% of the slope in the elastic region of the curve. However, if this slope is not reached before a story drift ratio of 0.10 is attained, it is assumed that the global drift capacity is equal to 0.10. Based on the calculated drift demands and assumed local and global capacities, Lee and Foutch conclude that all the buildings considered in the study satisfy the collapse prevention performance objective.

Shake Table Collapse Experiments

Only a few experiments have been conducted in which structural models are tested all the way to collapse. Kato et al. (1973) test simple models to collapse or near collapse on a shaking table and compare their response with the response obtained analytically considering strain hardening and P-Δ effects. The models are formed with 15 cm (5.9 in.) high H steel columns fixed at both ends and a concentrated mass on top of the columns. They conclude that except for some softening of the hysteresis loops due to Bauschinger effect, the test results are well predicted by the analytical model.

Vian and Bruneau 2003 also test to collapse on a shaking table 15 simple specimens, each built with four steel columns connected to a rigid mass. The column heights range from 91.7 mm (3.6 in.) to 549.5 mm (21.6 in.) and are designed to have a slenderness ratio of 100, 150, or 200. The tests are conducted under the N-S component of the 1940 El Centro ground acceleration record, progressively increasing the intensity of the motion until the specimens collapse. From their test results, they observe that the inelastic behavior of the specimens has a high dependence on the traditional stability factor [defined for a single-story structure as $P/K_0H$, where P= vertical load; $K_0$=initial stiffness; and H= story height]. Specimens with a stability factor of less than 0.1 are able to withstand the ground motions with larger ductility demands and accumulated drifts than those with a stability factor of more than 0.1. They also compare their experimental results against those obtained analytically using a simple hysteretic model. They find that the analytical results do not provide a good match to the experimental data.

On a shake table too, Kanvinde (2003) tests to collapse or near collapse 19 simple structures with a story height of 254 mm (10 in.), a bay width of 610 mm (24 in.), and a floor plan of 305 mm by 610 mm (12 in. by 24 in.). The structures are built with four steel flat columns connected to a rigid 1.425 kN (320 lb) steel mass. They are tested under two of the ground motions recorded during the 1994 Northridge earthquake: Obregon Park and Pacoima Dam. The specimens collapse after plastic hinges are formed at the top and bottom of the columns and a story mechanism is formed. Kanvinde also conducts a nonlinear, large displacement, dynamic analysis of the specimens to evaluate the ability of analytical tools to predict their response under very large displacements. For this purpose, he uses the open system for Earthquake Engineering simulation (OpenSEES) program being developed by the Pacific Earthquake Engineering Research Center (OpenSEES 2005). The hysteretic model used in the analyses is the Giufré-Menegotto-Pinto model, which is characterized by a yield envelope and a nonlinear hardening exponential law. The parameters for this model are determined from monotonic and cyclic tests of the columns used to build the experimental models. By comparing the displacement time histories obtained analytically and experimentally, Kanvinde finds that the analytical simulations predict with an average error of about 15% the results from the shake table tests.

Similarly, Elwood and Moehle (2003) conduct shake table tests of two one-half scale reinforced concrete plane frames to investigate the mechanisms that lead to the seismic collapse of reinforced concrete frames with nonductile or low-ductility columns. The focus of the tests is the loss of column axial capacity due to column shear failure and the redistribution of loads that takes place in a structure after the axial failure of a column. The two test specimens consist of three columns fixed at their base and interconnected by a beam at their upper end. The only difference between the two specimens is the magnitude of the axial force applied to the center column. This center column is designed with widely spaced transverse reinforcement to make it vulnerable to a shear failure and a subsequent axial failure during the tests. A scaled version of a ground motion recorded during the 1985 Chile earthquake is used as input. Elwood and Moehle also compare the measured response of the test specimens with results from a nonlinear dynamic analysis of an analytical model of the shake table specimens. This analytical model is formulated with a proposed column element that incorporates empirical models that predict the column drifts at the time of shear and axial load failures. These empirical models are developed to account for and evaluate the influence of such failures in a building frame analysis. They find that the analytical model provides a good estimate of the measured drifts up to the point of shear failure but fails to capture the large displacements that occur after that. Because of the underestimation of the column drifts, it also fails to detect the axial failure of the center column in the specimen with the larger central axial load.

Although not precisely a shake table experiment, it is also worthwhile describing the full-scale experiment conducted by Nakashima et al. (2006). Nakashima and coworkers test to collapse under quasistatic cyclic loading a full-scale model of a steel moment-resisting frame. The frame has three stories, two bays in the longitudinal direction, and one bay in the transverse direction. The plan dimensions are 12 m by 8.25 m (39.4 ft by 27.1 ft) and the total height is 8.5 m (27.9 ft.) The columns and beams are made with cold-formed square tubes and hot-rolled wide flange sections, respectively. The floors are formed with metal deck sheets and cast-in-place reinforced concrete. The horizontal loading is applied by two jacks placed at the center of the third floor along the frame's longitudinal direction. The loading protocol consists of displacement amplitudes that produce overall drift angles horizontal displacements at the loading point over frame height of 1/ 200, 1/ 100, 1/ 75, 1/ 50, 1/ 20, and 1/ 15 rad, with each amplitude repeated two or three times. In this test, they observe that in the last portion of the loading the first-story shearforce resistance decreases significantly owing to column local buckling, plastic elongation of the anchor bolts at the base of the columns, and crashing of the concrete underneath the column base plates. In the end, local buckling occurs at the top of the columns of this story and a story collapse mechanism develops.

Nakashima and coworkers also perform a series of numerical simulations with an analytical model of the tested frame to investigate how well commonly used methods of analysis can predict the observed experimental behavior of the frame. In this analytical model, the beams and columns are represented with elastic beam elements and bilinear rotational springs at the ends of the elements. A rotational spring is also inserted at the bottom of the first-story columns to account for the flexibility of the column base plates and anchors. P-Δ effects and the deformation of the panel zones at the beam-column joints are accounted for. They find that with the proper adjustment of the basic material properties, strain hardening after yielding, and the composite action between the steel beams and the reinforced concrete floor slabs, the numerical simulations are capable of accurately predicting the observed cyclic behavior of the frame up to a drift angle of 1/ 25 rad. Because the simple analytical model used cannot account for the deterioration that takes place in the structural elements at large drifts, the numerical simulations fail to reproduce the experimental results in the last portion of the loading.

## Needs and Challenges

It may be inferred from the foregoing literature review that the assessment of the capacity of a structure to resist an earthquake-induced collapse is a complicated task. The reason is that many factors are involved and it is difficult to account for many of these factors accurately. Factors that may significantly affect the collapse capacity of a structure are: (1) the characteristics of the ground motion e.g., intensity, frequency content, and duration exciting the structure; (2) the dynamic properties of the structure; (3) the geometry of the structure e.g., torsional effects; (4) the post-elastic and post-buckling behavior of its components; (5) the strength and stiffness of these components; (6) the degradation of this strength and this stiffness after several loading cycles; (7) the interaction between vertical loads and lateral drifts; (8) the interaction of the structure with its nonstructural components e.g., the effect of components such as stairways and cladding on structural stiffness and strength; (9) soil-structure interaction e.g., the additional lateral displacements and damping introduced by the flexibility of the foundation soil, or the changes in the configuration of the structure due to large soil settlements or localized soil failures; and (10) residual stresses and initial imperfections e.g., the influence of fabrication residual stresses and member out-of-straightness on member stiffness. It seems, therefore, that an accurate analytical prediction of the collapse capacity of a structure is possible only if the prediction is attained through a step-by-step dynamic finite-element analysis in which (1) the equations of motion are established on the basis of the deformed configuration of the structure; (2) this deformed configuration is updated at each step; (3) nonlinear large-deformation elements are used; (4) the mesh considered is sufficiently fine to accurately account for the spread of plasticity, local instabilities, and crack formation; (5) a characteristic response of the structure is monitored to identify a rapid increase in its magnitude; and (6) the analysis is repeated under a sufficiently large number of different ground motions with different characteristics to obtain statistically meaningful averages.

It may also be inferred from the presented literature review that the currently available methods to assess the collapse capacity of a structure are not completely satisfactory. Those based on single-degree-of-freedom models, although simple, are unreliable because, as pointed out by Bernal (1992, 1998), the collapse capacity of a structure strongly depends on the assumed shape of the failure mechanism and this shape cannot be predicted a priori, not even with the help of a pushover analysis. Nonlinear static methods are also unreliable because, as discussed earlier, they lack a theoretical foundation, are based on incorrect assumptions, and neglect important effects. The methods based on finite-element models seem to be fine except that to be reliable it is necessary to account properly for the many factors listed earlier. This renders them computationally demanding and therefore impractical. Likewise, the methods based on an incremental dynamic analysis, besides being also computationally demanding, may also be unreliable. The reason is that these methods are based on ground motion intensity versus structural response curves that are difficult to interpret as these curves may turn out to be non-monotonic and discontinuous, may not reach a plateau to clearly indicate a collapse occurrence, or may exhibit this plateau at several levels of the ground motion intensity (Vamvatsikos and Cornell 2001). They may also be substantially different from ground motion to ground motion and thus show a considerable dispersion. This means that the methods based on an incremental dynamic analysis may require the consideration of a large number of ground motions to obtain the pertinent statistical averages with a high level of confidence (Krawinkler et al. 2003), or the search for alternative intensity measures to reduce such dispersion. With methods based on an incremental dynamic analysis, the results may also depend on the selection of the parameter used to measure the ground motion intensity spectral acceleration at the fundamental period of the structure versus Arias intensity, for example.

Lastly, it may be observed that (1) the reliability of collapse assessment methods may be affected by the accuracy and convergence problems that are likely to occur due to the high levels of nonlinear behavior and large displacements a structure may experience when it is near collapse; (2) the adequacy of the available collapse assessment methods has not been verified through experimental or field studies; (3) only a few shake table experiments have been carried out to study the collapse of structures and verify the adequacy of collapse assessment methods; (4) realistic models of real structures have never been tested to collapse under earthquake excitations; (5) little information is currently available about the real capacity of structures to resist a collapse; and (6) little is known about the inherent safety margin against collapse in code-designed structures (Maison and Bonowitz 2004).

It is clear, thus, that further research is needed before the collapse capacities of structures and the associated safety margin against collapse may be evaluated with confidence. There is a need for experiments with realistic full-scale specimens subassemblies and whole structures in which the specimens are tested all the way to collapse. These experiments are needed to (1) advance the understanding of the conditions that lead to a collapse in real, full-scale, three-dimensional structures; (2) evaluate the capability of existing numerical analysis techniques to predict building behavior at the deformation levels involved when a structure is near collapse; (3) assess the adequacy of current collapse assessment methods; (4) generate data on the hysteretic behavior of structural components as they approach the failure state; and (5) calibrate simplified analysis models. There is also a need to evaluate the inherent safety margin in current seismic provisions against a structural collapse and establish whether or not this safety margin is adequate enough.

Ultimately, the great challenge for the profession is the development of simplified techniques that correlate well with experiments and advanced methodologies to estimate in a reliable way the aforementioned collapse capacities and safety margin. These techniques are urgently needed to facilitate (1) the scrutinization of present and future code provisions in regard to their ability to provide an adequate safety margin against collapse; (2) the development of design procedures that explicitly would make structures resist a collapse with a specified safety margin; and (3) the identification and strengthening of weak structural components that may compromise the collapse capacity of a structure. Undoubtedly, the availability of such techniques could help improve the seismic performance of building structures and minimize the number of catastrophic failures during earthquakes.

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