

Open Access

# Assessment of Collapse Approach Based on Pre-established Engineering Demand Parameters (EDP)s Limits

Hassan Moniri\*, Orod Zarrin, Mohammad Moniri and Zahra Ahooei

Eastern Mediterranian University Famagusta, Cyprus

#### Abstract

Traditionally, collapse potential was estimated by using non-deteriorating systems in order to predict the Engineering Demand Parameters (EDPs) and assigning judgment limits for these parameters. Recently, the deteriorating systems have been used for estimation of collapse but still based on pre-established EDPs limits. However, EDPs become very sensitive when the system is very near to collapse, and small disturbance of the input creates great variations in the response. In the earthquake engineering, the concept of global collapse denotes the lack of ability of a structural system for bearing the gravity loads in exposing the seismic excitation. In the earthquake engineering the concept of collapse denotes the lack of ability of a structural system or a part of it, for bearing the gravity load-carrying capacity under the seismic excitation. Collapse can be local or global; the local collapse can for example happen when a vertical load-carrying component is not successful in compression or when shear transfer is missed between the vertical and horizontal components (for instance shear failure between a column and a flat slab). But global collapse may have several reasons. The transference of a primary local failure from each component to another one can lead to progressive or cascading collapse. In this sturdy, try to investigate the different parts of collapse assessment methods to understand and quantify the effects and to develop nonlinear deteriorating component models which could duplicate the experimental results. P- $\Delta$ , degrading hysteretic model, and expected spectral shape effect on collapse methods assess the structural collapse capacity by nonlinear dynamic analysis occurs in case of ground motions selection and scaling for the analysis.

Keywords: Global collapse; P- $\Delta$  effects; Spectral; Spectral shape; Stiffness

### Introduction

A structure's appropriate seismic performance needs available strength and deformation capacities of the components to be more than the earthquake imposed necessities on the structure. Due to structural behaviour during an earthquake, performance evaluation should be carried out by nonlinear time history analysis procedure and according to selected ground motion. If encountered to nonlinear structural behaviour, displacements are more descriptive than forces to structure and more effective control is achieved if they are bounded instead of.

A shift in design approach from force-based to that of behaviour will create a new method named performance-based design; a scheme for designing to limit states. Nonlinear analysis is a way to pass over the elastic range of structure capacity. In order to assess the seismic requirements at low operational levels, e.g. life safe and collapse prevention of structure, inelastic behaviour should be taken into widespread consideration. One of the fundamental issues in performance-based earthquake engineering is determining the seismic demand and collapse capacity proportionate to earthquakes. Consequently, various methods have been proposed for assessing seismic structural performance in development of performance-based earthquake engineering. In the earthquake engineering, the concept of global collapse denotes the lack of ability of a structural system for bearing the gravity loads in exposing the seismic excitation. In the earthquake engineering the concept of "collapse" denotes the lack of ability of a structural system or a part of it, for bearing the gravity loadcarrying capacity under the seismic excitation (Figure 1).

Collapse can be local or global; the local collapse can for example happen when a vertical load-carrying component is not successful in compression or when shear transfer is missed between the vertical and horizontal components (for instance shear failure between a column and a flat slab). But global collapse may have several reasons. The transference of a primary local failure from each component to another one can lead to progressive or cascading collapse [1]. Incremental collapse happens when displacement of one story is very big, and the impacts of second order (P- $\Delta$ ) completely counterbalance the shear resistance of the first order story. In each of these cases the collapse replication requires modelling of the deterioration properties of structural components exposed to cyclic loading, as well as the inclusion of P- $\Delta$  impacts (Figure 2). Some buildings collapsed partially or entirely in the following earthquakes: in alparaiso, Chile



\*Corresponding author: Hassan Moniri, Eastern Mediterranian University Famagusta, Cyprus, Tel: +90 392 630 1111; E-mail: hsn.moniri@gmail.com

Received Januray 30, 2016; Accepted February 10, 2016; Published February 12, 2016

**Citation:** Moniri H, Zarrin O, Moniri M, Ahooei Z (2016) Assessment of Collapse Approach Based on Pre-established Engineering Demand Parameters (EDP)s Limits. J Civil Environ Eng 6: 214. doi:10.4172/2165-784X.1000214

**Copyright:** © 2016 Moniri H, et al. This is an open-access article distributed under the terms of the Creative Commons Attribution License, which permits unrestricted use, distribution, and reproduction in any medium, provided the original author and source are credited.



in 1985; Mexico City in 1985; Armenia in 1988; Luzon, Philippines in 1990; Guam in 1993; Northridge, Calif. in 1994; Kobe, Japan in 1985; Kocaeli, Turkey in 1999; Chi-Chi, Taiwan in 1999; and Bhuj, India in 2001. Bernal in the investigation of the instability of buildings in earthquakes, asserts that only by limiting the structure's maximum elastic story drifts we cannot guarantee a structure's immunity against inelastic dynamic instability. This conclusion is confirmed recently [1]. Also, Challa and Hall [2] in their study of the collapse capacity of a twenty story steel frame, see significant plastic hinging in the columns of the structure and the structure's possible collapse when exposed to ground motions in a great earthquake. Although according to what is needed in current code provisions, the flexural strength of the columns is more than its beams in all of the joints. It is worth mentioning that this remark is recently confirmed [3]. In this sturdy, try to investigate the different parts of collapse assessment methods to understand and quantify the effects and to develop nonlinear deteriorating component models which could duplicate the experimental results.  $P-\Delta$ , degrading hysteretic model, and expected spectral shape effect on collapse methods assess the structural collapse capacity by nonlinear dynamic analysis occurs in case of ground motions selection and scaling for the analysis (Figure 3).

# **Analytical Collapse Investigations**

Takizawa and Jennings [4] studied the final capacity of an RC frame in seismic excitations. This structural model was an equivalent SDOF system which involved degrading tri-linear and quadric-linear (or strength degrading) hysteretic curves. This is a primary effort to evaluate  $P-\Delta$  effects as well as material deterioration in collapse evaluation. They used some modified Takeda models to indicate that the SDOF systems which had negative post-yield stiffness tend to collapse, either if they had experienced the damage before or not. Mehanny and Deierlein examined collapse for some composite structures which had RC columns as well as the steel or composite beams. For a structure and ground motion (GM) intensity record, these researchers performed a second-order inelastic time history analysis (THA) for the undamaged structures and computed the cumulative damage indices, which were used to degrade stiffness and strength of the damaged sections. They reanalyzed the damaged structure via a second-order inelastic static analysis with respect to the residual displacements and involving just gravity loads. It was supposed that the Global collapse occurs in case the maximum vertical load that the damaged structure is able to endure is less than the applied gravity loads ( $\lambda u < 1$ ). In case the collapse did not occur, then the record would be scaled to determine the ground motion intensity in which the collapse happens(Figure 4).

Page 2 of 5

# P- $\Delta$ effects on Global Collapse

Several aspects of collapse assessment methods are improved nowadays. Researchers have tried independently to understand and quantify the P- $\Delta$  effects and to develop nonlinear deteriorating component models which could duplicate the experimental results. In addition, efforts have been done for integrating the factors that affect the collapse in an integrated methodology. The investigation of the global collapse initiated by P- $\Delta$  effects in seismic reaction. However, hysteretic models took a positive post-yielding stiffness into account the structure tangent stiffness turned negative in huge P- $\Delta$  effects that finally led to the system's collapse. For example, Jennings and Husid used a one story frame which had springs at the ends of the columns by the use of bilinear and hysteretic models. They inferred that the most significant factor in collapses is the structure's height, the ratio of the earthquake intensity to level of the yield of the structure, and the second slope of the bilinear and hysteretic model. They declared that the required motion intensity for collapse depended firmly on ground motion duration. This conclusion was drawn without consideration of cyclic deterioration behaviour, and simply because the probability of collapse increases when the loading path stays for a longer time on a backbone curve with a negative slope.

Sun et al. investigated the impact of gravity on the dynamic behaviour of the SDOF system and its impact on changing the system's



**Figure 3:** The response of an SDOF system represented by a peak-oriented model with rapid cyclic deterioration [16].



period. Bernal analyzed two-dimensional moment-resisting frames, and inferred that the least required strength (or base shear capacity) for enduring a ground motion without collapse absolutely depends on the form of the controlling mechanism (Figure 5).

# **Degrading Hysteretic Models**

In the degrading hysteretic model, degradation of the reloading stiffness depends on maximum displacement occurred in the loading path direction. As a result of this attribute, this model is frequently called the peak-oriented model. Song and Pincheira's model [5] can also represent the stiffness deterioration and cyclic strength on the basis of dissipated hysteretic energy. This model is basically a peak oriented model which regards the pinching on the basis of deterioration factors. The backbone curve contains a kind of post capping negative stiffness as well as a branch of residual strength. Due to the fact that the original backbone curve doesn't deteriorate, the unloading and accelerated cyclic deterioration are the mere modes, and before arriving to the peak strength, the model is not able to reproduce the strength deterioration. Ibarra et al. developed a tri-linear model similar to that of Song and Pincheira which was able to take strength deterioration in to account completely. Based on the results of 320 tests performed on columns around the world, relations are presented for seismic behavioral parameters of the beam-column elements (Figure 6). In order to study structural behaviour and determination of instabilities, Haselton et al. [6] utilized linear regression analysis on PEER dataset (collected at Washington university by Berry and Eberhard [7] including unilateral and reciprocating tests on 306 rectangular and 177 circular beamcolumns) to calibrate the data presented [8,9]. Finally some relations were derived for the necessary parameters to introduce monotonic





and cyclic behavior herein. These relations were somewhat suitable for modelling the elements of regulatory-designed buildings. The model utilized by Hazelton et al. [10] can be applied to consider the nonlinear behaviour of beam-column elements of trilinear model offered by Ibarra et al. [3]. One important attribute of this model is to have a negative branch after the hardening region which enables us to model strain softening appears in phenomenon like concrete crushing or buckling and failure of armatures.

# Evaluating the Expected Spectral Shape Effect on Collapse Assessment

Another challenge in assessing structural collapse capacity by nonlinear dynamic analysis occurs in case of ground motions selection and scaling for the analysis. Baker and Cornell indicated that the spectral shape, along with the ground motion intensity, is an important trait of ground motions which has an influence on the structural response. Especially, for a certain level of ground-motion hazard (for instance a 2 percent chance of exceedance in 50 years), the form of the Uniform Hazard Spectrum (UHS) may be totally different from the form of the mean or the anticipated response spectrum of an actual ground motion which has a similarly high spectral magnitude in one period [11,12].  $\boldsymbol{\epsilon}$  (i.e., epsilon) can be defined as the number of logarithmic standard deviations among the spectral value and the mean Sa prediction in a ground-motion prediction or "attenuation" model. In order to show the unique spectral form of some rare ground motions, the Loma Prieta spectrum includes a rare spectral intensity at 1.0 s of 0.9 g, which involves only a 2 percent chance of exceedance in 50 years. It is revealed that this extreme ground motion has a very different form than the mean anticipated spectrum. Especially, the spectrum of this record has a peak from nearly 0.6 to 1.8 s and lesser intensities in proportion to the predicted spectrum in other times. The intensity at 1.0 s, excelled with a 2 percent probability in 50 years, exists in the peak of the spectrum, and in this time the observed Sa (1 s) = 0.9 g is very higher than the mean expected Sa (1 s) = 0.3 g; in other points far from the peak, the spectral values are more similar to the mean expected Sa. This peaked shaped exists because the ground motions, which have an intensity above the average, do not always have equal and large intensities in other points [13]. In a 1.0 s period, the spectral value of the Loma Prieta record is 1.9 standard deviations higher than the anticipated mean spectral value from the attenuation connection, hence this record will have " $\epsilon$ =1.9 at 1.0 s."  $\epsilon$  (or epsilon) is defined as the number of logarithmic standard deviations between the spectral value observed and the mean Sa prediction from a ground-motion prediction or attenuation model. Correspondingly, this record has  $\varepsilon$ =1.1 in 1.8 s. Hence, the component  $\boldsymbol{\epsilon}$  is a function of the ground-motion record, the groundmotion prediction model which is compared, and the desirable period. Baker and Cornell investigated the effects of several ground-motion characteristics on the collapse capacity of a no ductile reinforced concrete (RC) frame 7-story building with an important period T1 of 0.8 s. They discovered that the average collapse capacity rose by a factor of 1.7 when a  $\epsilon$  (0.8s) =2.0.

#### **Selection of Ground Motions**

The global collapse method is based on the time history analysis. Therefore, a set of ground motions should be selected cautiously based on the specific goals. The set must be large enough to produce statistically reliable results (Figure 7).

#### **Deterioration Models**

Collapse evaluation is based on hysteretic models that account for



history-dependent strength and stiffness deterioration. Deteriorating models are developed for bilinear, peak-oriented, as well as pinching hysteretic models. These systems' monotonic backbone curve includes a negative tangent stiffness branch, an elastic branch, a strainhardening branch, and in some cases a residual strength branch of zero slope. In addition, cyclic deterioration is considered by making use of energy dissipation as a deterioration criterion. The following 4 modes of deterioration are involved: post-capping strength, basic strength, accelerated reloading stiffness deterioration, and unloading stiffness. It is shown the response of an SDOF system represented by a peakoriented model with rapid cyclic deterioration.

#### Structural Systems

In general, the collapse assessment methodology is identical for SDOF and MDOF systems. A variety of SDOF systems are used in Chapter 4 to determine the parameters that most affect global collapse. The information synthesized from SDOF systems is used to narrow the number of parameters to be studied in MDOF structures.

# **Collapse Capacity**

To obtain the collapse capacity related to a particular ground motion, the structural system is analyzed under increasing relative intensity values, expressed as  $(Sa/g)/\eta$  for SDOF systems. The intensity of the ground motion (Sa) is the 5% damped spectral acceleration in the elastic period of the SDOF system (without P- $\Delta$  effects), while  $\eta$ = Fy/W is the base shear strength of the SDOF system which is normalized by its seismic weight. The relative intensity can be plotted against the EDP of interest, resulting in  $(Sa/g)/\eta$ -EDP curves. For MDOF structures, the relative intensity is expressed as  $[Sa(T_1)/g]/\gamma$ , where  $Sa(T_1)/g$  is the normalized spectral acceleration in the structure's fundamental period without P- $\Delta$  effects, and the parameter  $\gamma$  is the base shear coefficient Vy/W, which is equivalent to  $\eta$ . These relative intensity definitions permit a dual interpretation:

(1) If there be an increase in the ground motion intensity and the system strength is kept constant, the resulting  $(Sa/g)/\eta$  - EDPor  $([Sa(T_1)/g]/\gamma$  - EDP) curves represent incremental dynamic analyses (IDAs) [17].

(2) In case the ground motion intensity is kept constant (given hazard) and the strength of the system is reduced, the resulting (Sa/g)/ $\eta$  - EDPor ([Sa(T\_1)/g]/ $\gamma$  - EDP) curves represent EDP demands for various strength levels and are referred to as "strength variation curves." In this case, (Sa/g)/ $\eta$  is equal to the conventional strength

reduction factor, R, for structures without over strength. Note that when the strength is decreased the entire backbone curve scales down.

#### Effects of Uncertainty in System Parameters

In the first part of the research, the collapse capacity is examined considering Record To Record variability (RTR) as the only uncertainty in the computation of the collapse capacity. However, system parameters like ductility capacity and post-capping stiffness can also be considered in a probabilistic framework, even though experimental information that can be used to define statistical properties of the parameters of the hysteresis model is rather limited. The first-order second-moment (FOSM) method is utilized for computation of the additional variance of collapse capacity resulting from the uncertainty in the system parameters, while Monte Carlo simulation is also utilized in order to verify some of the results. The FOSM method approximates the collapse capacity variance based on a Taylor's series expansion of a performance function (g) about the anticipated values of random variables. One of the main advantages of the method is that the first and second moments are appraised without any knowledge about distribution of the function "g". For instance, it is indicated that the contributions to the variance of collapse capacity from several sources, including RTR variability, ductility capacity, uncertainty in postcapping stiffness, and cyclic deterioration, considering a standard deviation of the log of the data of 0.60. Based on the system properties, the contributions of uncertainty in system parameters to the total variance can be small or comparable to the contribution due to RTR variability.

#### **Collapse Assessment of SDOF Systems**

Parameter studies on SDOF systems are easily implemented and help to identify the system parameters which can have an insignificant or prominent influence on MDOF structures. The small calculation effort required for analyzing the SDOF systems allows the investigation of so many systems. Furthermore, modification of a special parameter usually has a larger impact on SDOF systems than on MDOF structures. The latter structures usually have elements yielding in various times and some of the factors do not reach the inelastic range; thus, their global stiffness matrix has smaller modifications than the corresponding stiffness of SDOF systems. In the past, many studies have been conducted to evaluate the inelastic seismic demands of SDOF systems. Seismic demands have been studied by means of constant ductility inelastic displacement ratios [18] or by means of strength reduction factors for constant ductility [19]. The second study included the effect of strength and stiffness deterioration in hysteretic models with bilinear backbone curves. The results indicated that strength deterioration may greatly affect the response of SDOF systems, but the effects of unloading stiffness deterioration are relatively small. Gupta and Kunnath extended the investigation of Rahnama and Krawinkler, obtaining similar conclusions. However, these studies are based on systems without strength deterioration of the backbone curve and they do not address the collapse limit state. Song and Pincheira investigated the impact of stiffness and strength deterioration on the SDOF systems maximum inelastic displacement without including geometric nonlinearities. They discovered that the displacement proportion between a deteriorating and nondeteriorating system can be about two (particularly in the short-period range) and that it differs meaningfully with the deterioration rate and type of ground motion. They assumed that an SDOF system collapses if its remaining strength is less than 10% of the yield strength. They reported that many systems collapsed for one or more ground motions under low strength coefficients but they did not trace this limit state

for all the cases. Vamvatsikos performed the incremental dynamic analyses (IDAs) for pinched hysteretic SDOF systems which involved a negative post-capping stiffness and residual strength although without any cyclic deterioration. He detected that the cap displacement ( $\delta_{cc}$ ) and the slope of the post-capping stiffness constitute the two factors that have the greatest influence on the performance of the medium-period-systems. Ibarra and Krawinkler investigations aimed to have an innovation for global collapse assessment of deterioration-oriented structural systems.

#### Conclusion

In general, the collapse assessment methodology is identical for SDOF and MDOF systems. A variety of SDOF systems are the most affect global collapse. The information synthesized from SDOF systems is used to narrow the number of parameters to be studied in MDOF structures. Collapse evaluation is based on hysteretic models that account for history-dependent strength and stiffness deterioration. Deteriorating models are developed for bilinear, peak-oriented, as well as pinching hysteretic models. These systems' monotonic backbone curve includes a negative tangent stiffness branch, an elastic branch, a strain-hardening branch, and in some cases a residual strength branch of zero slope. In addition, cyclic deterioration is considered by making use of energy dissipation as a deterioration criterion. The following four modes of deterioration are involved: post-capping strength, basic strength, accelerated reloading stiffness deterioration, and unloading stiffness. It is shown the response of an SDOF system represented by a peak-oriented model with rapid cyclic deterioration. The global collapse generally refers to the lack of ability of a system to support gravity loads due to the extreme lateral displacement, which significantly reduces the story shear resistance and produces instability in the system. Traditionally, collapse potential was estimated by using non-deteriorating systems in order to predict the engineering demand parameters (EDPs) and assigning judgment limits for these parameters. Recently, the deteriorating systems have been used for estimation of collapse but still based on pre-established EDPs limits. The result of shake table conducted that a SDOF steel frame system exposed to earthquakes of gradually increasing intensity until the collapse are extremely precise for predicting the collapse for systems in which the P- $\Delta$  effect controls the beginning of collapse. The displacement proportion between a deteriorating and non-deteriorating system can be about two (particularly in the short-period range) and that it differs meaningfully with the deterioration rate and type of ground motion. Collapsing systems happened during one or more ground motions under low strength coefficients but they did not trace this limit state for all the cases. He detected that the cap displacement ( $\delta c$ ) and the slope of the post-capping stiffness constitute the two factors that have the greatest influence on the performance of the medium-period-systems. In the degrading hysteretic model, degradation of the reloading stiffness depends on maximum displacement occurred in the loading path direction. As a result of this attribute, this model is frequently called the peak-oriented model. Finally, despite the large amount of researches and studies on this topic, the response of structural systems under geometric nonlinearities and material deterioration has not been studied in details. Hence, there is a need for conducting systematic research about the global collapse with respect to all sources that result in this limit situation.

#### References

 Kaewkulchai G, Williamson EB (2003) Dynamic Behavior of Planar Frames during Progressive Collapse. 16th ASCE Engineering Mechanics Conference, University of Washington, Seattle.

- Murty Challa VR, Hall JF (1994) Earthquake collapse analysis of steel frames. Earthquake Eng & Struct Dynamics 23: 1199–1218.
- Ibarra LF, Medina RA, Krawinkler H (2005) Hysteretic models that incorporate strength and stiffness deterioration. Earthquake Eng and Struct Dynamics 34: 1489-1511.
- Haruo T, Paul CJ (1980) Collapse of a model for ductile reinforced concrete frames under extreme earthquake motions. Earthquake Eng & Struct Dynamics 8: 117–144.
- 5. Song, Pincheira (2000) Spectral Displacement Demands of Stiffness and Strength-Degrading Systems. Earthquake Spectra.
- Christos AZ, Dimitrios V, Panagiotis G, Kostas A (2007) Impact of FE Modeling in The Seismic Performance Prediction of Existing RC Buildings. Proceedings of the COMPDYN 2007 ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering.
- Berry MP, Eberhard MO (2003) Column Deformation Demands at Bar Buckling. Proceedings, 2003 Structures Congress. Am Soc of Civil Engineers, Seattle, Washington, May.
- Panagiotakos TB, Fardis MN (2001) A Displacement-based Seismic Design Procedure of RC Buildings and Comparison with EC8. J of Earthquake Eng and Struct Dynamics 30: 1439-1462.
- Ibarra LF, Krawinkler H (2005) Global collapse of frame structures under seismic excitations. Rep. No. TB 152, The John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA.
- Haselton C, Baker JW (2006) Ground motion intensity measures for collapse capacity prediction: Choice of optimal spectral period and effect of spectral shape. in Proceedings, 8th National Conference on Earthquake Engineering, San Francisco, California, p: 10.
- Baker JW, Cornell CA (2006) Correlation of Response Spectral Values for Multi-Component Ground Motions. Bulletin of the Seismological Society of Am 96: 215-227.
- Baker JW, Cornell CA (2006) Which Spectral Acceleration Are You Using? Earthquake Spectra 22: 293-312.
- 13. Baker JW (2011) Conditional Mean Spectrum: Tool for ground motion selection. J StructEng 137: 322-331.
- 14. David MB, William BJ, Thomas EF (1997) Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of recent Work. Seismological Research Letters 68: 128-153.
- Baker JW, Cornell CA (2005) A Vector-Valued Ground Motion Intensity Measure Consisting of Spectral Acceleration and Epsilon. Earthquake Eng & Struct Dynamics 34: 1193-1217.
- Luis FI, Krawinkler H (2004) Hysteretic models that incorporate strength and stiffness deterioration. Earthquake Eng & Struct Dynamics 34: 1489–1511.
- Vamvatsikos D, Cornell CA (2002) The Incremental Dynamic Analysis and Its Application to Performance-Based Earthquake Engineering. Elsevier Science Ltd. All rights reserved 12th European Conference on Earthquake Engineering.
- Miranda E (1993) Site-Dependent Strength Reduction Factors. J Struct Eng ASCE 119: 3505-3519.
- Nassar A, Krawinkler H (1991) Seismic Demands for SDOF Report No. 95, The John A. Blume Earthquake Engrg. Center, Stanford University.