A Practical Approach for Assessing Structural Resistance to Earthquake-Induced Collapse

Abbie Liel[1] and Hesham Tuwair[2]

[1]Assistant Professor, Dept. of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO 80309. Email: abbie.liel@colorado.edu
[2] Graduate Research Assistant, Dept. of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO 80309.

ABSTRACT

This paper proposes an alternative algorithm for estimating structural resistance to earthquake-induced collapse. The method utilizes nonlinear static (pushover) analysis to obtain an initial estimate of collapse resistance. This estimate serves as the starting point for dynamic analysis. Analyses presented demonstrate that the proposed algorithm is more computationally efficient than a standard incremental dynamic analysis approach, reducing analysis time by approximately 90%. Accuracy of the proposed approach is demonstrated through comparison with results obtained from incremental dynamic analysis, indicating good agreement between the two methods.

INTRODUCTION

Emerging performance-based design and assessment methodologies focus on evaluation of structural collapse capacity as a primary metric for seismic safety of individual buildings or building systems. In recent years, researchers have developed advanced procedures for assessing structural resistance to earthquake-induced collapse. These procedures require creation of a nonlinear analysis model capable of representing structural behavior as it becomes damaged. The simulation model is then subjected to a suite of recorded or synthetic ground motions, scaled to different levels of ground motion intensity according to incremental dynamic analysis. This process is computationally intensive, but represents a significant improvement over nonlinear static methods for assessing the collapse performance limit state.

This paper proposes a practical approach to collapse performance assessment utilizing dynamic analysis, but with critical modifications to reduce the computational effort. The method utilizes results from a nonlinear static (pushover) analysis to obtain a preliminary estimate of collapse capacity. This estimate serves as the starting point for dynamic analysis, in order to reduce computational time. As a result, the proposed practical approach takes advantage of information gained from pushover analysis, but retains the accuracy obtained from dynamic analysis of multiple degree-of-freedom building models. The accuracy and efficiency of the proposed approach is validated through the assessment of structural resistance to seismic collapse for several example buildings, using the *OpenSees* analysis software.

WHY QUANTIFY STRUCTURAL RESISTANCE TO SEISMIC COLLAPSE?

Increased computational power, improved nonlinear analysis tools and advancements in performance-based earthquake engineering, have made it possible to simulate the seismic response of building and bridge structures as they become damaged and collapse. These assessments of collapse resistance provide metrics of seismic risk and safety that are central to performance-based design and assessment of new and existing buildings and evaluation of code seismic provisions. The recent FEMA p695 publication for *Quantification of Building Seismic Performance Factors* requires that proponents of a new structural system show that the design rules for the new system provide adequate resistance to earthquake-induced collapse (FEMA 2009a). Quantitative assessment of collapse risk has also recently been incorporated into seismic design maps such that mapped spectral values for use in building design are now determined using a risk-targeted approach, which are envisioned to result in as-designed structures with a 1% probability of collapse in 50 years (Luco et al. 2007). The concept of assessing seismic collapse risk is an integral part of the ATC-58 project, which is developing next-generation procedures for performance-based design (ATC 2009). As a result of these and other developments, there is increasing interest from both research and practicing engineers to use nonlinear simulation models to evaluate the risk of earthquake-induced collapse.

HOW TO EVALUATE STRUCTURAL COLLAPSE RESISTANCE

The general procedure for evaluating the collapse resistance of a structure is as follows. First, a nonlinear analysis model of the building is created. The model may be two or three-dimensional, but should incorporate the key elements of the seismic resisting system. The simulation model should be capable of capturing the possible failure modes of the system and the critical aspects of strength and stiffness degradation as the structure collapses (Ibarra et al. 2005, FEMA 2009a). In concept, any structural analysis program can be used, provided that it has the material and geometric nonlinear features to represent earthquake-induced collapse and robust solution algorithms to achieve convergence (Haselton et al. 2009a).

Incremental dynamic analysis methods may then be used to assess resistance to earthquake-induced collapse (Vamvatsikos & Cornell 2002, 2004). In incremental dynamic analysis, a building model is subjected to a suite of recorded or synthetic ground motion accelerograms (see e.g. Haselton et al. 2009b). Each record is individually applied to the structure and scaled in intensity until collapse occurs. The structural response of the model is recorded each time the ground motions are scaled. Provided that the simulation model has the necessary nonlinear features, structural collapse may be predicted from runaway interstory drifts and dynamic instability. The outcome of this process is a seismic collapse probability ("fragility") function, which represents the probability of collapse as a function of the ground motion intensity for a particular structure. Variation in collapse capacity (termed "record-to-record" variability) occurs because of differences in frequency content and other ground motion characteristics. The median collapse capacity represents the spectral intensity at which 50% of the ground motion records cause the structure to collapse and may

be used to quantify structural collapse resistance. Ground motion intensity is generally quantified as the spectral acceleration at the first-mode period of the building, $Sa(T_1)$, but in concept other intensity measures could also be used.

In the discussion that follows, simulation models of reinforced concrete (RC) frame structures are used to illustrate the procedure and methods for assessing structural collapse resistance. These buildings are modern special moment frames that vary in height and other key design parameters. Design, models, and assessment have been discussed in detail elsewhere (Haselton et al. 2009c, FEMA 2009a). Structures are represented by two-dimensional models in *OpenSees* software platform (PEER 2009), with special attention given to the modeling of beam-column elements to represent the phenomena of yielding, spalling, bar buckling and crushing as these components experience flexural failure (Haselton et al. 2009c). Each model is subjected to a set of 44 ground motions recorded from large magnitude earthquakes and moderate fault-rupture distances (FEMA 2009a). (Note: since these ground motions were selected without regard for the unique spectral shape of rare ground motions, results should be adjusted as in Haselton et al. (2009d.). This adjustment is neglected here for simplicity in presentation of the proposed algorithm.)

Results of incremental dynamic analysis for a 4-story RC space frame building are shown in Figure 1. The building is designed according to ASCE 7-05 and ACI 318-05 for a high seismic Los Angeles site (Haselton et al. 2009c). The structure has 22 in. square columns and 22x24 in. beams and as-modeled first mode period, T_1, of 0.94 sec. Figure 1a shows results from incremental dynamic analysis for each of the 44 records, illustrating the relationship between spectral intensity and maximum interstory drift (IDR) in the structure as records are scaled. Collapse occurs when the incremental dynamic analysis curve for a particular record flattens out, indicating very large displacements in the building. Results are summarized in the collapse fragility curve in Figure 1b, which shows the probability of collapse as a function of ground motion intensity.
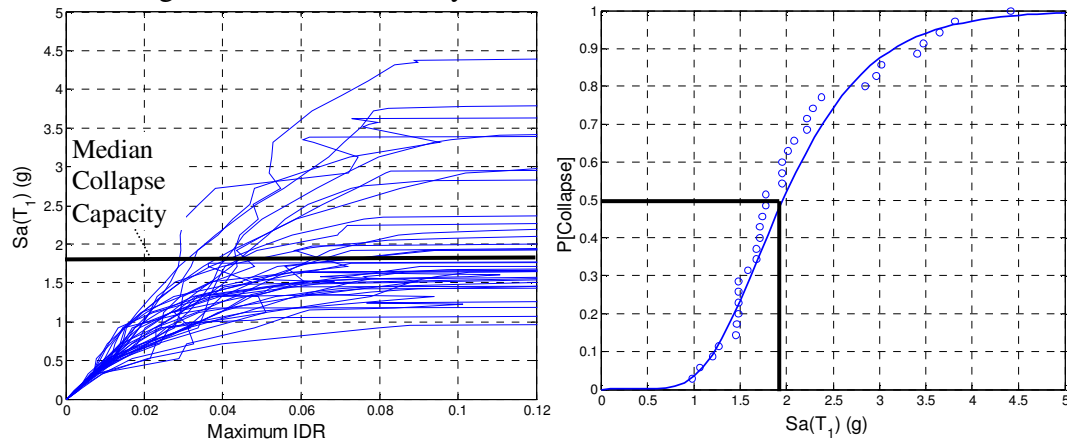

Figure 1. Evaluation of structural collapse resistance for a 4-story RC frame illustrating (a) incremental dynamic analysis and (b) collapse fragility function.

PROPOSED ALGORITHM FOR DYNAMIC ASSESSMENT OF COLLAPSE RESISTANCE

This paper proposes a modified algorithm for dynamic assessment of collapse resistance, which attempts to minimize the computational effort needed to determine

the collapse resistance of a given structure. As with incremental dynamic analysis, a robust nonlinear analysis model and set of ground motion records are required.

Step 1: Initial Estimate of Dynamic Collapse Resistance

Nonlinear static (pushover) analysis is used to obtain an initial estimate, Sa^*, of the dynamic collapse resistance of the structure. Selected methods relating pushover analysis to dynamic collapse resistance to obtain Sa^* are described below.

Step 2: Evaluate Probability of Collapse at Specified Spectral Acceleration Level

Once the initial estimate of dynamic collapse resistance is obtained, each record is scaled to a spectral acceleration of Sa^* to obtain the dynamic time history response of the structure. Key metrics of structural response are recorded, including peak interstory drift ratio, and whether or not the structure collapsed. The probability of collapse is determined from the ratio of the number of records that caused the structure to collapse, to the total number of records.

Results for the 4-story RC frame building are shown in Figure 2a-b. For this example, $Sa^*(T_1) = 1.92g$ is predicted from nonlinear static analysis according to the relationship proposed by FEMA p440a (2009). Sa^* is denoted *Sa, Level 1* on Figure 2. At *Sa, Level 1*, the probability of collapse is 0.61 (i.e. *p[collapse|Sa = 1.92g] = 27/44*).

Step 3: Revise Estimate of Dynamic Collapse Resistance

The goal of the proposed algorithm is to determine the median spectral intensity at which the structure collapses, which corresponds to $Sa(T_1)$ at which 50% of the ground motion records cause collapse. The probability of collapse computed in Step 2 is therefore used to obtain a revised estimate of the dynamic collapse resistance. If more than 50% of the records collapse, the median collapse capacity is lower than *Sa, Level 1* and the revised estimate, *Sa Level 2 = Sa,Level 1 − stepsize*. If fewer than 50% of the records collapse, the median collapse capacity is known to be larger than *Sa, Level 1* and the revised estimate, *Sa Level 2 = Sa,Level 1 + stepsize*. For the example building, the probability of collapse in the previous step is 0.61 and *stepsize* = 0.15g, such that *Sa Level 2* = 1.92g − 0.15g = 1.77g. The choice of the step size affects the computational efficiency of the algorithm, as discussed below.

Step 4: Update List of Ground Motions to Run

Once the revised estimate of the collapse resistance is obtained, it is also possible to reduce the list of ground motions to be run, further improving the computational efficiency of the method. Consider the following: if a particular ground motion record causes collapse at $Sa(T_1) = x$, it follows that the same ground motion will also cause collapse at any $Sa(T_1) > x$. Therefore, if the revised spectral intensity $Sa,Level 2 > x$, we know this record will collapse and we do not need to run the analysis. Conversely, if a ground motion record does not cause the structure to collapse at $Sa(T_1) = x$, it follows that the same ground motion will not cause collapse at any $Sa(T_1) < x$. For the example

shown in Figure 2, 17 of the original set of 44 records did not cause the structure to collapse at *Sa,Level 1* = 1.92g. At *Sa,Level 2* = 1.77g, these 17 records do not need to be rerun because they will not cause collapse at spectral values less than *Sa,Level 1*. Therefore only 27 records need to be run at *Sa,Level 2*. The other 17 records are assumed not to cause collapse in collapse probability computations.

Note: It has been observed that an earthquake record may cause collapse of a structure at a spectral intensity *x*, but not cause collapse when scaled to a higher spectral intensity (*y* > *x*). These are so-called "resurrecting records" and this phenomenon may occur when records scaled to different levels excite different failure modes in the structure. The proposed algorithm assumes that records do not resurrect, i.e. if it causes collapse at some ground motion intensity, it will also cause collapse at all higher ground motion intensities. This assumption may affect the accuracy of the algorithm, though it is reasonable for the structures and intensity measure considered. When very few records are used or the structure has many possible failure modes, such that resurrecting records may be important, is suggested to eliminate Step 4 and run all earthquake records at subsequent Sa Levels. This choice will reduce some of the computational advantages of the algorithm.

Step 5: Repeat Steps 2-4 until Median Collapse Point is Found

For each new spectral value, *Sa,Level i*, the algorithm described in steps 2-4 is repeated. All ground motions in the updated list are scaled to *Sa,Level i* and dynamic analyses are run. After the analysis is completed at each Sa level, the probability of collapse is computed. All records should be included in this calculation, even those that were not run at the spectral level of interest, e.g. number of collapsed records = number of assumed collapsed records (from Step 4) + number of observed collapsed records. This calculation is illustrated in Figure 2. At *Sa,Level 2* = 1.77g, 27 records were run and 23 caused the structure to collapse. The remaining 17 records were not run, but were assumed not to collapse according to the logic described above, resulting *p[collapse]* = 0.52. This process is repeated until the median collapse point is found, i.e. the probability of collapse is equal to 0.50 (+/- some tolerance).

For the example building, the process of finding the median collapse point is illustrated in Figure 2. The analysis is run at *Sa,Level 1* = 1.92g and *p[collapse]* = 0.61. Subsequently the analysis is run (with a smaller number of records) at *Sa,Level 2* = 1.77g and *p[collapse]* = 0.52. Since the probability of collapse is still larger than 0.50, the estimate of the collapse intensity is reduced to *Sa,Level 3* = 1.62g. At *Sa,Level 3* the collapse probability is 0.32. At this point, the algorithm knows that 1.62g < *Median Sa at Collapse* < 1.77g. An intermediate value is estimated, *Sa,Level 4* = 1.68g, resulting in *p[collapse]* = 0.38. This process is repeated until *Sa,Level 6* = 1.76g and *p[collapse]* = 0.50.

Step 6: Refine Estimate of Median Collapse Point

The goal of the analysis is to find the lowest spectral acceleration value at which 50% of the records collapse: the median collapse point. Due to the discrete number of ground motions used, there is a range of spectral values for which *p[collapse]* = 0.50. (Illustration: suppose, hypothetically, that 22 (of 44) records caused collapse at exactly

Sa = 1g and 22 records caused collapse at exactly *2g*. For all *Sa* between *1* and *2g*, the probability of collapse is 0.5.) Step 5 finds one possible value within this range.
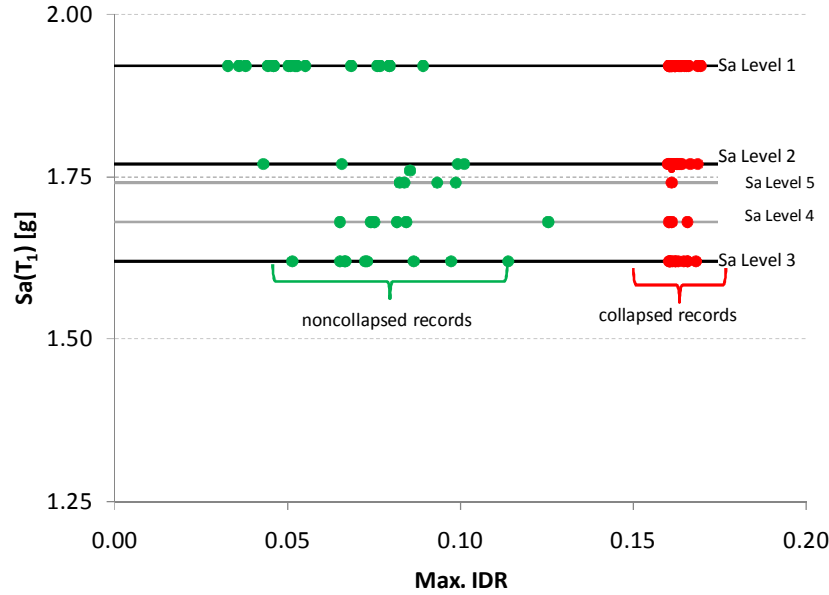

Figure 2a. Illustration of proposed algorithm for the 4-story RC Frame structure, showing relationship between ground motion intensity, i.e. Sa(T₁), and structural response, i.e. peak interstory drift.
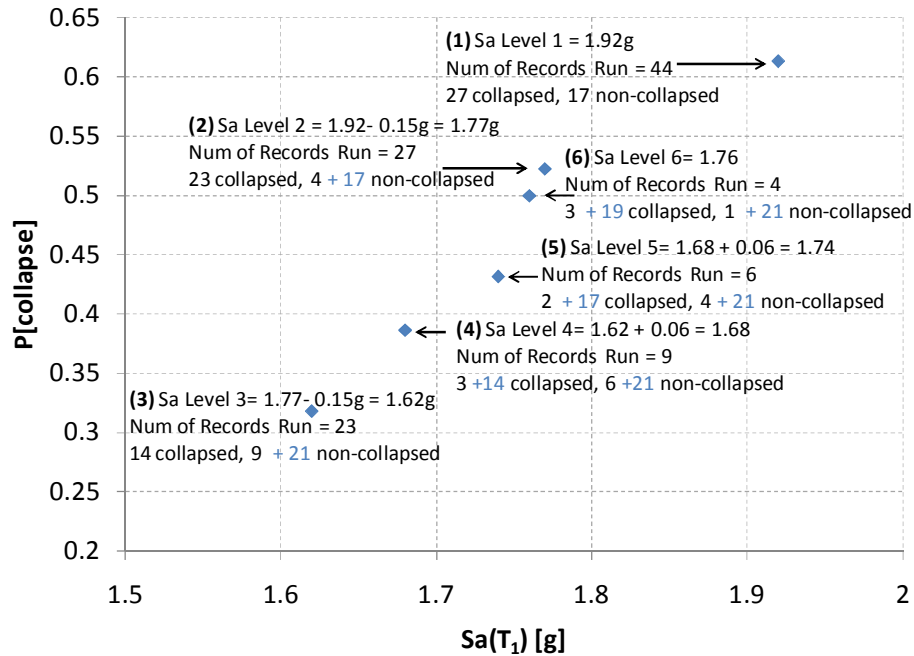

Figure 2b. Annotated collapse cumulative probability function obtained from proposed algorithm, showing how decisions were made to change the estimate of collapse resistance at each step.

To refine the estimate of the median collapse point, the median obtained in Step 5 is compared to the maximum spectral acceleration value which had *p[collapse]*

< 0.50. If these two values are more than some specified tolerance apart (e.g. *tolerance = 0.03g*) intermediate values are considered. For the example building, the median collapse capacity obtained in Step 5 is *1.76g*. The maximum spectral acceleration value which had *p[collapse]* < 0.50 is *1.74g*. Since these values are within the tolerance specified, the median value is taken as the average, such that *Sa,collapse = 1.75g*.

Algorithm Parameter Definition

To use the proposed algorithm, several parameters need to be defined. Firstly, an initial estimate of the collapse resistance of the structure is needed. This value may be obtained from static pushover analysis or any other reasonable method. Possible methods are discussed in the following section. In addition, the user must specify the step size used in separating subsequent estimates of collapse capacity. After the algorithm determines the bounds on the collapse capacity, the step size decreases. The tolerance in Step 6 must also be assigned. Each of these parameters affects the computational efficiency of the algorithm.

INITIAL ESTIMATE OF STRUCTURAL COLLAPSE RESISTANCE

The proposed algorithm uses nonlinear static (pushover) analysis to obtain an initial estimate of the collapse resistance. By utilizing information available from pushover analysis of a structure, which is not very computationally expensive, the approach is able to significantly reduce the effort needed to complete the dynamic assessment.

Inelastic pushover analysis is conducted by applying a pre-defined lateral load pattern to a structural model. The loads (displacements) on the structure are subsequently increased in order to observe the nonlinear load-displacement response of the structure. Pushover analysis therefore is able to represent redistribution of internal forces and deformation demands as the structure becomes damaged (Krawinkler & Seneviratna 1998, Mwafy & Elnashai 2001). It is most appropriate for structures where the first-mode dominates the response.

A variety of methods have been proposed to estimate the dynamic collapse resistance of a structure from nonlinear static analysis. We consider several previously published methods. FEMA p440a (also known as ATC-62) developed an estimate of the minimum strength to avoid dynamic instability, R_{max}, which is based on the building period and force-displacement quantities from pushover analysis (FEMA 2009b, Equation 4-4). R_{max} may be converted to a spectral ordinate to estimate the median collapse capacity. Chou and Kanvinde (2006) propose an equation to determine median collapse resistance from the ultimate roof displacement from static pushover analysis (defined as the roof displacement at which the base shear is equal to zero). This relationship depends on the number of stories and modification factors to account for multi-degrees of freedom, inelastic and hysteretic response and P-Δ effects. Another method is SPO2IDA, proposed by Vamvatsikos and Cornell (2006). The SPO2IDA tool, derived by relating pushover and dynamic analysis results for single-degree-of freedom systems, uses a building's pushover curve to predict fractile incremental dynamic analysis results. A final simplistic approach is developed here. It assumes that modern code-designed buildings are able to resist a spectral acceleration of 1.8 times the code-defined maximum considered

earthquake (MCE) at the building location. Older, non-code-conforming buildings are assumed to resist a spectral acceleration of 0.75 times the MCE. These values are based on previous research (FEMA 2009) which shows that modern buildings typically have a collapse margin of 1.5 to 2.0 relative to the MCE. Data for older buildings is based on Liel et al. (2009).

In practice, any of these methods (or many others) could be used to develop an initial estimate for the collapse capacity of a building since the algorithm uses dynamic analysis to refine the initial estimate. However, methods which provide better predictions of collapse resistance will demonstrate the most significant reductions in computational time. To investigate the methods described above, a set of 59 buildings was considered, including 25 modern RC frame structures, 26 older (non-ductile) RC frame structures and 8 wood frame structures (Haselton et al. 2009c, Liel et al. 2009, Christovasilis et al. 2008). Results of approximate analysis are compared to dynamic analysis to estimate the median collapse capacity. On average for the buildings considered, SPO2IDA gave median predictions of collapse capacity that were 11.4% higher than dynamic results for modern RC frame buildings, 2.2% lower for nonductile RC frames and 28.1% lower for wood buildings. The FEMA p440a equation over-predicted median capacity of the modern RC frames by 4% on average, and under-predicted the median capacity of the nonductile RC frames and wood buildings by 7.8% and 39.4% respectively. The approximate MCE method gave good results in all cases, over-predicting median capacity by 1.5 to 5.4% depending on the building type. The Chou method, which was developed based on RC frames, provides a better estimation of collapse capacity for the RC frame structures than the wood frame buildings.

ACCURACY AND EFFICIENCY OF PROPOSED ALGORITHM

The proposed algorithm is evaluated in terms of accuracy and efficiency. Accuracy is determined by comparing the results obtained using the proposed algorithm to those obtained by incremental dynamic analysis for the 4-story building. These results are shown in Tables 1 and 2. Key algorithm parameters, including step size and tolerance, are modified in each case to evaluate the robustness of the solution. For the incremental dynamic analysis results, a standard stepping algorithm is used increasing the intensity of ground motions by a constant step from a small value to collapse. Other algorithms, such as hunt-and-fill (Vamvatsikos and Cornell 2002), may provide slightly different results or a decrease in computational time.

As Tables 1 and 2 show, the median collapse capacity obtained using the proposed algorithm is very close to that obtained from incremental dynamic analysis. For the baseline cases (A1 and B1) the median collapse capacity differs by approximately 1%. Table 2 also shows that the accuracy of the proposed algorithm is not significantly affected by the initial estimate of the collapse resistance or the step size used in the analysis. These findings provide confidence that the proposed algorithm has reasonable accuracy to reproduce results from incremental dynamic analysis.

The computational efficiency is reflected in the analysis times reported in Tables 1 and 2. All analyses were conducted on the same Dell Precision quad core computer with two 2.00 GHz processors and 4GB of RAM, so the values should be

comparable. The incremental dynamic analysis ran for 14.2 to 22.8 hours, depending on the algorithm input start value and step size, as reported in Table 1. The proposed algorithm reduces the computational time by 90% on average, with the analyses taking from 1.6 hours to 2.1 hours, as reported in Table 2. Improvements in computational time are expected to be even more significant for commercially-available software packages which typically have much longer run times than the *OpenSees* models used in this example.

Table 1. Results of collapse assessment for 4-story RC frame building, obtained using incremental dynamic analysis with varying parameters.

Run ID	Start Sa Value [g]	Step Size [g]	Analysis Time		Median Collapse Sa [g]
			hr	min	
A1	0.15	0.15	22	47	1.77
A2	0.15	0.30	15	11	1.78
A3	0.15	0.60	14	7	1.77
A4	0.31	0.15	15	10	1.77
A5	0.61	0.15	16	21	1.77
A6	1.01	0.15	16	23	1.76

Table 2. Results of collapse assessment for 4-story RC frame building, analyzed using proposed algorithm with varying parameters.

Run ID	Initial Collapse Estimate, Sa [g]	Method used to Obtain Initial Collapse Estimate	Step Size [g]	Analysis Time		Median Collapse Sa [g]
				hr	min	
B1	1.92	FEMA p440a	0.15	1	42	1.75
B2	1.72	MCE-method	0.15	1	38	1.76
B3	2.14	Chou	0.15	2	4	1.76
B4	1.92	FEMA p440a	0.10	1	36	1.77
B5	1.92	FEMA p440a	0.30	1	38	1.75
B6	1.92	FEMA p440a	0.60	2	4	1.78

Table 3 reports the computational time for collapse assessment of three additional buildings using the proposed algorithm: 2, 8 and 12-story modern RC frames. As expected, there is an increase in computational time for the taller buildings with more degrees of freedom. In terms of accuracy, the analyses show good agreement with incremental dynamic analysis with the exception the 2-story building, for which the results of the proposed algorithm differ by approximately 8%. This error is likely introduced by the combination of a "resurrecting" record and inaccuracies in the initial collapse estimate.

The reduction in computational time comes from the use of an initial estimate of the collapse resistance. In addition, the algorithm takes advantage of information gained at each analysis level to reduce the number of ground motion records that need to be run at subsequent levels. For the example shown in Figure 2, the median collapse capacity is obtained from a total of 113 individual time history analyses (different records and different scaling factors). The same analysis using incremental dynamic analysis requires running approximately 530 individual time history

analyses. Note that all runs are not equivalent in terms of computational time as it takes longer to solve the underlying numerical system of equations when the building is close to collapse (Vamvatsikos and Cornell 2006).

Table 3. Results of collapse assessment for 2, 4, 8 and 12-story RC frames, using proposed algorithm.

Num of Stories	Initial Collapse Estimate, Sa [g]	Method used to Obtain Initial Collapse Estimate	Analysis Time		Median Collapse Sa [g]	
			hr	*min*	*Proposed Alg.*	*IDA*
2	4.19	FEMA p440a	1	28	3.36	3.09
4	1.92	FEMA p440a	1	42	1.75	1.75
8	0.71	FEMA p440a	3	27	0.78	0.80
12	0.62	FEMA p440a	5	24	0.66	0.62

ALGORITHM ADVANTAGES AND LIMITATIONS

A number of other researchers have developed improved incremental dynamic analysis algorithms. The simplest method to conduct incremental dynamic analysis is a 'stepping' algorithm, in which the intensity of the ground motion is increased by a constant step from zero to collapse (Vamvatsikos and Cornell 2002, Yun et al. 2002), as applied here. A 'hunt-and-fill' method expends fewer analyses in finding the initial estimate of the collapse point because the step size is increased at each step, until the structure collapses (Vamvatsikos and Cornell 2002). This approach is similar to that proposed here, except that a series of dynamic analyes is used to obtain the initial estimate of the collapse point (i.e. the "hunt" phase) rather than results of static pushover analysis, as suggested here. Vamvatiskos and Cornell (2002) also suggest filling in results at a number of lower intensities to obtain the whole IDA curve.

A number of researchers have advocated using nonlinear static analyses in lieu of incremental dynamic analysis where computational efficiency is important. The SPO2IDA method (Vamvatsikos & Cornell 2006), the N2 method (Fafjar & Gaspersic 1996) and Han and Chopra (2005)'s modal pushover approach use pushover results to approximate dynamic analyses results. The methods tend to provide reasonable accurate results for first-mode dominated buildings for which the single-degree-of-freedom pushover assumption is appropriate. Other researchers try to reduce the computational effort in incremental dynamic analysis by reducing the number of ground motions needed in the suite of records. Azarbakht and Dolsek (2007) use a SDOF model to establish a precendece list of ground motions to reduce the number of records used. The combination of static pushover analysis with nonlinear time history analysis in the proposed algorithm distinguishes it from other methods to reduce the computational effort involved in incremental dynamic analysis.

As a result of the modifications made to reduce computational time, the proposed algorithm has more limited application than incremental dynamic analysis. In incremental dynamic analysis, the median collapse capacity is computed from the spectral acceleration at which each ground motion record collapses. These statistics also permit computation of the standard deviation, which represents the uncertainty in the collapse capacity prediction associated with ground motion record-to-record

variability. The proposed algorithm cannot be used to quantify the standard deviation. However, it has been shown (e.g. Haselton et al. 2009, FEMA 2009) that the uncertainty in the collapse fragility function corresponds to a logarithmic standard deviation of approximately 0.40 in most cases and FEMA p695 does not require computation of this value.

In addition, incremental dynamic analysis is capable of representing structural response from elastic to yielding to global instability. Since the proposed algorithm does not consider the response of the structure under low levels of spectral acceleration, i.e. excitations significantly less intense than those that cause collapse, the method cannot be used to quantify earthquake-induced damage and economic losses. Application of the proposed algorithm should be carefully considered where resurrecting records may introduce errors.

CONCLUSIONS

This paper proposes a modified algorithm to determine the collapse capacity of a structure, utilizing results of pushover analysis to obtain an initial estimate of the collapse resistance. The proposed practical approach takes advantage of information gained from pushover analysis to reduce computational effort, but retains the accuracy obtained from dynamic analysis. For the buildings considered, the approach reduces computational time by approximately 90%. The algorithm is also robust, and provides stable results regardless of input parameters.

Assessments of structural resistance to earthquake-induced collapse have become increasingly accepted as metrics of seismic safety for evaluating new and existing buildings and building code provisions (ATC 2009, FEMA 2009a, Luco et al. 2007). The algorithm proposed here aims to reduce the computational effort expended in conducting these assessment. This reduction may improve the feasibility of using commercially-available software packages to predict collapse resistance as these methods become more widely used.

REFERENCES

ATC (2009). *Guidelines for Seismic Performance Assessment of Buildings,* (ATC-58 50% Draft) prepared for U.S. Department of Homeland Security and FEMA, Washington, D.C.

Azarbakht, A., Dolsek, M. (2007). "Prediction of the median IDA curve by employing a limited number of ground motion records." *Earthquake Eng. and Structural Dynamics,* 36(15), 2401–2421.

Chou, J.H. (2006). "Simplified methods to predict earthquake induced sidesway collapse in modern reinforced concrete special moment frames." *M.Sc. Thesis*, Dept. of Civil & Env. Eng., University of California, Davis.

Christovasilis, I., Filiatrault, A., Constantinou, M., Wanitkorkul, A. (2008). "Incremental dynamic analysis of woodframe buildings." *Earthquake Eng. and Structural Dynamics*, 38(4), 477-496.

Fajfar, P., Gaspersic, P. (1996). "The N2 method for the seismic damage analysis for RC buildings." *Earthquake Eng. and Structural Dynamics*, 25(1), 23–67.

FEMA (2009a) *Recommended Methodology for Quantification of Building System Performance and Response Parameters,* (FEMA P695). Prepared by ATC, Redwood City, CA.

FEMA (2009b) *Effect of Strength and Stiffness Degradation on Seismic Response*, (FEMA P440a). Prepared by ATC, Redwood City, CA.

Han, S. W., Chopra, A. K. (2006). "Approximate incremental dynamic analysis using the modal pushover analysis procedure." *Earthquake Eng. Structural Dynamics,* 35,1853–73.

Haselton, C.B., Liel, A.B., Deierlein, G.G. (2009a). "Simulating Structural Collapse due to Earthquakes: Model idealization, Model Calibration, and Numerical Solution Algorithms." *Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN)*, Greece, June 2009.

Haselton, C.B., Editor (2009b). "Evaluation of Ground Motion Selection and Modification Methods: Predicting Median Interstory Drift Response of Buildings.*" PEER (2009/01),* University of California, Berkeley.

Haselton, C.B., A.B. Liel, G.G. Deierlein, B.S. Dean, and J.H. Chou (2009c). "Seismic Collapse Safety of Reinforced Concrete Buildings: I. Assessment of Ductile Moment Frames.", *Journal of Structural Engineering* (accepted).

Haselton, C.B., J.W. Baker, A.B. Liel, and G.G. Deierlein (2009d). "Accounting for Expected Spectral Shape (Epsilon) in Collapse Performance Assessment", *Journal of Structural Engineering* (in press).

Ibarra, L. F., Medina, R. A., Krawinkler, H. (2005). "Hysteretic models that incorporate strength and stiffness deterioration." *Earthquake Eng. and Structural Dynamics,* 34(12), 1489-511.

Krawinkler, H., Seneviratna, G.D. (1998). "Pros and cons of a pushover analysis of seismic performance evaluation." *Engineering Structures*, 20(4–6), 452–64.

Liel, A.B., Haselton, C.B. and Deierlein, G.G. (2009). "Seismic Collapse Safety of Reinforced Concrete Buildings: I. Comparative Assessment of Non-ductile and Ductile Moment Frames.", *Journal of Structural Engineering* (accepted).

Luco, N., Ellingwood, B. R., et al. (2007). "Risk-targeted versus current seismic design maps." *SEAOC 2007 CONVENTION PROCEEDINGS*.

Mwafy, A. M., Elnashai, A. S. (2001). "Static pushover versus dynamic collapse analysis of RC buildings." *Engineering Structures,* 23 (2001), 407–24.

PEER (2009). Open System for Earthquake Engineering Simulation (*OpenSees*). http://opensees.berkeley.edu

Vamvatsikos, D., Cornell, C.A. (2002). "Incremental Dynamic Analysis." *Earthquake Eng. and Structural Dynamics*, 31 (3), 491-514.

Vamvatsikos, D., Cornell, C.A. (2004). "Applied incremental dynamic analysis." *Earthquake Spectra*, 20 (2), 523-53.

Vamvatsikos, D., Cornell, C. A. (2006). "Direct estimation of the seismic demand and capacity of oscillators with multi-linear static pushovers through IDA." *Earthquake Eng. and Structural Dynamics,* 35 (9),1097–117.

Yun, S.Y., Hamburger, R.O., Cornell, C.A., Foutch, D.A. (2002). "Seismic performance evaluation for steel moment frames." *Journal of Structural Engineering,* 128 (4), 534-5.