16th ASCE Engineering Mechanics Conference
July 16-18, 2003, University of Washington, Seattle

DYNAMIC BEHAVIOR OF PLANAR FRAMES DURING PROGRESSIVE COLLAPSE

Griengsak Kaewkulchai[1],
and Eric B. Williamson[2], Member ASCE

[1] Graduate Research Assistant, Department of Civil Engineering, University of Texas, Austin, TX 78712. E-mail: nguang@ mail.utexas.edu
[2] Assistant Professor, Department of Civil Engineering, University of Texas, Austin, TX 78712. E-mail: ewilliamson@mail.utexas.edu

ABSTRACT

This paper describes ongoing research concerning the dynamic progressive collapse of planar frame structures. Previously, the authors have described the development of analysis software to simulate the behavior of structural frames during a collapse event. The software considers the effects of both geometric and material nonlinearity, and it incorporates a damage index to account for strength and stiffness degradation in the members. The damage index is also used to determine the onset of member failure. Early research emphasized the importance of considering dynamic load redistribution following the failure of one or more members. In the current paper, sample analyses and results of damaged frames subjected to an initial collapse event are presented. The results show that the response of frames is not strongly dependant upon whether the analysis starts from a deformed or an undeformed configuration at the time of column failure. Furthermore, accounting for damage not only extends the structural period, but also increases the level of displacements and plastic rotations.

Keywords: progressive collapse, building collapse, damage, structural dynamics, nonlinear analysis

INTRODUCTION

Great attention has been focused on a type of failure known as progressive collapse since the Ronan Point apartment collapse in London in 1968 (Griffiths et al. 1968). Progressive collapse can be defined as a chain reaction of failure initiated by a loss of one or a few supporting elements. A key aspect is that the resulting damage is disproportionate to the original cause. Although there was a large amount of research on progressive collapse during the 1970s and early 1980s, research activity in this area slowed tremendously throughout most of the mid to late 1980s and 1990s. In recent years, however, terrorist attacks against the Alfred P. Murrah Building in Oklahoma City in 1995, and the World Trade Center in New York in 2001, have regenerated interest in progressive collapse of structures.

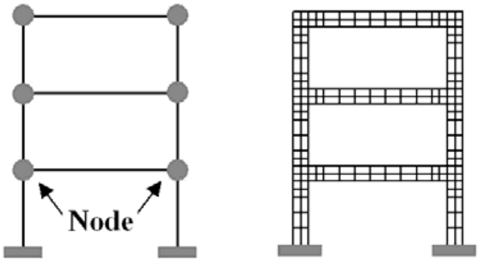
Many approaches have been proposed to prevent or reduce the risk of progressive collapse in buildings (Breen 1976, Leyendecker and Ellingwood 1977). In most current building codes (e.g., NBC 1995, ASCE 1996, IBC 2000), an indirect design approach has been integrated into the specifications through mandatory strength, ductility, and continuity requirements. In addition, a direct design procedure known as the 'Alternate Load Path' method is described by these codes for investigating the potential of progressive collapse in the design of buildings. This method is based on an assumed static redistribution of loads following the failure of a key structural element. Using a static approach for the analysis of progressive collapse, however, has been shown not to be conservative (Pretlove et al. 1991).

In order to overcome the limitations of the Alternate Load Path method, a framework for computing the dynamic response of frame structures during a progressive collapse event has been proposed. The current research focuses on developing an analysis tool for studying the dynamic response of planar frame structures subjected to an initiating localized failure. A computer program has been developed to simulate the dynamic behavior of planar frames up to, and through, collapse. This paper describes ongoing research on the dynamic progressive collapse of planar frame structures, and addresses some details not included in previous work by the authors (Kaewkulchai and Williamson 2002). Sample analyses and results of damaged frames subjected to an initial collapse event are presented.

SOLUTION PROCEDURE AND SOFTWARE DEVELOPMENT

A basic description of the solution procedure and software development used in the study of dynamic progressive collapse of planar frame structures is given in this section. Some information has already been discussed in previous work by the authors (Kaewkulchai and Williamson 2002). A more detailed explanation of these subjects has also been given in the literature by the authors (Kaewkulchai and Williamson (in press)).

For modeling of a planar frame structure, a discrete element model as shown in Figure 1(a) is used. In this model, a frame is formed by an assembly of discrete members (i.e., beam-column elements) that describe individually the hysteretic behavior of structural components. These discrete elements are generally connected at nodes located at the element ends in which degrees of freedom, corresponding to the nodes, represent the structural response. Note that a frame structure can be modeled using a finite element model in which the members and joints of the frame are divided into a large number of finite elements as shown in Figure 1(b). Use of this model, however, requires extensive computational resources for the nonlinear dynamic analysis of large structures. Given the inherent uncertainties in predicting the response of a large frame structure during a collapse event, a detailed finite element model is not well suited for conditions being considered in this research.



(a) (b)

Figure 1. Modeling of frame structures. (a) Discrete element model, (b) Finite element model.

The developed software utilizes the conventional direct stiffness method for the analysis of planar frame structures. An implicit direct integration scheme, the Newmark-beta method (Newmark 1959), is employed to solve the governing equations of dynamic equilibrium coupled with Newton-Raphson iterations for carrying out the nonlinear analyses. In this study, a classical or proportional (Rayleigh) damping matrix is assumed along with the use of a lumped mass matrix. Geometric nonlinearity (P-Δ effect) is taken into account by using a simplified geometric stiffness matrix. For material nonlinearity, the beam-column element originally developed by Kim (1995) is used. The element incorporates a lumped plasticity model to capture the nonlinear behavior of frame members. Inelasticity of the element due to yielding under a combination of axial force and moment is assumed to occur only at the element ends or hinges. The effects of strength and stiffness degradation of members are modeled by means of a damage model. The damage model, as described in the following section, utilizes a damage index at each member end to determine the onset of member end failure.

Damage Model

In the literature, several damage models have been proposed that depend upon a damage index, D, having a value ranging from 0 (no damage) to 1 (total damage). Examples include those of Park and Ang (1985), and Rao et al. (1998) for concrete members, and Krawinkler and Zohrei (1983), and Azevedo and Calado (1994) for steel members. For the current research, a damage index D that depends linearly upon the maximum deformation and the accumulated plastic energy is proposed. To wit,

$$D = \alpha U(\delta) + \beta W(\delta)$$

where α, β are constant (material) parameters, $U(\delta)$ is a function that depends on the maximum deformation, and $W(\delta)$ is a function that depends on the accumulated plastic energy.

By varying the values of α and β, different rates of damage accumulation can be obtained so that many of the models presented in the literature can be represented (Williamson and Hjelmstad 2001). During an analysis, the damage indices of frame members are updated at each time step. Thus, the subsequent response of the damaged frame being analyzed depends not only upon the previous stress-strain history, but also upon the rate at which damage accumulates.

Member End Failure

A damage index, D, employed to account for the effects of strength and stiffness degradation of members, is also used to determine the onset of member failure. For each member, a damage index associated with each member end, D_i at hinge i and D_j at hinge j, is calculated and updated at each time step. When the damage index of a hinge reaches a value of one (total damage), the hinge is assumed to fail or separate completely from the main structure (Figure 2). At this point, the failed hinge becomes discontinuous from its primary joint, but the hinge on the opposite end of the member can still be intact. Following component failure, a modified member stiffness procedure with releases of end forces is used to track the response of the failed end without the need of an additional node (Kaewkulchai and Williamson (in press)).

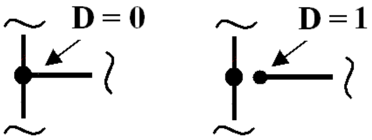


Figure 2. Hinge separation (total damage), *D* = 1.

Post Analysis after Member End Failure

As mentioned earlier, computer software has been developed to simulate the dynamic behavior of damaged planar frames up to, and through, collapse. Hence, the software is capable of performing an analysis even after one or more members have failed. While this capability is not new, it is one that is not widely available for implicitly-based analysis software. To continue analyzing a frame after one or more members have failed, various computational parameters must be updated. To illustrate, consider the structural frame in Figure 3 in which a beam member has the internal forces at end *i* equal to F_i, V_i and M_i, and at end *j* (before failure) equal to F_j, V_j and M_j. To maintain equilibrium at the onset of hinge separation, the internal forces of the failed hinge (end *j*) are released and applied back to the main structural frame. At this point, the externally applied load vector of the frame is modified to include those internal forces resulting from the failed end. At the failed end, equal and opposite internal forces are applied so that the member forces at this end become zero. Within that time step, the mass matrix is modified to represent the loss of mass contributing to the failed end of the beam member. Then, damping and stiffness matrices are modified to represent the current state of the structural configuration. In addition, jumps in acceleration at the node connecting to the failed hinge will occur due to the suddenly released forces. These jumps in acceleration can be obtained by satisfying the new equilibrium conditions of the node. After updating nodal accelerations, the analysis can then be carried forward to the next time step.

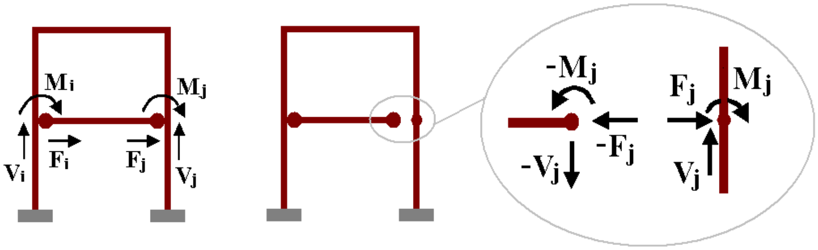


Figure 3. Post analysis after member end failure.

EXAMPLE ANALYSES AND RESULTS

In this section, example analyses and results for the dynamic response of planar frames subjected to an initiating collapse event are presented. Figure 4 shows member properties and dimensions of two structural configurations, A and B, having the same uniform load, *w,* of 0.4 kips/in acting on the beams. The failure of the first floor column on the right end forms the first initial collapse scenario, while the second initial collapse scenario is due to the failure of the first floor column that is second from the right end. Two cases of initiating collapse scenarios for the two-bay, two-story frame (C1-A and C2-A) are shown in Figure 5.

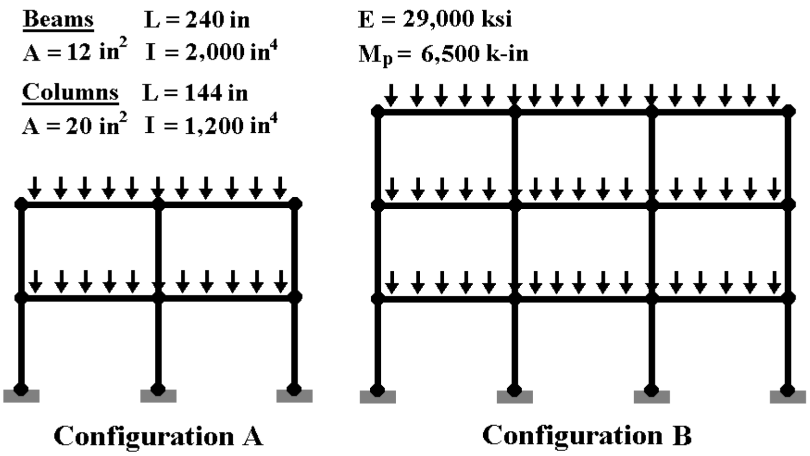


Figure 4. Frame structural configurations and member properties.

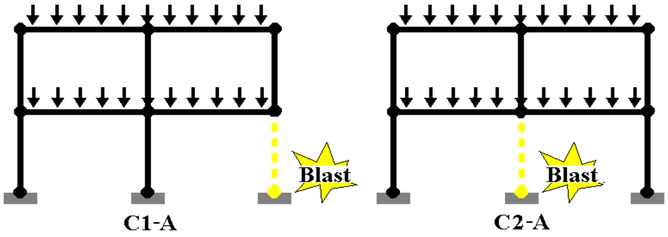


Figure 5. Two cases of initiating collapse scenarios.

For configuration A, the dynamic response of the frame obtained from analyses starting with a deformed configuration is compared with those starting with an undeformed configuration. Figure 6(a) shows analysis concepts of cases C1-A and C2-A that account for structural deformations prior to the assumed initiating collapse events, while Figure 6(b) shows analysis concepts in which the frame is assumed to be at rest in its original configuration, then subjected to a suddenly applied load *w*. To represent the case in which there are initial deformations, forces equal and opposite to the member forces of the failed column are applied to the node connecting to the failed column. In this case, both the uniform load *w* and the applied forces (*P*, *V*, and *M*) are slowly applied to the frame so that static deformations are obtained. At a time step where all loads reach their peak value, the applied forces (*P*, *V*, and *M*) are removed to simulate an initiating collapse event (see Figure 6a). For dynamic analyses performed in these examples, the time step size is set to be 0.01 sec, and a beam mass of 0.124 kips-s^2/in at each end is used for all beam members. In addition, as is typical when using a lumped mass procedure, rotational inertia of the ends is ignored. Damping is also ignored in these simulations due to the fact that material inelasticity tends to dominate energy dissipation for systems in which yielding of the elements occurs.

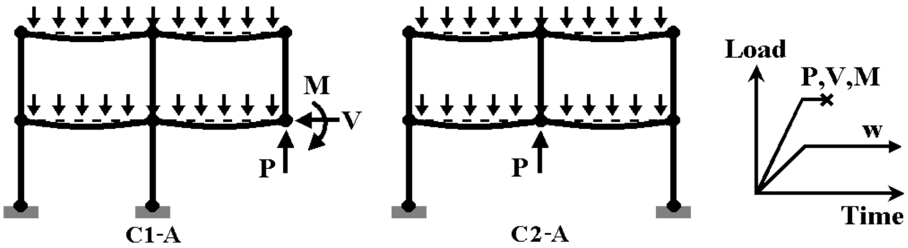


Figure 6(a). Analyses starting with deformed configurations.

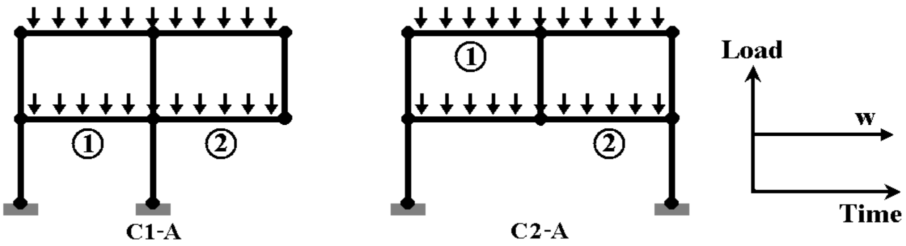


Figure 6(b). Analyses starting with undeformed configurations.

The results obtained from the dynamic analyses starting with deformed and undeformed configurations for cases C1-A and C2-A are summarized in Figures 7 and 8. Figure 7 shows displacement histories at the nodes connecting to the failed columns. In both cases, the displacement response of the frame starting from the original configuration shows slightly greater amplitude with the same vibration period when compared to the response obtained from the frame assuming failure occurred while it was in its deformed configuration. Moment histories of members 1 and 2, indicated in Figure 6(a), are given in Figure 8. In case C1-A, the moments obtained from the undeformed and deformed configurations in both members1 and 2 match very well during the course of the analysis and show very little discrepancy toward the end of the analysis. For the case C2-A, the results compare even more favorably. From these results, it is possible to conclude that there is little difference in the dynamic response of the frame obtained from analyses starting with either a deformed configuration or an undeformed configuration at the time of column failure.

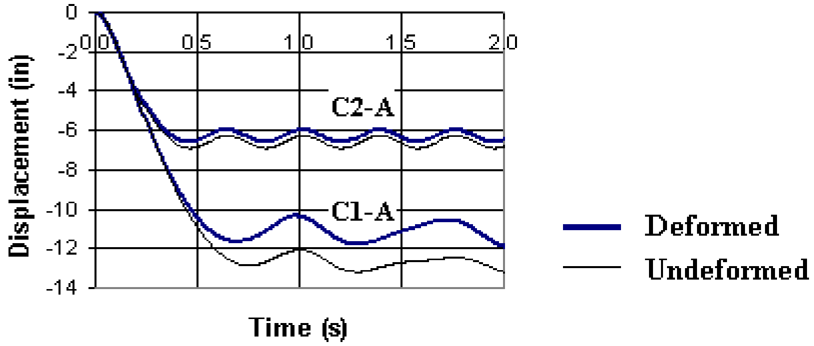
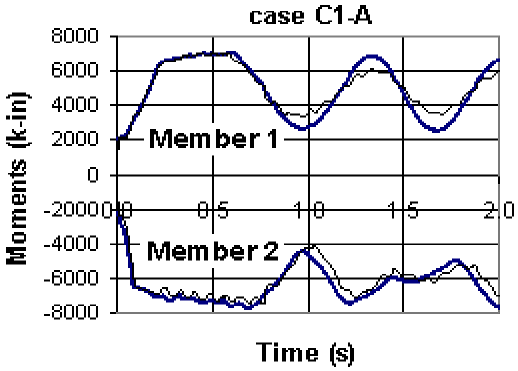


Figure 7. Displacement histories at the failed nodes for cases C1-A and C2-A.



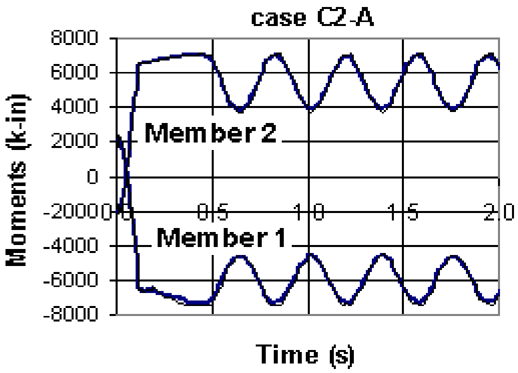


Figure 8. Moment histories of members 1 and 2 for cases C1-A and C2-A.

It is interesting to note that the dynamic analyses starting from the undeformed configuration, while conservative and simple to perform, produce results that show some vibration noise due to the contributions from higher-modes, especially for column axial forces as shown in Figure 9. The reason for this behavior is that the analyses started with a force that is suddenly applied, and hence produced axial vibration in the columns at the beginning of the analyses. Nonetheless, the results show that the global response of frames is not strongly dependant upon whether the analysis starts from a deformed or an undeformed configuration at the time of column failure.

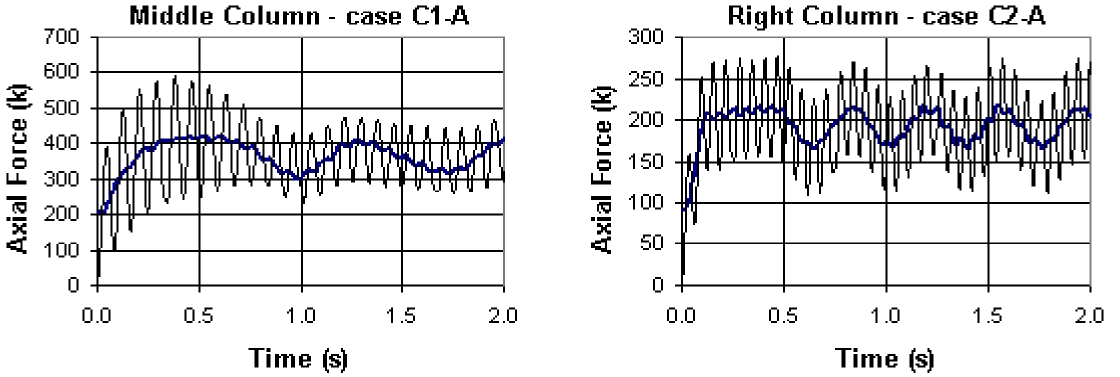


Figure 9. Axial force histories of 1st-story columns for cases C1-A and C2-A.

Based on the previous results, analyses starting with the original configuration will be used to study frame configuration B. The two initiating collapse scenarios (C1-B and C2-B) used in previous calculations are also considered for this case. Both static and dynamic analyses are performed, and the computed results are compared. In these examples, the damage parameters (α, β) are first set to be zero to allow a direct comparison between the results obtained from the static and dynamic analyses of the undamaged frame. Then, in subsequent dynamic analyses, the first damage parameter α is set to be 0.02 so that when the maximum rotation θ_{max} is equal to $50 \cdot \theta_y$, the damage index will equal to one. The second parameter β is assumed to vary from 0.00-0.03 to represent damage caused from energy dissipation. All other analysis parameters from the previous examples remain unchanged. By using higher values of α and β, a higher rate of damage accumulation is obtained. As previously discussed, the effects of strength and stiffness degradation of frame members are accounted for by means of the damage index. As a result, the response of damaged frames also depends on the rate at which damage accumulates. For a higher rate of damage accumulation, localized damage at member hinges reduces the member strength and stiffness, and therefore the overall frame strength and stiffness, at a faster rate. Hence, a higher rate of damage accumulation results in a more flexible frame.

The results obtained from the static and dynamic analyses without considering damage for cases C1-B and C2-B are summarized and compared in Figure 10 and Table 1. Figure 10 shows plastic hinge locations obtained from the analyses. As can be seen from the figure, including inertial effects results in a response behavior with greater numbers of plastic hinges. In these examples, during dynamic load redistribution, the plastic hinges do not spread to the additional bay. In Table 1, vertical displacements at the failure nodes and plastic hinge rotations at various points throughout the structure are compared for the static and dynamic analyses to assess the level of plasticity. In addition, dynamic increase factors (DIF) are determined by computing the ratio of the maximum response for the dynamic cases to the static cases.

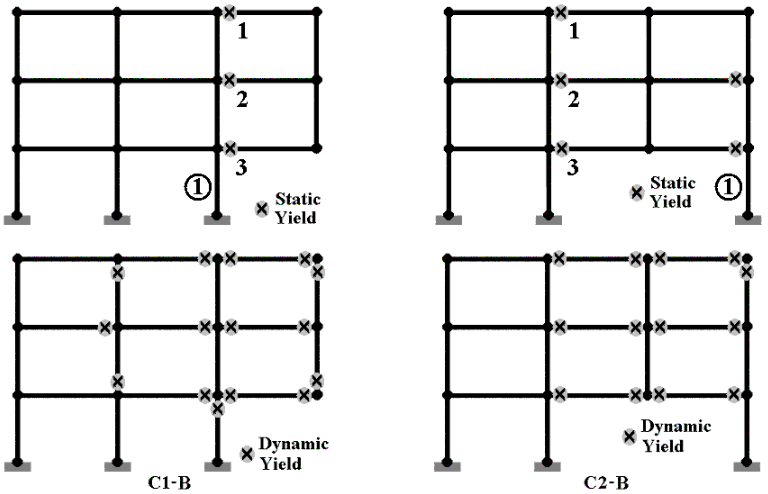


Figure 10. Plastic hinge locations obtained from static and dynamic analyses.

Table 1. Comparisons of displacement and plastic rotations.

	Case	Static Analysis	Dynamic Analysis	DIF
Vertical Displacement	C1-B	-3.277	-9.479	2.893
Plastic Rotation (Point 1)	C1-B	-0.006	-0.026	4.333
Plastic Rotation (Point 2)	C1-B	-0.008	-0.028	3.500
Plastic Rotation (Point 3)	C1-B	-0.008	-0.028	3.500
Vertical Displacement	C2-B	-1.728	-6.437	3.725
Plastic Rotation (Point 1)	C2-B	-0.002	-0.024	12.000
Plastic Rotation (Point 2)	C2-B	-0.004	-0.024	6.000
Plastic Rotation (Point 3)	C2-B	-0.004	-0.024	6.000

As can be seen from this table, the dynamic increase factors (DIF) for the vertical displacements range from 2.89 to 3.73, and those for the plastic rotations range from 3.50 to 12.00. In these sample cases, the results demonstrate that accounting for dynamic effects leads to considerably greater inelastic deformations throughout the frames. Consequently, the large amount of increased inelastic deformations in the members may result in more member failures and trigger a progressive collapse event. Thus, accounting for dynamic load redistribution appears to be an important feature in predicting the potential for progressive collapse of frames.

For the dynamic analyses considering damage, the computed results for cases C1-B and C2-B are summarized and compared through Figures 11 and 12 and Table 2. The displacement histories at the failed nodes and the moment history of column 1 (see Figure 10) are shown in Figure 11 for case C1-B, and in Figure 12 for case C2-B. In both cases, when considering the vertical displacement history, damage softened the frame strength and stiffness and resulted in larger amplitude response with a longer period of vibration. During the time history analysis, the moment of column 1 for case C1-B was slightly smaller for the cases in which damage is considered as might be expected due to less moment transfer from the connecting beams. However, in case C2-B, only a small change in structural period was noticeable. The most likely reason for this behavior is due to the fact that the horizontal displacement in case C2-B for the damaged frame was greater than that of the undamaged one, which results in some additional moment at the column due to sway.

In Table 2, comparisons of plastic rotations of damaged and undamaged frames are given. For both cases, accounting for damage increases the level plastic rotations at all locations. In addition, using higher rates of damage accumulation results in larger plastic rotations.

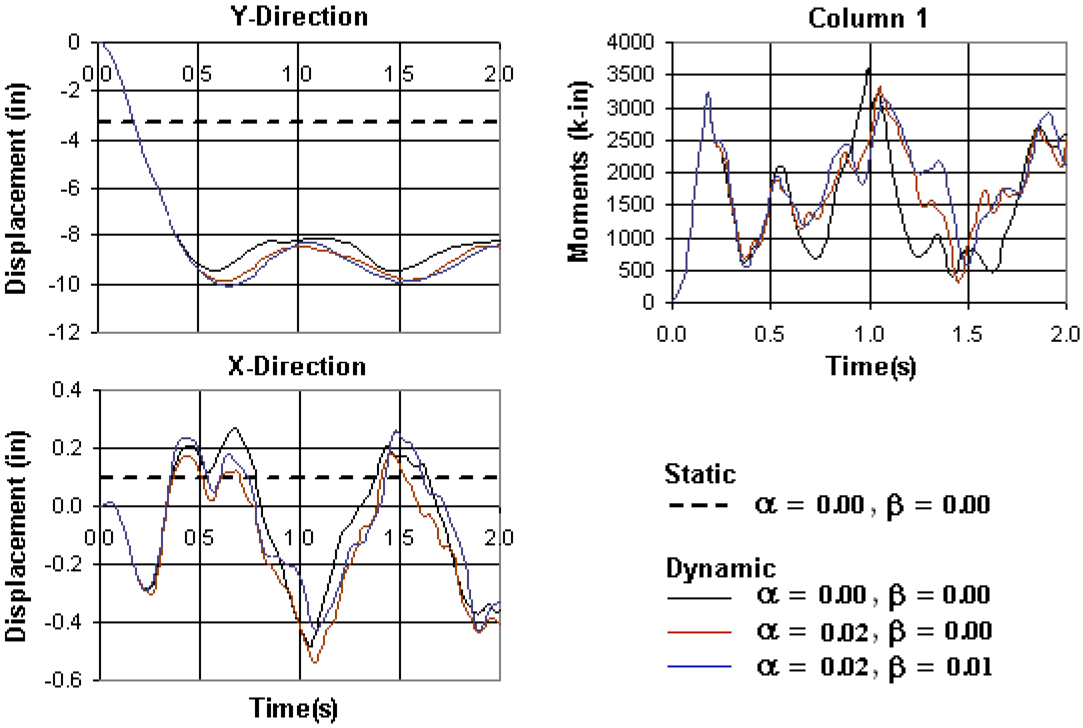


Figure 11. Displacement histories at the failed nodes and moment history of column 1 for case C1-B with damage parameters.

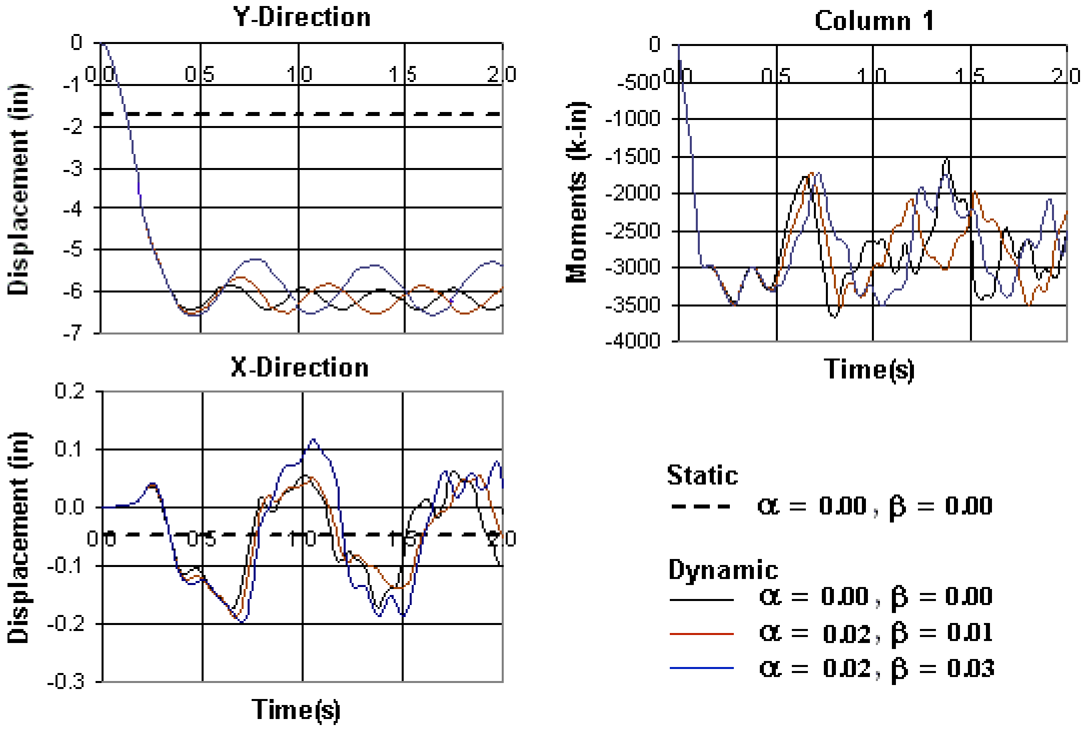


Figure 12. Displacement histories at the failed nodes and moment history of column 1 for case C2-B with damage parameters.

Table 2. Comparisons of plastic rotations of damaged and undamaged frames

	Case	No damage	Damage
Plastic Rotation (Point 1), α = 0.02, β = 0.00 α = 0.02, β = 0.01	C1-B	-0.026	-0.030 -0.036
Plastic Rotation (Point 2), α = 0.02, β = 0.00 α = 0.02, β = 0.01	C1-B	-0.028	-0.032 -0.037
Plastic Rotation (Point 3), α = 0.02, β = 0.00 α = 0.02, β = 0.01	C1-B	-0.028	-0.035 -0.045
Plastic Rotation (Point 1), α = 0.02, β = 0.01 α = 0.02, β = 0.03	C2-B	-0.024	-0.025 -0.034
Plastic Rotation (Point 2), α = 0.02, β = 0.01 α = 0.02, β = 0.03	C2-B	-0.024	-0.028 -0.036
Plastic Rotation (Point 3), α = 0.02, β = 0.01 α = 0.02, β = 0.03	C2-B	-0.024	-0.027 -0.034

SUMMARY AND CONCLUSIONS

Progressive collapse has been known to be an important design consideration since the 1970s, but interest in this topic waned until recently when several buildings failed by progressive collapse due to terrorist attacks. Current building codes and provisions have addressed the progressive collapse issue through standard structural requirements, i.e., strength, ductility, redundancy, and continuity requirements. In addition, a direct design procedure known as the 'Alternate Load Path' method is also recommended by the current building codes as an analysis technique for investigating the potential of progressive collapse in the design of buildings. However, the results obtained from this method can be unconservative as a result of neglecting inertial effects that play a dominant role following the sudden failure of one or more structural members. In the current research, our focus has been on developing an analysis tool for studying the dynamic response of planar frame structures subjected to an initiating localized failure.

This paper presents some information on the development of our progressive collapse analysis software and solution methodology. Sample analyses and results of damaged frames subjected to initial collapse events are presented. From example cases considering dynamic load redistribution, dynamic effects appear to have significant impact on the response behavior of the frames. Furthermore, damage appears to be contained only to the collapsing bay. The results also show that there is little difference in the response of frames whether or not the analysis starts from a deformed or an undeformed configuration at the time of column failure. In addition, accounting for damage not only extends the structural period but also increases the level of displacements and plastic rotations. Future research will focus on conducting parametric studies to identify key factors that contribute to the progressive collapse of planar frame structures.

REFERENCES

ASCE (1996), Minimum Design Loads for Buildings and Other Structures, ASCE 7-95, New York, USA.

Azevedo, J. and Calado, L. (1994), Hysteretic behavior of steel members: Analytical models and experimental tests, Journal of Construction Steel Research, 29, 71-94.

Breen, J. E., editor (1976), Research Workshop on Progressive Collapse of Building Structures, Summary Report, HUD-PDR-182, Department of Housing and Urban Development, Washington, D. C., 112 p.

Griffiths, H., Pugsley, A., and Saunders, O. (1968), Report of the Inquiry into the Collapse of Flats at Ronan Point, Canning Town, Her Majesty's Stationary Office, London, England.

IBC (2000), International Building Code, International Code Council, USA.

Kaewkulchai, G.., and Williamson, E. B. (2002), Dynamic Progressive Collapse of Frame Structures, The 15th Engineering Mechanics Division Conference. New York: ASCE.

Kaewkulchai, G.., and Williamson, E. B., Beam Element Formulation and Solution Procedure for Dynamic Progressive Collapse Analysis, Computers & Structures (in press).

Kim, K. D. (1995), Development of Analytical Models for Earthquake Analysis of Steel Moment Frames, Ph.D. Dissertation, University of Texas at Austin.

Krawinkler, H., and Zohrei, M. (1983), Cumulative Damage in Steel Structures Subjected to Earthquake Ground Motions, Computers & Structures, 16(1-4), 531-541.

Leyendecker, E. V., and Ellingwood, B. R. (1977), Design Methods for Reducing the Risk of Progressive Collapse in Buildings, NBS Building Science Series 98, National Bureau of Standards, Washington, D. C., 68 p.

NBC (1995), National Building Code of Canada, Canadian Commission on Building and Fire Codes, Canada.

Newmark, N. M. (1959), A Method of Computation for Structural Dynamics, Journal of Engineering Mechanics, 85, 67-94.

Park, Y. J., and Ang, A. H. S. (1985), Mechanistic Seismic Damage Model for Reinforced Concrete, Journal of Structural Engineering, 111(4), 722-739.

Pretlove, A. J., and Ramsden, M., Atkins, A. G. (1991), Dynamic Effects in Progressive Failure of Structures, International Journal of Impact Engineering, 11(4), 539-546.

Rao, P. S., Sarma, B. S., Lakshmanan, N., and Stangenberg, F. (1998), Damage model for reinforced concrete elements under cyclic loading, ACI Materials Journal, 95(6), 682-690.

Williamson, E. B., and Hjelmstad, K. D. (2001), Nonlinear dynamics of a harmonically-excited inelastic inverted pendulum, Journal of Engineering Mechanics, 127(1), 52-57.