COLLAPSE ANALYSIS MADE EASY (MORE OR LESS)

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Abstract

The purpose of a progressive collapse analysis is to determine whether a structure remains stable when one or more members (usually columns or bearing wall panels) are suddenly removed. Progressive collapse analyses can be performed using linear or nonlinear methods of structural analysis. This paper is concerned mainly with nonlinear methods.

This paper considers the following topics.
- The basic concepts of progressive collapse analysis.
- Linear vs. nonlinear analysis methods.
- Static vs. dynamic analysis.
- Modeling requirements for nonlinear analysis.
- Special features that are needed for progressive collapse analysis.
- Progressive collapse analysis vs. analysis for earthquake resistant design.
- The Perform-Collapse computer program, as a practical tool for use in design.

Introduction

Progressive collapse occurs when the loss or failure of one member in a structure leads to loss or failure of other members, progressing through the structure and leading to partial or full collapse. Progressive collapse analysis is a design tool that can be used to assess whether progressive collapse is likely to occur. This paper is concerned only with the effects of sudden member removal, not with the causes

It is important to keep the goal in mind. A collapse analysis is a merely a design tool. Its purpose is not to provide an exact simulation of structural collapse, but to provide the designer with useful information for assessing the performance of the structure and making reasonable decisions about its safety. The same is true of analyses for earthquake loads. The purpose is not exact simulation of the dynamic response of the structure (which is an impossible task), but a reasonable assessment of its performance.

Behavior Following Sudden Removal of a Column

The essential aspects of behavior following sudden removal of a column can be illustrated using the simple frame shown in Figure 1.
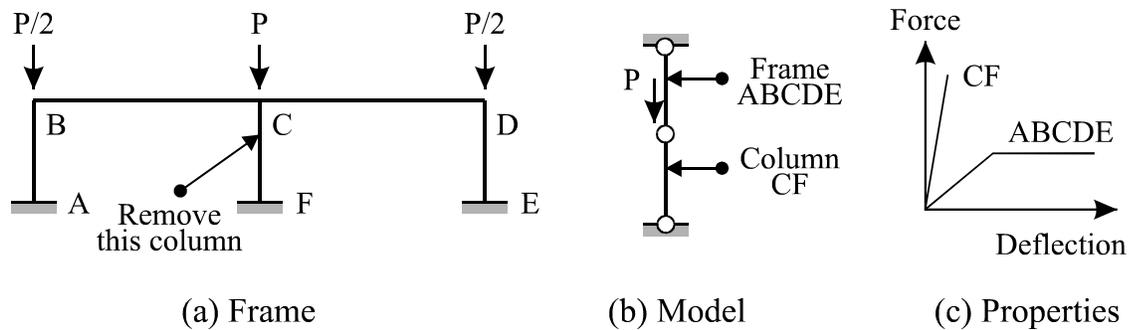
Figure 1. Simple Frame to Illustrate Behavior

Figure 1(a) shows the frame. When the frame is intact, column CF takes essentially all of the load P, and hence the force in the column is P. If the column is suddenly removed, its force is suddenly transferred to frame ABCDE, and the frame responds dynamically. Depending on its strength, the frame may remain essentially elastic, it may yield but not collapse, or it may collapse completely.

Figure 1(b) shows an analysis model consisting of two bars, one to model the column CF and the second to model the frame ABCDE. A mass at the load point models the vertical inertia of frame ABCDE. Figure 1(c) shows the properties for the column and frame. The column is much stiffer than the frame, and the column is elastic whereas the frame can yield.

The "correct" analysis sequence for sudden removal of the column is : (1) apply load P, then (2) suddenly remove the column CF and calculate the dynamic response. However, the following analysis sequence will give the same results : (1) remove the column CF, then (2) suddenly apply the load P. The results that are of most interest are as follows.

(1)   The maximum deflection at C. From this the following can be found.
(2)   If the frame remains elastic (or is to be designed to remain elastic), the maximum forces in the frame members.
(3)   If the frame is allowed to yield, the maximum deformations (e.g., plastic hinge rotations) in the frame members.

To get the dynamic response it is necessary to perform a dynamic analysis. However, the maximum deflection at C can be calculated without doing a dynamic analysis, by considering energy balance. As the structure deflects, the load P loses potential energy, and the frame ABCDE gains strain energy (in the elastic case) or gains strain energy and dissipates inelastic energy (in the yielding case). When the loss of external potential energy equals the gain in internal energy, the maximum displacement is reached.

Consider the case where the frame ABCDE has an elastic-perfectly-plastic relationship between load and deflection. For this case, Figure 2 shows the maximum deflections for three different frame strengths.
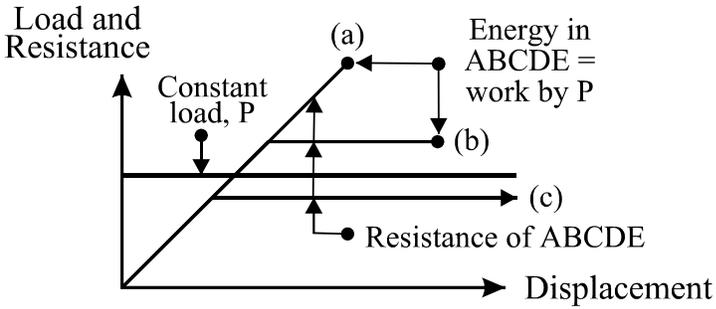
Figure 2. Energy Balance for Elastic-Perfectly-Plastic Behavior

In case (a) the frame ABCDE is very strong. In this case the maximum deflection is twice the deflection that would be obtained by applying the load statically. This deflection, and also the maximum forces in the frame members, can be calculated by performing a conventional linear analysis of the frame for a load equal to twice the actual load.

In case (c) the frame ABCDE has a strength smaller than P. In this case it is not possible to reach an energy balance, and the frame collapses completely.

In case (b) the frame ABCDE has a strength between P and $2P$. The frame yields until an energy balance is reached, and the maximum deflection is larger than the elastic deflection in case (a). This deflection, and also the plastic hinge rotations in the frame members, can be calculated by performing a nonlinear analysis (in this case a very simple one) until an energy balance is reached.

A relationship can be established between the frame strength and the displacement ductility ratio, μ, equal to the maximum displacement divided by the yield displacement. If the strength is $2P$ or larger, the structure remains elastic and μ is 1.0. if the strength is P, μ is infinite. For a strength aP, where a is between 1 and 2, μ is given by :

$$\mu = \frac{a}{2(a-1)} \tag{1}$$

For example, for a strength of 1.09P (a = 1.09), μ = 6. This shows that the ductility demand can be reasonable even if the strength is only slightly larger than the gravity load.

Equation (1) is for elastic-perfectly-plastic behavior. If the structure strain hardens after yield, the ductility demands are smaller. For example, Figure 3 shows the case where the yield strength of frame ABCDE is 1.0P but there is strain hardening. In this case the displacement ductility ratio is given by :

$$\mu = 1 + \sqrt{\frac{1}{h}} \tag{2}$$

where h is the strain hardening ratio. The ductility ratio is 6 when the strain hardening ratio is 0.04.

Load and
Resistance

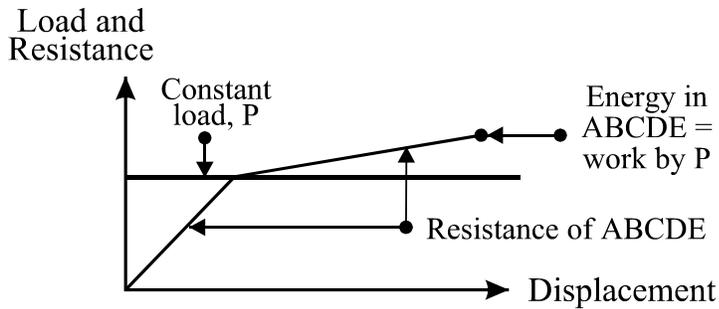Figure 3. Energy Balance for Strain Hardening Behavior

Dynamic Response

Although the actual behavior is dynamic, the analyses in the preceding section do not require calculation of the dynamic response., Figure 4 shows the type of dynamic response that would be calculated if a dynamic analysis were performed (assuming no viscous damping).
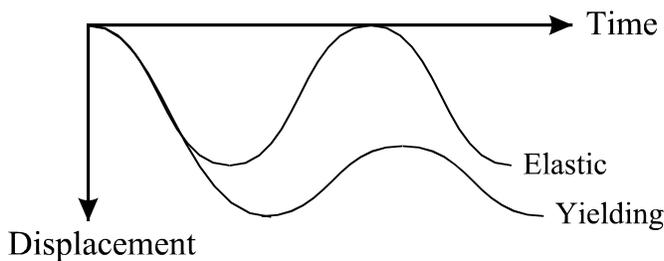
Figure 4. Dynamic Response of Simple Frame

Design Using Static Linear Analysis

The preceding sections show that if a simple structure remains essentially elastic, the deflections and member forces are essentially twice as large as would be obtained by applying the gravity loads statically. This suggests a design procedure as follows.

(1)   Build a linear analysis model.
(2)   Delete the removed element(s).
(3)   Apply double the expected gravity load, and perform a linear analysis to calculate the member strength demands.
(4)   Design the members to provide strength demand-capacity ratios no larger than 1.0. To account for effects such as increased material strength under dynamic loads, the nominal strength capacities might be multiplied by capacity (ϕ) factors that are larger than the usual values.

In this procedure, however, the goal of keeping the structure essentially elastic is a very conservative one. As shown in the preceding section, if the strength demand exceeds the strength capacity, the structure yields but does not necessarily collapse. As shown earlier, the ductility demand can be kept reasonable if the structure strength is only slightly larger than the gravity load. For example, if a ductility ratio of 6 is reasonable to prevent collapse, and if elastic-perfectly plastic behavior is assumed, the strength needs to be only 1.09 times the gravity load. In Step (4) of the above procedure, this corresponds to a demand-

capacity ratio of (2)/(1.09) = 1.8. Since most structures strain harden after yield, a demand-capacity ratio larger than 1.8 can be justified.

General Services Administration Guidelines

The General Service Administration guidelines for the design of government buildings [GSA 2000] provide fairly explicit guidance on the use of linear analysis. The goal is to prevent widespread progressive collapse, while allowing collapse in small areas. The report also allows nonlinear analysis, but does not give explicit guidelines.

The guidelines specify that one column or one section of wall is to be removed, and that gravity loads equal to 2(DL+0.25LL) are to be applied. Strength demands are calculated using static linear analysis, and the strength demand-capacity ratios, based essentially on nominal strength, must be no larger than 2.0 over most of the structure (or 1.5 for "atypical" structures). The demand-capacity ratios can be larger in the allowable collapse region, which is a limited area adjacent to the removed member.

Based on the discussions in the preceding sections, this procedure has a rational basis.

Possible Problems With Static Linear Analysis Method

The static linear analysis method has the advantage that it can be applied using any computer program for linear structural analysis. It has the disadvantage that it may not be very accurate. There are three reasons for this, as follows.

(1) *Load Sequence*. The "correct" load sequence is to apply the gravity load first, then remove a column. The linear analysis method assumes that essentially the same results will be obtained if the column is removed first and the gravity load is then applied. This is true for a simple frame, but not necessarily for a more complex case.

(2) *Dynamic Load*. The true behavior is dynamic. However, the method assumes that static analysis, with amplification of the load, is sufficiently accurate. Again, this is true for a simple frame, but not necessarily for a more complex case.

(3) *Nonlinear Behavior*. The true behavior is nonlinear. The method assumes that if the strength demand-capacity ratio in the linear analysis is kept below about 2.0, the ductility demands in the actual nonlinear structure will be reasonable, and will provide a consistent level of safety. This is not necessarily the case, even for a simple structure. As in earthquake resistant design, this is the greatest weakness of methods based on linear analysis.

These aspects are considered in the next three sections.

Load Sequence

For the simple model considered in the preceding sections, the behavior is the same whether the load sequence is (a) apply gravity, load then remove column (the "correct" sequence) or (b) remove column, then apply gravity load (a sequence which is more convenient for analysis). However, these two sequences are not necessarily the case for structures that are more complex. An example is the frame shown in Figure 5.

This is still a simple frame. However, it has more spans that the earlier frame, and the gravity load is distributed over the beams rather than concentrated at the columns. For simplicity assume that under gravity load the center column has only axial force, with a magnitude *P*. When this column is suddenly

removed, a downward concentrated load of magnitude P is suddenly applied to the frame, and the stiffness of the column suddenly becomes zero. This is not the same as removing the column from the unloaded frame then suddenly applying the distributed gravity load. The difference in response may not be important for design purposes, but it raises a concern
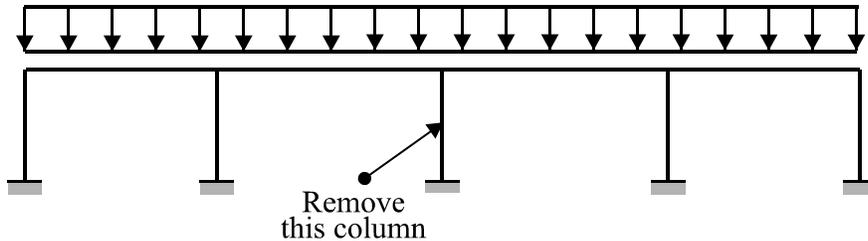

Figure 5. A More Complex Frame

Dynamic Load

When load is suddenly applied to the frame it responds dynamically. Based on energy balance considerations, the maximum dynamic displacement can be calculated by applying the load statically, and increasing it until there is an energy balance. However, this is accurate only for a simple frame with simple loading.

If the "correct" load sequence is used for the frame in Figure 5, the concentrated load should presumably be increased until an energy balance is obtained (at a concentrated load of $2P$ for an elastic structure, less for a yielding structure). This means that the distributed gravity loads remain constant, and the final value of the concentrated load is larger than P. If the alternative load sequence is used, the column is removed and the distributed gravity load is then increased until there is an energy balance (to twice the actual load for an elastic structure). For either of these static loads, it is not obvious that the resulting deflections, member forces and member deformations will be closely similar to those when a load of magnitude P is applied dynamically. Again, the difference in response may not be important for design purposes, but it raises a concern.

A potentially important point for the alternative load sequence and linear analysis is whether the distributed gravity load should be doubled in all spans of the frame or only in the spans adjacent to the removed column. Intuition suggests that it would be more accurate to apply two times the gravity load only in the bays adjacent to the removed column, and one times the gravity load in the other bays. In some respects this would tend to be less conservative (e.g., it would mean smaller axial compression loads in the remaining columns) and in other respects it would tend to be more conservative (e.g., it would mean larger bending moments in the columns, because the loads in adjacent spans are not balanced).

Nonlinear Behavior

When a structural member yields, the concern for design is not the forces in the member but its deformations. The most rational approach for design is to calculate deformation demands and capacities, and hence deformation demand-capacity ratios. This requires nonlinear analysis. We can try to achieve the same design goals using linear analysis and strength demand-capacity ratios, but this is a losing battle. Nonlinear analysis has the advantage is that it can account for two important effects, as follows (it also, of course, has several disadvantages).

Concentration of ductility demand. With linear analysis it may be possible to get reasonable estimates of the displacements of a nonlinear structure, and hence of the displacement ductility ratio (for example using the "equal displacements" assumption for earthquake response). However, if the displacement ductility ratio is μ, the maximum member deformation ductility is almost always substantially larger than μ. The reason is that yield deformation tends to be concentrated in the weaker members, rather than distributed uniformly throughout the structure. There is no unique relationship between structure displacement ductility and member deformation ductility. Methods based on linear analysis use a "one size fits all" approach to account for this effect.

Stiffening and strengthening due to sag. After a column is removed, the floor sags. As it does, tension forces can develop in the girders and floor slab, and it is possible for them to carry substantial load by "cable" and "membrane" actions. This is a geometrically nonlinear effect that can be taken into account only in a nonlinear analysis. It can lead to substantial increases in strength and stiffness, and hence substantial decreases in structure displacement and member deformation. It is, however, a double edged sword, since tension forces can develop in beam-to-column connections, and these connections may fail in tension. Analyses that do not consider cable action will not calculate these tension forces.

To obtain an indication of the magnitude of the "cable" effect consider a steel bar that is restrained horizontally and deflected vertically, as shown in Figure 6(a). A beam would have similar behavior. Figure 6(b) shows the relationship between vertical displacement and axial strain for this bar. If the yield strain for the steel is 0.002 (corresponding to a stress of 60 ksi), this strain is reached when the bar rotation is 0.063 radians. This is not an unreasonable rotation for a beam in a progressive collapse analysis. Hence, large axial forces can be developed, provided there is stiff horizontal restraint. In an actual frame the restraint would not be rigid, so Figure 6 exaggerates the effect. Nevertheless, it can be substantial.
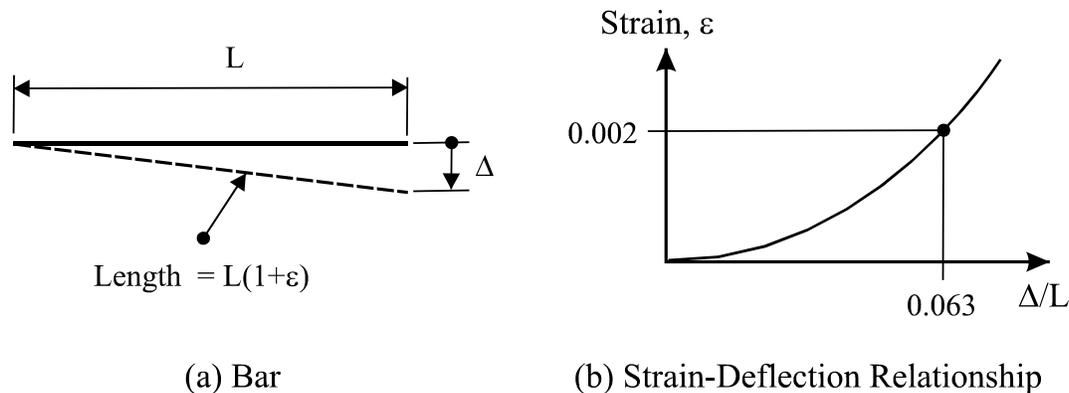

(a) Bar   (b) Strain-Deflection Relationship

Figure 6. Cable Effect in a Bar

To obtain an indication of the magnitude of the "membrane" effect in a slab, consider the flat plate structure shown in Figure 7. This consists of a uniform slab with a span-to-thickness ratio of 40 and self weight load. The slab has point supports as shown, rather than columns. These supports provide vertical support only, with no horizontal or rotational restraint. One of the supports, in the middle of one of the long edges, is missing. Note that since the supports do not provide any horizontal restraint, and hence do not provide any anchorage to resist membrane forces, anchorage must be developed within the slab itself. Also, when the missing support is at an edge, it is more difficult for the slab to develop anchorage than if this support were in the interior of the slab.

Each bay of the slab was modeled using a uniform 8 x 8 mesh of rectangular slab/shell elements (512 elements total). The analysis was run using the Perform-Collapse program described later in this paper.

The purpose of the analysis is to illustrate the effects of membrane action. This is not necessarily a practical structure or a practical finite element mesh, and this is not an example of a progressive collapse analysis.
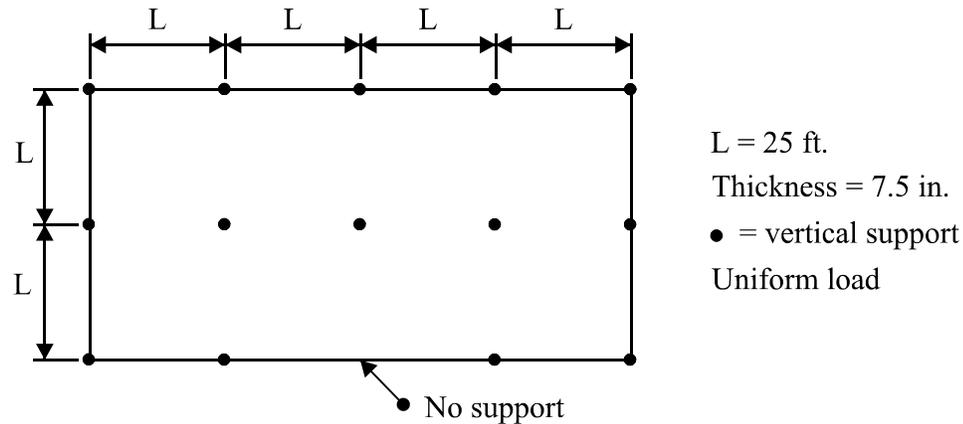

Figure 7. Slab for Study of Membrane Action (Plan View)

Four analyses were run, as follows.

(E1) Elastic slab, ignoring membrane effects (small displacements analysis). The material elastic modulus is 3000 ksi and Poisson's ratio is 0.2.
(E2) Elastic slab considering membrane effects (large displacements analysis).
(C1) Reinforced concrete slab ignoring membrane effects. The slab has 0.75% reinforcement top and bottom in both directions, with a steel yield stress of 60 ksi and elastic-perfectly-plastic behavior. The slab was modeled using a "layered" or "fiber" model of the cross section, with 5 concrete layers (with thicknesses of 1.0, 1.5, 2.5, 1.5 and 1.0 inches, respectively) plus layers for the steel reinforcement (0.5 inches cover to the center of the layer). The concrete modulus is 4000 ksi, the compression strength is 4 ksi, and the tension strength is zero.
(C2) Reinforced concrete slab including membrane effects.

For Case E1 the slab was loaded until its maximum deflection (at the location of the missing support) was 30 inches (4 times the slab thickness). Based on a material density of 150 lb/ft^3, this required a load of 14.7 times the slab self weight. The same load was then applied for Case E2. The calculated maximum deflection for Case E2 was 20.7 inches. Hence, membrane action reduced the calculated deflection by about 30%.

For Case C1 the slab was again loaded until its maximum deflection was 30 inches. This required a load of 3.3 times the slab self weight. The same load was then applied for Case C2. The calculated maximum deflection for Case C2 was 21.3 inches. Again, membrane action reduced the calculated deflection by about 30%.

These analyses show that a slab can be substantially stiffer when membrane effects are considered. It also shows, perhaps surprisingly, that there can be substantial membrane action even when the slab is not externally restrained.

It may be noted that the behavior of a reinforced concrete slab is complex. The membrane effect tends to stiffen the slab. However, yield of the reinforcement tends to make it less stiff. Also, as a slab cracks its neutral axis shifts toward the compression side, and because of this the slab extends (similar to axial growth in a reinforced concrete beam). In these analyses the slab was free to expand horizontally, so that axial growth was not restrained. For interest, Cases C1 and C2 were analyzed with supports that are rigid horizontally, so that (a) the supports provide external anchorage and (b) they restrain axial growth in the slab. Surprisingly, the calculated maximum deflection for Case C1 was reduced from 30 inches to 11.9 inches. Presumably the axial restraint suppresses cracking in the slab, and hence both stiffens and strengthens it. The calculated maximum deflection for Case C2 was 18.9 inches, somewhat smaller than the earlier 21.3 inches. For this case the slab extends as it deflects, and this allows the slab to crack.

An analysis with rigid horizontal supports is extreme, but it indicates that in-plane restraint (provided, for example, by walls) could significantly change the calculated behavior.

Design for Progressive Collapse Using Nonlinear Analysis

A linear analysis can not provide an accurate simulation of the behavior of a nonlinear structure. However, this does not mean, that linear analysis is unsuitable for design. It is worth re-emphasizing that the purpose of structural analysis is not to provide an accurate simulation of behavior, but to provide information that can be used in design. Linear analysis has been used for many years in earthquake resistant design, where it is understood that there can be substantial nonlinear behavior. Linear analysis also has a place in design against progressive collapse.

Nevertheless, earthquake engineers have long been aware that nonlinear analysis can provide better design information than linear analysis. As better nonlinear analysis tools are developed, nonlinear analysis becomes more practical, and it is being used increasingly often for earthquake resistant design. The same can be expected for design against progressive collapse.

There are three main steps in applying nonlinear analysis in design, as follows.

(1) Create an analysis model that captures the important aspects of the structural behavior.
(2) Perform a structural analysis of the model. Calculate displacement, deformation and strength demands.
(3) Calculate demand-capacity ratios, and hence evaluate the performance and make design decisions.

Some key aspects of these steps are considered in the following sections. As shown in these sections, the concepts for progressive collapse design are similar to those for earthquake resistant design, but there are substantial differences in the details.

Nonlinear Modeling : General

The most difficult step is creating a meaningful nonlinear model. As with a model for earthquake analysis, it is important to account for nonlinear material behavior, including yield, cracking, shear failure, etc., and to consider a variety of components, including beams, columns, connections, walls, etc. Linear analysis models are relatively simple because there are only a few types of linear behavior. Nonlinear models are more complex, because there are many types of nonlinear behavior with many different underlying mechanisms.

For example, in a linear model, beams and columns can both can be modeled using a generic frame element that has axial, bending and torsional stiffnesses. For nonlinear analysis, however, there can be major differences between beams and column. In an earthquake analysis (but not necessarily in a progressive collapse analysis) a beam will generally have small axial forces, and inelastic behavior can be modeled using relatively simple plastic hinges. However, a column can have large axial forces, with substantial P-M interaction. When the behavior is inelastic, it is a complex task to model P-M interaction.

Similarly, for linear analysis a reinforced concrete wall can be modeled using a generic shell element with membrane and plate bending stiffnesses. For nonlinear analysis it is necessary to consider yield of the reinforcement, cracking and crushing of the concrete, and interaction of the steel and concrete in resisting shear. Also, in a linear model it is not necessary to consider hysteresis loops, or aspects such as stiffness degradation and brittle strength loss. All of these can be important in a nonlinear model.

Nonlinear modeling is not a matter of fitting a curve to experimental results. It requires understanding the underlying mechanisms, deciding which of these mechanisms are important, and deciding how much accuracy is needed. It also requires understanding the capabilities of the nonlinear components that are available in a computer program, and choosing the components that are appropriate for the task. Building a meaningful nonlinear model is a challenging task, but not an impossible one. As computer programs get better and computers get faster, nonlinear analysis becomes increasingly feasible, not only technically but also economically.

Nonlinear Modeling : Floor Systems

For earthquake analysis the major concern is the lateral load system. For progressive collapse analysis the major concern is the gravity load system. One consequence of this is that for progressive collapse analysis, floor systems may need to be modeled in much greater detail. When a floor system is modeled for earthquake analysis, it is usually sufficient to consider inelastic behavior only in the girders, to omit the floor beams from the model, and to consider the floor slab only to the extent that it serves as a diaphragm. In a progressive collapse analysis it may be necessary to model all parts of the floor, and to consider inelastic behavior in the floor slab as well as in the girders. In the model of a building for earthquake analysis, a floor can usually be modeled using one beam element for each beam member, and it can often be assumed that the slab is rigid as a membrane. For progressive collapse analysis a detailed model along the lines shown in Figure 8 may be needed.
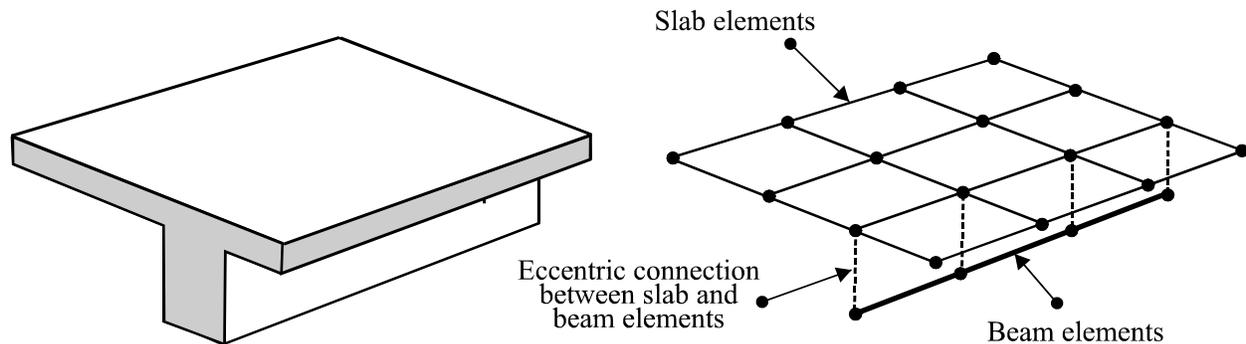

Figure 8. Floor System Model

For a slab that is composite with steel beams, it may even be necessary to model nonlinear behavior in the shear connectors between the beam and slab.

Nonlinear Modeling : Geometric Nonlinearity

Geometric nonlinearity is nonlinearity associated with change of shape of a structure. A small displacements analysis assumes that the change of shape is so small that it has a negligible effect. Geometric nonlinearity has two parts, as follows.

(1) *Equilibrium*. Equilibrium must be satisfied in the deformed position of the structure. A small displacements analysis assumes that equilibrium can be considered in the undeformed position.
(2) *Continuity*. The relationship between the deformation of an element and the displacements of the nodes is actually nonlinear (see Figure 6). A small displacements analysis assumes that the relationship is linear.

There are two types of analysis that account for geometric nonlinearity, namely P-Δ analysis and large displacements analysis. P-Δ analysis accounts for the effect of shape change on the equilibrium, but assumes linear continuity relationships. Large displacement analysis accounts for both effects.

P-Δ analysis is simpler than large displacement analysis, and can be much more efficient computationally. For earthquake analysis it is almost always sufficient to do only P-Δ analysis. However, if only P-Δ analysis is considered for a floor system in a progressive collapse analysis, the beams and slabs will not develop "cable" or "membrane" actions as they deflect, which could cause substantial inaccuracy. For progressive collapse analysis it is usually necessary to perform a large displacement analysis, where the beams and slabs develop "cable" or "membrane" actions

Nonlinear Structural Analysis : Loading

For earthquake resistant design the main design loads are gravity loads and horizontal inertia forces from ground accelerations. For progressive collapse analysis the main design loads are gravity loads and forces from removed elements. The loads for progressive collapse analysis tend to be simpler, and better defined, than for earthquake analysis.

The load sequence for earthquake analysis is gravity load followed by lateral load. The "correct" load sequence for progressive collapse analysis is gravity load on the full structure followed by suddenly applied load from the removed elements.

For linear analysis it may be sufficiently accurate to remove the elements first then apply the gravity loads. This may also be the case for nonlinear analysis, but as noted before there are potential problems with this load sequence. It adds little or no complexity to a nonlinear analysis if the loads are applied in the "correct" sequence.

Nonlinear Structural Analysis : Static vs. Dynamic

The loads for earthquake analysis and progressive collapse analysis are both dynamic, but in both cases it may be accurate enough to use static analysis.

For earthquake analysis there are two major incentives for the use of static analysis. First, not only is a dynamic analysis much more expensive computationally than a static push-over analysis, but because of the variability in ground motions it is usually necessary to run analyses for several different accelerograms. Second, with static push-over analysis the earthquake can be represented by a response spectrum, and an accelerogram is not needed. The incentives are not so strong for progressive collapse

analysis. First, a dynamic analysis is not much more expensive than a static analysis. Second, the dynamic load is well defined, so it is not necessary to run multiple dynamic analyses.

For earthquake analysis the static push-over method has some serious weaknesses, especially for taller structures, and its accuracy has been questioned. For progressive collapse analysis the energy balance method seems, from limited experience, to be fairly accurate.

For dynamic earthquake analysis, the horizontal mass dominates, and the vertical mass is usually ignored. Also, the mass can usually be lumped at a few mass points. For progressive collapse analysis the vertical mass tends to dominate,. However, the horizontal mass may still be important and it will be usual to consider both. For progressive collapse analysis it may not be accurate to lump the mass at a few mass points. A convenient method, which can be automated, is to base the mass at any node on the gravity load at that node.

Performance Evaluation

The performance evaluation methods are similar for earthquake analysis and progressive collapse analysis, based on deformation demand-capacity ratios for components that can yield, and on strength demand-capacity ratios for components that are required to remain essentially elastic. In both cases, strength should be used as the demand-capacity measure for brittle members or modes of behavior, such as columns with large axial forces or reinforced concrete members in shear.

In both cases, a major problem is choosing suitable deformation capacities for yielding components. For earthquake analysis the best source is FEMA 356 [FEMA 2000]. For progressive collapse, some guidance is provided by Table 2.1 of the GSA Guidelines [GSA 2000]. As an example, for a reinforced concrete beam the GSA Guidelines allows an end rotation of 6 degrees, or 0.1 radians. For the Collapse Prevention performance level, FEMA 356 allows an end rotation of about 0.02 radians. The allowable deformations for progressive collapse analysis are thus much larger than for earthquake analysis, as would be expected because a larger amount of damage is acceptable.

Implementation : RAM Perform-Collapse

This section addresses the "Made Easy" claim in the title.

Nonlinear analysis for progressive collapse does not require new technology, such as new finite element theories or new analysis techniques. It does, however, require effective implementation of existing technology. That is, it requires computer programs that can be used productively in a design office. Given the inherent complexity of nonlinear behavior, it may never be easy to construct a nonlinear analysis model. However, a well designed computer program can greatly simplify the task. Once the model has been built, a well designed computer program can also make the nonlinear analysis essentially automatic, so that the engineer does not need to worry about nonlinear analysis strategies, and can concentrate on the modeling and performance evaluation. There is at least one computer program that can claim to have these features.

For earthquake resistant design using nonlinear analysis, the computer program RAM Perform-3D has been developed specifically for use in design. As the developer of Perform-3D, the author is obviously biased. However, experience with real structures has shown that Perform-3D is indeed a practical tool. The program has the following key features.

(1) It has a comprehensive library of nonlinear and linear components for the modeling of beams, columns, walls, seismic isolators, viscous dampers, and other members. The nonlinear components all have essentially the same force-deformation relationships (a basic trilinear relationship with optional brittle strength loss).

(2) It incorporates a reliable and essentially automatic nonlinear analysis strategy. Static push-over and dynamic ground motion analyses can be run on the same model.

(3) It allows deformation capacities of a variety of types to be specified for inelastic components, calculates deformation demand-capacity ratios, and uses these ratios to display performance information. It also allows strength capacities to be specified, and calculates strength demand-capacity ratios.

Information on Perform-3D can be found on the RAM International web site (www.ramint.com). and a free student version of the program (maximum 100 nodes) can be downloaded.

RAM Perform-Collapse is an extension of Perform-3D, with modeling and analysis features specifically for progressive collapse analysis. Some of these features are as follows.

(1) The same library of nonlinear components as Perform-3D, with additional components to model floor systems, specifically a component for reinforced concrete slabs/shells, with a layered cross section (similar to a fiber cross section for a beam or column, but with 2D layers rather than 1D fibers).

(2) True large displacements analysis (Perform-3D has only P-Δ analysis).

(3) The ability to specify patterns of elements that are to be removed. The analysis sequence consists of applying gravity load to the complete structure, then suddenly removing an element pattern (either a single element or multiple elements). The program automatically applies the loads caused by element removal. Any number of removed element analyses can be run for a single gravity load analysis.

(4) Automatic static analysis. The analysis runs until the energy balance criterion is reached.

(5) Automatic dynamic analysis. The analysis can be set to stop shortly after the maximum displacement is reached, or it can be run for a specified time.

(6) Provision for deformation and strength capacities as in Perform-3D, with essentially the same tools for displaying performance results.

(7) The ability to read Perform-3D models. The model can then be edited to make it suitable for progressive collapse analysis.

Perform-Collapse will be available in a free student version. It will be demonstrated during the oral presentation of this paper.

Conclusion.

This paper has shown that the basic concepts for progressive collapse analysis are fairly simple. However, the details can be complex. As with design for earthquake resistance, it is possible to design for progressive collapse using linear structural analysis. However, since the behavior is nonlinear, more accurate results can be obtained using nonlinear analysis.

While nonlinear analysis is a complex task, with modern computers and computer programs it is definitely feasible, and the improved accuracy may well justify the additional effort. The computer program RAM Perform-Collapse is a practical tool intended specifically for progressive collapse analysis and performance evaluation. Nonlinear analysis may never be truly easy, but a well-designed tool such as RAM Perform can greatly simplify the task.

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