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Effective use of nonlinear analysis in bridge design

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EFFECTIVE USE OF NONLINEAR ANALYSIS

IN BRIDGE DESIGN

by

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1. ABSTRACT

Traditionally, structural analysis has been based on the assumption of linear behavior. There are good reasons for this, and for the foreseeable future the vast majority of structural analyses will continue to use linear models. However, as technology improves it becomes possible to use more sophisticated design procedures, with analyses which use nonlinear models. Such design procedures are emerging, but the technology is not well understood and the tools needed to apply the technology are not yet available. That is, at the present time we do not understand how to apply this new technology, and even if we did we would not have the tools to apply it effectively. Further, developing the understanding and the tools will be a complex and lengthy process.

This report assesses the present state of the art in the application of nonlinear structural analysis, with particular emphasis on bridge structures, and makes recommendations for future action. The report covers three broad areas, as follows.

1. Principles, with emphasis on modelling for analysis and the role of analysis in design.

2. Procedures, with emphasis on how nonlinear analysis can be incorporated into the design process.

3. Tools, with emphasis on the computer programs which are needed to support the design process.

The report concludes that for conventional strength-based design, procedures based on linear analysis have worked well. They have not worked so well, however, for seismic resistant design. With modern computer technology it is possible to use nonlinear analysis to obtain much more reliable information for use in design. However, the problems and the technology are complex and are not well understood. A concerted effort is needed to develop the tools and procedures for effective use of this technology.

1. INTRODUCTION

Traditionally, structural analysis has been based on the assumption of linear behavior. The main reasons for this are as follows.

1. It is relatively easy to model a structure for linear analysis.
2. The computations are relatively easy to perform.
3. The analysis results for different load cases can be superimposed.
4. The analysis results are sufficiently accurate for use in design.

The last reason is probably the key point. Structural analysis is not an end in itself, but merely a step in the design process. Ultimately, the analysis tools which engineers use are driven by the needs of design, and if linear analysis were not useful for design it would not be used. Hence, in order to determine the roles of both linear and nonlinear analysis it is necessary to understand the needs of design. Linear analysis is sufficiently accurate in most cases for conventional strength-based design. However, it may not be sufficiently accurate for design procedures in which deformations and damage need to be taken into account. Increasingly it is being recognized that such procedures are needed for rational seismic-resistant design, and that there is an important role to be played by nonlinear analysis.

Unfortunately, the tail may be wagging the dog. Because linear analysis is well understood and relatively easy to perform, engineers have relied almost exclusively on strength-based design procedures, with strength demands calculated by linear analysis. Even seismic-resistant design has been mostly strength-based, although it has been known for many years that this is not rational.

This does not mean that linear analysis is not useful. For the foreseeable future the vast majority of structural analyses will continue to use linear models. However, as technology improves it becomes possible to use more sophisticated design procedures, with analyses which use nonlinear models. Such design procedures are emerging, but the technology is not well understood and the tools needed to apply the technology are not yet available.

In the author's opinion, this last point is extremely important, and can not be emphasized too strongly. At the present time we do not understand how to apply this new technology, and even if we did we would not have the tools to apply it effectively. Further, developing the understanding and the tools will be a complex and lengthy process. This report does not provide answers because the answers do not yet exist. Rather, its purpose is to assess the present state of the art, and to make recommendations for future action.

The report covers three broad areas, as follows.

1. Principles, with emphasis on modelling for analysis and the role of analysis in design.

2. Procedures, with emphasis on how nonlinear analysis can be incorporated into the design process.

3. Tools, with emphasis on the computer programs which are needed to support the design process.

The report consists of four main chapters, each largely self-contained.

Chapter 2 examines the role of analysis in design. This chapter concludes that nonlinear analysis is needed in certain cases, but that many questions remain to be answered before clear procedures for using nonlinear analysis can be developed.

Chapter 3 reviews the particular problem of modelling beam and column members for nonlinear frame analysis. This chapter concludes that the available beam-column models are not adequate for the task.

Chapter 4 examines the nonlinear analysis computer programs which are available to the engineer, and identifies the features which are needed in these programs. This chapter concludes that the available programs are far from adequate.

Chapter 5 summarizes the key aspects of the overall problem.

This report concludes that seismic resistant design can be substantially improved if careful use is made of nonlinear analysis. However, considerable cost and effort will be needed to establish procedures and develop effective tools.

2. THE ROLE OF ANALYSIS IN DESIGN

2.1 OVERVIEW

This chapter begins with a review of the role of linear structural analysis in design, and points out that it has serious limitations. A number of simple examples are presented, to illustrate these limitations and to indicate when nonlinear analysis might be needed. A number of questions related to the role of nonlinear analysis are listed.

This chapter does not provide answers. Rather, its purpose is to emphasize that we first need a better understanding of the questions.

2.2 THE ROLE OF LINEAR STRUCTURAL ANALYSIS

2.2.1 Conventional Strength-Based Design

With modern computer programs linear structural analysis can be performed easily and routinely. Because of this, we tend to forget the approximations which are made, and perhaps to think that nonlinear analysis should also be routine. Before considering nonlinear analysis, it is useful to review the role of linear analysis and the assumptions on which it is based.

To limit the scope of the initial discussion, consider only frame structures, consider design for strength only (i.e., ignore serviceability and other criteria), assume that the structure is subjected only to gravity and wind loads, and ignore the dynamic effects of wind. The following is essentially the procedure.

1. Design decisions are based on comparisons of strength demand and strength capacity.

2. The comparison is usually made at the member level (other possibilities are the structure, substructure, cross section and material levels). Comparisons are also made for connections. For this report the term "member" includes connections as well as members such as beams and columns.

3. Linear structural analysis is used to calculate member strength demands (axial force, shear force, bending moment, torsional moment).

4. Code formulas are used to calculate member strength capacities. These formulas account for nonlinear behavior of the elements, and are based to a large extent on the results of experiments.

5. Code formulas are used to make the demand-capacity comparisons.

2.2.2 Approximations and Assumptions

One key aspect of this procedure is that the demands are calculated assuming linear structural behavior, whereas the capacities account for nonlinear behavior. This is not necessarily inconsistent. However, it assumes that the strength demands calculated by linear analysis are sufficiently accurate for use in design, even though the structure does not remain linear as it approaches collapse. For frame structures under gravity and wind loads, experience has shown that this assumption is generally reasonable.

A second key aspect is that for linear structural analysis, the only property of the structure which needs to be captured in the analysis model is its stiffness. Further, in order to obtain the correct member forces for static loads it is necessary to capture only the relative stiffnesses of the members, not their absolute stiffnesses (absolute stiffnesses are, however, needed to obtain accurate displacements). The definition of the analysis model is thus a reasonably easy task.

The role of structural analysis in this procedure, and the type of analysis to be performed, are thus well defined. It is important, however, to keep in mind the approximations which are being made. Some of these are as follows.

1. It is usual to assume that beam-to-column connections are either fully rigid or perfectly pinned. In reality, however, moment-resistant connections can have significant flexibility and "pinned" connections can have significant stiffness. Computer programs for frame analysis typically provide only a beam-column element and a support spring element, and hence do not allow direct modelling of semi-rigid connections.

2. Because concrete cracks at low load levels, cracking must be accounted for in modelling reinforced concrete members. Hence, analyses of reinforced concrete structures are "linearized" rather than truly "linear". This is important when absolute stiffnesses are needed, but can also be significant for relative stiffnesses, because a member in compression cracks less and hence is relatively stiffer than a member with little or no compression. For example, in a frame under gravity plus lateral load, some columns will have larger compression forces than others, and hence may be significantly stiffer and may resist more of the lateral load. This could have a significant effect on the bending moment and shear force demands.

3. For traditional allowable stress design (ASD), strength demands are calculated for service loads, and capacities are reduced by factors of safety. The implied assumption is that the structure is safe if the member capacities exceed the service load demands by appropriate safety factors. For ultimate strength design, or for load and resistance factor design, factors of safety are applied to the loads, and the capacities are realistic values (nominal values multiplied by capacity reduction factors). The strength demands are thus calculated for factored loads. There are two possible interpretations for the implied assumption, as follows.

    (a) The structure is safe if the member capacities exceed the service load demands by appropriate factors of safety, and different factors are applied for different loadings (e.g., 1.2 for dead load versus 1.6 for live load). This is essentially a refinement of the ASD assumption.
    (b) The structure is safe if the member capacities equal the demands under factored loads, when the structure is near its ultimate strength. This is quite different from the ASD assumption.

    If the relative stiffnesses of the members do not change much as the frame yields and becomes nonlinear, then the force distribution in the frame will not change much as the loads increase from service to factored values. A linear analysis can thus give sufficiently accurate strength demands for both service and factored loads. In this case it makes no difference whether the design philosophy is based on assumption (a) or assumption (b). However, if some members become nonlinear much sooner than others, the force distribution may change dramatically. The strength demands from a linear analysis may then be accurate for service loads but not for factored loads. In this case there are substantial differences between the two assumptions. In particular, if the design philosophy is based on assumption (b), it may be necessary to use nonlinear structural analysis for the demand calculations.

4. Ideally, strength demands and capacities would be compared at the structure level, in terms of load demand and load capacity. This is not generally done, (a) because it is more convenient to design on a member-by-member basis, and (b) because it requires complex nonlinear analysis to calculate load capacity at the structure level. In some cases, however, the strength of a member depends on how that member interacts with other members, and can not be calculated considering the member in isolation. An example is the buckling strength of a column, which depends on the restraint offered by adjacent members. The member capacity calculation may thus have to account for effects at the structure or substructure level. This is difficult to achieve in a code formula.

2.2.3 Extension to Seismic-Resistant Design

The preceding sections have reviewed the role of linear structural analysis for strength-based design considering only gravity and wind loading. Although the design procedure is not completely rational, it is relatively simple and experience has shown that it can lead to sound designs.

The situation is more complex, however, for seismic resistant design. There are two main reasons for this, as follows.

1. For strength-based seismic resistant design it is necessary to account for dynamic effects when calculating strength demands. These demands depend on the mode shapes and periods of the structure. The mode shapes depend on the relative stiffnesses of the members, and the periods depend on the absolute stiffnesses. Accurate modelling of stiffness is thus more important than for static loading.

2. If structures were designed to remain essentially linear in strong earthquakes, then strength-based design allowing for dynamic effects would be rational. However, most structures are designed to yield substantially, and hence simple strength-based design is not rational. Certainly, strength considerations are important. However, it is not sufficient to consider only strength, and it may not be accurate enough to use only linear analysis..

The second point is particularly important. In the next section, simple examples are used to illustrate this point.

2.3 SOME EXAMPLES

2.3.1 Example 1: Strength Design with Implicit Control of Ductility Demand

A strong earthquake will load a typical structure beyond yield. If the amount of damage is excessive, the structure may have insufficient strength to resist gravity loads, and may collapse. One measure of damage is ductility demand, expressed as a ratio of maximum deformation divided by yield deformation. The deformation measure might be total displacement, relative displacement (e.g., story-to-story drift), overall member deformation (e.g., relative rotation between member ends), local deformation (e.g., curvature), or even strain.

One design approach is to use strength as the limit state, but to account for the influence of ductility demand on strength capacity. The simplest procedure is essentially as follows for a reinforced concrete frame.

1. Calculate strength demands for gravity loads, using linear analysis.

2. Calculate strength demands for seismic loads, using linear dynamic analysis and realistically strong ground motions.

3. Combine the gravity demands with some fraction (1/N) of the seismic demands.

4. Choose axial force and bending moment capacities based on these demands.

5. Choose shear demands based on moment capacities, and hence design the shear reinforcement. This is to ensure that the yield mechanism will be primarily flexural with no significant ductility demand in shear, since shear failure tends to be brittle. Experience has shown that it is unwise to design for only the shear demands calculated in Step 3.

Implicit in this design procedure is that if the value of N in Step 3 is not too large, then the ductility demands will be modest, any strength loss under seismic loading will be small, and the strength capacity will be sufficient to prevent collapse. There is no explicit attempt to calculate the ductility demands or the amount of strength loss.

It is sometimes assumed that the ductility demand, expressed as a ratio, is approximately equal to N. However, this depends on the ductility measure which is chosen. If displacement is used as the measure, the assumption may be reasonable, but for measures such as curvature the ductility ratio is likely to be much larger than N, and can vary substantially from structure to structure. This aspect is considered in Example 5.

2.3.2 Example 2: Strength Design with Explicit Control of Ductility Demand

An improvement of the Example 1 procedure is being developed within Caltrans for the design of single bridge columns. The procedure is essentially as follows.

1. Calculate strength demands on the column for gravity loads, from a linear analysis of the bridge structure.

2. Calculate strength demands for seismic loads, using linear analysis of the bridge structure and realistically strong ground motions. Also calculate horizontal displacement demand at the top of the column.

3. Combine the gravity strength demands with some fraction (1/N) of the seismic demands.

4. Design the column for axial force and bending moment capacities based on these demands.

5. Choose strength demands for the foundation based on the designed column capacities, and hence design the foundation. This is to ensure that there will be no significant ductility demands within the foundation.

6. Using a <u>nonlinear</u> analysis model of the column plus the foundation, impose a displacement equal to that calculated in Step 2, and calculate the curvature ductility demand in the column. Design confining reinforcement to provide this ductility with no significant strength loss.

7. Choose shear demands based on the moment capacity, and hence design the shear reinforcement.

This is a more rational procedure than that outlined in Example 1, because it explicitly calculates a ductility demand. The procedure is, however, based on the assumption that the displacements for the yielding structure are essentially equal to those calculated in the linear analysis. This is not necessarily correct. Also, it may not be obvious how to select the element properties for the linear analysis. These aspects are illustrated by the next two examples.

2.3.3 Example 3: Choosing Properties for Linearized Analysis

In Example 2, the displacements were calculated using linear analysis. Consider the calculation of longitudinal displacements for a bridge with one column and two abutments, as indicated in Figure 2.1a. A simple analysis model is indicated in Figure 2.1b. This model assumes that the bridge superstructure is rigid longitudinally, and that the column and abutments are modelled using spring elements. Figure 2.1b also indicates the type of nonlinear behavior which might be exhibited by the column and abutments.

For a linear (or, more correctly, linearized) analysis it is necessary to choose linear stiffnesses for the column and abutment springs. One choice might be the initial stiffnesses, which would mean zero stiffness for the abutments. Intuitively this does not seem to be correct. Another choice might be secant stiffnesses, based on some assumed or calculated maximum displacement (perhaps obtained by iteration). Intuitively this is better, but it still fails to capture important aspects of the structural behavior.

There are thus serious difficulties in defining a linear analysis model for this problem. The obvious solution is to define a nonlinear model, which more accurately captures the true behavior. With such a model it is no longer possible to obtain the dynamic response using a response spectrum analysis. Instead, a step-by-step dynamic analysis is needed, and it may be necessary to perform analyses for several ground motions, to allow for the fact that the computed response can be sensitive to changes in the ground motion. However, the computed displacement is more reliable as a basis for design.

Note that the analysis model indicated in Figure 2.1 is a very simple one, and a more complex model might be needed.

2.3.4 Example 4: A Danger in the Equal Displacements Assumption

The procedure in Example 2 is based on the assumption that the displacements of a nonlinear structure are similar to those of the same linear structure. This is based on observations for single degree of freedom models, and is generally regarded as being reasonable for long period structures (the "displacement preserved" part of the response spectrum). A modified assumption, based on equal energy considerations, can be used for shorter periods (the "velocity preserved" part of the spectrum).

While the equal displacements assumption is not especially accurate, it is useful for design because it simplifies a complex problem. Example 3 showed, however, that even if the assumption were accurate it can not necessarily be used to avoid the need for nonlinear analysis. This example illustrates a further weakness.

Figure 2.2 shows an analysis model consisting of a rigid block supported by two springs. For ground motion in the transverse direction as shown, the model has two degrees of freedom, namely translation and (torsional) rotation. The model might represent a free-standing section of bridge with two columns, or a building with two lateral load-resisting elements.

First consider the case where the springs are linear. The model has two modes, translational and torsional. There can be complications if the periods of these modes are nearly equal, but assume that this is not the case. Further, assume for simplicity that the centers of mass and stiffness coincide, so that there is zero response in the torsional mode. Next consider the case with nonlinear springs. Assume that the springs are elastic-perfectly-plastic, as shown, and that one spring is slightly weaker than the other. At some point in the dynamic response this weaker spring will yield, while the stronger spring is still elastic. Hence, the instantaneous center of stiffness suddenly shifts from the center of the structure to the location of the stronger spring. The block thus tends to pivot about this location, and it significant torsional deformation may develop. The result can be that the displacement demand on the weaker spring is much larger than that computed for the linear model. That is, the equal displacements assumption may no longer hold.

This is an over-simplified example. However, it illustrates an important point, namely that multi degree of freedom structures are much more complex than those with only a single degree of freedom. There are serious potential dangers in applying single degree of freedom results to multi degree of freedom cases.

2.3.5 Example 5: Concentration of Ductility Demand

Consider the simple frame in Figure 2.3a, and make the following assumptions.

1. The only significant loads are lateral seismic loads.

2. The equal displacements assumption applies.

3. Before yield the frame displacement originates 50% in the columns and 50% in the girder (Figure 2.3b).

4. The columns are stronger than the girder, and the girder is essentially elastic-perfectly-plastic.

5. The measure of girder deformation is rotation of the girder ends.

6. The displacement ductility ratio (maximum displacement divided by yield displacement) is 4 (Figure 2.3b).

Figure 2.3b shows the load-displacement relationship. Before yield the displacement originates 50-50 in the columns and girder. After yield all additional displacement originates in the girder. The deformation ductility ratio for the girder is thus 3.5/0.5 = 7, or nearly twice the displacement ductility ratio. If the columns contributed, say, 75% of the flexibility before yield, and the girder only 25%, the deformation ductility ratio for the girder would be 3.25/0.25 = 13. Also, if the deformation measure were curvature (at the girder ends) rather than end rotation, the ductility ratios would be still larger.

This example indicates that there is no fixed relationship between displacement and deformation ductilities.

2.3.6 Summary

These examples illustrate some of the ways in which linear analysis might be used as a basis for design, and some of the problems which might arise. They suggest that, at least in some cases, design can be performed more rationally if use is made of nonlinear analysis.

Linear analysis is simple, and despite its limitations will continue to be used for the majority of design applications. There are many cases, however, where nonlinear analysis can and should be used. Unfortunately, nonlinear analysis is much more complex, and procedures for using it effectively are not yet in place. Some of the problems result from a mind-set which engineers have developed over the years. Some erroneous beliefs which can result from this mind-set are as follows.

1. Analysis produces "analysis results" which can be used for design.

   This is true for simple strength-based design, where the analysis results are simply the strength demands. It is not so simple for seismic resistant design. The concept of what constitutes "analysis results" needs to be examined in detail, and designers and analysts must work more closely together.

2. Models and computer programs for nonlinear analysis are similar to those for linear analysis.

   For a linear model of a frame structure, only a simple beam-column element is needed, and only the element stiffnesses need to be specified. A nonlinear model requires much more information, a much better understanding of structural behavior, and a computer program with much more complex elements.

3. Powerful computer programs for linear analysis are available at modest cost. Therefore, programs for nonlinear analysis should also be available, at similar cost.

   This is not the case because programs for nonlinear analysis are vastly more complex. Much of our mind-set was established years ago, when an engineer needed not much more than a slide rule, a sheet of paper and some books to do an effective job. These days we need much more powerful, and more expensive, tools. We must be prepared to invest in the development of these tools. It is worth noting that the lack of adequate knowledge and tools in past years has left us with many substandard structures which are extremely expensive to upgrade.

4. Nonlinear analysis is at best approximate, and hence linear analysis is more accurate.

   Linear analysis is accurate only if the behavior of the structure is essentially linear. If it is not, the "accuracy" of a linear analysis is an illusion.

2.4. WHAT IS THE ROLE OF NONLINEAR STRUCTURAL ANALYSIS?

2.4.1 General

At the present time the role of nonlinear analysis in design is far from clear. Research has been performed on nonlinear analysis for many years, the theory is fairly well established, and there have been many useful applications. However, the technology is relatively new for civil engineering applications, and the problem is extremely complex. Many questions need to be asked and answered before the proper role of nonlinear analysis can be established. That is, we need to clarify the questions before we can provide the answers.

This section contains a list of many questions. Clear answers to some of these questions already exist, in design codes, design manuals and elsewhere. For many of the questions, however, clear answers have not yet been formulated. This is particularly true for seismic resistant design.

2.4.2 Summary of the Basic Process

Before listing the questions it is useful to summarize the essential steps in the design process, as follows.

1. Define, in qualitative terms, the design limit states which are to be considered. Two simple examples are (a) collapse of a beam (a strength limit state), and (b) excessive deflection of a beam (a serviceability limit state).

2. Quantify each limit state in terms of one or more demand-capacity measures, for which demand side and capacity side values can be calculated. Examples are (a) beam bending moment under factored loads (a measure of strength demand), and (b) beam midspan deflection under service loads (a measure of deflection demand).

3. Specify how the demand value of each measure is to be calculated. This will usually involve structural analysis. The analysis may be linear or nonlinear, and may or may not require a computer program.

4. Specify how the capacity value of each measure is to be calculated. This will usually involve code equations, but could involve structural analysis.

5. Perform the calculations, and compare the demand and capacity values. If the demand is less that the capacity, the limit state is satisfied.

The process is, of course, not quite this simple. However, these steps serve as a basis for discussion.

2.4.3 Questions Related to Demand-Capacity Measures

The definition of limit states and demand-capacity measures is of fundamental importance in the design process. The purpose of these questions is to clarify what quantities are to be calculated, and why.

1. What are the limit states?

2. At what levels are the demand-capacity comparisons to be made (structure, member, cross section, etc.)?

3. What are the demand-capacity measures for each limit state?

4. Do any demands depend on capacities? This can affect the way the demands are defined and calculated. Some examples are as follows.
   (a) The shear strength demand in a column can depend on its moment strength capacity.
   (b) The column axial strength demand in a frame can depend on the maximum girder end shears, and hence on the girder moment capacities.
   (c) For earthquake loading, curvature ductility demand can depend on moment strength capacity.

5. Do any capacities depend on demands? This can affect the way the capacities are defined and calculated. Some examples are as follows.
   (a) Moment and shear strength capacities can both depend on curvature ductility demand.
   (b) Curvature ductile capacity can depend on axial strength demand (larger axial force, lower ductility).
   (c) For lateral-torsional buckling, moment strength capacity can depend on the shape of the bending moment diagram (i.e., the bending moment demand).

6. Exactly how are the demand values to be computed? For example, is it sufficiently accurate to use linear analysis, or must nonlinearities be taken into account?

7. Exactly how are the capacity values to be computed? These values will typically be obtained from code equations, but this is not essential. For example, the strength capacity of a reinforced concrete column section might be defined in terms of a P-M-M interaction surface, to be calculated by computer using structural analysis techniques.

8. Exactly how are the demand-capacity comparisons to be made?

2.4.4 Questions Related to the Purpose of the Structural Analysis

Structural analysis, whether linear or nonlinear, will always be an important part of design. In traditional strength-based design, structural analysis is used to calculate strength demands. However, analysis can be used for many purposes, and it is important to clarify the purpose.

Is the purpose of the analysis:

1. Calculation of strength demands for demand-capacity comparisons?

2. Calculation of deflections for comparison with allowable deflections (i.e., calculation of deflection demands)?

3. Calculation of more complex demand measures, such as ductility or damage, for demand-capacity comparisons?

4. Calculation of structural response for preliminary evaluation of structural behavior, without formal demand-capacity comparisons?

5. Calculation of capacities for an existing structure? This is almost always more difficult than the calculation of demands.

6. Validation and/or calibration of an analysis model? This aspect is addressed in Chapter 4.

2.4.5 Questions Related to the Behavior of the Structure

Structural analysis is performed not on the real structure but on an analysis model. The analysis results will be meaningful only if the model captures the behavior of the structure with sufficient accuracy for the problem at hand. It is important, therefore, to understand the behavior of the structure, especially its nonlinear behavior. Some questions which might be asked are as follows.

1. Which members are likely to become nonlinear, and under what loadings?

2. Which nonlinear members have significant effects on the behavior of the structure? If a particular nonlinearity has only a minor effect, it can be modelled approximately or ignored.

3. Is it necessary to perform dynamic analyses, or are static analyses sufficient?

4. How sensitive is the nonlinear behavior to the loading and to the material properties? For example:
   (a) If the relative strengths of the beams and columns in a frame are changed slightly, does the behavior change from weak-beam-strong-column to vice versa? Ideally, both the real structure and the analysis model should be insensitive to such factors, since they are rarely known precisely.
   (b) A key aspect of seismic resistant design is that the computed response can be sensitive to changes in the ground motion. If this is the case, a simple analysis model may give results which are just as reliable (or unreliable) as a more elaborate model, and the extra cost of an elaborate model may not be justified.

2.4.6 Questions Related to the Behavior of Structural Members

The behavior of a real structure depends greatly on the behavior of its members. Similarly, the behavior of an analysis model depends greatly on the behavior of the structural elements which make up the model. For a model to be accurate, the elements must capture the essential aspects of member behavior. In creating a model, therefore, it is important that the analyst have a thorough understanding of member behavior. For traditional linear analysis, it has not been necessary for analysts to have much knowledge of behavior. Also, it has not been necessary for designers to have much knowledge of how to create an analysis model. The analyst and the designer have been able to operate largely independently of each other, the designer giving structural dimensions to the analyst, and the analyst returning member strength demands to the designer. This is not the case when nonlinear analysis is to be used. The analyst must now have a stronger understanding of behavior, and the designer must have a clearer understanding of the limitations of structural analysis.

Some questions which might be asked about member behavior are as follows.

1. What are the causes of the nonlinear behavior? If the causes are clearly understood, it is easier to create a rational analysis model.

2. What measures are used to characterize the nonlinear behavior? For example, for a cable restrainer a simple force-extension relationship can be sufficient. For a bridge column, however, it may be necessary to consider more complex behavior, relating bending moment, axial force and shear force to curvature, axial deformation and shear deformation.

3. What interaction effects are present? Examples are (a) P-M interaction, and (b) the effect of ductility demand on strength. If there are strong interactions, it can be much more difficult to understand the behavior, to model it for analysis, and to choose demand-capacity measures.

4. To what extent can the nonlinear behavior be controlled by design? If the nonlinear behavior can be deliberately controlled, it can be more easily understood and more accurately modelled. For example, if a reinforced concrete column is designed so that shear failure can not occur (by basing shear demands on flexural capacities), it is not necessary to model inelastic shear behavior (which is extremely difficult). Because the behavior can be controlled in new designs, modelling and analysis can be simpler for new designs than for existing structures.

5. What experimental results are available for calibration of the analysis model? Has the model been adequately calibrated? Frequently, nonlinear analysis models are not fully rational, in the sense that they do not directly capture the underlying causes of nonlinear behavior. For example, the concept of a plastic hinge is not strictly rational for concrete structures. The more indirect the model, the greater the need for calibration against experimental results or field observations. Note that a linear model is the most indirect of all, and needs the most calibration.

2.4.7 Questions Related to a Specific Analysis Model

An analysis model must capture the behavior of the real structure, with sufficient accuracy for the practical problem at hand. It must also be feasible to perform the analysis, and the results must be useful. Some questions which might be asked are as follows.

1. Is the purpose to support a new design or to evaluate an existing structure? In a new design, the designer can control the behavior of the structure, and can build factors of safety into the design. Also, the purpose of the analysis is usually to calculate demands. Sufficiently accurate results can usually be obtained with fairly simple analysis models (of which a linear model is the simplest). In the evaluation of an existing structure, however, the behavior depends on the structure properties (which will be known only approximately), and the likely purpose of the analysis is to calculate strength capacity or damage demand. The analysis model must thus capture all modes of behavior which may affect strength or damage. This is a far more difficult task.

2. What results (demand and/or capacity measures) are to be calculated, and how are they to be used?

3. What aspects of behavior must be captured in the model. Some different situations are as follows.
   (a) Only stiffness is important (e.g., calculate strength demands using a linear or linearized model).
   (b) Only strength is important (e.g., calculate collapse load and collapse mechanism using a rigid-plastic model).
   (c) Both strength and stiffness are important, but not cyclic loading (e.g. calculate collapse load, sequence of plastic hinge formation and plastic hinge rotations using a simple elastic-plastic model).
   (d) Strength, stiffness and degradation of strength and stiffness under cyclic loading are all important (e.g., calculate ductility demands for a reinforced concrete structure under seismic loads).

4. Can the model be analyzed at reasonable cost with existing analysis tools? An important point is that we may need to raise our threshold for what is a reasonable analysis cost. We tend to underestimate the importance of analysis, and to expect that it can be performed easily and cheaply.

5. Has the model been calibrated, against experimental results or against a more accurate model? The more empirical the model, the greater the need for calibration.

6. Should any parameters (e.g., material properties, earthquake motions) be varied to account for uncertainty?

2.5. SUMMARY

This chapter has sought to show that there is a role for nonlinear analysis in design, but that there are many questions which remain to be answered before this role can be clearly identified.

Because of the apparent complexity of nonlinear analysis, and the apparent simplicity of linear analysis, it might be concluded that the use of nonlinear analysis is not warranted. It is important to remember that the purpose of structural analysis is to provide information for use in design. If sufficiently accurate information can be obtained from linear analysis, then linear analysis should be used. Nonlinear analysis should be used only when the added complexity is justified. The problem with linear analysis is that in many cases it can not give sufficiently accurate information.

It is also important to remember that the conventional strength-based design procedure may not lead to a sound design. Our understanding of seismic resistant design, and our design procedures, have improved greatly in recent years. However, there are large numbers of older structures that were designed using linear analysis and a simple strength-based philosophy, and are now known to have inadequate seismic resistance. Rational seismic resistant design requires consideration of deformation as well as strength, and has a much greater need for nonlinear analysis.

3. NONLINEAR MODELLING OF BEAM AND COLUMN MEMBERS

3.1 OVERVIEW

In the preceding chapter, general analysis and design issues were addressed, and it was shown that there is a role for nonlinear analysis. In order to use nonlinear analysis, however, it is necessary to construct a nonlinear model. For a linear analysis only the stiffness of the structure needs to be modelled. For a nonlinear analysis, properties such as strength, stiffness degradation, and hysteretic energy absorption may also need to be modelled.

Frame structures, and particularly reinforced concrete frames, pose particular modelling problems. This chapter reviews the modelling of frame structures, where the members resist axial force, bending, shear and possibly torsion. The emphasis is on reinforced concrete members, and on how to capture, in an analysis model, the important aspects of beam and column behavior.

The purpose of nonlinear analysis is first reviewed briefly, to keep modelling needs in perspective. The basic requirements for a reinforced concrete frame model are identified, particularly the need to model inelastic behavior both within a cross section and along the member length. Beam-column models based on lumped and distributed plasticity assumptions, and on force (flexibility) and displacement (stiffness) formulations, are summarized. Cross section models based on an analogy with plasticity theory are examined, and weaknesses in this approach are noted. Models based on "fiber" concepts are described, and it is argued that these models provide the best compromise between accuracy and computational effort. It is noted that although fiber models are superior to plastic hinge models, the basic fiber concept accounts only for axial and flexural effects, ignoring inelastic behavior in shear. An extension of the fiber concept to account for shear cracking and the yield of shear reinforcement is outlined. The importance of deformation in the beam-to-column connection region is also noted, and inelastic connection models are briefly discussed.

Most of the material in this chapter is taken from the following paper: Nonlinear Modelling and Analysis of Reinforced Concrete Frame Structures, by G. Powell, F.Filippou, K. Martini and S. Campbell, Proceedings of

Workshop on Recent Developments and Future Trends of Computational Mechanics in Structural Engineering, Beijing, China, September 1991, Elsevier Science Publications, 1992.

3.2 THE NEED FOR NONLINEAR ANALYSIS

As was noted in the preceding chapter, until relatively recently the design of reinforced concrete structures was based on strength considerations, assuming that the load magnitudes were well defined. Relatively little attention was given to behavior under extreme loading conditions such as earthquakes. One consequence of this is that many older reinforced concrete structures are brittle, and may not have sufficient ductility to survive strong earthquakes without serious damage. In future years it will be necessary to retrofit many structures to increase their level of safety. A major problem facing bridge engineers is that it is extremely difficult to compute the behavior of reinforced concrete structures under extreme loads, and hence it is extremely difficult to estimate the amount of damage which any particular structure might experience in a future earthquake. There is a need for better analysis methods in both new design and retrofit design, but the need for advanced nonlinear models is particularly strong in retrofit design.

It is important to remember that structural analysis can be used for two distinctly different reasons, as follows.

(1) Demand analysis: compute the <u>demands</u> on structures and structural members. As discussed in the preceding chapter, strength demands can often be calculated with sufficient accuracy using linear analysis. However, the calculation of deformation demands, such as curvature demands on a bridge column, is more likely to require nonlinear analysis.

(2) Capacity analysis: compute the <u>capacities</u> of structures and structural members. For example, given an existing structure designed to an old, and possibly inadequate, design code, estimate its true strength. For this type of analysis it is essential to consider nonlinear behavior. Also, if the analysis is to be a rational one, the analysis model must capture all those aspects of behavior which can influence the strength. For example, if shear failure is a possible cause of collapse, the analysis

model should ideally capture nonlinear shear effects. Capacity analysis is thus more difficult than demand analysis.

Capacity analysis of single members is often used as an aid in the development of design formulas for member strength. However, such analysis plays a secondary role, and most of the data is obtained from destructive tests of actual members, not from analysis. In retrofit design the engineer does not have the luxury of performing a destructive test, and must rely heavily on analysis.

A reinforced concrete frame structure consists essentially of beam, column and wall members, subjected to axial forces, bending moments, shear forces, and possibly torsional moments. There may also be significant deformation in the beam-to-column connection regions. Three dimensional behavior, of both the structure and the members, may need to be taken into account. This chapter is concerned mainly with the modelling of reinforced concrete beam and column members for capacity analysis of such frames. Similar principles apply for demand analysis, but the models can usually be simpler.

3.3 BASIC BEAM-COLUMN ELEMENT MODELS

Beams, columns and slender walls can be modelled using beam-column elements in which the effects of axial force, bending moment, shear force and possibly torsion are considered. Consider first the basic problem of modelling axial force and bending moment effects.

Figure 3.1 illustrates the effects of axial force and bending moment in a simple yielding member. For uniform bending the plastic zones extend along the full length and over part of the depth. For nonuniform bending the plastic zones extend partially along the length, but are still symmetrical. For combined axial force and nonuniform bending the plastic zones become unsymmetrical. Beam-column element models must capture these effects. For reinforced concrete they must also account for the fact that concrete is not a simple plastic material, but one which cracks and degrades.

Figure 3.2 shows alternative models for beam-column elements. Figure 3.2a shows a typical "lumped plasticity" model, in which the inelastic deformation is lumped in zero length plastic hinges at the element ends. Strain hardening effects can be modelled by specifying hardening properties for the hinges, or alternatively by adding an elastic element in parallel with the inelastic one. Figure 3.2b shows a typical "distributed plasticity" element, in which the inelastic behavior is monitored at several discrete sections along the element length. Assumptions are then made about the variation of inelastic behavior at all other sections (e.g., linear variation of flexibility between monitored sections). Figure 3.2c shows a substructured element, divided into a number of subelements. The element as a whole is of distributed plasticity type, although the subelements could be of lumped plasticity type. Fundamentally, there is no difference between a model which uses a single substructured element and one which uses several non-substructured elements. However, it is easier to prepare computer program input data for a substructured element, since the user needs to specify only one element rather than several. There can also be computational advantages with substructured elements.

In general, distributed plasticity models are more accurate than lumped plasticity models. The reason is that the distribution of plasticity along the element length varies with the loading on the element, as indicated in Figure 3.1, and it is not generally possible to determine lumped hinge properties which are accurate for all loadings. For example, if hinge properties are assigned to give accurate results for uniform bending, they will generally not be accurate for nonuniform bending. Elements with lumped hinges can still be useful, but their limitations must be recognized.

The two most important steps in nonlinear analysis are (a) given the current state of a nonlinear element, compute its tangent stiffness matrix (linearization), and (b) given the current state and a displacement increment at the end nodes, determine the new state (state determination). For the present purpose it is sufficient to focus on the tangent stiffness computation.

For a lumped plasticity model the hinge is typically assumed to be rigid-plastic-strain-hardening, and the main problem is to define its tangent (hardening) stiffness in the plastic range. The element tangent stiffness can then be obtained using conventional analysis techniques. For a distributed plasticity model the main problem is to define the tangent stiffness or flexibility of a monitored section. The element tangent stiffness can then be obtained by interpolating the stiffness or flexibility at intermediate sections, and integrating over the element length. It may be noted that for the element in Figure 3.2a, which models an entire member, it will generally not be accurate to assume a shape function and use a displacement formulation, because the shape function changes as the element yields. For example, the commonly used cubic displacement assumption (linear curvature) may be accurate before yield but is not accurate when plastic zones form at the element ends. A force (flexibility) formulation is thus indicated. For the subelements in Figure 3.2c, however, a displacement formulation may be suitable, since the subelements are short.

If the inelastic behavior of a hinge or monitored section were affected only by bending moment, then the constitutive relationship would be uniaxial (moment-rotation for a hinge, moment-curvature for a section). These relationships tend to be complex for reinforced concrete members, because of steel yielding and concrete cracking and strength loss. It is possible, however, to develop equations or rules to describe such uniaxial behavior. This approach is <u>empirical</u> at the hinge or section level, since the equations or rules are chosen to match experimentally observed behavior. If, however, there is P-M interaction, then the constitutive relationship becomes multi-axial instead of uniaxial. The hinge or section stiffness is thus defined not by a single number but by a matrix (2x2 for two dimensional P-M interaction, 3x3 for three dimensional P-M-M interaction). The problem of defining empirical rules for the <u>diagonal</u> terms of the constitutive matrix is the same as for the uniaxial case. However, it is virtually impossible to define empirical rules for the <u>off-diagonal</u> terms, because of the complexity of the underlying causes of the interaction. It is necessary, therefore, to base the interaction modelling on a <u>rational</u> theory of some kind rather than on empirical rules. A theory which has been commonly used is based on an analogy with the yield of plastic materials under multiaxial stress, as considered in the next section.

3.4 INTERACTION BASED ON PLASTIC THEORY ANALOGY

Figure 3.3a shows a von Mises yield surface for a plastic material (e.g., steel) in two dimensional principal stress space. If the well-known normality rule is assumed for plastic flow, it is easy to develop an expression for the tangent constitutive matrix after yield. Essentially, the von Mises yield criterion plus the normal flow assumption define a rational theory for the interaction between stresses. By analogy, the same approach can be used for stress resultants, as indicated in Figure 3.3b for two dimensional P-M space. Unfortunately there are problems with this analogy.

Figure 3.4a shows the yield surfaces for a linearly hardening plastic material, with an initial yield surface, a subsequent yield surface, and stress-strain curves for uniaxial stress along the x and y directions. With the assumptions which are usually made, the initial and subsequent yield surfaces have identical shapes, and the stress-strain curves are the same. Figure 3.4b shows the analogous case in P-M space for a reinforced concrete section. Now, the initial and subsequent yield surfaces do not have the same shape. Also, the action-deformation curves along the P and M directions are not the same. Further, for a complete theory it is necessary to provide empirical hardening rules, which define how the yield surfaces move, change size and possibly change shape as the material yields. Rules, such as kinematic hardening, can be defined fairly easily for yield of plastic materials, but not for P-M interaction.

In spite of these limitations, the plasticity analogy can capture some interaction effects, and it can serve as a basis for useful models. However, it must be recognized that such models are not based on a "correct" theory, and hence are empirical rather than rational. To develop a rational model it is necessary to model the underlying causes of inelastic behavior and interaction, by moving one level lower before introducing empirical rules. This can be done using "fiber" concepts.

3.5 FIBER REPRESENTATION OF A SECTION

Figure 3.5 shows a simplified reinforced concrete column section and a fiber representation of the section. In the fiber representation the section is divided into a number of steel and concrete longitudinal fibers, and each is assigned a uniaxial stress-strain relationship. These relationships are empirical, based on the observed behavior of steel and concrete. Note, in particular, that different stress-strain curves are assumed for confined and unconfined concrete. In order to assign the confined curve it is necessary to account for the amount of confinement and its effect on the stress-strain curve. A truly rational model would use a multiaxial model for concrete and consider the confining steel directly. However, such a model would be much more complex. For most practical purposes the effects of confinement can be accounted for empirically.

A major advantage of the fiber representation is that it allows P-M interaction to be considered without postulating multiaxial yield surfaces and flow rules.

Figure 3.6a shows that a two-fiber model with simple yielding fibers provides a P-M interaction surface that is a rough approximation of that for a steel I section. A three-fiber model (not shown) gives a hexagonal yield surface which is a closer approximation. Figure 3.6b shows that a model with two yielding steel fibers combined with two yielding and cracking concrete fibers gives a P-M interaction surface of reinforced concrete type (the four-fiber interaction surface is actually somewhat more complex than that shown). As the number of fibers is increased, the interaction surfaces more closely approximate those of actual cross sections. Strain hardening behavior is also captured, without the need for multiaxial hardening rules.

The fiber procedure can also be applied to hinges. In order to obtain the force-moment-strain-curvature behavior for a section, the fibers are assigned stress-strain curves. The force-moment-extension-rotation behavior of a lumped hinge can be obtained by assigning force-extension relationships to zero-length fibers. This is more empirical, since the hinge seeks to capture inelastic behavior along the element length as well as over the cross section, but the model can be useful.

Models of both hinges and cross sections have been developed with only small numbers of fibers (e.g., the "5 spring" hinge model developed by Saiidi). Since the number of fibers is small, the fiber stress-strain curves in such models are not those of the basic steel and concrete materials. Instead, they are empirically determined curves which combine steel and concrete properties into one fiber, and which also account for the small number of fibers. Although these curves must be determined empirically, elements based on small numbers of fibers are computationally much less costly than more rational elements with large numbers of fibers.

3.6 EXTENSION OF FIBER CONCEPT TO SHEAR DEFORMATION

The controlling mode of inelastic deformation for a frame member is ideally flexural, since this provides the greatest ductility. Some frame members may, however, be controlled by shear, particularly in structures designed to old design codes. Inelastic shear deformation is less ductile than flexural deformation, and it could be argued that shear failure will immediately be followed by structural collapse. However, a structure may be able to redistribute load in such a way that local shear failure does not lead to overall collapse. Also, inelastic shear deformation is not entirely brittle, and there may be sufficient ductility to avoid major damage. It may be important, therefore, to model inelastic shear deformation, and the interaction of shear forces with axial forces and bending moments. It is possible to extend the fiber modelling concept to include shear deformation. An outline of the procedure is as follows.

Figure 3.7a shows a rectangular column section, and Figure 3.7b shows its fiber representation. Figure 3.7b also shows a side view of a beam-column "slice" of infinitesimal length dx. This slice is a cuboid of dimension b by d by dx. Its behavior under axial force and bending moment is defined by the properties assigned to the longitudinal steel and concrete fibers. In addition, the slice may deform in shear. Shear resistance is provided by the concrete and by transverse shear reinforcement. In Figure 3.7b the shear reinforcement is shown as a transverse fiber. The fiber area is the shear reinforcement area per unit length multiplied by dx. Figure 3.7c indicates the way in which shear deformation is usually modelled in beam elements: the shear force produces shear deformation in the slice, based on the shear stiffness of the material. Since concrete has a large shear stiffness, the amount of shear deformation of this type is small and can usually be ignored. Figure 3.7d shows that shear deformation can also be caused by diagonal cracking. In this figure, the cracks are shown as opening with zero sliding parallel to the cracks, corresponding to perfect aggregate interlock. In fact there will be sliding as well as crack opening, but the essential behavior is similar to that shown. In particular, the cracking produces not only shear deformation but also longitudinal and transverse extension. There is thus interaction between shear cracking and axial effects (axial force and bending moment). Shear cracking also deforms the transverse reinforcement in tension, so that the reinforcement resists shear. By adding cracking degrees of freedom to the slice, it is possible to account for shear cracking, to include the shear reinforcement fiber in the model, and to account for interaction among shear force, axial force and bending moment. An important possible application is to model shear deformations in the plastic hinge zone of a bridge column. This is an important problem for new design as well as for retrofit design.

The theory to implement the above model has been partially developed and tested. A proposal to complete the development has been approved by Caltrans, but not yet funded. It appears to be possible to extend the model to include torsional shear cracking in three dimensional members, by allowing a spiral pattern of cracks.

3.7 CONNECTION DEFORMATION

In frame analysis it is usual to assume that the beam-to-column and column-to-foundation connections are rigid, in the sense that the members remain at right angles to each other as the frame deforms. It is well known, however, that there can be significant deformation in these connections. In steel frames this deformation is caused by high shear stresses in the "panel zone" of the connection. In concrete frames it is caused by compressive deformation, cracking and bond slip in the joint region. In some cases the connection may be the weakest part of the frame, and large deformations can be present. In order to develop a rational model for inelastic frame analysis it may be necessary to model these deformations.

Connection models which are similar in concept to a plastic hinge are the simplest. In these models the beam elements to the left and right of the connection are rigidly connected to each other, and similarly the column elements above and below the connection. However, the beams and columns are not joined rigidly but are connected by a deformable connection element. Any moments which are transferred from the beams to the columns thus pass through this element, causing it to deform. Inelastic connection behavior is modelled by assigning inelastic properties to the connection element. This type of element can be useful, but since its inelastic properties must be assigned empirically, it has the same weaknesses as other empirical elements. Reinforced concrete connections are complex, and it is desirable to capture the underlying behavior in the analysis model.

The main causes of deformation in a reinforced concrete connection are indicated in Figure 3.8. These causes are (a) yield of the main reinforcement, (b) bond slip, and (c) diagonal cracking. Reinforcement yield and bond slip can lead to significant crack opening in the beams at the column faces, and under cyclic loading cracks may open across the full beam depths. A connection element which directly models these effects has been partially developed. It is a complex substructured element which combines subelements for main reinforcement, column ties, concrete in the panel zone, bond links, and gaps at the column faces.

3.8 CONCLUSION

In order to provide realistic assessments of the safety of existing reinforced concrete structures, it is necessary to perform realistic capacity analyses. This requires more sophisticated inelastic modelling than is necessary for demand analyses. Unless the analysis model captures the important aspects of behavior, the analysis results may not be meaningful.

This chapter has reviewed a number of concepts for modelling beams, columns and connections in reinforced concrete frame structures. In order to capture the important aspects of behavior it may be necessary to use sophisticated, rational models for both members and connections, since simple models based on empirical concepts are suspect unless used very carefully. However, even with today's high powered computers, the computational cost of using fully rational models is too high for the analysis of most structures. Semi-empirical fiber models with small numbers of fibers seem to offer the best compromise for the immediate future. A "calibration" procedure, which permits empirical models to be used, is described in the next chapter.

The number of published papers and reports on frame element modelling is huge. Identifying the few most important ones is a tricky task, and this has not attempted for this report. Notable institutions at which work has been performed in the United States include the University of Illinois, Georgia Tech, SUNY Buffalo, the University of Nevada at Reno, and the University of California at Berkeley. Much excellent work has also been performed in Japan and in Europe.

It may finally be noted that an alternative approach to modelling reinforced concrete is to use solid finite elements for the concrete and embedded bar elements for the reinforcement, possibly with added bond link elements. Much excellent work has been done in this area, and the approach could be suitable for modelling single elements. However, it will be a long time before computers are fast enough to use this type of modelling for complete frames. The approach in this chapter has been on elements which are specifically intended for frame analysis, and which could be practical in the near future.

4. ANALYSIS TOOLS, AND THEIR INTEGRATION INTO THE DESIGN PROCESS

4.1 OVERVIEW

Because structural engineering decisions have major impacts on construction costs and safety, it can be argued that the best available analysis tools should be used in structural design. However, although it is clear that better tools are needed, it is not clear what forms these tools should take, how they should be applied, or how they should be developed. This chapter presents the author's opinions on the capabilities of existing analysis tools, and on the added capabilities which are needed.

The goals of the chapter are as follows.

1. Emphasize that the purposes of structural analysis need to be clarified.

2. Identify specific tasks to be performed.

3. Identify the tools which are needed for these tasks.

4. Indicate how these tools can be integrated into the overall design process.

4.2 PURPOSE OF ANALYSIS

Structural analysis is used for many purposes. The simplest is the calculation of strength demands. A number of others are as follows.

1. Calculate displacement demands for comparison with displacement capacities. Displacement capacities may be governed by minimum stiffness requirements, clearances between adjacent structures, etc.

2. Calculate complex demand measures, such as ductility or damage, for demand-capacity comparisons. This is required when more than just strength and stiffness are considered in the design process.

3. Determine structural response for evaluation of structural behavior, without formal demand-capacity comparisons. This may be required

during conceptual design, to ensure that the basic design concept is sound.

4. Calculate capacities for an existing structure. This is needed for safety evaluation of existing structures and for retrofit design. Capacity calculation tends to be much more difficult than demand calculation.

5. Validate and/or calibrate a modelling theory or a specific analysis model. This is an especially important issue for nonlinear analysis.

The type of analysis model to be used, and the means of performing the analysis, are strongly influenced by the purpose. For example, a simple model may be sufficient for preliminary evaluation of a structural concept, but not for safety evaluation of a critical structure.

The simplest model and method of analysis that will do the job are always the best choices. In some cases a back-of-the-envelope analysis will be sufficient, whereas in others a detailed nonlinear analysis may be called for. Ideally, the engineer will have available a full range of analysis tools, and the skills to choose the most appropriate tool. With existing theoretical knowledge and computer hardware it is technically possible to provide the engineer with these tools. However, not only are the existing tools far from adequate, but the procedures for applying them are not yet in place.

In the following sections the capabilities required of computer programs for practical nonlinear analysis are first considered. Tools to help integrate nonlinear analysis into the overall modelling-analysis-interpretation process are then described.

4.3 THE STATE OF THE ART

4.3.1 Existing Programs

There is a need for nonlinear structural analysis programs in the fields of nuclear, aerospace and offshore engineering, and a number of general purpose computer programs have been developed to satisfy these needs. These programs include, in alphabetical order, ABAQUS, ADINA, ANSYS, CAL-ANSR, FACTS, NASTRAN and SEASTAR. These programs are all difficult to use, and they generally lack the features needed to make them useful for bridge analysis. In particular, they generally do not include elements which are suitable for modelling components such as reinforced concrete columns and expansion joints. The NEABS program is specialized for bridge analysis, but is based on rather old technology and, like the general purpose programs, is not easy to use. The DRAIN-2D program is simple but has very limited features. DRAIN-2DX and DRAIN-3DX are more modern and more powerful than DRAIN-2D yet still relatively simple, but at present they do not have the elements needed for modelling bridge structures. Some specialized programs, such as the IDARC program for building analysis, have sophisticated elements for reinforced concrete members but are very narrow in scope.

A wish list of desirable features for a nonlinear analysis program for bridges is likely to include the following.

1. Simple input with graphic feedback.

2. Elements specially suited for bridge structures.

3. Loadings of a variety of types, including out-of-phase seismic motions at supports.

4. Reliable, automated nonlinear solution strategies.

5. Results of direct use in design.

6. Computationally efficient.

7. Easy to maintain and to modify.

8. Well documented with clear examples.

It is fair to say that the presently available programs fall far short of what is needed, in terms of modelling features, ease of use, and suitability for use in design. The main reason is that the technology is very complex.

4.3.2 Difficulties in Program Use and Development

Computer programs for nonlinear analysis are much more difficult to develop and to use than programs for linear analysis. Some particular aspects are as follows.

1. Given a structure and a computer program for linear structural analysis, different engineers are likely to obtain linear response results which are similar enough for engineering purposes. Given a structure and a computer program for nonlinear analysis, different engineers are likely to obtain nonlinear response results which are substantially different. The reason is that a nonlinear model requires much more input information than a linear model, and there is greater room for different engineers to make different modelling assumptions. This particularly true when empirical element models, rather than rational models, are used. Modelling rules are needed to guide the engineer, but such rules are generally not available.

2. In a nonlinear analysis program the input data required to describe the model and to control the analysis is complex, and preparing this data requires specialized skills. This may not be a serious problem for applications such as nuclear power structures and automobile crash simulations, where analysis is a dominant part of the overall process, and where analysts with special skills can be employed. For civil engineering design, however, analysis is a smaller part of the process, and it is desirable for design engineers to stay close to the modelling and analysis process. Design engineers generally do not have, and can not be expected to have, the specialized skills needed to use a complex general purpose program.

3. There are different reasons for performing a nonlinear analysis, each with specialized needs, and it is difficult to satisfy all of these needs in a single computer program. On the one hand, to avoid duplicating effort it is desirable to have one general-purpose program which can perform all of the required analysis tasks. On the other hand, special-purpose programs are easier to use and can be more efficient. If a well-defined specialized need exists, a special-purpose program can be justified. However, programs for nonlinear analysis are complex and expensive to develop, and require highly skilled programmers. A

generally better approach is to add special purpose features to a general purpose program. This requires, however, a general purpose program which can be easily customized.

4. As more is learned about nonlinear analysis, the programming technology is improving. Older programs tend to be less well organized, and to be costly to modify and maintain. They also tend to have limited or specialized capabilities. New programs can be more advanced technically, with easier maintenance and broader capabilities, but are costly to develop. It can be difficult to decide when to retire an old program and invest in a new one.

4.3.3 Needed Improvements

In the following sections the following aspects are considered in more detail.

1. Defining a nonlinear analysis model.

2. Preparing input data for the analysis program.

3. Customizing a general purpose analysis program to produce special purpose results output.

4. Processing time-history results.

In each case, a need is identified for tools which can help the engineer, and the features required in these tools are identified.

4.4 DEFINITION OF ANALYSIS MODEL

4.4.1 General Approach

The most critical task for nonlinear modelling is to define the properties of the elements which make up the model. Other tasks are to define the boundary conditions, the loadings and the load sequences, and to prepare the input data for the analysis program.

In simple cases (e.g., if the only nonlinearity is due to gap closure) it can be a simple task to choose the nonlinear elements and to define their properties. In more complex cases (e.g., if the nonlinearity is due to cracking and crushing of concrete members) this task is more difficult. The task will always be easier if the element model is a rational one, because the properties will then be in terms of the actual member geometry and material properties. It will always be more difficult if the element model is empirical.

With today's computers, nonlinear analysis using rational but complex element models of the types described in the preceding chapter is possible, but tends to be too expensive computationally for most structures. The cost can be reduced by using simpler and more empirical models. However, unless the correct empirical properties are specified for such models, the results of the analysis may not be meaningful. In academic studies, agreement with experimental results is often reported using simple models. However, such studies typically benefit from hindsight, and the models have often been adjusted, by trial and error, to obtain the agreement. For practical applications the engineer rarely has experimental data to support this "calibration" of empirical models.

One solution to this problem is to use rational, but complex, models to calibrate simpler, but empirical, models. The steps are essentially as follows.

1. Using available experimental results, calibrate the rational model. If the model is truly rational this should be relatively easy.

2. Set up two analysis models of a single element, using (a) the rational model and (b) the empirical model with a best estimate of the empirical properties.

3. Subject both analysis models to the same loadings, in the form of imposed forces and/or displacements. The load magnitudes should be those expected to act on the element in the complete structure.

4. Compare the results. If they agree within engineering accuracy, the empirical properties assigned in Step 2 are "correct". If not, revise the properties and repeat. That is, calibrate the empirical model against the rational model.

5. Repeat the process for a representative selection of elements.

6. Analyze the complete structure using the calibrated empirical models for the elements.

7. From the analysis results, confirm that the loadings on the elements are similar to those used in Step 3, and hence that the calibration is correct.

With this approach, the overall computational cost can be minimized, and the engineer can have confidence in the analysis results. In the future, as computers get faster, it will be possible to use rational models directly in the analysis of the complete structure.

There is a need for a computer program (a "calibration tool") to assist the engineer with this task. Such a program could be used (a) to calibrate an element model against experimental results, and (b) to calibrate an empirical element model against a rational one. The following sections describe the tasks to be performed and the features needed in a calibration tool.

4.4.2 Calibration Against Experiment

For calibration against experimental results the procedure is as follows.

1. The element to be calibrated must correspond to a member for which experimental results are available. Also, the experimental results must be for loadings which are similar in type and magnitude to those to which the member will be subjected in an actual structure. The engineer must choose nonlinear properties for the element.

2. Estimate a set of nonlinear properties for the element, based on prior experience, available guidelines, etc.

3. Analyze the element for the loadings (forces and/or displacements) used in the experiment, and compare the computed behavior with that observed in the experiment.

4. Based on this comparison, revise the nonlinear element properties, and repeat the process until an acceptably close comparison is obtained.

In general, experimental results will not be available for test specimens with the same dimensions and material properties as the members of the actual structure. Hence, this process will usually be used to provide modelling guidelines which are applicable to classes of elements, rather than modelling information on specific elements. The important point is that the process provides the engineer with a rational way of choosing element properties.

The process is essentially one of system identification, where nonlinear properties which give a best fit are to be found. The process can be informal and approximate, using trial and error. Alternatively, it might be possible to apply formal system identification techniques.

4.4.3 Calibration Against a Rational Element

For calibration of a simple but empirical element against a more complex but rational one, the procedure is as follows. The assumption is that a structure model using complex rational elements (e.g., fiber models) is too expensive to analyze, and that a model using simpler empirical elements (e.g., plastic hinge models) must be used.

1. The rational element is first calibrated against available experiments, to ensure that it gives accurate results and to assign values to the empirical parts of the element.

2. Estimate the loadings (forces and/or displacements) to which the member is likely to be subjected in the complete structure, and analyze the rational element for these loadings.

3. Estimate a set of nonlinear properties for the empirical element, based on prior experience, etc.

4. Analyze the empirical element for the estimated loadings, and compare its behavior with that of the rational element.

5. Based on this comparison, revise the empirical element properties, and repeat the process until an acceptably close comparison is obtained.

This process can be used to provide modelling guidelines which are applicable to classes of elements, or alternatively to provide modelling information on specific elements. As before, it might be possible to apply formal system identification techniques.

4.4.4 Calibration Against Subassembly Experiments

Experiments are often conducted on subassemblies rather than individual members, for example a beam-column-connection subassembly. The steps in the calibration process are the same as those in the preceding section, except that a subassembly (i.e., a small structure) must be modelled and analyzed, rather than a single element.

4.4.5 Required Features for Calibration Tool

A computer program which assists the engineer in calibrating a nonlinear model is a necessary tool for nonlinear analysis. The program must have the following features.

1. It must perform static analyses of nonlinear models of single elements and subassemblies.

2. It must be possible to apply specified displacements and/or specified forces to the model.

3. It must have plotting features which allow the response of the model to be compared with experimental data, and with results obtained from analyses of the same assembly modelled with different element types.

Such a computer program is similar in many respects to a general purpose analysis program. It does not need to consider large structures or dynamic loads, but it must have results plotting features which are flexible and easy to use.

4.5 DATA PREPARATION FOR ANALYSIS PROGRAM

After the analysis model has been defined, it must be described to the analysis program, usually by means of an input data file. With most general purpose analysis programs this is a complex task which requires a lot of training.

One solution to this problem is to develop a special purpose analysis program, with specialized input. This can be the most effective solution provided the special purpose is well defined and the analysis is to be performed sufficiently frequently to justify the development effort. The danger is that initial versions of special purpose programs frequently do not have all of the needed features, and they must be extended. The engineers who develop special purpose programs are often not skilled programmers, with the result that the programs can be difficult to modify.

A second solution is to use a general purpose analysis program, but to develop a special purpose preprocessor which creates input data files for the general purpose program. This will usually involve much less programming effort than the development of a complete special purpose program. However, it is also necessary to develop a special purpose postprocessor, to take the general purpose output and organize it for the specialized problem. This aspect is addressed in the next section.

A third solution is to use a general purpose analysis program without a preprocessor, but to prepare input data "templates" to guide the user for special problem types. A template might, for example, be prepared for a two-column bridge bent. The program user would fill in the parts of the input file which define the geometry of the bent, certain member properties, and the loading values, but the template would already contain the parts of the file which define the nonlinear solution strategy. Here again, however, it is necessary to provide special purpose postprocessing, as considered in the next section.

4.6 SPECIALIZED RESULTS OUTPUT

4.6.1 Form of Analysis Results

The results which can be saved during analysis for use in the interpretation phase are as follows.

1. Current state data. This data provides detailed "snapshots" of the structure state at specific times during the response. It includes node displacements and detailed information on the element states. The amount of data required for each saved state can be large, but the number of saved states is likely to be small.

2. Time-history and/or load-history data for node displacements. This can be used for plotting displacements against time or load, and for

drawing deflected shapes. For a 3D model the amount of data is essentially 6 displacements per node for each time or load step. For example, for a structure with 1000 nodes and 1000 time steps, there are essentially 6 x 1000 x 1000 = 6 million words of data, or 24 megabytes of disk storage (assuming single precision words).

3. Time-history and/or load-history data for element responses. This can be used for plotting element response quantities against time or load, plotting hysteresis loops, estimating damage, etc. The amount of data to be saved for each element can be large, especially for complex elements. Hence, the total amount of data can be immense for a large structure. For example, for a structure with 2000 elements (about 1000 nodes, as in the preceding example), an average of 30 items of response data per element (not a large amount), and 1000 time steps, the disk storage requirement is 240 megabytes (again assuming single precision words). A method for reducing the volume of this data is described later.

4. Envelope data (maximum and minimum values) for element responses, such as maximum element forces and deformations. This data needs to be saved only occasionally, for example at the end of each loading. The disk storage requirement is relatively small.

4.6.2 Need for Specialized Output

A particular problem for the practical application of nonlinear analysis is that different applications require different analysis results, which must be processed in different ways. For strength-based design, the required results are envelope values of member forces and moments, and it is relatively easy for a computer program developer to output these values in a convenient form. However, if ductility is to be considered, the output required by the designer is more complex, and it is more difficult for the developer of a general purpose analysis program to anticipate the designer's needs.

The most common purpose of an analysis is to calculate demands (not necessarily only strength demands) for use in design. For this type of analysis, the envelope data will ideally include all of the demand values which the designer needs. However, because it is difficult for a program designer to anticipate the designers's needs, there is likely to be a need for special purpose output and processing of the analysis results. One solution is to develop special purpose analysis programs, with specialized output. A second is to develop special purpose versions of general purpose programs. Both of these solutions, however, require highly skilled programmers, and both lead to proliferation of computer programs and duplication of effort.

A third solution is to use a general purpose program, but to allow a program user to customize its output, by modifying certain of the output routines without changing the body of the program. The body of a nonlinear analysis program is bound to be complex, and it should be changed as little and as infrequently as possible. A general purpose program can be set up, however, to allow the results output routines to be customized relatively easily, by a programmer who is familiar with the specialized application but not with the details of the general purpose program.

This approach has the following advantages.

1. A single analysis program can serve many applications, reducing duplication of effort.

2. The developers of the analysis program do not have worry about providing output which satisfies all needs, and can concentrate on refining the body of the program.

3. Engineers familiar with particular applications can write relatively simple routines to output specialized results and process them in specialized ways. These routines can be modified as needs change.

The type of results most likely to be affected are those of envelope type, since designs are usually governed by maximum effects. Some applications, however, may require specialized processing of time-history output. This poses additional problems, which are considered in the next section.

4.7 PROCESSING OF TIME HISTORY RESULTS

4.7.1 General Approach

Ideally the demand measures needed for design are known before the analysis is carried out, and the envelope output (which may be customized) includes the required values. In such cases it is not necessary to save time-history results, and the volume of output is relatively small. In some cases, however, the demand measures may not be known, so that it may be necessary to save time-history results and process them to obtain the required demand values or other data.

As already noted, for a large structure the amount of element time-history data to be saved can be immense, straining the capacity of even very large disks. Also, the specific element response data items to be saved must be specified before the analysis is carried out. Unless all response items are saved, some required results may be missing from the time-history.

A procedure which can be used to solve this problem is as follows. Assume that time history data from a dynamic analysis is to be processed to obtain detailed damage information.

1. During the analysis, save time-histories of node displacements, but not of element responses. The amount of data to be saved is thus greatly reduced.

2. Identify the elements which are likely to have the most damage. This can be done by examining the envelope values of element forces and deformations, which are output separately during the analysis.

3. Choose one of these elements. Extract, from the histories of node displacements, a history of the displacements at the nodes to which the element connects.

4. Set up an analysis model containing only this single element, and analyze it for these node displacements. During the analysis, calculate and output damage measures and other pertinent results.

5. Repeat for other potentially critical elements.

In Step 4 this process repeats calculations already performed during the main analysis (the state determination calculations). However, there can be a cost saving because histories of element response no longer need to be saved, and because the single element calculations can be performed on low cost microcomputers. Also, decisions do not need to be made on which element responses to save during the main analysis. The time history processor re-creates all of the detailed time history information on the element state, and can process this information in ways which may not have been foreseen during the main analysis.

A time history processor can also be used to check the calibration of empirical elements. In the calibration phase it is necessary to estimate the loadings (forces and/or displacements) to which the elements are likely to be subjected. Since the actual loadings on the elements, as computed in the analysis phase, may be different from those estimated, it may be necessary to check that the calibration is correct, using the computed loadings. This can be done by using a time history processor to plot and compare results of element behavior (e.g., hysteresis loops).

4.7.2 Required Features for Time History Processor

A computer program which assists the engineer in processing time-history results must have the following features.

1. It must perform analyses of single elements subjected to histories of forces and/or displacements.

2. It must have features for computing demands of a variety of types, including ductility and damage.

3. It must allow the responses of empirical and rational models to be compared, to help check element calibration.

As with the calibration tool, such a computer program is similar in many respects to a general purpose analysis program. It does not need to consider large structures or dynamic loads, but it must have the capability to calculate demand measures of a variety of types.

4.8 SUMMARY

Deformation-based design using nonlinear analysis is much more complex than strength-based design using linear analysis. In order to apply the technology effectively and efficiently, the engineer needs a variety of tools. These include the following.

1. A computer program to help calibrate nonlinear models.

2. A better general purpose analysis program, which is simpler and more reliable than existing programs. The program should allow easy customization for output of specialized results.

3. Better nonlinear elements for modelling bridge components.

4. Tools such as input preprocessors and input data templates to simplify the task of describing the model to the analysis program.

5. Special purpose programs for results processing and interpretation. One such program is a time-history processor.

5. SUMMARY AND CONCLUSION

5.1 OVERVIEW

The use of nonlinear analysis in design is a complex problem, which at the present time is not well understood. This report has examined several aspects of the overall problem. It has argued that nonlinear analysis is needed for rational seismic resistant design, but that a great deal of additional work is needed before it can be applied routinely and effectively.

There are five main areas where improvements in the technology are needed. These are:

(1) Theory. Engineers need better training in structural behavior, modelling for analysis, and design theory.

(2) General Purpose Software. Better general purpose computer programs are needed to perform the analyses.

(3) Special Purpose Software. Software specifically tailored for bridge engineering is needed, as special purpose stand-alone programs and/or as additions to general purpose programs.

(4) Policies. Structural analysis is not an end in itself, but merely an aid to design. Policies establishing when to use nonlinear analysis are needed.

(5) Guidelines. Performing a nonlinear analysis is not a simple task. The procedures to be followed in creating a nonlinear model, performing analyses, and interpreting results need to be spelled out.

These aspects are summarized in this chapter.

This chapter also describes related work being carried out by the author's group. Most of the work performed under the contract has been conceptual, as presented in this report. However, some work under the project has been performed on the development of nonlinear elements for bridge structures.

5.2 SPECIFIC ASPECTS

5.2.1 Theory

Nonlinear structural analysis makes use of the same basic theories and computational techniques as linear analysis. However, nonlinear analysis is much more complex, and there is a great deal of material which is not a included in linear analysis theory. Few engineers have been trained in nonlinear modelling and analysis, and there are currently no text books which deal with the subject from an engineering viewpoint (as distinct from a continuum mechanics viewpoint).

The author has taught a graduate course on nonlinear structures for several years. This is a relatively practical course, emphasizing structural engineering topics rather than finite element theory. However, two Caltrans engineers took the course in spring 1991, and it became clear that much more practical content was needed. The course was made more practical in spring 1992, but further improvements are needed and are being planned. The author also presented a more focused short course to a group of Caltrans engineers in spring 1992, with the intent of exploring practical nonlinear modelling issues. This course was useful in that it helped to clarify some major issues, but it did not provide definitive answers. The main reason is that the problems are extremely complex and the technology is very immature. Much more thought and discussion are needed to identify the important theoretical points and practical techniques.

Changes in design education are also needed. Most engineers receive education only in strength-based design, but for seismic resistant design it is not sufficient to consider only strength, and deformations must also be taken into account. Additional work needs to be done on developing deformation-based design concepts, and on incorporating these concepts into educational programs.

5.2.2 General Purpose Software

A general purpose computer program for nonlinear analysis is an essential tool. There are several candidate programs, but in the author's opinion they all fall well short of what is required. With the possible exception of NEABS, they are all much too cumbersome to use, and they lack the features needed for effective modelling of bridge structures. NEABS is clearly better suited for bridge analysis, but in the author's opinion it is based on outdated analysis theory and software technology.

A particular concern is that whatever general purpose program is used, special purpose nonlinear elements and output features will almost certainly need to be added. The program must be designed to make these tasks as easy as possible.

Because of unhappy past experiences, organizations such as Caltrans are understandably reluctant to spend money on computer program development. The cost and time always exceeds projections, and the final products never meet expectations. However, computer programs are essential tools for an engineer, with immense potential for increased productivity and improved engineering, especially with today's hardware and software technologies. If commercially available computer programs can not do the job effectively, and in the case of nonlinear analysis they can not, research and development on software may be justified.

One option which Caltrans might consider is the development of a bridge analysis program within the DRAIN series. In current work on the DRAIN series, the author's group is trying to provide (successfully, we believe) the needed compromise between simplicity and generality. The series includes DRAIN-2DX for general 2D structures, DRAIN-3DX for general 3D structures, and DRAIN-BUILDING for 3D building structures. DRAIN-BUILDING incorporates the analysis features of DRAIN-3DX, and uses the same element library, but takes advantage of some special features of building structures to simplify the input and organize the program data. In particular, a building is modelled as a number of floors connected by "interfloors" consisting of columns, walls, and braces. Floors are first defined essentially as plane structures, and are then positioned in space and connected by interfloors. If several floors or interfloors have the same geometry, only one of each type needs to be defined. A similar concept can be applied to bridge structures, with support structures connected by spans. One advantage is that nonlinearity will usually be present only in the support structures (bents and expansion joints), and decks can be modelled using linear elements. This can be used to speed up the computation.

5.2.3 Special Purpose Software

Since it is unlikely that a general purpose computer program will provide all of the features needed for bridge analysis, special purpose programs will almost certainly be needed. These may be stand-alone programs, or may add special purpose features to a general purpose program.

A special purpose program will almost always be preferable to a general purpose program. However, the problem to be solved must be sufficiently narrow, important and frequent to justify the development effort. If the problem involves nonlinear analysis, it may be better to add special purpose features to a general purpose analysis program.

A particular concern for bridge analysis is the need to investigate nonlinear effects in reinforced concrete bridge columns, including axial force, bending moment, shear deformation, bond slip, and the interactions among them. A special purpose program for column investigation may be justified. An alternative, which in the author's opinion would generally be more effective, is to add special purpose features to a general purpose program. These features would include special purpose input and results processing, but would avoid duplication of effort by using the modelling and analysis capabilities of the general purpose program. Of course, this requires that special purpose input and results processing be added to the general purpose program, and most such programs have not been developed with this in mind. This aspect is being considered in the development of the DRAIN series of programs.

Also, if a general purpose program is to be used for bridge columns, it must have a suitable element, or elements, in its library. For bridge column investigation, a "fiber" element of the type used in the RCCOLA and UNCOLA special purpose programs is needed. Such elements are becoming available. In particular, two 3D beam-column elements, one based on Saiidi's lumped plasticity "five-spring" model and the other based on a distributed plasticity fiber model, have been developed for DRAIN-3D, and extensions to account for shear cracking and connection bond slip have been partially developed. Caltrans funding is anticipated to incorporate these concepts into a comprehensive fiber beam-column element for the DRAIN and CAL-ANSR programs.

A 3D expansion joint element has also been developed under this project. It has been implemented in DRAIN-3D, and is in the process of being implemented in CAL-ANSR.

5.2.4 Policies

There are times when a nonlinear analysis is needed and times when it is not. The issue of whether or not to use nonlinear analysis is mainly a policy issue, based on engineering need. Technical and cost limitations must be considered, such as whether or not the needed software is available, and whether or not the modelling and computational costs are reasonable. However, need should be the driving force leading to improved technology.

For bridge engineering, it does not yet appear to have been firmly established as to when, where and why nonlinear structural analysis is needed. In order to establish firm policies a lot more work is required, involving the best brains from both the academic and practical sides of the profession.

5.2.5 Guidelines

Given a bridge structure and a computer program for linear structural analysis, different engineers are likely to obtain analysis results which are similar enough for engineering purposes. Given a bridge structure and a computer program for nonlinear analysis, different engineers are likely to obtain substantially different results. The main reason is that it is more difficult to set up a model for nonlinear analysis, and there is greater room for different engineers to make different modelling assumptions. If meaningful results are to be obtained, modelling guidelines must be established.

The problem is particularly important for reinforced concrete structures because its nonlinear behavior is so complex. Until recently there has been relatively little demand for nonlinear analysis of civil engineering structures, with the result that the topic has tended to be academic rather than practical. Also, because of limitations in computer hardware, the emphasis in modelling has tended to be on what is feasible computationally rather than on what is actually needed to obtain practical numbers. As a result, the theoretical foundations of nonlinear analysis are in place, but the available element models and computer programs are far from adequate for the task.

Finally, guidelines tend to be general and abstract. To make the process understandable, well documented examples are needed to lead an engineer through the process and explain the steps.

5.3 CONCLUSION

For conventional strength-based design, procedures based on linear analysis have worked well. They have not worked so well, however, for seismic resistant design. With modern computer technology it is possible to use nonlinear analysis to obtain much more reliable information for use in design. However, the problems and the technology are complex and are not well understood. A concerted effort is needed to develop the tools and procedures for effective use of this technology.

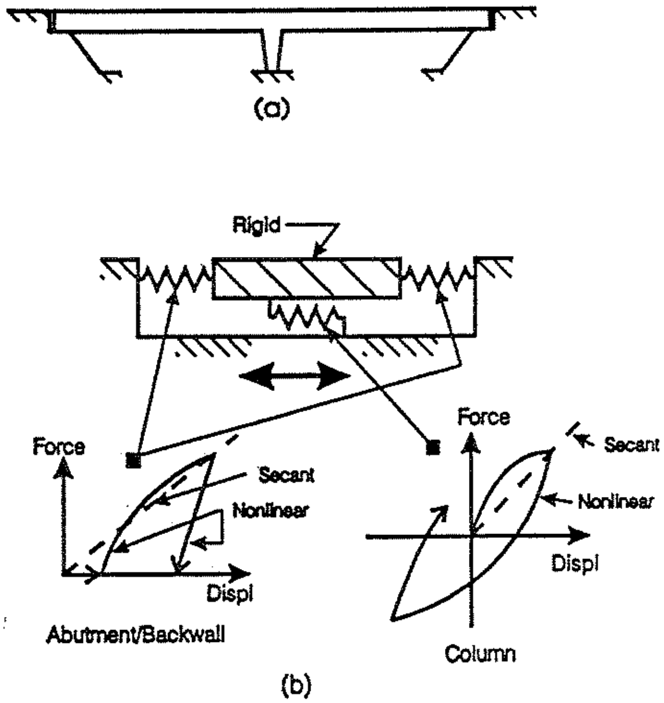


FIGURE 2.1 BRIDGE WITH LONGITUDINAL SHAKING

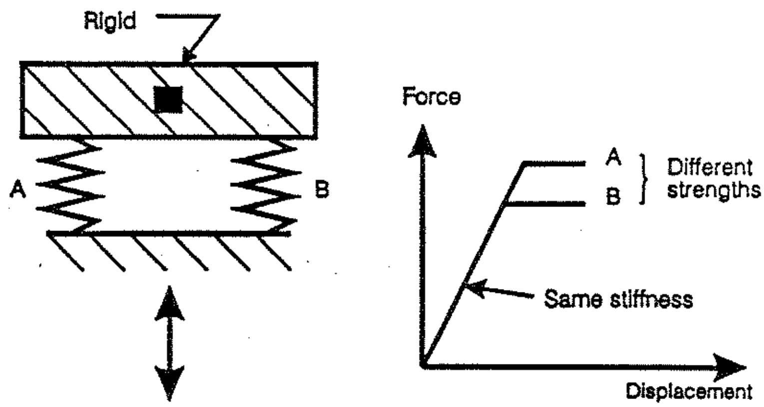


FIGURE 2.2 SIMPLE MODEL WITH TRANSLATION AND TORSIONAL ROTATION

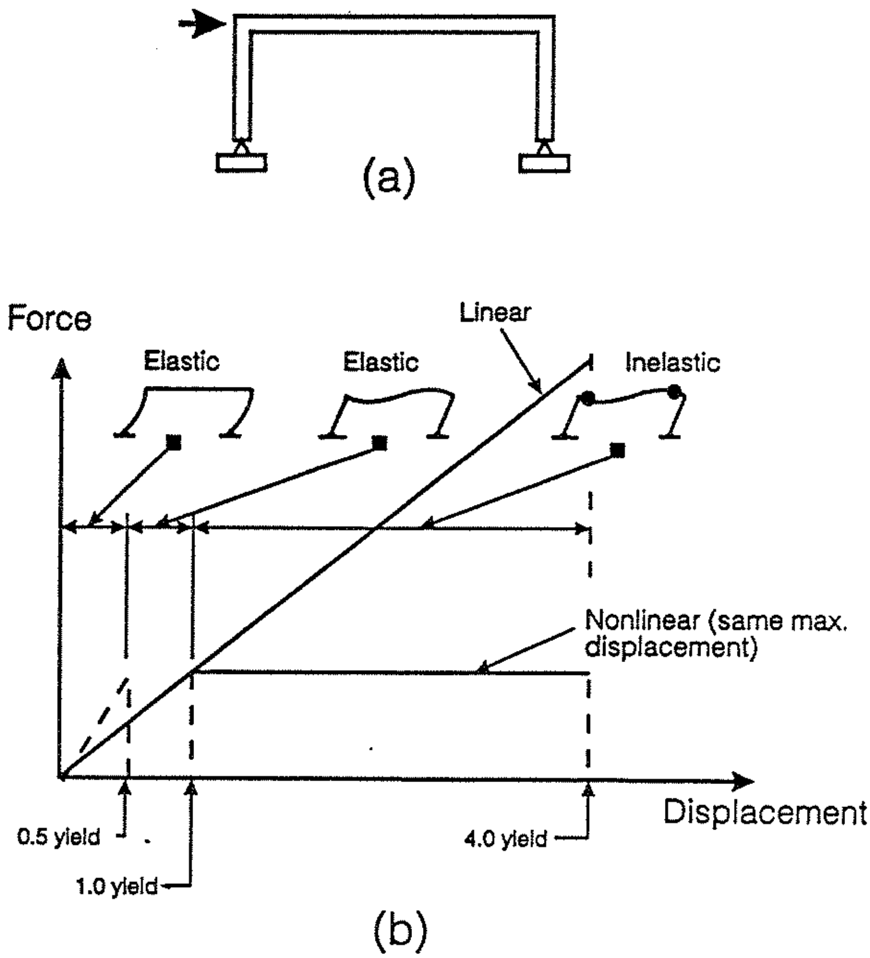


FIGURE 2.3 CONCENTRATION OF DUCTILITY DEMAND

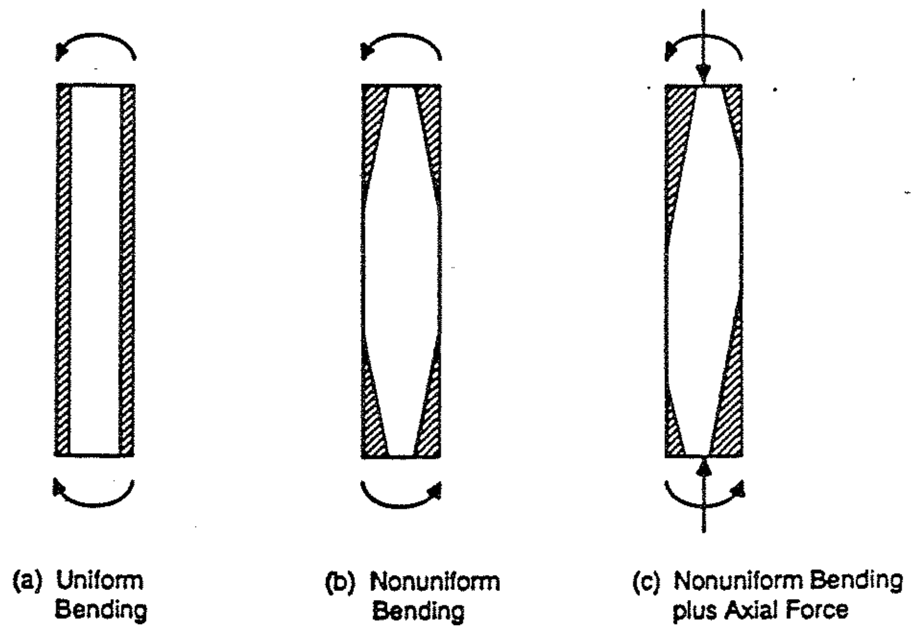


FIGURE 3.1 PLASTIC ZONES FOR DIFFERENT LOADINGS

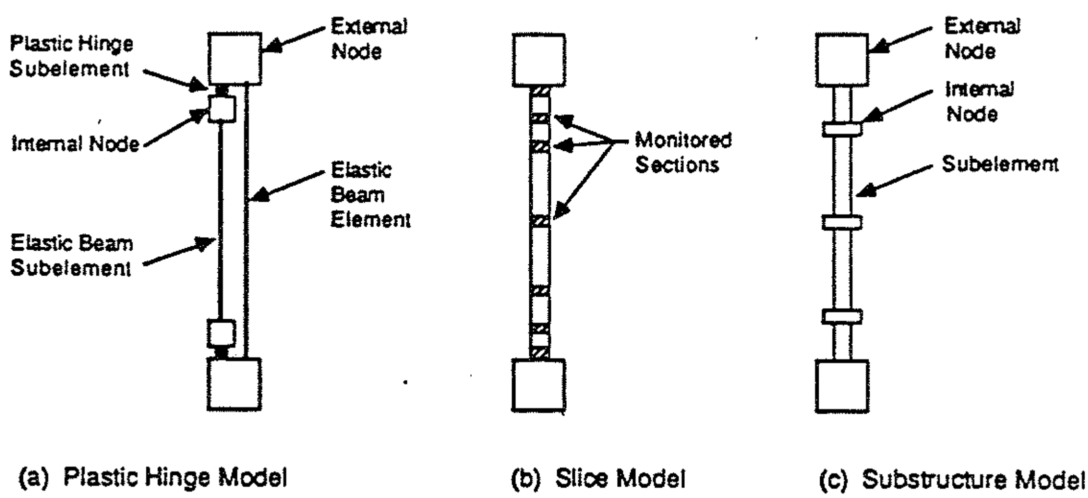
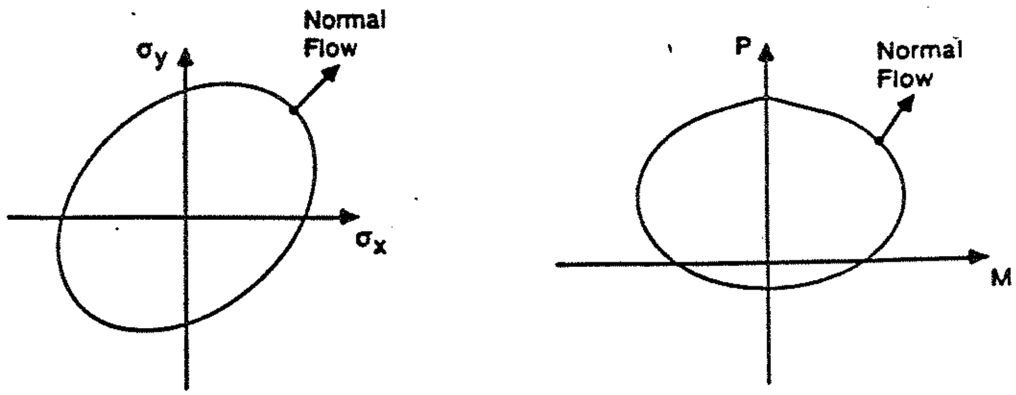


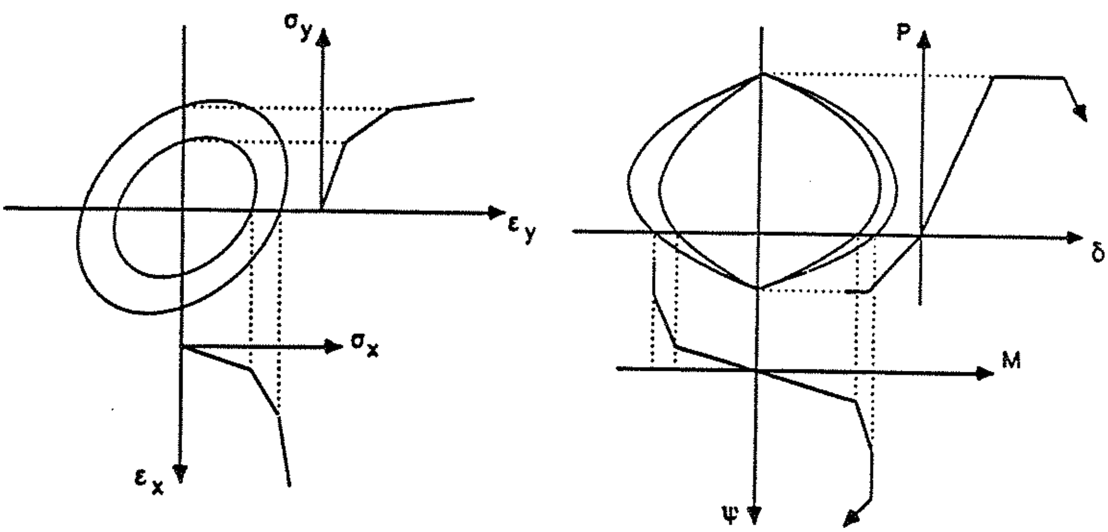
FIGURE 3.2 ALTERNATIVE ELEMENT MODELS



(a) Stresses

(b) Stress Resultants

FIGURE 3.3 PLASTICITY THEORY ANALOGY



(a) Stresses and Strains

(b) Stress and Strain Resultants

FIGURE 3.4 DIFFICULTIES WITH ANALOGY

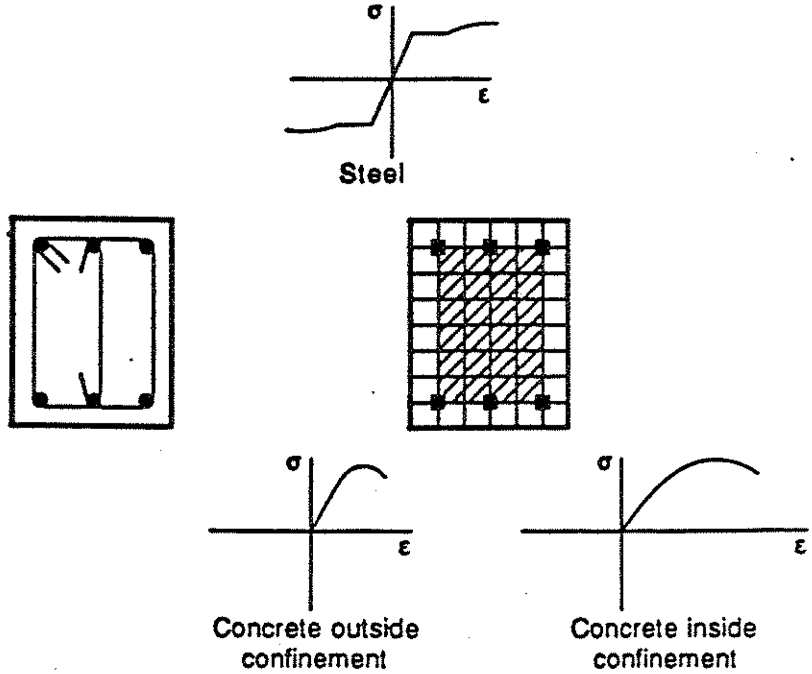


FIGURE 3.5 FIBER REPRESENTATION OF A SECTION

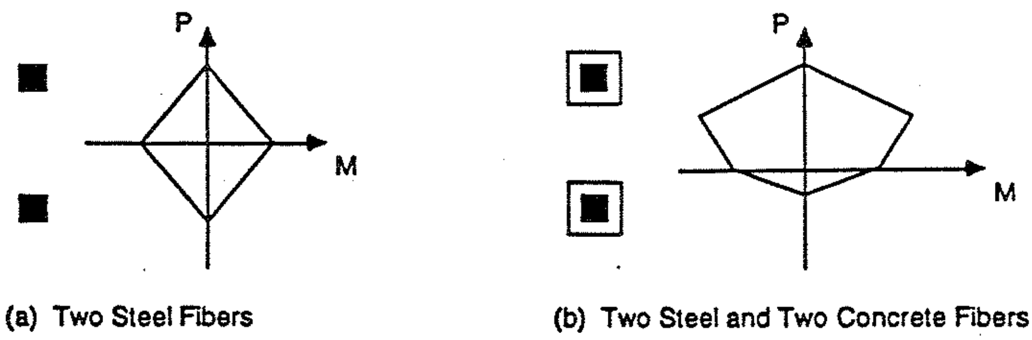


FIGURE 3.6 P-M INTERACTION SURFACES FOR SIMPLE SECTIONS

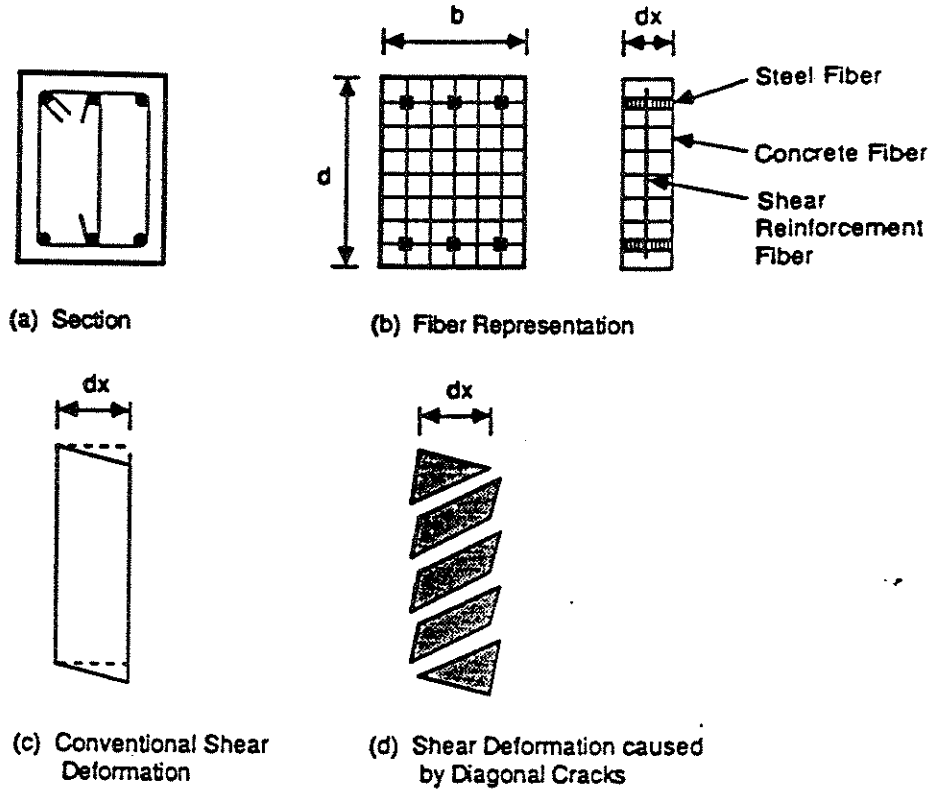


FIGURE 3.7 EXTENSION OF FIBER MODEL FOR SHEAR DEFORMATION

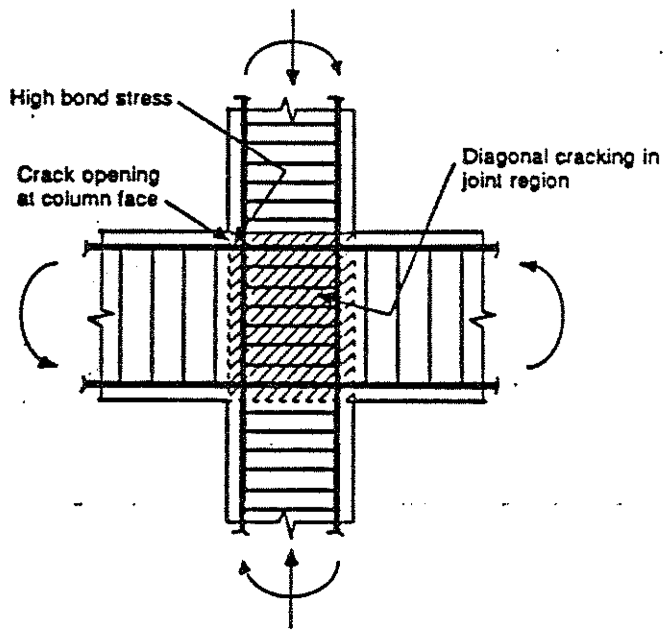


FIGURE 3.8 SOURCES OF DEFORMATION IN A BEAM-COLUMN CONNECTION