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Displacement-Based Design of RC Structures Subjected to Earthquakes

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Earthquake induced structural displacements are a main case of damage in structures subjected to earthquakes. Simple techniques for estimating structural displacements enable development of a design approach based explicitly on expected displacements. This approach is useful both in planning stages where decisions can be made to control displacement demands and in final design stages where details for structural and nonstructural elements are established. Comparison with more conventional force- or ductility-based approaches to design indicates that a displacement-based approach to design is both simple and effective.

INTRODUCTION

Damage in structures subjected to earthquakes may appear in many forms and may be attributed to a variety of causes. In some cases it is convenient to view damage in terms of the occurrence of excessive forces. Examples include the overturning of building contents and failure of reinforced concrete elements in apparent shear and anchorage modes. In other cases it is more convenient to view damage in terms of the occurrence of excessive deformations. Examples here include nonstructural or structural member damage due to exhaustion of material strain capacities, pounding of adjacent structures, and complete collapse due to P-delta effects. The engineer responsible for design or evaluation of a structure should be alert to the fact that both forces and deformations may have an important influence on eventual behavior.

Structural deformations and their contribution to damage may be represented indirectly through displacement ductility measures. Early analytical studies of earthquake response of simple structures in the inelastic range noted that displacement ductility demands for a given earthquake could be related to the strength of the structure. Given these relations, and the tradition of designing structures to resist externally applied loads, it is natural that design approaches emphasizing strength have gained prominence in earthquake engineering.

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These approaches have directed design attention away from the importance of structural deformation as a main determinant of damage in structures subjected to earthquakes.

This paper develops some fundamental ideas of earthquake resistant design and evaluation using structural displacements directly. The ideas, considered together, may be considered to constitute a general displacement-based, or displacement-oriented, design approach. It is assumed throughout that design and evaluation will be based on relatively simple analysis concepts and results; detailed nonlinear response analyses that might lead to insights beyond those illuminated by simple approaches are not considered here. The discussion focuses on applications to reinforced concrete systems, but the methods are more generally applicable.

DISPLACEMENT AS A DESIGN PARAMETER

The idea of using structural displacement information directly in earthquake resistant design and evaluation is not new. Among the notable early references are Veletsos and Newmark [1960] and Muto, et al. [1960]. Based on a limited analog study, Muto, et al. wrote that maximum inelastic displacements of SDOF (single-degree-of-freedom) systems are not significantly different from those of elastic systems having the same initial period and damping. Furthermore, it was noted that this tendency "...may suggest employment of a structural design method based upon the maximum displacement." Details of the design method were not stated, though it may be inferred that displacement information was intended to be used directly to gage adequacy of a structure.

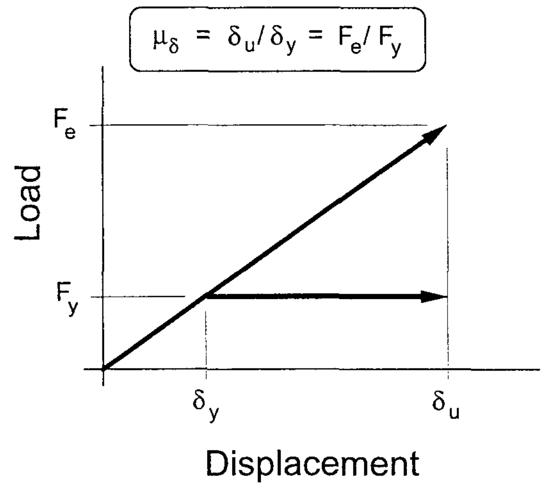

Figure 1 - Displacement ductility based on equal displacement rule

Veletsos and Newmark [1960] arrived at similar conclusions regarding displacements using numerical methods. However, they emphasized using displacement demands in conjunction with target ductility demands to determine required system strength. The pertinent relations between demands on an elastic system and on an equivalent inelastic system are illustrated in Figure 1. If maximum displacement demands for the equivalent elastic and inelastic systems are both equal to δ_u for a given earthquake ground motion, then the displacement ductility demand μ_δ is defined by Equation 1.

$$\mu_\delta = \frac{\delta_u}{\delta_y} = \frac{F_e}{F_y} \tag{1}$$

in which δ_u = expected maximum displacement, δ_y = yield displacement, F_e = force demand on the equivalent elastic system, and F_y = the provided force capacity (strength). Knowing the target ductility demand μ_δ and the calculated elastic force demand F_e, the required (design) strength F_y can be calculated readily from Equation 1.

The basis that maximum displacement demands for elastic and inelastic response are equal is commonly referred to as the "equal displacement rule," and the ratio F_e/F_y is commonly referred to as the "demand-capacity ratio." Several studies have shown that the equal displacement rule is not conservative in all cases, in particular in the short-period range [Newmark and Riddell, 1980; Otani, 1981]. Nonetheless, the simple concept of relating demand-capacity ratios to deformation demands has served as a useful expedient for gaging design requirements in a variety of applications [Park, 1986; Buckle, et al., 1987; Triservices, 1986; ATC-22, 1989].

Proposals for design and evaluation of concrete structures based directly on displacement information have been made previously. Sozen [1981] and Shimazaki and Sozen [1985] used numerical and shake table experimental results to promote displacement control in design, but they did not attempt to establish relations between displacement and required structural details. Moehle and Wallace [1989] and Qi and Moehle [1991] used computed displacement response to evaluate detailing requirements in walls and frames, respectively, but did not present the methodology in a general form. The present paper draws from these earlier studies to develop a general format for a displacement-based approach to design.

DISPLACEMENT RESPONSE OF SDOF SYSTEMS

The inelastic displacement response of SDOF systems has been studied extensively since the 1950s. Available tools range from approximate methods for deriving inelastic response from elastic response spectra [Newmark and Hall, 1982] to direct inelastic response analysis of SDOF systems [Biggs, 1964]. In the present paper, emphasis is on the former approach. The presentation here is intentionally light so as to not detract from the main focus of the paper, which is the use and usefulness of displacement information in seismic evaluation and design.

As summarized by Newmark and Hall [1982], elastic design response spectra may be defined by regions of roughly constant acceleration, velocity, and displacement response. In those period ranges of constant velocity and displacement response, it is commonly found that the inelastic displacement is approximately bounded by the elastic displacement value [Newmark and Riddell, 1980; Otani, 1981]. For shorter periods, maximum inelastic displacements tend to exceed the elastic values. Shimazaki and Sozen [1985] noted that the period corresponding to the intersection of the constant velocity and constant acceleration ranges also corresponded approximately to the peak on the input energy spectrum. They used this attribute of the energy spectrum to define a characteristic ground period T_g. Displacement response amplification for periods less than T_g are found to depend on the ground motion intensity and structural load-displacement characteristics. Techniques for estimating the relation between inelastic and displacement amplitude for periods shorter than T_g have been indicated by Shimazaki and Sozen [1985] and Qi and Moehle [1991].

Displacement amplitude, and overall dynamic stability, are influenced by P-delta effects. In structures where drift is reasonably controlled, P-delta effects are not a major consideration. Where large displacements are anticipated, P-delta effects should be incorporated in the analysis. The subject is discussed more thoroughly in the literature [e.g., Carr and Moss, 1980; Davidson, et al., 1991; Mahin and Boroschek, 1991].

When using a response spectrum to calculate elastic force demands for a reinforced concrete structure, it is common practice to calculate the structural periods based on the gross-section properties rather than a reduced stiffness that results from concrete cracking. This approach is often considered acceptable because a shorter period usually will result in conservatively higher design force levels. In an analysis for expected displacements, it is important to correctly estimate effective stiffness or else to err on the flexible side. As an approximation, it is suggested that the effective structural periods be calculated based on a flexural stiffness equal to half the gross-section flexural stiffness, with damping in the range from two and five percent of the critical value. These values are selected because they approximate typical effective properties for reinforced concrete structures loaded to near the yield level, and because they produce reasonable correlation with both experimental and refined analysis results [Shimazaki and Sozen, 1985; Qi and Moehle, 1991].

DISPLACEMENT RESPONSE OF MULTISTORY FRAMES

Discussion in the preceding section emphasized the estimation of displacement demands for SDOF systems. To extend the application of displacement information to multistory frames, it will be necessary to develop an understanding both of the global displacement of the structure as well as the distribution of that displacement over the height of the structure.

It is important to clarify terms related to displacement. Global displacement refers to the displacement relative to the base of an equivalent SDOF system representing the structure. Roof displacement refers to lateral displacement of the roof relative to the base. Interstory drift refers to the relative lateral displacement between two adjacent floors bounding the story. Drift and displacement are used interchangeably. Drift ratio refers to the drift divided by the height above the base, except interstory drift ratio refers to the difference in lateral displacements for two adjacent floors divided by the distance between the floors.

Numerical and experimental studies of planar frames of moderate height have indicated that displacement response is dominated by response in an apparent first mode [Sozen, 1981; Moehle, 1984]. Saiidi and Sozen [1979] demonstrated that this predominant component of the displacement response could be modeled using a SDOF oscillator having hysteretic properties similar to those of the constituent elements of the frame. This finding suggests that the global displacement maxima of multistory systems may be estimated using simplified response spectrum methods as discussed previously for equivalent SDOF systems. Numerous case studies confirm this view [Moehle, 1984; Qi and Moehle, 1991; Shimazaki and Sozen, 1985]. Experimental data are lacking for flexible structures in which apparent higher modes contribute significantly to displacement response.

In a laterally-loaded multistory framed structure having column strengths exceeding the beam strengths the relative lateral displacement will vary over the height approximately as illustrated in Figure 2b. For this shape, roof displacement may be estimated as being approximately 1.25 times the equivalent SDOF displacement.

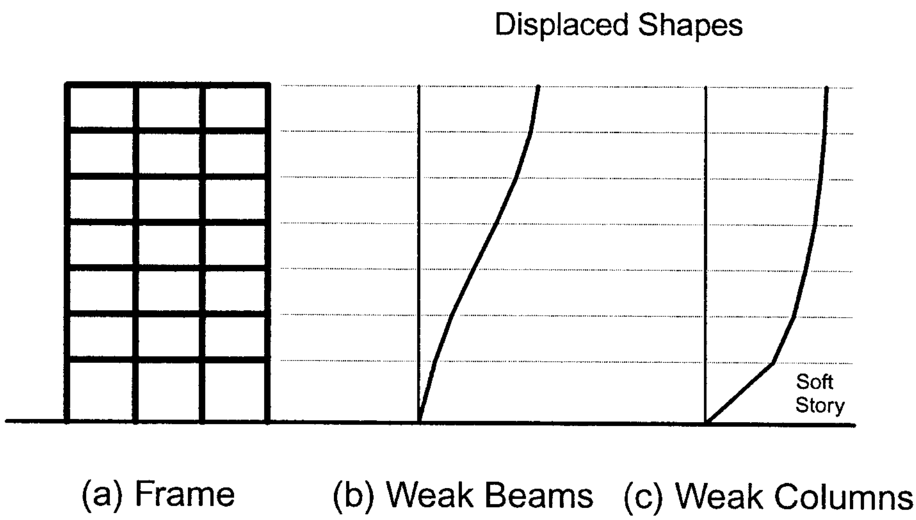

Figure 2 - Building frame displaced shapes

Design requirements will depend not only on roof drift but on the distribution of drift over the height. Elastic analyses may provide insight into drift distribution, but may result in nonconservative estimates due to concentration of inelastic action. Improved information about drift distribution can be obtained from inelastic analyses and model tests of multistory frames. Qi and Moehle [1991] report results of inelastic response analyses of specific five- and ten-story frames subjected to a range of earthquake motions. In the five-story frames that had been designed considering lateral loads (such that strength of framing members varied over the height) the maximum interstory drift ratio was approximately 1.5 times the roof value; in contrast, the frames designed primarily for gravity loads (and therefore having uniform strength over height) had maximum interstory drift ratios as high as 1.8 times the roof value. For the ten-story frames, the respective ratios between interstory and roof drift ratios were 2.1 and 2.8. Sozen [1981] reports measured drift ratios for a series of earthquake simulation tests of small-scale, nine- and ten-story frames. The maximum interstory drift ratio during design-level tests ranged from 1.3 (for regular frame-wall structures) to 2.0 (for irregular frames) times the roof drift ratio. In the analyses and tests reported above, the columns were designed so the sum of column strengths at a joint exceeded the sum of beam strengths at all but the roof level. In addition, it is worth noting that the results are for specific structures; values outside the range noted here are possible for different structures.

The values noted in the preceding paragraph vary widely, and suggest that the ratio between maximum interstory and roof drift ratio will depend on height (tending to increase with increasing number of stories), design basis (whether story strength is uniform or increases toward the base), and regularity. In addition, the ratio is likely to vary depending on characteristics of the base input. Values for a particular structure should be estimated with these tendencies and variations in mind. For typical, regular structures, a ratio of 2.0 may be taken as representative for strong-column/weak-girder frames, with the caveat that ratios exceeding 2.0 are credible.

Quantitative information on drift distribution in structures having columns relatively weaker than beams is limited [Schultz, 1990; Kelly, 1977]. Drift in such structures tends to concentrate in a single "weak" story (Figure 2c), and may be amplified considerably by P-delta effects. A conservative approach is to assume the maximum interstory displacement is equal to the maximum roof displacement, resulting in a relatively high interstory drift estimate for multistory structures. This approach provides a suitable deterrent for new designs having soft stories, but is unsatisfactory for existing construction where realistic solutions are needed for whatever conditions are encountered. Nonlinear static or dynamic analyses may provide additional insight into drift profile, but until verified through experimental or field experience, the results of such analyses should be viewed cautiously. Additional research on frame behavior, including P-delta effects, is necessary before reliable design guidelines for drift in weak-column structures can be made.

DISPLACEMENT AND DESIGN

Previous discussion has suggested methods for gaging the displacement demand on a structure. The utility of displacement information in design lies in the ease with which it may be related to damage. The relation is well illustrated by damage that occurred in the Olive View Hospital during the 1971 San Fernando earthquake (Figure 3) [Mahin, et al., 1976]. Distortion and damage may be identified in the nonstructural glazing and in the structural columns of the first story. Furthermore, the potential for collapse due to P-delta effects (gravity loads acting through lateral displacements) may be inferred. These actions suggest that lateral drift should be a focal point of earthquake engineering design.

Figure 3 - Olive View Hospital following the 1971 San Fernando Earthquake

Several possible applications of displacement information in design are discussed below. It is assumed that to some acceptable degree of accuracy (a) the design earthquake ground motions are known, (b) the resulting overall displacements to a given ground motion can be calculated, and (c) the distribution of the displacement over the height of the structure can be estimated. Awareness of these assumptions and the associated uncertainties is essential to competent application of the concepts that follow. Similar assumptions and uncertainties apply to other approaches for earthquake resistant design.

Nonstructural Elements

Nonstructural damage, disrupted operations, and life-safety considerations make protection of nonstructural elements a high priority. Nonstructural damage usually arises because of inertial forces on the element or because of distortion of the element due to imposed deformations. The latter is of primary interest in the present discussion; damage due to imposed inertial forces is not considered further here.

Racking tests of a variety of materials indicate the onset of damage at lateral distortions on the order of 0.005 times the height, with damage increasing as distortions increase [Sakamoto, et al., 1984; Freeman, 1985]. In a building, it is important to recognize that interstory drift and distortion of nonstructural elements are not necessarily equal, but depend on the nonstructural details [Sakamoto, et al., 1984; Wang, 1987] and the nature of interstory drift. Figure 4 illustrates extreme cases of the relation between interstory drift and distortion of a nonstructural panel. The situation illustrated in Figure 4a may be typical of that occurring in frames, where the distortion of the nonstructural panel relates almost directly to the interstory drift (assuming the panel is not isolated from the frame). Figure 4b illustrates the other extreme where an equal amount of drift occurs with no distortion of the nonstructural panel. Figure 4b may represent the situation in the upper floors of wall buildings. A familiar and classic example of drift without significant interstory distortion is the Pisa Tower.

The range of nonstructural materials and details in existing and new construction is so broad no attempt will be made to draw quantitative conclusions within the scope of the present paper. Nonetheless it is clear that structural deformation and nonstructural detailing should not be overlooked in design and evaluation.

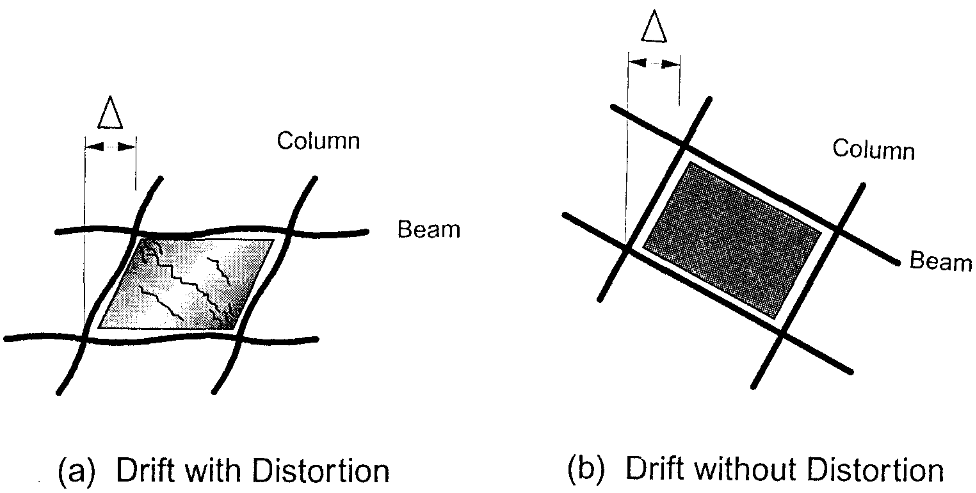

Figure 4 - Interstory drift and distortion of nonstructural elements

Non-lateral Load Resisting Structural Elements

Current design practice allows seismic resistance to be assigned entirely to select portions of a structure, the remaining portions being designed to carry vertical loads only. Examples in the latter category include precast floor systems, flat-plate floor systems, and complete existing framing systems in seismically rehabilitated structures. These "vertical load carrying elements" are susceptible to damage due to earthquake induced lateral deformations [e.g., Pan and Moehle, 1989], and should be checked to ensure they can continue to support vertical loads under the imposed lateral deformations.

When demand-capacity ratios are used to estimate ductility demands in a structure, the demand and the capacity refer to the lateral load resisting elements. The demand-capacity ratio provides no direct information regarding performance of the non-lateral load resisting elements. Analyses of behavior should be carried out by directly considering the expected displacements.

Lateral Load Resisting Elements

The displacement-based approach to design of reinforced concrete structural elements follows directly from fundamental concepts of element deformation. For clarity, only flexural deformations will be considered. The basic concepts are illustrated in relation to the simplified reinforced concrete single-column bridge bent of Figure 5. It is assumed that under the action of uniaxial lateral load the column yields in flexure at the base. As described in greater detail elsewhere [Priestley and Park, 1987], flexural deformation is modeled by elastic curvature over height and inelastic curvature concentrated in a plastic hinge of equivalent length l_p at the base. The ultimate displacement capacity according to this model is

$$\delta_u = \frac{\phi_y l^2}{3} + (\phi_u - \phi_y) l_p (l - \frac{l_p}{2}) \tag{2}$$

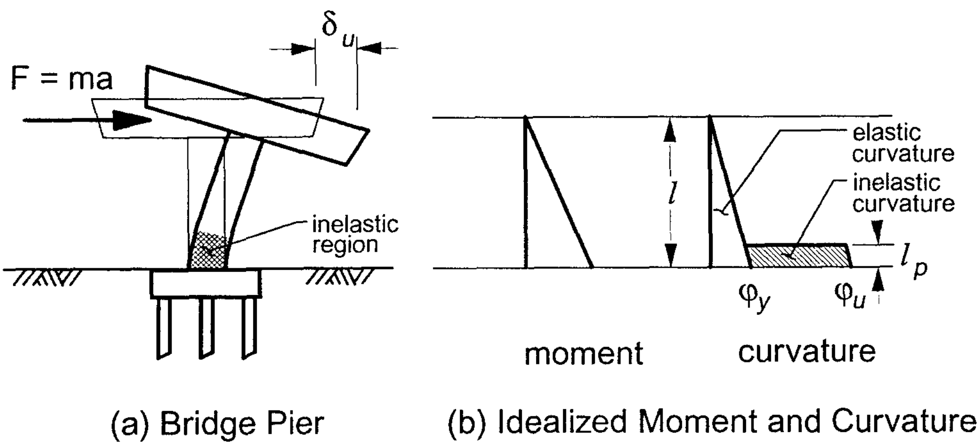

Figure 5 - Idealized flexural curvature in a cantilever bridge column

In Equation 2, ϕ_y = yield curvature, l = column height, and ϕ_u = ultimate curvature. The equivalent plastic hinge length l_p in a reinforced concrete member depends (in a manner not yet well understood) on section depth, column aspect ratio, bar diameter, and magnitudes of axial load and shear. Satisfactory correlation with laboratory test results can be obtained using a length equal to $0.5h$, where h = the section depth.

Equation 2 can be used to calculate ultimate displacement capacity based on calculated yield and ultimate curvature capacities. Alternately, Equation 2 can be solved for required ultimate curvature capacity ϕ_u in terms of the ultimate displacement demand δ_u as expressed below.

$$\phi_u = \left(\delta_u - \frac{\phi_y l^2}{3}\right)\left(\frac{1}{l_p(l - l_p/2)}\right) + \phi_y \tag{3}$$

Ultimate curvature ϕ_u indicated by Equation 3 can be compared directly with calculated ultimate curvature capacity to gage adequacy of a structural member.

So that relations between lateral drift and required details will become transparent, a further simplified model for flexural deformability is proposed (Figure 6). According to the simplification, elastic curvatures outside the plastic hinge zone (Figure 6b) are ignored, and the total curvature near the base is centered at the base of the member instead of just above

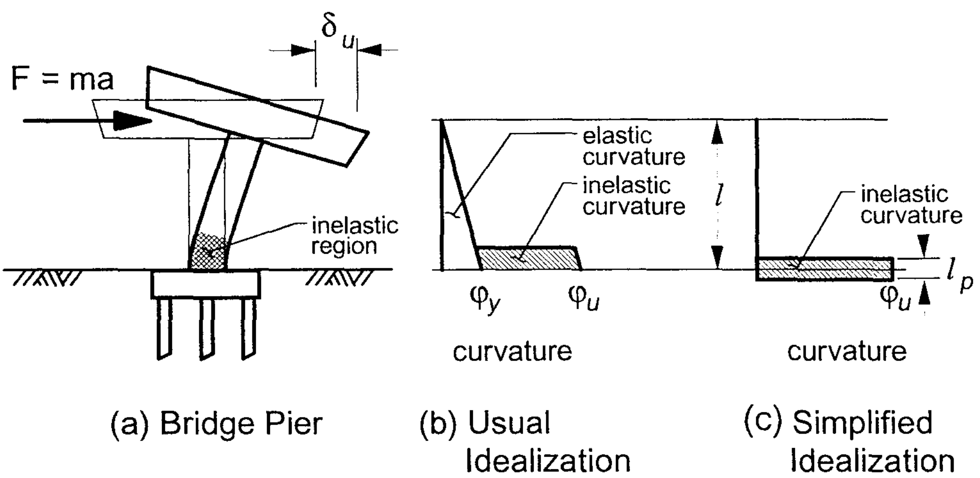

Figure 6 - Simplified curvature distribution in cantilever bridge column

the base (Figure 6c). The ultimate displacement capacity with this model is given by Equation 4.

$$\delta_u = \phi_u l_p l \tag{4}$$

The simplified model (Figure 6) produces displacement estimates not significantly different from those produced with the more commonly used model (Figure 5) for common ranges of section curvature ductility and member aspect ratio, as shown in Figure 7. In Figure 7, the plastic hinge length l_p is taken equal to $0.5h$.

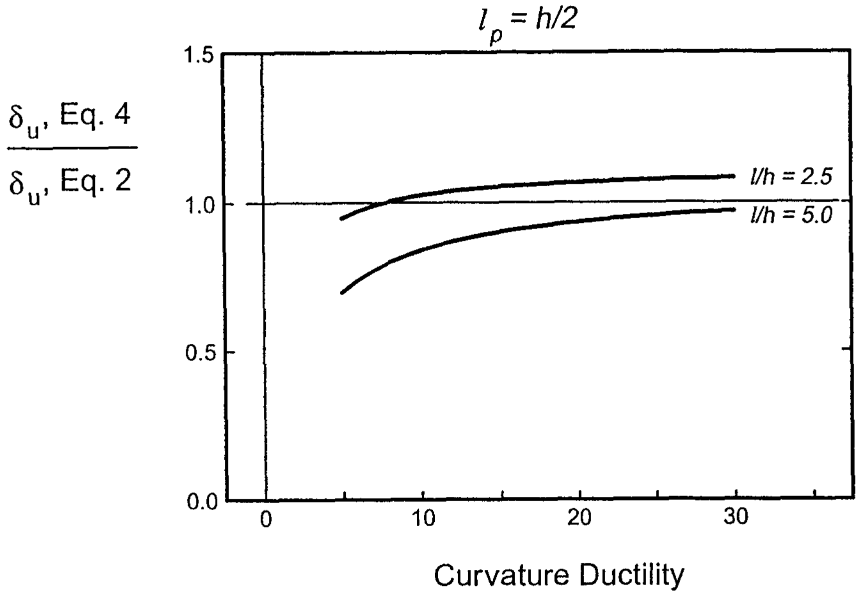

Figure 7 - Comparison of calculated displacements by Equations 2 and 4

Assuming plane sections remain plane, curvature in Equation 4 can be expressed as a ratio of strain and distance from the neutral axis, as depicted in Figure 8. Equation 5 expresses the relation.

$$\phi_u = \frac{\epsilon_u}{\beta h} \tag{5}$$

in which ε_u is limiting strain of a point on the cross section located a perpendicular distance βh from the neutral axis. Substituting Equation 5 in Equation 4, substituting $0.5h$ for l_p, and dividing by member length l results in Equation 6.

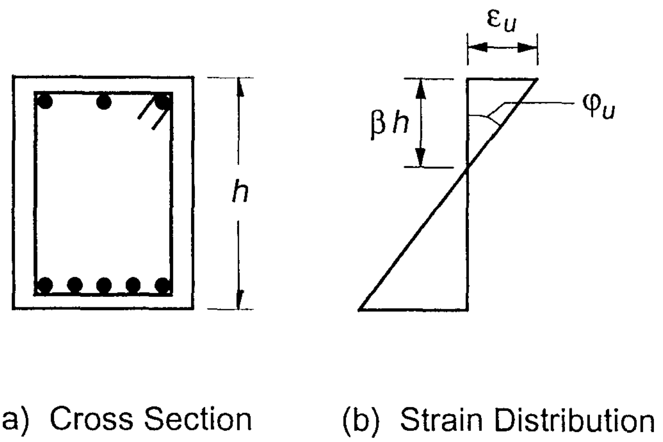

Figure 8 - Idealized cross-sectional strain distribution in flexural member

$$\frac{\delta_u}{l} = \frac{1}{2\beta}\epsilon_u \tag{6}$$

Equation 6 implies that drift ratio δ_u/l can be related directly to strain, and, consequently, to damage and required details.

As an example of the application of Equation 6, the drift capacity of the bridge column illustrated in Figure 6 will be evaluated. The column cross section is shown in Figure 9a. It is assumed that the transverse reinforcement is insufficient to confine the concrete. A conventional sectional analysis is carried out according to ACI 318-89 [1989], except maximum usable concrete compression strain is taken equal to 0.004. According to the analysis, the neutral axis under ultimate loading conditions is located at a depth of 14.4 inches (Figure 9b). Therefore, the quantity β in Equations 5 and 6 is β = (14.4 inches / 72 inches) = 0.2. Substituting $\beta = 0.2$ and $\varepsilon_u = 0.004$ into Equation 6 results in a drift ratio capacity $\delta_u/l = 0.01$.

Equation 6 is not limited to assessment based on compression strain capacity. It may also be used in assessing tensile strain demands, for example, in longitudinal reinforcement of a reinforced concrete member. As an example, consider Figure 9c, in which the term βh is taken to represent the depth of the tension zone rather than the compression zone. For this case, β = (54 inches / 72 inches) = 0.75. Therefore, according to Equation 6 the drift capacity is equal to two-thirds of the effective tensile strain capacity. The effective strain capacity may be determined by strain capacity under reversed cyclic loading of plain reinforcement, lap-spliced reinforcement, or mechanically-coupled reinforcement, as the case may be. For example, if effective tensile strain capacity was 0.06, effective drift ratio capacity would be 0.04 for this example.

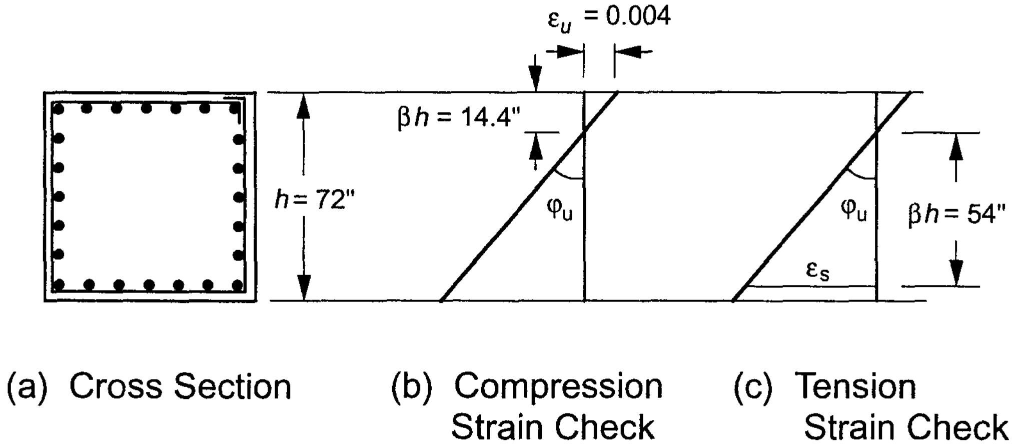

Figure 9 - Example of concrete bridge column

It is worth noting that the estimates of drift capacity made in the preceding two paragraphs are certainly imperfect. Although refinements to the calculation procedure are undoubtedly possible, it is doubtful that the added precision will add significantly to the overall accuracy of the estimate. Uncertainties associated with defining the subject earthquake, the structural response, and the member and material behavior under reversed cyclic loading are certain to eclipse minor efforts at refinement. Despite the imperfections, the analysis is still useful, if only to indicate that limits must be placed on bridge displacements.

In order to establish relations between interstory drift and strain in a multistory building it is necessary to understand the relative contributions of beam and column deformations to the total interstory drift. The discussion here is restricted to multistory frames with typical aspect ratios. Current design practice for frames in regions of high seismic risk requires that the columns in new construction be stronger than the beams. As a consequence, the majority of inelastic action in intermediate floors is expected to occur in the beams. Therefore, a conservative approach to establishing beam deformation demands is to assume that the columns are rigid (Figure 10a). According to this assumption the effective beam rotation (or drift ratio δ/l, Equation 4) is equal to the interstory drift ratio of the stories containing the beam. Columns at these same intermediate floors should be designed to have strength exceeding the beam strength, and should be capable of accommodating moderate inelastic action [Park, 1986]. The columns framing into the foundation cannot be protected easily from inelastic deformation demands. Neither can their deformations be easily determined because of uncertain flexibility of the foundation and first-floor beams. Given the critical nature of these columns, and the numerous failures reported in the literature, it is recommended that they be detailed conservatively to sustain an

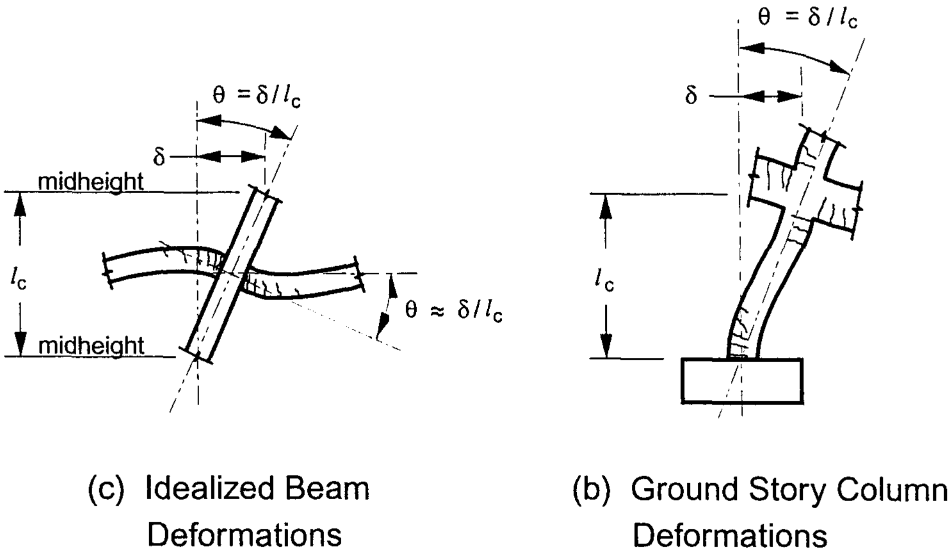

Figure 10 - Beam and column deformations in multistory building frame

effective column rotation (or drift ratio δ/l, Equation 4) equal to the expected interstory drift at that location.

A previous example related to a bridge column suggested that maximum ratio drift capacity considering tensile strain limits is on the order of 0.04. In a building where maximum interstory drift ratio may be twice the roof drift ratio, the corresponding maximum allowable roof drift ratio would be approximately 0.02. A building plan calling for roof drift ratio exceeding (or even approaching) this value is likely to be impractical. Roof drift ratios significantly less than this quantity would be desirable if the objective is to control nonstructural and structural damage.

Equation 6 can be used to evaluate general design and detailing guidelines for ductile frames. As an example, consider an unconfined beam. Flexural response is calculated according to ACI 318-89 [1989] except maximum usable concrete compression strain is taken equal to 0.004. Concrete strength is 4000 psi, and yield strength of elasto-plastic Grade 60 reinforcement is 75 ksi. Figure 11 plots results in which beam rotation capacity is δ_u/l as defined by Equation 6. According to Figure 10a, the beam rotation in Figure 11 may be taken equal to the interstory drift ratio in a building. The results indicate a maximum drift ratio capacity of approximately 0.01 for the permitted range of longitudinal reinforcement ratios. ACI 318-89 seismic provisions require transverse reinforcement at close spacing in plastic hinge regions of beams, resulting in an increase in the usable concrete compressive strain capacity [Park, 1986]. As a result, flexural drift ratio capacities in excess of those indicated in Figure 11 are likely in beams satisfying current code provisions. Experimental results support these analytical results [Qi and Moehle; 1991].

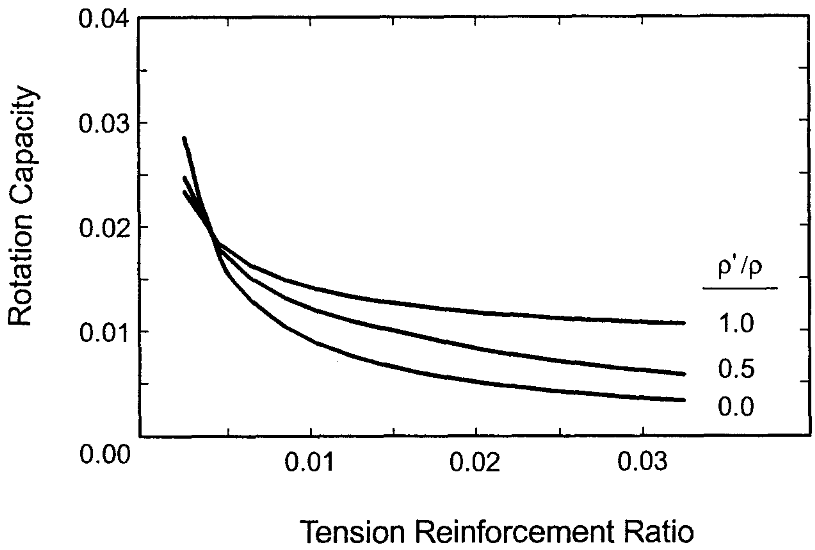

Figure 11 - Rotation capacity of reinforced concrete beam sections

The concepts described above can be applied to other structural elements. For example, reinforced concrete structural walls may be visualized as beams oriented vertically and having tension reinforcement equal to compressive reinforcement. The results of Figure 11 suggest that if walls are provided in sufficient quantity to limit the drift ratio to 0.01, confinement of the wall boundaries is unnecessary. A more-detailed analysis of wall boundary element requirements using this approach is discussed by Wallace and Moehle [1991]. Detailed analyses of other structural elements will not be pursued in this paper.

Overall System Response

Previous discussion has emphasized the importance of displacement control in relation to local behavior of nonstructural and structural elements. Displacement control is also essential to ensuring satisfactory overall system response, in particular, in relation to pounding of adjacent structures and incremental collapse due to P-delta effects.

Pounding between adjacent structures has been identified as a contributor to damage and collapse during earthquakes [e.g., Bertero, 1987]. Current US codes permit earthquake resistant design to be carried out using equivalent static force procedures. To reduce the likelihood of pounding these codes stipulate minimum building separations. Expected displacements during a strong earthquake are not obtained directly using the code static procedure because the loads do not relate directly to the earthquake induced actions and the structural model used for design does not represent stiffness properties expected during an earthquake. It is therefore difficult to assure that pounding will not occur if the code minimum requirements are applied. If a direct analysis of expected displacement response is made, pounding can be avoided with a higher degree of certainty.

Second order (P-delta) effects associated with large displacement response may result in displacement amplitudes exceeding those estimated by conventional analyses, and can therefore increase the potential for damage and collapse [Carr and Moss, 1980]. As indicated for a SDOF structure in Figure 12, equilibrium requires that $M = Vl + P\delta$, in which M is the base moment, V is the lateral inertial force acting on the mass, and P is the vertical force acting on the mass. For inelastic response, substituting the plastic moment strength M_p for M and solving for V results in the following expression defining the effective lateral load resistance V.

$$V = \frac{M_p}{l} - P\frac{\delta}{l} \tag{7}$$

From Equation 7 it is clear that the effective resistance is most significantly affected when the base shear strength M_p/l is low or the lateral drift ratio δ/l is high. Some studies of inelastic seismic response of frames satisfying current code strength requirements indicate that P-delta effects will not significantly impact response if drift levels (ignoring P-delta effects) are below 0.01 [Carr and Moss, 1980]. More detailed parameter studies of P-delta effects are reported elsewhere [Davidson, et al., 1991; Mahin, 1991].

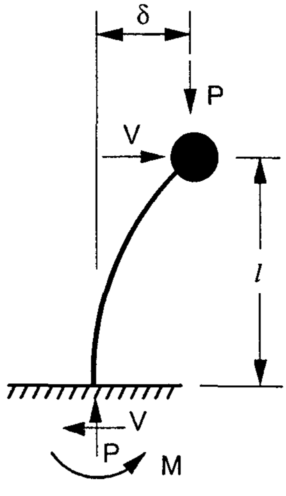

Figure 12 - P-delta effects on a SDOF system

COMPARISON OF DRIFT AND DUCTILITY APPROACHES TO DESIGN

The preceding discussion has described the connection between drift and expected performance of reinforced concrete structures subjected to earthquakes. The relevance of displacements has long been recognized, but seldom are displacements made the focal point of design. Instead, displacements are usually considered indirectly through ductility measures or demand-capacity ratios, or not considered at all. The paragraphs that follow compare the basic drift and ductility approaches to design, with the objective of emphasizing the relative merits of the approaches.

The displacement-based approach to design of structural elements has been described previously. The overall approach is summarized in Figure 13a in relation to a reinforced concrete bridge column. In Step 1, the strength and effective structure period are estimated. The expected inelastic displacement is then determined from a displacement response spectrum (Step 2). As described previously, for long-period structures an elastic response spectrum can be used to estimate expected inelastic displacements directly. For short-period structures, alternate procedures should be used [see, e.g., Shimazaki and Sozen, 1985; Qi and Moehle, 1991]. In step 3, the ultimate curvature demand ϕ_u is calculated from the displacement demand using Equation 3. Lastly, the section curvature capacity is checked for a given set of details and proportions using accepted section analysis methods (Step 4). As an alternate, Steps 3 and 4 may be replaced by Equation 6 which relates strain demands directly to expected drift demand.

In a typical ductility-based approach to design, the ratio of elastic force demand to structure strength (demand-capacity ratio) is used to gage the displacement ductility and, subsequently, the section detailing requirements. The procedure is illustrated by the four steps outlined in Figure 13b. In Step 1, the strength and effective structure period are estimated. In Step 2, the displacement ductility demand is estimated based on the demand-capacity ratio, that is, the ratio of elastic force demand (as read from an elastic response spectrum) to the structure strength. For long-period structures, the equal displacement rule indicates that the displacement ductility is equal to the demand-capacity ratio (Figure 1). In Step 3, the curvature ductility demand is calculated from the displacement ductility demand according to Equation 8. As a final step, the cross section is analyzed to determine if the available curvature ductility capacity for a given set of details is sufficient for the calculated curvature ductility demand (Step 4).

Step 3 of the preceding paragraph required calculation of the curvature ductility demand based on the expected displacement ductility demand. For a cantilever column (Figure 5), the relation between curvature ductility and displacement ductility can be derived from Equation 3. Assuming $l_p = 0.5h$, the resulting expression is

$$\mu_\phi = \frac{2}{3}(\mu_\delta - 1)\left(\frac{l}{h}\right)\left(\frac{1}{1 - h/4l}\right) + 1 \tag{8}$$

In Equation 8, the displacement and curvature ductilities are defined by $\mu_\delta = \delta_u/\delta_y$ and $\mu_\phi = \phi_u/\phi_y$.

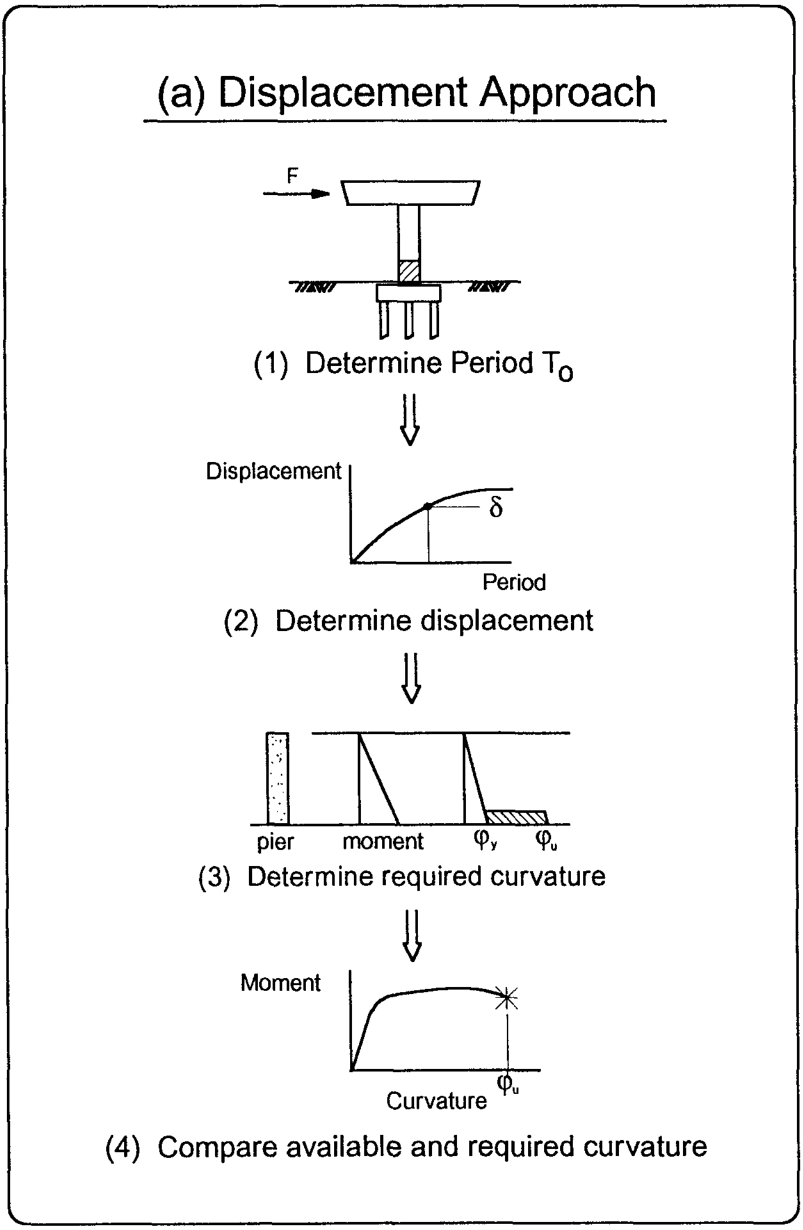

Figure 13 - Displacement and ductility approaches to design

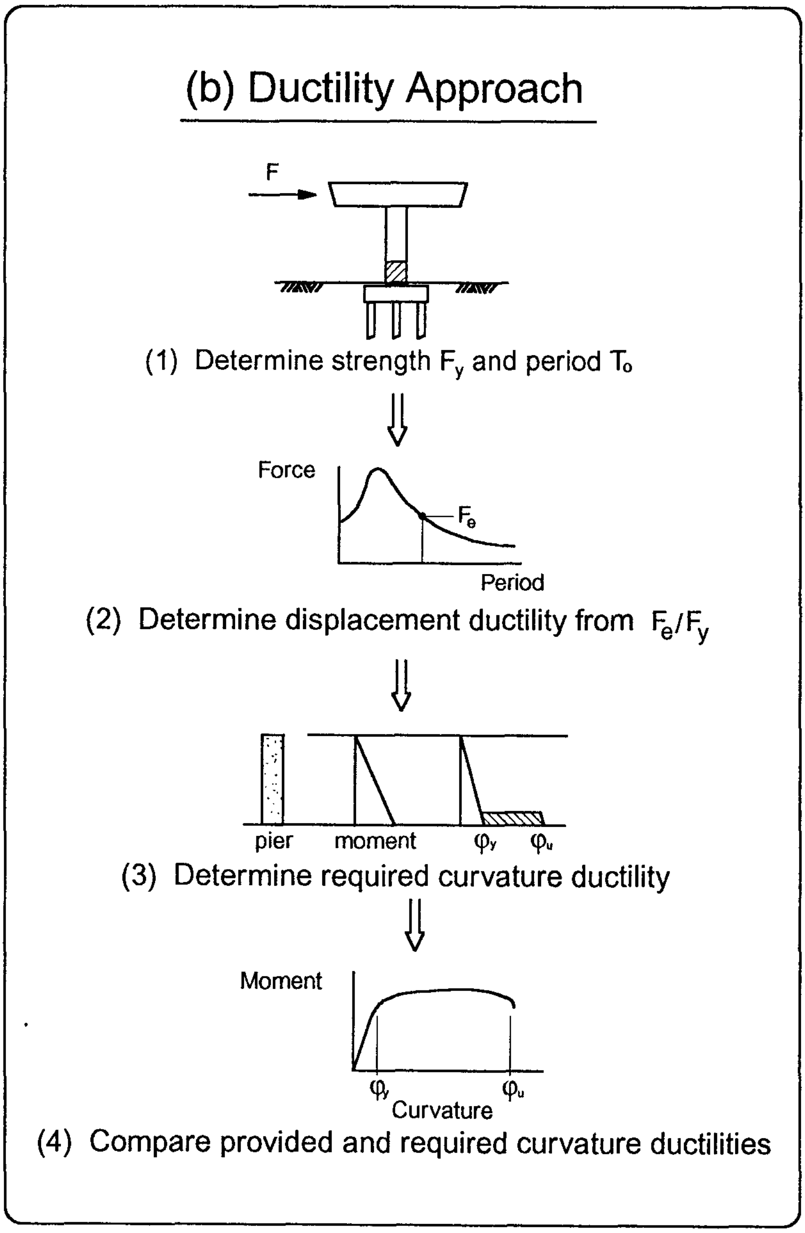

Figure 13 (continued) - Displacement and ductility approaches to design

The parallel nature of the ductility and displacement approaches should be apparent in Figure 13. Though the algorithms are similar, and the end results presumably the same, the information that is processed differs. The relative merits of the two approaches lie in the ease with which this information is available and interpreted. In the ductility-based approach, readily calculated elastic forces define the ductilities but do not provide a clear image of deformations. In the displacement-based approach, displacements are used directly to define element deformations.

The nature of the information used in the ductility-based and displacement-based approaches can influence decisions made in the design process. As an example, consider the existing structure illustrated in Figure 14a. It is assumed for simplicity that the effective structure period falls in the long-period range so that expected inelastic displacement amplitude can be read directly from an elastic displacement response spectrum. The columns of the structure possess a load-deformation behavior represented by curve "a" in Figure 14b, with yield occurring at a unit displacement and failure occurring at displacement δ_u equal to 3 units, such that the available displacement ductility is $\mu_\delta = 3$. The structure possesses an effective structure period T_a, with resulting displacement demand δ_a and acceleration demand A_a read from the linear elastic response spectra of Figure 14c. As indicated in Figure 14b, the displacement demand δ_a exceeds the deformation capacity of the columns, so that seismic upgrading of the structure is in order. It is assumed in the following discussion that the structure will be retrofit by addition of external structural elements rather than by direct modification of the columns.

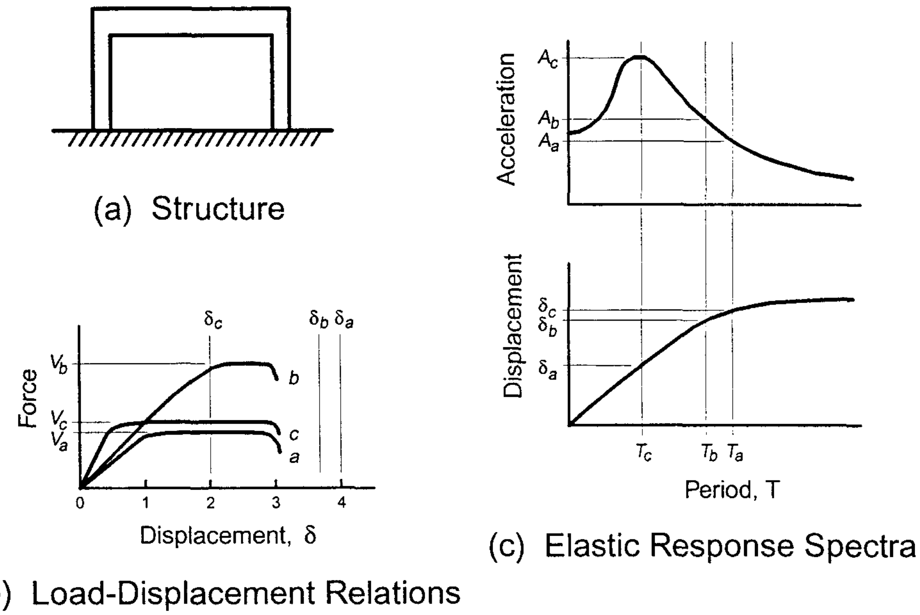

Figure 14 - Example redesign of existing frame

In a ductility-based approach the apparent goal in redesign of the structure (Figure 14) is to reduce the demand-capacity ratio (because doing so will reduce the nominal ductility demand). This goal may suggest that the successful redesign is one that (a) adds strength and (b) avoids significant stiffness increase because an increase in stiffness results in an increase in "demand." A plausible solution based on this specious goal is illustrated in Figure 14b and 14c, where the structure strength is increased to V_b and the stiffness is increased only slightly resulting in an effective structure period T_b. As illustrated in the figure, this solution may not adequately change the deformation demand on the columns and therefore might not provide the desired degree of safety. Specifically, for the period range of the subject structure, modifications that influence strength and not stiffness cannot be effective for protecting the critical columns.

A displacement-based approach indicates directly that if the columns are to be protected the structure must be stiffened so as to reduce the displacement demand. A solution satisfying this more realistic goal is illustrated in Figure 14, where the redesign results in a significant increase in stiffness (load-displacement relation "c"), with corresponding reductions in effective structure period T_c and displacement demand δ_c.

The example illustrated in the preceding paragraphs is obviously simplified. Seldom can the structure strength and stiffness be modified independently as has been assumed. Furthermore, there are cases (e.g., short-period structures situated on soft sites) where strength has a significant influence on deformation demand. Nonetheless, the example illustrates the advantage of operating directly with deformation quantities as opposed to demand-capacity ratios. The particular case considered here illustrates further that seismic strengthening frequently is more a matter of stiffening than it is of strengthening.

The ductility-based approach is well suited to cases where inelastic response is distributed uniformly throughout the structure; in such cases the local demand-capacity ratios provide a reasonably accurate picture of ductility distribution and magnitude. Where inelastic response is not uniformly distributed the local demand-capacity ratios do not provide information on local ductility demands, and the displacement-based approach may be preferred. As an example, consider a bridge column founded on a flexible foundation and having an long effective structure period (Figure 15). The elastic rotational stiffnesses of the foundation and of the column are assumed to be equal, and the column strength is assumed to be one fourth of the elastic force demand. During an earthquake, the maximum displacement of the superstructure is determined by the combined flexibility of the column and foundation. If the ductility-based approach is used, the column displacement ductility demand may erroneously be estimated to be equal to the demand-capacity ratio (that is, $\mu_\delta = 4$). If displacements are viewed directly, as in Figure 15, the correct displacement ductility for the column ($\mu_\delta = 7$) is obtained. Corrections to the demand-capacity approach have been proposed for conditions such as those encountered in this example [Chapman, 1982]; with the displacement-based approach these corrections are unnecessary.

The preceding discussion has emphasized either a displacement-based approach to the exclusion of ductility or force considerations, or a ductility-based approach to the exclusion of force or displacement considerations. A responsibly comprehensive analysis of

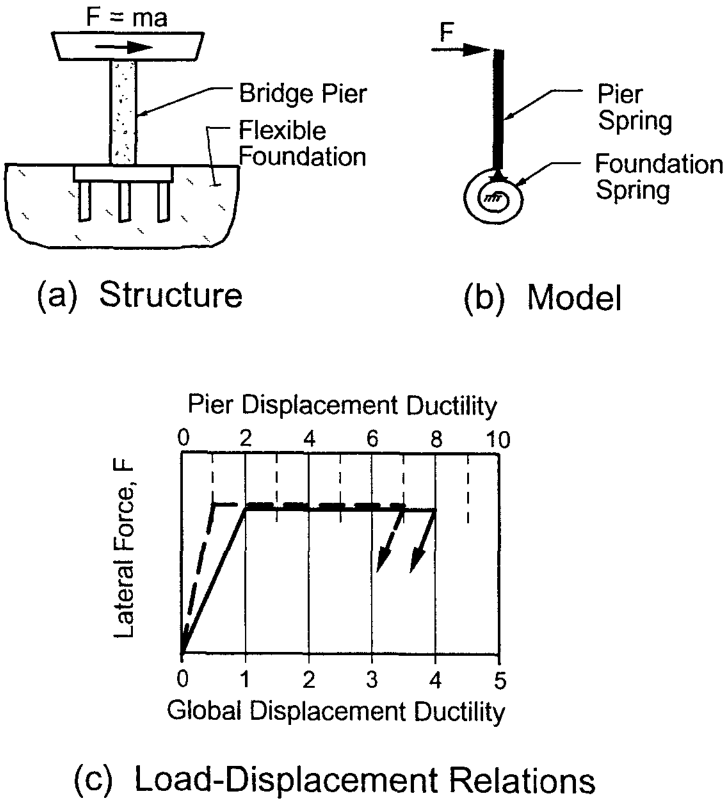

Figure 15 - Bridge pier with flexible foundation

seismic response should not follow either of these simplistic approaches. Instead, the analysis should consider displacements, forces, and ductilities, regardless of the basic emphasis (displacement or ductility) of the analysis. Lateral displacements result in P-delta effects that may be important. Forces associated with inelastic flexural response determine actions in shear and other less-ductile response modes, and should be used in a capacity design approach to establish required strengths [Park, 1986]. Flexural ductility plays an important role in capacity design where behavior in nonductile modes (e.g., shear in reinforced concrete columns) depends on the ductility level [Aschheim and Moehle; 1992]. For these reasons, displacements and forces should not be overlooked when using a ductility-based approach, and forces and ductilities should not be overlooked when using the displacement-based approach.

CONCLUSIONS

Studies of the inelastic response of simple and complex structures have resulted in the development of uncomplicated tools for estimating maximum lateral drift during a strong earthquake. Two approaches to design and evaluation using drift information are available. A ductility-based approach uses displacement information indirectly, establishing ductility requirements as a function of the provided strength and the strength required for elastic response. A displacement-based approach uses displacement information directly. The latter approach has been the main subject of the present paper.

The displacement-based approach can be used to establish proportions and layout that will control drift demand, and to determine structural and nonstructural details that will ensure adequate performance. Several examples illustrating its application have been presented. The examples demonstrate that the displacement-based approach is a simple and effective tool for design.

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