Myths and Fallacies in Earthquake Engineering

— Conflicts between Design and Reality

by M. J. Nigel Priestley

It has long been recognized that our codified approaches for seismic design bear a comparatively tenuous relationship to expected performance. Design is based on a static "snapshot" simulation of the dynamic event, using methods extrapolated from approaches felt to be adequate and conservative for gravity load design. A major difference between gravity load effects and seismic response is that ultimate strength should never be developed under gravity load, while it is almost certain to be developed under seismic response, typically at a level of excitation that may be a fraction of the design level for seismic attack. Further, although ductile seismic response implies greater dependency on displacements than forces, we still, as a matter of convenience and tradition, design for specified force levels, and treat displacements in a comparatively cursory way.

When the design approach is based on carefully considered philosophy, as in the capacity design approach pioneered in New Zealand and gradually becoming accepted in many other seismic regions, excellent results are to be expected because the design structure is comparatively insensitive to the various assumptions made.

It appears, however, that the enormous approximations involved in seismic design are perhaps becoming less appreciated, rather than more, as sophisticated analytical techniques become specified by codes and accepted into common design practice, as a matter of routine. In the United States, and I suspect elsewhere, this has resulted in a tendency for the functions of analysis and design to be separated and performed by different specialists. The analyst is responsible for modeling the structure and running the lateral force analysis — typically a 3-D modal analysis process. Results from the analysis are presented to the designer who determines member sizes, reinforcement quantities (if reinforced concrete construction), and detailing aspects. The analyst is typically more involved in the analytical process than in the correct simulation of member characteristics, with potential dangers. The result of the separation of design and analysis tends to be that analysis drives the design process, rather than the reverse, which might seem to be more appropriate.

There is also room to examine current design and detailing practice, much of which is also extrapolated from gravity load considerations. Occasionally, this process can lead us in directions that are inappropriate for seismic behavior. Even when tenets of structural performance are based on purely dynamic characteristics, such as energy dissipation under cyclic response, the directions we are accustomed to taking are not necessarily the best for survival and damage control.

In this paper, some of the accepted seismic design and analysis procedures are identified as "myth" or "fallacy" — an overstatement perhaps, to make a rather dry topic seem more interesting. Nevertheless, a critical examination of the bases of our design processes is always appropriate, since the origins of these are often obscure, and lost in the history of design practice, or worse, in code committee minutes. Some of the points to be made are well known, others perhaps less so.

The elastic spectral analysis fallacy

The fundamental basis of seismic design is still the assumption that an elastic (or modified elastic) acceleration re-
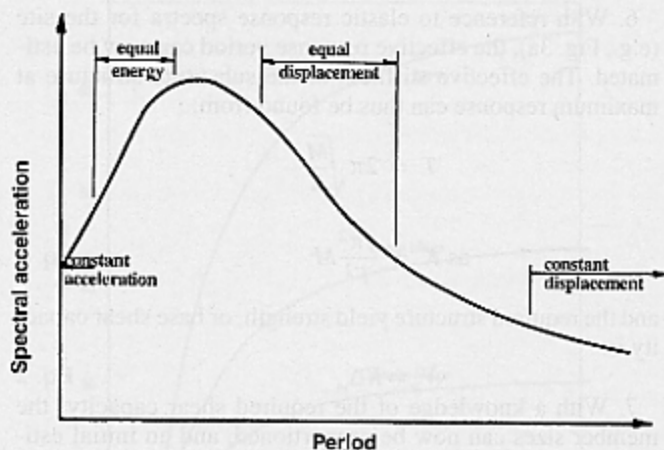
Fig. 1 — Design acceleration response spectrum.

sponse spectrum provides the best means for establishing required performance of a structure. The limitations of the approach are well known, and accepted because of the design convenience and the lack of a viable design alternative. A case can be made that this is a fallacy, and that viable design alternatives exist, or could be developed with comparative ease.

To summarize the limitations:

1. Response is based on a "snapshot" of structural response; that is, response at the moment of peak base shear for an equivalent elastically responding structure. Duration effects, which tend to be period-dependent, with short-period structures suffering a greater number of response cycles than long-period structures, are not considered. The merits of using modal combination rules to provide some insight into higher mode effects seems hardly worthwhile when these will have to be considered by largely empirical rules later in the capacity design process.

2. The relationship between peak displacement response of elastic and inelastic systems is complex, and more variable than commonly accepted. Various rules, such as the "equal energy" and "equal displacement" rules, are commonly employed, but without much consistency or logic. If we consider a typical elastic acceleration spectrum, as in Fig. 1, four distinct zones can be identified.

At zero period, displacements of elastic and ductile sys-

tems cannot, by definition, be related. The structure will be subjected to peak ground acceleration (PGA), regardless of ductility capacity, and will fail if a lesser strength than that corresponding to PGA is provided. In the rising portion of the acceleration spectrum, displacements of inelastic systems are greater than those of elastic systems with equivalent initial stiffness, and the "equal-energy" relationship has some application.

In the initial stages of the falling portion of the acceleration spectrum, elastic and inelastic displacement responses are often similar, leading to the "equal-displacement" rule. As the structural flexibility increases still further, the "equal-displacement" rule tends to become increasingly conservative. At very long periods, there is essentially no structural response to the ground motion, and the concept of an absolute displacement (independent of period or ductility) could be advanced, where the relative displacement of the center of mass of the structure is equal to the absolute peak ground displacement.

Different codes rely on startlingly different relationships between elastic and inelastic displacements. In the United States, the relationships between force reduction factors (R_w), seismic load factors (1.4), and design displacements $(\Delta_y \times \frac{3}{8} R_w)$ included in the Uniform Building Code[1] approach can be interpreted as implying that inelastic displacements are expected to be about 50 percent of equivalent elastic displacements. At the other end of the spectrum, many Central and South American codes rely on the equal energy approach, with a base ductility of about 4. This implies inelastic displacement about 150 percent of equivalent elastic displacements. Although we accept displacement capacity to be more fundamental to seismic response than strength, it appears that different groups of experts cannot agree within a factor of 3 as to what these should be, from a given elastic acceleration spectrum.

Although these points have been recognized and partially considered in codes that define inelastic spectra with variable ratios between elastic and ductile coordinates, such as NZS 4203,[2] confusion is still almost universal.

3. The elastic acceleration approach places excessive emphasis on elastic stiffness characteristics of the structure and its elements. As discussed subsequently, we are less careful than we should be in determining these characteristics. The question remains as to whether better alternatives might be

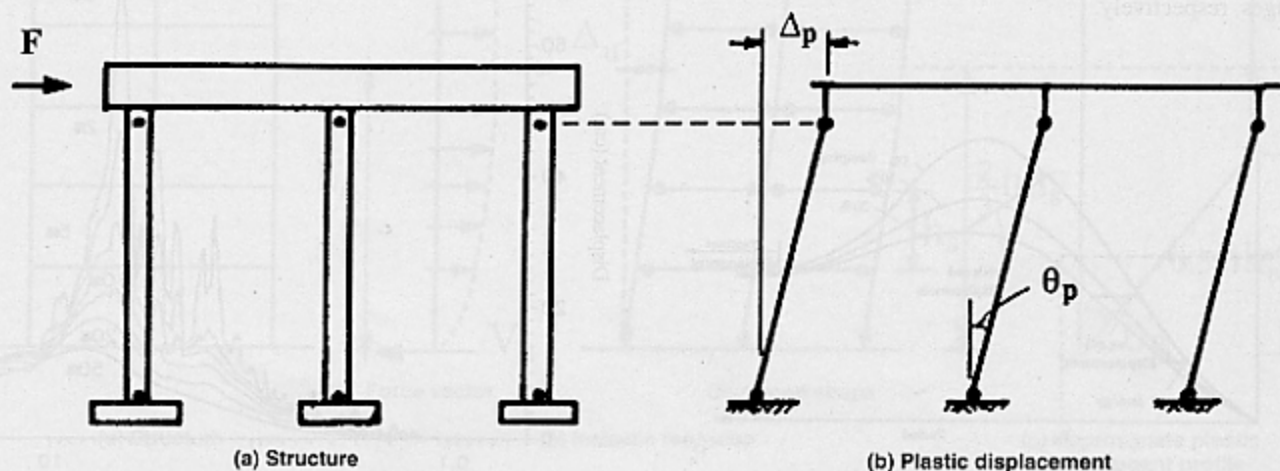(a) Structure

(b) Plastic displacement

Fig. 2 — Bridge bent example

considered. I believe that they can, and that a more consistent approach may be achieved by complete inversion of the design process.

It should be noted that although many researchers have discussed displacement-based design, the processes described are in fact still strength-based. Moehle,[3] for example, discusses the relative merits of ductility-based and displacement-based design, but in his comparison, the starting point is still a given strength and stiffness (and hence period) with the difference being whether displacements or ductilities are checked. As acknowledged by Moehle, when properly carried out, the two approaches are directly equivalent. In the approach described here, strength and stiffness are the end product of the design process, rather than the starting point.

The procedure is initially illustrated by reference to the simple multicolumn bridge pier shown in Fig. 2, for which displacement-based design is comparatively straight forward. A set of elastic displacement response spectra for different levels of equivalent viscous damping are required (Fig. 3a). These can be generated in much the same way as elastic acceleration response spectra. The shape, with a resonant region as shown in Fig. 3a, with reducing displacement response at large periods is characteristic of displacement spectra (Fig. 3b).

The sequence of operations involves the following steps:

1. An initial estimate for the structural yield displacement Δ_y is made. Since final results are not particularly sensitive to the value assumed, Δ_y could be based on a typical drift angle of about $\theta_y = 0.005$. For a building design, a lower value would generally be appropriate.

2. The limit to acceptable plastic rotation of critical hinges is determined. This will be a function of the importance of the structure, the section geometry, and an acceptable level of transverse reinforcement. For the bridge bent of Fig. 2, hinges are assumed to develop at top and bottom of the columns.

3. The maximum acceptable structural plastic displacement Δ_p at the center of seismic force, corresponding to the plastic rotation limit of the most critical hinge, is found from considerations of mechanisms deformation.

4. A first estimate of total acceptable structural displacement is thus $\Delta_m = \Delta_y + \Delta_p$.

5. An estimate of effective structural damping is made, based on the implied displacement ductility level $\mu_\Delta = \Delta_m/\Delta_y$ (Fig. 4), where curves are given based on typical hysteresis characteristics for structures with beam hinges or column hinges, respectively.
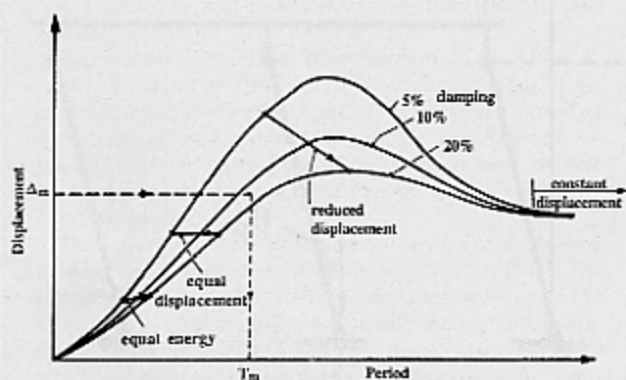
6. With reference to elastic response spectra for the site (e.g., Fig. 3a), the effective response period can now be estimated. The effective stiffness of the substitute structure at maximum response can thus be found from:

$$T = 2\pi \sqrt{\frac{M}{K}}$$

$$\text{as } K = \frac{4\pi^2}{T^2} M \qquad \text{Eq. 1}$$

and the required structure yield strength, or base shear capacity is:
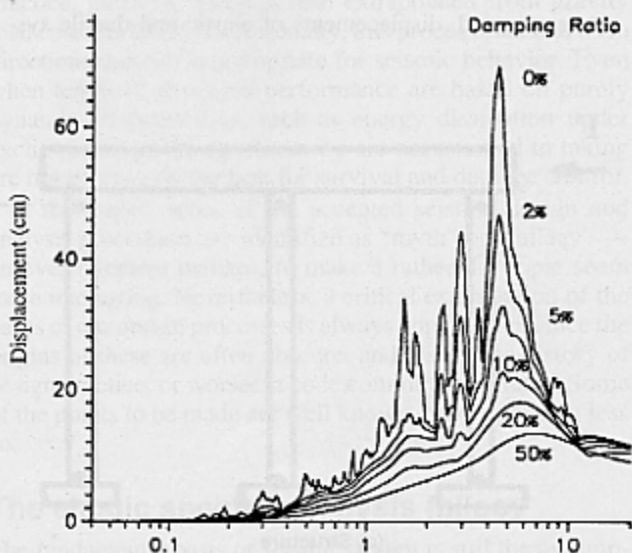
$$V_B = K\Delta_m \qquad \text{Eq. 2}$$

7. With a knowledge of the required shear capacity, the member sizes can now be proportioned, and an initial estimate of reinforcement made. The elastic stiffness can thus be calculated, and a refined estimate of the yield displacement obtained.

8. The total displacement, structure ductility, and hence effective structural damping are thus revised, and steps 4 - 7 repeated until a stable and satisfactory solution is obtained. Individual flexural strength requirements for potential plastic hinges are finalized, based on statics.

The approach outlined has considerable flexibility, since plastic hinge rotational capacity can be related to transverse detailing (or vice versa), and the design is not dictated by somewhat arbitrary decisions about force-reduction factors.

It would appear that this method of displacement-based design could also be applied to multi-story frame or shear wall buildings, provided some additional assumptions are made. The two critical pieces of information required are (1) the relationship between maximum interstory drift and structural displacement at the height of the center of seismic force; and (2) the shape of the lateral force vector to be applied. These aspects are illustrated in Fig. 5 for an idealized frame of n stories each of equal height h. The center of seismic force is approximately at 2/3 of the building height, and the maximum displacement at this height can thus be expressed as:

$$\Delta_u = \Delta_y + \frac{2}{3} nh\, \theta_p K \qquad \text{Eq. 3}$$

(a) Design displacement spectra

Fig. 3 a and b — Displacement response spectra for design.

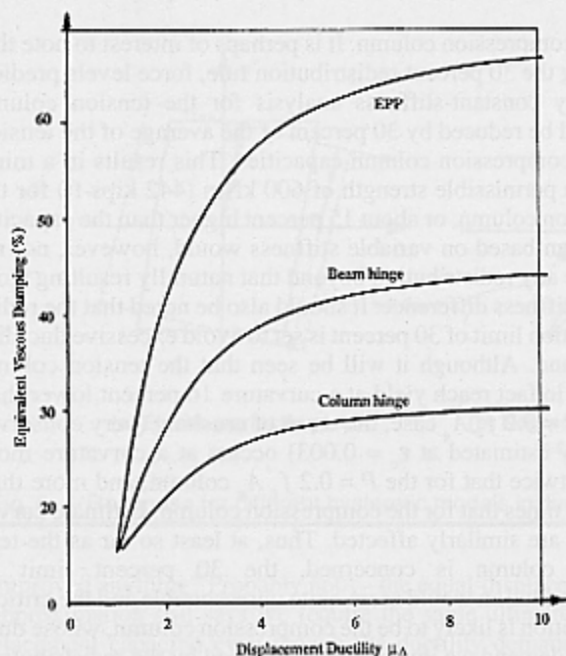(b) Actual displacement spectra (Brawley 1979, 315°)

Fig. 4 — Equivalent viscous damping vs. displacement ductility level.

where $K \leq 1$ defines the nonuniformity of drift up the building height and θ_p is the maximum acceptable rotation of the plastic hinges and hence the maximum story drift angle. On the basis of inelastic analyses of frames, Paulay and Priestley[4] recommend that the distribution of drift should be assumed to be that shown in Fig. 5c, where the drift in the lower half of the stories is equal to twice the average drift at the roof level. Assuming further that this distribution can also be applied to the plastic component of drift, Eq. 3 can be simplified to:

$$\Delta_u = \Delta_y + 0.5\ nh\ \theta_p \qquad \text{Eq. 4}$$

That is, $K = 0.75$.

It is suggested that improved estimates of plastic drift would be obtained by elastic analysis of a substitute struc-

ture,[5] where stiffness of members containing hinges is reduced in proportion to their expected ductility. Hence, if beam hinges are expected to have rotational ductilities of $\mu_\theta = 7$ (which might correspond to a structure displacement ductility of $\mu_\Delta = 4$) then the appropriate stiffness for the beams in the elastic analysis would be: $K_u = K_e/\mu_\theta = 0.14K_e$. The adequacy of the design can thus be checked by a lateral elastic analysis of the substitute structure.

If the inelastic displaced shape can be approximated by Fig. 5c, it follows that the vector of lateral inertial forces to be applied to the structure should also take the same shape.

The displacement-based design approach appears attractive in principle, but will need to be checked by specific examples covering a wide range of structural types and periods.

The refined analysis myth

It was noted at the start of this paper that structural analyses for design purposes have become more sophisticated in recent years, with the consequences that the analysis and design functions are frequently separated and carried out by different people. The reason for the increased sophistication in the analysis is principally related to the availability of powerful computers rather than a perceived inadequacy of earlier and simpler analysis techniques.

Although 3-D modal analysis is undoubtedly useful in structures with unusual or irregular geometry, it is doubtful if it produces better results than those obtained from simpler methods — say lateral analysis based on an assumed lateral force distribution. The myth here, then, is that refinement of the analysis produces more "accurate" results. It is appropriate to consider the refinement of the analytical process in light of the approximations still remaining.

Elastic modal analysis essentially relies on the equal-displacement approximation, since it is not feasible to use different force-reduction factors associated with different modes of elastic response. As noted previously in reference to Fig. 1, this is appropriate for a comparatively narrow band of periods.

Deflection profiles from elastic modal analyses tend to underestimate drift levels in the lower stories of a building. As

(a) Structure

(b) Inelastic response
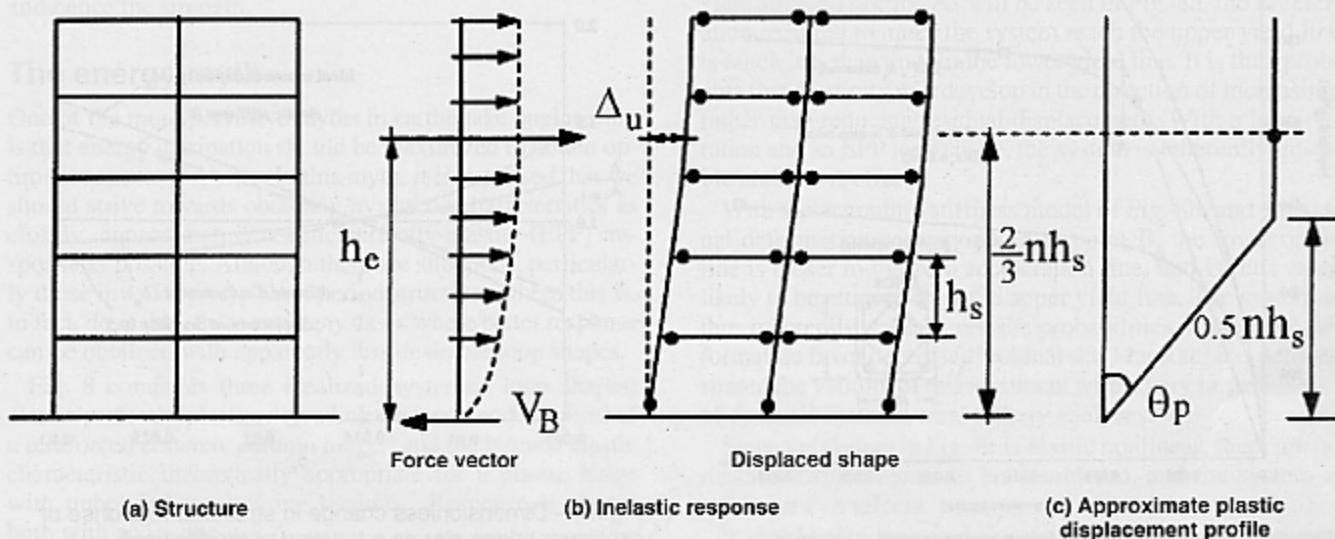
(c) Approximate plastic displacement profile

Fig. 5 — Maximum response of a frame building.

noted earlier, it is felt that agreement could be improved by use of a substitute structure approach.

Elastic analyses are generally based on approximations of member stiffness that should be considered gross, even in the elastic range. As an example of this, let us consider that the columns of the lowest story of the frame shown in Fig. 5a are reinforced concrete 600 x 600 mm (24 x 24 in.) columns, reinforced with 8 – D28 (28 mm [1.1 in.] diameter) bars with yield strength of 455 MPa (70 ksi). Concrete strength is f'_c = 31 MPa (4.5 ksi). It will be normal in the analysis to assume that all three columns at the lowest level have the same stiffness, though it is possible the central column might be allocated a slightly higher stiffness because of increased axial load. It would seem to be impossible to allocate different stiffnesses to the two outer columns when multi-modal response is considered.

For the sake of argument, it is assumed that the outer columns carry gravity loads of $P_{(D+L)} = 0.2 f'_c A_g$, and that seismic axial forces of $P_E = \pm 0.2 f'_c A_g$ can be expected. Fig. 6 shows moment-curvature relationships for the compression and tension columns, including the effects of varying axial force as the lateral base shear increases, and the relationship assuming a constant axial force of $P = 0.2 f'_c A_g$. Onset of first yield of tension reinforcement and attainment of a compression strain of $\varepsilon_c = 0.003$ are also noted.

Taking the yield condition to correspond to a curvature of about 0.0084/m (0.003/ft) — which is the yield point for a bilinear approximation to the $P = 0.2 f'_c A_g$ curve — we find that the effective stiffness of the compression column is more than twice that of the tension column. As a consequence, the distribution of elastic forces in the lower stories is likely to be substantially different from that predicted by the "refined" elastic analysis. Note, however, that it would be comparatively straight forward to consider these effects in an equivalent lateral force approach.

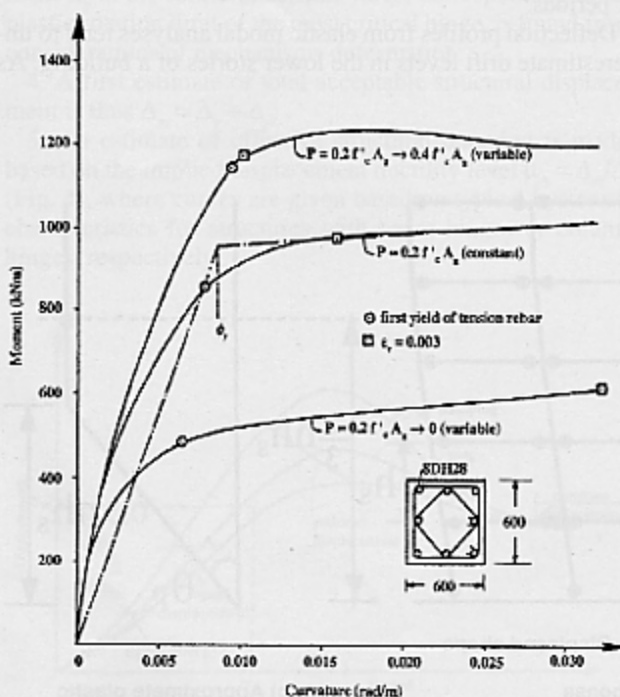In some design codes (e.g., ref. 2), it is permissible to redistribute up to 30 percent of moment from a tension column to a compression column. It is perhaps of interest to note that using the 30 percent redistribution rule, force levels predicted by constant-stiffness analysis for the tension column could be reduced by 30 percent of the average of the tension and compression column capacities. This results in a minimum permissible strength of 600 kNm (442 kips-ft) for the tension column, or about 15 percent higher than the capacity. Design based on variable stiffness would, however, not require any redistribution beyond that naturally resulting from the stiffness difference. It should also be noted that the redistribution limit of 30 percent is set to avoid excessive ductility demand. Although it will be seen that the tension column does in fact reach yield at a curvature 16 percent lower than the $P = 0.2 f'_c A_g$ case, the onset of crushing (very conservatively estimated at $\varepsilon_c = 0.003$) occurs at a curvature more than twice that for the $P = 0.2 f'_c A_g$ column, and more than three times that for the compression column. Ultimate curvatures are similarly affected. Thus, at least so far as the tension column is concerned, the 30 percent limit to redistribution would seem quite unreasonable and the critical condition is likely to be the compression column, whose ductility demand we imagine to be reduced by the redistribution process (herein appears another fallacy).

The strength/ductility trade-off fallacy

The current design emphasis on force-based design, together with the general adoption of the equal displacement approximation leads us to the natural conclusion that required strength, S, and displacement ductility demand, μ_Δ, are related by the expression:

$$S \cdot \mu_\Delta = constant \qquad \text{Eq. 5}$$

for a given structure or critical element, with the usual caveat that for short-period structures, the approximation may be inappropriate.

The fallacy of this observation becomes obvious when we invert the logic. Consider that we are designing a structural element, say a bridge column, and we decide that, as designed, the ductility capacity (and hence in a more basic sense the displacement capacity) is inadequate. As a consequence, we decide to increase the strength to reduce the ductility demand. We do this by increasing the longitudinal steel ratio, and keeping the section size constant. Have we really

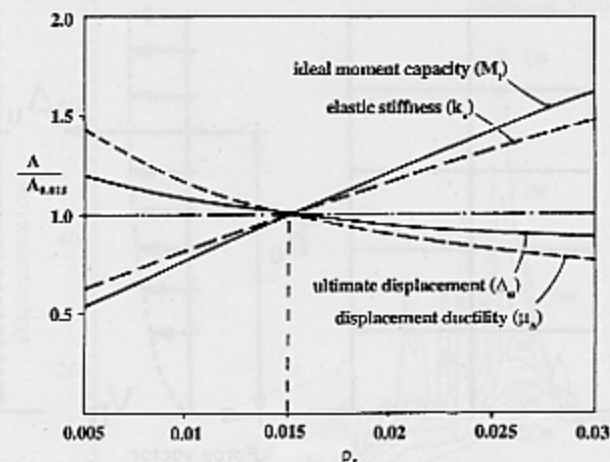
Fig. 6 — Influence of axial force on moment curvature relationship for rectangular column (1 kNm = 8.85 kip-in.).

Fig. 7 — Dimensionless change in structural response of cantilever bridge pier as a function of longitudinal reinforcement ratio.

(a) Elastic-perfectly plastic   **(b) Degrading stiffness (Takeda)**   **(c) Nonlinear elastic**
                                     [r.c. column hinge]   **[unbonded prestressing]**
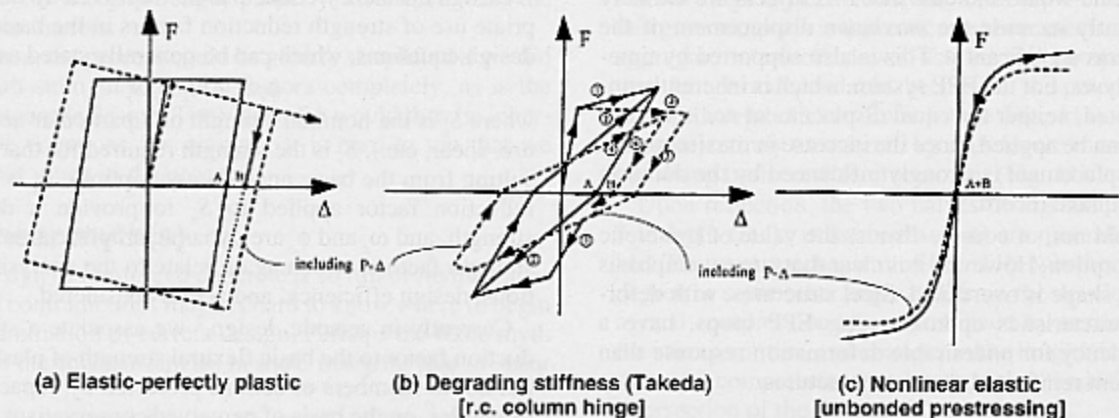
Fig. 8 — Response for different hysteretic models including P-Δ effect.

improved anything? Probably not — the equal-displacement approximation still says we require the same ultimate displacement capacity, even though the ductility demand has apparently been reduced, and it is certainly not clear that increasing the longitudinal reinforcement ratio has increased the ultimate displacement.

In fact, quite the opposite is more likely. Fig. 7 plots the results of varying longitudinal reinforcement ratio for circular columns with axial load ratio 0.1 $f_c' A_g$. Results for moment capacity, stiffness, ductility capacity and ultimate displacement are expressed in dimensionless form by reference to the value pertaining to a "standard" longitudinal ratio of ρ_l = 0.0015. It will be seen that as ρ_l is increased, the ultimate moment capacity (M_i) increases almost proportionately, but the ultimate displacement capacity reduces somewhat (by about 10 percent at ρ_l = 0.03), and the ductility capacity reduces even more. Of course, the argument also contains a fallacy, since the stiffness (K_e) has increased almost as much as the strength has, and thus the period will have changed. Nevertheless, it is unlikely that this stiffness variation will have been included in the original calculations, and the required increase in ultimate displacement has clearly not been achieved. It can even be reasonably argued that if we were worried about the ductility or displacement capacity, we would have been better off reducing the reinforcement ratio, and hence the strength.

The energy myth

One of the more pervasive myths in earthquake engineering is that energy dissipation should be maximized to obtain optimum seismic response. In this myth, it is supposed that we should strive towards obtaining hysteretic characteristics as closely approximating elastic/perfectly-plastic (EPP) response as possible. Although there are situations, particularly those involving very short period structures where this is, in fact, desirable, there are many cases where better response can be obtained with apparently less desirable loop shapes.

Fig. 8 compares three idealized hysteresis loop shapes: elastic/perfectly-plastic, degrading stiffness model typical of a reinforced concrete column hinge,[6] and the bilinear elastic characteristic theoretically appropriate for a plastic hinge with unbonded prestressing tendons.[7] Response is shown both with and without P-Δ.

Consider first the response without P-Δ effects. If response

is in the "equal-displacement" domain of Fig. 1, the peak response displacements of the three systems are likely to be very similar if each has the same initial stiffness. In actual fact, the displacements of the EPP system will on average be a little smaller than the other two, but the difference will not be great. Let us assume that a maximum drift of 2 percent, corresponding to a displacement ductility factor of μ_Δ = 5 is obtained. After the earthquake, the residual drift of the EPP system could be as high as 1.6 percent, that of the column hinge about 0.9 percent, and the bilinear system of Fig. 8c will return to its initial position. Which system has exhibited better response? It is at least arguable that residual displacements are ultimately more important than maximum displacements, given the difficulty of straightening a bent building after an earthquake.

Consider now the influence of hysteresis loop shape on response as affected by P-Δ effects. MacRae has rather convincingly shown[8] that the tendency for instability under P-Δ is strongly related to the shape of the loop. In Fig. 8, the influence of P-Δ moments on the inelastic response shape is shown by dashed lines. With the EPP loop of Fig. 8a, response at a given instant of the seismic response has resulted in a residual deformation corresponding to point B. The structure will oscillate with the elastic stiffness about this point until response acceleration sufficient to develop the yield strength occurs. As will be seen in Fig. 8a, the acceleration required to make the system reach the upper yield line is much less than that for the lower yield line. It is thus probable that plasticity will develop in the direction of increasing, rather than reducing residual displacement. With a long duration and an EPP loop shape, the system is inherently unstable under P-Δ effects.

With the degrading stiffness model of Fig. 8b, and a residual deformation corresponding to point B, the lower yield line is closer to the zero acceleration line, and is thus more likely to be attained than the upper yield line. The system is thus inherently stable, since the probabilities of inelastic deformation favor decreased residual set. MacRae[8] has demonstrated the validity of this argument with a very large number of dynamic inelastic time-history analyses.

Since the system in Fig. 8c is elastic nonlinear, there are no residual displacements to be considered, and the system is stable for P-Δ effects.

It should also be noted that for longer period structures, where P-Δ effects are likely to be significant, the equal dis-

placement rule would indicate that P-Δ effects are unlikely to significantly increase the *maximum* displacement of the stable systems significantly. This is also supported by time-history analyses. For the EPP system, which is inherently unstable as noted, neither the equal displacement nor equal energy rules can be applied, since the increase in maximum and residual displacement is strongly influenced by the duration of the earthquake record.

One should not, of course, dismiss the value of hysteretic energy absorption. However, it is clear that current emphasis on the loop shape is overstated. Steel structures, with deformation characteristics approximating EPP loops, have a greater tendency for undesirable deformation response than the equivalent reinforced concrete structures.

The distribution of flexural reinforcement fallacy

The discussions to this point have largely related to analysis issues. However, it is clear that many aspects of design and detailing could also bear critical review. The remainder of this paper will examine a few issues specifically related to reinforced concrete design, though more could be identified, both with reinforced concrete and with other materials.

One of the most pervasive fallacies relates to the way we distribute reinforcement in beams of ductile moment-resisting frames. By use of moment redistribution, we frequently end up with positive and negative moment demands that are equal, or nearly so. We then place reinforcement in two bands, as close to top and bottom respectively of the beam, as shown in Fig. 9a, in the mistaken view that this provides the most efficient distribution. Wong et al.[9] have shown that essentially the same moment capacity can be achieved by distributing the total amount of reinforcement down the sides of the beam (Fig. 9b). Fig. 10 compares the flexural strength of the alternative distributions of Fig. 9, as a function of the mechanical reinforcement ratio $\rho f_y / f_c'$, where $\rho = A_s/bh$ is the total reinforcement ratio. The strength differences between the distributions are insignificant.

There are, however, good reasons for adopting the distribution of Fig. 9b. Congestion at beam-column joints of two-way frame construction is considerably eased, a greater proportion of the joint shear force can be associated with the diagonal concrete strut, thus reducing the demand for joint shear reinforcement, and flexural overstrength resulting from strain-hardening of reinforcement is reduced. This latter point could be taken advantage of by reducing the overstrength ratio used to develop member forces in the capacity design process, thus resulting in design efficiencies.
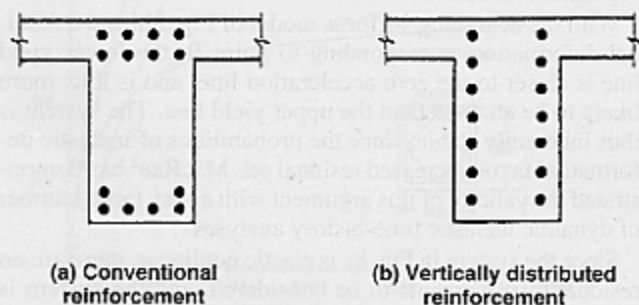
Design efficiency could also be improved by more appropriate use of strength reduction factors in the basic capacity design equations, which can be generally stated as:

$$\phi_s S_n \geq \omega_s \phi_o S_r \qquad \text{Eq. 6}$$

where S_n is the nominal strength of a particular action (flexure, shear, etc.), S_r is the strength required for that action resulting from the basic analysis assumptions, ϕ_s is a strength reduction factor applied to S_n to provide a dependable strength, and ω_s and ϕ_o are dynamic amplification and overstrength factors, which again relate to the analysis assumptions, design efficiency, and action considered.

Currently in seismic design,[2] we associate a strength reduction factor to the basic flexural strength of plastic hinges, but not to members or actions protected by capacity design principles, on the basis of perceived conservatism in the values of ω_s and ϕ_o currently specified.

It would appear that the logic may have thus been inverted. It is clear that we do not need a flexural strength reduction factor for plastic hinges, since small variations in strength from the specified value will only result in small variations in ductility demand. As noted in relation to Fig. 7, increasing the reinforcement content, which is the end result of application of a flexural strength reduction factor, may not improve overall safety.

However, if we wish to totally proscribe nonductile inelastic deformation (e.g., shear) we need a high degree of assurity that the dependable strength of that mode cannot be exceeded. If we believe strength reduction factors need to be associated with that action (e.g., shear) as a result of possible nonconservatism of design equations, or possible material under-strength, then these factors should be utilized in the capacity design process. If the product $\omega_s \phi_o$ is felt to be such that no strength reduction factor is needed, this implies that $\omega_s \phi_o$ is too high, and should be reduced.

This may seem to be a matter of semantics, since the end result would probably be little different in current design practice. However, if design is to be permitted in accordance with more advanced analytical processes (e.g., time-history analysis to determine expected dynamic influences once strength of plastic hinges has been determined) then values for ω_s and ϕ_o might become determined by the results of the analysis process. In this case, it would be better to have the

(a) Conventional reinforcement   (b) Vertically distributed reinforcement

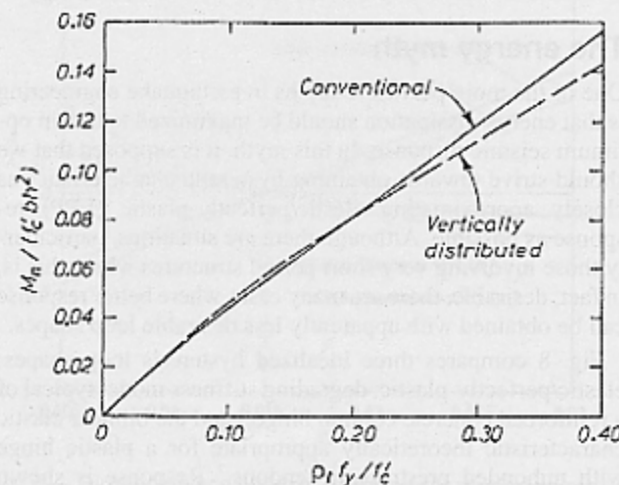Fig. 9 — Arrangements of longitudinal reinforcement in beams.

Fig. 10 — Dimensionless flexural strength for beams with conventional and vertically distributed reinforcement.

variability of design strength properly associated with the correct parameters.

Of course, the process could be greatly simplified if we did away with strength reduction factors completely, as is the case in Japanese design. The value of ϕ would thus be inherent in the equations for strength. It is hard to see that we would lose much in the process.

The shear myth(s)

Shear design of reinforced concrete is so full of myths, fallacies, and contradictions that it is hard to know where to begin in an examination of current design. Perhaps the basic myth central to our inconsistencies in shear design is that of shear itself.

It has been argued that we tie ourselves into intellectual knots by separating flexure and shear, and considering them essentially independent entities. Compression field theory as developed by Collins et al.[10] is an attempt to integrate the actions. Similar attempts have been made elsewhere.[11] The fact remains, however, that it is very convenient to separate the flexural and shear actions. The more fundamental approaches are not only inconvenient from a design viewpoint, but also do not produce notably better agreement with experimental results — particularly when shear strength of ductile linear members characteristic of framed structures is considered.

Our understanding of the mechanisms of shear transfer in plastic hinge regions seems particularly weak. If we consider

(a) Conditions at high ductility

(b) The 45 degree truss mechanism

Fig. 11 — Shear transfer in beam plastic hinges.

a beam hinge adjacent to a column, as depicted in Fig. 11, the design assumption is that concrete shear mechanisms such as aggregate interlock, dowel action, and compression shear transfer are undependable, because of the potential presence of a full depth flexural crack (see Fig. 11a). Hence shear must be entirely transferred by a 45 degree truss mechanism, involving transverse reinforcement (Fig. 11b).

Upon reflection, the two halves of this assumption (wide full depth crack; truss mechanism) are mutually incompatible. The truss mechanism of Fig. 11b, whether based on 45 degree or some other angle, relies on the development of diagonal compression struts, stabilized by vertical tension in the stirrups or ties, and changes in the longitudinal beam tension and compression resultants at the "nodes" (formed by intersection of the tie and diagonal forces).

If a vertical section is taken at any position, the vertical component of the diagonal compression forces *continuous across the section* must equal the shear force transferred by the truss mechanism. At the critical section at the column face, there is a full depth flexural crack formed by inelastic action in the two opposite directions of response. As a consequence, there can be no diagonal compression forces crossing this section, and the shear carried by the truss mechanism must also be zero.

This apparent, and obvious, dilemma has been "rationalized" by the assertion that the intersection of diagonal cracks (Fig. 11a) allows the cracks to dilate and close in the middle section of the beam, thus permitting diagonal compression struts to develop. There are two concerns to adopting this solution:

First, if diagonal compression can develop across this crack, then perhaps the assumption that $V_c = 0$ should not be made since at least part of the rationale for discarding V_c has been eliminated. Second, the assumption of diagonal compression implies a reduction to apparent flexural strength of the section.

Consider equilibrium of the stress resultants in Fig. 11b. The resultant of the diagonal compression forces may be assumed to act at midheight. If a 45 degree truss is assumed, its vertical and horizontal components must both be equal to V as shown. The flexural compression force acts at or near the center of compression reinforcement.

Without the diagonal compression, the moment capacity is:

$$M_i = T(d - d') \qquad \text{Eq. 7}$$

If the diagonal compression force is included, the moment capacity reduces to:

$$M_{iv} = T(d - d') - V\frac{(d - d')}{2} = \left[T - \frac{V}{2} \right](d - d') \quad \text{Eq. 8}$$

Neglecting the dead load shear, assuming equal moment capacity at opposite ends of the beam, and a beam length of $x(d - d')$:

$$V = \frac{2M_{iv}}{x(d - d')} \qquad \text{Eq. 9}$$

hence $M_{iv} = T(d - d') - \dfrac{M_{iv}}{x}$ or $M_{iv} = \left(\dfrac{1}{1 + \dfrac{1}{x}} \right) M_i$ Eq. 10

In a deep beam with (say) $x = 4$, this implies a 20 percent reduction in moment capacity, which is not supported by experimental results. Even with more slender beams (say $x = 10$) the moment reduction should be evident. Note that when
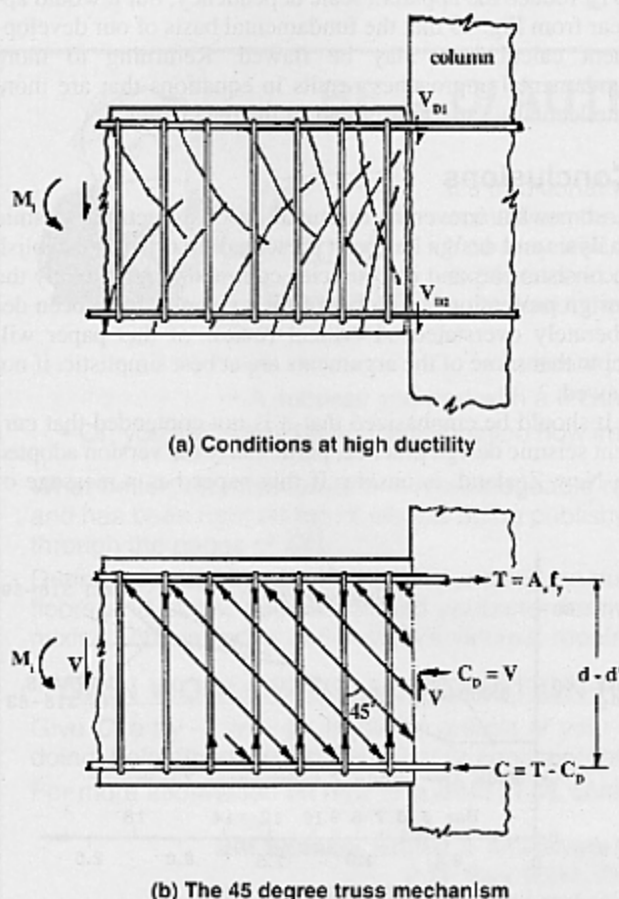
dead-load shear is added, negative moment capacity should be further eroded, if the truss mechanism were correct.

Since this behavior is not apparent in experimental results, it would appear that reliance on alternative mechanisms is needed. If a full depth crack can develop, it is difficult to escape the conclusion that all shear must be carried by dowel action. If this is the case, the primary shear transfer function of transverse reinforcement in the plastic hinge region must be to reduce the unsupported length over which dowel action of the longitudinal reinforcement occurs, hence increasing the shear that can be transferred.

Rational models for the amount of shear reinforcement required can be developed, which are very different from those resulting from the truss model. Critical aspects include diameter of longitudinal reinforcement (the bigger the better), and location of the first stirrup from the column face — which should be as small as possible.

As mentioned, there are many other inconsistencies in shear design. A particularly troublesome one is the way in which we treat the enhancement of shear strength by axial compression. Our codes indicate that the degree of enhancement depends on whether the structural element is a beam, a column, or a wall, and whether the axial compression comes from applied gravity forces, or from prestress. Differences in influence of more than 100 percent are possible. Since the differences appear to result from semantic definitions, a degree of skepticism is appropriate.

Development of reinforcement fallacy

The final example considered is the way in which we consider development of reinforcement. Development is a catch-all term used to describe embedment or anchorage, splicing, and flexural bond. The same equations, with modifiers in some cases, are used to describe the three situations. Although this is convenient for design, it is unlikely to be realistic in practice, considering the wide number of possible situations occurring, as illustrated for a bridge column in Fig. 12. A number of rather different conditions exist depending on location, whether or not confinement is present, and whether or not the splitting cracks assumed in the basic development length equation can actually develop.

The fundamental basis of the basic development length equation can also be questioned. Until recently in the ACI code, this length was given by:

$$l_{db} = 0.019 A_b f_y / \sqrt{f_c'} \qquad \text{Eq. 11}$$

Where A_b is in mm^2 and f_y and f_c' are in MPa. Modifiers are included to represent the influence of cover, bar spacing, location in the concrete pour, confinement, etc. Fig. 13 shows the ratio of basic development length to bar diameter d_b for U.S. bar sizes, for $f_y = 414$ MPa (60 ksi), and $f_c' = 22.5$ MPa (3.3 ksi). In Fig. 13, the equation definitions are in American Standard units. It will be seen that the basic development length increases in terms of number of bar diameters as the diameter increases. In fact, considering the range of bar sizes used in the United States (#3 - #18, .375 - 2.26 in. [10 - 57 mm] dia.), Eq. 11 implies the *dimensionless* development length increases by a factor of six, or the *actual* development length increases by a factor of 36. Tests on scale models do not support a scale dependency of this nature.

To some extent, of course, the modification factors applied to l_{db} reduce the apparent scale dependency, but it would appear from Fig. 13 that the fundamental basis of our development calculations may be flawed. Returning to more fundamental approaches results in equations that are more intellectually satisfying,[12] and fit the data better.

Conclusions

A somewhat irreverent examination of aspects of seismic analysis and design has been presented. In order to establish inconsistencies and inaccuracies commonly accepted by the design profession, a number of the examples have been deliberately overstated. A critical review of this paper will show that some of the arguments are at best simplistic, if not flawed.

It should be emphasized that it is not contended that current seismic design practice, particularly the version adopted in New Zealand, is unsafe. If this paper has a message or
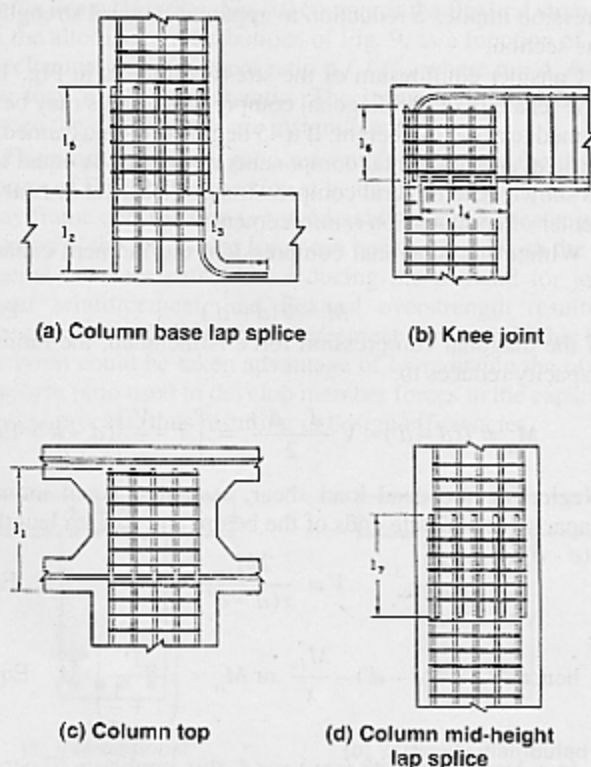

(a) Column base lap splice   (b) Knee joint

(c) Column top   (d) Column mid-height lap splice
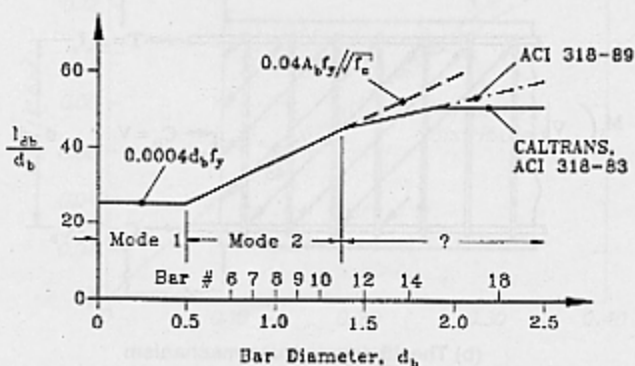
Fig. 12 — Anchorage and splicing of reinforcement.

Fig. 13 — Basic development length, for deformed bars in tension ($f_c' = 3.25$ ksi; $f_y = 60$ ksi). f_c

conclusion, it is simply the following cautionary note related to the tendency for increased complexity in analysis:

Given the wide range, and occasional gross nature of the assumptions and approximations inherent in seismic design, we might be better keeping the design and analysis processes simple enough so that we still understand what we are doing.

A second, and more serious (perhaps!) point brought out at the start of this paper related to displacement-based design: If we accept that displacements are more important than forces, it is time we started basing our designs on displacement, rather than acceleration spectra.

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M.J. Nigel Priestley is a professor of structural engineering at the University of California, San Diego. He was educated in New Zealand, and spent 10 years on the faculty of the Department of Civil Engineering, University of Canterbury, before moving to the United States in 1986. His research interests include seismic response of bridges and buildings.