REVIEW ARTICLE

The Philosophy and Application of Capacity Design

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A deterministic seismic design strategy, developed in New Zealand where it has been used for some 15 years, applicable particularly to reinforced concrete structures, is presented. Its application to ductile frames, buildings with structural walls and frame-wall systems is briefly summarized.

INTRODUCTION

It is established that structures can be designed and constructed so as to satisfy various seismic performance criteria, most importantly that of preventing collapse during an exceptionally large earthquake. For most engineers seismic design is synonymous with the complex analysis of elastic or inelastic dynamic response to random ground excitations. This presentation, reflecting the views of a structural designer, attempts to contrast analysis with design strategies that are suited to overcome difficulties that stem from inevitable uncertainties in the prediction of ground motions. A deterministic design philosophy is advocated whereby the designer can, within certain limits, choose the seismic response of a structure that is safe, rational, predictable and achievable in construction. The designer may thus enhance desirable, and suppress undesirable features of structural behavior. In this, the vital role of the quality of the design and detailing of critical regions of structural systems is emphasized because this alone can assure the very desirable characteristic of seismic response; tolerance with respect to the inevitable crudeness of predicting earthquake imposed displacements.

BASIC AIMS IN SEISMIC DESIGN

It is now well established that for buildings of normal usage it is not economical to provide strength sufficient to prevent structural damage during very large earthquakes that are likely to occur only once in a few hundred years.

Significant damage during such exceptional events, perhaps beyond repair, must thus be expected. However, design and construction must ensure that collapse resulting in loss of life will not occur. Therefore, the designer must concentrate on structural qualities which will ensure that for the expected duration of an earthquake, relatively large displacements can be accommodated without significant loss of lateral force resistance, and that the integrity of the structure to support gravity loads is maintained.

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The most important property associated with this survival limit state is ductility, that is, tolerance for large inelastic deformations without significant loss of resistance. The exploitation of this property is a relatively recent feature in the evolution of structural engineering.

The ductility capacity of a reinforced concrete building system is conveniently quantified by the ratio of the lateral displacement at a suitable elevation, such as the roof, to the yield displacement at the same level; that is, the displacement ductility factor. Because the transition from elastic to inelastic response is non-linear, acceptable simplifications need to be made, particularly with respect to the definition of the displacement at first yield. An example is shown in Figure 1. While such global ductility is indicative of inelastic response of the entire system, the designer must pay even more attention to ductility demands that arise in critical potential plastic regions of the structure.

Relatively frequent earthquakes inducing comparatively minor intensity of ground shaking should not interfere with functionality. This means that no damage to the building and its content needing repair should occur. To achieve this aim, displacements must be limited and resistance must be provided while the structure remains essentially elastic. The controlling property for this serviceability limit state is stiffness. Because the principles of the

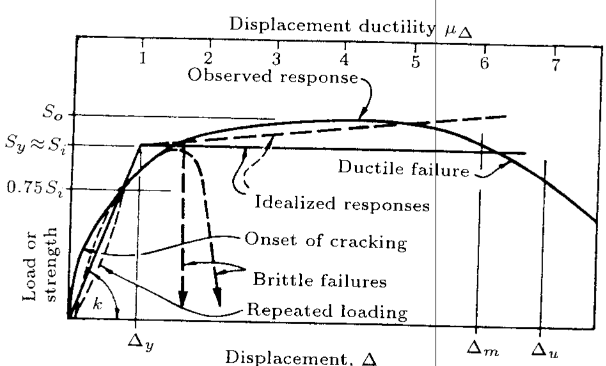

Figure 1. Typical force-displacement relationships for reinforced concrete elements and the definitions of stiffness and displacement ductility.

analysis of elastic systems are well established, no further attention is given here to this feature. Elastic response may utilize a major fraction of the ideal (nominal) strength, S_i, of the structure. Therefore, extensive cracking in concrete components must be expected and, this must be considered when estimating the stiffness of members [1].

A DETERMINISTIC LIMIT STATE DESIGN STRATEGY

It must be recognized that present probabilistic predictions at any site of the characteristics of ground motions generated by earthquakes are extremely crude. For common situations, the strength of the structure with respect to the resistance of lateral forces will have to be a fraction of the strength that corresponds to elastic dynamic response. Therefore, building codes are forced to make gross approximations in estimating the reduced intensity of acceptable lateral design forces. Consideration in design of elastic dynamic response characteristics, such as the contribution of the higher mode shapes of vibration to internal structural actions, are often emphasised and recommended. However, in the ultimate state inelastic response relying on ductility and ability to dissipate seismic input energy will primarily govern structural response. The crudeness resulting from predictions of ground motions and the results of elastic analyses of structural models, does not justify the often perceived accuracy aimed at, and claimed, in the design for an ultimate limit state.

These considerations suggest that gross approximations, particularly when they simplify routine design processes, are both attractive and justified. This is particularly the case when a structural system is rationally and deterministically chosen so as to be able to mobilize energy dissipating regions which, when intelligently detailed, will have ample reserve deformation capacity to accommodate significant departures from displacements associated with initial estimates for an intended displacement ductility capacity. Such an approach avoids the need for sophisticated techniques of analysis to evaluate the development of numerous possible plastic mechanisms in a complex structural system. Instead, the strategy invites the designer to "tell the structure" where plastic hinges are desirable or convenient and practicable at the ultimate limit state, and to proscribe plastification in all other regions. The strategy leads to the establishment of a suitable strength or capacity hierarchy between components of the total system. These are the conceptual ingredients of what has become known as capacity design philosophy.

In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen, suitably designed and detailed for energy dissipation under severe imposed deformations. All other elements are, then, to be protected against actions that could cause failure by providing them with strength greater than that corresponding to the maximum feasible strength in the potential plastic hinge regions.

Capacity design requires the maximum probable values of displacement-induced forces to be estimated. Such forces are associated with the development of the overstrength of potential plastic hinges. This is defined by Equation 3. For this, the strength properties of components as built, including strength enhancement of both steel and concrete under large imposed strains, need to be evaluated. The contribution to internal tension forces of all reinforcement, irrespective of its intended purpose, such as temperature or shrinkage control or to satisfy code and construction requirements, must be included wherever such bars can be subjected to earthquake-induced tensile strains. When, in critical regions, excess reinforcement is provided, this must not be interpreted as a feature resulting in increased safety. In such cases, which are not uncommon, overstrength will be developed, and as a consequence all elements intended to remain elastic must be designed for correspondingly-increased resistance.

FUNDAMENTAL STRENGTH RELATIONSHIPS

Definitions of Strength

Because strength hierarchies must be quantified, each level of strength must be clearly defined. The term strength used here expresses the resistance of the structure, or that of a member or of a particular section.

Required strength (S_u) corresponds to the demand resulting from the application of prescribed factored loads and forces at the ultimate limit state. A principal aim of structural design is to provide resistance, also termed design strength or dependable strength, to meet this demand.

Ideal strength (S_i), or nominal strength, is based on established theory of structural mechanics predicting the prescribed limit state with respect to failure of a section. It is based on final dimensions, reinforcing content and placement, and code-specified material strength properties. The relationship:

$$\phi S_i \geq S_u \ , \tag{1}$$

may be conveniently defined with a strength reduction factor $\phi \leqslant 1.0$. Alternatively material resistance factors are used.

Probable strength (S_p) takes into account the probable strength of materials utilized typically during moderate inelastic seismic displacements. It requires the knowledge of the mean strength of materials used in the construction. It may be quantified by:

$$S_p = \phi_p S_i \ , \tag{2}$$

where ϕ_p is the probable strength factor. Probable strength properties are used in estimation of the strength of existing structures, and in modelling for time-history dynamic analyses to predict the likely behavior of a structure exposed to selected earthquake records.

Overstrength (S_o) considers all possible sources that may contribute to strength exceeding the ideal strength. The most significant contribution to strength enhancement under severe ductility demand is material strength developed in excess of that assumed in or specified for routine design. This is conveniently expressed by the materials overstrength factor λ_o, so that:

$$S_o = \lambda_o S_i \ . \tag{3}$$

Because, the mean yield strength of steel is greater than the specified value f_y, and additionally due to the strain hardening under large ductility demand, the value of λ_o is typically in the range of 1.25 to 1.45. Where plastic hinges develop in heavily confined columns that are also subjected to large axial compression forces, the compression strength of the concrete and hence the flexural strength of the critical column section may also be significantly enhanced [2].

Relationships Between Strengths

In the establishment of a strength hierarchy, of prime interest to the designer is overstrength and its relation to the required strength, i.e.:

$$S_o = \lambda_o S_i \geq \lambda_o S_u/\phi \ . \tag{4}$$

The ideal strength, S_i, will be affected by rounding up in calculations; arrangements of reinforcement to suit practicality in construction; and the designer's decision to deviate from design actions derived from elastic analyses in recognition of the possible redistribution of internal structural actions during inelastic response. Therefore the final ideal strength obtained may be more or less than that derived from Equation 1.

In routine design, overstrength, for example in flexure, is conveniently quantified by the flexural overstrength factor:

$$\phi_o = \lambda_o M_i/M_E \ , \tag{5}$$

where M_E is the required strength derived by analysis only for the specified lateral earthquake design forces.

An illustrative analogy of a chain, intended to possess some ductility and shown in Figure 2, suggests that one might choose just one weak but very ductile (high quality) link, while all other links may be brittle (low quality). Clearly when the brittle links have strength, P_{is}, in excess of that of the weak link when this develops its maximum strength, $\lambda_o P_i$, the location of potential failure is known.

The chain is to be designed for an earthquake-induced tensile force of $P_u = P_E$. According to Equation 1 the ideal strength, P_i, of the link that controls the strength of the chain must be such that $P_i \geq P_E/\phi$. Having chosen the size of the ductile link, its

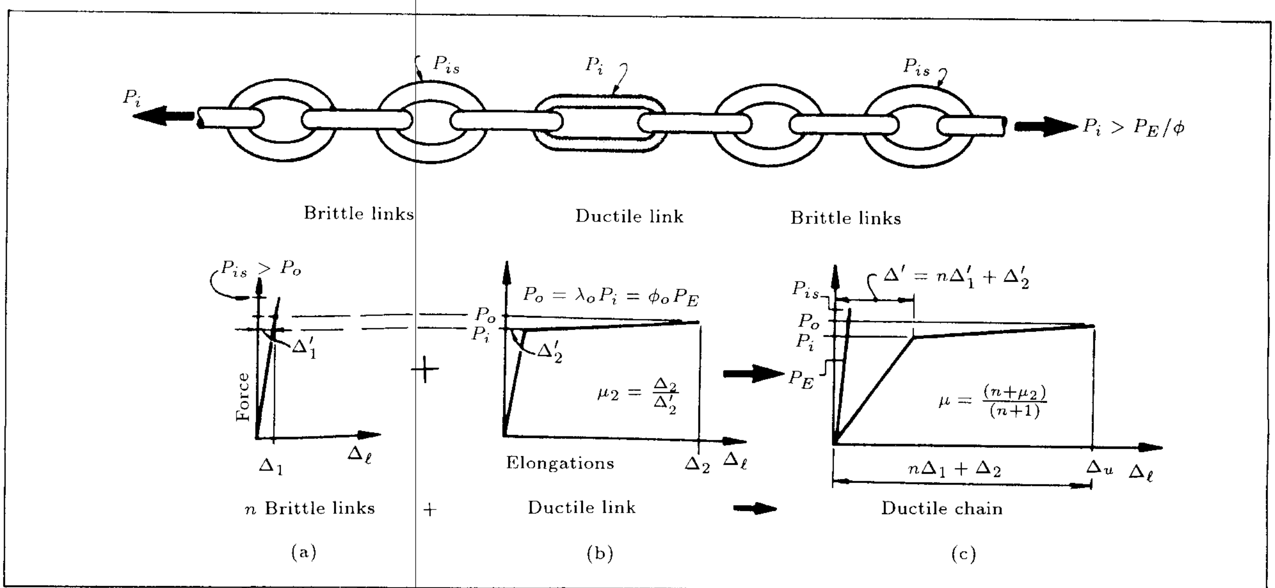

Figure 2. Strength limits in a ductile chain.

overstrength, $P_o = \lambda_o P_i = \phi_o P_E$, can be readily established. This force then determines the required strength, P_{us}, of the brittle links. Thus their ideal strength, P_{is}, must be:

$$P_{is} \geq P_{us}/\phi_s = P_o/\phi_s = \phi_o P_E/\phi_s \ . \tag{6}$$

The example in Figure 2 also draws attention to the important relationship between overall ductility capacity and corresponding local ductility demand. These are often and mistakenly taken as being identical. Figures 2a, 2b and 2c show the force-elongation relationships for a strong link, the weak ductile link and the chain as a whole, respectively. With the notation used in this figure it may be readily shown that:

$$\mu = (n + \mu_2)/(n + 1) \ , \tag{7}$$

where μ is the ductility capacity of the chain consisting of n strong links, and μ_2 is the large ductility capacity of the weak link. If for example the elongation of the ductile link is limited to 10 times its elongation at the onset of yield (i.e. $\mu_2 = 10$) and there are eight strong links in the chain shown in Figure 2, the ductility capacity of the chain is limited to only $\mu = 2$. The implication of this simple example is that in some structures the global ductility that can be relied on for reducing seismic response will be limited by the ductility capacity of the most critical ductile link in the system.

DUCTILITY DEMAND AND CAPACITY

One of the difficult tasks in the estimation of the inelastic response of a structure with given resistance in terms of lateral forces, is the gauging of the ductility demand that might be imposed on it by the design earthquake. The predicament is bypassed in building codes by the selection of elastic design spectra considered appropriate for the locality, and subsequent specification for the reductions of response in recognition of an assumed ductility capacity of the structural system.

Typical smoothed spectra cannot guarantee that transient ductility demands during a large earthquake will not exceed the assumed ductility capacity. While such design spectra, widely utilized in countries affected by seismicity [3], may be used with confidence when determining the required strength with respect to lateral design forces, S_u, they should not be considered as being reliable predictions of maximum ductility demands. The inherent crudeness of the recommendations of current seismic design procedures for the level of ductility capacities of different structural systems, to be used in establishing the intensity of lateral design forces, must thus be recognized. Fortunately with the implementation of thoughtful detailing of potential plastic regions of the structure, in most situations it is relatively easy to provide reserves in ductility capacities without incurring significant economic penalties. Moreover, in a well detailed structure the development of intended maximum ductility will always generate resistance somewhat greater than the required strength. By providing reserve ductility capacity, structures can be constructed that are very tolerant with respect to our inability to predict ductility demands more accurately. This concept is an essential ingredient of the seismic design strategy advocated in this presentation.

An example for the possibility to provide reserve ductility capacity is illustrated in Figure 3. This shows the hysteretic response of an isolated reinforced concrete interior beam-column assembly of a typical two-way ductile multistory frame. The near full size test specimens [4], consisting of two beams at right angles, monolythic with a 130 mm thick floor slab, was subjected to different types of cyclic displacements with stepwise increases of amplitudes in one or both principal directions of the framing system [5]. These displacements are expressed as a percentage of the story height. The outline of the specimen and the loading pattern in the East-West direction only is also seen in Figure 3. The displacement patterns imposed with larger ductilities, such as associated with the shaded area in Figure 3, were particularly severe because plastic hinges were developed simultaneously with all four

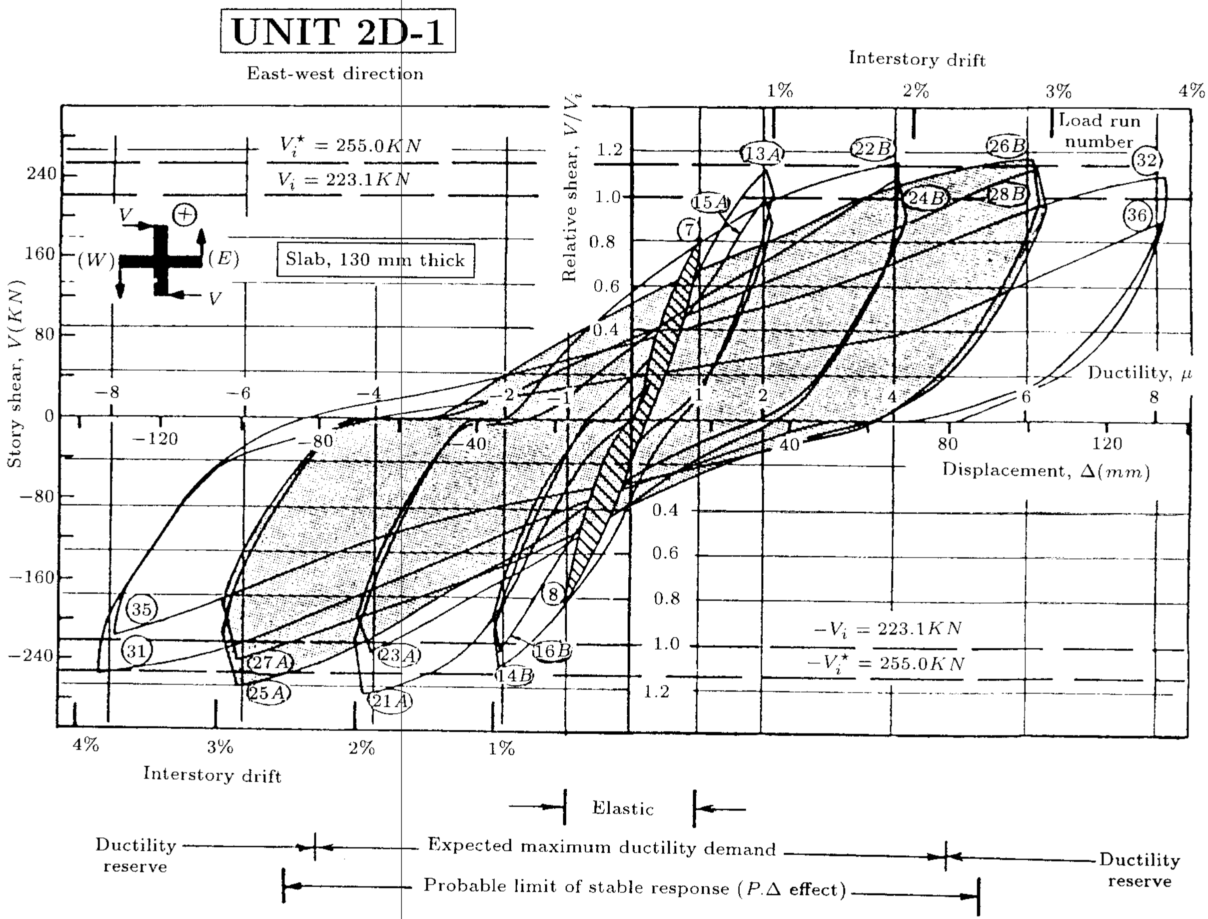

Figure 3. Hysteretic response of a two-way ductile frame subassemblage.

beams meeting at the joint.

The excellent hysteretic response of the unit is shown in Figure 3. It is seen that with progressively increasing inelastic displacements, stiffness degradation, as expected, was inevitable. However, strength reduction at peak displacements was found to be negligible. The normalized strength at unity in terms of the horizontal column shear force, V_i, corresponds to the ideal strength of the critical beam sections, including some contribution of flanges in tension, based on measured rather than specified material strength properties. The strength so defined corresponds to the probable strength, S_p, (Equation 2) of the prototype structure. It is also seen that at overstrength, 15 to 20% excess strength was achieved. This performance was achieved with judicious detailing of all plastic hinges and particularly the beam-column joint [1,2].

At the base of Figure 3 displacement ranges corresponding with elastic response, expected maximum ductility demand and ductility reserve are compared. It is considered that when interstory drift exceeds about 2.5%, significant reduction of frame resistance with respect to lateral forces must be expected due to P-delta effects.

REINFORCED CONCRETE DUCTILE FRAMES

The major steps of the previously stated capacity design approach, when applied to multistory buildings with only ductile frames, are outlined here.

Modelling for Analysis

The derivation of design actions is generally based on the elastic response of the structural systems. In order to obtain realistic results, for example for interstory deflections and estimation of periods of vibration, allowance for the effects of cracking on member stiffness, as implied in Figure 1, should be made. Typical reduction of flexural rigidity, based on the moment of inertia of gross concrete sections of prismatic members, is 60% in beams and 10 to 60% in columns, depending on the axial compression loads to be carried.

A Classification of Frame Response

One of the important tasks of the designer is to choose a suitable plastic frame mechanism for energy dissipation. For this purpose it will be necessary to determine whether design earthquake forces or those due to gravity loads will govern the required strength, S_u, of members.

In earthquake-dominated frames the required strength of beams will be controlled by seismically-induced actions when these are combined with those resulting from gravity loads. The maximum beam moments in such frames will generally occur at column faces. In such frames it is relatively easy to provide columns that are stronger than the beams.

In gravity load-dominated frames flexural strength of beams is controlled solely by the appropriate combination of factored gravity loads. However, design earthquake forces will control the strength of columns. This is typical in low-rise buildings with relatively long beam spans. In such frames it is difficult and often irrational to design columns to be stronger than beams. Therefore a different but suitable plastic frame mechanism needs to be selected.

Frame Mechanisms

In order to minimize plastic hinge rotations, particularly in columns, a suitable plastic mechanism for energy dissipation during a major earthquake must be chosen. Potential plastic hinges should be dispersed over the frames rather than allowed to concentrate only in a few locations.

Figure 4 compares possible mechanisms in a simple 6 story-frame subjected at roof level to a given displacement, Δ_u, at the ultimate state. It is now universally accepted that column sway mechanisms developing in a one story only (Figure 4a) should be avoided. Columns, being designed to be stronger than beams, can ensure that such "soft story" mechanisms will not develop.

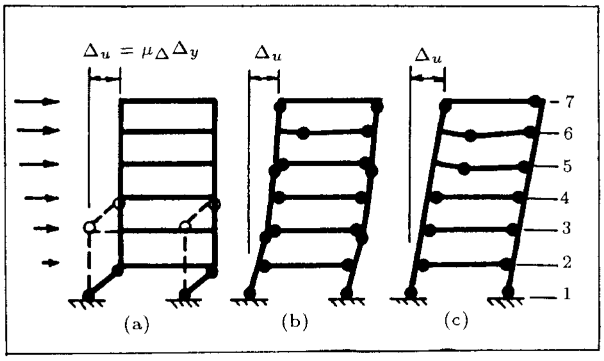

Figure 4. Frame mechanisms.

Figure 4b shows that some columns may develop plastic hinges at one end. This type of mechanism, widely used in the United States, requires all column ends to be appropriately detailed for ductility. However, if columns are provided with additional reverse strength, it is possible to restrict plastic hinge formation to the column base only, as shown in Figure 4c. This approach, extensively used in New Zealand, allows the relaxation in the detailing of columns in all upper storys, as the formation of plastic hinges is not expected.

Principles of Beam Design

When beams are chosen to be the weak links of the chain of resistance, all other parts of the structure, such as joints, columns and the foundations, need to be somewhat stronger. Therefore, the strength of beams influences the overall strength and cost of ductile frames. To this end the strength of beams should be kept as close as possible to the minimum that is required.

Beam actions resulting from gravity loads and the required earthquake forces are, as a rule, derived with the use of analysis of elastic structures. In recognition of the eventual ductile response of beams an intelligent redis-

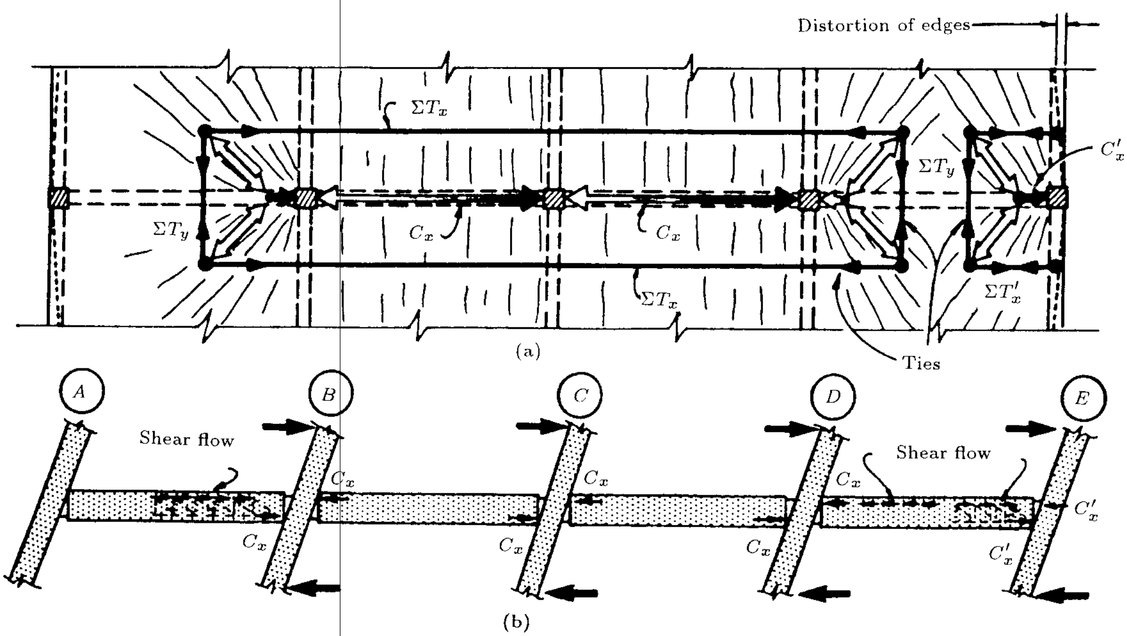

Figure 5. Flange mechanisms in continuous beams.

tribution of bending moments [2] within certain limits should be used. The general aim is to reduce peak values of design moments at the expense of increasing moments in regions where small values were predicted by the analysis of the elastic structure.

When the dimensioning of beams has been completed, the maximum moment that could be developed at critical sections of plastic hinges by large inelastic story displacements needs to be evaluated. This is termed the beam flexural overstrength (Equation 3). It must be based on the dimensions and reinforcement of the beam, as built.

In the evaluation of the flexural strength of beams the contribution to adequately anchored reinforcement in reinforced concrete slabs that are supported by the beams, must be included [2]. It should be remembered that large earthquake induced displacements will mobilize the strength of the majority of bars placed in slabs that interact monolithically with the beams. Neglecting the contribution of tension flanges will result in an underestimation of beam overstrength, which in turn may jeopardize the performance of components that were intended to remain elastic. Slab contributions to beam strength need also to be considered with the design of beam-column joints. While plastic hinges are being developed, beams become longer. Hence, all slab reinforcement placed parallel to such beams is strained in tension. The tension forces so generated are then balanced by equal compression forces at the beam-column junctions. A qualitative description of the mechanism [4] is given in Figure 5.

Once the flexural overstrength of both plastic hinges in the span of a beam, as shown in Figure 4, is determined for each of the two directions of earthquake attack, the maximum moment induced shear forces can then be found from consideration of equilibrium. After the superposition with gravity load-induced shear forces, shear envelopes can be determined and the required shear reinforcement provided. The overloading in shear of beams so designed is not possible. This approach demonstrates in the simplest form, the intent and application of the philosophy of capacity design.

PRINCIPLES OF COLUMN DESIGN

The Major Steps of the Design Process

The reasons for enforcing a preferred mechanism, such as the shown in Figure 4c, were enumerated. Thus, in accord with the specific approach to the design of ductile frames, columns should have adequate strength to remain elastic at all upper levels while the overstrengths of the beams that frame into them are being developed.

A magnification of column design moments is necessary to provide reserve flexural strength of columns at and above level 2. This can be evaluated in two simple steps. To enable a column to match the moment input at the overstrength of adjacent beams, quantified by the beam overstrength factor, ϕ_o, which was obtained with Equation 5, it is necessary that:

$$M_{i,col} \geq \phi_o M_{E,col} \ , \tag{8}$$

where $M_{E,col}$ is the column moment derived from the initial elastic analysis for the prescribed lateral design forces and $M_{i,col}$ is the required ideal strength, both taken at the same level. To enable, during manipulations, equilibrium requirements at beam-column joints to be continuously monitored, the above moment values refer to the centre of the joints, that is node points of the model frame.

Specifically for columns the overstrength factor is defined as:

$$\phi_o = \Sigma M_{b,o} / \Sigma M_{b,E} \ , \tag{9}$$

where $M_{b,o}$ is the flexural overstrength of the beams as detailed, and $M_{b,E}$ is the moment derived for the beams for the prescribed earthquake forces.

To clarify these relationships not encountered in gravity loaded structures, Figure 6 is presented. The moments due to seismic design action only in the vicinity of a joint, are shown in Figure 6a. Because beam and column moments are in equilibrium at the node point, they are used as reference quantities to assist in the preservation of equilibrium during subsequent changes. The beam moments resulting from the overstrength of each beam on either side of the column are shown in Figure 6b. To maintain joint equilibrium, the sum of the corresponding column moments must be equal and opposite. In this figure it has been assumed that the moments in the columns above and below the beam bear the same relation to each other as in Figure 6a.

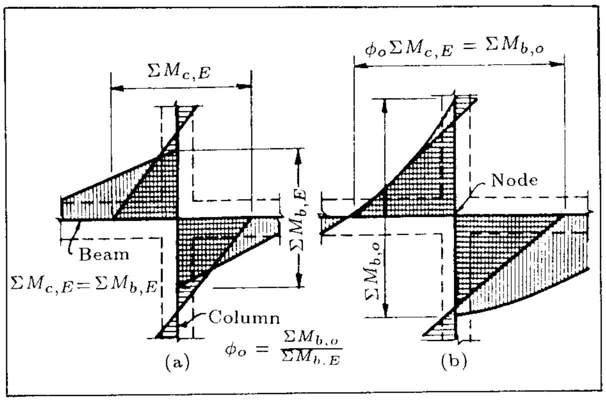

Figure 6. The relationship between beam and column moments at a node.

It is emphasized that the moments in Figure 6b are induced by large lateral seismic displacements. They are based on the properties of the beams as designed and detailed, and therefore include whatever considerations have been given to gravity load effects.

The moments induced in columns of a ductile multistory frame will be very different from those predicted for the elastic structure. Figure 7 compares moment patterns for such a column. The first figure shows the result of analysis for specified equivalent lateral static forces, while the others illustrate moment patterns encountered at different instants during the elasto-plastic dynamic response to a selected earthquake record.

In recognition of features associated with the inelastic dynamic response of a frame, as illustrated in Figure 7, column design moments must be further increased to accommodate significant departures of earthquake-induced moments above or below a level from those predicted by the initial elastic analysis. This may be achieved by a dynamic magnification factor, ω, so that:

$$M_{i,col} \geq \phi_o \omega M_{E,col} \ . \tag{10}$$

The value of ω was derived from a large number of analytical time history studies of the inelastic dynamic response of frames subjected to a variety of earthquake records [6]. Rec-

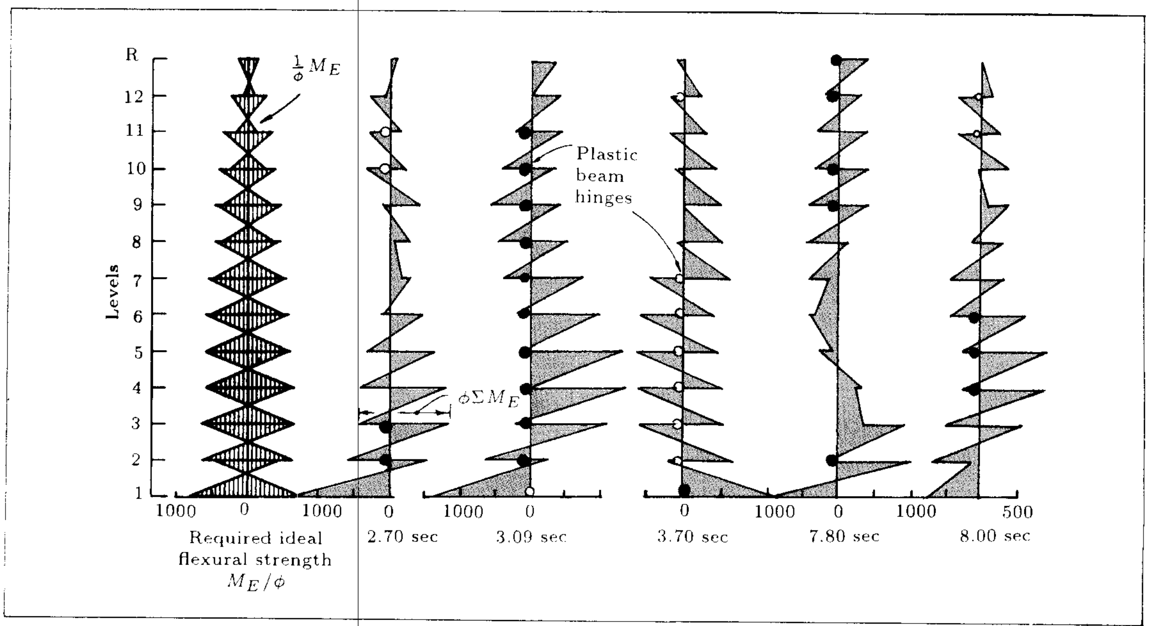

Figure 7. A comparison of column moment patterns due to horizontal static and dynamic forces.

ommended values [1] for the dynamic moment magnification factor applicable to columns of one-way frames are:

$$\omega = 0.6T_1 + 0.85 \ , \qquad (11a)$$

with the restriction that:

$$1.3 \leq \omega \leq 1.8 \ , \qquad (11b)$$

where T_1 is the fundamental period of vibration (in seconds) of the structure. The factor ω allows for the greater vulnerability of columns of long period structures where responses in the higher modes are more significant. This is seen in Figure 7.

In the third and final step in the process of estimating the necessary flexural strength of columns, the centre line moments determined from Equation 10 are reduced to correspond with the location of the critical section in relation to nodal points. The three steps used in arriving at the final column design moment are illustrated in Figure 8.

With conservative assumptions with respect to the gradient of column moments in a story, the final design moments for a critical column section, M_u, may be found from:

$$M_u = \phi_o \omega M_E - 0.3 h_b V_u \ , \qquad (12)$$

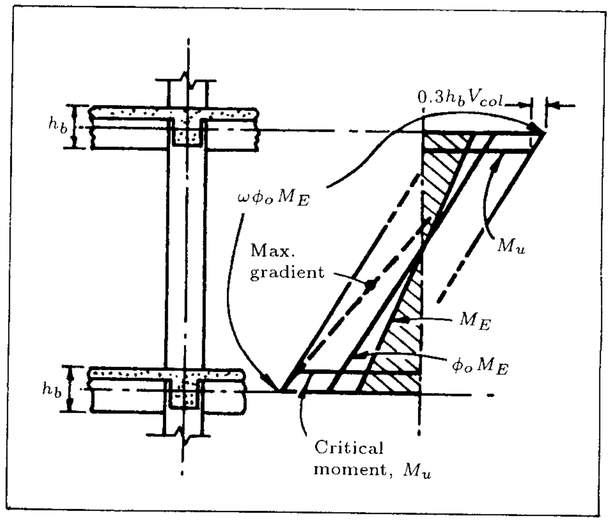

Figure 8. Moment magnifications for columns in the upper stories.

where h_b is the depth of the beam and V_u is the design shear force for the column. With certain limitation [1] some reduction of the design moments for exterior columns may be made when these are subjected to very low axial compression or to tensile forces. This concession may allow a considerable reduction in the longitudinal reinforcement that needs to be provided, and an optimization of it when the design moments concurrent with the extreme limits of axial forces, consistent with the direction of earthquake attacks, are considered.

It should be noted that the moments obtained in accordance with Equation 10 are magnified by the factor ω independently at each end of the column. This implies that the moment gradient, and hence column shear force, V_E, is not expected to differ very significantly from that derived from Equation 8, that is $V_u \approx \phi_o V_E$.

Column design shear forces should be based on the maximum beam moment input, $\phi_o \Sigma M_E$. However, to allow for exceptional wave forms during the response of a frame, it is recommended [1] that an increase in the moment gradient associated with frame overstrength should be considered, so that in upper stories:

$$V_u = \omega_v \phi_o V_E \ , \tag{13}$$

where ω_v is dynamic shear magnification factor with a value of 1.3 and 1.6 for columns of one-way and two-way frames respectively, and V_E is the column shear force derived from the initial elastic analysis for the prescribed seismic design forces. Although Equation 13 is considered to be intentionally conservative in order to provide added protection against an undesirable shear failure, it usually leads to moderate demands for shear reinforcement.

As Figure 5 implies, after the formation of two plastic hinges, the span length of beams increases. This is particularly significant when the two plastic hinges associated with the direction of earthquake attack are in different positions, as shown in Figure 9. Such hinges will not be subjected to reversable plasticity and hence deformation beyond the yield level will be cumulative. The corresponding progression of plastic hinge rotations and subsequent elongations of beams is illustrated in Figure 9.

The lengthening of beams during ductile response will affect particularly columns of the first story. As Figure 10 illustrates, in the first story the displacement of the top ends of the columns relative to the base will be different for each column. In the example shown, columns 3 and 4 are severely affected while columns 1 and 2 are relieved.

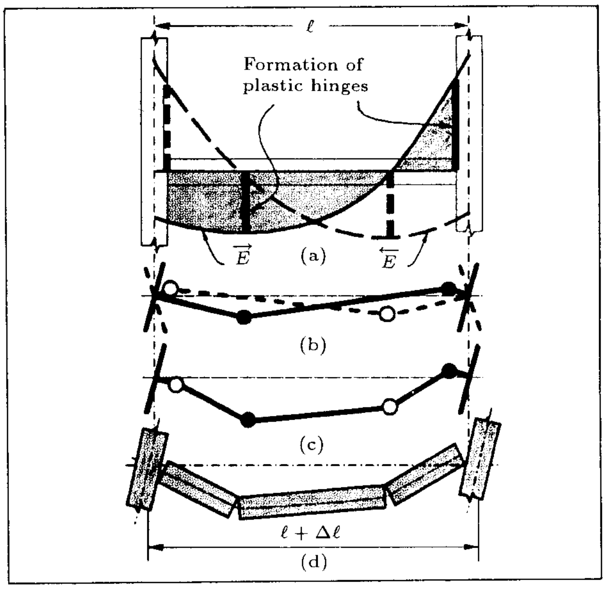

Figure 9. Beam elongations resulting from plastic hinge rotations.

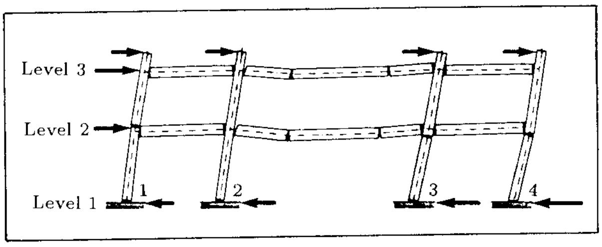

Figure 10. The effect of inelastic beam elongations on first story columns of a frame.

As a consequence, it should be assumed that because of beam elongation first story columns may develop plastic hinges also at the top end at level 2. Hence, these columns must be designed for shear and detailed accordingly [7].

Column axial design forces should be based on the summation of the maximum earthquake induced beam shear forces, that is, at the development of the overstrengths of the relevant beams (Equation 5). Some reduction of design axial forces may be considered in recognition of the likelihood that beams will not develop flexural overstrength at all levels above the story considered [1]. The reduction also takes into account the magnitude of the dynamic moment magnification factor, ω, and whether the column is part of a one-way or two-way frame [2]. With the earthquake induced column axial forces, so derived, the appropriate gravity loads need to be superimposed.

The design and detailing of columns can now proceed with the knowledge of the magnitudes of the moments, axial and shear forces corresponding to the development of the overstrength of the plastic mechanism adapted for the framing system. When this ideal (nominal) strength of the critical column section is made sufficient to resist these actions, no inelastic deformation in columns above level 2 should be expected.

Advantages and Limitations of the Capacity Design of Frames

The advantages claimed to result from the application of capacity design to reinforced concrete multistory ductile frames are:

1. In the event of a major earthquake, the development of the chosen energy dissipating mechanisms is assured.
2. In the presence of strong columns, beam plastic hinges will be dispersed over several stories of frames. Thereby, a concentration of inelastic deformations is prevented.
3. With the elimination of the possibility of plastic hinges developing at the ends of columns in and above the second story, the need for detailing of these regions to ensure significant curvature ductility capacity, particularly by confining compressed concrete with transverse reinforcement, does not arise.
4. The shear resistance of the end regions of columns is not adversely affected by inelastic deformations.
5. Lapped splices of column bars, which, as a general rule must not be located in plastic hinge regions [1], may be arranged immediately above a floor at the bottom end of columns. Where plastic hinges are to be expected, such as in the first story, lapped splices must be located in the mid-height regions of such columns [1].
6. Slightly increased column dimensions which may be resulted, will reduce congestion of the shear reinforcement in beam-column joints, and provide improved anchorage conditions for beam bars, a rather critical aspect of the detailing of reinforcement.

The recommended magnification of column design actions cannot be claimed to ensure that no yielding of the longitudinal bars at column ends can ever occur. Under extreme shaking rare transient yielding of column bars in the extreme fibers of the critical section can be expected. Such inelastic deformations, corresponding to negligible curvature ductility demands, should not be considered as being synonymous with the formation of a plastic hinge. The significance of restricted yielding of the longitudinal bars in one column of a story is in the reduction of its stiffness rather than ductility demand.

STRUCTURAL WALLS

The advantages of structural walls in the resistance of lateral forces, particularly in terms of deflection control, are well established. The term "shear wall", although a misnomer, is still widely used. Apart from shear, walls must also resist overturning moments and gravity loads, just like frames, and shear resistance is not necessarily a critical aspect of design. In seismic design special precautions must be taken to suppress shear failures under any circumstance.

Critical aspects of the design depend on the number and the length of walls available in one building to resist earthquake actions. In apartment buildings numerous walls may be utilized and, hence, demands on individual walls may be small. Often code-specified minimum amounts of reinforcement will suffice to satisfy strength requirements with modest ductility demands. Even elastic response may be assured. In office buildings, however, the entire lateral force resistance may be assigned to a few walls and these then will require special attention. Considerations of seismic design in the following sections address mainly this type of the walls. Specific design recommendation based on the philosophy of capacity design have been formulated in New Zealand [1] and also adopted for other countries [8]. Only fundamental issues, relevant to the inelastic response of wall systems can be reviewed here.

Cantilever Walls

In some multistory buildings the entire lateral force resistance may be assigned to a set of cantilever walls. When rigid diaphragms are present, an analysis of the elastic system readily provides the wall actions corresponding to story translations and torsion. To obtain realistic predictions for the serviceability limit state, it is essential to take the effects of wall cracking on wall stiffness into account. Because walls are generally lightly reinforced, the reduction of flexural rigidity after cracking is very significant [2,9]. A good estimate for the effective moment of inertia, I_e, of the gross wall section may be obtained from:

$$I_e = \left(\frac{100}{f_y} + \frac{P_u}{f'_c A_g}\right) I_g \ , \tag{14}$$

where the compression strength of the concrete, f'_c, and the yield strength of the reinforcement, f_y, are expressed in MPa, P_u, is the design gravity load on the wall and A_g and I_g are the area and moment of inertia respectively based on the gross concrete wall section. A typical relationship is $I_e \approx 0.3 I_g$.

Because walls will need to be detailed for significant curvature ductility capacity, in many situations designers may exploit inelastic redistribution of wall resistances to obtain more advantageous quantities of flexural reinforcement or demands on the foundations [2].

Failure Modes

Walls are very similar to ordinary beams. Hence, the well-established and understood simple principles of reinforced concrete are applicable and adequate. Common failure modes encountered in cantilever walls are shown in Figure 11.

Based on equilibrium and strain compatibility criteria, failure modes in flexure, shown in Figure 11b, are readily predicted [2,10]. It is important that in seismic design all vertical reinforcement presented in the stem and boundary elements or flanges of walls are to be taken into account. Typical strain profiles associated with ultimate limit states for flexure in channel shaped walls are shown in Figure 12. The example intends to illustrate the importance of wall geometry. The ultimate curvature capacity in one wall, controlled by concrete compression strains, such as $\varepsilon_c = 0.004$, may well exceed the maximum demand imposed by the design earthquake, while a reversed moment in the

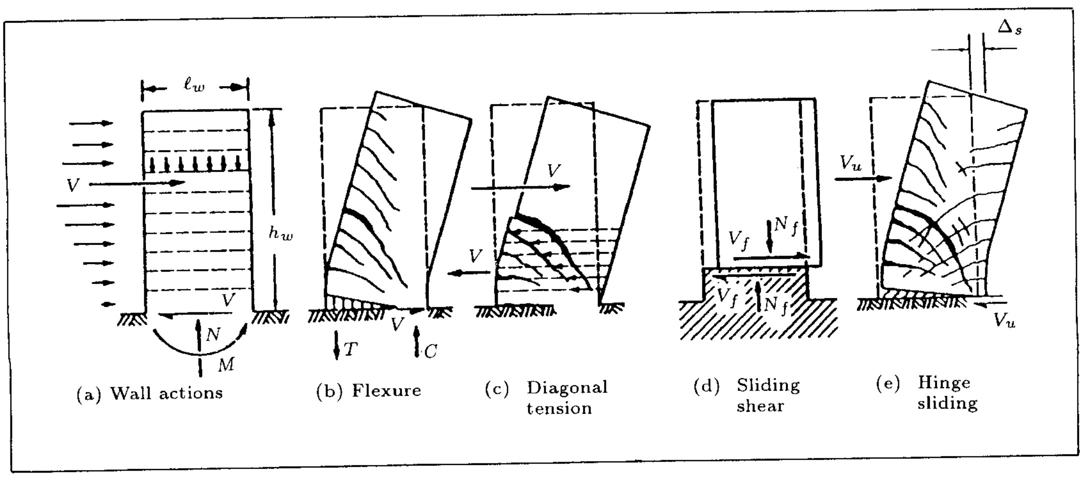

Figure 11. Failure modes in cantilever walls.

same wall with identical curvature demand may impose very large concrete compression strains. Figure 12 suggests that the ratio of the neutral axis depth c to wall length l_w is a unique parameter which gauges the curvature ductility capacity of a wall section. It takes into account the section configuration of the wall, the axial load intensity and the arrangement of the vertical reinforcement.

From first principles the relationship between the given displacement ductility demand for a wall, to be assumed by the designer when evaluating the magnitudes of seismic design forces, and the curvature ductility capacity of the critical section at the base of a cantilever wall, as detailed, can be readily established [2,10]. Such relationships for cantilever walls are shown in Figure 13. However, because of simplifications that can be made without significant loss of accuracy, such analysis need not be part of routine design. For example, it may be shown that when the ratio c/l_w is of the order of 0.1 or less, ample ductility capacity is available and premature crushing of concrete is not expected. For larger values of the ratio c/l_w, confinement of the concrete in part of the flexural compression zone of the wall sections may be necessary [1].

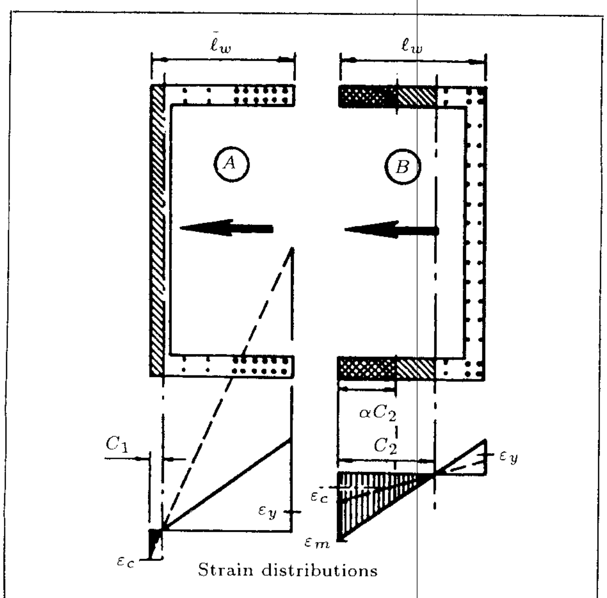

Figure 12. Strain profiles showing ductility capacities in channel-shaped walls.

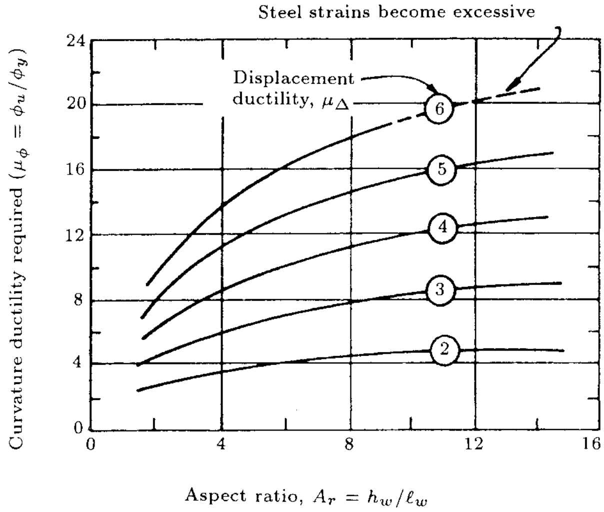

Figure 13. Required curvature ductility capacity of cantilever wall sections as a function of the displacement ductility demand and the aspect ratio of cantilevers.

Shear Strength of Walls

Because of the detrimental effects of inelastic shear mechanisms on the hysteretic response of walls, an aim of capacity design is to suppress in any event shear failures. This concern is particularly relevant to the potential plastic hinge region of a wall where reversing cyclic inelastic flexural strains may reduce shear strength that is generally relied on in the strength limit state associated with gravity loading.

The first task, therfore, is to make an upper bound estimate of the shear demand during the ductile response of the wall. As outlined previously, the shear generated while the flexural overstrength of the wall base is developed, is readily quantified using an overstrength factor defined by Equation 5. For walls this is $\phi_{o,w}$. In terms of typical code-specified lateral design forces, this step is illustrated in Figures 14a and 14b. While a plastic hinge develops at the base, the remainder of the wall is intended to remain elastic. Hence, the wall above the base plastic hinge remains sensitive to dynamic excitations in the higher modes. Figure 14c shows a typical distribution of inertia forces associated with such a situation, while a base hinge at overstrength is still active. Figure 14d compares moment patterns associated with the three sets

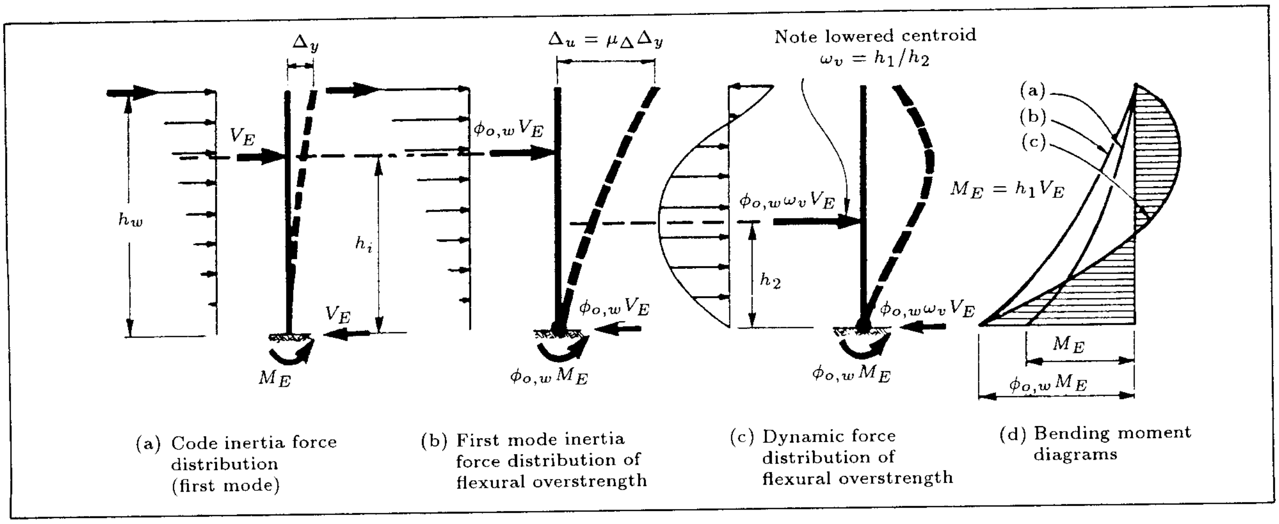

Figure 14. A comparison of code-specified and dynamic lateral forces acting on cantilever walls.

of lateral forces shown and it demonstrates that the critical base shear is that relevant to forces simulated in Figure 14c. It is seen in Figure 14 that the dynamic magnification of the base shear is simply $\omega_v = h_1/h_2$.

Accordingly, the maximum shear for cantilever walls can also be estimated from Equation 13 where the dynamic shear magnification factor, considered to be dependent on the number of stories, n, may be taken [1] as:

$$\omega_v = 0.9 + n/10 \ , \qquad (15a)$$

for buildings of up to six stories, and:

$$\omega_v = 1.3 + n/30 \leq 1.8 \ , \qquad (15b)$$

for buildings over six stories.

To protect the concrete against premature diagonal compression failure, the shear stress based on Equation 13 should be limited and reduced with increasing expected ductility demands [2].

Wall Instability

Aesthetic or functional requirements may necessitate the use of relatively thin rectangular walls. Clearly in the potential plastic region of such walls instability resulting from out-of-plane buckling may arise particularly in the first story. The failure mode is illustrated in Figure 15a. Recent studies [2] have shown that the

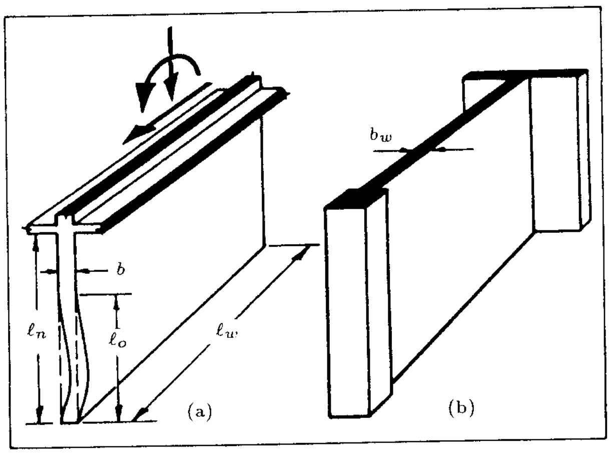

Figure 15. Typical sectional configurations of walls.

major parameter affecting buckling is the magnitude of the inelastic tensile strains imposed on the vertical flexural reinforcement during a preceding large displacement excursion. Buckling occurs while wide cracks are being closed and when softened reinforcement delays this closure. Accordingly the ratio of wall thickness at the end of a wall section to the clear height in the first story must be restricted [1] when the expected ductility demand is significant. Wherever possible enlarged boundary elements, as shown in Figure 15b, should be used.

Coupled Structural Walls

When walls of multistory buildings contain regular vertical rows of openings for windows or doors to elevator shafts, it is customary to refer to coupled walls. These are connected by coupling beams formed below and above such openings. Because of their significant stiffness and energy dissipation over the full height, coupled structural walls, when suitably detailed, possess optimal seismic properties. Only a cursory review of the aspects of seismic design of these structures can be presented here.

The plastic mechanism of two coupled walls is shown in Figure 16d. At the extreme, all coupling beams can be expected to yield while a plastic hinge forms at the base of each wall. The aim of capacity design in this case is the prevention of shear failure in any component, while large displacements are repeatedly sustained in a stable manner. The full line curves in Figures 16a to 16c show the wall moments, M_1 to M_2, and coupling beam shear forces, q, obtained with the analysis of the idealized elastic structure. The dashed curves show the design quantities which may be used after some inelastic redistribution of the previously derived moments and shears has been allowed for [2]. The justification of the redistribution of design actions stems from the large ductility capacity, which appropriate detailing of the structure will ensure. The moments, M_1' and M_2', so derived, when combined with the appropriate axial forces, can then be used to proportion the base sections of the walls.

In the first step in the design for shear resistance the overstrength of the coupling beams in terms of the shear force transmitted, Q_{io}, is determined. It is now well established [2,10] that to ensure the ductile response of the coupling beams, which are often short and relatively deep, diagonal reinforcement, extending in both directions from corner to corner of such beams and adequately anchored in the walls, should be provided [1].

The summation over the full height of the structure of the shear forces developed in the coupling beams at overstrength, will give an upper bound estimate of the earthquake induced axial force, P_{Eo}, at the base of the two walls. In tall structures some reduction of this force may be allowed for [2] in recognition of the likelihood that not all beams will develop simultaneously overstrength. This enables maximum axial forces, including gravity loads, in compression, P_{2o}, and tension P_{1o}, to be estimated. With allowance for the strength enhancement for the vertical wall reinforcement provided at the base, the flexural overstrength of the walls, M_{1o} and M_{2o}, in presence of the axial forces, P_{1o} and P_{2o}, can now be evaluated. Corresponding with Equation 5, the overstrength factor for the coupled wall system is obtained from:

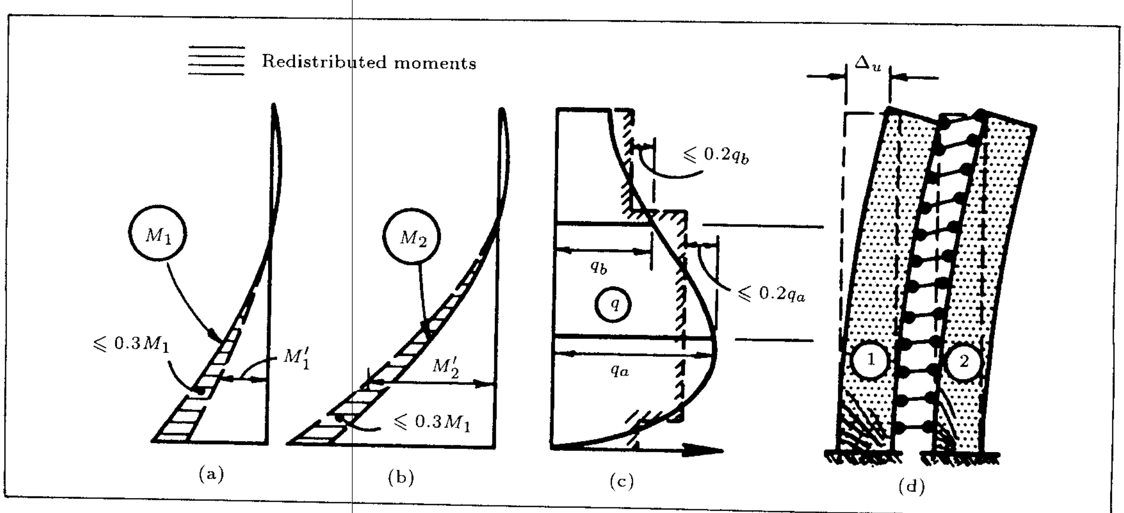

Figure 16. Design actions for a ductile coupled wall structure.

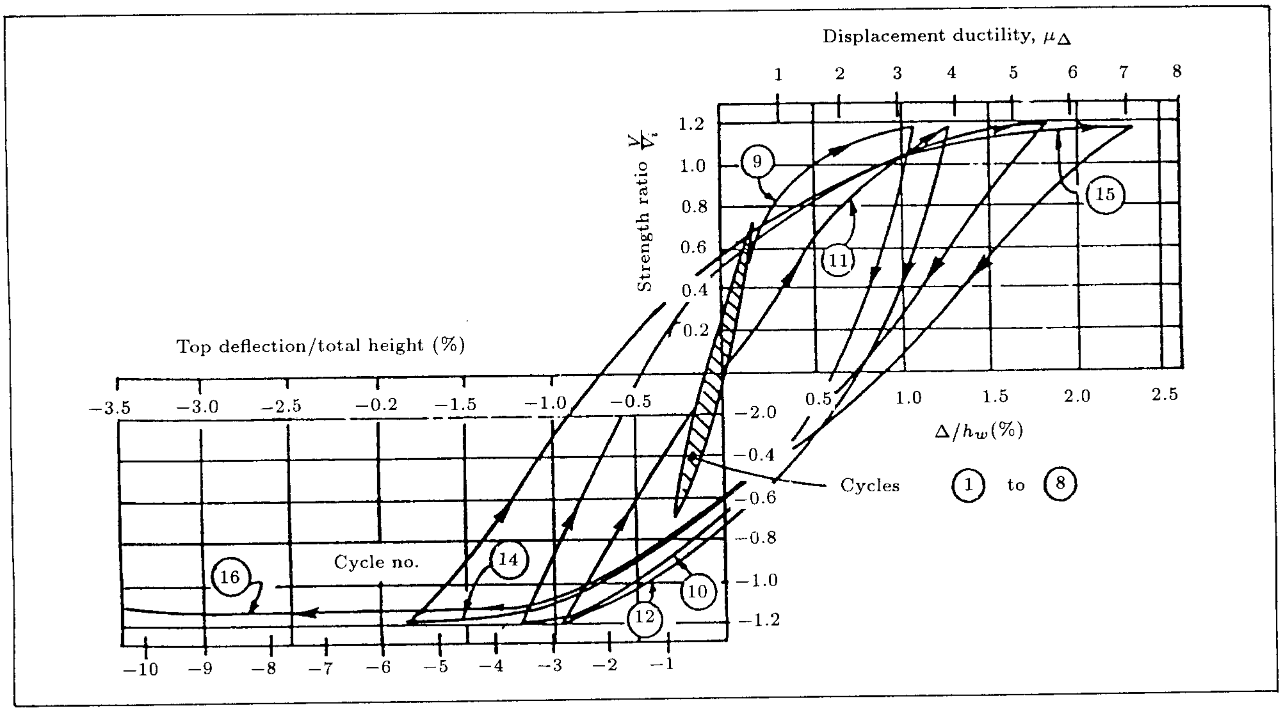

Figure 17. The response of a model coupled test wall structure.

$$\phi_{o,w} = \frac{M_{1o} + M_{2o} + lP_{Eo}}{M_E}, \tag{16}$$

where M_E is the overturning moment at the base corresponding to the base shear, V_E, resulting from the specified intensity of lateral design forces. The design shear force that is not expected to be exceeded in any seismic event is then estimated for each wall from:

$$V_{i,wall} = \omega_v \phi_{o,w} \left(\frac{M_{io}}{M_{1o} + M_{2o}} \right) V_E . \tag{17}$$

The response of a seven story one quarter full size test wall [11] with diagonally reinforced coupling beams [10] is shown in Figure 17.

It is seen that stable hysteretic response with displacement ductility demands in excess of six could be achieved with no sign of strength degradation. During the response with ductilities in excess of three, approximately 18% enhancement of the strength based on measured material properties was consistently achieved.

DUAL STRUCTURAL SYSTEMS

In many buildings, resistance to earthquake forces will be provided by both frames and walls. These are defined as dual or hybrid systems. The modelling of some prototype dual structures, as well as two acceptable energy dissipating mechanisms, are shown in Figure 18. It is well-established that the two systems, when compatible story deflections are assured by very rigid floors acting as diaphragms, offer efficient resistance to lateral forces in approximately the lower half of the structure. Because of the totally incompatible deflections of the two structures in the upper half of a building, when functioning independently, frames of dual sys-

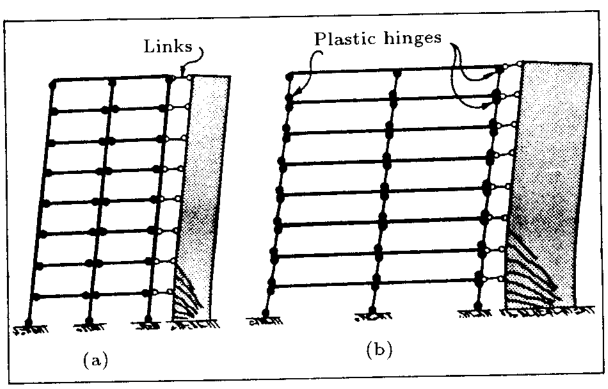

Figure 18. Energy-dissipating mechanisms in dual systems.

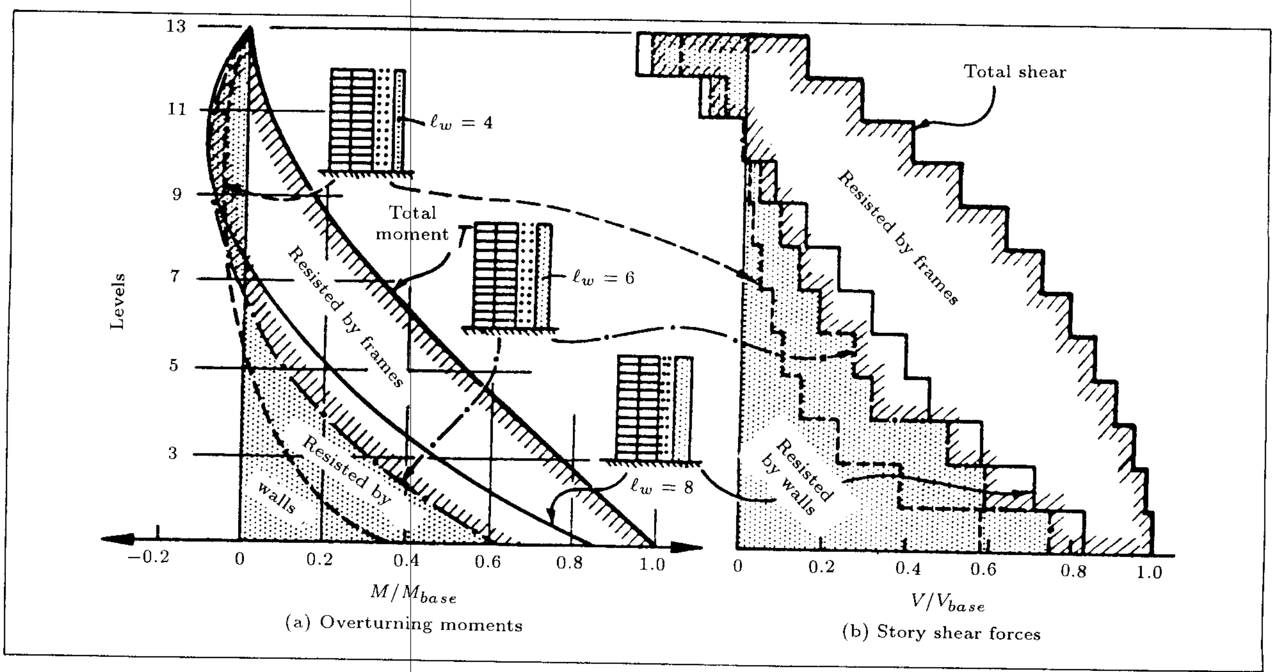

Figure 19. Wall and frame contributions to the resistance of overturning moments and story shear forces in three elastic example structures.

tems tend to resist at those levels forces that are larger than the total lateral floor forces. As a corollary, walls near the top of the building are subjected to negative forces. Figure 19 shows typical divisions of the resistance to overturning moments and to horizontal story shear forces for three example dual systems. These example structures consist of seven two-bay frames and two symmetrically positioned cantilever walls. While the frames are identical in the three structures, the length of the walls was assumed to be 4, 6 and 8m respectively. As expected, the stiffer walls with l_w = 8m offer maximum resistance to both moments and shear forces. However, their contribution to lateral strength dramatically are reduced in the upper half of the structure. These results may be obtained from routine analyses of elastic dual systems.

Some building codes specify that at least a fraction, typically 25%, of the lateral design forces should be assigned to the frames. As Figure 19 shows, this procedure may be satisfactory at the bottom stories of the building, but it is grossly unconservative in the upper half of the structure. At these levels more than 100% of the earthquake induced lateral forces may need to be resisted by the frames. Therefore such arbitrary allocation of lateral force resistance should not be used.

With appropriate relatively minor modifications capacity design procedures, outlined previously for ductile frames and walls, can be used [2]. It should be noted, however, that the participation of walls is sensitive to the degree of base fixity, that is, the type of foundation that is available. Because of the interaction of walls and frames, diaphragm forces may be generated that are significantly larger than those encountered in multistory systems with frames or walls only. These diaphragm forces do require special attention particularly when precast concrete floor systems are used.

DETAILING FOR DUCTILITY

It is re-emphasized that the aims of the seismic design strategy described for various structural systems can be fulfilled only with judicious detailing of the reinforcement in all potential inelastic regions that have been chosen for energy dissipation. This aspect of the design process, often neglected in the technical literature, should be considered as important as the analytical work leading to the derivation of design actions and the proportioning of structural members for strength. The causes of the majority of failures in earthquakes can be attributed to neglect in detailing for ductility and flawed structural concepts rather than to lack of precision in analysis.

Because large and repeated strain reversals are to be expected in both steel and concrete, the requirements for ductility differ from those relevant to the resistance of gravity loads and wind forces. Space limitations do not allow a detailed treatment of this very important topic to be presented here. Significant research efforts in New Zealand, based on simple rational analyses supplemented by extensive experimental work, were devoted to developing specific detailing techniques [1,2,4].

CONCLUSIONS

A design strategy and its application were outlined for reinforced concrete buildings in which earthquake resistance was provided by ductile frames, by ductile structural walls or by the interactive combined actions of these two systems. Emphasis was placed on design features which, over the past 25 years, were primarily developed in New Zealand. The design strategy described evolved from the following precepts:

1. In the context of the state-of-the-art in structural engineering, current predictions of the probable characteristics of large earthquake-generated ground motions are crude. Under these circumstances an aim to achieve a degree of precision in analytical techniques, comparable to those developed for structures to satisfy serviceability and "hypothetical" ultimate limit states, to predict both earthquake-induced actions and deformations within the structure, is not justified.

2. Provided that a reasonable level of resistance to lateral forces, such as prescribed for various seismic regions by relevant national building codes, is chosen, errors arising from crude estimations of the characteristics of ground motions will manifest themselves only in erronous predictions of earthquake-imposed displacements, that is, ductility demands. Thus deformation capacity is one of the most important structural property in areas of high seismic risk.

3. Types and localities of energy dissipation mechanisms need to be chosen as a part of the capacity design procedure, in which a unique hierarchy of strengths is established. All weak and necessarily ductile links must satisfy requirement of the stipulated level of lateral force resistance. The distribution of minimum strengths throughout a ductile structure associated with lateral forces, both horizontally and vertically, may be based on a simple analysis of elastic systems with subsequent redistribution of design actions, sometime quite significant, from less to more desirable locations.

4. As a general rule, rationally-detailed structures can be made very ductile with relative ease and little, if any, additional cost. Thereby, a considerable reserve in inelastic deformations, that is ductility capacity, can be imparted to structural systems. Detailing of reinforced concrete structures, very often considered a subordinate, depreciated drafting activity with apparent lack in intellectual appeal, deserves at least as much attention as the analytical work used to estimate design actions. Faults in detailing are the first that will be revealed during earthquakes. They are predominant causes of structural distress. The detailing of potential plastic regions is partly an art. It relies on feel for, and understanding of, the natural disposition of internal forces and often invites innovations. Judiciously-detailed ductile systems will be tolerant with respect to imposed seismic displacements, a valuable feature of structural response, that will compensate for the crudeness in predicting the magnitudes of such displacements.

5. Various steps in the description in previous sections of the design procedure were intended to emphasize the designer's determination to simply "tell the structure what to do". It is in this respect that the design strategy is deterministic. It inhibits the activation of mechanisms other than those chosen. Thereby, reinforced concrete buildings can be made very tolerant to a wide range of ductility demands. Hence, they can be expected with confidence to perform "as they were told to".

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