Ductility of Structural Concrete

Ductilité du béton armé

Duktilität von Konstruktionsbeton

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SUMMARY

Definitions for the required and available ductility of structural concrete are described. The need for ductility in the design of structures for earthquake resistance is considered, and possible modes of inelastic deformation of moment resisting frames and structural walls are outlined. Procedures for detailing members for ductility are discussed.

RÉSUMÉ

Des définitions concernant la ductilité nécessaire et disponible du béton armé sont présentées; il est nécessaire de tenir compte de la ductilité dans le dimensionnement des structures devant résister aux séismes. On esquisse les modes possibles de déformation plastique des cadres résistant aux moments et des parois armés. On discute des procédés qui permettent d'augmenter la ductilité des éléments de construction.

ZUSAMMENFASSUNG

Definitionen für die erforderliche und verfügbare Duktilität von Konstruktionsbeton werden beschrieben. Es wird die Notwendigkeit begründet, die Duktilität bei der Bemessung von Tragwerken im Hinblick auf eine Erdbebenbeanspruchung zu berücksichtigen und es werden die möglichen Arten nicht-elastischer Verformungen von Rahmentragwerken und Wandscheiben aufgezeigt. Die Vorgehensweise für die konstruktive Durchbildung von Bauteilen zur Errechnung der Duktilität werden diskutiert.

1. INTRODUCTION

The term "ductility" in structural design is used to mean the ability of a structure to undergo large deformations in the postelastic range without a substantial reduction in strength. This can be contrasted with "brittle" behaviour in which the load carrying capacity of the structure decreases sharply when the strength is reached.

In design, consideration of the available ductility of a structure is necessary for the following reasons: (1) to prevent brittle failure, (2) to use distributions of bending moments differing from that obtained from linear elastic structural analysis, and (3) to survive severe earthquake and blast loading.

This paper considers the ductility of structural concrete members and structures. Emphasis is given to aspects of the design of structures for earthquake resistance, since ductility considerations are of paramount importance in the design for earthquake loading.

2. DEFINITIONS FOR REQUIRED DUCTILITY

The required ductility of a structure, element or section can be expressed in terms of the maximum imposed deformation. Often it is convenient to express the maximum deformations in terms of ductility factors, where the ductility factor is defined as the maximum deformation divided by the corresponding deformation present when yielding occurs. The use of ductility factors permits the maximum deformations to be expressed in nondimensional terms as indices of postelastic deformation for design and analysis. Ductility factors have been commonly expressed in terms of the various parameters related to deformations, namely displacements, rotations, curvatures and strains.

The displacement ductility factor is $\mu = \Delta_{max}/\Delta_y$, where Δ_{max} is the maximum displacement and Δ_y is the displacement at yield. The displacement ductility factor μ is shown defined for ideal elastoplastic behaviour in Fig.1.

The rotation ductility factor required of members is θ_{max}/θ_y, where θ_{max} is the maximum rotation at the plastic hinge and θ_y is the rotation in the plastic hinge region at yield. The information most needed by designers is the required curvature behaviour of the critical sections of members in plastic hinge regions, expressed by the curvature ductility factor ϕ_{max}/ϕ_y, where ϕ_{max} is the maximum curvature at the section and ϕ_y is the curvature there at yield.
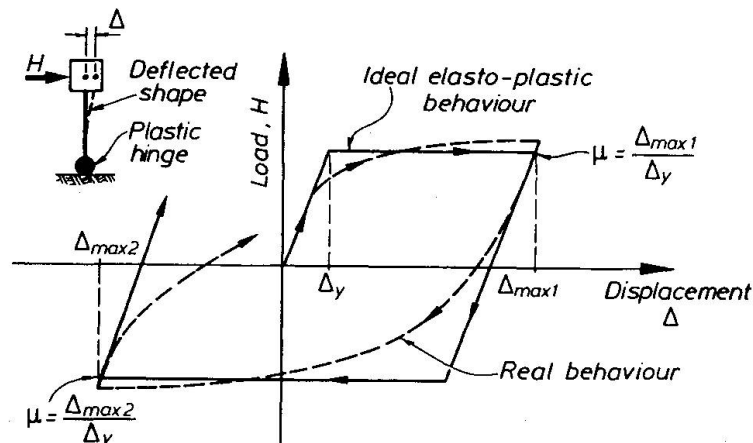

Fig.1   Displacement Ductility Factor

3. DEFINITIONS FOR AVAILABLE DUCTILITY

The ductility required of the structure when loaded into the postelastic range needs to be matched by the available ductility of the structure. Definitions which can be used to estimate the available ductility factor are considered below [2]:

(a) The Definition of the Yield Deformation - When calculating ductility factors the definition of the yield deformation (displacement, rotation or curvature) often causes difficulty since the force-deformation relation may not have a well defined yield point. This may occur, for example, due to nonlinear behaviour of the materials, or due to reinforcing bars at different depths in a structural concrete section reaching yield at different moment levels, or due to plastic hinges in different parts of a structure forming at different load levels. Various alternative definitions which have been used by investigators to estimate the yield displacement are illustrated in Fig.2. These are the displacement when yielding first occurs (Fig.2a), the yield displacement of the equivalent elastoplastic system with the same elastic stiffness and ultimate load as the real system (Fig.2b), the yield displacement of the equivalent elastoplastic system with the same energy absorption as the real system (Fig.2c), and the yield displacement of the equivalent elastoplastic system with reduced stiffness found as the secant stiffness at 75% of the ultimate lateral load H_u of the real system (Fig.2d). The latter definition (Fig.2d) takes the secant stiffness as described in order to include the reduction in stiffness due to cracking near the end of the elastic range. This latter definition is the most realistic definition for the yield displacement for reinforced concrete structures.
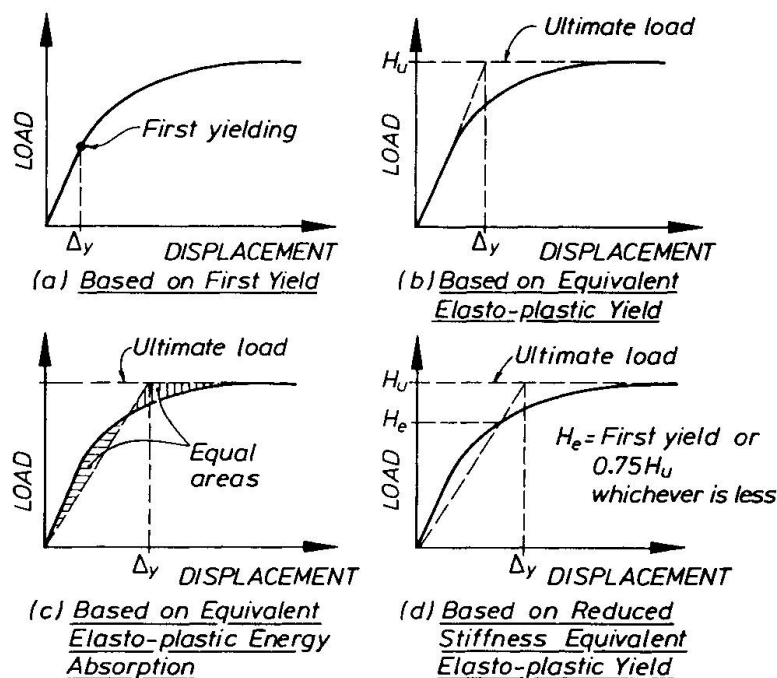
Fig.2   Alternative Definitions for Yield Displacement

(b) Definition of the Maximum Available (Ultimate) Deformation - The maximum available (ultimate) deformation has also been estimated using various assumptions by investigators. Some possible estimates for the maximum available displacement are shown in Fig.3. These are the displacement corresponding to a particular limiting value for the concrete compressive strain (Fig.3a), the displacement corresponding to the peak of the load-displacement relation

(Fig.3b), the postpeak displacement when the load carrying capacity has undergone a small reduction (Fig.3c), and the displacement when the transverse or longitudinal reinforcing steel fractures or the longitudinal compression reinforcement buckles (Fig.3d). When considering the most appropriate definition it should be recognized that most structures have some capacity for deformation beyond the peak of the load-displacement relation without significant reduction in strength. It would be reasonable to recognize at least part of this postpeak deformation capacity. Also, it is evident that the maximum available deformation does not necessarily correspond to a specified extreme fibre concrete compresssive strain. Hence the most realistic definition for the maximum available displacement are given by the criteria shown in Figs. 3c and 3d, whichever occurs first.
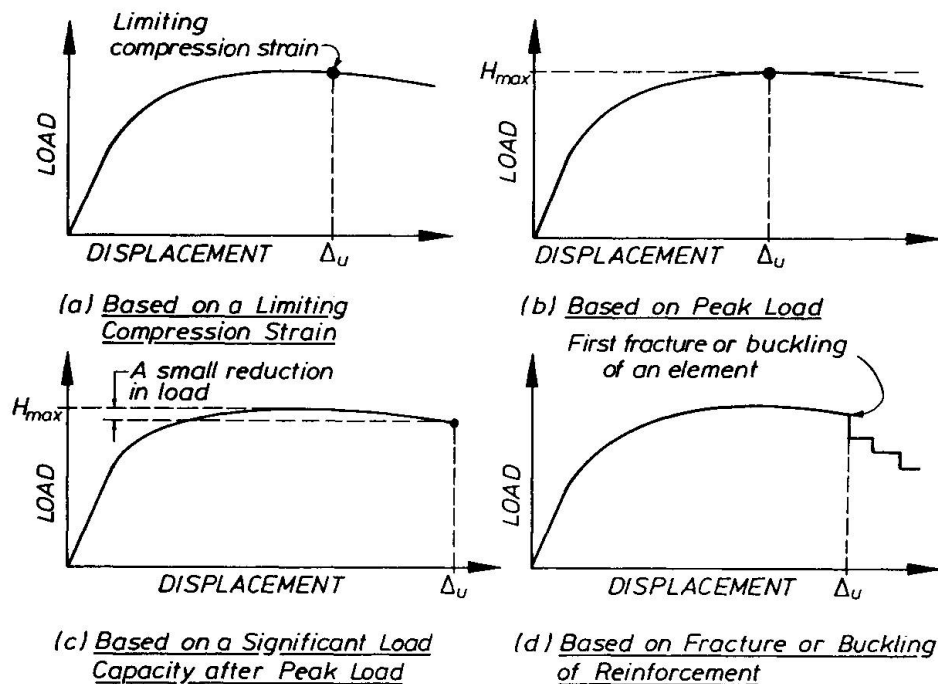

Fig.3 Alternative Definitions for Maximum Available (Ultimate) Displacement

4. DUCTILE DESIGN APPROACH FOR EARTHQUAKE LOADS

4.1 Design Strength and Ductility for Earthquake Loading

In the design of structures for earthquake resistance the emphasis should be placed on good structural concepts and detailing of reinforcement. It is well known that when a structure responds elastically to ground motions during a severe earthquake, the maximum response acceleration may be several times the maximum ground acceleration and depends on the stiffness of the structure and the magnitude of the damping. Generally it is uneconomical to design a structure with adequate strength to respond in the elastic range during a severe earthquake, since that would require very high design earthquake forces. Codes generally recommend lower levels of design earthquake loading which means that the critical regions of the structure need to be detailed to possess sufficient ductility to enable the structure to survive without collapse. For example, the design earthquake loads recommended by the New Zealand general structural design code [3] for ductile structures assumes that the structure is capable of deforming in the postelastic range to a displacement ductility factor of at least μ = 4 to 6 during several cycles of earthquake loading without significant loss of strength.

4.2 Capacity Design

The exact characteristics of the earthquake ground motions that may occur at a given site cannot be predicted with certainty and it is difficult to evaluate all aspects of the complete behaviour of a complex structure when subjected to a severe earthquake. Nevertheless it is possible to design the structure so that it has features that will ensure the most desirable behaviour. The rational approach for achieving this aim in earthquake resistant design is to choose the most suitable mechanism of postelastic deformation for the structure and to ensure by appropriate design procedures that yielding will occur only in the chosen manner during a severe earthquake. This approach is referred to as "capacity design" in New Zealand [3,4].

For moment resisting frames the best means of achieving ductile postelastic deformations is by flexural yielding at selected plastic hinge positions, since with proper design the plastic hinges can be made adequately ductile. The chosen plastic hinge positions are designed for adequate flexural strength and ductility for the code specified loading. Then when designing for shear the design shear forces are calculated on the basis of amplified plastic hinge moments, to take into account the actual quantity of longitudinal reinforcement present, the actual yield strength of that steel being higher than specified and strain hardening of steel at high deformations. If plastic hinges in columns of moment resisting frames are to be avoided, the design bending moments of columns may also need to be amplified to take into account the effects of higher modes of vibration and biaxial earthquake loading as well as beam overstrength [4].

4.3 Preferred Modes of Inelastic Deformation

Fig. 4 shows mechanisms of inelastic deformation which could form in moment resisting frames and structural walls due to the formation of plastic hinges during severe earthquakes. For tall moment resisting frames a beam sidesway mechanism is preferred since it makes more moderate demands on the curvature ductility required at the plastic hinges and ductility is more easily provided in beams than in columns. Collapse of frames during severe earthquakes due to a "soft storey" (column sidesway mechanisms) has commonly been observed. Ductile coupled structural walls (see Fig.4) should preferably be designed so that the coupling beams yield before the walls, and the coupling beams should be detailed for adequate ductility. Ductile cantilever walls should preferably be designed to ensure that flexural yielding occurs. For the substructures of bridges, the mechanisms inelastic deformations sought are similar to those for buildings during severe seismic loading [5].

4.4 Required Plastic Hinge Rotations

The required curvature ductility factor ϕ_u/ϕ_y which should be available at the plastic hinge locations in frames and walls will depend on the many variables involved, such as the geometry of the members and the relative strengths of sections. Codes do not generally expect designers to calculate the curvature ductility factors required at the plastic hinge regions. Instead, adequate ductility is considered to have been provided if the structure resisting seismic forces is detailed for ductility in accordance with the seismic provisions of the code [4].

## 5. DETAILING OF PLASTIC HINGE REGIONS

5.1 General

The most important design consideration for ductility in the plastic hinge regions of structural concrete members is the provision of adequate longitudinal compression reinforcement as well

as tension reinforcement, and the provision of adequate transverse reinforcement in the form of rectangular stirrups or hoops and cross ties or spirals, in order to act as shear reinforcement, to confine the compressed concrete, and to prevent premature buckling of the compressed longitudinal reinforcement. A centre to centre spacing of transverse bars not exceeding six longitudinal bar diameters in plastic hinge regions is considered to be necessary in order to prevent premature buckling of longitudinal bars during cycles of tension-compression yielding such as is caused by severe earthquake loading [4].
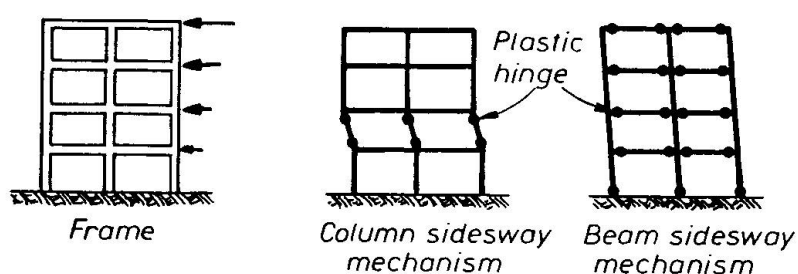
(a) Moment Resisting Frames
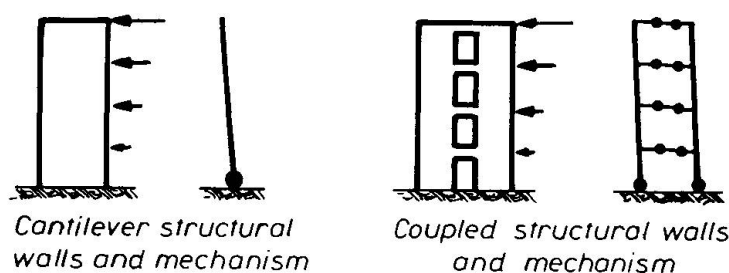
(b) Structural Walls

Fig. 4 Moment Resisting Frames and Walls and Mechanisms of Inelastic Deformation Due to Plastic Hinging During Severe Earthquake Loading

5.2 Ductility Enhancement by Concrete Confinement

The ductility (and strength) of structural concrete members can be greatly improved by confining the compressed concrete using arrangements of closely spaced transverse reinforcement in the form of spirals and circular hoops or rectangular hoops with adequate cross ties. The concrete becomes confined when at strains approaching the unconfined strength the transverse strains become very high and the concrete bears out against the transverse reinforcement, which then applies a passive confining pressure due to the arching of the concrete between the transverse bars and the longitudinal bars. The cover concrete, including that concrete outside the arching forces, is not confined and will be lost as in the case of unconfined concrete.

It is evident from Fig.5 that the confinement of concrete is improved if the transverse and longitudinal reinforcement is placed at relatively close spacing, and if there are a number of longitudinal bars well distributed around the column section and ties across the section, because then the arches between the bars are shallower and hence more of the concrete sections is confined.

(a) Circular hoops
or spiral

(b) Rectangular hoops
with cross ties

(c) Overlapping
rectangular hoops

(d) Confinement by
transverse bars

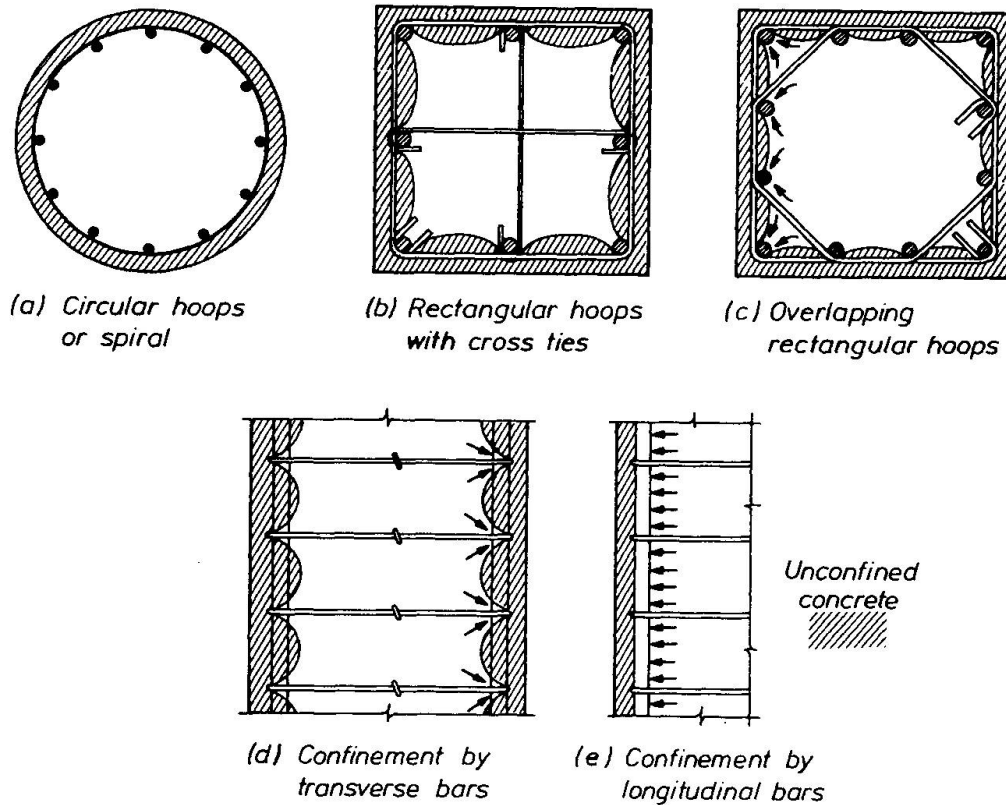(e) Confinement by
longitudinal bars

Fig.5 Confinement of Compressed Concrete by Reinforcement

Typical longitudinal stress-strain curves for well confined concrete and identical but unconfined concrete are shown in Fig.6. For confined concrete, eventual fracture of the transverse reinforcement limits the useful concrete compressive strain, but values in the range 0.02 to 0.08 are typically obtained [5,6]. The extent of the improvement in the stress-strain behaviour is a function of the lateral confining pressure, which in turn depends on the volume, yield strength, and efficiency of the arrangement of the transverse reinforcement.

Moment-curvature analyses incorporating models for the stress-strain relation of concrete confined by various quantities and arrangements of transverse reinforcement can be used to compute the quantities of transverse reinforcement required to achieve various curvature ductility levels. This procedure is the basis of several analytical approaches.
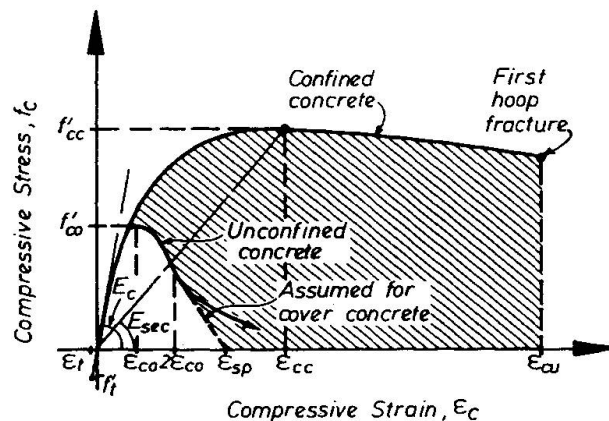

Fig.6 Typical Compressive Stress-Strain Curves for Confined and Unconfined Concrete [6]

5.3 Approach for Ductility Evaluation Based on a Limiting Concrete Compressive Strain

In the past, the maximum available (ultimate) curvature of a structural concrete member has usually been assumed to be reached when a specified "ultimate" compressive strain ε_{cu} has been attained by the concrete. Experimental research by a number of investigators has resulted in the development of several empirical equations for ε_{cu}. A difficulty with this approach is that ε_{cu} depends on several variables, including the ratio of neutral axis depth to member section depth, the shape of the compressed area of the member section, the confined concrete stress-strain relation and the steel stress-strain relation. It is difficult to include all the variables in an equation for ε_{cu}. Generally ε_{cu} values which have been proposed have resulted in conservative estimates of the ultimate curvature [1,5]. However the approach has the merit of simplicity, since the maximum available (ultimate) curvature can be readily calculated from $\phi_u = \varepsilon_{cu}/c$, where c is the neutral axis depth at that stage.

5.4 Approach for Ductility Evaluation Based on the Postpeak Moment-Curvature Behaviour

A more recent method imposes no limit on the concrete compression strain but seeks adequate moment-curvature behaviour. An example of this approach is that proposed by Zahn, et al [7]. Cyclic stress-strain relations for confined concrete were determined [6] which, along with cyclic stress-strain relations for reinforcing steel, permitted analytical predictions of the cyclic moment-curvature behaviour of reinforced concrete members. In addition, it was found that the longitudinal concrete compressive strain at first fracture of the transverse reinforcement could be estimated by energy considerations, by equating the increase in strain energy stored in the confined concrete (represented by the shaded area between the stress-strain curves for the unconfined and confined concrete in Fig.6) to that stored in the transverse reinforcing steel at fracture by tensile straining. These analytical procedures were used to determine the maximum available (ultimate) curvature of reinforced concrete columns containing various arrangements and quantitites of transverse reinforcement. To simulate the effect of severe earthquake loading, a sequence of four identical cycles of bending moment to peak curvatures of equal magnitude in each direction was imposed on the member. The peak curvature for which the moment reduced to 80% of the ideal moment capacity or for which fracture of the longitudinal or transverse reinforcement occurred, was defined as the maximum (available) ultimate curvature (see Fig.7). Design charts have been prepared which relate the maximum available curvature ductility factor ϕ_u/ϕ_y to the column axial load level and to the magnitude of the confining stress from the transverse reinforcement [7].

6. MECHANISMS OF SHEAR RESISTANCE

6.1 Shear Resistance in Plastic Hinge Regions

Tests [1] have demonstrated that cyclic flexure in plastic hinge regions of members can cause a degradation of the shear carried by the conventional shear resisting mechanisms. This is because full depth flexural cracks can exist in the plastic hinge regions, as well as inclined diagonal tension cracks, during much of the reversed loading range. This occurs because when longitudinal steel yields in tension for loading in one direction, open cracks will be present in the concrete "compression" zone when the load is applied in the opposite direction. These cracks will remain open until that steel yields in compression and allows the cracks to close and the concrete to carry some compression (see Fig.8). Thus for parts of the loading cycles the bending moment will be carried by a steel couple alone. If the shear stress at the section is high a sliding shear deformation can occur along a full depth vertical crack (see Fig.8) and the load-deflection hysteresis loops for the structure will show a strength and stiffness degradation.

(a) Moment Reduction Criterion
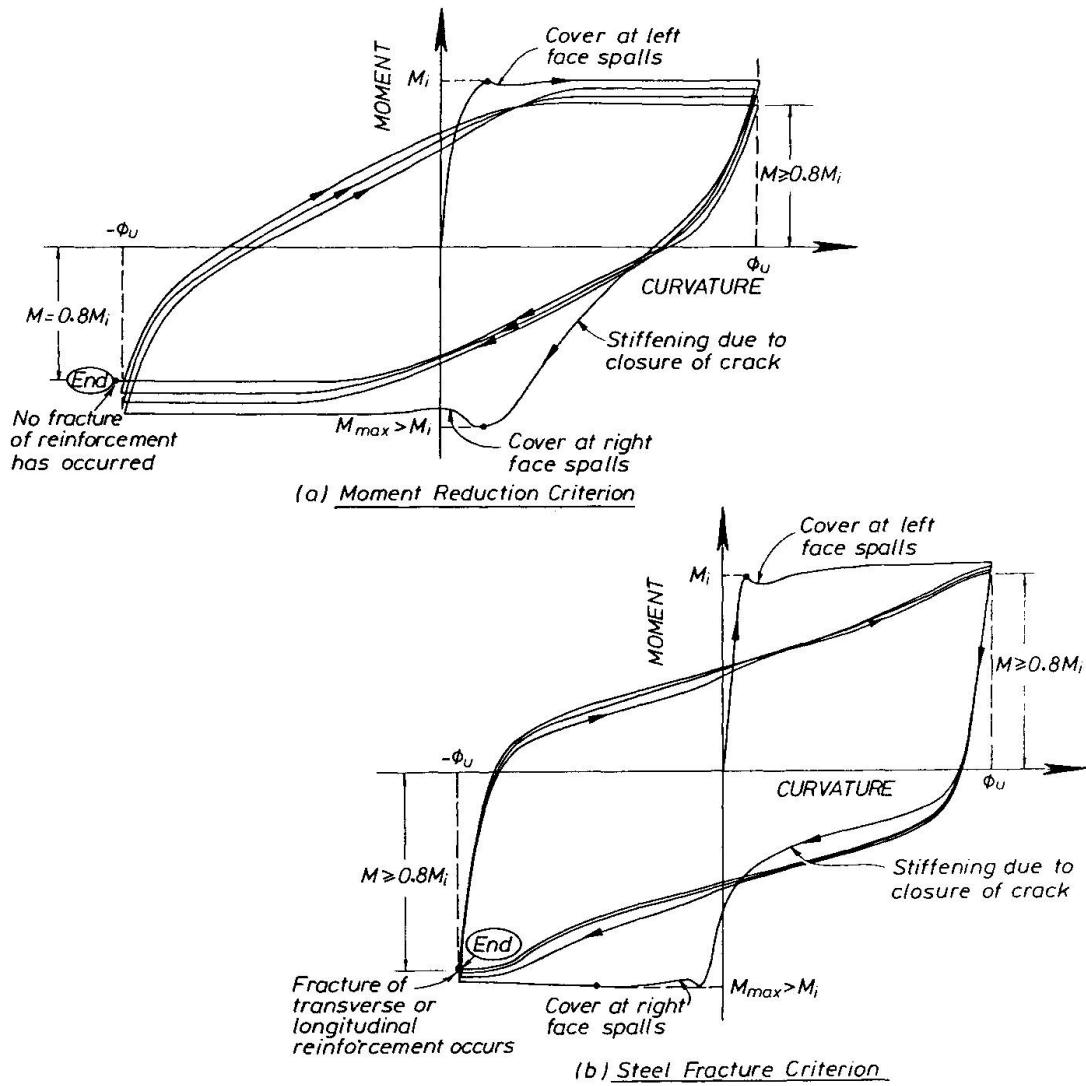
(b) Steel Fracture Criterion

Fig.7 Ductility Evaluation Based on Postpeak Cyclic Moment-Curvature Analysis [7]
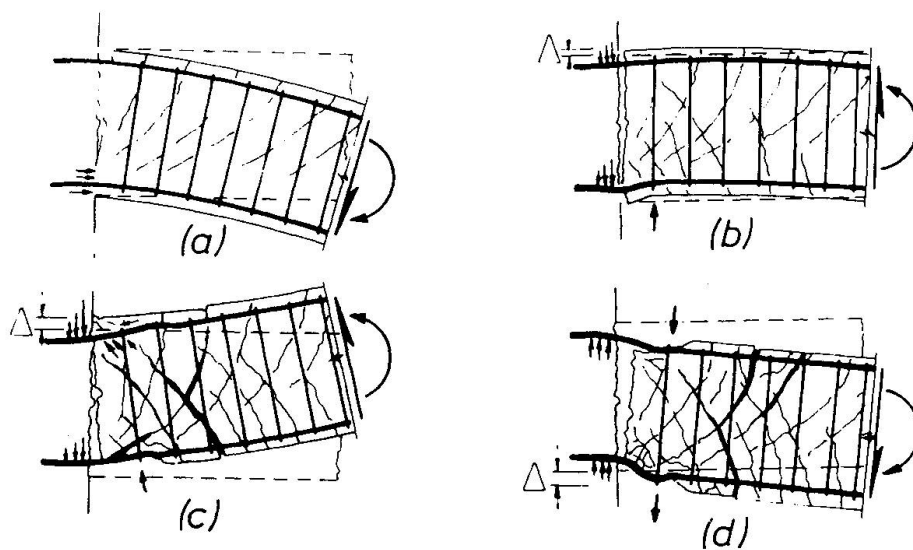

Fig.8 Significant Stages of Development of Deformations at a Plastic Hinge
During Cyclic Flexure with High Shear [8]

6.2 Shear and Bond Resistance in Beam-Column Joints of Moment Resisting Frames

Beam-column joint cores can be subjected to extremely high shear and bond stresses when subjected to earthquake loading. If the beams and columns are detailed for adequate ductility the joint cores could become the critical regions of the structure unless also carefully designed. Fig. 9 illustrates an interior beam-column joint core which forms part of a moment resisting frame subjected to earthquake seismic loading. Consideration of the concrete and steel forces acting at the boundaries of the joint core indicates that, to satisfy the equilibrium requirements of the joint core there must be two mechanisms of joint core shear resistance [1], namely :

(a)   a diagonal compression strut carrying the concrete compressive forces across the joint core.

(b)   a truss mechanism of joint core reinforcement carrying the longitudinal bar forces across the joint core.

(a) Internal Actions and
      Crack Pattern

Transfer of Concrete Forces   Transfer of Some Bond Forces

(b) Shear Transfer of Concrete Compression Forces and
      Some Bond Forces from Longitudinal Bars by
      Diagonal Compression Strut Mechanism.

Remaining Bond Forces   Truss Mechanism Involving
                                            Horizontal and Vertical Reinforcement
                                            and Concrete Struts.

(c) Shear Transfer of Remaining Bond Forces from
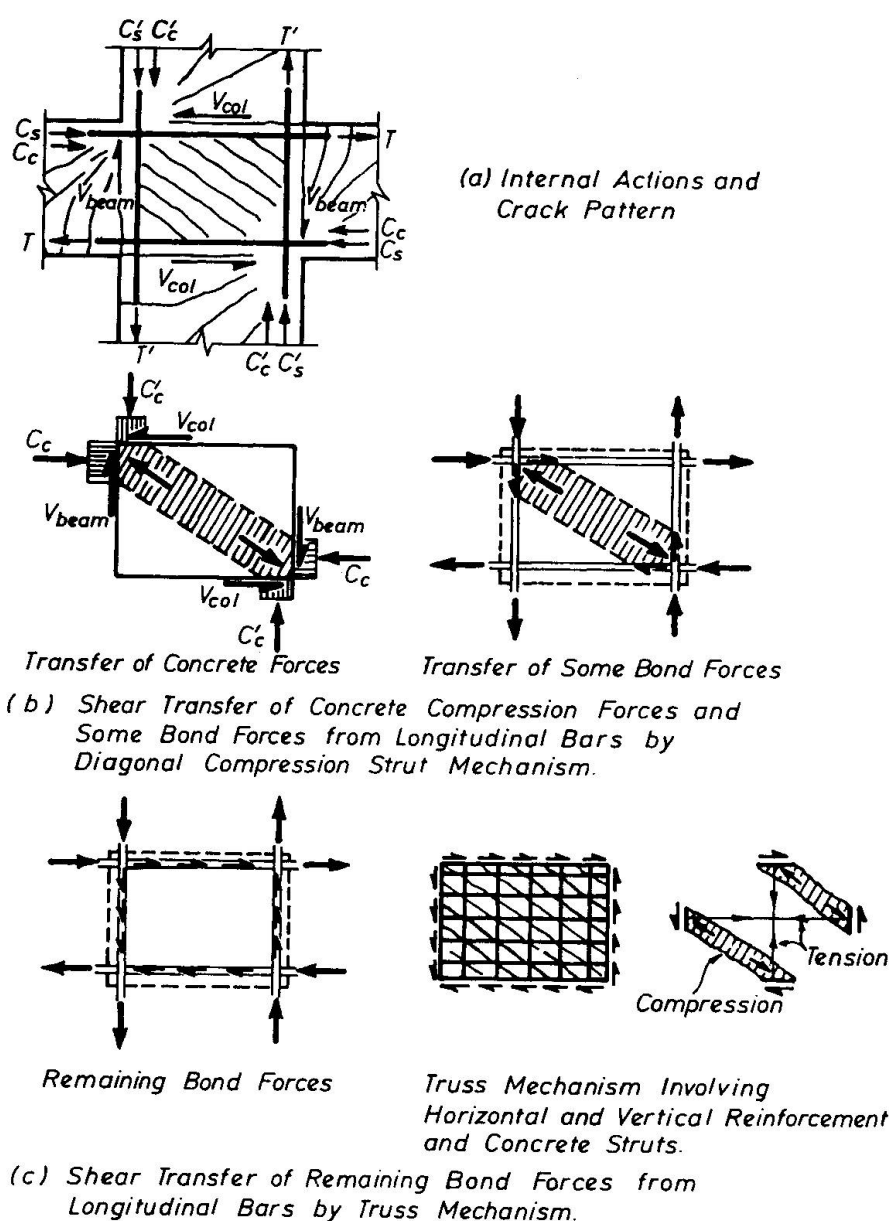   Longitudinal Bars by Truss Mechanism.

Fig.9 : Idealized Behaviour of a Reinforced Concrete Beam-Column Joint of a
         Moment Resisting Frame Subjected to Horizontal Loading [1]

It is evident that the truss mechanism requires the presence of both horizontal and vertical shear reinforcement and a diagonal concrete compression field in the joint core to satisfy the equilibrium requirements of the mechanism (see Fig.9c). This can be provided by horizontal column hoops and intermediate longitudinal column bars.

6.3 Strength and Stiffness Degradation Due to Shear and Bond Mechanisms

Significant postelastic deformations due to shear or bond mechanisms lead to severe degradation of strength and stiffness and to pinched hysteresis loops with reduced energy dissipation. Fig.10 shows typical measured experimental load-displacement hysteretic behaviour of two reinforced concrete beam-column assemblies, one controlled by ductile flexural plastic hinging in the beams (Fig.10a) and the other controlled eventually by bond slip of longitudinal beam bars through the joint core (Fig.10b), and a structural wall controlled by shear mechanisms (Fig.10c).

The extent to which shear and bond should be permitted to participate in the hysteretic behaviour is still a controversial matter. Although some variations in hysteresis loop shape may not be a major influence on the postelastic dynamic response of structures subjected to earthquake excitation, there is no doubt that it is much easier to repair flexural damage occurring at a well detailed plastic hinge in a member than to repair damage resulting from inelastic shear and bond mechanisms. Earthquake design codes in New Zealand [3,4] use the capacity design procedure to ensure that yielding of longitudinal flexural reinforcement occurs rather than inelastic shear and bond mechanisms.

(a) Reinforced Concrete Beam-Column Assemblage Controlled by Ductile Flexural Plastic Hinging in the Beams

(b) Reinforced Concrete Beam-Column Assemblage Eventually Controlled by Bond Slip of Longitudinal Beam Bars through Joint Core

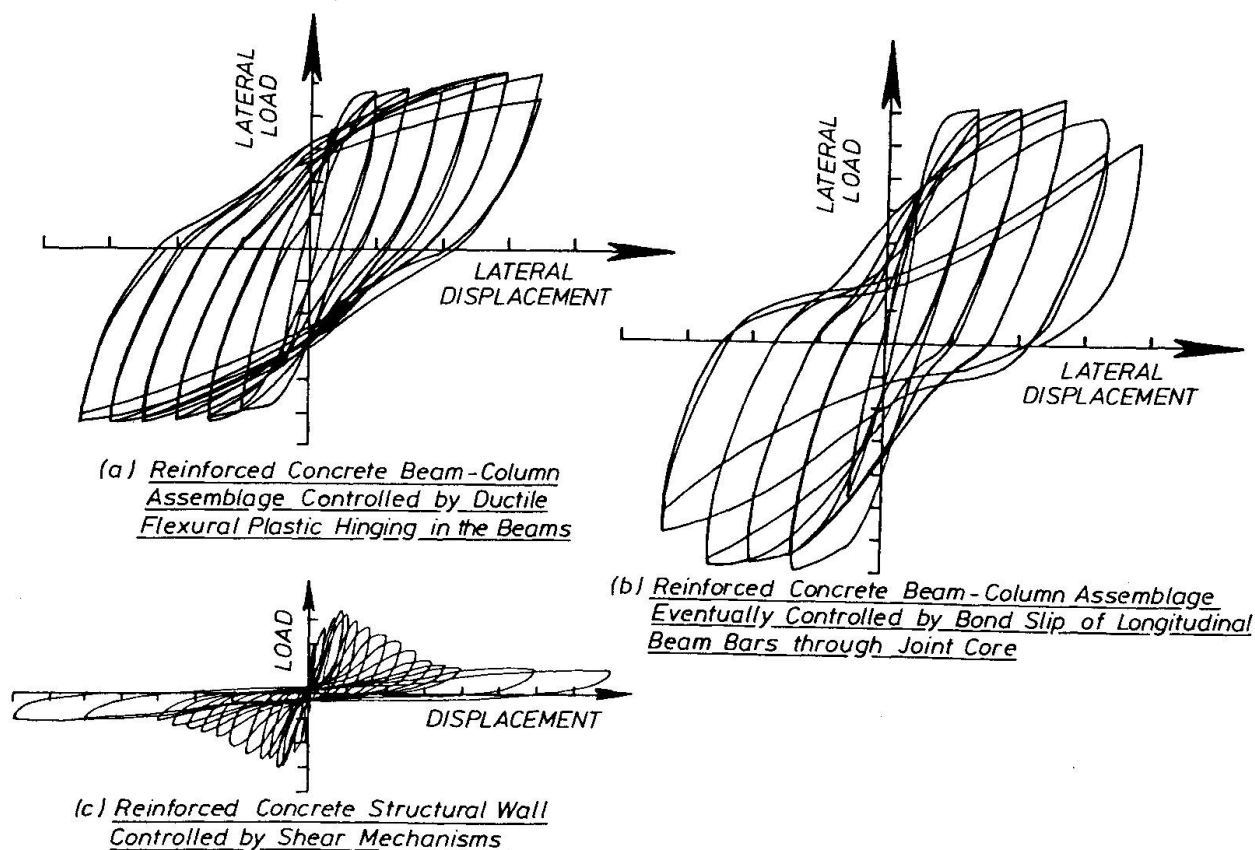(c) Reinforced Concrete Structural Wall Controlled by Shear Mechanisms

Fig.10 - Typical Measured Hysteresis Loops for Reinforced Concrete Subassemblages

7. CONCLUSIONS

1.   Evaluations of ductility have sometimes been confusing in the past due to the various possible definitions for the yield deformation and the maximum available (ultimate) deformation. It is suggested that the yield deformation should be estimated from an equivalent elastoplastic system with elastic stiffness which includes the effects of cracking and with the same ultimate load as the real system. The maximum available (ultimate) deformation should be estimated as that postpeak deformation when load has reduced by a small specified amount, or when the reinforcement fractures or buckles, whichever occurs first.

2.   The ductility of concrete structures required for earthquake resistance is best achieved by ensuring in design that it occurs by flexural yielding of plastic hinges. The longitudinal reinforcing steel should have a suitably large elongation at fracture and should be adequately restrained by transverse reinforcement so as to avoid premature buckling. The ductility and strength of compressed concrete can be significantly improved by the presence of well detailed arrangements of transverse reinforcement. The stress-strain relation of confined concrete can be written as a function of the quantity and arrangement of transverse reinforcement. Analytical procedures are available to determine the quantity of transverse reinforcement required to achieve specified levels of curvature ductility.

3.   In the design of ductile structures for earthquake resistance, failure modes to be prevented are those due to diagonal tension or diagonal compression caused by shear, excessive plastic hinge rotation of heavily loaded columns, sliding shear along jointing faces or in plastic hinge regions, and bond failures along longitudinal reinforcement. All of these undesirable failure modes lead to premature strength degradation and reduced ductility and can be avoided by use of the capacity design procedures.

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