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State-of-the Art Report
DUCTILITY EVALUATION FROM LABORATORY AND ANALYTICAL TESTING

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SUMMARY

Definitions for the required and available ductility in terms of displacements, rotations and curvatures are discussed. Methods for estimating the yield deformation and the maximum available deformation are described and suggestions are made for appropriate definitions. Examples are given of different imposed histories of inelastic displacement which have been used in the experimental testing of structures and structural assemblages in which cycles of quasi-static loading are applied. Analytical procedures for the ductility evaluation of the plastic hinge regions, including the enhancement of ductility due to transverse reinforcement, are discussed.

INTRODUCTION

The term "ductility" in seismic design is used to mean the ability of a structure to undergo large amplitude cyclic deformations in the inelastic range without a substantial reduction in strength. Ductile structures are generally able to dissipate significant amounts of energy during those cyclic deformations. The ductility required of a structure responding to a major earthquake can be estimated by nonlinear time-history dynamic analysis. Procedures for evaluating the available ductility of structural members and their connections are of importance to enable designers to ensure that structures have adequate available ductility to match the required ductility.

This report focusses on definitions which enable the determination of the required and available ductility, and methods for evaluating and enhancing ductility and designing for ductility levels.

DEFINITIONS FOR REQUIRED DUCTILITY

In the non-linear time-history dynamic analysis of structures responding to a major earthquake in the inelastic range it is usual 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 inelastic deformation for seismic design and analysis. Ductility factors have been commonly expressed in terms of the various response parameters related to deformations, namely displacements, rotations and curvatures.

The displacement ductility factor, $\mu = \Delta_{max}/\Delta_y$, where Δ_{max} is the maximum displacement and Δ_y is the displacement at yield, is the value normally determined in inelastic time history dynamic analyses. The displacement ductility factor μ is shown defined for ideal elasto-plastic behaviour in Fig.1.

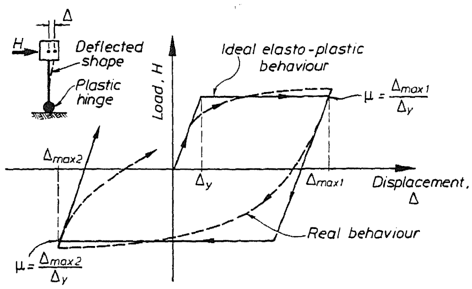


Fig.1 Displacement Ductility Factor

Dynamic analyses also commonly determine the rotation ductility factor required of members 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 structural 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.

It needs to be recognized that there can be significant numerical differences between the magnitudes of the required displacement, rotation and curvature ductility factors. This is because once yielding has commenced in a structure the deformations concentrate in the yielding regions. For example, for reinforced concrete moment resisting frames the required ϕ_{max}/ϕ_y at the plastic hinges may be several times the required Δ_{max}/Δ_y for the structure (Ref.1).

The displacement ductility factor required of typical code-designed structures may vary between 1 for elastically responding structures to as high as 7 for ductile structures, but is typically in the range 3 to 6.

DEFINITIONS FOR AVAILABLE DUCTILITY

The ductility required of the structure during response to a major earthquake 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.

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 longitudinal bars at different depths in a reinforced 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 elasto-plastic system with the same elastic stiffness and ultimate load as the real system (Fig.2b), the yield displacement of the equivalent elasto-plastic system with the same energy absorption as the real system (Fig.2c)(Ref.2), and the yield displacement of the equivalent elasto-plastic 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)(Ref.3) 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.

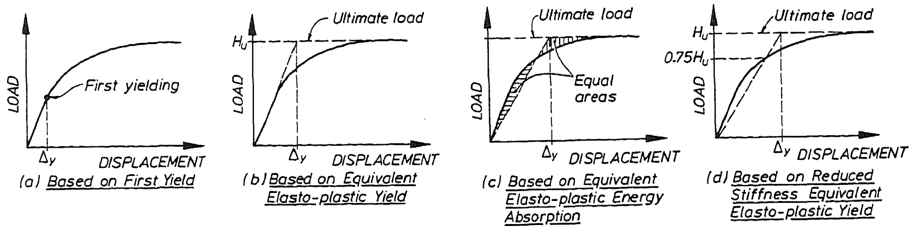


Fig.2 Alternative Definitions for Yield Displacement

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 post-peak 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 post-peak 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. The definition for maximum available deformation could also include a cyclic loading parameter, such as the maximum deformation when after 4 cycles of loading to that deformation the load carrying capacity has reduced by a small specified amount or the reinforcement has fractured or buckled.

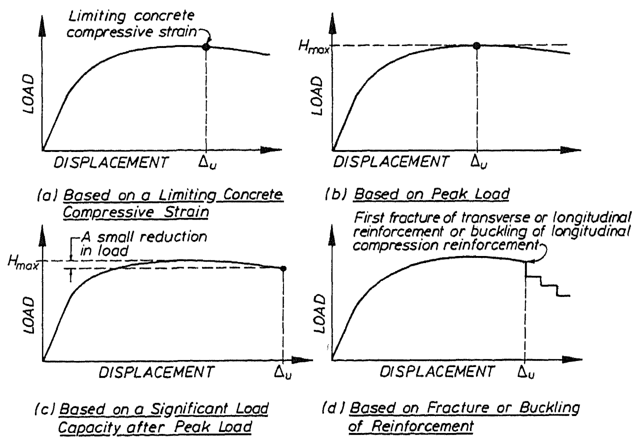


Fig.3 Alternative Definitions for Maximum Available (Ultimate) Displacement

Definition of Available Ductility Factor The available displacement ductility factor, rotation ductility factor and curvature ductility factor can be written as Δ_u/Δ_y, θ_u/θ_y and ϕ_u/ϕ_y, respectively, where the maximum available (ultimate) and yield quantitites are defined as in Figs. 2 and 3.

Cumulative Ductility Factor The cumulative ductility factor undergone by a structure during cycles of reversed loading is also of interest when assessing the effects of several cycles of reversed loading. For example a structure subjected to 4 cycles of loading to displacement ductility factors of 4 in each direction would undergo a cumulative displacement ductility factor of $\Sigma\mu = 32$. Care should be taken when assessing the effects on the structure of cumulative ductility factors. For example, 16 cycles of loading to $\mu = 1$ in each direction can result in significantly less damage to the structure than 2 cycles to $\mu = 8$ in each direction, although both loading histories give a cumulative displacement ductility factor of $\Sigma\mu = 32$.

A Hysteretic Energy Dissipation Index As illustrated in Fig.1, the behaviour of a real structure usually varies significantly from elasto-plastic behaviour. Hence, since ductility factors are written only in terms of deformations, they do not give a measure of the energy dissipation of real structural systems. Mahin and Bertero (Ref.2) have defined an index which measures the total hysteretic energy dissipation, which could be useful for systems which substantially degrade in stiffness and/or strength. Their energy dissipation index is the ratio of the total energy dissipated by the real system to the total energy dissipated by the elasto-perfectly plastic system with the same yield strength, when both systems are subjected to cyclic loading with the same imposed ductility history, where the energy dissipated is the area within the hysteresis loops of the loading cycles.

EXPERIMENTAL METHODS FOR DUCTILITY EVALUATION

The experimental testing of structures and structural assemblages in laboratories, to assess performance and available ductility during major earthquakes, requires decisions concerning the appropriate displacement history to be imposed to simulate seismic loading.

Shake Table Testing Shake table testing, with the table following the motions of a recorded earthquake, is a realistic experimental method for assessing the performance and the required and available ductility of structural systems. A major limiting factor is the mass, size and strength of structure that can be tested since these will depend on the capacity of the table. Often only scale models can be tested and scaling of the earthquake record may also be necessary.

Pseudodynamic Testing Pseudodynamic testing is an alternative which retains the realism of shake table testing but has the convenience of conventional quasi-static loading tests (Ref.4). In pseudodynamic testing experimental measurements are made of the restoring forces of the structure at each step during the testing, and this direct experimental feedback is used to calculate by inelastic dynamic computer analysis the displacements to be imposed on the structure by hydraulic actuators to closely resemble those that would occur if the building was subjected to the ground shaking of a particular earthquake.

Quasi-Static Load Testing Most experimental testing of structures and structural assemblages has used quasi-static cyclic loading, applied by hydraulic actuators, which has not attempted to follow the strain rate or the specific displacement history imposed by a particular earthquake. Instead the structure is subjected to predetermined numbers of displacement controlled quasi-static loading cycles to predetermined displacement ductility factors. The slow strain rate means that the test may take several days to conduct.

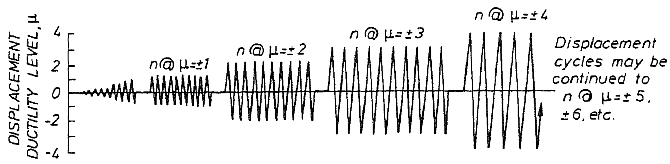
Time-history dynamic analyses of code-design structures responding inelastically to major earthquakes can be used to obtain a guide as to the quasi-static loading history to be applied. For example, Mahin and Bertero (Ref.5) have found that the number of yield excursions tends to increase with decreasing period of vibration, except in the very short period range. Yielding has been found to occur about the same number of times in each direction, but the maximum displacement is generally larger in one direction than the other. For stiffness degrading single-degree-of-freedom-systems, designed for a seismic force level corresponding to a displacement ductility factor of 4, responding to severe earthquake ground motions recorded on firm ground at moderate epicentral distances, such as the 1940 El Centro earthquake, the number of yield reversals in each direction did not generally exceed 4. For the unrealistic elasto-perfectly plastic systems the number of yield excursions in each direction varied from about 15 for a period of 0.2 seconds to about 3 for a period of about 2 seconds. It is to be noted that the major earthquake which occurred in Mexico City in September 1985 had most of its energy in relatively long period ground motions and the duration of the strong earthquake shaking was exceptionally long. Hence the number of yield excursions for that Mexico City earthquake would be expected to be several times that of the 1940 El Centro earthquake.

Quasi-static load testing gives conservative estimates of the real strength of the structure or structural assemblage, since earthquake loads are dynamic and an increase in the strain rate results in an increase in the strength of concrete and steel. However significant differences between the shapes of the hysteresis loops obtained from quasi-static and dynamic loading tests may not be observed. For example Iwasaki, et al (Ref.6) concluded that the effect of loading velocity on energy dissipation of reinforced concrete columns was less significant for displacements of 4 times that a first yield of the longitudinal reinforcement, but at higher displacements the energy dissipation capability was appreciably larger when the loading velocity was 100 cm/sec than when the loading velocity was 10 cm/sec.

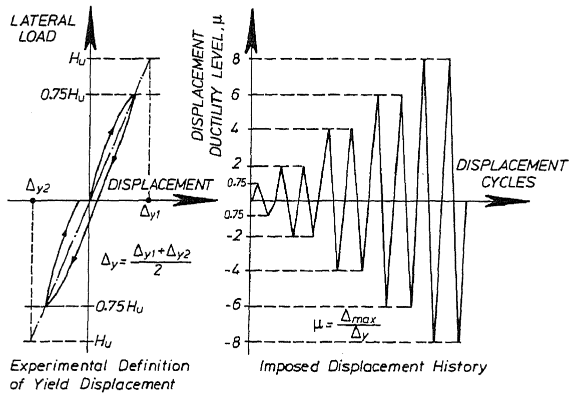
In quasi-static load testing the displacement history does not follow in detail the complex response of a structure to an actual earthquake. Instead a more simple displacement history is applied to enable an assessment to be made as to whether the structure is tough enough to be likely to perform satisfactorily during a major earthquake. Unfortunately, investigators in the past have used a range of displacement histories and various definitions of yield deformation which have made the comparison of results of different investigations difficult. As a result, values for ductility obtained from experimental tests have sometimes been misused in judging the likely performance of structures during major earthquakes. Agreement is needed for appropriate definitions of the main parameters describing inelastic behaviour for quasi-static load testing so that performance obtained from analytical and experimental investigations can be properly assessed and compared in terms of their application to the design of structures for earthquake resistance.

<u>Examples of Quasi-Static Cyclic Loading in Terms of Displacement Ductility</u> As an example of a quasi-static loading history, the Commentary of the New Zealand general design code (Ref.7), which incorporates a displacement ductility factor of about 4 for ductile structures, recommends as an approximate criterion for the adequate ductility of moment resisting frames that the structure should be able to undergo 4 loading cycles to a displacement ductility factor of 4 in each direction without the horizontal load carrying capacity reducing by more than 20%.

A quasi-static loading pattern which has been used for column tests at the Construction Technology Laboratories, Skokie, USA (Ref.8) and at the Public Works Research Institute, Ministry of Construction, Japan (Ref.6) is shown in Fig.4a. The displacement Δ_y has been taken as the displacement corresponding first yield of the outer longitudinal reinforcing bars. The ductility level is increased step-wise and the number of symmetrical loading cycles at each ductility level has been n = 2 to 10, typically n = 2 in the US and n = 10 in Japan. A quasi-static loading pattern which has been used for many years at the University of Canterbury, New Zealand, (Ref.3) is shown in Fig.4b. The yield displacement is found using the mean measured secant stiffness at 0.75 of the theoretical ultimate load as illustrated in the figure. Again the ductility level is increased step-wise and two symmetrical loading cycles have been applied at each level. Sometimes the ductility levels have been increased in steps of 2 cycles to μ = ±1, ±2, ±3 etc. if limited ductility is expected.



(a) Tests at the Construction Technology Laboratories, Skokie, USA and at the Public Works Research Institute of the Ministry of Construction, Tsukuba, Japan.



(b) Tests at the University of Canterbury, New Zealand

Fig.4 Examples of Displacement Histories Used for Quasi-Static Cyclic Loading Tests of Columns and Structural Subassemblages

A more detailed quasi-static loading history used for seismic load tests involving bi-directional earthquake loading is that agreed to by the principal investigators of the US-New Zealand-Japan-China collaborative research project on the seismic design of reinforced concrete beam-column joints (Ref.9). Again the yield displacement is determined to be 1.33 times the displacement measured at 0.75 of the theoretical ultimate load. The displacement controlled loading history imposed is illustrated in Fig.5 for the first 12 cycles. Obtaining that international agreement was a major step forward and has permitted proper comparison of the performance of the structures tested in the four countries.

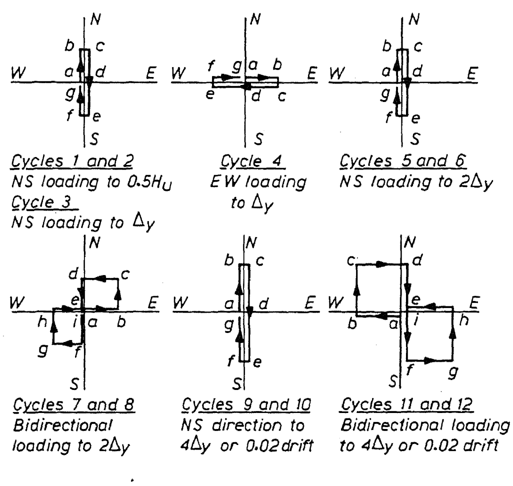


Fig.5 Bidirectional Displacement History Used for Quasi-Static Cyclic Loading Tests of the US-New Zealand-Japan-China Collaborative Research Project on Reinforced Concrete Beam-Column Joints

The above quasi-static loading histories used in New Zealand and in the US-NZ-Japan-China collaborative research project are suitable for earthquakes of typical duration. For a long duration earthquake, such as the September 1985 Mexico City event, more loading cycles at each ductility level would be necessary.

Care should be taken to ensure that the test structure or structural subassemblage is adequately stiff to satisfy the code limitations for interstorey displacements. If the test structure or assemblage is overly flexible the level of interstorey displacement required to achieve a given displacement ductility level may be unrealistically large.

Example of Quasi-Static Cyclic Loading in Terms of Interstorey Displacement It has been suggested by some investigators that the imposed deformation history should be based on the level of interstorey displacement rather than the level of displacement ductility factor. Zhu and Jirsa (Ref.10) have suggested that if the test structure or structural subassemblage can withstand imposed displacement cycles of up of $0.03\ell_c$ in each direction, where ℓ_c is the storey height, without substantial loss in strength, the structure is satisfactory. The concept of using this maximum interstorey displacement criterion has considerable merit since it avoids the difficulty of the definition of the yield displacement.

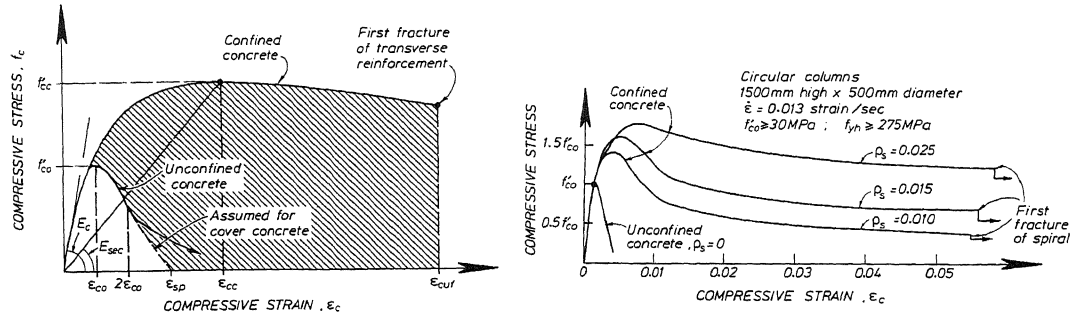
ANALYTICAL METHODS FOR DUCTILITY EVALUATION

Analytical Procedures Definitions which can be used for the yield deformation and the maximum available (ultimate) deformation in analytical methods are similar to those illustrated in Figs. 2 and 3.

For structures in which ductility is controlled by flexural plastic hinging of members the available displacement ductility factor will be limited by the available (ultimate) curvature ductility factor. The relationship between displacement ductility of the structure and the curvature ductility at the plastic hinges can be determined considering the geometry of the deformation of the structure, providing that the equivalent plastic hinge length, over which the ultimate curvature can be considered constant (Refs. 1 and 3), is known. In recent column tests (Ref.3) the equivalent plastic hinge length taking into account the spreading of plasticity due to bond deterioration and diagonal tension cracking was found experimentally to be on average close to $\ell p = 0.5h$ where h is the column depth. The plastic hinge rotation is given by $\theta_p = (\phi_u - \phi_y)\ell_p$.

Moment-curvature analyses can be used to determine the maximum available curvature ductility of structural concrete sections. The moment-curvature relations are dependent on the stress-strain characteristics of the reinforcing steel and the compressed concrete. In particular, the maximum available (ultimate) curvature of a beam or a column with a large neutral axis depth is dependent on the available ductility of the concrete. Also, in all cases the reinforcing steel needs to be capable of large plastic elongation, otherwise the ductility of the member may be restricted by the fracture strain of the steel.

Ductility Enhancement by Concrete Confinement The ductility 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 or circular hoops or rectangular hoops with adequate cross ties. Typical results are shown idealized in Fig.6, which compares the longitudinal stress-strain curves for confined concrete with the curves for identical but unconfined concrete. 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 (Refs. 3 and 11). 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.



(a) Idealized (b) Measured Experimentally

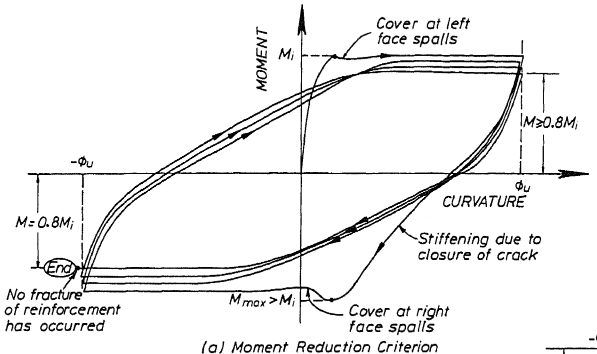
Fig.6 Typical Compressive Stress-Strain Curves for Confined and Unconfined Concrete (Ref.11)

It is evident that moment-curvature analyses incorporating models for the stress-strain curve 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 analytical procedure is the basis of several approaches which have been proposed for evaluating the ductility of structural concrete members.

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 relationship which in turn depends on the transverse reinforcement and the strength of the unconfined concrete, and the steel stress-strain relationships. 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. However the approach has the merit of simplicity, since the maximum available (ultimate) curvature can be readily calculated from $\phi_u = \epsilon_{cu}/c$, where c is the neutral axis depth at that stage.

An example of this approach is the method proposed by Muguruma, et al (Ref.12). On the basis of tests conducted on confined concrete specimens an idealized stress-strain curve for confined concrete was proposed. Then ϵ_{cu} was defined as the strain at the extreme compression fibre of the member in flexure when the mean stress in concrete compressive stress block, as found from the stress-strain curve, was a maximum. When compared with test results from flexural members the approach was shown to give a conservative estimate of the maximum available (ultimate) curvature.

Approach for Ductility Evaluation Based on the Post-Peak Moment-Curvature Behaviour A more recent evaluation method which has been proposed 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 (Ref.13). On the basis of tests conducted on confined concrete specimens cyclic stress-strain relations for confined concrete were determined (Ref.11) 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 and flexural strength of reinforced concrete columns containing various arrangements and quantitites of transverse reinforcement. 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). The ideal moment was the maximum moment reached, which could be significantly higher than the code calculated value, due to the enhancement of concrete strength resulting from confinement and of steel strength due to strain hardening. The maximum available curvature was increased incrementally until an ultimate limit condition, as defined above, was reached. Design charts have been prepared which relate the maximum available curvature ductility factor ϕ_u/ϕ_y to the column axial load level and the magnitude of the confining stress from the transverse reinforcement.



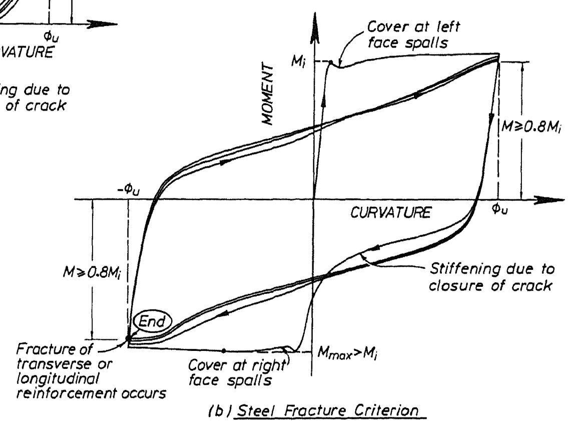


Fig.7 Ductility Evaluation Based on Post-Peak Cyclic Moment-Curvature Analysis (Ref.13).

EFFECT OF SHAPE OF HYSTERESIS LOOP ON RESPONSE

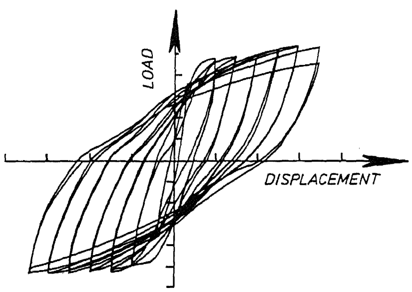
Fig.1 indicates that real load-deformation behaviour of structural concrete members varies significantly from ideal elasto-perfectly plastic behaviour. A number of shapes of hysteresis loops have been used to model the cyclic moment-curvature behaviour of reinforced concrete members, to be utilized in inelastic time-history dynamic analyses, such as bilinear with variable post-yield stiffness, Ramberg-Osgood, and stiffness degrading idealizations. Several investigators have studied the influence of the shape of the hysteresis loops on the response of structures to major earthquakes. It is difficult to reach specific conclusions due to the large number of variables involved.

Structures can undergo significant stiffness degradation when cycled in the inelastic range. However, on average, the differences in the ductility demand for elasto-perfectly plastic systems and stiffness degrading systems found by Mahin and Bertero (Ref.5) and Moss, et al (14) were small, except perhaps for short period structures where the ductility demand of the degrading stiffness system may be larger. Degrading stiffness systems were found to dissipate hysteretically about the same amount of energy as elasto-perfectly plastic systems, even though they do not reach their full strength as often (Ref.5). This is because energy is dissipated hysteretically by the elasto-perfectly plastic system only when the full strength is reached, but for the stiffness degrading system (for example, shown as real behaviour in Fig.1) energy is dissipated due to non-linear behaviour in almost all cycles after first yield.

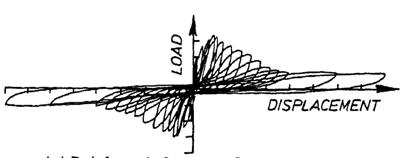
However Mahin and Bertero (Ref.5) have found that bilinear hysteretic loops with even a small negative post-yield slope (-5%) can substantially increase the ductility demand, particularly for short period structures and long duration earthquakes. It should be noted though that the bilinear model is not typical of real behaviour of structural concrete members. Stiffness degrading models are more typical and a reduction of strength seen in hysteresis loops generally occurs as an overall reduction in strength (as in Fig.7) or only at the end of the post-yield load-deformation branch. Moss, et al (Ref.14) have found using elasto-perfectly plastic hysteresis loops that strength degradation to 80% of the initial strength during seismic excitation in the inelastic range did not significantly influence the displacement response.

Prestressed concrete members have significantly narrower moment-curvature hysteresis loops, and hence lower hysteretic energy dissipation, than reinforced concrete members. The maximum displacements reached by code-designed prestressed concrete single-degree-of-freedom systems has been found to be on average approximately 30% greater than reinforced concrete systems of similar strength, initial stiffness and viscous damping, when responding to major earthquakes (see Ref.15).

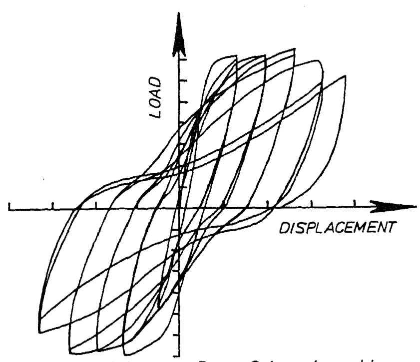
Significant inelastic deformations due to shear or bond mechanisms lead to severe degradation of strength and stiffness and to pinched hysteresis loops with reduced energy dissipation. Fig.8 shows typical measured experimental load-displacement hysteretic behaviour of two reinforced concrete beam-column assemblies, one controlled by ductile flexural plastic hinging in the beams (Fig.8a) and the other controlled eventually by bond slip of longitudinal beam bars through the joint core (Fig.8b), and a structural wall controlled by shear mechanisms (Fig.8c). Kitayama, et al (Ref.16) have investigated the inelastic



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



(c) Reinforced Concrete Structural Wall Controlled by Shear Mechanisms



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

Fig.8 Typical Measured Hysteresis Loops for Reinforced Concrete Subassemblages dynamic response of 4, 7 and 16 storey moment resisting frames with the plastic hinge behaviour in the beams modelled by stiffness degrading hysteresis loops with and without pinching behaviour caused by bond deterioration. The effect of significant pinching of the hysteresis loops on the response was found to be relatively small and it was concluded that some bond deterioration of beam bars within a beam-column joint may be tolerable. 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 inelastic 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.

CONCLUSIONS

1. Ductility factors of structures expressed as the maximum deformation divided by the corresponding deformation at yield are useful nondimensional indices of inelastic deformations. Ductility factors can be defined in terms of the required ductility during major earthquakes and the available ductility, and can be expressed in terms of displacements, rotations and curvatures.

2. Values of ductility factor have sometimes been misused in the past due to the various current definitions for the yield deformation and the maximum available (ultimate) deformation for structural concrete members when the shapes of the load-deformation hysteresis loops are not elasto-perfectly plastic. Agreement is needed as to the definitions of "yield" and "maximum available (ultimate)". It is suggested that the yield deformation should be estimated from an equivalent elasto-perfectly plastic 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 post-peak deformation when load has reduced by a small specified amount, or when the reinforcement fractures or buckles, whichever occurs first.

3. In the past experimental testing of structures and structural assemblages in which cycles of quasi-static loading are applied, has involved the use of many different inelastic displacement histories. The quasi-static loading history agreed to by the principal investigators of the US-NZ-Japan-China collaborative research project on the seismic design of reinforced concrete beam-column joints, for testing involving bidirection earthquake actions, represents a substantial step forward in agreement.

4. Experimental testing using imposed displacements based on the percentage of the storey height (that is, the drift) rather than displacement ductility factor has considerable merit since it avoids the difficulty of defining the yield deformation.

5. The ductility of concrete structures 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.

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