Bull. Ind. Soc. Earthq. Tech. Paper No.283, Vol.27, No.2, June 1990, pp.30-45.

SEISMIC DESIGN OF REINFORCED CONCRETE FRAMED STRUCTURES

By

S.K. Thakkar
Professor, Earthquake Engg., University of Roorkee

ABSTRACT

The development of safe and economic design of a reinforced concrete framed structure to withstand a severe earthquake motion is a challenging problem for a designer. There is a significant advancement in the earthquake resistant design practice of RC framed structures through the concept of ductility. Design of structure for ductility involves introducing energy/dissipating characteristics in it in a controlled manner. Though the concept of ductility in seismic design is known for over four decades now yet there are inadequacies in ductility based design. This paper presents state of art in the ductility based design approach of RC structures. The importance of definitions of ductility, methods of evaluating ductility and need of ductility and proper use in earthquake resistant design and its other aspects are highlighted. The research and development needed for improvement in ductility based design and modifications in this approach are also presented.

INTRODUCTION

It is very well recognized that most of the destruction and economic loss due to severe earthquake motion is due to failure of inadequately designed Civil Engineering Structures. One of the most effective ways of earthquake disaster mitigation is to improve existing methods of Earthquake Resistant Design (ERD), to develop new and better methods of design, construction and maintenance, and of repairing, retrofitting and maintaining existing buildings. The seismic performance of designed structures should be judged as the test of existing practice of ERD.

Two approaches for safety of structures against earthquakes have emerged over the years (i) to provide energy absorbing and dissipating capability to the structure, known as ductility based design (DBD) (ii) to provide isolation pads between the structure and foundation, thereby introducing a filter, to limit the forces and accelerations experienced by the structure, known as seismic-base-isolation (SBI). These two methods are entirely different in concept, ductility based design has been adopted by the seismic cods of many countries, will be a subject of discussion in this presentation. It is now evident that it would be highly uneconomical to design a structure to withstand the severe earthquake motion without damage. Also, the recent researches on earthquake safety of structure and seismic performance have clearly indicated that ductile structures can significantly dissipate energy in inelastic deformations and servive a severe shock. The inelastic deformation characteristics of structure depends on the elasto-plastic flexural behaviour of constituent members and thus the codes now pay special consideration to the provision of flexural ductility to the structural members and joints.

Key-note address presented, at the National Seminar on **Earthquake Resistant Structures** New Delhi, February, 22-23, 1990.

This paper presents the state of the art in the ductility based design of reinforced concrete framed structures. The importance of definition of ductility, methods of evaluating ductility, proper use of ductility concepts, uncertainties in seismic codes with regard to ductility, ductility enhancement method for attaining efficient earthquake resistant design are highlighted. The recent developments in improving earthquake resistant design procedures are discussed. The suggestions for further research and development are presented.

KEY POINTS OF EARTHQUAKE RESISTANT DESIGN

The essential feature of eartthquake resistant design of a structure is that it should be able to withstand earthquake motion without damage, maintains its function after the earthquake. It is possible to identify six key points for earthquake proof structures (Izumi, 1988) :

1. Select good foundation for the site
2. Make them light
3. Make them strong
4. Make them ductile
5. Shift the natural period of structures from predominant period of earthquake motion
6. Heighten the damping capacity

The ductility based design has been based on the concept (3) and (4). The earthquake protection of structure through seismic base isolation has emerged on the basis of concept (5) and (6).

BRIEF HISTORIC BACKGROUND

The advantage of designing ductile structures in earthquake resistant design was demonstrated in the early 1950. Housner (1956) discussed the use of limit design for earthquake resistant design. The application of ductile concept was presented in 1961 in the PCA manual, Design of Multistoreyed Reinforced Concrete Buildings for Earthquake motions (Blume, 1961). Significant experimental and analytical research efforts have since been devoted to the development of earthquake resistant design methods based on the considerations of strength and ductility. The computer programs had also been developed as early as in 1977 for earthquake resistant inelastic design of reinforced concrete, ductile moment resisting frames based on the concept of ductility (Zagajeski, 1977). There has been significant developments in the design for ductility of concrete members (Park, 1975; Watanabe, 1988). Even though these developments have taken place, the term ductility continues to be an ambiguous parameter because of its different definitions and problems in its quantification. Its value and significance in real behaviour of structure can be quite different. The inelastic design of frames for seismic forces is rarely carried out. Realising this problem special theme session was organised in 9th World Conference on Earthquake Engineering in Tokyo-Kyoto, Japan on August 2-9, 1988 where several aspects of ductility were discussed in different research papers. It was clearly stated (Bertero, 1988) that there is an urgent need to get a world wide agreement on ductility related terms, and of their evaluation and application to earthquake resistant design of structures.

In Mexico and Newzealand building codes have introduced explicity the use of ductility ratio in the estimation of seismic design forces using the limit state design method. In Newzealand seismic code, NZS 4203:1984 is based on capacity design procedure (Paulay, 1988). The ACI 318-83 has introduced detailed seismic design consideration based on ductility concept. In India IS:4326-1976 has given recommendations for ductility based design in construction of reinforced concrete structures.

PHILOSOPHY OF DUCTILITY BASED DESIGN

It is very well recognised now that because of economic reasons the structure is not designed to have sufficient strength to remain elastic in severe earthquake. The structure is designed to possess adequate ductility so that it can dissipate energy by flexural yielding and servive the shock.

In the ordinary reinforced concrete member subjected to the flexural moment. considerable plastic deformation can be obtained after the yielding of tension reinforcement and it attains an available limit when the compression fibre of concrete at the critical section reaches the ultimate value. Figure 1 shows a typical moment-curvature relationship for reinforced concrete beam section showing such a behaviour. Thus ductility is quite attainable characteristics in reinforced concrete section.

DESIGN PRINCIPLES

The general principles in the design for ductility are set out below.

1. The structural layour should be as regular and simple as possible. In general a highly indeterminate but regular rectangular frame will absorb large amounts of energy. The sudden change in stiffness from floor to floor should be avoided. Extra strength should generally be provided in areas of high stiffness.

2. Building as a whole and all of its constituent members should be designed to possess ductility, so that it can dissipate significant amounts of energy in plastic deformations under severe earthquake motions.

3. Non-ductile failure in beams should be avoided.

4. Ductile frames should be capable of dissipating seismic energy in flexural mode at a significant number of beam hinges.

5. Non-ductile failure of columns should avoided. Columns should be designed to have overstrength to avoid the formation of hinges and column hinge mechanisms. Figure 2a and 2b shows the undesirable and desirable mechanisms of failure of a building frame respectively, former imposes a high ductility demand while later makes only a moderate demand on curvature ductility at plastic hinges (Rosenblueth, 1980). This follows the concept of well known strong-column-weak-girder design. However should column hinges form at the top and bottom of a column within a storey, such column hinges should be designed and detailed to be ductile.

6. Failure of beam-column joints because of shear, buckling, bond failure and extensive yielding of reinforcement should be avoided. The strength of a joint should not be less than the maximum strength of the weakest member it connects. The joint should be prevented from becoming major source of energy dissipation in a ductile frame.

7. Both overstrong and understrong elements can prove to be dangerous in seismic design. The presence of overstrong parts of the structure will mean that the curvature ductility demand is concentrated into local region of the structure and may lead to collapse because of high inelastic

deformations enforced there. The weak part of the structure acts as a fuse once the strength of that part of the structure is reached, the rest of the frame may remain in the elastic range. There would be concentration of ductility demand in weak part of the structure.

DEFINITIONS OF DUCTILITY AND RELATED TERMS

There are several definitions of ductility factor, each definition has its own significance. Some definitions are presented below :

Ductility : The term ductility in seismic design implies the ability of a structure to undergo cyclic deformations in the inelastic range without any significant reduction in its initial strength, Fig. 3a.

Ductility factor : It is the ratio of maximum deformation to the initial yield deformation, Fig. 3b.

Displacement ductility factor : The displacement ductility factor is the ratio of maximum displacement to the yield displacement. This value is normally determined by inelastic time history analysis, Fig. 3c.

Cyclic displacement ductility factor : It is the ratio of cyclic displacement to the yield displacement as explained in Fig. 3c (Mahin, 1981). This definition has a significance when significant inelastic reversals occur. Real structures have limited capacities to sustain such cyclic deformations.

Permanent displacement ductility factor : It is the ratio of permanent displacement to displacement at yield Fig. 3c (Mahin, 1981).

Equivalent enetgy dissipation ductility factor : It is a convenient comparative index of hysteretic energy dissipation (Mahin, 1981). This is numerically equal to displacement ductility of a monotonically loaded system that dissipates the same energy, and has same yield strength and initial stiffness, as the actual system, Fig. 3c.

Rotational ductility factor : The rotational ductility factor is defined as the ratio of maximum rotation at plastic hinge to rotation there at yield. This factor is also determined by inelastic dynamic analysis, Fig. 3d.

Curvature ductility factor : The curvature ductility factor is defined as ratio of maximum curvature at plastic hinge to the curvature there at yield. This ductility factor is most needed by the designers, Fig. 3e.

Cumulative ductility factor : The cumulative ductility factor undergone by cycles of reversed loading is 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 factor of 4 in each direction would undergo a cumulative displacement ductility factor of = 32.

A hysteretic energy dissipation index : Mahin and Vertero (1976) have defined an index which measures total energy dissipation, which could be useful for system 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 elastic-perfectly plastic system with the same yield strength, when both systems are subjected to cyclic loading with the same imposed displacement history, where the energy dissipated is the area within the hysteresis loops of the loading cycles.

The difference between available ductility factor should also be noted.

Available ductility factor : The available displacement ductility factor, rotational 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 maximum available and yield quantities are defined in Fig. 4 and 5.

Required ductility factor : The ductility required of a structure responding to a major earthquake can be estimated by nonlinear time-history dynamic analysis. The ductility required needs to be matched with the available ductility in order to ensure that structures have adequate ductility.

DIFFERENCE BETWEEN VARIOUS KINDS OF DUCTILITY FACTORS

It must also be recognized that there can be significant numerical differences between the magnitude of the required displacement, rotation and curvature ductility factors. This is because once the yielding begins in a structure, the deformations concentrate in the yielding regions. For example, for reinforced concrete moment resisting frames the required ϕ_m / ϕ_y at the plastic hinges may be several times the required Δ_m / Δ_y for the structure. The displacement ductility factor required of typical code designed structures may vary typically between 3 and 6. The curvature ductility corresponding to above displacement ductility may vary between 6 and 10. The relationship between displacement ductility factor of the structure and the curvature ductility factor at the plastic hinges can be determined considering the geometry of the deformation of the structure, provided that the equivalent plastic hings length, over which the ultimate curvature can be considered constant is known.

The ductility can also be defined for a member, storey and the structure as a whole. It should be noted that the member ductility factor may be considerably higher than the storey ductility factor, which in turn may be somewhat higher than the overall ductility factor. In order to develop an overall ductility factor of 3 to 6 in a structure, the storey ductility factor may have to vary between 4 to 8 and member ductility factors may lie in the range of 5 to 15.

METHODS OF EVALUATING DUCTILITY FACTOR

The methods of ductility evaluation can be put in two categories : experimental and analytical (Part, 1988).

Experimental Methods : The experimental testing of structures and structural assemblages in laboratories have enabled determination of available ductility in constitutent members and structures. Some methods are briefly described below.

Shake table testing : The model tests under simulated earthquake motion provide reliable approach of determining ductility. A major limiting factor is the mass, size and strength of structure that depends upon table capacity. The time and amplitude scaling of earthquake record may be necessary.

Pseudo-dynamic testing : Pseudo-dynamic testing is alternative approach to shake table testing, in which no shaking of model is as such required. In pseudo-dynamic testing experimental measurements are made of the restoring forces of the structure at each step during the testing, and this 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 : This is widely used testing method of carrying out cyclic load test to determine hysteretic behaviour and ductility of structure and constitutent member. The strain rate and specific displacement history in an earthquake is not simulated in these tests. The investigators in the past have used a range of displacement histories and various definitions of yield deformation and ultimate deformation which has made comparison of results of different investigators difficult. As a result, values for ductility obtained from experimental tests have sometime been misused in judging the likely performance of structures during major earthquakes. Agreement is needed for defining main parameters describing inelastic behaviour for quasi-static load testing so that performance obtained from analytical and experimental investigation can be properly compared.

Analytical methods : The definitions which can be used for the yield deformation and ultimate deformation in analytical methods are similar to those illustrated in Figures 4 and 5. The following methods can be used for ductility evaluation.

Moment curvature analysis : Moment curvature analysis 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 concrete. The moment curvature analysis can also be carried out incorporating models for stress-strain curve of concrete confined by various quantities of transverse reinforcement (Mander, 1988), Fig. 6. Such an analysis in fact permits determining quantity of transverse reinforcement to achieve various curvature ductility levels.

Nonlinear dynamic analysis : The member ductility required of a structure can be determined by carrying out nonlinear dynamic analysis of a multistoreyed frame on the adequate idealization of hysteretic restoring force characteristics of earthquake resistant members and the moment redistribution due to progressive formation of plastic hinges. A number of multistorey frames responding nonlinear to earthquake have been analyzed by methods of nonlinear dynamic analysis, but it is difficult to draw general conclusions. The number of variables involved in determining nonlinear response of multistorey frames is so high that no more than qualitative statements can be made. For example, type of ground motion, yield parameters, shape of hysterisis loop, damping are some variables that can significantly effect the nonlinear response. The nonlinear dynamic analysis permits evaluation of rotational ductility of members, curvature ductility of members and displacement ductility of structure. These ductility ratios may be quite different than each other. It is evident that the curvature ductility is a far meaningful index for member ductility than the rotational ductility because of dependence of θ_y on the loading as well as the member properties.

A comparison of elastic and nonlinear response of a twenty storey building obtained by nonlinear dynamic analysis (Clough, 1966; Newmark, 1970) is given in Fig. 7, which clearly shows a variation of ductility demand of girders and column along the height of the structure.

DUCTILITY - AN AMBIGUOUS PARAMETER

The physical meaning of ductility is clearly understood yet it continues to be an ambiguous parameter because of following reasons:

1) The precise definition of ductility is for elasto-plastic behaviour and monotonic loading while real behaviour can be quite different. It does not include rate effects and reverse load effect on structures, although lately some attempts to include

cyclic effects in ductility definition have been made (Mahin, 1981). The real behaviour of structures could be of Ramberg Osgood for stiffness degrading type, Fig. 8a, b; the beam-column assemblages may have pinched hysteresis loops with reduced energy dissipation, Fig. 8c, d.

2) Its use in behaviour other than elastic-plastic causes ambiguity and confusion.

3) There are different possible variations in its definitions.

4) The quantification is a problem, no precise number can be associated with it. The different types of ductility ratios can have different values even for the same structure. The different ductility factors are not directly related because it is a nonlinear parameter. Ductility depends upon several parameters of design, thus quantifying it often poses a problem.

5) It is not a sole parameter to describe damage. Assigning a number of ductility ratio does not represent the extent of damage.

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 stress-strain curve for confined concrete is shown in Fig. 6. For confined concrete, eventual fracture of transverse reinforcement limits the useful concrete compressive strain, but the values in the range of 0.02 to 0.08 are typically obtained.

The extent of 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 transverse reinforcement. Confining of concrete is one of most practical methods for enhancing the flexural ductility of concrete members.

EFFECT OF SHAPE OF HYSTERESIS LOOP ON DUCTILITY

Figure 3b indicates that real load deformation behaviour of structural members which 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, for inelastic time history analysis such as bilinear with variable post yield stiffness, Ramberg Osgood and stiffness degrading idealizations, Fig. 8a, b.

Structures can undergo significant stiffness degradation when cycled in the inelastic range. However, on an average the differences in the ductility demand for elasto-perfectly plastic systems and stiffness degrading systems found by Mahin and Bertero (1981) were small, except perhaps for short period structures where the ductility demand of degrading systems may be larger. Degrading stiffness systems were found to dissipate hysteretically about the same amount of energy as elasto-plastic systems, even though they do not reach their full strength as often. This is because energy is dissipated hysteretically by the elastic-perfectly plastic system only when full strength is reached, but for the stiffness degrading system energy is dissipated due to nonlinear behaviour in almost all cycles after first yield.

UNCERTAINTY IN CODES WITH REGARD TO DBD

Although many of the world codes have adopted ductility based design for earthquake resistant structures but the recommendations are not explicit. The codes give detailing practice in implicit way. The codes at present do not indicate specifically the level of ductility plastic hinge should be capable of achieving but recommend detailing practice with the aim of ensuring adequate ductility. Rational and reliable method of energy dissipation is not yet included in the codes.

CURRENT PRACTICE IN EARTHQUAKE RESISTANT DESIGN

The current practice of earthquake resistant design consists of following steps :

1) The linear elastic design response spectrum (LEDRS) is first established on the basis of seismicity of site and ground conditions.

2) The inelastic design response spectrum (IDRS) is established from LEDRS using reduction factor (Mahin, 1981). One of the practice is to divide LEDRS ordinates by ductility factor. A value of overall ductility factor between 3 to 6 is adopted for this purpose.

3) The member forces in the frame is computed from linear elastic analysis.

4) The members of the structure are then designed by limit state design procedure. The design principles of ductility are employed and detailing for ductility is carried out.

USE OF DUCTILITY IN ESTABLISHING IDRS

The Linear Elastic Response Spectrum (LEDRS) is often modified using a factor to obtain Inelastic Design Response Spectrum (IDRS) (Bertero, 1988). The simplest method for doing this is to reduce the LEDRS by a factor which is independent of period. The ATC-3 has presented such a procedure. In chapter 4 of ATC-3 Commentary it is stated that R is an emperical response reduction factor intended to account for both damping and the ductility inherent in structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Figure 9a shows IDRS on the basis of ATC. Based on analytical studies, Newmark and Hall (1973) concluded that for short period structures, any significant reduction in design forces required for elastic response would result in unacceptably large ductilities for moderate period structures, the energy absorbed by an inelastic structure at its maximum displacement approximates that absorbed by an elastic system resulting in strength modification factor of $(2\mu -1)^{-1/2}$, and for relatively long period structures, the maximum displacements of elastic and inelastic systems are equal, so a strength modification of $1/\mu$ would be appropriate. Figure 9b shows IDRS obtained by this approach.

NEED FOR DUCTILITY AND ITS PROPER USE IN ERD

It is well known that all the structural members, joints and supports should be designed with largest feasible ductility and stable hysteresis behaviour so that entire structure would display ductile behaviour. There are two reasons for this requirement, (i) it allows structure to develop maximum potential strength, (ii) the large structural ductility allows structure to move as a mechanism under its maximum potential strength. Care should be taken to prevent too large reductions of linear elastic response spectrum (LEDRS) through indiscriminate use of ductility ratio.

CURVATURE DUCTILITY DESIGN OF RC STRUCTURE

In the seismic design of reinforced concrete ductile frames, it is necessary to provide curvature ductility to each critical section of constitutent members so as to satisfy the displacement ductility demand of the structure. Past many researches have shown that the lateral confining of concrete is one of the most practical methods for enhancing curvature ductility of concrete members. Mugurma (1988) on the basis of the idealized stress-strain curve of confined concrete has proposed design procedure of confining reinforcement to provide the required ultimate section curvature in reinforced concrete members. Figure 10 shows the proposed curvature ductility design procedure in simple flow chart. Design chart for obtaining the amount of confining reinforcement has been prepared to achieve specified level of curvature ductility.

SUGGESTED MODIFICATIONS IN DBD

The following steps are suggested for modifications in ductility based design :

1) Establish LEDRS for the site.
2) Establish IDRS from LEDRS using ductility factor.
3) Compute member forces in the structure by linear elastic analysis of frame.
4) Design the members using limit state method following the design principles of ductility.
5) The curvature ductility design should be done to determine confinement reinforcement.
6) The detailing for ductility should be done as per recommended practice.
7) Nonlinear dynamic analysis should be carried out to determine the ductility demand, the available ductility should match with the ductility demand.

The current procedures commonly ignore steps 5 and 7.

ENERGY APPROACH TO ERD

One of the promising approach for earthquake resistant design of future (Bertero, 1988) is energy approach. In this approach, it is recognized that the total energy input, E_I can be resisted by the sum of the kinetic energy E_k, the elastic strain energy E_{ES}, energy dissipated through plastic deformations (hysteretic damping) E_H, and the equivalent viscous damping E_ρ . The energy equation for a single mass vibrating system can be written as,

$$E_I = E_K + E_{ES} + E_H + E_\rho$$

The practical methods of design based on this approach are yet to be developed.

RESEARCH AND DEVELOPMENT IN DBD

It is evident that there are gaps in the knowledge of ductility based design which needs further improvement. The following further research and development effort is suggested.

1. To develop practical methods of earthquake resistant design based on energy approach.
2. More reliable engineering parameters are needed to define damage potential.
3. There is a need for agreement on definition and use of ductility ratio.
4. To improve quantification of ductility ratio.
5. To develop more reliable methods for estimating the values of reduction factor R.

CONCLUSIONS

The following conclusions are derived from the above study :

1. Ductility concept is valuable for safe and economic design of earthquake resiatant structures.
2. Presently ductility is used more in a qualitative manner. It needs quantification, more elaborate design procedures are needed.
3. There is a need for search of a more reliable parameter to define damage potential.
4. There is a need to develop more reliable methods of determining reduction factors for establishing IDRS.

REFERENCES

1. ACI Committee 318 (1983), Building Code requirements for reinforced concrete, ACI 318-83, A.C.I. Detroit, MI48219.
2. Bertero, V.V. (1986), Lessons learned from recent earthquakes on research implications for earthquake resistant design of building structures in U.S., Report No. UCB/EERC-86/03, Earthquake Engineering Research Centre, University of California, Berkeley, California.
3. Bertero, V.V. (1988), Ductility based structural design, Proceedings of 9 WCEE, Aug. 2-9 Tokyo-Kyoto, Japan, Vol. VIII.
4. Blume, J.A., Newmark, N.M. and Corning, L.H. (1961), Design of multistorey reinforced concrete buildings for earthquake motions, Portland Cement Association, Chicago, Illinois.
5. Clough, R.W. and Benuska, K.L. (1966), FHA study of seismic design criteria for high rise buildings, Report HUD TS-3, Fedral Housing Administration, Washington, D.C. Aug.

6. Housner, G.W. (1956), Limit design of structures to resist earthquakes, Proceedings, World conference on earthquake engineering, Berkeley, California.

7. IS:4326-1976 (1976), Code of practice for earthquake resistant design and construction of buildings, BIS, New Delhi.

8. Izumi, Masanori, (1988), Base isolation and passive seismic response control, Proceedings of 9 WCEE, Aug. 2-9, Tokyo-Kyoto, Japan, Vol. VIII.

9. Mahin, S.A. and Bertero, V.V. (1976), Problems in establishing and predicting ductility in structural design, Proceedings of the International Symposium on Earthquake Structural Engineering, St. Louis, Mo., Vol. I.

10. Mahin, S.A. and Bertero, V.V. (1981), An evaluation of inelastic seismic design spectra, Journal of Structural Division of the ASCE, 107, No. ST9, Sept.

11. Mander, J.B., Priestley, M.J.N. and Park, R., (1988), Theoretical stress-strain model for confined concrete, Journal of Structural Engineering, ASCE, ST8.

12. Mugurma, H., Watanabe, F., Nishiyama, M. (1988), Curvature ductility design of reinforced and prestressed concrete members, Proceedings of 9 WCEE, Aug. 2-9, Tokyo-Kyoto, Japan, Vol. VIII.

13. Newmark, N.M. (1970), Current trends in seismic analysis and design of high rise structures, Earthquake Engineering, Wiegel, R.L., Prentice-Hall, Englewood Cliffs, N.J.

14. Newmark, N.M. and Hall, W.J. (1973), Proceedings and criteria for earthquake resistant design, Building Practice for Disaster Mitigation, Building Science Series 45, NBS, Feb.

15. NZS 4203:1984 (1984), Code of practice for general structural design and design loadings for buildings, Standard Association of NewZealand.

16. Park, R. and Paulay, T. (1975), Reinforced concrete structures, John Wiley, New York.

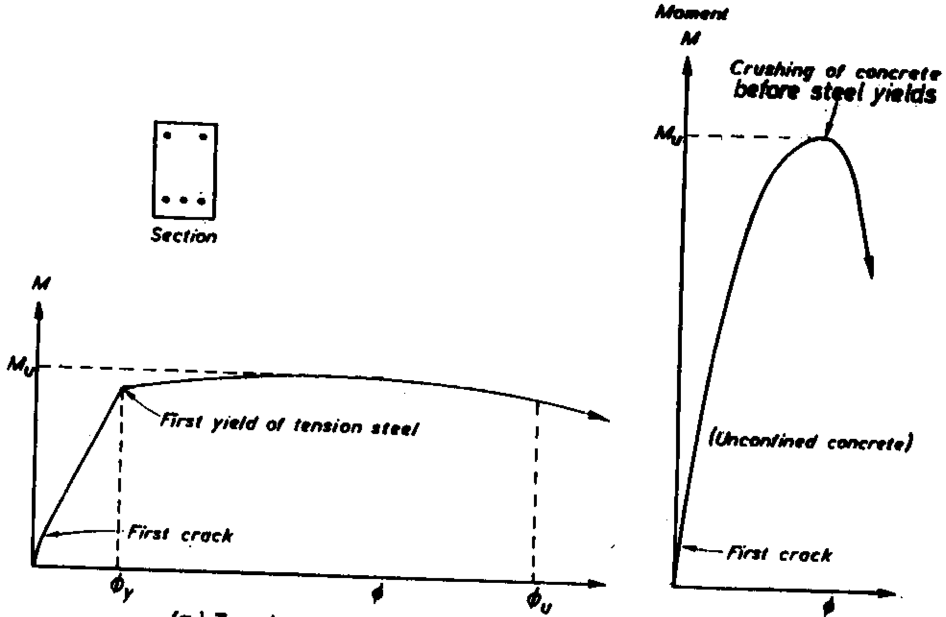
17. Park, R. (1988), Ductility evaluation from laboratory and analytical testing, Proceedings of 9 WCEE, Aug. 2-9, Tokyo-Kyoto, Japan, Vol. VIII.

18. Paulay, T. (1988), Seismic design in reinforced concrete the state of the art in NewZealand, Proceedings of 9 WCEE, Aug. 2-9, Tokyo-Kyoto, Japan, Vol. VIII.

19. Rosenblueth, Emilio (1980), Design of earthquake resistant structures, Pentech Press.

20. Wantanabe, F. (1988), Shear design of concrete members to meet ductility requirement, Proceedings of 9 WCEE, Aug. 2-9, Tokyo-Kyoto, Japan, Vol. VIII.

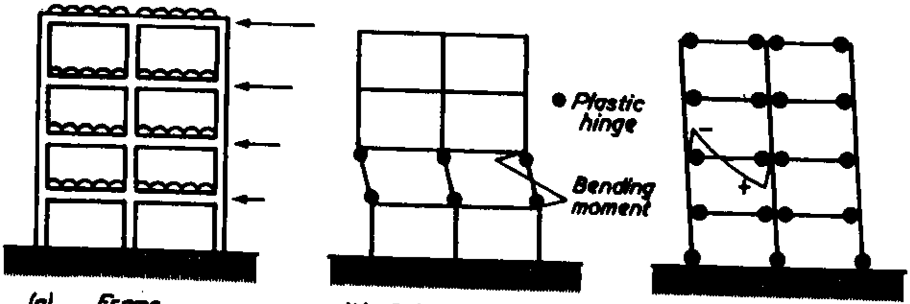
21. Zagajeski, S. and Bertero, V.V. (1977), Computer-aided optimum design of ductile R/C moment resistant frame, Proceedings, Workshop on earthquake resistant design of reinforced concrete building construction, University of California, Berkeley, California, Vol. II.



(a) Tension failure

(b) Compression failure

FIG. 1 – MOMENT CURVATURE RELATIONSHIP FOR R C BEAM SECTIONS

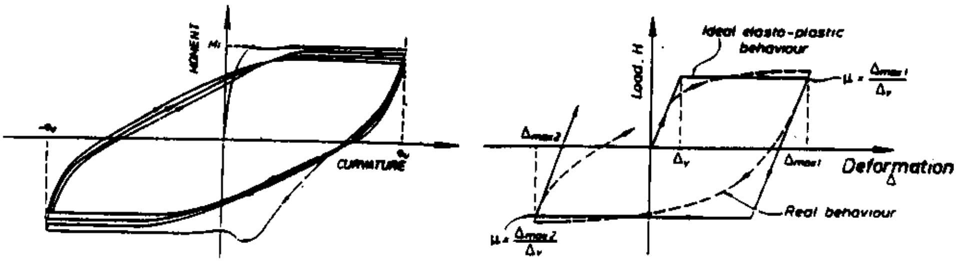


(a) Frame

(b) Column sidesway mechanism (Undesirable)

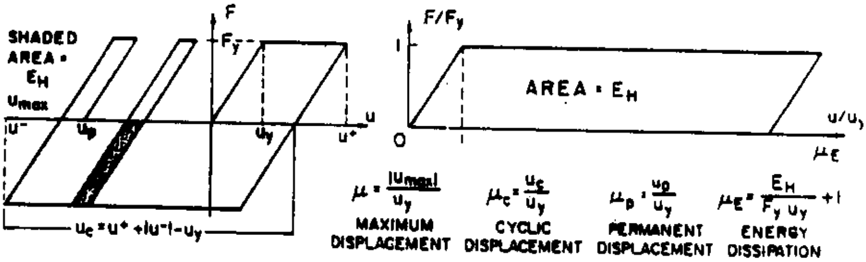
(c) Beam sidesway mechanism (Desirable)

FIG. 2 – FAILURE MECHANISMS IN A FRAME

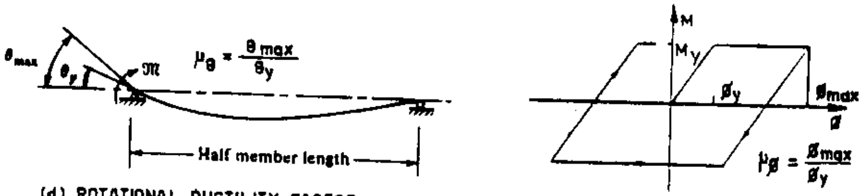


(a) DUCTILE BEHAVIOUR UNDER CYCLIC LOADING

(b) DUCTILITY FACTOR



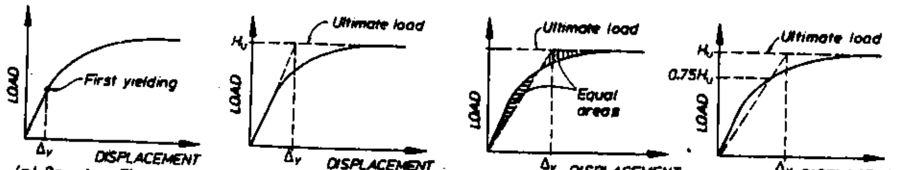
(c) DEFINITIONS OF ALTERNATE DUCTILITY FACTORS



(d) ROTATIONAL DUCTILITY FACTOR

(e) CURVATURE DUCTILITY FACTOR

FIG. 3 – DEFINITION OF DUCTILITY FACTORS



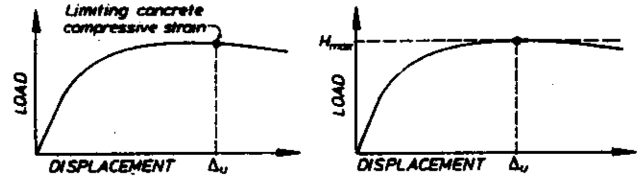
(a) Based on First Yield

(b) Based on Equivalent Elasto-plastic Yield

(c) Based on Equivalent Elasto-plastic Energy Absorption

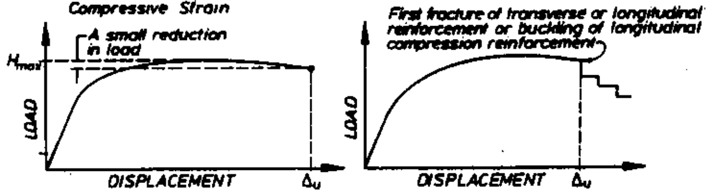
(d) Based on Reduced Stiffness Equivalent Elasto-plastic Yield

FIG. 4 – ALTERNATE DEFINITIONS OF YIELD DISPLACEMENT



(a) Based on a Limiting Concrete Compressive Strain

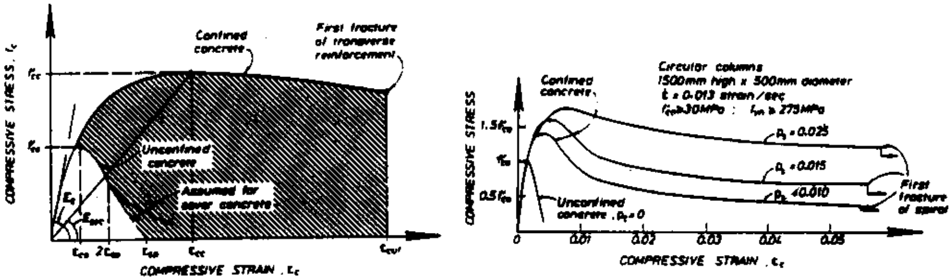
(b) Based on Peak Load



(c) Based on a Significant Load Capacity after Peak Load

(d) Based on Fracture or Buckling of Reinforcement

FIG. 5_ ALTERNATE DEFINITIONS FOR MAXIMUM AVAILABLE DISPLACEMENT



(a) Idealized (ref.16)

(b) Measured Experimentally (ref.16)

FIG. 6_TYPICAL COMPRESSIVE STRESS STRAIN CURVE FOR CONFINED CONCRETE

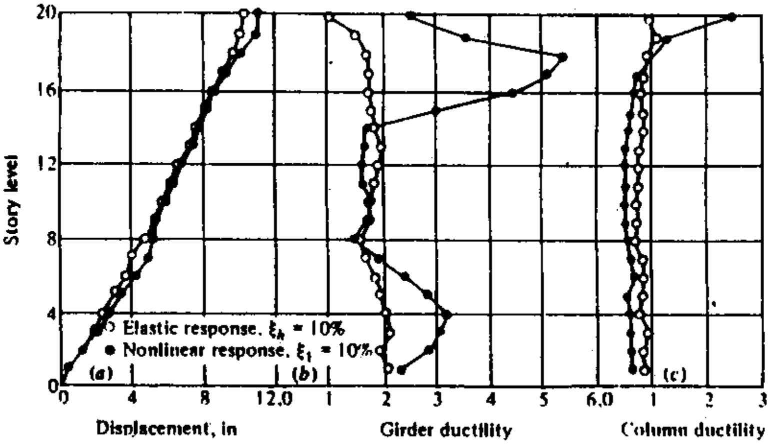
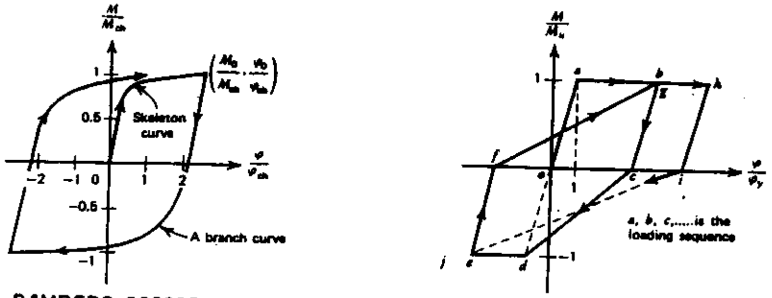


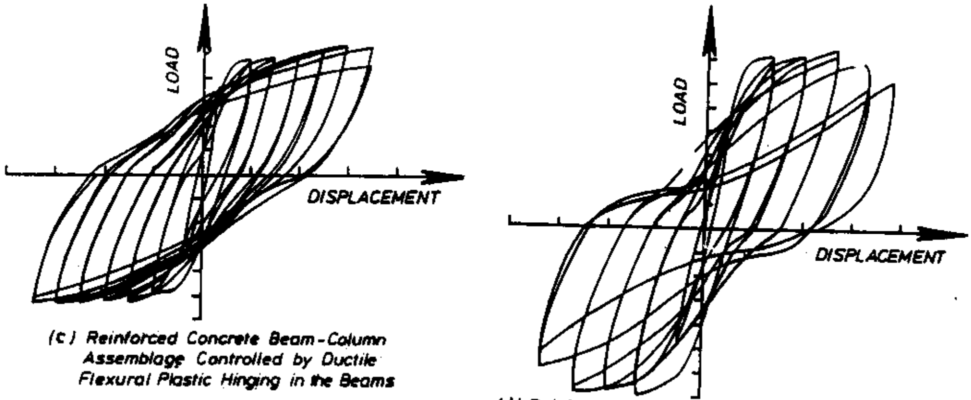
FIG. 7_COMPARISON OF ELASTIC AND NONLINEAR RESPONSE OF A MULTISTOREY BUILDING



(a) RAMBERG-OSGOOD

(b) CLOUGHS DEGRADING STIFFNESS

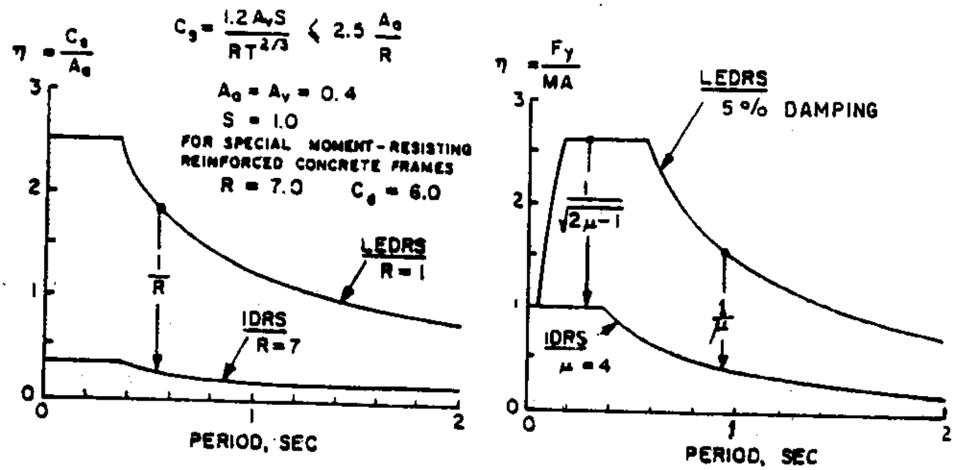
IDEALIZED MOMENT CURVATURE RELATIONSHIP



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

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

FIG. 8 _ HYSTERETIC BEHAVIOUR OF REINFORCED CONCRETE



(a) ATC METHOD

(b) NEWMARK-HALL METHOD

FIG. 9 _ METHODS OF CONSTRUCTING IDRS FROM LEDRS

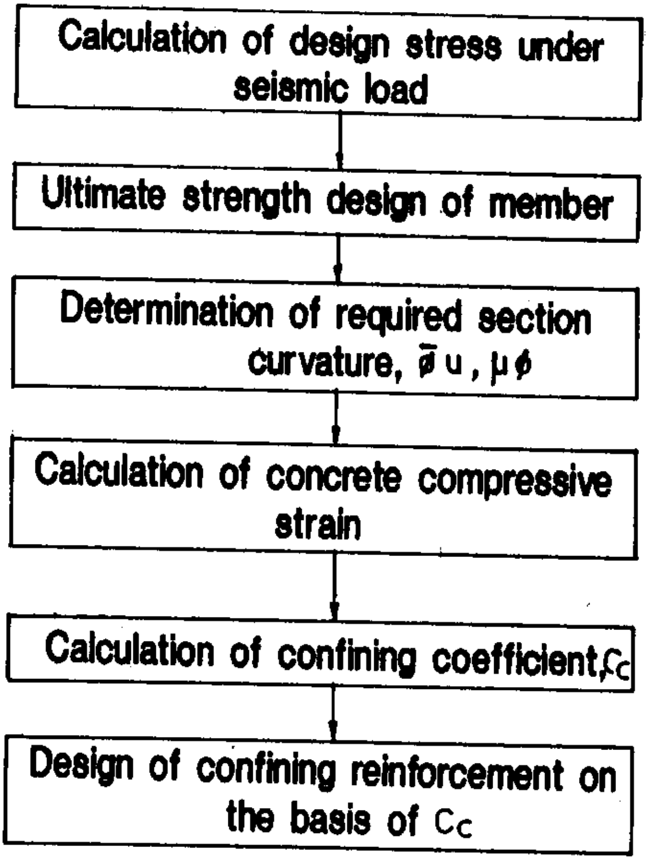


FIG. 10_ FLOW CHART FOR CURVATURE DUCTILITY DESIGN