2.3-MEMBERS: BEHAVIOUR AS RELATED TO DESIGN CRITERIA

by

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SYNOPSIS

The analytical and experimental behaviour of structural members and their connections as related to seismic design criteria are examined. Emphasis is given to reinforced concrete elements, but prestressed concrete, structural steel and masonry are briefly considered as well. Aspects of design, where it appears that information is lacking or where existing design criteria need to be reassessed, are discussed.

INTRODUCTION

It is well known that use of the static seismic design loads recommended by codes implies that the strength of critical members will be reached during a severe earthquake and that the critical members should have sufficient ductility to enable the structure to survive without collapse when subjected to several cycles of loading well into the inelastic range. This means avoiding all forms of brittle failure and achieving adequate ductility by flexural yielding of members.

Comprehensive reviews of the experimental behaviour of concrete and steel structures responding to seismic type loading have been given by Park and Paulay (1), Bertero (2), Fujimoto and Naka (3), and others. This paper will discuss design criteria for members and comment on aspects where it appears that information is lacking or existing design criteria need to be reassessed in the light of the actual behaviour of members.

DESIGN CRITERIA FOR STRENGTH

In the design of earthquake resistant structures, energy dissipating plastic hinge regions are detailed to ensure that they maintain near full flexural strength during the inelastic deformations that may occur, while sufficient reserve strength is provided against other types of failure to prevent undesirable modes of failure. The prevention of undesirable modes of failure requires a realistic assessment of possible beam flexural overstrength as well as the dependable strengths of the other failure mechanisms.

Design codes generally ignore the effect of rate of loading on the material strengths. Tests conducted at Berkeley (4) on reinforced concrete beams have indicated an increase in the first yield moment of about 20% due to high strain rate, but a reduction in the effect of high strain rate occurred at greater deformations, and after the first cycle of loading in which the member is yielded the hysteresis loops were little affected by the strain rate. Thus there is good justification for ignoring the effect of high strain rates on the material strengths in seismic design.

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DESIGN CRITERIA FOR DUCTILITY

A designer who thinks fundamentally will nave difficulty in deciding the level of ductility necessary in members. Codes have been vague on this point and definitions of "ductility factor" have been various and confusing. Nonlinear dynamic analyses of code-designed structures responding to typical severe earthquake motions have given an indication of the order of postelastic deformations required (see, for example, Ref. 5, 6 and 7) but the number of variables involved is so great that no more than qualitative statements have been made at present. The Californian (8) and New Zealand (9) codes have indicated that displacement ductility factors of the order of 3 to 5 are to be required of ductile earthquake resistant structures. The working draft seismic design recommendations of the Applied Technology Council (10) recommends ductility factors for ductile reinforced concrete and structural steel frames of 3 to 5, this presumably being the displacement ductility factor.

Confusion has existed in the minds of some designers regarding the definition of ductility factor, since it can be expressed in terms of displacements, rotations or curvatures. The displacement ductility factor, $\mu = \Delta_u/\Delta_y$ where Δ_u = maximum lateral deflection and Δ_y = lateral deflection at first yield, is the value commonly determined in nonlinear dynamic analyses. Some analyses have determined the rotational ductility factor of members θ_u/θ_y where θ_u = maximum rotation of end of member, and θ_y = rotation at end of member at first yield. However the information needed by the designer concerns the required member section behaviour expressed by the curvature ductility factor ϕ_u/ϕ_y, where ϕ_u = maximum curvature at the section and ϕ_y = curvature at the section at first yield. Thus the required ϕ_u/ϕ_y value is a far more meaningful index for ductility demand than the other possibilities. It needs to be recognised that there is a significant difference between the displacement, rotational and curvature ductility factors. This is because once yielding has commenced in a structure the deformations concentrate at the plastic hinge positions and further displacement occurs mainly by rotation of the plastic hinges. Thus the required ϕ_u/ϕ_y ratio will be greater than the Δ_u/Δ_y ratio.

When calculating ductility factors the definition of first yield deformation (displacement, rotation or curvature) often causes difficulty when the load or moment-deformation curve is not elastoplastic. This may occur for example due to plastic hinges in members not forming simultaneously or longitudinal bars in reinforced concrete members at different levels in the section yielding at different load levels. In such a case it is suggested that the "first yield" displacement be taken as the displacement calculated for the structure assuming elastic behaviour up to the strength of the structure in the first load application to yield, as illustrated in Fig. 1. A similar definition can be adopted for first yield rotation and curvature. Such a definition for first yield allows comparison of the effects of different loop shapes with the same initial stiffness and strength on the ductility demand.

It is apparent that agreement needs to be reached on the various definitions of ductility factor to avoid the looseness of definition which exists at present.

It is evident that the sequence of plastic hinge development in structures will influence the curvature ductility demand. Dynamic analyses have indicated that ductility demand concentrates in the weak parts of structures and that the curvature ductility demand there may be several times greater than for well proportioned frames. This can also be illustrated by examination of static collapse mechanisms. Fig. 2 shows a frame and shear walls which can be used for seismic resistance. Possible mechanisms which could form due to flexural yielding and formation of plastic hinges are also shown in the figure. If yielding commences in the columns of a frame before the beams a column sidesway mechanism can form. In the worst case the plastic hinges may form in the columns of only one storey, since the columns of the other storeys are stronger. Such a mechanism can make very large curvature ductility demands on the plastic hinges of the critical storey (1), particularly for tall buildings. On the other hand if yielding commences in the beams before in the columns a beam sidesway mechanism, as illustrated in Fig. 2, will develop (1), which makes more moderate demands on the curvature ductility required at the plastic hinges in the beams and at the column bases. Therefore a beam sidesway mechanism is the preferred mode of inelastic deformation, particularly since the straightening and repair of columns is difficult. Hence for frames a strong column-weak beam approach is advocated to ensure beam hinging. In the actual dynamic situation higher modes of vibration influence the moment pattern and it has been found (11) that plastic hinges in beams move up the frame in waves involving a few storeys at a time. For cantilever shear walls the static collapse mechanism involves a plastic hinge at the base and the curvature ductility demand for a given displacement ductility factor depends very much on the plastic hinge length as a proportion of the wall height. For coupled shear walls the mechanism shown in Fig. 2 can occur (1) and ideally the beams should yield before the wall bases to enable easier repair.

It is evident that many more nonlinear dynamic analyses need to be conducted on a range of building types using a variety of strong motion records to ascertain the likely curvature ductility demand on the critical sections to allow the designer to anticipate ductility requirements with more confidence.

TEST LOADING CRITERIA

A great deal of valuable information on the effects of severe earthquakes has been obtained from tests on structural assemblages in the laboratory using cycles of pseudo-static loading. Structural assemblages rather than complete structures have normally been tested due to difficulties with size. Fig. 3 shows a test specimen representing a beam-column joint of a frame. The members extend approximately to the points of contraflexure. It is impossible in such tests to simulate accurately all aspects of loading and ductility demand of the actual more complex structure. However if the loading sequence is severe enough, and if the strength, stiffness and energy dissipation of the test specimen are found to be acceptable, satisfactory performance of the actual structure can be confidently expected. In the past a variety of loading sequences and acceptance criteria have been used by various research laboratories, making the comparison of results difficult and resulting in different conclusions from tests being reached.

Two loading criteria which have been used in New Zealand laboratories are shown in Figs. 4 and 5. The displacement ductility factor is calculated using the first yield displacement for the first inelastic load run defined as in Fig. 1. A simple acceptance criterion is that the seismic load carrying capacity should not reduce by more than 20% during the test (9). It is suggested that the loading criterion of Fig. 5 be adopted since it allows observation of behaviour at various levels of imposed ductility during the test. The chosen magnitude of the imposed displacement ductility factor, the number of cycles of loading, and the centre of oscillation of the deflections, are debatable issues which can only be answered in detail by those who have conducted extensive nonlinear dynamic analyses. However there is no doubt that a standard loading criterion needs to be adopted to allow test results to be compared on a consistent basis.

BEHAVIOUR OF BEAMS AS RELATED TO DESIGN CRITERIA

Moment-curvature analyses for reinforced concrete beam sections shows that the available curvature ductility increases with increase in compression steel content and decrease in tension steel content. The seismic provisions of ACI 318-71 (12) require that the tension steel ratio should not exceed 0.5 of that producing balanced failure, and that at column faces the positive moment capacity of beams should be at least 0.5 of the negative moment capacity. It can be shown that this will ensure an available curvature ductility factor ϕ_u/ϕ_y of at least 6 for an extreme fibre maximum concrete strain of 0.004 (1). Hence if the curvature ductility factor demand is 2 or 3 times this value, as is likely in a severe earthquake, the concrete needs to be confined effectively by closely spaced closed stirrups and damage to the cover concrete must be expected. It would seem preferable to use lower tension steel contents than the limiting value allowed by ACI 318-71. With cyclic bending moments to yield in each direction a full depth crack can exist down the concrete section for much of the loading range (see Fig. 6) and the reinforcing bars may yield alternatively in tension and compression resulting in a lowering of the tangent modulus of elasticity of the steel owing to the Bauschinger effect. This could lead to buckling of reinforcing bars in compression at lower load levels than expected. It is recommended therefore (1) that to prevent bar buckling in plastic hinge zones the spacing of stirrups surrounding the compression steel should not exceed six compression steel bar diameters, a spacing which is much smaller than recommended in most current codes. The deterioration of the concrete due to the opening and closing of cracks in plastic hinge zones with cyclic bending moment decreases the concrete shear resisting mechanisms (aggregate interlock, dowel action and across the compression zone). In such regions only truss action of the stirrups should be relied on to carry shear, and where the shear force is high the full depth cracks should preferably be crossed by diagonal reinforcement. These precautions for shear resistance are not recommended by most codes. In order to avoid shear failure the design shear force used needs to be calculated on the basis of the design gravity loads on the members and the likely overstrength moment capacity of the plastic hinges at the ends of the members. The plastic hinge moments are calculated using a realistic steel strength bearing in mind the likely excess of the actual yield strength over the specified yield strength f_y and the strain hardening which may occur in developing the required ultimate curvature. For example, for intermediate grade steel (f_y = 40 ksi = 276 MPa) use of a steel strength of $1.25f_y$ would appear reasonable; for high strength steel (f_y = 60 ksi = 414 MPa) which shows considerable strain hardening after a short yield plateau use of a steel strength of $1.4f_y$ would appear to be necessary.

For prestressed concrete beams few codes give guidance for seismic design but recently the New Zealand Prestressed Concrete Institute has published recommendations (13) and the FIP Seismic Commission is at present drafting recommendations (14). Research has shown that properly detailed prestressed concrete members will give satisfactory seismic resistance, although the lower hysteretic energy dissipation of a prestressed concrete member compared with a reinforced member of the same strength and initial stiffness will generally lead to a greater deformation response of the prestressed concrete member to a severe earthquake. Good confinement of concrete and a small neutral axis depth (say less than 0.25 of the member depth) are the most important requirements for adequate curvature ductility.

For structural steel beams the design problems are not great providing that ductile welds can be achieved, lateral instability is avoided and the effect of local buckling of beam flanges is considered. Beam shears should be determined from the greatest probable shear force taking into account the overstrength moment capacities at plastic hinges.

BEHAVIOUR OF COLUMNS AS RELATED TO DESIGN CRITERIA

The strong column - weak beam design concept aims at having plastic hinges form in the beams rather than in the columns. Some codes, for example the seismic provisions of ACI 318-71 (12), require that at beam-column connections the sum of the moment strengths of the columns should exceed the sum of the moment strengths of the beams along each principal plane at the connection. This requirement unfortunately will not prevent plastic hinges forming in columns. Dynamic analyses have shown that in frames, due to higher mode effects, points of contraflexure may occur well away from the mid height of columns at various stages during an earthquake (1). Thus bending moment distributions in columns such as in Fig. 7 are possible. Hence the beam input moments $M_{b1} + M_{b2}$ may have to be resisted almost entirely by one column section. If the point of contraflexure lies outside the column height the strength of one column section needs to exceed $M_{b1} + M_{b2}$. This required column strength to prevent plastic hinges forming is much greater than the ACI 318-71 requirement. A general direction of seismic loading also causes a severe condition for the columns. In design it is customary to consider seismic loading to act in the direction of the principal axes of the structure and in one direction at a time. However a general direction of severe seismic loading can cause yielding of the beams in both directions simultaneously. For example, for the symmetrical building shown in Fig. 8, if a displacement ductility factor of 4 is reached in direction 2 it only requires $\Delta_1 = \Delta_2/4$ to cause yielding in direction 1 as well, and this occurs when θ is only 14°. Thus yielding in the beams in both directions may occur simultaneously for much of the loading. Biaxial bending may reduce the flexural strength of the column, and the resultant beam moment input applied biaxially to the columns is $\sqrt{2}$ times the uniaxial beam moment input. Therefore concurrent earthquake loading may cause the columns to yield before the beams unless columns are strengthened to take this effect into account. Similarly, concurrent earthquake loading will induce higher shear forces in columns when the beams yield than for loading in one direction only, and this higher shear force is to be resisted by sections loaded along a diagonal.

It is evident that column flexural strengths of greater than twice the ACI requirements would be needed if plastic hinges in columns are to be avoided. The difficulty of preventing plastic hinges from forming in columns is such that some column yielding must be considered to be inevitable. Note that yielding due to shift of the points of contraflexure will only occur at one end of the columns and therefore will not lead to a column sidesway mechanism in that storey. The degree of protection of columns against yielding is a debatable issue and needs to be approached on a probabilistic basis.

The possibility of plastic hinges forming in columns due to the effects discussed above makes it important to ensure that columns are capable of behaving in a ductile manner. Hence for reinforced and prestressed concrete columns adequate transverse steel in the form of hoops or spirals should be present at the potential plastic hinge regions at the column ends, to ensure ductile concrete behaviour and to prevent buckling of the longitudinal steel. Code provisions for confining steel are at present based on rather arbitrary assumptions. For example, the amount of spiral steel specified by ACI 318-71 (12) is based on preserving the axial load strength of the column after the cover concrete has spalled rather than aiming at a particular curvature ductility factor, and the quantity of rectangular hoops specified is based on the assumption that rectangular hoops are one half as efficient as spirals in confining the concrete. More accurate confining steel provisions are required with more emphasis on flexural ductility, and including as a variable the axial load level. There is also a scarcity of design criteria and a lack of experimental results for rectangular columns loaded by shear force along a diagonal.

BEHAVIOUR OF BEAM-COLUMN JOINTS AS RELATED TO DESIGN CRITERIA

Recent experimental investigations of reinforced and prestressed concrete beam-column joints have indicated that when the plastic hinges form in the members adjacent to the connection the joint core may be subjected to extremely high shear forces and bond stresses (see Fig. 9). Under cyclic loading the concrete in the joint core may break down due to alternating diagonal tension cracks and bond forces, and the bars may slip through the joint core due to bond deterioration. The joint core shear design provisions of ACI 318-71 have been shown to be inadequate to resist intense inelastic displacement cycles simulating the effect of a severe earthquake (1). It would appear to be erroneous to base a design procedure for joint cores on test results obtained from members as the ACI code has done. Shear carried by the concrete shear resisting mechanisms V_c should only be taken into account when the column axial load is high, or the joint core is crossed by at least one bonded prestressing tendon near mid-depth, or beams enter the column on all four faces, or plastic hinges form away from the joint core and the core remains elastic. In other cases the shear strength of the concrete should be ignored. The total horizontal shear force to be carried by the shear reinforcement

($T - V' - V_c$, in Fig. 9) should be carried across the corner to corner crack. Longitudinal column bars should exist around the perimeter of the section so that some cross the diagonal tension crack to help transfer vertical shear force. That is, four bar columns should not be used. Joint cores in two-way frames should be designed to carry the biaxial shear force resulting from beams yielding in two directions simultaneously. To avoid bond problems, reinforcing bars passing through joint cores should not be of excessively large diameter. At interior columns, for example, for bars with a yield strength of 40 ksi (276 MPa), the bar diameter should not exceed 0.04 of the joint core dimension in the bar direction. Designers should be encouraged to enforce plastic hinges in beams to occur away from the column faces, thus allowing better (elastic) joint core behaviour. It is evident that the simplistic methods of joint core design at present recommended by codes need to be revised to take account of the above factors.

Beam-column connections in structural steel do not appear to pose the same problems as structural concrete. The main problems for structural steel connections are achieving adequate strength and stiffness in the web of the joint core and avoiding brittle failure at welds.

BEHAVIOUR OF SHEAR WALLS AS RELATED TO DESIGN CRITERIA

Reinforced concrete shear walls provide an attractive means of seismic resistance, helping to reduce problems such as column yielding, beam-column joint detailing, and instability due to drift. Their stiffness also enables much non-structural damage during a severe earthquake to be minimized. Tests have shown that properly detailed reinforced concrete shear walls will provide adequate ductility in tall buildings (1). Cantilever walls are detailed essentially as reinforced concrete beams. The floor slabs provide support against lateral instability. Coupled shear walls, generally shear walls with vertical rows of openings, have as the critical design elements the coupling beams which may be short and deep. The shear strength of coupling beams should exceed the flexural strength to avoid brittle failure. Recent experiments (1) have shown that the strength and ductility of reinforced concrete coupling beams can be improved if the principal reinforcement is placed diagonally in the beams instead of the conventional steel arrangement of longitudinal bars and vertical stirrups. Many designers prefer shear walls without a thickening at the wall ends (rectangular, channel or I sections) but other designers have columns at the wall ends (dumb-bell shaped section). Others bury structural steel frames in reinforced concrete shear walls. The relative merits of shear walls of various shapes and the real need for structural steel within reinforced concrete walls needs further investigation.

Masonry shear walls with vertical reinforcement in grouted cavities in the walls are being used with more confidence by designers. Reinforced concrete block walls have been designed as unconfined reinforced concrete. The extent to which this design procedure can be justified needs further investigation.

CONCLUSIONS

The aspects of the behaviour of members as related to design criteria which are considered to be in need of clarification are:

1. Definitions of first yield deformation and types of ductility.
2. The likely curvature ductility demand on critical sections of members of structures responding to severe earthquakes.
3. The loading and performance criteria adopted in laboratory tests on structures with pseudo-static loading simulating seismic loading.
4. For concrete beams, the transverse steel required to confine the concrete and prevent buckling of the longitudinal reinforcement, and the design of shear reinforcement, in plastic hinge zones.
5. The degree of protection of columns against yielding. For concrete columns, the design of transverse steel required in potential plastic hinge zones, and the biaxial shear strength.
6. The design procedures for shear reinforcement in concrete beam-column joint cores. Criteria for anchorage of bars passing through joint cores.
7. The efficiency of various shapes of section of reinforced concrete shear walls. Adequacy of design procedures for reinforced masonry walls.

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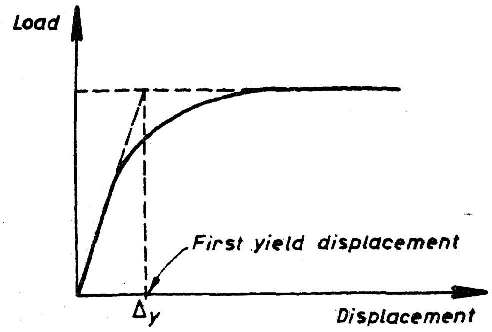

Fig. 1 Possible definition for "first yield" displacement when load-deflection relationship is curved

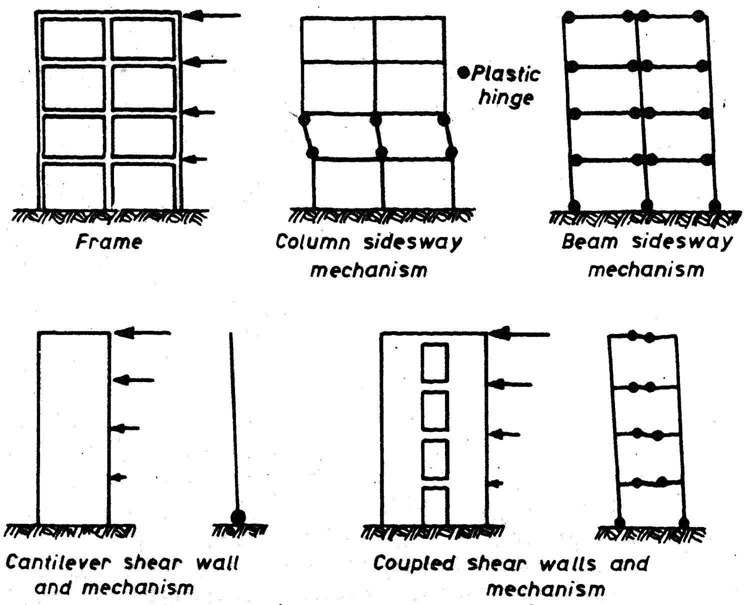

Fig. 2 Building structures under seismic loading and possible mechanisms

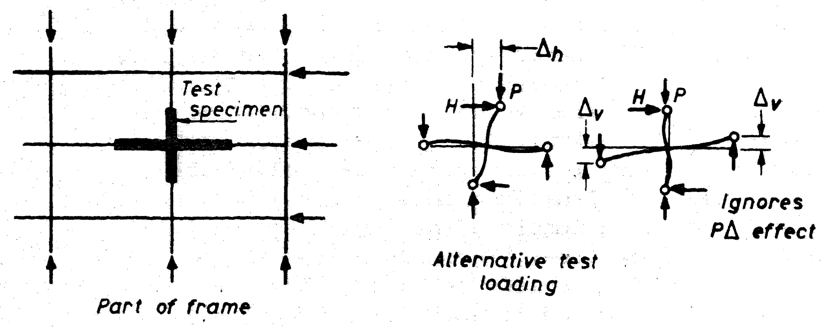

Fig. 3 Beam-column joint test specimen and load application

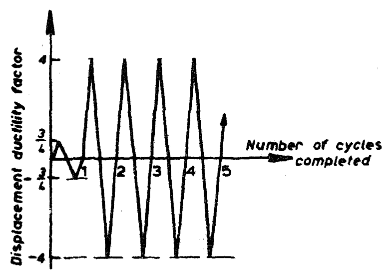

Fig. 4 Simple loading criterion

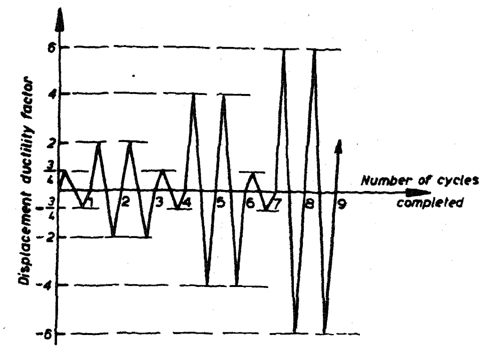

Fig. 5 More complex loading criterion

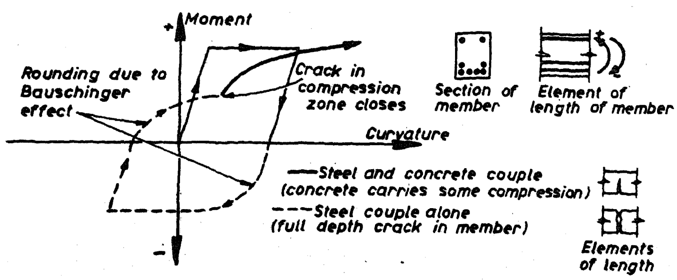

Fig. 6 Moment-curvature relationship for doubly reinforced concrete section with reversed flexure

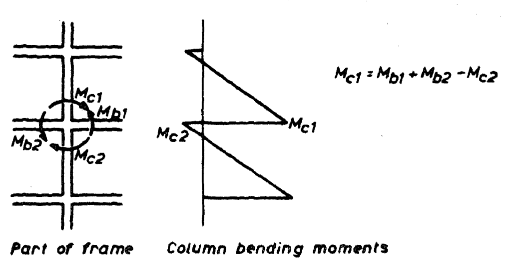

Fig. 7 Possible column moments during dynamic response

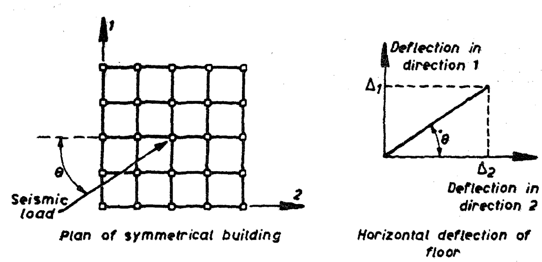

Fig. 8 General direction of earthquake loading on building

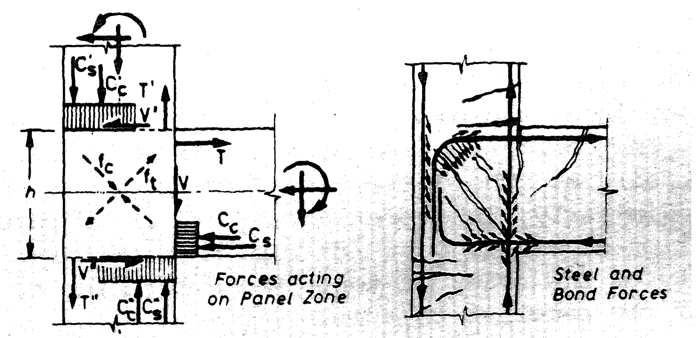

Fig. 9 Stress resultants, crack pattern and bond forces at reinforced concrete exterior beam-column joint

DISCUSSIONS

Hiroshi Akiyama (Japan)

Ductility is usually defined with regard to the maximum deformation. Ductility, however, should be also defined with regard to the accumulated plastic deformation (or accumulated plastic strain energy), since the accumulated plastic deformation is more likely to be related to Structural damages. The accumulated plastic deformation can develop within a limited deformation amplitude and can cause the collapse of the structure.

For this point of view, it must be done to examine the vertical distribution of the accumulated plastic deformation (or the concentration of damages) before reaching the conclusion of whether weak-beam strong-column type of structure is advantageous or not. My opinion is that the concentration of damages need to be always considered.

Ref: 1. Akiyama, H., 'An Application of High Strength Steels to Earthquake Resistant Buildings', Proc. 10th Congress of IABSE, 1976.

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Brijesh Chandra & A.R. Chandrasekaran (India)

In the design criteria for deflection ductility, it is mentioned that a factor of 3 to 5 is required for ductile earthquake resistant structures. Tests on steel frames have been reported (Ref.1) in which deflection ductilities even at failure do not exceed three. As the panelist mentioned there could be different definitions of ductility. In the tests, which were carried out on single bay, single storeyed four column steel frame designed such that yielding occured only in columns, the deflection at yield was defined as the one causing yield strain at the maximum fibre as measured by strain gauges. The deflection and strain was monitored continuously till failure. The deflection ductility did not exceed three. Similarly, interpreting fig. 5.11 of reference 2, deflection ductility does not exceed three even for strain ductility of the order of ten. We would like to have the comments of the panelist on this.

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B.R. Seth (India)

The importance of ultimate analysis under seismic loads has been nicely stressed by the authors, for R.C.C. frames. The following limit analysis is suggested, to overcome the problems and difficulties pointed out by them.

PLASTIC HINGE V/S MOMENT RELATION:

Yield point curvature and moment can be expressed as[II]

$$\phi_e.d = \frac{f_y}{E_s\,(1-k)} = \frac{f_y}{E_s\,(1 + pm - (2pm + p^2m^2)^{1/2})}$$

$$M_e = f_y p.bd^2(1-k/3)$$

The ultimate curvature and moment can be written as

$$\phi_u.d = \frac{e_u\,(0.85\,k_1\,f'_c)}{p.f_y} = \frac{0.00255\,k_1\,f'_c}{p.f_y}$$

taking $e_u = 0.003$

$$M_u = f_y\,p.b.d^2\,(1-p.f_y/\,1.70\,f'_c)$$

and

$$M_e/M_u = \frac{1+ (pm- (2pm + p^2m^2)^{1/2}\,)/3}{1 - p.\,f_y/\,1.70\,f'c}$$

From the plots of M_e/M_u v/s p for particular usual values of f_y and f'_c it can be seen that M_e/M_u does not vary much considering the effect of shear, M_e/M_u equal to 0.9 can be accepted. The general moment curvature curve can be approximated as bi-linear curve. If M/M_u v/s $\phi_e.d$ is plotted, M/M_u has constant ordinate and abscissa is also almost constant as for good ductility, section is under-reinforced and variation in percentage of steel is not much. As, A.C.I. Committee recommends the length of effective plastic zone as equal to the effective depth. Therefore, plastic hinge rotation

$$\theta = (\phi_u - \phi_e)d$$

NOTATIONS:

b, d	are width and effective depth of section.
e_u, f_y, f'_c	are the ultimate strain of concrete, yield stress of steel and cylinder strength of concrete.
k, k_1	are fraction for depth of neutral axis in elastic stage and factor for uniform ultimate stress block
m, p	are modulus ratio and fraction of area of steel
m_e, M_u	are the moment resistance capacity of the section at yield of steel and ultimate stage.
ϕ_e, ϕ_u, θ	are the curvature at yield and ultimate stage, and plastic hinge rotation.

Hawkins (U.S.A.)

You made a statement that the shear problem has been solved. I wonder which shear problem you are talking about. I believe it was in relation to joints. Which solution you are talking about ?

Saha, (India)

For the horizontal member of a coupled shear wall structure, Prof. Paulay recommended the diagonal reinforcement. In such a case would he say that the conventional design using stirrups would be insufficient for the shear design of the reinforcement concrete member ?

Author's Closure

The preference for the weak beam-strong column concept is because if plastic hinges develop at top and bottom in the columns of one storey of a multistory frame, resulting in a column sidesway mechanism forming in that storey, very large plastic hinge rotations are required at those hinges for the frame to reach the required displacement ductility factor and to survive a severe earthquake. Heavily loaded columns may not be sufficiently ductile, and P-Δ effects may become severe, resulting in collapse of the frame. On the other hand, a sidesway mechanism involving plastic hinges forming in the beams and at the bases of the bottom storey columns results in a smaller plastic hinge rotation required at those hinges to achieve the required displacement ductility factor, and ductility is more easily provided in flexural members. Hence it is easier to detail a structure to survive an earthquake when plastic deformations occur mainly in the beams. Also, damage to columns is difficult to repair. Hence a strong column-weak beam concept in design is preferred.

It is agreed that structural damage may be related to accumulated plastic strains. However for the few excursions into the inelastic range during an earthquake the monotonic moment-curvature relationships for plastic hinge region should give a good envelope curve for cyclic load behaviour. Thus the monotonic moment-curvature relationship gives a good indication of available ductility. Tests have shown that reinforced and prestressed concrete structural members subjected to a few cycles of inelastic deformations do not suffer a fatigue failure of the steel but may undergo strength and stiffness degradation of the concrete due to opening and closing of cracks in alternating directions. However, good detailing of reinforcement by way of confining steel and reasonable proportions of compression steel will ensure that the integrity of the member is maintained and will enable large numbers of cycles of inelastic deformation to be applied without excessive increase in damage.

The panel paper refers to the U.S. and New Zealand Codes which indicate that displacement ductility factors of the order of 3 to 5 are required of ductile earthquake resistant structures. Obviously the displacement ductility demand of a structure responding to a severe earthquake will depend on the level of seismic load it has been designed for (i.e. its strength), the actual ground motion the structure is subjected to, and the stiffness and the viscous and hysteretic damping characteristics of the structure. Hence wide variations in displacement ductility demand can exist, depending on these variables. The displacement ductility factors quoted in Codes are based on assumed values for these parameters and are approximate average values. It is to be expected that the curvature ductility factor required in a plastic hinge region will exceed the displacement ductility demand for the structure, since once yielding has commenced in the structure the deformations tend to concentrate at the plastic hinge positions and further displacement occurs mainly by rotation at the plastic hinges (see Ref. 1 of the paper). That is, before the stage of plastic hinging strains in all elements of the structure contribute to the displacement of the structures but beyond the stage of plastic hinging displacements occur mainly due to deformation in plastic hinge regions. Hence it will follow that the ratio of maximum strain reached in the steel to yield strain will be much greater than the displacement ductility factor.

Theory of this type to determine the ratio of ultimate curvature to curvature at first yield for reinforced concrete structures has also been derived elsewhere (see Ref. 1 of the paper). Use of an ultimate concrete strain of 0.003 is conservative in ultimate curvature calculations; a value of 0.004 is more reasonable for unconfined concrete and a higher value may be used for concrete confined by stirrup ties (see Ref. 1 of the paper). Assuming an equivalent plastic hinge length equal to the effective depth of the member may be optimistic if yielding occurs only to one side of the critical section such as in a beam of a frame where the maximum moment occurs at the column face. Empirical equations for the equivalent plastic hinge length have been given elsewhere (for example, see Ref. 1 of the paper).

The shear problem for beam-column joint cores has been solved in as much as it is possible to provide adequate shear reinforcement in a joint core to prevent a shear failure there during cyclic loading. Such an approach needs to take into account the degradation of the shear strength of the concrete shear resisting mechanism due to cyclic loading. This means ensuring that sufficient horizontal stirrup ties, and vertical column bars between the corner bars, exist in the joint core to form a truss mechanism to carry most of the joint core shear forces (see Ref. 1 of the paper). However the possible degradation of bond strength for longitudinal bars passing through the joint core also needs to be considered by the designer and rules need to be established to limit the maximum bar diameter which can be used to ensure that slip through the joint core does not occur. A suggestion is made in the paper that for Grade 40 (mild steel) deformed bars in beams the diameter should not exceed 1/25th of the column dimension in the direction of the bar.

This question has already been commented on by Professor Paulay in his reply to discussion on the paper "Ductile Behaviour of Shear Walls Subjected to Reversed Cyclic Loading".