CAPACITY DESIGN OF REINFORCED CONCRETE FRAMES FOR DUCTILE EARTHQUAKE PERFORMANCE+

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ABSTRACT - The basis of the design of reinforced concrete frames for fully ductile earthquake performance, applicable to low buildings as well as to major structures, is outlined in terms of capacity design criteria now considered essential to prevent non-ductile failures and enable the building to survive earthquake attack.

1. INTRODUCTION

Ductility is a basic requirement of the New Zealand loading code, NZS 1900: Chapter 8. In a major earthquake, a building frame designed to code levels of lateral strength will be forced well beyond yield. The seismic resisting frame system must develop plastic hinges and subsequently undergo large inelastic lateral displacements in a ductile manner, in order to satisfy the dynamic energy demand and thus survive the earthquake attack. Ductile reinforced concrete design is therefore primarily concerned with actual strength (i.e. capacity) of members and joints in the real structure when built, and aims to ensure that hinging develops only at suitable locations while effectively preventing any form of premature, non-ductile failure.

It is very evident from earthquake damage reports that the smaller framed buildings are just as vulnerable as high buildings, and will resist a major earthquake by deforming inelastically at hinge locations. Thus, all reinforced concrete framed buildings (excepting those of single storey with light roof) designed to code loadings should be detailed for fully ductile earthquake performance.

The general principles of ductile seismic design have been recognised for some years. Based on provisions of the SEAOC code, ductility design and detailing requirements are included in current New Zealand codes of practice (1), (2). Recent testing and research at the University of Canterbury has contributed valuable information on column and beam-column joint behaviour. The probability of concurrent hinging of all beams at a joint (diagonal or skew earthquake) is now recognised (3) and should be considered in design. This paper indicates current thinking and outlines criteria for the capacity design of component members and joints of ductile reinforced concrete frames, presently employed for public buildings in the design office of the Chief Structural Engineer, New Zealand Ministry of Works.

2. PLASTIC HINGES

The development of a "storey sway" column hinging mechanism (Fig. 1) during an earthquake can endanger a multistorey building by concentrating the whole of the seismic energy in the columns of only one storey. For buildings of more than two storeys, it is unlikely that the available ductility of the hinges of this mechanism would be sufficient (4). Ductile design therefore aims to prevent dangerous storey sway column mechanisms, and requires that except in special cases all multistorey frames be designed to develop beam hinging mechanisms for seismic resistance (fig. 2). It should be noted that, associated with beam hinging, a hinge will form at the effective base of each frame column. In addition, in the top storey, a column hinge at each top connection (where one column resists two beam moments) cannot always be avoided but is not considered dangerous if properly detailed.

A very real possibility exists that column hinges may develop, notwithstanding our design precautions aimed at ensuring beam hinging mechanisms (5). Test evidence and other design information is limited, and actual hinging behaviour of the real structure cannot presently be predicted with any certainty. Increased beam strength may enforce column hinging where actual strain hardening of bars, tensile capacity of slab concrete, or contribution of slab reinforcement are greater than envisaged in design. Column hinges may develop above, and possibly also below, the highest level at which hinges have actually formed in the beams. Where lower storey stiffnesses vary from those above, the column base hinge could be located above the actual base. The risk that unforeseen column hinges may develop makes it essential that frame columns be detailed for ductile hinging under all possible axial load conditions, while preventing brittle shear failure by providing strength to resist the column shears which act when column hinges develop.

3. FRAME ANALYSIS UNDER CODE LOADS

Using preliminary design information, the seismic resisting frame is analysed to obtain bending moments and deflections due to code specified seismic lateral loads (including real and accidental torsions) acting separately in each principal direction of the building.

These frame analyses should be examined

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carefully, noting any special features which could affect the way in which capacity design criteria should be applied. Reversed moment at the top of a lower storey column, due to the point of column contraflexure moving into the storey above, should be recognised. In some cases, bending moments in columns due to gravity loads (either acting alone or in combination with applied design lateral forces) may exceed moments obtained from capacity criteria, and reinforcement should be provided for this condition. Locations of expected column hinges, which may arise from column-hinging mechanisms where permitted by code or which may exist at column base or elsewhere as indicated previously, should be noted for special detailing consideration.

4. BEAM LONGITUDINAL REINFORCING BARS

Load-factored combinations of moments due to dead, live, and code specified seismic lateral loads from the frame analyses are used for cross-section strength design to obtain bar requirements for all beams. These are rationalised by the designer who selects suitable arrangements of bars. As finally detailed, actual sizes, numbers and steel grade of longitudinal bars are known at each end section of all beams of the seismic frame.

It should be noted at this point that when longitudinal beam reinforcement has been selected and detailed, normally the only subsequent use of the static analysis results concerns the checking of lateral deflections and inter-storey drifts, and related calculations. Except in special cases, strength design of all other frame reinforcement is based on ductility criteria using capacities as described in the following paragraphs.

5. BEAM HINGE MOMENT CAPACITY

The beam hinge moment capacities, at yield in the actual structure, are of great significance. They determine the level of seismic lateral load resistance built into the real structure (fig. 3), and thus they directly affect the ductility demand which the structure must sustain in order to survive the earthquake. They also largely govern the design of all other components of the frame.

To secure ductile performance in the real structure, design calculations must allow adequately for maximum likely overstrength which could be built in to the beam hinge locations. Beam hinge moment capacities must therefore be calculated from the known reinforcing bar details, making allowance for all foreseeable sources of such overstrength, and using, for hinge capacity calculation only, a ϕ factor of 1.0.

Overstrength moment capacity during hinging, as curvature increases during and after spalling of cover concrete, should be carefully assessed considering actual reinforcing bar strengths combined with all other factors contributing to overstrength in the real structure. For this purpose, the limited data presently available on New Zealand reinforcing bars is useful as a guide only. Otherwise, the designer should calculate hinge moment capacity considering overstrength sources listed below. A small margin (up to 10%) resulting from yield point recommendations allows for overstrength sources not considered.

(a) Higher actual yield point of main bars

Allow for maximum likely yield point 25% above the specified minimum value. This recommendation, based partly on reference (6), applies both to Grade 40 bars to NZS 1693 and to Grade HY60 bars to NZS 1879 in the size range of No. 8 to No. 12 inclusive.

(b) Strain hardening of main bars

For Grade 40 bars, which generally have a yield plateau extending to about 16 times yield strain, no strain hardening allowance appears necessary where the recommended maximum yield point in (a) above is used.

For Grade HY60 bars, which have a yield plateau of about 4 times yield strain only, a strain hardening allowance of at least 15 kips/in^2 is recommended. This is based partly on references (7) and (8) and represents a 20% increase on maximum actual yield point recommended in (a), or a 25% increase on specified minimum yield point.

The early strain hardening of HY60 bars during hinging causes a large hinge strength increase of uncertain amount which greatly reduces any benefits obtained from their use. The susceptibility of HY60 bars to strain-age embrittlement is also causing concern. The designer is advised not to use this grade of bar in members expected to develop plastic hinges.

(c) Slab reinforcement

Co-operation of slab reinforcing bars can significantly increase beam hinge capacity, and its contribution should be carefully assessed.

(d) Tensile strength of slab concrete

Where anchorage of slab connections near the column is effective, concrete tensions may be significant especially for a single cycle or for a single pulse loading.

6. DESIGN OF BEAM SHEAR REINFORCEMENT

Non-ductile shear failure must be prevented. In addition to the load-factored dead and live load shears, the beams must resist the maximum shears which can be induced under dynamic conditions as hinges develop and plastic zones spread inwards, usually from the beam ends. Some uncertainty exists as to the magnitude of these maximum shears, such uncertainty being greater where members have small span/depth ratios (e.g., beam between closely-spaced columns; also short columns between deep spandrel beams). Therefore, until test evidence becomes available, the calculation of induced shears should preferably contain a margin which varies with the member span/depth ratio. Thus, it is recommended that hinging shears be obtained from the calculated hinge capacity moments using a reduced member length (L - d), i.e. the clear span L reduced by 0.5d for each hinge existing in the member (fig. 4). These shears are of course of constant magnitude along the beam length. In cases where one plastic hinge occurs in the span at some distance from the joint (fig. 2), the value of L for hinging shears is the distance between the plastic hinge locations, being less than the clear span.

Normal design methods should be used (ϕ = 0.85 for shear; specified minimum f_y for steel) in calculating the shear reinforcement required. Over a length equal to twice the beam depth from the column face, i.e. at hinge locations, the web reinforcement should be able to resist the entire shear.

The consequences of shear failure under seismic attack and the status of present methods for predicting shear capacity fully justify the provision of a further margin of shear strength, additional to that recommended above, wherever practicable.

7. EFFECT OF CONCURRENT BEAM HINGING

Buildings designed to code loadings will undergo total lateral displacements of say 4 times yield displacement during a major earthquake. A 4 times yield displacement of a regular building, in a horizontal direction angled at only 15 degrees to one principal direction, could have a component displacement causing yield in the other principal direction of the building (fig. 5). Again, a 3 times yield displacement angled at only 19 degrees to one principal axis will cause concurrent yielding along the other axis. It therefore appears highly probable that beam hinges will form simultaneously in all beams framing in to a column joint for a significant fraction of the time of severe shaking. Williamson (3) has drawn attention to this effect, and Blume (9) has discussed the probability of concurrent response in both principal building directions.

Previously, space frames have usually been designed as plane frames by considering earthquake loading and performance separately in each of the two principal building axes. With the object of forcing beam hinging mechanisms, each column of these plane frames is designed to be stronger than the beams framing into it when they reach yield capacity. However, in the event, now considered very probable, that such columns and the beam-column joints will be subjected to concurrent hinging of all beams framing into the joint, it appears (fig. 6) that:

(a) column hinge mechanisms may form in place of the beam mechanisms envisaged;

(b) columns and beam-column joints will be acted upon by significantly higher moments and shears than those for which they were designed; and

(c) exterior columns and particularly corner columns will sustain greatly increased axial overturning forces.

Under these conditions there is a very real risk of brittle failure of columns and joints and possible collapse.

8. COLUMN AXIAL LOAD AND MOMENT

Ductile concrete frame design requires that, in addition to preventing shear failures, the serious consequences of column failure under axial load and bending moment should be avoided. Beam hinging causes large axial forces and moments to act on the column section, and adequate capacity must be provided for all possible load conditions.

Column axial forces considered are due to:

(a) Dead and Live loads

Apply load-factored combinations as specified.

(b) Vertical accelerations

Allow for additonal vertical loading, based on the contributory load W_v acting on the column, of $\pm 1.5CW_v$, where C = 0.20, 0.15, 0.10 for Zones $\bar{A}$, B, C, respectively.

(c) Beam hinging (i.e. overturning)

The maximum overturning force at an exterior column section is obtained by summing, for all beams above the section, the induced shears calculated from beam hinge capacities (allowing for over-strength). Concurrent hinging of beams along both principal axes should be considered where applicable, e.g. at a corner column, in computing maximum column axial forces.

To ensure that beam hinge mechanisms develop in the frame, the bending strength of a column section (above and below the joint) must be stronger than the maximum beam hinge capacities (summed on both sides of the joint) for all possible column axial force conditions. For the same reason, the column bending strength about the appropriate diagonal or skew axis should be greater than the resultant biaxial moment acting at the joint developed as a result of concurrent hinging of all beams framing into the joint. The total beam hinge capacity moment at the joint should be shared between the columns above and below the joint in proportion to their stiffnesses.

Column moment capacity reduces significantly when cover concrete becomes ineffective or spalls (8), (10). This can occur in the column close to the joint after one or more reversals (as a result of spalling of the hinging beam section), and the change in relative member strengths may then enforce further hinging in the column and not in the beam. Our design to ensure hinging of beams thus depends upon continuing effectiveness of cover concrete in the column section adjacent to the joint. To ensure that hinges formed in the beam do not shift from beam to column, the column core should be designed for sufficient strength to exceed the beam hinging capacities as above without assistance from the cover concrete. Special consideration of capacity reduction due to spalling should also be given at those column locations where hinges can be expected to develop.

The column core section and longitudinal reinforcement is designed for the axial forces and uniaxial and biaxial moments indicated above. The specified minimum steel yield point should be used, and this may provide a small margin (up to 10%). For design on the core section without assistance from cover concrete, a ϕ factor of not greater than 0.85 could be taken. Where biaxial bending design by computer is not available, useful aids are given in references (11), (12), (13).

It should be noted that extra longitudinal steel gives an added margin of column strength, but also requires additional column shear reinforcement to protect against shears which act if column hinges develop.

For exterior and corner columns subjected to overturning forces, the assumption that hinges develop in all beams above any level provides a necessary safety margin of axial load; this margin increases for the higher buildings, which have less predictable performance. There is little or no axial load margin in the case of two or three storeys where it is likely that all beams would actually develop hinges. There is no comparable axial load margin for interior columns. For all building heights, column bending moment requirements are based entirely on hinging conditions at the joint considered; the assumption of all beams hinging does not influence the design moment and therefore provides no margin. It should be noted that the assumption provides neither axial load margin nor bending moment margin in the case of the interior columns, regardless of building height.

9. COLUMN SHEAR DESIGN

Prevention of shear failure requires that the column have adequate shear strength to resist the full shears induced by plastic hinging adjacent to the joints above and below the storey column considered. It is particularly important to provide capacity to resist the resultant column shears due to concurrent hinging of all beams framing into the joints above and below the storey column (these hinges develop large resisting moments - in a biaxial direction - in the top and bottom column sections, inducing high shears acting in a diagonal or skew direction relative to the column axes).

Column shear design should also provide for the possibility that, notwithstanding our design aims, column hinging may be enforced (refer to Section 2 above). The column should therefore be designed to resist the maximum shears which can be induced under dynamic conditions, assuming that hinges develop and plastic zones spread inwards from the ends of the column. This requirement will generally govern column shear design, and applies to shears acting in various directions along and at skew angles to the column axes.

Maximum column hinge moment capacity should be calculated at both ends of the column for all possible loading conditions (maximum moment capacity is reached with the balance load P_b acting), and should allow for overstrength as for beam hinge capacity calculations using $\phi = 1.0$. Capacities should be calculated for the full cross-section (i.e. cover concrete effective) under both uniaxial and various biaxial bending directions. The present uncertainty in magnitude of the maximum shear induced during column hinging is greater than for beams (due to shorter clear distance between supports and to other effects, e.g. $P - \Delta$). To allow for this the maximum shear should be calculated from the column hinge capacities using the clear column height h reduced by 0.5d (where d is the column depth) for each column hinge (fig. 7).

The column shear reinforcement is designed using normal methods ($\phi = 0.85$ and specified minimum f_y), but only the column core area is considered effective (i.e. allow for spalling of cover concrete). This steel should be provided to resist the full calculated shear over the portion of column where special transverse (confining) reinforcement is provided - i.e., in likely hinge locations above and below the joint. In the column mid-height section some shear may be carried on the core concrete, provided axial compressive forces do not reduce below a nominal stress level of 0.12 f'c. Particular attention should be given to shear details at the expected location of the column base hinge.

The absence of test evidence applicable to the design of column shear reinforcement for the large shears acting under concurrent beam hinging or biaxial column hinging conditions is of concern, especially relative to the importance of preventing brittle column failure. The adequacy of normal tie shapes to resist large shears acting in a diagonal or skew direction on a square or rectangular column is doubtful and needs test investigation. The resistance of square or rectangular ties to diagonal shears should be improved by the use of supplementary diamond-shaped ties, but these diamond ties act with a different effective depth and may not co-operate with the rectangular ties immediately shear forces are imposed. The shear strength of circular tie or spiral reinforcement is also uncertain. Test investigation should include the effect of tensile axial column loads, and should examine shear resistance after cover concrete has spalled away.

10. COLUMN CONFINEMENT AND DUCTILITY

The possibility that column hinges may develop has been discussed (Section 2), and overcapacity loadings may also eventuate (5). Magnification of vertical ground accelerations can occur in some buildings, as evidenced by San Fernando 1971 earthquake data (although in what ways the recorded magnifications are structurally significant is not yet clear). Confinement of column core concrete by detailing special transverse reinforcement is therefore essential. This confining reinforcement should be provided over the full column height, not just at possible hinge locations above and below the joint. The need to compensate for loss of strength, should spalling of cover concrete occur at any section or over the whole column length, was demonstrated in the San Fernando earthquake. The presence of lapped splices at mid-height of the storey is a further reason for providing confining steel in this region of the column.

The amount of confining reinforcement required by the Ministry of Works Code of Practice is in line with SEAOC and the 1971 ACI requirements, excepting that a footnote indicates that for columns less than 30" square subject to moderate axial loads, a lesser amount than given by the formula (but not less than the minimum allowable) may be adequate. For these conditions we have proportioned single hoops (without supplementary crossties) and square spirals in the volumetric ratio P_s range from the minimum 0.12 f'c/fy up to 0.15 f'c/fy (refer to the 1971 ACI Code, clause A.6.4). SEAOC 1971 revisions have reduced by one-third previously required amounts of confining steel above the minimum, and these revised requirements now correspond closely with Ministry of Works practice for practical column sizes (16" and above). McKenzie (14) has recommended that the unsupported length of rectangular hoops should be kept not greater than 15" by the addition of crossties. SEAOC 1971 revisions restrict supplementary crossties or legs of overlapping hoops to spacing not more than 14" on centre transversely.

Research on reinforced concrete column ductility has been carried out by Sampson (10) and Norton (8) at the University of Canterbury under Professor Park. The work shows that the column confining steel required to achieve reasonable curvature ductility during column hinging depends very much on the level of column axial load. Subject to more detailed study of these reports, it appears that practical confining steel limitations may dictate an upper limit to column axial load for acceptable ductility at about 0.5 f'c x Ag (where "balance" on the interaction curve is approximately 0.3 f'c x Ag). This may tend to increase column sizes and may also, to a varying degree depending on seismic base shear coefficient, limit the height of reinforced concrete seismic-resistant building frames.

11. THE BEAM-COLUMN JOINT

Joint deterioration and the resulting brittle failure must be prevented. The joint must be designed to maintain integrity and capacity of the core section throughout a number of large inelastic reversals, even though the cover concrete may have spalled away. It seems impossible to prevent spalling and diagonal cracking during inelastic action, but this must be minimised to keep resulting stiffness degradation of the joint as small as possible.

Design practice has been based mainly on tests by the U.S. Portland Cement Association. An important series of tests by Patton (15) and others at the University of Canterbury, supervised by Professor Park and Dr. Paulay, has provided valuable further information which will aid joint design and detailing. It is hoped that further work in this series will be undertaken, to include beam hinging conditions, little or no column axial compressions, and to check the joint shear criteria that have been proposed in Patton's report. The principal joint design considerations are summarised below.

11.1 Core Confinement

The column core should be confined throughout the full joint depth. Confining steel is calculated as for column confinement just outside the joint. For the special case of interior joints with beams framing in on all four sides under certain limiting conditions, code clauses permit the amount of steel to be reduced to one-half of that calculated. Patton has proposed a rather severe restriction on the laterally unsupported length of column core ties in the joint, in order to maintain effective confining pressures and minimise the risk of joint disruption. This should be the subject of further tests.

11.2 Joint Shear Reinforcement

The joint ties provided between the top and bottom beam bars should be adequate to resist the entire joint shear resulting from the column shear together with the concentrated shears due to yielding of the beam reinforcement. With beam tension steel at yield, the joint shear V_j for the exterior joint (fig. 8a) is:

$$V_j = Asfy - H$$

and joint shear for the interior joint (fig. 8b) is:

$$V_j = (Asfy)_1 + (Asfy)_2 - H.$$

These joint shears act along the direction of the beam or beams which are hinging. Assuming corner-to-corner cracking across a diagonal of the joint, Patton gives the total area of joint ties A_v to be placed between top and bottom beam bars (fig. 8d), taking $\phi = 0.85$, as:

$$A_v = \frac{V_j}{\phi . fy}$$

Concurrent hinging of all beams framing into the joint must also be allowed for, if joint failure is to be prevented. The resultant of the joint shears in the two principal directions acts along a diagonal or skew direction (fig. 8c) and is resisted by ties between top and bottom beam bars as for the uniaxial case in the previous paragraph. Cover concrete is considered ineffective. Ties should be of suitable shape to resist the shears in the skew direction (refer to the section on column shear).

Similarly to the core confinement requirements, codes permit the joint shear steel to be reduced to one-half of that calculated, if the special case of interior joints with beams framing in on all four sides under certain limiting conditions is applicable. This provision requires test investigation.

It should be noted that tie requirements in the joint are taken as the greater of those required for confinement and for shear. These ties are thus assumed to serve a dual purpose. Patton recommends that ties should preferably enclose the column core and longitudinal column bars to form an effective strut mechanism in the core.

11.3 Anchorage of Beam Reinforcement At Exterior Joints

Yielding beam bars should be anchored adequately by proper joint detailing based on available test information. University of Canterbury tests indicate that anchorage of beam bars by bending down within the column core leads to joint disruption. The tests confirm that it is very necessary to continue beam reinforcement through the exterior joint and anchor in beam stubs beyond the column core. This has been Ministry of Works practice, wherever possible, for some years.

11.4 Eccentric Beam-Column Joints

The serious embrittlement caused by eccentricity of beams framing into the column can cause premature failure of an otherwise ductile joint. The induced shears, torsions and moments may cause shear disruption of the joint, or may result in column hinging through reduction in available column capacity. Mead (16) has drawn attention to this problem and has discussed its significance.

In ductile concrete frame design, these risks are not acceptable and offset connections should be avoided.

12. CONCLUSION

The real earthquake performance demands on a frame may be very severe compared with calculated code loading conditions. For low framed buildings as well as for major structures, it is essential to design for ductility - i.e., ability to deflect well beyond yield.

Starting with beam longitudinal bar details obtained from code loadings, the paper describes the capacity design of beams, columns, and frame joints for yielding conditions. This approach considers the maximum forces which can exist in the real structure, and aims to balance strength, ductility, shear resistance, and concrete confinement in order to control hinge locations and prevent any form of premature, brittle failure.

Further ductility criteria will be found in the codes referred to, giving requirements for other structural members, for drift control, and for detailing. By considering maximum, real forces during yielding, criteria can be developed for special cases to aid in eliminating possible sources of brittleness, thus aiming to achieve ductile earthquake performance.

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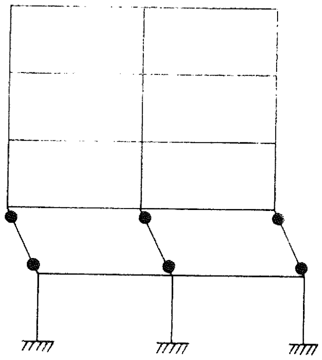


FIG. 1: "STOREY SWAY" COLUMN HINGING MECHANISM

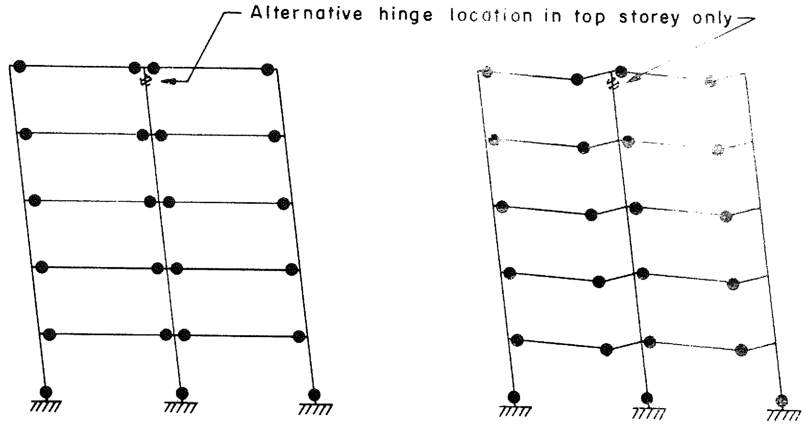


FIG. 2: BEAM HINGING MECHANISMS (Note hinges at base of columns)

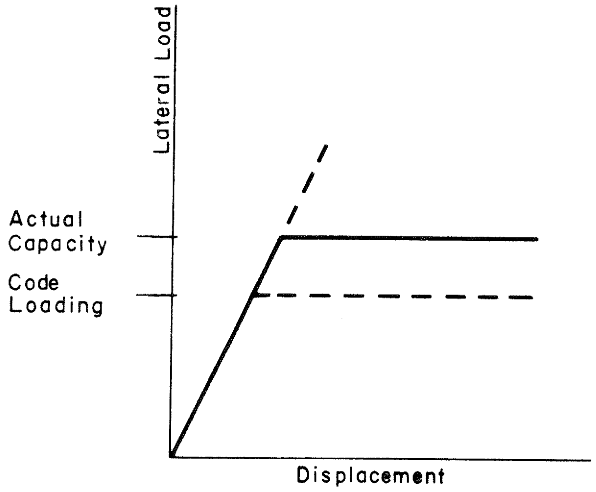
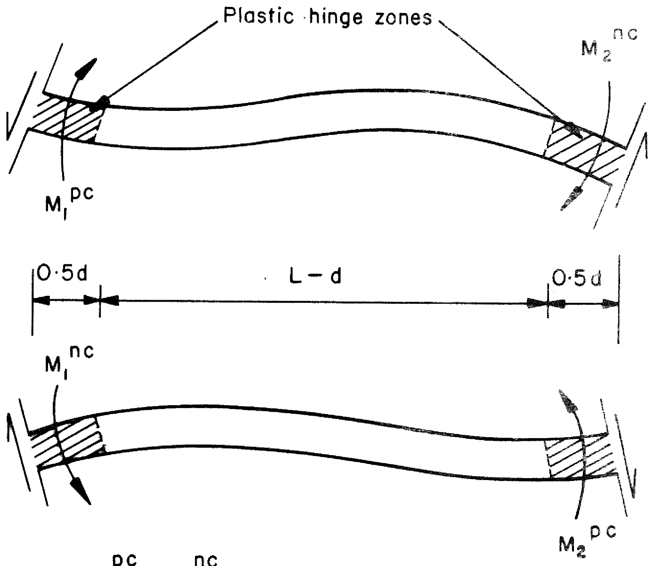


FIG. 3: LATERAL LOAD-DISPLACEMENT OF STRUCTURE



$$V = \frac{M_1^{pc} + M_2^{nc}}{L-d} + 1{\cdot}05\,V_D + 1{\cdot}27\,V_{LR}$$

$$V = \frac{M_1^{nc} + M_2^{pc}}{L-d} + 1{\cdot}05\,V_D + 1{\cdot}27\,V_{LR}$$

FIG. 4: BEAM HINGING SHEARS

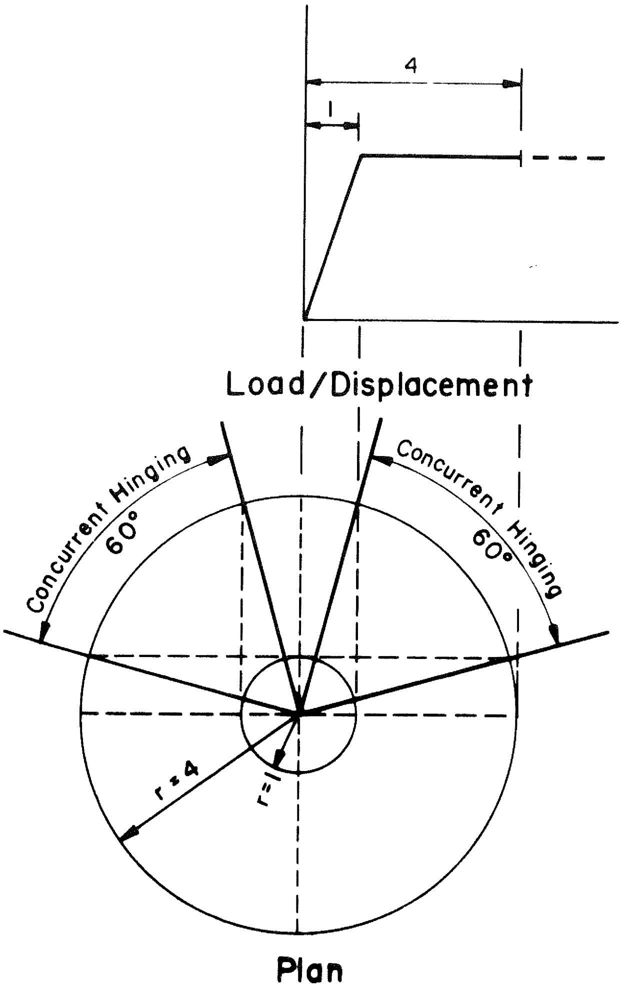


FIG. 5: CONCURRENT BEAM HINGING.

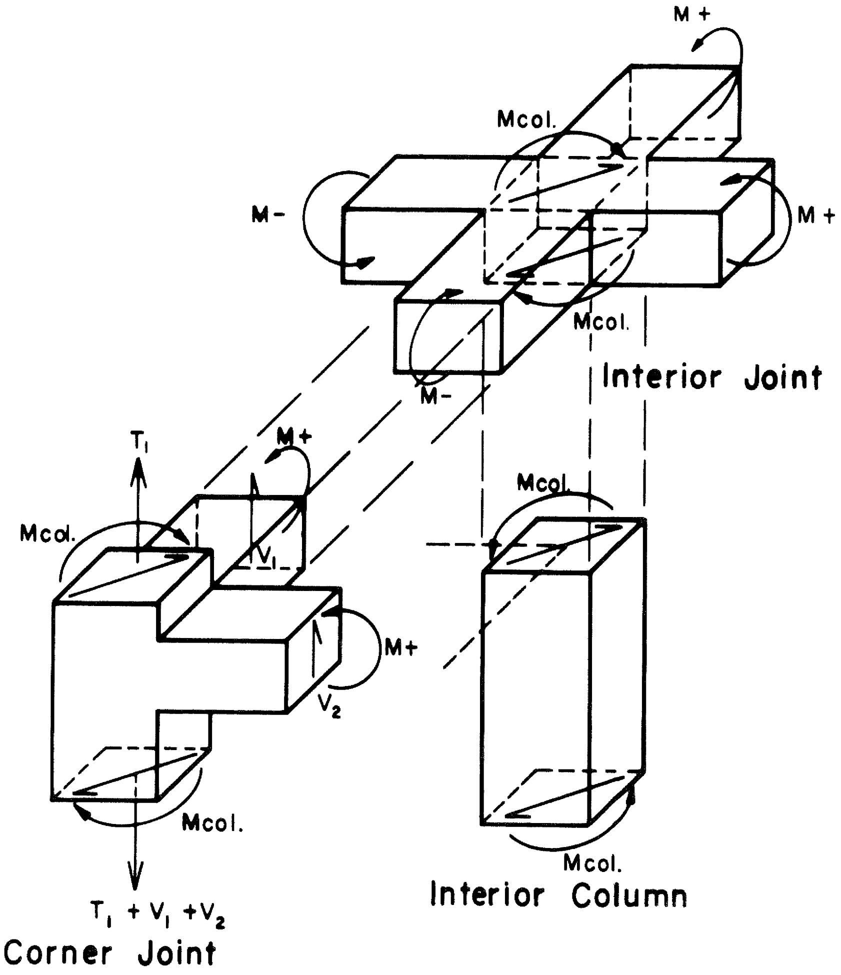


FIG. 6: EFFECT OF CONCURRENT BEAM HINGING ON COLUMNS AND JOINTS

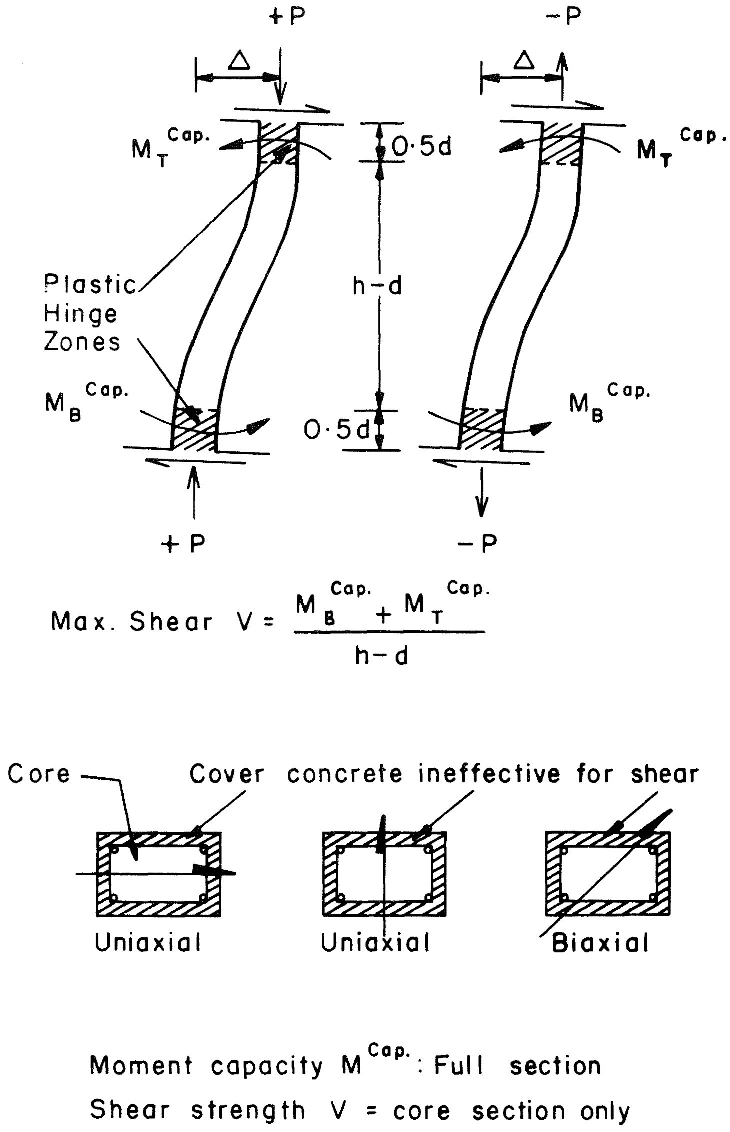
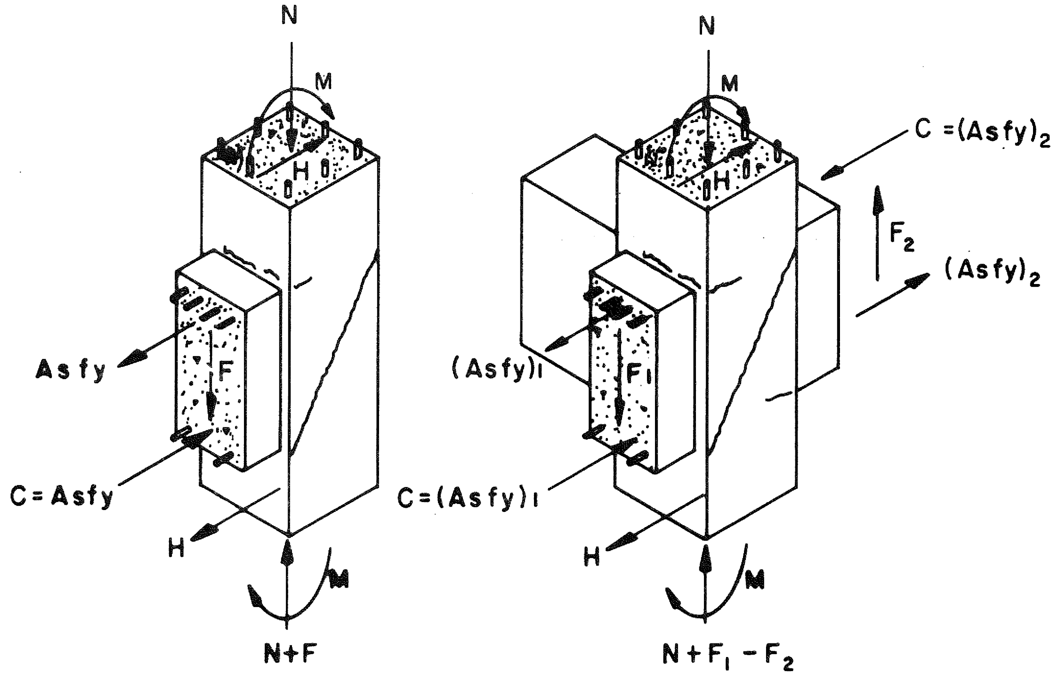


FIG. 7: COLUMN HINGING SHEARS

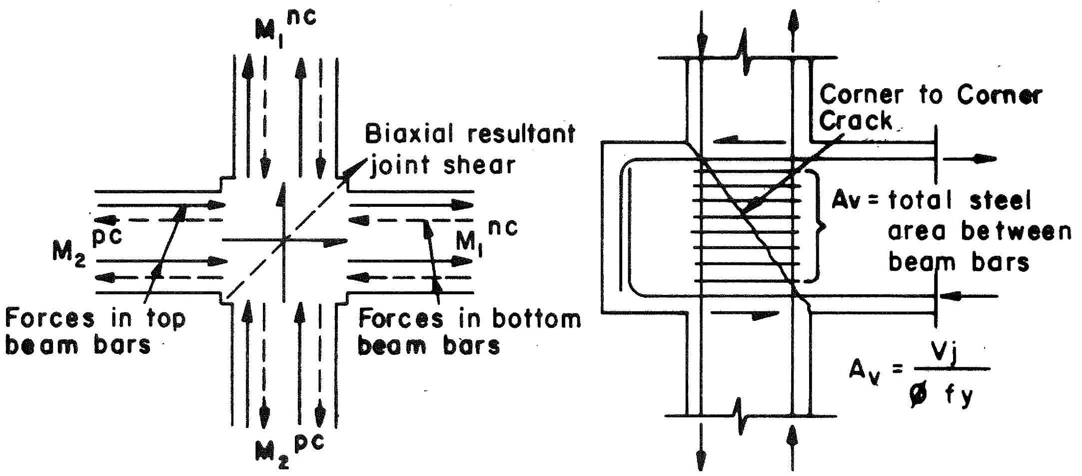


$V_j = Asfy - H$

$V_j = (Asfy)_1 + (Asfy)_2 - H$

a) Exterior Joint (Uniaxial Shear)

b) Interior Joint (Uniaxial Shear)



c) Large Biaxial Joint Shear due to Concurrent Beam Hinging

d) Joint Shear Reinforcement A_V

FIG. 8: BEAM-COLUMN JOINT SHEARS