

This paper is the result of deliberations of the Society's discussion group on
SEISMIC DESIGN OF DUCTILE MOMENT RESISTING REINFORCED CONCRETE FRAMES

SECTION H

COLUMNS SUBJECTED TO FLEXURE AND AXIAL LOAD

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H1.0 SCOPE

This section covers the design of columns of ductile frames subjected to flexure and ultimate axial compression exceeding $0.1f_c A_g$, where A_g = gross area of the column section and f_c = specified compressive cylinder strength of the concrete.

H2.0 DIMENSIONS

The dimensions of rectangular sections of continuous column members, where moments of opposite sign exist at each end, should be such that

$$\frac{l_n}{b} < 16 \quad (\text{H-1})$$

$$\frac{l_n h}{b^2} < 65 \quad (\text{H-2})$$

where l_n = clear height of column, h = column depth and b = column width. The depth and width of rectangular cantilever members should be such that

$$\frac{l_n}{b} < 10 \quad (\text{H-3})$$

$$\frac{l_n h}{b^2} < 38 \quad (\text{H-4})$$

where the notation is as above.

H3.0 MOMENT REDISTRIBUTION

The amount of redistribution of column moments found from elastic analysis for any combination of earthquake and appropriately factored gravity loading should not change the maximum end moment in any column, taken at the axis of the beam, by more than 15%. This limitation is satisfied if the redistribution of shear forces between columns is limited to 15% of the smaller shear force acting on any of the columns involved.

H4.0 COLUMN STRENGTH FOR FLEXURE AND AXIAL LOAD

The design bending moments and axial forces used should be determined from an elastic analysis which takes into account the maximum likely end moments and likely axial loads on the columns. Such an analysis should include the influence of probable beam overstrength and possible magnification of column moments due to dynamic effects, and should provide a high degree of protection against plastic hinging of columns during

the most severe seismic motions. A procedure which satisfies these requirements is given in Section G.

The full column cross section may be considered to contribute to the strength of the section.

H5.0 LONGITUDINAL REINFORCEMENT

H5.1 Longitudinal Reinforcement Ratio

The reinforcement ratio should not be less than 0.01. The reinforcement ratio should not be greater than 0.06 for grade 275 steel and not greater than 0.045 for grade 380 steel, except that in the region of lapped splices the total reinforcement ratio should not exceed 0.08.

H5.2 Distribution of Longitudinal Reinforcement

The longitudinal column bars in a potential plastic hinge region, defined as in Section H6.1.1, should not be spaced further apart between centres than one-third of the length of the section dimension in that direction, but need not be less than 200mm apart. The minimum bar diameter used should not be less than two-thirds of the maximum bar diameter used. Where longitudinal column bars are also utilised as vertical shear reinforcement in joint cores the distribution of bars should be in accordance with Section J4.3.

H5.3 Size of Longitudinal Bars

The maximum diameter of longitudinal column bars at any level should not exceed 1/20th of the depth of any beam framing into the column at that level for grade 275 steel and 1/25th of the depth of any beam framing into the column at that level for grade 380 steel.

H5.4 Anchorage and Splices of Longitudinal Bars

The anchorage lengths should be not less than that required by Chapter 12 of ACI 318-71. The allocation of longitudinal reinforcement should be such that yield can only occur at the potential plastic hinge regions defined in H6.1.1.

When longitudinal column bars are anchored at foundations or beam-column joint cores at the top of frame, the anchorage should be deemed to commence in the anchorage region a distance of one half of the relevant depth of the foundation member or beam, or 10 bar diameters, whichever is less, from the face at which the steel enters the foundation member or beam. Such bars should be terminated with at least a standard hook.

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Lap splices in longitudinal column bars should be located in the central region of the clear height of the column. If special transverse reinforcement, defined as in Section H6.1.2 or H6.1.3, exists over the full length of the lap splice the 0.75 factor of Section 12.5(d) of ACI 318-71 for bars enclosed within a specified steel spiral may be incorporated in the calculation for development length. Lap splices should be Class C tension splices (lap of $1.7l_d$), unless it can be shown that the maximum computed steel stress is always less than $0.5f_y$ throughout the splice length in which case Class B tension splices (lap of $1.3l_d$) may be used.

Welded splices or other positive connections may be used providing it can be shown that their performance is at least equivalent to that of lap splices.

H6.0 TRANSVERSE REINFORCEMENT

H6.1 Special Transverse Reinforcement in Potential Plastic Hinge Regions

Members should have special transverse reinforcement in the form of spiral or hoop reinforcement, with or without supplementary cross ties, in potential plastic hinge regions.

H6.1.1 Potential Plastic Hinge Regions

The potential plastic hinge regions should be considered to be at each end of a column above and below connections over a length from the faces of the connection of not less than the larger of (a) the longer column section dimension in the case of a rectangular column or the diameter of the section in the case of a circular column, (b) one-sixth of the clear height of the column, or (c) 450mm.

H6.1.2 Spiral Reinforcement

Where circular spiral reinforcement is used in potential plastic hinge regions the volumetric ratio of spiral reinforcement when $P_e < 0.7f'_c A_g$ should not be less than

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \left(0.375 + 1.25 \frac{P_e}{f'_c A_g} \right)$$

or (H-5)

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \left(0.375 + 1.25 \frac{P_e}{f'_c A_g} \right)$$

(H-6)

whichever is greater, where A_g = gross area of column section, A_c = area of core of column measured to outside of transverse confining steel, f'_c = specified compressive cylinder strength of concrete, f_{yh} = specified yield strength of transverse steel, and P_e = maximum design compressive load acting on column.

Columns with $P_e > 0.7f'_c A_g$ should not be used.

H6.1.3 Hoop Reinforcement

Where rectangular hoop reinforcement, including overlapping hoops and supplementary cross ties, is used in potential plastic hinge regions, the total area of hoop bars and supplementary cross ties in each of the

principal directions of the section when $P_e < 0.6f'_c A_g$ should be not less than

$$A_{sh} = 0.3s_h h'' \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \left(0.33 + 1.67 \frac{P_e}{f'_c A_g} \right)$$

or (H-7)

$$A_{sh} = 0.12s_h h'' \frac{f'_c}{f_{yh}} \left(0.33 + 1.67 \frac{P_e}{f'_c A_g} \right)$$

(H-8)

whichever is greater, where s_h = centre to centre spacing of hoop sets, h'' = dimension of the concrete core measured perpendicular to the direction of the hoop bars, and the other notation is as for Eqs. H-5 and H-6.

Columns with $P_e > 0.6f'_c A_g$ should not be used.

H6.1.4 Size, Distribution and Anchorage of Special Transverse Reinforcement

Circular Spirals

For circular spirals the minimum bar diameter should be 8mm. Anchorage should be provided by an extra one-half turn of the spiral bar. The spiral bar should be terminated either with at least a 135° bend around a longitudinal bar and an extension beyond the bend of at least ten spiral bar diameters, or by welding the spiral bar back on to itself. Spirals should not be lap splices but may be welded to develop the strength of the spiral bar. The clear spacing between spirals should not be less than 25mm. The centre to centre spacing between spirals should not exceed the smaller of (a) one-fifth of the column diameter, (b) 125mm, or (c) six times the diameter of the longitudinal bar.

Rectangular Hoops

For rectangular hoops and supplementary cross ties, the bar diameter should not be less than 8mm and the diameter of supplementary cross ties should not be less than two-thirds of the diameter of the peripheral hoop. Hoops should be anchored at longitudinal steel by at least a 135° bend with an extension beyond the bend of at least ten hoop bar diameters. Each end of a supplementary cross tie should engage either a longitudinal bar or the peripheral hoop beside a longitudinal bar with a bend of at least 135° and an extension beyond the bend of at least ten tie bar diameters. Alternatively, an equivalent welded anchorage may be used at the end of hoops and supplementary cross ties. Supplementary cross ties and legs of hoops should not be spaced transversely more than either 200mm or one-quarter of the column section dimension perpendicular to the direction of the transverse steel.

Each longitudinal column bar or bundle of bars should be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross tie, except that the following longitudinal bars are exempt from this requirement:

(a) Bars or bundles of bars which lie between two laterally supported bars or bundles of

bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed 200mm between centres.

(b) Inner layers of column bars within the concrete core centred more than 75mm from the inner face of hoops.

The yield force of the hoop bar or supplementary cross tie should be at least one-sixteenth of the yield force of the bar or bars it is to restrain, including the contribution from the bar or bars exempted under (a) above.

The spacing of hoop sets shall not exceed the smaller of (a) one-fifth of the smaller column section dimension, (b) 150mm, or (c) six times the diameter of the longitudinal bar to be restrained.

H6.2 Transverse Reinforcement Between Potential Plastic Hinge Regions

In regions of the columns between the potential plastic hinge regions the spacing of transverse steel should not exceed 300mm or one-half of the smaller column dimension or twelve times the diameter of the longitudinal bars, but other details shall comply with H6.1.4.

COMMENTARY

CH1.0 SCOPE

This section covers the seismic design requirements for columns of ductile frames. Columns when subjected to axial compression less than $0.1f_c A_g$ should be detailed as beams as in Section E except that the requirements of H5.0, and H6.1.4 in potential plastic hinge regions, should apply to all columns.

CH2.0 DIMENSIONS

The criteria for the limiting dimensions of rectangular sections are based on considerations of possible lateral instability of members and are similar to the criteria for beams. The background to the criteria is explained in Section CE4.1. No such requirements are necessary for circular sections.

CH3.0 MOMENT REDISTRIBUTION

It is permitted to vary the shear force acting on a column by up to 15%, but it is important that no shear force be "lost" in the process. That is, the total shear force acting across the columns of a storey should not be reduced. Moment redistribution in columns involves some plastic rotations in the members but the detailing for ductility for seismic actions will ensure sufficiently ductile members to achieve the permitted redistribution. Advantages to be gained by allowing some moment redistribution in design are discussed in section CE4.3.

CH4.0 COLUMN STRENGTH FOR FLEXURE AND AXIAL LOAD

The energy dissipation necessary for a multistorey frame to survive a severe earth-

quake should in general occur by the formation of ductile plastic hinges in beams. This is because the curvature ductility demand of the plastic hinges in columns can be unacceptably high if a column sidesway mechanism forms with plastic hinges at the top and bottom of the columns of one storey, whereas if plastic hinges develop in the beams up much of the height of the frame the curvature ductility demand of the beams is less severe and can be more easily provided⁽¹⁾. NZS 4203:1976 permits column sidesway mechanisms only in the case of single or two storey buildings, and in the top storey of multistorey buildings, because the curvature ductility demand of plastic hinges in columns in these cases is not high and can be met. Also, plastic hinges must be expected to develop in the columns at the base of multistorey buildings. Therefore, apart from these exceptions, a strong column-weak beam concept of design should be adopted. However, as discussed in Section CG.0, extremely high bending moments in columns can arise from the combined effects of overstrength beams, concurrent earthquake loading, and higher modes of vibration, so that the magnitude of column strength required to completely protect the columns is such that in a practical design some yielding of columns during a severe earthquake must be considered as inevitable. Note that column yielding resulting from shifts of the positions of the points of contraflexure away from the locations indicated from analysis for static code loading (caused by higher mode effects) will occur at only one end of columns in a storey and thus will not lead to a column sidesway mechanism in that storey. The procedure for the determination of design actions in columns recommended in Section G will give reasonable column protection but columns will still need to be detailed for ductile behaviour at potential plastic hinge regions since plastic hinges can be expected to form there for some instants during the earthquake.

The full cross section of the column may be considered to contribute to the strength of the section because column yielding is limited and if spalling of the concrete cover does occur the transverse confining steel present and strain hardening of the longitudinal steel will allow the column core to maintain a substantial moment and axial load.

CH5.0 LONGITUDINAL REINFORCEMENT

CH5.1 Longitudinal Reinforcement Ratio

The limiting amounts of longitudinal steel follow the ACI and SEAOC Codes except that the maximum quantity of grade 380 is reduced in view of the higher yield strength of that steel. In order to avoid excessive congestion of steel the total reinforcement ratios in the region of lapped splices is limited to 0.08. The minimum reinforcement ratio should not necessarily apply to the piles supporting the foundation members of framed structures.

CH5.2 Distribution of Longitudinal Reinforcement

In general bars should be distributed reasonably uniformly around the perimeter of the column section in order to assist

the confinement of concrete in potential plastic hinge regions. The column bars between the corner bars can also act as vertical shear reinforcement in joints if required (see Section CJ4.3).

CH5.3 Size of Longitudinal Bars

At any beam-column joint, except at the top of the upper storey columns and at the column-foundation joints, extremely high bond forces may develop in the longitudinal bars in columns due to the yield stress in tension possibly being reached at one beam face and a high compressive stress being reached at the other beam face when the frame develops large inelastic deformations during severe seismic motions. Hence the bar diameter needs to be restricted to a reasonable proportion of the bar length between top and bottom beam faces to ensure that a loss of column strength does not occur due to bond failure. As the probability of plastic hinges forming in a column above and below a beam simultaneously is smaller than the probability of plastic hinges forming in a beam each side of a column simultaneously, and because bond conditions are better for vertical bars, the maximum permitted bar diameter is greater than the values permitted for longitudinal steel in beams in Section E4.4.3.

CH5.4 Anchorage and Splices of Longitudinal Bars

Yielding of longitudinal column bars, when it occurs, should only take place at the ends of the columns where the special transverse steel is provided. To ensure that yielding does not occur elsewhere in the columns, the possibility of higher mode effects causing the locations of the points of contraflexure to shift away from the positions found from analysis for static code loading, should be taken into account when considering curtailment and splice location. The column end moments recommended in Section CG7.1 may be used to determine the distribution of column moments.

Plastic hinges are expected to form at the bottom of the lower storey columns and at the top of upper storey columns a significant number of times during a severe earthquake. Reversals of loading can lead to bond degradation which causes yielding of the longitudinal bars to penetrate into the foundation members at the column bases and into the beam-column joint cores at the top of the frame. Such yield penetration gradually reduces the effective development length of the bars. To ensure that anchorage is maintained the required development length is calculated assuming that yield has penetrate a distance of either one half of the relevant member depth or 10 bar diameters, whichever is less, into the relevant members.

Splices should be placed in the central region of columns, so as to be clear of potential plastic hinge regions if possible, because yield penetration into the lap region after cyclic loading could reduce the effective lap length and because cover concrete cannot be relied on in regions of high stress in a plastic hinge region to aid in the transfer of stress between lapped bars. If the special transverse reinforcement defined in either Sections H6.1.2 or H6.1.3 extends

throughout the whole height of the column, so as to exist over the full length of the lap splice, it may be deemed to have an equivalent confining effect as the steel spiral specified in Section 12.5(d) of ACI 318-71, leading to a 25% reduction in the required development length. Lap splices should generally be Class C tension splices because variations in column moments due to higher mode effects make it difficult to ensure that the steel stress will be always less than $0.5f_y$ in the splice region during severe earthquake motions. The steel stress in the column bars between the column ends may be calculated using the column moment gradient recommended in Section CG7.1.

In determining the criteria for welded splices and other positive connections the standard of workmanship, difficulty of inspection, and the final reliability of the splice should be taken into account. A site testing programme may be necessary.

CH6.0 TRANSVERSE REINFORCEMENT

CH6.1 Special Transverse Reinforcement in Potential Plastic Hinge Regions

CH6.1.1 Potential Plastic Hinge Regions

The potential plastic hinge regions occur at the ends of columns. The extent of the potential plastic hinge regions is based on SEAOC and ACI Code requirements.

CH6.1.2 and CH6.1.3 Spiral and Hoop Reinforcement

The special transverse steel to confine potential plastic hinge regions is based on the SEAOC Code requirements with the amount of transverse steel modified to take axial load level into account.

The amount of circular spiral steel specified in the SEAOC and ACI Codes is based on preserving the axial load strength of the column after the cover concrete has spalled rather than aiming to achieve a particular curvature ductility factor for the column. The amount of rectangular hoops specified is also based on the same criterion and assumes that rectangular hoops, because of their shape, are less efficient than circular spirals in confining the concrete. The philosophy of maintaining the axial load strength of columns after the spalling of cover concrete does not properly relate to the detailing requirements of adequate plastic rotation capacity of eccentrically loaded column sections. A more logical approach for the determination of the required content of transverse steel would be based on ensuring a satisfactory moment-curvature relationship and would include as variables the level of axial load on the column, the longitudinal steel ratio, the proportion of the column section confined, the stress-strain curve of the longitudinal steel, and the stress-strain curve of the confined concrete as a function of the amount of confining steel⁽¹⁾. Moment-curvature analyses of columns show a decrease in moment capacity when the cover concrete spalls but providing adequate confining steel is present sections can maintain substantial moment with further plastic rotation. A difficulty with refined moment-curvature

analyses of this type is that insufficient experimental data is available to properly establish the stress-strain curve for confined concrete including the effect of overlapping hoops and hoops with supplementary cross ties. Only moment-curvature analyses for monotonic loading based on approximate stress-strain curves for confined concrete are available at present (1,2,3). However, such analyses have shown that the equations for transverse steel content recommended in the 1973 SEAOC Code are generally conservative for moderate axial load levels but not conservative at high load levels. Thus the SEAOC equations have been modified to take axial load level into account. Eqs. H-5 and H-6 for spiral reinforcement result in the following amounts of spiral steel being placed as a percentage of the amount recommended in the 1973 SEAOC Code:

$P_e/f'_c A_g$	0.1	0.2	0.3	0.4	0.5	0.6	0.7
% of SEAOC ρ_s	50	63	75	88	100	113	125

Eqs. H-7 and H-8 for hoop reinforcement result in the following amounts of hoop steel being placed as a percentage of the amount recommended in the 1973 SEAOC Code:

$P_e/f'_c A_g$	0.1	0.2	0.3	0.4	0.5	0.6
% of SEAOC A_{sh}	50	66	83	100	117	133

The moment-curvature analyses (1,2,3) have indicated that use of the recommended equations H-5 to H-8 will result in curvatures being reached which generally are much greater than five times the yield curvature (defined as when the outermost tension bars first begin to yield) accompanied by a moment capacity which is generally not less than 0.8 times the moment capacity calculated at an extreme fibre concrete strain of 0.003 (for columns with longitudinal steel ratio of 0.02 or greater) providing that the axial compression does not exceed either $0.7f'_c A_g$ for columns with circular spiral reinforcement or $0.6f'_c A_g$ for columns with rectangular hoop reinforcement. Also, experimental evidence (4) from spirally reinforced cantilever columns tested under cyclic flexure with displacement ductility factors of up to six have confirmed that the spiral steel content recommended by Eqs. H-5 and H-6 for axial compression of $0.1f'_c A_g$ should result in adequate ductility. However the moment-curvature analyses have shown that the curvature ductility available from very heavily loaded columns is doubtful even with large quantities of confining steel (2,3) and it is recommended that spiral columns with $P_e > 0.7f'_c A_g$ and hooped columns with $P_e > 0.6f'_c A_g$ shall not be used unless special studies show they have adequate ductility. A greater limiting axial load for columns with spiral reinforcement compared with columns with rectangular hoop reinforcement is recommended as it has been found from analytical studies that the SEAOC specified spiral steel confines the concrete more effectively than the specified hoop steel. The moment-curvature analyses (2,3) have also shown that use of grade 380 steel as longitudinal reinforcement in columns improves the performance of the columns at high curvatures because the early strain hardening of that steel helps to compensate

for the loss of moment strength due to the reduction of contribution from the concrete.

CH6.1.4 Size, Distribution and Anchorage of Special Transverse Reinforcement

Circular Spirals

For circular spirals the details of minimum bar diameter and spacing given in H6.1.4 are mainly based on long standing ACI and SEAOC practice. An exception to ACI and SEAOC practice is that at each end of the spiral unit after an extra one-half turn of spiral the bar should be anchored either by bending around a longitudinal bar followed by a 10 diameter extension into the confined core, or by welding the spiral bar back onto itself. A further exception is that lap splices of spiral bars are not permitted, since if the cover concrete spalls the spiral will become ineffective due to loss of bond at the lap. If spirals cannot be constructed from a single length of bar, the bars should be welded together so as to be capable of developing the strength of the bar, which could be defined as the smaller of $1.6f_y$ or the breaking strength of the bar. A further exception to ACI and SEAOC practice is that the centre to centre spacing between spirals is not to exceed one-fifth of the column diameter or 125mm, rather than the maximum clear spacing of 75mm permitted by the ACI and SEAOC Codes. The maximum allowable spiral spacing should be kept reasonably small because concrete is confined mainly by arching between spirals and if the spiral spacing is large a significant depth of unconfined concrete will penetrate into the concrete core between the spirals and thus reduce the effective confined concrete section. The above allowable maximum spiral spacings are greater for larger columns since a greater penetration of unconfined concrete between the spirals is less significant for such columns. The additional requirement that the spiral spacing should not exceed six longitudinal bar diameters is to prevent buckling of longitudinal steel when undergoing yield reversals in tension and compression. It is well known that such stress reversals in the yield range cause a reduction in the tangent modulus of the steel at relatively low stresses, due to the Bauschinger effect, and therefore closely spaced transverse steel providing lateral support is required to prevent buckling of the longitudinal steel.

Rectangular Hoops

In most rectangular columns a single rectangular peripheral hoop will be insufficient to properly confine the concrete and laterally restrain the longitudinal steel against buckling. Therefore an arrangement of hoops with or without supplementary cross ties will be necessary. The restriction on the variation of bar diameter is to ensure a reasonable distribution of transverse steel across the section. Supplementary cross ties can only be expected to function effectively if fitted tightly around the bars, a rather difficult requirement in practice. Any gap left between the inside of the bend of the cross tie and the outside of the laterally supported bar will mean that outward expansion of the concrete needs to occur before the cross tie becomes fully effective, and thus some concrete confinement is lost.

It would appear to be better to use a number of overlapping hoops rather than a single peripheral hoop and supplementary cross ties. An example of alternative details and the preferred arrangement is shown in Fig. CH.1. Note from Fig. CH.1(a) and (b) that a supplementary cross tie can engage either the longitudinal bar or the peripheral hoop beside a longitudinal bar. That is, the concrete is confined by arching between hoops, supplementary cross ties and longitudinal bars. In a set of overlapping hoops it is preferable to have one peripheral hoop enclosing all column bars together with one or more hoops covering smaller areas of the column section. This is because such a detail is easier to construct since the longitudinal bars are held more firmly in place if they are all enclosed by one hoop. Thus the detail in Fig. CH.1 (c) which has a hoop enclosing all bars and a smaller hoop enclosing the middle four bars is to be preferred to the detail in Fig. CH.1(d) which has two hoops each enclosing six bars. Fig. CH.2 shows typical details using overlapping hoops for columns with a greater number of longitudinal bars. It is to be noted that the inclined hoop surrounding the four bars at the centre of each column face in Fig. CH.2(b) can be counted on making a contribution to A_{sh} in Eqs. H-7 and H-8 by determining the equivalent bar area of the component of forces in the required direction. For example, two such hoop legs inclined at 45° to the column sides could be counted as making a contribution of $\sqrt{2}$ times the area of one perpendicular bar in assessing A_{sh} . That is, in Fig. CH.2(b) A_{sh} may be taken as $5.41A_{te}$, where A_{te} is the area of each hoop bar. Note also that the spacing between adjacent hoop legs or supplementary cross ties should not exceed either 200mm or one-quarter of the section dimension, as is illustrated in Fig. CH.1 and CH.2, in order to ensure reasonable confinement from vertical and transverse steel.

The hoops and supplementary cross ties in columns are also necessary to provide lateral support to the longitudinal bars to prevent buckling. Hence, as for spiral columns, the spacing between centres of hoop sets should not exceed six longitudinal bar diameters. However not all bars need to be laterally supported by a bend in a transverse hoop or cross tie. If bars or groups of bars which are laterally supported by bends in the same transverse hoop or cross tie are less than or equal to 200mm apart, any bar or bundle of bars between them need not have effective lateral support from a bent transverse bar, as is demonstrated in Fig. CH.2(a).

Also, bars which lie within the core of the column centred more than 75mm from the inside face of the peripheral hoop need no special lateral support. When supplementary cross ties are anchored by bending around the hoops they need not enclose the adjacent longitudinal bars as well (see Fig. CH.1 (b)). However, such supplementary cross ties should be secured to longitudinal bars at each end so as to allow the hoop to give effective lateral support to those longitudinal bars. That is, although the supplementary cross ties do not pass around the four longitudinal bars in Fig. CH.1 (b) they can be regarded as providing effective lateral support to those longitudinal bars since they effectively restrain the hoop beside the bars. Alternatively,

the supplementary cross tie can engage the longitudinal bar directly as is illustrated in Fig. CH.1(a). The requirement that the yield force of the hoop bar or supplementary cross tie should be at least one-sixteenth of the yield force of the bar or bars to be restrained is similar to that for beams and an explanation of this requirement is given in sections E.5 and CE.5.

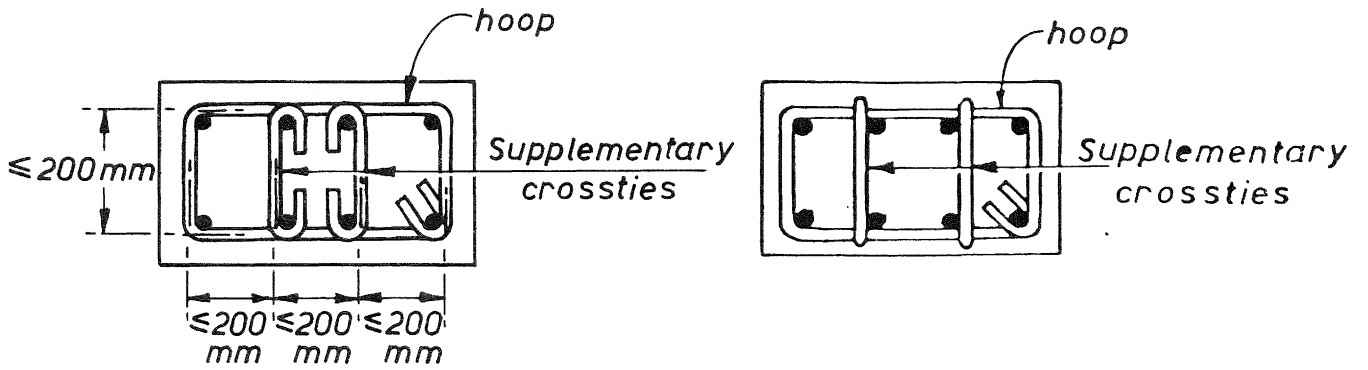
The spacing between centres of hoops should not exceed one fifth of the smaller column section dimension or 150mm in order to ensure effective arching of confined concrete between the hoop sets. The requirement that the hoop spacing should not exceed six longitudinal bar diameters is to prevent buckling of the longitudinal reinforcement when subjected to stress reversals in the yield range.

CH.6.2 Transverse Reinforcement Between Potential Plastic Hinge Regions

In regions where yielding of longitudinal reinforcement cannot occur the spacing of transverse steel can be increased because the confinement of the concrete and the prevention of buckling of steel become less critical items.

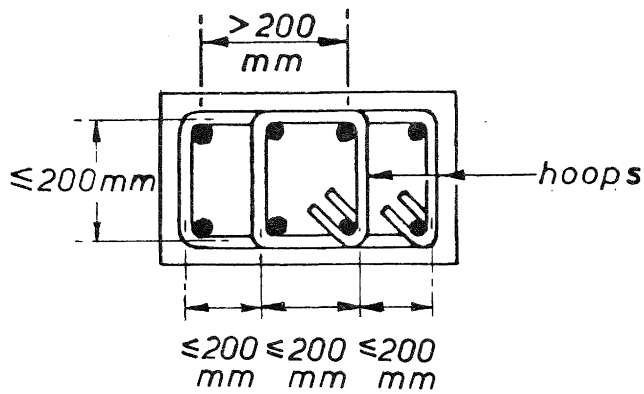
REFERENCES

1. R. Park and T. Paulay, "Reinforced Concrete Structures", John Wiley & Sons, New York, 1975.
2. J. A. Norton, "Ductility of Rectangular Reinforced Concrete Columns", Master of Engineering Report, University of Canterbury, 1972.
3. P. D. Leslie, "Ductility of Reinforced Concrete Bridge Piers", Master of Engineering Report, University of Canterbury, 1974.
4. M.J.N. Priestley, R. Park, B. E. Davey and I.R.M. Munro, "Ductility of Reinforced Concrete Bridge Piers", Proceedings of 6th World Conference on Earthquake Engineering, New Delhi, 1977.

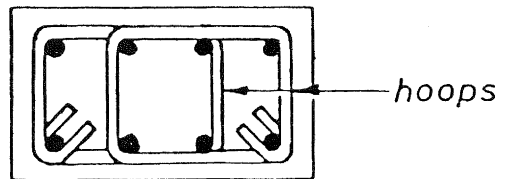


(a) Single hoop plus two supplementary cross ties bent around longitudinal bars

(b) Single hoop plus two supplementary cross ties bent around hoop

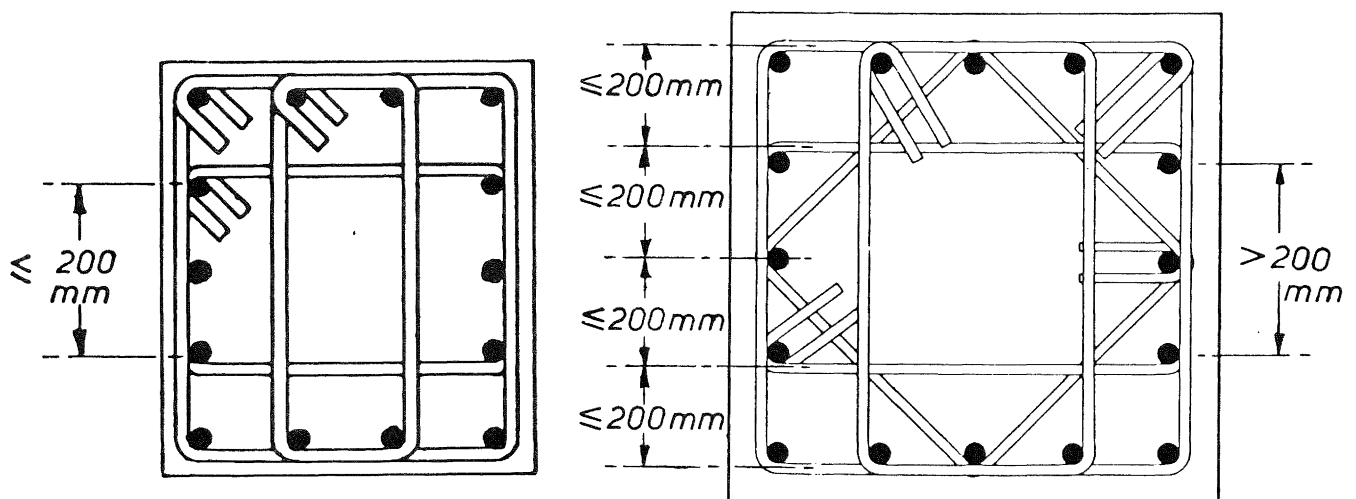


(c) Two overlapping hoops - preferred detail



(d) Two overlapping hoops - not preferred to (c)

FIGURE C.H.1: ALTERNATIVE DETAILS USING HOOPS AND SUPPLEMENTARY CROSS TIES



(a) Three overlapping hoops

(b) Four overlapping hoops

FIGURE C.H.2: TYPICAL DETAILS USING OVERLAPPING HOOPS.