Field experience section

A CRITICAL LOOK AT THE DESIGN OF REINFORCED CONCRETE FOR DUCTILITY

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1. Area in which More Information is Required

In designing adequate ductility into a frame structure to resist earthquake forces, the portion of the procedure which has the least satisfactory theoretical and experimental basis is the strictly design part. This is the section in which the designer, having assessed the ductility demands of the various members of the frame, proceeds to decide on the concrete dimensions and reinforcement pattern which will satisfy these demands. The reasons for the shortcomings are fairly obvious. Unlike frame analysis and frame response, which can be dealt with as tidy applied mechanics problems, investigation of the design problems requires research workers with an extensive design background, and an awareness of all the design criteria which have to be satisfied. Unfortunately, such research workers are rare, and consequently insufficient research has been carried out. What research has been done has obviously not been planned with a view to obtaining information that can be used in design.

2. Present Design Procedures

The design procedures most used in New Zealand at present are those set out in the Recommended Lateral Force Requirements of SEAOC (Structural Engineers' Association of California). These were originally published in 1966, and subsequently revised in 1967 and 1968. They are a development from an earlier set of design procedures published in the book, "Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions" by Blume, Newmark and Corning (which will henceforth be referred to as B.N.C.).

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3. Summary of SEAOC Requirements

The main provisions of the SEAOC requirements can be summarised as attempting to accomplish the following in frame design:

(1) To provide adequate plastic hinge rotation capacity in all members, adjacent to member junctions, by the addition of special transverse confining reinforcement at these locations.

(2) To ensure that, at any beam-column junction, the columns are strong enough to remain in the elastic range while the beams develop plastic hinges. (However, note comments later in this article about hinges at the bottom of ground floor columns).

(3) To ensure that the joint areas are heavily enough reinforced to remain unchanged when the beams form plastic hinges.

(4) To ensure that all members have sufficient shear capacity to develop plastic hinges without failing in shear.

(5) To prevent compression failures in flexural members, even when substantial hinge rotations occur, by using appropriate amounts of tension and compression reinforcement.

4. General Shortcomings of SEAOC Procedures

Once a designer starts to use the SEAOC procedures, he soon becomes aware of two major defects.

(1) Many of the reinforced concrete design criteria have a doubtful theoretical or experimental basis. What experimental evidence does exist generally derives from small test specimens. The extrapolation of this to cover large building members, without taking account of scale factors, gives cause for considerable concern. Again, the small scale tests carried out to date have <u>not</u> formed part of a co-ordinated programme of tests to check the effect of variations in all parameters on member ductility.

(2) Cases regularly arise where the design criteria result in impractical structural details which do not appear to be justified.

Below are set out some of the requirements which, in the opinion of the writer, appear to require modification or appear to have a doubtful basis.

5. Plastic Hinges at Bottom of Ground Storey Columns

There are several good reasons why plastic hinges should be developed in the beams of a structure rather than in the columns, when the structure is subjected to lateral seismic forces. Hence the SEAOC Requirements quite properly give design rules to ensure that hinges form in beams, rather than columns, at all levels of the structure except the base. However, this hinge mechanism does require formation of hinges at the bottom of the columns in the lowest storey - see fig. 1.

The designer should concentrate his attention on this bottom column hinge. At all other beam to column junctions, although column confinement is required adjacent to junctions, it is very unlikely that column hinges will form, if the SEAOC provisions governing relative ultimate strengths of beams and columns have been applied. In contrast, the hinge at the bottom of each ground storey column is certain to form, when the corresponding frame deflects into the plastic range under lateral seismic forces.

6. Severity of Hinge Rotations at Bottoms of Columns

To make matters worse, under dynamic conditions, this hinge could be subjected to particularly severe rotations. Figure 2 will illustrate this point. It is normally assumed that earthquake forces are fed in as static forces acting simultaneously at all floor levels, to give the deflected shape shown by the dotted lines. In actual fact, earthquake forces are fed to the structure as heavy dynamic ground pulses. The bottoms of the columns will move with the ground pulse, but the floors will show considerable delay in following this movement. Hence, dynamic deflection at a particular instant could be as shown by the full lines in figure 2, with greater distortion of the bottom storey and greater rotation of the hinge at the bottom of the column.

7. Doubt about SEAOC Procedures for Confining Steel for Large Columns

Unfortunately, the base hinge forms at the maximum column cross section, which, in a high rise building, is far greater than in the small laboratory specimens on which any rotation capacity tests for members subjected to axial load and bending have so far been carried out. This is really disturbing, because the SEAOC procedures for calculating confining steel tend to give much less effective confinement as the member size grows larger, particularly where rectangular ties are used.

The following effects illustrate this.

(1) Ratio of steel requirements in square and round columns of the same overall dimensions. (The formulae given in the 1968 revision of the SEAOC requirements are used here). Consider the columns shown in fig. 3(a) and (b), with the lateral bursting pressure H developed in each case by hinging. Use the same notation as is used in the SEAOC requirements. h" is the core dimension of the square column. Let the core diameter of the round column be D.

In the case of the round column, bursting pressure develops a tension in the spiral which is proportional to D.

In the case of the square column, bursting pressure develops a bending moment in the hoop steel which is proportional to $(h'')^2$.

Hence, if D is increased 3-fold, the area of spiral steel needs to be trebled. On the other hand, if "h" is increased by a factor of 3, the area of hoop steel needs to be increased by a greater factor than 3 (because it has to resist bending moment). Yet, the SEAOC code formula assumes that the ratio of area of rectangular hoop steel to area of spiral steel required for D = h", is 2 in both cases. If 2 is a safe value in the case of the smaller value of D & h", (which approaches the cross section of specimens used in research studies on columns), it is no longer a safe value when both these values are trebled, to approach the dimensions of high rise building columns.

(2) Formula for area of spiral steel in round spiral columns.

To resist a constant bursting pressure, the area of spiral steel required should be proportional to D, and the volume ratio of spiral steel (p") should be constant, regardless of column diameter. However, the actual formula used by SEAOC is:

$$p'' = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f''_y}$$

with a lower limit of

$$p'' = 0.12 \frac{f'_c}{f''_y}$$

Because of the term $(\frac{A_g}{A_c} - 1)$ in the formula, the volume ratio actually decreases as column diameter increases. The following illustrations show this.

Overall diameter of round column	Cover to spirals	Value of $p'' \frac{f''_y}{f'_c}$
15"	1½"	0.256
18"	1½"	0.196
24"	1½"	0.137
27" or greater	1½"	0.12

To summarise, using SEAOC design procedures for confining steel, the safety of spirally reinforced round columns against bursting pressure can be expected to decrease greatly as the dimensions increase, and the safety of square columns would drop to an even greater extent as dimensions increase. Hence the SEAOC procedures could design a small column for a laboratory experiment which would then show adequate confinement and rotation capacity, and yet could provide inadequate confining steel in a large building column.

8. Summary of Positions Regarding Present SEAOC Hoop Provisions for Large Columns

Thus we finish up with the position that, to develop overall frame ductility, the bottom of each column in a high rise building will certainly be subjected to severe plastic hinge rotations. Yet we do not know whether the SEAOC design procedures will give sufficient confining steel in this location to enable the column to withstand such rotations, because the only experimental evidence relates to much smaller sections.

9. Author's Recommended Design Procedure for Confinement of Plastic Hinge Areas in Large Columns

A designer faced with design of a building is not able to sit back and await additional research information appropriate to his problem. He has to solve the problem forthwith. In the case set out above he has to attempt to relate it to any experimentally based information which he has available, in this case information relating to smaller cross sections. Referring to figs. 3(c), 3(d) and 3(e), assume that in each case the bursting pressure developed by hinge rotation within the confined area is the same. In 3(c) the hoop tension developed is T and the steel area required in each leg of the spiral is A. In 3(d) the tension developed in each leg of the rectangle is T. There is, in addition, a lateral bending force, distributed over the span AB, which is equal to 2T and which causes a bending moment. Assume that, to resist this combination of tension and bending moment, the SEAOC provision of an area of 2A in each leg of the rectangular hoop, is adequate for a small section of the dimensions shown.

Now let us turn our attention to fig. 3(e). This is a rectangular column where the dimensions of the confined core A'D'D"A" is three times that of 3(d). Let us now add the ties B'B", C'C", EF and GH, welding the ends of the ties to the sides of the main rectangular hoop. Now compare A'B' in 3(e) with AB in 3(d). The tension will be the same in both cases, and the bending moment diagrams due to lateral forces will be exactly the same. Hence, if the same cross sectional area, 2A is used for the hoop leg in both cases, the confining effect will be exactly the same. The tension in each of the cross ties will be equal to 2T, and there will be no accompanying bending moment, and, therefore, from the case of fig. 3(c), a cross sectional area of 2A will be adequate for this case.

For a large size column, at the bottom of the ground storey, where severe hinge action can be expected, the reinforcing scheme shown in fig. 3(e) is recommended. The confining hoops for case 3(d) are calculated, and then adapted to give the corresponding scheme for 3(e). The welding makes the fabricated hoops in 3(e) expensive, but this one hinge is a very small part of the structure, so the effect on overall cost is negligible.

Ties in the form of "hairpins" which are not welded in place could not be relied upon to give the same performance under extreme conditions.

The point is that a designer can not afford to avoid taking special precautions at this critical point. If severe damage to the column occurred at this point, it is one part of the structure on which repair work would be almost impossible.

Note that the following normal code provision for the lateral support of longitudinal rods must be applied to the hinge area as well as to the remainder of the column. "The ties shall be so arranged that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie having an included angle of not more than 135 deg. and no bar shall be farther than 6 in. from such a laterally support bar". In 3(e), the welded junction of a cross tie and the side of the main rectangular hoop can be considered equivalent to the corner of a tie for providing lateral support.

10. Necessity for Additional Research

Note that the soundness of the design for 3(e) depends on the adequacy of confinement obtained in cases 3(c) and 3(d). Hence further research on small dimension rectangular and spiral columns is required, to check this point, and to check the effect of variation of all parameters, including shear stress and longitudinal steel ratios and applied axial load, on the confining steel required. In particular, a formula for calculating confining steel with a sound theoretical basis, is required, as pointed out in the next section. This could involve an investigation into whether bursting pressure for a given value of ϕ_u/ϕ_y was constant for different values of member cross section and whether the bursting pressure increased as the compressive strength of the concrete increased. It is emphasized that the investigations must be planned to give information which can be used in design. The big weakness in research which has been carried out to date is that it has been on scattered areas of the problem, it has not been comprehensive and it has not been planned to provide a design basis.

11. Reducing Restrictions on Member Proportions

The SEAOC provisions require that both flexural members and columns shall not have a width-depth ratio of less than 0.4. This restriction appears reasonable, because the stirrup shape corresponding to a deep narrow beam would appear to be less effective in confining concrete than a square stirrup. However, it appears possible to overcome this problem by providing a series of overlapping hoops of approximately square section, as shown in 4(a), or by welding in cross ties as in 4(b). The area A would be the same as in a square hoop round a single confined core of square section, with length of side equal to h".

This proposed provision would be very useful for deep membered frames (formed by a shear wall with a pattern of openings) and deep spandrel beams. The writer has initiated the use of this provision in Ministry of Works' designs several years ago.

12. Suggested Modifications to SEAOC Formula for Amount of Confining Steel

The 1968 Revisions to the SEAOC Requirements require that the total cross-sectional area (A''_{sh}) of rectangular hoop reinforcement shall not be less than

$$A''_{sh} = 0.45\ ah'' \frac{f'_c}{f''_{yh}} \left(\frac{A_g}{A_c} - 1\right) \quad \text{(Referred to below as equation (1)).}$$

nor $$A''_{sh} = 0.12ah'' \frac{f'_c}{f''_{yh}}$$

whichever is greater

where A_c = area of column core
A_g = gross area of column
A''_{sh}= total cross sectional area in square inches of hoop reinforcement having a spacing of (a) inches and crossing a section having a core dimension of h".

This gives very impractical results on small columns, particularly of a generous cover is provided, because of the factor $\left(\frac{A_g}{A_c} - 1\right)$ in equation (1).

For example, in a 16 inch square column with 1½" cover all round to outside of hoops, No. 6 hoops at 3 inch spacing are called for by equation (1). On the other hand, in a 15 inch square column with 1" cover all round to outside of hoops, only No. 4 bars at 3 inch spacing are called for by equation (1). Yet, once the outside cover has spalled off, due to severe hinge rotations, the remaining confined core has the same dimensions in both cases. It appears illogical to require that the amount of hoop steel to give ductility be dependent on the ratio of $\frac{A_g}{A_c}$ because the cover concrete will split off when extreme ductility is developed, and A_g will then no longer be a property of the column.

The only justification for equation (1) is that it does provide sufficient hoop steel to bring the strength to resist axial load, after the cover concrete has spalled off, up to the value for the full original concrete section.

However, a better basis for design appears to be <u>to design the confined core on its own, without any assistance assumed from the cover concrete</u>, to resist combined axial load and bending moment. The confining steel could then be designed to resist bursting pressure resulting from severe hinge rotations. Subject to check by laboratory testing, which is recommended in the section above, the following formula is suggested <u>for values of</u>

h" up to 15".

$$A''_{sh} = 0.15ah'' \frac{f'_c}{f''_{yh}}$$

For large columns, cross ties, as shown in figure 3(e), could be added to the hoops to keep the effective value of h" no greater than 15". For the bottom of the ground storey, the cross ties must be welded to the hoops. For all other localities where special transverse column reinforcement is called for, the cross ties may have hooks at each end which engage both horizontal and vetical bars. (This is because it is most unlikely that columns will develop hinges at these other locations). The straight portions at the ends of the hooks should have a length of at least 10 bar diameters.

13. Design of Shear Stirrups Adjacent to Beam-Column Junctions

-Column Junctions.

The SEAOC Requirements allow the concrete in these areas to take part of the shear, in accordance with the normal provisions in ACI 318-63. The writer feels very strongly that, in any areas where plastic hinges could form, shear stirrups or hoops should be designed to take the full shear. Successive reversals of severe plastic rotations can set up intersecting patterns of cracks, which open up progressively further with each reversal. In the absence of adequate research information on concrete shear strength in these circumstances, it appears prudent to design stirrups in these locations to take the full shear, without relying on a portion being carried "on the concrete alone".

14. Transverse Reinforcement in Beam-Column Joint

This is covered in section 2630(e)5 of the SEAOC Requirements, which points out that the transverse reinforcement in this area has to be designed for shear as well as for confinement. V_u is calculated by assuming that the beam reinforcing steel is up to yield stress, and the shear reinforcement required is calculated accordingly. Referring to fig 5(a), the shear in the joint area = Af_y.

This gives very heavy and closely spaced stirrups in some cases, which are almost impossible to place. A different approach has been introduced by the writer into Ministry of Works designs. Referring to fig. 5(b), the beam steel is taken to the far face of the column, and given full anchorage beyond point P. This can be considered to transfer the force from the beam steel into the column at point P. Then, provided that the depth of the beam is not greater than the depth of the column, a diagonal compression can be considered to carry the shear across the joint, from P to the centre of compression near the bottom of the beam (point N). This is consistent with the normal truss anology on which shear stirrups are calculated - if a stirrup was provided at P which could carry the whole of the shear force, no other stirrups would be required between P and N. Note that reversal of seismic moment in the beam must also be designed for. It has been suggested to University of Canterbury that research should be carried out on typical joints to check this mechanism.

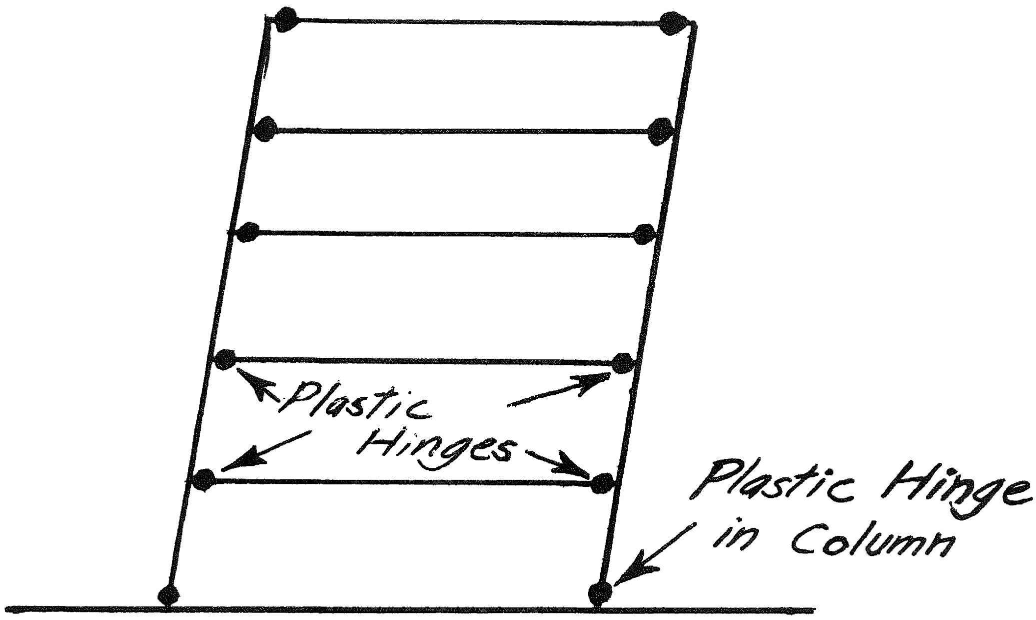

Plastic
Hinges
Plastic Hinge
in Column

Fig 1

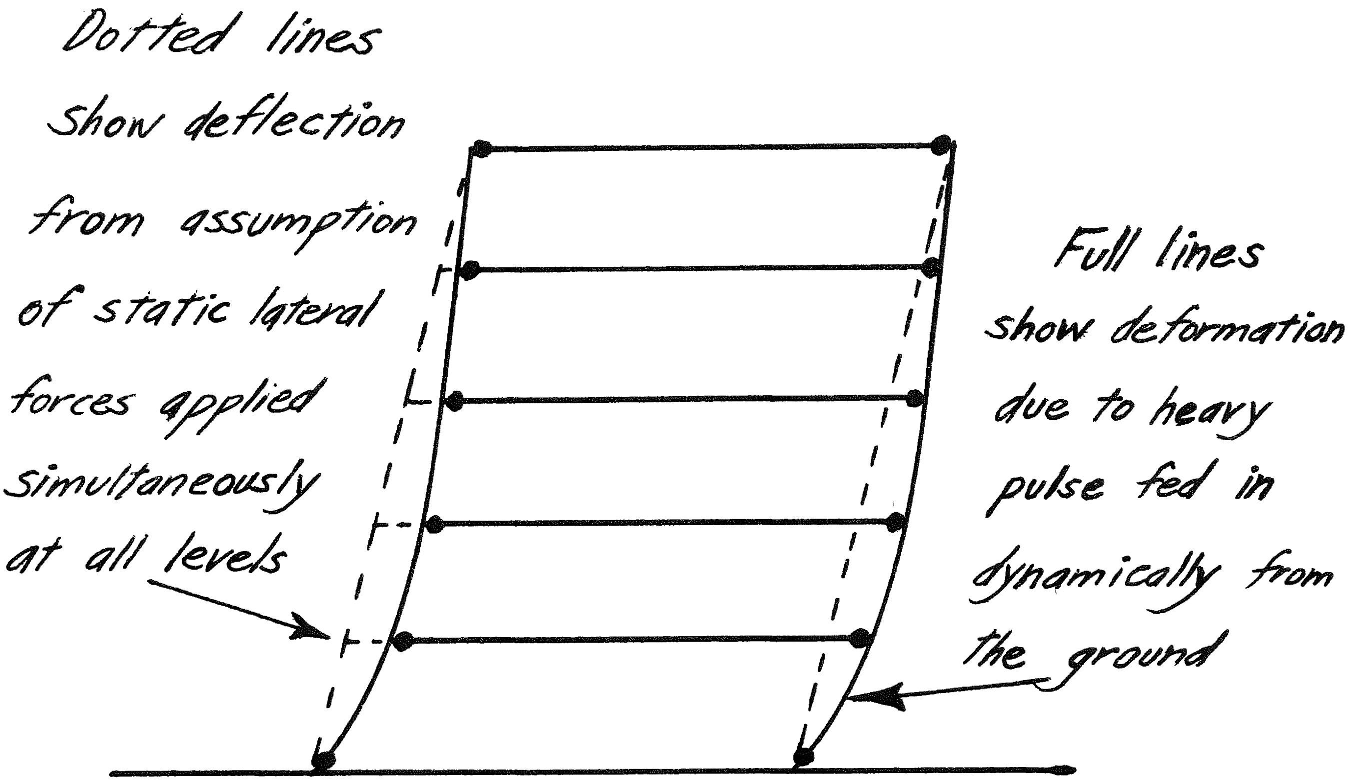

Dotted lines
show deflection
from assumption
of static lateral
forces applied
simultaneously
at all levels
Full lines
show deformation
due to heavy
pulse fed in
dynamically from
the ground

Fig 2

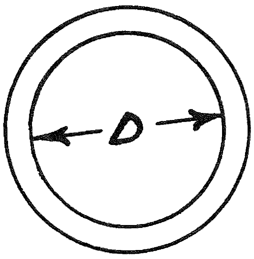
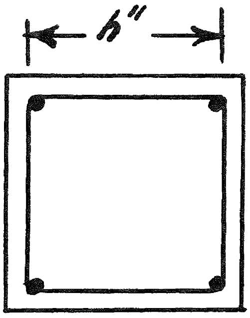


Fig 3(a)

Fig 3(b)



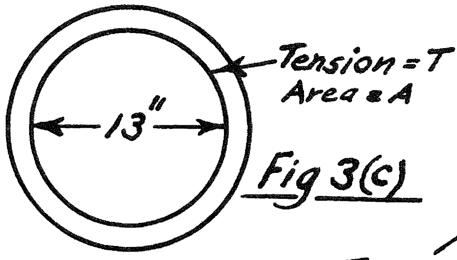


Fig 3(c)

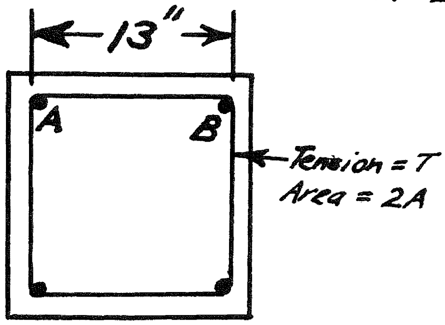


Fig 3(d)

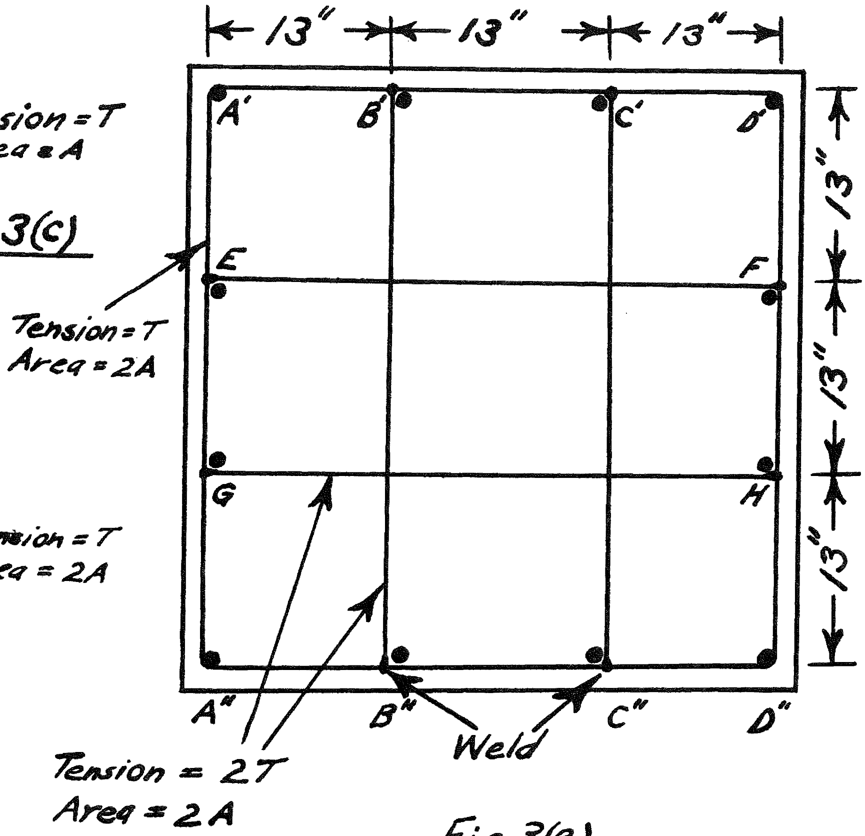


Fig 3(e)

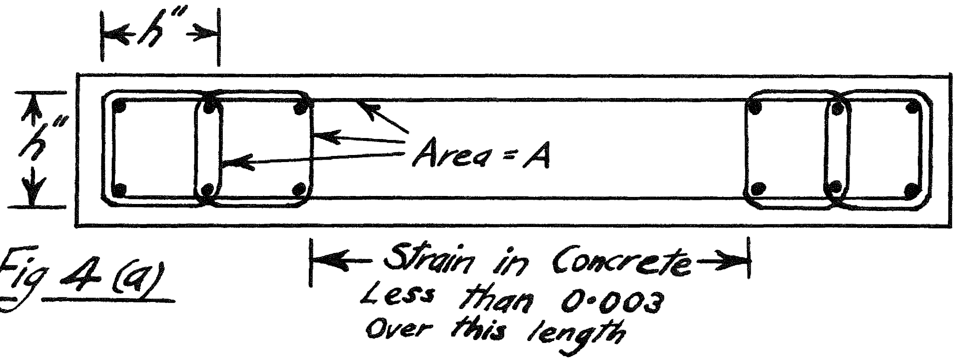


Fig 4 (a)

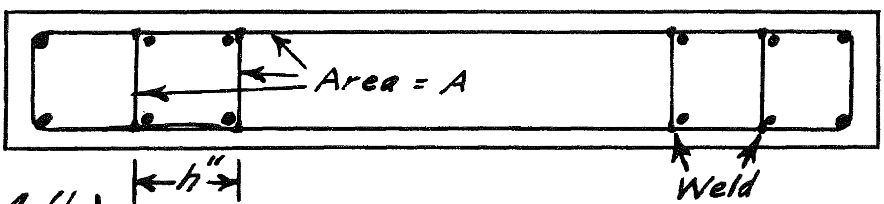


Fig 4 (b)

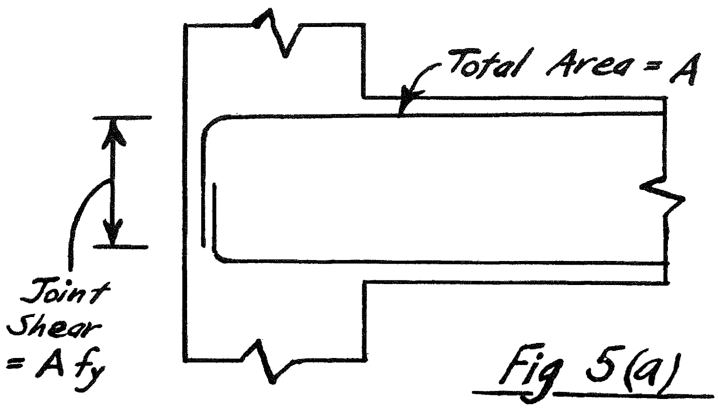

Total Area = A
Joint
Shear
= A fy

Fig 5(a)

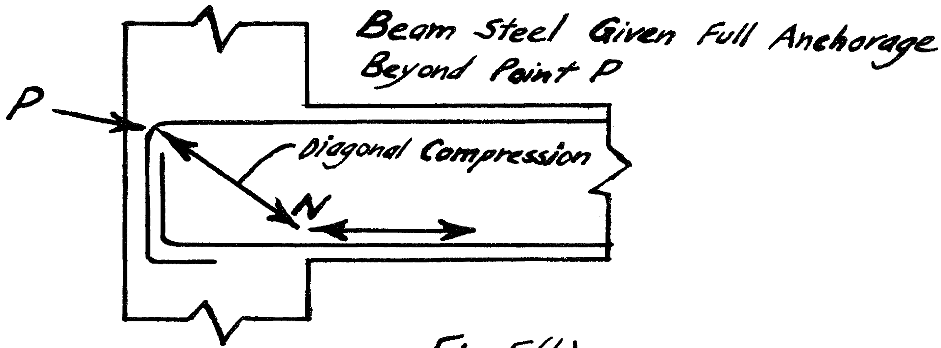

Beam Steel Given Full Anchorage
Beyond Point P
P
Diagonal Compression
N

Fig 5(b)