DIAGONAL BEAM REINFORCING FOR DUCTILE FRAMES

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ABSTRACT

This paper describes a system of diagonal beam reinforcing for reinforced concrete ductile frame buildings in seismic areas.

The development and application of the system is described with reference to the design of an 18 storey building.

The proposed reinforcing system has a number of major structural advantages over conventional reinforcing. Moments and shears within the beam span can be resisted entirely by the main reinforcing. Plastic hinge lengths are substantially increased and the hinging is kept away from the column face allowing concrete strut action to be effective in resisting forces within the elastic beam column joint.

1. INTRODUCTION

The principal objective in seismic design of reinforced concrete structures is to provide a system that allows significant amounts of energy to be dissipated in a controlled ductile mechanism. For frame structures the most desirable mechanism is a "strong column - weak beam" system with ductile flexural yielding occurring in the beams. A capacity design procedure can ensure that this mechanism is maintained throughout a major earthquake with reasonable protection against column yielding mechanisms or brittle failures.

In a conventionally reinforced frame structure under seismic loading, plastic hinges generally occur at the ends of the beams where moments are highest. Ductility requirements result in wide cracking, large steel strains and penetration of yield into the beam column joints particularly after a number of load reversals. Possible undesirable consequences of this behaviour are sliding shear failure in the beam, loss of bond of flexural steel in the joint, and a shear failure in the joint itself.

Various proposals have been made for reinforcing layouts that will reduce this undesirable behaviour. Sliding shear failures can be controlled by diagonal reinforcing in the plastic hinge zone[1]. Joint behaviour can be improved by diagonal reinforcing in the joint itself[2], or be relocating the beam hinge away from the joint so that it remains elastic[3].

This paper describes a system of beam reinforcing which has considerable advantages over both conventional reinforcing and previous proposals for improved behaviour. In the proposed system the main beam reinforcing crosses diagonally at mid-span. Secondary reinforcing ensures that hinging occurs only in the main diagonal reinforcing.

Major advantages are that plastic hinges are remote from the column face, plastic hinge lengths are long, the possibility of a sliding shear failure in the beams is eliminated and the beam-column joint always remains elastic. Some changes are required in detailing and placing of reinforcing, but no serious problems are introduced and a reduction in reinforcing costs is anticipated. The system can be used in all framed structures where gravity moments are small.

This diagonal reinforcing system has been developed for the design of a proposed eighteen storey building. This paper describes the reinforcing system in some detail, then outlines the design procedure for the building in which it will be used.

2. DIAGONAL REINFORCING SYSTEM

2.1 Description

The diagonal reinforcing system described in this paper was suggested by Professor Paulay of the University of Canterbury during discussion on design strategy for an 18 storey building that will be described in Section 3.

The beam is illustrated in diagrammatic form in figure 1. The main beam reinforcing crosses diagonally at mid-span between beam stuvs which are heavily reinforced to prevent beam hinging at the column face.

2.2 Beam Hinging

Figure 2 compares the beams flexural capacity with an applied bending moment diagram resulting from lateral loading. Gravity load moments are not shown, because they are relatively small in this building. Minor gravity load moments have little effect on this discussion.

The geometry and reinforcing steel areas are such that, under increasing lateral load, yielding occurs first at the end of the stuv at point A on figures 1 and 2. The moment capacity at the column face, point B, is sufficient to prevent yielding at that point. As the beam moments increase and strain hardening occurs, yielding

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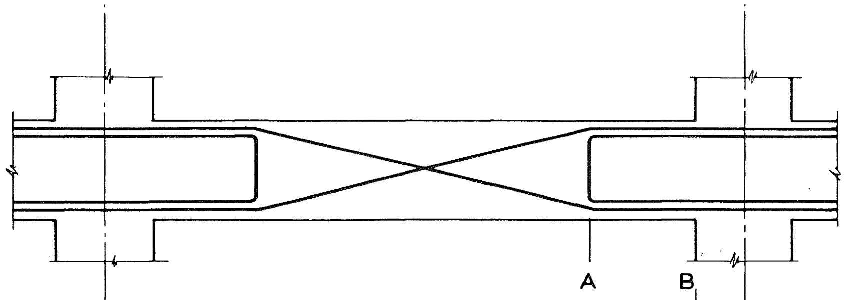


Fig. 1 Elevation of Beam Reinforcing

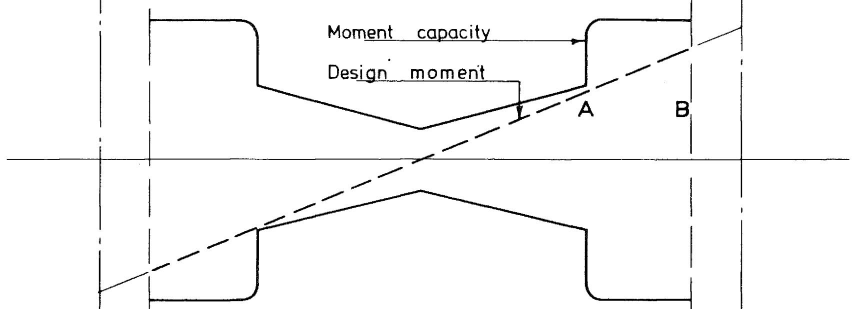


Fig. 2 Bending Moment Diagram

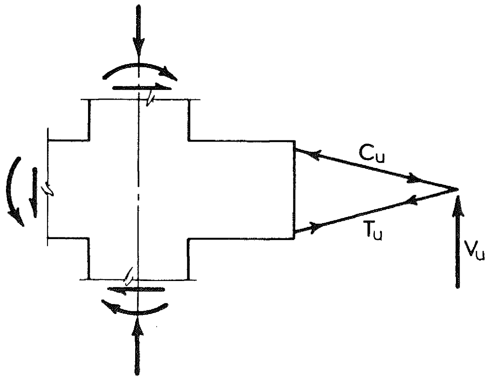


Fig. 3 Truss action in diagonal reinforcing

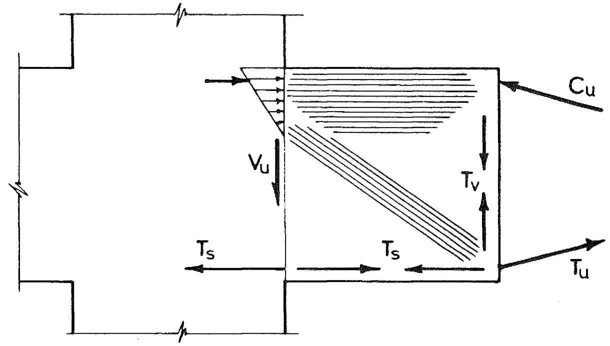


Fig. 4 External actions and internal forces on beam stub

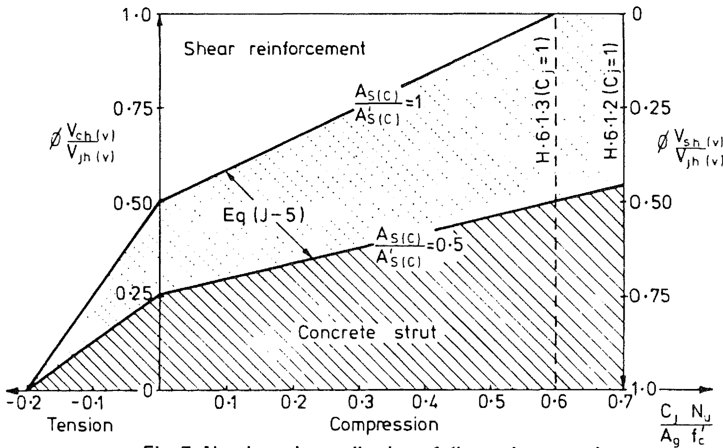


Fig. 5 Nominated contribution of diagonal strut actions to vertical and horizontal joint shear resistance

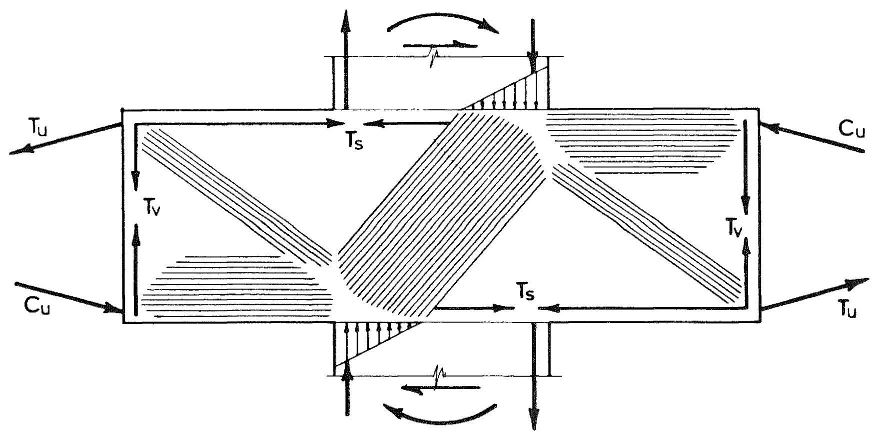


Fig. 6 Concrete strut action in beam stub and beam-column joint

progresses towards the centre of the beam.

The concrete and secondary reinforcing in the central section of the beam are required only to prevent buckling of compression bars. After cracking, the diagonal reinforcing creates a simple truss which can resist all applied forces as shown in figure 3.

2.3 Ductility

The need for a structure to possess a high degree of displacement ductility is well established. For any predictable yielding mechanism it is possible to calculate the amount of curvature ductility required in the plastic hinge for the building as a whole to achieve a given displacement ductility.(4)

A disadvantage of forcing beam hinging away from the columns is that the curvature ductility demand in those hinges will increase. The proposed diagonal beam reinforcing system however allows yielding to occur over the whole length of the diagonal reinforcing, hence steel strains required to achieve a given displacement ductility are much less than in a conventional reinforcing system. The reduced steel strains will ensure better performance of the reinforcing, and associated cracking will be better distributed with smaller crack widths.

2.4 Beam Stub

The forces in the diagonal reinforcing are transferred to the beam-column joint through the beam stub. The forces acting on and within the stub are illustrated in figure 4. A steel tie is provided at the end of the stub to resist the vertical component of the force in the diagonal compression strut and hence prevent buckling of the compression bars. The steel areas provided in the stub are such that they are never expected to yield, hence the integrity of the stuv is assured even under cyclic loading. Compression and shear forces within the elastic stub can be resisted by concrete strut action.

2.5 Beam Geometry

Beam geometry has several influences. The advantages of a longer beam stub are that anchorage is improved, the joint is given better protection and the diagonal reinforcing is at a more efficient angle. These advantages must be balanced against the greater ductility demand of the resulting short hinge, resulting in high steel strains and severe cracking at lower levels of load.

In practice it has been found suitable to provide a steel area at the column face twice that in the diagonal reinforcing. The length of the stub can be calculated from the required overstrength capacity and the length and depth of the beam.

For preliminary design purposes, assuming that all forces are resisted by reinforcing steel, it can be shown that

$$\frac{S}{L} = 0.5 - \frac{k \cos \theta}{2n}$$

where S = the length of the stub measured from the column face.
L = the clear length of the beam.
k = the desired factor of safety against yielding at the column face.
n = the ratio of the steel area at the column face to the steel area of the diagonal reinforcing.
θ = the angle to the horizontal of the diagonal reinforcing.

Typical values for k (1.25) and n (2.0) give S = 0.19 L, when $\theta = 12^{o}$.

For final design calculations a more accurate relationship can be derived from first principles considering actual beam geometry and the contribution of concrete to flexural strength at the column face.

2.6 Beam-Column Joint

Traditionally little attention has been given to the analysis and design of beam-column joints. Recent test evidence has indicated poor performance under post-elastic cyclic loading, particularly bond failure of the beam bars due to successive yeild penetrations into the joint, and deterioration of the joint concrete to the extent that it cannot contribute to the transfer of shear across the joint. Conventional reinforcing to prevent this behaviour is both expensive and difficult to place.

The proposed diagonal reinforcing system solves the problem by removing it. The plastic hinge regions in the beams are relocated away from the column faces so that the joint remains elastic under all loading conditions. Yield penetration into the joint is prevented, bond stresses are reduced, and the joint concrete can make a substantial contribution to shear transfer.

Recent proposals (3, 5) indicate that if the beam-column joint remains elastic, 50% or more of the horizontal shear resistance can be provided by diagonal strut action, varying with level of axial load. A suggested relationship for design purposes is shown in figure 5. Savings in reinforcing steel within the joint can be considerable. Concrete strut actions within the beam stubs and the elastic beam column joint are illustrated in figure 6.

2.7 Experimental Results

A number of experimental studies have produced evidence supporting the expected behaviour of the proposed diagonal reinforcing system.

These include tests of diagonally reinforced coupling beams, a diagonally reinforced beam-wall junction, a diagonally reinforced beam and stub, and elastic beam column joints.

2.7.1 Diagonally reinforced coupling beam tests

Diagonal bars are commonly used for reinforcing beams between coupled shear walls. As in the proposed system, reinforcing steel is able to resist all applied forces without any assistance required from concrete or secondary reinforcing. The major differences between the two systems are the angles of the inclined bars and the anchorage conditions.

Diagonally reinforced coupling beams have been tested extensively under post-elastic cyclic loading with excellent results[4]. Figure 7 illustrates geometry and behaviour of a typical coupling beam. Figure 8 compares the ductilities of a number of such beams with conventionally reinforced beams of similar geometry.

2.7.2 Diagonally reinforced beam wall junction test

The advantages of inclined reinforcing in plastic hinge zones have been demonstrated in a series of tests of a beam-wall junction by Paulay and Spurr[6]. In this series of tests it was found that inclined reinforcing greatly improved the cyclic post-elastic behaviour of the plastic hinge as illustrated in figure 9. The reinforcing angle and beam geometry of these tests are more similar to the proposed system than the coupling beams described above.

As a result of these tests, Paulay and Spurr conclude that the normal undesirable features of degrading stiffness, loss of energy dissipating capacity and eventual sliding shear failure in the plastic hinge zone can be avoided by the use of diagonal reinforcing, and further, that concrete damage such as wide full depth cracks affecting the response of the beam can be avoided.

2.7.3 Diagonally reinforced beam and stub test

Following the development and design of this reinforcing system for a proposed eighteen storey building described in Section 3, a testing programme was carried out at the School of Engineering, University of Canterbury, on a half beam and stub with similar geometry and reinforcing as shown in figure 10. The scale was about half full size. The unit was subjected to reversed cycle loading with increasing displacement ductility. The load-deflection relationship is shown in figure 11. Test results are reported in detail in reference 7.

The behaviour of this unit was very good with one exception. The special beam stirrups at the change in direction of the main reinforcing, were not strong enough to remain elastic. Some yielding occurred causing local crushing of the concrete in following cycles. This is illustrated diagrammatically in figure 12. If these stirrups had been strong enough to remain elastic, the gysteresis loops shown in figure 11 would have been even fuller than shown. This yielding appears to have been caused by stirrup misfit or by a larger force than anticipated in the compression strut. Despite this local problem the gysteresis loops were stable and the unit showed no degredation in strength throughout the test.

In all other respects the unit behaved very well. There was no appreciable yielding of main steel at the comumn face. Steel strains and curvature were well distributed towards the centre of the beam as shown in figure 13. At the same displacement ductilities, curvatures were considerably less than in similar units with more conventional reinforcing.

In the stub there was some spreading of yield and loss of bond towards the column face accentuated by the unpredicted stirrup yielding. Bond stresses are very high in this region because the stress on the bars has to decrease from an overstrength value at the plastic hinge to something less than yield at the column face, while the applied moment increases over this length. The beam-column joint was undamaged at the end of the test.

This test has confirmed the excellent behaviour predicted for a diagonal reinforcing system, and has highlighted specific areas which will require detailed design consideration.

2.7.4 Elastic beam-column joint tests

A recent series of tests at the University of Canterbury[8] has confirmed that within a beam-column joint which remains elastic, concrete strut action can be relied upon to resist a substantial percentage of joint shear. These tests support an increased reliance on concrete strut action allowing a reduction in joint reinforcing when plastic hinges are relocated away from the column face.

3. BUILDING

3.1 Description

This section describes the proposed eighteen storey building for which the diagonal reinforcing system was developed. The building is to be a retail and office building at 174-180 Lambton Quay, Wellington, for the Challenge Corporation Limited. The ground and first floors are retail shopping, the third floor is car-parking, with office space above. The sloping site allows access from Lambton Quay at ground floor and from The Terrace at the third and fourth floors. The exterior of the building is clad with precast concrete panels. A perspective elevation and typical floor plan are shown in figures 14 and 15.

3.2 Structural System

After consideration of a number of alternatives, a metal deck floor system with cast-in-place topping was selected. It spans between precast interior beams and cast-in-place exterior beams.

Lateral loads are resisted by a perimeter frame. A frame system was selected in preference to shear walls because of planning considerations on the lower floors where carparking and retail shopping precluded the use of a shear wall system. The main lift shaft, with walls of lightweight fire-rated construction, extends from the top floor down to second floor level. A perimeter frame was selected so that the interstorey height could be kept to a minimum with deep exterior beams and shallow interior beams. An advantage of the perimeter frame is that gravity moments are small compared with earthquake moments.

The building has been designed as a free-standing tower from the Lambton Quay level, the podium to The Terrace being separated with a seismic gap. The exterior precast concrete units will act as permanent formwork for the perimeter frames. Solid precast panels adjacent to the service core will be supported and separated to allow frame deformations under seismic loading. The foundations consist of drilled belled piles under all columns tied together at ground level with a deep foundation beam.

3.3 Geometry

The plan dimensions of the tower are 35.0 m x 25.0 m. Typical floor to floor height is 3.15 m.

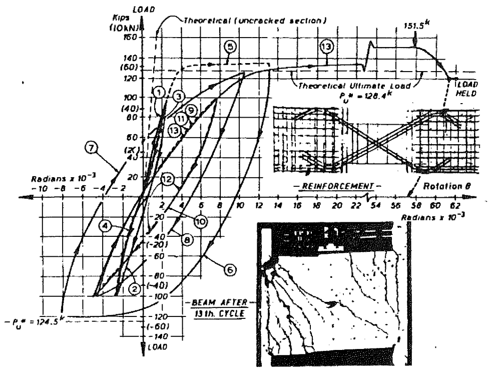


Fig. 7 Load-rotation relationship fcr a diagonally reinforced coupling beam.

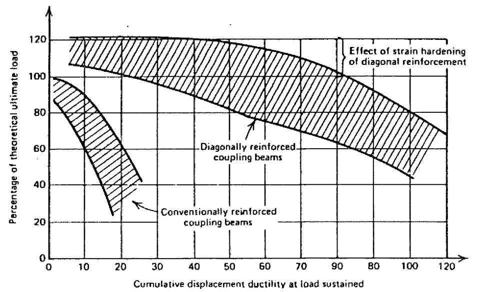
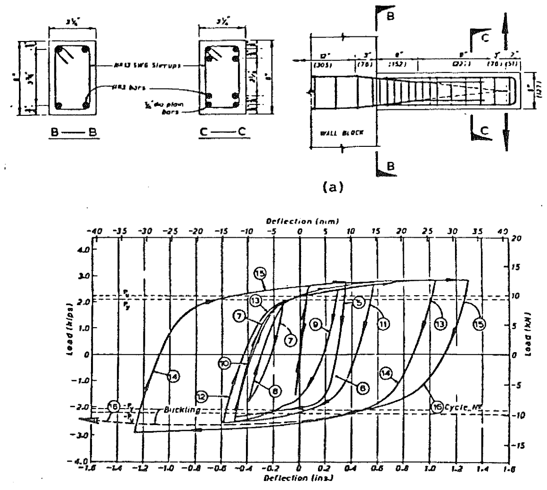


Fig. 8 Cumulative ductilities imposed on conventionally and diagonally rein-forced coupling beams.



(a)

(b)

Fig. 9 Details of a Diagonally Reinforced Beam-Wall Junction Specimen.
(a) Dimensions and Reinforcement.
(b) Load-Deflection Relationship.

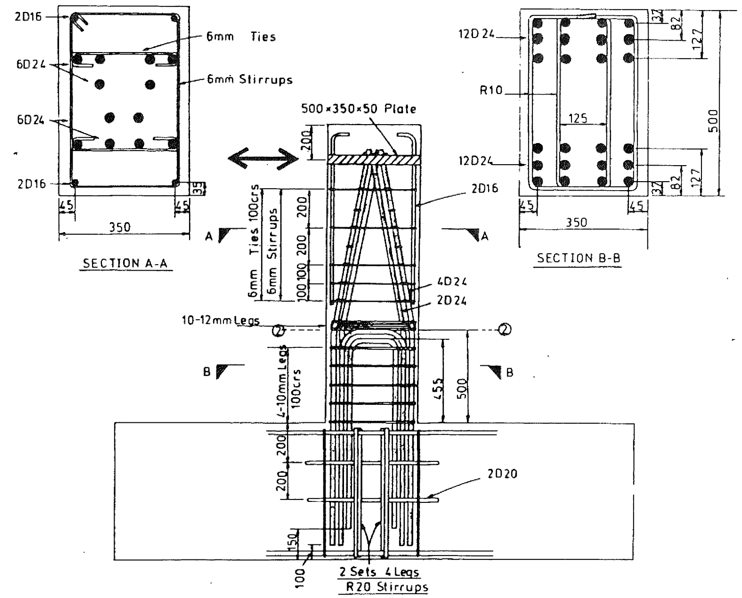


Fig. 10 Reinforcing in test unit.

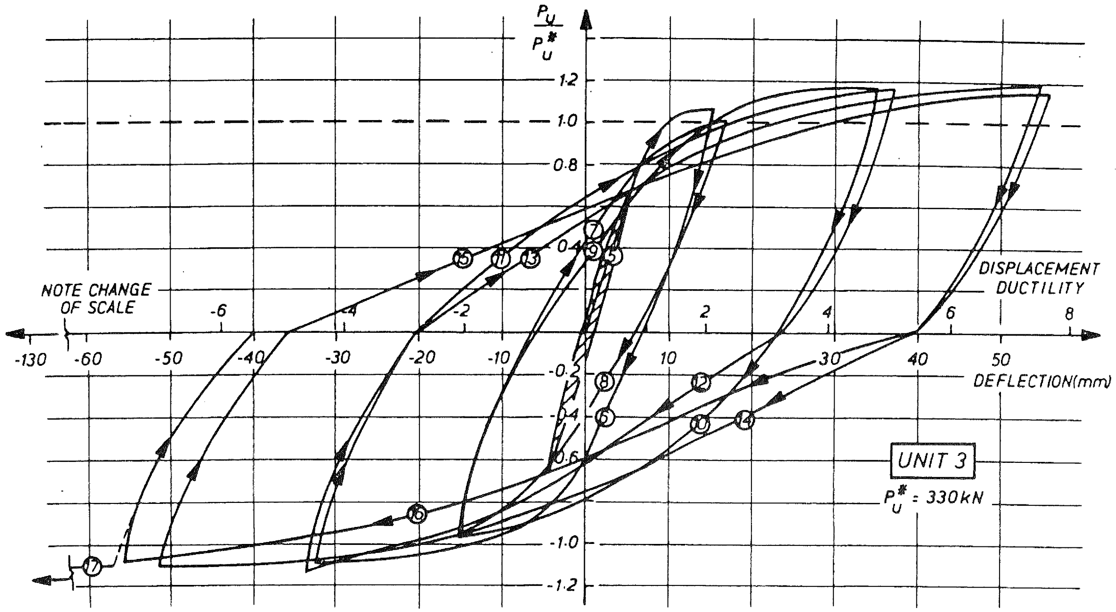


Fig. 11 Load-deflection relationship.

As the perimeter frame resists all lateral loads the members are fairly massive. At the base of the building the corner columns are 1200 mm square, other columns 1200 x 800 mm and 900 x 800 mm. The beams are 900 mm square. These sizes decrease in two steps up the building.

The beams of the perimeter frame have midspan diagonal reinforcing at all levels from the first floor level up to the 14th floor level. Beams of the top four levels are conventionally reinforced because gravity load moments become significant at those loads.

4. DESIGN

4.1 Design Philosophy

The perimeter frame has been designed using a capacity design procedure to resist the loads specified in NZS 4203 : 1976. The basic concept is a beam hinging mechanism.

The diagonal beam reinforcing forming the "fuse" of the energy dissipating mechanism has been designed to yield at the code level of load. An elastic computer analysis was used to calculate member actions under code loading. The beam stubs, beam-column joints and columns were all subsequently designed using a capacity design approach to ensure that the chosen mechanism is maintained throughout whatever deformations may be imposed in the event of a severe earthquake.

4.2 Discussion Group

Specific aspects of the design have generally followed the recommendations of the New Zealand National Society for Earthquake Engineering's Discussion Group on Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames[9]. This Discussion Group was in progress during development of the major design concepts early in 1977, so consultation was possible with members of the group, particularly Russell Poole of this office and Professor Paulay at the University of Canterbury.

4.3 Design Procedure

A brief summary of the design procedure is tabulated below:

Basic design information:

1. Size members from preliminary calculations.
2. Establish basic reinforcing geometry.
3. Calculate loads.
4. Computer analysis.

Design beams at each level:

1. Select critical output combination for beam design.
2. Redistribute beam moments.
3. Calculate beam shears at code level of load.
4. Design diagonal reinforcing to resist applied shears.
5. Calculate overstrength capacity of diagonally reinforced beam.
6. Design secondary reinforcing in beam and stub.
7. Design reinforcing in beam-column joint.

Design columns and foundations:

1. Calculate column actions from beam overstrengths.
2. Design columns for flexure, axial load, and shear.
3. Design foundations.

4.4 Basic Design Information

4.4.1 Member sizes

Basic geometry was established in consultation with the client; Challenge Corporation Limited, the Architects Warren and Mahoney, and contractors Civil and Civic (N.Z.) Limited. Frame member sizes were established at this stage following preliminary manual calculations.

4.4.2 Reinforcing geometry

Reinforcing geometry was defined in terms of all the design considerations described in Section 2. In particular the relationship between stub length, inclination of reinforcing and factor of safety against yielding at the column face received close attention to ensure that the structure behaves as intended.

4.4.3 Loads

Loads were calculated according to NZS 4203 : 1976. The base shear coefficient for equivalent static loads was :

Cd = CISMR	where	C = 0.075	(High period, Zone A)
= 0.066		I = 1.0	(Class III)
		M = 1.0	(Reinforced concrete)
		R = 1.1	(More than 1000 people)
		S = 0.8	(Ductile frame)

The spectral modal analysis was on the basis of the spectrum specified in figure 3 of NZS 4203 : 1976, for rigid and intermediate subsoils in Zone A.

4.4.4 Computer analysis

The perimeter frame was analysed as four interconnected one-way frames using a general computer programme for linear elastic analysis of three-dimensional building structures (TABS). Both an equivalent static force analysis and a spectral modal analysis were carried out in accordance with NZS 4203 : 1976. The structure was assumed to be symmetrical about both major axes which simplified the snalysis considerably. The torsion provisions of NZS 4203 : 1976 were followed.

The fundamental periods of vibration in the two principal directions were found to be 1.31 seconds and 1.07 seconds. Beam flexibility was such that "tube" action was found to be negligible. Calculated interstorey deflections at the code level of load were up to 5 mm, with total deflection at the top of the building of 50 mm.

The computer output provided elastic bending moments, shears and axial forces for all members. Gravity actions (dead and live) and seismic actions (static and dynamic) were all calculated separately then combined before selecting critical combinations for beam design. The critical lateral load case for most members was that resulting from the spectral modal analysis.

4.5 Beam Design at Each Level

4.5.1 Beam shears

At each level, bending moments produced by lateral loading were redistributed to allow the use of the same reinforcing layout in each bay. The beam shear associated with this bending moment was used as the design shear for each beam.

4.5.2 Diagonal reinforcing

The diagonal reinforcing in each beam was designed such that the vertical components of the tension and compression forces at yield equalled the design shear.

4.5.3 Gravity moments

Gravity moments are small compared with earthquake moments, because the perimeter frame carries little gravity load, but all of the lateral loads. For example, at the column face the worst combination of gravity moments was only 8% and 15% of the earthquake moment, at floors 2 and 15 respectively.

Gravity moments were not considered in the design of diagonal reinforcing because the critical beam section at the stub end is very close to the point of zero gravity moment. A small amount of additional reinforcing was provided in the bottom at midspan and at the top through the column to resist gravity moments. This additional reinforcing will not affect behaviour under seismic loading. The nominal stirrups in the beam span are sufficient to resist shears resulting from gravity loads.

4.5.4 Overstrength capacity

The overstrength capacity of the diagonal reinforcing was calculated on the basis of the actual steel area and expected yield strength, with allowance for strain hardening and a ϕ factor of unity. This overstrength capacity was used in all subsequent calculations for the rest of the frame.

4.5.5 Secondary reinforcing

Secondary reinforcing has been provided within the central section of the beam to prevent buckling of the individual compression bars and the whole compression strut, and to prevent concrete from falling out of the body of the beam during a severe earthquake.

Beam stirrups have been provided at the end of the stub to resist the vertical component of the force in the compressive strut with a conservative safety factor to prevent the behaviour observed in testing(7).

Beam stirrups within the stub were calculated to resist shears in excess of a modest allowance for concrete strut action. The stub length and reinforcing geometry had been selected to ensure that no yielding occurred in the main reinforcing at the column face, as described previously.

4.5.6 Beam-column joint

Shears in the beam-column joint are expected to be resisted mainly by concrete strut action. The concrete contribution was calculated in accordance with the recommendations of the Discussion Group(5). Column links have been provided only to confine the column through the joint. The rest of the reinforcing required to resist horizontal shears has been provided in the form of straight bars anchored in the elastic stubs. Some of these bars were extended into the diagonally reinforced zone to control cracking at the end of the stub and to resist gravity load moments. Vertical shears are resisted mainly be strut action with some contribution from intermediate column bars.

4.6 Design of Columns and Foundations

4.6.1 Column actions

The column actions were calculated on Paulays recommendations(10) which allow the difficult combination of capacity design, higher mode effects and concurrency to be considered relatively simply. The perimeter frame has the advantage that concurrent earthquake effects need be considered at only the corner columns.

4.6.2 Column design

The columns were designed for combined flexure and axial load using standard interaction charts. An advantage of the redistribution described in 4.5.1 above was that for all except corner columns, the overstrength beam shears on each side of the column cancelled out, and the only axial forces were from gravity loads. Tension was critical in the corner columns at the base, so standard charts had to be extended into the tension region for these columns.

Transverse reinforcing in the columns was designed on the basis of the Discussion Group's recommendations(11). The design shear at the base of the columns was 2.4 x the elastic shear under code loading.

4.6.3 Foundations

The foundations were designed to resist column base forces with a safety factor to ensure that hinges occur in the columns and not in the foundation system. Deep belled piles were required under the corner columns to resist axial tension during seismic loading.

5. REINFORCING

5.1 Splice Locations

Although the reinforcing system described in this paper is simple in concept, it raises some problems with layout and placing of reinforcing. The two inter-related problems are that top bars at one end of a beam become bottom bars at the other, and that splices cannot be located at mid-span.

A large number of alternative reinforcing layouts were considered. Some are shown diagrammatically in figure 15.

It is not possible to lap reinforcement at mid-span as in a conventional system because the diagonal reinforcing is expected to yield throughout its length under seismic loading. The joint and the stubs are the only possible locations for laps. Conventional lapping in this area will be difficult because of congestion but a solution is to allow

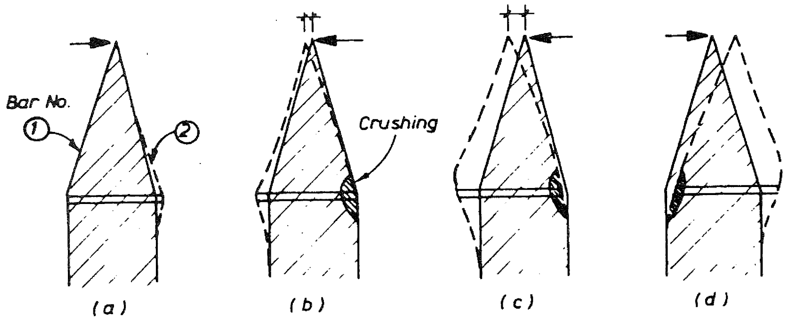


Fig. 12 Behaviour of unit under cyclic loading (7)

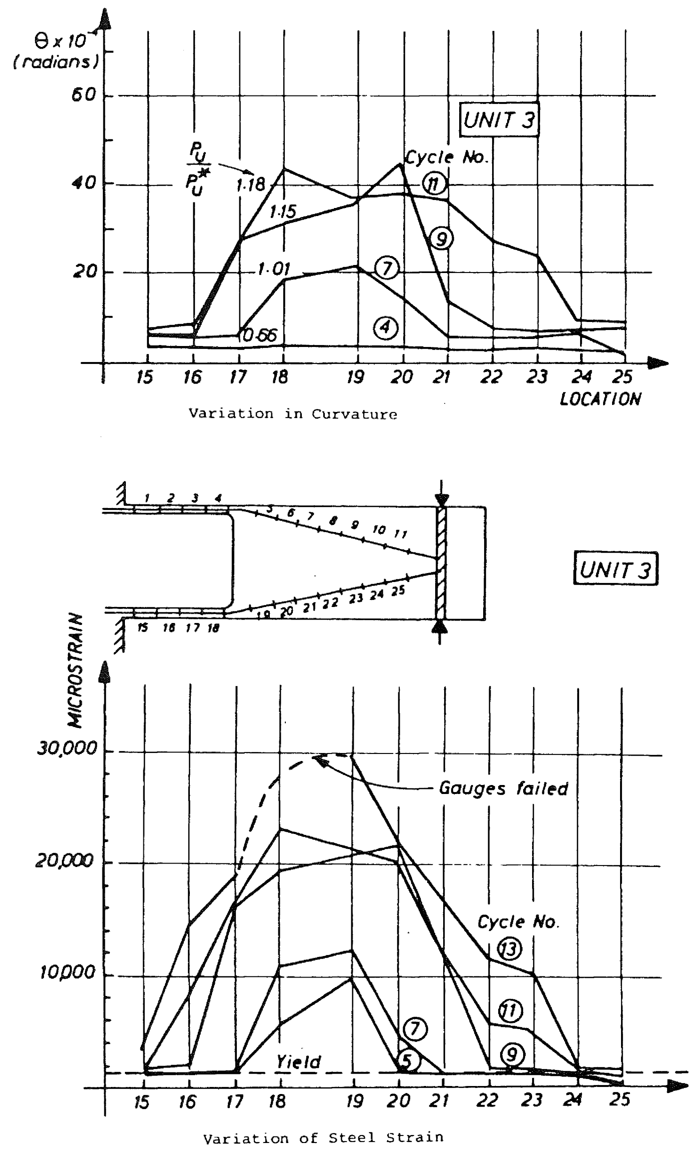


Fig. 13 Variation in curvature and steel strain along the beam (7)

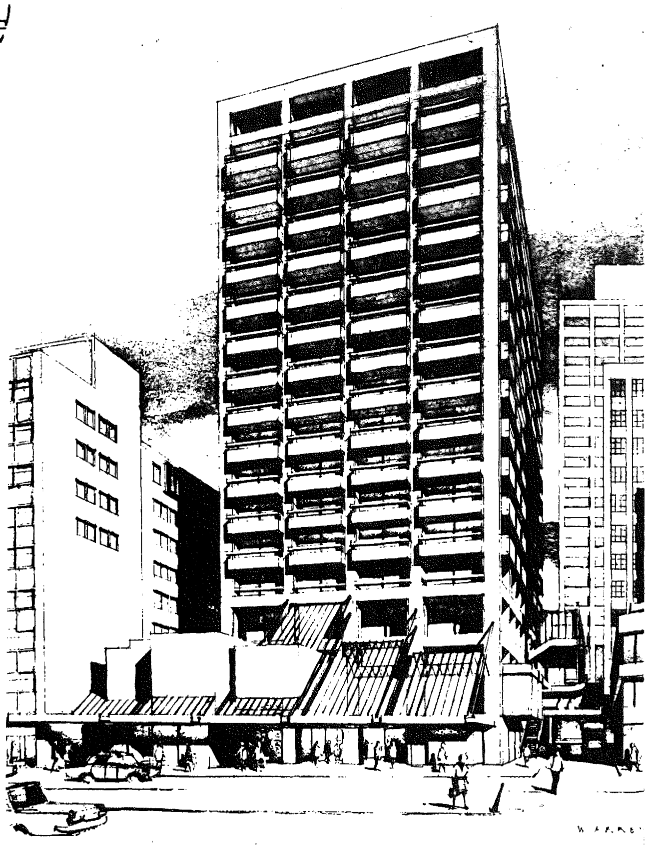


Fig. 14 Elevation of proposed building

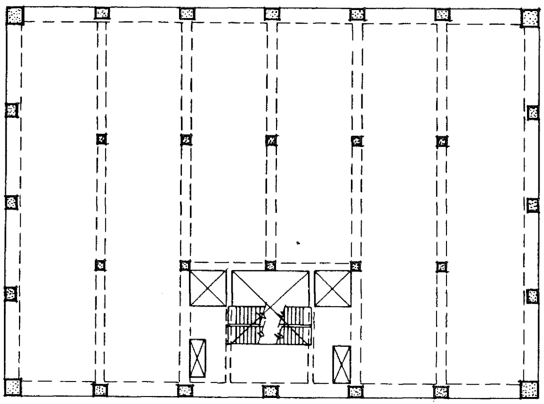


Fig. 15 Typical Floor Plan

diagonal steel on one side of the joint to become estra flexural steel on the other.

A possible arrangement using this principle is shown in figure 16(a). This is particularly suitable if the steel area at the column face is double the steel area provided at mid-span. Details such as shown in figure 16(b) and figure 16(c) are unsuitable because of the discontinuity in the centre of the joint. Mechanical splices could be used as shown in figure 16(d) but this was found to create additional construction problems. A system that requires alternating layers to be placed in opposite directions as shown in figure 16(e) will create construction difficulties.

5.2 Reinforcing Detailing

The reinforcing system selected was that shown in figure 16(a) which was the most suitable for prefabrication of reinforcing cages. It was considered essential that all the beam cages be prefabricated, preferably off the site, because of the complexity and location of the beams. Large scale details have been drawn to facilitate both fabrication and erection of reinforcing cages. The beams have been detailed with two types of prefabricated cages alternated bay by bay along the beam. Both inner and outer layers of reinforcing in a typical bay are illustrated in figure 17.

This reinforcing layout will be relatively easy to place, but it is bulky. When the beams were first sized, a wide beam width was selected such that shear stresses would be low enough to prevent a sliding shear failure using a conventional reinforcing system. When the diagonal reinforcing system was developed it was hoped to be able to reduce the beam width, but this was not possible because of the widths of the prefabricated reinforcing cages.

5.3 Joints

A major difficulty with the design and construction of conventional ductile moment resisting frames is providing and placing joint reinforcing. It is often almost impossible and very expensive to fit in all the reinforcing required by calculation. Even if all the reinforcing can be placed, it can prove impossible to get good concrete into the joint because of the congestion of the reinforcing.

In this design it was possible to reinforce the beam-column joint simple with several straight bars anchored in the beam stubs, with links only to confine the column concrete. A modest saving in steel will be amplified by reduced labour costs in placing both steel and concrete.

5.4 Costs

The proposed system of diagonal reinforcing was selected for this building after preliminary design and pricing had been carried out on the basis of conventional reinforcing. The decision to use the diagonal system followed a detailed study of material and labour costs and the construction programme. In comparison with a system of conventional reinforcing, this system has a similar quantity of main reinforcing but less secondary reinforcing. Cost savings occur because the secondary reinforcing deleted is expensive and difficult to place, and because the prefabricated reinforcing cages will speed construction.

6. CONCLUSIONS

A system of diagonal reinforcing has been proposed for the beams of multistorey reinforced concrete ductile frame buildings for seismic areas.

This paper has described the details and expected behaviour of the system which has major advantages over conventional reinforcing. The design of the reinforcing has been described with reference to a proposed eighteen storey building incorporating the diagonal reinforcing system.

This paper will provide useful information for designers considering the use of diagonal beam reinforcing in multi-storey buildings.

ACKNOWLEDGEMENTS

The reinforcing system described in this paper was developed at the suggestion of Professor T. Paulay of the University of Canterbury. Comments and design assistance from Professor Paulay, R.A. Poole and B.J. Wood are gratefully acknowledged.

REFERENCES

1. T. Paulay. "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames - Shear Strength Requirements". Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 10, No. 2, June 1977.
2. R.A. Poole. "Design of a 28 Storey Reinforced Concrete Building in Wellington". Paper presented at South Pacific Regional Conference on Earthquake Engineering, May 1976.
3. T. Paulay, R. Park, M.J.N. Priestley. "Reinforced Concrete Beam Column Joints Under Seismic Actions". ACI Journal, Vol. 75, No. 11, November 1978.
4. R. Park and T. Paulay. Reinforced Concrete Structures John Wiley & Sons, 1975.
5. R.W.G. Blakeley. "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames - Design of Beam Column Joints". Bulletin of the New Zealand National Society for Earthquake Engineering. Vo. 10, No. 4, December 1977.
6. T. Paulay and D.D. Spurr. "Frame-Shear Wall Assemblies Subjected to Simulated Seismic Loading". Proceedings of Sixth World Conference on Earthquake Engineering, New Delhi, January 1977.
7. I.N. Bull. "The Shear Strength of Relocated Plastic Hinges". Department of Civil Engineering Research Report No. 78/11. University of Canterbury. February 1978.
8. G.R. Birss. "The Elastic Behaviour of Earthquake Resistant Reinforced Concrete Interior Beam-Column Joints". Department of Civil Engineering Research Report No. 78-13. University of Canterbury. February 1978.
9. N.Z.N.S.E.E. Discussion Group. "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames". Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 10, No. 2, Vol. 10, No. 4 and Vol. 11 No. 2.
10. T. Paulay. "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames - Columns : Evaluation of Actions". Bulletin

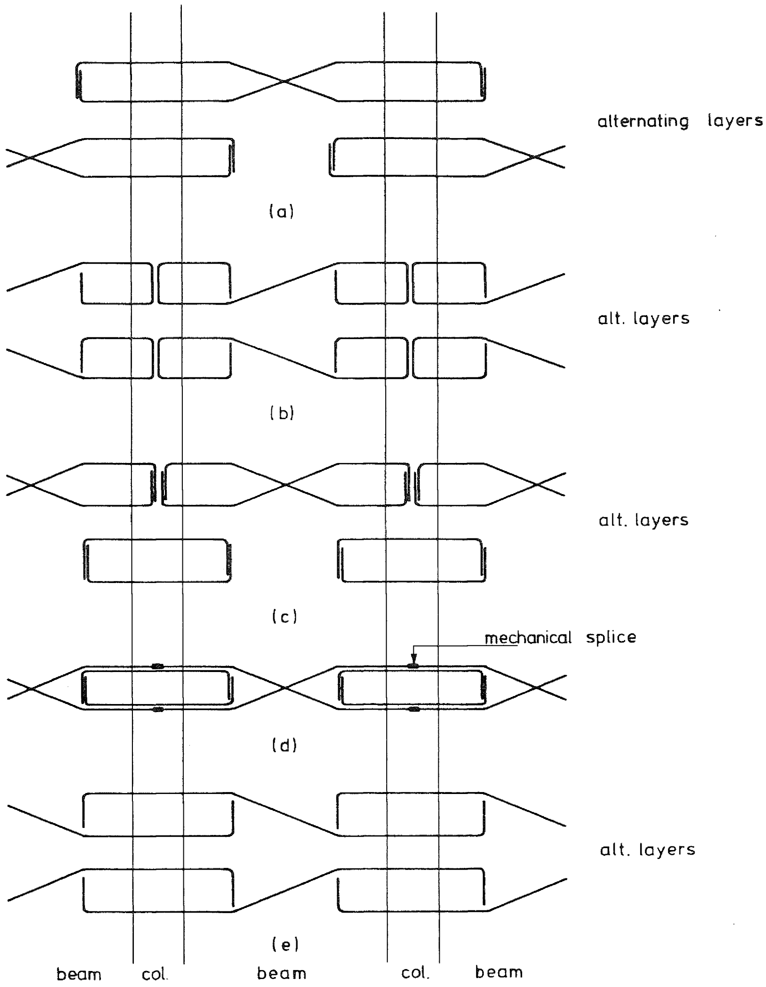


Fig. 16 Possible reinforcing layouts

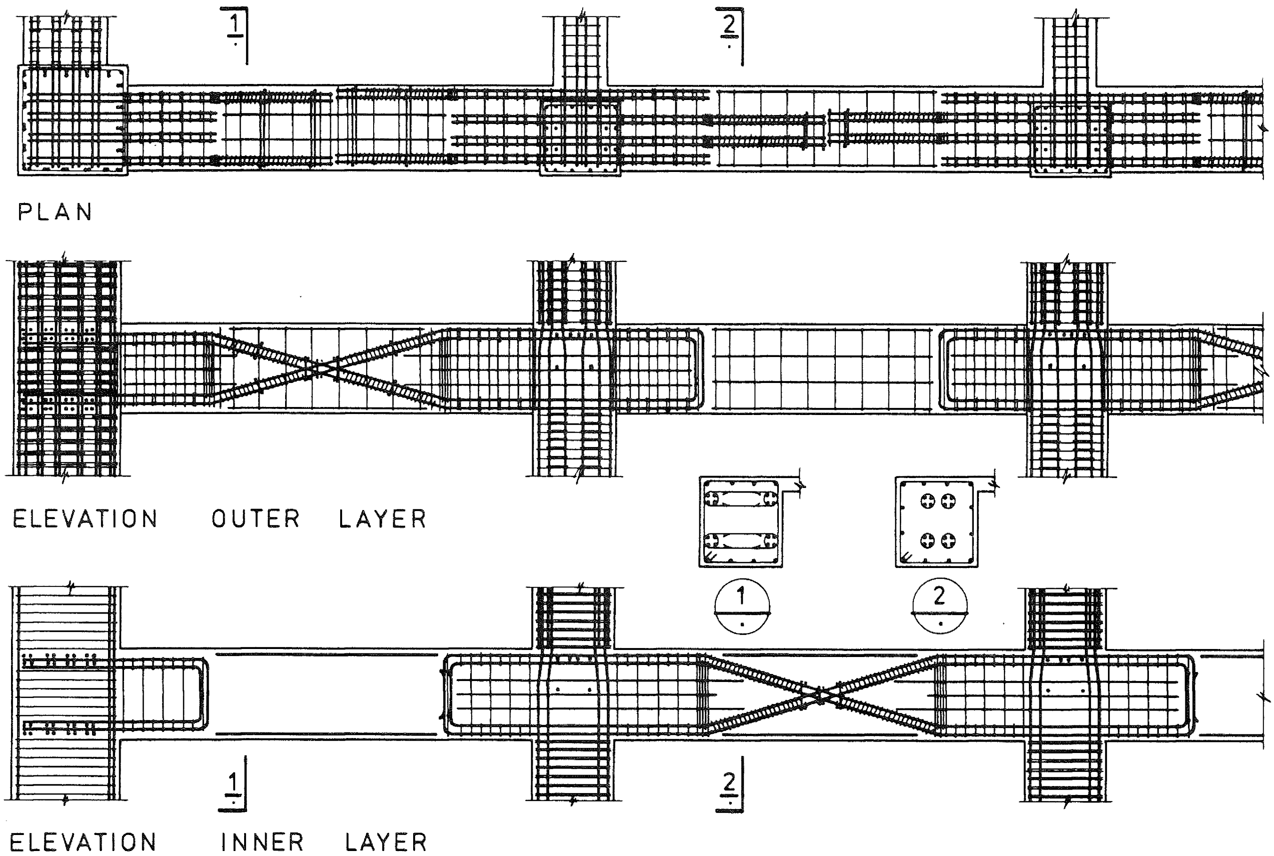


Fig. 17 Typical reinforcing layout

of the New Zealand National Society for Earthquake Engineering. Vo. 10, No. 2. June 1977.

11. R. Park. "Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames - Columns Subjected to Flexure and Axial Load". Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 10, No. 2. June 1977.

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