DIAGONALLY REINFORCED COUPLING BEAMS

A report submitted in partial fulfilment of the requirements for the degree of Master of Engineering at the University of Canterbury, Christchurch, New Zealand.

by

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ABSTRACT

A previous investigation by Paulay\textsuperscript{1} at the University of Canterbury cast doubts on the ability of coupling beams with an $\frac{L}{D}$ ratio of 1.5 or less, to perform satisfactorily in a coupled shear wall system. A theoretical study of the behaviour of coupled shear walls indicated that a ductility of at least twelve was required in the critically situated beams. The conventionally reinforced coupling beams could not attain this ductility, particularly at loads approaching the theoretical ultimate. A system of diagonal reinforcement for the beams was developed in an effort to resist the load by forces within the steel only, hence enhancing the ductility.

By testing one conventionally reinforced beam and three diagonally reinforced beams it was hoped to compare their behaviour. It was especially desired to establish any improvement of the ductility and load retention ability under repeated cyclic loading.

A comparison was made between theoretical and experimental values for the stiffness and elongation of the beams. This gives some indication of the predictability of beam and coupled shear wall behaviour.
ACKNOWLEDGEMENTS

I am most grateful to my Supervisor, Dr. T. Paulay, whose inspiration resulted in the diagonally reinforced coupling beam. His planning of the test program, his guidance and most of all, his endless optimism and personal encouragement throughout the course of the project, were invaluable.

I acknowledge Professor H.J. Hopkins, Head of the Department of Civil Engineering, for the use of facilities.

My very special thanks to Mr. K.L. Marrion, whose care in constructing the test specimen and in taking readings during the testing have ensured the reliability of the experimental results. My thanks are extended to Messrs. J.S. Sheard, J.M. Adams and J.G.G. Van Dyk, Technicians, for their help at various stages of the project.

The generosity of Certified Concrete Christchurch Ltd., for donating half the concrete for this project is gratefully acknowledged.

I am indebted to Mrs. D.E. Ball, who very kindly typed the script.

The financial assistance of Beca, Carter, Hollings and Ferner is most appreciated.
REFERENCES


5. A.C.I. 318-63, "Building Code Requirements for Reinforced Concrete".


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<table>
<thead>
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<tbody>
<tr>
<td>A</td>
<td>area</td>
</tr>
<tr>
<td>(A_s)</td>
<td>area of flexural steel</td>
</tr>
<tr>
<td>(A_v)</td>
<td>area of one stirrup</td>
</tr>
<tr>
<td>b</td>
<td>width of a beam</td>
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<tr>
<td>C</td>
<td>compression force in the main flexural steel</td>
</tr>
<tr>
<td>d</td>
<td>effective depth of a beam</td>
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<tr>
<td>E</td>
<td>Young's modulus</td>
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<tr>
<td>(f'_c)</td>
<td>cylinder crushing strength of concrete</td>
</tr>
<tr>
<td>(f'_{cu})</td>
<td>cube strength of concrete</td>
</tr>
<tr>
<td>(f_y)</td>
<td>yield strength of steel</td>
</tr>
<tr>
<td>H</td>
<td>storey height or building height</td>
</tr>
<tr>
<td>(H_i)</td>
<td>cumulative force induced in the horizontal steel by the stirrups</td>
</tr>
<tr>
<td>(h_e)</td>
<td>equivalent viscous damping factor</td>
</tr>
<tr>
<td>(I_i)</td>
<td>horizontal force induced in the horizontal steel by the stirrups</td>
</tr>
<tr>
<td>I</td>
<td>moment of inertia</td>
</tr>
<tr>
<td>M</td>
<td>moment on the beam</td>
</tr>
<tr>
<td>(P_w)</td>
<td>(A_s/bd)</td>
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<tr>
<td>p</td>
<td>load on beam or a point load applied at the top of the shear wall</td>
</tr>
<tr>
<td>(P_1)</td>
<td>load on the beam at increment</td>
</tr>
<tr>
<td>(P_d)</td>
<td>theoretical ultimate load</td>
</tr>
<tr>
<td>(P_{max})</td>
<td>maximum load applied in a particular cycle</td>
</tr>
<tr>
<td>s</td>
<td>spacing of stirrups or length of a coupling beam</td>
</tr>
<tr>
<td>T</td>
<td>tension force in the main flexural steel</td>
</tr>
<tr>
<td>V</td>
<td>shear on a coupling beam</td>
</tr>
<tr>
<td>(V_c)</td>
<td>shear capacity of the unreinforced beam</td>
</tr>
<tr>
<td>(V_{sd})</td>
<td>shear force resisted by the main diagonal steel</td>
</tr>
<tr>
<td>(V_{ss})</td>
<td>shear force resisted by the stirrups</td>
</tr>
<tr>
<td>(V_u)</td>
<td>total theoretical shear capacity of the beam</td>
</tr>
<tr>
<td>(V_u')</td>
<td>shear capacity of the beam neglecting the contribution of the stirrups</td>
</tr>
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$W$ external lateral load, usually of a triangular shape, on coupled shear wall structure

$W_p$ potential energy within a beam

$W_L$ energy loss which is equal to the area of the load rotation loop

$\phi$ capacity reduction factor

$\theta_{max}$ maximum rotation in a particular cycle

$\theta_p$ plastic rotation of a beam

$\theta_y$ yield rotation of a beam
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CHAPTER ONE

INTRODUCTION

1.1 Shear Wall Structures

The seismic resistance of numerous multistorey buildings is concentrated in reinforced concrete shear walls. These shear walls may be pierced by openings for windows, doors, service holes and other purposes. If these vertical rows of openings are regular then the shear walls are referred to as "coupled", the coupling consisting of a number of panels or beams formed between the vertically arranged openings. The nature of this coupling system will tend to govern the interaction between the shear walls thus an understanding of their performance is necessary for the successful utilisation of the shear wall as a structural element.

There is a certain amount of distrust in shear walls implied in some codes, particularly with respect to ductility. These place limitations upon the height of shear wall structures and also require the presence of rigid jointed flexible frames which are expected to be capable of resisting a certain fraction of the lateral load. An example of this is the height limit of 220\(^1\) to 280\(^1\), depending on the seismic zone, imposed by the New Zealand Ministry of Works\(^4\). These limitations are probably justified at present because of the limited knowledge of the postelastic behaviour, in particular the availability of ductility, in coupled shear walls. This project is orientated towards the examination and improvement of the ductility of the coupling beam system.

1.2 Aims of this Project

A series of tests was carried out by Paulay\(^1\) on
conventionally reinforced coupling beams but, unfortunately, because of the loading system employed, difficulties were encountered in following the postelastic behaviour of these beams. A modified loading system was developed and, using this, a further set of tests was carried out by Beekhuysen using 20" deep beams. As well as these tests another test was carried out using the modified loading system and a 31" deep beam to provide a better idea of its postelastic behaviour. The results of this test conducted in 1969 have not been reported previously. They are reproduced here for the sake of comparison with other results obtained during this project. At this stage the concept of the diagonally reinforced coupling beam (see Fig. 3.3) was developed by Paulay who then carried out a pilot test on the first of these beams (designated Beam 316). The philosophy behind this design is that the shear and moment on the coupling beam can be effectively resisted by a steel force alone. This is provided by the uniformly stressed diagonal reinforcement acting in compression along one diagonal and in tension along the other (see Fig. 1.1).

Subsequently, two modified diagonally reinforced beams were designed in such a way as to avoid the buckling of the bars while in compression; a problem observed in the first test. These two beams, one a 31" deep beam (Beam 317) and the other a 39" deep beam (Beam 395) were then constructed and later tested as part of this project.

The primary aim of this project was to evaluate the improvement in ductility obtained with the use of diagonal reinforcement and to provide some recommendations as to the use of these beams. To this end the test specimen were instrumented for rotation, elongation, and for strains of the main flexural steel along one diagonal only. In the first two tests more extensive readings were taken in an
FIG. 1.1  ACTIONS PRODUCED BY LOAD AND REINFORCEMENT IN A DIAGONALLY REINFORCED COUPLING BEAM.
effort to make a comparative study of other aspects of the behaviour of the two types of beams. In the final two tests an examination was also made of the crack widths at loads up to three quarters of theoretical ultimate. These were made in an effort to obtain some idea of the serviceability of the beams under a "working load" magnitude of earthquake.

Concurrently with this project, Santhakumar, at the University of Canterbury, developed theoretical expressions for the stiffness and elongation of the diagonally reinforced beams. These have been included in the report to test them against experimentally observed values.
CHAPTER TWO

DUCTILITY REQUIREMENTS FOR COUPLING BEAMS

2.1 Introduction

The major part of the earthquake resistant capacity of a building may be mobilized in the inelastic range. Because of this it is important to have ductility in the building in order to dissipate the energy inelastically without failure. The coupling beams, often short and relatively deep, may be subject to high shearing stresses when the ultimate flexural strength is to be developed. These high shear forces restrict the ductility obtainable, thus it is important to assess the ductility demand of the coupling system when the overall, ultimate strength of the coupled shear wall is being determined.

2.2 Derivation of Ductility Requirements for a Typical Shear Wall

There are basically six stages in the evaluation of the ductility requirements for a coupled shear wall system. Initially the wall loading is obtained from code requirements, and the assumptions of perfect elast-plastic, bilinear behaviour in the process of plastic hinge formation and uniformity of coupling beams is made. Fig. 2.1 shows a typical shear wall structure. Fig. 2.2(a) shows the curves obtained for various stages (the circled numbers) of the loading pattern for a 20 storey building with W = 600 k and P = 80 k (reproduced from Ref. 3).

Stage one is to carry out a laminar analysis of the structure under the code specified loading. From this analysis typical distributions of laminar shear, laminar
FIG. 2.1 A TYPICAL SHEAR WALL AND LOAD PATTERN
FIG. 2.2 LAMINAR SHEAR FORCE DISTRIBUTIONS, DUCTILITY REQUIREMENTS AND DEFLECTIONS FOR A COUPLED SHEAR WALL STRUCTURE WITH DIFFERENT DISTRIBUTIONS OF LAMINAR SHEAR STRENGTH
rotations and roof level deflections are obtained.

Stage two involves the choice of the amount of overload acceptable before the ultimate strength of the coupling system is attained. The quantities for this stage are simply those from stage one increased by this factor. This is the elastic limit of the structure and the overall ductility can be expressed in terms of the deflections at this load.

In stage three an additional load is applied so that the ultimate capacity of the laminae is reached over at least the upper 90% of the height of the wall. The walls are still elastic but the coupling beams have yielded. The axial loads on the walls have now reached a maximum as these loads are the summation of the shears on the fully plastified coupling beams. One of these loads is usually a net tension and the other a large compression force when allowance is made for existing gravity loads.

Stage four involves the addition of a further load increment to attain yield in the wall subject to axial tension. Both walls act elastically as coupled cantilevers up to this stage.

The final load increment is that applied to cause the ultimate capacity of the wall in compression to be reached. This is stage five and it is now that the collapse mechanism is completed. At this stage the reinforcement requirements for the elements of the structure can be calculated, and changes to the previously assumed wall properties may be made.

In order to evaluate the ductility demand for the coupling beams, the required overall ductility is imposed on the structure. In the example structure an inelastic roof level displacement of three times that of the elastic limit has been imposed in accordance with the currently
accepted value of four for a desired overall ductility factor (see Fig. 2.2). This is stage six of the behaviour. These curves indicate that a ductility factor of 11.8 is required in the critically situated coupling beams.

If variable strength coupling beams are used (their strength being proportional to the elastic laminar shear) then stages two and three are the same, thus there is a smaller ductility demand up to this stage. The distribution of the laminar ductility factors during the next three stages is presented in Fig. 2.2(b). From these curves it can be observed that very large ductilities are required in the upper half of the structure. This is not as serious a problem as may be suspected because these beams carry a lower load and hence a lower steel content, they possess a greater ability to undergo plastic rotations.

The ability of the more highly stressed deep coupling beams to meet the required ductility requirement was one of the major objectives of this project and as such will be discussed in greater detail in due course.
CHAPTER THREE

TEST SPECIMEN AND TEST PROCEDURE

3.1 The Test Specimen

Four beams were involved in this series of tests, three being 31" deep and one 39" deep, the respective length/depth ratios being 1.29 and 1.02. The central part of the specimen was 6" thick and the end blocks 8" thick (see Fig. 3.1). The depth to width ratio of the beams was larger than that commonly encountered in practice. Problems associated with this will be discussed at a later stage.

The load was applied to the beam through the end blocks and produced the moment pattern as shown in Fig. 3.1. The effect of the reversed loading is shown by the dotted line. The end blocks were designed to be fairly lightly stressed during the loading and although some cracking was observed these closed upon removal of the load. This would indicate that the steel did not approach yield.

All beams were cast from one side to ensure similar bond conditions for all the steel. Building the beams from one side was also necessary for ease of construction in the laboratory. Practical difficulties associated with the pouring of the diagonally reinforced beams will be discussed at a later stage.

3.2 The Concrete

The concrete was obtained from a local 'ready-mix' plant in Christchurch and is representative of the concrete used in and around the city on various building projects. The concrete mix proportions can be found in Table 3.1.

The concrete was compacted during placing using an
FIG. 3.1 PRINCIPAL DIMENSIONS AND LOAD/PATTERN OF THE TEST BEAMS
immersion type vibrator and after screeding and floating, the beam, together with the test cylinders, cubes and prisms, was cured for seven days under a layer of wet sacking covered with a polythene sheet to minimize evaporation losses.

The test cylinders, cubes and prisms belonging to each beam were tested either immediately before or after the beam itself. The relevant strengths and properties of the concrete used in each of the four beams have been tabulated in Table 3.11.

3.3 The Reinforcement

Details of the reinforcement for Beams 315 and 317 can be seen in Figs. 3.2 and 3.3 respectively. The reinforcement for Beam 316 was similar to that in Beam 317 except for the absence of confining ties around the main reinforcement. Beam 395 had a similar reinforcement cage to Beam 317 except for the greater depth of section, hence a slightly larger cage. Photographs of the reinforcement cages can be seen in Figs. 4.2, 4.25, 4.4 and 4.45. All reinforcement consisted of deformed bars except for the plain #2 bars used as stirrups and ties in Beams 317 and 395. The properties of the reinforcement are summarised in Table 3.11.

3.4 The Loading Frame

The loading frame used was the same as that employed by Paulay and this is fully described in his work.

3.5 Instrumentation

3.5.1 Steel Strain Measurements

Steel studs were welded on to the reinforcement
FIG. 3.2 DETAILS OF BEAM 315 REINFORCEMENT
FIG. 3.3 DETAILS OF BEAM 317 REINFORCEMENT
at 4" centres to form the gauge location over which the strain was measured using a Demec strain gauge. Drilled stainless steel plates were attached to the end of these studs to receive the points of the Demec gauges. Allowance for movement of these studs was incorporated by casting a $\frac{1}{4}$" diameter hole in the concrete around them.

The gauged reinforcement and gauge locations for Beams 315 and 317 are shown in Fig. 3.2 and 3.3. Beam 316 had the same studs along the diagonal reinforcement as Beam 317 but it was also instrumented along the stirrups as in Beam 315. Beam 395 was instrumented along the diagonal bars as in 317 but, because of its greater depth, two more gauge locations were required.

The numbering of stirrup gauge lengths was such that the tens digit indicates the number of the stirrup while the units digit indicates the particular location along the length of the stirrup.

3.5.2 Rotation Measurement

Dial gauges were used to measure the vertical and horizontal movement along a vertical reference line on the end block. Steel plates were attached to the concrete surface to receive the points of the dial indicators which were in turn fixed to a stationery frame.

From these readings the rotation and horizontal displacements along the beam centre line were calculated as outlined in Paulay's work.

Because of the high rotations and elongation of the beam and the limited travel of the dial gauges (2" in most cases), the gauges had to be altered several times during the course of the test in order to follow the end block movement. These adjustments were allowed for by altering the initial zero readings when the results were analysed.
3.5.3 **Load Application**

The load was applied to the beam by means of a 100 ton capacity hydraulic jack fed by a 10,000 psi capacity Riehle testing machine. Coupled with the 100 ton loading jack was an identical jack pushing against a 100 ton capacity load cell (see Fig. 3.4). The load cell and the jack were both mounted in the jaws of a testing machine which provided the necessary reaction. The load applied by the jack to the beam at any stage was measured by the load cell which was connected to a strain bridge calibrated to give the load directly in kips. The benefit of this indirect loading system is that at any one time when the testing machine is cut off from the circuit, it is a "fixed volume" system in which increasing deflections will cause a reduction in load. This prevents the beam from "running away" at yield as the testing machine tries to maintain the load resulting in a rapid and relatively uncontrolled increase in deflections which can end in premature failure.

The line of action of the ram was occasionally checked using a taut string line, as any eccentricity results in unequal moments acting on the beam.

3.5.4 **Crack Observations**

The development of cracks was carefully marked at regular intervals of loading and when there was a significant change, a photograph was taken, some of which are reproduced in the report.

The widths of some typical cracks on Beams 317 and 395 were measured using a microscope. These measurements were taken at loads up to 0.75 theoretical ultimate, and are intended only as a general guide to the size of the cracks sustained under an earthquake loading which does not cause yielding in the coupling beams. These measurements are not
FIG. 3.4 SCHEMATIC DIAGRAM OF LOADING AND LOAD MEASURING SYSTEM
necessarily the maximum crack widths.

3.5.5 The Testing Procedure

This was similar to that employed by Paulay but it is slightly simplified.

Initially several readings were taken to determine the "no load" values of the steel strains and dial gauge readings.

After this the following actions were taken at each load increment.

1. Valve closed between oil pump and jack lines after the required load has been applied.
2. Load and time recorded.
3. Dial gauges for rotation measurements read and recorded.

When necessary:

4. Strains along the main steel recorded and Demec gauges checked on standard bar.
5. Stirrup strains measured.
6. Cracks marked, crack widths measured.
7. Eccentricity of the jack recorded.
8. Photographs taken.

Finally, at each increment

9. Final value of time and load recorded.

The readings were recorded manually and processed by an electronic computer.

3.6 The Loading Procedure

The beams were loaded in several cycles to destruction.
In these cycles the load was increased by predetermined increments from zero load to its maximum and then reduced to zero. A new cycle commences when a similar sequence is followed with the load being applied in the other direction. The odd numbered cycles load the beam in one direction and the even numbered cycles load the beam in the reverse direction.
CHAPTER FOUR

DISCUSSION OF RESULTS

4.1 General

This chapter outlines the results obtained from the testing of the four beams and compares their behaviour. The results for Beam 315 have been treated first followed by Beams 316, 317 and 395 respectively. Finally the comparison of the four beams is made.

The failure of Beam 315 was restricted to a relatively narrow band near the supports. It was decided to repair the beam and reload it to failure. This repaired beam is referred to as Beam 1315 and has been treated in the section for Beam 315.

4.2 Beam 315

4.2.1 Beam Properties

This beam was included in the test series primarily to enable a comparison of its behaviour with the diagonally reinforced beams. It was also desired to gain some idea of the ductilities available when the beam was tested using the "constant volume" loading system.

The dimensions of the beam are given in Figs. 3.1 and 3.2, and the component properties in Tables 3.1, 3.11 and 3.111. The span to depth ratio was 1.29. The load sequence is presented in Fig. 4.1 and consists of four cycles of moderate magnitude loading. These are followed by two cycles in which the beam was loaded past yield and finally one cycle in which the beam could only carry about 25% of its theoretical ultimate before failing. These cycles
Theoretical Ultimate Load $P = 184.0$ kips

- Denotes a rotation reading
- $\bigcirc$ Denotes a steel strain reading

FIG. 4.1
LOAD SEQUENCE FOR BEAM 31.5

$[\text{Raw text converted to natural text}]$
of moderate magnitude will be referred to as "elastic cycles" as in these cycles the load is that anticipated from the design earthquake with appropriate load factors and capacity reduction factors included. The reduction factor applied for Beams 315 and 1315 is 0.85 as the beams fail in shear. With a load factor of 1.25 this gives an "elastic" load of 0.63 times the yield load. The mode of failure for the other beams is not clearly defined but a capacity reduction factor of 0.85 is assumed. The elastic load in this case is again 0.63 times the yield load.

The reinforcement cage is shown in Figs. 4.2 and 4.3. These figures show the welded studs used for strain measurements in their protective tubing. As a rule all photographs presented for this test view the beams from the East and references, such as to supports, apply to this side of the beam.

4.2.2 Behaviour of the Flexural Reinforcement

4.2.2.1 The distribution of strains.

The distribution of strains for both the top and bottom steel is shown in Fig. 4.4 for different load ratios (pi/psi) during the first and fifth cycles. Because of the random formation of cracks considerable differences were sometimes observed between the strains measured at either side of the beam. A crack extending across the width of the beam did not necessarily cross corresponding gauge lengths.

Generally the strains in the two layers of reinforcement at the top or at the bottom are very similar. The strain in the outer #8 bars is usually slightly larger, as would be expected with a linear distribution of strain across the section.

As was observed by Paulay, \(^1\) the tensile strains became
FIG. 4.2 REINFORCEMENT CAGE FOR THE END BLOCKS AND BEAM OF BEAM 315

FIG. 4.3 REINFORCEMENT CAGE FOR BEAM 315
FIG. 4.4 STEEL STRAIN DISTRIBUTION IN BEAM 315
rather large at the point of zero moment due to the formation of diagonal cracks at higher loads.

Near the ultimate load the tensile strains in the two layers of reinforcement passing through the compression zone may vary greatly. This is because, near ultimate, the beam undergoes severe shear displacements (see Fig. 4.17). There is also a tendency for the beam to separate along the shear cracks at the support and along the diagonal cracks inducing combined shear and flexural stresses in the steel. Superimposed on this are the stresses induced by "hinging" about the steel which induces compression in the outer bars and tension in the inner bars.

The uniformity of strains over the majority of the beam is due primarily to the presence of the secondary, horizontal reinforcement. This controls the formation of cracks which causes the "peaking" effect in the strain profile.

4.2.2.2 The tension force distribution.

The behaviour of the beam and the mode of load resistance can be more clearly visualised if the strains are translated into internal forces. The tension force distribution curves are shown in Fig. 4.5. These are obtained by taking the beam strain for the four bars in each face of the beam and by using 29,500 ksi for the Modulus of Elasticity of the steel.

The reason for the non-linearity of the curves is explained by Paulay in his thesis. The curves have a remarkable similarity except in the compression zone for reasons described previously. Yield has begun in both the top and bottom bars at 81% of the theoretical ultimate.

4.2.3 The Behaviour of the Stirrups

4.2.3.1 Load stress relationships.
FIG. 4.5 STEEL FORCE DISTRIBUTION IN BEAM 315
Six four inch gauge lengths were provided along the length of each of the five instrumented stirrups (see Fig. 3.1). The four largest measured stresses are recorded in Fig. 4.6. As indicated the first portion of the graphs is plotted for the first load cycle and the remainder of the graph for loads in the fifth cycle.

For all but the end stirrups the stresses are in good agreement with the A.C.I. recommendations. The stresses are negligible until cracking occurs then rise almost linearly with load. The continuity of the curves between the readings for the first and fifth cycles indicate very little strength degradation with cyclic loading. The fact that there is not a higher stress in the stirrup where it is crossed by the diagonal crack is because of the predominance of a shear slip failure over a separation failure. The separation failure is accompanied by high local stresses where the stirrup is crossed by the diagonal crack.

The end stirrup stress is low and not particularly uniform because of the load transfer into the adjacent end block at lower loads. The localized shear slip failure is apparent from the sudden increase in stress in stirrup nine as the concrete's shear capacity is destroyed and load is transferred to the steel.

All stirrups show very little strain until cracking occurs at approximately \( .22 \, P_u \).

The A.C.I. code prediction is that the maximum shear permitted for the concrete of an unreinforced web before diagonal cracking is

\[
V_c = \phi b d \left( 1.9 \sqrt{f_{tc}} + 2500 \frac{P_w}{M} \frac{V_d}{M} \right)
\]

with \( d = 28'' \), \( P_w = 1.64 \% \), \( \frac{V_d}{M} = 1 \) and \( \phi = 1 \).
FIG. 4.6 STIRRUP STRESSES IN BEAM 315
\[ V_c = 28 \times 6 \left( \frac{1.9}{5500} + 2500 \times 0.0164 \right) \]
\[ = 30.7 \text{ kip} \]
\[ = 0.167 P_u^* \]

For a section with web reinforcement the A.C.I. code predicts that the ultimate shear carried by the section is

\[ V_u = V_c + V_s \]
\[ V_s = \frac{A_{\text{f.d}}}{S} \left( \frac{584 \times 46.5 \times 28}{4} \right) \]
\[ = 190.5 \text{ kip}^* \]
\[ = 1.04 P_u^* \]
\[ V_u = 221.2 \text{ kip} \]
\[ = 1.20 P_u^* \]

The failure mechanism associated with separation along the main diagonal crack should not therefore form. This was borne out by observation.

4.2.3.2 The strength of the web reinforcement.

The forces in the stirrups, at points where they crossed the major diagonal crack, have been plotted in Fig. 4.47. There is a rather regular pattern which is consistent with the tendency of the beam to separate along the major diagonal. When the load reaches .94 \( P_u^* \) in the fifth cycle there is a departure from this pattern because of the shear failure, especially at the right hand support.

Fig. 4.48 is a plot of the contribution of all stirrups crossing the main diagonal against the load. As would be
FIG. 4.7  STIRRUP FORCE DISTRIBUTION ALONG THE CRITICAL DIAGONAL OF BEAM 315

FIG. 4.8  THE SHEAR FORCE CARRIED BY THE STIRRUPS ACROSS THE CRITICAL DIAGONAL OF BEAM 315
expected there is a discrepancy between the shear carried by the stirrups and the shear force on the beam. During the first cycle the stirrups and concrete share the shear force but by the fifth cycle the concrete's contribution may be neglected. The shear force in the stirrups at low load level is greater than the shear on the section. This is due to permanent displacements across the major diagonal crack which cause a force in the stirrups at zero load. These permanent displacements are carried by shear movement along the crack such that when the load is removed the sides of the crack do not match and thus will not fully close. Diagonal separation failure does not occur because the maximum force to be transferred across the diagonal crack does not exceed the combined load capacity of the stirrups (243 kips = 1.32\text{kip})

4.2.4 **Deformation**

4.2.4.1 Rotations.

The load rotation relationship is shown for all seven load cycles in Fig. 4.9. The behaviour of the two ends of the beam is very similar up to the end of the fifth cycle where the extensive crushing of the right hand support causes a noticeable discrepancy in the two rotations.

Theoretical stiffnesses were calculated for both the cracked and uncracked sections assuming infinitely rigid end blocks, but because of this assumption the rotations of the beam proper were overestimated by 57% when both parts were in the uncracked state. When the beam was cracked the overestimation was only about 5-12%. The theoretical beam rotations with these factors taken into account are shown in Fig. 4.9.

In the light of the assumptions made in their derivation the agreement can be considered quite satisfactory
FIG. 4.9 LOAD ROTATION RELATIONSHIP FOR BEAM 315
The onset of cracking can easily be observed in the first load cycle and also the onset of yield during the fifth and sixth cycles is quite marked. The stiffness of the beam during the first four cycles is almost constant with only minor deviations at low load caused by cracking. At the beginning of the sixth cycle the effects of cracking and subsequent yield are considerably more marked, with the stiffness being quite severely lowered. The seventh cycle has an extremely low stiffness due to the concrete falling from the end regions severely reducing the shear capacity of the section.

The most important function of the load-rotation diagram is the information it provides on the ductility of the beam. The yield rotation obtained from the sudden change of slope in the fifth cycle is $5 \times 10^{-3}$ radians. The largest rotation at a reasonable sustained load in the fifth cycle was approximately $12 \times 10^{-3}$ radians. This resulted in a ductility of approximately

$$\frac{12 \times 10^{-3}}{5 \times 10^{-3}} = 2.4$$

at a load ratio $P_1/P_u$ of .94. In the sixth cycle, if the fact that there is some rotation at zero load is considered, then a ductility of

$$\frac{25 \times 10^{-3}}{5 \times 10^{-3}} = 5$$

is obtained for a load ratio of .74.

The ductilities do not give a true indication of energy adsorption properties as may be seen in Fig. 4.10. The first cycle is close to the shape of the bilinear response but after this the curves become such that, for a certain ductility, they absorb considerably less energy than theoretically predicted. This may be expressed
FIG. 4.10
COMPARISON OF ACTUAL AND BILINEAR LOAD

Rotation (radians x10^-3)

Load (kips)

Cycle 5, Beam 315
Theoretical Bilinear Response
Deficit Energy
Cycle 6, Beam 315

0 4 8 12 16 20 24 28
as an equivalent viscous damping factor which will be
described more fully in Section 5.3.

If the shear walls connected by the coupling beam
are assumed rigid with neither flexural nor shear
distortions, then an estimate may be made of the inter-
storey deflections. The deflection prescribed by the
New Zealand loading code is \( \frac{H}{400} \) based on the "elastic"
lateral seismic force. Deflections of up to \( \frac{H}{200} \) may
be allowed "where adequate clearances are provided for
non-structural parts". These deflections correspond
to rotations of \( 2.5 \times 10^{-3} \) radians and \( 5 \times 10^{-3} \) radians
respectively. This gives only an approximate estimate
as wall elongations are not considered. A tension in
one wall and a compression in the other wall can result in
further rotations and inter-storey drifts of the structure.

The rotation at the working load for Beam 315 is
approximately \( 3.8 \times 10^{-5} \), thus special precautions would
have to be taken.

4.2.4.2 The variation of stiffness.

From the slope of the load rotation curve,
shown in Fig. 4.9, the beam stiffness may be obtained.
The stiffness so obtained has been plotted against load
ratio in Fig. 4.11.

The curve indicates that, after the initial cracking
in the first cycle, the beam’s stiffness is reduced by
approximately 75%. The curve for the second cycle is of
similar shape to that of the first cycle, but the reduction
in stiffness due to cracking is far less marked. The
following cycles commence with a low stiffness which
gradually increases as cracks are closed. Once the initial
cracking is complete the curves generally have three
definite stages.
FIG. 4.11  THE VARIATION OF STIFFNESS WITH CYCLIC LOADING IN BEAM 315
(a) A "hardening" range where the stiffness increases. This is associated with closure of the cracks.

(b) A "steady" range where the stiffness is approximately constant. This represents linearly elastic behaviour.

(c) A "softening" range where the stiffness is observed to decrease due to plastic concrete deformations and yielding of the reinforcement.

4.2.4.3 Elongation of the beam.

The curves obtained from the displacement of the end blocks are reproduced in Fig. 4.12. These curves also show the significant stages of the beams behaviour, such as cracking and yielding. The large deformations (hence lower stiffnesses) during the first two cycles are due to anchorage slips and initial formation of the cracks.

The drop in elongation in the seventh cycle is probably partially due to the failure of the arch or truss action as the concrete "compression members" are severely affected in the end regions. The main reinforcement is subject to dowel action which causes "kinking" of the bars near the supports. This "kinking" results in a reduction in length of the beam and may help cause the drop in elongation mentioned.

4.2.5 Crack Formation and Failure Mechanism

The first cracks formed at the tension corners of the beam and propagated in a near vertical direction. The diagonal cracks developed later at about 46% of $P_u^*$, the holes for the Demec studs tending to act as initiators. In the second cycle a similar crack pattern was observed
FIG. 4.12  THE ELONGATION OF BEAM 315
but with the diagonal cracks appearing slightly earlier at 40% $P^*$. As the loading progressed more cracks appeared near the corners of the beam, especially fairly steep cracks, which became very pronounced above 90% $P^*$ in the fifth cycle. In the third and fourth cycles there were very few new cracks formed and only minor propagation of existing ones. The development of the crack pattern is shown in Figs. 4.13 to 4.17.

The destruction of the concrete near the support is due to movement along shear cracks combined with high intensity diagonal compression. This is accelerated by reversal of loading and the accompanying large movements along the shear cracks. This type of failure is accompanied by excessive yielding of the end stirrups and local yielding of the main steel. The shear displacement can be clearly seen in Fig. 4.17, which shows the steel with surrounding crushed concrete removed. The concrete appears to have been crushed by sliding, although some lateral bursting has taken place near the top right hand corner of the beam.

4.2.6 The Repaired Beam

Although both the flexural reinforcement and the stirrups had yielded in the vicinity of the right hand support, it was decided to repair the beam. The failure was restricted to a fairly narrow band near the end of the beam, as may be seen in Fig. 4.17. At this stage a random sample of crack widths in the central portion of the beam was taken. These revealed that although the beam had reached almost 95% of ultimate, on the average, the crack widths were quite low. When both sides of the beam were examined, only 20% of the cracks were over 0.002" and none over 0.01", thus, in practice, this section of the
FIG. 4.13  THE CRACK PATTERN OF BEAM 315 AT 0.69P* IN CYCLE ONE

FIG. 4.14  BEAM 315 AT 0.69P* IN CYCLE TWO
FIG. 4.15  BEAM 315 AT 0.95\textsuperscript{P*} IN CYCLE FIVE

FIG. 4.16  BEAM 315 AT 0.77\textsuperscript{P*} IN CYCLE SIX
FIG. 4.17  THE CRACK PATTERN AFTER THE FINAL LOAD INCREMENT WHEN

\[ P_i = 0.24P_u \]

FIG. 4.18  BEAM 315 PRIOR TO REPAIRING WITH AREAS OF CRUSHED CONCRETE REMOVED
beam could quite easily be restored to a serviceable surface. The repairing procedure was similar to the one described by Paulay for Beam 313 in his test series. The reinforcement in the end regions was straightened and the concrete removed as shown in Fig. 4.18. The mix used was similar to the one used for repairing Beam 313 and had a cylinder strength of 8330 psi after 68 days.

A similar loading sequence to the original beam was followed as shown in Fig. 4.19. Only the rotations of the end blocks were measured and this gave the load rotation curve shown in Fig. 4.20. As might be expected, the repaired beam was not as stiff as the original one especially at the beginning of the elastic cycles, this low stiffness being associated with the closing of cracks, many of which were already present. The flexural reinforcement had also entered the plastic range at the failure of Beam 315, and this in itself would result in a softer type of response because of the Bauschinger effect in the steel.

The ultimate load is very close to the theoretical one and about 4% above the ultimate load found for Beam 315. This increase in strength is probably due in some measure to two effects, the increased strength of the concrete in the support region and the strain hardening of the steel.

The ductility of the repaired section in the sixth cycle was

\[
\frac{31 \times 10^{-3}}{5 \times 10^{-3}} = 6.2
\]

at a load ratio of 0.87. This is an improvement on that achieved in Beam 315 because of strain hardening of the reinforcement and stronger concrete in the end regions.

The elongation of Beam 1315 is recorded in Fig. 4.21. The effects of cracking are not as marked as in Beam 315.
FIG. 4.19
LOAD SEQUENCE FOR BEAM 1315

*\( P_* = 184.0 \text{ kips} \)

\[ P_* = 184.0 \text{ kips} \]

- Denotes a rotation reading

Increment No.

Cycle No.

Theoretical Ultimate Load \( P_* = 184.0 \text{ kips} \)
FIG. 4.20 LOAD ROTATION RELATIONSHIP FOR BEAM 1315
FIG. 4.21 THE ELONGATION OF BEAM 1315
and a near linear load elongation relationship is observed. As would be expected there is very little permanent displacement until yielding occurs in the fifth cycle because the crack formation is already well advanced. Another contributing factor is that the cracking in the end regions of the beam is not as extensive as in Beam 315. This could be for two reasons, either the stronger concrete in this area or the steel has strain hardened and so reduced flexural cracking. The development of the crack pattern can be seen in Figs. 4.22 and 4.23.

The failure mechanism is again sliding shear but in this case along the junction of the old central portion and the repaired section. There has been some spalling as can be see in Fig. 4.23. This was caused by the combined action of shear slip and diagonal compression.

4.3 **Beam 316**

4.3.1 **Beam Properties**

Beam 316 was the first of the diagonally reinforced specimens to be tested. It is similar to Beam 317 shown in Fig. 3.3, but does not have the ties around the main reinforcement. The component properties are given in Tables 3.I, 3.II and 3.III. The load sequence presented in Fig. 4.24 consists of a total of thirteen cycles. The first four of these are elastic cycles, followed by three cycles of "plastic" loading to yield, five more elastic cycles and, finally, one plastic load to failure.

The reinforcement cage is shown in Figs. 4.25 and 4.26, in which the stirrups instrumented for this test can be seen.

All photographs for this beam, unless stated otherwise, view the beam from the West and all references (for example
FIG. 4.22 THE CRACK PATTERN FOR BEAM 1315 AT 0.65P* IN CYCLE THREE

FIG. 4.23 BEAM 1315 AT FAILURE IN THE SIXTH CYCLE P₂ = 0.94P*
Theoretical Ultimate Load $P_U^* = 124.5$ kips

- Denotes a rotation reading

- Denotes a steel strain reading

Cycle No.

Increment No.
FIG. 4.25   REINFORCEMENT CAGE FOR BEAM 316 SHOWING END BLOCK REINFORCEMENT AND ANCHORAGE OF MAIN STEEL

FIG. 4.26   REINFORCEMENT CAGE FOR BEAM 316 SHOWING THE INSTRUMENTED STEEL
to the supports) apply to this side of the beam.

4.3.2 Behaviour of the Flexural Reinforcement

4.3.2.1 The distribution of strains.

The distribution of strains along the instrumented diagonal steel is shown in Fig. 4.27 for different load ratios during the first five cycles. As is to be expected, the strains in the two layers of bars are very similar near the centre of the beam with the strain in the outer layer of bars becoming greater near the end, due to the higher lever arm. The slightly unsymmetrical behaviour of the two ends of the beam is probably due to the presence of the stirrup gauge points at the right hand end. There is a more symmetrical distribution of strains in Beam 317 (see Fig. 4.17) in which there are no stirrup gauge points.

The similarity of strains for the load ratio of \(0.8 P_i/P_u\) when the bars are in both compression and tension indicates very little degradation with elastic cyclic loading.

4.3.2.2 The steel force distribution.

The distribution of the compressive and tensile forces in the steel are shown in Fig. 4.28. These forces are found as previously outlined in 4.2.2.2. When the load is above about 0.4 \(P_u\) there is a fairly uniform distribution of the force with a higher force in the right hand end. The higher force could be due to the effect of the holes for the stirrup Demec studs, which are present at that end only. The holes cause a reduction in concrete area and this may result in the neutral axis rising. This would reduce the lever arm and induce a higher force in the steel. This could explain the deeper crack free
FIG. 4.27 DISTRIBUTION OF STRAINS IN THE FLEXURAL REINFORCEMENT OF BEAM 316
FIG. 4.28 THE DISTRIBUTION OF STEEL FORCE IN BEAM 316.
'compression' zone at this end of the beam as seen in Fig. 4.38. The lower force in the central part of the bars at low load ratios is due to the moment being resisted by the secondary horizontal reinforcement. Consider this secondary reinforcement being close to yield along the full length of the beam, as it probably will be for moderate load ratios. This steel has very little effect near the end of the beam because the majority of the tension force necessary is applied by the main steel near the centre of the beam where the moment is low and the lever arm of the main steel is low, then the small contribution of the secondary reinforcement and the concrete tensile force at a high lever arm can be significant.

The compressive steel forces are low because the effective area in compression may be large and the effect on the steel small. Sometimes the 'compression' steel is not even in the compression zone as is the case for a load ratio of \(0.32 \frac{P_i}{P_u}\) near the centre of the beam. The lack of cracks near the centre of the beam (see Fig. 4.38) is illustrated by the low steel forces in this area, the majority of the load being taken by the concrete.

The theoretical ultimate load based on yield appears to be surprisingly accurate as the bars are just approaching tensile yield at a load ratio of \(0.97 \frac{P_i}{P_u}\).

4.3.2.3 The position of the internal forces.

By assuming that the tension force is applied along the centroid of the diagonal steel, the position of the centroid of the compression force can be determined. This is plotted in Fig. 4.29. At low load levels especially the computed compression centroid may be well outside the beam. This would indicate that the magnitude of the tension force resultant, and perhaps its position, may be incorrect.
FIG. 4.29 THE POSITION OF THE INTERNAL COMPRESSION FORCE IN BEAM 316
This is to be expected as no account has been taken of the horizontal steel (2 #4 bars, top and bottom). This not only increases the tension force at any section but it also moves the position of the tension force resultant towards the outside of the beam. The concrete also carries some tension at low loads before it cracks. As the load level increases the compression force resultant moves closer to the centroid of the main diagonal compression steel. This is because the effect of the secondary reinforcement becomes insignificant and the concrete reaches a stage where it is so cracked that it can no longer sustain its share of the internal tensile forces.

4.3.3 The Behaviour of the Stirrups

4.3.3.1 Load stress relationships.

The load stress relationship is shown in Fig. 4.30, the portions of the curve below a load of 0.8F* being obtained during cycle one and the remainder during cycle five. The development of the diagonal crack can easily be seen in stirrups two and three where gauge lengths four and six respectively are crossed by the crack. The effect of the diagonal crack is probably not as great because of the restraining effect of the main diagonal bars. These bars restrict the crack width and thereby reduce its effect on the stirrups.

The stresses in stirrup one are low because up until the formation of the vertical shear crack at the support the end block restricts the expansion of the beam in this region. If the expansion is small then the stress it induces in the stirrup is low. An exception to this is when the movement can occur across the diagonal crack inducing a locally high stress, as happened at location sixteen. Areas of low crack development show up quite
FIG. 4.30 LOAD-STRESS RELATIONSHIP FOR THE STIRRUPS IN BEAM 316
quite markedly on the curves as regions of low stirrup stress (compare Fig. 4.30 with the photographs Figs. 4.38 and 4.40).

The dramatic increase in stresses just after a load ratio of $1.0 \frac{p_1}{p_u^*}$ is reached is because the shear has previously been resisted by the main diagonal steel and, when it reaches yield, it can sustain no further increase. The stirrups are then called upon, quite suddenly, to resist all the increase in load and this sudden rise in stress results. The calculated shear taken by the unreinforced section and the main diagonal steel is 127.3 kips according to A.C.I. recommendations. This is very close to $p_u^*$ where the increase occurs. This would strongly support the above observation.

The non-uniformity of the stirrup strains is probably due, in some part, to the cracks in this beam being relatively steep as compared with Beam 315 (compare Figs. 4.40 and 4.15). The flexural cracks tend to run along the stirrup whereas the diagonal cracks tend to run across it. The absence of horizontal steel has probably made the crack pattern and hence the stirrup strains a lot less uniform.

The shear capacity predicted by the A.C.I. code are as follows:

$$V_c = b \cdot d \cdot \sqrt{f'_c}$$

$$= 23.6 \cdot 2 \cdot \sqrt{4825}$$

$$= 23.3 \text{ kips}$$

$$= 0.13 p_u^*$$

The value of $2 \sqrt{f'_c}$ has been adopted because the more accurate A.C.I. expressions are not applicable in this case. They cannot be strictly applied as the main
flexural steel is not horizontal.

The shear contribution of the main steel is

$$V_{sd} = 2A_s f_y \sin \theta = 99.6 \text{ kips}$$

where $\sin \theta$ is the angle of the diagonal steel to the horizontal.

The shear contribution of the stirrups would be

$$V_{ss} = A_s f_y \cdot d/s$$
$$= 21.53.3 \cdot 28/5.72$$
$$= 54.9 \text{ kips}$$

Thus the total theoretical shear capacity would be

$$V_u = 23.3 + 54.9 + 99.6$$
$$= 177.8 \text{ kips}$$
$$= 1.41 p^*_u$$

The shear taken by the section, ignoring the effect of stirrups would be

$$V'_u = 23.3 + 99.6$$
$$= 122.9 \text{ kips}$$
$$= .93 p^*_u$$

4.3.3.2 The strength of the web reinforcement.

The forces in the stirrups where they cross the potential diagonal crack are plotted in Fig. 4.31. As the stirrups in only one half of the beam have been instrumented the symmetrical behaviour shown has been assumed.

The diagonal cracking at a load ratio of between 0.31 and 0.4 is clearly shown by the sudden increase in the force. The restraining effect of the main reinforcement on the width
FIG. 4.32 SHEAR FORCE CONTRIBUTION OF ALL STEEL ACROSS THE MAIN DIAGONAL CRACK FOR BEAM 316

FIG. 4.31 FORCE IN THE STIRRUPS WHERE THEY CROSS THE MAIN DIAGONAL CRACK IN BEAM 316
of the diagonal crack is shown by the low force induced in stirrup four.

The shear force contribution of all steel across the main diagonal is plotted in Fig. 4.32. It has been assumed that the force in the main steel is tension along one diagonal and an equal compression along the other diagonal. In the first cycle it would appear that some load may be transferred by concrete mechanisms within the beam such as aggregate interlock, as the shear force carried by the stirrups is less than the load applied. This interlock action is possible as, although the diagonal crack is reasonably wide, large shear displacements occur creating the necessary bearing for load transfer. In the fifth cycle the apparent shear carried by the steel is greater than the shear applied to the section. This is probably caused by the force in the compression steel being less than that in the tension steel because of its interaction with the concrete. A reduction in the vertical component of the compressive force results and hence the shear resistance applied by the main steel overestimated.

4.3.4 Deformations

4.3.4.1 Rotations

The load rotation curves are shown in Fig. 4.33 for all thirteen cycles and in Fig. 4.34 for the first four elastic cycles plotted to a larger scale. The behaviour of the beam is reasonably symmetrical up to the first plastic load cycle when, because of more severe cracking, the left hand support became more flexible. The permanent displacements during the elastic cycles are very small, the maximum being only \(5 \times 10^{-4}\) radians at zero load at the end of the first cycle.
FIG. 4.33  LOAD ROTATION RELATIONSHIP FOR BEAM 316
FIG. 4.34 LOAD ROTATION RELATIONSHIP FOR THE FIRST FOUR CYCLES FOR BEAM 316
The theoretical stiffness of the uncracked section shown in Fig. 4.34 was calculated as outlined in Section 4.2.4.1. The agreement of the actual stiffness with the theoretical stiffness was very good. The theoretical stiffness for the cracked section, both considering and neglecting the effects of the compression steel were calculated using a formulation derived by Santhakumar. The stiffness calculated from theory underestimates the rotations produced at a load of 100 kips by about 12%. The calculated stiffnesses assume a certain area of concrete contained in the compression strut but this is rather arbitrary and if the area is less than that assumed the lower stiffness would result. The underestimation may also be due in part to slip and strain in the bar anchorages. A better comparison of stiffnesses can be gained in Section 4.3.4.2.

The yield rotation for Beam 316 is \( 4 \times 10^{-3} \) radians and the average rotation at the end of the thirteenth cycle is \( 49 \times 10^{-3} \) radians which corresponds to a ductility of 12.25 while sustaining theoretical ultimate load. Up to this stage the highest rotation would have been \( 17 \times 10^{-3} \) radians in the sixth cycle at a load of just under theoretical ultimate. This rotation corresponds to a ductility of 4.25.

If an elastic interstorey drift of \( \frac{H}{400} \) is desired (as prescribed by the N.Z. code), then the lowest load resisted is 92 kips in the fourth cycle which is close to the expected working load of \( 0.75 \frac{P_u}{P_u} = 96 \) kips. At failure of the beam the interstorey drift calculated as outlined in Section 4.2.2.1 is 7" over a storey height of 12' (this is a drift of \( \frac{H}{20.6} \)).

The maximum load sustained by the beam is 151.5 kips which is 23.1 kips above the theoretical ultimate. This
increase is mainly due to strain hardening of the reinforce-
ment. If the beam is assumed to deform symmetrically, and
the strain at the commencement of strain hardening is 15000
microstrains (found by test) then the rotation required is
16.7 x 10^{-3} radians. This corresponds quite well with
the rotation at which the ultimate load is exceeded. The
small increase of load over theoretical, especially
noticeable in cycle five, is due to the effects of dowel
action and aggregate interlock along the main diagonal crack.

4.3.4.2 The variation of stiffness.

The stiffnesses were obtained as previously
described and these have been plotted against the load ratio
in Fig. 4.35. After initial cracking in cycle one the
stiffness drops by 70%. Upon completion of the first
two cycles the stiffness rises slightly at moderate load
levels to a value close to that predicted by theory. The
similarity of the theoretical and actual stiffnesses is
good considering the actual stiffnesses can only be determined
to within an estimated 25%. The higher stiffnesses of
the third, fourth and fifth cycles may be explained if the
crack formation is considered. The initial cracking
occurs throughout the first two cycles and, in subsequent
cycles, the majority of the movement in the cracks occurs
at low load levels. This is characterised by the rising
stiffness with increasing load up to moderate load levels in
the third, fourth and fifth cycles.

The lower stiffness of the top left hand corner of
the beam can be seen when the stiffness of the odd numbered
cycles is compared with the higher stiffness of the even
numbered cycles. The lower stiffness is due to the fact
that the steel is taking most of the compression force in
this corner.
Theoretical Stiffness Relationships

- Uncracked Section
- Cracked Section with Compression Steel Considered
- Cracked Section without Compression Steel Considered

FIG. 4.35 THE VARIATION OF STIFFNESS WITH LOAD IN BEAM 316
4.3.4.3 The elongation of the beam.

The elongation of the beam is shown in Fig. 4.36 together with the theoretical elongations. The curves show quite marked changes of slope when cracking and yielding occur. After the first two cycles the beam has a permanent elongation of 0.02 in due to anchorage slip and residual stresses in the flexural steel. The sixth and seventh cycles are somewhat unusual as they have a decrease in elongation with increasing load. This results from a large contraction along the compression diagonal and a small elongation along the tension diagonal. This is expressed graphically in Fig. 4.37. The elongations for the last six cycles are very similar because the loads applied have not caused the tension steel to yield, this being the source of most of the permanent elongations. The elongation at failure was .953 in which would have a significant effect if the beam was incorporated in a coupled shear wall.

4.3.5 Crack Formation and Failure Mechanism.

The first significant cracks appeared at a load of 40 kips and propagated in a near vertical direction. These cracks are due to tension along the main diagonal steel, the holes for the Demac studs acting as crack initiators. By the end of the first cycle (Fig. 4.38) the diagonal crack is clearly developed and there are several steep flexural cracks as well as cracks due to tension in the main bars. In the second cycle a similar pattern is followed with the diagonal crack following the Demac studs (Fig. 4.39) at the completion of the fifth cycle major vertical cracks have developed at both supports of the beam (Fig. 4.40).

At the end of the thirteenth cycle there has been extensive cracking in the corners of the beam as well as severe crushing in the top left corner (Fig. 4.41).
FIG. 4.36 THE ELONGATION OF BEAM 316
FIG. 4.37  GRAPHICAL EXPLANATION OF BEAM CONTRACTION
FIG. 4.38  THE CRACK PATTERN FOR BEAM 316 AT A LOAD OF \(0.78p^*\) IN CYCLE ONE

FIG. 4.39  BEAM 316 AT \(0.80p^*\) IN CYCLE TWO
FIG. 4.40  BEAM 316 AT $1.08P_u$ IN CYCLE FIVE

FIG. 4.41  BEAM 316 AT FAILURE IN THE THIRTEENTH CYCLE WITH $P_1 = 1.18P_u$
The cracks, especially in the lower left corner, are quite large and there has been considerable shear displacement along them as may be seen by examining the holes for the Demec studs (Fig. 4.41).

The actual failure was due to the buckling of the three #8 bars in compression causing a local sideways displacement near the top left hand corner (Figs. 4.42 and 4.43).

4.4 Beam 317

4.4.1 Beam Properties

This beam contained the same main steel and stirrups as Beam 316 but incorporated ties around the main reinforcement in an effort to reduce the compression steel's susceptibility to buckling. The three #8 bars were surrounded by #2 bars in the tie corners as shown in Figs. 3.3 and 4.44. The #8 bars were welded to the ties to secure them within the cage. The four #7 bars were welded in the corners of #2 ties at 4'' centres. The reinforcement cage is shown in Figs. 4.44 and 3.3 and the component properties are given in Tables 3.1, 3.II and 3.III. The load sequence is presented in Fig. 4.46 and is comprised of four elastic cycles, four plastic cycles, two more elastic cycles and finally, three plastic cycles. The test was terminated at increment 150 in the thirteenth cycle when, because of large deformations, the test rig became unstable. In the plastic range a ductility factor of twelve was desired but, because these could not be easily calculated until after the test, the ductilities obtained at the time could only be estimated. This results in the apparent random nature of the rotations imposed, and found, in the load rotation curves.
FIG. 4.42  VIEW FROM ABOVE OF THE BUCKLED 
# 8 BARS IN THE TOP LEFT 
CORNER WITH LOOSE CONCRETE 
REMOVED

FIG. 4.43  A SIDE VIEW OF THE BUCKLING 
SHOWN ABOVE
FIG. 4.44  REINFORCEMENT CAGE FOR BEAM 317 SHOWING THE TIES AROUND THE MAIN STEEL

FIG. 4.45  REINFORCEMENT CAGE FOR BEAM 395
FIG. 4.46 LOAD SEQUENCE FOR BEAM 317

Theoretical Ultimate Load $P_u^* = 117$ kips

Increment No.

$P_u^* = 98.8$ kips

- Denotes a Rotation Reading
- Denotes a Steel Strain Reading
× Denotes a Crack Width Reading
In this test the main steel strains and beam deformations were recorded. Crack measurements were also made at various stages during the elastic cycles in an endeavour to provide an indication of the beam's serviceability. Several facets of this beam's behaviour were similar to those in Beam 316, thus, in general, only special points will be discussed.

4.4.2 Behaviour of the Flexural Reinforcement

4.4.2.1 The distribution of strains.

The distribution of strains along the instrumented diagonal steel is shown in Fig. 4.47. The pattern is very similar to that found for Beam 316 but the strains, especially near the ends, appear somewhat higher. The yield strain is reached at a load as low as 0.63 theoretical ultimate. This is due to the high local stresses induced in the corners by the horizontal #4 bars and will be described in more detail in Section 4.4.5.

The curves are quite symmetrical, as is to be expected, with the slight discrepancies between the strains in the two layers of bars being due to their different lever arms and irregularities in crack formation. There is quite a high residual strain at the end of the first cycle due to the incomplete closing of some of the cracks. Compression strains are again very low even at relatively high load ratios.

4.4.2.2 The steel force distribution.

The distribution of compressive and tensile steel forces are shown in Fig. 4.49. Here again the symmetrical behaviour of the steel may be seen. The force at the left hand end is slightly higher than at the right hand end as may be expected, as it is at this end that failure eventually occurred. Apart from the high forces
FIG. 4.47 DISTRIBUTION OF STRAINS IN THE FLEXURAL REINFORCEMENT IN BEAM 317
FIG. 4.48 DISTRIBUTION OF STEEL FORCE ALONG THE FLEXURAL REINFORCEMENT OF BEAM 317
near the ends the steel appears to behave much the same as in beam 316 although there is a less marked increase in force where the steel crosses the diagonal crack. This is because there is not one definite crack but several spread over about 10".

4.4.2.3 The position of the internal forces.

Assuming that only the main diagonal steel provides the resisting tensile forces then the compression resultant is as shown in Fig. 4.49. For all load ratios shown, the force is very close to the position of the compression steel but probably for lower load ratios the concrete could have a greater effect thus causing a more significant variation. The fact that the resultant lies close to the position of the compression steel means that the steel stresses increase almost linearly with load. The low lever arm at the ends is due to the high local steel stress caused by the failure mechanism described in Section 4.4.5.

4.4.3 Deformations

4.4.3.1 Rotations.

The load rotation curves are shown in Fig. 4.50 for all thirteen cycles and in Fig. 4.51 for the first four elastic cycles. The rotations shown are the average rotations of the two ends as these individually may differ considerably at high loads. In the elastic cycles the maximum difference is only 2 x 10^-4 radians but by the eighth cycle the difference is as large as 30 x 10^-3 radians. This is due to eccentricities in the line of action of the jack. When the jack was in position for the even numbered cycles no adjustment could be made in the line of action.
FIG. 4.49 THE POSITION OF THE INTERNAL COMPRESSION FORCE
FIG. 4.50. THE LOAD ROTATION RELATIONSHIP FOR
Theoretical Stiffness Relationships

- Uncracked Section
- Cracked Section with Compression Steel Considered
- Cracked Section without Compression Steel Considered

FIG. 4.51 THE LOAD ROTATION RELATIONSHIP FOR THE FIRST FOUR CYCLES FOR BEAM 317
and at one stage this was 9" to one side of the beam centre line. The result of this is that the moment at the left hand end is over twice that at the right hand end, hence the difference in rotations. For the odd numbered cycles the eccentricities were not as large because there was some latitude for jack alignment.

The first two cycles show considerable permanent deformations due mainly to local yielding of the steel at working loads, as outlined previously. The next two cycles show very little difference between the loading and unloading curves. This is because the movement occurred across existing cracks rather than new cracks being formed. As only one rotation reading was possible on the uncracked beam, the compression of theoretical and actual stiffnesses is not very meaningful. The increasing slope with load in the third and fourth cycles is due to the closure of cracks at low load levels when the concrete begins to sustain the compression.

The yield rotation is approximately $4 \times 10^{-3}$ radians which, with a rotation of $27.5 \times 10^{-3}$ radians in the fifth cycle, gives a ductility of 6.7 at a load ratio of 1.18. The maximum ductility obtained is 21.1 in the eighth cycle at a load ratio of 1.18. The interstorey drifts for the elastic cycles are close to the code prescribed $\frac{h}{400}$ thus no special precautions need be taken. The maximum drift in the plastic cycles is 8.9" over a 12' storey height - a drift of at least $\frac{h}{16.2}$. Some idea of the drift can be seen in Fig. 4.58 which shows the test rig at the final load increment.

The yield loads are higher than the theoretical loads because of truss action. This occurs because the horizontal #4 bars do not have sufficient anchorage at the end to
provide flexural resistance thus, these bars may be used for the alternative load resisting mechanism of truss action, as shown in Fig. 4.52. The compression struts of the truss, shown by the dotted lines, are provided by the concrete and the tension members (full lines) by the steel. The number of stirrups able to participate in the truss is governed by the cumulative horizontal force $H_i$ they induce in the #4 bars. Thus force $H_i$ cannot exceed the yield load of the horizontal bars.

Assuming a yield stress of 60 ksi, then the maximum tension per stirrup is 2.95 kips.

If $L_i$ = horizontal force induced in the #4 bars by stirrup $i$

$$L_i = \frac{V_i}{\tan \theta}$$

and $H_i$ = the cumulative horizontal force

$$H_i = L_i + H_{i-1}$$

and if the end stirrups are disregarded because of lack of anchorage for the #4 bars, then

$$H_2 = 6.95 \text{ kips}$$
$$H_3 = 12.67 \text{ kips}$$
$$H_4 = 16.89 \text{ kips}$$

The #4 bars can carry 17.3 kips in tension when yielding, thus $h_5$ and $h_6$ must be 0.41. This corresponds to a shear of 0.30 kips provided by the last two stirrups.

The total load resisted is then

$$V_2 + V_3 + V_4 + V_5 + V_6 = 5.9 + 5.9 + 5.9 + 0.9 + 0.9$$

$$= 18.6 \text{ kips}$$

There is some small amount of shear transfer by aggregate interlock and dowel action along the diagonal
FIG. 4.52 DIAGRAMMATIC REPRESENTATION OF TRUSS ACTION
crack but this is negligible. When the effects of truss action are taken into account the ultimate loads become 135.6 kips in the positive direction and 117.4 kips in the negative direction. The actual yield loads are 137.0 kips and 120.8 kips respectively which is a good correspondence. Subsequently in the eleventh cycle the beam had begun to suffer severe buckling in the top right hand corner reducing the load capacity to 30% theoretical ultimate and even further to 65% theoretical ultimate in the thirteenth cycle. When these buckled bars were in tension (in the even numbered cycles), there is a less severe reduction in load, the load remaining above $P_f$.

4.4.3.2 The variation of stiffness.

The stiffness is plotted against the load ratio in Fig. 4.53. As these curves are very sensitive to small reading errors, they should only be regarded as an indication of general trends. The first two elastic cycles show the effects of the cracking in the corners causing a reduction of stiffness at higher loads. In the next two elastic cycles the previously formed cracks are closing thus increasing the stiffness with increasing load. The last eight cycles are very similar as the load increases. This is caused by the majority of the load being resisted by the diagonal steel in which the Bauschinger effect is occurring.

4.4.3.3 The elongation of the beam.

The elongation of the beam with load is shown in Fig. 4.54. The curve is basically very similar to Fig. 4.36 for Beam 316, but the elongations are much larger after the sixth cycle. The sixth, seventh and eighth cycles all show a reduction in elongation with load as observed in the previous beam. The decrease in elongation
FIG. 4.53 THE VARIATION OF STIFFNESS WITH LOAD FOR BEAM 317
Theoretical with Compression Steel Considered
without

FIG. 4.54 THE ELONGATION OF BEAM 317
after the eighth cycle is because the tension steel is not
highly stressed (thus giving rise to only small elongations)
and the compression steel is buckling which causes a
contraction of the beam. The contraction becomes the
predominant movement from the ninth cycle onward. The
buckling failure in cycle thirteen causes the rapid reduction
in elongation as the compression diagonal shortens. The
maximum elongation of almost 1.4\" is very large and this
amount of movement would have a significant effect on the
coupled walls.

4.4.4 Crack Width Measurement and Crack Formation

The first significant cracks in the first cycle
appeared at a load of 30 kips and were near vertical
"flexural" type tension cracks at the top left and bottom
corners of the beam. These were accompanied by
cracks transversely across the line of the main diagonal
tension steel and the main vertical shear cracks at either
end. The main diagonal crack began to form at 40 kips
and by a load of 70 kips was well developed as shown in
Fig. 4.55. In the second cycle, there was a similar
pattern of formation with the major diagonal crack following
the line of the Demec studs at a load of 30 kips (see Fig.
4.56). By the end of the fifth cycle (Fig. 4.57), there
is severe cracking near the top left and bottom right
corners due to the first stage of the failure mechanism and
the diagonal cracking is very pronounced. In the sixth
cycle there is serious splitting along the line of the Demec
studs and buckling has caused a sideways movement of the
bottom face of the beam (Fig. 4.59). Cycles twelve and
thirteen caused a rapid deterioration in the beam with
extensive spalling due to buckling of the compression bars.
There is also severe shear displacements along the main
FIG. 4.55  THE CRACK PATTERN FOR BEAM 317 AT A LOAD OF $0.64p^* \overline{u}$ IN THE FIRST CYCLE

FIG. 4.56  BEAM 317 AT $0.71p^* \overline{u}$ IN THE SECOND CYCLE
FIG. 4.57  BEAM 317 AT 1.16P* IN THE FIFTH CYCLE

FIG. 4.58  THE TEST RIG SHOWING THE ANGLE OF THE END BLOCKS AT THE FAILURE OF BEAM 317
FIG. 4.59  A VIEW OF THE BOTTOM LEFT HAND CORNER OF BEAM 317 FROM BELOW AT A LOAD OF 1.21 P* IN THE SIXTH CYCLE

FIG. 4.60  THE CRACK PATTERN OF BEAM 317 AT A LOAD OF 1.1 P* IN THE SEVENTH CYCLE
vertical shear cracks especially at the left hand end. These displacements can be seen in Figs. 4.61 and 4.62 where the small circles on the shear cracks were once continuous. These would indicate shear displacements of approximately $\frac{1}{2}''$.

The crack widths measured on the West side of the beam are shown in Fig. 4.63 along with the locations at which the readings were taken. Consistency in the readings was hard to achieve because of the large local variations in the crack widths but the general indication is that after several cycles of high elastic loading, the permanent crack widths, when the load is released, are still below the .01" recommended in A.C.I. 318-63. The crack widths range up to 0.0175" under a working load but this is not excessive because of the short term nature of the loading.

4.4.5 **Failure Mechanism**

The failure of this beam was the result of two complementary causes. The first, due to bad detailing, resulted in the damage to the corner concrete. The two horizontal #4 bars had to be bent in to avoid the main steel and to enable them to be tied to it in order to hold the stirrup cage in place during concreting. Bearing pressures were exerted when the bars were stressed causing cracking and eventual spalling of the corner concrete (see Fig. 4.64). High local stresses were induced in the main steel by these bearing pressures and the steel reached yield at a load as low as 63% of theoretical ultimate.

In the fifth cycle severe cracking occurred in the top left hand and bottom right hand corners (Fig. 4.57) and the vertical shear crack at the support was up to $\frac{1}{2}''$ wide. In the sixth cycle the four #7 bars were placed in compression and buckled slightly near the corners where the concrete had severely cracked. The cracking reduces the beams
FIG. 4.61  BEAM 317 AT 0.81 P* IN CYCLE ELEVEN

FIG. 4.62  BEAM 317 AT 0.68 P* IN CYCLE THIRTEEN
FIG. 4.63 CRACK WIDTH MEASUREMENTS FOR BEAM 317
resistance to buckling as it reduces the stabilizing effect of the concrete. In the seventh cycle (Fig. 4.60) spalling began in the lower left hand corner accompanied by slight buckling of the three #3 bars. At the end of the eleventh cycle the concrete in the corners is severely cracked and spalled (Fig. 4.61) but transverse buckling displacements are still not large. The separation along the diagonal crack is over \( \frac{1}{2} \)" at the end of the twelfth cycle and the four #7 bars have buckled upward and outward by over 1". This was accompanied by severe cracking of the concrete confined between the bars. The concrete around the three #8 bars in the bottom left hand corner had also begun to spall.

The thirteenth and final cycle caused the three #8 bars to buckle toward the east. The confined concrete around the bars had been lost so their restraint to buckling was very low (see Fig. 4.62). At failure the #8 bars had buckled about two inches and the #7 bars, even though they were in tension, were still buckled by about an inch.

4.5 Beam 395

4.5.1 Beam Properties

Beam 395 has the same general layout as Beam 317 shown in Fig. 3.3, but has a depth of 39" rather than 31". The component properties are given in Tables 3.1, 3.11 and 3.111. The load sequence for all eleven cycles is shown in Fig. 4.65. The first four were elastic cycles, these being followed by four plastic cycles. The last of these cycles is not truly a plastic cycle but rather a cycle in which buckling was the factor that governed the load applied. The ninth and tenth cycles were elastic cycles and, on the eleventh cycle, the beam was taken to destruction because the buckling failure had become so
advanced. The reinforcement cage is shown in Fig. 4.45, the main diagonal instrumentation consisting of two more Demec points than in the other three beams. As in Beam 317, the main steel strains, deformations and crack widths have been recorded.

4.5.2 Behaviour of the Flexural Reinforcement

4.5.2.1 The distribution of strains.

The distribution of strains along the main diagonal steel is shown in Fig. 4.66. The pattern is generally very similar to that for Beam 317 but it has a more marked increase in strain near the centre of the beam where the steel is crossed by the main diagonal crack. As the crack formed at a relatively low load, the concrete can take more tension between the supports and the crack hence reducing the tension necessary in the steel. At higher loads a more uniform strain occurs as the tension contribution of the concrete becomes negligible. The steel has yielded at a load of 0.75 theoretical ultimate across the outer gauge lengths because of high local stresses accompanying the start of a similar failure mechanism to that found in the previous beam. The bars under compression show a fairly regular strain pattern with a tension near the centre. This is due to the diagonal crack causing permanent displacements and accompanying residual tensions where it crosses the main steel.

4.5.2.2 The steel force distribution.

Fig. 4.67 shows the distribution of the steel force, both compressive and tensile, along the diagonal steel. The onset of diagonal cracking at a load of between .25 and .42 theoretical ultimate is seen as a change in curve profile across gauge lengths 15 and 16. Repeated elastic loading
FIG. 4.66 DISTRIBUTION OF STRAINS ALONG THE FLEXURAL REINFORCEMENT FOR BEAM 395
FIG. 4.67 DISTRIBUTION OF STEEL FORCE ALONG THE FLEXURAL REINFORCEMENT FOR BEAM 395
appears to have had a more pronounced effect than in previous beams as there is a marked increase in the steel force between the first and fifth cycles at a load of .75 theoretical ultimate. An unusual feature in the second cycle is the tension over most of the beam at a load of .40 theoretical ultimate.

4.5.2.3 The position of the internal forces.

The position of the compression centroid has been plotted in Fig. 4.68 for various load ratios in the first and fifth cycles. At low load levels the position of the resultant would indicate that the magnitude of the tension force is higher than expected. If there was some tension force contributed by the concrete the tension resultant would move towards the centre of the beam and hence move the compression resultant more towards the compression steel at low loads. When Fig. 4.73 for Beam 395 is compared with Fig. 4.50 for Beam 317, it is found that for similar loads the steel forces are approximately the same. This is somewhat unusual as the tension force for Beam 317 acts as a smaller lever arm thus the force should be greater for a similar load. By a load of 1.09 theoretical ultimate the compression resultant passes almost directly along the compression bars as expected. The only significant result of this behaviour is the higher steel stresses at loads up to working load and hence, probably more extensive cracking. The reduction in lever arm at the ends is due to the high local steel stresses accompanying the first stage of the failure mechanism described in Section 4.5.5.

4.5.3 Deformations

4.5.3.1 Rotations.
FIG. 4.68 THE POSITION OF THE INTERNAL COMPRESSION FORCE IN BEAM 395
The load rotation curve for all load cycles is shown in Fig. 4.69 and is shown to a larger scale in Fig. 4.70 for the first four load cycles. The bias towards positive rotations, which was entirely unintentional, was due to the large permanent rotations applied in the first plastic cycle.

The first four elastic cycles are very similar in shape to those of previous beams but the rotations are correspondingly smaller because of the higher stiffness of the section. The permanent no load displacements are low, the maximum being only 5 x 10^-4 radians. The eighth, ninth and eleventh cycles exhibit the characteristic buckling pattern. This consists of a region of high initial stiffness as cracks along the new compression diagonal close followed by a lower gradually increasing stiffness, as the previously buckled steel straightens. This may be followed by a decrease in the stiffness (as in cycles ten and eleven) as the compression steel buckles. The second set of cycles at a load of 75% P_u (nine and ten), have much higher rotations than those in previous beams, because of the extensive buckling already present at this stage.

The yield rotation is approximately 4 x 10^-3 radians. The rotation at the end of the fifth cycle is 45 x 10^-3 thus a ductility factor of 11.25 was obtained at a load of 1.22 theoretical ultimate. In the sixth cycle a ductility factor of 10.5 was obtained and in the seventh cycle a factor of 12.25 was achieved. The interstorey drifts for the elastic cycles are less than \( \frac{H}{300} \) as may be expected because of the stiffer section. During the plastic cycles the corresponding drifts were in the region of \( \frac{H}{72} \).

The load achieved in the fifth cycle was 146.3 kips which is quite in excess of the theoretical load of 119.5 kips.
FIG. 4.69  THE LOAD ROTATION CURVE FOR BEAM 395
Theoretical Stiffness Relationships

- Uncracked Section
- Cracked Section with Compression Steel Considered
- Cracked Section Neglecting Compression Steel

FIG. 4.70 THE LOAD ROTATION RELATIONSHIP FOR THE FIRST FOUR CYCLES FOR BEAM 395
This increase is due to the truss action described in 4.4.3.1. In this case all the stirrups participate in the action and carry 29.5 kips of the load. This would give a revised theoretical ultimate of 149 kips which corresponds very well with the observed ultimate load. The fifth cycle caused extensive damage in the lower right and upper left corners of the beam as well as opening up large vertical shear cracks at the supports (see Fig. 4.75). This damage caused a reduction in the maximum load obtained in the sixth cycle by destroying the ends of the concrete compression struts for the truss action. This reduces the load taken by truss action to a negligible level. The ultimate load in the seventh and eleventh cycles are only reduced slightly with load reversal.

4.5.3.2 The variation of stiffness.

The plot of load ratio to stiffness is shown in Fig. 4.71. The curves are very similar to those for Beam 316 although there is a greater discrepancy between the theoretical and actual stiffnesses. The values for the uncracked stiffness appear close but as only one reading was taken in this region no significance could be placed on this. There is a large reduction of stiffness after the first plastic cycle but this is probably due to its severity. The buckling behaviour can be seen in the last four cycles.

4.5.3.3 The elongation of the beam.

The elongation of the beam is plotted in Fig. 4.72. The curve shows that the buckling commenced in the sixth cycle accompanied by a decrease in elongation. In the seventh cycle the compression strut was still reasonably intact so the beam lengthens as the tension steel straightens. The eighth and tenth cycles show large contractions which accompany the buckling of the compression strut. The ninth
Theoretical Stiffness Relationships

- Uncracked Section
- Cracked Section with Compression Steel Considered
- Cracked Section without Compression Steel Considered

FIG. 4.71 THE VARIATION OF STIFFNESS WITH LOAD FOR BEAM 395
FIG. 4.72 THE ELONGATION OF BEAM 395

- Theoretical with compression steel considered
- without

Load Ratio $P / P_0$

Elongation (ins.)

Cycle No.

Buckling of Compn. Steel
and eleventh cycles show an increase in length as the
tension bars straighten accompanied by very little
contraction of the compression strut. The beam fails at
the end of the eleventh cycle when the #3 bars buckled and
the beam rapidly shortens. The elongations occurring
during a particular cycle are quite large and would therefore
have a significant effect on the coupling beam-shear wall
interaction. The elongations of this beam are the highest
of the series, up to 1.6". A considerable amount of the
elongation occurs across the vertical shear cracks at the
supports, as may be seen in Fig. 4.76.

4.5.4 Crack Width Measurement and Crack Formation.

The sequence of crack formation is shown in
Figs. 4.73 to 4.78. The crack formation was very similar
to that in Beam 317, the vertical shear crack at the support
forming at a load of 30 kips and the main diagonal crack at
a load of 50 kips in the first cycle (Fig. 4.73). In the
second cycle the main diagonal crack formed at a load of
only 20 kips, the load being considerably lower than that
in the first cycle because of the efficacy of Demco points
(Fig. 4.74).

At the end of the fifth cycle the top right and bottom
left corners are damaged and the main diagonal and shear
cracks are very wide (Fig. 4.75). The diagonal cracking
is not as extensive as in Beam 317, there being only about
three major cracks. The development of cracks past this
stage will be outlined in the following section.

Crack width measurements for selected points on the
West side of the beam have been reproduced in Fig. 4.79.
The vertical shear cracks at locations eleven and seventeen
are the most significant with widths up to 0.025 ins. under
elastic loading. These two cracks also have the highest
FIG. 4.73 THE CRACK PATTERN FOR BEAM 395 AT A LOAD OF 0.42P_u IN CYCLE ONE.

FIG. 4.74 BEAM 395 AT A LOAD OF 0.72P_u IN CYCLE TWO.
FIG. 4.75 BEAM 395 AT A LOAD OF 1.22P
IN CYCLE FIVE

FIG. 4.76 BEAM 395 AT A LOAD OF 0.64P
IN THE TENTH CYCLE
FIG. 4.78 THE LEFT HAND SUPPORT OF BEAM 395 FROM THE EAST AT A LOAD OF 0.96\(P^*\) IN CYCLE ELEVEN

FIG. 4.77 THE BUCKLING OF THE BOTTOM LEFT HAND CORNER OF BEAM 395 AT 0.72\(P^*\) IN CYCLE EIGHT
crack widths when there is no load on the beam, the greatest being 0.076 ins. These crack widths could have been reduced with proper secondary, horizontal reinforcement as no steel crosses the main vertical shear cracks between points sixteen and eighteen.

4.3.5 The Failure Mechanism

The failure mechanism for this beam was the same as that for Beam 317 but the beam deteriorated much more rapidly.

At the end of the fifth cycle the concrete began to fall from the lower left hand corner and was severely cracked at the top right hand corner. This cracking and spalling was due to the high bearing pressures exerted by the horizontal reinforcement as described in Section 4.4.5. Reversal of the load caused the four #7 bars to begin to buckle in the lower left hand corners accompanied by spalling of the surrounding concrete. The seventh cycle, which induced tension in the #7 bars, caused further spalling in the lower left hand corner and destruction of the concrete between the bars. When the load was reversed for cycle eight the concrete between the #7 bars fell out, reducing the buckling restraint. As a result of this a lateral movement of about three inches occurred in the lower left corner causing more spalling (Fig. 4.77). At this stage there has been slight crushing in the top left hand corner and a small amount of spalling in the top right corner. The only buckling of any significance at this stage had occurred in the #7 bars in the bottom left hand corner.

The ninth and tenth cycles caused extensive cracking and spalling in the left hand section of the beam (Fig. 4.76). This damage is a result of tension, which causes cracking,
FIG. 4.79 CRACK WIDTH MEASUREMENTS FOR BEAM 395
followed by buckling which causes the previously cracked concrete to spall. The final cycle, which induced compression in the three #8 bars, caused most of the concrete cracked in cycle ten to fall out, as may be seen in Fig. 4.77. At moderate loads in this cycle, the concrete began to spall away from the compression bars in the top left corner. This caused a reduction in buckling resistance and resulted finally in the #8 bars moving rapidly sideways at a load of 115 kips. Basketing reinforcement could have reduced the spalling of the concrete but it was not included in these tests as it disguises the behaviour of the main steel.
CHAPTER FIVE

A COMPARISON OF THE BEAMS

5.1 Flexural Reinforcement

Because of the vastly different behaviour of the flexural reinforcement in the conventionally and diagonally reinforced beams, a comparison of all four beams would serve little purpose. However, the behaviour of the flexural steel in the three diagonally reinforced beams will be compared.

In all cases the three beams show relatively similar strains in the two layers of reinforcement with a slight discrepancy toward the ends of the beam where the bars at a greater lever arm have slightly higher strains.

The best way to compare the flexural load resistance of the beams is to study the variation of the lever arm and relate this to the steel stresses. The lever arm in Beam 316 is large at low load ratios and indicates a line of thrust approximately along the line of the compression reinforcement at loads approaching ultimate. In complete opposition to this behaviour is Beam 395, in which the lever arm is small at low load ratios. However, in this beam too the line of thrust approached the line of the compression reinforcement at approximately ultimate load. Intermediate between these two cases is Beam 317 in which the compression resultant is almost along the line of the compression steel for all load ratios.

This behaviour could be explained if, in Beam 316, there is only a small, highly stressed, area of concrete in compression. On the other hand in Beam 395, there should be a large area of concrete lightly stressed. In Beam 317
there should exist a situation between the two extremes. Unfortunately the Bernoulli-Navier concept of linear strain distribution does not apply to these beams. Furthermore concrete strains were not measured. The only guide available with respect to concrete compression strains, is to look at the stresses in the steel in cycle two when it is in compression. This can only be a guide as the bars have been under considerable tension in the previous cycle and their behaviour cannot accurately reflect that of the concrete. It is found that the steel in Beam 395 is the lowest stressed and that in Beam 316 the highest so it would appear that the above hypothesis, with respect to concrete compression stresses and the extension of compression zones, is correct.

Although this behaviour just outlined, affects the steel stress at loads up to ultimate, the load resisting mechanism at yield appears to be provided by forces acting along the line of the two sets of diagonal bars.

5.2 Shear Reinforcement

The increase of stirrup stresses with load has been observed for Beams 315 and 316 and reflect the difference in the mode of shear resistance of the two types of beams. In both beams the stresses are low until diagonal cracking occurs, a behaviour predicted by the A.C.I. code\(^5\). After cracking, the stress in the stirrups in Beam 315 approach the code predicted values quite accurately. There is one exception to this. Stirrup nine, is not so highly stressed as, in this region, the stirrup elongation is restrained by the end block. Yield is reached in stirrup eight and nine over most of their length at a load of approximately 90% theoretical ultimate (see Fig. 4.6).

In Beam 316 however, the stresses remain low except
where the stirrup is crossed by the diagonal crack in which case the stress may be a little higher (Fig. 4.30). The middle stirrup (number four) shows a scatter of stresses as there are a series of diagonal cracks spread along about fifteen inches of its depth. The gradual increase of stress in the stirrups situated towards the centre of the beam occurs because the end block's restraint of transverse beam expansion is less towards the middle. Larger elongations of the stirrups can occur, thus the stresses are higher. At a load of about 110% theoretical ultimate the stresses in the stirrups rapidly rise to yield at sections crossing the diagonal crack. The stirrups in Beam 315 reach yield at a load 10% below theoretical ultimate, whereas yield in the stirrups in Beam 316 occurs at a load 10% above theoretical ultimate.

This difference in behaviour is a result of the different roles played by the stirrups in load resistance. In Beam 315 the stirrups are an integral part in the load resisting mechanism, carrying a large proportion of the shear applied to the section. In contrast the stirrups in Beam 316 do not aid in shear resistance until approximately theoretical ultimate load when the load exceeds the shear capacity of the unreinforced section plus the diagonal steel.

The diagonal cracking appears to have had very different effects on the stirrups in the two types of beam. In Beam 315 the effects are spread over the length of the stirrup whereas in Beam 316 the effect is very localised. The reason for this can be seen by comparing Figs. 4.39 and 4.44. Beam 315 has a network of diagonal cracks whereas Beam 316 has a definite zone of cracking. This behaviour is typical of a separation failure as found by Paulay in Beam 311.

The shape of the stirrup force profiles along the beam (Figs. 4.7 and 4.31) show the same general pattern of
increasing force towards the centre line. The exception to this is the low stress in the centre stirrup in Beam 316 caused by the restraining effect of the main steel on the width of the diagonal crack. In Beam 315 at the maximum load in the fifth cycle, only three stirrups have yielded. These were in the area where the shear slip failure was imminent. Four stirrups had yielded at the same stage of loading in Beam 316, but in this case it was caused by the separation across the main diagonal crack.

5.3 Deformations

Rather than study the load rotation curves which have been described previously, this section will compare the energy absorption properties of all four beams. The importance of a large ductility factor has been outlined before but the beams must also retain much of their ability to transfer load. If they do not then the shear force transferred between the coupled shear walls is small and the efficiency of the structure is lost. When the load capacity of the beams fail the shear walls tend to act as two independent cantilevers rather than a coupled system in which the moment is resisted by the combined action of both walls.

Fig. 5.1 shows the load (expressed as a percentage of theoretical ultimate) plotted against the cumulative ductilities obtained by the beams at loads above that level. Beams 313 and 314 from a previous series of tests by Paulay have also been included for the sake of comparison. If the ductilities achieved at theoretical ultimate are considered, Beams 317 and 395 exhibit superior characteristics. It must however be considered that Beam 316 had not completely failed (see Fig. 4.33) as at least one further cycle could have been made at a load approaching ultimate with probably a large
FIG. 5.1 CUMULATIVE DUCTILITIES AT PARTICULAR LOADS FOR ALL THE BEAMS

Conventionally Reinforced Beams
Diagonally

Indicates a higher ductility may have been achieved if test taken to complete, controlled failure.
ductility factor obtained. Moreover, if the loading had been applied to this beam so that moderate ductilities were obtained in several cycles rather than just a large ductility in one cycle, then a more favourable curve may have been achieved. When considering the curves for the beams not tested in this project, it should be borne in mind that the beams were tested using a constant load rather than a constant displacement loading system. The result of this is that the post-elastic behaviour could not be followed and the actual ductilities would be higher than those shown.

To quantify the energy absorption characteristics, an equivalent viscous damping constant \( h_e \) is found

\[
h_e = \frac{1}{2\pi} \frac{W}{W_p}
\]

where \( W \) is the energy loss equal to the area of the load rotation loop and \( W_p \) is the potential energy

\[
W_p = F_{\text{max}} \theta_{\text{max}}
\]

If the equation for \( h_e \) is studied it is seen that it compares the actual energy absorbed with the theoretical energy absorbed assuming a bilinear load rotation curve (Fig. 4.10). Fig. 5.2 shows \( h_e \) plotted against the cumulative amount of energy absorbed at that particular value of \( h_e \) or greater. This means that the further towards the top right corner of the graph a particular curve lies the greater and more efficient the energy absorption of that beam. Efficient energy absorption is achieved when the load rotation curve is close to the bilinear response thus, for a certain amount of energy to be absorbed at a fixed load, the minimum deformation is required. The curve for Beam 316 again does not do it justice as the beam
FIG. 5.2  CUMULATIVE POST-ELASTIC ENERGY AT A PARTICULAR VALUE OF $\Delta W/W$. 
had not fully failed when the test was terminated.

Beam 317 is the most efficient beam followed by Beams 395, 316, 315, 394, 313, 393 and 314 in that order. The different loading system for the last four beams should be remembered as the values for energy absorbed using the constant displacement loading system would have been slightly higher than those shown.

The interstorey drifts at working load for the four beams are

\[
\begin{align*}
\text{Beam 315}: & \quad \frac{H}{260} \\
\text{Beam 316}: & \quad \frac{H}{370} \\
\text{Beam 317}: & \quad \frac{H}{380} \\
\text{Beam 395}: & \quad \frac{H}{490}
\end{align*}
\]

The latter three are quite satisfactory, being close to the recommended \( \frac{H}{400} \) but the drift for Beam 315 may necessitate special detailing of non-structural elements within the building.

The elongations for the beams are quite similar for the elastic cycles but for the plastic cycles they range from 0.2" maximum for Beam 315 to 1.6" maximum for Beam 395. The high elongations in the plastic cycles in Beam 317 and 395 could be due in part, to the fact that the jack applying the load was not vertical. This would induce a horizontal tension in the beams of approximately 20 kips maximum which could easily cause the higher elongations. This tension force would have had a detrimental effect on the behaviour of the beams.

The stiffness relationships for the four beams are very similar with approximately the same stiffness for the three 31" beams during the elastic cycles. There is a slightly
higher stiffness for the 39" beam as would be expected. For all the diagonally reinforced beams the actual stiffness of the cracked sections are less than the theoretical predictions, especially after the plastic cycles. This discrepancy is mainly due to the variation of the size of the compression strut from that assumed in the derivation of the theoretical relationship. The theoretical stiffness for the cracked section is about 40% of the theoretical value for the uncracked section. The actual values of stiffness for the cracked sections range about 60% and 95% of their theoretical stiffness.

5.4 The Failure Mechanism

These have been fully discussed for each beam in the previous chapter. In summarising, the failure of each beam was

Beam 315 - The destruction of the concrete at the supports under cyclic loading, resulting in a sliding shear failure.

Beam 1315 - A sliding shear failure along the joint between the original and repaired sections of the beam.

Beam 316 - Buckling of the three #3 bars while under compression near the top left corner.

Beams 317 and 395 - Extensive cracking near the corners due to the previously described action of the two #4 secondary bars. This caused a reduction in buckling resistance of the section and both sets of diagonal bars which eventually buckled sideways near the support.

The mechanism of the last three beams is far more desirable than that for the first two because of the far greater amounts of energy absorbed before failure. The
risk of a buckling failure is likely to be greatly reduced in the wider beams found in actual shear walls. In these the bars can be spaced further apart thus increasing the moment of inertia of the steel arrangement resisting buckling.

A buckling failure caused by static loading results in a dramatic collapse of the structure or the affected component. However, in a seismic type of dynamic loading, in which large deformations rather than loads are imposed, the instability as encountered in these beams, is essentially of a ductile nature. Figs. 4.50 and 4.69 show that, in spite of large buckling in the compression bars, considerable forces are sustained by the beams. It is to be noted that after reversal of the load, the buckled bars tend to straighten and are again capable of sustaining their full share of the tensile load. The compression bars, straightened in the previous cycle, then receive a reasonable force before they buckle and the load reduces.
CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

This series of tests demonstrated the marked improvement in behaviour of a coupling beam obtained by using a diagonal system of reinforcement. Not only were large ductilities obtained but these were accompanied by less degradation of load capacity under repeated cyclic loading. Unfortunately, it is still doubtful whether the beams could resist several cycles of high intensity loading with a ductility factor greater than twelve. They may prove adequate if the period of the wall system, with cracked coupling beams, becomes sufficiently high to markedly reduce the base shear on the structure. These large ductilities are accompanied by high interstorey drifts. A ductility of twelve in Beam 316 gave rise to a drift of \( \frac{H}{24} \) and the maximum drift in all beams was close to \( \frac{H}{16} \) where \( H \) is the storey height. This drift amounts to 9" in a 12' storey height. The drift is cumulative over the building height and it neglects elongations within the walls which add to the displacement. Thus the movement at the top of the wall, when maximum ductility is achieved within the coupling beams, would be quite alarming.

In all three diagonally reinforced beams tested, a stability failure rather than a material failure occurred. If the buckling failure could be eliminated an improvement in performance would probably result. The test beams were only 6" wide and the end blocks provided very little, if any, torsional resistance. In a real structure much wider beams would be used and considerable restraint provided by the
shear walls and the floor diaphragms. An example of this restraint is that provided to the top region of the beams above lift doors.

To reduce the possibility of a buckling failure, several precautions should be considered in the design of real beams.

i) The detailing of secondary reinforcement should be such that this steel in no way causes high local stresses in the concrete. In particular, high bond and bearing stresses caused by bends and anchorages should be avoided within the beams.

ii) The beams should be of a reasonable width (preferably a similar thickness to the wall). The main flexural steel should then be placed as close as possible to the faces of the beam to produce the highest possible moment of resistance. The buckling load is proportional to the moment of inertia, i.e. the distance between the bars squared. The amount of concrete in the beam is approximately equal to the cover plus the distance between the bars. If the spacing between the bars is doubled then the buckling resistance is increased fourfold but the volume of concrete is only increased by a factor of less than two.

iii) The flexural steel should be surrounded by ties or perhaps a spiral at the closest practical pitch. This is especially important near the beam supports in order to contain the concrete. This confined concrete is then available to act with the steel in resisting buckling. Even with extensive yielding in the steel it was found that the concrete remained reasonably intact. It was not until buckling occurred, that this concrete began to be lost.

iv) Sufficient 'basketing' reinforcement (stirrups and horizontal steel) should be provided to ensure a fine
mesh of cracks and to retain shattered chunks of concrete at the disaster stage. The stirrups should be of sufficient strength to reduce the possibility of a large separation crack along the diagonal.

There are a number of problems associated with diagonally reinforced beams which should be considered in design of shear wall structures.

i) The compaction of the concrete within the formwork may be difficult especially in a narrow deep beam. This problem arises because the ties around the main flexural steel make it difficult to compact the concrete near the bottom with an immersion type vibrator. This could be alleviated to some degree if the spacing of the ties was increased near the centre of the beam. In this area the degree of cracking is not as high as near the supports and hence the buckling restraint provided by the concrete is greater (see Figs. 4.41, 4.64 and 4.76). Also at the centre of the beam tension steel provides some lateral restraint for the compression strut.

ii) Unless the effect of all horizontal and diagonal steel is considered (including basketing reinforcement), the load induced in the shear walls by the beams, may be greater than that theoretically predicted. This would affect the ultimate load capacity of the walls as one wall would be subject to a higher compression than theoretically predicted. More importantly, the other wall would suffer a higher tension force which could seriously affect its behaviour.

iii) Accurate theoretical predictions for elongation and stiffness are difficult to make. These depend greatly on the properties assumed for the compression strut and elongations especially are sensitive to such second order effects as anchorage slips and crack formation.

The relative costs of the two types of coupling beam
were compared and it was found that these were very similar. When the improvement in behaviour of the wall system is taken into account, the diagonally reinforced beams would be far more preferable in buildings where seismic resistance is to be provided.
### TABLE 3.I  CONCRETE MIX PROPORTIONS

<table>
<thead>
<tr>
<th>CEMENT CONTENT</th>
<th>1bs./cu.yd.</th>
<th>500</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\frac{3}{4}) in. AGGREGATE</td>
<td>- &quot; -</td>
<td>1060</td>
</tr>
<tr>
<td>(\frac{1}{2}) in. AGGREGATE</td>
<td>- &quot; -</td>
<td>705</td>
</tr>
<tr>
<td>SAND</td>
<td>- &quot; -</td>
<td>1620</td>
</tr>
<tr>
<td>WATER CEMENT RATIO</td>
<td>-</td>
<td>.620</td>
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### TABLE 3.II  CONCRETE PROPERTIES

<table>
<thead>
<tr>
<th>BEAM NUMBER</th>
<th>-</th>
<th>-</th>
<th>315</th>
<th>316</th>
<th>317</th>
<th>395</th>
</tr>
</thead>
<tbody>
<tr>
<td>AGE WHEN TESTED</td>
<td>-</td>
<td>days</td>
<td>31</td>
<td>34</td>
<td>246</td>
<td>267</td>
</tr>
<tr>
<td>CUBE STRENGTH</td>
<td>(f_{cu}')</td>
<td>psi</td>
<td>6900</td>
<td>6073</td>
<td>-</td>
<td>-</td>
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<tr>
<td>CYLINDER STRENGTH</td>
<td>(f_c')</td>
<td>psi</td>
<td>5500</td>
<td>4825</td>
<td>7348</td>
<td>5150</td>
</tr>
<tr>
<td>STRENGTH RATIO</td>
<td>(f_c'/f_{cu}')</td>
<td>-</td>
<td>.796</td>
<td>.795</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MODULUS OF RUPTURE</td>
<td>(f_t')</td>
<td>-</td>
<td>660</td>
<td>.711</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CERTIFIED STRENGTH</td>
<td>(f_{c, cert}')</td>
<td>psi</td>
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<td>3250</td>
<td>3250</td>
<td>3250</td>
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<tr>
<td>DATE Poured</td>
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<td>-</td>
<td>10/11/69</td>
<td>13/3/70</td>
<td>4/2/71</td>
<td>24/2/71</td>
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<tr>
<td>DATE TESTED</td>
<td>-</td>
<td>-</td>
<td>10/12/69</td>
<td>16/4/70</td>
<td>7/10/71</td>
<td>17/11/71</td>
</tr>
</tbody>
</table>

NOTES:  
(1) 6" x 6" x 6" standard cubes  
(2) 12" x 6" standard cylinder  
(3) 3" x 3" x 12" standard prism
<table>
<thead>
<tr>
<th>BEAM NUMBER</th>
<th>315</th>
<th>316</th>
<th>317</th>
<th>395</th>
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<tbody>
<tr>
<td>FLEXURAL REINFORCEMENT</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NOMINAL SIZE</td>
<td>-</td>
<td>-</td>
<td>#7</td>
<td>#8</td>
</tr>
<tr>
<td>ACTUAL AREA</td>
<td>-</td>
<td>in²</td>
<td>.590</td>
<td>.740</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>$f_y$ ksi</td>
<td>44.7</td>
<td>43.0</td>
<td>41.8</td>
</tr>
<tr>
<td>ULTIMATE STRENGTH</td>
<td>$f_u$ ksi</td>
<td>67.8</td>
<td>68.0</td>
<td>66.2</td>
</tr>
<tr>
<td>SECONDARY REINFORCEMENT</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NOMINAL SIZE</td>
<td>-</td>
<td>-</td>
<td>#5</td>
<td>#4</td>
</tr>
<tr>
<td>ACTUAL AREA</td>
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<td>in²</td>
<td>.291</td>
<td>.105</td>
</tr>
<tr>
<td>YIELD STRENGTH</td>
<td>$f_y$ ksi</td>
<td>46.5</td>
<td>53.3</td>
<td>46.8</td>
</tr>
<tr>
<td>ULTIMATE STRENGTH</td>
<td>$f_u$ ksi</td>
<td>71.5</td>
<td>77.7</td>
<td>68.4</td>
</tr>
</tbody>
</table>

NOTES: (1) Plain untested bars
# Indicates nominal bar size in multiples of $\frac{1}{8}$ inch