DUCTILITY OF RECTANGULAR REINFORCED CONCRETE

COLUMNS WITH AXIAL LOAD

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

WAYNE DOUGLAS GILL

February 1979
ABSTRACT

An experimental investigation into the post-elastic ductile behaviour of reinforced concrete columns designed according to the requirements of the Draft New Zealand Code of Practice for the Design of Concrete Structures was undertaken. Four full size column sections were built and subjected to a static cyclic lateral load sequence over a range of axial compressive load levels.

Ductility requirements of reinforced concrete columns are discussed and the results and analyses of experimental results are presented. Results are presented in the form of measured lateral load - displacement and moment - curvature relationships, curvature and longitudinal bar strain profiles, and transverse confining steel strains. Analysis of results includes the comparison of measured ductilities, plastic hinge lengths, concrete spalling and maximum compression strains, and stress-strain curves for confined concrete with those obtained by existing analytical methods.

Conclusions are made about the effectiveness of the confinement provided by the draft code recommendations.
ACKNOWLEDGEMENTS

The research for this report was carried out in the Department of Civil Engineering, University of Canterbury, under the overall guidance of its Head, Professor R. Park.

The project was supervised by Professor R. Park and Dr. M. J. N. Priestly, whose advice and guidance is gratefully acknowledged.

I wish to thank the assistance provided by Mr. N. W. Prebble, Technical Officer, and the Technical staff of the Civil Engineering Department. Special thanks are due to Mr. G. Hill, Mr. G. Clark, Mr. A. Bell, and Mr. A. Stokes for their advice and assistance during construction and testing of the specimens.

The financial assistance provided by the New Zealand Ministry of Works and Development is gratefully acknowledged.

Finally, I thank my fiancee, Liz, for her encouragement and understanding.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>i</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>ii</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>x</td>
</tr>
<tr>
<td>NOTATION</td>
<td>xi</td>
</tr>
<tr>
<td><strong>SECTION ONE</strong></td>
<td></td>
</tr>
<tr>
<td>1.1 General</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Aim</td>
<td>1</td>
</tr>
<tr>
<td>1.3 Scope</td>
<td>1</td>
</tr>
<tr>
<td>1.4 Review of Previous Research</td>
<td>2</td>
</tr>
<tr>
<td>1.5 Format</td>
<td>3</td>
</tr>
<tr>
<td><strong>SECTION TWO</strong></td>
<td></td>
</tr>
<tr>
<td>2.1 Summary</td>
<td>5</td>
</tr>
<tr>
<td>2.2 Ductility Demand Criteria</td>
<td>5</td>
</tr>
<tr>
<td>2.3 Ductility Demands of Seismic Loading</td>
<td>7</td>
</tr>
<tr>
<td>2.4 Behaviour of Columns in Multistorey Frames</td>
<td>7</td>
</tr>
<tr>
<td>2.5 Ductility Demands for Columns</td>
<td>10</td>
</tr>
<tr>
<td>2.6 Historical Review of Ductility Provisions</td>
<td>11</td>
</tr>
<tr>
<td>2.7 DZ 3701 Requirements for Achieving Column Ductility</td>
<td>14</td>
</tr>
<tr>
<td><strong>SECTION THREE</strong></td>
<td></td>
</tr>
<tr>
<td>3.1 Summary</td>
<td>17</td>
</tr>
<tr>
<td>3.2 Specimen Size Criteria</td>
<td>1</td>
</tr>
<tr>
<td>3.3 Design of Specimens</td>
<td></td>
</tr>
<tr>
<td>3.3.1 Column Designation</td>
<td>18</td>
</tr>
<tr>
<td>3.3.2 Longitudinal Reinforcement</td>
<td>20</td>
</tr>
<tr>
<td>3.3.3 Transverse Reinforcement</td>
<td>21</td>
</tr>
<tr>
<td>3.3.4 Beam Stub</td>
<td>26</td>
</tr>
<tr>
<td>3.3.5 Anchorage</td>
<td>27</td>
</tr>
</tbody>
</table>
3.4 Material Properties
   3.4.1 Steel 27
   3.4.2 Concrete 29

3.5 Construction 34

SECTION FOUR - INSTRUMENTATION AND TESTING PROCEDURE
4.1 Summary 37
4.2 Instrumentation
   4.2.1 Strains and Curvatures 37
   4.2.2 Horizontal Displacements 42
   4.2.3 Load Measurement 42

4.3 Testing Procedure
   4.3.1 Test Equipment 42
   4.3.2 Specimen Installation 44
   4.3.3 Testing Proper 46

SECTION FIVE - TEST RESULTS FROM SPECIMENS
5.1 Summary 50
5.2 Specimen One
   5.2.1 Yield Displacement and Curvature 51
   5.2.2 General Description of the Behaviour of the Column During Testing 52
   5.2.3 Measured Load-Displacement and Moment-Curvature Relationships 55
   5.2.4 Measured Curvature Profile 55
   5.2.5 Measured Bar Strain Profiles 60
   5.2.6 Measured Confining Steel Strains 60

5.3 Specimen Two
   5.3.1 Yield Displacement and Curvature 63
   5.3.2 General Description of the Behaviour of the Column During Testing 63
   5.3.3 Measured Load-Displacement and Moment-Curvature Relationships 67
   5.3.4 Measured Curvature Profile 67
   5.3.5 Measured Bar Strain Profiles 67
   5.3.6 Measured Confining Steel Strains 74

5.4 Specimen Three
   5.4.1 Yield Displacement and Curvature 75
5.4.2 General Description of the Behaviour of the Column During Testing 75
5.4.3 Measured Load-Displacement and Moment-Curvature Relationships 77
5.4.4 Measured Curvature Profile 82
5.4.5 Measured Bar Strain Profiles 82
5.4.6 Measured Confining Steel Strains 82

5.5 Specimen Four
5.5.1 Yield Displacement and Curvature 86
5.5.2 General Description of the Behaviour of the Column During Testing 86
5.5.3 Measured Load-Displacement and Moment-Curvature Relationships 88
5.5.4 Measured Curvature Profile 91
5.5.5 Measured Bar Strain Profile 91
5.5.6 Measured Confining Steel Strains 91

SECTION SIX ANALYSIS OF TEST RESULTS FROM SPECIMENS

6.1 Summary 99

6.2 Specimen One
6.2.1 Comparison with EWD Estimation of Ductility 99
6.2.2 Plastic Hinge Length Estimation 100
6.2.3 Concrete Spalling Strain 103
6.2.4 Maximum Measured Concrete Compression Strain 103
6.2.5 Comparison of Analytical Confinement models 104

6.3 Specimen Two
6.3.1 Comparison with EWD Estimation of Ductility 109
6.3.2 Plastic Hinge Length Estimation 109
6.3.3 Concrete Spalling Strain 109
6.3.4 Maximum Measured Concrete Compression Strain 109
6.3.5 Comparison of Analytical Confinement models 111

6.4 Specimen Three
6.4.1 Comparison with EWD Estimation of Ductility 114
6.4.2 Plastic Hinge Length Estimation 114
6.4.3 Concrete Spalling Strain 116
6.4.4 Maximum Measured Concrete Compression Strain 116
6.4.5 Comparison of Analytical Confinement Models 116

6.5 Specimen Four
6.5.1 Comparison with MWD Estimation of Ductility 118
6.5.2 Plastic Hinge Length Estimation 118
6.5.3 Concrete Spalling Strain 118
6.5.4 Maximum Measured Concrete Compression Strain 120
6.5.5 Comparison of Analytical Confinement Models 120

SECTION SEVEN DISCUSSION, CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

7.1 Discussion of Results and Analysis
7.1.1 Measured Load-Displacement and Moment-Curvature Relationship 122
7.1.2 Measured Curvature and Longitudinal Bar Strain Profiles 123
7.1.3 CDP 810/A Ductility Calculations 123
7.1.4 Plastic Hinge Length Estimation 123
7.1.5 Concrete Spalling Strains and Maximum Measured Compression Strains 124
7.1.6 Comparison of Analytical Confined Concrete Stress-Strain Models 124

7.2 Conclusions 125
7.3 Recommendations for Future Research 126

APPENDIX A REFERENCES 127
APPENDIX B 132
APPENDIX C 136
LIST OF FIGURES

1.1 Moment-Curvature Curves for Aoyama's Specimen A-2 with Axial Load and Flexure 4
2.1 'First Yield' Displacement Criteria 6
2.2 Bending moments in Lower Columns of a 12 Storey Frame responding Nonlinearly to El Centro Earthquake 1940 N-S Component 8
2.3 Frame Structure under Seismic Loading and Possible mechanisms 9
2.4 Idealised Stress-Strain Curve for Concrete Confined by Rectangular Hoops 12 12
3.1 Moment Pattern 18
3.2 Column Specimen Dimensions 19
3.3 Longitudinal Steel Arrangement - all Specimens 20
3.4 Tie Arrangements 21
3.5 Reinforcing Cage Plans 24
3.6 Reinforcing Cage Plans 25
3.7 Beam Stub Steel 26
3.8 End Anchorage Details 27
3.9 Completed reinforcing Cages 28
3.10 Stress-Strain Curve for GR 380 D24 Bar (Longitudinal Bars - all Specimens) 30
3.11 Stress-Strain Curve for GR 275 R10 Bar (Ties for Specimens One and Three) 31
3.12 Stress-Strain Curve for GR 275 R12 Bar (Ties for Specimen Two) 52
3.13 Stress-Strain Curve for GR 275 -R12 Bar (Ties for Specimen Four) 33
3.14 Specimen Mould 36
4.1 Dial Gauge Frame Support 37
4.2 Dial Gauge Locations 39
4.3 Strain and Curvature Determination 40
4.4 Typical Strain Gauge Locations 41
4.5 Load Application 43
4.6 Load Bearings 45
4.7 Specimen Installation 47
4.8 Loading Criterion 46
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>Stress-Strain Curve for Confined Concrete [14]</td>
<td>107</td>
</tr>
<tr>
<td>6.4</td>
<td>Concrete Stress-Strain Curves for Specimen One</td>
<td>108</td>
</tr>
<tr>
<td>6.5</td>
<td>Concrete Stress-Strain Curves - Specimen wo</td>
<td>112</td>
</tr>
<tr>
<td>6.6</td>
<td>Concrete Stress-Strain Curves - Specimen Three</td>
<td>117</td>
</tr>
<tr>
<td>6.7</td>
<td>Concrete Stress-Strain Curves - Specimen Four</td>
<td>121</td>
</tr>
</tbody>
</table>
# List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Column Specimen Hoop Steel Requirements</td>
<td>22</td>
</tr>
<tr>
<td>3.2</td>
<td>Shear Steel Requirements</td>
<td>26</td>
</tr>
<tr>
<td>3.3</td>
<td>Steel Properties</td>
<td>29</td>
</tr>
<tr>
<td>3.4</td>
<td>Concrete Strengths</td>
<td>34</td>
</tr>
<tr>
<td>5.1</td>
<td>Comparison of Theoretical Ultimate and Experimental Maximum Moments - Specimen One</td>
<td>58</td>
</tr>
<tr>
<td>5.2</td>
<td>Comparison of Theoretical Ultimate and Experimental Maximum Moments - Specimen Two</td>
<td>70</td>
</tr>
<tr>
<td>5.3</td>
<td>Comparison of Theoretical Ultimate and Experimental Maximum Moments - Specimen Three</td>
<td>81</td>
</tr>
<tr>
<td>5.4</td>
<td>Comparison of Theoretical Ultimate and Experimental Maximum Moments - Specimen Four</td>
<td>94</td>
</tr>
<tr>
<td>6.1</td>
<td>Ductility Calculations - Specimen One</td>
<td>101</td>
</tr>
<tr>
<td>6.2</td>
<td>Plastic Hinge Length Calculations - Specimen One</td>
<td>102</td>
</tr>
<tr>
<td>6.3</td>
<td>Ultimate Compression Strain</td>
<td>103</td>
</tr>
<tr>
<td>6.4</td>
<td>Comparison of Analytical Confinement Models</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specimen One</td>
<td>109</td>
</tr>
<tr>
<td>6.5</td>
<td>Ductility Calculations - Specimen Two</td>
<td>110</td>
</tr>
<tr>
<td>6.6</td>
<td>Plastic Hinge Length Calculations - Specimen Two</td>
<td>111</td>
</tr>
<tr>
<td>6.7</td>
<td>Ultimate Compression Strain</td>
<td>111</td>
</tr>
<tr>
<td>6.8</td>
<td>Comparison of Analytical Confinement Models</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specimen Two</td>
<td>113</td>
</tr>
<tr>
<td>6.9</td>
<td>Ductility Calculations - Specimen Three</td>
<td>115</td>
</tr>
<tr>
<td>6.10</td>
<td>Plastic Hinge Length Calculations - Specimen Three</td>
<td>116</td>
</tr>
<tr>
<td>6.11</td>
<td>Ultimate Compression Strain</td>
<td>116</td>
</tr>
<tr>
<td>6.12</td>
<td>Comparison of Analytical Confinement Models</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specimen Three</td>
<td>116</td>
</tr>
<tr>
<td>6.13</td>
<td>Ductility Calculations - Specimen Four</td>
<td>119</td>
</tr>
<tr>
<td>6.14</td>
<td>Plastic Hinge Length Calculations - Specimen Four</td>
<td>118</td>
</tr>
<tr>
<td>6.15</td>
<td>Ultimate Compression Strain</td>
<td>120</td>
</tr>
<tr>
<td>6.16</td>
<td>Comparison of Analytical Confinement Models</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specimen Four</td>
<td>120</td>
</tr>
</tbody>
</table>
NOTATION

\( A_c \) = area of concrete section confined by hoops
\( A_g \) = gross area of section
\( A_s \) = area of longitudinal steel
\( A_{sh} \) = area of confining steel
\( A_v \) = area of shear steel
\( b \) or \( b' \) = core dimension
\( b_w \) = width of full section
\( c \) = neutral axis depth at ultimate
\( c_l \) = centre to centre distance between the longitudinal bars
\( C_c \) = total concrete compressive force
\( C_s \) = total force in longitudinal bars in compression
\( d \) = effective depth of section
\( d' \) = nominal diameter of longitudinal bar
\( d'' \) = nominal diameter of transverse reinforcement
\( \delta \) or \( \Delta \) = displacement
\( \delta_p \) = plastic displacement
\( \delta_y \) or \( \Delta_y \) = yield displacement
\( \Delta_u \) = ultimate displacement
\( E_c \) = modulus of elasticity of concrete
\( \varepsilon \) = strain
\( \varepsilon_c \) = concrete strain
\( \varepsilon_{cu} \) = ultimate concrete strain
\( \varepsilon_{sh} \) = strain hardening strain
\( \varepsilon_{s1}, \varepsilon_{s2} \) = minimum and maximum average longitudinal strains corresponding to the maximum stress in concrete
\( \varepsilon_y \) = steel yield strain
\( \varepsilon_{o}, \varepsilon_{oo} \) = concrete strain at maximum stress level
\( \varepsilon_{20c} \) = concrete strain at 20% maximum stress \([12]\)
\( \varepsilon_{50c} \) = concrete strain at 50% maximum stress for confined concrete \([12]\)
\( \varepsilon_{50h} \) = \( \varepsilon_{50c} - \varepsilon_{50u} \) \([12]\)
\( \varepsilon_{50u} \) = concrete strain at 50% maximum stress for unconfined concrete \([12]\)
\( f'_c \) = stress in concrete
\( f'_c \) = concrete cylinder strength
\( f'_s \) = stress in transverse steel
\( f_y \) or \( f'_y \) = yield stress in steel
\( h \) = depth of full section
\( h'' \) = core dimension
\( I_{cr} \) = cracked section modulus
\( k \) = maximum stress ratio \([13]\)
\( k_1, k_2 \) = coefficients used in calculating plastic hinge length
\( k_s \) = ratio of the strength of confined concrete to the strength of plain concrete \([14]\)
\( k_u \) = ratio \( c/t' \) at limit condition
\( k_y \) = ratio \( c/t' \) at yield condition
\( L \) = column length to point of contraflexure
\( L_p \) = plastic hinge length
\( m \) = cover concrete ratio for confined concrete section
\( m \) = moment
\( \mu_{u, theo} \) = theoretical ultimate moment
\( \mu \) = overall structure ductility
\( n \) = number of longitudinal bars in the specimen
\( N_A \) = neutral axis
\( P \) = lateral load
\( P_e \) = axial compressive load
\( P_{occ} \) = 0.85 \( f'_c (A_c - A_s) \)
\( P_{u, theo} \) = theoretical ultimate lateral load
\( \rho_s \) = volumetric transverse steel ratio
\( \rho_t \) = longitudinal steel percentage
\( \phi \) = capacity reduction factor
\( \phi \) = curvature
\( \phi_u \) = section curvature at ultimate
\( \phi_y \) = section curvature at yield
\( s \) or \( s_h \) = spacing of hoop sets
\( t' \) = core dimension
\( T \) = total force in longitudinal bars in tension
\( \theta_p \) = plastic rotation
\[ \theta_y \] = member end rotation at first yield
\[ \theta_u \] = maximum member end rotation
\[ v_c \] = nominal permissible shear stress carried by concrete
\[ v_u \] = nominal total design shear stress
\[ V_u \] = total applied design shear force
\[ Z \] = slope of falling branch of concrete stress strain curve [12]
SECTION ONE

INTRODUCTION

1.1 GENERAL

In New Zealand the code of practice for the design of reinforced concrete structures has traditionally been based on ACI publications. NZS 3101P [1] was based mainly on ACI 318-63 [2] with reference made to the 1968 SEAOC recommendations [3] for seismic design. With the realisation that much of the code was significantly out of date, designers in New Zealand have been using more recent editions of the ACI or SEAOC codes.

During 1973 a draft code for comment, DZ 3101 [4], that would be appropriate for use with the design loadings code NZS 4203: 1976 [5] was published. The "non seismic" chapters have been largely based on ACI 318-71 [6] and ACI 318-77 [7] but with extensive and significant alterations to suit New Zealand requirements. The additional seismic requirements are entirely new and markedly different from Appendix A of the ACI code.

1.2 AIM

The aim of the study reported herein was an experimental examination of the provisions of Chapter 17 of DZ 3101, "members subjected to flexure and axial load - additional seismic requirements", in an attempt to gain a measure of the post-elastic ductile performance of reinforced concrete columns.

1.3 SCOPE

Four full size column sections were designed to the
provisions of Chapter 17 of NZ 3101 for different axial load levels, nominally 0.15, 0.25, 0.40 and 0.55 $f'_c \text{ Ag}$, and subjected to a cyclic lateral load test sequence.

1.4 REVIEW OF PREVIOUS RESEARCH

Theoretical and experimental investigations into the ductile behaviour of reinforced concrete sections have been conducted at the University of Canterbury and elsewhere. A review, not claimed to be fully comprehensive, of previous related work is given in this section.

Fundamental to the response of structural concrete to seismic loadings are the stress-strain characteristics of steel and concrete. Cyclic load tests have been conducted on typical samples of reinforcing bar [8,9,10] and idealisations proposed to model the stress-strain curves. Idealisations for the stress-strain characteristics of confined concrete have also been proposed [9,11-14]. Based on available experimental data for monotonic behaviour, limited mainly to concrete specimens containing simple square hoops or circular spirals, the idealisations have also been assumed to act as the envelope for cyclic behaviour. (Details of Kent and Park's idealisations [12] are given in Section 2.6.)

Considerable work has been done on the investigation of circular reinforced concrete columns and bridge piers under seismic loading [11, 15-18]. Although concerned with circular sections the same general principles are applicable to the present study. Theoretical investigations have included the study of the effect of a number of parameters on the response of single stem and double stem piers to severe earthquakes and the formulation of methods for predicting the moment-curvature behaviour of circular confined reinforced concrete columns. Experimental programmes, with particular attention given to the load-deflection and moment-curvature relationships, have been conducted and the measured response compared with the theoretical predictions.
Investigation of the ductile behaviour of rectangular reinforced concrete column sections has been done [19 - 24]. These have been predominantly theoretical investigations of the effect of confinement from rectangular hoops on the curvature ductility under monotonic flexure. Approaches to the detailing of rectangular reinforced concrete columns for ductility were presented, based on the moment-curvature response of reinforced concrete sections derived from the stress-strain curves of concrete confined by rectangular hoops and of steel including strain hardening. The axial load level, quantity of longitudinal steel, the section properties and the material strengths were among the factors considered. Few investigators have attempted to determine the behaviour of reinforced concrete column sections under cyclic loading. Examples of theoretical investigations into the behaviour of columns under cyclic loading are Park, Kent and Sampson [24], Aoyama [25], and Bertero and Bresler [26]. FIG. 1.1 shows a comparison of Aoyama’s experimental results for a member subjected to axial load and cyclically varying bending moment and Park, Kent and Sampson’s theoretical approach for the same member.

1.5 FORMAT

Section Two contains a review of ductility requirements and demands of reinforced concrete column sections imposed by seismic loading, including basic definitions and codified requirements for achieving adequate column ductility.

The detailed design and construction of the specimens used in this experimental investigation are presented in Section Three, along with the properties of the materials used while Section Four is concerned with the instrumentation and testing procedure employed.
FIG. 1.1 Moment-curvature curves for Aoyama's specimen A - 2 with axial load and flexure [24].

The instrumentation is related to the information sought. Results from the experimental test are presented in Section Five, and analysis of test results in Section Six. Final discussion and conclusions are summarised in Section Seven and aspects of this study requiring further investigation are noted.

A bibliography appears in Appendix A, a record of the plans of the testing reaction frames in Appendix B, and a calculation summary of analysis details from Section Six is contained in Appendix C.
SECTION TWO

DUCTILITY REQUIREMENTS OF REINFORCED CONCRETE COLUMN SECTIONS

2.1 SUMMARY

Current seismic design for frame structures relies on energy absorption and dissipation by post-elastic deformation of plastic hinges to enable the structures to survive major earthquakes. The necessity for a ductility approach is clearly demonstrated by performance of reinforced concrete structures in recent earthquakes, in particular Tokachi-oki (1968) [27], where high member strength was insufficient to avoid total collapse of many structures, and San Fernando (1971) [28], where inadequate detailing for ductility was apparent.

In this section the ductility demands on columns imposed by seismic loading are discussed, displacement and curvature ductility factors as used in this text are defined and codified requirements for achieving adequate column ductility are reviewed.

2.2 DUCTILITY DEMAND CRITERIA

It should be noted that the definition of ductility factor can be expressed in terms of displacements, rotations or curvatures. This has lead to confusion in some designers minds when interpreting code ductility requirements.

The displacement ductility factor, $\mu$, is most commonly referred to in seismic design loading codes. This is defined as the ratio of lateral deflection of some representative point of the structure (centre of mass or top of the building) at ultimate, to lateral deflection at first yield.

\[
\mu = \frac{\Delta u}{\Delta y}
\]
However, the information needed by the designer in assessing member performance concerns the required member section behaviour expressed by the curvature ductility factor \( \phi_u/\phi_y \), where \( \phi_u \) = maximum curvature at the section and \( \phi_y \) = curvature at the section at first yield. Some dynamic analyses have determined the rotational ductility factor of members \( \Theta_u/\Theta_y \), where \( \Theta_u \) = maximum rotation of end of member and \( \Theta_y \) = rotation at end of member at first yield.

Problems arise in defining first yield deformation for calculating ductility factors when the load or moment-deformation curve is not elastoplastic. In reinforced concrete columns this may occur due to longitudinal bars at different depths in the section yielding at different load levels. The "first yield" criteria adopted for this study is that suggested by Park [29].

The first yield displacement is taken as the displacement calculated for the column assuming cracked-section elastic behaviour up to the theoretical ultimate strength of the column in the first load application, as shown in FIG. 2.1. A similar definition can be adopted for the first yield rotation and curvature.

FIG. 2.1 "First yield" displacement criteria.
2.3 **DUCTILITY DEMANDS OF SEISMIC LOADING**

Analysis of the records of severe earthquakes has shown that the lateral loads recommended by design standards for earthquake loading are considerably less than the theoretical elastic response inertia loads. However, it is well known that many structures designed to the lateral loads of design standards have survived severe earthquakes, this being attributed to the ability of ductile structures to absorb energy by post-elastic deformation. Hence the present use of lateral loads for seismic design that are less than the elastic response inertia loads can only be justified if structures have sufficient ductility. NZS 4203: 1976 [5] states "all seismic resisting systems, regardless of building height, designed to the seismic loadings of this standard must have ductility". (Clause C 3.2) This statement is qualified by providing for minor exemptions such as small buildings designed in accordance with other provisions of that code.

In an attempt to quantify the general statement about ductility NZS 4203: 1976 [5] gives the following approximate criteria: "the building as a whole should be capable of deflecting laterally in at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the code loadings calculated on the assumption of appropriate plastic hinges is at least four times that at first yield, without the horizontal load carrying capacity of the building being reduced by more than 20%." (Clause C 3.2)

2.4 **BEHAVIOUR OF COLUMNS IN MULTISTOREY FRAMES**

The New Zealand loadings code [5] favours the strong column - weak beam design concept which aims at dissipating seismic energy in a flexural mode at a significant number of beam hinges, rather than in column hinges. Some codes, for example the seismic provisions of
ACI 318 - 77 [2], attempt to satisfy this philosophy by merely requiring that at beam-column connections the sum of the moment strengths of the columns exceed the sum of the moment strengths of the beams along each principal plane at the connection. In practice this may provide insufficient protection as the formation of column hinges is enhanced by four factors:

a) The actual beam steel strength at high curvatures will be higher than the specified yield strength and this strength will be further enhanced by strain hardening. Therefore the beam input moment may be considerably higher than that calculated using the specified or dependable yield strength.

b) Higher mode effects may cause points of contraflexure to occur well away from the mid height of columns at various stages during an earthquake giving a largely different moment distribution than that assumed in design and placing a high demand on particular columns. Bending moment distributions in columns such as in FIG. 2.2 are possible.

![Diagram of moment distribution in columns](image)

**FIG. 2.2** Bending moments in lower columns of a 12 storey frame responding nonlinearly to El Centro earthquake 1940 N-S component [30].
In the extreme if the point of contraflexure lies outside the column height the strength of one column section needs to exceed the sum of the input beam moments. This required column strength to prevent plastic hinges forming is much greater than the ACI 318 - 77 requirement.

b) In design it is customary to consider seismic loading to act in the direction of the principal axes of the structure and in one direction at the time. However, biaxial effects can cause yielding of the beams in both directions simultaneously and will generally reduce the flexural strength of the column. Typically the flexural strength of a square column for bending about a diagonal may be 15% less than the flexural strength for uniaxial bending. Therefore concurrent earthquake loading may cause the columns to yield before the beams unless columns are strengthened to take this effect into account.

d) The necessity for base hinges to complete the static collapse mechanism of the desired beam sidesway mechanism. FIG. 2.3 shows a frame and possible mechanisms which could form due to flexural yielding and formation of plastic hinges.

![Frame](Image)

**FIG. 2.3** Frame structure under seismic loading and possible mechanisms.
The undesirable column sidesway mechanism which can make very large curvature ductility demands on the plastic hinges of the critical storey, particularly for tall buildings, is included for comparison.

The difficulty of preventing plastic hinges from forming in columns is such that some column yielding must be considered to be inevitable. A design procedure developed by Paulay [31] aimed at giving reasonable protection against column yielding (except at base columns), taking into account beam overstrength, higher mode and concurrent effects, is being recommended in New Zealand for determining column actions.

The possibility of yielding occurring at the column ends due to the effects discussed makes it important to ensure that columns are capable of behaving in a ductile manner. Hence for reinforced (and prestressed) concrete columns adequate transverse steel in the form of hoops or spirals should be present at the potential plastic hinge regions at the column ends. These provide for concrete confinement by arching between hoops and also laterally restrain the longitudinal steel against buckling. This combined action helps to ensure ductile concrete behaviour.

2.5 DUCTILITY DEMANDS FOR COLUMNS

Since post elastic deformations are concentrated in regions of plastic hinging, there can be significant differences between the magnitude of the displacement and curvature ductility factors. NZS 4203: 1976 [5] calls for an overall displacement ductility factor, $\Delta u/\Delta y$, of 4 which will require a curvature ductility factor of a significantly higher magnitude. The actual relationship between the ductility factor and the ductility of sections is complicated because, for most designs, yielding will not occur at the critical sections at the same load. Park [32] has developed an approxi-
mate analysis, based on simplifying approximations, that will give an indication of the order of section ductility required to achieve a given displacement ductility factor. Norton [20] suggests that a reasonable criterion for adequate performance would be to maintain a moment capacity, at $\phi_u/\phi_y = 16$, of 80-90% of the peak moment, avoiding brittle shear and anchorage failures.

2.6 **HISTORICAL REVIEW OF DUCTILITY PROVISIONS**

The general principles of ductile seismic behaviour have been recognised since the 1960's. Recommendations for amounts of special transverse steel in the ends of columns from the SEAOC and ACI Codes were incorporated in the New Zealand Ministry of Works Code of Practice for the design of Public Buildings, P.W. 81/10/1 [33] and later in the New Zealand Code of Practice for Reinforced Concrete Design NZS 3101 P: 1970 [1]. The provisions were based on the strength criteria for spirally reinforced columns that the column capacity for axial load after the spalling of the cover concrete should at least equal the capacity before spalling. It was assumed that rectangular hoops were only 50% as effective as circular spirals for confining reinforcement. The philosophy of maintaining the axial load strength of columns after the spalling of cover concrete does not properly relate to the detailing requirements of adequate plastic rotation capacity of eccentrically loaded column sections. A more logical approach for the determination of the required content of transverse steel would be based on ensuring a satisfactory moment-curvature relationship and would include as variables the level of axial load on the column, the longitudinal steel ratio, the proportion of the column section confined, the stress-strain curve of the longitudinal steel, and the stress-strain curve of the confined concrete as a function of the amount of confining steel.
mate analysis, based on simplifying approximations, that will give an indication of the order of section ductility required to achieve a given displacement ductility factor. Norton [20] suggests that a reasonable criterion for adequate performance would be to maintain a moment capacity, at $\phi_M/\phi_y = 16$, of 80-90% of the peak moment, avoiding brittle shear and anchorage failures.

2.6 HISTORICAL REVIEW OF DUCTILITY PROVISIONS

The general principles of ductile seismic behaviour have been recognised since the 1960's. Recommendations for amounts of special transverse steel in the ends of columns from the SEAOC and ACI Codes were incorporated in the New Zealand Ministry of Works Code of Practice for the design of Public Buildings, P.W. 81/10/1 [33] and later in the New Zealand Code of Practice for Reinforced Concrete Design NZS 3101 P: 1970 [1]. The provisions were based on the strength criteria for spirally reinforced columns that the column capacity for axial load after the spalling of the cover concrete should at least equal the capacity before spalling. It was assumed that rectangular hoops were only 50% as effective as circular spirals for confining reinforcement. The philosophy of maintaining the axial load strength of columns after the spalling of cover concrete does not properly relate to the detailing requirements of adequate plastic rotation capacity of eccentrically loaded column sections. A more logical approach for the determination of the required content of transverse steel would be based on ensuring a satisfactory moment-curvature relationship and would include as variables the level of axial load on the column, the longitudinal steel ratio, the proportion of the column section confined, the stress-strain curve of the longitudinal steel, and the stress-strain curve of the confined concrete as a function of the amount of confining steel.
Moment-curvature analyses of rectangular column sections for monotonic flexure have been conducted (see Section 1.4) using idealised stress-strain curves for steel and concrete. The stress-strain curve for the steel included the effect of strain hardening. The stress-strain curve for the concrete included the effect of the confining steel. FIG. 2.4 shows the assumed curve for concrete confined by rectangular hoops proposed by Kent and Park [12] on the basis of analysis of existing experimental data.

![Stress-strain curve diagram](image)

**FIG. 2.4 Idealised stress-strain curve for concrete confined by rectangular hoops. [12]**

The characteristics of the curve are described below.

**Region AB:** $0 \leq \varepsilon_c \leq 0.002$. A second degree parabola is assumed, following the curve for unconfined concrete.

$$f_c = f'_c \left[ \frac{2 \varepsilon_c}{0.002} - \left( \frac{\varepsilon_c}{0.002} \right)^2 \right]$$  \hspace{1cm} (1)

**Region BC:** $0.002 \leq \varepsilon_c \leq \varepsilon_{20c}$

$$f_c = f'_c \left[ 1 - Z \left( \varepsilon_c - 0.002 \right) \right]$$  \hspace{1cm} (2)
where \[ Z = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - 0.002} \]  \tag{3}

\[ \varepsilon_{50u} = \frac{3 + 0.29 f'_c}{145 f'_c - 1000} \]  \tag{4}

\[ \varepsilon_{50h} = \frac{3}{4} \rho_s \sqrt{\frac{h''}{s_h}} \]  \tag{5}

where \( f'_c \) is in MPa, \( s \) is the ratio of volume of hoops to volume of concrete core, \( h'' \) is the width of the confined core and \( s_h \) is the spacing of hoops.

Region CD: \( \varepsilon_c > \varepsilon_{20c} \). It is assumed that concrete can sustain a compressive stress of 0.2 \( f'_c \) indefinitely.

Recently published work by Valienas, Bertero and Popov [13] indicates that increased concrete stress and corresponding enhanced concrete strain is possible.

That is, the stress-strain curve of Kent and Park may be conservative. However, tests by the PCA [34] have indicated that Kent and Park's curve gives a good estimate of the actual behaviour, although slightly conservative. More tests are necessary to establish the stress-strain curve for concrete confined by various arrangements of overlapping hoops and supplementary cross ties.

The moment-curvature curves were calculated using the idealised stress-strain curves, assuming that plane sections remain plane and satisfying the requirements of strain compatibility and equilibrium. The cover concrete was assumed to become ineffective at a compressive strain greater than 0.004.

The approximate analyses referred to have shown that the equations for transverse steel content recom-
mended in the 1973 SEAOC Code [35] are generally conservative for moderate axial load levels but not conservative at high load levels. In New Zealand the SEAOC equations have been modified to take axial load level into account.

The SEAOC provisions as contained in the 1973 version of that code are given below.

The total cross-sectional area \( A_{sh} \) of rectangular hoop reinforcement shall not be less than

\[
A_{sh} = 0.3 \, s_h \, h'' \, \left[ \frac{A_g}{A_C} - 1 \right] \, \frac{f'c}{f_{yh}} \tag{6}
\]

or

\[
A_{sh} = 0.12 \, s_h \, h'' \, \frac{f'c}{f_{yh}} \tag{7}
\]

whichever is greater.

2.7 **DZR 3101 REQUIREMENTS FOR ACHIEVING COLUMN DUCTILITY**

The draft New Zealand Concrete Code [6] requires that column members shall have special transverse reinforcement in the form of spiral or hoop reinforcement, with or without supplementary cross ties, in potential plastic hinge regions considered to be at the ends of members above and below moment resisting connections over a length from the faces of the connection of not less than:

a) the longer member section dimension in the case of a rectangular section or the diameter in the case of a circular section,

b) one-sixth of the clear height of the member,

c) 450 mm.

Where rectangular hoop reinforcement is used, including overlapping hoops and supplementary cross ties,
in potential plastic hinge regions the total area of hoop bars and supplementary cross ties in each of the principal directions of the section where \( Pe \leq 0.6 \ f'_c \ A_E \) shall be not less than

\[
A_{sh} = 0.3 \ s_h \ h'' \left[ \frac{A_E}{A_c} - 1 \right] \ \frac{f'_c}{f_{yh}} \ (0.33 + 1.67 \ \frac{Pe}{f'_c A_E}) \quad (8)
\]

or \( A_{sh} = 0.12 \ s_h \ h'' \ \frac{f'_c}{f_{yh}} \ (0.33 + 1.67 \ \frac{Pe}{f'_c A_E}) \) \quad (9)

whichever is greater, where \( Pe \) shall not be taken as less than \( 0.1 \ f'_c A_E \).

Limits are placed on the upper axial load level and the longitudinal reinforcement ratio and distribution. The spacing of hoop sets shall not exceed the smaller of:

a) one-fifth of the smaller section dimension,

b) 150 mm,

c) six times the diameter of the longitudinal bar to be restrained.

The spacing of the hoop sets need not be less than 75 mm.

Each longitudinal reinforcing bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than \( 135^\circ \) or by a supplementary cross tie, except that the following bars are exempt from this requirement:

a) Bars or bundles of bars which lie between two laterally supported bars or bundle of bars supported by the same hoop where the distance between the laterally supported bars or bundle of bars does not exceed 200 mm between centres.

b) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inner face of hoops.
The recommended equation (8) results in the following amounts of transverse steel being placed as a percentage of the amount recommended in the 1973 SEAOC Code.

<table>
<thead>
<tr>
<th>$Pe \frac{f'<em>c A_g}{A</em>{sh}}$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>% of SEAOC $A_{sh}$</td>
<td>50</td>
<td>66</td>
<td>83</td>
<td>100</td>
<td>117</td>
<td>133</td>
</tr>
</tbody>
</table>

Note that $0.6 f'_c A_g$ is the upper limit of column axial load allowed.
SECTION THREE

DESIGN AND CONSTRUCTION OF SPECIMENS

3.1 SUMMARY

Chapter 17 of DZ 3101 [4] covers the design of ductile members subjected to flexure and axial loads in the range 0.1 to 0.6 \( f' \)c A_g. The initial aim of this project was to test four specimens designed to DZ 3101 requirements and subjected to cyclic lateral loads in conjunction with a range of axial load levels, nominally 0.15, 0.25, 0.40 and 0.55 \( f' \)c A_g.

This section describes the design of the four specimens, the properties of the materials used and the construction method employed.

3.2 SPECIMEN SIZE CRITERIA

With the installation in 1978 of the 10MN capacity servo-hydraulically controlled Dartec Universal Testing Machine at the University of Canterbury Civil Engineering Department, the testing of full size building components was made possible. Using the Dartec machine for applying the axial load, the choice of specimen size was limited mainly by the capacity, 1.5MN, of the jacks available for applying the lateral loads, and the clear height within the Dartec straining frame.

Although a reverse moment pattern through the 'joint' would have been desirable (to simulate that experienced during an earthquake), time and resources did not allow such a testing programme. A moment gradient, however, was felt as being necessary and so a simple mid height loading (giving a moment pattern and moment at critical section as shown in FIG. 3.1) was adopted. In order to achieve cyclic loading and experimental simplicity, a mid height loading point on each side of the specimen was used.
FIG. 3.1 Moment pattern.

The final column dimensions were obtained by considering the following factors:

a) lateral load jack capacity,
b) longitudinal steel ratio bearing in mind that GR 380 steel was to be used (see later),
c) suitable beam stub depth,
d) realistic column height (Dartec machine limited to 4m between loading platens),
e) accessibility to machine (maximum clear spacing between Dartec straining frame is 1.0m).

FIG. 3.2 shows the dimensions of the column specimens tested.

3.3 DESIGN OF SPECIMENS

3.3.1 Column designation

The column numbering system adopted in this report refers to the order in which the specimens were tested and is:

<table>
<thead>
<tr>
<th>SPECIMEN NUMBER</th>
<th>NOMINAL $\frac{P_e}{f'_c}$</th>
<th>$A_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ONE</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>TWO</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td>THREE</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>FOUR</td>
<td>0.55</td>
<td></td>
</tr>
</tbody>
</table>
FIG. 3.2 Column Specimen Dimensions.
3.3.2 Longitudinal reinforcement

The moment-curvature analyses mentioned in section 1.4 have demonstrated that use of high strength steel as longitudinal reinforcement in columns improves the performance of the columns at high curvatures because the early strain hardening of that steel helps to compensate for the loss of moment capacity due to the reduction of contribution from the concrete. For this reason it was decided to use GR 380 steel as the longitudinal reinforcement.

Conforming to the requirements of D4 3101 for GR 380 longitudinal steel, \((0.01 \leq \rho_t \leq 0.045\), spacing not greater than one third of the length of the section dimension, maximum bar diameter 1/25th of the depth of any beam framing into the column at that level), it was decided to use 12 DH24 bars equally spaced on the four sides of the column with 50 mm cover, as shown in FIG. 3.3

![Diagram of longitudinal steel arrangement](image)

FIG. 3.3 Longitudinal steel arrangement - all specimens.

This gave a \(\rho_t\) of 0.018 and a beam stub depth of 600 mm.
3.3.3 Transverse reinforcement

a) Steel area

The transverse steel in potential plastic hinge regions required by DZ 3101 is governed by equation (8) or (9). In this case equation (9) governs because of the section size and cover chosen.

It was felt that no decided disadvantage would be experienced by having two different arrangements of tie sets and so the two most logical arrangements for the longitudinal steel arrangement used, conforming to DZ 3101 requirements, were adopted. See FIG. 3.4.

(a) Specimen One and Two  
(b) Specimen Three and Four

FIG. 3.4 Tie arrangements.

GR 275 R10 bars were chosen for specimens One and Three, GR 275 R12 bars for specimens Two and Four.

Arrangement (a) gives 3.414 effective leg areas of confinement steel in each direction, arrangement (b) gives 4 effective leg areas.

Solving equation (9) for $s_n$ for each of the four specimens gives the confinement steel required in
the hinge zones, listed in TABLE 3.1. For comparison the steel areas required by the SEAOC provisions are included.

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Pe</th>
<th>$A_{sh}$ (mm$^2$)</th>
<th>$h''$ (mm)</th>
<th>$f'_{c}$</th>
<th>$s_{h}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.15</td>
<td>268.1</td>
<td>461.4</td>
<td>470</td>
<td>0.101</td>
</tr>
<tr>
<td>Two</td>
<td>0.40</td>
<td>386.1</td>
<td>386.1</td>
<td>474</td>
<td>0.095</td>
</tr>
<tr>
<td>Three</td>
<td>0.25</td>
<td>314.2</td>
<td>415.9</td>
<td>470</td>
<td>0.101</td>
</tr>
<tr>
<td>Four</td>
<td>0.55</td>
<td>452.4</td>
<td>359.7</td>
<td>474</td>
<td>0.102</td>
</tr>
</tbody>
</table>

**TABLE 3.1** Column specimen hoop steel requirements.

When the design was carried out the known yield stress of the bars was used but all the specimens were designed using a concrete strength, $f'_{c}$, of 30MPa. It was recognised that some adjustment of the axial load level to compensate for variations of actual concrete strength at the time of testing would be necessary to ensure equality of both sides of equation (9).

(b) **Spacing**

The centre to centre spacings of the confining steel sets as shown in TABLE 3.1 all conformed to the code requirements except that the specimen Four spacing was closer than need be. Rather than increase the bar size (and hence the spacing) it was decided to leave the spacing at 62 mm as this still allowed adequate room for concrete placement and vibration.

(c) **Potential hinge region**

Section 17.6.1.1 of the New Zealand Concrete Code requires that the potential hinge region be the larger of:
(i) the longer member section dimension (550 mm),
(ii) one-sixth the member clear height (450 mm),
(iii) 450 mm.

Thus special transverse reinforcement is required for a length of 550 mm from the beam stub.

(d) Between potential plastic hinge regions

The transverse steel in these areas was designed according to the maximum expected shear loading, and conforming to the requirements of Chapter 19 and Section 17.6.2 of DZ 3101.

The permissible nominal shear stress carried by the concrete, $v_c$, was computed by:

$$v_c = 0.25 \left(1 + \frac{f'_c}{25}\right) \sqrt{\frac{P_e}{A_c}} - \frac{f'_c}{25}\sqrt{\frac{P_e}{A_c}}$$  \hspace{1cm} (10)

A value of $f'_c = 30$ MPa was used for all specimens.

The total nominal design shear stress, $V_u$, was calculated using the expected theoretical maximum lateral force $P_u$. (see Section Five)

$$V_u = \frac{V_u}{\phi_{b_w} d}$$  \hspace{1cm} (11)

The area of shear reinforcement, $A_v$, within a distance $s$ was found from:

$$A_v = \frac{(V_u - v_c) b_w s}{f_{yh}}$$  \hspace{1cm} (12)

TABLE 3.2 summarises the design. In all cases the shear requirements were checked against the special transverse steel requirements.

Plans for the reinforcing cages are shown in FIGS. 3.5 and 3.6.
FIG. 3.5 Reinforcing Cage Plans
FIG. 3.6 Reinforcing Cage Plans.

SPECIMEN THREE

6 sets R10
triple ties-105

8 sets R10
triple ties-75

7 sets R10
triple ties-70

12 DH24
50 cover

SPECIMEN FOUR

3 sets R12
triple ties-200

10 sets R12
triple ties-62

8 sets R12
triple ties-60

12 DH24
50 cover
<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>$\frac{f'_c}{f_c}$</th>
<th>$v_c$ (MPa)</th>
<th>$v^*$ (kN)</th>
<th>$v_u$ (MPa)</th>
<th>$A_v$ (mm$^2$)</th>
<th>$s$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.15</td>
<td>0.67</td>
<td>5/6</td>
<td>2.52</td>
<td>514.2$^+$</td>
<td>92**</td>
</tr>
<tr>
<td>Two</td>
<td>0.40</td>
<td>1.65</td>
<td>665</td>
<td>2.91</td>
<td>452.4$^+$</td>
<td>206</td>
</tr>
<tr>
<td>Three</td>
<td>0.25</td>
<td>1.17</td>
<td>642</td>
<td>2.81</td>
<td>314.2</td>
<td>103</td>
</tr>
<tr>
<td>Four</td>
<td>0.55</td>
<td>2.02</td>
<td>606</td>
<td>2.66</td>
<td>452.4</td>
<td>378++</td>
</tr>
</tbody>
</table>

TABLE 3.2 Shear steel requirements.

$^*$ Design values were taken from Design Charts [36] with $\phi = 1, f'_c = 30$ MPa.

$^+$ Shear steel taken as four legs effective.

$^{**}$ Due to error in calculations, ties are at 135 mm centres in specimen.

$^{++}$ Greater than $b/2$. 200 used in specimen.

5.3.4 Beam stub

As well as simulating a beam, the stub was incorporated to provide a loading point for the lateral load and confinement of the central 'joint' so that hinging would be forced to occur in the desired area. The extra reinforcement in this case consisted of top and bottom D 24 'beam bars' and a 12 beam 'stirrups'. See FIG. 3.7.

FIG. 3.7 Beam stub steel.
3.3.5 **Anchorage**

Because the problem of longitudinal bar slip was of no interest in this project, positive mechanical anchorage of the longitudinal bars was provided at each end of the specimens by means of a welded angle iron frame, as shown in FIG. 3.8. Because GR 380 steel was used each bar had to be preheated before welding.

![Diagram of anchorage details](image)

**FIG. 3.8** End anchorage details.

The completed reinforcing cages for each specimen are shown in FIG. 3.9.

3.4 **Material Properties**

3.4.1 **Steel**

Prior to design and construction of the columns, tensile tests were carried out on randomly selected steel specimens. For each of the different steel sections
used six test specimens were chosen, two being subjected only to yield and ultimate strength determinations and the remaining four being subjected to full extensometer testing in order to establish the stress-strain relationships. Table 3.3 summarises the yield and ultimate stresses while Figs. 3.10 to 3.13 give the stress-strain curves.

It is of interest to note that two orders of R12 steel received (four months apart but probably from the same batch) were rejected because they did not exhibit a well defined yield plateau, a characteristic that was felt was generally unrepresentative of steel used in New Zealand construction.

<table>
<thead>
<tr>
<th>Bar</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D24</td>
<td>375</td>
<td>635.6</td>
</tr>
<tr>
<td>R10</td>
<td>297</td>
<td>416.8</td>
</tr>
<tr>
<td>(SP. ONE &amp; THREE)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R12</td>
<td>316</td>
<td>440.6</td>
</tr>
<tr>
<td>(SP. TWO)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R12</td>
<td>294</td>
<td>412.4</td>
</tr>
<tr>
<td>(SP. FOUR)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3 Steel properties.

3.4.2 Concrete

The concrete used in all specimens was delivered by one of the Christchurch Ready Mix suppliers with a specified nominal 28 day strength of 30 MPa, slump of 75 mm and maximum aggregate size of 20 mm.

In order to achieve the desired range of levels of axial load it was important that the concrete strength for all specimens be close to 30 MPa. It was expected that some adjustment of axial load level would be required to balance the governing equation (9) but not
FIG 3.10 'STRESS STRAIN CURVE FOR GR 380 D 24 BAR' (LONGITUDINAL BARS - ALL SPECIMENS)
FIG 3.11 "STRESS - STRAIN CURVE FOR GR 275 R10 BAR" (TIES FOR SPECIMENS ONE AND THREE)
FIG 3.12 "STRESS - STRAIN CURVE FOR GR 275 R12 BAR" (TIES FOR SPECIMEN TWO)
FIG 3.13 STRESS STRAIN CURVE FOR GR 275 R 12 BAR (TIES FOR SPECIMEN FOUR)
to the extent that was finally necessary. Without exception the concrete strength, as determined by compression tests on cylinders just prior to testing of each specimen, was vastly different from 30 MPa and hence lead to significantly different levels of axial loads than originally intended. (See TABLE 3.4) It highlights one of the major problems facing designers and researchers in reinforced concrete construction - the applicability of sophisticated design methods that are dependent on concrete material properties when there is no guarantee that the as built concrete is the same as that assumed in design. The possibility of concrete strength greater than the specified strength has potentially serious significance, as according to the requirements of D2 3101 it means that the amount of confining steel is insufficient for the level of axial load designed for.

TABLE 3.4 summarises the concrete cylinder strengths and the intended and actual axial load levels tested.

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Concrete Strength (MPa)</th>
<th>Pe/f'c</th>
<th>A_g</th>
<th>Intended</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>23.1</td>
<td>0.15</td>
<td>0.26</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Two</td>
<td>41.4</td>
<td>0.40</td>
<td>0.214</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Three</td>
<td>21.4</td>
<td>0.25</td>
<td>0.42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Four</td>
<td>23.5</td>
<td>0.55</td>
<td>0.60^x</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TABLE 3.4 Concrete strengths.

^x A value of 0.76 is required for balance of equation (9) but since 0.6 is the upper limit allowed by the code this was the value chosen.

3.5 CONSTRUCTION

Because of the relatively large mass of concrete in each specimen, the need for four specimens to be built,
and the choice (for practical reasons) of pouring in the horizontal position, it was felt that a rigid mould was required. Thus the mould consisted of a fixed steel channel section framework base and removable angle iron side frames, all sheathed with 17 mm construction ply. The finished inner surface received 3 layers of paint.

The construction process consisted of strain gauging (detail in Section Four) of the transverse reinforcement to be placed in the potential plastic hinge zones, tying of the transverse steel to the longitudinal bars (including beam stub steel), welding of end zone anchorage frames and placement of lifting inserts and dial gauge frame support rods before lowering of the cage into the mould. The ready mix concrete was placed into the mould and vibrated and six concrete cylinder samples taken. After initial set had occurred the exposed concrete surface was covered with damp sacks and a plastic 'jacket' for a minimum of seven days before the formwork was stripped. FIG. 3.14 shows the reinforcing cage for specimen Three in the mould prior to concrete placement.
FIG. 3.14 Specimen mould.
SECTION FOUR

INSTRUMENTATION AND TESTING PROCEDURE

4.1 SUMMARY

In order to obtain moment-curvature and load-displacement plots, curvature and longitudinal bar strain profiles, as well as an indication of transverse steel maximum strains, plastic hinge lengths and ultimate concrete compression strains, various instrumentation was required.

This section details the instrumentation employed to obtain the above information and also sets out the testing procedure.

4.2 INSTRUMENTATION

4.2.1 Strains and curvatures

A series of dial gauges mounted on external frames, which were in turn mounted on built-in stubs anchored to the concrete core of the column (see FIG. 4.1), enabled strains and curvatures to be determined.

FIG. 4.1 Dial gauge frame support.

By considering a pair of gauges mounted at the
same height on each side of the column specimen, and knowing the gauge length of the built in stubs, a strain distribution across the column section can be obtained. Interpolation of this distribution gave concrete face strains and longitudinal bar strains. Determination of the neutral axis depth from the strain distribution allowed curvatures to be calculated.

While this method assumes strain compatibility at the concrete-steel interface and the plane sections hypothesis such assumptions were felt accurate enough for the purposes of this project.

FIG. 4.2 shows the location of the dial gauges while FIG. 4.3 demonstrates the method of strain and curvature determination as described above.

Electric resistance strain gauges were mounted at various locations on the transverse confining steel in order to obtain an indication of the effectiveness of the confining steel by virtue of the extent of yielding that may have occurred. Typical locations (in this case for specimen Four) are shown in FIG. 4.4. Gauge locations were lightly filed, then smoothed with emery paper and cleaned thoroughly with Methyl Ethyl Ketone. Either 5 mm Kyowa KFG-5-C1-11 or 5 mm Showa N11-PA-5 electric strain gauges and Showa SFG-5 T terminal strips were glued to the gauge sites. Showa SC-22 adhesive was used for waterproofing but because of concern for mechanical protection a final layer of epoxy was added. For specimen Four Shinkoh SN/4 waterproofing compound was used, covered with Scotch 3M vinyl mastic for mechanical protection. All strain gauge leads were threaded through electrical "spaghetti" to reduce the possibility of leads fracture when concrete cracking occurred.

Despite the measures taken to ensure strain gauge protection the data that was obtained from them was far from perfect. A higher than expected proportion of
FIG. 4.2 Dial gauge locations.
strain, $\varepsilon$

\[
\text{curvature, } \phi = \frac{\varepsilon_{D1} + \varepsilon_{D2}}{750} \\
\text{NA} = \frac{\varepsilon_{D2}}{\phi} - 100 \\
\varepsilon_A = (488 - \text{NA}) \phi \\
\varepsilon_D = (\text{NA} - 62) \phi
\]

FIG. 4.3 Strain and curvature determination.
Fig. 4.4 Typical strain gage location.
gauges failed completely at relatively low strains, other gauges gave inconsistent results and most had low insulation resistance. Fortunately because a large number of gauges had been installed to obtain an extensive coverage, sufficient data was still available for an indication of the effectiveness of the confining steel.

It was felt that the reason for the poor performance of the electric strain gauges was due mainly to the poor mechanical protection. After construction it was discovered that the Showa SC-22 adhesive should be built up in layers and not applied as one layer.

4.2.2 Horizontal displacements

Four dial gauges were mounted on an independent frame, one each at the top and bottom of the specimen on the line of the lateral load reaction links and one each at the top and bottom of the beam stub (see FIG. 4.2). All four gauges were 0.01 x 50 mm Mitutoyo gauges. By an averaging process the mid point horizontal deflection relative to the column ends could be obtained.

4.2.3 Load measurement

The axial load was maintained at the desired level by the 'hold control' of the Dartec machine. Accurate measurement of the applied lateral load was achieved by means of two nominally rated 1.5 MN load cells, which prior to testing had been calibrated on an Avery Universal Testing Machine. (In practise the load cells remain linear up to 50% above their rated capacity.)

4.3 TESTING PROCEDURE

4.3.1 Test equipment

As mentioned previously axial load application was by means of the 10 MN Dartec machine and lateral
load by a pair of 1.5 mN jacks. The adopted loading pattern (Section 5.2) required a pair of reaction frames as shown diagrammatically in FIG. 4.5.

FIG. 4.5 Load application.

The reaction frames were each assembled from a pair of large steel I beams, welded together side by side and with appropriate stiffener and connection plates (see Appendix B for plans). Column top and bottom end caps were made from 32 mm steel plate with 75 mm diameter steel pin extensions to which the reaction link arms (200 x 20 steel section with local thickening round holes) were attached to corresponding pins on the reaction frames. Semi circular bearings welded to plates with matching scalloped plates acted as the axial loading points as well as allowing the end rotation necessary to achieve the simple moment pattern. Because of their weight and awkwardness the bearings were able to be bolted to the column end caps while the associated
scalloped plates could be bolted to the Dartec loading plattens. The lateral loads were applied through spherical bearings and flat loading plates, which allowed for any unsymmetrical rotation at mid height. FIG. 4.6 shows the two different types of bearings.

4.3.2 Specimen installation

This deserves mention because of the overhead lifting limitations and accessibility to the Dartec machine. (Overhead crane directly above machine had a 1500 kg capacity, whereas each column weighed approximately 3200 kg and reaction frames weighed 1600 kg each.)

The column test specimen was moved as close as possible to the Dartec machine by means of the 5700 kg capacity overhead crane in the preparation area of the laboratory (to within about 15 m), 'aimed' towards the machine and lowered on to a pair of trolleys. At this stage the end caps were fitted and locked into position (thus the reason for section enlargement at ends of columns) and then the column was manually pulled towards the machine by means of a Tirfor Tu 15 anchored through the far side of the test space of the Dartec machine. Because the strong floor is not continuous over the whole laboratory it was necessary to lay a steel plate path to avoid damaging the floor. Once one end of the column was located above the test machine lower platten, two 3 tonne chain block hooks permanently fixed to the crosshead of the machine were attached to the lifting brackets near the top of the specimen which was then winched into the vertical position, the bottom end riding in freely on its trolley. The process was repeated for the West side reaction frame (East side frame was permanent for duration of test series) except that it was lifted vertically by the overhead crane. The next step was to fit the link arms from the reaction frames to the end caps followed by instal-
FIG. 4.6 Load bearings.
lation of horizontal jacks and loading plates, final exact positioning and plastering of axial loading plates and connection of strain gauge leads to the data logger. The process described above is shown in FIG. 4.7.

4.3.3 Testing proper

(a) Preliminary

Just prior to commencement of testing, compression tests were carried out on the concrete cylinders to determine the as tested concrete strength and hence the required level of axial load. This load level was applied and held constant by the Dartec machine (direct digital readout of load) to within ± 5 kN.

(b) Horizontal load sequence

The initial horizontal load cycle was used to establish the first yield displacement and involved a load application in each direction in turn up to about 3/4 of the predicted theoretical ultimate (still in elastic range). Subsequent cycles followed the pattern commonly used at the University of Canterbury (see FIG. 4.8).

FIG. 4.8 Loading criterion.
FIG. 4.7a Specimen installation.
FIG. 4.7b Specimen installation.
For specimens Three and Four a third cycle to ductility factor six was performed to check on any further degradation. Testing was stopped at this stage rather than continuing to higher ductility levels, so that the specimens could in the future be repaired and retested.

(c) Data acquisition

At each load increment, all dial gauges and horizontal load were read (horizontal load by pre-calibrated Budd P350 strain indicator). At the peak of each load cycle and some intermediate increments, a full set of electric strain gauge readings were taken by a 200 channel Solartron DTU data logger and all crack formations were marked and photographed.
SECTION FIVE

TEST RESULTS FROM SPECIMENS

5.1 SUMMARY

This section contains the detailed results obtained from each of the four column specimens. The section is divided into four subsections, one for each of the specimens and all following the same format, describing the derivation of the yield displacement and yield curvature, general description of the behaviour of the columns during testing, and various plots made from the experimental data.

5.2 SPECIMEN ONE

Actual properties: \( f'_c = 23.1 \text{ MPa} \)
\( P_e = 0.26 f'_c \quad A_g = 1815 \text{ kN} \)
Long. Steel
\( f_t = 0.0179 \text{ with } f_y = 375 \text{ MPa} \)
Hoops = R10/80 with
\( f_{yh} = 297 \text{ MPa} \)

5.2.1 YIELD DISPLACEMENT AND CURVATURE

Yield displacement and curvature for the column specimen was determined as described in Section 2.2. The basis for the determination of the theoretical ultimate moment used in this calculation was the New Zealand Ministry of Works and Development publication CDP 801/A [36] which presents column design charts adapted from the charts originally given in ACI publication SP - 7 [37]. Using capacity reduction factor \( \phi = 1 \), we have the theoretical ultimate moment of resistance of 691.2 kNm using the actual material \( f'_c \) and \( f_y \) values, corresponding to a mid height ultimate lateral load of 1152 kN.

The load displacement relationship measured in the first elastic loading cycle was extrapolated to the theoretical ultimate lateral load of 1152 kN giving
FIG. 5.1 Yield displacement specimen one.
yield displacement, \( \delta_y \), of 5.7 mm. The process is shown in FIG. 5.1. A similar operation on the moment-curvature relationship gave a yield curvature, \( \phi_y \), of \( 7 \times 10^{-6} \) / mm.

5.2.2 GENERAL DESCRIPTION OF THE BEHAVIOUR OF THE COLUMN DURING TESTING

Elastic cycle (Cycle 1)

A crack at the beam stub.column interface, and two other fine horizontal cracks, appeared on the tension face at both the top and bottom plastic hinge regions. At the top the cracks were spaced at about 200 mm vertically apart, while at the bottom the spacing was about 150 mm.

\( \text{DF} = 1 \) (Cycle 2)

One further crack appeared at the bottom plastic hinge position at about the same spacing as before. At the top two further cracks appeared, one outside the hinge region. The initial cracks all lengthened, branched and widened.

\( \text{DF} = 2 \) (Cycles 3 and 4)

The number of cracks at the top remained the same and one further crack appeared at the bottom. All cracks had now noticeably extended and inclined towards the centre due to the influence of shear. Crushing of the column cover concrete near the beam stub was observed.

\( \text{DF} = 4 \) (Cycles 5 and 6)

One further crack was observed. Spalling of the cover concrete occurred and by the end of Cycle 6 extended for almost 200 mm from the beam stub. The cracks were evenly distributed over the plastic hinge length with considerable overlap from side to side. On the tension face cracks were up to 5 mm wide.
DF = 6 (Cycles 7 and 8)

Initial spalling of concrete left an inclined plane of intact concrete extending from the beam stub-column interface to approximately the first tie set. (See FIG. 5.2) Elsewhere along the plastic hinge spalling of concrete occurred right back to the longitudinal bars. Four tie sets were clearly visible, and although not all of the cover concrete had fallen off it had obviously separated from the core over a length of about 400 mm. Neither the confining transverse steel nor the longitudinal steel showed any sign of buckling. The concrete core appeared to be excellently confined.

FIG. 5.3 shows the condition of the column at various stages of testing. The final photograph at DF = 6 shows the column after most of the loose cover concrete had been removed.

FIG. 5.2 Typical inclined spalling plane.
FIG. 5.3 Testing of specimen One.
5.2.3 Measured Load-Displacement and Moment-Curvature Relationships

The lateral load-displacement and moment-curvature relationships obtained from the test are shown in Fig. 5.4 and 5.5 respectively. The total mid-height lateral loading is plotted against the average (top and bottom of beam stub) mid-height lateral displacement, while the total moment (includes P-\delta effect) at the beam stub-column interface is plotted against the average of the top and bottom curvatures obtained from the dial gauges over the first 100 mm from the beam stub. The corrected theoretical ultimate load for the P-\delta effect is shown.

Both plots confirm the good behaviour of the specimen observed during testing. Although there is some load (and moment) degradation on the second cycle to each of the ductility factor peaks it is not of major proportions and without exception the peak loads all exceed the predicted theoretical ultimate loads. A comparison between theoretical and experimental moments is given in Table 5.1. Some stiffness degradation (characterised by 'pinching' of the curves) during reversal of load in post elastic cycles can be observed but it is not of any major significance and is characteristic of columns, due to the compressive axial load. Thus the earthquake energy dissipation capability of this column detail appeared to be excellent.

5.2.4 Measured Curvature Profile

The curvature profile plotted in Fig. 5.6 shows the curvatures at the centre of the gauge lengths joined by straight lines. It can be seen that the high curvatures are concentrated near the beam stub. Although the curvatures at the bottom plastic hinge are a little greater during reverse loading compared with the top plastic hinge (i.e. the beam stub tilted slightly
Pe = 0.26 f'c Ag  
(f'c = 23.08 MPa)  
Curvature averaged at 100 mm from slab

FIG. 5.5 "MOMENT CURVATURE"  SPECIMEN ONE
<table>
<thead>
<tr>
<th>Displacement ductility factor</th>
<th>Cycle</th>
<th>FORWARD LOADING</th>
<th>REVERSE LOADING</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum moment (kN m)</td>
<td>Increase over theoretical moment (%)</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>783</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>771</td>
<td>11.5</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>821</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>797</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
<td>836</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>801</td>
<td>16</td>
</tr>
</tbody>
</table>

**TABLE 5.1** Comparison of theoretical ultimate and experimental maximum moments - Specimen One.

* Based on Ref. [56] with $\phi = 1$. 

58.
FIG. 56 "CURVATURE PROFILE" — SPECIMEN ONE
during testing) the reasonable agreement between top and bottom behaviour was the basis for using the average curvature in the moment curvature plot. The maximum curvature ductility factor obtained of 21 at displacement ductility factor 6 clearly demonstrates the significantly higher section ductility demands discussed in Section 2.5.

5.2.5 MEASURED BAR STRAIN PROFILES

The longitudinal bar strain profiles in FIGS. 5.7 and 5.8 were obtained on the basis of the method and assumptions described in Section 4.2.1. It can be seen that the region of tension yield extends almost over the full 550 mm length of the 'code hinge length' at displacement ductility factors of four and greater, and over a significant length at displacement ductility factor two. Of more interest is the region over which the steel is influenced by strain hardening, demonstrating the benefit of using GR 380 steel for the longitudinal bars in columns. The maximum steel strain obtained during the test corresponds to a steel strength enhancement of about 47% above the actual yield strength which will considerably offset any loss of column flexural strength due to spalling of concrete. The greater the tension strains obtained compared with compression strains are typical of what would be expected for a section under a relatively low level of axial compression, (i.e. the natural axis is closer to the extreme compression fibre than to the extreme tension fibre).

5.2.6 MEASURED CONFINING STEEL STRAINS

Section 4.2.1 mentioned the disappointing performance of the electric strain gauges that had been placed in the plastic hinge region on the confining steel. However, from the gauges that did give consistent results it was obvious that the confining hoops had performed their task adequately, further substantiated by the fact that there was no indication at all of buckling of the
FIG 5.7  "BAR STRAIN PROFILE" - SPECIMEN ONE - FORWARD LOADING
FIG. 58 "BAR STRAIN PROFILE" — SPECIMEN ONE — REVERSE LOADING.
longitudinal bars. The majority of gauge positions indicated the hoop yield strains had not been exceeded. In those positions where yield had occurred it appeared not to be extensive. A maximum strain of 4300 microstrains was measured, yield strain being established at 1400 microstrains, strain hardening commencing at 12460 microstrains.

5.3 SPECIMEN TWO
Actual properties: $f'_c = 41.4$ MPa
$P_e = 0.214 f'_c A_g = 2680$ kN
Long. steel
$\rho_t = 0.0179$ with $f_y = 375$ MPa
Hoops = R 12/75 with
$f_{yh} = 516$ MPa

5.3.1 YIELD DISPLACEMENT AND CURVATURE
Using the same method as for specimen One the theoretical ultimate moment of resistance at the critical section was calculated to be 905.0 kNm. This gave a total theoretical ultimate mid-height lateral load of 1508 kN.

Extrapolation of the load-displacement relationship measured in the first elastic loading cycle to the theoretical ultimate lateral load gave a yield displacement $\delta_y$ of 4.2 mm, (see FIG. 5.9). Yield curvature, $\gamma_y$, was established at $7 \times 10^{-6}$ / mm.

5.3.2 GENERAL DESCRIPTION OF THE BEHAVIOUR OF THE COLUMN DURING TESTING
Elastic Cycle (Cycle 1)
Cracking was initiated during the first load increment. At the completion of the cycle there was a total of six cracks at both the top and bottom plastic hinge regions on the east side (tension face for positive loading), and six and four cracks at the top and
FIG. 5.9 Yield displacement specimen Two.
bottom respectively, on the west side (tension face for negative loading). All cracks were predominantly horizontal and were spread at about 100 mm vertically.

\[ \text{DF} = 2 \] (Cycles 2 and 3)

During the first positive loading increments two additional cracks appeared at the top. All other cracks had extended and were markedly inclined and overlapping from side to side. Spalling of the compression face concrete at the beam stub column interface was observed during the first positive loading increments.

\[ \text{DF} = 4 \] (Cycles 4 and 5)

It became obvious that the plastic hinging was concentrated at the top. That is, the beam stub was tilting and resulting in large plastic rotation above the beam stubs. There was sudden substantial spalling of the cover concrete during the first positive loading increments and by the end of cycle five there was appreciable spalling of the cover concrete on both the east and the west faces above the beam stub. At the bottom there was only minor extension of existing cracks.

\[ \text{DF} = 6 \] (Cycles 6 and 7)

Although there was some new crack formation at the bottom, most of the inelastic deformation was still concentrated at the top. By the completion of the test, spalling of the cover concrete at the top extended over about 400 mm and although there was crushing at the bottom there was no spalling. As for specimen One, there was no indication of buckling of either the confining transverse or longitudinal steel.

FIG. 5.10 shows the condition of the column at various stages of testing.
FIG. 5.10 Testing of specimen Two
5.3.3 MEASURED LOAD-DISPLACEMENT AND MOMENT-CURVATURE RELATIONSHIPS

FIGS. 5.11 and 5.12 are the lateral load-displacement and moment-curvature relationships respectively obtained from the test. The average mid-height displacement was again plotted against the total mid-height lateral load. The moment plotted is that at the face of the beam stub, but because of the concentration of plastic hinging action at the top the curvature plotted is that obtained from the dial gauges over the first 100 mm at the top only.

The sudden spalling of cover concrete during the first load excursion to DF = 4 is clearly indicated in both plots. Despite this cover loss the maximum moment strength obtained during each of the cycles compares well with the theoretical ultimate moment of the section. TABLE 5.2 shows a detailed comparison between theoretical and experimental moments. Both plots show some strength degradation during repeat cycles to ductility factor peaks but as for specimen One it is not of major proportions. Stiffness degradation during load reversals is again apparent as is greater resistance in one direction (reverse loading).

5.3.4 MEASURED CURVATURE PROFILE

The curvatures plotted at the centre of the gauge lengths in FIG. 5.13 clearly demonstrate the concentration of plastic hinging action at the top and hence the reason for using the top curvatures only in the moment curvature plot. Curvatures are again concentrated near the beam stub and for a displacement ductility factor of 6 a maximum curvature ductility factor of 20 was obtained.

5.3.5 MEASURED BAR STRAIN PROFILES

Consistent with the observed behaviour and curvature distribution are the longitudinal bar strain pro-
FIG. 5.11  "LOAD - DISPLACEMENT"  SPECIMEN TWO
Pe = 0.214 f_c Ag.
(f_c = 41.4 MPa).
Top Curvature at
100 mm plotted.

FIG. 512
MOMENT CURVATURE
SPECIMEN TWO
<table>
<thead>
<tr>
<th>Displacement ductility factor</th>
<th>Cycle</th>
<th><strong>FORWARD LOADING</strong></th>
<th></th>
<th><strong>REVERSE LOADING</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum moment (kN m)</td>
<td>Increase over theoretical moment (%)</td>
<td>Reduction in repeat cycle from initial cycle (%)</td>
<td>Maximum moment (kN m)</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>910</td>
<td>0.5</td>
<td></td>
<td>942</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>876</td>
<td>-3</td>
<td>-4</td>
<td>935</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>957</td>
<td>6</td>
<td>-9.5</td>
<td>1006</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>895</td>
<td>-1</td>
<td>-6.5</td>
<td>956</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>956</td>
<td>6</td>
<td></td>
<td>1010</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>927</td>
<td>2.5</td>
<td>-3</td>
<td>974</td>
</tr>
</tbody>
</table>

**TABLE 5.2** Comparison of theoretical ultimate and experimental maximum moments—Specimen Two.

* Based on Ref. [36] with $\phi = 1$.

** Minimum values obtained after cover concrete crushing.
FIG. 513 "CURVATURE PROFILE" — SPECIMEN TWO
Fig. 514 "Bar Strain Profile" - Specimen Two - Forward Loading
Fig 5.15  "Bar Strain Profile" - Specimen Two - Reverse Loading
files as shown in Figs. 5.14 and 5.15. The steel at the top hinge region is subjected to significantly higher strains than the bottom region and hence the length over which the steel strains exceed either yield or strain hardening strain is correspondingly greater at the top. The maximum strain obtained during the test corresponds to a steel strength enhancement of about 40% above the actual yield strengths (c.f. 47% for specimen One). The relative magnitudes of the tension and compression steel strains are again indicative of the relatively low level of axial compressive column load on this specimen.

5.3.6 MEASURED CONFINING STEEL STRAINS

Confirmation of the good behaviour of the confining steel observed visually (no buckling of longitudinal steel was visible) was provided by the electric strain gauge readings. Despite the limitations on the accuracy of the readouts discussed in Section 4.2.1, it was apparent that yielding of the confining steel was again not extensive. Although yielding had occurred at several gauge positions, nowhere had strain hardening commenced and this in spite of the fact that the length of the yield plateau for the confining steel in this specimen was considerably less than that for the confining steel in specimen One. A maximum strain of 5200 microstrains was observed, yield strain being 1500 microstrains and strain hardening at 6090 microstrains.

5.4 SPECIMEN THREE

Actual properties: $f'_c = 21.4$ MPa

$P_e = 0.42 f'_c A_g = 2719$ kN

Long. steel

$\rho_t = 0.0179$ with $f_y = 375$ MPa

Hoops = $R$ 10/75 with

$f_{yh} = 297$ MPa
5.4.1 **Yield Displacement and Curvature**

The theoretical ultimate moment of resistance as determined by the CDP 810/A [36] column design charts was 645.9 kN.m, with a correspondingly theoretical ultimate mid-height lateral load of 1076.5 kN.

**FIG. 5.16** shows the extrapolation procedure used to obtain the yield displacement, $\delta_y$, of 3.5 mm. Yield curvature, $\phi_y$, was established at $7 \times 10^5$/mm.

5.4.2 **General Description of the Behaviour of the Column During Testing**

**Elastic Cycle (Cycle 1)**

During positive loading (east side tension face) a crack at the beam stub-column interface appeared at both the top and bottom plastic hinge regions. Formation of one crack at the top and two at the bottom plastic hinge regions was initiated during negative loading (west side tension face).

\[ DF = 2 \] (Cycles 2 and 3)

The number of cracks at both the top and bottom plastic hinge regions totalled five on each face at the completion of cycle 3, spaced at about 130 mm vertically. The initial cracks all lengthened, branched and widened, with noticeable inclination towards the centre. Crushing of the column cover concrete and minor spalling at the corners adjacent to the beam stub was observed.

\[ DF = 4 \] (Cycles 4 and 5)

Although there was some extension of existing cracks and overlap from side to side on early cracks, the major addition was the sudden spalling of the cover concrete at the top and bottom plastic hinge regions. This spalling was evident over a length of about 200 mm from the beam stub.
FIG. 5.16 Yield displacement specimen Two.
DF = 6 (Cycles 6, 7 and 8)
It became obvious that there was little difference between the top and bottom plastic hinge region behaviour. Spalling of the cover concrete right back to the longitudinal steel continued on all compression faces with some loss of cover concrete on the side faces. The maximum extent of cover concrete loss was about 500 mm from the beam stub. There was no apparent sign of buckling of either the confining transverse or longitudinal steel.

The condition of the column at various stages of testing is shown in FIG. 5.17.

5.4.3 MEASURED LOAD-DISPLACEMENT AND MOMENT-CURVATURE RELATIONSHIPS
The lateral load-displacement and moment-curvature relationships obtained from the test are shown in FIG. 5.18 and 5.19 respectively.

The displacement used is the average mid-height displacement. The moment at the face of the beam stub is plotted against the average of the top and bottom curvatures obtained from the dial gauges over the first 100 mm.

Both plots clearly show the sudden spalling of cover concrete during the first excursion to DF = 4. The measured maximum load and moment during each cycle exceed the predicted theoretical ultimate values. A comparison between theoretical and experimental moments at each cycle peak is shown in TABLE 5.3. The percentage increase is greater than for the previous two specimens, probably more a reflection on the design charts [36] than anything else. The strength degradation during repeat cycles to ductility factor peaks is small, as is the stiffness degradation during load reversals. There is only a small increase in average curvature values during positive loading when going from DF = 4
Fig. 5.17 Testing of specimen Three.
FIG. 5.18 LOAD DISPLACEMENT SPECIMEN THREE
FIG. 519  "MOMENT CURVATURE"  SPECIMEN THREE

\[ P_c = 0.42 \, f'_c \, A_g \]  
\[ (f_c = 21.4 \, MPa) \]

Curvature averaged at 100 mm from stub

ACI \( M_u \) theoretical

\[ \phi/\phi_y \]

CURVATURE (\( \phi \)) \( \times 10^{-6} / \text{mm} \)

Tie = 110
Sh = 75
Cover = 50 mm
Bars = 0.24
TABLE 5.3 Comparison of theoretical ultimate* and experimental maximum moments—Specimen Three.

<table>
<thead>
<tr>
<th>Displacement ductility factor</th>
<th>Cycle</th>
<th>FORWARD LOADING</th>
<th>REVERSE LOADING</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum moment</td>
<td>Increase over</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(kN m)</td>
<td>theoretical moment (%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reduction in repeat cycle from initial cycle (%)</td>
<td>Reduction in repeat cycle from initial cycle (%)</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>762</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>744</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>808</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>747</td>
<td>15.5</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>812</td>
<td>26</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
<td>760</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>735</td>
<td>17.5</td>
</tr>
</tbody>
</table>

* Based on Ref. [36] with \( \phi = 1 \).
to \( DF = 6 \) load cycles.

5.4.4 Measured Curvature Profile

The reason for the small average curvature increase in the moment curvature relationship mentioned above is clearly visible in the curvature profile, FIG. 5.20. During the positive load cycles to \( DF = 6 \) the curvatures at the top plastic hinge region have concentrated at the second dial gauge position and not at the beam stub-column interface. A maximum curvature ductility factor of 14 was obtained at displacement ductility factor -6.

5.4.5 Measured Bar Strain Profiles

The reason for the movement of maximum curvatures away from the beam stub-column interface during load cycles to \( DF = 6 \) is more obvious in the longitudinal bar strain profiles shown in FIGS. 5.21 and 5.22. There is a significant concentration of compression strain over the second dial gauge length at the top plastic hinge region during forward loading. Because of the higher level of axial compression on this specimen the neutral axis is now closer to the extreme tension fibre than to the extreme compression fibre, reflected in the greater compression strains than the tension strains. Strain hardening is again apparent with the maximum compression strain obtained during the test corresponding to a steel strength enhancement of about 42% above the actual yield strength.

5.4.6 Measured Confining Steel Strains

The disappointing behaviour of the electric strain gauges was still apparent in this specimen. The visual observation of the good behaviour was substantiated by those gauges that did give consistent readings. As before, yielding had occurred at several gauge po-
FIG. 5.20 "CURVATURE PROFILE" SPECIMEN THREE
FIG. 5.22 "BAR STRAIN PROFILE" SPECIMEN THREE REVERSE LOADING
sitions but appeared not to be extensive. A maximum strain of 6400 microstrains was measured, yield strain being 1400 microstrains and strain hardening at 12460 microstrains.

5.5 SPECIMEN FOUR

Actual properties:

\[ f'_c = 23.5 \text{ MPa} \]
\[ f'_{\text{c}} A_g = 4265 \text{ kN} \]
\[ f'_{\text{c}} A_g \text{ Long. steel} \]
\[ \rho_t = 0.0179 \text{ with } f_y = 375 \text{ MPa} \]
\[ \text{Hoops } = R 12/62 \text{ with } \]
\[ f_{\text{yh}} = 294 \text{ MPa} \]

5.5.1 YIELD DISPLACEMENT AND CURVATURE

The theoretical ultimate moment of resistance was calculated to be 597.7 kN.m, with a corresponding theoretical ultimate mid-height lateral load of 996 kN.

The extrapolation procedure to obtain the yield displacement, \( \delta_y \), of 2.5 mm is shown in FIG. 5.23. Yield curvature, \( \phi_y \), was established at \( 6 \times 10^{-6} \text{/mm} \).

5.5.2 GENERAL DESCRIPTION OF THE BEHAVIOUR OF THE COLUMN DURING TESTING

Elastic cycle (Cycle 1)

No cracks were observed during positive loading (west side tension) but one crack at the beam stub-column interface at both the top and bottom plastic hinge regions during negative loading was noted.

DF = 2 (Cycles 2 and 3)

At the completion of cycle 3 the cracks on the west side numbered five and three at the top and bottom plastic hinge regions respectively with two only at both the top and bottom plastic hinge regions on the east side. Indications of spalling of the cover concrete at the corners was apparent.
FIG. 5.23 Yield displacement - specimen Four.
DP = 4 (Cycles 4 and 5)
Although there was some new crack formation and extension of initial cracks, major spalling of cover concrete was the dominant feature.

DP = 6 (Cycles 6, 7 and 8)
Spalling of the cover concrete continued to dominate although several vertical cracks near the ends of the member were observed. Spalling back to the longitudinal bars extended for about 500 mm from the beam stub. As for all the previous specimens the concrete core appeared to be excellently confined with no buckling of either the confining transverse or longitudinal steel evident.

FIG. 5.24 shows the condition of the column at various stages of testing.

5.5.3 MEASURED LOAD-DISPLACEMENT AND MOMENT-CURVATURE RELATIONSHIPS
FIGS. 5.25 and 5.26 are the total mid-height lateral load–average mid-height displacement and moment-curvature relationships respectively obtained from the test. The moment plotted is that at the face of the beam stub, but because of concentration of curvatures away from the beam stub during load cycles to displacement ductility factor 6, the curvature plotted is that obtained from the average of the first dial gauges up to cycle 5 and the average of the first two dial gauges for subsequent cycles. This means that the moment-curvature relationship for cycles 6, 7 and 8 is not totally consistent with the earlier cycles but is accurate enough for the purpose of this study. The loss of cover concrete does not seem to have significantly affected the moment capacity of the column. A comparison between theoretical and experimental moments at each
FIG. 5.24b Testing of specimen Four.
cycle peak is shown in TABLE 5.4. The large increase over the theoretical ultimate moment is further evidence of the belief of the conservative nature of the design charts [36]. Strength and stiffness degradation is again small.

5.5.4 MEASURED CURVATURE PROFILE

The curvature profile shown in FIG. 5.27 clearly illustrates the movement of peak curvatures away from the beam stub-column interface at load cycles to DF = 6. A maximum curvature ductility factor of 16 was obtained at displacement ductility factor -6.

5.5.5 MEASURED BAR STRAIN PROFILES

The longitudinal bar strain profiles shown in FIGS. 5.28 and 5.29 again indicate the concentration of hinging action at the second dial gauge positions during load cycles to DF = 6. The significantly greater compression strains than tension strains indicate that the neutral axis depth is now much closer to the extreme tension fibre than to the extreme compression fibre as would be expected for a column with such a high level of axial compression. Nowhere has strain hardening occurred in the tension steel but the maximum level of strain hardening in the compression steel corresponds to a steel strength enhancement of about 45% above the actual yield strength.

5.5.6 MEASURED CONFINING STEEL STRAINS

The type of waterproofing and mechanical protection used on the electric strain gauges in this specimen seemed to be more effective than the type used in the other three specimens, as the incidence of total gauge failure was less and far more gauges gave consistent results. As might be intuitively expected the confining hoops in this specimen (with a high level of axial
FIG. 5.25 LOAD DISPLACEMENT - SPECIMEN FOUR
FIG. 526 MOMENT - CURVATURE - SPECIMEN FOUR
### TABLE 5.4 Comparison of theoretical ultimate and experimental maximum moments—Specimen Four.

<table>
<thead>
<tr>
<th>Displacement ductility factor</th>
<th>Cycle</th>
<th>Forward Loading</th>
<th>Reverse Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum moment (kN m)</td>
<td>Increase over theoretical moment (%)</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>808.9</td>
<td>35.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>764.3</td>
<td>28</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>875.4</td>
<td>46</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>839.7</td>
<td>40.5</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>910.7</td>
<td>52.5</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>853.0</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>856.4</td>
<td>43.5</td>
</tr>
</tbody>
</table>

* Based on Ref. [36] with $\phi = 1$. 
FIG. 527 CURVATURE PROFILE - SPECIMEN FOUR
FIG. 528 BAR STRAIN PROFILE  SPECIMEN FOUR  FORWARD LOADING
Fig. 5.29 Bar strain profile specimen four reverse loading
compression on the section) were subjected to greater strains than any of the previous specimens. Yielding occurred more often and a maximum strain of 8600 microstrains was recorded, only slightly less than the strain at which strain hardening commences, 8960 microstrains. Yield strain was 1400 microstrains.
SECTION SIX

ANALYSIS OF TEST RESULTS FROM SPECIMENS

6.1 SUMMARY

Comparisons can be made between the experimental data and various analytical methods that have been proposed to predict the response of members subjected to seismic loading. This section contains comparisons of the estimation of ductility, plastic hinge length, concrete spalling and maximum compression strains, and moment capacity.

6.2 SPECIMEN ONE

6.2.1 COMPARISON WITH MWD ESTIMATION OF DUCTILITY

The design charts in the MWD's publication CDP 810/A [38] give a method for the calculation of ductilities of rectangular columns. These MWD designed charts have adopted for ultimate curvature calculations the stress-strain curve for confined concrete proposed by Kent and Park [12], but with a limiting ultimate concrete strain $\varepsilon_{cu}$ using a modified form of Baker's equation for $\varepsilon_{cu}$ [39]. Thus the full extent of the stress-strain curve for confined concrete was not utilised in the derivation charts. To calculate the ultimate plastic displacement the equivalent plastic hinge length proposed by Baker [39] was incorporated in the charts. It is of interest to determine the accuracy of the charts by comparison with the measured test data. The ductility of the column was calculated using these charts and compared with the maximum ductility obtained during the test. Also the amount of confining steel required by this MWD method to achieve a displacement ductility factor of 6 was calculated. The calculations are summarised in TABLE 6.1.
The column properties assumed in the calculations were:

Concrete strength \( f'_c = 23.1 \text{ MPa} \)
Cover concrete ratio for confined core \( m = 0.047 \)
Longitudinal steel ratio \( \rho_t = 0.0179 \)
Axial load \( P_e = 1815 \text{ kN} \)
Core dimension \( b', t' = 470 \text{ mm} \)

6.2.2 **PLASTIC HINGE LENGTH ESTIMATION**

An estimate of the equivalent plastic hinge length at the various ductility factors was obtained from the measured curvatures.

With reference to **FIG. 6.1** the plastic displacement, \( \delta_p \), is the moment of the crosshatched portion of the curvature diagram between A and B about A..

\[
\delta_p = \phi_p \frac{L_p (L - \frac{L_p}{2})}{2} = (\phi_u - \phi_y) \frac{L_p (L - \frac{L_p}{2})}{2}
\]  

(13)

From the known plastic displacement \( \delta_p \), and the plastic curvature \( \phi_p \), the plastic hinge length can be calculated by solving Eq. (13) as a quadratic in \( L_p \).
**TABLE 6.1 Ductility calculations - Specimen One**

| | CDP 870/A |  |
|---|---|---|---|---|
| | \( \rho_s = 0.01 \) | \( \rho_s = 0.03 \) | interpolated \( \rho_s = 0.015 \) | experimental \( \rho_s = 0.015 \) |
| \( \frac{P_e}{f'_c b't'} \) | 0.356 | 0.356 |  |  |
| \( K_u \) | 0.46 | 0.44 | 0.455 | 0.33\* |
| c/d | 0.555 | 0.525 |  |  |
| \( \varepsilon_{cu} \) | 0.0075 | 0.0133 | 0.0088 | 0.013\* |
| \( \phi_u = \frac{\varepsilon_{cu}}{K_u t'} \) | 3.47 | 6.43 | 4.21 | 13.16 \* |
| \( (x10^{-5}/\text{mm}) \) |  |  |  |  |
| \( \frac{P_e}{f'_y b't'} \) | 0.022 | 0.022 |  |  |
| \( K_y \) | 0.45 | 0.45 |  |  |
| \( \phi_y = \frac{\varepsilon_y}{(1-K_y-m)t'} \) | 8.51 | 8.51 |  |  |
| \( (x10^{-5}/\text{mm}) \) |  |  |  |  |
| \( L_p = 8K_1K_2c/dL \) | 360 mm | 340 mm |  |  |
| \( \theta_p = (\phi_u - \phi_y) L_p \) | 0.0094 | 0.0190 |  |  |
| \( \delta_p = \theta_p (L - \frac{L_p}{2}) \) | 9.61 mm | 19.55 mm | 12.10 mm | 28.3 mm* |

* Measured at peak of load cycle 7.

**Note:**

1) Based on the CDP 870/A's chart values for \( E_{cr} \) and \( K_u = 691.2 \text{ kN.m} \), a yield displacement of 6.0 mm is obtained (compared with the 5.7 mm obtained experimentally).

2) To achieve a plastic displacement of 28.3 mm, CDP 870/A requires \( \rho_s = 0.048 \), equivalent to hoops of R 16/64 (compared with R 10/80 by DZ 3101), compared with the \( \rho_s \) of 0.015 used in the test.
The plastic curvature over the first 100 mm from the beam stub was used as being representative of the curvature over the hinge length. It is recognised that some error will be involved by using this approximation but in lieu of a more sophisticated analysis it is felt that a reasonable estimate of \( L_p \) will be obtained. A comparison is made with the value obtained from the following expression due to Baker [39]:

\[
L_p = 0.8 k_1 k_3 c/d L
\]  

(14)

where \( k_1 \) = parameter for influence of type of steel  
\( (0.7 \text{ for mild steel, } 0.9 \text{ for cold worked steel}) \)  
\( k_2 \) = parameter for influence of grade of concrete  
\( c \) = neutral axis depth at ultimate  
\( d \) = effective depth of member  
\( L \) = distance of critical section to the point of contraflexure

TABLE 6.2 summarises the results and it is evident that in the load cycles to high ductility factors Baker's \( L_p \) value was approached.

**TABLE 6.2 Plastic hinge length calculations - Specimen One**

<table>
<thead>
<tr>
<th>Structure ductility</th>
<th>( \delta_p = \delta - \delta_y ) (mm)</th>
<th>( \phi_p = \phi - \phi_y ) (x10^-6 /mm)</th>
<th>( L_p ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>calculated from Eq.(13)</td>
<td>Baker from Eq.(14)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5.31</td>
<td>28.65</td>
<td>166</td>
</tr>
<tr>
<td>4</td>
<td>17.03</td>
<td>59.48</td>
<td>269</td>
</tr>
<tr>
<td>6</td>
<td>28.11</td>
<td>109.75</td>
<td>237</td>
</tr>
<tr>
<td>-2</td>
<td>5.61</td>
<td>29.13</td>
<td>173</td>
</tr>
<tr>
<td>-4</td>
<td>17.1</td>
<td>63.25</td>
<td>252</td>
</tr>
<tr>
<td>-6</td>
<td>28.3</td>
<td>108.25</td>
<td>242</td>
</tr>
</tbody>
</table>

Note: \( L_p = 253 \text{ mm} \) represents \( L_p = 0.46h \), where \( h \) = column overall depth.
6.2.3 Concrete Spalling Strain

An estimate of the concrete spalling strain of the cover concrete was obtained by observing when spalling was first noticeable and noting the load at which it occurred and the corresponding strain. The spalling strain was estimated to be about 0.005.

6.2.4 Maximum Measured Concrete Compression Strain

A comparison is made between the maximum concrete compression strain measured at the surface of the confined concrete core obtained from the test (measured at DF = 6) and the theoretical ultimate compression strain obtained from equations from Baker [39], Corley [40], and CDP 810/A.

Baker's expression is

\[ \varepsilon_{cu} = 0.0015 \left[ 1 + 150 \rho_s + (0.7 - 10 \rho_s) \frac{d}{c} \right] \]  \hspace{1cm} (15)

Corley's expression is

\[ \varepsilon_{cu} = 0.003 + 0.02 \frac{h}{L} + \left( \frac{\rho_s f_y h}{138} \right)^2 \]  \hspace{1cm} (16)

CDP 810/A (Based on Baker's method) expression is

\[ \varepsilon_{cu} = 0.0021 \left[ 1 + 150 \rho_s + (0.7 - 10 \rho_s) \frac{d}{c} \right] \]  \hspace{1cm} (17)

Table 6.3 shows the comparison and indicates that a much greater concrete compressive strain was actually available for the confined concrete than predicted by any of the three equations.

<table>
<thead>
<tr>
<th>Method</th>
<th>( \varepsilon_{cu} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>0.016</td>
</tr>
<tr>
<td>Baker, Eq. (15)</td>
<td>0.007</td>
</tr>
<tr>
<td>Corley, Eq. (16)</td>
<td>0.013</td>
</tr>
<tr>
<td>CDP 810/A, Eq. (17)</td>
<td>0.010</td>
</tr>
</tbody>
</table>
6.2.5 **Comparison of Analytical Confinement Models**

Several analytical models have been developed to predict the stress-strain behaviour of concrete confined by rectangular hoops when subjected to axial compression. Although most models were based on experimental data from monotonically increasing axial compression tests on relatively small size specimens, it is often claimed that the stress-strain curves presented realistically define the envelope curve for cyclic compression simulating seismic action.

In order to test this hypothesis the models suggested by Kent and Park [12], Valienas, Bertero and Popov [13], and Sheikh and Uzumeri [14], were applied to the specimens tested.

To calculate the moment capacity at a relatively high curvature, an extreme fibre concrete compressive strain for the concrete core of 0.005 was assumed. Using an iterative procedure the neutral axis depth required for the internal forces to balance the external applied load was calculated. Cover concrete was assumed to be effective up to a strain of 0.004, and to be lost at higher strains. Taking moments of the internal forces about the centre line gave the predicted moment of resistance corresponding to the extreme fibre core strain of 0.005. These moments were then compared with the experimental moment corresponding to a extreme fibre core strain of 0.005.

The details of the models applied are given below.

**Kent and Park**

see section 2.6.

**Valienas, Bertero and Popov**

Similar in form to Kent and Park's model, it does however consider concrete stress enhancement to be applicable. With reference to FIG. 6.2 the relevant details are:
FIG. 6.2 Idealised stress-strain curve for confined concrete [13].
Region AB.

\( \varepsilon_c \leq \varepsilon_o \)

\[
\frac{f_c}{f'_{c}} = \frac{\frac{E_c}{f'_{c}} \frac{\varepsilon_c}{\varepsilon_o} \left( \frac{\varepsilon_c}{\varepsilon_o} \right) - k \left( \frac{\varepsilon_c}{\varepsilon_o} \right)^2}{1 + \left[ \frac{E_c}{k f'_{c}} \right] - 2 \left( \frac{\varepsilon_c}{\varepsilon_o} \right)}
\]

(18)

Region BC.

\( \varepsilon_o \leq \varepsilon_c \leq \varepsilon_{.3k} \)

\[
\frac{f_c}{f'_{c}} = k \left[ 1 - 2 \varepsilon_o \left( \frac{\varepsilon_c}{\varepsilon_o} - 1 \right) \right]
\]

(19)

Region CD.

\( \varepsilon_{.3k} \leq \varepsilon_c \)

\[
\frac{f_c}{f'_{c}} = 0.3k
\]

(20)

where \( \varepsilon_o = 0.0024 + 0.006 \left( 1 - \frac{0.734 s}{h''} \right) \frac{\rho_s f_{yh}}{\sqrt{f'_{c}}} \)

(21)

\[
k = 1 + 0.1096 \left( 1 - \frac{0.245 s}{h''} \right) \left( \frac{\rho_s + \frac{d'''}{d''} \frac{d'^{}}{d^'} f_{yh}}{\sqrt{f'_{c}}} \right)
\]

(22)

The above equations in S.I. units were converted from
the original equations which are given in Imperial Units.

Sheikh and Uzumeri

The stress-strain curve proposed is shown in
FIG. 6.3.
FIG. 6.3 Idealised stress-strain curve for confined concrete.\[14\]

The governing equations are:

\[ K_s = 1.0 + \frac{b^2}{140P_{occ}} \left[ \left( 1 - \frac{nc_{c}^2}{5.5b^2} \right) \left( 1 - \frac{0.5s}{b} \right)^2 \right] \sqrt{\frac{f_{c}'}{s}} \] \hfill (23)

\[ \varepsilon_{s1} = 80K_s f'_{c} \times 10^{-6} \] \hfill (24)

\[ \frac{\varepsilon_{s2}}{\varepsilon_{00}} = 1.0 + \frac{248}{c} \left( 1 - 5.0(s/b)^2 \right) \frac{\rho_{s} f'_{s}}{\sqrt{f'_{c}}} \] \hfill (25)

\[ \varepsilon_{s85} = \varepsilon_{s2} + 0.225 \rho_{s} \sqrt{b/s} \] \hfill (26)

where \( P_{occ} \) is in kN, \( f'_{c} \) and \( f'_{s} \) in MPa and linear dimensions are in mm.

The results of the comparison are summarised in TABLE 6.2. The three predicted stress-strain curves for this column are shown in FIG. 6.4. Calculation details are given in Appendix C. For discussion of results see section 7.
FIG. 6.4 Concrete stress-strain curves—specimen One.
Table 6.4 Comparison of analytical confinement models - Specimen One.

<table>
<thead>
<tr>
<th>Method</th>
<th>Moment at $\varepsilon_c = 0.005$ (kNm)</th>
<th>Neutral axis depth at $\varepsilon_c = 0.005$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>630</td>
<td>169</td>
</tr>
<tr>
<td>Kent and Park</td>
<td>605</td>
<td>196</td>
</tr>
<tr>
<td>Vallenlas et al</td>
<td>629</td>
<td>205</td>
</tr>
<tr>
<td>Sheikh and Uzumeri</td>
<td>651</td>
<td>165</td>
</tr>
</tbody>
</table>

6.3 Specimen Two

6.3.1 Comparison with MWD Estimation of Ductility

CDP 810/A calculations of the ductility capacity for specimen Two are summarised in Table 6.5. The column properties assumed in the calculations are:

- Concrete strength $f'_c = 41.4$ MPa
- Cover concrete ratio for confined core $m = 0.051$
- Longitudinal steel ratio $\rho_t = 0.0179$
- Axial load $P_e = 2680$ kN
- Core dimension $b', t' = 474$ mm

6.3.2 Plastic Hinge Length Estimation

The results of the plastic hinge length comparison calculations, using the method outlined in section 6.2.2, are given in Table 6.6

6.3.3 Concrete Spalling Strain

Observation of first concrete spalling established the concrete spalling strain to be about 0.005.

6.3.4 Maximum Measured Concrete Compression Strain

The comparison of maximum concrete compressive strain measured at the surface of the confined concrete
Table 6.5 Ductility calculations - Specimen Two.

<table>
<thead>
<tr>
<th></th>
<th>CDP 810/A</th>
<th></th>
<th>interpolated</th>
<th>experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\rho_s = 0.01$</td>
<td>$\rho_s = 0.03$</td>
<td>$\rho_s = 0.023$</td>
<td>$\rho_s = 0.023$</td>
</tr>
<tr>
<td>$\frac{P_e}{f'_c b't'}$</td>
<td>0.288</td>
<td>0.288</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_u$</td>
<td>0.38</td>
<td>0.37</td>
<td>0.374</td>
<td>$0.487^*$</td>
</tr>
<tr>
<td>$c/d$</td>
<td>0.475</td>
<td>0.455</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{cu}$</td>
<td>0.008</td>
<td>0.0135</td>
<td>0.0116</td>
<td>$0.027^*$</td>
</tr>
<tr>
<td>$\phi_u = \frac{\varepsilon_{cu}}{K_u t'}(x10^{-5}/\text{mm})$</td>
<td>4.44</td>
<td>7.70</td>
<td>6.59</td>
<td>$13.89^*$</td>
</tr>
<tr>
<td>$\frac{P_e}{f'_c b't'}$</td>
<td>0.0318</td>
<td>0.0318</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_y$</td>
<td>0.51</td>
<td>0.51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\phi_y = \frac{\varepsilon_y}{(1-K_y-m)t'}(x10^{-6}/\text{mm})$</td>
<td>9.59</td>
<td>9.59</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$L_p = 0.8K_1K_2dL$</td>
<td>249</td>
<td>239</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Theta_p = (\phi_u - \phi_y)L_p$</td>
<td>0.0087</td>
<td>0.0161</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\delta_p = \frac{\Theta_p (l - Ln)}{2}$</td>
<td>9.36</td>
<td>17.40</td>
<td>14.59</td>
<td>$21.29^*$</td>
</tr>
</tbody>
</table>

* Measured at the peak of load cycle 7.

Note: 1) Based on the CDP 810/A's chart values for $EI_{cr}$ and $K_u = 905.0$ kNm, a yield displacement of 5.54 mm is obtained (compared with 4.2 mm obtained experimentally).

2) To achieve a plastic displacement of 21.29 mm, CDP 810/A requires $\rho_s = 0.040$, equivalent to hoops of R 16/75 (compared with R 12/75 by DZ 3101), compared with the $\rho_s = 0.023$ used in the test.
### Table 6.6 Plastic hinge length calculations - Specimen Two

<table>
<thead>
<tr>
<th>Structure ductility</th>
<th>$\delta_p = \delta - \delta_y$ (mm)</th>
<th>$\phi_p = \phi - \phi_y$ (x10^-6/mm)</th>
<th>$L_p$ (mm) Calculated from Eq. (13)</th>
<th>Baker from Eq. (14)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.11</td>
<td>21.38</td>
<td>173</td>
<td>195</td>
</tr>
<tr>
<td>4</td>
<td>12.72</td>
<td>63.45</td>
<td>181</td>
<td>219</td>
</tr>
<tr>
<td>6</td>
<td>21.39</td>
<td>125.6</td>
<td>151</td>
<td>230</td>
</tr>
<tr>
<td>-2</td>
<td>4.10</td>
<td>18.04</td>
<td>207</td>
<td>209</td>
</tr>
<tr>
<td>-4</td>
<td>12.56</td>
<td>69.6</td>
<td>161</td>
<td>210</td>
</tr>
<tr>
<td>-6</td>
<td>21.38</td>
<td>102.3</td>
<td>189</td>
<td>218</td>
</tr>
</tbody>
</table>

Note: $L_p = 218$ mm represents $L_p = 0.4$ h, where h = overall column depth.

core (measured at DF = 6) and that calculated by the methods detailed in section 6.2.4 is given in Table 6.7.

### Table 6.7 Ultimate compression strain

<table>
<thead>
<tr>
<th>Method</th>
<th>$\varepsilon_{cu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>0.026</td>
</tr>
<tr>
<td>Baker, Eq. (15)</td>
<td>0.008</td>
</tr>
<tr>
<td>Corley, Eq. (16)</td>
<td>0.015</td>
</tr>
<tr>
<td>CDP 810/A, Eq. (17)</td>
<td>0.011</td>
</tr>
</tbody>
</table>

6.3.5 **Comparison of Analytical Confinement Models**

The comparison of the experimental and predicted moments by the three models discussed in section 6.2.5 is shown in Table 6.8.

The three predicted stress-strain curves for this column are shown in Fig. 6.5 while the calculation details are given in Appendix C.
FIG. 6.5 Concrete stress-strain curves - specimen Two.
<table>
<thead>
<tr>
<th>Method</th>
<th>Neutral axis depth at $\varepsilon_c = 0.005$</th>
<th>Moment at $\varepsilon_c = 0.005$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>166</td>
<td>810</td>
</tr>
<tr>
<td>Kent and Park</td>
<td>163</td>
<td>808</td>
</tr>
<tr>
<td>Vallenas et al.</td>
<td>180</td>
<td>792</td>
</tr>
<tr>
<td>Sheikh and Uzumeri</td>
<td>170</td>
<td>797</td>
</tr>
</tbody>
</table>
6.4 Specimen Three

6.4.1 Comparison with MWD Estimation of Ductility

CDP 810/A calculations of the ductility capacity for specimen Three are summarised in TABLE 6.9.

The column properties assumed in the calculations are:

- Concrete strength $f'_c = 21.4$ kPa
- Cover concrete ratio for confined core $m = 0.047$
- Longitudinal steel ratio $\rho_t = 0.0179$
- Axial load $P_e = 2719$ kN
- Core dimension $b', t' = 470$ mm

6.4.2 Plastic Hinge Length Estimation

TABLE 6.10 shows the comparison of plastic hinge lengths calculated by Eq. (13) and Baker's Eq. (14).

| Structure ductility | $\delta_p - \delta_y$ (mm) | $\phi_p - \phi_y$ (x10^6/mm) | $L_p$ (mm) calculated from Eq. (13) | $L_p$ from Baker
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>3.47</td>
<td>19.71</td>
<td>157</td>
<td>352</td>
</tr>
<tr>
<td>4</td>
<td>10.82</td>
<td>52.1</td>
<td>188</td>
<td>385</td>
</tr>
<tr>
<td>6</td>
<td>17.58</td>
<td>62.21</td>
<td>265</td>
<td>378</td>
</tr>
<tr>
<td>-2</td>
<td>3.70</td>
<td>21.95</td>
<td>150</td>
<td>366</td>
</tr>
<tr>
<td>-4</td>
<td>10.36</td>
<td>42.65</td>
<td>223</td>
<td>426</td>
</tr>
<tr>
<td>-6</td>
<td>17.48</td>
<td>77.45</td>
<td>206</td>
<td>414</td>
</tr>
</tbody>
</table>

Note: $L_p = 474$ mm represents $L_p = 0.75$ h, where $h =$ overall column depth.
Table 6.9 Ductility calculations - Specimen Three.

<table>
<thead>
<tr>
<th></th>
<th>CDP 810/A</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\rho = 0.01$</td>
<td>$\rho = 0.03$</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>0.007</td>
<td>0.013</td>
</tr>
<tr>
<td>$\frac{P}{f_{cp}b''t''}$</td>
<td>0.575</td>
<td>0.575</td>
</tr>
<tr>
<td>$K_u$</td>
<td>0.67</td>
<td>0.63</td>
</tr>
<tr>
<td>$c/d$</td>
<td>0.728</td>
<td>0.695</td>
</tr>
<tr>
<td>$\varepsilon_{cu}$</td>
<td>0.007</td>
<td>0.013</td>
</tr>
<tr>
<td>$\phi_u = \frac{\varepsilon_{cu}}{K_u t''}$</td>
<td>2.22</td>
<td>4.39</td>
</tr>
<tr>
<td>($x10^{-5} /mm$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\frac{P}{f_y b''t''}$</td>
<td>0.033</td>
<td>0.033</td>
</tr>
<tr>
<td>$K_y$</td>
<td>0.52</td>
<td>0.52</td>
</tr>
<tr>
<td>$\phi_y = \frac{\varepsilon_y}{(1-K_y-m)t''}$</td>
<td>9.90</td>
<td>9.90</td>
</tr>
<tr>
<td>($x10^{-5} /mm$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$L_p = 0.3K_1K_2c/dL$</td>
<td>488</td>
<td>466</td>
</tr>
<tr>
<td>$\theta_p = (\phi_u - \phi_y)L_p$</td>
<td>0.006</td>
<td>0.0158</td>
</tr>
<tr>
<td>$\delta_p = \theta_p (L - \frac{L_p}{2})$</td>
<td>5.7</td>
<td>15.3</td>
</tr>
</tbody>
</table>

* Measured at peak of load cycle 8

Note: 1) Based on the CDP 810/A's chart values for $E_{cr}$ and $w_u = 645.9$ kN/m, a yield displacement of 4.89 mm is obtained (compared with 3.5 mm obtained experimentally).

2) To achieve a plastic displacement of 17.66 mm, CDP 810/A requires $\rho_s = 0.035$, equivalent to hoops of R 12/63 (compared with R 10/75 by DZ 3101), compared with the $\rho_s = 0.0203$ used in the test.
6.4.3 **Concrete Spalling Strain**

Observation of first concrete spalling established the concrete spalling strain to be about 0.007.

6.4.4 **Maximum Measured Concrete Compression Strain**

Table 6.11 shows the comparison of maximum concrete compressive strain measured at the surface of the confined concrete core (measured at $DF = 6$) and that calculated by methods detailed in Section 6.2.4.

**Table 6.11 Ultimate compression strain.**

<table>
<thead>
<tr>
<th>Method</th>
<th>$\varepsilon_{cu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>0.018</td>
</tr>
<tr>
<td>Baker, Eq. (15)</td>
<td>0.007</td>
</tr>
<tr>
<td>Corley, Eq. (16)</td>
<td>0.014</td>
</tr>
<tr>
<td>CDP 810/A, Eq. (17)</td>
<td>0.010</td>
</tr>
</tbody>
</table>

6.4.5 **Comparison of Analytical Confinement Models**

Table 6.12 shows the comparison of the moments predicted by the models discussed in Section 6.2.5 and the experimental moment. Fig. 6.6 shows the three predicted stress-strain curves for this column and the calculation details appear in Appendix C.

**Table 6.12 Comparison of analytical confinement models—Specimen Three**

<table>
<thead>
<tr>
<th>Method</th>
<th>Neutral axis depth at $\varepsilon_c=0.005$</th>
<th>Moment at $\varepsilon_c=0.005$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>256</td>
<td>720</td>
</tr>
<tr>
<td>Kent and Park</td>
<td>264</td>
<td>639</td>
</tr>
<tr>
<td>Vallenas et al.</td>
<td>282</td>
<td>605</td>
</tr>
<tr>
<td>Sheikh and Uzumeri</td>
<td>223</td>
<td>698</td>
</tr>
</tbody>
</table>
FIG. 6.6 Concrete stress-strain curves—specimen Three.
6.5 SPECIMEN FOUR

6.5.1 COMPARISON WITH MWD ESTIMATION OF DUCTILITY

TABLE 6.13 summarises the CDP 870/A calculations of the ductility capacity for specimen Four. The column properties assumed in the calculations are:

- Concrete strength \( f'_c = 23.5 \text{ MPa} \)
- Cover concrete ratio for confined core \( m = 0.0051 \)
- Longitudinal steel ratio \( \rho_t = 0.0179 \)
- Axial load \( P_e = 4265 \text{ kN} \)
- Core dimension \( b,t' = 474 \text{ mm} \)

6.5.2 PLASTIC HINGE LENGTH ESTIMATION

The comparison of plastic hinge lengths calculated by Eq. (13) and by Baker's Eq. (14) is shown in TABLE 6.14.

<table>
<thead>
<tr>
<th>Structure Ductility</th>
<th>( \delta_p = \delta - \delta_y ) (mm)</th>
<th>( \phi_p = \phi - \phi_y ) (x10^-6/mm)</th>
<th>( L_p ) (mm)</th>
<th>Calculated from Eq. (13)</th>
<th>Baker from Eq. (14)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2.57</td>
<td>13.24</td>
<td>174</td>
<td>380</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>7.86</td>
<td>23.7</td>
<td>319</td>
<td>427</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>13.0</td>
<td>30.3</td>
<td>437</td>
<td>508</td>
<td></td>
</tr>
<tr>
<td>-2</td>
<td>2.68</td>
<td>13.70</td>
<td>176</td>
<td>423</td>
<td></td>
</tr>
<tr>
<td>-4</td>
<td>7.6</td>
<td>26.7</td>
<td>267</td>
<td>462</td>
<td></td>
</tr>
<tr>
<td>-6</td>
<td>12.9</td>
<td>45.4</td>
<td>266</td>
<td>525</td>
<td></td>
</tr>
</tbody>
</table>

Note: \( L_p = 525 \text{ mm} \) represents \( L_p = 0.95 \text{ h} \), where \( h \) = overall column depth.

6.5.3 CONCRETE SPALLING STRAIN

The concrete spalling strain was established to be about 0.005.
<table>
<thead>
<tr>
<th></th>
<th>CDP 810/A</th>
<th></th>
<th></th>
<th>Experimental</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\rho_s = 0.01$</td>
<td>$\rho_s = 0.03$</td>
<td>Extrapolated $\rho_s = 0.0349$</td>
<td>$\rho_s = 0.0349$</td>
<td></td>
</tr>
<tr>
<td>$\frac{P_e}{f'_c b't'}$</td>
<td>0.807</td>
<td>0.807</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_u$</td>
<td>0.89</td>
<td>0.86</td>
<td>0.85</td>
<td>0.84*</td>
<td></td>
</tr>
<tr>
<td>$c/d$</td>
<td>0.88</td>
<td>0.85</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_{cu}$</td>
<td>0.0067</td>
<td>0.0125</td>
<td>0.0139</td>
<td>0.020*</td>
<td></td>
</tr>
<tr>
<td>$\phi_u = \frac{E_{cu}}{K_u t'}$</td>
<td>1.59</td>
<td>3.07</td>
<td>3.43</td>
<td>6.06*</td>
<td></td>
</tr>
<tr>
<td>($x10^{-5}$/mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\frac{P_e}{f'_y b't'}$</td>
<td>0.051</td>
<td>0.051</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_y$</td>
<td>0.62</td>
<td>0.62</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\phi_y = \frac{E_y}{(1-K_y-m)t'}$</td>
<td>1.28</td>
<td>1.28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>($x10^{-5}$/mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$L_p = 0.8 K_y k_y c/dL$</td>
<td>568</td>
<td>548</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\delta_p = (\phi_u - \phi_y) L_p$</td>
<td>0.00176</td>
<td>0.0098</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\delta_p = \phi_p \left( \frac{L-L_p}{2} \right)$</td>
<td>1.61</td>
<td>9.08</td>
<td>10.91</td>
<td>13.00*</td>
<td></td>
</tr>
</tbody>
</table>

* measured at peak of load cycle 8.

Note: 1) Based on the CDP 810/A's chart values for $E I_c r$ and $m_u = 597.7\ kN m$ a yield displacement of 5.00 mm is obtained (compared with 2.5 mm obtained experimentally).

2) To achieve a plastic displacement of 13.00 mm, CDP 810/A requires $\rho_s = 0.0405$, equivalent to hoops of R 12/54 (compared with R 12/62 by D2 3101), compared with the $\rho_s = 0.0349$ used in the test.
6.5.4 Maximum Measured Concrete Compression Strain

The comparison of maximum concrete compressive strain measured at the surface of the confined concrete core (measured at $DF = 6$) and that calculated by the methods detailed in Section 6.2.4 is shown in TABLE 6.15.

<table>
<thead>
<tr>
<th>Method</th>
<th>$\varepsilon_{cu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>0.020</td>
</tr>
<tr>
<td>Baker, Eq. (15)</td>
<td>0.010</td>
</tr>
<tr>
<td>Corley, Eq. (16)</td>
<td>0.018</td>
</tr>
<tr>
<td>CDP 810/A. Eq. (17)</td>
<td>0.014</td>
</tr>
</tbody>
</table>

6.5.5 Comparison of Analytical Confinement Models

The experimental and predicted moment comparison in Section 6.2.5 is shown in TABLE 6.16. The three predicted stress-strain curves for this column are shown in FIG. 6.7 with the calculation details summarised in Appendix C.

<table>
<thead>
<tr>
<th>Method</th>
<th>Neutral axis depth at $\varepsilon_c = 0.005$ (mm)</th>
<th>Moment at $\varepsilon_c = 0.005$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>250</td>
<td>810</td>
</tr>
<tr>
<td>Kent and Park</td>
<td>345</td>
<td>639</td>
</tr>
<tr>
<td>Vallenas et al.</td>
<td>370</td>
<td>527</td>
</tr>
<tr>
<td>Sheikh and Uzumeri</td>
<td>280</td>
<td>780</td>
</tr>
</tbody>
</table>
FIG. 6.7 Concrete stress-strain curves - specimen Four.
SECTION SEVEN

DISCUSSION, CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

7.1 DISCUSSION OF RESULTS AND ANALYSES

Discussion of results has already been undertaken in the individual subsections for each specimen in Section Five. The discussion here is intended as a general one, considering all four specimens.

7.1.1 MEASURED LATERAL LOAD-DISPLACEMENT AND MOMENT-CURVATURE RELATIONSHIP

All specimens exhibited excellent hysteretic lateral load displacement and moment curvature relationships. Except for specimen Two (high concrete strength) the measured peak lateral loads and calculated moments sustained by the sections significantly exceeded the ultimate actions predicted by ACI based methods. The greater the level of axial load on the specimen the greater was the increase from predicted to measured lateral load and moment, pointing to the more conservative nature of ACI based methods at higher levels of axial load. Even when there was sudden spalling of cover concrete the reduction in section resistance was relatively small and recoverable. Strength degradation during repeated load cycles was not of major proportions.

All specimens had been confined using the approach recommended in DZ 3101 [4]. The excellent behaviour of the specimens up to the displacement ductility factor of six applied gives further justification for the DZ 3101 recommendations for confining steel for axial load level between 0.1 and 0.6 $f'_c A_g$. 
7.1.2 MEASURED CURVATURE AND LONGITU DINAL BAR STRAIN PROFILES

A distinct spreading of plastic hinging action was noticeable in the specimens with the higher levels of axial load, reflected in the curvature and longitudinal bar strain profiles.

The benefits of using grade 380 steel for the longitudinal bars in columns because of its 'early' strain hardening characteristics are easily identifiable. The steel strength enhancement considerably offsets the loss of strength due to any loss of cover concrete. Prevention of longitudinal bar buckling was obtained in all specimens.

7.1.3 CDP 810/A DUCTILITY CALCULATIONS

Without exception the measured available ductility significantly exceeded that predicted by CDP 810/A [38]. Although testing was stopped after either two or three full cycles to structure ductility six it was apparent that much higher structure ductilities could have been reached. The under prediction of available ductility by CDP 810/A is due mainly to the unrealistically low maximum extreme fibre concrete strain adopted for the calculations.

7.1.4 PLASTIC HINGE LENGTH ESTIMATION

At the lower levels of axial load there were no discernible trends in the plastic hinge length as determined from first principles. Although Baker's method and intuition, for the calculation of the plastic hinge length, predicts and increase in plastic hinge length with increase in ductility factor this was not born out by the experimental results for the specimens with lower axial load. There was reasonable agreement between first principles and Baker's method. At the higher levels of axial load both methods showed in-
creasing plastic hinge length with ductility factor, but Baker's method significantly overestimated the actual length.

7.1.5 CONCRETE SPALLING STRAINS AND MAXIMUM MEASURED COMPRESSION STRAINS

The measured values exceeded those traditionally assumed in design and analysis of reinforced concrete columns. Cover concrete spalling strain was in the range 0.005 to 0.007 while a maximum extreme fibre concrete compression strain of about 0.020 was measured. It is believed that a strain greater than this could have been attained if testing had proceeded to a higher structure ductility.

7.1.6 COMPARISON OF ANALYTICAL CONFINED CONCRETE STRESS-STRAIN MODELS

The accuracy of three analytical models for the stress-strain relationship of rectangular confined concrete column sections was tested by comparing the moment of resistance predicted by each of the models with the experimental values obtained for an extreme concrete core strain of 0.005. The models investigated were those due to Kent and Park, Vallenas, Bertero and Popov, and Sheikh and Uzumeri. A clear trend was observed. At the lower levels of axial load all three models gave approximately the same results, the predicted moments being 8-10% less than the experimental moments, but at the higher levels of axial load the discrepancy between the models was significant. While all three models underestimated the moment of resistance the amount by which they did was vastly different. At the highest level of axial load (0.6 $f'_{c,A}$) Sheikh and Uzumeri underestimated by 4%, Kent and Park by 21%, and Vallenas, Bertero and Popov by 55%. At an axial load of 0.42 $f'_{c,A}$ the same relative order of accuracy was maintained, the
percentage underestimations being 5, 11 and 16 respectively. It is felt that these results are an indication of the way in which the models were derived, from data from compression tests only on confined column sections.

7.2 CONCLUSIONS
(a) The amount of confining steel required by DZ 3101[4], which modifies the SEAOC recommendations according to the level of axial load, appears from this limited experimental study to result in excellently confined rectangular reinforced concrete columns. Good ductile behaviour over the whole intended range of axial loads (0.1 to 0.6 \( f'_c A_g \)) was exhibited. NZS 4203 : 1976[5] recommendations for a maximum of 20% lateral load carrying ability loss after eight load reversals to a structure ductility of 4 are easily attainable.

(b) Existing methods for predicting the ultimate concrete compressive strain, of column sections anyway, are unrealistic and unduly conservative, except for that of Corley[40] which approaches the values measured for the specimens with higher axial load levels. This conservatism leads to underestimation of the available structure ductility as predicted by CDP 810/A which utilises a modified Baker estimation of ultimate concrete compressive strain. It is suggested that Corley's approximation for \( E_{cu} \) be used in conjunction with the CDP 810/A charts.

(c) Grade 380 steel should be used as the longitudinal reinforcing in columns because of the compensation it provides for loss of the cover concrete, by virtue of its early strain hardening, and corresponding enhanced ductile performance.

(d) The level of axial load should be considered when applying existing analytical models
for the stress-strain behaviour of rectangular confined concrete sections. For levels of axial load
\[
\frac{\bar{f}_c A}{A_g} \leq 0.3
\]
any of the three models investigated in this study can be applied with confidence and reasonable accuracy. For levels greater than 0.3 it is recommended that the method by Sheikh and Uzumeri is used.

7.3 RECOMMENDATIONS FOR FUTURE RESEARCH

(a) more full size experimental tests to validate the conclusions reached on the limited number of specimens tested in this study.

(b) Possible inclusion of an axial load level modification in the analytical models for the prediction of the stress-strain behaviour of confined concrete sections so they can confidently be applied to flexure-axial load situations.
APPENDIX A

REFERENCES


2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete, ACI 318-63, American Concrete Institute, 1963


6. ACI Committee 318, "Building Code Requirements for Reinforced Concrete", ACI 318-71, American Concrete Institute, 1971

7. ACI Committee 318, "Building Code Requirements for Reinforced Concrete", ACI 318-77, American Concrete Institute, 1977


13. Vallenas, J. Bertero, V. V. and Popov, E.P., "Concrete Confined by Rectangular Hoops Subjected to Axial Loads", Report No. UCB / EER C - 77/13, Earthquake Engineering Research Centre, University of California, 1977


22. Park, R. and Norton, J.A., "Effects of Confining Reinforcement on the Flexural Ductility of Rectangular Reinforced Concrete Column Sections with High Strength Steel", Symposium on Design and Safety of Reinforced Concrete Compression Members, International Association for Bridge and Structural Engineering, Quebec, 1974

23. Park, R. and Sampson, R. A., "Ductility of Reinforced Concrete Column Sections in Seismic Design", Journal of American Concrete Institute, Vol. 69, No. 9, September 1972


27. Nielsen, N. N. and Nakagawa, K., "The Tokachi-Oki Earthquake, Japan, May 16, 1968; a preliminary report on damage to structures", International Institute of Seismology and Earthquake Engineering, Tokyo, 1968


32. Park, K., "Ductility of Reinforced Concrete Frames Under Seismic Loading", New Zealand Engineering, Vol. 23, No. 11, November 1968


35. SEAOC, "Recommended Lateral Force Requirements and Commentary", Seismology Committee, Structural Engineers' Association of California, San Francisco, 1973


37. ACI Committee 340, "Ultimate Strength Design of Reinforced Concrete Columns", Special Publication No. 7, American Concrete Institute, Detroit, 1964

38. "Ductility of Bridges with Reinforced Concrete Piers", Civil Division Publication, Ministry of Works and Development, CDP 810/A, April 1975


GENERAL VIEW OF TEST AREA
## APPENDIX C

### CALCULATION SUMMARY FOR ANALYTICAL CONFINEMENT MODELS IN SECTION SIX

<table>
<thead>
<tr>
<th>SPECIMEN NUMBER</th>
<th>METHOD</th>
<th>Pe (kN)</th>
<th>C_c (kN)</th>
<th>C_s (kN)</th>
<th>T (kN)</th>
<th>N.A (mm)</th>
<th>M (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ONE</td>
<td>KENT AND PARK</td>
<td>1815</td>
<td>2017</td>
<td>815</td>
<td>1017</td>
<td>196</td>
<td>625</td>
</tr>
<tr>
<td></td>
<td>VALLENAS ET AL.</td>
<td>1815</td>
<td>1992</td>
<td>840</td>
<td>1017</td>
<td>205</td>
<td>629</td>
</tr>
<tr>
<td></td>
<td>SHEIKJ &amp; UZUMERI</td>
<td>1815</td>
<td>2124</td>
<td>708</td>
<td>1017</td>
<td>165</td>
<td>651</td>
</tr>
<tr>
<td>TWO</td>
<td>KENT AND PARK</td>
<td>2680</td>
<td>3024</td>
<td>678</td>
<td>1022</td>
<td>163</td>
<td>808</td>
</tr>
<tr>
<td></td>
<td>VALLENAS ET AL.</td>
<td>2680</td>
<td>2950</td>
<td>747</td>
<td>1017</td>
<td>180</td>
<td>792</td>
</tr>
<tr>
<td></td>
<td>SHEIKJ &amp; UZUMERI</td>
<td>2680</td>
<td>3014</td>
<td>683</td>
<td>1017</td>
<td>170</td>
<td>797</td>
</tr>
<tr>
<td>THREE</td>
<td>KENT AND PARK</td>
<td>2719</td>
<td>2538</td>
<td>993</td>
<td>812</td>
<td>264</td>
<td>639</td>
</tr>
<tr>
<td></td>
<td>VALLENAS ET AL.</td>
<td>2719</td>
<td>2472</td>
<td>1017</td>
<td>770</td>
<td>276</td>
<td>605</td>
</tr>
<tr>
<td></td>
<td>SHEIKJ &amp; UZUMERI</td>
<td>2719</td>
<td>2774</td>
<td>919</td>
<td>974</td>
<td>223</td>
<td>698</td>
</tr>
<tr>
<td>FOUR</td>
<td>KENT AND PARK</td>
<td>4265</td>
<td>3679</td>
<td>1124</td>
<td>538</td>
<td>345</td>
<td>639</td>
</tr>
<tr>
<td></td>
<td>VALLENAS ET AL.</td>
<td>4265</td>
<td>3482</td>
<td>1150</td>
<td>367</td>
<td>370</td>
<td>527</td>
</tr>
<tr>
<td></td>
<td>SHEIKJ &amp; UZUMERI</td>
<td>4265</td>
<td>3998</td>
<td>1012</td>
<td>745</td>
<td>280</td>
<td>780</td>
</tr>
</tbody>
</table>

**Pe** = Axial compression on column

**C_c** = Total calculated concrete compressive force for extreme core strain of 0.005 (cover concrete assumed effective up to strain 0.004)

**C_s** = Total calculated force in longitudinal bars in compression

**T** = Total calculated force in longitudinal bars in tension

**Pe + T = C_c + C_s**
Classn:

DUCTILITY OF RECTANGULAR REINFORCED CONCRETE COLUMNS WITH AXIAL LOAD.

Wayne Douglas Gill

ABSTRACT:
An experimental investigation into the post-elastic ductile behaviour of reinforced concrete columns. Results are presented for four columns with different levels of axial load. Conclusions are made about the effectiveness of the confinement provided by DZ 3101.