Unsymmetrically Reinforced Concrete Columns Under Small Eccentricity of Loading

BY

Engineer Valentine Eziefule Ezetah
(B. Eng Civil, M. Eng Struct, MNSE, CE COREN)

A Dissertation Submitted to The Postgraduate School, Ahmadu Bello University, In Fulfillment For The Requirements For The Degree of Ph.D (Structures)

Department of Civil Engineering
Faculty of Engineering
Ahmadu Bello University, Zaria

May, 1991

Dedicated To Genuine

Quest For Knowledge

CERTIFICATION

This thesis entitled "UNSYMMETRICALLY REINFORCED CONCRETE COLUMNS UNDER SMALL ECCENTRICITY OF LOADING" by Engr. Valentine Eziefule Ezetah meets the regulations governing the award of the Degree of Ph.D (Structures) of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

Professor A. O. Abatan
Internal Examiner — Date: 15/7/91

Professor O. Adebisi
Internal Examiner — Date: 10.07.91

Dr. S. P. Bjeh
Internal Examiner — Date: 11th July, 1991

Dr. K. J. Osinubi
Internal Examiner — Date: 10.7.91

Dr. O. M. Sadiq
Internal Examiner — Date: 10.07.91

Professor O. Adebisi
Head of Department — Date: 10.07.91

Dean,
Postgraduate School — Date: 20/3/92

D E C L A R A T I O N

I hereby declare that this thesis
has been written by me and that it
is a record of my own research work.
It has not been presented in any
previous application for higher degree.

All sources of information are
acknowledged by means of references.



VALENTINE EZIEFULE EZETAH

ACKNOWLEDGEMENT

It is usual in works of this nature to acknowledge the goodwill of some of the many people who contributed to the successful completion of the thesis. My first thanks go to my Supervisors Professors A.O. Abatan and Adebisi for their guidance. For sponsoring this work, am grateful to Abubkar Tafawa Balewa University Bauchi. I thank Professor Buba Bajoga imensely for his encouragement and personal interest in the progress of this project. I am grateful to all the acedemic staff and graduate students of the department of Civil Engineering A.B.U., Zaria, for without their various criticisms during Seminars, I may not have been able to improve on the work. I am ever grateful to Alhaji Sonibare, Mr. Bello, Mr. Ismaila and all the staff of the structures lab, for their assistance. I wish also to thank Mr. Isaac Ogwuche and Mrs Aiyejumo for the production. I shall not end without mention of Professor Oleszkiewicz whose original idea on unified design approach for concrete structures formed the basis of this research. I am also very grateful to Professor Andrez Florek who sent his comments from Poland. Am grateful to Mr. Bon Okeke who made it possible for me to get in touch with some journal publications from Europe.

There are obviously people too numerous to list who have in one way or the other contributed to this thesis, am ever grateful to all.

ABSTRACT

Reinforced concrete columns under Small Eccentricity of loading have not been given adequate attention as a class of columns which require its own design method. The present assumption is that current theories on columns which regard eccentricity as a purely flexural parameter, are adequate for the design of this class of columns. The result is that columns under small eccentricity are either designed as flexural systems or as axially loaded systems with arbitrary reduction of the assumed material strengths. Available design charts for uniaxially and biaxially loaded columns have limited parameters as well as assume symmetrical arrangement of reinforcements which make their use for the design of this class of columns uneconomical.

New models for design and capacity estimation of uniaxially and biaxially loaded columns have been proposed. The models indicate lower reinforcement requirement and more flelxible use of design parameters, over existing methods. A total of seventy-two replica model and prototype columns were crushed to failure as well as a model frame, to test the assumptions of the propoised design and capacity estimation methods. The recorded failure loads were compared to both the

proposed theoretical loads and current theories of load estimation. The results show that the proposed load capacity estimation model is more realistic in determining the load capacity of unsymmetrically reinforced concrete short columns. With a hundred percent agreement between expected and recorded failure loads, the assumptions of the proposed design/load capacity estimation methods are fairly accurate. The proposed methods simplify the design problems of unsymmetrically reinforced concrete short columns under small eccentricity of loading.

TABLE OF CONTENTS

		PAGE
CHAPTER 1	Introduction	1
CHAPTER 2	Literature Review	9
CHAPTER 3	Test Set-Up And Methodology	117
CHAPTER 4	Proposed Uniaxial Design Method	123
CHAPTER 5	Theoretical And Experimental Load Capacities	142
CHAPTER 6	Eccentricity And Reinforcement Unsymmetry	167
CHAPTER 7	Concrete Confinement And Failure Patterns	186
CHAPTER 8	Model Test Frame	202
CHAPTER 9	Effect of Slenderness	208
CHAPTER 10	Biaxial Loading Design And Capacity Estimation	215
CHAPTER 11	Collapse of Some Buildings	226
CHAPTER 12	Unsymmetrical New Type Of Composite Columns	231
CHAPTER 13	Conclusions And Recommendations	243

LIST OF TABLES

PAGE

Table 2.1 Chiwuzie's Test Results 63

Table 3.1 Test Specimen Parameters 117

Table 3.2 Composite Column Parameters 118

Table 3.3 Reinforcement Bar Parameters 119

Table 4.1 Average Strains at Different Loadings 138

Table 4.2 Summary of Reinforcement Requirements Using Various Methods 139

Table 5.1 The Average Load Ratios Recorded For Given Reinforcement Ratio 150

Table 5.2 Analysis of Group G Specimens 151

Table 5.3 Observed Failure Loads And Theoretical Failure Loads of Column Specimens 152

Table 5.4 Despersion of Observed-Theoretical Load Ratios With Respect to Model Ratios for Different Eccentricities of Group H 161

Table 5.6 Observed/CP110 Load Variation With Eccentricity 163

Table 5.7 Dimensionless Load Capacity Group G 164

Table 5.8 Dimensionless Load Capacity Group H 165

Table 5.9 Dimensionless Load Capacity for All Specimens 165

Table 6.1 Total Difference In Strains At Different Loads 176

Table 6.2 Recorded Strains At Various Load Levels 177

Table 6.3 Total Strain, Less Strain For Zero Eccentricity At Various Load Levels 178

Table 6.4 Constants Of Proportionality For Equation 7.3.3 179

Table 6.5 Dimensionless Moment Response of Reinforcement For Group H Specimems 182

Table 6.6 Average Recorded Starins on Reinforcement at Various Load Levels For Group E 182

Table 7.1 Stresses in Tie And Main Reinforcement 188

Table 7.2 Stresses in Steel at Different Loads Levels for New Type of Columns 189

Table 7.3 Load Carried by Concrete at Various Levels for New Type Of Columns 190

Table 7.4 Effect of Tie Spacing on Failure Load 191

Table 7.5 Theoretical-experimental Failure Load Ratios 192

Table 8.1 Strain Variation With Age of Concrete Under a Given Load 203

Table 8.2 Differential Strains in Reinforcements Due to Increased Load 205

Table 9.1 Test Results of Slender Columns 213

Table 12.1 Test Results of New Type of Composite Columns 241

Table 12.2 Reduction Factors for New Type of Composite Columns 241

Table 10.1 Comparison of Observed Failure .....

LIST OF FIGURES

			PAGE
Fig	2.0	Optimal Reinforcement	85
Fig	2.1	Moment-Thrust Interaction Curves	86
Fig	2.2	Determination of Load Capacity by Oladapo	87
Fig	2.3	Design Charts or Aids	87
Fig	2.4	L-shaped Columns	88
Fig	3.1	Dimensionless Load Capacity And Eccentricity Group H	124
Fig	3.2	Dimensionless Moment Response Of Reinforcement	125
Fig	3.3	Variation of Dimensionless Load Capacity With Eccentricity	126
Fig	3.4	Dimensionless Load Capacity And Reinforcement Unsymmetry	127
Fig	3.5	Sketch of the Specimens And Load Application	128
Fig	4.1	Proposed Design Chart	144
Fig	4.2	Load Response Slope Variation With Eccentricity	145
Fig	4.3	Stress Distribution In A Column - Oleszkiewicz	146
Fig	4.4	Eccentrically Loaded Section	146
Fig	5.1	Variation of Effective Strain With Applied Load	147
Fig	5.2	Typical Reinforcement Response to Load At A Given Eccentricity	148
Fig	5.3	Effect of Reloading On Specimen Recorded Strains	163
Fig	5.4	Load Response of Symmetrical Reinforcements	165

Fig 5.6 Variation of Load Ratio With Eccentricity for Slender Columns 169

Fig 5.7 Typical Load Response Of Reinforcement At Critical Eccentricities 170

Fig 5.8 Load Capacity Variation With Eccentricity of Group H 171

Fig 5.9 Effect of Reinforcement Unsymmetry On Failure Loads 172

Fig 5.10 Variation of Load Ratio With Eccentricity for Short Columns 173

Fig 5.11 Effect of Total Reinforcement Area On Failure Load 164

Fig 5.12 Effect Of Total Reinforcement Area on Failure Load Ratio (Group G) 164

Fig 6.1 Strain Distribution Across a Section Of A Given Load Level 182

Fig 6.2 Variation of Total Difference In Strain With Eccentricity 184

Fig 7.1 Recorded Failure Patterns 189

Fig 7.2 Load Response of Ties 195

Fig 7.3 Load Ratio Variation With Tie Spacing 196

Fig 7.4 Variation of Confining Pressure With Applied Load 197

Fig 7.5 Variation of Load Carried By Concrete With Ac/As 198

Fig 7.6 Variation Of Failure Load With Area Ratio 199

Fig 8.1 Four Storey Model Test R-C Frame 210

Fig 8.2 Strain Variation In Less Compressed Steel Due To Reduction In Eccentricity 211

Fig 8.3 Strain Variation In More Compressed Steel Due To Reduction In Eccentricity 24

Fig 8.4 Effect Of Concrete Strength Development On Load Distribution 212

Fig 12.1 Sketch of New Type of Composite Sections 238

Fig 12.2 Unsymmetrical New Type Of Composite Columns 238

LIST OF PLATES

				PAGE
1.	Plate 1	The Test Load Frame		8
2.	Plate 2	The Strain Digital Meter		116
3.	Plate 3	The Model Frame		144
4.	Plate 4	Crushed Specimens		159
5.	Plate 5	The Reinforcement Of The Model Frame ...		185
6.	Plate 6	The Model Test Frame Under Load		214

.....

NOTATIONS

Ac - Area of Concrete

As - Area of Less Compressed Steel

A's - Area of More Compressed Steel

b - Breadth of Column Section

d - Depth of More Compressed Steel From Compression Face

d - Depth Steel From Tension Face

d - Depth of S

e - Eccentricity of Loading

fcu - Concrete Cube Strength

fs - Stress in Less Compressed Steel

fy - Steel Yield Strength

h - Total Depth of Column Section

hc - Depth of Concrete Core

L - Effective Height of Column

M - Moment

N - Axial Load

Ne - Observed Failure Load/Load Capacity

Nt - Theoretical Failure Load/Load Capacity

Sv - Tie Spacing

LCR - Less Compressed Reinforcement

MCR - More Compressed Reinforcement

CHAPTER 1

INTRODUCTION

The current proposals by Abatan (1986) and Oleszkiewicz (1985) on the Unified design approach for reinforced concrete was the "Vitamin Shot" for this research. The interest of the author in reinforced concrete columns increased with the collapse of some buildings in Lagos in 1985, one of which was situated at Iponri in Lagos Nigeria. The building at Iponri collapsed as the fourth floor slab was being cast, suggesting over-loading of the reinforced concrete columns (NSE 1985). Various reports released by the Nigerian Society of Engineers and the Nigerian Institute of Structural Engineers attributed the collapses mainly to poor materials and inadequate desigsns. As the building had collapsed, the determination of the load capacity of the columns or the load level at which they collapsed, was one interest of this research. Part of the research therefore was investigation of some collapses of reinforced concrete buildings hoping that these collapses would form part of the real life results. Unfortunately, structural collapses in Nigeria have been treated with a lot of secrecy, that Nigerian engineers would hardly gain any experience from them. All results used in the analysis within the text, inevitably, were limited to laboratory tests.

KASHIM IBRAHIM LIBRARY

1.1 Research Question

Reinforced concrete columns are supposed to primarily be compression members. Due however to the rigidity of Insitu concrete construction, moments are transferred to the columns from the beams. Construction inacuracy may also result in eccentric loading of the columns. Present design methods for reinforced concrete columns treat this moment purely as a flexural parameter. Thus making some assumptions which result in the design of reinforced concrete columns in high rise buildings (or other eccentrically-loaded compression members) being uneconomical. Present design methods either assume such column as axially loaded with arbitrary reduction of material strengths to take care of what is referred to as small eccentricity, or neglect the tensile strength of concrete and design columns as flexural systems with unbalanced axial load condition. Thus there is no "middle of the road" solution for columns in high rise buildings which carry heavy loads but small (relatively) moments. The arbitary reduction of material strengths to take care of small moments is uneconomical for columns of high strength concrete, which is good and effective in compressive stress

resistance. Available design charts assume symmetrical arrangement of reinforcements in addition to restricted design parameters which are often approximated by designers, to the detriment of cost. The present use of the terms "small eccentricity" and "moment dominance" in determining when to design a column as axially loaded, symmetrically or unsymmetrically reinforced, is not precise. The design of columns for unsymmetrical reinforcement arrangement involves rigorous iterative procedure in addition to the classical assumptions of bending as it affects strain distribution at failure, which may not represent strain distribution at failure (Lafe 1986; Parrot 1974). Presently there is no universally accepted simple method of design of biaxially loaded columns. All these make it worthwhile to provide an alternative simple and cheap method of design of reinforced concrete columns, based on the ultimate limit state of collapse, for columns under small eccentricity of loading. Most available literature on reinforced concrete emphasize the design aspect, neglecting each time, the importance of capacity estimation as a good supervision tool as well as in cases of arbitration.

By the current methods, while it may be easy to specify reinforcements for columns under small eccentricity, it may not be easy estimating the load capacity of such columns. Major question which needs to be answered is whether a column reinforced symmetrically with say A^1s = 3R12 and As = 3R12 will attain the same failure load with another column reinforced unsymmetrically with As = 4R12 and As = 2R12. The questionable nature of the assumption that ties merely prevent the buckling of reinforcements in a column; since it had been reported that no matter the closeness of ties in a column, the steel buckles at failure, calls for some research. Taylor (1983) in his work on new type of composite columns, concluded that there was need to study the performance of the columns with the variatioon of the core as a concrete confinement parameter.

1.2 Scope of Work

This research, which is intended to be part of a continous work on reinforced concrete with maximum concrete contribution of unity, involved

(i) The provision/preparation of some design and capacity estimation models for uniaxially and

biaxially loaded columns with unsymmetrical reinforcement arrangement, based on a number of assumptions.

(ii) Laboratory testing of replica model and prototype columns, and a model reinforced concrete frame, to test the accuracy of the assumptions of proposals.

1.3 Limitations

Apart from the limitations of test method reported in Chapter 3, other limitations of the research include

(i) Inability to obtain full technical details of collapsed buildings

(ii) Inability to deal with repeated cyclic loading condition

(iii) Tests are limited to the tropics.

(iv) The capacity of the machine for crushing the prototype columns limited both the size and strength of concrete. This also had the effect of reducing the eccentricities of loading.

1.4 General

The research is based on the ultimate limit state design method. Emphasis is therefore placed on the

British Code of practice CP110 as the reference code. The code of practice CP110 is also the most popular design code among Nigerians for reinforced concrete. All equations given as analytical equations of concrete sections by CP110 are rather implied from the respective suggested design equations, after due application of safety factors. In writing this text, some knowledge of reinforced concrete is assumed. Therefore the reader is expected to be familiar with such terms as cover, effective depth, cube strength etc. It is also expected that the reader will be able to differentiate between when a load means applied load and when it means load capacity of the section. In the text, eccentricity of loading is often used in place of moment to mean the same thing, however when the values are given, they are in their right dimensions. Notations are similar to those commonly used in most literature on reinforced concrete design. Where it is necessary and where a notation as used within a chapter is different from list of notations provided, the notation is defined or explained. It should be noted that the concrete cube strength (fcu) in the text refers not necessarily to the 28 day strength, but the cube strength at the time of testing the specimens.

All the strain values are given in micro-strains except where specified. In plotting the various graphs involving the strain values, the strains at low loads are sometimes ignored because at low load levels, the gauges may not have stabilized. The inevitable use of some words to near monotony is only an attempt at clearity.

1.5 Organisation of Text

The review of available and relevant literature on reinforced concrete columns is presented in Chapter 2 under twelve sub-chapters. The literature review is by no means exhaustive but rather highlights the directions of most researchers or authors. Attempt is again made within chapter 2 to emphasize in greater detail, some of the limitations of current design methods. Chapter 3 describes the laboratory test set-up and method of testing of the specimens and materials. The proposed method of design for uniaxially loaded columns is presented in Chapter 4 along with the derivation of the design chart. Chapters 5-9 present the results and analysis of model, prototype and frame tests. Biaxial load design method and capacity estimation are presented in Chapter 10

while the review of the collapse of some buildings is presented in Chapter 11. Chapter 12 is an attempt to extend the design proposals and principles to unsymmetrical New Type of composite columns. The conclusions of the research are presented in Chapter 13 along with some recommendations that will improve the knowledge and designs of reinforced concrete columns. Note that the reinforcements commonly referred to as compression and tension reinforcements are referred to as the more compressed steel and the less compressed steel respectively. The text presents mainly typical behaviours in form of graphs.

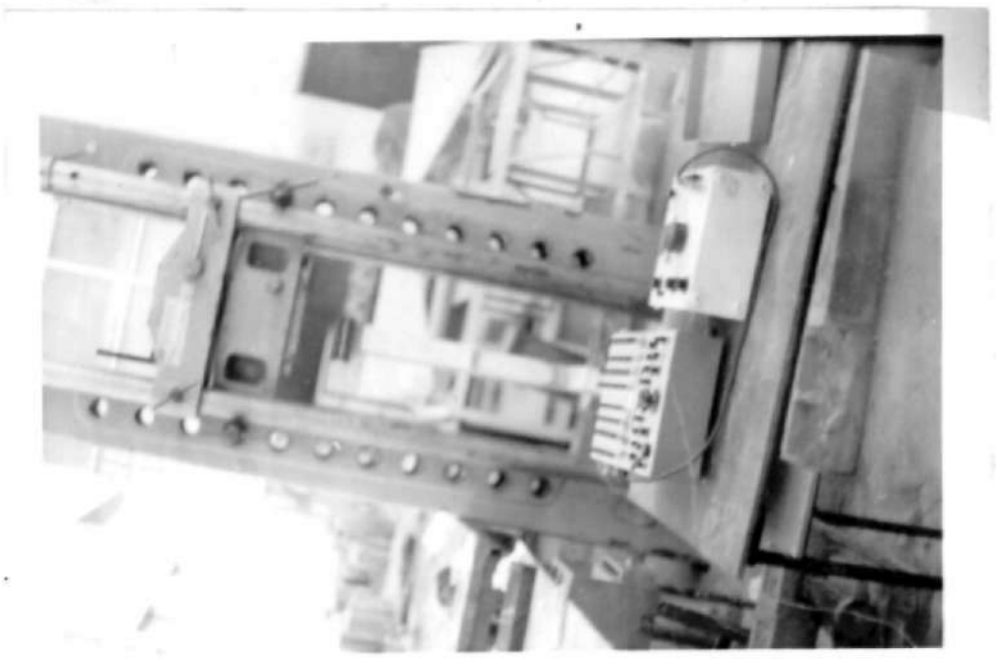


Plate 1 Load Frame

CHAPTER 2

2.1 Reinforced concrete Columns.

Reinforced concrete columns are usually vertical structural members made of concrete-steel composite. In some cases, columns may be inclined. The primary function of columns is to resist axial gravity loads in addition to some induced moments. Reinforced concrete columns are classified into short and slender columns. Short columns are columns with height/least-breadth ratio of 12 or less. Any Column with a higher ratio is a slender column (BSI 1972). The basic difference between short and slender columns is the inducement of additional eccentricity by load, in slender columns. The choice of 12 does not absolutely differentiate both columns, but is a convenient ratio.* While short columns are governed by the strength of the composite materials, slender columns are governed by its propensity to buckle. In other words, a slender column is likely to fail before it's constitutuent materials attain their maximum stresses (Oliva 1987; Davies 1962). Columns are further classified into braced and unbraced. Braced columns have their lateral loads

---

*This ratio was taken as 15 by CP114.

resisted for them by such other elements as walls and staircases while unbraced columns resist all imposed lateral loads such as wind loads, by their flexural action. Columns are similar to walls except that walls are vertical compression members with the greatest lateral dimension more than four times the least lateral dimension. (Astill et al 1975; Aktan et al 1984).

2.2 Reinforced Concrete Column Current Design Methods

There are basically two methods of design of reinforced concrete columns. These are the elastic design method and the limit state method. The third method which is the load factor method is similar to the limit state method with additional use of capacity reduction factor. This third method is common in the United States of America and is well explained by Park and Pauly (1975). The other two methods are used in the British Codes as well as draft code for Nigeria and are therefore briefly reviewed. (Armer et al 1982; Oladapo 1981; Winter et al 1986).

2.2.1 The Elastic Design Method

This method of design is based on the basic principles of bending theory of composite material. Some of the assumptions are

(i) Plane sections remain plane before and after bending. This implies linear distribution of strain which is directly proportional to distance from the neutral axis.

(ii) Both concrete and steel are elastic implying a constant ratio of elastic modulus.

(iii) All tensile stresses are taken by reinforcements.

(iv) There is perfect bond between concrete and steel.

The bending theory has been the method commonly used for assessing the resistance of reinforced concrete members to bending. Although this conventional method of design is inadequate and irrational in explaining the behaviour of concrete structures, it has survived because it produced simple and convenient design formulae and because structures designed with it have proved safe by time. (Oladapo 1981). While this method was noted to be inadequate, (Oladapo 1981) states that it will continue to be an asset for the prediction of the behaviour of concrete structures at service load condition. Essentially the elastic design method otherwise known as the

KASHIM IBRAHIM LIBRARY

permissible stress method, strives to ensure that at working load condition, the stresses in a member do not exceed prescribed allowable limits. These limits are the factored (reduced) strengths of the materials. The basic principle being the "Transformation" of the section into equivalent section in either concrete or steel units, on the basis of the elastic modulus ratio.

Design of Axially Loaded column

An unreinforced concrete column is capable of resisting axial compression, but a minimum amount of reinforcement is usually provided in these columns to take care of the effects of shrinkage, thermal stresses and incidental moments. Reinforcements may also be provided to reduce the size of the section.

The permissible load on a short column is given as $N = fcc\ Ac + fsc\ Asc$ ..... 2.2.1. Since the strain in concrete is equal to the strain in steel at the same point, $N = fcc\ (Ac + m\ Asc)$ ..... 2.2.2. Note that this method prescribes the allowable or permissible load and not the load capacity of the column.

Example

To design a short column to resist an axial load of 2500KN. The permissible concrete and steel stresses in direct compression are $5.3N/mm^2$ and $125N/mm^2$

respectively.

Solution Oladapo (1981)

Assume one percent reinforcement; minimum required

by code.* Using equation 2.2.1 and assuming a square column the column size is approximately 620mm x 620mm.

- Design of Eccentrically Loaded Columns (Cowan 1982; Park et al 1975; Sengupta 1983)

Two cases exist which are

(i) For an unreinforced rectangular section, for tension stress to be zero

$$\frac{N}{Ac} - \frac{Ne}{Z} \quad 0 \quad \ldots\ldots\ldots\ldots\ldots 2.2.3$$

Where Z is elastic section modulus.

$$\therefore \quad \frac{N}{bh}\left(1 - \frac{6e}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots 2.2.4$$

$$\therefore \quad e \quad h/6 \quad \ldots\ldots\ldots\ldots\ldots 2.2.5$$

For a reinforced concrete section similar equations as 2.2.3 - 2.2.5 apply except that the section modulus will now depend on percentage reinforcement. This results in a more complex formula especially for non-symmetrically reinforced sections. However for simplicity, Adepegba (1989) suggests there

will be no tensile streess or cracus for

1% reinforcement if e h/5

4% reinforcement if e h/4

8% reinforcement if e h/3

Adepegba suggests that generally for medium reinforced concrete column sections, there will be no crack for e less than or equal to between 0.2h-0.25h. The total stress at extreme fibres is given as

$$f = \frac{N}{Ae} + \frac{N*e*h}{2Ie} \quad \ldots\ldots\ldots\ldots 2.2.6$$

Where Ae and Ie are the transformed area and moment of inertia respectively. Oladapo 1981; Rashedi et al 1988; Wan 1988.

(ii) Tension Developed

The concrete in this case cracks, there therefore cannot be superposition of stress. From equillibrium of forces

$$N = \frac{1}{2} bn f_c + (m-1)A_c \left(\frac{n-dc}{n}\right) f_c - mA_c \left(\frac{d-n}{n}\right) f_c \quad \ldots\ldots 2.2.7$$

$$Ne = \frac{1}{2} bn f_c \left(\frac{h}{2} - \frac{n}{3}\right) + (m-1)A_c \left(\frac{n-d_c}{n}\right) f_c \left(\frac{h}{2} - d_c\right)$$

$$+ mA_c \left(\frac{d-n}{n}\right)\left(f_c\right)\left(\frac{h}{2} - dc\right) \quad \ldots\ldots 2.2.8$$

Where n is depth of compression

dc = Total cover to reinforcement

m = modular ratio

•2.2.2 Ultimate Strength Design Method

The ultimate strength design method is the alternative design method to the elastic method. This method applies partial factors of safety to both materials and loads. (Ferguson 1984; Park et al 1975). The ultimate strength design method which is essentially the limit state method was developed because of some reasons:

(i) Reinforced concrete section behave inelastically at high loads, hence the elastic method cannot give reliable prediction of the ultimate strength of a section.

(ii) Ultimate strength design allows a more rational selection of the load factor. A low factor may be used for quantities that can be determined with higher precision.

(iii) Since the stress-strain curve for concrete is non-linear and is time dependent, the constant value of modular ratio is a crude approximation.

(iv) The ultimate strength method indicates the elastic method to be conservative.

(v) The ultimate strength method lends itself easily to design for earthquake-resistant structures where the performance beyond Elastic range is very necessary.

The ultimate strength method which is design for collapse vary according to the country and the specified partial factors of safety.

2.2.2.1 Partial Factors of Safety

The limit state method applies partial factors of safety to both materials and loads. The following load factors are specified by the American concrete Institute (Park et al 1975).

(a) Dead load and live load only 1.4 and 1.7 respectively.

(b) Dead, live and wind loads 1.05, 1.275 and 1.275 respectively.

(c) Where live load having its full value or zero should be checked and actions resulting from the dead load and wind load are of opposite sign, 0.9 for dead load and 1.3 for wind load.

For the same conditions a, b, c above CP110

recommends

(a) 1.4 for dead load and 1.6 for live load.

(b) 1.2 for dead load, live load and wind load

(c) 0.9 for dead load and 1.4 for wind load.

The American concrete Institute recommends in addition, capacity reduction factors, as follows BSI 1972; Park et al 1975; Maruyama et al 1984.

(a) Beams 0.90

(b) Columns with spiral ties 0.75

(c) Columns with rectangular ties 0.70

(d) Shear and torsion 0.85

CP110 has no such capacity reduction factors but partial factors for material strengths given as follows

(a) For utimate limit state, 1.5 for concrete and 1.15 for steel.

(b) Servicibility limit state, 1.0 for both concrete and steel when checking for deflection.

(c) Servicibility limit state, 1.0 for steel and 1.3 for concrete when checking for cracks.

2.2.2.2 Axially Loaded columns

The load capacity of an axially loaded concrete column is estimated in a similar manner as in the

elastic method; the summation of the load capacities of the concrete and steel. Thus N = Acfcc + Asc fsc ..... 2.2.9. It was said that up to the value of N spiral and tied columns behave identically and the transverse steel adds very little to the strength of the column. (Chiwuzie 1975; Priestly 1983). While a tied column with ties not closely spaced will fail at a load value of N, accompanied by the crushing of concrete and buckling of bars, this may result only in the loss of the shell of concrete outside the spirals of a circular column. The enhancement of concrete strength due to confinement is discussed in chaps 2.6 and 8. The failure stress of concrete in a column section is about sixty-seven percent of the cube strength). Applying the partial factor of safety and allowing for buckling for the bars in equation 2.2.9

N = 0.45fcu Ac + 0.72fy Asc 2.2.10

To allow for ":small" eccentricity this equation is modified as

N = 0.4fcu Ac + 0.67fy Asc 2.2.11

For short columns supporting an approximately symmetrical arrangement of beams equation 2.2.11 is

further modified as

$$N = 0.35f_{cu}A_c + 0.6f_yA_{sc} \quad \ldots\ldots\ldots \quad 2.2.12$$

In example 2.2.1 assuming the dead load is 1670kN and the live load is 820kN, neglecting the weight of the column, the same problem could be solved in two ways. fcu=21, fy=250. Percentage reinforcement - one.

1. CP110 Method

Design load = 1.4x1670 + 1.6x820 = 3666kN

$\therefore 3666x10^3 = 0.45x21x0.99Ac + 0.72x250x0.01Ac$

If the column is a square, b=h = 573 mm.

2. ACI Method

Design load = 1.4x1670 + 1.7x820 = 3732kN

Applying a capacity reduction factor of 0.7,

The design load becomes 5356KN.

$\therefore 5356x10^3 = 0.99x0.67x21Ac + 0.01Acx250$

Assumming a square column then

b = h = 571mm.

For this problem, the ACI and CP110 methods approximate to the same value, with the design by elastic method requiring about eighteen percent more concrete.

2.2.2.3 Eccentrically Loaded Short Columns

The most popular and convenient method of design

of eccentrically loaded columns is by the use of charts. The charts in CP110 part 2 are produced based on the principles of rectangular-parabolic stress distribution as explained in 2.3. Appllied force will be resisted by forces developed within the section.

$$N = K_1 bX + f_{sc} A'_s - f_s A_s \quad \ldots\ldots\ldots\ldots \quad 2.2.13$$

taking moments about the mid section

$$M = K_1 bX (h/2 - K_2 X) + f_s A'_s (h/2 - d') + f_s A_s (h/2 - d) \ldots \quad 2.2.14$$

When the neutral axis depth X is more than h K^1bx is adjusted to 0.45fcubh and the concrete has no contribution to moment resistance. For symmetrical reinforcement, equation 2.2.13 may be written as

$$\frac{N}{bh} = K_1 \frac{x}{h} f_c + f'_c \frac{A's}{bh} \quad \frac{As}{bh} f_s \quad \ldots\ldots\ldots \quad 2.2.15$$

and equation 2.2.14 may be written as

$$\frac{M}{bh} = K_1 \frac{x}{h} (0.5 - x/h) + \frac{fsc\ As}{bh} \left(\frac{d}{h} - 0.5\right) + \frac{f_s A_s}{bh} (0.5 - \frac{d}{h}) \ldots \quad 2.2.16$$

For a given depth of neutral axis, the steel strains and hence stress are determined from the elasto-plastic stress-strain model. Thus for a given

reinforcement percentage, equations 2.2.15 and 2.2.16 combined produce a chart for symmetrical reinforcement arrangement. (Bennet 1973). The notable assumptions of the chart are the linear strain distribution in a cross section under bending and axial load, and the perfect bond assumption at high loads. It must also be noted that CP110 charts apply to columns with as little eccentricity as 0.02h (N/bh = 24 and M/bh^2 = 0.5) as well as high eccentricities of about 2.4h (N/bh = 5 and M/bh^2 = 12). Simple approximate equations for the design of rectangular and circular columns have been proposed based on a study of dimensionless load-moment interaction chart. (Meyer et al 1982).

Sometimes it is more convenient to use the eccentricity of load from the plastic centroid. The plastic centroid is the centroid of resistance of the section if all the concrete is compressed to the maximum stress of 0.67fcu (Ferguson 1984; Park 1975). In other words, the plastic centroid is the point of application of load (No) to produce an axially loaded condition at failure. Taking moments of the internal forces about the centroid of the more compressed steel and equating this to the moment of the

resultant forces gives

$N_o d'' = 0.85fsybh(d-0.5h) + A'sfy(d-d^1)$

$= (0.85fcubh + (A's + As)fy)d''$

$$\therefore \quad d'' = \frac{0.85fcybh(d-0.5h) + A'sfy(d-d^1}{0.85fcybh + (A's+As)fy} \quad \ldots\ldots 2.2.17$$

Where d'' is the distance from the plastic centroid to the less compressed steel. For symmetrically reinforced members, the plastic centroid corresponds to the centre of the cross section. Taking moments about the plastic centroid gives

$N_u e = 0.85fcybx(d-d''-0.5x) + A'sfy(d-d^1-d'') + Asfsd''$

$\ldots\ldots\ldots$ 2.2.18

where fcy is concrete cylinder strength, x is the depth of equivalent rectangular stress block. A balanced failure occurs when the less compressed steel yields, just as the concrete at the extreme fibre reaches ultimate strain value of 0.003. When charts are not used, especially when moment is said to dominate, it becomes uneconomical to design the column as symmetrically reinforced. The rigorous iterative procedure for unsymmetrical arrangement involves

(i) The choice of a trial neutral axis depth.

(ii) Determination of the strains and hence the

stresses of the reinforcements.

(iii) Determination of the areas of reinforcements from the equations 2.2.13 and 2.2.14.

(iv) Repeating the procedure in an iterative manner until a given depth of neutral axis produces minimum total area of reinforcements.

There is no precise load combination situation where the designer is required to design either as symmetrically reinforced or otherwise. The choice of which way to go appears to depend on the convenience of the designer (Park et al 1975; Moseley et al 1976). There is also the current tendency to assume that once a column is subjected to axial load and bending, the so called tensile reinforcements must be in tension (Adepegba 1989). Apart from costs, in the present design method it is not clear whether a column with say A's = 400mm^2 and As = 200mm^2 and reinforced symmetrically with A's = 300mm^2 and A's = 300mm^2 is safe or attains the same reliability level. Clement (1977) attempts to solve the problem of heavily loaded columns with small eccentricity. He suggests that such columns will be entirely under

compression and to take care of the moments, additional load is imposed on one line of steel while the other is reduced by the same amount.

Example

To design a column with the following parameters

N = 300KN, M = 100KNM, b = 300mm, h = 600mm

d = 540mm; $d^1=d_2$=60mm; fcu = 30; fy = 410

Solution by Clement (1977)

Lever arm = 600 - 2 x 60 = 480mm

$\therefore$ Force to resist moment of 100KNM

$= 100 \times 10^3/480 = 208$ KN

Force in reinforcements = $3000 - (0.45x30x300x600x10^3)$

= 560KN

Force on each reinforcement if there were to be no moment = 560/2 = 280KN.

$\therefore$ Force on reinforcement considering the moment

$= 280 \pm 208$ i.e 488KN or 72KN.

Provide symmetrical arrangement of reinforcement

$A's = A's = 488 \times 10^3/(0.74 \times 410) = 1610mm^2$

Note that the capacity of the less compressed steel is 488KN but the force on it is 72KN. Note also that if the moment is increased to 134KNM, while the force

in the less compressed steel will be zero, reinforcement capacity of 560KN will be installed. If however the reinforcement stress is increased by reducing the area of the less compressed steel, the static equillibrium will not be satisfied. Oladapo (1981) contends that in considering the ultimate strength of a concrete framed structure, the concept of eccentircity is not of much use. In estimating the strength of a concrete column which will fail by crushing the concrete in compression on one face while just about developing tension on the other face. Oladapo (1981) uses the elastic analysis of the section as shown in fig 2.2. The area of each of the eight bars is 645mm^2. Using stress distribution (Parabolic function) for concrete and that for steel as specified by CP110, the table below was produced.

Position	ϵ	fc	f_s
5	0.003	50	-
4	0.00265	49.32	410
3	0.0015		300
2	0.00035	10.99	70
1	0	0	0

Taking into consideration concrete displaced by steel

Ns2 = 645 (70-10.99) x 3 = 114/2kN

Ns3 = 645 (300-37.5) x 2 = 338.6kN

Ns4 = 645 (410-49.32) x 3 = 697.9kN

Nc = 2/3 x 300 x 50 = 300 kN

∴ Total failure load = 4150.7 kN

Taking moments about point 1

Mu = (5.3)(200)(200) - 114.2x35 + 338.6x150-697.9x165

= 302.24kNm

Recorded Failure Patterns

To determine the design load combination N and M of the above problem by CP110 specifications, partial factors of safety are applied to stresses. Therefore maximum concrete stress becomes $0.45 \times 50 = 22.5 N/mm^2$. The following table is obtained.

Position	fc	fs
2	4.95	60.9
3	16.875	260.9
4	22.19	356.5

Taking into consideration displaced concrete.

N = Ns2 + Ns3 + Ns4 + Nc = 2420KN

taking moments about 1 gives M = 475.6KNm. Using these load combination to estimate the required reinforcements by CP110 method will result in reinforcement requirement of about 10% as against 5.73% originally provided. This implies that using elastic analysis method at the ultimate load will be decietful

2.2.3 Design of Biaxially Loaded Columns

The load estimation and application of safety factors for design of biaxially-loaded reinforced concrete columns are similar to uniaxially loaded

columns. Due to the complex behaviour of concrete under load, biaxial load design is complex (Furlong 1961; Harik et al 1988). Current design methods are summerized under the various attempts to accurately predict the behaviour of concrete columns under biaxial loading. Hsu et al 1973; Linder et al 1984; Park et al 1975; Taylor 1985.

2.2.3.1 Method of Superposition

In this procedure, the biaxial problem is simplified as superposition of two uniaxial solutions. The same procedure for uniaxial loading is used while superposing the resultant reinforcements. This is the method used in the code of Venezuela. (Park et al 1975). It was observed that although the results lie on the safe side, the reinforcement requirement may be excessively conservative. Ih another method the biaxial load is replaced by two statically equivalent forces and taking the concrete strength each time as equivalent to that in ratio of the force with the applied load. That is, when considering Nx acting at an eccentricity of ex, the strength of the concrete is assumed equal to:

It was suggested that the procedure does not have any theoretical support although it produced satisfactory solutions.

2.2.3.2 Method of Equivalent Uniaxial Eccentricity

A number of approximate analytical expressions have been proposed to enable the determination of an equivalent uniaxial eccentricity, the equivalent uniaxial eccentricity is

$e_o = e_x + Ke_y$ where k is a constant which depends on the level of axial load and the percentage reinforcement. Park et al 1975; Hsu 1985.

2.2.3.3 Aprroximation of Interaction Surface

Various suggestions on the method of design approximating the Interaction Surface have been made and incoporated in some codes. Bresler (1960) suggests

$$\frac{1}{Nu} = \frac{1}{Nux} = \frac{1}{Nuy} = \frac{1}{No} = 1 \quad \ldots\ldots\ldots \quad 2.2.21$$

where

Nu is the ultimate biaxial load capacity.

Nux, Nuy are load capacities at eccentricities ex and ey respectively and each acting at a time alone.

No is the ultimate load when there is no eccentricity.

While the use of trignometric equation had been suggested. Bresler (1960) suggests interaction equation in the form

$$\frac{(Mx)^{\alpha}}{Mux} + \frac{(My)^{\beta}}{Muy} = 1 \quad \text{..... 2.2.21}$$

CP110 (1972) amended Breslers, 1960) equation specifying that $\alpha = \beta$ and that the value varies from 1 for lightly loaded columns to 2 heavily loaded columns. The equation given by CP110 for is

$$\alpha = 1.66 \frac{(N)}{Nuz} - 0.67 \quad \text{..... 2.2.22}$$

2.2.3.4 Design Charts

Weber (1966) produced design charts for diagonally loaded columns. The principle being the conversion of the biaxial problem into a uniaxial solution assuming that the maximum diviation of a constant load contour occurs for bending about the column diagonal. Sixteen charts were produced using the suggestions by Meek (1963), for four symmetrical reinforcement arrangement and only one set of material strengths. Park and Pauly (1975)

modified the assumptions of Weber and produced a design chart.

2.2.4 Summary

There are two methods of design of reinforced concrete columns. The elastic method was developed as an extension of the theories of bending in the early practice of reinforced concrete. The justification for the continued use of this method is the safety of structures designed by it. The major assumption of the elastic method is that reinforced concrete structures are elastic under service load condition. The other method of design is the ultimate strength method which designs a structure beyond the elastic range. The American concrete Institute (ACI) specifies load safety factors similar to those specified by CP110 under the ultimate limit state method, but in addition specifies capacity reduction factors. CP110 in addition to load partial factor of safety, specifies material factors of safety. The design charts of CP110 under the same assumptions provide reinforcements for columns of eccentricities as small as 0.02h and for eccentricities as big as 2.4h. The

design charts have many limitations for their practical use. In the design of biaxially loaded columns current methods either simplify the biaxial solutions or provide equations for approximating the biaxial failure stress surface. The methods do not lend themselves easily to capacity estimation. The only justification for their continued use is the observed safety of structures built with such designs. The current design methods are not flexible with design parameters as well as reliability levels. The charts assume symmetrical reinforcement arrangement.

2.3 Limit State Theory

As was stated in the previous chapter, the two methods of design differ from each other mainly from the application of safety factors. In the limit state method, partial factors of safety are applied to both materials and loads. There are two main types of limit state, servicibility limit state and the ultimate limit state. There are other limit states such as durability, excessive vibration, fatique etc which do not affect most designs of reinforced concrete structures but sometimes may be checked. Debernardi et

al 1989. The philosophy behind the limit state theory is the achievement of safety by limiting both the load and stresses on materials for design. Bennett 1973; Mosely et al 1976; Reynolds 1968.

2.3.1 Servicibilty Limit State

The servicibility limit state may be said to be a method of design of structures within limit state conditions so that the structure must remain unaffected during it's service life. The development of cracks in the reinforced concrete structure and the deflection of flexural elements are indeed the most important consideration for servicibility limit state. When structures are designed for the ultimate limit state however, they are checked against servicibility.

Deflection

Beams and slabs which typify flexural systems are normally designed so as to resist safely an expected maximum load over a safety margin although large deflections may occur, these must not affect adversely the appearance or efficiency of such structure. For instance no matter how safe a building may be, occupants will panic at the sight of beams or slabs with large deflections. Therefore the deflection of

KASHIM IBRAHIM LIBRARY

structural elements is limited by various specifications under service load.

The servicibility limit state relies on elastic behaviour of the materials at service loading. At one-third of the strength of concrete, it behaves in a manner that could be approximated elastic. Therefore at this stress level or load level, the deflection of the member is expected to be proportional to the load, although creep and shrinkage do not allow for perfect elasticity of the concrete (Neville 1963; Nandivaram et al 1987). The strain distribution across the sections of both plain and reinforced concrete members may be assumed linear provided that the depth of the section is small relative to the span. The deflected shape of a beam is related to the curvature by the equation

$$\frac{1}{R^{(x)}} = \frac{d^2y}{dx^2}$$

where $\frac{1}{R}$ is the Curvature of a beam at point x.

From simple bending theory where y is deflection,

$$\text{Curvature } \frac{1}{R} = \frac{M}{EI}$$

$$\therefore \quad y = \iint \frac{M}{EI} \, dx \, dx$$

Double integration of moment at a point will give the deflection at that point. Another method is the application of the moment-area theorem. Various expressions may be derived for deflection of a beam or slab depending on the support and load conditions. In

reinforced concrete, the second moment of Inertia I is affected by the cracking of the concrete and reinforcement ratio. As a simplification, the value Ic which represents the moment for cracked section should be used at regions of maximum bending moments. Oleshiewicz 1985.

Specification

To avoid excessive sagging of floors, beams jamming of doors and windows, cracking and buckling of partitions etc various codes specify the limit of deflection. Deflection requirements are translated into span/depth ratios for ease of designs but in some complex problems, the exact deflection values may be calculated.

Crack control

Crack control in a reinforced concrete member is necessary because reinforcements are optimally utilized

when strains in the tensile zone are high, the crack width should not be too high to allow for either the loss of stored (expensive) liquid in liquid-retaining structures, or hasten the corrosion and hence the weakening of the reinforcements. Assusming the crack width in a member under flexure is w at a regular spacing of s, therefore

$$w = \epsilon s.$$

ϵ_i is the average strain at xi level. One of the simplest expressions for average crack spacing which takes account of concrete cover and size of reinforcements is that proposed by welch. (Park et al 1975) given as

$$S = 1.5c + 30$$

where c is the cover to reinforcement measured to the bar centre, and 0 is the bar diameter. This leads to an expression for maximum crack width in the form

$$w = 1.5(1.5c + 30)\left(\frac{fc}{Es} - 0.0001\right)\left(\frac{h - x}{d - x}\right)$$

CEB (1982) also suggested that

$$w = (4.5 - 0.4)\left(\frac{0.4}{e}\right) d \frac{fs}{K,}$$

where K^2 is $328KN/mm^2$ for deformed bars. Kaar and Mattock (1984) later modified this to a simpler form.

From investigations on the modified formulae, results obtained showed a difference of $\pm$ 40 percent therefore the equation was further modified to read

$$W_{max} = 2.921(4\ A) f_s \left(\frac{h2}{h1}\right) \times 10^{-6} mm$$

where h2 and h1 are the distances of the extreme tension fibre and steel respectively to the neutral axis. A is the area of concrete surrounding each bar. CP110 (1972) recommends the maximum surface width of cracks be given as

$$W = \frac{3\ a\ m}{1+2\ (a-c)} \cdot \frac{}{(h-x)}$$

where a is the distance from the point considered to the surface of the nearest longitudinal bar. C is the minimum concrete cover to the tension steel.

$$\epsilon_m = \epsilon_i - \frac{1.2\ bth(a^1-x)}{1000\ As\ (h-x) fy}$$

where

ϵ_i is the strain at the level considered, calculated as for cracked section and the concrete modulus of elasticity as bt is the width of the section at the centroid of the tension reinforcement. a^1 is the distance from the compression fibre to the point at

which the crack width is being calculated.

h and x are the total depth, and depth of compression respectively.

The British Code requires in general that the surface, crack widths at service load do not exceed 0.3mm and this may be easily interpreted in terms of the minimum clear distances between bars. The recommendations of CEB-FIP require that the surface width at service load not exceed 0.1mm in agressive environment, 0.2mm in an unprotected environment and 0.3mm in protected environment. The maximum crack widths are calculated using the equation by Borges (Part et al 1975; Morrell 1977).

$$W = \frac{1}{Es}\left(2.5c + 0.066\,\frac{\phi}{ew}\right)\left(fs - \frac{107}{ew}\right) \quad (in)$$

where Es - modulus of elasticity of steel in psi

C - concrete cover thickness

ϕ - diameter of bar

ew - As/bwd

bw - width of web (inches)

All proposals and specifications on the calculation of the crack width appear to be effort to simplify the complex behaviour of concrete at service load. It is

not clear how the expected service load is related to elasticity of concrete even though concrete is approximately elastic at about one-third of its strength. Not much attention is given to cracking of concrete columns; perhaps no need for it. (92, 107).

2.3.2 Ultimate Limit State

Some assumptions of the elastic theory are stated under the servicibility limit state-major among the assumptions is that at service load the reinforced concrete element is elastic. At higher loads and just before failure, reinforced concrete is plastic and exhibits a non-linear nature. Thus the safety assessment of concrete structures in current design practice is based on the ultimate limit state. The ultimate limit state design allows therefore the cracking of concrete since the reinforcement is best utilized at high strains and stresses. Partial factors of safety are applied to both the expected load and the material strength. Numerous test results have shown that the creep and shrinkage effects are released during the plastic deformation of a member at failure, the concrete strength is not affected by the previous service loading. Thus the ultimate strength of reinforced members is practically independent on the load history and when compared with the maximum possible loading, allows for a satisfactory estimation of the structure safety (Polyokov 1985). The ultimate limit state have similar assumptions as stated under the servicibility limit state.

Ultimate Flexural Strength

Reinforced concrete members under flexure collapse in two ways, either by concrete crushing or by steel yielding. The mode of failure depends on the reinforcement. While a section with low reinforcement (under reinforced) will fail by steel yielding, an over reinforced section fails

by concrete crushing. Optimal design requires the failure of both materials at maximum loading. The steel yields while the section is able to carry the applied load until the concrete compressive strain reaches it's ultimate crushing value, taken as 0.0035. The stress distribution of concrete across its surface at failure is too complicated and needs to be idealized for practical application. A number of proposals have been made to represent the stress distribution of concrete but these are generalised as a shape with an area of (K_1fcu bx).

Sometimes the stress distribution is assumed parabolic. (Oleszkiewicz 1985; Oliva 1980). The stress distribution said to approach reality is the rectangular-parabolic stress distribution K_1 is the average stress and K_2X the depth of the stress-block centroid. Because of the comlexity of equations involving the rectangular-parabolic stress block in design calculations, the equivalent rectangular stress distribution is often adopted which gives results quite close to the rectangular-parabolic stress state yet simplifying the equations (Ferguson 1984; Meek 1990). The equivalent (uniform) stress is assumed to be about ten percent less than the maximum stress of rectangular-parabolic stress block. This results in the design strength of concrete under the ultimate limit state as forty-percent of the recorded cube strength. Oladapo (1981) states that the new limit state design approach is a logical and rational restatement of the basic concepts which have been used in design practice for several years. Account is taken of all factors which determine structural behaviour and structural safety. The inherent variation in material strengths and the uncertainties of loading justify a statistical probabilistic approach. It is not the intention however, that every design process should involve rigorous statistical analysis. The main use of

statistics will be in aiding those responsible for framing the codes of practice to specify suitable factors for use in design. In other words, the choice of risk is left to the designer. In laying down value of the safety factors, the codes of practice are infact specifying the levels of risks. It is important to note here that this level of risk chosen by the code depends on the structural element and the method of loading as is explained later.

In a singly reinforced section for instance, condition of equillibrium requires that; Asfy = 0.75fcubx assuming the uniform concrete stress over a rectangular area to be seventy-five percent of the cube strength as suggested the ultimate moment of resistance Mu is then given as:

$$Mu = Asfy\left(d-\frac{2}{3}\left(\frac{Asfy}{b\ fcu}\right)\right)$$

In an over reinforced section, the steel stress is nct known, but it has been found experimentally that the quantity of reinforcement in an over-reinforced sectiон has very little effect on the ultimate bending moment given by Park et all (1975) as

$$Mu = 0.3fcu\ bd^2.$$

This is equivalent to a depth of compression of 0.537d. CP110 (1972) specifies a maximum depth of compression of 0.5d and an equivalent rectangular stress distribution of 0.4fcu, resulting in ultimate moment of

$$Mu = 0.15fcu\ bd^2.$$

2.3.3 Assessing The Global Safety Factor Under The Ultimate Limit State

To assess the global safety factor of the ultimate limit state, assume a simply supported beam, span 4m; b = 200mm; h = 400mm; d = 375mm; fcu = 30; fy = 250n/mm2; $d_1 = d_2 = 25$.

Characteristic imposed load = 10 kN/m.
Characteristic dead load including self weight of beam = 15kN/m.
Total design load (ULS) = 1.4 x 15 + 1.6 x 10 = 37kN/m
M(max) = 37 x $4^4/8$ = 74 kNm.
Mu = 0.15fcu bd = 126.56 ¢ M applied.
Therefore from CP110 (1972) reinforcement is about 990mm/cbm.
Flexural capacity of the beam (minimum)
= 990 x 250 x (lever arm)
but lever 375 - 1/2 (990 x 250)

(0.603x30x200)
340.795
∴ Flexural capacity of beam = 84.35 kNm
∴ Mimimum load capacity of beam w
W = 84.35 x $8/4^2$ = 42.175kN.
Imposed load capacity = 42.175 - 15 = 27.175kN
∴ Global safety against overload = $\frac{27.175}{10}$ = 2.72.

Now if the beam span is changed to 5m; the new reinforcement requirement will be 1821mm^2. This will give an imposed load capacity (calculated as previously) as 30.44kN resulting in a global safety factor against over load of 3.044. This indicates that for a given set of design parameters, under the ultimate limit state, the applied safety factor increases with increase in span although the same partial factors of safety were employed.

for a given set of design parameters, under the ultimate limit state, the applied safety factor increases with increase in span although the same partial factors of safety were employed.

2.3.4 Summary

The limit state method consists of the servicibility limit state, ultimate limit-state and other limit states such as durability etc. The servicibility limit state is used in the design of liquid retaining structures and such others where the appearance or aestetic function of the structure is the limiting design criteria. The ultimate limit state is used in the design of reinforced concrete for performance beyond the service load condition and indeed up to the point of failure. The classical assumptions of bending theory are made in the design of reinforced concrete structures. The calculation of crack width in concrete section are rather simplification of complex equations, and the results are not always uniform. Considering more precise analyses and instrumentation, and also the fact that crack widths will be limited to small values further simplification of this aspect of limit state design may be necessary. The assumption of equivalent rectangular stress distribution is suitable. However

there is the need to determine with precision, the relation between the failure stress of concrete in an element and the cube strength of same concrete; sixty-seven percent is currently specified by CP110 (1972) while some researchers have recommended between 60 and 75 percent (Okuboyejo 1988). This discrepancy is quite enormous for high strength concrete sections. The global factor of safety against over loading under the limit state condition for flexural members as currently specified, depend on the loading and structural element. Okuboyejo 1988; Virdi et al 1982; Umaru 1989.

2.4 Concrete in Hot Climate

A brief review of the performance of concrete in hot climate as was mentioned earlier, is necessary because the research was carried out in Nigeria which is in the tropics noted for its high temperature.

Concrete is a conglomeration made from cement, water, sand and graded stone or coarse aggregate. The cement and water react chemcially to bind the aggregates. The main function of fine aggregate is the reduction of voids within the concrete. (Neville 1963). Concrete has remained one of the best structural materials in construction. Although it is

more than a hundred years old as a material the complex nature and behaviour has made continued research on it worthwhile. Some research have been made on various aspects of concrete and concrete behaviour in the tropics or hot climates (Oladapo 1981; Osha 1970). There is not much data on the effect of high temperature on the behaviour of actual concrete structures, although some results of a research show that changes in temperature will generally have two effects on concrete structures exposed directly to the sun. Increases in the concrete temperature will cause a linear expansion. Schodek 1980. In addition, structures in which one part is exposed to the sun and other parts are in the shade will experience a rotational movement caused by temperature differential between the exposed and the shaded parts. In the type of climatic conditions obtainable in most parts of the southern states of Nigeria temperature effects are not likely to present a serious problem to the designer, but in the northern states, adequate provision should be made to ensure that temperature stresses are accomodated in any part of the structure The positive effects of temperature is the increase in

strength at early age as against the strength attainable for the same period (within 28 days) in the temperate regions. Although Oladapo (1981) contends that published results of research into the strength-age relationship for concrete of the tropics do not lead to identical conclusions, it was stated elsewhere Ezetah (1988; Osha 1970) that although concrete not properly cured in the tropics* produce higher ratio of 3 and 7 days strength against 28 days strength than concrete that is properly cured, the overall 28 days strength of improperly cured concrete remains lower. It was also concluded that for a typical mix, the strength in 3 days in the tropics is about the expected 7 days strength of same concrete in the temperate climate. Another effect of high temperature on concrete is the reduction of the water content and workability during mixing. This does not allow for effective compaction of the concrete around the structural member or element. Suggestions have been made for concrete mix design method for the tropics. Osha (1970). Apart from the craking of concrete due to high differential temperature, it is not clear from the available literature the effect of temperature on the

behaviour of concrete under stress. The suggested methods for mix design for the tropics are interested

mainly in countering the effect of rapid dehydration during mixing. It was concluded that any concrete designed by road Note 4 method in Nigeria will under estimate the strength of the concrete. Oleszkiewicz (1985); Osha (1970).

Summary

From available data, high temperature has two major effects on concrete. Firstly, it results in rapid dehydration of the concrete thus reducing the effective workability of the concrete. Secondly the high early maturity increases the early strength of concrete made in the tropics, but producing an overall lower 28 day strength. There is presently no clear effect of temperature on the behaviour of concrete except that high differential temperature will cause cracking of the concrete. It was also concluded in a research paper that high evapouration rates in the gulf region have led to a substantial build-up of salts in the ground and this makes it difficult to keep the surface of fresh or immature concrete moist to provide effective curing.

2.5 Reinforced Concrete Short Columns by Oleszkiewicz

A brief review of the subject of concrete columns by Oleszkiewicz (1985) became necessary because a substantial basis for this research, and comparisons made within the text, originate from Oleszkiewicz's ideas.

Oleszkiewicz (1985) and Abatan (1986) are evolving a unified approach to reinforced concrete designs. The basis of which is the assumption of the rectangular-parabolic stress distribution in a flexural system. If K_1 is the average stress over the stress block, from equillibrium of forces (between concrete and reinforcements)

$$K_1 fcu\ bx + f^1_s A^1_s - fs\ As = 0. \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ 2.5.1$$

the ultimate flexural (moment) strength of the section may be obtained by taking moments any convenient level (for instance about the middle of the effective depth).

$$Mu = K_1 bxfcu\left(\frac{d}{2} - K_2 X\right) + f^1_s A^1_s \left(\frac{d}{2} - d^1\right) + fs\ As\ \left(-\frac{d}{2}\right) - \ldots\ldots\ldots\ldots\ldots\ 2.5.2$$

where k_1, k_2, x are as defined under chapter 2.3. Equations 2.5.1 and 2.5.2 may be written in

dimensionless terms by dividing 2.5.1 by K_1bdfcu and 2.5.2 by K_1kbd^{2}fcu. Thus equation 2.51 becomes

$$\frac{x}{d} + \frac{(f'_s)}{K_1 f_u}\frac{(A'_s)}{bd} - \frac{(f_s)}{K_1 f_{cu}}\frac{(A_s)}{bd} = 0 \quad \ldots\ldots\ldots 2.5.3$$

or $\alpha_c + \alpha_{1s} - \alpha_s = 0$ 2.5.4

and equation 2.52 becomes

$$\frac{M_u}{k\, f_{cu} bd^2} = \beta$$

$$= \left(\frac{x}{d}\right)\left(\frac{1}{2} - k_2\,\frac{x}{d}\right) + \left(\frac{f's}{k_1 fcu}\right)\left(\frac{A's}{bd}\right)\left(\frac{1}{2} - \frac{d'}{d}\right) + \left(\frac{f_s}{k_1 fcu}\right)\left(\frac{A_s}{bd}\right)\frac{1}{2} \cdots 2.5.5$$

hence

$$\beta = \alpha_c\left(\frac{1}{2} - k_2\,\alpha_c\right) + \alpha's\left(\frac{1}{2} - \frac{d'}{d}\right) + \frac{1}{2}\alpha_s \quad \cdots\cdots 2.5.6$$

Thus

$= \frac{x}{d} = \alpha_c$ is the ratio of compression zone depth to the effective depth.

α'_s represents the axial effect of compression reinforcements.

α_s represents the axial effect of tension reinforcement.

β_c is the flexural effect of the compression zone.

β'_s is the flexural effect of the compression reinforcement.

β_s is the flexural effect of the tensile reinforcement.

An assumed α_c value is defined at the α-axis the corresponding βc measured parallel to the B-axis defines a point representing the contribution of the compression zone to flexural resistance of the section. Assigning different values to x/d the complete α_c-β_c curve may be constructed as shown in fig. 2.0 for singly reinforced sections $\alpha c = \alpha_s$ and α'_s is zero.

From the observation of the ultimate strength diagram, it is noticed that the contribution of the concrete (compression zone) to the ultimate strength of the section increases with the increasing neutral axis depth to reach it's maximum at $\alpha_c = 0.5$. and thereafter decreases to reach zero at $\alpha_c = 1$. The stress in reinforcement however, increases with the

decreasing x/d to reach it's design yielding stress at x/d equal to Ecu/Ecut + Esd).

Generally the graph of -B relation is a second degree parabola. *Fig 2.0.* indicates the various reinforcement requirements of a concrete section under various combinations of axial load and bending structural members under axial load or combined axial load and bending are known as columns. These are normally vertical members supporting floors or beams, but horizontal or inclined members like structs may be considered as columns if the loading combinations are as stated above. When the ratio of the cross sectional dimensions of a vertical load-bearing member is more than four, the member although known as a wall can be designed as a narrow column. It is recommended that a minimum eccentricity of 0.05 times the minimum depth of section be applied to any cross section of concrete. Note that this recommendation does not consider the amount (or heaviness) of the axial load. The strength of a column is dependent on its effective length which is related to the critical buckling load given as $^{1}EI/1^{2}e$ where EI is the flexural rigidity. For short columns the deflection is negligible.

2.5.1 Axially Loaded Short Columns

Fig 4.3 shows idealised case of a short column under pure axial compressive load. The column is reinforced longitudinally with four bars linked by lateral ties to prevent their outward buckling. Due to the axial loading and bond between concrete and steel, the longitudinal strains of concrete and steel are uniform across the cross section and are equal.

From static equillibrium condition,

$$N = fc\ Ac - fs\ Asc$$

under service loading stresses in the concrete and in the steel are sufficiently low to assume linear deformation of both materials. With that range of loading then the elastic theory applies and it follows that

fc = N/Ac + cAsc) where c here means the modular ratio which is noted to increase with time of loading, due to creep and shrinkage of the concrete. This causes gradual redistribution of internal forces. The transfer of forces from concrete to steel may continue for years untill the effective modular ratio settles to an approximately constant value which is very difficult to determine with precision. When the column is subjected to several load applications and

removals, the residual stresses constitute further complications in elastic analysis of reinforced concrete columns. In fact it is almost impossible to determine actual stresses in the concrete and steel accurately therefore the elastic analysis does not provide satisfactory basis for design calculation.

2.5.2 Load Carrying Capacity

Ultimate strength of a concrete column is practically independent of the load history. Thus the load carrying capacity of a column may be expressed as $N = 0.67fcu\ Ac - fyAsc$ .....2.5.8a. Sometimes fy in equation 2.5.8a is reduced by 0.67fcu to take care of concrete displaced by reinforcements. When the eccentricity is small and the ultimate load N is acting between the longitidunal refinforcement (i.e $e = h/2 - d^1$), part of the column section may be considered as axially loaded. It is in this case assumed that the resultant of stresses occuring in the part of section behind the eccentricity is balanced by the resultant of stresses occuring on the remaining part of the section which may be expressed by

$$0.67f_{cu}\ b\ (\frac{h}{2} - c) + f'_{sy}A'_s = 0.67f_{cu}b(\frac{h}{2} + e) + f'_y\ A_s \quad ....2.5.7$$

assuming that both sides of the equation 2.5.7 are equal an approximate value of the ultimate resistance of a rectangular column is given by

$$N = 2\ (0.67f_{cu}b\ (\tfrac{h}{2} - e) + f'_y A'_s) \quad \ldots\ldots\ldots\ldots\ldots\ldots 2.5.8$$

provided the reinforcement at the less compressed side is not less than

$$A's - 2\ (0.67f_{cu}/f'_y)be \quad \ldots\ldots\ldots 2.5.9$$

which follows from equation 2.5.9 that for small eccentricities symmetrical reinforcement is then required.

2.5.3 Design for Ultimate Limit State

In design, the load carrying capacity is modified by the partial factors of safety as recommended by CP110. Thus for axially loaded column.

$$N = 0.45fcu\ Ac + 0.72fyAsc \quad \ldots\ldots\ldots\ 2.5.11$$

Due to construction tolerances of eccentricity of loading equation 2.5.11 is modified to be

$$N = 0.4fcu\ As + 0.67\ fyAsc \quad \ldots\ldots\ldots\ldots\ 2.5.12.$$

for columns where the eccentricity does not eceed the value of $(h/2 - d_1)$ and the ultimate load does not

exceed

N = 0.4fcu (h-2e) only nominal reinforcement is provided which is not taken into account in the resistance calculations. Thus equation 2.5.9 may be modified as

$N = 0.4fcub(h-2e) + 1.34fy\ A_1s$ ..... 2.5.13.

provided As $\geq$ A's - *(fcu/fy)be*

For short columns supporting approximately symmetrical arrangement of beams under uniformly distributed load equation 2.5.12 is further modified to give

N = 0.35fcuAc + 0.60fy Asc ..... 2.5.14.

Tests have shown that above formulae are not applicable to columns with a very low reinforcement. Therefore CP110 recommends that the minimum cross sectional area of longitudinal reinforcement should not be less than one percent of the concrete cross-sectional area. *For lightly loaded members*, it is sufficient to provide a minimum area of Asc 0.15N/fy.

2.5.4 Eccentrically Loaded Short Columns

When the eccentricity exceeds a certain value still small, the distribution of strain and stress in the section are rather similar to those in a beam

EASTERN LIBRARY

section and the columns undergo the same types of failure that occurs in beams. Therefore the analysis of a reinforced concrete column resisting both a moment and an axial load is based on the same assumptions as those used in bending strength theory. In particular it is assumed that the column is at failure when the concrete strain reaches a value of 0.0035.

2.5.5 Load Carrying Capacity of Eccentrically-Loaded Short Columns

Using the general dimensionless equations 2.5.4 and 2.5.7 but replacing d with h. For various assumed values of compression depth, the strains of the reinforcements and thus their stresses are calculated. Therefore for given values of α_c and reinforcement ratios, the curve of $\alpha - \beta$ relation is drawn which is known as interaction diagram.

In the design of eccentrically loaded section the principal equations are those of the section under flexure but with an unbalanced axial load. The principal design equations are:

$$A'_s = \frac{N\left(e + h/2 - d_2\right) - 0.2 f_{cu} b h^2 \left(0.75 - \frac{d^2}{h}\right)}{(0.72 f_y)(h - d_2 - d')} \quad \ldots\ldots 2.5.15$$

$$\text{and } As = \frac{0.2fcubh + 0.72fyAs - N}{0.87\ fy}$$

Oleszkiewicz (1985) stated that when A_1s is negative it means concrete compression zone is not fully utilized.

2.5.6 Summary

The assumptions of the unified general dimensionless load capacity of a reinforced concrete section under axial load and bending with the resulting α - β relation curves as shown in fig 2.0. are similar to that by CEB and quite unique in the analysis of reinforced concrete columns. The use of the word "small eccentricity" as has been noted by most designers is not clear since there is no definition of how small an eccentricity will be to be regarded small. The only attempt to define small eccentricity is in the specification for minimum construction tolerance of five percent of depth of section, and the provision of nominal reinforcement when eccentricity is less than half the concrete core.

$e \leqslant h/2$. The load capacity of a reinforced concrete column with small eccentricity is given as $2(0.67fcub\ (h/2-e))$ provided $As \geqslant A_1s - 2be$

(0.67fy/fy). Nothing is said of the case when As A_1s - 2be(0.67fcu/fy) nor what determines when As becomes more or less than 2be (0.67fcu)/fy. When N(e +h/2 - d_1) is less than the value of 0.2fcubh$_2$ (0.75 - d2/h), full use of the compression zone is not made. It must be noted that in the formular for estimating the load capacity of columns with "small" eccentricity, the position of the reinforcement (in terms of the effect of concrete cover) does not matter.

2.6 Concrete Confinement In Reinforced Concrete Columns

At low stress levels the transverse reinforcement referred to as hoops or ties remain relatively un-stressed in concrete sections. At high stresses, approaching the uniaxial strength of the concrete, the transverse steel provides some confinement to the concrete when under stress. The most common cause of collapse of a building in an earthquake is said to be insufficient ductility of the bearing walls and columns. (Barr et al 1982; Fischer 1984; Harris et al 1982). Most of the research into concrete confinement was as a result of effort to improve on the performance of concrete structures under

earthquake. Concrete confinement provides ductility to the structure which is a very essential requirement of earthquake resisting structure (Graig 1982; Basu 1983). The two common forms of transverse reinforcements are the rectangular ties and the circular spirals. For obvious constructional reasons each is associated with the rectangular and circular columns respectively. The initial intention of the provision of ties was as a guide to the main reinforcements. With increasing knowledge it became obvious that the spacing of the ties affected the strength of the concrete sections. It is generally agreed by most researchers that the circular spirals are more effective in concrete confinement than the rectangular ties. (Chiwazie 1975; Maruyama et al 1984; Oliva et al 1980; Sheikh 1982). The reason for the difference in effectiveness is said to be, because of the shape of circular spirals, they are in axial hoop tension and provide a continuous confining pressure around the circumference of the column. Rectangular ties on their own will apply confining pressure near the corners of the ties, since lateral pressure from the concrete will cause laternal bowing of the tie sides. A ratio in effectiveness

between the spirals and the rectangular ties has been suggested to be about 2. for the same volume of steel (Ahmad et al 1982). While most researchers agree to the difference in effectiveness between rectangular ties and spirals. There is a division in opinion on both the magnitude of this increased ductility and in the contribution of ties to strength enhancement of confined concrete. It had been observed that tied columns do not have much ductility espcially in tall buildings where eccentricity ratio may be small (Fintel 1974). Sheik (1980) studied the effect of ties to strength enhancement in short concrete columns. Twenty four columns with height-breadth ratio of about six were tested to failure. The spacing of the ties, which were multiple tie sets in various arrangements, range between 100mm to 30mm, thus producing spacing-depth ratio (Sv/h) of between 0.33 and 0.10. These of course were very high tie steel contents. It was observed that the concrete core and the cover behave differently at longitudinal strain values of 0.0015 - 0.002. Most of the columns attained about ninety percent of their failure loads at strain values of 0.002. The following conclusions among others were made by Sheikh (1980).

(a) Concrete when confined with rectangular ties and well distibuted longitudinal steel, exhibits a very significant strength gain of up to 70 percent, and increased ductility.

(b) A closely knit cage both in longitudinal and lateral directions increases the efficiency of confinement. Reduced tie spacing results in higher concrete strength and ductility even though the stiffness of the tie steel is reduced by reducing the size of the tie to maintain the same volumetric ratio.

(c) Within the range of values used in the tests, longitudinal reinforcement appears to have little effect on the behaviour of the confined concrete. Conclusion (b) above does not agree with earlier observation by Park (1975) that the ratio of the diameter of the transverse bar to the unsupported length of transverse bars is significance in the case of rectangular stirrups, because a larger bar diameter leads to more effective confinement. Equations for concrete strength enhancement by confinement were proposed (Ahmad et al 1982; Papia et al 1988). The strength of concrete cylinders loaded axially to

failure while subjected to confining fluid pressure is given by

$$fcc = fc + 4.1fl \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.6.1$$

where fc is the axial compressive strength of confined specimen and fl - lateral confining pressure given by

$$fl = \frac{2*Asp*fst}{ds*S} \quad \ldots\ldots\ldots\ldots\ldots \quad 2.6.2$$

Asp is the area of spiral steel; ds is diameter of spiral; fst is the stress in tie steel; S is pitch of spiral. The axial compressive strength of concrete confined by spiral steel is given as

$$fcs = fc + 8.2\ (fy\ Asp)/ds*S \quad \ldots\ldots\ldots \quad 2.6.3.$$

The volume of spiral steel per unit length of column core is given as

$$Vs = \frac{Asp*\prod*ds}{S} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.6.4$$

Thus the concrete confinement could be expressed in terms of the volume of spiral Chiwuzie (1975) studied the contribution of ties in short reinforced concrete columns. He observes that Nigerian Code of practice (NCP1) specifies that the permissible axial load should not exceed N where N = fcAc + fsc Asc for tied columns and N = fc Ak + fsc ACsc + 27000* Ab for spiral

reinforcement where Ak is concrete core area excluding the cover and the area of steel. Ab is the equivalent area of helical reinforcements that is volume of helical reinforcement per unit length of columns Chiwuzie (1975) crushed columns with height-breadth ratio of six. Three column sizes with single and doble tie arrangements were considered. The tie spacings were 40, 80 and 120mm resulting in spacing-depth ratio of between 0.4 and 1.2. Table 2.1 below presents Chiwuzie's (1975) test results.

27000 Psi is equivalent to $189n/mm_2$

Table 2.1: Chiwuzies Test Results

Specimen	Sv	Size of Col	fcu	e %	Average Ultimate Load
A4	40	100 x 100	25	1.88	28.02 tons
A8	80	100 x 100	"	"	26.47 "
A12	120	100 x 100	"	"	24.40 "
B4	40	100 x 150	"	1.25	38.40 "
B8	80	100 x 150	"	"	36.90 "
B12	120	100 x 150	"	"	36.60 "
C4	40	100 x 200	"	1.40	55.90 "
C8	80	100 x 200	"	"	52.17 "
C12	120	100 x 200	"	"	48.20 "

Group A specimens as well as group B, had four bars at the corners. While group C specimen had six bars; four at the corners and two at the middle. Chiwuzie (1975) observed that currently he could not say the lowest value of ultimate load with respect to ite spacing. For instance at high spacings of 140mm the graph from a research work gave the ratio of prism strength of unreinforced columns to the cube strength as 0.865 for specimen group A, 0.865 for specimen group B and 0.885 for group C. The results suggest that these ratios will be much lower for further increase in tie spacing. Chiwuzie (1975) observed that in the

tests carried out at Leeds University Orangun obtained scattered values from 0.56 to 0.89 with the average of 0.71. While tests on plain gravel concrete columns by another researcher varied from 0.56 to 0.92 with the average of 0.68. From his results, it was clear that for tie spacing of between 100 percent and 150 percent of the minimum depth of section, the effects of pitch on ultimate load becomes less important irrespective of column size and shape. This Chiwuzie (1975) contends, account for Hudson's conclusion that the effect of lateral ties on the ultimate load for short columns is neglibible, since his spacing was between 100 - 800 percent of the minimum depth of the columns tested. At the Technical University of Cracow (Chiwuzie 1975), the ultimate strains of confined concrete recorded were between 0.0063 - 0.0062 for spacing of 50mm, 0.0046 - for spacing of 125mm and 0.0036 - -.0045 for spacing of 200mm. It was observed that the ultimate strain almost doubled with decrease in spacing from 200mm to 50mm. Chiwuze (1975) observed that some researchers suggested emperical formular for concrete increase in strength due to confinement by ties. These are given in terms of the ultimate load capacity of short columns

(1) Blume et al

$$N = 0.67\ fcuAc + Asfy + \frac{2.05m\ Ak\ Asb\ fyv}{}$$

Asb is the area of cross-section of a leg of hoop reinforcement, a - is the longer outside dimension of hoop reinforcement; Sv - spacing.

(2) Oleskiewicz

$$N = Ac\ fc\ \left(2 - \frac{1}{Ebu}\right)\left(\frac{El}{Et}\right) + As\ Es\ Ei \quad \ldots\ 2.6.6$$

where fc is the characteristic strength of concrete as defined by adequate ratios of cylinder and cube strengths or measured samples of plain columns, or estimated from graph.

El is the actual unit deformation of the column

Ebu is the ultimate strain of concrete at each load.

Blume et al and Oleshiewicz also gave the formula for the loads carried by the ties as Blume - Ntie

$$= \frac{2.05m.Ak.Asb}{a.s}\ fyv \quad \ldots\ 2.6.7$$

$$Olesz - Ntie = \frac{m\ Ast\ at}{U\ Sv}\ fyv \quad \ldots\ 2.6.8$$

U is the poisson's ratio. It was observed that the tests carried out indicate higher load capacities than that suggested by Blumme et al.

The difference being between 8 - 21 percent. The results of Chiwuzie (1975) agree with the suggestion by Oleszkiewicz. The following specifications on ties are made by various codes. (Chiwuzie 1975).

(a) NCP1/CP110 - The spacing of tie should not be more than the minimum depth of the section, twelve times the mimum diameter of main bar or 300mm. While the diameter of the tie should not be less than one-quarter of that of the largest bar size.

(b) American Code - The spacing of tie should not be more than sixten times the diameter of longitudinal reinforcement, forty-eight times the diameter of the tie which is not less than 6mm, or minimum depth of column.

(c) Polish Code - Spacing should not be more than the minimum depth of section and not more 400mm. The diameter of tie should not be less than one-quarter that of main reinforcements nor 4.5mm.

(d) Russian Code - The spacing should not be more than fifteen times the diameter of longitudinal reinforcement, the minimum depth of section nor

500mm. The Russian Code adds that:-

(1) Binders are of great importance. They ensure that the concrete will attain cube crushing strength.

(2) Prevent buckling of the longitudinal reinforcements.

Chiwuzie (1975) concludes that his results agree with the suggestions of Oleszkiewicz and that the specifications on ties, by NCP1 require to be reviewed. The results of Chiwuzie (1975) indicate that using the traditional strength formula for short concrete column without the effect of confinement will give an average difference of about 25 percent from recorded values, for specimen groups A and B, while 31 percent for group C. Mander (1988) suggests a relation for confined compressive strength of concrete as

$$fcc = fco\left(1-1.254 + 2.254\sqrt{1 + \frac{7.94fc}{fco}} - 2\frac{fl}{fco}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 2.6.10$$

where fcc is the confined strength and fco the unconfined strength of concrete and fl is given as fl = 1/2Ke es fyh 2.6.10a

$$Ke = \left(1 - \frac{S}{----}\right)_2/1-Pcc$$

2ds)

The assumption here is that the confining force is equal to the hoop force in the tie. That is

$$2\ f_{yv}\ A_{sp} = f_l\ S.\ d_s \quad \text{..... 2.6.12}$$

where

S - spacing of ties

dc = diameter of spiral

ecc = ratio of longitudinal reinforcement to the area of the concrete core section.

es = ratio of transverse confining steel to the volume of confined concrete core.

$$e_s = \frac{4\ A_{sv}}{d_c\ S_v} = \frac{(A_{sv})}{(S_v)} \frac{(\text{Perimeter of tie})}{(\text{Area enclosed by tie})}$$

..... 2.6.13

The equation presented for the concrete strength increament due to confinement is rather not simple.

Summary

Transverse reinforcement in concrete columns enhance the strength of such columns by confining the concrete. Circular spirals are generally agreed to be more effective in the confinement of concrete than ties, this being mainly because of the continuity of the spiral around the circumference of the column.

There appear to be no agreement on a simple formular for the strength enhancement of concrete by ties, however most postulations may simply be represented as

$$fcc = fcu + K\ Sv \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.6.9$$

where K depends on the strength of the tie, area of tie and configuration. To achieve appreciable increase in strength of concrete due to ties, the nominal specifications of most codes would not apply and the amount of steel required will be considerably high, amounting to steel cages. For tie spacings, more than the minimum depth of the section, there is almost no contribution of ties to strength enhancement. The ultimate strains obtainable in a concrete section (column) depends greatly on the tie spacing. For tie spacings more than the depth of the column, the ultimate strains under axial loads may not be up to 0.002. Most researches carried out presently are on concentric loading of the short columns. Not much attention appear to have been given to confinement in eccentrically-loaded short columns, nor to columns of high rise buildings where the eccentricity is low due to high axial loads. From all the researches, it is clear that no matter the closenss of ties in reinforced

concrete columns, the steel longitudinal reinforcements buckle. Infact the buckling of the bars is a sign of attainment of high strains due to confinement. Without confinement, in axially loaded columns, the steel main reinformcement do not attain high strains and thus, do not buckle at column failure.

2.7 Uniaxial And Biaxial Stress Response of concrete

A brief summary of the short term uniaxial and biaxial response of concrete to stresses is presented. Concrete contains large number of microcracks especially between coarser aggregates and mortar interfaces and under load these microcracks increase in size this resulting in the non-linear benaviour of concrete. (Chen 1982; Mindess et al 1981). Some microcracks are said to be caused by segregation, shrinkage or thermal expansion in the mortar while others develope during loading due to the relative difference in the stiffness of the aggregates and morter. It is also possible that similar microcracks develope in reinforced concrete due to the relative stiffness of between the steel reinforcement and the concrete at their interface. (Chen 1982). The stress-strain relation is approximately linear at a

maximum stress of about thirty-percent of the strength of the concrete and below. For stresses above thirty percent of the strength, the curve shows a gradual increase in curvature up to about 0.75fc and 0.90fc after which it bends sharply and approaches the peak point fc. Beyond the peak the stress-strain curve decends until it approaches a point known as the ultimate strain and the concrete crushes. The exact criteria for failure of concrete is not known. However there are strong indications that failure occurs at a limiting strain of 0.002 - 0.004 in compression and 0.0001 to 0.0002 in tension. Various codes specify ultimate strain value (for comspression) of between 0.003 and 0.0035 (Parrott 1974). Since concrete under uniaxial compression does not crush at the ultimate strength, and since a reinforced concrete specimen, a column for instance, fails at the maximum load if under increasing montonic load. It appears therefore that at the attainment of maximum stress by the concrete, some of the load carried by concrete will begin to be transfered to the steel reinforcements in a manner similar to plastic redistribution of forces in a reinforced concrete joint. At the stress range of

0.5fc to 0.75fc, if the concrete specimen is unloaded and reloaded, the unloading curve exhibits some nonlinearity thus resulting in a characteristic hysteresis loop for continued loading and unloading. The true modulus of elasticity of concrete does not exist as concrete is not perfectly elastic. Approximations of the young's modulus of elasticity are expressed in various forms such as the tangent modulus etc. The determination of the modulus of elasticity is a very difficult exercise because it requires constant rate of stress application. (Avram et al 1981; Oladapo 1981). Various postulations have also be given as a relation between the short term modulus of elasticity and the strength of concrete for design purposes. The American concrete Institute proposed an emperical relation $Eo = 33w_{1.5}$ fcy psi where w is the uniaxial cosmpressive strength of concrete cylinder. CP110 specifies

$$E = 55 \frac{fcu}{m}$$

In respect of lateral deformation of concrete under stress, the poisson ratio for concrete is said to be between 0.15 and 0.22 a representative value being 0.2.

It was however observed that poissons ratio for concrete depends greatly on the moisture loss during pre-drying. The effects of porosity, stress and age of the concrete at the time of loading are insignificant. Poissons ratio for cement paste may be obtained using the experical formular

$$U = 0.214\ V$$

where V represents the moisture loss during predrying. Chen 1982; Pauly 1982.

The stress-strain relation for concrete in tension which is similar to concrete behaviour under compressive stress. However, the curve is nearly linear for higher levels of stress. The uniaxial tensile strength of concrete is between 0.05 to 0.1 of the uniaxial compressive strength. The direct tensile strength is difficult to measure and may be taken as

$fty = 4\ fcy$ (psi) where fcy is the compressive cylinder strength and fty the tensile cylinder strength.

2.7.2 Biaxial behaviour of Concrete

Considerable research has been done recently on the biaxial response of concrete to stress. It was observed that the maximum compressive strength

increases for the biaxial compression state, achieving a maximum strength increase of about 25 percent for a stress ratio, O/O2 of 0.5 (Parrott 1974.) As the failure point is approached, an increase in volume occurs as the compressive stress continues to increase. This inelastic volume increase called dilatancy is usually attributed to progressive growth of major microcracks in concrete. (Papia et al 1982). The widely used function for simulation of stress-strain curves of concrete under biaxial state of stresses is given

$$\text{as } O = \frac{aE}{1+((aEp(op)-2)(E/Ep) + (E/Ep)^2} \quad \ldots\ldots 2.7.1$$

where O, E are stress and strains in principal axis

Op, Ep are experimentally determined maximum principal stress and corresponding strain.

a - experimentally determined coefficient representing initial tangent modulus.

Summary

The stress strain behaviour of concrete under uniaxial and biaxial stress states are described. Concrete being a plastic material does not have a

real youngs modulus of elasticity. Approximations are adopted for short term design purposes. Concrete does not fail at it's maximum stress. In a reinforced concrete section under increasing monotonic load, since the section fails at maximum load, it appears therefore that after attaining maximum stress, the concrete distributes it's load to reinforcement to attain the maximum failure strain. The strength of concrete in biaxial stress state is observed to be higher than the strength under uniaxial stresses.

2.8 Finite Element Method For Concrete Columns

The concept of the finite element procedure is that the behaviour of a continuium whose degress of freedom is infinite can be analysed by discretization of the contium into elements with finite degrees of freedom. The individual behaviour is carefully described to follow known laws so that when the elements are reassembled their combined behaviour is the same as a continium. This method of analysis have become fashionable in concrete columns and indeed reinforced concrete, as a means of accurately approximating complex concrete behaviour due to material and geometrical nonlinearity. The basic steps involved in

the procedure are

(a) Discretization of the continium into finite elements.

(b) Definition of the behaviouur of the elements.

(c) Imposing compatibility conditions on the elements.

(d) Assembling the behaviour of elements.

(e) Testing the convergence of results.

Two methods have been commonly used for the numerical solution of the governing differential equation of a column subjected to an axial load and end moments. Both approaches require the moment-thrust-curvature relationships of the cross section. (Hsu 1985; Kayal 1984). For elastic cross-section of a single material the moment-thrust-curvature relation is adequately represented by the bending equation:

$$\frac{f}{y} = \frac{M}{I} = \frac{E}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 2.8.1$$

where f is the stress, y the distance of the most stressed fibre, M is the moment, I is the second moment of area, E is the youngs modulus and R is the curvature radius. However, in a composite section or a biaxial bending case, it becomes almost an impossible problem. Thus numerical procedures become unavoidable. Before

the use of the approach by Newmark (Meek 1963), the method of determining the deflected shape of a column under axial load and end moment by Von Karman was quite popular.

Two approahces can be adopted in studying the ultimate behaviour of columns with both material and geometrical non-linearities. The first is by applying a small fraction of the expected failure load to the column and determining the deflected shape. The process is repeated with higher load levels untill at a certain level, the deflected shape in equillibrium with the applied loading cannot be obtained. The highest load for which a delfected shape can be obtained is then taken as the ultimate load of the column. The other approach is to impose some strains on the column and to determine the forces and moments using the moment-thrust-curvature relation, untill a final value of load for which the equillibrium and the compatibility conditions are satisfied is attained. This highest load is the ultimate load capacity of the reinforced concrete columns. The convergence of the load value Ni for an impposed initial strain value in an iterative process to the ultimate load N can be

accelerated using a modification of the Newton-Raphson method (Hsu 1985). Rectangular elements have appeared to be very popular among researchers for it's ease.

Consider the element k with it's centroidal point Uk Vk referring to the principal axis. The strain Ek across the element k can be assumed to be uniform and since plain sections remain plain as is usually assumed for numerical analysis, $Ek = En + _ Ou\ Vk + Ov\ Uk$ .....2.8.2 where En is the uniform direct strain due to an axial load N, Ou is the curvature produced by the bending moment component Mu and Ov the curvature due to moment component Mv. Ou is considered positive when it produces compressive stresses in positive V direction also Ov is considered positive if it produces compression in the positive U direction. Having established the strain distribution across the section the axial load and bending moment components can be calculated using the following equations (Hsu 1985)

$$N(c) = \sum_{k=1}^{n} fk\ ak \quad \text{.....2.8.3}$$

$$Mu(c) = \sum^{n} fk\ ak\ Vk \quad \text{.....2.8.4}$$

$$Mv(c) = \sum_{k=1}^{n} \quad fu\ au\ Uu \quad \text{.....2.8.5}$$

subscript (c) indicates values of N1 Mu and Mv calculated in an iteration cycle. While ak and fu are the area and stress of the element k respectively. For a known geometry and material properties the stress resultants N1 Mu, Mv can be expressed as functions of Øu, Ø v, and En by the following equations. $N = N(Øu, Øv, Ew)$ 2.8.6

$Mu = (mu(Øu, Øv, En)$.....2.3.7

$Mv = Mv(Øu, Øv, En)$.....2.3.3

Since the principal axis are taken for analytical purpose, co-ordinates' transformation is an important procedure. The following steps can be used

(1) Find the moment of Inertia Ix, Iy and the product moment

of Inertia Ixy.

(2) Use the equation tan 20 = 2Ixy/(Iy-Ix) to determine the angle between the centroidal and principal axis:

$$\begin{pmatrix} u \\ v \end{pmatrix} = R \begin{pmatrix} x \\ y \end{pmatrix} = \begin{pmatrix} \cos\Phi & -\sin\Phi \\ \sin\Phi & -\cos\Phi \end{pmatrix} \begin{pmatrix} x \\ y \end{pmatrix} \quad \ldots\ldots\ldots 2.8.9$$

$$\begin{pmatrix} Mx \\ My \end{pmatrix} = R \begin{pmatrix} Mu \\ Mv \end{pmatrix} = \begin{pmatrix} \cos\Phi & \sin\Phi \\ -\sin\Phi & \cos\Phi \end{pmatrix} \begin{pmatrix} Mu \\ Mv \end{pmatrix} \quad \ldots\ldots\ldots 2.8.10$$

Two concrete stress-strain polynomials are currently proposed. The polynomials are

(a) $$\frac{fc}{fcp} = \sum_{i=1}^{4} Ci \left(\frac{E}{Eo}\right)^{i} \quad \ldots\ldots\ldots 2.8.11$$

where C_1= 2.41, C_2 = 1.865, C_3 = 0.5 and Cy = 0.4=045. This was proposed by Basu.

(b) The proposal by Cranston is given as

$$\frac{fc}{fcp} = \sum_{ic1}^{4} Ci \left(\frac{E}{Eo}\right)^{i} \quad \ldots\ldots\ldots 2.8.12$$

where C_1 = 2, C_2 = 0.6732, C_3 = 0.6536 and C_4 = 0.268. The strain at peak stress and the crushing strain are taken as Eo = 0.002 and Eu = 0.0035. For the steel on elasto-plastic stress strain curve is usually assumed. In his research on I-shaped

columns, HSU (1985) used rectangular elements of 0.5 inch sizes because the steel reinforcements are the No.3 bars which have their diameters about that size. Figs 2.4.a - 2.4.c show the section. Ix = 144.8in^4, and Iy = 81.5in^4; Ixy = -43.4in^4 , 0=27 of the tested ten specimens, each was six feet high. Six of the specimens were tested for monotonic loading while four were tested for cyclic loading. The monotonic loading gave an average ratio between experimental load and analysis load as 97 percent while 83 percent was recorded for cyclic loading.

Kayal (1984) made a number of assumptions which include

(1) Tensile strength of concrete is negligible.

(2) Failure of the column is by excessive bending. Instead of dividing the column into segments Kayal (1984) assumed a single beam column element with five degrees of freedom - two end rotations, two end diflections and a mid section deflection. Chen (1980) in his own work obtained the segment stiffness by solving the governing differential equations for equillibrium instead of assuming a displacement the finite segment method developed previously for wide flange steel columns under biaxial bending, material and geometric non-linearities overcome by iterative procedure.

2.8 Summary

The finite element method of analysis of concrete columns is an effort to solve numerical non-linear problems either as extension of numerical solutions to complex linear problems or numerical solutions of large continiums, each time overcoming the problem of non-linearity by iterative process. The basic process may be summerized in a few steps as:

(1) For unknown curvatures or assumed values the cartesion coordinates (x,y) for each element is determined thus also determining the distance to the neutral axis of the element and indeed all other elements.

(2) The imposed strains at the elements can then be calculated where there exists residual strains, the strains calculated have to be added to the residual strains interpolated for the particular element to obtain the total strain of the element.

(3) Using the material stress-strain curve as assumed the stresses of the elements are obtained.

(4) With the known natural coordinate for each point and the cartesion coordinate of the vertices the value of the determinant for each element is evaluated.

(5) The force and moments are determined and compatibility imposed on them with respect to the segments.

(6) The curvature is varied to produce a maximum load combination just before equillibrium cannot be obtained.

Generally, in the current finite element analysis procedures the assumptions of bending and the determination of the moment-thrust-curvature relation may not be suitable for the analysis of reinforced concrete short columns with small eccentricity where the radius of curvature approaches infinity.

Where it is difficult to predict the safety of a structure mathematical model tests serve alternative or supplementary role. Modelling is the process of representing a structure in a smaller scale for the pupose of effectively studing the behaviour of such structure. The use of physical models in studies, although an old process became more popular with increasing complexity and sophistication in engineering with it's increasing risks. For a model to adequately simulate the behaviour of a prototype, the relationship between the model and the prototype must be accurately established. This relationship represented in terms of scale depends on a number of factors which include (Abrams 1982; Gero 1973).

(a) Size of prototype

(b) Model material behaviour and sizes

(c) Level of precision required.

Having chosen a model scale ratio, the selection of model materials that will satisfy the simulation requirements is the most difficult step in the successful model investigation. The relevance of using small scale models for analysis of a particular structural form or loading may be seen with a few sample conclusions deduced from the measured response

(Abrams 1982; Gero 1973; Konishi et al 1981).

(a) Consideration of non-linear behaviour had no loss of servicibility.

(b) Distribution of lateral forces is more complex than may be assumed. Measured forces contained components at different frequencies which would make prediction of forces impossible.

(c) Stiffness and strength characteristics of small scale models were similar to those of large scale models. Comparison of measured behaviour revealed the same tendencies with respect to cracking, yielding of reinforcement.

(d) The use of models to study the response of reinforced concrete structures to short duration loads has been demonstrated to be an effective design tool. Where uncertainties, scale effects and incompatibilities arise, precise correlations can be determined by repeating the same experiment on enlarged scales although it must be stressed that there is no unique scaling law for any particular situation.

The use of replica models is the most reliable model test. Replica model scaling implies that the

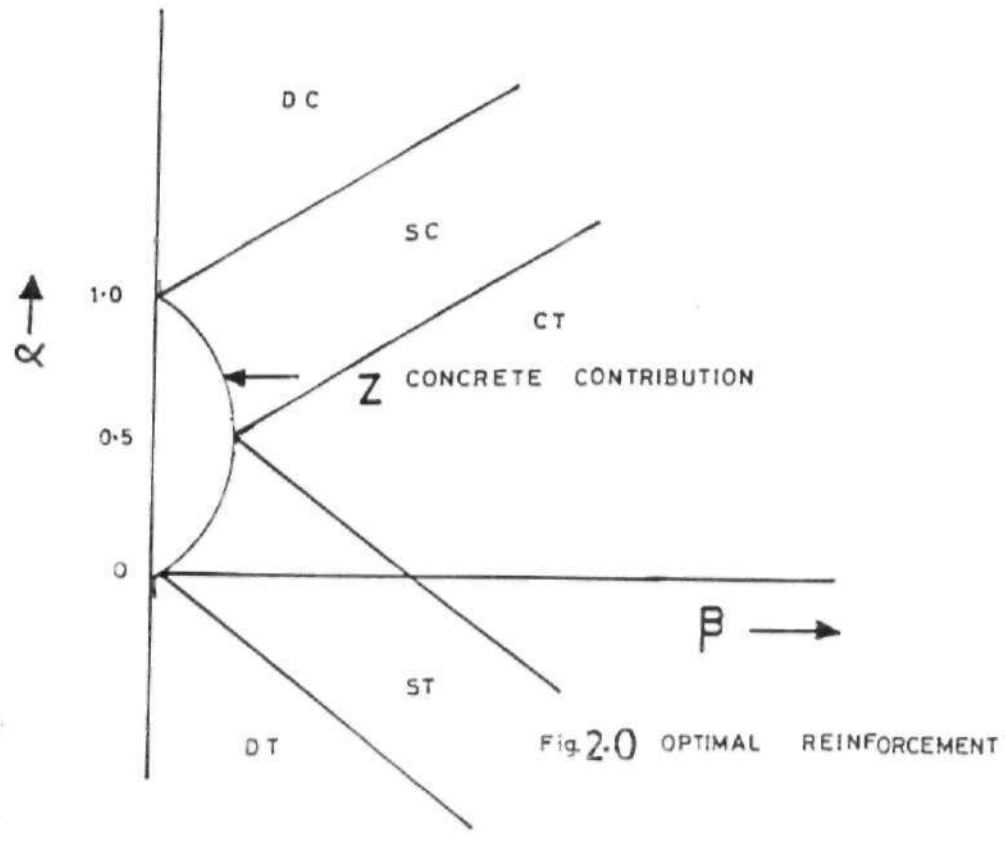
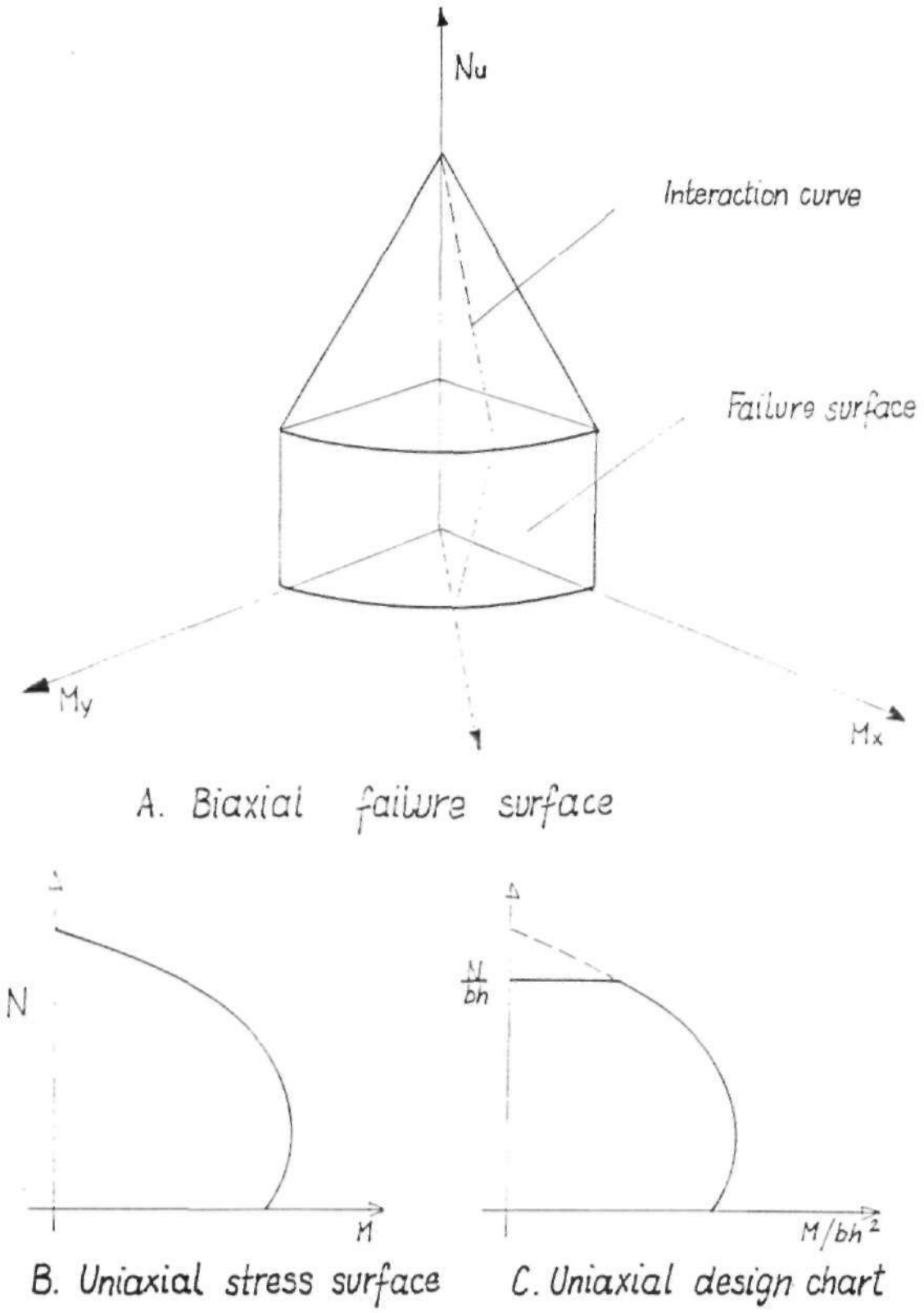


Fig. 2·0 OPTIMAL REINFORCEMENT

Plate 2 Digital Strain Meter



A. Biaxial failure surface

B. Uniaxial stress surface

C. Uniaxial design chart

Fig. 2.1 MOMENT-THRUST INTERACTION CURVES.

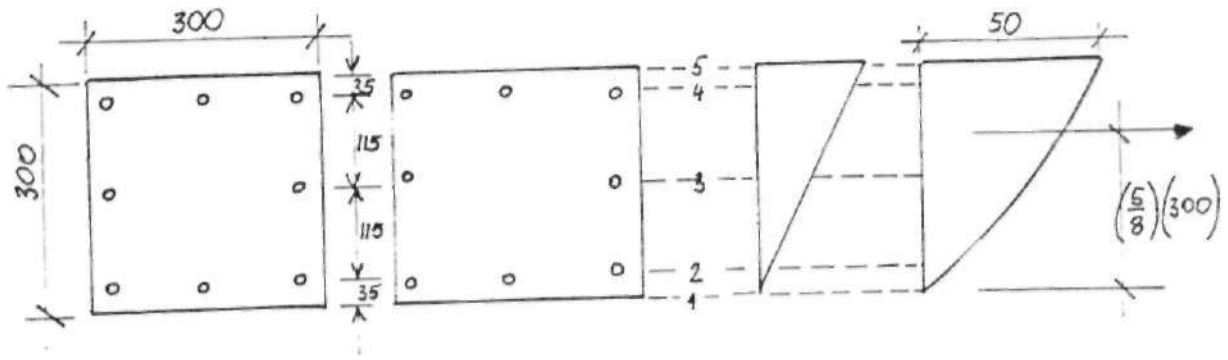
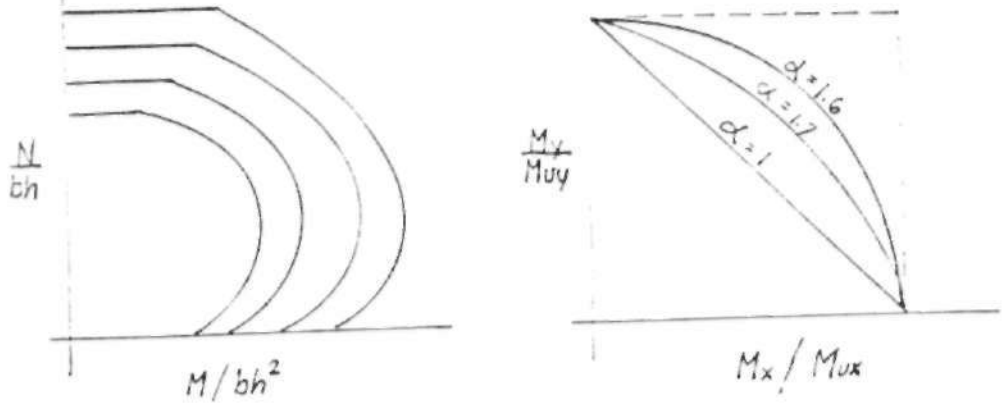


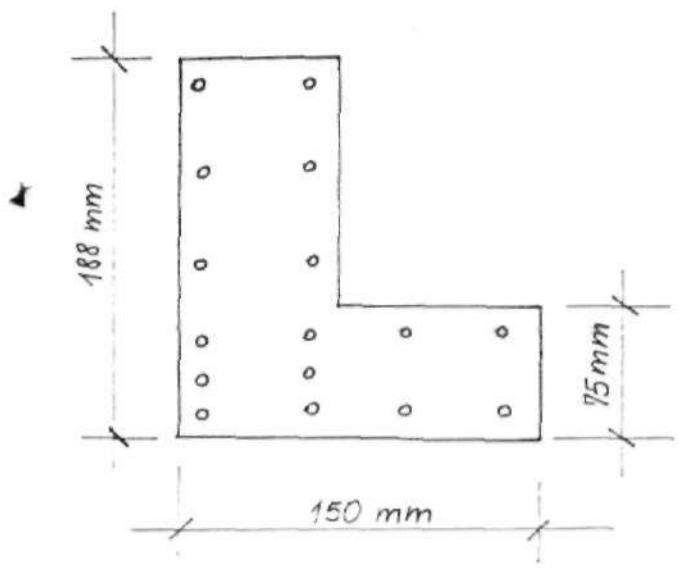
Fig. 2.2 DETERMINATION OF THE LOAD CAPACITY OF AN UNSYMETRICALLY REINFORCED COLUMN BY OLADAPO (1981)



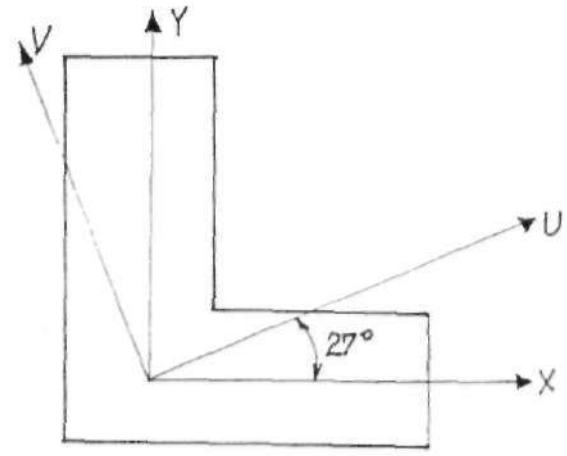
A. CP110 uniaxial design chart.

B. Biaxial load interaction contours.

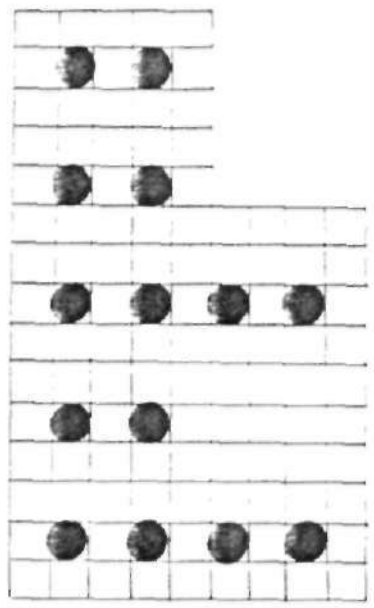
Fig. 2.3 DESIGN CHARTS OR AIDS.



A. Section indicating the arrangement of bars.



B. Global - local coordinates



C. Discredization of specimen

Fig. 2.4 L-SHAPED COLUMNS
(test specimens of Hsu etal)

dimensions of prototype and model, scale geometrically and the material properties of the model and prototype such as density and stress-strain relation are identical. Tasker (1947) as early as 1947 established the relationship between deflections and stresses in models to those of the prototype using dimensionless analysis. He concluded that the ratio of model stress to prototype stress was a one-to-one ratio for the applied loads. The analysis of scaling had tended to be on the basis of dimensionless quantity. In reinforced concrete column experiments, the scale ratio is measured in terms of the length-breadth ratio (Meek et al 1990). The definition of short columns as those with height-breadth ratios of between 7 and 12 does not allow for clear definition of replica model scale ratio.

2.10 Reliability in Reinforced Concrete Structures

To ensure the safety of a design or construction of a structure the safety coefficient as a ratio of expected load measured in terms of stresses, and the load capacity of the structure must be more than unity. With respect to the various codes of practice, such structure is said to be safe only when these

coefficients are larger than the prescribed design values in the absence of dynamic effects. (Brown 1988). The safety coefficient more frequently refered to as safety factor, depend on the following

(a) Quality of materials used

(b) Conditions of the building

(c) Risks which are run in using the building especially in terms of human lives*

(d) Precision of the mathematical calculation and the structural theories.

Reliability levels specified by various codes evolve after adequate understanding of the behaviour of such materials or structures as the case may be. Reliability based codes of practice are almost eclusively concerned with the design of single elements in a structure. The overall reliability of a structural system is an ideal alternative because the structural system is usually different from the elements. It has been reported that as many as one in fourteen (1/14) suspension bridges topples before its design life expires (World 1984), while other types of bridge failure rates range from one in ten to one in one thousand - one percent of Arch dams fail while two percent of rock fill dams give way.

About one in one hundred off shore structures fail as well as steel towers.

Typical failure rates are better for reinforced concrete and steel buildings; about one in ten thousand. Experience therefore show that the failure rate of a structure is perculiar to the type of structure. Below are the common causes of building failures . Values given are in percentages.

Foundation movement	29
Steel corrosion	22
Masonry failure	10
Deflection	9
Fire damage	5
Timber dry rot	4
Inadequate bearings	2
Movement joint malfunction	2
Underground roof leakage	2
Floor screed	1
Others	14

It was noted that inadequate supervision resulted into 47 percent of structural failures during construction. Not real standards on reliability principles existed

untill 1950 when a committee to specify design safety margins for the institution of structural engineers was inaugurated. (World 1984). The reliability of reinforced concrete elements as specified by various codes of practice have similar phylosophsy. (Pham 1984; Tasker 1947).

(1) The safety margins are higher for quantities that are less likely to be predicted accurately. For instance CP110 (1972) specifies higher margin for live load than for dead load as well as higher margins for concrete strengths than steel strengths because the later isw more predictable.

(2) Concrete strength is a variable which follow the normal distribution law. For every hundred concrete cubes made, not more than five is expected to have strengths less than prescribed. As was shown in example 2.3. the reliability or safety margin of a reinforced concrete element designed using the specifications of CP110 is variable depending on the intrinsic properties and tendencies of the member. Since the reliability or the failure rate of structures has been shown to depend on the type of structure, it shows that

using the same reliability level for similar reinforced concrete elements in all structures need to be reveiwed. However this also indicates that the low failure rate of reinforced concrete structures may be due to conservative nature of the design assumptions. Developing countries such as Nigeria for instance apply reliability levels specified by Britain and America religiously without considering the need for local conditions. In a research report (World 1986) it was observed that the chloride concentration of 0.6 - 1.0kg/m^3 typical in concrete made with local materials in eastern Saudi Arabia and 0.6 - 3.0 kg/m^3 in core samples taken from existing 15 - 20 years old structures is significantly higher than the 0.6 - 0.9 kg/m^3 levels necessary to initiate erosion. As the steel rusts it expands creating a mechanical bursting pressure that cracks the concrete cover eventually causing spalling that exposes the reinforcement. Countries like Saudi Arabia cannot precisely specify reliability levels of concrete structures based on specifications and design safety margins. The difference in habit of the user of the structure is also an important factor in reliability

analysis.

Conclusion

It was observed that failure rates of structures depend on the type of structure with reinforced concrete having the least failure rate. It may be wasteful to design the same reinforced concrete element with the same safety margin inrespective of the type of structure. The reliability of structures in the developing countries designed on the basis of foreign codes may not possess the same reliability level as envisiaged. It was concluded for instance that the maximum percentage reinforcement as specified by western codes is not suitable to the Gulf region. A maximum of six percent was suggested. In the Gulf region and the tropics cover is important in ensuring the reliability of structures. By ensuring good cover, clean joints, good concrete mix and adequate supervision, the life span of a concrete building could be increased from 5 to 30 years.

2.12 Current Researches on Reinforced Concrete Columns

Research on reinforced Concrete Columns started seriously in the early fifties, most of which were geared towards the evolvement of the various national

and international codes of practice. Also at that time, the understanding of the behaviour of reinforced concrete was still at the primary stage, thus most of the researches were either assisting the production of the codes or evolving simple methods of design of reinforced concrete columns. Reinforced Concrete Columns have to date, not attracted as much interest as beams and slabs have, due partly to it's relative cheapness or effectiveness in load carrying, partly because less column failures have been recorded (Brown 1988) and because the problem of reinforced concrete is tension and bending (flexure) which are easily studied and amplified in beams and slabs. Recent researches on R.C. columns have been either to evolve earthquake resistant designs or on finite element method of analysis.

Abatan (1986) and Oleszkiewicz (1985) in their various works have tried to evolve unified system of design of reinforced concrete sections for ultimate limit state. Some of the principles adopted by Oleszkiewicz are explained in chapter 2.5 However he makes some classical assumptions one of which is that the distribution of strain in the concrete and

reinforcement across the section at failure is a linear function of the depth of the column. Also assumed is that in ultimate strength analysis the tensile concrete strength is ignored.

On the design charts and methods, Meek (1963) studied rectangular columns loaded axially with bending moments about both principle axis. He observed that the analysis of rectangular reinforced concrete columns for axial loads and bending moments about both axes attracted more attention because it is a more difficult problem in reinforced concrete theory and only limited test results have been published. No satisfactory equation has been developed to define the ultimate column load as a function of the principal bending moments. Several writers suggested the extension of the interaction curve for axial loaod and bending moment, to the problem of multi-axial eccentricity, to form what is referred as interaction surface. Meek (1963) compared the theoretical contour line of such interaction surface with results obtained by experimental testing. The classical assumption of triangular strain distribution across the section is assumed, a rectangular stress distribution. The

interaction surface shown in fig. 2.0. as assumed for the charts by Meek are mere extension of the uniaxial interaction curves which were noted to have been substantiated by test results. Denpending on what is made constant, horizontal or vertical curves can be generated within the interaction surface. Nine columns 125mm square with height/breadth ratio of 12 were tested to failure. The moments applied range between 28.3 and 95.6 in-kips (3.2 - 10.8 knm) producing eccentricity of between 34 and 26mm. He concludes that

(1) The concept of interaction surface is useful. If sets of contour lines could be drawn then biaxcally loaded columns would present no further difficulties. A designer should therefore produce interaction curves as these form the set of charts.

(2) Interaction curves are valuable for columns with equal areas of reinforcement in all faces.

(3) To obtain a reliable prediction of the ultimate load of columns with a set of loads (N, Mx, My), steel strains must be considered and this may be tedious for columns with many bars but a graphical solution appears suitable.

Weber (1966) on the basis of the uniaxial interaction curves for load and moments also suggests a method of approximating the failure surface based on linear interpolation between the capacity for bending on the diagonal and the capacity for bending on a principal axis. The objective being to provide an easily understandable, convenient method for the design of biaxially loaded columns that it similar in form to methods used for Uniaxial loading. The basic assumptions are

1) Plane sections remain plain, (which is one of the classical assumptions of the flexural or bending theory).
2) Compatible concrete and steel strains.
3) Maximum concrete strain at ultimate load equals 0.003.
4) Capacity reduction factor of 0.7

The sixteen charts produced assume symmetrical arrangement of reinforcements in square columns. Although various symmetrical arrangement of reinforcements are considered, the charts are not suitable in cases requiring unsymmetrical arrangement of reinforcements as well as for sections that are

rectangular. The charts are bound to have limited use. For reinforcements of between 1 and 2 percent, the charts are observed to be more linear than curvelinear and with negative gradients.

Furlong (1961) on the ultimate strength of square column under biaxially eccentric loads states that the design of reinforced concrete members to carry compression and biaxial bending is quite complex, any generalized procedure for designing such members will necessarily involve some estimating of cross sections followed by rather tedious checking of the estimates untill an optimum cross section is obtained. To reduce the labour required to predict accurately, the behaviour of columns with skew bending, some insight can be developed by studying the behaviour of typical column shapes. Similar shapes will exhibit similar characteristics which when correlated can save much time in design and can indicate such rules for guidance with specifications governing design.

Furlong (1961) studied columns with equal reinforcements at the all the faces and those with reinforcement at two faces alone*. Although the author is supposed to propose a method of estimating the

ultimate strength of square columns under biaxial loads,he makes the same classical assumptions of the bending theory which is always extended (wrongly or rightly) to column problems. Also assumed is the recommendation by ACI-ASCE Committee 327 that concrete may be depended on to resist crushing. Concrete was also assumed to have no tensile strength. Furlong (1961) made the following conclusions

1) The capacity of square reinforced columns to resist eccentric compression loads is a maximum when loads cause bending about the major axis of the cross section.
2) The capacity of a column to reist moments about any axis is virtually constant for eccentricities less than one-tenth of the thickness of the column.
3) The capacity for square columns with equal amounts of steel in each of four faces to resist eccentric compression loads is smallest when loads cause bending about an axis of 45 degree from a major axis.
4) Greater reductions in Nn and Nu from an eliptical arc may be necessary for rectangular shapes under skew bending.

319952
TA
660·C6
Ege

319952

2.13 Summary

Most recent researches on reinforced concrete columns were mainly for

a) The development of various national and international codes of practice and specifications.

b) The development of specifications for earthquake-resistant designs and better understanding of response of r.c. columns to dynamic loads.

c) The application of the finite element methods of analysis to columns.

Adequate knowledge the behaviour reinforced concrete columns under uniaxial load condition appear to be assumed. Researchers have been attracted to the precision of the use of finite element method of analysis on the assumptions of flexural systems imposed on columns.

More work appear to have been done on reinforced concrete beams and slabs than on columns. Perhaps because of the relative ease in modelling. All the classical assumptions and infact research results on slabs and beam theories have been super-imposed on

KASHIM ... 1992 ZARIA, NIGERIA

columns, thus eccentric loading is regarded purely as bending, in columns. Not much attention appear to have been given to unsymmetrically reinforced concrete columns since symmetrical systems simplify complex but accurate mathematical models. Since bending theories and its assumptions have been super-imposed on reinforced concrete columns, current designs and analysis have remained for pure axial load or bending with axial load. Adequate attention have not been given to loadings of small eccentricity which is typified by columns in high rise buildings. The only attempt so far made for this class of columns is the arbitrary reduction in material strengths, for designs only. All emphasis have been on design. The only justification for some assumptions being the continued safety of most structures. The mean ratio of theoritical failure loads and observed failure loads appears to be the most popular method of assessing experimental results among researchers.

Bresler (1960) proposed two criteria for the design of biaxially loaded columns based on approximations of failure surfaces. Bresler (1960) noted that exact mathematical expressions defining the

failure surfaces cannot be established, the two criteria for the design of columns under compression and biaxial bending proposed were observed to provide a *good approximation of analytical results when* compared to test results. Eight columns were crushed to failure.

Roberts (1983) describes simplified method of design of uniaxial and biaxially loaded concrete and composite columns. It was noted by Roberts (1983) that earlier researcher had compared the ultimate failure load of reinforced concrete columns predicted using cp110 with a porposed method and got an accuracy *of 66 percent mean value, and coefficient of variation* of 21 percent. However comparing the test results with the computer aided analysis on accuracy of 99 percent was reported (Robert et al 1983).

The proposed method is an extension of proposal for steel columns for which a test result of 16 columns gave an accuracy of 94.4 percent and coefficient of variation of 10.9 percent. It is noted that this researcher like many others use means and coefficient of variations of column capacities as a criteria for assesing experimental/analythical result.

Chen (1982) worked on Box and Cylindrical columns under biaxial loading. He noted that current codes do not consider these large welded built-up box members that are commonly used in off-shore structures and tall buildings. The finite element principle was adopted in the analysis of the columns. In the present analysis each element is treated independently and each contributes to the total sectional stiffness. The author did not consider interaction among elements.

Most of the current researches on the design of reinforced concrete columns are usually on the biaxial loading condition "Al Noury et al 1982". The axial load-moment interaction curves are extended in various forms, to biaxial solution. The assumption of course is that the uniaxial load design for reinforced

concrete columns has been well treated. Some of the mathematical transformation of the uniaxial curves to biaxial surface have little practical value. (Breysse et al 1988; Ramamurthy et al 1983; Weber 1966). Researches on Earthquake related designs.

The collapse of structures and the consequent loss

of human and material resources has necessiated vigorous researches into earthquake reistant designs. Current researches are either on the (shear capacity of reinforced concrete columns) effect of tie spacing on the ultimate failure load of such columns or on the dynamic response of reinforced concrete frames (Balendra et al 1984; Oliva et al 1987). In one study analytical and experimental studies were conducted on physical and analytical models of a 15-storey r.c. frame coupled wall structure which was identified as typical medium-rise office in the United States. The primary objective being to evaluate the state of the art practice in relation to proposals for seismic design code, and also to determine if the state of the art analysis provides reliable assessment of seismic response of concrete structures. It is believed by many researchers that the collapse of a building in an earthquake is insufficient ductility in the bearing

walls and columns (Ali 1987; Polyakov 1985; Sheikh et al 1980). This led to investigations into the effect of concrete confinement or more specifically, tie spacing, on the ultimate strength of reinforced concrete columns.

The ductility of a reinforced concrete is entirely a function of it's reinforcement. It was observed that inspite of the various works done on the effect of tie spacing on the strength of concrete columns, it has not been conclusive about this effect on the strength of columns. Various shapes and spacings were tried. It appears however that it is generally accepted that circular or spiral ties produce more effective confinement than rectangular ties (Ahmad et al 1982; Chiwuzie 1975). This is attributed to the fact that rectangular ties apply confining pressure near the corners only. Another reason which has not been commonly given which may affect the effectiveness of rectangular ties when compared to spiralo ties is the continuity of the later around the column. Various proposals were made for estimating the effect of tie spacing on the strength of concrete columns. These are given in Chapter 7. Virdi (1982) used the finite element method for the analysis of the varicus sections described. With the increased use of the computers and micro computers by engineers and scientists, the finite element method of analysis has gained a lot of ground in reinforced concrete

researches around the world (kayal 1984; Olatunji 1985). Kayal (1984) presents for instance, a method of analysis of braced and unbraced columns based on the finite element principles. This method considers nonlinear behaviour of reinforced concrete columns arising out of material nonlinearity and large displacements. The method uses a single beam-column element with five degrees of freedom, two end rotations, two end deflections and one deflection at mid length position. Analytical integration leading to closed form solution is used to evaluate the contribution of concrete area to the axial load and moment acting on the cross section. The iterative procedure for solutions of non-linear equations used is based on modified Newton-Raphson technique. In another effort to simplify the problem of biuaxial loading, Hsu (1973) introduced an unusual column shape for his experiments. The analysis of the specimens was by the finite element principle, using the rectangular element and the extended Newton-Raphson technique. Some of the assumptions are the classical assumptions of the bending theory, creep and shrinkage have no effects on the stresses sof the descrete

elements and buckling does not take place before attainment of the ultimate load of the column. In all the methods, the various suggested stress-strain relation of concrete as proposed by Basu (1984) and Cranston were used and produced fair accuracies. Virdi (1982) crushed nine composite columns under biaxial load condition. The test results were analysed using the finite element principlels. He found the results to be close to observed strengths. The maximum compressive stress of concrete at failure was taken as 0.64 times the cube strength. The mean of the theoritical strength/observed strength ratio was 104 percent and a standard deviation of 10 percent.

In another research, the column is treated as a space structure. This is because general solution of a biaxially loaded column problem must be approached from the stand point of a three dimensional situation. The colomn is sgmented and each segmented is divided into descrete elements of steel and concrete in order to calculate it's tangent stiffness properties at different levels of strain. The segment stiffness relationship is computed by solving the governing differential equations about the principal axes of the

cross section. Direct stiffness method is used to construct the whole column stiffness matrix. Material and geometrical non-linearities are handled by iterative procedures based on the defined tangent stiffness approach. Comparisons of theoritical loads and observed failure loads of elevlen columns gave a good corelation (Al-noury et al 1982).

2.14 The Limitations of Current Design Methods

2.14.1 General

The major elements of a structure are the slabs, beams and columns. Each element is designed with respect to its deformation method (or failure mode). Although the failure mode of these reimfprced concrete elements may be complex, it may be approximated to a simple form or idealization (Astill 1975; Bennett 1973). While slabs are best designed as plates, beams are designed as purely flexural prismatic sections. Columns carry mainly gravity loads and it's buckling - prospensity. This is the reason for classifying columns into short columns and slender columns. Columns are supposed to be primarily compression members, but when eccentrically loaded, are currently designed either as a purely presmatic flexural system

with unbalanced axial load condition, or as axially loaded system with arbitrary reduction of the assumed material strengths when the eccentricity of loading is assumed to be small. This chapter reviews the flows of current design methods (and charts) of reinforced concrete columns in respect of columns under small eccentricity of loaoding, which typify the class of columns of high rise structures. (Fintel 1974).

2.14.2 Optimum Use of Concrete

In the dsign of reinforced concrete sections, because concrete can easily be made into various shapes, the optimal utilization of concrete is easily achieved. The most important property of concrete is its high compressive strength. Because concrete is poor in tension the primary objective of reinforcement is to resist within a concrete element, all tensile forces. Secondly because of the high relative modular ratio of steel, it serves to reduce the size of concrete elements . In the current design methods for eccentrically load columns, classical assumptions made include

a) The strains are dependent on the distance of point of consideration from the neutral axis.

b) There is perfect bond between the concrete and reinforcements, infering that there inforcement strains are equal to the concrete strains at the same point.

c) Concrete is weak in tension and the tensile strength is zero.

d) The reinforcement response to load is elastoplatic.

The assumption (c) above is not compatible to the appearance of cracks (on a concrete column) as signifying the on-set of failure. (Chen 1982). While this assumption may be suitable for the design of slabs and beams, it may be questionable for columns. In addition, in beams and slabs, optimum utilization of the material requires the provision of voids within the tensile zone. Similar provision in a column will obviously imp[air drastically, the lood capacity of such column. The contribution of concrete to load resistiance as an interaction of axial load and bending, has been indicated to be parabolic. The CEB (1982) suggests a maximum value of about 0.8 while Oleszkiewicz (1985) modifies this value to one. This implies that it is possible to desiign eccentrically loaded columns with optimum concrete contribution of one. In a new design concept, since the application

of void in the tensile zone will reduce the load capacity of a column, it is possible using high strength concrete to assume a concrete contribution of one (Oleskiewicz 1985; Virdi et al 1982).

2.14.3 The Balanced Load Concept

The balanced load condition of a column is the point at which the column fails by both crushing of concrete and yielding of reinforcements. This point is used (as a measure) to determine the failure mode of a column. When the eccentricity is more than that at which balanced failure occurs, the column is said to fail by yielding of reinforcements. If on the other hand the eccentricity is smaller, failure is said to be by crushing of concrete. From all recorded load tests on concrete columns, failure has always been either by crushing (or shearing) of concrete alone indicating low utilization of reinforocements or by buckling of reinforocements accompanied by concrete crushing. Thus in the design of concrete columns it may be more relevant to relate the eccentricity of loading with respect to the "balance point", with utilization and non-utilization of reinforcements. It should also be noted that from current assumptions about the "balanced point", the

depth of compression zone and hence the force load-resistance contribution of concrete remains the same irrespective of the amount of reinforcement unsymmetry (Park et al 1975; Wan 1988).

• 2.14.4 The Concept of Small Eccentricity

Available CP110 design charts apply the same principles to columns of eccentricities as small as two percent of the concrete depth, and those of two hundred percent. There is no "middle of the road" solution for columns under small eccentricities. The question of which eccentricity should universally be regarded as small by all designers is a problem. Various eccentricities, from five to twenty-two percent of depth of section has been suggested as small. Since by current methods, symmetrical arrangement of reinforocements by charts or unsymmetrical arrangement using the rigorous iterative method depend on the level of eccentricity, it is usually not clear to most designers when to reinforce symmetrically or otherwise. This approximation increases the cost of reinforcements, and more when columns are biaxcally loaded. The arbitrary ten percent reduction in material design strength when the eccentricity is perceived to be small by the designer has no direct mathematical

relation. Thus for a varied number of eccentricities, a given loaod and column concrete section, the same amount of reinforcement is provided. This lacks precision especially when high strength concrete is used. It does not also lend itself to capcity estimation as may be required in cases of arbitration. This underscores the need to study the concept of small eccentricity in relation to the design of class of columns which cannot adequately be described as axially loaded nor as flexural systems with finite depth of neutral axis. Avram et al 1981; BS1 1972; Brondium 1987; Lafe 1986.

2.14.5 Reinforcement Requirement

In the current design method when moment is said to dominate, the section is designed by the rigorous iterative procedure resulting in unsymmetrically reinforced section. Otherwise the dsign charts are used. There is considerable difference in the required reinforcements when both methods are usued for columns of small eccentricity. Since the designer decides if moment dominates or not the reinforocement requirement is left to approximation. One important question which current design methods have not answered is whether a

column with symmetrical reinforcement arrangement of say As = 5R12 and As = 2R12. This implies that for short columns under small eccentricity, unsymmetrical arrangement of reinforcements may infact achieve both reduced reinforcement requirement and increased reliability.

2.14.6 Flexibility of Current Design Charts

Available design charts for reinforced concrete columns assume both reinforcement arrangement and design parameters such as concrete grade, steel grade, concrete cover etc. These do not allow the designer much flexibility. The designer often approximates the situation to the detriment of reinforcement requirement and concrete utilization.

2.14.7 Compatibility of Strains at Failure

The strains in reinforcements and concrete at a given point are assumed equal at failure. The concrete attains it's peak stress at a strain value below the ultimate and relinquishes some of it's load to the reinforcements. This sudden increase in the load carried by the reinforcement may affect the bond between concrete and steel. In addition with the concept of failure of concrete in compression as the

ultimate of increasing microcracks, and because the micro-cracks develope during loading due to the relative stiffness between concrete and reinforcements at their interface, the assumption of perfect bond between steel and concrete at failure becomnes questionable (Chen 1982; Nandivaram 1988; Robert 1983). Design model which does not make such assumption may approach the ideal.

2.14.3 Strain Distributiuon and Depth of Neutral Axis

The middle third rule of development or not, of tension on the face of an eccentrically loaded concrete column is based on superposition of pure axial load condition and pure bending condition. By this method, there is considerable difference in the depth of neutral axis for the pure bending condition and the combined loading condition achieved from superposition. This difference in the depth of neutral axis is not cartered for in the load estimation of columns under small eccentricity of loading (Moseley 1976; Morrell 1977; Oladapo 1981).

CHAPTER 3

TEST SET-UP AND RESEARCH METHODOLOGY

3.1 Specimen Description

Table 3.1: Presents The Parameters of The Test Specimens as shown in Fig. 3.5.

Table 3.1: Test Specimen Parameters

Specimen/ Specimen Group	b	h	l.	l/h	fcu	A^1s	A_s	d^1	d_2
Group A	150	150	750	5	53	177	177	30	30
" B - 1	150	150	750	5	42	203	203	25	25
" B - 2	150	150	750	5	43	312	203	25	25
" C	100	100	1400	14	39	265	177	25	25
" D	100	100	310	3	56	265	177	25	25
" E	100	100	300	3	57	265	177	25	25
Specimen G1-1	100	100	300	3	46	3320	190	20	20
" G12	100	100	300	3	46	260	157	20	20
" G13	100	100	300	3	46	197	106	20	20
" G14	100	100	300	3	46	167	106	20	20
" G21	100	100	300	3	54	232	190	20	20
" G22	100	100	300	3	54	260	157	20	20
" G23	100	100	300	3	54	197	106	20	20
" G24	100	100	300	3	54	107	106	20	20
" G31	100	100	300	3	57	232	190	20	20
" G32	100	100	300	3	57	260	157	20	20
" G33	100	100	300	3	57	197	106	20	20
" G34	100	100	300	3	57	167	106	20	20
Group H1	46	103	760	10	27	140	96	10	24
" H2	76	108	760	10	31	143	96	10	24
" H3	76	108	1170	15	27	143	96	10	24
" H4	76	108	1170	15	31	143	96	10	24
" H5	100	100	300	3	57	-	-	14	14

The last column in table 3.1, shows the number of specimen in the group Table 4.1.2 shows the parameters of the new type of composite column specimens tested, as illurstrated in fig 13.1.

Table 3.2: Composite Column Parameters

	Column Section	Size of Angles	L	Space Between Angles	l/b	Area of Steel	Area of Concrete
CC1	100X100	30X30	750	40 mm	7.50	720mm	9280mm^2
CC2	120x120	"	750	60 mm	6.25	720mm	13680 "
CC3	140X140	"	750	80 "	5.36	720"	18880 "
CC4	160X160	"	750	100 "	4.69	720"	24880 "
CC5	180X180	"	750	120 "	4.17	720"	31680 "
CC6	200X200	"	750	140 "	3.75	720"	39280 "
CC7	120X120	40X40	750	20 "	6.25	1198	13202 "
CC8	140X140	"	750	20 "	5.36	1198	18402 "
CC9	180X180	"	750	20 "	4.17	1198	31202 "

There were a total of 72 reinforced concrete columns and nine composite columns*. All the columns were cast horizontally, in either steel moulds or plywood moulds, each time attempting to achieve the depths of reinformcements as accurate as possible. Apart from group G specimens where different reinforments were provided, each specimen group had the same reinforcements arrangement.

---

*These are referred to as new type of composite columns by earlier researches, Taylor et al (1983).

3.1.2 Materials

Table 3.2. shows the yields and the actual diameters of the reinforcement bars as against the nominal diameters quoted in table 3.1. All the steel are mild steel. The angle iron for the composite columns were not tested for yeild strength but were assumed to have strengths of 250n/mm^2.

Table 3.3: Reinforcement Bar Parameters

Specimen	Nominal Diameter	Actual Diameter	Yeild Strength
G	8 mm	7.80mm	286n/mm^2
	10 mm	10.60mm	250n/mm^2
	11 mm	11.00mm	250n/mm^2
	6	5.80mm	290n/mm^2
	12	11.5mm	250n/mm^2
	16 mm	15.6mm	250n/mm^2

Concrete for all the columns was made from crushed aggretate of 10mm maximum size. The sand is well within zone 3. Vibration table was used for the compaction of concrete. The specimens were removed from the moulds a day after casting. For the composite columns, the steel angles used were of two types; 30mm x 30mm equal angles and 39.4mm x 39mm x 3.8mm equal angles for the encased composite columns.

The angles were held together as illustrated in fig. 12.1, with 6mm mild steel.

3.2 Instrumentation

The instrumentation and test procedure for the specimens were relatively simple. For the specimens with height-breadth ratio of 3, the Elect 2000 compression machine served as the loading device, while the universal testing machine was the loading device for specimens with height-breadth ratios more than 3. Digital strain meter and the analogue meter were both used at different stages in monitoring the strains in the reinforcements at various stages of the research. The strain meter work on the principle of Wheatston bridge while the guages form the resistors.

The strain guages used were the Tokyo Sokki electrical resistance strair gauges of various gauge lengths between 5mm and 20mm. Fig 3.5 and Plate 2 show the test set up and digital strain meter respectively. Three things had to be ensured:

(a) Perfectly flat ends. This was provided for in the machines, by having self-adjusting bases as load is applied.

(b) Uniform load distibution. To avoid load

concentration and enhance uniform load distribution over the concrete cross section, cardboard paper was placed at both ends of the column specimen, in addition, steel plate 12mm thickness was placed on the cardboard before a roller which came directly in contact with the loading pattern.

(c) Accuracy of the eccentricities. Due to the low values of eccentricities used in the tests, the accuracy of the point of loading was paramount and carefully modelled. Round bar 12mm diameter was used to transfer the load from the machine to the specimen. The bar which was as long as the breadth of the column section was marked with vertical lines at each end to coincide with the required line of action of the load, on the steel plate and specimens. Thus, to achieve a given value of eccentricity, the line of action of the load is clearly marked on both the specimen and plate respectively, to be away from the centroid of the specimen by the value of eccentricity required. For specimens with zero eccentricity, the steel roller was removed, thus transfering the

load directly through the steel plate and cardboard to the specimen.

3.3 Test Procedure

All the columns were tested by monotonic increasing static loading, to failure. Specimen groups G1, G2, H2, H4 and H5 were loaded at the rate for concrete cube tests. All other specimens were loaded at very slow rates to enable the reading of the strains to be taken at various load levels. However all were intended to represent short-term concrete tests. The failure of the specimen was taken as the maximum load recorded by the specimen. At load levels of about eighty percent of the failure load, and above, the strain values become unsteady and unreliable. The gain control of the digital strain meter was kept at high value of eighty - percent.

Although the meter has facility for direct recording, bridge-balancing was adopted.

CHAPTER 4

PROPOSED UNIAXIAL DESIGN METHOD

4.1 Assumptions

A number of assumptions are made both in the formulation of the proposed design charts and analytical capacity estimation methods. These assumptions are:

(1) Columns are primarily under compressive load. Since concrete is good in compression, and for small eccentricity in short columns, the moments do not produce reactions within the section that are compatible with the classical theories of bending with respect to strain distibution. This implies that the strain distibution in short columns is not similar to the current strain distribution assumption (Nilson 1986; Taylor 1984; Wan 1988).

(2) The contribution of concrete to load resistance is parabolic as stated by the CEB and modified by Oleszkiewicz (1985). The optimal reinforcement requirement of a concrete section is as illustrated by fig 2.0. The minimum dimensionless concrete contribution to load resistance is unity and can be achieved in short columns.

Fig. 3.1

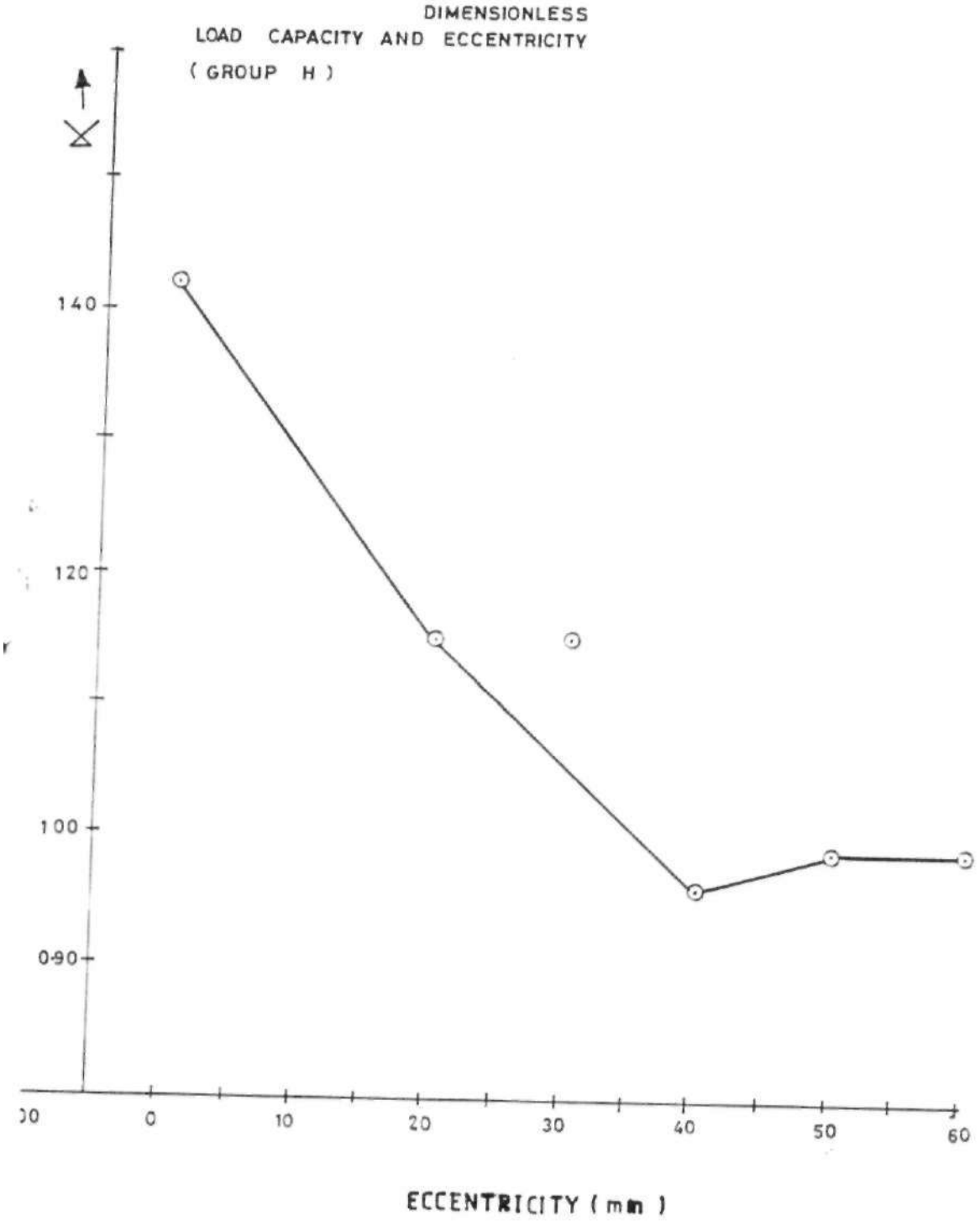

DIMENSIONLESS
LOAD CAPACITY AND ECCENTRICITY
(GROUP H)
140
120
100
0·90
)0
0
10
20
30
40
50
60
ECCENTRICITY (mm)

Fig. 3.2

DIMENSIONLESS MOMENT
RESPONCE OF REINFORCEMENTS

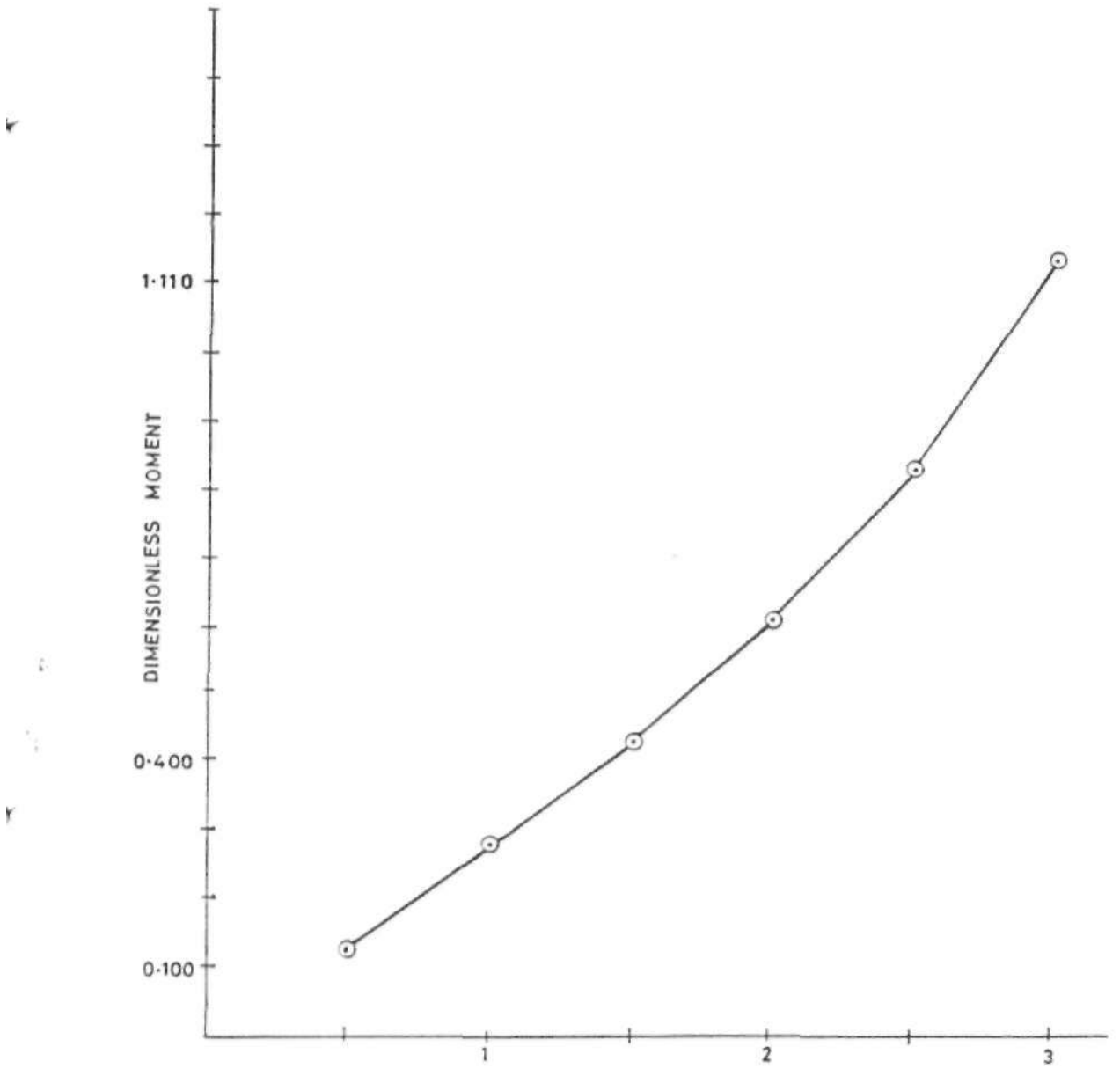


Fig. 3·3

VARIATION OF DIMENSIONLESS LOAD CAPACITY WITH ECCENTRICITY (ALL SPECIMENS).

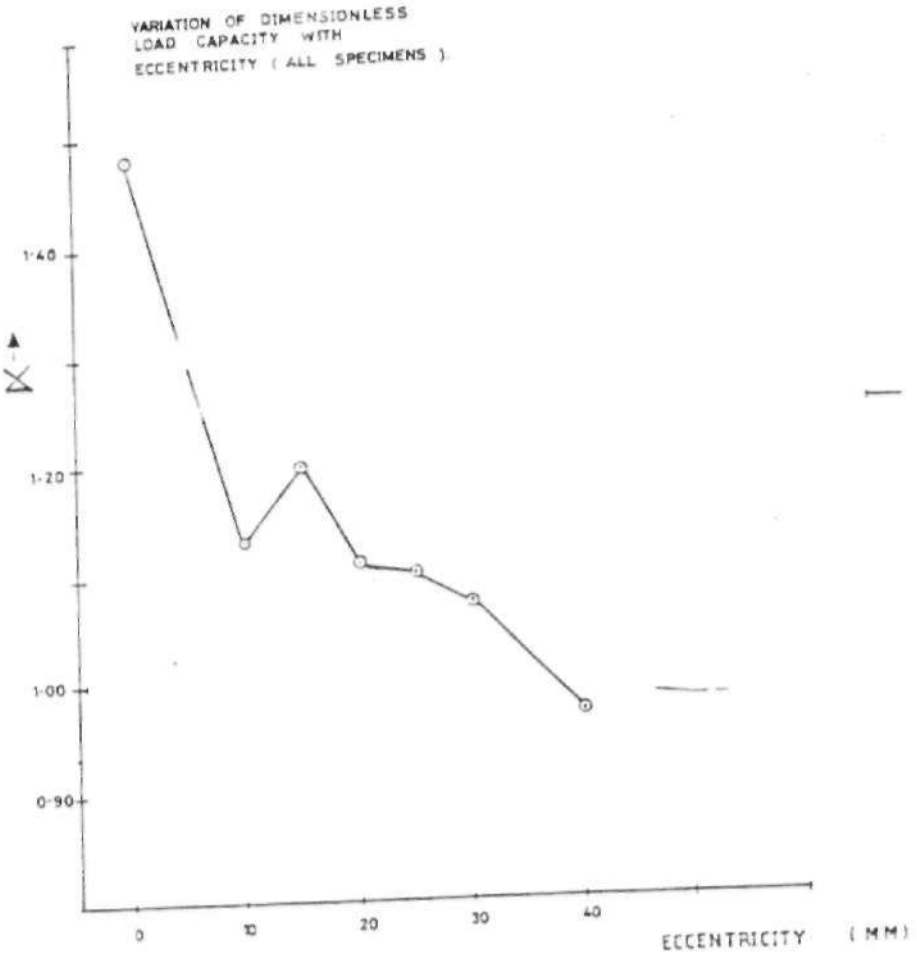
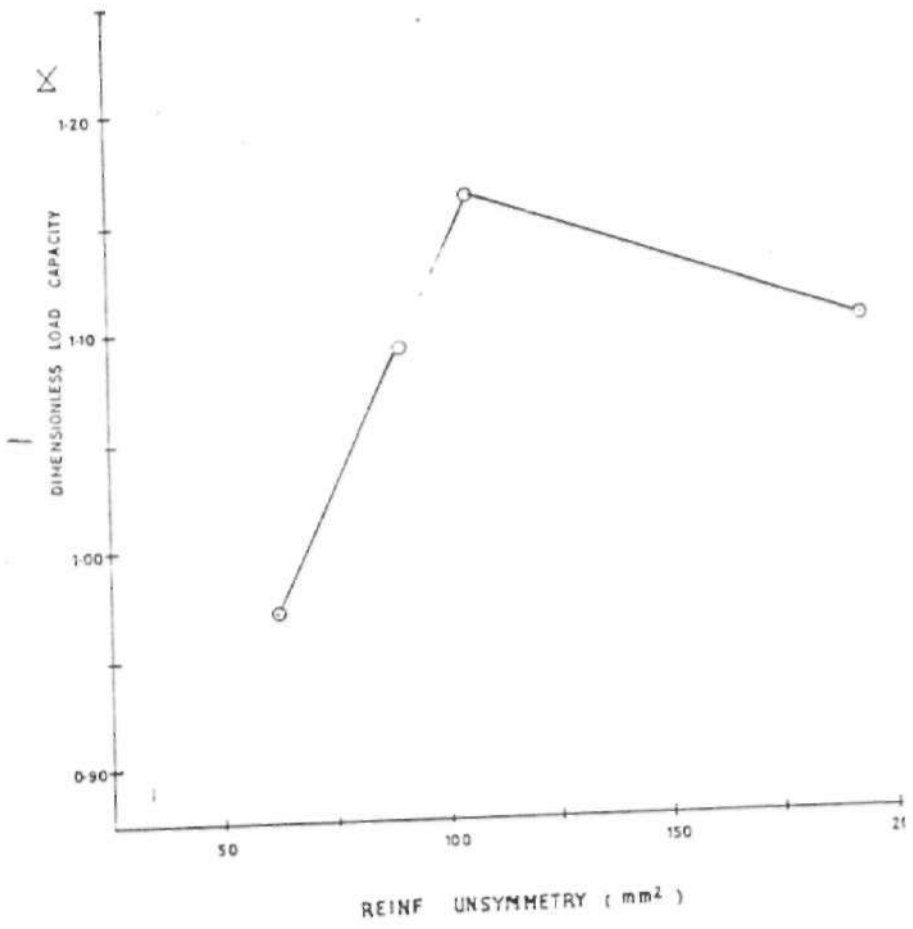
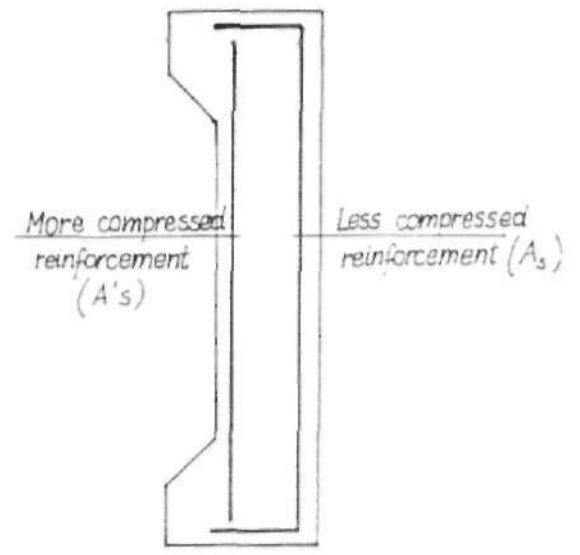


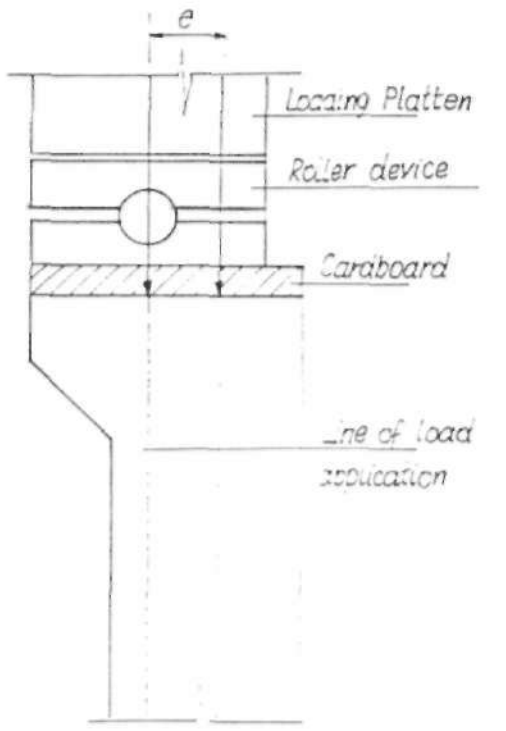
Fig 3.4

DIDMENSIONLESS LOAD CAPACITY
AND REINFORCEMENT UNSYMMETRY
GROUP G.

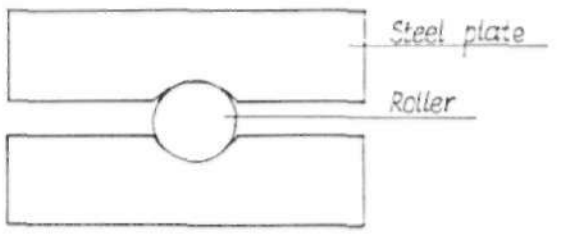




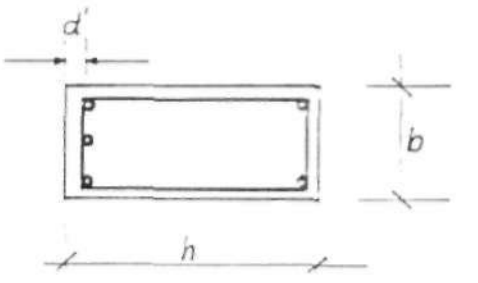
A COLUMN SPECIMEN



B LOAD APPLICATION AT ECCENTRICITY e



C. ROLLER DEVICE



D. COLUMN CROSS-SECTION

FIG. 3.5

SKETCH OF THE SPECIMEN AND LOAD APPLICATION

(3) Only axially loaded short columns can be reinforced symmetrically.

(4) Although the ratio of the stresses in concrete and steel is known to be a variable due to shrinkage and creep (Mindess et al 1981; Debernardi et al 1989), for any given steel and concrete strengths, this ratio at failure is a constant.

(5) The additional deformations due to load in a short column is zero. This implies that buckling does not occur before the crushing of the materials (Bernitsas et al 1983).

(6) The bending moment is applied about the principal axes of the cross section. The minimum moment capacity of the concrete column is equal to the moment capacity of the reinforcements, and $d^1 = d_2$.

(7) There is a relation between reinforcement unsymmetry and the eccentricity of loading.

(8) As in (1) above, the moment-thrust-curvature relation as are currently being used, may be suitable only for slender columns where deformations are considerable.

(9) For slender columns, the additional eccentricity due to the interaction of load and the slenderness

of the section may be estimated using suggestions by CP110 (1972).

(10) The cover of concrete to reinforcement affect the moment capacity of the short columns.

(11) The longitudinal stress of an element is dependent only on the longitudinal strains. The effect of shrinkage and creep are neglected.

(12) All design and analytical loads are short-term loads and no consideration is given to fatique.

(13) Static equillibrium is more relevant than compatibility of strains at failure.

4.2 Formulation of Design Chart

Concrete columns are effective in resisting axial compression, this efficiency reduces as the height-breadth ratio increases. However for height-breadth ratios of 12 and less, it has been found that this efficiency remains fairly constant. (Oleszkiewicz 1985; Vallenas et al 1977). With the introduction of eccentricity there is a complex interaction of axial load and flexure. The proposal presented are based on simplification of this interaction.

Consider a concrete section with axial load and moment and assuming the unified design approach of

Abatan (1986) and Oleshiewicz (1985).

$$N = fc\ Ac + A'_s\ f'_s + A_s f_s \quad \ldots\ldots\ldots\ldots\ldots\ 4.2.1$$

$$N = f_c bx + A'_s\ f'_s + A_s\ f_s \quad \ldots\ldots\ldots\ldots\ldots\ 4.2.1a$$

dividing equation 4.2.1a by fcbh results in

$$\frac{N}{dcbh} = \frac{x}{h} + \frac{(As)}{(bh)}\frac{(fs)}{(fc)} + \frac{(As)}{(bh)}\frac{(fs)}{(fc)} \quad \ldots\ldots\ldots\ 4.2.2$$

If N/fcbh is written as and the contribution of concrete and steel written in like manner, equation 4.2.2 becomes

$$= \alpha_c + \alpha_s + \alpha'_s \quad \ldots\ldots\ldots\ldots\ 4.2.3.$$

Note that α_s and α'_s are products of reinforcement ratio and stress ratio. The assumption at this stage is that all the reinforcements have compressive stress. If the stress in As is tensile, 4.2.3 would have been minus α_s.

Taking moments at the centroid of section and reducing the equation to dimensionless form as in chap. 2.5 by dividing through with $fcbh^2$ the moment equations below result

$$M = f_c b\ X\ (h/_2 - \tfrac{X}{2}) + A'_s\ f'_s\ (h/_2 - d') - A_s f_s\ (h/_2 - d_2)$$

$$\ldots\ldots\ldots\ldots\ldots\ 4.2.4$$

and

$$\beta = \beta_c + \beta'_s - \beta_s \quad \ldots\ldots\ldots \quad 4.2.5$$

where

$$\left.\begin{aligned} \beta_c &= \alpha_c\,(0.5 - \alpha_{c/2}) \\ \beta_s &= \alpha'_s\,(0.5 - d^1/h) \\ \beta_s &= \alpha_s\,(0.5 - d_2/h) \end{aligned}\right\} \quad \ldots\ldots\ldots \quad 4.2.6$$

From assumptions above, when maximum concrete contribtion is attained, $\beta_c = 0.$ and $\alpha_c = 1$ therefore equation 4.2.3 becomes $\alpha = 1 + \alpha_s - \alpha_s$ 4.2.7

let the axial force to be resisted by the reinforcements after concrete has tried enough be $\hat{\alpha} = \alpha - 1$ therefore equation 4.2.7 may be written as $\alpha_s = \hat{\alpha} - \alpha'_s$ 4.2.8

substituting equation 4.2.8 in equation 4.2.5 gives

$$\beta = \alpha'_s\left(0.5 - \frac{d}{h}\right) - \left(\hat{\alpha} - \alpha'_s\right)\left(0.5 - \frac{d}{h}\right) \quad \ldots\ldots\ldots \quad 4.2.9$$

dividing through by $(0.5-d^1/h)$ gives

$$\hat{\beta} = \frac{\beta}{\dfrac{(0.5-d^1)}{h}} = 2\,\alpha'_s - \hat{\alpha} \quad \ldots\ldots\ldots\ldots \quad 4.2.10$$

rearranging equation 4.2.10 gives

$\hat{\beta} = -\hat{\alpha} + 2\,\alpha'_s$ OR

$\hat{\alpha} = -\hat{\beta} + 2\,\alpha'_s$ 4.2.11

For a given reinforcement percentage (ratio) and

failure stress ratio α^{1}s becomes a constant in equation 4.2.11. Plotting the graph of $\hat{\alpha}$ against $\hat{\beta}$ will produce a straight line with negative gradient and

intercept of 2 α s. Note that the difference between β and $\hat{\beta}$ is the effect of cover on the moment capacity of the section. Equation 4.2.7 may be written as

$$\alpha^{1}s = \hat{\alpha} - \alpha_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.2.12$$

substituting equation 4.2.12 into equation 4.2.5 gives

$$\beta = (\hat{\alpha} - \alpha_s)(0.5 - \frac{d^1}{h}) - \alpha_s(0.5 - \frac{d_2}{h})$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.2.13$$

dividing through by $(0.5 - d^1/h)$ gives

$$\hat{\beta} = \frac{\beta}{(0.5 - \frac{d^1}{h})} = \hat{\alpha} - 2\alpha_s \ldots\ldots\ldots \quad 4.2.14$$

rearranging equation 4.2.14 gives

$$\alpha = \hat{\beta} - 2\alpha_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.2.15$$

For a given reinforcement percentage and stress ratio, α_s becomes a constant. Thus plotting a graph of $\hat{\alpha}$ against $\hat{\beta}$ will produce straight lines with positive slope. Combining the two straight lines of positive and negative slopes produce a design chart for short reinforced concrete columns as shown in fig

4.1. For practical production of chart, the units for graph are , the stress ratio. Thus and B are converted to stress ratios. Any point on the chart shows the reinforcement condition and may be determined by linear interpolation.

4.3. Method of Design for Uniaxial Loading

The design method assumes the ultimate limit state therefore all relevant safety factors and determination of the loading is similar to current methods.

For a set of design data, the proposed uniaxial loading design are explained in the following steps.

(1) Determine the value of where = - 1 and = N/(fcbh)

(2) If is less than one, reduce the cross sectional area of the concrete column.

(3) Determine B where $B = B/0.5-d^1/h$ and $B = M/bh^1fc$. Note that fc = (0.67fcu)(S.F). where S.F is the partial factor of safety applied to the concrete material usually taken as 1.5.

(4) Determine which is the ratio between steel failure stress and concrete failure stress.

(5) Convert and B to - values.

(6) Locate the reinforcement position on the chart.

(7) Arrange the reinforcements as indicated on the two

sides of the column section.

Example. To design a column with the following data.

- Design axial load N = 2460KN
- Design moment M = 91.2KNM.
- Column size = 200mm x 450mm

- Depth of more compressed reinforcement for compression face d^1 = 45mm.
- Reinforcement yeild stress fy = 425 n/mm^2
- Concrete grade = 50

Solution

Assume fs = 0.72x425 = $306n/mm^2$

fc = 0.45x50 = $22.5n/mm^2$

$$= \frac{2460x10^3}{22.5x450x200} - 1 = 0.2148$$

$$B = \frac{(91.2x10^6)}{(22.5x200x450^2)} \cdot \left(0.5 - \frac{45}{425}\right) = 0.2538$$

$$= 306/22.5 = 13.6$$

$$\therefore = 0.2148/13.6 \text{ - values} = 0.0158 \text{ -values}$$

$$\therefore B = \frac{0.2538}{13.6} = 0.0187 \text{ - values.}$$

from the Chart A^1s = 1.775% ($1598mm^2$)

As = 0.15% ($135mm^2$)

1.925% - $1733mm^2$

It will be observed that if the cover of the above design is reduced by 15mm, the reinforcement requrement will be reduced by about ten percent. The same cover had to be used here in order to relate this method to that by CP110. Refer to Chapter 4.4.

4.4 Comparison of Proposed Method With Current Design Methods

Two typical design problems will be done with the various methods of designs. The only parameter for comparison here will be the total reinforcement requirement.

Problem 1 To design a column given the following data (see chap 4.3).

N = 2640KN, M = 91.2KNM, d^{1}= 45

b = 200, h = 450, fy = 425, fcu = 50

A CP110 Design Chart

$$M/bh_2 = \frac{91.2 \times 10^6}{200 \times 4502} = 2.25$$

$$\frac{N}{bh} = \frac{2640 \times 10_3}{200 \times 450} = 27.4$$

d_1/h = 0.1, from CP110/2/88

As = $1550mm_2$

As = $1550mm_2$

Total = $3100mm_2$

B Reigorous Iterative Method.*

For fy = 425 E1 = 0.00148 and E2 = 0.00385

1 = 0.389x50; U2 = 0.438

- for x = 400 As = 1859, As = 57652; total = $59511mm_2$

- for x = 350 As = 1992 As = 5973; total = $7965mm_2$

- for x = 450 As = 1803 As = 10904; total = $5468mm_2$

C Clement's Method (1977).

Concrete force = $0.45x50x200x450x10-_3$ = 2025KN

∴ force in either arm to form a couple to resist moment of 91.2KNM

$$F = \frac{91.2x10_6}{360} = 253333N = 253.3KN$$

∴ Force in compression = $\frac{615}{2} + 253.3 = 560.8KN$

∴ $A_1s = As = \frac{560.8x10_{3n}}{312\ n/mm^2} = 1797.44mm^2$

D American (ACI) Method (Park et al 1975).

For a capacity reduction factor of 0.70 and reinforcements equally distributed at four faces.

$$\frac{N}{fcbh} = \frac{2460\times10^3}{0.85\times50\times200\times450} = 0.64314 \quad 0.64$$

$$\frac{M}{fcbh^2} = \frac{91.2\times106}{0.85\times50\times200\times450} = 0.0529847 \quad 0.0053$$

from design chart assuming reinforcements at four faces, Area of reinforcement = $2058mm^2$

E Oleszkiewicz Method (1985).

Since $e = M/N = \frac{91.2\times106}{2640\times10^3} = 34.55mm$ $\frac{h}{2} - d^1$

$2640\times10^3 = 0.4\times50\times200(450-69.10) + 1.14\times425As$

$\therefore As = 1116.4\times10^3/569.5 = 1960mm^2$

$\therefore As \quad 1960 - \frac{(50)}{(425)}(200 \times 34.55) = 1147mm^2$

also

$As \quad 1960 - 2(0.67\times50\times200(34.55)/312) = 476mm^2$

$\therefore As = 1960; \quad As = 1147; \text{ total } = 3107mm^2$

The Summary of the reinforcement Requirements of the same design with respect to the various methods compared, is presented in table 4.2.

Table 4.2: Summary of Reinforcement Requirements Using

Various Methods for the Same Problem

S/No	Method	A's	As	Total
1	Proposed Method	1598mm^2	135mm^2	1733 mm^2
2	CP110 Design Chart	1550mm^2	1550 "	3100 "
3	Rigorous method	2706	2762	5468 "
4	Clement's method	1797	1797	3594 "*
5	ACI method	1029	1029	2058 "
6	Oleszkiewicz Method	1960	1147	3107 "
7	Approximate method	1159	1159	2318 "

It will be observed that by clement's method, the actual force on the less compressed reinforcement is 307.5 - 253.3 = 54.2KN. If the bars are stressed to a maximum stress of 312n/mm^2 the area of reinforcement required at the less compressed face ill be 174mm^2 resulting in total reinforcement requirement of 1971mm^2, which will be close to the proposed method. Also the ACI method would have resulted in design load was split into imposed and dead loads as the partial factor of safety on imposed load by that method is higher than ultimate limit state method. By the approximate method, neglecting the eccentricity of loading and reducing the strength of the materials,

$N = 0.4fcubh + 0.67fyAsc$ which gives $Asc = 2318mm^2$.

Table 4.1: Average Strains at Different Loadings

	50KN	100	150	300	450KN	700KN
A 0	25	78	122	360	412	1033
A - 25	118	206	315	602	978	1460
B	117	189	283	585	-	-

Plotting the graph of load against the recorded strains for A.0. the slope is approx. $0.97x10^6$ plotting same for A 25 shows that the slope is (0.466). Lower. Since youngs modulus of the reinforcement remains constant, this suggests that the amount of load carried by the reinforcements depends on the eccentricity. The lower slope suggest that the reinforcements carry more (additional) load.

4.5 Advantages and Limitations of Proposed Method

4.5.1 Advantages

The major advantage of the proposed method of design is reduction in the reinforcement requirement and the size of the concrete by making optimum use of concrete in compression. In addition, the method is simple, thus cheaper for a designer.

4.5.2 Limitation

The limitations of the method are

(1) The method has the tendency to crowd the reinforcements on one side of the column.

(2) It also encourages the use of small cover to reinforcement.

4.6. Summary

The chapter presents the assumption of the model design and load capacity estimation methods, the formulation of the design chart, method of design using the chart and a comparison of various methods of design using the same problem. The comparison shows that the proposed design method is cheaper and simpler than other design methods.

CHAPTER 5

Theoretical And Experimental

Load Capacities Of Short Columns

5.1 Proposed Uniaxial Load Estimation Method

In consonance with the assumptions stated in Chap. 5.1, the static equiillibrium of forces is more relevant than the strain compatibility or perfect bond critieria. Also with unit concrete contribution α_c, which is the flexural capacity of concrete becomes zero. Therefore equation 5. becomes $\alpha = \alpha's + \alpha_s$..... 5.1.1.

Taking moments about the less compressed steel gives the equation

$$\alpha\left(\frac{e}{h}+0.5-\frac{d'}{h}\right)=\alpha c\left(0-d'/h\right)+\alpha_s\left(1-\frac{d'}{h}-\frac{d_2}{h}\right)$$

For maximum concrete contribution, $\alpha c = 1$

equation 5.1.2 becomes

$$\alpha\left(\frac{e}{h}+0.5\ d/h\right)=(0.5-d/h)+\alpha's\left(1-d/h-d_2/h\right)\ldots\ldots\ 5.1.3$$

For given eccentricity of loading the dimensionless quantity of the failure load. Thus the failure load becomes

$$N = fcbh \quad \text{.....5.1.4}$$

Load capacity estimation of short concrete columns by Oleshiewicz is reviewed in chapter 2.5

5.2 Oleszkiewicz And Proposed Method

Both methods are compared using the set of parameters for concrete columns, of group H. fig 5.3 presents the comparison. It will be observed that Oleszkiewicz method varies linearly while the proposed method is parabolic. Both methods are about the same at eccentricities of 0 and 50mm with maximum difference between e= 20 and 30mm. Oleszkiewicz method is not affected by the cover to reinforcement while this was observed to affect the strength of the columns.

5.3 Failure Load And Reinforcement Unsymmetry

From the assumptions of the proposed design method, the minimum moment capacity of a column is fixed for a given amount of reinforcement unsymmetry. The specimens of group G were tested at the same eccentricity of 10mm for various reinforcement area unsymmetry. The specimens were three sub groups of concrete grade 46,54

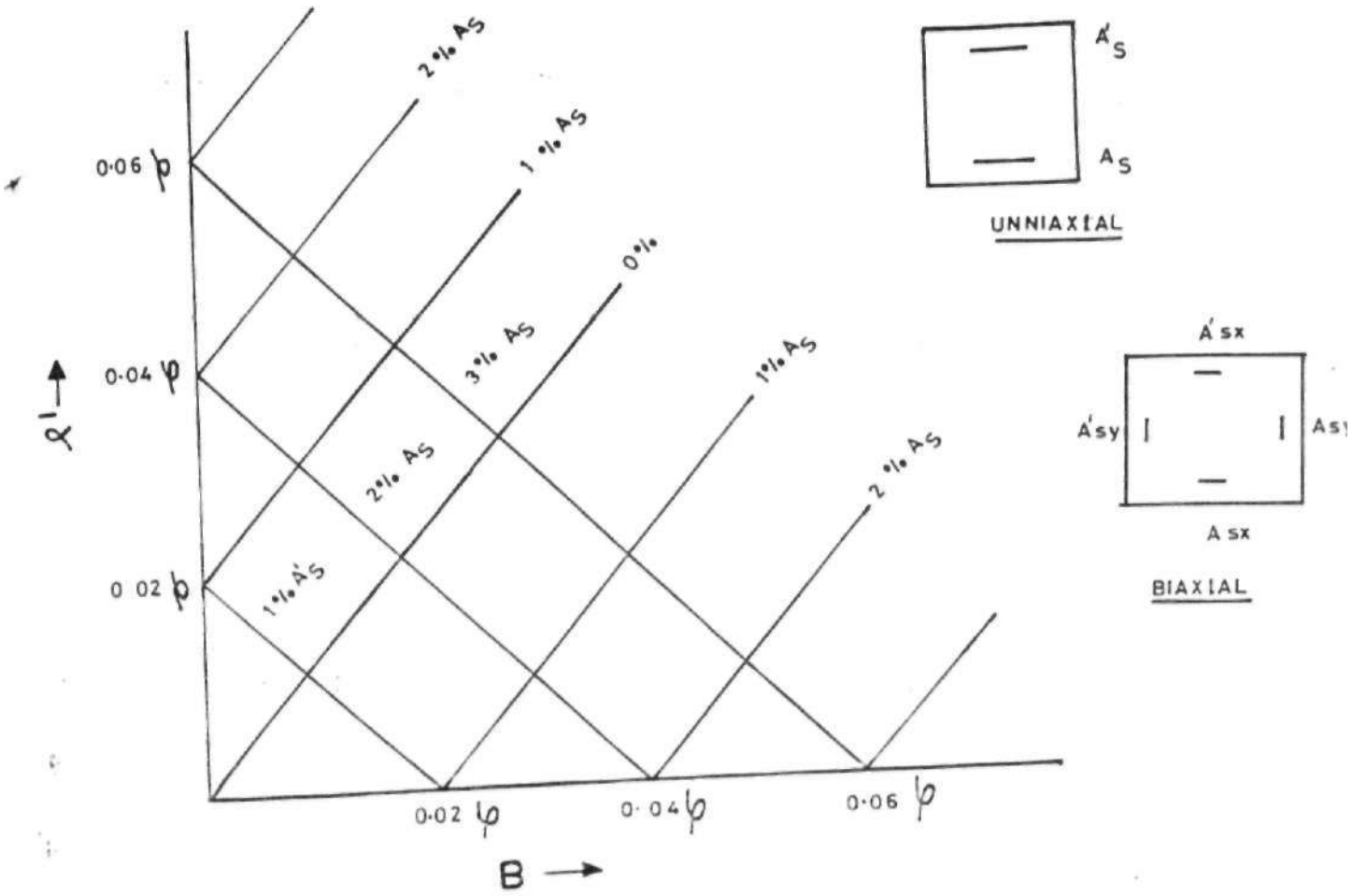
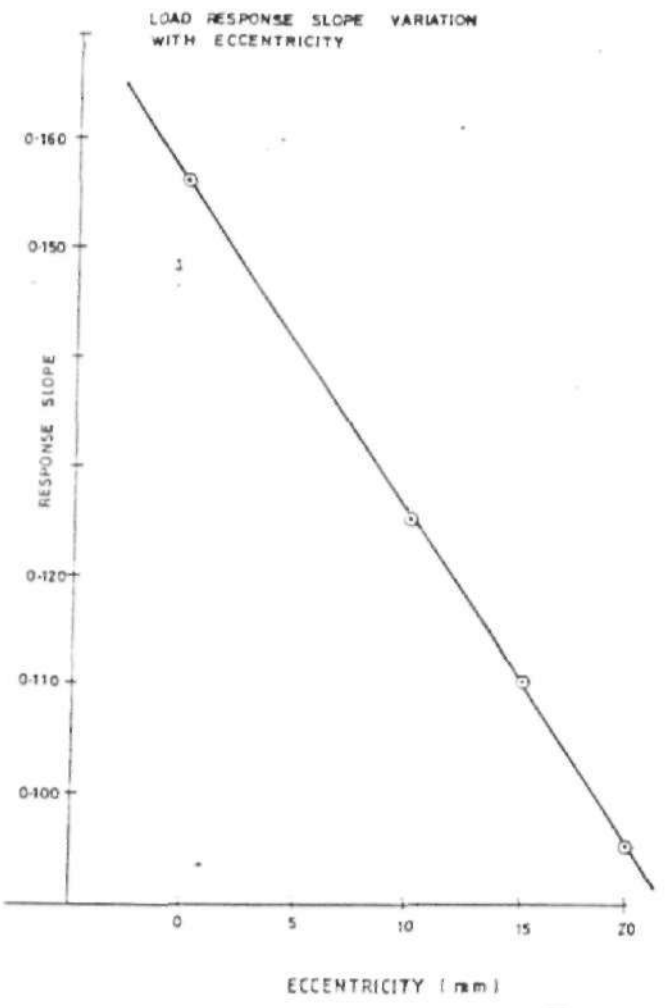


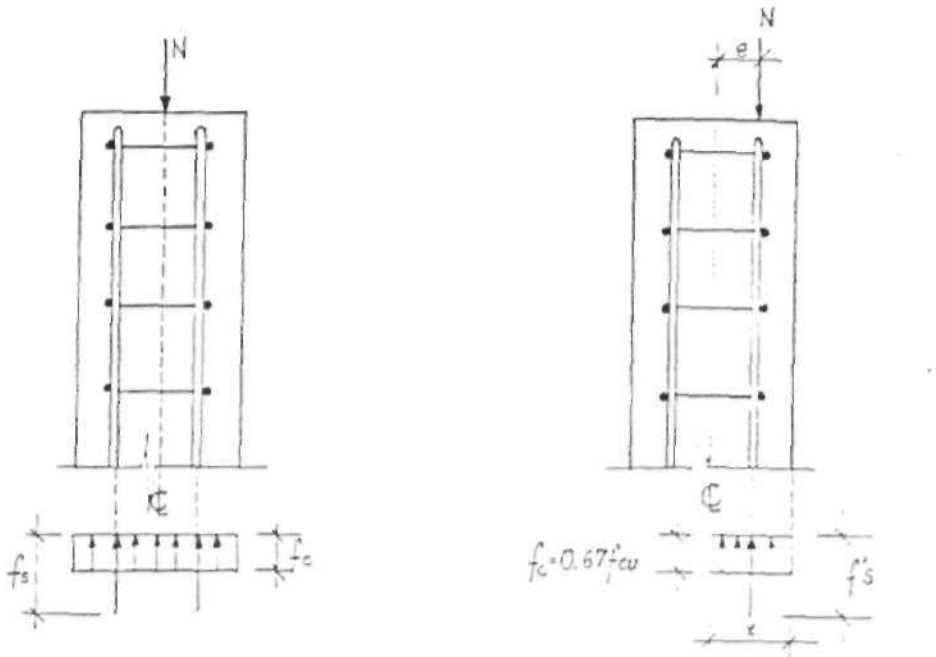
Fig 4.1 New Design Chart



Plate Model Frame Under Construction

Fig. 4.2


LOAD RESPONSE SLOPE VARIATION WITH ECCENTRICITY
RESPONSE SLOPE
0·160
0·150
0·120
0·110
0·100
0
5
10
15
20
ECCENTRICITY (mm)



A. AXCALLY LOADED SECTION B. ECCENTRICALLY LOADED

Fig. 4.3 STRESS DISTRIBUTION IN A COLUMN SECTION UNDER LOAD (OLESZKIEWICZ)

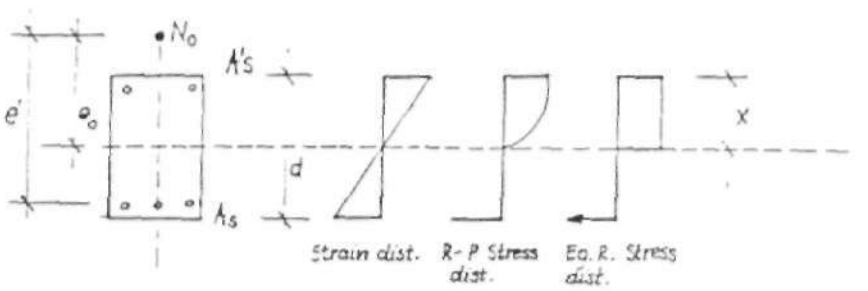
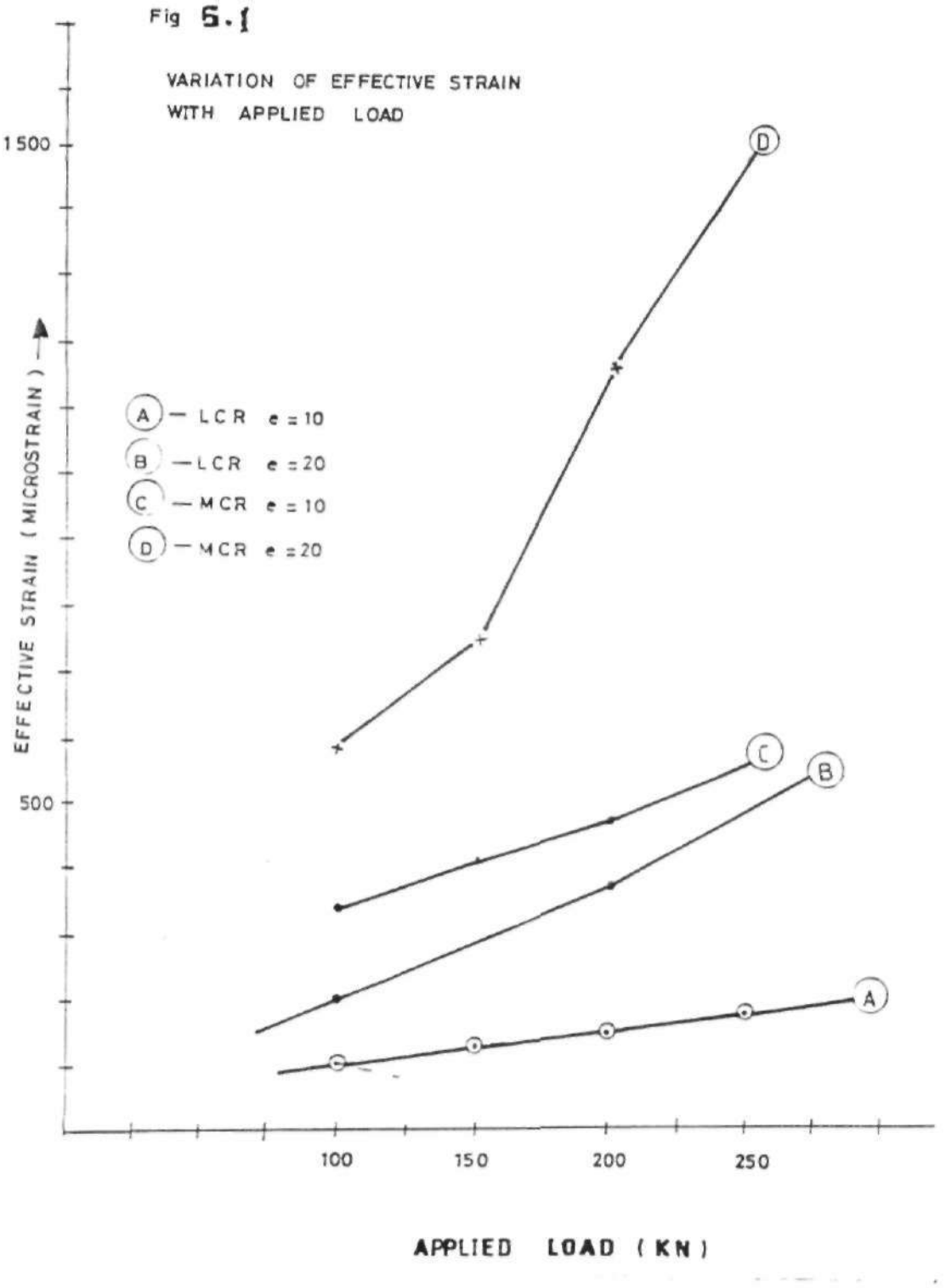


Fig. 4.4 ECCENTRICALLY LOADED COLUMN SECTION AT ULTIMATE LOAD AS CURRENTLY ASSUMED

Fig 5.1

VARIATION OF EFFECTIVE STRAIN
WITH APPLIED LOAD



and 57. The results of the test presented in figs 5.9. and 5.11 show that there is a particular reinforcement unsymmetry at which maximum failure load was recorded. This indicates that higher amount of reinforcement in a steel section may not necessirily attain higher failure load for a given eccentricity of loading. It may be concluded therefore that at an eccentricity of loading, there is an optimum reinforcement requirement for a given concrete column section. This is quite in agreement with the assumption that the moment capacity of a section is equal to the moment capacity of the reinforcements.

5.4 Reinforcement Response to Applied Load

Fig 5.1 shows the response of reinforcement to applied load, for two set of reinforcements. (A) and (B) are the load-strain relation for the less compressed steel while (C) and (D) are for the more compressed steel. The results show that the load strain relation are divergent. Inspite of the effect of shrinhage and creep, the angle of diversion remain approximately constant. This implies that the load unsymmetry in the reinforcements for a given eccentricity is

approximately a constant. Fig 5.7.1 shows a typical strain response of the less compressed steel at higher loads. The slope of the relation which is approximately linear, changes from positive to negative at higher load levels and eccentricity. Fig 5.2 shows the typical reinforcement response to applied load. This is similar to the load carried by steel for increasing applied load in the new type of composite columns. This typified the responce of reinforcement in a concrete section. The response is approximately linear just before the appearance of cracks, and then curvilinear. The appearance of visible cracks in the concrete tend to coincide with the attainment of maximum stress and the subsequent drop in load carried by the concrete similar to previous reports on columns (Furlong 1979; Ferquson 1984).

Plotting the graph of average observed failure load (Ne) against the eccentricity of loading (e)for specimen groups C and H which are prototype columns, as presented in fig 5.6. It will be seen that there is an eccentricity value at which the column attains a minimum load capacity, for a given reinforcement unsymmetry. This implies that for a given reinforcement unsymmetry,

there is a value f eccentricity at which the less compressed steel is minimally utilized. At an eccentricity above this value, the utilization of the less comressed steel increases. This agrees with the results presented in fig 3.1.

Table 5.1: The Average Load Ratios Recorded For given Reinforcement Ratio Percentages

e%	Ne/NT	- Fc / AV - Ne	- AV
5.7	85	1.30	1.08
4.2	105	1.20	1.19
3.0	102	1.35	1.07
2.7	109	1.37	1.06

Table 5.3. gives the average observed theoritical load ratio irrespective of the concrete grade, for the various reinforcement percentages. It will be observed that although columns group G with 2.7 percent reinforcement recorded highest observed/thecritical load ratios, they did not also show the highest concrete contribution to load resistance.

Table 5.2: Analysis of Grop G Specimens.

	fcu	d'	d^2	Fc	Ne	NT	Ne / NT	N*T	Fc - Nc	Ne - bhfc	Reinf. ratio
G1	46	20	46	291	220	283	78%	330	1.32	0.71	5.72
G2	46	20	43	295	259	243	107	294	1.14	0.34	4.17
G3	46	20	40	299	228	237	96	277	1.31	0.74	3.03
G4	46	20	41	300	236	216	109	266	1.27	0.77	2.73
G5	54	20	46	341	260	297	88	370	1.31	0.72	5.72
G6	54	20	41	347	281	279	101	338	1.24	0.78	4.17
G7	54	20	41	351	250	253	99	314	1.40	0.69	3.03
G8	54	20	41	352	260	232	112	303	1.35	0.72	2.73
G9	57	20	44	360	281	320	88	388	1.28	0.74	5.72
G10	57	20	42	366	298	281	107	350	1.23	0.78	4.17
G11	57	20	42	370	279	254	110	327	1.33	0.73	3.03
G12	57	20	42	372	248	231	107	314	1.5	0.65	2.73

Table 5.3.:Variation of Dimensionless Load Capacity with Reinforcement Ratio

Reinforcement ratio	AV - Ne / NT	Fc (AV) - Ne	Average Dimensionless Load Capacity
5,72	85%	1.30	1.08
4.17	105%	1.20	1.19
3.03	102%	1.35	1.07
2.73	109	1.37	1.06

5.5. Statistical Analysis of Result Observed Failure Loads

It is not a common practice to subject test results on structural members, especially the load capacities, to statistical analysis for the obvious reason of limitation on the specimen population (Brown 1988; Macgregor 1983). However in this research a faily large number of model columns were crushed to failure to estimate their load capacities. With such a large number, sixty-six, it may be expected to be subjected to statistical analysis.

The strength of a reinforced concrete member may vary from the calculated or nominal strength when tested in the laboratory due to variations in the material strength and dimensions of the member as well

as uncertainties inherent in the mathematical model used to compute the member strengths (Macregor et al 1983). The basic variables affecting the strength of reinforced concrete members are the actual concrete strength in compression and tension, yield strength of reinforcement and dimensions of the cross section. There had been various attempts to determine the coefficient of variation of concrete, which has been suggested to be between 12 and 18 percent. Macgregor (1981) had estimated the coefficient of variation to be 20.7 percent. Attempts were made to relate in mathematical terms, the errors in an experimental research. This may be simplified with the equation (Jeary et al 1982; Lepert et al 1982; Macgregor 1983).

$$V_{TC} = \sqrt{V_m^2 + V_t^2 + V_s^2} \quad \ldots\ldots 5.1$$

where V_{TC} is the coefficient of variation error obtained directly from the comparison of the measured and calculated strengths; Vm is the variability of the model; Vt represents the uncertainties on the measured loads due to inaccuracy of the gauges and errors in the definition of failure; Vs is the other erros that may depend on the physical test parameters. Thus the error due to mathmetical model may be given as

$$V_m = \sqrt{V_{TC}^2 - V_s^2 V_t^2} \quad \ldots\ldots\ldots\ldots 5.2$$

where Vs is taken as equal to the material variability.

Concrete strength variability is of major importance in the analysis of variability of structural member than other factors. Variability of concrete increases as the strength increases as was observed by Road Research laboratory in London. Nandivaram et al 1988. "It may be stated that observed error = Model error - material variability".

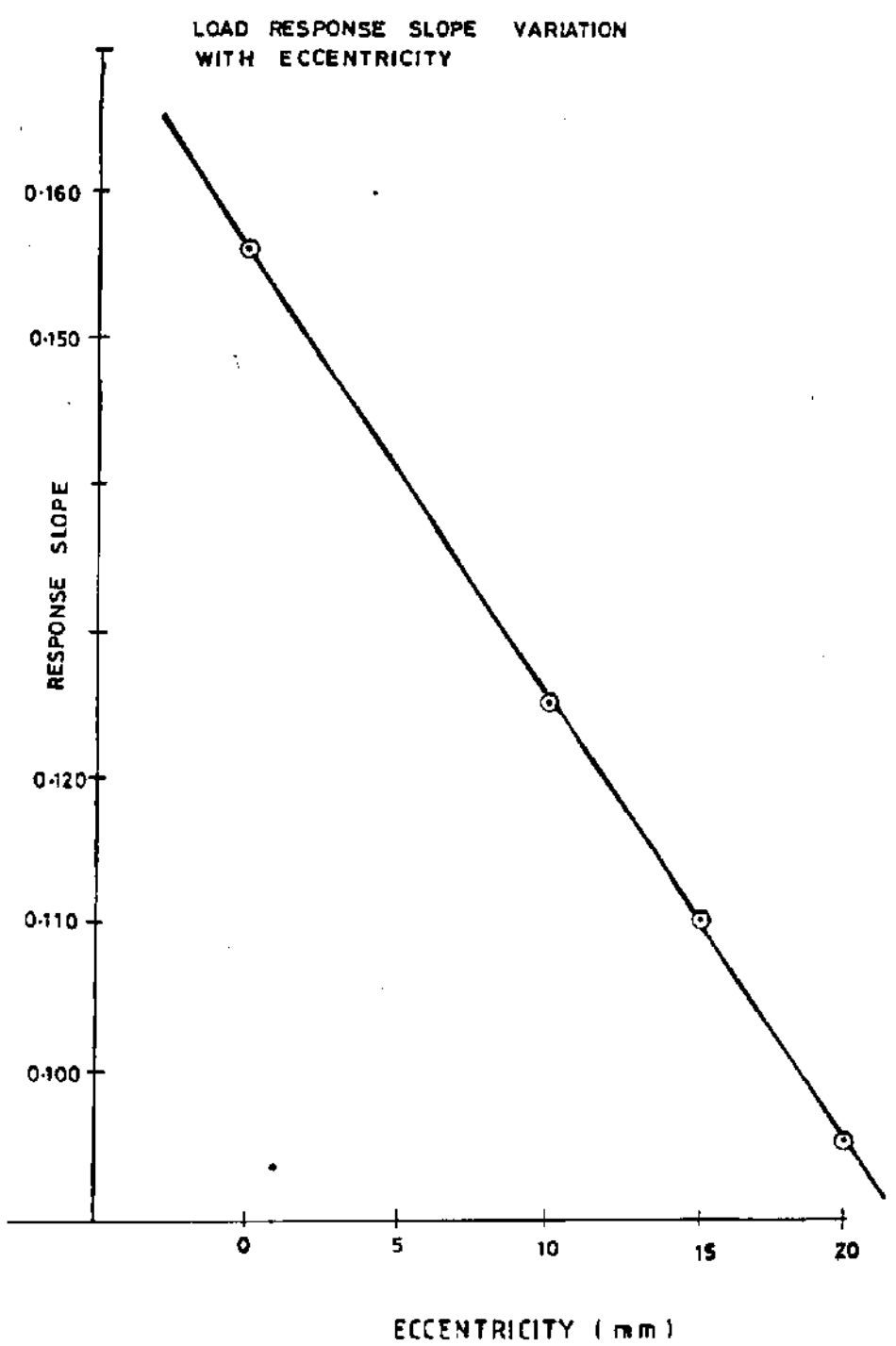
Table 5.5 shows the dispersion of the observed results with respect to variation in model ratio.

Table 5.5: Dispersion of Observed-Theoretical Load Ratios With Respect to Model Ratio.

l/h	Average Ne/Nt	Standard Deviation	Model Ratio	No
3	98%	15%	1/3	19
5	96%	14%	1/2	26
10	105%	21%	1	16
15	111%	16%	1	5

The dispertion of the results does not appear to be affected in a definite manner, by the model ratio of the test specimens. For the sixty-six specimens tested uniaxially, the average ratio of observed failure load/theoretical failure load is 99.77 percent with a

Fig. 4.2


LOAD RESPONSE SLOPE VARIATION
WITH ECCENTRICITY
RESPONSE SLOPE
0·160
0·150
0·120
0·110
0·100
0
5
10
15
20
ECCENTRICITY (mm)

standard deviation of 17.22 percent. If a coefficient of variation of concrete strength is taken as 20 percent as suggested (Macgregor et al 1983), applying equation 5.3.

It is observed that the model error is negative indicating very high accuracy of the mathematical load-estimation model. This is in addition to the average observed-theoretical load ratio of 100 percent. It can therefore be said that there is very good agreement between theory and observation Florek (1985).

Using the equation suggested by Oleszkiewicz (1985) to determine the load capacity of the columns and comparing with the observed failure loads. The average observed/theoretical failure load ratio is obtained as 83 percent with a deviation of 19 percent resulting in a coefficient of variation of 23 percent. It will be inferred therefore that the method by Oleszkiewicz over estimates the capacities of short reinforced concrete columns under small eccentricity.

5.6 Effect of Eccentricity

For the specimen group H, the observed/theoretical failure load ratios were compared in terms of it's

variation with eccentricity. Table 5.6 presents this ratio for different eccentricities. This is plotted in fig 5.10.

Table 5.6: Observed/Theoretical Failure Load Ratios for Different Eccentricities of Group H

e (mm)	Ne/N_T %	Ne/Nolez
0	69	64
20	86	70
30	99	86
40	101	83
50	116	109
60	121	-

The graph shows an approximately linear variation of the Ne/N^T ratio with eccentricity. This relation is approximated as

$$Ne/N_T = 0.69 + 0.98(e/h) \quad \ldots\ldots\ldots\ldots \quad 5.6.1$$

This realtion may be used to modify the expected failure loads or design reinforcement requirement using the proposed methods. The low value of the ratio for e = o is attributed to the low utilization of steel in unsymmetrically reinforced columns subjected to axial

loads only. Low Ne/Nt values recorded agrees with the results of Umaru (1989).

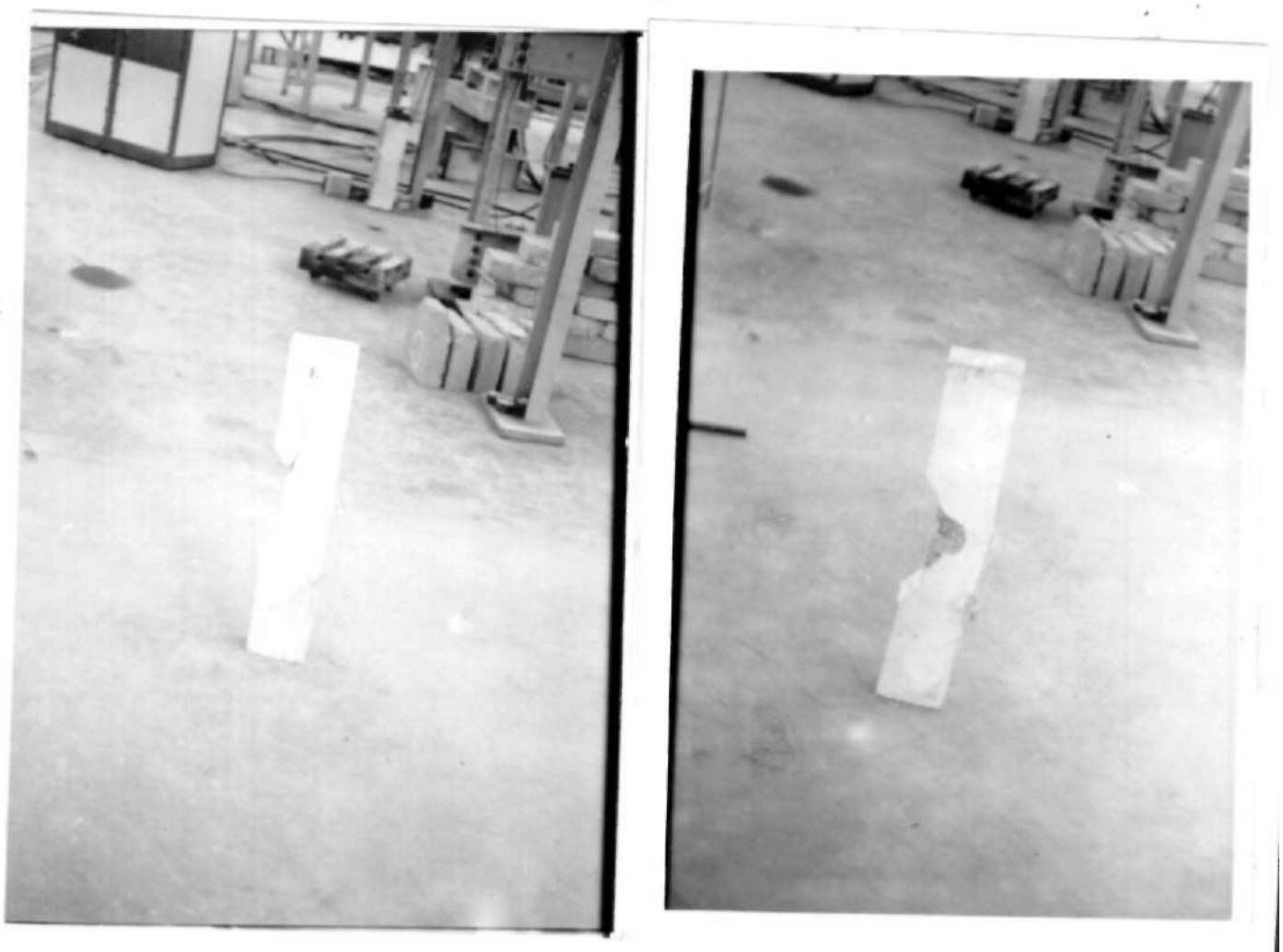


Plate 4 crushed specimens

Table 5.7 Observed/CP110 Load Variation With Eccentricity

e (mm)	Ne/NCP110
10	68
20	65
30	62
35	66
40	52
50	53

Umaru (1989) tested nineteen short columns to flailure at zero eccentricity. The spacing of the ties varies from 0.66h to 0.17h. Inspite of the closeness of the ties, the average ratio of observed/expected load was 82 percent. This appears to be in agreement that reinforced concrete column may not attain its maximum load capacity due to under-utilization of the reinforcements. This may also suggest that the failure stress of concrete in uniaxially loaded short columns is about fifty percent of the cube strength.

From fig 3.1, the observed dimensionless load capacity of the tested columns were all about one and this did not appear to vary with reinforcement unsymmetry. In the design of short R.C Columns under

small eccentricity of loading. It may be observed that at a given eccentricity of loading, the recorded dimensionless load capacity of the column is dependent on the reinforcement unsymmetry.

Table 5.8: Dimensionless Load Capacity Group G

	Fcu=46	fcu = 54	fcu = 57	- AV
A=192	1.06	1.07	1.10	1.08
A=103	1.25	1.16	1.16	1.19
A= 91	1.10	1.03	1.09	1.07
A= 61	1.14	1.07	0.97	1.06
- AV	1.14	1.08	1.08	-

Table 5.9: Dimensionless Load Capacity Group H

Eccentricity	fcu=27	fcu = 31	- Average
e = 0	1.16	1.68	1.42
e = 20	-	1.15	1.15
e = 30	1.45	0.86	1.15
e = 40	1.0	0.92	0.96
e = 50	0.99	0.98	0.99
e = 60	0.98	0.99	0.99
- AV	1.92	1.10	-

Table 5.10: Dimensionless Load Capacity All Specimens

e	α	n
0	1.48	12
10	1.13	17
15	1.20	3
20	1.11	6
25	1.10	7
30	1.07	8
40	0.97	3
50	0.95	6
60	0.99	2

For the 66 specimens tested, the average recorded value of α is 1.17 with a standard deviation of 0.259 (resulting in a coefficient of variation of 22%).

It will be observed that the highest average value of α was recorded for eccentricity of zero. This is significant when compared to the lowest Ne/N^T ratio as which indicates that the low value of Ne/N^T for zero eccentricity is as a result of low utilization of reinforcement. This is also indicated by low strains of reinforcement.

However the results justify the assumption of α = 1 and above for designs.

Fig. 5.3

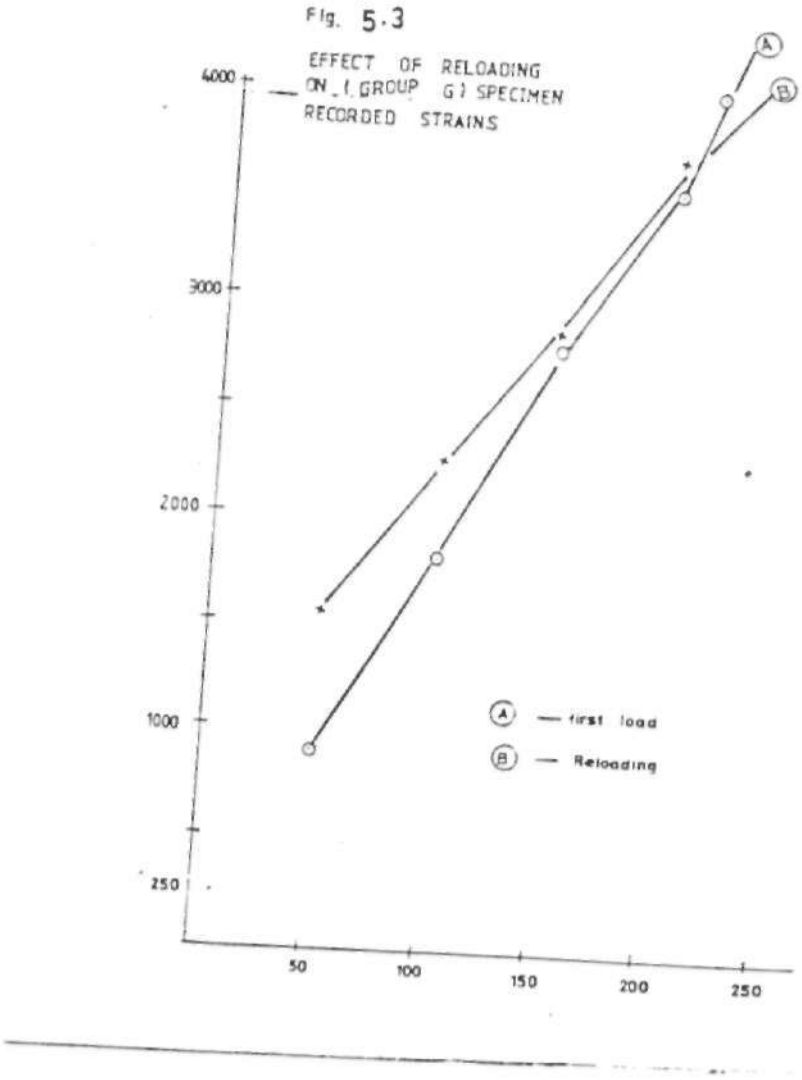

EFFECT OF RELOADING
ON I GROUP G1 SPECIMEN
RECORDED STRAINS
4000
3000
2000
1000
250
50
100
150
200
250
A — first load
B — Reloading

Fig 5.11

FAILURE LOAD VARIATION
WITH TOTAL REINFORCEMENT

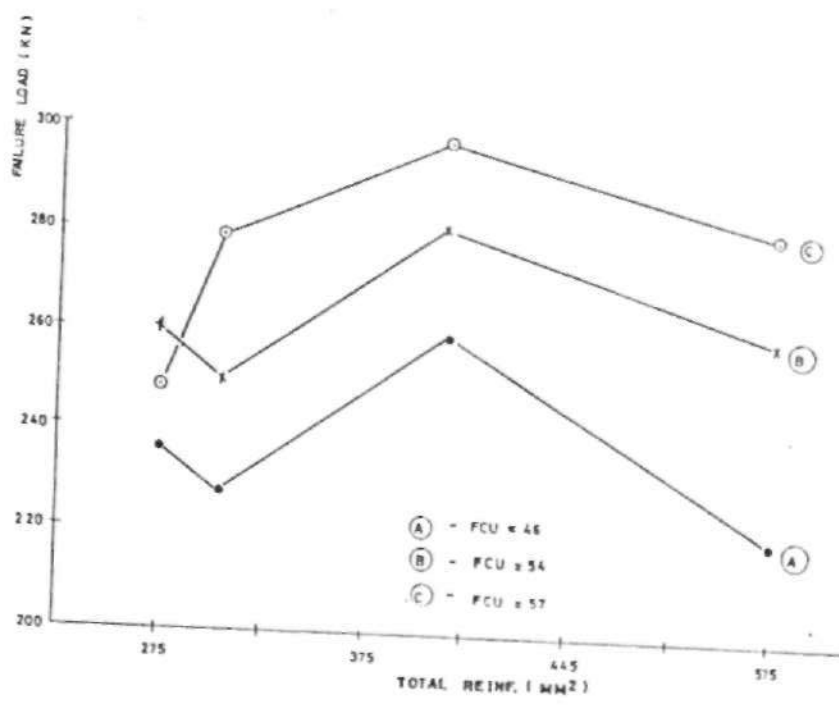


Fig 5.12

EFFECT OF TOTAL REINF AREA
ON FAILURE LOAD RATIO (group G).

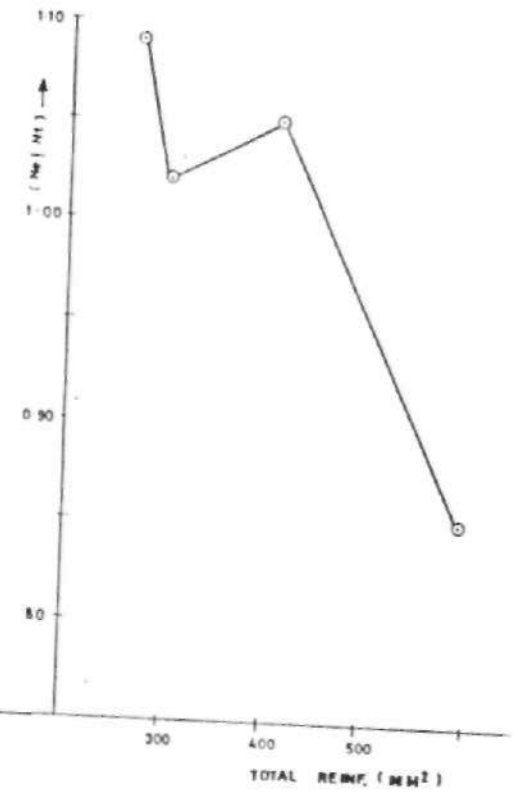


Fig. 5.4

LOAD RESPONSE OF SYMMETRICAL Reinforcement AT DIFFERENT ECCENTRICITIES

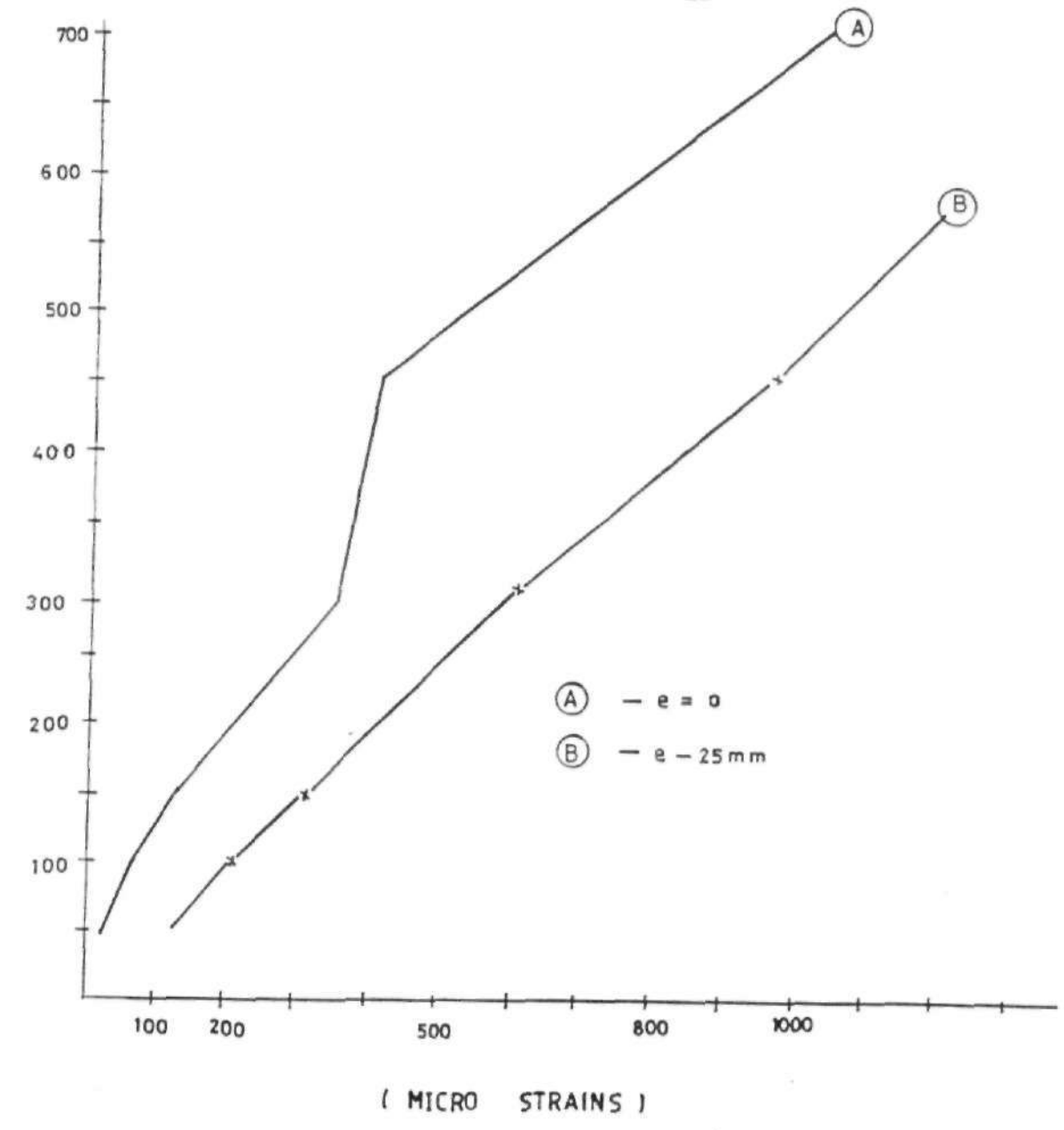
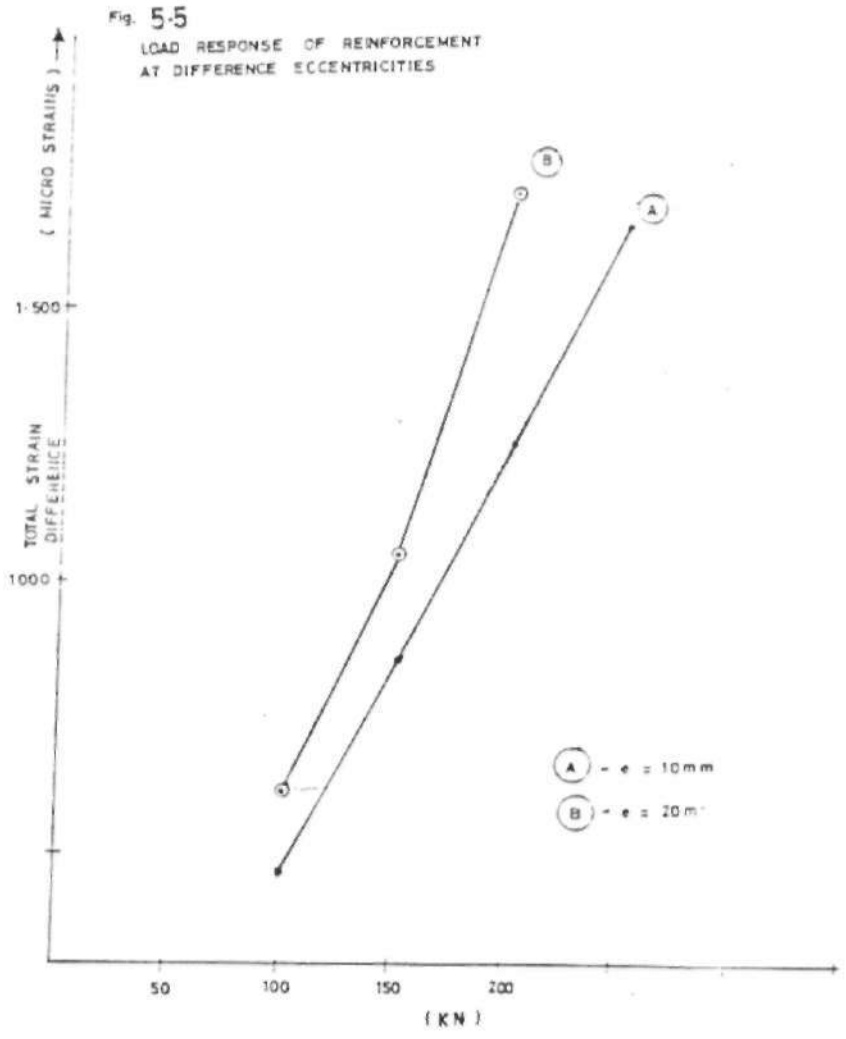


Fig. 5·5
LOAD RESPONSE OF REINFORCEMENT
AT DIFFERENCE ECCENTRICITIES



CHAPTER 6

ECCENTRICITY AND REINFORCEMENT UNSYMMETRY

6.1 General

One major assumption of current design method is linear strain distibution within the section depending on the point of consideration from the neutral axis (Bungey 1982, Priestly et al 1983; Baikov 1978). In the proposed design method, it is assumed that in a concrete column, static equillibrium of forces is more relevant than compatibility of strains at failure. Thus it is assumed that the strain distribution may not be linear at failure. In addition the minimum moment capacity of the reinforced concrete column is the moment capacity of the reinforcements. For a given section, as the applied moment increases, the reinforcement difference or unsymmetry, increases. This implies that there is a relation between reinforcement unsymmetry and the eccentricity of loading. This was earlier implied by the suggestions of Oleszkiewicz (1985)*. It is also important to determine wheather a symmetrical reinforcement arrangement of say A^1s = 4R12 and As = 4R12 is less safe than one with unsymmetrical arrangement of A^1s = 5R12 and As = 2R12. In other words, reinforcement unsymmetry could achieve reduced cost and enhanced

Fig 5-2

TYPICAL REINFORCEMENT (MC)
RESPONSE TO LOAD AT
A GIVEN ECCENTRICITY

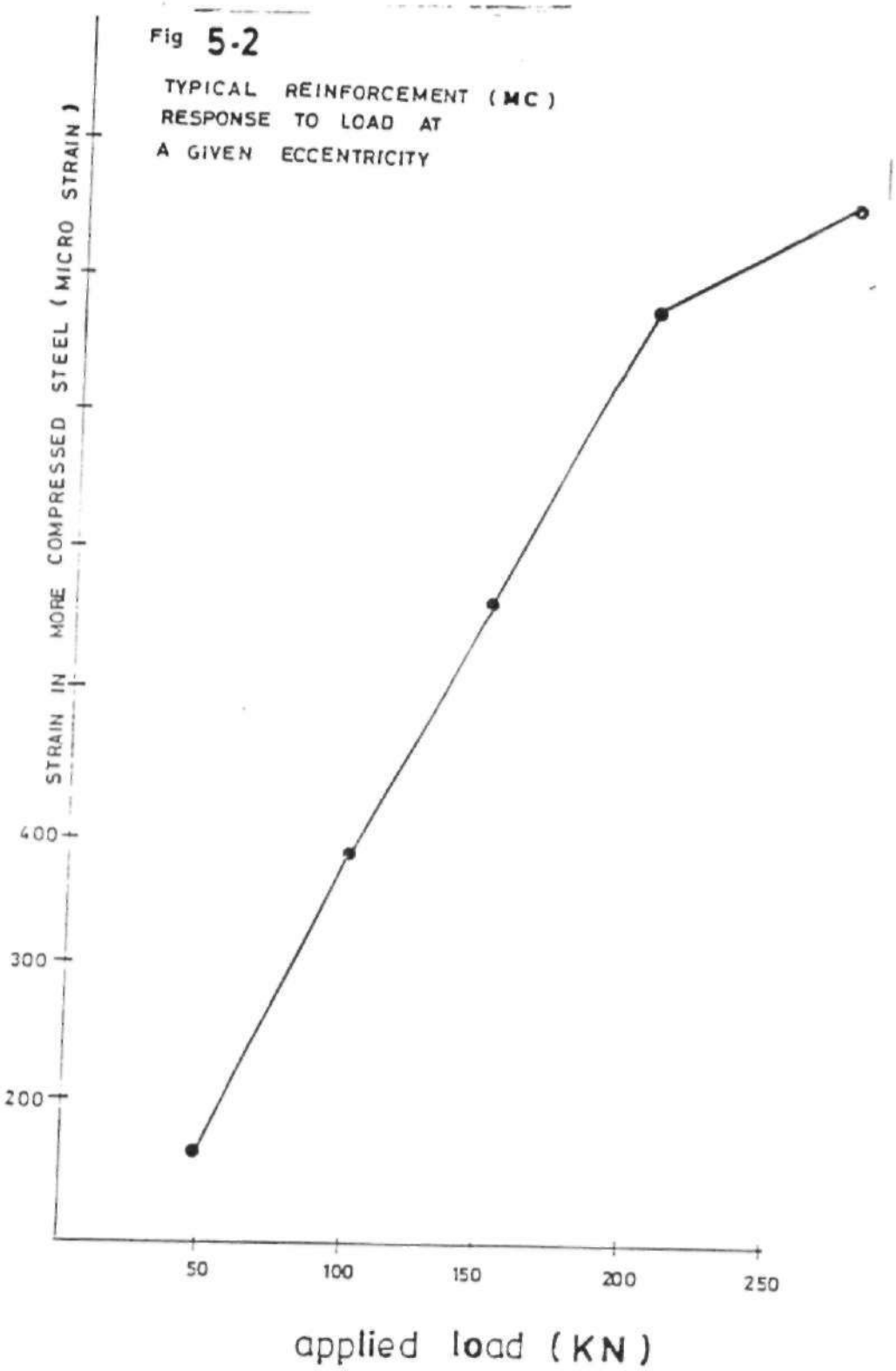
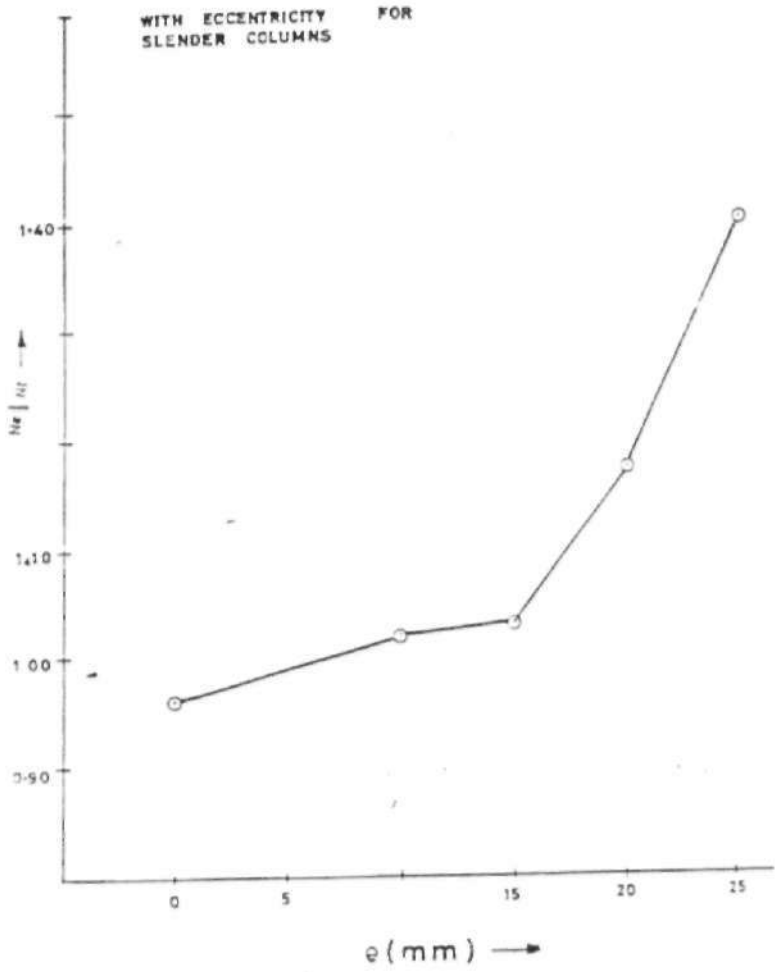


Fig. 5.6

VARIATION OF LOAD RATIO (N_e/N_t) WITH ECCENTRICITY FOR SLENDER COLUMNS



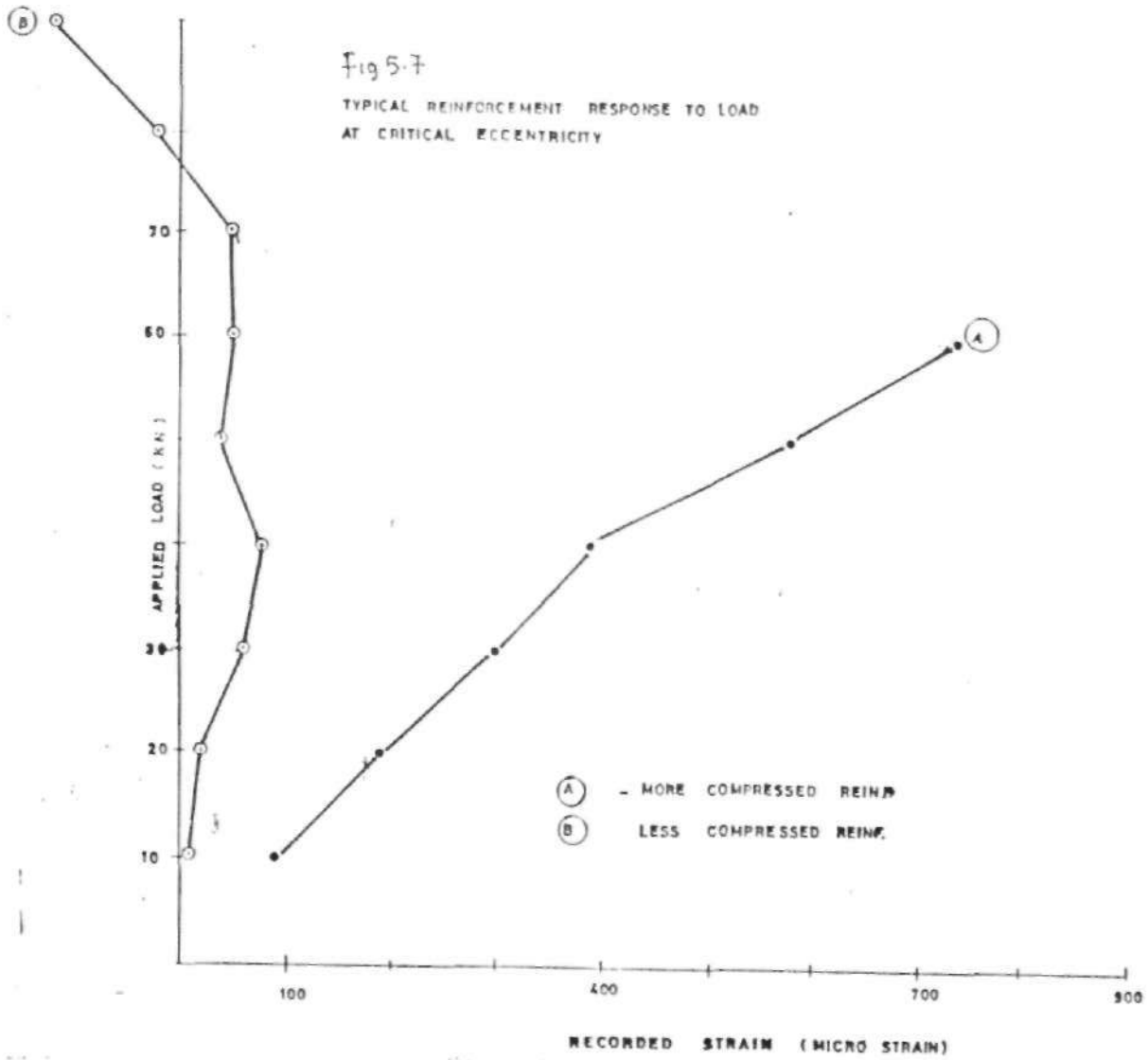
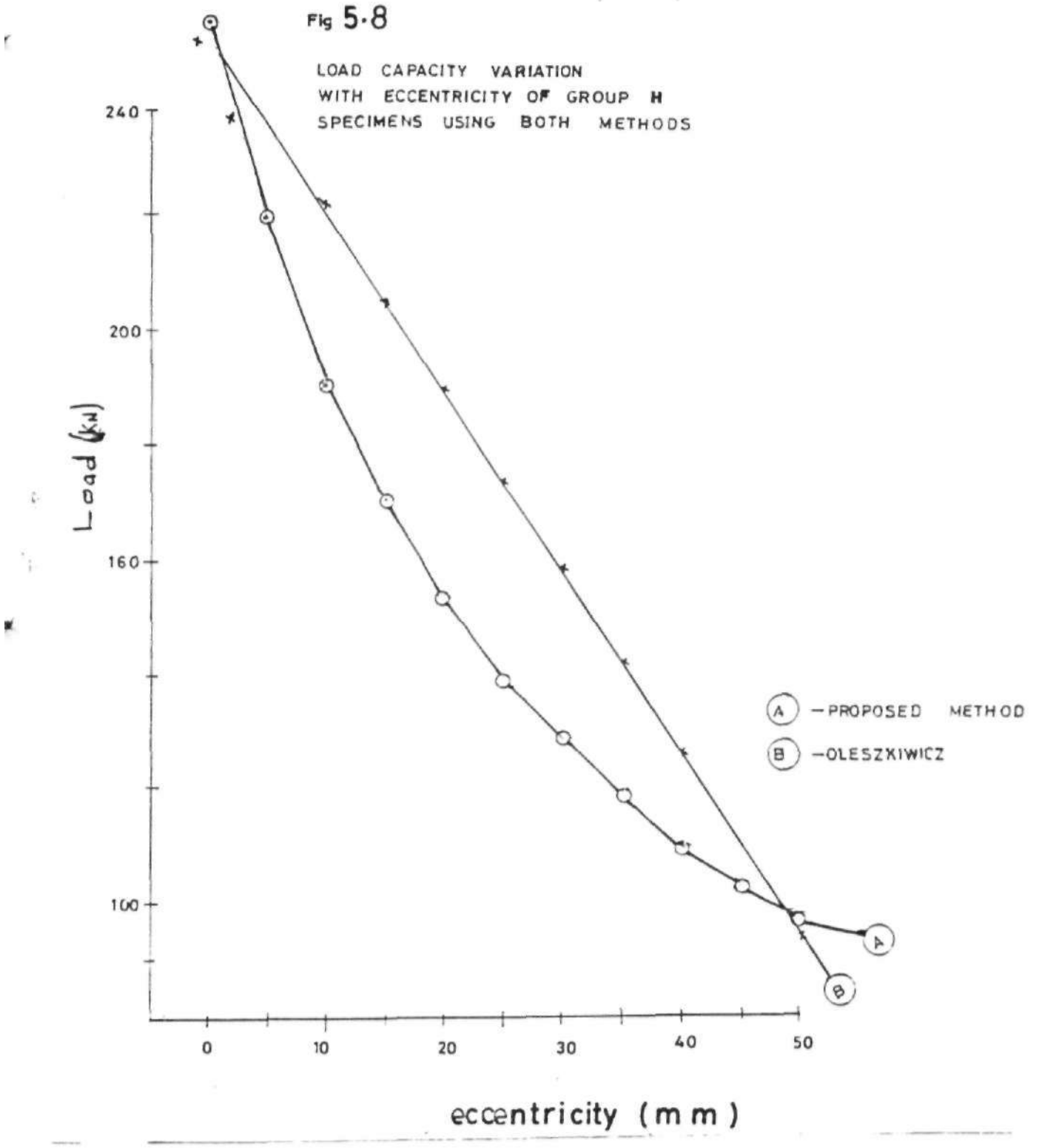


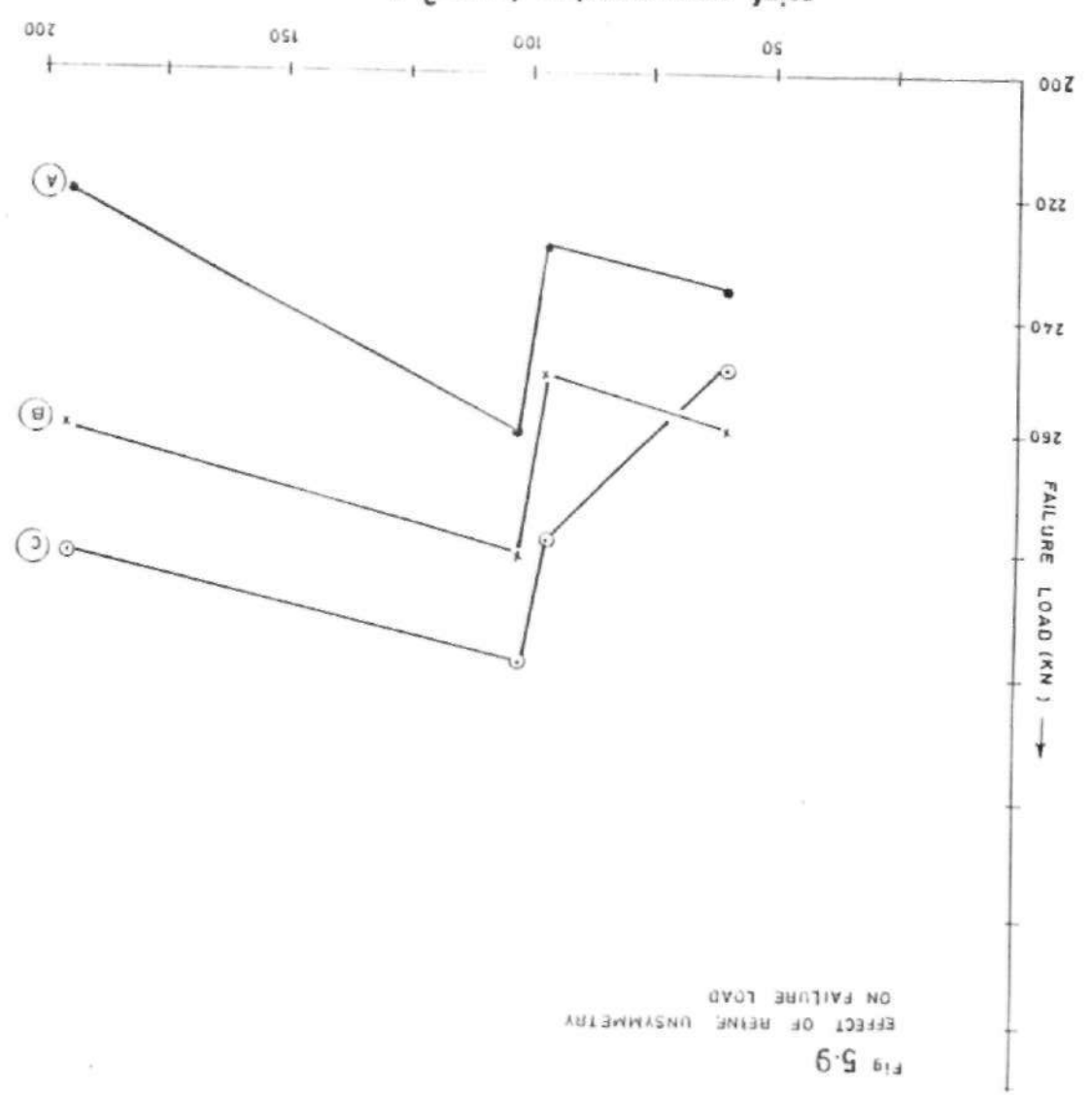
Fig 5.7

TYPICAL REINFORCEMENT RESPONSE TO LOAD AT CRITICAL ECCENTRICITY

Fig 5.8

LOAD CAPACITY VARIATION
WITH ECCENTRICITY OF GROUP H
SPECIMENS USING BOTH METHODS



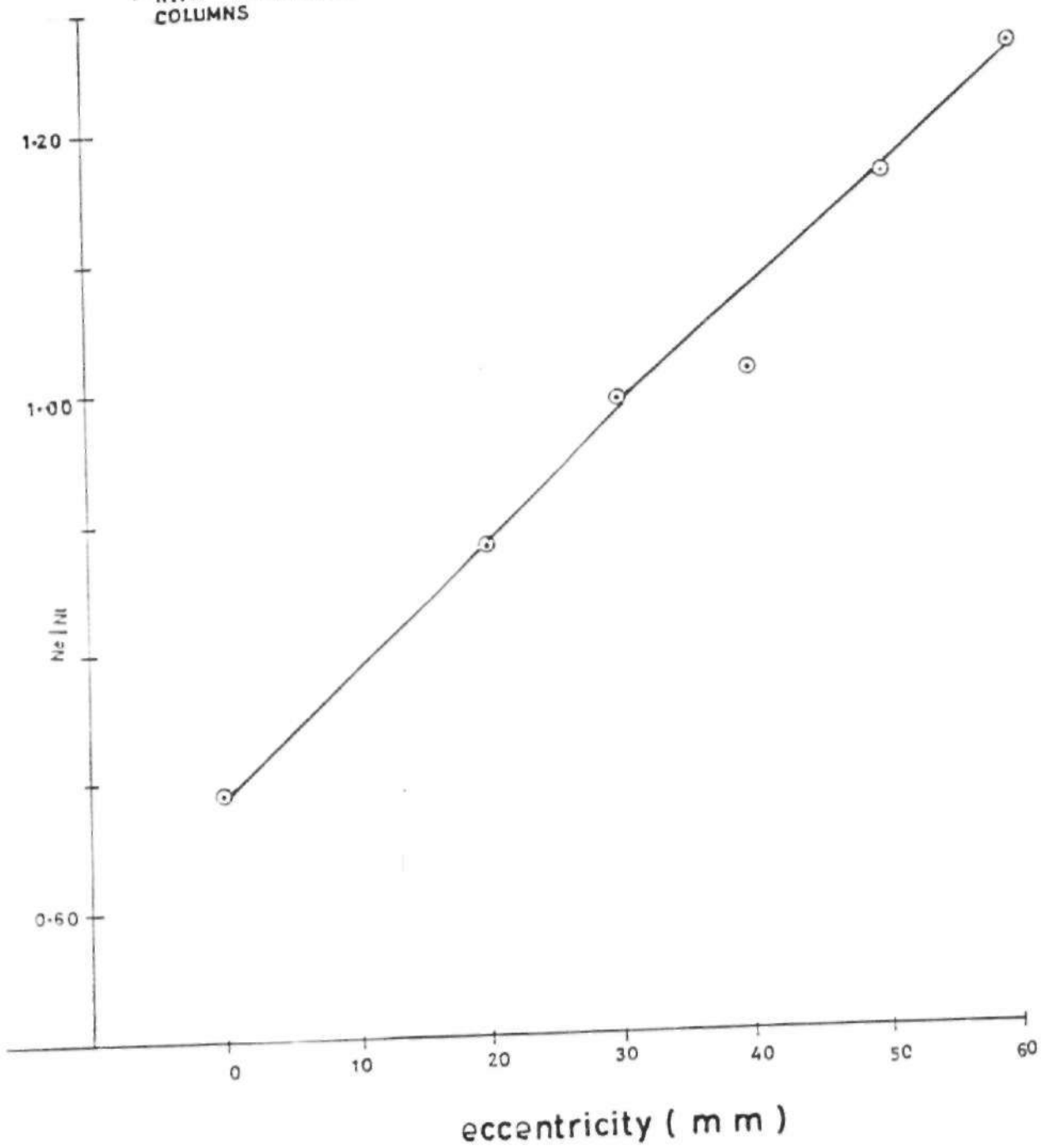


(A) - f_{fu} = 46
(B) - f_{cu} = 54
(C) f_{fu} = 57

Fig 5.9
EFFECT OF REINF. UNSYMMETRY
ON FAILURE LOAD

Fig. 5.10

VARIATION OF LOAD RATIO (Ne/Nt) WITH ECCENTRICITY FOR SHORT COLUMNS



strength simultaneously. Taylor et al (1983) in their tests on new type of composite columns estimated the theoretical strength of columns neglecting small eccentricities. The flailure patterns suggested that for such eccentrically loaded composite columns, the steel sections should have been unsymmetrically arranged to achieve higher failure loads.

6.2 Strain Distribution

Fig 6.1. shows the representation of the strains recorded at various points; more compressed face, more compressed steel, less compressed steel and the tensile face of the concrete, at a given load level. Although this cannot be said to represent the strain distribution across the section, it however shows that for the class of columns tested, the strain distribution at the more compressed section of the column may not be linearly dependent on the distance of the reinforcement from the compression face as is currently being assumed by designers (Adepegba 1989; Cedeolin et al 1984; Oliva et al 1987; Papia et al 1989). The strains in the tensile face appear to be linearly dependent on the distance of the point from the neutral axis. In the current design practice, under the limit state method, the reinforcement is

assumed to be a point not considering the relative modulus of elasticity with respect to concrete. Plotting the strain distribution across the transformed composite section produces much lower depth of neutral axis than without due regard to modular ratio. It may be inferred therefore that the depth of neutral axis and the strain distribution across a reinforced concrete section depends on the reinforcement unsymmetry.

6.3 Difference in Strain

The difference in strains of the reinforcements represent the difference in loads carried by the reinforcements at a given load level (Meek 1963; Papia 1988). Table 6.1 give the total difference in strains at different load levels for specimen group G. This is plotted in fig 6.2 and show near parallels.

---

*Refer to chapter 2.5

Table 6.1: Total difference in Strains at Different Loads

e	50KN	100KN	150KN	200 KN	250KN
0	68	364	699	1024	1340
5	70	413	770	1120	1460
10	79	465	854	1250	1650
15	790	1260	1740	1930	-
20	150	615	1049	1713	2260

The total difference in strains is the difference of the sum of the strains on the More compresses steel and the less compressed steel taking congisance of area of reinforcement. If the values for eccentricity of 15 are neglected, it shows that the gradients of the relation between eccentricity and total strain difference changes with increase in load. This indicates that as the load carried by the column itself increases, the difference in the load carried by the more compressed steel and the less compressed steel increase in same manner, for a given eccentricity. Table 6.1 provides the average strain recorded for group B specimens, at various load levels. Plotting the graph of the applied load against the recorded strains give two straight lines with different slopes.

Table 6.2: Recorded Strains at Variousw Load Levels for Symmetrically Reinforced Columns of B2 Group

Eccentricity (mm)	Average Strains at Different Loads					
	50KN	100KN	150KN	300KN	450KN	700KN
0	25	78	122	360	412	1033
25mm	118	206	315	602	978	1460

The slope for zero eccentricity is almost double that for eccentricity of 25mm. This indicates that the recorded strain depend on eccentricity. From fig 4.2. and table 6.1. It will be observed that the slope for the relation between total strain difference and applied load is not zero. Table 6.3 which is an attempt to compare the loads carried by the reinforcements due entirely to the eccentricity, gives the values of total strain difference less the total strain difference for zero eccentricity, for group E specimens.

Table 6.3: Total Strain less Strain for Zero Eccentricity at Various Loads

	Strain Values in Microstrains				
	Eccentricity	100KN	150KN	200KN	250KN
LCR*	10mm	94	124	140	170
MCR	10mm	195	275	366	480
LCR	20mm	334	394	466	550
MCR	20mm	585	744	1155	1470

The graph of load against recorded strain produced from table 6.3.3 produced straight lines in the form of the equation

$\text{Log } N_1 = C_2 + K_1 \log (E_1 - E_2 \ldots 3.1$

$\text{Log } N_2 = C_2 + K_2 \log (E_2 - E_2) \ldots . 3.2$

where N_1 and N_2 are the applied load level and E_1 is the strain recorded for the less compressed steel at eccentricity of e. while E_2 is the strain recorded for the more compressed steel at eccentricity of e. It was observed that $K_1 - K_2 = 0.052$ and is a constant. This shows that the slopes of the relation K depend on the eccentricity of load. Plotting the graph of applied load against recorded strains for table 6.1

produce straight lines in the form $N = C_e + Ke\ (\ Ee)$

..... 3.3

where N is the load level and Ce and Ke are constants of proportionality. Ee is the total strain difference for a given eccentricity. The constants are given in table 6.4.

Table 6.4: Constants of Proportionality for Equation 6.3.3.

e	Ce	Ke
0	40	0.156
5	41	0.141
10	40	0.110
15	-45	0.110
20	40	0.0948

From the suggestion by Oleszkiewicz (1985) given that the parameters of group E specimens are: fcu = 57, fy = 250; b = 100 reinforcement unsymmetry is proportional to eccentricity and the constsant of proportionality K,

$$K = 2 \times \left(\frac{0.67 \times d57}{250}\right)(100) = 30.552$$

Therefore unsymmetry = 30.552 e 3.6.4

For a given material property, load on reinforcement depend on the strain of reinforcement. For a given

load level, assuming full steel utilization the stress in steel is proportional to the area. Therefore it can be observed from table 6.4 that

$K_o - K_e)$ (277) = 30.552*e 6.3.5

277 is about the area of more compressed steel. It will be concluded that within the tested specimen values, the reinforcement unsymmetry is directly proportional to the eccentricity of load, as was suggested by Oleszkiewicz (1985).

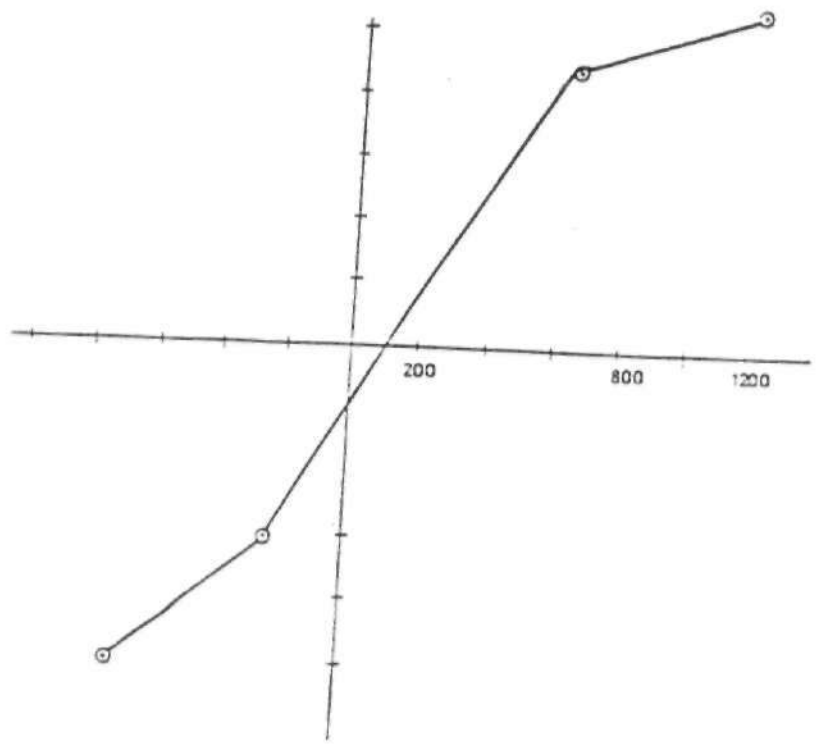
From the results presented, it will be inferred that for unsymmetrically reinforced concrete columns, the rate of increase in the difference in the loads carried by the more compressed and less compressed steel is not zero for zero eccentricity, the ultimate load capacity of an unsymmetrically reinforced concrete column may not be equal to the simple summation of material strengths when loaded axially. A simple equation may be suggested for the load capacity of an axially loaded unsymmetrically reinforced concrete column as Nu = fcbh + f^1sA^1s + K(fsAs).... 3.6.6

where K is a reduction factor dependent on the unsymmetry of reinforcements. This may also be justified by the fact that the plastic centroid of a transformed section is eccentrically located with

respect to the physical centroid of the column section at which an axially applied load is supposed to act, thus producing an eccentrircity in the transformed section. (Oladapo 1981). From the results presented in the text, it shows that at a given eccentricity, an increase in load produce increasing difference in the utilization of the steel reinforcement. This implies that the effectiveness of reinforcement in load resistance depends on the eccentricity of loalding. The product of the reinforcement ratio of less and more compressed steel, their respective distances from the centroid of the column and their respective observed strains, is proportional to the dimensionless moment capacities of the reinforcements. The constant of proportionality depends on the effective dynamic modular ratio between concrete and steel which is indeterminable due to the effects of creep and shrinkage. Table 6.5 presents this ratio in comparison with the applied moment. This is plotted in fig 6.2. and shows an approximate linear relation, justifying the assumption that at optimum utilization of concrete under small eccentricity, the reinforcement moment capacity is equal to the applied moment.

Fig. 6·1

STRAIN DISTRIBUTION ACROSS A SECTION AT A GIVEN LOAD LEVEL (GROUP H).



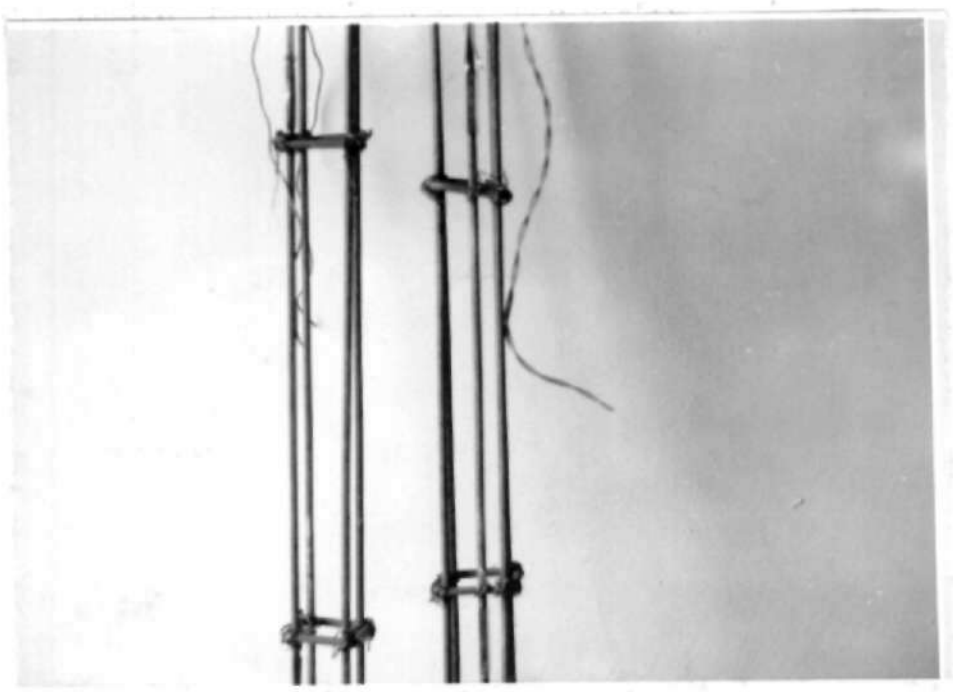


Plate 5 Reinforcement of Model frame

Table 6.5: Dimensionless Moment Response of Reinforcements for Group H Specimens, e = 50mm

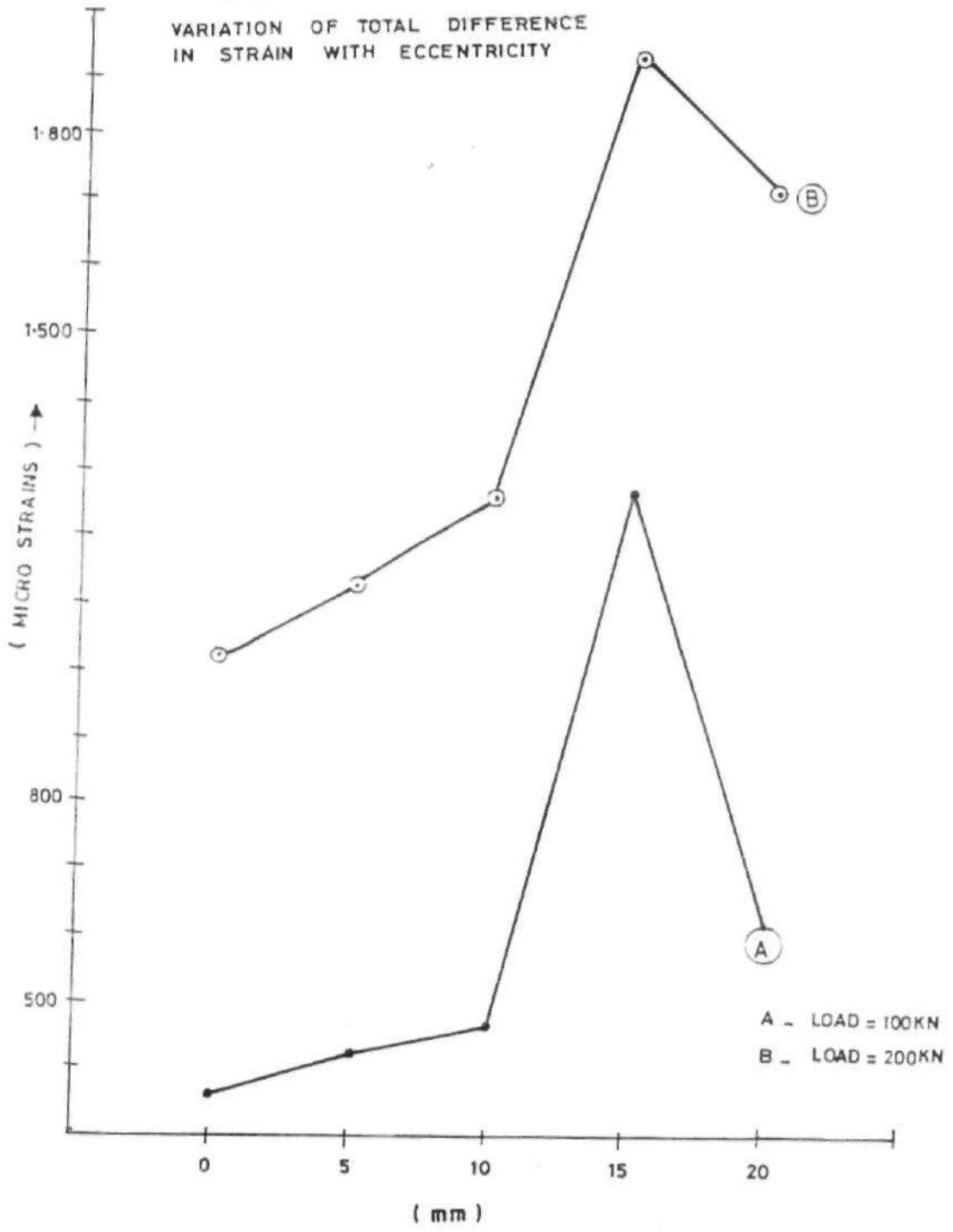
M	$B^1s + Bs$
0.5 KNM	0.122 mm
1.0	0.275 mm
1.5	0.420 mm
2.0	0.602 mm
2.0	0.828 mm
3.0	1.133 mm

Table 6.6: Average Recorded Strains on Reinforcements of Various Load Levels for Specimen Group E.

	Recorded Strains in Micro Strains					
	e = o		e = 10mm		e = 20mm	
Load (KN)	LCR	MCR	LCR	MCR	LCR	MCR
50	46	48	85	83	165	160
100	88	180	135	245	255	375
150	123	315	185	408	320	560
200	160	580	230	570	393	830
250	200	580	285	740	475	1070

The results indicate that irrespective of the eccentricity of loading, at a load of 50KN, the strains in the less compressed steel are equal to the strains

Fig. 6.2


VARIATION OF TOTAL DIFFERENCE
IN STRAIN WITH ECCENTRICITY
1·800
1·500
800
500
(MICRO STRAINS)
0
5
10
15
20
(mm)
A
B
A _ LOAD = 100KN
B _ LOAD = 200KN

in the more compressed steel. At higher loads however, the difference in strains between the reinforcements increase considerably. This shows that the distribution of load among the reinforcing bars depend on the level of loading. Designers are however interested in the collapse point. Brant 1984; Mander set al 1988.

CHAPTER 7

Concrete Confinement And Failure Patter Of Test Specimens

7.1 General

Current designs of reinforced concrete columns assume that ties prevent the buckling of the reinforcements (Ahmad 1982; Behan 1982 . Researchers have found that no matter the closeness of the ties in the column, the column fails by the buckling of the reinforcements (Chiwuzie 1975; Ferguson 1984; Sheikh 1980). Ties therefore, have come to be accepted rather as assisting in the strength enhancement through concrete confinement. Another dimension was added in investigating confinement in this research, by studying the confinement of tie spacing and that by the new type of composite columns (Taylor et al 1983).

7.2 Specimen Failure Patterns

There were four basic failure patterns recorded in the tests as shown in fig 7.1.

(a) The crushing of the concrete and buckling of the steel reinforcement or steel angle sections of the new type of composite columns. This is described as a compressive unbalanced failure of a column section (Bennett 1973).

(b) The shearing of the conrete section without the buckling of the steel similar to the failure of an unreinforced conrete section thus suggesting lower utilization of reinforcement or steel angle section (Husseini 1984, Matock et al 1984; Heger et al 1982).

(c) The collapse of the concrete cover with the core intact. This indicates high concrete confinement but overall load reduction of the column section due to the failure of the cover.

(d) The failure of the column section with approximately uniform vertical crack. This indicates uniform strain at failure along the length of the specimen.

The major observation in the research is that the failure patterns for both the reinforced concrete columns and for the new type of compsite coilumns were similar. These are related to the confinement of conret by tie spacing and by concrete steel area ratio for the new type of columns as described by Taylor (1983). When the tie spacing is less than the depth of the core section, patterns (a) and (c) are the likely failures, while (b)and (d) are likely for spacing more than the depth of the core sections. This is in agreement with earlier researchs. Ali 1987, Chiwuzie 1975, Sheikh

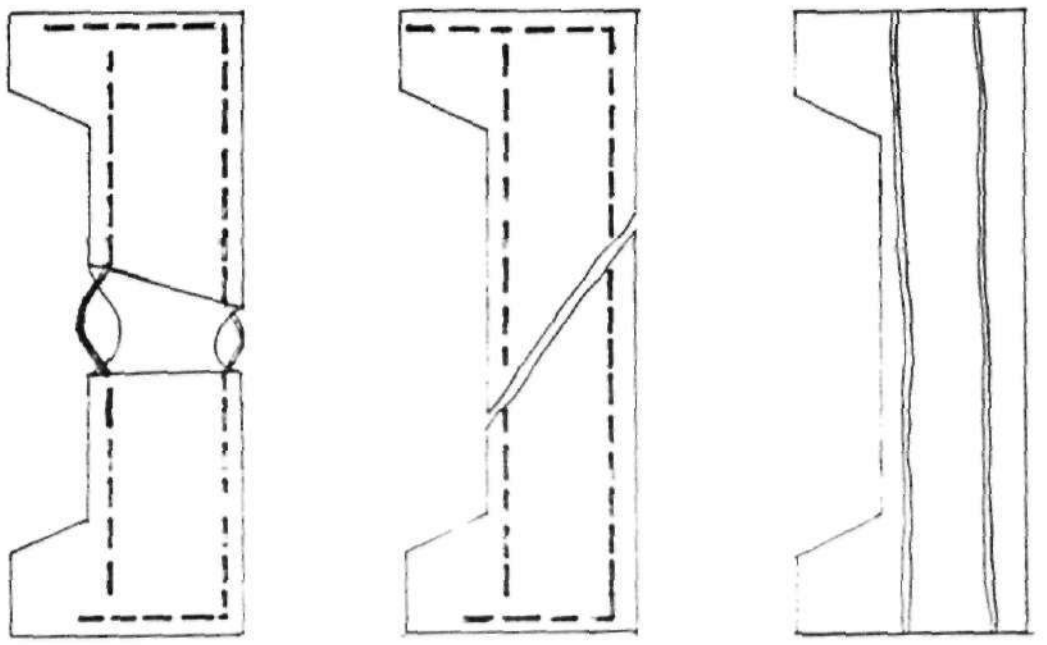
7.3. Stresses In Ties And Main Reinforcement

Table 7.1. presents the stresses in the ties and main reinforcements at various load levels and for two tie spacings.

Table 7.1 Stresses in tie and main reinforcement

Tie Spacing (mm)	Load (KN)	Tie-Stress n/mm^2	Stress in Main Reinf (n/mm^2)
100	50	4	4
	100	8	8
	450	10	122
	700	45	214
250	50	2	2
	100	4	4
	200	8	50
	250	12	72
	700	35	205

The results suggest that at a closer spacing higher stress is induced in the tie. This implies that lower tie spacing contributes more to load resistance. Table 7.2. shows the stresses in the steel section of the composite columns at various load levels and for different concrete-steel area ratios.



D and E are new type of composite columns.

Fig. 7.1 RECORDED FAILURE PATTERNS.

Table 7.2. Stresses in Steel at Different load levels for New Type of Columns

Ac/As	Steel Stress At Various Loads (n/mm^2)			
	50kn	100kn	150kn	300
13	54	78	94	-
19	10	40	100	-
26	15	20	32	84
36	92	102	112	142
44	13	35	49	92
55	8	16	27	58
*11	9	36	72	-
*15	14	21	37	67
*26	8	16	27	46

Table 7.3. Load Carried by Concrete at Various Load Levels

Ac/As	Load Carried by Concrete (KN)				
	Load Level - 50KN	100KN	150KN	300KN	1/b
13	11	44	82	-	
19	43	71	78	-	
26	41	86	127	240	
36	-	27	69	198	
44	41	75	115	234	
55	44	88	131	258	
*11	39	57	64	-	
*15	33	75	106	220	
*26	40	78	118	245	

*Encased Sections

Some tendencies which were not clearly indicated by graphical relations were observed as the behaviour of new type of concrete columns in respect of confinement.

(a) For the encased sections, increasing the area ratio of concrete and steel which is also reducing the effective height/depth ratio.

(b) The relation between steel utilization and concrete-steel area ratio appears to be a tripple polynomial function.

(c) For the specimens tested, there appears to be a value which is the interaction point between slenderness and concrete confinement, at which steel is optimally utilized.

7.4 Confinement and Failure Loads

The effect of tie spacing on the failure loads of reinforced concrete specimens is presented in table 7.5 for specimen group A, and fig 7.3.

Table 7.4: Effect of Tie Spacing on Failure Load

Eccentricy (mm)	Sv/hc	Sv/h	$Ne:N^T$
0	0.5	0.33	14%
0	1.0	0.67	98%
0	1.5	1.00	82%
0	2.0	1.33	97%
0	2.5	2.67	96.6
25	0.5	0.33	118%
25	1.0	0.67	102%
25	1.5	1.00	87%
25	2.0	1.33	78%
25	2.5	2.67	87%

It will be observed that for specimens with tie spacing more than the concrete core, the experimental failure loads recorded were less than the theoretical

value. The effect of tie spacing on the strength of concrete short columns, from the results above been taken into account by most researchers (Ali 1987; Chiwuzie 1978; Sheikh 1982). It was suggested that for a short column to attain its maximum theoretical strength, the tie spacing should be about 26 percent of the concrete core depth for axially loaded columns and 34 percent for columns with small eccentricity (Ali 1987). Fig 7.6 is the relation between the ultimate failure load of new type of columns and concrete confinement measured in terms of the concrete at about an area ratio of 36, the steel is optimally utilized as well as maximum concrete confinement attained. The low experimental-theoretical failure load ratios obtained is ignored.

Table 7.5 Theoretical-Experimental Failure Load Ratios for Various Ac/As Ratios

Ac/Ac	Ne/N	Ne/Ny
13	54%	68%
19	58%	72%
26	66%	94%
36	79%	98%
44	69%	86%
55	46%	57%

because it has been observed that composite columns capable of attaining their squash loads are too short for any practical purpose. Taylor et al 1983; Yusuph 1987. The strain values recorded at failure accompanied by buckling of reinforcements were lower than the predictable values using the suggested method by Papia et all (1988, 1989). It was observed that for the same tie spacing, the buckling propensity of the longitudinal reinforcements increase with increasing eccentricity of loading. The eccentricity of loading was not considered as affecting the buckling strains of the reinforcements by Papia et al (1988, 1989). Fig 5.1 shows that the strains of longitudinal reinforcements for a given tie spacing depend very much on the eccentricity. The suggested equations for estimating the critical strains at buckling, of main column reinforcements appear too complex for the behaviour of reinforced concrete columns under small eccentricity. It has been suggested that the exact criteria of failure of concrete is not known, but there are strong indications that failure occurs at a limiting strian of 0.002 - 0.004 in compression and

Fig. 7·2

LOAD RESPONSE OF TIES

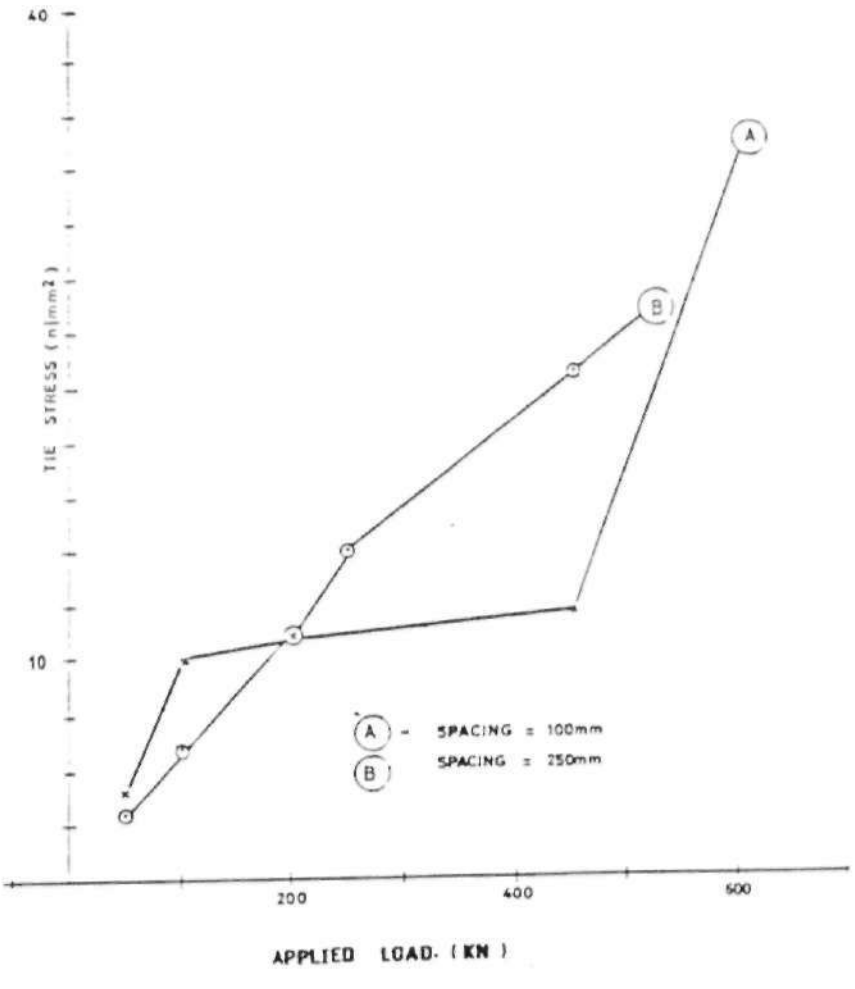


Fig. 7·3

LOAD RATIO VARIATION
WITH TIE SPACING

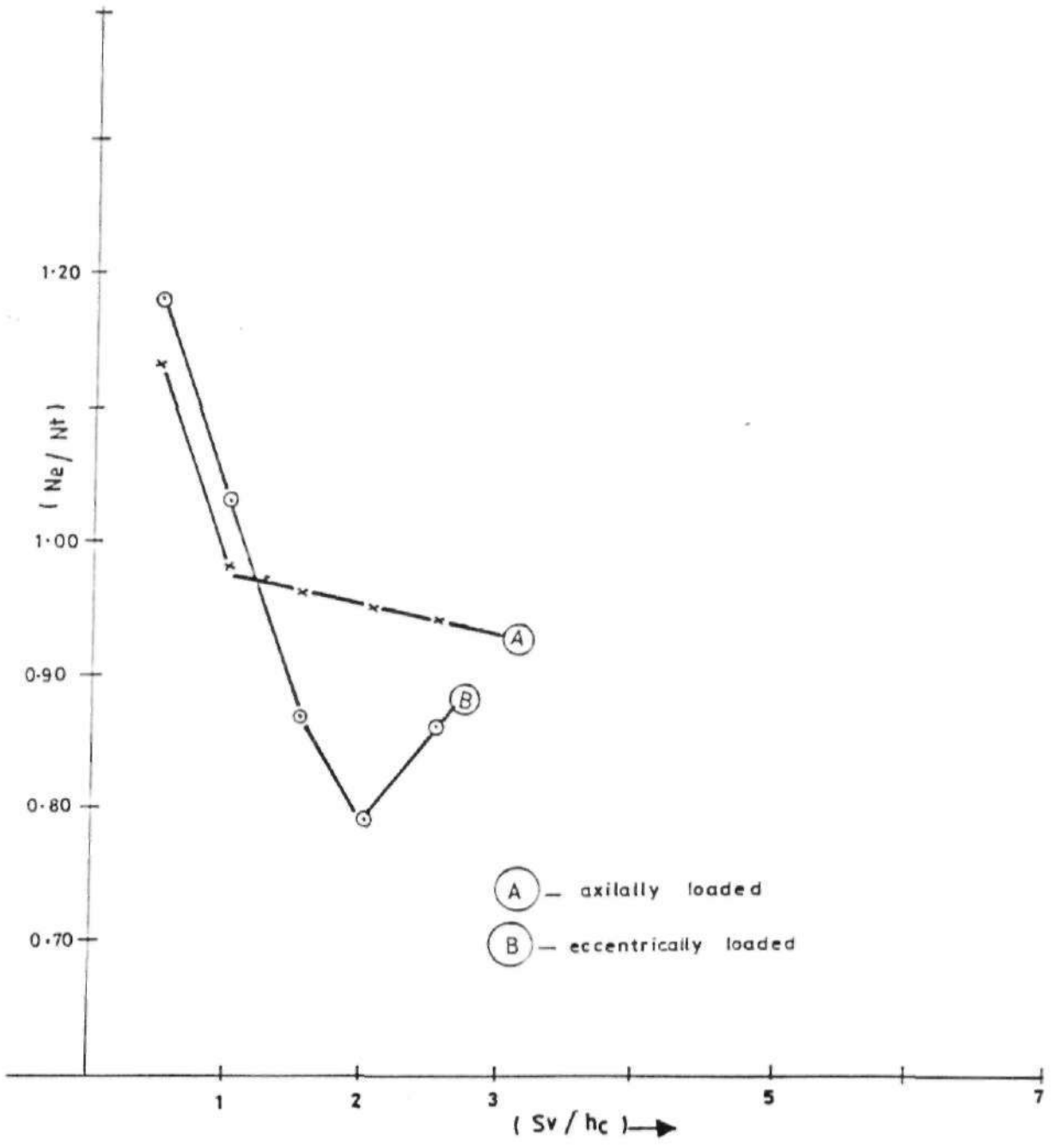


Fig 7.4
VARIATION OF CONFINING
PRESSURE WITH APPLIED LOAD

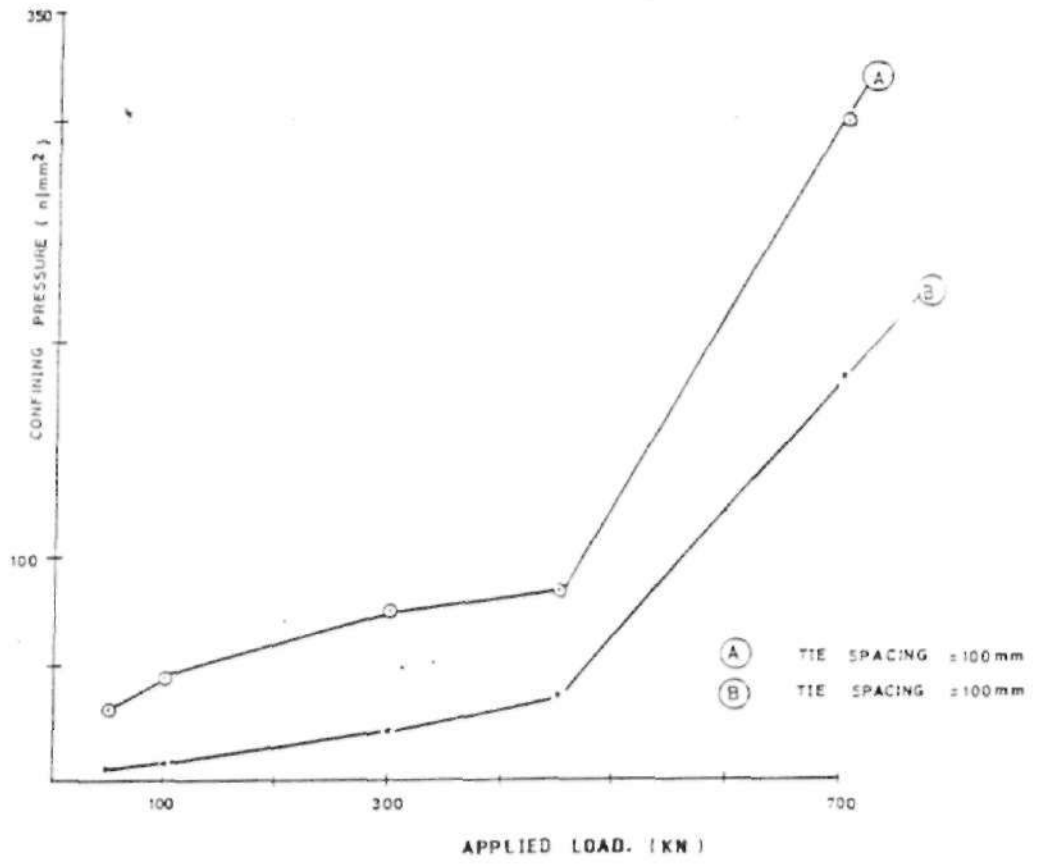


Fig 7.5

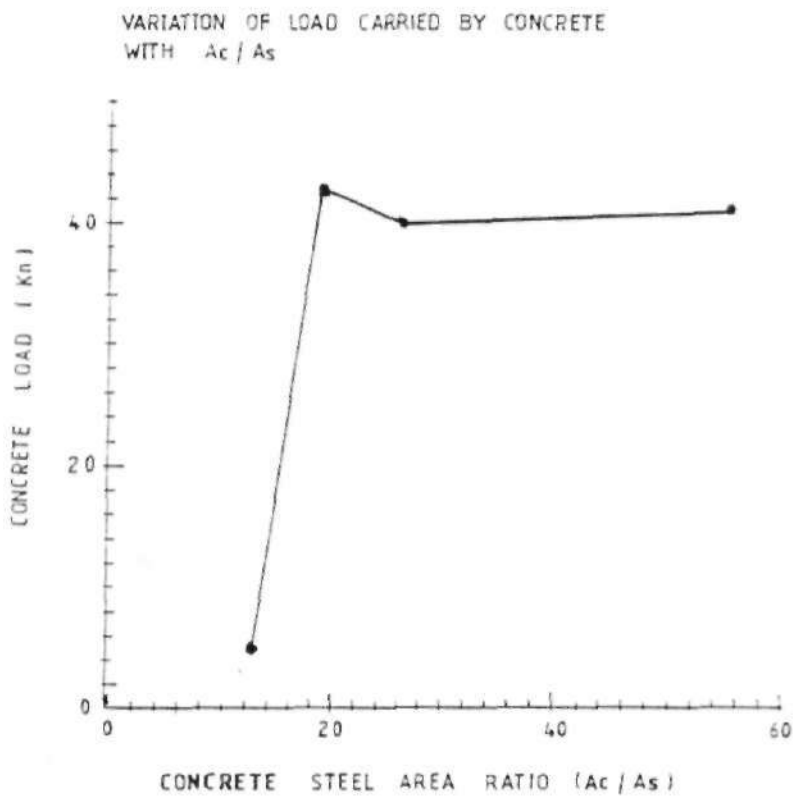
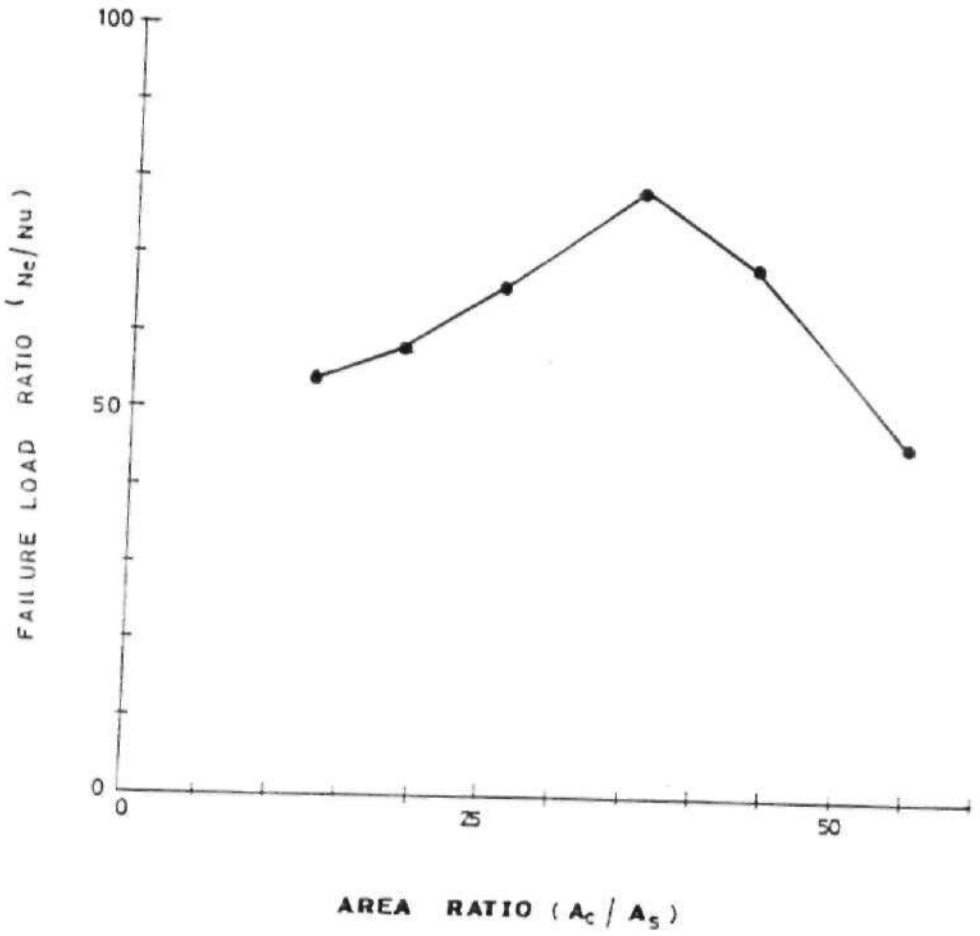

VARIATION OF LOAD CARRIED BY CONCRETE
WITH Ac / As
CONCRETE LOAD (Kn)
40
20
0
0
20
40
60
CONCRETE STEEL AREA RATIO (Ac / As)

Fig 7.6

VARIATION OF FAILURE LOAD WITH AREA RATIO



0.001 - 0.002 in tension Nandivaran et al 1988; Priestly et al 1981). Previous research by Umaru 1989 indicate that for the 19 specimens tested, for zero eccentricity, the relation

$$Ne/Nt = 0.775 - 0.0608 \frac{Sv}{h}$$ may be suggested.

Summary

Concrete confinement were studied in the reinforced concrete columns by varying the tie spacing while for the new type of columns, the confinement was studied by varying the concrete-steel area ratio. The results suggest that the failure patterns of reinforced concrete columns and the new type of columns were similar and were related to the confinement of concrete in the coloumn. Some other conclusions can be made from the text.

(1) The stress in steel in both reinforced concrete columns and new type of composite columns, depend on the concrete confinement. Steel utilization therefore depends on concrete confinement.

(2) The work of ties in a reinforced concrete column may not be to prevent the buckling of reinforcement, but to confine the concrete. Ties

KASHIM IBRAHIM LIBRARY

KASHIM I... RECEIVED ... 1992 ZARIA, NIGERIA Ahmadu Bello University

aid in axial load resistance. There is therefore the need to review the specifications on tie spacing to incoporate it's contribution to axial load resistance.

(3) The contribution of ties to axial load resistance depend on the eccentricity of loading.

(4) For columns with tie spacing more than the concrete core, the concrete strain at failure may be less than 0.2 percent.

(5) There exists an optimum concrete-steel area ratio at which stability and confinement are optimally utilized in the new type of composite columns. This ratio is about 36.

(6) For large concrete-steel area ratios, the present design equations for the new type of composite columns may not be suitable.

CHAPTER 8

MODEL FRAME TEST

8.1 General

The relation between eccentricity and reinforcement unsymmetry was studied in Chapter 6 using the results of the model columns. The importance of this relation had been emphasized in that chapter. However, it will be stated here that one of the inferences of that chapter is that reinforcement unsymmetry may achieve both reduced cost and increased reliability of a given column. Therefore the objective of the model frame test was to confirm the results of the model elements. A concrete model frame as shown in fig 8.1 was cast in the laboratory with four of the columns reinforced differently using 6mm mild steel. The reinforcements were cut-off at each floor level in order to approximate a simply supported joint of the next floor. Also in order to approximate a uniaxial situation, the secondary beams were made shorter. The loads were placed between the primary beams. The frame reprsents a heavily loaded structure such as a warehouse. The strains were measured using the digitial strain meter. Concrete strains were measured only in

No. 1 column. The concrete strength at test was $37.5N/mm^2$.

8.2 Transfer of Stresses With Age

Before the sixth day after casting of the frame, the strains in the reinforcements could not be recorded due to the water within the columns which short-circuits the strain gauges. Table 8.1 and fig 8.4 present the strain variation of the reinforcements with the age of concrete.

Table 8.1: Strain Variation With Age of concrete Under a Given Load. Strains in Reinforcement

Age	Less Compressed	More Compressed
6 days	-2530	-925
7 "	-2351	-750
8 "	-2280	-500
9 "	-2190	-420
10 "	-2065	-300
11 "	-1950	- 50
12 "	-2000	-120
13 "	-1896	-428
14 "	-1590	-720

The results indicate that initially both reinforcements were in tension and as the concrete

acquires strength there is gradual redistribution of the stresses such that (the system is similar to change of eccentricity) there is increased compression in both reinforcements which tend to reduce the stress on the less (tensile) compressed steel and increase the stress in the more compressed steel. It should be observed that under this load which represents the weight of the columns and overlying beams and slabs, the less compressed steel has exceeded the design ultimate strain. Papia et al 1989. One remarkable point here is the fact that both reinforcements are under tension. The stress transfer agrees with the results presented in chapter 6. In respect of the variation of reinforcement strains with eccentrcity. This transfer of stresses is not attributable to creep but to maturity of the concrete. Chen 1980; Priestly et al 1983.

Table 8.2: Differential Strains in Reinforcements Due to Increased Load

Load(n)	Reduction in Eccentricity	Es1	Es1	Ecc	Ect	Es2	Es4
407n	29mm	-563	837	195	+86	-100	-1780
492"	48 "	-552	850	212	-100	- 94	-1796
565"	60 "	-535	865	140	-101	- 92	-1842
638"	70 "	-476	865	140	-103	- 85	-1850
828"	86 "	-114	886	78	-126	- 73	- 417
959"	94 "	-108	956	54	-140	- 64	- 413
1163"	101 "	-291	959	135	132	49	- 450
1283"	105 "	-270	1000	153	163	40	- 336
1346"	107 "	-	1000	114	167	38	- 290
1625"	113 "	-130	805	8	28	-12	+ 806
1890"	117 "	-146	820	21	50	115	973
2121"	119 "	-144	841	13	25	80	950
2342"	121 "	-140	806	13	82	96	963
2574"	123 "	-754	814	128	100	102	946
2765"	127 "	-594	720	720	265	60	930
3007"	114 "	+ 2	775	51	207	65	1010
3238"	108 "	- 4	839	78	120	56	1072
3354"	107 "	- 9	773	145	127	64	956

The results indicate that column no. 4 shows the highest strains for a given load level. This implies that it will likely be the first to fail if the frame is loaded to failure. This is remarkable because column No. 4 as shown in fig 8.1 has the highest amount of reinforcement. This result confirms the fact that higher reinforcement ratio does not automatically imply greater safety in a column. Reinforcement arrangement is as important as the reinforcements. Another remarkable point is that the strain distribution across

the column Cross Section as measured in column No. 1 does not follow the simplified elastic theory of bending as would be expected . There does not appear to be a simple relation between the positions of the reinforcements and the strains in concrete and steel. This implies as was earlier suggested in chapters 2 and 5 that the elastic theories of bending may not apply to concrete columns under small eccentricity of loading.

Figs 8.2 and 8.3 show the variation of the recorded strains with reduction of eccentricity, in the less compressed and more compressed steel respectively. The diagrams are gquite similar tc the typical load response of reinforcement at critical eccentricity as shown in fig 5.7.

As the eccentricity of the load is reduced, the load in the less compressed steel reduces while that in the more compressed steel increases. These results are quite in agreement with the results of the test replica model columns presented earlier in Chapters 4 - 6.

8.3 Summary

The results of a model test reinforced concrete frame are briefly presented in this chapter. The results indicate that in a loaded concrete frame, the

stress distribution among reinforcements and concrete for the different columns, subjected to equal loading differ. This difference is attributable to the arrangement of reinforcements within the column. The results also indicate that higher amount of reinforcement does not necessarily imply higher safety factor of a concrete column. At load levels less than one-tenth the load capacity of the columns, the recorded strains are high compared to values expected based on current theories. This indicates that for columns under small eccentricity of loading, the currently assumed failure strains may be conservative and unrealistic of the situation.

CHAPTER 9

EFFECT OF SLENDERNESS ON REINFORCED CONCRETE COLUMNS

9.1 General

A total of seven columns with height-breadth ratios more than 12 were tested to failure. These columns were regarded as slender columns as specified by CP110. The observed failure loads were compared with failure loads obtained using the proposed new method. The additional moments were estimated using the specifications by CP110 (1972).

9.2 Additional Eccentricity

The approximate design approach for slender columns is the moment magnifier method. The moment magnification factor as given in American codes is (Bernitsas et al 1980; BSI 1972 Gary 1980.

$$\frac{C_m}{1 - \frac{Nu}{PNc}} \geqslant 1$$

where Cm is end effect factor taken as equal to 0.6 - 0.4 (M_1/M_2) $\geqslant$ 0.4 for columns braced against side sways.

M_1 is the smaller of the ultimate moments at the ends of the column found in the first order analysis,

positive if member is bent in single curvature, negative if bent in double curvature M_2 - is the larger of the ultimate moments at the ends of the column found in the first order analysis and is always positive

Nu is the ultimate load on column

is reduction factor

Nc is elastic critical Euler buckling load.

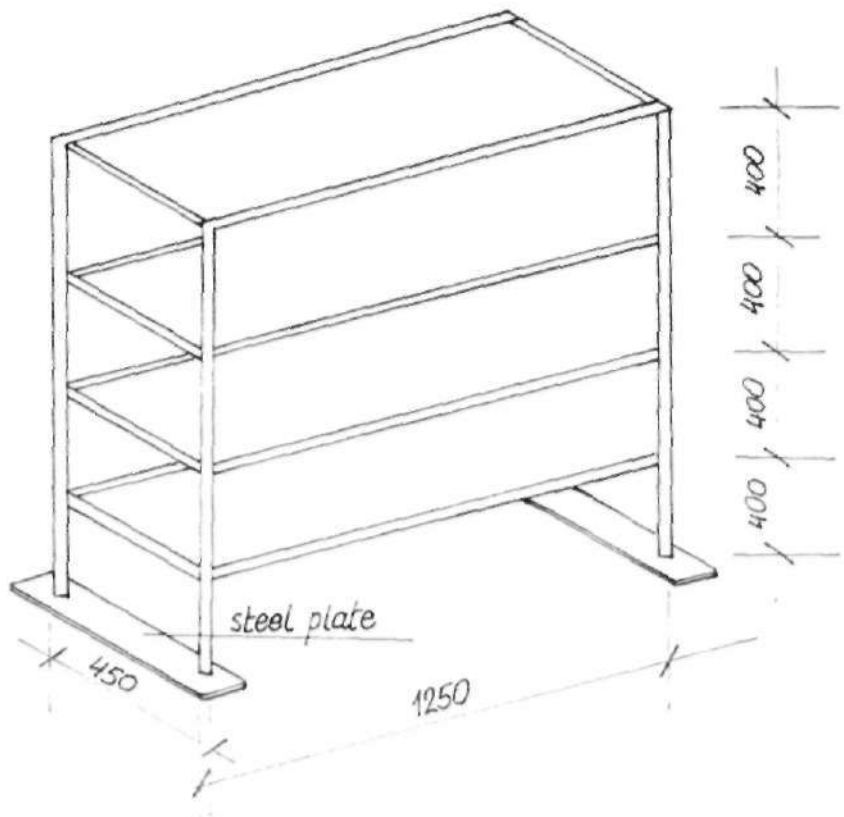
The determination of the additional moment in a slender column is complicated. CP110 specifies a simple method as given below.

$$Madd = \frac{Nh}{1750} \left(\frac{le}{h}\right)^2 \left(1 - 0.0035 \frac{le}{h}\right)$$

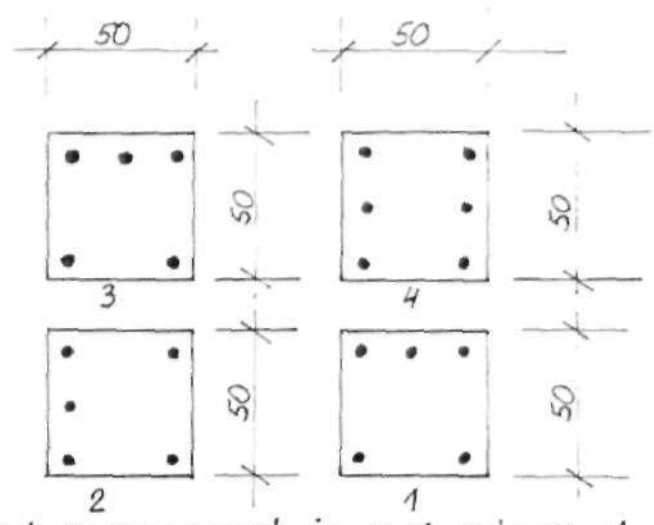
where Madd is the additional moment and le is the effective height of column. In the tests carried out, the effective height of the columns was assumed to be equal to their respective heights. Timoshenko (1961) on the basis of elastic analysis suggested the additional eccentricity as

$$Eadd = \frac{4}{\pi} eo \left(1 - \frac{1}{Ne/Nc}\right)$$

where Nc is the Euler critical buckling load given as $\pi^2 Ec\ Ic/l^2$, Ne is the applied load, and L is the height of the column. It is not convenient to use



A. Isometric view of R-C frame



B. Reinforcement arrangement in each column 1-4

Fig. 8.1 FOUR STOREY MODEL TEST R-C FRAME

Fig 8.2

STRAIN VARIATION IN LESS COMPRESSED STEEL DUE TO REDUCTION IN ECCENTRICITY (TEST FRAME)

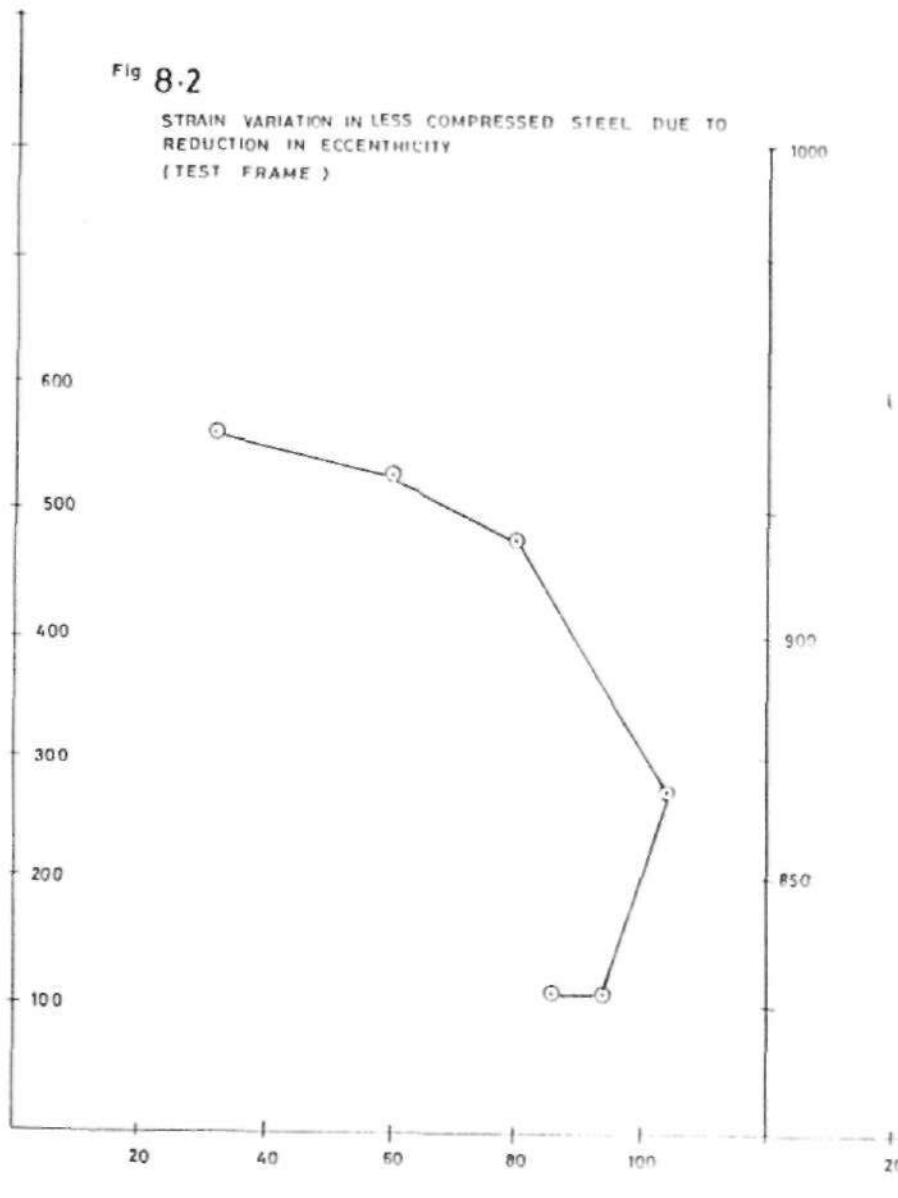


Fig 8.3

STRAIN VARIATION IN MORE COMPRESSED STEEL DUE TO REDUCTION IN ECCENTRICITY (TEST FRAME)

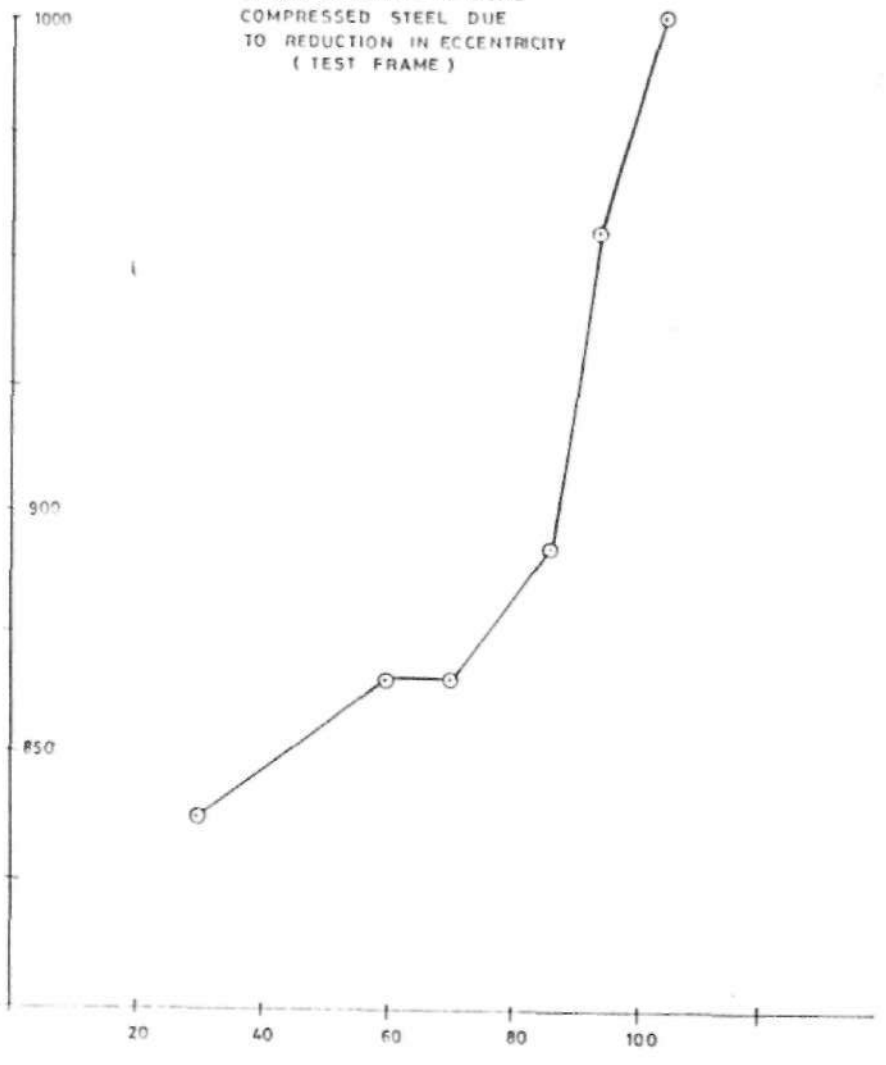
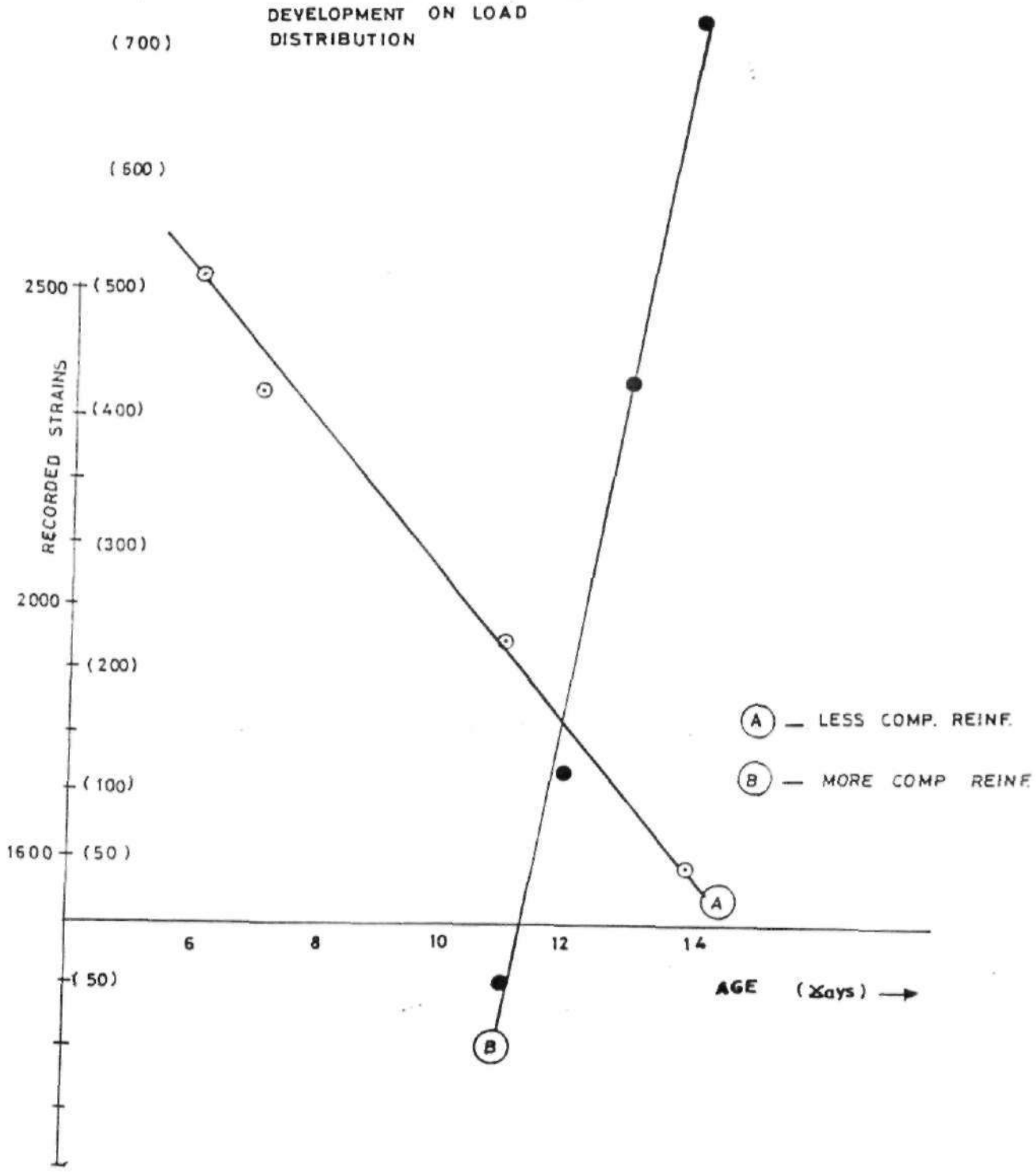


Fig 8·4

EFFECT OF CONCRETE STRENGTH DEVELOPMENT ON LOAD DISTRIBUTION



the Timoshenho suggestions to estimate the theoretical failure load of the columns tested. Table 9.1 is a summary of the seven slender columns tested to flailure. The results indicate close agreement between theory and observations. The suggested method of estimating the additional eccentricity of a slender column by the CP110 method appears to be very convenient and fairly accurate. Therefore in the design of concrete columns under small eccentricity of loading, the additional eccentricity should be estimated

Table 9.1: Test Results of Slender Columns

Specimen	eo	l/h	Ne	Nm	Ne/Nr
C1	0	14	244	255	0.96
C2	10	14	202	199	1.02
C3	15	14	186	180	1.03
C4	20	14	190	163	1.17
C5	25	14	210	150	1.40
H3	50	15	90	77	1.17
H4	50	15	105	84	1.14
				Average	1.14

using the CP110 method

The difference in response of reinforcements at a

given load level between slender and short columns of group H specimens is presented in graphical form in figs 5.6. and 5.10 It shows an almost linear relation with the applied laod. This suggests that for a given column section, the additional eccentricity is a constant, and this agrees with CP110. Hag 1989; Robert et al 1983.



Plate 6 the Model Frameunder load

CHAPTER 10

BIAXIAL LOADING AND DESIGN

10.1 General

A brief review of the current practice and research on biaxially loaded columns is provided in chapter 2.2. It will be recalled that most current researches on concrete columns are either on finite element method as a precise method of estimating biaxial load capacity of columns, or the design of biaxially loaded concrete columns. Inspite of all the work done, biaxial loading continues to be complicated. In this chapter, the proposed design method for biaxially loaded columns is presented as well as method of Analysis.

10.2 Design of biaxially loaded Columns with Small Eccentricities

The proposed design method is based on the extension of the assumptions and principles of the uniaxial loading design and analysis as presented in chapter 5. The proposed design method may be given in the following steps.

Step 1

Determine the eccentricities ex and ey where ex is more than ey, and acting as to produce a torque in the yy - axis.

Step 2

The design axial load is resolved into two equivalent uniaxial loads Nx and Ny in the ratio of the eccentricities.

$$Nx = \frac{(ex}{(ex + ey)} \quad (N) = k \times N.$$

Step 3

Determine the strength of concrete in both axis

fcx = kx fc

fcy = ky fc.

Determine also, the stress ratios φx and φy

$$\varphi x = \frac{fc}{fcx}; \quad \varphi y = \frac{fc}{fcy}$$

Step 4

Using the proposed design chart for uniaxial loading determine reinforcements with respect to the two axes xx and yy for load combinations

(Nx, Mx) and Ny, My)

Step 5

Place the reinforcements as suggested by the chart.

10.3 Comparison of Design Methods

For the given set of design parameters below, current methods of design are compared with the

proposed design method in terms of the reinforcement requirement only. Design parameters are same with example 10.1.

Method	Total Reinforcement Required
1. Proposed Method	5,550mm^2
2. Super position CP110	16,950mm^2
3. Weber, Park, Paulay	12,126mm^2
4. Modified Proposed Method	8,550mm^2

The modification above is achieved by arbitrarily changing (increasing) the amount of reinforcement in order to achieve ratios of Mx/Mux and My/Muy as defined by CP110, to be less than unity thus satisfying the specification by the code. The results show that the proposed method of design reduces the reinforcement requirement, and hence cost of columns biaxially loaded with small eccentricities. The higher reinforcement in current methods is due mainly to the magnification of the error of approximation of the biaxial problem for small eccentricities.

10.4 Estimating The Biaxial Load Capacity of a column

The current methods of design of biaxially-load

columns have been reviewed in chapter 2.7. Although these methods result in higher reinforcement requirement for columns with small eccentricities, they do not lend themselves easily to capacity estimation. This chapter provides a simple method of capacity - estimation for reinforced concrete columns with small eccentircity. The method is an extension of the principles and assumptions of the uniaxial method as produced in chapters 4. Tests on replica models as presented in chaopters 4 - 8 have shown that the assumption of the minimum moment capacity of a column being equal to the moment capacity is in agreement with the results. The reinforcement unsymmetry of the column is also proportional to the eccentricity of loading. The proposed method of load estimation is presented in steps as follows.

Step 1

Input all the design parameters of the column for which capacity is to be estimated. The eccentricities of load in the two principal directions xy should be within the concrete section. If they are not, use current methods.

Step 2

. The materials' strengths are divided into two, in the ratio of the eccentricities ex and ey

$$fcx = kx\ fc$$
$$fcy = ky\ fc$$
$$fsx = kx\ fs \quad \text{..... 11.4.1}$$
$$fsy = ky\ fs$$

where

$$Kx = \frac{ex}{ex + ey};\quad Ky = 1 - Kx \text{ ..... 11.4.2}$$

Step 3

Consider each axis xx and yy. From the equations of the uniaxial load capacity and taking moments about the less-compressed steel, the capacity is estimated from the following equations.

$$Fcx = Ac\ fcx$$
$$Fsx = As\ fsx \quad \text{..... 11.5.3}$$
$$fsx = as\ fx$$

$$Mx = Fcx\left(\frac{nx}{2} - d^1x\right) + F^1sx(dx - d^1x) +$$

$$fsx\ (dx - hx/2) \text{ ..... 8.4.4}$$

$$Nx = Mx/Cex + \frac{hx}{2} - d^1x) \text{ ..... 11.5.5}$$

$$Fcy = Ac\ fcy$$

$$Fsy = As\ fsy \quad \text{..... 11.5.6}$$

$$Fsy = As\ fsy$$

$$My = fcy\left(\frac{hy}{2} - d^{1}y\right) + F^{1}sy(dy-d^{1}y) + Fsy\left(dy - \frac{hy}{2}\right) \quad \text{..... 11.4.7}$$

$$Ny = My/\left(ey - \frac{hy}{2} - d^{1}y\right) \quad \text{..... 11.4.8}$$

$$N = Nx + Ny \quad \text{..... 11.4.9}$$

It should be noted that Nx produces a toque parallel to the xx axis while acting along yy axis. Also Ny acts along xx axis. When considering YY axis to determine N_y, Asx = A1 and As = A3 - A4. When considering YY axis to determine Ny, Asy = A3 and As = A^1 - A^2. It is also assumed that A^1 A2 and A3 A4.

10.5 Experimental Varification

Five replica model columns with parameters reported in table 3.1. where tested to failure biaxially. The biaxial loading condition was achieved by using an "ell-shaped" roller device. The roller device transfered the load from the crushing machine

to the column specimen at given eccentricities. The columns were loaded to failure and the maximum observed loads recorded as the failure load. The recorded failure loads of the columns are given in table 10.1.

Table 10.1: Comparison of Observed Failure Load to Theoretical* Failure Load of biaxially Loaded Concrete Columns

Specimen	Failure Load (Ne)	Theoretical* Load (Nt)	Ne/Nt
H51	276KN	284KN	0.97
H52	280KN	"	0.99
H53	260KN	"	0.92
H54	281KN	"	0.99
H55	283KN	"	1.00
		Average	0.97

The average observed/theoretical failure load ratio for the specimens is 0.97. This indicate high accuracy of the theoretical load estimation method.

Example 10.1

To design using the proposed design method, the reinforced concrete column given the following design

parameters.

(1) Applied axial load = 3016KN

(2) Applied moments Mx = 199.26KNM; My = 90KNM

(3) b = 300mm; h = 500mm; $d^1 = d_2$ = 25mm.

(4) fcu = 25; fy = 410

Solution

Step 1 $ex = \frac{199.26x106}{3016x10^3} = 66.07mm = 66mm$

$ey = \frac{90 \times 10^6}{3016x10^3} = 29.84mm \quad 30mm$

Step 2/3 $Nx = (3016)\frac{(66)}{(96)} = 2074KN$

$Ny = (3016)\frac{(30)}{(96)} = 942KN$

$fcx = \frac{(66)}{(97)}(25)(0.45) = 7.734\ n/mm^2$

fcy = (30)

$$\frac{(25)(0.45)}{(96)} = 3.516 \text{ n/mm}^2$$

$$\varphi_x = \frac{(0.72 \times 410)}{7.734} = 38.17$$

$$\varphi_y = \frac{(0.72 \times 410)}{3.516} = 83.96$$

Step 4 $$\hat{\alpha}_x = \frac{(2074 \times 10^3}{300 \times 500 \times 7.734} - 1 = 0.7878$$

$$\hat{\alpha}_x \quad \begin{matrix} -- \\ -- \\ -- \end{matrix} \quad 0.0206 \varphi_x$$

$$\hat{\beta}_x = \left(\frac{(199.26 \times 10^6)}{(300 \times 500^2 \times 7.734)}\right) / \left(0.5 - \frac{25}{500}\right)$$

$$= 0.763$$

$$\hat{\beta}_x \quad \begin{matrix} -- \\ -- \\ -- \end{matrix} \quad = 0.019995 \varphi_x \quad 0.02 \varphi_x$$

Reinforcements for xx axis are (from chart)

As = $3075 mm^2$

As = $75 mm^2$

$$\hat{\alpha}_y = \left(\frac{(942 \times 10^3)}{(300 \times 500 \times 3.516)}\right) - 1 = 0.78612$$

$$\hat{\alpha}_y \quad \begin{matrix} -- \\ -- \\ -- \end{matrix} \quad 0.0094 \varphi_y$$

$$\hat{\beta}_y = \left(\frac{(90 \times 10^6)}{(500 \times 300^2 \times 3.516)}\right) / \left(0.5 - \frac{25}{300}\right) = 1.3652$$

From the chart, for - = 0.094 and B = 0.01634 the reinforcements for yy axis are.

Acy = 1950mm^2

Asy = 450mm^2

Total reinforcement requirement = 5550mm^2

It will be observed that the above solution does not satisfy the simplified check provided by CP110. If the reinforcements are arbitralily increased by about fifty-five percent. It now satisfies CP110 provision, still lower than current design requirements by more than twenty-nine to fifty percent. It must however be noted that the provision by CP110 is for simplification. The basis for such simplification has not been evaluated with respect to the proposed design method and may not apply for columns with small biaxial moments.

Example 10.2:

To estimate the biaxial load capacity of a reinforced concrete column with the following parameters. Bresler 1960. b = h = 381mm. Reinforcements - 4y22 at corners. fy = 345n/mm^2, fcu = 25n/mm^2. ex = ey = 102mm. $d^1 = d_2$ = 62mm.

<u>Solution</u>

fsx = fsy = (1)

$$(-)\ (0.86x345) = 148.35n/mm^2$$
$$(2)$$

$$fcx = fcy = (1)$$
$$(-)\ (0.67x25) = 8.37n/mm^2$$
$$(2)$$

$$Fcx = (381x381-1552)(8.375) \times 10_3 = 1202.73KN$$

$$Fsx = 776x148.35x10_3 = 115.10KN$$

$$Mx = My = (1202.73)(190-62)+(115.10x256) = 0.1835KNM$$

$$Nx = Ny = \frac{Mx}{(102+190-62)} = \frac{Mx}{230} = 798KN$$

$$\therefore N = Nx + Ny = 798 \times 2 = 1595KN$$

10.6 Summary

The chapter presents the propcsed methods of design as well as load capacity estimation of biaxially loaded columns under small eccentricity. Also presented is the results of biaxial load test on five replica model columns. Attempt is made to show that the proposed design method and proposed capacity estimation method are simple and efficient for this class of columns. The test results indicate very good agreement be.ween theory and experimental observation.

CHAPTER 11

Some Structural Failure

11.1 General

As was earlier stated in chapter one, a brief review of some structural failures was supposed to serve as real life results for this research but unfortunately due to the present attitude of the various governments and estate developers, all documents relating to collapsed structures are usually quickly destroyed to evade prosecution. This chapter therefore is limited to a descriptive essay. The main objective of this chapter therefore is to highlight the importance of collapse as a source of knowledge and also as a specialist field of civil engineering, just as Morbid Anatomy is in Medicine. Nine cases are briefly reviewed in the text. The erection of building in modern Nigeria undergoes a number of processess through the Architect Engineer and relevant government ministry. Because buldings now symbolise prestige and status, high rise buildings have become a commonplace. The increase in collapse is related to this tendency. All the reviewed failures are buildings.

11.2. When a Building Collapses

A building is designed by an architect and engineer and constructed by a contractor who should

be an engineer (or builder) depending on the complexity. Therefore the failure (collapse may imply extreme case) of a building may be as a result of any or combination of the following.

(a) Wrong designs due either to professional incompetence or negligence.

(b) Poor or wrong construction due either to bad supervision by the resident or consulting engineer or incompetence on the apart of the contractor.

(c) Natural cause.

(d) Non use of the relevant professionals, by the client.

The reviewed cases will relete to the above.

11.3. Iponri Collaspe Western Avenue Lagos

The building collapsed when the fourth floor was being cast. From the drawings found under debris, columns were placed from the first floor on contilever beams (NSE Report 1985). The concrete was of poor quality. The house was said to have been designed by an engineer. The columns at the ground floor were 255mm square with 4R20 reinforcements,

while the subsequent floors were reinforced with 4R16. The ties were of diameter 10mm and at 255mm between centres. The maximum span of the slabs was 5 meters. At the time of collapse the walls had not been erected. 6mm diameter ties at half the spacing provided would have enhanced tremondously, the strength of the columns. The observed concrete strength was 10N/mm^2. The eccentricity that will be produced by the dead weight of the slab and beams on the column will be about 46mm. Therefore the load capacity of such column will be about 487kN which is about the load that will be produced by dead weight of concrete alone in the ground floor columns at the completion of the fourth floor. Without any settlement analysis, it is likely the collapse of the building was due to the overloading of the provided columns. The same reason will be attributed to the collapse of Ayilara-Ojuelegba and PortHarcourt private school building. These structures were previously in use until the owners increased the number of floors without adequate reinforcement of columns (NISE report 1985). These are typical cases of Professional negligence in the Iponri case, and non-use of the engineer in the other two cases.

11.4. Bauchi State Government Schools

GGSS Tafawa Balewa was sited at a place previously used for rice cultivation. The drawings show wrong reinforcement of the combined foundation footing. In this case, settlement should have been a more important factor than bearing capacity consideration. With some differential settlement, wide cracks appeared at the walls emphasizing the need for expansion/settlement joint as required by CP110 (1972). GSS Cham is sited on black cotton soil and the wide cracks on ordinary ground was indicative of foundation problem for the buildings in addition to vee-cracks on the walls. At GSS Futuk, wrong design of the cantilever beams carrying the balcony led to its failure and the subsequent collapse of the balcony. The collapse of the buildings at GSS Doma was quite interesting. Every wall including surface sewer-manholes cracked excepting walls carrying water tanks. The explanation is that there was some form of vibration leaving all the sandcrate walls cracked except those carrying water tanks which served as dampening weight (force). This postulation is reinforced by the fact that Doma is close to Pindiga where a minor earth movement had been

reported. This may represent a typical case of natural disaster. The collapse at Barnawa Kaduna, may also be attributed to settlement.

Summary

The cases reviewed indicate a general pattern of failure of reinforced concrete structures attributable to respective causes. Sudden collapses are associated with overloading of the columns while vee and inclined cracks on walls indicate foundation problem.

Collapses of most of the government buildings were as a result of professional negligence of the engineering consultants while those of the private estate owners revolve around non-use of the engineer.

CHAPTER 12

DESIGN OF UNSYMMETRICAL NEW TYPE OF COMPOSITE COLUMNS

12.1 General

A composite steel-concrete column is a member with cross section consisting of steel section (or sections) and concrete which act together to resist axial compression and moments. Previously composite sections were designed as steel sections with protective weak concrete covering it (Liu 1984; Pham 1984). Greater understanding of the behaviour of these columns led to the modification of the design method from the approach of individual materials acting on their own to the approach of combined or composite action of both materials.

For a short composite column the squash load is the load at which the concrete crushes and the steel yields. It was suggested that any composite column capable of attaining its squash load will be too short for any practical use (Virdi et al 1982). The other limiting load capacity of a composite column is the Euler's load which is the load capacity of a slender column. For a given steel or composite section, the squash load and Euler's load can therefore be used to obtain the first approximation

for the buckling curve which is a relation between the maximum load capacity and the slenderness ratio (Timoshenko et al 1961). The current design method of composite columns under axial load and bending entails mainly estimating if a given section will fail under axial load and moment. Thus two composite sections with different cost implications may satisfy the same problem (Chen 1982; Paolo 1990). This of course indicates some sort of approximation in the design of composite columns. In addition, the moment-thrust-curvature relation assumed in the designs as earlier pointed out, may not be suitable for the design of columns under small eccentricity of loading (Chen 1982). All current design charts for composite steel-concrete columns assume symmetrical arrangement of sections (Weber 1966), which may not be economical for optimal material utilization. Some tests were recently carried out on what was described as new type of composite columns (Taylor et al 1983). The columns were made of channel sections filled with concrete. The composite columns described in this text are similar. These are angle sections filled with concrete. For high rise buildings, the columns are normally loaded with heavy axial loads and

comparatively low moments. In addition, the height-breadth ratio's of these columns are usually low, decreasing as the axial loads increase. Unsymmetrical new type of composite columns therefore will be suitable for high rise structures for which columns have small eccentricity.

This chapter therefore presents a proposed design method which is the same method earlier proposed in chapter 5 but extended to composite columns. The design philosophy is that for high-rise structures, the height-breadth ratios of the columns are usually low. Therefore such columns may be designed to attain a reasonable proportion of their squash loads. The definition of short columns remain as in CP110 (1972).

12.2 Some Tests On New Type of columns

Taylor et al (1983) carried out a number of tests on the new type of composite columns. The applied loads were at various eccentricities of between 0 and 50mm. The average ratio of the observed load/theoretical load was 0.84 with a coefficient of variation of 21 percent. It was suggested that for such columns the design load Ny is equal to the squash load multiplied by a reduction factor.

Fig 7.6 shows the variation of failure load with the concrete steel area ratio. The results of the tests earlier discussed in chapter 8 indicate that for axially loaded composite columns, at concrete - steel area ratio of 36 the column attains the maximum load capacity. At this point the concrete height-breadth ratio and the concrete confinement are optimally utilized. Fig 7.5 also indicates that concrete contribution to load resistance appears to be at a constant maximum for concrete-steel area ratios of twenty and above. From these two results, it is therefore suggested that for an unsymmetrically reinforced new type of composite columns, the concrete-steel area ratio should be between 20 and 36. Also the effect of eccentricity on reinforcement load response as discussed in chapter 6 indicates that unsymmetrical arrangement of reinforcements (and steel sections, for composite columns) may achieve not only reduced cost, but also enhanced reliability of the column. Therefore, unsymmetrical arrangement of section should be seen not only in terms of cost but also of safety.

13.3 Design of Uniaxially Loaded Column

The design procedure is the same as suggested in chapter 6 for reinforced concrete columns. The only difference is that, instead of choosing bars, having determined the reinforcement ratio, angle sections are chosen. In addition, a final check has to be carried out to ensure that the concrete-steel area ratio is between 20 and 36.

Example 1

To design an unsymmetrically arranged new type of composite column for load combination of N = 3000KN; M = 101.25KNM given that the concrete 28th day strength is 22n/mm^2 and steel yield stress is 208n/mm^2. Height of column = 3.5m.

Solution

The procedure is taken in steps.

1. Choose a column section ensuring that the height-breadth ratio is less than 12. Try 500 x 400mm concrete section.
2. Determine the design dimensionless quantities.

   fc = 0.45x22 = 10

   fs = 0.72x208 = 150

$\therefore \varphi = 150/10 = 15$

$$\therefore \quad \alpha = \frac{3000\text{x}10^3}{400\text{x}500\text{x}10} = 1.5$$

$$\therefore \quad \hat{\alpha} = 0.5 = 0.03\varphi$$

assume $d_1 = 0.05h$

$$\beta = \frac{101.25\text{x}10_6}{400\text{x}500^2\text{x}10} = 0.10125$$

$$\therefore \quad \hat{\beta} = 0.10125/0.45 = 0.225 = 0.015\varphi$$

3. Determine reinforcement requirement from chart

$$\therefore \quad As = 2.4\%(bh) = 4800mm^2$$

$$As = 0.9\%(bh) = 1800mm^2$$

4. Provide/Arrange chosen sections.

   Provide two angles; 150x75x12 at more compressed side.

   Provide two angles; 75x50x8 at less compressed side.

5. Check against concrete-steel area ratio provided; 33 is provided in this case. O.K.

If the same design is to be made using the suggestion by Taylor (1783); that is assuming the eccentricity small awnd applying a reduction factor. The reduction factor by Taylor et al (1983) may be approximated as follows.

$$R = 0.8 - 3.3+92 \begin{matrix}(e)\\(-)\\(h)\end{matrix} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 13.1$$

Therefore for an eccentricity of 33.75mm and depth of 500mm, the reduction factor R is about 0.572.

$\therefore$ $300x10^3 = (Acx\ 10 + As\ x\ 150)R.$

$\therefore$ As $21,632mm^2$

Observe that this is about three times the solution above. The alternative method is rather to "check" the load capacity of a chosen section (Alnoury et al 1982; Chen 1982; Virdi et al 1982).

12.4 Design of Biaxially Loaded Column

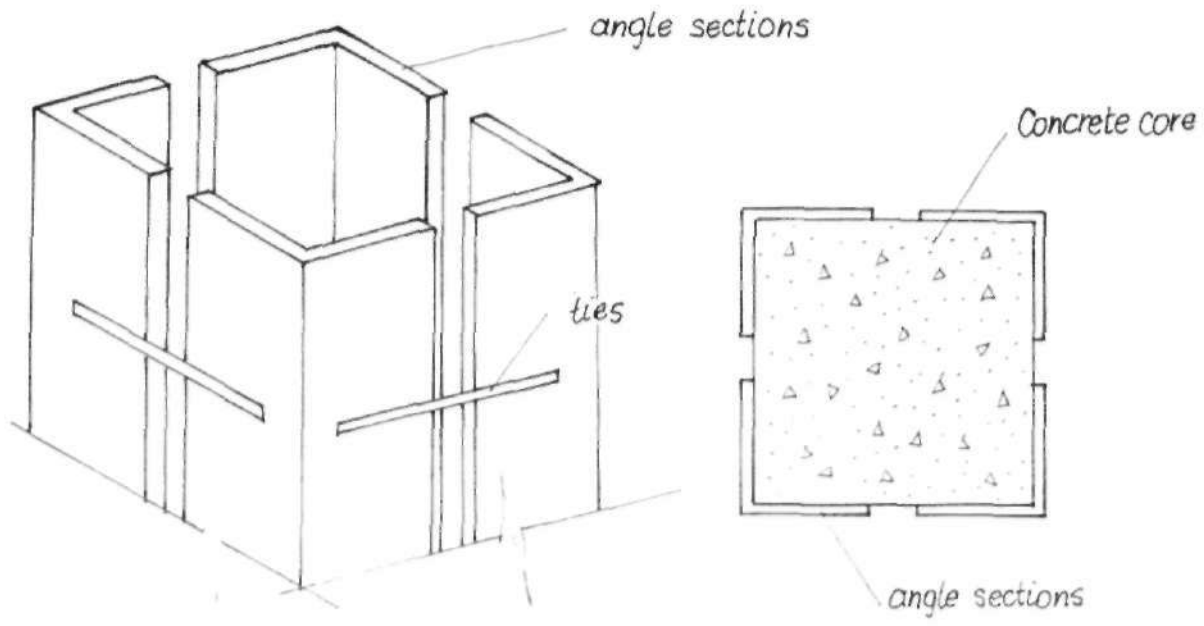
The design is as proposed in chapter 10. The loads and material design strengths are proportioned with respect to the two load axes, in the ratio of the eccentricities in those axes. Thereafter, the design using the chart is carried out in the same manner as in uniaxial loading and finally superimposing the steel areas determined.

Example 2

Tod design a composite column 3.5m high given the following parameters

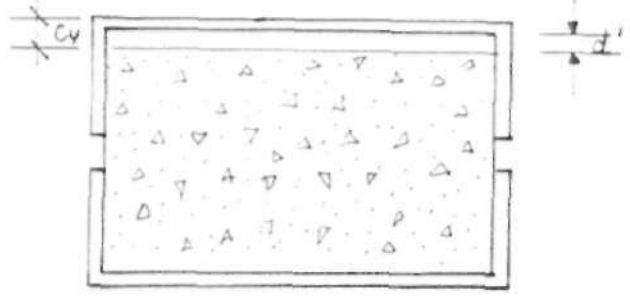
(a) Applied load N = 8000kN

(b) Applied moments Mx = My = 100kNm



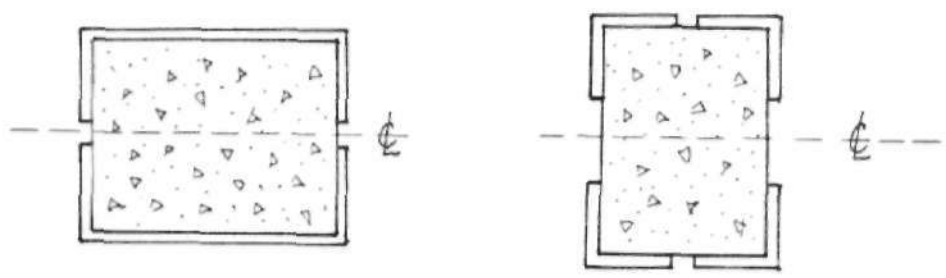
A. Isometric view
(assembly of angle sections)

B. Cross section



C. Alternative cross section
using chanels.

Fig. 12.1 SKETCH OF (SYMETRICAL) NEW TYPE OF COMPOSITE COLUMNS.



A. using channel sections

B. using angle sections

Fig. 12.2 UNSYMETRICAL NEW TYPE OF COMPOSITE COLUMNS.

(c) $f_{cu} = 44N/mm^2$; $f_y = 416N/mm^2$

Solution

Choose a concrete section 500 x 500mm

$f_c = 0.45 \times 44 = 20N/mm^2$

$f_s = 0.72 \times 416 = 300N/mm^2$

$e_x = e_y = 100x10^6/8000x10^3 = 12.5mm$

$\therefore$ $K_x = K_y = 12.5/25 = 0.5$

$$f_{cx} = f_{cy} = \begin{matrix}(1)\\(-)\\(2)\end{matrix}(20) = 10n/mm^2$$

$$f_{sx} = \begin{matrix}(1)\\(-)\\(2)\end{matrix}(300) = 150n/mm^2$$

$\varphi x = 150/10 = 15$

$$N_x = \begin{matrix}(1)\\(-)\\(2)\end{matrix}(8000) = 4000KN$$

$\therefore$ $\alpha_x = 4000x10^3/500x500x10 = 1.6$

assume $d^1x = 0.005hx$

$$\beta_x = \left(\frac{100x10^6}{500x500^2x10}\right)/0.05 = 0.296$$

converting the dimentionless applied load and moment to φ- values

$\acute{\alpha}_x = 0.6/15 = 0.041\ \varphi$

$\acute{\beta}x = \frac{0.296}{15} = 0.2\varphi x$

from the chart

Asx = 3%(bh) = $7500mm^2$

Asx = 1%(bh) = 2500mm^2

The procedure is repeated with respect to the other axis, and the resulting areas of steel superimposled. In this case since all the parameters are equal in the two axes, the steel requirement will be

same. Therefore the resulting arrangement of the four angle sections will be such that their respective areas are

(i) 3750+3750 - Provide 200x200x20le (7630mm^2)

(ii) 3750+1250 - Provide 150x150x18le (5100mm^2)

(iii) 3750+1250 - Provide 150x150x18le (5100mm^2)

(iv) 1250+1250 - Provide 100x100x15le (2790mm^2)

The arrangement above means that the first angle section combines $\frac{A^1sx}{2}$ and $\frac{A^1sy}{2}$ while

the last combines $\frac{Asx}{2}$ and $\frac{Asy}{2}$

It will be observed that the depth d^1 is slightly greater than the assumed value (25mm). This may be ignored since the steel also increases the sectional dimension of the column; this increase should off-set any additional moment.

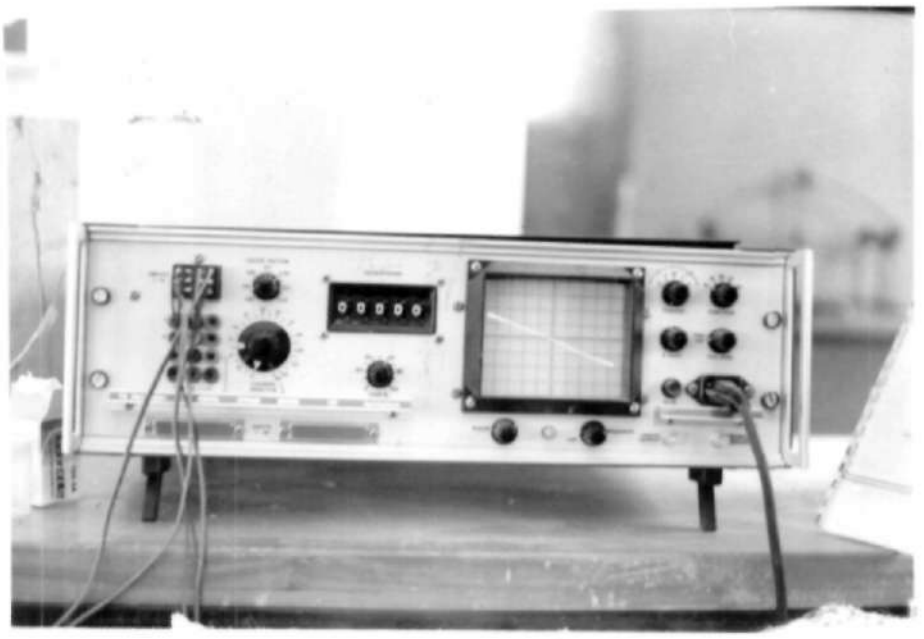


PLATE 2 DIGITAL STRAIN METER

Table 12.1: Test Results of New Type of Composite Columns

Specimen	fcu	Ne	Nu	Ny	Ne/Ny	Ne/Nu
CC1	38N/mm^2	212 kN	391 kN	313 kN	0.54	0.68
CC2	"	290KN	503"	402 "	0.58	0.72
CC3	"	478 "	634"	507 "	0.75	0.94
CC4	"	620 "	788"	630 "	0.79	0.98
CC5	"	663 "	961"	769 "	0.68	0.36
CC6	"	526 "	1155"	924 "	0.46	0.57
CC7	"	517 "	726"	581 "	0.71	0.89
CC8	"	648 "	1052"	842 "	0.62	0.77
CC9	"	292 "	594"	475 "	0.49	0.62
				Average	0.78	

Table 12.2: Reduction Factors for New Type of Composite Columns

Eccentricity e(mm)	0	8	10	12	25	30	40	50
Reduction Factor	0.08	0.62	0.68	0.73	0.58	0.48	0.34	0.32

CHAPTER 13

Conclusions And Recommendations

13.1 Conclusions

From the analysis of existing literature and test results presented in this text, a number of conclusions are made. These conclusions are made mainly for better understanding and design of reinforced concrete columns under loads of small eccentricity. The effect of the spacing or the contribution of ties to axial load resistance has not been included in the various deductions and conclusions.

13.1.1 Present Design Methods And Charts

The present assumption of elastic moment thrust-curvature relation in the design of reinforced concrete columns does not apply to columns under small eccentricity of load which have infinite depth of neutral axis. The code CP110 design charts for columns apply the same principles to the design of both columns with small eccentricity as well as columns with large eccentricity; 0.0Zh - Zh. (25) and this is not economical.

13.1.2 Current Research Trends On Concrete Columns

Most current researchers on reinforced concrete columns assume adequate knowledge of the uniaxial load response and behaviour of concrete columns.

This is erroneous for columns of small eccentricity which may not suitably and economically be referred to as either axially loaded systems or flexural systems.

13.1.3 Small or Large Eccentricity in Column Designs

When the eccentricity of load is less than or equal to half the depth of concrete section, this eccentricity should be assumed small, or otherwise, when more than half the depth of column section.

13.1.4 Load Capacity Estimation by Oleszkiewicz

The capacity estimation of a concrete column under small eccentricity as suggested by Oleszkiewicz over estimates the section when the effect of concrete confinement by ties is not taken into consideration. Also the assumption by Oleszkiewicz that for columns with small load eccentricity, the concrete cover has no effect on the strength of the column is not suitable for this class of concrete columns. The position of the reinforcement, hence the cover, affects the strength of this class of columns.

13.1.5 Concrete Confinement in Columns

The contribution of ties to axial load resistance in a concrete column depends on the eccentricity of loading. The various suggested mathematical equations for the increment of concrete strength due to ties spacing need to incorporate the effect of eccentricity. The

various suggestions therefore cannot be said to be adequate. The function of ties in a concrete columns is not to prevent the buckling of the reinforcements as is currently believed, but to increase the concrete confinement. The maximum failure strain of concrete depends on the tie spacing. The effectiveness or utilization of reinforcement in short columns under loads of small eccentricity depends on the concrete confinement by ties.

13.1.6 Reinforced Concrete Columns in the Tropics

The performance of concrete has been closely related to the strength. The strength of concrete in the tropics has been observed to differ from similar concrete in the temperate regions. The long term load transfer between concrete and steel and the time-dependent variation of the modular ratio, is different as is currently assumed.

13.1.7 Eccentricity and Reinforcement Unsymmetry

For concrete columns under small eccentricity of loading, there is a linear relation between eccentricity of loading and the unsymmetrical reinforcement requirement measured in terms of unsymmetrical load response of reinforcements.

At a given eccentricity of loading, there is an optimal reinforcement unsymmetry at which maximum column capacity is achieved.

13.1.8 Proposed Design/Estimation Method

The proposed biaxial and biaxial design methods of concrete columns are simpler and cheaper than current methods for the design of columns under small eccentricities of loading. With a hundred percent agreement between theory and observed test results for the over seventy columns, the assumptions and principles of the proposed design and capacity estimation methods will reduce the cost of reinforcement in a concrete column.

13.1.9 Unsymmetrical Reinforcement

In terms of reinforcement requirement only, it is not economical to design any concrete column under eccentric load as symmetrically reinforced. The proposed design chart is an asset to design engineers, as an alternative to current design charts which assume symmetrical reinforcement, or arrangement of steel sections in the new type of composite columns.

13.1.10 Moment Capacity

For optimum utilization of concrete in a reinforced concrete column under small eccentricity of load. The moment capacity is an intrinsic property of the section, depending on the reinforcement unsymmetry.

13.1.11 Reinforcement Utilization

In an unsymmetrically reinforced concrete short column under small eccentricity of loading, reinforcement utilization in load resistance depends on the eccentricity of loading. For a given concrete grade and reinforcement unsymmetry, there exists an eccentricity value at which the less compressed steel is least utilized in load resistance. Thus for this class of columns, an increase in the eccentricity of loading (moment) does not necessarily imply a reduction in the expected failure load when concrete grade and reinforcement arrangement remain constant.

13.1.12 The Dimensionless Load Capacity of Specimens

For the sixty-six specimens tested, the average dimensionless load capacity recorded was 1.17. The assumption that the minimum dimensionless load capacity of concrete columns under small eccentricity of loading as unity is justified by the results.

13.1.13 Change of Reinforcement Stresses With Time

From the moment when the concrete column formwork is removed, the stresses in the less compressed steel

decrease with increase in strength of concrete, while the stresses in the more compressed steel increase. This corresponds to the change in stresses with change in eccentricity.

13.1.14 Position of Higher Reinforcement Area

In an unsymmetrically reinforced concrete column, the position of higher reinforcement area depend on the eccentricity of loading. For small eccentricities the more compressed steel is more in area while the less compressed steel is more for large eccentricities.

13.2 Recommendations

13.2.1

Most researchers on concrete confinement worked on axially loaded sections. There is the need to study this subject with respect to eccentricity, as concrete confinement was observed to depend on eccentricity.

13.2.2

In the design of reinforced concrete columns under small eccentricity of loading, the cross-section should be rectangular instead of square. Rectangular section with respect to the applied-moment-direction optimize the use of concrete.

13.2.3
The assumption of 0.002 as the design failure strain in reinforced concrete columns under small eccentricity of loading may be conservative. There is need to investigate the failure strain in columns.
13.2.4
Composite columns under small eccentricity of loading should be designed as unsymmetrically arranged section using the suggested design method.
13.2.5
Following the proposal by Oleszkiewicz as illustrated in fig 2.0. It may be possible to design prestressed beams (or axially loaded flexural systems) with a concrete contribution of unity. Research should therefore be carried out in this respect.

11. Stress Ratio - The ratio of design steel strength and design concrete strength.

12. Ties - Lateral reinforcement.

APPENDIX IV

GROUP E TEST RECORD

Load	KN	Recorded Strains at Load Levels 0 50	60	70	80	90	100	200	Specimen
e = 0	MC	0 100	108	124	165	186	211		E5
	LC	0 228	218	200	177	173	156		
e = 10	MC	0 -	244	253	315	350	370		E5
	LC	0 -	201	194	170	152	165		
e = 20	MC	0 266	255	288	333	382	431		E5
	LC	0 171	179	187	199	202	191		
e = 30	MC	0 358	368	410	478	-	-		E5
	LC	0 240	242	262	271	-	-		
e = 10	MC	0 10	-	40	-	-	50	100	E5
	LC	0 160	-	140	-	-	150	130	
e = 15	MC	0 270	-	360	-	-	480	770	E5
	LC	0 10	-	40	-	-	90	190	

APPENDIX I

SPECIMENS-TEST RECORD

Recorded Strains

Specimen		Balance	50KN	100KN	150KN	300KN		Load at Crushing
B11	MCR	12685	12700	12650	12610	12400		360KN
	LCR	12035	11840	11710	11655	11300		
B12	MCR	3050	2960	2885	2800	2479	3350	450KN
	LCR	630	595	500	400	150	360	
B13	MCR	4460	4390	4340	4250	4100	4000	502KN
	LCR	3930	3855	3760	3640	3300	3000	
B21	MCR	9160	13970	14200	14030	14100		250KN·
	LCR	-	-	-	-	-	-	
B14	MCR	2920	2850	2710	2570	2210		387KN
	LCR	3860	3725	3550	3400	2770		
B22	MCR	-	-	-	-	-	-	
	LCR	2250	2170	2110	2250	2125	1190	534KN
B15	MCR	4320	4160	4075	4040	3740		327KN
	LCR	-	-	-	-	-	-	
B23	MCR	-	-	-	-	-	-	778KN
	LCR	3690	3550	3445	3345	3110	-	
B16	MCR	3050	2960	2885	2800	2470	3350	550KN
	LCR	1	2960					
CC1		4000	3730	3610	3530			250KN
CC8		10,000	9810	9745	9550	9350		646KN
CC9		3460	3415	3230	3100			292KN
CC3		3120	3045	3000	2960	2870		418KN
CC7		1035	965	930	850	700	580	517KN
CC6		540	405	500	410	310	230	526KN

appendix I Cont.

A24	6015	5820	5730	5560	5320	5020	788KN
A25	1920	1870	1845	1800	1650	1440	502KN
A22	5430	5330	5200	5110	4765	4000	584KN
CC3	3470	3345	3210	3070	2690	2395	498KN
CC3	2950	2830	2770	2670	2350	2040	678KN

LCR - Less Compresses Steel.

* MCR - More Compressed Steel;

APPENDWIII

FRAME TEST RECORD

• N(n)	(KNM)	fe (MM)	N(n)	IT	1C	1 4T	3T	ICC	1CT
0	0	—	—	9870	5070	—	—	—	—
61	' 90	149		9850	5085	-	-	-	-
97	14.4	149		9848	5095	-	-	-	-
63	9.3	149		7380	4946	-	-	-	-
167	26	159		.7330	4981	-	-	-	-
364	26	73		J7270	J5020	3850	-	-	-
425	26	61		7120	5070	-	-	-	-
260	37	142			5660	8575	17435	, -	-
322	45	140		.9550	j3350	J24320	J6820J	' 7610	8240
407	45	111	i 85n	8987	4187	22540'.	6720	7415	.8326
492		92	, S5n	J899S	4200	, 22524.	:6726]	7398	.8140
565		80	i 73 J	9015 I	4215	22478.	6728.	• 7470	.8139
638		71	i 73 .	9074]	4215	22470.	6735	7470	.8137
828		55	,190	9436 .	4236	23903.	6747.	7532	.8114
959		47	,131 .	9442 •	4306	239 07,	675 6	7556	.3100
1163		39	(204 •	9259 :	4309	23870	6771.	7475	,8108
1283		35	35	9280 J	4350	23934.	6860	7457	.8077
1346		34	.63	-	4350	24 03 0r J	6858.	7496	.8073
1625		28	979	9420 \	4155	15126.	6808.	7618	3212
1890		24	265	9404	4170 ;	15193,	6935.	7631	8190
2121		21	231	9406 '	4191	151-70,	6900	7597	3215
2342		19	221 .	9410	4156	151S3[1],	6 916'	7597	8158
2574		17	232	8796 .	4164	1516 5,	6922.	7482	8140
2765	37	± J	191	8956	4070	15150'.	6880	7340.	7975
3007	o —	2 /	242		4125 .	15220.	6885 :	7559'	8033
3238	106		231	9546	4189	152S2-	6876.	7532.	8120
3254	106	32	116	9541	4123	15276j	6884	7465.	8113
				9536 \	4123 J	15268',	6886.	7472	8130

APPENDIX V

TEST RESULTS OF GROUP H FOR ECCENTRICITY OF 50MM

Recorded Strain (Microstrain)*

Load	MCR	Concrete (Comp)	LC	Concrete (ten)
10KN	146	735	-30	-70
20	315	1185	-95	-220
30	471	1070	-165	-650
40	663	1215	-265	-730
50	864	-	-470	-2020
60	1151	-	-710	-2775
70	1283	-	-825	-3625
80	1518	-	-1205	-4650
90	2167	-	-1610	-

APPENDIX VI

TEST RESULTS OF GROUP H FOR ECCENTRICITY OF 40MM

Recorded Strains (Microstrains)*

Load	MCR	Concrete (Comp)	LCR	Concrete (ten)
10KN	-	90	10	-30
20KN	-	190	20	-30
30KN	-	300	60	-45
40KN	-	390	80	-125
50KN	-	575	40	-90
60KN	-	740	50	-115
70KN	-	910	50	-160
80KN	-	1090	-20	-185
90KN	-	1415	-120	-200
Lonad release	-	210	-170	-30

*Minus sign indicates tension

MCR means, more compressed reinforcement.

LCR means less compressed reinforcement.

APPENDIX VIV

TEST RESULTS OF GROUP H SLENDER SPECIMENS

Load	Strain MCR	LCR
10KN	135	-35
20KN	260	-170
30KN	400	-310
40KN	565	-480
50KN	735	-600
60KN	915	-785
70KN	1100	-945
80KN	1335	-1470
90KN	1370	-1835

APPENDIX IV FRAME TEST RECORD II

N	ΔN	e (mm)	E1	E2	E1	E2	E3	E4
407	85	111	-883	-883	-563	837	-1780	-100
492	85	92	-872	-870	-552	850	-1796	- 94
565		80	-855	-855	-535	865	-1842	- 92
638		71	-796	-855	-476	865	-1850	- 85
828		55	-434	-834	-114	886	- 417	- 73
959		47	-428	-764	-108	956	- 413	- 64
1163		39	-611	-761	-291	959	- 450	- 49
1283		35	-590	-720	-270	1000	- 336	+ 40
1346		34	-	-720	-	1000	- 290	+ 38
1625		28	-450	-915	-130	805	+ 806	- 12
1890		24	-466	-900	-146	820	+ 973	+115
2121		21	-464	-879	-144	841	+ 950	+ 80
2342		19	-460	-914	-140	806	+ 963	+ 96

2574		17	-1074	-906	-754	814	+ 946	+102
2765		13	- 914	-1000	-594	720	+ 930	+ 60
3007		27	- 318	- 945	+ 2	775	+1010	+ 65
3238		33	- 324	- 881	- 4	839	+1072	+ 56
3354		32	- 329	-947	- 9	773	+ 956	+ 64
-		-	- 334	-947	- 14	773	+ 948	+ 66
61		149	- 20	+ 15				
63		149	- 490	-124				
97		149	- 22	+ 25				
167		159	-2540	- 89				
364		73	-2600	- 50				
435		61	-2750	0				
260		142	-1598	+ 590				
322		140	-320	-1720				

REFERENCES

1. Abatan A (1986) "Computer Aided Design of Reinforced Concrete Columns" Proc. AMSE Conf. Nigeria. Vol C P150-163 AMSE France.

2. Ali M J (1987) "The Effect of the Spacing on the Strength and Ductility of Concrete Short Columns" B. Eng. Thesis, ATBU.

3. Astill A W and Martin L H (1975) Reinforced Concrete Structures. John Wiley Pub. 272pp

4. Armer G S T and Garas F.K (1982) Design for Dynamic Loading Construction Press London. 362pp.

5. Allaire Paul E (1985) Basics of Finite Element Method Brown. Pub. Iowa

6. Aktan A E and Bertero V V (91984) "Seismic Response of Reinforced Concrete Frame Wall Structures" Journ. Struct Engry vol 110 No. 8 ASLE PP 1803-1821

7. Abrams Daniel P (1982) "Small Scale Modelling of Reinforced Concrete Structures to Resist Earthquake Motions" Design for Dynamic Loading PP86-92 Const Press.

8. Adepegba D. (1989) "Reinforced and Prestressed Concrete Design for Students and Engineers" Unpublished Unilag.

9. Avram, Facaoaru, Mirsu, Filimon and Testa (1981) Concrete Strength and Strains. Elsevier Pub. Amsterdam 558pp.

10. Al-Noury S and Chen W F (1982) "Finite Segment Method for Biaxially loaded R.C. Columns" Journ. Struct. d.v. ASCE Vol 108 No ST4 pp780-799.

11. Ahmad S H and Shah S P (91982) "Stress-Strain Curves of Concrete Confined by Spiral reinforcement" Journ. ACI Vol 79 No 6 PP484-490.

12. Brown Colin B (1988) "Errors in Structural Engineering" Journ. Struct Engrg ASCE Vol 114 No 11 PP2575-2593.

13. Bernitsas MM and Kokkinis T (1983) "Buckling of Columns with Non Movable Boundaries" Journ. Struct. Engrg ASCE Vol 109 No. 9 PP 2113-2128.

14. Brondium-Nielsen Troels (1987) "Concrete Sections under biaxial bending" Journ Struct Engrg Vol 113 No 10 ASCE pp 2137-2144.

15. Bungey J H (1982) The Testing of Concrete in Structures Surrey Univ. Press London 207 pp

16. Behan J E and O'connor C (1982) "Creep buckling of reinforced concrete columns" ASCE Struct div vol 108 ST 12 pp 2799-2818.

17. Baikov V and Sigalov E (1978) Reinforced Concrete Structure Vol 1 MIR Moscow.

18. Bennett E W (1973) Structural Concrete Elements Chapman and hall London. 314 pp.

19. Barr, Howe, Neilson, Carter, Nachtshein, Riecn and Rudiger (1982) "An experimental investigation of scalling of reinforced concrete structures under impact loading" Proc Int. Seminar ISE London pp220-233.

20. Brant N F A (1984) "Reinforced Concrete Columns of variable cross section. Ph.D thesis city UK.

21. Balendra T and Tay C H (1984) "Dynamic Instability of Braced columns" Journ Struct (India) vol 4 No. 1 pp 13-26.

22. Breysse Dennis and Mazars Jacky (1988) "Simplified Approach of non-linearity in R.C. Beams" Journ. Struct. Engrg ASCE Vol 114 No 2 pp 251-268.

23. Basu A.K. (1984) "Charts for Seismic Design of Framewall Systems" Journ. Struct Engrg ASCE Vol 110 No. 1 pp 31-46.

24. Bresler B ((1960) "Design Criteria for Reinforced Concrete Columns Under Axial Load And Biaxial Bending" Journ. ACI Vol 57 pp 481-490.

25. British Standard Institution (1972) "CP110 The Structural use of Concrete"

26. Chen W F (1982) "Plasticity In Reinforced Concrete Mcgraw Hill 474 pp

27. Clements R W (1977) Design of Reinforced Concrete Elements MD and Evans Pub. 127 pp.

28. Cowan H J (1982) Design of Reinforced Concrete Structures Englewood Cliff. 286 pp.

29. Chen W F (91982) "Box and Cylindrical Columns Under Biaxial bending" Axially Compressed Structures App. Sc. Pub. London 316pp.

30. Craig W H (91982) "Conflicts in Dynamic Modelling in Geomechanics" proc. Int. Seminar ISE London pp1-6.

31. GEB-FIP (1982) "Manual on Bending and Compression" Construction. Press London. 111p.

32. Coker E.B. (1987) "Effect of eccentricity on unsymmetrically reinforced Concrete Slender Columns Using CP110 and Timosnenko Approaches" B. Eng Thesis ATBU Bauchi.

33. Gedolin, L. MUlas M¢GC (91984) "Biaxial Stress-Strain Relation for Concrete" Journ Engrg Mech ASCE Vol 110 No. 2 pp 187-206.

34. Chiwuzie V.M. (91975) "Transverse Reinforcements in Short Concrete Columns" M.Sc. Thesis ABU Zaria.

35. Davies Li I. (1982) "Studies of the Response of Reinforced Concrete Structures to Short Duration Dynamic Loads". Proc. Int. Seminar ISE London pp 198-219.

36. Dyrbe, Jensen, Neilsen and Nitlegaard-Neilsen 1982) "Dynamic Loading of a Guyed Mast" proc. Int. Seminar ISE London. pp75-83.

37. Dow RcS¢ and Smith C.S. (91984) "Effects of Localized Imperfections on Compressive Strength of Long Rectangular Plates" Journ. Const. Steel Research (GB) Vol 4 No. 184 pp51-76.

38. Debernardi, Pier Gaorgio (1989) "Behaviour of Concrete Structures in Service" Journ. Struct. Engrg. Vol. 115 No. 1 pp32-50.

39. Ezetah V.E. and Bhalla S.N. (1989) "Materials Standards for Nigeria: Review" Proc. Int. Conf dev. Yola.

40. Furlong R.W. (1979) "Concrete Columns under Biaxially eccentric thrust" Journ ACI Vol 76 No 46 pp 1093-1194.

41. Ferguson Phil (1984) Reinforced Concrete Fundamentals John Wiley NY. 694pp.

42. Florek A. (1985) "Quality of Sandcrete Hollow blocks manufactured in Northern Nigeria" Proc. Conf. NTRC Fin & H Lagos.

43. Fintel Mark (1974) Handbook of Concrete Engineering Van Nonstrand New York. 801pp.

44. Fischer M (1984) "Load Carrying Behaviour and Design of Compressed Plates with longitudinal Stiffners" Journ. Stah Vol. 53 No.4 pp111-117.

45. Furlong R.W. (1961) "Ultimate Strength of Square Columns with Biaxially Eccentric Loads" Journ. ACI No. 57 pp 1129-1140.

46. Gary G. (1983) "Dynamic buckling of an Elastoplastic Column" Int. Journ. Engrg (GB) Vol 1 No. 4 pp357-378.

47. Grimault J.P. (1984) "Stability in the face of fire of Hollow Steel Columns Filled with Concrete" Journ. Inst. Concrete Tech and Public Highway France No. 423 pp1-18.

48. Gero J.S. (1973) "Structural Model testing of MCL Centre Tower Building" Model Lab report MR10 Univ. Sydney.

49. Harik I and Gesund H.(1988) "Flow Charts for Biaxial bending in resinforced conrete columns". Journ. Struct. Engrg Vol 114 No.6 pp 1230-1249.

50. Herzog M. (1984) "A New Formular for Slender Steel Columns under Eccentric Compression" Journ. Stan Vol 53 No.9 pp281-282.

51. Harris H.G. Bertero V.V. and Clough R.W. (1982) "One-fifth Scale Model of a Seven-storey R.C. Frame wall building under Earthquake" Proc. Int Seminar ISE London pp126-140.

52. Head M.C. and Aristizabal-Ochoa D.J. (1987) "Analysis

of Prismatic and Linearly Tapered Reinforced Concrete Columns". Journ. Struct. Engrg ASCE Vol 113 No.3 pp575-589

53. heger F.J. and Mcgrath T.S. (1982) "Shear Strength of Pipe, Box Sections and Other one-way Flexural Members." Journ. ACI Vol 79 No6 pp470-483.

54. Hsu C.T. and Mirza M.S. (1973) "Structural Concrete Biaxial bending and Compression" Juorn Struct. Div. ASCE Vol.99 No.ST2 pp285-290.

55. Hsu C.T. (1985) "Biaxially Loaded L-shaped reinforced Concrete Columns" Journ. Struct Engrg Vol.111 No.12 ASCE pp 2576-2595.

56. Jeary A.P. and Ellis B.R. (1982) "The Accuracy of Mathematical Models of Structural Dynamics" Proc Int. Seminar ISE London pp48-56.

57. Husseini A. (1984) "Limit State of Diagonal Cracking at the Support Zone in the Reinforced Flexural Elements" Ph.D Thesis ABU Zaria.

58. Kayal S. (1984) "Finite Element Method For Uniaxially Loaded Reinforced Concrete Columns' Journ. Struct Engrg ASCE Vol. 110 No.5 pp1114-1133.

59. Konishi I. and Takaoka N. (1981) "Some Comments on the Reliability Analysis of Civil Eng. Structures" Proc. ICOSSAR Norway pp341-358.

60. Liu x L and Chen W.F. (1984) "Reinforced Concrete pipe Columns: Behaviour and Design" Journ-Struct. Engrg ASCE Vol110 No.6 pp1356-1373.

61. Linder J. and Geitzelt R. (1984) "Biaxial bending and Thrust-Comparison of Design Concepts" Journ. Stan Vol 53 No.11 pp328-333.

62. Lepert P and Narzul P. (1982) "Adjustment of Platforms' Model Using experimental Results" Proc. Int Seminar ISE pp 7-20.

63. Lafe O¢ (1986) "Elements of R.C. Design" Macmillan Pub. 227pp

64. Mosley W.H. and Bungey J.H. (1976) Reinforced Concrete Design Macmillan Press Ltd. 350pp.

65. Morrell P. (1977) Design of Reinforced Concrete Elements. Granada London 214pp.

66. Mindess Sidney and Young J. Francis (1981) Concrete Prentice Hall New Jersey 207 pp.

67. Mander J.B. Priestly M.J.N. and Park R. (1988) "Observed Stress-Strain Behaviour of confirned concrete" Journ. Struct Engrg ASCE Vol.114 No.3 pp 1827-1849.

68. Maruyama K. Ramirez H. and Jirsa O.J. (1984) "Short R.C. Columns under Bilateral Load Histories" Journ. Sturct. Engrg. ASCE Vol.110 No.1 pp120-137.

69. Meek J.L. and Lin W.J. (1990) "Geometric and Material non Linear Analysis of thin Walled Beam Columns" Journ. Struct. Engrg ASCE Vol.116 No.6 pp1473-1490.

70. Meyer Christian and Bathe Klaus-Jurgen (1982) "Non Linear Analysis of r/c Structures in Practice" Journ. Struct. Div. ASCE Vol.108 No ST7 pp1605-1622.

71. Mekonnen B. (1984) "Reinforced Concrete Column Design Equations" Journ. ACI Vol 81 No.3 pp 242.250.

72. Mattock H.A. and Wang Z. (1984) "Shear Strength of Reinforced Concrete Members Subjects to High Axial Compressive Stress" Journ.Act Vol 81 No.3 pp 211-230.

73. Martinez S., Nilson A.H. and Slate F.O. (1984) "Shear Tests of High and Low Strength Concrete Beams without Stirrup" Journ. ACI Vol 81 No.4 pp 350-357.

74. Meek J.L. (1963) "Ultimate Strength of Columns with Biaxially Eccentric Loads" Journ ACI Vol 60 Part 8 pp 1053-1064.

75. Haq K. (1989) "Eccentric Loading Effect On Strength and Reinforcement Unsymmetry in Reinforced Concrete Columns" B.Eng Thesis ATBU Bauchi.

76. Macgregor J.G. Mirza S.A. and Ellingwood B (1983) Statistical Analysis of Resistance of reinforced and prestressed Concrete Members". Journ ACI No.3 Proc. V80

pp167-176.

77. Nandivaram Shanmugam and Sing-ping Chiew (1988) "Thin-walled Box Columns Under Arbitrary End Loads" Journ. of Struct. Engrg ASCE Vol.114 No.6 PP 1390-1402.

78. Neville A.M. (1963) PROERTIES OF CONCRETE. Pitmans 687 PP.

79. Nilson A.H. and Winter G. (1986) DESIGN OF CONCRETE STRUCTURES. Macgraw Hill Pub. 730pp.

80. Nandivaram E.S., Sing-Ping Chiew and Seng-lip Lee (1987) "Experimental Study of Thin-walled Steel Box Columns" Journ. Struct. Engrg ASCE. Vol 113 No. 10 pp. 2208-2220.31.Neils C. Lind (1981) "Structural Codes and Prototype Testing" Proc. ICOSSAR 81 Norway. Elsevier Oxford. pp. 639-648.

82. Nigerian Society of Engineeers (1985) Report on Collapsed Building Lagos.

83. Nigerian Institute of Structural Engineers (1985) Report on Collapsed Buildings. Lagos.

84. Oliboyejo A.L. (1988) "Eccentricity and Reinforcement Unsymmetry in a Reinforced Concrete Column" B. Eng. Thesis ATBU Bauchi.

85. Oliver M.G. and Clough R.W. (1987) "Biaxial Seismic Response of R/C Frames" Journ. Struct. Engrg ASCE Vol. 113 No. 6 pp. 1264-1281.

86. Oladapo I.O. (1981) Fundamentals of The Design of Concrete Structures. Evans Pub. Co. 213 pp.

87. Oleszkiewicz S. (1985) "Reinforced Concrete Structures" Unpublished A.B.U. Zaria.

88. Oliva M.G. and Bertero V.V. (1980) "Shaking Table Testing of a Reinforced Concrete Frame with Biaxial Response". Report No UCB/EERC 80/82 Univ. Calif. EERC.

89. Osha A.R.A. (1970) "Concrete Mix Design in The Tropics" Unpublished MSc. Thesis A.B.U. Zaria.

90. Olatunji R.O. (1985) "Finite Element Analysis of Panel Buildings Under Differential Support Settlement" Unpub. M.Eng. Thesis A.B.U. Zaria.

91. Priestly M.J. and Elder D.M. (1983) "Stress-Strina Curves for Unconfined and Confined Concretee Masonry" Journ. ACI Vol. 80 No. 3 pp.192-200.

92. Park R. and Paulay T. (1975) Reinforced Concrete Structures John Wiley New York. 769 pp.

93. Priestley K,J.N. Park R and Potangaroa R.T. (1981) "Ductility of Sirally Confined Concrete Columns" Journ. Struct. Div. ASCE Vol. 107 STI pp. 181-202.

94. Papia Maurizo and Russu Gaetano (1989) "Cojmpressive Concrete Strain at Buckling of Lonititudinal Reinforcement" Journ. Struct. Engrg ASCE Vol. 115 No. 2 pp 382-397.

95. Paulay T. (1982) "Lapped Splices in Earthquake Resisting Columns" Journ. ACI Vol. 79 No. 6 pp 458-469.

96. Parrott L.J. (1974) "Lateral Strains in Hardened Cement Paste Under Short and Long Term Loading" Mag. Conc. Research Vol. 26 No. 89 pp. 198 - 202.

97. Polyakov S.V. (1985) Design of Earthquake Resistant Structures Miv. Pub. Moscow 374 pp.

98. Papia M. Russo G. and Zingone G. (1988) "Instability of Longitudinal Bars in Reinforced concrete columns" Journ. Struct. Engrg. ASCE Vol. 114 No. 2 pp 445-461.

99. Pado Riva and Cohn M.Z. (1990) "Engineering Apprcach to Non-Linear Analysis of concrete Structures" Journ. Struct. Engrg. ASCE Vol. 116 No. 8 pp 2162 - 2186.

100. Pham L. (1984) "Reliability Analysis of Simply Supported Composite Beams" Civ. Eng. Trans IE Aust. Vol. Le 26 No.1 pp 41-48.

101. Perme A.L., Nieves J.M and Gouwers A. (1966) "Capacity of Reinforced Rectangular Columns Subject to Biaxial Bending" Journ. ACI No. 63 pp. 911 - 923.

102. Ramamurthy L.M. and Hafeezs Khan T.A. (1983) "L-Shaped Column Design for Biaxial Eccentricity" Journ. Struct. Engrg ASCE Vol. 109 No. 8 pp 1903 - 1917.

103. Robert E.H. and Yam L.C.P. (1983) "Some Recent Methods for the Design of Steel, Reinforced concrete and composite Steel-Concrete Columns in the UK" Journ. ACI March/April pp 139-149.

104. Rashedi Reza and Moses Fred (1988) "Identification of Failure Modes in System Reliability" Journ. Struct. Engrg ASCE Vol. 114 No. 2 pp 292 - 313.

105. Reynolds Chas E. (1968) Basic Reinforced concrete Design CPL London 264 pp.

106. Sheikh S.A. and Uzumeri S.M. (1982) "Analytical Model for concrete Confinement in Tied Colums" Journ. Struct Div. ASCE Vol. 108 ST 12 pp. 2703 - 2722.

107. Schodek L.D. (1980) Structures Prentice Hall. 572 pp.

108. Shrive N.G. (1983) "Compression Testing and Cracking of Plain Concrete" Mag. Conc. Research Cement and concrete Aslsociation Vol. 35 No. 122 pp 27 - 39.

109. Sengupta S.K. (1983) "Reinforced concrete Member Under Combined Torsion and Axial Compression" Jorn. Inst. Engrs (India) Vol. 64 Part C12 pp. 97 - 103.

110. Sheikh S.A and Uzumeri S.M (1980) "Strength and ductility of Tied concrete columns" Journ. Struct. Div. ASCE Vol. 106 No ST5 pp. 1079-1101.

111. Sheikh A.S. and Uzumeris M. (1982) "Analytical Model for concrete Confinement in Tied columns" Jorn. Struct. Div. ASCE Vol. 8 No ST12 pp. 2703-2722.

112. Sridharan S. and Benito R. (1984) "Columns: Static and Dynamic Interactive Buckling" Jorn. Engrg Mech. ASCE Vol 110 No. 1 pp. 49 - 65.

113. Strachan C.D. (1981) "Construction of The Humber Bridge" Proc. British Ass. for The Advancement of Science Vol. 2 pp. 57 - 67.

114. Taylor M.A. (1985) "Direct Biaxial Design of columns" Journ. Struct. Engrg Vol. 111 No. 1 ASCE pp. 158-174.

115. Taylor R, Shakir-Khalil H, Yee K.M. (1983) "Some Tests on a New type of composite columns" Proc. ICE Part 2 pp. 283 - 296.

116. Timoshenko P. and Gere J.M. (1961) Theory of Elastic Stability Mcgraw Hill 541 pp.

117. Tasker H.E. (1947) "A Dimensional Analysis Study for Model Test Walls" Duplicated Document No. 6 Commonwhealth Experimental Building Station Sydney Asustralia.

118. Tso K. Wai (1990) "Staltic Eccentricity concept for Tortional Moment Estimations" Journ. Struct. Eng. ASCE Vol. 116 No.5 pp 1199 - 1212.

119. Vallenas J., Bertero V.V and Popov E.P. (1977) "Concrete confined by Axial Loads" Report No UCS EERC 77/13 Univ. Calif. Berkeley.

120. Virdi K.S. and Dowling P.J. (1982) "Composite columns in Biaxial Loading" Axially compressed Structures. Applied Science Pub. pp. 129 - 147.

121. Vinnakota S. (1982) "Planar Strength of Restrained Beam columns" J. Struct div. ASCE Vol. 108 No ST 11 pp. 2496-2516.

122. Watson A.J. and Ang. T.H. (1982) "Impact Resistance of Reinforced concrete Columns" Proc. Int. Seminar ISE London. pp. 253.

123. Weber D.C. (1966) "Ultimate Design Charts for columns with Biaxial Bending" Journ. ACI Vol. 63 No. 11 pp. 1205 - 1230.

124. Winter G. and Arthur H.N. (1986) Design of concrete Structures Mcgraw Hill 730 pp.

125. Wearing J.L. and Patterson C. (1981) "Vibration Testing of composite Materials" Proc. Int. conf. composite Material IME London. pp. 463 - 474.

126. Walley F. and Bate S. (1961) "Guide to The BS Code of Practice for Prestressed Concrete. Cement and concrete Association London 95 pp.

127. World construction (1986) "Gulf concrete Under Scrutiny" Vol. 39 No, 7 pp. 22 - 24.

128. World Construction (1984) "Good and Bad News About Structural Failures" Vol. 37 No. 7 pp. 14 - 16.

129. Wu J.W and Gould P.L. (1984) "Pure Bending of Thin-Walled Beams" Journ. Engrg Mech. ASCE Vol. 110 No. 7 pp. 1076 - 1985.

130. Umaru Bello Sanda (1989) "Effectiveness of Overlapping Hoops as Ductility Reinforcement in Short concrete columns" MSc Thesis A.B.U., Zaria.

131. Wilby C.B. (1983) Structural concrete. Butterworths U.K. 256 pp.

132. Yusuph Azeez A. (1987) "Effect of concrete-Steel Area Ratio on The Strength of New type of composite columsn" B. Eng. Thesis ATBU Bauchi.

133. Wan T. Tasi (1988) "Uniaxial compressional Stress-Strain Relation in Concrete" Journ. Struct. Engrg vol. 114 No. 9 pp. 2133 - 2138 ASCE.

134. Yong Yook-Kong, Nour Malakah G and Nawy Edward (1988). Behaviour of Laterally confined High Strength Concrete Under Axial Loads" Journ. Struct. Engrg ASCE Vol. 114 No. 2 pp. 332-351.

135. Yam Lloyd C.P. (1981) Design of Compsite Steel-Concrete Structures. Surrey Univ. Press 168 pp.

136. Zhou S.P and chen W."F (1985) "Design Criteria for Box columsn Under Biaxial Loading" Journ. Struct. Engrg ASCE Vol. 111 No.2 pp. 2643 - 2658.

137. Ziekienwicz O.C. (1977) The finite Element Method Mcgraw Hill London 787pp.

138. Zuk William (1988) "Structures Beyond Tomorrow" Journ. Prof. Issues ASCE. Vol. 114 No. 3 pp. 344 - 347.

ADDENDUM

Please note that due to production problems.

1. Pages 6 and 7 are blank and therefore not included.

2. Plate 2 is in page 241 and not 116.

V.E. Ezetah

KASHIM IBRAHIM LIBRARY