STUDIES ON POLYMER MODIFIED STEEL FIBRE REINFORCED CONCRETE

A Thesis

submitted in fulfilment of the requirement for the award of the Degree

DOCTOR OF PHILOSOPHY

IN
FACULTY OF ENGINEERING
(CIVIL ENGINEERING)

OF THE UNIVERSITY OF CALICUT

BY

SHIVANANDA. K.P

DEPARTMENT OF CIVIL ENGINEERING
REGIONAL ENGINEERING COLLEGE
CALICUT - 673 601, KERALA
DECEMBER - 1998

REGIONAL ENGINEERING COLLEGE

CALICUT - 673 601, KERALA

CERTIFICATE

This is to Certify that the report entitled **"STUDIES ON POLYMER MODIFIED STEEL FIBRE REINFORCED CONCRETE" is** a bonafide record of the work done by **Mr. SHIVANANDA. K. P.**, under my supervision and guidance. The report is submitted to the **University of Calicut** as per the requirements of qualifying examination of the degree of **DOCTOR OF PHILOSOPHY** in Faculty of Engineering (Civil Engineering) .

N. Ganesan
5/1/99

R.E.C., Calicut

December 1998

Dr. N. GANESAN
Supervisor and Guide
Professor of Civil Engineering
R.E.C., Calicut. 673 601

Forwarded

5/1/99

I/C HEAD OF THE DEPARTMENT
CIVIL ENGINEERING
REGIONAL ENGINEERING COLLEGE

ABSTRACT

Strength and ductility are the two important factors to be considered in the design of structures subjected to seismic and other dynamic forces. Hence many attempts have been made in the recent past to develop new materials which exhibit higher strength and ductility than conventional concrete, so that they could be used in the case of structures subjected to seismic loading, impact or blast loading, cyclic loading etc. It has been understood from the literature that many of the engineering properties such as tensile strength, compressive strength, flexural strength, fracture toughness, energy absorption capacity, strain at peak load etc. of the conventional concrete could be improved by the addition of steel fibres. Similarly incorporation of polymers into the concrete also have been found to enhance the foresaid properties significantly. However no attempts have been come across on the combined of effect of steel fibres and polymers on the strength and behaviour of conventional concrete.

Considering this gap in the existing knowledge, an attempt has been made to study the combined effect of polymers and steel fibres on the strength and behaviour of conventional concrete. The polymer considered in this study is natural rubber latex. The preliminary studies on small size specimens and extension of the same to large-scale flexural members indicate that several engineering properties of the conventional concrete could be significantly improved by the latex modification and incorporation of steel fibres into the same.

Also, in the limit state design of reinforced concrete structures, cracking is one of the limit states, which the design has to satisfy to ensure serviceability of the structure. Limiting the width of cracks is important from the aesthetic point of view, to ensure water tightness and to safeguard the reinforcement against corrosion. For this purpose, suitable methods for estimating the width of cracks are required. Many variables influence the width and spacing of cracks in reinforced concrete members. Due to the complexity of the problem, a number of methods have been developed in the past to determine the width and spacing of cracks. These methods are generally based on the theoretical basis and partly on the test results. Some investigators have also developed empirical equations from the statistical analysis of the test results.

Some of these methods for predicting maximum width of crack have been introduced into the International Codes of practice with or without modification. While several methods are available for predicting the width and spacing of cracks in reinforced concrete flexural members, no information is available in literature for predicting the width and spacing of cracks in steel fibre reinforced concrete flexural members and polymer modified steel fibre concrete members. Hence an attempt is also made to propose a method for predicting the maximum width and spacing of cracks in steel fibre concrete conventional members. The same method has been suitably modified to account for the presence of polymer in the latex modified steel fibre reinforced concrete flexural members.

Taking note of the above points, the present investigation has been planned with an aim towards:

i. Obtaining the physical properties such as compressive and flexural strength, strain at peak load, energy absorption capacity etc. of latex modified concrete using small specimens like cubes, cylinders and prisms

ii. Studying the effect of latex modification and addition of steel fibres on the strength and behaviour of conventionally reinforced concrete flexural members

iii. Investigating effect of confined steel fibre reinforced concrete in the compression zone of latex modified reinforced concrete flexural members and

iv. Developing a method for determining the spacing and maximum width of cracks in
    a. Reinforced concrete flexural members additionally reinforced with steel fibres
    b. Latex modified reinforced concrete flexural members additionally reinforced with steel fibres.

The studies undertaken are as follows

1. Preliminary Studies on Latex Modified Concrete

An experimental programme was carried out to study the effect of natural rubber latex as polymer on the strength and behaviour of conventional concrete under compression and flexure. Also the effect of confinement on the strength and behaviour of latex modified concrete under Uni.-axial compression was studied. This preliminary investigation was restricted to small-scale specimens and the outcome of this experimental programme was suitably made use of in the later stages, for prototype flexural members.

The preliminary study revealed that the addition of small quantities of Dry Rubber Content (DRC) marginally improved the compressive and flexural strength of conventional concrete. A significant increase in the strain at peak load, energy absorption capacity was observed with the addition of 0.5% to 1.0% DRC. Further it was observed that the foresaid properties could be increased appreciably by providing confinement. Also the studies reveal that the reduction in strength due to the addition of higher percentage of DRC (more than 1.0%) can be markedly reduced by providing higher volumetric ratio of confinement.

2. Latex Modified Steel Fibre Reinforced Concrete Flexural Members

An attempt is made to study combined effect of latex and steel fibres on the strength and behaviour of conventionally reinforced concrete flexural members. Totally sixteen beams of 125 x 200 x 2000mm size were cast and tested. Out of these, twelve beams were latex modified steel fibre concrete beams and four were latex modified beams. Three different values of percentages of volume fraction of steel fibres (V_f) i.e.0.5, 1.0 and 1.5% and three different percentages of DRC i.e. 0.5, 1.0 and 1.5% were used as variables. The test results revealed that the addition of latex (0.5 to 1.0% of DRC) improve the first crack load and ultimate moment of resistance of the flexural members. The overall improvement in the ductility, toughness index and energy absorption capacity achieved due to the addition of latex and steel fibres indicate that latex modified steel fibre reinforced concrete appears to be an appropriate material for structures subjected to cyclic, impact, dynamic loading etc. Also an attempt is made to predict the first crack load and ultimate moment of resistance of latex modified steel fibre reinforced concrete flexural members.

3. Latex Modified Reinforced Concrete Beams with Confined Steel Fibre Reinforced Concrete in the Compression Zone

An attempt is made to study the effect of confined steel fibre concrete in the compression zone of latex modified reinforced concrete flexural members. The experimental programme consisted of casting and testing sixteen beams of 125 x 200 x 2000 mm size. The spiral hoops were used to confine the concrete in the compression zone. The studies reveal that the provision of confined steel fibre concrete in the compression zone of the latex modified reinforced concrete beams increases the load carrying capacity, ductility, the energy absorption capacity and toughness index of the specimens upto certain level and then decreases as the percentage of DRC increases.

4. Studies on Cracking of Latex Modified Steel Fibre Reinforced Concrete Flexural Members:

a. Comparison of International Codes

An attempt is made to compare the methods adopted in the International Codes of practice for predicting the maximum width of cracks in reinforced cement flexural members using the test results available in the literature. A total number of 732 test results reported by Hognestad, Clark, and Base et al have been used for comparing their experimental maximum width of cracks with those computed from the equations of (i) British Code (BS 8110 -1985), (ii) Model Code -1990, (iii) Gergely Lutz equation (ACI - 318-1995) and (iv) Chinese Code (GBJ10-89, 1989). From the comparison it was found that Gergely Lutz equation (ACI Code equation) predicts the width of cracks better when compared to the other equations. As these International Equations for predicting the maximum width of cracks did not compare satisfactorily with the experimental maximum width of cracks in the case of steel fibre concrete flexural members, the following method has been proposed.

b. Spacing and Width of Cracks in Steel Fibre Reinforced Concrete Flexural Members

A method has been proposed to determine the spacing and width of cracks in steel fibre reinforced concrete flexural members. The above method is the extension of the method proposed for reinforced concrete flexural members with the appropriate modification introduced to account for the presence of steel fibres in the reinforced concrete flexural members. The test results available in the literature have been used for evaluating the empirical constants appearing in the equation proposed in this study. The computed values of spacing and maximum width of cracks are found to compare satisfactorily with the test results.

c. Spacing and Width of Cracks in Latex Modified Steel Fibre Reinforced Concrete Flexural Members

A method is proposed to determine the spacing and maximum width of cracks in latex modified steel fibre reinforced concrete flexural members. The above method is an extension of the method proposed earlier for steel fibre reinforced concrete flexural members. Computation indicated that the presence of latex influences the strain in steel. A correction factor F to that effect was introduced in the proposed method. With the corrected values of steel strain, the spacing and maximum width of cracks were computed and compared with the experimental test results obtained in this study. The proposed method for computing the spacing and maximum width of cracks was found to compare satisfactorily with the experimental values.

The above studies are expected to be useful in a better understanding of strength and behaviour of latex modified steel fibre reinforced concrete flexural members with and without confinement. Also the methods proposed to predict the spacing and maximum width of cracks for steel fibre reinforced concrete flexural members and latex modified steel fibre reinforced concrete flexural members would be useful in the formulations related to limiting crack width criterion in the design of these structural members.

Based on the present studies, the following papers have been published so far along with the guide:

Journals

1. *"Comparison of International Codes for the Prediction of Maximum Width of Cracks in Reinforced Concrete Flexural Members"*, The Indian Concrete Journal, Vol. 70, No.11, November 1996, pp. 635-641.

2. *"Prediction of Spacing and Maximum Width of Cracks in Steel Fibre Reinforced Concrete Flexural Members"*, Journal of Structural Engineering, Vol. 24, No.3, October 1997, pp. 143-148

3. *"Strength and Ductility of Latex Modified Steel Fibre Reinforced Concrete Flexural members"*, Paper accepted for publication in the Journal of Structural Engineering.

Conferences:

4. *"Latex Modified Steel Fibre Reinforced Concrete for Seismic Resistant Structures"* Paper presented at the National Seminar on Civil Engineering in Disaster Management, held at Trivandrum, during 7-8, December 1995. pp. 5-1 -5-10.

5. *"Effect of Latex Modification on the Strength and Behaviour of Confined Concrete under Uni-Axial Compression"*, Paper presented at the International Seminar on Civil Engineering Practices in the Twenty-first Century, held at Roorkee during 26-28, February - 1996. pp. 757-764.

6. *"Prediction of First Crack Load and Ultimate Moment of Resistance of Polymer Modified Steel Fibre Reinforced Concrete Flexural members"*. Paper presented at the National seminar on High Performance Concretes, held at Chennai during 21-22 May 1998, pp. TS2-33- 41.

7. *"Performance of Latex Modified Reinforced Concrete Flexural Members with Confined Steel Fibre Concrete in the Compression Zone"*, Paper presented at the Sixth NCB International Seminar on Cement and Building Materials, held at New Delhi, during 24-27, November 1998.

ACKNOWLEDGEMENTS

The author wishes to express his deep sense of gratitude to **Dr. N.Ganesan,** P.G. Professor in Civil Engineering, for his invaluable guidance and encouragement throughout this investigation. It has been a unique pleasure to do Research work under Dr. Ganesan.

The author express his sincere thanks to **Dr. Vincent Paul,** Professor and Head of the Department of Civil Engineering, for providing the facilities for carrying out this research work.

The author wishes to sincerely thanks Prof. **Nagaraj, B.N.**, Dean Planning and Prof. **V.S.Prasad,** Prof. of Civil Engg. for their interest and encouragement given during the course of this investigation.

The author acknowledges with thanks the help rendered by his friends M/s **Joseph, J.X., V.V. Prakashan, M.V. Praveen, H.D.Sriram and Ms. Subha Lekshmi,** Post Graduate students during the various stages of the experimental work.

The service rendered by the staff of the structures laboratory during the course of experimental work are gratefully acknowledged.

The author thanks the **Management and Principal, Siddaganga Institute of Technology, Tumkur, Karnataka,** for sponsoring him for higher studies under the Deputation Scheme.

Finally, the author wishes to thank his wife **Mrs. Renukalatha** for her patience and encouragement and support throughout the period of work.

SHIVANANDA. K.P.

9

NOTATIONS

A_e - Effective area of concrete in tension

A_f - Aspect ratio of fibre

A_s - Area of main steel reinforcement

a_c - Average crack spacing

a'_m - Average spacing of cracks when they have just formed

b - Width of the slab or beam

c - Distance from the point at which the crack is to be determined to the surface of the nearest reinforcement

C_1 - A Constant

d - Effective depth of slab or beam

E_c - Modulus of Elasticity of concrete

E_s - Modulus of Elasticity of Steel

f_s - Steel stress

f_{bu} - Ultimate bond strength

f_y - Yield strength or 0.2% proof stress of steel binder.

f_c - Ultimate strength of unconfined concrete specimen.

f'_c - Ultimate strength of confined concrete specimen.

f'_{lc} - Ultimate strength of Latex modified confined concrete

f_{ct} - Tensile strength of concrete

f_{ck} - Characteristic strength of concrete

h - Overall depth of cross section

h_1 - distance from the centroid of the tension steel to the neutral axis

h_2 - distance from the point at which the crack width is to be determined to the neutral axis

I_{cr} - Moment of inertia of cracked section

K_b - Factor giving average bond stress

K_t - Factor giving average tensile stress

M - Bending moment

m - modular ratio (E_s/ E_c)

M_u - Ultimate bending moment

M_{cr} - Moment at cracking

V_f - Volume fraction of fibres

W_{max} - Maximum crack width on the surface of the beam

W_{cal} - Calculated Maximum crack width

W_{exp} - Maximum crack width obtained from experiment

W_{bt} - maximum crack width at the lower extreme fibre

x	- Neutral axis depth of a cracked section
ε_s	- Strain in steel
$\varepsilon_{s(cal)}$	- Calculated steel strain
$\varepsilon_{s(cor)}$	- Corrected steel strain
γ	- A constant.
ϕ	- Diameter of bar
ρ_b	- Volumetric ratio of confinement, ie, ratio of volume of binder to volume of core concrete.
ρ_b	- Particular volumetric ratio of confinement when the pitch of binder is equal to the least lateral dimension of the specimen.
a_{min}	- minimum crack spacing
u	- average bond stress
ΣO	- sum of the perimeters of the bar.

CONTENTS

			Page No.
Abstract			i
Acknowledgment			vii
Notations			viii
Contents			x
CHAPTER 1 INTRODUCTION			
1.1	General	....	1
1.2	Polymer Concrete Composites	....	2
1.3	Fibre Reinforced Concrete	....	3
1.4	Polymer Modified Steel Fibre Reinforced Concrete	....	4
1.5	Current Research Trends	....	4
1.6	Concluding Remarks	....	5
1.7	Structure of the Report	....	5
CHAPTER 2 REVIEW OF LITERATURE			
2.1	Introduction	....	8
2.2	Steel Fibre Reinforced Concrete	....	9
	2.2.1 Comments on the earlier works on steel fibre reinforced concrete	....	21
2.3	Polymers in Concrete	....	22
	2.3.1 Comments on earlier works on polymer modified concrete	....	28
2.4	Confined Concrete	....	29
	2.4.1 Comments on earlier works on confined concrete	....	35
2.5	Cracking of Reinforced Concrete	....	36
	2.5.1 Studies on cracking of reinforced concrete	....	37
	2.5.2 Classical theory or bond-Slip hypothesis	....	37
	2.5.3 No-slip theory - studies from UK	....	38
	2.5.4 General theory of cracking	....	38
	2.5.5 Studies of Broms	....	39
	2.5.6 Statistical approach - studies from USA	....	39
	2.5.7 Other studies	....	40

2.6 International Code Specifications Related to Cracking 42
2.6.1 I.S.Code 456-1978 42
2.6.2 B.S. 8110-1985 42
2.6.3 Model Code-1990 43
2.6.4 ACI Code 318-1995 45
2.6.5 Chinese Code equation (GBJ10-1989) 46
2.6.6 Comments on the earlier work on cracking of reinforced concrete 47
2.7 Scope of the Present Investigation 47

CHAPTER 3 PRELIMINARY STUDIES ON LATEX MODIFIED CONCRETE

3.1 Introduction 49
3.2 Natural Rubber Latex 50
3.2.1 Composition of latex 50
3.2.2 Molecular structure of latex 52
3.2.3 Workability of latex modified concrete 53
3.3 Effect of Latex Modification on the Strength of Concrete 54
3.3.1 Experimental programme 54
3.3.2 Variables considered 57
3.3.3 Testing of specimens 58
3.3.4 Discussion of test results 63
3.4 Effect of Confinement on Latex Modified Concrete 66
3.4.1 Introduction 66
3.4.2 Experimental programme 67
3.4.3 Casting of specimens 69
3.4.4 Testing of specimens 70
3.4.5 Behaviour of specimens under load 70
3.4.6 Results and discussion 71
3.5 Conclusions 76

CHAPTER 4. LATEX MODIFIED STEEL FIBRE REINFORCED CONCRETE FLEXURAL MEMBERS

4.1 Introduction 78
4.2 Experimental Programme 79
4.3 Testing of Specimens 83
4.3.1 Test setup 83
4.3.2 Testing procedure 84
4.4 Behaviour of Specimens Under Load 87
4.5 Discussion of Test Results 91
4.5.1 Load deflection behaviour 94
4.6 Analysis of Test Results 100
4.6.1 Variation of ultimate load with latex fibre index 100
4.6.2 Energy absorption capacity and toughness index 103
4.6.3 Ductility factor 106
4.7 Prediction of First Crack Load and Ultimate Moment of Resistance 114
4.7.1 First crack load 114
4.7.2 Ultimate moment of resistance 117
4.7.3 Modification proposed 120
4.8 Load factors 122
4.9 Conclusions 125

CHAPTER 5. LATEX MODIFIED REINFORCED CONCRETE BEAMS WITH CONFINED SFRC IN THE COMPRESSION ZONE

5.1 Introduction 126
5.2 Experimental Programme 127
5.3 Details of Specimens 127
5.3.1 Materials used 129
5.3.2 Preparation of the specimen 131
5.4 Test setup and Testing Procedure 132
5.4.1 Testing procedure 134
5.5 General behaviour of Specimens in Flexure 135

5.6 Discussion of Test Results 139
5.6.1 Variation of ultimate load with volume fraction of steel fibres 149
5.6.2 Variation of deflection at ultimate load 149
5.7 Energy Absorption Capacity and Toughness Index 152
5.7.1 Energy absorption capacity 152
5.7.2 Toughness index 154
5.8 Ductility Factor 161
5.9 Introduction of Constitutive Parameter 165
5.10 Load Factor 170
5.10.1 Load factor with respect to Limit state of deflection 170
5.10.2 Load factor with respect to limit state of cracking 173
5.11 Conclusions 175

CHAPTER 6 STUDIES ON CRACKING OF LATEX MODIFIED STEEL FIBRE REINFORCED CONCRETE FLEXURAL MEMBERS

6.1 Introduction 177
6.2 Comparison of International Codes for the Prediction of Maximum Width of Cracks in Reinforced Concrete Flexural Members 178
6.3 Test Results 178
6.3.1 Test results of Hognestad 178
6.3.2 Test results of Clark 179
6.3.3 Test results of Base etal 179
6.4 Comparison of International Equations with the Test Results 181
6.4.1 BS - 8110 - 1985 equation 181
6.4.2 Model Code - 1990 equation 182
6.4.3 ACI Code 318-1995 equation 183
6.4.4 Chinese Code equation (GBJ10-89 1989) 185

6.5 Results 185

6.6 Discussion of Test Results 204

6.7 Conclusions 205

6.8 Prediction of Spacing and Maximum width of Cracks In Steel Fibre Reinforced Concrete Flexural Members........... 206

6.8.1 Introduction 206

6.8.2 Test results 207

6.8.3 Comparison of International equations 210

6.8.4 Significance of choosing Desayi-Ganesan equation 211

6.8.5 Proposed method 212

6.8.6 Modifications proposed in this study 214

6.8.7 Spacing of cracks 222

6.8.8 Conclusions 230

6.9 Prediction of Spacing and Maximum Width of Cracks in Latex Modified Steel Fibre Reinforced Concrete Flexural Members 231

6.9.1. Introduction 231

6.9.2 Modification proposed in this study 231

6.9.3 Spacing of cracks 239

6.9.4 Conclusions 245

CHAPTER - 7 CONCLUSIONS AND SCOPE FOR FURTHER WORK

7.1 Preliminary studies on Latex Modified Concrete 246

7.2 Latex Modified Steel Fibre Reinforced Concrete Flexural Members 248

7.3 Latex Modified Reinforced Concrete Beams with Confined SFRC in the Compression Zone 249

7.4 Studies on Cracking of Reinforced Concrete Flexural Members 251

7.5 Scope for Further Work 252

REFERENCES 253

APPENDIX – I Model calculation for the Prediction of Spacing and maximum width of cracks in Latex modified SFRC Flexural members 262

1

APPENDIX - II Test Results Used for Comparison of International Codes and Model calculation for the Comparison of International Codes for the Prediction of maximum width of cracks in reinforced Concrete Flexural members 267

APPENDIX - III Model calculation for the Prediction of Spacing and maximum width of cracks in SFRC Flexural members 298

APPENDIX - IV Model calculation for the Prediction of Spacing and maximum width of cracks in Latex Modified SFRC Flexural members 306

CHAPTER - 1

INTRODUCTION

1.1 General

In the construction industry, *Concrete Technology* is heading towards an entirely new era by way of using polymers and fibres along with superplasticizers in concrete. Increasing interest is being shown in the area of new materials in the past two decades. This is quite understandable because, it is slowly, but increasingly being recognised that economic progress in construction depends more on an intelligent use of the materials and constant improvement of available materials than on extreme refinements of structural analysis.

Reinforced concrete structures, unlike steel structures, tend to fracture or fail in a relatively brittle fashion, as the ductility or deformation capacity of conventional concrete is limited. In such structures the brittle failure as a result of inelastic deformation can be avoided only if the concrete is made to behave in a ductile manner so that the member can absorb and dissipate large amount of energy.

The aforesaid reasons attracted several researchers and their investigations have resulted in the development of new materials like polymer composites, fibre composites, ferrocement etc. Also the investigations have revealed that providing suitable confinement to the concrete in the compression zone could further enhance the peak strength, strain at peak load and ductility of conventional reinforced concrete members. While several investigations are available in literature on the strength and behaviour of steel fibre reinforced concrete (SFRC) and polymer composites, attempts on polymer modified SFRC, which reflects the combined effect of polymers and steel fibres on the conventional concrete specimens are scanty. This gap in the existing knowledge suggests that a research programme on the strength and behaviour of polymer modified fibre reinforced concrete would be of much relevance. Hence an attempt has been made to conduct an experimental investigation to study, first, the effect of addition of polymers and steel fibres on the strength and behaviour of conventional concrete and subsequently extend this investigation to study the combined effect of polymer modification and addition of steel fibres on prototype conventional reinforced concrete structural members such as flexural members.

1.2 Polymer Concrete Composites

Though concrete is the most widely used construction material, it suffers from three major weaknesses - low tensile strength, high porosity and susceptibility to chemical and environmental attack. Most of these disadvantages found in ordinary structural concrete are removed by using polymer concrete composites. Polymer concrete composites are relatively a new development and are extensively used in many structural applications. They possess very high strength (up to 200 N/mm^2 in compression) and are more durable and resistant to most of the chemicals and acids [68].

Various types of polymer concrete composites make use of what are known as monomers. A monomer is an organic molecule, which is capable of combining chemically with similar or different molecules to form a higher molecular weight compound known as a polymer. A polymer consists of several monomers, which are linked, together in a chain like structure and the chemical process is called polymerisation.

Polymers are chemically inert materials having higher tensile and compressive strengths than conventional concrete. However, polymers have a lower modulus of elasticity and a higher creep, and may be degraded by heat, oxidising agents, ultra violet light, chemicals and micro-organisms. Also, certain organic solvents may cause stress cracking. Many of these disadvantages can be overcome by choosing a suitable polymer and by adding substances to the polymer which suppress the harmful effects.

There are different types of polymer and are being used in cement concrete construction. They are aqueous polymers, powder emulsions, water soluble polymers, liquid polymers etc. Rubber latex comes under aqueous polymer category. Recent studies indicate that both natural and synthetic rubber latex improve the engineering properties of concrete markedly.

The technology of polymer concrete is still, to a large extent, in the experimental stage. Extensive work is being done presently in different parts of the world to arrive at practically acceptable technologies and optimum concrete polymer combinations suitable for convenient commercial applications. More practical applications and their performance studies instil more confidence for field applications of these materials.

1.3 Fibre Reinforced Concrete

Fibre reinforced concrete is the concrete made of hydraulic cement, containing fine or fine and coarse aggregates and discontinuous discrete fibres [1]. Mainly three types of fibres - glass, polypropylene and steel are currently being used as concrete reinforcement. Due to the low effectiveness, poor alkaline resistance or high cost, the use of other fibres such as nylon, rayon carbon etc. has been almost ruled out after initial investigation.

Inclusion of randomly distributed steel fibres has been found to enhance the tensile strength and fracture toughness of cementitious materials significantly [77]. These improvements can be attributed to the arrest of micro cracks by fibres and also to the restraint against widening of cracks provided by the fibres bridging these cracks.

Steel fibre reinforced concrete has found many interesting field applications in the mass concrete work. They include bridge decks and overlays, highways and airfield pavements, repair work in dam's etc.

1.4 Polymer Modified Steel Fibre Reinforced Concrete

Polymer modified steel fibre reinforced concrete is made of hydraulic cement, containing fine or fine and coarse aggregate, discontinuous discrete fibres and polymers.

When fibres and polymers are added to conventional concrete they improve mechanical properties of conventional concrete significantly. Recent tests on polymer modified steel fibre concrete indicate that they are more durable. In literature only a few studies are available on polymer modified steel fibre concrete and these are restricted to small-scale specimens only. However studies on the effect of latex modified steel fibre concrete on the flexural behaviour of conventional RCC members have not been encountered in literature.

1.5 Current Research Trends

Strength and ductility are two important factors to be considered in the design of structures subjected to seismic forces and dynamic forces. Hence many attempts have been made in the recent past to develop new materials which exhibit higher strength and ductility, so that they could be used in the disaster resistant structures. Some of the materials, which have been developed, are Steel Fibre Reinforced Concrete (SFRC); polymer modified concrete etc. Incorporation of either steel fibres or polymers has been found to improve many of the mechanical properties of conventional concrete. Research activities are still in progress in this area with different types of fibres and polymers. However, no attempts have been made so far to study the combined effect of steel fibres and polymers on the behaviour of conventionally reinforced cement concrete members.

1.6 Concluding Remarks

It has been understood from the literature that many engineering properties like tensile and flexural strength, fracture toughness, energy absorption capacity, strain at peak load etc. could be improved by the addition of steel fibres and polymers to conventional concrete. However, no attempts have been made in the past to study the combined effect of steel fibres and polymers on the flexural behaviour of conventional cement concrete. Hence an attempt has been made through the present investigation to study the effect of latex modification and inclusion of steel fibres on the physical properties like compressive strength, flexural strength, strain at peak load etc. using control specimens first and extend the investigation to study the effect of polymer modified steel fibre reinforced concrete on the behaviour of structural members. The structural members considered in this study are flexural members subjected to point loads.

The investigation consists of three major parts: The first part deals with the studies on the effect of polymer (latex) modification and the inclusion of steel fibres on the flexural behaviour of conventionally reinforced concrete beams. The second part deals with the flexural behaviour of polymer modified reinforced concrete beams with confined SFRC in the compression zone. The third part deals with the studies carried out on the cracking behaviour of reinforced concrete flexural members with steel fibres. A method has been proposed for estimating the width and spacing of cracks in these members. Also the proposed method has been further modified to account for the addition of polymer (latex) in the case of latex modified steel fibre reinforced concrete flexural members.

1.7 Structure of the Report

The report consists of *Seven Chapters*. The first Chapter gives a brief introduction to the investigation carried out and explains the research significance of the present investigation. The previous studies in the field of fibre reinforced concrete, polymer-modified concrete and confined concrete have been critically discussed in Chapter 2 under the heading "*Review of Literature*". Towards the end of this Chapter, scope of the present investigation is discussed in detail.

Chapter 3 deals with the *preliminary studies* on latex modified concrete. In this Chapter attempts have been made to study the effect of natural rubber latex as polymer on the strength and behaviour of conventional concrete in compression and flexural members. The conclusions arrived at, based on this investigation has been discussed in this Chapter.

Chapter 4 deals with the *studies on latex modified steel fibre reinforced concrete flexural members.* In this Chapter, effect of addition of latex and steel fibres on the flexural behaviour, ductility, toughness index, energy absorption capacity has been discussed in detail. A method has been proposed to predict the first crack load and ultimate moment of resistance of latex modified steel fibre reinforced concrete flexural members. Conclusions arrived at, based on this study has been discussed in detail.

Chapter 5 deals with the *studies on latex modified reinforced concrete flexural members with confined SFRC in the compression zone.* In this Chapter, effect of addition of latex and confined steel fibre reinforced concrete in the compression zone of the conventionally reinforced concrete flexural members has been discussed in detail.

Chapter 6 deals with *the studies carried out on the cracking behaviour of reinforced concrete flexural members with steel fibres and latex.* To start with, attempts have been made to compare the International equations available in the literature for the prediction of maximum width of cracks with available test results in literature. Subsequently a method has been proposed for estimating the width and spacing of cracks in steel fibre reinforced concrete flexural members. Also an attempt has been made to extend the proposed method for predicting the width of cracks in the case of latex modified SFRC flexural members.

In Chapter 7, *the conclusions* arrived at, based on the experimental investigation have been given.

In Appendix I, II, III, and IV model calculations for

(i) Prediction of first crack load and ultimate moment of resistance in the case of latex modified steel fibre reinforced concrete flexural members.

(ii) Details of test results used for comparing the International Code equations and prediction of maximum width of crack in the case of reinforced concrete

(iii) Prediction of spacing and maximum width in the case of steel fibre reinforced concrete flexural members and

(iv) Prediction of spacing and maximum width in the case of latex modified steel fibre reinforced concrete flexural members

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction

The present investigation deals with the studies on polymer modified steel fibre reinforced concrete. Already polymer composites, steel fibre composites and polymer modified steel fibre reinforced concrete have been introduced briefly in Chapter 1. As flexural members play an important role in a structural system, specific attention has been given to study the effect of polymer modified steel fibre reinforced concrete on the behaviour of the same. An attempt has been made to review briefly the available literature on the following topics with special reference to flexural members.

i) Steel fibre reinforced concrete
ii) Polymer modified concrete
iii) Confined Concrete and
iv) Cracking of reinforced concrete

A large number of investigations are available in literature on the mechanical properties, durability etc. on the above topics and have been reviewed by other researchers [4,42,65,77]. As they are not within the purview of the scope of the present investigation, they are not reviewed in this Chapter. Since the aim of the present investigation is to study the flexural behaviour of polymer modified steel fibre reinforced concrete members, only those investigations related to the flexural behaviour of conventional reinforced concrete members additionally reinforced with steel fibres or polymers and provided with confinement in the compression zone have been discussed in detail. The scope of the present investigation is given at the end of this Chapter.

2.2 Steel Fibre Reinforced Concrete

Fibre reinforced concrete is made of hydraulic cement containing fine or fine and coarse aggregates and discontinuous discrete fibres [1].

It appears that *Romualdi* and *Batson* [81] were the first to incorporate aligned steel fibres to serve as a crack arresting mechanism. The continuous steel wires were aligned on the tension side of the beams and they found that the fracture arrest could be achieved by reducing the spacing of reinforcement to a suitable scale. Also from theoretical studies, they showed that the tensile cracking strength increased proportionately to the inverse square root of the spacing of reinforcement.

Based on their experimental investigation, *Shah* and *Rangan* [89] proposed a model to explain the reinforcing mechanism of fibre reinforced concrete. Different volumes, lengths, orientation and types of fibres were used. They compared fibre reinforced concrete with conventional reinforced concrete in flexure, tension and compression. They observed a significant increase in reinforcing effect of fibres after the cracks are initiated in the matrix, just as with conventional tensile and stirrup reinforcement. The post cracking resistance of fibres was considerably influenced by their length, orientation and stress-strain relationship. They have analytically predicted the reinforcing action of fibres by using composite materials approach based on the properties of individual components.

An experimental and analytical investigation of the mechanical properties of cement mortar reinforced with randomly oriented short steel fibres was carried out by *Pakotiprapha* et al [66]. Analytically, the material was treated as a composite and explicit expressions were derived for its properties in flexure, torsion, axial compression and tension by the law of mixtures.

An investigation into the mechanics of steel fibre reinforced concrete and its behaviour under uniaxial tension was carried out by *Rajagopalan* et al [76]. Equations were derived to predict the ultimate strength in flexure of steel fibre reinforced concrete beams with uniformly dispersed and randomly oriented fibre reinforcement. The effect of fibre reinforcement on the strength at first crack and also on ductility and ultimate strength was also studied. From their study, they have observed a significant increase in flexural strength by the inclusion of steel fibres in the tension zone only.

Ramakrishnan et al [77] carried out an extensive experimental investigation on the performance characteristics of fibre reinforced concrete which included compressive strength, static flexural strength including deflection, modulus of rupture, load deflection curves, determination of first crack load, post cracking strength, flexural fatigue, ultimate failure, pulse velocity and static and dynamic moduli of elasticity and plastic mixing. They concluded that there was no "*balling*" effect of fibres during mixing and placing. Fibre reinforced concrete had better finishability and was easy to work with even at higher fibre concentrations. Due to the addition of steel fibres, the ductility and the post crack energy absorption capacity were greatly increased. There was a tremendous increase in static flexural strength and a very significant improvement in the flexural fatigue strength. They have noticed a considerable improvement in the endurance limit due to the addition of steel fibres to conventional concrete.

Experiments by *Swamy* and *Al-Ta'an* [96], reported the influence of fibre reinforcement on the deformation and ultimate strength in flexure of reinforced concrete beams. The fibre concrete was provided either over the whole depth of the beam or in the effective tension zone surrounding the steel bars. An ultimate strength theory based on the British and American Codes was given, taking into account the increased steel strain at failure. This theory was based on the conventional compatibility and equilibrium conditions used for normal reinforced concrete, except that the effects of steel strain hardening and the contribution of steel fibres in the tension zone are recognised.

Henager [34] described a method for estimating the ultimate strength of reinforced concrete beams with randomly oriented steel fibres. Strength of two conventionally reinforced beams and six full size reinforced fibrous beams were tested experimentally to verify the method. Both cold drawn steel fibres and newly developed melt extracted fibres were used. About 25% increase in the moment capacity has been noticed in fibrous concrete beams. The post cracking stiffness of the reinforced fibrous beams was greater than conventionally reinforced beams. The crack width and crack spacing were less in reinforced steel fibre reinforced concrete beams and the first cracks occurred at loads 1.33 to 1.67 times greater than the conventional beams.

In order to find the application of fibre reinforced concrete in the design of blast resistant structures, *Parameswaran* et al [69] studied the behaviour of steel fibre reinforced concrete beams having equal compression and tension steel, adequately designed to avoid shear failure. SFRC beams were reported to have much better load dispersion characteristics as compared to normal reinforced concrete beams. Empirical expressions for the static rigidity of the beams have also been derived.

Paramasivam et al [68] have given idealised stress- strain curves for steel fibre concrete and reinforcing steel and derived simplified analytical expressions for the moment curvature and load deflection behaviour of simply supported reinforced steel fibre concrete beams in flexure. Analytical expressions were verified by testing experimental beams. Analytically predicted moment curvature and load deflection curves for test beams were found to agree well with the experimental data. The reinforced steel fibre concrete beams showed higher flexural strength and curvature and ductility at ultimate load when compared to similarly reinforced plain concrete beams. Their analytical approach seems to be very useful in the study and design of steel fibre reinforced concrete flexural members.

Santhakumar et al [85] conducted experimental studies on flexural behaviour of prestressed fibrous concrete beams. They tested sixteen beams of prestressed fibrous concrete and conventional prestressed concrete. The beams were tested under monotonic and reversed cyclic loading. The results of beams tested under monotonic loading exhibited superior performance with respect to load carrying capacity, stiffness, ductility and energy absorption capacity for prestressed fibrous concrete members, when compared to the conventional prestressed concrete members.

Dwarakanath and *Nagaraj* [22] conducted experimental studies on the flexural behaviour of large size reinforced concrete beams of 1800mm x 208mm x 100mm size. The beams reinforced with high strength deformed bars, both under-reinforced and over-reinforced, with fibres over the entire depth and over half the depth of the beam in tension side were considered for the investigation. They noticed that partial inclusion of steel fibres over the half depth, in the case of under reinforced beams, was equally beneficial on the full depth inclusion in controlling cracking and deflection and in increasing the stiffness of beam. In the case of over-reinforced beams, fibres used in small quantities were not found to be effective in any appreciable modification in the deformation behaviour of the beams.

Moens and *Nemegeer* [62] explain the procedure for designing fibre reinforced concrete based on toughness characteristics of steel fibre reinforced concrete. They commented that the basic properties of steel fibre are to be well known like for conventional reinforcement, to aid the design of steel fibre concrete. According to the type of application, various requirements like minimum flexural strength, compressive strength, minimum energy absorption capacity of steel fibre concrete etc. must be established for designing the steel fibre concrete. They specify that, in each of these cases, the quantity of fibres needed to ensure that a given reference concrete will comply with the relevant quality requirements can be inferred from the identity charts drawn for a specific fibre type.

Experimental results based on the bond behaviour of normal and high strength concrete with and without fibres have been reported by *Samen Ezeldin* and *Balaguru* [84]. A total of 18 mix proportions were investigated. In their study, they considered silica fume content, fibre length, and fibre content and bar size as variables. The bond tests were conducted using a modified pullout test in which concrete surrounding the bar was in uniform tension. Their experimental investigation revealed that the presence of silica fume resulted in higher bond strength but caused a brittle bond failure. Fibres improved the ductility of concrete to a considerable extent. The slip values corresponding to the maximum pullout load increased with the addition of steel fibres. The addition of steel fibres contributed very little to the bond strength of specimens with small bar diameter than the larger bars.

Antonio Nanni [7] has reviewed the existing literature on torsional performance of steel fibre reinforced concrete with and without conventional reinforcement and proposed design formulas for torsion following the format adopted by ACI 318-1989 [3]. Based on experimental evidence from the survey of literature he found that steel fibre inclusion was shown to improve the torsional resistance of rectangular beams up to approximately 60% compared to beams with or without conventional stirrups plus longitudinal reinforcement. This effect has been considered in the studies to modify the existing ACI 318-1989 torsional formulae.

Parviz Soroushian and *Ziad Bayasi* [73] have reported the results of an experimental study on the relative effectiveness of different types of steel fibres in concrete. The fibres considered in their study included straight-round, crimped-round, crimped-rectangular, hooked-single and hooked collated fibres with an aspect ratio of 75. A constant fibre volume fraction of 2% was used in their investigation. The fresh fibrous mixes were characterised by their slump, inverted slump cone time, workability and their compressive and flexural load deformation relationships. They concluded that the overall workability of fresh mix was found to be largely independent of the fibre type, with crimped fibres producing only higher slump. Hooked fibres were found to enhance slightly the flexural and compressive behaviour of concrete. The crimped fibres have shown slightly less effectiveness in improving strength and energy absorption capacity of concrete compared to straight fibres.

Dwarakanath and *Nagaraj* [23] have studied different methods available for predicting the flexural strength of steel fibre concrete composites. The existing methods have been reviewed and a modified empirical approach has been developed to predict the flexural strength of the composite. The direct tensile strength of the composite has been used as the basic parameter in their approach. The comparative study of the test values of the earlier investigation on fibre reinforced concrete and the computed values from their investigation have given a better correlation and accuracy. The specific advantage of their method is that it requires only the determination of direct tensile strength of the composite, which reflects the combined effect of volume fraction and aspect ratio of steel fibre reinforced concrete composites.

Kumar et al [53] have conducted a comparative study to determine the rotational capacity and toughness of reinforced concrete beams with and without fibres. The beams had equal reinforcement on both the faces and also reinforced in the web. Experimental investigation on sixteen beams with fibres only in the tension zone and fibres in the entire shear span have shown better performance than the beams with steel fibres only in the tension zone. This choice resulted in improved rotational capacity; toughness and ultimate strength compared to conventionally reinforced concrete beams.

Espion et.al [25] have reported the results of experimental research focusing on two possible methods to enhance the service load behaviour of ordinary reinforced concrete structural elements, i.e. the use of steel-fibre concrete and the use of high-strength concrete. The programme involved the testing of nine reinforced concrete beams with rectangular cross-section (b=250mm, h=150mm) and span L=1400mm. The reinforcement ratios considered are ρ = 0.33, 0.52 and 0.75%. They also tested a fibre-reinforced concrete beam with no bar reinforcement to determine the material properties of fibre concrete. Test results show that the use of high-strength concrete seems to be more effective than the use of fibre concrete to reduce the deflections at service load level. They conclude from these tests that the tensile properties of cracked fibre-reinforced concrete are too limited to reach the level of deformation that is required in safe reinforced concrete design and that the use of steel fibres as complementary reinforcement of rebars should be avoided.

Trottier et al [99] describes an experimental program in which four deformed commercial fibres with widely different geometries were investigated in steel-fibre reinforced concrete. Three matrices with compressive strengths of 42, 52, and 85 MPa were reinforced with fibres at a dosage rate of 40 kg/m^3. Compressive and flexural strengths were measured along with the elastic moduli. The focus of the study, however, was to measure and characterise the toughness improvements in the basic matrices due to the addition of various fibres. To this end, flexural load-deflection curves were analysed in accordance with the ASTM and Japan Society of Civil Engineers (JSCE) standard methods and also using a proposed-analysis scheme. Their study points out the limitations of the current techniques of toughness characterization and identifies this, as an area with immediate research needs. For the fibres and the matrices investigated, a strong influence of both fibre geometry and matrix strength on the toughness characteristics of fibre-reinforced concrete was observed. End-deformed fibres were, in general, found to perform superior to those deformed throughout the length.

Gopalaratnam et al. [32] have presented a summary of the available methods of characterising the flexural toughness of fibre reinforced concrete (FRC), with a review of most of the toughness standards and guidelines from standards institutions and other professional agencies in North America, Europe and Japan. Also they have reviewed other significant proposals available in the published literature. They have also discussed merits and drawbacks of these measures. Other related issues discussed include the fundamental significance, problems with regard to experimental measurements and the potential for practical design implementation of a toughness measure.

Chenkui et al. [16] conducted experimental investigations to study the properties, such as tensile, compressive, flexural strength, flexural toughness and flexural fatigue strength, of steel fibre reinforced concrete containing larger aggregate with maximum size of 40 mm. More than 400 specimens were tested, the results of the tests showed that the properties of the fibre concrete might approach those of fibre concrete containing small aggregate, when the size of steel fibre and the grading of aggregates were rationally selected for the mixture of fibre concrete. Based on the test results, some formulae were proposed to predict the properties of steel fibre concrete with larger crushed stone.

Ezeldin et al [26] conducted the analytical studies on immediate and long-term deflections of fibre-reinforced concrete beams. They report that the Addition of discrete steel fibres to concrete enhances its properties, especially in the areas of serviceability and toughness. With the increasing use of shallow sections made of high-strength fibre concrete and capable of meeting the strength requirements, deflection behaviour becomes an important factor that can control the design. They have presented an analytical method that predicts the moment-curvature and load-deflection relationships for beams made of fibre concrete and containing conventional reinforcement. The proposed method evaluates the immediate deformation as well as the long-term deformation as affected by creep and shrinkage. The tension stiffening effect is incorporated to obtain a better prediction of the curvature and deflection. The analytical algorithm proposed to generate the complete moment-curvature and load-deflection curves provides a good correlation between predicted values and experimental test data reported in the literature.

Wang et al [100] conducted studies on Fibre reinforced concrete beams under impact loading. Impact tests were carried out on small concrete beams reinforced with different volumes of both polypropylene and steel fibres. The drop height of the instrumented drop-weight impact machine was so chosen that some specimens failed completely under a single drop of the hammer, while others required two blows to bring about complete failure. It was found that, at volume fractions less than 0.5%, polypropylene fibres gave only a modest increase in fracture energy. Steel fibres could bring about much greater increases in fracture energy, with a transition in failure modes occurring between steel fibre volumes of 0.5% and 0.75%. Below 0.5%, fibre breaking was the primary failure mechanism and the increase in fracture energy was also modest; above 0.75% fibre pullout was the primary mechanism with a large increase in fracture energy.

Li et al [55] conducted studies on tensile behaviour of cement-based composites with random discontinuous steel fibres. The tensile properties of cement-based composites containing random discontinuous steel fibres are reported. Direct tensile tests were performed to study the effects of fibre length (hence fibre aspect ratio), interfacial bonding, and processing conditions on composite properties. Composite tensile strength and ductility are highlighted and discussed.

Hughes et al [36] studied on the impact energy absorption at contact zone and supports of reinforced plain and fibrous concrete beams. The total energy absorbed by a reinforced concrete beam when struck by a hard impactor depends in part on the local energy absorbed both in the contact zone and by the impactor. Test have been performed on both beam segments and reinforced concrete beams to ascertain the extent to which the local effects in nominally hard impacts affect the total energy absorbed. The beam segments were rigidly supported along their length and were impacted by the same solid impactor as that used for the flexural beam tests. The energy absorption on initial indentation of the concrete in the contact area is shown to be very small for either plain or fibre concrete. A simplified approach for designing reinforced concrete beams for impact by quantifying the energy absorption from the moment-rotation characteristics is also outlined.

Tan et al [97] investigated the behaviour of partially prestressed beams with steel fibres, alone or in combination with stirrups as shear reinforcement. A test program was carried out with the partial prestressing and the shear span-to-effective depth ratio, and the steel fibre content of the beam as major parameters. The influence of the various parameters on beam behaviour is discussed. Test results indicate that stirrups may be replaced by an equivalent amount of steel fibres without affecting the stiffness, shear strength, and cracking behaviour of the beam. The equivalence of steel fibres to stirrups is determined from the consideration of equilibrium of a cracked element, and a simple equation is proposed for the prediction of the shear-carrying capacity of partially prestressed steel fibre concrete (SFC) beams.

Filiatrault et al [28] conducted studies on seismic behaviour of steel-fibre reinforced concrete interior beam-column joints. The use of steel-fibre reinforced concrete to improve the behaviour of beam-column joints during earthquake excitation was investigated. Results of quasi-static tests on three full-scale interior beam-column joints and part of a prototype building designed according to the National Building Code of Canada are presented. The first specimen was made of normal concrete but ignored the special seismic recommendations related to the spacing of lateral reinforcement in the beams, column, and joint. The second specimen was also made of normal concrete and included full seismic details. The third specimen was similar to the first one but incorporated hook-end steel fibres in the joint region. Experimental results indicated that steel fibres bridging across cracks in the concrete mix increase the joint shear strength and can diminish requirements for closely spaced ties.

Spadea and *Bencardino* [92] studied the behaviour of composite concrete sections reinforced with conventional steel bars and steel fibres, and subjected to flexural cyclic loading beyond the yield point of steel bars, and analysed by means of a mechanical model. The stress-strain relationships for the concrete, for the steel fibre, and for the steel bars are assumed to be piecewise linear. These constitutive laws are used to obtain the primary moment-curvature relationship of the section for a monotonically increasing load up to failure. On this basic curve, the successive stiffness degradation is created as a function of stress and strain levels reached in the section at each load cycle. Strain hardening of steel and the combined effect of confinement ensured by the stirrups and metal fibres are also taken into account. The numerical results, obtained at first for ordinary reinforced concrete sections, are compared with experimental results available in literature. Subsequently, the model is applied to study concrete sections reinforced with steel bars and steel fibres, subjected to a flexural cycle, with identical mechanical and geometrical specifications to the reinforced concrete sections. The equality of the maximum curvature reached to each load cycle for both kinds of sections is imposed, and comparisons are drawn in terms of energy dissipation.

Parameswaran [70] has reported the Research and developmental work in fibre reinforced concrete (FRC) composites. The use of FRC composites started in India during early 1970s has now reached a stage when fibre concrete technology no longer remains confined to laboratory experiments alone but has found significant application in the production of precast concrete components and in in-situ strengthening and repairs of concrete structures. The current applications include flooring and roofing components, pipes, manhole covers and frames, precast thin wall elements, tunnel lining, construction of blast-resistant structures and currency vaults. He concludes that the large-scale application of this material is however, yet to catch-up in India.

Toughness characterisation of fibre-reinforced concrete has been studied by *Taylor* et al [98]. They have reported the strength and toughness measurements on a range of normal and high strength concrete mixes, with and without fibre reinforcement. Cube strength, modulus of rupture, cylinder splitting and torsional-tension test results are reported together with toughness measurements for polypropylene and steel fibre-reinforced concrete. The toughness measurements were carried out via two fracture-type test specimens rather than the traditional four-point loading arrangement on un-notched beams. In the toughness tests, crack mouth opening displacement (CMOD) was measured and used in a closed loop-testing mode to achieve complete load/displacement curves. Three different concentrations of polypropylene and steel fibres were investigated for each nominal grade of concrete (40, 60, 80, 100 and 120 N/mm^2), making 40 mixes in total. The results show that the load/CMOD curves are a good basis for defining toughness since CMOD is measured easily and with less errors than that observed with the more traditional toughness measurements based on load/displacement relationships obtained in deflection-controlled testing machines. Good correlation was observed in the strength tests and the two toughness tests showed similar load/CMOD curves and toughness indices. The effect of fibre reinforcement on high strength and normal strength concretes were found to be similar.

Khaloo and *Kim* [51] conducted studies on the mechanical properties of normal to high-strength steel fibre-reinforced concrete. A total of 84 specimens were tested to study the effect of concrete strength on the mechanical properties of concrete reinforced with randomly distributed steel fibres. The concrete strengths investigated include 25 MPa for normal-strength (NSC), 50 MPa for medium-strength (MSC), and 69 MPa representing high-strength concrete (HSC). Fibre content ranges from 0 to 1.5% by volume of the concrete matrix. The influence of concrete strength on the compressive strength, splitting tensile strength and modulus of rupture of steel fibre-reinforced concrete (FRC) was presented. Based on the limited number of specimens tested, it was concluded that HSC provides considerable improvement in compressive strength for fibre content of up to 1% compared to that of NSC and MSC. Also, modulus of rupture of NSFRC considerably improves due to fibre compared to those of MSFRC and HSFRC. Splitting tensile strength results do not indicate a clear dependency to concrete compressive strength.

2.2.1 Comments on earlier works on steel fibre reinforced concrete

The review of literature on earlier works on steel fibre reinforced concrete and the flexural behaviour of conventional reinforced concrete additionally reinforced with steel fibres reveal the following:

i) The inclusion of steel fibres to cementitious materials improves many of the engineering properties such as first crack strength, tensile strength, fracture toughness, energy absorption capacity etc. and appears to be a useful material and could be applied to typical situations which require high ductility and toughness of the composite.

ii) In the case of conventional reinforced concrete, when steel fibres are added either over the full depth or half depth of the beam, the moment rotation / load-deflection behaviour of the flexural members is significantly improved because of the increase in stiffness of the concrete matrix when reinforced with fibres.

iii) Reviews indicate that there exists an optimum value of fibre content. When fibres are added beyond this value, the overall improvement is not appreciable.

iv) Since the steel fibres intercept the cracks, which propagate from the soffit of the flexural members, the spacing and width of cracks have been found to be influenced by the presence of fibres. This in turn results in the higher fracture toughness of the material, which is one of the basic properties of the material like Poisson's ratio and modulus of elasticity. Also no attempts on studies which represent the physical behaviour of cracking in the conventional reinforced cement concrete members with steel fibres and for determining the spacing and maximum width of cracks in such members are available in literature.

v) In all the previous studies, attempts have been made by different authors to improve the behaviour of the conventional reinforced cement concrete members by suitably adding steel fibres and no attempts to study the combined effect of steel fibres and other additives like polymers are available in literature. Since the addition of polymers also improve many engineering properties of cementitious materials, there is lot of scope for studying the combined effect of steel fibres and polymers on the strength and behaviour of conventional reinforced concrete flexural members.

2.3 Polymers in Concrete

Polymers, a relatively new breed of materials, like in other fields, have found their way into civil Engineering with irresistible benefits. Today they are making a rapid headway in to the arena of concrete. This is happening because, in-spite of their newness, thėy are vast in numbers, and possess the quick and better flexibility to be introduced to meet practically any requirement in the field of concrete construction. In recent years, the use of polymers in concrete is expanding due to the increasing demands from construction industry. They are today used as substitutes or as partial substitutes for cement and as effective materials both individually and in combination with cement.

Polymer is a natural or synthetic chemical compound or mixture of compounds formed by polymerisation and consisting essentially of repeating structural units. In simple words, they are molecules of a particular organic chemical, linked together to form a macromolecule. Polymers are either of biological origin like wood, rubber etc. or non-biological in origin like plastic, nylon, polyester etc.

Broadly there are three types of polymer-concrete composites. They are: -

* Polymer cement concrete (PCC) or
  Polymer modified cement concrete.
* Polymer impregnated concrete (PIC) and
* Polymer concrete (PC)

In polymer cement concrete (PCC), monomers are introduced right at the mixing stage of concrete and they turn into polymers parallely as the hydration of cement proceeds. In polymer impregnated concrete (PIC), the monomers are introduced into the concrete after it is hardened and they polymerize subsequently. Polymer concrete (PC) is a different class of material, in which aggregates are bonded together in a dense matrix of thermosetting polymer binder. Hence it is cementless concrete.

Though there are several investigations available in literature on the modification of cement mortar/ concrete by synthetic rubber latexes, there are only a few investigations on modification of concrete by natural rubber latex.

Joseph A. Lavelle [47] studied the various properties of acrylic latex modified Portland cement. He conducted experiments on the effect of mortar density and curing conditions on the strength of acrylic latex modified Portland cement. The maximum strength properties from a cement mortar were obtained from a highly dense and well-cured mortar. The use of acrylic latex (or any other latex in general) caused a certain amount of air entrainment and lowered the density of the resulting cement mortar. Thus, when modifying a cement mortar with acrylic latex, especially during mechanical mixing, an appropriate amount of defoamer was used to minimize air entrainment and to maximize mortar density. His studies indicate that, to obtain the maximum physical properties, acrylic latex modified cement mortar should be air-dry cured, i.e., they should be cured at ambient room temperature and humidity. One of the reasons is that for the latex to beneficially modify the cement, it must be allowed to coalesce and form a thin film. The loss of water is a key step in this thin film formation process. Once the latex has been allowed to under go film formation, the basic strength properties of the mortar will be achieved.

Reports on the study of feasibility and techniques of impregnating and in-place polymerising of liquid monomers in pre- formed concrete and the use of monomers in a fresh concrete mix, followed by polymerisation are discussed by Dikeou et al [21]. Their studies include development of impregnation techniques and determination of the effect of various polymer loadings on resultant properties. The radiation techniques have shown greater improvements in properties than the thermal-catalytic method. Dramatic improvement in concrete properties resulting from the impregnation with methyl methacrylate is discussed.

Gerry Walter [30], in his study, compared the properties of latex modified Portland cement mortars made using five different types of latex. The latexes used were (1) plastisized polyvinyl acetate homopolymer (PVA), (2) co-polymer of vinyl acetate and ethylene (VAE), (3) carboxylated styrene-butyl acetate co-polymer (SA) and (4) carboxylated butyl acrylate - methyl methacrylate co polymer (SB). These latex modified mortars were compared with w/c ratio, permeability, adhesion, compressive strength and flexural strength, weathering resistance etc.

Sridharan et al [93] conducted investigation on the use of polymer latex for foundation blocks subjected to dynamic loads. Experiments were conducted using ordinary concrete and latex modified concrete footings of three different thicknesses, for three static loads at four excitation levels. The polymer used was natural rubber latex. Experimental results have revealed that the amplitude of resonance is reduced considerably in the latex modified concrete footings. It was also observed that the damping factor of the latex footing soil system is considerably larger than that of the ordinary concrete footings. Hence the use of latex in concrete foundations for machines may lead to more economical designs.

Five different types of polymer dispersions have been studied by *Mangat* and *Swamy* [60]. The strength, stiffness and shrinkage characteristics of polymer modified plain and fibre reinforced concretes have been evaluated. The effect of dry, wet and dry-wet curing on these properties have also been studied. It was also shown that, with proper selection of the type of polymer, modification of the water content and possible use of defoaming agents, polymer dispersion could be used advantageously to improve the properties of fibre- cement composites. The polymers used were DOW Latex 460, DOW Latex 464 (Saran), Corda, Revinex, and Revertex.

Ohama et al [61] carried out extensive experimental investigations on the effect of the monomer ratio on the typical properties of the polymer modified mortars with styrene butyl acrylate latexes. The polymer modified mortars using the styrene butyl acrylate latexes (polymerised with various styrene/ butyl acrylate monomer ratios) were prepared with different polymer cement ratios and tested for pore size distribution, flexural and compressive strength, water absorption and drying shrinkage. The results indicated that superior flexural and compressive strength of polymer modified mortars using styrene butyl acrylate latexes can be obtained. The water absorption characteristics were greatly affected by the polymer-cement ratio rather than the styrene content. By using about 35% styrene butyl acrylate, the drying shrinkage of polymer modified mortars can be reduced to an appreciable extent.

Soroushian et al. [73] conducted experiments on the effect of latex modification on the performance characteristics of carbon fibre reinforced mortars incorporating silica fume. Two styrene butadiene latexes were considered in their investigation. The effects of latex modification on the following properties of CFRC were investigated: Fibre to matrix interfacial bond, flexural and compressive performance, impact resistance, specific gravity, drying shrinkage, freeze-thaw durability and acid resistance. The results indicated that major gain in the bond strength between carbon fibres and cementitious materials were achieved from latex modification. Flexural toughness was also increased through latex modification, but the effect of latex modification on the flexural strength was relatively small. Latex modification was observed to cause reduction in the compressive strength of CFRC composites. The impact strength of unmodified and latex modified composites was comparable. Latex modification resulted in reductions in water absorption, drying shrinkage and specific gravity of CFRC.

Ravindra Rajah [78] investigated the effect of natural rubber latex on the properties of Portland cement paste, mortar and concrete. The results indicated that the latex modification causes retardation of setting: the intensity of the effect depends on the volume concentration of latex in the system. Change in the mix stability improved to a great extent. There was a reduction in the compressive, flexural and tensile strength, increase in air content and decrease in density being the main contributing factors. A reduction in the compressive to tensile strength ratio and an improvement in the extensibility and relatively larger strain at failure were also observed.

Nagaraj et al. [63] conducted an extensive investigation on the development of a method of incorporating natural rubber latex into concrete. Earlier studies had revealed that inevitable drastic reduction in the compressive strength takes place upon the incorporation of natural rubber latex, in relation to plain concrete strength. Hence an attempt was made to increase the strength level of plain concrete mix to offset the strength reduction upon incorporation of latexes. For this, the water-cement ratio was reduced without sacrificing the workability and compactability of concrete by adding high range water reducing admixture (Superplasticizer). The quantity of natural rubber latex was expressed as the dry rubber content (DRC) by percentage of volume of concrete. Their investigation indicated that, with natural rubber latex as an admixture, the ductility of concrete is enhanced with the retention of strength level of plain concrete. The use of Superplasticizer helped to increase the strength of natural rubber latex concrete and the delay in the coagulation of the latex, until the concrete was properly mixed and placed in moulds. It was found that 2.0% dry rubber content induced optimal levels of improvement in ductile behaviour without any reduction in compressive and tensile strengths over those of plain concrete.

Limaye & Kamat [57] conducted some experimental studies by incorporating six types of epoxy polymer combinations in two dosages (10% and 20%) with cement mortar. Five systems were based on epoxy resins and the sixth one was SBR latex modified with surfactants, stabilisers and anti-foaming agents. The experimental setup was planned to study the properties like flexural strength, split tensile strength, stress-strain behaviour. The test results revealed that the addition of latex and epoxy to cement mortar makes it more workable and thus facilitates preparation of low water-cement ratio. Modification with latex has high workability than epoxy. Strength increase was quite significant in epoxy and latex modification system over control specimens. Polymer modification improved the flexural and tensile strength of cement mortar considerably, whereas increase in compressive strength was marginal.

Daniel Bordeleau et al [19] conducted comparative studies of latex modified concrete and normal concrete subjected to freezing and thawing in the presence of deicer salt solution. Latex modified concrete was prepared with 7.5 and 15% of solid polymer to cement ratio. Deterioration of the concrete surfaces was evaluated by measuring mass of the scaled-off particles and by visual rating. Their results indicated that SBR in concrete improves very significantly the resistance of the concrete surface to freezing and thawing in the presence of dicer salts. This improvement depends on the quantity of SBR, the air void spacing factor and the water cement ratio used in the mix. Results also revealed that conventional concrete with a good air void spacing factor and low water-cement ratio can be almost as resistant to salt scaling as latex modified concrete.

Karim et al [48] studied the shear behaviour of steel reinforced polymer concrete using a resin based on recycled poly (ethylene terephthalate) (PET) plastic waste. Tests were conducted on 25 beams that were reinforced in tension zone with longitudinal bars. Steel fibres were also used in some of the beams. The parameters considered in their investigation include modes of failure and the effect of shear span to depth ratio, reinforcement ratio, compressive and flexural strength and steel fibres on the shear strength of the beams. The results indicate good shear strength in reinforced polymer concrete beams using unsaturated polymer resins based on re-cycled PET. The shear span to depth ratio had the greatest effect on the shear behaviour of reinforced concrete beams. The addition of steel fibres to reinforced polymer concrete beams resulted in a ductile failure with no shattering or spalling of concrete. The number of cracks also decreased as the fibre content increased.

2.3.1 Comments on earlier works on polymer modified concrete

Based on the review of literature on polymer modified concretes, the following points are noted:

i) Review of literature indicates that several researchers have tried different polymers ranging from natural to synthetic for improving the properties of cementitious materials.

ii) Most of the studies have been directed towards the understanding of the basic properties of the composites when polymers are used. Some researchers have done extensive durability studies on polymer modified concrete.

iii) These earlier works indicate that the strength and durability of concrete could be enhanced appreciably by adding polymers.

iv) The survey of literature indicates that most of the studies are limited to short term behaviour of polymer composites with polymers alone as additives.

v) Studies on natural polymers like latex modified concrete indicate that the ductility of the composite could be increased by adding natural rubber latex and hence could be used in situations, which require adequate strength and high ductility.

vi) In the case of latex modified concrete, the review indicates that addition of polymers beyond 2% DRC (Dry Rubber Content), does not improve the strength of the composite and infect leads to a reduction in strength at higher values of DRC.

vii) As stated in an earlier section (Section 2.2.1) most of the studies deal with only one additive and only in a few investigations, the combined effect of polymers and fibres on the behaviour of concrete have been tried.

viii) In the case of steel fibre-natural polymer concrete, till date only one investigation on the strength and behaviour of steel fibre-latex modified concrete is available [63] and it is limited to smaller size plain concrete specimens only. No attempts have been made to study the combined effect of steel fibres and polymers on the flexural behaviour of conventional reinforced cement concrete beams.

2.4 Confined Concrete

Confined concrete is that concrete, which is confined by transverse reinforcement in the form of closely spaced steel spirals of square or circular hoops. The concrete gets confined when at stresses approaching the Uni.-axial strength; the transverse strain becomes very high because of the progressive internal cracking and the concrete bears out against the transverse reinforcement, which then applies a confining reaction to the concrete. Thus transverse reinforcement provide passive confinement to concrete [71].

From the experimental investigations on confined concrete prisms and columns, *Chan* [15] proposed an equation for the strength of confined concrete in terms of strength gain factor (k) and another one for strain at peak/ultimate load (ε_u) in the columns when concrete carries the maximum load. These parameters were suggested as functions of the volumetric ratio of the steel to concrete core.

Roy and *Sozen* [82] carried out tests on concrete prisms confined with rectangular ties and concluded that the confinement by rectangular ties doesn't enhance the strength of concrete. Only two variables viz. volumetric ratio of confining steel to concrete core and the ratio of the shorter side dimension of the compressed concrete section to the spacing were considered by them.

Soliman and *Yu* [91] conducted studies on confined concrete prisms under eccentric loading. They concluded that the effect of confinement couldn't be solely expressed as a function of the volumetric ratio of confinement. Based on their studies, they proposed the stress strain relationship of the confined concrete. It consisted of a parabola and two straight lines with stresses and strains at the critical points related to transverse steel content, spacing and the confined area.

Sargin et al [86] investigated the effect of confinement of concrete by rectangular lateral reinforcement on the strength of concrete. The main variables considered were strength of concrete, size, spacing and grade of lateral reinforcement, strain gradient and thickness of cover. They concluded that the effect of confinement becomes negligible when the spacing of lateral reinforcement is larger than the dimension of the concrete core. Also they have proposed a general equation that gives a continuous stress-strain curve related to the spacing of ties, content and yield strength of transverse steel, the strain gradient across the section and concrete strength.

The experimental studies by *Scott* et al [87] revealed that substantial enhancement of peak strength of concrete was obtained as a result of confinement of concrete. At low strain rate, the strength enhancement was about 70% related to the unconfined cylindrical compressive strength. They modified the stress strain model proposed by *Kent* and *Park* [50] for high strain rate, in order to account for the increase in strength and ductility.

Shah and *Ahmed* [88] conducted experimental studies on concrete confined by spiral reinforcement and developed a mathematical model for the stress strain relationship based on the properties of hoop reinforcement and constitutive relationship of the plain concrete.

Mander et al [59] has developed a theoretical stress strain model for confined concrete, applicable for members with either circular or rectangular cross section and with static or dynamic axial compressive loading either monotonically or cyclically applied.

Base and *Read* [8] conducted tests on reinforced and prestressed concrete beams by mid span loading to investigate the efficiency of helical reinforcement in the compression zone as a means of improving the moment rotation characteristics of the resulting plastic hinges.

Iyengar et al. [42,43,44,45] developed a stress block for confined concrete in compression from the test results of confined concrete cylinders and prisms, subjected to uniaxial compression and flexure respectively. They concluded that:

1. The confinement offered by steel binders can be quantitatively expressed by factor termed the confinement Index (C_i) which is defined by

$$C_i = (\rho_b - \bar{\rho}_b)\frac{f_y}{f_c} \qquad \text{...(2.1)}$$

2. For specimens with circular spiral confinement, the ultimate strength of confined concrete $\bar{f}_c$ is given by

$$\frac{\bar{f}_c}{f_c} = 1 + 2.30\,(\rho_b - \bar{\rho}_b)\frac{f_y}{f_c} \qquad \text{...(2.2)}$$

These results and the data of strain at 90% of the maximum stress were used in developing a stress block.

Ziaria et al [104] conducted extensive studies on flexural members with confinement in the compression zone. A method for the design of over reinforced beams utilising the ductility resulting from the confinement has also been outlined and investigated experimentally using four types of over-reinforced beams. The results obtained have shown that, although the beams with confinement were able to achieve a flexural capacity of up to 246% of the value corresponding to the maximum longitudinal reinforcement ratio (r_{max}) allowed by ACI code, they still failed in a ductile manner.

Irawan et al [39] conducted studies on the three-dimensional finite element analysis of concrete columns laterally confined by steel ties and hoops. The strength gain is numerically investigated in using the elasto-plastic and fracture model for concrete. The uniformity of the confinement stress and the damage induced are enlightened in consideration of the confinement efficiency by the discretely distributed lateral steel ties and hoops. The sectional averaged lateral stress in concrete, the minimum of which along the axis of columns governs the capacity of the entire confined columns, is found to be affected by the volumetric averaged lateral stress of concrete as well as the spacing of the tie associated with the uniformity of stress states. The authors demonstrate that the spacing of lateral ties also influences the volumetric averaged confinement of concrete, which mathematically corresponds to the axial mean value of the sectional averaged confinement stress.

Watson et al [101] conducted studies on confining reinforcement for concrete columns. Previously derived stress-strain relationships for compressed concrete confined by various quantities and arrangements of transverse reinforcement are used in cyclic moment-curvature analysis of a range of reinforced concrete columns to derive design charts. The design charts permit the enhanced flexural strength of confined columns to be obtained. They also permit the quantities of transverse reinforcement required to achieve particular curvature-ductility factors in the potential plastic-hinge regions of reinforced concrete columns to be determined. The column section is considered to have reached its available ultimate curvature when either the moment resisted has reduced to 80% of the ideal flexural strength, or the strain energy absorbed in the transverse reinforcement has reached its strain energy absorption capacity, or when the longitudinal steel has reached its limiting tensile or compressive strain, whichever occurs first. Refined design equations to determine the quantities of transverse reinforcement required for specified ductility levels were derived on the basis of design charts. The equations are an improvement on the current provisions of concrete design codes.

An analytical model was developed to predict the complete stress-strain relationship of normal and high-strength concrete subject to uniaxial compressive loading and confined by transverse reinforcement by *El-dash,* and *Ahmad,* [24]. The confinement pressure is assumed to be uniform within the core of the column. The internal force equilibrium, the properties of the materials, and the geometry of the section were used to evaluate the pressure. The model utilises a single fractional equation, which satisfies realistic behaviour of the ascending portion, the peak point, and the post-peak descending portion of the stress-strain relationship. The predictions of the model and the available experimental results are compared over a range of concrete strengths. The model shows good predictive capability and is applicable for a wide range of variables such as the diameter of the circular section, the pitch, diameter and yield stress of the spiral, and the volumetric ratio of lateral reinforcement.

Fang et al [27] conducted studies on the strength and ductility of high strength tied columns subjected to uniaxial compressive load. 40 numbers of 250 x 250 x 1000mm HSC columns, in which 24 were laterally reinforced, and 16 were without reinforcement, were tested. Five additional normal strength concrete columns of the same size were tested as reference specimens. The compressive strengths of HSC varied from 48.3 MPa to 82.7 MPa (7,000 psi to 12,000 psi). The main variables were concrete strength (f_c), type of lateral reinforcement, and spacing of ties. The behaviour of tie and cross tie as well as the effectiveness of confinement is discussed. Test results revealed that the beneficial effect of lateral reinforcement on strength increase and ductility improvement was not so pronounced in the HSC specimens as in the normal strength concrete specimens The strains of tie and cross tie increased as the spacing of ties decreased and could be stretched to yield value if smaller spacing was adopted. An empirical equation is proposed to predict the peak stress and the stress-strain curve of confined HSC tied columns. Reasonable correlation between the predicted and tested results is obtained.

Hirasawa et al [33] conducted tests and analysis on the ultimate strength of short columns with confining reinforcement under biaxial bending. Main object of this study is to estimate the shape effects of internal hoops on ultimate capacity of the R/C column under biaxial bending. Nine specimens, each 300 mm square by 1,000 mm high, containing either 8 or 12 longitudinal steel bars and different arrangements of square or octagonal steel hoops, were tested under uniaxial or biaxial eccentric loads. From the results of the test and analysis, it is found that the analytical values calculated by element division method agree well with that of experiments.

Cusson et al [18] developed a stress-strain model for confined high-strength concrete and calibrated against the test results from 50 large-scale high-strength concrete tied columns tested under concentric loading. The effects of the concrete compressive strength, tie yield strength, tie configuration, transverse reinforcement ratio, tie spacing and longitudinal reinforcement ratio are accounted for in the proposed stress-strain model. The determination of the strength and ductility of confined concrete is based on the computation of the effective confinement pressure, which depends on the stress in the transverse reinforcement at maximum strength of confined concrete, and on the effectively confined concrete area. A method is proposed to compute the stress in the transverse reinforcement at maximum strength of confined concrete.

Papadopoulos [67] studied truss model for the confinement of concrete columns. In order to investigate the dependence of the ductility of a concrete column on its confinement by transverse reinforcement, a part of a column, subjected to axial compression, between two successive confined sections, is simulated by a plane truss, with bars obeying non-linear sigma - epsilon laws of concrete or steel. The equilibrium conditions are written with respect to the deformed structure, so that instability phenomenon is taken into account. The proposed model was applied on a column with square section, first unconfined and then with increasing confinement, for various values of spacing of transverse reinforcement. Finally, a method of preliminary design, based on the truss model, was proposed, for the pre-estimation of minimum required section of transverse reinforcement assuring ductility.

2.4.1 Comments on the earlier work on confined concrete

The review of literature on confined concrete reveals the following:

i) In general, confining the concrete in the compression zone with either square or spiral hoops can increase the strength and ductility of conventional reinforced cement concrete beams.

ii) Addition of steel fibres to the concrete in the confined compression zone significantly increases the strength and energy absorption capacity. The addition of steel fibres does not seem to increase the strength of the specimen significantly. However, ductility was found to increase considerably with the increase in volume fraction of steel fibres.

iii) The inclusion of steel fibres in the compression zone has significant influence on the ductility only at higher values of confinement. Whereas this influence is very little at low confinement.

iv) The effect of combination of several material properties like volume fraction of fibres, volumetric ratio of confinement, yield strength of confining steel and strength of concrete can be represented by a single parameter called "Confinement Fibre Index".

v) The experimental investigations have revealed that the brittle behaviour of over-reinforced flexural members can be converted to ductile by confining the compression zone along with the addition of steel fibres.

vi) Both synthetic and natural rubber latex can be used to improve the various properties of concrete like strength, ductility, energy absorption capacity etc.

vii) Modification by natural rubber latex causes retardation of the setting of concrete and the intensity of this effect depends on the volume concentration of latex in the system.

viii) The earlier studies have indicated that, the addition of natural rubber latex resulted in enhancement of ductility of concrete with the retention of strength level of plain concrete. It was found that 2.0% dry rubber content induced optimal levels of improvement in ductile behaviour without any reduction in compressive and tensile strengths over those of plain concrete.

2.5 Cracking of Reinforced Concrete

An attempt is made to review the literature available on the studies of cracking of reinforced concrete. A brief review of specifications given in certain International Codes of practice related to cracking is also made. Even though a large number of investigations available on cracking of two way slabs, tension members etc., they have not been reviewed in this Chapter, as they are not within the purview of the scope of the present investigation. Hence only the cracking behaviour of reinforced concrete flexural members such as beams and one way slabs are considered in this study.

2.5.1 Studies on cracking of reinforced concrete flexural members

Development of cracks in the tension zone of members subjected to flexure or axial tension has always been viewed with concern. The occurrence of cracks in reinforced concrete structures is inevitable because of the low tensile strength of concrete. With the advent of high strength steel, tensile strain in the concrete surrounding such reinforcement will be of the order of 0.001 even under service loads. Also, reinforcement becomes effective only when the surrounding concrete cracks. However, large scale cracking is not acceptable in view of aesthetic considerations, to ensure water tightness or gas tightness and to safe guard the reinforcement against corrosion. For efficient control of cracking, accurate prediction of crack width under different stages of loading is essential. Hence studies on cracking have attracted many investigators and different theories have been proposed for explaining the mechanism of cracking. Since many variables influence the width and spacing of crack in reinforced concrete members, the theories proposed are of approximate, semi-empirical and empirical nature and some are reviewed below.

2.5.2 Classical theory or bond - slip hypothesis

This is the earliest theory proposed for the mechanism of cracking developed from the observation of surface cracks. It is based on the assumption that the bond stress on the concrete reinforcement interface leads to the development of tensile stress in the concrete. Tensile stresses are uniformly distributed over the concrete section and a critical section fails when the average tensile stress exceeds the tensile strength of concrete. The bond between steel and concrete principally controls the width and spacing of cracks.

2.5.3 No slip theory - Studies from UK

BASE et al [8] proposed a fundamentally different approach from the classical theory of bond slip hypothesis. This theory assumes that there is no slip of the steel relative to concrete, for the range of crack width normally permitted in the reinforced concrete. The crack is assumed to have zero width at the surface of the reinforcing bar and an increase in the width as the surface of the member is approached. Crack width is dependent on the deformation of the surrounding concrete. Theory of elasticity can be used to determine the stress and strain in the concrete between the cracks.

Based on the test results of 105 beams, BASE et al proposed the following formula for predicting the maximum crack width on the surface of the beam. Their studies also revealed that the type of reinforcing steel had a much smaller influence on the crack width.

$$W_{\max} = 3.3\ c\left(\frac{f_s}{E_s}\frac{h_2}{h_1}\right) \qquad \ldots(2.3)$$

2.5.4 General theory of cracking

The studies of BEEBY [10] have resulted in a clear understanding of the mechanism of cracking. Beeby measured crack widths and spacing at various points across the bottom of one-way slabs. He found that the crack spacing and crack width increased with the distance from the bar and at some distance from the bar approached constant values, which were dependent on the crack height rather than the distance from the bar.

Based on this hypothesis, BEEBY [10] proposed equations for spacing and maximum width of cracks. Since the theory has been developed for cracks that cross the reinforcing bars at right angles, it is not directly applicable to a situation when the cracks cross the reinforcing bars at an angle.

2.5.5 Studies of Broms

Broms [11] proposed a cracking mechanism based on the elastic analysis of concrete structures. The effect of concrete cover on the spacing and width of cracks was investigated. He showed the presence of high tensile stresses within an area located inside a circle that is inscribed between two adjacent pre-existing cracks. Propagation of cracks was related to the sequence of cracking and spacing between cracks. Based on his studies, Broms proposed equations for the average crack spacing and average crack width, which consider the depth of concrete cover as an important variable.

2.5.6 Statistical approach - Studies from USA

The maximum crack width measurements made by number of investigators were examined using statistical methods. The equations proposed earlier were compared and new equations were proposed with the result of this analysis. Due to the relatively large scatter in the width of largest cracks and due to the large number of variables present, agreement was lacking among the investigators as to the most important variables influencing the size of the crack.

Gergely and *Lutz* [31] made extensive statistical analysis of crack width observed in the experimental investigations and proposed equations for predicting the maximum crack widths on the surface of the members reinforced with deformed bars. These equations are explained in Section 2.6.4.

2.5.7 Other studies:

Desayi and *Ganesan* [20] proposed a method to determine the crack spacing and maximum crack width in reinforced concrete flexural members and the constants appearing in the equations were determined from the statistical analysis of test results available in literature.

Zonjian and *Dajun* [105] developed new formulae for predicting crack width on the basis of test data obtained from 205 reinforced concrete members. Based on their investigation, they concluded that

i) For section having same concrete cover and other variables such as concrete strength and steel strength being same, the average crack spacing increases approximately linearly with increasing d/ρ_{te} where d is the diameter of the reinforcing bar and ρ_{te} is the percentage of reinforcement in relation to the effective of tensile concrete

ii) If d/ρ_{te} and other variables remain the same, the average spacing of cracks increases approximately linearly with increase in thickness of concrete cover.

The average crack spacing and maximum width of cracks calculated using the equations proposed by them have been compared with the test results and found to compare satisfactorily. These equations have been incorporated in the Chinese Code for concrete structures.

Hwan - Oh and *Young - Jin Kang* [37] proposed formulae for predicting the maximum crack width and average crack spacing in flexural members based on the recently developed cracking theory by non-linear Finite Element analysis. A series of tests on R.C beams were conducted. The test results were compared with the proposed equations and a good correlation was observed.

Ishibashi et al [41] conducted studies on estimation of bending crack width on the surfaces of concrete girders. Crack width on the surfaces of concrete girders is increased by partial drying shrinkage near the surfaces after initial cracking. And, reinforcement stress, that is the main factor of bending crack width, is varied by the influence of drying shrinkage and creep. A practical method of calculation of the bending crack width on the surfaces of concrete girders has been reported.

Zhao et al [102] have re-evaluated the present equations for flexural crack width of RC beams. Several equations have been developed for estimating the flexural crack widths of RC beams. Since most of them are semi-empirical, their applicability should be examined whenever new types of structure come into consideration. In their study they have discussed firstly the applicability of present equations for flexural crack widths to the beams with multi-layers of longitudinal bars. The examination of applicability is extended using test results of 86 beam specimens, which are deliberately selected so as to represent wide variety of arrangement of reinforcing bars. Based on the discussion the authors proposed the equations for crack spacing and crack width

Zhao et al [103] conducted experimental study of flexural cracking of RC beams with multi-layers of longitudinal bars. With request for larger RC structures and structural members in these days, longitudinal reinforcing bars are required to be large in size and to be placed in multi-layers in the cross section of members. This trend makes it necessary to examine the applicability of present code equations for flexural crack widths of RC beam. Based on the experimental test results, it was discussed in their study, how the multi layers of longitudinal bars influence the flexural cracking of RC beams. Special attention has been given to how the location and the size of bars as well as the bounded bars influence the cracking behaviour. The test results show that the crack spacing and the crack widths of beams depend upon how the longitudinal bars are placed although the bars nearest to the concrete surface have dominant effects on cracking.

2.6 International Code Specifications Related to Cracking

2.6.1 I.S Code - 456 - 1978 [40]

Clause 34.2.2 of I.S 456 - 1978 Code deals with cracking. It suggests that cracking should not adversely affect the appearance or durability of the structure. Acceptable limit of cracking would vary with the type of the structure and the environment and the Code suggest that the surface crack width should not in general, exceed 0.3 mm. For structures lying in aggressive environment, the surface crack width at points nearest to the main reinforcement should not exceed 0.004 times the nominal cover to the reinforcement as per the Code.

I.S 456- 1978 Code does not suggest any method for computing the crack width. Clause 42.1 suggests that the flexural members compliance with spacing requirements of the reinforcement as per the clause 25.3.2 should be sufficient to control flexural cracking.

2.6.2 BS 8110 - 1985 Equation [12]

The British Code suggests an expression for calculating the design surface crack width, provided the strain in the tension reinforcement is limited to $0.8f_y/E_s$ and the design surface crack width, which should not exceed the appropriate values given for the appearance and corrosion (0.3 mm) as-

$$W_{bt} = \frac{3\, a_{cr}\, \varepsilon_m}{1 + 2\,\dfrac{(a_{cr} - c_{min})}{(h - x)}} \qquad \text{...(2.4)}$$

where a_{cr} = distance from the point considered to the nearest longitudinal bar

ε_m = average strain at the level where cracking is being considered

C_{min} = minimum cover to tension steel

h = overall depth of member

x = depth of neutral axis.

Average steel strain ε_m is calculated from the equation:

$$\varepsilon_m = \varepsilon_l - \frac{b_t (h - x)(a' - x)}{3 E_s A_s (d - x)} \qquad \ldots(2.5)$$

where ε_l = strain in steel at the level considered ignoring the stiffening effect of concrete in the tension zone

b_t = width of the section at the centroid of tension steel

a' = distance from the compression face to the point at which the crack width is being measured.

2.6.3 Model Code 1990 equation [13]

As per the Model Code 1990, for all stages of cracking, the design crack width may be calculated according to the following expression:

$$W_k = l_{s,max} (\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \qquad \ldots (2.6)$$

where $l_{s,max}$ = length over which slip between steel and concrete occurs

ε_{sm} = average strain in steel within $l_{s,max}$

ε_{cm} = average strain in concrete within $l_{s,max}$

ε_{cs} = strain of concrete due to shrinkage which has to be introduced algebraically

$l_{s,max}$ is calculated from the following conditions:

If $\rho_{sef}\ \sigma_{s2} > f_{ctm}(t)(\ 1 + \alpha_e\ \rho_{s,ef})$, it may be assumed that the stabilised cracking condition has been reached, otherwise the formation of single crack should be considered.

Where $f_{ctm}(t)$ = mean value of the tensile strength of concrete at the time 't' when the crack appeared

α_e = ratio E_s/E_c

$\rho_{s,ef}$ = effective reinforcement ratio (= $A_s/A_{c,ef}$)

$A_{c,ef}$ = effective area of concrete in tension (area of concrete surrounding the tension reinforcement)

σ_{s2} = steel stress at crack

$l_{s\ max}$ is calculated from the following equations:

$$l_{s,\max} = \frac{\phi}{3.6\ \rho_{s\ eff}} \qquad \textit{(for stabilized cracking)}$$

....(2.7a)

$$l_{s,\max} = \frac{\sigma_{s2}}{2\ \tau_{bk}}\ \phi\ \frac{1}{1+\alpha_e\ \rho_{s,ef}} \qquad \textit{(for single crack formation)}$$

....(2.7b)

where φ = diameter of bar

According to the Code, in the absence of a more refined model the effective area of concrete in tension $A_{c,ef}$ is to be taken as

$A_{c,ef}$ = 2.5(h-d)b subjected to a maximum of ((h-x)/3)b

In equation (2.6), $\varepsilon_{sm} - \varepsilon_{cm} = \varepsilon_s - \beta\, \varepsilon_{sr2}$

Where ε_s = steel strain at the crack

β = 0.6 for short term / instantaneous loading.

and ε_{sr2} is given by the equation

$$\varepsilon_{sr2} = \frac{f_{ctm}\,(t)}{\rho_{s,\,ef}\;\; E_s}\,(1 + \alpha_e\, \rho_{s,\,ef}) \qquad \ldots(2.8)$$

2.6.4 ACI Code 318 - 1995 equation [3]

The specification given in ACI 318-1989 for control of cracking is based on the equation proposed by Gergely and Lutz [28] and hence this equation is considered for comparison. Gergely and Lutz made an extensive statistical analysis of crack width and developed equations which are as follows:

The side and bottom crack width are given by:

$$W_s = 0.091\,\frac{\sqrt[3]{(C_s\;A)}}{1 + \dfrac{C_s}{(d - x)}}\,(f_s - 5)x\;10^{-3} \qquad \ldots(2.9)$$

$$W_{bt} = 0.091\,\sqrt[3]{C_b\,A}\,\frac{(h - x)}{(d - x)}\,(f_s - 5)\;10^{-3} \qquad \ldots(2.10)$$

where C_s = side cover measured from the centre of outer bar

C_b = bottom cover measured from the centre of lower bar

A = area of concrete surrounding one bar

d = effective depth of tension reinforcement

h	=	overall depth of cross section
x	=	neutral axis of cracked section
f_s	=	steel stress in kips per square inch and all other units are in inches.

2.6.5 Chinese Code equation (GBJ 10-1989) [105]

Chinese code for concrete structures (GBJ 10-89) proposes the following equation for the maximum width of cracks in flexural members under short-term load:

$$W_{max} = 1.41\, \psi \frac{\sigma_s}{E_s} \left(2.7\, c + 0.11 \frac{d}{\rho_{te}} \right)^{\gamma} \quad \ldots(2.11)$$

where

σ_s	=	tensile steel stress at the crack
E_s	=	modulus of elasticity of steel
ψ	=	non uniformity coefficient of tensile steel
c	=	thickness of concrete cover
d	=	diameter of steel bar
ρ_{te}	=	A_s / A_{te}
A_s	=	area of steel
A_{te}	=	effective area of tensile steel
γ	=	Coefficient related to the bond properties of steel bar (γ = 1.0 for plain bars and 0.7 for deformed bars)

The non-uniformity coefficient of tensile steel is calculated from the following relation and is,

$$\Psi = 1.1 - 0.65 \frac{f_{ct}}{(\rho_{te}\ \ \sigma_s)} \quad \ldots(2.12)$$

where f_{ct} = tensile strength of concrete

If $\psi \leq 0.4$, take $\psi = 0.4$

$\psi \geq 0.4$, take $\psi = 1.0$

2.6.6 Comments on the earlier work on cracking of reinforced concrete

The following observations have been made during the review of literature:

i) The review of literature indicates that there are a number of equations for predicting the spacing and maximum width of cracks in reinforced cement concrete flexural members. These equations may be theoretical, semi theoretical or empirical in nature. However an extensive comparison of these methods for a given test data is not available in literature. Such a comparison will be very much useful in understanding the reliability of these equations.

ii) Though a large number of equations are available for predicting the spacing and maximum width of cracks in conventional reinforced cement concrete flexural members, only a very few attempts on estimation of crack width in fibre reinforced concrete flexural members are reported in literature.

iii) Besides the above, it may be noted that the equations proposed for the estimation of crack width in fibre reinforced concrete members are either theoretical or empirical in nature. An equation which represents the physical behaviour of cracking and at the same time takes care of its random behaviour, is still not available.

2.7 Scope of the Present Investigation

The review of literature indicate that the combined effect of polymers and steel fibres on the strength and behaviour of conventional concrete have not been studied in detail. In addition to this literature survey indicate that the strength and strain at peak load of conventionally reinforced concrete beams could be enhanced by the addition of steel fibres and also by confining the compression zone of the flexural members. Also, the addition of polymers like natural rubber latex to concrete improve the ductility, energy absorption capacity and other durability parameters of concrete. While a large number of investigations are available in literature on the individual effect of either polymers or steel fibres on the flexural behaviour of conventionally reinforced concrete beams, the combined effect of polymers and steel fibres on the flexural behaviour of RCC beams has not been come across.

Taking note of the above gap in the existing knowledge, an attempt is made to study first the individual and combined effect of polymers and steel fibres on the strength and behaviour conventional concrete and subsequently extend the investigation to study the strength and behaviour of conventionally reinforced concrete flexural members with and without confinement.

CHAPTER 3
PRELIMINARY STUDIES ON LATEX MODIFIED CONCRETE

3.1 Introduction

The efficient use of any material for construction depends on its mechanical properties. These properties have to be studied in depth for assessing the structural behaviour and durability characteristics of the same. Incorporation of steel fibres into the cementitious materials like mortar and concrete, has been found to improve the mechanical properties of the composite such as first crack strength, ductility, energy absorption capacity, fracture toughness, dimensional homogeneity etc. [68,77]. Also recent studies on polymer modified concrete indicate that addition of synthetic polymers up to a certain percentage enhances the density, strength, toughness, post peak load deflection characteristics (strain softening zone) and durability of the concrete [65,74]. However, only very few attempts have been made so far to study the effect of natural polymers like rubber latex on the engineering properties of concrete. Also it is possible to improve the strain at peak load and ductility of concrete by providing suitable confinement to the concrete under compression. Considering this, an experimental programme was carried out to study the effect of natural rubber latex as polymer on the strength and behaviour of conventional concrete under compression and flexure. This preliminary investigation was restricted to small specimens like cubes, cylinders and prisms, in order to investigate the behaviour of cement concrete when polymers like natural rubber latex was added to it. The findings of the preliminary investigation will be useful in interpreting the behaviour of prototype structural elements like beams when latex modified concrete is used in it.

---

* Based on the study presented in this Chapter, a technical paper entitled " **Effect of Latex Modification on the Strength and Behaviour of Confined Concrete Under Uni-axial Compression**", has been presented at the International Seminar on Construction Practices in Twenty first Century, held at Roorkee during 26-28, Feb. 1996, pp.757-764.

3.2 Natural Rubber Latex

Hevia Brasiliensis, commonly known as rubber tree, is the most important source of natural rubber and more than 97% of the natural rubber produced is from this tree. Rubber is extracted from a milky white liquid known as latex, which is obtained form the bark of rubber tree by a process called "*tapping*". It is a process of controlled wounding of the plant in which a thin layer of bark is removed. The latex vessels in the region of the wound are opened by tapping and latex flows out from the tree, which is channelled into a container, attached to it.

3.2.1 Composition of latex [58]

Latex is a white or slightly yellowish opaque liquid with a specific gravity in the range of 0.96 to 0.98 and having a variable viscosity. Latex in the latex vessels of rubber tree is sterile, but as it comes out of the tree is slightly alkaline or neutral and it gets contaminated by bacteria. These micro-organisms grow in the latex as it contains proteins and carbohydrates. As a result, volatile organic acids are produced and the latex gets coagulated on keeping. Field latex is a negatively charged colloidal dispersion of rubber particles suspended in an aqueous serum. The size of the rubber particles range from 0.025 to 0.3 microns. These rubber particles are surrounded by a layer of proteins and phospholipids. Latex contains a variety of other non-rubber constituents also. The proportion of these constituents varies according to season, soil, atmospheric conditions, clone, stimulation particles, tapping system etc.

In general the composition of latex is as follows:

Rubber	30 - 40 %
Protein	2.0 - 2.5 %
Resin	1.0 - 2.0 %
Sugar	1.0 - 1.5 %
Ash	0.7 - 0.9 %
Water	55 - 60 %

Fig. 3.1 shows composition of typical latex

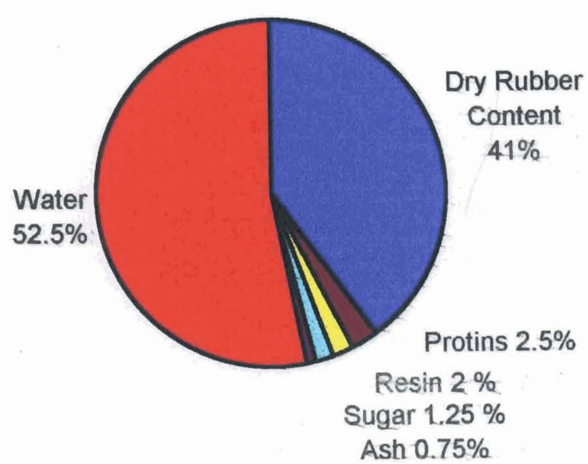


Fig. 3.1 Composition of Latex

3.2.2 Molecular structure of latex

The molecular structure of latex is shown in Fig. 3.2.

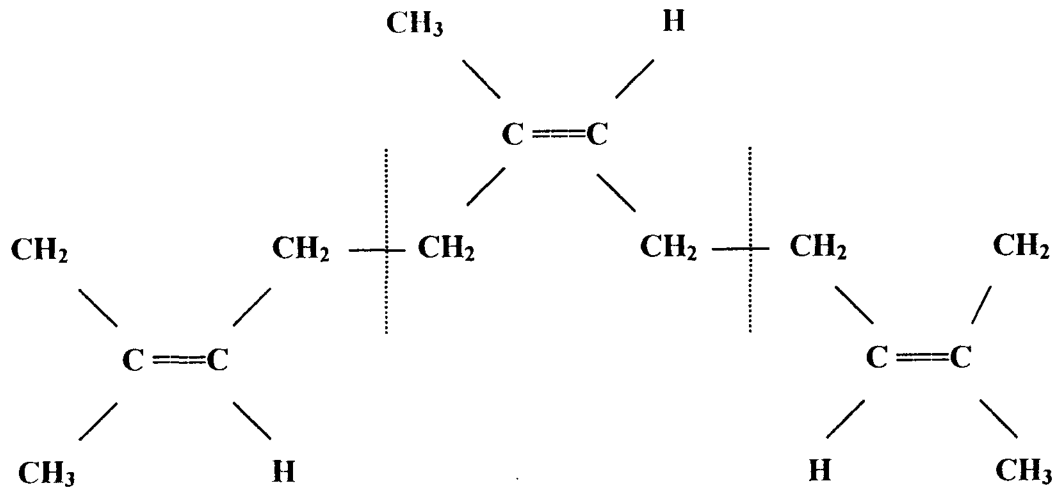


Fig. 3.2 Molecular Structure of Natural Rubber Latex

The rubber latex when extracted from the tree will contain isoprene molecules dispersed in water. In Fig. 3.2 , the molecular arrangement of isoprene molecules are shown in between two dotted lines. These isoprene molecules when come in contact with acidic environment they start coagulating and the chain reaction of coagulation is as shown in the figure 3.2.

3.2.3 Workability of latex modified concrete

When natural rubber latex is added to concrete, the mix becomes stiff and sticky which in turn reduces the workability of concrete mix. Also, the direct addition of rubber latex into the concrete mix leads to the sudden coagulation of latex and the concrete mix becomes harsh and less workable. To overcome this premature coagulation, latex is mixed first with a water reducing Superplasticizer and then water is added to the dry cement concrete mix as suggested by Nagaraj et al [63].

To fix the quantity of Superplasticizer to be added for different percentages of **D**ry **R**ubber **C**ontent DRC (i.e., 0.0, 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0%), flow table test was conducted for a fixed water cement ratio of 0.45. For the required workability of 40 cm flow, the quantity of Superplasticizer required was obtained and the same is given in Table 3.1.

Table 3.1

Superplasticizer Requirement for Different Percentages of DRC

% DRC	Superplasticizer in ml for 50 kg of Cement
0.00	300
0.50	520
1.00	740
1.50	900
2.00	1165
2.50	1350
3.00	1580

3.3 Effect of Latex Modification on the Strength of Concrete

3.3.1 Experimental programme

The experimental programme consists of casting and testing of latex modified concrete cubes, cylinders and prisms for obtaining the basic properties of the composite.

Cement : The cement used throughout the study is Ordinary Portland Cement of 43 grade. Table 3.2 gives the properties of cement used.

Sand : River sand passing through 4.75mm IS sieve and confirming to zone III of the IS 383-1970 is used. Table 3.3 gives the details of the sieve analysis.

Coarse Aggregate : The maximum size of coarse aggregate used in this study is 20mm and fineness modulus of coarse aggregate is 6.80. Table 3.3 gives the details of the sieve analysis.

Water : Water-cement ratio is kept constant and is 0.45 by weight.

Mix : The ratio of cement, sand and coarse aggregate used is 1:2:4 by weight.

Polymer : The polymer used in this investigation is natural rubber latex, which was procured from a nearby place, Mukkam. To prevent coagulation of natural rubber latex, adequate ammonia (0.2% by weight) was added immediately after tapping of natural rubber latex. As the water content of natural rubber latex varies from season to season and place to place, the Dry Rubber Content (DRC) of the latex was found out from the liquid latex and the amount of DRC was used as one of the variables in this study. Table 3.4 gives the properties of the natural rubber latex.

Table 3.2 Details of Cement Used

Sl. No.	Characteristics	Details
1	Trade Name	Malabar Super 43 Grade
2	Normal Consistency	29%
3	Initial Setting time	200 minutes
4	Final Setting time	415 minutes
5	Fineness (specific surface by Blaine's Air Permeability test)	240 M^2/kg

Table 3.3 Sieve Analysis of Aggregates used

Sl No	I.S. Sieve Size	Fine Aggregate				Coarse Aggregate			
		Wt. retained	% retained	Cum % retained	Cum % passing	Wt. retained	% retained	Cum. % retained	Cum % passing
1	40.00 mm	--	--	--	--	0.00	0.00	0.00	100.00
2	20.00 mm	--	--	--	--	0.00	0.00	0.00	100.00
3	10.00 mm	--	--	--	--	3984.00	79.68	79.68	2.32
4	4.75 mm	0.00	0.00	0.00	100.0	1016.00	20.32	100.00	0.00
5	2.36 mm	21.90	4.38	4.38	95.62	0.00	0.00	100.00	0.00
6	1.18 mm	44.80	8.96	13.34	86.66	0.00	0.00	100.00	0.00
7	0.60 mm	128.20	25.64	38.98	61.02	0.00	0.00	100.00	0.00
8	0.30 mm	240.90	48.18	87.16	12.84	0.00	0.00	100.00	0.00
9	0.15 mm	62.90	12.58	99.74	0.26	0.00	0.00	100.00	0.00
10	residue	1.30							
	Total	500.00				5000.0			

Fineness Modulus = Fine aggregates : 2.43 **Course Aggregate : 6.80**

Table 3.4

Properties of Natural Rubber latex

Polymer Type	Dry Rubber Content (DRC)	p^H	Unit Weight Kg/Lit
Natural rubber Latex	41 %	9.5	0.98

Superplasticizer: The Superplasticizer used in this experimental study is CONPLAST P211 and the dosage used for different percentages of DRC is given in Table 3.1. The mixing procedure is as follows:

* Cement and sand were first mixed thoroughly for two minutes.
* 70% of water by weight (corresponding to a W-C ratio of 0.45) was then added to this and again mixed thoroughly.
* Coarse aggregate was then added to the mixer gradually and mixing was continued.
* The remaining 30% of the water, mixed with latex and Superplasticizer was poured into the mixer and mixing was done till a uniform mix was obtained.

3.3.2 Variables considered

The variables considered in this study include different percentages of DRC viz., 0.0%, 0.5%, 1%, 1.5%, 2%, 2.5% and 3%. For each values of DRC, 3 cubes, 3 cylinders, and 3 prisms were cast and tested and the average value has been used for the discussion. Table 3.5 gives the overall dimensions of specimens. For casting cubes, cylinders and prisms, cast iron moulds were used. After 24 hours of casting, the specimens were demoulded and cured for 28 days. At the end of the curing period, they were white washed and kept ready for testing.

3.3.3 Testing of specimens

Cubes : The cubes were tested in a compression testing machine of 3000kN capacity as per IS 516-1959. The load was gradually applied at a rate of 145 kg/cm^2/min. till the failure of the specimen and the ultimate load was recorded.

Cylinders : The cylinders were tested in a compression testing machine of 3000 kN capacity as per IS 516-1959. The strains were computed for each loading stage using a compressometer of 200mm-gauge length fitted with a dial gauge of 0.01mm least count and 10mm travel. The load was gradually applied at a rate of 145 kg/cm^2 /min. till the failure of the specimen. The compressometer readings were recorded at an interval of 5kN load. These readings were made use of in calculating and plotting the stress versus strain plots for all the specimens.

Prisms : The prisms were tested under third point loading (four point bending) in a flexure-testing machine of 100 kN capacity as per IS 516-1959. The load was gradually increased till the failure of the specimen and the ultimate load was recorded and the failure location was recorded to determine the modulus of rupture of the specimen.

Table 3.5

Overall Dimensions of Specimens

Sl. No	Type of specimen	Overall dimensions (mm)	No. of specimens tested
1	Cubes	150 x 150 x 150	21
2	Cylinders	150 x 300	21
3	Prisms	150 x 150 x 700 with 500mm major span	21

The test results thus obtained were used for preparing the following plots:

i) Fig. 3.3 Compressive strength variation for different values of percentage DRC

ii) Fig. 3.4 Stress strain curve in compression for different values of percentage DRC

iii) Fig. 3.5 Flexural strength variation for different values of percentage DRC

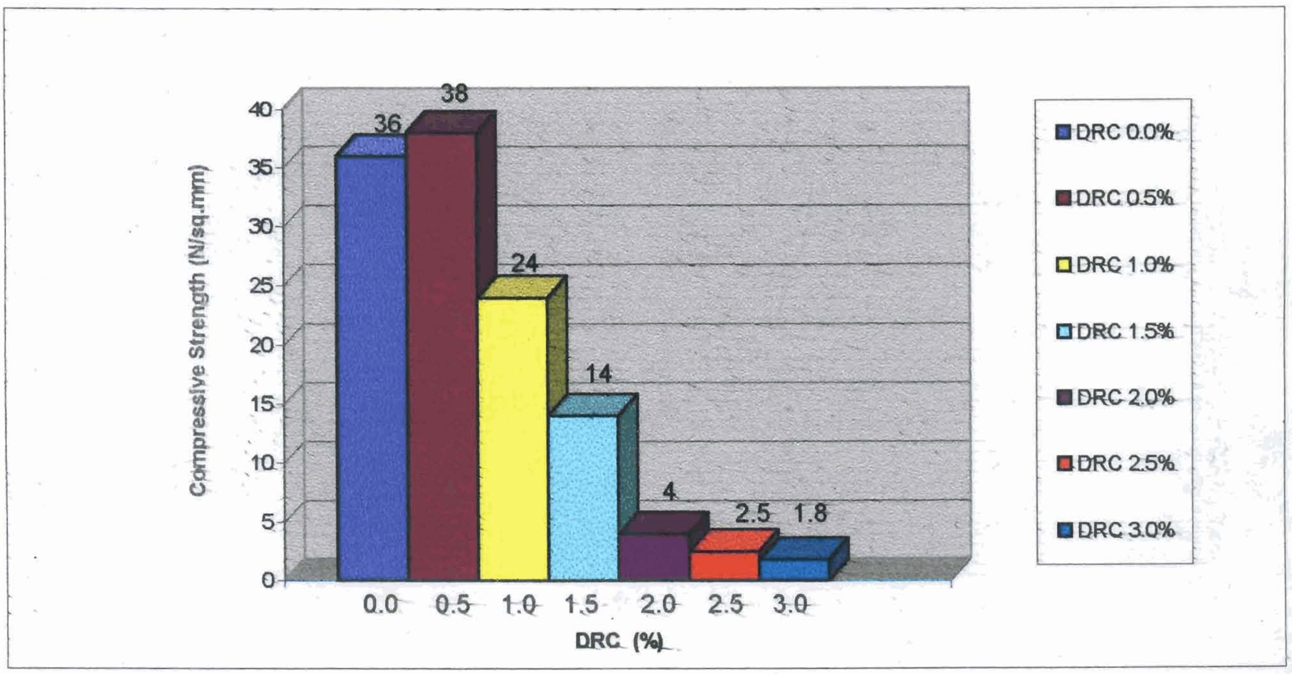


Fig. 3.3 Cube compressive strength variation for different percentage of DRC

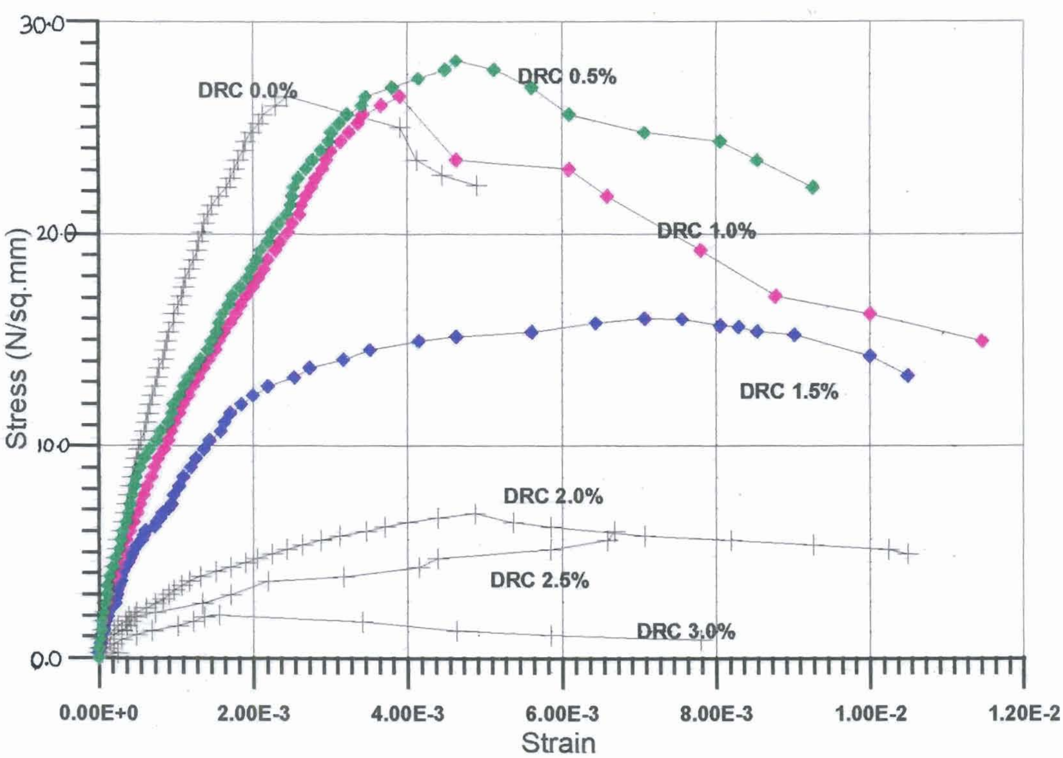


Fig. 3.4 Stress Strain Curve in Compression for different DRC (%)

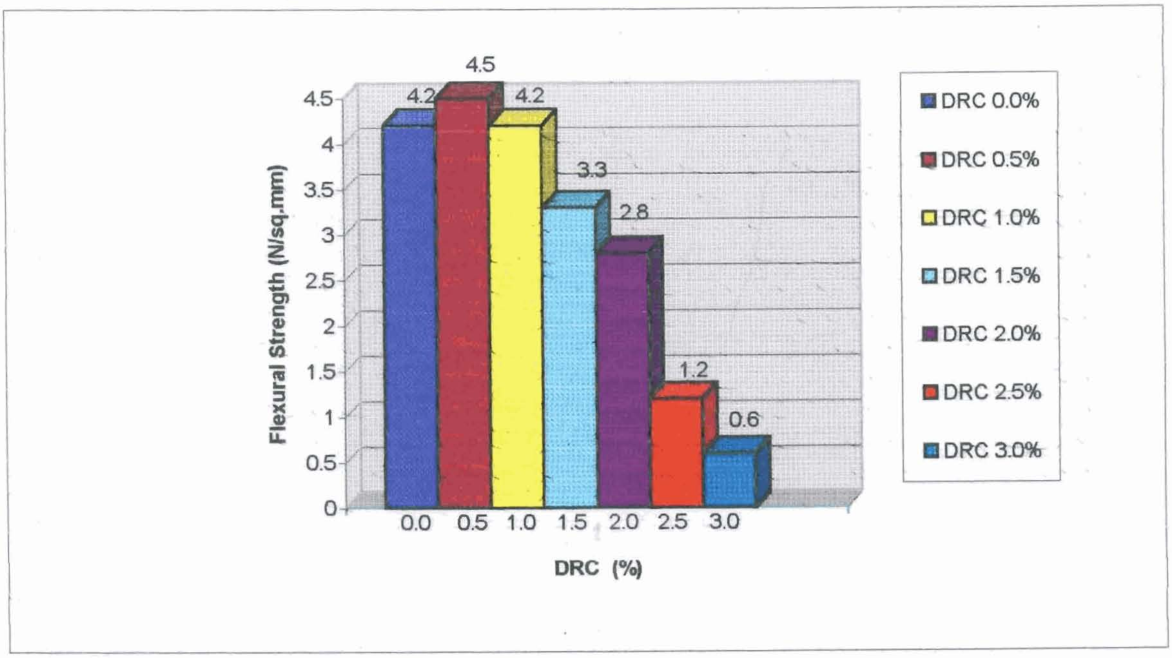


Fig. 3.5 Flexural Strength variation for different percentages of DRC

3.3.4 Discussion of test results

Referring to Fig. 3.3 it may be noted that, addition of smaller percentages of DRC (0.5%) improves the cube compressive strength of plain concrete marginally. The improvement was found to be about 3%. At 1% DRC, the strength was reduced by about 25%. At higher values of DRC ie., at 1.5%, 2.0%, 2.5% and 3.0% drastic reduction in values namely: 46%, 82% and 91% have been obtained. The reasons for increase in strength at lower values of DRC and reduction in strength at higher values of DRC are discussed in detail in the subsequent sections.

Referring to the stress versus strain plots of Fig. 3.4 which are obtained from the tests on cylindrical specimens, the following observations could be made.

1. The initial slope of the stress versus strain curve of plain concrete specimen is steeper than those with latex modification. As the DRC content increases the initial slope becomes more and more mild and practically the curve becomes flat at higher values of DRC. This softening behaviour of the material at higher values of DRC could be due to the effect of DRC, which transforms the material to behave in a ductile manner, and also introduces a higher degree of compressibility. However, for engineering purposes, both strength and ductility are essential properties of a material. From the Figure it appears that when DRC ranges from 0.5% to 1.0% both strength and ductility are higher than that of conventional plain concrete. Hence the optimum or useful values of DRC seems to be from 0.5% to 1.0%.

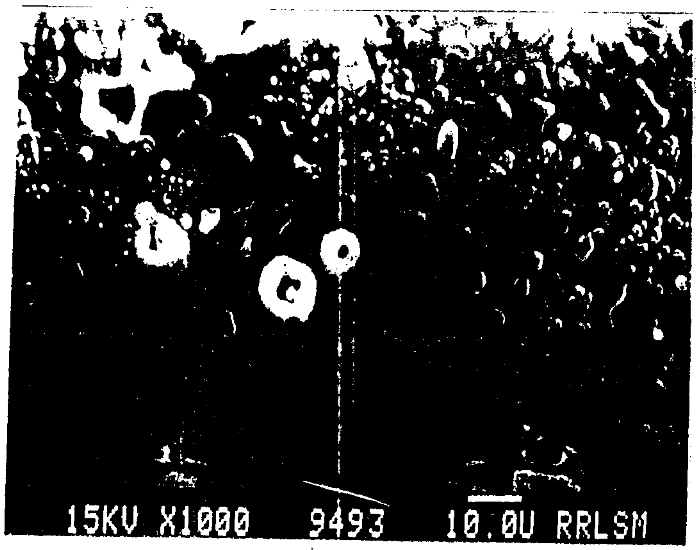
2. Secondly the strain at peak load (ε_p) of the specimen was found to be higher when DRC increases and the values of ε_p for useful values of DRC namely 0.5%, 1.0% are 0.0046 and 0.0038 which are significantly higher than that given by the conventional plain concrete specimen (ε_p = 0.0022). In the case of seismic resistant/ blast resistant/ cyclically loaded structures, the strain at peak load is an important parameter because the plain concrete starts crushing as and when the axial strain

exceeds 0.002, which is a well accepted value [71]. Hence it appears from the test results that latex modification of concrete could improve the axial strain by about two folds even without any confinement.

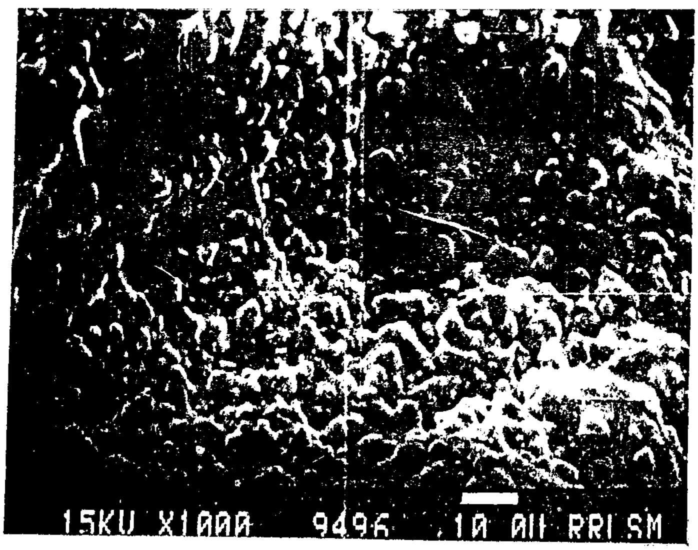
3. Also the Fig. 3.4reveals that the energy absorption capacity, as given by the area of the stress strain curve, improves significantly with the addition of DRC within the range of 0.5% to 1.0%. This further reinforces the earlier statements that the material becomes ductile with the addition of DRC.

It may be noted from Fig. 3.3and Fig. 3.4that the strength of plain concrete improves at lower values of DRC (0.5%) and the strength was found to be decreasing at higher values of DRC. The reasons for the above behaviour is due to the following:

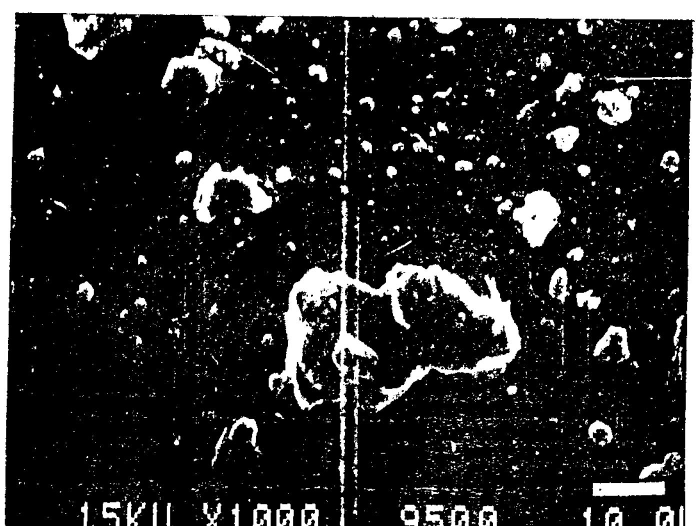
When lower percentage of DRC is added to plain concrete, the polymer fills all the voids existing in concrete and make the same dense which in turn improves both the compressive and flexural strength of concrete. At higher percentages of DRC, the excess quantity of latex other than those required for filling the voids of concrete form weak spots or interfaces which in turn causes reduction in density and strength. This phenomenon can be explained using the photographs taken on the latex modified concrete specimens using Scanning Electron Microscope (SEM). Fig. 3.6(a,b & c) show the photographs taken on the latex modified concrete specimens using Scanning Electron Microscope (SEM) with different percentages of DRC, viz., 0.0%, 0.5% and 1.0% at 1000 times magnification. These photographs were taken at the Regional Research Laboratory (RRL) Trivandrum.



DRC 0.0%



DRC 0.50%



DRC 1.0%

Fig. 3.6 Scanning Electron Microscopic studies
(1000 times magnified)

From Fig. 3.6 a it may be seen that, in the case of plain concrete specimen, lot of micro voids exist. When 0.5% latex was added to the concrete, the latex fills most of the voids existing in the cement concrete as shown in Fig.3.6b . This in turn improves the density of the specimen. Fig. 3.6c shows the specimen with 1.0% DRC. It can be seen from the figure that the excess latex other than that required to fill the voids covers the aggregates and cement mortar forming weak spots which in turn reduces the density and causes a reduction in strength.

The flexural strength of plain concrete and latex modified concrete with different values of DRC is obtained using the test results on prisms. These values are compared and are shown in Fig. 3.5 As in the case of cubes and cylindrical specimens, an increase in strength could be observed at lower values of DRC (0.5%) and further increase in DRC causes reduction in strength. The reasons for this behaviour may be the same as the one explained in the previous sections.

3.4 Effect of Confinement on Latex Modified Concrete

3.4.1 Introduction

When concrete structures are subjected to earthquake forces and blast loads the critical sections of the structural members must be able to absorb strain energy, if sudden failures are to be avoided. This is possible only if the material is capable of withstanding considerable deformations without a reduction in its load carrying capacity.

It is possible to improve the ductile property of the concrete if the material is prevented from disruption and crushing into pieces when compressed to its ultimate capacity. This can be done by providing suitable confinement to the concrete under compression.

A more practicable method of confining concrete in structural members appears to be the use of steel spirals around the periphery of the core concrete in compression.

Considering the above, a preliminary experimental programme is carried out to study the effect of confinement on the strength and behaviour of latex modified concrete under uni-axial compression. The outcome of this preliminary experimental programme has been suitably made use of in the studies on the effect of confinement in compression zone of reinforced concrete flexural members discussed in the subsequent sections.

3.4.2 Experimental Programme

The experimental work include casting and testing of 16 series of cylindrical specimens with different amount of confinement and latex. For each series, two specimens have been cast and tested and the average value of two test results has been used for the analysis. Thus a total of 32 cylindrical specimens have been tested. Out of these 16 series, 12 series of cylinders were confined by circular spirals having three different values of pitch viz., 30mm, 40mm and 50mm. Three percentage of DRC of latex viz. 1.0, 2.0 and 3.0% were used in this study. Plain bars of 6mm diameter with yield strength of 275 N/mm^2 have been used for fabricating the spiral, which is used as confinement. The overall dimensions of the cylindrical specimens are 150mm diameter and 300mm height. The volumetric ratio of confinement ρ_s which is the ratio of volume of binder to the volume of core concrete for 30mm 40mm and 50mm pitch are 4.83%, 3.62% and 2.90% respectively. Table 3.6 shows the details of confinement and latex used in the cylindrical specimens.

Table 3.6

Details of Confinement and Latex used in the specimens

Sl. No	Specimen Designation	% DRC Used	ρ_s (%)	Remarks
1	$C_0 L_0$	0.00	0.00	UC
2	$C_1 L_0$	0.00	4.83	C
3	$C_2 L_0$	0.00	3.62	C
4	$C_3 L_0$	0.00	2.90	C
5	$C_0 L_1$	1.00	0.00	UC-L
6	$C_0 L_2$	2.00	0.00	UC-L
7	$C_0 L_3$	3.00	0.00	UC-L
8	$C_1 L_1$	1.00	4.83	C-L
9	$C_1 L_2$	2.00	4.83	C-L
10	$C_1 L_3$	3.00	4.83	C-L
11	$C_2 L_1$	1.00	3.62	C-L
12	$C_2 L_2$	2.00	3.62	C-L
13	$C_2 L_3$	3.00	3.62	C-L
14	$C_3 L_1$	1.00	2.90	C-L
15	$C_3 L_2$	2.00	2.90	C-L
16	$C_3 L_3$	3.00	2.90	C-L

Note:
C - Confined and without latex
UC - Un-confined without latex
C-L - Confined with latex
UC-L - Un-Confined with Latex

3.4.3 Casting of specimens

The natural rubber latex used in this study was brought from a near by place called Mukkam. Immediately after tapping the latex, adequate quantity of ammonia was added (0.2% by weight) to prevent premature coagulation and to preserve the latex in liquid state for long time, to use in the laboratory. For casting specimens, cast iron moulds of 150mm diameter and 300mm height was used. Circular spirals of internal diameter 100mm were fabricated and placed inside the mould. The concrete mix used was 1:2:4 (Cement : Sand : Coarse aggregate) by weight. The water cement ratio used throughout the investigation was 0.45 by weight.

During the trial mixes, it was noticed that, as and when the latex was added to the green concrete, the ammonia present in the latex evaporates, causing instantaneous coagulation of the latex, which results in a harsh mix. To overcome this problem and to improve the workability of latex modified concrete mix, a Superplasticizer, CONPLAST P211 was added to the concrete latex mix.

The mixing procedure adopted is similar to the one explained in Chapter 3 under article number 3.3. Before casting, machine oil was smeared on the inner side of the mould and spiral reinforcement cage was placed inside the mould. The concrete mix was placed into the mould in two layers and the mould was vibrated on a vibrating table for thorough compaction. After 24 hours of casting, the specimens were demoulded and air dried for 24 hours and then moist cured for 27 days in a curing tank. The air drying of the specimens for 24 hours was done to allow for the coalescence and film formation, which is very essential in the polymer modification process.

Some of the specimens (those with higher latex content) took more time even up to 48 hours for setting. This may be due to the fact that the addition of higher percentage of latex affects the hydration of cement.

3.4.4 Testing of specimens

All the specimens were tested in a universal testing machine of 300 ton capacity. The specimens were subjected to a monotonic axial compression until failure. The rate of loading was kept constant for all the specimens. The ultimate load taken by the specimens were recorded. In all the cases the tests were terminated when the load was suddenly dropped to the fraction of the maximum and resulted in a complete destruction of the specimen.

3.4.5 Behaviour of specimen under load

In the case of unconfined specimens without latex, just before the ultimate load, cracks appeared in the vertical direction. As the load increased, the cracks widened and resulted in complete destruction of the specimen.

In the case of unconfined latex modified concrete specimens, irregular fine cracks appeared on the surface as the load increased. At higher stages of loading, cracks widened, cover concrete did not spall-off in the case of specimens with 1% DRC. This is due to the increased post-cracking tensile resistance and improvement in the dimensional stability of the specimen due to the addition of DRC. However with 2% and 3% DRC, the specimens crumbled all of a sudden without forming any cracks.

The specimens with latex modification and confinement have shown a better performance. In this case as the load increased, fine vertical cracks appeared on the surface. Further increase of loading widened the cracks. The cover concrete was found to be sticking to the core concrete and did not spall off at loads prior to the ultimate load. The cover and core concrete was found to be intact even at the ultimate load stage. The specimens with 1% DRC were found to give higher values of peak load and strain at peak load when compared to other specimens with 2% and 3% DRC for a given confinement.

3.4.6 Results and discussion

From the Table 3.7, it may be noted that

1. In general, addition of latex to unconfined concrete did not improve the strength of concrete. However with lower quantity of DRC (1%), the strength was found to improve marginally. Higher percentages of DRC (Viz. 2% and 3%), infact, have resulted in reduction in strength.

Table 3.7

Test Results

Sl. No.	Specimen Designation	Ultimate compressive strength (N/mm^2)
1	$C_0\ L_0$	19.25
2	$C_1\ L_0$	24.75
3	$C_2\ L_0$	21.36
4	$C_3\ L_0$	20.47
5	$C_0\ L_1$	21.08
6	$C_0\ L_2$	8.87
7	$C_0\ L_3$	3.05
8	$C_1\ L_1$	37.18
9	$C_1\ L_2$	22.25
10	$C_1\ L_3$	16.75
11	$C_2\ L_1$	24.97
12	$C_2\ L_2$	19.72
13	$C_2\ L_3$	14.60
14	$C_3\ L_1$	19.49
15	$C_3\ L_2$	10.88
16	$C_3\ L_3$	8.82

2. For a given quantity of DRC, as volumetric ratio of confinement increases, the strength increases. This effect is significant at lower values of DRC i.e, at 1%.

3. The test results reveal the fact that addition of 1% DRC to either confined or un-confined concrete improves the strength. Further addition of latex results in the reduction of strength. From this behaviour addition of DRC up to 1.0 % DRC appears to be in the useful range.

Figure 3.7shows the plots of ultimate load of latex modified specimens versus volumetric ratio of confinement (ρ_s). It can be seen from figure that, at 1% DRC, a noticeable increase in the strength could be seen as the value of ρ_s increases. At 2% DRC and 3% DRC, infact, a reduction in strength is noticed. This is higher at lower values of ρ_s. At higher values of ρ_s, a steep increase in strength can be seen in the case of specimens with 2% and 3% DRC. This indicates that by increasing ρ_s, strength could be increased even with higher percentage of DRC (2% and 3%). On the other hand, in the case of un-confined latex modified concrete, as the latex content increases beyond 1.0% of DRC, the strength decreases rapidly.

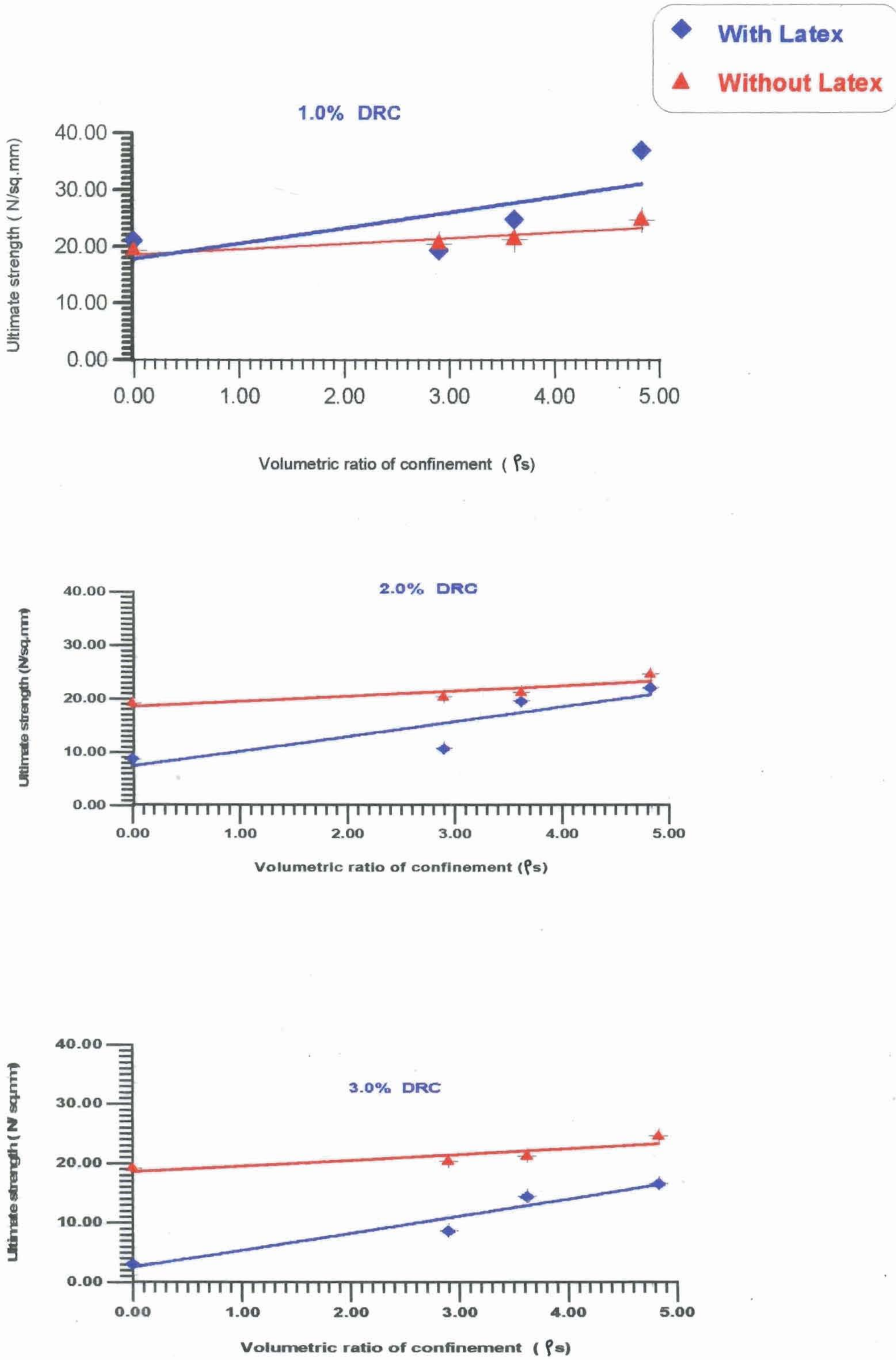


Fig. 3.7 Ultimate strength versus Volumetric ratio of confinement (ρs)

An attempt is made to study the combined effect of latex modification and confinement on the strength of concrete. Several combinations of DRC and Confinement have been tried to relate them with the strength of concrete. An index $(1+L_i)(1+C_i)$ known as Latex Confinement Index (L-C Index) was found to fit the data satisfactorily

where

$$L_i = \frac{DRC}{[1+(\frac{DRC}{0.5})^2]} \qquad \text{.... (3.1)}$$

and

$$C_i = (\rho_s - \bar{\rho}_s)(\frac{f_y}{f_c}) \qquad \text{... (3.2)}$$

where DRC = Dry Rubber Content in percentage

ρ_s = Volumetric ratio of confinement, i.e. the ratio of the volume of the binder to the volume of confined concrete

$\bar{\rho}_s$ = Particular volumetric ratio of confinement when the pitch of binder is equal to the least lateral dimension of the specimen

f_y = Yield strength or 0.2% proof stress of steel binder

f_c = Ultimate strength of unconfined concrete specimen.

Figure 3.8 shows the plot of Ultimate strength of confined latex modified specimens versus L-C Index. A best-fit curve was drawn for the plots. It can be seen from Fig.3.8 that, as the L-C Index increases, strength also increases. The equation for the best-fit curve, thus obtained is,

$$\textit{Ultimate Strength } (f_{l-c}) = 6.1 * (L\text{-}C\ Index)^{2.96} \qquad \text{...(3.3)}$$

with a correlation coefficient of 0.8

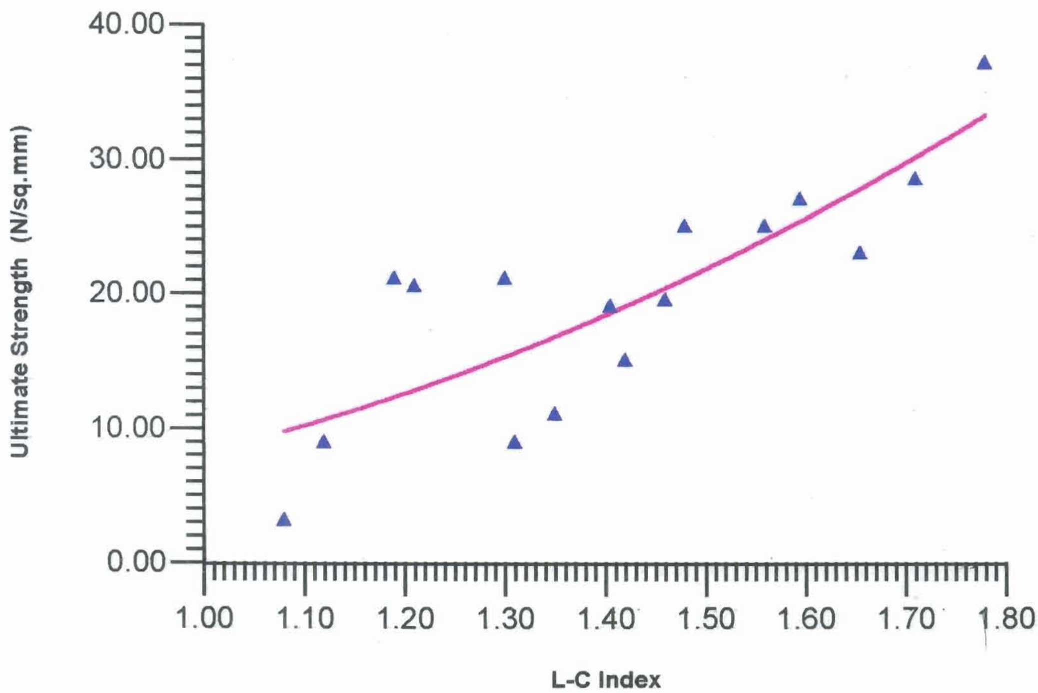


Fig. 3.8 Ultimate Strength versus L- C Index

This empirical equation could be used for predicting the ultimate strength of confined latex modified concrete specimens for the range of values of latex and confinement considered in this study.

3.5 CONCLUSIONS

From the preliminary experimental investigation, following conclusions are arrived at:

1. The addition of small quantities of DRC (0.5%) *marginally* improves the compressive and flexural strength of plain concrete (about 7.0%). Higher values of DRC causes drastic reduction in the strength of concrete.

2. The strain at peak load which is one of the important properties to be considered in the design of seismic resistant/ blast resistant/ cyclically or repeatedly loaded structures improves *significantly* (by two folds), with the addition of DRC.

3. The energy absorption capacity of the material enhances *markedly* (2- 2.5 times), with the addition of DRC within the range of 0.5% to 1.0%.

4. The foresaid properties of plain concrete namely strength, strain at peak load and energy absorption capacity have been found to improve with the addition of DRC within the range of 0.5% to 1.0%. This indicates that this range seems to be an optimum value of DRC.

5. The scanning electron microscopic studies reinforce the above findings and indicate that, at smaller percentages of DRC, density and hence the compressive strength and flexural strength of composite increases.

6. This preliminary investigation was restricted to small specimens like cubes, cylinders and prisms in order to investigate the behaviour of cement concrete when polymers like natural rubber latex was added to it. The findings of the preliminary investigation will be useful later in interpreting the behaviour of

prototype structural elements like beams when latex modified concrete is used in it.

7. The strength of concrete increases as the volumetric ratio of confinement increases. This increase is further improved by the addition of lower values of DRC (1%).

8. At higher values of DRC, infact, a reduction in strength is noticed for a given confinement.

9. The reduction in strength due to higher percentages of DRC can be appreciably reduced by providing higher volumetric ratio of confinement ρ_s.

10. From the study, DRC up to a value of 1.0 % appears to be a useful value in the case of latex modified confined concrete.

CHAPTER 4

LATEX MODIFIED STEEL FIBRE REINFORCED CONCRETE FLEXURAL MEMBERS

4.1 Introduction

*

Strength and ductility are the two important factors to be considered in the design of structures subjected to seismic and dynamic forces. Hence many attempts have been made in the recent past to develop new materials which exhibit higher strength and ductility, so that they could be used in the structures subjected to dynamic forces. Incorporation of steel fibres into the cementitious materials like mortar and concrete has been found to improve the structural properties like ductility, energy absorption capacity/toughness, post crack resistance, dimensional stability etc. of the composite [1,2,3]. Also the recent studies on polymer modified concrete indicate that the addition of natural or synthetic polymers up to a certain percentage enhances the strength, toughness, post peak load deflection characteristics (strain softening zone), and durability of concrete [4,5]

However, large scale investigations are not reported in literature so far on the combined effect of latex and randomly oriented steel fibres on the strength and behaviour of conventionally reinforced concrete beams. and no attempts have been

---

• Based on the studies presented in this Chapter, the following technical papers have been published:

1 . **"Latex Modified Steel Fibre Reinforced Concrete for Seismic Resistant Structures."** Paper presented at the National Conference on Civil Engineering Disaster Management, held at College of Engineering, Trivandrum during Dec. 7-8. 1995,pp.5-1 to 5-10.

2 **"Strength and Ductility of Latex Modified Steel Fibre Reinforced Concrete Flexural Members", Paper** accepted for publication in the Journal of Structural Engineering, SERC Madras.

3 Prediction of First Crack Load and Ultimate Moment of Resistance of Polymer Modified **Steel Fibre Reinforced Concrete Flexural Members",** Paper presented at the National Seminar on High Performance Concretes, held at Chennai during 21-22 May 1998, ppTS2-33-41

made in the recent past to study the combined effect of steel fibres and polymers on the first cracking load and ultimate moment of resistance of conventional reinforced cement concrete flexural members. Therefore an experimental programme was carried out to study the effect of latex modification and inclusion of steel fibres on the first crack load, ultimate moment of resistance, ductility and toughness characteristics of conventionally reinforced concrete flexural members.

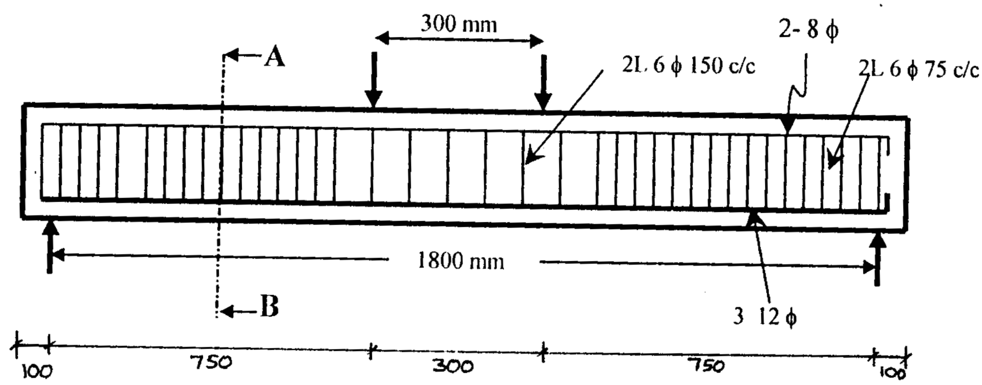
4.2 Experimental Programme

The experimental programme consisted of casting and testing of 16 latex modified reinforced concrete beams of 125 x 200 x 2000 mm size (Fig. 4.1). To start with trial specimens having a span of 1800mm were tested under third point loading (four point bending). The tests indicated that higher shear span to depth to depth ratio (a_v/d) are required in order to ensure flexural failure. Hence the point loads were applied at 750mm from the end supports so that a minor span (flexural span) Of 300mm and major span of 1800mm could be obtained. This testing arrangement is similar to the one adopted by the other researchers [46,52]. Out of these sixteen beams, 12 beams were additionally reinforced with steel fibres. The main variables considered in this study are.

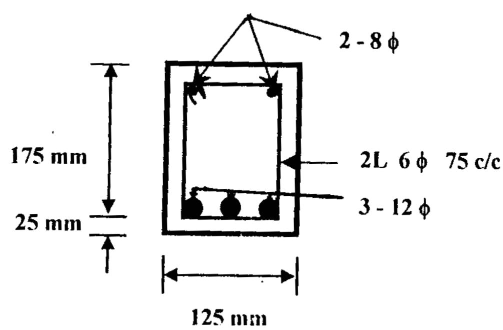
(I) Three different values of percentage of volume fraction (V_f) of steel fibres. (0.5, 1.0 & 1.5%)

(ii) Three different values of percentage volume fraction of dry rubber content (DRC) of latex (0.5, 1.0 and 1.5%)

The aspect ratio (A_p) of steel fibres used in this investigation was kept constant for all the specimens and is 50. Table 4.1 show the details of variables used in this study. The reinforcement used were high yield strength deformed (HYSD) bars of 12mm diameter in the tension side and 8mm diameter in the compression zone. The shear reinforcement was designed so that the shear capacity of the specimen is higher than the flexural strength.



Longitudinal Section of beam



Section AB

* All dimensions in mm

Fig. 4.1 Details of cross section of beam and reinforcement used.

Table 4.1
Details of variables used

Sl.No.	Beam designation	DRC (%)	Fibres V_f (%)
1	$BL_0 F_0$	0.0	0.0
2	$BL_0 F_1$	0.0	0.5
3	$BL_0 F_2$	0.0	1.0
4	$BL_0 F_3$	0.0	1.5
5	$BL_1 F_0$	0.5	0.0
6	$BL_2 F_0$	1.0	0.0
7	$BL_3 F_0$	1.5	0.0
8	$BL_1 F_1$	0.5	0.5
9	$BL_2 F_1$	1.0	0.5
10	$BL_3 F_1$	1.5	0.5
11	$BL_1 F_2$	0.5	1.0
12	$BL_2 F_2$	1.0	1.0
13	$BL_3 F_2$	1.5	1.0
14	$BL_1 F_3$	0.5	1.5
15	$BL_2 F_3$	1.0	1.5
16	$BL_3 F_3$	1.5	1.5

Note : L_0 represents beams without DRC
$L_1 \rightarrow$ 0.5% DRC, $L_2 \rightarrow$ 1.0% DRC, $L_3 \rightarrow$ 1.5% DRC,
F_0 represents beams without steel fibres
$F_1 \rightarrow$ 0.5% V_f, $F_2 \rightarrow$ 1.0% V_f, $F_3 \rightarrow$ 1.5% V_f,

This is done in order to ensure flexural failure. In the shear span 2 legged 6mm dia. stirrups were given at 75mm centres. At the middle the spacing was at 150mm centre to centre. The steel fibres had a diameter of 0.88mm and were obtained by chopping GI wire to the required length. The mechanical properties of the reinforcement and steel fibres used in this investigation are given in Table 4.2. Ordinary Portland cement of 43 grade conforming to IS 8112-1989 was used. River sand passing through 4.75mm IS sieves confirming to grading zone III and crushed stone with a nominal maximum size of 20mm and having a fineness modulus of 6.8 were used as aggregates. It may be noted that as per the ACI 544[2] guidelines the maximum size of coarse aggregate to be used in steel fibre reinforced concrete is limited to 19mm. However in this experimental programme the maximum size of coarse aggregate used is 20 mm. This is due to the following reasons. The main objective of this study is to investigate the strength and behaviour of conventional reinforced concrete flexural members and how to improve the fore-said properties by the addition of steel fibres and polymers. It may be noted that in practice, in all these conventional structural elements, the maximum size of the coarse aggregates used will be normally 20mm. In view of the above, the maximum size of aggregate is retained as 20mm. A concrete mix proportion of 1:2:4 (cement: sand: coarse aggregate) by weight with a water cement ratio of 0.45 by weight was used. The polymer used in this investigation was natural rubber latex having a dry rubber content of 41% by weight. To prevent the early coagulation of latex and to disperse the latex molecules uniformly in concrete, a water reducing superplasticizer (CONPLAST P211) was used while mixing the natural rubber latex with concrete.

All the ingredients were first mixed in dry condition in a concrete mixer. 70% of the calculated amount of water was added to the dry mix and mixed thoroughly to get a uniform mix. Later required quantities of steel fibres were sprinkled over the concrete mix and mixing was continued. At the end, the remaining 30% of water, mixed with latex and superplasticizer was poured into the mixer and mixed well to get a uniform concrete mix.

Before casting, machine oil was smeared on the inner surfaces of the mould and the concrete was poured into the mould in layers, and each layer was compacted using a needle vibrator. Along with each specimen, 150mm cubes, 150 x 300mm cylinders and 150 x 150 x 700 mm prisms were also cast. After 24 hours of casting, the specimens were de-moulded and air dried for 24 hours and then moist cured for 27 days using wet gunny bags. The air drying was done to allow for the coalescence and film formation which is essential in the polymer modification process. After the curing period was over, the specimens were white washed and kept ready for testing.

Table 4.2

Mechanical Properties of Steel Reinforcement and Fibres Used

Nominal dia. of Bar (mm)	Actual diameter (mm)	0.2% proof stress (N/mm^2)	Ultimate strength (N/mm^2)	Modulus of elasticity (N/mm^2)
12 mm	12.24	470.00	480.0	2.127E5
8 mm	8.31	330.00	340.0	1.923E5
6 mm	6.58	275.00	320.1	1.093E5
0.88mm fibres	0.88	86.00	330.0	0.330E5

4.3 Testing of Specimens

4.3.1 Test Set-up

All the specimens were tested in a compression and bending testing machine of capacity 300Ton (2942kN). In order to note the applied load precisely, a load cell of 25 Ton (245 kN) was used. A special steel frame arrangement called rotation meter was fabricated to measure the longitudinal strains. Three linear variable differential transducers (LVDT) were used for measuring the longitudinal strains (at top, bottom, and middle). The longitudinal deformations at the top were measured using an LVDT with a range of ± 0.5mm and a resolution of 0.01mm. This LVDT was placed at 25mm from the extreme compression fibre. The deformations at the middle was measured using an LVDT of ± 1.0 mm range and a resolution of 0.001mm and was positioned at 75mm from the extreme compression fibre. The bottom measurements were made using an LVDT of range ± 10 mm range and a resolution of 0.01mm and was placed at 25mm from the extreme tension fibre. The vertical deflections were measured using a dial gauge of 50mm and a least count of 0.01mm.

The beams were supported on two rollers (30mm dia.) of which one was fixed and the other was capable of rotation. The effective span was kept as 1800mm. The specimens were tested under two point loading. Two rollers, each 30mm diameter served as load points and were kept on the beam at a distance of 300mm, kept in position by plaster- of- paris. Fig. 4.2 shows schematic diagram of the the test setup and Fig. 4.3 shows the photograph of the test setup.

4.3.2 Testing procedure

The load was applied in stages. For every stage of loading, the following readings have been noted.

i) deflection at the mid span

ii) LVDT readings at 25mm, 75mm and 175mm from the extreme compression fibre of the specimen.

All the control specimens viz. Cubes, cylinders and prism were tested. The compressive strength, stress-strain behaviour of concrete and flexural strength have been obtained from the testing of control specimens.

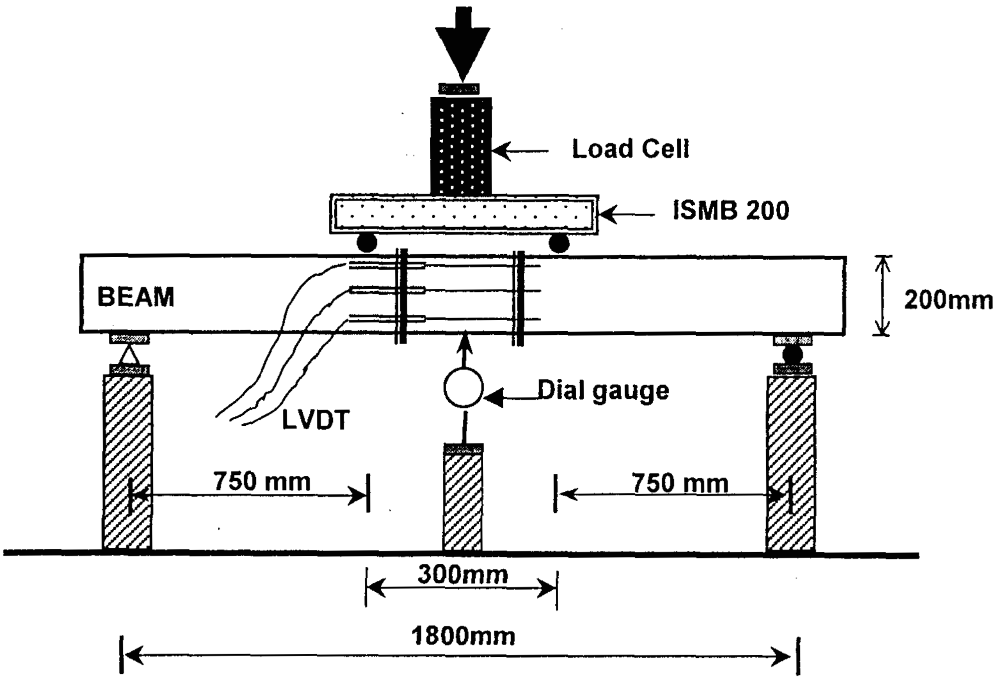


Fig. 4.2 TEST SET UP

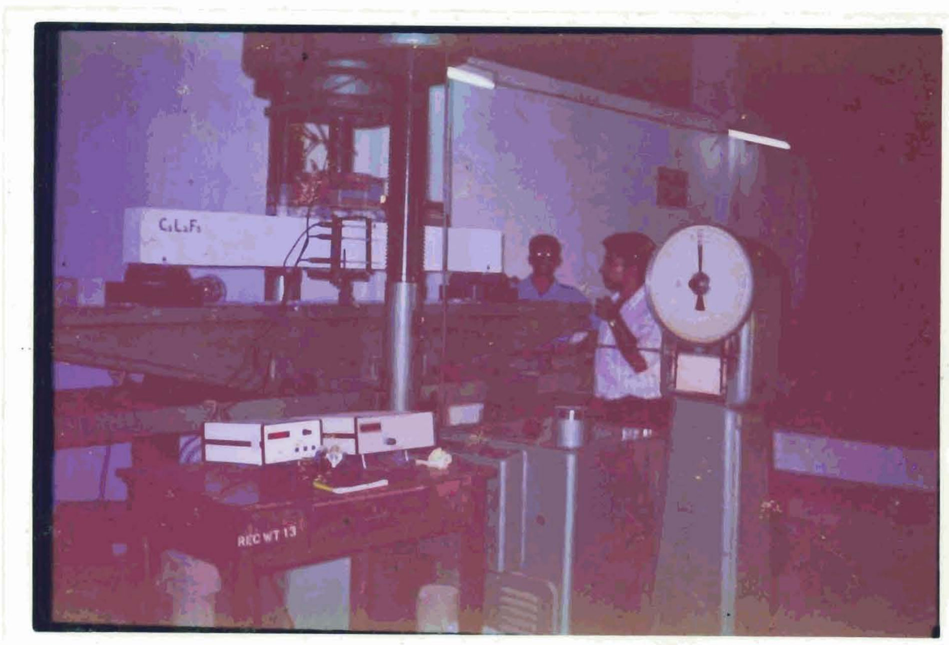
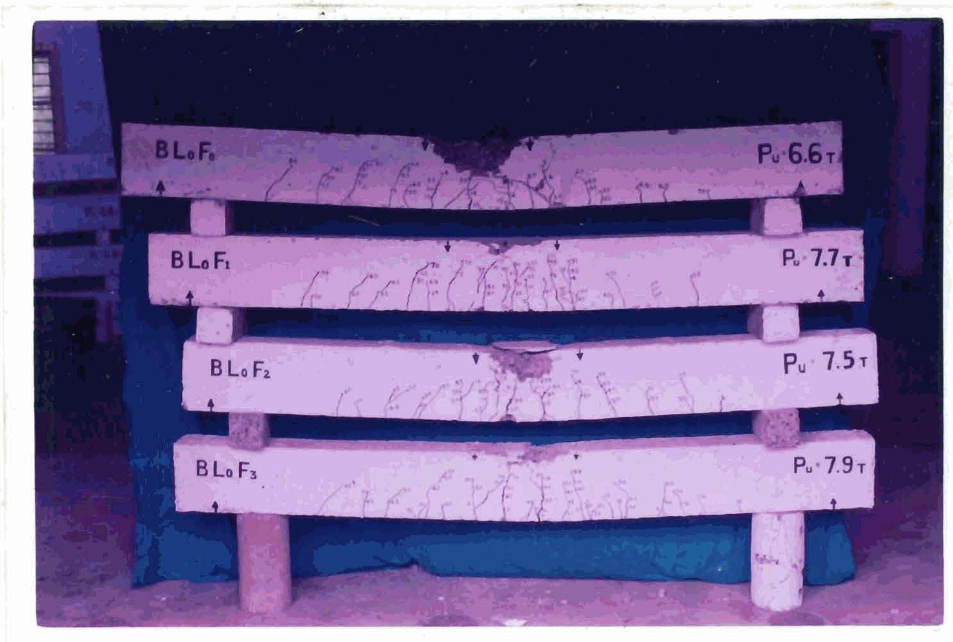


Fig. 4.3 Photograph of the test setup

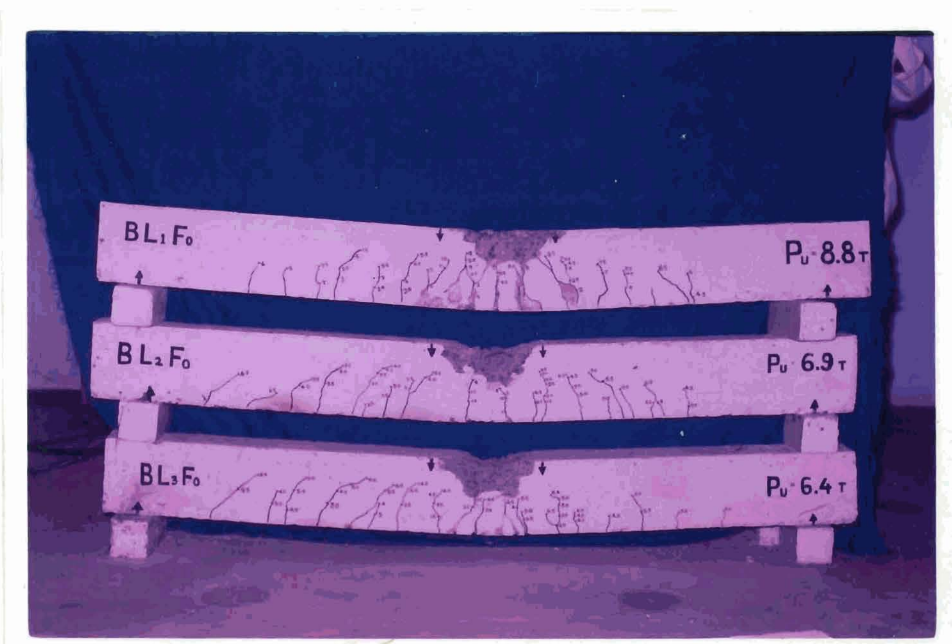
4.4 Behaviour of the Specimen Under Load

In the case of reinforced concrete specimens without latex and steel fibres, cracks appeared as the loading reached the first crack load of the specimen. Further increase of load resulted in the formation of additional cracks and widening of some of the earlier cracks. At ultimate stage, most of the cracks traversed up the top face of the beam and the steel bar in the compression zone in between the loading points appeared to be buckled and spalling of compression concrete took place. In the case of specimens additionally reinforced with steel fibres, a larger number of finer cracks developed. At ultimate stage, all the cracks traversed up the top face of the beams and some portion of compression concrete was found to have crushed. However, spalling of side and cover concrete did not occur and were found to be intact with the core concrete. In the case of latex modified concrete specimen and latex modified steel fibre concrete specimen similar behaviour was noticed.

One major observation noticed is that addition of fibres and latex improved the first crack and ultimate load in most of the specimens. Also deflection at ultimate load was found to be higher. Only when the latex content is increased to higher percentages, a reduction in load carrying capacity was noticed. Figs 4.4 (a,b,c,d and e) show the crack pattern of the tested beams.

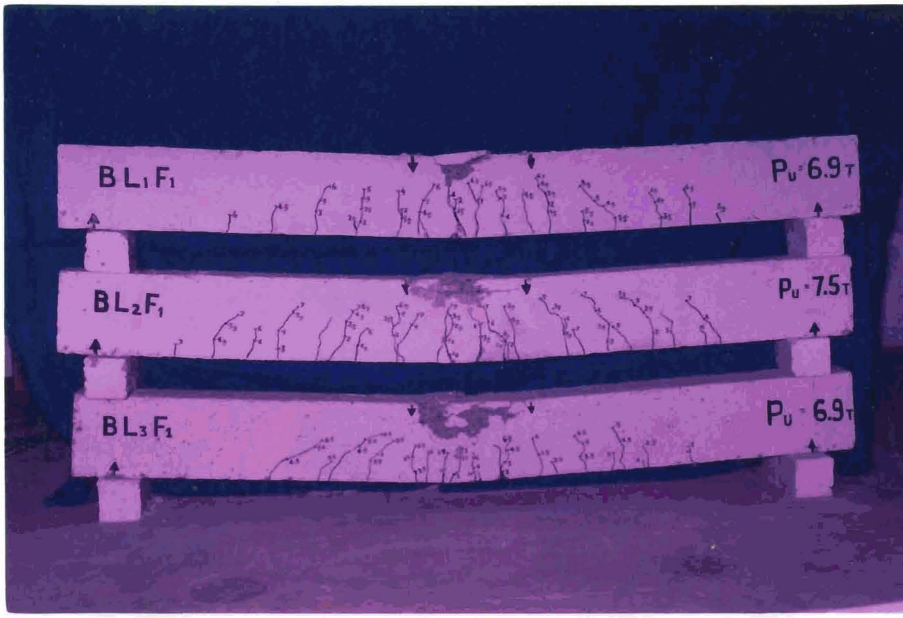


(a)

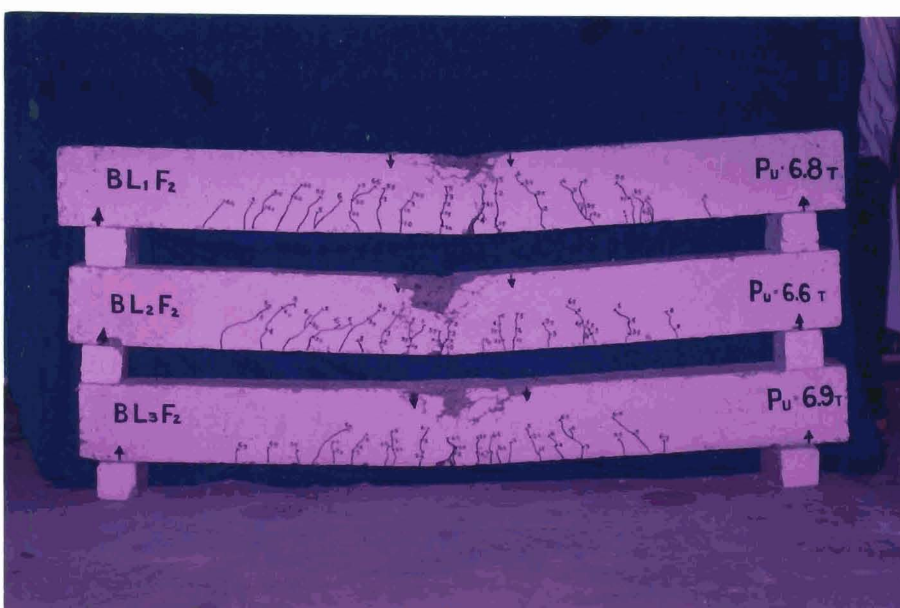


(b)

Fig. 4.4 Photograph of tested beams



(c)



(d)

Fig. 4.4 Photograph of tested beams

(e)

Fig. 4.4 Photograph of tested beams

4.5 Discussion of Test Results

The test results such as first crack load and ultimate load are given in Table 4.3 And the strength of control specimens are given in Table 4.4.

Table - 4.3

Results of beam test

Sl. No	Beam Designation	First Crack Load (KN)	Ultimate Load (KN)	Strength gain factor
1	$BL_0\ F_0$	14.70	64.68	1.00
2	$BL_0\ F_1$	15.68	75.46	1.16
3	$BL_0\ F_2$	15.68	73.50	1.14
4	$BL_0\ F_3$	17.64	77.42	1.19
5	$BL_1\ F_0$	16.00	86.24	1.33
6	$BL_2\ F_0$	14.70	67.62	1.04
7	$BL_3\ F_0$	9.80	62.72	0.96
8	$BL_1\ F_1$	11.76	67.62	1.04
9	$BL_2\ F_1$	9.80	73.50	1.14
10	$BL_3\ F_1$	9.80	67.62	1.04
11	$BL_1\ F_2$	13.72	66.64	1.03
12	$BL_2\ F_2$	11.76	64.68	1.00
13	$BL_3\ F_2$	14.70	67.62	1.04
14	$BL_1\ F_3$	14.70	69.58	1.07
15	$BL_2\ F_3$	15.68	70.56	1.09
16	$BL_3\ F_3$	14.70	67.62	1.04

Note : Strength gain factor $= \dfrac{\text{Ultimate load of the specimens}}{\text{Ultimate load of the specimen } BL_0\ F_0}$

Table - 4.4

Test results of control specimen

Sl. No	Beam designation	Cube strength (N/mm^2)	Cylinder strength (N/mm^2)	Mod. of rupture (N/mm^2)
1	$BL_0 F_0$	39.10	30.84	4.97
2	$BL_0 F_1$	42.88	31.40	4.44
3	$BL_0 F_2$	32.44	28.96	4.44
4	$BL_0 F_3$	40.20	36.21	4.97
5	$BL_1 F_0$	38.88	27.16	4.17
6	$BL_2 F_0$	29.33	24.90	4.08
7	$BL_3 F_0$	30.22	20.93	4.35
8	$BL_1 F_1$	44.44	32.54	4.71
9	$BL_2 F_1$	37.55	28.86	4.08
10	$BL_3 F_1$	32.44	22.35	3.73
11	$BL_1 F_2$	37.55	29.43	4.71
12	$BL_2 F_2$	26.22	21.79	3.70
13	$BL_3 F_2$	26.66	20.08	3.55
14	$BL_1 F_3$	44.44	37.91	4.88
15	$BL_2 F_3$	34.88	32.54	5.51
16	$BL_3 F_3$	28.88	27.73	4.08

Referring to Table 4.3, the following points may be noted.

i. As the fibre content increases, the first crack load and ultimate load increases by about 15% on an average.

ii. In the case of specimens with latex alone, the addition of small percentages of latex viz.,0.5%, improved the first crack load and ultimate load significantly. The ultimate load was found to be about 33% higher than that of the plain specimen. However at higher percentages of latex, viz., 1.0%, the improvement was found to be only marginal (4%). When the percentage of DRC was further increased i.e. at 1.5% infact a reduction in ultimate load occurred. This may be due to the following reasons. When latex is added to the fresh concrete, the polymer particles get dispersed in the cement paste. As the hydration of cement proceeds and the water in the pores drain out, the polymer particles fill the micro pores. Ultimately, the removal of water by cement hydration, the closely packed polymer and cement particles coalesce into continuous films. Thus a monolithic network is formed in which the polymer and cement hydrate phases inter-penetrate throughout. This will increase the density, which in turn increased the strength of concrete. However, as the latex content increased, the excess latex will lead to the formation of weak spots in the specimen due to air entrainment and resulting in the reduction in density. At higher values of latex content this reduction is significantly high.

iii. In the case of specimens with steel fibres and latex, the first crack and ultimate load of the specimen have been found to improve marginally and no definite trend has been observed as the quantity of latex and fibre content increases. However these specimens have shown better post- peak load deflection performance. The energy absorption capacity and the ductility factor given by these specimens were significantly higher than conventional specimens. The details of computations of energy absorption capacity and ductility factor are given in subsequent sections.

iv. The improvement in the behaviour of specimens as the fibre content increases may be due to the following reasons. When fibres are added to concrete, they intercept the cracks, and will not allow them to propagate in the same direction. Hence the cracks intercepted by the fibres have to take a meandering path which requires more energy for further propagation resulting in higher load carrying capacity.

v. In the case of latex modified concrete specimens, the addition of smaller percentages of DRC fill all the voids present the concrete and there by improves the density of concrete. Hence an improvement in the first crack load and ultimate load was noticed. The addition of higher percentages of DRC (above 1.0%) than those required for filling the voids leads to zones of weak spots which in turn reduces the ultimate load carrying capacity of the members.

4.5.1 Load deflection behaviour

The recorded values of load and deflection have been used to obtain the P-δ plots. Fig. 4.5 to 4.9 show the load deflection plots for :

- i. Plain beam and SFRC beams (Fig. 4.5)
- ii. Plain beam and latex modified beams (Fig. 4.6)
- iii. Latex modified SFRC beams with V_f=0.5% (Fig. 4.7)
- iv. Latex modified SFRC beams with V_f=1.0% (Fig. 4.8)
- v. Latex modified SFRC beams with V_f=1.5% (Fig. 4.9)

Referring to Figs. 4.5 to 4.9, it may be noted that :

i. All the curves are linear up to the formation of first crack and then they become non linear due to the formation of multiple cracks and propagation of the same up to ultimate load. In the case of conventionally reinforced concrete specimen a sudden drop in the load was noticed beyond the peak load and then the load drops at a reduced rate.

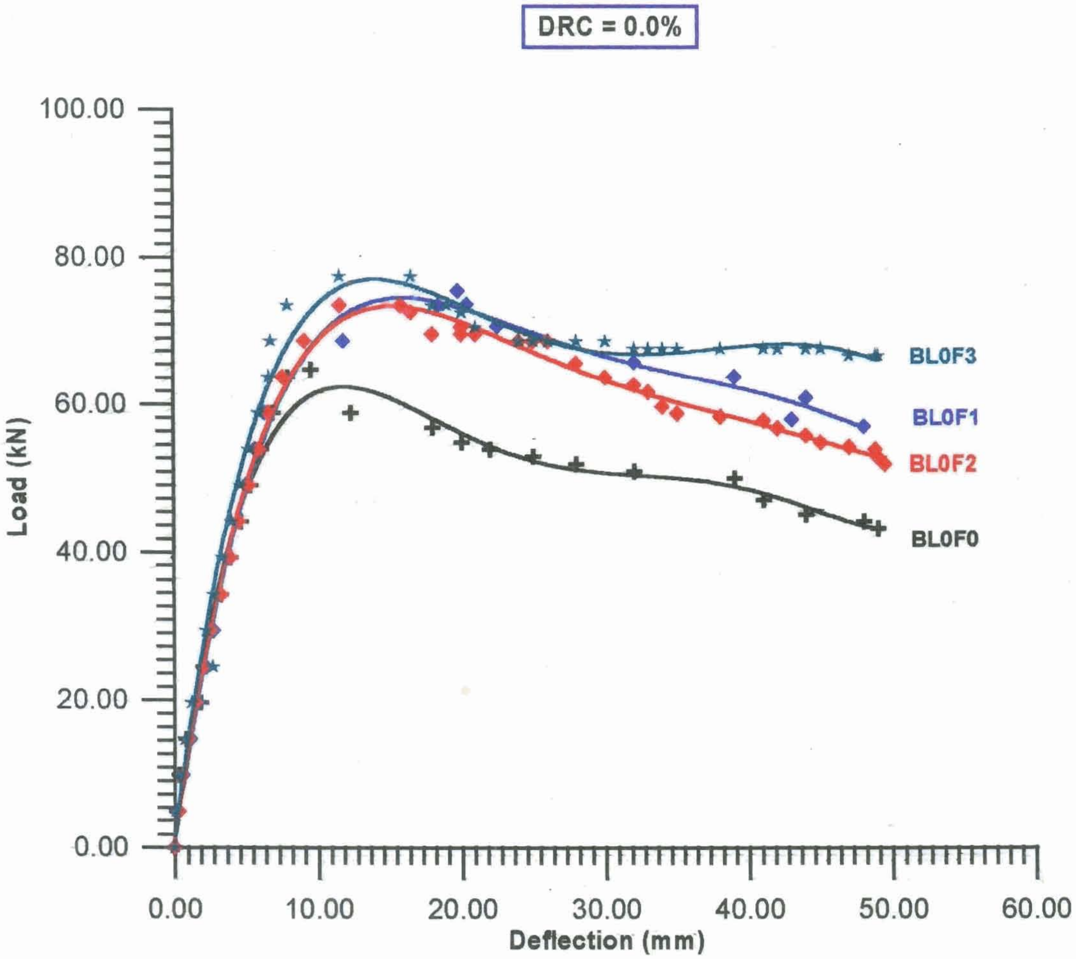


Fig. 4.5 Load versus Deflection plot for beams with steel fibres

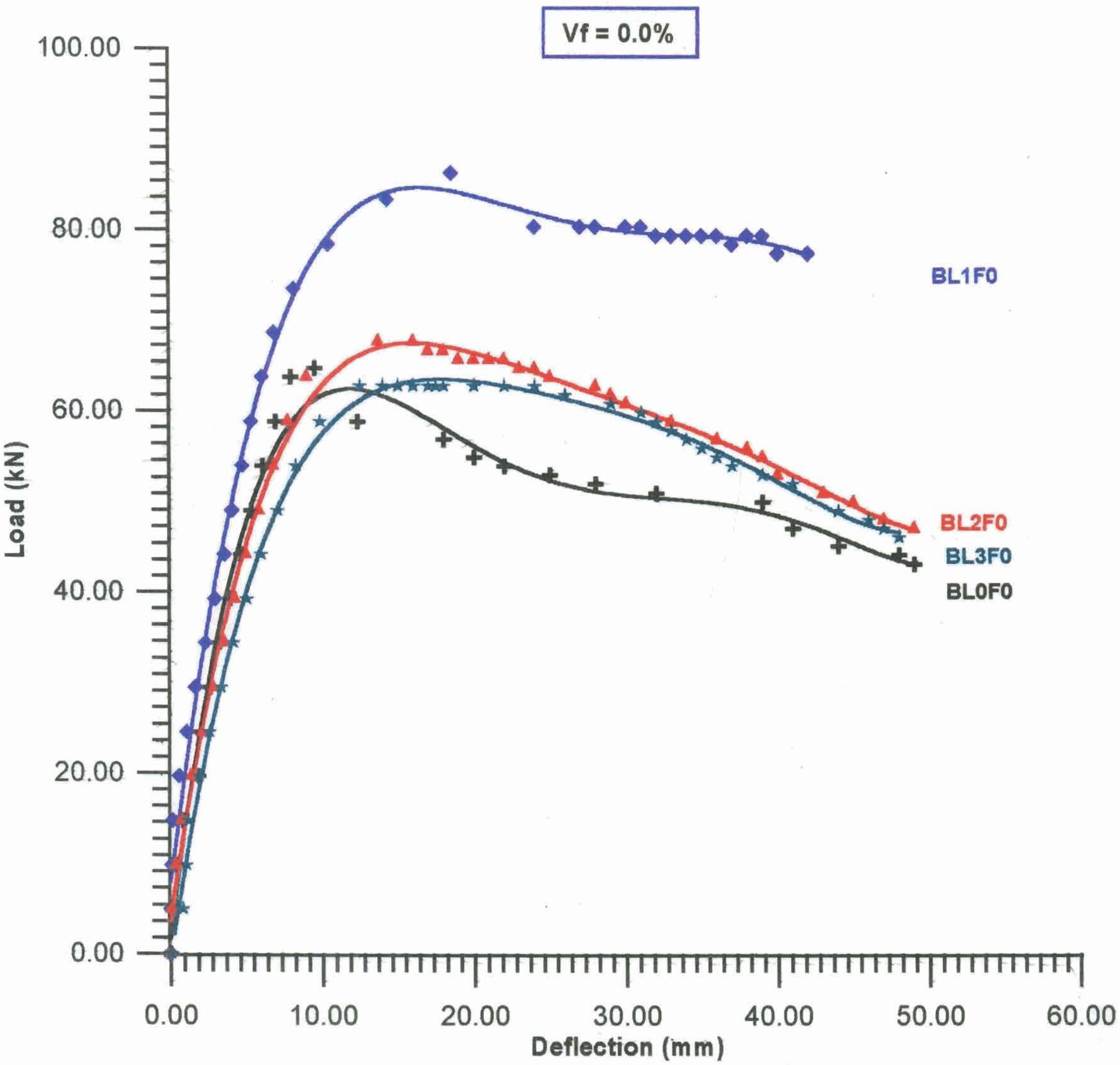


Fig. 4.6 Load versus Deflection plot for beams with different % DRC

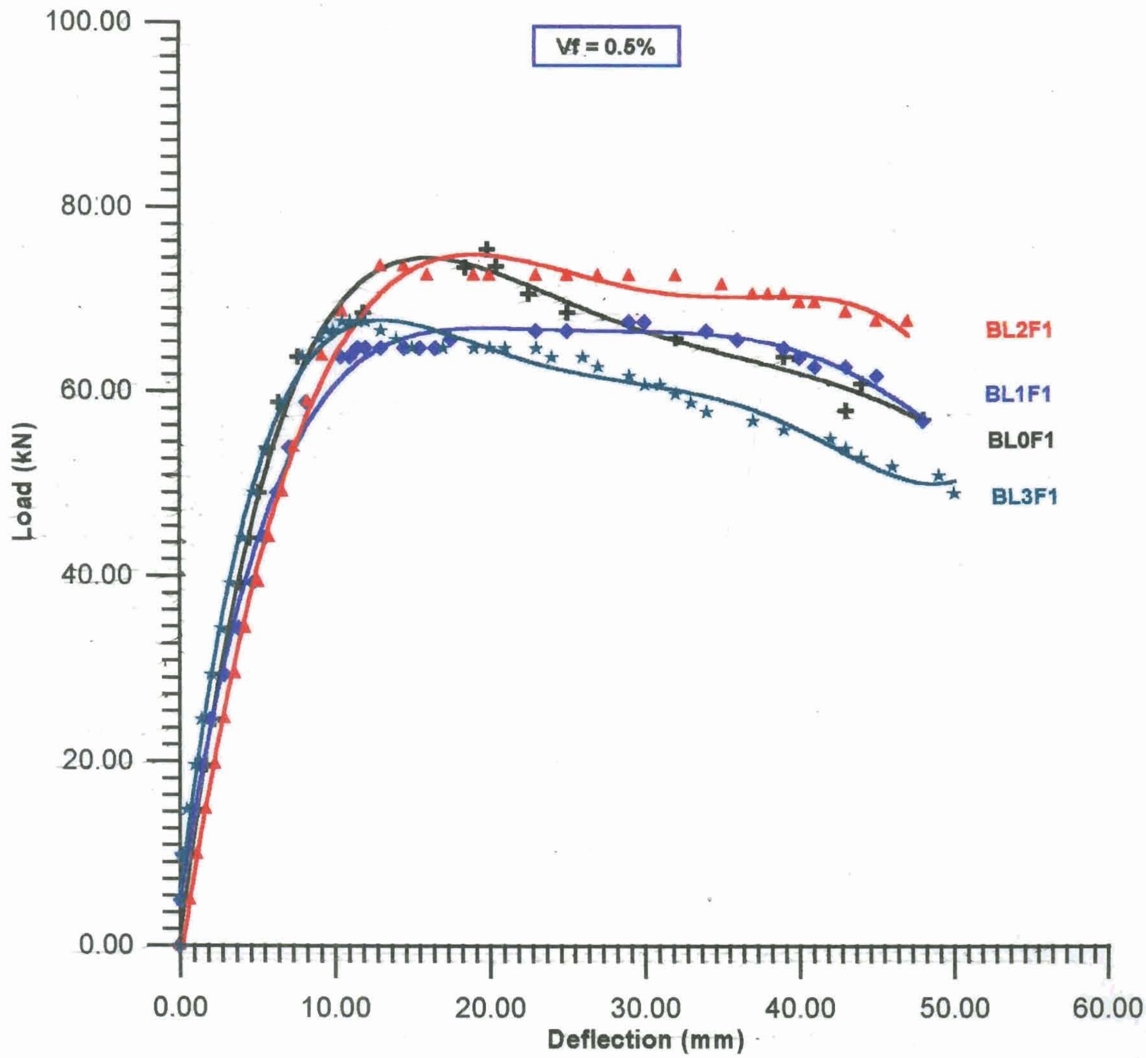


Fig. 4.7 Load versus Deflection plot for beams with Vf=0.5% and different DRC %

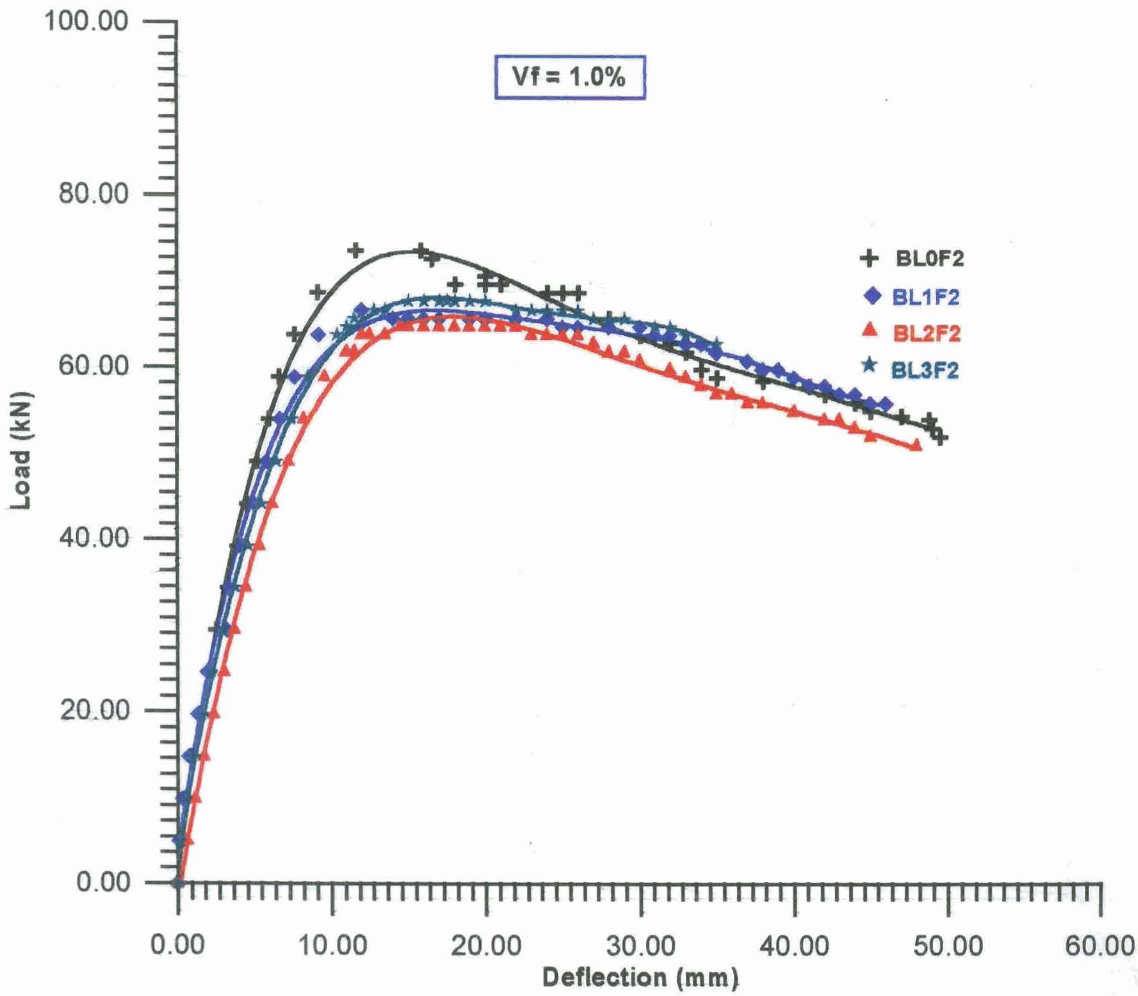


Fig. 4.8 Load versus deflection plot for beams with Vf=1.0% and different DRC (%)

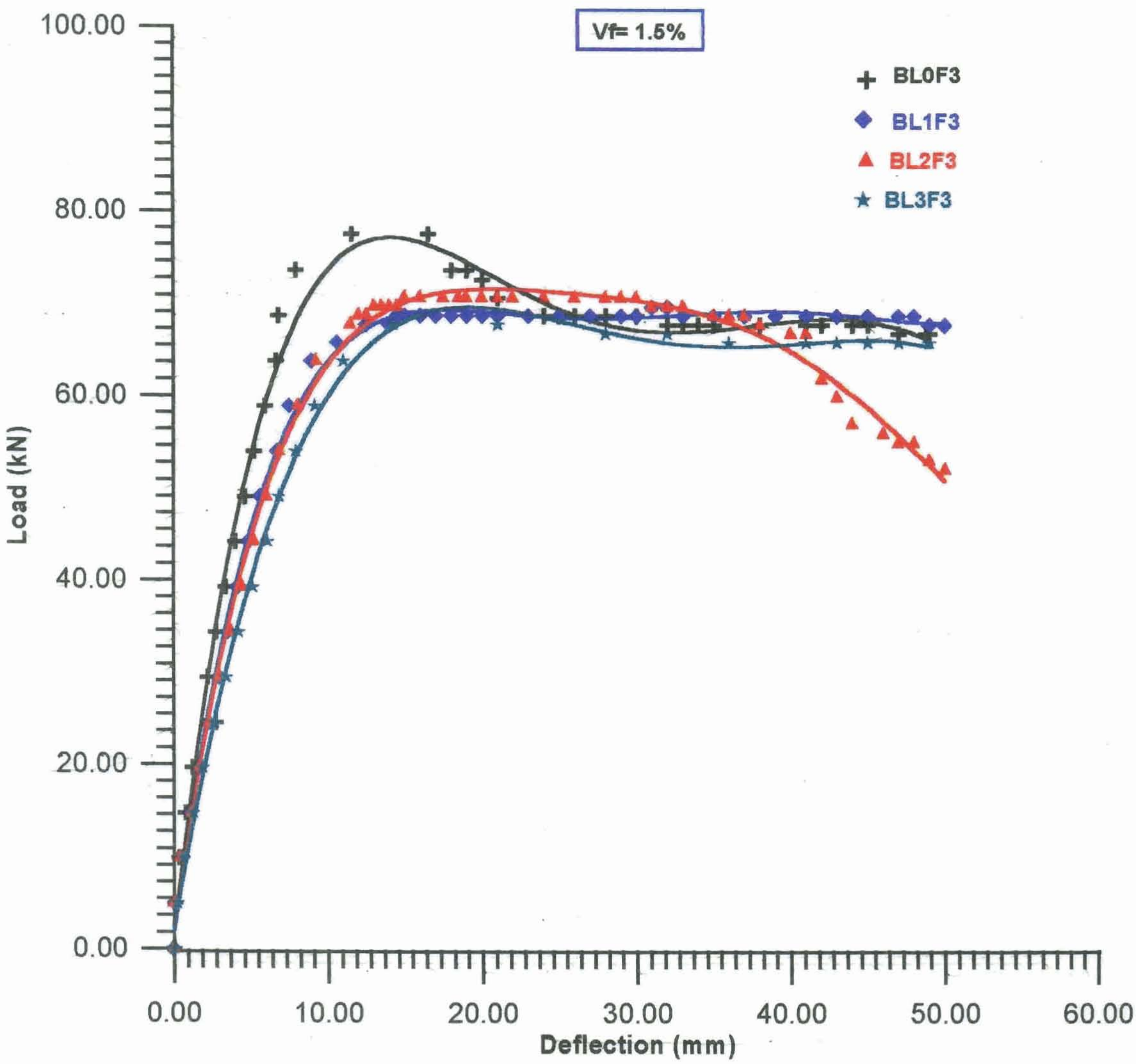


Fig. 4.9 Load versus deflection plot for beams with Vf=1.5% and different DRC (%)

On the other hand, the latex modified and steel fibre concrete specimens exhibit more or less a flat descending portion of the curve beyond peak load. This indicate the improvement in the dimensional stability and structural integrity of the specimen even beyond the peak load when steel fibres are added and modified by latex polymers.

ii. In general, the energy absorption capacity or toughness as indicated by the area under the load deflection plot is higher for the latex modified specimens and steel fibre reinforced concrete specimens.

iii. Referring to Fig. 4.5, it may be noted that the peak load, deflection at peak load, the area under the P-δ curve are found to be higher than that of the conventionally reinforced concrete specimen. The specimens with V_f=1.5% fibres exhibit better behaviour than other specimens.

iv. Fig. 4.6 indicate that specimens with 0.5% DRC improve the peak load and deflection at peak load significantly. On the other hand those with 1.0% and 1.5% marginally improve the above behaviour.

v. Figs. 4.7, 4.8 and 4.9 show the load deflection plots of latex modified steel fibre concrete conventionally reinforced concrete specimens with different combinations of volume fractions of steel fibres and DRC. It may be noted from these figures that the plots are close to each other and not differing appreciably. However, the specimen with V_f=0.5% and DRC=1.0% has shown better performance in terms of peak load, deflection at peak load and energy absorption capacity (area under the p-δ curve).

4.6 Analysis of Test Results

4.6.1 Variation of ultimate load with latex fibre index

The values of ultimate load of the beams are influenced by several parameters such as volume fraction of steel fibres, amount of DRC and compressive strength of concrete. Hence an attempt is made to obtain a relationship between the ultimate load given by the specimens and these parameters. After the trial of several combinations of V_f and DRC, a parameter called **L**atex **F**ibre **I**ndex was developed and is given by

$$LFi = (1+DRC) + (1+V_f) \qquad \text{....(4.1)}$$

Since the amount of compaction, type of mixing, small variation in the water-cement ratio affect the ultimate load, it is normalised by dividing it by a factor bdf_{ck}. Then the normalised ultimate load is given by

$$\text{Normalised } P_u = \frac{P_u}{bdf_{ck}} \qquad \text{...(4.2)}$$

where b and d are the breadth and effective depth of beams and f_{ck} is the cube compressive strength. The values of $\frac{P_u}{bdf_{ck}}$ were plotted against the values of LF_i as shown in Fig.4.10. From the figure it may be noted that, $\frac{P_u}{bdf_{ck}}$ increases as the value of LF_i increases in a non linear form up to $LF_i = 2.9$. Beyond this value, normalised Pu decreases because of the presence of high values of DRC which reduces the load carrying capacity as explained in section 3.3.4. A best fit equation is obtained for the plot and is given by

$$\frac{P_u}{bdf_{ck}} = 0.033\, LF_i - 0.0055\, (LF_i)^2 + 0.144 \qquad \text{...(4.3)}$$

From the above equation for a give value of LF_i, P_u can be determined.

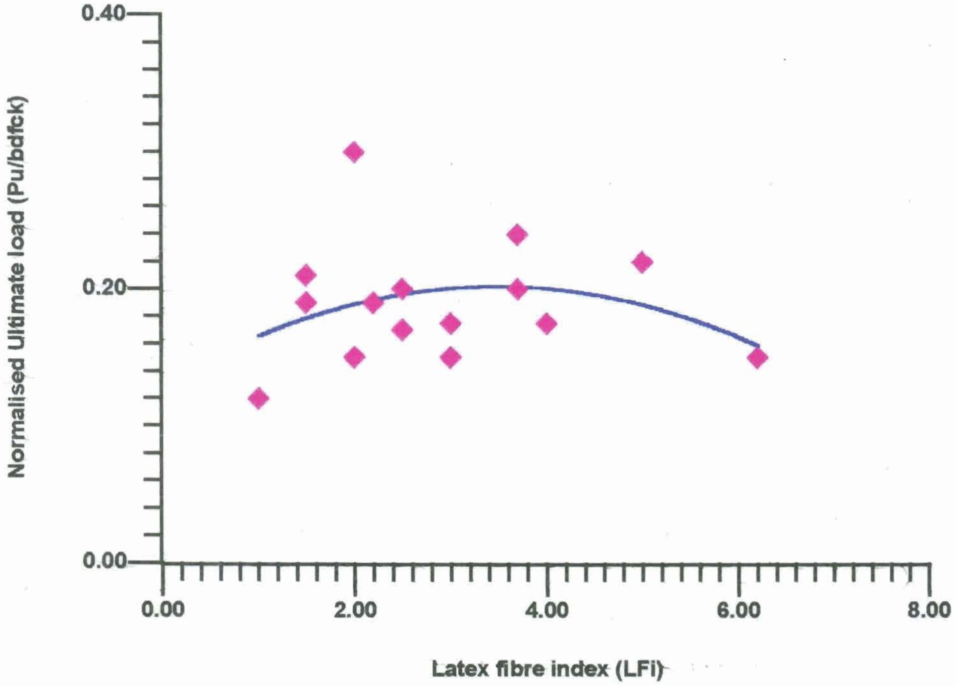


Fig. 4.10 Plot of normalised ultimate load versus Latex fibre index (LFi)

4.6.2 Energy absorption capacity and toughness index.

Energy absorption capacity

An attempt was made to obtain the energy absorption capacity of the specimens. In general, the term "Energy absorption capacity" of a given material could be obtained only from the full Load versus Deflection (P-δ) curve of the specimen. But due to the inherent limitations of the testing machine, the load (P) and the deflection (δ) readings beyond the peak load could not be noted for all stages of post peak loading. However, the post peak readings were recorded till the load reduced to 80% of the peak load (P_u). Hence the area under the P- δ curve considered in this study consists of the area under the ascending portion up to the peak load and under descending portion up to 0.80 P_u. The values of energy absorption capacity were computed from the area of (P-δ) curve for each specimen and are given in Table 4.5

From Table 4.5, it can be seen that the values of energy absorption capacity (U) computed from the P- δ curve using 80% of the post peak load as cut-off point, for all the specimens indicate that, as the values of V_f and DRC increases, energy absorption capacity increases significantly and this increase does not follow any particular trend.

Toughness index

It is well known that concrete will be effective in resisting the load until the formation of first crack. At this stage concrete is relieved of its tensile stress and steel takes the entire load at the cracked section. Hence an attempt is made to obtain the area under the P-d curve up to first crack load. Then the area obtained from the P- δ curve with 80% of post peak load as cut off point was divided by the area computed up to the first crack load, and this is termed as Toughness Index i.e.,

$$\text{Toughness Index} = \frac{\text{Area under the post peak P- } \delta \text{ curve up to } 0.8\ P_u}{\text{Area under the P - } \delta \text{ curve up to } P_{cr}} \quad ....(4.4)$$

This is similar to the procedure adopted by Japanese concrete Institute (JCI) for obtaining toughness index for steel fibre concrete [95] Table 4.5 gives the values of Toughness Index. From Table 4.5 it can be seen that the addition of latex improve the toughness index of conventionally reinforced concrete beams. The beam with 0.5% DRC has shown higher toughness index compared to 1.0% and 1.5% DRC. This is due to the reason that addition of higher quantities of latex, other than that required for filling the voids in the concrete, form weak spots/interfaces and crack originate from these locations and propagation of the same cause immediate failure leading to reduction in ultimate load, toughness etc. A similar trend has been noticed in the case of conventionally reinforced concrete beams additionally reinforced with steel fibres. The combined effect of latex and steel fibres indicate that the beams with DRC ranging from 0.5 to 1.0% and volume fraction of steel fibres ranging from 0.5 to 1.0% have given higher values of toughness index when compared to other combination of latex and steel fibres.

Table - 4.5
Energy Absorption Capacity (U) and Toughness Index (T-I) of Beams

Sl.No	Beam Designation	Energy Absorption Capacity(KN-mm)		Toughness Index	
		Absolute	Relative	Absolute	Relative
1	$BL_0 F_0$	941.92	1.00	92.80	1.00
2	$BL_0 F_1$	4188.02	4.45	309.37	3.33
3	$BL_0 F_2$	2549.43	2.70	229.92	2.48
4	$BL_0 F_3$	5055.86	5.37	336.69	3.63
5	$BL_1 F_0$	4601.72	4.88	389.93	4.20
6	$BL_2 F_0$	2379.10	2.52	333.21	3.59
7	$BL_3 F_0$	2466.32	2.62	319.90	3.45
8	$BL_1 F_1$	3810.80	4.04	292.20	3.15
9	$BL_2 F_1$	4374.26	4.64	607.50	6.54
10	$BL_3 F_1$	2755.32	2.92	306.10	3.30
11	$BL_1 F_2$	27040.31	2.87	380.92	4.09
12	$BL_2 F_2$	2568.58	2.73	294.92	3.18
13	$BL_3 F_2$	4028.61	4.27	424.02	4.56
14	$BL_1 F_3$	5524.18	5.86	424.90	4.58
15	$BL_2 F_3$	2614.26	2.77	244.90	2.64
16	$BL_3 F_3$	3842.75	4.07	269.28	2.90

Note : Values of relative $= \dfrac{\text{Energy absorption / Toughness of Specimen}}{\text{Energy absorption / Toughness of Specimen } BL_0 F_0}$

4.6.3 Ductility factor :

Ductility may be defined, in general, as the ability of a structure to undergo inelastic deformations beyond the initial yield deformation with no decrease in the load resistance as shown in Fig. (4.11).

The ductility of a member can be measured using load – deformation response. The deformation may be strain / rotation / curvature / deflection etc.. The ratio of ultimate deformation to the deformation at the first yield is defined as ductility factor.

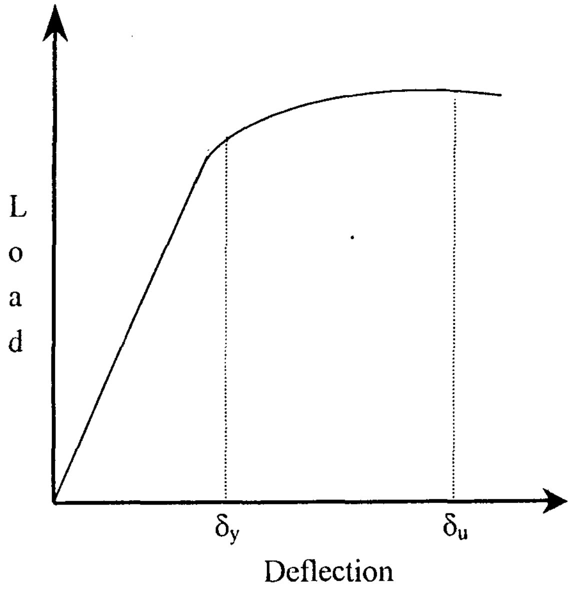


Fig. 4.11 Typical load versus deflection plot for determination of ductility factor

Ductility factor is an important parameter considered in the design of structures subjected to large deformation. Generally it is defined in the case of members subjected to flexure as:

$$\text{Ductility factor} = \frac{\text{Ultimate deformation}}{\text{Deformation at first yield}} = \frac{\delta_u}{\delta_y} \qquad \text{... (4.5)}$$

Since the addition of steel fibres and latex impart high ductility to concrete, an attempt was made to obtain the ductility factor for the specimens tested in this study. The value of δ_u were obtained for each specimen from the test results. For obtaining δ_y, the load at which the steel yields (P_y) has to be calculated and then from (P-δ) curve, δ_y is obtained.

For obtaining the load at which the steel yields (P_y), the following procedure was adopted.

From the elastic cracked section theory [20, 71], assuming that the steel was not stressed beyond its yield strength, the strain in steel can be obtained as follows:

$$\varepsilon_s = \frac{f_s}{E_s} = \frac{M\,(d - x)}{I_{cr} E_c} \qquad \text{...(4.6)}$$

$$\therefore\ f_s = \frac{M}{I_{cr}}(d - x)\frac{E_s}{E_c} \qquad \text{...(4.7)}$$

At yielding $f_s = f_y$ and the above equation becomes

$$f_y = \frac{M_y}{I_{cr}}(d - x)\frac{E_s}{E_c} \qquad \text{...(4.8)}$$

$$\therefore\ M_y = f_y\left(\frac{E_c}{E_s}\right)\frac{I_{cr}}{(d - x)} \qquad \text{...(4.9)}$$

From the geometry of the loading scheme (Fig. 4.2)

$$M_y = 0.375\ P_y \qquad \ldots(4.10)$$

$$\therefore\ P_y = \frac{M_y}{0.375} \qquad \ldots(4.11)$$

where I_{cr} = Cracked moment of Inertia of the transformed section
m = modular ratio
M_y = moment at which yielding starts
P_y = load at which yielding starts
x = depth of neutral axis
d = effective depth of beam

The values of load P_y at which steel starts yielding were computed using equation (4.11). Then the deflection corresponding to this load at which steel starts yielding (δ_y) was obtained from the load deflection curve of the specimen. Using equation (4.5) the values of ductility factor for the specimens tested were obtained. A relative comparison of ductility factor is given in Table 4.6. It can be seen that latex modification up to certain percentages improve the ductility of the conventionally reinforced concrete beams. This is very significant in beams with 0.5% DRC. In fact a marginal improvement in ductility factor has been noticed in the case of beams with 1.0 and 1.5% DRC. Beams with 0.5% volume fraction of fibres have shown higher ductility compared to that of beams with 1.0 and 1.5% fibres. The combined effect of latex and steel fibres on the conventionally reinforced concrete beams indicate that, the beams with 0.5% DRC and 0.5 to 1.0% volume fraction of steel fibres have given higher values of ductility factors. This may be due to the following reasons. When small percentage of latex is added to concrete, it fills the voids in the concrete and enhances the density and strength of concrete. Similarly when steel fibres of 0.5 to 1.0% are added, the fibres arrest the crack propagation by bridging across the cracks. Due to this, the cracks could not propagate in the same plane and have to take a deviated path resulting in higher energy demand for further propagation. This in turn increases the load carrying capacity and deflection at ultimate load.

Table - 4.6
Ductility Factors

Beam desgn.	Ultimate load P_u (KN)	Deflection at ultimate load δ_u (mm)	Yield load P_y (KN)	Defl. At yield load δ_y (mm)	D.F. = δ_u/δ_y	
					Absolute	Relative
BL_0 F_0	64.68	9.20	54.12	6.05	1.520	1.00
BL_0 F_1	75.46	19.80	53.66	5.50	3.600	2.37
BL_0 F_2	73.50	11.57	53.49	5.20	2.225	1.46
BL_0 F_3	77.42	11.52	50.16	4.80	2.400	1.58
BL_1 F_0	86.24	18.52	57.47	5.20	3.561	2.34
BL_2 F_0	67.62	13.70	59.88	7.70	1.780	1.17
BL_3 F_0	62.72	22.00	61.02	9.90	2.220	1.46
BL_1 F_1	67.62	33.00	52.76	6.90	4.782	3.15
BL_2 F_1	73.50	14.00	55.90	7.50	1.867	1.23
BL_3 F_1	67.62	10.50	63.02	7.80	1.346	0.88
BL_1 F_2	66.64	12.00	55.33	6.80	1.765	1.16
BL_2 F_2	64.68	14.50	63.78	13.00	1.115	0.73
BL_3 F_2	67.62	18.00	66.29	11.70	1.538	1.01
BL_1 F_3	69.58	17.00	49.08	5.60	3.035	1.99
BL_2 F_3	70.56	22.00	52.76	6.70	3.283	2.16
BL_3 F_3	67.62	14.90	56.91	8.30	1.795	1.18

When higher percentages of latex i.e., 1.0 and 1.5% are added, the excess quantity other than that required for filling the inherent voids in the concrete mass form weak spots in concrete. Cracks originate from these weak spots and cause immediate failure. Also higher percentage of volume fraction of steel fibres (i.e., V_f=1.5%), cause balling effect in the fibre concrete mix and due to this workability gets reduced and density of concrete decreases. This in turn affects the ultimate load and deflection at ultimate load. The fore-said developments, due to the addition of higher quantities of DRC and steel fibres, lead to the reduction in ductility factors. These results indicate that addition of 0.5% DRC and steel fibres up to V_f = 1.5% appears to improve the ductility of conventionally reinforced concrete flexural members significantly.

An attempt is made to relate the values of energy absorption capacity (U) and toughness index (TI) and ductility factor (DF) with the values of latex fibre index (LF_i) Fig.4.12, 4.13 and 4.14 shows the plots relating to U, TI and DF with LF_i. It may be seen from these plots that U, TI and DF varies in a non linear form and increases as the value of LF_i = 4.1, 3.9 and 3.0 respectively. Beyond these value, U, TI and DF decreases. This is mainly due to the addition of higher percentages of DRC which causes reduction in the strength of specimens and has been already discussed in section 3.3.4. The regression equation obtained for these plots are as follows

a) $U = 0.88\,(LF_i) - 0.1\,(LF_i)^2 + 2.1$

b) $TI = 2.06\,(LF_i) - 0.28\,(LF_i)^2 + 0.36$

c) $DF = 0.21\,(LF_i) - 0.036\,(LF_i)^2 + 1.4\,9$

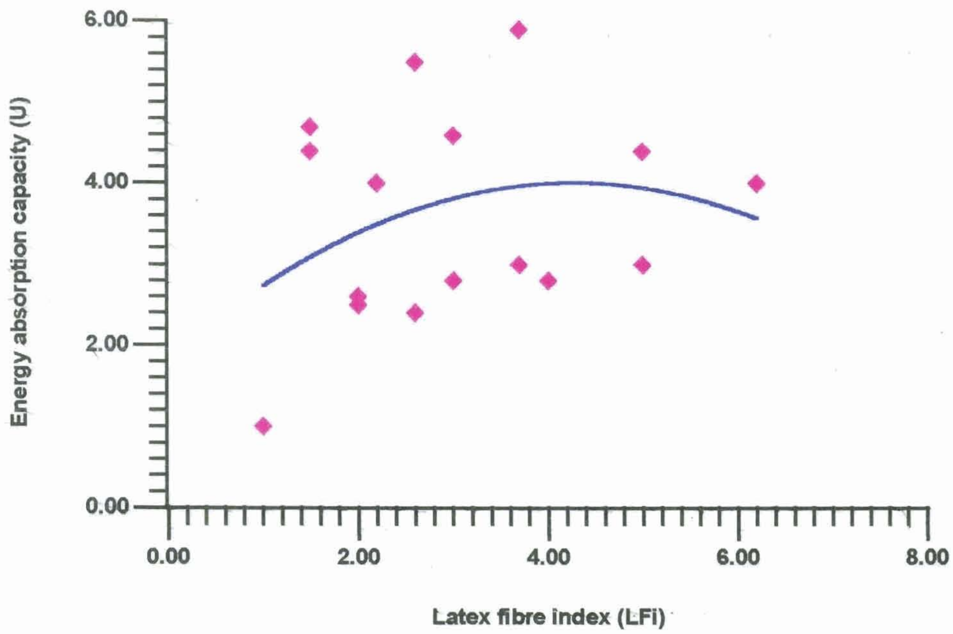


Fig. 4.12 Plot of energy absorption capacity versus Latex fiibre index (LFi)

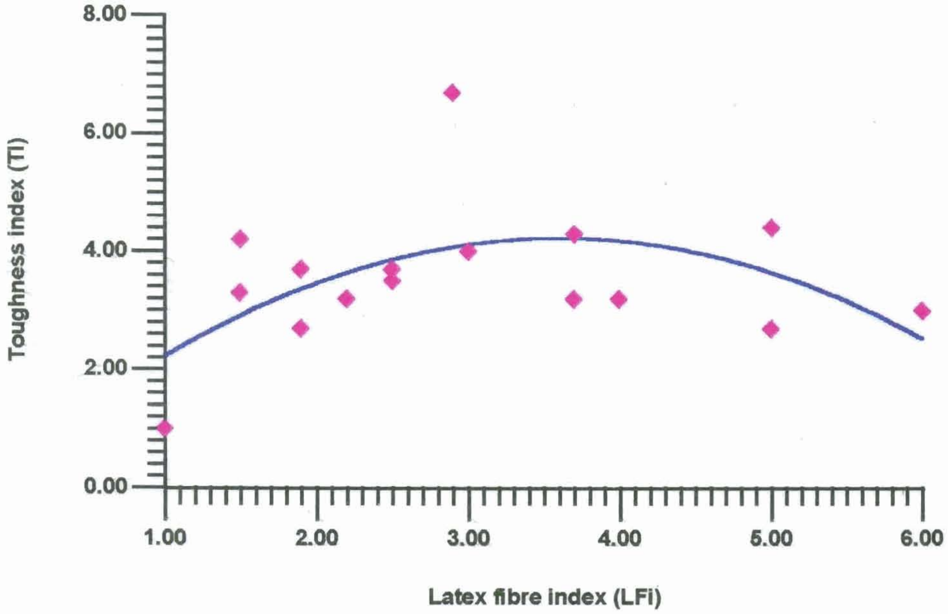


Fig. 4.13 Plot of Toughness index versus Latex fibre index (LFi)

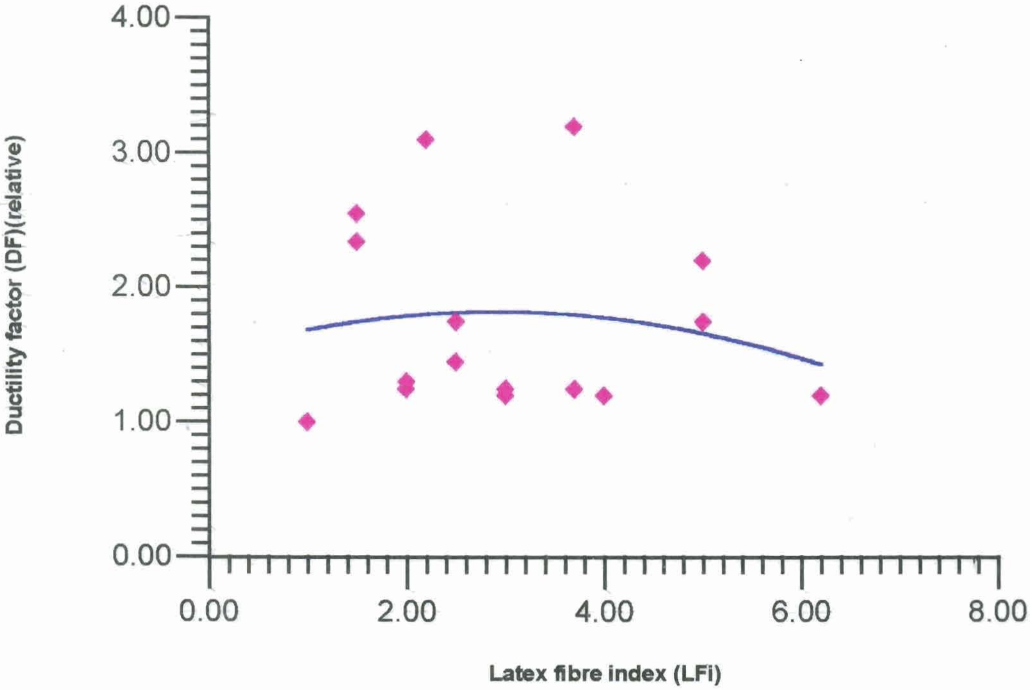


Fig. 4.14 Plot of Ductility factor (DF) versus Latex fibre index (LFi)

4.7 Prediction of First crack load and Ultimate Moment of Resistance

An attempt is made to predict the first crack load and ultimate moment of resistance of latex modified steel fibre reinforced concrete flexural members.

4.7.1 First crack load : An attempt is made to predict the first crack load in the latex modified steel fibre reinforced concrete flexural members. The following procedure is adopted for predicting the first crack load.

The cracking moment M_{cr} of any flexural member can be determined using the following equation

$$M_{cr} = \frac{f_r \, I_g}{y_t} \qquad \text{...(4.12)}$$

Where f_r = Modulus of rupture of the material used

I_g = Moment of inertia of the gross transformed section

y_t = Distance of extreme tension fibre from the neutral axis

In equation (4.12) the only unknown is f_r. While equations are available for obtaining f_r for plain concrete, no equations are available for obtaining f_r in latex modified steel fibre concrete. An attempt is made to obtain a relation between f_r and other physical parameters such as volume fraction of steel fibres (V_f), Dry Rubber Content (DRC) etc. The value of f_r were obtained from the prism tests according to IS 516-1959 and are shown in Table 4.4. As the variables considered in this study are different percentages of DRC and volume fraction of steel fibres, an attempt is made to obtain a parameter which takes into account he combined effect of DRC and V_f. After trying several combinations of DRC and V_f, a parameter called Latex- Fibre Index LF_i was obtained which is given by the equation

$$LF_i = (1 + DRC)*(1 + V_f) \qquad \text{....(4.13)}$$

Fig.4.15 shows the plot of f_r versus LF_i. The best fit equation for the plot is given by

$$f_r = 0.0167\,(LF_i)^2 - 0.19\,(LF_i) + 4.79 \qquad \ldots(4.14)$$

The values of f_r for different combinations of DRC and V_f can be determined from the equation (4.14). After determining the values of f_r, M_{cr} can be found from the equation (4.12). Then the first crack load (P_{cr}) is computed for the loading scheme considered in this study as follows.

For the given loading scheme,

$$M_{cr} = 0.375\,P_{cr} \qquad \ldots(4.15)$$

$$P_{cr} = \frac{M_{cr}}{0.375} \qquad \ldots(4.16)$$

The computed values of $P_{cr(theo)}$ were compared with the experimental values of $P_{cr(expt)}$. The comparison is given in Table 4.7. It may be noted from Table 4.7 that, the ratio of $P_{cr(theo)}$ / $P_{cr(exp)}$ ranges from 0.815 to 1.63. The average value of the ratio $P_{cr(theo)}$ / $P_{cr(exp)}$ is 1.117 and the coefficient of variation is 22.1%. This shows that the proposed method overestimates the first crack load by 11.7%. The coefficient of variation obtained ie.22.1% could be considered small for materials like latex modified steel fibre concrete, which is highly heterogeneous. Therefore the comparison could be considered satisfactory.

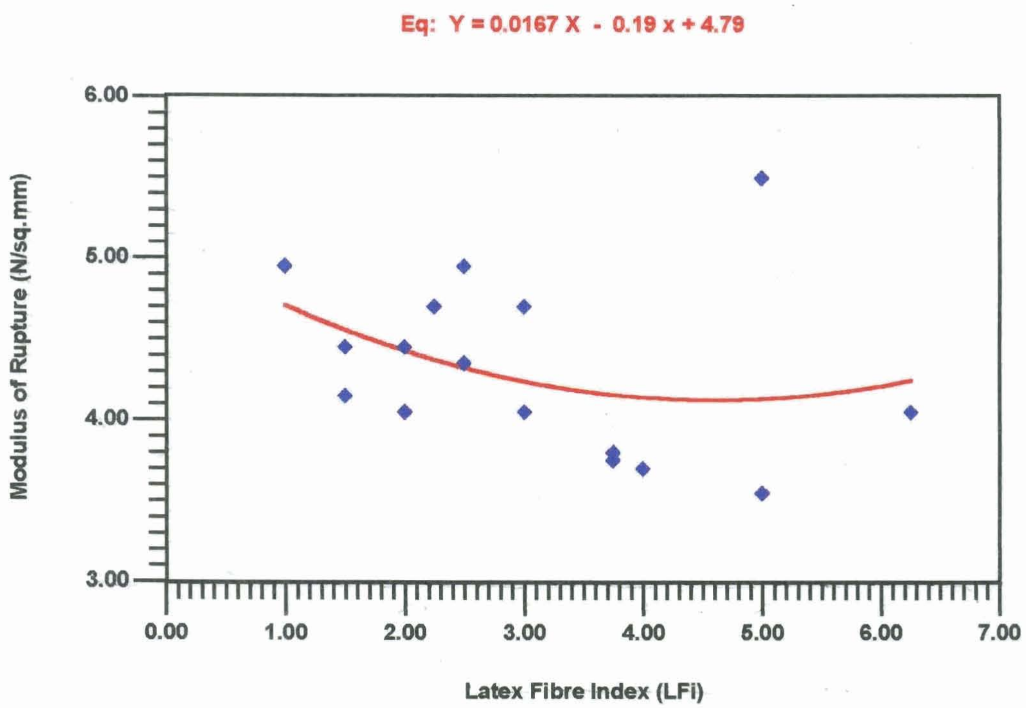


Fig. 4.15 Plot of Modulus of Rupture (fr) versus Latex Fibre Index (LFi)

Table - 4.7

Comparison of theoretical with experimental First Crack Load

Sl.No	Beam Designation	$P_{cr\,(theoritical)}$ (KN)	$P_{cr\,(Experimental)}$ (KN)	Ratio of $\frac{P_{cr\,(theoritical)}}{P_{cr\,(Experimental)}}$
1	$BL_0\,F_0$	16.23	14.70	1.104
2	$BL_0\,F_1$	13.79	15.68	0.879
3	$BL_0\,F_2$	14.04	14.70	0.955
4	$BL_0\,F_3$	14.38	17.64	0.815
5	$BL_1\,F_0$	15.48	14.70	1.053
6	$BL_2\,F_0$	15.99	09.80	1.630
7	$BL_3\,F_0$	16.71	14.70	1.136
8	$BL_1\,F_1$	14.07	11.76	1.196
9	$BL_2\,F_1$	14.22	09.80	1.451
10	$BL_3\,F_1$	14.45	09.80	1.474
11	$BL_1\,F_2$	15.42	13.72	1.124
12	$BL_2\,F_2$	18.42	11.76	1.566
13	$BL_3\,F_2$	19.36	14.70	1.317
14	$BL_1\,F_3$	13.90	14.70	0.945
15	$BL_2\,F_3$	14.69	15.68	0.937
16	$BL_3\,F_3$	18.38	14.70	1.250

Average **1.117**
COV **22.10 %**

4.7.2 Ultimate moment of resistance

An attempt is made to predict the ultimate moment of resistance of latex modified steel fibre reinforced concrete flexural members. Referring to literature it was noted that Paramasivam et al [68] appears to be the first to develop an expression for flexural strength of reinforced steel fibrous concrete beams. Theoretical values of ultimate moment were computed using the method proposed by Paramasivam et al [68] as follows.

Based on the idealised stress block shown in Fig.4.16, they obtained expression for ultimate moment and is

$$M_u = \sigma_{tu} \frac{bh h_t}{2} + \frac{A_s f_y}{2} (h + h_t) \quad \text{...(4.17)}$$

where σ_{tu} = ultimate tensile strength of composite and is given by

$$\sigma_{tu} = \eta_l \eta_0 V_f l_f \frac{\tau_u}{2r} \quad \text{...(4.18)}$$

where η_l = length efficiency factor

η_0 = orientation factor due to the realignment of fibres bridging the crack

V_f = volume fraction of fibres

l_f = length of fibre

τ_u = ultimate bond stress

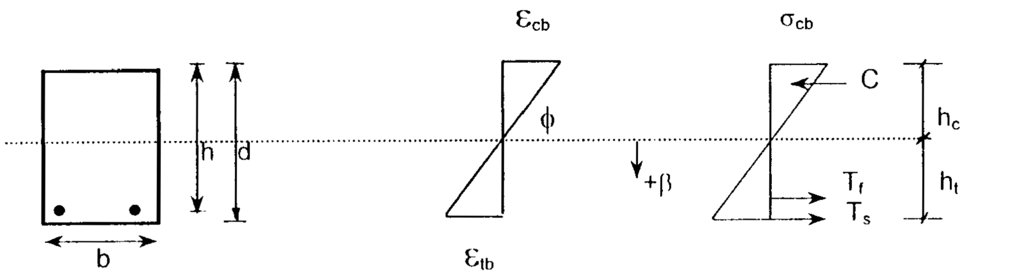
r = radius of fibre

The length efficiency factor is calculated based on the following

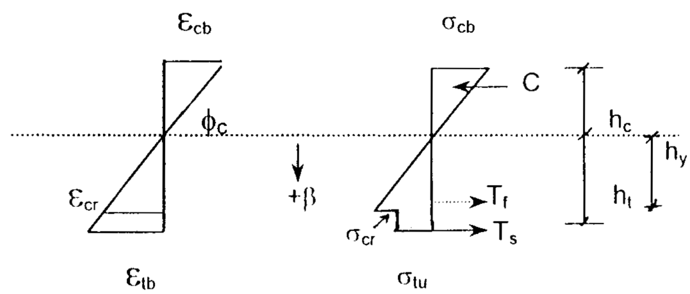
$\eta_l = 0.5$ for $l_f < l_c$

$\eta_l = 1-(l_c/2l_f)$ for $l_f > l_c$

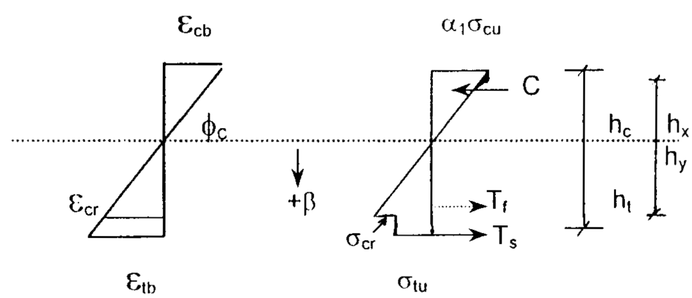
l_c is the critical length of fibre and is given by



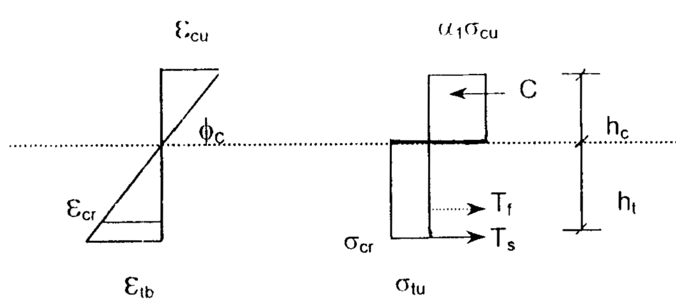
a. Elastic Range $(0 \leq \phi \leq \phi_{cr})$



b. Cracked Range $(\phi_{cr} \leq \phi_c \leq \phi_{cr1})$



c. Cracked Range $(\phi_{cr1} \leq \phi_c \leq \phi_{cr2})$
$(\phi_{cr2} \leq \phi_c \leq \phi_u)$



d. Ultimate Condition

Fig. 4.16 Idealised stress block considered for the analysis [68]

$$l_c = 0.5 \frac{\sigma_{fu} \; d_f}{\tau_u} \qquad \ldots(4.19)$$

η_0 = orientation factor due to the realignment of fibres bridging the crack and is given by

$$\eta_0 = \frac{\int_0^{\rho}\int_0^{\theta} \cos\theta \cos\rho \; d\theta \; d\rho}{\int_0^{\rho}\int_0^{\theta} d\theta \; d\rho} \qquad \ldots(4.20)$$

Where $\theta = \sin^{-1}(h/l_f)$

$\rho = \sin^{-1}(b/l_f)$

h_t = depth of zone under tension and is given by

$$h_t = \frac{\alpha_l \; \sigma_{cu} \; d - \frac{A_s \; f_y}{b}}{(\alpha_l \; \sigma_{cu} + \sigma_{tu})} \qquad \ldots(4.21)$$

where α_l = 0.9 for fibrous concrete

σ_{tu} = ultimate bond strength (=2.0 from CP:110 Part-1)[14]

The values of M_u computed using the above equations are compared with the experimental values. The comparison is given in Table 4.8. It may be seen form the Table 4.8 that the method proposed by Paramasivam et al [68], under-estimates the ultimate moment by 21%. The coefficient of variation of the ratio $\frac{M_{u(theo)}}{M_{u(\exp)}}$ is 7.27%.

Table - 4.8

Comparison of theoretical with experimental moment of resistance

Sl.No	Beam Designation	Ultimate Moment by Paramasivam et. al (KN-m)	Ultimate Moment (Experimental) (KN-m)	Ratio of $M_{u(theo)}/M_{u(exp)}$
1	$BL_0\ F_0$	20.90	24.25	0.86
2	$BL_0\ F_1$	21.30	28.29	0.75
3	$BL_0\ F_2$	19.70	27.56	0.71
4	$BL_0\ F_3$	22.10	29.03	0.76
5	$BL_1\ F_0$	20.50	32.34	0.63
6	$BL_2\ F_0$	20.20	25.35	0.79
7	$BL_3\ F_0$	19.50	23.52	0.83
8	$BL_1\ F_1$	21.40	25.35	0.84
9	$BL_2\ F_1$	21.00	27.56	0.76
10	$BL_3\ F_1$	20.20	25.35	0.79
11	$BL_1\ F_2$	21.30	25.99	0.82
12	$BL_2\ F_2$	20.30	24.25	0.83
13	$BL_3\ F_2$	19.90	25.35	0.78
14	$BL_1\ F_3$	22.00	26.08	0.84
15	$BL_2\ F_3$	21.80	26.46	0.82
16	$BL_3\ F_3$	21.30	25.35	0.84

Mean	**0.79**
COV	**7.27 %**

Hence there is a discrepancy between the predicted values and the experimental values of ultimate moments. This discrepancy may be due to the following reason. The method propose by Paramasivam et al [68] is applicable to reinforced fibrous concrete flexural members. On the other hand in the present study, the concrete was modified by the addition of polymers such as latex and additionally reinforced with steel fibres. This would have caused the discrepancy between experimental and theoretical values. Hence further attempts have been made to modify the method proposed by Paramasivam et al [68] taking into account the effect of latex modification for the reinforced fibrous concrete flexural members.

4.7.3 Modification proposed :

This method follows the one proposed by Paramasivam et al earlier for reinforced steel fibre concrete flexural members with suitable modifications in order to account for the addition latex along with steel fibres.

In the above equation (4.18) the σ_{tu} (the ultimate tensile strength of the composite) was suitably modified in order to account for the addition of latex in this study and is given by

$$\sigma_{tu} = \eta_l \ \eta_0 \ V_f \ l_f \ V_{DRC} \ \frac{\tau_u}{2r} \qquad ..(4.22)$$

The calculated value of tensile strength of composite was substituted in the equation (4.17) to find the ultimate moment of resistance M_u The values of M_u computed using the above equations are compared with the experimental values. The comparison is given in Table 4.9. Referring to Table 4.9, it may be noted that the ratio of $M_{u(theo)}$ / $M_{u(exp)}$ ranges from 0.750 to 1.024. The average value of the ratio $M_{u(theo)}$ / $M_{u(exp)}$ is 0.930 and the coefficient of variation is 7.44 %. This shows that the proposed method estimates the ultimate moment of resistance satisfactorily.

Table - 4.9

Comparison of theoretical with experimental moment of resistance (after modification)

Sl.No	Beam Designation	Ultimate Moment after modification (KN-m)	Ultimate Moment (Experimental) (KN-m)	Ratio of $M_{u(theo)}/M_{u(exp)}$
1	$BL_0 F_0$	24.83	24.25	1.024
2	$BL_0 F_1$	25.12	28.29	0.888
3	$BL_0 F_2$	22.57	27.56	0.819
4	$BL_0 F_3$	25.66	29.03	0.884
5	$BL_1 F_0$	24.25	32.34	0.750
6	$BL_2 F_0$	23.83	25.35	0.940
7	$BL_3 F_0$	22.83	23.52	0.971
8	$BL_1 F_1$	25.28	25.35	0.991
9	$BL_2 F_1$	24.77	27.56	0.899
10	$BL_3 F_1$	23.55	25.35	0.929
11	$BL_1 F_2$	25.88	25.99	0.996
12	$BL_2 F_2$	23.43	24.25	0.966
13	$BL_3 F_2$	22.94	25.35	0.905
14	$BL_1 F_3$	26.50	26.08	0.989
15	$BL_2 F_3$	24.80	26.46	0.955
16	$BL_3 F_3$	24.61	25.35	0.971
			Mean	0.93
			COV	7.44 %

4.8 Load Factors

Load factor with respect to limit state of deflection are calculated to understand whether the load factor with respect to strength or with respect to deflection governs the design. The following procedure is adopted to calculate the load factor.

a) For serviceability conditions, the allowable total deflection δ_t is limited to span/250. The total deflection is the sum of short-term and long-term deflections.

b) The deflection obtained from the experiment is short time or immediate deflection δ_i, long term deflection δ_L is calculated as [71]

$$\delta_l = (2.0 - 1.2\frac{A_{SC}}{A_{ST}})\delta_i \qquad \text{....(4.23)}$$

where δ_i is the short time deflection and

$$(2.0 - 1.2\frac{A_{SC}}{A_{ST}})\delta_i \qquad > 0.60 \qquad \text{....(4.24)}$$

where A_{sc} = Area of steel in compression

A_{st} = Area of steel in tension

c) The total deflection (δ_t) of the specimen is equal to the sum of short time deflection δ_i and long time deflection δ_l

$$\delta_t = \delta_i + \delta_l \qquad \text{.....(4.25)}$$

$$= \delta_i + \delta_i\ (2.0 - 1.2\ (A_{sc}/A_{st}))\ \delta_i$$

The values of δ_t obtained form equation (4.25) is equated to (span/250) and δ_i is computed. Corresponding to this δ_i, the load P_δ is obtained form the experimental plots.

e) The load factor with respect to limit state of deflection can be calculated using,

$$L.F. = P_u / P_\delta \qquad \text{....(5.26)}$$

The values of load factors with respect limit state of deflection obtained for all the specimens are given in Table 4.10. It can be seen that the load factor for all the specimens are more than 1.5, which indicate that the deflection controls the design. Hence, while designing the latex- modified SFRC beams, sufficient attention is to be paid to the deflection criterion, in addition to the strength considerations.

Table 4.10

Load factor with respect to limit state of deflection

Sl. No.	Beam designation	Ultimate Load P_u(KN)	Load at δ_l=2.724 mm (KN) P_δ	Load Factor P_u/P_δ
1	$B_0L_0F_0$	64.68	33.50	1.93
2	$B_0L_0F_1$	75.46	32.50	2.32
3	$B_0L_0F_2$	73.50	31.00	2.37
4	$B_0L_3F_3$	77.42	35.50	2.18
5	$B_1L_1F_0$	86.24	38.00	2.27
6	$B_1L_2F_0$	67.62	29.50	2.29
7	$B_1L_3F_0$	62.72	26.00	2.41
8	$B_1L_1F_1$	67.62	29.00	2.33
9	$B_1L_2F_1$	73.50	24.00	3.06
10	$B_1L_3F_1$	67.62	37.00	1.82
11	$B_1L_1F_2$	66.64	28.00	2.38
12	$B_1L_2F_2$	64.68	23.50	2.75
13	$B_1L_3F_2$	67.62	28.00	2.41
14	$B_1L_1F_3$	69.58	28.50	2.44
15	$C_1L_2F_3$	70.56	28.50	2.47
16	$C_1L_3F_3$	67.62	27.00	2.50

4.9 Conclusions

Based on the experimental and analytical studies, the following conclusions are arrived at:

1. In general, addition of latex (0.5 to 1.0% DRC) improve the first crack load and the ultimate strength of flexural members.

2. The addition of steel fibres to latex modified concrete flexural members improve the cracking behaviour significantly. However, the load carrying capacity is only marginally improved.

3. The addition of latex improves the energy absorption capacity, toughness index and ductility factor significantly. This is more pronounced in the case of beams with DRC ranging from 0.5% to 1.0% and steel fibres up to a volume fraction of 1.0%

4. The overall improvement in energy absorption capacity, toughness index and ductility, achieved due to the addition of latex and steel fibres to conventionally reinforced concrete flexural members indicate that the latex modified steel fibre reinforced concrete is an appropriate material in the case of structures which are subjected to large deformations, cyclic loading etc.

5. The method proposed in this investigation predicts the first crack load and ultimate moment resistance of latex modified steel fibre reinforced concrete flexural members satisfactorily.

6. Load factor with respect to limit state of deflection controls the design of the latex modified steel fibre reinforced concrete beams when compared to those with respect to the limit state of collapse against flexure and the limit state of cracking.

CHAPTER 5

LATEX MODIFIED REINFORCED CONCRETE BEAMS WITH CONFINED SFRC IN THE COMPRESSION ZONE

5.1 Introduction

Strength and ductility are the two important factors governing the design of seismic resistant structures. Under seismic loading, the structures are subjected to large deformations. To resist these inelastic deformations effectively without resulting in the sudden failure of the structure, the concrete structural members must be able to absorb strain energy. This is possible only if the material is capable of withstanding considerable deformation without any reduction in its load carrying capacity. Structures, which resist dynamic loading, must be designed for energy absorption capacity in addition to its strength. Hence for concrete structures located in seismic zones, the improvement of ductility is of paramount importance. Confining the concrete in the compression zone increases its strength and ductility. Addition of steel fibres also adds to this type of improvement to a great extent. Further modification of concrete by natural rubber latex improves the ductility of the material with the retention of strength level of plain concrete.

It was felt that it would be worthwhile to study the combined effect of the three components on the strength and ductility of reinforced concrete flexural members. Hence an attempt was made in this direction.

The review of literature indicates that the strength and strain at peak load of conventionally reinforced concrete beams could be enhanced by providing confinement in the compression zone. The above mentioned properties could be further improved by the addition of steel fibres in the confined zone. Also, the addition of polymers like

---

* Based on the study presented in this Chapter, a technical paper entitled **"Performance of Latex Modified Concrete with Confined SFRC in the Compression Zone"**, has been presented at the Sixth NCB International Seminar on Cement and Building Materials held at New Delhi during 24-27, November 1998.

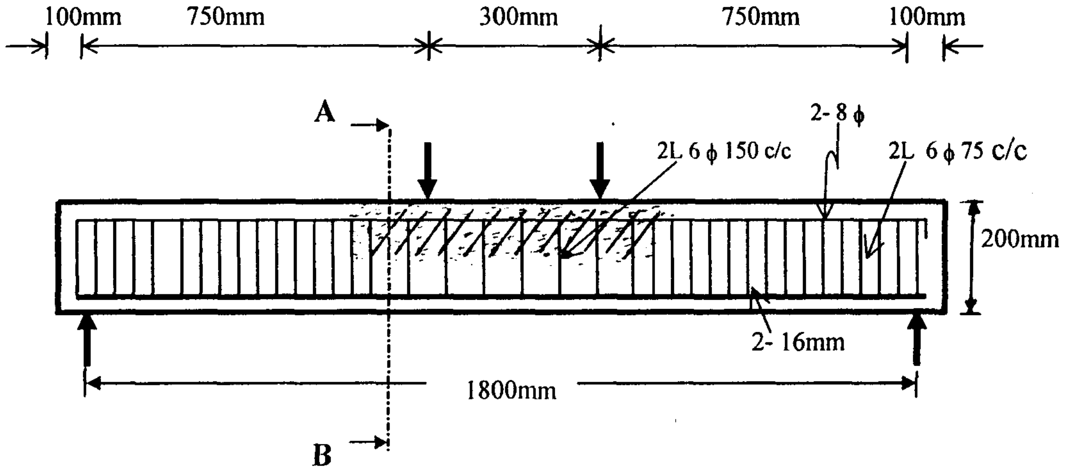
natural rubber latex to concrete improve the ductility, energy absorption capacity and other durability parameters. From the review of literature, it may be noted that no attempt has been made so far to study the combined effect of foresaid parameters such as polymer modification and the incorporation of confinement and steel fibres in the compression zone, on the strength and flexural behaviour of conventionally reinforced concrete beams. It is also shown by the previous researchers that the incorporation of one or more these parameters converts the brittle behaviour of concrete into ductile one. Hence an attempt has been made to study the flexural behaviour of latex modified reinforced concrete beam specimens with confined SFRC in the compression zone.

5.2 Experimental Programme

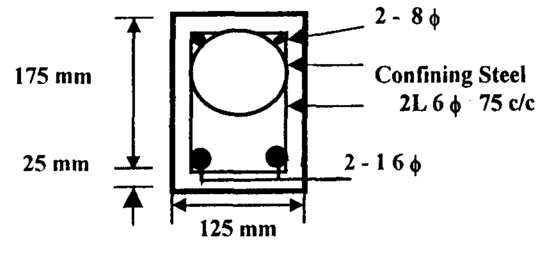
The experimental programme consisted of casting latex modified reinforced concrete beams with confined and fibrous compression zone and testing them under flexure. The polymer used was natural rubber latex. The spiral hoops were used to confine the concrete in the compression zone. Besides the confinement, short discrete steel fibres were randomly distributed in the compression zone. The variables considered in this study were: i) Volumetric ratio of confinement (ρ_s) ii) Volume fraction of steel fibres (V_f) and iii) Dry rubber content of the natural rubber latex (DRC). Aspect ratio (A_p) of the steel fibres was kept the same for all the specimens.

5.3 Details of Specimen

The overall dimensions of the specimens used were 125 x 200 x 2000mm. Fig 5.1 gives the sectional details of the specimens. In order to understand the behaviour of the specimens under flexural loading and ensure flexural failure, the beams were loaded with a minor span of 300 mm and a major span of 1800mm. Hence the shear span in this case is 750mm. By providing higher value of shear span/depth ratios the flexural failure of the specimen can be ensured. Similar procedure had been adopted by other researchers [45,52]. The shear reinforcement was designed in such a way that the shear capacity of the specimen was higher than the flexural strength, so that the specimens would not fail in shear.



Longitudinal Section



Section AB

* All dimensions in mm

Fig. 5.1 Overall dimensions of specimens and details of reinforcement

In the present investigation, 16 rectangular beams were cast and tested under two point loading. The volumetric ratio of confinement considered was 6.33%. Three different values of volume fraction of steel fibres 0.5, 1.0 & 1.5% and three different values of dry rubber content viz. 1.0, 2.0 & 3.0% were considered. Table 5.1 gives the details of the parameters used in each specimen.

5.3.1 Materials used

Cement : Cement used was ordinary Portland cement confirming to IS 269-1989.

Fine aggregate : Fine aggregate used was river sand passing through IS 4.75mm sieve, having a fineness modulus of 2.43.

Coarse aggregate : The coarse aggregate considered was locally available quarry crushed granite stones passing through IS 20mm sieve and retained on IS 4.75mm. The fineness modulus of the coarse aggregate used was 6.8.

Reinforcement : Main tension reinforcement consisted of 2 nos. of 16 mm dia. HYSD bars: compression reinforcement consisted of 2 nos. of 8mm dia HYSD bars. The shear reinforcement consisted of 2 legged vertical stirrups made of 6.58mm diameter plain bars at a spacing of 75mm c/c in the shear span and 150mm c/c in the flexural span.

The objective of this study was to understand the effect of confined latex modified SFRC in the compression zone on the flexural behaviour of beams. The confinement was provided in the form of circular spirals over a depth of 90mm from the top extreme compression fibre. The external diameter of spirals coil used was 75mm and were made of 6.58mm diameter plain bar. It was provided for a length of 650mm (flexural span (300mm) + one effective depth on either side) in the middle portion of the specimen. The pitch of the spirals was kept constant and was 40mm.

Table 5.1

Details of Latex, Fibre and Confinement used in the beams

Sl. No.	Beam Designation	Latex DRC (%)	Volume fraction of steel fibres V_f (%)	Confinement		
				Dia of bar (mm)	Pitch of spiral (mm)	Volumetric ratio ρ_s (%)
1	$C_0L_0F_0$	0.0	0.0	--	--	--
2	$C_0L_1F_0$	1.0	0.0	--	--	--
3	$C_0L_2F_0$	2.0	0.0	--	--	--
4	$C_0L_3F_0$	3.0	0.0	--	--	--
5	$C_1L_0F_1$	0.0	0.5	6.58	40	6.33
6	$C_1L_0F_2$	0.0	1.0	6.58	40	6.33
7	$C_1L_0F_3$	0.0	1.5	6.58	40	6.33
8	$C_1L_1F_1$	1.0	0.5	6.58	40	6.33
9	$C_1L_1F_2$	1.0	1.0	6.58	40	6.33
10	$C_1L_1F_3$	1.0	1.5	6.58	40	6.33
11	$C_1L_2F_1$	2.0	0.5	6.58	40	6.33
12	$C_1L_2F_2$	2.0	1.0	6.58	40	6.33
13	$C_1L_2F_3$	2.0	1.5	6.58	40	6.33
14	$C_1L_3F_1$	3.0	0.5	6.58	40	6.33
15	$C_1L_3F_2$	3.0	1.0	6.58	40	6.33
16	$C_1L_3F_3$	3.0	1.5	6.58	40	6.33

Note : 1. Volumetric ratio of confinement ρ_s (%) = $\dfrac{\text{Volume of binder}}{\text{Volume of core concrete}}$

2. Specimen $C_0L_0F_0$ means the specimens without confinement, latex and fibres

3. $C_0 \rightarrow$ without confinement, $C_1 \rightarrow$ confinement with ρ_s =6.33%

4. $L_0 \rightarrow$ without latex (DRC), $L_1 \rightarrow$ DRC=0.5 %, $L_2 \rightarrow$ DRC=1.0 %, $L_3 \rightarrow$ DRC=1.5 %

5. $F_0 \rightarrow$ without fibres (V_f=0.0%), $F_1 \rightarrow V_f$=0.5%, $F_2 \rightarrow V_f$=1.0%, $F_3 \rightarrow V_f$=1.5%

Fibres : The fibres used were galvanized steel fibres of diameter 0.88mm with an aspect ratio 25. The short discrete fibres were randomly distributed in the compression zone of the flexural span of the specimen. Three different volume fractions of steel fibres viz. 0.5, 1.0 and 1.5% were used.

Natural Rubber Latex : The main aim of this investigation was to study the flexural behaviour of latex modified reinforced concrete beams with confined SFRC in the compression zone. The properties of natural Rubber Latex used in this investigation are given in Chapter 3. The Dry Rubber Content (DRC) considered in this study were 1%, 2% and 3%.

Water: For the preparation of the specimens, potable water available in the laboratory was used.

The ratio of the concrete mix used was 1:2:4 by weight and the water cement ratio was 0.45 by weight. As mentioned in Chapter 3, when latex is added to concrete, the mix becomes harsh and hence to improve the workability, adequate quantity of water reducing Superplasticizer (CONPLAST-211) was also used in the mix as per Table 3.1. In the case of latex modified specimens, the entire volume was modified with latex. However the addition of steel fibres and confinement in the form of spirals was restricted to compression zone of the flexural span only.

5.3.2 Preparation of the specimens

Preparation of moulds : Two wooden moulds of 125 x 200 x 2000mm inner dimension were made to cast the specimens. Smooth aluminum sheets were nailed to the inner surfaces of the moulds for the easy removal of the specimens. The concrete floor of the casting yard itself was used as the base of the mould. To prevent the buckling of the sides of the mould during casting and to keep the dimensions intact, U shaped metal clamps were used. Along with these specimens, control specimens viz. 150mm cubes, 150 x 300mm cylinders and 150 x 150 x 700mm size prisms were also cast.

Casting of specimens : All the materials were weighed using an AVERY 300 kg. platform balance. For the easy removal of the specimens, oil was applied to the inner surfaces of the mould and on the floor on which the specimens were cast. The mixing of aggregates and cement was done in a mechanical mixer of capacity 0.5 m^3. The dry materials were mixed thoroughly and then the required quantity of water was added to the mix (w/c ratio 0.45 by weight) and mixing is continued till uniform mix was obtained. The amount of Superplasticizer used for different percentage DRC is given in Table 3.1 Chapter 3. Required quantities of cement, fine aggregate, coarse aggregate, water, natural rubber latex and Superplasticizer were weighed and kept ready for mixing. The mixing and placing of concrete was done as explained in the earlier Chapter (Chapter3, Section 3.4.3)

Curing of specimens: The specimens were removed from the mould after 24 hours of casting and air dried for another 24 hours and then moist cured for 27 days using wet gunny bags. The air drying of the specimens for 24 hours was done to allow for the coalescence and film formation, which is very essential in the latex modification process. Some of the specimens (those with higher latex content) took more time, even up to 48 hours, for setting. This may be due to the fact that the addition of higher percentages of latex affects the hydration of cement. After the curing period was over, the specimens were white washed and kept ready for testing. The control specimens were also cured under the same conditions of environment.

5.4 Test Setup and Testing Procedure

Test setup: All the specimens were tested in a universal Compression Testing Machine of capacity 300 tonnes (2942 kN). In order to note down the applied load precisely,. a load cell of 25 tons (245 kN) capacity, with a least count of 0.01 tonne (0.098 kN) was used as shown in Fig. 5.2. A special steel frame arrangement called rotation meter was fabricated out of angle sections to measure the longitudinal strains. Three Linear Variable Differential

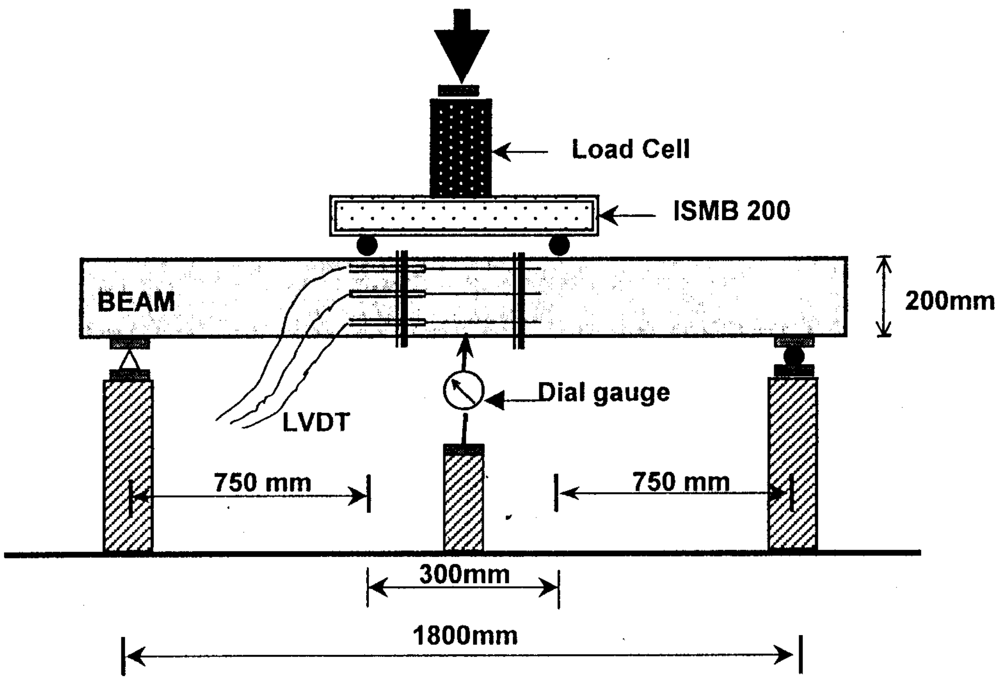


Fig. 5.2 TEST SET UP

Transducers (LVDT) were used for measuring the longitudinal strains (at the top, bottom and middle) and to find out the moment - curvature relationship. The longitudinal deformations at the top were measured using an LVDT with a range of ±5mm and a resolution of 0.001mm. This LVDT was placed at a distance of 25mm from the extreme compression fibre. The deformations at the middle was measured using an LVDT of ± 1mm range and a resolution of 0.001 mm and was positioned 75mm from the extreme compression fibre. The bottom measurements were made using an LVDT of range ±10mm with a resolution of 0.01mm and were placed 25mm above the extreme tension fibre. The vertical deflections at the middle were measured using a dial gauge having a travel of 50mm and a least count of 0.01mm.

The beams were supported on two rollers (30mm dia.) of which one was fixed and the other was capable of rotation. The effective span was kept as 1800mm. The specimens were tested under two point loading. Two rollers, each of diameters 30mm served as load points and were kept on the beams at a distance of 300mm, kept in position by plaster of Paris. A rolled steel joist was used to transmit the load from the machine to the two locations through the rollers.

5.4.1 Testing procedure: The load was applied in stages. For every stage of loading, the following readings have been noted:

i) Deflection at the mid span of the specimen.

ii) LVDT readings at 25mm, 75mm and 175mm from the extreme compression fibre of the specimens.

iii) Width of cracks at the soffit and the propagation of the cracks after the first cracking load. For measuring the crack width an imported microscope with a magnification of 50X and a resolution of 0.02mm was used.

The above readings/measurements have been made upto the ultimate load of the specimens. Beyond the ultimate load, only the deflections could be recorded that too only upto 80percent of the peak load. As the testing machine was stress controlled type, it was not possible to record all the deflections in the post peak load range as sudden failure was observed beyond 80% of the peak load. Fig.5.3 (a, b, c, d & e) shows the crack pattern of the tested specimens.

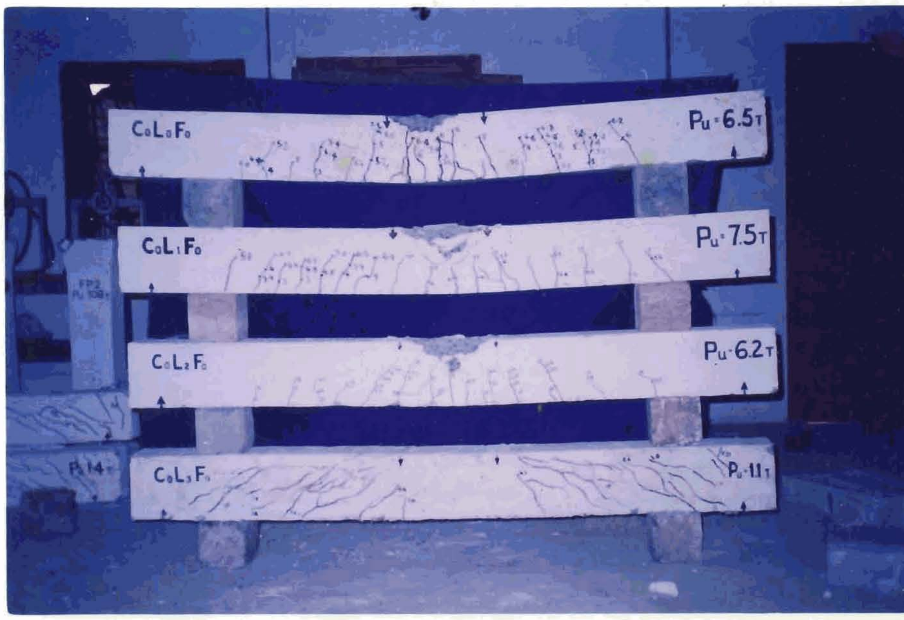
All the control specimens viz. cubes, cylinders and prisms were tested according to IS 516-1959. The compressive strength, stress strain behaviour of concrete and flexural strength have been obtained from the testing of control specimens.

5.5 General Behaviour of Specimens in Flexure

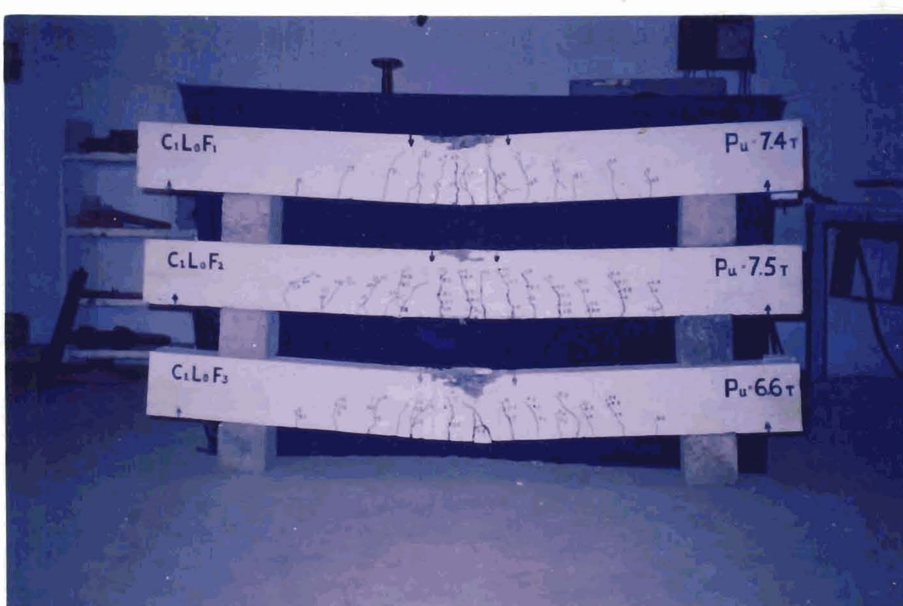
In all the specimens, cracks appeared at the soffit of beam within the flexural span. Then more number of cracks appeared in the flexural and shear spans and the cracks moved upwards. As the load increased, the cracks began to widen. At higher loads some of the cracks propagated up the beam and the tension steel started yielding at these cracks.

The unconfined plain beam $C_0L_0F_0$ failed all of a sudden just after the ultimate load was reached. This indicates a brittle failure. But the latex modified; confined SFRC and latex modified confined SFRC beams resisted considerably even after the ultimate load was reached. They deformed considerably before the load came down. This indicates ductile behaviour of the specimens when the conventional reinforced concrete is modified by latex and confined with steel fibre reinforced concrete in the compression zone.

Almost all the beams failed in flexure except those with a DRC of 3%, which failed in shear- flexure. This was due to the fact which is already explained in Chapter 3 under the section 3.3.4 that, at a DRC of 3%, the strength of concrete reduces drastically and hence specimens failed before the steel started yielding.

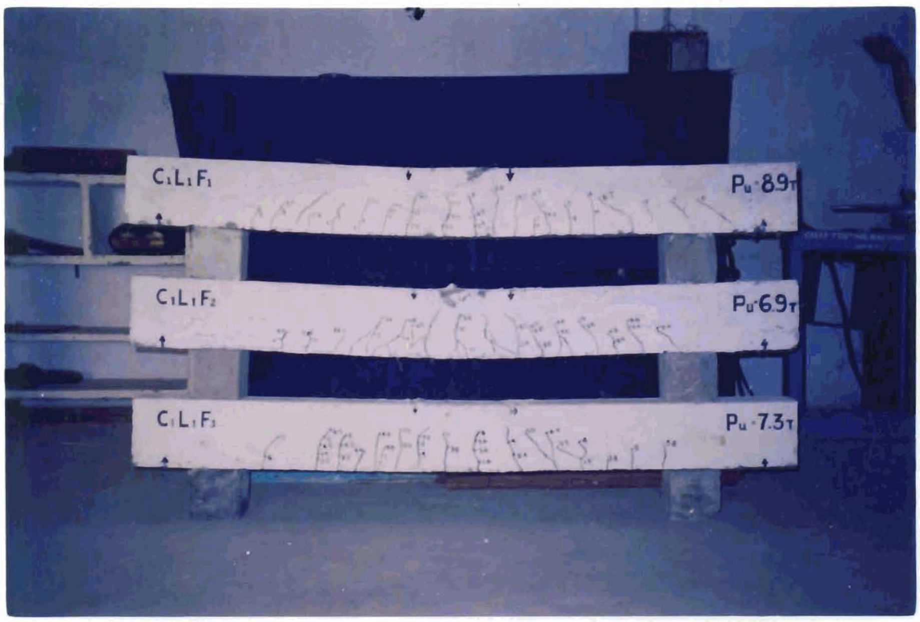


(a)

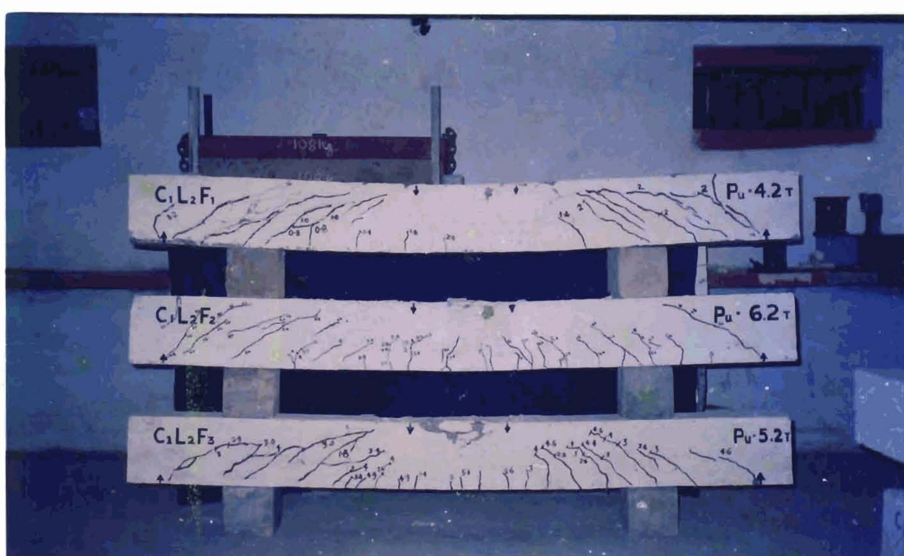


(b)

Fig. 5.3 Photograph of tested beams

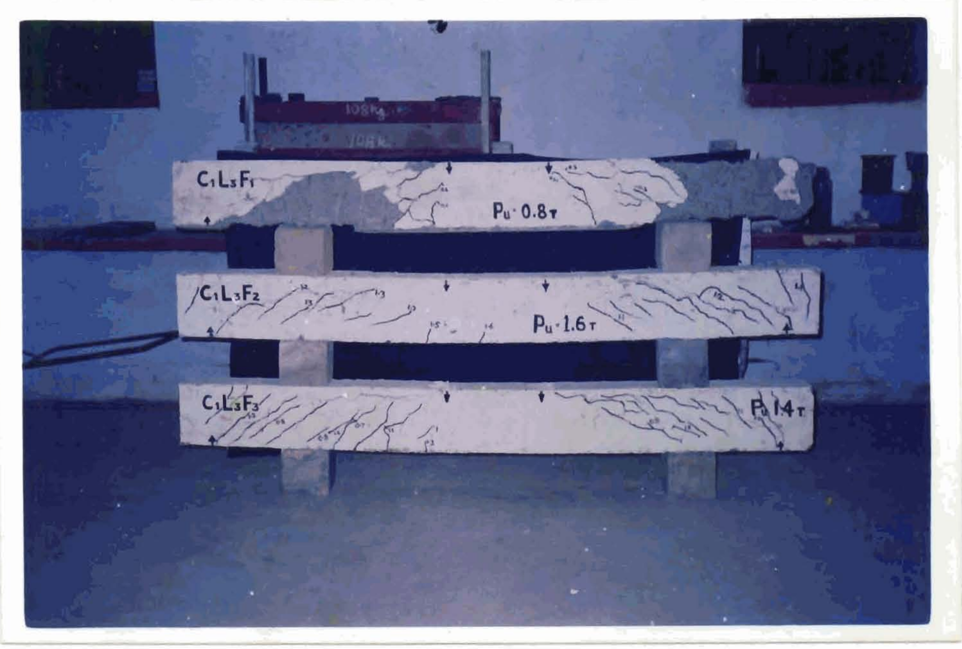


(c)



(d)

Fig. 5.3 Photograph of tested beams



(e)

Fig. 5.3 Photograph of tested beams

5.6 Discussion of Test Results

In this section, the experimental results obtained from the tests conducted on both the beams and control specimens are presented and discussed. The recorded values of load, deflection, strains etc. have been used to obtain plots relating them with the various physical parameters of the specimens.

Table 5.2 shows the first crack load and the ultimate load of the flexural specimens along with the compressive strength and flexural strength of the control specimens. Referring to the Table, the following points can be noted:

The first crack load (the load at which the first crack appears) of the specimens increase as latex is added to concrete. This increase is found to be significant for a DRC of 1%. At higher values of DRC viz. 2% and 3%, in fact, a reduction in the first crack load is observed. This may be due to the following reasons. When latex is added to the fresh concrete, the polymer particles get dispersed in the cement paste. As the hydration of cement proceeds and the water in the pores drain out, the polymer particles fill the micro pores. Ultimately by the removal of water by cement hydration, the closely packed polymer and cement particles coalesce into continuous films. Thus a monolithic network is formed in which the polymer and cement hydrate phases inter-penetrate throughout. This will increase the density, which in turn increased the strength of concrete. However, as the latex content increased, the excess latex will lead to the formation of weak spots in the specimen due to air entrainment and resulting in the reduction in density. At higher values of latex content this reduction is significantly high. This phenomenon has been already explained using the photographs taken on the latex modified concrete specimens using Scanning Electron Microscope (SEM) in Chapter 3 under the section 3.3.4.

Table 5.2

Test Results

Sl. No.	Beam designa-tion	First crack load (P_{cr}) (KN)	Ultimate load (KN)	Control Specimens			Strength gain factor
				Cube strength (N/mm^2)	Cyl. Strength (N/mm^2)	Mod. of rupture (N/mm^2)	
1	$C_0L_0F_0$	15.69	63.74	44.38	34.59	5.11	1.00
2	$C_0L_1F_0$	25.50	73.55	38.79	34.40	4.22	1.15
3	$C_0L_2F_0$	15.69	60.80	14.82	08.88	2.20	0.95
4	$C_0L_3F_0$	03.92	10.79	02.83	02.61	0.49	0.17
5	$C_1L_0F_1$	19.61	72.56	45.83	37.73	4.64	1.14
6	$C_1L_0F_2$	17.65	73.55	46.92	34.54	4.18	1.15
7	$C_1L_0F_3$	23.53	64.72	47.29	37.09	4.90	1.02
8	$C_1L_1F_1$	23.53	87.27	40.31	37.18	4.64	1.37
9	$C_1L_1F_2$	17.65	67.66	31.60	37.18	4.64	1.06
10	$C_1L_1F_3$	15.69	71.58	45.98	21.09	3.54	1.12
11	$C_1L_2F_1$	07.85	41.19	24.19	09.43	3.17	0.65
12	$C_1L_2F_2$	17.65	60.80	27.89	22.75	2.96	0.95
13	$C_1L_2F_3$	11.77	50.99	23.97	07.88	2.75	0.80
14	$C_1L_3F_1$	01.96	07.85	02.83	01.28	0.59	0.12
15	$C_1L_3F_2$	09.81	15.69	04.58	02.28	0.42	0.25
16	$C_1L_3F_3$	05.88	13.73	04.36	03.05	1.36	0.22

Note : $$\text{Strength gain factor} = \frac{\text{Ultimate load of the specimen}}{\text{Ultimate load of specimen } C_0L_0F_0}$$

ii) The specimens with DRC of 1% have given higher values of ultimate load than those with values of DRC of 2% and 3%. This reinforces the earlier findings that the addition of latex above a particular amount of DRC decreases the strength of concrete, as explained earlier.

iii) When fibres are added to latex, in most of the cases ultimate load of the specimens have been found to increase. This could be attributed to the following effects of the fibre bridging. As and when micro cracks develop in the matrix, the fibres in the vicinity of such micro cracks will try to arrest these cracks and prevent further propagation. Hence the cracks which appear inside the matrix have to take a meandering path, resulting in the demand for more energy for further propagation, which in turn increases the ultimate load. However, at higher values of fibre content, in fact, a reduction in ultimate load has been noticed. This is mainly due to the balling effect of fibres at higher percentages and difficulty experienced in fully compacting the specimens. From the above, it is seen that the upper limit of the fibre content which gives higher load carrying capacity is 1.0%.

iv) Comparing the specimens with and without confinement, it may be observed that the inclusion of confinement in the compression zone increases the load carrying capacity significantly.

v) From the experiments, it appears that the optimum value of DRC is 1% and the maximum strength gain is achieved at this percentage. At higher values of DRC viz.. 2% and 3% the first crack load and ultimate load decreases due to the reasons mentioned earlier.

As previously mentioned, the recorded values of load, deflection, strains etc. have been used to obtain plots relating them with the physical properties of the specimens. Fig.5.4. to 5.8 show the load deflection plots for :

1. Beam without confinement and fibres but with different percentages of DRC (Fig. 5.4)
2. Beams with confinement and different volume fractions of fibres but without latex (Fig.5.5)
3. Beams with 1% DRC, confinement and different volume fraction of steel fibres (Fig. 5.6)
4. Beams with 2% DRC, confinement and different volume fraction of steel fibres (Fig. 5.7)
5. Beams with 3% DRC, confinement and different volume fraction of steel fibres (Fig. 5.8)

Referring to these plots, the following points can be noted:

i) In the case of specimens without confinement and fibres but only with latex (Fig.5.4), at 1% DRC the ultimate load and the deflection at peak load is higher than those of specimens with 2% and 3% DRCs. At higher values of DRC viz. at 2% and 3%, the ultimate load and deflection at peak load decrease. This decrease is drastic in the case of specimen with 3% DRC. The reason for the above behaviour has been already explained.

ii) In the case of specimens with confinement and fibres and no latex, (Fig.5.5) as the volume fraction of steel fibres increases from 0 to 1%, the ultimate load and deflection at peak load also increases. Beyond $V_f = 1\%$ i.e. at $V_f = 1.5\%$, in fact a reduction in ultimate load is noted. This may be due to the balling effect of fibres and the difficulty in compacting during casting the specimens with higher values of V_f.

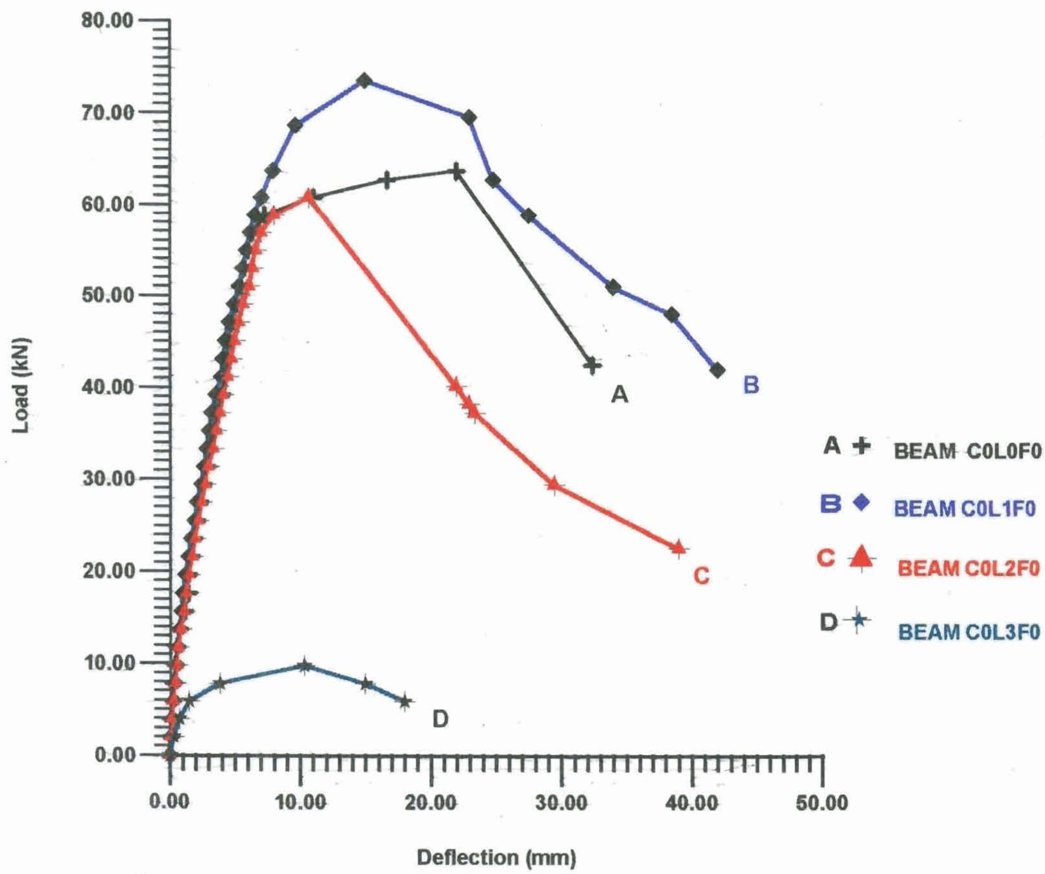


Fig. 5.4 Load Deflection plots for Latex Modified Beams
(without steel fibres)

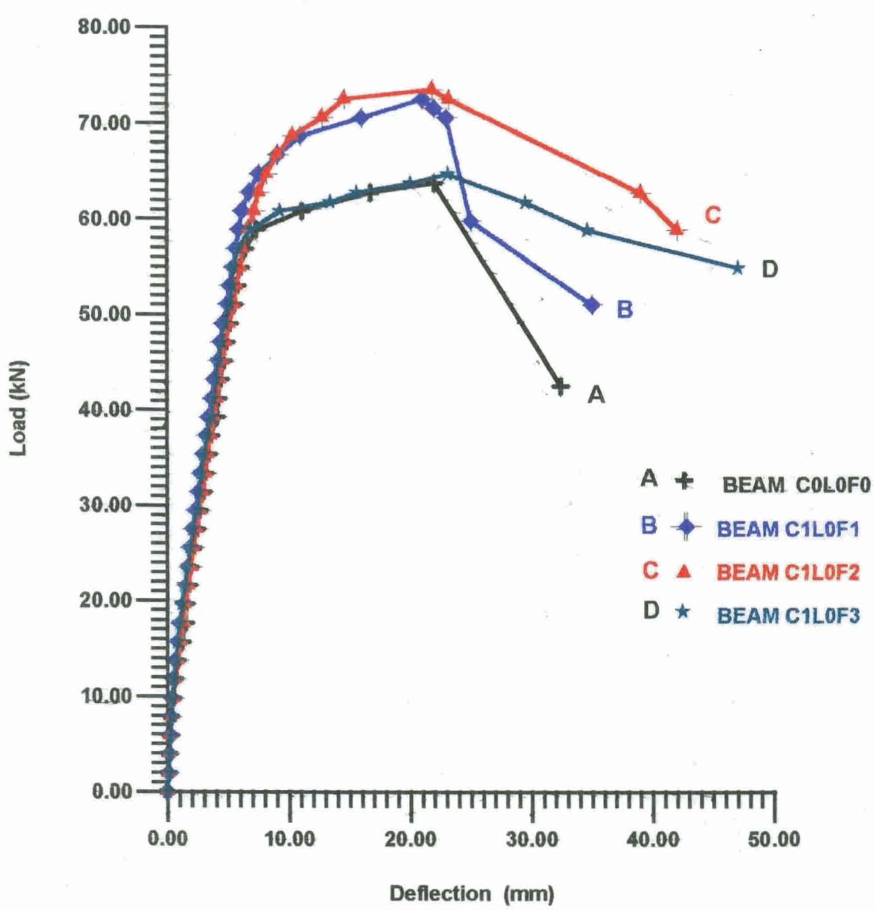


Fig. 5.5 Load Deflection plots for beams with confinement and Steel Fibres

iii) Referring to Fig.5.6, the ultimate load and the deflection at peak load of the confined specimens with 1% DRC and 0.5% V_f are significantly higher than those of the plain beam. The confined specimens with Vf=1% and 1.5% have given lower values of ultimate load than the specimens with Vf=0.5%. This may be due to the following reasons. As mentioned earlier, when the quantity of latex added is small (of the order of 1% DRC), the concrete mix becomes denser and hence higher load carrying capacity is obtained. Addition of small quantities of steel fibres (V_f =0.5%) to these specimens enhances the overall behaviour of the material by arresting the micro-cracks which form in the matrix during loading. When higher percentages of volume fraction of fibres are added (V_f =1% and 1.5%), it might have caused difficulty in compacting the specimens properly due to balling effect and thus resulting in reduced strength.

iv) Referring to Fig. 5.7, the confined specimens with 2% DRC and different volume fraction of fibres have given lower values of ultimate load when compared to the plain beam. Out of the three specimens, the one with a volume fraction of 1% gives a better performance than those with V_f=0.5% and 1.5%.

v) Referring to Fig. 5.8, when the value of DRC is 3% the confined specimen with fibres have given very low value of ultimate load when compared to that of the plain beam. The provision of confinement and the addition of fibres have no effect when the DRC is 3%.

From the above behaviour of the specimens, it may be noted that there is a general improvement in the performance of latex modified specimens and latex modified specimens with confined SFRC in the compression zone. Also, it can be seen that the plain beam failed after the peak load without giving substantial deflections. However, the latex modified specimens (upto a DRC of 2%) with confined SFRC in the compression zone failed in a ductile manner and large deflections were noticed at the ultimate stage. This indicates that the latex modification and the provision of confined SFRC in the compression zone improves the ductility markedly. The computation of ductility factor were given in the subsequent sections.

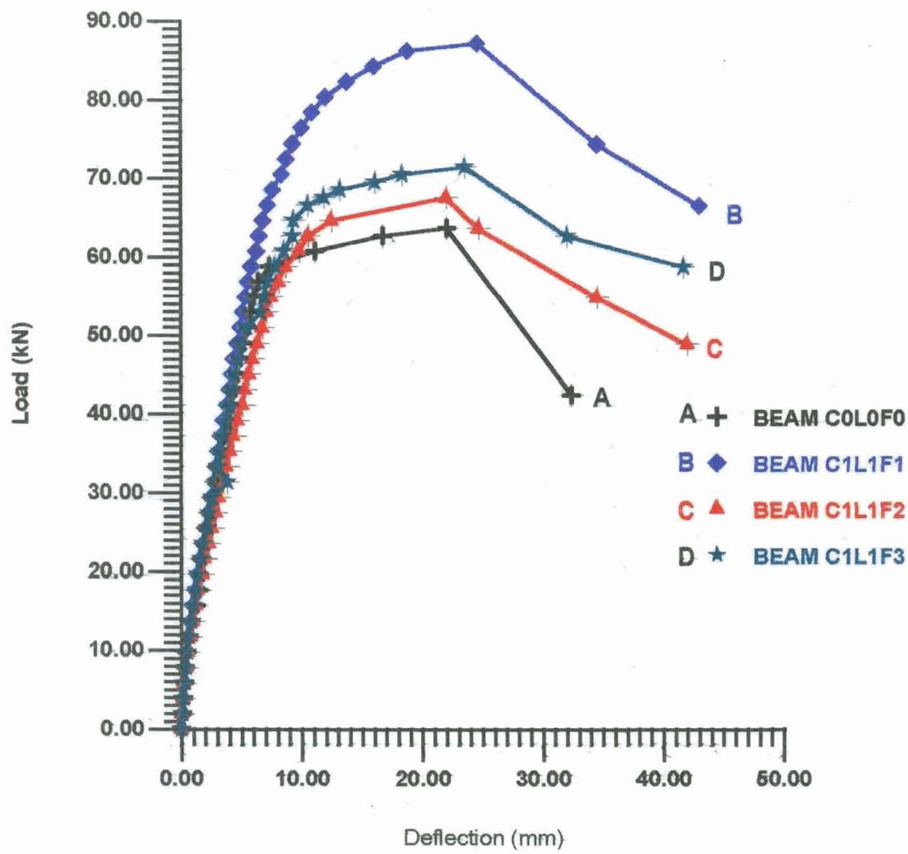


Fig. 5.6 Load Deflection plot for Latex Modified SFRC Beams

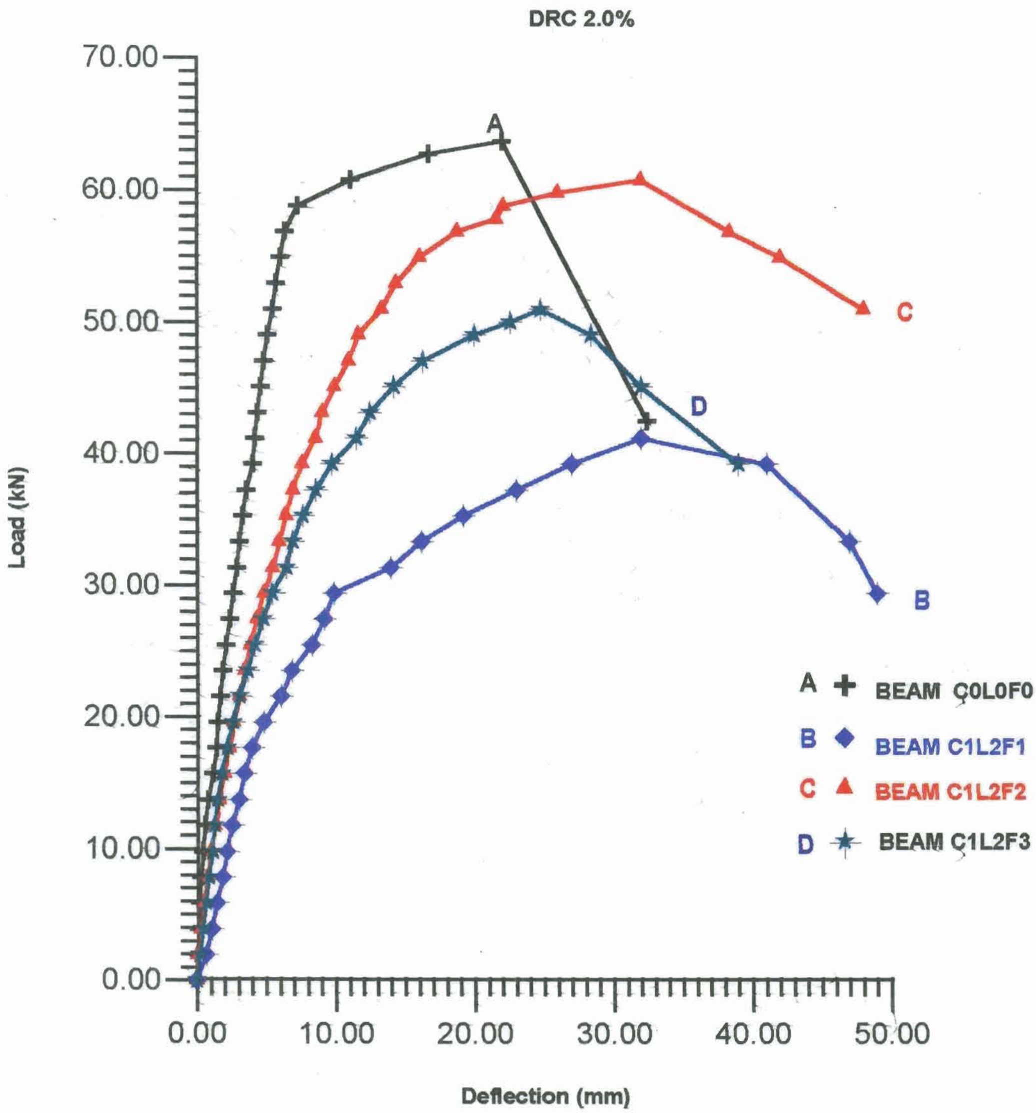


Fig. 5.7 Load Deflection plots for Latex modified SFRC Beams

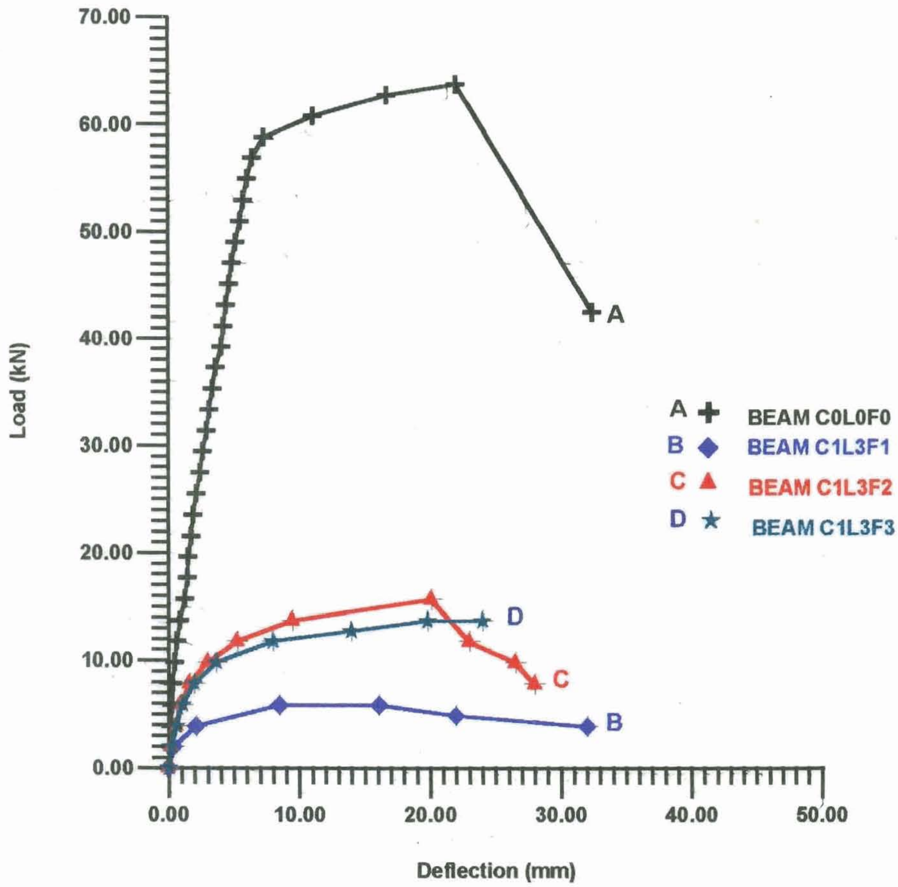


Fig. 5.8 Load Deflection Plots for Latex Modified SFRC Beams

5.6.1 Variation of ultimate load with volume fraction of steel fibre

Several parameters like percentage of DRC, confinement and volume fraction of steel fibres affect the strength of concrete. Hence in order to account for these variables, the ultimate load was normalised by dividing P_u by bdf_{ck}. The normalised values of ultimate load (P_u / bdf_{ck}) were plotted against values of DRC. Fig. 5.9 shows the variation of normalised ultimate load with volume fraction of steel fibres for specimens with different values of DRC.

From the Figure it may be noted that the maximum value of ultimate load is given by the specimens with V_f = 0.5% and DRC = 1.0% or in general, for a range of V_f = 0.5% to 1.0% and DRC from 0.0% to 1.0% as indicated by the hatched portion in the figure .

5.6.2 Variation of deflection at ultimate load

The deflection at ultimate load (δ_u) is an important parameter and is used to obtain the ductility factor in the case of flexural specimens Fig. 5.10 relates the deflection at ultimate load with volume fraction of steel fibres for specimens with different values of DRC. It may be seen from the figure that the values of δ_u , in general, increases with increase in value of volume fraction of steel fibres. The exception to this is those specimens with DRC = 2.0% where, infact, a reduction in δ_u is noticed as V_f increases. The specimens with 3.0% DRC gave low value of δ_u. In the case of specimens with 1.0% DRC, δ_u decreases slightly as V_f increases from 0.5% to 1.0%, then onwards it increases as V_f reaches 1.5%. Even though the specimens with DRC = 2.0% give a higher value of δ_u, the ultimate load of these specimens were low compared to those with DRC =1.0%. Hence those specimens with DRC value of 2.0% do not satisfy the dual requirements strength and ductility in the case of structures subjected to cyclic or repeated loadings. On the other hand those with a value of DRC = 1.0% improves both strength and ductility (Fig. 5.9 and 5.10)

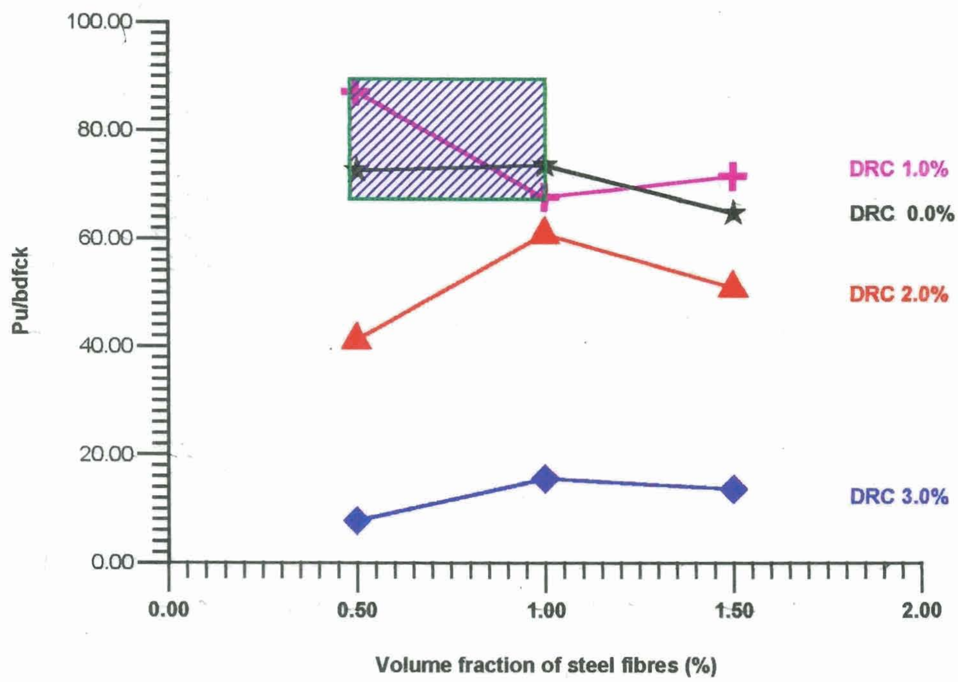


Fig. 5.9 Plot of Ultimate load versus Volume fraction of Steel fibres

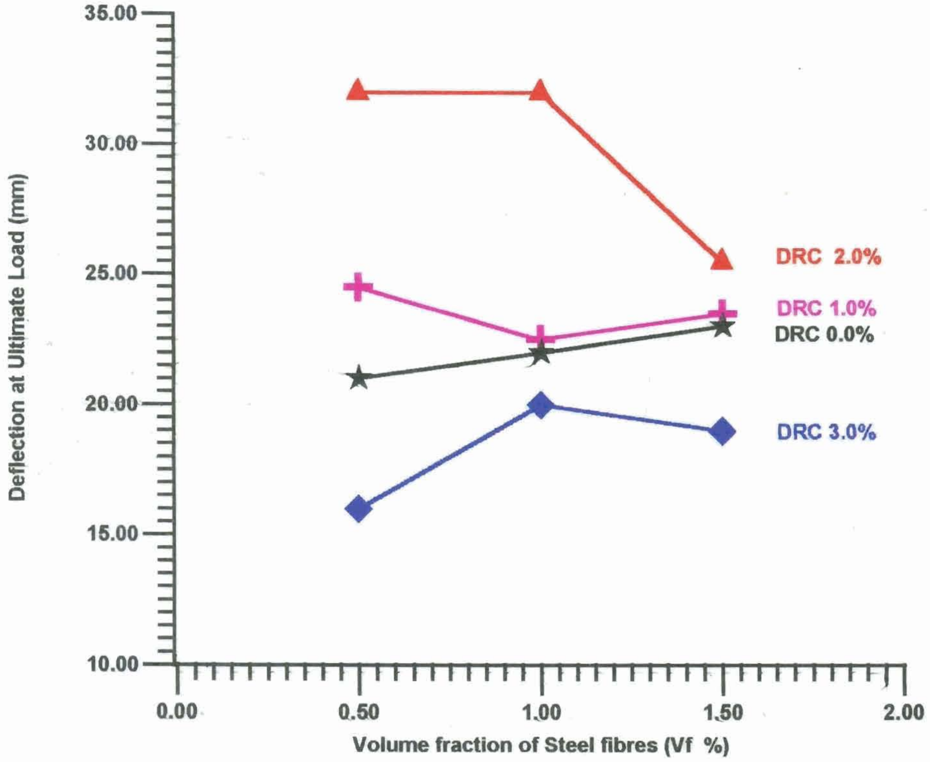


Fig. 5.10 Deflection at Ultimate Load versus Volume fraction of steel fibres

5.7 Energy Absorption Capacity and Toughness Index

One of the aims of this thesis is to improve the structural behaviour of conventionally reinforced concrete flexural members either by the addition of steel fibres or by the modification of concrete by polymer like latex or by the combined effect of fibres and latex along with confining of the compression zone of such latex modified steel fibre concrete. From the literature, it was noted that the individual effect of the above actions generally improved the structural behaviour such as strength, ductility, energy absorption capacity, toughness etc.. The combined effect of steel fibres, latex and confinement on the structural behaviour of conventionally reinforced concrete flexural members have been discussed in the subsequent sections. Hence attempts have been made to obtain the important material properties like energy absorption capacity, toughness index, ductility factor for the specimens tested in this study. The following sections explain in detail the method used for determining these parameters and variation of the same with fibres and latex.

5.7.1 Energy absorption capacity

In order to determine the energy absorption capacity or toughness of the specimens, the following procedure was adopted. The area under the full load deflection curve represents the energy absorption capacity of the specimen. But due to the inherent limitations of the testing machine (Stress controlled), the load and deflection readings could not be taken for all the stages of post-peak loading. However, the post-peak readings were recorded till the load reduced to 80% of the peak load (P_u). Hence the energy absorption capacity of the specimens were calculated from the P- δ curve considering the area under the ascending portion and upto 0.80 P_u in the descending portion.

Table 5.3 gives the values of energy absorption thus computed for all the specimens tested in this study. From Table 5.3 it can be seen that in general, the energy absorption capacity increases with slight fluctuation for confined SFRC specimens and latex modified (up to a DRC 2%) specimens with confined steel fibre reinforced concrete in the compression zone. In the case of specimens with 3% DRC, there is large reduction in the energy absorption capacity and the reasons for the drastic reduction in the energy absorption capacity is the same as the one explained in section 3.3.4. The maximum value of energy absorption capacity obtained was 2.651 KN-m and is given by the specimen with $V_f = 0.5\%$ and DRC = 1.0%.

Table 5.3
Energy absorption capacity

Sl. No	Designation	Energy absorption capacity	
		Absolute (KN-m)	Ratio
1	$C_0L_0F_0$	1.742	1.00
2	$C_0L_1F_0$	1.608	0.92
3	$C_0L_2F_0$	0.872	0.50
4	$C_0L_3F_0$	0.108	0.06
5	$C_1L_0F_1$	1.916	1.10
6	$C_1L_0F_2$	2.155	1.24
7	$C_1L_0F_3$	2.271	1.30
8	$C_1L_1F_1$	2.651	1.52
9	$C_1L_1F_2$	1.939	1.11
10	$C_1L_1F_3$	2.355	1.35
11	$C_1L_2F_1$	1.487	0.85
12	$C_1L_2F_2$	2.104	1.21
13	$C_1L_2F_3$	1.390	0.80
14	$C_1L_3F_1$	0.132	0.08
15	$C_1L_3F_2$	0.298	0.17
16	$C_1L_3F_3$	0.258	0.15

Fig. 5.11 and 5.12 shows the variation of energy absorption capacity of confined specimens with the volume fraction of steel fibres and values of DRC respectively. For comparison purposes only confined specimens have been considered for these plots. Considering both figures 5.11 and 5.12, it may be seen that the combination of V_f = 0.5% to 1.0% and DRC = 0.0% to 1.0% improve the energy absorption capacity of latex modified reinforced concrete flexural members with confined SFRC in the compression zone.

5.7.2 Toughness index

As several parameters like strength of concrete, percentage of DRC, percentage of steel fibres influence the energy absorption capacity, an attempt was made to normalise the energy absorption capacity as follows. The major parameter, which affects the first crack strength of reinforced concrete, is the strength of concrete and which depends on the ingredients of concrete, mixing of concrete, workmanship/ quality of work etc. Beyond the first crack stage, multiple cracks develop as the loading increases and finally yielding of reinforcement occurs. The (p-δ) curve also softens after the first cracking stage. Therefore, the computed values of energy absorption capacity were normalised by dividing them by the energy absorption capacity computed up to the first crack load. This is similar to the procedure adopted in JCI (Japanese Concrete Institute) [95] for obtaining the toughness index of steel fibre concrete. This ratio was termed as toughness index (T.I). Thus toughness index is given by:

$$\text{T.I.} = \frac{\text{area under the P-}\delta\text{ curve up to 80\% of the post peak load}}{\text{area under the P - }\delta\text{ curve up to first cracking}} \quad \text{....(5.1)}$$

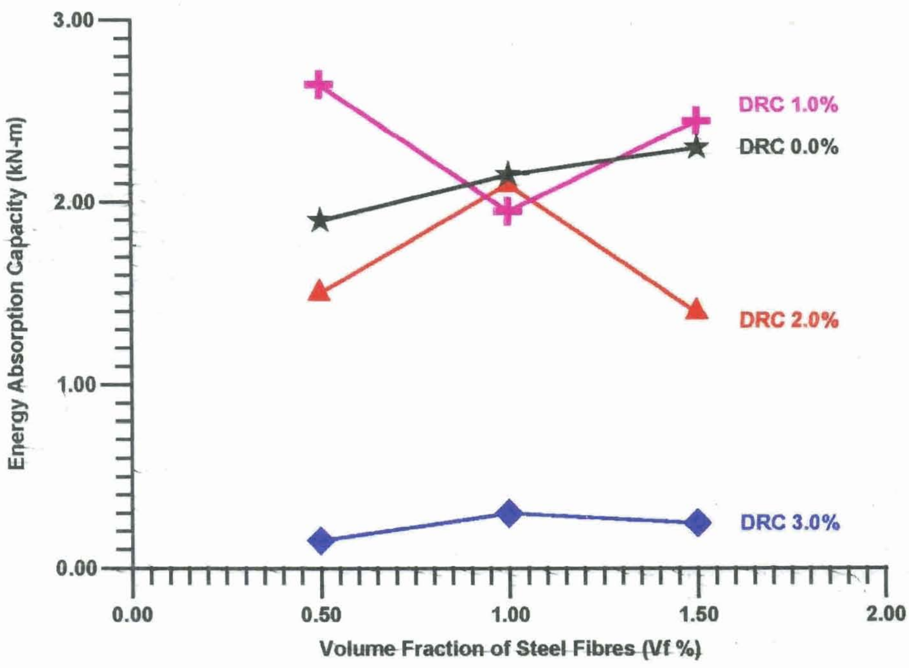


Fig. 5.11 Energy Absorption Capacity versus Volume Fraction of Steel Fibres

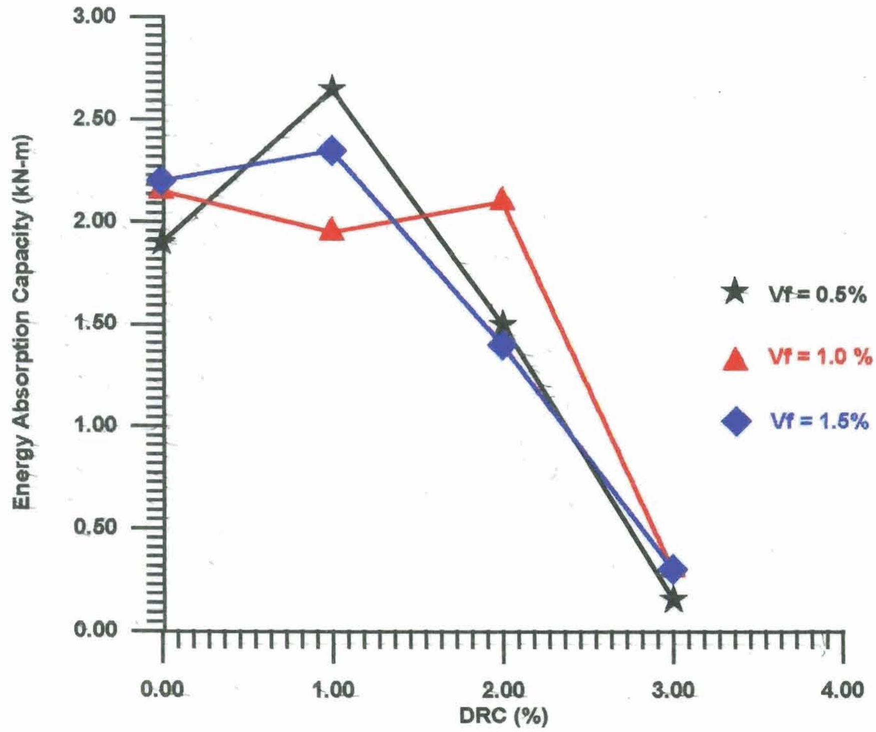


Fig. 5.12 Energy Absorption Capacity versus DRC (%)

Table 5.4 shows the values of toughness indices calculated for the specimens. It can be seen that, due to the incorporation of confinement and steel fibres in the compression zone, there is an increase in the toughness index. For latex modified (DRC 1%) specimens with confined SFRC, there is an increase in the toughness index and the relative value ranges from 1.2 to 2.04. But for specimens with higher latex addition (DRC 2% and 3%) the toughness indices reduces considerably due to the reasons discussed earlier.

Fig. 5.13 and 5.14 shows the variation of toughness index of confined specimens with volume fraction of steel fibres and DRC respectively. From these figures it may be noted that the combination of $V_f = 0.5\%$ to 1.0% and DRC = 0.0% to 1.0% enhances the value of T.I. markedly when compared to other combinations of V_f and DRC.

Table 5.4

Toughness Index

Sl No.	Designation	Toughness Index	
		Absolute	Relative
1	$C_0L_0F_0$	207.82	1.00
2	$C_0L_1F_0$	111.08	0.54
3	$C_0L_2F_0$	104.09	0.50
4	$C_0L_3F_0$	56.60	0.27
5	$C_1L_0F_1$	359.29	1.73
6	$C_1L_0F_2$	377.07	1.81
7	$C_1L_0F_3$	283.81	1.37
8	$C_1L_1F_1$	248.50	1.20
9	$C_1L_1F_2$	424.17	2.04
10	$C_1L_1F_3$	281.00	1.35
11	$C_1L_2F_1$	48.79	0.24
12	$C_1L_2F_2$	62.75	0.30
13	$C_1L_2F_3$	76.00	0.37
14	$C_1L_3F_1$	86.50	0.42
15	$C_1L_3F_2$	71.09	0.34
16	$C_1L_3F_3$	61.46	0.30

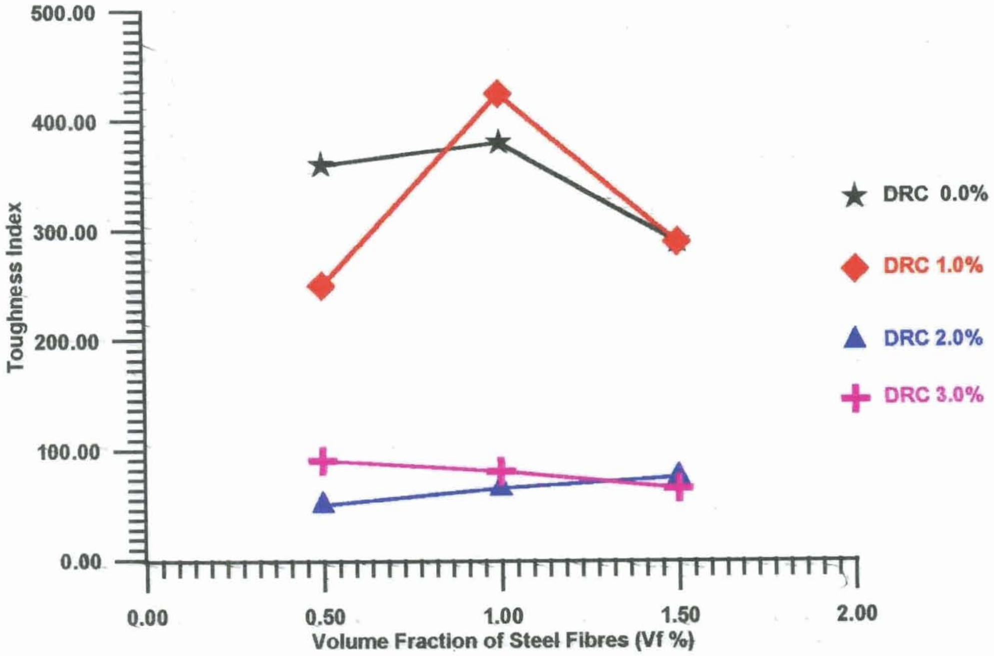


Fig. 5.13 Toughness Index versus Volume Fraction of Steel Fibres (Vf %)

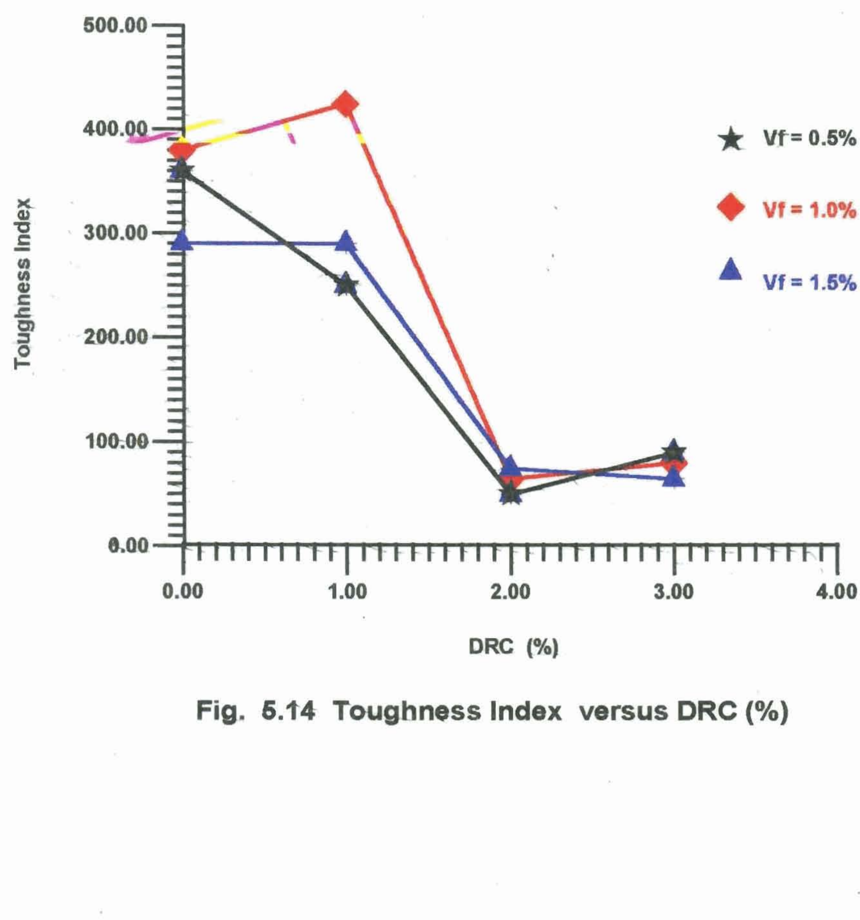


Fig. 5.14 Toughness Index versus DRC (%)

5.8 Ductility Factor

As explained in section 4.5.2, ductility is the ability of a structure to undergo inelastic deformations beyond the initial yield deformations with no decrease in the load resistance. The ductility of a member is usually expressed by the ductility factor, which is the ratio of the ultimate deformation to the deformation at the first yield.

$$\text{Ductility factor (D.F)} = \frac{\delta_u}{\delta_y} \qquad \ldots(5.2)$$

Where δ_u = ultimate deformation (deformation at ultimate load)

δ_y = deformation at first yield.

The procedure for computing the ductility factor has already been explained in Chapter 4 under the section 4.5.2

Table 5.5 shows the ductility factors thus calculated for all the specimens with values of DRC up to 2.0%. As the specimens with DRC =3.0% failed before the steel started yielding, they were not considered. It may be seen from the Table that, specimens with confinement, latex and fibres have improved the ductility factor markedly. The usable range of Vf varies form 1.0% to 1.5% and DRC varies from 0.0 to 1.0% .

Fig. 5.15 and 5.16 shows the variations of ductility factor of confined specimens with volume fraction of steel fibres and DRC respectively. From the figure, it may be noted that even though a definite trend is not obtained from these figures, it may be observed that the useful range of volume fraction of steel fibres is 0.5% to 1.0% and DRC is 0 to 2.0%.

Table 5.5 Ductility Factor

Sl.No.	Designation	Ductility Factor	
		Absolute	Relative
1	$C_0L_0F_0$	2.961	1.00
2	$C_0L_1F_0$	2.167	0.73
3	$C_0L_2F_0$	1.542	0.52
4	$C_1L_0F_1$	2.875	0.97
5	$C_1L_0F_2$	3.125	1.06
6	$C_1L_0F_3$	3.857	1.30
7	$C_1L_1F_1$	3.440	1.16
8	$C_1L_1F_2$	3.136	1.06
9	$C_1L_1F_3$	3.762	1.27
10	$C_1L_2F_1$	3.500	1.18
11	$C_1L_2F_2$	4.148	1.40
12	$C_1L_2F_3$	3.185	1.08

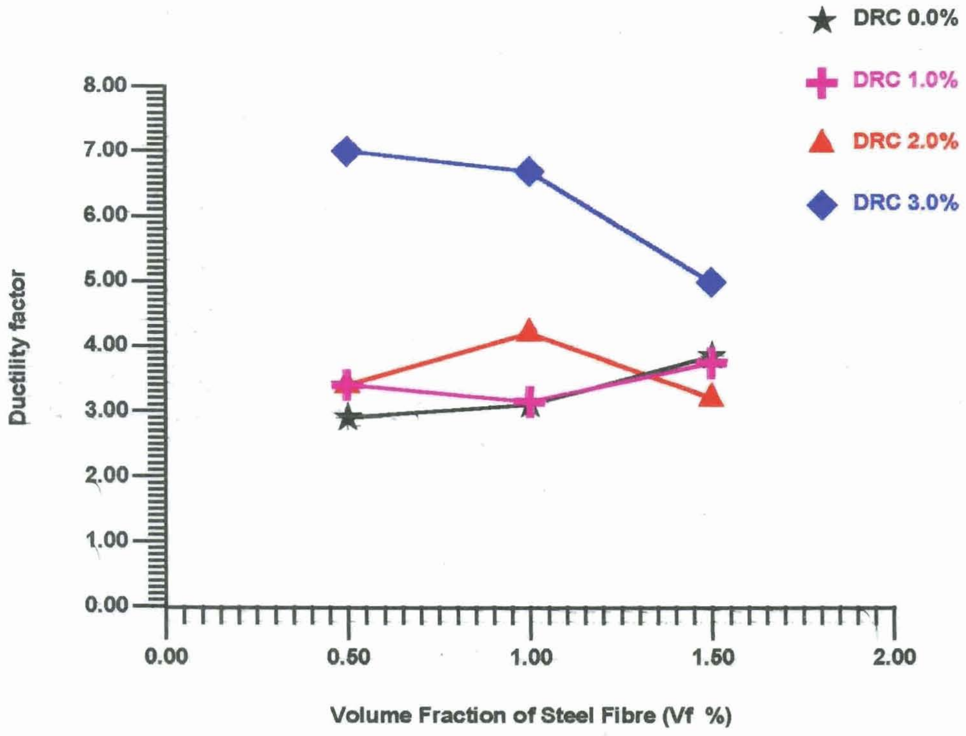


Fig. 5.15 Ductility Factor versus Volume Fraction of Steel Fibres (Vf %)

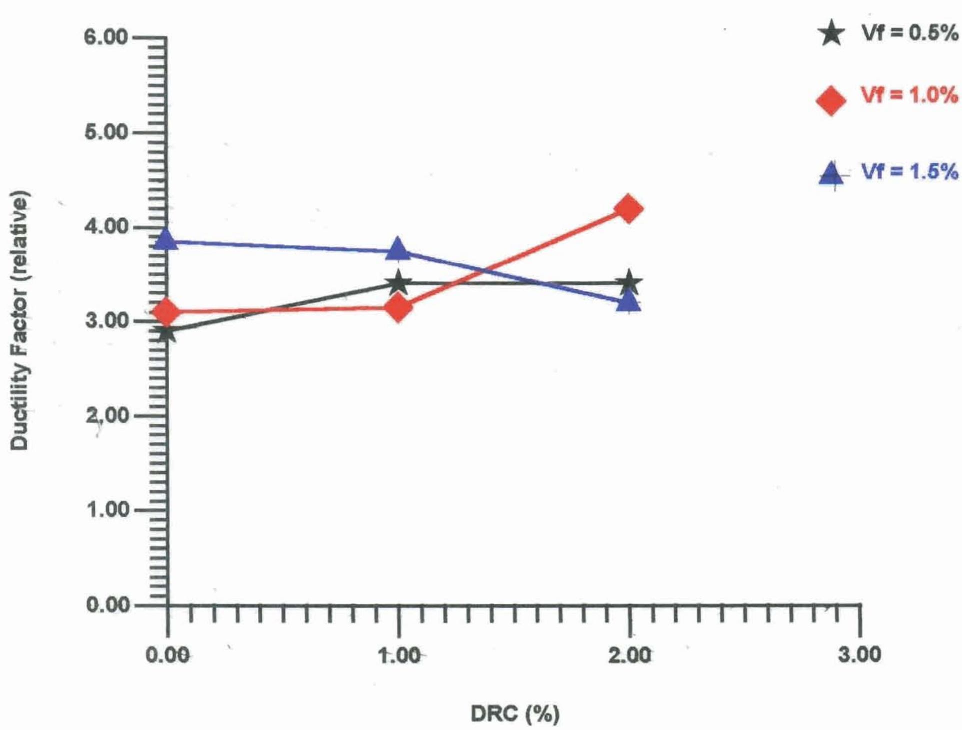


Fig. 5.16 Ductility Factor versus DRC (%)

5.9 Introduction of Constitutive Parameter

All the earlier attempts were concentrated in making plots to understand the effect of the variation in (i) confinement, (ii) volume fraction of fibres and (iii) DRC on the test results such as ultimate load (P_u), deflection at ultimate load (δ_u), energy absorption capacity, toughness index (T.I) and ductility factor. However the combined effect of all the three parameters cannot be obtained from such plots. Hence an attempt was made to define a single parameter which takes in to account the variation of all the three physical parameters considered in this study. Thus a parameter called the constitutive parameter (C_p) was developed which is given by

$$C_p = \frac{1}{(1-\rho_s)(1-V_f)(1-DRC)} \qquad \ldots(5.3)$$

This C_p is obtained after trying several combinations of ρ_s, V_f and DRC on the mechanical properties of confined latex modified reinforced concrete flexural members. This parameter was compared with the test results. Fig. 5.17 shows the plot of the Ultimate Load versus the constitutive parameter. It can be seen that, as the value of this parameter C_p increases the ultimate load also increases in a non-linear form. In this case a second-degree polynomial was found to be the best fit. The regression equation relating the ultimate load and C_p was obtained and is,

$$P_u = 26225\, C_p - 12500\, C_p^{\,2} - 13679 \qquad \ldots(5.4)$$

The constitutive parameter Cp was related to energy absorption capacity, toughness index and ductility factor. Fig. 5.18, 5.19 and 5.20 shows the plots relating them. The regression equation obtained for these plots are as follows:

a. Energy absorption capacity (U) $= 890.63\, C_p - 421.88\, C_p^2 - 467.95$

b. Toughness Index (TI) $= 148812.5\, C_p - 70312.5\, C_p^2 - 78445$

c. Ductility Factor (DF) $= 18.52\, C_p - 17.09$

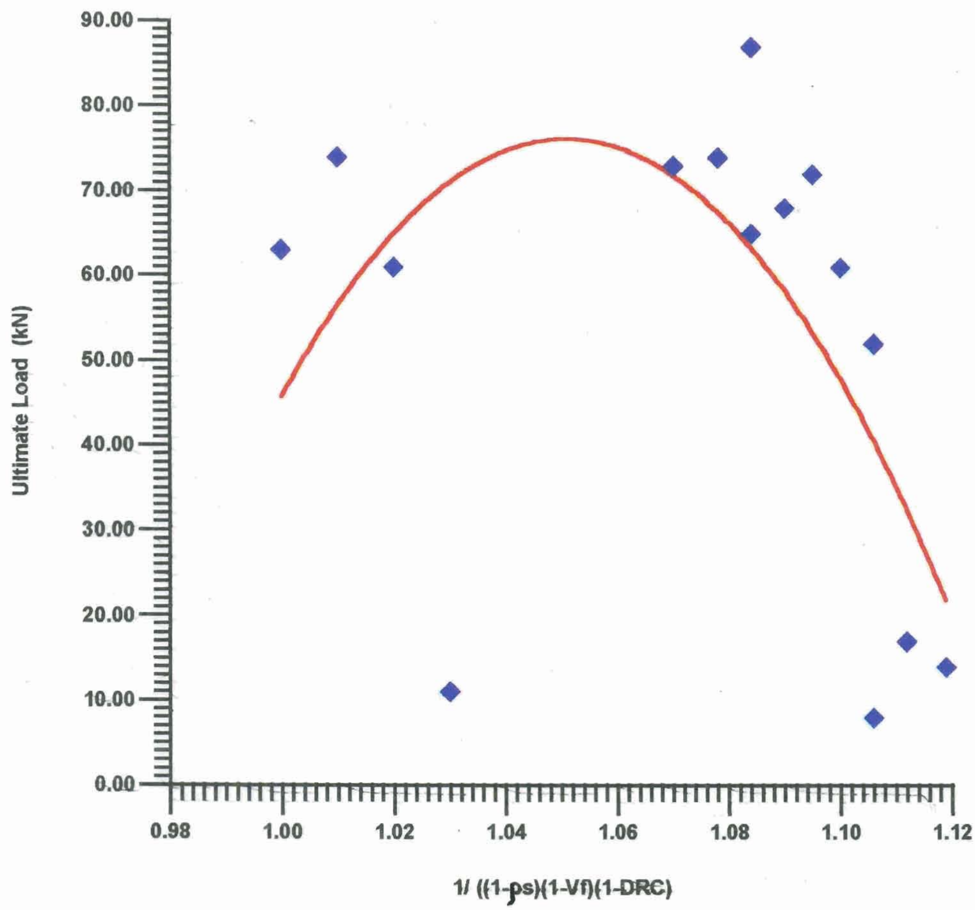


Fig. 5.17 Ultimate Load versus Constitutive Parameter

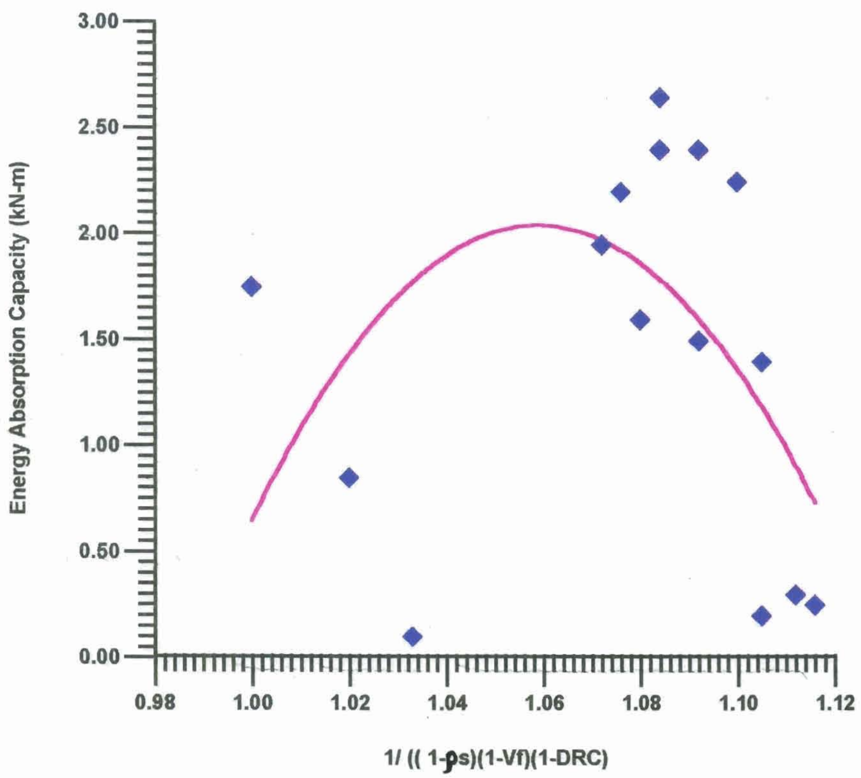


Fig. 5.18 Energy Absorption Capacity (U) versus Constitutive Parameter

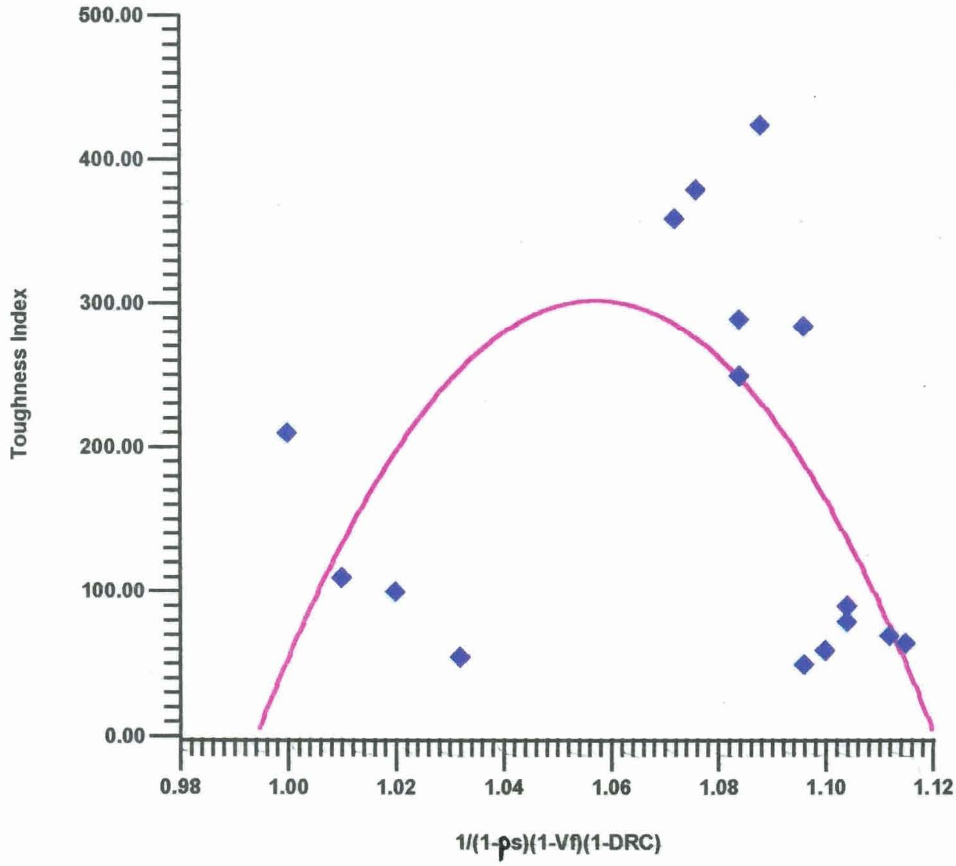


Fig. 5.19 Toughness Index (TI) versus Constitutive Parameter

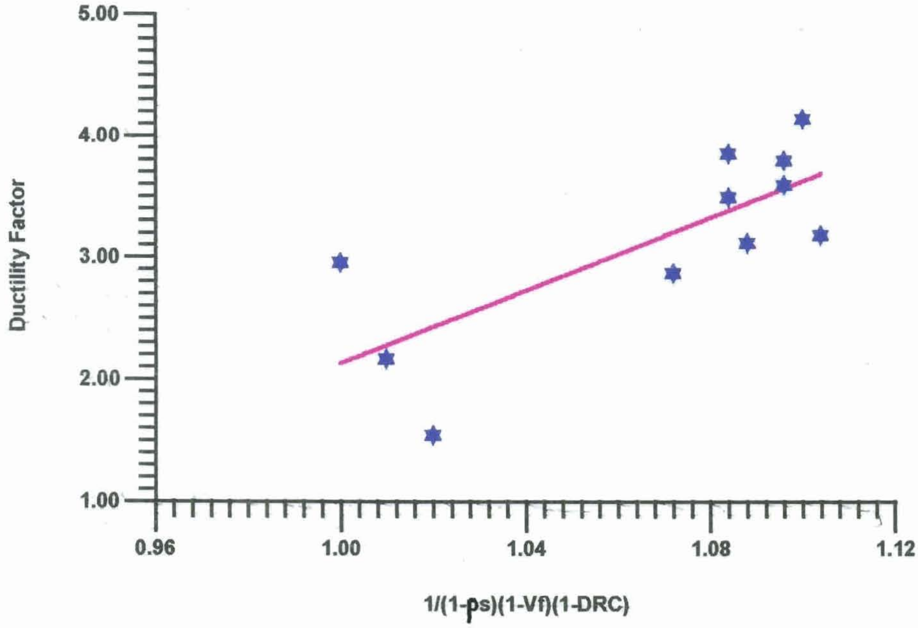


Fig. 5.20 Ductility Factor versus Constitutive Parameter

It may be seen from these figures that as Cp increases the energy absorption capacity (U) and Toughness Index (TI) increases for C_p upto 1.05. Beyond these points, the value of U, and TI decreases. This is mainly due to the addition of DRC more than 1.0% . The reasons for the reduction in values of Pu , U, and TI at higher percentages of DRC has been already explained in section 3.3.4.

5.10 Load Factor

In the limit state design of reinforced concrete structures, the design should both the safety and serviceability criteria. As far as safety is concerned, the member subjected to bending should be safe against limit state of collapse against flexure and shear. The laod factor considered in the case of limit state of collapse against flexure and shear is 1.5. Regarding the serviceability, limit states of deflection and cracking are the important ones. An attempt is made to obtain the load factors with respect to limit state of deflection and cracking in the case of latex modified reinforced concrete flexural members with confined SFRC in the compression zone. Load factors with respect to the limit state of deflection and the limit state of cracking were calculated to understand whether the load factor with respect to strength or with respect to deflection or with respect to cracking governs the design. The following procedure was adopted to calculate the various load factors.

5.10.1 Load factor with respect to limit state of deflection

The load factor with respect to the limit state of deflection was calculated considering the following points.

a) For serviceability conditions, the allowable total deflection δ_t is limited to span/250.

The total deflection is the sum of short-term and long-term deflections.

b) The deflection obtained from the experiment is short time or immediate deflection δ_i, long term deflection δ_l is calculated as [71]:

$$\delta_l = (2.0 - 1.2\ (A_{sc}/A_{st}))\ \delta_i \qquad \ldots(5.11)$$

provided,

$$(2.0 - 1.2\ (A_{sc}/A_{st})) \geq 0.60 \qquad \ldots..(5.12)$$

c) The total deflection of the specimen is equal to the sum of experimental deflection and the deflection obtained from the equation (5.11).

$$\delta_t = \delta_i + \delta_l \qquad \ldots(5.13)$$

$$= \delta_i + \delta_i\ (2.0 - 1.2\ (A_{sc}/A_{st}))$$

$$= \delta_i\ (3.0 - 1.2\ (A_{sc}/A_{st})) \qquad \ldots..(5.14)$$

where A_{sc} = area of steel in compression

A_{st} = area of steel in tension

d) The value of δ_i is obtained from the equation (5.14) by substituting

$$\delta_t = \text{Span}/250$$

Corresponding to this δ_i, the load P_δ is obtained from the experimental load deflection plot.

e) The load factor with respect to limit state of deflection can be calculated using,

$$\text{L.F.} = P_u / P_\delta \qquad \ldots.(5.15)$$

The values of load factors with respect limit state of deflection obtained for all the specimens are given in Table 5.6. It can be seen that the load factor for all the specimens are more than 1.5, which is normally considered as strength factor. This indicate that while designing the confined latex- modified SFRC beams, sufficient attention is to be paid to the deflection criterion, in addition to the strength criterion.

Table 5.6
Load Factor with respect to Limit state of Deflection

Sl. No.	Beam designation	Ultimate Load P_u(KN)	Load at δ_l=2.724 mm (KN) P_δ	Load Factor P_u/P_δ
1	$C_0L_0F_0$	63.74	32.00	1.99
2	$C_0L_1F_0$	73.55	30.00	2.45
3	$C_0L_2F_0$	60.80	28.00	2.17
4	$C_0L_3F_0$	10.79	07.14	1.51
5	$C_1L_0F_1$	72.56	34.29	2.12
6	$C_1L_0F_2$	73.55	30.29	2.43
7	$C_1L_0F_3$	64.72	32.00	2.02
8	$C_1L_1F_1$	87.27	30.00	2.91
9	$C_1L_1F_2$	67.66	27.71	2.44
10	$C_1L_1F_3$	71.58	30.00	2.39
11	$C_1L_2F_1$	41.19	12.00	3.43
12	$C_1L_2F_2$	60.80	20.00	3.04
13	$C_1L_2F_3$	50.99	20.00	2.55
14	$C_1L_3F_1$	07.85	04.00	1.96
15	$C_1L_3F_2$	15.69	10.00	1.57
16	$C_1L_3F_3$	13.73	08.57	1.60

5.10.2 Load factor with respect to limit state of cracking

Cracking in reinforced concrete structures is a random phenomenon. Crack occurs as and when the modulus of rupture of concrete is exceeded. As per the limit state of crack width, structural member is said to violate this condition when the width of crack appears at the soffit exceeds the permitted value of crack width. As per IS 456-1978, the allowable maximum crack width is 0.3 mm. To find the load factor with respect to the limit state of cracking, the data obtained from the crack width measurements taken during the experiment was used. From this, the load (P_{cr}) corresponding to a crack width of 0.3 mm was found out for each specimen. Then the load factor was calculated using the equation

$$L.F. = P_u / P \qquad \text{....(5.16)}$$

Referring to Table 5.7, it can be seen that the load factor with respect to the limit state of cracking is less than 1.5 for most of the specimens, the exceptions being the plain beam and those with 3.0% DRC. The reason may be due to the following : In polymer modified concrete, at low values of DRC (i.e, up to 1.0%) the monolithic network formed by the interpenetration of polymer phase and cement hydrate phase increases the crack resistance of the specimens. But for higher dosage of polymer addition, the air entrainment causes discontinuities in the monolithic network structures and excess latex form weak spots, which reduces the crack resistance.

Hence in short, it can be said that the limit state of deflection controls the design of polymer modified beams with Confined SFRC in the compression zone.

Table : 5.7

Load factor with respect to the Limit state of Cracking

Sl. No.	Beam designation	Ultimate Load P_u(KN)	Load at Crack width w = 0.3 mm P_{cr} (KN)	Load Factor P_u/P_{cr}
1	$C_0L_0F_0$	63.74	26.80	2.38
2	$C_0L_1F_0$	73.55	58.84	1.25
3	$C_0L_2F_0$	60.80	31.38	1.94
4	$C_0L_3F_0$	10.79	06.86	1.57
5	$C_1L_0F_1$	72.56	39.22	1.85
6	$C_1L_0F_2$	73.55	60.80	1.21
7	$C_1L_0F_3$	64.72	50.99	1.27
8	$C_1L_1F_1$	87.27	70.60	1.24
9	$C_1L_1F_2$	67.66	52.95	1.28
10	$C_1L_1F_3$	71.58	62.76	1.14
11	$C_1L_2F_1$	41.19	27.46	1.50
12	$C_1L_2F_2$	60.80	50.99	1.19
13	$C_1L_2F_3$	50.99	37.26	1.37
14	$C_1L_3F_1$	07.85	01.96	4.00
15	$C_1L_3F_2$	15.69	07.85	2.00
16	$C_1L_3F_3$	13.73	07.85	1.75

5.11 Conclusions

Based on the experimental and analytical studies conducted, the following conclusions are arrived at:

1. The provision of confined SFRC in the compression zone of polymer modified reinforced concrete beams, in general, increases the load carrying capacity and the ductility of the specimens. This is more predominant in the case of ductility of the specimens.

2. The investigation indicate that, the optimum dosage of DRC is 1.0%. Beyond this limit, there is infact, reduction in the load carrying capacity was noticed. For specimens with 3.0% DRC the reduction in strength is drastic.

3. Load versus deflection plots indicates that there is considerable improvement in the ductility of the specimen upto 1.0% DRC. Also in the case of beams with confined SFRC alone (no polymer modification) this improvement is marginal. Hence it can be said that the addition of latex (to the optimum level) contributes to the ductility of the specimens in a major way.

4. The energy absorption capacity and the toughness index of the specimens increases upto certain level and then decreases as the DRC increases.

5. The latex modification (upto 1% DRC), the incorporation of confinement and the addition of steel fibres in the compression zone have enhanced the strength and ductile behaviour. This indicate that these parameters can be introduced in converting the brittle behaviour of over reinforced concrete flexural members into a ductile one. Hence the maximum longitudinal reinforcement ratio for the flexural members prescribed by the Code of practice could be raised to increase the flexural capacity of beams. This would be beneficial in situations where there is a restriction on the overall depth of beams, particularly if the beams are subjected to large bending moments. Use of over-reinforced beams with polymer modification and confined SFRC in the compression zone can be considered in this case as alternative to the use of prestressed concrete construction.

6. latex modification, the incorporation of confinement and the addition of steel fibres in the compression zone, enhances the strength and ductility of RCC beams and hence seems to be appropriate for seismic resistant structures.

7. Load factor with respect to the limit state of deflection controls the design of the latex modified reinforced concrete beams with confined SFRC in the compression zone, when compared to those with respect to the Limit State of collapse against flexure and the Limit state of cracking.

STUDIES ON CRACKING OF LATEX MODIFIED STEEL FIBRE REINFORCED CONCRETE FLEXURAL MEMBERS

6.1 Introduction

The occurrence of cracks in reinforced concrete structures is inevitable because of the low tensile strength of concrete. Cracks form when the tensile stress in concrete exceeds its tensile strength. In the limit state design of concrete structure, the width of crack is an important parameter to be considered for the serviceability of the member. Limiting the crack width is important from the aesthetic point of view, to ensure water tightness and to safe guard the reinforcement against corrosion [4]. Many variables influence the width and spacing of cracks in reinforced concrete members. Due to the complexity of the problem, a number of methods have been developed in the past to determine the width and spacing of cracks. These methods are generally based partly on theoretical basis and partly on test results. Some investigators also developed empirical equations from statistical analysis of test results.

Some of these methods for predicting maximum width of cracks have been adopted by International Codes of Practice with or without modifications. In this Chapter, an attempt is made to compare the methods adopted in the International Codes of Practice for predicting the maximum width of cracks using the test results reported in literature. Similar comparison involving earlier version of British Code, viz. CP

Based on the studies presented in this Chapter, the following technical papers have been published:

1. **"Comparison of International Codes for the Prediction of maximum Width of Cracks in Reinforced Concrete Flexural Members"** Paper published in The Indian Concrete Journal, Nov. 1996, pp-635-641.
2. **"Prediction of Spacing and Maximum Width of Cracks in Steel Fibre Reinforced Concrete Flexural Members"**, Paper published in the Journal of Structural Engineering, SERC Madras. Vol. 24, No.3, Oct. 1997, pp. 143- 148.

110-1972 and Model Code 1978 have been already reported [20]. As these equations have been revised recently, the revised ones are considered in this study. Also an attempt is made to propose a method for predicting the spacing and maximum width of cracks in reinforced concrete flexural members additionally reinforced with steel fibres. Further this proposed me iod have been extended to predict the spacing and maximum width of cracks in the cas of latex modified steel fibre reinforced concrete flexural members.

6.2 Comparison of International Codes for the Prediction of Maximum Width of Cracks in Reinforced Concrete Flexural Members

To start with, the International equations available in literature for the prediction of maximum width of cracks in reinforced concrete flexural members have been c red and these equations have been compared with the available test results.

6.3 Test Results

Test results of Hognestad [35], Clark [17] and Base et al [8] available in literature have been used for comparing their experimental maximum width of cracks with those predicted by International equations. Only beams having rectangular cross section and reinforced with deformed bars are used. A total of 732 test results are considered for comparison. A brief description of these test results is given in the following sections.

6.3.1 Test results of Hognestad [35]

Hognestad conducted experimental study on the nature of cracking of reinforced concrete beams. He reported crack widths at centroid of reinforcement for steel stresses ranging from 20000 psi (137.9 N/mm^2) to 50000 psi (344.7 N/mm^2) for every 10000 psi (68.9 N/mm^2) increments. Out of 36 beams tested, 4 beams were reinforced with plain bars and are not considered in this analysis. Thus a total of 121 crack width readings on the sides of the beam at the centroid of reinforcement level are used here. Details of the data collected are given in Table 6.1.

6.3.2 Test results of Clark [17]

Clark tested 54 specimens and reported maximum crack width and spacing for steel stresses ranging from 15000 psi (103.4 N/mm^2) to 35000 psi (241.3 N/mm^2) at every 5000 psi (34.5 N/mm^2) increment. A total of 250 maximum crack width readings are available. Out of these 250 maximum crack widths, the following three first maximum crack widths are not used in the analysis.

(i) 0.0172 mm (0.00068 in) in the case of specimen No. 6-71/2-4-1 at a steel stress of 103.4 N/mm^2 (15,000 psi)

(ii) 0.0373 mm (0.00147 in) in the case of specimen No. 6-12-3-2 at a stress of 206.8 N/mm^2 (30,000 psi)

(iii) 0.0408 mm (0.00161 in) in the case of specimen No. 6-12-3-1 at a steel stress of 241.3 N/mm^2 (35,000 psi)

The above three values of crack widths gave very high values of the ratio of calculated crack width to the experimental crack width. This may be due to the difficulty involved in the recognition of the location of the first crack and measurement of the width as soon as it appears -the width being too small for accurate measurement. Thus, a total of 247 maximum crack widths from the studies of Clark have been used in this analysis. The details of the data collected are given in Table 6.1

6.3.3 Test results of Base et al. [8]

Base et al. of Cement and Concrete Association of London carried out an extensive investigation on the control of flexural cracking of rectangular beams. Tests were carried out on 133 beams using plain and different types of deformed bars incorporating several variables like size of bar, side cover, bottom cover, effective reinforcement ratio, concrete strength, curing condition etc. The maximum crack widths at the lower extreme tensile fibres and the corresponding surface strain were interpolated from the graphs given in the supplement part of their report. Thus a total of 364 values of maximum crack width reported are considered for comparison. The details of the test results used are given in Table 6.1.

TABLE 6.1

Details of Test Results Used in the calculations.

Source	Type of specimen	No. of observations	Steel Stress in MPa	Steel Stress in lb/sq.in
Hognestad:		28	137.9	20,000
Crack width at	Reinforced	32	206.8	30,000
the level of	concrete	32	275.8	40,000
reinforcement	beams	29	344.7	50,000
Clark:	Reinforced	45	103.4	15,000
Crack width at	concrete	49	137.9	20,000
the soffit	beams and	52	172.9	25,000
	slabs	51	206.8	30,000
		50	241.3	35,000
Base et.al:		75	46.90	6,800
F_s corresponds	Reinforced	75	112.4	16,300
to the average	concrete	77	177.2	25,700
stress in steel	beams	74	241.3	35,000
for these obser		63	303.4	44,000
vations				

6.4 Comparison of International Equations with the Test Results

The equations available for estimating maximum crack width in reinforced concrete flexural members have been already given in the literature survey. The same has been reproduced here for clarity.

6.4.1 BS 8110 - 1985 equation [12]

The British code suggests an expression for calculating the design surface crack width, provided the strain in the tension reinforcement is limited to $0.8f_y/E_s$ and the design surface crack width, which should not exceed the appropriate values given for the appearance and corrosion (0.3 mm) as

$$W_{bt} = \frac{3\, a_{cr}\, \varepsilon_m}{1 + 2\,\dfrac{(a_{cr} - c_{min})}{(h - x)}} \qquad \ldots(6.1)$$

where a_{cr} = distance from the point considered to the nearest longitudinal bar

ε_m = average strain at the level where cracking is being considered

C_{min} = minimum cover to tension steel

h = overall depth of member

x = depth of neutral axis.

Average steel strain ε_m is calculated from the equation:

$$\varepsilon_m = \varepsilon_l - \frac{b_t\,(h - x)\,(a' - x)}{3\, E_s\, A_s\,(d - x)} \qquad \ldots(6.2)$$

where ε_l = strain in steel at the level considered ignoring the stiffening effect of concrete in the tension zone

b_t = width of the section at the centroid of tension steel a' = distance from the compression face to the point at which the crack width is being measured.

When using equation (6.2), in the case of test results of Hognestad, ε_l is calculated as:

$$\varepsilon_l = \frac{f_s}{E_s} \qquad \ldots(6.3)$$

6.4.2 Model Code 1990 equation [13]

The Model Code 1990 suggests, for all stages of cracking, the design crack width may be calculated according to the following expression.

$$W_k = l_{s,\max} \left(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs} \right) \qquad \ldots(6.4)$$

where $l_{s,max}$ = length over which slip between steel and concrete occurs

ε_{sm} = average strain in steel within $l_{s,max}$

ε_{cm} = average strain in concrete within $l_{s,max}$

ε_{cs} = strain of concrete due to shrinkage which has to be introduced algebraically

$l_{s,max}$ is calculated from the following conditions

If $\rho_{s\ eff}\ \sigma_{s2} > f_{ctm}(t)(1 + \alpha_e\ \rho_{s,ef})$ it may be assumed that the stabilized cracking condition has been reached, otherwise the formation of single crack should be considered.

where $f_{ctm}(t)$ = mean value of the tensile strength of concrete at the time 't' when the crack appeared

α_e = ratio E_s/E_c

$\rho_{s,ef}$ = effective reinforcement ratio (= $A_s/A_{c,ef}$)

$A_{c,ef}$ = effective area of concrete in tension (area of concrete surrounding the tension reinforcement)

σ_{s2} = steel stress at crack

$l_{s\,max}$ is calculated from the following equations:

$$l_{s,\max} = \frac{\phi}{3.6\,\rho_{s\,eff}} \qquad \text{(for stabilised cracking)} \qquad \text{...(6.5 a)}$$

$$l_{s,\max} = \frac{\sigma_{s2}}{2\,\tau_{bk}}\,\phi_s\,\frac{1}{1+\alpha_e\,\rho_{s,ef}} \qquad \text{(for single crack formation)} \qquad \text{...(6.5 b)}$$

where ϕ_s = diameter of bar

According to the Code, in the absence of more refined model the effective area of concrete in tension $A_{c,ef}$ is to be taken as

$$A_{c,ef} = \frac{2.5(h-d)}{b} \quad \text{subjected to a maximum of } \left(\frac{h-x}{3}\right)b$$

In equation (6.4) $\quad \varepsilon_{sm} - \varepsilon_{cm} = \varepsilon_s - \beta\,\varepsilon_{sr2}$

Where ε_s = steel strain at the crack

β = 0.6 for short term / instantaneous loading.

and ε_{sr2} is given by the equation

$$\varepsilon_{sr2} = \frac{f_{ctm}\,(t)}{\rho_{s,ef}\quad E_s}\,(1 + \alpha_e\,\rho_{s,ef}) \qquad \text{...(6.6)}$$

6.4.3 ACI Code 318 - 1995 equation [3]

The specification given in ACI 318-1995 for control of cracking is based on the equation proposed by Gergely and Lutz [31] and hence this equation is considered for comparison. Gergely and Lutz made an extensive statistical analysis of crack width and developed the equations which are as follows:

The side and bottom crack width are given by :

$$W_s = 0.091 \frac{\sqrt[3]{(C_s A)}}{1 + \dfrac{C_s}{(d - x)}} (f_s - 5) x\ 10^{-3} \qquad \ldots(6.7)$$

$$W_{bt} = 0.091 \sqrt[3]{C_b A} \frac{(h - x)}{(d - x)} (f_s - 5)\ 10^{-3} \qquad \ldots(6.8)$$

where C_s	=	side cover measured from the centre of outer bar
C_b	=	bottom cover measured from the centre of lower bar
A	=	area of concrete surrounding one bar
	=	effective depth of tension reinforcement
h	=	overall depth of cross section
x	=	neutral axis of cracked section
f_s	=	steel stress in kips per square inch and all other units are in inches.

While comparing the test results of Base et al, the steel stress f_s are obtained from the surface strain assuming a linear variation.

Thus

$$\varepsilon_s = \text{Surface strain } \frac{(d-x)}{(h-x)}$$

$$f_s = \varepsilon_s\ E_s$$

6.4.4 Chinese Code equation (GBJ 10-89 1989)[105]

Chinese code for concrete structures (GBJ 10-89) proposes the following equation for the maximum width of cracks in flexural members under short-term load.

$$W_{max} = 1.41\,\psi \frac{\sigma_s}{E_s}\,(2.7\,c + 0.11\,\frac{d}{\rho_{te}})^{\gamma} \qquad \ldots(6.9)$$

where	σ_s	=	tensile steel stress at the crack
	E_s	=	modulus of elasticity of steel
	ψ	=	non uniformity coefficient of tensile steel
	c	=	thickness of concrete cover
	d	=	diameter of steel bar
	ρ_{te}	=	A_s / A_{te}
	As	=	area of steel
	A_{te}	=	effective area of tensile steel
	γ	=	coefficient related to the bond properties of steel bar (1.0 for plain bars and 0.7 for deformed bars)

The non-uniformity co-efficient of tensile steel is calculated from the following relation and is,

$$\Psi = 1.1 - 0.65\,\frac{f_{ct}}{(\rho_{te}\ \ \sigma_s)} \qquad \ldots(6.10)$$

where f_{ct} = tensile strength of concrete

If $\psi \leq 0.4$, take $\psi = 0.4$

$\psi \geq 0.4$, take $\psi = 1.0$

6.5 Results

Equations (6.1), (6.4), (6.8) and (6.9) were used to compute the maximum width of cracks (W_{cal}) for each specimen. The computed values were compared with the experimental values (W_{exp}). Results of the comparison are given in Table 6.2 to 6.5. The value of the average and coefficient of variation of ratio W_{cal}/W_{exp} are given in

i. Table 6.2 for the test results of Hognestad
ii. Table 6.3 for the test results of Clark
iii. Table 6.4 for the test results of Base et al.
iv. Table 6.5 for the test results of Hognestad, Clark and Base et al for all the stages of steel stress

The consolidated results are given in Table 6.6. The model calculations for determining the maximum width of crack using B.S.8110-1985 equation, Model Code 1990 equation, ACI 318-1995 equation and Chinese code GBJ 10-89 equation are given in Appendix - II. Fig. 6.1 to 6.12 gives the comparison of calculated crack width with the experimental values for all the equations.

Table 6.2

Comparison of Calculated crack width with the test results of Hognestad [35]

Sl. No.	Equation Used	No. of Obser-vations	Value of σ_s N/mm^2	W_{cal}/W_{exp} Average	Std. Dev.	Coeff. of Variation (%)
1	B.S. Equation	28	137.9	0.914	0.284	31.08
		32	206.8	0.773	0.204	26.33
		32	275.8	0.726	0.214	29.46
		29	344.7	0.713	0.200	28.08
2	Model Code equation	28	137.9	0.640	0.278	43.49
		32	206.8	0.639	0.274	42.82
		32	275.8	0.620	0.270	43.55
		29	344.7	0.658	0.262	39.74
3	Gergely Lutz equation	28	137.9	1.020	0.305	29.93
		32	206.8	0.940	0.232	24.64
		32	275.8	0.892	0.210	23.57
		29	344.7	0.906	0.198	21.81
4	Chinese Code equation	28	137.9	0.928	0.244	26.26
		32	206.8	0.836	0.193	22.43
		32	275.8	0.833	0.200	24.02
		29	344.7	0.845	0.195	23.02

Table 6.3

Comparison of Calculated crack width with the test results of Clark [17]

Sl. No.	Equation Used	No. of Observa -tions	Value of σ_s N/mm^2	W_{cal} / W_{exp}		
				Average	Std. Dev.	Coeff. of Variation (%)
1	B.S. Equation	45	103.4	0.882	0.235	26.63
		49	137.9	0.830	0.242	29.21
		52	172.4	0.812	0.247	30.36
		51	206.8	0.800	0.198	24.80
2	Model Code equation	45	103.4	0.635	0.277	35.74
		49	137.9	0.649	0.245	37.82
		52	172.4	0.677	0.306	45.17
		51	206.8	0.683	0.243	35.65
		50	241.3	0.664	0.243	36.55
3	Gergely Lutz equation.	45	103.4	1.026	0.305	29.68
		49	137.9	1.018	0.263	25.78
		52	172.4	1.033	0.289	28.02
		51	206.8	1.037	0.218	21.03
		50	241.3	1.013	0.241	23.83
4	Chinese Code equation.	45	103.4	0.595	0.175	29.47
		49	137.9	0.591	0.182	30.75
		52	172.4	0.591	0.177	29.93
		51	206.8	0.592	0.143	24.08
		50	241.3	0.573	0.138	24.11

Table 6.4

Comparison of Calculated crack width with the test results of Base et al [8]

Sl. No.	Equation Used	No. of Observa -tions	Value of σ_s N/mm^2	W_{cal} / W_{exp}		
				Average	Std. Dev.	Coeff. of Variation
1	B.S. Equation	75	46.90	0.933	0.355	38.04
		75	112.40	1.071	0.420	39.22
		77	177.20	1.094	0.416	38.04
		74	241.30	1.091	0.434	39.73
		63	303.40	1.139	0.433	38.04
2	Model Code equation	75	46.90	0.105	0.078	74.32
		75	112.40	0.269	0.141	52.41
		77	177.20	0.314	0.151	47.88
		74	241.30	0.330	0.173	52.47
		63	303.40	0.358	0.183	50.98
3	Gergely Lutz equation.	64	46.90	0.301	0.214	71.31
		75	112.40	0.860	0.325	37.72
		77	177.20	1.029	0.340	33.04
		74	241.30	1.103	0.403	36.55
		63	303.40	1.183	0.389	32.88
4	Chinese Code equation.	75	46.90	0.286	0.136	47.56
		75	112.40	0.535	0.248	46.34
		77	177.20	0.612	0.263	42.95
		74	241.30	0.637	0.282	44.20
		63	303.40	0.678	0.285	42.08

Table 6.5

Comparison of Calculated crack width with the test results for all the stages of steel stress

Source	Equation Used to calculate theoretical crack width	No. of Observations	W_{cal} / W_{exp}		
			Average	Std. Dev.	Coeff. of Variation
Hogne-stad Data	B.S. Equation	121	0.779	0.239	30.74
	Model Code Eq.	121	0.639	0.271	42.46
	Gergely Lutz eq.	121	0.938	0.243	25.95
	Chinese Code eq.	121	0.866	0.211	24.40
Clark Data	B.S. Equation	197	0.829	0.233	28.12
	Model Code Eq.	247	0.662	0.256	38.64
	Gergely Lutz eq.	247	1.026	0.264	25.78
	Chinese Code eq.	247	0.588	0.164	27.86
Base & others	B.S. Equation	364	1.063	0.418	39.28
	Model Code Eq.	360	0.275	0.173	63.16
	Gergely Lutz eq.	353	0.904	0.457	50.56
	Chinese Code eq.	364	0.545	0.284	52.15

Table 6.6

Comparison of Calculated crack width with all the reported test results

Sl. No.	Equation Used to calculate theoretical crack width	No. of Observations	[illegible]		
			Average	Std. Dev.	Coefficient of Variation
1	B. S. Equation	682	0.945	0.368	38.90
2	Model Code Eq.	728	0.467	0.297	62.66
3(a)	Gergely Lutz eq.	721	0.951	0.373	39.22
(b)	Omitting the first stages of Base et al	657	1.015	0.321	31.63
4	Chinese Code eq.	732	0.613	0.264	43.08

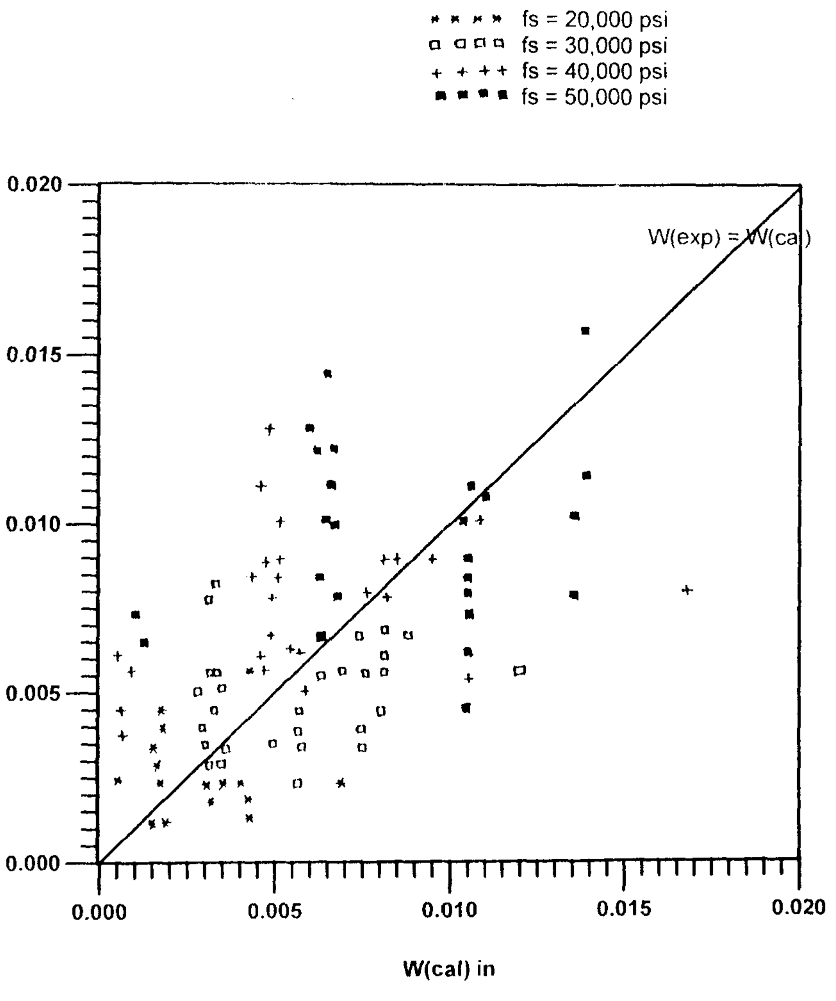


Fig. 6.1 Comparison of calculated crack width using BS 8110 equation with the experimental crack width of Hognestad

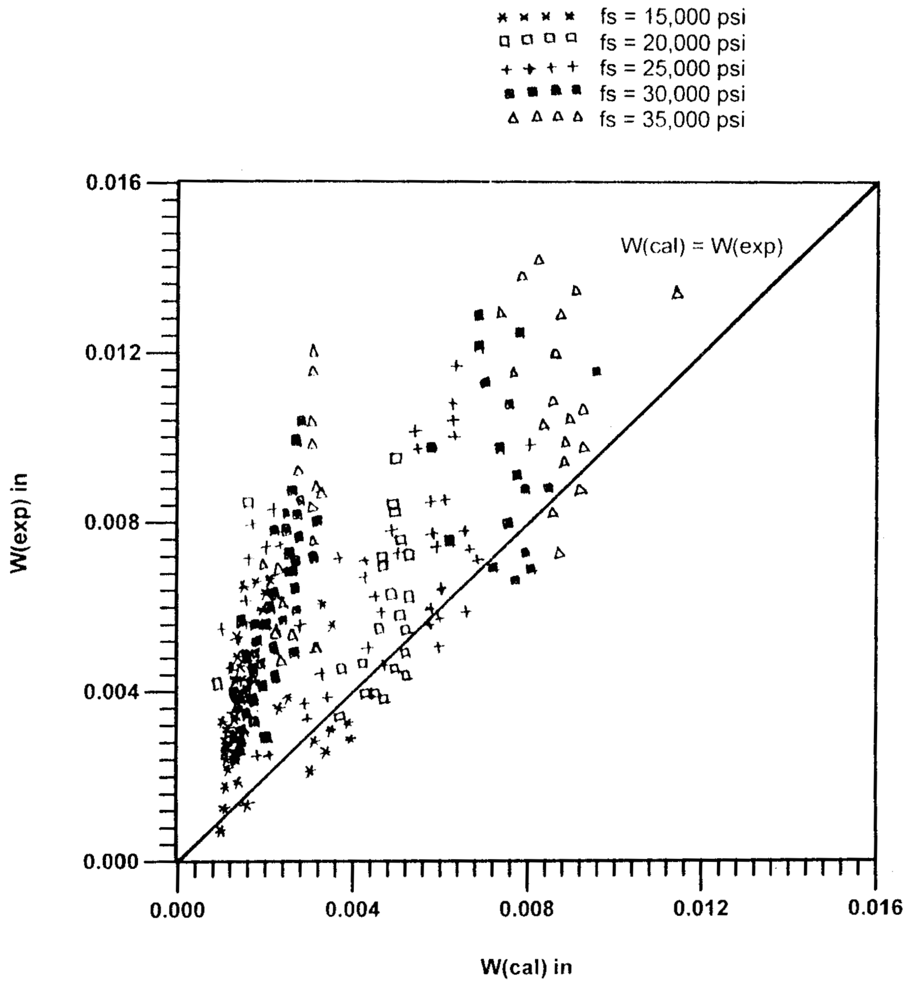


Fig. 6.2 Comparison of calculated crack width using BS 8110 equation with the experimental crack width of Clark

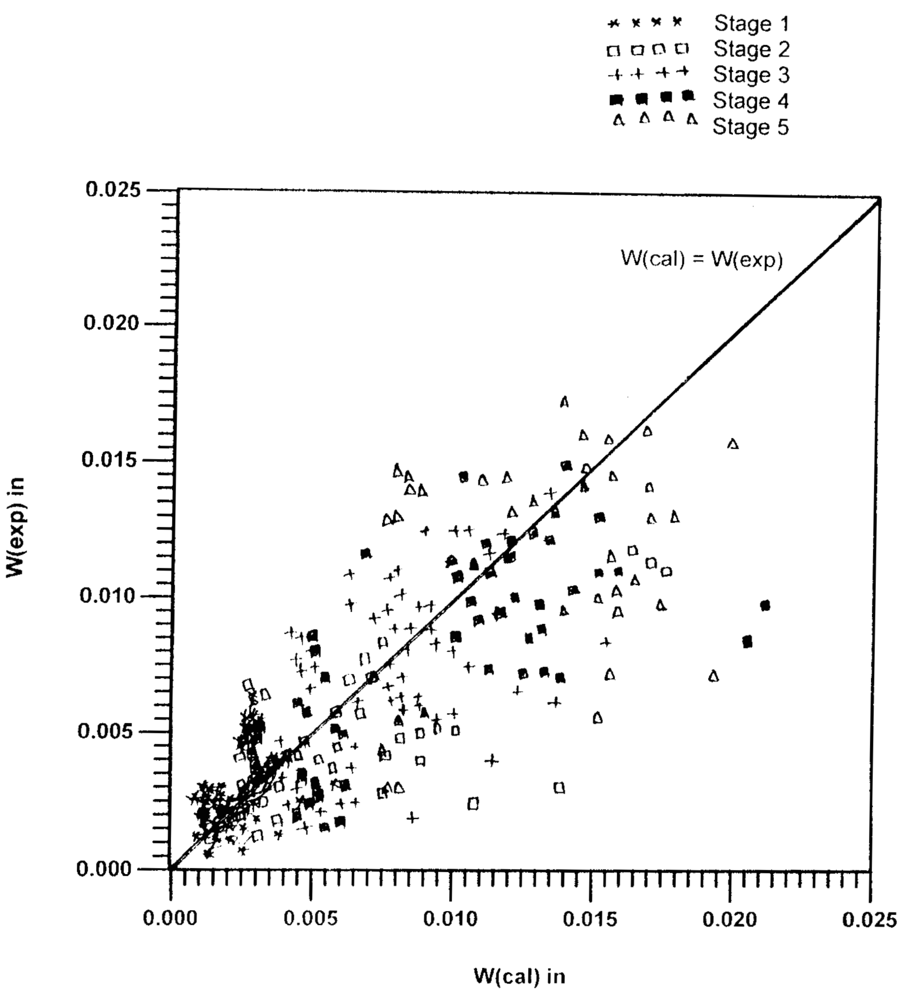


Fig. 6.3 Comparison of calculated crack width using BS 8110 equation with the experimental crack width of Base et al

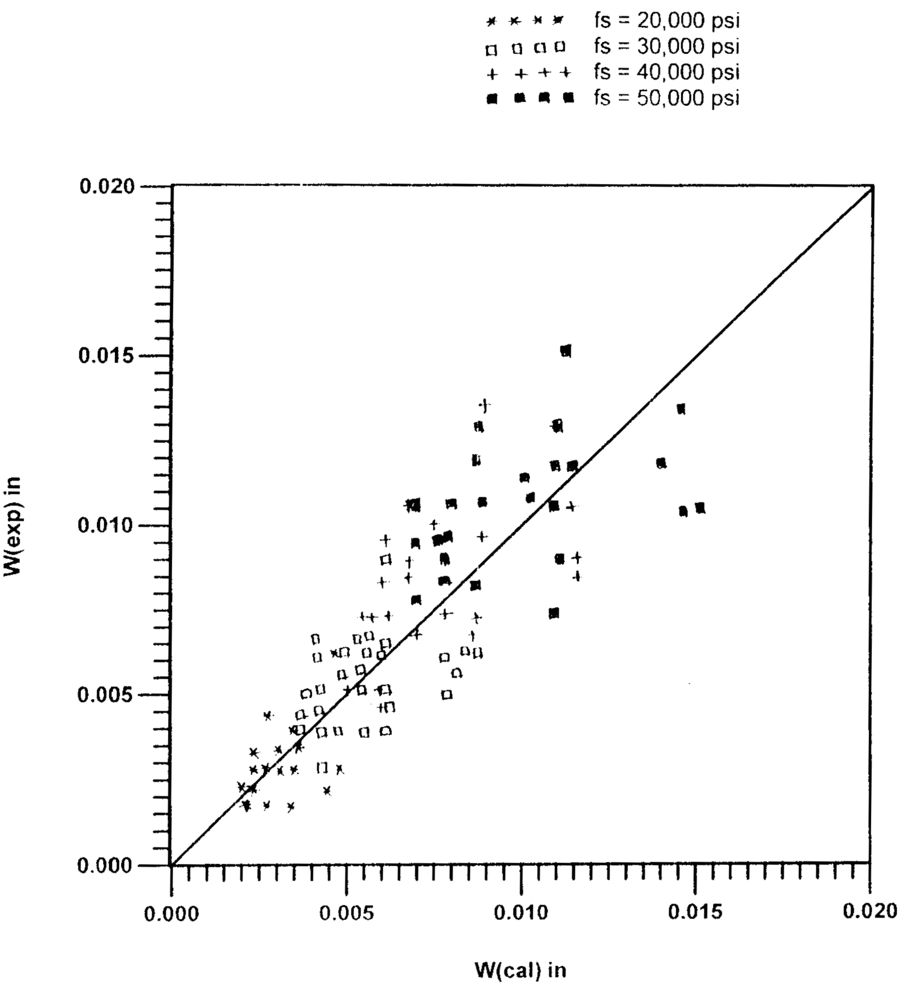


Fig. 6.4 Comparison of calculated crack width using Gergely - Lutz equation with the experimental crack width of Hognestad

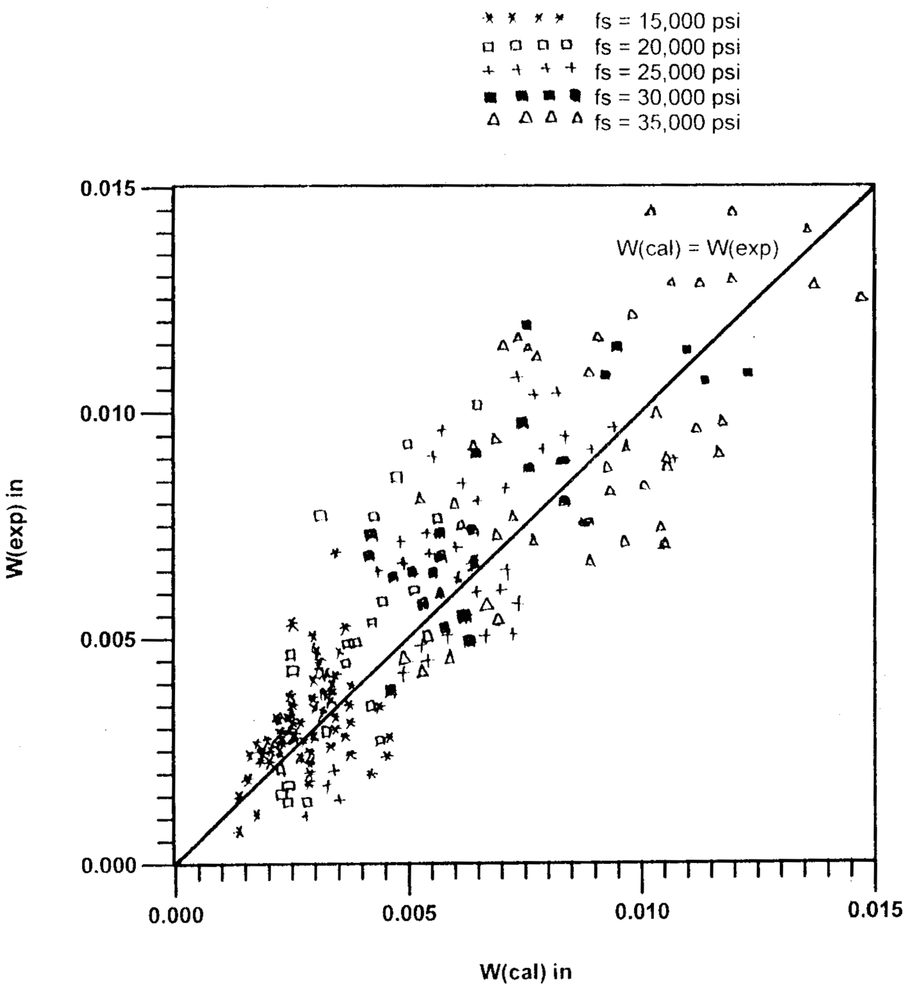


Fig. 6.5 Comparison of calculated crack width using Gergely - Lutz equation with the experimental crack width of Clark

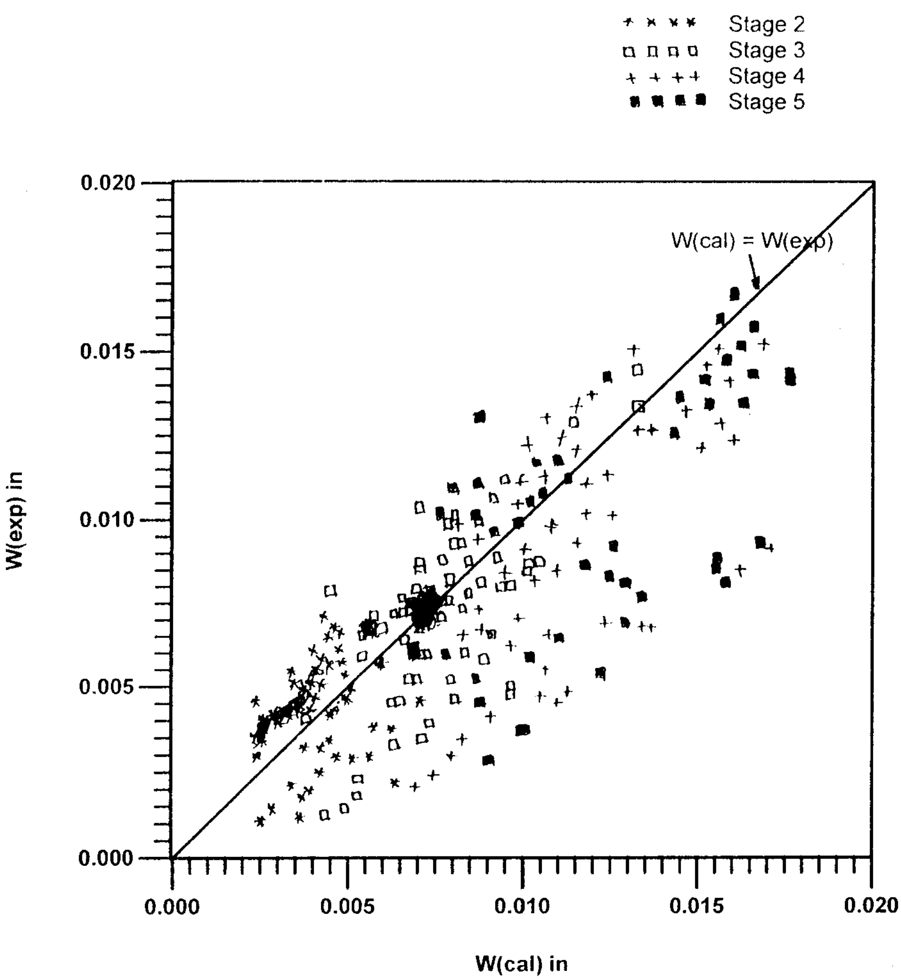


Fig. 6.6 Comparison of calculated crack width using Gergely - Lutz equation with the experimental crack width of Base et al

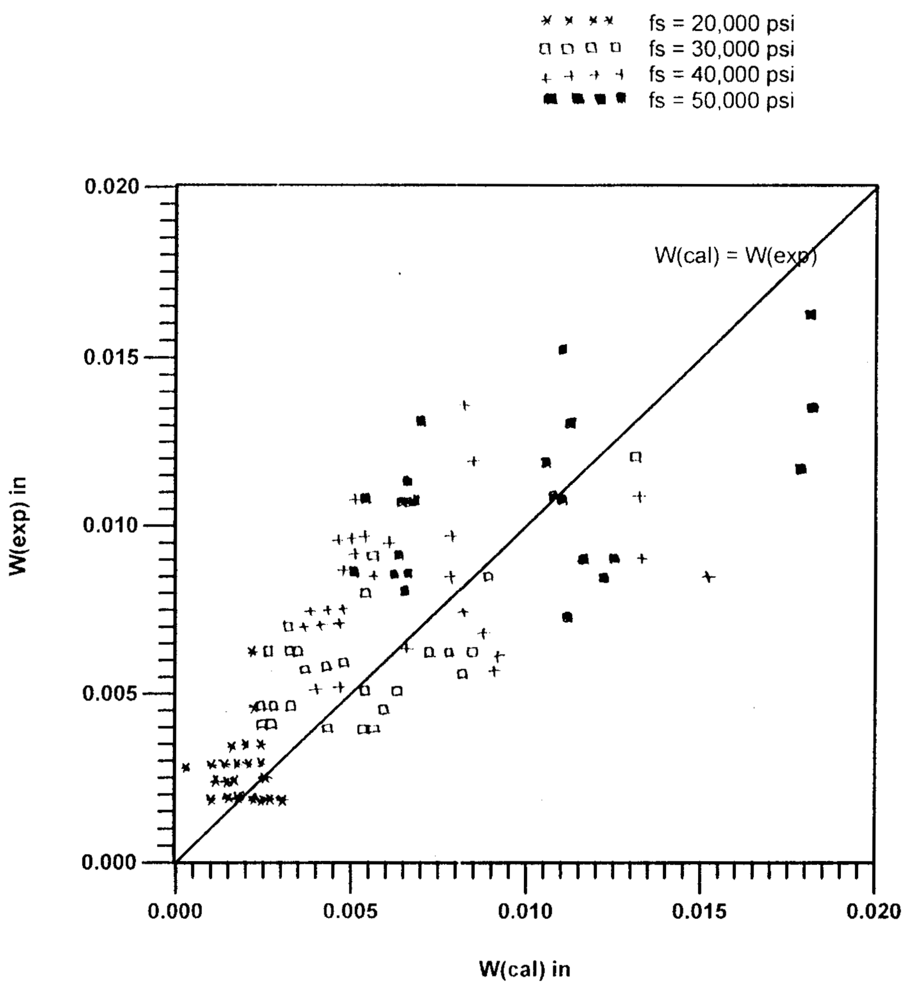


Fig. 6.7 Comparison of calculated crack width using Model Code equation with the experimental crack width of Hognestad

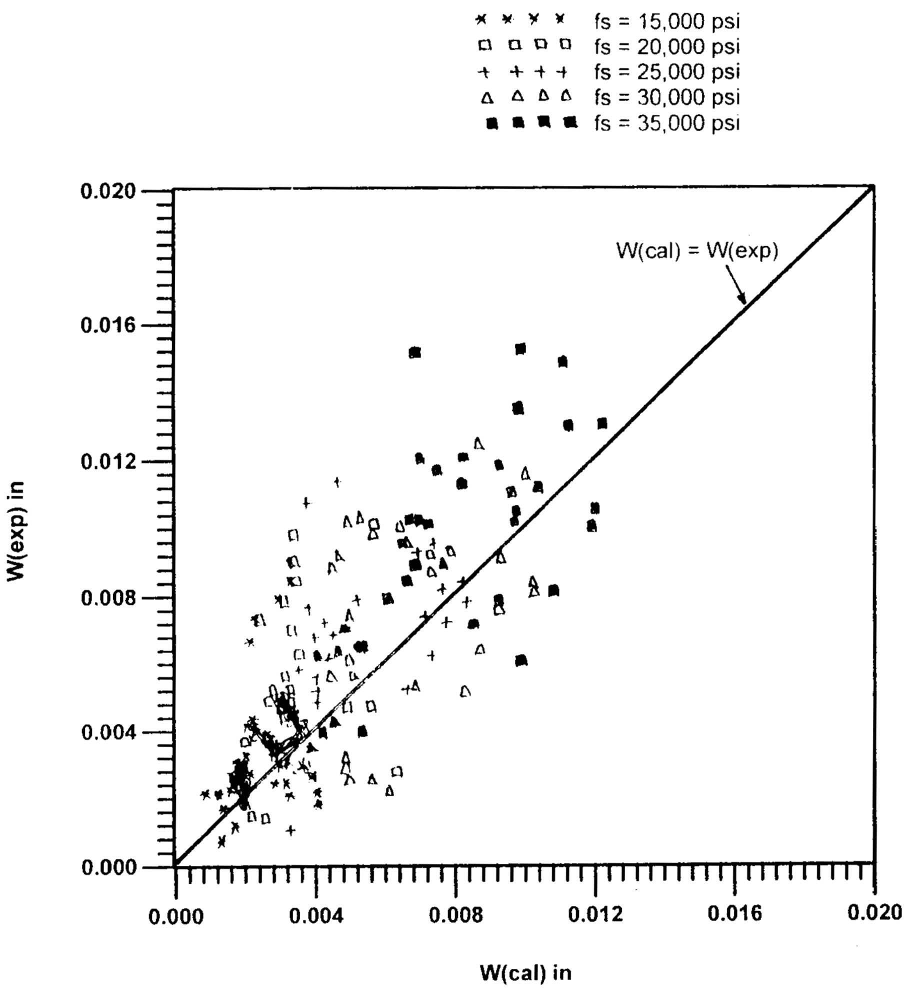


Fig. 6.8 Comparison of calculated crack width using Model Code equation with the experimental crack width of Clark

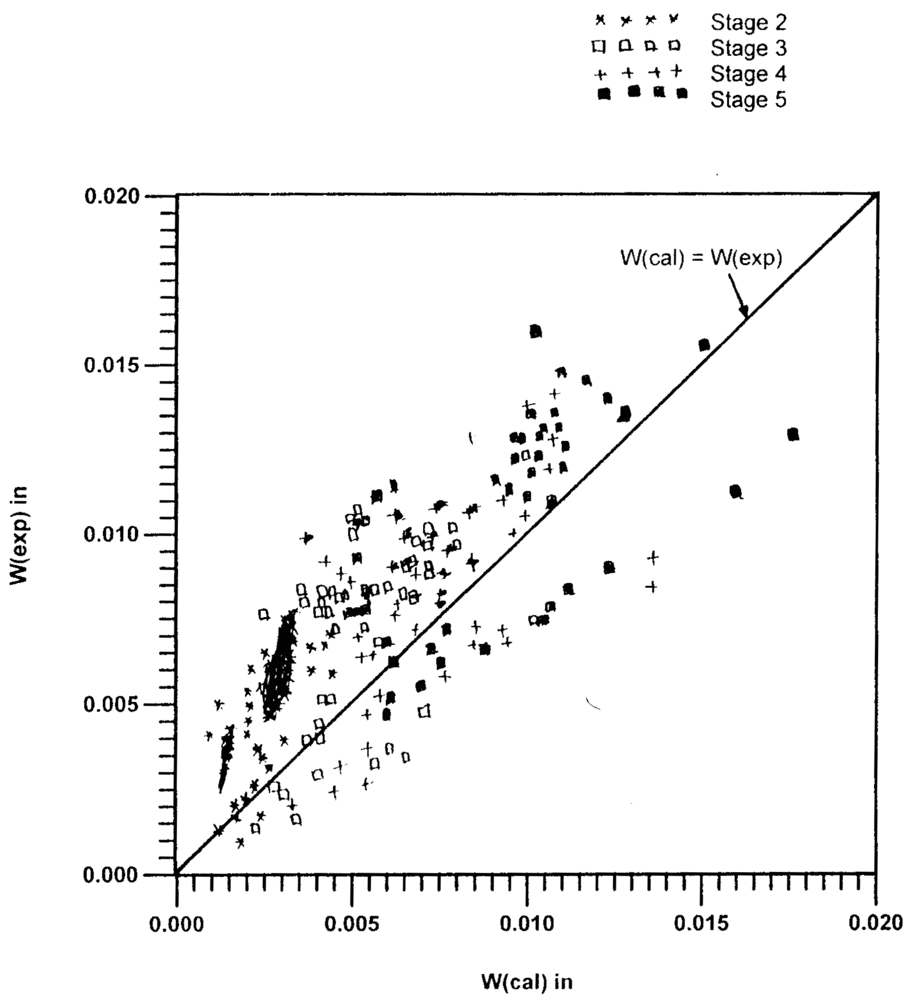


Fig. 6.9 Comparison of calculated crack width using Model Code equation with the experimental crack width of Base et al

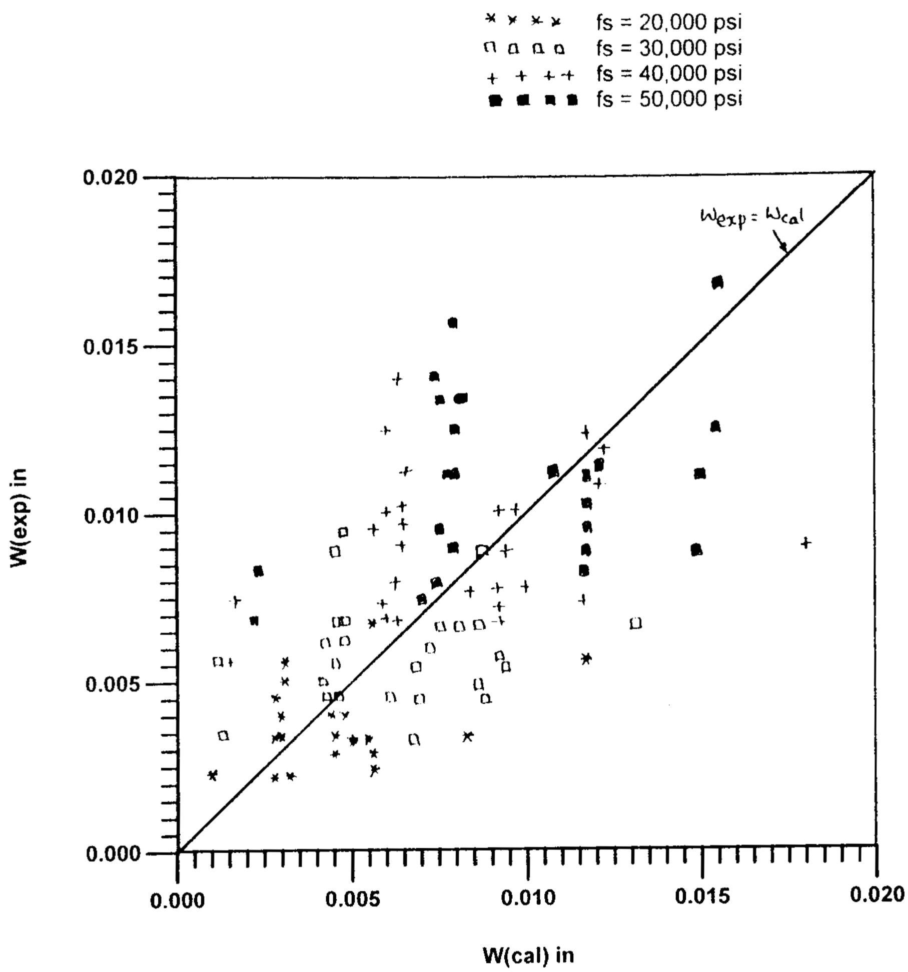


Fig. 6.10 Comparison of calculated crack width using Chinese Code equation with the experimental crack width of Hognestad

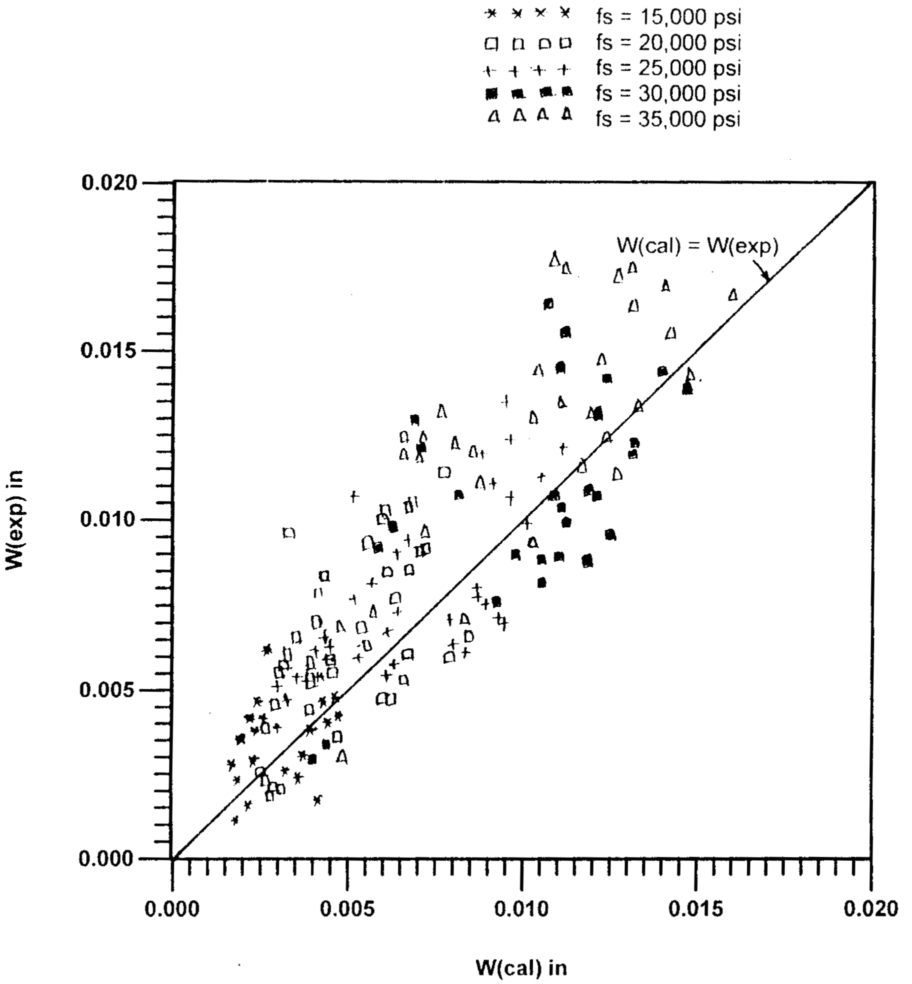


Fig. 6.11 Comparison of calculated crack width using Chinese Code equation with the experimental crack width of Clark

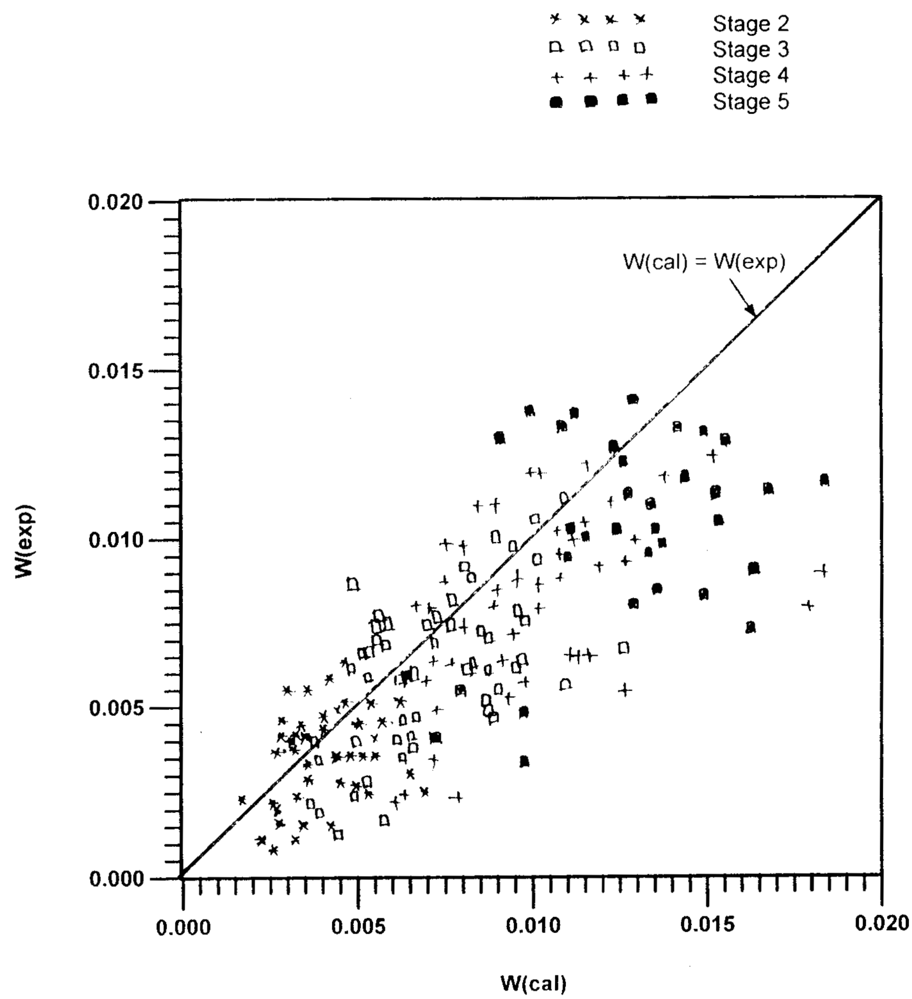


Fig. 6.12 Comparison of calculated crack width using Chinese Code equation with the experimental crack width of Base et al

6.6 Discussion of Test Results

From the results obtained (Table 6.2 to 6.5) the following points are noted.

1. **B.S 8110-1985 equation** underestimates the crack width (i) in the case of test results of Hognestad and Clark and (ii) in the case of test results of Base et al. at steel stress level of 46.9 N/mm^2. However the calculated crack widths compare better with the test results of Base et al. than the test results of other two sources. For all the test results of Base et al., B.S equation (Table 6.5) has given the average ratio of W_{cal}/W_{exp} as 1.063 and the coefficient of variation of 39.28%. However when all the test results from the three sources are put together (Table 6.6) B.S equation under estimates the crack width by 5.5% ; W_{cal}/W_{exp} is 0.945 on an average with a coefficient of variation of 38.90%.

2. **Model Code equation (1990)** under-estimates the values of crack width for all the test results of Hognestad, Clark and Base et al. With all the test results put together (Table 6.6), Model Code equation under-estimates the crack width by 53.3% and the value of coefficient of variation of W_{cal}/W_{exp} is 62.66%.

3. **ACI : 318-1995 Code equation** over estimates the values of crack width (i) in the case of test results of Hognestad at the steel stress level 137.9 N/mm^2 (ii) in the case of test results of Clark, at all steel stress levels (iii) in the case of test results of Base et al at steel stress levels of 177.2, 241.3 and 303.4 N/mm^2. In the case of test results of Base et al, the average of the ratio of W_{cal}/W_{exp} at steel stress 46.9 N/mm^2 is very low viz., 0.301 and the coefficient of variation is 71.31%. This may be attributed to the reason that the equation (6.7) and (6.8) were obtained by Gergely and Lutz from statistical analysis omitting the test results at steel stress below 96.5 N/mm^2. From Table 6.6 it can be seen that Gergely and Lutz equation with 657 test results (omitting the test results of the first stage of steel stress in the case of Base et al.) over estimates the crack width by 1.5 % and the coefficient of variation of W_{cal}/W_{exp} is 31.63 %.

However with all the 721 test results the equation under estimates the crack width by 4.9% and the coefficient of variation of W_{cal}/W_{exp} is 39.22%.

4. **Chinese Code Equation** under estimates the crack width in the case of all the test results of Hognestad, Clark and Base et al. Also when comparing with all the test results (Table 6.6), Chinese code equation under-estimates the crack width by 38.7% ; W_{cal}/W_{exp} is 0.613 on an average with a coefficient of variation of 43.08%.

6.7 Conclusions

1. In this study, the International Equations for the determination of crack width in reinforced concrete flexural members are compared with the test results of Clark, Hognestad and Base et al. available in literature. From the comparison it is found that the Gergely Lutz equation predicts the width of cracks better when compared to the other equations.

2. It may be noted that, all the equations available in the International Codes are based on certain theories involving strength of materials approach and evaluation of empirical constants from statistical analysis of test results appearing in the equation.

3. Considering the random behaviour of cracking and dynamic propagation of cracks, it is felt that, appropriate methods involving concepts of fracture mechanics have to be developed for estimating the width of cracks in reinforced concrete members.

6.8 Prediction of Spacing and Maximum Width of Cracks in Steel Fibre Reinforced Concrete Flexural Members

6.8.1 Introduction

In the limit state design of reinforced concrete structures cracking is one of the important limit states which the design has to satisfy to ensure serviceability of the structure. Cracking in reinforced concrete members under service load is due to the low tensile strength of concrete. The development of cracks in reinforced concrete members has a major influence on the structural performance, energy absorption capacity, ductility, corrosion resistance of reinforcement, etc. Hence it is necessary to mitigate the cracking in reinforced concrete structures [37]. Recent investigations [6] indicate that addition of steel fibres improves the cracking behaviour of reinforced concrete members significantly.

In the case of fibrous composite material, the steel fibres act as crack arresters that restrict the growth of flaws already existing in the matrix and control them from enlarging. Test results reported by Swamy et al [96] indicate that the ability of the fibre reinforcement to control cracking and deflection is more important than improvement in the strength characteristics. It has been already established [68] that the fibres influence the cracking behaviour of matrix and convert the brittle behaviour into ductile one. In addition to this, the presence of fibres increases the stiffness of matrix, which in turn results in reduction of deflection. Also from the recent investigation, it has been learnt that addition of polymer improves many of the engineering properties of conventional concrete.

Even though a large number of investigations are available in literature on strength and behaviour of steel fibre reinforced concrete (SFRC) members subjected to flexure, only a few attempts have been made to predict the width of cracks in such members. These methods are either purely theoretical or empirical in nature. Since cracking in concrete member is a random phenomenon and subjected to large degree of scatter, a method which represents the physical behaviour of cracking and at the same time take care of this large degree of scatter is the most suitable one.

In this Chapter, attempts have been made first to propose a method for predicting the spacing and width of cracks in the case of steel fibre reinforced concrete flexural members using available results in literature. Then the same method has been extended to predict the spacing and width of cracks in latex modified SFRC flexural members with suitable modification.

Results obtained using the equations available in literature for predicting the crack widths in reinforced concrete flexural members have been compared with the test results [83], to verify the applicability of these equations in predicting the width of cracking in SFRC members. As these equations did not give a satisfactory comparison, an attempt is made to modify the analytical equation for spacing and maximum width of crack in SFRC flexural members with conventional reinforcement.

6.8.2 Test results

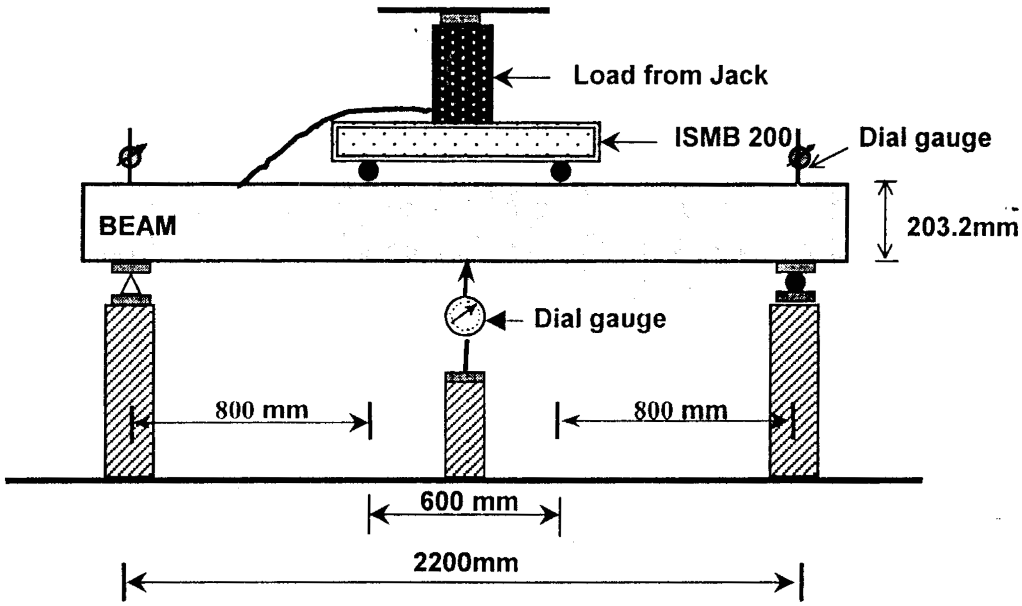
In the literature it has been found that Achyutha and Sabapathi [6] have carried out an extensive study on strength and behaviour of SFRC flexural members. They conducted tests on RCC beams additionally reinforced with steel fibres. The variables considered in their study include (i) area of reinforcement, (ii) compressive strength of concrete, (iii) aspect ratio of fibres, (iv) volume fraction of fibres, and (v) yield strength of steel. A total of 46 beams have been tested in their study. The test results reported by Sabapathi [83] have been used for evaluating the empirical constants appearing in the equation proposed in this study. Out of 46 specimens, those reinforced with steel fibres over the entire cross section only have been considered. The details of the specimen and fibre reinforcement used are given in Table 6.7. The beams were tested under two point loading as shown in Fig. 6.13.

Table 6.7

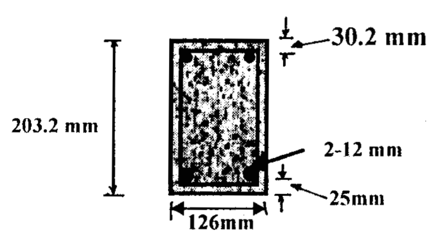
Details of specimens and reinforcement [83]

Sl. No.	Beam No.	Dia of Bars (mm)	No. of bars	A_f	V_f	f_{ck} (N/mm²)	f_y (N/mm²)
1	1	10	2	80	1.6	32.16	375.0
2	2	10	2	80	1.0	34.84	615.0
3	3	10	2	65	1.3	29.82	400.0
4	4	8	2	65	1.3	29.82	386.0
5	7	10	2	50	1.6	32.57	400.0
6	8	10	2	50	1.0	32.18	400.0
7	9	10	2	65	1.3	28.43	580.0
8	10	12	2	65	1.3	28.43	432.0
9	14	10	2	80	1.3	31.17	365.0
10	15	10	2	65	1.6	34.59	402.0
11	16	10	2	65	1.0	36.11	402.0
12	17	10	2	65	1.3	35.82	426.0
13	18	16	2	65	1.3	35.82	510.0
14	21	16	3	50	1.9	36.95	479.0
15	47	16	2	80	1.5	46.65	443.0
16	46	16	3	80	1.5	46.65	443.0

Note: 1. Dimensions of all specimens are 126.0 x 203.2mm
2. Clear cover = 25 mm
3. Span = 2200 mm



Test Setup



Cross Section

Fig. 6.13 Experimental setup and cross section of the beam used [83]

6.8.3 Comparison of International equations

To start with, an attempt is made to compare the results obtained from existing equations available in literature for predicting the maximum crack width in conventionally reinforced concrete members with the test results [83]. The equations given by BS 8110 [12] ACI equation [3] (Gergely Lutz equation), Model code [13] and Desayi - Ganesan [20] have been used to calculate the maximum crack width (W_{cal}) and compared with the experimental values. Details of calculation of maximum crack width are given in the Appendix III. Table 6.8 gives the values of average of W_{cal}/W_{exp} for observations collected from the test data [83].

Table 6.8

Comparison of results from present equations with the test results

Equation		No. of Observations [83]	Ratio of W_{cal}/W_{exp}	
			Average	Coefficient of variation (%)
B.S. 8110	[12]	97	0.808	51.75
Gergely & Lutz	[3]	97	1.342	47.50
Model Code 1990	[13]	97	0.832	50.42
Desayi & Ganesan	[20]	97	1.623	53.19

From Table 6.8, it may be noted that equations of BS 8110 and Model Code 1990 under- estimate the width of crack by 19.2% and 16.8% respectively. On the other hand, equations of Gergely Lutz and Desayi and Ganesan over estimate the width of crack by 34.2% and 62.3% respectively. Also all the equations have given larger values of coefficient of variation of W_{cal}/W_{exp} and do not give a satisfactory comparison. These equations have been developed to estimate the width of cracks in reinforced concrete flexural members only. In the case of reinforced concrete beams with additional steel fibres, the presence of steel fibres affect the cracking behaviour significantly [96]. The effect of steel fibres has not been incorporated in the above equations. This might have resulted in large difference between computed and experimental values of crack width. This indicates that suitable modifications have to be introduced in the above equations to account for the effect of steel fibres for predicting the width of cracks.

6.8.4 Significance of choosing Desayi and Ganesan equation

It may be noted that the equations given by B.S.8110, Model Code 1990 and Gergely and Lutz are generally based on partly on theoretical and partly on the test results and are semi-empirical equations. These equations are obtained using different combinations of variables that were thought to affect the cracking behaviour and which correlate better with the test results. Such equations may not give a physical picture of the cracking behaviour. However a hypothesis for cracking behaviour with assumed behaviour of some of the parameters and evaluation of the same using statistical analysis of the large number of data may give a better understanding of the physical behaviour and at the same time take care of the random behaviour of the cracking. The equations proposed by Desayi and Ganesan [20] is based on the classical bond slip hypothesis and the constants appearing in the equation are evaluated using a large number of test results [20] and direct observation of the equation reveals a clear picture of the cracking behaviour of reinforced cement concrete.

In addition to this, it may be noted that the B.S 8110 and Model Code equation under-estimate the width of cracks in the case of reinforced concrete members with additional steel fibres (Table 6.8). Therefore, the incorporation of stiffening effect of the fibres in the crack width equation will lead to further reduction in the computed values of the crack width. Then the next choice is Geregely and Lutz equation, which is purely empirical and does not represent the physical behaviour of cracking. Therefore attempts were made to modify Desayi and Ganesan equation.

In view of the above, the method proposed by Desayi and Ganesan [20] has been chosen and the same has been modified in the case of Reinforced Cement concrete members with additional steel fibres.

6.8.5 Proposed method

The method proposed in this study follows the one proposed earlier by Desayi and Ganesan [20], for reinforced concrete flexural members which is primarily based on the bond slip hypothesis. In order to account for the effect of fibres, suitable modifications are introduced.

In their method, Desayi and Ganesan [20] considered a concrete member with a reinforcing bar under tension and explained the formation of new crack at section xx (Fig. 6.14) which is at a distance l_1 from an already formed crack. A new crack could form at section xx when the bond force developed by the bar causes a maximum stress equal to the tensile strength of concrete at section xx.

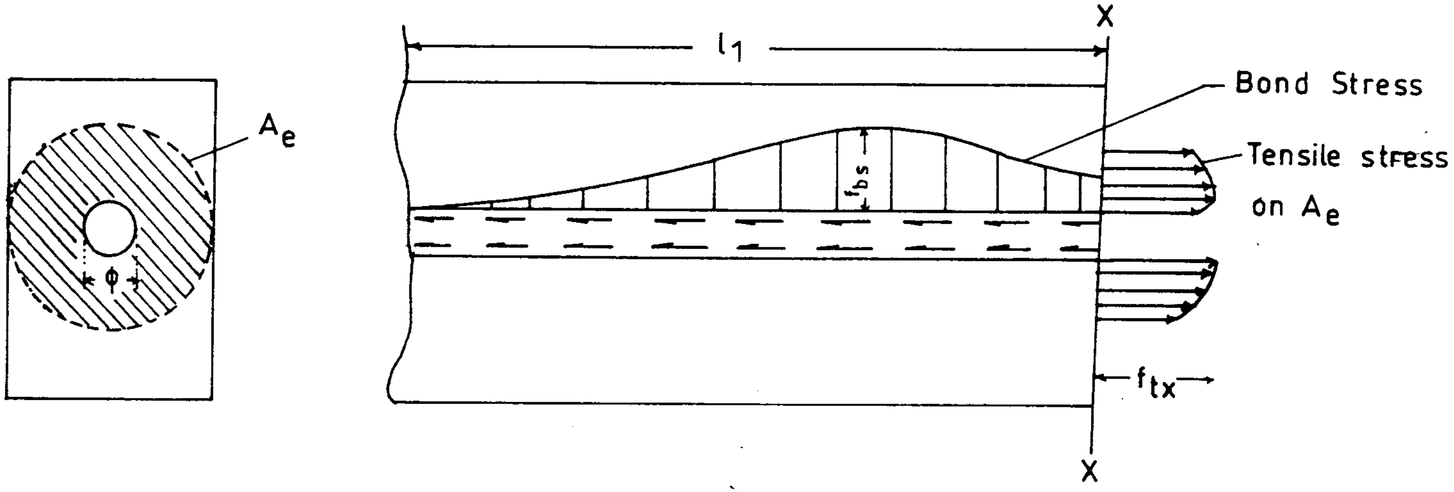


Fig. 6.14 Bond and tensile stress distribution in concrete at a section distance l_1 from a crack formed in a reinforced concrete member[20]

Based on the statistical analysis of a number of test results, they obtained the following equations for spacing and maximum width of cracks in reinforced concrete flexural members.

Average crack spacing a_c :

$$a_c = \frac{k_t \; f_{ct} \; A_e}{k_b \; f_{bu} \left[\dfrac{M}{M_u}\right]^{\gamma} \; \Sigma \pi \phi} \qquad \text{........(6.12)}$$

Maximum crack spacing is :

$$a_m = \frac{k_t \; f_{ct} \; A_e}{k_b \; f_{bu} \left[\dfrac{M_{cr}}{M_u}\right]^{\gamma} \; \Sigma \pi \phi} \qquad \text{.........(6.13)}$$

Maximum crack width at lower extreme fibre W_{bt} :

$$W_{bt} = a_m \; \varepsilon_s \; \frac{(h - x)}{(d - x)} \qquad (6.14)$$

6.8.6 Modifications proposed in this study

It may be noted from equation (6.14) that, for a given beam, all the parameters of equation (6.14) are constant except the strain in steel ε_s which is a variable and depends on the external load. This was obtained based on elastic cracked section theory [20]. Since steel strain is an important parameter for estimating the width of cracks in the case of reinforced concrete members, an attempt is made to compare the values of strain computed based on the cracked section theory with that of experimental values. Strain in steel have been calculated as follows

The stress in steel (f_s) at any moment 'M' after cracking is,

$$f_s = m \frac{M}{I_{cr}}(d - x) \qquad \text{.........(6.15)}$$

The strain in steel ε_s corresponding to the stress f_s is,

$$\varepsilon_s = \frac{f_s}{E_s} \qquad \text{.........(6.16)}$$

$$where \quad m = \frac{E_s}{E_c}$$

The computed values of ε_s from equation (6.15) and (6.16) are compared with the experimental values obtained from the test results [83], and are shown in Fig. 6.15. The experimental values of strain have been obtained for different values of M/M_w ranging from 0.2 to 1.8 where M represents the moment corresponding to a particular stage of loading and M_w represents the working load moment computed [20] assuming the permissible stress in steel is 230 N/mm^2. From this figure it may be noted that, the method used (Eq. 6.15 and 6.16) for calculating the values of strain in tension reinforcement in the case of SFRC members over estimates the value of ε_s . This may be due to the following reasons. In the case of SFRC members, the steel fibres in the tension zone increase the elastic deformation of the material surrounding the main reinforcement. In addition to this the stiffening effect of concrete between cracks at the initial stages is high when compared to that of plain concrete. This results in a significant reduction of strain in steel . The conventional method for computing strain in steel based on cracked section theory over estimates the values of strain in case of SFRC members, as the stiffening effect in between cracks is not taken in to account. Hence a correction factor F is introduced to evaluate the strain in steel in the case of SFRC members. The correction factor F represents the combination of different geometrical and mechanical properties, which were thought to affect the cracking behaviour of the member.

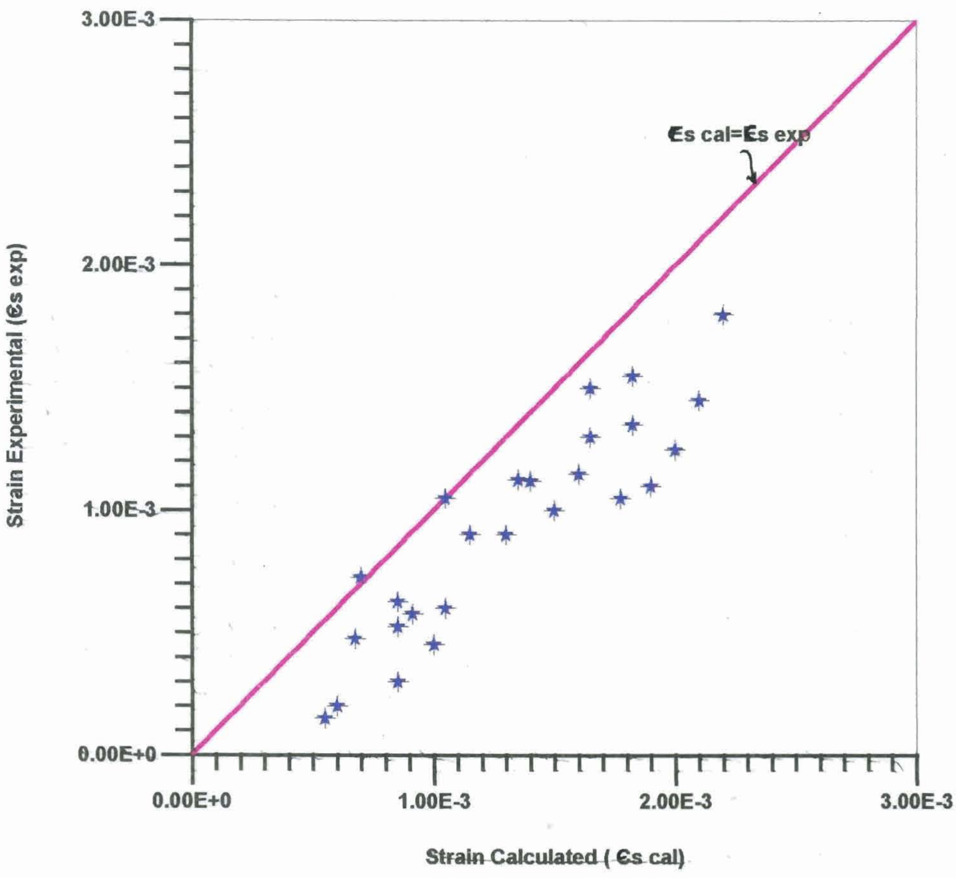


Fig. 6.15 Comparison of calculated strain with experimental strain

Equation (6.17) gives the relation between F and the geometrical and mechanical properties of the member.

$$F = \frac{b(h-x) f_{ct} A_f V_f}{A_s f_s 10^6} \qquad \text{...(6.17)}$$

In equation (6.17) the contribution of matrix is incorporated in the equation in the form of $b(h\text{-}x)f_{ct}$ which represents the tensile force in the matrix. The amount of steel fibres which increases the elastic deformation of the matrix, surrounding the tensile reinforcement, is an important parameter and is represented by the combination of volume fraction and aspect ratio of fibres, V_f and A_p respectively.

As tensile force in steel reinforcement is also another important parameter, it is incorporated in the equation by A_s f_s. It may be noted that the factor F is introduced in order to account for the stiffening effect of steel fibre concrete matrix in between the cracks, which causes significant reduction of strain in steel [89].

The values of ($\varepsilon_{s(cal)}$ - $\varepsilon_{s(exp)}$) are plotted against F and is shown in Fig. 6.16. The best fit line is drawn relating ($\varepsilon_{s(cal)}$ - $\varepsilon_{s(exp)}$) with F and the regression equation thus obtained is:

$$\varepsilon_s(cal) - \varepsilon_s(\exp) = m_l(F) + c_l \qquad \text{.........(6.18)}$$

Where $m_l = 1.414$

$c_l = 2.831 \times 10^{-4}$

As the experimental strain should be equal to the corrected strain, $\varepsilon_{s(exp)}$ is replaced on the LHS by $\varepsilon_{s(cor)}$ and thus,

$$\varepsilon_{s(cor)} = \varepsilon_{s(cal)} - (1.414\ '\ F + 2.837\ '\ 10^{-4}) \qquad \text{...(6.19)}$$

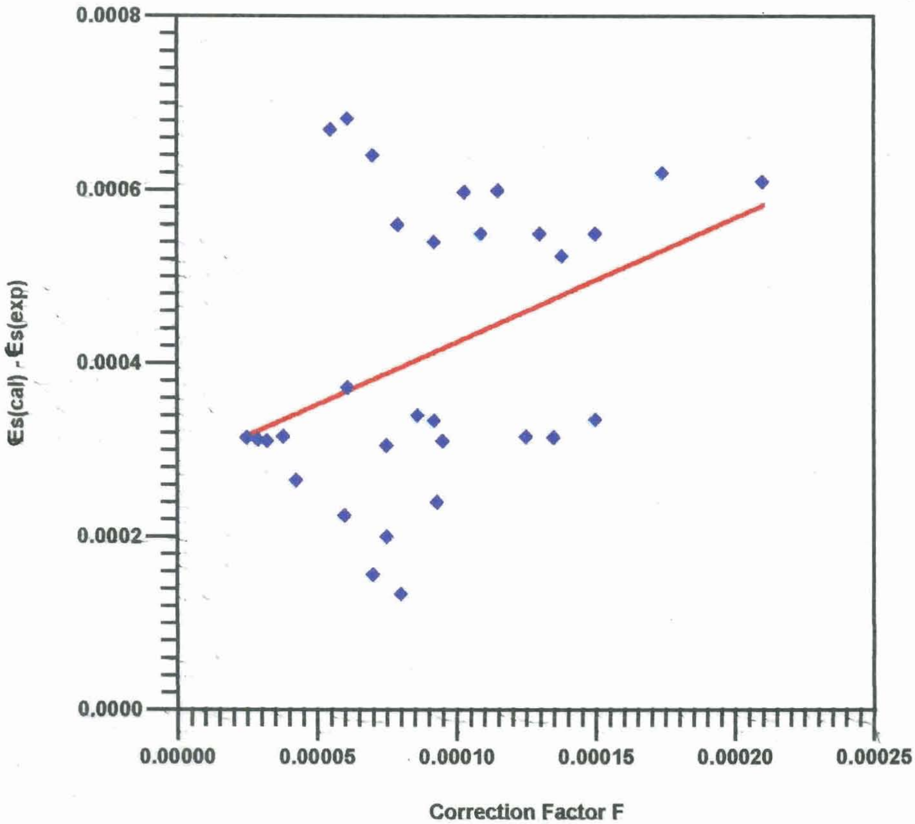


Fig. 6.16 Єs (cal) - Єs(exp) versus Correction Factor F

Fig. 6.17 shows the variation of the corrected strain $e_{s(cor)}$ with the experimental strain. From the figure it can be seen that the equation (6.19) satisfactorily predicts the strain in steel in the case of SFRC members and all the points are lying within ±15% variation from the line of equality.

The equation (6.14) for determining the maximum crack width at the soffit is modified by replacing ε_s by $\varepsilon_{s(cor)}$ and is given as follows:

$$W_{bt} = a_m \, \varepsilon_{s(cor)} \frac{(h - x)}{(d - x)} \qquad \ldots (6.20)$$

Equation (6.20) is used to determine the maximum bottom crack width and these values are compared with the measured width of cracks. Table 6.9 gives the values of W_{cal}/W_{exp} and coefficient of variation. It may be noted that the proposed method under-estimate the values of crack width by 0.8% with a coefficient of variation of 35.30%. Comparing Table 6.8 and 6.9, it is seen that the proposed method estimates the maximum crack width better than the equations available in literature for reinforced concrete flexural members. Fig.6.18 shows the plot relating the values of W_{cal} with W_{exp} and the computed values compare satisfactorily with the experimental values.

Table 6.9

Comparison of Maximum crack width computed using the proposed method with the test results

Equation	No of Observations	Ratio of W_{cal}/W_{exp}	
		Average	Coefficient of variation (%)
Proposed method	97	0.992	35.30

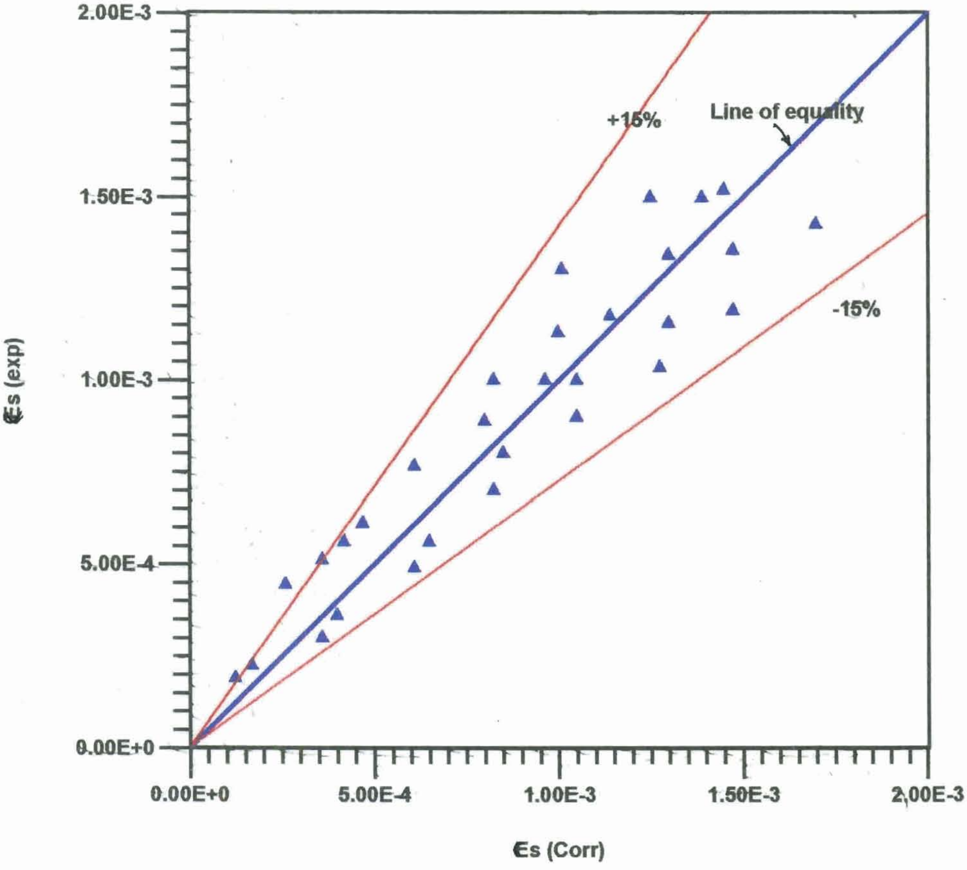


Fig. 6.17 Comparison of corrected strain with the experimental strain

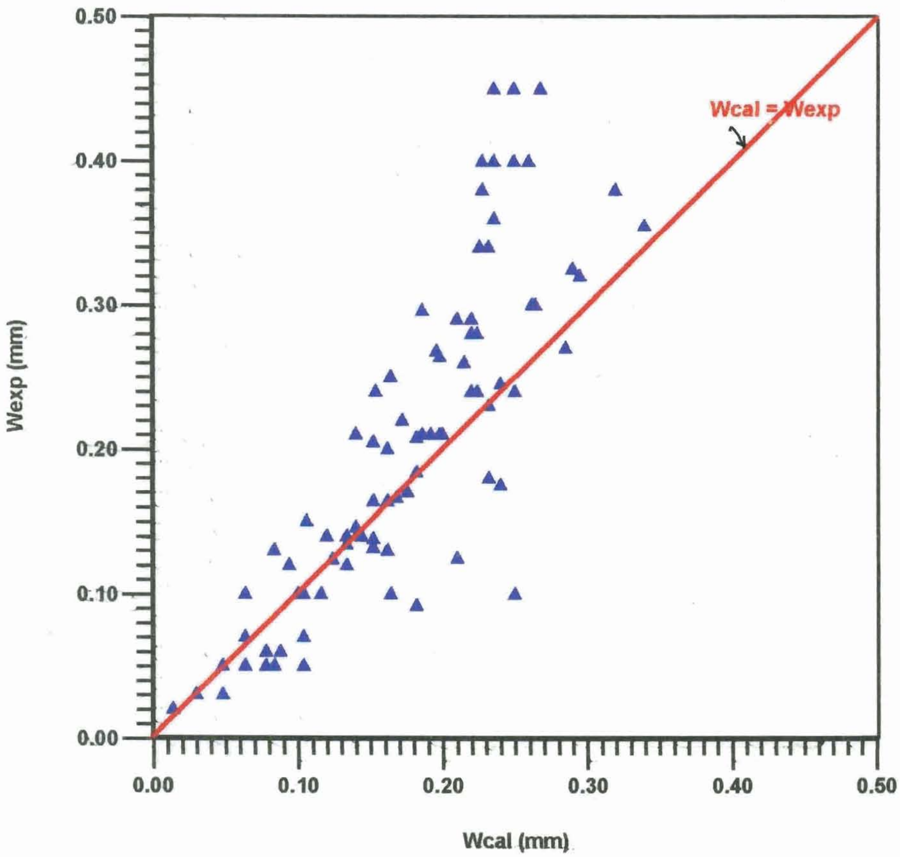


Fig. 6.18 Comparison of calculated Crack width with the experimental Crack width

6.8.7 Spacing of cracks

. The earlier studies on spacing of cracks in the case of steel fibre reinforced concrete indicate that the addition of steel fibres have negligible effect on the crack initiation [89]. The fibres play a vital role only after the cracking of matrix in extreme tensile zone by bridging across the faces of cracks which in turn causes reduction in the widening of cracks and delay the propagation of cracks.

In view of the above, the existing equation for spacing of cracks in the case of plain reinforced cement concrete (without SFRC) has been used to compute the crack spacing. The computed values of spacing of cracks using equation (6.12) and (6.13) have been compared with the experimental values. Fig. 6.19 shows the plot of computed and experimental values of spacing of cracks. From the figure it may be noted that the points lie around the line of equality and this indicate that the equations (6.12) and (6.13) predict the spacing of cracking in SFRC members satisfactorily. Table 6.10 and 6.11 give the details of values of ratio of computed mean spacing to the experimental mean spacing and the values of ratio of computed maximum spacing. The average values of $\frac{\text{spacing}_{(cal)}}{\text{spacing}_{(exp)}}$ values are 0.93 and 0.96. The coefficient of variation of the $\frac{\text{spacing}_{(cal)}}{\text{spacing}_{(exp)}}$ are 29.22% and 18.12%. The above values indicate that the comparison is satisfactory.

Fig. (6.20 a) to (6.20 d) shows the plot of spacing of cracks at different stages of loading with steel stress. It is seen from the figure that the measured and calculated values of spacings compare satisfactorily except at the initial stages of loading. This discrepancy may be due to the reason that there is a possibility that one or two hairline cracks would have been missed during the visual observation at the initial stages of cracking which leads to higher values of experimental spacing.

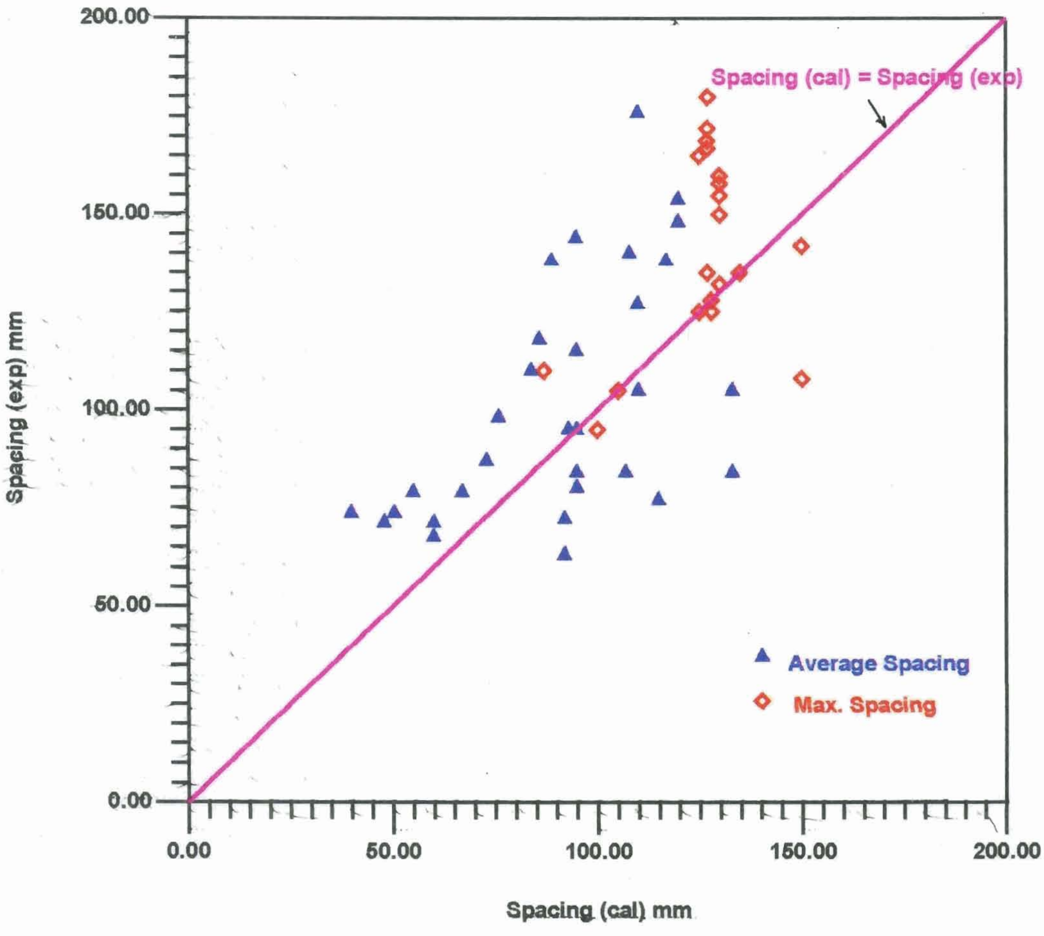


Fig. 6.19 Comparison of calculated crack spacing with experimental crack spacing

Table 6.10

Comparison of calculated and experimental mean spacing of cracks [83]

Beam No.	Spacing (cal) (mm)	Spacing (exp) (mm)	Ratio $\frac{Spacing_{(cal)}}{Spacing_{(exp)}}$
1	114.70	75.60	1.52
1	96.67	84.20	1.15
2	132.63	84.70	1.56
2	94.00	71.00	1.32
3	109.18	139.00	0.78
3	89.96	139.00	0.64
7	93.56	144.30	0.64
9	119.41	154.00	0.77
9	86.38	117.80	0.73
14	109.65	88.00	1.24
14	93.28	61.10	1.52
15	97.23	142.50	0.68
16	117.60	138.30	0.85
16	96.72	79.00	1.22
17	120.46	147.50	0.81
17	97.08	115.00	0.84
4	134.14	104.00	1.28
4	111.91	104.00	1.07
10	92.89	129.00	0.72
10	74.50	86.20	0.86
18	83.38	109.6	0.76
18	63.10	67.00	0.94
8	111.27	176.00	0.63
8	91.67	111.20	0.82
45	77.56	97.50	1.00
45	77.57	97.50	0.79
21	53.69	73.10	0.73
21	41.53	73.10	0.56
46	63.63	70.60	0.90
46	50.59	70.60	0.71
13	111.63	125.30	0.89
13	94.96	94.30	1.00
23	68.49	78.10	0.87
23	56.68	78.10	0.72

Average = 0.93

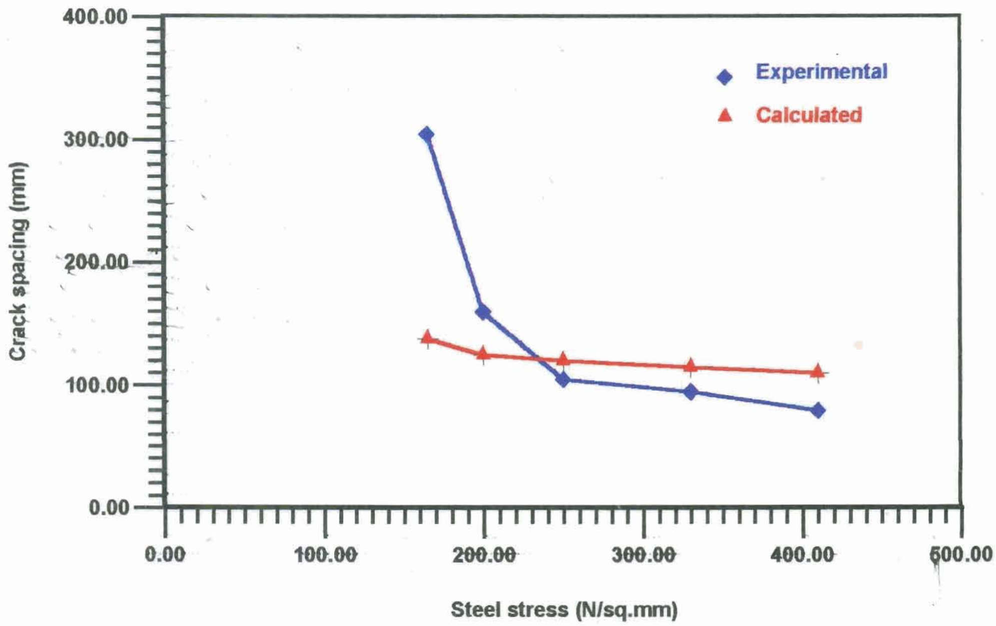
COV = 29.22

Table 6.11

Comparison of calculated and experimental maximum spacing of cracks [83]

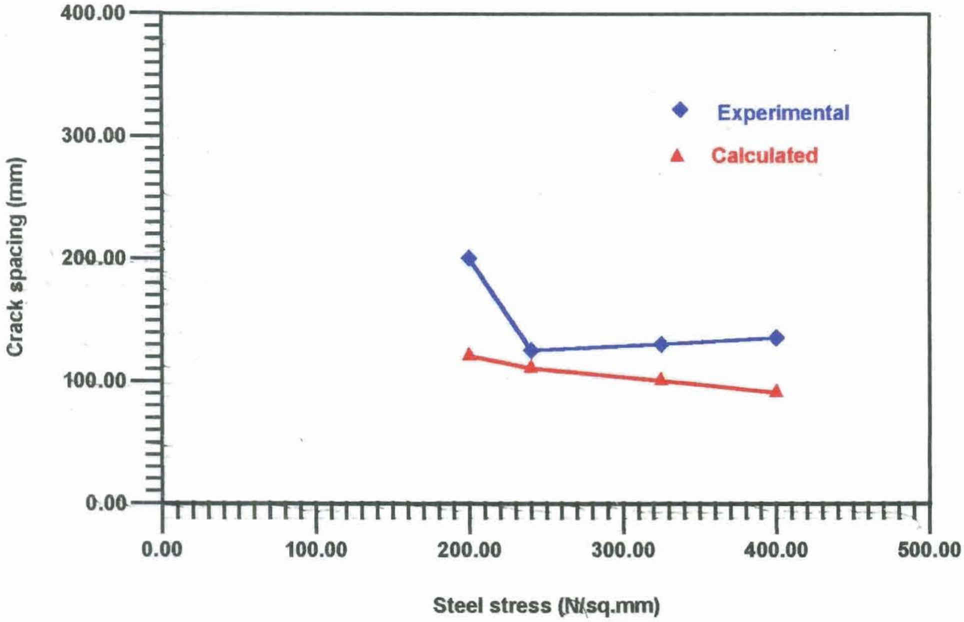
Beam No.	Spacing (cal) (mm)	Spacing (exp) (mm)	Ratio $\frac{Spacing_{(cal)}}{Spacing_{(exp)}}$
1	134.28	128.0	1.04
1	134.28	128.0	1.04
2	153.89	144.0	1.06
2	153.89	107.0	1.04
3	129.88	175.0	0.74
3	129.88	175.0	0.74
9	143.13	147.0	0.97
14	129.52	170.0	0.76
14	129.52	170.0	0.76
15	137.32	150.0	0.91
16	135.67	162.0	0.83
17	139.15	150.0	0.92
17	139.15	165.0	0.84
4	138.29	138.0	1.00
4	138.29	138.0	1.00
18	128.50	165.0	0.77
18	128.50	100.0	1.28
8	130.76	137.0	0.95
45	144.58	135.0	1.07
45	144.58	135.0	1.07
21	92.68	110.0	0.84
46	106.34	95.00	1.11
46	106.34	95.00	1.11
13	130.16	176.0	0.73
13	130.16	167.0	0.77
23	110.22	105.0	1.04
23	110.22	105.0	1.04

Average = **0.96**
COV = **18.12**



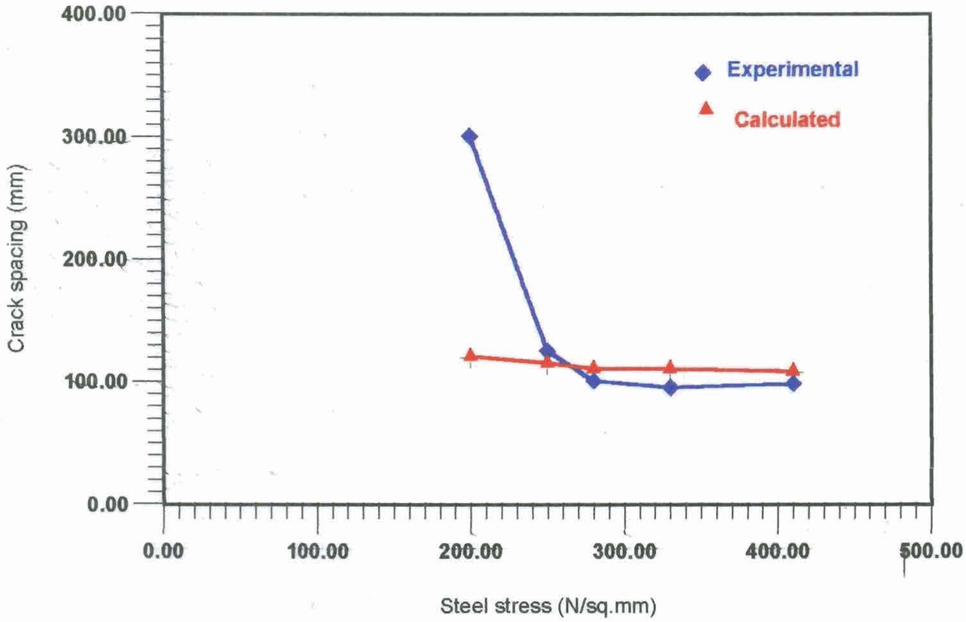
BEAM No. 1

Fig. 6.20 (a) Variation of crack spacing with calculated steel stress



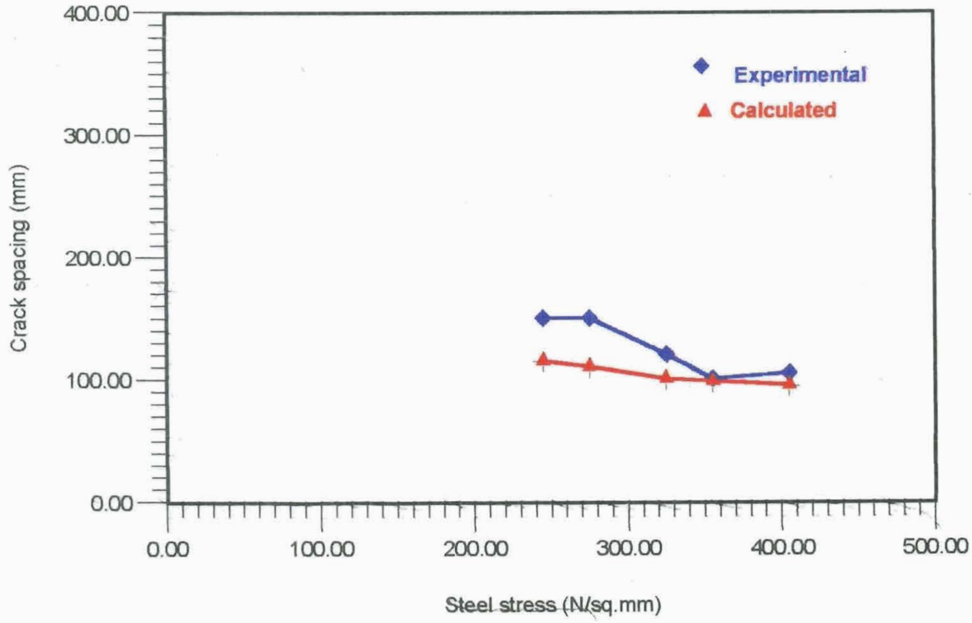
BEAM No. 3

Fig. 6.20 (b) Variation of crack spacing with calculated steel stress



BEAM No. 13

Fig. 6.20 (c) Variation of crack spacing with calculated steel stress



BEAM No. 15

Fig. 6.20 (d) Variation of crack spacing with calculated steel stress

6.8.7 Spacing of cracks

The earlier studies on spacing of cracks in the case of steel fibre reinforced concrete indicate that the addition of steel fibres have negligible effect on the crack initiation [89]. The fibres play a vital role only after the cracking of matrix in extreme tensile zone by bridging across the faces of cracks which in turn causes reduction in the widening of cracks and delay the propagation of cracks.

In view of the above, the existing equation for spacing of cracks in the case of plain reinforced cement concrete (without SFRC) has been used to compute the crack spacing. The computed values of spacing of cracks using equation (6.12) and (6.13) have been compared with the experimental values. Fig. 6.19 shows the plot of computed and experimental values of spacing of cracks. From the figure it may be noted that the points lie around the line of equality and this indicate that the equations (6.12) and (6.13) predict the spacing of cracking in SFRC members satisfactorily. Table 6.10 and 6.11 give the details of values of ratio of computed mean spacing to the experimental mean spacing and the values of ratio of computed maximum spacing. The average values of $\frac{\text{spacing}_{(cal)}}{\text{spacing}_{(exp)}}$ values are 0.93 and 0.96. The coefficient of variation of the $\frac{\text{spacing}_{(cal)}}{\text{spacing}_{(exp)}}$ are 29.22% and 18.12%. The above values indicate that the comparison is satisfactory.

Fig. (6.20 a) to (6.20 d) shows the plot of spacing of cracks at different stages of loading with steel stress. It is seen from the figure that the measured and calculated values of spacings compare satisfactorily except at the initial stages of loading. This discrepancy may be due to the reason that there is a possibility that one or two hairline cracks would have been missed during the visual observation at the initial stages of cracking which leads to higher values of experimental spacing.

6.8.8 Conclusions

In this study a method is proposed for predicting the spacing and maximum width of cracks in conventionally reinforced concrete flexural members with steel fibres. The values of spacing and width of cracks computed using the proposed method compare satisfactorily with the test results available in literature.

As cracking is a random phenomenon and subjected to a large degree of scatter, more number of test results are required to obtain a statistically best fit equation for predicting the spacing and maximum width of cracks in reinforced steel fibre concrete beams

The above investigation indicated that addition of steel fibres to conventional reinforced concrete flexural members improve the cracking behaviour. Similarly the studies available in literature on polymer modified concretes also show that the polymers play a vital role in delaying the cracking phenomenon. However no attempts on the combined effect of steel fibres and polymers on the cracking behaviour of conventional reinforced concrete members have been come across in literature. Hence an attempt is made in this direction to fill the gap in the existing knowledge.

6.9 Prediction of Spacing and Maximum Width of Cracks in Latex Modified Steel Fibre Reinforced Concrete Flexural Members

6.9.1 Introduction

As it has been already known that combined effect of steel fibres and latex improve the strength, ductility, energy absorption capacity of conventional concrete, an attempt is also made to extend the proposed method for steel fibre reinforced concrete members to predict the spacing and maximum width of cracks in latex modified steel fibre reinforced concrete flexural members. The details of experimental programme carried out were already presented in Chapter 4.

6.9.2 Modification proposed in this study

In this section an attempt is made to modify the method given in the previous section (6.8) for estimating the width and spacing of cracks of steel fibre reinforced concrete flexural members so that the same can be extended to the latex modified steel fibre reinforced concrete flexural members.

Referring to equation (6.12) to (6.14), it may be noted that the only variable in the equation is ε_s. If the computed values of ε_s compare with the experimental values of strain in steel, modification is not required. To verify this, the computed values of ε_s have to be compared with the experimental values. Already details of experimental programme carried out and the measurement of strain at the level of steel has been explained in Chapter -4. The test results obtained were used in this sections for comparison purposes.

Fig. 6.21 shows the plot relating to the experimental values of strain at the level of steel with the computed values of strain ε_s. From the plot, it may be noted that, the equation for computing the values of strain (eq. 6.16) over estimate the strains.

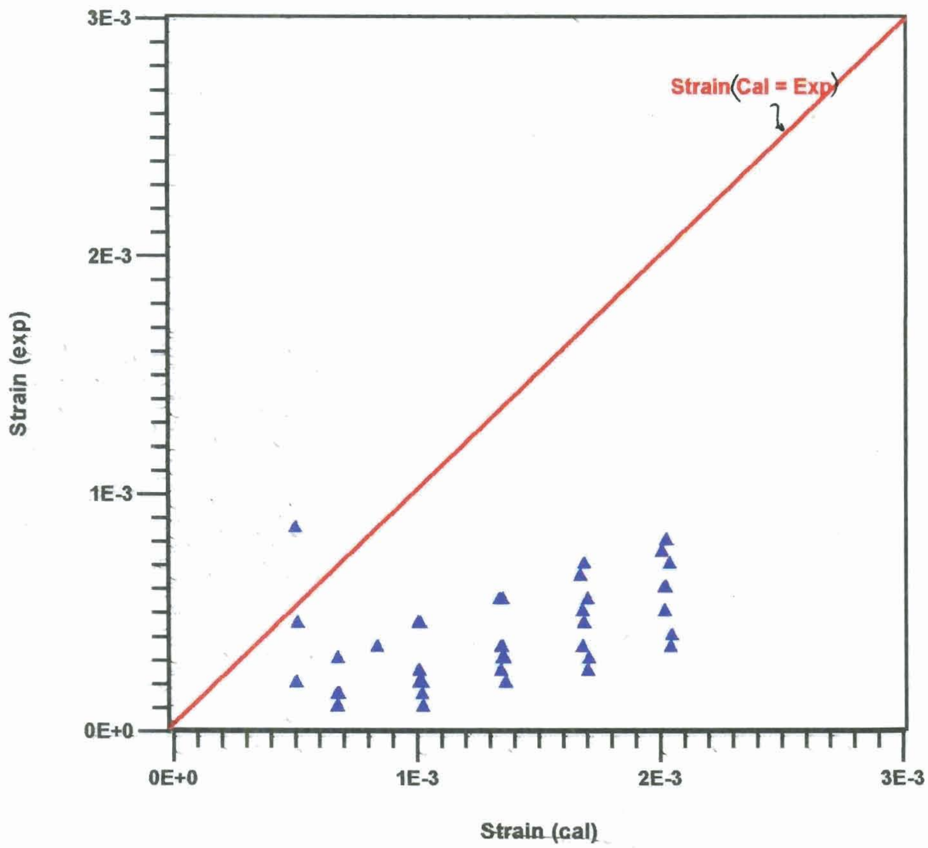


Fig. 6.21 Comparison of Experimental strain with calculated strain

Hence ε_s need to be modified. The reasons for this discrepancy is due to the addition of latex and steel fibres, which impart ductility to concrete, which is one of the outcome of the earlier Chapters (Chapter 4) resulting in higher experimental values.

The computed values of ε_s is corrected as follows :

In order to correct the computed values of ε_s, a correction factor (dimensionless) (F) which takes in to account different geometrical and mechanical properties which were thought to affect the cracking behaviour of the member was introduced. Equation (6.21) gives the relation between F and the geometrical and mechanical properties of the member.

$$F = \frac{b(h-x)\,f_{ct}\,A_f\,V_f L_e}{A_s\,f_s\,10^6} \qquad \text{... (6.21)}$$

where $$L_e = 1 + \frac{1}{1.5 * DRC} \qquad \text{.....(6.22)}$$

In the above equation the contribution of matrix is incorporated in the form of $b(h\text{-}x)f_{ct}$ which represents the tensile force in the matrix. The amount of steel fibres which increases the elastic deformation of the matrix, surrounding the tensile reinforcement, is an important parameter and is represented by the combination of volume fraction and aspect ratio of fibres, V_f and A_p respectively. The presence of latex is represented by the L_e. As tensile force in steel reinforcement is also another important parameter, it is incorporated in the equation by A_s f_s. It may be noted that the factor F is introduced in order to account for the stiffening effect of latex modified steel fibre reinforced concrete matrix in between the cracks, which causes significant reduction of strain in steel .

The values of $(\varepsilon_{s(cal)} - \varepsilon_{s(exp)})$ are plotted against the correction factor (F) and is shown in figure 6.22. The best fit line is drawn relating $(\varepsilon_{s(cal)} - \varepsilon_{s(exp)})$ with F and the regression equation thus obtained is:

Best fit equation : $1400 F^2 - 5.0 F + 5.5 \times 10^{-4}$

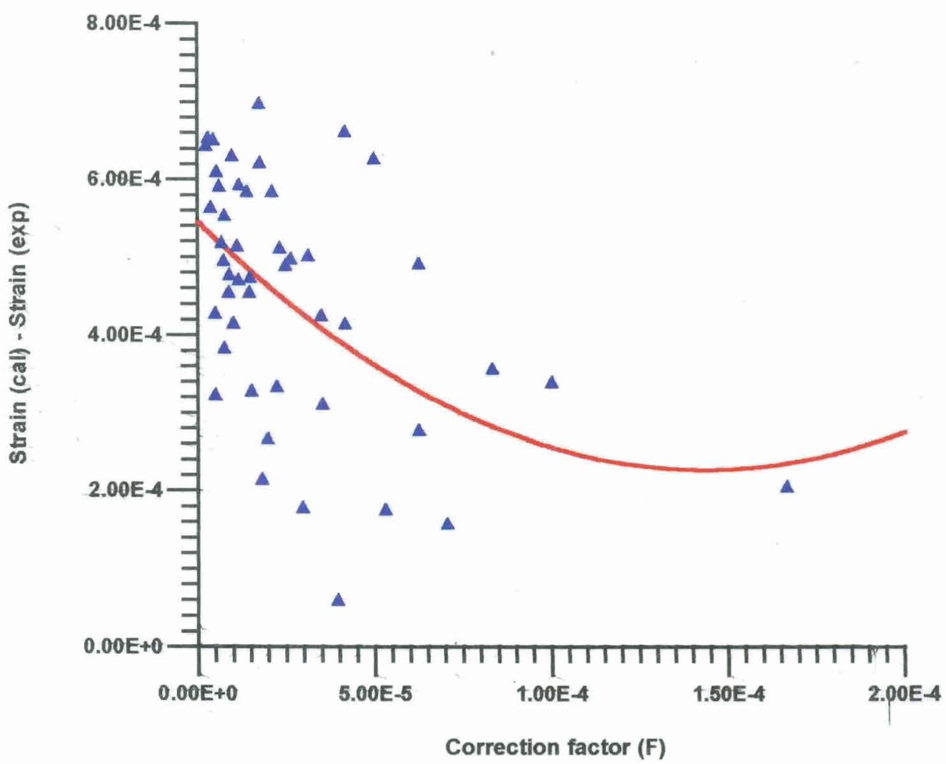


Fig. 6.22 Plot of [Strain (cal) - Strain (exp)] versus Correction factor (F)

$$\varepsilon_{s(cal)} - \varepsilon_{s(exp)} = 1400\ F^2 - 5.0\,F + 5.5 \times 10^{-4} \qquad \ldots.(6.23)$$

As the experimental strain should be equal to the corrected strain, $\varepsilon_{s(exp)}$ is replaced on the LHS by $\varepsilon_{s(cor)}$ and thus,

$$\varepsilon_{s(cor)} = \varepsilon_{s(cal)} - (1400\ F^2 - 5.0\,F + 5.5 \times 10^{-4}) \qquad \ldots\ldots\ldots(6.24)$$

Fig. 6.23 shows the variation of the corrected strain $\varepsilon_{s(cor)}$ with the experimental strain. From the figure it can be seen that the equation (6.24) satisfactorily predicts the strain in steel in the case of SFRC members.

The equation (6.14) for determining the maximum crack width at the soffit is modified by replacing ε_s by $\varepsilon_{s(cor)}$ and is given as follows:

$$W_{bt} = a_m\,\varepsilon_{s(cor)}\frac{(h-x)}{(d-x)} \qquad \ldots(6.25)$$

Equation (6.25) is used to determine the maximum bottom crack width and these values are compared with the measured width of cracks. Table 6.12 gives the values of W_{cal}/W_{exp} and coefficient of variation. It may be noted that the proposed method under-estimate the values of crack width by 13 % with a coefficient of variation of 19.20%. Fig.6.24 shows the plot relating the values of W_{cal} with W_{exp} and the computed values compare satisfactorily with the experimental values.

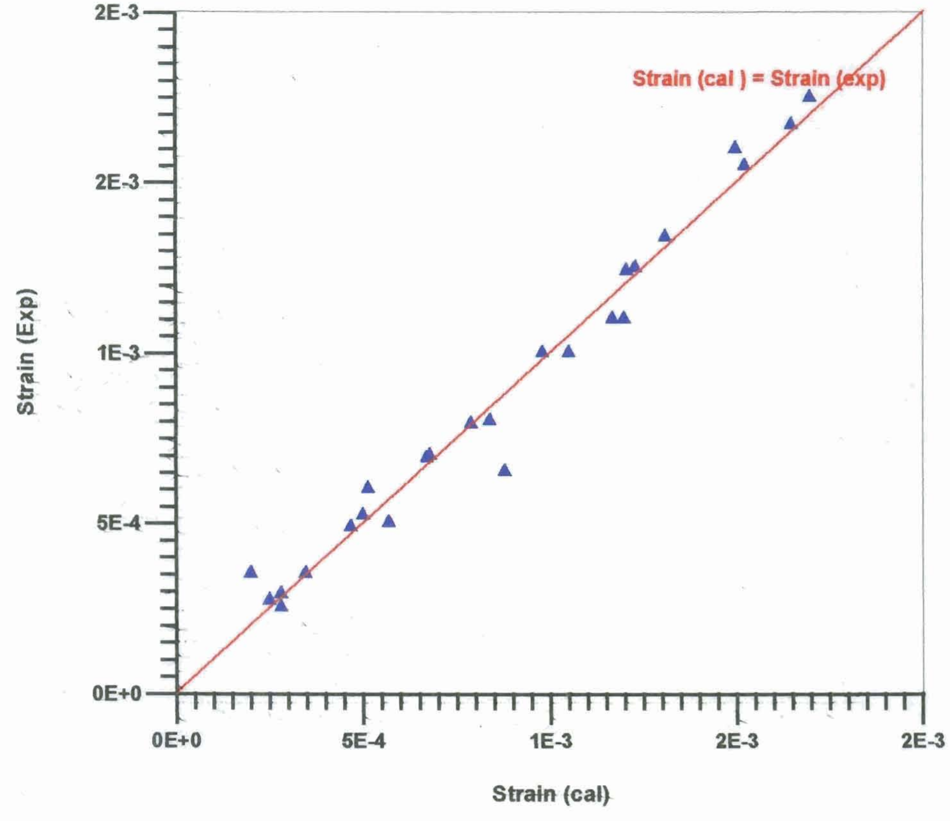


Fig. 6.23 Comaprison of Experimental strain with Calculated strain (after modification)

Table 6. 12

**Comparison of Maximum crack width computed using
The proposed method with the test results**

Equation	No. of Observations	Ratio of [illegible]	
		Average	Coefficient of Variation (%)
Proposed method	47	0.87	19.20

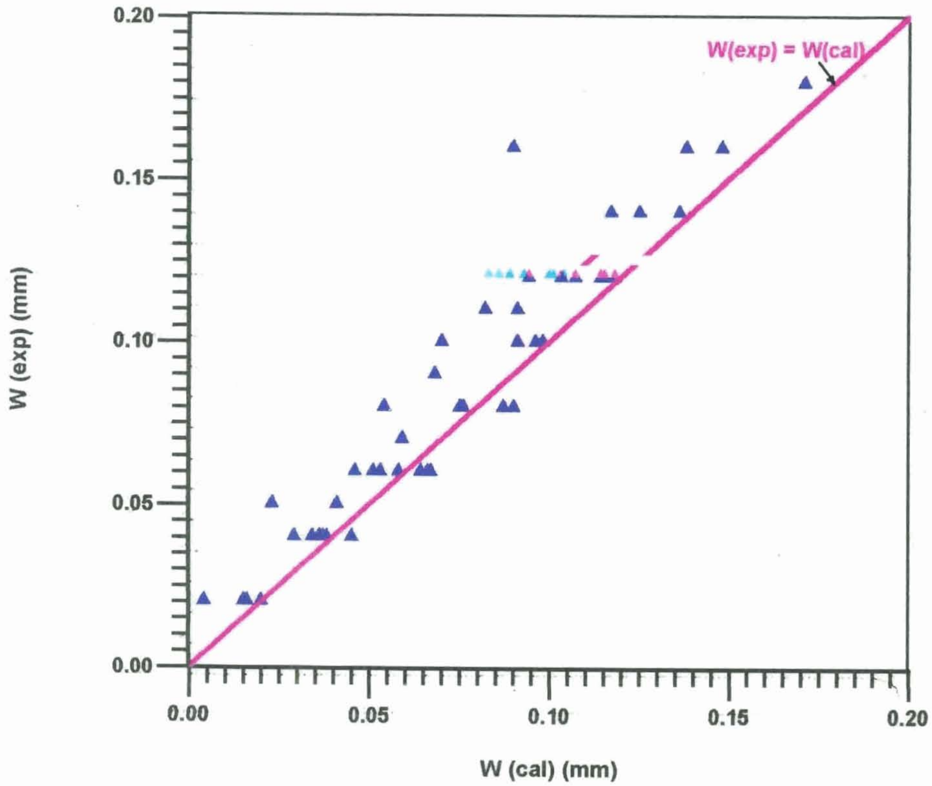


Fig. 6.24 Comparison of W(exp) with W(cal) after modification

6.9.3 Spacing of cracks

The earlier studies on spacing of cracks in the case of steel fibre reinforced concrete indicate that the addition of steel fibres have negligible effect on the crack initiation [89]. The fibres play a vital role only after the cracking of matrix in extreme tensile zone by bridging across the faces of cracks which in turn causes reduction in the widening of cracks and delay the propagation of cracks. Also the addition of latex improve the strength, ductility, energy absorption capacity of conventional concrete.

In view of the above, the existing equation for spacing of cracks in the case of plain reinforced cement concrete (without steel fibres and latex) has been used to compute the crack spacing. The computed values of average spacing of cracks using equation (6.12) have been compared with the experimental values. Table 6.13 gives the average values of Spacing of cracks $_{(cal)}$ / Spacing of cracks $_{(exp)}$ and coefficient of variation. It may be noted that the proposed equation under- estimates the values of spacing of crack by 6 % with a coefficient of variation of 29.25 %. Fig. 6.25 shows the plot of computed and experimental values of average spacing of cracks. Details of the comparison of spacing of cracks are given in Table 6.14. From the figure it may be noted that the points lie around the line of equality and this indicate that the equation (6.12) predict the average spacing of cracking in latex modified SFRC flexural members satisfactorily.

Table 6. 13

Comparison of Spacing of cracks computed with the test results

Equation	No. of Observations	Ratio of $\frac{spacing_{(cal)}}{spacing_{(exp)}}$	
		Average	Coefficient of variation (%)
Proposed method	47	0.94	29.25

Fig. (6.26) and (6.27) show the plot of spacing of cracks at different stages of loading with steel stress. It is seen from the figure that the measured and calculated values of spacings compare satisfactorily except at the initial stages of loading. This discrepancy may be due to the same reason as explained in the earlier sections (Section 6.8.7).

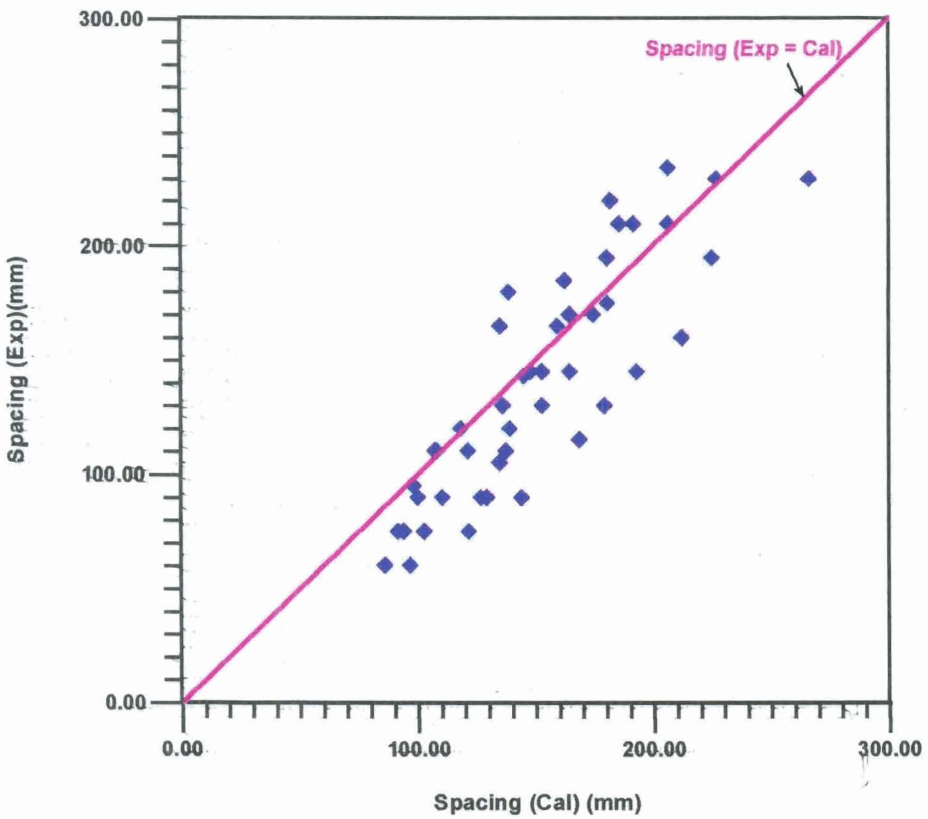
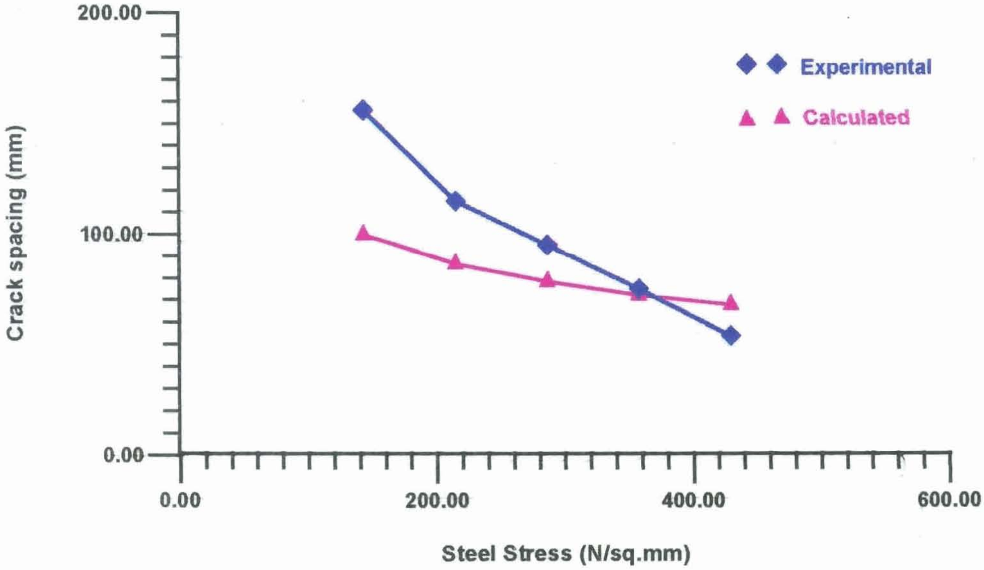
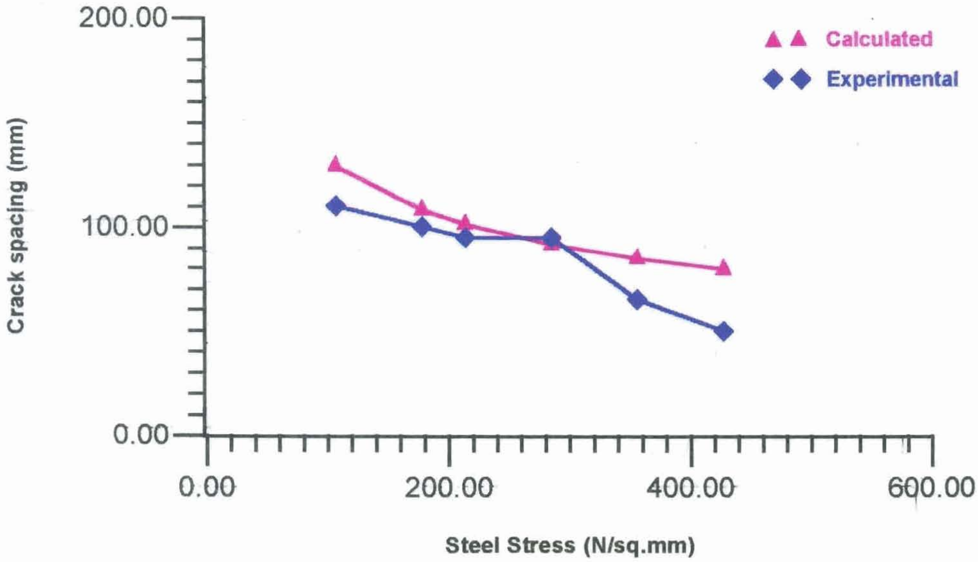


Fig. 6.25 Comparison of Computed Crack spacing with Experimental Crack spacing



Beam No. 8

Fig. 6.26 Variation of crack sapcing with calculated steel stress



(Beam No. 14)

Fig. 6.27 Variation of crack spacing with calculated steel stress

Table 6.14

Details of Calculated & Experimental Crack Spacing of Latex Modified Steel Fibre Reinforced Concrete Flexural Members

Beam No.	Load (KN)	Spacing (cal) mm	Spacing (exp) mm	Ratio (cal/exp)
8	19.62	99.51	156.00	0.64
8	29.43	86.34	115.00	0.75
8	39.24	78.07	95.00	0.82
8	49.05	72.21	75.00	0.96
8	58.86	67.74	54.00	1.25
9	19.62	87.67	110.00	0.80
9	29.43	76.07	103.00	0.74
9	39.24	68.78	85.00	0.81
9	49.05	63.61	68.00	0.94
9	58.86	59.68	46.00	1.30
10	19.62	66.68	115.00	0.58
10	29.43	57.86	96.00	0.60
10	39.24	52.32	73.00	0.72
10	49.05	48.39	54.00	0.90
10	58.86	45.40	42.00	1.08
11	19.62	89.47	110.00	0.81
11	29.43	77.63	90.00	0.86
11	39.24	70.20	74.00	0.95
11	49.05	64.92	57.00	1.14
12	19.62	64.88	152.00	0.43
12	29.43	56.29	123.00	0.46
12	39.24	50.90	96.00	0.53
12	49.05	47.08	65.00	0.72
12	58.86	44.17	59.00	0.75
13	14.72	65.65	123.00	0.53
13	29.43	51.51	96.00	0.54
13	39.24	46.58	65.00	0.72
13	49.05	43.08	45.00	0.96
13	58.86	40.41	35.00	1.15
14	14.72	129.15	110.00	1.17
14	24.53	108.00	100.00	1.08
14	29.43	101.33	95.00	1.07
14	39.24	91.62	95.00	0.96
14	49.05	84.74	65.00	1.30
14	58.86	79.50	50.00	1.59
15	14.72	110.02	110.00	1.00
15	19.62	99.48	90.00	1.11
15	29.43	86.32	83.00	1.04
15	39.24	78.05	76.00	1.03
15	49.05	72.19	53.00	1.36
15	58.86	67.72	42.00	1.61
16	14.72	92.90	103.00	0.90
16	19.62	84.00	80.00	1.05
16	29.43	72.89	65.00	1.12
16	39.24	65.91	60.00	1.10
16	49.05	60.95	58.00	1.05

NO OF TEST RESULTS = 47
AVERAGE 0F (Cal / Exp) = 0.94
STANDARD DEVIATION = 0.28
COEFF. OF VARIATION = 29.25

6.9.4 Conclusions

In this study, the method proposed earlier for the predicting the spacing and maximum width of cracks in conventionally reinforced concrete flexural members with steel fibres has been extended to latex modified SFRC flexural members. The values of average spacing and maximum width of cracks computed using the proposed method compare satisfactorily with the test results.

CHAPTER - 7

CONCLUSIONS

An experimental investigation was carried out to study the combined effect of natural rubber latex as polymer and steel fibres on the strength and behaviour of conventionally reinforced concrete flexural members with and without confinement in the compression zone. The conclusions drawn from the above study are summarised in this Chapter.

7.1 Preliminary Studies on Latex Modified Concrete

An experimental programme was carried out to study the effect of natural rubber latex as polymer on the strength and behaviour of conventional concrete under compression and flexure. This preliminary investigation was restricted to small specimens like cubes, cylinders and prisms, in order to investigate the behaviour of cement concrete when polymer like natural rubber latex was added to it. The findings of this preliminary investigation was made use of in interpreting the behaviour of prototype structural elements like beams when latex modified concrete is used in it.

From the preliminary experimental investigation, following conclusions are arrived at:

1. The addition of small quantities of DRC (0.5%) <u>marginally</u> improves the compressive and flexural strength of plain concrete. Higher values of DRC causes drastic reduction in the strength of concrete.

2. The strain at peak load which is one of the important properties to be considered in the design of seismic resistant/ blast resistant/ cyclically or repeatedly loaded structures improves <u>significantly</u> (by two folds) with the addition of DRC.

3. The energy absorption capacity of the material enhances markedly with the addition of DRC within the range of 0.5% to 1.0%.

4. The foresaid properties of plain concrete namely strength, strain at peak load and energy absorption capacity have been found to improve with the addition of DRC within the range of 0.5% to 1.0%. This indicates that this range seems to be an optimum value of DRC.

5. The scanning electron microscopic studies reinforce the above findings and indicate that, at smaller percentages of DRC, density and hence the compressive strength and flexural strength of composites increases.

6. This preliminary investigation was restricted to small specimens like cubes, cylinders and prisms in order to investigate the behaviour of cement concrete when polymers like natural rubber latex was added to it. The findings of the preliminary investigation will be useful later in interpreting the behaviour of prototype structural elements like beams when latex modified concrete is used in it.

7. The strength of concrete increases as the volumetric ratio of confinement increases. This increase is further improved by the addition of lower values of DRC (1%).

8. At higher values of DRC, infact, a reduction in strength is noticed for a given confinement.

9. The reduction in strength due to higher percentages of DRC can be appreciably reduced by providing higher volumetric ratio of confinement ρ_s.

10. From the study, DRC up to a value of 1% appears to be a useful value in the case of latex modified confined concrete.

7.2 Latex Modified Steel Fibre Reinforced Concrete Flexural Members

An experimental investigation was carried out to study the effect of latex modification and inclusion of steel fibres on the first crack load, ultimate moment of resistance, toughness and ductility characteristics of conventionally reinforced concrete flexural members.

Based on the experimental and analytical studies, the following conclusions are arrived at:

1. In general, addition of latex (0.5 to 1.0% DRC) improve the first crack load and the ultimate strength of flexural members.

2. The addition of steel fibres to latex modified concrete flexural members improves the cracking behaviour significantly. However, the load carrying capacity is only marginally improved.

3. The addition of latex improves the toughness, energy absorption capacity and ductility factor significantly. This is more pronounced in the case of beams with 0.5% DRC and steel fibres up to a volume fraction of 1.0%

4. The method proposed in this investigation predicts the first crack load and ultimate moment resistance of latex modified steel fibre reinforced concrete flexural members satisfactorily.

5. The overall improvement in ductility, toughness index and energy absorption capacity achieved due to the addition of latex and steel

fibres to conventionally reinforced concrete flexural members indicate that the latex modified steel fibre reinforced concrete is an appropriate material in the case of structures which are subjected to large deformations, cyclic loading etc.

6. Load factor with respect to limit state of deflection controls the design of the latex modified steel fibre reinforced concrete beams when compared to those with respect to the limit state of collapse against flexure and the limit state of cracking.

7.3 Latex Modified Reinforced Concrete Beams with Confined SFRC in the Compression Zone

An attempt was made to study the combined effect of the three components i.e, i) addition of latex, ii) fibres and iii) confining the compression zone of flexural members, on the strength and ductility of reinforced concrete flexural members.

Based on the experimental and analytical studies conducted, the following conclusions are arrived at:

1. The provision of confined SFRC in the compression zone of polymer modified reinforced concrete beams, in general, increases the load carrying capacity and the ductility of the specimens. This is more predominant in the case of ductility of the specimens.

2. The investigation indicate that the optimum dosage of DRC is 1.0%. Beyond this limit, there is infact, reduction in the load carrying capacity was noticed. For specimens with 3.0% DRC the reduction in strength is drastic.

3. Load versus deflection plots indicates that there is considerable improvement in the ductility of the specimens upto 1.0% DRC. Also in the case of beams with confined SFRC alone (no polymer modification) this improvement is marginal. Hence it can be said that the addition of latex (to the optimum level) contributes to the ductility of the specimens in a major way.

4. The energy absorption capacity and the toughness index of the specimens increases upto certain level and then decrease as the DRC increases.

5. The latex modification (upto 1% DRC), the incorporation of confinement and the addition of steel fibres in the compression zone have enhanced the strength and ductile behaviour. This indicates that these parameters can be introduced in converting the brittle behaviour of over reinforced concrete flexural members into a ductile one. Hence the maximum longitudinal reinforcement ratio for the flexural members prescribed by the Code of practice could be raised to increase the flexural capacity of beams. This would be beneficial in situations where there is a restriction on the overall depth of beams, particularly if the beams are subjected to large bending moments. Use of over-reinforced beams with polymer modification and confined SFRC in the compression zone can be considered in this case as alternative to the use of prestressed concrete construction.

6. Latex modification, the incorporation of confinement and the addition of steel fibres in the compression zone, enhances the strength and ductility of RCC beams and hence seems to be appropriate for seismic resistant structures.

7. Load factor with respect to the limit state of deflection controls the design of the Latex modified RC beams with confined SFRC in the compression zone, when compared to those with respect to the Limit State of collapse against flexure and the Limit state of Cracking.

7.4 Studies on Cracking of Latex Modified Steel Fibre Reinforced Concrete Flexural Members

An attempt was made to compare the methods adopted in the International Codes of Practice for predicting the maximum width of cracks using the test results reported in literature. Also, attempt has been made to propose a method for predicting the spacing and width of cracks in the case of steel fibre reinforced concrete flexural members using available results in literature. Then the same method has been extended to predict the spacing and width of cracks in latex modified SFRC flexural members with suitable modification.

Based on these studies the following conclusions are arrived at

1. The International Equations for the determination of crack width in reinforced concrete flexural members are compared with the test results of Clark, Hognestad and Base et al. available in literature. From the comparison it was found that the Gergely Lutz equation predicts the width of cracks better when compared to the other equations.

2. All the equations available in the International Codes are based on certain theories involving strength of materials approach and evaluation of empirical constants from statistical analysis of test results appearing in the equation. Considering the random behaviour of cracking and dynamic propagation of cracks, it is felt that, appropriate methods involving concepts of fracture mechanics have to be developed for estimating the width of cracks in reinforced concrete members.

3. A method has been proposed for predicting the spacing and maximum width of cracks in conventionally reinforced concrete flexural members with steel fibres. The values of spacing and width of cracks computed

using the proposed method compare satisfactorily with the test results available in literature.

4. The method proposed earlier for the predicting the spacing and maximum width of cracks in conventionally reinforced concrete flexural members with steel fibres has been extended to latex modified SFRC flexural members. The values of spacing and width of cracks computed using the proposed method compare satisfactorily with the test results.

5. The computed values of spacing of cracks using equations available in literature for conventionally reinforced concrete flexural members predict the spacing of cracks in latex modified SFRC flexural members satisfactorily.

7.5 Scope for Further Work

The present work could be extended to study

1. Effect of different types of polymers on the strength and behaviour of the conventional concrete.
2. Influence of aspect ratio of fibres on the strength and ductility of polymer modified concrete.
3. Studies on other structural members like columns and column beam joints subjected to both monotonic and cyclic loading.
4. Durability studies on polymer modified steel fibre reinforced concrete.

REFERENCES

1. ACI COMMITTEE REPORT NO. 544., " State of the Art Report on Fibre Reinforced Concrete", ACI-544 IR-82 (Re-approved), American Concrete Institute, Detroit, Part 5, May 1982, pp 544.IR-22.

2. ACI COMMITTEE REPORT NO. 544., " Measurement of properties of Fibre Reinforced Concrete", American Concrete Institute, ACI Materials Journal, Vol. 85, No. 6, Nov.- Dec. 1988, pp. 583-593.

3. ACI COMMITTEE REPORT NO. 318, " Building Code Requirements for Reinforced Concrete, (ACI-318-89)and commentary- ACI 318R-89" American Concrete Institute, Detroit-1989, Chapter-II, Shear and Torsion, pp.137-180.

4. ACI COMMITTEE 224. " Control of cracking in concrete structures". Journal of American Concrete Institute. Vol.69, No. 12., December 1972, pp:717- 752.

5. ACI COMMITTEE REPORT NO. 548., " Guide for the Use of Polymers in Concrete," ACI Journal, Vol. 83, No.5, Sep.- Oct. 1986. pp. 798- 829.

6. ACHYUTHA. H., and SABAPATHI, P., " Cracking characteristics of R.C. beams with steel fibres." Proceedings of the ISFRC, Vol. 1, Dec. 16-19, 1987. Madras, India. pp 2.59- 2.68.

7. ANTONIO NANNI., " Design for torsion using steel fibre reinforced concrete." ACI Materials Journal, Vol. 87, No.6, Nov.- Dec. 1990. pp. 556-564.

8. BASE, G.D., READ, J.B., BEEBY, A.W., and TAYLOR, H.P.J. " An Investigation of the Crack Control Characteristics of Various Types of Bars in Rein-forced Concrete Beams". Research report No. 18, Cement and Concrete Association, London, December 1966.

9. BASE, G.D., and READ, J.B., " Effectiveness of Helical Binding in the Compression Zone of Concrete Beams," ACI Journal, Vol. 62, 1965, pp. 763-780.

10. BEEBY, A.W., " The prediction of Crack Width in Hardened Concrete," The Structural Engineer, Vol. 57, Jan. 1979.

11. BROMS, B.B., " Stress Distribution in Reinforced Concrete Members with Tension Cracks," Journal of the American Concrete Institute Proceedings, Vol. 62, Sept. 1965.

12. B.S. 8110: PART 2. 1985. Serviceability calculation, Section 3, British Standard Institution, London.

13. CEB - FIP MODEL CODE 1990. Bulletin d' Information No. 203, Comite Euro-International du. Beton Lansanne, July 1991.

14. CP-110, Part-1, 1972, "The structural use of concrete, Design materials and workmanship. British Standards Institution.

15. CHAN, W.W.L., " The Ultimate Strength and Deformation of Plastic Hinges in Reinforced Concrete Frame works," Magazine of Concrete Research (London), Vol. 7, No. 21, pp 121-132.

16. CHENKUI, HUANG; GUOFAN, ZHAO " Properties of steel fibre reinforced concrete containing larger coarse aggregate", Cement & Concrete Composites v 17 n 3 1995. pp. 199-206

17. CLARK, A.P., " Cracking in Reinforced Concrete Flexural Members". Journal of the American Concrete Institute, Proceeding vol-52, April 1956.

18. CUSSON, DANIEL; PAULTRE, PATRICK ,"Stress-strain model for confined high-strength concrete ,Journal of Structural Engineering, Vol. 121, No. 3, Mar 1995. pp. 468-477

19. DANIEL BORDELEAU, MICHEL PIGEON, and NEMKUMAR BANTHIA, " Comparative Study of Latex Modified Concretes and Normal Concretes Subjected to Freezing and Thawing in the Presence of Deicer Salt Solution," ACI Materials Journal, Vol. 89, No.6, Nov.- Dec. 1992. pp. 547- 553.

20. DESAYI. P AND GANESAN .N , " An Investigation on Spacing of Cracks and Maximum Crack Width in Reinforced Concrete Flexural Members." Materials and Structures (RILEM, Paris), Vol 18, March April 1985. pp 123 - 133.

21. DIKEOU, J.T., KUKACKA, L.E., BACKSTROM, J.E., and STEINBERG, M., "Polymerization Makes Tougher Concrete," ACI Journal, October 1969, pp 829- 839.

22. DWARAKANATH, H.V., and NAGARAJ, T.S., " Flexural Behaviour of Reinforced Fibre Concrete Beams", Proceedings of the International Symposium on Fibre Reinforced Concrete, Madras, India, Dec. 16-19, 1987, Vol.1, pp 2.49-2.58.

23. DWARAKANATH, H.V., and NAGARAJ, T.S., " Comparative Study of Prediction of Flexural Strength of Steel Fibre Concrete." ACI Materials Journal, Vol. 88, No.6, Nov.- Dec. 1991. pp. 714-720.

24. EL-DASH, K.M.; AHMAD, S.H., "Model for stress-strain relationship of spirally confined normal and high-strength concrete columns", Magazine of Concrete Research, Vol.47 No.171 Jun 1995. pp 177-184

25. ESPION, B.; DEVILLERS, J.-J.; HALLEUX, P. "On the use of steel fibres as complementary reinforcement of reinforced concrete beams in bending" Materials and Structures Vol- 26 No. 162 Oct 1993. pp 479-485

26. EZELDIN, A. SAMER; SHIAH, T.W. "Analytical immediate and long-term deflections of fiber-reinforced concrete beams", Journal of Structural Engineering, Vol. 121 No. 4 Apr 1995. pp 727-736

27. FANG, I. KUANG; HONG, KUN-CHIANG; WU, JUN YU, " Strength and ductility of high strength tied columns", Proceedings of the National Science Council, Republic of China, Part A: Physical Science and Engineering Vol.18, No. 1 Jan 1994. pp 63-74

28. FILIATRAULT, ANDRE; PINEAU, SYLVAIN; HOUDE, JULES , "Seismic behavior of steel-fiber reinforced concrete interior beam- column joints, ACI Materials Journal (American Concrete Institute), Vol. 92 No. 5 Sep-Oct 1995. pp 543-552

29. GANESAN, N. and RAMANA MURTHY, J.V., "' Strength and Behaviour of Confined Steel Fibre Reinforced Concrete Columns," ACI Materials Journal, Vol. 87, No.3, May - June, 1990. pp. 221-227.

30. GERRY WALTER, D., " Comparison of Latex Modified Portland Cement Mortars," ACI Materials Journal, Vol. 87, No.4, July.- Aug. 1990. pp. 371-377.

31. GERGELY.P. and LUTZ. L.A. " Maximum crack width in reinforced concrete flexural members " Paper No.6, Causes mechanism and control of cracking in concrete". ACI Publication SP- 20. ACI. Detroit, Michigan, USA. 1968.

32. GOPALARATNAM, VELLORE S.; GETTU, RAVINDRA, "On the characterization of flexural toughness in fiber reinforced concretes", Cement & Concrete Composites Vol. 17 No. 3 1995. pp 239-254

33. HIRASAWA, IKUO; MOTOYAMA, SIGEKI; FUJISHIRO, MASAHIRO, "Test and analysis on the ultimate strength of short columns with confining reinforcement under biaxial bending, Proceedings of the Japan society of Civil Engineers, No. 490, pt 5-23 1994-1995. pp 91-100

34. HENAGER, C.H., "Ultimate Strength of Reinforced Steel Fibrous Concrete Beams," Proceedings of the Conference held in London, March 23-24, 1977, pp. 165-173.

35. HOGNESTAD, E., "High strength bars as concrete reinforcement. Part 2 : Control of Flexural Cracking". Journal of the PCA. Research and Development laboratories, Vol. 4, January 1962.

36. HUGHES, B.P.; AL-DAFIRY, H., "Impact energy absorption at contact zone and supports of reinforced plain and fibrous concrete beams, Construction and Building Materials Vol. 9, No. 4, Aug 1995. pp 239-244

37. HWAN OH, B. and YOUNG - JIN KANG , " New Formulas for Maximum Crack Width and Crack Spacing in Reinforced Concrete Flexural Members." ACI Structural Journal, March - April 1987. pp 103 - 111.

38. IBRAHIM AND LUXMOORE, " Control of Crack Width by inclusion of Fibres in Conventionally Reinforced Concrete". Journal of Cement Composites, Vol.1, July 1979, pp 77-89.

39. IRAWAN, PAULUS; MAEKAWA, KOICHI "Three-dimensional analysis on strength and deformation of concrete confined by lateral reinforcement", Proceedings of the Japan Society of Civil Engineers No.472 pt 5-20 Aug 1993. pp 107-118

40. I.S. 456- 1978, "Code of Practice for Plain and reinforced Concrete," Indian Standard Institution, New Delhi. 1984.

41. ISHIBASHI, TADAYOSHI; TSUYOSHI, TAKESHI, " Study on the estimation of bending crack width on the surfaces of concrete girders", Proceedings of the Japan Society of Civil Engineers, No 484, pt 5-22 Feb 1994. pp 33-40.

42. IYENGAR, K.T.S., DESAYI,P., and NAGIREDDY, K., " Stress Strain Characteristics of Concrete Confined in Steel Binders." Magazine of Concrete Research, (London), Vol.22, No.72, 1970, pp.173-184.

43. IYENGAR, K.T.S., DESAYI,P., and NAGIREDDY, K., " Flexure of Reinforced Concrete Beams with Confined Compression zones," ACI Journal, Vol.68, No.9, Sep.- Oct. 1971, pp.719-725.

44. IYENGAR, K.T.S., DESAYI,P., and NAGIREDDY, K., " Confined Concrete and its Applications in R.C. Beams and Frames," Building Science, Vol. 7, 1972, pp.105-120.

45. IYENGAR, K.T.S., DESAYI,P., and NAGIREDDY, K., " Deflections in Reinforced Concrete Beams with Confined Compression zones," ACI Journal, SP-43, 19 pp. 279-294.

46. IYENGAR, K.T.S., DESAYI,P., and NAGIREDDY, K., " Ductility of Reinforced Concrete Sections with Confined Compression Zones," Earthquake Engineering and Structural Dynamics, Vo. 4, 1975, pp. 111- 118.

47. JOSEPH. A. LAVELLE., "Acrylic Latex Modified Portland Cement," ACI Materials Journal, Vol. 85, No.1, Jan.- Feb. 1987. pp. 41-48.

48. KARIM.S REBEIZ, SLEIMAN.P.SERHAL, and DAVID W. FOWLER, "Recommended Design Procedure in Shear for Steel Reinforced Polymer Concrete," ACI Structural Journal, Vol. 90, No.5, Sep.- Oct. 1993. pp. 562-567.

49. KAUSHIK, S.K., " Behaviour of SFRC Under Tension, Compression and Flexure," Proceedings of the National Seminar on Properties and Application of Fibre Reinforced Concrete, Madras, Dec. 21-22, pp. p4.1- p4.51.

50. KENT, DUDLEY CHARLES, and PARK ROBERT., " Flexural Members with Confined Concrete, " Proceedings of ASCE, Vol. 97, ST7, pp. 1969-1990.

51. KHALOO, A.R.; KIM, N., "Mechanical properties of normal to high-strength steel fiber-reinforced concrete", Cement, Concrete and Aggregates, Vol.18, No. 2, Dec 1996. pp 92-97

52. KONG, F.K., AND EVANS, R.H, Reinforced and Pre-Stressed Concrete, Third Edition, 1987, ELBS Publications.

53. KUMAR.V., VINOD KUMAR GUPTA, DHIRENDRA SINGHAL and ROY, B.N., " Rotational Capacity of SFRC Beams with Equal Reinforcement on Tension and Compression Faces," Journal of Structural Engineering, Vol.-22, No.3, Oct.-1995, pp. 129-134.

54. LECH CZARNECKI, " The status of Polymer concrete," Concrete International, July 1985, pp 47-53.

55. LI, V.C., and STANG. H. KRENCHEL., " Micromechanics of Crack Bridging in Fibre reinforced Concrete," Materials and Structures, (RILEM), No. 26, 1993, pp 486-494.

56. LI, VICTOR C.; WU, HWAI-CHUNG; MAALEJ, MOHAMED; MISHRA, DHANADA K.; HASHIDA, TOSHIYUKI "Tensile behavior of cement-based composites with random discontinuous steel fibers ", Journal of the American Ceramic Society Vol. 79 No. 1 Jan 1996. pp 74-78

57. LIMAYE, R.G., and KAMATH, M.K., " Experimental Studies on Polymer Modification of Cement Mortar," Indian Concrete Journal, March 1992, pp 153- 158.

58. MAC- GREGOR, and GREENWOOD, C.T., " polymers in Nature," John Wiley and Sons, pp 329-340.

59. MANDER,J.B., PRIESTLEY, M.J.N, and PARK, R., " Theoretical Stress-Strain Model for Confined Concrete beams," Journal of The ACI, Vol. 62, 1965, pp. 1804-1825.

60. MANGAT,P.S., and SWAMY,R.N., " Some Properties of Fibre Polymer Concrete System", RILEM, Materials and Structures, Vol. 10, No.60, pp.1-12.

61. MARIANO VALLE and ORAL BUYUKOZTURK., " Behaviour of Fibre Reinforced High - Strength Concrete under Direct Shear." ACI Materials Journal, Vol. 90, No.2, Mar.- Apr. 1993. pp. 122-133.

62. MOENS, J. and NEMEGEER, D., " Designing Fibre Reinforced Concrete Based on Toughness Characteristics," Concrete International, Nov. 1991, pp 38-43.

63. NAGARAJ,T.S., IYENGAR, K.T.S., AND KAMESWARA RAO, B., "Superplasticized Natural Rubber Latex Modified Concrete", International Journal of Cement and Concrete Research., 18, No.1, Jan.1988, pp 138-144.

64. NEVILLE, A.M., Properties of Concrete , Third Edition, 1981, ELBS Publications.

65. OHAMA, Y, DEMURA, K., MASUMI, H., and MASURA, K., " Properties of Polymer Modified Mortars Using Styrene Butyl Acrylate latexes with Various Monomer Ratios." ACI Materials Journal, Vol. 88, No.1, Jan.- Feb. 1991. pp. 56-61.

66. PAKOTIPRAPHA,B., PAMA, R.P. and LEE, S.L. " Mechanical Properties of Cement Mortar with Randomly Oriented Short Steel Wires," Magazine of Concrete Research, Vol. 26, No.86, 19, pp. 3-15.

67. PAPADOPOULOS, P.G., "Truss model for the confinement of concrete columns", Proceedings of the 1996 1st International Conference on Earthquake Resistant Engineering Structures, ERES 96, Thessaloniki, Greece, Earthquake Resistant Engineering Structures Advances in Earthquake Engineering, Vol. 2, 1995. pp 503-511

68. PARAMASIVAM, P., T.Y. LIM., and S.L. LEE , " Flexural behaviour of reinforced steel fibre concrete beams " Proceedings of the ISFRC, Vol. 1, Dec. 16-19, 1987. Madras, India. pp 1.79 - 1.90.

69. PARAMESWARAN, V.S, KRISHNAMURTHY, T.S, BALASUBRAMANIYAN, K. and LAKSHMANAN, N. "Behaviour of Fibre Reinforced Concrete Beams with Equal Tension and Compression Reinforcement", Proceedings of the International Symposium on Fibre Reinforced Concrete, Madras, India, Dec.16-19, 1987, Vol.1, pp 2.3-2.24.

70. PARAMESWARAN, V.S., "Research and applications of FRC - Indian scenario", Indian Concrete Journal Vol. 70 No.10 Oct 1996. pp 553-557

71. PARK,R., and PAULAY, T., " Reinforced Concrete Structures," 1975, John Wiley and Sons, New York.

72. PARVIZ SOROUSHIAN, FADHEL AOUADI AND MOHAMAD NAGI., "Latex Modified Carbon Fibre reinforced Mortar", ACI Materials Journal, Vol. 88, No.1, Jan.- Feb. 1991. pp. 11-18.

73. PARVIZ SOROUSHIAN and ZIAD BAYASI., " Fibre - Type effects on the Performance of Steel Fiber Reinforced Concrete." ACI Materials Journal, Vol. 88, No.2, Mar.- Apr. 1991. pp. 129-134.

74. PARVIZ SOROUSHIAN, ATEF TLILI, MAKOTO TEHENA, and BEN.L.TILSEN, " Durability Characteristics of Polymer Modified Glass Fibre Reinforced Concrete", ACI Materials Journal, Vol. 90, No.1, Jan.- Feb. 1993. pp. 40-49.

75. PERUMALSAMY N. BALAGURU AND SURENDRA P. SHAH., Fibre Reinforced Cement Composites - (Eds.), Mc Graw Hill International Edition, Civil Engineering Series. 1992.

76. RAJAGOPALAN, K, PARAMESWARAN, V.S., and RAMASWAMY, G.S., " Strength of Steel Fibre Reinforced Concrete Beams," Indian Concrete Journal, Vol.48, No. 1, pp. 17-25.

77. RAMAKRISHNAN.V., " Materials and Properties of Fibre Reinforced Concrete", Proceedings of the International Symposium on Fibre Reinforced Concrete, Madras, India, Dec. 16-19, 1987, Vol.1, pp 2.3-2.24.

78. RAVINDRA RAJHA., R. " Effect of Natural Rubber Latex on the Properties of Portland Cement Paste, Mortar, and Concrete," Procedings of the Third International Congress on Polymers in Concrete, 19 , Vol-1, pp. 380-394.

79. REBEIZ, K.S., SERHAL, S, and FOWLER, D.W., " Shear Behaviour of Steel Reinforced Polymer Concrete Using Recycled Plastic," ACI Structural Journal, Vol. 90, No.6, Nov.- Dec. 1993. pp. 675-682.

80. RICHERT, F.E, BRANDTZAEG, A, and BROWN, R.L., " A Study of the Failure of Concrete Under Combined Compressive Stress," University of Illinois Engineering Experiment Station, Bulletin, No. 185, 1928.

81. ROMUALDI, J.P., and BATSON, G.B., " Behaviour of R.C. beams with closely spaced reinforcement," ACI Journal, Proceedings V.60, June 1963, pp. 775-790.

82. ROY, H.E.H., and SOZEN, METE, A., " Ductility of Concrete," Flexural Mechanics of Reinforced Concrete, SP-12, ACI-ASCE, Detroit, pp. 213-224.

83. SABAPATHI.P, " Flexural Behaviour of Reinforced Concrete Beam with Steel Fibres " Ph.D thesis submitted to the Department of Civil Engineering, IIT Madras. Oct. 1983.

84. SAMEN EZELDIN and BALAGURU, P.N., " Bond Behaviour of Normal and High Strength Fibre Reinforced Concrete," ACI Materials Journal, Vol. 86, No.5, Sep.-Oct. 1989. pp. 515-524.

85. SANTHAKUMAR, A.R., LAKSHMIPATHY, M. and HARIRAJ, P. " Flexural Behaviour of Prestressed Fibre Concrete Beams", Proceedings of the International Symposium on Fibre Reinforced Concrete, Madras, India, Dec. 16-19, 1987, Vol.1, pp 2.179-2.190.

86. SARGIN, M., GHOSH, S.K., and HANDA, V.K., " Effect of Lateral Reinforcement upon the strength and Deformation Properties of Concrete," Magazine of Concrete Research (London), Vol. 23, No. 75-76, pp 99-118.

87. SCOTT, B.D., PARK,R., and PRIESTLEY, M.J.N., " Stress- Strain Behaviour of Concrete Confined by Overlapping Hoops at Low and High Strain Rate," ACI Journal, Vol 79, 1982, pp 484-490.

88. SHAH,S.P., and AHMED, S.H., " Stress-Strain Curve of Concrete Confined by Spiral Reinforcement," ACI Journal Proceedings, Vol. 79, 1982, pp. 484-490.

89. SHAH.S.P AND VIJAYARANGAN.B, " Fibre Reinforced Concrete Properties" ACI Journal, Vol.1, Feb. 1971, pp. 126-135.

90. SHAH, S.P., LUDIRDJA, D., DANIEL, J.I., and MOBASHER, B., "Toughness-Durability of glass fibre reinforced concrete system." ACI Materials Journal, Vol. 85, No.5, Sep.- Oct. 1988. pp. 352-360.

91. SOLIMAN, M.T.M., and YU, C.W., " The flexural Stress Strain Relationship of Concrete Confined by Rectangular Transverse Reinforcement," Magazine of Concrete Research (London), Vol. 19, No. 61, pp 223-238.

92. SPADEA, G.; BENCARDINO, F., "Behavior of fiber-reinforced concrete beams under cyclic loading Journal of Structural Engineering Vol. 123 No. 5, May 1997. pp 660-668

93. SRIDHARAN, A., NAGARAJ, T.S., and CHANDRAKARAN, S., " Dynamic Response of latex modified concrete machine foundations," magazine of Concrete Research, No. 42, Mar. 1990, pp 37- 43.

94. Standard test method for flexural toughness of fibre reinforced concrete (using beam with third point loading) (C-1018-84), 1984 Book of ASTM standards, part-04.02, ASTM, Philadelphia. pp 637-644.

95. Standard test method for flexural toughness of fibre reinforced concrete (Standard-SF4), JCI standards for test methods of fibre reinforced concrete Japan Concrete Institute, 1983, pp45-51.

96. SWAMY. R.N., AL - TAAN S and SAMY A.R., ALI, " Steel fibres for controlling cracking and deflection. " Concrete International, August 1979. pp 41 - 49.

97. TAN, KIANG-HWEE; PARAMASIVAM, P.; MURUGAPPAN, K. "Steel fibers as shear reinforcement in partially prestressed beams, ACI Structural Journal Vol. 92, No. 6, Nov-Dec 1995. pp 643-652

98. TAYLOR, M.R.; LYDON, F.D.; BARR, B.I.G. "Toughness characterisation of fibre-reinforced concrete, Indian Concrete Journal, Vol. 70, No. 10, Oct 1996. pp 525-531

99. TROTTIER, JEAN-FRANCOIS; BANTHIA, NEMKUMAR Toughness characterisation of steel-fibre reinforced concrete", Journal of Materials in Civil Engineering, Vol. 6 No. 2, May 1994. pp 264-289

100. WANG, NIANZHI; MINDESS, SIDNEY; KO, KEITH, "Fibre reinforced concrete beams under impact loading Cement and Concrete Research Vol. 26, No. 3 Mar 1996. pp 363-376

101. WATSON, S.; ZAHN, F.A.; PARK, R., "Confining reinforcement for concrete columns", Journal of Structural Engineering, Vol.120, No. 6, Jun 1994. pp 1798-1824

102. ZHAO, WEIJIAN; MARUYAMA, KYUICHI, " Reevaluation of present equations for flexural crack width of RC beams Proceedings of the Japan Society of Civil Engineers, No.490, pt 5-23 1994-1995. pp 147-156

103. ZHAO, WEIJIAN; MARUYAMA, KYUICHI, "Experimental study of flexural cracking of RC beams with multi-layers of longitudinal bars", Proceedings of the Japan Society of Civil Engineers n 490, 1995. pp 137-145

104. ZIARIA, M. MOHAMMED, DAVID HALDANE and ATTALLAH S. KUTTUB, " Flexural Behaviour of Beams with Confinement," ACI Structural Journal, Vol. 92, No. 1, 1995, pp. 103-114.

105 ZONGIAN, L., and DAJUN, D., " Crack Width in Reinforced Concrete Members." International Journal of Structures. Vol. 12, December 1992. pp.

APPENDIX - I

Prediction of First Crack load and Ultimate Moment of Resistance of latex Modified Steel fibre reinforced concrete

Beam No. 8

Data available

Compressive strength of concrete (Cylinder strength)	f_c	=	32.54 N/mm^2
Volume fraction of steel fibres	V_f	=	0.5%
Dry rubber content	DRC	=	0.5%
Aspect ratio of Steel fibres	A_p	=	50
Length of fibres	l_f	=	44mm
Ultimate tensile strength of fibre	σ_{fu}	=	330.0 N/mm^2
Ultimate Load	Pu(exp)	=	67.62 KN
	α	=	0.9 for fibre concrete
Elastic Modulus of Concrete	E_{cc}	=	18077.77 N/mm^2

To find the First Crack Load P_{cr} :

Neutral Axis (N-A) was calculated form the expression

$$bh(Nd - \frac{h}{2}) + (m_8 - 1)(Nd - d_2) + (m_f - 1)\, V_f\, bh(Nd - \frac{h}{2})$$

$$= (m_{12} - 1)\, A_{st} 12(d - Nd)$$

$$Nd = \frac{bh\frac{h}{2} + (m_8 - 1)\, A_{sc}\, d_c + (m_f - 1) V_f\, bh\frac{h}{2} + (m_{12} - 1)\, A_{st}\, d}{bh + (m_8 - 1)\, A_{sc} + (m_f - 1) V_f\, bh + (m_{12} - 1)\, A_{st}}$$

Here	b = 125 mm ,	h	= 175mm ,
	E_{8mm} = 1.923 x 10^5	E_{12mm}	= 2.127 x 10^5
	E_{fibre} = 0.30 x 10^5	E_{cc}	= 18077.77 N/mm^2
	m_8 = E_{8mm} / E_{cc}	m_{12}	= E_{12mm} / E_{cc}
	m_f = E_{fibre} / E_{cc}	A_{sc}	= 108.4 mm^2
	A_{st} = 352.0 mm^2	V_f	= 0.5/100

Substituting all these values in the above equation and
after simplification we get depth of NA (Nd) = 105.27 mm

To find gross M.I.

$$M.I._{(Concrete)} = 125 \times \frac{200^3}{12} + (125 \times 200 \times (105.27 - 100)^2)$$

Gross M.I. = M.I.$_{concrete}$ + M.I.$_{Steel}$ + M.I.$_{Fibres}$

Gross M.I. = 0.109×10^9 mm^4

$$M.I._{(steel)} = (m_{12} - 1) \times 352.0 \times (175 - 105.27)^2 + (m_8 - 1) \times 108.4 \times (105.27 - 20)^2$$

$$AF = \frac{(\pi \times 0.88^2)}{4} \times \frac{V_f}{100} \times 125 \times 200$$

$$M.I.(Fibre) = (m_f - 1) \times AF \times (105.27 - 100.0)^2$$

Modulus of rupture of Latex Modified SFRC flexural members is given by :

$$f_r = 0.0167\,(LF_i)^2 - 0.19\,(LF_i) + 4.79$$

$$Where \quad LF_i = (1 + DRC) \times (1 + V_F)$$

$$= (1 + 0.005) \times (1 + 0.005) = 1.010025$$

$$\therefore f_r = 4.6 \text{ N/mm}^2$$

$$M_{cr} = \frac{f_r \; I_g}{y_t} = \frac{4.60 \times 0.109 \times 10^9}{(200 - 105.27)}$$

$$= 5.29 \times 10^6 \; N\text{-}mm$$

$$= 5.29 \; KNM$$

$$P_{cr(theo)} = \frac{4.8 M_{cr}}{Span} = \frac{4.8 \times 5.29}{1.8} = 14.10 \text{ kN}$$

$P_{cr\ Experimental}$ = 11.76 kN

Ratio $P_{cr\ theo} / P_{cr\ exp}$ = 1.198

Ultimate Moment of Resistance (M_u)

Method proposed by Paramasivam et.al. :

The ultimate moment of resistance (M_u) is given by

$$M_u = \sigma_{tu} \frac{bhh_t}{2} + \frac{A_s \; f_y}{2} (h + h_t)$$

Where σ_{tu} = ultimate tensile strength of composite and is given by

$$\sigma_{tu} = \eta_l \; \eta_0 \; V_f \; l_f \; \frac{\tau_u}{2r}$$

and h_t is the depth of zone under tension and is given by :

$$h_t = \frac{\alpha_1 \; \sigma_{cu} \; d \; \frac{A_s \; f_y}{b}}{(\alpha_1 \; \sigma_{cu} + \sigma_{tu})}$$

Here η_0 is the orientation factor and is given by

$$\eta_0 = \frac{\int_0^{\rho} \int_0^{\theta} \cos\theta \cos\rho \; d\theta \; d\rho}{\int_0^{\rho} \int_0^{\theta} d\theta \; d\rho}$$

η_l is the length efficiency factor and is calculated based on the following

$$\eta_l = 0.5 \qquad \text{for} \quad l_f < l_c$$

$$\eta_l = 1-(l_c/2l_f) \qquad \text{for} \quad l_f > l_c$$

Where l_c is the critical length of fibre and is given by

$$l_c = 0.5 \frac{\sigma_{fu} \; d_f}{\tau_u}$$

$$l_c = \frac{0.5 \times 330.0 \times 0.88}{2.0} = 72.60mm$$

For the available data

Here l_f is less than l_c, Hence $\eta_l = 0.5$

Orientation factor $\eta_0 = 0.405$ (after integrating the equation between the limits (0 to $\pi/2$)

$$r = \frac{C/s\ area\ of\ fibre}{peremeter\ of\ fibre} = \frac{0.6084}{2.7657} = 0.22$$

Ultimate tensile strength of composite is given by

$$\sigma_{tu} = 0.5 \times 0.405 \times \frac{0.5}{100} \times \frac{0.5}{100} \times \frac{2}{2 \times 0.22}$$

$$= 0.0000227 \ N/mm^2$$

Depth of zone under tension h_t is give by :

$$h_t = \frac{0.9 \text{ x } 32.54 \text{ x } 175.0 - (\frac{353.0 \text{ x } 470.0}{125})}{(0.9 \text{ x } 32.54) + 0.0000227}$$

$$= 129.68 \text{ mm}$$

Ultimate Moment M_u and Ultimate load P_u is given by :

$$M_u = 0.0000227 \text{ x } \frac{125 \text{ x } 175 \text{ x } 129.68}{2} + \frac{353.00 \text{ x } 470.0}{2} (175 + 129.68)$$

$$= 25274761.0 \text{ N - mm} \quad (25.27 \text{ KN - M})$$

$$P_u = \frac{2 \text{ x } M_u}{0.75} = \frac{2 \text{ x } 25.27}{0.75}$$

$$= 67.39 \text{ KN}$$

Ratio $P_{u(theo)} / P_{u(exp)}$ = (67.39 / 67.62) = 0.997

APPENDIX -II

TEST RESULTS USED FOR THE COMPARISON OF INTERNATIONAL CODE EQUATIONS FOR THE PREDICTION OF MAXIMUM WIDTH OF CRACKS IN R.C. FLEXURAL MEMBERS

C & CA DATA (Base. et al. [8])

Stage - 1

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain in steel x 10^{-3}	Crack width (x 10^{-3}) (in)
1	15.125	13.125	8.00	1.25	2	1.375	1.375	5050	0.27	2.20
2	15.125	13.125	8.00	1.25	2	1.375	1.375	5400	0.27	2.49
3	15.125	13.125	8.00	1.25	2	1.375	1.375	5350	0.26	2.21
4	15.125	13.125	8.00	1.25	2	1.375	1.375	4980	0.22	1.25
5	15.125	13.125	8.00	1.25	2	1.375	1.375	5180	0.26	2.05
6	15.125	13.125	8.00	1.25	2	1.375	1.375	4580	0.25	1.93
7	15.125	13.125	8.00	1.25	2	1.375	1.375	4500	0.25	2.16
8	15.125	13.125	8.00	1.25	2	1.375	1.375	4520	0.22	1.82
9	15.125	13.125	8.00	1.25	2	1.375	1.375	4550	0.30	2.16
10	15.125	13.125	8.00	1.25	2	1.375	1.375	4600	0.25	2.10
11	15.750	13.125	8.00	0.875	4	1.375	1.375	4840	0.23	2.05
12	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	0.23	1.59
13	15.750	13.125	8.00	0.875	4	1.375	1.375	4810	0.32	1.59
14	15.750	13.125	8.00	0.875	4	1.375	1.375	4940	0.21	1.36
15	15.750	13.125	8.00	0.875	4	1.375	1.375	4880	0.32	2.27
16	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	0.30	2.27
17	15.750	13.125	8.00	0.875	4	1.375	1.375	4780	0.25	2.73
18	15.750	13.125	8.00	0.875	4	1.375	1.375	4750	0.27	2.27
19	15.750	13.125	8.00	0.875	4	1.375	1.375	4870	0.33	2.05
20	15.750	13.125	8.00	0.875	4	1.375	1.375	4830	0.27	2.05
21	16.000	13.125	8.00	0.50	12	1.375	1.375	4380	0.23	1.82
22	16.000	13.125	8.00	0.50	12	1.375	1.375	4410	0.32	2.05
23	16.000	13.125	8.00	0.50	12	1.375	1.375	4300	0.23	1.59
24	16.000	13.125	8.00	0.50	12	1.375	1.375	4340	0.23	2.50
25	16.000	13.125	8.00	0.50	12	1.375	1.375	4440	0.32	2.05
26	16.000	13.125	8.00	0.50	12	1.375	1.375	5490	0.25	1.59
27	16.000	13.125	8.00	0.50	12	1.375	1.375	5540	0.25	1.82
28	16.000	13.125	8.00	0.50	12	1.375	1.375	5450	0.25	1.36
29	16.000	13.125	8.00	0.50	12	1.375	1.375	5580	0.32	1.36
30	16.000	13.125	8.00	0.50	12	1.375	1.375	5740	0.27	1.14
31	15.125	13.125	8.00	0.875	4	0.750	0.750	4175	0.33	1.82
32	15.125	13.125	8.00	0.875	4	1.375	1.375	4203	0.36	2.50
34	15.750	13.125	8.00	0.875	4	1.375	1.375	4216	0.30	2.50
35	17.125	13.125	8.00	0.875	4	2.750	1.375	3800	0.27	1.59

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain in steel x 10^{-3}	Crack width (x 10^{-3}) (in)
36	17.125	13.125	8.00	0.875	4	2.750	1.375	3880	0.39	2.27
37	16.125	13.125	8.00	0.875	4	1.750	1.375	3950	0.25	1.14
38	16.125	13.125	8.00	0.875	4	1.750	1.375	4000	0.32	1.36
39	14.875	13.125	8.00	0.875	4	0.500	1.375	4190	0.30	0.68
40	14.875	13.125	8.00	0.875	4	0.500	1.375	4240	0.28	0.91
41	14.875	13.125	8.00	0.875	4	0.500	0.688	4600	0.39	1.25
42	15.750	13.125	8.00	0.875	4	1.375	1.563	4800	0.41	1.36
43	15.750	13.125	8.00	0.875	4	1.625	3.250	4850	0.32	2.16
44	14.875	13.125	8.00	0.875	4	0.500	0.688	4700	0.32	1.14
45	15.750	13.125	8.00	0.875	4	1.375	1.563	4950	0.39	1.70
46	15.750	13.125	8.00	0.875	4	1.625	3.250	5160	0.34	2.73
47	15.750	13.125	8.00	0.875	4	1.375	1.375	2950	0.36	2.73
48	15.750	13.125	8.00	0.875	4	1.375	1.375	2985	0.48	2.61
49	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	0.30	2.27
50	15.750	13.125	8.00	0.875	4	1.375	1.375	2827	0.52	3.86
51	15.750	13.125	8.00	0.875	4	1.375	1.375	3475	0.37	3.75
52	15.750	13.125	8.00	0.875	4	1.375	1.375	4900	0.50	2.95
53	16.000	13.125	8.00	0.500	12	1.375	1.375	4850	0.18	1.93
54	16.000	13.125	8.00	0.500	9	1.375	1.375	4775	0.41	2.27
55	16.000	13.125	8.00	0.500	6	1.375	1.375	5020	0.25	2.05
56	15.750	13.125	8.00	0.875	4	1.375	1.375	5190	0.25	1.93
57	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	0.34	2.27
58	15.750	13.125	8.00	0.625	4	1.375	1.375	4900	0.39	2.50
59	16.000	13.125	8.00	0.500	12	1.375	1.375	4340	0.36	2.05
60	16.000	13.125	8.00	0.500	9	1.375	1.375	4150	0.36	2.73
61	16.000	13.125	8.00	0.500	6	1.375	1.375	4540	0.25	2.05
62	15.750	13.125	8.00	0.875	4	1.375	1.375	4085	0.34	2.27
63	15.750	13.125	8.00	0.875	4	1.375	1.375	4048	0.25	1.82
64	15.750	13.125	8.00	0.875	4	1.375	1.375	4150	0.43	2.50
65	15.750	13.125	8.00	0.875	4	1.375	1.375	4279	0.37	3.72
66	15.750	13.125	8.00	0.875	4	1.375	1.375	3410	0.24	1.59
67	15.750	13.125	8.00	0.875	4	1.375	1.375	4000	0.30	1.59
68	15.750	13.125	8.00	0.875	4	1.375	1.375	4670	0.36	2.61
69	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.00	0.00
70	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.39	1.59
71	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.34	2.16
72	16.000	14.375	8.00	0.50	6	1.375	1.375	4110	0.34	1.14
73	16.000	13.125	6.25	0.50	12	1.375	0.500	5210	0.36	0.68
74	16.000	13.125	8.00	0.50	12	1.375	1.375	5300	0.48	1.59
75	16.000	13.125	9.75	0.50	12	1.375	2.250	5340	0.25	1.14
76	16.000	14.375	6.25	0.50	6	1.375	0.500	4040	0.00	0.00
77	16.000	14.375	9.75	0.50	6	1.375	2.250	3940	0.20	0.91

Stage - II

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain in steel x 10^{-3}	Crack width (x 10^{-3}) (in)
1	15.125	13.125	8.00	1.25	2	1.375	1.375	5050	0.64	4.47
2	15.125	13.125	8.00	1.25	2	1.375	1.375	5400	0.69	5.57
3	15.125	13.125	8.00	1.25	2	1.375	1.375	5350	0.66	4.54
4	15.125	13.125	8.00	1.25	2	1.375	1.375	4980	0.57	3.52
5	15.125	13.125	8.00	1.25	2	1.375	1.375	5180	0.63	4.09
6	15.125	13.125	8.00	1.25	2	1.375	1.375	4580	0.65	4.32
7	15.125	13.125	8.00	1.25	2	1.375	1.375	4500	0.66	4.55
8	15.125	13.125	8.00	1.25	2	1.375	1.375	4520	0.68	4.55
9	15.125	13.125	8.00	1.25	2	1.375	1.375	4550	0.71	4.80
10	15.125	13.125	8.00	1.25	2	1.375	1.375	4600	0.61	4.30
11	15.750	13.125	8.00	0.875	4	1.375	1.375	4840	0.71	5.00
12	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	0.68	4.55
13	15.750	13.125	8.00	0.875	4	1.375	1.375	4810	0.75	3.18
14	15.750	13.125	8.00	0.875	4	1.375	1.375	4940	0.68	4.32
15	15.750	13.125	8.00	0.875	4	1.375	1.375	4880	0.75	5.23
16	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	0.70	4.65
17	15.750	13.125	8.00	0.875	4	1.375	1.375	4780	0.70	4.77
18	15.750	13.125	8.00	0.875	4	1.375	1.375	4750	0.77	5.23
19	15.750	13.125	8.00	0.875	4	1.375	1.375	4870	0.73	4.55
20	15.750	13.125	8.00	0.875	4	1.375	1.375	4830	0.75	3.86
21	16.000	13.125	8.00	0.50	12	1.375	1.375	4380	0.66	3.86
22	16.000	13.125	8.00	0.50	12	1.375	1.375	4410	0.74	4.09
23	16.000	13.125	8.00	0.50	12	1.375	1.375	4300	0.65	3.41
24	16.000	13.125	8.00	0.50	12	1.375	1.375	4340	0.64	3.86
25	16.000	13.125	8.00	0.50	12	1.375	1.375	4440	0.80	4.60
26	16.000	13.125	8.00	0.50	12	1.375	1.375	5490	0.64	3.18
27	16.000	13.125	8.00	0.50	12	1.375	1.375	5540	0.65	3.64
28	16.000	13.125	8.00	0.50	12	1.375	1.375	5450	0.66	2.95
29	16.000	13.125	8.00	0.50	12	1.375	1.375	5580	0.77	5.00
30	16.000	13.125	8.00	0.50	12	1.375	1.375	5740	0.74	3.86
31	15.125	13.125	8.00	0.875	4	0.750	0.750	4175	0.73	3.18
32	15.125	13.125	8.00	0.875	4	0.750	0.750	4190	0.69	3.25
33	15.750	13.125	8.00	0.875	4	1.375	1.375	4203	0.77	5.91
34	15.750	13.125	8.00	0.875	4	1.375	1.375	4216	0.70	5.80
35	17.125	13.125	8.00	0.875	4	2.750	1.375	3800	0.77	3.64
36	17.125	13.125	8.00	0.875	4	2.750	1.375	3880	0.89	4.43
37	16.125	13.125	8.00	0.875	4	1.750	1.375	3950	0.68	2.05
38	16.125	13.125	8.00	0.875	4	1.750	1.375	4000	0.76	3.18
39	14.875	13.125	8.00	0.875	4	0.500	1.375	4190	0.66	1.59
40	14.875	13.125	8.00	0.875	4	0.500	1.375	4240	0.73	2.05
41	14.875	13.125	8.00	0.875	4	0.500	0.688	4600	0.86	2.50
42	15.750	13.125	8.00	0.875	4	1.375	1.563	4800	0.89	2.73
43	15.750	13.125	8.00	0.875	4	1.625	3.250	4850	0.73	4.55

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain In steel x 10^{-3}	Crack width (x 10^{-3}) (in)
44	14.875	13.125	8.00	0.875	4	0.500	0.688	4700	0.77	1.93
45	15.750	13.125	8.00	0.875	4	1.375	1.563	4950	0.84	3.18
46	15.750	13.125	8.00	0.875	4	1.625	3.250	5160	0.68	4.77
47	15.750	13.125	8.00	0.875	4	1.375	1.375	2950	0.86	5.57
48	15.750	13.125	8.00	0.875	4	1.375	1.375	2985	0.93	5.34
49	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	0.74	4.32
50	15.750	13.125	8.00	0.875	4	1.375	1.375	2827	0.95	6.59
51	15.750	13.125	8.00	0.875	4	1.375	1.375	3475	0.77	5.56
52	15.750	13.125	8.00	0.875	4	1.375	1.375	4900	0.75	5.00
53	16.000	13.125	8.00	0.500	12	1.375	1.375	4850	0.58	4.32
54	16.000	13.125	8.00	0.500	9	1.375	1.375	4775	0.77	4.32
55	16.000	13.125	8.00	0.500	6	1.375	1.375	5020	0.00	0.00
56	15.750	13.125	8.00	0.875	4	1.375	1.375	5190	0.66	4.43
57	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	0.86	6.02
58	15.750	13.125	8.00	0.625	4	1.375	1.375	4900	0.75	5.57
59	16.000	13.125	8.00	0.500	12	1.375	1.375	4340	0.77	4.10
60	16.000	13.125	8.00	0.500	9	1.375	1.375	4150	0.95	4.32
61	16.000	13.125	8.00	0.500	6	1.375	1.375	4540	0.95	6.14
62	15.750	13.125	8.00	0.875	4	1.375	1.375	4085	0.77	4.77
63	15.750	13.125	8.00	0.875	4	1.375	1.375	4048	0.52	3.64
64	15.750	13.125	8.00	0.875	4	1.375	1.375	4150	0.86	5.45
65	15.750	13.125	8.00	0.875	4	1.375	1.375	4279	0.77	6.36
66	15.750	13.125	8.00	0.875	4	1.375	1.375	3410	0.00	0.00
67	15.750	13.125	8.00	0.875	4	1.375	1.375	4000	0.66	3.98
68	15.750	13.125	8.00	0.875	4	1.375	1.375	4670	0.77	5.68
69	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.73	3.18
71	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.81	4.43
72	16.000	14.375	8.00	0.50	6	1.375	1.375	4110	0.77	2.16
73	16.000	13.125	6.25	0.50	12	1.375	0.500	5210	0.64	1.25
74	16.000	13.125	8.00	0.50	12	1.375	1.375	5300	0.92	2.619
75	16.000	13.125	9.75	0.50	12	1.375	2.250	5340	0.66	2.95
76	16.000	14.375	6.25	0.50	6	1.375	0.500	4040	0.86	1.36
77	16.000	14.375	9.75	0.50	6	1.375	2.250	3940	0.57	2.61

Stage - III

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain in steel x 10^{-3}	Crack width (x 10^{-3}) (in)
1	15.125	13.125	8.00	1.25	2	1.375	1.375	5050	1.00	7.79
2	15.125	13.125	8.00	1.25	2	1.375	1.375	5400	1.09	7.27
3	15.125	13.125	8.00	1.25	2	1.375	1.375	5350	1.14	7.21
4	15.125	13.125	8.00	1.25	2	1.375	1.375	4980	0.98	5.64
5	15.125	13.125	8.00	1.25	2	1.375	1.375	5180	1.02	6.82
6	15.125	13.125	8.00	1.25	2	1.375	1.375	4580	1.07	7.05
7	15.125	13.125	8.00	1.25	2	1.375	1.375	4500	1.02	6.59
8	15.125	13.125	8.00	1.25	2	1.375	1.375	4520	1.07	7.27
9	15.125	13.125	8.00	1.25	2	1.375	1.375	4550	1.06	7.16
10	15.125	13.125	8.00	1.25	2	1.375	1.375	4600	1.07	6.40
11	15.750	13.125	8.00	0.875	4	1.375	1.375	4840	1.05	7.27
12	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.07	7.05
13	15.750	13.125	8.00	0.875	4	1.375	1.375	4810	1.18	5.46
14	15.750	13.125	8.00	0.875	4	1.375	1.375	4940	1.13	7.27
15	15.750	13.125	8.00	0.875	4	1.375	1.375	4880	1.14	6.81
16	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.12	8.75
17	15.750	13.125	8.00	0.875	4	1.375	1.375	4780	1.14	7.27
18	15.750	13.125	8.00	0.875	4	1.375	1.375	4750	1.19	7.05
19	15.750	13.125	8.00	0.875	4	1.375	1.375	4870	1.22	7.73
20	15.750	13.125	8.00	0.875	4	1.375	1.375	4830	1.14	5.68
21	16.000	13.125	8.00	0.50	12	1.375	1.375	4380	1.09	5.91
22	16.000	13.125	8.00	0.50	12	1.375	1.375	4410	1.14	6.25
23	16.000	13.125	8.00	0.50	12	1.375	1.375	4300	1.01	5.23
24	16.000	13.125	8.00	0.50	12	1.375	1.375	4340	1.07	6.36
25	16.000	13.125	8.00	0.50	12	1.375	1.375	4440	1.24	6.59
26	16.000	13.125	8.00	0.50	12	1.375	1.375	5490	1.36	6.82
27	16.000	13.125	8.00	0.50	12	1.375	1.375	5540	1.09	5.34
28	16.000	13.125	8.00	0.50	12	1.375	1.375	5450	1.07	5.23
29	16.000	13.125	8.00	0.50	12	1.375	1.375	5580	1.09	5.23
30	16.000	13.125	8.00	0.50	12	1.375	1.375	5740	1.10	5.00
31	15.125	13.125	8.00	0.875	4	0.750	0.750	4175	1.14	4.32
32	15.125	13.125	8.00	0.875	4	0.750	0.750	4190	1.09	4.32
33	15.750	13.125	8.00	0.875	4	1.375	1.375	4203	1.25	8.64
34	15.750	13.125	8.00	0.875	4	1.375	1.375	4216	1.05	8.18
35	17.125	13.125	8.00	0.875	4	2.750	1.375	3800	1.17	5.45
36	17.125	13.125	8.00	0.875	4	2.750	1.375	3880	1.34	6.36
37	16.125	13.125	8.00	0.875	4	1.750	1.375	3950	1.09	4.55
38	16.125	13.125	8.00	0.875	4	1.750	1.375	4000	1.18	4.77
39	14.875	13.125	8.00	0.875	4	0.500	1.375	4190	1.05	2.50
40	14.875	13.125	8.00	0.875	4	0.500	1.375	4240	0.98	2.73
41	14.875	13.125	8.00	0.875	4	0.500	0.688	4600	1.30	3.86
42	15.750	13.125	8.00	0.875	4	1.375	1.563	4800	1.36	4.77

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain in steel x 10^{-3}	Crack width (x 10^{-3}) (in)
43	15.750	13.125	8.00	0.875	4	1.625	3.250	4850	1.09	6.59
44	14.875	13.125	8.00	0.875	4	0.500	0.688	4700	1.15	3.41
45	15.750	13.125	8.00	0.875	4	1.375	1.563	4950	1.36	4.89
46	15.750	13.125	8.00	0.875	4	1.625	3.250	5160	1.11	7.16
47	15.750	13.125	8.00	0.875	4	1.375	1.375	2950	1.37	7.61
48	15.750	13.125	8.00	0.875	4	1.375	1.375	2985	1.45	8.30
49	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	1.05	5.57
50	15.750	13.125	8.00	0.875	4	1.375	1.375	2827	1.50	9.43
51	15.750	13.125	8.00	0.875	4	1.375	1.375	3475	1.16	7.95
52	15.750	13.125	8.00	0.875	4	1.375	1.375	4900	1.23	6.25
53	16.000	13.125	8.00	0.500	12	1.375	1.375	4850	0.93	6.82
54	16.000	13.125	8.00	0.500	9	1.375	1.375	4775	1.27	7.27
55	16.000	13.125	8.00	0.500	6	1.375	1.375	5020	1.15	7.95
56	15.750	13.125	8.00	0.875	4	1.375	1.375	5190	1.02	7.16
57	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	1.39	10.2
58	15.750	13.125	8.00	0.625	4	1.375	1.375	4900	1.20	8.64
59	16.000	13.125	8.00	0.500	12	1.375	1.375	4340	1.14	6.14
60	16.000	13.125	8.00	0.500	9	1.375	1.375	4150	1.52	7.73
61	16.000	13.125	8.00	0.500	6	1.375	1.375	4540	1.75	12.1
62	15.750	13.125	8.00	0.875	4	1.375	1.375	4085	1.36	7.73
63	15.750	13.125	8.00	0.875	4	1.375	1.375	4048	1.02	6.25
64	15.750	13.125	8.00	0.875	4	1.375	1.375	4150	1.32	7.95
65	15.750	13.125	8.00	0.875	4	1.375	1.375	4279	1.16	9.20
66	15.750	13.125	8.00	0.875	4	1.375	1.375	3410	1.02	5.80
67	15.750	13.125	8.00	0.875	4	1.375	1.375	4000	1.68	11.4
68	15.750	13.125	8.00	0.875	4	1.375	1.375	4670	1.18	7.27
69	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.02	5.68
70	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.23	5.68
71	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.07	5.68
72	16.000	14.375	8.00	0.50	6	1.375	1.375	4110	1.11	3.18
73	16.000	13.125	6.25	0.50	12	1.375	0.500	5210	0.98	1.59
74	16.000	13.125	8.00	0.50	12	1.375	1.375	5300	1.34	3.52
75	16.000	13.125	9.75	0.50	12	1.375	2.250	5340	1.00	5.00
76	16.000	14.375	6.25	0.50	6	1.375	0.500	4040	1.18	1.82
77	16.000	14.375	9.75	0.50	6	1.375	2.250	3940	0.91	3.98

Stage - IV

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain In steel x 10^{-3}	Crack width (x 10^{-3}) (in)
1	15.125	13.125	8.00	1.25	2	1.375	1.375	5050	1.40	9.88
2	15.125	13.125	8.00	1.25	2	1.375	1.375	5400	1.52	7.95
3	15.125	13.125	8.00	1.25	2	1.375	1.375	5350	1.82	9.88
4	15.125	13.125	8.00	1.25	2	1.375	1.375	4980	1.32	8.52
5	15.125	13.125	8.00	1.25	2	1.375	1.375	5180	1.45	9.77
6	15.125	13.125	8.00	1.25	2	1.375	1.375	4580	1.56	10.6
7	15.125	13.125	8.00	1.25	2	1.375	1.375	4500	1.38	10.0
8	15.125	13.125	8.00	1.25	2	1.375	1.375	4520	1.50	10.3
9	15.125	13.125	8.00	1.25	2	1.375	1.375	4550	1.42	10.0
10	15.125	13.125	8.00	1.25	2	1.375	1.375	4600	1.36	8.64
11	15.750	13.125	8.00	0.875	4	1.375	1.375	4840	1.71	10.9
12	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.52	9.55
13	15.750	13.125	8.00	0.875	4	1.375	1.375	4810	1.59	7.50
14	15.750	13.125	8.00	0.875	4	1.375	1.375	4940	1.57	10.7
15	15.750	13.125	8.00	0.875	4	1.375	1.375	4880	1.55	8.86
16	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.48	11.2
17	15.750	13.125	8.00	0.875	4	1.375	1.375	4780	1.50	10.2
18	15.750	13.125	8.00	0.875	4	1.375	1.375	4750	1.65	10.5
19	15.750	13.125	8.00	0.875	4	1.375	1.375	4870	1.66	10.1
20	15.750	13.125	8.00	0.875	4	1.375	1.375	4830	1.57	8.64
21	16.000	13.125	8.00	0.50	12	1.375	1.375	4380	1.48	7.27
22	16.000	13.125	8.00	0.50	12	1.375	1.375	4410	1.52	8.41
23	16.000	13.125	8.00	0.50	12	1.375	1.375	4300	1.36	7.27
24	16.000	13.125	8.00	0.50	12	1.375	1.375	4340	1.47	8.75
25	16.000	13.125	8.00	0.50	12	1.375	1.375	4440	1.70	9.07
26	16.000	13.125	8.00	0.50	12	1.375	1.375	5490	1.82	8.86
27	16.000	13.125	8.00	0.50	12	1.375	1.375	5540	1.39	7.16
28	16.000	13.125	8.00	0.50	12	1.375	1.375	5450	1.52	7.49
29	16.000	13.125	8.00	0.50	12	1.375	1.375	5580	1.44	6.82
30	16.000	13.125	8.00	0.50	12	1.375	1.375	5740	1.57	7.95
31	15.125	13.125	8.00	0.875	4	0.750	0.750	4175	1.57	5.91
32	15.125	13.125	8.00	0.875	4	0.750	0.750	4190	1.48	6.14
33	15.750	13.125	8.00	0.875	4	1.375	1.375	4203	1.68	10.9
34	15.750	13.125	8.00	0.875	4	1.375	1.375	4216	1.44	11.2
35	17.125	13.125	8.00	0.875	4	2.750	1.375	3800	1.77	7.55
36	17.125	13.125	8.00	0.875	4	2.750	1.375	3880	1.82	8.64
37	16.125	13.125	8.00	0.875	4	1.750	1.375	3950	1.49	5.91
38	16.125	13.125	8.00	0.875	4	1.750	1.375	4000	1.61	6.59
39	14.875	13.125	8.00	0.875	4	0.500	1.375	4190	1.48	3.41
40	14.875	13.125	8.00	0.875	4	0.500	1.375	4240	1.36	2.61
41	14.875	13.125	8.00	0.875	4	0.500	0.688	4600	1.76	5.68
42	15.750	13.125	8.00	0.875	4	1.375	1.563	4800	1.82	6.36

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain In steel x 10^{-3}	Crack width (x 10^{-3}) (in)
43	15.750	13.125	8.00	0.875	4	1.625	3.250	4850	1.55	8.86
44	14.875	13.125	8.00	0.875	4	0.500	0.688	4700	1.57	4.32
45	15.750	13.125	8.00	0.875	4	1.375	1.563	4950	1.80	6.36
46	15.750	13.125	8.00	0.875	4	1.625	3.250	5160	1.45	9.32
47	15.750	13.125	8.00	0.875	4	1.375	1.375	2950	1.98	10.2
48	15.750	13.125	8.00	0.875	4	1.375	1.375	2985	2.00	11.9
49	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	1.52	8.64
50	15.750	13.125	8.00	0.875	4	1.375	1.375	2827	1.82	11.2
51	15.750	13.125	8.00	0.875	4	1.375	1.375	3475	1.61	10.2
52	15.750	13.125	8.00	0.875	4	1.375	1.375	4900	1.66	8.52
53	16.000	13.125	8.00	0.500	12	1.375	1.375	4850	0.30	9.55
54	16.000	13.125	8.00	0.500	9	1.375	1.375	4775	1.55	10.0
55	16.000	13.125	8.00	0.500	6	1.375	1.375	5020	1.64	11.1
56	15.750	13.125	8.00	0.875	4	1.375	1.375	5190	1.45	9.55
57	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	1.98	13.4
58	15.750	13.125	8.00	0.625	4	1.375	1.375	4900	0.00	0.00
59	16.000	13.125	8.00	0.500	12	1.375	1.375	4340	1.55	8.64
60	16.000	13.125	8.00	0.500	9	1.375	1.375	4150	0.00	0.00
61	16.000	13.125	8.00	0.500	6	1.375	1.375	4540	0.00	0.00
62	15.750	13.125	8.00	0.875	4	1.375	1.375	4085	1.89	11.6
63	15.750	13.125	8.00	0.875	4	1.375	1.375	4048	1.41	9.55
64	15.750	13.125	8.00	0.875	4	1.375	1.375	4150	1.83	10.8
65	15.750	13.125	8.00	0.875	4	1.375	1.375	4279	1.58	12.8
66	15.750	13.125	8.00	0.875	4	1.375	1.375	3410	1.49	8.64
67	15.750	13.125	8.00	0.875	4	1.375	1.375	4000	2.14	13.7
68	15.750	13.125	8.00	0.875	4	1.375	1.375	4670	1.41	7.95
69	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.41	7.50
70	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.64	8.18
71	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.68	9.09
72	16.000	14.375	8.00	0.50	6	1.375	1.375	4110	1.46	4.20
73	16.000	13.125	6.25	0.50	12	1.375	0.500	5210	1.25	2.27
74	16.000	13.125	8.00	0.50	12	1.375	1.375	5300	1.77	5.00
75	16.000	13.125	9.75	0.50	12	1.375	2.250	5340	1.37	6.59
76	16.000	14.375	6.25	0.50	6	1.375	0.500	4040	1.47	2.27
77	16.000	14.375	9.75	0.50	6	1.375	2.250	3940	0.52	5.23

Stage -V

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain In steel x 10^{-3}	Crack width (x 10^{-3}) (in)
1	15.125	13.125	8.00	1.25	2	1.375	1.375	5050	1.84	12.6
2	15.125	13.125	8.00	1.25	2	1.375	1.375	5400	2.02	11.1
3	15.125	13.125	8.00	1.25	2	1.375	1.375	5350	2.24	13.0
4	15.125	13.125	8.00	1.25	2	1.375	1.375	4980	1.75	9.77
5	15.125	13.125	8.00	1.25	2	1.375	1.375	5180	1.77	11.6
6	15.125	13.125	8.00	1.25	2	1.375	1.375	4580	1.92	12.7
7	15.125	13.125	8.00	1.25	2	1.375	1.375	4500	1.83	13.3
8	15.125	13.125	8.00	1.25	2	1.375	1.375	4520	1.91	12.5
9	15.125	13.125	8.00	1.25	2	1.375	1.375	4550	1.89	13.0
10	15.125	13.125	8.00	1.25	2	1.375	1.375	4600	1.73	11.6
11	15.750	13.125	8.00	0.875	4	1.375	1.375	4840	2.14	14.3
12	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.90	12.5
13	15.750	13.125	8.00	0.875	4	1.375	1.375	4810	2.07	10.7
14	15.750	13.125	8.00	0.875	4	1.375	1.375	4940	2.05	13.4
15	15.750	13.125	8.00	0.875	4	1.375	1.375	4880	2.02	11.6
16	15.750	13.125	8.00	0.875	4	1.375	1.375	4910	1.95	15.7
17	15.750	13.125	8.00	0.875	4	1.375	1.375	4780	1.95	13.0
18	15.750	13.125	8.00	0.875	4	1.375	1.375	4750	2.80	13.9
19	15.750	13.125	8.00	0.875	4	1.375	1.375	4870	2.16	13.0
20	15.750	13.125	8.00	0.875	4	1.375	1.375	4830	2.07	11.1
21	16.000	13.125	8.00	0.50	12	1.375	1.375	4380	1.99	10.2
22	16.000	13.125	8.00	0.50	12	1.375	1.375	4300	1.84	8.75
24	16.000	13.125	8.00	0.50	12	1.375	1.375	4340	1.83	10.4
25	16.000	13.125	8.00	0.50	12	1.375	1.375	4440	0.00	0.00
26	16.000	13.125	8.00	0.50	12	1.375	1.375	5490	2.02	11.6
27	16.000	13.125	8.00	0.50	12	1.375	1.375	5540	1.77	9.09
28	16.000	13.125	8.00	0.50	12	1.375	1.375	5450	1.89	9.43
29	16.000	13.125	8.00	0.50	12	1.375	1.375	5740	1.93	10.5
31	15.125	13.125	8.00	0.875	4	0.750	0.750	4175	2.02	7.73
32	15.125	13.125	8.00	0.875	4	0.750	0.750	4190	1.94	9.75
33	15.750	13.125	8.00	0.875	4	1.375	1.375	4203	2.20	14.1
34	15.750	13.125	8.00	0.875	4	1.375	1.375	4216	2.03	14.5
35	17.125	13.125	8.00	0.875	4	2.750	1.375	3800	2.27	10.0
36	17.125	13.125	8.00	0.875	4	2.750	1.375	3880	2.44	11.6
37	16.125	13.125	8.00	0.875	4	1.750	1.375	3950	1.95	7.84
38	16.125	13.125	8.00	0.875	4	1.750	1.375	4000	2.10	8.64
39	14.875	13.125	8.00	0.875	4	0.500	1.375	4190	1.89	4.43
40	14.875	13.125	8.00	0.875	4	0.500	1.375	4240	2.36	6.14
41	14.875	13.125	8.00	0.875	4	0.500	0.688	4600	2.27	7.27
42	15.750	13.125	8.00	0.875	4	1.375	1.563	4800	2.14	7.5
43	15.750	13.125	8.00	0.875	4	1.625	3.250	4850	1.82	10.5
44	14.875	13.125	8.00	0.875	4	0.500	0.688	4700	2.03	6.02

Beam No.	Depth (in)	eff. Depth (in)	breadth (in)	bar dia (in)	No.	Side cover (in)	bottom cover (in)	Cube streng. (psi)	Strain In steel x 10^{-3}	Crack width (x 10^{-3}) (in)
45	15.750	13.125	8.00	0.875	4	1.375	1.563	4950	2.09	7.5
46	15.750	13.125	8.00	0.875	4	1.625	3.250	5160	1.74	11.1
47	15.750	13.125	8.00	0.875	4	1.375	1.375	2950	2.36	12.8
48	15.750	13.125	8.00	0.875	4	1.375	1.375	2985	0.00	0.00
49	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	1.98	10.0
50	15.750	13.125	8.00	0.875	4	1.375	1.375	2827	1.89	12.3
51	15.750	13.125	8.00	0.875	4	1.375	1.375	3475	2.12	12.2
52	15.750	13.125	8.00	0.875	4	1.375	1.375	4900	2.20	10.5
53	16.000	13.125	8.00	0.500	12	1.375	1.375	4850	1.64	12.3
54	16.000	13.125	8.00	0.500	9	1.375	1.375	4775	1.80	10.5
55	16.000	13.125	8.00	0.500	6	1.375	1.375	5020	0.00	0.00
56	15.750	13.125	8.00	0.875	4	1.375	1.375	5190	1.86	11.9
57	15.750	13.125	8.00	0.875	4	1.375	1.375	4960	0.00	0.00
58	15.750	13.125	8.00	0.625	4	1.375	1.375	4900	0.00	0.00
59	16.000	13.125	8.00	0.500	12	1.375	1.375	4340	1.86	10.0
60	16.000	13.125	8.00	0.500	9	1.375	1.375	4150	0.00	0.00
61	16.000	13.125	8.00	0.500	6	1.375	1.375	4540	0.00	0.00
62	15.750	13.125	8.00	0.875	4	1.375	1.375	4085	0.00	0.00
63	15.750	13.125	8.00	0.875	4	1.375	1.375	4048	1.86	12.3
64	15.750	13.125	8.00	0.875	4	1.375	1.375	4150	0.00	0.00
65	15.750	13.125	8.00	0.875	4	1.375	1.375	4279	0.00	0.00
66	15.750	13.125	8.00	0.875	4	1.375	1.375	3410	0.00	0.00
67	15.750	13.125	8.00	0.875	4	1.375	1.375	4000	0.00	0.00
68	15.750	13.125	8.00	0.875	4	1.375	1.375	4670	1.80	11.6
69	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.00	0.00
70	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	0.00	0.00
71	15.750	13.125	8.00	0.875	4	1.375	1.375	4360	1.95	10.9
72	16.000	14.375	8.00	0.50	6	1.375	1.375	4110	1.80	5.45
73	16.000	13.125	6.25	0.50	12	1.375	0.500	5210	1.61	2.84
74	16.000	13.125	8.00	0.50	12	1.375	1.375	5300	2.30	6.48
75	16.000	13.125	9.75	0.50	12	1.375	2.250	5340	1.66	8.64
76	16.000	14.375	6.25	0.50	6	1.375	0.500	4040	1.81	2.84
77	16.000	14.375	9.75	0.50	6	1.375	2.250	3940	1.82	6.59

HOGNISTAD DATA

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel dia (in)	Steel No. of bars	Steel No. of tiers	Concrete strength (Psi)	Cover side (in)	Cover bottom (in)	Steel stress (Psi)	Crack width (in)
8	16	14.1	1.00	2	1	4570	1.375	1.375	20000	0.003
8	16	14.2	0.75	4	1	3970	1.375	1.375	20000	0.002
8	16	13.6	0.50	8	2	4320	1.375	1.375	20000	0.0025
8	16	13.1	0.375	15	3	3840	1.375	1.375	20000	0.0025
8	16	14.2	0.75	4	1	3855	1.375	1.375	20000	0.0045
8	16	14.1	1.00	2	1	3525	1.375	1.375	20000	0.005
8	16	13.6	0.50	8	2	3250	1.375	1.375	20000	0.0035
8	16	14.1	1.00	1	1	4750	1.375	1.375	20000	0.000
8	16	14.2	0.75	2	1	3940	1.375	1.375	20000	0.000
8	16	13.6	0.50	4	2	4010	1.375	1.375	20000	0.000
8	16	13.1	1.00	4	2	3700	1.375	1.375	20000	0.003
8	16	13.4	0.75	8	2	4470	1.375	1.375	20000	0.003
8	16	14.1	1.00	2	1	7040	1.375	1.375	20000	0.0035
8	16	14.2	0.75	4	1	7240	1.375	1.375	20000	0.003
8	16	13.6	0.50	8	2	7070	1.375	1.375	20000	0.003
8	16	13.1	0.375	15	3	6785	1.375	1.375	20000	0.002
12	16	13.6	0.50	12	2	4730	0.75	1.375	20000	0.0025
8	16	13.6	0.50	8	2	4490	0.75	1.375	20000	0.0025
6	16	13.6	0.50	6	2	4430	0.750	1.375	20000	0.003
4	16	13.6	0.50	4	2	5870	0.750	1.375	20000	0.000
8	24	21.4	0.75	6	2	4320	1.375	1.375	20000	0.0030
8	16	14.2	0.75	4	1	4460	1.375	1.375	20000	0.003
8	12	10.2	0.75	3	1	4430	1.375	1.375	20000	0.002

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel			Concrete strength (Psi)	Cover		Steel stress (Psi)	Crack width (in)
			dia (in)	No. of bars	No. of tiers		side (in)	bottom (in)		
8	08	06.2	0.75	2	1	4110	1.375	1.375	20000	0.002
8	16	15.2	0.875	2	1	4030	1.375	0.375	20000	0.002
8	16	14.2	0.875	2	1	3040	1.375	1.375	20000	0.002
8	16	12.7	0.875	2	1	3640	1.375	2.875	20000	0.0025
8	16	11.2	0.875	2	1	3640	1.375	4.375	20000	0.003
8	16	14.2	0.875	2	1	4360	0.375	1.375	20000	0.002
8	16	14.2	0.875	2	1	4360	2.875	1.375	20000	0.004
8	16	15.2	0.875	2	1	3940	0.375	0.375	20000	0.002
8	16	12.7	0.875	2	1	3940	2.875	2.875	20000	0.006
8	16	14.1	1.00	2	1	4570	1.375	1.375	30000	0.005
8	16	14.2	0.75	4	1	3970	1.375	1.375	30000	0.006
8	16	13.6	0.50	8	2	4320	1.375	1.375	30000	0.005
8	16	13.1	0.375	15	3	3840	1.375	1.375	30000	0.004
8	16	14.2	0.75	4	1	3855	1.375	1.375	30000	0.0055
8	16	14.1	1.00	2	1	3525	1.375	1.375	30000	0.0085
8	16	13.6	0.50	8	2	3250	1.375	1.375	30000	0.0045
8	16	14.1	1.00	1	1	4750	1.375	1.375	30000	0.0055

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel dia (in)	Steel No. of bars	Steel No. of tiers	Concrete strength (Psi)	Cover side (in)	Cover bottom (in)	Steel stress (Psi)	Crack width (in)
8	16	14.2	0.75	2	1	3940	1.375	1.375	30000	0.0045
8	16	13.6	0.50	4	2	4010	1.375	1.375	30000	0.0040
8	16	13.1	1.00	4	2	3700	1.375	1.375	30000	0.006
8	16	13.4	0.75	8	2	4470	1.375	1.375	30000	0.006
8	16	14.1	1.00	2	1	7040	1.375	1.375	30000	0.006
8	16	14.2	0.75	4	1	7240	1.375	1.375	30000	0.006
8	16	13.6	0.50	8	2	7070	1.375	1.375	30000	0.0045
8	16	13.1	0.375	15	3	6785	1.375	1.375	30000	0.0045
12	16	13.6	0.50	12	2	4730	0.750	1.375	30000	0.005
8	16	13.6	0.50	8	2	4490	0.750	1.375	30000	0.003
6	16	13.6	0.50	6	2	4430	0.750	1.375	30000	0.0065
4	16	13.6	0.50	4	2	5870	0.750	1.375	30000	0.004
8	24	21.4	0.75	6	2	4320	1.375	1.375	30000	0.0055
8	16	14.2	0.75	4	1	4460	1.375	1.375	30000	0.006
8	12	10.2	0.75	3	1	4430	1.375	1.375	30000	0.004
8	08	06.2	0.75	2	1	4110	1.375	1.375	30000	0.004
8	16	15.2	0.875	2	1	4030	1.375	0.375	30000	0.005
8	16	14.2	0.875	2	1	3040	1.375	1.375	30000	0.004
8	16	12.7	0.875	2	1	3640	1.375	2.875	30000	0.006
8	16	11.2	0.875	2	1	3640	1.375	4.375	30000	0.006
8	16	14.2	0.875	2	1	4360	0.375	1.375	30000	0.004

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel			Concrete strength (Psi)	Cover		Steel stress (Psi)	Crack width (in)
			dia (in)	No. of bars	No. of tiers		side (in)	bottom (in)		
8	16	14.2	0.875	2	1	4360	2.875	1.375	30000	0.008
8	16	15.2	0.875	2	1	3940	0.375	0.375	30000	0.003
8	16	12.7	0.875	2	1	3940	2.875	2.875	30000	0.008
8	16	14.1	1.00	2	1	4570	1.375	1.375	40000	0.007
8	16	4.2	0.75	4	1	3970	1.375	1.375	40000	0.010
8	16	13.6	0.50	8	2	4320	1.375	1.375	40000	0.007
8	16	13.1	0.375	5	3	3840	1.375	1.375	40000	0.005
8	16	14.2	0.75	4	1	3855	1.375	1.375	40000	0.009
8	16	14.1	1.00	2	1	3525	1.375	1.375	40000	0.0125
8	16	13.6	0.50	8	2	3250	1.375	1.375	40000	0.006
8	16	4.1	1.00	1	1	4750	1.375	1.375	40000	0.0085
8	16	14.2	0.75	2	1	3940	1.375	1.375	40000	0.006
8	16	13.6	0.50	4	2	4010	1.375	1.375	40000	0.007
8	16	13.1	1.00	4	2	3700	1.375	1.375	40000	0.009
8	16	13.4	0.75	8	2	4470	1.375	1.375	40000	0.007
8	16	14.1	1.00	2	1	7040	1.375	1.375	40000	0.009
8	16	4.2	0.75	4	1	7240	1.375	1.375	40000	0.008
8	16	13.6	0.50	8	2	7070	1.375	1.375	40000	0.007
8	16	13.1	0.375	5	3	6785	1.375	1.375	40000	0.0065
12	16	13.6	0.50	12	2	4730	0.75	1.375	40000	0.009
8	16	13.6	0.50	8	2	4490	0.75	1.375	40000	0.005
6	16	13.6	0.50	6	2	4430	0.750	1.375	40000	0.008

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel			Conrete strength (Psi)	Cover		Steel stress (Psi)	Crack width (in)
			dia (in)	No. of bars	No. of tiers		side (in)	bottom (in)		
4	16	13.6	0.50	4	2	5870	0.750	1.375	40000	0.0065
8	24	21.4	0.75	6	2	4320	1.375	1.375	40000	0.009
8	16	14.2	0.75	4	2	4460	1.375	1.375	40000	0.0085
8	12	10.2	0.75	3	1	4430	1.375	1.375	40000	0.006
8	08	06.2	0.75	2	1	4110	1.375	1.375	40000	0.0055
8	16	15.2	0.875	2	1	4030	1.375	0.375	40000	0.0065
8	16	14.2	0.875	2	1	3040	1.375	1.375	40000	0.0065
8	16	12.7	0.875	2	1	3640	1.375	2.875	40000	0.01
8	16	11.2	0.875	2	1	3640	1.375	4.375	40000	0.008
8	16	14.2	0.875	2	1	4360	0.375	1.375	40000	0.006
8	16	14.2	0.875	2	1	4360	2.875	1.375	40000	0.011
8	16	15.2	0.875	2	1	3940	0.375	0.375	40000	0.005
8	16	12.7	0.875	2	1	3940	2.875	2.875	40000	0.011
8	16	14.1	1.00	2	1	4570	1.375	1.375	50000	0.01
8	16	14.2	0.75	4	1	3970	1.375	1.375	50000	0.00
8	16	13.6	0.50	8	2	4320	1.375	1.375	50000	0.0085
8	16	13.1	0.375	15	3	3840	1.375	1.375	50000	0.01
8	16	14.2	0.75	4	1	3855	1.375	1.375	50000	0.012
8	16	14.1	1.00	2	1	3525	1.375	1.375	50000	0.014
8	16	13.6	0.50	8	2	3250	1.375	1.375	50000	0.008
8	16	14.1	1.00	1	1	4750	1.375	1.375	50000	0.0125

Beam Width (in)	Overall depth (in)	Effective depth (in)	Steel			Conrete strength (Psi)	Cover		Steel stress (Psi)	Crack width (in)
			dia (in)	No. of bars	No. of tiers		side (in)	bottom (in)		
8	16	14.2	0.75	2	1	3940	1.375	1.375	50000	0.0085
8	16	13.6	0.50	4	2	4010	1.375	1.375	50000	0.010
8	16	13.1	1.00	4	2	3700	1.375	1.375	50000	0.000
8	16	13.4	0.75	8	2	4470	1.375	1.375	50000	0.000
8	16	14.1	1.00	2	1	7040	1.375	1.375	50000	0.011
8	16	14.2	0.75	4	1	7240	1.375	1.375	50000	0.010
8	16	13.6	0.50	8	2	7070	1.375	1.375	50000	0.009
8	16	13.1	0.375	15	3	6785	1.375	1.375	50000	0.008
12	16	13.6	0.50	12	2	4730	0.75	1.375	50000	0.0105
8	16	13.6	0.50	8	2	4490	0.75	1.375	50000	0.0075
6	16	13.6	0.50	6	2	4430	0.750	1.375	50000	0.010
4	16	13.6	0.50	4	2	5870	0.750	1.375	50000	0.008
8	24	21.4	0.75	6	2	4320	1.375	1.375	50000	0.0105
8	16	14.2	0.75	4	1	4460	1.375	1.375	50000	0.0100
8	12	10.2	0.75	3	1	4430	1.375	1.375	50000	0.008
8	08	06.2	0.75	2	1	4110	1.375	1.375	50000	0.0080
8	16	15.2	0.875	2	1	4030	1.375	0.375	50000	0.0075
8	16	14.2	0.875	2	1	3040	1.375	1.375	50000	0.0085
8	16	12.7	0.875	2	1	3640	1.375	2.875	50000	0.011
8	16	11.2	0.875	2	1	3640	1.375	4.375	50000	0.010
8	16	14.2	0.875	2	1	4360	0.375	1.375	50000	0.007
8	16	14.2	0.875	2	1	4360	2.875	1.375	50000	0.012
8	16	15.2	0.875	2	1	3940	0.375	0.375	50000	0.006
8	16	12.7	0.875	2	1	3940	2.875	2.875	50000	0.015

CLARK DATA

Steel stress = 15,000 PSI

Beam No.	Depth (H) (in)	Width (b) (in)	Eff. depth (in)	No. of bars	bar dia	Cyl. strength (psi)	Crack width (in)
1	6	12	5.31	2	.3	4260	0.00000
2	6	12	5.31	2	.3	4140	0.00000
3	6	08	5.31	2	.3	3390	0.00000
4	6	08	5.31	2	.3	3860	0.00000
5	6	7.5	5.31	2	.3	4180	0.00000
6	6	09	5.31	3	.3	3640	0.00000
7	6	09	5.31	3	.3	3330	0.00000
8	6	7.5	5.31	3	.3	3720	0.00073
9	6	7.5	5.25	2	.4	3550	0.00000
10	6	7.5	5.25	2	.4	3570	0.00109
11	6	7.5	5.25	2	.4	3810	0.00166
12	6	15	5.13	2	.6	3890	0.00226
13	6	9	5.19	2	.5	4300	0.00214
14	6	11	5.13	2	.6	4100	0.00278
15	6	11	5.13	2	.6	3750	0.00266
16	6	15.0	5.06	2	.7	3870	0.00246
17	6	15.0	5.06	2	.7	3850	0.00278
18	6	7.5	5.06	1	.7	3600	0.00342
19	6	7.5	5.06	1	.7	3460	0.00299
20	6	7.5	5.19	2	.5	3330	0.00281
21	6	7.5	5.19	2	.5	3200	0.00290
22	6	7.5	5.19	2	.5	4190	0.00252
23	6	9.0	5.00	1	.8	4500	0.00250
24	6	9.0	5.00	1	.8	3780	0.00370
25	6	6.0	5.00	1	.7	4250	0.00332
26	6	7.5	5.13	2	.6	3980	0.00198
27	6	9.5	5.06	2	.7	3410	0.00484
28	6	9.5	5.06	2	.7	4100	0.00314
29	15	6.0	13.0	1	.8	3690	0.00203
30	15	6.0	13.0	1	.8	3750	0.00264
31	15	6.0	13.38	2	.6	4360	0.00228
32	15	6.0	13.38	2	.6	4240	0.00000
33	15	6.0	13.13	2	.6	3940	0.00308
34	15	6.0	14.06	2	.7	3890	0.00190
35	15	6.0	13.06	2	.7	3250	0.00282
36	15	6.0	13.06	2	.7	4280	0.00241
37	15	6.0	13.06	2	.7	4210	0.00355
38	15	6.0	13.06	2	.7	4040	0.00329
39	15	6.0	12.86	1	1.0	3570	0.00442

40	15	6.0	12.86	1	1.0	4160	0.00513
41	15	6.0	13.00	2	.80	3860	0.00654
42	15	6.0	13.00	2	.80	3870	0.00345
43	15	6.0	13.56	2	.90	4080	0.00235
44	15	6.0	12.94	2	.90	4140	0.00415
45	15	6.0	12.94	2	.90	3950	0.00302
46	15	6.0	12.94	2	.90	4140	0.00292
47	15	6.0	12.94	2	.90	3540	0.00443
48	23	6.0	20.86	1	1.0	3960	0.00331
49	23	6.0	20.86	1	1.0	3620	0.00401
50	23	6.0	20.30	1	1.1	3930	0.00414
51	23	6.0	20.94	2	.90	3650	0.00361
52	23	6.0	20.94	2	.90	3560	0.00332
53	23	6.0	20.80	2	1.1	3590	0.00340
54	23	6.0	20.80	2	1.1	4040	0.00490

Steel stress = 20,000 PSI

Beam No.	Depth (H) (in)	Width (b) (in)	Eff. depth (in)	No. of bars	bar dia	Cyl. strength (psi)	Crack width (in)
1	6	12	5.31	2	.3	4260	0.00
2	6	12	5.31	2	.3	4140	0.000
3	6	08	5.31	2	.3	3390	0.000
4	6	08	5.31	2	.3	3860	0.000
5	6	7.5	5.31	2	.3	4180	0.000
6	6	09	5.31	3	.3	3640	0.0014
7	6	09	5.31	3	.3	3330	0.00175
8	6	7.5	5.31	3	.3	3720	0.00156
9	6	7.5	5.25	2	.4	3550	0.00303
10	6	7.5	5.25	2	.4	3570	0.00146
11	6	7.5	5.25	2	.4	3810	0.00248
12	6	15	5.13	2	.6	3890	0.00477
13	6	9	5.19	2	.5	4300	0.00337
14	6	11	5.13	2	.6	4100	0.00397
15	6	11	5.13	2	.6	3750	0.00444
16	6	15.0	5.06	2	.7	3870	0.00417
17	6	15.0	5.06	2	.7	3850	0.00499
18	6	7.5	5.06	1	.7	3600	0.00608
19	6	7.5	5.06	1	.7	3460	0.00497
20	6	7.5	5.19	2	.5	3330	0.00440
21	6	7.5	5.19	2	.5	3200	0.00432
22	6	7.5	5.19	2	.5	4190	0.00391
23	6	9.0	5.00	1	.8	4500	0.00407
24	6	9.0	5.00	1	.8	3780	0.00496
25	6	6.0	5.00	1	.7	4250	0.00531
26	6	7.5	5.13	2	.6	3980	0.00298
27	6	9.5	5.06	2	.7	3410	0.00749
28	6	9.5	5.06	2	.7	4100	0.00476
29	15	6.0	13.0	1	.8	3690	0.00464
30	15	6.0	13.0	1	.8	3750	0.00459
31	15	6.0	13.38	2	.6	4360	0.00441
32	15	6.0	13.38	2	.6	4240	0.00301
33	15	6.0	13.13	2	.6	3940	0.00369
34	15	6.0	14.06	2	.7	3890	0.00274
35	15	6.0	13.06	2	.7	3250	0.00430
36	15	6.0	13.06	2	.7	4280	0.00362
37	15	6.0	13.06	2	.7	4210	0.00431
38	15	6.0	13.06	2	.7	4040	0.00568
39	15	6.0	12.86	1	1.0	3570	0.00659
40	15	6.0	12.86	1	1.0	4160	0.00739

41	15	6.0	13.00	2	.80	3860	0.00860
42	15	6.0	13.00	2	.80	3870	0.00515
43	15	6.0	13.56	2	.90	4080	0.00342
44	15	6.0	12.94	2	.90	4140	0.00553
45	15	6.0	12.94	2	.90	3950	0.00409
46	15	6.0	12.94	2	.90	4140	0.00444
47	15	**6.0**	12.94	2	.90	3540	0.00404
48	23	**6.0**	20.86	1	1.0	3960	0.00641
49	23	**6**.0	20.86	1	1.0	3620	0.00633
50	23	6.0	20.30	1	1.1	3930	0.00689
51	23	6.0	20.94	2	.90	3650	0.00523
52	23	6.0	20.94	2	.90	3560	0.00369
53	23	6.0	20.80	2	1.1	3590	0.00495
54	23	6.0	20.80	2	1.1	4040	0.00730

Steel stress = 25,000 PSI

Beam No.	Depth (H) (in)	Width (b) (in)	Eff. depth (in)	No. of bars	bar dia	Cyl. strength (psi)	Crack width (in)
1	6	12	5.31	2	.3	4260	0.000
2	6	12	5.31	2	.3	4140	0.000
3	6	08	5.31	2	.3	3390	0.00207
4	6	08	5.31	2	.3	3860	0.00143
5	6	7.5	5.31	2	.3	4180	0.00258
6	6	09	5.31	3	.3	3640	0.00246
7	6	09	5.31	3	.3	3330	0.00340
8	6	7.5	5.31	3	.3	3720	0.00309
9	6	7.5	5.25	2	.4	3550	0.00436
10	6	7.5	5.25	2	.4	3570	0.00226
11	6	7.5	5.25	2	.4	3810	0.00402
12	6	15	5.13	2	.6	3890	0.00669
13	6	9	5.19	2	.5	4300	0.00457
14	6	11	5.13	2	.6	4100	0.00559
15	6	11	5.13	2	.6	3750	0.00627
16	6	15.0	5.06	2	.7	3870	0.00616
17	6	15.0	5.06	2	.7	3850	0.00709
18	6	7.5	5.06	1	.7	3600	0.00806
19	6	7.5	5.06	1	.7	3460	0.00457
20	6	7.5	5.19	2	.5	3330	0.00607
21	6	7.5	5.19	2	.5	3200	0.00589
22	6	7.5	5.19	2	.5	4190	0.00508
23	6	9.0	5.00	1	.8	4500	0.00571
24	6	9.0	5.00	1	.8	3780	0.00671
25	6	6.0	5.00	1	.7	4250	0.00708
26	6	7.5	5.13	2	.6	3980	0.00403
27	6	9.5	5.06	2	.7	3410	0.00531
28	6	9.5	5.06	2	.7	4100	0.00406
29	15	6.0	13.0	1	.8	3690	0.00669
30	15	6.0	13.0	1	.8	3750	0.00769
31	15	6.0	13.38	2	.6	4360	0.00668
32	15	6.0	13.38	2	.6	4240	0.00420
33	15	6.0	13.13	2	.6	3940	0.00504
34	15	6.0	14.06	2	.7	3890	0.00362
35	15	6.0	13.06	2	.7	3250	0.00578
36	15	6.0	13.06	2	.7	4280	0.00465
37	15	6.0	13.06	2	.7	4210	0.00582
38	15	6.0	13.06	2	.7	4040	0.00774
39	15	6.0	12.86	1	1.0	3570	0.00905

40	15	6.0	12.86	1	1.0	4160	0.00941
41	15	6.0	13.00	2	.80	3860	0.01053
42	15	6.0	13.00	2	.80	3870	0.00690
43	15	6.0	13.56	2	.90	4080	0.00454
44	15	6.0	12.94	2	.90	4140	0.00670
45	15	6.0	12.94	2	.90	3950	0.00537
46	15	6.0	12.94	2	.90	4140	0.00589
47	15	6.0	12.94	2	.90	3540	0.00528
48	23	6.0	20.86	1	1.0	3960	0.00920
49	23	6.0	20.86	1	1.0	3620	0.00886
50	23	6.0	20.30	1	1.1	3930	0.00884
51	23	6.0	20.94	2	.90	3650	0.00702
52	23	6.0	20.94	2	.90	3560	0.00509
53	23	6.0	20.80	2	1.1	3590	0.00675
54	23	6.0	20.80	2	1.1	4040	0.00967

Steel stress = 30,000 PSI

Beam No.	Depth (H) (in)	Width (b) (in)	Eff. depth (in)	No. of bars	bar dia	Cyl. strength (psi)	Crack width (in)
1	6	12	5.31	2	.3	4260	0.000
2	6	12	5.31	2	.3	4140	0.000
3	6	08	5.31	2	.3	3390	0.00304
4	6	08	5.31	2	.3	3860	0.00281
5	6	7.5	5.31	2	.3	4180	0.00492
6	6	09	5.31	3	.3	3640	0.00347
7	6	09	5.31	3	.3	3330	0.00414
8	6	7.5	5.31	3	.3	3720	0.00322
9	6	7.5	5.25	2	.4	3550	0.00623
10	6	7.5	5.25	2	.4	3570	0.00329
11	6	7.5	5.25	2	.4	3810	0.00517
12	6	15	5.13	2	.6	3890	0.00708
13	6	9	5.19	2	.5	4300	0.00500
14	6	11	5.13	2	.6	4100	0.00732
15	6	11	5.13	2	.6	3750	0.00816
16	6	15.0	5.06	2	.7	3870	0.00501
17	6	15.0	5.06	2	.7	3850	0.00892
18	6	7.5	5.06	1	.7	3600	0.00615
19	6	7.5	5.06	1	.7	3460	0.00609
20	6	7.5	5.19	2	.5	3330	0.00506
21	6	7.5	5.19	2	.5	3200	0.00752
22	6	7.5	5.19	2	.5	4190	0.00653
23	6	9.0	5.00	1	.8	4500	0.00654
24	6	9.0	5.00	1	.8	3780	0.00840
25	6	6.0	5.00	1	.7	4250	0.00941
26	6	7.5	5.13	2	.6	3980	0.00502
27	6	9.5	5.06	2	.7	3410	0.00639
28	6	9.5	5.06	2	.7	4100	0.00730
29	15	6.0	13.0	1	.8	3690	0.00823
30	15	6.0	13.0	1	.8	3750	0.00793
31	15	6.0	13.38	2	.6	4360	0.00889
32	15	6.0	13.38	2	.6	4240	0.00536
33	15	6.0	13.13	2	.6	3940	0.00632
34	15	6.0	14.06	2	.7	3890	0.00463
35	15	6.0	13.06	2	.7	3250	0.00735
36	15	6.0	13.06	2	.7	4280	0.00558
37	15	6.0	13.06	2	.7	4210	0.00721
38	15	6.0	13.06	2	.7	4040	0.00982
39	15	6.0	12.86	1	1.0	3570	0.01047
40	15	6.0	12.86	1	1.0	4160	0.01130

41	15	6.0	13.00	2	.80	3860	0.01243
42	15	6.0	13.00	2	.80	3870	0.00824
43	15	6.0	13.56	2	.90	4080	0.00572
44	15	6.0	12.94	2	.90	4140	0.00802
45	15	6.0	12.94	2	.90	3950	0.00673
46	15	6.0	12.94	2	.90	4140	0.00632
47	15	6.0	12.94	2	.90	3540	0.00666
48	23	6.0	20.86	1	1.0	3960	0.01170
49	23	6.0	20.86	1	1.0	3620	0.01104
50	23	6.0	20.30	1	1.1	3930	0.01049
51	23	6.0	20.94	2	.90	3650	0.00882
52	23	6.0	20.94	2	.90	3560	0.00626
53	23	6.0	20.80	2	1.1	3590	0.00881
54	23	6.0	20.80	2	1.1	4040	0.00000

Steel stress = 35,000 PSI

Beam No.	Depth (H) (in)	Width (b) (in)	Eff. depth (in)	No. of bars	bar dia	Cyl. strength (psi)	Crack width (in)
1	6	12	5.31	2	.3	4260	0.000
2	6	12	5.31	2	.3	4140	0.00287
3	6	08	5.31	2	.3	3390	0.00464
4	6	08	5.31	2	.3	3860	0.00434
5	6	7.5	5.31	2	.3	4180	0.00523
6	6	09	5.31	3	.3	3640	0.00455
7	6	09	5.31	3	.3	3330	0.00528
8	6	7.5	5.31	3	.3	3720	0.00445
9	6	7.5	5.25	2	.4	3550	0.00774
10	6	7.5	5.25	2	.4	3570	0.00616
11	6	7.5	5.25	2	.4	3810	0.00637
12	6	15	5.13	2	.6	3890	0.00888
13	6	9	5.19	2	.5	4300	0.00615
14	6	11	5.13	2	.6	4100	0.00902
15	6	11	5.13	2	.6	3750	0.00950
16	6	15.0	5.06	2	.7	3870	0.00563
17	6	15.0	5.06	2	.7	3850	0.01093
18	6	7.5	5.06	1	.7	3600	0.00734
19	6	7.5	5.06	1	.7	3460	0.00765
20	6	7.5	5.19	2	.5	3330	0.00618
21	6	7.5	5.19	2	.5	3200	0.00903
22	6	7.5	5.19	2	.5	4190	0.00657
23	6	9.0	5.00	2	.8	4500	0.00793
24	6	9.0	5.00	2	.8	3780	0.01062
25	6	6.0	5.00	2	.7	4250	0.01109
26	6	7.5	5.13	2	.6	3980	0.00651
27	6	9.5	5.06	2	.7	3410	0.00793
28	6	9.5	5.06	2	.7	4100	0.00924
29	15	6.0	13.0	1	.8	3690	0.00947
30	15	6.0	13.0	1	.8	3750	0.00980
31	15	6.0	13.38	2	.6	4360	0.01075
32	15	6.0	13.38	2	.6	4240	0.00649
33	15	6.0	13.13	2	.6	3940	0.00799
34	15	6.0	14.06	2	.7	3890	0.00557
35	15	6.0	13.06	2	.7	3250	0.00883
36	15	6.0	13.06	2	.7	4280	0.00666
37	15	6.0	13.06	2	.7	4210	0.00859
38	15	6.0	13.06	2	.7	4040	0.01170
39	15	6.0	12.86	1	1.0	3570	0.01221
40	15	6.0	12.86	1	1.0	4160	0.01399

41	15	6.0	13.00	2	.80	3860	0.01427
42	15	6.0	13.00	2	.80	3870	0.00958
43	15	6.0	13.56	2	.90	4080	0.00673
44	15	6.0	12.94	2	.90	4140	0.00965
45	15	6.0	12.94	2	.90	3950	0.00000
46	15	6.0	12.94	2	.90	4140	0.00884
47	15	6.0	12.94	2	.90	3540	0.00805
48	23	6.0	20.86	1	1.0	3960	0.01436
49	23	6.0	20.86	1	1.0	3620	0.01289
50	23	6.0	20.30	1	1.1	3930	0.01221
51	23	6.0	20.94	2	.90	3650	0.01061
52	23	6.0	20.94	2	.90	3560	0.00737
53	23	6.0	20.80	2	1.1	3590	0.00000
54	23	6.0	20.80	2	1.1	4040	0.00000

Comparison of International Equations for the Prediction of Maximum width of cracks in R.C. Flexural members (Chapter 6)

HOGNESTAD DATA : **Beam No. 29**

Width of beam = 8 inch
Total depth = 16 inch
Effective depth = 15.2 inch
Size of bars = 7/8 inch
Concrete cylinder comp. strength = 4030 psi
No. of bars = 2
Clear side cover = 1.375 inch
Clear bottom cover =0.375 inch

Maximum crack width (measured on the sides of the beam at the centroid of the reinforcement) corresponding to a steel stress of 50000 psi = 0.0075 inch.

$E_c = 57000 \sqrt{f_c} = 3.62 \times 10^6$ psi (2.496×10^4 N/mm^2)

$m = E_s/E_c = 8.01$ $\quad A_s = 1.203$ in^2

Neutral axis depth from the extreme compression face = 4.965 inch

B.S. EQUATION : $\varepsilon_1 = f_s / E_c = 1.724 \times 10^{-3}$

$$\varepsilon_m = \varepsilon_1 - \frac{b_t\,(h - x)\,(a' - x)}{3\ E_s\ A_s\,(d - x)}$$

$$= 1.6016 x 10^{-3}$$

$$a_{cr} = 1.375 \text{ inch} \qquad c_{min} = 0.375 \text{ inch}$$

$$W_{bt} = \frac{3\ a_{cr}\ \varepsilon_m}{1 + 2\dfrac{(a_{cr} - c_{min})}{(h - x)}}$$

$$= 5.5929x\,10^{-3}\ inch$$

$$Ratio = \frac{W_{cal}}{W_{exp}} = \frac{5.5929x\,10^{-3}}{0.0075} = 0.7457$$

MODEL CODE 1990 EQUATION

$A_{c,ef}(1) = 2.5\ (H\text{-}d)b = 16.0\ in^2$

$A_{c,ef}(2) = ((H\text{-}x)/3)\ b = 29.42\ in^2$

$\therefore\ A_{c,ef} = 16.0\ in^2$

$\rho_{s.ef} = A_s/A_{c,ef}) = 0.0752$

$f_{ctm}(t) = 6\ \sqrt{f'_c} = 380.9$ psi

$\rho_{s\ eff}\ \sigma_{s2} = 0.0752\ x50,000 = 3760$ .....(1)

$f_{ctm}(t)(\ 1 + \alpha_e\ D_{s,ef}) = 380.9(\ 1 + 8.01\ x\ 0.0752)$

$= 610.34$ .....(2)

$$l_{s,max} = \frac{\phi}{3.6\ \rho_{s\,eff}}$$

$$= \frac{(7/8)}{(3.6x0.0752)} = 3.23$$

Here (1) is > (2) therefore stabilised cracking condition has been reached

$$\varepsilon_{sr2} = \frac{f_{ctm}(t)}{\rho_{s,ef}\ x\ E_s} x(1 + \alpha_e\ \rho_{s,ef})$$

$$= 2.8x\,10^{-4}$$

$$\varepsilon_{s2} = \frac{50000}{(2x\,10^5\,x145)} = 1.72\ x\ 10^{-3}$$

β = 0.6 from Table 7.4.2 of Code [13]

$\varepsilon_{s2} - \beta\ \varepsilon_{sr2} = 5.16 \times 10^{-3}$

$W_k = l_{s,max} (\varepsilon_{s2} - \beta\ \varepsilon_{sr2}) = 5.02 \times 10^{-3}$

Ratio $W_{cal} / W_{exp} = 5.02 \times 10^{-3} / 0.0075 = 0.67$

ACI CODE 318 - 1995 EQUATION

$A_{c,ef} = 2\ (h-d)b = 12.8\ in^2$

$A = A_{c,ef}/n = 6.4\ in^2$

$C_s = 1.375 + (0.5 \times 0.375) = 1.8125$

$$W_s = 0.091 \frac{\sqrt[3]{(C_s \ A)}}{1 + \frac{C_s}{(d - x)}} (\sigma_s - 5) x\, 10^{-3}$$

$$= 7.875 x\, 10^{-3} \ \text{inch}$$

$$Ratio = \frac{W_{cal}}{W_{\exp}} = \frac{7.875x\, 10^{-3}}{0.0075} = 1.05$$

CHINESE CODE EQUATION (GBJ 10-89)

$A_{te} = 2(h\text{-}d)b = 12.8\ in^2$

$\rho_{te} = A_s / A_{te} = 0.094$

$f_{ct} = 6 /f'_c = 380.9$ psi

$$\Psi = 1.1 - 0.65 \frac{f_{ct}}{(\rho_{te} \ \ \sigma_s)} = 1.047$$

Therefore $\psi = 1.00$

$$W_{max} = 1.41 \, x 1.00 \, \frac{50000}{2x\,10^5\,x145} \left(2.7\,x0.375 + 0.11\,\frac{0.875}{0.094} \right)^{0.7}$$

$$= 3.99x\,10^{-3} \ \ inch$$

$$Ratio = \frac{W_{cal}}{W_{exp}} = \frac{3.99x\,10^{-3}}{0.0075} = 0.533$$

APPENDIX - III

Prediction of Spacing and Maximum width of cracks in SFRC flexural members

Beam No.10 [Ref. 72] (Crack measured at the extreme tension face of the beam)

$b = 126\ mm\ (4.96\ in.)$

$h = 203.2\ mm\ (8.0\ in.)$

$d = 172.2\ mm\ (6.79\ in.)$

$A_{st} = 237.82\ mm^2$ (2-12mm dia)

$m = E_s/E_c$

$W_{exp} = 0.17\ mm$

$f_{ck} = 28.43\ N/mm^2\ (4122.35\ psi)$

$f_y = 432\ N/mm^2\ (62640\ psi)$

$M/M_w = 1.20$

$E_s = 200 \times 10^3\ N/mm^2$

$E_c = 23120.098\ N/mm^2$

$M_u = 16.48 \times 10^6\ N\text{-}mm$

For a cracked section $x = 59.21\ mm$ (N.A)

$M_w = 8.34 \times 10^6\ N\text{-}mm$

$M = 10.00 \times 10^6\ N\text{-}mm$

$I_{cr} = 34.98 \times 10^6\ mm^4$

$f_s = 279.45\ N/mm^2$

B.S.Equation $\varepsilon_l = (f_s/E_s) . ((h-x)/(d-x)) = 1.78 \times 10^{-3}$

$$\varepsilon_m = \varepsilon_l - \frac{b_t\ (h - x)\ (a' - x)}{3\ E_s\ A_s\ (d - x)}$$

$$= 1.618 \times 10^{-3}$$

$$W_{cal} = \frac{3\ a_{cr}\ \varepsilon_{min}}{1 + 2\ \dfrac{a_{cr} - C_{min}}{h - x}}$$

$$= 0.150mm$$

$$Ratio = \frac{W_{cal}}{W_{exp}} = 0.880$$

Model Code Equation

$$Ae_1 = 9450\ mm^2$$

$$Ae_2 = 6380.64\ mm^2 \qquad \therefore\ Ae = Ae_2$$

$$\rho_{seff} = 0.0246$$

$$f_{ct(nt)} = 2.433\ N/mm^2$$

$$\rho_{seff} . f_s = 5.488 \qquad ...(A3.1)$$

$$f_{ct(nt)} .(1 + m.\ \rho_{seff}) = 3.17 \qquad ...(A3.2)$$

since $(A3.1) > (A3.2)$ *Stabilised Cracking Condition has reached*

$$l_{s\ \max} = \frac{\phi}{3.6x\ \rho_{s,eff}} = 85.47mm$$

$$\varepsilon_{sr2} = \frac{f_{ct(nt)}\ (1 + m\ x\ \rho_{s,eff})}{\rho_{s,eff}\ x\ E_s}$$

$$= 4.06\ x\ 10^{-4}$$

$$\varepsilon_{S2} = 1.397\ x\ 10^{-3}$$

From Code $\beta = 0.6$; $\varepsilon_{s2} - \beta\ \varepsilon_{sr2} = 1.1534\ x\ 10^{-3}$

$$W_k = (l_{(s\ max)}\ x\ (\varepsilon_{s2} - \beta\varepsilon_{sr2})) = 0.10\ mm$$

$$Ratio = W_k/W_{exp} = 0.588$$

Gergely Lutz Equation

$f_s = 279.4\ N/mm^2$ (40513 psi)

$T_s = 31$ mm (1.22 in)

$H_1 = 112.99$ mm (4.45 in)

$A_e \quad 7812\ mm^2$

$A = A_e / n = 3906\ mm^2 \ (6.05\ in^2)$

$$W_{cal} = 0.0913 \sqrt[3]{T_s . A}\ x \frac{(h - x)}{(d - x)}\ x\ (f_s - 5000)\ 10^{-6}$$

(FPS units)

$W_{cal} = 8.016\ x\ 10^{-3}\ in\ \ (0.2036\ mm)$

Ratio $W_{cal} / W_{exp} = 1.198$

Desayi and Ganesan Equation

$f_{cr} = 0.7\ (f_{ck})^{0.5} = 3.732\ N/mm^2$

For the gross cross section

$$D_n = \frac{\frac{bh^2}{2} + (m-1)\ A_{st}\ (h - d)}{bh + (m-1)\ A_{st}}$$

$= 96.92\ mm$

$$I_{gross} = \frac{bh^3}{12} + bh\ (\frac{h}{2} - D_n)^2 + A_{st}\ (m-1)\ (d_n - (h-d))^2$$

$= 99.6\ x\ 10^6\ mm^4$

$$M_{cr} = f_{cr} \frac{I_g}{d_n} = 3.47 x\ 10^6 \quad N - mm$$

$A_{e1} = 7812\ mm^2$

$A_{e2} = 18142.74\ mm^2$

$\therefore \quad A_e = 7812\ mm^2$

$$a_m = \frac{k_t\ f_{ct}\ A_e}{k_b \left[\dfrac{M_{cr}}{M_u}\right]^{\gamma} f_{bu}\ \Sigma\ (\pi\phi)}$$

$k_t = k_b = 2/3 \quad and\ \gamma = 0.35$

$f_{ct} = 2.27\ N/mm^2$

f_{bu} (from CP 110-1972) $= 3.3856\ N/mm^2$

$\therefore\ a_m = 119.77\ mm$

$\varepsilon_s = f_s/E_s = 1.397 \times 10^{-3}$

$$W_{cal} = a_m\ \varepsilon_s \times \frac{(h-x)}{(d-x)} = 0.212\ mm$$

Ratio $W_{cal}/W_{exp} = 1.247$

Modification proposed

$$\varepsilon_{s(corr)} = \varepsilon_s - (m_1 X + c_1)$$

$$where \quad X = \frac{b\ (h-x)\ f_{ct}\ A_f\ V_f}{A_s\ f_s \times 10^6} = 5.51 \times 10^{-5}$$

$m_1 = 1.414$

$c_1 = 2.873 \times 10^{-4}$

$\varepsilon_{s(corr)} = 1.032 \times 10^{-3}$

$$W_{cal} = a_m\ \varepsilon_{s,corr} \frac{(h-x)}{(d-x)}$$

$$= 0.157\ mm$$

Ratio $W_{cal}/W_{exp} = 0.93$

....

Experimental Crack widths obtained from Graph. (Ref. [83])

Sl No.	Beam No	M/M_w	$W_{(exp)}$ mm
1	1	1.04	0.07
2	1	1.38	0.09
3	1	1.72	0.10
4	1	1.85	0.31
5	2	0.80	0.05
6	2	1.28	0.22
7	2	1.59	0.30
8	2	1.90	0.35
9	3	0.84	0.06
10	3	0.97	0.10
11	3	1.12	0.14
12	3	1.24	0.18
13	3	1.35	0.22
14	3	1.50	0.29
15	3	1.58	0.34
16	3	1.66	0.40
17	3	1.75	0.45
18	4	0.85	0.05
19	4	1.20	0.03
20	4	1.44	0.23
21	4	1.70	0.25
22	4	1.90	0.33
23	7	0.96	0.05
24	7	1.05	0.17
25	7	1.20	0.24

Sl No.	Beam No	M/M_w	$W_{(exp)}$ mm
26	7	1.40	0.30
27	7	1.58	0.35
28	8	0.90	0.10
29	8	1.05	0.15
30	8	1.22	0.20
31	8	1.40	0.22
32	8	1.56	0.30
33	8	1.77	0.40
34	9	0.84	0.06
35	9	0.97	0.10
36	9	1.12	0.14
37	9	1.24	0.18
38	9	1.35	0.22
39	9	1.50	0.29
40	9	1.58	0.34
41	9	1.66	0.40
42	9	1.75	0.45
43	10	0.83	0.05
44	10	1.05	0.13
45	**10**	**1.20**	**0.17**
46	10	1.30	0.25
47	10	1.43	0.27
48	10	1.56	0.30
49	10	1.66	0.35
50	10	0.74	0.03

Sl No.	Beam No	M/M_w	$W_{(exp)}$ mm
51	14	0.90	0.05
52	14	1.10	0.13
53	14	1.25	0.17
54	14	1.42	0.22
55	14	1.60	0.30
56	15	1.28	0.10
57	15	1.45	0.13
58	15	1.63	0.17
59	15	1.80	0.30
60	16	0.62	0.03
61	16	1.08	0.19
62	16	1.26	0.25
63	17	0.84	0.06
64	17	0.97	0.10
65	17	1.12	0.14
66	17	1.24	0.18
67	17	1.35	0.22
68	17	1.50	0.29
69	17	1.58	0.34
70	17	1.66	0.40
71	17	1.70	0.45
72	18	1.10	0.13
73	18	1.27	0.18
74	18	1.41	0.23
75	18	1.58	0.25

Sl No.	Beam No	M/M_w	$W_{(exp)}$ mm
76	18	1.72	0.30
77	18	1.85	0.34
78	18	1.98	0.38
79	21	1.44	0.02
80	21	0.55	0.03
81	21	0.76	0.10
82	21	0.92	0.13
83	21	1.20	0.22
84	21	1.30	0.22
85	21	1.45	0.23
86	21	1.65	0.27
87	45	0.64	0.05
88	45	0.82	0.12
89	45	1.12	0.17
90	45	1.40	0.24
91	45	1.68	0.28
92	46	0.55	0.03
93	46	0.75	0.07
94	46	0.95	0.10
95	46	1.15	0.12
96	46	1.33	0.13
97	46	1.70	0.24

APPENDIX -IV

Prediction of Spacing and Maximum width of cracks in Latex modified SFRC flexural members

Beam No.14 (Width of Crack measured at the extreme tension face of the beam)

$b = 125\ mm$	$f_{ck} = 21.79\ N/mm^2$
$h = 200\ mm$	$f_y = 470\ N/mm^2$
$d = 169\ mm$	$A_{st} = 353\ mm^2$ (3 -12.24 mm dia)
$E_s = 2.127 \times 10^5\ N/mm^2$	$m = E_s/E_c$
$W_{exp} = 0.06\ mm$	$M_u = 25.81\ KNM$

$Pcr = 14.72\ KN$ (load at which the width of crack is measured)

$$E_c = 57{,}000\sqrt{0.8\,F_{ck}} \quad \text{......(FPS units)}$$

$$F_{ck} = 21.79\ N/mm^2 \quad or \quad 3159.55\ \ PSI$$

$$\text{Therefore}\quad E_c = 2865711.4\ PSI \quad or \quad 19{,}763.54\ \ N/mm^2$$

$$\text{Modular ratio}\ (m) = E_s/E_c = 10.76$$

$$f_{cr} = 0.7\,(f_{ck})^{0.5} = 3.27\ N/mm^2$$

For the cracked section depth of NA is calculated as :

$$\frac{bn^2}{2} = m\ A_{st}\ (d - n)$$

$$n = 68.18\ mm$$

$$M_{cr} = 0.5 \times P_{cr} \times 0.75 = 5.52\ KNM \ or\ 5.52 \times 10^6\ N\text{-}mm$$

$$I_{cr} = \frac{bn^3}{3} + A_{st}\ m\,(d - n)^2$$

$$= 42.49 \times 10^6\ mm^4$$

$$f_s = m \frac{M_{cr}}{I_{cr}} (d-n)$$

$$= 106.00 \ N/mm^2$$

Strain in steel (ε_s) = f_s / E_s = 5.022×10^{-4}

Experimental Strain ($\varepsilon_{s\ cal}$) = 0.3×10^{-3}

$$A_e = 2((h-d)\,b) = 7750 \ mm^2$$

$$a_m = \frac{k_t \, f_{ct} \, A_e}{k_b \left[\dfrac{M_{cr}}{M_u}\right]^{\gamma} f_{bu} \, \Sigma \, (\pi\phi)}$$

$k_t = k_b = 2/3$ and $\gamma = 0.35$

$f_{ct} = f_c / 10 = 2.17 \ N/mm^2$

f_{bu} *(from CP 110-1972)* = $3.35 \ N/mm^2$

therefore $a_m = 129.15 \ mm$

$$W_{cal} = a_m \, \varepsilon_s \frac{(h-x)}{(d-x)} = 0.084 \ mm$$

Ratio W_{cal} / W_{exp} **= 0.084 / 0.06 = 1.4**

Modification proposed

$$\varepsilon_{s(corr)} = \varepsilon_{s(cal)} - (14000\, F^2 - 5.0\, F + 5.5 \times 10^{-4})$$
$$= 0.3967 \times 10^{-3}$$

where

$$F = \frac{b(h - n)\, f_{ct}\, A_f\, L_i V_f}{A_{st}\, f_s\, 10^6}$$

$$F = 0.22445 \times 10^{-4}$$

and

$$L_i = 1 + \frac{1}{1.5 \times DRC} = 134.33$$

$$W_{cal} = a_m\, \varepsilon_{s,corr} \frac{(h - x)}{(d - x)}$$

$$= 0.067 \text{ mm}$$

Ratio W_{cal}/W_{exp} = 1.12

....

Test results of latex modified SFRC Flexural members obtained from the test results

Beam No.	F_{ck} N/mm^2	Ultimate Moment (KN-M)	P_{cr} KN	DRC (%)	V_f (%)	Strain (exp)	W_{exp} (mm)
8	32.54	25.28	19.62	0.50	0.50	0.25E-03	0.04
8	32.54	25.28	29.43	0.50	0.50	0.50E-03	0.06
8	32.54	25.28	39.24	0.50	0.50	0.65E-03	0.08
8	32.54	25.28	49.05	0.50	0.50	0.11E-02	0.12
8	32.54	25.28	58.86	0.50	0.50	0.16E-02	0.14
9	28.86	24.80	19.62	1.00	0.50	0.35E-03	0.05
9	28.86	24.80	29.43	1.00	0.50	0.60E-03	0.06
9	28.86	24.80	39.24	1.00	0.50	0.80E-03	0.08
9	28.86	24.80	49.05	1.00	0.50	0.11E-02	0.10
9	28.86	24.80	58.86	1.00	0.50	0.16E-02	0.12
10	22.35	23.56	19.62	1.50	0.50	0.30E-03	0.02
10	22.35	23.56	29.43	1.50	0.50	0.70E-03	0.04
10	22.35	23.56	39.24	1.50	0.50	0.80E-03	0.06
10	22.35	23.56	49.05	1.50	0.50	0.10E-02	0.08
10	22.35	23.56	58.86	1.50	0.50	0.14E-02	0.11
11	29.42	24.88	19.62	0.50	1.00	0.40E-03	0.04
11	29.42	24.88	29.43	0.50	1.00	0.60E-03	0.06
11	29.42	24.88	39.24	0.50	1.00	0.85E-03	0.08
11	29.42	24.88	49.05	0.50	1.00	0.12E-02	0.12
12	21.79	23.42	19.62	1.00	1.00	0.35E-03	0.02
12	21.79	23.42	29.43	1.00	1.00	0.55E-03	0.05
12	21.79	23.42	39.24	1.00	1.00	0.85E-03	0.07
12	21.79	23.42	49.05	1.00	1.00	0.12E-02	0.08
12	21.79	23.42	58.86	1.00	1.00	0.16E-02	0.16
13	20.08	22.95	14.72	1.50	1.00	0.30E-03	0.02
13	20.08	22.95	29.43	1.50	1.00	0.55E-03	0.04
13	20.08	22.95	39.24	1.50	1.00	0.85E-03	0.06
13	20.08	22.95	49.05	1.50	1.00	0.11E-02	0.09

….. contd

Beam No.	F_{ck} N/mm^2	Ultimate Moment (KN-M)	P_{cr} KN	DRC (%)	V_f (%)	Strain (exp)	W_{exp} (mm)
13	20.08	22.95	58.86	1.50	1.00	0.14E-02	0.11
14	37.91	25.81	14.72	0.50	1.50	0.30E-03	0.06
14	37.91	25.81	24.53	0.50	1.50	0.50E-03	0.10
14	37.91	25.81	29.43	0.50	1.50	0.65E-03	0.12
14	37.91	25.81	39.24	0.50	1.50	0.85E-03	0.14
14	37.91	25.81	49.05	0.50	1.50	0.10E-02	0.16
14	37.91	25.81	58.86	0.50	1.50	0.13E-02	0.18
15	32.53	25.28	14.72	1.00	1.50	0.35E-03	0.04
15	32.53	25.28	19.62	1.00	1.50	0.50E-03	0.06
15	32.53	25.28	29.43	1.00	1.50	0.70E-03	0.10
15	32.53	25.28	39.24	1.00	1.50	0.85E-03	0.12
15	32.53	25.28	49.05	1.00	1.50	0.11E-02	0.14
15	32.53	25.28	58.86	1.00	1.50	0.14E-02	0.16
16	27.72	24.63	14.72	1.50	1.50	0.45E-03	0.02
16	27.72	24.63	19.62	1.50	1.50	0.50E-03	0.04
16	27.72	24.63	29.43	1.50	1.50	0.75E-03	0.08
16	27.72	24.63	39.24	1.50	1.50	0.90E-03	0.08
16	27.72	24.63	49.05	1.50	1.50	0.11E-02	0.10
16	27.72	24.63	58.86	1.50	1.50	0.14E-02	0.12

Note : 1. Strain values obtained from the rotation meter at the level of steel

2. Ultimate Moment Calculated from Paramasivam etal method in the

Comparison of calculated crack width with experimental crack width of latex modified SFRC Flexural members after modification

Beam No	W_{cal} (mm)	W_{exp} (mm)	Ratio W_{cal} / W_{exp}
8	0.037	0.040	0.92
8	0.064	0.060	1.07
8	0.090	0.080	1.13
8	0.114	0.120	0.95
8	0.136	0.140	0.97
9	0.023	0.050	0.46
9	0.051	0.060	0.86
9	0.076	0.080	0.95
9	0.098	0.100	0.98
9	0.118	0.120	0.98
10	0.015	0.020	0.75
10	0.038	0.040	0.95
10	0.058	0.060	0.96
10	0.075	0.080	0.94
10	0.091	0.110	0.83
11	0.045	0.040	1.13
11	0.066	0.060	1.10
11	0.087	0.080	1.09
11	0.107	0.120	0.89
12	0.020	0.020	1.00
12	0.041	0.050	0.81
12	0.059	0.070	0.84
12	0.075	0.080	0.94
12	0.090	0.160	0.56
13	0.004	0.020	0.21
13	0.036	0.040	0.89
13	0.053	0.060	0.88
13	0.068	0.090	0.76

Beam No	W cal (mm)	W exp (mm)	Ratio W cal / W exp
13	0.082	0.110	0.75
14	**0.067**	**0.060**	**1.12**
14	0.091	0.100	0. 14
14	0.103	0.120	0.85
14	0.125	0.140	0. 14
14	0.148	0.160	0.93
14	0.171	0.180	0.95
15	0.034	0.040	0.86
15	0.046	0.060	0.76
15	0.070	0.100	0.70
15	0.094	0.120	0.78
15	0.117	0.140	0.83
15	0.138	0.160	0.86
16	0.016	0.020	0.82
16	0.029	0.040	0.73
16	0.054	0.080	0.67
16	0.076	0.080	0.95
16	0.096	0.100	0.96
16	0.115	0.120	0.96

No. of test results	:	47
Average Wcal/Wexp	:	0.87
Std. Deviation	:	0.17
Coefficient of variation	:	19.20