



University of Bahrain
College of Engineering
Department of Civil and Architectural Engineering

**PARTIAL USE OF HIGH PERFORMANCE
CONCRETE IN RECTANGULAR
REINFORCED CONCRETE
BEAMS**

BY

HUSSAIN S. M. FALAH AL-OMARI



Thesis Submitted in Partial Fulfillment of the Requirement for the Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

1998

DECLARATION

I declare that this work is the result of my own investigations and that it has neither been accepted nor being submitted in substance for any other degree.

Signed:



Hussain S. M. Falah. Al-Omari
(Candidate)

Date: 23. 11. 1998

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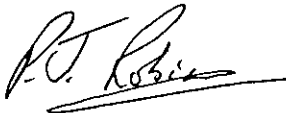
Submitted by Hussain S. Al-Omari to the Department of Civil and Architectural Engineering satisfies the requirement for the Master of Science degree in Civil Engineering.

Oral Examining Committee:



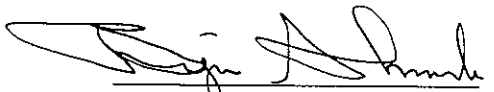
Prof. M. P. Saka
(Supervisor)

Department of Civil and Architectural
Engineering, University of Bahrain



Dr. Peter Robins
(External Examiner)

Department of Civil Building Engineering
Loughborough University
UK



Dr. Bashar Ahmadi
(Internal Examiner)

Department of Civil and Architectural
Engineering, University of Bahrain

AKNOWLEDGMENT

This work would not have been possible without the help given so generously by so many persons. It is not possible for me to thank each and every individual here. However I wish to specially mention members of the faculty of Bahrain University, my colleagues at work and my family. Amongst faculty members I would like to express my sincere gratitude, thanks and appreciation to Prof. Dr. M. P. Saka who guided me in my thesis for his valuable comments and continuous support, and Dr. F. Malhas for initiating this work. Thanks to my superiors and colleagues at work, for their enormous support directly or indirectly, my parents and my wife were very supportive and patient when I needed them most. They encouraged me and gave me moral support when required. I can not be grateful enough to them.

Hussain S. AL-Omari

To my parents, my wife and my children

ABSTRACT

This study presents an experimental study of the behavior of rectangular reinforced concrete beams made out of different overlays. For each overlay two types of concrete grade is used. One is a normal concrete with the compressive strength of $f'_c = 44 \text{ N/mm}^2$ and the other is a high performance concrete with the compressive strength of $f'_c = 65 \text{ N/mm}^2$.

The aim of the study is to investigate the effect of the partial use of high performance concrete in normal concrete beams as different layers. For this purpose, normal beams were casted having different parts made out of high performance concrete. The investigation covered the effect of this combination on the cracking moment, yielding moment, the ultimate moment of the beam. Furthermore, the mid-span deflection, ductility, crack propagation and pattern were also studied. In order to carry out these investigations and evaluate the above parameters, sixty specimen beams were prepared with a cross section of 200 mm x 350 mm and 2 m long span. High performance concrete was used as a top overlay, bottom overlay and middle segment in the normal beam. Specimens were divided into four main groups. In the first group HPC was placed on the top layer of 120 mm. In the second, the bottom layer of 120 mm was casted as HPC. The third group was fully made of normal concrete. Finally the fourth group of beams were made using HPC in the middle segment of the normal concrete beam. Each group consisted of 18 beams, which was divided into six subgroups each having a different steel ratio.

The experiments carried out on sixty specimens beams have shown that the tension overlay being high performance concrete increases the cracking moment, reduces the mid-span deflection at service load in beams with low steel ratios. However, no improvement was observed on the

ductility of those beams. In beams with compression overlay being high performance concrete the ductility factor was improved 43% while reduction was observed in the mid-span deflection. A reasonable improvement in ductility, mid-span deflection and in cracking moment was reduced in beams with middle segment is made out of high performance concrete.

TABLE OF CONTENTS

CHAPTER ONE INTRODUCTION

1.1.	HIGH PERFORMANCE CONCRETE (HPC)	1
1.2.	ADVANTEGES OF USING HPC	2
1.2.1.	ECONOMY	3
1.2.2.	DURABILITY	4
1.2.3.	MATERIAL PROPERTIES	5
1.3.	COMPOSITE BEAMS AND STRUCTURES	8
1.3.1.	COMPOSITE ACTION	8
1.3.2.	SHEAR TRANSFER IN COMPOSITE ACTION	9
1.4.	LITERATURE REVIEW	10
1.5.	PARTIAL USE OF HPC	14
1.6.	OBJECTIVE OF THIS STUDY	15

CHAPTER TWO BEHAVIOR OF REINFORCED CONCRETE BEAMS WITH HPC OVERLAYS (PRELIMINARY STUDY)

2.1.	OBJECTIVES OF THE PRELIMINARY STUDY	17
2.2.	BASIC ASSUMPTIONS	19
2.3.	THEORETICAL CALCULATIONS	19
2.3.1.	MOMENT CURVATURE	19
2.3.2.	COMPUTER PROGRAM	20
2.4.	ANALYSIS AND DISCUSSIONS	21
2.4.1.	CRACKING MOMENT	21
2.4.2.	MID-SPAN DEFLECTION	26
2.4.3.	DUCTILITY	26
2.5.	CONCLUSIONS	27

CHAPTER THREE

EXPERIMENTAL PROGRAM

3.1.	INTRODUCTION	29
3.2.	PREPARING OF STEEL AND SHUTTER	30
3.3.	CONCRETING AND CURING	31
3.4.	COMPRESSIVE TESTS	33
3.4.1.	TESTING PROCEDURE	34
3.5.	TENSILE TESTS	36
3.5.1.	TESTING MACHINE	36
3.5.2.	TESTING PROCEDURE AND RESULTS	37
3.6.	FLEXURAL TESTING	37
3.6.1.	TEST PROCEDURE	38
3.7.	TEST PROCEDURE AND DATA RESULTS	41
3.7.1.	GROUP 1 : BOTTOM OVERLAY BEING HPC	41
3.7.1.1.	TEST ARRANGEMENT AND DATA RESULTS OF GR. 1	42
3.7.2.	GROUP 2 : TOP OVERLAY BEING HPC.	50
3.7.2.1.	TEST ARRANGEMENT AND DATA RESULTS OF GR. 2	51
3.7.3.	GROUP 3 : MIDDLE SEGMENT BEING HPC	58
3.7.3.1.	TEST ARRANGEMENT AND DATA RESULT OF GR.3	59
3.7.4.	GROUP 4: NORMAL CONCRETE BEAMS	66
3.7.4.1.	TEST ARRANGEMENT AND DATA RESULTS OF GR. 4	67

CHAPTER FOUR

DISCUSSION AND ANALYSIS

4.1.	INTRODUCTION	74
4.2.	THE EFFECT OF STEEL CONTENT ON THE BEAM BEHAVIOR USING DIFFERENT TYPE OF OVERLAYS	74
4.3.	THE EFFECT OF OVERLAYS ON THE BEAM PROPERTIES	81
4.4.	MODE OF FAILURE, CRACK DEFORMATION AND PATTERN	95
4.4.1.	BEAMS WITH BOTTOM OVERLAYS	95
4.4.2.	BEAMS WITH TOP OVERLAYS	97
4.4.3.	MIDDLE SEGMENT BEING HPC	99
4.4.4.	NORMAL CONCRETE BEAMS	101

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

5.1.	BOTTOM OVERLAYS BEING HPC	103
5.2.	TOP OVERLAYS BEING HPC	104
5.3.	MIDDLE SEGMENT BEING HPC	105
5.4.	COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS	105
5.5.	COST IMPLICATION OF PARTIAL USE OF HPC	106

REFERENCES	107
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APPENDICES

APPENDIX A	EXPERIMENTAL DATA RESULTS	110
APPENDIX B	COMPUTER PROGRAM “RESPONSE”	170
APPENDIX C	NUMERICAL EXAMPLES	193

LIST OF TABLES

CHAPTER 1

Table 1.1	Material properties as given in ref.[19]	7
-----------	--	---

CHAPTER 2

Table 2.1	Different reinforcement ratio considered for each group	18
Table 2.2	Mid-span deflection at service load	26
Table 2.3	Ductility factors	27

CHAPTER 3

Table 3.1	Groups of reinforcement	31
Table 3.2	Concrete mix design	32
Table 3.3	Compressive strength of cylinders specimens	35
Table 3.4	Experimental tensile properties of steel bars	37
Table 3.5	Details of specimens (bottom overlay)	42
Table 3.6	Average value of strain, curvature and mid-span deflection for beams B11,B21 and B31	44
Table 3.7	Average value of strain, curvature and mid-span deflection for beams B12,B22 and B32	45
Table 3.8	Average value of strain, curvature and mid-span deflection for beams B13,B23 and B33	46
Table 3.9	Average value of strain, curvature and mid-span deflection for beams B14,B24 and B34	47
Table 3.10	Average value of strain, curvature and mid-span deflection for beams B15,B25 and B35	48
Table 3.11	Average value of strain, curvature and mid-span deflection for beams B16,B26 and B36	49
Table 3.12	Details of specimens (top overlay)	51
Table 3.13	Average value of strain, curvature and mid-span deflection for beams T11,T21 and T31	52
Table 3.14	Average value of strain, curvature and mid-span deflection for beams T12,T22 and T32	53
Table 3.15	Average value of strain, curvature and mid-span deflection for beams T13,T23 and T33	54
Table 3.16	Average value of strain, curvature and mid-span deflection for beams T14,T24 and T34	55
Table 3.17	Average value of strain, curvature and mid-span deflection for beams T15,T25 and T35	56
Table 3.18	Average value of strain, curvature and mid-span deflection for beams T16,T26 and T36	57
Table 3.19	Details of specimens (middle segment)	59

Table 3.20	Average value of strain, curvature and mid-span deflection for beams M11,M21 and M31	60
Table 3.21	Average value of strain, curvature and mid-span deflection for beams M12,M22 and M32	61
Table 3.22	Average value of strain, curvature and mid-span deflection for beams M13,M23 and M33	62
Table 3.23	Average value of strain, curvature and mid-span deflection for beams M14,M24 and M34	63
Table 3.24	Average value of strain, curvature and mid-span deflection for beams M15,M25 and M35	64
Table 3.25	Average value of strain, curvature and mid-span deflection for beams M16,M26 and M36	65
Table 3.26	Details of specimens (normal concrete beams)	66
Table 3.27	Average value of strain, curvature and mid-span deflection for beams N11	68
Table 3.28	Average value of strain, curvature and mid-span deflection for beams N22	69
Table 3.29	Average value of strain, curvature and mid-span deflection for beams N33	70
Table 3.30	Average value of strain, curvature and mid-span deflection for beams N44	71
Table 3.31	Average value of strain, curvature and mid-span deflection for beams N55	72
Table 3.32	Average value of strain, curvature and mid-span deflection for beams N66	73

CHAPTER 4

Table 4.1	Mid-span deflection of normal concrete beams at service load compared to beams with different overlay	80
Table 4.2	The cracking moment (M_{cr}) in kN.m as obtained from the experiments for each group of reinforcement	84
Table 4.3	Yielding moment (M_y) in kN.m as obtained from the experiments	89
Table 4.4	Yielding moment (M_u) in kN.m as obtained from the experiments	89
Table 4.5	Curvature at ultimate (ϕ_u) / curvature at yielding (ϕ_y) as obtained from the experiments	89
Table 4.6	Ductility factor as calculated experimentally	90
Table 4.7	Mid-span deflection (δ) in mm as obtained experimentally at service load	94

CHAPTER 5

Table 5.1	Comparison of theoretical and experimental results	105
-----------	--	-----

LIST OF FIGURES

CHAPTER 1

Fig. 1.1	Stress –Strain curves for beams with different type of concrete	6
----------	---	---

CHAPTER 2

Fig. 2.1	Beam loading	18
Fig. 2.2	Cross section of reinforced concrete beams	18
Fig. 2.3	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.0046$)	23
Fig. 2.4	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.0057$)	23
Fig. 2.5	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.0066$)	24
Fig. 2.6	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.0088$)	24
Fig. 2.7	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.01$)	25
Fig. 2.8	Comparison of moment –curvature curves for different type of beams with similar steel ratio ($\rho = 0.026$)	25

CHAPTER 3

Fig. 3.1	Grouping of shutter	30
Fig. 3.2	Concrete of bottom layer (stage one)	33
Fig. 3.3	Capping of cylinders specimens	34
Fig. 3.4	Compressive strength testing machine	35
Fig. 3.5	Universal testing machine (FTS LFV 1000)	36
Fig. 3.6	Electronic desk	39
Fig. 3.7	Data logger machine (3535D-Scorpio)	40
Fig. 3.8	Dimension of beam with bottom overlay	41
Fig. 3.9	Test arrangement of beams with bottom overlay	43
Fig. 3.10	Dimension of beam with top overlay	50
Fig. 3.11	Test arrangement of beams with top overlay	51
Fig. 3.12	Dimension of beam with middle segment being HPC	58
Fig. 3.13	Test arrangement of beams with middle segment	60
Fig. 3.14	Dimension of beam – Normal concrete beams	66
Fig. 3.15	Test arrangement of normal concrete beams	67

CHAPTER 4

Fig. 4.1	Comparison of moment-curvature curves of beams with tension overlays	75
Fig. 4.2	Comparison of moment-curvature curves of beams with compression overlays	76
Fig. 4.3	Comparison of moment-curvature curves of beams with middle segment being HPC	76

Fig. 4.4	Comparison of moment-curvature curves of normal concrete beams	77
Fig. 4.5	Comparison of load Vs mid-span deflection for beams with tension overlays	78
Fig. 4.6	Comparison of load Vs mid-span deflection for beams with compression overlays	79
Fig. 4.7	Comparison of load Vs mid-span deflection for beams with middle segment being HPC	79
Fig. 4.8	Comparison of load Vs mid-span deflection for normal concrete beams	80
Fig. 4.9	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.0046$)	81
Fig. 4.10	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.0057$)	82
Fig. 4.11	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.0066$)	82
Fig. 4.12	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.0088$)	83
Fig. 4.13	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.010$)	83
Fig. 4.14	Comparison of moment-strain for beams with different overlays and the same steel ratio ($\rho = 0.026$)	84
Fig. 4.15	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.0046$)	86
Fig. 4.16	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.0057$)	86
Fig. 4.17	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.0066$)	87
Fig. 4.18	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.0088$)	87
Fig. 4.19	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.010$)	88
Fig. 4.20	Comparison of moment-curvature for beams with different overlays and the same steel ratio ($\rho = 0.026$)	88
Fig. 4.21	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.0046$)	91
Fig. 2.22	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.0057$)	91
Fig. 4.23	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.0066$)	92
Fig. 4.24	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.0088$)	92
Fig. 4.25	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.010$)	93
Fig. 4.26	Comparison of load Vs mid-span deflection for beams with different overlays and the same steel ratio ($\rho = 0.026$)	93
Fig. 4.27	Crack development and pattern for beams with tension overlay with different steel ratios (groups 1-6)	96

Fig.	4.28	Crack development and pattern for beams with compression overlay with different steel ratios (groups 1-6)	98
Fig.	4.29	Crack development and pattern for beams with middle segment being HPC.with different steel ratios (groups 1-6)	100
Fig.	4.30	Crack development and pattern for normal concrete beams with different steel ratios (groups 1-6)	102

CHAPTER 5

Fig.	5.1	Cost implication of partial use of HPC in normal concrete beams as different layers	105
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NOMENCLATURE and ABBREVIATION

A'_s	- Compression reinforcement, mm ²
A_s	- Tension reinforcement, mm ²
b	- Width of compression face of member, mm.
c	- Distance from extreme compression fiber to neutral axis, mm.
C_{cu}	- Creep coefficient.
d	- Distance from extreme compression fiber to centroidal tension reinforcement, mm ² .
d'	- Distance from extreme compression fiber to centroidal of compression reinforcement, mm
F_r	- Modulus of rupture = 0.7 sq. root of f'_c
M	- Applied moment, kN.m.
M_{cr}	- Cracking moment, or moment causing flexural cracking at section due to externally applied loads, kN.m
M_y	- Yielding moment, at which tension reinforcement reaches the tensile strain, kN.m
I	- Moment of inertia of section resisting externally applied loads, mm ⁴
I_{ut}	- Moment of inertia of un-cracked section transformed to concrete, mm ⁴
I_{cr}	- Moment of inertia of cracked section transformed to concrete, mm ⁴
ϵ_c	- Extreme compressive strain
ϵ_t	- Extreme tension strain
ϵ_s	- Tension reinforcement strain
ϕ	- Curvature = M / EI
φ	- Unit curvature = $1 / r$
r	- Radius of curvature
δ	- Mid-span deflection, mm
$\delta_{1,2}$	- Deflection as measured at the bottom face of the beam under the two point's load.
ρ	- Reinforcement ratio = A_s / bd
ρ_b	- Reinforcement ratio producing balanced strain condition.
ρ_{min}	- Minimum reinforcement ratio = $200 / f_y$
ρ_{max}	- Maximum reinforcement ratio = $0.75 \rho_b$
n_1	- Modular ratio of concrete = $E_{c(HPC)} / E_{c(Nor)}$
n	- Modular ratio of steel being transformed to concrete = $E_s / E_{c(Nor)}$
$E_{c(Nor)}$	- Modulus of elasticity of normal concrete, N/mm ²

$E_{c(HPC)}$	- Modulus of elasticity of HPC.concrete,N/mm ²
E_s	- Modulus of elasticity of reinforcement, N/mm ²
EI	- Flexural stiffness of compression member
f'_c	- Specified compressive strength of concrete, N/mm ²
f_u	- Ultimate stress of reinforcement,N/mm ²
h_1	- The thickness of overlay being HPC, mm
h_2	- The thickness of normal concrete, mm
h	- Thickness of the cross section of concrete beam, mm
L_e	- Effective span length, mm
Y_t	- Distance from centroidal axis of gross section to extreme fiber in tension, mm
HPC	- High performance concrete
Nor.	- Normal concrete
Top	- Top overlay being HPC.
Bot.	- Bottom overlay being HPC.
LVDT	- Linear variable differential transducer
OPC.	- Ordinary Portland Cement
RAK.	- Ras-al-khayma aggregate, Type of aggregates, commonly used in concrete industry in Bahrain, imported from Ras-al-khayma, Dubai, U.A.E.
P	- Externally applied load, kN.

CHAPTER 1

INTRODUCTION

CHAPTER 1

INTRODUCTION

1.1 High Performance Concrete (HPC)

Concrete is one of the very widely used civil engineering material which is characterized by its compressive strength. In recent years, there has been a rapid growth of interest in high strength concrete. Those of concrete having uniaxial compressive strength, in the range of 80-120 Mpa is termed as high strength concrete. Such concrete are obtained by using certain admixtures including water-reducing agents, fly ash and micro-silica in addition to other ingredients of cement, sand and aggregates. Furthermore extreme attention is paid to the quality control during its production. High-strength concrete also has higher modulus of elasticity, tensile strength and lower creep coefficient. They are more durable and resist corrosion and abrasion better than normal concrete. Due to these properties, the high strength concrete is also termed as high performance concrete. The increase in the compressive strength is achieved by using extremely fine chemically reactive materials such as silica fume, calcareous fillers and even

black carbon, so that they fill the microvoids in grain packing, thus improving the compactness of the material and at the same time improving the rheological properties of the fresh concrete mix. [5].

1. 2 Advantages of using HPC

The practical use of HPC has preceded full knowledge of its properties as a material and of the behavior of structural members using the material. There are good reasons, both technical and economical in which higher-compressive strength correlates with improvements in other engineering properties such as tensile strength, creep coefficient.

The classic and earliest applications of high strength concrete were for piles, columns, and other axially loaded members. But, in current practice much more attention is given to flexural members. With flexural members, high strength concrete may provide greater deflection control, reduce prestressing losses, and/or permit use of less mild steel and prestressed reinforcing.[2]. These advantages can be summarized as follow:

- Ability to use more slender, lighter sections.
- Longer spans and/or wider member spacing.
- Greater stiffness and better control of deflection.
- Reduction in pre-stressing losses due-to a better control over creep and shrinkage.
- Reduction in the amount of shear reinforcement required in some flexural members.
- Reduction impact of high early stresses due-to prestressing and earlier application of service loading and the effects it may have on members at early ages.

1.2.1 Economy

In flexural reinforced concrete members, the maximum amount of tension reinforcement is limited by $\rho_{max.} \leq 0.75 \rho_b$ (ρ_b -reinforced ratio producing balanced strain condition) as defined by design codes such as ACI 318-83 [12] to insure a level of ductile behavior where the steel ratio of the beam must be less or equal to $\rho_{max.}$ Any increase in load capacity is only possible either by increasing the steel percentage, or using higher strength concrete. If the steel percentage is increased, the section size will be increased accordingly to maintain the ductility of the section. On the other hand, using higher strength concrete favorably reduces the section size, which results in a saving in the cost especially in multistory buildings. Having smaller cross sections for beams and columns yield to less amount of steel. In other words, the cost of high strength concrete is largely balanced by concrete quantity savings and by productivity improvements. Indirect benefits are also gained through durability improvements leading to maintenance savings.

The use of high strength concrete in compression members for a given cross section results in carrying more loads. As a result column sections can be selected smaller. In flexural members, smaller sections reduce dead loads involved and longer spans can be achieved [7]. For longer span bridges, the combination of high performance concrete (to reduce dead load) with pre-stress beams (to control deflections) extended the range of concrete bridge span to over 274 m [11].

1.2.2 Durability

In general the effect of environment on the durability of concrete structures depends on the permeability of gaseous and dissolved substances. Permeability is much influenced by the pore structure of the cement-paste that is dependent on relevant porosity and pore size distribution, where the resistance is usually reduced by the increase in the quantity of pores. If the acids or salts are able to reach the reinforcement through the cracks or pores, corrosion of the steel can be induced causing cracking and spalling of concrete.

The durability of high strength concrete from the aggressive chemical agent's point of view is higher than that of normal concrete. This improvement is related with the alkali-aggregate reactions due to the following reasons:[5]

- High performance concrete has a fine discontinuous pore structure which reduces ionic diffusions through the mix and concrete paste-aggregate interface.
- A reduction of $Ca(OH)_2$ due to the pozzolanic reaction of silica fume.
- Frost resistance as well as deicer-salt resistant.
- A reduction of ions OH concentration in the pore solution.

The use of superplasticizers is another way of increasing the concrete compressive strength. These materials reduce the flocculation of cement grains in which the cement grains in suspension in water can recover their initial grain size. This approach leads to an appreciable reduction in the quantity of water necessary, since quite a lot of this water is no longer trapped in the cement grain flakes. However, superplasticizers fabrication technology has improved and there

are great number of commercial superplasticizers such as, sulphonate melamine and sulphonate naphthalene.[5].

The addition of 5,10,15 and 20% silica fume increases the strength of the concrete, and reduces both permeability and workability. Carbonation is reduced to about 35% of that of the normal concrete beams with out silica fume. If silica fume percentage is greater than 15% and that of the superplasticizer is greater than 1.5% the pozzolan action reduces carbonation by 70% after a month [20].

1.2.3 Material properties

Compressive Strength: This is the most important functional property and it can be attained faster and greater than that of normal concrete. It has been possible to produce in the laboratory, concrete having a crushing strength exceeding 200 Mpa [5]. Concrete with mean strength values between 100 and 120 Mpa at 28 days measured on a cylinders can be easily produced, by making optimum use of locally available materials and water reducing agents.

Tensile Strength: The tensile strength is higher than that of normal concrete due to the higher strength and densification of the matrix and of the paste-aggregate interface. A better tensile strength, which results in an increase of the cracking moment which in turn favorably, affect the appearance of the cracks. For example in ordinary concrete structures, the micro cracking begins as soon as the compression stress reaches about 30-40% of the ultimate compressive strength and at 80-90% of this stress, cracks become inter-connected. In High Performance Concrete a little micro cracking develops when the compressive strength reaches about 65-75% of the ultimate stress and at 80-90% of this stress the cracks are not fully inter-connected. [11]

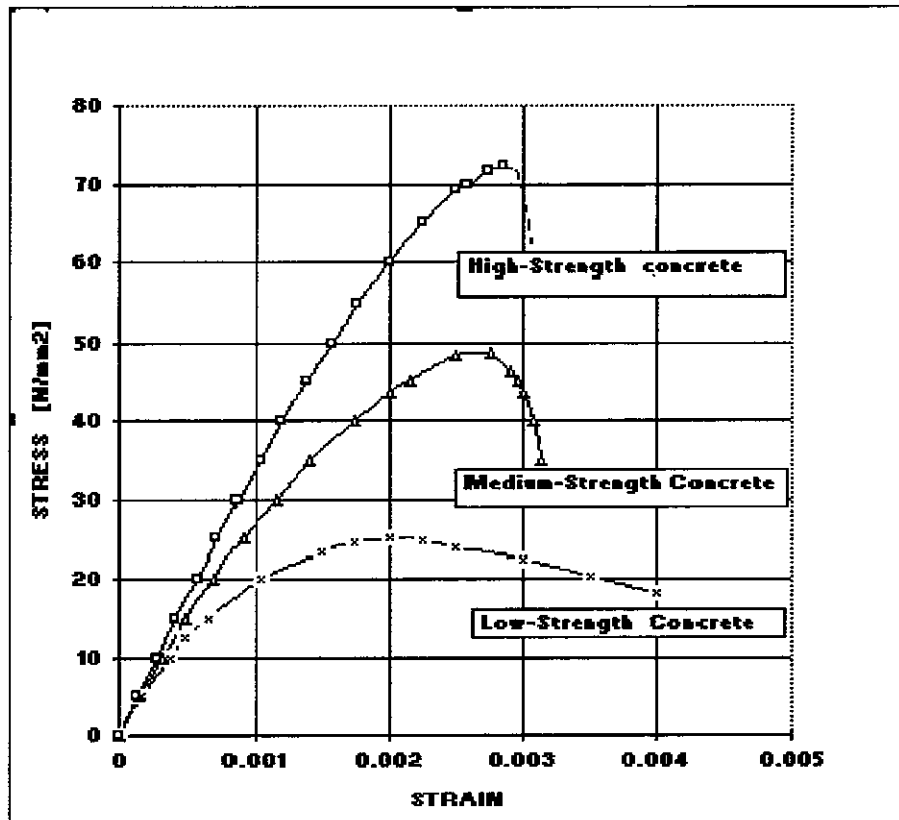


Fig. 1.1. Stress - Strain curve for different type of concrete

Young's Modulus (E_c): High performance concrete has higher modulus of elasticity as given in Table 1.1 which certainly results in smaller deflections in beams.

Shrinkage and Creep: Some of the studies made on HPC show that [19] the total shrinkage of HPC is approximately half that of the normal concrete. Since the material contains a very little free water after hydration, the creep of HPC is smaller than that of ordinary concrete. The creep coefficient C_{cu} decreases while the strength increases as shown in Table 1.1. In other words while the w/c ratio decreases the creep coefficient decreases too.

Bond with reinforcement: The bond stress between concrete and reinforcement is proportional to the tensile stress. It is therefore a function of compressive strength. With an increase of approximately 50% in the compression

strength, the steel-concrete bond increases approximately 40%, which stiffens the section and reduces the crack widths. [5].

The use of condensed silica fume doubles the steel-concrete bond by reducing the porosity and thickness of the transition zone adjacent to the steel. This in turn improves the adhesion-type bond at small slip level and allows larger stress and strain levels. [22].

Table 1.1 *Material Properties as given in [19].*

<i>Material</i>	<i>Compressive strength f'_c N/mm^2</i>	<i>Modulus of Elasticity (E_c) N/mm^2</i>	<i>CreepCoeff. C_{cu}</i>	<i>C_{cu}/C_{cu}</i>
Low strength concrete	20.7	23320	3.1	1.00
Medium strength concrete	41.4	32970	2.4	0.77
High strength concrete	70.0	42570	1.6	0.52

Ductility: In general, it is known that the stronger the concrete the more brittle it is. But, since structural engineers deal with reinforced or pre-stressed concrete, then the ductility of which can be adjusted accordingly to any particular needs. Flexural tests run on reinforced HPC beams show that their ductility is similar to that of beams made with ordinary concrete. [5].

Permeability: The combined use of ultrafine particles and a water reducing agent result in concrete with a very low permeability. With increased concrete strength, the permeability becomes consistently low to very low according to both *AASHTO T277* and water permeability classifications. [21].

1.3 Composite Beams and Structures

In construction there is an essential way of improving the economy and functionality of the structures by using a composite construction. The basic principle underlying composite construction is that, certain materials may be used more effectively in certain types of stressed conditions, Thus, the combination of a material strong in compression with the strong in tension makes a very economical structure.

It can be said that a reinforced concrete beam or slab is a composite member of steel and concrete, Also, a precast beam with a cast-in-site concrete slab or steel beam with a concrete slab are a composite constructions associated with the use of two materials.

Currently the uses of composite construction are growing rapidly due to economic reasons. Composite elements yield lighter beams which in turn result in smaller columns and consequently less steel tonnage to be supported. This results in savings in the cost of foundations and in the building height especially in the multistory buildings.

1.3.1 Composite Action

The composite action in structures is considered as the interaction of different structural elements, and may be developed using either different or similar structural materials. Composite construction became generally accepted by engineers for bridges during the 1950s and for buildings during the 1960s. [1]

During the past decade, research on continuous composite beams and connections has led to even greater economy through the use of composite design in continuous structures due to increased strength through continuity and also the increased stiffness due to composite action.

In 1975 Johnson discussed the behavior of flitched beams with no shear connectors with full slip and shear connectors with no slip (full interaction). The result showed that the maximum bending stress was reduced by 50% when shear connectors were provided, and the mid-span deflection was 25% of that in the case with no shear connectors. [1]

1.3.2 Shear Transfer in Composite Action

Composite action between steel and concrete or between structural elements of the material implies interaction between them and a transfer of shear at the connections. In reinforced concrete beams, this is taken of by the natural bond between concrete and deformed reinforcing bars. In the common type of composite beams, there is some shear transfer by bond and friction at the interface between steel beams and concrete slabs. Hence, shear connectors are needed to give reliable composite action with the following two objectives:

- To transfer shear between the steel and the concrete, thus limiting the slip at the interface and to make the structure to act as a unit.
- To prevent an uplift and separation between the steel beams and concrete slab at interface.

In the case of reinforced concrete monolithic beam-slab constructions, the longitudinal shear is taken by the concrete and the steel stirrups.

1.4 Literature review

Considerable amount of research is carried out on the use of concrete overlays in damaged or spalling structural members to restore or increase one or more of the followings:

- Compressive and/or tensile strength.
- Stiffness of the members.
- Ductility.
- Functional performance.
- Chemical attack and/or fire resistance.

The behavior of strengthened beams by using different type of concrete mix and/or different materials is similar to the partial use of high performance concrete from the flexural behavior point of view as well as the study's objective of this study.

M. Saiidi, S. Vrontinos, and B. Douglas [13] have developed a model to determine the yield and ultimate behavior of reinforced concrete beams strengthened by concrete overlays. The overlay may be on the compression or tension side. The model was evaluated by comparing analytical results with the test data obtained from 14 beams. A reasonably close agreement between the calculated and measured moments due to applied load and displacements at yielding of steel was noted. The most interesting results and conclusions derived in this research were as follows:

- The observed mode of failure in different type of specimens varied from flexural to beam shear and to separation of the overlay.
- The use of thick overlays for heavily reinforced beams increases the interface failure.

- A slight increase of ultimate moment and ductility ratio was observed as concrete strength increases.
- Both ductility and ultimate moment increased as the depth ratio of overlay increased.
- Higher moment capacity and lower ductility observed as the steel ratio increased.

Esam M. Ali [17] has conducted research to study the flexural behavior of strengthened reinforced beams in which, part of this study dealt with different types of concrete mixes as an overlay. The following conclusions were reported:

- The mid-span deflection of strengthened beams was smaller than that of control beams (normal concrete beams).
- The compressive and tensile strain of strengthened beams was lower than that of control beams.
- The ultimate load of strengthened beams was higher than that of control beams.

H. H. Abrishami, W. D. Cook, and Denis Mitchell [14] have conducted a study in which the flexural behavior of both normal and high performance reinforced concrete beams were investigated. The following are some of the results and conclusions derived from this research.

- High-strength concrete beams had slightly higher stiffness and about a 5% higher ultimate strength than normal - strength concrete beams.
- Crack width in high - strength concrete beams were smaller than that in normal-strength concrete beams at service load level.

- After flexural yielding, normal - strength concrete beams displayed diagonal tension cracks with final failure occurring by flexural crushing.
- After flexural yielding high - strength concrete beams experienced sudden failure with diagonal tension cracking.

M.Maalej and Victor C.Li [23] have proposed a new design for reinforced concrete flexural members to improve durability. The design makes use of unique properties of strain-hardening cementitious composite to limit crack width. The composite is used as a replacement for the concrete material that surrounds the main reinforcement in a regular reinforced concrete member. With this design, it was shown that crack widths under service load conditions could be limited to values never before achieved using conventional steel reinforcement and concrete. Under these conditions, it was concluded that it would be possible to prevent the migration of aggressive substances into the concrete or reinforcement. Furthermore, accelerated corrosion due-to longitudinal cracking or spalling can be reduced, if not eliminated, and spalling and delaminating problems common to many of today's reinforced concrete structures can be prevented.

S. Setunge, M. M. Attard, and P. Lep Darvall [24] have carried out an experimental research of ultimate strength using several mixes ranging from normal strength concrete to very high strength concrete. Other parameters in the mixes were the use of silica fume and type of crushed coarse aggregate. A two-parameter model for the failure envelope for confined concrete was adopted. Empirical expression for the failure envelope were derived for normal strength concrete and very high strength concrete with and without silica fume. A simple lower bound expression for the ultimate strength of concrete under confinement is

presented for any grades of concrete. The following outcomes were concluded:

- Very high strength concrete produced with silica fume shows higher normalized ultimate strength than the one without silica fume.
- The failure envelope of very high strength concrete is similar for all three types of coarse aggregate used.

S. E. Swartz, A. Nikaeen, H. D. Narayan Babu, N. Periyakaruppan, and T. M. E. Refai [25] have constructed and tested four reinforced concrete rectangular beams with a nominal strength of 82.7 Mpa. The beams were reinforced longitudinally with grade 60 steel at 50% and 150% of balanced ratio and had no stirrups of one beam, 50% stirrups of one beam, and full-required stirrups of two beams based on ACI requirements. The results of this study were summarized briefly as follows:

- The compressive stress block in the beam at failure is curved and may be represented by parabola with a maximum fiber stress of f'_c
- The strain at ultimate load in the extreme compression fiber may be less than 0.003 mm/mm(the maximum usable strain). A recommended value by the author for design is 0.0025 mm/mm.
- The secant modulus of elasticity may be estimated reasonably well by the ACI formula.
- The uni-axial compressive stress-strain curve is almost linear up to stress value of about $0.5 f'_c$.
- The beam and cylinder compressive stress-strain curves are similar.

Arthur H. Nilson [26] the main purpose of this paper is to summarize more recent research at Cornell University dealing with the behavior of reinforced and prestressed concrete structural members made using high strength concrete. Tests

have included axially loaded members with varying amount of spiral reinforcement steel, flexure-critical beams with varying amount of tensile and compressive reinforcement and stirrups, reinforced concrete beams under sustained load of 3 years duration. It was found that while many provisions of the 1983 building code requirements for reinforced concrete (ACI 318-83) are applicable to high strength concrete materials and members, certain code provisions must be re-examined, modified or limited to insure structural safety and serviceability. It was concluded that:

- Flexural strength and short-term deflection of reinforced concrete beams are predicted very well by the present ACI code procedure.
- Beams ductility is satisfactory, provided tensile steel ratios do not exceed the current maximum values, inspite of characteristic lack of ductility of high strength concrete.
- Because of the much lower creep coefficient, sustained load deflections of high strength beams are less than predicted by the present ACI code equations.

1.5 Partial use of high performance concrete

In many instances normal concrete is more than adequate to provide the required strength of flexural structural members, Therefore, it is not justified to fully use the high performance concrete instead of normal concrete in every day structural design due to the fact that the high performance concrete cost more than double the normal concrete, However, there are categories where the normal concrete can use the advantages of high performance concrete to improve one or more of the following features :

- Serviceability
- Durability
- Ductility

To attain these objectives, this study proposes the partial use of high performance concrete in normal concrete beams. This can be reasonably accomplished by either having different layers of high performance concrete in the beam cross section in the compression or tension zone, or by having different segments of the beam of different concrete grades.

1.6 Objectives of this study

The main objective of this study is to investigate the feasibility of the partial use of HPC in reinforced concrete beams. For this purpose two grades of concrete, the first is normal concrete with $f'_c = 44 \text{ N/mm}^2$ and the second is high performance concrete $f'_c = 65 \text{ N/mm}^2$ are used to produce four different group of reinforced concrete beams with rectangular cross section. The beams had length of 2 meters and cross section of 200 mm x 350 mm. In the first group HPC was placed on the top layer of 120 mm. In the second, the bottom layer of 120 mm was casted with HPC. The third group of beams was fully made of normal concrete. Finally the fourth group of beams were made using middle segment being HPC. Each group consisted of eighteen beams, which is divided into six sub groups each having a different steel ratio. Each sub group had three identical beams. Total number of beams considered in the study is sixty.

The objective of the study is to investigate the affect of partial use of high strength concrete in normal reinforced concrete beams subjected to flexural loading to serviceability, durability and ductility. Before initiating the experimental program, an analytical model is developed to determine the response of simply reinforced concrete beams with HPC overlay either at the compression or tension zone. The results obtained are outlined in chapter 2.

The description of experimental program and the results obtained in the tests carried out on the beam specimens are given in chapter 3. The discussion and analysis of test results are outlined in chapter 4. Comparisons of cracking moments and yielding moments are illustrated in the graphs, which contains the results of all tested beams. Moment curvature diagrams are also included in the same chapter. Mode of failure, crack propagation and pattern are shown in the pictures, which are given in chapter 4. Conclusions obtained from this study are listed in chapter 5.

CHAPTER 2

BEHAVIOR OF REINFORCED CONCRETE BEAMS WITH HIGH PERFORMANCE CONCRETE OVERLAYS (PRELIMINARY STUDY)

CHAPTER 2

BEHAVIOR OF REINFORCED CONCRETE BEAMS WITH HPC OVERLAYS (PRELIMINARY STUDY)

2.1. Objectives of the preliminary study

The main objective of this investigation is to study the effect of partial use of high strength concrete in normal reinforced concrete beams subjected to a flexural loading with particular attention to evaluate the followings:

- Cracking moment.
- Mid-Span deflection at service load.
- Ductility.

To attain those objectives, before initiating the experimental program an analytical model was developed to determine the response of simply supported reinforced concrete beam, with concrete overlay either at the compression or tension zone. Four different groups of beams were considered. In the first group HPC was placed on the top layer of 120 mm as shown in Fig. 2.2.a. In the second group the bottom layer of 120 mm was casted with HPC as shown in Fig.2.2.b. The third group beams were fully made of normal concrete as shown in Fig.2.2.c.

Finally the fourth group of beams were made using fully HPC as shown in Fig2.2.d.

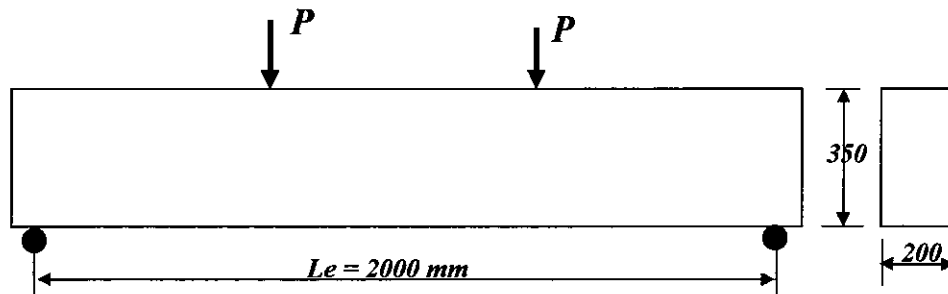


Fig. 2.1 *Beam loading*

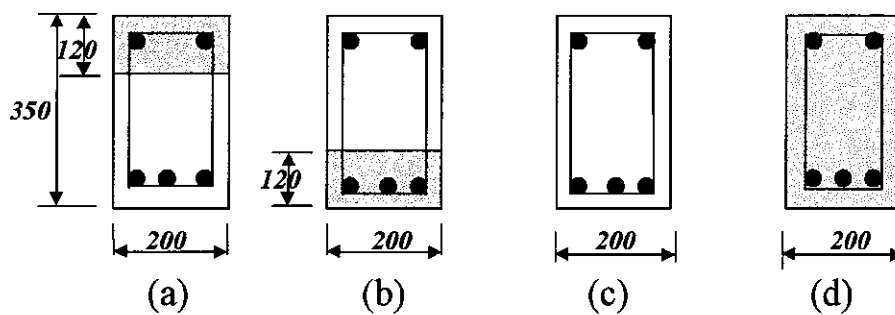


Fig. 2.2. *Cross Sections of reinforced concrete beams.*

Each group is subdivided into another six groups, each of which had different reinforcement ratios as shown in Table.2.1.

Table 2.1. *Different reinforcement ratios considered for each group.*

Reinforcement ratios	Compression steel	Tension steel	Stirrups
Ratio (1)	2 ϕ 12	2 ϕ 10+1 ϕ 12	ϕ 8 @ 150
Ratio (2)	2 ϕ 12	3 ϕ 12	ϕ 8 @ 150
Ratio (3)	2 ϕ 12	2 ϕ 16	ϕ 8 @ 150
Ratio (4)	2 ϕ 12	2 ϕ 16+ 1 ϕ 12	ϕ 8 @ 150
Ratio (5)	2 ϕ 12	3 ϕ 16	ϕ 8 @ 150
Ratio (6)	2 ϕ 12	3 ϕ 25	ϕ 8 @ 150

2.2. BASIC ASSUMPTIONS

The reinforced concrete beams considered in the analytical model has a layer of HPC either at compression or tension zone with the following assumptions and idealizations:

- Full composite action is assumed between the layers of concrete.
- Consequently plane section is assumed to remain plane after deformation.
- Stirrups are assumed to be capable of transferring shear from one layer to another.
- The tensile reinforcement is assumed to have yielded when the tensile steel strains reaches (f_y/E_s) mm/mm.
- The ultimate load is reached when the extreme compression concrete strain reaches the maximum usable strain equal to 0.003 mm/mm.
- Strain-hardening effect is ignored.
- Cracked concrete in tension (below the Neutral Axis) zone is ignored.
- Temperature and shrinkage stresses are ignored.

2.3. Theoretical Calculation

2.3.1. Moment-Curvature

The relation between applied moment in a given beam section and resulting curvature through the full range of loading to failure, is important in several contexts. It is basic to the study of member ductility, understanding of the developments of plastic hinges, and accounting for the redistribution of the elastic moments that occurs in most of the reinforced concrete structures before collapse.

The curvature is defined as the angle change per unit length at any given location along the axis of a member subjected to bending loads:

$$\phi = 1/r \quad (2.1)$$

where ϕ = unit curvature and r = radius of curvature. By using the usual assumptions regarding perfect bond between steel and concrete and plane section remain plane, it is possible to calculate the relation between applied moment and curvature for reinforced concrete beam section subject to flexural moment.

2.3.2. Computer Program

A computer program called “Response” developed at the University of Toronto [10] in fulfillment of a M.Sc. thesis used to carry out the computation of moment-curvature relationship. This program determines the load-deformation response of reinforced concrete cross-sections. By using the concept of moment-curvature, the program can calculate the following four aspects:

1. Moment and Axial load.

- calculate moment-curvature relationship for a given axial load and/or axial load-to-moment ratios.
- find the strain distribution for a given axial load and/or axial load to moment ratio and a specific strain anywhere within the section.
- find the strain for a given moment and axial load.
- find the axial load and moment for a given strain.

2. Shear and axial load.

- calculate shear force - strain relationship.
- find the biaxial strain state for a given axial load and shear force.

3. All loads.

- calculate axial load, shear, and moment - strain relationship.
- find the strain state due to applied moment, axial load and shear.

4. Rough Capacities.

- calculate estimates of the compressive and tensile capacities and the positive and negative moment capacities.
- calculate estimates of the cracking shear force and the post-cracking shear strength.

The program can perform three different types of analysis, these are:

- Load - deformation relationship.
- The value of moment for a given axial load at which the bottom fiber cracks.
- The strain for given loads.

The program is designed to calculate up to five different concrete layers and ten reinforcement layers. It also counts for tension stiffening, displaced concrete by reinforcement, thermal and shrinkage strains, and strain discontinuities for composite members. More details on program capability and calculation can be found in appendix B.

2.4. Analysis and Discussion

2.4.1. Cracking Moment.

As external force is applied to a section the moment - curvature relationship remains linear up to the cracking moment. This segment of moment-curvature represents the elastic range based on uncracked transformed section. In which the tension stress is smaller than that of modulus of rupture (F_r). of the cross section. The cracking moment can be written as follow:

$$M_{cr} = F_r I_{ut} / y_t \quad (2.2)$$

where F_r = modulus of rupture, I_{ut} = moment of inertia of uncracked transformed section, y_t = the distance from the neutral axis to the extreme tension fiber. Once the modulus of rupture is reached the section will crack. Appearance of cracks will reduce the flexural rigidity of the section. The reduction in rigidity will increase the curvature with no change in moment. But if the concrete in tension is neglected then a drop in applied moment will occur. This reduction is greater for the lightly reinforced sections as shown in Fig.2.3 and smaller for heavily reinforced sections as in Fig 2.8. The reduction of rigidity will continue until the steel in tension begins to be active in taking the tension stresses. This results in linear moment-curvature curve up to the point of the yielding of steel. The steel starts yielding when the strain in steel reaches a value of

$$\mathcal{E}_s = f_y / Es \quad (2.3)$$

where f_y = the yielding stress of the steel, Es = the modulus of elasticity of steel. When the steel yields, a large increase in curvature occurs at nearly constant moment.

The analytical results obtained in this study showed that the beams with bottom overlay, top overlay, and complete section being *HPC* have larger cracking moments. This can be encountered due to the larger moment of inertia of the section compared to that of normal concrete sections.

The cracking moment has increased with approximately 8% for beams with top overlay, 35% for the beams with bottom overlay, and 58% for the beams being completely made out of *HPC*, compare to normal beams.

It is found that the moment-curvature curves for both beams with normal concrete cross section and bottom overlay being *HPC* follow the same curve after cracking. Similarly the beams with full *HPC* cross section and the top overlay being *HPC* follow the same curve after cracking. This can be explained by the

fact that, the normal concrete zone in the top overlay beams and the HPC in the bottom overlay beams are ignored being in the tension zone of the cross section of the beam. The remaining active zone being either normal concrete as the case of bottom overlay or HPC as the case of top overlay.

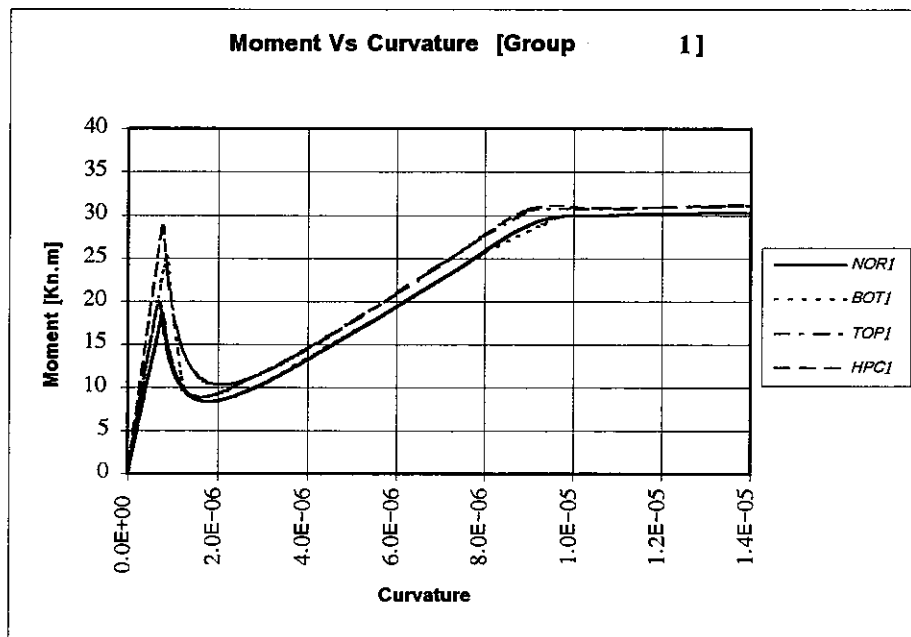


Fig. 2.3. *Moment - Curvature Curves ($\rho = 0.0046$)*

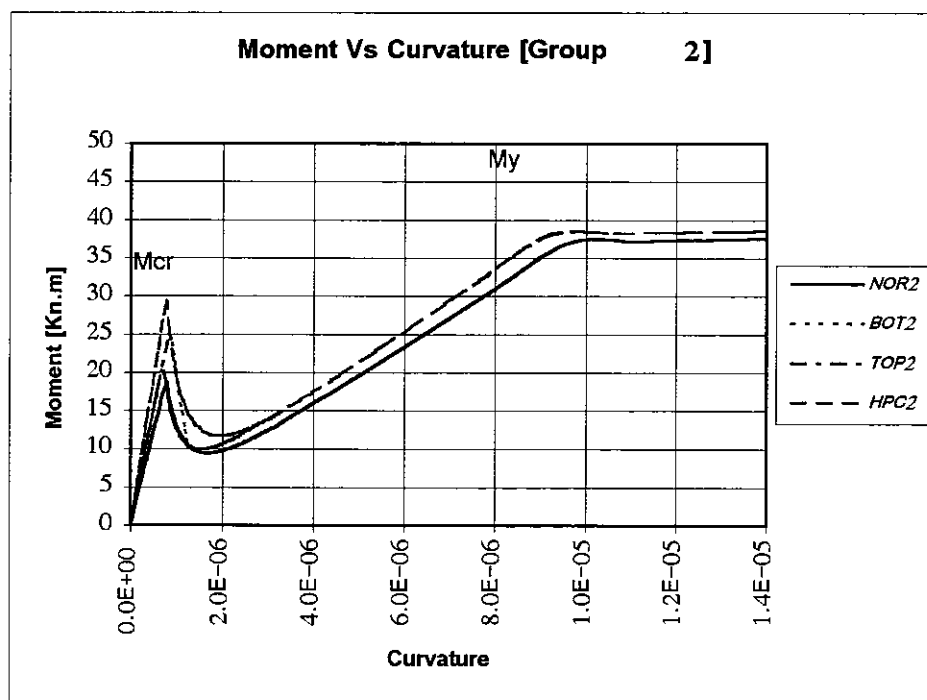


Fig. 2.4. *Moment - Curvature Curve ($\rho = 0.0057$)*

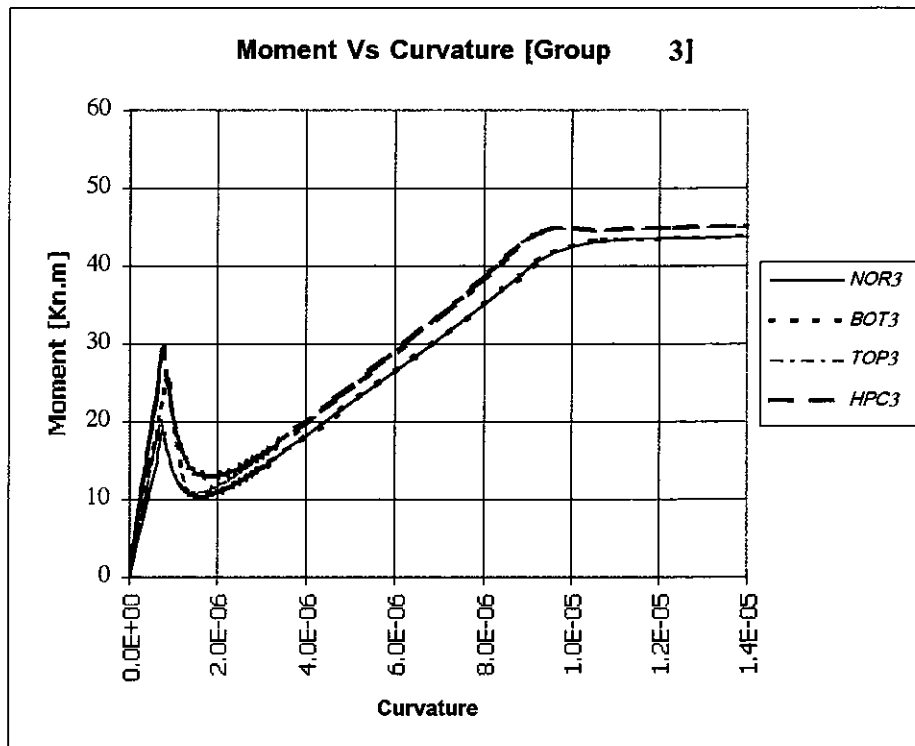


Fig. 2.5. *Moment - Curvature Curve ($\rho = 0.0066$)*

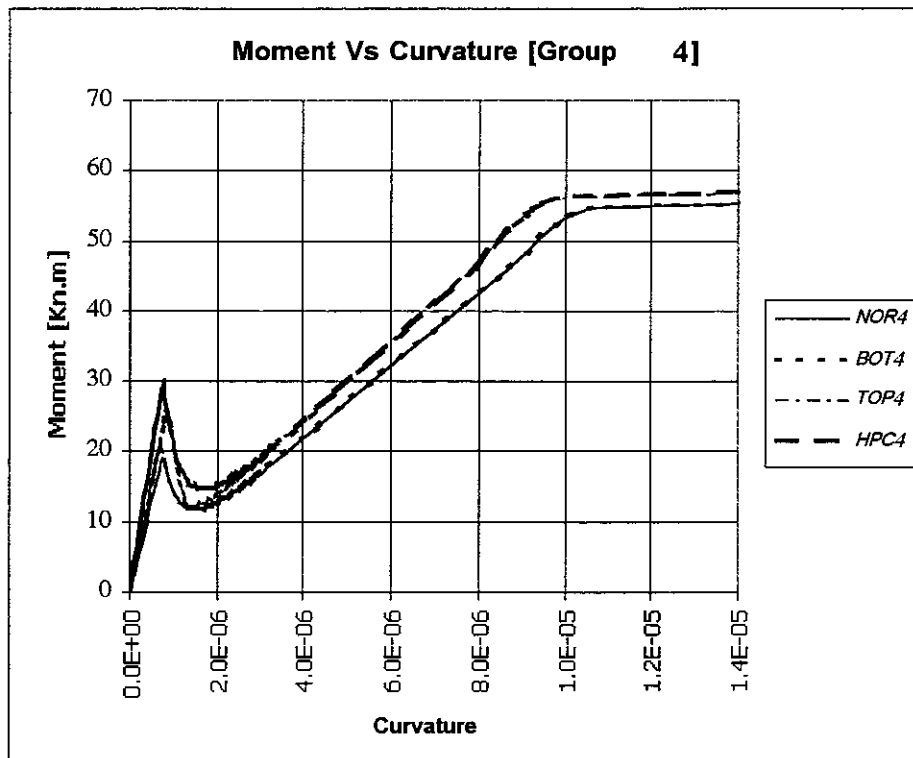


Fig. 2.6. *Moment - Curvature Curve ($\rho = 0.0088$)*

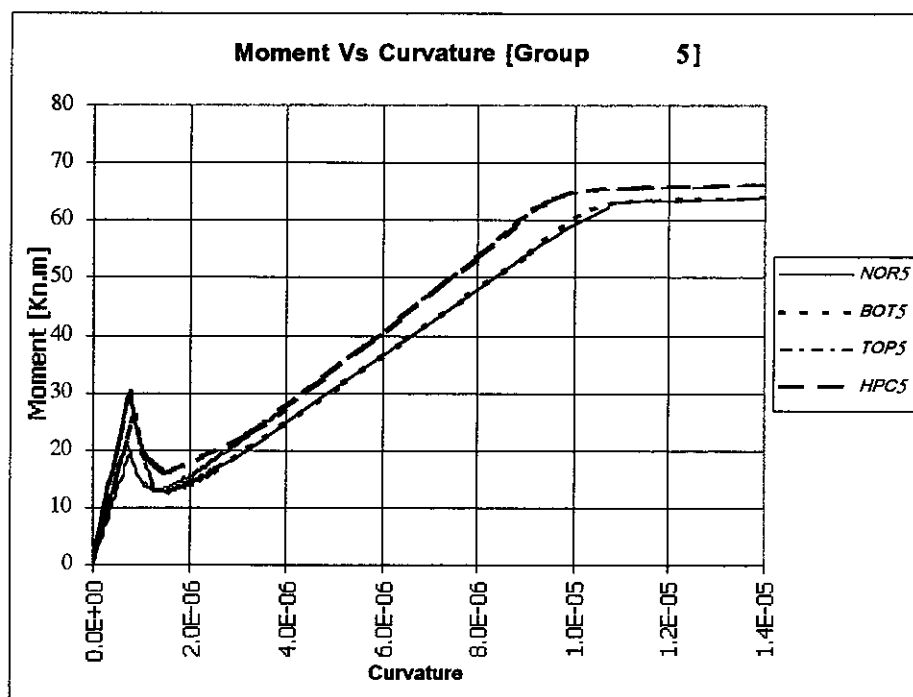


Fig. 2.7. *Moment - Curvature Curve ($\rho = 0.0100$)*

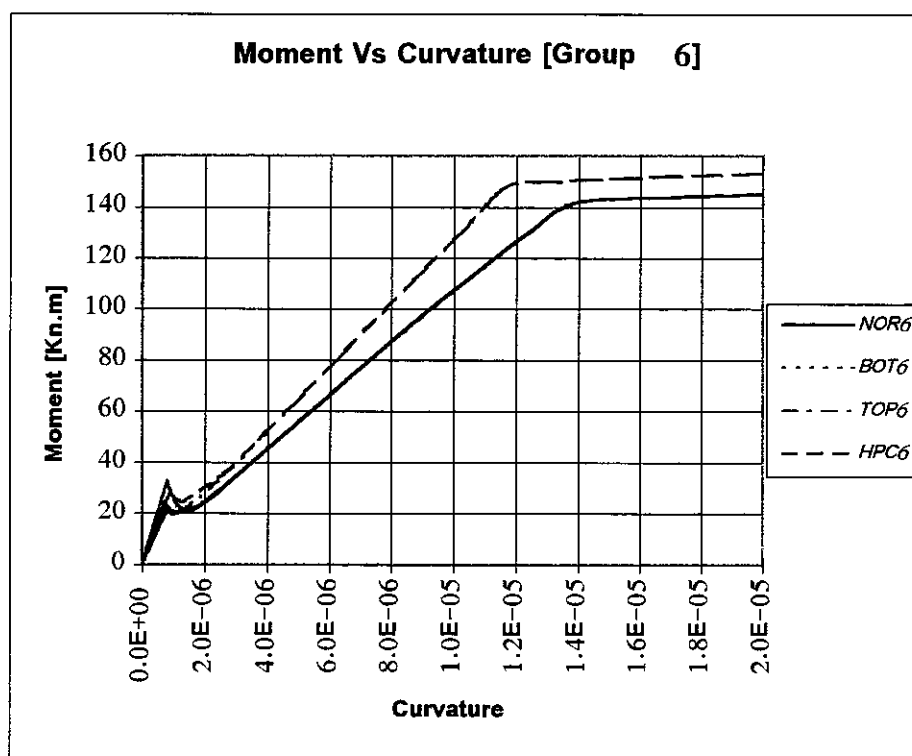


Fig. 2.8. *Moment - Curvature Curve ($\rho = 0.0260$)*

2.4.2. Mid-Span deflection

The mid-span deflections were calculated for all beams at the same loading level under the service load of normal concrete beams as given in Table 2.2. In beams with the top overlay being HPC has approximately 11% lower deflection at lower steel ratio and about 16% at higher steel ratio. In the beams with the bottom overlays, the mid-span deflection was reduced approximately by 80% at the lower steel ratio and no improvement was found at high steel ratio. The beams made fully of HPC showed an improvement of 89% at lower steel ratio and about 16% at higher steel ratio.

Table. 2.2. *Mid-Span Deflection at Service load*

Mid-Span Deflection (mm)				
Steel Ratio	Normal	Tension Overlay	Compression Overlay	HPC
0.0046	2.30	0.32	2.05	0.25
0.0057	2.55	2.05	2.10	0.29
0.0066	2.63	2.28	2.35	0.33
0.0088	2.75	2.56	2.44	2.08
0.0100	2.87	2.72	2.57	2.36
0.0260	3.55	3.54	2.99	2.98

2.4.3. Ductility

Ductility factor is taken as the ratio of curvatures at the ultimate stress and yield stress which are the stresses at the fiber in extreme compression with the strain of 0.003 and at the steel with the strain of 0.0021 mm/mm respectively.

The beams with compression overlays as well as the beams made fully of HPC showed about 40 % improvement in ductility factor, while the ones with bottom overlay showed no improvement as given in Table 2.3 This can be explained by the definition of ductility factor. When steel starts yielding, the

neutral axis will be located above the interface of bottom layer. In the case of top composite, the neutral axis will be within the top layer. By considering the fact that the concrete below neutral axis is ignored, the beams will behave as a normal concrete beam in the case of bottom composite and as a HPC in the case of top composite.

Table 2.3. Ductility factors

Steel Ratio	Ductility		Factors	
	Normal	Tension overlay	Compression overlay	HPC
0.0046	9.20	9.20	15.11	15.10
0.0057	8.17	8.10	13.17	13.18
0.0066	7.29	7.27	12.00	12.00
0.0088	6.54	6.54	10.55	10.50
0.0100	5.92	5.94	9.67	9.67
0.0260	2.29	2.22	4.67	4.67

2.5. Conclusions

The study carried out clearly shows that when HPC is used as compressive overlays the following improvements are obtained:

- Approximately 40 % improvement in ductility.
- Approximately 11-16 % improvement in mid-Span deflection.
- Approximately 6 % improvement in cracking moment.

On the other hand the effects of using HPC as tension overlays are:

- No improvement in ductility.
- Approximately 20% improvement in mid-span deflection at low steel ratio.

- No improvement in mid-Span deflection at heavily reinforced beams.
- Approximately 26 % improvement in cracking moment.

It can be concluded that the use of tension overlays is worthy whenever the cracking moment is to be increased, While if the ductility and/or the mid-span deflection were the subject of improvement, the compressive overlays is recommended.

By ignoring the concrete in tension, the result will be approximated and will not reflect the actual behavior of the beams. Hence, in order to verify the analytical outcome an experimental program was prepared in order to evaluate the actual behavior of overlays.

CHAPTER 3

EXPERIMENTAL PROGRAM

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

Recent years have witnessed a substantial increase in the use of composite construction, and enhanced research efforts aiming at developing techniques for combining steel and concrete effectively. From the structural performance point of view, as shown in the previous chapters that several gains can be achieved by casting reinforced concrete beams with two different grades of concrete. The behavior of composite section using two layers of cast-in-situ concrete made of two different grades of concrete is still not widely used. The concept of overlaying or strengthening of an old concrete member is commonly used in practice.

In the present study introducing a new technique of using two types of concrete with different grades both cast at site is introduced. An experimental program is carried out to study the flexural behavior of sixty composite beams

represented in three models. Each model consists of eighteen beams divided into six sub-groups, and each sub-group consists of three identical beams.

3.2 Preparing of shutter and steel

The experimental program covers testing of sixty beams. In order to minimize the number of individual forms three beams combined as one which resulted in twenty forms as shown in Fig.3.1. All forms were produced using 12mm plywood for bottom, sides and partitions, and 50x50-mm softwood for supports and 25 x 50 mm for plywood side supports. All forms cut and assembled within the Department of Civil Engineering workshop and then shifted to a place near to structures laboratory for concreting and easy handling during the test program. Before introducing the steel inside the forms, all forms were painted using a chemical release agent in order to minimize absorption of water and easy removal of the beams out of the wooden forms.



Fig 3.1. *Grouping of shutter*

Six groups of reinforcement ratios were decided to be used for the specimen beams. The reinforcement covered the range between minimum ratio

and 0.75 of maximum ratio as shown in Table. 3.1. The higher reinforcement ratio was decided in such a way that two layers of tension reinforcement were avoided. Steel bars with 10, 12, 16, and 25-mm diameter of grade 60 were used throughout. Fifteen nos. 8 mm closed stirrups were utilized to connect the tension and compression bars. For handling purpose two numbers of hooks were introduced at a distance of about 300-mm from the margins. Plastic spacers were used to hold the steel bars in place and maintain adequate cover.

3.3 Concreting and curing

It was decided to use ready mix concrete supplied by a local agent of producing ready mix concrete due to the following:

- The quantity of concrete required for 60 specimens were approximately 10 m^3 which was beyond the production capability of the structure laboratory.
- Uncertainty of completing the whole processes within the same day as well as maintaining the same mix design for all beams.

Table. 3.1. Groups of reinforcement

Reinforcement groups	Compression steel A_s'	Tension steel A_s	Stirrups	Steel ratio (ρ)
1	2 ϕ 12	2 ϕ 12 + 1 ϕ 10	15 ϕ 8	0.0046
2	2 ϕ 12	3 ϕ 12	15 ϕ 8	0.0057
3	2 ϕ 12	2 ϕ 16	15 ϕ 8	0.0066
4	2 ϕ 12	1 ϕ 12 + 2 ϕ 16	15 ϕ 8	0.0088
5	2 ϕ 12	3 ϕ 16	15 ϕ 8	0.0100
6	2 ϕ 12	3 ϕ 25	15 ϕ 8	0.0260

Two different grades of concrete, one is normal $f'_c=40 \text{ N/mm}^2$ and the other high strength $f'_c=75 \text{ N/mm}^2$, were requested from the Ready Mix Plant of Haji Hassan. This plant uses locally available aggregates, which are brought from Ras Al Khayma (RAK) of UAE, Ordinary Portland Cement (OPC), fine

aggregate and superplasticizers. Micro-silica was added to produce high strength concrete as shown in Table.3.2.

Table.3.2. *Concrete mix design.*

Target f_c [N/mm ²]	Cement Content [Kg/m ³]	W/C ratio	Aggregate 10/20 mm	Density [Kg/m ³]	Additives
40	330	0.5	RAK	2456	-
75	400	0.4	RAK	2462	M.silica

Concreting was planned to be completed within the same day in two stages, first stage in the morning and second stage in the afternoon as follows:

In stage one, a concrete mix of normal strength was used to fill the first layer of top composite beams, normal beams and ends of middle composite beams. Second concrete mix of high strength was used to fill bottom layer of bottom composite beams as shown in Fig. 3.2.

In stage two, normal concrete mix was used to complete second layer of bottom composite as well as middle segment of middle composite beams. Second mix of high strength concrete used to complete top layer of top composite beams. The concrete mixers were reaching the site in pre-scheduled timing to avoid maximum allowable setting time of concrete.

The interval of time between the two stages of concrete was needed to prepare the followings:

- To give the first layer of concrete enough time to gain some strength before receiving the next layer so that mixing of two layers of concrete can be avoided.
- To roughen the top surface of the bottom layer to form a key for the next layer.

- To dismantle and remove the temporary partitions fixed for the middle composite beams as well as roughen the vertical surface of interaction.

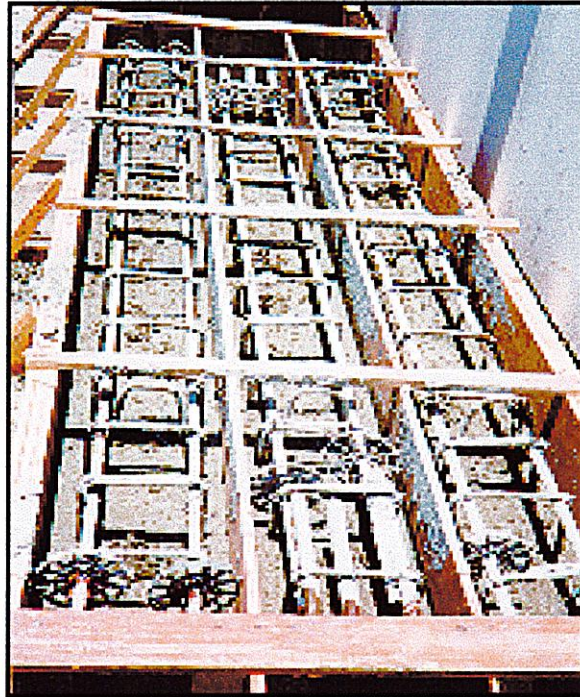


Fig.3.2. *Stage one- concreting of bottom layer*

After concreting, all forms were covered with wet gunny bags and covered with a layer of polythene sheets. For the following three days the specimens were watered continuously. The watering process was later reduced to three times a day for the following four days. The specimens were kept in forms until the test program was started.

3.4 Compressive Test

For the purpose of determining the compressive strength of cylindrical concrete specimen (f'_c), eleven specimen were prepared using 150 x 300 mm cylinders of standard moulds. Six specimens were prepared from the normal concrete mix and the other five specimens were prepared from high strength

concrete mix. All specimens were prepared and fully immersed in a water tank in the laboratory for 28 days. One day prior to testing, specimens were taken out from the water and capped using a heated sulfur mortar to provide plane surface complying with the *ASTM C39-86*. Specimens then were returned back to water tank and kept until the testing day as shown in Fig. 3.3.



Fig. 3.3. *Capping of cylinders*

3.4.1 Testing Procedure

A high stability compression machine shown in Fig. 3.4 was used to evaluate the cylindrical compressive strength. The machine complies with *BS 1881* and *ASTM E4* with a maximum capacity of 5000 kN. and it has a capability of having an increment of 1 kN. in loading.

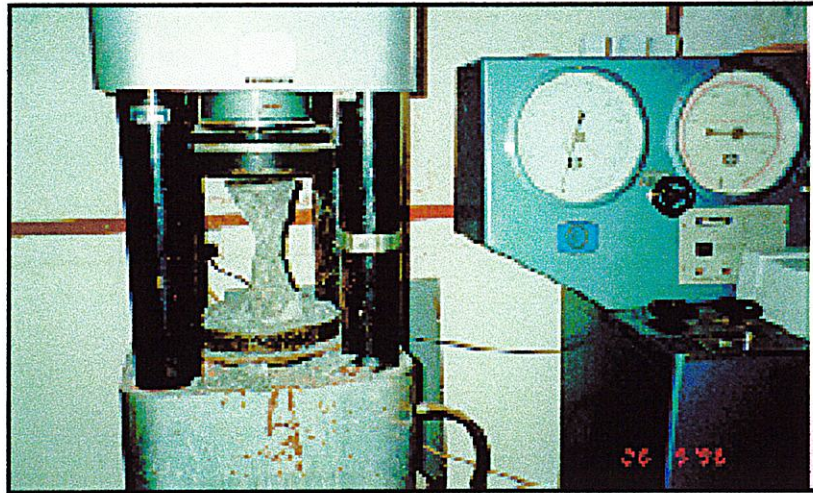


Fig.3.4. Compressive Strength Testing Machine

Testing procedure was carried out in accordance with the *ASTM 39-86*, which is listed in the following:

- Cylinder was removed from the curing tank and placed in a lower bearing block of the machine.
- The axes of specimen were aligned with the center of the machine.
- Continuous load was applied with a rate of 0.14 Mpa/sec.
- The maximum load carried by the specimen was recorded.

First, the compressive strength was calculated for each specimen individually. An average of compressive strength was then computed in two groups, one for the normal concrete and another for the high strength concrete as summarized in Table.3.3.

Table 3.3. Compressive strength of cylinder specimens (f'_c).

No.	Dimension [mm]	Load (P) [KN.]	Area [mm ²]	f'_c [N/mm ²]	Average f'_c [N/mm ²]
1	150 x 300	812	17671.5	45.9	44.2
2	150 x 300	788	17671.5	44.6	
3	150 x 300	806	17671.5	45.6	
4	150 x 300	740	17671.5	41.9	
5	150 x 300	764	17671.5	43.2	
6	150 x 300	780	17671.5	44.1	
7	150 x 300	1092	17671.5	61.8	65.0
8	150 x 300	1234	17671.5	69.8	
9	150 x 300	1112	17671.5	62.9	
10	150 x 300	1167	17671.5	66.0	
11	150 x 300	1145	17671.5	64.8	

3.5 Tensile Tests

For the purpose of determining the tensile properties of steel bars, twelve deformed steel bars with a length of 600 mm were prepared. For each evaluation, three bars of the same diameter were taken from different steel bars. The test includes $\phi 10$, $\phi 12$, $\phi 16$, $\phi 25$ as shown in Table.3.1.

3.5.1 Testing Machine

A universal testing machine type, *FTS LFV 1000* with a maximum capacity of 1000 kN, for both in compression and tension was used to determine the tensile strength of the bars. The accuracy of the machine is granted according to *DIN 51221*, class 1 and 2. The pressure is generated by an electronic desk type *SRG-5000*. The machine is accompanied by X-Y recorder of A4 size which plots the load/displacement or deformation diagrams according to the test performed. Fig.3.5 shows the general view of the machine.

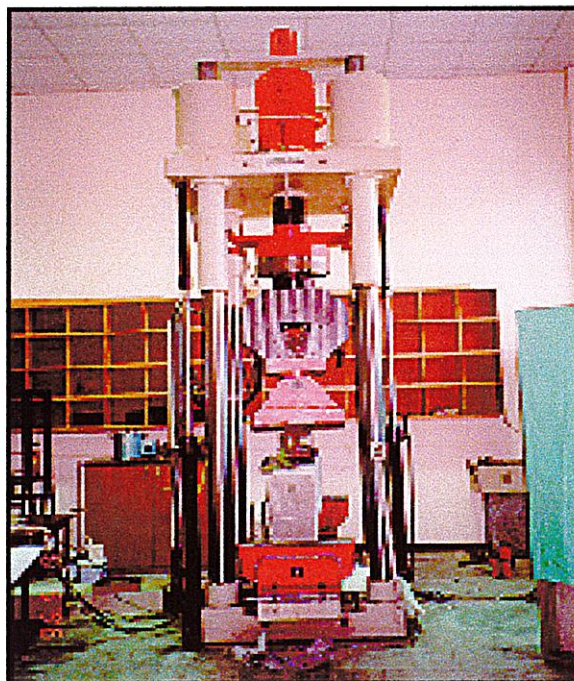


Fig.3.5. *Universal Testing Machine [FTS LFV 1000]*

3.5.2 Test procedure and results.

Each bar was gripped to the gripping plates of the testing machine, and an extensometer was attached to the bar and at the same time connected to the electronic desk. A continuous load was applied with a rate of 10 N/mm^2 of the bar cross section per second, depending on the diameter of the bar under the test. An automatic load-displacement diagram was produced. The extensometer was removed before the ultimate load was reached not to damage the extensometer. The ultimate load was recorded and tensile properties were calculated according to *BS 4449-1988* and the results are given in Table. 3.4.

Table.3.4 Experimental Tensile Properties of Steel bars.

Diameter ϕ [mm]	Yielding Stress (f_y) [N/mm ²]	Average (f_y) [N/mm ²]	Ultimate Stress (f_u) [N/mm ²]	Average (f_u) [N/mm ²]	M. of elasticity (E_s) [N/mm ²]	Average (E_s) [N/mm ²]
10	446	444	710	694	203000	200000
	439		686		201000	
	446		686		196000	
12	451	451	666	667	205000	204000
	451		668		202000	
	450		667		205000	
16	443	453	670	682	207300	205000
	465		706		200000	
	452		671		207300	
25	440	440	701	700	203700	204000
	440		699		203700	
	440		699		203700	

3.6 Flexural Testing

The flexural tests were divided into four main groups. The first group was those beams with bottom overlay HPC. The second group was those with top HPC overlay. The third was middle composite and the fourth group was the beams casted with normal concrete. Each group having eighteen beams divided

into six sub-groups with six different steel ratios covering the range between the $\rho_{\min}=0.0046$ and $0.75\rho_{\max.}=0.026$

3.6.1. Test Procedure

All beams were tested under displacement control in a universal hydraulic testing machine type [*FTS LFV 1000*] with a 1000-KN. capacity. The pressure is generated by an electronic desk type [*SRG-5000*], as shown in Fig.3.6. Third point loading was used throughout the whole program by introducing a steel channel. The effective span was 2000 mm, and a distance of 720 mm between the load points.

All measurements were taken and stored in a data acquisition system type [*SI 3535D scorio*]. The data logger machine measures the analog signal inputs with a high degree of accuracy and linearity, using an analog-to-digital converter as shown in Fig.3.7. The data logger machine was programmed to store data on a computer disk at every time by pressing the <enter> button at the front panel of the machine. This manual recording procedure was programmed to allow the followings:

- To pause the test periodically to allow manual measurement of crack width by means of a microscope.
- To monitor and record the crack developments and extensions.
- To mark the crack pattern on the surface of the beam.
- To record the load of the data saved at that level of loading from the machine cell .

All beams were painted with a lime solution to provide a brittle white coating on the beam surface for easy and clear visual monitoring of the cracks. The surfaces of the beams were also marked with a grid of 50x50 mm to follow the crack developments and patterns.

For strain measurements two groups of linear variable differential transducers (*LVDT*) were used, first type *GTX-2500* with a linear stroke of 2.5 mm used to measure compressive strains, second one type *GTX-5000* with a linear stroke of 5 mm used to measure tensile strains at steel level. The signals of variation in resistance were transformed to a variation in DC voltage by connecting the strain gages into bridge circuit energized by an external constant voltage.

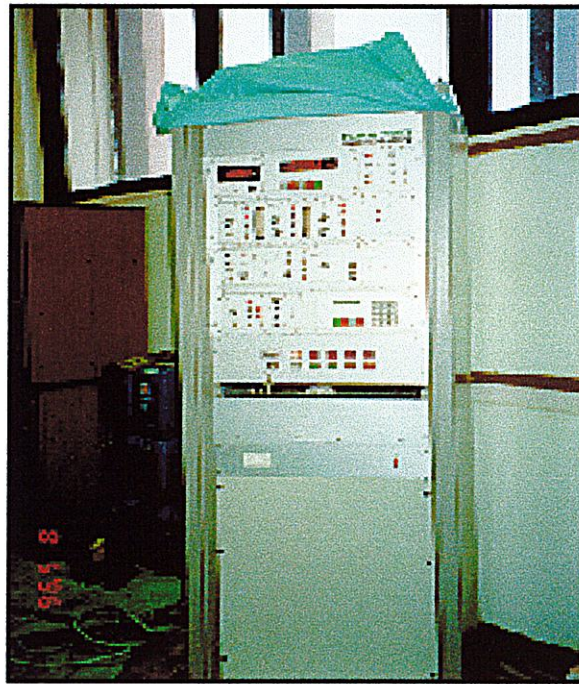


Fig.3.6. *Electronic Desk*

The reason behind this is that the resistance changes are very small, they can only be detected by the bridge which allows small changes in voltage to be read directly. The strain gauge length was extended by the mean of wooden pieces. Numbers of strain gages were fixed at the top compression level, and another two were fixed at the tension steel level. For group (2) the same arrangement was used with an additional two strain gages fixed at the line of

inter-action between the two layers of concrete in order to detect whether any slippage occurs during the test procedure. For the purpose of measuring deflection, three LVDT's were fixed at the mid-span as well as under the two point loads. Periodically, all transducers were calibrated using standard calibration procedure to assure their workability and accuracy of reading.

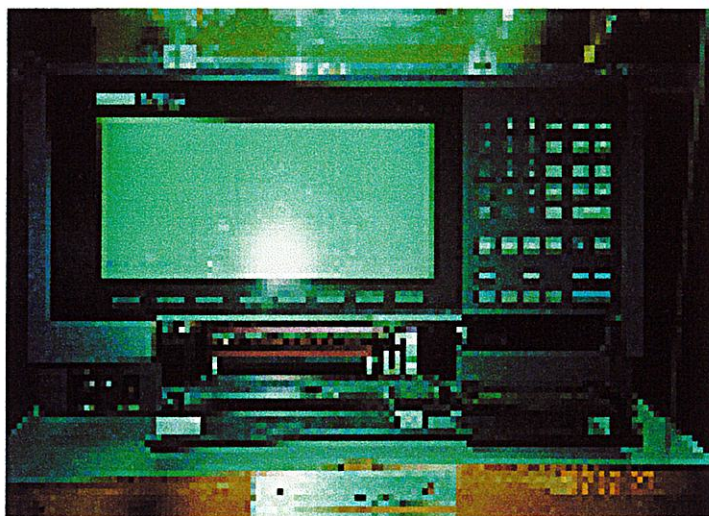


Fig.3.7. Data logger machine (3535D-Scorpio)

Unfortunately, due to the limited number of transducers available and as protective procedure all measuring devices were removed after the yielding of compression steel which is monitored through the data logger to prevent them from the failure of the beam sudden shock.

At the end of each test, the data stored in the disk was edited to introduce the load recorded for each reading. The files was then transformed into a table using excel software and saved with its own identification for later analysis and study.

3.7. Test procedure and data results of experimental program

3.7.1. Groups 1: Bottom overlay being HPC

This group consists of a series of eighteen reinforced concrete beams, with dimensions of 200 mm wide, 350 mm deep, and length of 2200 mm. The effective span was 2000 mm. The clear concrete cover depth provided was 50 mm. Fig. 3.8 shows the overall dimensions of the beams.

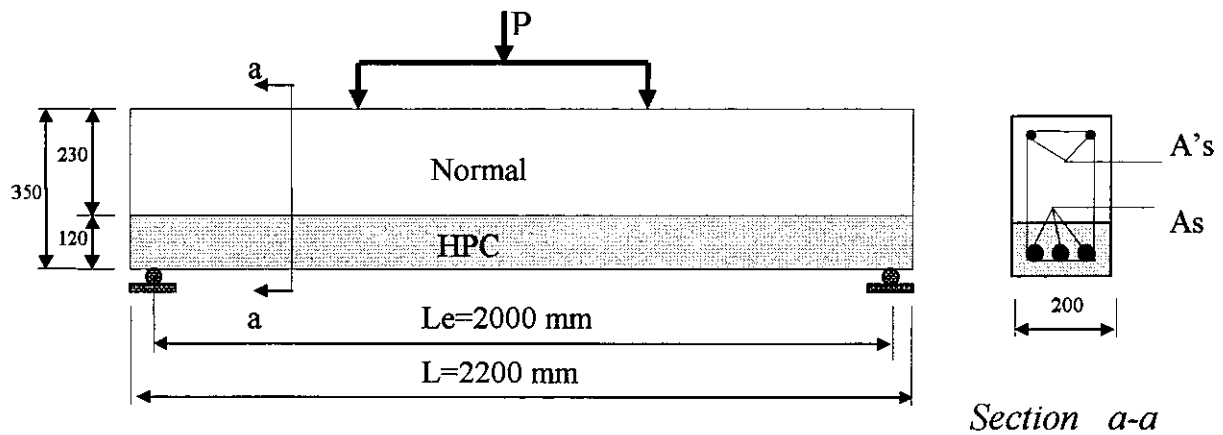


Fig. 3.8. Dimensions of beam (Bottom overlay).

All the beams in this group were cast using a two-step procedure. In the first, high strength concrete ($f'_c = 65 \text{ N/mm}^2$) was casted up to a depth of 120 mm. After allowing sufficient time for hardening in order not to allow mixing of two types of concrete, the second layer of normal strength concrete ($f'_c = 44 \text{ N/mm}^2$) was cast in the remaining depth of 230 mm. The surface of the first layer

was intentionally roughened by the mean of steel brush to provide a key for the top layer of concrete to prevent slippage at the interface.

The total number of eighteen beams were divided into six categories of reinforcement, each category consist of three identical beams. All the beams were provided with a tension and compression reinforcement as given in Table.3.5. Furthermore the double leg stirrups of $\phi 8$ mm diameter with an average yielding stress of 440 N/mm² are provided with spacing of 153 mm.

Table 3.5. : Details of specimens (group 1)

Beam	No. of specimen	b mm	h_1 mm	h_2 mm	H mm	$A's$ mm ²	A_s mm ²	ρ	ρ/ρ_{max}	ρ/ρ_b
B11 , B21 , B31	3	200	120	230	350	226	275	0.0046	0.135	0.101
B12 ,B22 , B32	3	200	120	230	350	226	339	0.0057	0.167	0.125
B13 , B23 , B33	3	200	120	230	350	226	402	0.0068	0.199	0.149
B14 , B24 , B34	3	200	120	230	350	226	515	0.0088	0.256	0.193
B15 , B25 , B35	3	200	120	230	350	226	603	0.0100	0.293	0.220
B16 , B26 , B36	3	200	120	230	350	226	1473	0.0260	0.762	0.571

$$\rho_{max} = 0.75\rho_b (\rho_{balance}). \rho = A_s/bd$$

All the beams were equipped with two hooks for carrying and handling during the test process. Each beam was marked with a letter and two numbers, B-stands for bottom, first number stands for the number of the beam within the same sub-group (1,2 or 3), and the second number stands for the reinforcement ratio within the group (1 through 6).

3.7.1.1. Test arrangement and data results of group 1

Four transducers were attached to one surface of the beam. Two of them are used to measure the compressive strain which were fixed at about 15 mm from the extreme compression fiber. The other two transducers were fixed at the

tension steel level to measure the tension strain as shown in Fig. 3.9. For measuring the deflection three transducers were fixed at the bottom face of the beam, one at the center of the beam and the other two under the two points load as shown in Fig.3.9

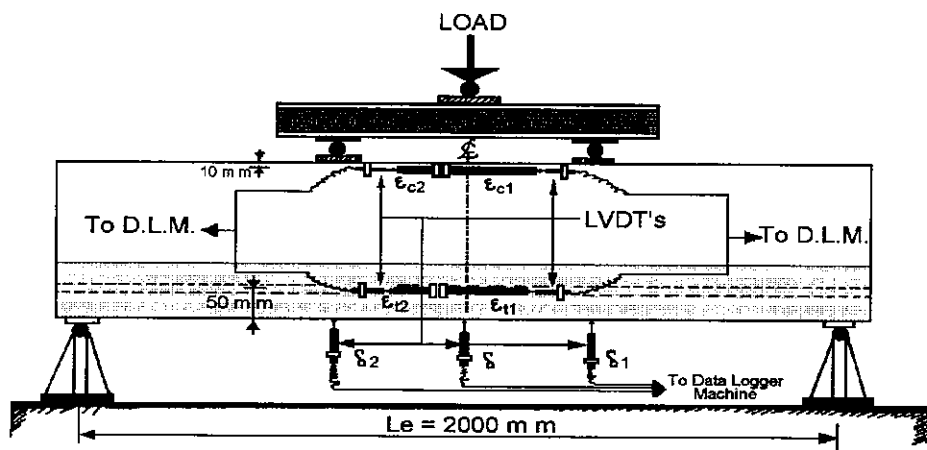


Fig. 3.9. Test arrangement of bottom overlay Hpc

The data results were analyzed and an average value of compression strain (ϵ_c), tension strain (ϵ_t) and mid-span deflection (δ) were calculated for all beams with the same steel ratio as represented in the following tables. The original data as measured for individual beams are listed in tables as given in appendix A

Table 3.6. Average values of strain, curvature and mid-span deflection for beams B11, B21, and B31

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	34	32	1.87E-07	0.24
30	57	46	2.95E-07	0.36
40	76	64	3.98E-07	0.58
50	95	92	5.33E-07	0.74
60	209	153	1.03E-06	1.16
70	519	255	2.21E-06	1.87
80	902	355	3.59E-06	2.39
90	1324	449	5.07E-06	3.03
100	1609	689	6.56E-06	3.87
110	2435	983	9.77E-06	5.41
120	3739	1291	1.44E-05	9.18
130	11373	2034	3.83E-05	15.28
135	14422	2368	4.80E-05	19.93
140	20996	3083	6.88E-05	24.96
150	24254	4825	8.31E-05	27.33

Table 3.7. Average values of strain, curvature and mid-span deflection for beams B12, B22, and B32

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0.00
20	31	29	1.69E-07	0.22
30	53	43	2.73E-07	0.32
40	71	61	3.77E-07	0.54
50	118	85	5.80E-07	0.68
60	125	148	7.79E-07	0.95
70	469	227	1.99E-06	1.56
80	759	307	3.05E-06	2.11
90	1080	405	4.24E-06	2.62
100	1322	480	5.15E-06	3.16
110	1491	589	5.94E-06	3.47
120	1672	710	6.80E-06	3.98
130	1891	799	7.68E-06	4.41
140	2385	867	9.29E-06	5.27
150	3132	960	1.17E-05	7.01
160	5889	1231	2.03E-05	8.96
170	13597	1587	4.34E-05	12.93
180	14354	2506	4.82E-05	19.70
190	19841	2835	6.48E-05	22.70

Table 3.8. Average values of strain, curvature and mid-span deflection for beams B13, B23 and B33

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	28	26	1.52E-07	0.19
30	46	41	2.48E-07	0.30
40	64	57	3.44E-07	0.47
50	95	78	4.94E-07	0.60
60	113	131	6.96E-07	0.87
70	160	189	9.95E-07	1.18
80	399	239	1.82E-06	1.62
90	652	290	2.69E-06	2.26
100	932	380	3.75E-06	2.66
110	1123	452	4.50E-06	3.05
120	1291	507	5.14E-06	3.47
130	1490	568	5.88E-06	3.95
140	1680	624	6.58E-06	4.45
150	1939	709	7.56E-06	5.01
160	2674	839	1.00E-05	5.62
170	4436	1301	1.64E-05	6.84
180	7494	1915	2.69E-05	9.60
190	12865	2363	4.35E-05	20.16
200	16458	2835	5.51E-05	22.56

Table 3.9. Average values of strain, curvature and mid-span deflection for beams B14, B24 and B34

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	25	24	1.41E-07	0.18
30	42	36	2.22E-07	0.29
40	57	51	3.07E-07	0.40
50	81	72	4.37E-07	0.56
60	97	101	5.67E-07	0.83
70	138	158	8.46E-07	1.08
80	494	221	2.04E-06	1.51
90	612	279	2.54E-06	1.96
100	750	356	3.16E-06	2.25
110	904	404	3.74E-06	2.65
120	1016	446	4.17E-06	3.08
130	1133	496	4.65E-06	3.25
140	1285	553	5.25E-06	3.90
150	1419	620	5.82E-06	4.19
160	1555	664	6.34E-06	4.78
170	1695	745	6.97E-06	5.17
180	2010	834	8.12E-06	5.48
190	2438	1000	9.82E-06	5.96
200	3427	1178	1.32E-05	7.58
210	6882	1318	2.34E-05	9.63
220	10922	1521	3.56E-05	13.87
230	12324	2147	4.13E-05	18.64
240	16583	2787	5.53E-05	23.60

Table 3.10. Average values of strain, curvature and mid-span deflection for beams (B15, B25, B35)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (φ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	22	21	1.20E-07	0.17
30	37	32	1.95E-07	0.27
40	46	52	2.80E-07	0.38
50	72	68	3.99E-07	0.53
60	104	100	5.80E-07	0.68
70	114	154	7.67E-07	0.89
80	162	202	1.04E-06	1.26
90	488	243	2.09E-06	1.55
100	608	326	2.67E-06	2.19
110	782	390	3.35E-06	2.38
120	961	428	3.97E-06	2.67
130	1048	482	4.37E-06	2.97
140	1146	523	4.77E-06	3.43
150	1244	568	5.18E-06	3.73
160	1347	623	5.63E-06	3.93
170	1499	704	6.29E-06	4.24
180	1613	755	6.77E-06	4.67
190	1739	807	7.27E-06	4.96
200	1865	878	7.84E-06	5.28
210	1977	947	8.35E-06	5.94
220	2148	1018	9.05E-06	7.01
230	2571	1121	1.05E-05	8.62
240	4568	1235	1.66E-05	11.98
250	9176	1946	3.18E-05	16.72
260	14180	2462	4.75E-05	22.23
270	16245	2645	5.40E-05	*

Table 3.11. Average values of strain, curvature and mid-span deflection for beams (B16, B26 and B36)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	18	20	1.09E-07	0.21
30	26	29	1.55E-07	0.37
40	35	40	2.15E-07	0.50
50	54	62	3.29E-07	0.65
60	79	89	4.81E-07	0.81
70	111	122	6.63E-07	1.00
80	206	196	1.15E-06	1.2
90	408	238	1.84E-06	1.39
100	535	300	2.39E-06	1.61
110	622	342	2.75E-06	1.81
120	702	386	3.11E-06	2.02
130	796	461	3.59E-06	2.25
140	881	510	3.97E-06	2.48
150	985	555	4.40E-06	2.71
160	1087	613	4.86E-06	2.95
170	1174	669	5.27E-06	3.19
180	1268	728	5.70E-06	3.51
200	1432	842	6.50E-06	4.08
220	1606	963	7.34E-06	4.67
240	1814	1088	8.29E-06	5.28
250	1902	1183	8.82E-06	5.62
260	1995	1249	9.27E-06	5.85
280	2197	1340	1.01E-05	6.69
300	2396	1429	1.09E-05	7.43
320	2618	1542	1.19E-05	8.28
340	2923	1701	1.32E-05	9.27

3.7.2 Group (2): Top overlay being HPC

This group consist of a series of eighteen reinforced concrete beams, with width of 200 mm, depth of 350 mm, a total length of 2200 mm, and an effective span of 2000 mm. The clear concrete cover depth provided was 50 mm as shown in Fig. 3.10

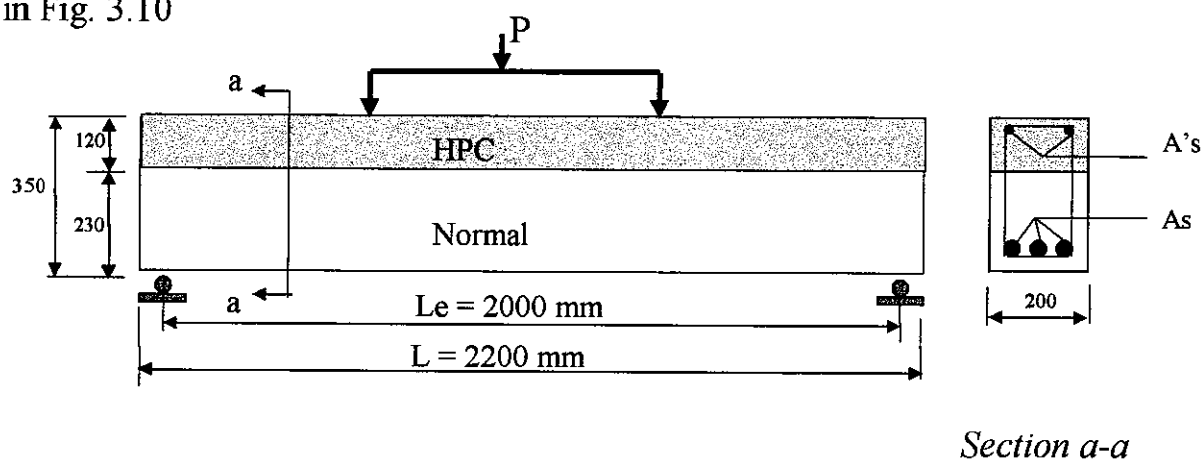


Fig. 3.10. Dimensions of beam (Top Overlay).

All the beams in this group were cast using a two-step procedure in which a depth of 230 mm made of normal strength concrete ($f'_c = 44 \text{ N/mm}^2$) was cast first, then, the remaining depth of 120 mm was filled with high strength concrete ($f'_c = 65 \text{ N/mm}^2$). In order to prevent mixing of the two different concrete, sufficient time was allowed for the first layer to gain some strength. The surface of the first layer was intentionally roughened by the mean of steel brush to provide a key for the top layer of concrete to prevent slippage at the interface.

The total number of eighteen beams were divided into six categories of reinforcement, each category consist of three identical beams. All the beams were provided with a tension and compression reinforcement as given in Table.3.6. Furthermore double leg stirrups at every 153 mm c/c of ϕ 8 diameter

with an average yielding stress of 440 N/mm^2 were also used to connect the top and bottom reinforcement as given in Table.3.4

Table 3.12. : Details of specimen (group 2)

Beam	No. of specimen	b mm	h_1 mm	h_2 mm	H mm	A's mm ²	A _s mm ²	ρ (A _s /bd.)	ρ/ρ_{max}	ρ/ρ_b
T11, T21, T31	3	200	230	120	350	226	275	0.0046	0.135	0.101
T12, T22, T32	3	200	230	120	350	226	339	0.0057	0.167	0.125
T13, T23, T33	3	200	1230	120	350	226	402	0.0068	0.199	0.149
T14, T24, T34	3	200	230	120	350	226	515	0.0088	0.256	0.193
T15, T25, T35	3	200	230	120	350	226	603	0.0100	0.293	0.220
T16, T26, T36	3	200	230	120	350	226	1473	0.0260	0.762	0.571

3.7.2.1 Test arrangement and data results of group 2

Four transducers were attached to one surface of the beam. Two of them are used to measure the compressive strain which were fixed at about 15 mm from the extreme compression fiber. The other two transducers were fixed at the tension steel level to measure the tension strain as shown in Fig. 3.11. For measuring the deflection three transducers were fixed at the bottom face of the beam, one at the center of the beam and the other two under the two points load as shown in Fig.3.11.

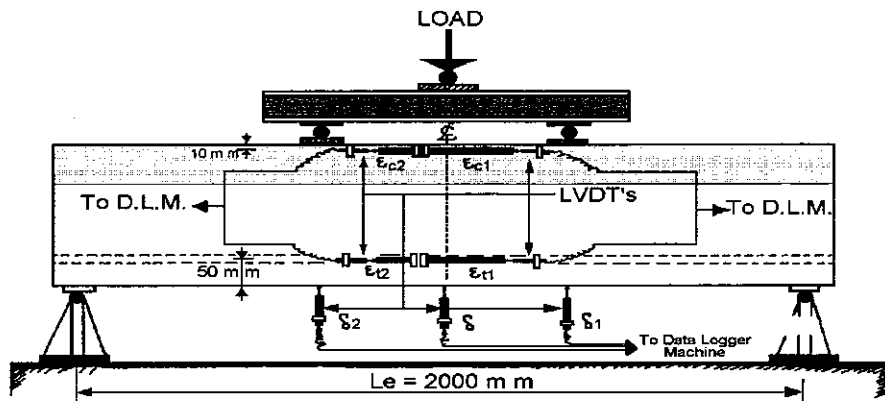


Fig. 3.11. Test arrangement of beams with top overlay

The data results were analyzed and an average value of compression strain (ϵ_c), tension strain (ϵ_t) and mid-span deflection (δ) were calculated for all beams with the same steel ratio as represented in the following tables. The original data as measured for individual beams are listed in tables as given in appendix A

Table 3.13. *Average values of strains, curvature and mid-span deflection for beams (T11, T21, and T31)*

<i>Applied Load</i> [KN]	<i>Average Tension strain (ϵ_t)</i> [x10E-6 mm]	<i>Average Compression Strain (ϵ_c)</i> [x10E-6 mm]	<i>Average Curvature (ϕ)</i> [x10E-6 mm/mm]	<i>Measured Mid-Span Deflection</i> δ [mm]
0	0	0	0	0
20	40	32	2.07E-07	0.27
30	59	58	3.34E-07	0.44
40	123	76	5.68E-07	0.60
50	136	104	6.86E-07	0.76
60	319	173	1.41E-06	1.11
65	468	215	1.95E-06	1.40
70	729	266	2.84E-06	1.75
80	923	344	3.62E-06	2.12
90	1150	440	4.54E-06	2.60
100	1433	530	5.61E-06	3.05
110	1826	631	7.02E-06	3.65
120	2535	753	9.40E-06	4.59
130	6325	1131	2.13E-05	8.46
140	10375	1745	3.46E-05	15.65
150	16371	2195	5.30E-04	24.48
160	23906	2687	7.60E-04	32.59

Table 3.14. Average values of strains, curvature and mid-span deflection for beams (T12, T22, and T32)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0.00
20	32	30	1.77E-07	0.23
30	53	46	2.84E-07	0.34
40	75	66	4.03E-07	0.57
50	118	88	5.87E-07	0.67
60	214	111	9.28E-07	0.87
70	442	192	1.81E-06	1.43
80	572	281	2.44E-06	1.89
90	755	362	3.19E-06	2.47
100	932	411	3.84E-06	2.75
110	1087	463	4.43E-06	3.37
120	1224	511	4.95E-06	3.76
130	1377	566	5.55E-06	4.20
140	1538	613	6.15E-06	4.91
150	1711	661	6.78E-06	5.34
160	2115	716	8.09E-06	6.00
170	2577	777	9.58E-06	7.60
180	3386	887	1.22E-05	9.90
190	7496	1302	2.51E-05	13.26
200	12551	1701	4.07E-05	17.65
210	19692	2554	6.36E-05	23.93

Table 3.15. Average values of strains, curvature and mid-span deflection for beams (T13, T23, and T33)

Applied Load [KN]	Average Tension strain (ϵ) [$\times 10^{-6}$ mm]	Average Compression Strain (ϵ_c) [$\times 10^{-6}$ mm]	Average Curvature (ϕ) [$\times 10^{-6}$ mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	29	27	1.57E-07	0.21
30	47	42	2.53E-07	0.33
40	67	60	3.63E-07	0.47
50	101	80	5.16E-07	0.63
60	152	106	7.36E-07	0.78
70	404	157	1.60E-06	1.08
80	516	227	2.12E-06	1.50
90	726	305	2.94E-06	1.97
100	846	385	3.52E-06	2.34
110	968	423	3.97E-06	2.79
120	1055	467	4.35E-06	3.19
130	1145	509	4.73E-06	3.64
140	1294	561	5.30E-06	4.02
150	1404	606	5.74E-06	4.47
160	1633	656	6.54E-06	4.83
170	1831	703	7.24E-06	5.40
180	2097	760	8.16E-06	6.03
190	2473	874	9.56E-06	7.03
200	3533	1160	1.34E-05	9.54
210	6716	1540	2.36E-05	14.43
220	16716	1810	5.29E-05	19.31
230	19437	2631	6.31E-05	27.89

Table 3.16. Average values of strains, curvature and mid-span deflection for beams (T14, T24, and T34)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	26	25	1.43E-07	0.20
30	42	40	2.32E-07	0.32
40	58	59	3.35E-07	0.44
50	81	76	4.47E-07	0.59
60	115	103	6.21E-07	0.72
70	201	138	9.70E-07	0.92
80	384	198	1.66E-06	1.40
90	497	238	2.10E-06	1.69
100	606	288	2.55E-06	1.94
110	840	328	3.34E-06	2.34
120	982	366	3.85E-06	2.66
130	1091	404	4.27E-06	2.99
140	1243	439	4.81E-06	3.27
150	1378	474	5.29E-06	3.63
160	1457	513	5.63E-06	4.03
170	1576	554	6.09E-06	4.30
180	1744	599	6.70E-06	4.69
190	1877	644	7.20E-06	5.11
200	2142	687	8.08E-06	5.58
210	2463	803	9.33E-06	6.15
220	3864	1082	1.41E-05	7.52
230	6798	1376	2.34E-05	10.80
240	14615	1736	4.67E-05	16.11
250	17499	2563	5.73E-05	23.75

Table 3.17. Average values of strains, curvature and mid-span deflection for beams (T15, T25, and T35)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	24	23	1.34E-07	0.18
30	38	39	2.18E-07	0.29
40	51	57	3.07E-07	0.46
50	74	72	4.15E-07	0.56
60	106	98	5.81E-07	0.69
70	136	125	7.46E-07	0.82
80	285	171	1.30E-06	1.16
90	411	211	1.78E-06	1.47
100	520	270	2.26E-06	1.75
120	708	353	3.03E-06	2.35
140	908	423	3.80E-06	2.95
160	1108	500	4.59E-06	3.64
180	1267	572	5.25E-06	4.34
200	1474	655	6.08E-06	4.87
210	1634	724	6.74E-06	5.20
220	1779	788	7.33E-06	5.57
230	2034	873	8.30E-06	6.13
240	2856	959	1.09E-05	7.26
250	4385	1145	1.58E-05	10.46
260	7607	1585	2.63E-05	12.48
270	12938	2040	4.28E-05	18.16
280	16587	2512	5.46E-05	*

Table 3.18. Average values of strains, curvature and mid-span deflection for beams (T16, T26, and T36)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	23	22	1.28E-07	0.22
30	35	31	1.88E-07	0.38
40	43	42	2.45E-07	0.51
50	64	65	3.68E-07	0.68
60	119	94	6.09E-07	0.76
70	194	136	9.44E-07	0.97
80	206	209	1.18E-06	1.17
100	474	296	2.20E-06	1.61
120	659	374	2.95E-06	2.06
140	834	466	3.71E-06	2.47
160	1011	560	4.49E-06	2.85
180	1195	643	5.25E-06	3.36
200	1352	715	5.90E-06	4.01
220	1517	808	6.64E-06	4.56
230	1631	896	7.22E-06	4.87
240	1740	971	7.75E-06	5.15
260	1917	1071	8.53E-06	5.73
280	2081	1171	9.29E-06	6.43
300	2239	1285	1.01E-05	7.04
320	2420	1422	1.10E-05	7.66
340	2713	1562	1.22E-05	8.39
360	3001	1879	1.39E-05	9.31
370	3312	1926	1.50E-05	9.94

3.7.3 Group (3): Middle segment being HPC

This group consists of a series of eighteen reinforced concrete beams, with dimensions of 200 mm wide, 350 mm deep, and length of 2200 mm. The effective span of 2000 mm. The clear concrete cover depth provided was 50 mm as illustrated in Fig.3.12.

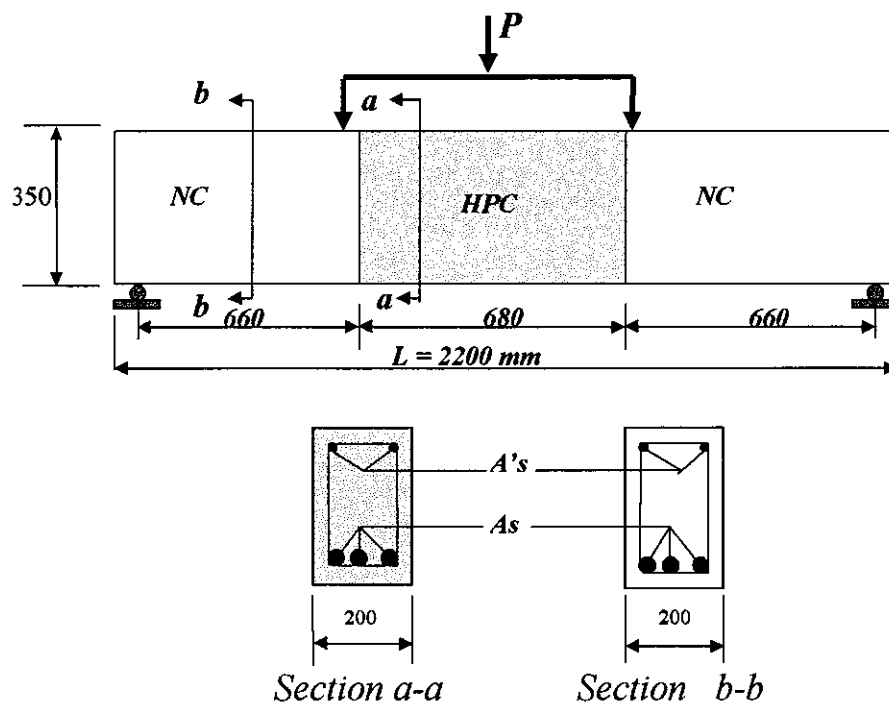


Fig. 3.12. Dimensions of beam (Middle segment).

All beams in this group were cast using a two-step procedure. Each beam was divided into three segments. One middle segment of 680 mm long and two outer segments of 760 mm long. In the first stage, outer segments was concreted using normal strength concrete ($f'_c = 44 \text{ N/mm}^2$). After allowing sufficient time

for hardening so two types of concrete do not mix, the temporary partitions was removed and the middle segment was filled with high- strength concrete ($f'_c = 65 \text{ N/mm}^2$). The vertical surfaces of the first segment were intentionally roughened by the mean of steel brush to provide a key for the next segment of concrete to prevent slippage at the interface.

The total number of eighteen beams were divided into six categories of reinforcement, each category consists of three identical beams. All beams were provided with a tension and compression reinforcement as given in Table.3.2. and in addition 14 numbers $\phi 8$ closed double leg stirrups was provided at every 153 mm c/c was provided with an average yielding stress of 440 N/mm^2 .

Table 3.19. Details of specimen (group 3)

Beam	No. of specimen	b mm	l_1 mm	l_2 mm	H mm	A's mm ²	As mm ²	ρ (As/bd.)	ρ/ρ_{max}	ρ/ρ_b
M11, M21, M31	3	200	660	680	350	226	275	0.0046	0.135	0.101
M12, M22, M32	3	200	660	680	350	226	339	0.0057	0.167	0.125
M13, M23, M33	3	200	660	680	350	226	402	0.0068	0.199	0.149
M14, M24, M34	3	200	660	680	350	226	515	0.0088	0.256	0.193
M15, M25, M35	3	200	660	680	350	226	603	0.0100	0.293	0.220
M16, M26, M36	3	200	660	680	350	226	1473	0.0260	0.762	0.571

3.7.3.1. Test arrangement and data results of group 3

Four transducers were attached to one surface of the beam. Two of them are used to measure the compressive strain which were fixed at about 15 mm from the extreme compression fiber. The other two transducers were fixed at the tension steel level to measure the tension strain as shown in Fig. 3.13. For measuring the deflection three transducers were fixed at the bottom face of the beam, one at the center of the beam and the other two under the two points load as shown in Fig.3.13.

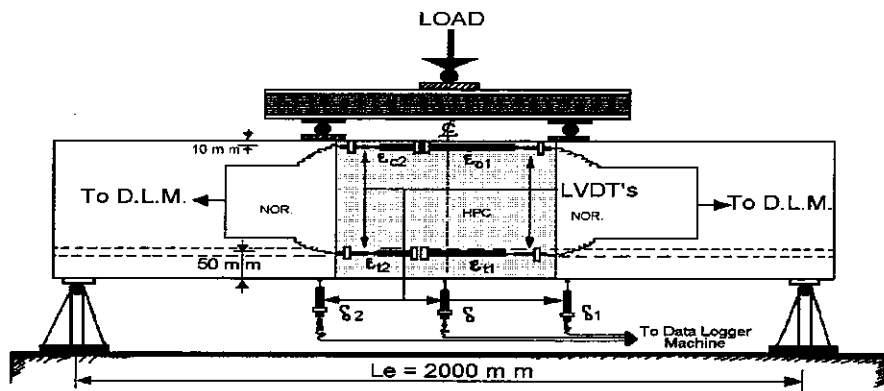


Fig. 3.13. Test arrangement of beams (middle segment)

The data results were analyzed and an average value of compression strain (ϵ_c), tension strain (ϵ_t) and mid-span deflection (δ) were calculated for all beams with the same steel ratio as represented in the following tables. The original data as measured for individual beams are listed in tables as given in appendix A

Table 3.20. Average values of strains, curvature and mid-span deflection for Beams (M11, M21, and M31)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid- Span Deflection δ [mm]
0	0	0	0	0
20	43	49	2.63E-07	0.32
30	64	68	3.76E-07	0.47
40	81	87	4.79E-07	0.68
50	111	112	6.38E-07	0.93
60	184	173	1.02E-06	1.22
65	334	249	1.67E-06	1.50
70	437	317	2.15E-06	1.75
75	581	385	2.76E-06	2.00
80	720	438	3.31E-06	2.35
85	860	479	3.82E-06	2.55
90	1036	530	4.47E-06	2.85
95	1182	658	5.26E-06	3.23
100	1393	744	6.10E-06	3.55
110	1901	897	7.99E-06	4.25
120	3252	1074	1.24E-05	5.70
130	8481	1699	2.91E-05	17.42
140	12682	2123	4.23E-05	26.83
150	21456	2567	6.86E-05	*

Table 3.21. Average values of strains, curvature and mid-span deflection for Beams (M12, M22, and M32)

Applied Load [kN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0.00
20	36	48	2.38E-07	0.26
30	54	64	3.38E-07	0.44
40	72	85	4.46E-07	0.64
50	96	107	5.79E-07	0.86
60	118	155	7.81E-07	1.16
65	251	248	1.43E-06	1.28
70	343	291	1.81E-06	1.46
75	449	367	2.33E-06	1.72
85	597	444	2.97E-06	2.12
90	656	513	3.34E-06	2.40
95	731	550	3.66E-06	2.63
100	801	586	3.96E-06	2.89
110	937	655	4.55E-06	3.27
120	1092	735	5.22E-06	3.78
130	1271	820	5.97E-06	4.13
140	1412	895	6.59E-06	4.69
150	1722	1144	8.19E-06	5.46
160	2189	1279	9.91E-06	6.58
170	3016	1899	1.40E-05	8.32
180	6386	1907	2.37E-05	10.97

Table 3.22. Average values of strains, curvature and mid-span deflection for Beams (M13, M23, and M33)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	33	46	2.23E-07	0.27
30	52	63	3.27E-07	0.40
40	68	76	4.10E-07	0.60
50	88	102	5.42E-07	0.83
60	108	144	7.20E-07	1.04
65	132	214	9.90E-07	1.22
70	300	267	1.62E-06	1.33
75	379	296	1.93E-06	1.57
80	533	342	2.36E-06	1.75
85	560	386	2.70E-06	1.98
90	678	412	3.11E-06	2.27
95	750	488	3.53E-06	2.43
100	845	490	3.82E-06	2.58
110	969	532	4.29E-06	2.97
120	1103	587	4.83E-06	3.37
130	1252	681	5.52E-06	3.79
140	1347	719	5.90E-06	4.24
150	1527	814	6.69E-06	4.92
160	1725	930	7.59E-06	5.29
170	2106	1063	9.05E-06	5.84
180	3092	1186	1.22E-05	6.51
190	5204	1768	1.99E-05	7.92
200	7140	1902	2.58E-05	13.47
210	16394	2155	5.30E-05	20.12
220	18248	2545	5.94E-05	*

Table 3.23. Average values of strains, curvature and mid-span deflection for Beams (M14, M24, and M34)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	32	38	2.00E-07	0.25
40	65	72	3.90E-07	0.56
45	78	82	4.54E-07	0.58
50	85	96	5.16E-07	0.75
60	105	133	6.81E-07	0.97
65	126	160	8.16E-07	1.15
70	164	201	1.04E-06	1.22
75	377	239	1.76E-06	1.43
80	453	266	2.05E-06	1.56
90	605	316	2.63E-06	1.84
100	735	385	3.20E-06	2.13
120	872	482	3.87E-06	2.76
140	1018	585	4.58E-06	3.69
160	1182	658	5.26E-06	4.26
180	1352	746	5.99E-06	4.98
190	1490	809	6.57E-06	5.49
200	2161	935	8.83E-06	6.01
210	2878	996	1.11E-05	6.85
215	3300	1271	1.31E-05	8.07
220	4209	1501	1.63E-05	9.73
230	5195	1790	2.00E-05	14.20
240	8892	1913	3.09E-05	16.68
245	12851	2099	4.27E-05	*
250	16159	2518	5.34E-05	*

Table 3.24. Average values of strains, curvature and mid-span deflection for Beams (M15, M25, and M35)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	28	33	1.75E-07	0.23
30	46	48	2.68E-07	0.38
40	60	68	3.64E-07	0.53
50	78	92	4.85E-07	0.64
60	96	123	6.26E-07	0.77
70	112	181	8.35E-07	1.06
80	382	256	1.82E-06	1.34
90	553	311	2.47E-06	1.84
100	654	360	2.90E-06	1.98
110	762	406	3.34E-06	2.26
115	817	426	3.55E-06	2.42
120	863	451	3.75E-06	2.57
130	945	494	4.11E-06	2.91
140	1033	534	4.48E-06	3.18
150	1127	581	4.88E-06	3.48
160	1226	618	5.27E-06	3.78
170	1325	660	5.67E-06	4.11
180	1470	698	6.19E-06	4.41
190	1559	737	6.56E-06	4.70
200	1657	788	6.99E-06	5.00
210	1793	842	7.53E-06	5.30
220	1960	912	8.20E-06	5.97
230	2344	1013	9.59E-06	7.13
240	3039	1301	1.24E-05	8.90
250	5719	1652	2.11E-05	11.60
260	8455	1923	2.97E-05	14.49
270	12272	2214	4.14E-05	17.08
280	15847	2524	5.25E-05	19.56

Table 3.25. Average values of strains, curvature and mid-span deflection for Beams (M16, M26, and M36)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	25	31	1.57E-07	0.28
30	38	40	2.22E-07	0.40
40	56	63	3.40E-07	0.53
50	79	87	4.73E-07	0.66
60	99	116	6.13E-07	0.82
70	141	162	8.65E-07	0.99
80	228	219	1.28E-06	1.17
90	326	289	1.76E-06	1.35
100	442	345	2.25E-06	1.52
110	540	392	2.66E-06	1.70
120	640	454	3.12E-06	1.91
130	720	495	3.47E-06	2.11
140	788	528	3.76E-06	2.32
150	904	566	4.20E-06	2.54
160	985	612	4.56E-06	2.76
180	1136	704	5.26E-06	3.25
190	1213	757	5.63E-06	3.50
200	1332	796	6.08E-06	3.80
220	1565	904	7.05E-06	4.33
230	1693	943	7.53E-06	4.60
240	1817	997	8.04E-06	4.96
250	1936	1066	8.58E-06	5.23
260	2062	1139	9.15E-06	5.50
270	2184	1238	9.78E-06	5.87
280	2311	1322	1.04E-05	6.13
290	2430	1400	1.09E-05	6.47
300	2567	1466	1.15E-05	6.86
310	2666	1573	1.21E-05	7.25
320	2796	1657	1.27E-05	7.64
330	2939	1753	1.34E-05	8.03

3.7.4 Group (4): Normal Concrete Beams

This group consists of a series of six reinforced concrete beams, all beams dimensions were 200 mm wide, 350 mm deep and 2200 mm long. The effective span was 2000 mm. The clear concrete cover depth provided was 50 mm. As represented in Fig. 3.14.

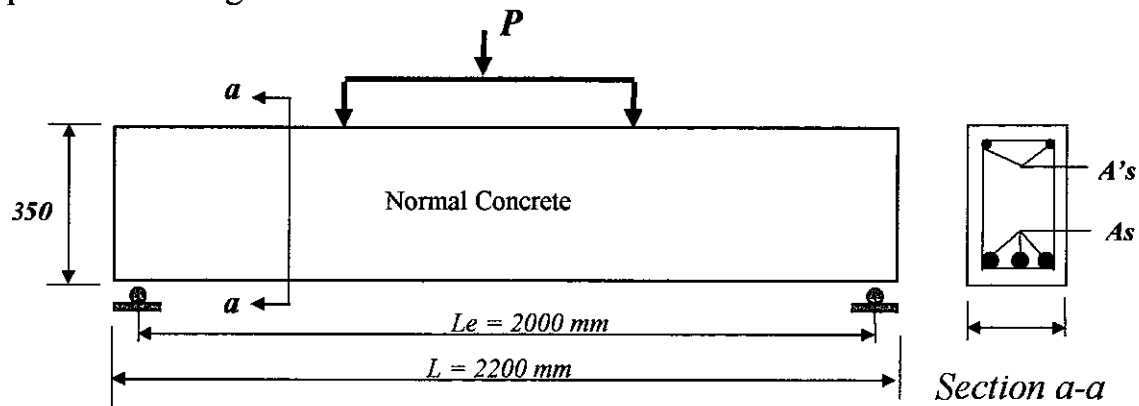


Fig. 3.14. Dimensions of beam (Normal).

All beams in this group were casted using normal strength concrete ($f'_c = 44 \text{ N/mm}^2$).

The total number of six beams were divided into six categories of reinforcement, each category consist of one beam. All beams were provided with a tension and compression reinforcement as given in Table.3.8. Additionally closed double leg stirrups of 8-mm diameter was provided at every 153-mm c/c with an average yielding stress of 440 N/mm^2 . All beams were equipped with two hooks for carrying and handling the beam during the test process.

Table 3.26. Details of specimen (group 4)

Beam	No. of specimen	b mm	h_1 mm	h_2 mm	H mm	$A's$ mm ²	A_s mm ²	ρ (A_s/bd)	ρ/ρ_{max}	ρ/ρ_b
N11	1	200	230	120	350	226	275	0.0046	0.135	0.101
N22	1	200	230	120	350	226	339	0.0057	0.167	0.125
N33	1	200	1230	120	350	226	402	0.0068	0.199	0.149
N44	1	200	230	120	350	226	515	0.0088	0.256	0.193
N55	1	200	230	120	350	226	603	0.0100	0.293	0.220
N26	1	200	230	120	350	226	1473	0.0260	0.762	0.571

3.7.4.1. Test arrangement and data results of group 4

Four transducers were attached to one surface of the beam. Two of them are used to measure the compressive strain which were fixed at about 15 mm from the extreme compression fiber. The other two transducers were fixed at the tension steel level to measure the tension strain as shown in Fig. 3.15. For measuring the deflection three transducers were fixed at the bottom face of the beam, one at the center of the beam and the other two under the two points load as shown in Fig.3.15.

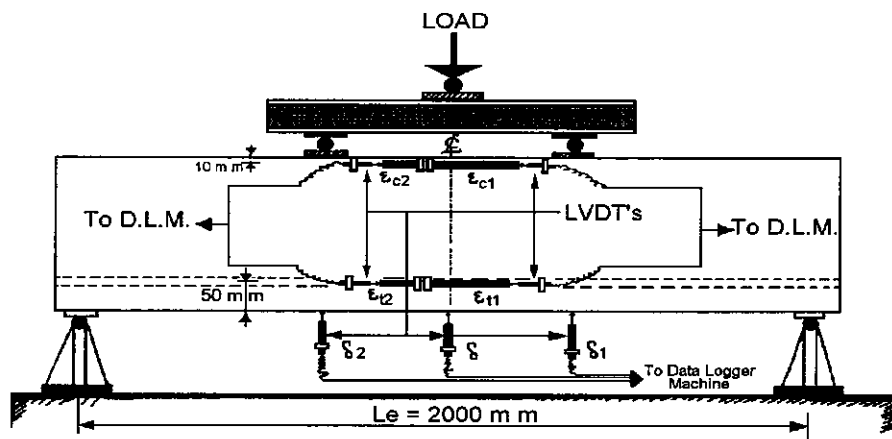


Fig. 3.15. Test arrangement of beams (normal concrete beams)

The data results were analyzed and an average value of compression strain (ϵ_c), tension strain (ϵ_t) and mid-span deflection (δ) were calculated for all beams with the same steel ratio as represented in the following tables. The original data as measured for individual beams are listed in tables as given in appendix A

**Table 3.27. Average values of strains, curvature and mid-span deflection
for Beam (N11)**

Applied Load (P) [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	52	68	3.41E-07	0.32
30	81	103	5.26E-07	0.46
40	101	127	6.51E-07	0.62
50	269	188	1.30E-06	0.78
60	522	246	2.19E-06	1.24
65	631	310	2.69E-06	1.62
70	836	393	3.51E-06	1.95
75	896	447	3.84E-06	2.22
80	935	485	4.06E-06	2.51
85	1034	517	4.43E-06	2.79
90	1326	545	5.35E-06	3.12
95	1452	592	5.84E-06	3.63
100	1631	744	6.78E-06	4.17
110	3010	1259	1.22E-05	6.09
120	4215	1645	1.67E-05	10.64
130	12703	1960	4.19E-05	16.64
140	18926	2938	6.25E-05	27.2

Table 3.28. Average values of strains, curvature and mid-span deflection for Beam (N22)

Applied Load (P) [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0.00
20	52	59	3.17E-07	0.25
30	85	90	5.00E-07	0.42
40	115	117	6.60E-07	0.59
50	153	145	8.49E-07	0.72
60	433	215	1.85E-06	1.09
65	535	278	2.32E-06	1.55
70	798	320	3.19E-06	1.84
75	856	359	3.47E-06	2.02
80	913	420	3.81E-06	2.28
85	978	448	4.07E-06	2.46
90	1095	478	4.49E-06	2.81
95	1153	511	4.75E-06	3.08
100	1332	570	5.43E-06	3.29
110	1519	643	6.18E-06	3.72
120	1768	726	7.12E-06	4.18
130	1894	812	7.73E-06	4.78
140	2318	997	9.47E-06	5.35
150	3002	1080	1.17E-05	7.01
160	6012	1146	2.05E-05	9.19
170	10010	1546	3.30E-05	12.06
180	12957	1957	4.26E-05	16.92
190	16327	2822	5.47E-05	23.03

Table 3.29. Average values of strains, curvature and mid-span deflection for Beam (N33)

Applied Load (P) [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	44	50	2.66E-07	0.25
30	70	78	4.23E-07	0.37
40	100	108	5.93E-07	0.53
50	132	141	7.80E-07	0.66
60	243	207	1.28E-06	0.97
65	324	260	1.67E-06	1.28
70	506	293	2.28E-06	1.40
75	647	340	2.82E-06	1.69
80	768	384	3.29E-06	2.01
85	833	431	3.61E-06	2.15
90	883	469	3.86E-06	2.39
95	965	523	4.25E-06	2.54
100	1041	563	4.58E-06	2.70
110	1289	621	5.46E-06	3.03
120	1432	678	6.03E-06	3.53
130	1675	762	6.96E-06	3.99
140	1846	826	7.63E-06	4.43
150	2160	889	8.71E-06	4.94
160	2559	951	1.00E-05	5.48
170	2984	1387	1.25E-05	7.14
180	7420	2174	2.74E-05	10.45
190	12703	2315	4.29E-05	15.90
200	20853	2746	6.74E-05	22.51

Table 3.30. Average values of strains, curvature and mid-span deflection for Beam (N44)

Applied Load [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	46	43	2.54E-07	0.24
40	81	85	4.71E-07	0.46
50	106	117	6.36E-07	0.59
60	180	188	1.05E-06	0.93
65	280	224	1.44E-06	1.13
70	371	256	1.79E-06	1.26
75	459	285	2.13E-06	1.48
80	524	323	2.42E-06	1.79
90	655	381	2.96E-06	2.05
100	812	441	3.58E-06	2.36
120	1106	576	4.81E-06	3.19
140	1256	620	5.36E-06	3.98
160	1420	758	6.22E-06	4.83
170	1597	820	6.90E-06	5.21
180	1829	873	7.72E-06	5.58
190	2215	960	9.07E-06	6.23
200	2937	1045	1.14E-05	8.15
210	5844	1332	2.05E-05	10.71
220	10101	1848	3.41E-05	16.11
230	12991	2268	4.36E-05	*
240	16315	2764	5.45E-05	*

Table 3.31. Average values of strains, curvature and mid-span deflection for Beam (N55)

Applied Load (P) [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	39	39	2.10E-07	0.22
30	60	64	3.53E-07	0.32
40	78	82	4.57E-07	0.43
50	97	109	5.89E-07	0.56
60	144	146	8.27E-07	0.80
70	269	196	1.33E-06	1.07
80	463	253	2.04E-06	1.50
90	576	314	2.54E-06	1.93
100	705	371	3.07E-06	2.27
110	806	440	3.56E-06	2.47
115	877	474	3.86E-06	2.64
120	1016	494	4.31E-06	2.84
130	1055	549	4.58E-06	3.04
140	1176	563	4.97E-06	3.53
150	1222	614	5.24E-06	3.73
160	1313	682	5.70E-06	4.08
170	1440	748	6.25E-06	4.32
180	1587	818	6.87E-06	4.79
190	1661	878	7.25E-06	5.06
200	1805	936	7.83E-06	5.37
210	1921	1028	8.42E-06	5.72
220	2058	1148	9.16E-06	7.13
230	2435	1244	1.05E-05	8.64
240	5604	1415	2.01E-05	10.88
250	8442	1796	2.92E-05	14.68
260	12453	2103	4.16E-05	18.26
270	15372	2544	5.12E-05	22.35

Table 3.32. Average values of strains, curvature and mid-span deflection for Beam (N66)

Applied Load (P) [KN]	Average Tension strain (ϵ_t) [x10E-6 mm]	Average Compression Strain (ϵ_c) [x10E-6 mm]	Average Curvature (ϕ) [x10E-6 mm/mm]	Measured Mid-Span Deflection δ [mm]
0	0	0	0	0
20	32	32	1.80E-07	0.24
30	46	56	2.91E-07	0.38
40	67	80	4.20E-07	0.55
50	97	105	5.77E-07	0.71
60	146	128	7.83E-07	0.87
70	212	183	1.13E-06	1.07
80	334	221	1.58E-06	1.26
90	437	304	2.12E-06	1.48
100	522	354	2.50E-06	1.66
110	639	418	3.02E-06	1.86
120	753	462	3.47E-06	2.06
140	906	524	4.08E-06	2.48
150	1033	564	4.56E-06	2.74
160	1102	618	4.91E-06	3.04
170	1222	687	5.45E-06	3.30
180	1294	742	5.82E-06	3.57
190	1400	816	6.33E-06	3.90
200	1490	861	6.71E-06	4.09
210	1547	917	7.04E-06	4.34
220	1640	973	7.47E-06	4.76
230	1758	1043	8.00E-06	5.11
240	1852	1195	8.70E-06	5.39
250	1907	1268	9.07E-06	5.68
260	2018	1345	9.61E-06	5.94
270	2081	1396	9.93E-06	6.31
280	2184	1433	1.03E-05	6.65
290	2291	1464	1.07E-05	7.05
300	2419	1540	1.13E-05	7.42
310	2542	1672	1.20E-05	8.00

CHAPTER 4

DISCUSSION AND ANALYSIS

CHAPTER 4

DISCUSSION AND ANALYSIS

4.1. Introduction

The results obtained from the experimental program are analyzed in this chapter. The effect of different overlays and different steel ratios on the behavior of concrete beam is determined by comparing the values of cracking moments, yielding moments and mid-span deflections. Moment-curvature graphs are plotted for each test in order to determine the effect of different overlays on these parameters.

4.2. The effect of steel ratio on beam with different types of overlays

A comparison between beams with different steel ratios and its effect on the mechanical properties and the behavior of the beam are discussed in the following .

- **Cracking Moment (M_{cr})**

The beam starts cracking when the strain at the extreme tension zone reaches a strain equal to F_r/E_c where (F_r) is the modulus of rupture and (E_c) is the modulus of elasticity of the cross section.

The cracking moment then becomes

$$M_{cr} = \frac{F_r I_g}{Y_t} \quad (4-1)$$

where (I_g) is the moment of inertia of the transformed cross section and (Y_t) is the lever arm measured from the neutral axis of the section to the extreme tension fiber.

If the steel in the tension zone is increased, the neutral axis drops down to balance the section, which results in smaller Y_t and higher moment of inertia and larger cracking moment.

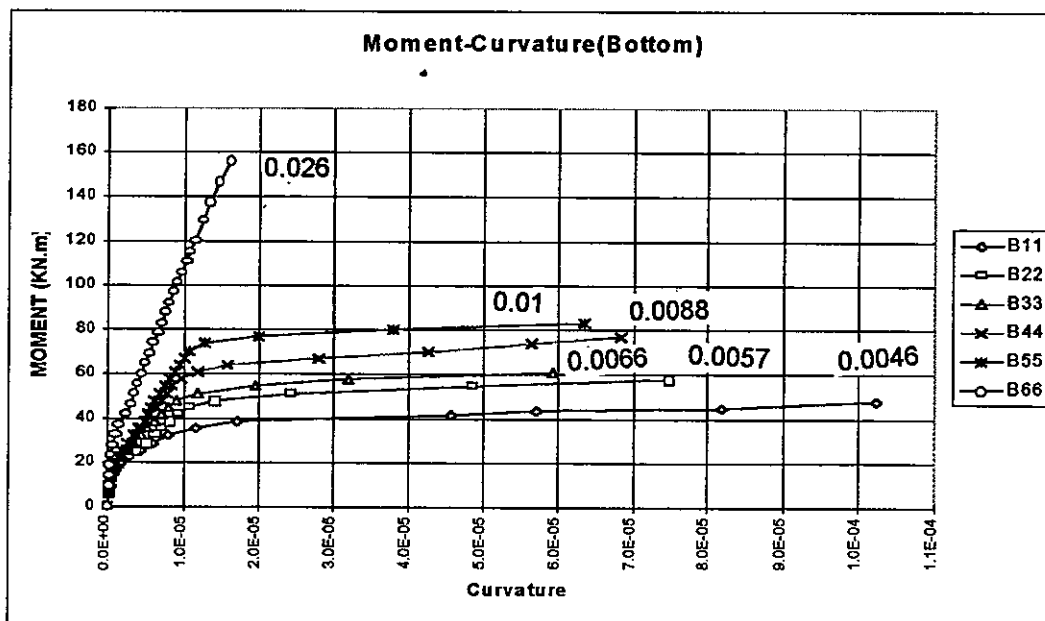


Fig. 4.1.. Moment vs Curvature for Bottom overlays with different steel ratios

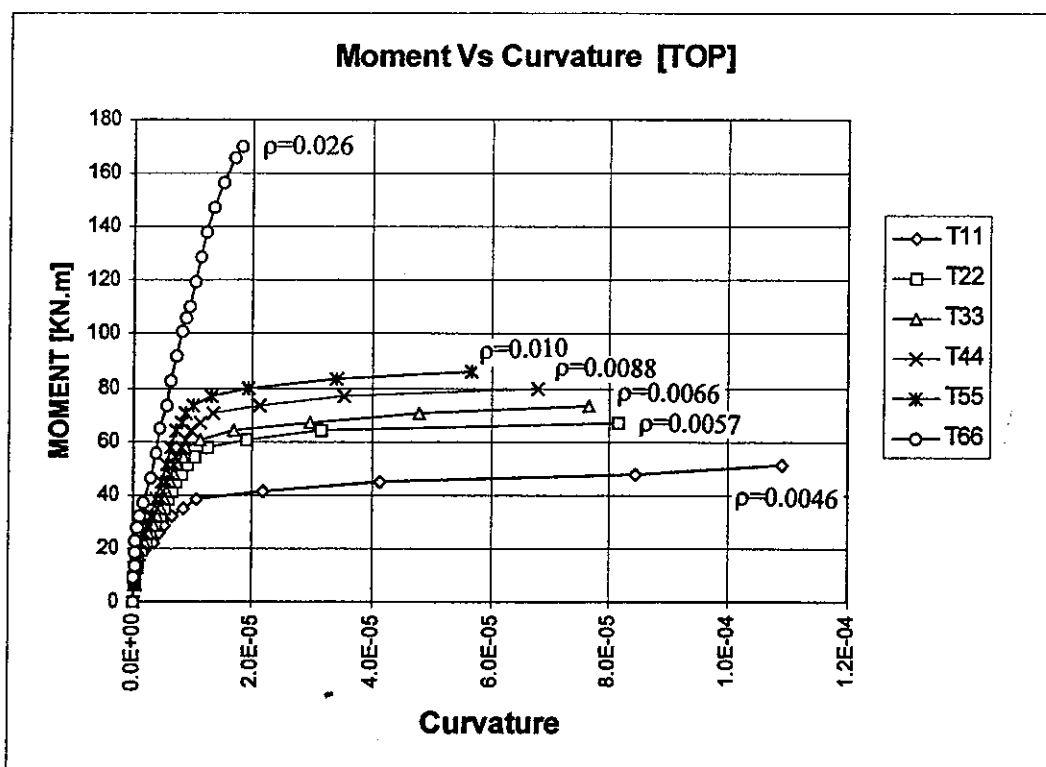


Fig. 4.2..Moment vs Curvature for Top overlays with different steel ratios

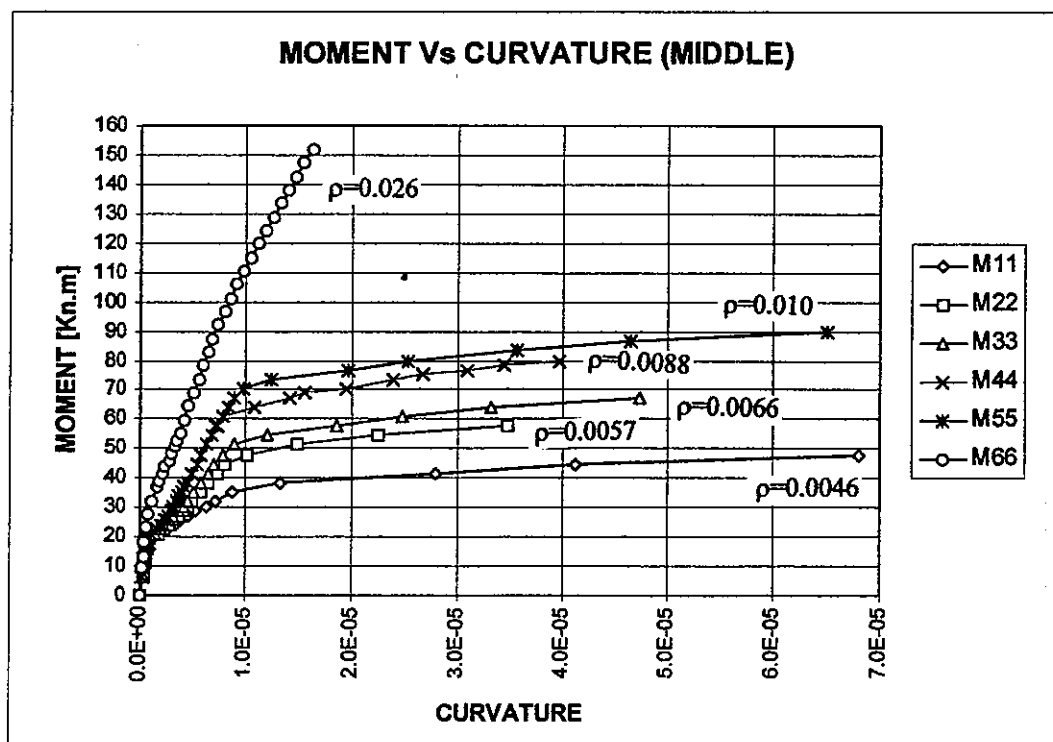


Fig. 4. 3..Moment vs Curvature for Middle Segment with different steel ratios

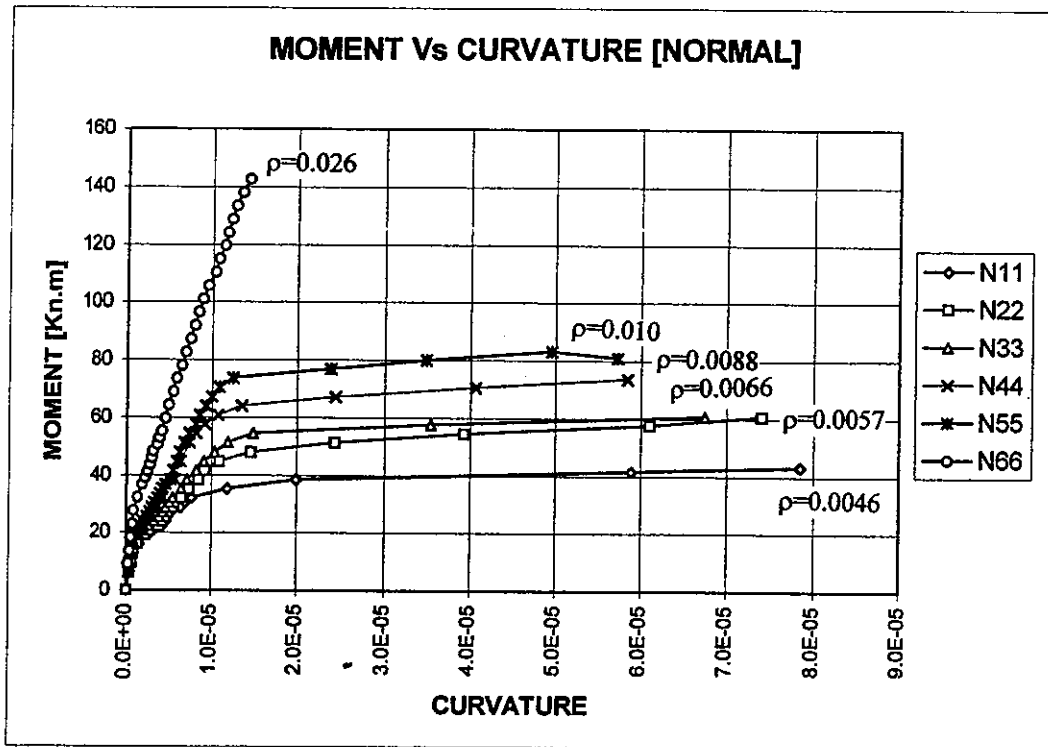


Fig. 4.4..Moment vs Curvature for Normal Beams with different steel ratios

The moment-curvature diagrams shown in Figs. 4.1, 4.2 and 4.3 for beams with bottom and top overlays being HPC and middle segment being HPC clearly verifies the fact that the larger the steel content, the smaller the curvature and the higher the cracking moment. The computation of the cracking moment and the curvature is given in detail in appendix C. Fig. 4.4 shows the same pattern for the normal beams.

- **Yielding Moment (My)**

The Figures 4.1 to 4.4 also verify that the increase in the steel content increases the yielding moment. This is certainly the result of the following equation

$$M_y = A_s \cdot F_y \cdot (d - k_2 c) \quad (4-2)$$

where (A_s) is the tension steel in (mm^2), (F_y) the yielding stress of the steel in (N/mm^2), (d) the distance from the center of the tension steel to the extreme compression fiber, (c) the depth of concrete block measured from the extreme compression fiber to the neutral axis and (k_2) is a stress block parameter at the flexural strength of rectangular section.

The increase of steel in tension zone directly increases the concrete stress block depth (c) to balance the section resulting in higher yielding moment. For more detail refer numerical examples presented in appendix C.

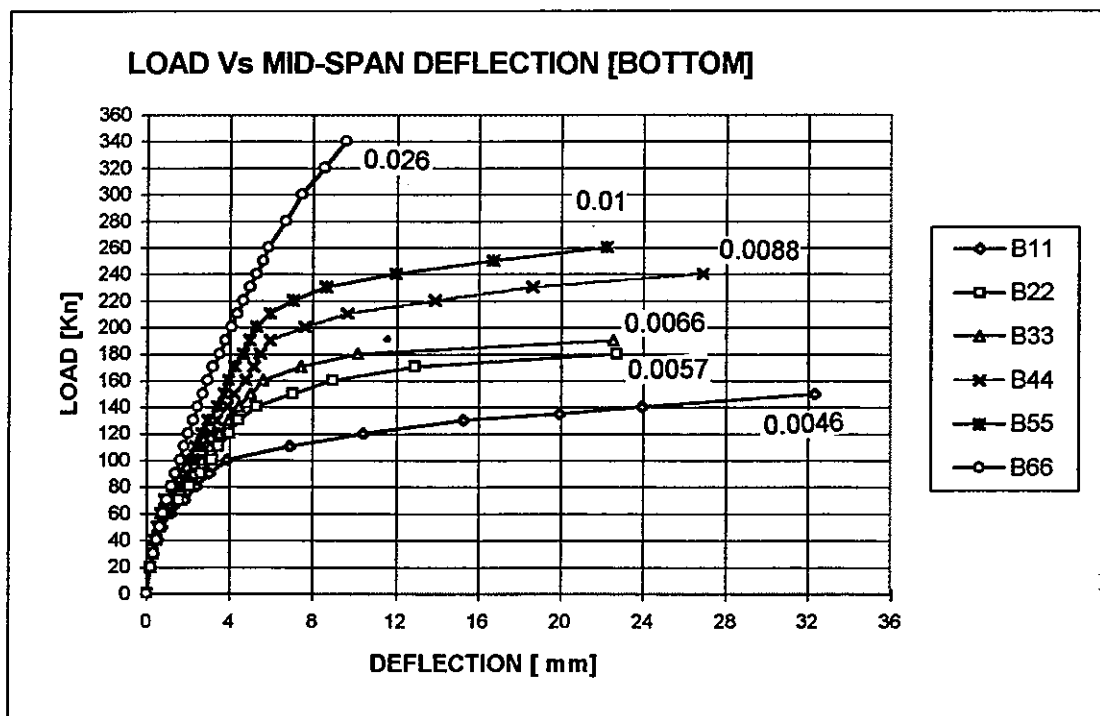


Fig 4.5. Load Vs mid-span deflection for bottom overlay being HPC with different steel ratio

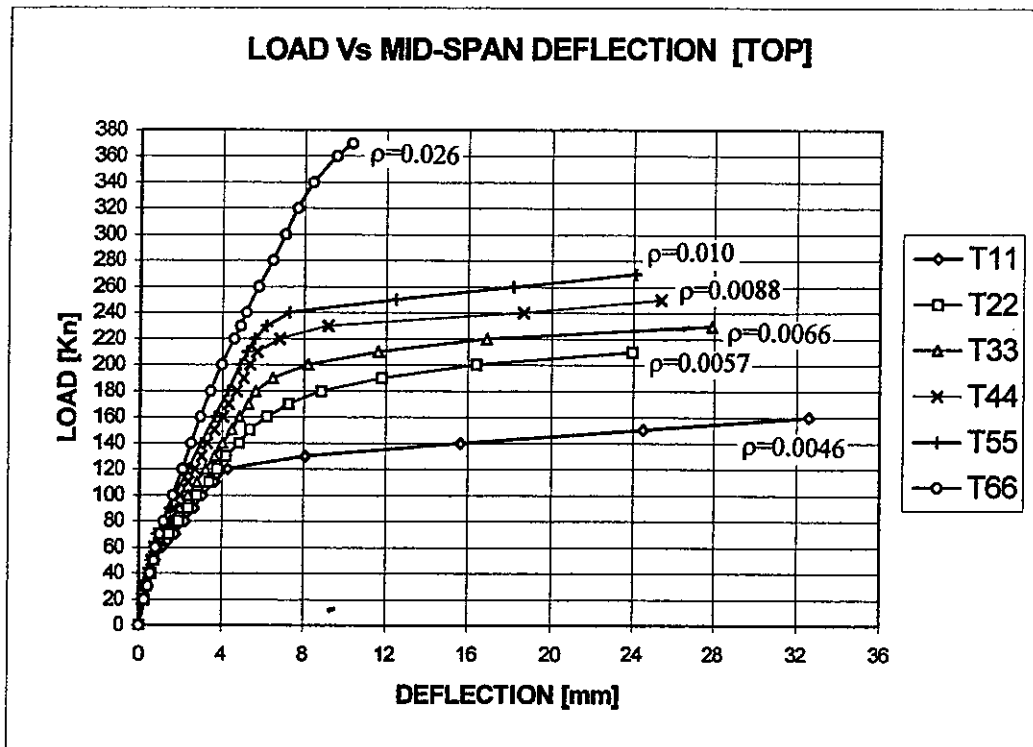


Fig 4.6. Load Vs mid-span deflection for Top overlay being HPC with different steel ratio

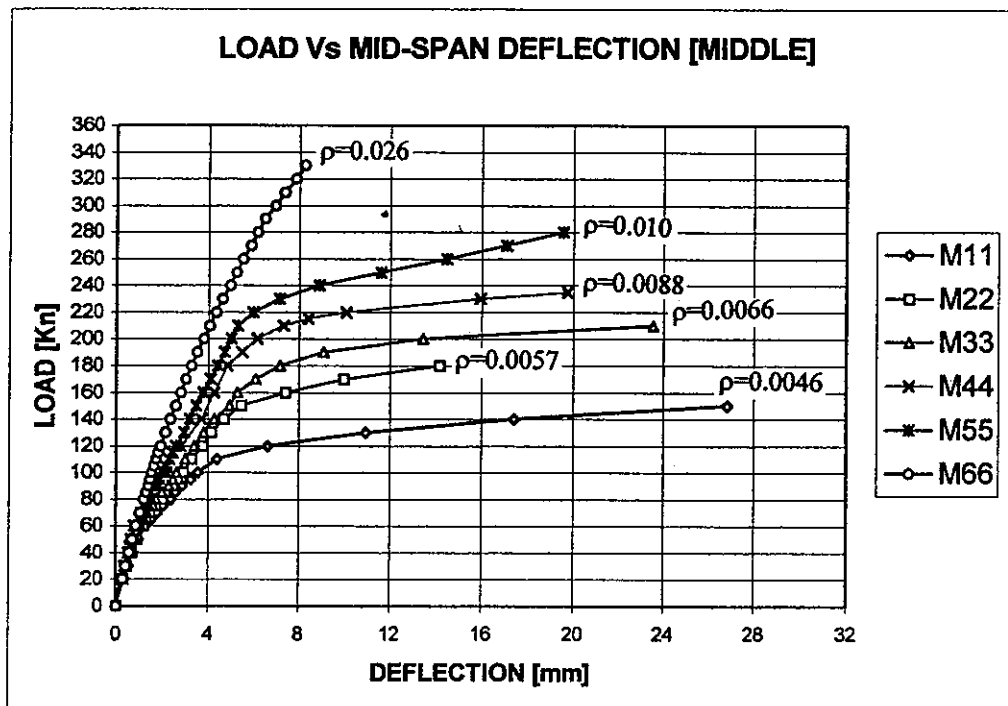


Fig 4.7. Load Vs mid-span deflection for Middle Segment being HPC with different steel ratio

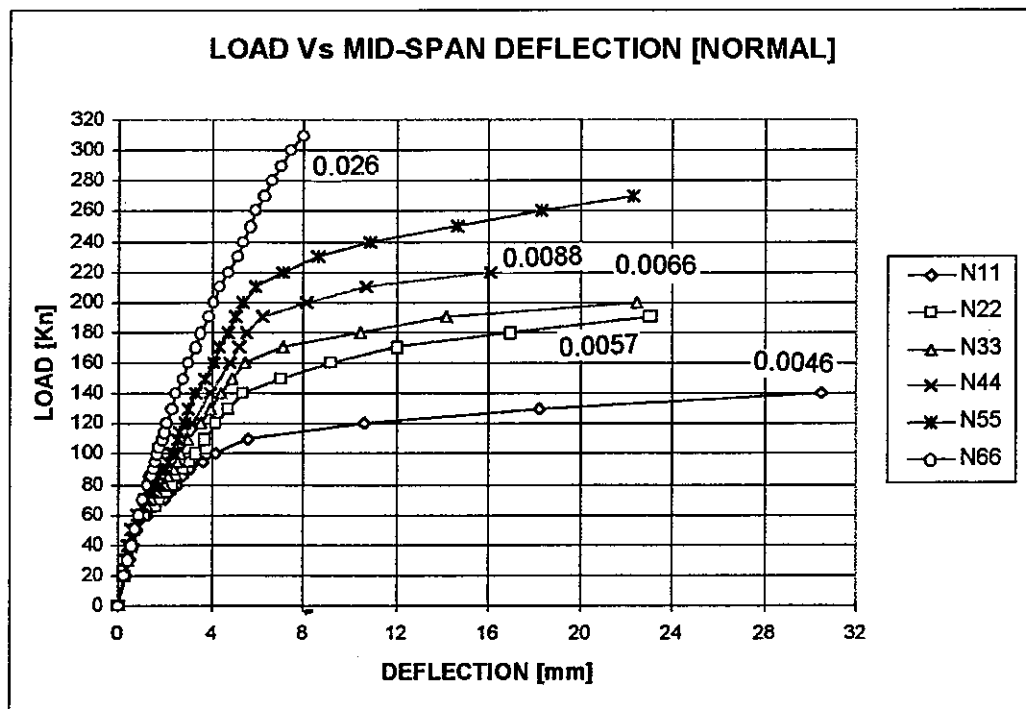


Fig 4.8. Load Vs mid-span deflection for Normal concrete with different steel ratio

• Mid-Span Deflection.

The curves shown in Figs. 4.5 to 4.8 give mid-Span deflections for beams with different type of overlays. They clearly show the effect of steel content on the mid-span deflection. It can be concluded from these graphs that the higher the steel contents the smaller the deflection. The values of the mid-span deflections for different steel ratios are given in Table 4.1.

Table 4.1. Mid-Span Deflection at Service Load of Normal Concrete Beam Compared to beams with different overlays .

Steel Ratio	Service Load [kN.]	Normal [mm]	Bottom [mm]	Top [mm]	Middle [mm]
0.0046	67	1.91	1.56	1.56	1.54
0.0057	85	2.66	2.18	2.12	2.16
0.0066	101	2.92	2.57	2.46	2.41
0.0088	132	3.72	3.34	3.22	3.15
0.0100	155	4.01	3.78	3.55	3.47
0.0260	246	5.54	5.40	5.00	5.08

4.3. The effect of overlays on the beam properties

- Cracking Moment (M_{cr})

The strains measured at the extreme top and extreme bottom fiber of the beam are plotted against the moment in Figures 4.9, 4.10, 4.11, 4.12, 4.13, and 4.14. Both compressive and tensile strains are shown in the same graphs where the strains with negative values represent the compressive strain. Each figure contains total of eight curves. Each curve is represented with two letters and one number. The capital letter represents the type of beam overlay, B-for bottom overlay, T-is for top overlay, M-for middle segment being HPC, N-stands for normal concrete. The following number represents the steel ratio group and the last small letter stands for the strain, where c-is for compression and t-is for tension.

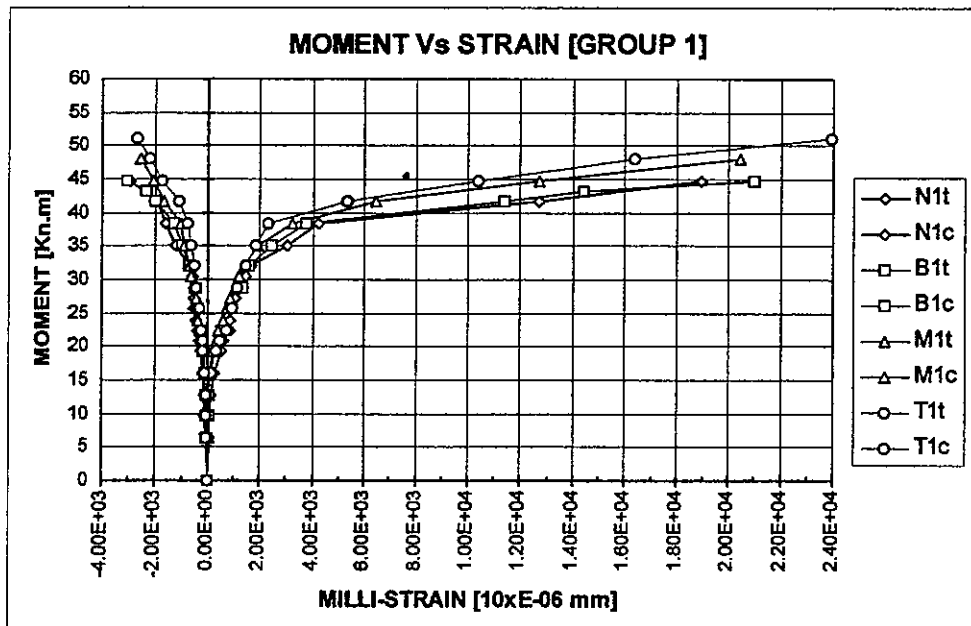


Fig. 4.9. Moment Vs. Strain for different overlays, reinforcement group no.1
($\rho = 0.0046$)

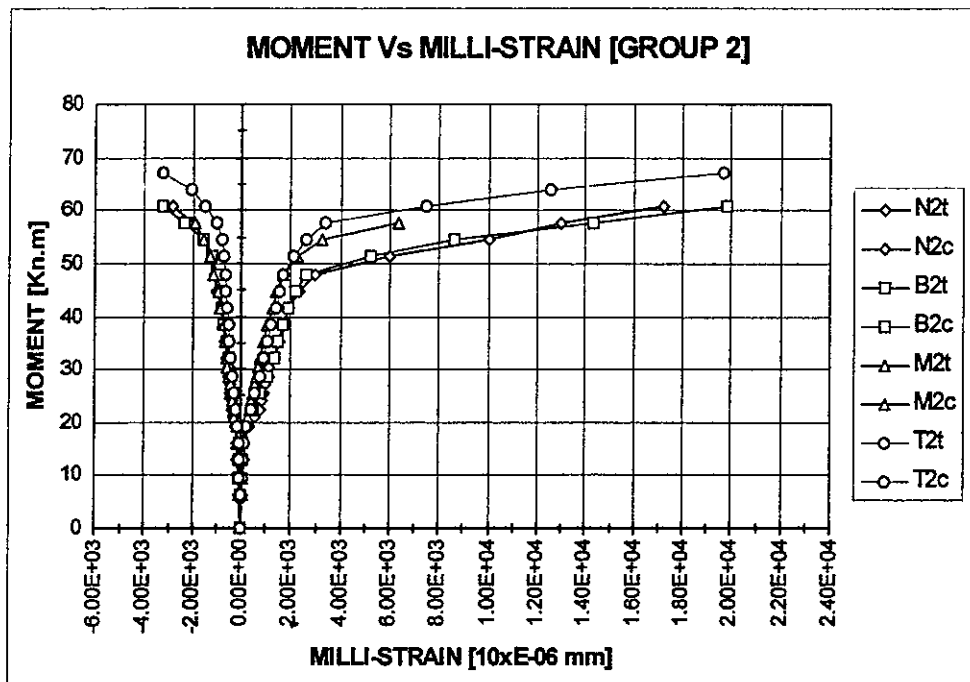


Fig. 4.10. *Moment Vs. Strain for different overlays, reinforcement group no.2*
($\rho = 0.0057$)

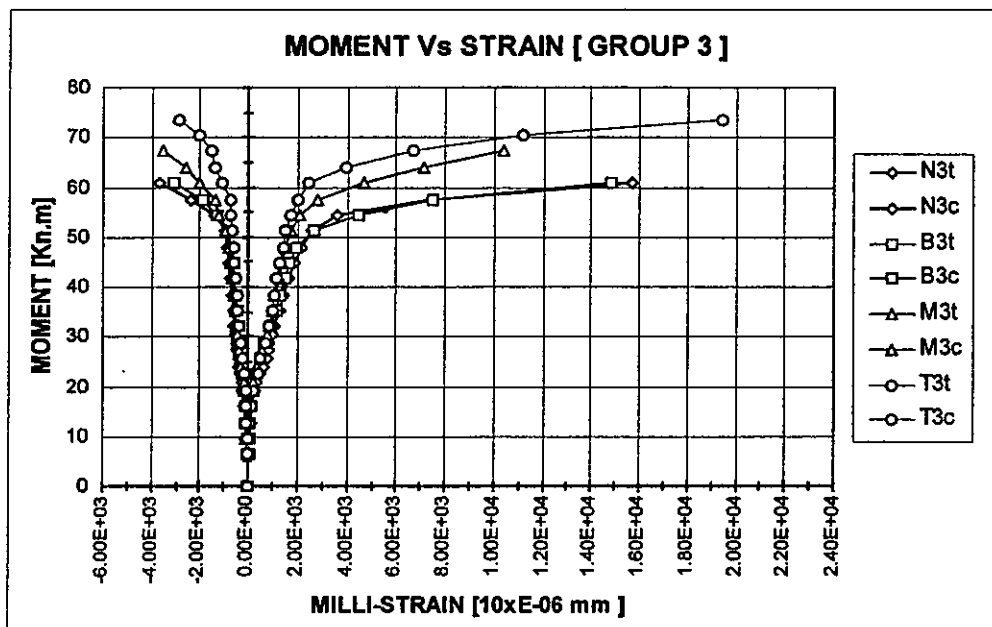


Fig. 4.11. *Moment Vs. Strain for different overlays, reinforcement group no.3*
($\rho = 0.0066$)

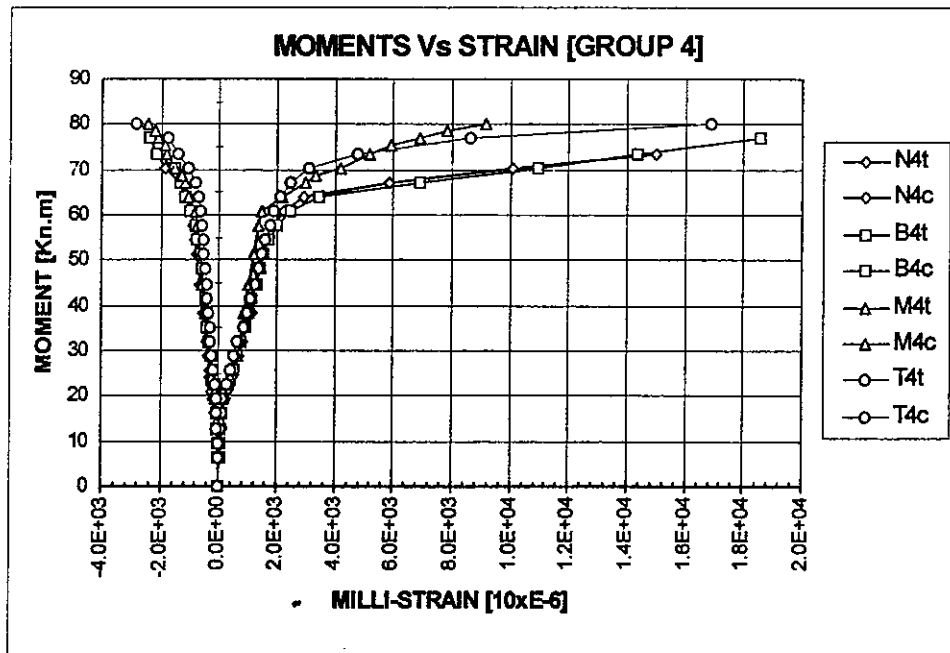


Fig. 4.12. Moment Vs. Strain for different overlays, reinforcement group no.4
($\rho = 0.0088$)

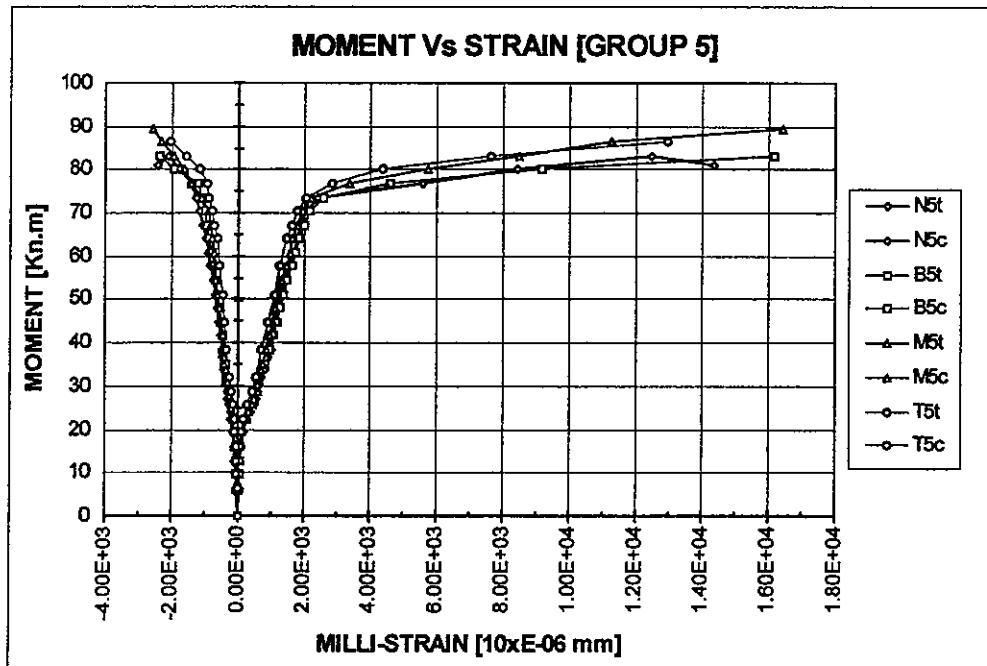


Fig. 4.13. Moment Vs. Strain for different overlays, reinforcement group no.5
($\rho = 0.010$)

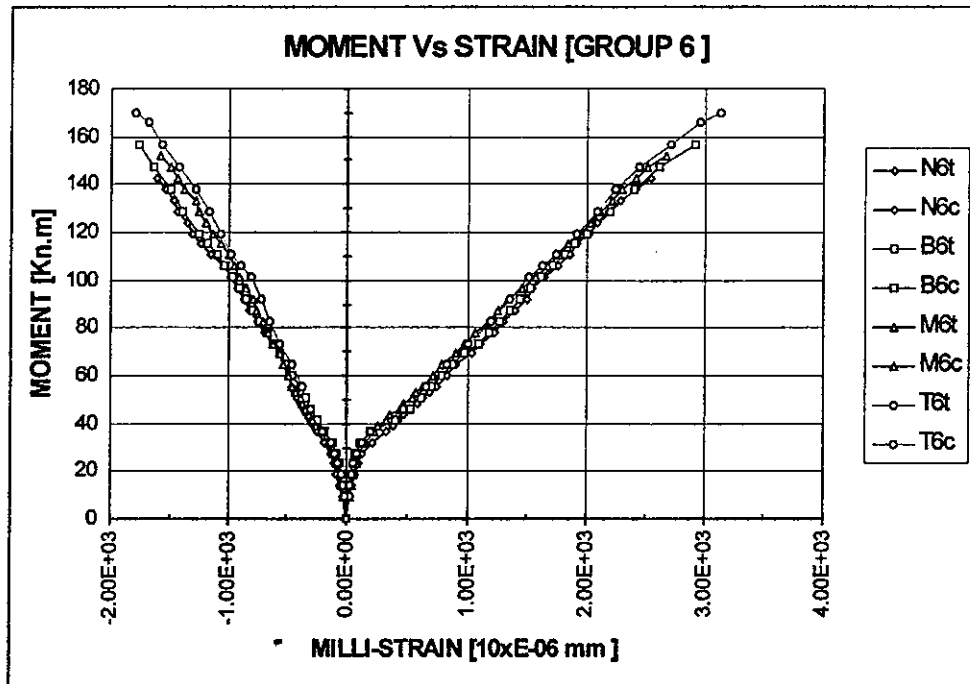


Fig. 4.14. *Moment Vs. Strain for different overlays, reinforcement group no.6 ($\rho = 0.026$)*

The cracking moment (M_{cr}) has been calculated using (4.1) and listed in Table 4.2. for each reinforcement group.

Table 4.2. *The cracking Moment (M_{cr}) in kN.m obtained from the experiments for each group of reinforcement*

Type of overlay	Group (1)	Group (2)	Group (3)	Group (4)	Group (5)	Group (6)
	$\rho = 0.0046$	$\rho = 0.0057$	$\rho = 0.0066$	$\rho = 0.0088$	$\rho = 0.010$	$\rho = 0.026$
Normal	13.72	15.71	16.49	17.87	19.33	27.81
Bottom	17.16	19.30	20.80	22.23	23.85	33.43
Top	16.22	17.03	19.01	20.47	22.68	29.45
Middle	17.10	19.41	20.84	21.23	23.05	31.66

From the above table the following can be concluded:

- The beams with bottom overlay being HPC gives 24% increase in the cracking moment value.
- The beams with top overlay being HPC yields 13% increase in the cracking moment value.
- The beams with middle segment being HPC resulted in 21% increase in the cracking moment value.

- **Ductility**

Ductility is a factor represented by the ratio of the deformation at the ultimate load to the deformation at yielding. In the preliminary study it was assumed that the ultimate load is the load at which the extreme compression strain reaches a value of 0.003. In the experimental program this value was not recorded in most of the beams due-to some difficulties in holding the strain gages up to the end of the test. In some cases strain gages fall down, or started giving some uncertain readings. This was mostly happened because of the extended length of the strain gages. Due-to the above mentioned reasons, It was assumed that the beam reaches to its ultimate strength whenever the compression strain reaches to a value of 0.0025.

The curvature was calculated from average data and represented in the following charts. Each chart represents a group of four beams with different overlays with the same steel content.

Moment- curvature graphs are plotted in figs. 4.15, 4.16, 4.17, 4.18, 4.19 and 4.20 for each group of reinforcement ratio. Each graph contains curves belonging to each type of the beam. In these graphs N-stands for beams with normal concrete, B-stands for beams with bottom overlay HPC, M-stands for middle segment HPC and T-represents beams with top overlay HPC.

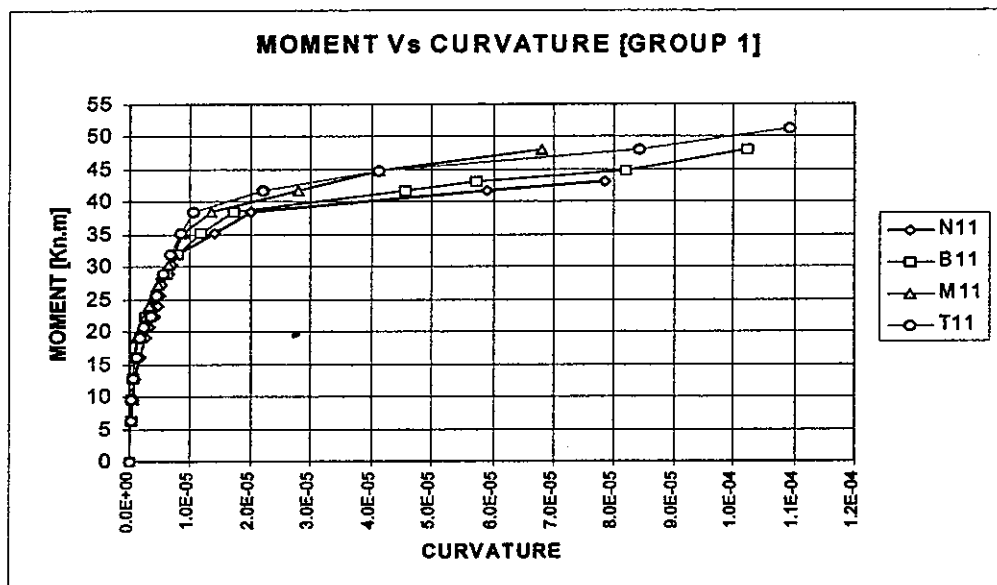


Fig. 4.15. Comparison of moment *Vs* Curvature of beams with different overlays having the same steel ratio ($\rho = 0.0046$)

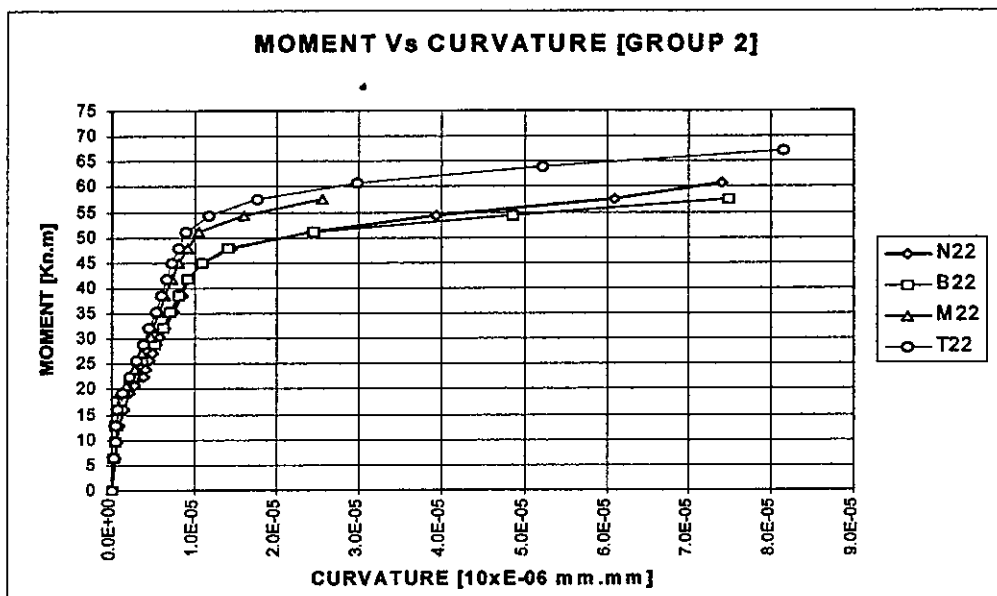


Fig. 4.16. Comparison of moment *Vs* Curvature of beams with different overlays having the same steel ratio ($\rho = 0.0057$)

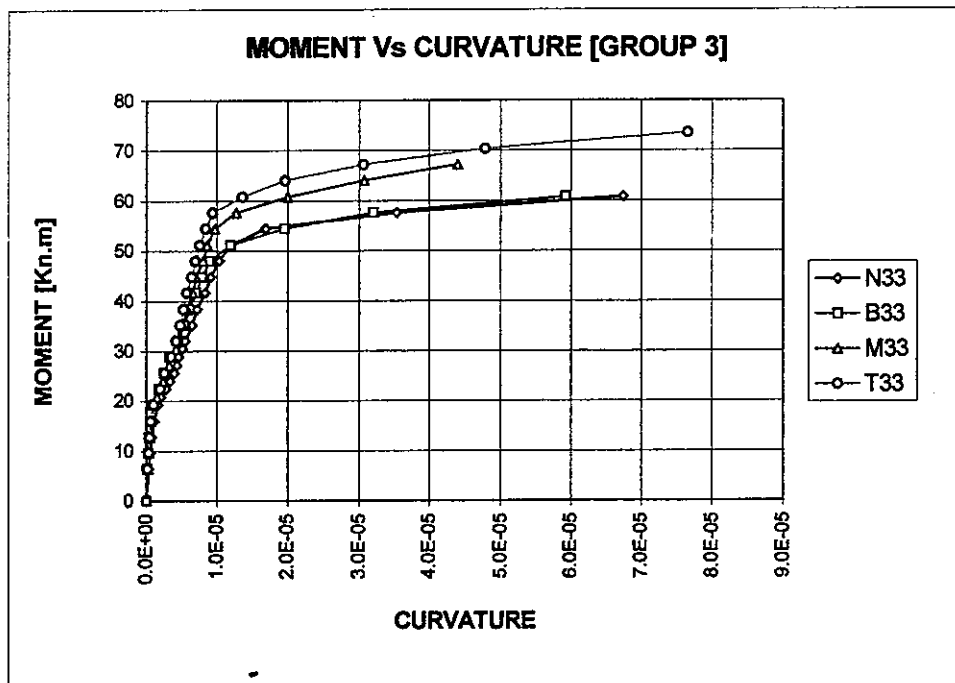


Fig. 4.17. Comparison of moment Vs Curvature of beams with different overlays having the same steel ratio ($\rho = 0.0066$)

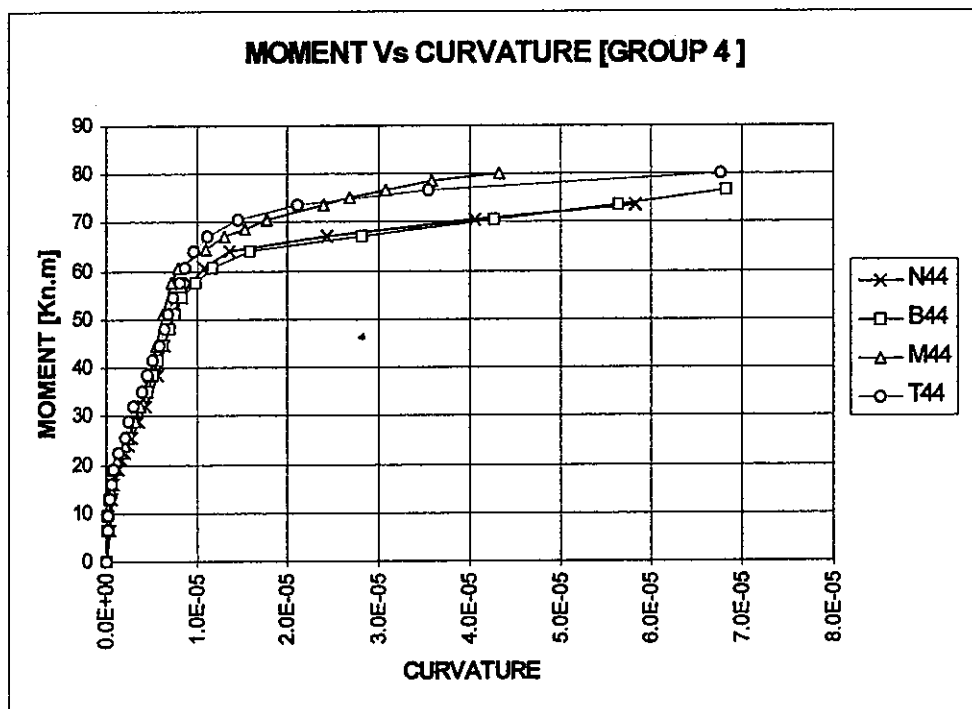


Fig. 4.18 Comparison of moment Vs Curvature of beams with different overlays having the same steel ratio ($\rho = 0.0088$)

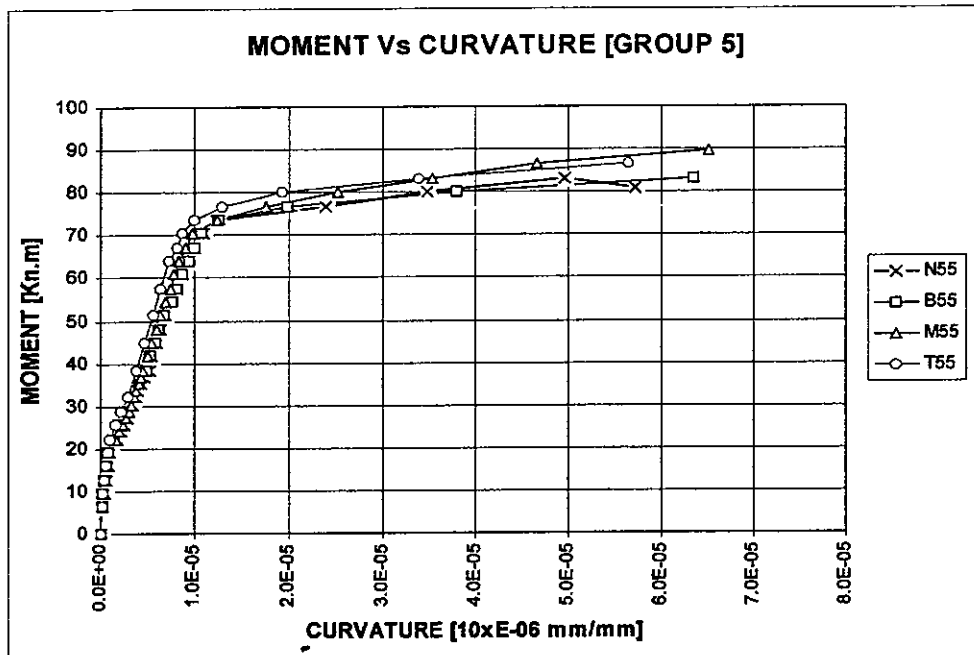


Fig. 4.19. Comparison of moment Vs Curvature of beams with different overlays having the same steel ratio ($\rho = 0.010$)

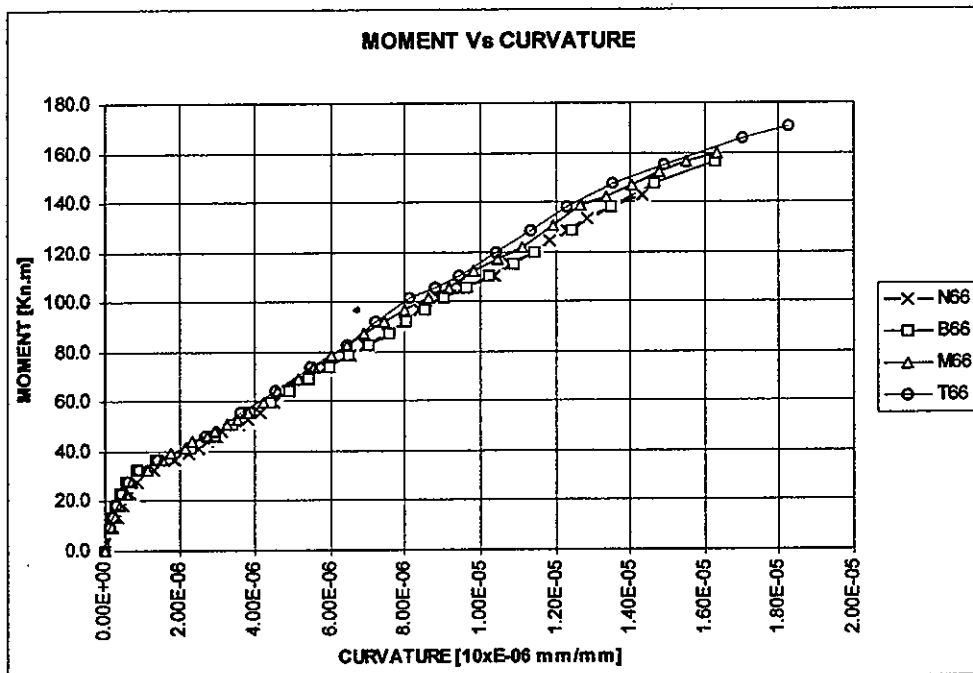


Fig. 4.20. Comparison of moment Vs Curvature of beams with different overlays having the same steel ratio ($\rho = 0.026$)

The curvature at yielding moment (M_y) as well as the curvature at ultimate moment (M_u) were calculated from the above charts and were represented in the following table for easy comparison and evaluation of ductility

Table 4.3. *Yielding Moment (M_y) in kN.m as calculated experimentally*

Type of overlay	Group (1)	Group (2)	Group (3)	Group (4)	Group (5)	Group (6)
	$\rho=0.0046$	$\rho=0.0057$	$\rho=0.0066$	$\rho=0.0088$	$\rho=0.01$	$\rho=0.026$
Normal	34.82	47.15	53.72	63.58	74.03	*
Bottom	36.16	46.55	51.45	62.13	74.02	*
Top	38.60	55.17	61.88	67.99	76.58	167.05
Middle	37.36	54.24	56.87	67.00	75.94	*

* Data is not available

Table 4.4. *Ultimate Moment (M_u) in kN.m as calculated experimentally for normal concrete at compression strain of 0.0025*

Type of overlay	Group (1)	Group (2)	Group (3)	Group (4)	Group (5)	Group (6)
	$\rho=0.0046$	$\rho=0.0057$	$\rho=0.0066$	$\rho=0.0088$	$\rho=0.01$	$\rho=0.026$
Normal	43.37	59.61	62.17	75.10	86.08	*
Bottom	43.50	57.55	61.73	75.36	83.87	*
Top	49.98	67.00	73.09	79.76	89.52	*
Middle	47.52	*	70.03	79.93	89.35	*

* Data is not available

Table 4.5. *Curvature at ultimate (ϕ_u) / curvature at yielding (ϕ_y) as obtained experimentally*

Type of overlay	$\rho=0.0046$	$\rho=0.0057$	$\rho=0.0066$	$\rho=0.0088$	$\rho=0.01$	$\rho=0.026$
	ϕ_u/ϕ_y	ϕ_u/ϕ_y	ϕ_u/ϕ_y	ϕ_u/ϕ_y	ϕ_u/ϕ_y	ϕ_u/ϕ_y
Normal	5.33E-05	5.02E-05	5.34E-05	4.87E-05	5.02E-05	*
	1.16E-05	1.11E-05	1.20E-05	1.11E-05	1.18E-05	*
Bottom	5.18E-05	4.79E-05	4.69E-05	4.91E-05	4.89E-05	*
	1.11E-05	1.06E-05	1.05E-05	1.12E-05	1.13E-05	*
Top	6.73E-05	6.21E-05	6.14E-05	5.65E-05	5.43E-05	*
	1.01E-05	1.02E-05	1.09E-05	1.05E-05	1.07E-05	1.43E-05
Middle	6.47E-05	*	5.87E-05	5.29E-05	5.16E-05	*
	1.09E-05	1.38E-05	1.15E-05	1.09E-06	1.16E-05	*

* Data is not available

Table 4.6. *Ductility factor as calculated experimentally*

Group of steel	Normal concrete	Bottom overlay	Top overlay	% Improve.	Middle overlay	% Improve.
1	4.60	4.67	6.66	44.78%	5.94	29.13%
2	4.52	4.52	6.10	34.96%	*	*
3	4.45	4.47	5.63	26.52%	5.10	14.61%
4	4.39	4.38	5.38	22.55%	4.85	10.48%
5	4.25	4.33	5.07	19.29%	4.45	4.71%
6	*	*	*	*	*	*

* Data is not available

The above tables of data represent the effect of different types of overlays on ductility. It can be concluded the following:

- The beams with bottom overlay HPC behave like normal beams.
- The beams with top overlay HPC and low steel ratio increases the ductility factor with almost 45% while beams with high steel ratios yields 19% increase in ductility.
- The middle segment being HPC increases the ductility factor by almost 29% for beams with low steel content and only 4.7% for beams with high steel content.

• Mid-Span Deflection at Service Load

The mid-span deflections against the applied loads recorded during the experiments of the beams with different overlays and different steel ratios are plotted in figures 4.21, 4.22, 4.23, 4.24, 4.25 and 4.26.

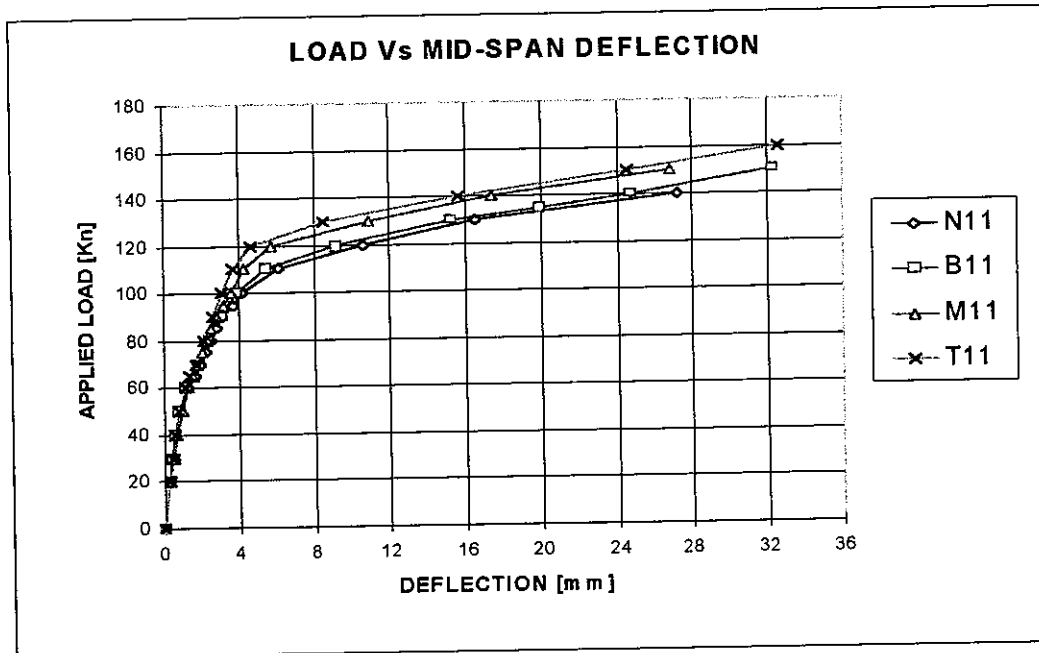


Fig. 4.21. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 1, $\rho=0.0046$).

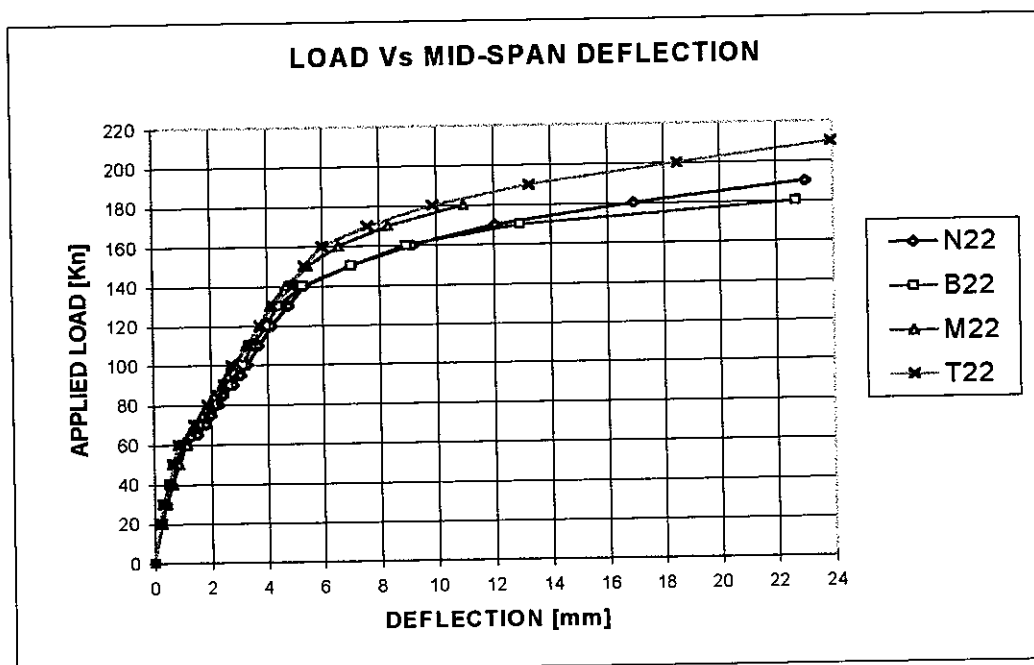


Fig. 4.22. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 2, $\rho=0.0057$).

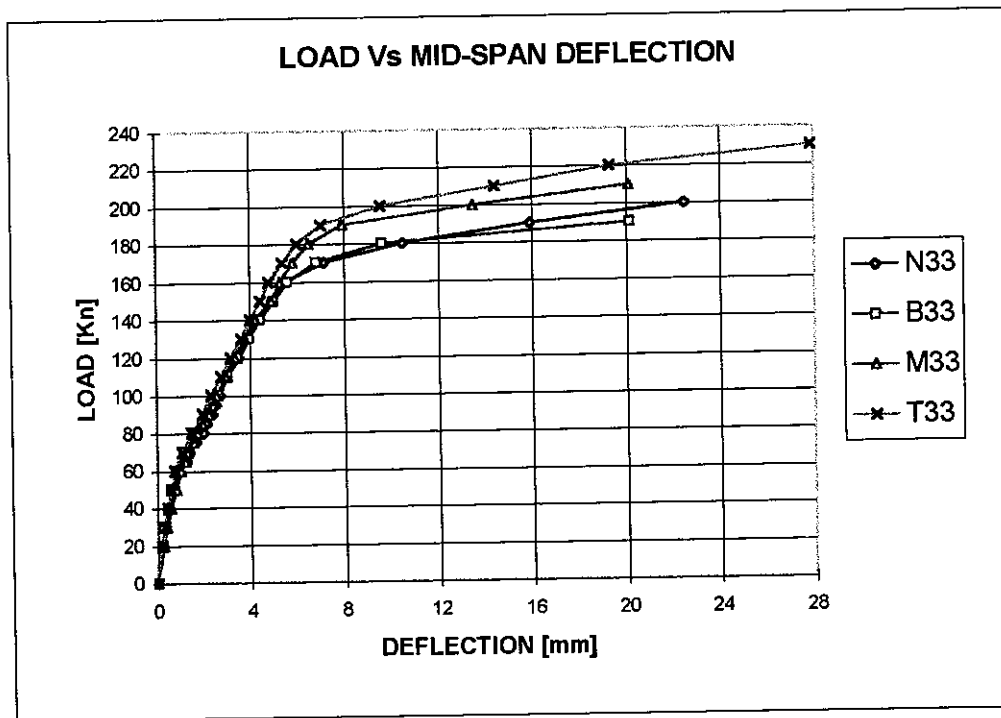


Fig. 4.23. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 3, $\rho=0.0066$).

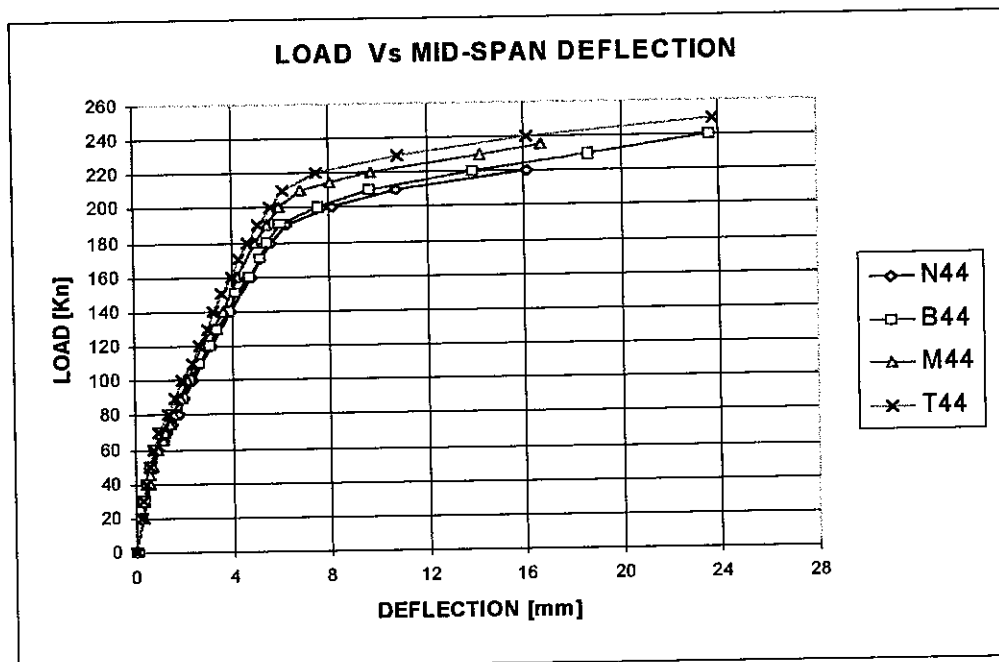


Fig. 4.24. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 4, $\rho=0.0088$).

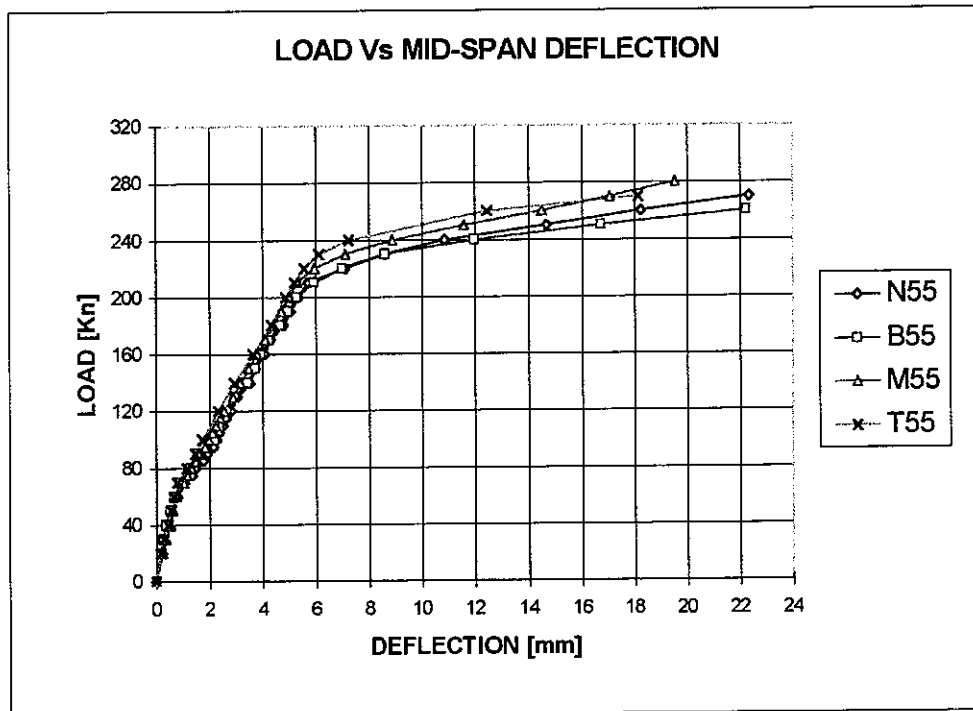


Fig. 4.25. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 5, $\rho=0.01$).

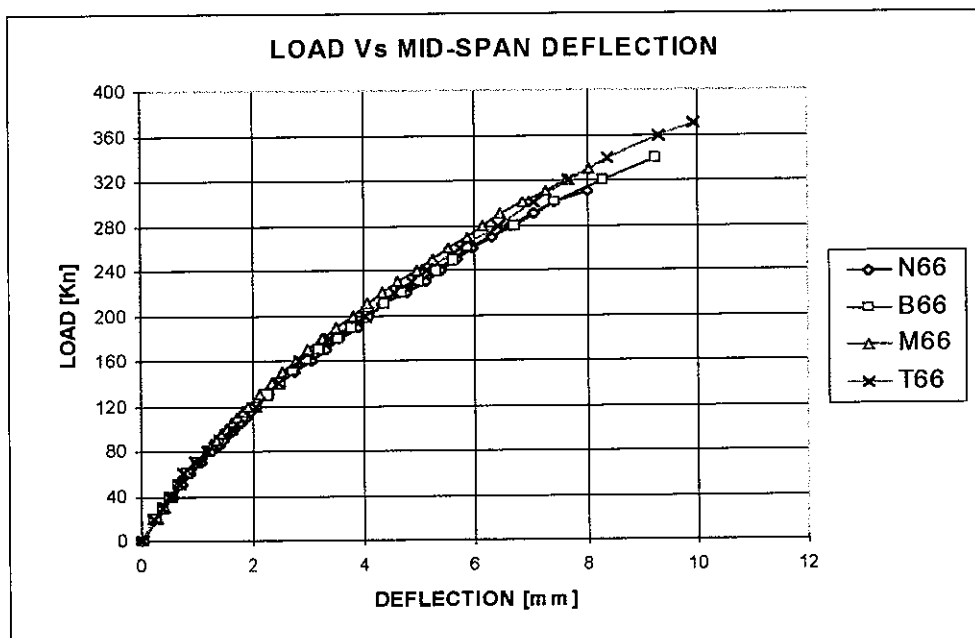


Fig. 4.26. Load Vs Mid-Span Deflection for Different type of overlays with the same steel content (Group 6, $\rho=0.026$).

Table 4.7. *Mid-span Deflection (δ) in mm. as obtained experimentally at service load*

Type of overlay	Group (1)	Group (2)	Group (3)	Group (4)	Group (5)	Group (6)
	$\rho=0.0047$	$\rho=0.0057$	$\rho=0.0069$	$\rho=0.0088$	$\rho=0.01$	$\rho=0.026$
Normal	1.91	2.66	2.92	3.82	4.01	5.54
Bottom	1.56	2.18	2.57	3.34	3.78	5.40
Top	1.52	2.12	2.46	3.22	3.55	5.00
Middle	1.54	2.16	2.41	3.15	3.47	5.08

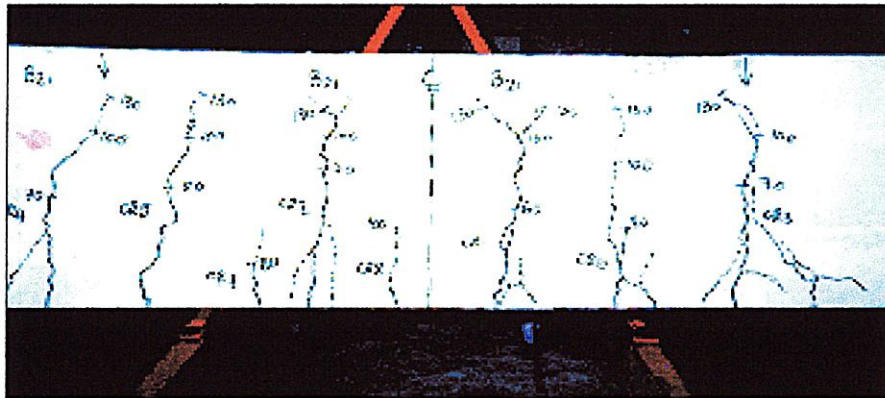
Table 4.7. represents the effect of different overlays on mid-span deflection obtained at service load ($M_{\text{service}}=(0.6-0.8)M_u$). The following conclusions are drawn from the data shown in table 4.7.

- The bottom overlay shows smaller deflection at service load for beams with low steel content and almost no improvement for beams with higher steel content. An improvement of almost 22.4% for lower steel content and 2.6% for higher steel content were recorded.
- The top overlay shows slightly better behavior from the deflection point of view and a value of 25.6% and 10.8% for lower and higher steel content simultaneously
- The middle segment being HPC behaves like the top overlays beams and exhibit an improvement of 24% and 9.1% for lower and higher steel content simultaneously.
- It can be concluded that the higher the steel contents the smaller the improvement in mid-span deflection.

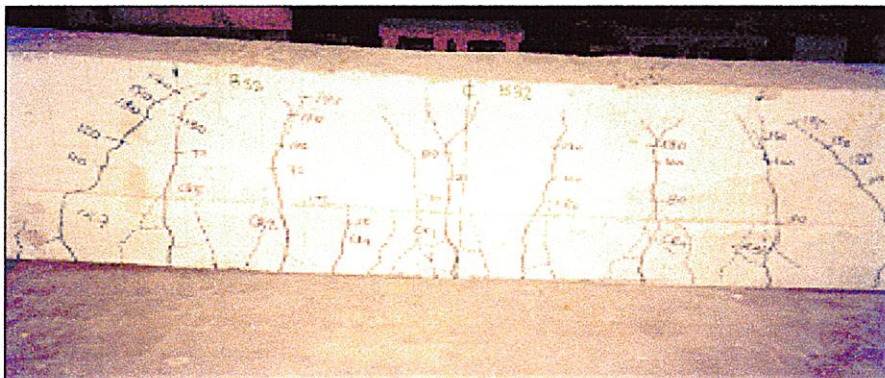
4.4. Mode of failure, Crack propagation and pattern

It is noticed that the behavior of the beams under the loading was not affected with overlays. All beams behaved normal as expected and failed in flexure, except for some beams with middle segments, the cracks first started at the line of interaction between the two types of concrete. However these cracks at interface have not affected the general behavior of the beam. The following pictures clearly reveal the above fact.

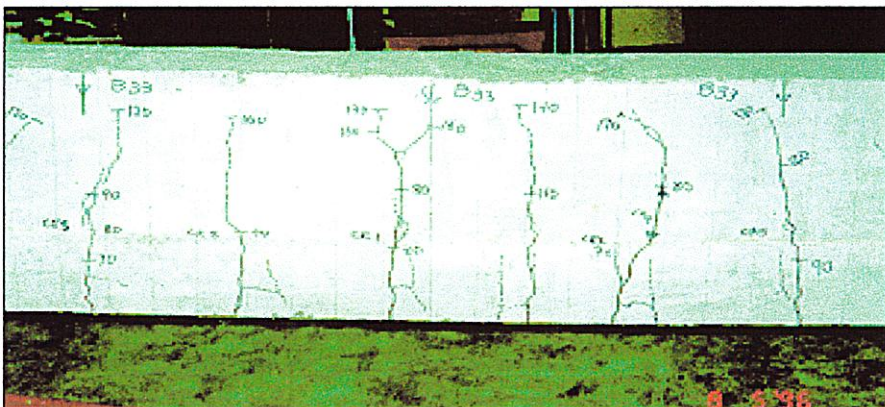
4.4.1. Beams with bottom overlays



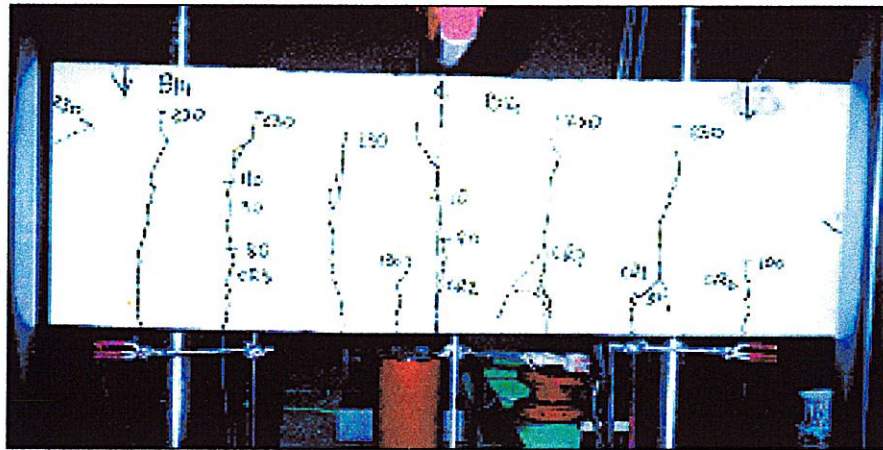
(1)



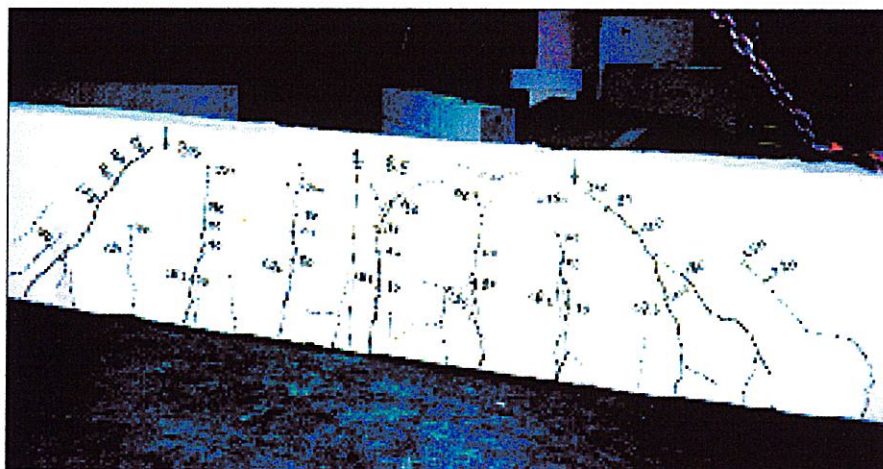
(2)



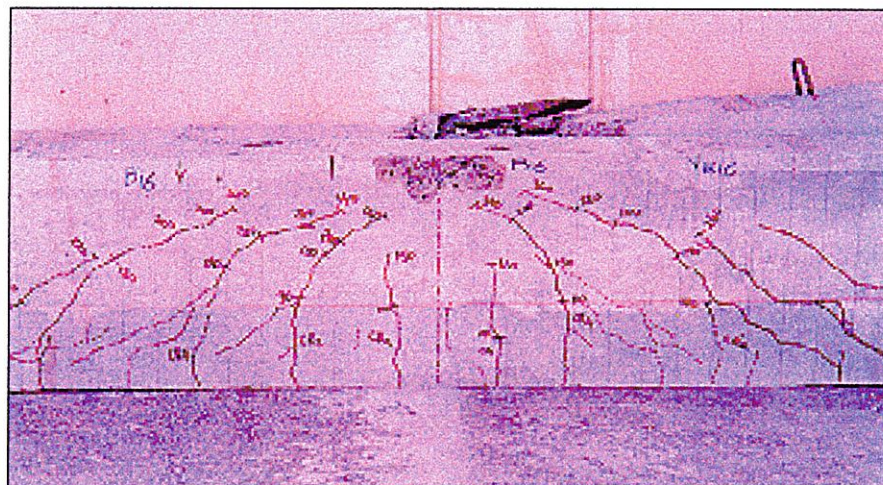
(3)



(4



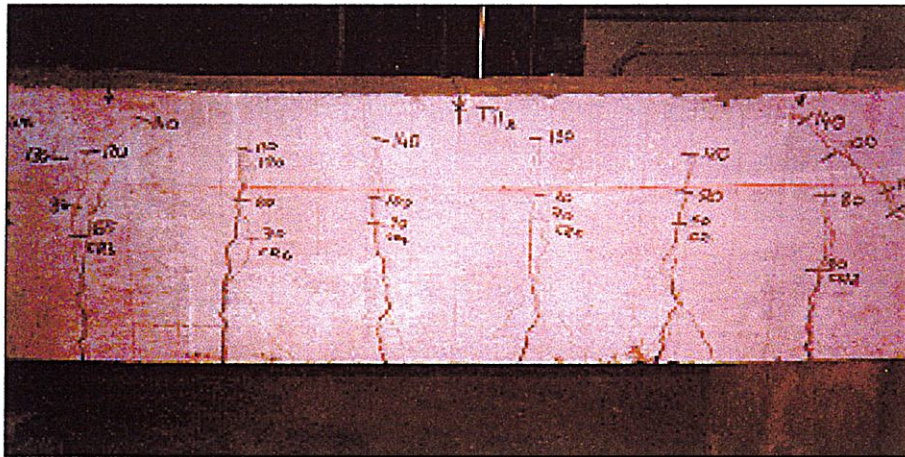
(5



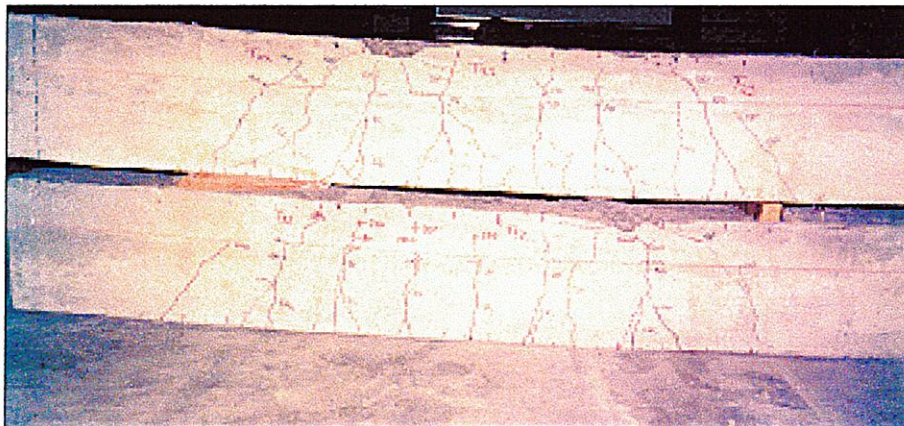
(6

Fig 4.27. Crack developments for beams with bottom overlays for groups of reinforcement 1,2,3,4,5,&6

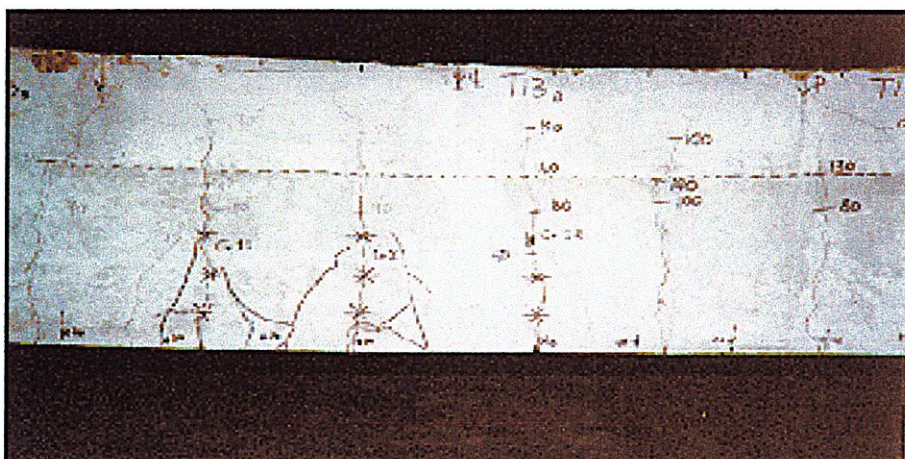
4.4.2. Beams with top overlays



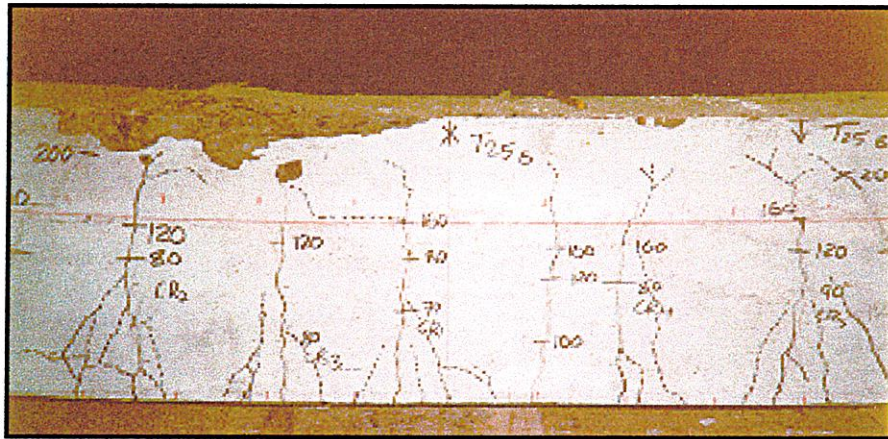
(1)



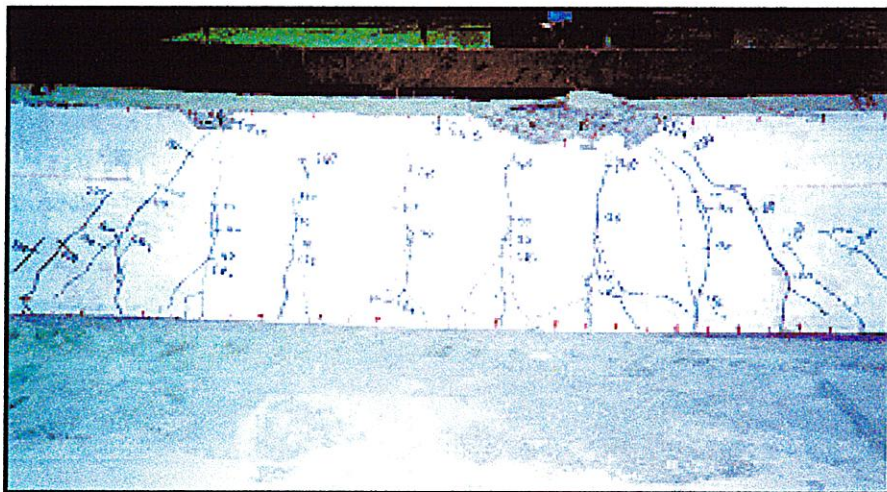
(2)



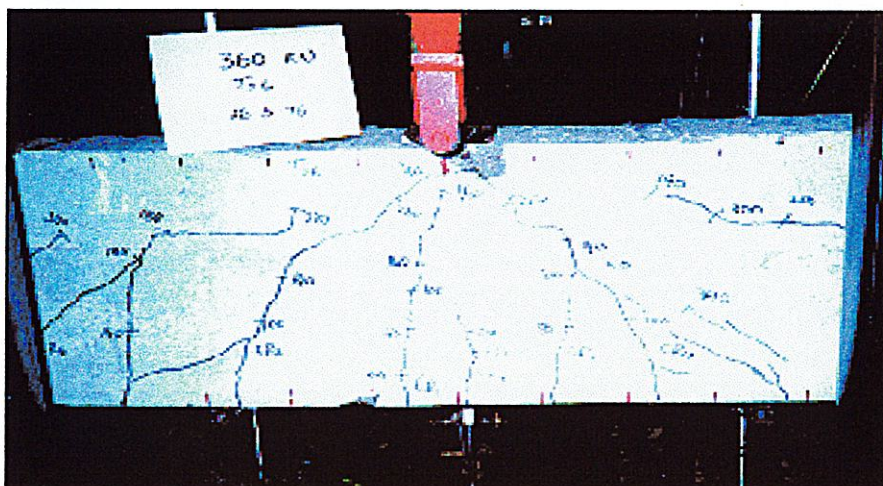
(3)



(4)



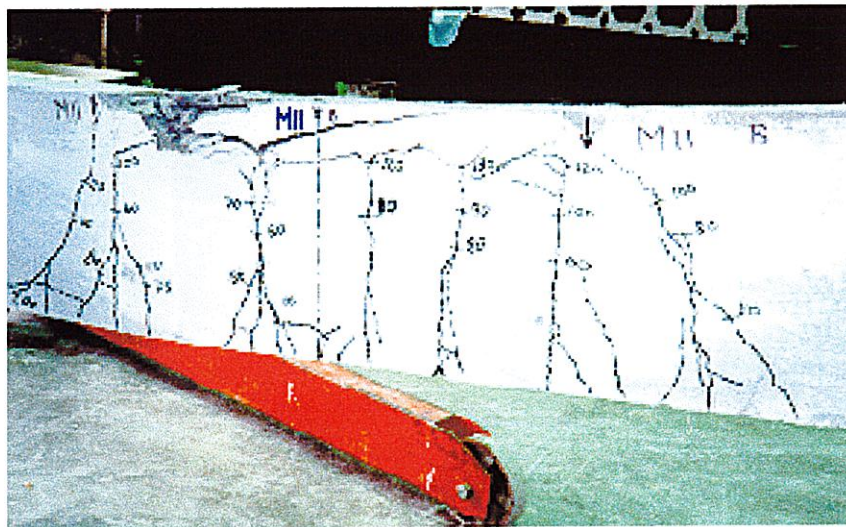
(5)



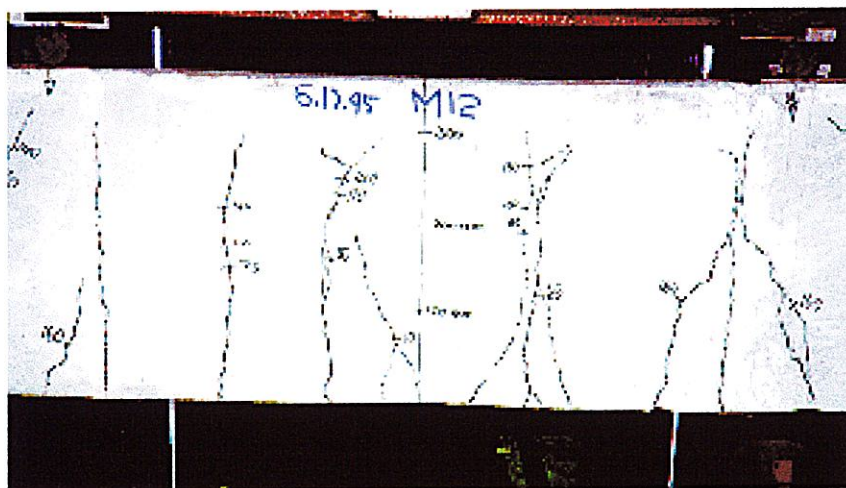
(6)

Fig 4.28 Crack developments for beams with top overlays for groups of reinforcement 1,2,3,4,5,&6

4.4.3. Middle segment being HPC



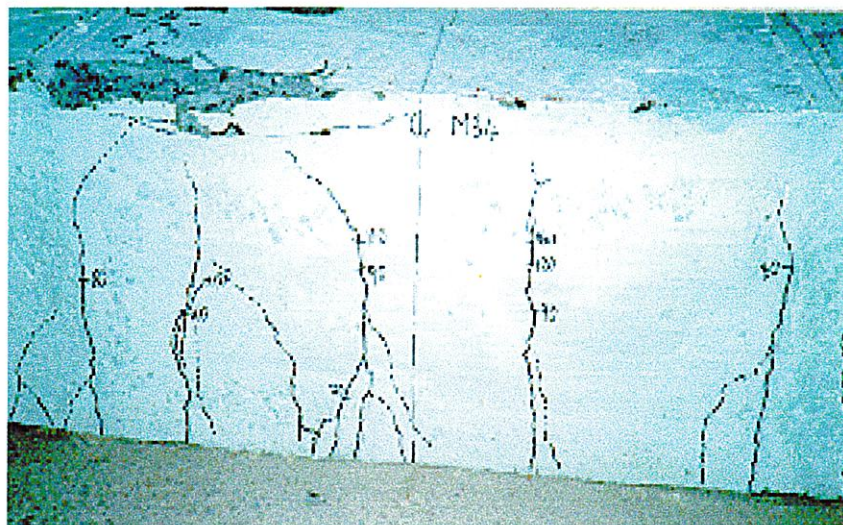
(1)



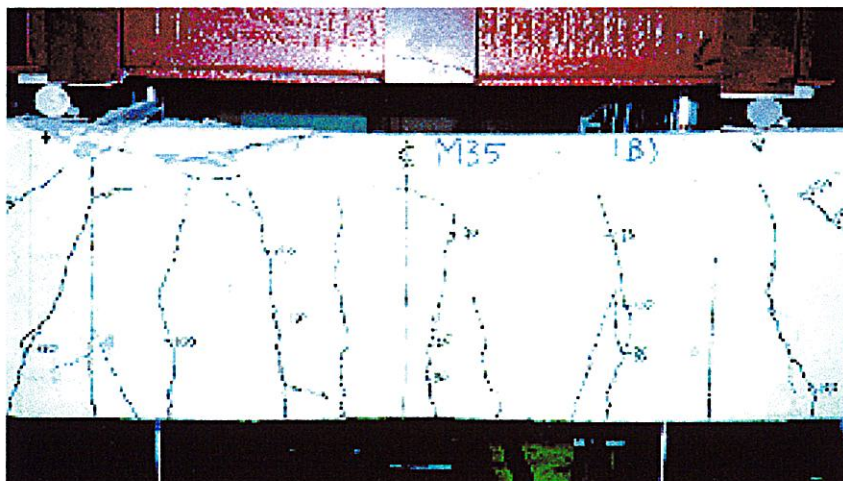
(2)



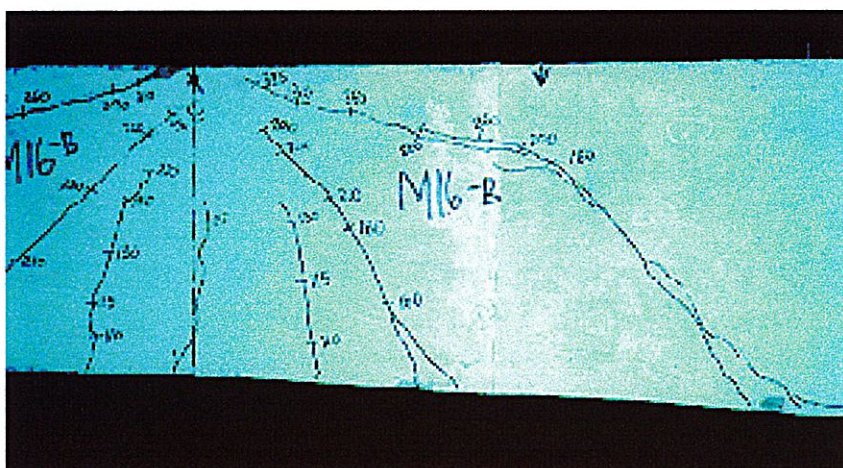
(3)



(4)



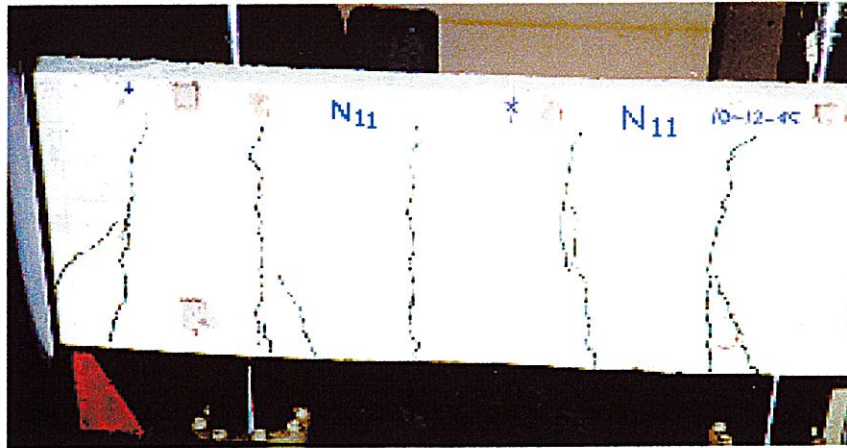
(5)



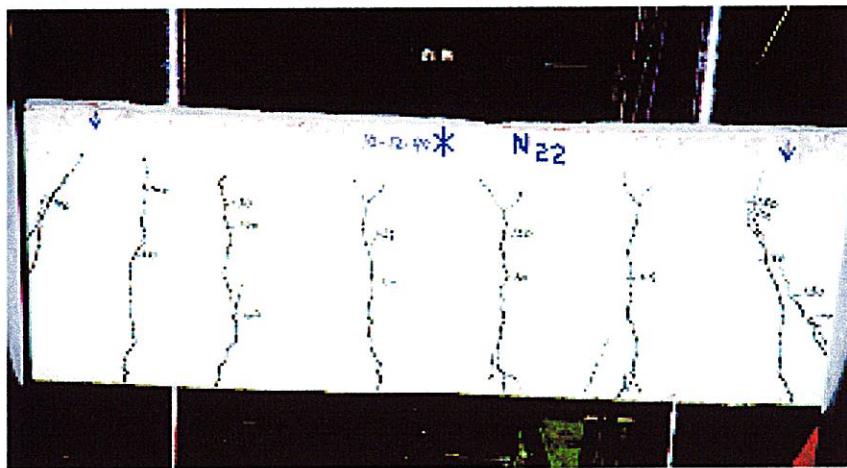
(6)

Fig 4.29. Crack developments for beams with middle segment being HPC. for groups of reinforcement 1,2,3,4,5,&6

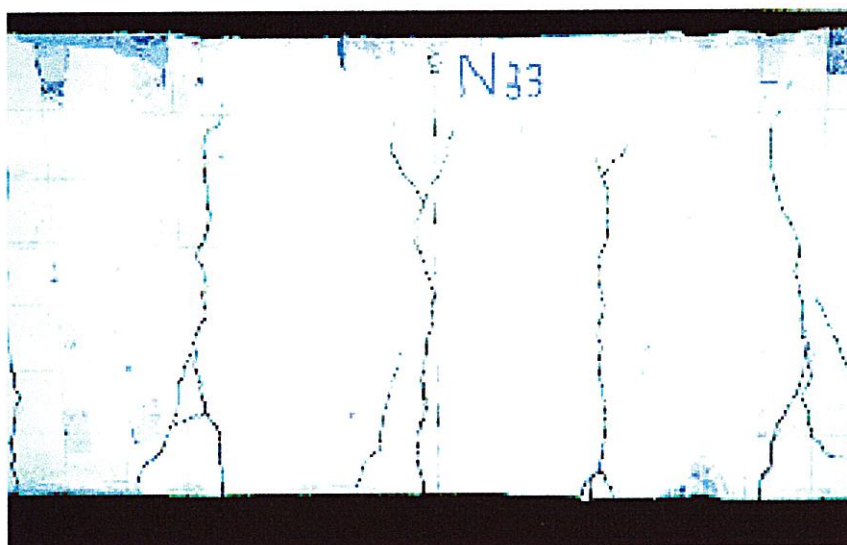
4.4.4. Normal concrete beams



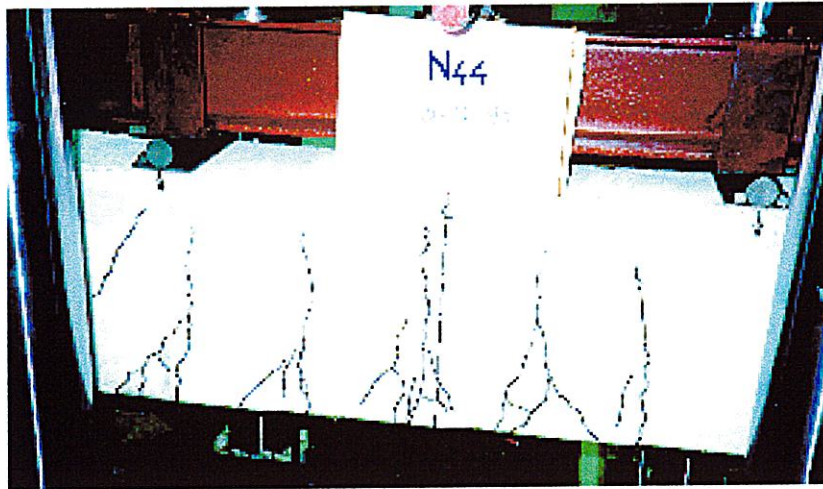
(1)



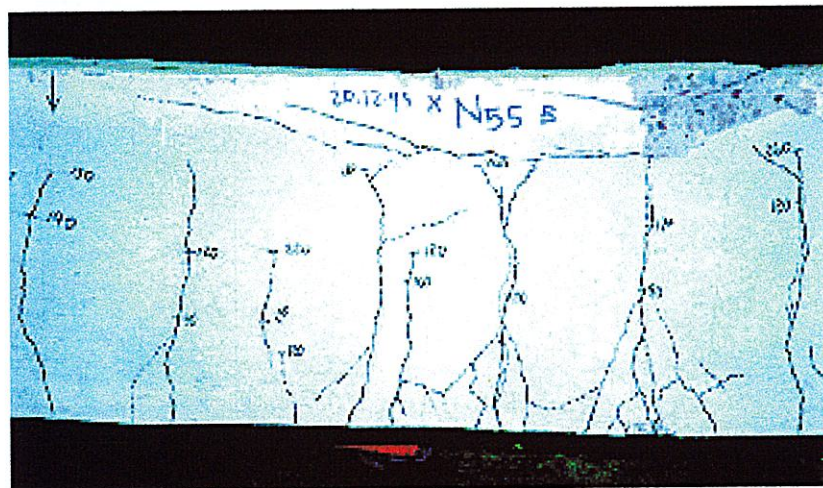
(2)



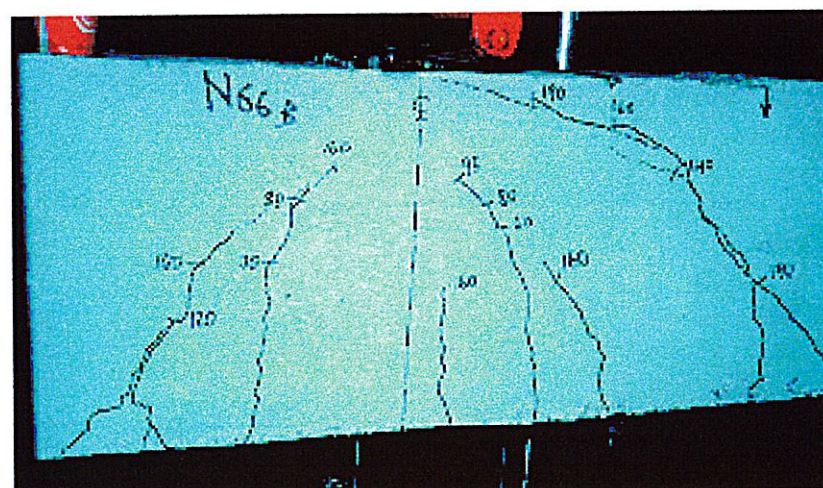
(3)



(4)



(5)



(6)

Fig 4.30. Crack developments of normal concrete beams for groups of reinforcement 1,2,3,4,5,&6

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION

From the analytical and experimental studies carried out in this work, the following conclusions can be outlined depending on the location of the HPC overlays.

5.1. Bottom overlays being HPC

It is observed that in beams having HPC overlay as their bottom part the cracking moment is improved about 24% regardless of the steel ratio. Hence, from the durability point of view, in cases where the environment is arid, bottom overlay is to be preferred.

It was noticed that there was no improvement in ductility and all beams with bottom overlay behaved like beams with normal concrete.

Hence, it is not advisable to use the bottom overlay where the ductility is the governing factor in the design of the beam.

Mid-span deflection at service load was improved with almost 22.4% for the beams with low steel content and 2.6% for the beams with high steel content. It is not recommended to use bottom overlays in heavily reinforced concrete beams since that the improvement is negligible.

5.2. Top overlays being HPC

For all the beams with top overlays the cracking moment was improved only 13%. The top overlays are not recommended for improving the cracking moment since the improvement percentage is small.

Ductility factor for beams with top overlays were improved by 43% for beams with lower steel content and 19% for beams with high steel content. It is worth to use top overlays in beams with low steel content and fairly acceptable to be used for beams with high steel content whenever the ductility is the main issue.

Mid-span deflection was improved by almost 25.6% for beams with low steel content and by 10.8% for beams with high steel content.

There is an advantage of using the top overlays to improve the mid-span deflection in beams with low as well as high steel content.

5.3. Middle segment being HPC.

In beams having HPC in their middle segment cracking moment was improved by 21% regardless the steel ratio. This is an important increase and

can be recommended in cases where the cracking moment is required to be improved.

Beams ductility with low steel content were improved by 27% and only by 4% for beams with higher steel content. It is not recommended to use middle segment to improve the ductility in beams with high steel content. It is apparent that this arrangement is beneficial in beams with low steel content.

The tests carried out on the beams with middle segment being HPC have shown that mid-span deflection was improved by 24% for the beams of low steel content and by 9.1% for the beams with high steel content.

5.4. Comparison of theoretical and experimental results

Table 5.1. summarizes the results of improvements obtained in the selected parameters through theoretical and experimental studies. It is clear that the results are close to each other.

Table 5.1. Comparison of theoretical and experimental results.

Beam Type	Bottom overlay		Top overlay		Middle Segment	
Parameters	Theoretical	Experimental	Theoretical	Experimental	Theoretical	Experimental
M_{cr}	26 %	24 %	6 %	13 %	N.A*	21 %
Ductility	0 %	0 %	40 %	43-19 %	N.A*	27-4 %
Deflection	20-0 %	23-3 %	11-16 %	26-11 %	N.A*	24-9 %

* (N.A-this type of beam was not considered in theoretical study)

The field application of this study at small sites is not appropriate for cast-in-situ concrete due-to the difficulties of handling the concrete stages and quality control.

The overlays can be recommended for precast beams for special structures where the workmanship and casting of different type of concrete can easily be controlled.

5.5. Cost implication of the partial use of HPC in normal concrete beams as a different layer

Based on the actual cost of one complete specimen including the cost of shutter, steel and concrete it can be concluded that the beams with overlays costs only 6 % more than the beams fully made of normal concrete, while the beams fully made of HPC costs as 25 % more. Hence, it is clear that this application economically is viable.

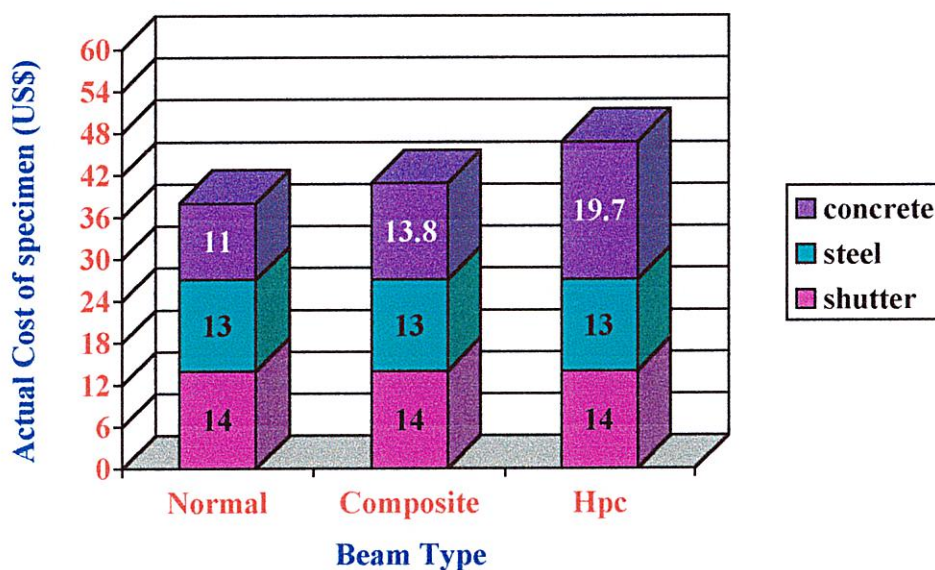


Fig. 5.1. Cost implication of concrete specimen

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REFERENCES

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APPENDICES

APPENDIX A

EXPERIMENTAL DATA RESULT OF BEAMS

Table A-1.1 *Experimental data result of beam (N11).*

Load P [kN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [x10 ⁶]	ϵ_{t2} [x10 ⁶]	ϵ_{c1} [x10 ⁶]	ϵ_{c2} [x10 ⁶]
0	0	0	0	0	0	0	0
20	0.26	0.29	0.32	48	55	72	64
30	0.38	0.43	0.46	78	84	100	106
40	0.49	0.51	0.62	118	124	122	132
50	0.59	0.69	0.78	298	283	171	205
60	1.10	1.09	1.24	502	542	233	258
65	1.32	1.37	1.62	624	638	288	332
70	1.64	1.65	1.95	775	896	374	412
75	1.98	1.94	2.22	871	921	415	478
80	2.12	2.05	2.51	912	958	447	522
85	2.32	2.28	2.79	1012	1056	478	556
90	2.71	2.67	3.12	1284	1368	506	584
95	3.15	3.05	3.63	1386	1518	564	620
100	3.31	3.29	4.17	1576	1685	602	686
110	5.71	5.7	6.09	2950	3069	995	1122
120	9.2	9.14	10.64	4078	4352	1644	1645
130	*	*	16.64	12036	13369	2037	1883
140	*	*	27.2	17928	19924	3094	2782

Table A-1.2 Experimental data result of beam (N22).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
P [KN]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_t [$\times 10^6$]	ϵ_c [$\times 10^6$]	ϵ_t [$\times 10^6$]	ϵ_c [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.23	0.22	0.25	56	48	64	54
30	0.35	0.39	0.42	92	78	94	86
40	0.56	0.54	0.59	132	117	125	108
50	0.68	0.69	0.72	302	223	162	127
60	1.09	1.06	1.09	472	394	234	196
65	1.39	1.54	1.55	607	463	287	268
70	1.62	1.75	1.84	851	744	334	306
75	1.76	1.88	2.02	916	795	376	342
80	1.98	2.01	2.28	978	847	438	402
85	2.06	2.26	2.46	1048	908	466	429
90	2.37	2.49	2.81	1154	1036	499	457
95	2.59	2.71	3.08	1212	1094	537	485
100	2.77	2.88	3.29	1423	1241	592	548
110	3.12	3.20	3.72	1617	1421	672	613
120	3.54	3.59	4.18	1822	1713	764	688
130	4.10	4.02	4.78	1957	1830	852	771
140	4.58	4.49	5.35	2421	2215	948	845
150	5.78	5.69	7.01	3425	3179	1031	928
160	6.53	6.26	9.19	6381	5642	1169	1123
170	8.23	8.79	12.06	10706	9313	1651	1424
180	11.61	11.88	16.92	12466	13448	2026	1888
175	17.16	16.73	23.03	16167	16486	2897	2746

Table A-1.3 Experimental data result of beam (N33).

Load P [KN.]	Deflection		Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	ϵ_{t1} [$\times 10^{-6}$]	ϵ_{t2} [$\times 10^{-6}$]	ϵ_{c1} [$\times 10^{-6}$]	ϵ_{c2} [$\times 10^{-6}$]
0	0	0	0	0	0	0
20	0.23	0.24	48	39	54	45
30	0.34	0.36	74	66	82	74
40	0.45	0.48	102	98	117	98
50	0.56	0.57	162	176	154	128
60	0.84	0.86	313	212	222	192
65	1.05	1.02	385	263	318	202
70	1.16	1.11	558	454	354	232
75	1.48	1.35	680	614	406	274
80	1.65	1.56	788	748	462	306
85	1.75	1.67	841	824	529	333
90	2.07	1.8	887	878	564	374
95	2.2	1.91	961	969	598	448
100	2.34	2.02	1030	1051	641	484
110	2.62	2.24	1270	1307	703	539
120	2.98	2.68	1409	1454	776	579
130	3.44	2.94	1634	1716	840	684
140	3.80	3.29	1810	1882	914	738
150	4.14	3.55	2117	2203	987	791
160	4.51	3.83	2485	2633	1056	846
170	5.93	5.08	2854	3114	1424	1349
180	8.44	8.06	8144	6695	2214	2133
160	9.79	9.87	13042	12364	2410	2219
180	*	*	21372	20334	2867	2624

Table A-1.4 Experimental data result of beam (N44).

Load P [kN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^{-6}$]	ϵ_{t2} [$\times 10^{-6}$]	ϵ_{c1} [$\times 10^{-6}$]	ϵ_{c2} [$\times 10^{-6}$]
0	0	0	0	0	0	0	0
20	0.2	0.11	0.24	48	44	41	44
40	0.38	0.28	0.46	78	83	85	84
50	0.48	0.37	0.59	119	112	122	112
60	0.77	0.63	0.93	213	167	210	166
65	0.91	0.77	1.13	325	234	229	219
70	1.01	0.87	1.26	403	339	258	253
75	1.1	0.95	1.48	470	448	278	292
80	1.49	1.19	1.79	521	526	301	344
90	1.72	1.38	2.05	629	681	334	428
100	1.99	1.63	2.36	788	835	383	498
120	2.7	2.3	3.19	1045	1167	559	593
140	3.35	3.01	3.98	1223	1289	598	642
160	4.06	3.72	4.83	1435	1405	724	792
170	4.39	4.06	5.21	1552	1642	792	847
180	4.72	4.37	5.58	1778	1880	848	898
190	5.08	4.67	6.23	2193	2236	929	991
200	5.61	5.17	8.15	2801	3072	1005	1084
210	7.16	7.46	10.71	5216	6472	1279	1385
220	8.33	8.64	16.11	9649	10552	1768	1927
230	*	*	*	12498	13484	2278	2258
240	*	*	*	15414	17216	2733	2795

Table A-1.5 Experimental data result of beam (N55).

Load	Deflection		Tension Fiber Strain		Compression Fiber Strain	
P [kN]	δ_1 [mm]	δ_2 [mm]	ϵ_1 [$\times 10^{-6}$]	ϵ_2 [$\times 10^{-6}$]	ϵ_{c1} [$\times 10^{-6}$]	ϵ_{c2} [$\times 10^{-6}$]
0	0	0	0	0	0	0
20	0.22	0.26	34	36	35	42
30	0.33	0.38	58	62	61	66
40	0.44	0.45	74	82	83	81
50	0.55	0.59	92	102	114	104
60	0.69	0.74	126	146	167	124
70	1.08	1.07	278	260	209	182
80	1.31	1.32	461	464	268	237
90	1.58	1.58	595	556	342	286
100	1.94	1.81	727	682	402	340
110	2.16	2.01	825	787	488	391
120	2.4	2.25	1036	996	562	426
130	2.64	2.46	1125	985	602	496
140	2.95	2.75	1214	1137	620	505
150	3.22	3.03	1299	1144	681	547
160	3.52	3.26	1347	1278	745	618
170	3.78	3.47	1492	1388	829	667
180	4.23	3.73	1616	1557	910	726
190	4.48	3.94	1725	1597	952	804
200	4.74	4.16	1858	1751	1002	869
210	5.06	4.41	1957	1885	1153	902
220	5.42	4.72	2140	1975	1297	998
230	6.87	6.08	2478	2392	1345	1142
240	7.82	7.94	5683	5525	1465	1364
250	10.71	10.71	8656	8227	1871	1720
260	16.77	17.59	13849	11056	2112	2094
270	*	*	15767	14977	2561	2527

Table A-1.6 *Experimental data result of beam (N66).*

Load P [kN.]	Deflection			Tension Fiber Strain			Compression Fiber Strain		
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^{-6}$]	ϵ_{t2} [$\times 10^{-6}$]	ϵ_{c1} [$\times 10^{-6}$]	ϵ_{c2} [$\times 10^{-6}$]		
0	0	0	0	0	0	0	0		
20	0.20	0.00	0.24	29	34	29	34		
30	0.30	0.03	0.38	44	48	50	62		
40	0.42	0.14	0.55	62	72	74	86		
50	0.56	0.29	0.71	92	102	98	112		
60	0.70	0.43	0.87	116	136	122	134		
70	0.86	0.61	1.07	192	231	174	192		
80	1.03	0.77	1.26	301	366	210	232		
90	1.23	0.93	1.48	425	449	322	286		
100	1.38	1.10	1.66	498	546	387	321		
110	1.55	1.27	1.86	588	689	461	374		
120	1.72	1.44	2.06	732	774	522	402		
130	1.88	1.61	2.27	796	879	542	432		
140	2.05	1.81	2.48	829	982	576	472		
160	2.59	2.27	3.04	1010	1193	664	572		
180	3.03	2.72	3.57	1201	1386	798	686		
200	3.48	3.15	4.09	1441	1538	919	802		
220	3.90	3.57	4.76	1612	1668	1064	882		
230	4.18	3.92	5.11	1751	1764	1128	957		
240	4.43	4.15	5.39	1893	1810	1216	1174		
250	4.68	4.38	5.68	1940	1874	1292	1243		
260	4.93	4.61	5.94	2091	1945	1385	1305		
270	5.25	4.88	6.31	2138	2023	1445	1346		
280	5.58	5.15	6.65	2277	2091	1482	1384		
290	5.97	5.47	7.05	2304	2277	1502	1426		
300	6.36	5.78	7.42	2450	2387	1622	1458		
310	6.98	6.22	8.00	2593	2490	1777	1567		

Table A-2.1 Experimental data result of beam (B11).

Load P [KN.]	Deflection			Tension Fiber Strain			Compression Fiber Strain		
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.24	0.22	0.27	30	34	36	39		
30	0.33	0.32	0.4	52	56	58	52		
40	0.47	0.45	0.51	63	86	84	64		
50	0.6	0.62	0.67	119	105	114	95		
60	0.93	0.94	1.13	363	323	221	177		
70	1.20	1.17	1.50	562	423	336	297		
80	1.66	1.69	2.17	777	713	465	383		
90	2.21	2.15	2.82	1226	1193	552	483		
100	2.69	2.61	3.44	1406	1318	669	566		
110	3.01	2.98	3.93	3160	2944	863	712		
120	3.45	3.41	4.59	4811	4697	1025	854		
130	8.52	8.63	11.79	8719	6389	1887	1630		
135	11.55	11.09	15.93	14964	13880	2444	2292		
140	15.03	14.01	19.45	21425	19786	3487	3264		
150	18.48	18.51	27.33	25517	24991	4830	4819		

Table A-2.2 *Experimental data result of beam (B21).*

Load P [KN.]	Deflection		Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0
20	0.17	0.19	0.24	35	27	29
30	0.28	0.29	0.34	61	32	38
40	0.39	0.4	0.56	78	47	59
50	0.49	0.49	0.68	112	69	82
60	0.71	0.71	1.20	226	133	125
70	1.22	1.18	1.96	418	214	224
80	1.81	1.8	2.25	1325	358	345
90	2.14	2.14	2.64	1576	469	439
100	2.72	2.74	3.34	1885	617	530
110	3.18	3.16	3.86	2429	875	780
120	3.97	4.03	4.73	3457	1195	1016
130	7.37	7.29	9.11	14068	2308	1940

Table A-2.3 Experimental data result of beam (B31).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.18	0.17	0.22	34	36	28	32
30	0.26	0.24	0.33	58	59	47	48
40	0.43	0.42	0.55	76	78	57	72
50	0.58	0.52	0.64	169	175	98	94
60	0.75	0.74	1.14	324	325	150	122
70	1.27	1.28	1.83	595	570	265	215
80	1.65	1.64	2.18	737	719	344	287
90	2.10	2.18	2.86	1226	1259	387	366
100	2.87	2.95	3.38	1610	1712	903	848
110	3.36	3.47	3.94	1873	1976	1392	1278
120	5.97	6.01	6.9	3169	3246	1892	1766
130	-	-	15.93	12964	13688	2244	2192
140	-	-	20.48	20474	22300	2839	2743

Table A-2.4 Experimental data result of beam (B12).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
P							
0	0	0	0	0	0	0	0
20	0.16	0.16	0.17	31	28	30	34
30	0.24	0.24	0.28	52	49	38	48
40	0.33	0.33	0.37	71	69	54	73
50	0.48	0.5	0.56	163	202	83	96
60	0.74	0.76	0.88	273	324	186	203
70	1.11	1.11	1.29	490	628	289	319
80	1.47	1.36	1.61	781	886	332	438
90	1.83	1.81	2.17	1114	1280	405	512
100	2.14	2.04	2.51	1388	1444	530	612
110	2.46	2.31	2.86	1545	1636	568	643
120	2.81	2.65	3.23	1807	2076	682	764
130	3.25	3.08	3.74	2084	2186	740	786
140	3.71	3.58	4.37	2494	2653	814	861
150	4.41	4.39	5.13	2815	3139	907	945
160	5.36	5.29	6.11	3232	3458	1037	1116
170	6.62	6.81	7.46	4232	4968	1439	1554
177	-	-	17.79	9120	10506	2281	2462
180	-	-	18.69	18520	20469	2418	2689

Table A-2.5 Experimental data result of beam (B22).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.17	0.16	0.18	33	28	29	26
30	0.25	0.21	0.29	58	51	46	44
40	0.31	0.29	0.54	77	64	56	60
50	0.36	0.38	0.66	83	73	78	86
60	0.45	0.44	0.92	286	226	142	104
70	0.70	0.67	1.48	383	474	177	189
80	1.14	1.12	2.14	896	645	273	230
90	1.46	1.32	2.61	1247	972	368	361
100	1.90	1.87	2.83	1326	1170	404	418
110	2.28	2.26	3.54	1466	1374	584	586
120	2.58	2.54	3.84	1593	1405	688	726
130	3.14	3.08	4.62	1771	1688	827	865
140	3.51	3.47	5.03	1948	2169	918	896
150	4.47	4.42	5.14	3228	3410	986	964
160	7.48	7.42	9.85	6816	6247	1354	1253
170	-	9.78	12.9	13644	12579	1758	1591
180	-	-	20.71	20162	18263	2524	2391

Table A-2.6 *Experimental data result of beam (B32).*

Load P [kN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0.00	0	0	0	0
20	0.18	0.15	0.22	29	34	30	23
30	0.26	0.26	0.34	51	56	47	35
40	0.35	0.37	0.58	75	72	62	59
50	0.57	0.54	0.71	85	140	76	90
60	0.78	0.75	0.96	246	316	110	141
70	1.08	1.12	1.62	477	540	204	181
80	1.23	1.21	2.12	576	771	343	228
90	1.76	1.77	2.52	998	871	422	364
100	2.22	2.18	2.80	1246	1357	482	432
110	2.61	2.51	3.38	1394	1531	602	568
120	2.88	2.81	3.68	1459	1689	719	678
130	3.25	3.19	4.16	1751	1865	804	770
140	3.64	3.56	5.12	2114	2334	912	803
150	8.85	8.91	11.10	2935	3263	1014	945
160	-	-	16.00	7461	8122	1328	1298
170	-	-	18.95	13497	14235	1651	1529

Table A-2.7 Experimental data result of beam (B13).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.15	0.15	0.16	26	31	24	27
30	0.25	0.25	0.27	43	48	28	34
40	0.34	0.35	0.38	58	64	41	54
50	0.48	0.47	0.55	113	190	62	86
60	0.66	0.65	0.77	244	262	166	163
70	1.02	1.02	1.20	467	578	187	259
80	1.35	1.28	1.56	628	672	205	308
90	1.74	1.68	2.03	742	808	250	360
100	2.07	1.99	2.39	932	1015	368	413
110	2.42	2.39	2.89	1059	1223	415	487
120	2.78	2.63	3.27	1268	1320	512	535
130	3.22	3.16	3.84	1402	1507	584	568
140	3.57	3.49	4.19	1617	1697	628	617
150	4.15	3.98	4.95	2034	2127	722	784
160	5.27	5.16	6.51	3680	3933	881	1005
170	14.14	12.57	16.25	7602	8264	1662	1913
180	-	-	-	10336	11875	1925	2291
190	-	-	-	13286	13678	2296	2486

Table A-2.8 Experimental data result of beam (B23).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
P							
[KN.]							
0	0	0	0	0	0	0	0
20	0.16	0.16	0.19	23	16	29	23
30	0.25	0.26	0.30	43	40	44	50
40	0.33	0.32	0.49	63	58	63	73
50	0.42	0.41	0.58	70	77	74	88
60	0.55	0.56	0.78	98	90	115	108
70	0.88	0.79	1.15	138	126	169	152
80	1.20	1.08	1.62	172	357	222	189
90	1.65	1.44	2.32	241	569	264	250
100	2.10	1.78	2.45	454	680	386	331
110	2.48	2.13	2.84	580	796	456	425
120	2.86	2.53	3.27	799	946	527	468
130	3.35	3.04	3.86	985	1226	602	542
140	3.73	3.42	4.27	1080	1451	696	604
150	4.10	3.81	4.91	1204	1636	764	695
160	4.48	4.16	5.16	1497	1823	886	795
170	5.37	4.79	6.30	2435	1954	1052	999
180	6.57	5.67	7.63	2828	3247	1337	1292
190	-	-	-	15015	13226	2630	2568

Table A-2.9 Experimental data result of beam (B33).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.15	0.18	0.2	33	36	28	24
30	0.24	0.25	0.32	51	53	41	46
40	0.32	0.34	0.43	68	72	52	57
50	0.43	0.44	0.66	112	108	74	85
60	0.58	0.59	0.81	217	124	112	122
70	0.98	0.95	1.19	484	204	212	153
80	1.39	1.25	1.64	652	510	272	238
90	1.78	1.69	2.28	845	708	336	279
100	2.30	2.15	2.74	1304	1224	403	377
110	2.92	2.74	3.43	1649	1430	480	448
120	3.35	3.08	3.87	1815	1598	515	486
130	3.75	3.48	4.45	2016	1801	569	543
140	4.14	3.83	4.88	2218	2014	618	582
150	4.53	4.19	5.34	2411	2220	667	622
160	4.95	4.56	5.79	2616	2493	728	737
170	5.55	5.11	6.51	3275	3085	1047	1130
180	7.65	7.84	8.65	9319	7360	2282	2361
190	-	-	22.56	17996	15989	2588	2691

Table A-2.10 Experimental data result of beam (B14).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.16	0.18	0.19	28	24	31	31
30	0.24	0.23	0.28	47	44	47	47
40	0.33	0.33	0.38	61	58	59	59
50	0.42	0.4	0.54	86	91	85	86
60	0.5	0.5	0.69	121	161	100	145
70	0.71	0.71	0.97	329	334	126	197
80	1.19	1.24	1.59	670	542	268	292
90	1.61	1.67	2.01	848	625	307	404
100	1.85	1.87	2.17	1020	699	436	476
110	1.86	1.97	2.65	1096	983	470	522
120	2.28	2.25	2.84	1248	1096	500	568
130	2.46	2.48	3.36	1397	1223	536	608
140	2.76	2.85	3.96	1562	1350	664	719
150	3.32	3.21	4.14	1700	1476	799	842
160	3.51	3.49	4.38	1856	1619	829	902
170	3.85	3.77	4.62	2033	1783	872	984
180	4.08	4.12	4.97	2251	2092	957	1089
190	4.38	4.29	5.27	2971	2809	1143	1124
200	4.48	4.36	6.28	4274	4026	1352	1462
210	5.02	5.06	6.67	6695	6450	1412	1594
220	5.85	5.73	7.20	10488	11242	1586	1668
230	-	-	10.92	14039	12440	1876	1905
240	-	-	15.08	18272	17304	2428	2501

Table A-2.11 *Experimental data result of beam (B24).*

Load P [KN.]	Deflection			Tension Fiber Strain			Compression Fiber Strain		
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.17	0.19	0.21	26	24	17	22		
30	0.27	0.26	0.28	38	43	21	34		
40	0.38	0.4	0.42	51	56	26	59		
50	0.52	0.53	0.57	72	88	39	77		
60	0.64	0.65	0.76	124	142	56	101		
70	0.82	0.85	1.02	274	284	83	149		
80	1.13	1.13	1.42	394	469	129	197		
90	1.35	1.32	1.97	543	620	148	239		
100	1.71	1.67	2.13	654	752	282	277		
110	2.01	1.98	2.58	867	908	329	307		
120	2.23	2.19	2.89	941	1072	385	342		
130	2.56	2.51	3.21	1022	1227	453	412		
140	2.88	2.86	3.82	1192	1375	484	445		
150	3.30	3.29	4.01	1366	1539	508	484		
160	3.58	3.55	4.28	1530	1695	568	519		
170	3.87	3.86	4.61	1699	1852	712	688		
180	4.26	4.19	4.82	1885	2020	877	740		
190	4.45	4.45	5.13	2064	2164	999	971		
200	4.81	4.73	6.38	3051	2832	1181	1052		
210	5.07	5.01	7.58	6027	5811	1312	1226		
220	9.08	9.03	11.07	8339	7117	1472	1362		
230	10.54	10.53	18.51	9158	8473	1840	1458		

Table A-2.12 Experimental data result of beam (B34).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.2	0.18	0.22	22	27	26	19
30	0.28	0.26	0.3	38	40	38	29
40	0.35	0.36	0.41	55	58	60	42
50	0.43	0.42	0.56	83	76	88	56
60	0.52	0.48	0.73	173	163	131	73
70	0.81	0.77	0.95	336	352	212	181
80	1.04	0.99	1.53	442	444	245	194
90	1.16	1.10	1.90	564	472	296	278
100	1.45	1.37	2.16	702	670	348	315
110	1.65	1.64	2.73	814	758	401	396
120	2.01	1.96	2.92	911	825	439	440
130	2.22	2.23	3.18	1019	910	486	480
140	2.76	2.67	3.91	1130	1099	514	514
150	2.98	2.91	4.13	1241	1190	560	524
160	3.25	3.26	4.36	1350	1280	599	568
170	3.57	3.48	4.79	1448	1352	630	586
180	3.83	3.76	4.84	1962	1848	674	667
190	4.11	4.06	5.37	2381	2239	916	846
200	4.59	4.62	6.49	3229	3151	1044	978
210	6.46	6.56	6.84	8413	7895	1225	1140
220	8.77	8.68	9.79	14532	13815	1546	1494
230	-	-	11.49	18384	16935	2816	2684

Table A-2.13 Experimental data result of beam (B15).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.19	0.18	0.21	18	23	24	22
30	0.3	0.31	0.37	38	35	32	33
40	0.42	0.41	0.48	47	55	61	66
50	0.52	0.51	0.6	71	76	72	74
60	0.65	0.63	0.74	87	97	110	102
70	0.88	0.82	0.98	163	202	156	182
80	1.07	1.05	1.33	314	350	220	256
90	1.20	1.24	1.55	451	490	259	302
100	1.55	1.56	1.88	515	674	390	342
110	1.92	1.83	2.21	706	784	424	381
120	2.17	2.12	2.56	897	984	478	426
130	2.37	2.38	2.86	928	1168	591	463
140	2.78	2.71	3.23	1067	1235	629	506
150	3.25	3.19	3.50	1132	1318	662	540
160	3.57	3.52	3.88	1286	1396	703	678
170	3.86	3.75	4.17	1394	1593	844	715
180	4.13	4.06	4.50	1502	1718	876	759
190	4.50	4.42	4.83	1619	1936	911	869
200	4.80	4.72	5.16	1729	2160	945	918
210	5.07	5.12	5.53	1832	2274	971	957
220	5.24	5.26	5.93	2003	2363	1025	984
230	6.46	6.56	7.06	2600	2897	1124	1018
240	-	6.88	8.75	4670	4875	1226	1155
250	-	10.23	13.08	7208	7869	1295	1512
250	-	-	15.09	8574	9937	1713	1975
260	-	-	20.32	15371	16561	2249	2615

Table A-2.14 Experimental data result of beam (B25).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.15	0.16	0.18	20	19	15	17		
30	0.24	0.23	0.25	36	36	29	34		
40	0.34	0.33	0.38	45	40	42	45		
50	0.45	0.44	0.52	71	70	63	59		
60	0.54	0.56	0.76	65	152	93	95		
70	0.72	0.74	0.86	190	272	150	156		
80	0.98	0.92	1.34	270	333	174	183		
90	1.19	1.22	1.58	474	411	220	231		
100	1.46	1.44	2.02	556	616	359	267		
110	1.83	1.78	2.23	801	762	401	398		
120	2.21	2.12	2.52	894	968	429	425		
130	2.36	2.26	2.85	948	1072	498	467		
140	2.66	2.64	3.21	1021	1186	588	473		
150	2.95	2.89	3.63	1124	1321	608	503		
160	3.25	3.27	3.95	1248	1436	645	540		
170	3.64	3.58	4.26	1424	1554	677	571		
180	3.78	3.67	4.58	1523	1661	708	689		
190	4.09	3.92	4.88	1624	1770	737	728		
200	4.28	4.27	5.21	1729	1882	862	748		
210	4.64	4.58	5.60	1845	2007	895	878		
220	4.92	4.87	5.93	2117	2233	927	999		
230	5.63	5.6	6.30	3034	2389	1080	1114		
240	8.37	8.31	10.21	5266	5864	1215	1320		
250	-	-	14.87	8428	8803	2179	2375		
260	-	-	18.35	14128	16103	2451	2657		

Table A-2.15 Experimental data result of beam (B35).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.18	0.16	0.18	24	25	28	18		
30	0.3	0.28	0.25	38	37	37	24		
40	0.41	0.40	0.38	51	40	54	42		
50	0.52	0.48	0.52	77	64	83	57		
60	0.62	0.61	0.76	128	92	113	84		
70	0.75	0.74	0.86	242	168	165	116		
80	1.11	1.12	1.34	458	329	212	169		
90	1.30	1.28	1.58	561	543	245	203		
100	1.60	1.62	2.02	662	622	342	258		
110	1.84	1.86	2.23	875	765	377	361		
120	2.14	2.10	2.52	1062	958	410	401		
130	2.49	2.40	2.85	1178	996	442	432		
140	2.84	2.80	3.21	1264	1104	484	455		
150	3.13	3.08	3.63	1366	1203	599	496		
160	3.42	3.34	3.95	1409	1304	641	529		
170	3.70	3.64	4.26	1615	1413	765	650		
180	3.97	3.88	4.58	1794	1480	807	690		
190	4.13	4.10	4.88	1906	1578	848	751		
200	4.50	4.40	5.21	2024	1667	942	850		
210	4.80	4.70	5.60	2144	1759	1004	977		
220	5.02	4.95	5.93	2284	1885	1145	1030		
230	5.34	5.26	6.30	2374	2130	1204	1184		
240	6.08	5.98	11.21	3540	3193	1285	1211		
250	8.86	8.67	18.35	5137	5939	1487	1310		
260	-	-	13.91	8941	8591	1657	1474		
270	-	-	15.1	9551	9765	1756	1677		

Table A-2.16 Experimental data result of beam (B16).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		0
20	0.16	0.18	0.20	16	19	18	22		22
30	0.28	0.27	0.34	22	24	26	31		31
40	0.38	0.40	0.48	32	33	39	54		54
50	0.6	0.57	0.68	45	58	66	77		77
60	0.77	0.78	0.85	75	79	96	102		102
70	0.92	0.95	1.05	118	182	102	168		168
80	1.10	1.09	1.27	255	208	183	232		232
90	1.26	1.25	1.47	449	500	244	264		264
100	1.43	1.51	1.69	535	572	297	291		291
110	1.60	1.69	1.89	617	677	358	317		317
120	1.74	1.81	2.11	701	771	407	357		357
130	1.93	2.02	2.34	785	872	482	393		393
140	2.14	2.13	2.58	873	965	522	462		462
150	2.34	2.35	2.79	962	1146	568	508		508
160	2.56	2.61	3.03	1059	1266	607	583		583
180	3.07	3.12	3.57	1244	1485	782	717		717
200	3.53	3.60	4.17	1398	1667	866	842		842
220	4.01	4.06	4.69	1579	1869	999	928		928
230	4.23	4.27	4.93	1677	1985	1075	989		989
240	4.47	4.50	5.24	1785	2115	1125	1049		1049
250	4.74	4.76	5.58	1899	2232	1236	1188		1188
260	5.02	5.12	5.60	1980	2383	1328	1222		1222
280	5.57	5.67	6.51	2147	2675	1368	1295		1295
300	6.01	6.13	7.12	2347	2911	1495	1399		1399
320	-	-	7.81	2615	3165	1544	1497		1497
340	-	-	9.01	3298	3504	1931	1856		1856

Table A-2.17 Experimental data result of beam (B26).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		0
20	0.21	0.26	0.23	16	14	19	17		17
30	0.33	0.39	0.37	22	23	26	28		28
40	0.45	0.52	0.5	31	35	32	32		32
50	0.56	0.64	0.63	48	61	49	58		58
60	0.72	0.79	0.81	69	82	82	96		96
70	0.86	0.94	1.00	106	112	129	109		109
80	1.03	1.11	1.20	122	126	164	211		211
90	1.17	1.26	1.39	305	432	213	235		235
100	1.35	1.46	1.63	471	651	273	258		258
110	1.57	1.64	1.84	610	698	294	314		314
120	1.71	1.81	2.05	696	770	339	369		369
130	1.91	1.99	2.31	831	870	467	400		400
140	2.10	2.20	2.54	926	953	488	438		438
150	2.30	2.37	2.77	1026	1031	548	492		492
160	2.47	2.45	3.03	1167	1119	626	561		561
180	2.96	2.97	3.63	1374	1313	721	685		685
190	3.13	3.17	3.86	1462	1393	784	738		738
200	3.36	3.43	4.15	1555	1480	814	784		784
220	3.78	3.91	4.70	1745	1667	924	865		865
240	4.22	4.38	5.32	1942	1955	1027	951		951
260	4.88	4.92	6.03	2173	2053	1148	1068		1068
280	5.34	5.34	6.56	2420	2205	1264	1230		1230
300	5.91	5.88	7.27	2650	2382	1322	1285		1285
320	6.49	6.46	7.95	2819	2593	1467	1372		1372
340	7.39	7.21	8.84	3074	2958	1528	1515		1515
350	-	-	9.88	3287	3592	1741	1612		1612

Table A-2.18 Experimental data result of beam (B36).

Load	Deflection		Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	ε_H [$\times 10^6$]	ε_2 [$\times 10^6$]	ε_1 [$\times 10^6$]	ε_2 [$\times 10^6$]
0	0	0	0	0	0	0
20	0.29	0.26	19	25	17	26
30	0.41	0.35	29	33	26	36
40	0.52	0.43	42	38	42	42
50	0.65	0.54	64	46	54	65
60	0.77	0.65	117	52	67	93
70	0.94	0.78	129	76	92	129
80	1.14	0.92	338	124	182	206
90	1.31	1.06	547	212	231	242
100	1.50	1.21	659	324	389	290
110	1.69	1.35	750	377	441	328
120	1.89	1.5	843	430	478	367
130	2.09	1.67	923	495	507	514
140	2.31	1.83	1020	549	587	563
150	2.56	2.03	1133	612	612	600
160	2.80	2.21	1226	684	678	623
180	3.32	2.60	1393	799	786	674
200	3.91	3.10	1578	914	897	849
220	4.46	3.54	1705	1070	1085	979
240	4.99	3.98	1847	1242	1250	1127
250	5.29	4.23	1921	1316	1397	1187
260	5.63	4.51	2004	1377	1454	1274
280	6.10	4.99	2162	1571	1567	1314
300	6.71	5.46	2328	1760	1612	1462
320	7.45	6.06	2539	1976	1736	1633
340	8.09	6.55	2772	2108	1975	1806

Table A-3.1 Experimental data result of beam (T11).

Load P [KN.]	Deflection		Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]
0	0	0	0	0	0	0
20	0.26	0.24	0.2	40	36	34
30	0.27	0.29	0.32	62	59	59
40	0.31	0.35	0.43	69	74	62
50	0.45	0.47	0.57	114	147	95
60	0.85	0.79	1.01	216	298	132
65	1.12	1.11	1.38	418	402	184
70	1.60	1.56	1.89	868	624	263
80	1.98	1.92	2.35	1124	828	365
90	2.56	2.47	2.97	1308	1095	432
100	2.87	2.86	3.47	1827	1248	496
110	3.46	3.35	4.02	2156	1413	572
120	4.14	4.12	5.03	3339	1628	690
130	5.89	5.87	7.23	5515	3363	928
140	10.97	11.00	14.3	9743	8964	1574
150	-	-	25.6	26784	22151	2514
160	-	-	32.6	30307	28471	3346

Table A-3.2 Experimental data result of beam (T21).

Load P [KN.]	Deflection		Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	ε_{t1} [$\times 10^6$]	ε_{t2} [$\times 10^6$]	ε_{c1} [$\times 10^6$]	ε_{c2} [$\times 10^6$]
0	0	0	0	0	0	0
20	0.27	0.31	43	54	32	28
30	0.48	0.53	59	68	70	39
40	0.69	0.75	201	233	108	65
50	0.9	0.96	393	323	148	82
60	1.14	1.18	495	434	195	134
65	1.23	1.22	675	466	213	172
70	1.33	1.36	766	516	234	268
80	1.65	1.63	862	612	278	314
90	1.88	1.91	1004	737	406	421
100	2.17	2.26	1279	963	477	498
110	2.86	2.92	1392	1233	539	665
120	3.28	3.3	1550	1450	594	804
130	7.04	7.09	5700	5236	1099	891
140	-	-	11728	10234	1502	1388
150	-	-	-	16270	-	1859

Table A-3.3 Experimental data result of beam (T31).

Load	Deflection		Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_t [$\times 10^6$]	ϵ_c [$\times 10^6$]	ϵ_c [$\times 10^6$]
0	0	0	0	0	0	0
20	0.21	0.25	0.28	33	35	32
30	0.32	0.34	0.40	52	58	58
40	0.43	0.45	0.50	79	82	67
50	0.53	0.57	0.61	121	164	94
60	0.82	0.84	0.94	268	385	171
65	1.07	1.12	1.31	435	412	199
70	1.45	1.47	1.74	822	776	243
80	1.86	1.99	2.15	1052	1057	322
90	2.31	2.41	2.66	1363	1393	421
100	2.76	2.80	3.13	1617	1665	540
110	3.23	3.30	3.69	1977	2188	689
120	3.61	3.65	4.14	2209	2434	786
130	6.59	6.84	8.69	5781	6354	1262
140	10.72	11.00	15.10	10660	10920	1888
150	-	-	21.90	25684	27068	2498
160	-	-	30.00	27049	29795	2536

Table A-3.4 Experimental data result of beam (T12).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{d1} [$\times 10^6$]	ϵ_{d2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.18	0.19	0.22	34	28	28	30
30	0.3	0.32	0.37	62	58	45	50
40	0.45	0.43	0.52	78	64	67	71
50	0.54	0.54	0.66	152	131	89	95
60	0.71	0.71	0.85	252	215	140	121
70	1.13	1.11	1.45	467	465	182	246
80	1.63	1.57	2.06	722	595	280	299
90	2.08	1.91	2.62	901	695	335	356
100	2.24	2.24	3.07	1124	810	379	405
110	3.10	3.02	3.64	1246	969	421	462
120	3.38	3.29	4.13	1388	1095	480	496
130	3.80	3.72	4.52	1485	1164	526	588
140	4.04	3.98	4.95	1616	1384	575	648
150	4.47	4.43	5.36	1825	1606	622	664
160	4.96	4.85	5.88	2020	1884	691	721
170	5.26	5.12	6.37	2134	2084	754	760
180	6.36	6.32	7.62	2491	2286	910	893
190	7.73	7.81	8.63	4450	2477	1424	1317
200	10.20	10.16	10.7	8124	6428	2083	1899
210	-	-	-	24327	20315	2623	2480

Table A-3.5 Experimental data result of beam (T22).

Load	Deflection		Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0
20	0.17	0.19	29	36	32	27
30	0.27	0.24	48	44	45	42
40	0.46	0.45	86	77	63	64
50	0.56	0.54	144	113	85	88
60	0.64	0.62	324	248	98	101
70	0.86	0.9	417	364	132	137
80	1.38	1.36	469	575	189	337
90	1.83	1.78	661	735	282	411
100	2.29	2.24	829	903	326	451
110	2.68	2.66	958	1067	375	503
120	3.02	2.95	1081	1257	421	549
130	3.32	3.29	1202	1459	469	595
140	3.60	3.57	1327	1643	513	632
150	4.05	3.96	1451	1823	569	690
160	4.49	4.44	1558	2095	616	734
170	4.82	4.76	1706	2137	670	788
180	5.76	5.68	1907	2472	738	874
190	6.24	5.18	2262	3289	1138	1288
200	12.8	12.78	6700	11103	1613	1771
210	16.2	16.13	18696	22231	2511	2600

Table A-3.6 Experimental data result of beam (T32).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		0
20	0.17	0.18	0.22	34	31	29	34		34
30	0.25	0.29	0.33	56	51	45	51		51
40	0.41	0.43	0.53	78	68	62	68		68
50	0.52	0.52	0.64	92	88	82	86		86
60	0.64	0.61	0.75	242	303	98	106		106
70	0.97	0.98	1.46	415	523	206	250		250
80	1.39	1.41	1.85	497	573	261	321		321
90	1.68	1.65	2.32	823	717	399	389		389
100	1.87	1.84	2.35	992	935	453	449		449
110	2.57	2.63	3.24	1178	1102	509	507		507
120	3.00	2.92	3.47	1250	1270	552	565		565
130	3.39	3.28	3.85	1517	1436	598	618		618
140	3.76	3.72	5.24	1677	1583	644	663		663
150	4.17	4.07	5.72	1838	1724	693	726		726
160	4.44	4.32	5.11	2033	1900	747	787		787
170	4.87	4.85	5.6	2284	2138	822	867		867
180	5.43	5.44	6.36	2461	2697	915	993		993
190	6.85	6.89	7.57	3551	3745	1357	1287		1287
200	10.00	10.12	11.8	5983	6970	1615	1525		1525
210	12.7	12.54	15.2	8421	9484	1806	1716		1716
220	13.6	13.61	16.4	9086	10497	1937	1918		1918
200	-	-	-	14181	18347	2539	2652		2652

Table A-3.7 Experimental data result of beam (T13).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0	0	0
20	0.17	0.18	0.21	30	27	26	27		27
30	0.26	0.27	0.3	52	41	44	40		40
40	0.36	0.37	0.41	72	62	64	56		56
50	0.48	0.47	0.54	112	89	84	76		76
60	0.59	0.58	0.67	208	168	116	95		95
65	0.76	0.79	0.92	324	354	158	102		102
70	0.92	0.89	1.05	382	426	185	129		129
80	1.21	1.26	1.38	471	560	219	234		234
90	1.73	1.68	1.92	674	777	325	284		284
100	2.04	2.06	2.34	791	901	370	400		400
110	2.36	2.41	2.77	845	1091	406	439		439
120	2.77	2.74	3.17	965	1144	449	485		485
130	3.07	3.16	3.46	1026	1264	493	525		525
140	3.4	3.52	3.84	1185	1402	538	584		584
150	3.75	3.82	4.24	1252	1556	579	633		633
160	4.13	4.19	4.65	1412	1653	624	688		688
170	4.53	4.56	5.12	1678	1784	667	738		738
180	5.02	5.12	5.63	1754	1839	709	811		811
190	7.46	7.46	8.75	2409	2536	1017	1130		1130
200	8.68	8.68	10.20	3463	3602	1341	1378		1378
210	9.76	9.71	11.62	4246	4646	1465	1614		1614
210	-	-	15.57	6565	6867	1743	2076		2076
220	-	-	16.9	8570	9438	1865	2192		2192
225	-	-	25.3	16275	16551	2496	2967		2967
230	-	-	27.89	20616	20258	2691	3170		3170

Table A-3.8 Experimental data result of beam (T23).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.17	0.17	0.19	24	26	24	27
30	0.26	0.26	0.29	43	42	40	38
40	0.38	0.38	0.42	62	61	62	64
50	0.42	0.4	0.53	87	86	88	82
60	0.51	0.48	0.62	116	97	96	94
70	0.61	0.58	1.112	161	125	117	114
80	1.17	1.19	1.42	596	353	213	196
90	1.61	1.59	2.01	714	432	326	270
100	1.91	1.88	2.34	819	515	364	299
110	2.43	2.36	2.89	1061	607	408	333
120	2.68	2.69	3.33	1116	693	463	375
130	3.06	3.05	3.77	1333	814	511	417
140	3.42	3.38	4.16	1415	979	551	456
150	3.8	3.77	4.55	1621	1347	584	508
160	4.23	4.18	5.01	1888	1622	625	556
170	4.66	4.67	5.48	2072	1889	674	611
180	6.42	6.39	8.50	6451	2128	1087	778
190	-	-	19.08	11135	10722	1714	1684
200	-	-	24.84	16476	15109	2005	2122
210	-	-	34.06	19705	20960	2408	2569

Table A-3.9 *Experimental data result of beam (T33).*

Load	Deflection				Tension Fiber Strain		Compression Fiber Strain	
P [kN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	
0	0	0	0	0	0	0	0	
20	0.15	0.16	0.22	25	31	25	26	
30	0.23	0.27	0.32	43	46	36	42	
40	0.37	0.35	0.42	62	66	61	59	
50	0.41	0.43	0.52	86	91	75	68	
60	0.52	0.54	0.68	123	138	98	101	
70	0.83	0.88	1.08	459	489	129	183	
80	1.33	1.34	1.70	618	586	187	243	
90	1.58	1.61	1.97	782	708	227	285	
100	1.99	1.96	2.35	867	848	261	323	
110	2.30	2.26	2.70	1179	1016	299	370	
120	2.61	2.57	3.06	1298	1167	337	413	
130	3.14	3.19	3.69	1468	1331	375	454	
140	3.47	3.45	4.06	1606	1464	420	497	
150	3.94	3.96	4.63	1839	1693	514	575	
160	7.95	7.91	9.78	7606	6175	1135	1083	
170	-	-	16.51	10044	9853	1688	1612	
180	-	-	23.61	16059	14457	2255	2123	
190	-	-	31.34	19336	17829	2696	2512	
210	-	-	36.84	25519	22904	2752	6425	

Table A-3.10 Experimental data result of beam (T14).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.13	0.13	0.16	25	24	25	28
30	0.21	0.2	0.27	44	38	44	47
40	0.3	0.28	0.38	61	59	63	66
50	0.39	0.37	0.47	81	86	86	79
60	0.49	0.47	0.59	108	95	124	97
70	0.75	0.65	0.85	176	141	158	152
80	1.23	1.14	1.44	306	199	265	220
90	1.36	1.34	1.69	382	250	310	260
100	1.75	1.68	2.08	458	310	365	294
110	2.13	2.09	2.49	802	724	401	350
120	2.43	2.37	2.82	955	834	439	382
130	2.87	2.83	3.33	1021	960	478	429
140	3.18	3.12	3.68	1252	1063	515	453
150	3.52	3.44	4.05	1395	1173	558	489
160	3.62	3.56	4.46	1452	1290	598	519
170	4.32	4.28	4.83	1611	1411	641	564
180	4.71	4.64	5.26	1884	1631	685	616
190	5.17	5.12	5.76	2078	1886	729	658
200	5.74	5.68	6.38	2706	2513	778	713
210	7.52	7.48	8.56	3324	3227	1062	902
220	-	-	16.22	6425	7783	1760	1445
230	-	-	24.62	10908	11991	2576	1842

Table A-3.11 Experimental data result of beam (T24).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.15	0.17	0.2	34	26	25	24
30	0.25	0.23	0.28	48	44	32	32
40	0.35	0.34	0.36	63	69	54	51
50	0.45	0.44	0.52	90	94	64	60
60	0.54	0.54	0.68	136	197	75	72
70	0.72	0.74	0.92	298	397	83	88
80	0.98	1.02	1.29	516	445	94	124
90	1.31	1.31	1.62	648	674	109	166
100	1.56	1.52	1.90	773	823	190	213
110	1.92	1.89	2.33	891	968	220	243
120	2.14	2.13	2.60	1026	1135	250	294
130	2.42	2.38	2.88	1146	1294	268	334
140	2.68	2.63	3.21	1268	1361	290	373
150	3.04	3.00	3.63	1381	1485	305	411
160	3.34	3.28	3.95	1494	1565	357	450
170	3.61	3.57	4.27	1609	1639	391	492
180	4.01	3.99	4.60	1726	1792	429	542
190	4.23	4.21	4.94	1845	1842	463	582
200	4.49	4.42	5.27	1970	1924	495	614
210	4.92	4.89	5.68	2101	2054	532	669
220	5.29	5.23	6.11	2353	2533	563	709
230	6.27	6.23	7.23	2805	3652	762	964
240	7.28	7.26	8.64	4192	4815	962	1147
250	8.75	8.78	11.70	8594	7555	1264	1457
260	-	-	15.31	16033	14151	1592	1804

Table A-3.12 Experimental data result of beam (T34).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
P							
0	0	0	0	0	0	0	0
20	0.16	0.15	0.21	22	22	21	25
30	0.27	0.26	0.29	36	40	39	43
40	0.36	0.37	0.41	48	50	58	62
50	0.44	0.47	0.54	66	69	75	89
60	0.53	0.57	0.65	91	91	117	132
70	0.80	0.83	1.00	386	169	174	177
80	1.22	1.24	1.46	487	351	229	257
90	1.46	1.46	1.75	580	447	276	307
100	1.69	1.68	2.03	688	586	320	345
110	2.01	2.02	2.40	824	832	367	384
120	2.32	2.32	2.75	951	989	407	425
130	2.59	2.63	3.07	1070	1055	446	469
140	2.91	2.91	3.42	1277	1239	493	511
150	3.14	3.16	3.69	1363	1471	533	550
160	3.42	3.39	3.99	1425	1515	573	581
170	3.69	3.70	4.31	1557	1630	613	625
180	3.97	3.95	4.62	1666	1766	659	665
190	4.34	4.29	5.01	1723	1887	723	708
200	4.60	4.56	5.32	1813	1928	778	746
210	4.88	4.86	5.68	1979	2092	852	798
220	-	-	11.5	3217	3353	1598	1459
230	-	-	15.18	6592	6142	2049	1728
240	-	-	20.65	12235	13218	2537	2297
250	-	-	25.37	15387	18410	3046	2679

Table A-3.13 Experimental data result of beam (T15).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.15	0.16	0.19	21	24	27	23
30	0.23	0.24	0.3	24	36	41	39
40	0.32	0.31	0.41	44	42	56	42
50	0.4	0.42	0.51	71	64	71	62
60	0.49	0.49	0.62	107	102	92	81
70	0.67	0.66	0.86	135	168	132	96
80	1.00	1.03	1.26	256	239	168	134
90	1.23	1.22	1.56	409	448	215	186
100	1.46	1.45	1.85	556	565	318	288
120	2.01	1.98	2.47	633	701	432	378
140	2.38	2.36	3.02	804	843	512	427
160	2.97	3.07	3.58	978	1037	575	488
160	3.15	3.14	3.67	1088	1143	600	576
180	3.68	3.73	4.29	1139	1213	675	626
200	4.36	4.34	4.85	1219	1406	742	708
210	4.48	4.57	5.15	1421	1511	812	804
220	4.79	4.81	5.47	1558	1631	883	964
230	5.44	5.49	6.40	1997	1869	1057	1131
240	-	-	16.98	8199	7211	2281	2313
250	-	-	22.97	11964	10924	2647	2889

Table A-3.14 Experimental data result of beam (T25).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
P							
0	0	0	0	0	0	0	0
20	0.15	0.16	0.18	31	18	21	24
30	0.2	0.21	0.24	46	47	38	36
40	0.28	0.27	0.32	54	64	56	67
50	0.36	0.36	0.45	68	87	60	74
60	0.45	0.44	0.55	120	108	87	96
70	0.68	0.69	0.75	224	142	103	142
80	0.92	0.87	1.04	489	315	139	174
90	1.15	1.15	1.39	577	415	183	224
100	1.36	1.33	1.63	669	516	228	233
120	1.95	1.92	2.20	844	750	287	314
140	2.44	2.41	2.79	1048	967	357	405
160	3.22	3.18	3.58	1225	1067	374	492
180	3.74	3.76	4.14	1305	1263	461	562
200	4.36	4.33	4.79	1488	1441	548	663
210	4.6	4.55	5.08	1580	1634	692	692
220	4.92	4.93	5.46	1686	1718	735	750
230	5.23	5.22	5.84	1930	2003	785	791
240	6.05	5.98	6.99	2440	2643	1089	982
250	-	-	10.76	5256	4771	1451	1340
260	-	-	18.48	9962	8869	1931	1842
270	-	-	24.16	12916	13581	2574	2461

Table A-3.15 Experimental data result of beam (T35).

Load P [KN.]	Deflection			Tension Fiber Strain			Compression Fiber Strain		
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.15	0.14	0.16	24	25	21	23		
30	0.2	0.21	0.25	42	31	37	41		
40	0.28	0.31	0.36	55	44	58	62		
50	0.38	0.37	0.48	72	79	77	86		
60	0.48	0.52	0.6	94	102	103	128		
70	0.72	0.73	0.84	119	106	126	152		
80	1.03	0.98	1.19	238	174	185	227		
90	1.23	1.21	1.47	352	263	226	233		
100	1.49	1.48	1.76	460	353	279	273		
120	2.01	1.95	2.37	619	703	351	354		
140	2.42	2.43	3.03	820	964	421	417		
160	3.28	3.24	3.67	1013	1113	471	487		
180	3.89	3.85	4.30	1229	1453	552	557		
200	4.31	4.29	4.97	1675	1612	641	629		
210	4.68	4.62	5.37	1845	1813	686	657		
220	5.07	5.06	5.78	2076	2002	715	681		
230	5.62	5.58	6.15	2242	2164	754	717		
240	6.43	6.42	7.42	2874	3468	879	887		
240	-	-	-	3659	3854	1091	1097		
250	-	-	-	6313	7284	1605	1862		
260	-	-	-	13144	16111	2446	2679		

Table A-3.16 Experimental data result of beam (T16).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		0
20	0.12	0.10	0.17	21	25	24	27		27
30	0.20	0.19	0.30	34	35	31	34		34
40	0.32	0.31	0.43	39	45	39	47		47
50	0.45	0.44	0.58	58	62	64	71		71
60	0.61	0.59	0.76	88	97	88	93		93
70	0.74	0.73	0.95	137	119	160	136		136
80	0.92	0.89	1.16	202	193	282	201		201
100	1.29	1.26	1.58	387	392	363	323		323
120	1.65	1.58	2.02	653	456	411	400		400
140	2.03	1.98	2.54	747	693	511	486		486
160	2.41	2.38	2.99	864	1016	624	580		580
180	2.79	2.78	3.48	1160	1167	784	647		647
200	3.19	3.16	4.00	1381	1222	829	723		723
220	3.72	3.67	4.53	1552	1372	924	804		804
230	3.95	3.93	4.81	1745	1546	1075	948		948
240	4.19	4.17	5.08	1834	1745	1127	1031		1031
260	4.81	4.78	5.75	2119	1974	1207	1159		1159
280	5.40	5.36	6.33	2227	2140	1307	1234		1234
300	5.95	5.93	6.97	2433	2358	1415	1306		1306
320	6.47	6.42	7.59	2656	2557	1532	1477		1477
340	7.13	7.11	8.35	2815	2768	1616	1566		1566
360	8.06	8.05	9.37	3193	3012	1755	1649		1649
370	-	-	10.31	3350	3274	1988	1864		1864

Table A-3.17 Experimental data result of beam (T26).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.16	0.17	0.20	27	20	25	20
30	0.21	0.21	0.23	37	37	35	33
40	0.36	0.33	0.38	47	41	47	39
50	0.59	0.56	0.62	69	65	67	66
60	0.74	0.76	0.82	94	92	118	92
70	0.91	0.90	1.08	120	134	145	128
80	1.05	1.05	1.28	213	219	225	212
100	1.42	1.43	1.76	479	540	335	315
120	1.78	1.78	2.21	625	734	428	405
140	2.16	2.19	2.76	762	953	534	494
160	2.71	2.68	3.28	952	1110	648	584
180	3.21	3.25	3.87	1128	1291	708	641
200	3.67	3.66	4.39	1286	1474	786	712
220	4.02	3.98	4.95	1435	1675	913	815
230	4.42	4.39	5.21	1514	1750	976	910
240	4.65	4.64	5.5	1609	1831	1137	985
260	5.01	4.98	6.05	1761	1976	1254	1085
280	5.63	5.64	6.70	1948	2167	1389	1197
300	6.14	6.12	7.29	2019	2259	1525	1305
320	6.69	6.66	7.87	2233	2350	1702	1474
340	7.23	7.25	8.48	2369	2422	2031	1570
360	7.80	7.84	9.14	2485	2595	2329	1821

Table A-3.18 Experimental data result of beam (T36).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
P							
0	0	0	0	0	0	0	0
20	0.16	0.18	0.20	22	24	16	17
30	0.24	0.27	0.32	33	31	26	28
40	0.32	0.35	0.43	47	41	39	43
50	0.4	0.43	0.55	66	62	56	67
60	0.51	0.53	0.70	98	88	68	106
70	0.65	0.63	0.87	128	106	112	137
80	0.74	0.79	1.08	212	197	145	186
100	1.09	1.11	1.50	561	487	198	241
120	1.48	1.45	1.94	728	760	279	323
140	1.82	1.81	2.40	905	945	368	401
160	2.17	2.23	2.87	1045	1078	448	477
180	2.69	2.71	3.45	1205	1220	532	546
200	3.15	3.14	3.98	1360	1388	621	616
220	3.49	3.56	4.5	1518	1551	710	681
230	3.85	3.83	4.88	1623	1605	748	717
240	3.99	4.05	5.17	1729	1689	793	755
250	4.47	4.51	5.7	1896	1774	895	823
280	4.94	4.95	6.27	2058	1947	1004	894
300	5.49	5.45	6.87	2235	2128	1115	1043
320	5.93	5.96	7.51	2409	2317	1218	1131
340	6.68	6.67	8.35	2556	3350	1328	1263
360	7.94	7.95	10.18	2754	3966	1643	2075

Table A-3.18 Experimental data result of beam (T36).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.16	0.18	0.20	22	24	16	17
30	0.24	0.27	0.32	33	31	26	28
40	0.32	0.35	0.43	47	41	39	43
50	0.4	0.43	0.55	66	62	56	67
60	0.51	0.53	0.70	98	88	68	106
70	0.65	0.63	0.87	128	108	112	137
80	0.74	0.79	1.08	212	197	145	186
100	1.09	1.11	1.50	561	487	198	241
120	1.48	1.45	1.94	728	760	279	323
140	1.82	1.81	2.40	905	945	368	401
160	2.17	2.23	2.87	1045	1078	448	477
180	2.69	2.71	3.45	1205	1220	532	546
200	3.15	3.14	3.98	1360	1388	621	616
220	3.49	3.56	4.5	1518	1551	710	681
230	3.85	3.83	4.88	1623	1605	748	717
240	3.99	4.05	5.17	1729	1689	793	755
260	4.47	4.51	5.7	1896	1774	895	823
280	4.94	4.95	6.27	2058	1947	1004	894
300	5.49	5.45	6.87	2235	2128	1115	1043
320	5.93	5.96	7.51	2409	2317	1218	1131
340	6.68	6.67	8.35	2556	3350	1328	1263
360	7.94	7.95	10.18	2754	3966	1643	2075

Table A-4.1 Experimental data result of beam (M11).

Load P [KN.]	Deflection		Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]
0	0	0	0	0	0	0
20	0.34	0.30	0.35	37	46	46
30	0.52	0.50	0.56	66	62	67
40	0.73	0.70	0.78	83	72	94
50	0.90	0.90	0.97	122	148	142
60	1.08	1.10	1.18	192	246	204
65	1.27	1.30	1.46	262	399	353
70	1.39	1.50	1.63	368	454	444
75	1.53	1.60	1.82	406	523	487
80	2.21	2.20	2.48	732	630	526
85	2.42	2.40	2.71	895	717	601
90	2.65	2.70	2.97	1222	1007	715
95	2.98	3.00	3.30	1338	1077	816
100	3.19	3.20	3.60	1457	1153	913
110	3.48	3.50	4.35	1651	1360	1051
120	6.53	6.60	7.77	2758	2682	1261
130	9.37	9.30	11.55	5771	4909	2525
140	10.50	11.00	13.05	6184	5504	3507
150	-	-	19.50	14025	13034	4491
						... 3270

Table A-4.2 Experimental data result of beam (M21).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ε_{t1} [$\times 10^6$]	ε_{t2} [$\times 10^6$]	ε_{c1} [$\times 10^6$]	ε_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.27	0.25	0.32	39	48	48	59
30	0.33	0.28	0.42	54	61	75	73
40	0.37	0.35	0.52	78	84	89	87
50	0.50	0.52	0.66	116	129	126	93
60	0.83	0.80	1.46	247	218	184	226
65	1.15	1.16	1.91	401	348	259	326
70	1.28	1.27	2.11	489	445	288	402
75	1.48	1.46	2.38	580	536	356	467
80	1.67	1.69	2.59	645	640	383	546
85	1.81	1.79	2.76	794	801	411	579
90	2.19	2.19	3.19	914	933	446	625
95	2.48	2.46	3.55	1160	1091	502	758
100	2.71	2.69	3.82	1344	1482	583	833
110	3.16	3.15	4.42	1634	1798	863	930
120	5.53	5.48	7.28	2416	2714	1044	1241
130	6.86	6.75	9.25	5592	6056	1208	1432
140	8.99	8.96	13.15	8080	8734	1671	1851
150	-	-	26.28	16850	18163	3238	3517

Table A-4.3 Experimental data result of beam (M31).

Load P [KN.]	Deflection			Tension Fiber Strain			Compression Fiber Strain		
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0.00	0.00	0.00	0	0	0	0	0	0
20	0.28	0.26	0.30	41	48	47	51	47	51
30	0.36	0.34	0.42	68	72	64	72	64	72
40	0.56	0.53	0.64	78	88	84	95	84	95
50	0.73	0.66	0.78	93	117	107	99	107	99
60	0.91	0.86	1.03	121	142	132	128	132	128
65	1.02	0.99	1.13	289	306	148	132	148	132
70	1.24	1.19	1.51	409	454	227	189	227	189
75	1.66	1.61	1.80	582	858	286	298	286	298
80	1.80	1.75	1.98	693	979	309	329	309	329
85	1.89	1.86	2.19	827	1124	332	378	332	378
90	2.06	2.02	2.38	917	1222	358	413	358	413
95	2.33	2.37	2.83	1054	1371	487	680	487	680
100	2.82	2.87	3.23	1291	1628	526	826	526	826
110	3.64	3.63	4.37	1752	2012	786	917	786	917
120	6.45	6.45	7.81	3062	3478	862	952	862	952
130	9.64	9.61	12.00	7959	8599	1304	1555	1304	1555
140	-	-	20.05	14239	15353	2455	2765	2455	2765
150	-	-	25.72	17646	19018	3262	3624	3262	3624

Table A-4.4 Experimental data result of beam (M12).

Load P [KN.]	Deflection		δ [mm]	Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]		ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.26	0.23	0.32	40	30	62	53
30	0.45	0.47	0.56	62	58	78	63
40	0.61	0.61	0.73	72	69	94	88
50	0.75	0.73	0.91	104	84	112	125
60	0.88	0.85	1.06	121	114	203	237
65	0.95	0.90	1.14	224	298	289	326
70	1.12	1.00	1.35	335	548	333	416
75	1.28	1.13	1.57	455	648	447	552
80	1.39	1.21	1.71	517	738	492	608
85	1.56	1.46	1.98	629	814	538	652
90	1.82	1.56	2.25	703	872	590	936
95	1.93	1.64	2.38	742	939	645	992
100	2.05	1.73	2.52	791	1020	702	1060
110	2.35	2.00	2.85	882	1218	864	1208
120	2.65	2.19	3.15	981	1437	925	1361
130	3.05	2.48	3.64	1241	1458	1041	1485
140	3.40	2.79	4.04	1388	1585	1211	1640
150	5.11	4.29	6.17	1832	2645	1618	2026
160	6.21	5.10	7.51	2477	5523	1825	2485
170	9.78	8.32	12.3	5427	10052	2875	3283

Table A-4.5 Experimental data result of beam (M22).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.23	0.23	0.27	28	34	56	34
30	0.29	0.32	0.45	51	37	77	58
40	0.42	0.47	0.61	79	49	91	75
50	0.64	0.61	0.87	90	76	105	105
60	0.89	0.91	1.15	114	81	136	141
65	1.07	1.02	1.40	189	232	212	250
70	1.17	1.20	1.51	294	235	261	273
75	1.38	1.40	1.82	401	243	373	303
80	1.49	1.49	1.95	469	248	396	313
85	1.79	1.73	2.18	525	262	427	364
90	1.96	1.93	2.39	604	270	459	392
95	2.15	2.17	2.6	727	447	488	454
100	2.28	2.27	2.76	771	553	510	482
110	2.58	2.58	3.23	849	698	559	537
120	3.04	3.02	3.69	967	819	643	602
130	3.33	3.28	4.07	1064	914	708	673
140	3.74	3.69	4.55	1199	1061	778	733
150	4.46	4.44	5.54	1321	1240	930	863
160	6.78	6.74	8.24	2798	2234	1350	1174
170	7.70	7.69	9.43	3248	2759	1589	1392
180	11.00	11.00	13.9	8916	6262	2249	1941
143	-	-	14.4	8990	7438	2484	1786

Table A-4.6 Experimental data result of beam (M32).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.18	0.17	0.20	42	41	42	38
30	0.24	0.26	0.31	59	57	64	46
40	0.41	0.43	0.52	74	87	86	73
50	0.57	0.56	0.69	94	127	95	98
60	0.72	0.70	0.86	116	163	104	111
65	0.82	0.83	1.08	335	228	225	187
70	0.97	0.92	1.21	406	339	248	216
75	1.07	1.00	1.33	456	493	278	249
80	1.30	1.25	1.58	464	682	312	284
85	1.34	1.30	1.78	691	771	364	340
90	1.64	1.60	2.16	739	949	379	361
95	1.98	1.91	2.31	796	1035	404	385
100	2.11	2.08	2.46	847	1121	434	425
110	2.36	2.37	2.83	1009	1265	482	479
120	2.62	2.65	3.26	1129	1418	529	551
130	3.16	3.15	3.66	1263	1785	588	624
140	3.63	3.66	4.14	1402	1937	633	673
150	4.1	4.04	4.68	1696	2199	682	745
160	5.81	4.83	6.50	1875	2427	970	1072
170	7.15	7.12	8.24	2920	3292	1280	1377
180	10.51	10.5	14.5	8192	9172	1797	2139

Table A-4.7 Experimental data result of beam (M13).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.24	0.26	0.27	34	25	58	52
30	0.34	0.32	0.39	49	44	71	64
40	0.45	0.42	0.65	64	56	81	74
50	0.62	0.68	0.81	79	81	96	117
60	0.74	0.79	1.02	102	115	148	196
65	0.98	1.02	1.24	168	185	222	285
70	1.14	1.19	1.45	280	385	317	366
75	1.32	1.33	1.61	325	453	348	421
85	1.44	1.44	1.86	374	628	428	496
90	1.62	1.68	2.24	415	666	546	568
95	1.83	1.89	2.46	534	749	576	598
100	1.98	2.05	2.75	690	816	648	669
110	2.17	2.22	2.99	784	945	690	721
120	2.82	2.86	3.64	905	1161	788	801
130	3.09	3.06	4.05	1097	1256	851	862
140	3.14	3.19	4.52	1203	1396	962	1043
150	3.57	3.52	5.58	1467	1505	1134	1212
160	4.26	4.32	6.73	1677	1848	1308	1311
170	4.72	4.81	9.39	2444	2999	1668	1615
190	6.46	6.52	13.25	4639	4960	2576	2864
200	7.44	7.53	16.01	6090	7044	3257	3280
210	9.10	9.08	23.54	10771	11185	4421	4136

Table A-4.8 Experimental data result of beam (M23).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.24	0.22	0.29	38	30	40	36
30	0.34	0.31	0.41	63	51	62	54
40	0.44	0.46	0.64	90	71	82	71
50	0.71	0.68	0.88	113	78	97	98
60	1.03	0.96	1.15	219	92	159	119
65	1.13	1.12	1.27	289	167	178	172
70	1.31	1.26	1.61	324	212	192	194
75	1.44	1.47	1.83	414	323	205	209
80	1.75	1.68	2.13	650	442	296	291
85	2.03	2.03	2.37	713	525	309	311
90	2.17	2.18	2.54	861	693	333	368
95	2.4	2.33	2.79	931	784	351	425
100	2.54	2.46	2.96	1094	896	367	447
110	2.88	2.84	3.33	1238	946	400	474
120	3.20	3.17	3.70	1369	1021	432	508
130	3.60	3.62	4.21	1475	1178	460	549
140	4.10	4.05	4.69	1589	1329	528	586
150	4.44	4.45	5.08	1714	1534	567	678
160	4.81	4.77	5.51	1864	1726	728	906
180	6.34	6.28	7.44	3889	4628	1363	1439
188	9.85	9.78	11.81	5436	6321	1697	1718
200	10.33	10.3	12.88	6907	7863	1859	2469
184	-	-	-	8739	10879	2553	3109

Table A-4.9 *Experimental data result of beam (M33).*

Load P [kN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.21	0.20	0.24	38	31	40	47
40	0.46	0.45	0.66	63	61	72	76
60	0.86	0.85	1.01	113	91	117	125
80	1.70	1.63	2.04	528	512	273	306
90	2.03	2.01	2.43	639	793	312	347
100	2.36	2.35	2.95	703	873	331	479
110	2.84	2.79	3.49	918	982	386	522
120	3.35	3.37	4.01	1058	1103	427	568
140	4.08	3.97	4.85	1316	1246	503	690
150	4.45	4.42	5.29	1446	1496	542	748
160	4.82	4.76	5.73	1574	1658	528	801
170	5.58	5.57	6.53	1824	1956	744	1026
180	7.06	6.98	8.5	3817	4034	1144	1599
190	8.23	8.27	10.07	4646	5221	1470	2082
200	9.26	9.28	11.51	6058	8879	1650	3099

Table A-4.10 *Experimental data result of beam (M14).*

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.22	0.21	0.26	36	31	41	36
40	0.51	0.59	0.69	63	58	76	68
50	0.68	0.65	0.76	92	84	98	97
60	0.75	0.71	0.85	118	98	118	129
70	0.83	0.79	0.95	248	212	187	193
80	1.02	0.95	1.15	448	386	242	218
90	1.08	1.09	1.42	536	494	282	300
100	1.28	1.24	1.53	662	610	351	344
120	1.76	1.77	2.08	781	698	452	385
140	2.72	2.68	3.16	926	846	534	449
160	4.22	4.14	4.81	1083	912	608	524
180	5.23	5.2	6.09	1208	1059	694	596
190	5.8	5.78	6.7	1460	1090	714	609
200	7.61	7.63	7.53	3215	2655	1240	1127
210	-	-	10.7	3605	3285	1467	1393
220	-	-	-	4687	4453	1849	1841
230	-	-	-	5481	5295	2184	1884
240	-	-	-	6784	6808	2274	2074

Table A-4.11 *Experimental data result of beam (M24).*

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.2	0.21	0.24	37	35	41	34
40	0.35	0.32	0.42	68	75	87	82
45	0.41	0.43	0.56	83	84	85	85
50	0.6	0.61	0.78	87	92	93	97
60	0.84	0.87	1.12	128	107	147	132
65	1.04	0.98	1.25	208	129	162	168
70	1.18	1.19	1.46	318	234	230	200
75	1.28	1.28	1.54	370	310	251	232
80	1.42	1.39	1.72	563	412	302	268
90	1.76	1.77	2.11	736	680	352	312
100	2.14	2.15	2.48	868	708	404	388
120	2.72	2.38	3.16	926	846	562	466
140	3.46	3.41	4.08	1058	878	689	584
160	4.26	4.30	4.85	1183	1012	728	687
180	4.93	4.94	5.67	1242	1108	855	768
190	5.37	5.38	6.12	1308	1259	994	896
200	5.85	5.82	6.74	1460	1390	1014	1009
210	7.64	7.65	8.58	3215	2855	1240	1127
215	-	-	10.9	3805	3485	1467	1393
220	-	-	-	7887	6653	1749	1741

Table A-4.12 Experimental data result of beam (M34).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.19	0.2	0.26	27	28	39	36
40	0.44	0.41	0.56	60	63	58	61
45	0.51	0.52	0.6	69	74	79	77
50	0.61	0.62	0.72	77	77	92	98
60	0.79	0.79	0.94	88	92	147	127
65	0.88	0.9	1.05	122	164	162	147
70	1.05	1.02	1.26	235	399	181	215
75	1.24	1.26	1.52	382	445	227	246
80	1.36	1.40	1.67	415	492	278	285
90	1.63	1.62	1.99	576	609	322	330
100	1.94	1.97	2.38	775	784	416	404
120	2.47	2.49	3.03	1031	950	505	524
140	3.10	3.12	3.83	1289	1109	593	660
160	3.87	3.86	4.62	1575	1328	686	712
180	4.48	4.43	5.34	1852	1641	779	782
190	4.79	4.74	5.74	2010	1814	837	804
200	5.18	5.19	6.24	2225	2020	920	901
210	5.63	5.66	6.85	2579	2330	1018	932
215	6.10	6.08	7.58	3097	2812	1145	1079
220	6.82	6.85	8.59	4015	3682	1367	1147
230	-	-	9.96	5124	4880	1669	1422
235	-	-	10.8	6056	5809	1964	1776
240	-	-	-	7281	6695	2184	1920
245	-	-	-	8284	7418	2274	2124
250	-	-	-	9450	8868	2460	2375

Table A-4.13 Experimental data result of beam (M15).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.23	0.20	0.23	28	25	46	27
30	0.28	0.29	0.42	52	42	52	40
40	0.39	0.36	0.48	68	62	64	58
50	0.54	0.52	0.75	84	76	87	71
60	0.68	0.64	0.91	112	134	125	87
70	1.01	1.00	1.15	192	216	183	152
80	1.24	1.21	1.41	324	402	241	207
90	1.49	1.52	1.69	526	511	323	257
95	1.6	1.61	1.81	567	602	338	266
100	1.75	1.77	1.98	607	686	364	296
110	2.01	2.05	2.27	715	798	425	333
120	2.26	2.24	2.54	841	925	467	368
130	2.58	2.56	2.87	901	986	526	400
140	2.84	2.79	3.14	1033	1038	571	440
150	3.1	3.04	3.40	1131	1152	623	482
160	3.38	3.41	3.74	1251	1229	667	518
170	3.65	3.64	4.05	1380	1280	719	555
180	3.94	3.93	4.35	1494	1326	761	590
190	4.21	4.23	4.67	1598	1387	808	632
200	4.48	4.51	4.97	1745	1413	854	672
210	4.47	4.54	5.26	1829	1493	892	701
220	5.06	5.03	5.64	2120	1682	940	743
230	6.11	5.88	6.57	2261	1807	982	821
240	8.27	8.02	8.96	4061	4842	1290	1410
250	10.50	10.50	12.42	7139	7586	1821	1755
260	-	-	14.48	8005	9167	2068	1962
270	-	-	15.96	10842	9842	2373	2064
280	-	-	19.76	14396	12880	2656	2318

Table A-4.14 Experimental data result of beam (M25).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.25	0.21	0.24	33	27	38	39
30	0.31	0.28	0.33	48	45	73	54
40	0.41	0.4	0.46	63	58	102	66
50	0.55	0.57	0.67	84	76	123	96
60	0.69	0.63	0.83	103	89	156	114
70	1.07	1.05	1.31	329	472	215	184
80	1.26	1.23	1.58	456	575	304	230
90	1.45	1.41	1.82	587	676	347	282
100	1.71	1.72	2.09	664	752	388	327
110	1.93	1.94	2.36	773	860	428	359
115	2.06	1.58	2.52	825	920	450	374
120	2.22	2.17	2.69	886	975	466	404
130	2.51	2.54	3.04	988	1103	518	458
140	2.72	2.69	3.29	1047	1214	554	489
150	2.96	2.94	3.56	1091	1321	600	551
160	3.19	3.11	3.83	1167	1441	642	599
170	3.55	3.48	4.19	1244	1561	684	637
180	3.82	3.88	4.50	1325	1652	724	671
190	4.04	4.09	4.79	1438	1738	772	686
200	4.28	4.31	5.07	1549	1827	809	731
210	4.55	4.58	5.38	1772	1964	875	835
220	4.91	4.98	5.81	1941	2107	965	984
230	7.14	7.2	8.53	3425	3688	1309	1632
240	9.02	9.12	11.2	6368	6568	1766	1920
250	9.71	9.73	12.82	7384	7459	1948	2162
260	-	-	13.19	7546	7794	2080	2225
270	-	-	17.6	9732	12497	2382	3135
255	-	-	24.36	13817	16499	3580	3900

Table A-4.15 Experimental data result of beam (M35).

Load P [KN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0	0	0	0	0
20	0.25	0.18	0.22	26	30	28	21
30	0.31	0.26	0.38	42	46	41	27
40	0.42	0.38	0.42	52	56	62	54
50	0.64	0.58	0.71	72	78	92	81
60	0.78	0.77	0.86	98	100	150	106
70	0.88	0.91	1.02	198	184	185	184
80	1.18	1.21	1.33	455	260	283	268
90	1.39	1.41	1.60	569	450	342	317
100	1.58	1.65	1.87	634	580	387	396
110	1.86	1.88	2.16	742	682	428	464
115	1.98	2.00	2.32	771	724	448	491
120	2.08	2.12	2.47	806	745	471	531
130	2.28	2.37	2.82	883	810	485	576
140	2.57	2.6	3.11	957	910	523	628
150	2.76	2.84	3.48	1038	1030	547	685
160	3.13	3.06	3.78	1117	1150	567	712
170	3.38	3.29	4.09	1217	1270	617	746
180	3.66	3.58	4.37	1466	1559	654	786
190	3.91	3.89	4.64	1526	1684	698	824
200	4.13	3.93	4.97	1601	1808	744	918
210	4.41	4.33	5.25	1771	1927	787	964
220	4.65	4.59	5.65	1841	2067	828	1012
230	4.98	4.92	5.94	1905	2177	871	1060
240	5.25	5.16	6.25	2084	2309	916	1106
250	5.51	5.52	6.57	2276	2468	968	1257
260	6.82	6.95	9.79	8026	10191	1548	1653
270	7.41	7.52	11.69	11259	13463	2022	1907
280	-	-	15.35	18735	19778	2801	2440

Table A-4.16 Experimental data result of beam (M16).

Load	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{tl} [$\times 10^6$]	ϵ_{cl} [$\times 10^6$]	ϵ_{cl} [$\times 10^6$]	ϵ_{cl} [$\times 10^6$]
0	0	0	0.00	0	0	0	0
20	0.28	0.24	0.31	21	23	26	34
30	0.4	0.38	0.48	28	36	36	42
40	0.57	0.56	0.65	44	48	61	68
50	0.71	0.73	0.80	65	67	92	96
60	0.86	0.88	0.97	96	70	117	132
70	1.04	1.02	1.17	167	154	196	182
80	1.20	1.16	1.35	248	195	232	213
90	1.36	1.30	1.55	393	230	294	264
100	1.46	1.43	1.73	575	283	365	299
110	1.67	1.67	1.90	643	393	414	334
120	1.93	1.87	2.10	704	423	487	371
130	2.14	2.08	2.32	752	486	512	424
140	2.35	2.28	2.56	782	567	538	458
150	2.52	2.46	2.78	826	628	553	495
160	2.64	2.58	3.02	893	768	587	555
180	3.13	3.04	3.47	1063	876	655	632
200	3.6	3.59	4.04	1276	996	731	694
220	4.11	4.07	4.64	1527	1220	821	841
240	4.56	4.42	5.17	1862	1482	886	945
260	5.05	5.08	5.71	2010	1710	1115	1060
280	5.53	5.55	6.26	2214	2034	1451	1280
290	5.96	5.87	6.72	2311	2151	1561	1393
300	6.21	6.25	7.00	2475	2310	1639	1425
320	7.08	7.05	7.90	2657	2536	1864	1696
312	8.8	8.75	9.57	2718	2820	2005	1804

Table A-4.17 Experimental data result of beam (M26).

Load P [kN.]	Deflection			Tension Fiber Strain		Compression Fiber Strain	
	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]
0	0	0	0.00	0	0	0	0
20	0.27	0.25	0.31	24	27	29	39
30	0.37	0.36	0.41	31	40	36	45
40	0.47	0.47	0.52	61	53	63	66
50	0.58	0.59	0.65	82	64	81	87
60	0.72	0.73	0.80	102	93	123	114
70	0.87	0.89	0.97	219	138	164	153
80	1.02	1.04	1.15	293	184	247	229
90	1.17	1.20	1.33	373	193	337	298
100	1.32	1.34	1.49	500	326	449	343
120	1.69	1.68	1.90	669	438	594	432
140	2.04	2.02	2.29	820	624	662	509
160	2.42	2.43	2.73	1076	785	789	596
180	2.93	2.91	3.26	1247	912	887	697
200	3.38	3.36	3.77	1434	1116	987	817
220	3.82	3.81	4.26	1624	1380	1092	921
230	4.06	4.05	4.52	1718	1500	1163	938
240	4.30	4.29	4.78	1815	1627	1221	998
260	4.86	4.91	5.37	2092	1889	1309	1184
270	5.10	5.16	5.62	2187	2014	1387	1212
280	5.35	5.42	5.89	2281	2139	1414	1365
290	5.62	5.70	6.16	2368	2251	1473	1399
300	5.91	6.00	6.47	2471	2378	1511	1469
310	6.20	6.31	6.78	2578	2522	1563	1513
320	6.54	6.70	7.15	2668	2684	1627	1574
330	6.88	7.10	7.54	2765	2868	1683	1597
292	-	-	-	3134	3074	2158	1854

Table A-4.18 Experimental data result of beam (M36).

Load	Deflection			Tension Fiber Strain			Compression Fiber Strain		
P [KN.]	δ_1 [mm]	δ_2 [mm]	δ [mm]	ϵ_{t1} [$\times 10^6$]	ϵ_{t2} [$\times 10^6$]	ϵ_{c1} [$\times 10^6$]	ϵ_{c2} [$\times 10^6$]		
0	0	0	0	0	0	0	0		
20	0.2	0.19	0.21	26	26	31	24		
30	0.29	0.29	0.31	44	47	45	36		
40	0.39	0.38	0.42	67	64	61	59		
50	0.49	0.48	0.54	101	96	87	76		
60	0.62	0.61	0.68	118	113	112	98		
70	0.74	0.72	0.82	130	176	142	134		
80	0.89	0.87	1.00	184	263	206	187		
90	1.02	1.00	1.16	344	421	281	261		
100	1.16	1.13	1.34	446	524	321	294		
110	1.32	1.28	1.53	546	725	371	324		
120	1.47	1.44	1.73	783	821	453	385		
130	1.61	1.59	1.91	859	948	487	429		
140	1.77	1.75	2.12	912	1023	535	463		
160	2.1	2.08	2.53	1139	1250	617	526		
180	2.54	2.50	3.03	1310	1408	684	669		
200	3.02	3.00	3.58	1488	1684	784	764		
220	3.47	3.41	4.09	1659	1977	882	868		
240	3.94	3.89	4.64	1955	2162	984	947		
260	4.37	4.30	5.13	2259	2414	1118	1046		
280	4.83	4.74	5.65	2512	2688	1288	1133		
290	5.07	4.95	5.92	2699	2801	1365	1207		
300	5.30	5.18	6.18	2775	2995	1454	1296		
320	5.80	5.73	6.70	2939	3294	1613	1569		
340	-	-	7.40	3182	3543	1822	1753		
360	-	-	8.55	3465	3870	2529	2395		

Appendix B

Computer Program “Response”

B.1. INTRODUCTION

RESPONSE can be used to determine the load-deformation response of a prestressed concrete cross section subject to moment, axial load and shear force. RESPONSE is a user friendly program and facilitates many different ways of analyzing concrete sections.

The following titles explain the potential of the program RESPONSE in more details.

B.2. CONFIGURE-OPTIONS

RESPONSE can be configured so that many of the options will be selected automatically every time the program runs. In order to specify the options to be used, the configure menu is selected . If you

want the options to be remembered the next time the program is run use the “Update” option to save your selections.

B.2.1 Concrete Model

RESPONSE can be run using two different stress-strains curves for concrete. Throughout the program the stress-strain relationship proposed by Thorenfeldt, Tomaszewicz and Jensen is referred to as the “High-Strength Model” and the parabolic stress-strain curve is referred to as the “Parabolic-Model”. When results are displayed or stored, the models are referred to as “High” and “Para” respectively.

To change the stress-strain relationship select “Configure” from the main menu, select “Concrete-Model” and then select either the “Parabolic-Model” or the “High-Strength-Model”. When the parabolic stress-strain model is used the program will run somewhat faster because the computations of stresses and strains are simpler particularly in cases involving shear.

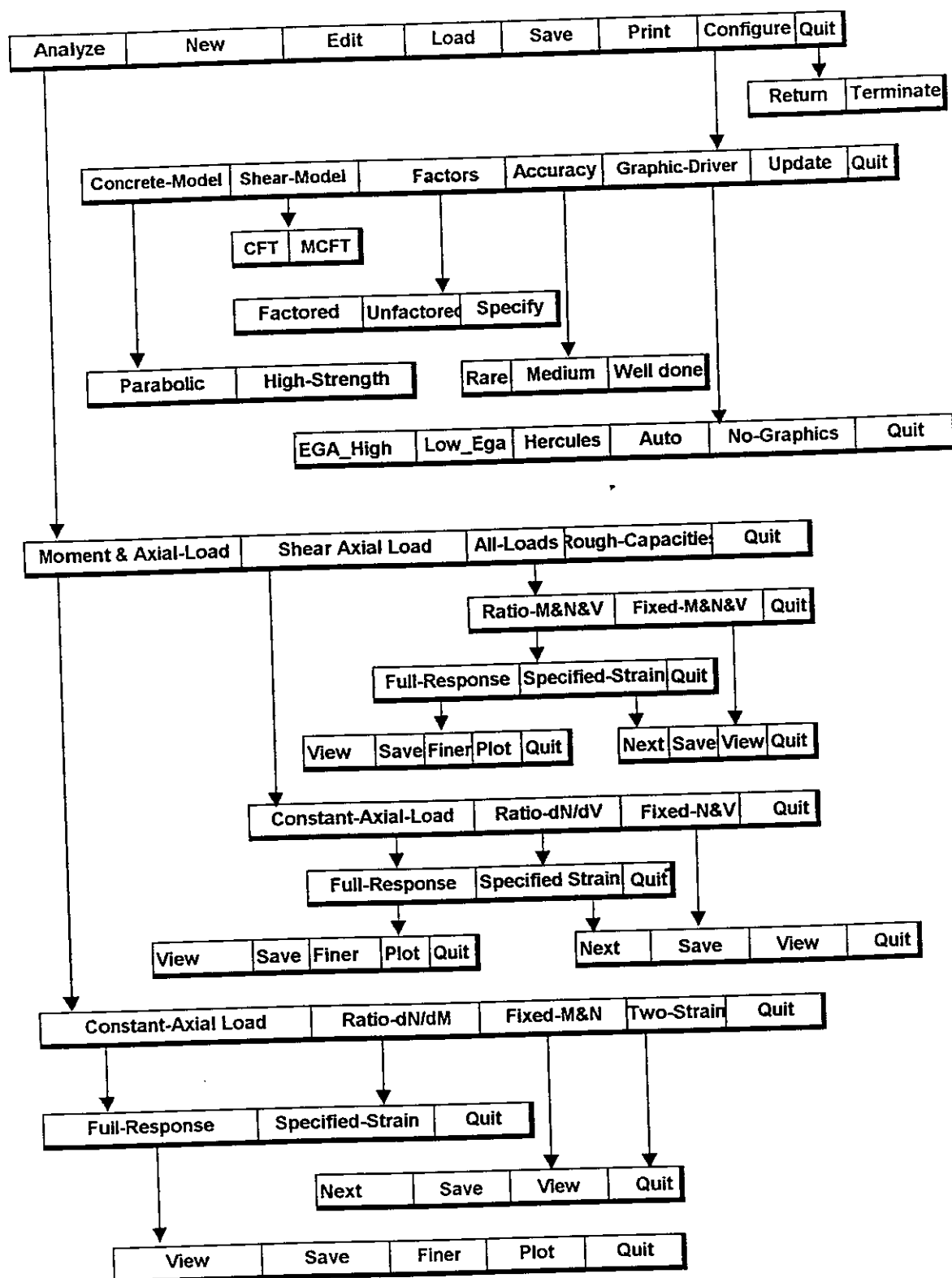


Figure B-1 Menu Structure for Program RESPONSE

B.2.2 Shear Model

RESPONSE can be run using two different theories for shear analysis. These two models are the modified compression field theory (MCFT) and the compression field theory (CFT)

B.2.3 Factored

In the ACI code the design strength of a section is found by multiplying the nominal strength (i.e. the unfactored strength) by a strength reduction factor ϕ . In the Canadian concrete design code the factored strength of a section is found by applying a material performance factor to each material and then calculating the strength using factored stress-strain curves for the materials. These material performance factors are: 0.60 for concrete, 0.85 for reinforcing steel, and 0.90 for pre-stressing steel and structural steel. The individual material factors can be changed using the “Specify” option, to accommodate other codes that use a factored-strength approach with different material factors.

To apply these factors to the individual material stress-strain curves select “Factored”. To use no material factors select “Unfactored”.

B.2.4 Accuracy

RESPONSE uses a number of different iteration routines to converge on the answer. To determine when the program has come sufficiently close to the answer the difference between the calculated result and the desired result are compared to a fraction of the axial or the flexural capacity of the section.

There are three levels of convergence, which can be specified: Rare, Medium or Well-Done. The convergence limits used are specified in Table B-2.

Table B-2 *Convergence limits*

Accuracy	Limit for Axial Load	Limit for Moment	Limit for Shear
Rare	$\pm$ <u>Compressive capacity</u> 200	$\pm$ <u>Flexural capacity</u> 200	$\pm$ <u>Compressive capacity</u> 600
Medium	$\pm$ <u>Compressive capacity</u> 1000	$\pm$ <u>Flexural capacity</u> 1000	$\pm$ <u>Compressive capacity</u> 3000
Well-done	$\pm$ <u>Compressive capacity</u> 5000	$\pm$ <u>Flexural capacity</u> 5000	$\pm$ <u>Compressive capacity</u> 15000

B.3. DEFINING A CROSS SECTION

The following sections describe the parameters used to define a reinforced section for analysis. Limits have been placed on parameters describing section geometry and material properties. The limitations on material properties were selected to allow reasonable material properties. These limitations were imposed to avoid problems with the display of very large force resultants and to provide a check on input data.

B.3.1 Units

Units can only be selected when a new section is entered. Once either Metric or U.S. Customary units has been specified, the units are fixed for the cross section and cannot be changed later. To select U.S. Customary units enter “*u*” and to select Metric units enter “*m*”. Data is entered and displayed in the units specified in Table B-3.

Table B-3 *Units used for input and output*

Unit System	Metric	U.S. Customary
Dimensions	millimeter	Inches (in.)
Stresses and Elastic Moduli	megaPascals (Mpa)	Kips per square in. (ksi)*
Strains	millistrain	Millistrain
Forces	kiloNewton (kN)	Thousand pounds (kips)
Moments	kiloNewton-meters (kNm)	Foot-kips (ft-kips)

*Concrete stresses in U.S. Customary units are entered and displayed in pounds per square inch (psi).

B.3.2. Number of Concrete Types

The program can use up to five different concrete types to model the response of a section. By default, it assumes that you have one type of concrete and it displays the parameters describing it. Typically, one layer of concrete is used for the concrete which does not participate in tension stiffening, and another type is used to represent the concrete, which is close to the reinforcing steel. A third type of concrete could be used to model topping concrete for a composite section. Note: The shear analysis routines use the properties of the first concrete type.

There has to be at least one type of concrete specified. To facilitate error checking of subsequent parameters, the number of concrete types can not be reduced at a later stage. To increase the number of concrete types, simply enter the number of concrete types, which you want to use for your section.

To change the parameters describing a concrete type, activate the cell containing the parameter and enter the new parameter. For each type of concrete enter the parameters defining the stress-strain relationship.

The parameter ϵ'_c refers to the strain when f_c reaches f'_c . If this is unknown enter “0” and the program will estimate the strain.

For the “Parabolic” concrete stress-strain model, the tangent modulus, E_{ct} , is estimated to be 1.1 times the secant modulus. Therefore, the following equations are used to determine ϵ'_c :
 $E_c = 62,700 / f'_c$, psi units $E_c = 5200 / f'_c$, Mpa units, ϵ'_{ct} .

Tension Stiffening Factor refers to the product of the factor accounting for bond characteristics and the factor for the type of loading (see Section 4.10 of the text). Suggested values for this factor are obtained by pressing “FI” (for help).

B.3.3 Number of Reinforcing Bar Types

By default, the program assumes that the section contains no reinforcing bars. The program can use up to five different types of mild reinforcing steel. Each steel type is described by a trilinear stress-strain relationship described in Fig. B-2. Enter the number of reinforcing steels which you want to use for your section and then enter the parameters describing the stress-strain relationships. If you wish to ignore strain hardening of the reinforcement, enter the same value for f_u and f_y . Note: To facilitate error checking, the number of reinforcing bar types can only be increased.

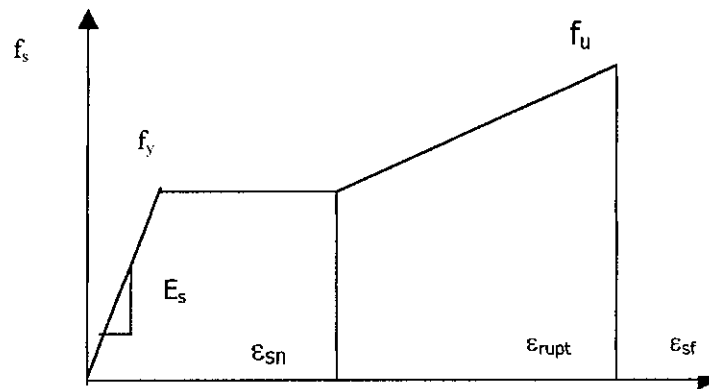


Fig.B-2 Stress-strain relationship of reinforcing bar

B.3.4 Number of Tendon Types

Again, by default, the program assumes that the section contains no prestressing tendons. The program can use up to five different types of prestressing steel. Prestressing steel types are defined using a modified Ramberg-Osgood curve for the stress-strain relationship. To specify tendon types (prestressing steels) enter the number of different tendon types you want to consider for the section. Once one or more tendon types have been specified, the parameters describing the stress-strain curve (s) are displayed. These parameters can then be modified as described previously. Note: To facilitate error checking, the number of tendon types can only be increased.

B.3.5 Height and Moment Axis

Once all the material properties have been specified the section geometry can be entered. The program uses the total height of the section in a number of data checks. Because of this, the total height is another quantity, which can only be increased. Therefore, the default height of section is 2 (inches) or 50 (millimeters).

The distance from the bottom of the section to the moment axis is used to define the location about which moments are taken. In many cases, the moment axis is taken at half the height of the section or as centroidal axis specified from the bottom of the section. If you do not know the location of the centroidal axis enter an initial estimate, which can be corrected after the program calculates the centroidal axis location.

B.3.6 Concrete Layers

The geometry of the cross section is defined by trapezoid layers which can be combined to make up a large variety of cross sections. All sections are assumed to be symmetrical about a vertical plane and are displayed as such on the screen by pressing “F9”. The minimum number of trapezoids required to define the cross-sectional geometry determines the numbers of layers used to model a cross section. Additional layers may have to be used to differentiate between different concrete types. Note that because of the techniques used in the numerical integration, it is not necessary to further subdivide the layers in an attempt to improve numerical accuracy. Each individual layer is defined using the following parameters: distance from the bottom of the section to the bottom of the layer, bottom width, top width, and height (See Fig. B-3)

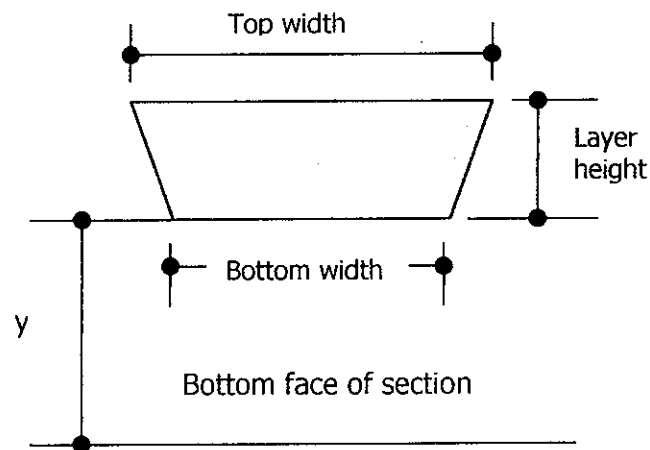


Figure B-3 Parameters defining a concrete layer.

The number of concrete layers used to describe a cross section is limited to 20. The number of concrete layers can be changed later, but a minimum of one layer has to be specified. Enter the number of concrete layers used and then modify the parameters for each layer to describe your section. At any time during this process you can press “F9” to see a plot of the cross section.

B.3.7 Reinforcing Bar Layers

If no reinforcing bar types were specified previously, no reinforcing bar layers can be entered. By default the number of reinforcing bar layers is zero. To specify reinforcing bar layers enter the number of layers you want to use (limited to a maximum of ten). Then define the location of each layer, the cross-sectional area of each layer, and the type of reinforcing bar used for each layer.

B.3.8 Displaced Concrete

This option will account for the amount of concrete displaced by reinforcing bar and tendon layers. If you do not want to account for the displaced concrete enter “*n*” otherwise enter “*y*” and the concrete type replaced by each steel layer

B.3.9 Strain Discontinuous

Composite members, such as precast beam with a cast-in-place topping, generally have a strain discontinuity at the interface between the component parts of the section. To account for this discontinuity, it is necessary to treat the total strain as the sum of the initial strains, which existed when the final piece was placed and the additional strains occurring after the cross section had been completed.

Note that for members with strain discontinuities the program reports the strains and curvatures associated with the additional strains and curvatures that occur after completion of the section rather than the total strains.

B.4 ANALYSIS OPTIONS

Once a section has been loaded or entered it can be analyzed. The many different analysis options (see Fig. B-1) can be divided into the following three categories:

1. Moment and Axial Load

- * Calculate moment-curvature relationships for given axial loads and/or axial load-to-moment ratios.

- * Find the strain distribution for a given axial load and/or axial-load-to-moment ratio and a specific strain anywhere within the section.
 - * Find the strains for given moment and axial load.
 - * Find the axial load and moment for given strains.
2. Shear and Axial Load
- * Calculate shear force vs. strain relationship for given axial loads and/or axial load-to-shear ratios.
 - * Find the biaxial strain state for given axial load and/or axial-load-to-shear ratio with a specified principal strain.
 - * Find the biaxial strain state for given axial load and shear force.
3. All Loads
- * Calculate axial load, shear and moment vs. strain relationships of a variety of linear combinations of shear-to-axial-load and shear-to-moment ratios.
 - * Find the strain state due to applied moment, axial load and shear.
4. Rough Capacities
- * Calculate estimates of the compressive and tensile capacities and the positive and negative moment capacities.
 - * Calculate estimates of the cracking shear force and the post-cracking shear strength.

The available analysis options are described in more detail in the sections below.

B.4.1 Moment & Axial Load

The “Moment & Axial-Load” option has several analysis features which all use plane sections analysis. The moment and axial load option can be selected by first selecting “Analyze” from the main menu and then by selecting “Moment & Axial-Load” from the next menu.

Once “Moment & Axial-Load” has been selected the options outlined below are available.

B.4.1.1. Constant-Axial Load

For a selected axial load the full moment-curvature response or the curvature and moment for a specified strain anywhere within the section can be calculated using this option.

To proceed with this analysis selects “Constant-Axial-Load” from the menu and then specify the following:

1. Axial load
2. Whether to account for tension stiffening or not
3. Now the “Full-Response Specified-Strain Quit” menu will appear. Select an option from the menu.
4. The iteration will now be displayed on screen
5. Depending on your choice in the previous menu either the “View Save Finer Plot Quit” or the “Next Save View Quit” menu will appear.

B.4.1.2. Ratio dN/dM

This analysis can be used to get the full response for a section loaded by an axial load which is a function of the moment. In addition, a moment and axial load pair satisfying the relationship described below can be calculated for a specified strain anywhere within the section. The relationship between moment and axial load is described by the following equation: axial load = (axial load at zero moment) + dN/dM * moment.

To proceed with this analysis select “ dN/dM ” from the menu and then specify the following:

1. Axial load when zero is moment
2. Ratio dN/dM
3. Whether to account for tension stiffening or not
4. Now the “Full-Response Specified-Strain Quit” menu will appear. Select an option from the menu.
5. The iterations will now be displayed on screen
6. Depending on your choice in the previous menu either the “View Save Finer Plot Quit” or the “Next Save View Quit” menu will appear.

B.4.1.3. Fixed – M & N

This option is used to find the strain distribution across the section under a given moment and axial load.

B.4.1.4. Two-Strains

This option calculates the moment and axial load corresponding to a given strain distribution. Entering two locations within the section and the strains at these locations specifies the strain distribution.

B.4.2 Shear & Axial-Load

The “Shear & Axial-Load” option has analysis features, which can be performed using either the compression field theory or the modified compression field theory or the modified compression field theory

B.4.2.1. Constant-Axial-Load

For a specified axial load the full shear-deformation response or the shear force and the biaxial Strain State for a specified principle tensile strain can be calculated using this option.

B.4.2.2. Ratio dN/dV

This analysis can be used to get the full response for a section loaded by a shear force and an axial load, which is a function of the shear force. In addition a shear and axial load pair satisfying the relationship described below can be calculated for a specified principal strain. The relationship between axial load and shear is described by the following equation: $\text{axial load} = (\text{axial load at zero shear}) + dN/dV * \text{shear}$.

B.4.2.3. Fixed-N & V

This option is used to find the biaxial distribution for a given shear and axial load. The analytical method used for this analysis is a gradient approach, which may run into convergence problems.

B.4.3 All-Loads

This option enables the load-deformation response to be produced for a given ratio between shear and moment and between shear and axial load. In addition, the strain state for a given combination of loads can be found. Once “All-Loads” has been selected the following options are available.

B.4.3.1. Ratio-M&N&V

This analysis can be used to get the full response for a section loaded by a specific loading regime. Both the axial load and the moment are functions of the shear force.

$$\text{Axial Load} = (\text{axial load at zero shear}) + dN/dV * \text{shear}$$

$$\text{Moment} = (\text{moment at zero shear}) + dM/dV * \text{shear}$$

B.4.3.2. Fixed-M&N&V

This analysis is used to find the strains corresponding to a specified axial load moment and shear.

B.5 NUMERICAL INTEGRATION

In determining the axial load and moment on the section, it is necessary to evaluate the following two integrals:

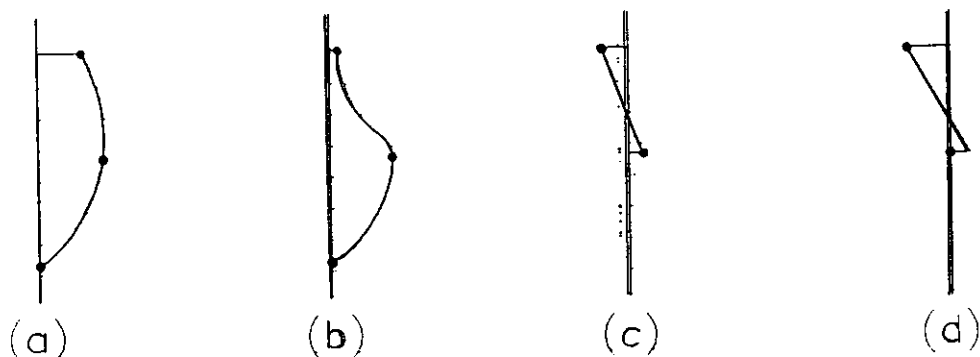
$$N = \int_A f(y) dA$$

$$M = \int_A f(y) y dA$$

Because these integrals cannot be evaluated in closed form, numerical integration is used. For the numerical integration, a parabolic stress distribution across each individual layer is assumed. The shape of the parabolic distribution is determined from the calculated stresses at the bottom, middle and top of the layer. This method of integration is very accurate for layers subject to compressive stresses (based on a "Parabolic" stress-strain relationship) over the depth of the layer.

However, the approach can become inaccurate if the stresses are not compressive over the entire layer or if the "High Strength" stress-strain relationship is used. Figure B-4 shows examples of the difference between the actual stress distributions and the parabolic distributions used in the numerical integration.

Actual stress distribution



Assumed stress distribution

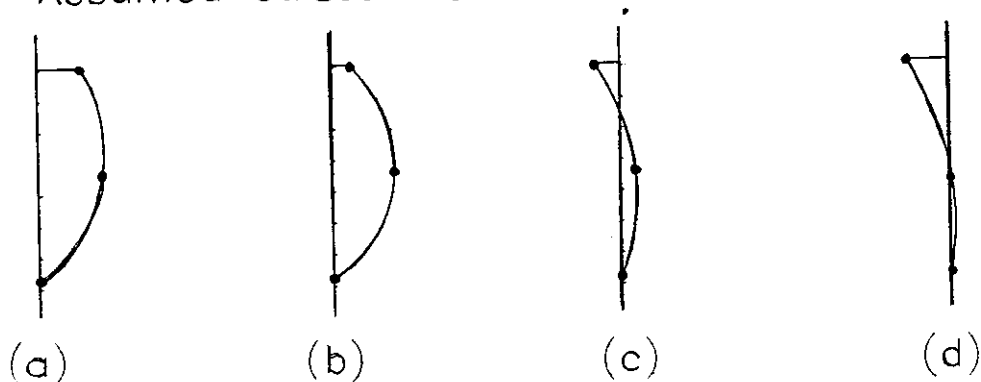


Fig.B-4 Actual and assumed stress distribution across individual layers

To determine if the integration is sufficiently accurate, the program uses the following procedure:

1. Calculate the stress at the top, middle, and bottom of the layer. Determine the parabolic stress distribution based on these three stresses.
2. Calculate the actual stresses at the fifth points within the layer and compare these to the stresses obtained from the parabolic stress distribution.

3. If the two sets of stresses do not agree within about 1% then split the layer into three equal sub-layers.
4. Repeat steps 1, 2 and 3 for each sub-layer.

This process, outlined in Figure B-5 is repeated up to eight times if necessary.

Because of this automatic subdivision procedure, the number of layers used for the numerical integration does not have to be more than the minimum number of trapezoids necessary to describe the section geometry and material distribution. This algorithm is efficient because it increases the number of layers only where and when necessary.

B.6 ITERATION PROCEDURES

B.6.1. Iterating on One Strain Variable, Members with no Shear

For a specified axial load and a strain specified anywhere within the section (e.g. cracking strain at the bottom face), the corresponding moment is found using the following procedure:

1. Given the location of the specified strain select a location for a second strain and guess strain.
2. Calculate the stress resultants for the strain distribution defined by the specified strain and the second strain.
3. Increment the second strain by a small amount.
4. Calculate the axial stress resultant and the moment for the new

strain distribution

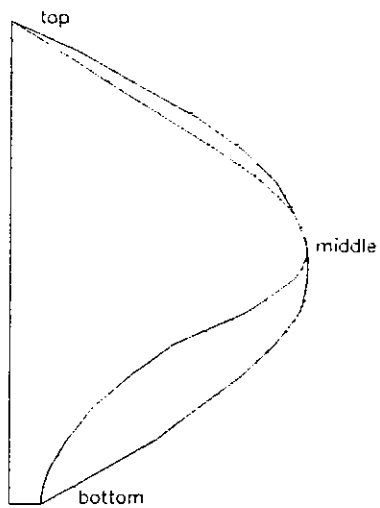
5. Compare the specified axial load to the new axial stress resultant obtained in step 4. If they are close enough then the moment obtained in step 4 is the moment corresponding to the specified axial-load and the specified strain. If they are not close enough proceed to step 6.
6. Find the rate of change of the axial stress resultant with respect to the second strain. Use this rate of change to make a new estimate of the second strain. Return to step 4.

The procedure is a simplification of the actual algorithm used by the program

Another single iteration algorithm is used to develop moment-curvature diagrams. The moments corresponding to a given axial load and specified curvatures are calculated. Then the curvature is incremented until the full response has been calculated. The procedure used to find the moment is similar to that outlined above. The difference is that the curvature is the fixed strain variable and the strain at the center is iterated on until the axial stress resultant converges to the specified axial load.

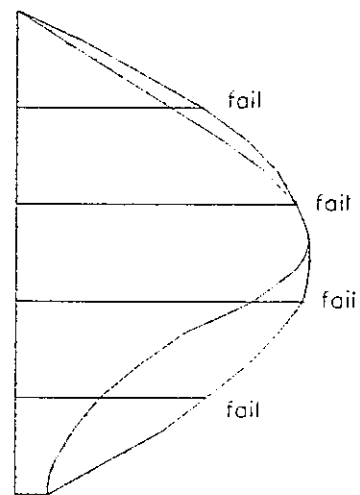
A third single-iteration algorithm is used to find the axial load corresponding to a given moment and a specified axial strain. This algorithm is used as a subroutine for calculations involving moment, axial load and shear. Again the numerical procedure is similar to the one outlined above. The axial strain is fixed and the curvature is changed until the moment resultant converges on the given moment.

Step 1



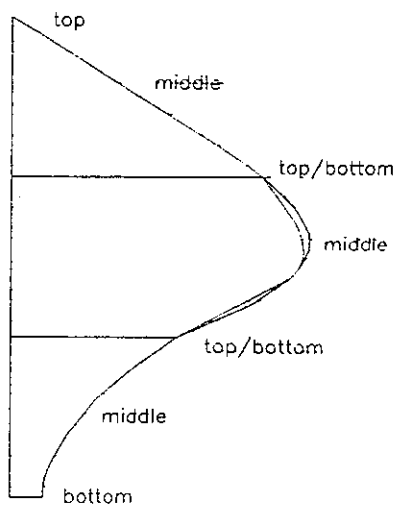
concrete stress distribution
and parabolic stress distribution

Step 2



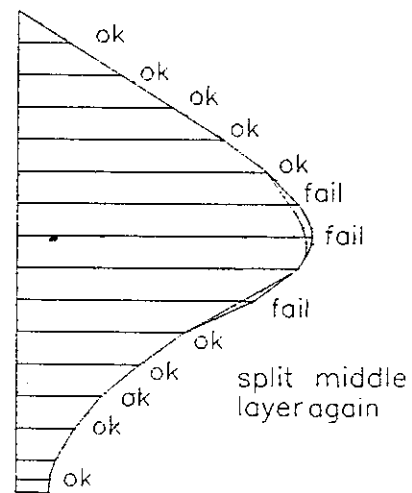
check numerical integration

Step 3



split into 3 sublayers

Step 4



check numerical integration
for sublayers

Fig. B-5 Numerical integration check and automatic subdivision of layers.

B. 6.2. Iterating on Two Strain Variables, Members with No Shear

To determine the strain distribution corresponding to a given axial load and a given moment, the strain at the centroid and the curvature have to be found by iteration. This can be achieved by two methods. The first method iterates on both variables simultaneously until stress resultants have converged on the given loads. This approach is called the "Gradient" approach. Another approach called the "Two-Stage" algorithm is used when the "Gradient" approach runs into difficulties. The "Two-Stage" approach uses two nested single iterations to find the strain distribution.

B.6.2.1. The Gradient Approach

The "Gradient" approach starts with a guess of initial strains and then calculates the stress resultants and the stiffness of the section. Using these resultants and the stiffness a new estimate of the strains is calculated. This process is repeated until the resultants match the specified moment and axial load. This procedure can run into numerical difficulties when the given loads are close to those loads causing cracking of the section. Under these conditions the stiffness of the section changes suddenly from one iteration to the next. If this procedure has not converged after 20 attempts, the program then uses the "Two-Stage" approach. The gradient approach is outlined in more detail in Ref. B-1.

B.6.2.2. The Two-Stage Approach

The "Two-Stage" approach is more stable than the "Gradient" approach because only one variable is iterated on at a time. The iteration routine fixes the strain at the bottom of the section and finds the strain distribution for the given axial load using the routine described in Section B.6.1. The second iteration routine changes the bottom strain until the moment resultant converges to the given moment. If this method does not converge after 50 iterations a variant of the "Two-Stage" approach is used to iterate on the top strain. The "Two-Stage" algorithm is outlined in more detail in Ref. B-1.

B.6.4. Convergence Limits

The iterative algorithm described above are used to converge on stress resultants. To determine whether a stress resultant is close enough to the given loads, convergence limits have to be specified. Convergence limits used by the program are linked to the capacity of the section. In addition, the user can choose among three levels of accuracy, which have been provided to give the user some control over the limits used by the program. If answers do not have to be very exact, a lower accuracy can be used and less iteration will be required to perform calculations. This can translate into significant timesaving on slow microcomputers.

APPENDIX C

NUMERICAL EXAMPLES

C.1 Introduction

As long as the tensile stress in the concrete is smaller than the modulus of rupture (F_r), so that no tension cracks develop, the strain and stress distribution is essentially the same as in an elastic, homogeneous beam. The only difference is the presence of another material, the steel reinforcement. In the elastic range, for any given value of strain, the stress in the steel is “ n ” times that of the concrete eq.(1).

$$f_s = \frac{E_s}{E_c} f_c = n f_c \quad (1)$$

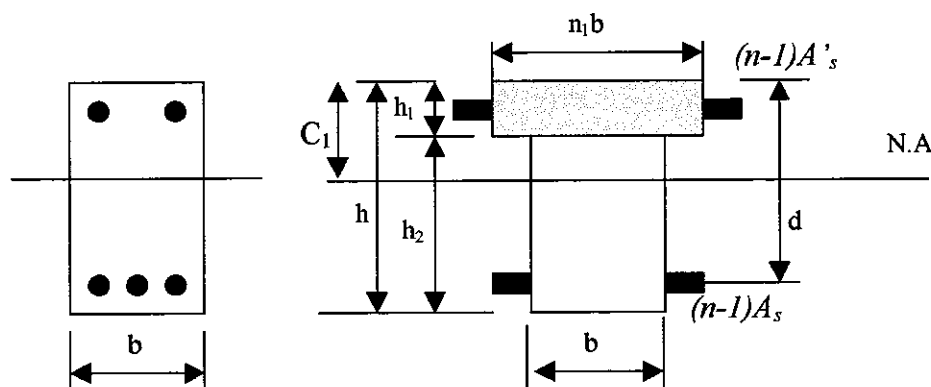
where $n = E_s/E_c$ is known as the *modular ratio*.

In calculations account can be taken of this fact by replacing the actual steel and concrete cross section with a fictitious section thought of as consisting of concrete only.

In this transformed section the actual area of the reinforcement is replaced with an equivalent concrete area equal to nA_s located at the level of the steel. Once the transformed section has been obtained, the usual methods of analysis of elastic homogeneous beams apply. That is, the section properties (location of neutral axis, moment of inertia, section modulus, etc.) are calculated in the usual manner.

C.1.1. Neutral axis location for transformed, uncracked section.

Example 1: Rectangular reinforced concrete beam with top overlay)



a) Actual section

b) Transformed, uncracked section

Fig.C.1. Actual and transformed, uncracked section of example 1.

Knowing the section geometry as presented below :

b	$= 200$	$[\text{mm}]$	d'	$= 50$	$[\text{mm}]$
h_1	$= 120$	$[\text{mm}]$	d	$= 292$	$[\text{mm}]$
h_2	$= 230$	$[\text{mm}]$	f_r	$= 4.64$	$[\text{N/mm}^2]$
h	$= 350$	$[\text{mm}]$	A'_s	$= 226$	$[\text{mm}^2]$
$E_{c(\text{Nor})}$	$= 33200$	$[\text{N/mm}^2]$	A_s	$= 515$	$[\text{mm}^2]$
$E_{c(\text{HPC})}$	$= 44800$	$[\text{N/mm}^2]$	f_y	$= 440$	$[\text{N/mm}^2]$
E_s	$= 200000$	$[\text{mm}^3]$	n	$= E_s/E_{c(\text{Nor})}$	$= 6$
f'_c	$= 44$	$[\text{N/mm}^2]$	n_l	$= E_{c(\text{HPC})}/E_{c(\text{Nor})}$	$= 1.35$

The neutral axis location can be calculated from the geometry of the section as shown in Fig. c.1.b. for transformed, uncracked section as follow:
Taking moments of the area about neutral axis

$$n_l b h_1 (c_1 - h_1/2) + b (c_1 - h_1) (c_1/2 - h_1/2) + (n-1) A'_s (c_1 - d') = b (h - c_1) (h/2 - c_1/2) + (n-1) A_s (d - c_1) \quad (2)$$

substituting the known values in eq. (2-a) results in:

$$c_1 = 165.18 \text{ mm} \quad (2-a)$$

$$Y_t = (h - c_1) = (350 - 165.18) = 184.82 \text{ mm} \quad (3)$$

C.1.2. Calculation of moment of inertia (I_{ut}) for transformed, uncracked section

From the section geometry as shown in fig.c.1.b. the moment of inertia can be calculated as follows:

$$I_{ut} = (n_l b (h_1)^3)/12 + n_l h_1 b (c_1 - h_1/2)^2 + A'_s (n-1) (c_1 - d')^2 + (b (c_1 - h_1)^3)/12 + b (c_1 - h_1) ((c_1 - h_1)/2)^2 + (b (h - c_1)^3)/12 + b (h - c_1) ((h - c_1)/2)^2 + A_s (n-1) (d - c_1)^2 \quad (4)$$

Substituting the values of the parameters in eq.4 results in:

$$I_{ut} = 8.807 \times 10^8 \text{ [mm}^4\text{]}. \quad (4-a)$$

C.1.3. Calculation of cracking moment (M_{cr}).

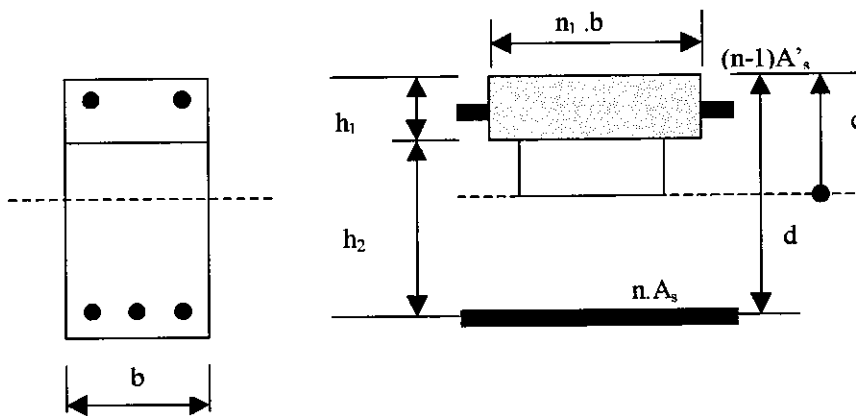
$$M_{cr} = F_r \cdot I_{ut} / Y_t \quad (5)$$

Substituting the values of parameters of eq. 3& 4-a in eq. 5. Result:

$$\begin{aligned} M_{cr} &= (4.64 \times 8.807 \times 10^8) / 184.82 = 22110421 \text{ N.mm} \\ &= 22.11 \text{ kN.m} \end{aligned} \quad (5-a)$$

C.2. Calculation of yielding moment (M_y).

C.2.1. Calculation of neutral axis for transformed, cracked section.



a) original, untransformed section b) transformed, cracked section

Fig. C.2. Geometry of transformed, cracked section

From the geometry of the section the neutral axis for transformed, cracked section can be calculated as follow:

$$n_1 b h_1 (c - h_1/2) + b(c - h_1)((c - h_1)/2) + A'_s (n - 1)(c - d') = n A_s (d - c) \quad (6)$$

$$(bc^2)/2 + (n_1 b h_1 h_1 - b h_1 + n A'_s - A'_s + n A_s) c - (n - 1) b h_1^2/2 - (n - 1) A'_s d' - n A_s d = 0 \quad (6-a)$$

substituting the known parameters as shown previously, it follows

$$c^2 + 32.74 c - 7102.07 = 0 \quad (6-b)$$

Solving eq.(6-b) leads to :

$$c = 70 \text{ mm} \quad (6-c)$$

C.2.2. Calculation of yielding moment (M_y)

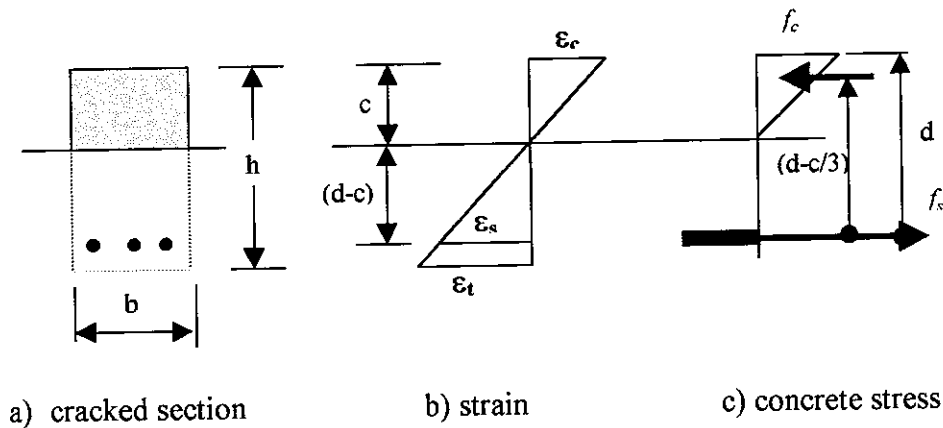


Fig.C.2.2. Cracked section

$$\varepsilon_c = \varepsilon_s c / (d - c) = (.0022 \times 70) / (222) = 0.000694$$

$$f'_c = \varepsilon_c E_c = 0.000694 \times 33200 = 23 \text{ kN}$$

$$f'_c / f_c = 23 / 44 = 0.52$$

The compressive stress in the concrete at the extreme fiber is obtained almost equal to $0.5 f'_c$. In such a case according to ACI code ^[1], the linear-elastic behavior can be assumed for the concrete. Hence, the stress

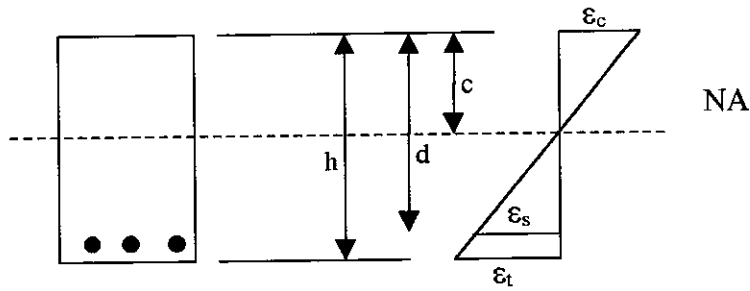
distribution can be approximated as triangular. The yielding moment (M_y) then can be found as:

$$M_y = A_s F_y (d - c/3) \quad (7)$$

Substitute the value of “c” as obtained from eq.6-c :

$$\begin{aligned} M_y &= 515 \times 440 (292 - 70/3) = 60879867 \text{ N.mm} \\ &= 60.88 \text{ kN.m} \quad (7-a) \end{aligned}$$

C.3. Calculation of strains (ϵ_c), (ϵ_t) and (ϵ_s) at a given moment.



a) Concrete section

b) strains

Fig C.3.1 Strain distribution along the concrete section.

- Calculation of compression strain (ϵ_c) at a given moment in elastic range.

$$\phi = M / Ec \cdot I_{ut} = \epsilon_c / c \quad (8)$$

$$\epsilon_c = c \cdot M / Ec \cdot I_{ut} \quad (8-a)$$

Where ϕ is the curvature, M - is the applied moment, I_{cr} is the moment of inertia of the transformed, cracked section and c is the depth of concrete block measured from the extreme compression fiber to the neutral axis.

- Calculation of moment of inertia (I_{cr}) for transformed, cracked section.

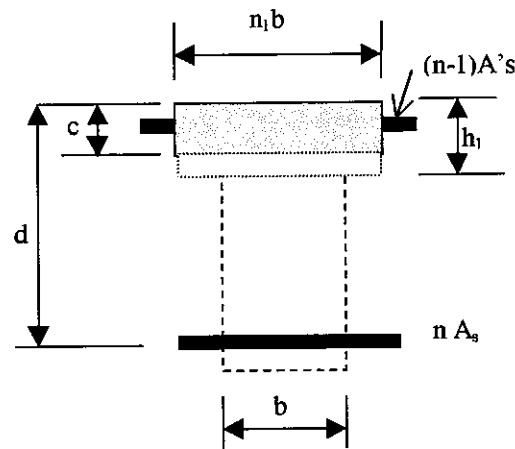


Fig.C.3.3. Geometry of transformed, cracked section

$$I_{cr} = (n_1 b c^3)/12 + n_1 b c (c/2)^2 + A'_s (n-1) (c-d')^2 + n A_s (d-c)^2 \quad (9)$$

Substituting for the values of the parameters results in

$$I_{cr} = 1.836 \times 10^8 \quad \text{mm}^4 \quad (9-a)$$

For given " M "=10 kN.m , " c_1 "= 157.6 mm and " I_{ut} "=8.807x10⁸ mm⁴ as calculated in previous example and substituting in eq.8-a.in which:

$$\begin{aligned} \epsilon_c &= c_1 \cdot M / E c \cdot I_{ut} = 165.18 \times 10 \times 10^6 / 3.32 \times 10^4 \times 8.807 \times 10^8 \\ &= 56.49 \times 10^{-6} \text{ millistrain} \end{aligned} \quad (10)$$

-Calculating of tension strain (ϵ_t)

from the triangle similarity *fig. C.3.1-b*. One can write the followings :

$$\epsilon_c/c_1 = \epsilon_t/(h-c_1) \quad (11)$$

$$\epsilon_t = \epsilon_c(h-c_1)/c_1 \quad (11-a)$$

Substitute for the value of ϵ_c as calculated from eq.10

$$\epsilon_t = 56.49 \times 10^{-6} (350 - 165.18) / 165.18 = 63.2 \times 10^{-6} \text{ millistrain}$$

-Calculation of steel strain (ϵ_s)

From the triangle similarity *fig. C.3.1-b*. One can write the followings:

$$\epsilon_c/c_1 = \epsilon_s/(d-c_1) \quad (12)$$

$$\epsilon_s = \epsilon_c(d-c_1)/c_1 \quad (12-a)$$

Substitute for the value of " ϵ_c " as calculated from eq.10

$$\epsilon_s = 56.49 \times 10^{-6} (292 - 165.18) / 165.18 = 43.4 \times 10^{-6} \text{ millistrain}$$

-Calculation of curvature (ϕ)

$$\phi = \epsilon_c/c = \epsilon_s/(d-c) \quad (13)$$

$$\phi = (56.49 \times 10^{-6}) / (70) = 8.07 \times 10^{-7} \text{ mm/mm} \quad (13-a)$$