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Structural Behaviour of Concrete Beams Reinforced with Polymer Grids and High Strength Steel

By

**Ali A. Elgabroni
B.Sc. (Civil Engineering)**

**A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of**

MASTER OF ENGINEERING

**DEPARTMENT OF CIVIL ENGINEERING
CARLETON UNIVERSITY
OTTAWA, CANADA
DECEMBER, 1989.**

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with Polymer Grids and High Strength Steel"**

submitted by

Ali A. Elgabroni

in partial fulfilment of the requirements for the degree of

Master of Engineering

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January, 1990

Abstract

The research work presented in this thesis was designed to provide a basic understanding of the structural behaviour of concrete beams reinforced with ultra high tensile strength Tensar polymer grids (*UX - 1600*) either as primary reinforcement or as secondary reinforcement in conjunction with high strength steel. The primary objectives of the test program were to study the effectiveness of polymer grids in controlling the cracks in concrete beams as well as the effect of grid reinforcement on the ductility of such beams. The experimental program included testing of thirteen full scale rectangular concrete beams $150 \times 300 \times 2200$ mm in size under monotonic repeated vertical loads.

Test results showed that the use of polymer grids as a secondary reinforcement in concrete beams imparts a marginal increase in the ultimate strength, but it increases their ductility significantly. The ductility increased by 75% over that in the conventional steel reinforced beams.

The use of the polymer grids also provides a better crack control. Both crack spacings and widths decrease in Tensar and steel reinforced beams significantly, when compared to those of conventional reinforced beams. It should be noted that with the introduction of high tensile steel reinforcing bars, the problem of crack control has become even more critical because of the high service loads.

Beams reinforced with polymer grids as a main reinforcement had low strength, but they showed good recovery after unloading. Results showed that a crack width of 15 mm decreased to 4 mm after 5 minutes unloading.

A conceptual model to explain the increase in ductility provided by polymer grids used as a secondary reinforcement in conjunction with steel reinforcement is presented.

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Table of Contents

Abstract	ii
Acknowledgements	iii
List of Figures	viii
List of Tables	xiii
Notation	xv
1 Introduction	1
1.1 General	1
1.2 Literature Review	3
1.3 Objectives and Scope	25
2 Experimental Program	27
2.1 General	27
2.2 Test Specimens	28
2.3 Material Properties	47
2.3.1 Concrete	47
2.3.2 Steel Reinforcement	52
2.3.3 Tensar Grids	52
2.4 Loading Apparatus	64
2.5 Test Set-up	65
2.6 Instrumentation and Data Acquisition System	67
2.6.1 Strain Guages	67
2.6.2 LVDTs	71
2.6.3 Crack Measurements	72
2.6.4 Data Acquisition System	72
2.7 Test Procedure	75
3 Observed Behaviour and Test Results	80
3.1 Introduction	80
3.2 General Observed Behaviour	80

3.3	Set 1 - Steel Reinforced Concrete Beams	
	- SRC beams (B1 and B8)	82
3.3.1	General	82
3.3.2	Observed Behaviour	85
3.4	Set 2 - Steel and Tensar Reinforced Concrete beams	
	- STRC beams (B3 and B4)	90
3.4.1	General	90
3.4.2	Observed Behaviour	95
3.5	Set 3 - Steel and Tensar Reinforced Concrete beams	
	- STRC beams (B5 and B6)	103
3.5.1	General	103
3.5.2	Observed Behaviour	110
3.6	Set 4 - Steel and Tensar Reinforced Concrete beams	
	- STRC beams (B7)	116
3.6.1	General	116
3.6.2	Observed Behaviour	122
3.7	Set 5 - Steel and Tensar Reinforced Concrete beams	
	- STRC beams (B9 and B10)	130
3.7.1	General	130
3.7.2	Observed Behaviour	131
3.8	Set 6 - Tensar Reinforced Concrete beams	
	- TRC beams (B11 and B12)	139
3.8.1	General	139
3.8.2	Observed Behaviour	146
3.9	Set 7 - Tensar Reinforced Concrete beams	
	- TRC beams (B13 and B14)	154
3.9.1	General	154
3.9.2	Observed Behaviour	155
3.10	Strain Readings	161
	3.10.1 Strain Readings in SRC and STRC Beams	161
	3.10.2 Strain Readings in TRC Beams	163
3.11	Summary of the Test Results	163
	3.11.1 Loads and Deflections	163
	3.11.2 Cracks in SRC and STRC Beams	163
	3.11.2.1 Cracks Number	163
	3.11.2.2 Maximum Crack Width	163
4	Discussion of Test Results	167
4.1	Introduction	167

4.2	Deflections for SRC and STRC beams	167
4.2.1	General	167
4.2.2	Deflections at first crack	168
4.2.3	Deflections at yield loads	173
4.2.4	Deflections at ultimate loads	175
4.2.5	Deflections at failure loads	177
4.2.6	Ductility	180
4.2.7	Strain readings	182
4.2.8	Spread of plasticity	182
4.2.9	Conclusions	183
4.3	Deflections for TRC beams	184
4.3.1	General	184
4.3.2	Deflections at first crack	185
4.3.3	Deflections at ultimate loads	187
4.3.4	Deflections at failure loads	189
4.3.5	Ductility	190
4.3.6	Strain readings	191
4.3.7	Conclusions	191
4.4	Cracking behaviour of SRC and STRC beams	192
4.4.1	General	192
4.4.2	Number of cracks	192
4.4.3	Maximum crack width	193
4.4.4	Conclusions	194
4.5	Cracking behaviour of TRC beams	194
5	A Conceptual Model for Increased Ductility in STRC Beams	198
5.1	Introduction	198
5.2	The State of the Stresses and Cracks in the Surrounding Concrete around the Deformed Steel Bar	199
5.3	Effect of Tensar Grids on the Ductility of STRC Beams	204
5.3.1	Bond Stresses	204
5.3.2	Bond Failure and Failure Mechanisms	217
5.3.3	Parameters Influencing the Bond Stress and Bond Failure in STRC Beams	222
5.3.4	Conclusions	223
6	Summary and Conclusions	225

6.1 Summary	225
6.2 Conclusions	226
6.3 Recommendations for Future Work	228
References	230

List of Figures

Figure	Page
1.1 Experimental Moment-Rotation Curves for Reinforced Concrete Beams, [9]	4
1.2 Variations of Curvature Ductility Factor ϕ_u/ϕ_y for Reinforced Concrete Beams with Unconfined Concrete, [55]	6
1.3 Moment-Curvature Diagrams, [66]	7
1.4 Comparison of Reinforcements-Single Loads, [68]	9
1.5 Deflection and Cracking Characteristics of Reinforced Beams with Fibre Concrete in the Compression Zone and Ordinary Concrete in the Tension Zone, [75]	11
1.6 Deflection and Cracking Characteristics of Beam with High Strength Steel (700 MPa) and Fibre Concrete, [75]	12
1.7 Typical Load-Deflection Curves of Beams without and with Fibres, [73] and [74]	15
1.8 Effect of Increasing Fibre Content on the Crack Width, [62]	17
1.9 Tensar Geogrid SS1 (Courtesy TENSAR Corporation	21
1.10 Increased Flexural Cracking with Increased Polymer Grids, [76]	22
1.11 Typical Load-Deflection Behaviour, [76])	23
2.1 Details of Beam Dimension	30
2.2 Reinforcement Details for Beam B1 (Set 1)	31
2.3 Reinforcement Details for Beam B8 (Set 1)	32
2.4 Reinforcement Details for Set 2 (Beam B3 and Beam B4)	33
2.5 Reinforcement Details for Set 3 (Beam B5 and Beam B6)	34
2.6 Reinforcement Details for Set 4 (Beam B7)	35
2.7 Reinforcement Details for Set 5 (Beam B9 and Beam B10)	36
2.8 Reinforcement Details for Set 6 (Beam B11 and Beam B12)	37
2.9 Reinforcement Details for Set 7 (Beam B13 and Beam B14)	38

2.10	Locations of the Plastic Chairs	41
2.11	Locations of Fiber Glass Pipes	42
2.12	Beam B1 Before Casting	43
2.13	Beam B9 Before Casting	44
2.14	Beam B5 Before Casting	45
2.15	Beam B14 Before Casting	46
2.16	Test Set-up for Uniaxial Compression Test	50
2.17	Stress-Strain Curve for Concrete in Compression after 70 Days	51
2.18	Stress-Strain Curve for Deformed Steel Bars	53
2.19	Stress-Strain Curve for Plain Steel Bars	54
2.20	Dimensions of Tensar Grid (UX-1600)	55
2.21	Tensile Load vs. Strain for Tensar Grid (UX 1600) (Tensar Corporation)	59
2.22	Close-up for End Clamps Used for Tensile Test of Tensar	60
2.23	Test Set-up for Tensile Test of Tensar	61
2.24	Tensile Load vs. Strain for Tensar Grid (UX 1600) (Carleton University)n	62
2.25	A Complete Tensar Grid Used in the Test	63
2.26	Four Layers Tensar Grid Connected together	63
2.27	Front View of Testing Frame	66
2.28	Test Set-up	68
2.29	Locations of the Strain Gauges on the Beam	70
2.30	Locations of the LVDTs	73
2.31	Data Acquisition System	74
2.32	Load-Time Curve for SRC and STRC Beams	76
2.33	Load-Time Curve for TRC Beams	77
2.34	: a) Beam B7 at a Total Load of 25 kN b) Beam B7 after Failure	79
3.1	Close-up of the Midspan Section after Failure for Beam B1	83
3.2	Close-up of the Bottom of Midspan Section after Failure for Beam B8	84
3.3	Load vs. Midspan Deflection for Beam B1	87
3.4	Beam B1 after Failure	88
3.5	Maximum Crack Width at Different Cycles for Beam B1	89

3.6	Number of Cracks at Different Cycles for Beam B1	89
3.7	Load vs. Midspan Deflection for Beam B8	91
3.8	Development of Crack Patterns for Beam B8	92
3.9	Maximum Crack Width at Different Cycles for Beam B8	94
3.10	Number of Cracks at Different Cycles for Beam B8	94
3.11	Load vs. Midspan Deflection for Beam B3	97
3.12	Development of Crack Patterns for Beam B3	98
3.13	Maximum Crack Width at Different Cycles for Beam B3	101
3.14	Number of Cracks at Different Cycles for Beam B3	101
3.15	Load vs. Midspan Deflection for Beam B4	104
3.16	Development of Crack Patterns for Beam B4	105
3.17	Close-up of the Middle for Beam B4	108
3.18	Maximum Crack Width at Different Cycles for Beam B4	109
3.19	Number of Cracks at Different Cycles for Beam B4	109
3.20	Load vs. Midspan Deflection for Beam B5	112
3.21	Beam B5 after Failure	113
3.22	Close-up of the Midspan Section after Failure, Beam B5	114
3.23	Number of Cracks at Different Cycles for Beam B5	115
3.24	Load vs. Midspan Deflection for Beam B6	117
3.25	Development of Crack Patterns for Beam B6	118
3.26	Maximum Crack Width at Different Cycles for Beam B6	121
3.27	Number of Cracks at Different Cycles for Beam B6	121
3.28	Concrete Cover at Tension Side. a) Beam B1 b) Beam B7	124
3.29	Load vs. Midspan Deflection for Beam B7	125
3.30	Development of Crack Patterns for Beam B7	126
3.31	Maximum Crack Width at Different Cycles for Beam B7	129
3.32	Number of Cracks at Different Cycles for Beam B7	129
3.33	Close-up of the Midspan of Beam B9	133
3.34	Load vs. Midspan Deflection for Beam B9	134
3.35	Development of Crack Patterns for Beam B9	135
3.36	Maximum Crack Width at Different Cycles for Beam B9	138

3.37	Number of Cracks at Different Cycles for Beam B9	138
3.38	Close-up of the Midspan Section after Failure for Beam B10	140
3.39	Load vs. Midspan Deflection for Beam B10	141
3.40	Development of Crack Patterns for Beam B10	142
3.41	Number of Cracks at Different Cycles for Beam B10	145
3.42	Load vs. Midspan Deflection for Beam B11	148
3.43	Beam B11 before Failure	149
3.44	Close-up of Cracked Section of Beam B11 after Failure	149
3.45	Close-up of Midspan Section of Beam B12 after Unloading and after Unloading by 5 Minutes	150
3.46	Load vs. Midspan Deflection for Beam B12	151
3.47	Development of Crack Patterns for Beam B12	152
3.48	Load vs. Midspan Deflection for Beam B13	157
3.49	Beam B13 During Unloading Stage	158
3.50	Load vs. Midspan Deflection for Beam B14	159
3.51	Development of Crack Patterns for Beam B14	160
3.52	Strain Profile for SRC and STRC Beams	162
4.1	Load Deflection Curves for Beams in Set 1 and Set 2	169
4.2	Load Deflection Curves for Beams in Set 1 and Set 3	170
4.3	Load Deflection Curves for Beams in Set 1 and Set 4	171
4.4	Load Deflection Curves for Beams in Set 1 and Set 5	172
4.5	Load Deflection Curves for Beams in Set 6 and Set 7	186
4.6	Number of Cracks at Different Cycles for SRC and STRC Beams	196
4.7	Maximum Crack Width at Different Cycles for SRC and STRC Beams	197
5.1	Pull out Test on Deformed Bar, after Goto [29]	201
5.2	Deformation of Concrete Around Ribbed Reinforcing Bar (Schematic Diagram), after Goto [29]	202
5.3	Schematic Representation of How Bond Forces Balanced by Tensile Stress Rings in Concrete in Anchorage Zone, after Goto [79]	203

5.4	Section through Part of the Tensile Zone in A Reinforced Concrete Beam. The Dimensions of the Concrete Ring are Determined by the Lowest Value of C_x and C_y , after Tephers [79]	203
5.5	Distribution of Tensile Ring Stresses a) Elastic b) Partly Cracked Elastic, and c) Plastic, after Tephers [79]	205
5.6	Effect of Thickness of Concrete Cover upon Bond Capacity of the Pull-out Specimens on Occurrence of Longitudinal Cracking in the Concrete Cover, after Tephers [79]	206
5.7	Concrete Ring in Beam B1	208
5.8	Concrete Ring in Beams B3 and B4	208
5.9	Concrete Ring in Beams B5 and B6	209
5.10	Concrete Ring in Beam B7	209
5.11	Concrete Ring in Beams B9 and B10	210
5.12	Distribution of Stresses around the Steel Bar	211
5.13	Bearing Stresses on the Interlocked Concrete in A Polymer Grid Stress under Tension	214
5.14	Contraction of A Polymer Grid Stressed under Tension	215
5.15	Close-up of the Bottom of Midspan Section of Beam B10 shows the Bond Failure	219
5.16	Beam Action Mechanism	220
5.17	Arch Action Mechanism	221

List of Tables

Table	Page
1.1 Details of Test Beams, [73] and [74]	14
1.2 Details of Test Beams, [53]	20
1.3 Ductility of SFRC Beams, [53]	20
1.4 Flexural Load Test Results on Tensar Composites, [76]	23
2.1 Details of Test Beams	39
2.2 Concrete Compressive Strength after 70 Days	49
2.3 Tensile Strength after 70 days	49
2.4 Properties of Tensar Grid (<i>UX</i> – 1600)	56
2.5 Locations and Number of the Strain Gauges	71
3.1 Loads and deflections for the beams	164
3.2 Cracks Number for SRC and STRC Beams	165
3.3 Maximum Crack Width for SRC and STRC Beams at Different Cycles	166
4.1 Comparison between Deflections for SRC and STRC Beams at First Crack	174
4.2 Comparison between Deflections of SRC and STRC Beams at Yield Loads	175
4.3 Comparison between Deflections for SRC and STRC Beams at Ultimate Loads	176
4.4 Comparison between Deflections for SRC and STRC Beams at Failure	178
4.5 Comparison between Ductilities for SRC and STRC Beams	181
4.6 Comparison between Deflections for TRC Beams at First Visible Crack	187
4.7 Comparison between Deflections for TRC Beams at Ultimate Loads	188
4.8 Comparison between Deflections for TRC Beams at Failure Loads	189
4.9 Comparison between Ductilities for TRC Beams	190

5.1 Effective Areas of Concrete Rings and theoretical Bearing Bond Stresses at The Middle of SRC and STRC Beams	212
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Notation

A_s	- Area of Longitudinal Steel in Tension Zone.
A'_s	- Area of Longitudinal Steel in Compression Zone.
C_x	- Concrete Cover in X direction.
C_y	- Concrete Cover in Y direction.
d	- Steel Bar Diameter.
D	- Concrete Cylinder Diameter.
E_c	- Modulus of Elasticity of Plain Concrete.
E_s	- Modulus of Elasticity of Steel.
f_b	- Bearing Bond Stress.
f_{bc}	- Bearing Stress Developed against Junctions of Polymer Grids.
f_{br}	- Bearing Stress Developed against Ribs of Polymer Grids..
f'_c	- Concrete Cylinder Stress.
f_{cbc}	- Bond Stress when Concrete Cover Cracks.
f_r	- Tensile Strength of the Concrete.
f_{rcb}	- Radial Component of Bearing Bond Stress.
f_s	- Steel Stress.
f_t	- Concrete Cylinder Splitting Strength.
f_{tcb}	- Tangential Component of Bearing Bond Stress.
f_y	- Yield Strength of Longitudinal Steel.
L	- Concrete Cylinder Length.
LW	- Light Weight Mortar Matrix.

LVDTs	- Linear Variable Differential Transformers.
P_u	- Ultimate load on Concrete Cylinder.
OW	- Normal Weight Mortar Matrix.
M_y	- Bending Moment at Yielding.
M_u	- Bending Moment at Ultimate.
SRC	- Steel Reinforced Concrete.
STRC	- Steel and Tensar Reinforced Concrete.
SWG	- Steel Welded Grids.
TRC	- Tensar Reinforced Concrete.
UX	- Uniaxial.
ϵ_c	- Strain in Plain Concrete.
ϵ_{cu}	- Ultimate Strain in Plain Concrete.
ϵ_o	- Strain Corresponding to Maximum Stress in Plain Concrete.
ϵ_s	- Strain in Longitudinal Steel.
Δ	- Deflection.
Δ_{max}	- Maximum Deflection.
Δ_u	- Ultimate Deflection.
Δ_y	- Yield Deflection.
μ	- Ductility index.
ϕ	- Curvature.
ϕ_y	- Curvature at Yield.
ϕ_u	- Curvature at Ultimate.
σ_t	- Tensile Splitting Stress.

Chapter 1

INTRODUCTION

1.1 General

During the last few decades, structural engineers have been paying increasing attention to the behaviour of structures subjected to earthquake excitation. In view of the tremendous potential of earthquakes in causing loss of life and damage to property, it is important that research be carried out to establish safe and reliable design techniques for structures situated in seismically active regions.

The importance of ductility and associated energy absorption capacity of a structure in resisting earthquake has long been recognized. The need for ductility was emphasized in recent years in light of the damage sustained by ductile buildings subjected to severe earthquake. In concrete structures, it is normal to provide a strong column-weak beam framing. In such a frame, if yielding has commenced at the critical sections of all beams before the columns reach yield curvature, further deformations in beams take place at constant beam-end moments due to plastic rotation occurring in the beam hinges. It is advantageous to design the floor beams so that they become the major energy dissipating devices and a large amounts of energy is dissipated in such beams before hinging of the columns commences. This ensures that the columns generally remain in the elastic range and the structure enjoys a higher degree of protection against permanent damage. Yielding in columns may lead to a permanent misalignment of the building which makes it impossible to repair. It is considerably easier to repair damaged beams.

Ductile behaviour means the ability to undergo large inelastic deformations with little loss in the strength. The available ductility of reinforced concrete section is mainly dependent on the amount of longitudinal tension and compression steel reinforcement, the concrete and steel strengths and the amount of transverse steel for concrete confinement and restraint against buckling of bars.

In addition to ductility, the control of crack width in reinforced concrete is an important design consideration. Cracks can develop in a reinforced concrete structure as the internal stresses exceed certain limits. Crack widths have to be limited to acceptable limits for a variety of reasons such as control of deflection, durability, corrosion prevention and maintenance of integrity and appearance of the structure. It should be noted that with the introduction of high tensile steel reinforcing bars, the problem of the control of cracking has become even more critical because of the high service loads.

In recent years, research on reinforced concrete has focussed on the development of new construction materials. An outcome of this research is the use of fibers of steel, glass, polypropylene, polyethylene and others for reinforcing of concrete. These fibers are mixed with the fresh concrete before casting. Many investigations had shown that the incorporation of these fibers in concrete improves several properties of reinforced concrete beams including flexural strength, ductility and crack control.

Recently, the development of ultra high strength polymer grids has opened up the possibility of using them as reinforcement in concrete. One of the main characteristics of polymer grids is the high elongation of the grids at failure. The author believed that the use of polymer grids as a secondary reinforcement in conjunction with high strength steel reinforcement in tension, which has a low elongation at failure should improve the ductile behaviour of reinforced concrete beams. Also, the mechanical interlocking provided by the polymer grid should reduce crack widths and spacing. An experimental investigation of the effect of supplementary polymer grid reinforcement on the ductility and cracking characteristics of concrete beams

reinforced by high-strength steel was undertaken to verify the potential of such grid reinforcement. To the author's knowledge, no other experiment or investigation of a similar nature has been carried out before.

1.2 Literature Review

Previous research on ductility capacity and energy absorption capacity has been concentrated on improving the ductility of concrete beams by improving the ductility of concrete in the compression zone. Ductility of concrete in compression zone can be improved either by using spirals or conventional stirrups, or by providing longitudinal compression reinforcement, or by both of these means.

The effectiveness of helical or closely spaced stirrups in the compression zone of concrete beams was tested by J. Romualid and G. Batson [64], A. Mattock [44] in 1964, H. E. Roy and M. A. Sozen [66] in 1964, G. Base and J. Read [9] in 1965, W. G. Corley [18] in 1966, M. Soliman and C. Yu [70] in 1967, H. G. Geymayer [28] in 1968, E. Nawy et al. [50], D. Kent and R. Park [34] in 1971, and others. They have indicated qualitatively the beneficial effect of confinement by transverse reinforcement and helical confining on the ductility of reinforced concrete flexural members. As an example, Figure 1.1 shows experimental moment-rotation curves from a series of beams tested by G. Base and J. Read [9]. They stated that "the beams had a rectangular section 152 mm wide by 279 mm deep and were loaded by a single point load at midspan over a simply supported span of 3050 mm. Figure 1.1a gives the moment-rotation curves obtained for beams having a tension steel content of approximately one-half the balanced failure value. Figure 1.1b shows the moment-rotation curves obtained for beams designed for a balanced failure. The helics and/or the closely spaced stirrups are shown to give a substantial increase in ductility. With curves obtained for beams that had a tension steel content greater than the balanced value (Figure 1.1c), not only is an increase in ductility obtained

(c)

Figure 1.1: Experimental Moment-Rotation Curves for Reinforced Concrete Beams, [9]

by using transverse reinforcement apparent, there is also an increase in strength due to the large influence of the enhanced compressive strength of the concrete on this type of failure”.

The effect of concrete and steel properties on the ductility of unconfined concrete beams was discussed by R. Park and T. Paulay [54] in 1975 and R. Park and D. Ruitong [55] in 1988. The following conclusions were reported:

- An increase in the compression steel content increases the ductility.
- An increase in the concrete strength increases the ductility.
- An increase in the extreme fiber concrete strain at ultimate load increases the ductility. Figure 1.2 shows the effect of compression steel and concrete strength on the ductility factor (ϕ_u/ϕ_y) for reinforced concrete beams with unconfined concrete reported by R. Park and D. Ruitong [55].

Figure 1.3 shows in the form of moment curvature curves some of the results obtained in a series of tests carried out by H. E. Roy and M. A. Sozen [66] in 1964 on beams subjected to pure bending moment in their center parts. All beams were made from concrete having the same concrete strength. The longitudinal reinforcement consisted of ASTM-A432 bars with a well defined yield point of about 427 MPa. It was found that the addition of 3/16 in. ties spaced at 1-1/2 in. increased the ductility considerably. A further improvement in the overall inelastic behaviour of the member was obtained when the 3/16 in. ties were supplemented by two #3 bars placed to serve as compressive reinforcement. In the case of beams without transverse and compression reinforcement, inelastic rotation was concentrated in one small region whose length was smaller than the effective depth. When ties were used, the inelastic rotation spread over a longer length but most of the damage was still concentrated in a region of length approximately equal to the effective depth. On the other hand, when two #3 bars were used as compressive reinforcement

Figure 1.2: Variations of Curvature Ductility Factor ϕ_u / ϕ_y for Reinforced Concrete Beams with Unconfined Concrete, [55]

Figure 1.3: Moment-Curvature Diagrams, [66]

together with ties, the inelastic deformations spread over the whole region subjected to constant moment. Also, in the case where ties were spaced so closely that they prevented the buckling of the compression steel, no visible damage was observed in the confined concrete.

Considerable research has been carried out over the last three decades on concrete reinforced with fibers (steel, glass, asbestos, carbon, nylon, polypropylene, polyethylene, syenthetic fibers and others) to verify whether fiber reinforcement improved the cracking behaviour, ductility and energy absorption capacity of reinforced concrete beams.

In 1970, S. Shah and P. Rangan [68] studied the relative efficiency of conventional stirrups and steel fibers in improving the ductility of concrete beams. Twenty four simply supported beams measuring $(50 \times 75 \times 900)$ mm were tested under two-point loading. The beams were designed to ensure that failure took place in the central constant moment zone. This zone contained closed stirrups of varying volumes, fibers of different amounts, or compression longitudinal reinforcement of different volumes. The following conclusions were drawn from the results of the tests:

a: For over-reinforced beams:

- The addition of stirrups, fibers, or compression reinforcement did not significantly influence the load-deflection curves up to the maximum load of these beams
- The addition of stirrups, fibers, and compression reinforcement each increased ductility of over-reinforced beams. For example, Figure 1.4 indicates that the ductility of compression concrete $(\epsilon_{cu} - \epsilon_{ci})/\epsilon_{ci}$ is increased 10 times by the addition of 0.9% by volume of stirrups, while the same volume of fibers increases ductility 4.5 times. Compression steel increases this ratio two times compared to beams without any ductility reinforcement.

Figure 1.4: Comparison of Reinforcements-Single Loading, [68]

- The addition of stirrups and fibers retards the internal crack growth of compression concrete itself but the addition of compression steel influences crack growth to a much lesser degree.

b: For under-reinforced beams:

- The addition of stirrups or fibers does not change either the overall behaviour of beams or the behaviour of compression concrete.
- No volume dilation of the compression zone was observed and the concrete compressive strains were considerably lower than those for over-reinforced beams.

In 1975, R. N. Swamy and K. Al-Noori [75] showed that the presence of steel fibers enables high strength steel with yield strength of 700 MPa to be used with both crack width and deflection being controlled to within acceptable limits. Figure 1.5 and Figure 1.6 show load-deflection relationship and the maximum crack width for two beams with different kind of steel reinforcement. The first beam had steel fiber mixed concrete in the compression zone and was reinforced with steel having a yield strength of 410 MPa. The second beam was reinforced by steel fiber concrete in the tension zone and was reinforced with steel having a yield strength of 700 MPa. As shown in Figures 1.5 and 1.6, the beams with high yield steel (700 MPa) showed the same degree of ductility as beams with deformed bars (410 MPa). The presence of fibers in the tension zone enabled the beams to develop much higher rotation at failure.

In 1976, C. Henager [30], tested four full scale size beams measuring $203 \times 305 \times 3950$ mm to study the flexural and cracking behaviour of fiber reinforced beams. It was concluded that the use of steel fibrous concrete in reinforced concrete beams increased their ultimate flexure strength by 25%. Use of fibrous concrete also resulted in a reduction in both crack width and crack spacing, and delayed occurrence of the first cracks to loads 1.33 to 1.67 times greater than those for conventional beams.

Figure 1.5: Deflection and Cracking Characteristics of Reinforced Beam with Fibre Concrete in the Compression Zone and Ordinary Concrete in the Tension Zone, [75]

**Figure 1.6: Deflection and Cracking Characteristics of Beam
with High Strength Steel (700 MPa) and
Fibre Concrete, [75]**

In 1979 and 1981, R. N. Swamy, S. Al-Ta'an, and S. Ali [73] and [74], tested fifteen reinforced concrete beams of size $130 \times 203 \times 2500$ mm. The beams were reinforced with hot rolled steel reinforcement with a specified minimum yield strength of 460 MPa and cold-worked high tensile steel with a specified minimum yield strength of 617 MPa. The concrete used in the test was normal concrete and fiber concrete with different volumes of fibers. Table 1.1 shows the details of the beams tested. Typical load deflection curves for beams with fibers and without fibers are illustrated in Figure 1.7. It was concluded that:

- The cracks were more closely spaced in all the fiber concrete beams at all stages of loading and this resulted in substantial reductions in crack width at all stages of loading.
- The deflection of beams reinforced with steel of 617 MPa specified minimum yield strength were of the same order as those of identical beams with steel of 460 MPa specified minimum yield strength when the volume of steel fiber in concrete was 0.5 percent.
- Fiber concrete beams showed significant inelastic deformation and ductility at failure. The concrete strain at crushing varied from 0.0052 to 0.0066.
- Fiber reinforcement not only gave a better distribution of the tensile cracks in the beams but also minimized the penetration of the shear cracks into the compression zone.
- The addition of steel fibers to over-reinforced concrete beams increased the ductility of the concrete in the compression zone and transferred the brittle mode of failure to ductile one.

In 1986, O. Ibrahim and A. Luxmore [32], tested twenty steel fiber reinforced concrete beams to study the control of cracks in reinforced concrete beams. It was concluded that steel fibers can reduce the maximum crack widths by 40%.

Table 1.1: Details of Test Beams, [73], [74]

Beam no.	DR10	DR11	DR12	DR13	DR14	DR20	DR21	DR22	DR23	DR24	DR30	DR31	DR32	DR33	DR34
Type of concrete	A	B	B	C	C	A	B	B	C	C	A	B	B	C	C
Fiber percentage	0.0	0.5	1.0	0.5	1.0	0.0	0.5	1.0	0.5	1.0	0.0	0.5	1.0	0.5	1.0
Cube strength, N/mm ²	36.97 ¹	39.49 ²	40.0 ³	37.50 ⁴	39.24 ⁵	38.88 ¹	37.3 ²	40.20 ³	38.14 ⁴	38.04 ⁵	38.65 ¹	39.03 ²	40.88 ³	37.00 ⁴	37.80 ⁵
Modulus of rupture, N/mm ²	3.48 ¹	4.72 ²	6.30 ³	4.00 ⁴	4.55 ⁵	4.23 ¹	4.38 ²	5.91 ³	4.27 ⁴	4.29 ⁵	4.23 ¹	4.99 ²	5.90 ³	3.67 ⁴	4.24 ⁵
Effective depth, mm	176	176	176	176	176	174	174	174	174	174	176	176	176	176	176
Tension reinforcement	2-12 ⁶	2-12 ⁶	2-12 ⁶	2-12 ⁶	2-12 ⁶	2-16 ⁶	2-16 ⁶	2-16 ⁶	2-16 ⁶	2-16 ⁶	2-12 ⁶	2-12 ⁶	2-12 ⁶	2-12 ⁶	2-12 ⁶
Tension reinforcement, percent	0.99	0.99	0.99	0.99	0.99	1.78	1.78	1.78	1.78	1.78	0.94	0.94	0.94	0.94	0.94

*Tortised bars $f_y = 460$ N/mm².¹High tensile steel bars $f_y = 617$ N/mm², crimped steel fibers 0.5 x 50 mm.²Plain concrete³Fiber concrete

All beams were provided with two 10 mm diameter Tortised bars in the compression zone

Overall dimensions of the beams: 130 x 203.2 mm

Span = 2250 mm.

Concrete type: A = plain concrete.

B = fibers located over whole depth of beam.

C = fibers located in effective tension zone only.

Figure 1.7: Typical Load-Deflection Curves of Beams without and with Fibres, [73] and [74]

In 1987, D. Rao, M. Kadir and K. Al-Awffa [62] tested ten beams of size $75 \times 150 \times 1850$ mm to study the influence of steel fibres on deformational behaviour. It was concluded that steel fibres arrested advancing cracks and reduced the crack widths. The improvement of crack control due to the addition of fibers was more pronounced with deformed bars than with plain bars. Figure 1.8 shows the effect of increasing fiber content on the crack width.

In 1987, H. Acliyutha and P. Sabapathi [2], tested ten beams to study the influence of steel fibers on cracking characteristics of reinforced concrete beams. The following conclusions were reported :

- The presence of fibers reduces the crack height by about 25%.
- There is a reduction of 50% to 90% in the maximum crack width at working load depending on the fiber aspect ratio.
- An increase in the fiber content and aspect ratio increased the percentage increase in service load at a specific crack width.

In 1987, R. J. Craig, J. Decker, L. Dombrowski, R. Laurencelle, and J. Federovich, of New Jersey Institute of Technology [19], investigated the increase in ductility obtained in conventional reinforced concrete beams through the inclusion of steel fibers. They tested four full-sized beams. The following conclusions were reported:

- The use of fibers helps to improve the ductility.
- The higher the tensile stresses due to the presence of fibers, the higher is the ultimate moment.
- The benefits obtained by the addition of fibers decreased with the addition of compression steel.

Figure 1.8: Effect of Increasing Fibre Content on the Crack Width, [62]

Earlier investigations on reinforced and laced reinforced concrete structure elements have shown that beam elements provided with equal compression and tension steel reinforcement exhibit vastly superior ductile characteristics at failure. In these cases, the failure was triggered by the rupture of the main tension steel and the strain in the steel at failure was generally very large. One of the major drawbacks in using higher percentages of steel levels however was the large-scale spalling of concrete in the compression zone which cause a buckling of the compression bars.

In 1987, V. S. Parameswaran, T. S. Krishnamoorthy, K. Balasubramanian, and N. Lakshmanan [53] studied the behaviour of steel fibre reinforced concrete beams having equal compression and tension steel. Their idea was that by using steel fibers in doubly-reinforced beams, it may be possible to confine the concrete in the compression zone, thereby preventing its spalling at failure stage. They tested six full scale simply supported beams of size $100 \times 200 \times 2060$ mm in flexure in two point loading test. Details of the test beams are given in Table 1.2. All the failures were caused by the rupture of tensile steel. Table 1.3 gives the yield deflection, failure deflection, failure rotation at the plastic hinge, and the ductility ratio of all the tested beams.

The following conclusions were drawn from the test program :

- The commonly observed failure by buckling of compression steel at higher percentages of reinforcement in reinforced concrete beams is not observed in steel fiber reinforced concrete beams.
- For the range of main percentage reinforcement up to 2.5%, the failure of steel fiber reinforced concrete beams is by the rupture of tensile steel.
- The failure ductility and rotation at the plastic hinge of steel fiber reinforced concrete beams is found to be significantly different from normal reinforced concrete beams.
- Steel fiber reinforced concrete beams give far higher load dispersion for

concentrated loads. The dispersion can be taken as three horizontal to one vertical up to the level of tensile steel.

A large amount of experimental work has been done on other kind of fibers such as natural fibers, glass fibers, polypropylene fibers, polyethylene fibers, synthetic fibers, metallic fibers and others. In all cases the fibers used were of small sizes and were dispersed in the concrete matrix. Use of alternative materials to supplement or replace steel bar reinforcement in concrete has not yet been tried. However, the recent development of high strength polymer grids has opened up the possibility of using them as a reinforcement in cement and concrete composites to produce a new reinforced composite material.

Very little experimental work has been carried out to study the behaviour of polymer grid reinforced cement and concrete composites. In 1984, R. N. Swamy, R. Jones and P. N. Oldroyd, of University of Sheffield (England) [76] investigated the behaviour of polymer grids (known as Tensar) in cement composites under static loads. Tensar geogrid is the proprietary name for a polymer grid manufactured by Netlon Ltd. Corporation (England). Tests were carried on six slabs specimens of size $700 \times 300 \times 20$ mm in flexure. The specimens were reinforced with increasing amount of biaxial grids (SS1). Figure 1.9 shows Tensar grid (SS1). Two types of cement matrices were used, a normal weight mortar matrix (OW) and a light weight mortar matrix (LW). For the purpose of comparison, two specimens reinforced with a welded steel mesh (SWG) of 1 mm diameter wire at 50 mm centers and another two plain cement specimens were also tested. All the specimens were loaded beyond first cracking, to deflection of 20 mm. The test results are shown in Table 1.4 and Figures 1.10 and 1.11. The following conclusions were reported :

- As the number of the polymer grid layers was increased, the first crack appeared at progressively earlier loads.

Table 1.2: Details of Test Beams, [53]

Serial No.	Beam No.	Main Reinforcement (at top and bottom)	Volume Percentage of Fibres	Transverse Steel
1	B1	2 ϕ 10	0.50	2 legged ϕ 8 @ 150mm centres
2	B2	2 ϕ 10	0.75	2 legged ϕ 8 @ 150mm centres
3	B3	2 ϕ 10	1.00	2 legged ϕ 8 @ 150mm centres
4	B4	2 ϕ 12	1.00	2 legged ϕ 8 @ 150mm centres
5	B5	2 ϕ 16	1.00	2 legged ϕ 8 @ 150mm centres
6	B6	2 ϕ 10	0.00	2 legged ϕ 8 @ 125mm centres

Table 1.3: Ductility of SFRC Beams, [53]

Serial No.	Beam No.	Yield Deflection Δ_y (mm)	Ultimate Deflection Δ_u (mm)	Ductility Ratio μ	Plastic Hinge Rotation (Degrees)
1	B1	10.4	60.1	5.87	6.76
2	B2	8.7	60.0	6.9	6.65
3	B3	8.5	73.0	8.59	8.09
4	B4	8.5	70.0	8.24	7.76
5	B5	9.0	113.0	12.56	12.52

Figure 1.9: Tensar Geogrid SS1
(Courtesy TENSAR Corporation)

Figure 1.10: Increased Flexural Cracking with Increased Polymer Content, [76]

Table 1.4: Flexural Load Test Results on Tensar Composites, [76]

SPECIMEN	PLAIN		25/25 SWG		1 LAYER SS1		4 LAYERS SS1		8 LAYERS SS1	
	OW	LW	OW	LW	OW	LW	OW	LW	OW	LW
LOAD TO 1ST CRACK KN	0.64	0.48	0.63	0.48	0.61	0.48	0.52	0.47	0.49	0.46
LOAD TO 20mm DEFL KN	-	-	-	-	0.50	0.26	0.76	0.81	1.11	0.80
GROSS STRESS AT 1ST CRACK N/mm ²	3.20	2.40	3.15	2.40	3.05	2.40	2.60	2.35	2.45	2.30
GROSS STRESS AT 20mm DEFL N/mm ²	-	-	-	-	3.50	1.30	3.80	4.05	5.55	4.00

Figure 1.11: Typical Load-Deflection Behaviour [76]

- The plain, unreinforced specimens, and the specimens with one layer of steel mesh were unable to reach 20 mm deflection, and failed to show any post-cracking ductility. On the other hand, Tensar composites were able to show remarkable post-cracking ductility. The following conclusions were reported:
- In many cases, it was impossible to fail the specimens completely, increasing load producing further large deflections, and maintaining the integrity of the test specimens.
- The initial crack spacing in flexure was dominated by the transverse rib spacing in the outermost layer of the grid.
- The crack spacing was controlled by the geometry of the reinforcing grids.
- Increasing amounts of grid reinforcement also increased the number of cracks.
- After loading and unloading the Tensar specimens, the specimens would not only recover, even after extensive cracking and deformation, but also retain their shape and integrity in a structural system.
- The energy absorption capability increased with increasing volume content of grid reinforcement.

In 1984, A. J. Watson, B. Hobbs and P. Oldroyd, of University of Sheffield (England) [81], and also in 1986, A. O. Abd El-Halim, G. T. Suter and J. Winterink, of Carleton University [1], investigated the behaviour of Tensar reinforced cement composites under dynamic loads. They tested concrete slabs, which were either reinforced with Tensar polymer grids or a steel mesh or were unreinforced under dynamic loads by using drop hammer. They concluded that the structural performance of slabs reinforced with polymer grids under dynamic loading is comparable with that of steel reinforcement but that polymer grids have potential advantages in many circumstances due to their inherent resistance to water and chemical corrosion.

1.3 Objective and Scope

The test program reported in this thesis was designed to provide full information on the structural behaviour of concrete beams reinforced with ultra high tensile strength Tensar polymer grids ($UX - 1600$) either as primary reinforcement or as secondary reinforcement in conjunction with steel. The objectives of the test program included a study of the effectiveness of polymer grids in controlling the cracks in concrete beams. This study was carried out by measuring cracks widths. The other objective was to study the effect of grid reinforcement on the ductility of beams. The experimental program included testing of thirteen full scale rectangular beams $150 \times 300 \times 2200$ mm in size. To study the effectiveness of Tensar grids ($UX - 1600$) on crack control, flexural moment and ductility of the concrete beams, the thirteen beams were classified into seven sets according to their reinforcement configuration as follows:

- Set 1: Two beams, one reinforced with steel only in compression and tension, and the other one reinforced with steel in compression and with steel and two disconnected layers of Tensar grids in tension. These beams were control beams.
- Set 2: Two beams reinforced with steel in compression and steel and two layers of Tensar grids in tension. The Tensar layers were placed above the steel.
- Set 3: Two beams reinforced with steel in compression and steel and four layers of Tensar grids in tension. The layers were placed above the steel.
- Set 4: One beam reinforced with steel in compression and steel and two layers of Tensar grids in tension. The layers were placed under the steel.
- Set 5: Two beams reinforced with steel in compression and steel and four layers of Tensar grids in tension. Two layers of Tensar grids were placed above the steel and the other two layers were placed under the steel.

Set 6: Two beams reinforced with two layers of Tensar grids in tension. No compression or shear reinforcement of any kind was used.

Set 7: Two beams reinforced with four layers of Tensar grids in tension. There was no compression or shear reinforcement.

The thesis has been organised into six chapters and the contents of each chapter are briefly as follows:

Chapter 1 presents an introduction and a review of the existing literature relevant to the present study.

The details of the experimental program, the test loading facility and the instrumentation are given in Chapter 2.

Chapter 3 is devoted to a presentation of the experimental results obtained from the tests. The results include the measured load, deflection, maximum crack width, cracks number and the variations of strains in longitudinal reinforcement and on the concrete surface.

Chapter 4 discusses the test results obtained and compares the measured loads, deflections and strains between different sets.

Chapter 5 explains a conceptual model for the test results.

The conclusions drawn from the test results and a summary of the work done are presented in Chapter 6 together with suggestions for further research in this new field of study.

Chapter 2

EXPERIMENTAL PROGRAM

2.1 General

The experimental program was designed to investigate the effect of ultra high strength Tensar polymer geogrids (*UX-1600*) reinforcement on the flexural strength, the load-deflection, cracking characteristics, and the ductility capacity of reinforced concrete beams. It included testing of thirteen simply supported beams under monotonic repeated vertical loads. Two control beams were reinforced with high-strength steel having a yield stress of 610 MPa (88.5 ksi). One of them was reinforced also by two layers of discontinuous polymer grids on the tension side of the beam. These beams were denoted as SRC beams. Seven beams were reinforced with the same high-strength steel as used in the control beams and varying number of layers of the polymer grids (*UX - 1600*) as secondary reinforcement on the tension side of the beam. These beams were denoted as STRC beams. In order to study the behaviour of beams reinforced with polymer grids only, two beams were reinforced with two layers of polymer grids and another two beams were reinforced with four layers on the tension side of the beams. These beams were termed as TRC beams. Details of the test specimens, the test set-up, instrumentation, and test procedure are described in this chapter. In addition, the materials properties and reinforcement arrangement are also reported.

2.2 Test Specimens:

Thirteen rectangular reinforced concrete beams, having a constant width of 150 mm, a constant depth of 300 mm, and a length of 2440 mm, were tested on a simply supported span of 2200 mm, center-to-center of support. Figure 2.1 illustrates the overall geometry of the specimens. Different reinforcement configurations were used in the test program. The specimens were labelled on the basis of their reinforcement configuration. All specimens had an average concrete strength of 40 MPa (determined on 6- by 12- in. cylinders at the age of 70 days). SRC beams and STRC beams were reinforced with vertical stirrups (6.00 mm diameter at 125 mm spacing) except for a length of 300 mm at the middle of the beam.

The thirteen beams are classified into seven sets as follows:

- **Set 1 (SRC beams) :** This set consists of two pilot beams, beam B1 and beam B8. Beam B1 is reinforced with equal tension and compression high-strength steel (2 No. 15). Figure 2.2 shows the reinforcement details of beam B1. Beam B8 reinforcement is similar to beam B1 but it has two layers of polymer grids placed under the tension steel. These layers were discontinued at the middle of the beam as shown in Figure 2.3. They were placed to study the effect of polymers on crack control only. Figure 2.3 shows the reinforcement details of beam B8.
- **Set 2 (STRC beams) :** This set consists of two beams (beam B3 and beam B4). The beams are reinforced with equal tension and compression high-strength steel (2 No. 15) and two layers of polymer grids placed above the tension steel as a secondary reinforcement. Figure 2.4 shows the reinforcement details of beam B3 and beam B4.
- **Set 3 (STRC beams) :** This set consists of two beams, beam B5 and

beam B6. The beams are reinforced with equal tension and compression high-strength steel (2 No. 15) and four layers of polymer grids placed above the tension steel as a secondary reinforcement. Figure 2.5 shows the reinforcement details of beam B5 and beam B6.

- **Set 4 (STRC beams) :** This set consists of one beam, beam B7. This beam is reinforced with equal tension and compression high-strength steel (2 No. 15) and two layers of polymer grids placed under the tension steel as a secondary reinforcement. Figure 2.6 shows the reinforcement details of beam B7.
- **Set 5 (STRC beams) :** This set consists of two beams, beam B9 and beam B10. The beams are reinforced with equal tension and compression high-strength steel (2 No. 15) and four layers of polymer grids, two layers above the tension steel and two layers under the tension steel as a secondary reinforcement. Figure 2.7 shows the reinforcement details of beam B9 and beam B10.
- **Set 6 (TRC beams) :** This set consists of two beams, beam B11 and beam B12. The beams are reinforced with two layers of polymer grids on the tension side only as a main reinforcement. There is no compression or shear reinforcement . Figure 2.8 shows the reinforcement details of beam B11 and beam B12.
- **Set 7 (TRC beams) :** This set consists of two beams, beam B13 and beam B14. The beams are reinforced with four layers of polymer grids in the tension side only as a main reinforcement. There is no compression or shear reinforcement . Figure 2.9 shows the reinforcement details of beam B13 and beam B14.

Table 2.1 provides a summary of the test specimens and their properties.

Beam Dimension

Cross Section

Figure 2.1: Details of Beam Dimension

Elevation

Cross Section

Figure 2.2: Reinforcement Details for Beam B1
(Set 1)

Elevation

Section A-A

Section at Midspan

**Figure 2.3: Reinforcement Details for Beam B8
(Set 1)**

Elevation

Cross Section

**Figure 2.4: Reinforcement Details for Set 2
(Beams B3 and B4)**

Elevation

Cross Section

**Figure 2.5: Reinforcement Details for Set 3
(Beams B5 and B6)**

Elevation

Cross Section

**Figure 2.6: Reinforcement Details for Set 4
(Beam B7)**

Elevation

Cross Section

**Figure 2.7: Reinforcement Details for Set 5
(Beams B9 and B10)**

Elevation

Cross Section

**Figure 2.8: Reinforcement Details for Set 6
(Beams B11 and B12)**

Elevation

Cross Section

**Figure 2.9: Reinforcement Details for Set 7
(Beams B13 and B14)**

Table 2-1: Details of Test Beams

Set No.	Beam No.	Type and Amount of Tensile Reinf.	Type and Amount of Compressive Reinf.	Vertical Reinf. (Stirrups)	Sketch of Cross Section at Midspan
1	B1 & B8	2 No. 15	2 No. 15	6 mm @ 125mm
2	B3 & B4	2 No. 15 & 2 Layers Tensar		6 mm @ 125mm
3	B5 & B6	2 No. 15 & 4 Layers Tensar	2 No. 15	6 mm @ 125mm
4	B7	2 No. 15 & 2 Layers Tensar	2 No. 15	6 mm @ 125mm
5	B9 & B10	2 No. 15 & 4 Layers Tensar	2 No. 15	6 mm @ 125mm
6	B11 & B12	2 Layers Tensar
7	B13 & B14	4 Layers Tensar

Note(1): All the steel used as reinforcement is high strength steel with yield strength of 610 MPa.

Note(2): Steel bar No. 15 is 16 mm nominal diameter.

Note(3): Stirrups are 6 mm in diameter and are spaced at 125 mm.

Strain gauges were attached to the longitudinal (tensile and compressive) reinforcement bars and the polymer grids at the middle section of the beam (see paragraph 2.6.1). For SRC beams and STRC beams, plastic chairs were wired to the reinforcement cage to provide the required bottom cover. Figure 2.10 shows the location and spacing of the plastic chairs for SRC beams and STRC beams. For providing the required bottom cover for TRC beams, fiber glass pipes with outer diameter of 13 mm and inner diameter of 9 mm were used. Fiber glass pipes were placed inside the plywood form. Threaded bars were then inserted into the fiber glass pipes and secured to the outside of the form by nuts. Following this, polymer grids were fixed to the pipes through steel wires as shown in Figure 2.11. When the forms were demolded, threaded bars were removed but fiber glass pipes were left in the concrete. Figure 2.11 shows the location, spacing and the method of fixing the polymer grids layers.

For SRC beams and STRC beams, the reinforcement cage was carefully lowered into the plywood formwork. The alignment was checked and corrected, if required. Concrete was placed in the forms in layers. Compaction of each layer was effected by internal vibration using an electrically operated vibrator and by hand tamping. Side covers and end covers for SRC beams and STRC beams were ensured by placing wooden strips of thickness equal to the cover thickness and by carefully and slowly withdrawing the same as the casting continued. The specimens were finished to a smooth and level surface and left in the mould for about 28 days. During this time, the specimens were watered well, covered all around with plastic sheets and maintained in a wet condition. After 28 days of curing, beams were demolded and the specimens were removed to a protected place with shade until the time of testing. During the casting of beam specimens, twelve standard concrete test cylinders (6 by 12 in.) were also cast. These cylinders were used to determine the properties of unconfined concrete. Figure 2.12 through 2.15 show the fabrication of the specimens.

Plane

Cross Section

Note: All dimensions in mm

Figure 2.10: Locations of the Plastic Chairs

Figure 2.11: Locations of Fiber Glass Pipes

Figure 2.12: Beam B1 Before Casting

Figure 2.13: Beam B9 Before Casting

Figure 2.14: Beam B5 Before Casting

Figure 2.15: Beam B14 Before Casting

2.3 Material properties

2.3.1 concrete

Ready mixed concrete was used for casting the concrete beams. To ensure that concrete was able to pass through the smallest spacing of polymer grids, which was 14 mm, a maximum aggregate size of 10 mm was used in all beams. The ready-mixed concrete was mixed in the lab with superplasticizers before casting and vibrated very well to avoid any possible honeycombing problems. A single batch of concrete was used to cast all the specimens. A total of twelve test cylinders were cast concurrently with the beams. This number was deemed to yield a representative sample of the concrete used in constructing the beams. It must be mentioned that the twelve test cylinders were taken at the beginning, middle and end of the casting of the beams. The cylinders were vibrated very well on a shake table. The cylinders were finished to a smooth and level surface and left in the mold for about twenty four hours. They were then demolded and transported to the curing water tank. The tank was replenished with fresh water at the end of every week. After 28 days of curing, the cylinders were removed from the water tank. Eight electrical strain gauges, two per cylinder, were fixed on four of the twelve cylinders.

A *Tinius Olsen 60,000 lb* universal testing machine was used for uniaxial compression test and the split cylinder test. During all tests, an *X-Y* plotter was used to obtain approximate load-deflection relationship instantly as the test proceeded. Figure 2.16 shows the test set up used for uniaxial compression test for cylinders. After 70 days, ten concrete cylinders were tested under uniaxial compression load. As stated earlier, four of these had strain gauges attached to them. Strain readings were recorded manually using a digital strain recorder as shown in Figure 2.16.

Table 2.2 summarize the results of the test. As can be seen, the average compressive strength $f'_c \simeq 40.0$ MPa. Figure 2.17 shows the average stress-strain curve for concrete in compression.

Two of the twelve cylinder specimens were tested using the split cylinder test method to determine the tensile strength of the concrete. Table 2.3 summarize the results of the test.

As can be seen from Table 2.2, the average splitting strength f_t was 2.9 MPa. Since the modulus of rupture f_r is about 1.5 times the splitting strength,

$$\begin{aligned} f_r &= 1.5 f_t \quad \text{MPa} \\ &= 4.35 \quad \text{MPa} \end{aligned}$$

According to the CSA standard A23.2-M77, the modulus of rupture, f_r , of normal density concrete can be taken as

$$\begin{aligned} f_r &= 0.6\sqrt{f'_c} \quad \text{MPa} \\ &= 3.8 \quad \text{MPa} \end{aligned}$$

For the purpose of theoretical calculations of strength, the modulus of rupture, f_r , has been considered to be 4.0 MPa.

Table 2-2: Concrete Compressive Strength after 70 Days

Cylinder No.	Diameter D	Length L	Area $A = \frac{\pi \times D^2}{4}$	Total Load P_u	Ultimate Stress $f_c = \frac{P_u}{A}$
	mm	mm	mm ²	N	MPa
1	152.0	305.0	18,146	776,649	42.8
2	148.6	305.0	17,343	679,846	39.2
3	150.2	305.0	17,718	714,035	40.3
4	147.8	305.0	17,157	658,829	38.4
5	151.2	305.0	17,955	712,813	39.7
6	150.8	305.0	17,860	714,400	40.0
7	149.6	305.0	17,577	683,745	38.9
8	151.4	305.0	18,003	716,519	39.8
9	150.0	305.0	17,671	708,607	40.1
10	148.8	305.0	17,390	716,468	41.2

Table 2-3: Tensile Strength after 70 days

Cylinder No.	Diameter D	Length L	Area $A = \pi \times D \times L$	Total Load P_u	Ultimate Stress $f_t = \frac{P_u}{A}$
	mm	mm	mm ²	N	MPa
1	151.0	305.0	144,877	239,660	3.3
2	152.0	305.0	145,644	181,250	2.5

**Figure 2.16: Test Set-up for Uniaxial
Compression Test**

Figure 2.17: Stress-Strain Curve for Concrete in Compression after 70 Days.

2.3.2 Steel Reinforcement

High strength deformed steel bars of 16 mm diameter (No. 15), were used as longitudinal reinforcement for beams. Plain bars of 6 mm diameter were used as shear reinforcement. The stress-strain characteristics of the reinforcing bars were determined by testing four specimens for each type of reinforcing steel cut from bars used in the beams. The tensile strains were measured using an extensometer (clip-on gauge) with a 50.8 mm base length. The extensometer was connected to the X-Y plotter. Six points were marked on each sample and the distances between them were measured carefully before and after testing to calculate the extension. Figure 2.18 and Figure 2.19 show the stress-strain relationship for the deformed bars and plain bars.

2.3.3 Tensar Grids

The high strength polymer with the trade name Tensar and manufactured in the form of grids was selected as the reinforcing material for this experimental program. The grid used had the designation uniaxial *UX* – 1600. The selected grid comes in 16.7 mm by 137.16 mm mesh made by punching holes through a stretched high density polyethelene sheet. Figure 2.20 shows the dimensions of the mesh. A full description of the properties of the grids as supplied by the Tensar Corporation is presented in Table 2.4.

**Figure 2.18: Stress-Strain Curve for
Deformed Steel Bars**

Figure 2.19: Stress-Strain Curve For Mild Steel (Stirrups)

Figure 2.20: Dimensions of Tensar Grid (UX-1600)

Table 2-4: Properties of Tensar Grid (UX-1600)

Property	Test Method	Value
Reinforcement and Interlock		
• Tensile modulus		
MD dimension	ASTM D 638 Modified (1)	138,000 lb/ft (min) [2013.8 N/mm]
• Apertures		
MD dimension	I.D. Calipered	5.4 in (nom) [137.16 mm]
CMD dimension	I.D. Calipered	0.66 in (nom) [16.7 mm]
open area	COE Method Modified (2)	60 % (nom)
• Junctions		
thickness	O.D. Calipered	0.23 in (nom) [5.8 mm]
Strength	ASTM D 638 Modified (3)	80 % (min)
• Long term design load		
MD dimension	Note (4)	300 lb/ft (nom) [43.7 N/mm]
Material		
• high density polyethelene	ASTM D 1248 Type III/Class A/Grade 5	97.5 % (nom)
• carbon black	ASTM 4218	2.5 % (nom)
Dimensions		
• roll length		98ft [29.87 m]
• roll width		3.3 ft [1.006 m]
• roll weight		75 lb [378 N]

Notes

- 1) Secant modulus at 2% elongation measured by tensile loading test ASTM D 638 modified to clamp single ribs of the grid structure at junctions and apply a constant rate of extension of the rib of 4 inches per minute (101.6 mm per minute) at a temperature of 68° F (20 C°). No offset allowances are made

in calculating secant modulus. MD dimension is along roll length. CMD dimension is across roll width.

- 2) Percent open area measured without magnification by Corps of Engineers method as specified in CW 02215 Civil Works Construction Guide, November 1977.
- 3) Junction strength is an index measure of the ability of the junction of a grid structure to resist shear and tensile stresses. Junction strength is measured as a percent of ultimate single rib tensile strength by tensile loading test ASTM D 638 modified to clamp the horizontal and vertical ribs of a "T" shaped specimen, with the grid junction forming the cross of the "T", and with a constant rate of extension of the specimen applied across the junction at a rate of 4 inches per minute (101.6 mm per minute) at a temperature of 68°F (20 C°).
- 4) Long term design load is the maximum sustained tensile force recommended for the geogrid in permanent engineered structures to ensure that deformation of the structure remains within acceptable limits over its service life. It is determined by measuring deformation of the geogrid at various loadings in 1000 hour constant load tensile tests which correlate to deformations expected over a 100 year service life.

Supplied by :

The TENSAR Corporation
1210 Citizens Parkway
Morrow, GA 30260

Figure 2.21 shows the tensile load-strain relationship for Tensar polymer grid UX – 1600. This relationship was also supplied by the Tensar Corporation. It should be noted that the test was carried out at a constant rate of extension of the rib of 101.6 mm per minute (4 inches per minute) at a temperature of 20 C°.

A tensile load test was carried out at Carleton University to determine the tensile load-strain relationship for Tensar polymer grid UX – 1600 under a rate of extension of the rib of 50.8 mm per minute (2 inches per minute) at a temperature of 20 C°. Three ribs were tested under uniaxial tension in a Instron 100 kN universal testing machine. Specialized end clamps were used, as shown in Figure 2.22. The top and bottom junctions were held directly by these clamps. Deformation was measured directly on the Tensar rib with a detachable extensometer with a 50.8 mm base length. Figure 2.23 shows the overall test set-up. In all tests, an X-

Y plotter was used to record the load-extension relationship instantly as the test proceeded. Figure 2.24 shows the average tensile load-strain relationship for Tensar geogrid obtained in the lab at Carleton University. It was expected that the testing under a faster rate of extension would yield higher ultimate tensile load value. On comparing the results of two tests, the faster rate test done by Tensar Corporation and the slower rate test done by Carleton University, this trend is not apparent. The ultimate tensile load carried out at Carleton under a rate of extension of 2 inches per minute (50.8 mm per minute) was approximately 1.6 times the ultimate tensile load under a rate of extension of 4 inches per minute (101.6 mm per minute). Figure 2.25 shows a Tensar grid used in the test beams. As shown in the figure, the grid contains five ribs. Figure 2.26 shows four layers of grids connected together as a reinforcement for beam B5 in set 3.

**Figure. 2.21: Tensile Load vs. Strain for Tensar
Grid (UX-1600)
(Tensar Corporation)**

Figure 2.22: Close-up for End Clamps Used for Tensile Test of Tensar

Figure 2.23: Test Set-up for Tensile Test of Tensar

**Figure. 2.24: Tensile Load vs. Strain for Tensar
Grid (UX-1600)
(Carleton University)**

**Figure 2.25: A Complete Tensar Grid Used
in the Test**

Figure 2.26: Four Layers Grid Connected Together

2.4 Loading Apparatus

An MTS closed-loop servo controlled, electro-hydraulic structural testing system was used . The 667-kN (150-*kip*) capacity (tension / compression) system consisted of four main units:

- (1) Control Console Assembly.
- (2) Hydraulic Actuator.
- (3) Hydraulic Power Supply.
- (4) Testing Frame.

(1) *Control Console Assembly* was a combination of the following modules:

- a: Control panel - to provide centralized manual and automatic control for console power on/off, hydraulic pressure on/off, and the simultaneous starting and stopping of programmer and recorder.
- b: Operator's control - to provide power and control for servo controller, DC Transducer Conditioner, AC Transducer Conditioner and Feedback Selector. Servo controller is an 11 DC, solid state device providing a signal for servo-valve control depending on the difference between a program signal input and feedback signal. DC Transducer Conditioner providing DC excitation and output amplification for the strain gauge bridge in the load cell. Three distinct operating ranges are available : ± 30 kip, ± 75 kips, ± 150

kips.

AC Transducer Conditioner provides 10 kHz excitation for use with displacement transducer. Its output is 10 volts DC signal full scale for feedback input to the servo controller or for monitoring purposes. Feedback Selector allows switching of up to two feedback signals (load and stroke) to be applied to servo controller.

- (2) *Hydraulic Actuator* was normally rated at 150 kips (667 kN) static with a 62.7 square-inch (40,451.5 square-mm) piston area. The actuator assembly also included an integrally mounted, 6-inch (152.4 mm) *LVDT (Linear Variable Displacement Transformer)* type of displacement transducer. The maximum stroke of the piston was 6 inches (152.4 mm).
- (3) *Hydraulic Power Supply* unit was a constant volume, 3000 psi (20.68 MPa) and 20 gpm flow capacity system equipped with a 40 hp, 460 volts, 3-phase and 60 Hz motor.
- (4) *Testing Frame* was a 14-foot (4267.2 mm) tall, 9-foot (2743.2 mm) wide structural steel frame in which the hydraulic actuator was mounted. . Figure 2.27 shows details of the testing frame. Steel pads, 6in (152.4 mm) wide, between the rollers and the beam served to distribute the applied load and the support reactions.

2.5 Test Setup

In general, the beams required various markings for positioning the loading plates and for fixing the deflection strips. The test beam was placed in the testing frame, and the loading plate was placed on top of the beam at the center of the span. A layer of hydrostone was applied between the loading plate and the beam in order to have complete contact between the plate and the upper face of the beam and to reduce stress concentration. Particular attention was given to an exact alignment

Figure 2.27: Front View of Testing Frame

alignment of the test beam, the supports, and the loading assembly to ensure true symmetric bending. Deflection strips made of an aluminium channel were fixed at the center of the span and at 600 mm from the center of the span. Six *LVDTs*, three on either side of the beam, were then attached to a fixed joist and adjusted to read against the aluminium channels. The *LVDTs* and strain gauges were connected to the data acquisition system. The overall test setup is illustrated in Figure 2.28.

2.6 Instrumentation and Data Acquisition System

2.6.1 Strain Gauges

Measurements of strains in the steel reinforcement, polymer grids and concrete surface were made using electrical strain gauges. Foil strain gauges measuring 10 mm and 30 mm were used. The strain gauges used to measure strains on the longitudinal steel reinforcement and the grids had the following specifications:

Type: *N11 - FA - 10 - 120 - 11*

Gauge length (mm) : 10

Resistance (ohm) : 119.9

Thermal output : $\pm 2 \mu/C^\circ$

The strain gauges used to measure strains on the concrete surface had the following specifications:

Type: *N11 - FA - 30 - 120 - 11*

Gauge length (mm) :30

Figure 2.28: Test Set-up

Resistance (ohm) : 119.9

Thermal output : $\pm 2 \mu/C^\circ$

All the strain gauges on the reinforcement as well as on the concrete surface were placed at the middle section of the beam. Figure 2.29 shows the locations of the strain gauges on the beams. The number of strain gauges for each beam is different. The locations and number of strain gauges for each beam are summarized in Table 2.5. All strain gauges leads were organized in such a way that they exited far from the middle part of the cage to avoid weakening the middle section.

Extreme care was taken in installing the strain gauges, specially those on the reinforcement bars and the polymer grids. A layer of protective coating (silicon) was applied to every gauge to protect it against moisture and damage during casting. Prior to actual testing, all gauges were checked for proper functioning with a digital strain recorder.

Figure. 2.29: Locations of the Strain Gauges on the Beam

Table 2-5: Locations and Number of the Strain Gauges

Beam											Total
No.	GSBL	GSBU	GSTL	GSTU	GTU	GTL	GCT	GCS1	GCS2	GCS3	No.
B1	2	2	2	2			2	1	1		12
B3					3		2	1	1		10
B4	1	1					2	1	1		6
B5	2	2	1	1	3	3					12
B6	2	2	1	1			2	2	2		12
B7	2	2	1	1	3	3					12
B8	2	2	1	1			2	2	2	2	14
B9	2	2	1	1	3	3	2	1	1		16
B10	2	2	1	1							6
B11					3	3	2	1	1		10
B12											
B13											
B14							2	1	1		4

2.6.2 LVDTs

Vertical deflections of the beam were measured using *LVDTs* (Linear Variable Differential Transformers). Six *LVDTs* with 150 mm stroke were used, three on either side of the beam. The deflection strips of aluminium channels were fixed at the center of the beam and at 600 mm from the center of the beam. Figure 2.30 shows the locations of the *LVDTs*. The *LVDTs* were mounted on magnetic bases fixed to a loading joist. A routine check of all *LVDTs* was carried out prior to every test. The *LVDT* readings were collected automatically through the data acquisition system and an *HP* microcomputer which also displayed the deflections during the test. It should be mentioned that all the *LVDTs* were removed from their locations prior to the ultimate failure in order to avoid damage to the *LVDTs*.

2.6.3 Crack measurements

The first visible crack and further cracks were identified by carefully observing the concrete surface through magnifying telescope. The maximum crack width was measured every 5 kN load step and also at each load reversal. The cracks were also marked by a pen. The magnifying telescope had a least count of 0.127 mm (0.005 inch).

2.6.4 Data Acquisition system

The data acquisition system consisted of a hard disk storage and *HP* microcomputer. The hardware was capable of collecting 20 channels of strain readings and 10 *LVDT* readings. A number of software programs in Basic were used to collect and calibrate strain and *LVDT* measurements from all reading channels and to display them on the monitor for reference. The movement of the actuator stroke was plotted against the applied load on the screen for direct reference. All data collected was stored on the hard disk and transformed to another computer systems for data reduction, plotting and reporting purposes.

In all tests, an *X-Y* plotter was used to draw the load-stroke relationship. The plotter was connected to the *HP* microcomputer. Figure 2.31 shows the *HP* microcomputer, the *X-Y* plotter and the control unit.

Figure 2.30: Locations of the LVDTs.

Figure 2.31: Data Acquisition System

2.7 Test Procedure

The following precautions were taken while testing the beams:

- i) The loading device was aligned to be truly vertical.
- ii) The test beam was supported on the roller assemblies such that the markings of the beam span were just on the rollers, the horizontal axis of the beam passed through the center of the rollers and the center of the beam span was directly below the center of the loading device.
- iii) It was ensured that the loading joist was aligned centrally and parallel to the horizontal axis of the beam.
- iv) The load cell was centered with respect to the centers of loading joist and the test beam.
- v) The probes of the *LVDTs* were positioned at equal distances from the axis of the beam symmetrically on the deflection aluminium strips on either side of the beam.
- vi) Two masonry blocks were placed between the beam and the loading joist to prevent any mishap.
- vii) The *LVDTs* were balanced to ensure an initial reading of zero.
- viii) Each electrical strain gauge was checked to ensure an initial reading of zero.
- ix) Any cracks due to shrinkage or handling were carefully observed.

Cycles of loading, unloading and reloading were carried out on each test beam. Figure 2.32 shows schematically a typical load-time curve for SRC and STRC beams. The ultimate loads in the different cycles were 25 kN, 50 kN, 75 kN, 100 kN and failure load. There was a period of 10 minutes between any two cycles to study the elastic behaviour of the reinforcement. Figure 2.33 shows the load-time curve for TRC beams. The ultimate loads in the different cycles were 5 kN, 10 kN

Figure 2.32: Load - Time Curve for SRC and STRC

Figure 2.33: Time-Load Curve for TRC Beams

and failure load. Also, there was a period of 10 minutes between any two cycles to study the elastic behaviour of the reinforcement. Loads were applied at a constant rate of 5 kN/minute during both loading and unloading. Beam deflections and strain readings were recorded at steps of 5 kN during both loading and unloading. Cracks were observed throughout the test, and all hairline cracks were marked with ink. The cracking loads were also recorded. All beams were tested to failure and whenever feasible tests were continued beyond the maximum load-carrying capacity into the descending branch of the load-resistance function. Figure 2.34 shows beam B9 in the testing machine at a total load of 25 kN and at failure.

2

MICRO

a)

b)

**Figure 2.34: a) Beam B9 at a Total Load of 25 kN
b) Beam B9 after Failure**

Chapter 3

OBSERVED BEHAVIOUR AND TEST RESULTS

3.1 Introduction

The beam specimens described in Chapter 2 were subjected to monotonic load tests. The behaviour of each specimen during testing is discussed in this chapter. The principal results of the testing program, including ultimate load, deflection, cracking characteristics and the mode of failure are presented in detail. Summary of the test results are presented at the end of this chapter. The analysis of test results will be presented in Chapter 4. The specimens were divided into seven sets according to the reinforcement configuration and were individually labelled. These labels, described earlier in Chapter 2, will be used in the discussions that follow.

3.2 General Observed Behaviour

Beams labelled SRC and STRC were under-reinforced beams. At failure, even after the spalling of the concrete cover in the compression zone, most of these

beams did not collapse, but went on carrying slowly decreasing loads with increased deflections until the beams failed by fracture of the reinforcement (steel and Tensar) in the tensile zone.

STRC beams showed that the use of Tensar grids as a secondary reinforcement in conjunction with steel imparts only marginal increase in ultimate flexural strength, but leads to considerable increase in the energy absorption capacity, improved crack characteristics and greatly increased ductility. These improvements become more significant as the content of the Tensar grids increases. The ultimate loads of STRC beams were only 1.05 to 1.07 times the ultimate loads of SRC beams. However, the deflections of STRC beams at failure varied between 1.5 to 2 times the deflections of SRC beams, indicating a marked increase in ductility. Another important phenomenon was that the presence of Tensar grids had a beneficial effect on the crack width and crack spacing. The TRC beams showed that the use of Tensar grids as the principal reinforcement gives large deflections at small loads. As would be expected for a reinforcing material with a Young's modulus as low as that of Tensar grids, TRC beams started to manifest visible cracks under relatively low loads. The first visible crack load decreased with increasing polymer grids content, although the post-cracking load capacity and deflection increased with increasing volume content of Tensar grids. The ultimate loads of TRC beams ranged from only 0.13 to 0.15 times the ultimate loads of SRC beams. The maximum deflections of TRC beams ranged from 2 to 4 times the maximum deflections of STRC beams and SRC beams.

3.3 Set 1 - Steel Reinforced Concrete Beams -SRC beams (B1 and B8)

3.3.1 General

Specimens in this set (beam B1 and beam B8) were designed as control beams. Beam B1 was reinforced with 2 No. 15 high-strength longitudinal steel bars as tension reinforcement and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement. Beam B8 was reinforced with 2 No. 15 high-strength longitudinal steel bars as tension reinforcement and two layers Tensar grid , discontinued at the middle of the beam, and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement.

Beams B1 and B8 failed within 1.13 times of their predicted failure load, and the failure was preceded by yielding of the steel. Both Beams , with a distinctly under-reinforced section, exhibited the failure pattern of initial reinforcement yield, followed by fracture of steel in tension (Figure 3.1). There was no significant difference between B1 and B8 in the loads or deflections. The only difference was in the crack characteristics. Beam B8 showed increased number of cracks as compared to those in beam B1 under the same load. Also, the maximum crack width in beam B8 was less than that of beam B1. The difference in the crack widths was slight. Longitudinal cracks (bond cracks) became visible after yielding of the steel for a very short distance at the middle of the beams. The polymer grids in beam B8 kept the bottom concrete cover from falling after failure as shown in Figure 3.2. The deformation behaviour observed for these beams is similar to results obtained in tests carried out by other researchers [53]. Individual beam behaviour is described below.

Figure 3.1: Close-up of the Midspan after Failure for Beam B1

**Figure 3.2: Close-up of the Bottom of Midspan Section
after Failure for Beam B8**

3.3.2 Observed Behaviour

Beam B1

- i) The first cycle (25 kN): First hairline crack became visible at midspan under 15.00 kN total load. This crack became more prominent as the loads increased. At a total load of 25.00 kN, 2 cracks could be observed within the midhalf of the beam and the maximum crack width was 0.30 mm. Crack widths became approximately zero after the beam was unloaded.
- ii) The second cycle (50 kN): At a total load of 50.00 kN, 4 cracks were observed and the maximum crack width was 0.52 mm. At the end of the cycle, crack widths became approximately zero.
- iii) The third cycle (75 kN): At a total load of 70.00 kN, shear cracks appeared near the supports. At a total load of 75.00 kN, 10 flexural cracks were observed along the beam. The maximum crack width was 0.89 mm.
- iv) The fourth cycle (100 kN): After the load had reached 75.00 kN, the number of cracks increased until it reached 17 at a total load of 100.00 kN. The maximum crack width was 1.15 mm.
- v) The fifth cycle (Failure load): Yielding of tension reinforcement began at a load of approximately 110.00 kN. The midspan deflection of the beam at the beginning of reinforcement yielding was 13.20 mm. Shortly afterwards, the maximum load of 128.50 kN was reached. With increasing displacement, the load decreased to 126.40 kN at which time the tensile steel reinforcement fractured. After yielding, it was observed that some longitudinal cracks (bond cracks) became visible at the middle of the beam. These cracks occurred at the level of the

tension steel reinforcement. It should be noted that these cracks extended for a very short distance at the middle of the beam. After failure, the concrete cover in the compression zone was investigated and it was found to be completely spalled. No buckling was observed for the compression steel. The maximum deflection of the beam was 66.40 mm.

Figure 3.3 illustrates the load versus midspan deflection relationship obtained during the test. The failure mechanism in a plastic hinge as shown in Figure 3.4. The maximum crack widths recorded at maximum load in each cycle are shown in Figure 3.5. Figure 3.6 shows the number of cracks at different cycles. .

Beam B8

- i) The first cycle (25 kN) : Hairline cracks started at about 15.00 kN total load at midspan. At a total load of 25.00 kN, 4 cracks were detected within the midhalf of the beam (Figure 3.8a). The maximum crack width was 0.26 mm. Crack widths became approximately zero when the beam was unloaded.
- ii) The second cycle (50 kN) : Cracks grew more prominent and numerous as the load increased. Ten cracks were observed at a total load of 50.00 kN and the maximum crack width was 0.40 mm (Figure 3.8b). At the end of the cycle, crack widths became approximately zero.
- iii) The third cycle (75 kN) : At a total load of 70.00 kN, 2 shear cracks appeared near the right support. At a total load of 75.00 kN, 19 cracks were observed along the beam. The maximum crack width was 0.82 mm. The crack width became approximately zero when the beam was unloaded.
- iv) The fourth cycle (100 kN) : When the load reached 75.00 kN, another two shear cracks became visible near the left support. The number of cracks increased

Figure 3.3: Load vs. Midspan Deflection for Beam B1

Figure 3.4: Beam B1 after Failure

Figure 3.5: Maximum Crack Width at different Cycles for Beam B1

Figure 3.6: Number of Cracks at Different Cycles for Beam B1

until it reached 30 at a total load of 100.00 kN. The maximum crack width was 0.95 mm.

- v) The fifth cycle (Failure load): Beyond a load of 100.00 kN, the number of cracks increased until yielding of the tensile reinforcement began at a total load of 115.00 kN. The midspan deflection of the beam at the beginning of yielding of the reinforcement was 13.10 mm. The maximum load was reached at 131.00 kN at which time the tensile steel reinforcement fractured. The maximum deflection of the beam was 52.10 mm. Again, the longitudinal cracks became visible after yielding of the steel and extended for a very short distance only. After failure, the concrete cover in the compression zone was observed and it was found to be completely spalled. No buckling was observed for the compression steel.

Figure 3.7 shows the load versus midspan deflection relationship obtained during the test. The maximum crack width was measured in each cycle during loading and after releasing of the load. Figure 3.8 shows the development of crack patterns under loads of 25 kN, 50 kN, 75 kN, 100 kN and 131 kN. Measurements of the maximum crack width are given in Figure 3.9. Figure 3.10 shows the number of cracks at different cycles.

3.4 Set 2 - Steel and Tensar Reinforced Concrete Beams - STRC beams (B3 and B4)

3.4.1 General

This set of beams was designed to study the effect of Tensar grids on the behaviour of beams when the Tensar grids were placed above the tension steel.

Figure 3.7: Load vs. Midspan Deflection for Beam B8

a) At 25 kN

b) At 75 kN

Figure 3.8: Development of Crack Patterns for Beam B 8

c) At 100 kN

d) At Failure

Figure 3.8 (Contd.): Development of Crack Patterns for Beam B 8

Figure 3.9: Maximum Crack Width at Different Cycles for Beam B8

Figure 3.10: Number of Cracks at Different Cycles for Beam B8

Specimens (beam B3 and beam B4) were reinforced with 2 No. 15 high-strength longitudinal steel bars and two layers of Tensar grid placed above the steel as tension reinforcement for the beam and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement. Ultimate loads of these beams varied from 1.05 to 1.06 times the average ultimate load of the control beams in set 1. At failure, the maximum deflection of beams in this set ranged between 1.40 to 1.50 the average maximum deflection of beams in set 1 (control beams). Beams B3 and B4 failed by fracture of tension steel as for beams in set 1. Complete rupture of tension steel and complete tearing for Tensar grids were observed. The number of cracks in beam B3 was approximately similar to that of beam B1 but the maximum crack width was less than that of beam B1. Beam B4 showed increase in the number of cracks and a decrease in the crack width. It was observed that the longitudinal cracks (bond cracks) became visible after yielding of the steel. These cracks extended for a distance wider than that of beams in set 1.

Individual beam behaviour is described below.

3.4.2 Observed Behaviour

Beam B3

- i) The first cycle (25 kN): The first visible crack was detected under 15.00 kN total load at midspan. At a total load of 25.00 kN, 3 cracks could be observed within the midhalf of the beam as shown in Figure 3.12a. The maximum crack width was 0.27 mm. Crack widths became approximately zero when the beam was unloaded.
- ii) The second cycle (50 kN): Cracks grew more prominent and numerous as loads increased. Four cracks were observed at a total load of 50.00 kN. The maximum

crack width was 0.43 mm, see Figure 3.12b.

- iii) The third cycle (75 kN): At a total load of 70.00 kN, 2 shear cracks were observed between the supports and the load bearing plate. At a total load of 75.00 kN, 9 cracks could be seen along the beam and the maximum crack width was 0.75 mm. Crack widths became approximately zero when the beam was unloaded.
- iv) The fourth cycle (100 kN): Beyond the load of 75.00 kN, the number of cracks increased until it reached 13 at a total load of 100.00 kN. The maximum crack width was 1.10 mm.
- v) The fifth cycle (Failure load): Shear cracks became visible near the supports at a total load of 110.00 kN. At a total load of 118.00 kN, yielding of the tensile reinforcement began, the midspan deflection of the beam at the beginning of yielding was 12.90 mm. Shortly after, the maximum load of 137.80 kN was reached. It rapidly decreased to 127.00 kN. The maximum midspan deflection was 89.00 mm. The beam failed by rupture of the reinforcement (steel reinforcement and Tensar grids) in the tensile zone. There was no observed buckling of compression steel. The longitudinal cracks (bond cracks) became visible after yielding of the steel and extended for a distance greater than that in SRC beams as shown in Figure 3.12e.

Observation after failure showed that the concrete cover in the compression zone was completely spalled.

Figure 3.11 illustrates the load-midspan deflection relationship obtained during the test. Crack patterns for 25 kN, 50 kN, 75 kN, 100 kN and 127.00 kN are shown in Figure 3.12. Measurements of the maximum crack widths are shown in Figure 3.13. The number of cracks at different cycles is shown in Figure 3.14.

Figure 3.11: Load vs. Midspan Deflection for Beam B3

a) At 25 kN

b) At 50 kN

Figure 3.12: Development of Crack Patterns for Beam B3

c) At 75 kN

d) At 100 kN

Figure 3.12 (Contd.): Development of Crack Patterns for Beam B3

e) At Failure

Figure 3.12 (Contd.): Development of Crack Patterns for Beam B3

Figure 3.13: Maximum Crack Width at Different Cycles for Beam B3

Figure 3.14: Number of Cracks at Different Cycles for Beam B3

Beam B4

- i) The first cycle (25 kN) : Beam cracking started at 14.00 kN . At a total load of 25.00 kN, only one crack was observed within the midhalf of the beam (Figure 3.16.a). The maximum crack width was 0.23 mm.
- ii) The second cycle (50 kN) : Eleven cracks were observed at a total load of 50.00 kN. The maximum crack width was 0.38 mm (Figure 3.16b). At the end of the cycle, crack widths became approximately zero.
- iii) The third cycle (75 kN) : Shear cracks became visible near the left support at a total load of 70.00 kN. At the maximum load in the cycle (75 kN), 19 cracks were observed along the beam. The maximum crack width was 0.70 mm.
- iv) The fourth cycle (100 kN) : The number of cracks increased until it reached 24 at a total load of 100.00 kN. The maximum crack width was 0.90 mm. Again, crack widths became approximately zero when the beam was unloaded.
- v) The fifth cycle (Failure load) : Yielding of the tensile reinforcement began at a load of approximately 118.00 kN. The midspan deflection of the beam at the beginning of yielding was 13.40 mm. The beam went on to carry more load until the maximum load of 137.00 kN was attained following which it failed by rupturing of steel. The maximum central deflection obtained was 83.00 mm. The beam failed by a complete failure of the reinforcement (Steel bars and Tensar grids) in the tensile zone. No buckling was observed for the compression steel. The concrete cover in the compression zone was observed after failure and it was found to be completely spalled. Again, the longitudinal cracks (bond cracks) appeared after yielding of the steel at the level of the tension steel and extended for a distance greater than that in SRC beams as shown in Figure 3.16e.

Figure 3.15 shows the load-midspan deflection relationship for beam B4. Crack patterns for 25 kN, 50 kN, 75 kN, 100 kN and 137 kN are shown in Figure 3.16. Figure 3.17 shows a close-up of the plastic hinge after failure. Measurements of the maximum crack widths are shown in Figure 3.18. Figure 3.19 shows the number of cracks at different cycles.

3.5 Set 3 - Steel and Tensar Reinforced Concrete Beams - STRC beams (B5 and B6)

3.5.1 General

This set was similar to set 2 but had a different content of Tensar grids. Specimens in this set (beam B5 and beam B6) were reinforced with 2 No.15 high-strength longitudinal steel bars and four layers of Tensar grid placed above the steel as tension reinforcement, and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement. Beam B5 failed by rupturing of tension steel in a manner similar to that of beams in sets 1 and 2. Beam B6 failed in a shear mode (diagonal tension) near the bearing plate at the middle of the beam (Figure 3.18). The ultimate load of beams B5 and B6 were respectively 1.048 and 1.077 times the average maximum load for beams in set 1 (control beams). The maximum midspan deflection of beam B5 was about 1.747 the average maximum midspan deflection of beams in set 1. The maximum deflection of beam B6 was less than the average maximum deflection of beams in set 1. The number of cracks in beam B5 was more than the number of cracks in beam B1 but the number of cracks in beam B6 was less than that of beam B1. Crack widths were not measured for beam B5 but were measured for beam B6. The maximum crack width in beam B6 was less than that

Figure 3.15: Load vs. Midspan Deflection for Beam B4

a) At 25 kN

b) At 50 kN

Figure 3.16: Development of Crack Patterns for Beam B4

c) At 75 kN

d) At 100 kN

Figure 3.16 (Contd.): Development of Crack Patterns for Beam B4

e) At Failure

Figure 3.16 (Contd.): Development of Crack Patterns for Beam B4

Figure 3.17: Close-up of the Middle Section for Beam B4

Figure 3.18: Maximum Crack Width at Different Cycles for Beam B4

Figure 3.19: Number of Cracks at Different Cycles for Beam B4

both beams, B5 and B6.. These cracks extended for a distance greater than those of beams in set 1 and set 2. Individual beam behaviour is described below.

3.5.2 Observed Behaviour

Beam B5

The maximum crack widths were not measured for this beam.

- i) The first cycle (25 kN): First hairline crack was observed at midspan under 15.00 kN total load and subsequently grew more prominent as the load increased. At a total load of 25.00 kN, 7 cracks could be observed within the midhalf of the beam.
- ii) The second cycle (50 kN): At a total load of 50.00 kN, 10 cracks were observed. At the end of the cycle, crack widths were approximately zero.
- iii) The third cycle (75 kN): Shear cracks became visible near the right support at a total load of 65.00 kN. Similar shear cracks appeared near the left support at a total load of 70.00 kN. Sixteen cracks were observed along the beam when the load reached 75.00 kN.
- iv) The fourth cycle (100 kN): Beyond a load of 75.00 kN, the number of cracks increased until it reached 20 at a total load of 100.00 kN .
- v) The fifth cycle (Failure load): Yielding of tension reinforcement began at a load of 115.00 kN. The midspan deflection of the beam at the beginning of yielding of reinforcement (steel bars) was 13.20 mm. Shortly afterwards, the maximum load of 139.70 kN was reached, it diminished to 138.00 kN at which time the tensile steel reinforcement failed by rupture. The longitudinal cracks (bond

cracks) became visible after yielding of the steel and extended for a distance greater than those for beams in set 2 as shown in Figure 3.22.

After failure, the concrete cover in the compression zone was found to be completely spalled. No buckling was detected for the compression steel. The maximum deflection of the beam at failure was 103.50 mm. Figure 3.20 illustrates the load versus central deflection relationship obtained during the test. Figure 3.21 shows beam B5 after failure. Figure 3.22 shows a close-up of the midspan section of the beam after failure.

The number of cracks at different cycles is given in Figure 3.23.

Beam B6

- i) The first cycle (25 kN) : Initial cracking again started at about 15.00 kN total load at midspan. At a total load of 25.00 kN, one crack was observed within the midhalf of the beam (Figure 3.25a) and the maximum crack width was 0.30 mm. Crack widths became approximately zero when the beam was unloaded.
- ii) The second cycle (50 kN) : The only crack observed during the previous cycle grew more prominent as loads increased. At a total load of 50.00 kN, no additional cracks were observed. The maximum crack width was 0.63 mm.
- iii) The third cycle (75 kN) : At a total load of 65.00 kN, shear crack appeared to the left of load bearing plate, near the middle of the beam. At a total load of 75.00 kN, 5 cracks were observed along the beam and the maximum crack width was about 0.95 mm.
- iv) The fourth cycle (100 kN) : After the load had reached 85.00 kN, new shear crack became visible to the right of load bearing plate. At a total load of 90.00 kN, shear cracks appeared near the right support. The number of cracks

Figure 3.20: Load vs. Midspan Deflection for Beam B5

Figure 3.21: Beam B5 after Failure

Figure 3.22: Close-up of Midspan Section after Failure, Beam B5

Figure 3.23: Number of Cracks at Different Cycles for Beam B5

increased until it reached 22 at a total load of 100.00 kN. The maximum crack width was about 1.27 mm.

- v) The fifth cycle (Failure load): The yielding load was observed as 115.00 kN and the maximum deflection of the beam at the beginning of yielding of the reinforcement was 12.60 mm. Under a total load of 136.00 kN, a sudden shear failure occurred close to the bearing plate as shown in figure 3.18e. Subsequent investigation revealed that during beam fabrication one stirrup had inadvertently been placed in a wrong position, and this negligence caused the beam to fail in shear before its flexural capacity was quite exhausted. After yielding, it was observed that the longitudinal cracks (bond cracks) became visible at the level of the tension steel as seen in Figure 3.25e.

Figure 3.24 shows load-midspan deflection relationship. Crack patterns at different cycles is indicated in figure 3.25. Maximum crack width at the maximum load of each cycle is given in Figure 3.26. The number of cracks at different cycles are given in Figure 3.27.

3.6 Set 4 - Steel and Tensar Reinforced Concrete Beams - STRC beams (B7)

3.6.1 General

Beams in set 2 and set 3 showed that the presence of Tensar grids placed above the tensile steel reinforcement had improved the flexural strength, ductility, and cracking characteristics of the conventional reinforced concrete beams. For set 4, Tensar grids were placed under the tensile steel reinforcement in the tension concrete

Figure 3.24: Load vs Midspan Deflection for Beam B6

a) At 25 kN

b) At 50 kN

Figure 3.25: Development of Crack Patterns for Beam B6

c) At 75 kN

d) At 100 kN

Figure 3.25 (Contd.): Development of Crack Patterns for Beam B6

Figure 3.25 (Contd.): Development of Crack Patterns for Beam B6

Figure 3.26: Maximum Crack Width at Different Cycles for Beam B6

Figure 3.27: Number of Cracks at Different Cycles for Beam B6

cover. It was believed that if Tensar grids were placed under steel reinforcement, cracking behaviour of beams will be improved in comparison with beams in sets 2 and 3.

Beam B7 was reinforced with 2 No.15 high-strength longitudinal steel bars and two layers of Tensar grid placed under the steel as tension reinforcement and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement. The beam failed in a tension mode (complete fracture of steel bars and tensar grids) similar to the failure mode of beams in set 1 (control beams). The maximum load of the beam was about 1.059 times the average maximum load of beams in set 1. The maximum midspan deflection was 56% more than the average maximum midspan deflection of beams in set 1. It was observed that the longitudinal cracks (bond cracks) became visible after yielding of the steel. These cracks extended for a distance wider than that of beams in set 1 (Figure 3.30e). The most significant advantage of the presence of Tensar grids in the concrete cover in tension appeared to be in the crack distribution. Also, the Tensar grids kept the bottom concrete cover from falling after failure. Figure 3.28 shows the bottom of beam B7 after failure. As seen in the figure, very small parts from the bottom concrete cover of beam B7 had failed in comparison with the bottom concrete cover of beam B1.

Description of beam behaviour is given below.

3.6.2 Observed Behaviour

Beam B7

- i) The first cycle (25 kN): Beam cracking again started at a total load of 15.00 kN. At the end of the cycle 2, cracks at the middle of the beam were observed as shown in figure 3.30a. The maximum crack width was 0.20 mm.

- ii) The second cycle (50 kN): As the load was increased to 50.00 kN, the initial two cracks grew and another 7 cracks were observed (figure 3.30b). The maximum crack width was 0.33 mm.
- iii) The third cycle (75 kN): At a total load of 75.00 kN, 16 cracks were observed along the beam. No shear cracks were observed during this cycle. The maximum crack width was 0.60 mm.
- iv) The fourth cycle (100 kN): At a total load of 80.00 kN, shear cracks became visible near the supports. When the load reached 100.00 kN, the number of cracks increased until it reached 29. The maximum crack width was 0.85 mm.
- v) The fifth cycle (Failure load): Yielding of tension reinforcement began at a load of approximately 115.00 kN. The midspan deflection of the beam at the beginning of yielding of reinforcement was 13.85 mm. Shortly afterwards, the maximum load of 137.40 kN was reached, this decreased to 125.00 kN at which time the tensile steel reinforcement ruptured. The maximum midspan deflection of the beam was 92.80 mm. The longitudinal cracks (bond cracks) became visible after yielding of the steel and extended for a wide distance. These cracks occurred at the level of the tension reinforcement as shown in Figure 3.30e.

Figure 3.29 illustrates the load versus midspan deflection relationship. The concrete cover in the compression zone was observed after failure and it was found to be completely spalled. No buckling was observed for compression steel. Also, the concrete cover in the tension zone was investigated and it was noted that Tensar grids kept the concrete cover from falling as shown in figure 3.28. The crack patterns under 25 kN, 50 kN, 75 kN, 100 kN and 125 kN are shown in Figure 3.30. Figure 3.31 indicates the maximum crack widths recorded at the maximum load levels in each cycle. The number of cracks at different cycle is shown in Figure 3.32.

a) Beam B1 (SRC)

b) Beam B7 (STRC)

Figure 3.28: Concrete Cover at Tension Side

a) B1 b) B7

Figure 3.29: Load vs. Midspan Deflection for Beam B7

Figure 3.30: Development of Crack Pattern for Beam B7

c) At 75 kN

d) At 100 kN

Figure 3.30 (Contd.): Development of Crack Patterns for Beam B7

e) At Failure

Figure 3.30 (Contd.): Development of Crack Patterns for Beam B7

Figure 3.31: Maximum Crack width at Different Cycles for Beam B7

Figure 3.32: Number of Cracks at Different Cycles for Beam B7

3.7 Set 5 - Steel and Tensar Reinforced Concrete Beams - STRC beams B9 and B10

3.7.1 General

For beams in set 2 and set 3, Tensar grids were placed above the tensile steel reinforcement and for beam in set 4, Tensar grids were placed under the tensile steel reinforcement. In set 5, Tensar grids were placed both above and under the tensile steel reinforcement to study the effect of such a disposition of Tensar grids on the behaviour of beams.

Beams in set 5 (beam B9 and beam B10) were reinforced with 2 No.15 high-strength longitudinal steel bars and four layers of Tensar grid, two layers placed above the steel and two layers placed under the steel, as tension reinforcement, and 2 No. 15 high-strength longitudinal steel bars as compression reinforcement.

Ultimate loads of these beams varied from 1.056 to 1.065 times the average ultimate loads of beams in set 1 (control beams). At failure, the maximum midspan deflections of beams in this set were 62% and 74% higher than the average maximum midspan deflection of control beams (set 1). Beam B9 failed in a tension mode with a complete rupture of tension steel and complete tearing of Tensar grids. For beam B10, the failure mode initiated by compression followed by shear after the tensile reinforcement had yielded. Beams in this set showed an increase in number of cracks and a decrease in the cracks widths in comparison with beam B1. The longitudinal cracks (bond cracks) in these beams were observed after yielding of the steel. These cracks extended for a wide distance as shown in Figure 3.35e. Description of the behaviour for each beam is given below.

3.7.2 Observed Behaviour

Beam B9

- i) The first cycle (25 kN): Initial cracking started at about 15.00 kN total load. As the load increased the cracks became more prominent and numerous. At a total load of 25.00 kN, 11 cracks could be observed within the midhalf of the beam as shown in Figure 3.35a. The maximum crack width was 0.21 mm. Crack widths became zero when the beam was unloaded.
- ii) The second cycle (50 kN): At a total load of 50.00 kN, 19 cracks were observed. The maximum crack width was 0.35 mm (figure 3.35b).
- iii) The third cycle (75 kN): At a total load of 65.00 kN, shear cracks appeared near the right support. At a total load of 75.00 kN, 23 cracks were observed along the beam. The maximum crack width was about 0.66 mm.
- iv) The fourth cycle (100 kN) : When the load reached 80.00 kN, shear cracks became visible at the left support. After that, the number of cracks increased until it reached 30 at a total load of 100.00 kN. The maximum crack width was 0.90 mm.
- v) The fifth cycle (Failure load) : After the load had reached 100 kN, the number of cracks increased until yielding of the tensile reinforcement began at a total load of approximately 115.00 kN. The midspan deflection of the beam at the beginning of yielding was 14.00 mm. Shortly afterwards, the maximum load of 138.20 kN was reached; the load then decreased to 132.60 kN at which time the tensile reinforcement ruptured. The beam failed by rupture of the reinforcement (steel reinforcement and Tensar grids). After failure, the concrete cover was

observed and it was found to be completely spalled. No buckling was observed for the compression steel. The maximum midspan deflection of the beam was 96.00 mm. The longitudinal cracks (bond cracks) became visible after yielding of the tension steel. It was noted that these cracks extended for a distance wider than those of beams in set 2 and set 4 as shown in Figure 3.33 and Figure 3.35e.

Figure 3.34 shows the load-midspan deflection relationship obtained during the test. Figure 3.35 shows the crack patterns at loads of 25 kN, 50 kN, 75 kN, 100 kN and at failure. Measurements of the maximum crack widths are shown in Figure 3.36. The number of cracks at different cycles is shown in Figure 3.37.

Beam B10

It should be noted that crack widths were not measured for this beam.

- i) The first cycle (25 kN): The first hairline crack was detected under 14.00 kN total load at midspan. At a total load of 25.00 kN, 7 cracks could be observed within the midhalf of the beam as shown in Figure 3.40a.
- ii) The second cycle (50 kN): Ten cracks were observed at a total load of 50.00 kN.
- iii) The third cycle (75 kN): At a total load of 70.00 kN, shear cracks appeared near the supports. Eighteen cracks were observed along the beam at the maximum load of the cycle (75 kN).
- iv) The fourth cycle (100 kN): Shear cracks became visible near the supports and at the middle of the beam at a total load of 90.00 kN. The number of cracks increased until it reached 22 at a total load of 100.00 kN.

Figure 3.33: Close-up of Midspan of Beam B9

Figure 3.34: Load vs. Midspan Deflection for Beam B9

a) At 25 kN

b) At 50 kN

Figure 3.35: Development of Crack Patterns for Beam B9

c) At 75 kN

d) At 100 kN

Figure 3.35 (Contd.): Development of Crack Patterns for Beam B9

e) At Failure

Figure 3.35 (Contd.): Development of Crack Patterns for Beam B9

Figure 3.36: Maximum Crack Width at Different Cycles for Beam B9

Figure 3.37: Number of Cracks at Different Cycles for Beam B9

- v) The fifth cycle (Failure load): The yielding load was observed to be 120.00 kN. The midspan deflection of the beam at the beginning of yielding was 13.90 mm. The maximum load was reached at 137.00 kN, this later decreased to 132.00 kN at which time the beam failed by sudden compression failure followed by shear failure as shown in figure 3.38. The maximum midspan deflection just before the compression failure was 103.00 mm. Again, the longitudinal cracks appeared after yielding of the steel tensile reinforcement. These cracks extended for a distance wider than that of beams in set 2 and set 4 as shown in Figure 3.40.

Observation after failure showed that there was no tearing of Tensar grids or rupture of the tensile steel.

Figure 3.39 illustrates the load versus deflection relationship obtained during the test. The crack patterns are shown in figure 3.40. The number of cracks at different cycles is shown in Figure 3.41.

3.8 Set 6 - Tensar Reinforced Concrete Beams - TRC beams (B11 and B12)

3.8.1 General

Beams in sets 6 and 7 were designed to study the behaviour of concrete beams when Tensar grids alone were used as the main reinforcement. The concrete cover in the tension side of the beam was the same as that of SRC beams (38 mm).

Specimens in set 6 (beam B11 and beam B12) were reinforced with 2 layers of Tensar grids on the tension side of the beam as main reinforcement. There was no compression or shear reinforcement. As would be expected for a reinforcing

Figure 3.38: Close-up of the Midspan Section after Failure for Beam B10

Figure 3.39: Load vs. Midspan Deflection for Beam B10

a) At 25 kN

b) At 50 kN

Figure 3.40: Development of Crack Patterns for Beam B10

c) At 75 kN

d) At 100 kN

Figure 3.40 (Contd.): Development of Crack Patterns for Beam B10

e) At Failure

Figure 3.40 (Contd.): Development of Crack Patterns for Beam B10

Figure 3.41: Number of Craks at Different Cycles for Beam B10

material with Young's modulus as low as that of Tensar grids, both beams started to exhibit visible cracks under relatively low loads. The cracks became rather large when the loads increased. The beams showed an increasing post cracking load capacity. Ultimate loads of beam B11 and beam B12 were 11.75 kN and 11.95 kN respectively. The average ultimate loads for these beams were less than 10% of the average ultimate loads of SRC beams. To examine the post cracking ductility of the test specimens, all the specimens were loaded beyond first cracking until a complete failure of Tensar grids occurred. For all beams, the maximum stroke of the test machine (6 in-125.40 mm) was reached and yet the beams did not fail. So, a wood block with 95.00 mm height was placed under the steel bearing plate in order to get enough stroke. Each beam failed into two parts after complete tearing of the grids had occurred.

3.8.2 Observed Behaviour

Beam B11

- i) The first cycle (5 kN): No cracks were observed during this cycle.
- ii) The second cycle (10 kN): Again, no crack was detected during this cycle.
- v) The third cycle (Failure load): Initial crack began at a total load of 10.50 kN right near to the load bearing plate. The cracks grew larger while the load was increasing. The maximum load of 11.75 kN was reached. With further increase in displacement the load decreased until failure took place at 9.20 kN. The maximum deflection of the beam at failure was 95.00 mm. No more cracks other than the first one were observed. The beam separated into two parts along the first crack. Crack widths were not measured for this beam.

Figure 3.42 illustrates the load versus midspan deflection relationship obtained during the test. Figure 3.43 shows the crack patterns before failure. Close-up of the beam after failure is shown in figure 3.44.

Beam B12

- i) The first cycle (5 kN) : No crack was observed during this cycle.
- ii) The second cycle (10 kN) : Again, no cracks were detected during this cycle.
- v) The third (Failure load) : At a total load of 10.30 kN, three initial cracks at the middle of the beam were observed. The cracks grew larger as the load increased. After the deflection had reached 105.00 mm, the beam was unloaded to put the wood block under the steel bearing plate. Five minutes after unloading, the deflection was measured and it was observed that it had decreased to 84.00 mm due to the elastic behaviour of Tensar grids. The beam was loaded again until a complete tearing of Tensar grid occurred at which the beam split into two parts as shown in figure 3.47. At failure, the load was 13.43 kN and the maximum deflection was 166.00 mm

Figure 3.45a shows the crack pattern just after unloading and Figure 3.45b shows the crack pattern 5 minutes after unloading. Figure 3.46 shows load-midspan deflection relationship. The crack patterns at different cycles are indicated in Figure 3.47.

Figure 3.42: Load vs. Midspan Deflection for Beam B11

Figure 3.43: Beam B11 before Failure

Figure 3.44: Close-up of Cracked Section of Beam B11 after Failure

a) At Time 0

b) After 5 Minutes

**Figure 3.45: Close-up of Midspan Section of Beam
B12 after Unloading and after Unloading
by 5 Minutes**

Figure 3.46: Load vs. Midspan Deflection for Beam B12

a) At 5 kN

b) At 10 kN

Figure 3.47: Development of Crack Patterns for Beam B12

c) After Unloading

d) At Failure

Figure 3.47 (Contd.): Development of Crack Patterns for Beam B12

3.9 Set 7 - Tensar Reinforced Concrete Beams - TRC beams (B13 and B14)

3.9.1 General

Beams in this set were similar to beams in set 6, but had different amount of Tensar grids. Specimens in this set (beam B13 and beam B14) were reinforced with 4 layers of Tensar grids on the tension side only as a main reinforcement. There was no compression or shear reinforcement. The concrete cover on the tension side of the beam was the same as that of SRC beams (38.00 mm). The beams started to manifest visible cracks under relatively low loads, less than those of beams in set 6. The beams in set 7, demonstrated not only higher post-cracking load capacity, but also higher loads and deflection at failure when compared with beams in set 6. Thus, both the post-cracking load capacity and deflection increased with increasing volume content of Tensar grids. Ultimate loads of these beams were 17.10 kN and 16.70 kN respectively. The average ultimate load of these beams was about 1.42 times the average ultimate load of beams in set 6 and about 13% the average ultimate load of beams in set 1. Again, it was impossible to fail the beams completely within the limited stroke of the test machine that is 125.40 mm (6 in.). So, a wood bearing block of 95.00 mm height was placed above the steel bearing plate in order to get enough stroke. Beam B13 failed in a tension mode by the rupture of Tensar grids but it was impossible to get a complete rupture for beam B14. The maximum midspan deflection for the beams in this set ranged between 1.33 to 1.35 the average maximum midspan deflection of beams in set 6. Individual beam behaviour is described below.

3.9.2 Observed Behaviour

Beam B13

- ii) The first cycle (5 kN): No cracks were observed.
- ii) The second cycle (10 kN): Again, no cracks were detected.
- v) The third cycle (Failure load): Initial cracks were detected at a total load of 9.70 kN (6 cracks). These cracks widened as the load increased. After the deflection had reached 115.00 mm, the beam was unloaded to place the wood bearing plate above the steel bearing plate in order to get enough stroke. The maximum crack width before unloading the beam was 15.00 mm (measured with an ordinary scale having a least count of 1.0 mm). Five minutes after unloading, the crack width of 15 mm decreased to 4.00 mm, and after 15 more minutes it became 3.50 mm only. In the same time, the deflection diminished from 115 mm to 56 mm. After two hours, the maximum crack width was about 3.30 mm. Three days after unloading, the maximum crack width was about 3.10 mm which means that the change in crack width is very small after 15 minutes of unloading. At the end of three days, the beam was reloaded until failure. The ultimate load of the beam was 16.70 kN. The maximum deflection of the beam at failure was 170.00 mm. Observation after failure showed that the cracks occurred at the junctions of the polymer grids.

Figure 3.48 illustrates the load versus midspan deflection relationship obtained during the test. Figure 3.49 shows the beam before failure, during unloading stage.

Beam B14

- i) The first cycle (5 kN) : No cracks were observed.
- ii) The second cycle (10 kN): The first visible crack was detected near the fiber glass pipe at the middle at a total load of 7.00 kN and the maximum crack width was 1.50 mm. When the load reached 10.00 kN, another 2 cracks at the middle of the beam were observed. The maximum crack width at 10.00 kN was 4.30 mm. After the beam was unloaded, the crack widths and deflection decreased conspicuously.
- v) The third (Failure load) : The beam was loaded again until the deflection reached 103 mm. The beam was then unloaded and left in that state for 10 minutes. The maximum crack width before loading was 7.62 mm. Ten minutes after unloading, the maximum crack width decreased to 3.20 mm. The wood bearing block was placed above the steel bearing plate and the beam was loaded again until failure. The ultimate load of the beam was 19.20 kN, then decreased to 16.50 kN at which time a sudden shear failure near the bearing plate occurred. The maximum deflection at ultimate load was 163.00 mm and the maximum deflection at failure was 193.00 mm. It was impossible to get a complete tearing of Tensar grids.

Figure 3.50 shows load-midspan deflection relationship. The crack patterns at different cycles are shown in figure 3.51.

Figure 3.48: Load vs. Midspan Deflection for Beam B13

Figure 3.49: Beam B13 During Unloading Stage

Figure 3.50: Load vs. Midspan Deflection for Beam B14

a) At 5 kN

b) At Failure

Figure 3.51: Development of Crack Patterns for Beam B14

3.10 Strain Readings

3.10.1 Strain Readings in SRC and STRC Beams

The strain gauges were located at the middle of SRC and STRC beams to investigate the effect of the polymer grids on the distribution of the curvature of the beams. This was mainly done to study the elastic behaviour of the beams before yielding of the steel. It was expected that the strain gauges fixed on the steel reinforcement and Tensar grids will become inoperative after yielding of the reinforcement.

The average reading of the strain gauges fixed on the compression steel (GSTU and GSTL), were in fair agreement with the readings of strain gauges fixed on the concrete surface at the level of the reinforcement (GCS1), see Figure 2.29.

Most of the readings of the strain gauges fixed on the polymer grids deviated up to $\pm 50\%$ percent from the readings of the steel reinforcement before yielding of the steel. For unknown reasons, some of the strain gauges gave unreasonable results even before yielding. After yielding, all readings varied extremely and the strain gauges became inoperative.

Before yielding of the steel reinforcement, the strain profiles of the middle section in SRC beams were in a fair agreement with those of STRC beams. The strain profiles were not exactly linear, especially at loads near to the yielding loads.

After yielding of the steel reinforcement, the strain gauges fixed on the tensile reinforcement were broken due to high strains developed in the reinforcement. Also, the strain gauges fixed on the concrete became inoperative because of the surface spalling of the concrete.

Figure 3.52 shows the average strain profiles for SRC and STRC beams at 25 kN, 50 kN, 75 kN, 100 kN and 110 kN.

Figure 3.52: Strain Profile for SRC and STRC Beams

3.10.2 Strain Readings in TRC Beams

The strain gauges fixed on Tensar grids became inoperative after the first visible crack occurred. This is due to the high extension in the polymer grids.

3.11 Summary of the Test Results

3.11.1 Loads and Deflections

Loads and deflections at the first visible crack, yielding, ultimate and failure are summarized in Table 3.1.

3.11.2 Cracks in SRC and STRC beams

3.11.2.1 Crack Numbers

Table 3.2 shows the number of cracks for SRC and STRC beams.

3.11.2.2 Maximum Crack Width

Table 3.3 shows the maximum crack width for SRC and STRC beams measured during the tests for SRC and STRC beams at 25 kN, 50 kN, 75 kN and 100 kN.

Table 3.1: Loads and deflections for the beams

Set No.	Beam No	Load at first visible crack (kN)	deflection at first visible crack (mm)	Yield load (kN)	Yield Deflection (mm)	Ultimate load (kN)	Deflection at ultimate load (mm)	Failure load (kN)	Deflection at failure load (mm)
1	B1	15.00	1.30	110.00	13.20	128.50	58.20	126.40	66.40
	B8	15.00	1.10	115.00	13.10	131.00	52.10	131.00	52.10
2	B3	15.00	1.39	118.00	12.90	137.80	77.00	127.00	89.00
	B4	14.00	1.25	120.00	13.40	137.00	83.00	137.00	83.00
3	B6	15.00	1.50	115.00	13.20	139.70	78.40	138.00	103.50
	B6	15.00	1.30	115.00	12.60	136.00	59.00	136.00	59.00
4	B7	15.00	1.50	115.00	13.80	137.40	80.00	125.00	92.80
5	B9	15.00	1.45	115.00	14.00	138.20	67.30	132.60	96.00
	B10	14.00	1.40	120.00	13.90	137.00	88.00	132.00	103.00
6	B11	10.50	1.00			11.75	31.00	9.20	95.00
	B12	10.30	0.90			13.43	166.00	13.43	166.00
7	B13	9.70	0.70			16.70	114.00	14.10	170.00
	B14	7.00	0.50			19.20	162.00	18.50	193.00

**Table 3.2: Crack Numbers for SRC and STRC Beams
at Different Cycles**

Set No.	Beam No.	Number of cracks at 25 kN	Number of cracks at 50 kN	Number of cracks at 75 kN	Number of cracks at 100 kN
1	B1	2	4	10	17
	B8	4	10	19	30
2	B3	3	4	9	13
	B4	1	11	19	24
3	B5	7	10	16	20
	B6	1	1	5	22
4	B7	2	9	16	29
5	B9	11	19	23	30
	B10	7	10	18	22

Table 3.3: Maximum Crack Width for SRC and STRC Beams at Different Cycles

Set No.	Beam No.	Max. crack width at 25 kN (mm)	Max. crack width at 50 kN (mm)	Max. crack width at 75 kN (mm)	Max. crack width at 100 kN (mm)
1	B1	0.30	0.52	0.89	1.15
	B8	0.26	0.40	0.82	0.95
2	B3	0.27	0.43	0.75	1.10
	B4	0.23	0.38	0.70	0.90
3	B5				
	B6	0.30	0.63	0.95	1.27
4	B7	0.20	0.33	0.6	0.85
5	B9	0.21	0.35	0.66	0.90
	B10				

Chapter 4

DISCUSSION OF TEST RESULTS

4.1 Introduction

In this chapter, the test results reported in chapter 3 will be examined to draw relevant conclusions. Loads, deflections, ductility, failure modes and crack characteristics observed for the different sets will be compared.

A possible explanation for the increased ductility observed in the STRC beams will be discussed in chapter 5.

4.2 Deflections for SRC and STRC beams:

4.2.1 General:

There was a very slight difference in the deformations between SRC and STRC beams until yielding of the beams. The first visible crack loads, yielding loads and corresponding deflections were approximately equal for both SRC and STRC beams. The presence of Tensar grids in STRC beams improved the strength for some of these beams but only marginally. However, in all cases there was significant increase

in the ultimate deflection. There was a slight difference in the strength between STRC beams containing two layers of Tensar grid and STRC beams containing four layers of Tensar grid. There was a more significant difference in the deflection - of the order of 20 %. The effect of the position of Tensar grids on the deflections, that is either above the steel reinforcement or under, has been examined. It was believed that the placement of Tensar grids under the steel reinforcement would increase the ultimate deflection in comparison to the case when the grids were placed above the steel reinforcement. This is due to the expected increase in the number of transverse cracks in the case when Tensar grids were placed under the steel reinforcement which can in turn increase the length of the plastic hinge of the beam.

The ductility index for SRC and STRC beams is defined as the ratio of the deflection at failure load to the deflection at yield load. The ductility indices of STRC beams ranged between 6.19 to 7.84 (except for beam B6 which failed in shear), while the ductility indices for the two SRC beams were 5.03 and 3.98 respectively.

The mode of failure of SRC beams was similar to that of STRC beams except for beams B6 and B10. As mentioned in Chapter 2, beam B6 failed in shear and beam B10 failed in compression followed by shear.

Figures 4.1 through 4.4 compare the load-deflection curves of beams in set 1 with those of beams in other sets.

4.2.2 Deflections at first crack:

The loads at first visible crack in SRC and STRC beams varied from 0.1 to 0.12 of the experimental failure loads. Table 4.1 shows the loads at first visible crack and the corresponding deflections of SRC and STRC beams. Also, it compares the loads at first visible crack and the corresponding deflections of STRC beams with the average load at first visible crack and the corresponding average deflection of SRC beams. From Table 4.1, it is seen that the loads at first visible crack for STRC

Figure 4.1: Load-Deflection Curves for Beams in Set 1 and Set 2

Figure 4.2: Load-Deflection Curves for Beams in Set 1 and Set 3

Figure 4.3: Load-Deflection Curves for Beams in Set 1 and Set 4

Figure 4.4: Load-Deflection Curves for Beams in Set 1 and Set 5

beams are approximately equal to the average loads at first visible crack of SRC beams. Also, the differences in the deflections at first crack in SRC and STRC beams are small. It was expected from the predicted calculations that the presence of Tensar grids in STRC beams will decrease the load at first visible crack because of the low modulus of elasticity of Tensar grids especially at the junctions of the polymer grids. This trend was not apparent in the experimental results. From Table 4.1, it can be concluded that the presence of Tensar grids in conventional reinforced concrete beam has no effect on the load at first visible crack or on the corresponding deflections of these beams.

4.2.3 Deflections at yield loads:

The loads at yielding of reinforcement in SRC and STRC beams ranged between 0.83 and 0.92 times the experimental failure loads. The deflections at yield ranged between 0.13 and 0.25 times the maximum deflections at failure. Comparison between the yield loads and the corresponding deflections for SRC and STRC beams are shown in Table 4.2. The comparison is based on the average load and average deflection of beams B1 and B8.

As seen from the data presented in Table 4.2, in comparison to SRC beams the percentage increases in the yield loads for STRC beams are marginal. Also, the percentage increases in the yield deflections are small. This means that the presence of Tensar grids does not affect the yield deformation of conventional beams. On the basis of data presented so far, it can be concluded that the presence of Tensar grids in the conventional beams has no significant effect on the strength or deflection of the beams until yielding of the reinforcement.

**Table 4.1: Comparison between Deflections for SRC
and STRC Beams at First Crack**

Set No.	Beam No.	Load at First Visible Crack (kN)	Deflection at First Visible Crack (mm)	Increase Percentage at First Crack in Load † (%)	Increase Percentage at First Crack in Deflection ‡ (%)
1	B1	15.00	1.30		
	B8	15.00	1.10		
2	B3	15.00	1.39	0.00	15.83
	B4	14.00	1.25	-6.67	4.17
3	B5	15.00	1.50	0.00	25.00
	B6	15.00	1.30	0.00	8.33
4	B7	15.00	1.50	0.00	25.00
5	B9	15.00	1.45	0.00	20.83
	B10	14.00	1.40	-6.67	16.67

† Compared to the average load at first visible crack (15 kN) of SRC beams (B1&B8).

‡ Compared to the average deflection at first visible crack (1.20 mm) of SRC beams (B1&B8).

Table 4.2: Comparison between Deflections of SRC and STRC Beams at Yield Loads

Set No.	Beam No.	Yield Load (kN)	Yie'd Deflection (mm)	Increase Percentage at Yield in Load † (%)	Increase Percentage at Yield in Deflection ‡ (%)
1	B1	110.00	13.20		
	B8	115.00	13.10		
2	B3	118.00	12.90	4.89	-1.90
	B4	120.00	13.40	6.67	1.90
3	B5	115.00	13.20	2.22	0.38
	B6	115.00	12.60	2.22	-4.18
4	B7	115.00	13.80	2.22	4.94
5	B9	115.00	14.00	2.22	6.46
	B10	120.00	13.90	6.67	5.70

† Compared to the average yield load (112.5 kN) of SRC beams (B1&B8).

‡ Compared to the average yield deflection (13.15 mm) of SRC beams (B1&B8).

4.2.4 Deflections at ultimate loads:

Table 4.3 shows the ultimate loads and the deflections at ultimate loads of SRC and SRTC beams. Also, it gives a comparison between the ultimate loads and corresponding deflections for STRC beams and the average ultimate load and corresponding deflection for SRC beams (beams B1 and B8). It should be noted that most of the beams continued to deflect beyond the ultimate with decreasing

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Table 4.2: Comparison between Deflections of SRC and STRC Beams at Yield Loads

Set No.	Beam No.	Yield Load (kN)	Yield Deflection (mm)	Increase Percentage at Yield in Load † (%)	Increase Percentage at Yield in Load ‡ (%)
1	B1	110.00	13.20		
	B8	115.00	13.10		
2	B3	118.00	12.90	4.89	-1.90
	B4	120.00	13.40	6.67	1.90
3	B5	115.00	13.20	2.22	0.38
	B6	115.00	12.60	2.22	-4.18
4	B7	115.00	13.80	2.22	4.94
5	B9	115.00	14.00	2.22	6.46
	B10	120.00	13.90	6.67	5.70

† Compared to the average yield load (112.5 kN) of SRC beams (B1&B8).

‡ Compared to the average yield deflection (13.15 mm) of SRC beams (B1&B8).

4.2.4 Deflections at ultimate loads:

Table 4.3 shows the ultimate loads and the deflections at ultimate loads of SRC and STRC beams. Also, it gives a comparison between the ultimate loads and corresponding deflections for STRC beams and the average ultimate load and corresponding deflection for SRC beams (beams B1 and B8). It should be noted that most of the beams continued to deflect beyond the ultimate with decreasing

load until failure took place.

Table 4.3: Comparison between Deflections for SRC and STRC Beams at Ultimate Loads

Set No.	Beam No.	Ultimate Load (kN)	Deflection at Ultimate Load (mm)	Increase Percentage at Ultimate in Load † (%)	Increase Percentage at Ultimate in Deflection ‡ (%)
1	B1	128.50	58.20		
	B8	131.00	52.10		
2	B3	137.80	77.00	6.20	39.62
	B4	137.00	83.00	5.58	50.50
3	B5	139.70	78.40	7.67	42.16
	B6	136.00	59.00	4.81	6.98
4	B7	137.40	80.00	5.89	45.06
5	B9	138.20	67.30	6.51	22.03
	B10	137.00	88.00	5.58	59.56

† Compared to the average ultimate load (129.75 kN) of SRC beams (B1&B8).

‡ Compared to the average deflection at ultimate load (55.15 mm) of SRC beams (B1&B8).

From Table 4.3, it is seen that the maximum percentage increase of the ultimate load of SRC beams was 7.67 % in comparison to the average ultimate load of SRC beams. The deflection at ultimate load of STRC beams increased by a maximum of 59.56 % over the average deflection at ultimate load for SRC beams. This means that the presence of Tensar grids was very effective in increasing the deflections at

ultimate loads of STRC beams although it caused only a marginal increase in the ultimate strength.

In summary, the following conclusions can be drawn:

- The presence of Tensar grid increased the ultimate loads marginally but it increased the deflections at ultimate loads quite significantly.
- An increase in the number of Tensar layers from two to four did not cause an increase in the strength of the beams but it did cause an increase in the deflection.
- The position of Tensar grids, whether above or under the steel reinforcement, had no significant effect on the ultimate loads or deflections. The reason for this will be discussed later in Chapter 5.

4.2.5 Deflections at failure loads:

From a comparison between the load-deflection curves of SRC and STRC beams shown in Figures 4.1 through 4.4, it can be noted that for most of the beams the failure load is slightly less than the ultimate load. Failure loads of STRC beams varied between 0.98 to 1.072 times the average failure load of beams in set 1. Table 4.4 compares the failure loads and the corresponding deflections for STRC beams and the average failure load and corresponding deflection for SRC beams.

Table 4.4: Comparison between Deflections for SRC and STRC Beams at Failure

Set No.	Beam No.	Failure Load (kN)	Deflection at Failure Load (mm)	Increase Percentage at Failure in Load † (%)	Increase Percentage at Failure in Deflection ‡ (%)
1	B1	126.40	66.40		
	B8	131.00	52.10		
2	B3	127.00	89.00	-1.32	50.21
	B4	137.00	83.00	6.45	40.08
3	B5	138.00	103.50	7.23	74.68
	B6	136.00	59.00	5.67	-0.42
4	B7	125.00	92.80	-2.87	56.62
5	B9	132.60	96.00	3.03	62.02
	B10	132.00	103.00	2.56	73.84

† Compared to the average Failure load (129.75 kN) of SRC beams (B1&B8).

‡ Compared to the average deflection at Failure load (55.15 mm) of SRC beams (B1&B8).

From the results presented in Table 4.4, the following observations can be made:

- The presence of Tensar grids in STRC beams caused a marginal increase in the failure load.
- The presence of Tensar grids in STRC beams caused a significant increase in the maximum deflection.
- The failure load of STRC beams containing two layers of Tensar grid above

the steel reinforcement (set 2) ranged between 1.016 and 1.096 times the failure load for STRC beams containing two layers of Tensar grid under the steel reinforcement (set 4).

- The failure load of STRC beams containing four layers of Tensar grid above the steel reinforcement (set 3) was about 1.04 times the failure load of STRC beams containing two layers of Tensar grid under the steel reinforcement and another two layers of Tensar grid above the steel reinforcement (set 5).
- The maximum midspan deflections of STRC beams containing two layers of Tensar grid (set 2 and set 4) ranged between 1.4 and 1.56 times the average maximum midspan deflection of SRC beams.
- The maximum midspan deflections of STRC beams containing four layers of Tensar grids (set 3 and set 5) ranged between 1.62 and 1.75 times the average maximum midspan deflection of SRC beams, with the exception of beam B6.
- The maximum midspan deflections of STRC beams containing two layers of Tensar grids above the steel reinforcement (set 2) were less than the maximum midspan deflection of STRC beams containing two layers of Tensar grids under the steel reinforcement (set 4). The difference was however small.
- With the exception of beam B6, the maximum midspan deflections of STRC beams containing four layers of Tensar grids above the steel reinforcement were close to the maximum midspan deflections of STRC beams containing two layers of Tensar grids under the steel reinforcement and another two layers of Tensar grids above the steel reinforcement (set 4).
- The maximum midspan deflections of STRC beams containing two layers of Tensar grids (set 2 and set 4) were less than the maximum midspan deflections of STRC beams containing four layers of Tensar grids (set 3 and set 5). The difference was less than 20 %.

4.2.6 Ductility:

The ductility index is defined as the ratio of the deflection at failure load to the deflection at yield load. An increased ductility is a desirable characteristics from the point of view of design.

Table 4.5 shows the variation of the ductility indices for SRC and STRC beams. The ductility indices for STRC beams range from 6.19 to 7.84, except for beam B6 which failed in shear. In comparison, the average ductility index for control beams is 4.5 only.

From the results presented in Table 4.5, the following observations can be made:

- The ductility indices of STRC beams ranged between 1.37 to 1.74 times the average ductility index for control beams.
- The ductility indices of STRC beams containing four layers of Tensar (set 3 and set 5) , with the exception of beam B6, were greater than the ductility indices for STRC beams containing two layers (set 2 and set 4).

**Table 4.5: Comparison between Ductilities for SRC
and STRC Beams**

Set No.	Beam No.	Maximum Deflection Δ_{max} (mm)	Yield Deflection Δ_y (mm)	Ductility Index Δ_{max}/Δ_y (μ)	Increase Percentage in Ductility Index $\uparrow$ (%)
1	B1	66.40	13.20	5.03	
	B8	52.10	13.10	3.98	
2	B3	89.00	12.90	6.90	53.16
	B4	83.00	13.40	6.19	37.40
3	B5	103.50	13.20	7.84	74.03
	B6	59.00	12.60	4.68	3.89
4	B7	92.80	13.8	6.72	49.17
5	B9	96.00	14.00	6.86	52.27
	B10	103.00	13.90	7.41	64.48

$\uparrow$ Compared to the average ductility index (4.50)
of SRC beams (B1 and B8).

4.2.7 Strain readings:

Strain measurements were made at the midspan of the beams to compare the distribution of curvature between SRC beams and STRC beams. It was believed that Tensar grids may affect the distribution of curvature in STRC beams. As was shown in Subsection 3.10.1, the strain profile at the midspan section is nonlinear, especially at higher loads near yielding loads. The strain readings of SRC and STRC beams were very similar until the yield load. This means that the presence of Tensar grids has no effect on the strain profile of STRC beams until yielding. Because of the nonlinearity of strain profile even prior to yielding of reinforcement, the calculated deflection for SRC beams based on linear variation of strain underestimated the actual deflection obtained during the test. This nonlinearity increases after crushing of the concrete in the compression zone. It has been found in a study by Thomas and Sozen [80] and Corley [18] that beyond crushing of the concrete in the compression zone, hinging action at the critical sections gives rise to nonlinear strain profile whose trend is complex.

Because of the nonlinearity of the strain profile, the calculated deflection at ultimate load also underestimated the actual deflection at ultimate load. It should be noted that most of the strain gauges became inoperative after yielding of the tensile steel reinforcement. So, it was not possible to get the actual curvature of the beam at ultimate load.

It is recommended that in any future work Demec gauges (mechanical gauges) be used to get the strain readings until failure loads.

4.2.8 Spread of plasticity:

In order to study the spread of plasticity in the region of maximum moment, it is essential to know the exact curvature at these sections at yielding and ultimate

loads. Because of the nonlinearity of the strain profile before yielding and the missed readings of the strain at ultimate loads, it is impossible to calculate the exact plastic hinge length at the middle of the beams.

Since all SRC and STRC beams were similar in their dimensions and the steel reinforcement and there was no difference in their yield loads, the yield curvatures could be the same. Also, because the presence of Tensar grids resulted in only a marginal increase in the ultimate loads, the curvatures at these loads would be approximately the same for SRC and STRC beams. On the other hand, since the deflections of STRC beams were more than the deflections of SRC beams, it is obvious that the inelastic rotation of STRC beams increased due to the presence of Tensar grids. This means that lengths of plastic hinges for STRC beams were greater than the lengths of plastic hinges for SRC beams under the same load. The reasons for the increased inelastic rotation in STRC beams will be discussed later in Chapter 5.

4.2.9 Conclusions:

Based on the observations made in the preceding sections, the following conclusions can be drawn:

- 1) The presence of Tensar grids in STRC beams has no effect on the deformation behaviour of the beams before yielding of the steel reinforcement.
- 2) Tensar grids impart a marginal increase in the ultimate load of concrete beams.
- 3) The presence of Tensar grids causes a significant increase in the deflection at failure.
- 4) The ductility of STRC beams is increased significantly because of the polymer grids.
- 5) An increase in the number of Tensar grids does not increase the strength of the beams but increases the deflection subsequent to the yielding of tensile steel

reinforcement. The ultimate deflection in the present thesis increased by less than 20 % as number of layers increased from 2 to 4.

- 6) The position of Tensar grids, that is, their placement above or below the steel reinforcement has no significant effect on the strengths or deflections of STRC beams.
- 7) Beams reinforced with high strength steel can be made as ductile as the beams reinforced with ordinary strength steel by using Tensar grids as a secondary reinforcement in conjunction with steel reinforcement on the tension side of the beams.

4.3 Deflections for TRC beams:

4.3.1 General:

The behaviour of this type of beams was different from the behaviour of conventional reinforced concrete beams. The load at first visible crack was lower than those of SRC or STRC beams. It decreased with an increase in the number of Tensar grids.

The slope of the load-deflection curve changed after the cracking load in a manner similar to that of conventional reinforced concrete beams after yielding of the steel reinforcement. Such a behaviour was expected because the stiffness of the beam would reduce significantly after cracks have appeared. Also, the heights of cracks in the TRC beams were large. The depth of the compression zone was small because of the low equilibrium force of the polymer grids in the tension zone. After cracking, the load-deflection curve became alternately ascending and descending. This phenomena was observed for all beams except for beam B11. The deviations in the ultimate loads for beams in the same set were small but were high in the failure loads. The deviations in the deflections were high both at ultimate loads and

at failure loads, especially between beams in set 6. Due to the good interlocking between the concrete and the polymer grids, the bond between the two materials was good. Figure 4.5 compares between the load-deflection curves for beams in sets 6 and 7.

4.3.2 Deflections at first crack:

The loads at first visible crack of these beams were much smaller in comparison to those of SRC or STRC beams. An increase in the number of layers resulted in a decrease in the load at first visible crack. This trend was expected because of the low Young's modulus of elasticity of Tensar grids. In fact, the transformed moment of inertia of the concrete section containing Tensar grids would be less than the moment of inertia of the concrete section without Tensar grids. Also, most of the initial cracks occurred at the junctions of the polymer grids where the total thickness is more than the thickness of the ribs away from the junctions.

Table 4.6 compares the load at first visible crack and the corresponding deflections for beams in set 7 with the average load at first visible crack and the corresponding deflection for beams in set 6 (beam B11 and B12). As seen from Table 4.6, the load at first visible crack for beams B13 and B14 was lower by 6.73 % and 32.26 % respectively in comparison to the average load for beams B11 and B12. Also, the deflections for beam B13 and B14 was 26.31 % and 47.37 % less than the average deflection for beams B11 and B12.

Figure 4.5: Load-Deflection Curves for Beams in Set 6 and Set 7

Table 4.6: Comparison between Deflections for TRC Beams at First Visible Crack

Set No.	Beam No.	Load at First Visible Crack (kN)	Deflection at First Visible Crack (mm)	Decrease Percentage at First Crack in Load † (%)	Decrease Percentage at First Crack in Deflection ‡ (%)
6	B11	10.50	1.00		
	B12	10.30	0.90		
7	B13	9.70	0.70	6.73	26.31
	B14	7.0	0.50	32.26	47.37

† Compared to the average load at first visible crack

load (10.40 kN) of beams in set 6 (B11 & B12)

‡ Compared to the average deflection (0.95 mm) of beams in set 6 (B11 and B12).

4.3.3 Deformations at ultimate loads:

The ultimate loads of TRC beams ranged between 0.09 to 0.15 times the ultimate loads of SRC beams. This was because of the low Young's modulus of the polymer grids compared to the Young's modulus of the steel reinforcement. Table 4.7 shows the ultimate loads and corresponding deflections of TRC beams. Also, it gives the average load and deflection for each set.

Table 4.7: Comparison between Deflections for TRC Beams at Ultimate Loads

Set No.	Beam No.	Ultimate Load (kN)	Deflection at Ultimate Load (mm)	Average Load (kN)	Average Deflection (mm)
6	B11	11.75	31.00	12.59	98.50
	B12	13.43	166.00		
7	B13	16.70	114.00	17.95	138.00
	B14	19.20	162.00		

On the basis of results presented in Table 4.7, the following observations can be made:

- The ultimate load increases with an increase in the content of polymer grids. The average ultimate load of set 7 is higher than the average ultimate load of set 6 by 42.57 %.
- Large differences can exist in deflections between similar beams in the same set. The deflection of beam B12 is 5.35 times the deflection of beam B11 and the deflection of beam B14 is 1.42 times the deflection of beam B13.
- The deflection of beam B12 (reinforced with two layers of Tensar) is higher than the deflection of beams B13 and B14 (reinforced with four layers of Tensar).

It can be observed from Table 4.5 that the effect of the number of layers on the beam deflection at ultimate load does not follow a specific trend.

4.3.4 Deflections at failure loads:

Table 4.8 shows the failure loads and deflections for each TRC beams and the average failure load and deflection for each set.

Table 4.8: Comparison between Deflections for TRC Beams at Failure Loads

Set No.	Beam No.	Failure Load (kN)	Deflection at Failure Load (mm)	Average Load (kN)	Average Deflection (mm)
6	B11	9.20	95.00	11.31	130.50
	B12	13.43	166.00		
7	B13	14.10	170.00	16.30	181.50
	B14	18.50	193.00		

On the basis of results presented in Table 4.8, the following observations can be made:

- Failure load increases with an increase in the number of grids.
- Differences exist in the failure loads and deflections even for similar beams. The failure load of beam B12 is 1.46 times the failure load beam B11 and the failure load for beam B14 is 1.31 times the failure load of beam B13.
- Differences exist in the maximum deflections even for similar beams. The maximum deflection of beam B12 is 1.75 times the maximum deflection of beam B11. The maximum deflection of beam B14 is 1.14 times the

maximum deflection of beam B13.

Again, it can be observed from Figure 4.5 that the effect of the number of layers on the beam deflections at failure loads does not follow any specific trend.

4.3.5 Ductility:

The ductility index for TRC beams is defined as the ratio of the deflection at the failure load to the deflection at the end of initial linear portion of the load-deflection responses.

Table 4.9 shows the variation of the ductility indices for TRC beams.

**Table 4.9: Comparison between Ductilities
for TRC Beams**

Set No.	Beam No.	Maximum Deflection Δ_{max} (mm)	Yielding Deflection Δ_y (mm)	Ductility Index Δ_{max} / Δ_y (μ)	Average Ductility Index
6	B11	95.00	1.00	95.00	139.72
	B12	166.00	0.90	184.44	
7	B13	170.00	0.70	242.86	314.43
	B14	193.00	0.50	386.00	

The ductility indices for TRC beams range between 95 to 386. These values are extremely high when compared with the conventional reinforced concrete beams.

It is evident that the low values of yield deflections and the high values of failure deflections are the reasons of these extremely high values of ductility indices.

The ductility indices for beams in set 7 were higher than those of beams in set 6. there was two reasons for that:

- i) The yielding deflections of beams in set 7 are lower than those in set 6.
- ii) The maximum deflection of beams in set 7 are higher than those in set 6.

It is evident that there is a big difference in the ductility indices between beams in the same set. Again, due to the big deviation in the results, it is difficult to draw any conclusions about the effect of the polymer content on the ductility.

4.3.6 Strain readings:

Most of the strain gauges fixed on the polymer grids became inoperative immediately after the first crack. Because of this, it is not possible to study the curvature of TRC beams .

4.3.7 Conclusions:

Based on the observations made in the preceding sections, the following conclusions can be drawn:

- 1) The use of Tensar grids as a main reinforcement in concrete beams leads to a low value of load at the first visible crack.
- 2) Increasing the number of Tensar grids increases the strength of the beams.
- 3) Increasing the number of the layers does not have any well defined effect on beam deflections. However this conclusion needs to be investigated further.

4.4 Cracking behaviour of SRC and STRC beams:

4.4.1 General:

From the tests results, it is evident that the number of cracks in beams containing Tensar grids was significantly larger than the number of cracks in conventional reinforced beam, B1. An Increase in the number of cracks led to a decrease in the spacing between cracks leading to a regular uniform crack pattern. It is well known that a regular crack spacing implies a smaller crack width. This was clear in the measured maximum cracks width. The maximum cracks width in beams containing Tensar grids were less than those observed in beams without Tensar grids (conventional beams). Increasing the number of polymer grids resulted in an increase in the number of cracks and a decrease in the maximum crack width.

4.4.2 Number of cracks:

A comparison between the number of cracks at different cycles for SRC and STRC beams is shown in Figure 4.6. On the basis of the data presented in the figure, the following observations can be made:

- For all STRC beams, with the exception of beams B3 and B6, the number of cracks was larger than that of beam B1 under the same load.
- In spite of the discontinuity of the polymer grids in beam B8, the beam showed increased number of cracks.
- Beam B7, containing two layers of Tensar grids placed under the steel reinforcement, had a larger number of cracks than beam B3, but it had a lesser number of the cracks than beam B4. Beams B3 and B4 contain two layers Tensar of grids placed above the steel reinforcement.
- For the beams containing four layers of Tensar grids, beams B9 and B10,

which had two layers of Tensar grids placed under the steel reinforcement and two layers placed above the steel reinforcement, showed an increase in the number of cracks when compared to beams B5 and B6, containing four layers of Tensar grids placed above the steel reinforcement.

4.4.3 Maximum crack width:

An increase in the number of cracks is expected to be accompanied by a decrease in the crack spacing and a decrease in the crack width. This trend was evident in the measured maximum crack width for beams containing Tensar grids, even when the grids were discontinued (beam B8). Figure 4.7 compares the maximum cracks width at different loads for SRC and STRC beams.

On the basis of data presented in the figure, the following observations can be made:

- In most of the beams, the maximum crack width was less than that of beam B1 under the same load.
- In general, at working load a reduction of 30 to 60 % in the maximum crack width was observed in beams containing Tensar grids when compared to beams without Tensar, beam B1.
- Beam B6 showed an increase in the maximum crack width when compared with beam B1. This was presumably due to incorrect positioning of stirrups in beam B6.
- The maximum crack width of beam B7 (having two layers of Tensar grids placed under the steel reinforcement) was less than those of beams B3 and beam B4, having two layers of Tensar grids placed above the steel reinforcement.
- The maximum crack width of beam B9 (having two layers of Tensar grids placed under the steel reinforcement and another two layers placed above

the steel) was less than those of beams B3 and beam B4 (having two layers Tensor grids above the steel reinforcement).

4.4.4 Conclusions:

Based on the observations presented in this section, the following conclusions can be drawn:

- 1) The presence of Tensor grids in reinforced concrete beams on the tension side provides a better crack control.
- 2) In general, the maximum crack width in beams containing Tensor grids is less than that in conventional beams.
- 3) Additional work is required to study the following:
 - (i) The effect of the number of layers on crack spacing and crack width.
 - (ii) The effect of the position of the Tensor grids, that is either placed above the tensile steel or under the tensile steel.
 - (ii) The effect of the dimensions of the Tensor grids on crack pattern and width.

4.5 Cracking behaviour of TRC beams:

It was observed from the tests that the cracks in most of the TRC beams initiated at the intersections of the polymer ribs (junctions). These cracks became evident at a low load due to the low modulus of elasticity of the polymer grids and became wide after that. An increase in the number of layers increased the number of cracks. When cracks occur, the tensile force is carried by the polymer grids at such cracks. Between these cracks, concrete carries some tension force by bond. The bond between the concrete and the polymer grids is developed by the good

interlocking between the concrete and the grids. Due to the elastic behaviour of the polymer grids, the beams showed a good recovery on unloading even when the cracks were very wide. For a conventional reinforced concrete beam, only a small recovery occurs when the beam is unloaded after yielding of the steel.

**Figure 4.6: Cracks Number at Different Cycles
for SRC and STRC Beams.**

Figure 4.7: Maximum Crack Width at Different Cycles for SRC and STRC beams

Chapter 5

A CONCEPTUAL MODEL FOR INCREASED DUCTILITY IN STRC BEAMS

5.1 Introduction

Test results revealed that STRC beams had more ductility than SRC beams. There was however no significant difference in the deflections of SRC beams and STRC beams before yielding. Also, the yield loads of SRC and STRC beams were approximately equal. This means that, the presence of Tensar grids in STRC beams has no significant effect on the strength or deflection of STRC beams until yielding of the tension steel. However, based on the deflection at failure, the ductility of STRC beams was 75 % higher than that of SRC beams. The STRC beams also exhibited a higher strength in comparison to SRC beams, but the difference was less than 7 %.

A conceptual model to explain the increase in ductility caused by the presence of polymer grids is proposed here. At the present, these are only hypothesis which need to be verified by more extensive tests.

As mentioned in Chapter 4, an increase in the maximum deflections in STRC beams means an increase in the inelastic rotation of these beams. It is believed that

the increased inelastic rotation in STRC beams was due to the early deterioration of bond between the steel bar and the surrounding concrete after yielding of the steel. It has been observed by Parameswaran et al. [53] that bond stress between reinforced bar and the surrounding concrete is a critical factor in the inelastic rotation of flexural beams. A higher bond strength reduces the inelastic rotation while higher bond stresses or early bond deterioration increases it. The presence of Tensar grids affects the bond stresses developed between the deformed steel bar and the surrounding concrete. The bond stress in STRC beams is higher in comparison to that in SRC beams under the same level of load. This can lead to an early bond failure between the steel and the concrete. When bond failure (deterioration) occurs (locally or totally), the tensile force in the steel increases. This increases the length of the yielded portion of steel at the middle of the beam which in turn causes an increase in the inelastic rotation of the beam before failure of tensile steel. The presence of Tensar grids can also lead to a change from the beam action mechanism to the arch action mechanism if a total deterioration of bond between steel and concrete occurs before fracture of the tension steel.

5.2 The State of the Stresses and Cracks in the Concrete Surrounding the Deformed Steel Bar

In order to study the effect of Tensar grids on the bond stresses between the steel and concrete, it is important to understand the state of stresses and cracks in the concrete around the deformed steel bar.

After the formation of flexural cracks (transverse cracks) in the concrete around the steel bar, the steel stresses increase at these cracks and decrease between them. The reduction of steel stresses between cracks is accompanied by a transfer of an equivalent force from the steel bar to the surrounding concrete by bond forces. As observed by Goto [29], stress transfer occurs through inclined compressive force

radiating out from the bar. Goto showed that the radiating forces cause internal secondary cracks to form in addition to the primary cracks (transverse cracks) which are visible on the surface of the member. Goto carried out some tests where he injected ink into concrete specimens subjected to tensile stress as shown in Figure 5.1. By cutting the specimens after the test, it was possible to study the pattern of internal cracks. These internal cracks are inclined toward the primary cracks as shown in Figure 5.2. The movement of the bar is resisted by compressive struts in the concrete acting on the lugs as shown in Figure 5.2. The outer ends of these struts are equilibrated by a hoop tension in the surrounding concrete. If this hoop tension exceeds the tensile capacity of the concrete, longitudinal cracks develop.

These ideas were developed into a unified theory for bond by Tephers [79]. Tephers stated that "the inclined bond forces on the lugs can be resolved into radial components and tangential components as shown in Figure 5.2. The radial components will be balanced by tensile stressed concrete rings as shown in Figure 5.3. The concrete cover over the reinforcement usually represents the thinnest part of these rings. When the ring is stressed to rupture, it fails at this point, and longitudinal cracks appear. However, these cracks may start as internal longitudinal cracks which cannot be seen on the surface of the concrete before the ultimate load capacity of the ring is reached. When a longitudinal crack appears, displacement between the bar and the concrete increases considerably in comparison with that before cracking and leads to evenly distributed bond stresses along the cover cracked anchorage length. The radial components of the anchorage force then impose a load on the concrete cantilevers, which are the remnants of the cracked concrete ring, surrounding the reinforcing bar. When these cantilevers are stressed to their ultimate capacity, they fail according to the minimum stressed surface failure pattern".

Tephers stated that "the radial stress due to bond action on the concrete, can be regarded as a hydraulic pressure acting on a thick wall concrete ring. This concrete ring approximately represents the effect of the surrounding concrete. The wall thickness of the ring is determined by the smallest possible dimension, the least

**Figure 5.1: Pull out Test on Deformed Bar,
after Goto [29]**

a) Forces on Concrete

b) Longitudinal Section of axially loaded specimen

c) Cross-section

Figure 5.2: Deformation of Concrete Around Ribbed Reinforcing Bar (Schematic Diagram), after Goto [29]

Figure 5.3: Schematic Representation of How Bond Force are Balanced by Tensile Stress Rings in Concrete in Anchorage Zone, after Tephers [79]

Figure 5.4: Section through Part of the Tensile Zone in A Reinforced Concrete Beam. The Dimensions of the Concrete Ring are Determined by the Lowest Value of C_x and C_y , after Tephers [79]

of the concrete covers, (Figure 5.4)".

Tepfers suggested three theories that can be applied to the concrete ring for determination of the cracking resistance. Based on the distribution of the stresses, these theories are referred to as:

- (1) Elastic, as shown in Figure 5.5a.
- (2) Partly cracked elastic, as shown in Figure 5.5b.
- (3) Plastic, as shown in Figure 5.5c.

Figure 5.5 shows the distribution of the tensile ring stresses according to the three theories. Tepfers compared some experimental results obtained by himself and others with the suggested theories as shown in Figure 5.6. On the basis of such comparison, he concluded as follows:

- i) If C_x / d is approximately equal to one, the cover can be assumed to act plastically. This means that the cylinder will not break until the stresses in the tangential direction at every part of the cylinder have reached the ultimate tensile concrete stress, f_r .
- ii) If C_x / d is more than one, the concrete cover will be assumed to act between the plastic stage and partly cracked elastic stage.

In the above,

C_x is the minimum concrete cover,

d is the bar diameter,

and f_r is the tensile strength of the concrete.

5.3 Effect of Tensar Grids on the Ductility of STRC Beams

5.3.1 Bond Stresses

Figure 5.7 shows the concrete rings, suggested by Tepfers [79], around a steel

Figure 5.5: Distribution of Tensile Ring Stresses
(a) Elastic, (b) Partly Cracked Elastic, and (c) Plastic
, after Tephers [79]

Figure 5.6: Effect of Thickness of Concrete Cover upon Bond Capacity of the Pull-out Specimens on Occurrence of Longitudinal Cracking in the Concrete Cover, after Tephers [79]

bar in SRC beams and Figures 5.8 through 5.11 show the concrete ring around a bar in STRC beams at the junctions of Tensar grids (rib intersections). Since the side concrete cover, C_x , is less than the bottom concrete cover C_y , the radius of the concrete ring will be governed by C_x . As seen in Figures 5.8 through 5.11, the presence of polymer grids will decrease the effective area in the concrete ring, especially at the intersections of ribs (junctions). The decrease in the effective area imposes increased bearing bond stress, f_b , on the concrete ring. These bearing bond stresses were clearly higher than that in beam B1 (see Figure 5.7) under the same load level.

The effective areas of the concrete rings around the bars are shown in Table 5.1. These areas are calculated according to the following assumptions:

- 1) The steel bar is a corner bar (i.e., at the corner of the stirrup).
- 2) The grid width is 100 mm.
- 3) The concrete ring radius in STRC beams will be similar to that of beam B1.
- 4) The area is the net area of the concrete ring minus the area of the rectangular portion of Tensar grids at junctions.

It should be noted that the bearing bond stresses in STRC beams will not be uniform around the steel bar. The assumption made by Tephers [79] that the radial component of bearing bond stress, f_{rcb} , acts like a hydraulic pressure inside a concrete ring tube will not be the same when polymer grids are present. Therefore, the radial component stress, f_{rcb} , will not be uniform for STRC beams as in the case of beam B1 (see Figure 5.12).

Figure 5.12b shows the uniform radial stress distribution suggested by Tephers [79] when Tensar grids are not used, while Figure 5.12c shows the expected nonuniform radial stress distribution as proposed by the model when Tensar grids are used.

Due to the assumptions proposed by Tephers [79] suggesting that the radial

Figure 5.7: Concrete Ring in Beam B1

Figure 5.8: Concrete Ring in Beams B3 and B4

Figure 5.9: Concrete Ring in Beams R5 and B6

Figure 5.10: Concrete ring for Beams B7 and B8

Figure 5.11: Concrete Ring in Beams B9 and B10

a) Concrete Stresses at a steel lug

b) Uniform Radial Stresses when Tensar Grids are not used

c) Nonuniform Radial Stresses when Tensar Grids are used

Figure 5.12: Distribution of Radial Stresses around the Steel Bar

stress in concrete around the bar can be regarded as a hydraulic pressure acting on a thick wall concrete ring, therefore, Tephers theories can not be applied to STRC beams without modification. In other words, in case of STRC beams, the affected arch around the bar will not be a circle. For the purpose of a quantitative explanation of the effect of bond deterioration on the ductility, the concrete ring shape has been assumed to be circular. Determination of the exact shape of the concrete ring in the presence of Tensar grids requires further investigations.

**Table 5.1: Effective Areas of Concrete Rings
and theoretical Bearing Bond Stresses
at The Middle of
SRC and STRC Beams**

Set No.	Beam No.	Effective concrete ring area (mm ²)	Theoretical bearing bond stress
1	B1	1825	f_b
	B8	1430	$1.27 f_b$
2	B3 & B4	1430	$1.27 f_b$
3	B5 & B6	1430	$1.27 f_b$
4	B7	1430	$1.27 f_b$
5	B9 & B10	1035	$1.76 f_b$

Note: effective areas of concrete rings and theoretical bearing bond stresses for STRC beams and beam B8 are calculated at the junction of Tensar grid near the middle of the beam.

As shown in Table 5.1, the calculated bearing bond stress in STRC beams is higher than the bearing bond stress in beam B1. It ranges between 1.27 to 1.76

the bearing bond stress in beam B1. As mentioned before, splitting tensile stress, σ_t , develops due to the radial component of bearing bond stress (see Figures 5.2, 5.3 and 5.4). This will cause the maximum tensile stress, f_r , in the concrete ring of STRC beams to occur at an early stage when compared with that in beam B1.

After yielding of the tension steel, a high strain will develop in the steel. A similar strain will exist in the polymer grids. Due to the high strain in the grids after yielding, the force in the grids will increase with increasing strain (see the stress-strain curve of the grids in Figures 2.21 and 2.24). This increased force will cause the following :

- a) At the intersection of the ribs (junctions), there will be a bearing stress, f_{bc} , acting on the interlocked concrete in front of the junction, as shown in Figure 5.14. This bearing stress tends to push the concrete against the steel lugs. As mentioned before, the bearing bond stress, f_b , increases at the junctions of polymer grids due to the decreased area of the concrete ring around the steel bar (see Table 5.1). The presence of bearing stress at the junctions, f_{bc} , will increase the bearing bond stress, f_b , in front of the steel lugs. An increase in the bearing bond stress, f_b , will increase both the shearing bond stress and the splitting bond stress, σ_t (see Figure 5.4).
- b) Between junctions, as the tensile strain increases in the grid, the grid tends to contract as shown in Figure 5.14. This contraction will impose a horizontal bearing stress, f_{br} , acting on the interlocked concrete besides the ribs as shown in Figure 5.13. This stress tends to separate the concrete ring from the steel bar. The function of this bearing stress is similar to the function of the radial component, f_{rcb} , shown in Figure 5.4. Due to the increased bearing stress, f_{br} , the splitting stresses in the concrete ring, σ_t , will increase causing longitudinal cracks at early stages. It should be noted that the longitudinal cracks may not appear on the outer surface of the concrete cover.

The bearing stresses f_{bc} and f_{br} , as well as the increased bearing stress f_b seem to play a major role in causing early bond deterioration. The increased value of f_b

Figure 5.13: Bearing Stresses on the Interlocked Concrete in A Polymer Grid Stressed under Tension

Figure 5.14: Contraction of A Polymer Grid Stressed under Tension

alone, i.e. in the absence of f_{bc} and f_{br} , is not expected to cause bond deterioration and hence increased beam ductility, unless the splitting bond stress, σ_t , in the concrete ring reaches the maximum tensile stress, f_r . Beam B8 had discontinuous Tensar grids at the middle of the beam which was expected to increase the value of f_b . Due to the discontinuity of Tensar grids, the tensile stress in the grids at midspan was very small. Hence, the bearing stresses f_{bc} and f_{br} were not expected to be fully mobilized near the midspan. No increase in ductility was observed in beam B8 (Table 4.5). This result supports the idea that the mobilization of the bearing stresses f_{bc} and f_{br} together with f_b is quite necessary for early bond deterioration. This idea is again supported by results from beams B5, B9, and B10, containing the same number of polymer grids. Therefore the bearing stresses f_{bc} and f_{br} will be the same for these beams. Location of Tensar grids with respect to the concrete ring around the steel bar means a higher value of f_b in beams B9 and B10 comparing with that in beam B5 (see Table 5.1). For beams B9 and B10 containing two layers of Tensar grids above and under the steel bar, all Tensar grids were within the concrete ring. For beam B5, containing four layers of Tensar grids above the steel bar, part of the grids was outside the concrete ring. This arrangement means 27% increase in the bearing stress, f_b , with respect to beam B1. In the case of beams B9 and B10, an increase of 76% in the bearing stress, f_b , with respect to beam B1 was computed. Test results showed similar ductility for these beams. This took place irrespective of the high value of f_b in beams B9 and B10. This means that the bearing stresses f_{bc} and f_{br} are crucial for increased ductility.

According to Tepfers [79], the concrete ring in the test program should behave in a plastic manner when C_x/d is approximately equal to one (see Figures 5.7 through 5.11). It means that the longitudinal cracks will appear on the outer surface of the concrete ring. SRC and STRC beams showed longitudinal cracks on the side of the beam, after yielding of the tension steel where all the beams had $C_x/d \simeq 1$.

The bearing stresses, f_{bc} and f_{br} , acting on the interlocked concrete depend on

the thickness of the bearing area, i.e., the number of the polymer grids. Increasing the number of the polymer grids is expected to increase the bearing area in front of the junctions or at the side of the ribs. As mentioned above, this will cause the splitting tensile stresses in the concrete ring, σ_t , to increase, leading to an early bond deterioration and subsequently increased ductility. This was observed when four layers of Tensar grids were used instead of two layers.

5.3.2 Bond Failure and Failure Mechanisms

The interlocking stresses in STRC beams lead to a failure of bond between the steel and the surrounding concrete at early stages of loading after yielding of the tension steel. Bond failure can occur locally or totally. Local bond failure occurs early at the junctions of the polymer grid ribs near the critical section at the middle of the beam. Total bond deterioration may follow the local bond failure. This will happen if the bond failure at the junctions extends to the ribs between the junctions. Figure 5.15 shows the bottom of beam B10 after failure. From the figure, it is obvious that complete bond deterioration between the steel and the concrete occurred in this case. When the bond between the steel bar and the surrounding concrete fails, two different kinds of structural mechanisms may exist: the beam action mechanism and the arch action mechanism. These mechanisms depend on the degree of the bond deterioration, and whether it is local or total.

- a) **Beam action mechanism:** This mechanism occurs if the bond deterioration happens at the junctions of the polymer grid ribs only and there is no bond failure at ribs between intersections. Due to the high shear stresses after yielding of the steel, number of shear cracks (diagonal tension cracks) will increase. These cracks will extend near the middle of the beams for a longer distance in STRC beams than that in SRC beams because of the bond failure at the polymer's intersections. After bond failure at the intersections of the polymer grids, shear cracks will extend from these intersections toward the point load, P , as shown in Figure 5.16a. The diagonal tension cracks at the middle of

the beams cause a redistribution of the bending moment as shown in Figure 5.16. This results in a spreading of the plastic hinge length and therefore an increase in the inelastic rotation of the beam. In the test program, this failure mechanism occurred only in SRC beams, B1 and B8. The difference between STRC and SRC beams is in the distance that diagonal tension cracks spread at the middle of the beams.

b) **Arch action mechanism:** This mechanism follows the beam action mechanism. It should be noted that the transition from beam action mechanism to arch action mechanism is gradual. It will happen only if a complete bond deterioration between the steel and the concrete occurs for a distance at the middle of the beam. The steel bar in arch action mechanism will behave as a tie (see Figure 5.17). This means that the entire steel bar is yielding along the deteriorated bond length. The inelastic rotation will increase significantly in this mechanism compared to that in beam action mechanism.

In the test program, the arch action mechanism occurred in all STRC beams, except for beam B6. For STRC beams, after crushing of the concrete cover in the compression zone, the available area of the concrete under the bearing plate became too small to resist the compression force in conjunction with the compression steel. Hence, most of the compression force is resisted by the compression steel only. If the strength of the compression steel was enough, the tension steel would continue yielding until one of the following occurs:

- Tension failure of the tensile steel before yielding of the compression steel. This occurred in beams B3, B4, B5, B7, and B9.
- Yielding of the compression steel before tension failure in the tensile steel, as in the case of beam B10.

Figure 5.15: Close-up of the Bottom of Midspan Section of Beam B10 Shows the Bond Failure

a) Beam with diagonal cracks

b) Bending moment diagram
(without diagonal tension cracks)

c) Bending moment diagram
(with diagonal tension cracks)

Figure 5.16: Beam Action Mechanism

Figure 5.17: Arch Action Mechanism

5.3.3 Parameters Influencing the Bond Stress and Bond Failure in STRC Beams

In addition to the parameters that influence the bond stresses in a conventional reinforced beam, there are others that affect such stresses in STRC beams. These parameters are:

- 1) The thickness of the polymer grid.
- 2) The number of grid layers.
- 3) The geometry of the polymer grid.

The effect of these parameters as observed from the tests described in this work can be summarized as follows:

- 1) The thickness of the polymer grids: Increasing the thickness of polymer grids decreases the effective area of the concrete ring which increases the bearing bond stress, f_b . It also increases the bearing stresses, f_{bc} and f_{br} , which are the main factors that cause for bond deterioration. It can be concluded that beams reinforced with thick polymer grids will have early bond deterioration after yielding of the steel reinforcement when compared to those reinforced with thin polymer grids. Consequently, the former will also have a higher ductility.
- 2) The number of polymer layers: Increasing the number of polymer layers will increase the total thickness of the grids. As mentioned above, increased thickness causes early bond failure. It should be noted that the increased thickness at the junctions may not be effective in increasing the bond stress if the layers exist outside the concrete ring.
- 3) The geometry of the layers: The distance between the transverse ribs plays an important part in the bond failure. As mentioned before, bond failure begins at junctions, and then extends to areas between the junctions. Decreasing the distance between the junctions (transverse ribs) in the polymer grids means increasing the number of these junctions, which is expected to lead to early

bond deterioration. It is recommended that in any future work this effect may be studied by experimenting with other types of Tensar grids.

5.3.4 Conclusions

Based on the discussions presented in Chapter 4 and the proposed conceptual model, the following conclusions can be drawn:

- The maximum deflections for beams in set 2 and set 4 were approximately equal. It is possible to conclude that the position of the polymer layers, whether they are placed above the steel reinforcement or under the steel reinforcement has no effect on the ductility of the beam. As mentioned in subsection 5.3.1, the percentage increase in the bearing bond stress is independent of the position of grids. Also, the values of bearing stresses, f_{bc} and f_{br} , are independent of the position of grids. So, the length of the bond failure will be approximately the same whether the grids are placed above or below the steel.
- The maximum deflections for beams containing four layers of Tensar grids, sets 3 and 5, were higher than those of beams containing two layers, sets 2 and 4. Again, in the case of beams with four layers, the bond stresses, f_{bc} and f_{br} , will be higher than those in the case of beams with two layers. This will lead to an increase in the length of bond deterioration at the middle of the beam, and hence to an increase in the deflections of these beams. However the increase in the deflections was not high, being less than 20%.
- There was no difference in the maximum deflections between beam B5, containing four layers of Tensar grids above the steel and beams B9 and B10, containing two layers above the steel and another two layers under the steel. However, as shown in Table 5.1, there may be a difference in the bond stress. Since the total thickness of the polymer grids for these beams (B5, B9 and B10) will be the same, the bearing stresses on the interlocked concrete, f_{bc} and f_{br} , in beam B5 will be equal to those in beams B9 and

B10. As mentioned above, these bearing stresses, f_{bc} and f_{br} , are the main factors contributing in bond deterioration.

- The presence of Tensar grids in beam B8 was not effective in increasing the ductility because of the discontinuity of the grids at the middle section. The tensile stresses in the grids near the middle were very small. Therefore, the bearing stresses f_{bc} and f_{br} were not fully mobilized at the middle of the beam. These stresses being the main factors in improving ductility. Beam B8 did not show an increased ductility when compared with beam B1.

Chapter 6

SUMMARY AND CONCLUSIONS

6.1 Summary:

The research work presented in this thesis was motivated by a desire to explore the possibility of using new materials to produce a more efficient and reliable engineering system. Conventional steel reinforced beams have been used in many applications for decades. However, researchers as well as engineers have devoted most of their efforts to increase the ductility of concrete structures and improving their service performance. The work presented in this thesis deals with a new approach to improve the ductility. It investigates the use of a relatively high strength tensile strength Tensar polymer grid (*UX-1600*) for the purpose of improving the ductility of concrete beams.

The experimental program reported here included testing of thirteen full scale rectangular beams $150 \times 300 \times 2200$ mm in size. These beams were reinforced with steel, polymer grids, and combination of both. Test results demonstrated that use of polymer grids did result in an improvement in ductility. This increase was as much as 1.75 times that in steel reinforced beams.

The use of polymer grids also led to a better crack control which is very important in the serviceability stage of concrete beams especially when high strength steel is used.

Finally, a conceptual model was presented to explain the increased ductility when polymer grids are used as a secondary reinforcement in conjunction with steel reinforcement.

While these results confirmed the hypothesis of this thesis, more research work is needed to realize the potential of this new technology.

6.2 Conclusions:

The following conclusions can be drawn from the tests carried out in this research program:-

- (1) The use of Tensar grids as a secondary reinforcement in conjunction with the steel reinforcement on the tension side of reinforced concrete beams has the following effects:
 - i) Tensar grids do not change the deformation behaviour of the beams prior to the yielding of the tensile reinforcement.
 - ii) Tensar grids cause a marginal increase in the ultimate loads of the beams. The increase was less than 7% in the tests carried out.
 - iii) Presence of Tensar grids increases the maximum deflections significantly. The percentage increase in the maximum deflection reached 75% when compared to that of conventional reinforced beams.
 - iv) Tensar grid reinforcement improves the ductility significantly. The ductility increased by 75% over that of conventional reinforced beams.
 - v) The presence of polymer grids as supplementary tension reinforcement leads

to bond deterioration between the steel and the concrete, locally or totally. Such bond deterioration increases the inelastic rotation of Tensar and steel reinforced beams.

- vi) Use of Tensar reinforcement allows a better crack control. The crack spacing and crack width both decreased significantly in Tensar and steel reinforced beams, when compared to conventional steel reinforced beams.
 - vii) Location of Tensar grids in Tensar and steel reinforced beams had no effect on their behaviour. Thus beams with equal amount of grid reinforcement behaved in similar manner whether the grids were placed above or below the steel reinforcement. It should be noted that this observation is true for the loading condition adopted in this testing program.
 - viii) Increasing the number of Tensar grids increases the ductility of the beams.
- (2) Beams reinforced with high strength steel with a yield strength of 610 MPa can be made as ductile as the beams reinforced with ordinary reinforced steel by using polymer grids as supplementary reinforcement.
- (3) The use of Tensar grids alone as the main reinforcement in concrete beams is ineffective in increasing load carrying capacity of such beams. Such beams have low first visible crack loads and their strength is low. In the tests reported here, these strengths varied between 0.13 and 0.15 times the strength of conventional steel reinforced beams. An increase in the amount of grid reinforcement did increase the strength of the beam. The Tensar reinforced beam did show good recovery after unloading, so that the cracks induced during loading reduced to a small size after the load was removed. Test results showed that a crack width of 15 mm decreased to 4 mm 5 minutes after unloading.

6.3 Recommendations for Future Work:

The work presented in this thesis can be considered as the first of its kind in the field of reinforced concrete. The reluctance of many researchers to consider plastic meshes as a reinforcement element in concrete systems is due to the following reasons:

- 1) Doubts about the bond between the plastic grids and concrete.
- 2) The perception that low modulus of elasticity material could not provide any significant advantage over steel reinforced concrete.
- 3) The complexity associated with the modeling of such systems.

While the results of the work presented in this thesis have provided answers to some of these concerns, more research work is needed in the following areas:

- 1) The effect of the number of the layers on crack spacing and crack width should be studied.
- 2) The effect of the dimensions of the layers on crack pattern and crack width needs to be investigated.
- 3) Influence of rib dimensions of steel bar on bond failure when Tensar grids are used as a secondary reinforcement needs examination.
- 4) The behaviour of STRC beams under impact and cyclic loading should be investigated. Clearly, more advantages could be realized when these polymer grids are used as a secondary reinforcement in conjunction with steel reinforcement for concrete beams subjected to impact and cyclic loadings. The use of the polymer grids in earthquake regions can provide an attractive alternative to other more expensive solutions.
- 5) The effect of concrete strength on the behavior of STRC beams should be studied.
- 6) The effect of the concrete cover on the ductility of STRC beams needs exami-

nation.

- 7) The use of Tensar grids as stirrups in concrete beams needs examination.
- 8) Bond between different types of polymer grid and concrete mixes needs studying. More concentrated effort should be focused on developing a standard test to evaluate the bond between the grid and the concrete. This research should lead to a clear understanding of the main parameters that contribute to an improved bond.
- 9) Development of an analytical model is required; the conceptual model presented in Chapter 5 of this thesis needs more in-depth research to be fully developed. This will require larger experimental program to produce sufficient information to fully understand the parameters presented in the conceptual model.
- 10) The possibility of replacing secondary steel reinforcement by polymer grid reinforcement needs examination. Based on the results of this study, it appears that certain steel reinforcement that are used as secondary reinforcement can be replaced with suitable polymer grids. Loading and creep tests should be performed to evaluate the proposed changes on the overall performance of the structure.
- 11) Using polymer grids for ferrocement reinforcement. "Ferrocement are used for certain structures such as geodesic domes, wind tunnels, circular storage structures and swimming pools. In short, Ferrocement can be considered as a type of thin reinforced concrete construction where instead of discreetly-placed reinforcing bars, large amount of smaller diameter wire meshes are used uniformly throughout the cross-section, and instead of concrete, portland cement mortar is used", [72]. Polymer grids can be used instead of wire meshes or in conjunction with wire meshes to provide better crack control and higher corrosion resistance.

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