1

**PM-1 3 1/2" x 4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT**

PRECISIONSM RESOLUTION TARGETS

Triaxial compression of soils reinforced with discrete synthetic fibres

by

Alexandru Oancea

M. Sc., B. Sc., Civil Engineering, Bucharest Institute

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

Master of Engineering

in
Civil Engineering*

Carleton University
Ottawa, Canada

August, 1996

© 1996, Alexandru Oancea

***The Master of Engineering in Civil Engineering Program is a joint program with the University of Ottawa, administrated by the Ottawa - Carleton Institute for Civil Engineering**

National Library
of Canada

Acquisitions and
Bibliographic Services Branch

395 Wellington Street
Ottawa (Ontario)
K1A 0N4

Bibliothèque nationale
du Canada

Direction des acquisitions et
des services bibliographiques

395 rue Wellington
Ottawa (Ontario)
K1A 0N4

For a full reference

see Appendix A

The author has granted an irrevocable non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of his/her thesis by any means and in any form or format, making this thesis available to interested persons.

L'auteur a accordé une licence irrévocable et non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de sa thèse de quelque manière et sous quelque forme que ce soit pour mettre des exemplaires de cette thèse à la disposition des personnes intéressées.

The author retains ownership of the copyright in his/her thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without his/her permission.

L'auteur conserve la propriété du droit d'auteur qui protège sa thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

ISBN 0-612-19502-3

Canada

4.3.2	Type and properties of discrete fibres	43
4.3.2.1	General	43
4.3.2.2	Separation of the fibre bundle	44
4.3.2.3	Concentration and fibre length	46
4.4	Preparation of soil specimens	46
4.5	Testing procedures	48
4.5.1	Types of tests	48
4.5.2	The strain rate	48
4.5.3	Porewater pressure measurements	49
4.6	The compression test	49
4.7	Experimental programme (triaxial tests)	51
CHAPTER 5 : EXPERIMENTAL RESULTS.....		59
5.1	General	59
5.2	Triaxial tests on silty sand specimens	59
5.2.1	Unreinforced silty sand specimens	59
5.2.2	Silty sand specimens with 0.1% fibre content	60
5.2.3	Silty sand specimens with 0.2% reinforcement	61
5.2.4	Silty sand specimens with 0.3 and 0.5% reinforcement	62
5.3	Triaxial tests on sandy silt specimens	62
5.3.1	Unreinforced sandy silt specimens	62
5.3.2	Reinforced silty sand (0.1% fibre content)	63
5.4	Triaxial tests on medium sand specimens	64
5.4.1	Unreinforced medium sand specimens	64
5.4.2	Medium sand reinforced with 0.1% fibres	64
5.5	Unreinforced uniform sand specimens	64
5.6	Compression tests on medium sand specimens using the oedometer	65
5.6.1	Natural and reinforced medium sand	65
5.6.2	Compression of polypropylene fibres	65
CHAPTER 6: ANALYSIS AND DISCUSSION OF RESULTS.....		83
6.1	General	83
6.2	Mohr-Coulomb failure envelope	83
6.2.1	Influence of fibre concentration	84
6.2.2	Influence of fibre length	87
6.3	Comparison of deviator stress and volume change variations of natural and reinforced specimens	88
6.4	Volume change	91
6.5	Porewater pressure dissipation	93
6.6	Secant modulus	93
6.7	Strain increase with fibre content	95
6.8	Influence of fibres on different types of soil	97
6.9	Application of analytical model	99

Carleton University
Ottawa, Canada K1S 5J7

301. 1992

Thesis contains black and white and/or coloured graphs/tables/photographs which when microfilmed may lose their significance. The hardcopy of the thesis is available upon request from Carleton University Library.

The undersigned hereby recommend to the
Faculty of Graduate Studies and Research
acceptance of the Thesis entitled

**“Triaxial Compression of soils reinforced with
discrete synthetic fibres”**

submitted by
Alexandru Oancea

in partial fulfillment of the requirements for the degree of
Master of Engineering

Thesis Supervisor

Professor G.E. Bauer

Chair, Department of Civil and Environmental Engineering

Professor J.L. Humar

Carleton University

ACKNOWLEDGMENT

The author wishes to express his gratitude and appreciation to his supervisor, Professor G.E. Bauer for suggesting the subject of the thesis. The guidance, financial assistance and encouragement given by Professor G.E. Bauer made this study possible. His criticism was an invaluable source of inspiration and motivation. His continuous and effective support accelerated the progress of this study.

Special thanks are due to Professor K.T. Law for his constructive comments during the research. His encouragement definitely increased the extent of research and consequently the quality of the thesis.

The author would like to acknowledge the assistance of Messrs. K. Richter and P. Trudel in machining and fixing parts of the equipment. Without their considerable effort and willingness it would have been difficult to finish this work in time.

The author is grateful for the help of S. Conely in setting up the data acquisition system. Assistance and co-operation during the experimental phase of the study given by Messrs. K. McMartin and J. Plavka are also appreciated.

Acknowledgments are also extended to the Department of Civil and Environmental Engineering of Carleton University for support and assistance.

ABSTRACT

Discrete polymer fibres were used to reinforce granular soils subjected to triaxial compression. Discrete and randomly dispersed fibres are an alternative to continuous fibres or geogrids to improve the mechanical properties of soils. In this study two lengths of discrete polypropylene fibres and four concentrations were tested in conjunction with various soils. Four soil types with different granulometry were used ranging from a uniform silica sand to a fine silt. Specimens of natural and reinforced sands and silts were tested in static and cyclic triaxial compression. The stress, vertical strain, volume change and pore pressures were monitored. The compression behaviour in oedometer tests was used for comparison to the deformation in the triaxial tests. The theoretical model proposed by Gray and Ohashi (1983) was used to verify the experimental results. It is concluded that randomly dispersed polypropylene fibres increased the strength properties of the four soils tested. This increase is more pronounced in fine silts than in medium or uniform sands. This increase in strength is directly proportional to the fibre concentration. The higher fibre concentration is also associated with higher strains to reach peak strength and with greater volume change.

Triaxial compression of soils reinforced with discrete synthetic fibres

TABLE OF CONTENTS

CHAPTER 1: INTRODUCTION.....	1
1.1 General	1
1.2 Use of fibres to reinforce soils	2
1.3 Problem definition	3
1.4 Objectives of the research	4
1.5 Organization of the thesis	5
CHAPTER 2: LITERATURE REVIEW.....	7
2.1 General	7
2.2 Strength and deformation	7
2.3 Mechanism of reinforcement	10
2.4 Modulus of composite	12
2.5 Type and shape of fibres	14
2.6 Orientation of fibres	15
2.7 Length of fibres	16
2.8 Concentration of reinforcement	17
2.9 Preparation of laboratory specimens	17
CHAPTER 3: THEORETICAL CONCEPTS	27
3.1 The Mohr-Coulomb failure law	27
3.2 The stress calculation in a triaxial test	28
3.3 Analytical model	30
3.4 Volume change behaviour during triaxial compression of granular soils	33
3.5 Definition of secant modulus	35
3.6 The compression test	35
CHAPTER 4: EXPERIMENTAL PROGRAMME.....	40
4.1 General	40
4.2 Equipment	40
4.2.1 Triaxial cell	41
4.2.2 Volume change device	41
4.2.3 The compaction equipment	42
4.2.4 The data acquisition system	42
4.3 Materials	42
4.3.1 Soils	42

4.3.2	Type and properties of discrete fibres	43
4.3.2.1	General	43
4.3.2.2	Separation of the fibre bundle	44
4.3.2.3	Concentration and fibre length	46
4.4	Preparation of soil specimens	46
4.5	Testing procedures	48
4.5.1	Types of tests	48
4.5.2	The strain rate	48
4.5.3	Porewater pressure measurements	49
4.6	The compression test	49
4.7	Experimental programme (triaxial tests)	51
CHAPTER 5 : EXPERIMENTAL RESULTS.....		59
5.1	General	59
5.2	Triaxial tests on silty sand specimens	59
5.2.1	Unreinforced silty sand specimens	59
5.2.2	Silty sand specimens with 0.1% fibre content	60
5.2.3	Silty sand specimens with 0.2% reinforcement	61
5.2.4	Silty sand specimens with 0.3 and 0.5% reinforcement	62
5.3	Triaxial tests on sandy silt specimens	62
5.3.1	Unreinforced sandy silt specimens	62
5.3.2	Reinforced silty sand (0.1% fibre content)	63
5.4	Triaxial tests on medium sand specimens	64
5.4.1	Unreinforced medium sand specimens	64
5.4.2	Medium sand reinforced with 0.1% fibres	64
5.5	Unreinforced uniform sand specimens	64
5.6	Compression tests on medium sand specimens using the oedometer	65
5.6.1	Natural and reinforced medium sand	65
5.6.2	Compression of polypropylene fibres	65
CHAPTER 6: ANALYSIS AND DISCUSSION OF RESULTS.....		83
6.1	General	83
6.2	Mohr-Coulomb failure envelope	83
6.2.1	Influence of fibre concentration	84
6.2.2	Influence of fibre length	87
6.3	Comparison of deviator stress and volume change variations of natural and reinforced specimens	88
6.4	Volume change	91
6.5	Porewater pressure dissipation	93
6.6	Secant modulus	93
6.7	Strain increase with fibre content	95
6.8	Influence of fibres on different types of soil	97
6.9	Application of analytical model	99

CHAPTER 7: SUMMARY AND CONCLUSIONS.....	132
7 1 Summary	132
7 2 Conclusions	133
7 2 1 Strength	133
7 2 2 Strain	134
7 2 3 Soil Modulus	134
7 2 4 Volume Change and Porewater Pressures	135
7 2 5 Soil Granulometry	135
7 2 6 Fibre Content	135
7 2 7 Analytical Model	136
7 2 8 Recommendations for Future Research	136
REFERENCES.....	137
APPENDIX A.....	143
APPENDIX B.....	150
APPENDIX C.....	154
TABLE OF REFERENCES IN CHRONOLOGICAL ORDER.....	159
D 1 Triaxial compression tests on fibre reinforced soil.....	159
D 2 Triaxial compression tests on reinforced soil (other reinforcements than fibres)	159
D 3 Direct and simple shear tests on fibre reinforced soil	159
D 4 Properties of fibres and geotextiles used in soil reinforcement	160

LIST OF FIGURES

Fig 2.1	Sand reinforced with aluminum foil (after Schlosser and Long, 1972)	18
Fig 2.2	Sand reinforced with fibreglass nets (after Yang, 1972)	18
Fig 2.3	Sand reinforced with different layers of fabric reinforcement (after Gray et al., 1982)	19
Fig 2.4	Sand reinforced with geotextiles (after Atmatzidis et al., 1994)	19
Fig 2.5	Sand reinforced with Palmvra fibres (after Gray, 1978)	20
Fig 2.6	a) Shear strength increase vs. concentration of fibres, b) Influence of fibre concentration, c) Influence of fibre orientation (after Gray and Ohashi, 1983)	21
Fig 2.7	a) Skidding Mechanism, b) Rolling Mechanism (after O'Rourke et al., 1990)	22
Fig 2.8	a) Effect of confining pressure, b) test temperature, c) test strain rate for continuous filament melt-bonded polypropylene and polyethylene (after Andrawes et al., 1984)	22
Fig 2.9	In-air and in-soil behavior of a geogrid (after Juran, 1991)	23
Fig 2.10	a) Comparison of perpendicular vs. random oriented fibres b) Influence of fibre length on the shear strength, c) Influence of sand relative density (after Gray and Ohashi, 1983)	24
Fig 2.11	Sand reinforced with wood shavings (after Lee et al., 1973)	25
Fig 2.12	Kaolinite reinforced with pulp (cellulose) (after Andersland and Khattak, 1979)	25
Fig 2.13	Reinforced sand with discrete fibres (after Al-Refaei, 1985)	26
Fig 3.1	Assumed stress conditions on the triaxial specimen	37
Fig 3.2	Model of aligned fibres. (a) fibres normal to shear plane, (b) inclined fibres (after Fatani et al., 1991)	38
Fig 3.3	Compression of stress-strain curves for three types of compression	39
Fig 4.1	Schematic diagram of the triaxial apparatus (after Holtz and Kovacs, 1981)	52
Fig 4.2	Grain size distribution of test soils	53
Fig 4.3	Stress-strain curve and secant modulus of "Forta" fibres	54
Fig 4.4	(a) Bundled fibres, (b) and (c) Separated or unbundled fibres	55
Fig 4.5	(a) Unfolded fibre screen, (b) Bundle separated due to mixing friction in concrete (after Biddle, 1994)	56
Fig 4.6	Fibre separator	57
Fig 4.7	Lever type loading stand (figure after Kézdi, 1980)	58
Fig 4.8	Fixed-ring oedometer (after Holtz and Kovacs, 1981)	58
Fig 5.1	Stress-strain relations of natural silty sand specimens	67
Fig 5.2	Stress-strain relations of silty sand specimens (0.1% fibre content)	68
Fig 5.3	Stress-strain relations of silty sand specimens (0.2% fibre content)	69

Fig 5 4	Stress-strain relations of silty sand specimens (0.3% fibre content)	70
Fig 5 5	Stress-strain relations of silty sand specimens (0.5% fibre content)	71
Fig 5 6	Porewater pressures for silty sand specimens (0 and 0.1% fibre content)	72
Fig 5 7	Porewater pressures for silty sand specimens (0.2 and 0.3% fibre content)	73
Fig 5 8	Porewater pressures for silty sand specimens (0.5% fibre content)	74
Fig 5 9	Stress-strain relations of natural sandy silt specimens	75
Fig 5 10	Stress-strain relations of sandy silt specimens (0.1% fibre content)	76
Fig 5 11	Porewater pressures for sandy silt specimens (0 and 0.1% fibre content)	77
Fig 5 12	Stress-strain relations for natural medium sand specimens	78
Fig 5 13	Stress-strain relations of medium sand specimens (0.1% fibre content)	79
Fig 5 14	Stress-strain relations of natural uniform sand specimens	80
Fig 5 15	Oedometer comparison of compression of medium sand with different fibre contents.	81
Fig 5 16	Oedometer compression of polypropylene fibres	82
Fig 6 1	Mohr-Coulomb failure envelopes for natural and reinforced silty sand	102
Fig 6 2	Mohr-Coulomb failure envelopes for natural and reinforced sandy silt	103
Fig 6 3	Mohr-Coulomb failure envelopes for natural and reinforced medium sand	104
Fig 6 4	Mohr-Coulomb failure envelopes for natural and reinforced uniform sand	105
Fig 6 5	Analysis of parallelism	106
Fig 6 6	Apparent cohesion increase with fibre concentration	107
Fig 6 7	Comparison of "apparent cohesion" and peak strain with soil's content in fine particles	108
Fig 6 8	Stress-strain relations of silty sand specimens at 100 kPa cell pressure (0, 0.1, 0.2, and 0.5% fibre content).	109
Fig 6 9	Stress strain relations of silty sand specimens at 300 kPa cell pressure (0, 0.1 and 0.2% fibre content)	110
Fig 6 10	Stress-strain relations of silty sand specimens at 50 kPa cell pressure (0.2, 0.3 and 0.5% fibre content)	111
Fig 6 11	Stress-strain relations of sandy silt specimens (0 and 0.1% fibre content)	112
Fig 6 12	Stress-strain relations of medium sand specimens (0 and 0.1% fibre content).	113

Fig 6 13	Stress-strain relations of uniform sand specimens at 200 kPa cell pressure (0, 0.1 and 0.3% fibre content)	114
Fig 6 14	Cyclic load test on sandy silt specimens (0 and 0.1% fibre content)	115
Fig 6 15	Cyclic load test on medium sand specimens at 600 kPa cell pressure (0 and 0.1% fibre content)	116
Fig 6 16	Cyclic load test on a uniform sand specimen at 200 kPa cell pressure	117
Fig 6 17	Volume change for silty sand specimens (0, 0.1, 0.2, 0.3 and 0.5% fibre content)	118
Fig 6 18	Volume change for sandy silt specimens (0 and 0.1% fibre content)	119
Fig 6 19	Porewater pressures for silty sand specimens (0 and 0.1% fibre content)	120
Fig 6 20	Porewater pressures for sandy silt specimens (0 and 0.1% fibre content)	121
Fig 6 21	Comparison of secant moduli for silty sand specimens at 100 and 300 kPa cell pressure	122
Fig 6 22	Oedometer comparison of secant moduli for medium sand specimens (0, 0.1, 0.2, 0.3 and 0.5% fibre content)	123
Fig 6 23	Comparison of secant moduli for medium sand specimens at 100 and 400 kPa cell pressure (0 and 0.1% fibre content)	124
Fig 6 24	Comparison of secant moduli for uniform sand specimens at 200 kPa cell pressure (0, 0.1 and 0.3% fibre content)	125
Fig 6 25	Comparison of secant moduli for silty sand specimens (0 and 0.1% fibre content)	126
Fig 6 26	Comparison of secant moduli for silty sand specimens (0 and 0.1% fibre content)	127
Fig 6 27	Comparison of secant moduli for medium sand specimens (0 and 0.1% fibre content)	128
Fig 6 28	Comparison of secant moduli for uniform sand specimens (0% fibre content)	129
Fig 6 29	Oedometer compression of medium sand and polypropylene fibres (0 and 0.5% fibre content)	130
Fig 6.30	Gray-Hashi model (1983) applied to reinforced (0.1%) silty sand	131
Fig A 1	Results for test no 63	143
Fig A 2	Results for test no 64	144
Fig A 3	Results for test no 65	145
Fig A.4	Results for test no 68	146
Fig. A.5	Results for test no 69	147
Fig. A.6	Results for test no 70	148
Fig A 7	Results for test no 71	149
Fig. C 1	Comparison of peak deviator stress and strain for silty sand specimens (0, 0.1 and 0.2% fibre content)	154
Fig C.2	Comparison of peak deviator stress and strain for sandy silt specimens (0 and 0.1% fibre content)	155

Fig. C 3	Comparison of peak deviator stress and strain for medium sand specimens (0 and 0.1% fibre content)	156
Fig. C 4	Comparison of peak deviator stress and strain for uniform sand specimens (0, 0.1 and 0.3% fibre content)	157
Fig. C.5	Comparison of peak deviator stress and strain for all four test soils at 100 kPa cell pressure (0 and 0.1% fibre content)	158

LIST OF TABLES

Table 2.1	Degradation resistance of various synthetic fibres (after Cannon, 1976).....	26
Table 4.1	Soil classification results.....	53
Table 6.1	Comparison of cohesion increase at given vertical strains.....	99

LIST OF PLATES

Plate B.1	Polypropylene fibres 63 and 126 mm long	150
Plate B.2	Fibre count technique.....	150
Plate B.3	Examin. of the failure plane from a sandy silt specimen (0.1% fibre content).....	151
Plate B.4	Examin. of the failure plane from a sandy silt specimen (0.1% fibre content).....	151
Plates B.5 and B.6	Examination of the failure plane from a medium sand specimen (0.1% fibre content)	152
Plate B.7	Fibre investigation in a failed sandy silt specimen	153

To my professors

CHAPTER 1:

INTRODUCTION

1.1 General

The principle of reinforcing soil is used by some birds and animals which mix vegetal materials with soil to construct their habitat. There is proof that humans have used these techniques since the 5th and 4th millennium B.C. (Jones, 1985). Natural materials such as bamboo, timber, straw and reeds have been employed as reinforcements. Babylonians constructed ziggurats made of soil mixed with plant stems more than 3000 years ago. Chinese constructed tide embankments using fascines about 2000 years ago. Mats made of ditch reed were used by the Romans. Around the 14th century the Dutch used fascines widely to treat soft ground. The roots of trees can also be considered as natural reinforcement materials. Preventing erosion of slopes by planting grass and trees is a widely used practice today.

The modern concept of soil reinforcement was introduced on a commercial basis by Vidal (1966) who inserted horizontal flat strips of metal between layers of compacted soil (registered trademark of the company "Terre Armée"). The reinforcement of "Terre Armée" earth structures is purely by friction. New materials such as fabrics (geotextiles) and meshes (geogrids) are now widely used. In 1979, Leflaive mixed long continuous fibres with sand simulating the concept of plantroot reinforcement. This method is patented under the name "TEXSOL" and in 1987 was introduced also in Japan (Fukoka, 1988).

1.2 Use of fibres to reinforce soils

The use of discrete fibres are an alternative method to geotextiles and geogrids in soil reinforcement. Fibres can better model the natural roots of plants, which have been used for a long time in shallow depth soil stabilization. Present synthetic fibres offer good protection from biodegradation and attack from organisms. Vegetal fibres in contrast may become rotten in time if the surface vegetation disappears and, hence, the reinforcing effect is lost and the soil's porosity increases. Capillary voids are also present in plant fibres structure. When using synthetic fibres and an adequate compacting technique no additional void spaces are introduced in the composite.

Fibre reinforcement, in general, and the random distribution of fibres in particular, can be tested in the laboratory using standard test equipment to model field conditions. Consequently, the values obtained in laboratory testing may be used in designing prototype reinforced earth structures with more confidence.

The main drawback in employing randomly dispersed fibres for soil improvement may lie in the cost of mixing the fibres uniformly into the soil before compaction. Fabrics and geogrids have been preferred in the past as reinforcements, because of the ease of placement in embankments. Mixing fibres into clay in a large operation may prove to be extremely difficult due to the soils cohesive behaviour. Even in dry cohesionless sand, the only feasible technique for ensuring a random and uniform dispersion seems to be the hydraulic transportation and mixing of soil (Fukoka et al., 1991 and Kuwano et al., 1994).

Discrete fibres are a viable alternative to continuous fibres such as used in the Texsol process. In this laboratory study discrete fibres were mixed uniformly and in a random three-dimensional distribution into dry granular aggregates. It is, therefore, believed that this process of introducing discrete random oriented fibres in the field is feasible as an alternate method to using geotextiles, geogrids and Texsol.

1.3 Problem definition

There are two distinct concepts of placing reinforcement in a soil system. The first consists of placing oriented or aligned reinforcing elements, such as geogrids or membranes into the soil at strategic locations, usually in horizontal layers. The second method consists of mixing randomly distributed reinforcing fibres with the soil and compacting the mixture to a specified density. Researchers (Fatani et al., 1991) state that the concept of employing randomly dispersed fibres in the soil is new in the geotechnical field. No structures using discrete and randomly distributed fibres have yet been built. One main use of fibre reinforcement is to improve slope stability and to built steeper slopes. Also, the soil's lateral pressure will diminish behind a retaining wall if the soil is reinforced. A major problem in a fibre reinforced system is the uniform dispersion of fibres into the soil as was explained in the preceding paragraph.

Another problem is the determination of the proper length and type of fibre for a given soil. If the inclusion has a low tensile modulus and deforms too much in comparison to the natural soil, no reinforcement takes place. Moreover, the reinforcement must have stable mechanical properties with time, at least until the soil's strength increases naturally (i.e. consolidation). The interface friction and anchoring capacity of the fibres will play

a major role in mobilizing the tensile strength of the composite. The anchorage can be improved by changing the length and / or the shape of the fibres. A composite reinforced with discrete fibres may fail by fibre pullout or by breakage of the fibres in tension. Therefore, fibre length and tensile strength are two important factors to be considered. Of the same interest is the reinforcement concentration and the resulting increase in strength of the composite. Two fibre lengths were tested in this programme. It is also believed that the soil granulometry plays a major role in the performance of the polymer fibres. Four soil granulometries were used in this research. In conclusion, this study provided answers to the applicability of synthetic fibres, having different lengths and concentrations, as reinforcing elements for different types of soil. These answers, based on an extensive testing programme and analysis contribute to the further understanding of soil improvement using discrete fibres.

1.4 Objectives of the research

The main objective of this research was to investigate the reinforcing effect of discrete and randomly distributed polypropylene fibres on the mechanical properties of different soils. The specific objectives of this thesis were as follows:

- a) To investigate the amount or concentrations of reinforcement of polypropylene fibres on the strength of various soils in triaxial compression.
- b) To study the influence of fibre concentrations on volume change and soil compressibility.
- c) To evaluate the influence of fibre length on reinforcement and anchorage capacity.

- d) To analyze the behaviour of composite soil specimens in static and cyclic triaxial loading
- e) To correlate the influence of the soil's gradation on the performance of polypropylene fibres.
- f) To correlate the change in soil moduli to compression of the composite.
- g) To explain the mechanism of load transfer in contracting and expansive soils.
- h) To verify the validity of existing analytical models with experimental results.

1.5 Organization of the thesis

The first chapter will introduce the specific problems associated with discrete and randomly distributed fibres. The second chapter presents and discusses theories and experimental results of previous researchers. Pertinent analytical methods and experimental results of previous researchers will be used as basis for comparison in Chapter 6. The third chapter states the concepts and analytical methods used to analyze the experimental data. The concept of using the compression behaviour in oedometer tests to study composite soils is given. The applicability of the Gray-Ohashi analytical model (1983) to fibre reinforced triaxial specimens is analyzed. The fourth chapter presents the mechanical properties of fibres and the test soils. The test apparatus and procedures are discussed. The preparation of the specimens is described and also the extent of the experimental programme is outlined. The fifth chapter describes the different parameters obtained for the four types of soil. The results from the triaxial tests, in particular the three main parameters : deviator stress, volume change and porewater pressure, are presented as function of vertical strain. The test results for each

type of soil are presented. Relations which emphasize the fibre concentration and the confining stress are given. In the sixth chapter the test results are analyzed and discussed. The following types of analysis are presented: Mohr-Coulomb failure envelopes; principal stress versus confining pressure; secant modulus versus vertical strain; change of secant moduli; volume change and pore pressure variations; elastic settlement due to fibre compression and peak deviator stress versus vertical strain. The applicability of the Gray-Ohashi (1983) analytical model is tested. The seventh chapter gives a review of the thesis and contains the main results and conclusions. At the end of the thesis, the cited references are listed and the appendices are given.

CHAPTER 2: LITERATURE REVIEW

2.1 General

An extensive literature review had to be made due to the fact that few publications are available regarding the use of discrete and randomly distributed fibres in soils. Moreover, polypropylene is a material which has not been used before to the author's knowledge as discrete fibres in soil reinforcement. The polypropylene fibres have been, however, used in geofabrics and geogrids as reinforcement or for drainage filters in soil. Polypropylene fibres are commonly employed as secondary reinforcement in concrete panels.

Previous researchers have tested vegetal Palmyra or reed fibres as reinforcement in soil due to the relative ease of preparing test specimens with these fibres and due to the high tensile modulus of some vegetal inclusions. The literature review focused on types of problems which are similar to the topics dealt in this research. Because most of the research on soil reinforcement with discrete fibres and the corresponding analytical models were developed based on shear box results, and not on results from triaxial testing, references include also the results of direct shear tests. Reference is also made to the works of other researchers which have tested fabrics in conjunction with soils using the triaxial apparatus.

2.2 Strength and deformation

The idea of reinforcing soil with single fibres comes from the analogy of roots in plants which reinforce the top layers of soil. The increase in shear strength by root

reinforcement has been investigated by several researchers (Bishop, 1964, Kassif and Kopelovitz, 1968; Endo and Tsuruta, 1969; Brown and Sheu, 1975; Wu, 1976, Waldron, 1977, Borroughs and Thomas, 1977, Gray, 1978, Gray and Megahan, 1981 and Gray and Leiser, 1982). These researchers all agree that the root reinforcement increases the shear strength of soil. Gray and Ohashi (1983) have noticed that the post peak loss in strength is also reduced in dense sand. Two concepts, based on limit equilibrium theory and on triaxial test results of reinforced sand with oriented inclusions, have been proposed. In 1972, Schlosser and Long conducted a series of triaxial tests on sand with layers of circular aluminum foil disks and the “pseudo cohesion” concept was proposed to account for the reinforcement. They assumed that the increase of strength in the reinforced soil specimens was induced by an anisotropic or pseudo cohesion, C_R , which was a function of spacing and tensile strength of the disks. In the same year, Yang conducted a series of triaxial tests on specimens reinforced with horizontal layers of fibreglass nets and he developed the “equivalent confining pressure” concept. He concluded that the increase in axial stress, σ_{1f} , at failure, in the reinforced triaxial sample was due to an equivalent increase in confining stress $\Delta\sigma_3$. Both concepts can be used with the same result for analyzing the behaviour of composite earth reinforced with layers of fabric. At present, researchers use both concepts to define the increase in strength derived from triaxial or direct shear tests (Long et al., 1972; Juran and Schlosser, 1978, Schlosser and Elias, 1978; Hock and Brown, 1980; Leflaive, 1982, 1988; Fatani et al., 1991, Atmatzidis et al., 1994). Moreover, the above concepts seem to apply also for reinforced soil with discrete and randomly dispersed fibres. Therefore, the conclusions drawn from the

triaxial tests of soil reinforced with fabric layers can be considered valuable contributions in the research of soil reinforced with randomly distributed fibres. However, the failure modes of both systems are quite different. Few researchers report results of increased strength of soil reinforced with fibres. Leflaive (1982) estimated that fibre inclusion is equivalent to a cohesion of 150 to 250 kPa in the case of 0.1 to 0.2% reinforcement with continuous fibres for granular soils

The strain developed during triaxial or direct shear tests may be related to the settlements expected in the field. Therefore, the increased strain at failure of reinforced specimens, when compared with natural ones, is of great importance. This strain increase was observed by Hoare (1979), Brooms (1977), McGown et al. (1981), Gray and Ohashi (1983) and Bouzaza et al. (1994). Some other researchers observed an increase in strain with increasing confining pressure (McGown et al., 1981 and Schlosser et al., 1985).

The plot of vertical stress versus vertical strain usually shows an increase in the residual strength of the composite when compared to the residual strength of the natural soil. It is believed that only fibres penetrating or transgressing the shear plane are effective to carry tensile stresses. As mentioned by Fatani et al. (1991), 10-20% of the randomly distributed inclusions crossed the shear plane.

It was observed by Long et al. (1972), Gray et al. (1982), Bouzaza et al. (1994), that the failure or strength envelopes remain parallel with increasing fibre content and that the friction angle of the sand was, therefore, not affected by the reinforcement concentration. This fact led to the pseudo-cohesion concept as exhibited in a Mohr-

Coulomb failure plot. For the case of short fibres or spaced layers of fabric (or foil), previous researchers have found a critical confining stress below which the cohesion intercept tends toward zero. The strength envelopes were found to be curved by Schlosser and Long (1972), Yang (1972), Gray et al. (1978 and 1982), Al -Refeai (1985), Fatani et al. (1991), Bouzaza et al. (1994), Atmatzidis (1994) and Gray and Ohashi (1983) (Fig. 2.1, 2.2, 2.3, 2.4, 2.5, 2.6 (b)). Gray and Ohashi (1983) confirmed that below this critical confining stress the fibres tend to pull out from the composite

2.3 Mechanism of reinforcement

The most general relationship between length, friction, confining pressure and pull-out resistance was formulated by Gray (1978) and Gray and Ohashi (1983). To resist pull-out, and therefore reach their failure limit, the fibres must be long enough and develop sufficient frictional bond. The bonding force may be increased by increasing the all-round confining stress σ_3 . The conclusion of Schlosser and Long (1972), Gray et al. (1982) and Juran et al. (1988) is that there are two types of failure of a composite regarding the fibres, namely, pull-out and breakage of the inclusion. The same authors relate the tension failure of the fibres to their stiffness (elastic modulus). Gray and Ohashi (1983) found that low modulus fibres (ideally extensible), increase the shear strength when compared to inextensible ones which tend to fail in pullout. Other researchers show that ideally extensible reinforcements do not increase the strength of the soil but the amount of strain at failure (McGown et al., 1978 and Juran et al., 1988). Juran et al. (1988) found by numerical modeling that quasi-inextensible reinforcements are not suitable for soil reinforcement. As a conclusion, the length and the stiffness of

the fibres must be balanced in order to give optimum results in a given soil (stiffer reinforcements must be long enough to avoid pull-out). The four parameters of an inclusion discussed above (length, friction, confining pressure and stiffness) must be considered in problems of soil reinforcement using discrete fibres.

The load transfer mechanism from the soil particle to the fibre reinforcement is due to interface friction in granular soils. The fibre / soil interface friction is a function of the surface roughness of the fibre and the intrinsic soil friction. The shape of the fibres may also influence the load transfer mechanism due to interlock. This interlock given by the shape will increase the anchorage capacity. Therefore, the components of bond between the soil and the reinforcement are given by the surface friction of the fibre and the interlock given by the shape of the fibre. Therefore, a wavy shaped fibre has a better anchorage capacity than a straight fibre of the same length. Both components, surface friction and interlock will lead to the failure by breakage (and not by pull-out) of the reinforcement when the composite fails.

The amount of strain necessary to tension the fibres is controlled by the relative movement of soil particles. Juran et al. (1988) showed that high dilatant sands better mobilize the tension forces in the inclusions. Dyer and Milligan (1984), based on X-ray investigation, observed that the increase of tension forces in the inclusion can be related to the measured extension rate of the sand in the direction of the reinforcement. Both groups have assumed a perfect soil-inclusion adherence in their analytical model and in the interpretation of experimental results. An explanation of how adherence works in reality was given by O'Rourke et al. (1990), using scanned electron microscope

photographs. They found that a hard polymer surface would promote grain sliding (skidding) and soft surfaces would promote particle rolling and would result in recoverable fibre indentation (Fig 2.7 (a) and (b)). The same authors found that the interface friction increased with soil density. Juran et al (1988) stated that less compressible soils, beside being more strong in general, generate larger tension forces in the reinforcement. Gray and Ohashi (1983) showed that the increase in shear strength was the same for loose and dense sand, but larger strains were required in loose sand to reach peak strength. Gomes et al. (1994) have tested a variety of soils from crushed rock to cohesive soil and believe that the grain angularity and the roughness of the inclusion create interlocking and apparent adhesion.

2.4 Modulus of composite

The deformation of most soils under load is both elastic and plastic. Consequently the stress-strain curve of a soil is non linear. The procedure for calculating immediate settlement, of a structure for instance, usually involves some aspect of elastic theory. However, the “elastic” modulus used in these considerations may be better termed compression modulus. For small soil strains one could use the initial tangent modulus of the stress-strain curve determined in a triaxial test. At higher stress levels the secant modulus or the tangent modulus drawn at some predetermined stress level would be more appropriate. The choice which modulus to use depends on the problem to be investigated. In this study the secant modulus analysis was used. In the case of cycled load tests the hysteresis loops were connected by straight lines. These lines were parallel to one another after the second load cycle and were used for an “elastic” modulus

determination. A similar definition of moduli, as for natural soils, could be used in reinforced composites. McGown et al. (1978) classified the reinforcement as ideally inextensible (metal strips, bars, etc.) ($E_R / E_s > 3000$), and ideally extensible (natural and synthetic fibres, roots, fabrics, geotextiles) ($E_R / E_s < 3000$). E_R / E_s represent the ratio of reinforcement modulus (longitudinal stiffness) to average soil modulus.

However, the moduli of natural soil and reinforcement should be compatible within certain limits. If the reinforcement is too ductile, its mobilization of strength is associated with large soil strains. At such strain levels the natural soil might have already exceeded its peak strength. In such case the reinforcement is ineffective. On the other hand, brittle reinforcements are not suitable for soil reinforcement because compression forces develop in the inclusions and reduce the material resistance to shearing (Juran et al. 1988). Some metal and plastic fibres with lower rupture strains than the granular material in which they are embedded are effective in reinforcing the natural soil. McGown et al. (1978) have observed that ideally extensible reinforcements increase the soil's ductility (the strain before failure) as compared to quasi-inextensible (ideally inextensible or brittle) reinforcements which brake or slip. Andersland and Khattak (1979) using Kaolin reinforced with pulp (almost pure cellulose) and Al-Refeai (1985) using glass and reed fibres showed that the stiffness of the composite is increased for a dense sand when compared with the natural sand. Gray et al. (1982) tested triaxial dense dry sand reinforced with spun bonded polypropylene non-woven geotextile. They observed a reduction in stiffness of the composite (increased strain to reach peak stress) compared to the natural case. Juran et al. (1988) reported that stiff reinforcement having

a high elastic modulus yields a low initial secant modulus of the reinforced soil. The same authors have demonstrated that the amount of strain of the inclusion at rupture determines the mechanism of load transfer from the soil to the inclusion. Many researchers have shown that the mechanical properties of a reinforcement are influenced by the level of confinement (σ_3 in a triaxial test). El-Fermaoui and Novatzki (1982), McGown et al. (1982), Andrawes et al. (1984) (Fig. 2.8(a)) and Juran et al. (1991) have found that the stiffness of a geogrid embedded in soil increases with increasing confining pressure. Juran et al. (1991) reports data of the true extension of an embedded geogrid under high confining pressure. The stiffness of a synthetic material tested in air is lower than the corresponding stiffness in-soil (Fig. 2.9). The test temperature and test strain rate change the behaviour of polypropylene fibres (Fig. 2.2 (b) and (c)).

The density of the composite also plays a great role upon the soil's modulus. O'Rourke, Druschell and Netravali, (1990), found that the composite's modulus should be affected by the interface friction which increases with soil density. Broms (1977) and Gray and Ohashi (1983) show that loose sands require larger strains to reach peak stress for the same increase in shear strength. The shear stress is also affected by the relative density of the composite (Fig. 2.10(c)).

2.5 Type and shape of fibres

Natural fibres can consist of wood shavings, reed, cellulose, Palmyra and others. Effects of discrete natural fibre reinforcing soil are presented in Figs. 2.11, 2.12 and 2.13. Synthetic fibres which have been used as inclusions are made of polypropylene, PVC, polyamide and polyester. Steel wires, copper wires and glass fibres were also used

by previous investigators. As summarized by Giroud (1984) the fibre characteristics which make them suitable for soil reinforcement are strength, specific gravity, strain at failure, modulus of elasticity, stability and durability. Some porous fibres, such as wood shavings used by Hoare (1979), increased the composite's porosity and resistance to compaction, and decreased its strength. Degradation of reinforcements is caused by biological and chemical agents. The response of some synthetic fibres to the above factors was summarized by Cannon (1976) (Table 2.1).

Another important property of the reinforcement is given by the surface roughness of the fibre. Fatani et al. (1991) make a distinction between surface roughness and geometry of the reinforcement. Jewell (1980) has created ideally rough surfaces by changing the reinforcement's shape (e.g. by using a grid). Therefore, shape and roughness are interdependable variables. Gray and Ohashi (1983) showed that ideally "rough" surfaces mobilize entirely the sand's internal friction angle along the surface of the reinforcement.

2.6 Orientation of fibres

As concluded by Hayashi et al. (1988), Fatani et al. (1991) and Bauer and Zhao (1994) no reinforcing effects can be observed when the inclusions are placed parallel to the failure plane. Moreover the strength of the soil can decrease due to the lubricating effect the reinforcement can provide (Fig. 2.6(c)). Jewell (1980), Gray and Ohashi (1983) and Bauer and Zhao (1994) found that the reinforcement must be placed in the direction of the tensile strain. Therefore, a compression or tension force can develop in the reinforcement depending on its inclination with respect to the failure plane.

Moreover, even fibres transgressing the failure plane in a normal direction can undergo compression when the sand contracts during the initial stage of loading. As mentioned by Juran et al. (1988) and also discussed by Dyer and Miligan (1984), as dense sand starts to dilate, the reinforcing fibres will be subjected to tension. Fatani et al. (1991) have shown in their experimental study that discrete randomly oriented fibres will improve the shear strength. Gray and Ohashi (1983) (Fig. 2.10(a)) reported that the shear strength obtained in direct shear tests for randomly oriented fibres was approximately the same as that obtained when the fibres were normal to the shear plane.

2.7 Length of fibres

As mentioned previously, little research data concerning fibre reinforced soils is available. Despite the fact that the "TEXSOL" reinforcing system is using continuous fibres, the interest of present researchers has shifted toward the application of short and discrete fibres. The length of fibres used as inclusions in soil varied between 20 and 250 mm by Gray and Ohashi (1983) (Fig. 2.10(b)), 13 to 38 mm by Al-Refeai (1985) and 20 mm by Bouzaza et al. (1994). They determined a critical fibre length and confining pressure as discussed earlier in section 2.2. The critical confining stress was found to be dependent on the interface friction (fibre length or "roughness"). Al-Refeai (1985) mentioned that by increasing the length of fibres, the strength and stiffness of the composite increases. Gray and Ohashi (1983) showed that the increase of fibre length also increases the strength until a constant threshold value is reached .

2.8 Concentration of reinforcement

The upper concentration of plant roots to have a reinforcing effect in a permeated natural soil is probably 1.7% (Gray and Ohashi, 1983). Consequently, using this concentration as a guide, soils reinforced with discrete synthetic fibres should probably not exceed this concentration. Some researchers have used higher concentrations. Concentrations in the range of 1 to 5 % were employed by Gray and Ohashi (1983) (Fig. 2.6 (a) and (b)), Al-Refeai (1985) (Fig. 2.13 (b)) and Bouzaza et al. (1994). Gray and Ohashi (1983) found that the strength increased proportional to the concentration at least up to 1.7% (estimated upper limit of root concentration in a permeated soil). Yang (1972) and Al-Refeai (1985) also showed that the strength increase is proportional to a limiting amount of reinforcement.

2.9 Preparation of laboratory specimens

For the preparation of test specimens, different techniques were used by various researchers to achieve a random three-dimensional distribution of fibres and a maximum dry density. To ensure a random orientation the fibres and the sand were mixed by hand (Bouzaza et al., 1994), or by using a helical ascendant mixer (Al-Refeai, 1985). Fukoka et al. (1991) and Kuwano et al. (1994) used the water jet technique to mix continuous fibres with sand. When preparing a specimen, the shear box was inclined at different angles of deposition in order to obtain "random" distributions. Khay et al. (1990) using the same technique of preparing the specimens, showed that continuous fibres are rather arranged two-dimensionally in the plane of deposition and the composite material has a strong anisotropy.

Fig. 2.1 Sand reinforced with aluminum foil (after Schlosser and Long, 1972)

Fig. 2.2 Sand reinforced with fibreglass nets (after Yang, 1972)

Fig. 2.3 Sand reinforced with different layers of fabric reinforcement (after Gray et al., 1982)

Fig. 2.4 Sand reinforced with geotextiles (after Atmatzidis et al., 1994)

Fig. 2.5 Sand reinforced with Palmyra fibres (after Gray, 1978)

(a)

(b)

(c)

Fig. 2.6 a) Shear strength increase vs. concentration of fibres.
 b) Influence of fibre concentration.
 c) Influence of fibre orientation.
 (after Gray and Ohashi, 1983)

Fig. 2.7 a) Skidding Mechanism, b) Rolling Mechanism
(after O'Rourke et al., 1990)

Fig. 2.8 a) Effect of confining pressure, b) test temperature, c) test strain rate for continuous filament melt-bonded polypropylene and polyethylene
(after Andrawes et al., 1984)

Fig. 2.9 In-air and in-soil behavior of a geogrid (after Juran, 1991)

Fig. 2.10 a) Comparison of perpendicular vs. random oriented fibres
 b) Influence of fibre length on the shear strength.
 c) Influence of sand relative density.
 (after Gray and Ohashi, 1983)

Fig 2.11 Sand reinforced with wood shavings (after Lee et al., 1973)

Fig. 2.12 Kaolinite reinforced with pulp (cellulose) (after Andersland and Khattak, 1979)

Fig 2.13 Reinforced sand with discrete fibres (after Al-Refeai, 1985)

Resistance to Attack by	Types of Synthetic Fibres				
	Polyester	Polyamide	Polyethylene	Polypropylene	PVC
Fungus	Poor	Good	Excellent	Good	Good
Insects	Fair	Fair	Excellent	Fair	Good
Vermin	Fair	Fair	Excellent	Fair	Good
Mineral Acids	Good	Fair	Excellent	Excellent	Good
Alkalies	Fair	Good	Excellent	Excellent	Good
Dry heat	Good	Fair	Fair	Fair	Fair
Moist heat	Fair	Good	Fair	Fair	Fair
Oxidizing Agents	Good	Fair	Poor	Good	-
Abrasion	Excellent	Excellent	Good	Good	Excellent
Ultraviolet Light	Excellent	Good	Poor	Good	Excellent

Table 2.1 Degradation resistance of various synthetic fibres (after Cannon, 1976)

CHAPTER 3: THEORETICAL CONCEPTS

3.1 The Mohr-Coulomb failure law

The strength of soils is usually defined in terms of the stress developed at the peak of the stress-strain curve. One method of representing the strength is to draw the Mohr circles representing the states-of-stress at the peak points of the stress-strain curves.

The Mohr-Coulomb failure law expresses the strength as

$$\tau_{ff} = c + \sigma_{ff} \tan \phi \quad (3.1)$$

where σ_{ff} and τ_{ff} are the normal and tangential stresses on the failure plane, c is termed cohesion or intercept and ϕ is the angle of internal friction of soil particles. The parameter "tangent ϕ " includes sliding and rolling friction and the resistance offered by interlocking action. The triaxial tests yield data to plot the Mohr's stress circles. The cell pressure is the minor principal stress, σ_3 , and the cell pressure added to the applied vertical load is the major principal stress, σ_1 ; both at failure of the specimen. By using specimens of soil with approximately the same density and water content, at different cell pressures, an approximate tangent (failure envelope) to the circles can be drawn (Bowles, 1970). The slope of this tangent will be the angle of internal friction of the soil, ϕ ; the intercept of the failure envelope on the y-axis will be the cohesion c in the Mohr-Coulomb equation. The shear strength of cohesive materials may be expressed by Coulomb's equation

$$s = c + \sigma_n \tan \phi \quad (3.2)$$

where s = shear strength; c = soil cohesion; ϕ = angle of internal friction and σ_n = normal stress on the shear plane. For granular soils $c = 0$. Therefore, the shear strength for sands becomes

$$s = \sigma_n \tan \phi \quad (3.3)$$

In a reinforced soil, the reinforcement induces an “apparent cohesion” (Schlosser and Vidal, 1969). Long et al. (1972), Gray et al. (1982) and Bouzaza et al. (1994) observed that the angle of internal friction of reinforced soils was not affected by the reinforcement. Consequently, based on the Mohr-Coulomb failure law, the shear strength of a reinforced granular soil may be written as

$$s_R = c_R + \sigma_n \tan \phi \quad (3.4)$$

where s_R = shear strength of the reinforced soil; c_R = “apparent cohesion” produced by the reinforcement; σ_n = normal stress and ϕ = angle of internal particle friction.

3.2 The stress calculation in a triaxial test

In the triaxial test a soil specimen is compressed and maintained at a given lateral confining pressure that would exist in the ground at a given depth. Figure 3.1 is reproduced from Holtz and Kovacs (1981) and presents the assumed stress conditions on the triaxial specimen.

$$\sigma_{\text{axial}} = \sigma_1 - \sigma_3 = \frac{P}{A} \quad (3.5)$$

$$\sigma_{\text{cell}} = \sigma_2 = \sigma_3 \quad (3.6)$$

P = the instantaneous vertical load on the specimen as measured by loading ring or cell, for a corresponding value of vertical deformation ΔL .

A' = corrected cross-sectional area of the specimen for the corresponding load P .

$\sigma_1 - \sigma_3$ = is termed the stress difference or deviator stress.

Medium and very soft materials, as opposite to hard and rigid ones, arrive at the state of failure after a considerable deformation, assuming a “bulged” shape. The volume of the specimen after some decrease in length by $-\Delta L$, are given as follows:

$$A' (L_o - \Delta L) = V_o - \Delta V \quad (3.7)$$

$$A' = \frac{1 - \Delta V / V_o}{1 - \Delta L / L_o} \times \frac{V_o}{L_o} \quad (3.8)$$

$$\varepsilon = \frac{-\Delta L}{L_o} \quad (3.9)$$

$$A_o = \frac{V_o}{L_o} \quad (3.10)$$

The corrected area of the specimen derived from above equations will be:

$$A' = \frac{1 - \Delta V / V_o}{1 - \varepsilon} \times A_o \quad (3.11)$$

ε = the unit strain computed from the deformation-dial (LVDT) reading.

ΔL = total axial sample deformation.

L_o = original specimen length.

$-\Delta V$ = the decrease of the specimen volume.

V_o = initial sample volume.

A_o = original specimen area.

For drained test the volume of the specimen is changing (increase or decrease), and the deviator stress acting on the specimen becomes:

$$\sigma_1 - \sigma_3 = \frac{P(1 - \epsilon)}{A_0(1 \pm v)} \quad (3.12)$$

$$v = \frac{\pm \Delta V}{V_0} \quad (3.13)$$

where v = the specific volume change (volumetric strain).

3.3 Analytical model

Complex load-transfer phenomena from soil to the inclusions are taking place during the deformation and at failure of a reinforced soil. Finite element analyses have been used to describe the interaction between sand and reinforcement by Naylor et al. (1978) and Andrawes et al. (1980). Modeling the state of stress and strain were developed based mainly on direct shear test results. A simple model, based on the force-equilibrium method, to predict the strength increase of a composite reinforced with oriented fibres was developed by Gray and Ohashi (1983) and Gray and Al-Refeai (1986). The latter model has also been used to describe the behaviour of randomly and oriented metallic fibres tested in direct shear by Fatani et al. (1991). The tensile force generated in the reinforcement crossing the failure plane may be resolved into normal and tangential components. The normal component mobilize additional shear resistance by increasing the confining stress on the failure plane, whereas the tangential component directly resists shear. The fibre is assumed to be thin enough that it offers little if any resistance due to bending stiffness. The shear strength increase of a composite due to the perpendicular-oriented fibres, (Fig. 3.2) can be estimated from the following expression:

$$\Delta S_R = t_R (\sin i + \cos i \tan \phi) \quad (3.14)$$

where ΔS_R = strength increase of the composite per unit area of soil,

t_R = tensile strength mobilized in the fibres per unit area of soil,

i = angle of shear distortion,

ϕ = angle of internal friction of the soil.

In Fig. 3.1, it can be estimated from the shear's zone thickness z and the horizontal shear displacement x . For fibres or inclusions which are initially inclined to the shear plane (Fig. 3.1 b), the increase in shear resistance is given by:

$$\Delta S_R = t_R [\sin(90 - \beta) + \cos(90 - \beta) \tan \phi] \quad (3.15)$$

$$\beta = \tan^{-1} \left[\frac{1}{r + (\tan \alpha)^{-1}} \right] \quad (3.16)$$

where r = shear distortion ratio ($r = x / z$, Fig. 3.1 b),

x = horizontal of shear displacement,

z = thickness of shear zone,

α = the initial inclination of the fibre with the shear plane,

β = the angle of the deformed fibre with the shear plane.

Maher and Gray (1990) have statistically predicted that random oriented fibres are perpendicular to the failure plane, and therefore the relation (3.14) may be applicable to the case of this study. A fibre area ratio expressed as the ratio of total sheared cross-sectional area of the fibre to the total sheared area of the composite. The mobilized tensile strength per soil unit area (t_R) is the product of fibre's tensile stress and the area ratio (or fibre concentration in the shear plane).

$$t_R = \sigma_R \left(\frac{A_R}{A_C} \right) \quad (3.17)$$

where A_R = the reinforcement's cross-sectional area,

A_C = total composite's sheared area,

σ_R = tensile stress in the reinforcement present on the shear surface,

A_R / A_C = the fibre area ratio (FAR).

In the preceding four equations of the analytical model, the following conditions are assumed to be known: the thickness of the shear zone, the shape of the deformed fibre (as shown in Fig. 3.1), the stress distribution along the fibre's length and that the fibres will not fail by pull-out. During the process of load transfer from the composite to the fibres, (therefore before the fibres fail), a linear elastic relationship between tensile force and tensile strain can be assumed:

$$\sigma_R = E_R \epsilon_R \quad (3.18)$$

where E_R = the fibre's modulus of elasticity,

ϵ_R = the average tensile strain in the reinforcement.

If the rupture of the fibres has to be modeled, a non-linear stress-strain relationship of the polypropylene fibres may be used. Sometimes, the fibre volume ratio (FVR) index parameter becomes an important consideration if the fibre's volume within the composite is considerable. This ratio can be expressed as:

$$FVC = \frac{V_R}{V_S} \quad (3.19)$$

where V_R = volume of reinforcement,

V_S = volume of soil solids in the composite.

Substituting equation (3.14) into (3.4) results:

$$s_R = t_R (\sin i + \cos i \tan \phi) + \sigma_n \tan \phi \quad (3.20)$$

where $t_R (\sin i + \cos i \tan \phi)$ is the "apparent cohesion" (c_R) of the reinforced soil as discussed by Schlosser and Vidal (1969).

3.4 Volume change behaviour during triaxial compression of granular soils

In a triaxial test the specimen is first compressed isotropically by increasing the chamber pressure. Then the vertical stress is increased while the chamber pressure is held constant. If the deviator stress is plotted against the vertical strain, the stress-strain relation becomes curved even at small strains, achieves a peak and then gradually decreases past the peak. Eventually the curve levels off at a constant value of stress.

Before yielding, the volume of a sand specimen will decrease slightly. In this stage the particles are being pushed into a denser arrangement. This behaviour can be also observed during confined or isotropic compression. At and near the peak the soil fails. During this stage the volume of the specimen increases. A dense sand when compressed in one direction increases in volume (it may be defined as dilatant). The shearing strength of sands is composed of (1) the friction between adjacent grains and (2) the resistance to moving individual grains, called interlocking action. For a given sand, the portion of the shearing strength that is due to interlock increases as the unit dry weight increases. Interlocking effect itself is affected to some degree by particle shape and grain size distribution (it is generally greater for angular particles than for round ones, and greater for a well graded sand than for a narrowly graded soil). The angle of internal friction, as generally used, includes the effect of interlocking. If one refers to arrays of sand particles, in order to have a shear failure between particles, it is not only necessary to overcome the mineral-to-mineral frictional resistance, but also it is necessary to make

particles move up and over one another. Dense arrays of particles start to move apart just as soon as shear motion between arrays commences. The particles (arrays) moving apart lead to soil dilatancy. As shear motion continues, the degree of interlocking decreases, and consequently, the shear force necessary to continue the motion also decrease (the specimen fails). A loose sand will undergo more initial decrease in volume than a dense sand. The loose specimen first decreases in volume, then expands. A dense specimen expands in volume to a very marked degree at very low strains. The denser the sand, the greater the interlocking, hence the greater the deviator stress and consequent friction angle. The denser the sand, the greater the volume increase which occurs. Loose sands experience little volume change. However, at higher confining pressures, dense sand exhibits the behaviour of loose sand by showing a decrease in volume (or compression) with strain. The post peak decrease in deviator stress is more marked in dense specimens. At ultimate stress condition (constant deviator stress), the interlocking between the soil particles has decreased at a point at which continuous shear deformation can continue without further volume change. The causes of decrease in volume of grained soils are a combined effect of the following mechanisms: (1) the grains tend to slide on one another into a tighter packing and (2) individual soil particles undergo elastic deformations due to tension, compression, bending and shear caused by stresses within a granular mass. Under excessively large compressive stresses crushing of grains occurs. The fragments fill the remaining void spaces and this results in a volume decrease. This effect becomes important when a granular soil is subjected to very heavy loads or when it contains a large amount of calcareous shells. (3) For water-saturated

fine-grained soils, (for instance silts) the volume change is due mainly to the escape of water from the voids

3.5 Definition of secant modulus

The theory of elasticity uses a constant stress-strain ratio known as Young's modulus, E , to characterize the behaviour of linearly-elastic materials. For soils, whose behaviour is typically non-linear, the "modulus" analogous to E is not constant and holds only for a limited, narrow range of pressures. Consequently, for small soil strains one could use the initial tangent modulus of the stress-strain curve determined in a triaxial test. At higher stress levels the secant modulus (slope of a straight line drawn from the origin at some predetermined stress level) or the tangent modulus may be used. The choice of which modulus to use depends on the problem to be investigated. In the study on hand the secant modulus analysis, which is applicable to materials of non-constant "elastic" modulus, was used. This method of analysis offers better interpretation possibilities than the secant modulus defined at a predetermined strain and stress level. Fellenius (1990) used the tangent modulus analysis in his interpretation of results for static pile loading tests.

3.6 The compression test

A soil deposit loaded by a wide structure or a man-made fill will deform vertically. This vertical deformation at the surface resulting from the load is termed settlement and is due to the compressibility of the soil. The compressive properties of soil are usually determined in the laboratory using the apparatus known as a consolidometer. As it was mentioned in Section 3.4, during the initial stage of a triaxial

test (at very small strains 0.25%), the soil particles are pushed into a denser arrangement and the vertical stress-vertical strain behaviour is similar to that during confined or isotropic compression (Fig 3.3) (Lambe and Whitman, 1979)

Karol (1960) defined the term compression as a rapid almost instantaneous change in height of a soil mass due to applied loads. The same author showed that the term consolidation should be reserved for the slow change in height caused by the time-lag necessary to permit water to flow out of a loaded soil mass. Soil masses consisting mainly of large particles (except fine sands) have relatively large pores through which water can flow freely, and consequently the time-compression relationship of dry or saturated soils show approximately the same data. The void ratio-pressure curves obtained for sands are typical for those obtained from silts and clays with the exception that the void ratio changes are minute in comparison (Karol, 1960). The first clarification of the soil compression behaviour has been devised by Terzaghi (1925) and is known as the Terzaghi compressibility test. The apparatus used for this purpose is called oedometer (or consolidometer). If a soil specimen is subjected to compression in an oedometer the stress is computed using the following relation:

$$\sigma = \frac{P}{A_0} \quad (3.21)$$

where σ = vertical stress on the specimen.

P = vertical force given by the applied load .

A_0 = specimen area.

It should be noticed that when using polypropylene fibres mixed with soil, void ratio and porosity are not reliable parameters of the composite. The polypropylene fibres have a

modulus in compression about 100 times lower than the sand and, therefore, compress more under the same loading conditions. This fact may be interpreted as a decrease in void ratio. The fibres, due to their low specific gravity (0.78), as compared with the stone specific gravity (2.65), may be interpreted as compressible voids between soil particles. Basically, the fibres and soil particles should be studied as a unit. For this purpose, the compression of a specimen, or its vertical strain, are more significant parameters than initial void ratio. For instance, Indraratna et al. (1991) presented consolidation tests where fabrics were involved using the compression of the specimen as an indicative parameter for soil behaviour.

The following concept was used in testing soil and fibres in an oedometer either as a composite or taken separately as will be shown later in chapter 6. The compression difference of a reinforced and natural soil specimen will most probably represent the compression of the fibres. It may be assumed that the elastic deformation of the stone particle is negligible compared with the compression of polypropylene fibres.

Fig. 3.1 Assumed stress conditions on the triaxial specimen

Fig. 3.2 Model of aligned fibres: (a) fibres normal to shear plane, (b) inclined fibres (after Fatani et al., 1991).

Fig. 3.3 Compression of stress-strain curves for three types of compression

CHAPTER 4:

EXPERIMENTAL PROGRAMME

4.1 General

Only a few researchers have reported results of triaxial tests on soil specimens reinforced with continuous or discrete randomly distributed fibres. Leflaive (1982, 1988) used sand reinforced with continuous filaments. Al-Refeai (1985) carried out triaxial tests on uniform sand reinforced with discrete randomly mixed glass fibres (lengths of 13-25-38 mm and concentrations 0.5-5% by weight) and reed fibres (lengths of 13-25-38 mm and concentrations 0.5-2%). Bouzaza et al (1994) used dry sand reinforced with polyamide randomly distributed fibres 20 mm long. No research was reported on discrete randomly distributed polypropylene fibres. Moreover, none of the previous researchers tested silt or well graded sand. No volume change or pore pressures were recorded in these previous studies and also no cyclic triaxial loading tests were performed. The results presented in this study consisted of using fibrillated polypropylene fibres with different types of well graded soils (silts and sand). The results from these well graded soils were compared with corresponding values for uniform sand. The volume change and the pore pressures of the specimens were recorded under static and cyclic triaxial loading conditions.

4.2 Equipment

A standard triaxial cell commonly used in soil investigations was used. The main parts of the equipment consisted of the loading frame, the triaxial cell, the constant pressure apparatus, the volume change apparatus and the data acquisition system. The

loading was strain-controlled. Supplementary devices were used such as a vacuum pump for applying suction to the specimens during preparation of non-cohesive soil and a shaking table for compaction of granular specimens.

4.2.1 Triaxial cell

The triaxial cell was made of a Perspex cylinder sitting on a pedestal fitted with valves. The soil specimen was confined with porous stones, a bottom and a top cap and placed into the cell. The sample was encased in a thin rubber membrane sealed with O-rings. Leads from the data acquisition system were attached to electrical pressure transducers, a LVDT for measuring the vertical displacement and the load cell. The triaxial cell could accommodate specimens 100 mm in diameter and 200 mm high. The membranes were made of high durability natural latex having a 0.3 mm average thickness. Figure 4.1 shows a schematic diagram of the triaxial cell.

4.2.2 Volume change device

The change in volume of a specimen is an important parameter during the consolidation and shear phases in a drained triaxial test. After consolidation, the volume change measurements were used to calculate the actual density of the specimen before shearing. The volume change measurement was also used to compute the actual cross-sectional area of the specimen and average stresses. A volume change apparatus with the capability to measure up to 80 cm³ was utilized. The volume line can be reversed to obtain continuous volume change measurements. The volume change of a sandy-silt reinforced specimen was up to 100 cm³. The volume change apparatus, was calibrated at

the beginning of the test programme using a burette. Full saturation of specimens is necessary for ensuring the accuracy of volume change data.

4.2.3 The compaction equipment

The method of compaction was in accordance with ASTM D-2049. The principal components of the compaction assembly were the vibrating table, the lower platen, the mould and the bottom plate. A surcharge weight was placed on the top of the specimen during vibration. Details of the procedure will be provided under section 4.4, "Preparation of Specimens".

4.2.4 The data acquisition system

The data acquisition system was composed of a loading cell, a LVDT for measuring the vertical displacement, electrical transducers for recording the cell and pore pressures and a second LVDT as part of the volume change device. The internal load cell and transducers were using electrical resistance strain gauges. The transducers, LVDT's and load cell were connected to a signal amplifier which sent the signals to a computer. A computer program was used for displaying, managing and recording the data. A calibration of all measuring devices was made at the beginning of each test series.

4.3 Materials

4.3.1 Soils

Four types of soil were investigated: a silty sand, a sandy silt, a well graded sand and a uniform sand. The grain size analysis and the characteristics of the soils are given in Fig. 4.2 and in Table 4.1. They were determined according to procedures specified by

ASTM D-422-63. The objective in choosing these soils was to test various types and compare their behaviour in triaxial compression. The silty sand and sandy silt were gray coloured and were sieved down from a crushed limestone. The sandy silt was a mixture of 51% sand (above 0.075 mm particle size) and 49% silt. The silty sand had percentages of silt between 30-40%. The uniform graded sand, commercially sold as "Silica 16", was a crushed white quartz sand. The well graded sand had rounded particles and it is commonly used in mortars and concrete. The modified Proctor compaction test of the sandy silt yielded an optimum moisture content of 10.5% with a corresponding dry density of 20 kN/m^3 . The silty sand had a dry density of 19 kN/m^3 at a water content of 9.8%. The medium sand yielded a dry density of 20.2 kN/m^3 at a water content of 9.1%. The uniform sand yielded a dry density of 16 kN/m^3 at a water content of 3 to 4 percent. Some of the results of soil classification and identification are given in Table 4.1.

4.3.2 Type and properties of discrete fibres

4.3.2.1 General

A commercially available synthetic fibre (FORTA CR) was used in this study (heavy-duty grade of fibrillated polypropylene). Polypropylene is produced by the catalytic polymerization of propylene, or may be copolymers with ethylene or other materials. Propylene is a methyl ethylene, produced in the cracking of petroleum. Polypropylene has a crystalline structure with high hardness, toughness, and melting point. Polypropylene fibres have been used as secondary reinforcement in concrete since 1978. The polypropylene fibres are non-corrosive and chemically inert, hydrophobic (do

not absorb water), and consequently they are stable and durable. The specific gravity of polypropylene used in this study was 0.78, and its ultimate tensile stress was $\sigma_u = 93$ MPa (Fig. 4.3).

4.3.2.2 Separation of the fibre bundle

Jewell (1980) in his investigation created ideally rough surfaces by changing the reinforcement's shape. Gray and Ohashi (1983) showed that ideally "rough" surfaces mobilize entirely the sand's internal friction angle along the surface of the reinforcement. Between length, shape, three-dimensional interlocking and friction is a strong correlation. The three-dimensional orientation, and therefore better interlocking (three-dimensional reinforcement), is a main feature of discrete randomly distributed fibres. The fibrillated fibres were further processed in this study. A curly or spiral shape fibre resulted which improved the interlocking capability of the fibres with the sand. In their initial state the fibres appeared twisted and intertwined as shown in (Fig. 4.4). Unfolded and discrete fibres are an improved alternative to straight fibres and it is believed that discrete and wavy fibres will provide better interlocking to prevent pullout. Fatani et al. (1991) also referred to the geometry of the reinforcement as being vital in its performance. The "Forta" fibres used in this study came as twisted bundles in specified lengths (Fig. 4.4a). If the fibres were untwisted, a net or screen pattern of fibres resulted (Fig. 4.5 a). The "Forta" fibres are primarily used as secondary reinforcement in ready-mix concrete. The ingredients in ready-mix concrete are mixed continuously. This action will unfold the fibres (Fig. 4.5 b). Bigger aggregates will be able to provide the energy necessary to separate longer fibres. In Fig. 4.5.(b) it can be seen that the

fibres in concrete have different levels of separation. Non-separated bundles will provide higher porosity in concrete and might cause voids. Eventually the cement paste will penetrate the non-separated bundles and fill the voids between fibres. It is assumed that the soil, (compared to concrete), is less frictional, and the chances of separating the bundles by mixing are lower. For maximum reinforcement performance the discrete fibres should be randomly and uniformly distributed within the soil mass. Discrete fibres will not provide additional porosity by entrapping air, especially in soils with fine particle fractions and continuous gradations which are able to penetrate the fibre structure. If there are sufficient fines, all the void spaces between the unfolded fibres will be filled with soil (Fig. 4.5 a). If the fibres are already separated by a preseparation process, they are more pliable and can be easier dispersed in the soil. Therefore, a process where the fibre bundles are separated into individual strands will ensure uniformity of fibre distribution. It was observed that the shape of the fibres became curly or spirall when using the separation process. Such shapes tend to decrease the pull-out tendency of short fibres and ensure a more uniform distribution within the soil mass. The simple device to separate the fibrillated fibre bundless into single strands is shown in Fig. 4.6. The device consisted of two boards with protruding nails. The bottom part was shaped like a rectangular trough with arrays of protruding nails. The top part was a board which also had arrays of protruding nails. Placing the unbundled fibres in the trough and sliding the top board back and forth separated the fibrillation into single fibres (Figs. 4.4 and 4.6). The device will be referred to in the following section as "fibre-

separator". As can be seen in Figs. 4.4 and 4.6, the separated polypropylene fibres have the appearance of unspun wool.

4.3.2.3 Concentration and fibre length

The proposed polypropylene fibre concentration as secondary reinforcement in architectural precast panels, is about 0.1% *by volume* (27900 linear metres of reinforcement per cubic metre of concrete). A reason of using a fibre concentration of 0.1% *by weight* (the lower limit in this study), was the experience of other researchers, especially Leflaive (1982). A concentration of 0.1% polypropylene fibres *by weight*, represents an equivalent length of 5716 metres of continuous thread in one cubic meter of soil. In this study four fibre concentrations were used: 0.1, 0.2, 0.3 and 0.5 %. For one cubic meter of soil, the fibres (in concentration 0.1% *by weight*), have a weight of about 1.6 Kg. The length of fibres is an important factor from the point of view of fibre reinforcement performance. Two fibre lengths were considered in this investigation. 62.5 mm (2.5 inches) and 125 mm (5 inches).

4.4 Preparation of soil specimens

Preparation of specimens to ensure a uniform and random fibre dispersement is not an easy task. Al-Refeai (1985), showed that mixing the fibres and soil was a major difficulty. Previous researchers have prepared small specimens by hand mixing the fibres with sand. Some other researchers have used the water-jet technique in building specimens with continuous fibres (Kuwano et al. 1994). Eventually, the laboratory technique of unbundling or separating and mixing fibres with soil may suggest a field technology. It was necessary in this research to ensure a random and uniform

distribution of fibres throughout the specimens. First, the fibres were unbundled in the fibre-separator. In order to have a uniform distribution throughout the relatively small volume of soil specimens, the fibres were separated in 16 equal tridimensional bunches by weighing them. The already tridimensional interlocked bunches of fibres were successively placed into the triaxial mould, at equal spacing. After placement of a fibre bundle in the mould a measured quantity of dry soil was rained slowly over the fibres into the vibrating mould. The specimen was then vibrated to a specified density on a shaking table. Previously, tests were conducted to determine how the speed of pluviation and shaking intensity affect the dry density of the composite. The calibration of the system yielded a uniform density with deviation of less than 1%. The specimen density achieved in using dry soil, yielded about 85% of modified Proctor density. In the subsequent stage, after saturation and consolidation, 90% of the modified Proctor density was achieved. The preparation technology of introducing the soil in a dry state was preferred because it ensured the three-dimensional distribution of fibres. Even in the case of continuous fibres and hydraulic deposition of soil, researchers have found that the distribution is rather two-dimensional than three-dimensional and consequently the composite material has a strong anisotropy (Khay et al. 1990).

The specimens were vibrated in a split mould. A membrane (0.3 mm thick) was placed inside the mould and a suction was applied to keep the membrane tight to the mould wall. The preparation of cohesionless specimens on the testing machine was also possible by applying a suction to the specimen's interior. Paper drains were placed around the specimen, with porous stones at either end of the specimen. After

preparation, the specimen was submerged in water for 24 hours to saturate. The Skempton pore pressure parameter B was found from the triaxial test to range between 0.95 to 1.00 which is considered as an indication of full saturation (Craig, 1994)

4.5 Testing procedures

4.5.1 Types of tests

All tests were performed as consolidated-drained tests. A pore pressure sensor was placed at the centre of the specimens to monitor the excess porewater-pressure, if any, which could be generated during the shearing phase. After the confining pressure was applied, consolidation of the specimen followed. The consolidation of the specimens was considered to be complete when the pore pressures at the centre of the sample was zero. After consolidation the specimens were sheared under drained triaxial conditions. The strain rate for shearing a particular soil was determined as the maximum rate of shear at which no excess porewater pressure was developed. Several consolidated-undrained tests (CU), were performed as checks of the porepressure measurement. The effective soil parameters c' and ϕ' derived from both types of tests were similar.

4.5.2 The strain rate

For sand the chosen strain rate was 0.66 mm/minute due to the high permeability of the soil. Complete and instantaneous drainage occurred. In the case of the silty-sand samples, several triaxial tests were performed at different strain rates. The pore pressures were measured at the centre of the specimen. The rate of 0.33 mm/minute was chosen as satisfactory to ensure complete drainage.

It should be noted that the pore pressure at the center of specimen is the highest generated within the specimen since both ends and the sides were free to drain. A small hydraulic gradient is necessary to ensure the flow (drainage), between center and boundaries of specimen. In a consolidated-drained test zero pore pressures is a good assumption in particular when the rate of strain is chosen carefully.

4.5.3 Porewater pressure measurements

The pore pressure measurements in undrained tests can be made at the ends of a specimen or at the center by using a syringe needle (Bowles, 1978). The use of a needle at the center of specimen makes possible to monitor pore pressure in consolidated-drained triaxial tests of cohesive soils. In this way the strain rate can be adjusted during a test for drained conditions (zero porewater pressures). Different syringe type needles were used in this research in order to observe any possible time lag in the measurement. The best needle which yielded instantaneous response and minimum clogging was chosen.

4.6 The compression test

The compression of a soil occurs because of the deformation of soil grains, compression of air and water in the voids and / or squeezing out of water and air from the voids. The amount of compression of the soil mineral grains themselves is small and usually can be neglected. The compression of the pore fluid can also be neglected. It is assumed that in the case of a dry sand reinforced with polypropylene fibres, the rearrangement of soil particles and the deformation of the fibres will cause most of the soil compression. The dry sand was chosen for the purpose of making obvious the vertical

deformation due to two components, namely the soil grains and the synthetic fibres. Squeezing out of air in a permeable dry sand occurs instantaneously and does not affect the time of vertical deformation of the specimen to occur.

In the triaxial test, the vertical strain and the volume change are indicators of the soil compressibility. The triaxial test is not usually used for investigation of soil compressibility for engineering practice in which one-dimensional compression is dominant. Moreover, for the purpose of this study, in which a synthetic material other than soil was involved, a suitable compression of this material itself cannot be made using the triaxial apparatus. The soil's one-dimensional compressive properties are usually determined in the laboratory in a consolidation test. The investigation of the soil compressibility by using dead loads applied incrementally was considered relevant for strains of 0.25%, due to the same behaviour in triaxial compression and confined compression as was presented in Section 3.6. The compressibility of soils was discussed in Section 3.4., and the concept of the compression test in Section 3.6. The procedure given ASTM D-2435-70 was followed. The test device was a standard consolidometer (Fig. 4.7). A fixed-ring oedometer was used, in which the specimen dimensions were 100 mm in diameter and 19 mm in height (Fig. 4.6). The specimens, in a dry state, were prepared with the same well graded sand as used in the triaxial tests. The specimens were also reinforced with the same fibre contents as used in the triaxial tests and were compacted to the same dry density. A series of 24 load increments were used in order to obtain a load-unload type of compression curve. Ten uniaxial compression tests were performed using the oedometer.

4.7 Experimental programme (triaxial tests)

A total of 87 triaxial tests were carried out but only half of these were used in the analysis. A number of 10 tests were used to set up and calibrate the data acquisition system and to choose the proper strain rate. A number of tests were also used as repeat tests at the same confinement and fibre concentrations in order to check the reliability of results. About 15 tests yielded relatively low peak stress and were duplicated two, and sometime three times. The lowest peak stress represented about 90% of the maximum which could be obtained. It was realized that the amount of shear developed on the weakest shear plane along which the specimen fails is highly dependable on the uniformity of the reinforcement distribution. The random distribution is an aleatory process by definition. A relatively small quantity of fibres crossing the probable shear zone can make a huge difference in terms of stresses. Fatani et al. (1991) reported that 10-20% of the fibres are intersecting the shear plane, and that only these reinforcements are effective to carry the tensile stresses. Consequently, according to this supposition, the stress carried by the fibres can vary considerable. However, because the weakest plane is unique by definition, the strength cannot increase beyond a maximum point which is given by an ideal random distribution of fibres. Ideal random distributions of reinforcement into the specimens insure constant reliable results. Due to the inevitable drawbacks of the technique used in preparing laboratory specimens, the easiest way of check up was by repeating several times a given test under the same conditions (duplicating it). The variation of the ratio silt to sand from one test to another should be

also considered as a source of data non-uniformity. An extensive programme is necessary, in the case of a natural (real) soil due to the scatter of granulometry distribution encountered in nature. This tedious work was considered compulsory for the approach of a new research field as the use of discrete random distributed fibres is

Fig. 4.1 Schematic diagram of the triaxial apparatus (after Holtz and Kovacs, 1981)

Fig. 4.2 Grain size distribution of test soils

Soil type	D ₅₀ mm	C _u	C _c	USCS Classification
Sandy Silt	0.075	6.11	0.031	ML
Silty Sand	0.140	9.21	1.082	SM-ML
Medium Sand	0.480	4.29	0.933	SW
Uniform Sand	1.500	1.68	1.112	SP

Soil type	ϕ^3	Max. unit dry weight kN/m ³	A _v test unit dry weight kN/m ³
Sandy Silt	37	20	18
Silty Sand	41	19	17.4
Medium Sand	41	20.2	18.2
Uniform Sand	39	16	15.6

Table 4.1 Soil classification results

Fig. 4.3 Stress-strain curve and secant modulus of "Forta" fibres

Fig. 4.4 (a) Bundled fibres, (b) and (c) Separated or unbundled fibres

(a)

(b)

Fig 4.5 (a) Unfolded fibre screen
(b) Bundle separated due to mixing
friction in concrete (after D T Biddle, 1994)

Fig. 4.6 Fibre separator

Fig. 158. Lever type loading stand:

1 — loading lever, 2 — loading yoke, 3 — measuring frame with dial gauge, 4 — loading bridge, 5 — counterweight, 6 — weights, transmission: $P/Q \approx r_2/r_1$.

Fig. 4.7 Lever type loading stand (figure after Kézdi, 1980)

Fig. 4.8 Fixed-ring oedometer (after Holtz and Covacs, 1981)

CHAPTER 5 : EXPERIMENTAL RESULTS

5.1 General

This chapter presents the experimental results of static triaxial and oedometer tests. The results of triaxial tests yielded three main parameters, namely the deviator stress, volume changes and porewater pressures as function of vertical strain. Each type of soil in its natural state or as a composite with a given reinforcement concentration was tested at different confining pressures. Results are presented for each type of soil. The results are organized in a manner to show the deviator stress, volume change and porewater pressures at different confinements and same fibre concentration. Comparisons at different fibre concentrations, analyses and discussions will be presented in chapter 6.

5.2 Triaxial tests on silty sand specimens

5.2.1 Unreinforced silty sand specimens

The constituent characteristics of the silty sand were presented in section 4.3.1. The silty sand was considered to be well graded. The unreinforced soil specimens were tested in triaxial compression as reference and Figs. 5.1 and 5.6 show the results. A number of observations can be made. The stress-strain curves exhibited well defined peak values at about 13% axial strain. The stress-strain curves decreased gradually with increase of axial strain beyond peak. The residual deviator stress was reached at about 20% of vertical strain. The volume of the specimens decreased more rapidly at the beginning of the tests. The change of rate is between 5 and 12% of strain for cell

pressures of 100-500 kPa. At higher cell pressures the rate of volume change occurs at higher axial strains. The porewater pressures of the silty sand specimens, (Fig. 5.6), show a peak value between 3-5% of vertical strain and decreases rapidly afterwards. When the failure stress of the soil is reached, the porewater pressures are close to zero. At failure, a defined change of porewater pressures can be observed which is indicated as a second peak point. The porewater pressures show a tendency toward zero after failure of the specimens. The peak porewater pressures are less than 1% of the maximum deviator stress and consequently can be neglected in the analysis. The magnitude of porewater pressures do not show any dependence on the confining stress. As can be observed from Fig. 5.1, the initial tangent modulus increases with increasing the cell pressure. Generally speaking, all specimens developed uniform lateral bulges before sliding on a distinctive failure plane occurred.

5.2.2 Silty sand specimens with 0.1% fibre content

An amount of fibres of 2.5g (or 0.1% reinforcement per weight) was added to the soil. The following observations can be made from Figs. 5.2 and 5.6 which show the stress-strain relation and porewater pressures, respectively. By adding fibres, the stress-strain plot definitely changed when compared to the natural soil. The peak deviator stress increased by 54 to 133% depending on the amount of confining stress. The strain at which the peak strength occurs (between 17-22%) also increased by 30 to 69%. The deviator stress decreases gradually after the peak value. A residual deviator stress is reached at about 30% of vertical strain. The reduction in volume is pronounced until 12% vertical strain; beyond that the decrease is less pronounced. The volume change

decreases with increasing chamber pressure. The porewater pressures reached a peak point (at about 3% of vertical strain) of magnitude less than 1% of the corresponding peak deviator stress. As the specimens reach peak deviator stress, the porewater pressures showed a negative value; afterwards they start to increase toward zero. For most specimens the measured porewater pressures are in most cases virtually zero at peak deviator stresses. The secant modulus is increasing with increasing the chamber pressure. Failure occurred usually along a single shear plane at an angle of 68 to 70° with the horizontal. On this particular plane the fibres transgressing the shear plane were broken. In the mass of the specimen the fibres were intact (Appendix B). All specimens developed uniform bulges (barrel shape) before the distinctive shear plane developed.

5.2.3 Silty sand specimens with 0.2% reinforcement

The following discussion is based on the results shown in Figs. 5.3 and 5.7. The observations are in general the same as for 0.1% fibre concentration. Peak deviator stress occurs between 18-28% of vertical strain. The residual deviator stress was reached at 35% of vertical strain. The secant modulus is increasing with chamber pressure. The volumetric strain versus vertical strain plot shows a change of rate of decrease at about 12% strain (the volume decreases at a lower rate after failure occurs, even during the residual phase). The porewater pressures (Fig. 5.7) show a second peak point of lesser magnitude than the first one, which marks the failure of the specimen. Failure of the specimen due to fibre breakage occurred along a single shear plane at 68-70° with the horizontal after bulging initially.

5.2.4 Silty sand specimens with 0.3 and 0.5% reinforcement

By increasing the amount of reinforcement to 0.3 and 0.5%, the recording capacity of the load cell was exceeded even for low applied confining pressure. Due to the low confinement stress which could be applied only two triaxial tests were possible at a 0.3% fibre concentration and only one test at 0.5% fibre concentration (Fig. 5.4, 5.5, 5.7 and 5.8). The secant modulus is increasing with chamber pressure. Peak strength occurred at 26% vertical strain for 0.3% reinforcement and at 30% vertical strain for 0.5% reinforcement. The volume change becomes more gradual at about 12% vertical strain. The residual deviator stress was reached at about 35% vertical strain. The porewater pressures at failure of the specimens with 0.5% reinforcement (Fig. 5.8) are close to zero. Failure occurred along a single shear plane at 68-70° with the horizontal due to fibres breakage on this particular plane. Specimens also developed uniform bulges before a distinctive single shear plane was developed.

5.3 Triaxial tests on sandy silt specimens

5.3.1 Unreinforced sandy silt specimens

Figures 5.9 and 5.11 show the deviator stress, volume change and porewater pressures of the natural soil (not reinforced) tested in triaxial compression. The following observations are made based on five test results at different chamber pressures. The peak deviator stress increased with increasing chamber pressure. The strain at which the peak deviator stress occurred also increased by increasing the cell pressure. The deviator stress increased until a well defined peak value was reached between 11-15% axial strain. After peak, the deviator stress decreased gradually and the residual deviator

stress started to be constant between 15 and 22% axial strain. The volume of the specimens decreased continuously during shear. The volume change of the specimens decreased with increasing confining pressure. The porewater pressures (Fig. 5.14) increased at test begin, reached a peak point between 2 to 4% vertical strain and dissipated afterwards. The tendency of porewater pressures to increase at higher chamber pressures can be observed, but not in a consistent manner (because of one exception at 700 kPa) (Fig. 5.11). At about 15% vertical strain the porewater pressures was zero and showed some tendency to become negative.

5.3.2 Reinforced silty sand (0.1% fibre content)

Results of six tests, reinforced with 0.1% of fibres are presented in Figs. 5.10 and 5.12. It can be observed that by increasing the concentration of reinforcement the specimens yielded higher peak deviator stresses. These peak stresses were developed between 19-24% of axial strain. The axial strain which corresponded to peak deviator stresses increased with the cell pressures. After peak, the deviator stress constantly decreased and become residual starting at 26 to 32% vertical strain. The volume of the specimens decreased constantly and more pronounced at the beginning of tests. The porewater pressures (Fig. 5.12) increased at specimen loading, showed a peak between 2 to 3% vertical strain and dissipated afterwards. Before peak stress, the porewater pressures become zero in the case of four tests and they were less than 1% of peak deviator stress in the case of the other two tests. After failure of specimens the porewater pressures (of this two tests) decreased continuously and had a tendency to dissipate.

5.4 Triaxial tests on medium sand specimens

5.4.1 Unreinforced medium sand specimens

The medium sand presented in section 4.3.1 and Fig. 4.2 had a well and continuous gradation. In Fig. 5.12 the deviator stress and volume change parameters versus axial strain are presented for this sand at different confining pressures. It can be observed that the deviator stress increased at higher chamber pressures. At the same time the corresponding strain increased at higher cell pressures. The peak deviator stress is well defined between 5-7% vertical strain. After peak the deviator stress decreased gradually and residual deviator stress values were reached at 12 to 14% vertical strain. The specimens contracted at the beginning of tests and started to dilate at strains between 2% to 5% of axial strain. The dilatant behaviour was more pronounced at a low confining pressure of 100 kPa as shown in Fig. 5.12.

5.4.2 Medium sand reinforced with 0.1% fibres

The medium sand specimens reinforced with 0.1% fibres (Fig. 5.13), showed a similar behaviour as the natural sand specimens. The reinforced specimens yielded higher peaks of deviator stresses at corresponding higher vertical strains (8-12%). The decrease in deviator stress after failure was quite sudden compared to the behaviour of the unreinforced sand. The residual deviator stress occurred at higher vertical strains than for the natural soil (13 to 15%).

5.5. Unreinforced uniform sand specimens

In Fig. 5.14 it can be observed that a peak deviator stress values were reached between 6-8% vertical strain. The residual stress occurred at 17 to 21% vertical strain.

(for confining stresses of 100 to 300 kPa). The specimens experienced quite a pronounced dilative behaviour except for a confining stress of 1400 kPa. At this high confining pressure possible crushing of sand grains occurred explaining the contractive behaviour. This phenomenon was presented in section 3.4.

5.6 Compression tests on medium sand specimens using the oedometer

5.6.1 Natural and reinforced medium sand

The reasons and principles of performing oedometer tests were presented in sections 3.6 and 4.6. The results of compression in oedometer will be analyzed and compared with the triaxial test results in Chapter 6. In Fig. 5.15 the vertical strain-vertical stress relationships of the medium sand specimens are presented. The medium sand specimens had the same composition of particles (granulometry) and density as in the triaxial compression tests. Specimens were tested in natural and reinforced states. The amount of fibre concentrations were the same as in the triaxial compression tests namely 0.1, 0.2, 0.3 and 0.5%. A number of observations can be made from Fig. 5.15. The vertical strain of specimens was clearly increasing with fibre concentration at corresponding stresses. The specimen with 0.5% fibres underwent double the vertical strain of natural soil for maximum loading condition. The secant moduli of specimens at any given strain or stress levels are definitely decreasing with increasing fibre content. The secant moduli corresponding to the unloading curve are however constant.

5.6.2 Compression of polypropylene fibres

In a triaxial compression test the fibres were most probably compressed by the sand particles during loading. Lambe and Whitman (1979) showed that during the initial

phase of a triaxial compression test the sand particles are pushed into a denser state. The sharp sand particles also penetrate into the polymer structure and at the same time increase the confinement of the fibres. At small strains (0.25%), according to Lambe and Whitman (1979), the sand should show the same behaviour in confined or triaxial compression tests. This fact was presented in detail in sections 3.4 and 3.6. Only the given amount of fibres corresponding to the composite specimen was compressed in the oedometer. It was to simulate the compression of the fibres within the soil mass at the beginning of a triaxial test (before the dense sand increased in volume and the fibres were tensioned). In Fig. 5.16 the compression of 0.75g of unbundled polypropylene fibres were presented for a fibre concentration of 0.5%. The fibres were dispersed uniformly within the ring. The dispersement or porosity of fibres was about the same as that of the composite soil-fibre specimen. The fibres were then compressed under repeated load-unload cycles. No hysteresis was observed and the fibres behaved perfectly elastic (the rebound of fibres at unloading followed the same loading path). This test was therefore considered to give the stress-strain properties in compression of the fibre assembly.

Fig. 5.1 Stress-strain relations of natural silty sand specimens

Fig 5.2 Stress-strain relations of silty sand specimens
(0.1% fibre content)

Fig. 5.3 Stress-strain relations of silty sand specimens (0.2% fibre content)

Fig 5.4 Stress-strain relations of silty sand specimens
(0.3% fibre content)

Fig. 5.5 Stress-strain relations of silty sand specimens
(0.5% fibre content)

Fig. 5.6 Porewater pressures for silty sand specimens (0 and 0.1% fibre content)

Fig. 5.7 Porewater pressures for silty sand specimens
(0.2 and 0.3% fibre content)

Fig. 5.8 Porewater pressures for silty sand specimens
(0.5% fibre content)

Fig 5.9 Stress-strain relations of natural sandy silt specimens

Fig. 5.10 Stress-strain relations of sandy silt specimens (0.1% fibre content)

Fig. 5.11 Porewater pressures for sandy silt specimens
(0 and 0.1% fibre content)

Fig. 5.12 Stress-strain relations for natural medium sand specimens

Fig. 5.13 Stress-strain relations of medium sand specimens
(0.1% fibre content)

Fig. 5.14 Stress-strain relations of natural uniform sand specimens

Fig. 5.15 Oedometer comparison of compression of medium sand with different fibre contents

Fig. 5.16 Oedometer compression of polypropylene fibres

2

OF/DE

2

PM-1 3 1/2" x 4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTION TARGETS

CHAPTER 6: ANALYSIS AND DISCUSSION OF RESULTS

6.1 General

In this chapter the experimental results are compared and discussed. The following types of analyses are made: Mohr-Coulomb failure envelopes, principal stress versus confining pressure, deviator stress versus vertical strain, volume change and porewater pressures, elastic compression of fibres and secant modulus versus vertical strain. The applicability of the Gray-Ohashi (1983) theoretical model will also be tested.

Above analyses are made for the four types of soil namely, silty sand, sandy silt, medium and uniform sands. Three more variables will be considered: fibre concentration, fibre length and cell pressure.

6.2 Mohr-Coulomb failure envelope

The failure envelopes were drawn as best fit tangents to the Mohr circles (Figs 6.1 to 6.4). A rubber membrane correction was applied to all triaxial test results. The failure envelopes of the unreinforced soil passed through the origin of axes (zero cohesion). As can be seen from figures 6.1 and 6.2, for the silty sand and sandy silt specimens the failure envelopes for the composites remain parallel to the corresponding envelopes of the unreinforced specimens regardless of fibre content. There was no pronounced tendency of the failure envelopes for medium sand specimens to be parallel (Fig. 6.3). In some cases there were insufficient Mohr circles and the envelopes were assumed to be parallel to that of the unreinforced specimens.

This is in agreement with the observations of other researchers mentioned earlier (Fig. 6.4). It should be noted that all Mohr circles for a fibre concentration of 0.1% were verified by at least two repeat tests.

The parallelism of the envelopes suggests that the angle of internal friction is not affected by the amount of fibres added within the range of 0.1 to 0.5%. Some researchers observed similar results (Long et al., 1972, Gray et al., 1982, Gray and Ohashi, 1983 and Bouzaza et al., 1994). Other researchers have found that both cohesion and the angle of internal friction were increasing with an increase in fibre content (Bauer and Al-Joulani, 1991). Figure 6.5 presents the relation of major principal stress to minor principal for the natural and reinforced specimens. This mode of presentation was used by Schlosser and Long (1972), Yang (1972), Gray et al. (1982) as was pointed out in Figures 2.4, 2.5 and 2.6 and illustrates the parallelism of the relation with fibre content.

The apparent cohesion is increasing with increasing the fibre content for all four soils and fibre contents investigated. Similar results have been reported by Schlosser and Long (1972), Long et al. (1972), Schlosser and Elias (1978) and Hock and Brown, (1980). Other researchers referred to this apparent cohesion as an "equivalent" increase in confining pressure (Yang, 1982; Juran and Schlosser, 1978 and Atmatzidis et al., 1994).

6.2.1 Influence of fibre concentration

According to equation 3.4 the strength increase of a reinforced soil at a given normal stress is given by the increase in apparent cohesion. The angle of soil internal friction was not affected by the reinforcement. This was also found by some of previous

researchers and verified by this study. The apparent cohesion for silty sand was found to be in the range of 120 to 140 kPa, for each 0.1% of fibre concentration increase until 0.3% (Figs. 6.1 and 6.6). For a concentration of 0.5%, an increase of about 140 kPa was observed when compared to the concentration 0.3%. The sandy silt showed an increase of 120 kPa for 0.1% fibre concentration (Fig. 6.2). For medium sand, an increase of 80 kPa in cohesion was noticed at 0.1% fibre concentration and, at the same time, a decrease of the friction angle by 2° (Fig. 6.3). In the case of the uniform sand specimens reinforced with 0.1% fibres, a 40 kPa increase in cohesion was obtained compared to the unreinforced sand (Fig. 6.4). The increase in apparent cohesion, therefore, seems directly proportional to the quantity of reinforcement for 0.1 and 0.3% fibre content. An increased apparent cohesion of 140 to 250 kPa, using continuous fibres in sand, was reported by Leflaive (1982).

In summary, an increase in apparent cohesion (Figs. 6.1 to 6.6) was found to be related to the amount of reinforcement used up to a fibre concentration of 0.3%. A few tests were performed at a fibre content of 0.5% using low confining pressures. This envelope was based upon a single Mohr circle (due to the limitations of the equipment as was explained in chapter 4), and consequently the envelope was assumed to be parallel to those of lower fibre contents. A diminishing increase in cohesion intercept at this high fibre content (0.5%), (Fig. 6.6) might suggest the existence of a limiting or threshold amount of reinforcement which may be used to increase the apparent cohesion of soils. Yang (1972) and Al-Refeai (1985) have also found an increase in strength of soil

proportional to a limiting amount of reinforcement, after which the cohesion intercept tends to become constant

Lambe and Whitman (1979) stated that at the contact point of soil particles in a granular mass "for a given total normal load, the normal load per contact increases as the particle size increases" Figure 6.7 which presents on the vertical axis the gain in cohesion which is obtained from Figs. 6.1 to 6.4 (for a constant fibre concentration of 0.1%), suggests a decrease in apparent cohesion for sands with a lower limit for the uniform sand. From Fig. 4.2 it can be observed that the particle sizes of the four soils used in this study were increasing from sandy silt to silty sand to the medium sand and the uniform sand. It can be observed from Fig. 6.7 that the apparent cohesion of reinforced soil decreased with an increase in the percentage of larger size particles. A second effect, that of reduced strength for gap graded soils compared to continuous graded soils, was suggested by the uniform sand soil test results when compared to the medium sand apparent cohesion (Fig. 6.7). In the case of a coarse uniform sand less contact points between the coarse particles are present per unit volume of soil than in silt. If for the same volume of soil a given amount of fibres is introduced, more contact points between soil and fibres per unit volume for a fine soil are generated. Contraction and dilation of soil causes relative rolling and sliding movements between fibres and particles. A good reference to these types of particle movement on the polymer surface is provided by O'Rourke, Druschell and Netravali (1990) (Fig. 2.11). Lambe and Whitman (1979) showed that "larger particles are able to roll more easily than the smaller particles, perhaps as a result of their center of gravity being further away from

the plane of shear". It may be concluded that in a soil with continuous gradation a more uniform distribution of shear stresses between particles and fibres occurs (per unit area of fibre), and the interlocking of particles on fibres is reduced compared to a soil with uniform gradation. There is also a more uniform distribution of contact stresses in soils with finer particles because they develop more contacts per unit area of fibres (i.e. silts and fine sands). The above observations can be made in reference to Fig. 6.7. A comparison of well graded silts to well graded medium sand will reveal the influence of different particle sizes for continuous gradation curves, (see Fig. 4.2). A comparison of a well graded medium sand to a gap graded uniform medium sand will reveal the influence of the gradation. A reduction of the rupture strength due to the breakage of fibres by the sharp particles in contact with the fibres was also suggested by Murray and McGown (1982). The preceding observation may explain that less reinforcement was provided by the polypropylene fibres as the distribution of particles become more and more uniform and coarse.

6.2.2 Influence of fibre length

The Mohr-Coulomb envelopes were drawn for the sandy silt specimens reinforced with 63 and 126 mm long fibres and were shown to be congruent (Fig. 6.2). The envelopes also yielded the same internal friction angle and the same increase in apparent cohesion for both fibre lengths. The Mohr circles for both types of specimens are tangent to the same failure envelope. This is due to the fact that even the short fibres (63 mm) provided sufficient frictional and interlock resistance. As was pointed out in the literature review, fibre friction and length as well as applied confining pressure are inter-

dependable variables. By shortening the fibres, it is quite obvious that the apparent cohesion component will decrease toward zero and whereas the angle of internal friction will remain fairly constant. By making the fibres shorter and consequently decreasing the fibre surface area for frictional contact with the soil, the envelopes become curved at very low confining stress as shown in Figures 2.4, 2.5 and 2.6 (Gray et al., 1982, Fatani et al., 1991 and Al-Refaat, 1985, Bouzaza et al., 1994 and Atmatzidis, 1994). Or, in other words, more confinement is necessary to mobilize the friction component of short fibres (Jewell, 1980 and Gray and Ohashi, 1983). This is due to the fact that the pullout resistance of the fibres in a soil mass is directly dependent on the normal or confining stress at the fibre-soil interface.

6.3 Comparison of deviator stress and volume change variation of natural and reinforced specimens

In Figures 6.8, 6.9 and 6.10 the deviator stress and volume change versus vertical strain change relationships are presented for different fibre concentrations for the silty sand specimens. Each comparison of corresponding parameters was made at the same confining stress. The peaks of strain and stress plots are also compared in Appendix C. The above three sets of data show that the strength of the reinforced specimens when compared to the natural soil is increasing with fibre concentration (more than the double of the natural soil strength at 0.1% fibre concentration and six times higher at 0.5% fibre content). At the same time the strain of the reinforced specimens at failure increases (at 0.5% fibre concentration it represents the double of the natural soil strain at failure).

The volume of the reinforced specimens decreases at higher deviator stresses. Therefore, the volume decreases (contraction) with increasing fibre concentration (by 3 to 7%).

In Fig. 6.11 the deviator stress versus vertical strain relationships at four different confining stresses for a fibre content of 0.1% are presented for the sandy silt specimens. The following observations can be made. The deviator stress increases with fibre concentration. At the same time, the strain to failure of the reinforced specimens increases (it doubles at almost all confining stresses) if compared with the corresponding natural soil. The peak strains and stresses were identified and are compared in Appendix C.

In Fig. 6.12 the deviator stress and volume change versus vertical strain relationships were presented for 0.1% fibre concentration and different confining stresses for the medium sand. Three comparisons of natural and reinforced soil were made for the various confining stresses. At all confining stresses, the deviator stress of the reinforced specimens increased compared to the corresponding natural soil. At the same time, the strain to failure increased (it doubled compared to the strain of natural soils at all confining stresses). At higher chamber pressures the volume change was less pronounced. In Appendix C, the peak deviator stresses and corresponding strains were identified and presented at different cell pressures.

In Fig. 6.13 the effect of 0.1 and 0.3% reinforcement concentrations on deviator stress and volume change are presented for a given cell pressure for the uniform sand specimens. The specimen with 0.1% fibres showed a peak stress at about 11% of vertical strain. This increase in strain of the dense uniform sand specimen reinforced with 0.1%

fibre was also plotted in Fig. 6.7 (as well as for the other three corresponding reinforced soils). A discussion of the increased strains and expected compression will be given in a later section. The residual deviator stress occurred at about 25% vertical strain. For a 0.3% fibre concentration, the peak deviator stress was reached at about 16% of vertical strain and the test was stopped before the residual was reached. The higher fibre concentration (0.3%) caused a larger contractile volume change. This decrease may be attributed to the fact that for a uniform soil gradation the fibres increased the porosity of the particulate soil phase (the unit dry weight of the particulate soil phase decreased from 16.0 kN/m^3 for the unreinforced specimens to 14.7 kN/m^3 for the specimens reinforced with 0.3% fibres).

The particulate soil phase refers to the volume occupied by the soil, water and air voids. The fibrous phase (polypropylene fibres) has a low specific gravity (0.78) as compared to the soil (2.65) and may be interpreted as voids between soil particles. This particularity of the soil-polypropylene as a composite material was also explained in section 3.6. Therefore, the unit dry weight of the particulate soil phase was computed by subtracting the volume occupied by the fibres from the total volume of the composite and by subtracting the weight of the fibres from the total weight of the composite.

Consequently, more volume contraction resulted with increasing fibre concentration. It is believed that the contraction was due (1) to the decrease of the unit dry weight of the particulate soil phase and (2) to the specific behaviour of the fibrous polypropylene phase (as identified by the oedometer tests, and discussed in section 6.7). It should be remembered that a dense sand shows an initial contractile behaviour

followed by a dilative phase. The effect of increasing the reinforcement concentration also resulted in a reduced dilative behaviour of the composite.

It can also be observed that the deviator stress increased by increasing the fibre concentration. At the same time the strain at failure of reinforced specimens increased (compared to the natural soil, the strain at failure more than tripled for 0.3% fibre concentration) (Fig. 6.13).

6.4 Volume change

The volume change results were presented in chapter 5. The sand specimens decreased in volume at small initial strains and thereafter dilated (Figs. 5.12, 5.13, 5.14). This is a typical behaviour for dense sand. However, a dense sand subjected to increased confining stress starts to behave as a loose sand which typically decreases in volume. Figure 5.14 confirms this fact for the uniform sand tested at a confining stress of 1400 kPa. This observation was also reported by Bishop and Henkel (1957). The compressibility of the fibre structure (as a distinct phase in the soil) contributes also to an additional decrease in volume during shear. This phenomenon will be explained in a following paragraph.

Of particular interest was the volume change of the coarse uniform sand (presented in the preceding paragraph). A supplementary resistance to compaction and lower unit dry weight were achieved in the case of a coarse uniform graded sand reinforced with 0.3% polypropylene fibres (unit dry weight of the particulate soil phase of 14.7 kN/m^3) as compared with the unreinforced specimens (unit dry weight of 16.0 kN/m^3). As the unit dry weight of a given soil particulate phase decreased, additional

decrease in volume is likely to occur under the same loading conditions. A comparison of natural and reinforced well graded sand specimens revealed the influence of the reinforcement upon the volume change behaviour (Fig. 6.12). The same comparison was made for the specimens of natural and reinforced uniform sand (Fig. 6.13).

Cyclic loaded tests were performed on specimens of well graded and uniform sands, (Figs. 6.14, 6.15 and 6.16). They showed that the volume of specimens decreased further at each cycle of loading. Repeated loading increased the density of the composite by pushing the particles in a denser arrangement. The load-unload tests also showed an increase in maximum deviator stress after each cycle with corresponding decrease in volume (and subsequent increase in density). Cyclic load tests on medium and uniform sand showed an increase in volume at unloading (Fig. 6.16). As was explained in section 3.4, this fact was due to the partial rebound of the arrays of stone grains after reduction of load. A small amount of water was drawn back into the soil as the applied load decreased, due to the dilative behaviour and some negative porewater pressures which were generated. Only a small fraction of the volume change can be recovered upon load reduction (Karol, 1960).

In the case of sandy silt and silty sand specimens no dilatation took place in both, natural and reinforced samples, and the volume decreased continuously (Figs. 5.1, 5.2, 5.3, 5.4, 5.5, 5.9, 5.10, 6.8, 6.9 and 6.10.) Best fit curves of volume change at the same fibre content and vertical strain was used in order to identify the trend of the volume change for reinforced silty sand and sandy silt specimens (Figs. 6.17 and 6.18). These plots show an increase in volume contraction with fibre content. The sandy silt

specimens exhibited a greater rate of volume contraction with strain than the corresponding silty sand samples at the same fibre content

6.5 Porewater pressure dissipation

Previous researchers have not reported porewater pressure measurements. The majority of their tests were performed on sand and were considered to be drained (in some cases the sand was tested dry). However, in the case of silts and fine sands, porewater pressures can develop during loading or unloading (Karol, 1960). This fact was also explained in section 4.5.2. Figures 6.19 and 6.20 show the behaviour of porewater pressures during a triaxial compression test under quasi drained conditions of silt and sand mixes. It can be observed (relating to the strain at peak deviator stress), that the excess porewater pressures were always dissipated before the peak deviator stress was reached, or they were low enough (1-2% of the peak deviator stress) to be considered negligible. It may also be observed that the porewater pressures of reinforced silt specimens during triaxial consolidated-drained compression tests (at constant loading rate) are higher than those of the natural soil and that these pressures are increasing with an increase in fines. A reinforced soil sample contracts more (decrease in volume) under higher loads than natural soil (Figs. 6.8 and 6.17). As the rate of strain in a given series of triaxial tests was kept constant, a larger volume of water was expelled or sucked in from the composite specimens as compared to unreinforced specimens.

6.6 Secant modulus

The secant modulus concept was used for both unreinforced and reinforced specimens since the stress-strain relations were nonlinear. The concept consists of

determining the secant modulus at successive vertical strains from deviator stress-strain relations obtained from triaxial compression tests. The method has the advantage of viewing and comparing the moduli of unreinforced and reinforced specimens at corresponding strains ranging from 0.2 to 6% for sands and 12% for silts.

In Figure 6.21 the secant modulus for silty sand specimens reinforced with 0.1, 0.2, 0.3 and 0.5% fibres is presented and compared. It should be noted that the initial composite secant moduli are lower than the corresponding unreinforced specimens for vertical strains ranging from 0.2 to 9%. The vertical strains at which the secant moduli relations for the reinforced specimens are intersecting the corresponding behaviour of the unreinforced specimens increase with fibre content (Fig. 6.21). Once the tensile strains are sufficiently mobilized in the fibres they restrain the movement of soil particles and reinforce the soil. The pronounced decrease of the reinforced secant modulus at small strains followed by a constant value may suggest a different behaviour of the composite at small and large strains. This behaviour may be due to the effect that at large strains the fibres become fully tensioned. In contrast, the secant modulus of the unreinforced specimens decreased continuously with strain.

The results of the oedometer tests are presented in Fig. 5.15. These data were used to plot the secant moduli presented in Fig. 6.22. As was pointed out in section 3.6, at small strains the behaviour of soils in confined compression and triaxial compression tests is similar. The plots of secant moduli from the oedometer tests showed the same behaviour as the moduli at the beginning of a triaxial test, namely a decrease of secant modulus of the composite with an increase in fibre concentration.

The comparison of secant moduli for unreinforced and reinforced specimens for medium and uniform sand (Figs. 6.23 and 6.24) revealed the same trends as for the silty sand. As presented in Figs. 6.25, 6.26, 6.27 and 6.28, the comparison of specimens at different confining stresses and corresponding fibre concentrations revealed that the stiffness of the reinforced and natural soil specimens increased with an increase of confining pressure.

In summary, a comparison of secant moduli for the four unreinforced soils with the corresponding reinforced specimens revealed the following general trend. The stiffness (secant modulus) of the composite decreased at small strains and increased slightly regardless of the fibre content. With respect to the reinforcement concentrations, the compression behaviour in the oedometer showed a decrease of secant moduli for the composite with fibre content. The results from triaxial tests also showed a decrease of secant moduli with fibre content.

6.7 Strain increase with fibre content

From Figs. 6.8, 6.11, 6.12 and 6.13, which were also discussed in section 6.3, it can be observed that the vertical strain for the reinforced specimens at peak stress increases with fibre content. This observation is true for all four soils. In Appendix C the peaks deviator stresses were plotted against the vertical strain and consequently the increase in corresponding strains can be more easily evaluated. The fact that the strains at failure of reinforced specimens increased, when compared to the natural soil, was also reported by other researchers (Hoare, 1979, Brooms, 1977, McGown et al., 1981, Gray and Ohashi, 1983 and Bouzaza et al., 1994). It was reported that the strain

corresponding to peak stress is also increasing with increasing confining pressure (McGown et al., 1981 and Schlosser et al., 1985). Therefore, it is expected that introducing fibres to a soil will increase the compression of the composite when subjected to loading.

It is suggested that this increase in vertical strain with fibre content is due to the compression of fibres themselves. This assumption is based on the observation discussed in the preceding paragraph, namely that the tangent moduli of reinforced specimens are decreasing markedly at small strains when the fibre content is increased. For the well graded sand the "small" strains were in the order of 0.2 to 1.5% (Fig. 6.23). The second factor which contributes to an increased peak strain with fibre content is due to the contraction of the soil under greater loads carried by the reinforced composite.

As presented in section 3.6, the behaviour of sand in confined compression is similar to the behavior experienced in triaxial compression until strains of 0.25% are reached. The advantage of the oedometer test consists in its simplicity in separating the compression of soil and fibres. This is not possible in triaxial compression (Figs. 5.15 and 5.16). In Fig. 5.15 it was shown that the vertical strain of the specimens increased with fibre content. In Fig. 6.29 the results of the immediate compression of the natural and 0.5% reinforced (composite) soil were presented (replotted from Fig. 5.15). The comparison of these results emphasized (the bold plot) the compression of fibres for a concentration of 0.5% (replotted from Fig. 5.16). Both compressions taken separately, when added, equalled the total immediate compression of the reinforced (0.5%) composite specimen. Based on the oedometer compression tests, it was found that the

compression of fibres plays a significant role in the global compression of the composite. The compression of fibres may also be a cause for the lower secant moduli of the composite (compared to the natural soil) at low strains in the triaxial test. Polypropylene fibres show a larger compressibility than the sand itself for the same vertical load (Figs. 5.15 and 5.16). The compression of the fibres only in the oedometer showed that the rebound after unloading was 100 times greater than the amount of rebound of the sand. The rebound secant moduli (unloading portion of the plot) of sand are the same in successive cycles (the slopes of the cords to the unloading portion are parallel) (Fig. 5.15).

Since the fibre concentration varied but the rebound moduli of the composites were constant, it may be concluded that the recoverable strain is independent of fibre concentration. From the preceding observations one may envisage the following soil-fibre interaction mechanism. In a triaxial compression test, at low initial strains, the fibres were deformed only in compression. This resulted in a decrease in the secant modulus.

6.8 Influence of fibres on different types of soil

The effect of fibre concentration on apparent cohesion and peak strain was compared for the four soils in Fig. 6.7. The vertical axis in Fig. 6.7 (a) shows the gain in apparent cohesion for a fibre concentration of 0.1%. The data were compiled from Figs. 6.1 to 6.4. The sandy silt and silty sand specimens showed a larger increase in apparent cohesion than the medium and uniform sand specimens. By computing the average strain at peak points of the stress-strain relations for each soil, a second relationship was

established in Fig. 6.7 (b). The average strains were compiled from Figures 5.2, 6.11, 6.12 and 6.13. The average strain increase at failure for the reinforced (0.1%) specimens was compared with the corresponding average strain of the natural soil at failure (taken as 100%). The average strain increase was 67% for both sands, 57% for sandy-silt and 38% for silty-sand specimens. The reinforced silty-sand exhibited the greatest increase in strength (cohesion) at the lowest increase in strain when compared to the corresponding unreinforced soil. This is the most desired effect for a reinforced soil, namely high tensile strains in the reinforcement at low vertical compression. The reinforced uniform sand specimens exhibited only 25% increase in apparent cohesion when compared to the silty sand at twice the vertical strain when compared to the unreinforced soil. It should be noted however that the strains at failure for the four types of soil were not compared with one another, but to the strains of the corresponding unreinforced soil specimens.

The increase in apparent cohesion at different values of vertical strain for the four test soils is given in Table 6.1. The highest values of apparent cohesion (last values in each row) correspond to the peak strain of each type of soil. It may be observed that the apparent cohesion increases more for fine soils (silty sand and sandy silt), but at larger amounts of vertical deformation. Another observation can be made at 10% vertical strain. This is the maximal strain at which the four soils can be compared. Above 10% vertical strain the comparison cannot be made because the medium sand and uniform sand failed. The silty sand shows the highest increase in apparent cohesion .

Table 6.1 Comparison of cohesion increase at given vertical strains.

	Vertical Strain (%)						
	2	5	10	15	18	20	22
	Apparent Cohesion Increase for 0.1% Fibre Content (kPa)						
Silty Sand	0	20	100	150	160	-	-
Sandy Silt	0	20	60	120	130	140	150
Medium Sand	0	60	80	-	-	-	-
Uniform Sand	0	20	40	-	-	-	-

6.9 Application of analytical model

In this section the application of the analytical model proposed by Gray and Ohashi (1983) is presented for the silty sand soil. The theoretical concept was presented in section 3.3. The application of this model requires the calculation of the cross-sectional area of the fibres transgressing the shear plane. The measurement of the cross-sectional area of the large number of fibres on the shear plane of a test specimen can be considered a drawback in the validation of the model. The technique of determining the cross-sectional area of discrete fibres is presented in appendix B. It was based on the counts of fibres on the shear plane of four specimens at a concentration of 0.1%. The total cross-sectional area of the fibres transgressing the shear plane was found to be 17 mm². The total area of the shear plane of the triaxial specimens was 13000 mm². The ultimate tensile stress of polypropylene fibres was found to be 93 MPa (Fig. 4.3). Hardly any pull-out of fibres occurred on the shear plane. The model takes into account the different angles of inclination of fibres with the failure plane (Fig. 3.2). As is shown in Fig. 6.30, different initial inclinations of the fibres with the shear plane were assumed.

The model yielded a peak value of 161 kPa for an angle of fibre inclination of 48 to 50° to the shear plane. Therefore the relations 3.17 and 3.14 which give the shear strength increase per unit area of soil yield:

$$t_R = \sigma_R \left(\frac{A_R}{A_C} \right) = 93000 \left(\frac{17}{13000} \right) = 121.6 \text{ kPa} \quad (3.17)$$

$$\Delta S_R = t_R (\sin i + \cos i \tan \phi) = 121.6 (\sin 50^\circ + \cos 50^\circ \tan 41^\circ) = 161 \text{ kPa} \quad (3.14)$$

The increase in strength for the silty sand was found experimentally to be 160 kPa (Figs. 6.1, 6.6 and 6.7). The theoretical and experimental values are in good agreement for an inclination of fibres with the failure plane of 48 to 50°. For inclinations of fibres ranging from 0 to 90° the theoretical values of shear strength ranged from 105 to 120 kPa. Therefore, it was concluded that the model offers sufficient validity to predict the increase in strength for a silty sand soil reinforced with 0.1% of polypropylene fibres if the average inclination of fibres with the shear plane was considered to be between 45 to 55°. Two other counts of fibres on shear planes were made for fibre concentrations of 0.2 and 0.3%. The experimental data showed the same proportionality of strength increase for a fibre concentration of 0.3% (Fig. 6.6). Consequently it was concluded that the model can be used for predicting the composite ultimate strength at any fibre concentration (up to the limiting 0.3% fibre concentration used in this study).

It should be noted that the Gray-Ohashi model (1983) was based on direct shear tests on sand. In this study the model was applied to triaxial compression test results on silty sand. Assuming the same fibre cross sectional area for a sandy silt the model

showed sufficient validity to predict the increase in shear strength (Figs 6.2 and 6.30). However it can be observed from Figs 6.3, 6.4 and 6.30 that the model yields higher values of shear strength for the sand at any fibre inclination if the same fibre cross sectional area is considered.

Fig. 6.1 Mohr-Coulomb failure envelopes for natural and reinforced silty sand

Fig. 6.2 Mohr-Coulomb failure envelopes for natural and reinforced sandy silt

Fig. 6.3 Mohr-Coulomb failure envelopes for natural and reinforced medium sand

Fig. 6.4 Mohr-Coulomb failure envelopes for natural and reinforced uniform sand

Fig. 6.5 Analysis of parallelism

Fig. 6.6 Apparent cohesion increase with fibre concentration

Fig 6.7 Comparison of "apparent cohesion" and peak strain with content in fine particles

Fig. 6.8 Stress-strain relations of silty sand specimens at 100 kPa cell pressure (0, 0.1, 0.2, 0.3 and 0.5% fibre content)

Fig. 6.9 Stress-strain relations of silty sand specimens at 300 kPa cell pressure (0, 0.1 and 0.2% fibre content)

Fig. 6.10 Stress-strain relations of silty sand specimens at 50 kPa cell pressure (0.2, 0.3 and 0.5% fibre content)

Fig. 6.11 Stress-strain relations of sandy silt specimens (0 and 0.1% fibre content)

Fig. 6.12 Stress-strain relations of medium sand specimens
(0 and 0.1% fibre content)

Fig. 6.13 Stress-strain relations of uniform sand specimens at 200 kPa cell pressure (0, 0.1 and 0.3% fibre content)

Fig. 6.14 Cyclic load test on sandy silt specimens
 (0 and 0.1% fibre content)

Fig. 6.15 Cyclic load tests on medium sand specimens at 600 kPa cell pressure (0 and 0.1% fibre content)

Fig. 6.16 Cyclic load test on a uniform sand specimen at 200 kPa cell pressure

Fig. 6.17 Volume change for silty sand specimens
(0, 0.1, 0.2, 0.3 and 0.5% fibre content)

Fig. 6.18 Volume change for sandy silt specimens
(0 and 0.1% fibre content)

Fig. 6 19 Porewater pressures for silty sand specimens
(0 and 0.1% fibre content)

Fig. 6.20 Porewater pressures for sandy silt specimens (0 and 0.1% fibre content)

Fig. 6.21 Comparison of secant moduli for silty sand specimens at 100 and 300 kPa cell pressure

Fig. 6.22 Oedometer comparison of secant moduli for medium sand specimens (0, 0.1, 0.2, 0.3 and 0.5% fibre content)

Fig. 6.23 Comparison of secant moduli for medium sand specimens at 100 and 400 kPa cell pressure (0 and 0.1% fibre content)

Fig. 6.24 Comparison of secant moduli for uniform sand specimens at 200 kPa cell pressure (0, 0.1 and 0.3% fibre content)

Fig. 6.25 Comparison of secant moduli for silty sand specimens (0 and 0.1% fibre content)

Fig. 6.26 Comparison of secant moduli for silty sand specimens (0 and 0.1% fibre content)

Fig. 6.27 Comparison of secant moduli for medium sand specimens (0 and 0.1% fibre content)

Fig. 6.28 Comparison of secant moduli for uniform sand specimens (0% fibre content)

Fig. 6.29 Oedometer compression of medium sand and polypropylene fibres (0 and 0.5% fibre content)

Fig. 6.30 Gray-Ohashi model (1983) applied to reinforced (0.1%) silty sand

CHAPTER 7: SUMMARY AND CONCLUSIONS

7.1 Summary

Discrete and randomly dispersed polypropylene fibres were considered to improve the mechanical properties of granular soils. This study consisted of carrying out series of triaxial compression and oedometer tests. The triaxial tests were performed under static and cyclic loading conditions. Four types of soils with different granulometries were tested. Two polypropylene fibre lengths were investigated as reinforcement. The triaxial specimens had a size of 100 mm in diameter and were 200 mm high. The unreinforced and reinforced soils were tested under confining pressures ranging from 50 to 700 kPa. The stress-strain, volume change and pore pressure relations were compared and analyzed in order to emphasize the effect of the reinforcement. The strength increase resulting from the "apparent cohesion" and the friction angles of the soils were investigated. The increase in apparent cohesion was correlated with the granulometry of the test soils in order to reveal the soil which would yield the highest increase in strength compared to the corresponding unreinforced soil. The secant modulus, the vertical deformation and volume change were compared and discussed. The variation of secant modulus with fibre content was obtained from oedometer and triaxial compression tests and the corresponding values were compared. The pore pressure values were used in order to establish strain rates for drained shearing conditions. The model proposed by Gray and Ohashi (1983), was validated for the experimental results obtained in this study.

These results have shown that the addition of polypropylene fibres to natural soils will significantly increase the strength and at the same the vertical strain of the test specimens. The examination of the failure planes of triaxial specimens showed that even relatively short fibres do not fail in pullout but in tension. A mechanical device for fibre unbundling was built. A technique of fibre count on the failed surface of a specimen was developed. A method of photographing fibres in the interior of the failed specimens was proposed (Appendix B). Based on the experimental results and the analysis conducted in this study, a number of specific and general conclusions are presented in the next section

7.2 Conclusions

7.2.1 Strength

The discrete and randomly dispersed polypropylene fibres increased the strength of all four soils tested. The increase in strength was determined using the Mohr-Coulomb failure law. This strength increase manifested itself as an increase in apparent cohesion. The angle of internal friction of the fine grained specimens of sandy silt and silty sand were not affected and the failure envelopes were parallel for all fibre concentrations. Above a certain length of fibres (which supplied sufficient friction and interlocking to prevent fibre pullout), the strength of reinforced specimens was not affected (the failure envelopes were parallel). The medium sand specimens exhibited an increase in apparent cohesion but also a slight decrease of the angle of internal friction (of about 2°). The increase in strength was proportional to the reinforcement concentration for 0.1, 0.2 and 0.3% of soil dry weight. Polypropylene fibres increased the residual deviator stress for all four reinforced soils. The maximum increase in

apparent cohesion was in the range 120 to 140 kPa for each 0.1% fibre concentration considered. Initially the specimens deformed in uniform bulging before a distinct failure plane developed.

7.2.2 Strain

The strain to failure increased with an increase in fibre concentration for specimens reinforced with fibres. The failure strains of both reinforced and natural soil specimens increased with an increase in the confining pressure. The oedometer test results showed an increase in vertical strain (specimen compression) when increasing the fibre content. The oedometer test was used to investigate the compressibility of the reinforced specimens and also to separate the compressibility of soil and fibres. The increased compressibility of the reinforced soil, might not be an important consideration for reinforced embankments, slopes and retaining walls, but could be important where settlement criteria apply.

7.2.3 Soil Modulus

The stiffness of reinforced and natural soil specimens increased with increasing confining pressure. A comparison of the four natural soil specimens with the corresponding reinforced ones revealed, as general trend, that the stiffness of the composite decreased at small strains and increased at further loading for all fibre contents. The compression behaviour in the oedometer cell showed a decrease in secant moduli of the reinforced soil with fibre content. The triaxial test results also showed a decrease in secant moduli with fibre content.

7.2.4 Volume Change and Porewater Pressures

All reinforced soil specimens exhibited larger contraction with an increase in fibre content. Dense sand specimens showed a smaller increase in volume with an increase in fibre content. The porewater pressures developed in reinforced specimens were higher during loading and dissipated at the same rate as those in unreinforced specimens. The measurement of the volume changes and pore pressure dissipation were valuable indicators in visualizing the mechanism of soil drainage at failure for specimens during static and cycling loading stages.

7.2.5 Soil Granulometry

Compared to the unreinforced specimens, reinforced fine grained soil specimens showed a larger increase in strength than the coarser grained specimens. Compared to the unreinforced soil specimens, fine grained reinforced specimens showed smaller values in vertical strain at peak stress than coarser grained specimens. Well graded specimens with large size particles, exhibited a smaller increase in strength than finer grained specimens. The increase in strength was greater for well graded specimens than for uniform graded specimens. The strength of the reinforced specimens increased with content in soil fines. Therefore it can be concluded that the angularity of particles and the granulometry plays an important role in strength increase. This conclusion is also valid for reinforced soil specimens.

7.2.6 Fibre Length

All fibres transgressing the failure plane (63 and 126 mm long), failed in tension (breakage) and not by pullout. Within the soil mass and away from the shear plane the

fibres were intact. No pullout type of failure was observed. It is concluded that if the fibres were properly dimensioned, as a function of cross-section, length and shape, a pullout type of failure can be avoided. The curly or unbundled shape of the fibres improved the friction, interlocking and three-dimensional orientation.

7.2.7 Analytical Model

The analytical model proposed by Gray and Ohashi (1983) was capable of predicting the strength increase of soils reinforced with polypropylene fibres if the angle of the fibres to the failure plane (at failure of the specimens) is known. In this study the average inclination of fibres was assumed to be 50° .

7.2.8 Recommendations for Future Research

The laboratory work upon which the conclusions of this study are based should be followed by field investigation and building a prototype soil structure reinforced with discrete polypropylene fibres. Different existing or new mixing techniques of the soil other than dry sand should be investigated. A mixing technique at optimum moisture content should be investigated in order to compact the reinforced soil at the standard or modified Proctor density. The compressibility and bearing capacity of soil reinforced with discrete polypropylene fibres may be conveniently tested in-situ using the traditional plate-load test. The plate-load test is useful for obtaining the soil modulus for the purpose of estimating the settlement of foundations on reinforced soil. Full scale tests of embankments with slopes ranging from 3:1 to 1:1 may be considered for soils reinforced with different fibre content.

REFERENCES

- Al-Refeai T.O. (1985), "Constitutive Behaviour of Fabric vs. Fiber Reinforced Sand", Ph.D. Thesis, Civil Engineering Department, University of Michigan, Michigan
- Andersland, O.B. and Khattak, A. S. (1979), "Shear Strength of Kaolinite / fiber Soil Mixtures", Proceedings, International Conference on Soil Reinforcement Reinforced Earth and Other Techniques, Paris, Vol 1, pp. 11-16.
- Andrawes, K.Z., McGown, A., and Kabir, M H., (1984), "Uniaxial Strength Testing of Woven and Nonwoven Geotextiles", Geotextiles and Geomembranes, Vol 1, No. 1, pp. 41-56.
- Arpad Kézdi, (1980), "Handbook of Soil Mechanics, Volume 2, Soil Testing", Elsevier Scientific Publishing Company, Amsterdam pp. 152-171.
- Atmatzidis, D.K , Athanasopoulos, G A and Papantonopoulos, C.I., (1994), "Sand-Geotextile Interaction by Triaxial Compression Testing", Proceedings, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, Vol 1, pp 377-380
- Bauer G E and Zhao Y J., (1994), "Effect of Soil Dilatancy on Shear Strength of Reinforced Composites", Proceedings, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, pp 369-372.
- Bishop A.W. and Henkel D J., (1957), "The Measurement of Soil Properties in the Triaxial Test", Edward Arnold, London
- Bishop, D.M., and Stevens, M.E , (1964), "Landslides on Logged Areas in South-east Alaska", USDA Forest Service Research Paper NOR-1, U.S Department of Agriculture, Juneau, Alaska, .
- Bowes H.W., Russell, L T and Suter G T., (1990), "Mechanics of Engineering Materials", BRS Publications
- Bowles J.E., (1978), "Engineering Properties of Soils and Their Measurement", 2nd ed., McGraw-Hill Book Company, New York
- Bouzaza, A., Amorkane K. and Aberkane T , (1994), "Granular Soil Reinforced with Geotextile and Randomly Oriented Fibers. A Comparison", Proceedings, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, 5-9 September, Vol 1, pp. 391-394.

- Broms, B.B., (1977), "Triaxial Tests with Fabric Reinforced Soil", Proceedings, International Conference on Use of Fabrics in Geotechnics, Vol. 3, pp. 129-133.
- Brown, C.B., and Sheu, M.S., (1975), "Effects of Deforestation on Slopes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No GT1, Jan., pp. 147-165.
- Burroughs, E.R., and Thomas, B.R., (1977), "Declining Root Strength in Douglas Fir After Felling as a Factor in Slope Stability," USDA Forest Service Research Paper INT-190, U.S. Department of Agriculture, Juneau, Alaska.
- Cannon, E.W., (1976), "Fabric in Civil Engineering", Civil Engineering, (U.K.), March, pp. 39-42.
- Cernica, J.N., (1995), "Soil Mechanics", John Wiley and Sons, Inc.
- Cheng L. and Jack B.E., (1984), "Soil Properties: Testing, Measurement and Evaluation", Prentice-Hall, Inc., Englewood Cliffs, New Jersey.
- Craig, R.F., (1992), "Soil Mechanics", Fifth edition, Chapman and Hall, London, UK.
- Dyer, N.R., and Milligan, G.W.E., (1984), "A Photoelastic Investigation of the Interaction of a Cohesionless Soil with Reinforcement Placed at Different Orientations", Proceedings, Session 2, International Conference on In-situ Soil and Rock Reinforcement, pp. 257-262.
- El-Fermaoui, A., and Nowatzki, E., (1982), "Effect of Confining Pressure on Performance of Geotextiles in Soil", Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, Vol. III, pp. 799-804.
- Endo, T., and Tsuruta, T., (1969), "The Effect of Tree Roots Upon the Shearing Strength of Soil," Annual Report No. 18 of the Hokkaido Branch, Tokyo Forest Experiment Station, Tokyo, Japan, pp. 168-179.
- Fatani, M.N., Bauer, G.E., and Al-Joulani, N., (1991), "Reinforcing Soil with Aligned and Randomly Oriented Metallic Fibres", Geotechnical Testing Journal, Vol. 14, No. 1, March, pp. 78-87.
- Fellenius, B.H., (1990), "Guidelines for the static loading test", Deep Foundations Institute, Sparta, NJ, USA, pp. 26-33.
- Fukoka, M., (1988), "Earth reinforcement-West and east", Proceedings, International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukoka, Japan.

- Fukoka M., Nakayama K., Ihara S., Okedoi K. and Ozaki K. (1991), "Stability of Fiber Reinforced Retaining Walls", Proceedings, 2nd Int. Conf. on Recent Advances in Geotech. Earthquake Eng. and Soil Dynamics, St. Louis, pp. 643-648.
- Giroud, J.P., (1984), "Geotextiles and Geomembranes", Geotextiles and Geomembranes Journal, Vol. 1, No. 2, pp. 5-40.
- Gomes et al., (1994), "Soil Reinforcement Interaction by In-Soil Tensile Tests and Direct Shear Tests", Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, pp. 395-400.
- Gray, D.H., (1978), "Role of Woody Vegetation in Reinforcing Soils and Stabilizing Slopes", Proceedings, Symposium on Soil Reinforcing and Stabilizing Techniques in Engineering Practice, The New South Wales Institute of Technology, Sydney, Australia, Oct. 16-19, pp. 253-306.
- Gray, D.H., and Megahan, W.F., (1981) "Forest Vegetation Removal and Slope Stability in the Idaho Batholith," USDA Forest Service Paper INT-271, Intermediate Forest and Range Experiment Station, Ogden, Utah.
- Gray, D.H., Athansopoulos, G., and Ohashi, H., (1982), "Internal / External Fabric Reinforcement of Sand", Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, Vol 3, pp. 611-616.
- Gray, D.H., and Leister, A.T., (1982), "Biotechnical Slope Protection", Van Nostrand Reinhold Co., New York, N.Y.
- Gray, D.H., Ohashi, H. (1983), "Mechanics of Fiber Reinforcement in Sand", Journal of the Geotechnical Engineering Division, ASCE, Vol. 109, No. 3, pp. 335-353.
- Gray, D.H. and Al-Refai, T., (1986), "Behavior of fabric versus fiber reinforced sand", Journal of Geotech. Eng., ASCE, Vol. 112, GT 8, August, pp. 804-820.
- Hayashi, S., Ochiai, H., Yoshimoto, A., Sato, K. and Kiamura, T., (1988), "Functions and Effects of Reinforcing Materials in Earth Reinforcement", Proceedings, International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukoka, Japan, pp. 99-104.
- Hoare, D.J., (1979), "Laboratory Study of Granular Soils Reinforced with Randomly Oriented Discrete Fibers", Proceedings, International Conference on Soil Reinforcement, Paris, Vol. 1, pp. 47-52.

- Hock, E., Brown, E.T., (1980), "Empirical Strength Criterion for Rock Masses", American Society for Civil Engineers, Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, GT 9, pp. 1013-1035.
- Holtz D.R. and Kovacs W.D., (1981), "An Introduction to Geotechnical Engineering", Prentice-Hall, Inc., Englewood Cliffs, New Jersey.
- Jewell R.A., Wroth C.P. (1987), "Direct Shear Tests on Reinforced Sand", Geotechnique, 37, No.1, pp. 53-68.
- Jewell, R.A., (1980), "Some Factors which Influence the Shear Strength of Reinforced Sand", Cambridge University Eng. Dept., Cambridge, England, CUED / D-Soils / TR 85, 37 pp.
- Jones, C.J.F.P., (1985), "Earth Reinforcement and Soil Structure", Butterworth and Co. Ltd.
- Juran, I., Schlosser, F., (1978), "Theoretical Analysis of Failure in Reinforced Earth Structures", Proceedings, ASCE Symposium Earth Reinforcement, Pittsburgh, pp. 528-555
- Juran, I., Guermazi, A., Chen, C.L. and Ider, M.H., (1988), "Modelling and Simulation of Load Transfer in Reinforced Soils: Part 1", International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 12, pp. 141-155.
- Juran, I., Ider, M.H., Chen, C.I. and Guermazi, A., (1988), "Numerical Analysis of the Response of Reinforced Soils to Direct Shearing: Part 2", International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 12, pp. 157-171.
- Juran, I., Farrag, K. and Richmond, L., (1991), "Short and Long term Performance of Polymeric Geogrids", Proceedings, Geosynthetics '91, Atlanta, p. 13.
- Karol, R.H., (1960), "Soils and Soil Engineering", Prentice-Hall, Inc.
- Kassif, G., and Kopelovitz, A., (1968), "Strength Properties of Soil-Root Systems", Technion Institute of Technology, Haifa, Israel.
- Khay M., Gigan J.P., Ledelliou M., (1990), "Reinforcement with continuos thread: Technical developments and design methods", Proceedings 4th Int. Conf. on Geotextiles, Geomembranes and Related Products, pp. 21-26.

- Kuwano, J., Imamura, Y., Sakurai, M., Ogawa, K. and Ozaki, K., (1994), "Simple Shear Test on Sand Reinforced by Continuous Fibers", Proceedings, Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, Vol. 1, pp. 357-360.
- Lambe, T.W. and Whitman, R.V., (1979), "Soil Mechanics, SI Version", John Wiley & Sons, pp. 61-70, 116-160.
- Lee, K.L., Adams, B.D. and Vagneron, J.J. (1973), "Reinforced Earth Retaining Walls" Journal of Soil Mech. and Found. ASCE, Vol. 99, SM 10, pp. 745-764.
- Leflaive, E. (1982), "The Reinforcement of Granular Materials With Continuous Fibers", Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, Vol. 3, pp. 721-726.
- Leflaive, E. and Liausu, Ph., (1986), "The Reinforcement of Soils by Continuous Threads", Proceedings, 3rd ICG, Vol. 2, pp. 523-528.
- Leflaive, E. (1988), "Texol: Already More than 50 Successful Applications", Proceedings, 1st International Symposium Earth Reinforcement Practice, Fukuoka, pp. 541-545.
- Long, N.T., Guegan Y. and Legeay G., (1972), "Etude de la Terre Armee a l'Appareil Triaxial", Rapport de Recherche No. 17, Laboratoire Central des Ponts et Chaussees, Paris, France.
- Maher, M.H. and Gray, D.H. (1990), "Static Response of Sands Reinforced with Randomly Distributed Fibers", Journal of Geotechnical Engineering, ASCE, Vol. 116, pp. 1661-1677.
- McGown, A., Andrawes, K.Z., Wilson Fahmy, R.F. and Brady, K.C. (1981) "A new Method of determining the Load Extension Properties of Geotechnical Fabrics, TRRL, Report no. 104, Crowthorne, England.
- McGown, A. and Andrawes, K.Z., (1977), "The Influence of Non-Woven Fabric Inclusions on the Stress-Strain Behaviour of a Soil Mass", Proceedings, International Conference on the use of Fabrics in Geotechnics, Paris, Vol. 1, pp. 161-166
- McGown, A., Andrawes and Hasani, M.M., (1978), "Effect of Inclusion Properties on the Behaviour of Sand", Geotechnique, No. 3.

- McGown, A., Andrawes, K.Z. and Kabir, M. H., (1982), "Load-extension Testing of Geotextiles Confined in-soil", Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, Vol. 3, pp. 793-798.
- Murray, R.T., and McGown, A., (1982), "The Selection of Testing Procedures for the Specification of Geotextiles", Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, Vol. II, pp. 291-296 and discussion Vol. IV, pp. 62-64.
- O'Rourke, T.D., Druschell, S.J. and Netravali, A.N., (1990), "Shear Strength Characteristics of Sand-Polymer Interfaces", Journal of Geotechnical Engineering, Vol. 116, No. 3, pp. 451-469.
- Schlosser, F., Magnan, J.P. and Holtz, R.D. (1985) "Geotechnical Engineered Construction", Proceedings 11th Int. Conf. Soil Mech. Found. Eng., San Francisco, pp. 2499-2539.
- Schlosser, F. and Long, N.T., (1972), "Behaviour of Reinforced Earth for Retaining Structures", Proceedings, 5th, European Conference on Soil Mechanics and Foundations Engineering, Madrid, Vol 1, pp. 299-306.
- Schlosser, F. and Long, N.T., (1974), "Recent Results in French Research on Reinforced Earth", Journal of Constructions Div. ASCE, Vol. 100, pp. 223-237.
- Schlosser, F. and Elias, V., (1978), "Friction Reinforced Earth", Proceedings, ASCE Symposium on Earth Reinforcement, Pittsburgh.
- Vidal, H., (1966) "La Terre Armee", Annales de l'Institut Technique des Batiments et des Travaux Publiques, Paris, pp. 888-938.
- Waldron, L.J., "Shear Resistance of Root-Permeated Homogeneous and Stratified Soil," Soil Science Society of America Proceedings, Vol. 41, 1977, pp. 843-849.
- Wu, T.H., (1976), "Investigation of Landslides on Prince of Wales Island, Alaska," Geotechnical Engineering Report No. 5, Department of Civil Engineering , Ohio State Univ., Columbus, Ohio.
- Yang, Z., (1973), "Strength and Deformation Characteristics of Reinforced Sand", Ph. D. Thesis, University of California, Los Angeles.

TEST 63

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = shear plane

Max. vertic stress ($\sigma_1 - \sigma_3$) = 2379.31 kN/m²
 Min. pr. stress (σ_3) = 201.04 kN/m²
 Maj. pr. stress (σ_1) = 2580.35 kN/m²

Specimen Data

Weight of silt = 2512.1 g
 Weight of fibers = 5 g
 Specimen weight = 2517.1 g
 Height = 203 mm
 Diameter = 101 mm
 Original area = 80.07 cm²
 Specimen volume = 1625.5 cm³
 Volume of fibers = 6.4 cc
 Volume of silt = 1619.11 g
 Silt's unit dry weight = 1.55 g/cm³

Fig A.1 Results for test no. 63

TEST 64

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = shear plane

Max. vertic. stress ($\sigma_1 - \sigma_3$) = 1815.82 kN/m²
 Min. pr. stress (σ_3) = 301.17 kN/m²
 Maj. pr. stress (σ_1) = 2116.99 kN/m²

Specimen Data

Weight of silt = 2622.2 g
 Weight of fibers = 2.5 g
 Specimen weight = 2724.7 g
 Height = 197.5 mm
 Diameter = 98.5 mm
 Original area = 76.16 cm²
 Specimen volume = 1504 cm³
 Volume of fibers = 3.2 cc
 Volume of silt = 1501.5 g
 Silt's unit dry weight = 1.74 g/cm³

Fig. A 2 Results for test no. 64

TEST 65

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = shear plane

Max vertic stress ($\sigma_1 - \sigma_3$) = 1206.91 kN/m²
 Min. pr. stress (σ_3) = 101.7 kN/m²
 Maj. pr. stress (σ_1) = 1308.61 kN/m²

Specimen Data

Weight of silt = 2526.8 g
 Weight of fibres = 2.5 g
 Specimen weight = 2529.3 g
 Height = 203 mm
 Diameter = 98.5 mm
 Original area = 76.16 cm²
 Specimen volume = 1546 cm³
 Volume of fibres = 3.2 cc
 Volume of silt = 1542.8 g
 Silt's unit dry weight = 1.64 g/cm³

Fig. A.3 Results for test no. 65

TEST 68

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = shear plane

Max vertic stress ($\sigma_1 - \sigma_3$) = 2108.5 kN/m²
 Min. pr. stress (σ_3) = 401.3 kN/m²
 Maj. pr. stress (σ_1) = 2509.8 kN/m²

Specimen Data

Weight of silt = 2637.9 g
 Weight of fibres = 2.5 g
 Specimen weight = 2640.4 g
 Height = 201 mm
 Diameter = 98.25 mm
 Original area = 75.77 cm²
 Specimen volume = 1523.1 cm³
 Volume of fibres = 3.2 cc
 Volume of silt = 1519.9 g
 Silt's unit dry weight = 1.73 g/cm³

Fig. A 4 Results for test no. 68

TEST 69

Material = silty sand
 Type of test = C.D
 Loading rate = 0.33 mm/min
 Type of failure = multiple shear
 planes+vert cracks
 Max. vertic stress ($\sigma_1 - \sigma_3$) = 1601.69 kN/m²
 Min. pr. stress (σ_3) = 201.04 kN/m²
 Maj. pr. stress (σ_1) = 1802.73 kN/m²

Specimen Data

Weight of silt = 2593.6 g
 Weight of fibres = 2.5 g
 Specimen weight = 2596.1 g
 Height = 201.5 mm
 Diameter = 98 mm
 Original area = 75.39 cm²
 Specimen volume = 1519.1 cm³
 Volume of fibres = 3.2 cc
 Volume of silt = 1515.9 g

Silt's unit dry weight = 1.71 g/cm³

Fig. A.5 Results for test no. 69

TEST 70

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = parallel shear planes (double)
 Max vertic stress ($\sigma_1 - \sigma_3$) = 1991.65 kN/m²
 Min. pr. stress (σ_3) = 300.78 kN/m²
 Maj. pr. stress (σ_1) = 2292.43 kN/m²

Specimen Data

Weight of silt = 2609.1 g
 Weight of fibres = 2.5 g
 Specimen weight = 2611.6 g
 Height = 203 mm
 Diameter = 98.5 mm
 Original area = 76.16 cm²
 Specimen volume = 1546.1 cm³
 Volume of fibres = 3.2 cc
 Volume of silt = 1542.9 g
 Silt's unit dry weight = 1.69 g/cm³

Fig. A.6 Results for test no. 70

TEST 71

Material = silty sand
 Type of test = C D
 Loading rate = 0.33 mm/min
 Type of failure = parallel shear
 planes (double)
 Max vertic stress ($\sigma_1 - \sigma_3$) = 2393.84 kN/m²
 Min. pr. stress (σ_3) = 400.90 kN/m²
 Maj. pr. stress (σ_1) = 2794.74 kN/m²

Specimen Data

Weight of silt = 2555.3 g
 Weight of fibers = 2.5 g
 Specimen weight = 2557.8 g
 Height = 204 mm
 Diameter = 98 mm
 Original area = 75.39 cm²
 Specimen volume = 1538 cm³
 Volume of fibers = 3.2 cc
 Volume of silt = 1534.8 g
 Silt's unit dry weight = 1.66 g/cm³

Fig. A.7 Results for test no. 71

Plate B.1 Polypropylene fibres 63 and 126 mm long

Plate B.2 Fibre count technique

Plate B.3 Examin. of the failure plane from a sandy silt specimen (0.1% fibre content)

Plate B.4 Examin. of the failure plane from a sandy silt specimen (0.1% fibre content)

Plates B.5 and B.6 Examination of the failure plane from a medium sand specimen (0.1% fibre content)

Plate B.7 Fibre investigation in a failed sandy silt specimen

Fig. C.1 Comparison of peak deviator stress and strain for silty sand specimens (0, 0.1 and 0.2% fibre content)

Fig. C.2 Comparison of peak deviator stress and strain for sandy silt specimens (0 and 0.1% fibre content)

Fig. C.3 Comparison of peak deviator stress and strain for medium sand specimens (0 and 0.1% fibre content)

Fig. C.4 Comparison of peak deviator stress and strain for uniform sand specimens (0, 0.1 and 0.3% fibre content)

Fig. C.5 Comparison of peak deviator stress and strain for all four test soils at 100 kPa cell pressure (0 and 0.1% fibre content)

TABLE OF REFERENCES IN CHRONOLOGICAL ORDER**D.1 Triaxial compression tests for fibre reinforced soil**

Lee, et al. (1973)
Andersland and Khattak (1979)
Hoare (1979)
Leflaive (1982, 1988) (French Ministry of Public Work)
Al-Refeai (1985)
Maher and Gray (1990)
Bouzaza et al. (1994)

**D.2 Triaxial compression tests for reinforced soil
(other reinforcements than fibres)**

Vidal (1966)
Long and co-workers (1972)
Schlosser and Long (1972)
Yang (1972)
Brooms (1977)
McGown and Andrawes (1977)
Juran and Schlosser (1978)
McGown et al. (1978)
Schlosser and Elias (1978)
Hock and Brown (1980)
McGown et al. (1981)
Gray et al. (1982)
Schlosser et al., (1985)
Mitchell and Villet (1987)
Juran, (1988) (Part one)
Atmatzidis et al (1994)

D.3 Direct and simple shear tests for fibre reinforced soil

Gray (1978)
Jewell (1980)
Gray and Ohashi (1983)
Dyer and Milligan (1984)
Hayashi et al. (1988)
Juran et al (1988) (Part two)
Khay et al. (1990)
O'Rourke, Druschell and Netravali (1990)
Fatani et al. (1991)

Fukoka et al (1991)
Bauer and Zhao, (1994)
Gomes et al. (1994)
Kuwano et al. (1994)

D.4 Properties of fibres and geotextiles used in soil reinforcement

Cannon (1976)
Finnigan (1977) and Klobbie (1977)
Van Leeumen (1977) and Allen et al. (1982)
Schwab et al (1977)
McGown et al. (1978)
Bell and Hicks (1980)
Allen et al. (1982)
El-Fermaoui and Nowatzki (1982)
Andrawes et al. (1984)
McGown et al. (1982)
Murray and McGown (1982)
Giroud (1984)
Juran et al (1991)

END

1 8-0 6-9 7

FIN