

Centrifuge modelling of rainfall-induced landslides in unsaturated soil slopes



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Nottingham

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This dissertation is submitted for the degree of
Doctor of Philosophy

June 2019

*This is dedicated to my beloved father
who's thoughts, ideas and vision will always be alive...*

Declaration

I hereby declare that except where specific reference is made to the work of others, the contents of this dissertation are original and have not been submitted in whole or in part for consideration for any other degree or qualification in this, or any other university. This dissertation is my own work and contains nothing which is the outcome of work done in collaboration with others, except as specified in the text and Acknowledgements. This dissertation contains fewer than 100,000 words including appendices, bibliography, footnotes, tables and equations.

Vasileios Matziaris

June 2019

Acknowledgements

My first and foremost thanks is going to my supervisors, Alec Marshall and Charlie Heron. Alec gave me the opportunity to be involved in the ITN Marie Curie project MUMOLADE and do my PhD studies at the University of Nottingham. His support, advice and especially patience were offered to me very generously at all times. He managed to keep me in track but, at the same time, he gave me an extraordinary amount of freedom to apply my ideas to the project. Charlie was always there to support me and feed me with many new ideas. His guidance was very helpful and his comments helped me to improve significantly. Moreover, I would like to thank my former supervisor, Prof. Hai-Sui Yu, for his encouragement. His experience and support was continuous.

I am grateful to the ITN Marie Curie research project MUMOLADE funded by the European Commission for funding my research. All the people from the MUMOLADE project, who created a very strong research group, are kindly acknowledged. Especially, I would like to thank Prof. Wei Wu and the people from BOKU university whose ideas turned this research into a great experience. Furthermore, all my colleagues (ESRs and ERs) who were great friends during that journey. Finally, to all people who organised the workshops and summer/winter schools during the project.

The difficult part of this study, centrifuge modelling, would not be able to be conducted if there was no great support by the technicians. Most of the acknowledgements are going

to Craig, unbelievable Jim, *young* Steve, Luke and Solomon who supported me every time I needed their help. Without them, this work would be still in the planning phase! I feel, also, the need to thank Mike, Nigel, Lee, *big* Steve and Gary.

I am thankful to my colleagues for their help, friendship and support and for the incredible times we had in and out of the university. Andrea, Mo, Itai, Bo, Mohsen, Edward, John, Amid and Ali are only some of those. I am grateful to Piotr Osinski for his help in the design and implementation of the tensiometer method in the laboratory and to David Toll for his advices and support though the project. A great thanks goes also to Nathan Vimalan, CEO of VJTech Ltd, for his endless support since I joined the company in 2017.

Last, but not least, the biggest thank-you goes to my family; Georgia who is my support in life for the last 20 years and my sons, Alexandros and Filippas, for their love, the big big hugs and incredible smiles they gave me every time they felt I was stressed. I own you everything...

Abstract

Landslides and debris flows are geohazards common to countries with mountainous terrains. A significant number of rainfall-induced landslides occurs in regions with tropical and sub-tropical climates where residual and colluvial soils are widespread. This study is in the context of the Innovative Training Network project MUMOLADE (Multiscale Modelling of Landslides and Debris Flows) which aims to study the multi-scale and multi-phase analysis of landslides and debris flows. The research presented here aims to investigate the initiation conditions of landslides in soil slopes under varied rainfall conditions.

A physical modelling experimental investigation on the initiation of rainfall-induced landslides in unsaturated soil slopes was performed in this study using the geotechnical centrifuge at the University of Nottingham. To accomplish the objectives of the study, a testing apparatus was designed and built. The tests were conducted in a plane-strain centrifuge box with embedded systems for controlling rainfall and groundwater conditions. The container has a transparent Perspex window at one side which provided the ability to measure subsurface ground displacements using digital image analysis. Pore Pressure Transducers (PPTs) were used for measuring pore water pressures within the slope models during the tests.

Initially, the study focused on the behaviour of a fine, uniform sand. Classification tests of the soil were performed in the laboratory, including the Soil Water Retention Curve (SWRC) determined using the tensiometer method. Sand slope models were built at a high inclination angle, about 1.5 times higher than the soil's angle of friction, with suction providing the necessary cohesive component for stability at 1g. The simulation of two failure mechanisms

was attempted in this study. The first mechanism involves the loss of suction due to wetting in a soil slope of an angle higher than the angle of friction due to wetting. The second mechanism involves the collapse of the saturated voids due to increased pore water pressure which reduces the effective stresses within the soil mass. The second mechanism was not able to be reproduced in the tests conducted in initially unsaturated soil slopes with a low GWT, since the models showed unexpected resistance to failure. On the other hand, soil slope models failed under a condition of an elevated (i.e. shallow) GWT.

In order to study further the unsaturated soil conditions under increased gravity tests were performed in which drainage of the soil under these conditions was enabled and measurements of the retained water content and degree of saturation at different g-levels were made. Results indicated that even medium-grained soils retain an amount of water content in their mass and that the retained water content at each g-level strongly depends on the grain size distribution of the soil. The implication of these tests is that soil slopes retain their stability when gravity increases even at extreme conditions, i.e. at an angle 50% higher than the angle of friction.

In order to overcome the scaling issue of the increased seepage velocity, slope models were built from a slightly finer soil which was scaled down by $\sqrt{N}$. These soil slopes were tested under similar boundary conditions at the sand slopes. The results indicated that the reduction of the permeability caused instability initiation under different combinations of rainfall intensity and duration. Slope centrifuge tests were simulated using limit equilibrium analysis in order to validate the experimental results. The analysis revealed the previous statement, that the elimination of suction was not sufficient to reduce the shear strength of the soil to a level that would cause instability in the slopes. Failure was attributed to the small increase of positive pore water pressures which were generated only to the slopes with reduced seepage velocity (i.e. in slopes made of M_{90-10}). Therefore, grain size scaling in soil slope models was found to provide the necessary conditions for the simulation of rainfall-induced landslides in centrifuge tests. Rainfall thresholds for the occurrence of rainfall-induced landslides were obtained and

compared to existing data from the literature. The comparison showed that the centrifuge tests conducted in this study provided an over-estimated rainfall threshold.

Table of contents

List of figures	xiv
List of tables	xxvii
1 Introduction	1
1.1 Overview	1
1.2 Research aim and objectives	2
1.3 Outline of the thesis	3
2 Literature review	5
2.1 Introduction	5
2.2 Unsaturated soils	5
2.2.1 Effective stress and shear strength	5
2.2.2 Pore water pressure measurements	10
2.2.3 Moisture, pore pressure and stress profiles	14
2.2.4 Soil Water Retention Curve (SWRC)	15
2.2.5 Flow in unsaturated soils	15
2.3 Slope stability	17
2.3.1 Infiltration in unsaturated soil slopes	17
2.3.2 Triggering mechanisms of landslides due to rainfall	22
2.3.3 Rainfall characteristics	24

2.4	Centrifuge modelling	25
2.4.1	General aspects of centrifuge modelling	25
2.4.2	Scaling laws	26
2.4.3	Scale effects	29
2.5	Centrifuge modelling of rainfall-induced landslides	34
3	Instrumentation and apparatus	40
3.1	Geotechnical centrifuge facility	40
3.2	Testing apparatus	42
3.2.1	Strong box	42
3.2.2	Rainfall system	44
3.2.3	Groundwater control system	51
3.2.4	Image capture system	54
3.3	Tensiometers	56
3.3.1	Saturation and calibration system	60
3.3.2	Saturation of tensiometers	65
3.3.3	Calibration of tensiometers	68
3.3.4	Tensiometer cavitation	70
3.4	Deformation measurements	72
3.4.1	geoPIV	73
3.4.2	PIVlab	77
3.5	Summary	81
4	Soil characterisation and slope model preparation	83
4.1	Introduction	83
4.2	Soil classification	84
4.2.1	Scanning Electron Microscope	85

4.2.2	Particle size distribution	87
4.2.3	Particle density	92
4.2.4	Atterberg limits	93
4.2.5	Compaction tests	94
4.2.6	Saturated permeability	98
4.2.7	Capillary rise tests	101
4.2.8	Soil Water Retention Curve (SWRC)	105
4.2.9	Strength properties	126
4.2.10	Summary from laboratory testing programme	138
4.3	Soil model preparation	141
4.3.1	Sandy slope models	141
4.3.2	Silty sand models	145
4.3.3	PPT installation	146
4.4	Testing programme	146
5	Slope model tests	149
5.1	Introduction	149
5.1.1	Pore water pressure and deformation measurements	150
5.2	Rainfall effect in slopes with deep groundwater table	153
5.2.1	Effect of gravity increase	154
5.2.2	Effect of rainfall intensity on loose slopes	155
5.2.3	Effect of rainfall intensity in dense slopes	160
5.2.4	Summary of tests with deep GWT	166
5.3	Slope stability tests with shallow groundwater table	170
5.3.1	Loose slope	171
5.3.2	Dense slope	172
5.3.3	Summary	178

5.4	Flow path in centrifuge modelling	183
5.4.1	Model preparation	183
5.5	Chapter summary	188
6	Soil column tests	190
6.1	Introduction	190
6.2	Experimental setup	193
6.2.1	Stress distribution and error	195
6.2.2	Measuring system	198
6.2.3	Calibration	199
6.2.4	Materials and model preparation	206
6.3	Dry tests	208
6.4	Static wet tests	213
6.4.1	Retaining moisture content	213
6.4.2	Pore water pressure variations	224
6.5	Infiltration tests	228
6.6	Discussion	230
6.6.1	Conclusions	231
7	Centrifuge tests in silty sand slopes	234
7.1	Introduction	235
7.2	Theoretical approach	236
7.3	Experimental validation	238
7.3.1	Methodology	238
7.3.2	Results	239
7.3.3	Numerical validation of tests CSI1 and CSI3	251
7.3.4	Discussion	262

7.3.5	Rainfall thresholds	264
7.4	Chapter summary	266
8	Conclusions and future work	268
8.1	Conclusions from the current study	268
8.2	Future work	271
8.2.1	Soil column tests	271
8.2.2	Slope stability	271
	REFERENCES	272

List of figures

2.1	Experimental results showing the relationship between the effective stress parameter χ and the degree of saturation S_r for various soils (Lu and Likos, 2004).	6
2.2	Extended Mohr-Coulomb failure surface for an unsaturated soil (Fredlund and Rahardjo, 1993)	7
2.3	Microstructural analysis of interparticle interaction in a a) dry, b) saturated, c) unsaturated soil. F_x and F_y are external forces acting on the x- and y-plane (Zhou et al., 2016)	9
2.4	Example of a 10-m thick sandy layer and profiles of volumetric water content, pore water pressure, total and effective stresses under a) Saturated and b) Unsaturated conditions (Lu and Likos, 2004).	14
2.5	Conceptual Soil-Water Retention Curve (SWRC) for unsaturated soils and respective desaturation states (modified after Fredlund, 2000).	16
2.6	Suction profiles for a weathered rhyolite slope in Hong Kong during the rainy season (Sweeney, 1982).	18
2.7	Pore water pressure responses to rainfall infiltration at different depths (Toll, 2001).	19

2.8	Typical pore water pressure profiles in unsaturated soils with various flux conditions: (a) steady-state conditions, (b) transient conditions ($q/k_{sat} < 1$) and (c) transient conditions ($q/k_{sat} \geq 1$) (Zhang et al., 2011).	20
2.9	Variation of FoS with time for 4 different scenarios (Matziaris et al., 2005). .	21
2.10	Comparison between stress paths for rain-induced slope failure and for triaxial tests (Brand, 1982).	22
2.11	Stages in first-time slope failures (Leroueil, 2001).	24
2.12	Self-weight stress in the prototype scale and in the model scale at rest and under centrifugal acceleration.	27
2.13	Representation of the vertical stress distribution in a soil model under g and its prototype (after Taylor, 1995).	30
2.14	Effect of centrifugal acceleration in droplet trajectories, at an initial velocity of 20m/s: (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015). . . .	32
2.15	Effect of initial velocity of droplet V_o in droplet trajectories, at an acceleration of 20g: (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015). . . .	33
2.16	Effect of the droplet size in droplet trajectories, at an acceleration of 20g: (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015).	33
2.17	Observed seasonal pore pressure cycles (Take, 2003)	35
2.18	Threshold curves: rainfall duration versus rainfall intensity relationships (Ling et al., 2009b)	38
3.1	Main components of the geotechnical centrifuge (Ellis et al., 2006).	41
3.2	Centrifuge box used for slope model tests.	43
3.3	Bottom side of the lid.	44
3.4	Rainfall distribution system.	45
3.5	Overview of the slope model and the location of the spraying nozzles	46
3.6	Calibration results for the flow meter.	48

3.7	a) Layout of spraying nozzles, type ‘M’ (Take, 2003), b) Types M1, M5 and M15.	49
3.8	(a) Calibration of the flow rate for nozzle M1, (b) flow rates for nozzles M1, M5 and M15.	50
3.9	(a) Top view and (b) side view of the groundwater control system.	52
3.10	Boundary conditions applied to the slope model tests. (a) deep groundwater table (b) shallow groundwater table.	53
3.11	Prosilica camera and 8 mm high resolution lens.	55
3.12	Position of the Prosilica camera for capturing images during the centrifuge tests.	56
3.13	(a) Druck PDCR-81 Pore Pressure Transducer (after König et al., 1994), (b) EPB-PW tensiometer (Take, 2003).	59
3.14	X-ray view of the saturation and calibration rig.	60
3.15	Saturation and calibration system for the PPTs (not to scale). Numbers 1-11 indicate valves.	62
3.16	Performance of PPTs a) before initial saturation and b) after 1 initial saturation cycle.	69
3.17	Cavitation of PPTs 2 and 5.	71
3.18	Image manipulation during PIV analysis (White et al., 2003).	74
3.19	(a) Chosen region for determination of pixel intensity for PIV analysis. (b) Distribution of the normalized intensity for the pixels of the chosen region. . .	76
3.20	Photogrammetric target for the accurate calibration of the reference points (White et al., 2003).	77
3.21	Overview of the workflow and the features implemented in PIVlab (Thielicke and Stamhuis, 2014).	78
3.22	Validation of the obtained vectors and elimination of wild vectors (Thielicke, 2014).	80

3.23	Example of PIVlab analysis of a slope subjected to an elevated groundwater table condition.	81
4.1	Scanning Electron Microscope images of Leighton Buzzard sand, fraction E. In photos a and b the uniformity of the sand is shown whereas in photos c and d the angular shape of the soil grains is shown	86
4.2	Spectrum analysis of Leighton Buzzard fraction E sand.	86
4.3	Scanning Electron Microscope images of Congleton sand.	87
4.4	Scanning Electron Microscope images of the sandy silt.	88
4.5	Spectrum analysis for silt. The dominant element in cases a, b and d is Silica while in the case c Calcium is the main element.	88
4.6	Grain size distribution curves of employed soils. Size ranges for the categories shown (i.e. Silt, fine/medium/coarse Sand) are based on <i>BS1377 : 1990</i>	90
4.7	Classification of the M-Soil in the triangular classification chart	91
4.8	Grain size distributions of the sand-silt mixtures.	92
4.9	Cone penetration depth - water content graph for the determination of the LL of the M-Soil.	94
4.10	Compaction test results for light, heavy and vibrating hammer methods (Leighton Buzzard, fraction E)	96
4.11	Compaction curve for the M_{90-10} mixture and air void lines	97
4.12	Compaction curve for the M_{70-30} mixture and air void lines	98
4.13	Saturated coefficient of permeability versus void ratio for LB fraction E sand.	100
4.14	Arrangement for falling head permeability test in the BS compaction mould (Head, 2006).	101
4.15	Capillarity model.	102
4.16	Capillary rise tests in Leighton Buzzard sand - fraction E at equilibrium. . . .	104
4.17	Typical Soil Water Retention Curve (Toll, 2012).	106

4.18	a) Primary drying and wetting SWRCs for two different fractions of LB sand, determined with the pressure plate method. "vG" are the van Genuchten fitting curves to the experimental data. b) Grain-size distribution of LBS1 and LBS2 (Kasangaki, 2012).	107
4.19	Results from the SWRC determination of a sandy clay using various testing methods (Toll et al., 2015).	108
4.20	Pedestal used for the development of the SWRC. Plastic cap, metallic ring and soil specimen are not shown in (b) for simplicity.	110
4.21	Experimental setup for determining SWRC.	111
4.22	Evaporation curves for LB-fraction E sand specimens. Values on the y-axis refer to the bulk gravimetric water content as it was determined by the balance readings. The gradients of the curves refer to the linear branch.	112
4.23	Experimental drying SWRC for LB sand, fraction E.	114
4.24	Variation of pore water pressure with time for a SWRC test using tensiometer.	115
4.25	Experimental data for the drying and wetting SWRC for the M_{90-10} mixture.	117
4.26	Experimental drying SWRC for the M_{70-30} mixture.	118
4.27	Experimental drying SWRC for the M-Soil.	119
4.28	Drying Soil Water Retention curves for LB sand, fraction E, determined using the van Genuchten method. (a) primary drying curve at loose state ($e = 0.78$) and (b) primary drying curve at dense state ($e = 0.66$)	122
4.29	Drying SWRC for the M_{90-10} mixture, determined using the van Genuchten method.	122
4.30	Drying SWRC for the M_{70-30} mixture, determined using the van Genuchten method.	123
4.31	Drying SWRC for the M-Soil, determined using the van Genuchten method.	124

4.32	Wetting branch (desorption curve) of the SWRC for LB sand, fraction E. (a) primary wetting curve and (b) secondary wetting curve.	125
4.33	Absorption curve for the M_{90-10} mixture.	126
4.34	ShearScan2 apparatus, by VJ Tech Ltd, used for direct and residual tests on LB fraction E sand.	128
4.35	Shear stress - shear strain graph for the LB fraction E sand.	129
4.36	Shear stress vs normal stress graph for the LB fraction E sand.	130
4.37	Volume change of LB fraction E specimens during shearing.	130
4.38	Triaxial setup used for CU test in M_{90-10} mixture, provided by VJTech. . . .	132
4.39	B-value determined at each cell pressure increment, during the saturation of the M_{90-10} specimen	134
4.40	Consolidation graphs for the M_{90-10} soil. (a) Pwp dissipation and (b) Volume change.	135
4.41	Stress-strain curves for the M_{90-10} soil	136
4.42	Stress ratio variation of the M_{90-10} soil	137
4.43	Mohr-Coulomb failure envelope for M_{90-10} soil	137
4.44		138
4.45	Pore water pressure variation within the sample during the four shearing stages in respect to the axial strain.	139
4.46	Schematic of the single-holed sand hopper (Mo, 2014).	142
4.47	Use of a fine mesh to protect slope surface from erosion.	145
5.1	Cavitation of PPTs installed in fraction E sand due to gravity increase.	152
5.2	Boundary conditions for the slope models tested under a deep ground water table condition	153
5.3	Displacement vectors in the loose slopes at gravity increase from 1g to 60g (for tests CSA8.1 to CSA8.4).	155

5.4	Displacement vectors in the dense slopes at gravity increase from 1g to 60g (for tests CSA8.1 to CSA8.4).	156
5.5	Deformation of loose slope after low rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.1).	157
5.6	(a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.1)	158
5.7	Deformation of loose slope after medium rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.2).	159
5.8	(a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.1).	160
5.9	Deformation of loose slope after high rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.3).	161
5.10	(a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.3)	162
5.11	Deformation of dense slope after low rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.1).	163
5.12	(a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.1)	164
5.13	Deformation of dense slope after medium rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.2).	165

5.14 (a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.2).	166
5.15 Deformation of dense slope after high rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.3).	167
5.16 (a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.3).	168
5.17 Boundary conditions applied to the slope model at test CSA8.4, with a shallow groundwater table (test CSA8.4).	171
5.18 Water table rise in test CSA8.4 and progressive slope failure (model time). . .	173
5.19 (a) Deformation vectors and (b) deformation contours after 86.3 s of rainfall (before failure No 1) (test CSA8.4).	174
5.20 (a) Deformation vectors and (b) deformation contours after 92 s of rainfall (before failure No 2) (test CSA8.4).	175
5.21 (a) Deformation vectors and (b) deformation contours after 95.3 s of rainfall (before failure No 3) (test CSA8.4).	176
5.22 Boundary conditions applied to the slope model at test CSA9.4	177
5.23 Model layout at test CSA9.4	177
5.24 Water table rise in test CSA9.4 and progressive slope failure.	179
5.25 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 1.	180
5.26 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 2.	181
5.27 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 3.	182

5.28 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 3.	183
5.29 Layout of the slope model used for the determination of the flow net in the centrifuge (all dimensions in mm).	184
5.30 Flow net test at 10g. Blue lines indicate the movement of the dye from points 1, 2 and 3.	186
5.31 Flow lines within the slope at 10g.	186
5.32 Flow paths at 60g with rainfall conditions applied.	187
5.33 Flow lines at 60g with rainfall conditions applied. (a) Horizontal GWT (b) "bulged" GWT.	188
6.1 Testing apparatus for soil column tests.	194
6.2 Distribution of total and effective stresses for a saturated LB fraction E sandy model tested in the soil column apparatus at 60g.	196
6.3 Radii and dimensions of the soil column model on the geotechnical centrifuge.	197
6.4 Graduated measuring cylinder (standpipe) for the determination of the water volume extracted from the soil.	200
6.5 Illustration of the measuring system with height ℓ associated to the water volume in the tank	201
6.6 Calibration of the measuring system for the soil column apparatus. ℓ stands for the distance (measured in pixels) between the water level within the measuring cylinder and the reference point	201
6.7 Impact of trapped air bubbles to the measurement of the water volume within the tank.	203
6.8 Illustration of the water level distortion caused by centrifugal acceleration. (a) 1g conditions, (b) Ng conditions.	204
6.9 $\Delta\ell$ measured at different g-levels.	205

6.10	Increment of the water meniscus radius due to the increased gravity, causing a change to the reading of the water level.	207
6.11	Stages of creep after initial application of load (Kuhn and Mitchell, 1993). . .	208
6.12	Displacement of the upper surface of the soil column models at 60g centrifugal acceleration.	211
6.13	Fitting curve for the dry LB fraction E sand with $e = 0.84$	212
6.14	Fitting curve for the dry LB fraction E sand with $e = 0.8$	212
6.15	Desaturation curves for the Leighton Buzzard sand, fraction C (model scale). . .	215
6.16	Desaturation curves for the Leighton Buzzard sand, fraction E (model scale). . .	215
6.17	Desaturation curves for the Congleton sand (model scale).	216
6.18	Desaturation curves for the M_{90-10} mixture (model scale).	217
6.19	Desaturation curves for the M_{70-30} mixture (model scale).	218
6.20	Desaturation curves for the M_{50-50} mixture (model scale).	218
6.21	Desaturation curves for the M-Soil (model scale).	219
6.22	Variation of w with depth for the soil columns at the end of the centrifuge test. . .	221
6.23	Variation of S_r with depth for the soil columns at the end of the centrifuge test. . .	222
6.24	Variation of S_r at different g -levels for all soils tested.	223
6.25	Pore water pressure variation caused by drainage due to gravity increase, on Leighton Buzzard sand, fraction E (model scale). The red point in the subplot corresponds to the value of suction determined in the soil column test at 60g.	225
6.26	Pore water pressure variation caused by drainage, due to gravity increase, on M_{70-30} (time in model scale)	226
6.27	Pore water pressure variation caused by drainage, due to gravity increase, on M-Soil (model scale).	227
6.28	Pore water pressure variation caused by three rainfall events (i.e. 30 s, 30 s and 60 s in model scale).	229

6.29	Pore water pressure variation caused by a major rainfall event (model scale). .	230
6.30	Water content and pore water pressure evolution due to gravity increase for fraction E sand, in terms of its SWRC.	231
6.31	(a) Typical SWRC. (b) Shear strength behaviour of the soil as it relates to the SWRC (Vanapalli et al., 1996)	232
7.1	Distribution of moisture content with depth at the end of the compaction process for M_{90-10} soil in test CSI1.	240
7.2	Location of PPTs for test CSI1 (Matziaris et al., 2017)	241
7.3	Pore water pressure variation during gravity turn-on for test CSI1 (model time). .	242
7.4	Settlement of the slope, due to gravity increase, at $t=0$ (dimensions in model scale).	243
7.5	Initial failure of slope in test CSI1 after 1.25 s of rainfall (model scale). . . .	243
7.6	Secondary failure of slope in test CSI1 after 22.1 sec of rainfall (model scale). .	244
7.7	Pore water pressure changes at 60g for CSI1 test (model time).	245
7.8	Deformation vectors in the slope of Test CSI1, indicating deformation caused by rainfall. a) At $t = 1.12$ s (67.2 min in prototype) b) At $t = 1.18$ s (70.8 min in prototype) c) At $t = 1.25$ s (75 min in prototype). Failure surface indicated in c) corresponds to the observed <i>initial failure</i> (Fig 7.6).	246
7.9	Deformation vectors in the slope of Test CSI1, indicating deformation caused by rainfall after the 1st failure. a) Deformation at $t = 5s$, b) Deformation at $t = 10s$, c) Deformation at $t = 15s$, d) Deformation at $t = 20s$, e) Deformation at $t = 22s$, f) Deformation at $t = 22.1s$ ($t = 0$ when first failure occurred). The red line indicates the <i>secondary failure</i> observed in the end of the test (Fig 7.6). .	248
7.10	Maximum shear strain of the slope 0.1 s before the initiation of the first failure. .	249
7.11	Volumetric strain 0.1 s (model time) before the initiation of the first failure. .	249

7.12	Effect of gravity increment to the slope CSI3 at $t = 0$ (dimensions in model scale). The red line represents the critical slip surface as this was determined by the numerical analysis using Slope/W (section 7.3.3)	250
7.13	Deformation vectors in the slope of Test CSI3, indicating deformation caused by rainfall - $t = 0$ at rainfall initiation (model time).	252
7.14	Instability in the slope of Test CSI3 (model time).	253
7.15	Instability at Test CSI3.	253
7.16	Shear strain distribution in the slope of Test CSI3. Left: at $t = 170$ s Right: at $t = 1049$ s (time in model scale).	254
7.17	Geometry of the slope model used in the numerical analysis of tests CSI1 & CSI3.	256
7.18	Stability of the slope model prior to the application of rainfall.	257
7.19	Evaluation of the actual slip surface of the slope in test CSI1 before rainfall. .	258
7.20	Evaluation of the actual slip surface of the slope in test CSI1 after rainfall. Boundary conditions simulate the impact of a rainfall event with relative intensity of 1.	259
7.21	Evaluation of the actual slip surface of the slope in test CSI1 after rainfall. Boundary conditions represent the case of full saturation of the soil profile affected by a rainfall event with a relative intensity greater than 1.	260
7.22	Evaluation of the actual slip surface of the slope in test CSI3 before rainfall. .	260
7.23	Evaluation of the actual slip surface of the slope in test CS31 after rainfall. Boundary conditions simulate the impact of a rainfall event with relative intensity of 1	261
7.24	Evaluation of the actual slip surface of the slope in test CSI3 after rainfall. Boundary conditions represent the case of full saturation of the soil profile affected by a rainfall event with a relative intensity greater than 1.	261

7.25 Rainfall threshold obtained from the current study. 266

List of tables

2.1	Rainfall rate classification (Tokay and Short, 1996).	25
2.2	Scaling laws for rainfall parameters (Askarinejad, 2013)	29
3.1	Specifications and rainfall conditions applied with the three nozzle types. . .	51
3.2	Configuration of the Prosilica CG2450 camera for the slope tests.	56
3.3	Specifications of the tensiometers and porous filters used in this study. . . .	59
3.4	Dissolved oxygen in three types of water.	63
4.1	Classification characteristics of employed soils.	91
4.2	Particle density of the sand/silt mixtures.	93
4.3	Coefficient of saturated permeability k for the soils used in this study at specific density	102
4.4	Moisture distribution results of soil specimens. <i>Top</i> (T), <i>middle</i> (M) and <i>bottom</i> (B) refer to the vertical position of the samples, while <i>centre</i> (Ce) and <i>circumference</i> (Ci) to the horizontal position. Bulk w is the mean water content and the one determined by the balance readings.	113
4.5	Fitting parameters of the van Genuchten model for the desorption process (drying curve).	124
4.6	Fitting parameters of the van Genuchten model for the absorption process (wetting curve).	126

4.7	Properties of soils and mixtures used in this study. DS: Direct Shear tests. . .	140
4.8	Centrifuge testing programme of soil slopes. Prototype rainfall intensities were 7.4 mm/hr or 2.05×10^{-6} m/s (low), 27.2 mm/hr or 8×10^{-6} m/s (medium) and 87.7 mm/hr or 2.44×10^{-5} m/s (high).	148
6.1	Specifications of wet soil column tests.	214
6.2	Retained values of bulk gravimetric water content, w , at different g-levels. . .	219
6.3	Retained values of degree of saturation, S_r , at different g-levels.	220
6.4	Calculated fitting parameters and goodness of fit, R^2 for the fitting curves. . .	223
6.5	Normalized ($w_n = w/w_o - S_{r_n} = S_r/S_{r_o}$) values for water content and degree of saturation	224
7.1	Parameters used in the numerical analysis of tests CSI1 & CSI3.	254
7.2	Comparison of the fitting parameters between different rainfall thresholds. . .	265

Chapter 1

Introduction

The current project is integrated with the Innovative Training Network project MUMOLADE (Multi-scale Modelling of Landslides and Debris Flows). Within this context, the current study aims to investigate the initiation conditions of rainfall-induced landslides. The methodology selected for the investigation is the centrifuge testing of small-scaled slope models under well defined initial and boundary conditions. This chapter presents an overview of the MUMOLADE project and the contribution of the current study. Furthermore, the aim and objectives of this research are presented. Finally, a brief description of the chapter content is given at the end of this section.

1.1 Overview

Landslides and debris flows are serious geohazards common to countries with mountainous terrains. A significant number of rainfall-induced landslides is observed in regions under tropical and sub-tropical climates ([Brand, 1982](#); [Fourie, 1996](#)) where residual and colluvial soils are widespread. The likelihood and time of occurrence, in respect to a triggering rainfall event, are not easy to be defined leading to a major difficulty in predicting small and large-scale disasters. Rainfall characteristics, i.e. duration, intensity and distribution, play a significant

role in the pore water pressure changes and therefore influence the stability of natural and man-made slopes. Moreover, slope failures due to rainfall are often rapid phenomena and occur without warning leading to a major lack of reliable field data.

Rainfall-induced landslides are initiated due to the infiltration of rain water which moves downwards within the soil mass in the form of a wetting front. In this way, a saturated zone is formed between the wetting front and the soil surface with its size and characteristics depending on the intensity and total amount of precipitation as well as soil hydraulic properties. The saturation of the soil pores leads to the increase of pore water pressures and a subsequent decrease of effective stresses within the soil mass. Therefore, shear strength is reduced significantly leading to a potential reduction of the stability and the occurrence of a failure.

Landslides and debris flows involve very complex processes and can be best described as multiphase and multi-scale. Frequently landslides and debris flows are of multiphase nature involving air, fluid and solid. The complex interaction among these phases means that simple constitutive equations with constant strength and viscosity are no longer feasible ([Iverson, 2003](#)). Moreover, landslides and debris flows present multi-scale problems, both temporal and spatial. The temporal multi-scale aspect can be best appreciated by comparing rainfall induced landslides with the runout of debris flows. While the infiltration and saturation of a soil slope may take weeks, the runout of debris flows is often a matter of seconds. The spatial multi-scale aspect is manifested by some intriguing phenomena in landslides and debris flows, where the deformation is often dominated by intense shearing within thin shear bands. Further phenomena include the segregation of coarse particles and the local interaction between fluid and solid.

1.2 Research aim and objectives

The aim of this research study is to investigate the initiation conditions of rainfall-induced landslides in soil slopes. For the accomplishment of the study, a number of centrifuge tests on

small-scaled slope models were performed under well defined initial and boundary conditions. Different failure mechanisms were investigated and techniques applied in order to overcome scaling issues created by the increased gravity field in the centrifuge.

The objectives of the current study are:

- The study of the initiation of rainfall-induced landslides due to suction elimination
- The determination of the retained moisture content of soil columns subjected into centrifugal acceleration and study of the effect of grain size on that retained moisture content
- To validate the hypothesis of scaling down the permeability of a soil in order to match model and prototype seepage velocity
- To study the importance of the relative rainfall intensity on slope stability
- Determination of rainfall thresholds for landslide initiation based on the results from the centrifuge tests.

1.3 Outline of the thesis

An outline of the thesis chapters is presented in this section.

Chapter 1 presents an overview of the research work, the context and highlights the aim and objectives of the study.

In Chapter 2, a literature review and the state of the art in centrifuge modelling of rainfall-induced landslides is presented.

Description of the testing apparatus and the instrumentation used during the centrifuge tests is presented in Chapter 3. Moreover, the chapter presents the methods for the determination of the deformation fields during the centrifuge tests.

In Chapter 4, the results from the soil classification tests are presented. These tests were conducted in the soil mechanics laboratory at the University of Nottingham. Furthermore, the Soil-Water Retention Curve of two soils was determined in the laboratory. Finally, model preparation techniques are described and a summary of the centrifuge tests conducted is presented in this chapter.

Chapter 5 includes the initial findings from the slope stability tests performed on sand slopes under varied initial and boundary conditions.

The experimental setup and the results from the soil column tests are presented in Chapter 6. The results, together with the initial findings in slope models, provided knowledge about the moisture conditions and suctions enabled in a soil mass when tested in the centrifuge.

In Chapter 7, the results from the initial findings and soil column tests (obtained in Chapters 5 and 6) are combined and a new approach is adopted for the centrifuge modelling of rainfall-induced landslides. Two centrifuge slope tests are presented in which the seepage velocity was been scaled under varied rainfall intensities. Moreover, rainfall thresholds for the initiation of landslides are obtained.

Chapter 8 includes a summary of the most important points of this research and proposed further work.

Chapter 2

Literature review

2.1 Introduction

The current research involves three different disciplines, i.e. unsaturated soil mechanics, slope stability and centrifuge modelling. The basic concepts for each one of them is briefly described in this chapter. Moreover, recent research studies concerning the centrifuge modelling of rainfall-induced landslides are also presented.

2.2 Unsaturated soils

2.2.1 Effective stress and shear strength

When a soil is saturated, pore water is positive and the net effect of the water is to reduce the effective stress. Unlike saturated soils, which comprise a two-phase system of solids and water (or solids and air in totally dry soils), unsaturated soils are three-phase systems with solids, water and air co-existing in the mass. In this case, pore water sustains negative pore pressures,

thus creating tensile forces acting to increase the effective stress and pull the soil grains together. This is expressed by the following equation which was introduced by Bishop (1959):

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (2.1)$$

where σ' is the effective stress in an unsaturated state, $\sigma - u_a$ the net normal stress, u_a the pore air pressure, χ the effective stress parameter and $(u_a - u_w)$ the matric suction.

Equation 2.1 is the most widely used expression for effective stress in unsaturated soils. Parameter χ is a material property which depends on the degree of saturation or the soil suction and determines the degree of suction contribution to the increment of effective stress. Its value ranges between 0 and 1 with the two extremes indicating totally dry and fully saturated conditions, respectively. In a totally dry soil, $\chi = 0$, $u_a = o$ and $\sigma = \sigma'$. Equally, in a fully saturated soil, $\chi = 1$ and $u_a = 0$ resulting in effective stress given by Terzaghi's equation of effective stresses. Even though there is a clear dependency of χ to the degree of saturation, S_r , they do not coincide. Their relationship is illustrated in Figure 2.1 (Lu and Likos, 2004).

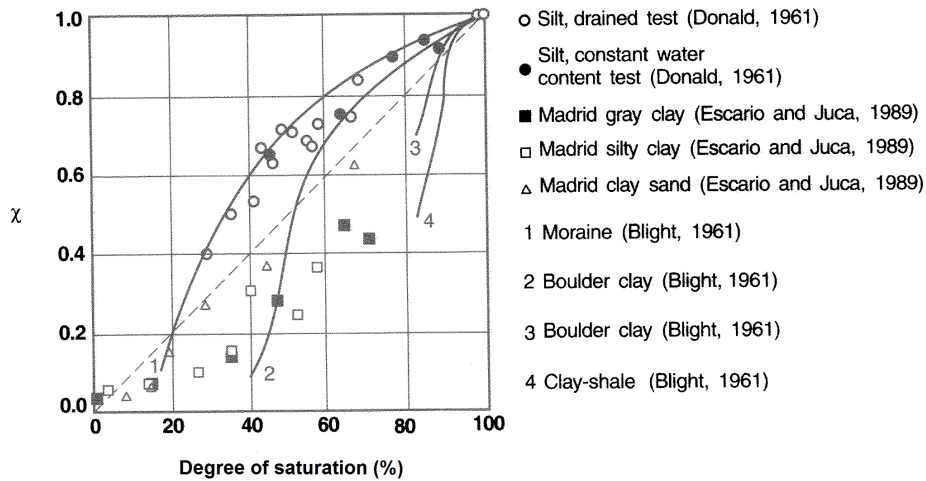


Fig. 2.1 Experimental results showing the relationship between the effective stress parameter χ and the degree of saturation S_r for various soils (Lu and Likos, 2004).

Fredlund et al. (1978) presented the extended Mohr-Coulomb failure criterion for unsaturated soils. In his criterion, a new variable was introduced, angle ϕ^b , which defines the

contribution of soil suction to the soil's shear strength and ranges between 0 and ϕ . The extended Mohr-Coulomb failure criterion was formulated by [Fredlund et al. \(1978\)](#) and describes the shear strength of an unsaturated soil in terms of two stress-state variables: $\sigma - u_a$ and $u_a - u_w$ and is given by the following equation:

$$\tau_f = c' + (\sigma - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi^b \quad (2.2)$$

where c' is the cohesion at zero matric suction and zero net normal stress, $(\sigma - u_a)_f$ is the net normal stress on the failure plane at failure, ϕ' the angle of internal friction associated with the net normal stress variable, and $(u_a - u_w)$ is the matric suction at failure. In the case of unsaturated soils, the failure envelope is represented by a surface on a Shear stress - Net normal stress - Matric suction plot, as shown in Figure 2.2.

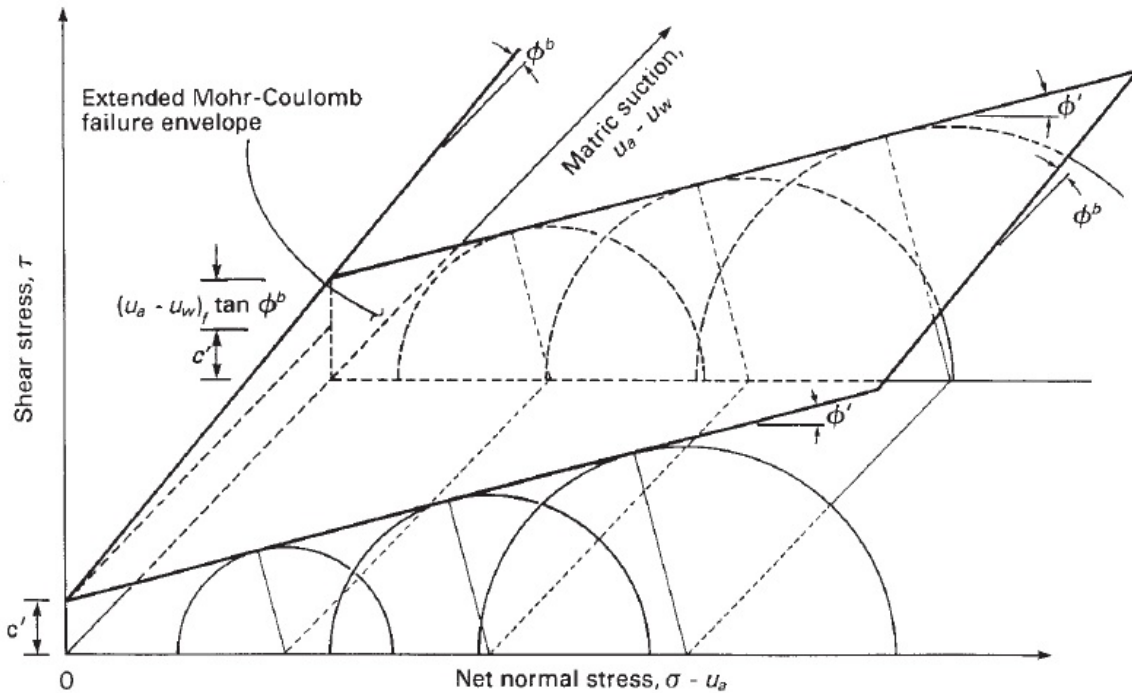


Fig. 2.2 Extended Mohr-Coulomb failure surface for an unsaturated soil ([Fredlund and Rahardjo, 1993](#))

In practice, equation 2.2 is difficult to be implemented due to the presence of the angle ϕ^b . This angle is difficult to be derived in a laboratory and most of the times it needs to be assumed.

The equation for the shear strength of an unsaturated soil can be re-written in terms of the micro-scale of an unsaturated soil (Zhou et al., 2016). While for dry and fully saturated conditions the shear strength equation is straightforward, depending on the total and effective stress respectively, in unsaturated soils the presence of partially filled voids requires an effective stress approach that incorporates the influence of soil suction. Looking at a system of two spherical soil grains in an unsaturated soil (Figure 2.3), water exists in two forms, i.e. absorbed and capillary form. Absorbed water is held around the soil grains due to the presence of the van der Waal's forces between solid and liquid molecules and is treated as an internal stress which primarily contributes to the cohesion of the soil mass. The absorbed water is not affecting the effective stress acting between the two particles. Therefore, only the capillary water pressure contributes to the shear strength of the unsaturated system. The shear strength equation can then be written as:

$$\tau = c + (\sigma - u_a)\tan\phi + (u_a - u_w)\frac{A_w^{cap}}{A}\tan\phi \quad (2.3)$$

where A_w^{cap} is the area of the capillary water perpendicular to the normal stress and A_w^{cap}/A the normalised area of the capillary water. The presence of the absorbed water, compared to the capillary water, is extremely small (Öberg and Sällfors, 1997; Vanapalli et al., 1996; Xu, 2004); therefore the area of the capillary water can be considered as the area of the pore water. The normalised area of water is assumed to be in direct proportion to the normalised volumetric water content of the soil, by applying Greens theorem (Vanapalli et al., 1996). Since

the normalised volumetric water content can be considered equal to the degree of saturation, equation 2.3 can be written as:

$$\tau = c + (\sigma - u_a)\tan\phi + (u_a - u_w)S_r\tan\phi \quad (2.4)$$

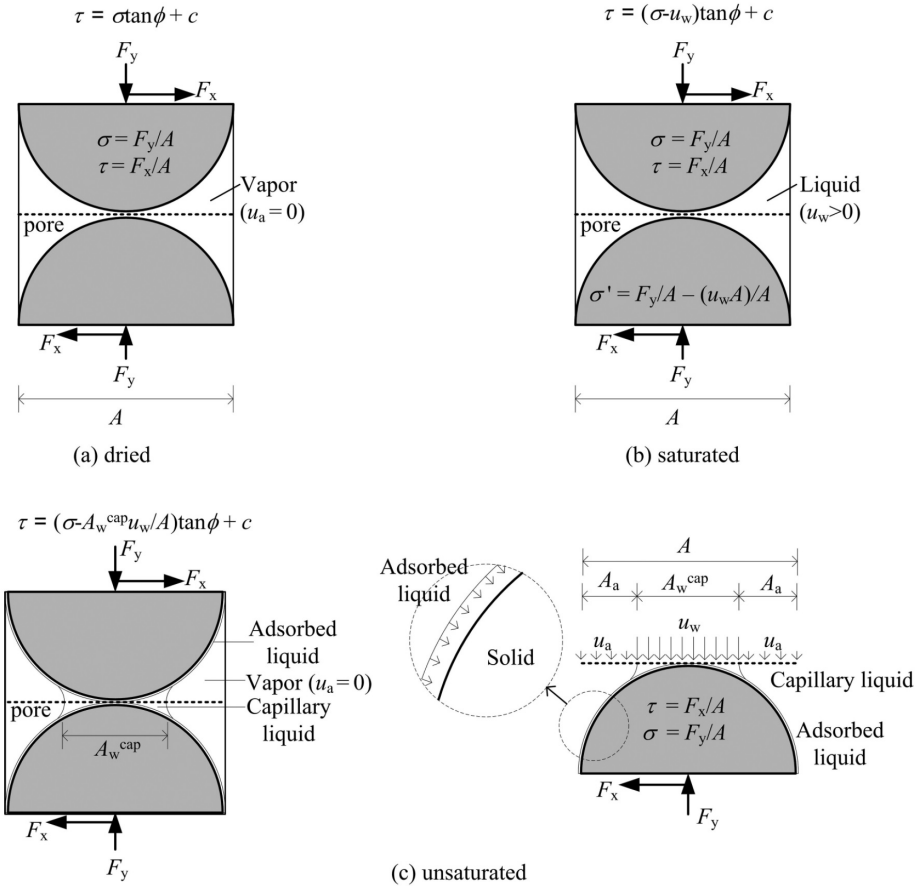


Fig. 2.3 Microstructural analysis of interparticle interaction in a) dry, b) saturated, c) unsaturated soil. F_x and F_y are external forces acting on the x- and y-plane (Zhou et al., 2016)

Equation 2.4 is more practical for the determination of the shear strength of an unsaturated soil compared to Equation 2.2. Moreover, Equation 2.4 reveals that the shear strength of an unsaturated soil is related directly to its Soil Water Retention Curve (SWRC) characteristics, i.e. to the relationship between suction and degree of saturation (section 2.2.4). Equation 2.4 has been widely adopted in the literature (Zhou et al., 2016), even though it is shown that it

significantly overestimates the shear strength of unsaturated clayey soils at high suctions.

The comparison between Equations 2.4 and 2.2 produces the following relationship:

$$\tan\phi^b = S_r \tan\phi \quad (2.5)$$

When $S_r = 0$ (dry soil), $\tan\phi^b = 0$ and for fully saturated conditions ($S_r = 1$), $\tan\phi^b = \tan\phi$. These two extreme values are generally true since angle $\tan\phi^b$ varies between 0 and $\tan\phi$ at dry and fully saturated conditions, respectively (Fredlund and Rahardjo, 1993). However, this is not necessary true for interim saturation states (i.e. partially saturated conditions) as experimental evidence has shown that the relationship between $\tan\phi^b$ and $\tan\phi$ is not linear (Lu and Likos, 2004).

2.2.2 Pore water pressure measurements

Pore water pressure changes during rainfall conditions are very important for the evaluation of the stability of slopes. Their accurate and reliable measurement in centrifuge models is an ongoing question among researchers. Below the groundwater table, pore water pressures within the soil are positive. Positive pore water pressures cause the reduction of the shear strength of soil and, consequently, the reduction of the stability of a slope. On the other hand, negative pore water pressures exist above the groundwater table with soils being in either a saturated or partially saturated state. Negative pore water pressure contributes to the shear strength of the soil due to the tension applied on the air - water interface. Therefore, soil slopes have higher stability when they exist in a partially saturated state. During rainfall, water infiltration in partially saturated soils causes pore water pressure to increase (from their initial negative value) due to an increased degree of saturation. This has an implication to the shear strength of the soil, which decreases and therefore affects the stability of the slope.

The difference between the pore air pressure u_a and pore water pressure u_w is defined as the *matric suction*, i.e:

$$s = u_a - u_w \quad (2.6)$$

Usually pore air pressure is equal to atmospheric pressure in partially saturated soils lying near the surface. Therefore matric suction coincides with the negative value of the pore water pressure. Since pore water pressure is negative in unsaturated soils, soil suction has a positive value.

Osmotic suction is related to the salt differences between the pore water and pure water (Fredlund and Rahardjo, 1993). The osmotic suction component can be described by the following equation:

$$\pi = RT\Delta_c \quad (2.7)$$

where R is the universal gas constant (8.314 J/mol·K), T the absolute temperature in Kelvins ($T = 273.16 + t^\circ$, where t° is the temperature in $^\circ\text{C}$) and Δ_c the difference in the concentration between two solutions. Osmotic suction changes have an effect on the mechanical behaviour of a soil. If the salt content in a soil changes, there will be a change in the overall volume and shear strength of the soil.

Total suction is the sum of the two components, the matric suction (or capillary component) and the osmotic suction (or solute component). Total suction quantifies the thermodynamic potential of soil pore water relative to a reference potential of free water (Lu and Likos, 2004). Soil suction is enabled when physical and physiochemical mechanisms cause the decrease of the thermodynamic potential of the soil pore water relative to the reference case. These mechanisms are:

1. Temperature

2. Inertial effects
3. Gravity
4. Capillary effects
5. Short-range adsorption effects
6. Osmotic effects

The first three mechanisms cause minor changes to the thermodynamic potential, therefore they can be neglected. Capillary and short-range adsorption effects may occur under either saturated or unsaturated conditions; their combination gives rise to the matric suction. The osmotic effect is unique to unsaturated soils and is responsible for the osmotic suction component. According to [Fredlund and Rahardjo \(1993\)](#), total suction changes are closely related to changes in the matric suction, while the osmotic component between the fluid phases remains more or less constant, in particular for higher water contents which are of interest for slope stability issues. Therefore, measurements of negative pore water pressures coincide with the values of matric suction. The term *suction* will be used to refer to the *matric suction* component throughout this thesis.

Many methods and techniques have been developed for the measurement of soil suction, either the total or osmotic components. The choice of the most suitable method depends on the nature of the measurement and the desired measurement range. The most common techniques for the measurement of matric suction are the following:

- Thermal conductivity sensors;
- Axis translation technique;
- Tensiometers.

Thermal conductivity sensors relate changes of thermal conductivity of a porous medium to the matric suction. The thermal conductivity of the sensor is directly dependent on the volumetric water content of the soil and is evaluated by measuring the rate of internal heat dissipation following a heat pulse. However, a soil's water content is directly related to the matric suction in a relationship known as the Soil Water Retention Curve (Section 2.2.4). This implies that changes in the water content of the porous medium of the thermal conductivity sensor can be indirectly related to the matric suction of the soil. Therefore, they are not appropriate to be used in centrifuge tests because they do not provide accurate measurements of suction.

Axis translation is a very common technique for the control or measurement of suction in pressure plate, triaxial, direct shear and oedometer tests (Marinho et al., 2008; Vanapalli et al., 2008). Its principle is based on the elevation of the air pressure which causes rise to the water pressure in order to maintain a constant value of suction. For the application of the technique, soil needs to be placed on a saturated porous disc with a high air entry (HAE) value. There are two procedures described in the literature when using the axis translation technique. In the first, both air and water pressures are controlled with their difference giving the value of suction within the soil mass. The soil is left to drain through the HAE disc and the water content at equilibrium corresponds to the suction applied. In the second procedure, air pressure in a soil sample with known water content is elevated and the water drain is blocked. The value of suction is determined as the difference between the applied air pressure and the measured water pressure. These two procedures are mainly used for the determination of the SWRC of a soil for suctions ranging between 0 and 1500 kPa. Although the wide use of the axis translation technique, it is not suitable for centrifuge model tests for two main reasons. 1) The measurement or control of suction is done in the whole soil mass and cannot be used for point measurements within the model; 2) the impact of transient seepage on pore water pressure cannot be determined.

2.2.3 Moisture, pore pressure and stress profiles

An example of the stress profiles that occur in a sandy layer under saturated and unsaturated conditions is illustrated in Figure 2.4 (Lu and Likos, 2004). In the first case, the groundwater table is located at the soil surface while in the second case it is at a depth z . While in the saturated state the volumetric water content θ remains constant ($\theta = \theta_s$), this is not the case in unsaturated conditions in which θ varies from a residual value θ_r at the surface to the saturated value θ_s near the depth of the groundwater table.

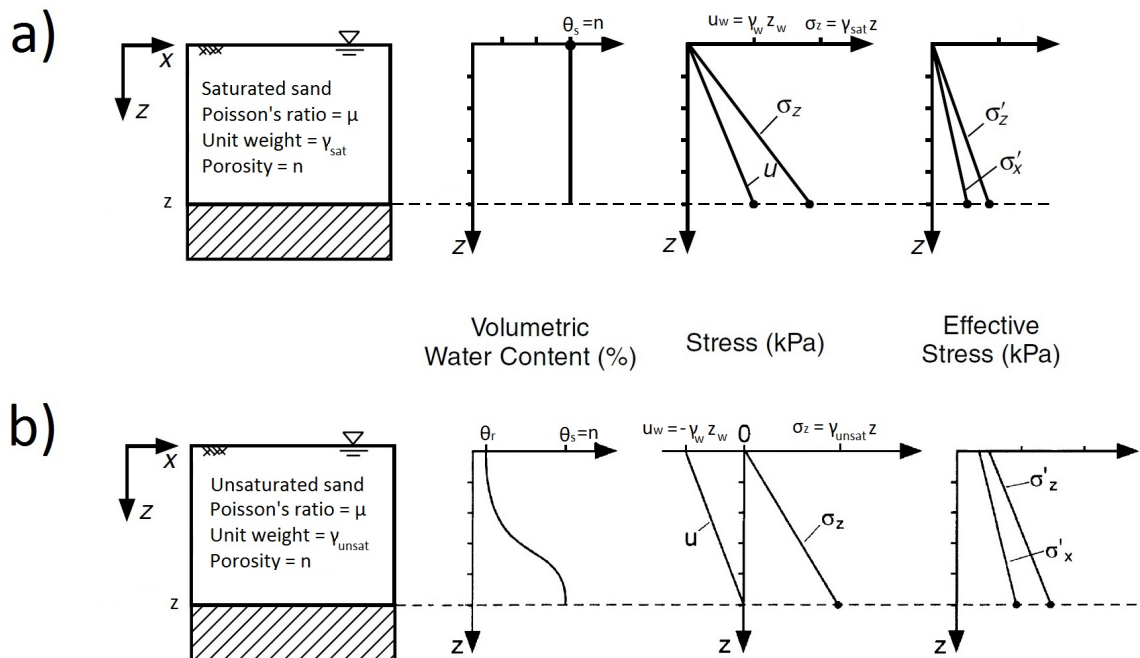


Fig. 2.4 Example of a 10-m thick sandy layer and profiles of volumetric water content, pore water pressure, total and effective stresses under a) Saturated and b) Unsaturated conditions (Lu and Likos, 2004).

In the case of a lower groundwater table (Figure 2.4b), pore water pressures above the Ground Water Table (GWT) are negative and under hydrostatic equilibrium they are distributed linearly with depth. They range from 0 kPa at the GWT level to a minimum value at the surface.

2.2.4 Soil Water Retention Curve (SWRC)

The Soil-Water Retention Curve (SWRC) or Soil-Water Characteristic Curve (SWCC) defines the relationship between the moisture content and matric suction of a soil. It provides a conceptual understanding between the mass (and/or volume) of water in a soil and the energy state of the water phase (Fredlund et al., 2012). The SWRC can be expressed as a relationship between matric suction and gravimetric water content $((u_a - u_w) - w)$, matric suction and volumetric water content $((u_a - u_w) - \theta)$ or matric suction and degree of saturation $((u_a - u_w) - S_r)$.

A conceptual SWRC with the respective desaturation states is illustrated in Figure 2.5 (Fredlund, 2000). A typical SWRC includes three zones, i.e. the saturation zone, the transition zone and the residual zone. Within the saturation zone, suction increases significantly when the soil loses just a small amount of moisture. This part of the curve indicates a state where the soil remains almost saturated while matric suction increases; it is representative of the capillary zone. The point at which matric suction is high enough to allow air to intrude into the soil mass is called the Air Entry Value (AEV). After the AEV, the SWRC becomes almost a straight line indicating a smooth increment of suction and a respective reduction of soil saturation. The slope at this part of the curve, into the transition zone, depends on the grain size of the soil and increases for coarser soils. The residual zone starts at the residual water content or degree of saturation and is the part of the curve which indicates a significant increment of suction with a small reduction of saturation.

2.2.5 Flow in unsaturated soils

Several concepts have been used to explain the flow of water through unsaturated soils. These concepts describe the driving potential which causes the movement of water within the unsaturated soil mass. The most common are (Fredlund and Rahardjo, 1993):

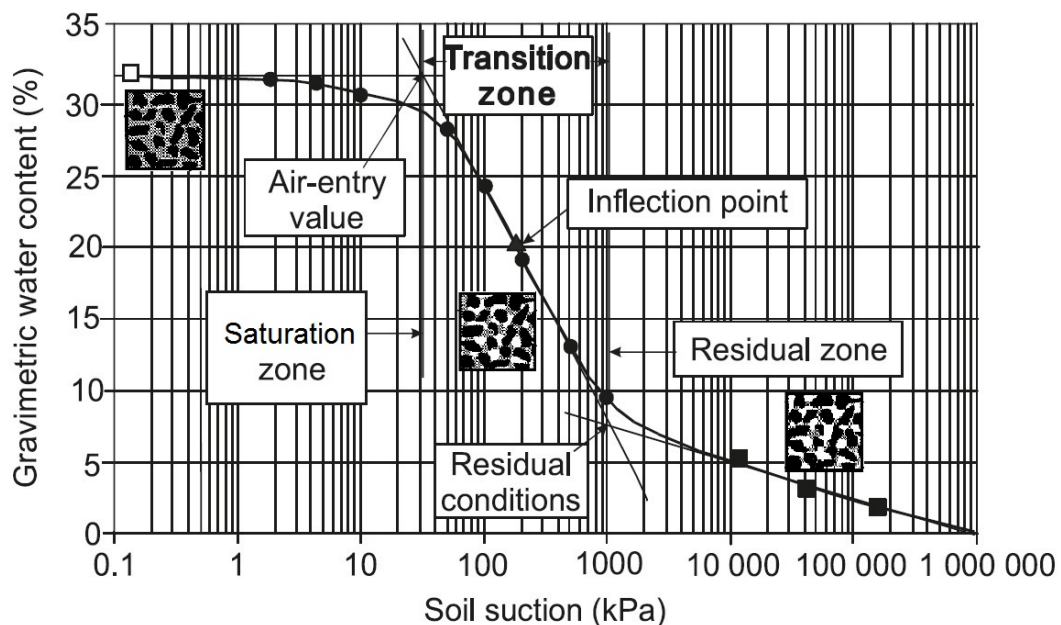


Fig. 2.5 Conceptual Soil-Water Retention Curve (SWRC) for unsaturated soils and respective desaturation states (modified after [Fredlund, 2000](#)).

- **Water content gradient** - This concept assumes that water moves within an unsaturated soil from a point of higher water content to a point of lower water content. However, this cannot be a fundamental behaviour, since there is also movement from lower to higher water content areas when there are variations in the soil types or under hysteretic effects and stress history variations.
- **Matric suction gradient** - According to this concept, water in unsaturated soils will flow from a point of lower suction to a point of higher suction. However, this is not always the case, as the water flows due to the pressure gradient of the individual phases of the unsaturated soil, i.e. air and water. Therefore, flow of water is possible even if the suction gradient is equal to zero, concerning the fact that air and water pressure gradients are changing.
- **Hydraulic head gradient** - The most reliable concept considers the hydraulic head gradient (or pressure head gradient) as the driving potential for water flow within the

unsaturated soil mass. The pressure and the elevation gradients are combined to give a hydraulic head gradient as the fundamental driving potential for flow in saturated and unsaturated soils ([Fredlund and Rahardjo, 1993](#)).

2.3 Slope stability

2.3.1 Infiltration in unsaturated soil slopes

Infiltration plays a significant role in slope stability analysis during rainfall. Rainfall does not necessarily cause total elimination of soil suction even after prolonged periods of rainfall. The effect of infiltration depends on the hydraulic properties of the soil (i.e. hydraulic conductivity), rainfall characteristics (rainfall intensity and duration), suction profile within the soil mass and the boundary conditions.

[Johnson and Sitar \(1990\)](#) used an array of tensiometers in a slope to record the responses of pore pressures to rainfall events that caused flowslides at adjacent sites. They found that at shallow depths suctions changed very little during storm events which was attributed to the fast water drain at the same rate as the infiltration. Below a specific depth, tensiometers tracked rapid changes in the pore pressures to about 5 kPa. The response was very transient and the difference at the arrival times between the shallow and deeper layers indicated a downslope migration of the wetting front of about 0.05 m/s. In contrast, [Sweeney \(1982\)](#) reported significant changes in suction at shallow depths in a 30 m high slope made of completely weathered rhyolite in Hong Kong (Fig. 2.6). More specifically, measurements of pore water pressures at depths 0 – 5m showed significant reduction of suction during the rainy season whereas at 5 – 24m depth the impact of rainfall to pore water pressures was very small. In fact, pore water pressures remained negative at deeper layers and the shear strength remained enhanced by suction.

In addition, [Toll \(2001\)](#) came to the same conclusion measuring pore water pressure distribution in a slope during rainfall (Fig. 2.7). Rainfall event caused a reduction in matric

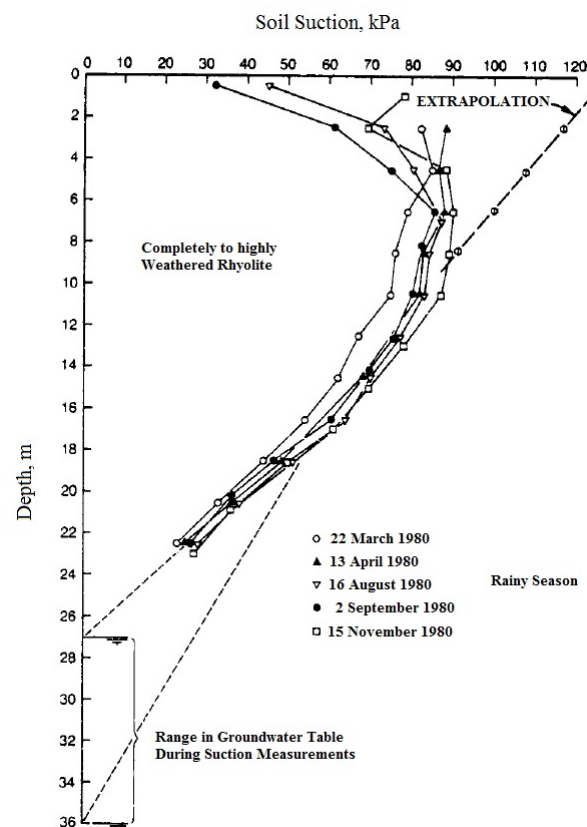


Fig. 2.6 Suction profiles for a weathered rhyolite slope in Hong Kong during the rainy season (Sweeney, 1982).

suction up to a depth between 0.5m and 1.1m, while the impact on the pore pressures at the depth of 1.1m is almost negligible. This is attributed to the fact that the top soil layer has higher permeability due to cracks and fissures developed by prolonged evaporation. Under this layer, the uniform unsaturated soil has a significantly decreased permeability, preventing the downward movement of the water. In case infiltration continues, it is more likely for the water to drain along the interface rather than moving downwards and generating seepage forces.

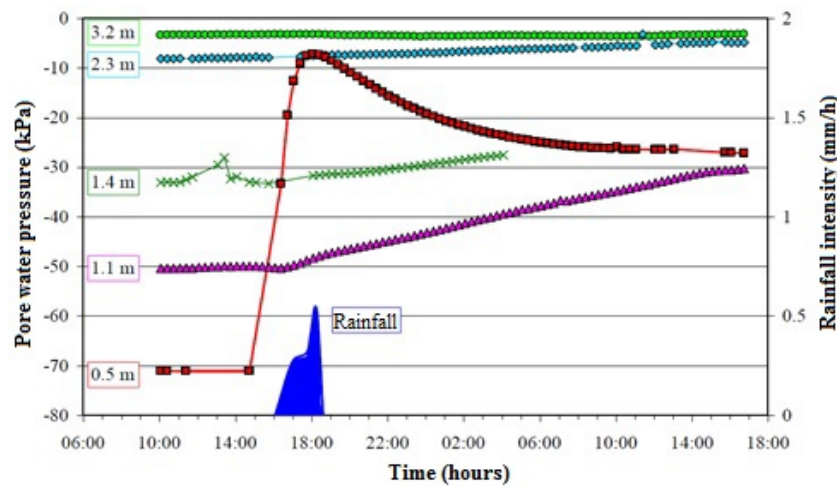


Fig. 2.7 Pore water pressure responses to rainfall infiltration at different depths (Toll, 2001).

Typical pore water pressure profiles in unsaturated soils under varied flux conditions on the soil surface are presented in Figure 2.8 (Zhang et al., 2011) revealing the importance of the rainfall conditions to suction elimination. Under hydrostatic conditions, $q = 0$ and pore water pressures distribute linearly and are negative above the groundwater table. When infiltration starts, matric suction decreases in a rate proportional to the ratio between rainfall intensity and saturated permeability (q/k_{sat}). As the ratio increases, i.e. rainfall becomes more intense and suction decreases more. In a steady-state condition, distribution of pore water pressures with depth would be linear up to the soil surface (Figure 2.8a). Depending on the initial state before of the soil, before rainfall, suction profiles will vary between the left and the right side of Figure 2.8b. In the first case, the initial conditions are hydrostatic and infiltration causes an increase in pore water pressures which depends on the intensity. The time needed to reach the

steady-state conditions depends on the rainfall duration, the coefficient of permeability and the water storage of the soil. However, if the initial conditions are not hydrostatic (i.e. initial flux is not zero, as in the case of antecedent rainfall) the change pattern is different with time. Eventually, the final distribution reaches the steady-state condition.

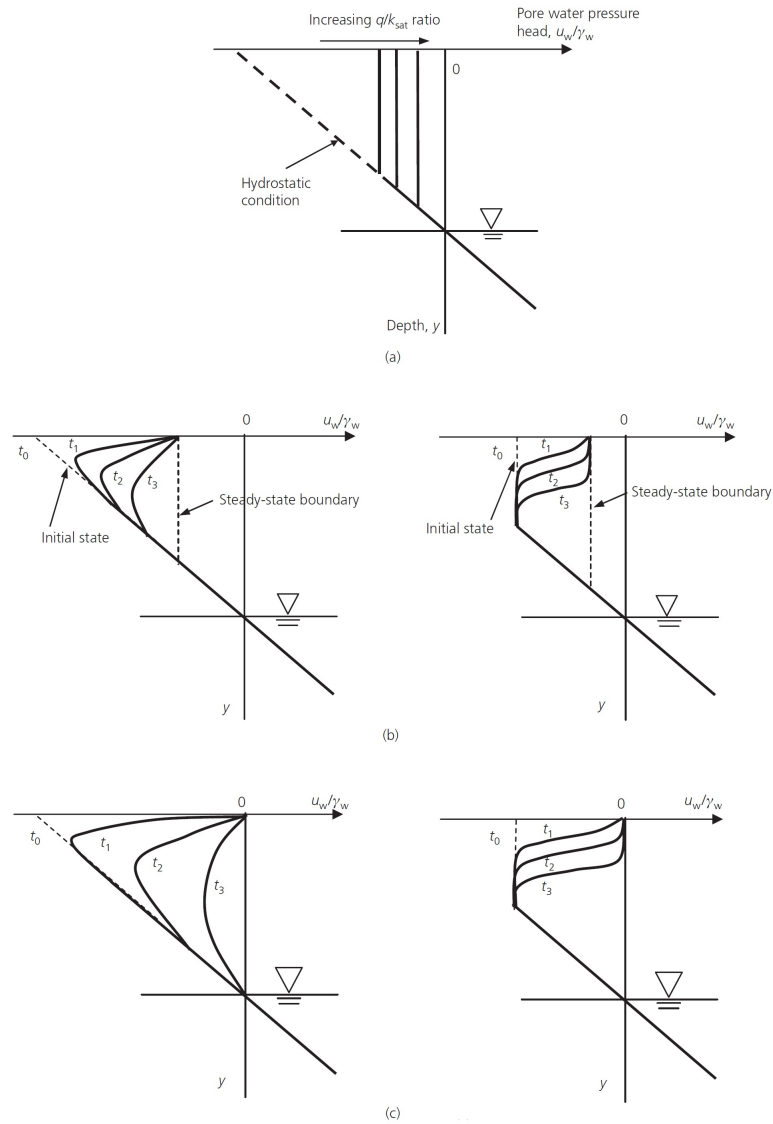


Fig. 2.8 Typical pore water pressure profiles in unsaturated soils with various flux conditions: (a) steady-state conditions, (b) transient conditions ($q/k_{sat} < 1$) and (c) transient conditions ($q/k_{sat} \geq 1$) (Zhang et al., 2011).

The importance of rainfall intensity can also be seen when comparing Figures 2.8b and 2.8c. When the ratio q/k_{sat} is lower than 1, then suction is not eliminated; rather a proportion remains in the soil contributing to the shear strength. When this ratio becomes equal to 1, theoretically suctions become zero and their contribution to the shear strength is lost. Again, the hydraulic conductivity or permeability of the soil is important in terms of the time required for the steady-state condition to be applied.

Slope stability has a strong dependency on the seepage velocity of the rain water, i.e. on the soil permeability. Matziaris et al. (2005) presented numerical modelling results showing the variation of FoS with time, using CHASM (Combined Hydrology And Stability Model, Anderson and Howes (1985)) (Fig. 2.9). Two different rainfall scenarios were tested, i.e. 24 and 8 hours of rainfall duration. Three different hydraulic conductivities were selected for the simulation, 10^{-7} , 10^{-6} and 10^{-5} m/s. The rainfall intensity was chosen to be 2×10^{-6} m/s for the 24h event and 4×10^{-6} m/s for 8h event. In the case of the lower permeability, at both scenarios, the slope behaved in the same manner, showing a reduced FoS that crossed the failure line 17 hours after rainfall initiation.

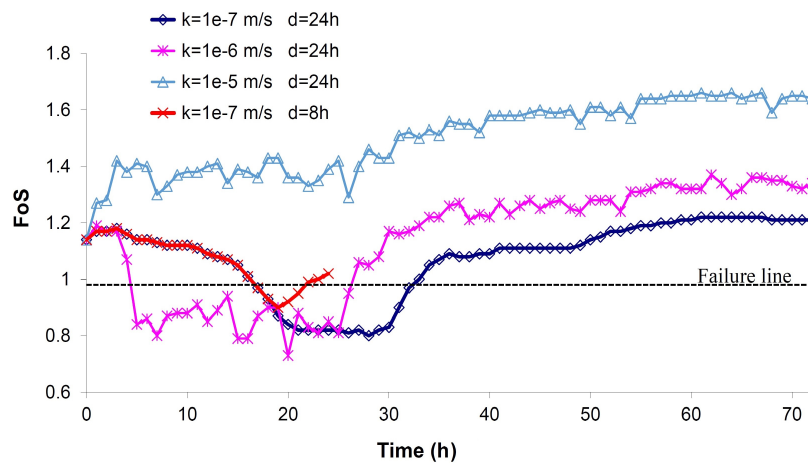


Fig. 2.9 Variation of FoS with time for 4 different scenarios (Matziaris et al., 2005).

2.3.2 Triggering mechanisms of landslides due to rainfall

Two main triggering mechanisms for rainfall-induced landslides have been recognised and are related to changes in pore pressures and seepage forces (Collins and Znidarcic, 2004; Zhang et al., 2011). In the first mechanism, significant build-up of positive pore pressures takes place during infiltration within the soil mass, below the slope surface and on the potential failure surface. Movements of the soil along this surface lead to liquefaction and potentially to rapid movements and failure of the slope. Brand (1982) emphasized that the field stress path, which corresponds to the change in stress resulting from the water infiltration, is much different than the one developed during a standard triaxial compression test (Fig. 2.10). In fact, he pointed out that the stress path can be described by a constant shear stress path as saturation of the soil progresses. Once the soil reaches saturation, further increase in pore water pressure does not change the magnitude of shear stress; however this can be affected by a change in either the seepage direction or the hydraulic gradient (Anderson and Sitar, 1995). This mechanism is also referred to as static liquefaction (Askarinejad et al., 2014; Take et al., 2004).

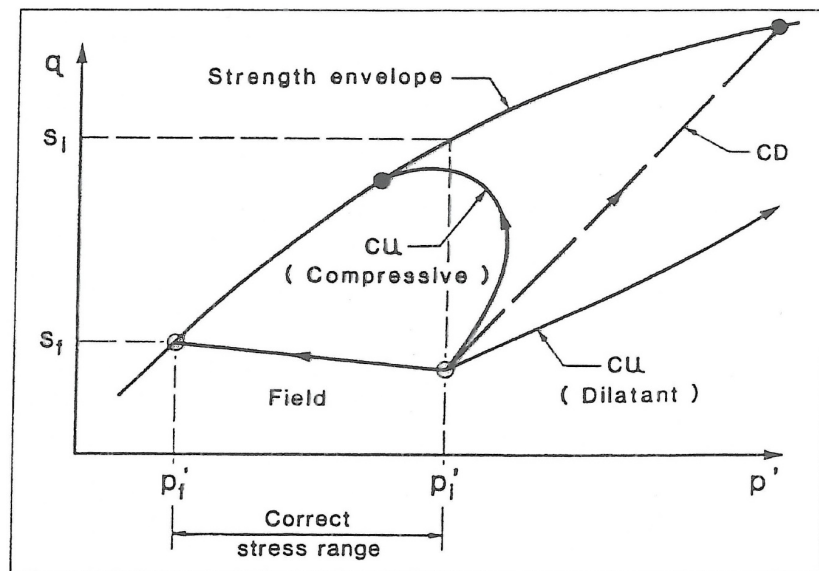


Fig. 2.10 Comparison between stress paths for rain-induced slope failure and for triaxial tests (Brand, 1982).

In the second mechanism, slope failure occurs due mainly to rainfall infiltration which causes a decrease in matric suction. Further rainfall creates conditions of near saturation in the soil, causing suction to be eliminated and effective stresses along the potential failure surface to decrease. Consequently, this results in a loss of shear strength which might trigger instability (Fourie, 1996; Fredlund and Rahardjo, 1993).

A crucial consideration, when dealing with slope stability problems, is to define the state which indicates failure. Cascini et al. (2010) classified rainfall-induced shallow landslides based on the stages of failure. *Failure stage* was defined as the formation of a continuous shear surface through the entire soil mass. *Post-failure stage* was defined as the rapid movement of the failed soil mass. The acceleration of the failed mass determines the type of failure, i.e. if it is classified as slide, slide to flow or flowslide. Leroueil (2001) recognized three different stages in a first-time slope failure, namely the pre-failure stage, the onset of failure and the post-failure stage (Figure 2.11). The pre-failure stage is characterized by a progressive deformation process which might lead to small-scale individual failures. At the onset of failure, a continuous shear surface is formed and global failure is achieved. In the post-failure stage, large deformations take place and the deformation field becomes discontinuous. Take (2003) noticed a gradual softening of clayey slopes, under the repeated action of winter (rainfall) and summer (evaporation) seasons, which occurred by sequences of shrinkage and swelling behaviour and, finally, global instability. During the pre-failure stage (softening), small-scale displacements were observed at different areas within the slope. A large scale movement of a soil block was considered as the initiation of the landslide. On landslide triggering tests, performed by O et al. (2004) and Moriwaki et al. (2004) in sandy soil slopes, a stress redistribution phenomenon was observed during the pre-failure stage and on the initiation of failure.

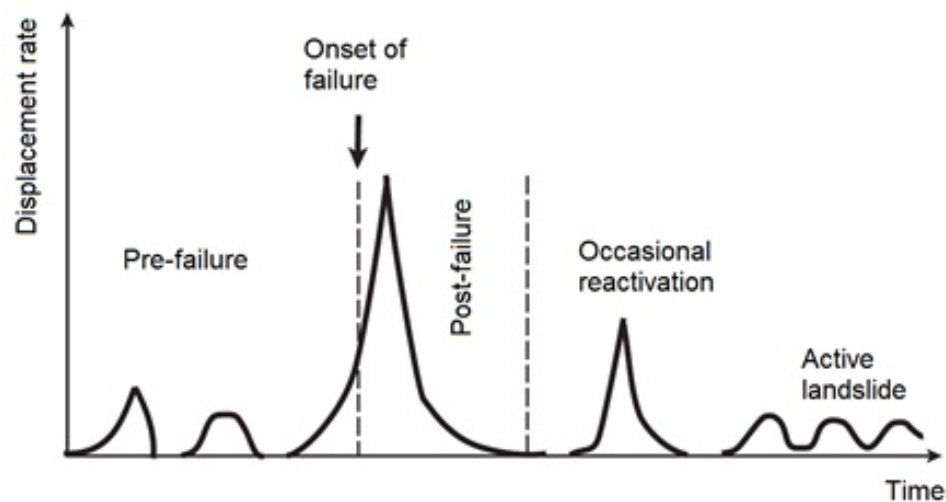


Fig. 2.11 Stages in first-time slope failures (Leroueil, 2001).

2.3.3 Rainfall characteristics

In the absence of the net changes in average atmospheric moisture content, rain balances the upward evaporative flux from the land and oceans (Trenberth, 1998). Although the upward flux due to evaporation is relatively steady, the rate of rainfall exhibits unsteady behaviour, with repeated cycles of rainless and wetted periods, lasting from minutes up to months. The duration and intensity of these phenomena may depend on the regional climatic characteristics, individual atmospheric processes (e.g. tropical storms, extended dry periods) and the landscape (elevation) at the area of concern. Analysis of rain gauge recordings, with 1-minute resolution, has shown that the maximum variation of rainfall intensity can reach the 68 mm/h for mid-latitude continental interiors and up to 238 mm/h for maritime subtropical conditions (Jones and Sims, 1978).

Several classifications of rainfall rates have been proposed with the most representative being that of Tokay and Short (1996), as presented in Table 2.1. This categorizes rainfall events exceeding 5 mm/h as *heavy*, while *extreme* events to be considered those with rainfall rates greater than 20 mm/h.

Table 2.1 Rainfall rate classification (Tokay and Short, 1996).

Descriptive rain rate category	Mean rainfall rate R (mm/h)	Mean rainfall rate R (m/s)
Very light	<1	$<2.78 \times 10^{-7}$
Light	$1 \leq R < 2$	$2.78 \times 10^{-7} \leq R < 5.56 \times 10^{-7}$
Moderate	$2 \leq R < 5$	$5.56 \times 10^{-7} \leq R < 1.39 \times 10^{-6}$
Heavy	$5 \leq R < 10$	$5.56 \times 10^{-7} \leq R < 2.78 \times 10^{-6}$
Very heavy	$10 \leq R < 20$	$2.78 \times 10^{-6} \leq R < 5.56 \times 10^{-6}$
Extreme	$R \geq 20$	$R \geq 5.56 \times 10^{-6}$

The size of the rain droplets in nature range between 0.3 and 1 mm in diameter for light rain (Tamate et al., 2012) and can reach up to 5 mm in diameter for heavy rainfall events. However, they rarely grow larger than 5 mm owing to the stability of the droplet under drag forces (Take, 2003). The size of the droplets depends also on the rain intensity.

2.4 Centrifuge modelling

2.4.1 General aspects of centrifuge modelling

The mechanical principle that underpins centrifuge modelling is the following: when an object is rotating around a centre, at a radius r and with a steady speed u , a radial force is acting on it. In order to stay in the circular orbit, the object is subjected to a constant radial centripetal acceleration equal to u^2/r or $\omega^2 r$, where ω is the angular velocity. For simplicity, the centripetal acceleration can be normalized with the Earth's gravity g giving $N = \omega^2 r/g$ as the number which express how many times higher is the acceleration field than Earth's gravity.

An important issue on the centrifuge modelling is the scaling laws. Scaling laws are relationships which relate the characteristics of the model to its prototype and can be derived by using dimensional analysis or from a consideration of the governing differential equations (Taylor, 1995). Also, the limitations which occur from the non-uniform acceleration field

within a centrifuge model must be taken into account. These are called scaling errors and include the error in the stress distribution within the model and Coriolis effect.

2.4.2 Scaling laws

Scaling laws provide relationships for connecting the parameters from centrifuge testing to the prototype scale. A catalogue of scaling laws for geotechnical centrifuge modelling applications was presented by [Garnier et al. \(2007\)](#).

The basic principle of centrifuge modelling in geotechnical applications is stress similitude, as illustrated in Figure 2.12. In the prototype scale self-weight stresses at a depth H are given by:

$$\sigma_p = \gamma H = \rho g H \quad (2.8)$$

where σ_p is the self-weight stress in prototype, γ the unit weight of the soil, ρ the density of the soil and g the acceleration of gravity.

In a model which has been scaled down by N times compared to the prototype, self-weight stresses at a corresponding point will be:

$$\sigma_m = \gamma \frac{H}{N} = \rho g \frac{H}{N} = \frac{1}{N} \sigma_p \quad (2.9)$$

Under centrifugal acceleration the self-weight stresses at the same point within the model become:

$$\sigma_m = \gamma \frac{H}{N} = \rho g N \frac{H}{N} = \rho g H = \sigma_p \quad (2.10)$$

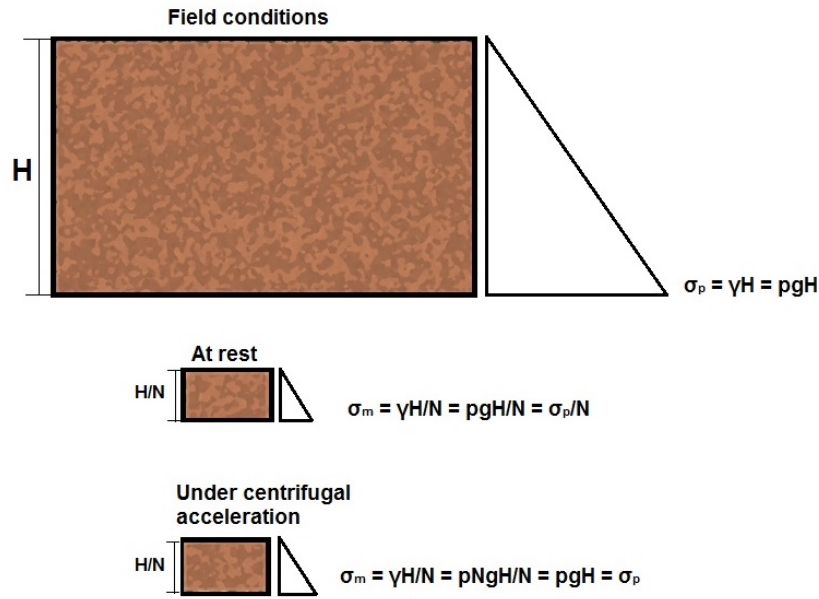


Fig. 2.12 Self-weight stress in the prototype scale and in the model scale at rest and under centrifugal acceleration.

Since the same self-weight stresses are applied at two corresponding points in the model and prototype at depth H/N in the model and H in the prototype, then:

$$h_p = N h_m \quad (2.11)$$

Equations 2.10 and 2.11 provide the basic scaling laws of centrifuge modelling and state that a soil model experiences the same self-weight stresses within its mass with a corresponding prototype, N times larger, when it is subjected to a centrifugal acceleration Ng .

Seepage

Darcy's law for seepage flow is shown in Equation 2.12.

$$v = \frac{q}{A} = -ki \quad (2.12)$$

where k is the coefficient of permeability and i the hydraulic gradient. In terms of the intrinsic permeability, K , which is subject to the soil grain size, shape and packing, the coefficient of permeability is written:

$$k = \frac{K\rho g}{\nu} \quad (2.13)$$

where ν is the dynamic viscosity, ρ the density of the fluid and g the gravitational acceleration. If the same pore fluid is used in the model and prototype, then the coefficient of permeability will depend on the gravitational acceleration and therefore:

$$k_p = \frac{1}{N}k_m \quad (2.14)$$

The hydraulic gradient i is dimensionless which implies that:

$$i_p = i_m \quad (2.15)$$

Therefore:

$$v_m = i_mk_m = i_pNk_p = Nv_p \quad (2.16)$$

Rainfall specifications

The velocity of flow is N times greater in the model than in the prototype. The total precipitated rainfall in the model will be N times less than that in the prototype, as this is measured in mm of water height (linear dimension). Rainfall intensity, which is the ratio between the total rain and the rain duration, will be N times higher in the model ([Askarinejad et al., 2012](#)). As an example, the simulation of a prototype rainfall event for 24 h, with intensity of 15 mm/h at 60g centrifugal acceleration requires a rainfall intensity $15 \times 60 = 900$ mm/h for a period of

Table 2.2 Scaling laws for rainfall parameters ([Askarinejad, 2013](#))

Term (dimension)	Prototype	Model
Total rainfall [L]	1	$1/N^2$
Rainfall duration [T]	1	$1/N^2$
Rainfall intensity [L/T]	1	N

$\frac{24 \times 3600}{60^2} = 24s$ in the model. The scaling of the rainfall parameters at centrifuge modelling is presented in Table 2.2.

2.4.3 Scale effects

Error in stress variation

In any centrifuge model, the distribution of stresses within the soil mass is not uniform as in the prototype. This is due to the variation of the acceleration in the space the model occupies. For the practical range of the soil depths involved in geotechnical engineering, Earth's gravitational field can be considered uniform.

The radially varying acceleration in the centrifuge model creates a non-linear stress distribution. The vertical stress at a depth z within the soil mass can be determined from ([Taylor, 1995](#)):

$$\sigma_{v,m} = \int_0^z \rho \omega^2 (R_t + z) dz = \rho \omega^2 z (R_t + \frac{z}{2}) \quad (2.17)$$

where $\sigma_{v,m}$ is the vertical stress in the model, ρ the soil's density, ω the angular rotational speed and R_t the radius between the centre of rotation and the surface of the soil model.

Figure 2.13 illustrates the depth distribution of the gravity stress in the prototype and the inertial stress in the centrifuge model. The comparison of the two curves shows two characteristic states which occur within the centrifuge soil model: i) An under-stress state, at the upper layers, where inertial stresses are lower than the gravity stresses and ii) an over-stress

state where stresses in the model are greater than in the prototype. Also, three characteristic depths can be identified, i.e. at the maximum under-stress (h_u), at the maximum over-stress (h_o) and the depth where the two stresses coincide (h_i). The maximum under stress occurs half-way between the soil surface and the depth h_i , i.e. $h_u = \frac{1}{2}h_i$. The maximum over-stress state always occurs at the bottom of the centrifuge model.

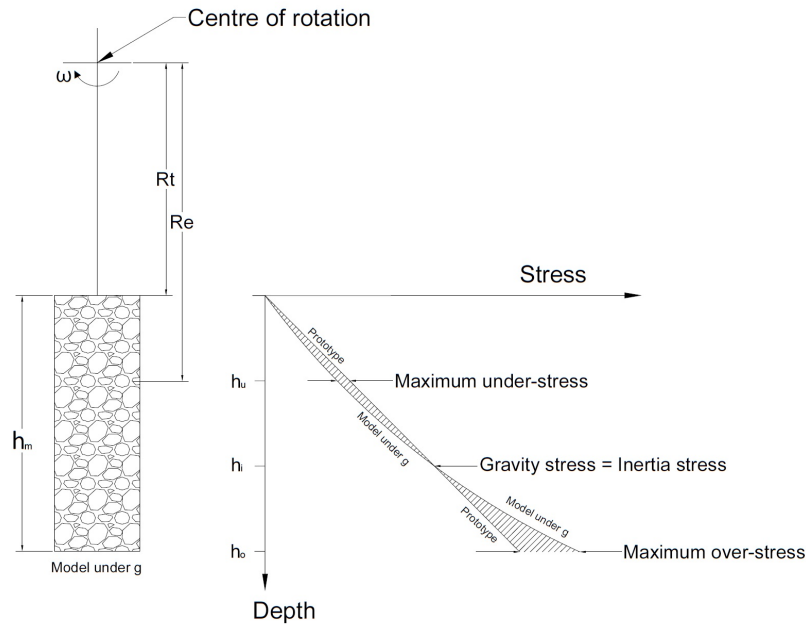


Fig. 2.13 Representation of the vertical stress distribution in a soil model under g and its prototype (after Taylor, 1995).

The maximum error in the model is given by:

$$r_u = r_o = \frac{h_m}{6Re} \quad (2.18)$$

According to Taylor (1995), the maximum error in the stress profile for most geotechnical centrifuges is generally less than 3% of the prototype stress, which is considered minor. The magnitude of the error in a centrifuge model must be kept low; however there is no specific guideline for the level of unacceptable error in a model.

Coriolis effect

Coriolis effect is a type of error in centrifuge models which involves the movement of an object with a velocity v in any direction within the plane of rotation of the arm (Schofield, 1980). The Coriolis acceleration, a_c is related to the angular velocity of the model, ω , and the velocity v and is given by the equation:

$$a_c = 2\omega v \quad (2.19)$$

On the other hand, the inertial acceleration of the model is given by:

$$a = \omega^2 R = \omega V \quad (2.20)$$

where V is the velocity of the model in the centrifuge flight.

The ratio between the Coriolis acceleration and the inertial acceleration of the model gives the error caused by Coriolis effect. It is generally assumed that an error less than 10% is acceptable, which implies $v < 0.05V$.

Droplet trajectories

Coriolis acceleration affects the trajectory of water droplets when rainfall simulation at g takes place. The deviation of water droplets injected under Ng by a spraying nozzle, compared to their trajectory at $1g$, was mathematically modelled by Caicedo et al. (2015) based on conservation laws for moments and mass. In all simulations, droplets were ejected in a cone at a 45° angle with respect to the y -axis (vertical axis of the model), and with angles in the horizontal plane of the model which vary in 30° steps. The radius from the axis of the centrifuge to the spraying nozzle was 1.4 m and the distance between the nozzle and the horizontal surface was 200 mm. The derived trajectories are illustrated in Figures 2.14 to 2.16. Results indicate that the spray patterns depend on the centrifugal acceleration (Fig. 2.14), the initial velocity of the droplet

(Fig. 2.15) and the droplet mass (Fig. 2.16). A general rule is that the smaller the size of the droplet the easier it is for it to deviate from its original trajectory. The same rule applies when the centrifugal acceleration increases or when the initial velocity is small. Based on these criteria, there are droplets which might not reach the ground during the centrifuge flight under specific conditions due to the Coriolis effect. It is essential to evaluate the maximum position of the spraying nozzle above the soil surface so as to avoid large distortion of the spraying cone, based on the specifications of the spraying system. However, soil erosion caused by large droplets or droplets with high initial velocity must also be avoided during the centrifuge modelling. Therefore, the optimum height of the spraying system must be determined for soil models subjected to rainfall under g .

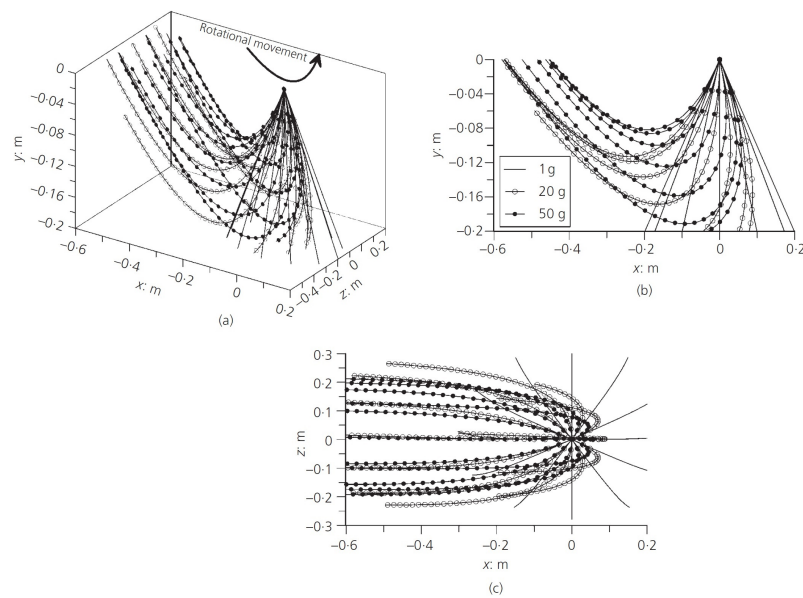


Fig. 2.14 Effect of centrifugal acceleration in droplet trajectories, at an initial velocity of 20 m/s : (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015).

Askarinejad et al. (2012) performed calibration tests in order to define the minimum distance between the spraying nozzles and soil surface so as to minimize both erosion and Coriolis effect. They determined this distance to be between 70 and 100 mm for centrifuge tests performed at 50g and for a mean droplet diameter of $40\text{ }\mu\text{m}$.

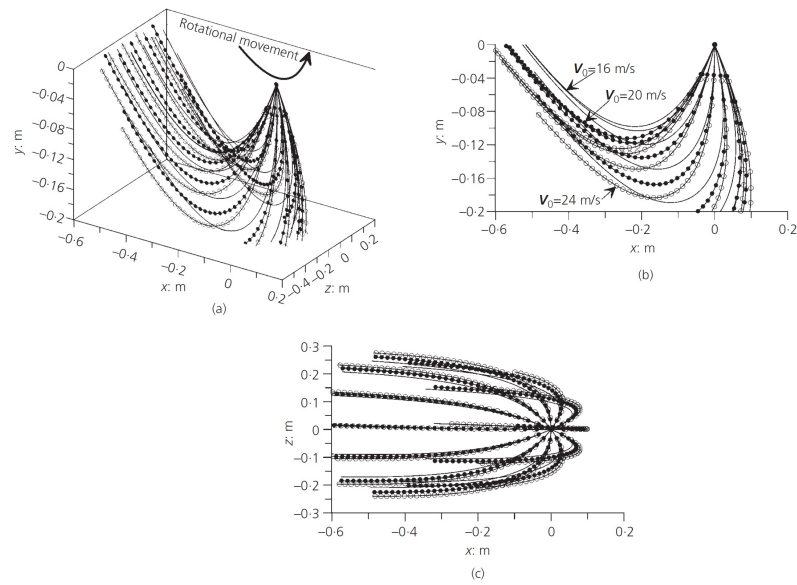


Fig. 2.15 Effect of initial velocity of droplet V_0 in droplet trajectories, at an acceleration of $20g$: (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015).

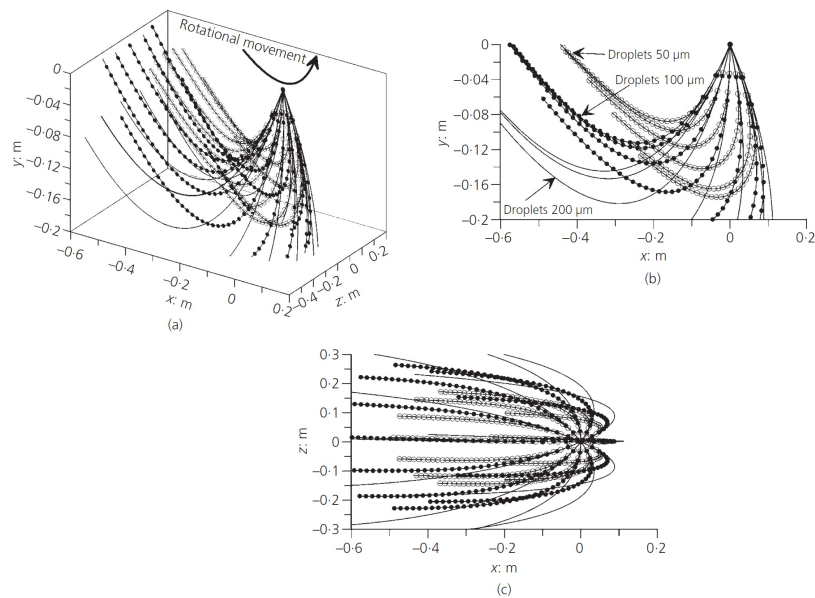


Fig. 2.16 Effect of the droplet size in droplet trajectories, at an acceleration of $20g$: (a) 3D view, (b) lateral view, (c) top view (Caicedo et al., 2015).

2.5 Centrifuge modelling of rainfall-induced landslides

During the last decade, many efforts have been made to model rainfall-induced landslides under centrifugal acceleration. For this reason, atmospheric chambers have been built in order to accurately simulate climatic conditions during the tests. Atmospheric conditions include rainfall characteristics (duration and intensity), evaporation, and temperature control. [Craig et al. \(1991\)](#) were the first to suggest the need for rainfall simulation in centrifuge models in order to conserve the moisture content of the soil. The idea came after noticing that during the centrifuge flight, slope models were dried considerably. The use of a simple rainfall generator was used in a slope model, representing an embankment in Malaysia, highlighting the importance of prevention of soil drying during the centrifuge flight. However, not much attention was paid either to the control of the rainfall characteristics or the control of the size of the droplets and the respective impact on the soil surface.

[Take \(2003\)](#) simulated seasonal pore pressure cycles of over-consolidated clay embankments in an increased gravity field produced by a geotechnical centrifuge (Fig. 2.17). The material used for the slope models was reconstituted kaolin clay. To control the atmospheric boundary conditions, they developed an atmospheric chamber which was capable of simulating dry and wet seasons ([Take and Bolton, 2003a](#)). Results revealed a progressive loss of the soil strength from season to season, with the wetting season causing swelling of the slope (due to negative pore pressure dissipation) and the dry season causing shrinkage. The repeated cycles produced accumulated damage to the slope which eventually failed during a wetting season.

[Hudacsek et al. \(2009\)](#) performed a similar investigation to [Take \(2003\)](#), describing the behaviour of a compacted clay embankment under climatic cycles. This time, glacial till was compacted to form small-scaled embankments which tested in a geotechnical centrifuge. The environmental chamber used in this study was able to control rainfall and evaporation so as to model wet (winter) and dry (summer) seasons. Even though the first seasonal cycles produced accumulated displacements of about 17 mm (at prototype), after eight simulated

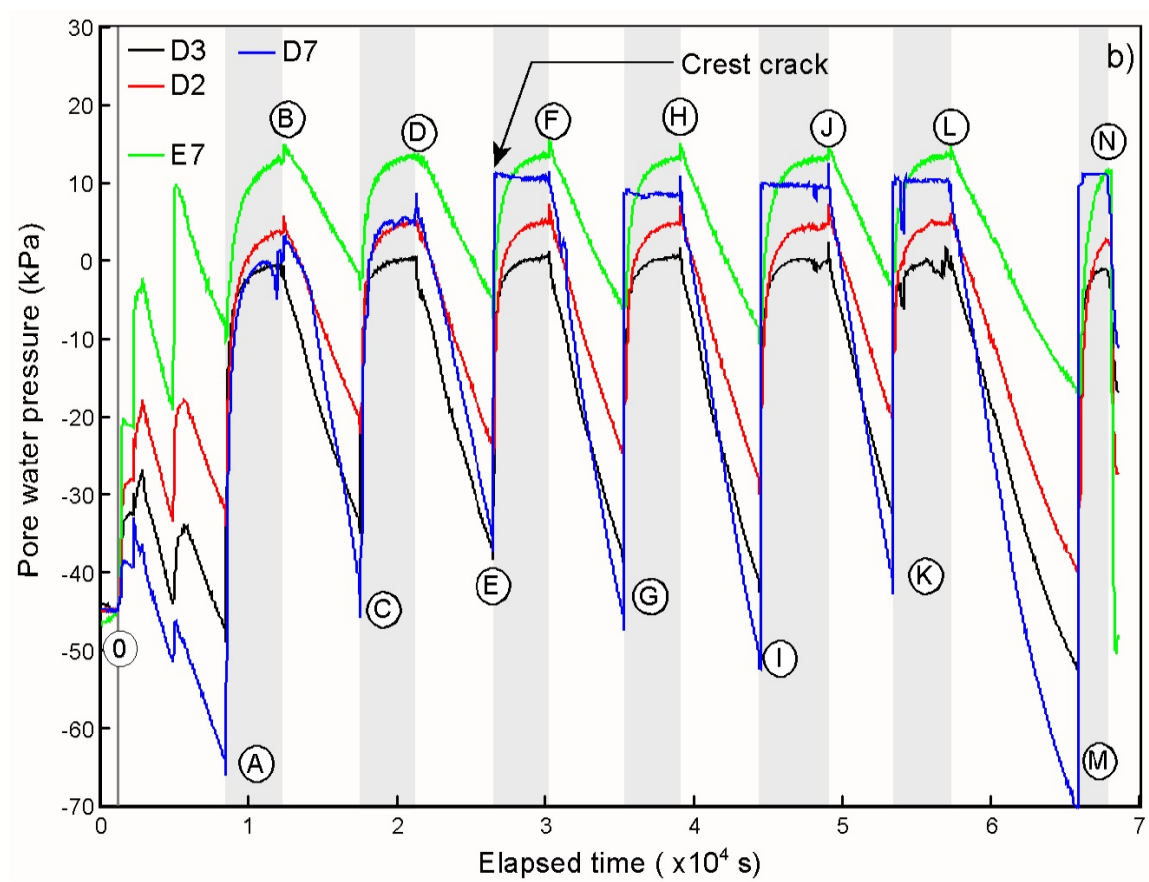


Fig. 2.17 Observed seasonal pore pressure cycles (Take, 2003)

seasons, the deformation rate decreased significantly. This was attributed to the stress history and mechanical properties of the model.

[Tamate et al. \(2012\)](#) studied the behaviour of sand slopes under rainfall conditions in a centrifuge. A comparison between the use of water and a viscous liquid for rainfall simulation was made. The inclination of the slope models were 30° and 45°. Results from a series of tests revealed that failure was driven by the advance of the wetting front in the slope profile and the respective saturation of the soil, leading to shear strength reduction. A very interesting conclusion from this study was the formation of different failure types in terms of the viscosity of the liquid used for rainfall simulation, even when equivalent permeability conditions were maintained. Shallow flow failures occurred with the use of water, while viscous liquid seemed to produce larger instabilities with deeper slip failures.

The simulation of two potential failure mechanisms in a completely decomposed granite was attempted by [Take et al. \(2004\)](#), i.e. static liquefaction and slide to flow. Decomposed granite was described as slightly clayey, silty, very gravely sand. Rainfall conditions were applied to the slopes by two rows of 5 mist nozzles which were placed close to the soil surface. In the case of the static liquefaction triggering mechanism, results indicated that failure is unlikely to occur in unsaturated soil slopes with a deep impermeable base. The reason is the high compressibility and mobility of the air in the unsaturated voids which allows the slope to accommodate wetting collapse without initiating undrained failure, despite of the soil behaviour in triaxial undrained tests where liquefaction took place. However, [Askarinejad et al. \(2012\)](#) pointed out that static liquefaction can only be initiated by a more viscous liquid than water in the centrifuge tests, as will be described below. In the case of the slide-to-flow triggering mechanism, loose and dense slope models were used, with the fine fraction of the material overlying the coarse one. In this experiment, rainfall infiltration was simulated by supplying water at the crest of the slope as a line source, allowing it to form a thin seepage layer which followed the contours of the low permeability material. High-speed with low-angle run-outs

are shown to be easily triggered in model fill slopes from initially slow moving slips driven by localized transient pore pressures arising from constricted seepage and material layering.

Wang et al. (2010) used silty sand to build heterogeneous slope models containing weak layers of montmorillonite and filter paper scrap mixtures having high permeability. Their intention was to evaluate the stability of heterogeneous slopes subjected to rainfall conditions, especially when these include highly permeable weak layers, and to compare them with uniform slopes. The results have shown that the wetting front moves faster when entering the weak layers, causing faster dissipation of suctions and the formation of shear zones within the slope. This zone became discontinuous at weak layers and, initially, widened due to the development of the deformation. As rainfall continued, the zone localized swiftly, leading to failure.

Ling et al. (2009b) performed centrifuge testing in very steep slope models with an angle of 60° consisting of sand and clay mixture. Rainfall was simulated by a system with mist spray nozzles with rainfall intensity varied between 5.3 mm/h and 11.8 mm/h, for centrifugal acceleration at 80g, while for centrifugal acceleration at 60g the intensity ranged between 6.1 mm/h and 13.6 mm/h. This parametric study allowed for the derivation of a rainfall intensity-duration threshold (local) curve which was compared with other local and global curves obtained by other studies (Figure 2.18).

Askarinejad et al. (2012) performed centrifuge model tests in a silty sand simulating the boundary conditions of a natural slope in Switzerland which was used as a full scale experiment. The natural slope was subjected to rainfall conditions for several hours until it failed due to a bottom-up saturation mechanism (Springman et al., 2012). Moreover, the static liquefaction failure mechanism was simulated by reducing the seepage velocity using a viscous liquid. The authors recommended the use of a liquid which was $\sqrt{N}$ times more viscous than water for rainfall simulation.

Static liquefaction failure mechanism was also studied by Min (2006) who performed centrifuge tests in loose fill slopes. The material used to build the loose fill slopes was a mixture

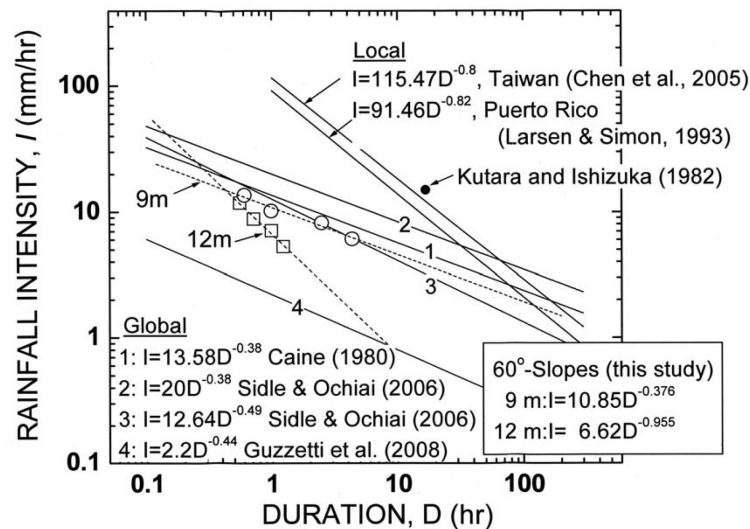


Fig. 2.18 Threshold curves: rainfall duration versus rainfall intensity relationships (Ling et al., 2009b)

of sand and completely decomposed granite (CDG). Groundwater boundary conditions included the rise of the water table and precipitation simulation. Rainfall conditions were applied to slope models by a rainfall simulation system which included an outlet panel consisting of four small compartments, while the bottom plate had 220 holes 0.5 mm in diameter. This configuration was designed in order to supply water only to the slope crest. Instead of minimizing the size of the droplets, in order to prevent the erosion from the impact force, a cover was applied on the slope crest consisting of an iron fine mesh, in the case of the sand slopes, or of a 5 mm layer of coarse CDG placed on top of the crest, in the case of the slopes formed by CDG. In the case of the ground water table rising, rapid flow failure was observed in a 24° loose sand fill slope which was caused by static liquefaction resulting from the strain-softening of the sand under undrained conditions. However, the triggering factor seemed to be a sliding on the edge of the slope crest, caused by a seepage flow failure under drained conditions due to rainfall application.

Caicedo et al. (2010) performed studies concerning physical modelling of the soil atmosphere interaction in the centrifuge, particularly concerning the process of evaporation. They

analysed the scaling factor for evaporation and concluded that to preserve the same equilibrium between the soil and the atmosphere, in the prototype and the model, the potential evaporation has to be increased by a factor of N .

Chapter 3

Instrumentation and apparatus

3.1 Geotechnical centrifuge facility

Physical modelling tests were performed in the geotechnical centrifuge facility based at the Nottingham Centre for Geomechanics (NCG), University of Nottingham. This is a 50 g - Tonne, beam type, centrifuge with 2.0 m radius; its main components are shown in Figure 3.1. The maximum acceleration is 150 g with a maximum payload of 500 kg at 100 g acceleration. For safety reasons, the machine is housed above ground in a circular chamber with internal diameter 5.0 m and 500 mm thick reinforced concrete walls, constructed on a 6.5×6.5 m floor slab.

Coarse balancing of the payload placed on the swing cradle is achieved by moving a fixed counterweight using a detachable screw jack prior to centrifuge flight. The location of the counterweight, before each test, is calculated using the payload mass. During the test, an *in-flight* balancing system of ± 50 kgm is activated when the deflection of the support pedestal at the location of one of the legs is higher than a specific level. This is achieved by movement of oil in the tubular rotor arms from one side of the main axis to the other. If the in-flight balancing system cannot compensate sufficiently for imbalance between the payload and counterweight,

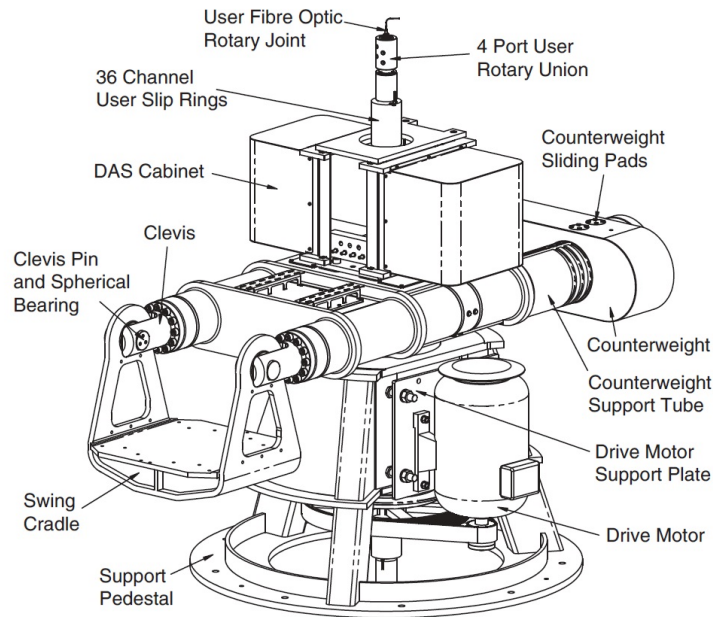


Fig. 3.1 Main components of the geotechnical centrifuge (Ellis et al., 2006).

the centrifuge stops automatically so that the counterweight position can be adjusted.

A number of electronic systems at the centre of the centrifuge and at the side of the arm allows for experimental control and data acquisition during tests. The data acquisition system includes channels for strain gauge and load cell connections power supply slots to solenoid valves. All channels are fed to the electronics rack, located in the DAS cabinet, and then they are transferred through a fibre optic link to the control room PC network (via the fibre optic rotary joint). Data acquisition happens on board and experiment control is carried out by two PCs located in the control room using Labview. Finally, a 4 port rotary union, located at the top of the control axis of the centrifuge, allows the transfer of water, air or vacuum to the centrifuge containers.

3.2 Testing apparatus

Two types of tests were performed in this study. Slope models tested under varying rainfall and boundary conditions are described in Chapters 5 and 7. Auxiliary centrifuge tests on soil column models are described in Chapter 6. The apparatus used for the slope model tests is described in this chapter whereas the one used for the soil column tests is described in Chapter 6.

Slope models were prepared and tested within a centrifuge model consisting of a strong box which accommodates the soil slope and a number of systems to control rainfall, groundwater and drainage conditions. A number of sensors were incorporated within to the centrifuge model for the measurement of positive and negative pore water pressure and rainfall rate. Moreover, a digital camera was used to capture images of the model during the centrifuge tests, thus allowing for optical measurement of slope displacements using the Particle Image Velocimetry (PIV) technique (Section 3.4).

3.2.1 Strong box

A plane strain strong box with internal dimensions of $600 \times 400 \times 200$ mm (length $\times$ height $\times$ width) was used for the slope model tests, as shown in Figure 3.2. It consists of an aluminium frame at the front which supports a transparent Perspex wall. The Perspex wall has a thickness of 80 mm and allows for observation of the model deformation. The out of plane strains are negligible under the increased stress level present during the centrifuge flights so as to maintain plain strain conditions. At its inner side, the Perspex wall contains control (or reference) points which are used for the PIV analysis (described in Section 3.4). The canal U-section part of the container is made of stainless steel and has a network of drainage grooves at the bottom from which water can flow to two drainage valves at either side of the box. These valves were also used for the saturation of the soil block before it was cut into a slope, as described in Section 4.3.1. At either side of the slope model, two perforated aluminium boxes assisted in

the drainage of the slope during tests. Also excess water accumulated on the model surface was removed through the drainage boxes. Finally, the aluminium back wall was modified in order to allow PPTs to be installed within the slope. Eight holes with a diameter of 8 mm were drilled at the back wall and cable glands, for the installation of PPTs, were fitted.

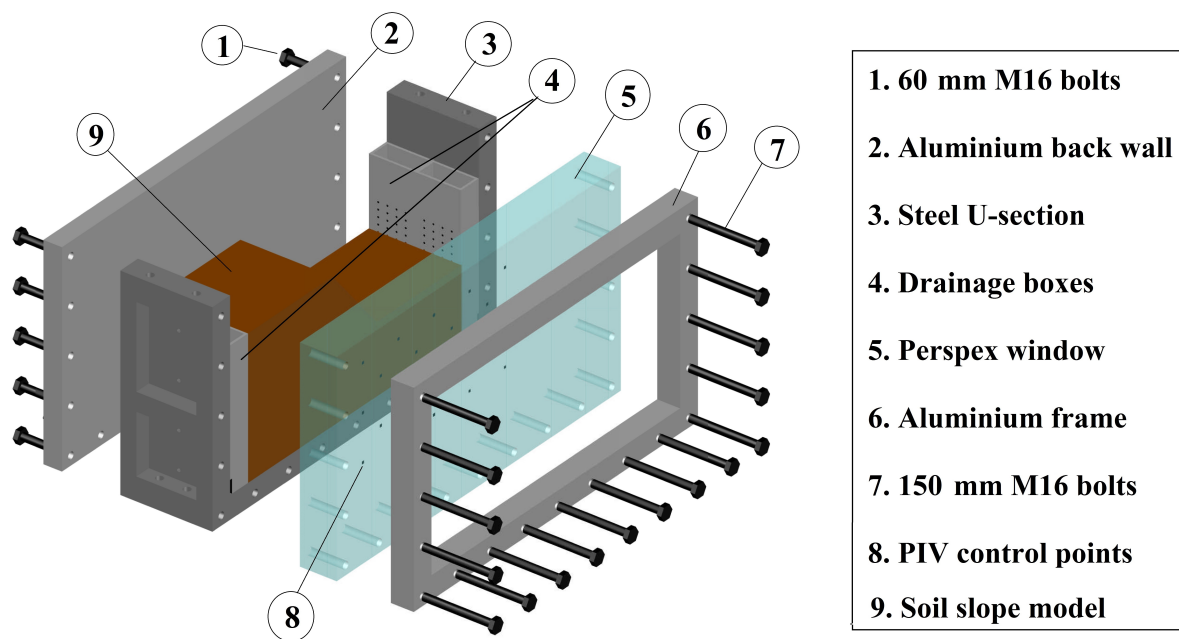


Fig. 3.2 Centrifuge box used for slope model tests.

The lid which was placed at the top of the box is shown in Figure 3.3. This is a 10 mm thick aluminium plate which is fitted with a 6 mm diameter o-ring along the bottom. The o-ring acts to seal the model during the centrifuge tests. Aluminium tubes acting as extensions for the spraying nozzles were also attached to the bottom of the lid. The top side of the lid housed a rainfall distribution system and a number of instruments, i.e. relative humidity and temperature sensor, webcam, flow meter and solenoid valves.

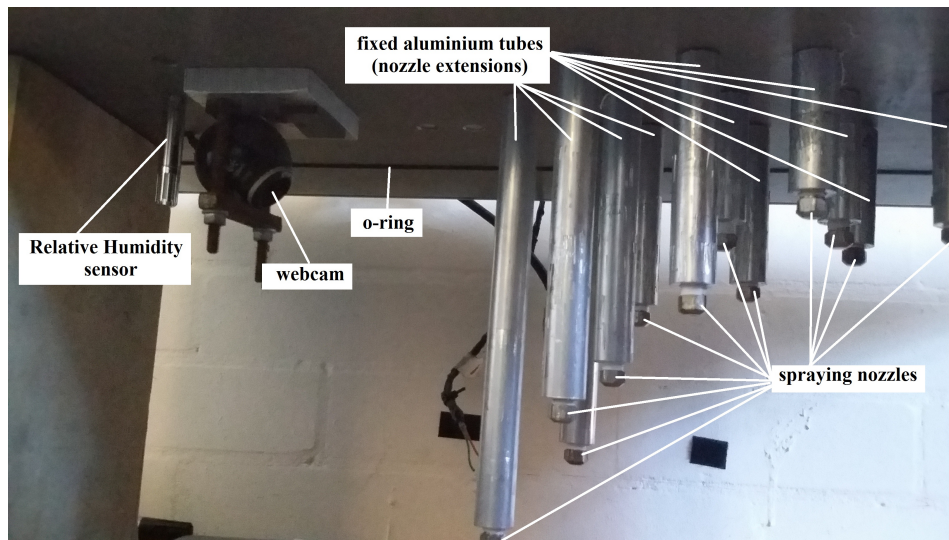


Fig. 3.3 Bottom side of the lid.

3.2.2 Rainfall system

The simulation of rainfall conditions on centrifuge slope models is described in this section. The rainfall system consists of the water distribution system, spraying nozzles, a solenoid valve and a flow meter (Figure 3.4). An overview of the slope model and the location of the spraying nozzles is shown in Figure 3.5. Rainfall was simulated using a series of spraying/misting nozzles attached to aluminium rods (extensions) and located below a sealed lid. An overview of the slope models and the relevant location of spraying nozzles is shown in Figure 3.5. The distance between the nozzles and the soil surface should be such that Coriolis effect does not incur significant impact to the distribution of rainfall on the model, which is translated in placing the nozzles very close to the model surface. On the other hand, the impact force of the droplets, due to the high pressure of the water, can erode the soil if the spraying nozzles are placed very close to the model. Based on previous studies about the optimum distance of the spraying nozzles (Askarinejad et al., 2012; Caicedo et al., 2015; Tamate et al., 2012), they were installed in such a way that the distance from the model surface was between 70 and 100 mm. Nozzles were connected to a network of pipes which distributes the water used in

rainfall simulations. A supply water tank, located outside the centrifuge, was connected to the nozzles via the hydraulic rotary joint. A flow measurement sensor and an on/off solenoid valve were located between the water tank and the nozzles.

The area in which the PPTs were installed before each centrifuge test and its relevant location in respect to the spraying nozzles is also shown in Figure 3.5. The exact location of the PPTs for each individual test is presented in Chapters 5 and 7.

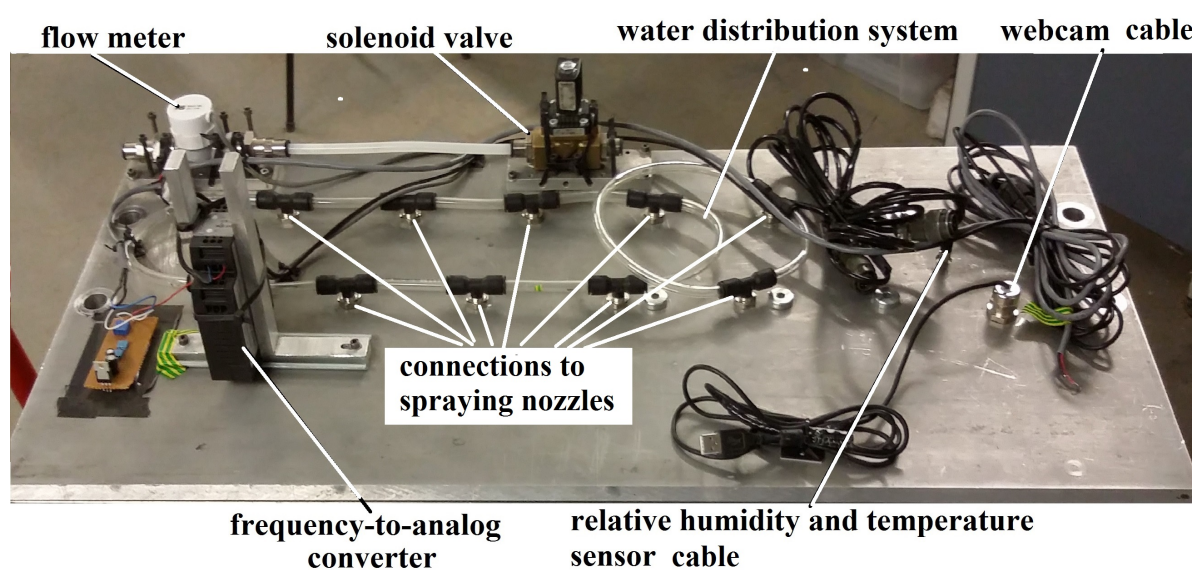


Fig. 3.4 Rainfall distribution system.

Water supply

Water was provided by the mains supply tap to the testing package through the slip rings of the centrifuge. A flow regulator was installed to the mains supply in order to restrict the flow rate to the maximum value measured by the flow meter ($= 6.5 \text{ l/min}$). The water pressure provided by the mains was measured at 600 kPa at the maximum flow rate. However, this was reduced to a maximum of 300 kPa before entering the centrifuge package.

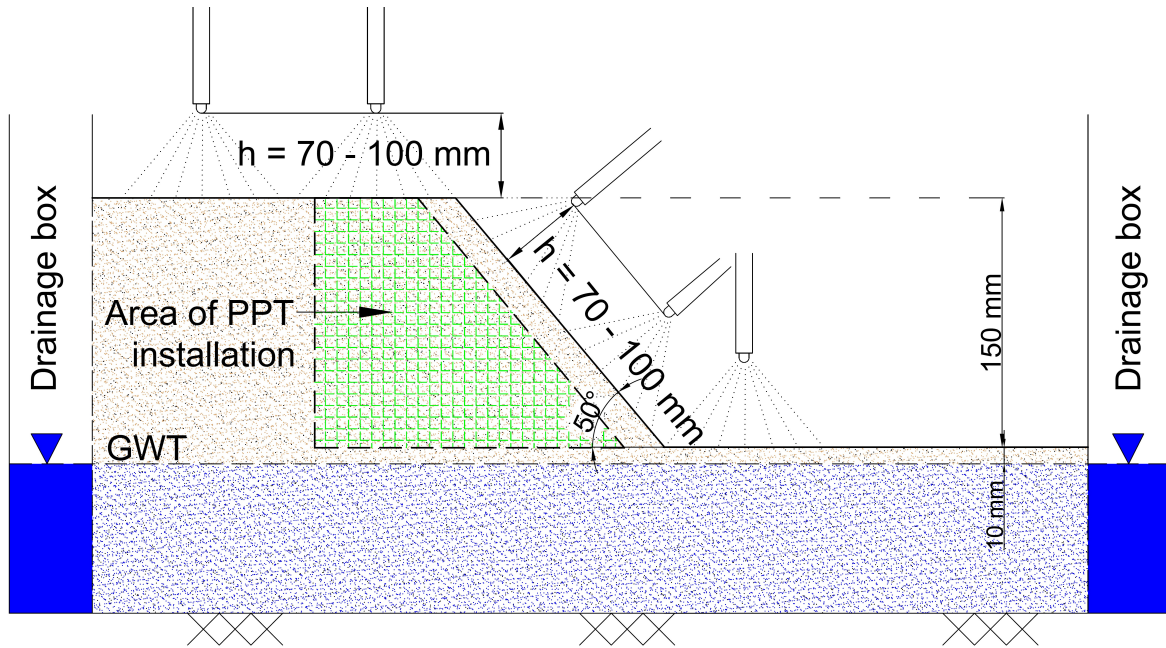


Fig. 3.5 Overview of the slope model and the location of the spraying nozzles

The water line was transferred from the slip rings to the testing box using a pipe with internal diameter of 10 mm. The pipe was mounted to the centrifuge arm and fixed to the inflow using a push-in fitting. Although the water pressure was reduced near the mains, centrifugal acceleration caused a significant pressure increase due to the increment of the water column self-weight. Slope centrifuge tests were operated at 60 g, therefore the experienced water pressure at the nozzle delivery system was almost 900 kPa. This could have an implication to the flow rate of the rainfall system since the flow rate of the nozzles depend on the pressure of the water delivered. To avoid uncertainties related to the flow rate and therefore the rainfall intensity, a flow meter was installed at the rainfall system so as to measure accurately the volume of water delivered to the nozzles.

Flow meter

A RS Pro Liquid Acetal Copolymer 4.5 → 16 V dc sensor (RS 257-149) flow meter was placed on the top of the lid in order to monitor the water flow delivered to the nozzles. The whole setup shown in Figure 3.4. The device is able to measure liquid flow with rates between 0.25 l/min and 6.5 l/min, at a maximum pressure of 2000 kPa.

The flow meter includes an infra-red light emitting diode. A turbine, which rotates by the flowing water, periodically breaks the light beam producing light flushes and eventually pulses (approximately 4600 pulses/litre) detected by a photo-detector circuit. The LED is directed at the receiver which has a built-in voltage regulator, photo diode, amplifier, schmitt trigger and output. The pulse signal is of high frequency (typically greater than 300 Hz) and therefore it cannot be detected reliably by the data acquisition system. For this reason, a frequency-to-analogue converter was connected to the flow meter in order to convert the pulse signal to a linear D.C. signal. A Red Lion IFMA0035 converter was used which operated under D.C. power and converted the signal into a standard 0-5 V analogue signal across the full range of flow rates.

The flow meter was powered by an external power supply, located outside the centrifuge, passing through the slip rings to the model. The power to the LED was limited so as the supply current did not exceed the recommended value by the manufacturer ($= 30 \text{ mA}$). For this reason, a voltage regulator circuit with a series of resistors and potentiometers was used to regulate the power supply and protect the flow meter.

The flow meter was calibrated in order to define the relationship between the flow rate q and the voltage signal. The calibration results are shown in Figure 3.6. Results show that there is a linear relationship between the flow rate and the signal acquired from the data acquisition system.

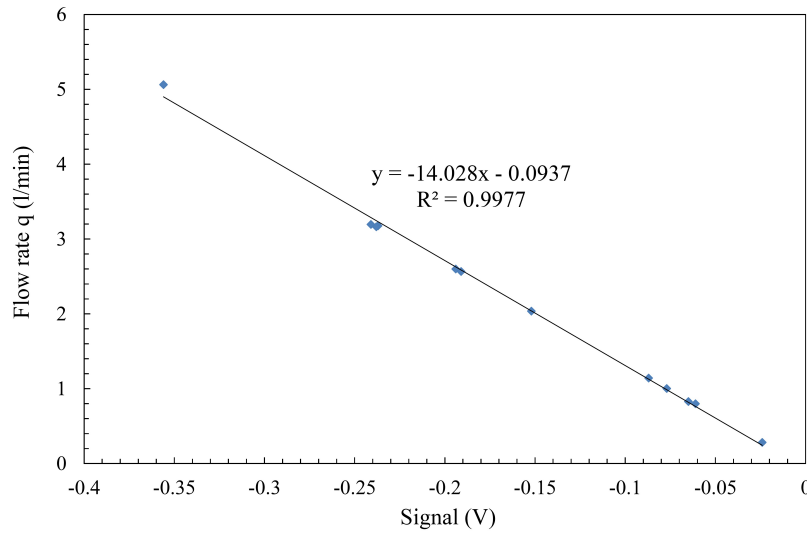


Fig. 3.6 Calibration results for the flow meter.

Spraying nozzles

Type ‘M’ spraying nozzles manufactured by Hago Manufacturing Co. and purchased from ICS Industrial Services Ltd were used for the tests. The general layout of the nozzles is presented in Figure 3.7a. The nozzles are made of stainless steel or brass and have a fine mesh (filter) at the inlet which prevents the outlet hole from clogging. They have a small size which makes them ideal for simulating rainfall conditions in centrifuge models. Their spraying pattern is solid cone-shaped providing a fairly uniform supply of water over the entire spray area (Fig. 3.7b). There are two main characteristics of the spraying nozzles which need to be taken into account when simulating rainfall in centrifuge models: the flow rate and the mean size of the water droplets.

Flow rate is related to the prototype rainfall intensity simulated in the centrifuge tests. As mentioned in Section 2.4.2, the scale factor for rainfall intensity is $I_{r,prototype}/I_{r,model} = 1/N$. This indicates that the applied rainfall intensity to the model must be N times higher than in the prototype, for similitude. Three nozzle types were chosen for this study. Nozzle M1 provided

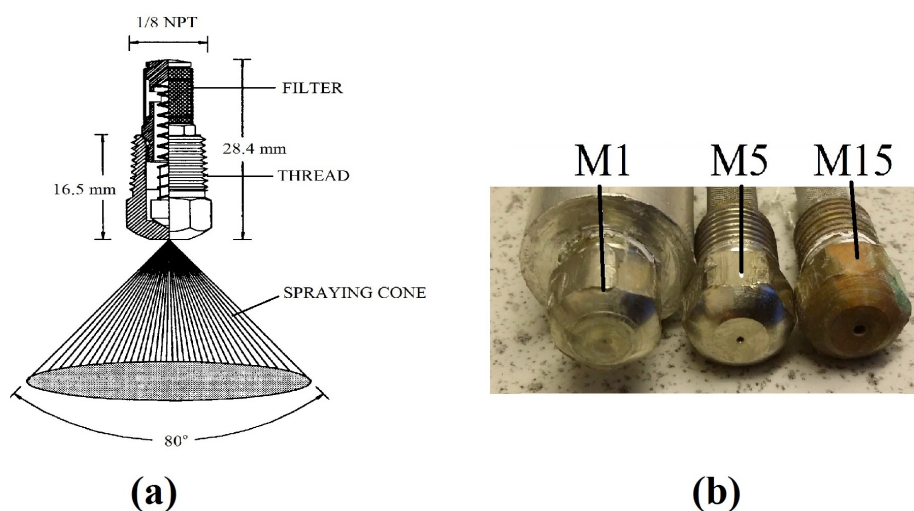


Fig. 3.7 a) Layout of spraying nozzles, type 'M' (Take, 2003), b) Types M1, M5 and M15.

the lowest flow rate, M15 the highest flow rate and M5 with an intermediate flow rate (Figure 3.7c).

The flow rate of the spraying nozzles depends on the applied water pressure. At low pressures, below 200 kPa, nozzles do not operate properly with the water dripping rather than spraying. The lowest operating pressure recommended by the manufacturer is 300 kPa. Figure 3.8a presents the results from calibration tests for nozzle M1 in order to relate the flow rate with the operating pressure. The values of the flow rates provided by the manufacturer are also shown in the graph. The calibration results show that there is a small deviation between the actual flow rates and the expected ones with the actual values being lower. This can be attributed to either a mistake in the manufacturer's estimation or to the blockage of the nozzles due to dirt within the water. Also, accumulation of limescale deposited on the nozzle filter or around the orifice, due to a high concentration of ions in the water, may cause a reduction of the flow rate of the nozzles with time. Nozzles were frequently submerged in a descaling solution (mixture of water and vinegar) so as to retain their initial flow capacity. In Figure 3.8b, the flow rates for the three types of nozzles used in this study are shown with respect to the

pressure of the provided water.

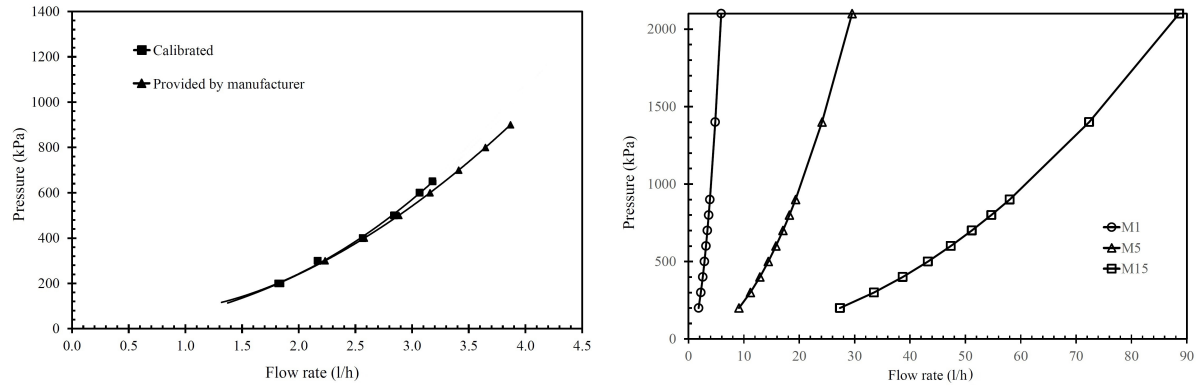


Fig. 3.8 (a) Calibration of the flow rate for nozzle M1, (b) flow rates for nozzles M1, M5 and M15.

Rainfall characteristics

The intensity of a prototype rainfall event is scaled to the model by N . This means that model rainfall intensity must be N times larger for similitude.

As shown in Figure 3.7, the cone of the spraying nozzles is 80° . For a mean distance between a spraying nozzle and the soil surface of 70 mm, the diameter of the circular spraying area is 117 mm and the area of spray is equal to $10,838 \text{ mm}^2$. If the flow rate of a nozzle is equal to q (in l/h), then the total precipitation in the model is expressed by the following equation:

$$I_r(\text{model}) = \frac{q}{A_s} \times 10^6$$

where $I_r(\text{model})$ is the rainfall intensity in the model in mm/h and A_s the spraying cover area in mm^2 .

Based on these calculations, the specifications of the spraying nozzles concerning the rainfall characteristics applied to the slope models in this study are presented in Table 3.1.

Table 3.1 Specifications and rainfall conditions applied with the three nozzle types.

	M1	M5	M15
Model rainfall intensity (mm/h)	450	1660	5262
Prototype rainfall intensity (mm/h)	7.4	28.8	87.7
Prototype rainfall intensity (m/s)	2.05×10^{-6}	8×10^{-6}	2.44×10^{-5}
Characterisation of rain	Light	Moderate	Heavy

3.2.3 Groundwater control system

Slope stability tests in the centrifuge enable very well defined boundary conditions. These include the location of the groundwater table below the slope toe and its variation during rain infiltration. To accomplish this, a groundwater control system was designed and fitted to the centrifuge testing package (Figure 3.9). The system consists of water reservoirs at each side of the model and two aluminium standpipes located outside the centrifuge box with overflow valves fitted at pre-defined locations.

The water reservoirs were made using perforated aluminium plates with 2 mm diameter holes at 20 mm intervals. The plates measured 400 mm $\times$ 200 mm (*height* $\times$ *width*) and were located between the soil and the water reservoir. The water reservoirs allowed water to drain from the soil at both sides of the model. A fine mesh (0.36 mm aperture) was placed on the plates to protect the holes from clogging. For slope tests with silty sand, filter paper was also used for the protection of the holes since there were grains smaller than 0.36 mm in the soil.

The level of water within the reservoirs defined the location of the groundwater table within the slope. The reservoirs were connected to the standpipes, which were fixed outside the testing box, through push-in valves and plastic pipes. The predefined locations in the standpipes maintained a constant water level during tests.

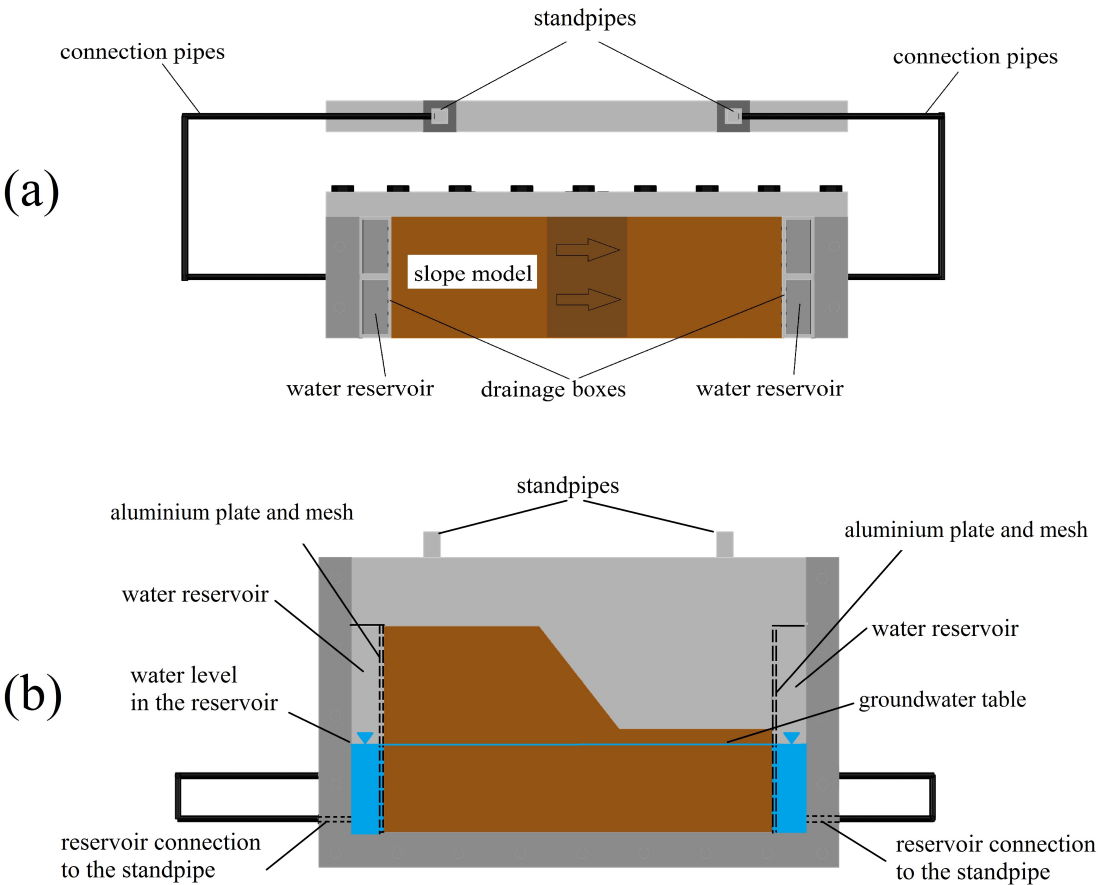


Fig. 3.9 (a) Top view and (b) side view of the groundwater control system.

The control of the groundwater conditions in the slope models considered two different boundary conditions. In the first condition, the groundwater table was kept constant and deep during the centrifuge spin-up and rainfall periods, as shown in Figure 3.10a. The water flowing out of the soil above the water table (due to de-saturation) was removed from the soil while the gravity field was increased. Pore water drained into the reservoirs which retained a constant water level. In this way, the groundwater table was kept constant at a predefined depth below the slope toe. During rainfall conditions the infiltrated water also drained into the reservoirs and therefore the water table was once again kept deep.

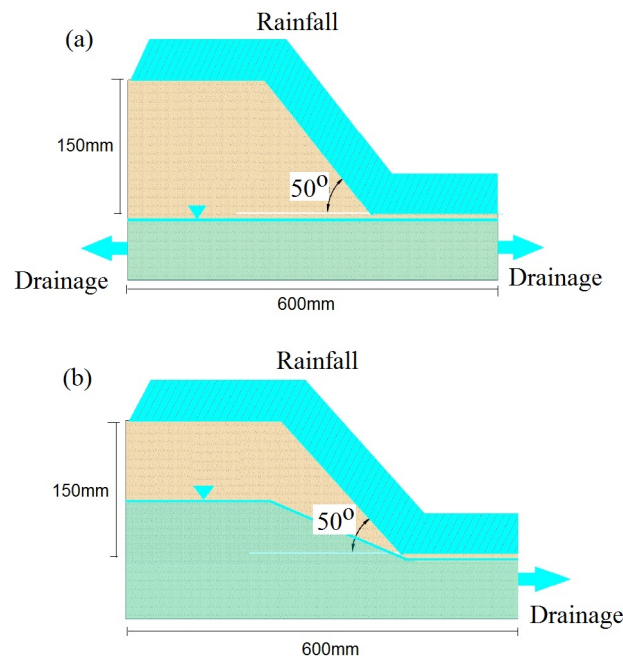


Fig. 3.10 Boundary conditions applied to the slope model tests. (a) deep groundwater table (b) shallow groundwater table.

The second boundary condition involved the elevation of the groundwater table during rainfall. This condition was achieved by closing the connection between the standpipe and the upslope reservoir, therefore allowing the water to drain only through the down slope valve (right side). Consequently, a shallow water table was formed in the slope, as shown in Figure 3.10b.

The rise of the water table was achieved by introducing rainfall water to the slope, therefore it occurred during and after the rainfall events. This boundary condition is typical in soil slopes, especially in residual soil formations which experience negative pore water pressures (Fredlund and Rahardjo, 1993; Rahardjo et al., 2010). Finally, this boundary condition was achieved also by manually inserting water to the upslope drainage box, with no rainfall applied to the slope model.

3.2.4 Image capture system

Images of the slope model were recorded using a monochrome Prosilica GC2450 digital camera (Figure 3.11). This camera has a compact and stiff construction which makes it ideal for use within centrifuge modelling applications. It has a maximum resolution of 5 MP (2448 (H) $\times$ 2050 (V) pixels) and 3.45 μm pixel size. There are two key features of the camera which are very important for the PIV analysis, as described by Stanier et al. (2015); the type of sensor and the type of shutter. The GC2450 incorporates a 2/3" Progressive-scan-CCD sensor which uses one amplifier for all pixels (whereas CMOS sensors have separate amplifiers for each sensor). Therefore, CCD sensors are very consistent which leads to more precise analysis using PIV. Another critical feature of this camera is the use of a global shutter instead of a rolling shutter. A global shutter allows the camera to capture the intensities of all pixels simultaneously rather than the intensity of each pixel in turn in a row to row basis. Consequently, a better precision is achieved when analysing frames of fast moving objects due to a reduced apparent distortion.

At full resolution, the GC2450 captures images at a rate of 15 Hz. It has a digital output via GigE Local Area Connection (LAN) which offers the ability to transfer large data packets in a short time period. An 8 mm lens was attached to the camera.

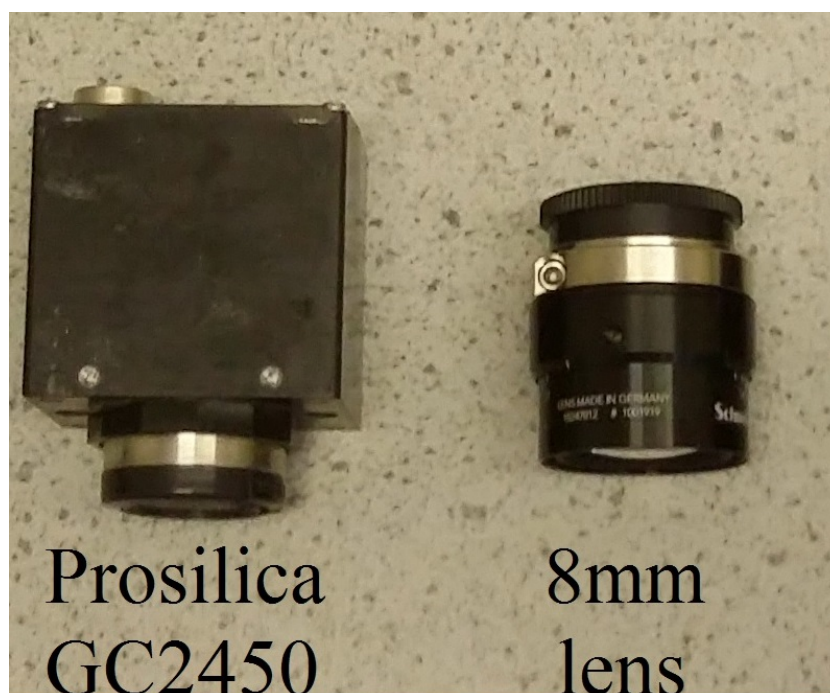


Fig. 3.11 Prosilica camera and 8 mm high resolution lens.

The position of the camera during the centrifuge tests is shown in Figure 3.12. The camera was clamped to a cradle which was fixed at the side of the centrifuge platform. Light Emitting Diode (LED) strips were mounted on the aluminium frame of the strongbox. These provided uniform illumination to the model and allowed the camera frame rate to be set to its maximum value (i.e. 15 frames per second).

The most important of the camera settings are shown in Table 3.2. The exposure time was in accordance with the PIV needs described in Section 3.4. The control of the camera was performed remotely using a Labview subroutine (also called Virtual Instrument or VI). The Labview subroutine included a live preview of the camera field which offered the ability to observe the model in real time during tests. Another preview window showed images immediately after capture. The frame rate was set at the time of operation allowing the choice

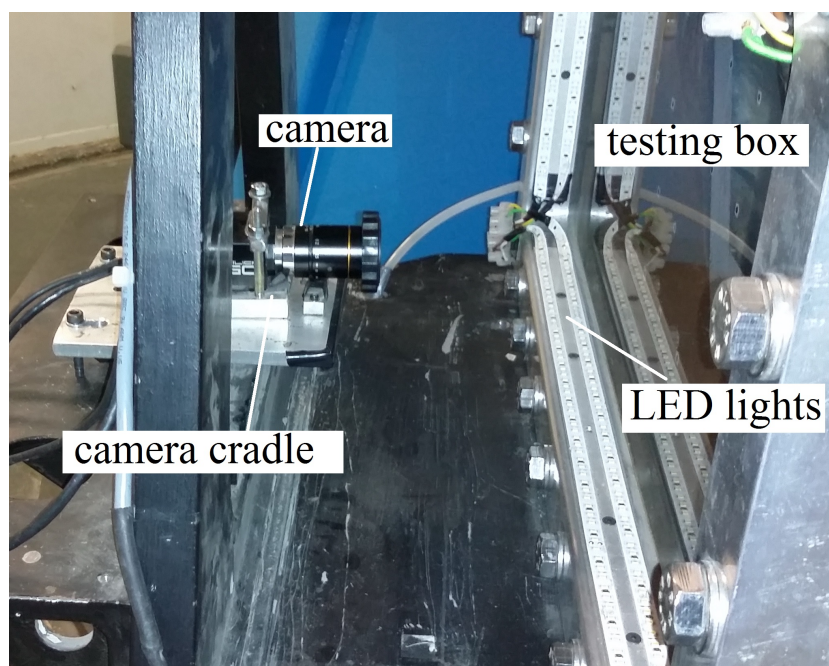


Fig. 3.12 Position of the Prosilica camera for capturing images during the centrifuge tests.

Table 3.2 Configuration of the Prosilica CG2450 camera for the slope tests.

Lens	8 mm
Exposure time	30000 μ s
Frame rate	15 Hz
Window dimensions (in px)	2448 (H) $\times$ 2050 (V)

of different frame rates at different stages of the centrifuge test.

3.3 Tensiometers

Tensiometers have been widely used in geotechnical centrifuge applications (e.g. [Askarinejad et al., 2014](#); [Hudacsek et al., 2009](#); [Min, 2006](#); [Take and Bolton, 2002, 2011](#); [Take et al., 2004](#); [Tamate et al., 2012](#)). The main advantage of these sensors is the direct and continuous measurement of matric suction. The osmotic suction component cannot be measured by tensiometers

as soluble salts can move freely through the porous filter (Murray and Sivakumar, 2010).

The main principle of tensiometers is based on the exchange of water between the soil mass and the tensiometer, so as to equalize the difference between the pore water pressure and the water pressure within the instrument. Over the last few decades, a number of tensiometers have been developed that are able to measure suctions above 100 kPa. Mair (1979) used a rigorous saturation program and a high air-entry filter element in order to measure suctions above 100 kPa in soils upon unloading from high pressures. Ridley and Burland (1993) developed an instrument which is able to measure suctions above 1500 kPa. These are also called *High Capacity Tensiometers*.

Pore Pressure Transducers (PPTs) can be used as tensiometers for the direct measurement of matric suction if they are fitted with a ceramic porous stone. In general, a PPT consists of the main body, the transducer (diaphragm), a small water reservoir, and a ceramic porous stone. The porous stone, often called filter, needs to remain saturated at all times in order to ensure good contact is maintained between the pore water and the PPT reservoir. When this condition is met, water pressure is transmitted to the water in the reservoir and is measured by the deformation of the diaphragm. The diaphragm is strain gauged and can provide measurements of both positive and negative pressures.

There are three key factors affecting the performance of PPTs in measuring suction: the permeability of the porous stone, the sensitivity of the pressure gauge, and the saturation ratio. The permeability of the porous stone is expressed with its *Air Entry Value* (AEV), which refers to the level of negative water pressure at which air starts to enter the pores. The AEV of a soil can be determined by the SWRC as the pressure which initiates soil desaturation (see Section 2.2.4). Since the contact between the pore water and the water within the porous stone

can be lost under this pressure, it is argued that the AEV of the porous stone is also the limit of suction measurement for the PPT. Higher suction will cause desaturation of the porous stone and eventually cavitation of the instrument (see Section 3.3.4). Furthermore, the level of saturation of the porous stone is very important for the performance of the tensiometer. The maximum measurable suction can be limited by the expansion of trapped air bubbles into the porous stone or the nucleation of vapour bubbles and release of trapped bubbles in crevices of the tensiometer body (Take and Bolton, 2003b).

Druck PDCR-81 Pore Pressure Transducers (PPTs), fitted with porous stones were used in the current study for the measurement of positive and negative pore water pressures (Fig. 3.13a). PPTs had a full scale capacity of either 300 kPa or 700 kPa and each one of them was equipped with a porous stone (sometimes called *filter*). The porous stone was fitted at the top of the sensor, while epoxy was used to seal the space between the stone and the steel casing. The selection of the AEV for the porous stone was made based on the maximum expected suction that would be measured during the centrifuge tests. Since suction rarely goes above 50 kPa in sandy soils, porous stones of 1 bar (i.e. 100 kPa) AEV were considered sufficient for the current study. Moreover, the selection of low AEV for the porous stones was made in order to increase the resolution of the sensors at the measurement of negative pore water pressures. Since the AEV of the porous stone limits the maximum suction capacity of these sensors, PPTs were able to measure a maximum positive pressure of 300 kPa or 700 kPa and a maximum suction of 100 kPa.

Furthermore, two EPB-PW tensiometers, manufactured by Measurement Specialties, Inc. and purchased from StrainSense Limited were used (Fig. 3.13b). These sensors had a maximum pressure capacity of 1500 kPa and were also fitted with a 1 bar porous filter. Therefore, these sensors had the ability to measure positive water pressure of 1500 kPa and maximum suction of

Table 3.3 Specifications of the tensiometers and porous filters used in this study.

Sensor ID	Pressure range (kPa)	AEV of porous filter (kPa)	Max. suction measured (kPa)	Excitation voltage (V)	Sensitivity (mV/V/bar)
PPT2	700	1	100	10	2.313
PPT5	700	1	100	10	2.326
PPT7	300	1	100	10	5.023
PPT9	300	1	100	10	4.962
PPT11	300	1	100	10	5.514
PPTP	1500	1	100	10	
PPTN	1500	1	100	10	

100 kPa. The specifications of the tensiometers and porous filters used are presented in Table 3.3.

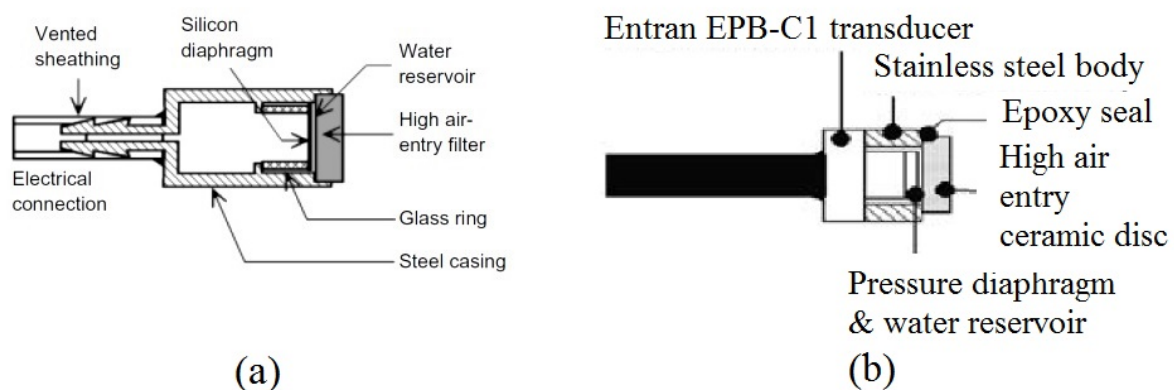


Fig. 3.13 (a) Druck PDCR-81 Pore Pressure Transducer (after König et al., 1994), (b) EPB-PW tensiometer (Take, 2003).

Both sensors (Druck PPTs and EPB-PW tensiometers) were used for the measurement of suction in this study. To distinguish them in the tests, Druck PPTs were named as PPTXX, where XX was the serial number of each sensor. On the other hand, EPB-PW tensiometers

were named as PPTP or PPTN. For each centrifuge test, the type and name of the sensors used as well as their location is clearly stated at the model plan.

3.3.1 Saturation and calibration system

The reliable measurement of negative pore water pressures from tensiometers strongly depends on the degree of saturation of the porous stone. The saturation process which was followed is described in Section 3.3. In order to ensure the reliable saturation of the tensiometers, a saturation and calibration rig was built, based on the work of [Take and Bolton \(2003a\)](#). This apparatus is shown in Figure 3.14. It is made of brass and includes 8 ports for the simultaneous saturation of 8 tensiometers and 4 valves which can be connected to air/water/vacuum lines or used to measure the internal pressure of the air/water.

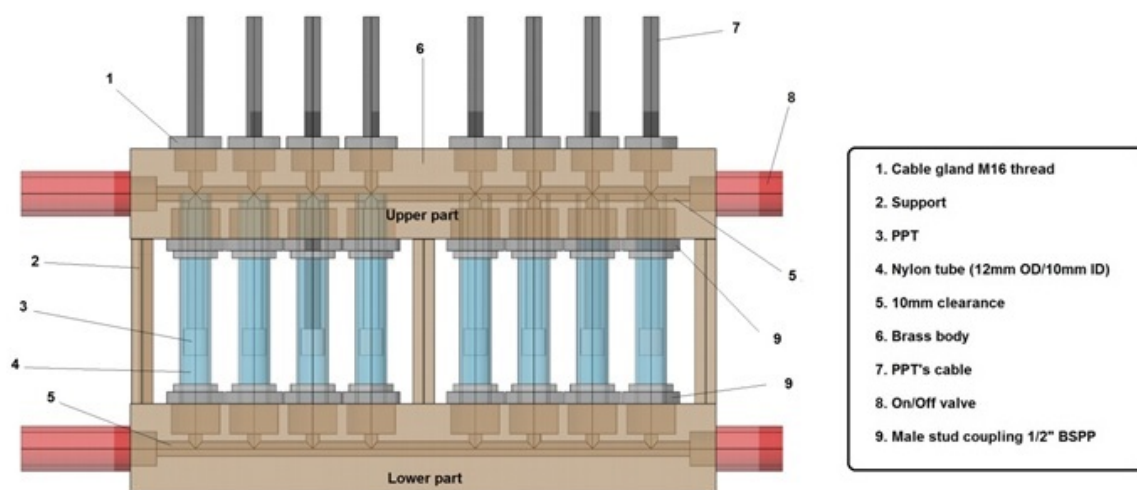


Fig. 3.14 X-ray view of the saturation and calibration rig.

The apparatus consists of upper and lower parts which are supported by three steel columns. Both parts are drilled from one side to the other forming two internal horizontal pipes 10 mm in diameter. At their ends, these pipes are connected to on/off valves which are connected to the vacuum line, the de-aired water supply and the water pressure line. Also, one of the valves is used to connect a pre-calibrated digital pressure meter (DPI) which determines the air or

water pressures within the chamber. This was a Comark digital pressure meter (RS 327-7461) with a range of ± 700 kPa which can be used for both air and water. Eight vertical transparent tubes, with internal diameter of 12 mm, are connected to the upper and lower parts using stud couplings. These provide connection between the horizontal pipes and form a closed circuit, as shown in Figure 3.14. Tensiometers are placed in one of the eight slots and their cables sealed with a cable gland fitted on the upper part of the apparatus. The cable glands hold the tensiometers and provide isolation for the inner space of the chamber. When all available tensiometers have been inserted to the saturation rig, the empty slots are sealed using blank plugs.

The saturation and calibration rig fits into a system which provides de-aired water as well as positive and negative water pressures, as shown in Figure 3.15. The different procedures involving the saturation and calibration of the PPTs are described in the next sections.

Water de-airing

Dissolved Oxygen (DO) refers to the level of free, non-compound oxygen present in water. DO enters water through the surrounding atmosphere with a slow diffusion process. This process is quickened through aeration which can be either natural (wind, waves, waterfalls, groundwater discharge) or man-made. Water will absorb oxygen and other gases from the atmosphere until it reaches equilibrium. The amount of DO in water can be expressed as either a percentage or as milligrams of air per litre of water (mg/l). 100% DO means that the water is holding as many dissolved gas molecules as it can in equilibrium. It states that at equilibrium the percentage of each gas in the water would be equivalent to the percentage of the gas in the atmosphere, i.e. its partial pressure. However, the percentage of DO in water can exceed 100% when super-aeration occurs, usually in waterfalls, or at rapid temperature changes. The actual amount of DO in water (expressed in mg/l) varies depending on the temperature, pressure and salinity (Wetzel,

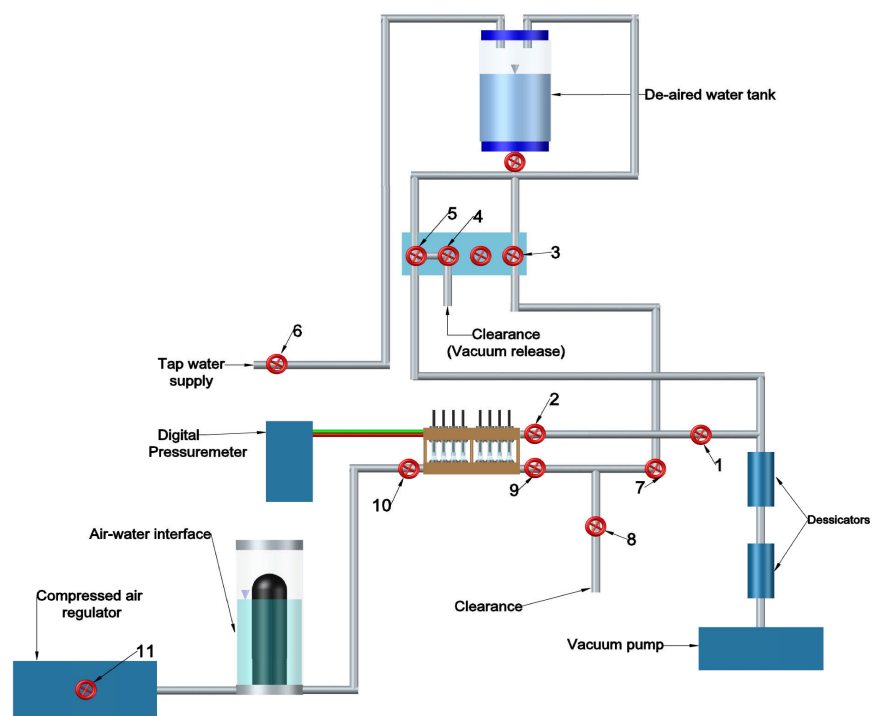


Fig. 3.15 Saturation and calibration system for the PPTs (not to scale). Numbers 1-11 indicate valves.

Table 3.4 Dissolved oxygen in three types of water.

Water type	D O	D O
	(%)	(mg/l)
Tap	-	12.22
Distilled	88.1	7.46
Distilled/De-aired	56.1	4.89

2001) and can reach values close to 14 mg/l at saturation. One of the key factors to ensure full saturation of the tensiometers is the low air content of the water used (Lourenco, 2008). Therefore, it is essential to reduce the DO of the water used for saturation.

The assessment of the amount of oxygen contained within water is usually performed by examining the water mass when it is under vacuum. By applying negative air pressure close to absolute zero (i.e. -101 kPa), oxygen in the form of air bubbles develops within the water. When the development of the air bubbles stops, the water is considered to be de-aired.

However, in order to more accurately assess the amount of oxygen within water and the performance of the de-aired system, DO was measured using a portable DO meter. These instruments are frequently used in water quality analysis and provide a measurement of both the percentage of DO and the actual amount of oxygen expressed in mg/l.

Three types of water were assessed for their oxygen content, i.e. tap water and distilled water. Measurements of the dissolved oxygen were taken before and after the de-airing process. The results are shown in Table 3.4.

Results show that de-airing of the distilled water is more effective since in this case DO reached a value of 4.89 mg/l, compared to the tap water which after de-airing had 12.22 mg/l

of DO. The achieved DO level of the distilled water can be considered satisfactory compared to the results from [Lourenco \(2008\)](#). Therefore, it was decided to use de-aired - distilled water for the tensiometer saturation. At later stages of the research, de-aired - de-ionized water was used for the saturation process, however the level of oxygen of this type of water was not measured. De-ionised water is de-mineralised and free from ions which contribute to the increment of the osmotic suction component. The use of de-ionised water in this study matrix prevented the development of osmotic suction and therefore suction measurements with tensiometers were not affected by the osmotic suction component.

De-airing was performed in the de-aired water tank shown in Figure [3.15](#). Distilled or de-ionized water was transferred into a large vessel and connected to the de-aired water tank through valve No 6. The vacuum pump was then turned on and negative air pressure was applied inside the tank. Valve No 6 was opened and the water was transferred into the water tank under vacuum. The distilled water was sprayed as it was inserted into the tank using a spraying nozzle in order to increase the effectiveness of the de-airing process. The water tank was left under vacuum until no air bubbles were visible. At that point, water was considered to be de-aired and ready to be used for the saturation of the PPTs.

Negative air pressure was applied to the system through a 180 W vacuum pump with the ability to displace 75 l of free air per minute. The pump had the ability to create absolute pressure conditions close to 0.01 kPa. Before entering the pump, air passed through a pair of desiccators filled with dry silica gel.

To apply positive water pressure to the calibrating rig, an air-water interface system was used.

Negative water pressure was applied within the saturation rig using the vacuum pump. While filled with water, the saturation rig was connected with the pump through valves No 1 and 2. To prevent excess amounts of water going to the pump, valve No 1 was opened for a few seconds and then closed again. Consequently, the space between valves No 1 and 2 was under tension. Valve No 2 was then opened slowly and tension was applied inside the chamber. In this way, water was kept under negative pressure. The actual pressure inside the chamber was measured using the DPI which was connected to the chamber.

3.3.2 Saturation of tensiometers

The reliable measurement of suction from tensiometers strongly depends on the effectiveness of the adopted saturation method. Researchers have applied different methods for saturation of the ceramic porous stone (e.g. [Lourenco, 2008](#); [Mendes, 2011](#); [Min, 2006](#); [Ridley et al., 2003](#); [Take, 2003](#)). [Guan and Fredlund \(1997\)](#) pointed out that the maximum measured suction strictly depends on the saturation process followed and that different procedures are associated with different maximum suction measurement. Most of the procedures described in the literature include the following general steps for the saturation of the porous stone: 1) initial saturation, by subjecting the tensiometer to vacuum close to -100 kPa, 2) submergence of the tensiometer into water while still under vacuum and 3) pressurization with positive water pressure close to the maximum range of the tensiometer.

The importance of the initial saturation of the porous stone was pointed out by [Take and Bolton \(2003b\)](#). The theoretical background was described by the same authors and is based on Boyle's law which links the pressure and volume of a bubble with Henry's law of solubility. The same principle has been applied to the saturation of triaxial specimens through the use of back pressure ([Bishop and Eldin, 1950](#); [Lowe and Johnson, 1960](#)). According to Boyle's law, air within a porous element, with an initial degree of saturation of S_i and at an initial

absolute pressure P_i , will be removed from the porous element and be replaced by water if a water pressure increment δP is applied. In this case, the assumption of a constant volume of the porous element is made. Then, the pressure change δP which is required to change the degree of saturation of the element from S_i to a final S is given by:

$$\delta P = P_i \frac{(S - S_i)(1 - H)}{1 - S(1 - H)} \quad (3.1)$$

where H is Henry's constant, which is approximately 0.02 ml of air per ml of water at room temperature. According to Henry's law, higher pressures will result in additional air being dissolved into the element.

Therefore, the theoretical change in pressure δP_{100} required for full saturation of the porous element ($S = 1$) is given by:

$$\delta P_{100} = 49 \times P_i(1 - S_i) \quad (3.2)$$

Equation 3.2 provides the theoretical background for the reliable saturation of the porous element. If a tensiometer with an initially dry porous stone (i.e. $S_i = 0$) is submerged into the water (at atmospheric conditions, i.e. $P_i \cong 100$ kPa) and pressurized, then the required change in pressure which will ensure full saturation will be approximately equal to 4,900 kPa or 4.9 MPa. However, if the initial pressure is close to absolute zero (e.g. $P_i = -98$ kPa), then the required pressure for full saturation would be approximately 98 kPa, equivalent to a return from vacuum to atmospheric conditions. Therefore, if a tensiometer is able to be submerged into water under vacuum, a return to atmospheric conditions would be sufficient to provide fully saturated conditions.

From Equation 3.2 it becomes clear that if the initial degree of saturation is not equal to zero, the pressure change required to saturate the element will be a fraction of that required

in the case of an initially dry element. Since it is difficult in practise to determine the initial degree of saturation of a porous stone, it is dried out before each initial saturation to ensure that $S_i = 0$.

Based on the above analysis, the procedure followed for the saturation of the tensiometers, before each laboratory test, was the following:

1. Tensiometer was inserted into the saturation rig and fixed using a cable gland.
2. Dry warm air was inserted from valve No 2 while valve No 10 was left open so as to allow for any moisture to be removed. The air was heated slightly in order to prevent damaging the sensor. Tensiometers were left to dry for about 30 minutes.
3. Valve No 10 was closed and the vacuum line was connected to valve No 2, decreasing the air pressure to a value of approximately -99 kPa within the chamber. Vacuum was applied for about 30 minutes.
4. While under vacuum, valve No 9 was opened slowly to allow de-aired water to flow into the saturation rig and flood the tensiometers.
5. The vacuum line was disconnected from valve No 2 which was then opened in order for the pressure within the chamber to return to atmospheric conditions. This process ensured that the pressure increment took place while tensiometers were submerged in the water.
6. Finally, valve No 10 was connected to the air-water interface and positive water pressure was applied to the chamber. The maximum pressure applied was about 80 % of the capacity of the tensiometer. 300 kPa PPTs were saturated at a pressure of 240 kPa, while 700 kPa and 1500 kPa tensiometers were saturated at a pressure of 600 kPa.

Positive water pressure was applied to the tensiometers for two reasons: 1) to ensure that the porous stone became fully saturated and 2) to dissolve any trapped air bubbles within the tensiometer body and the water reservoir. After their saturation, PPTs were kept submerged in water. Their installation into the model was performed with caution in order to prevent cavitation.

Figure 3.16 presents the results from tensiometer testing before and after applying initial saturation. Figure 3.16a shows the performance of the PPTs without any initial saturation cycle applied. In this case, PPTs were saturated only by applying a positive pressure of about 80 % of their capacity. Results show that PPTs Q1414N and Q1414P do not exactly follow the pressure changes applied within the saturation and calibration chamber (actual pressure measured with the external DPI). The rest of the PPTs show a rather good agreement with the changes of pressure, however at low pressure (near -100 kPa) they tend to show a reduction of pressure even though the actual pressure was kept constant. When initial saturation was applied (Figure 3.16b), all the tested PPTs show good agreement with the applied pressure changes.

3.3.3 Calibration of tensiometers

The calibration of the tensiometers took place in the saturation rig, with the tensiometers submerged in de-aired water. Calibration procedure included the correlation between the actual water pressure, measured using the DPI, and the electrical signal (voltage) received by the sensor. Both, positive and negative water pressures were applied inside the saturation rig using the air-water interface and vacuum pump, respectively. The signal which was produced by the sensors was recorded using a data acquisition system and Labview module Signal Express. The increments of pressure during calibration were recorded manually. Therefore, a relationship between the sensor signal and the actual pressure was established.

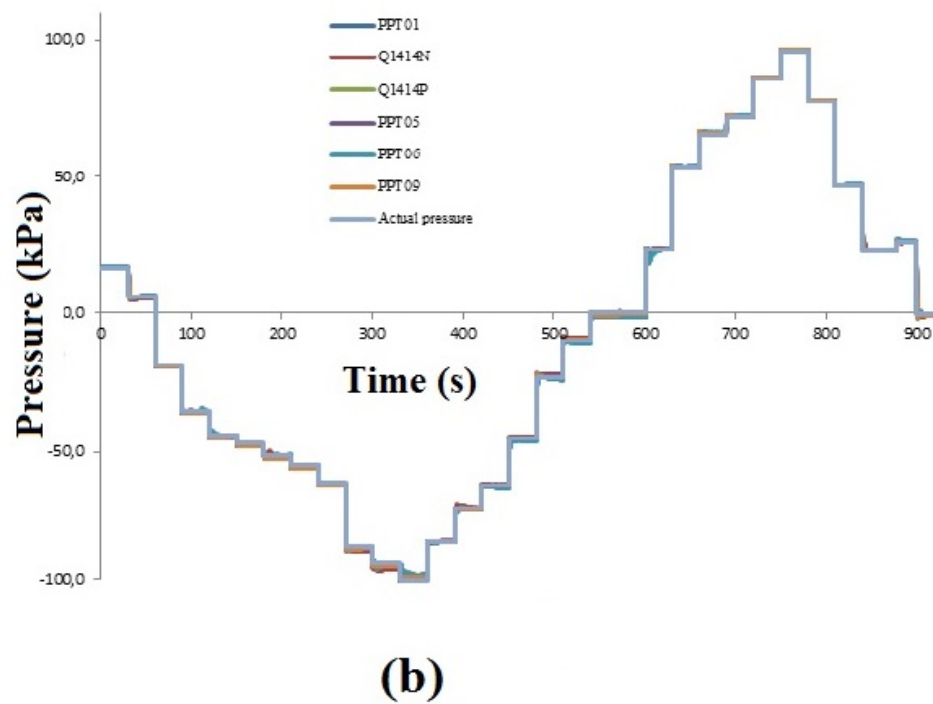
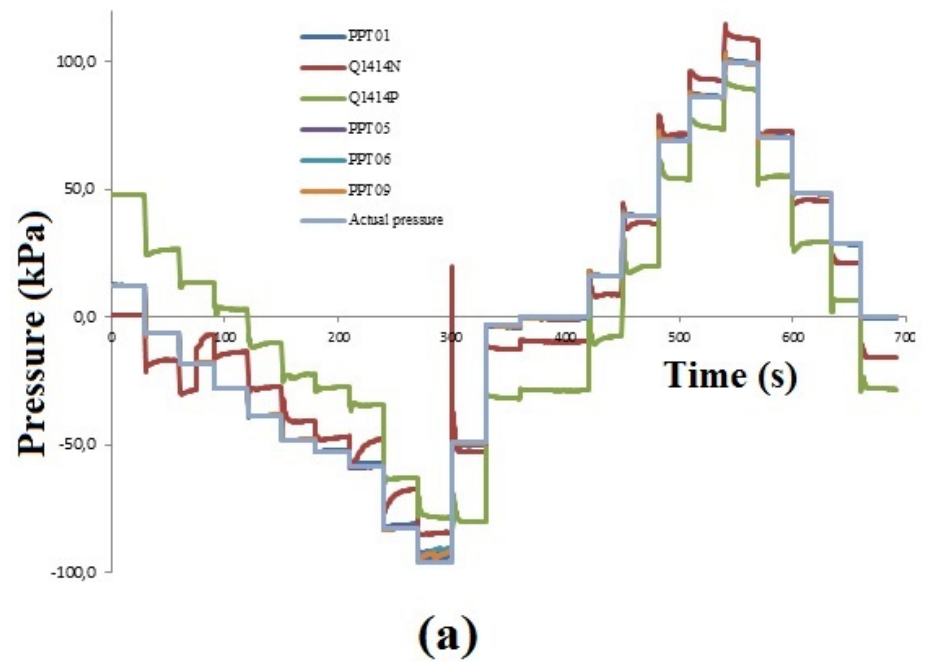


Fig. 3.16 Performance of PPTs a) before initial saturation and b) after 1 initial saturation cycle.

3.3.4 Tensiometer cavitation

Cavitation refers to the process of vapour nucleation in a liquid when the absolute pressure falls below the vapour pressure (Lu and Likos, 2004). In tensiometers, cavitation might occur in the porous stone in the form of an expanded air bubble under negative pore pressure. If cavitation occurs in the tensiometer, then the continuity of the liquid phase between the tensiometer reservoir and the pore water might be lost. Under these conditions, the measurement of soil suction becomes unreliable.

Apart from vapour nucleation in the water, very small air bubbles can also be trapped within crevices of the tensiometer body (Take, 2003). According to this hypothesis, imperfect construction of the tensiometer might produce discontinuities and crevices in the body of the sensor. These discontinuities can trap gas in the form of small bubbles under atmospheric conditions. Under tension, however, these bubbles grow until they reach a critical radius, as described by Young (1989). At that moment, stability of the bubble is lost and its growth becomes uncontrolled, leading to cavitation.

Usually, cavitation is an indication of the maximum suction that the tensiometer can measure. Theoretically, the cavitation pressure coincides with the air entry value of the porous stone which is the air pressure needed to allow air flow through it. However, there are also other factors which control the cavitation pressure of the tensiometer, like the dissolved oxygen in the water, the degree of saturation, and the quality of sealing around the porous stone.

Cavitation can be triggered in an initially saturated tensiometer by exposing it to atmospheric conditions and letting the porous stone dry out by evaporation. With this procedure, the maximum measurement range of the tensiometer can be found. An example of cavitation for

two tensiometers (PPT2 and PPT5) used in the centrifuge tests is shown in Figure 3.17.

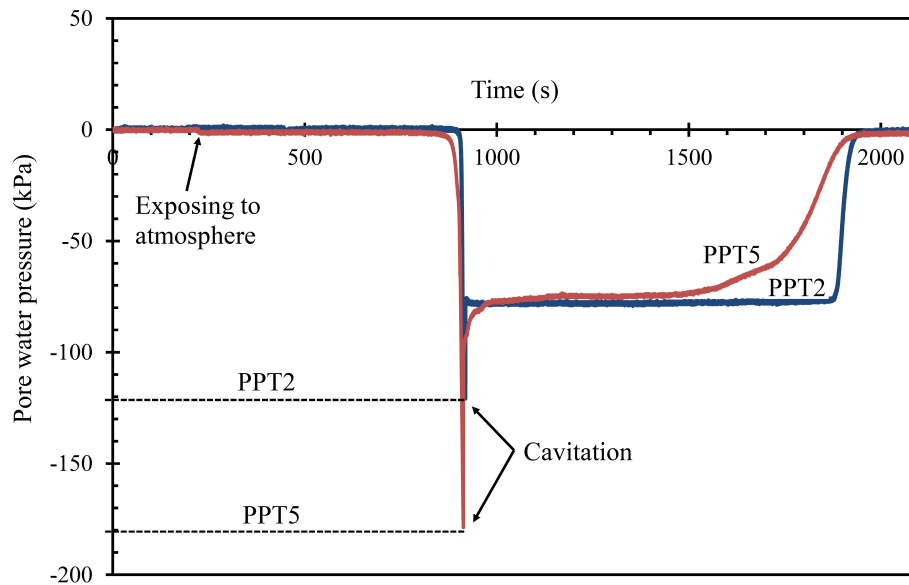


Fig. 3.17 Cavitation of PPTs 2 and 5.

Cavitation of the PPTs occurs in three stages. Initially, when the PPT is left exposed the measured pressure remains constant at about zero. After some time, evaporation of the water from the porous stone creates tension in the water reservoir which is expressed as a sharp decrease of pressure. When the negative pressure becomes higher than the maximum measured suction, which is controlled by the AEV of the porous stone, cavitation potentially takes place and the measurement returns to a small value. As shown in Figure 3.17, the maximum suction level achieved for the two PPTs is different, with PPT5 reaching a pressure of -180 kPa and PPT2 a pressure of -120 kPa. The pressure at which both instruments return after cavitation is the same and approximately equal to -80 kPa. In the last stage, pressure remains constant or almost constant at this value until it returns back to 0 kPa, indicating an almost dry porous stone. Given that the expected suctions of the tested soils do not exceed -30 kPa, the performance of

the PPTs as tensiometers was judged to be satisfactory.

Tensiometer cavitation was proved to be a major issue in centrifuge models when sandy soil was used. Soil models, made of sand, and subjected to centrifugal acceleration tend to desaturate extensively, as it will be presented at Chapter 6 of this thesis. This desaturation causes the porous stone of the tensiometer to lose contact with the pore water and, therefore, to cavitate. Under these conditions, suction can not be measured reliably. The issue was less important when more fine soils were used in centrifuge modelling. Sandy silt soils, that were used at the centrifuge column tests, were proved to retain higher levels of saturation during rotation and, therefore, cavitation did not occur in these soils.

The problems caused by tensiometer cavitation in sandy slope models are discussed in Section 5.1.1.

3.4 Deformation measurements

The assessment of the behaviour of slope models under rainfall conditions requires reliable monitoring of the soil deformation at different stages (e.g. before, during and after rainfall). Therefore, slope models have been prepared so as to have plain strain behaviour which can be observed through the transparent Perspex window. Slope movements in centrifuge models experience $1/N$ times smaller deformations than in the prototype due to scaling. Consequently, measurement of soil movement as small as 0.1 mm in the model are sometimes required for slope stability analysis.

In this study, Particle Image Velocimetry (PIV) was used to measure the deformation field of the slope. PIV is an image analysis technique which was first used in fluid mechanics in

order to track the movement of fluid and determine the flow field velocity (Adrian, 1991). The PIV process tracks marked particles in successive images with the velocity determined by the measurement of the distance covered and the time interval between the images.

White et al. (2003) and Take (2003) developed an appropriate PIV system for soil deformation (geoPIV). Moreover, PIVlab uses the same principles for tracking soil movements while it incorporates a user-friendly environment and useful features which help to improve the analysis. Both applications were used in the current study and are briefly described below.

3.4.1 geoPIV

The geoPIV measurement technique works by processing a sequence of images captured during a test. A monochrome image, in its digitized form, is a matrix of intensities recorded at each pixel (White et al., 2003). Intensity ranges between 0 and 255, with 0 corresponding to black and 255 corresponding to white. The intensity matrix is defined as $I(U)$, where $U = (u, v)$ is the pixel coordinates with u defining the horizontal and v the vertical coordinate (White et al., 2003). Colour images consist of three intensity matrices, one for each colour channel, i.e. red, green and blue.

GeoPIV tracks patches at pixels through a series of images to determine their movement compared to their location in the first image. The first image (image 1 in Figure 3.18) is divided into a grid of test patches with a size of $L \times L$ pixels located at coordinates (u_1, v_1) . Each of the patches is searched for in successive images within a larger patch (search patch). The location at which the highest correlation is found indicates the displaced position of the patch (u_2, v_2) . The location of the correlation peak is established to sub-pixel precision by fitting a bicubic interpolation around the highest integer peak. The operation is repeated for the entire mesh of patches within the image and then repeated for each image within the sequence so as to

produce complete trajectories of each test patch. These trajectories are visualised as vectors on the analysed images.

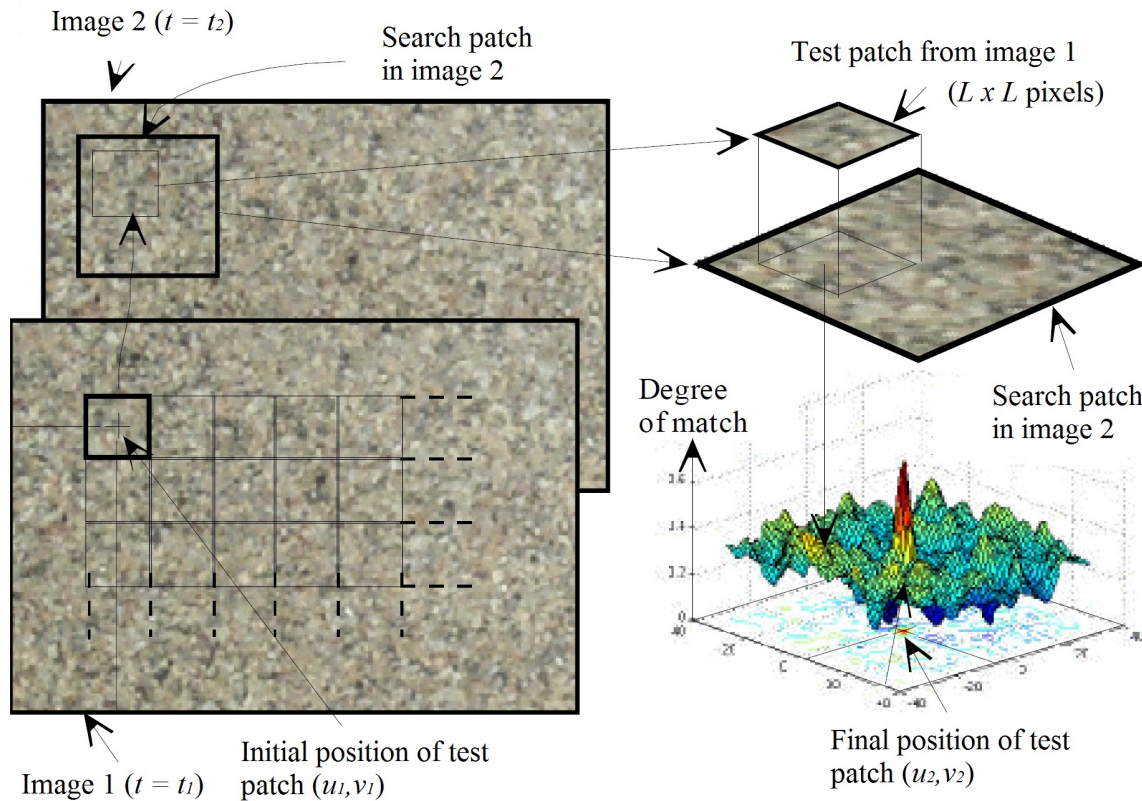


Fig. 3.18 Image manipulation during PIV analysis (White et al., 2003).

Validation experiments made by White et al. (2003) revealed that the precision of PIV analysis is a strong function of the patch size L and can be increased significantly if the soil has a random texture, which ensures very sharp correlation peaks are created. During the analysis of the current study, the size of the patches was determined by trial and error. For each analysis, L was modified so as to produce as few wild vectors as possible.

In this research, Leighton Buzzard (LB) sand, fraction E was used for some centrifuge slope tests. Although the natural sand has some inherent texture, it was important to enhance

it for geoPIV analysis (Mo, 2014). In order to increase the randomness of the soil texture, natural sand was mixed with a small portion (approximately 5 %) of dyed LB fraction E sand. Therefore, the final sandy model had sufficient texture for tracking, as suggested by White and Take (2002). In some of the centrifuge tests, silty sand was used to build the slope model. The texture of the silty sand was satisfactory for the geoPIV analysis mainly due to the higher distribution of particle sizes.

The accuracy of the PIV analysis depends also on the intensity which is related to the different intensities of the pixels comprising each patch. The texture in the captured images can be improved by setting an appropriate exposure time in the digital camera. To determine the exposure time which gives the best results for PIV, an analysis of the intensity distribution of the captured images was performed before the tests. As shown in Figure 3.19a, a specific region (strip) of the image which contains only the soil was chosen for the analysis. For this region, the distribution of the normalized intensity for different exposure times is shown in Figure 3.19b. Normalized intensity ($I_{normalised}$) values between 0 and 1 for each pixel.

The evaluation of the quality of the image and its appropriateness for PIV analysis is based on the intensity range. Each patch is made of a number of pixels which ideally must have distinct gray levels and can be recognised easily by geoPIV. In this case, a broad distribution pattern is created which can be easily tracked in successive images. The ideal case would be a uniform distribution for the whole range of intensities. The exposure time which gives the best results for the PIV analysis was chosen to be 30 ms because it produced the broadest distribution of intensities.

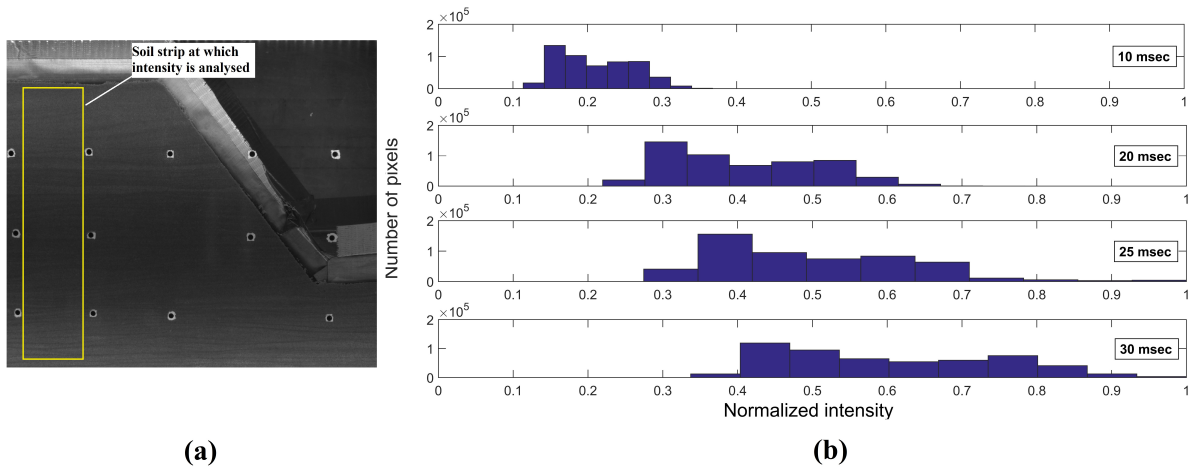


Fig. 3.19 (a) Chosen region for determination of pixel intensity for PIV analysis. (b) Distribution of the normalized intensity for the pixels of the chosen region.

Calibration

The initial results obtained from the analysis with geoPIV include measurements in pixel units, i.e. in the *image-space*. However, for geotechnical engineering applications it is more useful to measure displacements in real units (millimetres) in the *object-space*. This transformation from pixels to millimetres is performed using fixed points drawn on the inner side of the Perspex window. These points, known as reference targets, consist of a circular black dot, with a diameter of 3 mm, surrounded by a slightly bigger rectangle of white paint. The object-space coordinates of these points were obtained before each test with high accuracy using the photogrammetric target shown in Figure 3.20. The principles of close-range photogrammetry were used to develop a robust transformation from u, v to x, y coordinates (White et al., 2003)

Apart from the transition between image and object-space coordinates, calibration also accounts for picture distortion (or fish-eye lens effect). The sources of image distortion causing a deformed field of view are described in detail by White et al. (2003). According to the authors, 14 transformation parameters are needed in order to convert the coordinates and eliminate distortion. Therefore, a minimum number of 14 reference points are needed for the calibration

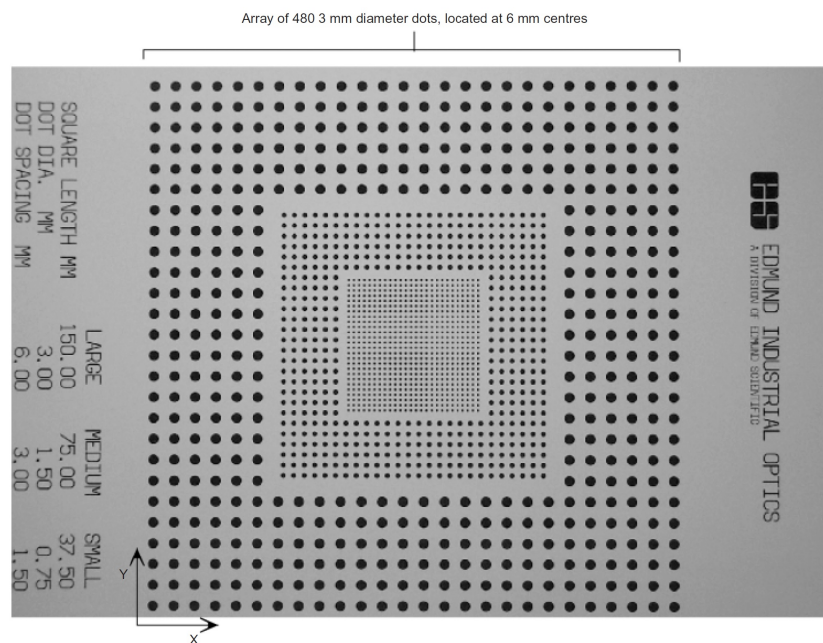


Fig. 3.20 Photogrammetric target for the accurate calibration of the reference points (White et al., 2003).

procedure. Finally, the enhanced gravity field created during centrifuge testing may cause movements of the camera from its initial position.

3.4.2 PIVlab

Another software used for PIV analysis of the soil deformation was PIVlab (Thielicke, 2014). This is a GUI-based open-source tool for digital PIV analyses which is programmed in Matlab. The main features of the PIVlab tool are presented in Figure 3.21. The three main steps for the deformation analysis are image pre-processing, image evaluation and post-processing.

Pre-processing

Pre-processing includes the preparation of the testing images for the analysis. PIVlab uses a number of techniques in order to improve the measurement quality before the actual image

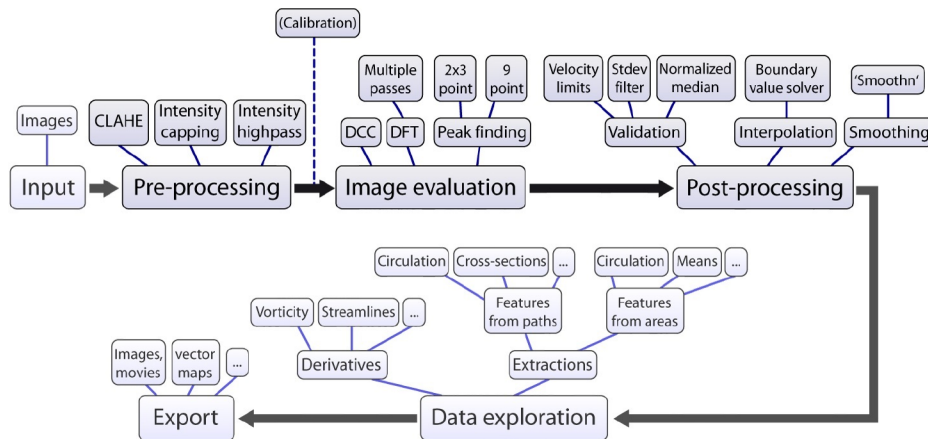


Fig. 3.21 Overview of the workflow and the features implemented in PIVlab (Thielicke and Stamhuis, 2014).

correlation takes place. These techniques allow the adjustment of the contrast and intensity of the images by applying several techniques to provide higher accuracy while producing fewer wild vectors. The techniques which are implemented into PIVlab are the following:

- **Histogram equalization:** with this optimisation technique, the range of intensities at each of the patches is spread out over the whole range of values (between 0 and 255). Potentially, this creates patches with more discrete texture and increases the number of valid (i.e. not wild) vectors by $4.7 \pm 3.2 \%$ (Shavit et al., 2007).
- **Intensity highpass:** this technique applies a high-pass filter that conserves mostly the high frequency information from the particle illumination. Therefore, the effects of inhomogeneous lighting to the low frequency background information is removed.
- **Intensity capping:** an upper limit of the pixel intensity can be selected with this technique. All pixels that exceed this threshold are replaced by this upper limit. This optimisation technique works well in cases of excessive illumination of the soil model which can be caused by soil drying during the spin up of the centrifuge. The importance of this technique is highlighted by the fact that in PIV analyses bright spots within the patches

will contribute statistically more to the correlation signal which may bias the results (Shavit et al., 2007).

Image evaluation

PIVlab uses a discrete cross correlation function which can be solved in two ways: either by using the Direct Cross Correlation (DCC) approach or the Discrete Fourier Transform (DFT) and advanced DFT techniques (Thielicke and Stamhuis, 2014). DCC has been shown to produce more accurate results than the standard DFT technique, however it requires increased computational effort.

Based on experimental evidence, Thielicke (2014) argued that the Fast Fourier Transformation (FFT) *window deformation* algorithm delivers more accurate results with less computational cost. Using this algorithm, data are analysed in several passes of decreasing interrogation area (patch size). In the first pass, the patch size is relatively large, with the results showing a high signal-to-noise ratio. However, the resolution of the analysis (i.e. number of vectors in the image) is kept low. Therefore, subsequent passes are performed using a more fine mesh of patches. Moreover, the displacement data obtained from the first pass are used to offset the patches in the second pass and so on. Following this procedure, a high vector resolution with a high signal-to-noise ratio is produced. Finally, the algorithm takes into account not only the displacement but also the deformation of the patches and incorporates these data to the subsequent passes.

Post-processing and results

The deformation field obtained from the PIVlab analysis can be filtered in order to remove wild vectors. All produced vectors are plotted in a $u - v$ graph where the user can define

acceptable limits depending on the specific application. All vectors which lie outside the limits are discarded and replaced by interpolated ones (Fig. 3.22).

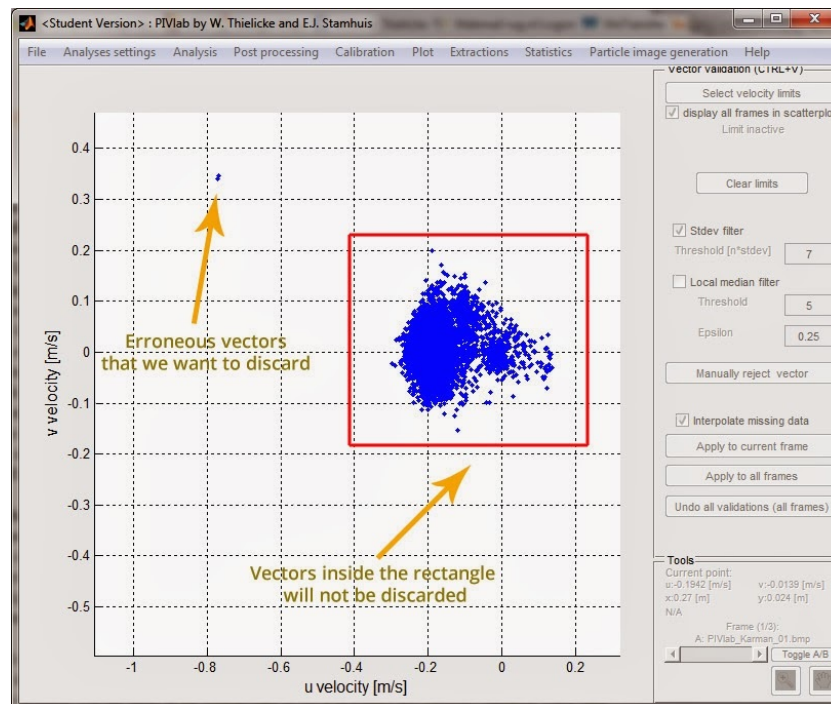


Fig. 3.22 Validation of the obtained vectors and elimination of wild vectors (Thielicke, 2014).

The final displacement field is plotted on the original testing image, as shown in Figure 3.23.

Comparison between PIVlab and geoPIV

The most important drawback of PIVlab, when used in geotechnical applications, is the inability for calibration against image distortion. The distortion of the captured images causes a slight deformation of the analysed model resulting in misleading results. Furthermore, plain strain analysis, based on captured images, requires the minimisation of the lens eye effect before the PIV analysis. To overcome this issue at PIVlab analysis, images were captured using a global shutter instead of a rolling shutter. This feature offers a better precision when analysing frames of fast moving objects due to a reduced apparent distortion.

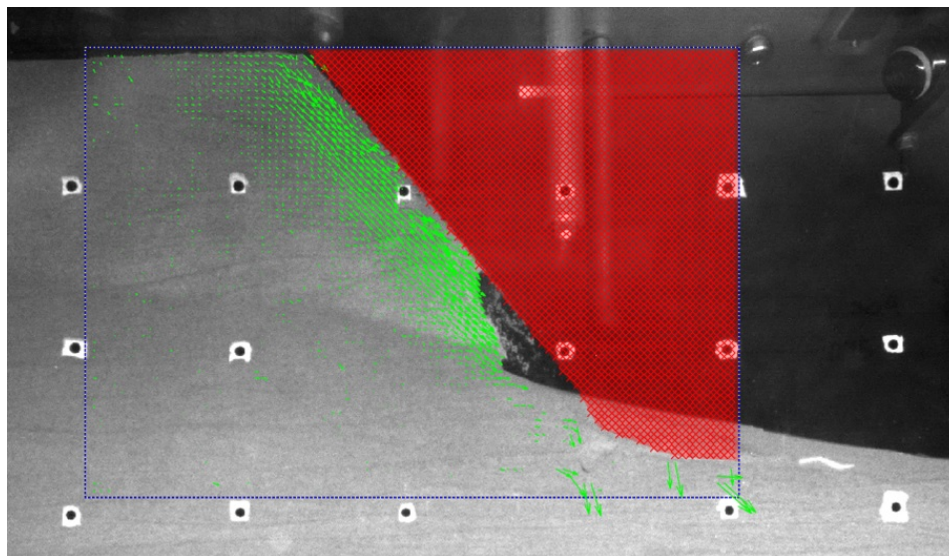


Fig. 3.23 Example of PIVlab analysis of a slope subjected to an elevated groundwater table condition.

Despite the lack of distortion correction, PIVlab offers great pre-processing tools to enhance the quality of the images. These tools proved to offer a very good analysis of the images taken in slope centrifuge tests CSA8.4 and CSA9.4. In these tests, the rising of the water level in slope models was causing a dramatic discolour of the initially bright sand grains. Wet sand was appearing very dark in the captures, which resulted in the loss of the most part of the image texture. The result of this phenomenon was a large number of wild vectors occurring when the analysis was performed with geoPIV. The loss of the image texture was eliminated with the use of PIVlab.

3.5 Summary

For the study of the behaviour of soil slopes under rainfall conditions, a testing apparatus was designed. This consisted of a plane-strain strong box with a Perspex window at the front which allowed the observation of soil deformation at all stages of the test. A rainfall system was designed in order to provide varied rainfall conditions in slopes at g . Moreover, a groundwater

system provided well-defined boundary conditions for the slopes. Digital images of the models can be taken during the tests through the Perspex window. The determination of the deformation field was performed using two PIV techniques: geoPIV and PIVlab.

Pore water pressure measurements involved the use of miniature PPTs which were fitted with porous stones in order to act as tensiometers and measure suction. The reliable measurement of suction requires a sufficient degree of saturation of the porous stones. A saturation and calibration apparatus was designed and used for the saturation of the porous stones and calibration of the sensors before each test. The procedure followed for the efficient saturation of the PPTs was presented. Also, cavitation of the PPTs was discussed and will be related to the determination of the Soil Water Retention Curve of soils in [Chapter 4](#).

Chapter 4

Soil characterisation and slope model preparation

4.1 Introduction

In this chapter, the results from the laboratory tests for the classification of the soils used in this study are presented.

Soil slope models consisted of fine sand (Leighton Buzzard sand, fraction E) and silty sand. Fine sand was selected in order to simulate the behaviour of cohesionless soil slopes under rainfall conditions. The lack of slope failure mechanisms observed in uniform sandy slopes is not consistent with the numerical analysis of similar cases. Therefore, more evidence was necessary in order to understand this behaviour. Moreover, the influence of the seepage velocity in rainfall-induced slope failures was examined in this study. To accomplish this objective, centrifuge model tests in silty sand were performed and compared to the results obtained from the sandy slope testing. The silty sand used in these tests had similar strength properties with the fine sand, however due to a lower hydraulic conductivity caused slower infiltration of the rain water.

The laboratory tests was performed on the following soil types: 1. Leighton Buzzard sand, Fraction E, 2. Congleton sand and 3. Mixtures of Congleton sand and silty sand. The results from the following classification tests are presented in this chapter:

- Scanning Electron Microscope analysis;
- Grain-size distribution;
- Particle density;
- Compaction;
- Saturated permeability;
- Capillary rise;
- Strength properties, including direct shear tests and triaxial tests;
- Soil-Water Retention Curve (SWRC) for the absorption and desorption processes.

The preparation of the soil slope models for centrifuge testing is also presented in this chapter. This includes the initial formation of the soil block, the installation of the PPTs and the final cut of the slope models. Finally, this chapter presents the centrifuge testing program which was adopted in order to accomplish the objectives of the study.

4.2 Soil classification

Classification of the soils used in the current study is presented in this chapter. Leighton Buzzard (LB) fraction E sand is a fine, uniform, silica sand which was used to form slope models tested in the centrifuge (Chapter 5). LB fractions C and E were used for element testing with the soil column apparatus described in Chapter 6. Congleton sand is another type of fine sand used at the element testing with the soil column apparatus (Chapter 6). Mixtures

of Congleton sand and a sandy silt were used in the soil column tests forming soils with varied grain size distribution. The testing program was conducted within the soil mechanics laboratory, in the Department of Civil Engineering at the University of Nottingham. This includes grain-size distribution, compressibility tests and saturated permeability tests. Scanning electron microscopy was also performed to define the grain shape, while spectrum analysis gave an indication about the chemical composition of the grains. The maximum height of the capillary rise in LB fraction E sand was determined as well as the relationship between moisture content and suction (SWRC-Soil Water Retention Curve).

4.2.1 Scanning Electron Microscope

Leighton Buzzard sand - fraction E

A series of LB fraction E sand photos were taken using a Scanning Electron Microscope (SEM) as shown in Figure 4.1. SEM images confirm that LB fraction E sand is a clean, uniform sand with particle sizes ranging between 0.06 mm and 0.2 mm. The shape of the particles can be characterized as sub-angular, based on BS EN ISO 14688-1:2002.

Results from the particle spectrum analysis are presented in Fig. 4.2. Spectrum analysis shows that silicon and oxygen are the two dominant chemical elements of the soil grains. Therefore, the dominant chemical compound in the soil grains is silicon dioxide (SiO_2), also known as silica, the main constituent of quartz mineral. Since silica is the major constituent of LB fraction E sand, it is characterised as *silica sand*.

Congleton sand

In Figure 4.3 an SEM image of Congleton sand is shown. This is a uniformly graded sand with sub-angular and sub-rounded particle shape. Congleton sand is also a silica sand with the grains mainly containing silicon and oxygen.

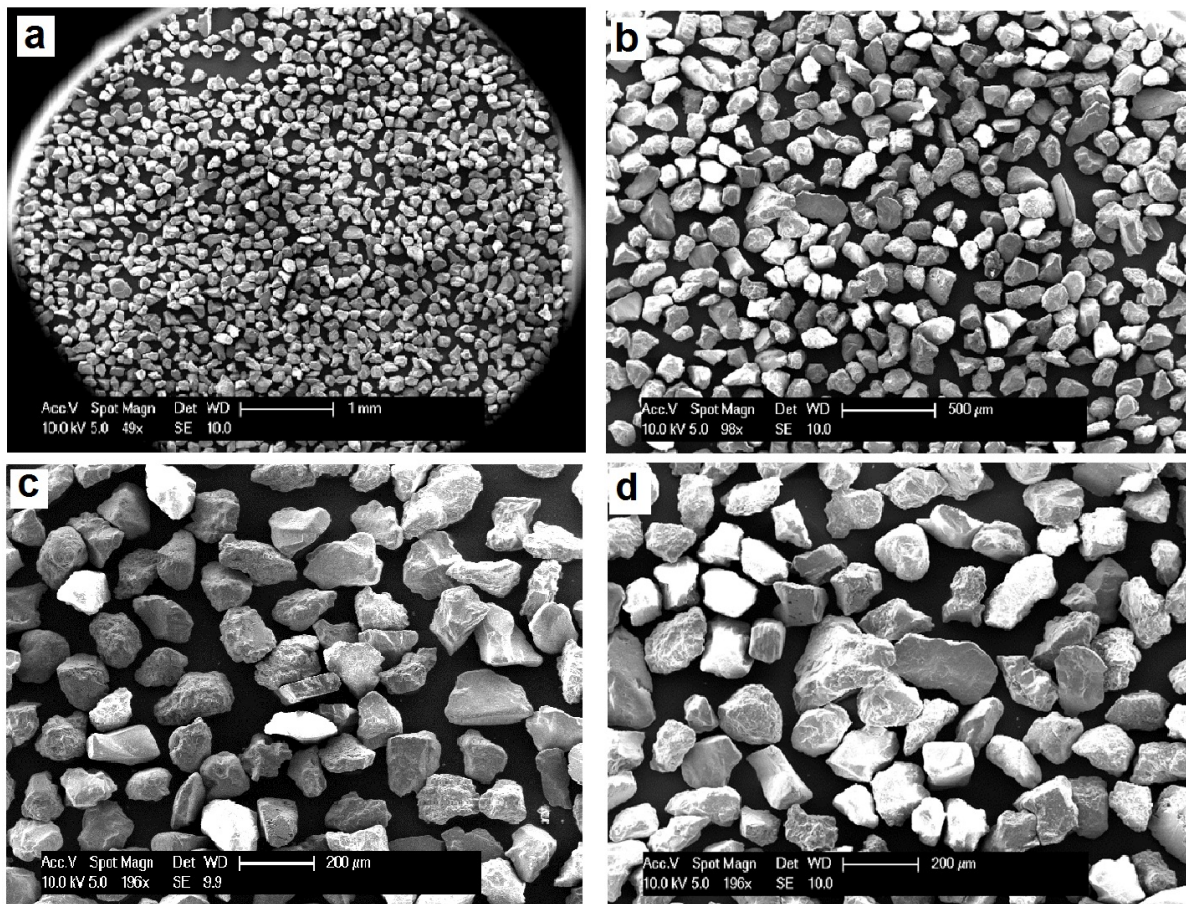


Fig. 4.1 Scanning Electron Microscope images of Leighton Buzzard sand, fraction E. In photos a and b the uniformity of the sand is shown whereas in photos c and d the angular shape of the soil grains is shown

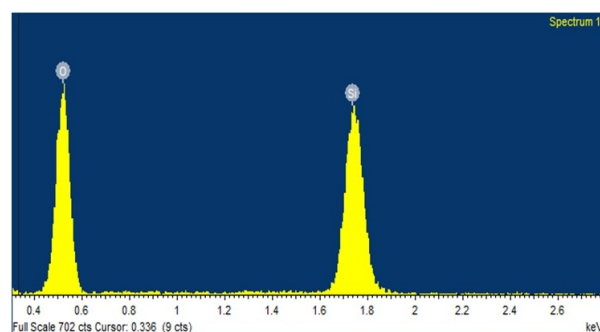


Fig. 4.2 Spectrum analysis of Leighton Buzzard fraction E sand.

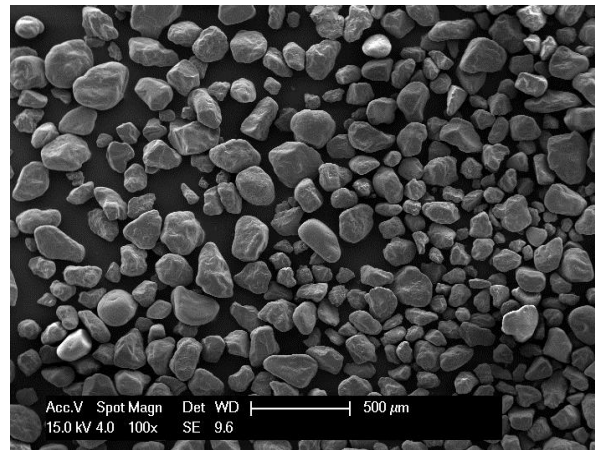


Fig. 4.3 Scanning Electron Microscope images of Congleton sand.

Sandy silt

The SEM images of the sandy silt soil are shown in Figure 4.4. This is a non-uniform soil with a wide range of particle sizes. The dominant elements of the particles can be seen in Figure 4.5 which presents the spectrum analysis results. As can be seen, silica is the dominant compound of the particles revealing that this is, also, a silica soil. Moreover, there are particles which contain other chemical elements, mainly calcium.

4.2.2 Particle size distribution

In this section, the particle size distributions of the soils employed in this study are presented. Dry sieving was conducted for the sandy soils while sedimentation analysis was used to determine the particle size distribution of the finest portion of the silty soil. The sieving procedure is described in *BS1377 : Part2 : 1990 : 9.3*, while the hydrometer analysis, which was used for sedimentation, is described in *BS1377 : Part2 : 1990 : 9.5*.

The sieves used for the dry sieving method had apertures of 0.063, 0.15, 0.212, 0.3, 0.425 and 0.6 mm. Moreover, a receiver was placed under the finest sieve to retain the material

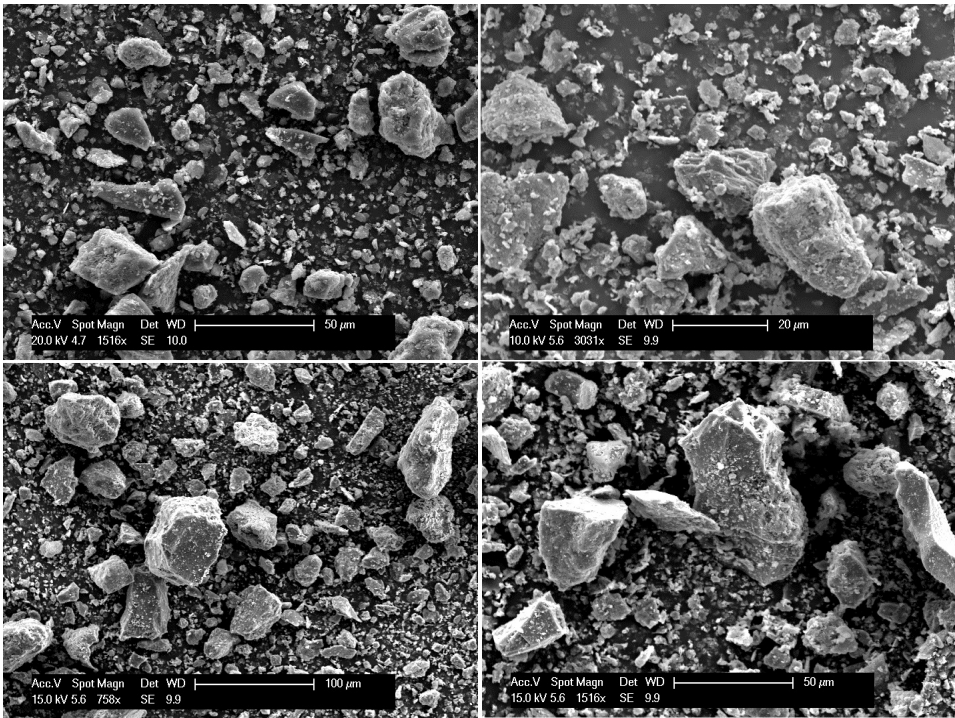


Fig. 4.4 Scanning Electron Microscope images of the sandy silt.

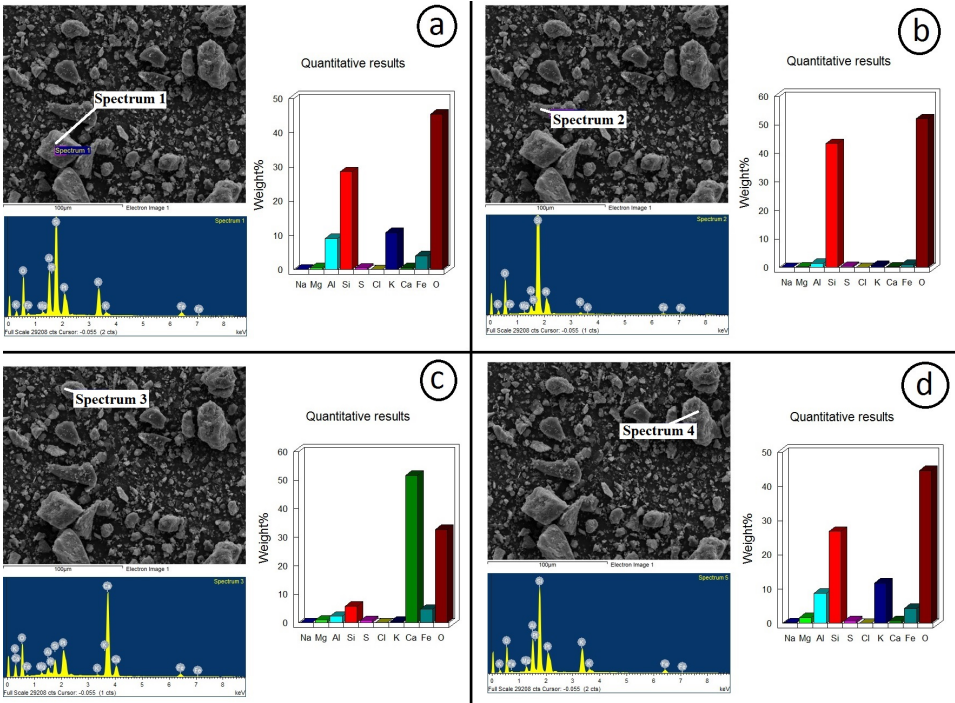


Fig. 4.5 Spectrum analysis for silt. The dominant element in cases a, b and d is Silica while in the case c Calcium is the main element.

passing the finest mesh. The soil material was inserted into the coarser sieve and a lid was placed on the top. Then, the sieves were placed on a mechanical sieve shaker and left for approximately 10 minutes until all the material was distributed to the sieves according to the particle size. In the case of LB fraction E and Congleton sand, no fine material was found to pass the finest sieve, therefore testing was considered completed. For the silty soil, the fine material passing the 0.063 mm sieve was used to perform hydrometer analysis and determine the silty and clayey constituents.

Sedimentation analysis for the fine material is based on Stoke's law, which indicates the proportionality of the velocity of falling particles (with an approximately spherical shape) with their diameter, when suspended in a liquid. Practically, this means that the larger the particles the quicker they will settle down out of the suspension. By determining the viscosity of the suspension at different time intervals, the percentage of the finer than the settled particles can be determined.

The method used in this study for the determination of the grain size distribution of the fine material was the hydrometer analysis. A hydrometer with specifications in accordance to the *BS1377 : Part2 : 1990 : 9.5.2.1* was calibrated and used to define the density of the solution at different time intervals. The suspension was prepared in a glass measuring cylinder with a volume of 1000 ml. At the start of the test, the suspension was shaken up until the particles were uniformly distributed within the solution ($t=0$). At $t=1000$ s, all coarse silty particles had settled down to the bottom of the solution. At $t=30000$ s, all the silty particles had settled down leaving only clayey particles in suspension. The procedure was ended 48 hours after the initial shaking of the suspension. The complete grain size distribution curve was obtained by combining the results from the dry sieving and hydrometer tests. The whole procedure was

repeated two times with the final curve determined using mean values.

Grain size distribution curves for LB-fraction E sand, Congleton sand and silt are shown in Figure 4.6. Table 4.1 presents classification characteristics of the soils, i.e. the effective diameter D_{10} , the mean diameter D_{50} , and D_{60} which indicates the particle size of which the 60 % of the soil is coarser. Moreover, the uniformity coefficient ($U = D_{60}/D_{10}$) is shown. The LB and Congleton sands are uniform sands with very narrow grain-size distribution curves and low coefficient of uniformity. This reveals that these soils consist only of grains with sizes within a narrow size band. On the other hand, the silty soil has a uniformity coefficient of 7.3, as it contains grains with varied sizes, from clay (< 0.002 mm) up to medium sand. Therefore, silty soil is characterised as well-graded.

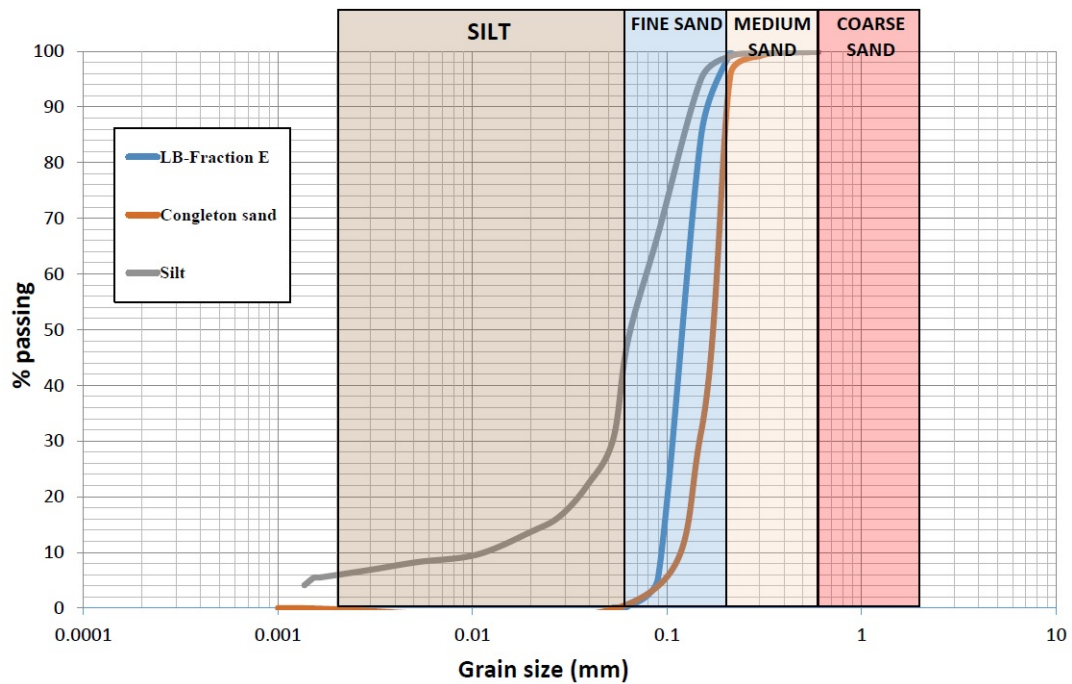


Fig. 4.6 Grain size distribution curves of employed soils. Size ranges for the categories shown (i.e. Silt, fine/medium/coarse Sand) are based on BS1377 : 1990.

Table 4.1 Classification characteristics of employed soils.

	D_{10}	D_{50}	D_{60}	U	Description
	mm	mm	mm	-	
LB-fraction E	0.095	0.12	0.13	1.37	Uniform fine sand
Congleton sand	0.13	0.18	0.18	1.38	Uniform fine sand
Silt (M-Soil)	0.011	0.065	0.08	7.27	Well-graded sandy silt

The grain size distribution curve of the silt reveals that it is comprised of 8 % clay, 40 % silt and 52 % sand. The classification of this soil can be done using the triangular classification chart shown in Figure 4.7. The projection of the portions for each constituent is shown with the red lines drawn on the chart. Consequently, the silty soil is projected to the area indicated as *sandy silt*. For convenience and to avoid confusion with the mixtures of Congleton sand and sandy silt, it will be hereafter referred simply as *silt* or M-Soil.

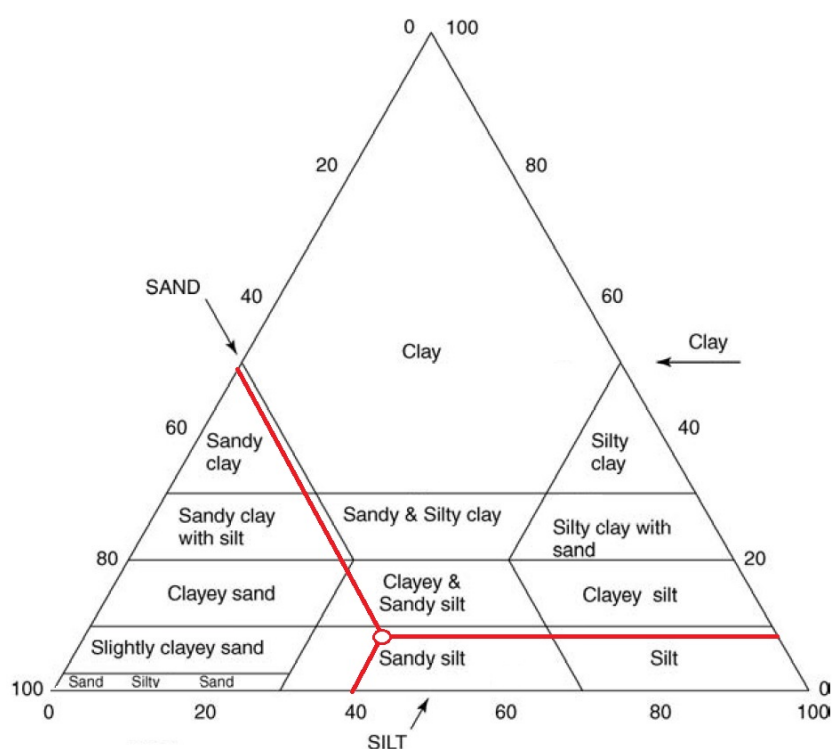


Fig. 4.7 Classification of the M-Soil in the triangular classification chart

As previously mentioned, mixtures of Congleton sand and silt were used for the element testing presented in Chapter 6. These are named as M_{c1-c2} , where c_1 is the percentage of the Congleton sand and c_2 the percentage of the silt. Three soil mixtures were tested, namely M_{90-10} , M_{70-30} and M_{50-50} . Moreover, pure Congleton sand and Silt (M-Soil) were tested. The grain size distribution for these mixtures was determined proportionally, using the grain size distribution of the constituent soils. The obtained curves for the mixtures are shown in Figure 4.8.

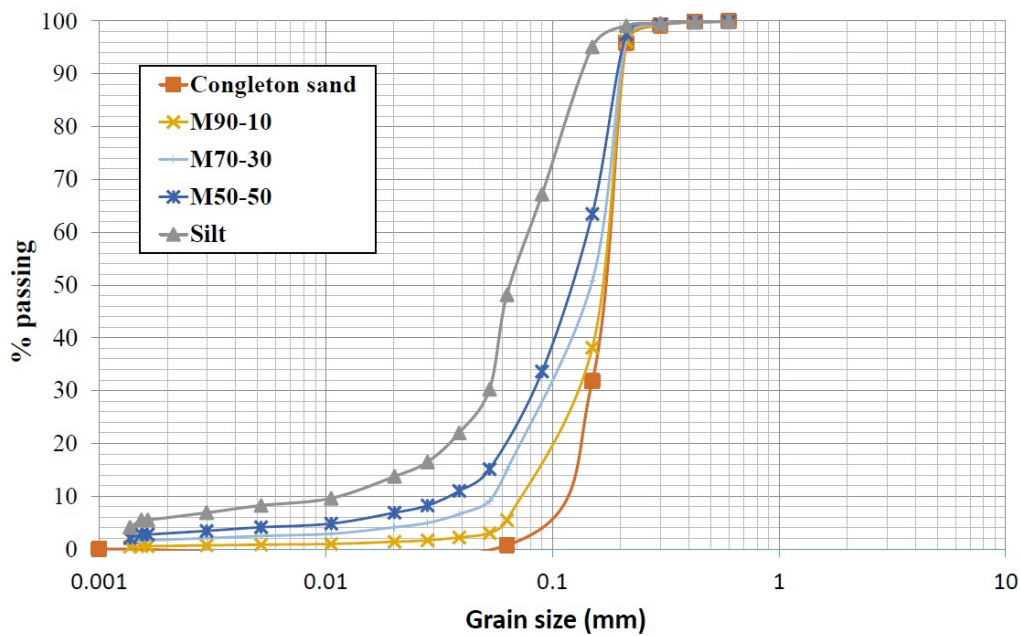


Fig. 4.8 Grain size distributions of the sand-silt mixtures.

4.2.3 Particle density

Particle density (ρ_s) refers to the average density of the grains which make up the soil mass and is expressed in Mg/m^3 . For practical reasons, the density of water used in the calculations throughout this study was taken as 1 Mg/m^3 .

Table 4.2 Particle density of the sand/silt mixtures.

	Congleton sand	M_{90-10}	M_{70-30}	M_{50-50}	M-Soil
ρ_s (Mg/m ³)	2.62	2.62	2.63	2.64	2.65

Particle density for Congleton sand and Leighton Buzzard fraction E sand were determined in the laboratory using the gas jar method (BS1377: Part 2: 1990: 8.2). For Congleton sand it was found to be 2.62 Mg/m³. For LB fraction E sand, particle density was found to be equal to 2.65 Mg/m³. The same value is also referred to in the literature ([Dihoru et al., 2010](#); [Mcdowell et al., 2012](#); [Tan, 1990](#)).

Using the same method (i.e. gas jar), the particle density of the silt was found to be equal to 2.65 Mg/m³. The particle density of soil mixtures consisting on different fractions of Congleton sand and silty sand was obtained using the following relationship ([Ling et al. \(2009b\)](#)):

$$\rho_{s,mix} = \alpha \rho_{s,cong} + (1 - \alpha) \rho_{s,silt} \quad (4.1)$$

where $\rho_{s,mix}$, $\rho_{s,cong}$ and $\rho_{s,silt}$ are the particle densities of the soil mixture, Congleton sand and silty sand, respectively, and α the fraction of Congleton sand in the soil mixture.

The values of particle density for the soil mixtures used in the centrifuge tests in Chapter 6 are presented in Table 4.2.

4.2.4 Atterberg limits

The Liquid Limit (LL) and Plastic Limit (PL) of the M-Soil was determined in the laboratory according to the procedures described in BS 1377: Part 2: 1990 (Clause 4.3 - Liquid Limit using the Cone Penetrometer Test and Clause 5.3 - Plastic Limit). For the LL, the cone penetration method was selected. The preparation of the sample included the removal of the material retained on a 435 μ m sieve under drying sieving. Then, the soil was brought to an initial

gravimetric water content lower than the expected LL and left to mature in a sealed plastic bag for 24 h. Before each test, a controlled amount of water was added to the soil and mixed thoroughly to make a uniform specimen. In this way, all specimens were prepared on a known gravimetric water content.

For the Liquid Limit (LL), five specimens at different water content were tested and the results are shown in Figure 4.9. The Atterberg limits for the M-Soil were found to be: LL = 32.9 % PL = 28.0 % PI = 4.9 %

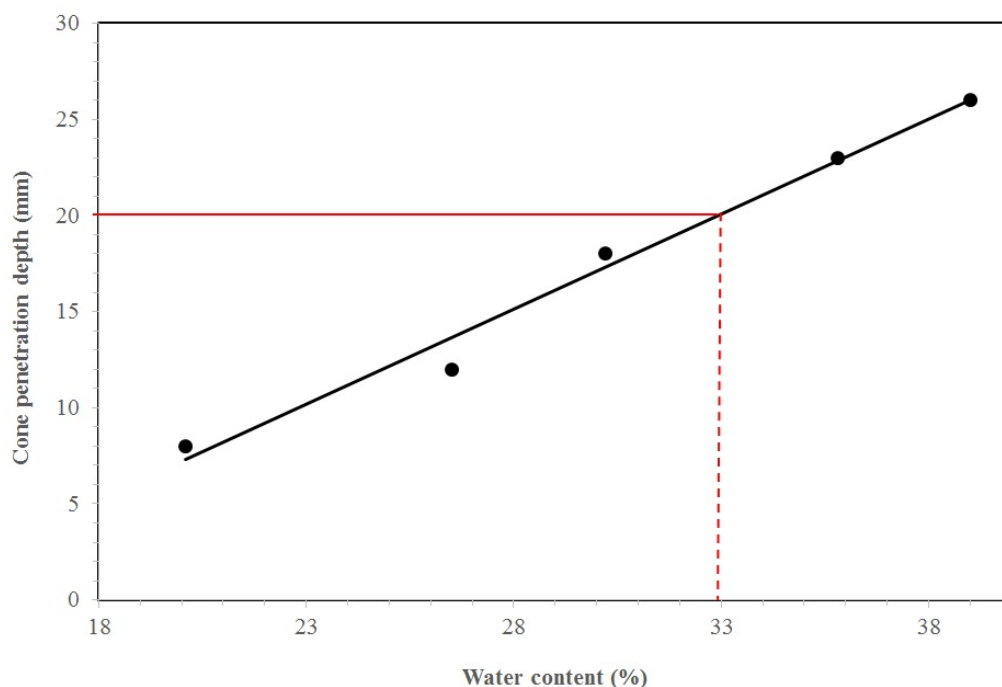


Fig. 4.9 Cone penetration depth - water content graph for the determination of the LL of the M-Soil.

4.2.5 Compaction tests

It is well known that the maximum dry density of a soil depends on its moisture content and also on the compaction effort. For the determination of the maximum dry density and the

optimum moisture content of LB fraction E sand, three different tests were conducted: the Light Compaction Test (LCT) (BS1377: Part 4: 1990: 3.3), the Heavy Compaction Test (HCT) (BS1377: Part 4: 1990: 3.7) and the Vibrating Hammer Test (VHT) (BS1377: Part 4: 1990: 3.7). The work done per unit volume of soil is 596 kJ/m^3 for the LCT and 11739 kJ/m^3 for the VHT (Head and Epps, 2011). This means that the energy applied with the second method is almost twenty times higher than the one used in the LCT. Finally, for the HCT, the work done per unit volume of soil is 2682 kJ/m^3 , almost 4.5 times higher than the LCT.

Results are shown in Fig. 4.10 and include all three methods of compaction. Even if the VHT incorporates higher compaction energy, the achieved density was significantly lower compared to the other two methods. This is attributed to the vibrations produced by the hammer which create a condition similar to the liquefaction of saturated sands, even though the compacted soil is in a partially saturated condition. Therefore, this method was not considered suitable for determining the maximum dry density. The HCT produces soil models with higher density, so it was selected as the appropriate one for determining the maximum dry density of LB fraction E sand and the corresponding optimum moisture content.

As shown in Figure 4.10, the HCT produced a double peak curve which is common for uniformly graded fine sands (Head, 2006). The maximum dry density was found to be $\rho_{d,max}=1.61 \text{ Mg/m}^3$, corresponding to a void ratio $e = 0.65$, and the optimum moisture content was $w_{opt}=11 \%$.

The maximum dry density can also be determined with the vibrating hammer method, in accordance with BS1377:Part 4:1990:4.2. The method is similar to the one described in BS1377:Part 4:1990:3.7 for the determination of the dry density/moisture content relationship, however it applies only to sands with very little or no silt content. In this case, the soil is compacted in three layers under saturated conditions, using the energy enabled by the vibrating hammer. The maximum dry density of LB-fraction E sand was determined to be 1.57 Mg/m^3 and the maximum dry density of Congleton sand was found to be equal to 1.70 Mg/m^3 .

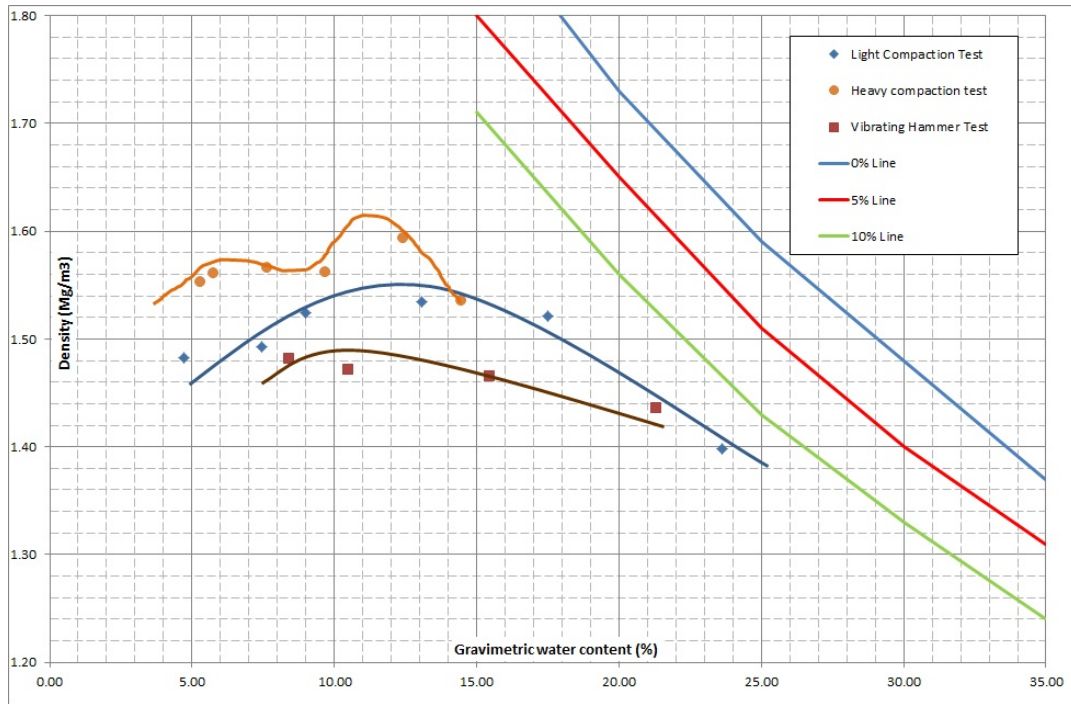


Fig. 4.10 Compaction test results for light, heavy and vibrating hammer methods (Leighton Buzzard, fraction E)

The minimum density was determined in accordance to the BS1377:Part 4:1990:4.4 and was found to be equal to 1.31 Mg/m^3 for the LB-fraction E sand and 1.47 Mg/m^3 for the Congleton sand. The minimum and maximum void ratios can be determined by the equations:

$$\rho_d = \frac{\rho_b}{1 + w} \quad (4.2)$$

$$p_b = \frac{p_s(1 + w)p_w}{1 + e} \quad (4.3)$$

where p_d is the dry density, p_b is the bulk density, w the moisture content, p_s the particle density, p_w the water density ($=1.0 \text{ Mg/m}^3$) and e the void ratio. Considering that in dry conditions

$w=0$ and combining equations 4.2 and 4.3:

$$e = \left(\frac{p_s}{p_d} \right) - 1 \quad (4.4)$$

For the LB-fraction E sand, $e_{min} = 0.69$ and $e_{max} = 1.01$ and for the Congleton sand, $e_{min} = 0.54$ and $e_{max} = 0.78$.

The compaction curve for the mixture M_{90-10} , determined using the light compaction test, is shown in Figure 4.11. The optimum moisture content was found to be 10.5 % and the maximum dry density 1.66 Mg/m^3 .

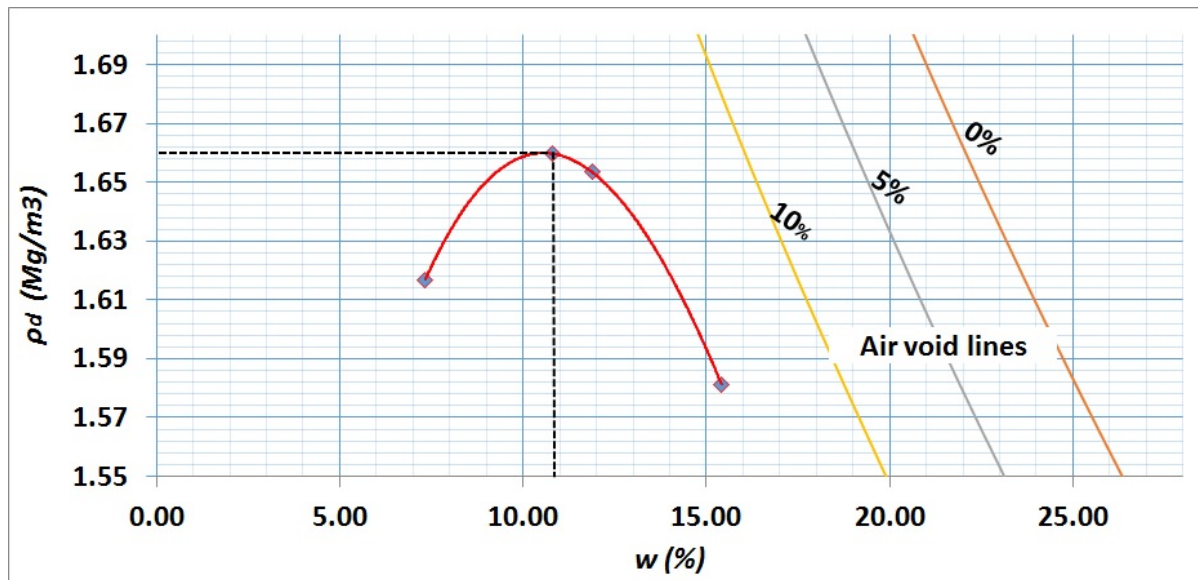


Fig. 4.11 Compaction curve for the M_{90-10} mixture and air void lines

The compaction curve for the mixture M_{70-30} , determined using the light compaction test, is shown in Figure 4.12. The optimum moisture content was found to be 12.5 % and the maximum dry density 1.67 Mg/m^3 .

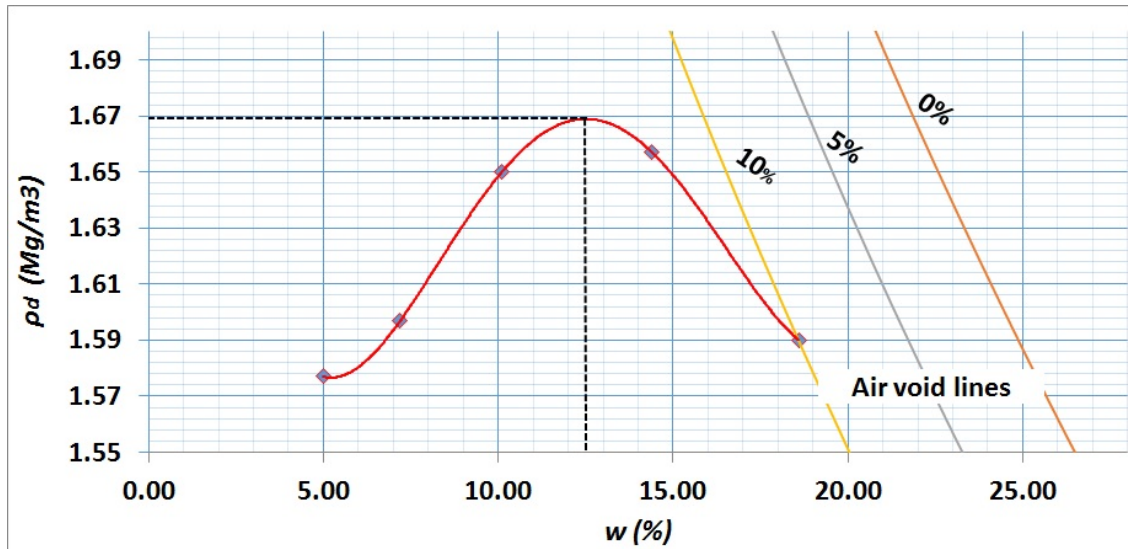


Fig. 4.12 Compaction curve for the M_{70-30} mixture and air void lines

Air void lines, added to the compaction curves in Figures 4.10 and 4.11, represent the dry density - moisture content relation for soil containing a constant percentage of air voids. This relationship is given by the following equation (Head, 2006):

$$\rho_d = \frac{(1 - \frac{V_a}{100})\rho_w}{\frac{\rho_w}{\rho_s} + \frac{w}{100}} \quad (4.5)$$

where ρ_d the dry density, V_a the volume of air voids expressed as a percentage of the total volume, ρ_w the water density, ρ_s the particle density and w the water content (expressed as percentage). Zero air void lines represent the fully saturated condition and are also called the saturation lines.

4.2.6 Saturated permeability

Saturated permeability is an important soil parameter for slope stability as it controls the seepage velocity during infiltration. The permeability of a soil is expressed by the hydraulic conductivity k which represents the velocity of water through a soil element with specific dimensions and hydraulic gradient. There are various methods to determine the hydraulic

conductivity of soils in the laboratory, depending on, primarily, the grain size distribution of the soil. The constant head method is used for the measurement of permeability in sands and gravels containing little or no silt and is described in *BS1377 : Part5 : 1990 : 5*. For finer soils, the falling head method is applied.

The saturated permeability of all soils used in this study were determined in the laboratory using the above mentioned methods. Fine sands (i.e. LB-E and Congleton) were tested in a large permeameter cell of 114 mm diameter and 460 mm length. The permeameter cell was modified appropriately in order to prevent washing of the fine particles away. For this reason, fine wire gauze was placed into the piezometer glands thus preventing the sand from entering. Also, two wire gauze discs with the same diameter as the permeameter cell and an aperture of 0.5 mm were placed at the top and bottom of the soil sample.

Figure 4.13 presents the results from the measurement of the saturated permeability for LB fraction E sand at different densities. As it was expected, permeability reduces with decreased void ratio. A general rule states that the relationship between void ratio e and the logarithm of saturated coefficient of permeability $\log k$ is linear, following experimental evidence (Huang et al., 1998; Lambe and Whitman, 1969; Wang and Sassa, 2003). In the case of fraction E sand, this is given by the following equation:

$$k = 0.0005 \times e^{7.3173} \quad (4.6)$$

The coefficient of saturated permeability for Congleton sand was determined at $\rho_d = 1.59 \text{ Mg/m}^3$ and $e = 0.65$, similar to the dry density of the soil at column tests (Section 6.4). Under these conditions, the saturated permeability of Congleton sand was determined to be equal to $7.51 \times 10^{-5} \text{ m/s}$.

Furthermore, the coefficient of saturated permeability of the sand/silt mixtures and the M-Soil was determined using the falling head method. This method was used due to the small grain size of the mixtures and the low drainage which was expected. The procedure followed

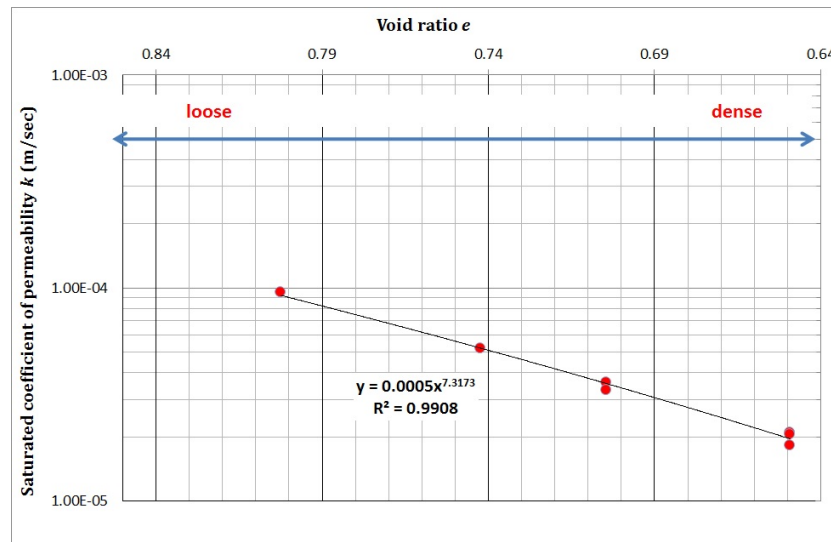


Fig. 4.13 Saturated coefficient of permeability versus void ratio for LB fraction E sand.

is not described either in BS or ASTM standards but it stands as a common practice for the testing of the hydraulic conductivity for fine soils (Head, 2006). The sand/silt mixtures tested using the falling head method were M_{90-10} , M_{70-30} and M_{50-50} .

For these series of tests, the conventional permeameter cell was replaced with a modified permeability cell (a BS compaction mould shown in Fig. 4.14). It consists of a compaction mould with end fittings similar to those used in the conventional falling head permeability tests. These consists of a perforated base and a top cap with an inlet tube which are secured to the mould and extension collar by tie-rods and wing nuts of the correct length. With this apparatus, soil is able to be compacted to the desired density using the Proctor test equipment inside the mould. Then, the sample is trimmed and the extension collar is fitted. At the top and bottom of the soil sample, a porous stone and wire gauge are inserted with the latter be in contact with the soil. In this way, the fine particles of the soil are not washed away. Steel wool is inserted in the space between the top cap and the soil to reduce the impact force of the water when inserted into the cell to fill it up. The base is perforated and a trench collects the water and removes it from the cell through an extraction valve.

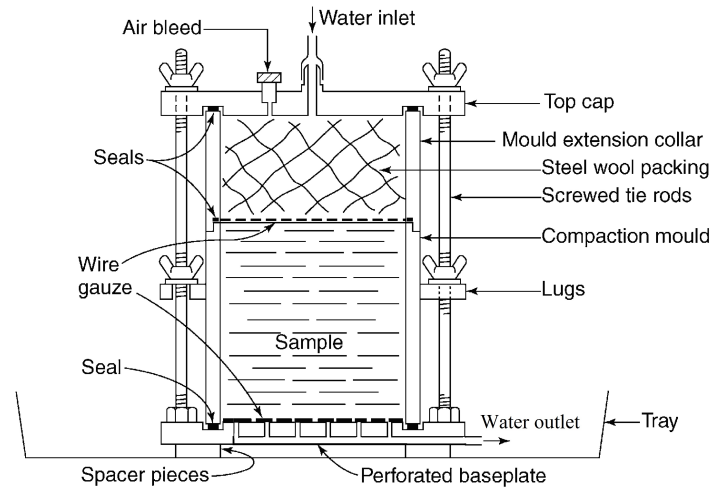


Fig. 4.14 Arrangement for falling head permeability test in the BS compaction mould (Head, 2006).

The results from the laboratory testing for all soils used in the current study, as well as the density at which each test was performed, are shown in Table 4.3.

4.2.7 Capillary rise tests

Capillary rise tests were performed on fraction E sand in order to determine the maximum height of the capillary rise and its dependency on soil density.

Capillary height strongly depends on the pore radius of the soil. Therefore, it depends on the grain size distribution but also on the packing density, i.e. void ratio. The maximum height of the water in a capillary tube is given by the following equation (Fredlund and Rahardjo, 1993):

$$h_c = \frac{2T_s}{\rho_w g R_s} \quad (4.7)$$

where h_c is the maximum height of the water, T_s is the surface tension of the water within the tube acting at an angle a from the vertical, ρ_w the water density and $R_s = r/\cos a$ is the radius of curvature of the meniscus formed in the tube with a diameter of $2a$ (Fig. 4.15).

Table 4.3 Coefficient of saturated permeability k for the soils used in this study at specific density

	Coefficient of permeability, k	Void ratio, e
	m/s	-
LB-E	5×10^{-5}	-
Congleton	7.5×10^{-5}	0.65
M_{90-10}	6×10^{-6}	0.56
M_{70-30}	3×10^{-6}	0.66
M_{50-50}	1×10^{-7}	0.62
M-Soil	8×10^{-8}	0.74

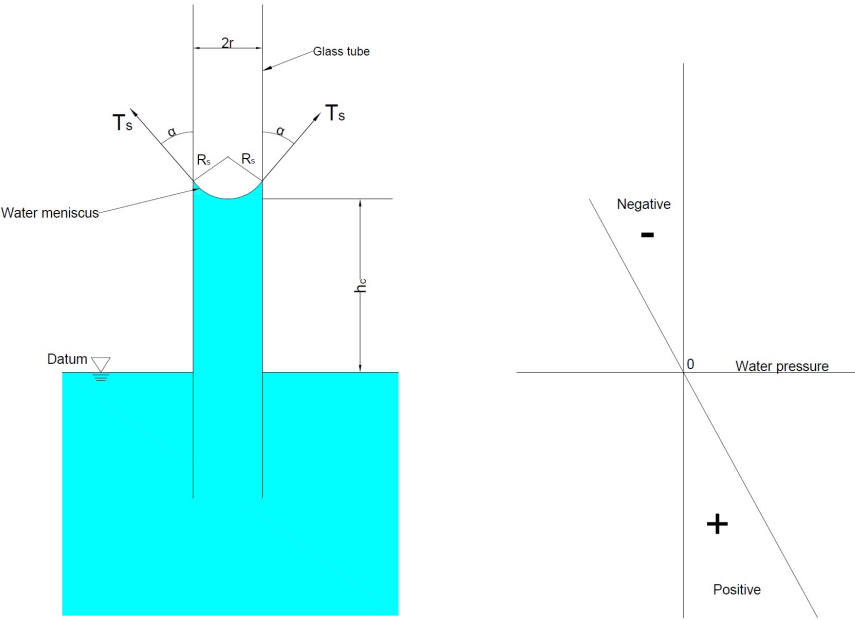


Fig. 4.15 Capillarity model.

Water pressure below the datum is positive due to hydrostatic pressure conditions. Above datum, water is held under tension and water pressure is negative following a "reverse" hydrostatic distribution (Fig. 4.15). This means that in the ideal case of water rise within a tube, water pressure at a height h above the datum will have an equivalent but negative value of water pressure at a point located at a depth h below datum.

If we impose values of water density as 1 g/cm^3 , gravitational acceleration $g = 980 \text{ cm/s}^2$, surface tension of the water as $T_s = 72 \text{ mN/m}$ at 25° and a zero contact angle, then Equation 4.7 becomes:

$$h_c = \frac{0.03}{r} \quad (4.8)$$

with h_c and r measured in mm.

The experimental setup comprised of a Perspex tube, 725 mm in height and 31 mm in diameter, which contained a soil column at a known density. The soil model was prepared using the dry pluviation method (described in Section 4.3.1). Preparation of the model using dry pluviation allowed for controlled soil density. The tube containing the dry soil mass was placed within a vessel full of water and the bottom 55 mm of the soil was kept submerged. This created a water level condition within the soil mass that was kept constant due to an overflow at the edges (Fig. 4.16). The soil column remained submerged in the water for 48 hours to allow time for the soil water to rise due to tension. After 48 hours, the system was in equilibrium (i.e. water front remained stable and capillary rise had reached its maximum level).

The results for two different dry densities can be seen in Figure 4.16. For the loose sand, with void ratio $e = 0.82$, the maximum capillary height was determined to be 428 mm. This was increased to 497 mm for the dense sand ($e = 0.65$) confirming the strong dependency of h_c

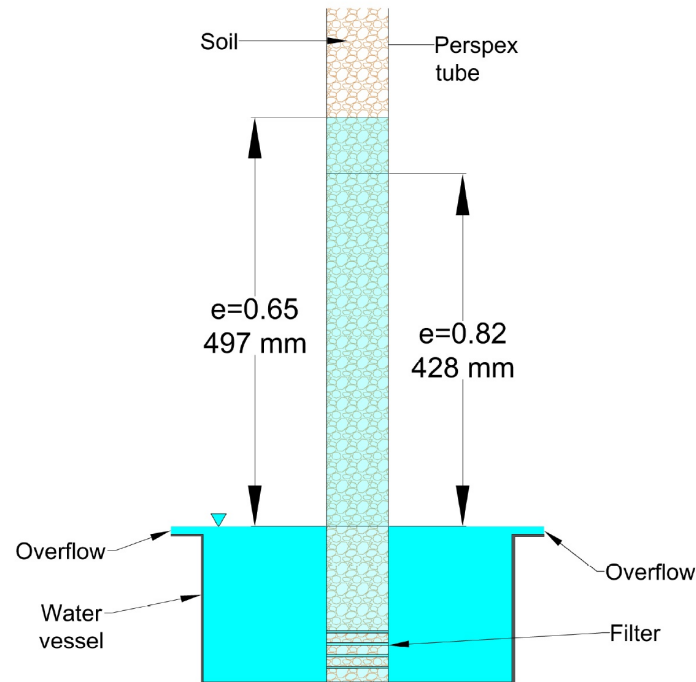


Fig. 4.16 Capillary rise tests in Leighton Buzzard sand - fraction E at equilibrium.

on void ratio. This relationship is described by the empirical equation introduced by [Peck et al. \(1974\)](#) linking capillary height with void ratio and the effective diameter of the soil (D_{10}):

$$h_c = \frac{C}{eD_{10}} \quad (4.9)$$

where h_c and D_{10} are in units of millimetres and C a constant which depends on the surface impurities and grain shape and ranges between 10 and 50 mm^2 .

Based on Equation 4.9, h_c for the dense state range between 128 mm and 642 mm for the loose state and between 162 mm and 810 mm. This is in accordance with the results obtained experimentally. Considering the measured capillary height, constant C for Leighton Buzzard sand (fraction E) should be considered to be equal to 32 mm^2 in order to match the experimental and theoretical values.

4.2.8 Soil Water Retention Curve (SWRC)

The relationship between the moisture content and suction in a soil is known as the Soil's Water Retention Curve (Section 2.2.4). This is an important empirical relationship in unsaturated soil mechanics that describes the thermodynamic potential of the soil pore water relative to that of free water as a function of the amount of the water adsorbed by the soil system (Lu and Likos, 2004). Much of soil's behaviour during centrifuge tests on soil slopes is explained by combining its SWRC with the test's parameters. For example, gravity increase causes desaturation of the soil mass which, in turn, causes soil suction to be increased. This desorption process which is best described with the drying branch of the SWRC. On the other hand, wetting of the soil during rainfall and the subsequent increase of water content is an adsorption process and is explained from the wetting branch of the SWRC. The wetting and drying branches of the SWRC do not coincide; they show hysteresis. This implies that more water is generally retained during the drying process than during the wetting process.

Typical drying and wetting retention curves of a soil, in terms of volumetric water content θ , are shown in Figure 4.17. When a soil dries out from a saturated state (i.e. $\theta = \theta_s$) it follows the primary drying curve up to the residual water content ($\theta = \theta_r$). On the reverse procedure it will follow the primary wetting curve. These two curves form an envelope which includes all the scanning curves obtained under different circumstances, i.e. initial $\theta \neq \theta_s$ for the drying curve or initial $\theta \neq \theta_r$ for the wetting curve. In both cases, the obtained curves will be flatter than the primary ones until the primary curves are met (Toll, 2012). Finally, the wetting curve rarely ends up at the saturated water content, either because of entrapped air bubbles during the wetting process or due to irreversible shrinkage behaviour of the soil during drying (Toll, 2012).

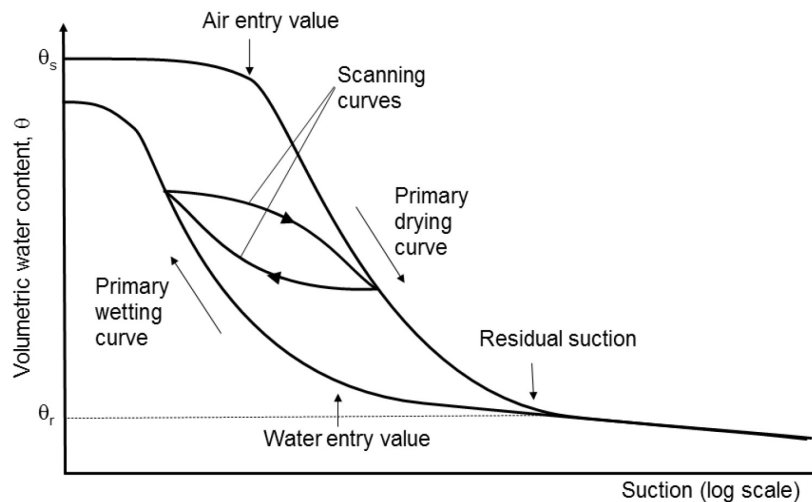


Fig. 4.17 Typical Soil Water Retention Curve (Toll, 2012).

Drying and wetting curves for two different fractions of LB sand have been determined by Kasangaki (2012) with the pressure plate method, as shown in Figure 4.18a. The grain size distribution of the two fractions, LBS1 and LBS2, is shown in Figure 4.18b. Finer soils experience higher suctions for the same saturation level (Brooks and Corey, 1964; Fredlund and Xing, 1994b). As expected, the drying SWRC of fraction LBS1 is below the drying curve of LBS2 since LBS1 is coarser than LBS2. Furthermore, the air entry value in LBS1 is lower than that in LBS2. The residual value of moisture content in both soils is close to 1%. When desaturation of the soil enters the residual zone, suction becomes significantly higher and reaches values close to 300kPa. This reveals the higher energy which is required to remove further amounts of water from the soil. Under this condition, water is discontinuous within the soil mass and exists only at localised locations at interparticle contacts.

Tensiometer method

The use of the tensiometer technique for the determination of the SWRC in the laboratory has been applied in the past by Cunningham (2000), Boso et al. (2005), Toker et al. (2004), Lourenço et al. (2007), Lourenço et al. (2011), Lourenço et al. (2015) and Toll et al. (2015). In

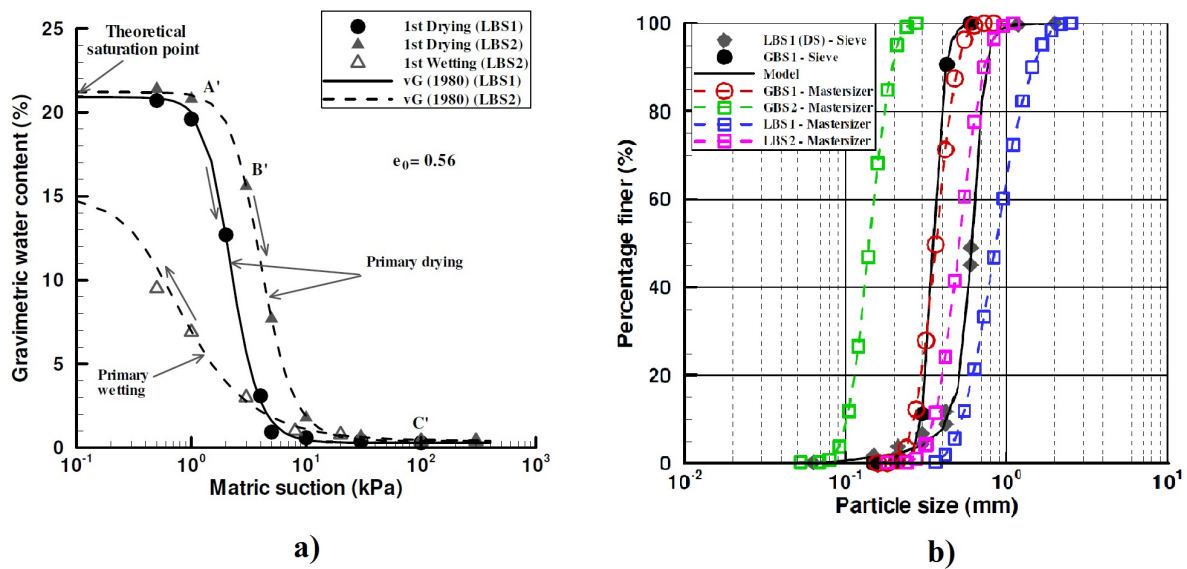


Fig. 4.18 a) Primary drying and wetting SWRCs for two different fractions of LB sand, determined with the pressure plate method. "vG" are the van Genuchten fitting curves to the experimental data. b) Grain-size distribution of LBS1 and LBS2 (Kasangaki, 2012).

general, this technique involves the drying of the soil by exposing it to atmospheric conditions while the induced suction is measured by a PPT within the soil mass. The major advantage of this technique is that soil drying can be imposed naturally, where negative pore water pressures are created (Toll et al., 2013). In contrast with the axis translation technique, where desaturation occurs due to artificially elevated air pressure and air intrusion at the boundaries of the sample, in the tensiometer technique the internal pores desaturate by cavitation when the pore water pressure becomes highly negative. Therefore, the soil drying process and suction-induced replicate the processes occurring in nature. Moreover, the tensiometer technique provides a quick, reliable and inexpensive way to determine the SWRC of a soil in the laboratory, compared with other methods.

The tensiometer technique for the determination of the SWRC has been compared to other methods by Toll et al. (2015). The results are shown in Figure 4.19. At low suctions (up to 400 kPa) the tensiometer technique seems to provide results in a good agreement with the

other methods. Suction values obtained from the pressure plate method are always higher than the values from the tensiometer method. This is attributed to the fact that cavitation is been prevented in the axis translation technique whereas it can take place during natural drying when using the tensiometer method [Toll et al. \(2013\)](#).

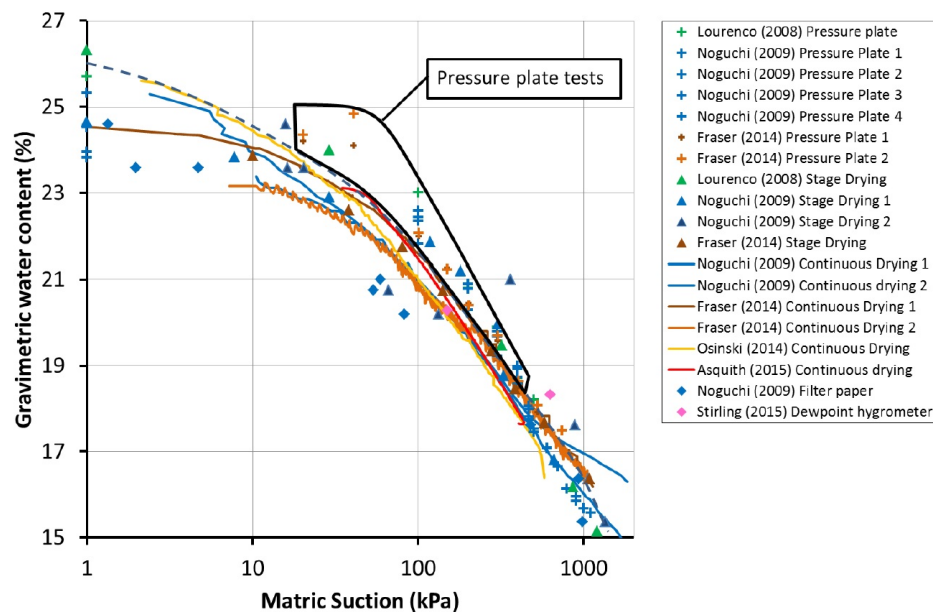


Fig. 4.19 Results from the SWRC determination of a sandy clay using various testing methods ([Toll et al., 2015](#)).

Two different procedures can be followed for the determination of the SWRC using tensiometer technique: the stages and the continuous methods. In the stages procedure, soil is allowed to dry exposed to the atmosphere until a specific water content is reached. Then it is sealed and allowed to equalise internally. After it has reached a homogeneous distribution of moisture, suction is measured using the tensiometer reading. Therefore, a single point of the SWRC is produced. Using this procedure, the moisture content determined by recording the changes of the soil mass corresponds to the bulk water content of the soil sample. However, from a practical point of view, this method might not produce very accurate results due to the potential disturbance of the soil sample when it is moved from the sealing chamber to the

pedestal. On the other hand, the continuous approach is based on the continuous evaporation of the pore water when the sample is exposed to the atmosphere. Changes of the total mass, which indicate the loss of pore water and the reduction of the gravimetric water content, and suction are constantly recorded and, therefore, a continuous SWRC can be produced. However, faster evaporation near the boundaries of the sample leads to a non-uniform distribution of moisture in the soil specimen. Consequently, the measured suction may not match with the moisture content determined by the mass changes.

Testing equipment and specimen preparation

Both drying and wetting SWRCs were determined in the laboratory using a specially designed pedestal (Fig. 4.20). The experimental setup comprises an aluminium base, on which the soil sample was placed, with a fitted PPT for measuring pore suctions. The PPT was kept in place using a cable gland which also provides insulation of the space occupied by the PPT cable. In order to ensure good contact between the soil sample and the aluminium base, rubber was placed at the margins of the soil sample. Also, an o-ring was fitted between the PPT and the aluminium base so as to prevent any exposure of the PPT head (porous stone) to the atmosphere.

Prior to testing, the PPT was saturated (as described in Section 3.3.2) and calibrated (Section 3.3.3). The PPT was carefully fitted to the pedestal and its porous stone was kept wet at all times with a wet piece of cotton in order to prevent cavitation. When the soil specimen was ready to be placed on the pedestal, cotton was removed and soil specimen was pushed slightly on the PPT to ensure a good contact. The insertion depth was selected to be between 3 and 5 mm so as to achieve a good contact and prevent soil disturbance or even cracking of the specimen.

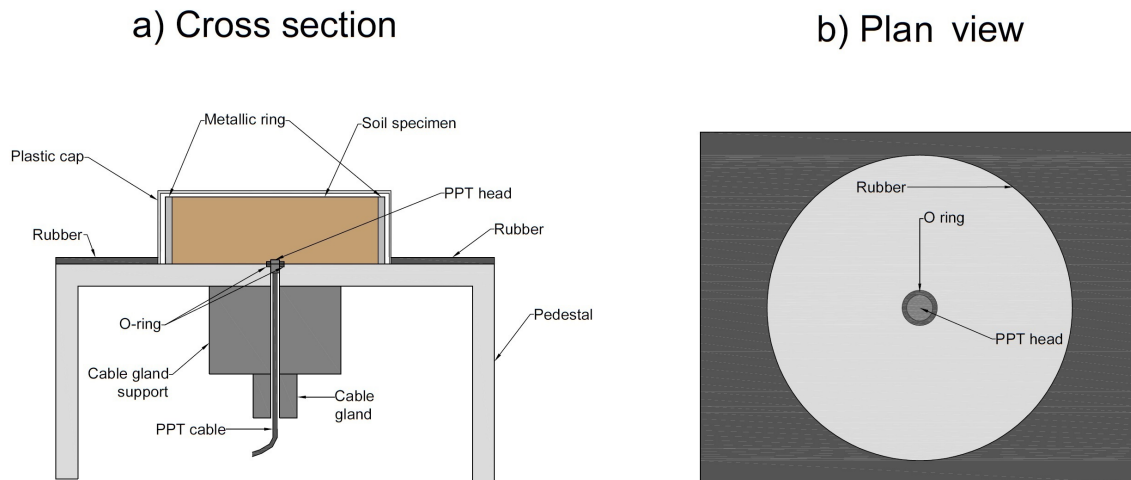


Fig. 4.20 Pedestal used for the development of the SWRC. Plastic cap, metallic ring and soil specimen are not shown in (b) for simplicity.

The preparation of the soil specimens for testing consisted of the following steps: initially, soil was mixed with water in order to achieve the desired moisture content and then left sealed in a bag for 24 hours in order to equalise its moisture. Soil was then compacted in a mould using the standard Proctor test (BS1377: Part 4: 1990: 3.3) in order to achieve the desired density. A soil block was then extruded from the mould and cylindrical specimens of 75 mm in diameter and 18.5 mm in height were taken using a trimmer. Soil specimens were kept in sealed vessels until the testing time in order to preserve their moisture. Just before the test, a small sample was taken from the specimen and dried in an oven (at 105°) in order to determine the initial gravimetric water content.

The final specimen was weighed and then placed on the circular base with the lower face touching the PPT. The specimen was slightly pushed onto the base to ensure a good contact between the transducer and the soil. The whole setup (pedestal, soil and PPT) was then placed on a balance with a precision of 0.01 g and was covered with a plastic cap (Fig. 4.21). By plotting the changes of the gravimetric water content w with suction, the SWRC of the soil was

obtained. Volume changes of the specimen during drying and wetting cycles were not recorded, therefore the SWRC was plotted only in terms of w .

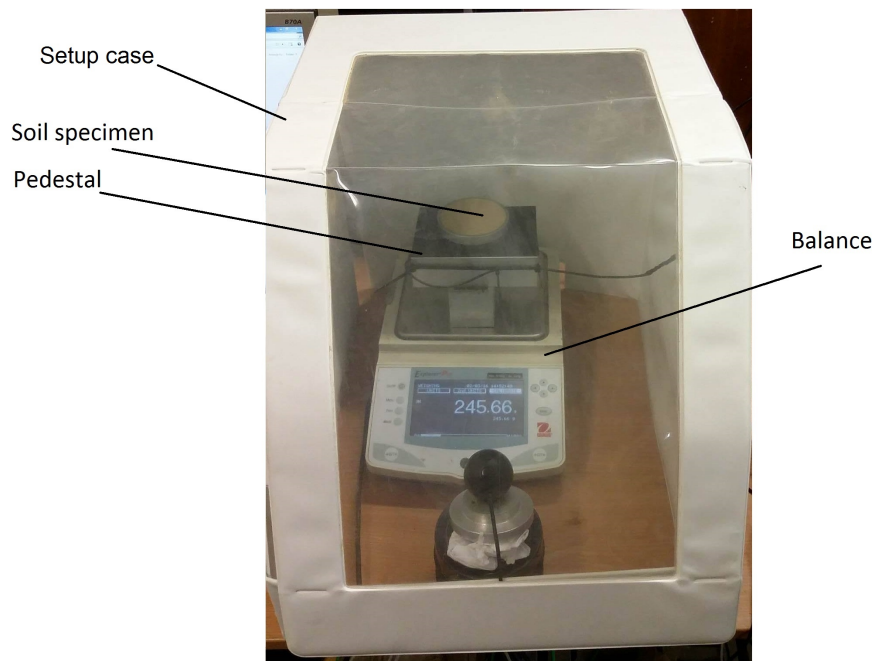


Fig. 4.21 Experimental setup for determining SWRC.

Figure 4.22 presents the change of the bulk gravimetric water content due to evaporation as determined by the balance readings. Two exposed specimens and one specimen covered with a plastic lid are shown. Results show that there are generally two branches of the desaturation curve, an initial linear part followed by an asymptotic to the x-axis curve. This shows that initially the pore water is evaporated at a constant rate which is not effected by the initial water content or the packing. The rate of water mass reduction is shown in Figure 4.22 and was 0.006% / min. After the soil reaches a water content of approximately 3%, evaporation starts becoming slower with a lower gradient. At this point, most of the top part of the specimen is already dried and evaporation occurs mainly at the bottom layers. On the other hand, the covered specimen retains the moisture throughout a 24 hour period with the evaporation rate

being 0.001 gr of water per minute or a reduction of 0.0008% / minute. Comparing the two methods, the exposed specimen seems to evaporate eight times faster than the covered one.

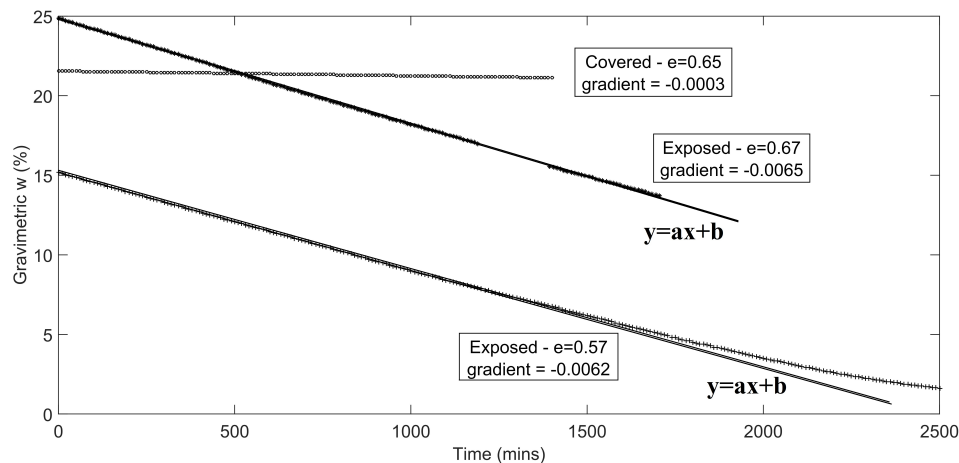


Fig. 4.22 Evaporation curves for LB-fraction E sand specimens. Values on the y-axis refer to the bulk gravimetric water content as it was determined by the balance readings. The gradients of the curves refer to the linear branch.

Distribution of moisture in the soil specimen due to drying

Evaporation causes the reduction of the bulk water content of the soil specimen, however a question about the uniformity of moisture within the specimen is raised. It is obvious that the top layer which is exposed to the atmosphere is evaporating faster than the bottom one and this might create a condition of non-uniform distribution of moisture. Therefore, more work was conducted on the determination of the moisture distribution within the soil specimen. To answer this question, soil specimens made of LB-fraction E sand were left to dry by evaporation under three circumstances: a) exposed to the atmosphere, left for 28 hours b) exposed to the atmosphere, left for 42 hours and c) covered with a plastic cap, left for 24 hours. After the above mentioned time periods, samples were taken from the centre and the circumference of the specimens at three different depths, i.e. top, middle and bottom, and dried in the oven to determine w .

Table 4.4 Moisture distribution results of soil specimens. *Top* (T), *middle* (M) and *bottom* (B) refer to the vertical position of the samples, while *centre* (Ce) and *circumference* (Ci) to the horizontal position. Bulk w is the mean water content and the one determined by the balance readings.

	T - Ce	T - Ci	M - Ce	M - Ci	B - Ce	B - Ci	Bulk w
	%	%	%	%	%	%	%
Exposed for 28h	7.59	8.42	9.34	8.64	10.49	9.08	10.09
Exposed for 42h	0.43	0.21	2.33	1.52	3.12	2.63	1.62
Covered for 24h	18.03	18.75	16.83	17.77	16.51	18.92	17.96

In Table 4.4 the water content distribution within the soil specimen determined by oven drying is shown. The last column shows the bulk or the mean values of moisture content for the whole sample and represents the values of moisture content determined by the balance readings. The covered sample shows a more uniform distribution with depth compared to the exposed ones. The exposed specimens show a gradual increment of w with depth which reveals that the upper layer evaporates first with the lower layers showing a delay. Even after 42 hours, when the top layer is almost dry, the bottom layer retains a small but significant amount of moisture, especially at the centre. There are, also, differences between the centre of the sample and the sides which are generally drier. It is worth noticing that the PPT is located at the centre of the specimen and at the bottom layer; therefore, suction readings correspond to the water content of the Bottom-Centre part.

Based on the two procedures described (in stages & continuous) for the SWRC determination using tensiometers and the preliminary results shown, it was decided to follow a combined procedure for the test. For the drying path the continuous method was followed, allowing the soil specimen to dry out exposed to the atmosphere. At specific time intervals, the specimen was covered with the plastic lid so as to allow the soil to equalise internally. After equalisation, readings of suction and mass were taken. For the determination of the wetting path of the curve, drops of water were added to the specimen and subsequently it was covered with the plastic lid

until suctions stabilized. At that point, the reading of suction value and mass provide a single point on the wetting branch of the SWRC. Wetting of the specimen continued until the soil reached the initial mass or until there were no significant change in suction.

Drying SWRC

Figure 4.23 shows the drying paths of the SWRC for the LB sand (fraction E), used in the current study, for two different packings. The dense specimen had an initial gravimetric water content of 22.2% with $e = 0.66$ while the loose specimen had an initial $w = 17\%$ and $e = 0.78$. Results show that the dense soil experiences higher suction than the loose soil for the same gravimetric water content. This is attributed to the smaller voids of the dense soil which need higher suctions in order to cavitate and allow water extraction through the evaporation process. On the other hand, the residual gravimetric water content is almost equal for both specimens, showing that it does not depend on packing.

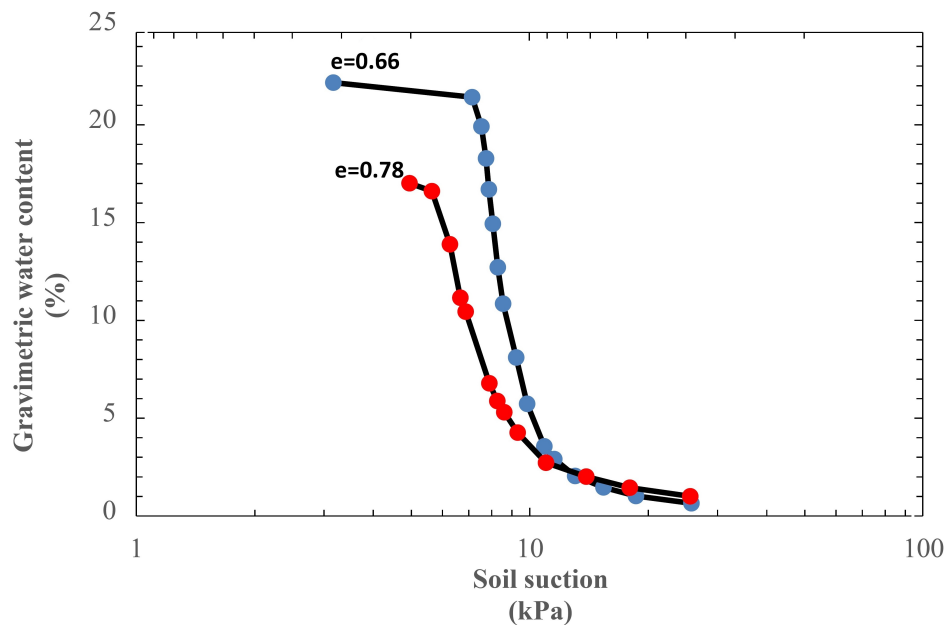


Fig. 4.23 Experimental drying SWRC for LB sand, fraction E.

At low moisture content, close to the residual ($w < 1\%$ at Figure 4.23), the measurement of suction becomes more complicated. Due to the nature of the PPTs and the cavitation of their pore stone when exposed to the atmosphere, suction increment is difficult to be attributed either to a real soil suction rise or the cavitation of the PPT. Figure 4.24 shows the variation of pore water pressure measurements with time for the specimen with $e = 0.66$. As can be seen, suction increased almost at a steady rate from the beginning of the test up to the point that the soil approaches the residual water content. Then, there is a sharp increment which ends up with the cavitation of the PPT.

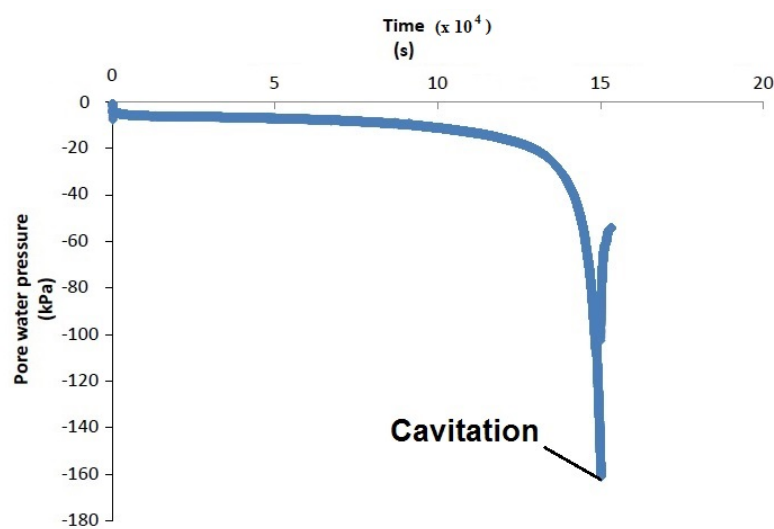


Fig. 4.24 Variation of pore water pressure with time for a SWRC test using tensiometer.

The transition from the actual reading of negative pore pressure to the PPT cavitation is not clear and cannot be determined. Two hypotheses are most probable here. The first one is that suction within the soil increases significantly when w approaches the residual value, in the same manner as shown in Figure 4.18a. At this stage, pore water exists as a thin film around soil particles and it is not held by the menisci (Toll et al., 2015). Consequently, PPTs are measuring the residual suction which is increased above the capacity of the tensiometer. Consequently, the tensiometer cavitates due to the induced high suction.

The second hypothesis is that PPTs are cavitating when desaturation reaches the residual zone. This is attributed to the technique of direct measurement of suction with tensiometers. When desaturation of the soil is extended and pore water exists on thin films around the soil grains, continuous paths to the atmosphere are created. Therefore, the porous stone of the tensiometer is actually exposed to the atmosphere. Consequently, the tension of the water in the porous stone is increased and therefore a non-real increment of suction is recorded. Eventually, this results in the cavitation of the PPT, in the way described in Section 3.3. In this case, the part of the SWRC which lies in the residual zone could be considered as non-realistic.

Comparing the results between the tensiometer method applied in this study and the pressure plate method (Figure 4.18a) for the same soil, it can be stated that suction readings within the residual zone using the PPTs are reliable.

The experimental data derived for the drying and wetting SWRC for the M_{90-10} mixture, using the tensiometer method, are shown in Figure 4.25. Two drying and one wetting cycles were applied to the same specimen. The initial void ratio was equal to 0.54 and the initial gravimetric water content 20.5%. The specimen was left to dry by evaporation until suction started to increase significantly, indicating the transition to the residual water content. Then, water was added to the specimen in order to define the wetting curve. Finally, the sample was left again to dry (2nd cycle) until it reached the residual water content.

As a result of the two drying cycles which were applied to the specimen, two drying SWRCs were determined for the M_{90-10} mixture. The two curves were identical, despite the fact that in the second cycle the specimen was dried after a wetting cycle. This result is in accordance to the statement presented in Figure 4.17 which describes the following drying curves as scanning curves between the primary drying and wetting curves. Also, it provides a validation of the tensiometer method for the determination of the SWRC since the same soil, under the same

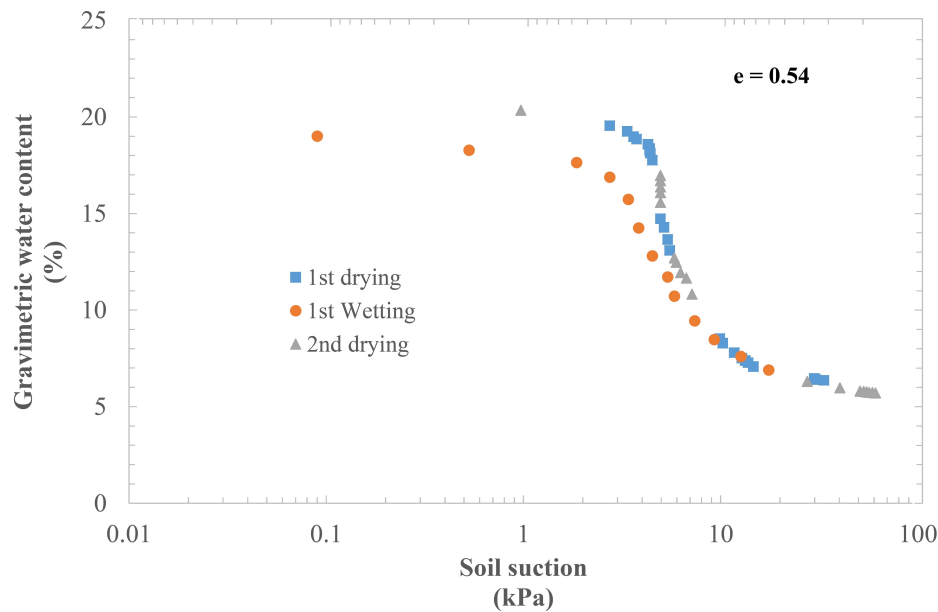


Fig. 4.25 Experimental data for the drying and wetting SWRC for the M_{90-10} mixture.

density, produced similar drying SWRCs.

The experimental SWRC for M_{70-30} mixture is shown in Figure 4.26. The initial void ratio of the soil specimen was 0.68 and the initial gravimetric water content was 22.6 %, which corresponded to an initial degree of saturation equal to 86.8 %. The specimen was placed on the base pedestal and left to dry by evaporation. Water content and suction measurements were taken at regular intervals until the gravimetric content reached 4.8 %. The shape of the experimental curve revealed that at the time the test was stopped, water content was not residual. However, due to a fault on the balance, no further data of the specimen mass were collected and, therefore, the test had to be terminated prematurely.

The experimental SWRC for M-Soil is shown in Figure 4.27. The specimen had an initial void ratio of 0.73 and initial gravimetric water content equal to 24.6 %, which corresponded to an initial S_r equal to 89.3 %. The sample was placed on the pedestal and left to dry exposed to

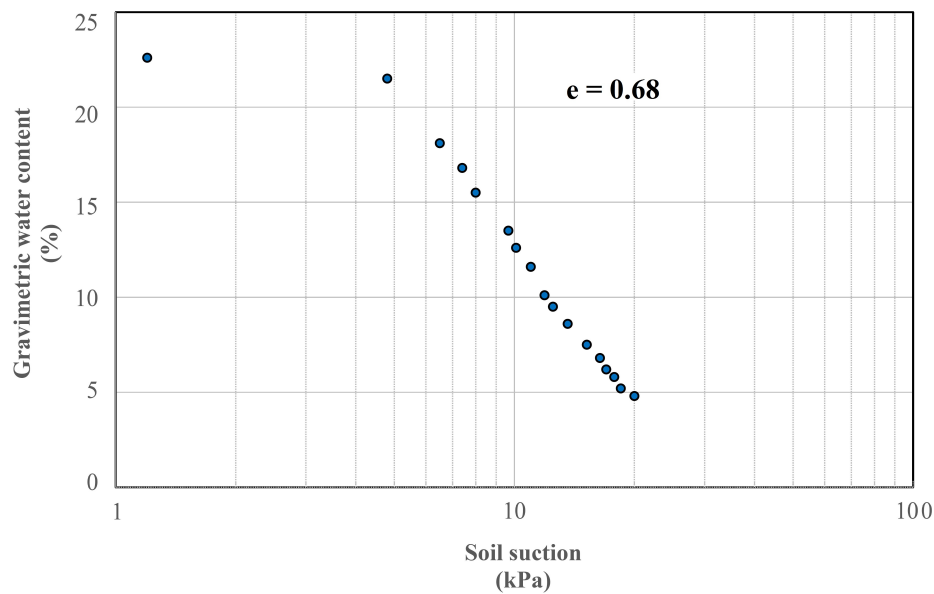


Fig. 4.26 Experimental drying SWRC for the M_{70-30} mixture.

atmospheric conditions while data for the gravimetric water content and suction were recorded at regular intervals. Initially, suction increased from 1 kPa to 8.1 kPa, with the gravimetric water content reducing only by 0.8 %. This reveals that the experimental curve is actually a scanning curve and not a primary drying curve. Therefore, in order to determine the actual primary drying and wetting curves of the tested soils, experimental data were fitted into a mathematical expression for the SWRC.

Mathematical description of the measured SWRC

A number of mathematical equations which describe the SWRC of a soil exist in the literature. These equations provide a full representation of the retention curve and make use of soil and fitting parameters. Moreover, they allow for the determination of retention parameters of the soil which can be easily extracted from the theoretical curves using mathematical expressions. The most commonly used relationships which have been described in the literature are: [Gardner \(1958\)](#), [Brooks and Corey \(1964\)](#), [van Genuchten \(1980\)](#) and [Fredlund and Xing \(1994a\)](#). It is not the intention of this thesis to fully describe these mathematical expressions for obtaining the

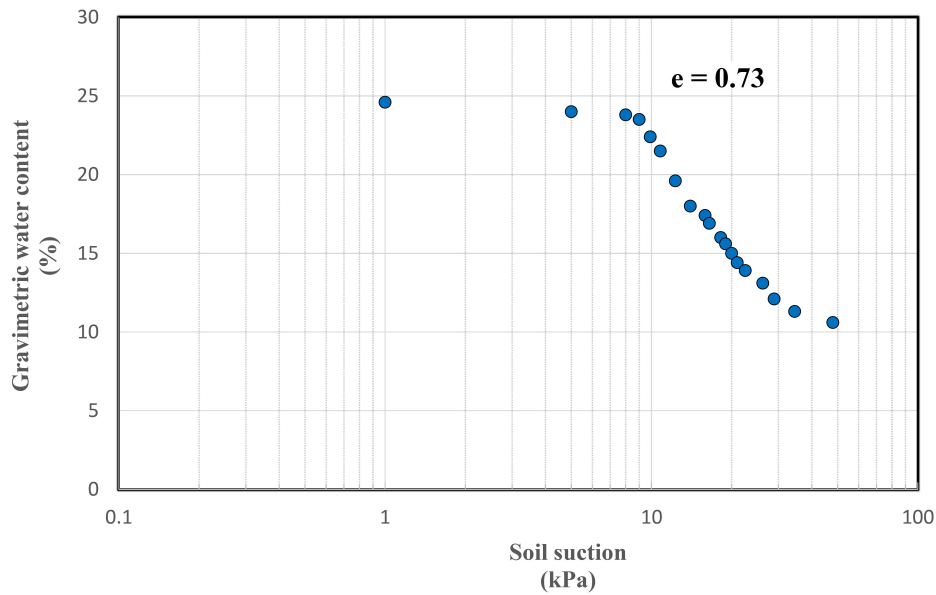


Fig. 4.27 Experimental drying SWRC for the M-Soil.

SWRC. A comprehensive review of these methods can be found in [Leong and Rahardjo \(1997\)](#). In this thesis, the van Genuchten method was chosen to fit the experimental data obtained from the tensiometer method.

In their review about the mathematical expressions for the SWRC, [Leong and Rahardjo \(1997\)](#) demonstrated the validity of the van Genuchten method. The authors concluded that in the case of cohesionless soils, this method provides the best results. Therefore, the van Genuchten equation was used in the current study in order to fit the experimental data obtained from the tensiometer method. Although experimental data for the SWRC were available, the mathematical formulation of the curve was considered necessary. This is because using Equation 4.11 gives the full curve of the soil neglecting the fact that the initial water content of the specimens tested was less than the saturated one. Moreover, the mathematical representation of the SWRC allowed for the determination of important retention parameters, such as the air entry value α_{ev} and the residual water content.

The mathematical formulation of the van Genuchten curve is the following:

$$\theta = \theta_r + \frac{\theta_s - \theta_r}{(1 + (\alpha\psi)^n)^m} \quad (4.10)$$

where θ is the volumetric water content of the soil, θ_r the residual and θ_s the saturated volumetric water content, ψ is the matric suction and α , n and m fitting constants.

The same expression can also be written in terms of gravimetric water content w instead of the volumetric water content, i.e.:

$$w = w_r + \frac{w_s - w_r}{(1 + (\alpha\psi)^n)^m} \quad (4.11)$$

with w_r and w_s are the residual and saturated gravimetric water content, respectively.

In the work of [van Genuchten \(1980\)](#), the saturated and residual water content were considered as knowns since they are easily determined. The saturated gravimetric water content of the soil can be calculated by using the equation:

$$S_r = G_s w / e \quad (4.12)$$

where S_r is the degree of saturation, G_s the specific gravity and e the void ratio of a given packing. At saturation:

$$w = e / G_s \quad (4.13)$$

The residual water content is the water content at which the desorption path becomes constant with increment in suction. Even though the residual water content can be easily determined graphically from the experimental data, it was considered as an independent fitting parameter to Equation 4.11 in order to be determined numerically.

The fitting parameters used in Equation 4.11 are α , n and m . Leong and Rahardjo (1997) argued that parameter α is related to the air entry value α_{aev} of the soil. Based on experimental evidence, Kasangaki (2012) showed that there is a linear relationship between the reciprocal of the fitting parameter α and α_{aev} which is given by:

$$\alpha_{aev} = 0.692 \times \alpha^{-1} - 0.284 \quad (4.14)$$

The value of the parameter n controls the slope of the linear part of the SWRC, with higher values giving steeper slopes. Practically, this parameter indicates the impact of suction to the desorption process. In order to obtain a closed-form empirical equation for the unsaturated hydraulic conductivity, van Genuchten (1980) related parameters n and m with the expression $m = 1 - n^{-1}$. For the accurate determination of the fitting parameters to the experimental data, the least squares curve fitting procedure was applied.

The results from the curve fitting procedure for the LB fraction E sand are shown in Figure 4.28, while the values of the fitting parameters and the coefficient of determination R-squared for the two soil packings are shown in Table 4.5. The initial gravimetric water content of the loose sample was calculated at 29.4% and for the dense sample at 24.9%. At the experimental procedure, the initial water content of the samples was less than the saturated one. This could lead to a false interpretation of the results. In fact, the experimental curve is a scanned curve and not the primary drying curve.

The results from the curve fitting procedure for the two drying SWRCs of the M_{90-10} mixture are shown in Figure 4.29. Using Equation 4.13, the saturated gravimetric water content was determined to be equal to 20.6 % while the residual one was determined from the curve fitting procedure and found at 6.2 % for the first cycle and 5.9 % for the second. Fitting parameters were determined as for the LB fraction E sand and are also shown in Table 4.5.

The determined drying SWRC for M_{70-30} , using van Genuchten fitting curve method, is shown in Figure 4.30. The gravimetric water content at saturation was determined at 25.8 %

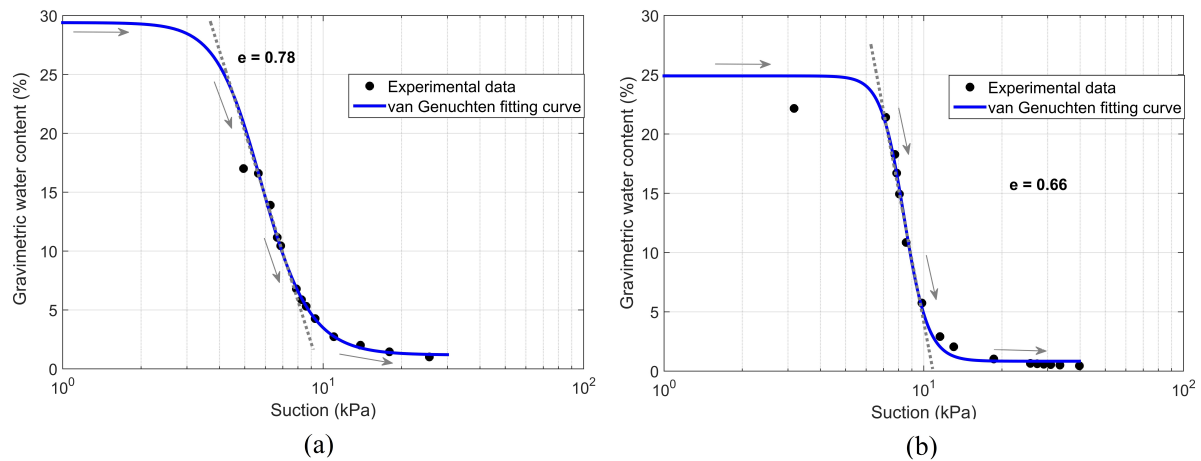


Fig. 4.28 Drying Soil Water Retention curves for LB sand, fraction E, determined using the van Genuchten method. (a) primary drying curve at loose state ($e = 0.78$) and (b) primary drying curve at dense state ($e = 0.66$)

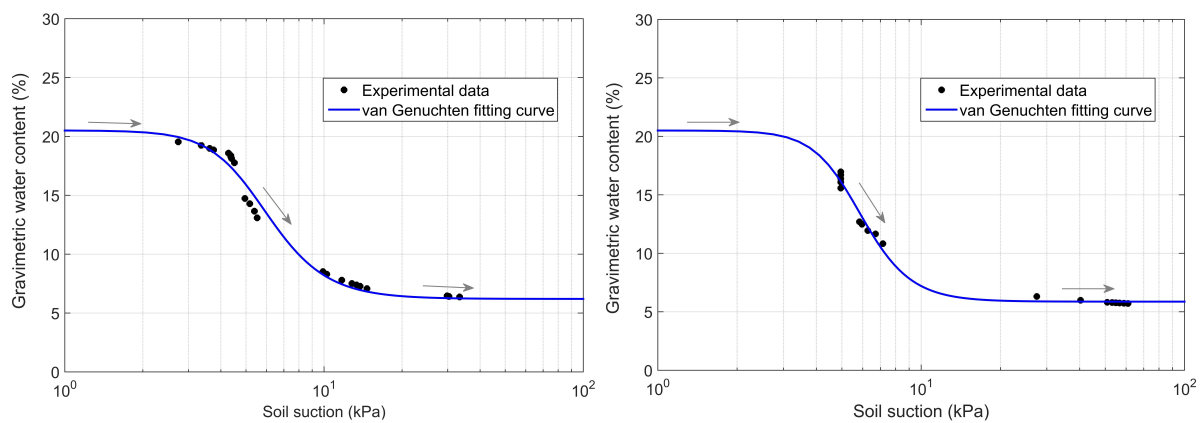


Fig. 4.29 Drying SWRC for the M_{90-10} mixture, determined using the van Genuchten method.

and the residual was found to be 4 %. The fitting parameters of the van Genuchten method are shown in Table 4.5.

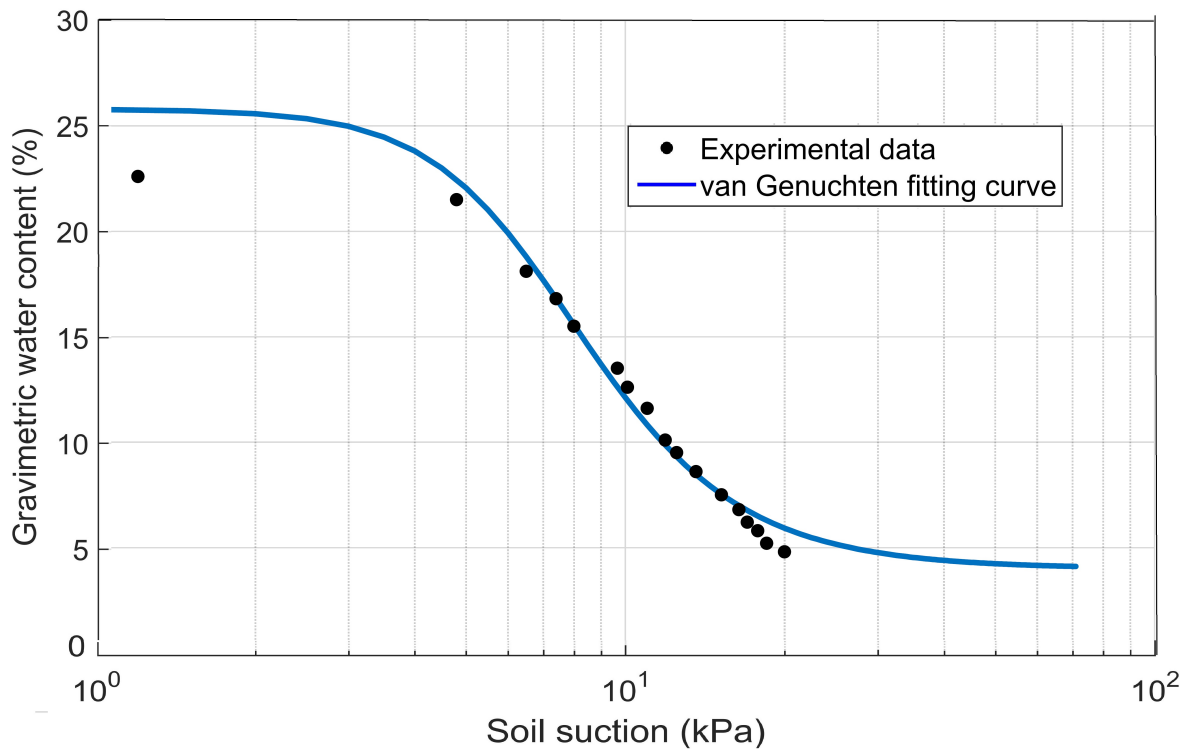


Fig. 4.30 Drying SWRC for the M_{70-30} mixture, determined using the van Genuchten method.

Finally, the fitting dry curve for the M-Soil was determined and presented in Figure 4.31. The gravimetric water content at saturation and residual state was found to be 27.5 % and 10.89 %, respectively. The fitting parameters are shown in Table 4.5.

Wetting SWRC

The wetting of the soil specimen was performed by gently adding water droplets on the soil mass using a syringe. When the total mass was increased so as to indicate an increment to the moisture content of about 1%, the soil specimen was covered with the lid and let to equalise for a few minutes. The time required for the moisture to distribute uniformly to the soil specimen was determined by the readings of the pore water pressure. When wetting was applied to the

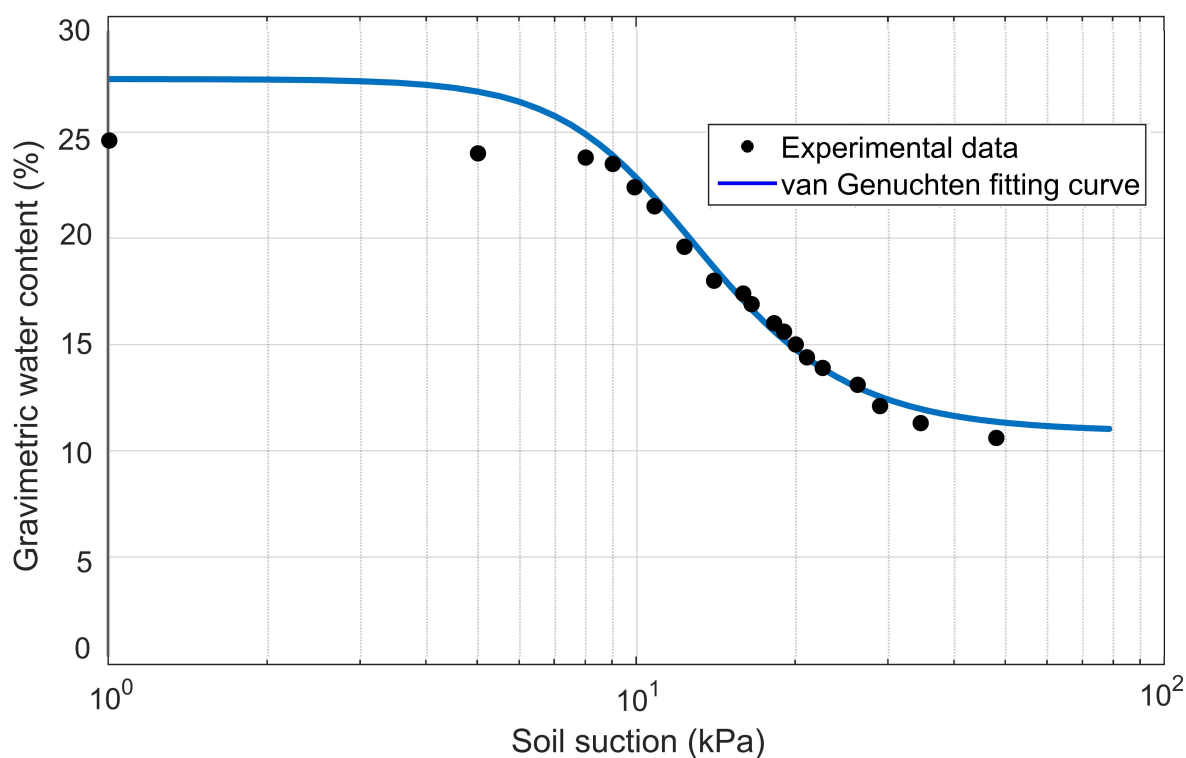


Fig. 4.31 Drying SWRC for the M-Soil, determined using the van Genuchten method.

Table 4.5 Fitting parameters of the van Genuchten model for the desorption process (drying curve).

Soil type	e	α	n	m	w_r	α_{ev}	R^2
	-	(kPa)	-	-	(%)	(kPa)	-
LB fraction E (dense)	0.66	0.121	9.886	0.899	0.8356	5.43	0.9889
LB fraction E (loose)	0.78	0.181	5.162	0.806	1.178	3.54	0.9871
M_{90-10} (cycle 1)	0.54	0.1816	4.192	0.761	6.2	3.53	0.9763
M_{90-10} (cycle 2)	0.54	0.1796	5.019	0.8	5.9	3.57	0.9925
M_{70-30}	0.68	0.1405	3.331	0.699	4.006	4.47	0.9757
$M - Soil$	0.73	0.086	3.5	0.714	10.89	7.76	0.9664

soil, suction was decreased gradually and reached a steady value after a while. When suction started to increase again slightly, moisture and pressure were recorded and wetting was applied again following the same procedure.

Two wetting curves of the LB fraction E sand were determined with the tensiometer technique and presented in Figure 4.32. Figure 4.32a shows the results after the first wetting process (primary wetting curve) while Figure 4.32b shows the results from a subsequent wetting (secondary wetting curve). In both cases, the initial void ratio of the specimens was 0.71. Curve fitting was applied to the experimental data using the Van Genuchten method (Equation 4.11). The fitting parameters and R^2 values are shown in Table 4.6.

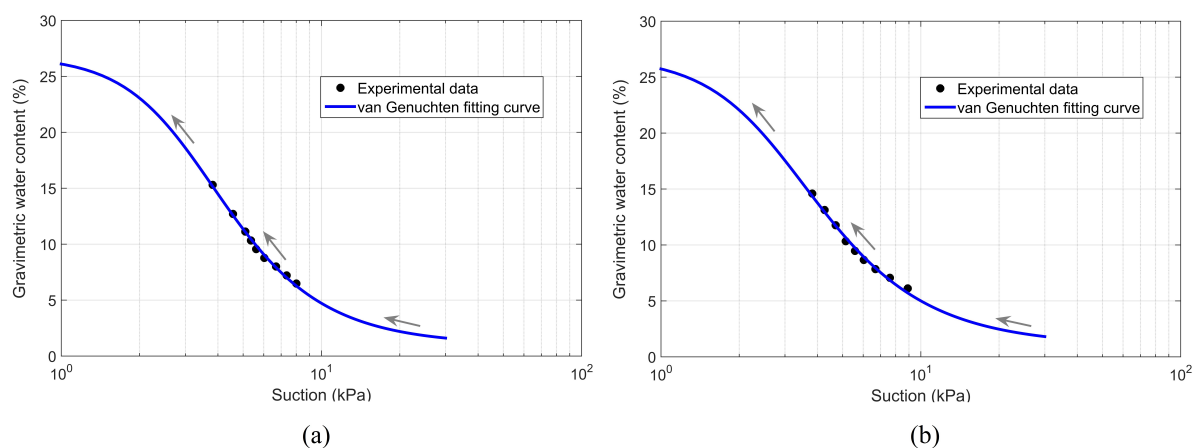


Fig. 4.32 Wetting branch (desorption curve) of the SWRC for LB sand, fraction E. (a) primary wetting curve and (b) secondary wetting curve.

The wetting brunch of the SWRC for the M_{90-10} soil, as was determined by the van Genuchten fitting curve, is shown in Figure 4.33 and the fitting parameters are presented in Table 4.6.

To determine the van Genuchten wetting curves, the residual water content was kept at a fixed value equal to the w_r obtained from the drying curves. For both cases, $w_r \approx 1\%$, so the

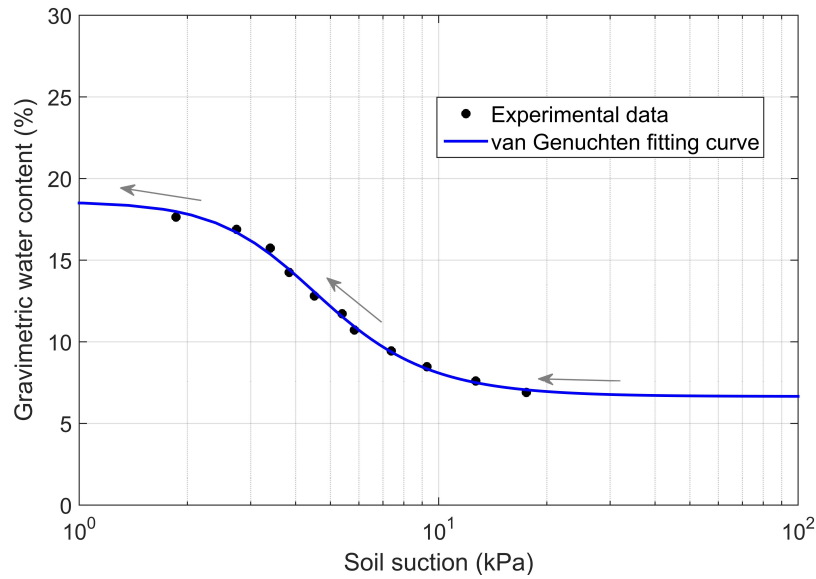


Fig. 4.33 Absorption curve for the M_{90-10} mixture.

Table 4.6 Fitting parameters of the van Genuchten model for the absorption process (wetting curve).

Soil type	e	α (kPa)	n	m	α_{ev} (kPa)	R^2
LB fraction E (cycle 1)	0.71	0.3126	2.674	0.626	1.93	0.995
LB fraction E (cycle 2)	0.71	0.3474	2.475	0.596	1.71	0.991
M_{90-10}	0.54	0.248	3.294	0.696	2.51	0.996

same value was used also for the wetting curves. Also, w_s had a fixed value of 26.7 % which corresponds to saturated conditions for $e = 0.71$.

4.2.9 Strength properties

The stability evaluation of the slope models requires the accurate determination of the shear strength properties of the soils used in this study. This section includes the results from a number of laboratory tests for the determination of the shear strength properties of these soils. In particular, Leighton Buzzard sand (fraction E) specimens were tested on a direct shear

apparatus in order to determine the peak and ultimate angle of friction. Moreover, M_{90-10} mixture triaxial specimens were used to determine cohesion and angle of friction in Consolidated Undrained (CU) tests performed on Triaxial testing equipment.

Direct shear tests

Direct shear tests were performed on specimens of LB fraction E sand. The objective of these tests was to determine the peak and residual angle of friction. Even though a small apparent cohesion was obtained from the test results, LB fraction E sand is considered a cohesionless material. The test procedures followed are described in BS 1377:1990-7. The direct shear tests were conducted on the ShearScan2 apparatus by VJ Tech Ltd (Fig. 4.34). This is an automated direct shear apparatus which uses a pneumatic loading concept for applying vertical load to the specimen, while horizontal load is applied using a stepper motor. Therefore, the control and measurement of normal and shear stresses is more accurate than in conventional shear box apparatuses. Two load cells with 15 kN capacity were used for the measurement of vertical and horizontal load applied to the specimen. Two LVDTs were, also, used for the measurement of vertical and horizontal displacement and the determination of axial and volumetric strain during shearing.

The test was conducted on dry samples and consisted of two stages, consolidation and shearing. Since the specimens were dry, consolidation data were not used on the determination of the maximum shearing because the settlement due to vertical loading was rapid. The speed of the horizontal displacement was set to be 0.1 mm/min. Four different normal stresses were applied to the specimens, namely 50 kPa, 100 kPa, 200 kPa and 400 kPa, with the latest simulating the stress state at the highest slope depth during centrifugal acceleration of 60g. Soil specimens were initially compacted to a high density, therefore the peak and residual shear

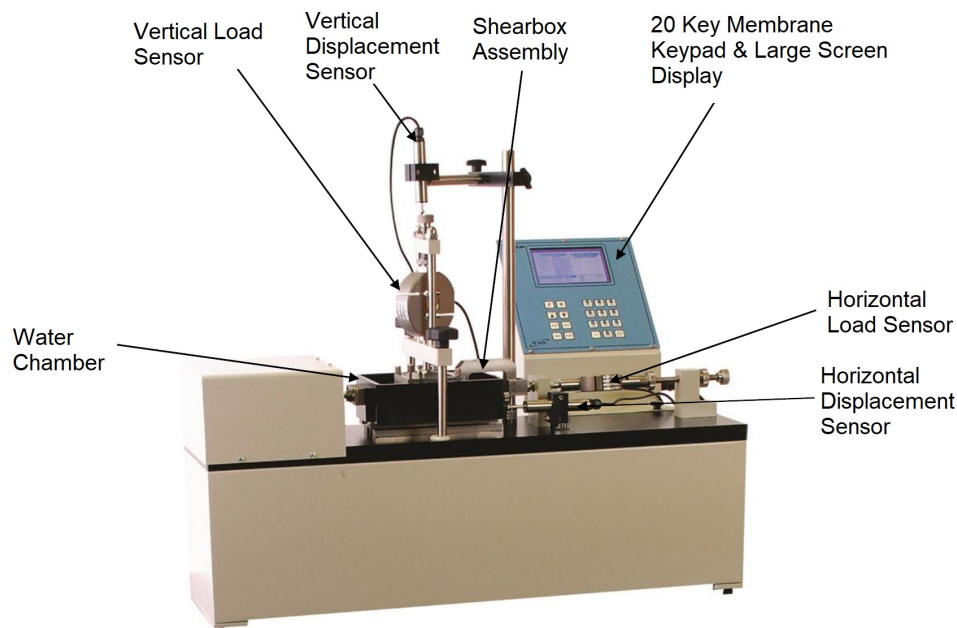


Fig. 4.34 ShearScan2 apparatus, by VJ Tech Ltd, used for direct and residual tests on LB fraction E sand.

strength were determined. The initial dry density of the LB fraction E sand specimens was 1.58 Mg/m^3 .

In addition, the residual shear strength was studied by performing multi-reversal direct shear tests. These consist of several repeatable shearing stages at the same specimen until a steady (residual) shear stress is observed. This value of stress defines the residual strength of the soil and can be compared to the residual stress defined from the single shearing of the dense samples. In this series of residual shear tests, each specimen was subjected to 4 subsequent shearing stages which proved to be satisfactory in achieving the residual state.

The stress-strain graph for the LB fraction E sand is shown in Figure 4.35. In the same figure, the results from the residual shear tests are shown. The maximum travel of the split box for all stages was the equivalent to 14 % of axial strain, corresponding to a displacement of 8.4 mm. That displacement was considered sufficient to capture also the residual part of the curves. In Figure 4.36, the peak and residual values for the shear stress are shown and the respective strength envelopes have been plotted. Therefore, the determined values for the peak

and residual angle of friction were 32° and 30° , respectively. Figure 4.37 presents the volume change of the specimens during shearing.

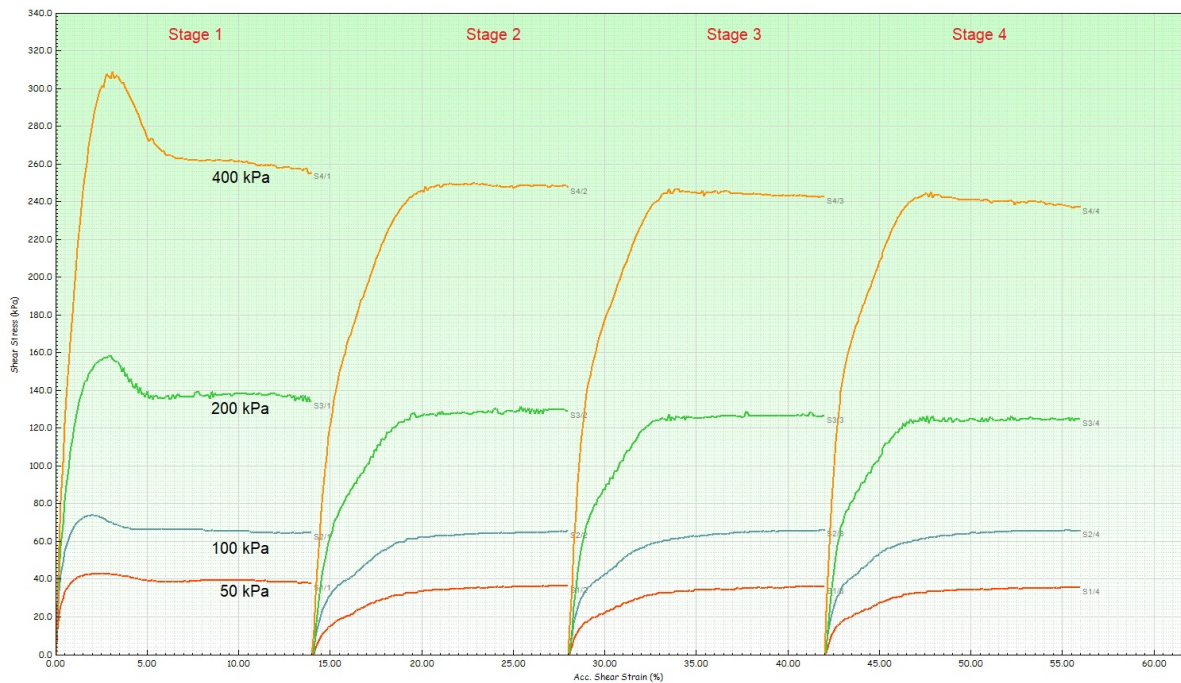


Fig. 4.35 Shear stress - shear strain graph for the LB fraction E sand.

A small apparent cohesion was observed in both peak and residual shear strength of LB fraction E sand. This is attributed to the unsaturated conditions of the tested sand and the contribution of suction to the shear strength. Despite that, LB fraction E is a clean, uniform sand and as such is considered a cohesionless material.

Triaxial tests

In this section the results from the triaxial testing of the M_{90-10} mixture are presented. Consolidated Undrained test (CU) was conducted and the effective strength parameters c' and ϕ' were determined.

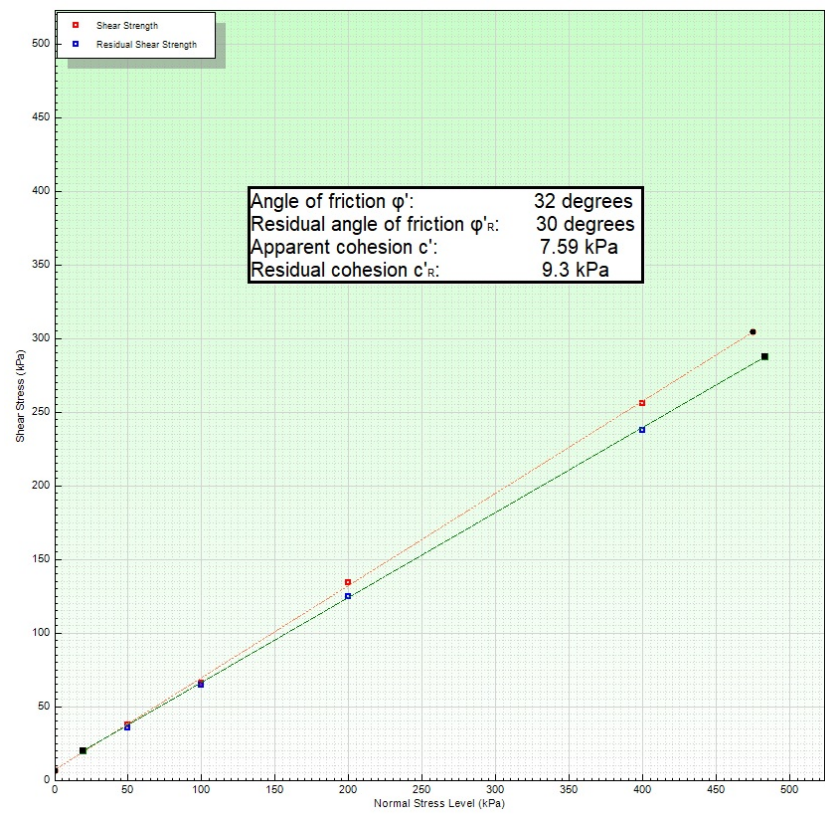


Fig. 4.36 Shear stress vs normal stress graph for the LB fraction E sand.

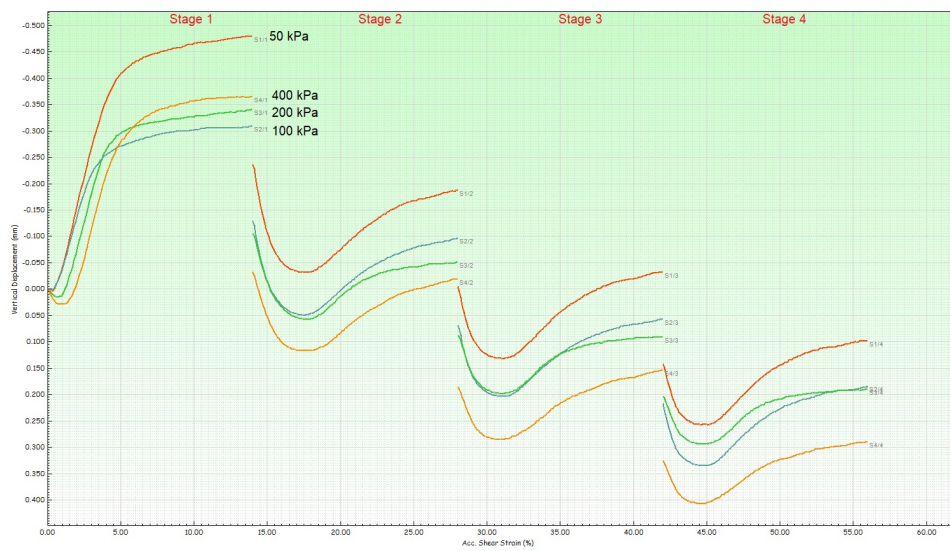


Fig. 4.37 Volume change of LB fraction E specimens during shearing.

The triaxial tests were performed based on the procedure described in BS 1377 - 1990, part 8. The equipment used in these tests was provided by VJTech limited and the setup is shown in Figure 4.38. This included:

- A load frame with a maximum capacity of 50 kN with a built-in four channel data logger.
- A triaxial cell able to accommodate soil samples with a maximum diameter of 50 mm and height of 100 mm. It brings three zero strain valves which allows to control cell pressure, back pressure and pore water pressure at the base of the specimen. The pedestal and top cap include a pair of ports which allow to apply back pressure at the top of the sample and measure the pore water pressure at the bottom.
- Two Automatic Pressure Controllers (APCs) for controlling and measuring water pressure in the cell (cell pressure APC) and in the sample (back pressure APC). Furthermore, the volume of water which flowed in the sample during saturation and out during consolidation and drained shearing was measured by the back pressure APC.
- An internal load cell (submersible load cell) located inside the triaxial cell with a maximum capacity of 5 kN. The submersible load cell was used instead of an external one as it offers the advantage to be placed close to the specimen and to be insensitive to the cell confining pressure and friction between the loading ram and the cell. Therefore, it offers more accurate measurement of the applied load.
- A pressure transducer used to measure pore water pressure at the bottom of the soil sample. The maximum capacity of the transducer was 1000 kPa.
- A Linear Strain Conversion Transducer (LSCT) for determination of strain.
- De-aired water supply provided to the sample through a pressure distribution panel.

The M_{90-10} specimen used at the triaxial testing had a density of 1.42 Mg/m^3 . This density was selected in order to achieve a similar packing with the soil used in the centrifuge slope



Fig. 4.38 Triaxial setup used for CU test in M_{90-10} mixture, provided by VJTech.

tests. In this way, the behaviour of the soil under the same initial conditions will be evaluated. The soil was compacted within a cylindrical mould with height of 100 mm and diameter of 50 mm, i.e. the ratio height:diameter was 2:1. Prior to the compaction, the soil was mixed with water and left for 24 hours to equalise. The initial moisture content of the specimen was determined by drying out remnants of the soil and was 12%. After compaction, the specimen was removed from the mould and carefully placed onto the pedestal, covered with a rubber membrane and fixed in place using two rubber o-rings. The top cap, providing the specimen with the back pressure, was fixed on the specimen using another two o-rings.

The method of testing which was selected for the CU triaxial testing was the multistage. Multistage effective stress tests have been fully described by [Head and Epps \(2014\)](#). Instead of testing three specimens in order to determine the effective strength parameters, multistage tests involve testing of a single specimen which is consolidated and sheared three or even four times. The first step includes saturation of the soil sample using the procedure described in

BS 1377:1990, part 8. Then, the specimen is consolidated at the smallest effective stress with the shearing stage to follow. During the shearing stage, it is important to define the stress level which is just enough to cause shearing. At that point, compression of the sample stops and the specimen is consolidated again at the next effective stress.

In the preparation of sandy specimens, it is usually very difficult to achieve the same density and, therefore, void ratio in all the specimens tested. The main advantage of multistage testing is that the inconsistency of different soil specimens, regarding the initial conditions, is avoided. An essential factor in multistage testing is the selection of the appropriate criterion for failure at the subsequent shearing stages. Excessive deformation is usually avoided in the first stages, while in the third stage the soil is left to either reach a maximum shear stress or strain to reach 20 %.

In the first stage, the specimen was subjected to saturation. Initially, de-aired water was flashed into the sample through the pore water pressure valve, importing water at a pressure of 40 kPa while the cell pressure was kept equal to 50 kPa. In this way, an effective stress of 10 kPa was applied to the specimen during this process which prevented its disturbance. Water flushing lasted until constant water flow was noticed through the back pressure line. Proper saturation followed the flushing stage, with increasing steps of cell and back pressure increments. Cell pressure was always kept 10 kPa above back pressure, imposing again an effective pressure within the sample equal to 10 kPa. Between the saturation steps, B-check was performing by elevating the cell pressure by 50 kPa and measuring the response of the pore water pressure. The B-value determined in every step is given by:

$$B = \delta u_w / \delta \sigma_3 \quad (4.15)$$

The relation between the B-value and the applied cell pressure is shown in Figure 4.39. The specimen was considered to be saturated as soon as the B-value was determined to be 0.95.

Saturation level was constantly increasing as the cell and back pressures were elevated. The desired B-value level was achieved at a back pressure of 440 kPa. For the B check, cell pressure was elevated at 500 kPa and the B value was determined to be equal to 0.95. Following saturation, the back pressure was elevated to 500 kPa and remained constant throughout the consolidation and saturation stages. The induce of effective pressure within the sample was done by elevating only the cell pressure.

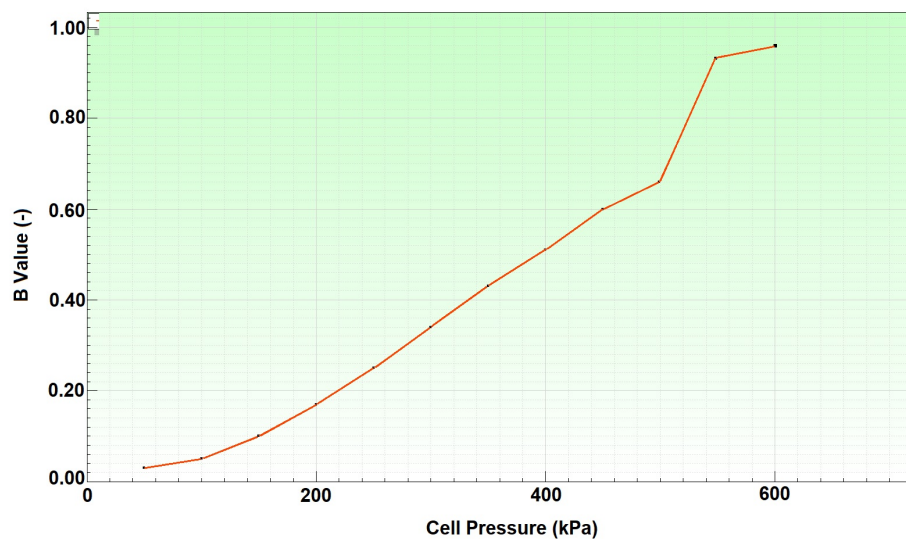


Fig. 4.39 B-value determined at each cell pressure increment, during the saturation of the M_{90-10} specimen

As already mentioned, the specimen was sheared in four stages. Before each shearing, consolidation took place at different effective pressures. These were selected as follows:

- Stage 1: 50 kPa
- Stage 2: 100 kPa
- Stage 3: 200 kPa
- Stage 4: 400 kPa

The specimen was left to consolidate until the excess pore pressure was dissipated by at least 98 % (Figure 4.40a). The change in volume was also recorded using the volume readings of the back pressure controller (Figure 4.40b). To achieve the desired effective pressures at each stage, the confining pressure (cell pressure) was raised above the back pressure by equivalent amounts.

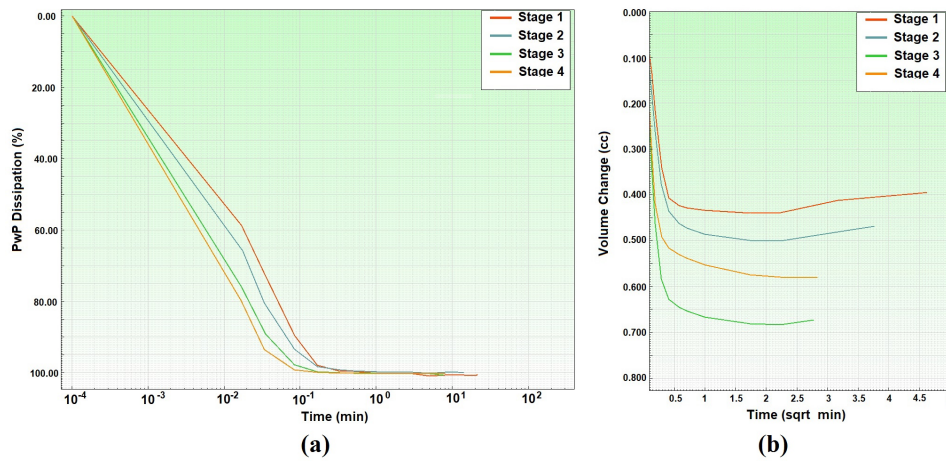


Fig. 4.40 Consolidation graphs for the M₉₀₋₁₀ soil. (a) Pwp dissipation and (b) Volume change.

The stress-strain curves are shown in Figure 4.41. The deviator stress has been corrected in order to account for the rubber membrane restrains, based on the procedure described in BS 1377 - 1990, part 8. The criterion which was selected as the stopping condition for each shearing stage was the maximum Stress Ratio (SR). SR is given by the following relationship:

$$SR = \sigma'_1 / \sigma'_3 \quad (4.16)$$

where σ'_1 the major effective stress and σ'_3 the minor effective stress.

The selection of the maximum SR as a failure criterion is very practical in multistage CU tests. This is because usually the maximum SR is achieved before the maximum deviator stress, therefore it allows to stop the shearing stage in a timely manner. The failure criterion for all the

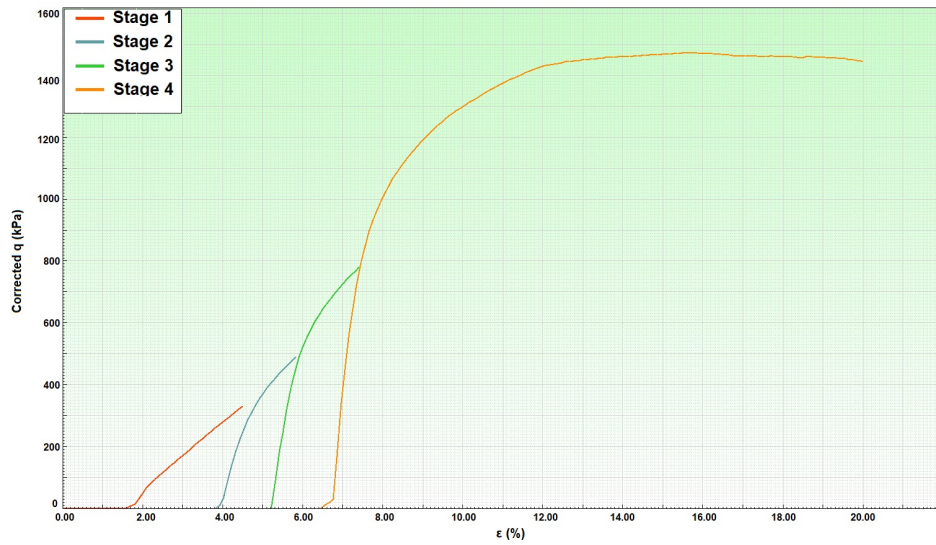


Fig. 4.41 Stress-strain curves for the M_{90-10} soil

shearing stages was a reduction to the SR for the time period required for the axial strain to change by 0.4 %. At the final shearing stage, the specimen was left to deform until the axial strain achieved 20 %.

The representation of the stress state for the M_{90-10} soil can be summarised in Figures 4.43 and 4.44. Figure 4.43 presents the Mohr circles obtained from the maximum deviator stress at the four shearing stages. Stresses plotted on the Mohr circles graph are the effective stresses. The Mohr-Coulomb failure envelope has been drawn and allows to determine the values of cohesion and angle of friction which were found to be 7 kPa and 32.5° .

Shear strength parameters can be also derived from the stress path plot shown in Figure 4.44. This has been plotted in terms of effective stresses on a MIT stress field with axis t' and s' , with:

$$t' = t = (\sigma_1 - \sigma_3)/2 \quad (4.17)$$

and

$$s' = (\sigma'_1 + \sigma'_3)/2 \quad (4.18)$$

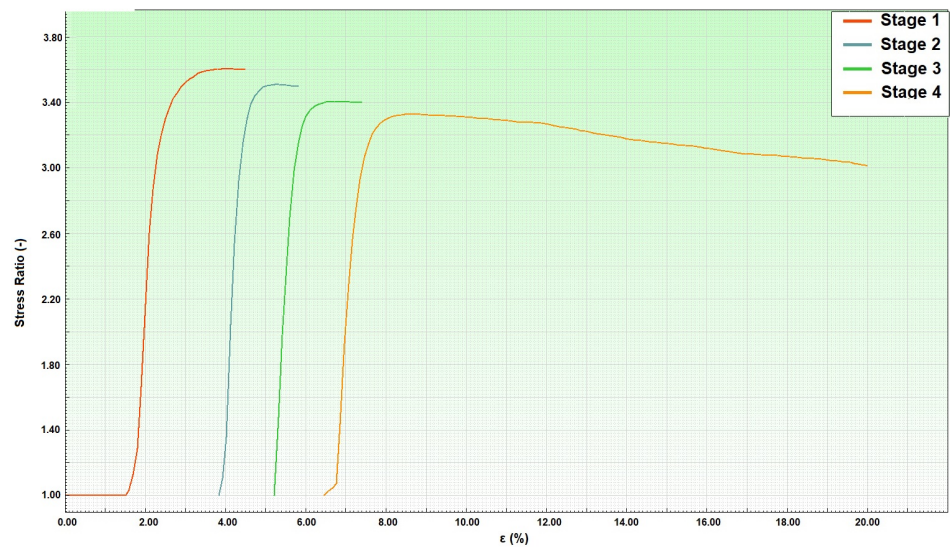


Fig. 4.42 Stress ratio variation of the M₉₀₋₁₀ soil

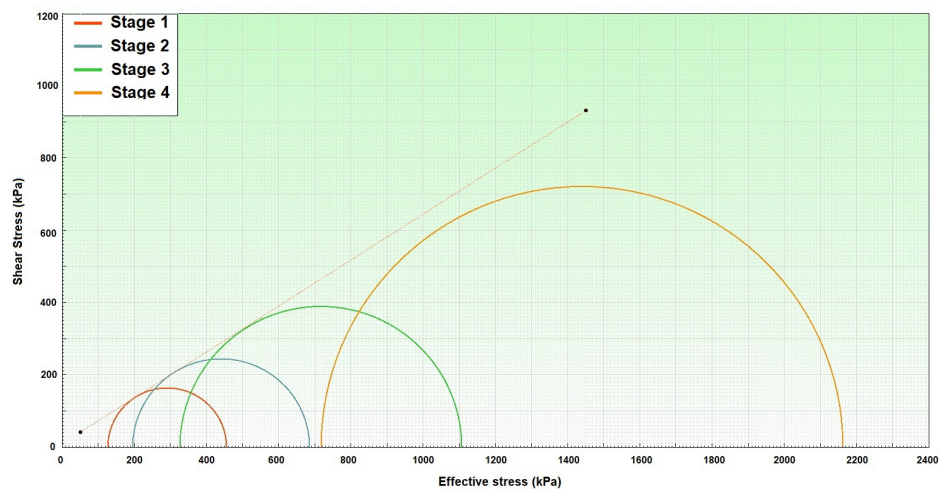


Fig. 4.43 Mohr-Coulomb failure envelope for M₉₀₋₁₀ soil

where σ'_1 and σ'_3 are the major and minor effective principal stresses, respectively.

The failure envelope (K_f line) has been drawn from the failure points for the four stress paths. The derived shear strength parameters from the stress path plot are 6 kPa and 32.4° . Finally, the variation of the pore water pressure during the four shearing stages is shown in Figure 4.45.

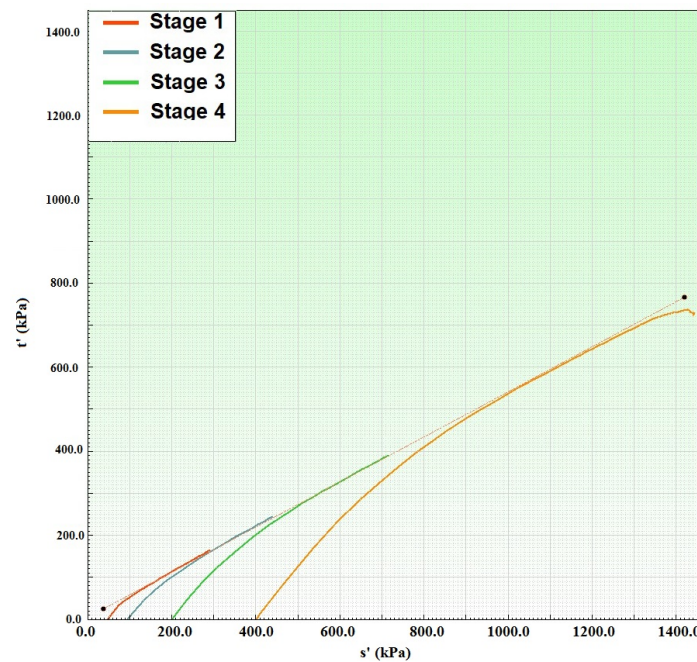


Fig. 4.44

4.2.10 Summary from laboratory testing programme

Table 4.7 presents a summary of the properties for the soils involved in this study under the laboratory testing program described in this chapter. Regarding the shear strength properties, there is a distinguish between the effective strength parameters determined from direct shear tests and those determined from triaxial tests. Moreover, the shear strength parameters deter-

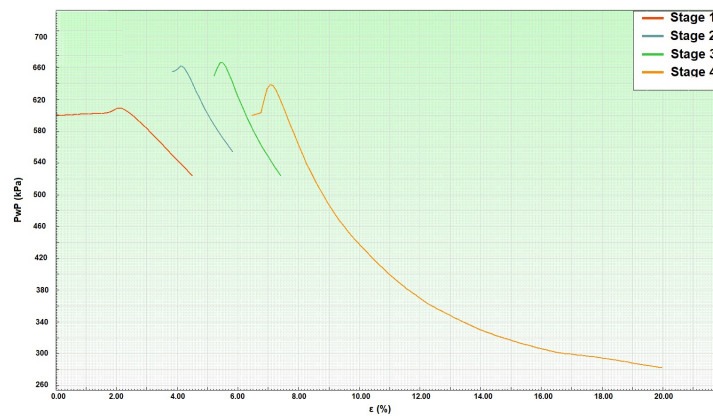


Fig. 4.45 Pore water pressure variation within the sample during the four shearing stages in respect to the axial strain.

mined at CU tests are presented based on the method used to derive them, i.e. if these were determined using the Mohr-Coulomb stress plot or the Stress Path plot.

Table 4.7 Properties of soils and mixtures used in this study. DS: Direct Shear tests.

	LB-E	Congleton	M ₉₀₋₁₀	M ₇₀₋₃₀	M ₅₀₋₅₀	M-Soil
Description	Fine sand	Fine sand	Silty sand	Silty sand	Silty sand	Silt
D ₁₀ (mm)	0.095	0.13	0.072	0.055	0.036	0.011
D ₅₀ (mm)	0.12	0.18	0.16	0.145	0.13	0.065
ρ_s (Mg/m ³)	2.65	2.62	2.62	2.63	2.64	2.65
$\rho_{d,max}$ (Mg/m ³)	1.61	1.70	1.66	1.665	1.672	-
$\rho_{d,min}$ (Mg/m ³)	1.31	1.47	1.30	1.31	1.34	-
w _{opt} (%)	11	-	10.5	12.5	14.3	-
e _{max} (-)	1.01	0.78	1.000	1.008	0.955	-
e _{min} (-)	0.69	0.54	0.578	0.579	0.579	-
k _{sat} (m/s)	5 × 10 ⁻⁵	7.51 × 10 ⁻⁵	6 × 10 ⁻⁶	8.11 × 10 ⁻⁶	1.05 × 10 ⁻⁷	5.34 × 10 ⁻⁷
c' (DS tests) (kPa)	0	-	-	-	-	-
φ' (DS tests) (°)	32	-	-	-	-	-
c' (CU tests -	-	-	7.15 kPa	-	-	-
Mohr-Coulomb) (kPa)						
φ' (CU tests -	-	-	32.5°	-	-	-
Mohr-Coulomb) (°)						
c' (CU tests -	-	-	6 kPa	-	-	-
Stress Path) (kPa)						
φ' (CU tests -	-	-	32.4°	-	-	-
Stress Path) (°)						

4.3 Soil model preparation

4.3.1 Sandy slope models

Slope models were prepared in the centrifuge strong box at an inclination of 50° . The selection of the slope angle was based on the soil characteristics and the failure mechanism that was attempted to be simulated. An over-steepened granular slope, with angle greater than the peak angle of internal friction, would remain stable only under the effect of suction. Since rain infiltration is causing the saturation of the soil profile and, thus, the elimination of suction, rainfall simulation was expected to cause instability. For this reason, the angle of the slope should need to be always greater than the angle of internal friction.

The preparation procedure consisted of the following steps:

1. Dry pluviation of the fine sand into the strong box to form the soil block;
2. Saturation of the soil from the bottom of the centrifuge box;
3. Desaturation of the soil by allowing free drainage;
4. Slope cut in the unsaturated soil mass;
5. Final adjustments and loading of the strong box on the centrifuge platform for testing.

Dry pluviation

Dry pluviation was used to prepare the slope models for centrifuge testing. It is also referred to as multiple-sieving air pluviation method ([Miura and Toki, 1982](#); [Zhao, 2008](#)) and provides a reliable method for preparation of granular soil models at dry densities between 50% and 90%. The final relative density of the model depends on the flow rate of the soil (i.e. the mass of soil per unit time) and the height of drop. In general, higher relative dry densities can be achieved by

increasing the height of the soil drop and decreasing the flow rate (Zhao, 2008). The advantage of dry pluviation is that allows to build soil models with quite uniform distribution of relative density with depth. Mo (2014) provided experimental evidence for the uniformity of the soil models achieved with this method by performing Cone Penetration Tests (CPT) in dry sandy models, at Ng conditions, prepared by dry pluviation. Furthermore, by adjusting the flow rate and drop height of the sand, the achieved dry density of the model can be controlled. Finally, this method provides satisfactory levels of repeatability concerning the density of the models (Mo, 2014).

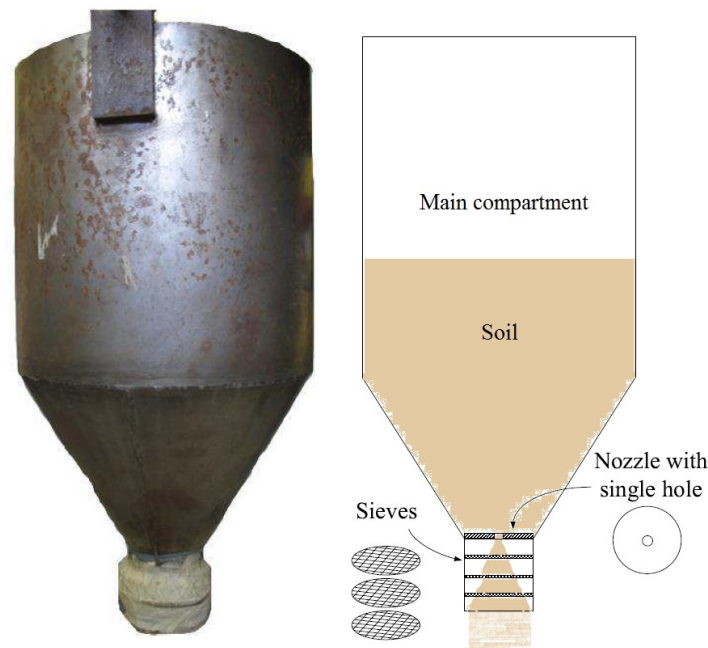


Fig. 4.46 Schematic of the single-holed sand hopper (Mo, 2014).

The sand hopper used for dry pluviation during this study is shown in Figure 4.46. It consists of the main compartment which was semi-filled with the soil, a nozzle with a single hole (orifice) and a series of sieves. The size of the hole controls the flow rate of the sand and the sieves allow the soil to distribute more evenly in the strong box. In this study, two different orifice diameters were used, 5 mm and 9 mm with average flow rates of 0.239 kg/min and

1.048 kg/min, respectively. The drop height was adjusted by moving the sand hopper vertically using a crane. Also, it could move horizontally and, therefore, spread the soil inside the testing box. Loose samples ($D_R = 50 \% \pm 10 \%$) were prepared using the large nozzle with a pouring height of 0.5 m, while dense samples ($D_R = 90 \% \pm 5 \%$) could be achieved with the small nozzle with 1 m pouring height. The accurate overall dry density was determined at the end of the pouring procedure by calculating the total mass of the sand and the volume it occupied. Prior to the preparation of each model, calibration took place in order to define the correct flow rate and drop height for the desired dry density. Pluviation of the dry sand took place inside the strong box up to a height of 300 mm from the bottom of the box. The dimensions of the soil block were $600 \times 300 \times 200$ (length $\times$ height $\times$ width).

Saturation of the soil block

The dry soil block was saturated from the bottom using the drainage valves and the standpipes described in Section 3.2.3. The water level, inside the standpipes, was adjusted so as to keep the upward hydraulic gradient in the soil to a low level. Consequently, there was no disturbance of the upward flux to the soil mass. The saturation of the soil block was ended as soon as the soil surface was submerged under the water level.

Desaturation

After the saturation of the soil block, the connection with the standpipes was stopped and no more water was allowed to flow in the box. The drainage valves at the bottom of the box were then opened. Subsequently, the excess water was drained out of the soil model and the soil mass was left in an unsaturated state. In order to accelerate the process, positive air pressure of 20 kPa was applied to the testing box for about 8 hours. The desaturation of the soil mass enabled uniform soil suction in the slope following drainage. After 8 hours, the soil block

remained in unsaturated state with an increased shear strength. Consequently, a slope was able to be shaped at an angle of 50° , higher than the angle of friction (32°).

Slope cut

Slopes were formed using a long aluminium piece with a sharp edge. An aluminium angle piece was fixed at the top of the centrifuge box in order to be used as a guide for the cutting tool. From the part of the soil which was removed, samples were taken and oven dried to determine the initial mean gravimetric moisture content. The final slope angle was determined by using a digital inclinometer on the slope face, making sure that it was within $\pm 2^\circ$ of the desired angle and keeping the consistency of the slope dimensions for all tests.

Final adjustments and load into the centrifuge platform

After the slope was shaped, the soil surface was covered by sheets of fine mesh (Figure 4.47) to prevent the erosion of the slope due to the impact from rain droplets at 60g. This was a stainless steel mesh with wire diameter 0.036 mm and an aperture of 0.058 mm, provided by The Mesh Company Ltd.

After the final adjustments, the strong box was weighed and loaded onto the centrifuge platform together with the auxiliary equipment (standpipes, digital camera and top lid). The counterweight of the centrifuge was set to the correct setting, based on the final weight of the apparatus. All the sensors were connected to the junction box and a final check on their performance was made using the Labview code. Water supply, coming from the slip-rings of the centrifuge, was connected to the rainfall distribution system through a 10 mm pipe. Finally, the two standpipes were connected to the centrifuge box through the side valves.

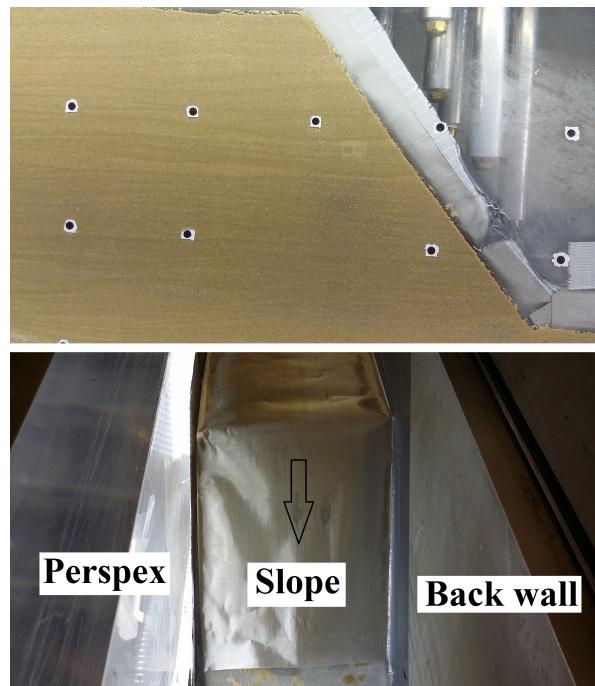


Fig. 4.47 Use of a fine mesh to protect slope surface from erosion.

4.3.2 Silty sand models

Silty sand slope models could not be prepared using the dry pluviation method due to the wide range of particle sizes. The free fall of particles with varied size, from fine sand to clay, can lead to grain segregation due to the differences in velocity. Consequently, the deposited soil would have different consistency from the soil of interest. For this reason, moist tamping was used for preparing silty sand models.

Prior to model preparation, dry soil was mixed with water at the optimum moisture content. Then, it was placed in bags and left sealed for more than 24 hours to equalise. Afterwards, it was compacted in layers within the strong box until the upper surface reached the 300 mm height indicator. The soil block was then left overnight in the strong box in order to equalise. The slope was formed again following the same procedure as in the sandy slope models, using

the sharp-edged aluminium piece and the angle guide.

The compaction of the soil within the strong box was made in 10 layers, each layer having a final thickness of approximately 30 mm. Compaction was performed with the small hammer which was used for the light compaction method, described in Section 4.2.5. A wooden plate with dimensions 600 mm $\times$ 200 mm (*length* $\times$ *width*) was placed on the soil surface to ensure that the applied energy was evenly distributed. Each layer received five blows, one at the centre of the plate and four at the sides.

4.3.3 PPT installation

PPTs were buried within the slope model and their exact locations were decided prior to the model preparation. During the soil deposition, either using the dry pluviation or moist tamping methods, PPTs were placed at the predefined locations while their porous stone were covered with a saturated silty paste in order to keep them moist and not cavitate.

4.4 Testing programme

The current experimental research study consists of two centrifuge testing programmes. The first includes the centrifuge tests on soil slope models under varied rainfall and boundary conditions and it is shown in Table 4.8. Two soils were used for these tests, LB fraction E sand and M_{90-10} soil mixture. Slopes made of LB fraction E sand were prepared at two different densities, representing a loose (tests 1.1-1.3) and a dense state (tests 2.1-2.3). Both were subjected into varied rainfall conditions, with rainfall intensity ranging between 7.4 mm/hr (or 2.05×10^{-6} m/s, for tests 1.1 & 2.1) and 87.8 mm/hr (or 2.44×10^{-5} m/s, for tests 1.3 & 2.3). Moreover, the case of an elevated ground water table was simulated into the loose and dense slopes (tests 1.4 and 2.4). All slope models were prepared at an inclination of 50° and a

height of 150mm. An overview of the slope models tested in the geotechnical centrifuge was presented in Figure 3.5. The centrifuge acceleration at which the rainfall was applied was 60g; therefore, the prototype height was 9m. M_{90-10} soil slope models were prepared at almost identical packing, while high (test 3) and low (test 4) rainfall intensities were applied to them. In all tests, apart from tests 1.4 and 2.4, the groundwater table was kept horizontal and deep below the slope, at a small depth below the slope toe by applying drainage conditions.

The second testing programme includes centrifuge tests performed in soil column models of varied grain size distribution for the determination of the retained water content at different g. The testing programme for these tests as well as the results are presented in Chapter 6. The objective of this testing programme was to study the initiation of negative pore water pressures during the centrifuge spinning and their distribution during rainfall. Soils with different permeabilities were used and the effect of different seepage velocity to the elimination of suction at rainfall conditions was studied. The results from the two testing programmes are combined and discussed in Chapter 7.

Table 4.8 Centrifuge testing programme of soil slopes. Prototype rainfall intensities were 7.4 mm/hr or 2.05×10^{-6} m/s (low), 27.2 mm/hr or 8×10^{-6} m/s (medium) and 87.7 mm/hr or 2.44×10^{-5} m/s (high).

Test	Code	Relative density	Rainfall intensity	Soil	Chapter
1.1	CSA8.1	52%	Low	LB fraction E	5
1.2	CSA8.2		Medium		
1.3	CSA8.3		High		
2.1	CSA9.1	94%	Low		
2.2	CSA9.2		Medium		
2.3	CSA9.3		High		
1.4	CSA8.4	52%	-		
2.4	CSA9.4	94%	High		
3	CSI1	57 %	High	M_{90-10}	7
4	CSI3	65 %	Low		

Chapter 5

Slope model tests

This chapter presents the initial findings from slope stability tests in sand models at Ng. The simulation of two failure mechanisms was attempted: loss of suction due to wetting with subsequent instability and static liquefaction. Slopes were tested under varied initial and boundary conditions. Due to the poor performance of the Druck PPTs, which were used for measuring suction in the slope models, pore water pressure measurements are not presented. Therefore, test results include only the deformation of the slope models during the tests.

All distances and timings presented in this chapter concerning the results from the centrifuge tests are in model scale unless stated otherwise.

5.1 Introduction

Centrifuge slope tests were divided into two groups, based on the applied boundary conditions. In the first group, rainfall conditions were applied to the slope models at three different intensities. The groundwater table (GWT) was set at a constant depth, below the slope toe during the tests. This case is characterised as "deep GWT". Rainfall did not cause its elevation to rise due to the drainage conditions applied at the side of the model. At the same time, the soil mass above the GWT experienced wetting and the degree of saturation, S_r , increased. Suction,

s, is responsible for retaining the slope at an angle higher than the angle of friction. Upon wetting, suction reduces, with implications to the effective stresses in the soil.

The second group of tests involved a shallow groundwater table formed within the slope. By restricting flow out of the upslope drainage box, the GWT was raised either during rainfall (test CSA8.4) or without rainfall (test CSA9.4). These cases are characterised as "shallow GWT". The water was added to the specimen by simulated rainfall caused the GWT to gradually rise, becoming closer to the slope surface due to and during rainfall simulation. In rainfall conditions, rain water fed the groundwater table while in static conditions water head was increased at the upslope side by inserting water to the upslope standpipe. The difference in head between the two sides of the model caused an increased hydraulic gradient and, therefore, initiated flow conditions within the slope.

5.1.1 Pore water pressure and deformation measurements

The measurement of pore water pressures (PWP) in slope models was performed using the two types of tensiometers (Druck PPTs and EPB-PW tensiometers) which were described in Section 4.3.1. These sensors were used successfully for the measurement of suction in sandy specimens and the determination of the SWRC at 1g, as described in Section 4.2.8. However, their performance at increased gravity conditions was not satisfactory. During centrifugal acceleration, the soil mass was desaturating extensively, something which caused these sensors to cavitate early, before the model was reached 60 g acceleration. Figure 5.1 presents the layout of a slope model with three installed PPTs at different depths. The measurement of the PWPs for the particular model are plotted and shown in the same figure.

The initial pore pressure at the three locations was close to 0 kPa revealing a condition near saturation. At the moment of the gravity increase, PWPs at all three locations started to

decrease. When PWP readings reached a value between -3 and -4 kPa, the readings returned to values close to the initial readings. This increment did not occur simultaneously for all sensors but in the following order: first at PPT01 which was located near the crest and at the lowest vertical distance from the soil surface, then at PPT09 located in between the other two PPTs and finally at PPT05 which was installed at the highest vertical distance from the surface. The increment caused the pore pressures to rise to a value about 1 kPa lower than the initial pressure. Pore pressures remained constant for the rest of the test with only minor variations.

The initial change of PWP readings at gravity increase is attributed to the desaturation that is caused to the soil mass due to the increased gravity conditions. As the water moves downwards, the S_r of the soil mass reduces and, therefore, s starts to increase. In this way, a zone of low S_r starts to form from the soil surface and moves downwards to deeper layers. The relationship between S_r and s is given by the SWRC of the soil (Section 4.2.8). Therefore, it was expected to observe continuous increments of s as the g level increased and desaturation progressed. However, it seems that starting from the PPT closest to the surface of the model (i.e. PPT01), desaturation becomes higher and the contact between the pore water and the PPT head (porous stone) is lost. Therefore, cavitation of the tensiometer occurred and the measurement of s became unreliable.

This example shows that the performance of the PPTs for measuring suction in sandy soils, which are subjected to centrifugal acceleration, is poor. Tensiometers start to cavitate even at low g -levels. The residual S_r of fraction E at 60 g was investigated further by conducting element centrifuge tests in soil columns. The results from these tests are presented in Chapter 6. Moreover, a new technique for preventing PPT cavitation under these conditions was studied and validated during the soil column tests. The technique was applied to the slope model tests which are presented in Chapter 7.

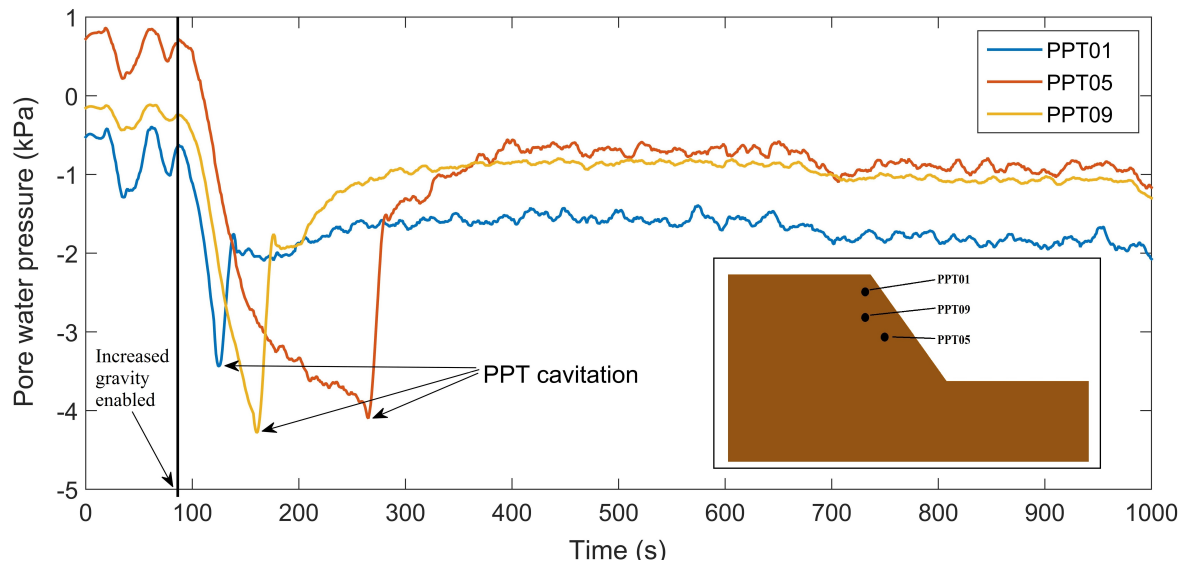


Fig. 5.1 Cavitation of PPTs installed in fraction E sand due to gravity increase.

In the centrifuge slope tests which were conducted with fraction E sand and presented in this chapter, suction measurements were not reliable. Therefore, the results of the tests concerned only the deformations that occurred before, during and after rainfall conditions.

The deformation field of the slope models was determined with the use of geoPIV and PIVlab software. Due to the nature of the physical modelling tests, which involves the desaturation of the soil mass at Ng, captured images often suffered from lack of consistent texture. This affected the PIV analysis and resulted in the development of a large number of wild vectors. In order to avoid this phenomenon, many techniques were used and many parameters were altered during the PIV analysis. Also, several trials took place, testing different values for the patch size, overlap, leapfrog, search zone, number and sequence of images, until the best possible result was obtained. However, the results in some tests were suffered by large numbers of wild vectors which had to be eliminated in order to avoid large patches of unreliable data on the strain distribution.

The gaps from the deleted vectors were filled using an interpolation method, as this was described in Section 3.4. In order to achieve consistent results, the interpolation method was applied to all PIV analyses run for the slope model tests.

5.2 Rainfall effect in slopes with deep groundwater table

In this section the results from the centrifuge tests on soil slopes under rainfall conditions with a deep, horizontal water table are presented. The boundary conditions applied to the models are illustrated in Figure 5.2. At both sides of the model, drainage was open by using the side valves of the centrifuge box. Groundwater table remained horizontal throughout the test procedure by overflow ports in the standpipes. The depth of the groundwater table was set at 10 mm below the slope toe. Therefore, the soil profile above the GWT remained in unsaturated conditions during the test.

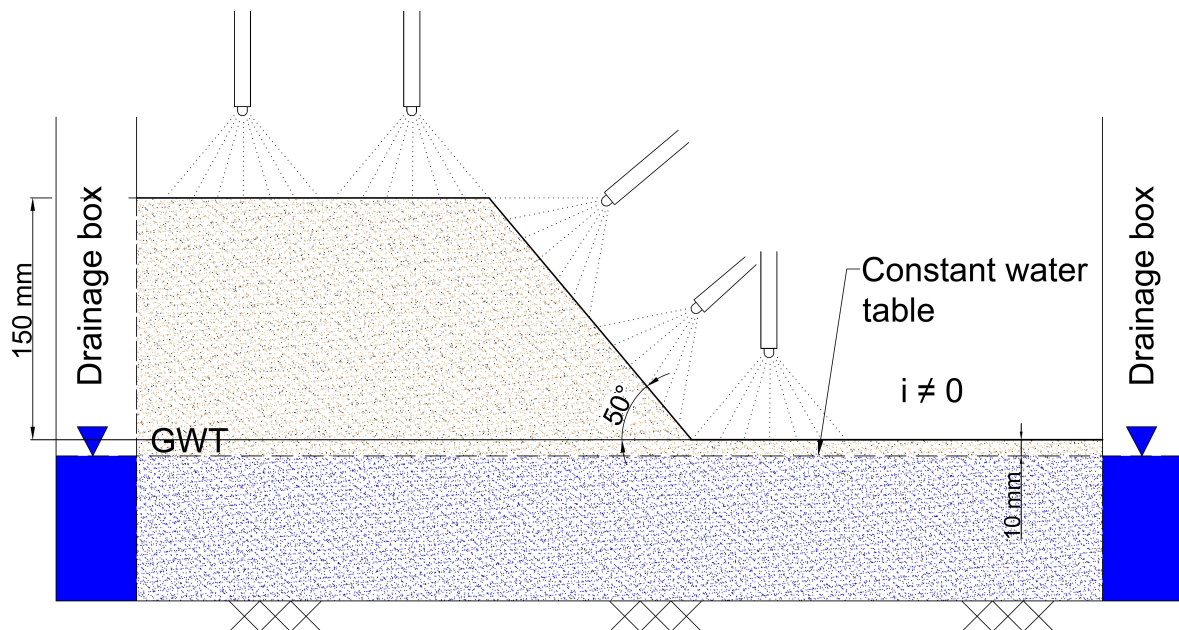


Fig. 5.2 Boundary conditions for the slope models tested under a deep ground water table condition

The inclination of all slope models was 50° and the height of the slopes was 150 mm. At the testing acceleration (60g), the corresponding prototype height is 9 m. The slope models were prepared under two different densities, representing a loose and a dense state. The loose slopes had an initial $I_d = 52\%$ ($\pm 2\%$) (tests CSA8.X) and dense slopes an initial $I_d = 94\%$ ($\pm 2\%$) (tests CSA9.X). Each model was subjected to three different rainfall intensities. The low intensity was 2.05×10^{-6} m/s (tests CSAX.1), medium intensity was 8×10^{-6} m/s (tests CSAX.2) and the high intensity was 2.44×10^{-5} m/s (tests CSAX.3), all in prototype scale. The maximum duration of each rainfall event was set to 30 min (in model scale) if failure would not occur, which represents a prototype duration of 1800 h or 75 days. Therefore, extended wet seasons were simulated in these tests.

5.2.1 Effect of gravity increase

Slope models were brought to the target centrifugal acceleration (i.e. 60g) in steps, pausing at 10g, 20g, 30g, 40g and 50g. The model was left at each g level for 5-10 min in order to allow dissipation of the excess pore water pressures. Thus, loading increments could be considered as drained. The desaturation of the soil mass due to g increment and the formation of the GWT below the slope toe was observed through the Perspex window.

The effect of the increased stresses, during the transition from 1g to 60g, within the loose slope model is shown in Figure 5.3. The initial dry density of the soils (in average) was $\rho_d = 1.46 \text{ Mg/m}^3$, the void ratio $e = 0.81$ and the relative density of the slopes was $I_d = 52\%$. Displacement vectors reveal that the soil mass behind the slope crest settled vertically downwards by approximately 2 mm. This settlement caused the change of the soil density, with the average values at 60g become $\rho_d = 1.47 \text{ Mg/m}^3$, void ratio $e = 0.8$ and relative density $I_d = 56\%$. Near the slope surface, displacement vectors had different orientation and became parallel to the slope. However, this settlement did not cause any failure of the slope model.

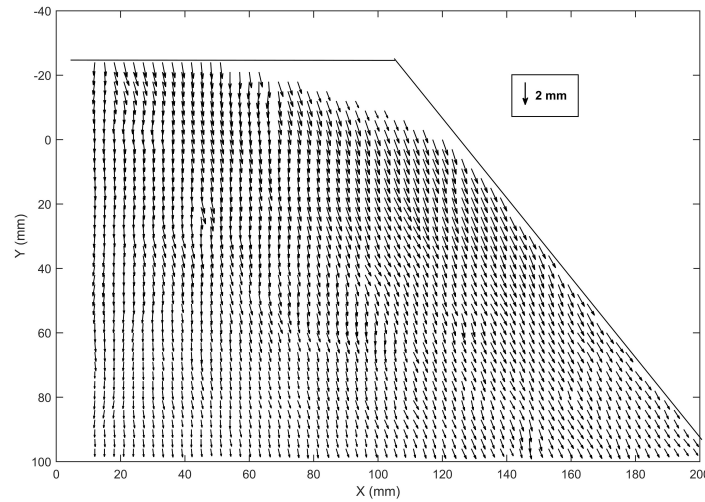


Fig. 5.3 Displacement vectors in the loose slopes at gravity increase from 1g to 60g (for tests CSA8.1 to CSA8.4).

After the end of the model preparation at 1g, the soil density of the dense slope was $\rho_d = 1.588 \text{ Mg/m}^3$, void ratio $e = 0.669$ and relative density $I_d = 94.4\%$. The maximum settlement in the dense slope models during spin-up to 60g was approximately 0.5 mm (Figure 5.4). This caused the change of dry density to $\rho_d = 1.591 \text{ Mg/m}^3$ and relative density at $I_d = 95.6\%$. Displacement vectors near the crest were again parallel to the slope but they become vertical near the toe.

5.2.2 Effect of rainfall intensity on loose slopes

The deformation results from tests CSA8.1, CSA8.2 and CSA8.3 are presented in this section. Deformation analysis was performed using geoPIV version 8 and involves the comparison of the slope profile just before the initiation of rainfall and immediately after the end of rainfall.

Low intensity - Test CSA8.1

Deformation of the loose slope under light rainfall is shown in Figure 5.5. Displacement vectors reveal that the sandy slope experienced swelling at the end of the rainfall simulation. The horizontal displacement contours show that swelling is more pronounced in the upper layers of

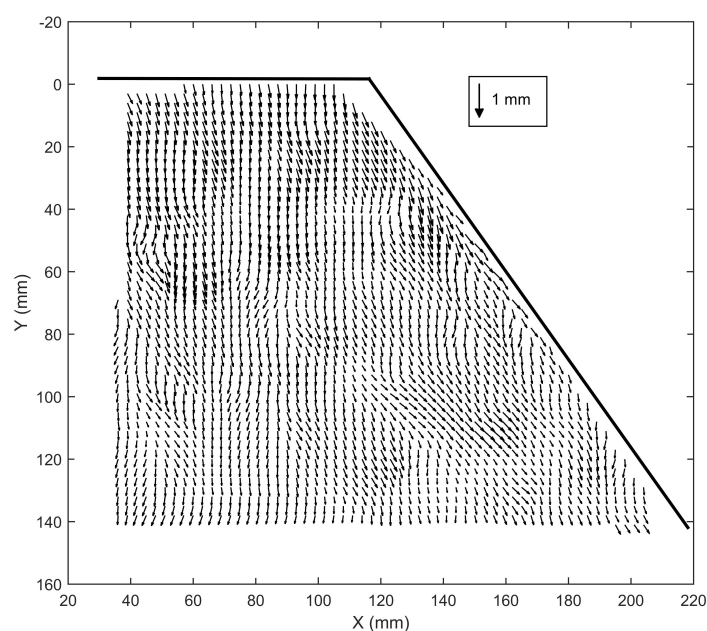
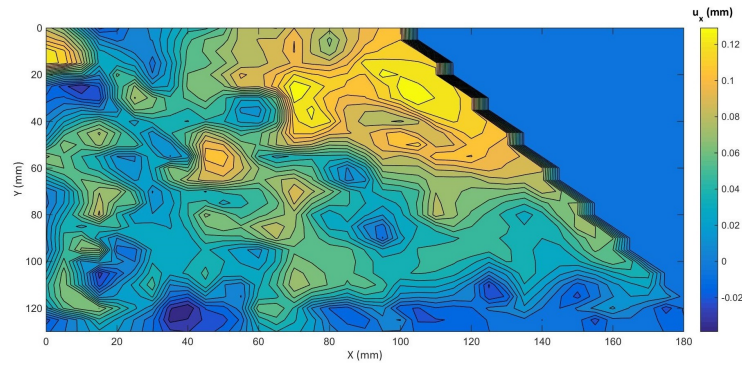


Fig. 5.4 Displacement vectors in the dense slopes at gravity increase from 1g to 60g (for tests CSA8.1 to CSA8.4).

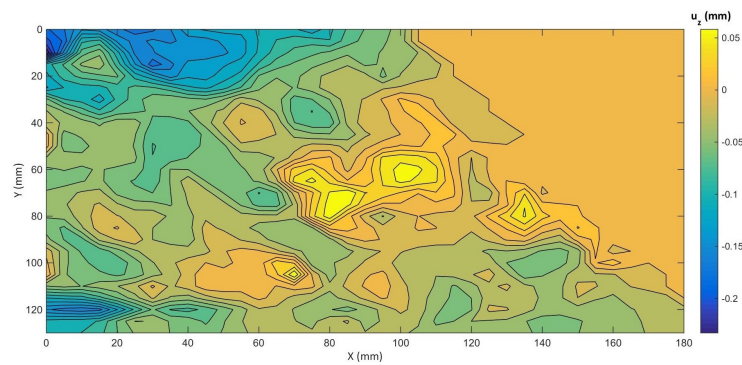
the slope, near the crest. The maximum displacement measured in the slope was 0.12 mm. The extended wetting period, with low rainfall intensity, was not sufficient to drive the soil slope to global or local failure. Shear and volumetric strain of the slope at the end of the rainfall period is shown in Figure 5.6.

Medium intensity - Test CSA8.2

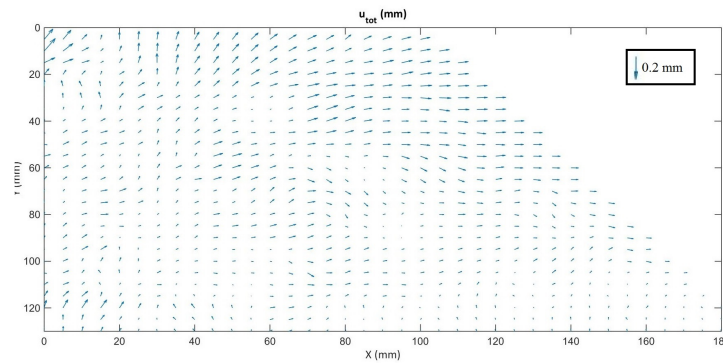
Very small displacements were also measured after the medium intensity event. The results are shown in Figure 5.7. In this case, the slope model did not experience swelling as in the case of the light rainfall simulation. Displacement vectors showed that the soil mass settled almost parallel to the slope face. However, the maximum displacement was only 0.1 mm. The results indicate that extended rainfall periods with medium rainfall intensity were not sufficient to cause significant disturbance to the slope. Again, no global or local failures occurred. Shear and volumetric strain of the slope at the end of the rainfall period is shown in Figure 5.8.



(a)



(b)



(c)

Fig. 5.5 Deformation of loose slope after low rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.1).

High intensity - Test CSA8.3

In the case of the highest rainfall intensity, the slope model experienced more intense deformation at the end of the wetting period. The results from the geoPIV analysis are shown in

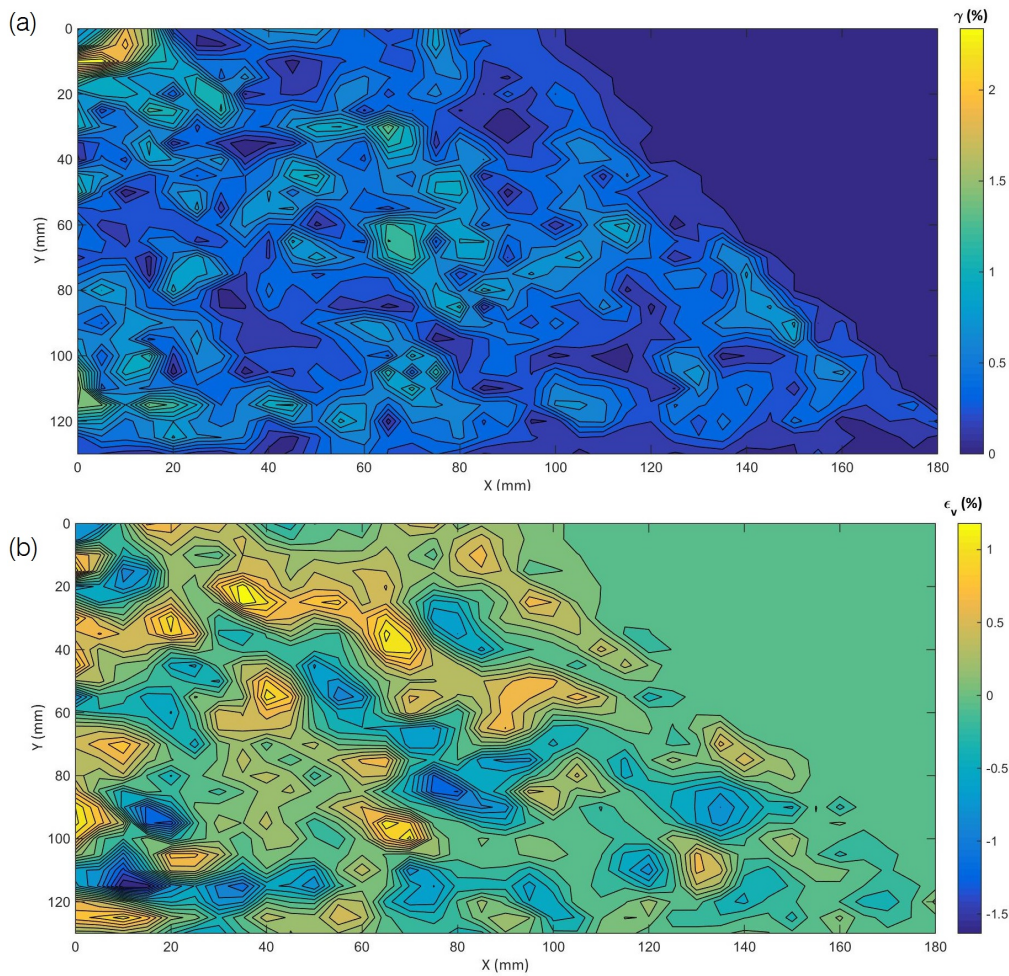
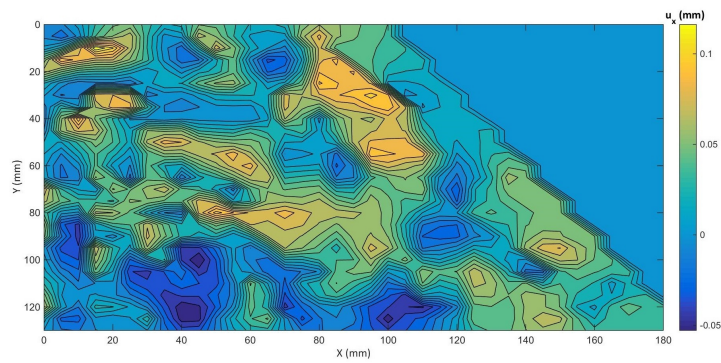
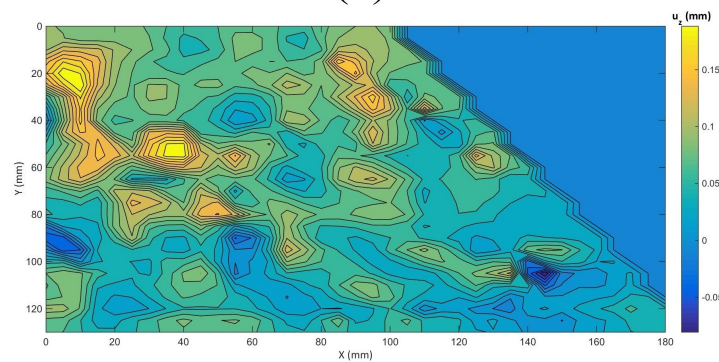


Fig. 5.6 (a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.1)

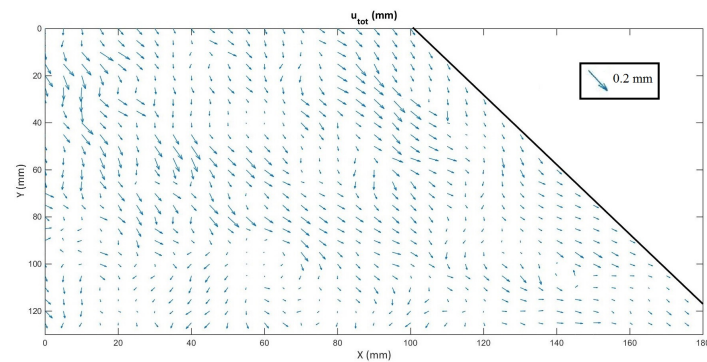
Figure 5.9. Horizontal displacement reached a maximum of 0.3 mm near the slope toe. Vertical displacement was found to be higher at the area near the slope crest and reached a value close to 0.5 mm, while it was reduced near the toe. This behaviour is highlighted from the total displacement vectors shown in Figure 5.9c. The soil mass seems to settle almost vertically at higher layers and change to a more horizontal orientation at deeper layers and close to the slope toe. Moreover, displacements show a gradual increase moving towards the slope face. Shear and volumetric strain of the slope at the end of the rainfall period is shown in Figure 5.10.



(a)



(b)



(c)

Fig. 5.7 Deformation of loose slope after medium rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.2).

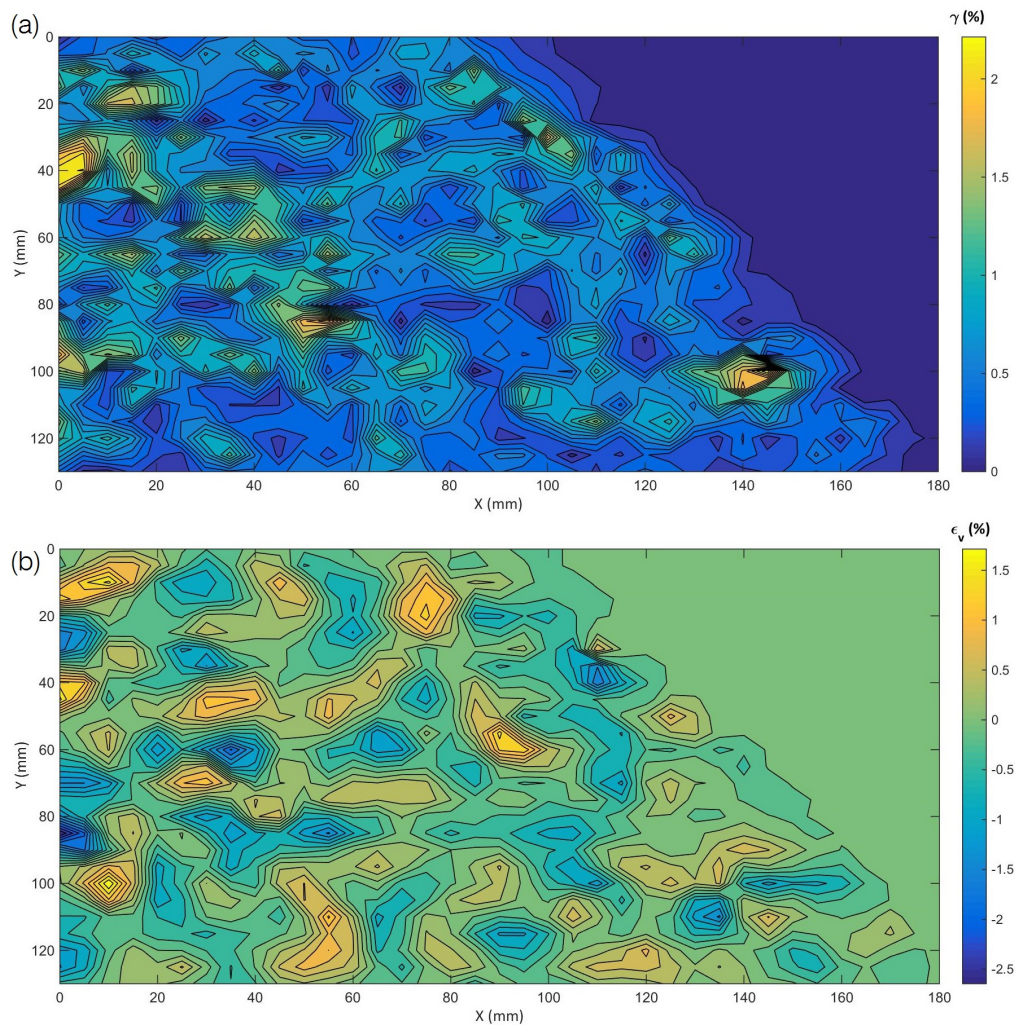
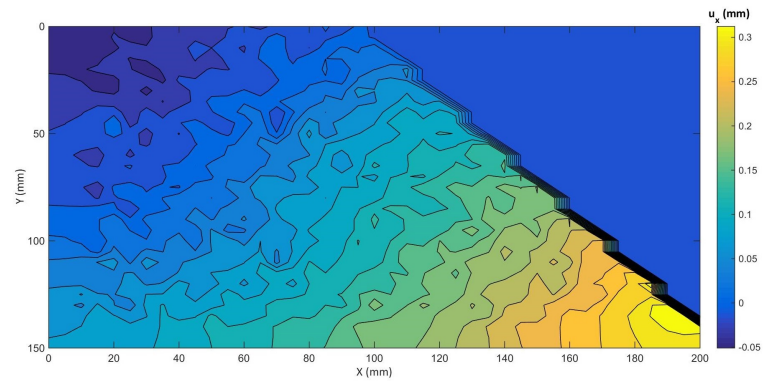


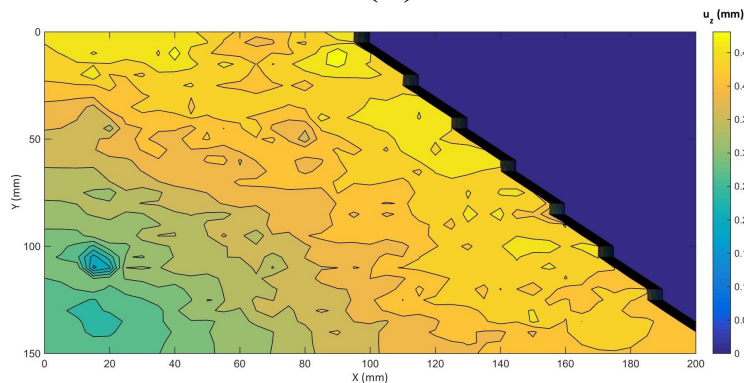
Fig. 5.8 (a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.1).

5.2.3 Effect of rainfall intensity in dense slopes

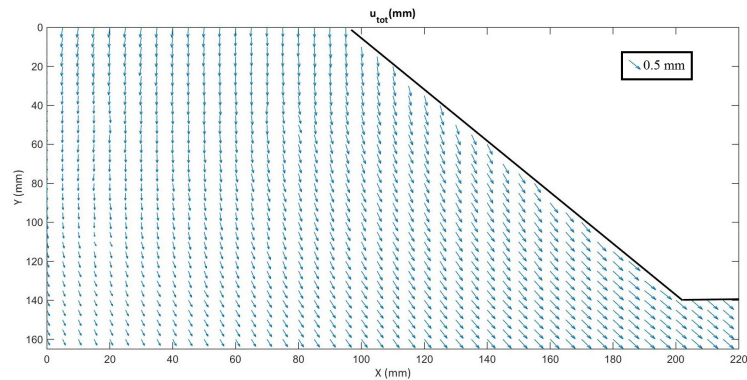
The deformation results from tests CSA9.1, CSA9.2 and CSA9.3 are presented in this section. Deformation analysis was performed using geoPIV version 8 and involves the comparison of the slope profile just before the initiation of rainfall and immediately after the end of rainfall period.



(a)



(b)



(c)

Fig. 5.9 Deformation of loose slope after high rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA8.3).

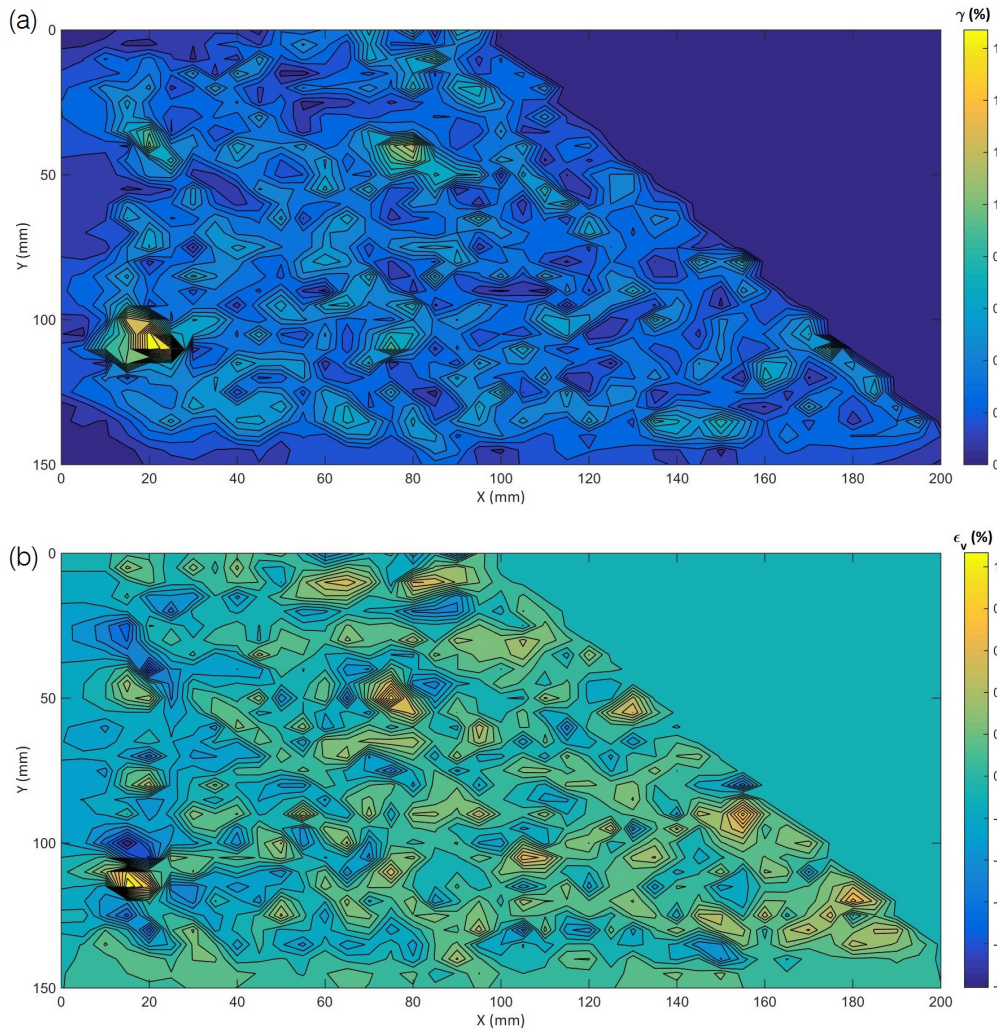


Fig. 5.10 (a) Shear strain and (b) volumetric strain contours for the loose slope model at the end of the rainfall period (test CSA8.3)

Low intensity - Test 9.1

The effects of the low rainfall intensity on the dense slope model are shown in Figure 5.11. Horizontal displacement is higher at the bottom half of the slope profile while vertical displacement is dominant in the upper half. The measured displacements were found to be greater than in loose slopes. The total displacement vectors reveal that the slope showed a tendency to create a circular failure. However, the level of displacement was not sufficient to cause a slope

failure. This is also highlighted from the level of shear and volumetric strains which remain low at the end of the rainfall period (Figure 5.12).

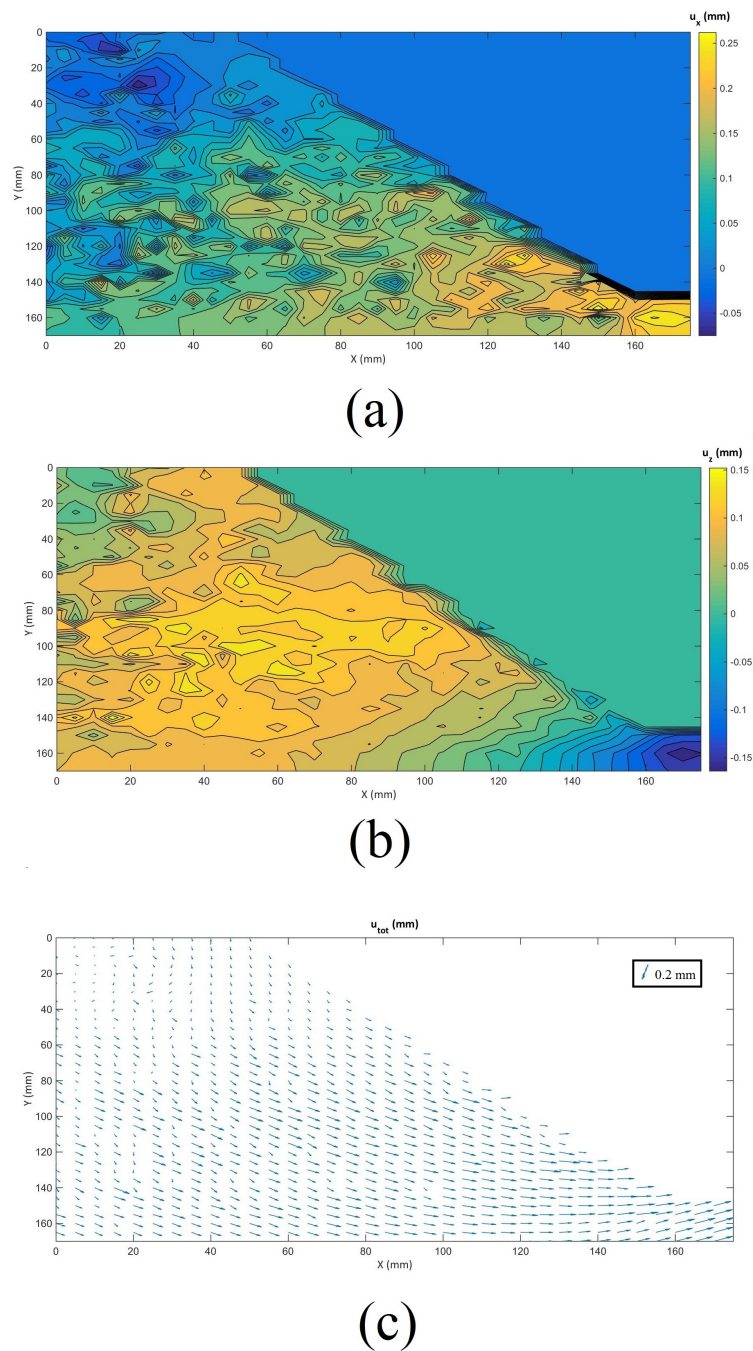


Fig. 5.11 Deformation of dense slope after low rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.1).

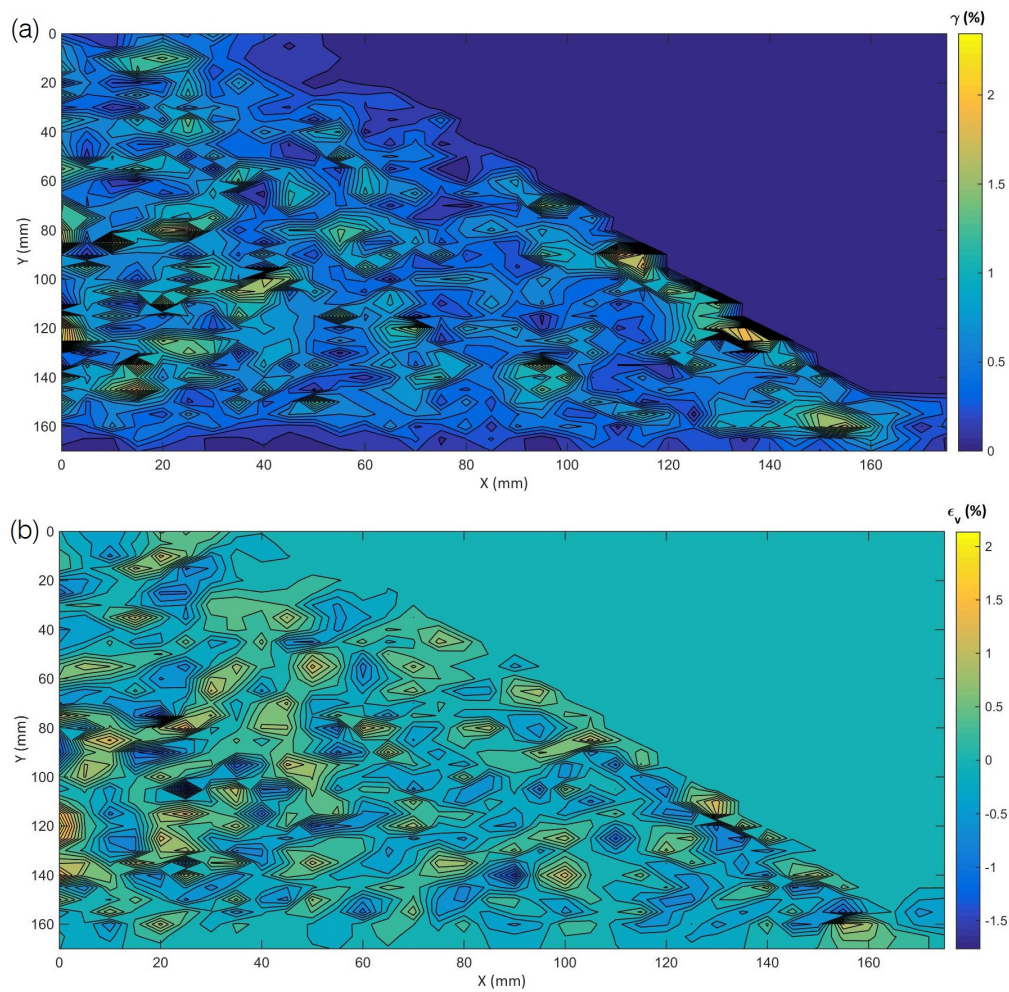
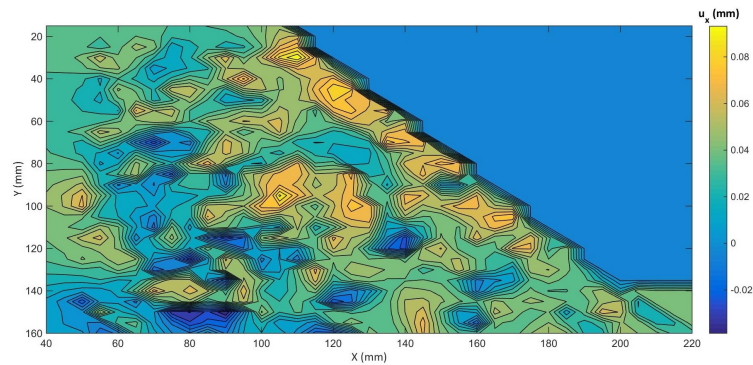


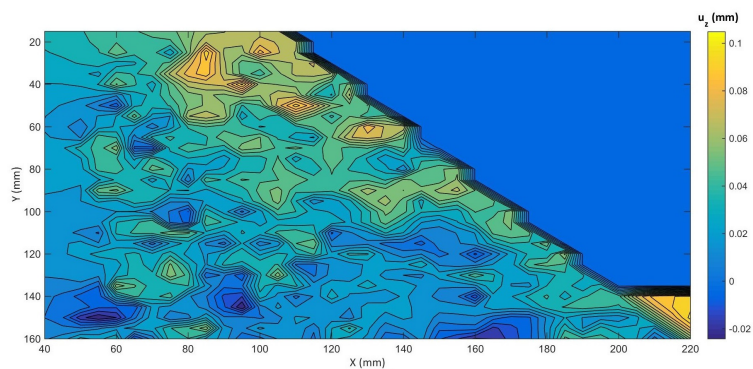
Fig. 5.12 (a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.1)

Medium intensity - Test 9.2

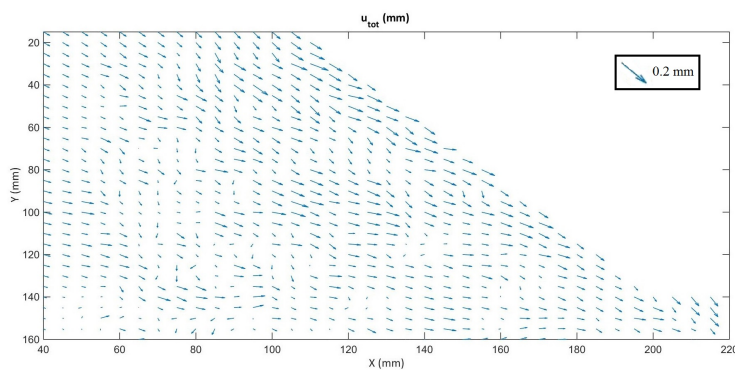
Under medium rainfall intensity, the dense slope model experienced very low displacements. These are shown in Figure 5.13. Even though the displacement vectors are parallel to the slope, revealing a tendency for failure, the deformation is not so high in order to initiate instability. Shear and volumetric strain of the slope at the end of the rainfall period is shown in Figure 5.14.



(a)



(b)



(c)

Fig. 5.13 Deformation of dense slope after medium rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.2).

High intensity - Test 9.3

The deformation of the dense slope under the most intense rainfall event is shown in Figure 5.15. The distribution of the horizontal displacements shows an increased deformation near

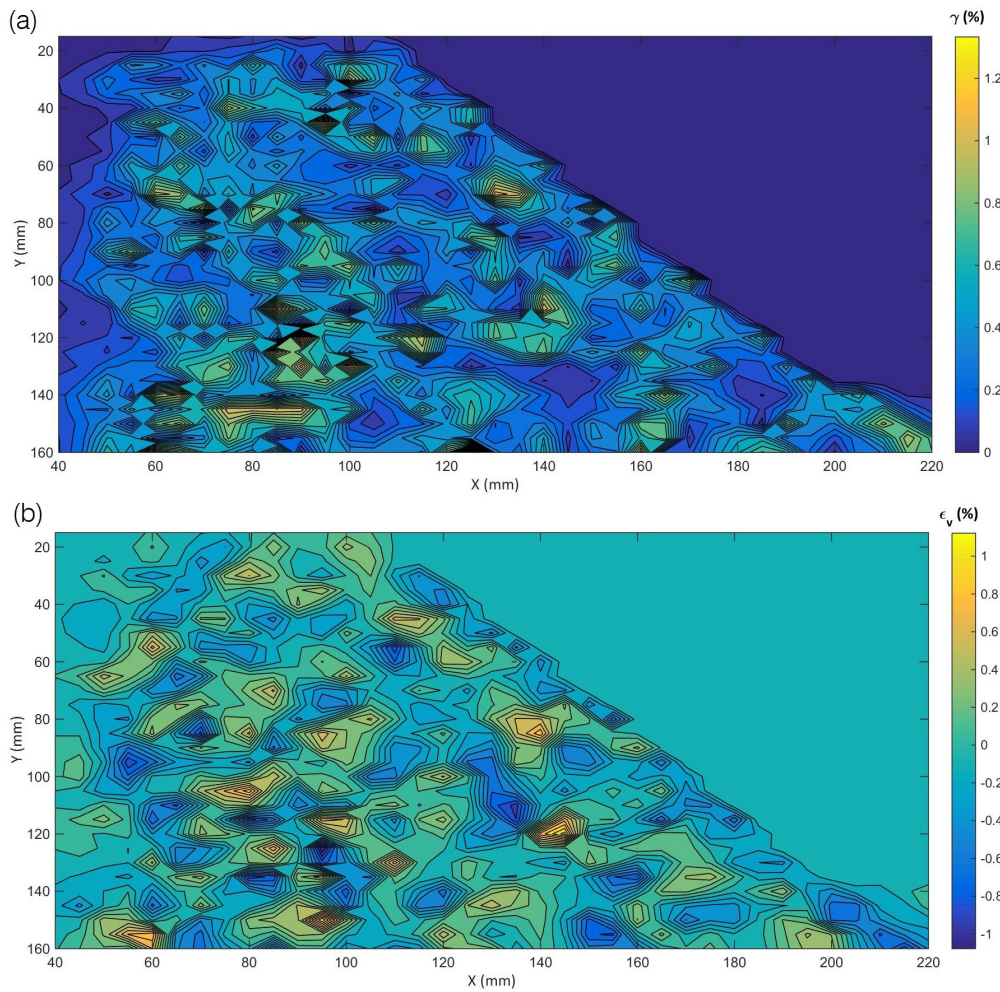


Fig. 5.14 (a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.2).

the slope toe. Rainfall caused an increased vertical displacement at the slope crest. The level of strain, both the shear and volumetric, was again sufficiently low and therefore no global or local failure occurred. Shear and volumetric strain of the slope at the end of the rainfall period is shown in Figure 5.16.

5.2.4 Summary of tests with deep GWT

The centrifuge modelling of loose and dense slopes under varied rainfall conditions for deep horizontal GWT revealed that the deformations were not significant. Despite the high incli-

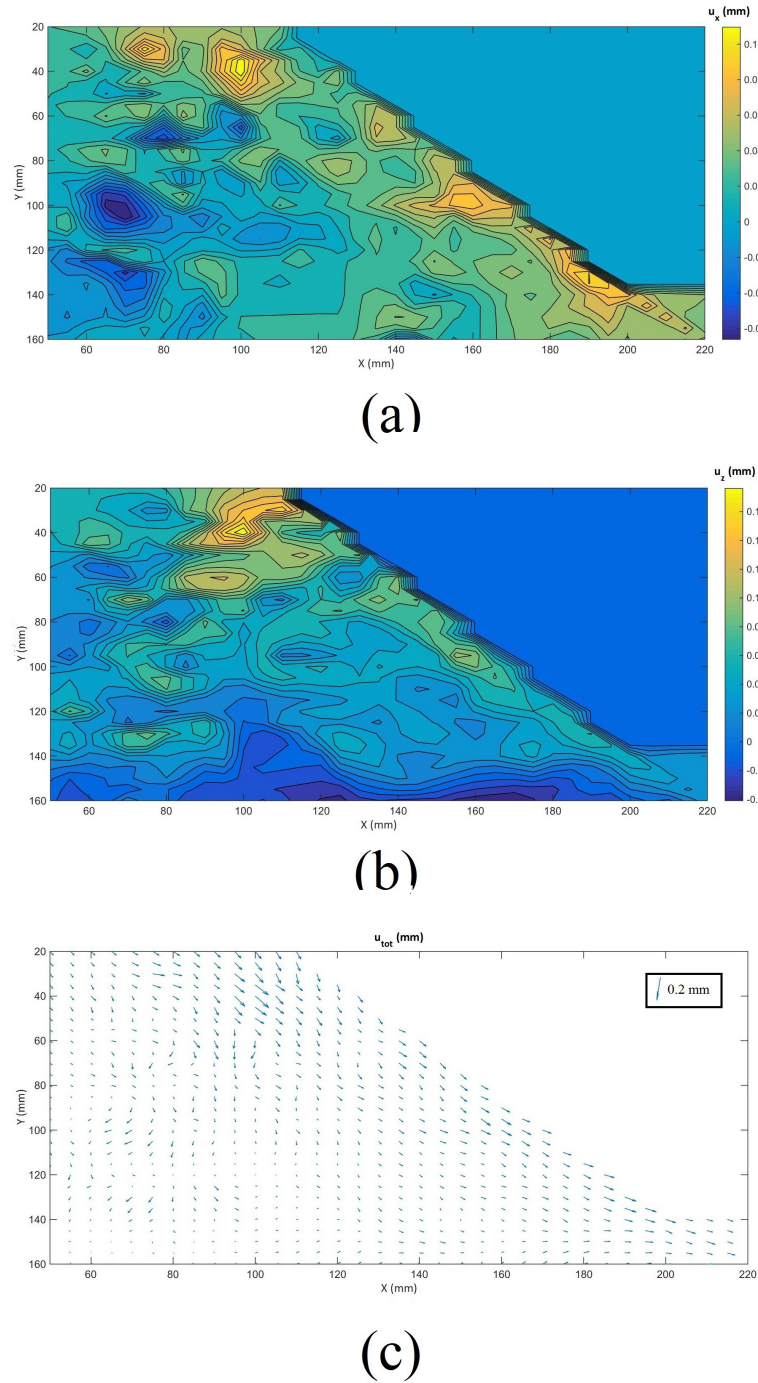


Fig. 5.15 Deformation of dense slope after high rainfall intensity: (a) horizontal displacement contours (b) vertical displacement contours (c) total displacement vectors (test CSA9.3).

nation of the slope, the high intensity of the rainfall and the long duration of the wet season, slopes did not experience any global or local failures. The prototype rainfall conditions which

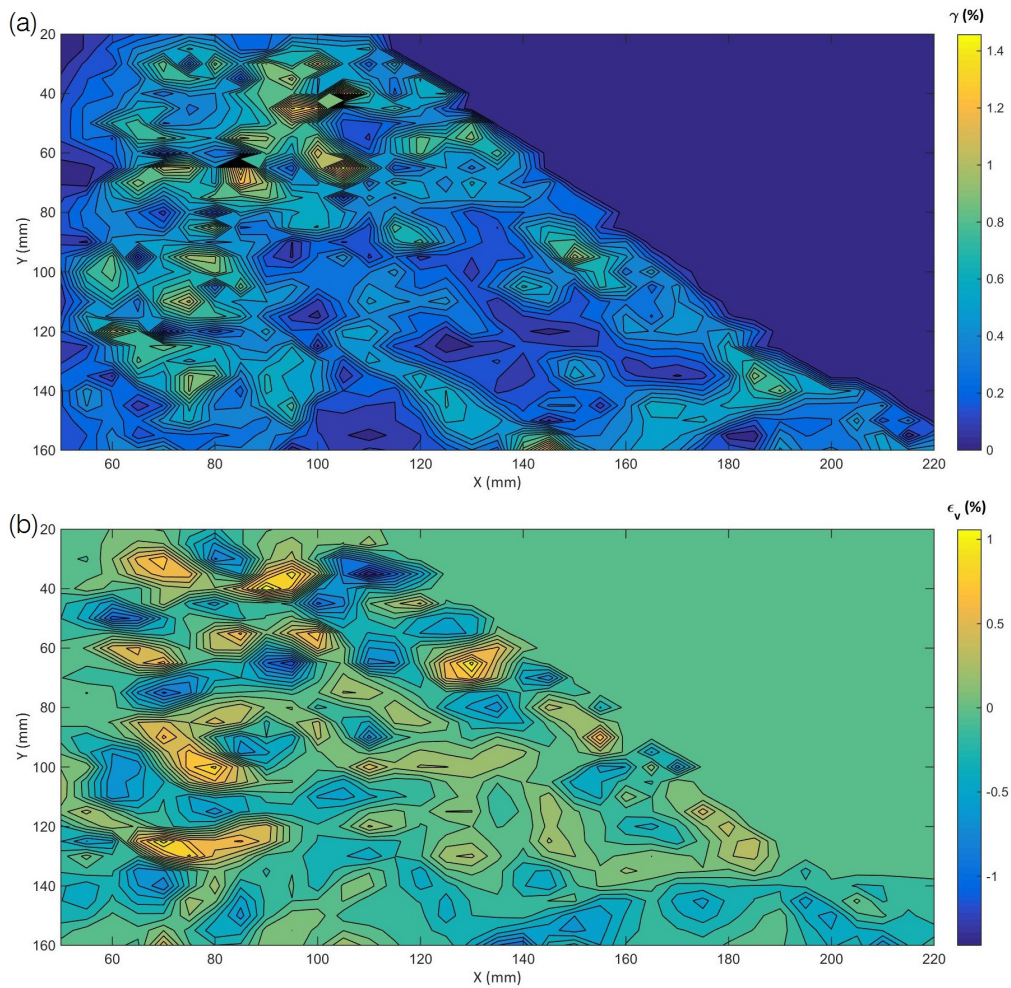


Fig. 5.16 (a) Shear strain and (b) volumetric strain contours for the dense slope model at the end of the rainfall period (test CSA9.3).

applied to the slopes were extreme and sometimes unrealistic. Moreover, the inability for suction measurements from the PPTs in the sandy models did not allow for the determination of the initial pore water pressures and the impact of rainfall infiltration.

Centrifuge modelling of granular soil slopes under extreme rainfall conditions showed that large deformations do not occur. This behaviour can be attributed to the increased seepage velocity in the centrifuge model due to increased gravity, which accelerates the dissipation of the excess PWP, inducing drained conditions. Therefore, collapse is prevented if the same soil

and pore fluid viscosity are used in the centrifuge modelling procedure. This issue is discussed further in Chapter 7.

Similar observations in sandy slopes have been made in the past. Min (2006) conducted centrifuge tests in loose slope models, made of Leighton Buzzard fraction E sand, at an angle of 30° . The simulated rainfall intensity was either 8.7×10^{-6} m/s and applied for 10.4 weeks duration or 1.8×10^{-5} m/s and applied for 2.3 weeks (in prototype scale). Under these conditions, slope models did not experience any local or global failure. The same tests were repeated with the groundwater table allowed to rise horizontally. Even under full submergence under the water level, slopes did not experience failure. Take et al. (2004) conducted centrifuge tests in 45° slopes made of completely decomposed granite in order to investigate the triggering mechanism of static liquefaction. After applying rainfall with an intensity of 1.2×10^{-6} m/s, the slope model experienced only minor settlement at the upper layers but not an instability. The authors stated that static liquefaction was unlikely to occur in uniform unsaturated slope models due to the high compressibility and mobility of air in the unsaturated void spaces. Therefore, undrained failure is restricted by the void spaces which allow the slope to accommodate wetting collapse.

A dry slope made of cohesionless soil will be able to remain stable only if its inclination is less than the soil friction angle. Since the inclination of the tested slope models was almost 1.5 times higher than the friction angle of fraction E (i.e. $\beta = 50^\circ$, $\phi = 32^\circ$), it would be expected to experience extended deformation soon after the soil was left to drain at high g . However, this was not the case and reveals that even a coarse soil does not become completely dry when it is tested in the centrifuge. This implies that the coarse soil remains under unsaturated state and suction contributes to its shear strength. Therefore, it is important to understand what is the residual water content and degree of saturation of the soil, at high g , and how this moisture affects soil strength. This issue is addressed in Chapter 6.

Moreover, conventional wisdom predicts that such a slope will fail if seepage flows parallel to the slope and its inclination exceeds half of its critical state friction angle (Take, 2003). This is the case at 1g conditions, however the effect of the increased gravity should also affect the flow paths of the infiltrated water. To understand the consequences of centrifuge modelling to the deformation of the flow net in the slope models, additional tests were performed and presented later in this Chapter 7.

5.3 Slope stability tests with shallow groundwater table

Instabilities in soil slopes may also occur due to seasonal climatic variations which affect the depth of the GWT below the soil surface. The rise of the GWT, due to rainfall or communication with a hydraulic boundary (e.g. a river or a lake), cause a change to the pore water pressure distribution affecting soil strength. Therefore, the study of rainfall-induced instabilities in the geotechnical centrifuge also includes the case of the water table rise.

Slope models were prepared at two different densities which indicate a loose (test CSA8.4) and a dense (test CSA9.4) state. The boundary conditions which were applied to the models initiated flow within the soil mass due to the creation of a hydraulic gradient.

For the particular tests, slope deformation analysis was performed using PIVlab, which offers the ability for better visualisation of the progressive failure, since the displacement vectors were projected on the original pictures. Moreover, the ability of the software to use filters for image improvement allowed to produce less wild vectors and therefore more accurate results.

5.3.1 Loose slope

The effect of rainfall with subsequent rise of the groundwater table was studied in this test. The boundary conditions which were applied to the model are illustrated in Figure 5.17. The slope height was 150 mm and had an inclination of $52^\circ (\pm 2\%)$, while the relative density of fraction E sand was 50%. The height of the slope in the prototype was 9 m. The model was brought to centrifugal acceleration of 60g in 10g increments. Soil started to drain during gravity increase and a horizontal water table formed just below the slope toe. Rainfall conditions were applied to the slope with the highest intensity (i.e. 2.44×10^{-5} m/s) and the left-side drainage turned off. Unlike the horizontal water table tests, the head difference created between the two sides of the slope increase the hydraulic gradient. Therefore, seepage flow was initiated within the soil.

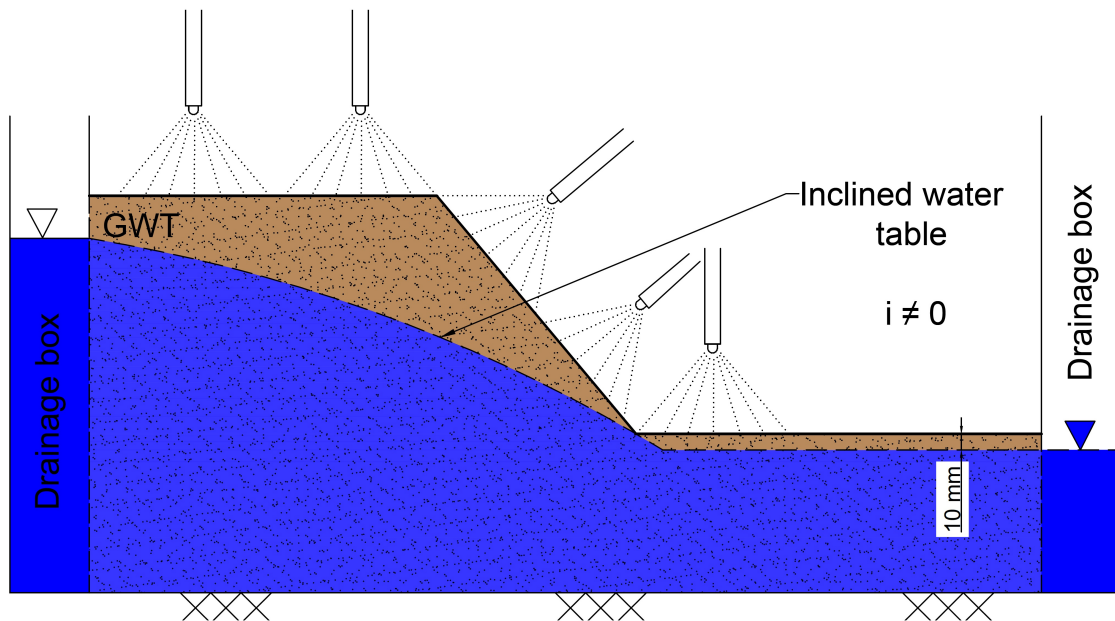


Fig. 5.17 Boundary conditions applied to the slope model at test CSA8.4, with a shallow groundwater table (test CSA8.4).

Pore water pressures were intended to be measured using three PPTs buried within the soil mass. However, the elevated GWT did not reach the installed PPTs, therefore positive pressures

were not able to be measured. The GWT profile was determined by observation through the Perspex window.

At 60g, a horizontal groundwater table was formed below the slope toe, as can be seen in Figure 5.18. Rainfall conditions were initiated at $t = 0$ s. Rainwater infiltrated the slope and reached the water table which then started to rise. After 72 s, the water table had reached the height of the slope toe and had a fairly curved shape. The result of an elevated GWT was the occurrence of a progressive failure which included 4 local and a final global failure. The first failure appeared 86.4 s after rainfall initiation. This was a small-scale failure occurring at the slope toe. The second failure took place at $t=92.1$ s, the third one at $t=96$ s and the fourth failure occurred at $t=120.1$ s. These failures caused the erosion of the slope toe, however the slope did not experience a global failure for the next 21.5 s. At time $t=141.6$ s, a large-scale failure occurred which corresponds to a prototype rainfall duration of 141.6 hr. The final failure surface indicated a rotational landslide passing through the slope toe.

Deformation vectors and contours are shown in Figures 5.19 - 5.21. After 86.3 s of rainfall and before progressive failure starts (Figure 5.19), a zone parallel to the slope shows increased deformation. The displacement of this zone indicates its susceptibility to failure and coincides with the final failure surface at the end of the test. The displacement increases as the failure progresses and the local instabilities occur (Figures 5.20 and 5.21).

The initial displacement at $t=86.3$ s occurred due to the erosion of the slope toe, as shown in Figure 5.19a. This erosion was caused by the seepage force which was initiated by the head difference between the two sides of the model.

5.3.2 Dense slope

The effect of the groundwater table rise, without rainfall, in the stability of a dense slope was evaluated in these tests. The boundary conditions which were applied to the model are

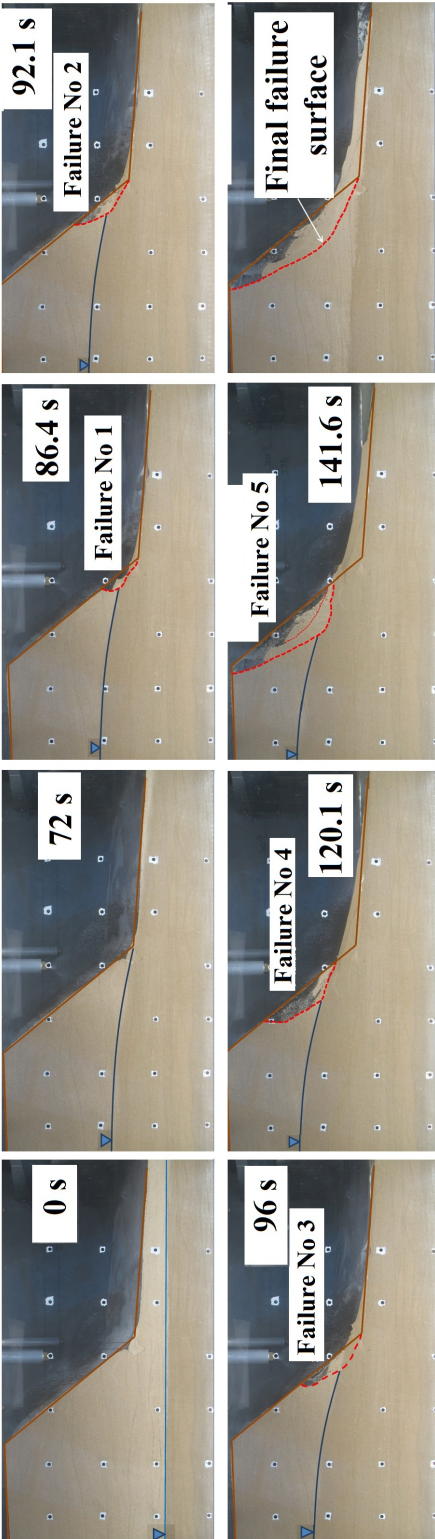
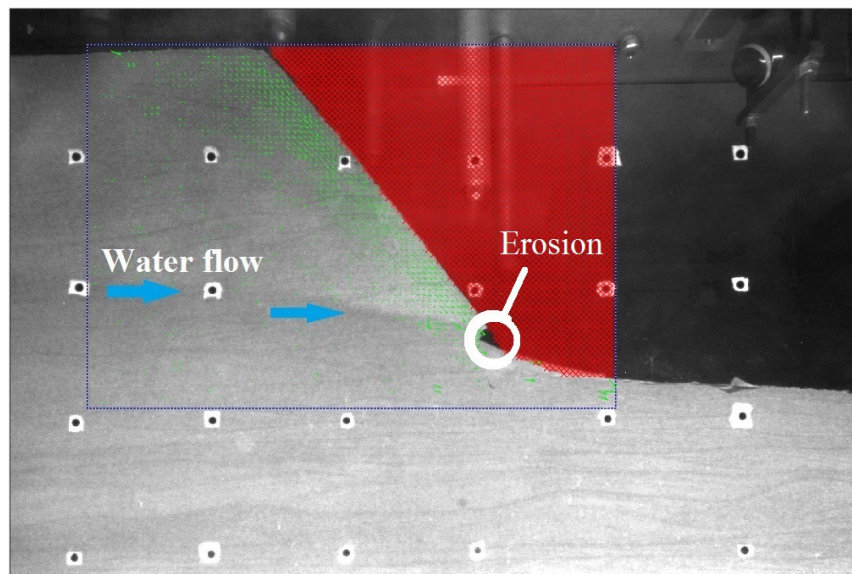
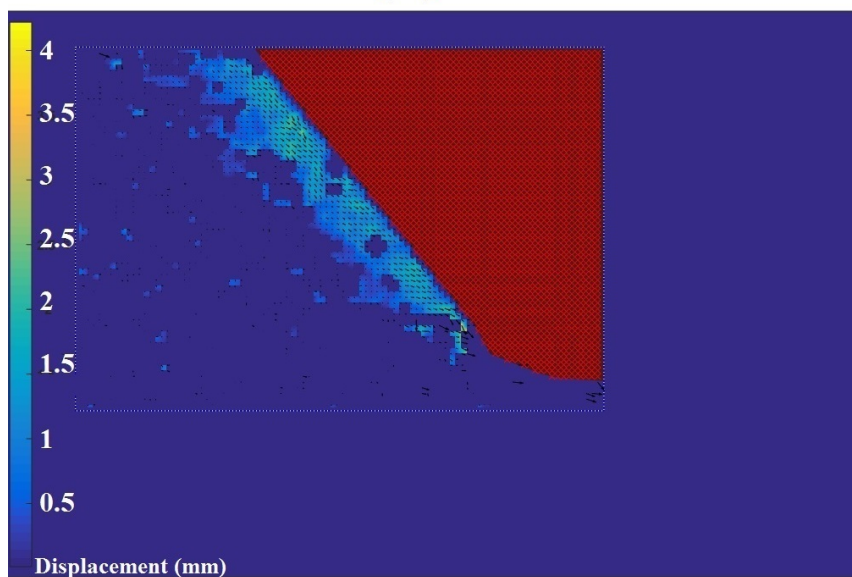


Fig. 5.18 Water table rise in test CSA8.4 and progressive slope failure (model time).



(a)



(b)

Fig. 5.19 (a) Deformation vectors and (b) deformation contours after 86.3 s of rainfall (before failure No 1) (test CSA8.4).

illustrated in Figure 5.22. The slope height was 150 mm and had inclination of 50° while the relative density of fraction E sand was 95%. The height of the slope in the prototype was 9 m. The model was brought to centrifugal acceleration of 60g at 10g increments. Soil started to

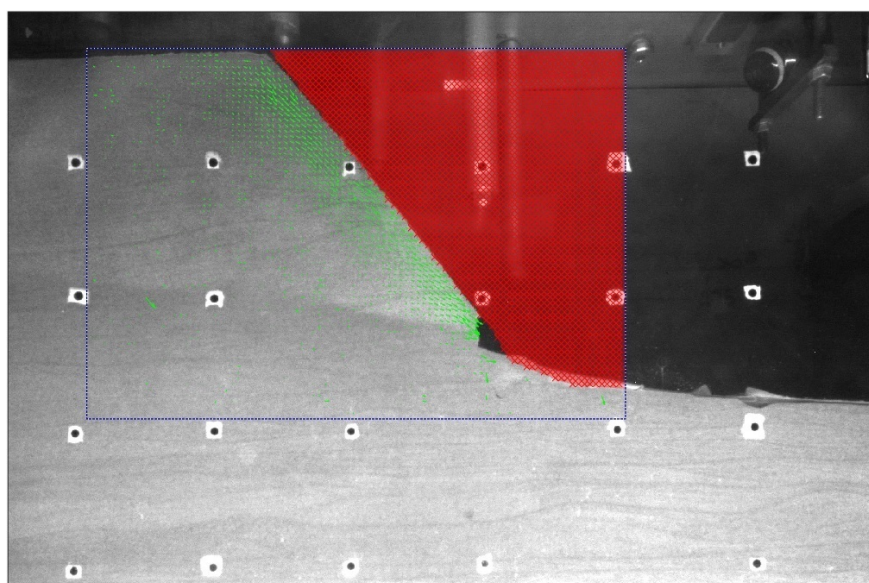
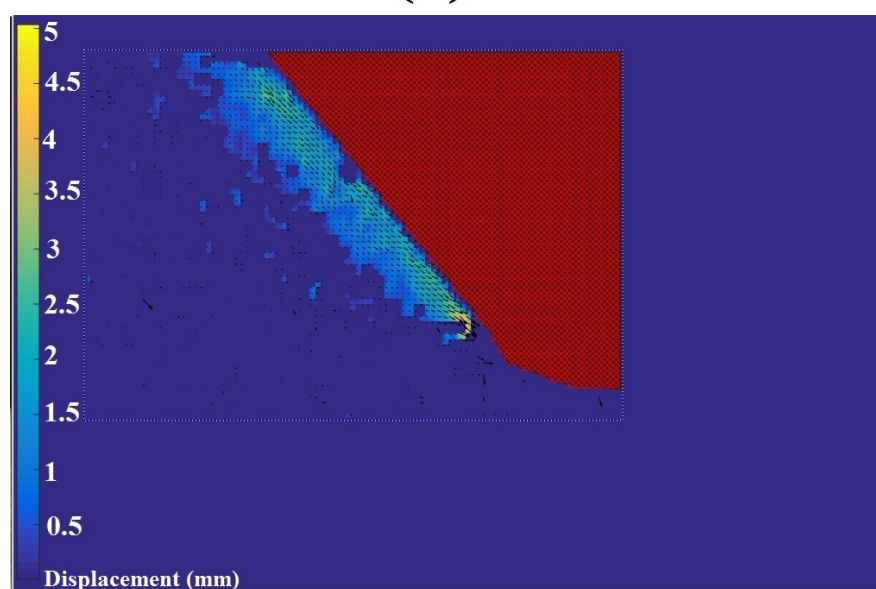
**(a)****(b)**

Fig. 5.20 (a) Deformation vectors and (b) deformation contours after 92 s of rainfall (before failure No 2) (test CSA8.4).

drain when centrifuge gravity increased. No rainfall conditions were applied to the model. The head difference between the two sides of the model was created by providing water to the

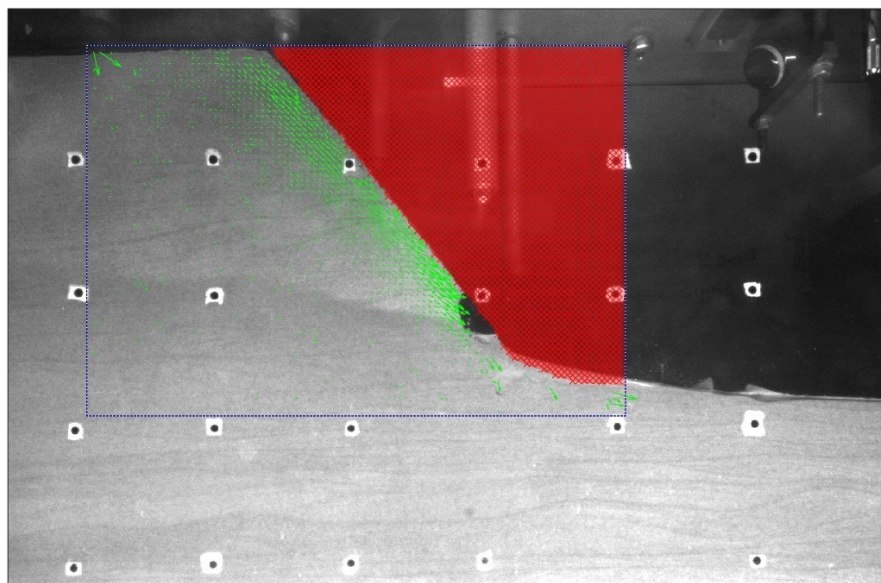
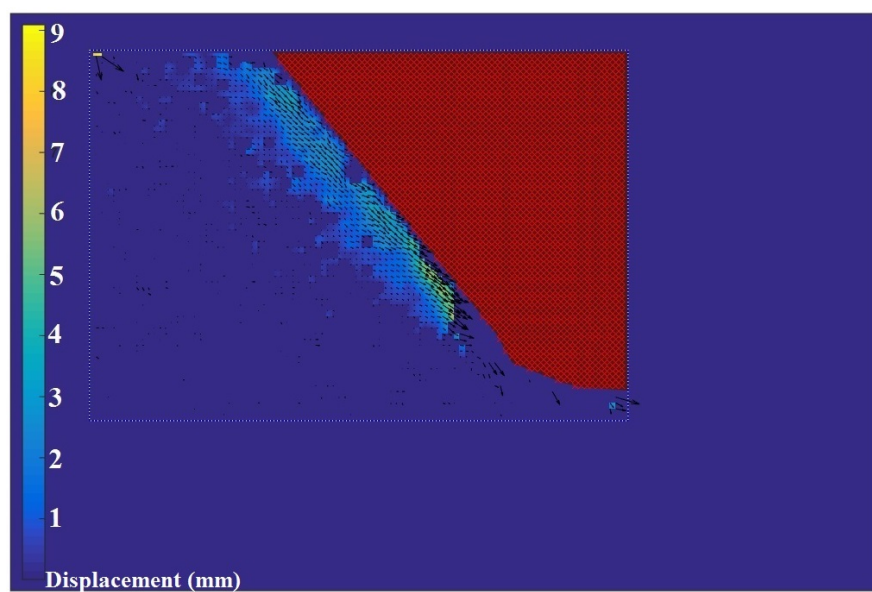
**(a)****(b)**

Fig. 5.21 (a) Deformation vectors and (b) deformation contours after 95.3 s of rainfall (before failure No 3) (test CSA8.4).

upslope drainage box and allowing drainage from the downslope drainage box.

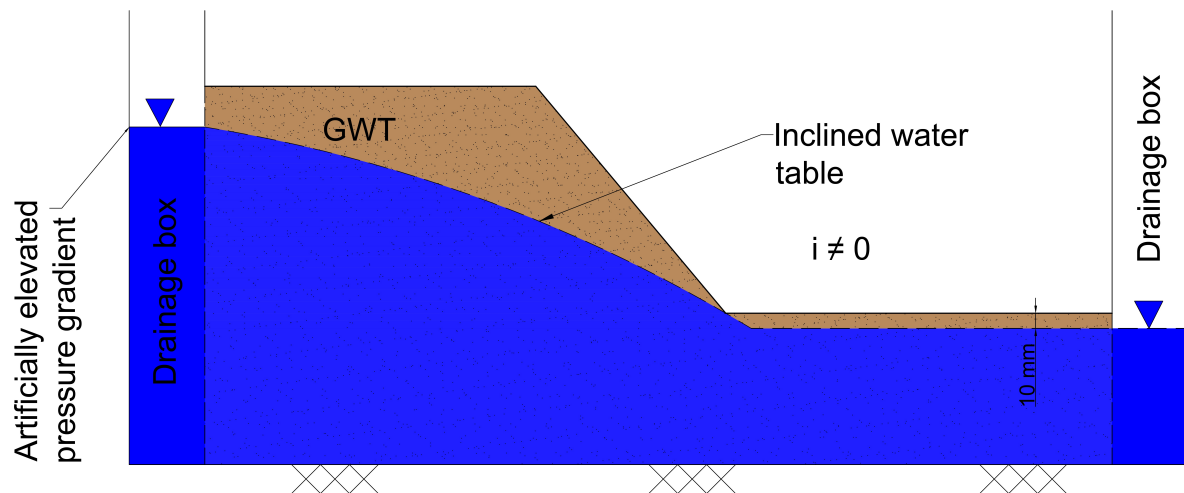


Fig. 5.22 Boundary conditions applied to the slope model at test CSA9.4

Four PPTs were buried within the slope model in order to measure positive PWPs induced by the elevation of the GWT. A layout of the model is shown in Figure 5.23.

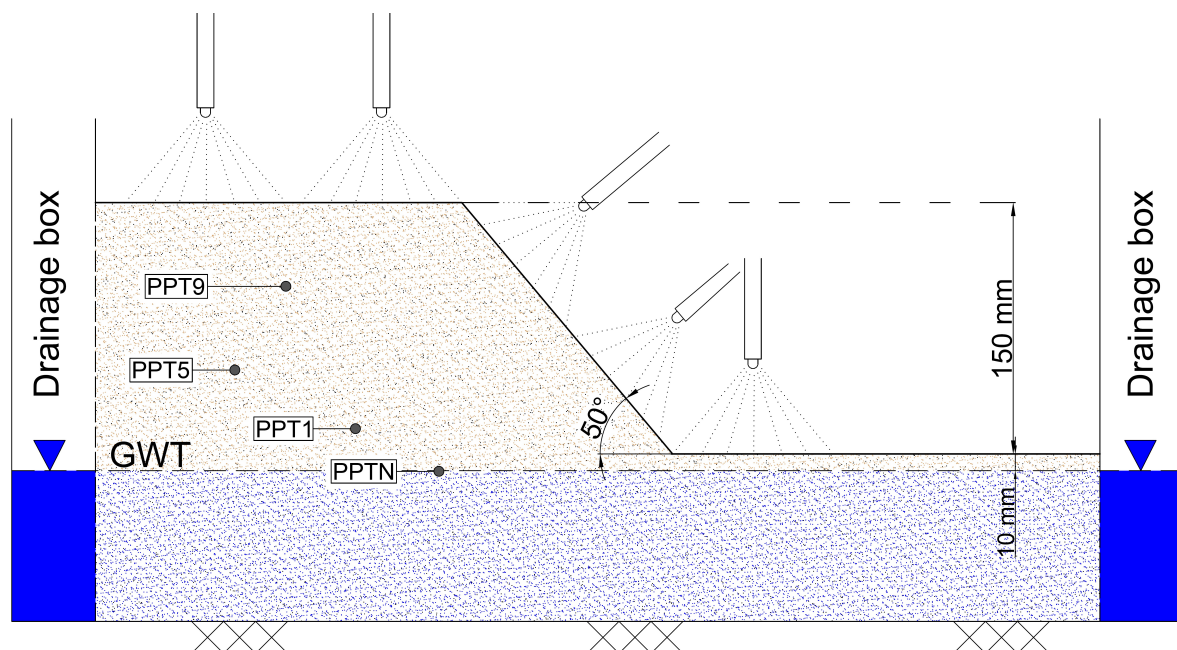


Fig. 5.23 Model layout at test CSA9.4

The behaviour of the slope model under elevated GWT was characterised by a progressive failure which included 4 local and one global instability. The progress of the failure is shown in

Figure 5.24. Progressive failure initiated from the high hydraulic gradient established between the slope toe and the back side of the model. The first local failure at the toe occurred when the groundwater table rose 15 mm above the slope toe. Another three local instabilities took place before a global failure occurred.

The results from the PIV analysis are shown in Figures 5.25 - 5.27. Just before the first failure occurred (Fig. 5.25), the behaviour of the slope is rigid and no settlements were caused even after the erosion of the slope toe. The same observation was made just before failure No 2 (Fig. 5.26). Deformation was restricted only to the area of the local deformation while the overall settlement of the slope was minor. Only at failure No 3 (Fig. 5.27) the final failure surface appeared in the deformation field. This reveals the resistance of the slope to a global failure even after extended erosion of the toe.

The PWP variations during the test are shown in Figure 5.28. The slope experienced an increase in pore pressures as the GWT continued to rise. The delay to the response of the PPTs which were located higher in the slope is evidence of the movement of the wetting front to the surface. At the moment of the global failure, PWPs at the highest depth reached a value of almost 60 kPa. The water supply to the model was stopped soon after the slope experienced failure No 5. The response was the dissipation of the excess PWPs due to soil drainage.

5.3.3 Summary

Comparison of results from tests CSA8.4 and CSA9.4 indicate a slightly different behaviour of the models. In the case of the loose slope, the maximum depth of the failure surface was 39 mm or 2.3 m in the prototype. The failure surface in the denser slope was shallower and found to be 29 mm or 1.7 m in prototype. Furthermore, the final failure surface was formed in the loose slope soon after the erosion of the toe. The denser slope was more resistant to a global failure even after three local failures at the toe and the subsequent loss of support. This can be explained by the increased strength of the dense slope, which was enhanced by potential

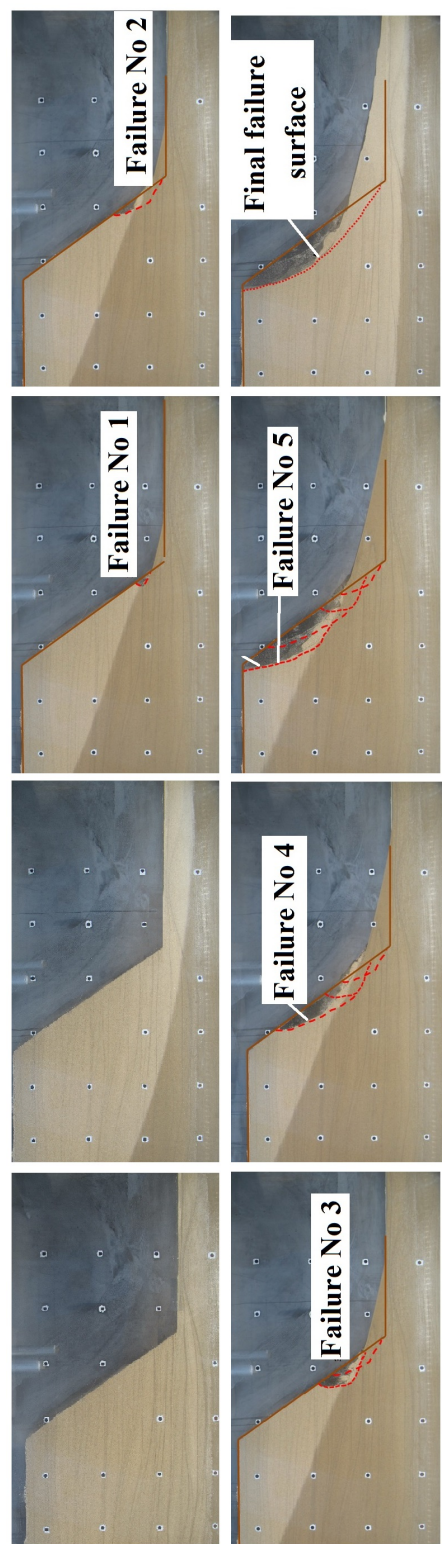


Fig. 5.24 Water table rise in test CSA9.4 and progressive slope failure.

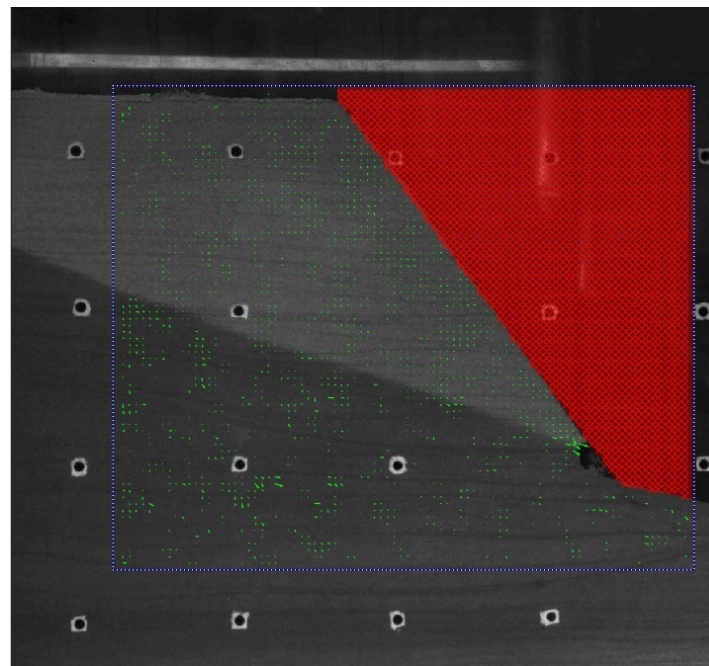
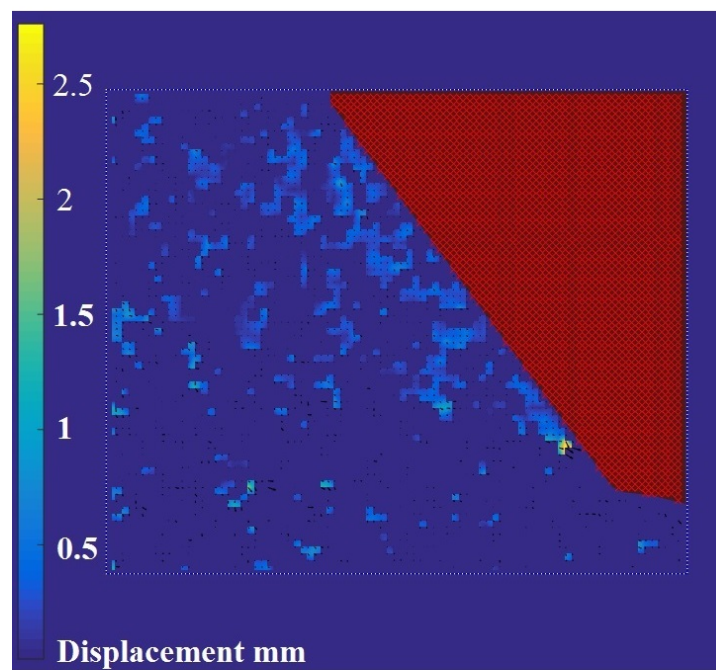
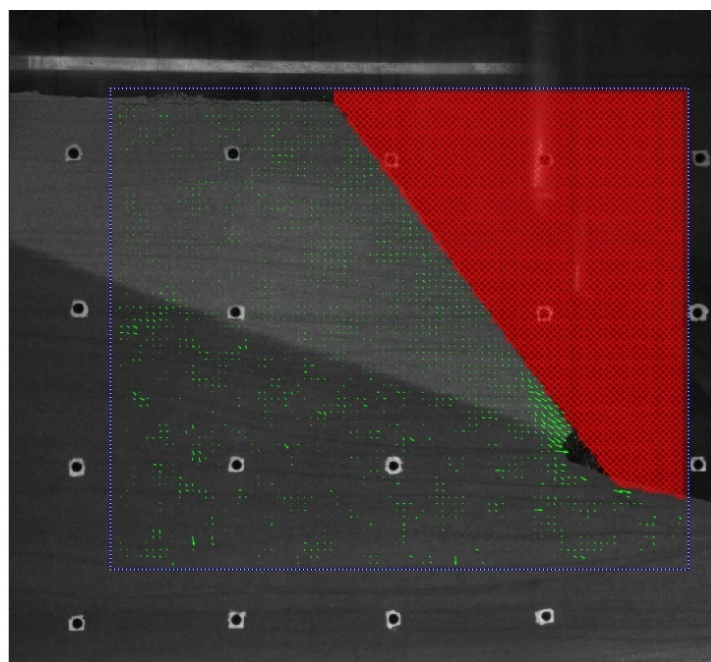
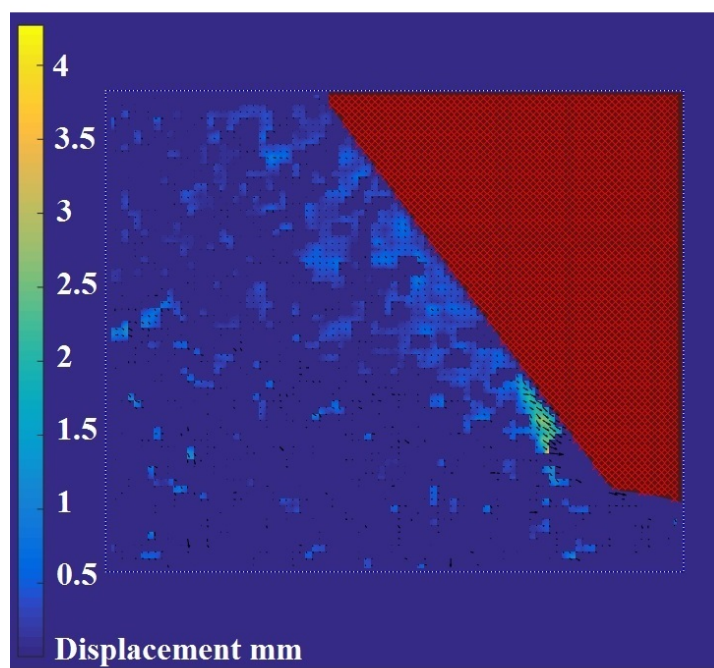
**(a)****(b)**

Fig. 5.25 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 1.



(a)



(b)

Fig. 5.26 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 2.

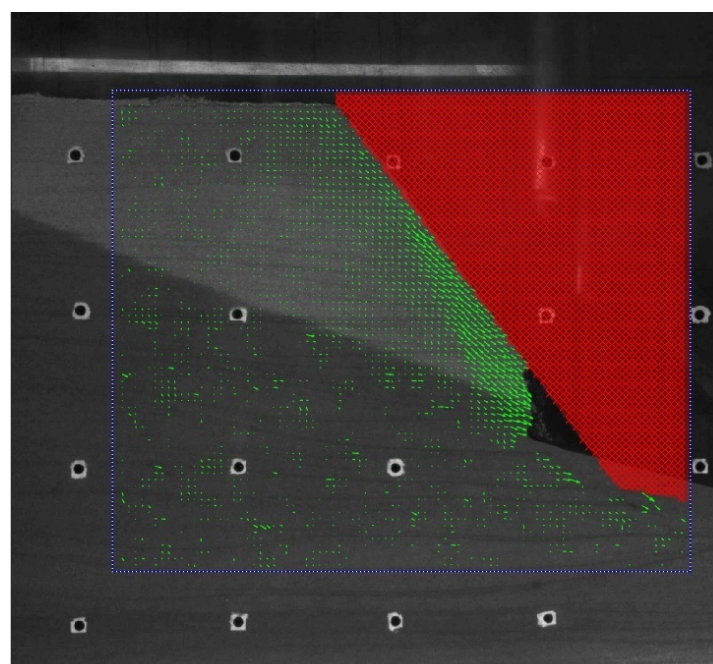
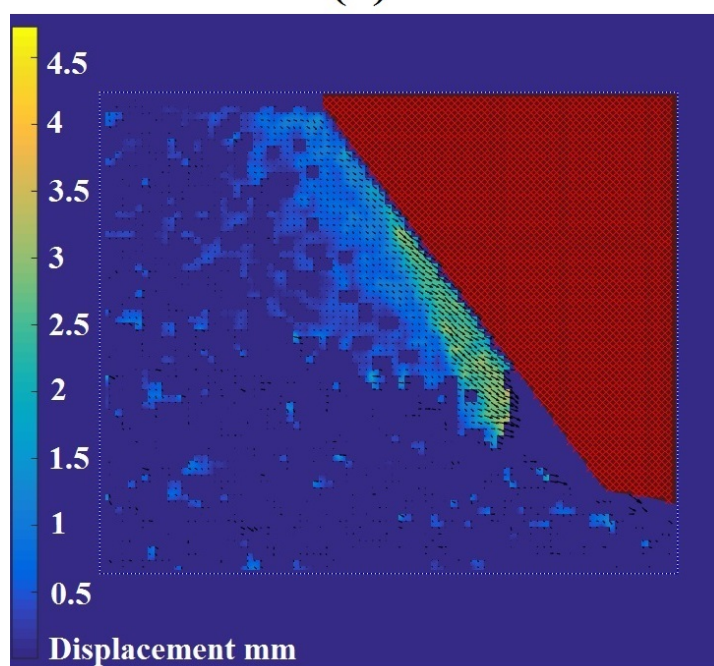
**(a)****(b)**

Fig. 5.27 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 3.

suction increment due to desaturation of the soil mass, but also by the loss of strength for the loose slope due to rainfall.

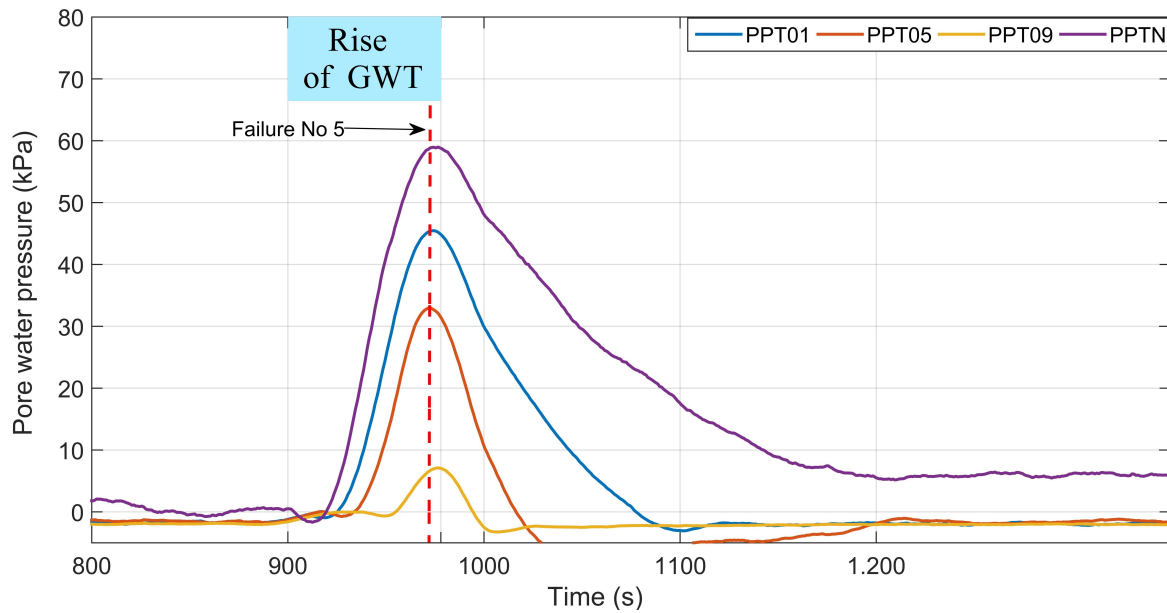


Fig. 5.28 (a) Deformation vectors and (b) deformation contours for test CSA9.4 at failure No 3.

5.4 Flow path in centrifuge modelling

One question which arises from the slope stability tests in sandy soil slopes is whether or not the flow net within the model changes under g . To investigate this issue, a flow net test was performed in the sandy slope under increased gravity.

5.4.1 Model preparation

A sandy model was prepared inside the centrifuge box using the dry sand pouring method and subsequent wetting. The saturated soil block was forced to drain by applying a positive air pressure of 15 kPa in the box and opening the valves. In this way, partial saturation was achieved which allowed the slope to be cut at an angle of 50° . The relative density of the model was 74%.

The layout of the model is shown in Figure 5.29. Eight stainless steel tubes with an internal diameter of 1 mm were inserted in the soil surface at a depth of about 5-10 mm. The tubes

were attached to a larger pipe 8 mm in diameter which was installed above the soil surface and on the Perspex window. One side of the pipe was connected to a tank which was located on the top lid of the centrifuge box. The tank was filled with a dye solution of water and potassium permanganate (KMnO_4). This type of solution is used widely in laboratory demonstrations of flow nets because of its characteristic colour and its ability to be spread into the soil mass following water flow. Therefore, it is used to track flow lines within a soil, involving models like retaining walls, diaphragmatic walls and dams. The other side of the 8 mm pipe was blocked so as to create a liquid pressure in the dye solution which then would exit from the tubes.

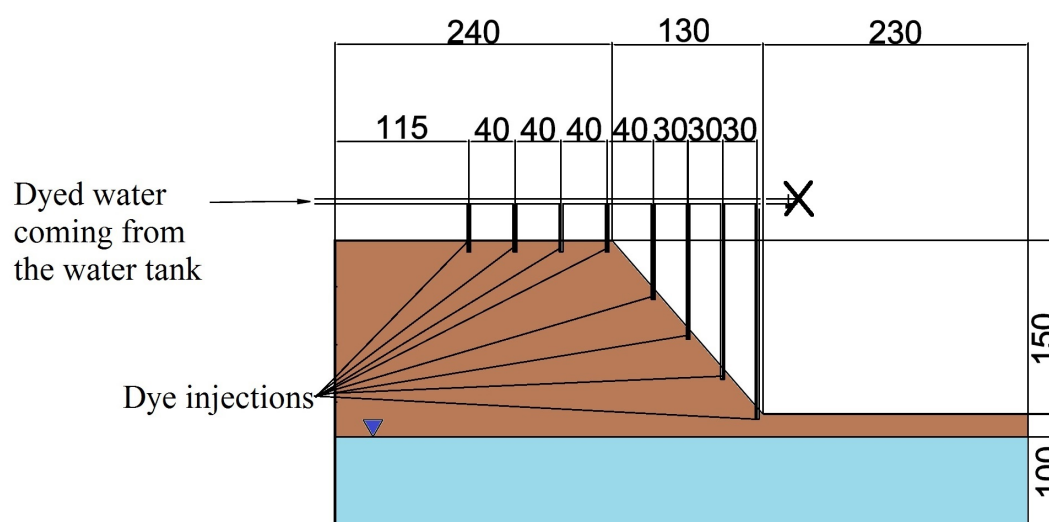


Fig. 5.29 Layout of the slope model used for the determination of the flow net in the centrifuge (all dimensions in mm).

The tubes had a separation of 40 mm at the horizontal crest of the slope and 30 mm on the slope. An on/off solenoid valve was installed between the tank and the tubes. By opening the valve, dye flowed from the tank into the 8 mm pipe and then exited from the tubes. In order to reduce the impact force and subsequent erosion of the soil mass due to the liquid pressure, the tank was placed as close to the injection system as possible. The distance between the upper

surface of the liquid inside the tank and the tubes was 260 mm for those located at the crest and 410 mm for those located deeper, near the slope toe.

The aim of this test was to observe the flow lines within the slope model at low g level (10g) and at high g level (60g) under rainfall conditions. The procedure which was followed included the gravity increase and a first injection of dye at 10g. The model was gradually brought to 60g and rainfall conditions were applied. The rainfall intensity used in these tests was the highest one, i.e. 2.44×10^{-5} m/s in prototype. Dye injections were again made to the slope under these conditions. The movement of the coloured liquid was observed through the Perspex window of the centrifuge box.

The movement of the water and the flow lines observed in 10g are shown in Figure 5.30. The red spots in the figure represent the points at which dye was injected. Under these conditions, dye was diffused in the soil driven by the downwards movement of the drainage front due to gravity increase. The flow line which starts from position 1 moves vertically downwards. The flow path from point 2 was curved and almost parallel to the slope. The same observation was made also for point 3, which seemed to move towards the drainage box, at the side of the model.

Figure 5.31 illustrates the flow lines as they were observed from the eight injection locations at 10g and with a horizontal water table. The flow path of the water flowing from the horizontal surface is vertical and water flows directly to the water table. Near the slope, flow lines start to curve and they tend to move parallel to the slope until they reach the water table.

At 60g, rainfall conditions were applied in order to create flow conditions within the slope. Dye solution was injected into 5 points, as shown in Figure 5.32a. In Figure 5.32b, the flow lines from points 1-4 indicate a vertical movement of the water to the water table. The movement of dye from point No 5 was different and again parallel to the slope. This is attributed to the rise of the water table above the slope toe and the drainage of the water to the drainage boxes.

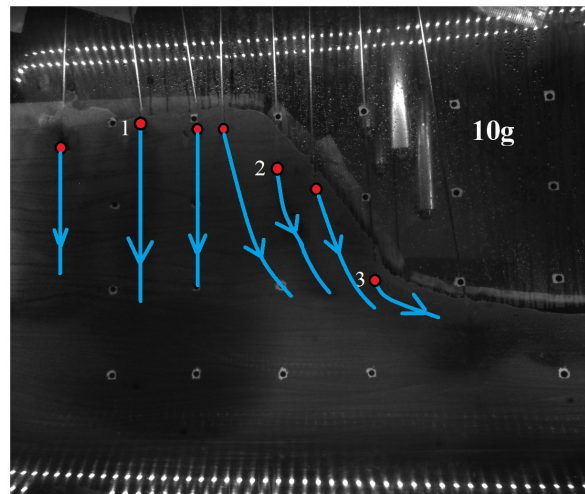


Fig. 5.30 Flow net test at 10g. Blue lines indicate the movement of the dye from points 1, 2 and 3.

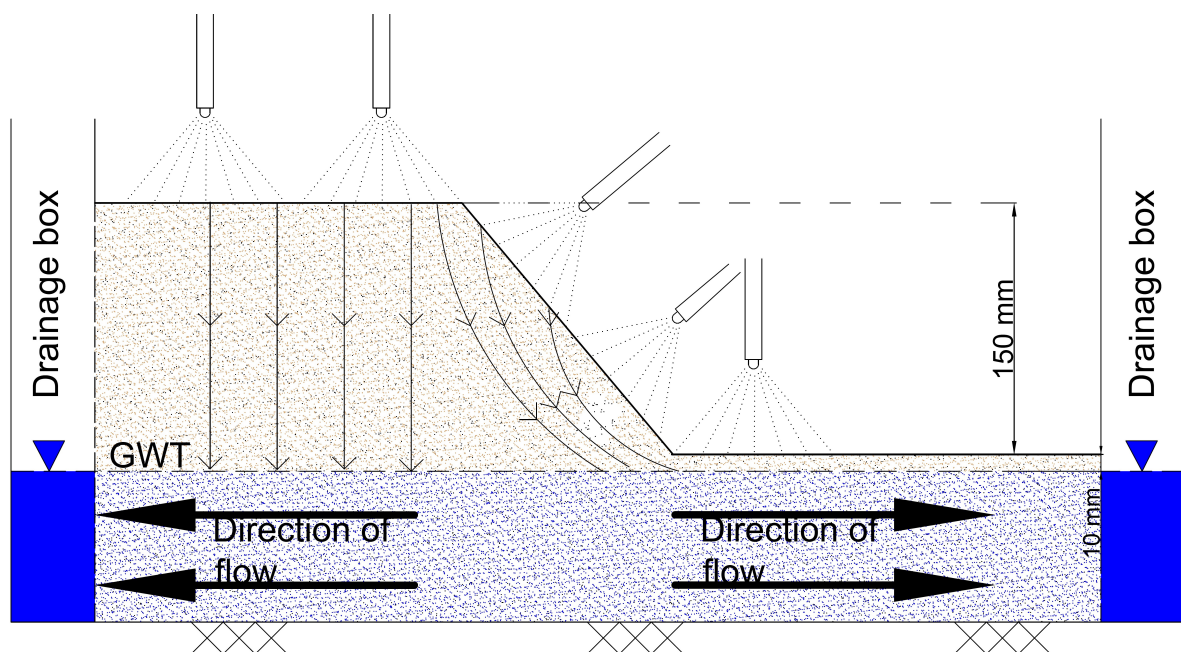


Fig. 5.31 Flow lines within the slope at 10g.

An illustration of the flow lines in the model at 60g is given in Figure 5.33. Part (a) presents the flow regime with a horizontal GWT, while part (b) illustrates the flow lines in the case of a "bulged" GWT. In the case of a horizontal GWT, water moves vertically downwards and feeds the water table. The same condition applies also at the area close to the slope, where at 10g

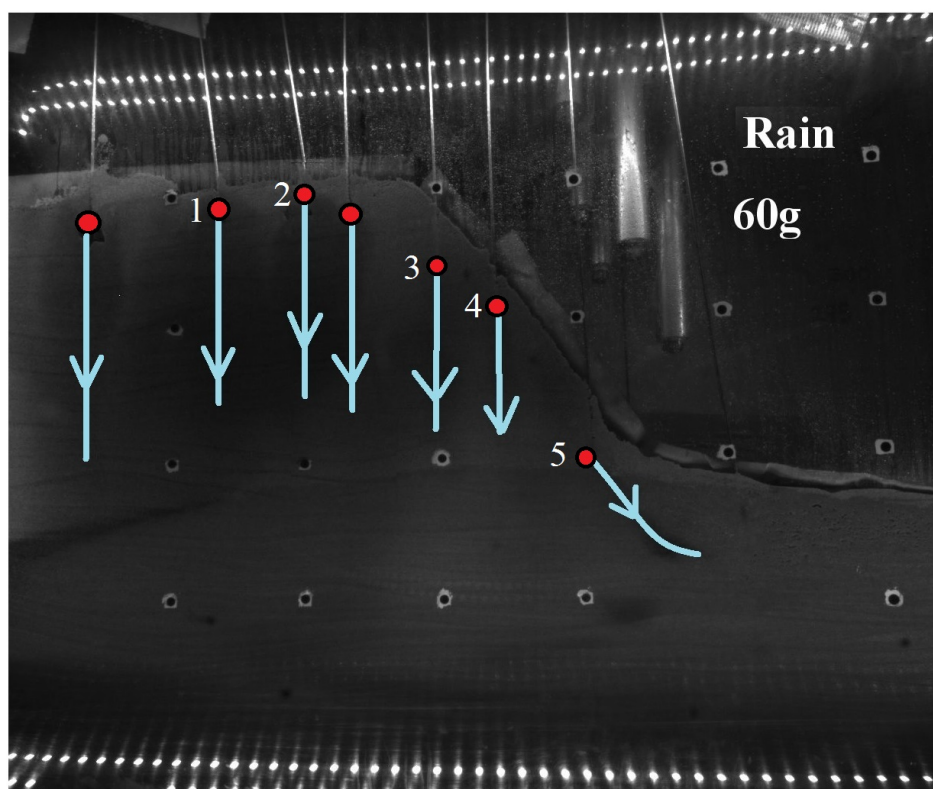


Fig. 5.32 Flow paths at 60g with rainfall conditions applied.

flow lines were curved and parallel to the slope. In the case of a "bulged" GWT, the movement of the water is dominated by the discharge of water into the drainage boxes.

Based on the above results, it can be stated that increased gravity conditions change the flow paths within the slope model with the vertical flow being dominant. However, the formation of a hydraulic gradient within the slope, similar to the one created in the slopes with the shallow GWT, causes the deviation of the flow lines from vertical, as the water moves to the discharge points at the two sides of the model.

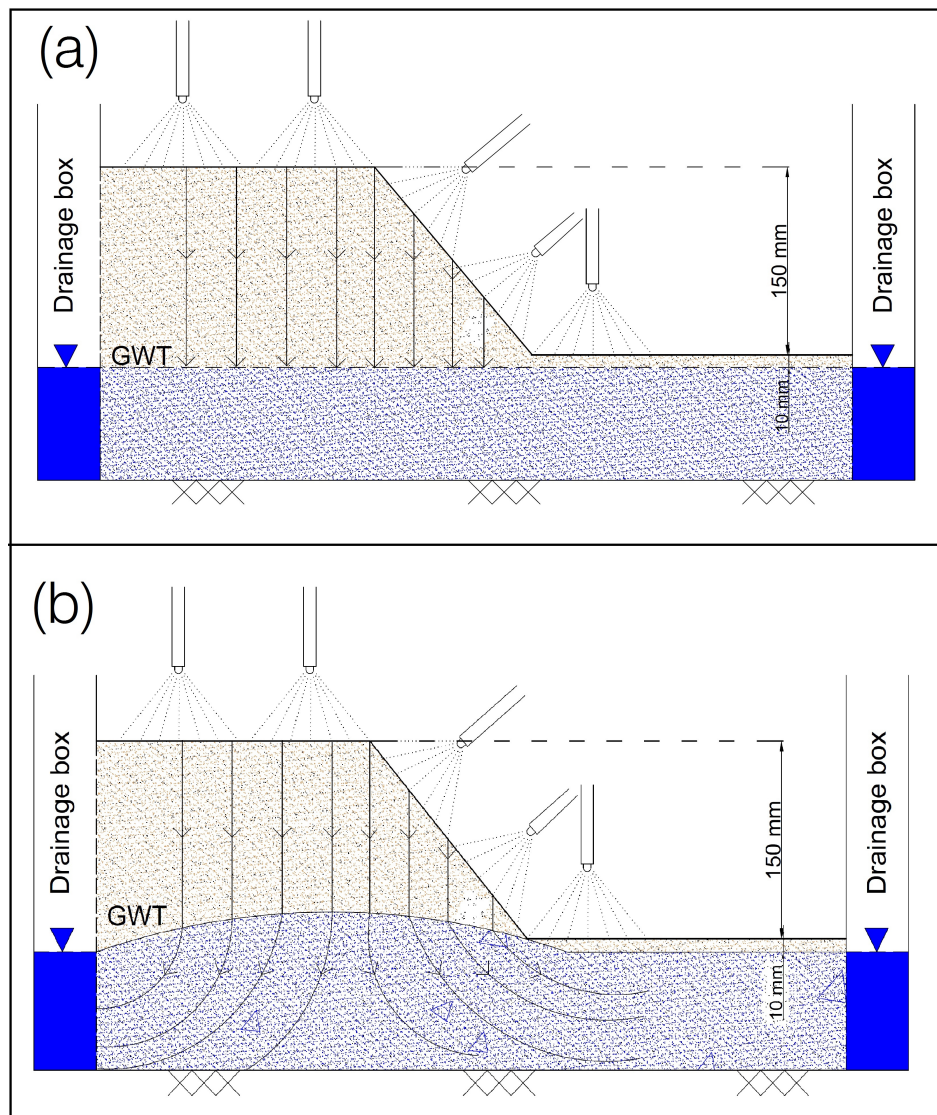


Fig. 5.33 Flow lines at 60g with rainfall conditions applied. (a) Horizontal GWT (b) "bulged" GWT.

5.5 Chapter summary

Loose and dense sandy slope models were subjected to various initial and rainfall conditions at 60g while the GWT was kept deep below the toe. The occurrence of a global instability is expected, however either due to dryness of the soil at 60g or due to rainwater flow parallel to the slope surface, this was not the case. Only minor deformations occurred and no local or

extended failures took place even under extreme rainfall conditions. The questions arising from the interpretation of the results are:

1. What is the residual water content and degree of saturation for the soil mass which lies above the groundwater table at increased gravity conditions?
2. What is the implication of the residual moisture conditions to the soil suction and therefore to the shear strength?
3. How is suction affected by water infiltration?

These questions are explored in Chapter 6.

A shallow GWT boundary condition was applied to a loose and a dense slope. GWT rose in the loose slope under the effect of rainfall while in the dense slope GWT rose by providing water to the upslope side. The implication of this boundary condition was the occurrence of progressive failures at both slopes.

Flow path tests, performed in LB fraction E slope models indicated that increased gravity affects significantly the flow paths of the infiltrated water. Flow paths follow the slope inclination at low g-level (5g-10g) but they move vertically downwards at higher g-levels.

Chapter 6

Soil column tests

6.1 Introduction

The initial conditions applied to a soil model which is subjected to centrifugal acceleration are very important for the evaluation of its behaviour. A number of physical modelling tests involve the presence of water in the soils, such as the study of unsaturated soil slopes ([Askarinejad et al., 2012](#); [Ling and Ling, 2012](#); [Matziaris et al., 2015](#); [Tamate et al., 2012](#)). The implication of rainfall to the stability of slopes involves water infiltration through the unsaturated soil profile and changes to the pore water pressures.

The increased gravity field which is created by centrifugal acceleration causes desaturation of a soil mass if drainage conditions are enabled. When a saturated mass starts to dry, suction develops inside the pore fluid due to attractive forces between water and solid grains and the development of surface tension ([Mitchell and Soga, 2005](#)). Suction contributes to the effective stress or skeletal forces within the soil and increases its shear strength. Therefore, centrifugal acceleration applied to a saturated soil, which drains freely, causes an increase of its shear strength.

A typical assumption is that granular soils drain completely under an increased gravity field. However, this was not the case in the slope model tests presented in Chapter 5. The understanding from the slope model tests is that fine sand retains some of its moisture at 60 g which leads to negative pore pressure and hence contributes to its shear strength. In order to investigate the implications of increased gravity to the retained moisture content of the soil, a series of soil column tests were performed.

Soil columns with different grain size distributions were subjected to a gradually increasing gravity field with a new experimental apparatus which allows the real-time measurement of the soil column average water content during centrifugal acceleration. Moreover, volume changes in the soil elements during drainage were monitored. Consequently, desaturation curves of the gravimetric water content w and degree of saturation S_r against g-level were produced. The distribution of w and S_r of the soil model was also determined at the end of each centrifuge test by extruding soil samples from different depths and drying them in an oven. The final bulk (mean) water content of the soil mass was then compared with the values obtained by the oven dried samples.

The aim of the centrifuge column tests is to investigate the behaviour of a wet soil mass when it is subjected to increased gravity conditions. The main objectives of these tests were:

- To observe the desaturation process produced by gravity increase and determine the residual moisture content;
- To determine the evolution of suction during centrifuge tests and correlate it with the SWRC data obtained in the laboratory tests presented in Chapter 4;
- To investigate the impact of rainfall conditions on the elimination of suction and the reduction of soil shear strength.

- To determine deformations that occur in soil samples under dry and wet conditions due to the increased gravity field;

The centrifuge tests presented in this chapter can be organised into the following categories:

1. Dry tests: initially dry soil columns were subjected to centrifugal acceleration for several hours to determine displacement evolution.
2. Static wet tests: soil samples, initially saturated or in a high saturation state, were subjected to increased gravity while settlement and water drainage were recorded in-flight.
3. Infiltration tests: soil samples, initially saturated or at high saturation, were subjected to increased gravity while infiltration (rainfall) was applied at 60g.

Centrifugal acceleration has been used in the past to induce desaturation and increase suction within a soil model. [Singh and Kuriyan \(2002\)](#) studied the impact of centrifugal acceleration on the development of the unsaturated state in a silty soil. Based on the retained w at different g levels, they presented a method for the estimation of the unsaturated hydraulic conductivity of the soil. Moreover, centrifugal acceleration has been used for the determination of the unsaturated hydraulic conductivity in sands ([Nimmo et al., 1987](#)) and silts ([Singh and Gupta, 2000](#)). In terms of boundary problem studies, [Take \(2003\)](#) used centrifugal acceleration in order to raise suction levels in clayey slope models which were then subjected to rainfall conditions. In this way, the impact of rain infiltration to the seasonal behaviour of slopes was studied.

However, the evolution of the desaturation process in the soil at gravity increase and the impact of g -level to the soil drainage is not very well determined, especially in granular soils. This study aims to determine what are the residual values of w and S_r at different g -levels for soils with different grain size distribution. The results contribute to the understanding of the drainage behaviour of the soil mass when used in centrifuge modelling.

6.2 Experimental setup

For the needs of the particular testing program, an experimental rig was designed and implemented (Figure 6.1). It consists of an 890 mm long aluminium tube, with 25.4 mm thick walls, which is separated into two compartments. The top compartment accommodates the soil column sample (soil chamber), with a height of 500 mm. However, the maximum soil height was reduced to 410 mm due to the space needed at the top for the installation of auxiliary equipment. The aluminium walls of the apparatus were thick enough to prevent deformation due to loading, at the stress level of the tests. The inner diameter of the tube was 127 mm, giving a total volume of the soil column sample of 0.021 m^3 . The floor of the soil chamber was a cone-shaped aluminium base with an 8 mm hole in the middle which acts as a funnel to gather the water extracted from the soil sample. Filter paper, coupled with a fine steel mesh, was fitted between the soil and the cone-shaped base in order to block the movement of grains and allow only water to be extracted.

Removal of water from the soil mass, due to drainage, causes a reduction of the total volume within the soil chamber. Depending on the volume change, this can cause a pressure drop within the chamber if it kept sealed. Therefore, a ventilation valve was fitted to the top lid to allow for pressure equalisation between the soil chamber and atmosphere. In this way, water drainage and suctions were not influenced during the tests. At the same time, the top lid was fixed to the soil chamber with the use of an o-ring. Consequently, air gusts produced from the centrifuge spinning did not affect the soil mass or initiate evaporation.

The bottom part of the soil column apparatus accommodated a steel water tank. This was fixed exactly below the soil chamber via a solenoid valve. The tank was used for collection of the extracted water from the soil-water system. Drainage of the soil mass was enabled at the moment that the solenoid valve was turned on. The tank was connected through its base to a

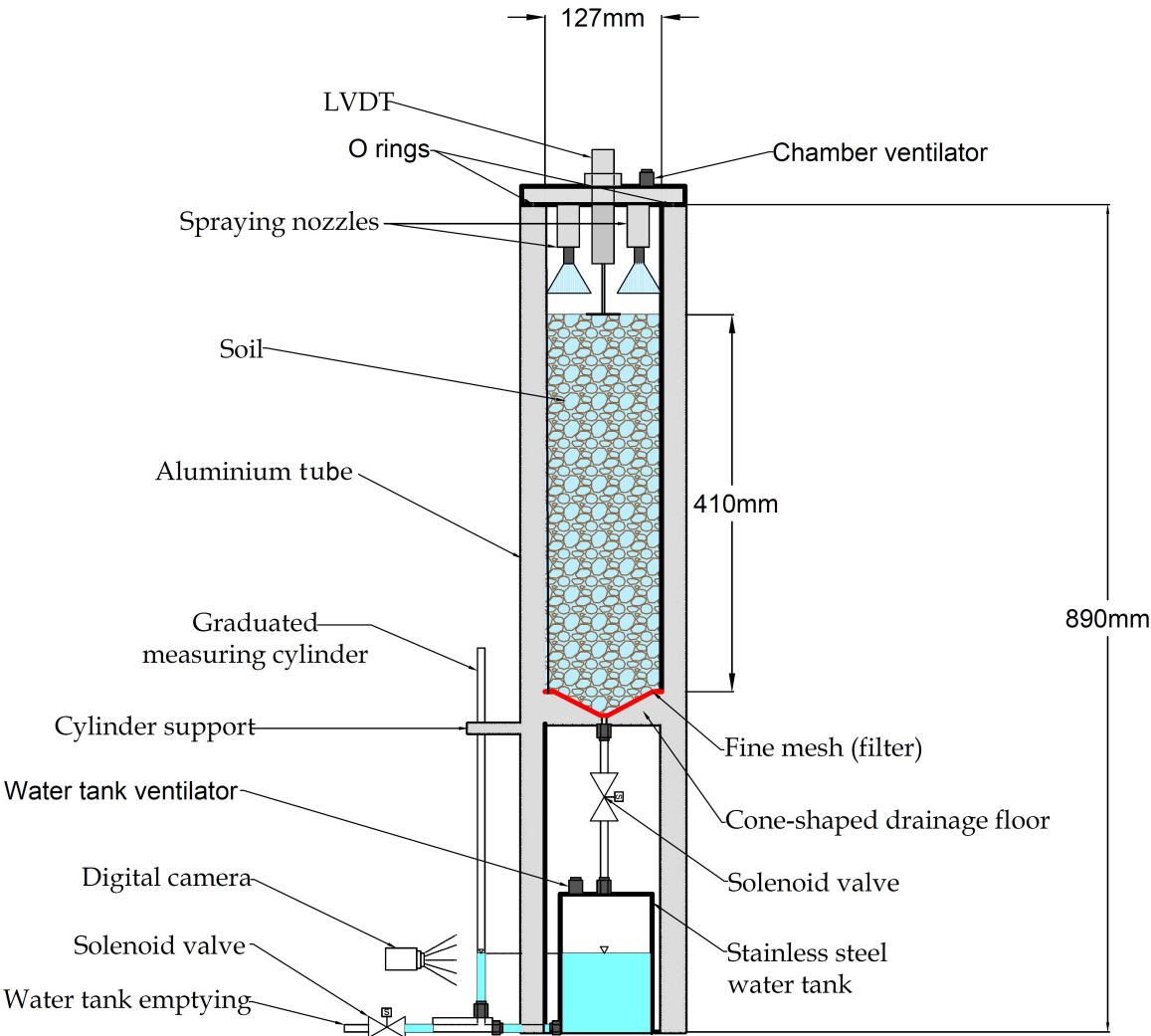


Fig. 6.1 Testing apparatus for soil column tests.

measuring system which allowed the accurate determination of the extracted water volume. A solenoid valve, connected to the bottom of the tank, was used to empty the tank whenever it became full. The volume of the water tank was limited to 1500 ml, which was usually less than the volume of the water removed from the soil during centrifuge tests. Therefore, during the tests, the solenoid valve connecting the tank with the soil chamber had to be turned off and the tank emptied by opening the lower solenoid valve.

The top lid of the chamber accommodated an LVDT which was installed at the centre of the soil and used to measure the deformation of the surface. The armature of the sensor was placed on a square aluminium plate, placed on the soil surface, which reduced the applied pressure arising from the increased weight of the armature at high g level. Moreover, the lid accommodated a pair of spraying nozzles which were used when applying rainfall to the models. The nozzles used for the infiltration (rainfall) tests were the M15 model which was able to provide a flow rate of 58 l/h at 900 kPa of water pressure. At 60g, this flow rate corresponds to a prototype rainfall intensity of 2.44×10^{-5} m/s provided that the height of the nozzle from the soil surface is 70 mm. These were fitted on aluminium extension rods and were placed 70 mm above the soil surface to reduce the Coriolis effect.

6.2.1 Stress distribution and error

The theoretical background concerning the distribution of vertical stresses in a centrifuged model was presented in Section 2.4.3. Due to the dependence of the gravity field on the radius r from the centre of rotation, this is not uniform throughout the model depth, which is in contrast to the prototype which experiences Earth's gravity (considered uniform). Therefore, stresses acting within the soil model do not coincide with the prototype stresses. However, it must be stated that the particular set of tests is not representation of a specific prototype, rather it

aims to simulate soil behaviour under saturated and unsaturated conditions when subjected to centrifugal acceleration.

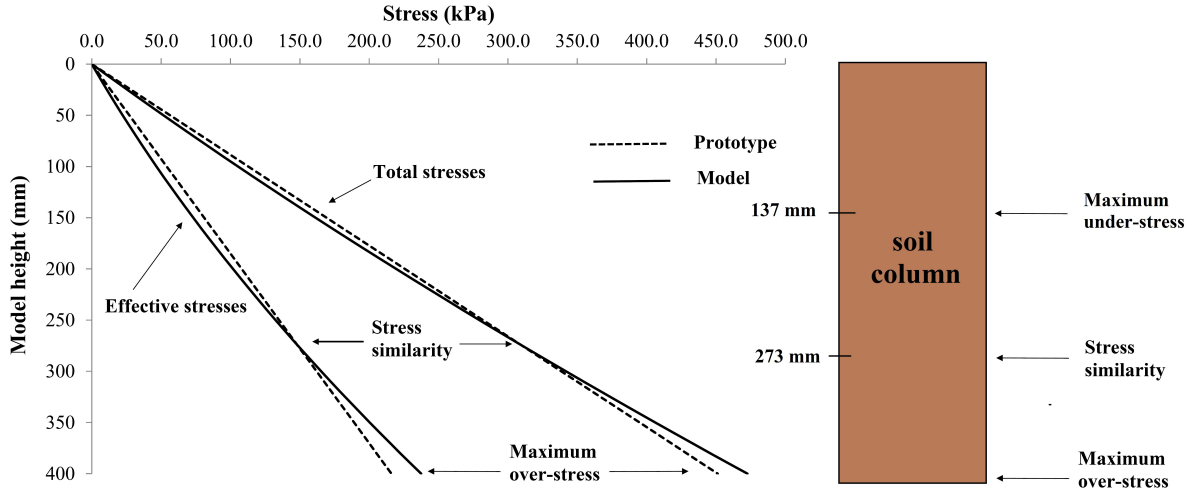


Fig. 6.2 Distribution of total and effective stresses for a saturated LB fraction E sandy model tested in the soil column apparatus at 60g.

Figure 6.2 illustrates the distribution of total and effective stresses for a saturated LB fraction E sand model (subscript m) tested in the soil column apparatus under 60g acceleration, having a total height $h_m = 410$ mm. Similar conditions apply to all soil models tested in the soil column apparatus under 60 g. In a prototype (subscript p) soil column with a height of $h_p = Nh_m$, total stresses are given by:

$$\sigma_{v,p} = pgh_p = pgNh_m \quad (6.1)$$

In the soil column model there is a slight variation of acceleration with radius, since the highest point of the model (i.e. top of soil surface) is at a radius of 1190 mm and the bottom point at a radius of 1600 mm from the centre of rotation. In the centrifuge model, the vertical stresses acting at a depth z from the surface of the model are given by:

$$\sigma_{v,m} = \int_0^z p\omega^2(R_t + z)dz = p\omega^2z(R_t + \frac{z}{2}) \quad (6.2)$$

where R_t is the radius from the centre of rotation to the surface of the model (Figure 6.3).

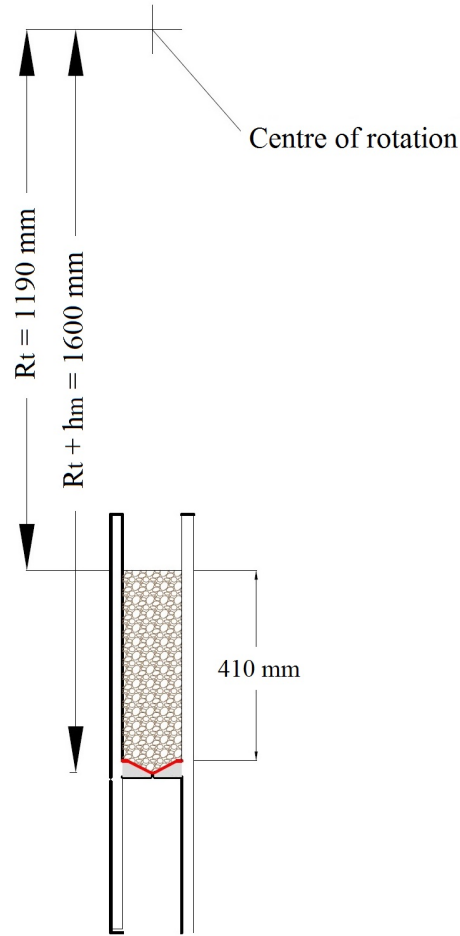


Fig. 6.3 Radii and dimensions of the soil column model on the geotechnical centrifuge.

In order to ensure that over and under levels of stresses are equalised, the effective radius R_e needs to be calculated from the following equation:

$$R_e = R_t + \frac{h_m}{3} \quad (6.3)$$

Therefore, the effective radius for soil column models is $R_e = 1327 \text{ mm}$. Based on this, inertial gravity of 60g corresponds to an angular rotational velocity $\omega = 21.06 \text{ rad/s}$ or, in terms of frequency, 201 revolutions per minute (RPM).

The maximum error in stresses is given by:

$$e_{max} = \frac{h_m}{6R_e} \quad (6.4)$$

which corresponds to the maximum over-stress at the bottom of the soil column (i.e. at depth h_m). Therefore, from Equation 6.4 it is derived that the maximum error in the stress profile is approximately 5% of the prototype stress. The depth at which there is exact correspondence in stress between the model and the prototype is at $\frac{2}{3}h_m = 273$ mm, as shown in Figure 6.2. Finally, at depth $\frac{1}{3}h_m = 137$ mm the maximum under-stress state occurs.

6.2.2 Measuring system

A system for the accurate measurement of the water volume extracted from the soil mass during the tests was designed and placed on the testing apparatus. This system comprises of a standpipe (measuring cylinder), located outside the aluminium tube, which was connected to the bottom of the water tank (Figure 6.4). The measuring cylinder was kept stable during the centrifuge tests by using an aluminium support attached to the chamber.

The water tank is connected to the measuring cylinder (standpipe) so as when the liquid in the tank settles it balances out to the same level in the standpipe. A ventilation valve ensured that air pressure inside the tank remains atmospheric during tests, therefore equal to the air pressure inside the standpipe. The water level was recorded during the centrifuge tests using the digital camera described in Section 3.2.4. For these tests, the camera was fitted with an 8 mm lens which offers better resolution over a smaller field of view. The camera was fixed using the cradle which was designed to support it during centrifugal acceleration. The rate of image capture was set to 1 Hz and the distance between a fixed point at the base of the cylinder and

the water level was measured in pixels in subsequent images. This distance was linked to the water volume in the tank by applying the calibration factor determined during the calibration process. Therefore, the water volume extracted from the soil was directly measured and could be easily connected to the gravimetric water content of the soil. Moreover, the variation of the degree of saturation was determined by using the deformation data and calculating volume changes.

Two types of calibration were performed for the soil column apparatus. The first involved the establishment of a calibration factor between the height of the water level in the measuring cylinder and the water volume inside the tank. The second calibration involved the effect of water level distortion due to the increased gravity conditions. Calibration procedures are described in the next section.

6.2.3 Calibration

Water volume

The calibration of the measuring system involved relating the water volume in the tank to the water level in the standpipe. This was achieved by using known volumes of water and measuring the water level in the measuring cylinder. The height of the water level in the standpipe was determined from the captured digital images using ImageJ software ([Schneider et al., 2012](#)) and measured in pixels. A specific point at the base of the measuring cylinder was selected to be the reference point for all tests (Figure [6.5](#)). Calibration results are shown in Figure [6.6](#).



Fig. 6.4 Graduated measuring cylinder (standpipe) for the determination of the water volume extracted from the soil.

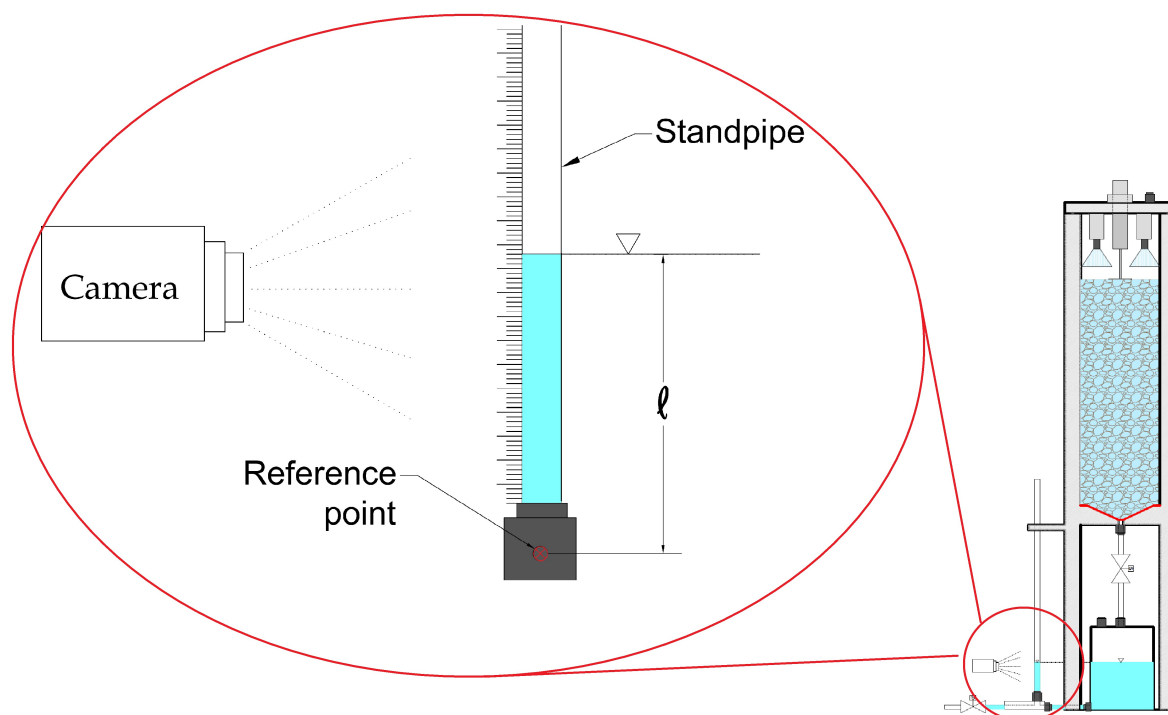


Fig. 6.5 Illustration of the measuring system with height ℓ associated to the water volume in the tank

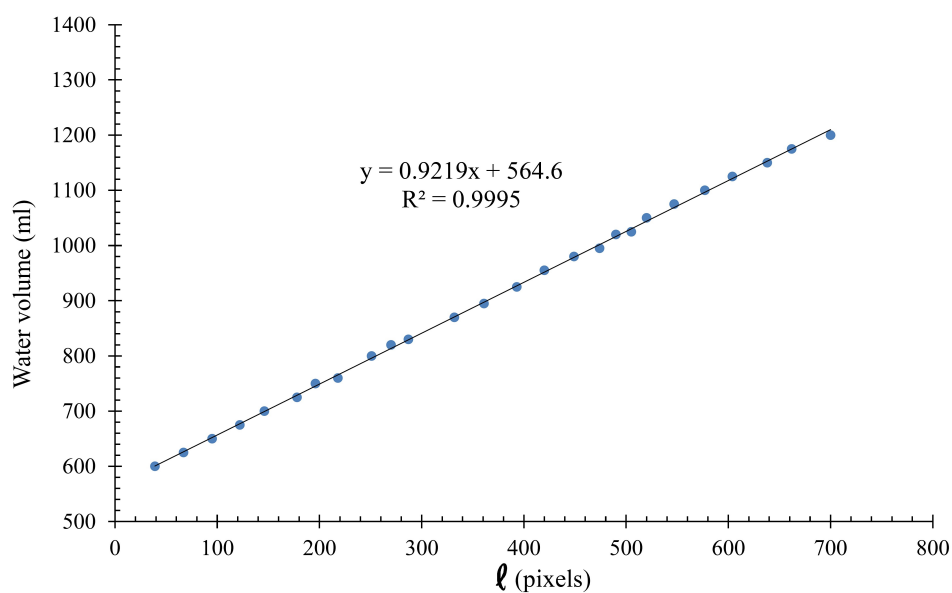


Fig. 6.6 Calibration of the measuring system for the soil column apparatus. ℓ stands for the distance (measured in pixels) between the water level within the measuring cylinder and the reference point

The calibration factor for the measuring system was determined as follows:

$$V_w = 0.9219 \times \ell + 564.6 \quad (6.5)$$

where V_w is the water volume (in ml) inside the water tank and ℓ the length of the water column inside the standpipe (in pixels). Equation 6.5 also defines the resolution of the measuring system. Since a change in the water level by 1 pixel corresponds to a change in water volume of approximately 0.9 ml, the accuracy of the measurement was determined to be ± 0.9 ml. In the case of LB fraction E sand, this water volume corresponds to a change in w of approximately ± 0.008 %.

An important consideration for the measuring system was the existence of trapped air bubbles. These bubbles can be formed during the initial filling of the water tank and can be trapped within the system. During the centrifuge testing, air bubbles can move into the system due to the vibrations of the centrifuge platform and cause disturbance of the water level. Moreover, at Ng conditions, water pressure increases and air bubbles are reduced in volume. Therefore, the water level might change even if there is no water transferred into the tank. An example of this effect can be seen in Figure 6.7. Initially, water was added very slowly to the tank until it reached a volume of approximately 900 ml. Up to this point, the relationship between water volume and ℓ was linear. However, the development of air bubbles within the system caused deviation from the linear relationship. After the system was shaken, data points complied again with the initial linear relationship.

In order to address this issue, de-aired water was added to the tank and measuring system prior to the centrifuge test. The water tank emptying valve was then opened and flow removed any trapped air bubbles within the pipes. By closing the emptying valve, the system was saturated with a volume of water remained within the tank and the standpipe.

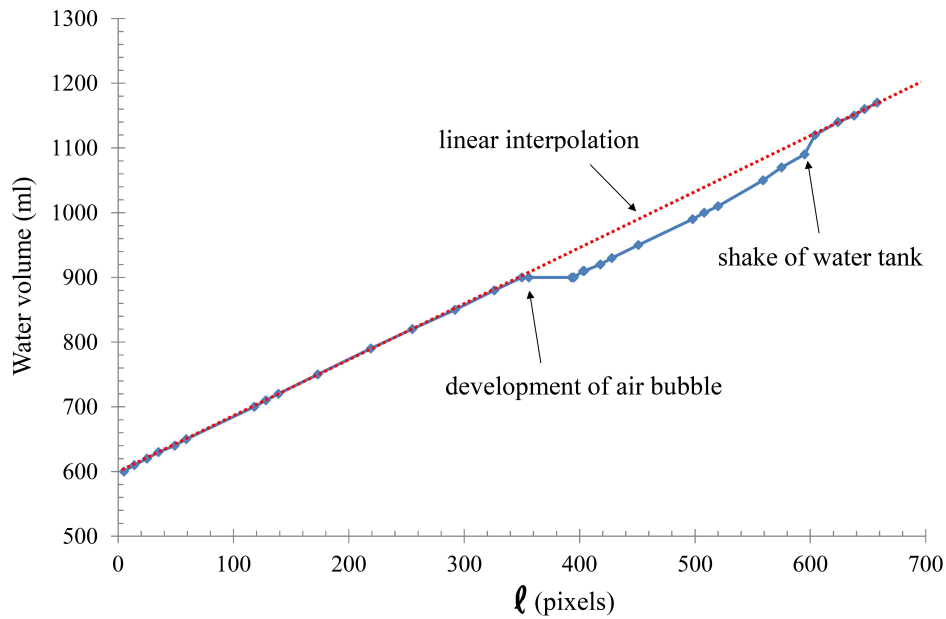


Fig. 6.7 Impact of trapped air bubbles to the measurement of the water volume within the tank.

Distortion

When the soil column system is put into a centrifugal acceleration field, the water level in the tank is distorted from horizontal, as shown in Figure 6.8. This is generated by the variation of the acceleration field which depends on the radius from the centre of rotation. Therefore, the reading in the measuring cylinder might not exactly match the water level inside the water tank. In order to minimise this error, the water tank was placed at the centre of the centrifuge platform while the measuring cylinder was positioned as close to the water tank as possible. Moreover, calibration was performed in order to determine the impact of the centrifuge acceleration field to the water level. The procedure followed in this case included the filling of the water tank with a known water volume and the observation of the water level indication at different g-levels. The water level in the standpipe was recorded at 1g and 5g and then at 5g intervals up to a final acceleration of 70g, thus exceeding the maximum acceleration of the actual tests (i.e. 60g).

The results from the calibration process are shown in Figure 6.9. In this plot, $\Delta \ell$ refers to the change of the water level caused by the increased gravity. The calibration procedure was

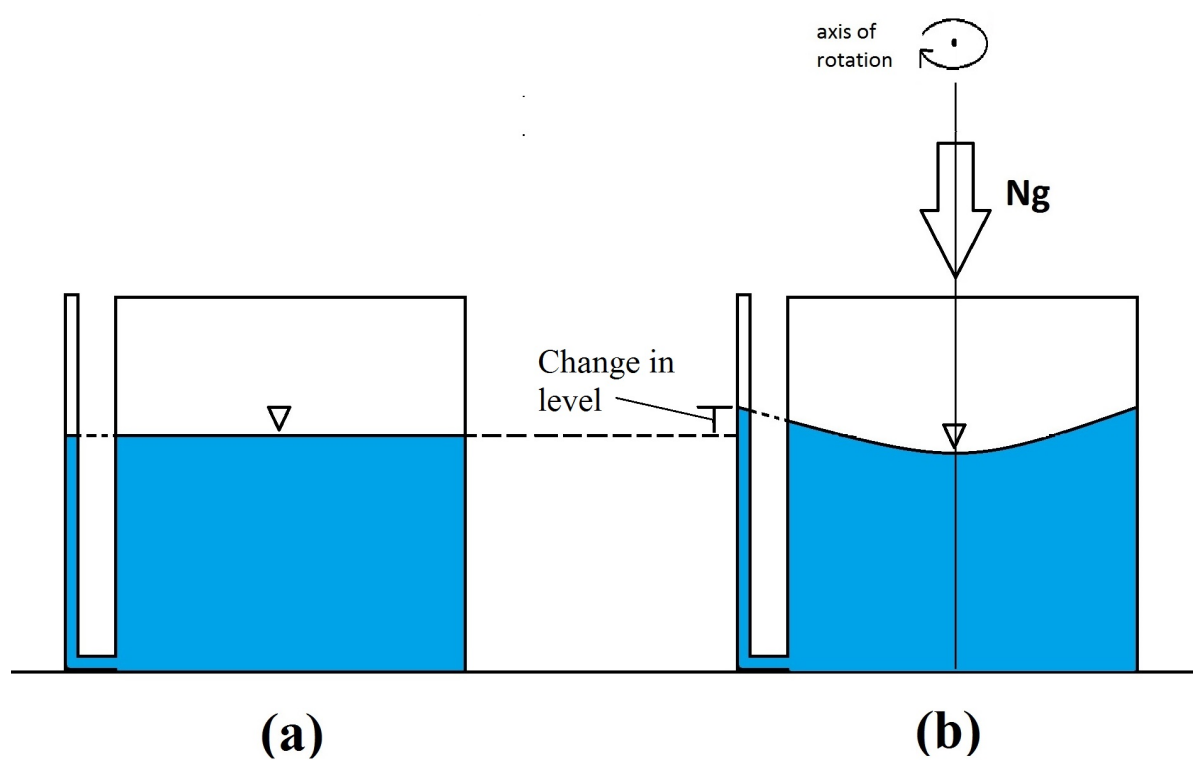


Fig. 6.8 Illustration of the water level distortion caused by centrifugal acceleration. (a) $1g$ conditions, (b) Ng conditions.

performed for three initial water volumes, i.e. 700 ml, 800 ml and 900 ml. Results reveal that for all three cases, there was a significant change in ℓ which was caused by the increased gravity induced distortion up to 10g. This increment is approximately equal for all three volumes of the water tank. At all other increments of g, water level does not seem to be affected significantly.

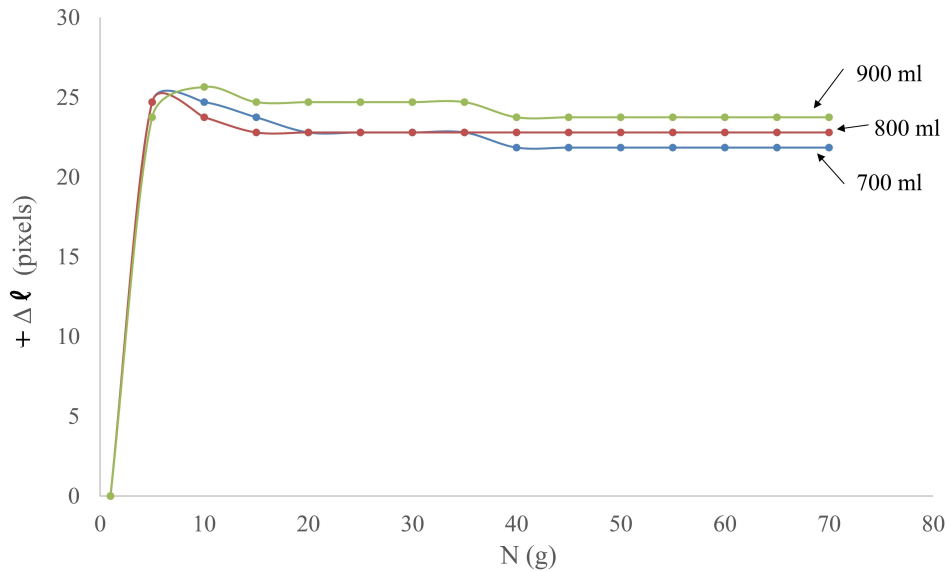


Fig. 6.9 $\Delta\ell$ measured at different g-levels.

The increment of ℓ at the start of centrifuge spinning is attributed to the increment of the water meniscus radius which is formed in the standpipe at 1 g, similar to a capillary tube model. The change to the indication which takes place during the transition from 1g to 5g is shown in Figure 6.10. The water meniscus was reduced when gravity was increased to 5 g and the level of water moved from ℓ_{1g} to ℓ_{5g} . This phenomenon is described by [Lu and Likos \(2004\)](#) as *capillary condensation* which can be caused by the saturation of the ambient air above the water level. In the current study, capillary condensation is not related to the vapour pressure change but to the gravity increment. Centrifugal acceleration causes an increase to the water pressure within the tube. Since the air pressure u_a does not change during the centrifuge spinning, it balances with the water pressure just below the air water interface which then becomes flat.

Apart from this phenomenon, gravity increment does not cause any other significant effect to the water volume readings, therefore distortion of the water level can be neglected. For practical reasons, water level is considered to be the lower point of the air-water interface within the tube, i.e. the base of the water meniscus.

Based on the calibration results described above, the following considerations were taken into account during testing:

1. Water tank was filled with an initial amount of water equal to 600 ml. Then, the system was shaken to ensure a full saturation of the pipes and the individual components (i.e. remove any trapped air bubbles).
2. At the beginning of the centrifuge test, the soil column was brought to a small initial acceleration of 5g with the drainage valve closed. When the centrifuge reached 5g, the initial water level was recorded and the drainage valve was opened to allow drainage and consequent water transfer from the soil into the water tank.

6.2.4 Materials and model preparation

Three types of silica sand and mixtures of Congleton sand and M-Soil, with varied proportions, were used in these series of tests. The types of silica sand used were LB fraction C, LB fraction E and Congleton sand. The soils produced from Congleton sand and M-Soil had varied grain size distributions (Chapter 4). The mixtures that were used in the soil column tests were M_{90-10} , M_{70-30} and M_{50-50} . In this way, the impact of grain size on the retained moisture was investigated.

The test procedure involved the measurement of water volume extracted from the soil at Ng, using the water level changes in the standpipe. Therefore, it was important to have an accurate

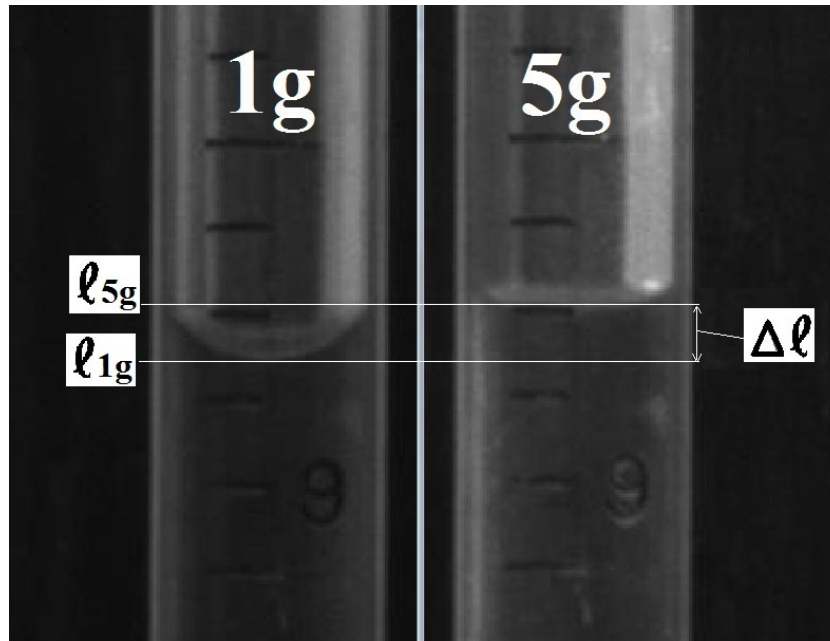


Fig. 6.10 Increment of the water meniscus radius due to the increased gravity, causing a change to the reading of the water level.

initial determination of the mass of water mass added to the soil. Preparation of the saturated sandy models was performed inside the soil chamber by first adding water and then pluviating dry sand from an average height of 500 mm from the water level. During the procedure, the mass of the water and the mass of the soil was recorded; therefore, the initial w , e and S_r could be determined. Furthermore, the determination of the initial water content was checked by oven drying samples.

Preparation of the silty sand models was performed by compaction within the soil chamber. Initially, the soil was mixed with water at the optimum water content and left in sealed bags for approximately 24 hours to equalise. The soil was then compacted in 5 layers using the light hammer method. Water was inserted from the bottom of the soil chamber to achieve a high degree of saturation. The determination of the initial gravimetric water content was performed using the oven drying method.

6.3 Dry tests

Dry granular soils exhibit minor deformation under typical levels of constant loading. [Kuhn and Mitchell \(1993\)](#) proposed that many soils exhibit viscous creep behaviour in which deformation and movement proceed under a state of constant loading or stress. Moreover, they divided creep into three distinct stages caused by stresses high enough to induce creep rupture, i.e. primary, secondary and tertiary (Fig. 6.11). [Leung et al. \(1996\)](#) conducted pile creep tests in a fine uniform sand under dry and saturated conditions. They observed that pile settlement in sand under constant loading continues at a rate proportional to the logarithm of time. In addition, no significant difference was noticed at the settlement rate between dry and saturated sand. These findings were in accordance with [Kuhn and Mitchell \(1993\)](#) who pointed out that the presence of water did not appear to be active in promoting the sliding between soil particles. [Bowman and Soga \(2003\)](#) stressed the importance of the presence of impurities that may either lubricate or cement the wet soil under loading.

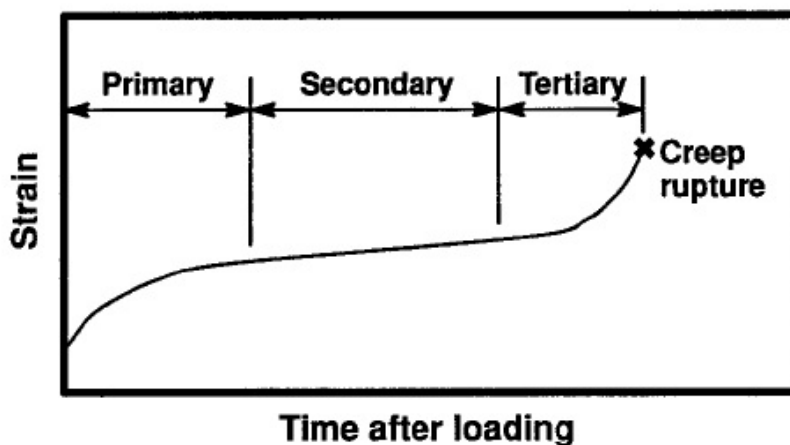


Fig. 6.11 Stages of creep after initial application of load ([Kuhn and Mitchell, 1993](#)).

According to [Mitchell and Soga \(2005\)](#), time-dependent deformation of soils can be attributed to:

- Particle breakage, which can lead to time dependent deformation of sands (Leung et al., 1996; McDowell, 2003; T. et al., 2001). Using experimental evidence from 1-dimensional consolidation tests, Leung et al. (1996) described particle breakage of a dry silica sand under descend loading. Results showed a significant breakage of soil grains after 5 days of loading at a pressure of 15.4 MPa. Vesic and Clough (1968) found that there is very little particle breakage for sand for a mean normal stress below 100 kPa, with dilatancy effect being more significant. On the other hand, they found that dilatancy becomes less significant for stresses above 1 MPa because of higher degree of particle breakage. Stresses caused extensive crushing when they exceeded 10 MPa.
- Ageing, which refers to structural changes occurring in soils during creep which cause an increase of soil stiffness. Until recently, ageing was observed and studied only in fine soils. However, granular soils also exhibit the same behaviour, especially in recently distributed or deposited soils. At constant effective stress, these soils have shown gains in stiffness and strength over time. The phenomenon is attributed to creep processes as well as chemical and biological cementation processes (Mitchell and Soga, 2005).
- Time-dependent changes in soil fabric under loading. Experimental evidence by Bowman and Soga (2005) showed that soil grains have a preferred horizontal orientation along their long axis during pluviation. This is increased when loading is applied to the soil and grains try to orient themselves horizontally. Therefore, particle re-arrangement occurs towards more stable configurations.

In the current study, the deformation of dry sand columns of fraction E sand at constant loading was investigated. The objective of these tests was to determine potential creep behaviour of the dry material at elevated g . The experimental setup consisted of soil column models which were prepared at two different relative dry densities: 45 % ($e = 0.84$) and 56 % ($e = 0.8$). Models were prepared using the sand pouring method and remained dry

throughout the test procedure. Models were subjected to a centrifuge acceleration of 60g for approximately 6 hours each. Soil settlement was measured using an LVDT which was fixed to the top lid. The initial height of each soil column was 400 mm before the start of the test.

The displacement of the dry sandy model at 60g conditions is shown in Figure 6.12. In the case of the initially looser sample, the transition from 1g to 60g caused an initial settlement of approximately 5.3 mm. Due to this settlement, dry density was increased from 1.44 Mg/m^3 to 1.46 Mg/m^3 and void ratio decreased to 0.815. On the other hand, the denser sample was settled by 1.7 mm, due to the gravity increment, and the dry density increased from the initial value (1.47 Mg/m^3) to a final value of 1.48 Mg/m^3 . Void ratio reduced to a final value of 0.79. Initial settlement of the two models brought them into a practically similar density. The initial displacement was followed by a smoother settlement for as long as the soil was subjected to 60g acceleration.

Initial settlement of the soil is caused by the increase in the self-weight stress within the soil mass. During this stage, soil grains reach a stable configuration, subsequently, since total and effective stresses remain stable, no further deformation was expected.

Settlement continued at a steady rate for the duration of the 60g period. Settlement is shown in Figures 6.13 and 6.14 plotted against $\log t$ (where t = time in minutes). A linear curve has been fitted to the experimental data. The time-dependent deformation of the dry sand, which is subjected to a constant loading, is characterized as *creep*. The rate of settlement can be expressed by the *creep rate* C_a in mm/log time cycle. In the case of the initially loose sample, creep rate was found to be 0.056 and for the initially dense sample 0.058. Since the two models were brought to a practically similar density, after centrifugal spinning, creep rate was expected to show similar values for the two tests.

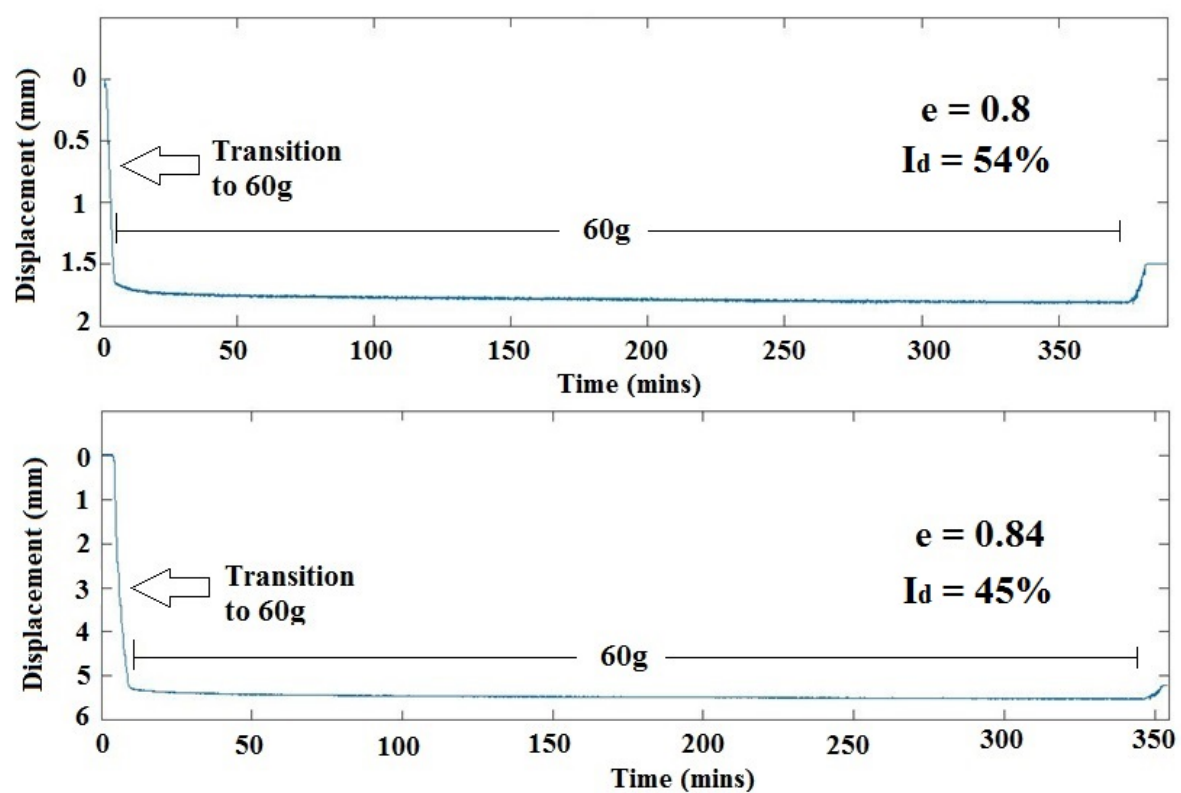


Fig. 6.12 Displacement of the upper surface of the soil column models at 60g centrifugal acceleration.

Comparison between the two tests shows that creep rate was essentially the same regardless of the initial void ratio. This means that the initial density of the soil does not affect its creep rate, rather this is directly dependent on the applied/stress and the current density.

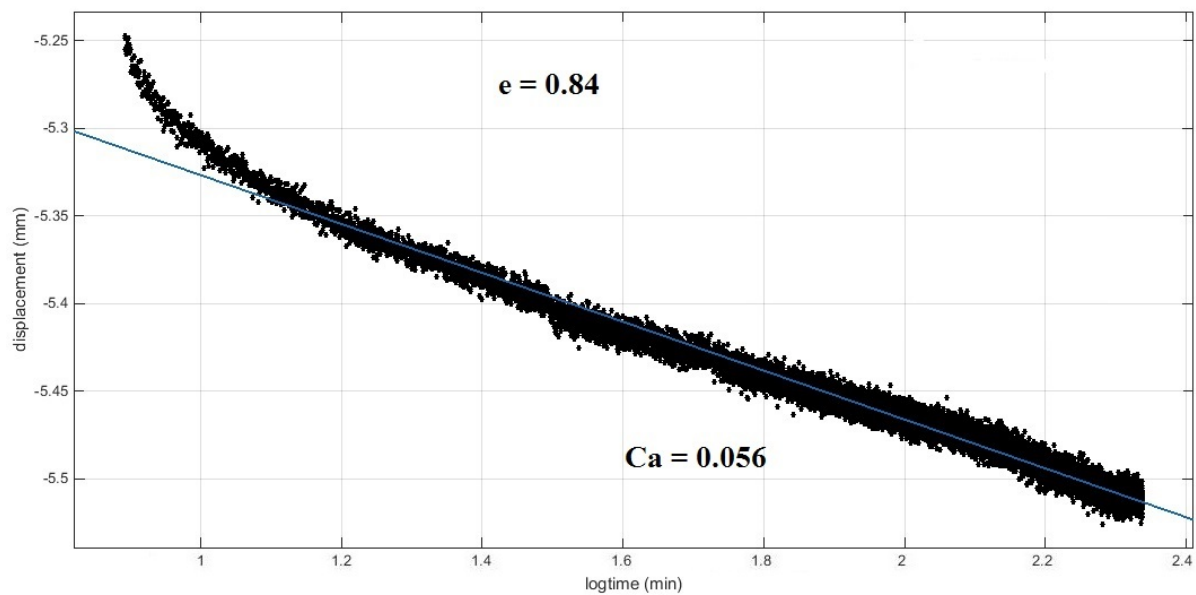


Fig. 6.13 Fitting curve for the dry LB fraction E sand with $e = 0.84$.

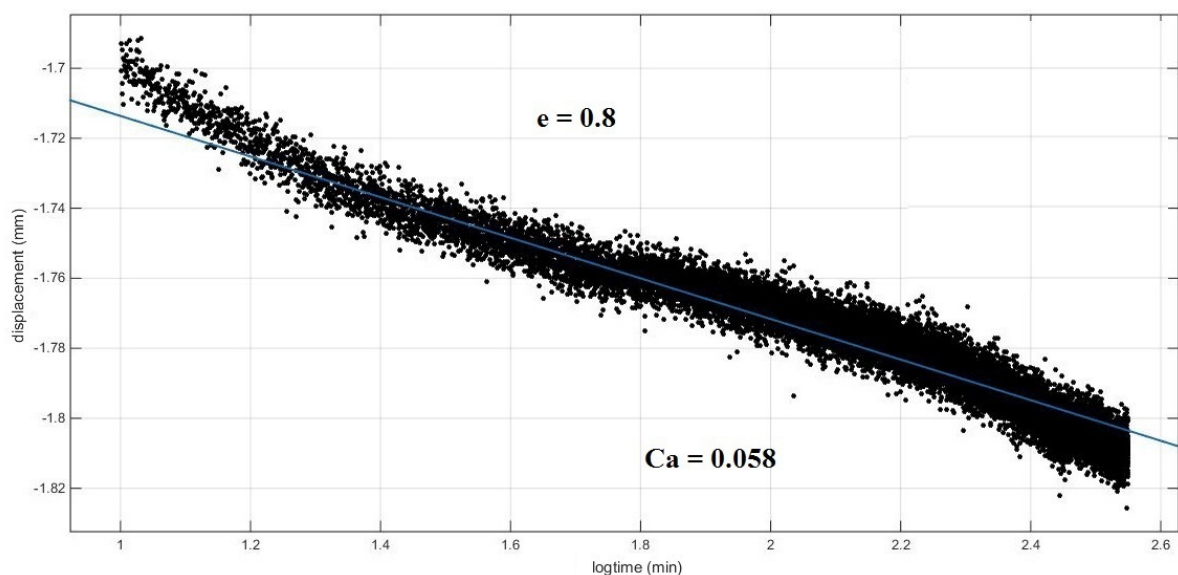


Fig. 6.14 Fitting curve for the dry LB fraction E sand with $e = 0.8$.

Based on the theoretical background presented in this section, the behaviour of dry sand could be attributed to either particle re-arrangement or particle breakage. The maximum stresses acting on the soil column (i.e. ≈ 500 kPa) do not exceed 1MPa which, according to Vesic and Clough (1968) causes little particle crushing. Therefore, creep should be attributed to particle re-arrangement and changes in the orientation of the long particle axis during the constant loading.

6.4 Static wet tests

This section presents the results from the static wet tests (i.e. without rainfall) performed in the soil column models.

6.4.1 Retaining moisture content

Soil column models were brought to 60g in the following order: 5g - 10g - 20g - 30g - 40g - 50g - 60g. In order to keep the consistency of the procedure for all tests, soil models were subjected to each g-level until the settlement of the surface was less than 0.01 mm for at least 10 minutes in model time. The deformation rate was selected instead of the drainage rate due to the slow drainage for fine soils.

Desaturation curves for the seven soils tested (Table 6.1) are shown in Figures 6.15 - 6.21. These present the variation of the gravimetric water content w with time with respect to changes of g-level. Moreover, desaturation curves were also plotted in regards to the degree of saturation S_r . Degree of saturation was calculated based on the variation of the gravimetric water content and volume change, as measured by the settlement of the soil model. The *steps* which appear in Figures 6.15 - 6.17 were caused by the closure of the drainage valve required to empty the

Table 6.1 Specifications of wet soil column tests.

Test ID	Soil type	Initial w	Initial S_r	Initial e
		%	%	-
CHSA8	LB-fraction C	25.9	100	0.69
CHSA12	LB-fraction E	29.4	98.3	0.79
CHSA7	Congleton	24.2	100	0.62
CHSA9	M_{90-10}	13.9	66.7	0.59
CHSA13	M_{70-30}	21.2	81.8	0.68
CHSA10	M_{50-50}	20.5	86.9	0.62
CHSA14	M-Soil	23.8	87.5	0.73

water tank. The displacement of the soil surface during the tests is also shown in these figures. Table 6.1 presents the specifications of each soil column test.

For the coarser LB-fraction C sand (Figure 6.15), the desaturation occurs very rapidly even from 5g when the water content reduced from 25.9% to 2.9%. Increasing centrifugal acceleration to 10g, cause the water content to reduce to 2.6%. Further increments of gravity did not affect moisture content. At the end of the test, LB-fraction C retained a water content $w = 2.5\%$ corresponding to $S_r = 9.7\%$. This is evidence that even medium sands have the ability to retain a small amount of water when subjected to elevated gravity.

Finer sands, i.e. LB-fraction E and Congleton, tend to retain more water during increased gravity. Initial spinning of fraction E caused a rapid drop of the moisture content by 18.5 %, from 29.4 % to 10.9 % (Figure 6.16). Comparing the progress of desaturation in the two LB fractions at 5 g acceleration, drainage in the coarser fraction C was completed much faster than in the finer fraction E. Further increments of gravity in fraction E caused more desaturation, up to 20 g. Between 30g and 60g, the change in acceleration did not cause significant changes to w . The retained values for fraction E at 60g were $w = 3.0\%$ and $S_r = 9.9\%$.

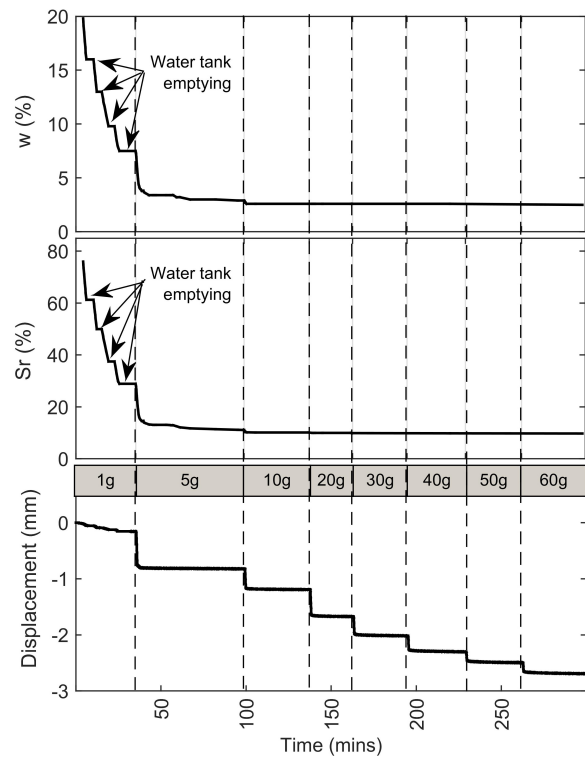


Fig. 6.15 Desaturation curves for the Leighton Buzzard sand, fraction C (model scale).

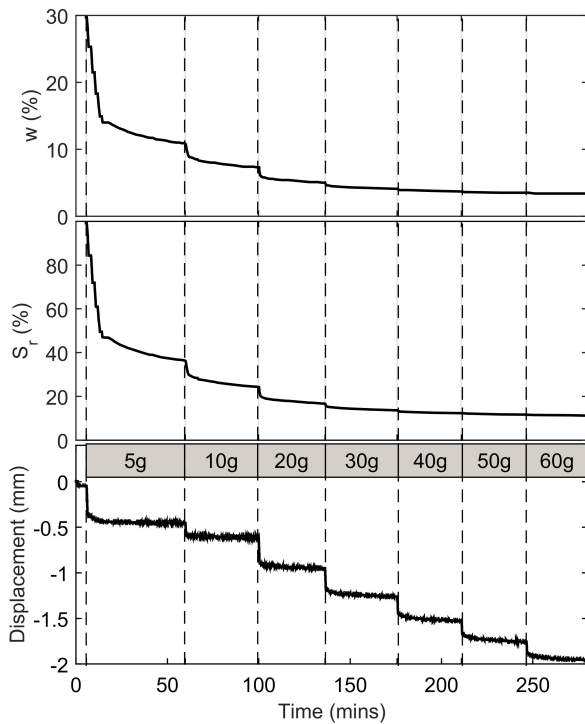


Fig. 6.16 Desaturation curves for the Leighton Buzzard sand, fraction E (model scale).

Figure 6.17 illustrates the desaturation curve for Congleton sand. Under 5g, w reduced to 7.7% from the initial 24.2% while the increment to 10 g caused a further loss of moisture of $w = 1.9\%$. Further increments of gravity cause more desaturation of the soil mass up to 40 g, when w was reduced to 4 %. The final values for w and S_r were 3.4 % and 14.2 %, respectively. Comparing the results for the two fine sands, the final retained w was found to be nearly the same for both cases. However, S_r in Congleton sand was found to be higher than in LB-fraction E sand. This can be attributed to the higher density in which the Congleton sand model was prepared ($e = 0.62$) than LB-fraction E ($e = 0.79$).

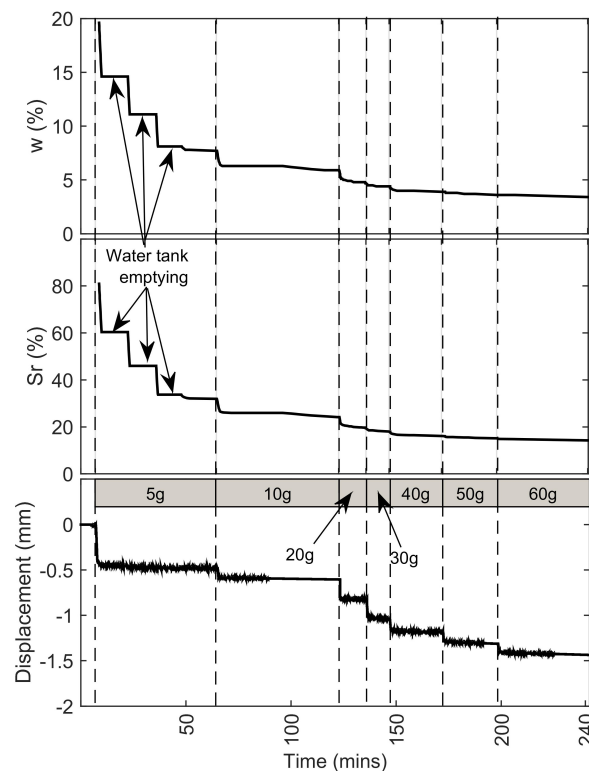


Fig. 6.17 Desaturation curves for the Congleton sand (model scale).

For the silty sands, the retained water at 60g increases considerably with the proportion of finer material added and reaches $w = 4\%$ and $S_r = 18.5\%$ for the M_{90-10} (Figure 6.18), $w = 5.7\%$ and $S_r = 22.1\%$ for the M_{70-30} (Figure 6.19) and $w = 11\%$ and $S_r = 49\%$ for the M_{50-50} (Figure 6.20). In the case of the M-Soil, the 60g values of w and S_r were 13.8% and 52.8%, respectively (Figure 6.21). As desaturation continues, the considerable reduction in hydraulic

conductivity slows the desaturation rate and results in the saturation ratio continuing to change over a prolonged period. This process is sensitive to the particle size distribution of the soil, as silty sands tend to desaturate much slower than clean sands.

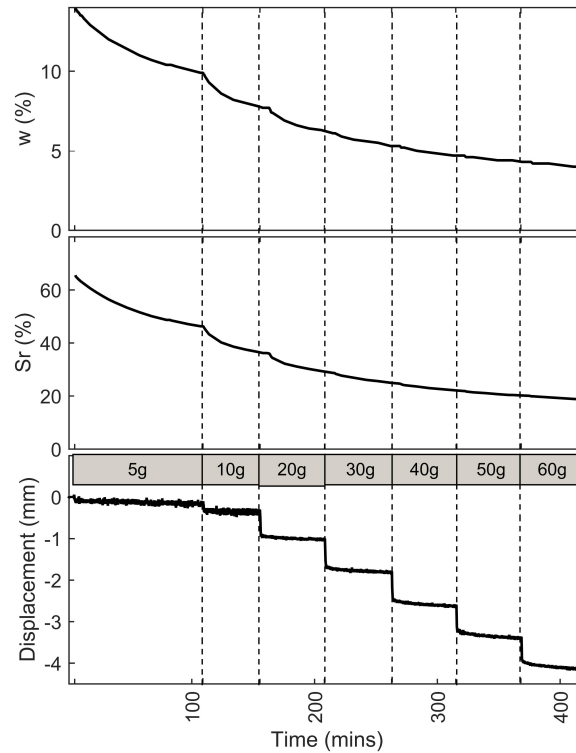


Fig. 6.18 Desaturation curves for the M_{90-10} mixture (model scale).

Tables 6.2 and 6.3 present the retained values of w and S_r , respectively, for the soils tested, at various g-levels. The results show the tendency of finer soils to retain greater amounts of water than coarser soils. On the other hand, medium sands (i.e. fraction C) show a rather unexpected high retained moisture content at 60g, having $S_r \approx 10\%$. Furthermore, the time required to reach equilibrium is significantly higher in fine soils. Based on the fact that hydraulic conductivity reduces significantly under partially saturated conditions, decreasing water content causes drainage to take longer to occur.

At the end of each test (i.e. at 1g), the distribution of the gravimetric water content w within the soil mass was determined by extruding soil samples from different depths and drying them

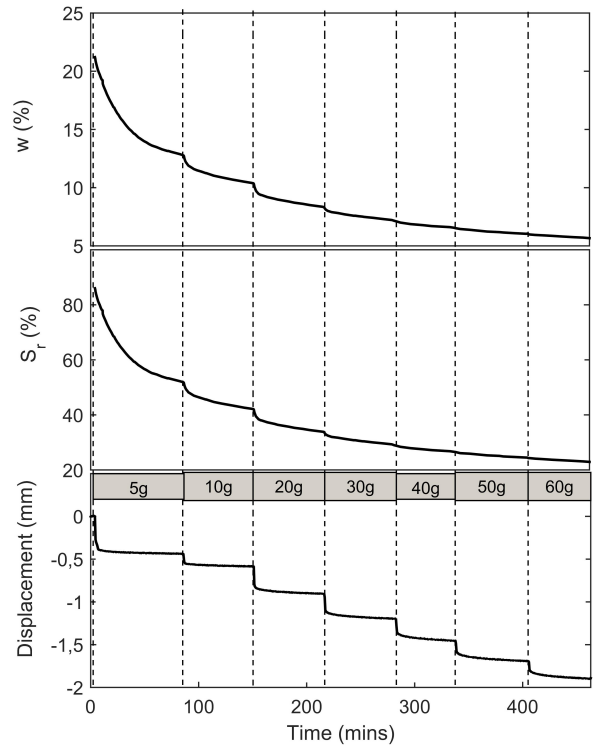


Fig. 6.19 Desaturation curves for the M_{70-30} mixture (model scale).

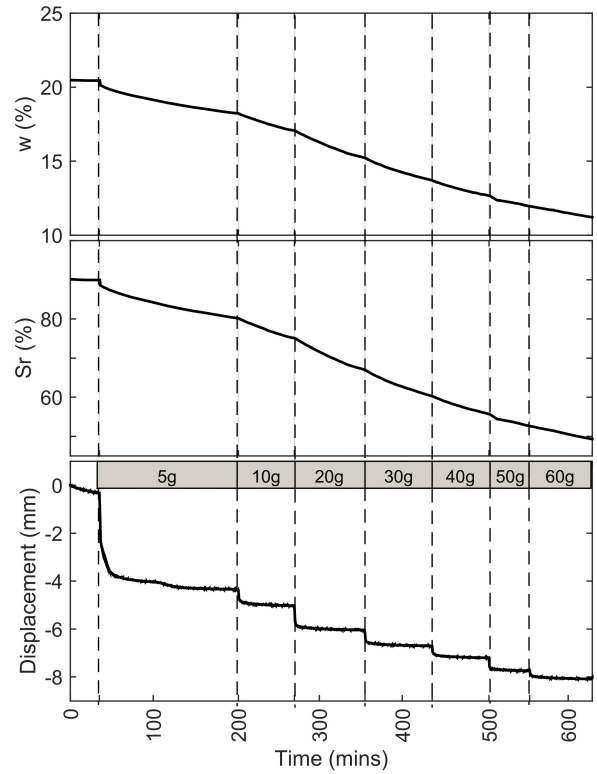


Fig. 6.20 Desaturation curves for the M_{50-50} mixture (model scale).

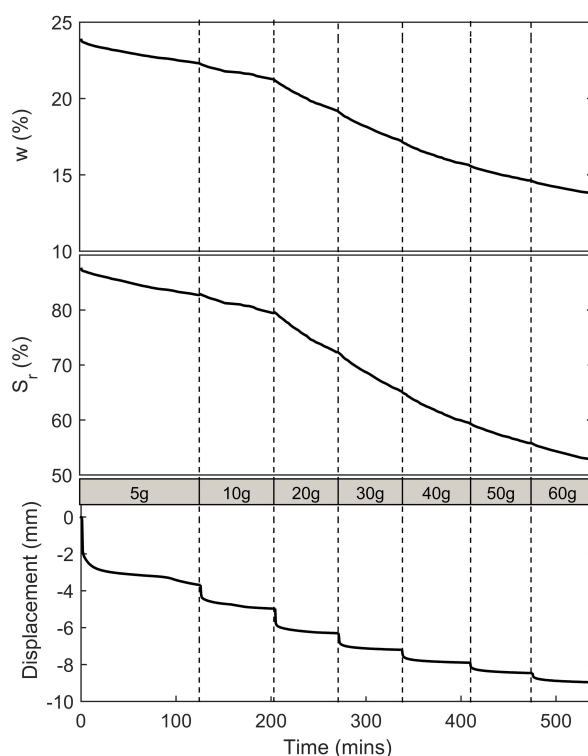


Fig. 6.21 Desaturation curves for the M-Soil (model scale).

Table 6.2 Retained values of bulk gravimetric water content, w , at different g-levels.

Soil name	Initial	5g	10g	20g	30g	40g	50g	60g
–	%	%	%	%	%	%	%	%
LB-fraction C	25.9	2.9	2.6	2.6	2.6	2.6	2.6	2.5
LB-fraction E	29.4	10.9	7.3	5	4.1	3.7	3.5	3.4
Congleton	24.2	7.7	5.8	4.8	4.4	3.9	3.6	3.4
M_{90-10}	13.9	9.9	7.7	6.3	5.3	4.7	4.2	3.9
M_{70-30}	21.2	12.9	10.4	8.4	7.2	6.6	6.0	5.7
M_{50-50}	20.5	18.2	17.1	15.3	13.8	12.7	12.0	11
M-Soil	23.8	22.3	21.3	19.2	17.2	15.7	14.6	13.8

Table 6.3 Retained values of degree of saturation, S_r , at different g-levels.

Soil name	Initial	5g	10g	20g	30g	40g	50g	60g
—	%	%	%	%	%	%	%	%
LB-fraction C	100	11	10.1	10	10	9.9	9.8	9.7
LB-fraction E	98.3	36.4	24.3	16.7	13.6	12.3	11.7	11.2
Congleton sand	100	31.6	23.8	19.7	18	15.9	14.8	14.2
M_{90-10}	66.7	46.3	35.9	29.4	24.6	21.9	19.5	18.5
M_{70-30}	81.8	49.8	40.1	32.4	27.8	25.5	23.6	22.1
M_{50-50}	86.9	79.6	74.9	67.4	61	56.1	53.5	49
M-Soil	87.5	82.7	79.4	72.3	65.2	59.5	55.8	52.8

in the oven at 105°C. The obtained distribution of w with depth for all soils is shown in Figure 6.22. The same type of plots are shown in Figure 6.23 in terms of S_r . The dashed line in the plots corresponds to the values of w and S_r determined using the soil column apparatus system at 60g, thus expressed as the bulk (or mean) values of the soil mass at the end of each test.

The distribution of moisture content in the soil model with depth was not expected to be uniform at the end of the test. This was observed in finer soils, while in coarser soils water distribution was more or less uniform with depth. In finer soils, w and S_r increased with depth. This shows the accumulation of water within a zone near the bottom of the soil mass. This zone started from about half the height for the M_{90-10} mixture while for the M_{50-50} the zone started near the upper section of the soil. M-Soil, on the other hand, showed a gradual increment of water content from the top of the soil model to a depth of 200 mm. Below that depth, the water content of the soil remains practically constant. Singh and Kuriyan (2002) observed the same non-uniform distribution of water content in silty soil columns 50 mm long, after testing them in the centrifuge for 480 and 960 min.

Comparison of w and S_r between the values obtained from the soil column apparatus at 60g and the oven dried samples is also illustrated in Figures 6.22 and 6.23. The comparison shows that the determined mean moisture content for each soil lies between the minimum and maximum values of its moisture profile. In the case of M_{90-10} , the determined w is close to its minimum value while for Congleton sand it is close to its maximum value. However, there is a good agreement between the two types of results which provides evidence of the reliability of the measuring method using the soil column apparatus.

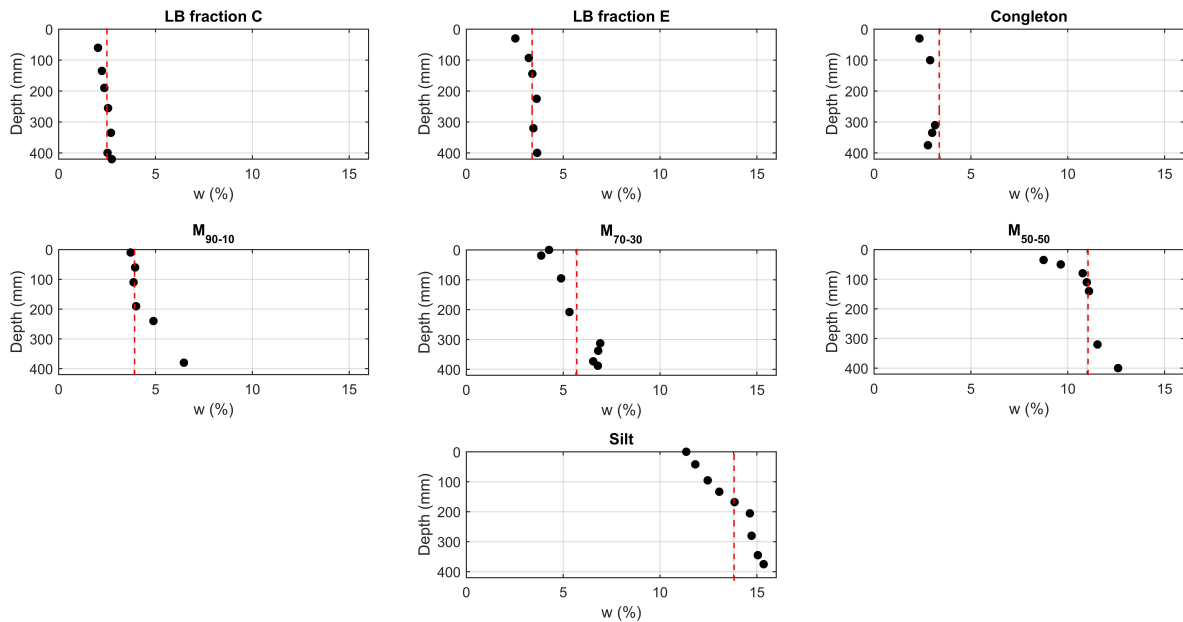


Fig. 6.22 Variation of w with depth for the soil columns at the end of the centrifuge test.

The experimental data of the retained S_r values at each g-level for the soils tested are plotted in Figure 6.24. Apart from LB fraction C sand, which seems to retain a consistent amount of water throughout gravity variation, all soils show a similar response to increased gravity. Also, the experimental results show that there is a discrete difference in S_r for mixtures M_{70-30} and M_{50-50} . This indicates that the proportion of M-Soil in the mixtures becomes significant, in terms of drainage, when it is between 30 % and 50 %.

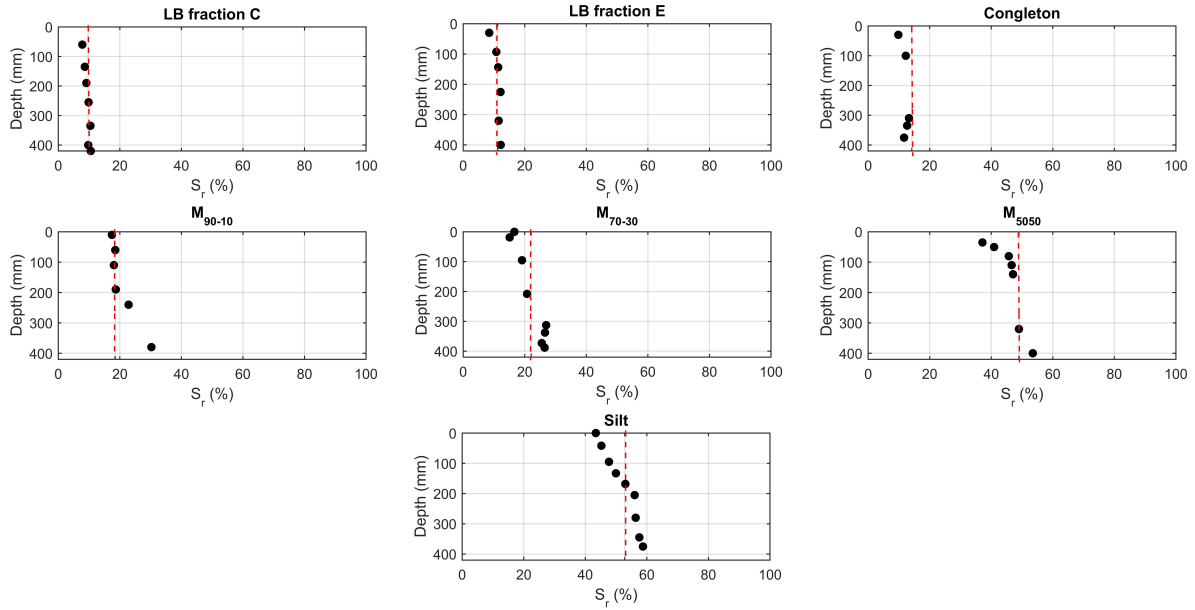


Fig. 6.23 Variation of S_r with depth for the soil columns at the end of the centrifuge test.

The experimental points can be fitted well with a power law of the form:

$$S_r = aN^{-b} \quad (6.6)$$

where N is the gravitational acceleration scale factor and a, b are curve fitting parameters.

The calculation of the fitting parameters for each soil as well as the R^2 are presented in Table 6.4. These results show that there is a general trend of parameter a to increase as the soil becomes finer. Also, parameter b reduces as the grain size becomes smaller. The R^2 for the M_{50-50} and M-Soil is reduced indicating higher scatter of the experimental results. This, however, can be attributed to the fact that these soils showed a continuous drainage behaviour at the end of each g -level, due to the reduced permeability. Therefore, equalisation did not occur and the final retained values for w and S_r could slightly differ from those recorded.

The normalised values for gravimetric water content (w_n) and degree of saturation (S_{r_n}), with respect to the initial values w_o and S_{r_o} , respectively, are presented in Table 6.5. The differences of S_{r_n} between the soils is characteristic of their ability to retain water at high

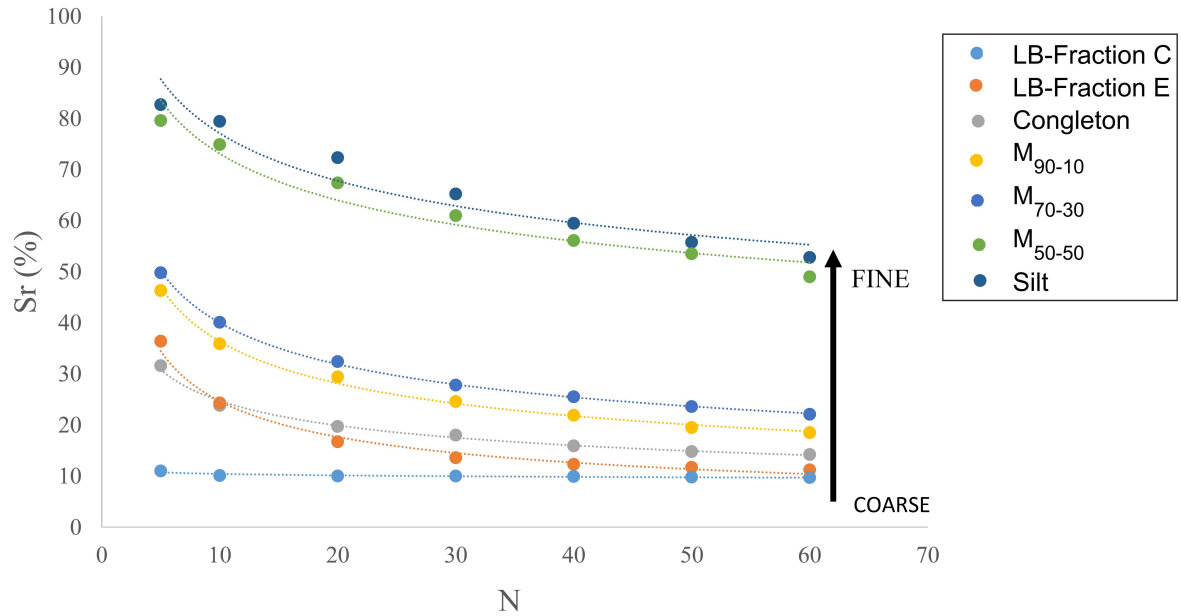


Fig. 6.24 Variation of S_r at different g-levels for all soils tested.

Table 6.4 Calculated fitting parameters and goodness of fit, R^2 for the fitting curves.

Soil name	a	b	R^2
LB-fraction C	11.5	0.04235	0.8178
LB-fraction E	81.88	0.5143	0.992
Congleton sand	52	0.3209	0.992
M_{90-10}	83.22	0.3617	0.996
M_{70-30}	84.26	0.3242	0.999
M_{50-50}	110.5	0.1821	0.95
M-Soil	114.3	0.1741	0.926

Table 6.5 Normalized ($w_n = w/w_o - S_{r_n} = S_r/S_{r_o}$) values for water content and degree of saturation

	w_n	S_{r_n}
LB-fraction C	0.0965	0.097
LB-fraction E	0.1156	0.1139
Congleton sand	0.1405	0.1420
M_{90-10}	0.2804	0.2774
M_{70-30}	0.2669	0.2702
M_{50-50}	0.5361	0.5639
M-Soil	0.5789	0.6036

g-levels. For the coarser soil (fraction C sand), the final S_r was reduced by 90.3 %. The reduction for the fine sands (i.e. fraction E and Congleton) was 88.6 % and 85.8 %, respectively. Soils M_{90-10} and M_{70-30} had similar reduction of S_r at 60 g, with 72.3% and 73% respectively. Finally, the finest soils retained the highest proportion of their initial water with only 43.6 % reduction for the M_{50-50} and 39.6 % for the silt. The similarity of the S_{r_n} values for the finest soils confirms that the behaviour of the sandy silt mixture is dominated by the presence of M-Soil when the silt makes up at least 30% of the soil. The increase of silt content by 20%, from M_{70-30} to M_{50-50} , results in approximately twice the amount of retained water at 60g.

6.4.2 Pore water pressure variations

The evolution of the pore water pressure in the soil mass during the gravity increase and the consequent drainage was investigated for three soils: fraction E, M_{70-30} and M-Soil. Saturated PPTs were placed in the middle of the soil columns at a depth of 205 mm below the soil surface. As mentioned in Chapter 5, the PPTs which were installed in fraction E sand models cavitared when the model started to spun on the centrifuge. In this section, PPT head (porous stone) was covered with a saturated silty paste when installed in fraction E and M_{70-30} soils

in order to avoid cavitation at Ng. This method was evaluated in the soil column centrifuge tests.

Pore water pressure measurement for LB fraction E sand is shown in Figure 6.25. At the first increment of g-level, desaturation causes the rise in suction to about 8 kPa. Closure of the drainage valve, for emptying the water tank causes sharp increments of pore water pressure due to the accumulation of water which moves downwards from the top layers to the bottom of the model. Further increments of g-level cause a gradual increment of suction which evolves as water drainage continues.

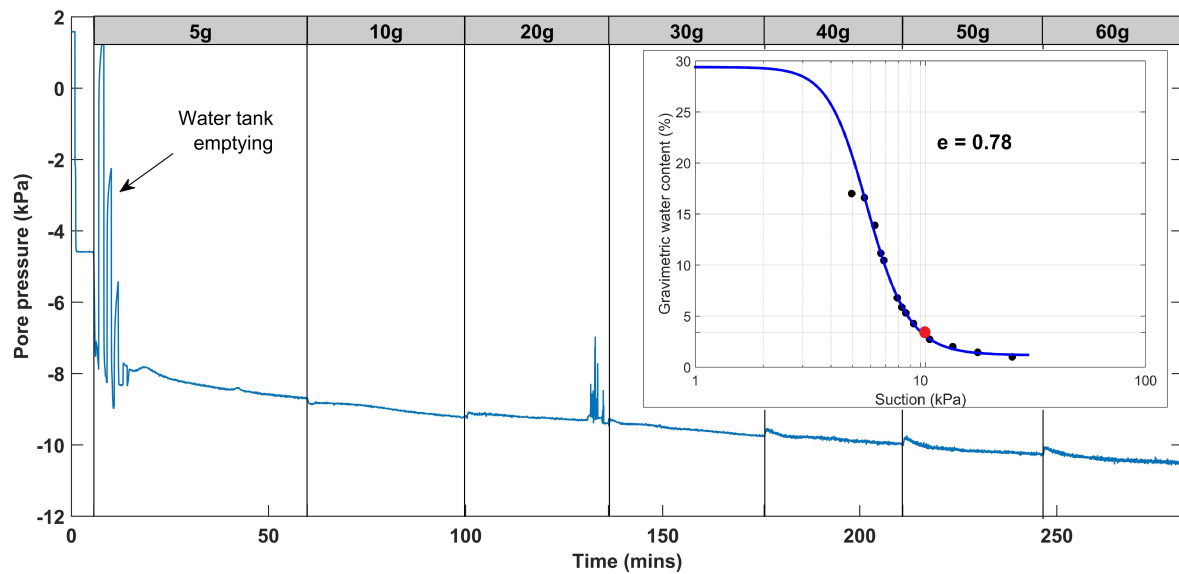


Fig. 6.25 Pore water pressure variation caused by drainage due to gravity increase, on Leighton Buzzard sand, fraction E (model scale). The red point in the subplot corresponds to the value of suction determined in the soil column test at 60g.

At the end of the 60 g acceleration, suction was measured to be 10.5 kPa. From the desaturation curve for fraction E sand, (Figure 6.16) the residual gravimetric water content was determined to be equal to 3.4%. The values of suction and w were plotted on the drying SWRC of fraction E sand which was determined in Section 4.2.8. As can be seen in the subplot (Figure 6.25), the determined suction value for the residual w fits well to the SWRC of the soil. This

indicates that the use of the silty paste to cover the porous stone did not adversely affect the ability of the PPT to measure suction, whilst protecting the porous stone from drying. For this reason, the silty paste cover for the PPTs was used in the slope model tests which are described in Chapter 7.

The evolution of negative pore water pressure in the M_{70-30} mixture is shown in Figure 6.26. In this case, the rise of suction was higher than in fraction E and reached a value of 18.5 kPa at 60g. This corresponds to a residual w of 5.66 % as determined from the desaturation curve. Higher values of suction in the M_{70-30} than in fraction E sand were expected, since it is a finer soil.

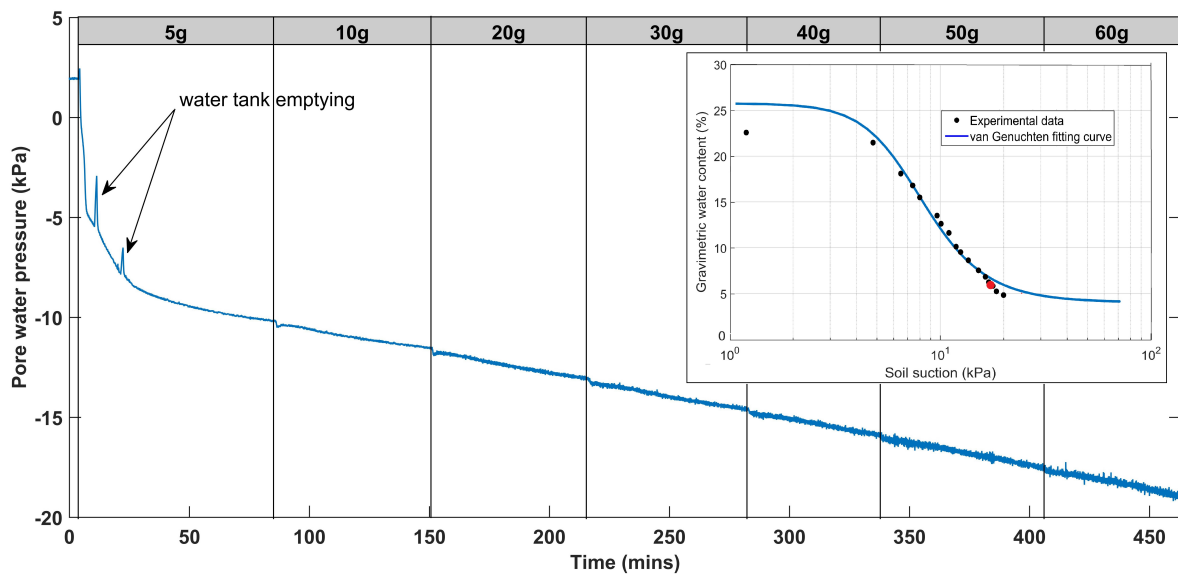


Fig. 6.26 Pore water pressure variation caused by drainage, due to gravity increase, on M_{70-30} (time in model scale)

In Figure 6.27, the determined values of suction for the M-Soil are presented. M-Soil shows significantly higher values of suction at 60g, up to 27 kPa. The residual w determined from the desaturation curve was found to be 13.8 %. As can be seen in the plot, increments in the g level caused significant increases to the pore water pressure. This is attributed to the low

permeability of the M-Soil which reduces the seepage velocity of the water and instantly allows to develop an undrained condition. Therefore, total stress increments, caused by a change in the g-level, initiate excess pore water pressures.

The undrained conditions were also observed at the start of the centrifuge test. As gravity was increased, pore water pressure was increased by approximately 8 kPa before dissipation. Comparing with the behaviour of M_{70-30} and fraction E sand, the excess pore water pressure was more significant for the finer soil. Consequently, this phenomenon strongly depends on the speed of the excess pore pressure dissipation and therefore on the soil permeability. Fine sand was able to dissipate this increment instantly, while for M_{70-30} the excess pore water pressure was only 1 kPa.

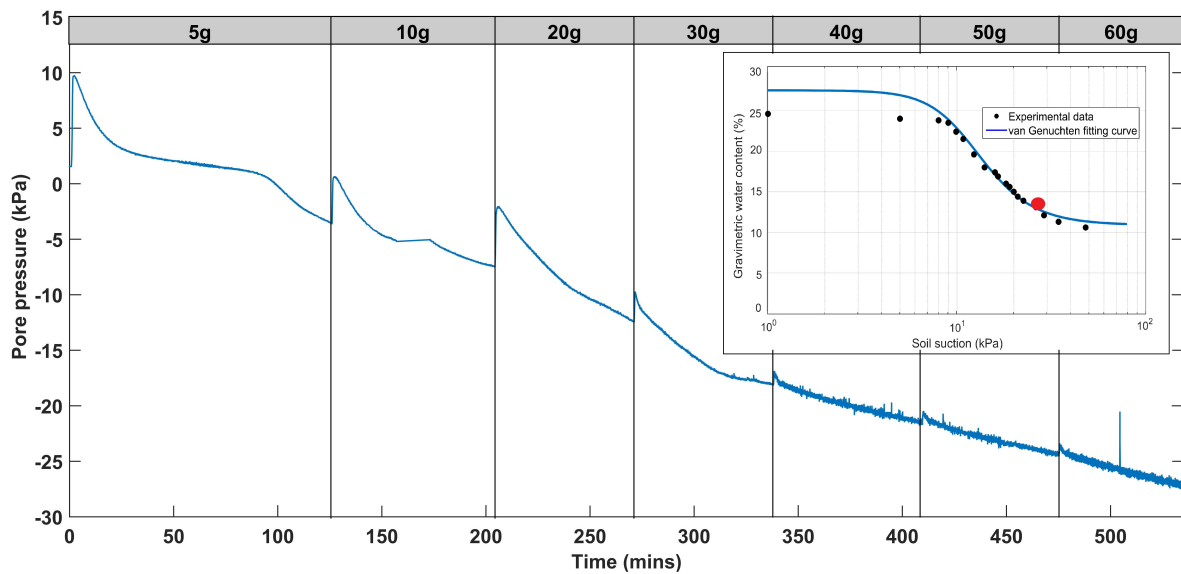


Fig. 6.27 Pore water pressure variation caused by drainage, due to gravity increase, on M-Soil (model scale).

6.5 Infiltration tests

Rainfall conditions were applied to fraction E sand columns in order to investigate the implication of water infiltration to the pore water pressures, especially the elimination of suction. For these tests the soil mass was instrumented with a saturated PPT at a depth of 205 mm below the soil surface. The porous stone of the PPT was covered with a silty paste to prevent cavitation. Moreover, a PPT (without a porous stone attached) was used to measure the air pressure conditions within the soil chamber. The soil model was brought to 60g in increments of 10g. At 60g, rainfall conditions were applied to the model using the M15 type spraying nozzles illustrated in Figure 6.1. Three different rainfall durations were applied: 30 s, 60 s and 300 s. In prototype scale, these correspond to rainfall durations of 30 h, 60 h and 12.5 d, respectively.

The results from the infiltration tests are presented in Figures 6.28 and 6.29. Rainfall initiation causes a minor increase of the air pressure within the chamber. This increment was less than 0.2 kPa which caused a minor increase to the pore air pressure and, consequently, suction within the soil.

Pore water pressure experienced an initial slight increment of less than 1 kPa at rainfall initiation. Two potential reasons for this increment can be identified. The first is the air pressure change due to the spraying nozzle operation. The second source is due to the impact force of the wetting front as it moves downwards from the top of the soil model to the position of the tensiometer. The rapid filling of the partially saturated soil pores with water expels the air causing an increase of pore air pressure. The arrival of the wetting front to the depth where the tensiometer is located is identified by the sharp increment of the pore water pressure.

Results show that the specific rainfall rate was able to cause elimination of suction and the increase of pore water pressures to a marginally positive value (i.e. ≈ 2 kPa) for a rainfall

duration of 30 s (in model time). At the end of the infiltration period, soil drainage causes suction to gradually rise back to the same value as before rainfall. For a slightly longer rainfall event (i.e. rainfall duration of 60 s in model time), pore water pressures reached the same value (i.e. 2 kPa), but then slightly reduced and finally remained approximately constant at 0.8 kPa. Again, at the end of the rainfall period, pore water pressure reduced gradually to its original value.

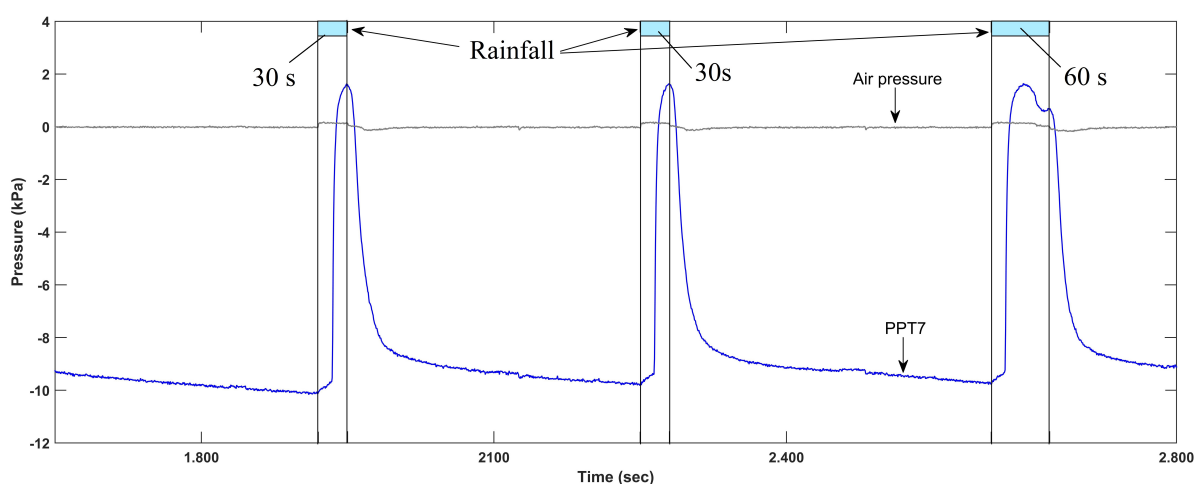


Fig. 6.28 Pore water pressure variation caused by three rainfall events (i.e. 30 s, 30 s and 60 s in model scale).

The pore water pressure change for the 300 s rainfall event are shown in Figure 6.29. Once again, the initial slight increment of pore water pressure was followed by a sharp rise to almost 2 kPa. Then, a pressure drop took place which can be attributed to the departure of the wetting front at the depth of the PPT and the establishment of steady state flow conditions in the soil. For the rest of the rainfall period, pore water pressures increased very slowly. This indicates that rainfall conditions applied to the soil cause a sharp increment of pore water pressures and the elimination of suction at the time of the water front arrival; longer rainfall periods do not contribute significantly to further changes in pore water pressure.

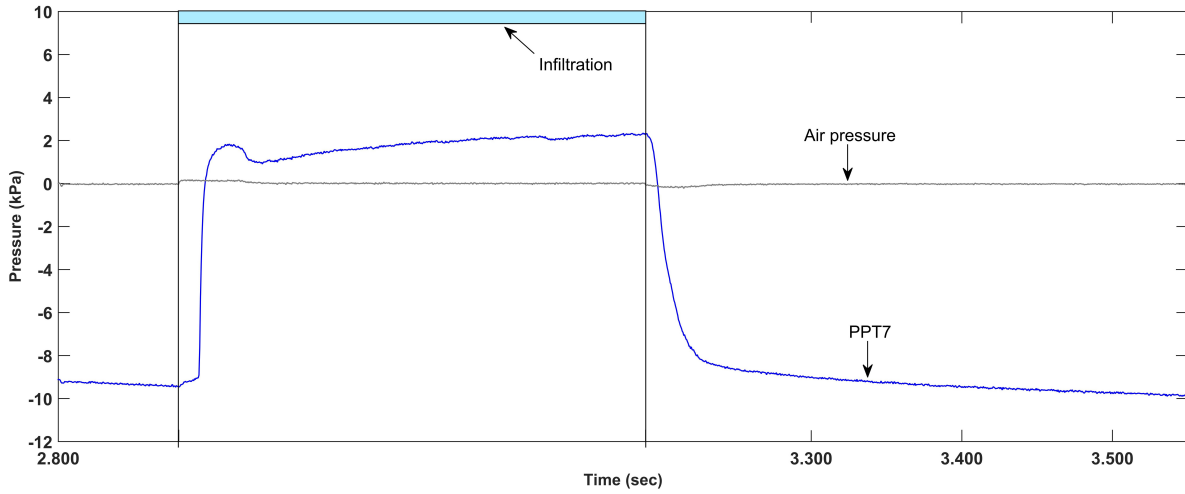


Fig. 6.29 Pore water pressure variation caused by a major rainfall event (model scale).

6.6 Discussion

In Figure 6.30, the evolution of gravimetric water content (taken by the soil column tests) at different g-levels in plotted on the SWRC of fraction E, obtained with the tensiometer method (Chapter 4). The soil starts from an initial water content at near saturation which implies a small value for suction. As desaturation due to g-level takes place, the SWRC path is followed and suction is increased.

The shear strength of unsaturated soils can be written, in terms of the degree of saturation of the soil as follows (Zhou et al., 2016):

$$\tau = c + [\sigma - u_a + (u_a - u_w)S_r] \tan \phi \quad (6.7)$$

Equation 6.7 highlights the effect of hydraulic behaviour (water retention) on the mechanical behaviour (shear strength) of unsaturated soils. Also, it indicates that shear strength of unsaturated soils is related directly to its SWRC. Therefore, it is concluded that shear strength of unsaturated soil slopes is increased when they are subjected to centrifuge acceleration and drainage conditions apply.

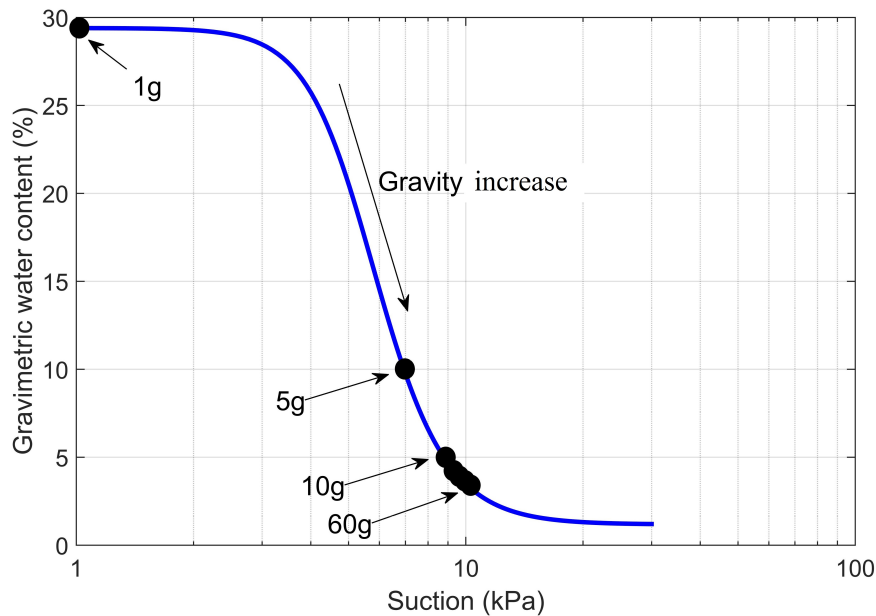


Fig. 6.30 Water content and pore water pressure evolution due to gravity increase for fraction E sand, in terms of its SWRC.

Figure 6.31 shows a typical relationship between the SWRC and the shear strength of a soil. Initially, there is a linear increment of the shear strength with respect to suction up to the air-entry value. As desaturation continues and suction increases beyond the air entry value, air intrudes into the soil pores and the amount of water at the soil particle contacts reduces. This condition indicates that water exists in a discontinuous form within the pores and the area of contact between the grains and the water meniscus is reducing. Therefore, the increase of the shear strength becomes non linear within the transition and residual regimes.

6.6.1 Conclusions

The conclusions drawn from the soil column tests presented in this chapter are the following:

- When a soil is subjected to centrifugal acceleration and drainage conditions are enabled, the degree of saturation will be reduced in a way which depends on the acceleration level and the grain size distribution of the soil. Finer soils tend to retain greater amounts of

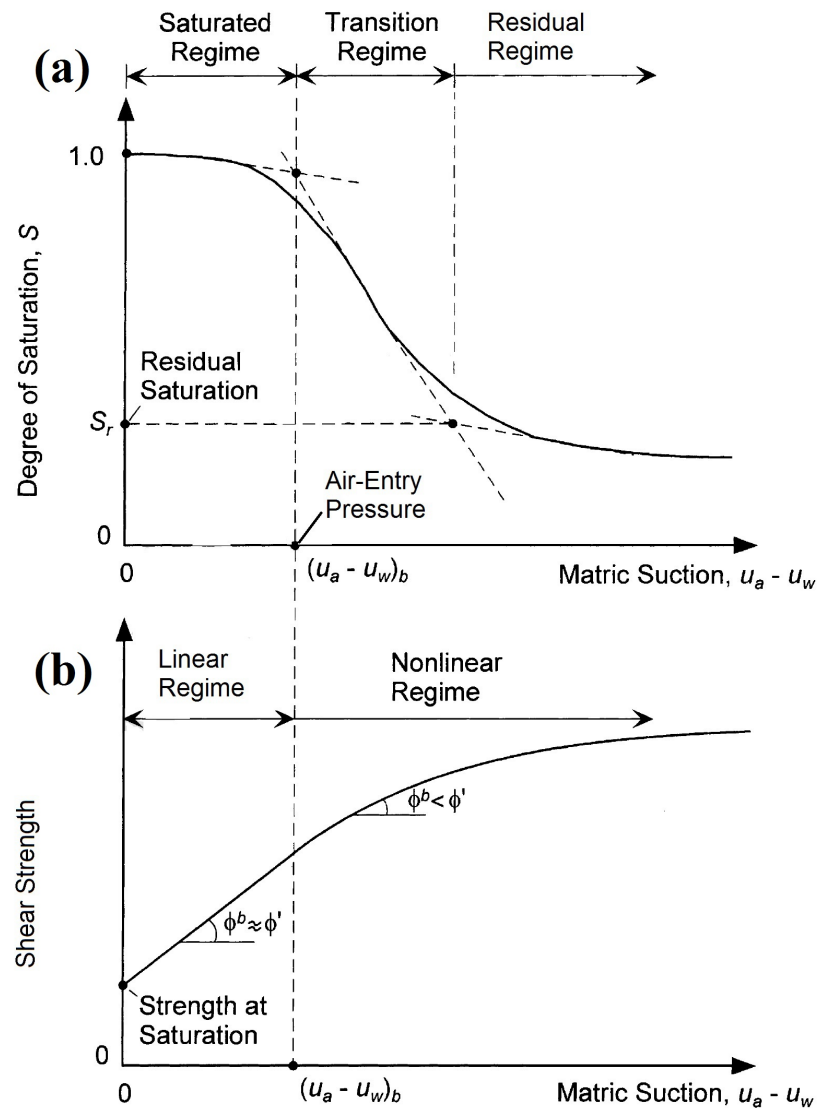


Fig. 6.31 (a) Typical SWRC. (b) Shear strength behaviour of the soil as it relates to the SWRC (Vanapalli et al., 1996)

water at the same g-level. On the other hand, coarse soils (i.e. medium and fine sands) retain a small but still significant amount of water at elevated g-levels.

- Water content profiles in LB fraction E sand at the end of the centrifuge spinning were fairly uniform with depth. Therefore, pore water pressure distribution with depth should be uniform and it can be concluded that suctions are more or less equal with depth for medium grained soils. On the other hand, finer soils exhibited non-uniform distributions of water content with depth. This indicates that suction is not constant with depth for the fine-grained soil; it increases with depth;
- Desaturation due to the increased gravity field causes a rise in the level of suction. Therefore, effective stresses and shear strength of the soil are also increased due to the increased gravity.
- Rainfall infiltration conditions in fraction E sand cause a reduction and even elimination of suctions at high rainfall intensity. However, the duration of rainfall does not have significant implications to the pore water pressure profile. Even with a high duration, pore water pressures are not affected significantly after the initial increment which indicates the arrival of the wetting front. Therefore, it can be stated that pore water pressures are affected mainly by the rainfall intensity and less by rainfall duration for steady state where drainage can accommodate quantity of rainfall. This was confirmed by the deformation of the sand slopes, shown in Chapter 5, which was not affected by the duration of rainfall.

Chapter 7

Centrifuge tests in silty sand slopes

The simulation of two failure mechanisms were attempted in Chapter 5. The first mechanism involves the loss of suction due to wetting in a soil slope of an angle higher than the angle of friction due to wetting. The second mechanism involves the collapse of the saturated voids due to increased pore water pressure which reduces the effective stresses within the soil mass. The second mechanism was not able to be reproduced in the tests conducted in initially unsaturated soil slopes with a low GWT, since the models showed unexpected resistance to failure. These results confirm the findings by [Take et al. \(2004\)](#) who suggested that static liquefaction is not occurring in initially unsaturated slope models under centrifuge testing. On the other hand, soil slope models failed under a condition of an elevated (i.e. shallow) GWT.

The initiation of instability in soil slopes subjected to rainfall, under centrifugal acceleration, has been performed in the past by applying various boundary conditions. One of the most common practise is the presence of layering in the soil model. [Take et al. \(2004\)](#) studied the performance of a soil slope when flow is initiated within the soil due to an inclined and impermeable boundary at a low depth below the soil surface. [Min \(2006\)](#) applied the same technique by introducing a dense layer with decreased hydraulic conductivity below the slope surface. While this technique has been proved effective, it does not answer to the following questions: (1) why a uniform sand slope does not fail even under extreme rainfall conditions and

(2) why centrifuge modelling does not reproduce the prototype conditions in uniform slopes. The use of almost impermeable boundaries within the slope leads to provoked instabilities which occur on this surface, therefore the shear plane is usually defined prior to the centrifuge testing.

7.1 Introduction

The stability of a granular slope can be described by the factor of safety, which is given by the following equation:

$$FoS = \frac{\tan\phi}{\tan\beta} \quad (7.1)$$

where FoS is the factor of safety, ϕ is the friction angle and β is the angle of the slope. The inclination of the sandy slope models was 50° while the peak angle of friction for the LB fraction E sand was found to be 32° (section 4.2.9). Therefore, factor of safety is calculated at 0.52, indicating instability. However, this is not the case in unsaturated soil slopes, as suction contributes the the soil's shear strength, thus increasing significantly the FoS.

When a coarse soil, like the fine sand used in the centrifuge tests, was tested at a high acceleration (i.e. 60g) it was expected to dry out almost completely. Concerning the extremely high slope angle, models should fail at 60 g even without the application of rainfall conditions. However, this was not the case for the slope model tests with a horizontal water table below the toe.

Soil column tests, presented in Chapter 6, revealed that fine sand retains a significant amount of water when tested at 60g, having a retained $S_r \approx 10\%$. Under these conditions, suction reached a value close to 10 kPa.

The presence of suction in soils contributes to the soil's shear strength and therefore to the slope stability. When a soil model, initially saturated or near saturation, is subjected to a

centrifugal acceleration, pore water drains, driven by the increased gravity. Therefore, the soil comes to a partially saturated condition and suction increases. The relationship between S_r and suction is provided by the SWRC of the soil. The contribution of suction to the shear strength increment depends on the level of suction and the degree of saturation and is given by Equation 7.2:

$$\tau = c' + (\sigma - u_a)\tan\phi' + (u_a - u_w)S_r\tan\phi' \quad (7.2)$$

where τ is the shear strength, c' is the cohesion in terms of effective stresses, $\sigma - u_a$ the total net stress, u_a is the pore air pressure, $u_a - u_w$ is the matric suction, S_r the degree of saturation and ϕ' is the effective angle of friction.

It is concluded that soil desaturation led to a positive increment in shear strength and therefore the slope was able to maintain its stability under static conditions. Furthermore, rainfall did not cause any significant deformation to the slope even at very high rates and for long periods. This implies that PWP's did not significantly increase which can be explained by the increased seepage velocity which is created in the model.

7.2 Theoretical approach

The experimental results shown in Chapter 5 raises questions about the effectiveness of centrifuge modelling to simulate prototype conditions, regarding rainfall-induced landslides. In this section, the influence of centrifugal acceleration to the seepage velocity and, therefore, to the slope stability under rainfall conditions will be investigated.

Scaling of Darcy's velocity is described in the following equation:

$$v_m = N \times v_p \quad (7.3)$$

i.e. flow velocity in the centrifuge model is N times greater than that in the prototype.

The length of the flow path for the rain water which infiltrates the soil is scaled by $1/N$. Therefore, seepage time in the model is reduced by $1/N^2$. Practically, this means that water drains faster in the centrifuge model compared to the prototype condition. This has implication to the build of the excess pore water pressures induced by infiltration. Faster infiltration produces faster dissipation of excess pore water pressures, thus soil collapse, due to pore water pressure increment, does not occur. The internal mechanism leading to this type of failure can be explained by the collapse of the saturated voids and the resulting local and abrupt increase of the pore water pressure (Take et al., 2004). Based on a micro-scale analysis by Askarinejad et al. (2014), if the same soil and structure is used at model and prototype, the scaling factor for the length at the grain scale will be equal to 1. The scaling factor for time for the gravitational falling of a particle at grain scale of:

$$t_{impact,m}/t_{impact,p} = \sqrt{1/N} \quad (7.4)$$

If the same structure is reproduced in the model and prototype, hydraulic conductivity of the soil remains the same. Then, the scaling factor of the time needed for the dissipation of the excess pore water pressures generated by the infiltration is given by:

$$t_{dissipation,m}/t_{dissipation,p} = 1/N \quad (7.5)$$

It is concluded that centrifuge modelling creates the conditions under which the dissipation of the excess pore water pressures occur $\sqrt{N}$ times faster than the collapse of the saturated voids. Therefore, it is practically impossible to induce collapse of the slope in the centrifuge with a horizontal GWT unless the seepage time in the model is reduced. In the case of a shallow GWT, seepage flow is initiated within the soil mass due to the increased hydraulic gradient. Under this condition, pore water pressure can increase dramatically and failure can occur.

To overcome this scaling issue in centrifuge modelling of rainfall-induced landslides, seepage velocity needs to be decreased in the model by $\sqrt{N}$. [Askarinejad et al. \(2014\)](#) performed centrifuge slope tests in a silty sand increasing the kinematic viscosity of the pore fluid by $\sqrt{N}$ times compared to the viscosity of pure water. While this provides a good practise to simulate rainfall impact, keeping the same grain size distribution for both model and prototype soils, it must be accompanied by auxiliary laboratory tests which will identify the influence of viscosity on soil strength properties. Moreover, due to the complexities of unsaturated soils tested in the centrifuge and in order to conserve the SWRC and the profile of the water content at equilibrium, [Caicedo and Thorel \(2014\)](#) proposed the use of the same pore fluid in the model and prototype.

7.3 Experimental validation

7.3.1 Methodology

To investigate this scaling issue in soil models made of fine sand, a new series of tests were designed and performed. In these tests, seepage velocity was reduced by reducing the grain size of the soil. The reduction of the grain size affects the hydraulic conductivity of the soil which reduces as well. In order to meet the $1/\sqrt{N}$ time difference between the collapse and the dissipation of the pore water pressures, the soil used to form slope models had a scaled hydraulic conductivity which was reduced by about $\sqrt{N}$.

To reduce the hydraulic conductivity of the soil, fine sand (Congleton sand) was mixed with a small proportion of M-Soil at a rate 9 : 1, respectively. This small amount of fine soil reduces the saturated hydraulic conductivity of the fine sand. The mean value of k_{sat} for the LB fraction E sand was determined at 5×10^{-5} m/s while for the M_{90-10} mixture that was found

to be 6×10^{-6} m/s, respectively. The ratio $k_{sat,frE}/k_{satM}$ was 8.3, slightly larger than $\sqrt{N}$ (i.e. 7.7). The maximum dry density of the soil was determined in the laboratory using the BS light hammer method and was found to be 1.66 Mg/m^3 . The optimum moisture content was found to be 10.5%. G_s was calculated proportionally using the G_s values of Congleton sand and M-Soil and was determined at 2.62. If this soil is used to build the same slope geometry and tested under the same rainfall conditions, it is possible to overcome the scaling issues described previously.

PPTs were installed in the models in order to measure suction variations during the test. Before the installation of the PPTs, they were saturated and calibrated in the saturation rig. During installation, PPT heads were covered with a small amount of a saturated silty paste in order to retain the contact between the pore water and the porous stone. The effectiveness and reliability of the method was validated in the soil column tests, discussed in Chapter 6.

Two tests were performed in silty sand slope models under rainfall conditions. Both slopes were prepared in the same way and with identical boundary conditions applied. Furthermore, slopes retained the same geometry as in the slope models made of LB fraction E sand from Chapter 5. In test CSI1, the maximum rainfall intensity was 2.44×10^{-5} m/s at the prototype scale, representing an extreme rainfall event. In test CSI3, the minimum rainfall intensity was applied, i.e. 2.05×10^{-6} m/s at prototype scale.

7.3.2 Results

CSI1

The preparation of the slope model took place using the moist tamping method. Soil was mixed with water so as to achieve the optimum moisture content of 10.5% (Section 4.2.5).

Then, batches of the wet soil were sealed in plastic bags for about 24 hours in order to allow the moisture to distribute uniformly. A soil block was prepared in the centrifuge box by compacting the soil into 10 layers with a minimum compaction effort to achieve a relatively loose state. Simultaneously, PPTs were installed in the soil mass at predefined locations during the compaction. The final achieved dry density was 1.49 Mg/m^3 .

Soil samples were taken from different depths at the end of the compaction procedure and dried in an oven. Sampling was done in the soil column which was removed from the soil block in order to cut the slope. Therefore, the slope itself was not affected by the drilling. The distribution of the gravimetric water content from the soil surface up to the level of the slope toe was determined and shown in Figure 7.1.

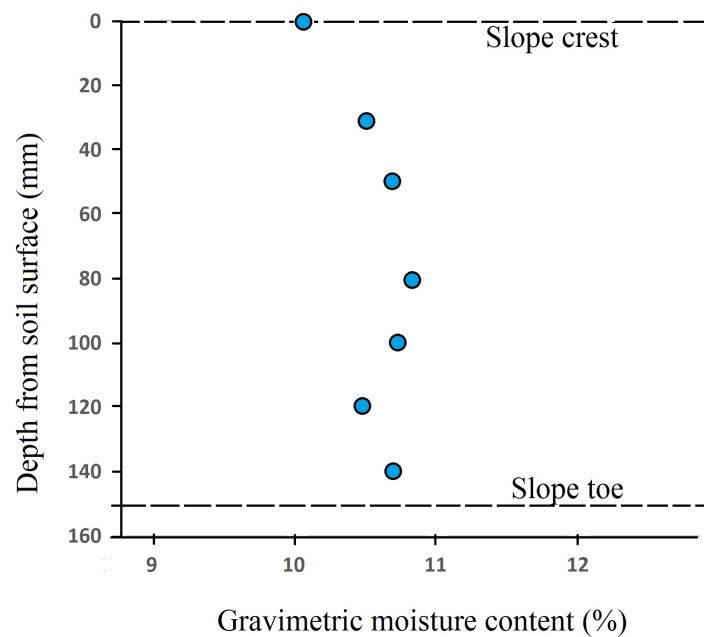


Fig. 7.1 Distribution of moisture content with depth at the end of the compaction process for M_{90-10} soil in test CSII.

Figure 7.2 shows the layout of the slope model and the location of the PPTs. PPT2 and PPT7 were installed 20 mm above the GWT and at the same level with the toe. PPT7 was installed at a depth of 50 mm below the horizontal soil surface. PPT5 was installed 92 mm below the ground level. The geometry of the slope models was similar to the one of the sandy

slope models tested under rainfall conditions (Chapter 5). The slope height was 150 mm with an inclination of 50° . Test was performed at 60g acceleration, so the prototype slope height was 9 m. Soil was left to drain upon gravity increment and the GWT kept horizontal using the side drainage boxes to remove the excess water from the box.

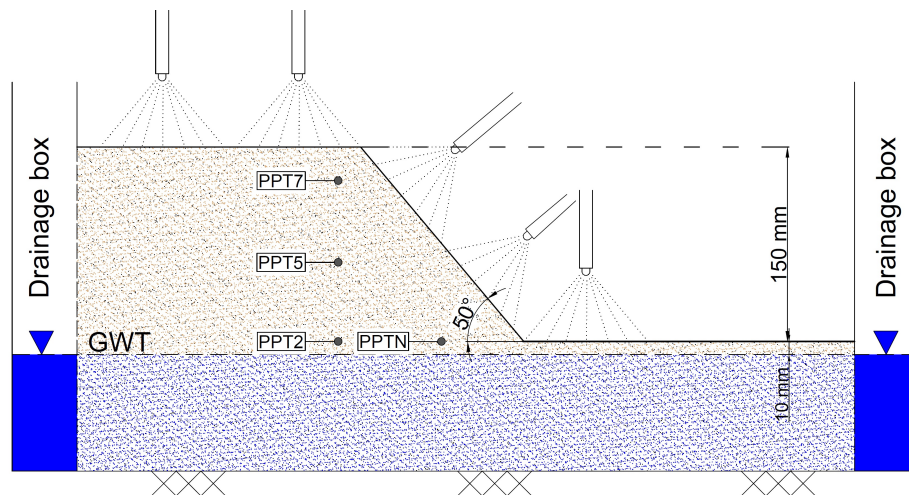


Fig. 7.2 Location of PPTs for test CS11 (Matziaris et al., 2017)

The slope was brought to the testing acceleration of 60g in the following sequence: 1g - 5g - 10g - 20g - 30g - 40g - 50g - 60g. Figure 7.3 presents the variation of the pore water pressures at the four locations during the gravity increase. Pore pressure started to reduce at PPTs 7 and 5 immediately after the model started to spin. Pressure increased in the locations of PPT2 and PPTN showing that water draining from the soil started to accumulate at greater depths before it was removed to the drainage boxes. At 20g acceleration, pore pressure in these locations started to decrease indicating that slope started to desaturate at all its height. This behaviour indicates, also, the impact of the lower permeability since water drains slower than in sandy soil. At an acceleration of 30g, pore pressures at all locations were negative and have almost the same value.

The effect of gravity increase to the stability of the slope is shown in Figure 7.4. The slope model settled due to the gravity increment, causing compaction of the soil. At 60g, the model

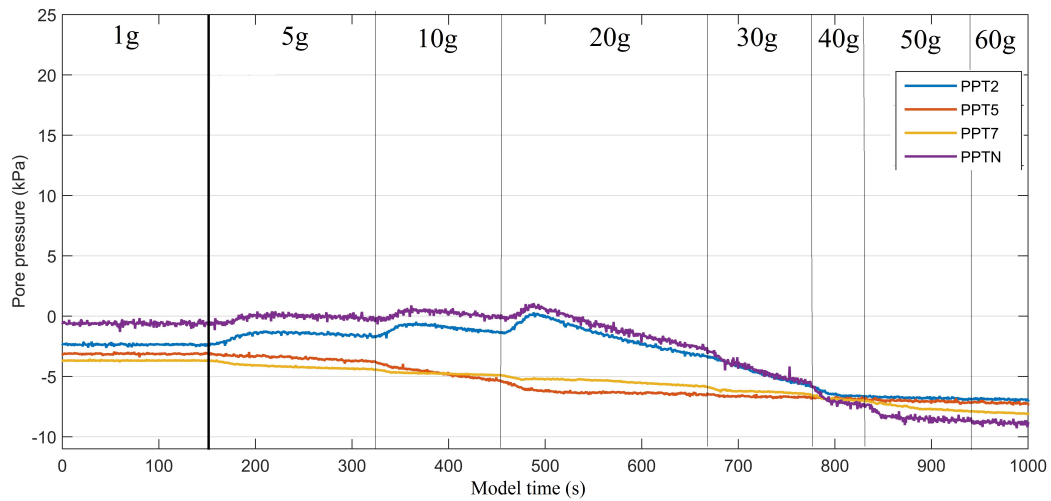


Fig. 7.3 Pore water pressure variation during gravity turn-on for test CSI1 (model time).

settled by a maximum of 1 mm. At a zone close to the slope, displacement vectors are oriented parallel to the slope, delimiting a potential failure surface. In the rest of the slope, displacement vectors are vertical. Comparing the displacement field with the flow lines determined at the flow path test in Chapter 5, it seems that failure surface is parallel to the flow lines of rain water. Therefore, it could be argued that rain water drainage plays an important role on the deformation of the slope at increased gravity conditions.

Rainfall was applied to the slope model at 60g with the highest intensity (2.44×10^{-6} m/s in prototype, corresponding to 2.44×10^{-5} m/s in model). In this case, the relative intensity (i.e. ratio between the rainfall intensity and soil permeability) was 1.35. An initial superficial failure below the crest occurred after 1.25 s of rainfall (75 min in prototype), as shown in Figure 7.5. As rainfall continued, the GWT started to rise showing the difficulty of the ground water to drain towards the drainage boxes due to the reduced permeability. A second failure occurred at the slope crest 22.1 s (1326 min in prototype) after the first failure and 23.7 s (1422 min in prototype) after the initiation of rainfall (Fig. 7.6).

Pore water pressure variation is plotted in Figure 7.7. Just before rainfall, desaturation enabled suctions up to 10 kPa. Immediately after rainfall application, pore water pressure

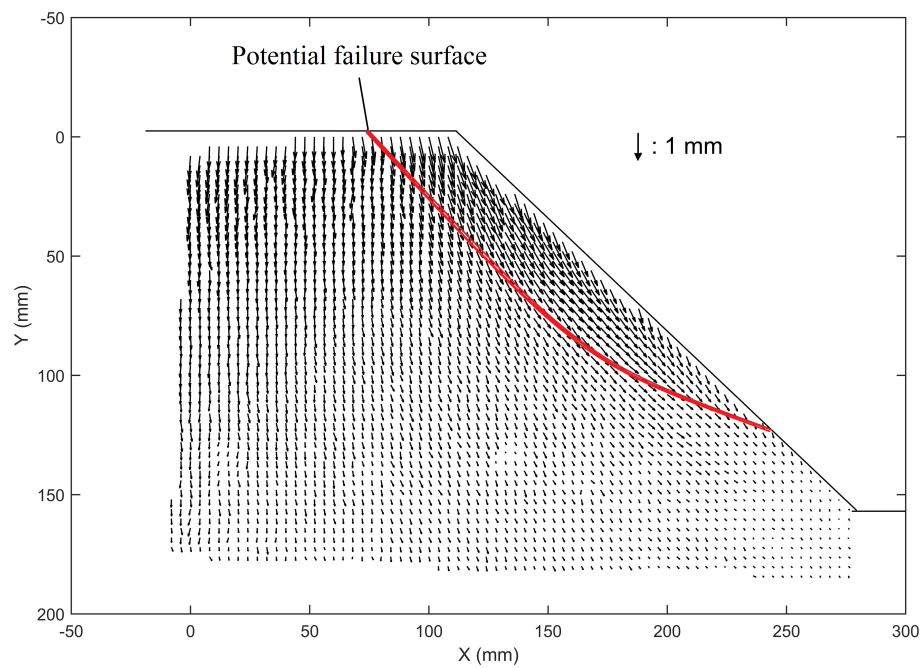


Fig. 7.4 Settlement of the slope, due to gravity increase, at $t=0$ (dimensions in model scale).

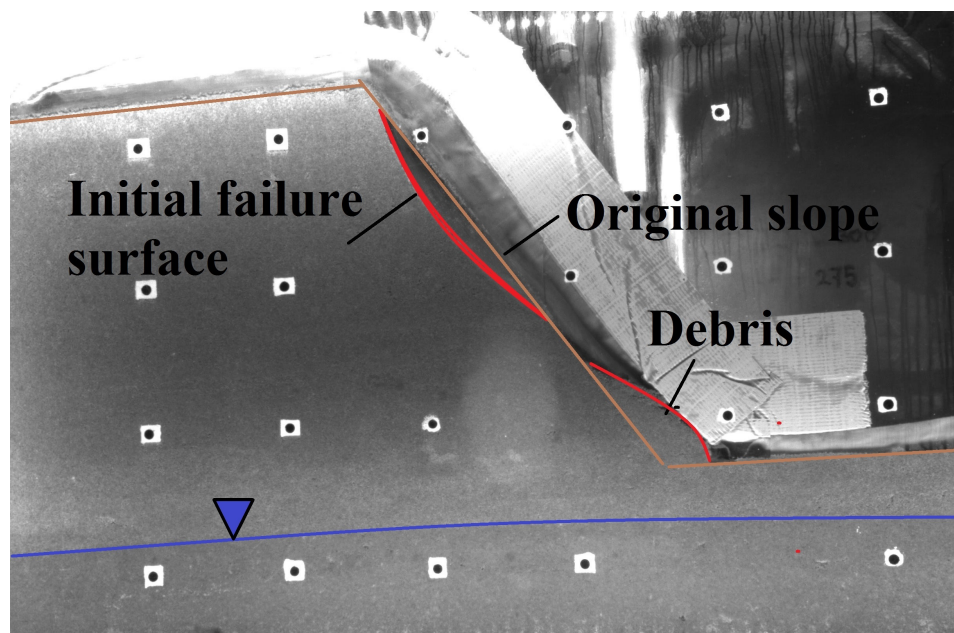


Fig. 7.5 Initial failure of slope in test CSI1 after 1.25 s of rainfall (model scale).

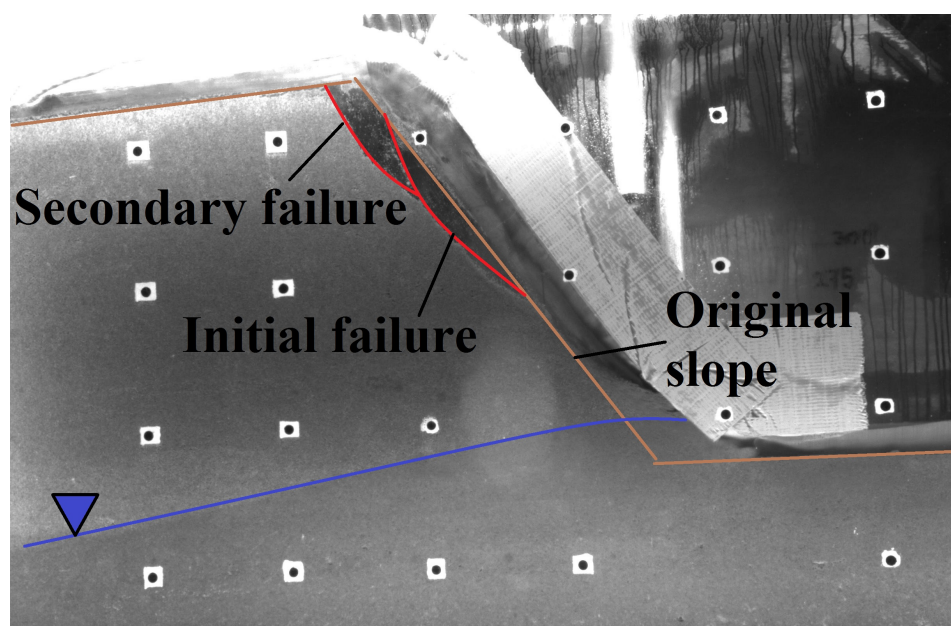


Fig. 7.6 Secondary failure of slope in test CSI1 after 22.1 sec of rainfall (model scale).

started rising simultaneously at all locations which reveals a small increase in the air pressure within the chamber. The first sharp increment occurred at the location of PPT7, which was located near the slope crest and indicated the arrival of the wetting front at this depth. Failure occurred about 1.5 s before pore water pressure at the location of PPT7 increased. This can be explained by the fact that PPT7 was installed deeper than the observed instability. The wetting front which caused an increase in pore water pressures and the subsequent failure reached PPT7 after the saturation of the sliding mass. The next increment in PWPs was at the location of PPT5 and then at the location of PPTs 2 and N. PPTs 2 and N were located near the slope toe and they show increases up to 25 kPa and 23 kPa, respectively due to the rise of the GWT.

Another conclusion drawn from Figure 7.7 is the different response of the pore water pressures in this soil compared to the fine sand (Fraction E). The arrival of the wetting front causes a sharp increment of pore water pressure (i.e. decrease of soil suction) which is followed by a gradual increase in pore pressure until the end of rainfall. This shows that the reduction of the seepage velocity, induced by the reduced permeability, causes higher pore water pressures

than in Fraction E. The actual pore pressures are different at each depth, with deeper layers experiencing higher pressures.

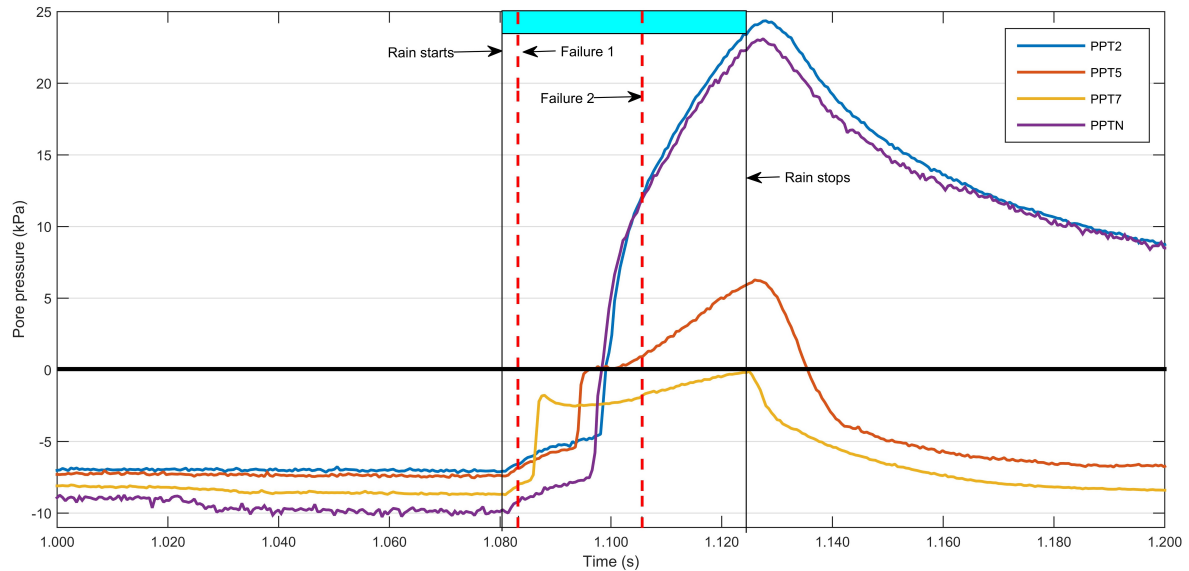


Fig. 7.7 Pore water pressure changes at 60g for CSI1 test (model time).

The deformation evolution of the slope profile during rainfall conditions, before the first failure, is shown in Figure 7.8. Time $t = 0$ s was the initiation of rainfall conditions. The deformation of the slope at $t = 1.12$ s was negligible and no signs of failure were obvious. At $t = 1.18$ s, the spatial distribution of the displacement vectors clearly define a region of deformation, near the middle of the slope (Fig. 7.8b). This was indicated as a movement of the soil mass directing outwards. Then, at $t = 1.25$ s, the soil mass above that point started to move and the first failure observed to the slope model (*initial failure*) at the point indicated by the brown line in Fig. 7.8c. The maximum depth of the failure surface was 15 mm, corresponding to a prototype depth of 0.9 m. The slope crest showed also an increased shear strain even though it did not collapse at this point.

The evolution of deformation prior to the second failure is shown in Figure 7.9. The results from the deformation analysis show a gradual movement of the upper part of the slope in a wedge shape. The soil movement is higher at $t = 22.1$ s. The results indicate that there was

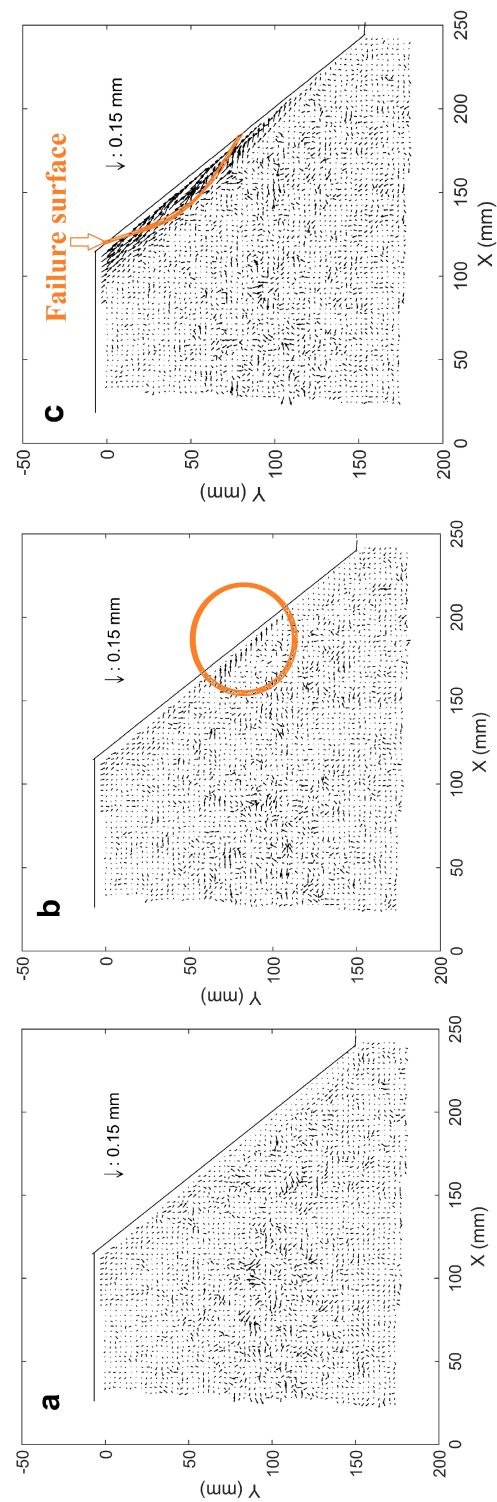


Fig. 7.8 Deformation vectors in the slope of Test CSI1, indicating deformation caused by rainfall. a) At $t = 1.12$ s (67.2 min in prototype) b) At $t = 1.18$ s (70.8 min in prototype) c) At $t = 1.25$ s (75 min in prototype). Failure surface indicated in c) corresponds to the observed *initial failure* (Fig 7.6).

a broader area which was deforming before the instability at the crest. The observed failure surface (*secondary failure*) is indicated by the red line in Figure 7.9f.

The deformation analysis showed that the area around the slope toe was not affected by rainfall. Even though the GWT rose significantly and the pore water pressures reached a value of 25 kPa, there was no deformation. This confirms the statement that the failure occurred due to the saturation of the slope profile and the loss of suction which reduced the shear strength. Furthermore, the slope failed without any significant deformation occurring prior to the instability. This indicates a brittle response to rainfall with a sudden failure. The maximum shear strain and volumetric strain 0.1 s before the occurrence of the first failure are shown in Figures 7.10 and 7.11, respectively.

CSI3

The impact of a rainfall event with lower intensity on a silty sand model was studied in this test. In order to compare the results, the model for this test was prepared in a similar way to the model CSI1. After the compaction of the soil within the centrifuge box, the final achieved dry density was 1.52 Mg/m^3 , thus it had almost the same density with the model in test CSI1. The response from the PPTs was not able to be recorded due to the malfunction of the data acquisition system. The slope was brought to the testing acceleration of 60g in the following sequence: 1g - 5g - 10g - 20g - 30g - 40g - 50g - 60g. The consequences of the gravity increment to the deformation of the slope is shown in Figure 7.12.

The slope settled by a maximum of 1 mm at the top of the soil surface. The same rate of deformation was observed in the slope model CSI1. Deformation vectors near the slope surface were again directed parallel to the slope. This confirms the statement that during gravity increase the behaviour of the slope is governed by the water drainage which is initiated due to the centrifugal spinning.

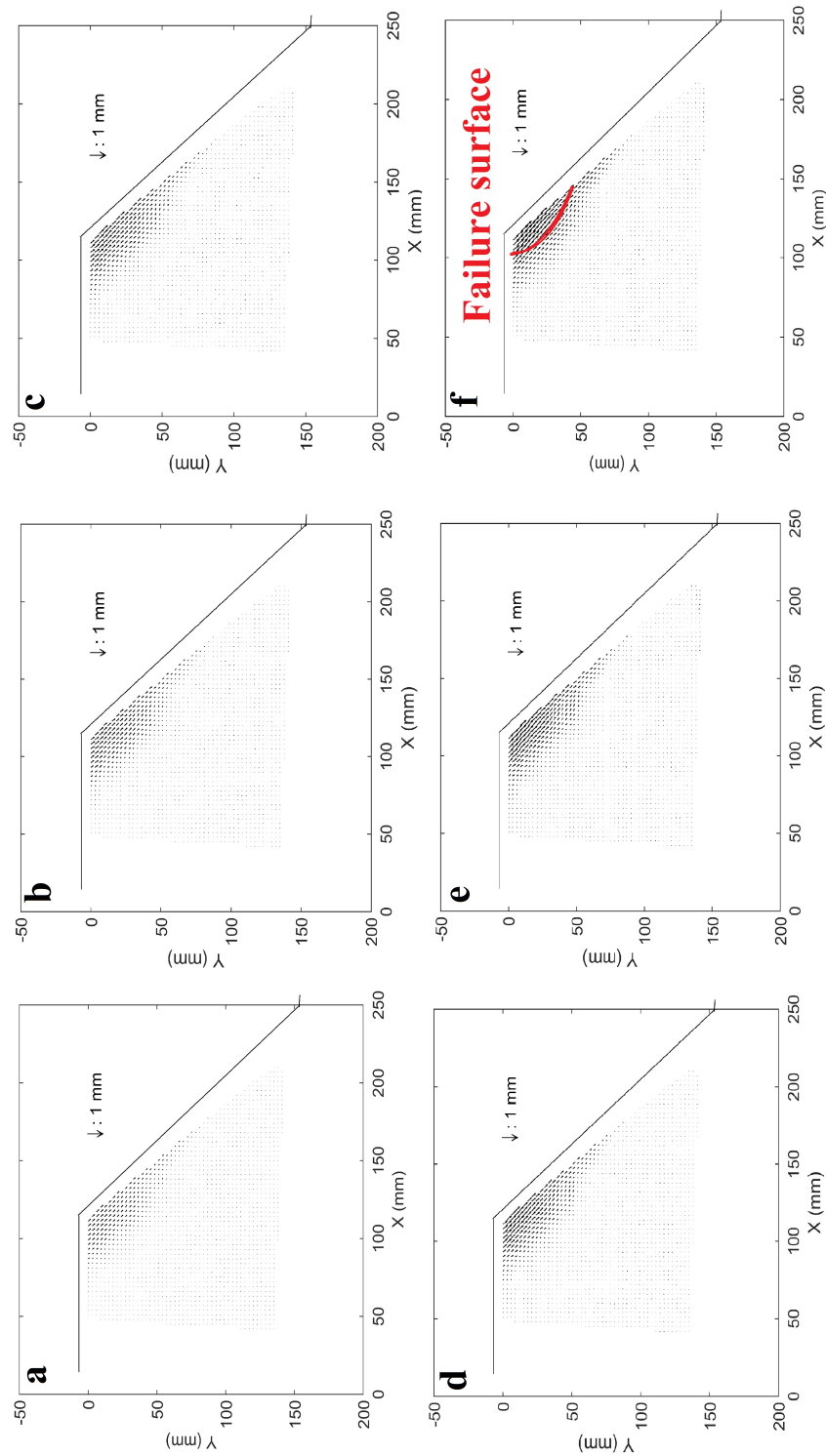


Fig. 7.9 Deformation vectors in the slope of Test CSI1, indicating deformation caused by rainfall after the 1st failure. a) Deformation at $t = 5s$, b) Deformation at $t = 10s$, c) Deformation at $t = 15s$, d) Deformation at $t = 20s$, e) Deformation at $t = 22s$, f) Deformation at $t = 22.1s$ ($t = 0$ when first failure occurred). The red line indicates the *secondary failure* observed in the end of the test (Fig 7.6).

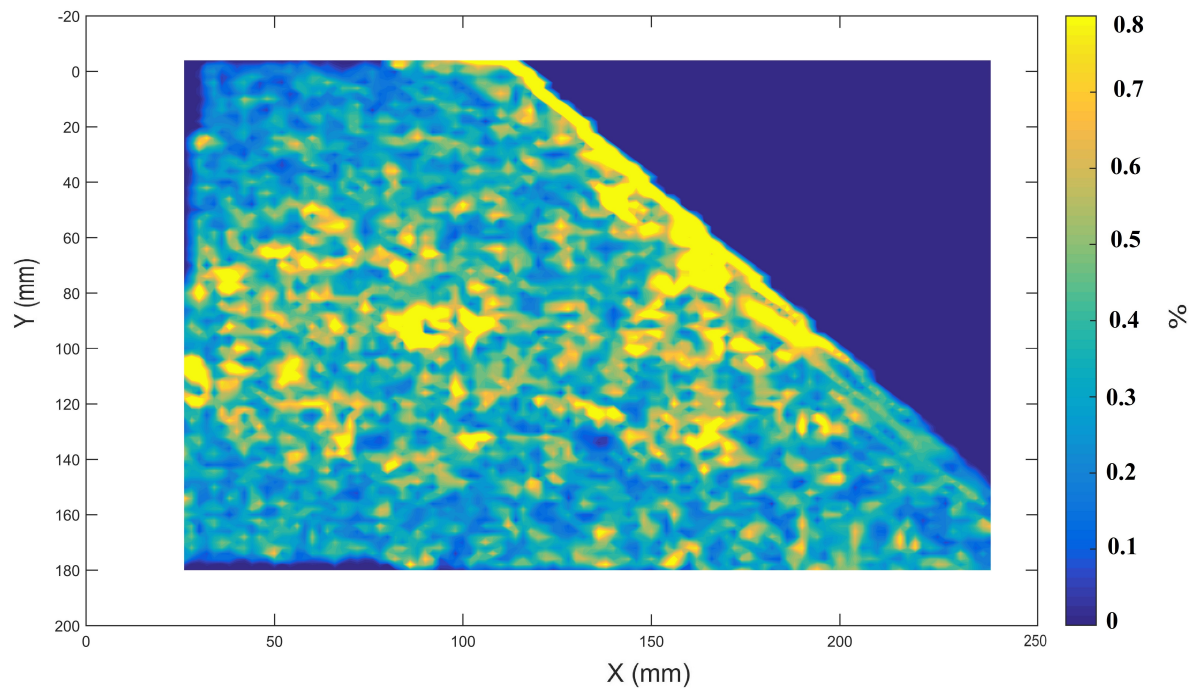


Fig. 7.10 Maximum shear strain of the slope 0.1 s before the initiation of the first failure.

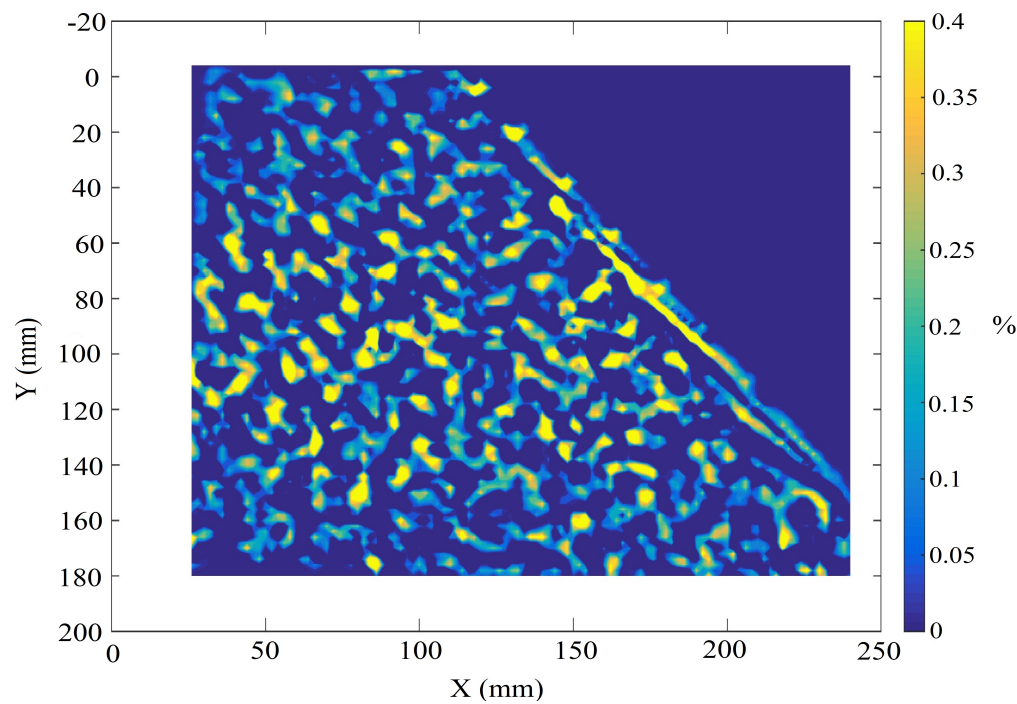


Fig. 7.11 Volumetric strain 0.1 s (model time) before the initiation of the first failure.

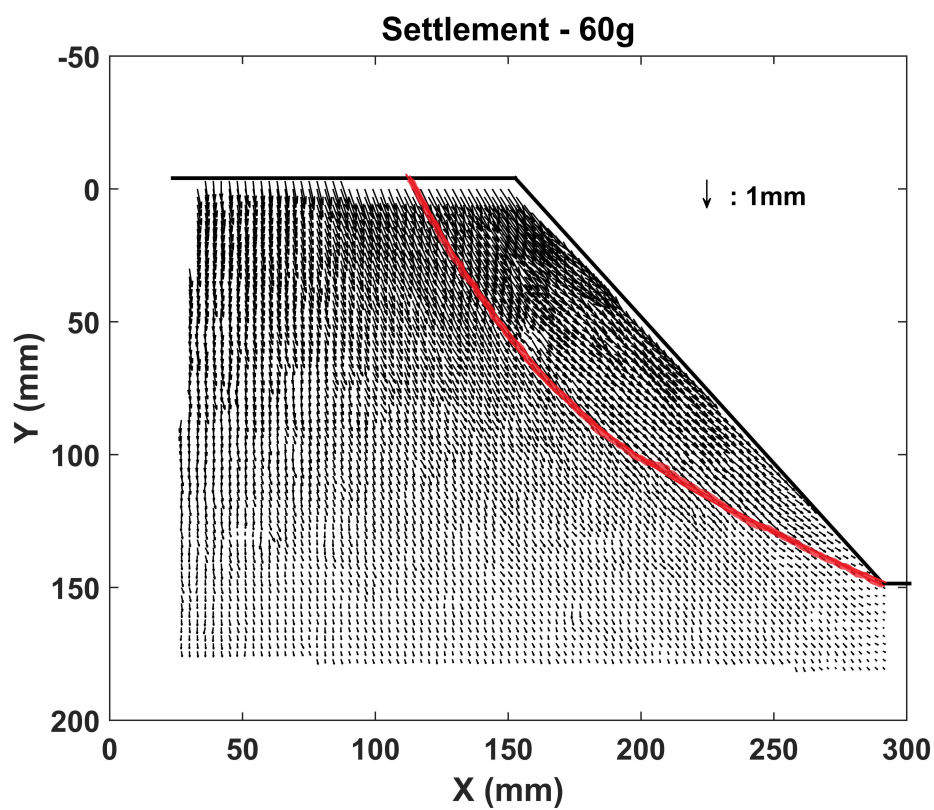


Fig. 7.12 Effect of gravity increment to the slope CSI3 at $t = 0$ (dimensions in model scale). The red line represents the critical slip surface as this was determined by the numerical analysis using Slope/W (section 7.3.3)

The evolution of the displacements in the slope profile is shown in Figure 7.13. 170 s after the initiation of rainfall a localised deformation occurred around the crest delimiting an unstable area. 700 s later, the unstable area expanded at 2/3 of the slope height. Finally, at $t = 1042$ s, the unstable soil mass is clearly delimited. Just 0.06 s later, initiation of the instability occurred. The deformation field is shown in Figure 7.14. The duration of rainfall causing the instability was 1042.06 s which corresponds to a prototype duration of 43.42 days.

The failure surface was circular with the maximum depth being 20 mm (1.2 m in the prototype), therefore it can be characterised as a superficial type of landslide. An image of the failure is shown in Figure 7.15. The GWT remained almost horizontal, with a slight curved shape due to the delayed drainage.

An interesting observation comes from the spatial distribution of the shear strain during rainfall. The maximum shear strain in the slope at $t = 170$ s and just before the initiation of the instability (i.e. $t = 1046$ s) are shown in Figure 7.16. At $t = 170$ s, the maximum shear strain on the shear plane was nearly 5%. At this stage, the shear plane formed up to a depth of 1/3 of the slope height. At time $t = 1049$ s (prior to failure), the shear plane was formed and the failure surface was delimited. Just before the initiation of rainfall, the maximum shear strain was approximately 10%. This behaviour indicates a landslide with prior consecutive smaller failures in contrast to the case of the high rainfall intensity where failure was sudden.

7.3.3 Numerical validation of tests CSI1 and CSI3

The results from the centrifuge tests CSI1 and CSI3 were further evaluated by performing numerical validation. The software Slope/W (Geostudio 2019) was used and limit equilibrium type of analysis was performed. The properties of the M_{90-10} soil, as they have been presented in Chapter 4 were used in the analyses. Slope models were designed in their prototype dimensions with the slope height at 9 m. The inclination of the slope models for the numerical

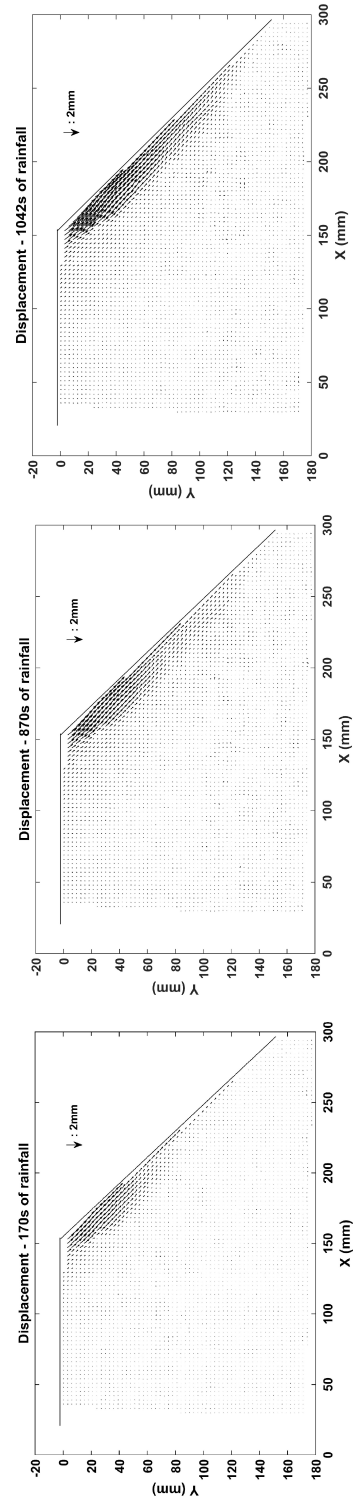


Fig. 7.13 Deformation vectors in the slope of Test CS13, indicating deformation caused by rainfall - $t = 0$ at rainfall initiation (model time).

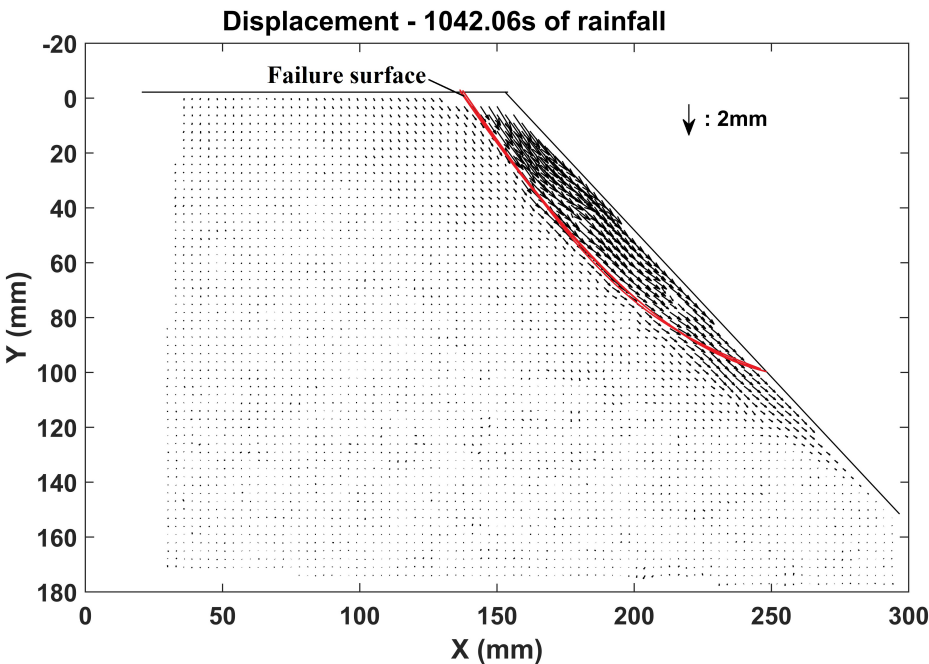


Fig. 7.14 Instability in the slope of Test CSI3 (model time).

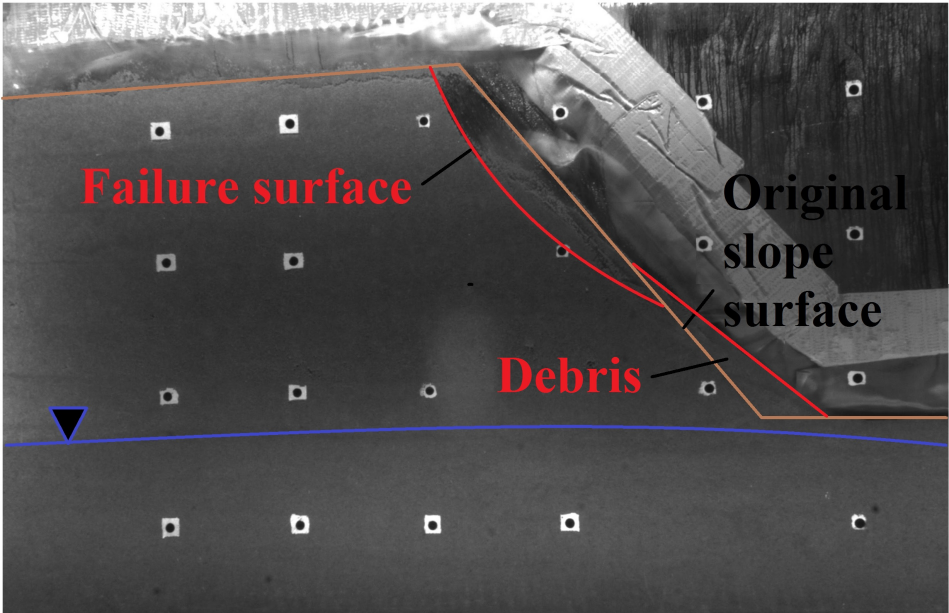


Fig. 7.15 Instability at Test CSI3.

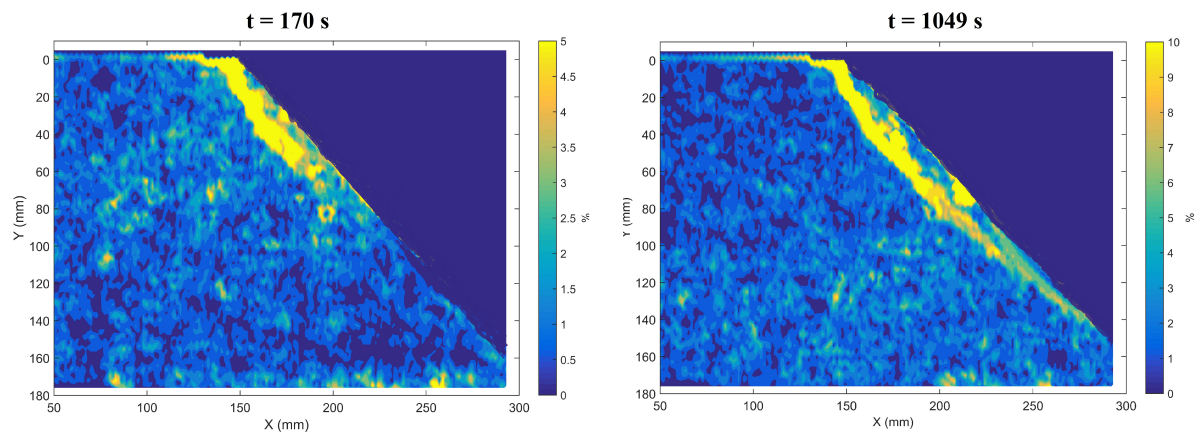


Fig. 7.16 Shear strain distribution in the slope of Test CSI3. Left: at $t = 170$ s Right: at $t = 1049$ s (time in model scale).

Table 7.1 Parameters used in the numerical analysis of tests CSI1 & CSI3.

Test	γ_{sat} (kN/m^3)	γ_{unsat} (kN/m^3)	S_r (-)	e (-)	c' (kPa)	ϕ' ($^\circ$)
CSI1	18.8	15.4	0.185	0.76	7.15	32.5
CSI3	19	15.7	0.185	0.72	7.15	32.5

analysis was 50° . Mohr-Coulomb material model has been used in the analyses, while Equation 7.2 was used to calculate the contribution of suction to the soil's shear strength. Slope stability analysis was performed using the Spencer method of slices. Groundwater conditions were simulated through the use of a phreatic surface lying underneath the slope toe.

To perform the numerical analysis, the following parameters needed to be defined: unit weight, degree of saturation, void ratio and strength parameters (c' and ϕ'). A summary of these parameters for the two tests can be seen in Table 7.1, where γ_{sat} and γ_{unsat} are the unit weight of the M_{90-10} soil in the saturated and unsaturated state, respectively, S_r the degree of saturation (ratio), e the void ratio, c' the cohesion and ϕ' the friction angle in terms of effective stresses.

The unit weight of the soil is calculated as:

$$\gamma = \frac{\rho_s + \gamma_w S_r e}{1 + e} \quad (7.6)$$

where γ_w is the unit weight of water (9.81 kN/m^3) and ρ_s the soil's particle density. In the unsaturated state, the degree of saturation was 0.185 and corresponds to the residual value of S_r for the M_{90-10} soil as this was determined in the column centrifuge tests in Chapter 6.

The effect of suction to the stability of the slope was modelled through the suction contribution to the shear strength. For this purpose, the SWRC of M_{90-10} , determined in Chapter 4, was input into the analysis. The maximum suction above the phreatic surface (groundwater table) was capped at 20 kPa, value corresponding to the residual water content of the soil as this was determined in Chapter 4. Moreover, the unsaturated unit weight was assigned to the soil located within the unsaturated zone, i.e. above the groundwater table.

Static factor of safety

Initially, the safety factor of the slope models prior to rainfall was determined. Since the two slopes were prepared in almost identical initial dry density, the numerical results are expected to be similar. Therefore, the analysis of only one slope model was sufficient to describe the behaviour of both physical models in tests CSI1 & CSI3. The geometry of the model is shown in Figure 7.17. A grid was used to define the location of the centre point for the circular failure surface with the lowest factor of safety. The radius of the potential slip surfaces was delimited by the lines parallel to the slope face.

The stability of the slope models, prior to the application of rainfall, is shown in Figure 7.18. The minimum factor of safety was found to be 1.19 which indicates that the slope was

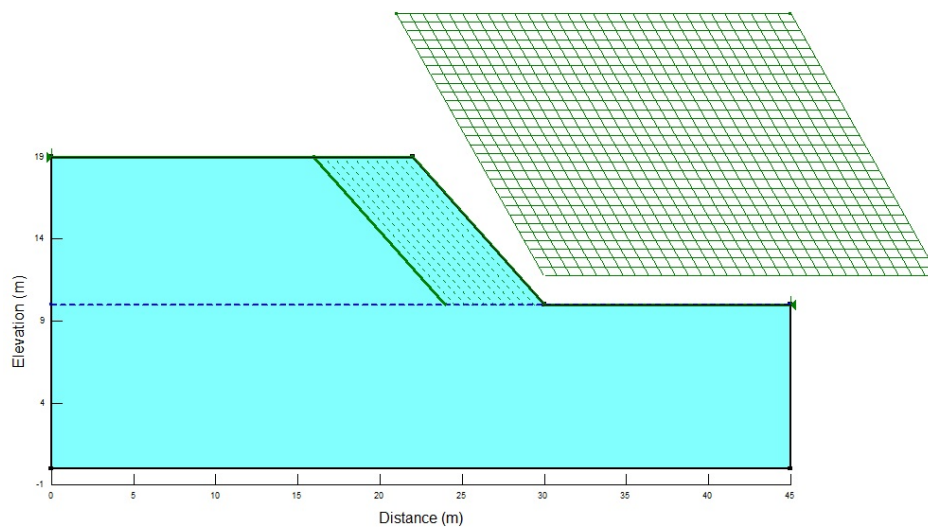


Fig. 7.17 Geometry of the slope model used in the numerical analysis of tests CSI1 & CSI3.

marginally stable without rainfall. The critical failure surface was found to be approximately 3 m deep in the middle of the slope height and passes from the slope toe. The critical slip surface can be compared to the deformation field obtained for the slope at test CSI3, at the moment this was brought to a 60 g acceleration and before the start of rainfall simulation. The critical slip surface is marked with a red colour in Figure 7.12 and the comparison with the deformation field obtained through the geoPIV analysis indicates that gravity increase caused dislocation of a soil block, similar to the predicted one, but about 1.5 times larger.

CSI1

In test CSI1 the slope model experienced instability in two stages, as it was indicated in Figures 7.6, 7.8 and 7.9. The final failure surface determined in the centrifuge test was drawn on the model analysed in Slope/W and its factor of safety was determined. To simulate the test conditions in the most accurate way, the following boundary conditions were applied:

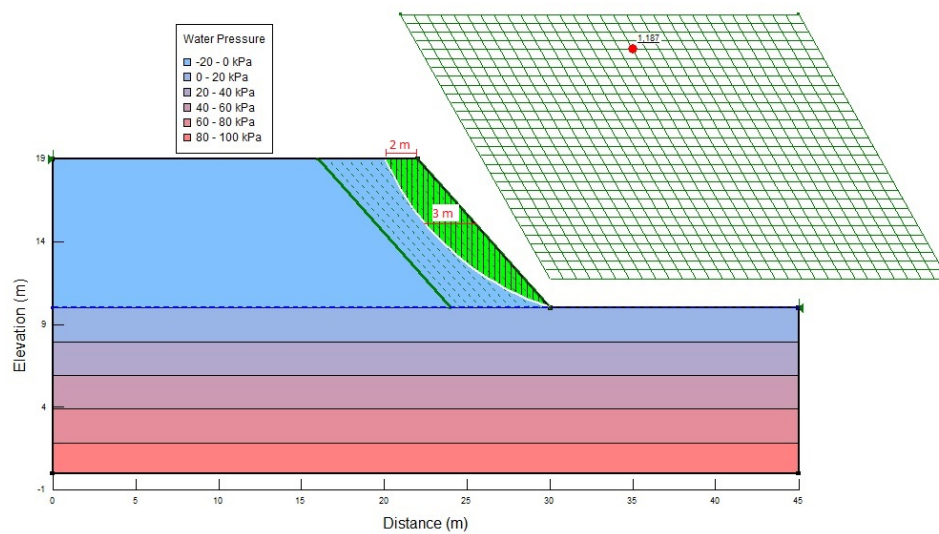


Fig. 7.18 Stability of the slope model prior to the application of rainfall.

- Suction applied to the unsaturated zone above the groundwater table. Phreatic surface is horizontal and passes from the slope toe (Figure 7.19). This condition represents the state of the slope prior to the application of rainfall.
- Suction above the groundwater table was limited to 0 kPa. The phreatic surface was raised by 2 m above the slope toe (Figure 7.20), representing a simplistic groundwater condition of the slope at the moment of failure. In this scenario, rainfall has caused the elimination of suction in the unsaturated zone, above the groundwater table, however no positive pore pressures have been generated. This case corresponds to the impact of a rainfall event, with relative intensity of 1, to the stability of the slope.
- The phreatic surface was raised up to the slope surface, representing the condition of a full saturation of the soil profile (Figure 7.21). In this scenario, relative intensity of the rainfall event is higher than 1 and dictates the saturation of the soil profile. In this sense, positive pore pressures are generated to the previously unsaturated zone causing significant reduction to the shear strength and eventually slope stability.

Before rainfall, the actual slip surface appears to have a significantly high factor of safety (1.42) which indicates stability. Safety factor is enhanced by the fact that suction applies to the zone above the groundwater table, where the slip surface is located. Suction appears to reach nearly 87 kPa at the top of the slope surface which causes a reinforcement of the soil as the shear strength increases.

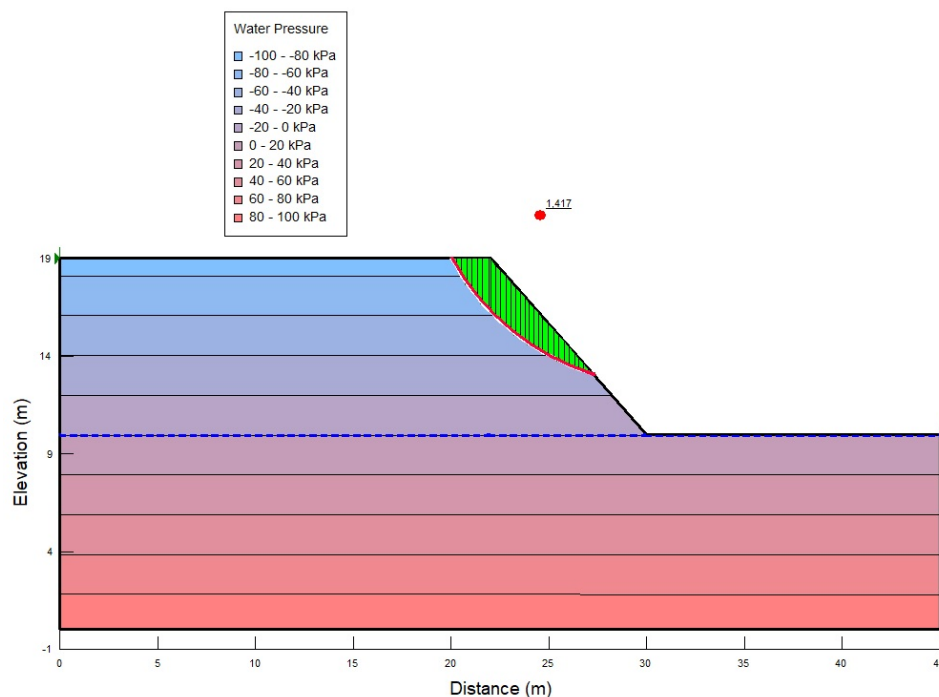


Fig. 7.19 Evaluation of the actual slip surface of the slope in test CSI1 before rainfall.

In the case of rainfall, with the slight elevation of the groundwater table and zero suction condition, the factor of safety for the actual slip surface was determined at 1.31 or 8% less than the factor of safety without rainfall. This reduction in the stability of the slope was caused due to the loss of suction contribution to the shear strength of the soil, as the slip surface is located within the *zero-suction* zone. However, despite the loss of strength, these boundary conditions are not capable of causing the instability of the slope on the particular slip surface.

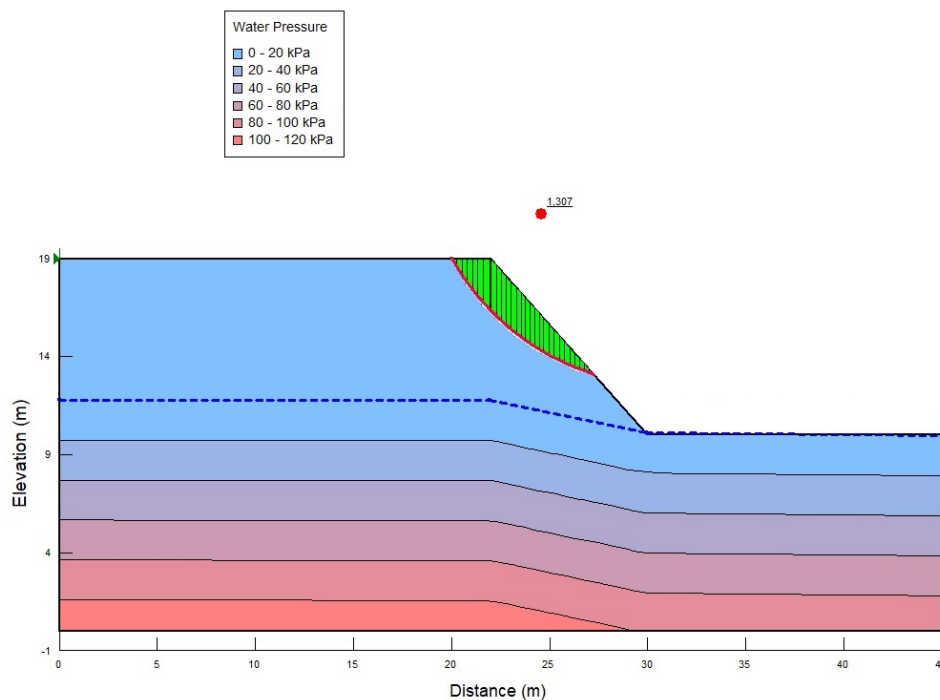


Fig. 7.20 Evaluation of the actual slip surface of the slope in test CSI1 after rainfall. Boundary conditions simulate the impact of a rainfall event with relative intensity of 1.

Finally, in the case of a full saturation of the soil profile, the slope seems unstable as the factor of safety has been determined at 0.761. In this case, as shown from the distribution of the pore water pressures in Figure 7.21, the slip surface is located within the zone where pore water pressures range between 0 and 20 kPa. This change in pore water pressures has caused a drop of the FoS by 46%.

CSI3

The slip surface, appeared in the slope of test CSI3, is shown in Figure 7.14 and is quite similar to the slip surface occurred in test CSI1. Therefore, the numerical analysis results are identical for the two tests and the factor of safety for both models is similar for the boundary conditions described above (Figures , 7.23 and 7.24).

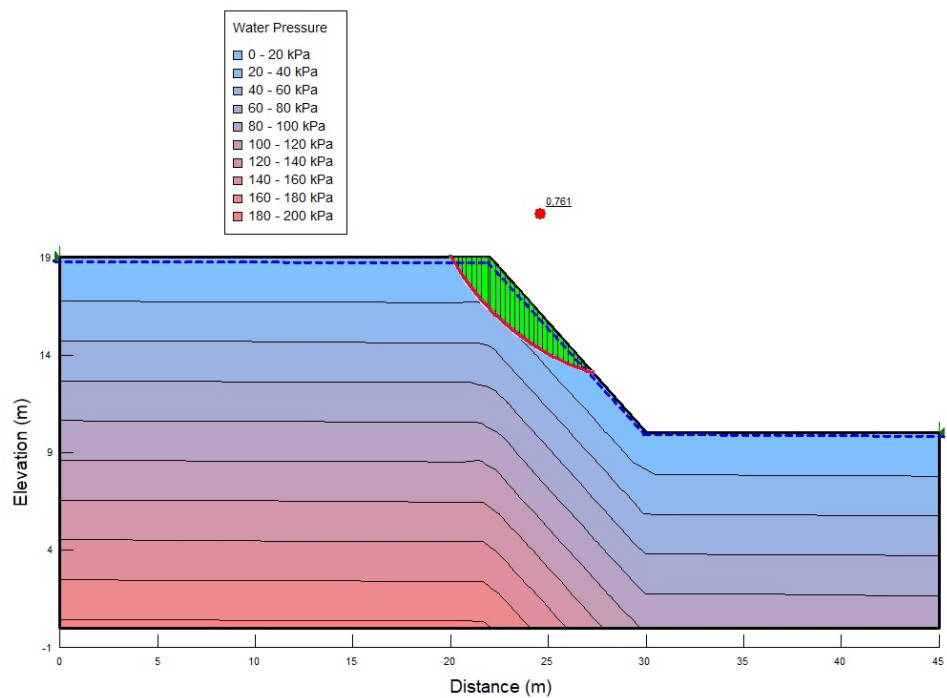


Fig. 7.21 Evaluation of the actual slip surface of the slope in test CSI1 after rainfall. Boundary conditions represent the case of full saturation of the soil profile affected by a rainfall event with a relative intensity greater than 1.

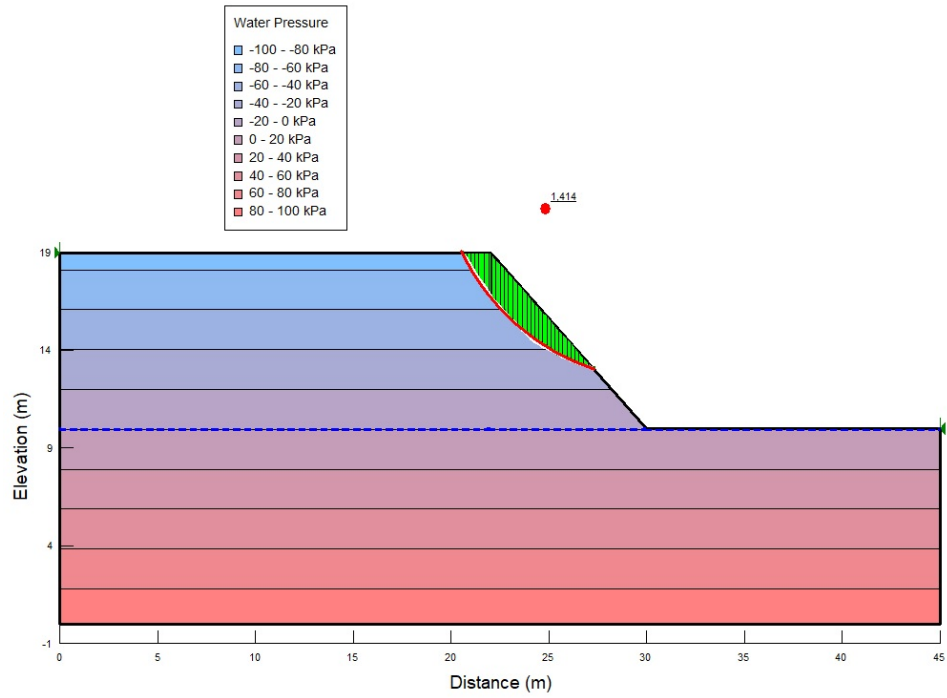


Fig. 7.22 Evaluation of the actual slip surface of the slope in test CSI3 before rainfall.

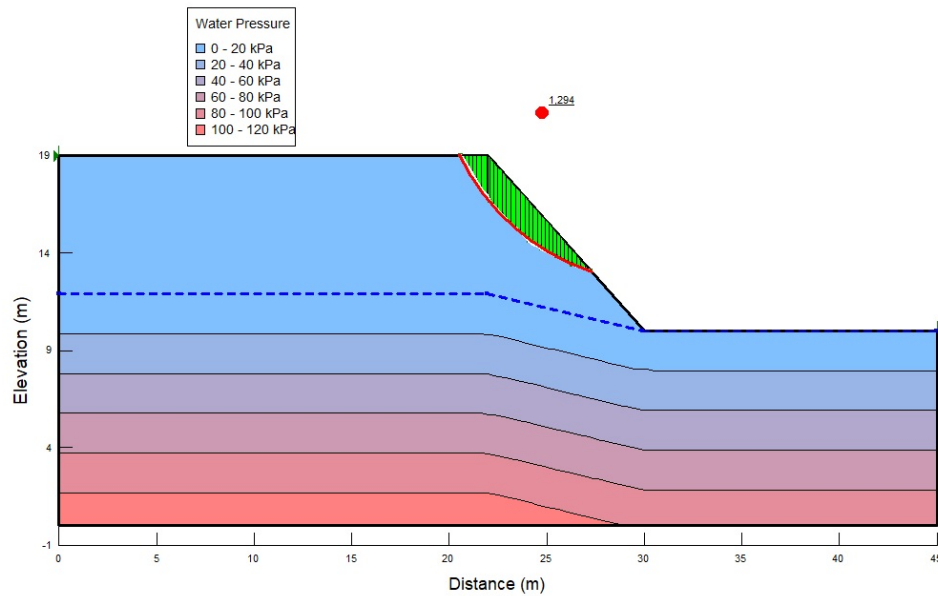


Fig. 7.23 Evaluation of the actual slip surface of the slope in test CS31 after rainfall. Boundary conditions simulate the impact of a rainfall event with relative intensity of 1

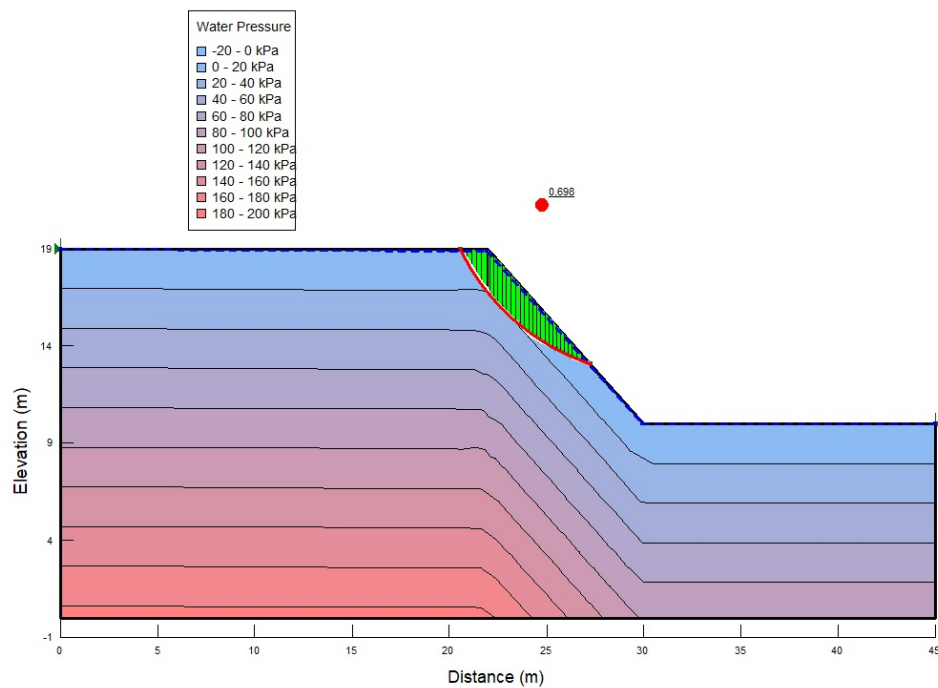


Fig. 7.24 Evaluation of the actual slip surface of the slope in test CSI3 after rainfall. Boundary conditions represent the case of full saturation of the soil profile affected by a rainfall event with a relative intensity greater than 1.

In the scenario without rainfall, the factor of safety was determined at 1.41 (7.22), indicating once more a significant resistance of the slope to failure. In the case of a rainfall with relative intensity of 1, factor of safety was found to be 1.29 and therefore suction elimination did not affect significantly its stability. Finally, in the case of a rainfall with high relative intensity, the slope seems to be prone to failure, once the factor of safety dropped down to 0.7.

7.3.4 Discussion

Introducing rainfall to the two silty sand slope models led to development of displacements and eventually failure, due to the infiltration of water. In the tests performed in fine sand models (Chapter 5) no failure mechanism was observed, whereas silty sand exhibited failures under both low and high intensities of rainfall (CSI1 and CSI3).

In order to explain the differences between the results obtained for sand and silty sand, one may consider the effect of silt content on permeability and Darcy velocity. The conventional scaling law considering the macroscopic response of the soil under increased gravity deduces that the Darcy velocity (v_m) in the N-g model is N times larger than that in the prototype (Taylor, 1995), where the flow path is considered to be N times larger in the prototype. Askarinejad et al. (2015) analysed the grain scale phenomenon of voids collapse and concluded that, if the structure of soil in an N-g model and prototype were the same, the scaling factor of length at the grain scale is equal to one (i.e. $L_p/L_m=1$). Thus, the scaling law for pore pressure dissipation time in the model and the prototype can be written as follows

$$t_{dissipation,m} = L_m/v_m = L_p(1/Nv_m) = t_{dissipation,p}/N \quad (7.7)$$

In addition, [Askarinejad et al. \(2015\)](#) considered the collapse of the voids analogous to the free fall of particles, for which the time scaling law can be written as:

$$t_{\text{impact},m} = \sqrt{2L_m/\alpha_m} = \sqrt{2L_p/N\alpha_p} = \sqrt{1/N} \times t_{\text{impact},p} \quad (7.8)$$

where a represents the acceleration, and L is the falling height. These relationships show that the pore water dissipation is $\sqrt{N}$ times faster than the collapse of the voids, which may also be related to the grain-scale movements required to initiate failure along a slip-plane.

These scaling factors may be used to explain the differences between the two types of soil used in this study. In case of sand, the pore water seepage is faster than the collapse of the voids and hence the time scale between the two coupled processes of dissipation and void collapse are incompatible. Thus, the pore pressure is dissipated before the void collapse occurs. In order to rectify this issue, [Askarinejad et al. \(2015\)](#) increased the viscosity of the pore fluid by a factor of $\sqrt{N}$. In this study, water is used as the pore fluid, but the use of silty sand provides models with a permeability ratio of 8.3 compared to the sandy models, nearly equal to $\sqrt{N}=7.75$. This suggests that adding 10% of silt essentially reduces the permeability of the soil and is equivalent to the case where the fluid viscosity is increased by the same factor. In other words, if the viscosity of the fluid increases by a factor of 8.3 in case of sandy soil tests (Chapter 5), failure is expected, similar to those observed in tests CSI1 and CSI3. This may also explain why the test CSI1 with relative intensity of 1.35 led to slope failure whereas test CSA8.3 with a higher relative intensity (2.44) did not result in a failure. In tests CSA8.3 and CSA8.4, the pore pressure dissipation phenomenon is 7.75 times faster than that of a free falling particle, and hence the pore pressure dissipates before the void collapse can happen. On the other hand, in test CSI1, pore pressure and void collapse have the same time scales which allow the test to properly simulate the prototype conditions. The presented interpretation and discussions, however, need further testing to validate the theoretical fundamentals discussed

above, where pore pressure data are collected from a larger number of tests with enhanced spatial resolution.

Numerical analysis of tests CSI1 & CSI3 indicated that the loss of suction from the soil profile was not enough to trigger instability under rainfall. A small positive pore pressure (up to 20 kPa) was sufficient to cause significant loss of the shear strength and trigger instability. In the case of CSI1, with a higher relative intensity than in test CSI3, instability triggered very fast compared to the slope in CSI3. This can be attributed to the faster formation of the wetting front in test CSI1 which caused faster saturation of the soil profile and eventually a fast instability. On the other hand, in CSI3, the relative intensity was small enough and therefore the saturation of the soil profile slower causing a delayed instability.

7.3.5 Rainfall thresholds

The initiation of rainfall-induced landslides strongly depend on the rainfall characteristics, i.e. rainfall intensity I and rainfall duration D . The combination of I and D provide rainfall thresholds for the initiation of landslides. These thresholds strongly depend on the regional conditions, i.e. landscape, slope geometry, geological formations, groundwater conditions, and climate. Rainfall thresholds provide a good solution when predicting the occurrence of rainfall-induced landslides. Moreover, they can be used in prediction models for the design of real-time warning systems which are installed in areas prone to failures.

Rainfall thresholds for landslide initiation have been proposed in the past on regional (C.Y et al., 2005; Larsen and Simon, 1993; Ling et al., 2009a) or global (Caine, 1980; Guzzetti et al., 2008) scales. The obtained rainfall threshold from the current study has been plotted on $\log(I)$ - $\log(D)$ scales in Figure 7.25. In the same graph, rainfall thresholds from other studies have

Table 7.2 Comparison of the fitting parameters between different rainfall thresholds.

Source	a	b	Type
Current study	95.197	-0.368	Centrifuge modelling of a 9 m slope at an angle of 50°
Chien-Yuan et al. (2005)	115.47	-0.8	Regional (Taiwan)
Larsen and Simon (1993)	91.46	-0.82	Regional (Puerto Rico)
Ling et al. (2009a)	10.85	-0.376	Centrifuge testing of a 9 m slope at an angle of 60°
Caine (1980)	13.58	-0.38	Global
Guzzetti et al. (2008)	2.2	-0.44	Global

also been plotted. These thresholds are presented in the form $I = aD^b$, where a and b are fitting parameters. Therefore, the relationship between the rainfall intensity and rainfall duration for the initiation of a landslide follows a power law. Table 7.2 provides a comparison of several rainfall thresholds obtained from the literature.

Comparing the different approaches, it appears that the obtained threshold for the higher rainfall rate falls within the area of two regional thresholds from Taiwan ([Chien-Yuan et al., 2005](#)) and Puerto Rico ([Larsen and Simon, 1993](#)). On the other hand, low intensity needed longer rainfall events for instability to occur, compared to the values obtained from the literature. In general, the centrifuge tests provided an over-estimated rainfall threshold. This could be attributed to the conditions and the scaling issues which govern the centrifuge modelling technique. Even though centrifuge modelling provides a very useful tool for the study of the slope behaviour under rainfall conditions, it is important to highlight the difficulties and uncertainties which might provide non realistic prototype conditions.

The rainfall thresholds obtained in this study concern a specific soil type, slope geometry, initial and boundary conditions. More work is needed in order to define more accurate

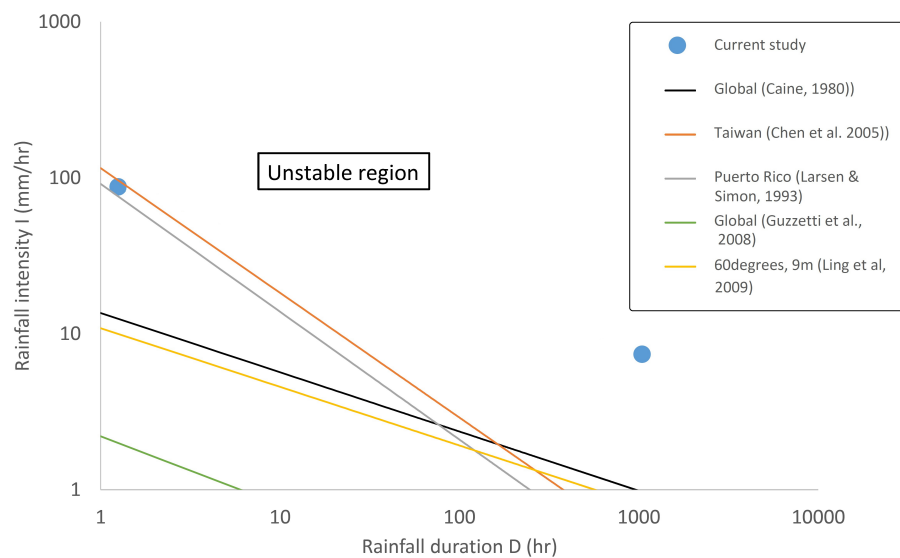


Fig. 7.25 Rainfall threshold obtained from the current study.

thresholds which would take into account different geometries, initial conditions, boundary conditions and soil types.

7.4 Chapter summary

The initiation of failure in soil slopes made of fine sand, tested under rainfall, was not able to be reproduced in the centrifuge. This was attributed to the increased seepage velocity in the centrifuge which provides conditions of fast dissipation of the excess pore water pressures. The technique used in this study to overcome this issue was the scaling of the hydraulic conductivity by $\sqrt{N}$. This was achieved by producing a slightly finer soil (on average) which consisted on a fine sand and silt at proportions 9:1. In this way, pore water pressures were developed in the model while the dissipation process was delayed due to the decreased seepage velocity. This created conditions of failure in the slope model.

The results from two centrifuge tests in the silty sand slopes revealed that model behaviour strongly depends on the rainfall characteristics. Higher rainfall rate initiated an instability in a

shorter period of time compared to the lower rainfall intensity. Moreover, the behaviour of the slope under high rainfall intensity was brittle causing a fast and sudden landslide. On the other hand, low intensity caused an instability with consecutive slides after extended deformation for a long duration. Finally, a rainfall threshold was obtained from the results of the centrifuge tests and compared with thresholds produced from local and regional scales. Comparison showed that centrifuge modelling produced an over-estimated prediction for the initiation of rainfall-induced landslides.

Chapter 8

Conclusions and future work

8.1 Conclusions from the current study

An experimental investigation on the initiation of rainfall-induced landslides in unsaturated soil slopes was performed in this study. Different initial and boundary conditions were applied to the slope models which were tested in the geotechnical centrifuge at 60g. To accomplish the objectives of the study, a centrifuge box was modified in order to provide the necessary rainfall and groundwater conditions.

Initially, the study focused on the behaviour of a fine, uniform sand, Leighton Buzzard fraction E. Classification tests of the soil were performed in the laboratory. Sand slope models were built at a high inclination angle, about 1.5 times higher than the angle of friction, with suction providing the necessary cohesive component for stability at 1g. At 60g, slopes showed an unexpected resistance to instability which was attributed to the soil suction enabled due to drainage. Therefore, the conditions of the soil mass when tested at 60g were studied further.

A series of tests in soil columns were designed and performed in order to define the conditions of the soil mass under increased g-levels. To accomplish the objectives of these tests,

a new centrifuge apparatus was designed. Results indicated that even medium-grained soils retain an amount of water content in their mass, even at 60g. The retained water content at each g-level strongly depends on the grain size distribution of the soil. The development of partially saturated conditions in soils tested in the centrifuge enables suction within the soil mass. The level of suction depends on the SWRC of the soil. Suction contributes to the soil strength increasing the effective stresses and, thus, the shear strength. Therefore, soil slopes retain their stability when gravity increases even at extreme conditions, i.e. at an angle 50% higher than the angle of friction.

For a more comprehensive knowledge of the behaviour of LB fraction E sand, under unsaturated conditions, the Soil Water Retention Curve (SWRC) was determined in the laboratory. The tensiometer method was selected for these tests as a fairly easy method which does not require special equipment, like unsaturated triaxial cell and pressure plates. The results of the tests performed on fraction E sand were reasonable and comparable to other techniques which were used to obtain the SWRC of the same soil in the past. The obtained SWRC was then correlated to the retained water content data obtained in the soil column tests.

The measurement of negative pore water pressures (i.e. suctions) in the soils was performed using Pore Pressure Transducers (PPTs) which were fitted with a porous stone. The reliable measurement of suction can be achieved if the contact between the water inside the porous stone and the pore water is retained. Therefore, it was important to achieve a high level of saturation of the porous stone. An experimental setup was designed and built in order to be used for the adequate saturation of the PPTs. These sensors were also used to obtain the SWRC of the soils using the tensiometer method. However, at increased g-levels, the performance of the PPTs in measuring suction in granular soils was poor, as they cavitated quickly as the g-level increased. Therefore, a new technique was developed and tested in the soil column tests, which involved

the use of a saturated silty paste as a cover for the porous stone which prevented cavitation. The technique allowed the use of the PPTs in the sand models and provided reasonable results.

The failure mechanism that was simulated in this study involved the saturation of the unsaturated soil slope due to water infiltration. By this means, the suction which retains slope stability under 1g should be eliminated and instability should occur. However, the test results indicated that this was not the case even under high relative rainfall intensity. This was attributed to the increased seepage velocity due to the centrifuge spinning which provided conditions of fast pore water pressure dissipation. Therefore, the reduction of the effective stresses and, thus, shear strength was not sufficient to initiate an instability.

In order to overcome the scaling issue of the increased seepage velocity, slope models were built from a slightly finer soil. In this way, the permeability of the soil was scaled down by $\sqrt{N}$ times and provided slower excess pore water pressure dissipation. These soil slopes were tested under similar boundary conditions as the sand slopes. The results indicated that the reduction of the permeability caused instability initiation under different combinations of rainfall intensity and rainfall duration. Slope centrifuge tests were simulated using limit equilibrium analysis in order to validate the experimental results. The analysis revealed the previous statement, that the elimination of suction was not sufficient to reduce the shear strength of the soil to a level that would cause instability in the slopes. Failure was attributed to the small increase of positive pore water pressures which were generated only to the slopes with reduced seepage velocity (i.e. in slopes made of M_{90-10}). Therefore, grain size scaling in soil slope models was found to provide the necessary conditions for the simulation of rainfall-induced landslides in centrifuge tests.

The set of rainfall-induced landslides, simulated at increased gravity conditions, was compared with existing data from the literature regarding rainfall thresholds of landslides. The

comparison showed that the centrifuge tests conducted in this study provided an over-estimated rainfall threshold.

8.2 Future work

8.2.1 Soil column tests

The development of the soil column tests in the centrifuge can be used for further analysis of the retention characteristics of soils. Spinning of a soil under increased g-level initiates soil drainage which can be used for the determination of the SWRC and the unsaturated hydraulic conductivity of the soil. Moreover, the relationship between the grain size and the retained water content at elevated gravity could be investigated further in order to obtain useful correlations.

8.2.2 Slope stability

Centrifuge slope tests presented in this study provided a good experimental basis on the behaviour of soil slopes under elevated gravity. However, the numerical validation of these results could provide verification on the reasons for the mismatch between model and prototype response. In addition, other methods to overcome scaling issues of the seepage velocity could be used, like the scaling of the kinematic viscosity of the liquid used to simulate rainfall.

The investigation of the initiation conditions for the rainfall-induced landslides can be studied further using the geotechnical centrifuge facility. More soil types can be tested under various rainfall conditions in order to obtain rainfall thresholds for landslide initiation. The rainfall thresholds could be related to the soil type, initial or boundary conditions and provide useful correlations for the improvement of early-warning systems for landslide initiation.

REFERENCES

- R. J. Adrian. Particle-imaging techniques for experimental fluid mechanics. *Annual Review of Fluid Mechanics*, 23(1):261–304, 1991. doi: 10.1146/annurev.fl.23.010191.001401.
- M. G. Anderson and S. Howes. Development and application of a combined soil water-slope stability model. *Quarterly Journal of Engineering Geology and Hydrogeology*, 18(3): 225–236, 1985.
- S.A. Anderson and N. Sitar. Analysis of Rainfall-Induced Debris Flows. *J. Geotech. Eng.*, 121 (7):544–552, jul 1995.
- A. Askarinejad. *Failure mechanisms in unsaturated silty sand slopes triggered by rainfall*. PhD thesis, ETH, 2013.
- A Askarinejad, J Laue, A Zweidler, M Iten, E Bleiker, H Buschor, and S M Springman. Physical modelling of rainfall induced landslides under controlled climatic conditions. In *Eurofuge 2012*, volume -, Delft, The Netherlands, 2012.
- A Askarinejad, A Beck, and S M Springman. Scaling law of static liquefaction mechanism in geo-centrifuge and corresponding hydro-mechanical characterisation of an unsaturated silty sand having a viscous pore fluid. *Can. Geotech. J.*, 2014. ISSN 0008-3674. doi: 10.1139/cgj-2014-0237.
- A. Askarinejad, A. Beck, and S.M. Springman. Scaling law of static liquefaction mechanism in geocentrifuge and corresponding hydromechanical characterization of an unsaturated silty sand having a viscous pore fluid. *Canadian Geotechnical Journal*, 52(6):708–720, 2015.
- A. W. Bishop and G. Eldin. Undrained triaxial tests on saturated sands and their significance in the general theory of shear strength. *Géotechnique*, 2(1):13–32, 1950. doi: 10.1680/geot.1950.2.1.13.
- A.W. Bishop. The principle of effective stress. *Teknisk Ukeblad*, 106(39):859–863, 1959.
- M. Boso, E. Romero, and A. Tarantino. The use of different suction measurement techniques to determine water retention curves. pages 169–181, Berlin, Heidelberg, 2005. Springer.
- E Bowman and K Soga. Creep, ageing and microstructural change in dense granular materials. *Soils and Foundations*, 43(4):107–117, 2003.
- E T Bowman and K Soga. Mechanisms of setup of displacement piles in sand: laboratory creep tests. *Can. Geotech. J.*, 42(5):1391–1407, 2005.

- E.W. Brand. Analysis and design in residual soils. In *ASCE Geotechnical Engineering Division Specialty Conference - Engineering and Construction in Tropical and Residual Soils*, pages 89–141, 1982.
- R H Brooks and A T Corey. Hydraulic properties of porous media. *Hydrol Pap Fort Collins CO Colo State Univ*, 3(3):27, 1964. ISSN 0309-1708. doi: citeulike-article-id:711012.
- B Caicedo and L Thorel. Centrifuge modelling of unsaturated soils. *J. Geo-Engineering Sci.*, 2:83–103, 2014. doi: 10.3233/JGS-130013.
- B Caicedo, J Tristanchó, and L Thorel. Centrifuge modeling of soil atmosphere interaction using a climatic chamber. In *Phys. Model. Geotech. - Springman, Laue Seward*, number Figure 1, pages 299–305, 2010. ISBN 9780415592888.
- B. Caicedo, Tristanchó J., and L. Thorel. Mathematical and physical modelling of rainfall in centrifuge. *Int. J. Phys. Model. Geotech.*, 15(3):150–164, 2015. ISSN 1346-213X. doi: 10.1680/ijpmg.14.00023.
- N Caine. The rainfall intensity: Duration control of shallow landslides and debris flows. *Geografiska Annaler. Series A, Physical Geography*, 62(1/2):23–27, 1980.
- L Cascini, S Cuomo, M Pastor, and G Sorbino. Modeling of Rainfall-Induced Shallow Landslides of the Flow-Type. *J. Geotech. Geoenvironmental Eng.*, 136(1):85–98, 2010.
- C. Chien-Yuan, C. Tien-Chien, Y. Fan-Chieh, Y. Wen-Hui, and T. Chun-Chieh. Rainfall duration and debris-flow initiated studies for real-time monitoring. *Environ. Geol.*, 47(5): 715–724, 2005.
- Brian D Collins and Dobrosław Znidarcic. Stability Analyses of Rainfall Induced Landslides. *J. Geotech. Geoenvironmental Eng.*, 130(4):362–372, 2004.
- W H Craig, B K H Bujang, and C M Merrifield. Simulation of Climatic Conditions in Centrifuge Model Tests. *Geotech. Test. J.*, 14(4):406–412, 1991.
- M.R. Cunningham. *The mechanical behaviour of a reconstituted unsaturated soil*. PhD thesis, Imperial College of Science, Technology and Medicine, London, 2000.
- Chen C.Y, Chen T.C., Yu F.-C., Yu W.-H., and Tseng C.-C. Rainfall duration and debris-flow initiated studies for real-time monitoring. *Environmental Geology*, 47(5):715–724, 2005.
- L. Di Horu, C. A. Taylor, S. Bhattacharya, D. M. Wood, F. Moccia, A.L. Simonelli, and G. Mylonakis. *Stiffness design for granular materials: a theoretical and experimental approach*. 2010.
- EA Ellis, C Cox, HS Yu, A Ainsworth, and N Baker. A new geotechnical centrifuge at the University of Nottingham, UK. In *Phys. Model. Geotech. - 6th ICPMG - Ng, Zhang Wang*, pages 129–133, 2006.
- a B Fourie. Predicting Rainfall-Induced Slope Instability. *Proc. ICE - Geotech. Eng.*, 119(4): 211–218, jan 1996. ISSN 1353-2618. doi: 10.1680/igeng.1996.28757.

- D G Fredlund. The 1999 r.m. hardy lecture: The implementation of unsaturated soil mechanics into geotechnical engineering. *Canadian Geotechnical Journal*, 37(5):963–986, 2000. doi: 10.1139/t00-026. URL <http://www.nrcresearchpress.com/doi/abs/10.1139/t00-026>.
- D. G. Fredlund, N. R. Morgenstern, and R. A. Widger. The shear strength of unsaturated soils. *Canadian Geotechnical Journal*, 15(3):313–321, 1978. doi: 10.1139/t78-029.
- D. G. Fredlund, H. Rahardjo, and M. D. Fredlund. *Soil-Water Characteristic Curves for Unsaturated Soils*, pages 184–272. John Wiley & Sons, Inc., 2012. ISBN 9781118280492. doi: 10.1002/9781118280492.ch5. URL <http://dx.doi.org/10.1002/9781118280492.ch5>.
- D.G. Fredlund and H. Rahardjo. *Soil Mechanics for Unsaturated Soils*. John Wiley & Sons Inc. New York, 1993.
- D.G. Fredlund and Anqing Xing. Equations for the soil-water characteristic curve. *Canadian Geotechnical Journal*, 31(4):521–532, 1994a. doi: 10.1139/t94-061.
- D.G. Fredlund and Anqing Xing. Equations for the soil-water characteristic curve. *Canadian Geotechnical Journal*, 31(4):521–532, 1994b. doi: 10.1139/t94-061.
- W. R. Gardner. Some Steady-State Solutions of the Unsaturated Moisture Flow Equation With Application To Evaporation From a Water Table. 85(4):228–232, 1958. ISSN 0038-075X. doi: 10.1097/00010694-195804000-00006.
- J. Garnier, C. Gaudin, S.M. Springman, P.J. Culligan, D. Goodings, D. Konig, B. Kutter, R. Phillips, M.F. Randolph, and L. Thorel. Catalog of scaling laws and similitude questions in geotechnical centrifuge modeling. *Int. J. Phys. Model. Geotech.*, 7(3):01–23, 2007.
- Y Guan and D G Fredlund. Use of the tensile strength of water for the direct measurement of high soil suction. *Canadian Geotechnical Journal*, 34(4):604–614, 1997. doi: 10.1139/t97-014.
- F Guzzetti, S Peruccacci, M Rossi, and C P Stark. The rainfall intensity–duration control of shallow landslides and debris flows: an update. *Landslides*, 5(1):3–17, 2008.
- K.H. Head. *Manual of Soil Laboratory Testing*, volume 1. Whittles Publishing, 3rd edition, 2006.
- K.H. Head and R.J. Epps. *Manual of Soil Laboratory Testing*, volume 2. Whittles Publishing, 3rd edition, 2011.
- K.H. Head and R.J. Epps. *Manual of Soil Laboratory Testing*, volume 3. Whittles Publishing, 3rd edition, 2014.
- Shangyan Huang, S L Barbour, and D G Fredlund. Development and verification of a coefficient of permeability function for a deformable unsaturated soil. *Canadian Geotechnical Journal*, 35(3):411–425, 1998. doi: 10.1139/t98-010.
- P Hudacsek, M F Bransby, P D Hallett, and A G Bengough. Centrifuge modelling of climatic effects on clay embankments. *Eng. Sustain.*, 162(2):91–100, 2009. doi: 10.1680/ensu.2009.162.

- R.M. Iverson. The debris-flow rheology myth. In *Proceedings of the Int. Conf. on Debris-Flow Hazards Mitigation: Mechanics, Prediction, and Assessment*, volume 1, pages 303–314, 2003.
- K. A. Johnson and N. Sitar. Hydrologic conditions leading to debris-flow initiation. *Canadian Geotechnical Journal*, 27(6):789–801, 1990. doi: 10.1139/t90-092.
- Douglas M. A. Jones and Arthur L. Sims. Climatology of instantaneous rainfall rates. *Journal of Applied Meteorology*, 17(8):1135–1140, 1978.
- G J Kasangaki. *Experimental Study of Hydro-Mechanical Behaviour of Granular Materials*. PhD thesis, Heriot-Watt University, 2012.
- D. König, H.L. Jessberger, M.D. Bolton, R. Phillips, G. Bagge, R. Renzi, and J. Garnier. Pore pressure measurement during centrifuge model tests: Experience of five laboratories. In *Leung, C.F., Lee, F.H., Tan, T.S. (eds.) Eurofuge '94, Balkema, Rotterdam*, pages 101–108, 1994.
- Matthew R. Kuhn and James K. Mitchell. New perspectives on soil creep. *Journal of Geotechnical Engineering*, 119(3):507–524, 1993.
- William T Lambe and Robert V Whitman. *Soil Mechanics*. John Wiley & sons, Massachusetts Institute of Technology, 1969.
- M. C. Larsen and A. Simon. A rainfall intensity-duration threshold for landslides in a humid-tropical environment, puerto rico. *Geografiska Annaler: Series A, Physical Geography*, 75 (1-2):13–23, 1993. doi: 10.1080/04353676.1993.11880379.
- E.C. Leong and H. Rahardjo. Review of soil water characteristic curve equations. *J. Geotech. Geoenvironmental Eng.*, 123(12):1106–1117, 1997.
- S. Leroueil. Natural slopes and cuts: movement and failure mechanisms. *Géotechnique*, 51(3): 197–243, 2001. doi: 10.1680/geot.2001.51.3.197.
- C F Leung, F H Lee, and N S Yet. The role of particle breakage in pile creep in sand. *Can. Geotech. J.*, 33(6):888–898, 1996.
- H Ling and HI Ling. Centrifuge Model Simulations of Rainfall-Induced Slope Instability. *J. Geotech. Geoenvironmental Engineering*, (138(9)):1151–1157, 2012. doi: 10.1061/(ASCE)GT.1943-5606.0000679.
- Hoe I. Ling, Min-Hao Wu, Dov Leshchinsky, and Ben Leshchinsky. Centrifuge modeling of slope instability. *Journal of Geotechnical and Geoenvironmental Engineering*, 135(6): 758–767, 2009a.
- Hoe I. Ling, Min-Hao Wu, Dov Leshchinsky, and Ben Leshchinsky. Centrifuge Modeling of Slope Instability. *Journal of Geotechnical and Geoenvironmental Engineering*, 135(6): 758–767, jun 2009b. doi: 10.1061/(ASCE)GT.1943-5606.0000024.
- S. Lourenço, D. Gallipoli, D. Toll, F. Evans, and G. Medero. *Experimental Unsaturated Soil Mechanics*, chapter Determination of the Soil Water Retention Curve with Tensiometers, pages 95–102. Springer Berlin Heidelberg, Berlin, Heidelberg, 2007. ISBN 978-3-540-69873-9. doi: 10.1007/3-540-69873-6_9.

- S.D.N. Lourenço, D. Gallipoli, D.G. Toll, C.E. Augard, and F.D. Evans. A new procedure for the determination of soil-water retention curves by continuous drying using high-suction tensiometers. *Canadian Geotechnical Journal*, 48(2):327–335, 2011. doi: 10.1139/T10-062.
- S.D.N. Lourenço, N. Jones, C. Morley, S.H. Doerr, and R. Bryant. Hysteresis in the soil water retention of a sand–clay mixture with contact angles lower than ninety degrees. *Vadose Zone Journal*, 14(7), 2015. doi: 10.2136/vzj2014.07.0088.
- Sergio D N Lourenco. *Suction Measurements and Water Retention in Unsaturated Soils*. Phd thesis, University of Durham, 2008.
- J. Lowe and T.C. Johnson. User of back-pressure to increase degree of saturation of triaxial test specimens. In *ASCE research conference on shear strength of cohesive soils*, pages 819–836, Bolder, Colorado, 1960.
- Ning Lu and William J Likos. *Unsaturated soil mechanics*. Wiley, 2004.
- R.J. Mair. *Centrifugal modelling of tunnel construction in soft clay*. PhD thesis, University of Cambridge, 1979.
- F. a. M. Marinho, W. a. Take, and A. Tarantino. Measurement of Matric Suction Using Tensiometric and Axis Translation Techniques. *Geotech. Geol. Eng.*, 26(6):615–631, may 2008. ISSN 0960-3182. doi: 10.1007/s10706-008-9201-8.
- V. Matziaris, M.D. Ferentinou, and M.G. Sakellariou. Slope stability assessment in unsaturated soils under rainfall conditions. In *International Workshop in Geoenvironment and Geotechnics*, pages 91–97, Milos, Greece, 2005. Helliotopos Conferences.
- V. Matziaris, A. M. Marshall, and H.S. Yu. Centrifuge model tests of rainfall-induced landslides. In Wei Wu, editor, *Recent Advances in Modeling Landslides and Debris Flows*, Springer Series in Geomechanics and Geoengineering, pages 73–83. Springer International Publishing, 2015. ISBN 978-3-319-11052-3. doi: 10.1007/978-3-319-11053-0_7.
- V. Matziaris, M. S. Masoudian, A. M. Marshall, and C.M. Heron. Centrifuge modelling of rainfall-induced slope instability in sand and silty sand. In *Proceedings of the 19 Int. Conf. on Soil Mechanics and Geotechnical Engineering*, pages 2179–2182, 2017.
- G. R. McDowell. Micromechanics of creep of granular materials. *Géotechnique*, 53(10): 915–916, 2003. doi: 10.1680/geot.2003.53.10.915.
- G. R. McDowell, O. Falagush, and H.-S. Yu. A particle refinement method for simulating dem of cone penetration testing in granular materials. *Géotechnique Letters*, 2(3):141–147, 2012. doi: 10.1680/geolett.12.00036.
- J Mendes. Assessment of the impact of climate change on an instrumented embankment: an unsaturated soil mechanics approach. 2011.
- Zhang Min. *Centrifuge modelling of potentially liquefiable loose fill slopes with and without soil nails*. Phd thesis, Hong Kong University of Science and Technology, 2006.
- James K Mitchell and Kenichi Soga. *Fundamentals of soil behaviour*. John Wiley & Sons, New Jersey, 2005.

- Seiichi Miura and Shosuke Toki. A sample preparation method and its effect on static and cyclic deformation - strength properties of sand. *Soils and Foundations*, 22(1):61–77, mar 1982. ISSN 03851621.
- Pin-Qiang Mo. *Centrifuge modelling and analytical solutions for the cone penetration test in layered soils*. PhD thesis, University of Nottingham, 2014.
- H. Moriwaki, T. Inokuchi, T. Hattanji, K. Sassa, H. Ochiai, and G. Wang. Failure processes in a full-scale landslide experiment using a rainfall simulator. *Landslides*, 1(4):277–288, 2004.
- E.J. Murray and V. Sivakumar. *Unsaturated soils: A fundamental interpretation of soil behaviour*. John Wiley & Sons Inc. New York, 2010.
- J. R. Nimmo, J. Rubin, and D. P. Hammermeister. Unsaturated flow in a centrifugal field: Measurement of hydraulic conductivity and testing of Darcy’s Law. *Water Resour. Res.*, 23(1):124–134, 1987. ISSN 19447973. doi: 10.1029/WR023i001p00124.
- Hiroataka O, Yasuhiko O, Gen F, Yoichi O, Takuro M, Toshiaki S, Tomomi T, and Kyoji S. A fluidized landslide on a natural slope by artificial rainfall. *Landslides*, 1(3):211–219, 2004. doi: 10.1007/s10346-004-0030-4.
- A. Öberg and G. Sällfors. Determination of shear strength parameters of unsaturated silts and sands based on the water retention curve. *Geotechnical Testing Journal*, 20(1):40–48, 1997. doi: <https://doi.org/10.1520/GTJ11419J>.
- R.B. Peck, W.E. Hansen, and T.H. Thornburn. *Foundation Engineering*. Wiley, New York, 2nd edition, 1974.
- H. Rahardjo, A. Satyanaga N., E. C. Leong, and N Y Song. Effects of groundwater table position and soil properties on stability of slope during rainfall. *Journal of Geotechnical and Geoenvironmental Engineering*, 136(11):1555–1564, 2010. doi: 10.1061/(ASCE)GT.1943-5606.0000385.
- A M Ridley and J B Burland. A new instrument for the measurement of soil moisture suction. *Géotechnique*, 43(2):321–324, 1993.
- A. M Ridley, K. Dineen, J. B. Burland, and P. R. Vaughan. Soil matrix suction: some examples of its measurement and application in geotechnical engineering. *Géotechnique*, 53(2): 241–253, jan 2003. ISSN 0016-8505. doi: 10.1680/geot.2003.53.2.241.
- Caroline A Schneider, Wayne S Rasband, and Kevin W Eliceiri. NIH Image to ImageJ: 25 years of image analysis. *Nat Meth*, 9(7):671–675, 2012.
- A. N. Schofield. Cambridge geotechnical centrifuge operations. *Géotechnique*, 30(3):227–268, 1980. doi: 10.1680/geot.1980.30.3.227.
- U. Shavit, R. J. Lowe, and J V. Steinbuck. Intensity capping: a simple method to improve cross-correlation piv results. *Experiments in Fluids*, 42(2):225–240, 2007. doi: 10.1007/s00348-006-0233-7.
- D N Singh and S J Kuriyan. Estimation of hydraulic conductivity of unsaturated soils using a geotechnical centrifuge. *Can. Geotech. J.*, 39:684–694, 2002.

- Devendra N Singh and Ashok K Gupta. Modelling hydraulic conductivity in a small centrifuge. *Canadian Geotechnical Journal*, 37(5):1150–1155, 2000.
- S. M. Springman, A. Askarinejad, F. Casini, S. Friedel, P. Kienzler, P. Teyssere, and A. Thielen. Lessons learnt from field tests in some potentially unstable slopes in Switzerland. *Acta Geotech. Slov.*, 9(1):5–29, 2012. ISSN 18540171.
- S A Stanier, J Blaber, W A Take, and D J White. Improved image-based deformation measurement for geotechnical applications. *Can. Geotech. J.*, 13(October 2015):1–35, 2015.
- D.J. Sweeney. Some in situ soil suction measurements in hong kong’s residual soil slopes. In *7th Southeast Asian Geotech. Conf.*, volume 1, pages 91–106, Hong Kong, 1982. Southeast Asian Geotechnical Society and Hong Kong Institution of Engineers.
- Masataka T., Osamu K., and Taketo H. Time-dependent behavior of crushable materials in one-dimensional compression tests. *Soils and Foundations*, 41(1):97–121, 2001. doi: 10.3208/sandf.41.97.
- W A Take and M D Bolton. An atmospheric chamber for the investigation of the effect of seasonal moisture changes on clay slopes. In *Int. Conf. Phys. Model. Geotech.*, pages 765–770, St. John’s, Newfoundland, Balkema, Rotterdam, 2002.
- W. A. Take and M. D. Bolton. Tensiometer saturation and the reliable measurement of soil suction. *Géotechnique*, 53:159–172(13), 2003a.
- W A Take and M D Bolton. Tensiometer saturation and the reliable measurement of soil suction. *Géotechnique*, 53(2):159–172, jan 2003b. ISSN 0016-8505. doi: 10.1680/geot.2003.53.2.159.
- W A Take and M D Bolton. Seasonal ratcheting and softening in clay slopes, leading to first-time failure. *Géotechnique*, 61(9):757–769, sep 2011. ISSN 0016-8505. doi: 10.1680/geot.9.P.125.
- W. a. Take, M. D. Bolton, P. C. P. Wong, and F. J. Yeung. Evaluation of landslide triggering mechanisms in model fill slopes. *Landslides*, 1(3):173–184, sep 2004. ISSN 1612-510X. doi: 10.1007/s10346-004-0025-1.
- William Andrew Take. *The influence of seasonal moisture cycles on clay slopes*. Phd thesis, University of Cambridge, 2003.
- S. Tamate, N. Suemasa, and T. Katada. Simulation of precipitation on centrifuge models of slopes. *Int. J. Phys. Model. Geotech.*, 12(3):89–101, sep 2012. ISSN 1346-213X. doi: 10.1680/ijpmg.11.00009.
- F.S.C. Tan. *Centrifuge and theoretical modelling of conical footing on sand*. PhD thesis, Cambridge University, 1990.
- R. N. Taylor. *Geotechnical Centrifuge Technology*. Chapman & Hall, London, 1995.
- W Thielicke and E.J. Stamhuis. Pivlab-towards user-friendly, affordable and accurate digital particle image velocimetry in matlab. *Journal of Open Research Software*, 2(1):e30, 2014. doi: <http://doi.org/10.5334/jors.bl>.

- William Thielicke. *The flapping flight of birds: Analysis and application*. PhD thesis, 2014.
- A. Tokay and D. A. Short. Evidence from tropical raindrop spectra of the origin of rain from stratiform versus convective clouds. *Journal of Applied Meteorology*, 35(3):355–371, 1996.
- N. K. Toker, J. T. Germaine, K. J. Sjoblom, and P. J. Culligan. A new technique for rapid measurement of continuous soil moisture characteristic curves. *Géotechnique*, 54(3):179–186, 2004. doi: 10.1680/geot.2004.54.3.179.
- D. G. Toll. Briefing: Rainfall-induced landslides in Singapore. *Proc. ICE - Geotech. Eng.*, 149(4):211–216, jan 2001. ISSN 1353-2618. doi: 10.1680/geng.2001.149.4.211.
- D. G. Toll, S.D.N. Lourenço, and J. Mendes. Advances in suction measurements using high suction tensiometers. *Engineering Geology*, 165:29–37, oct 2013. doi: 10.1016/j.enggeo.2012.04.013.
- D G Toll, J D Asquith, A Fraser, A A Hassan, G Liu, S D N Lourenço, J Mendes, T Noguchi, P Osinski, and R Stirling. Tensiometer techniques for determining soil water retention curves. In *Unsaturated Soil Mechanics - from Theory to Practice*, pages 15–22. CRC Press, oct 2015. ISBN 978-1-138-02921-7. doi: doi:10.1201/b19248-4.
- D.G. Toll. The behaviour of unsaturated soil. In *Handb. Trop. Residual Soils Eng.*, pages 117–145. CRC Press, 2012. doi: doi:10.1201/b12302-5.
- Kevin E. Trenberth. Atmospheric moisture residence times and cycling: Implications for rainfall rates and climate change. *Climatic Change*, 39(4):667–694, 1998. ISSN 1573-1480. doi: 10.1023/A:1005319109110.
- M Th. van Genuchten. A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils1. *Soil Sci. Soc. Am. J.*, 44:892–898, 1980. doi: 10.2136/sssaj1980.03615995004400050002x.
- S K Vanapalli, D G Fredlund, D E Pufahl, and A W Clifton. Model for the prediction of shear strength with respect to soil suction. *Canadian Geotechnical Journal*, 33(3):379–392, 1996. doi: 10.1139/t96-060.
- Sai K. Vanapalli, M. V. Nicotera, and Radhey S. Sharma. Axis translation and negative water column techniques for suction control. *Geotechnical and Geological Engineering*, 26(6):645–660, 2008.
- A.S. Vesic and G.W. Clough. Behavior of granular materials under high stresses. *Journal of the Soil Mechanics and Foundations Division*, 94(3):661–688, 1968.
- Gonghui Wang and Kyoji Sassa. Pore-pressure generation and movement of rainfall-induced landslides: effects of grain size and fine-particle content. *Engineering Geology*, 69(1–2):109–125, 2003. ISSN 0013-7952. doi: http://dx.doi.org/10.1016/S0013-7952(02)00268-5.
- R. Wang, G. Zhang, and J.-M. Zhang. Centrifuge modelling of clay slope with montmorillonite weak layer under rainfall conditions. *Appl. Clay Sci.*, 50(3):386–394, 2010.
- R.G. Wetzel. *Limnology: Lake and River Ecosystems (3rd ed.)*. Academic Press, San Diego, CA, 2001.

- D J White and W A Take. GeoPIV : Particle Image Velocimetry (PIV) software for use in geotechnical testing. Technical Report October, 2002.
- D. J. White, W. A. Take, and M. D. Bolton. Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry. *Géotechnique*, 53(7):619–631, jan 2003. ISSN 0016-8505. doi: 10.1680/geot.2003.53.7.619.
- Y. F. Xu. Fractal approach to unsaturated shear strength. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(3):264–273, 2004. doi: 10.1061/(ASCE)1090-0241(2004)130:3(264).
- F.R. Young. *Cavitation*. Mc Graw - Hill, London, 1989.
- L L Zhang, L M Zhang, J Zhang, and W H Tang. Stability analysis of rainfall- induced slope failure : a review. *Geotech. Eng.*, 164(GE5):299–316, 2011.
- Y. Zhao. *In situ soil testing for foundation performance prediction*. PhD thesis, University of Cambridge, 2008.
- Annan Zhou, Run-Qiu Huang, and Daichao Sheng. Capillary water retention curve and shear strength of unsaturated soils. *Can. Geotech. J.*, 14(January):cgj–2015–0322, 2016. ISSN 0008-3674. doi: 10.1139/cgj-2015-0322.