Evaluation of the Performance of Pervious Concrete Pavement in the Canadian Climate

By

Vimy Henderson

A thesis
presented to the University of Waterloo
in fulfillment of the
thesis requirement for the degree of
Doctor of Philosophy
in
Civil Engineering

Waterloo, Ontario, Canada, 2012

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

ABSTRACT

Pervious concrete pavement has the capacity to perform as two types of infrastructure: a pavement; and a stormwater management solution. It is a low impact development as it does not alter the natural hydrological cycle when implemented, unlike a conventional impermeable pavement. This research represents some of the initial investigations into pervious concrete pavement in Canada. The two research hypotheses of this research were the following:

1. Pervious concrete pavement can be successfully planned, designed, constructed and maintained in Canada for successful performance based on surface evaluations of permeability rate and surface condition.
2. Verification that the subsurface drainage capabilities of pervious concrete pavement are as described in literature and can be quantified using instrumentation.

Through monitoring of the design, construction, performance and maintenance of five field sites across Canada and various laboratory pavement slabs, the behaviour of pervious concrete pavement in freeze-thaw conditions has been evaluated. This thesis presents the findings from the various phases of the life cycle of pervious concrete pavement: planning; design; construction; and maintenance. An interpretation of the performance of pervious concrete pavement both from the perspective of the surface and subsurface is included.

The various field sites led to pervious concrete being used in areas exposed to static or parked traffic and areas with slow moving traffic. At the two sites that included static and slow moving traffic, the permeability performance was better in the areas of static traffic than those with moving traffic. Each of the field sites had a unique mix design and some had multiple variations of one basic mix design. The relationship between the void content and hardened density of the pervious concrete cores was linear with none of the cores being visually identified as outliers.

Substantial deterioration in pavement structure performance was identified at one site. Other field sites showed changes in structural capacity over the monitoring timeline. However, no locations of substantial decreases in structural capacity were identified.

The surface condition of the sites over the analysis period indicated that compaction to the surface during construction was helpful in constructing a quality pavement. The results of the project indicated that pervious concrete will crack when joints are not included and may also crack similarly to conventional impermeable concrete pavements if joints are spaced too widely or do not match joints of adjacent pavement.

Washing the pervious concrete pavement surface with a large hose or garden hose was found to be the most effective in improving permeability across a site and also in increasing the permeability of the pervious concrete. The initial permeability of the pervious concrete pavement was found to influence future performance.

Freeze-thaw cycling and moisture were found to alter the internal structure of pervious concrete. However, did not generally lead to surface distress development. The application of sand as a winter maintenance method decreased the permeability, as did the use of a salt solution. However, neither winter maintenance method led to the permeability rates of laboratory slabs dropping below an acceptable level. All three slabs loaded with a salt solution deteriorated to a point where the slabs had failed. The initial permeability of the field sites proved to be important and although some sites started with what appeared to be very high permeability rates, these sites were successful in the multiple year evaluation in maintaining adequate permeability rates. The types of surface distresses that developed in the cores and slabs in the laboratory were generally not substantially worse at the field sites, suggesting that pedestrian and vehicle traffic do not necessarily escalate distresses caused by the Canadian climate and corresponding winter activities.

The subsurface drainage that was quantified by the instrumentation included in three field sites confirmed observations from the surface of the pavement and exceeded other expectations. Two field sites exhibited limited drainage capabilities on the surface of the pervious concrete pavement, one shortly after construction, and the other within a year following construction. The subsurface analysis quantified and confirmed that moisture was not able to drain completely vertically through the pavement structures at these two sites due to the limited access in the pervious concrete pavement surface. In comparison, the subsurface drainage at another site surpassed the assumed behaviour of pervious concrete pavement structures. The pavement structure in general at this site was highly permeable and this was identified as moisture was not observed to be collecting in the bottom of the storage base layer at any time or for any period of time. The successful overall drainage performance of this site demonstrates the ability to effectively use pervious concrete pavement in Canada.

ACKNOWLEDGEMENTS

The completion of this research and dissertation was possible only because of the continued support and encouragement from my friends, colleagues and family. The ongoing motivation that all of these individuals provided was tremendous and greatly appreciated. The interest in the research developments from family and friends was very encouraging.

I would like to thank my committee members for their assistance and efforts throughout the completion of this work: Dr. Jean Andrey, Dr. Leo Rothenburg, Dr. Ludomir Uzarowski, and Dr. Susan Tighe.

I am very appreciative of the funding provided by the Cement Association of Canada and Rico Fung. I would also like to highlight the industry partners who made this research possible: Dufferin Concrete (Derek Lapierre, Ross Bain, Gord Cawker, Kelly Nix); Ontario Ministry of Transportation (Chris Raymond, Linda Fischer, Becca Lane); Rempel Bros. Concrete (Larry Sunnus, Mike Carter, Brian Ball, Doug Blender); Lafarge (Wayne Tutt, Tom Richards, Ayman Maarouf, Paul Lum); and Béton Mobile du Québec inc. (Jacques Bertrand, Gerard Dube).

I am very grateful for the guidance and support from my supervisor, Dr. Susan Tighe, throughout my undergraduate and graduate studies. Her continual encouragement has led to many unique and valuable experiences and future opportunities to pursue.

The support that I received from Dr. Ludomir Uzarowski, a committee member, was greatly appreciated. Dr. Ludomir Uzarowski provided significant experience and encouragement on the work which I am grateful for.

I would like to thank Brian Mills for his insight and continued willingness to contribute to this research.

The assistance from members of the Centre for Pavement and Transportation Technology (CPATT) and various industry groups was greatly appreciated. Without this support this project would not have been possible. Ms. Jodi Norris at the CPATT was very supportive of my graduate research and very helpful on a multitude of aspects!

The patience, support, insight and encouragement provided by Ms. Rabiah Rizvi was invaluable and I am forever grateful for this.

Finally, I would like to extend a sincere thank you to everyone involved in this work. The contributions from each individual during both undergraduate and graduate work were appreciated and allowed for this project to be undertaken and completed.

DEDICATION

I would like to dedicate my thesis to my family in appreciation of their continued support. Thank you Dad, Mom, Jackie and Granny Vim!

TABLE OF CONTENTS

Author's Declaration ii
Abstract iii
Acknowledgements vi
Dedication viii
List of Figures xiv
List of Tables xvi
1.0 Introduction 1
1.1 Pervious Concrete Pavement 4
1.2 Benefits of Using Pervious Concrete Pavement 6
1.2.1 Environmental Benefits 6
1.2.2 Societal Benefits 8
1.2.3 Economic 9
1.3 Challenges in Using Pervious Concrete Pavement 10
1.4 Research Scope and Hypotheses 12
1.5 Research Objectives 13
1.6 Research Contribution 13
1.7 Organization of Thesis 16
2.0 Pervious Concrete Pavement in Cold Climates 18
2.1 Stormwater Management and Pervious Concrete 19
2.2 Pervious Concrete Pavement 22
2.2.1 Cold Climate Experience 23
2.2.2 Mix Design 24
2.2.2.1 Aggregate 25
2.2.2.2 Cement Content 26
2.2.2.3 Admixtures and Additional Cementitious Materials 27
2.2.3 Pavement Structure 29
2.2.3.1 Subgrade 30
2.2.3.2 Granular Base 32
2.2.3.3 Pervious Concrete Surface 34
2.2.4 Construction 34
2.2.4.1 Site Preparation and Material Placement 35
2.2.4.2 Curing 37

2.2.4.3 Joints ... 38
2.2.4.4 Opening to Traffic ... 39
2.2.5 Surface Distresses ... 39
2.2.5.1 Slab Ravelling ... 40
2.2.5.2 Joint Ravelling ... 41
2.2.5.3 Cracking ... 42
2.2.5.4 Sealing ... 42
2.2.6 Maintenance ... 43
2.2.6.1 Winter Maintenance ... 43
2.2.6.2 Permeability Renewal Maintenance ... 45
2.2.6.3 Distress Maintenance ... 45
2.3 Summary ... 46
3.0 Data Sources and Testing Materials ... 49
3.1 Concrete Testing Procedures ... 49
3.1.1 Slump Test ... 49
3.1.2 Air Content ... 51
3.1.3 Density ... 52
3.1.3.1 CSA A23.2-6C “Density, Yield and Cementing Materials Factor of Plastic Concrete” ... 53
3.1.3.2 ASTM C1688 “Standard Test Method for Density and Void Content of Freshly Mixed Pervious Concrete” ... 53
3.1.4 Temperature ... 55
3.1.5 Compressive Strength ... 55
3.1.6 Flexural Strength ... 57
3.1.7 Void Content ... 58
3.1.7.1 Corelok ® ... 59
3.1.7.2 T-040R ... 59
3.1.8 Freeze-Thaw Durability ... 59
3.1.9 Permeability ... 60
3.1.9.1 Permeability with Gilson Permeameter ... 60
3.2 Surface Distress Evaluation Form ... 65
3.3 Instrumentation ... 65
3.3.1 Weather Data ... 66
3.3.2 Moisture Gauges ... 67

3.3.3 Maturity Sensors ... 67
3.3.4 Temperature Sensors ... 68
3.4 Field Sites ... 69
3.4.1 Site 1 ... 71
3.4.2 Site 2 ... 71
3.4.3 Site 3 ... 72
3.4.4 Site 4 ... 72
3.4.5 Site 5 ... 73
3.4.6 Surface Distress Evaluations ... 73
3.4.7 Permeability ... 73
3.4.8 Portable Falling Weight Deflectometer ... 74
3.4.9 Permeability Renewal Maintenance Methods ... 74
3.5 Summary ... 74
4.0 Planning and Design of Pervious Concrete Pavement ... 76
4.1 Planning ... 77
4.1.1 Application ... 77
4.2 Design ... 79
4.2.1 Mix Design ... 80
4.2.1.1 Fine Aggregate ... 82
4.2.1.2 Water Cement Ratio ... 83
4.2.1.3 Aggregate Type ... 84
4.2.1.4 Aggregate Size ... 85
4.2.1.5 Admixtures ... 86
4.2.2 Mix Design Summary ... 87
4.2.3 Pavement Structure ... 88
4.2.3.1 Pavement Structure Designs ... 89
4.2.3.2 Structural Integrity of Field Sites ... 90
4.3 Summary ... 99
5.0 Construction ... 101
5.1 Source ... 101
5.2 Method ... 102
5.3 Joints ... 105
5.3.1 Performance ... 106

5.3.1.1 Formed Joints ... 107
5.3.1.2 Saw Cut ... 109
5.3.2 Construction Joints ... 111
5.3.3 No Joints and Ineffective Joints ... 115
5.3.4 Curing ... 117
5.4 Maintenance ... 118
5.4.1 Permeability Renewal Maintenance ... 118
5.4.1.1 Permeability Renewal Maintenance Methods ... 118
5.4.2 Results ... 120
5.4.3 Analysis ... 125
5.5 Summary ... 129
6.0 Pervious Concrete Pavement Surface Performance ... 131
6.1 Samples and Field Sites in Analysis ... 131
6.2 Factors ... 134
6.3 Testing ... 135
6.4 Results ... 136
6.4.1 Permeability of Cores ... 136
6.4.2 Surface Condition of Cores ... 137
6.4.3 Permeability of Slabs ... 139
6.4.4 Surface Condition of Slabs ... 145
6.4.5 Permeability of Field Sites ... 149
6.4.6 Surface Condition of Field Sites ... 151
6.5 Analysis ... 152
6.5.1 Permeability ... 152
6.5.2 Surface Condition ... 165
6.6 Summary ... 168
7.0 Subsurface Drainage Behaviour of Pervious Concrete Pavement ... 171
7.1 Moisture Gauge Data Computation ... 171
7.2 Weather Data ... 174
7.3 Site 3 ... 175
7.3.1 Instrumentation Functionality Verification ... 178
7.3.2 Analysis ... 187
7.4 Site 4 ... 192

7.4.1 Instrumentation Functionality Verification ... 193
7.4.2 Maintenance at Site 4 in Spring 2010 ... 197
7.4.3 Analysis ... 200
7.5 Site 5 ... 206
7.5.1 Instrumentation Functionality Verification ... 206
7.5.2 Analysis ... 211
7.6 Findings ... 214
7.7 Summary ... 216
8.0 Conclusions and Recommendations ... 218
8.1 Recommendations to Practioners for the Use of Pervious Concrete Pavement ... 224
8.2 Recommendations for Future Research ... 225
References ... 229

APPENDICES

Appendix A ... 237
Appendix B ... 239
Appendix C ... 249
Appendix D ... 256
Appendix E ... 261

LIST OF FIGURES

Figure 1-1: Descriptions of Sustainability (Hart, 1999) ... 1
Figure 1-2: Pervious Concrete Pavement Sustainability Flow Chart (Henderson & Tighe, 2011) ... 4
Figure 1-3: Pervious Concrete Pavement ... 5
Figure 1-4: Flow Chart of Research Scope ... 14
Figure 2-1: Trends in Water Yield in Southern Canada (Statistics Canada, 2010) ... 18
Figure 2-2: Treatment Train for Stormwater and the Inclusion of Pervious Concrete Pavement (MOE, 2003; Walker, 2009) ... 21
Figure 2-3: Pervious Concrete Pavement Structure ... 30
Figure 2-4: Underdrain Pipe for Low Permeability Subgrade Soil ... 31
Figure 3-1: Slump Testing ... 50
Figure 3-2: Measuring the Slump of Pervious Concrete ... 51
Figure 3-3: Air Meter Bucket ... 52
Figure 3-4: Proctor Hammer Compacting Pervious Concrete for Density Testing ... 54
Figure 3-5: Cylinder Sample End Preparation by End Grinding ... 56
Figure 3-6: Flexural Beam in Testing Frame ... 58
Figure 3-7: Gilson Permeameter ... 61
Figure 3-8: Graduated Measurement Markings on Permeameter ... 61
Figure 3-9: Permeability Testing of a Slab in the Laboratory ... 62
Figure 3-10: Permeability of Cores and Cylinders in the Laboratory ... 63
Figure 3-11: Weather Station at a Field Site ... 66
Figure 3-12: Sensor Tree including three Moisture Gauges ... 67
Figure 3-13: Maturity sensor being placed in a slab during casting ... 68
Figure 3-14: Temperature Probes Being Installed ... 68
Figure 3-15: Field Site Locations ... 69
Figure 4-1: Permeability Based on Application ... 78
Figure 4-2: Void Content and Density of Cores ... 82
Figure 4-3: Void Content and Density of Cores with and without Fine Aggregate ... 83
Figure 4-4: Pervious Concrete Water Cement Ratio and Void Content ... 84
Figure 4-5: Void Content and Density of Cores with Different Aggregate Types ... 85
Figure 4-6: Void Content and Density of Cores with Different Sizes of Aggregate ... 86
Figure 4-7: Void Content and Density of Cores with Admixtures ... 87
Figure 4-8: PFWD Results at Site 5 ... 91
Figure 4-9: PFWD Results from Sites 2, 4 and 5 at 22 to 24 Months ... 97
Figure 4-10: PFWD Results at Site 2 ... 98
Figure 5-1: Varying Density in a Core from Site 5 ... 103
Figure 5-2: Core 2U1 from Site 3 ... 104
Figure 5-3: Joint Former ... 105
Figure 5-4: Formed Joint after Construction Site 1 October 2008 (16 Months) ... 107
Figure 5-5: Formed Joint August 2009 Site 1 (26 Months) ... 107
Figure 5-6: Formed Joint March 2010 (33 Months) Site 1 ... 108
Figure 5-7: Formed Joint April 2011 (46 Months) Site 1 ... 108
Figure 5-8: Saw Cut Joint After Construction Site 4 August 2009 (10 Months) ... 110

Figure 5-9: Saw Cut Joint Site 5 August 2010 (22 Months) 110
Figure 5-10: Saw Cut Joint Site 5 June 2011 (32 Months) 111
Figure 5-11: Construction of Site 4 113
Figure 5-12: Construction Joint A/B at Site 4 November 2008 (1 Month) 113
Figure 5-13: Construction Joint A/B at Site 4 June 2010 (20 Months) 114
Figure 5-14: Construction Joint A/B at Site 4 March 2011 (29 Months) 114
Figure 5-15: Effectiveness of Permeability Renewal Maintenance Methods 123
Figure 5-16: Average Permeability of Field Sites 126
Figure 6-1: Core Permeability (Surface to Bottom) 137
Figure 6-2: Core Permeability (Bottom to Surface) 137
Figure 6-3: Weight of Cores During Freeze-Thaw Cycling 138
Figure 6-4: Failure Progression of Mix 5B Core 139
Figure 6-5: Permeability of Slabs Loaded with Sand 140
Figure 6-6: Permeability of HSaltHP and HP Slab Groups 142
Figure 6-7: Weight of Slabs throughout Freeze-Thaw Cycles 146
Figure 6-8: Weight of Slab Group HSaltHP throughout Freeze-Thaw Cycling 148
Figure 6-9: Weight of Slab Group HP 148
Figure 6-10: Field Site Permeability 149
Figure 7-1: Example of Moisture Gauge Data 173
Figure 7-2: Moisture Gauge Sensor Tree A at Site 3A 176
Figure 7-3: Moisture Gauge Sensor Tree B at Site 3A 176
Figure 7-4: Moisture Gauge Sensor Tree C at Site 3B 177
Figure 7-5: Moisture Gauge Sensor Tree D at Site 3B 177
Figure 7-6: Sensor Trees at Site 3 178
Figure 7-7: Sensor Tree B Response to Freezing Temperatures at Site 3A 182
Figure 7-8: Sensor Tree B Response to Freezing Temperatures (Smaller Scale) at Site 3A 182
Figure 7-9: Sensor Tree A Winter 2009 to Spring 2011 184
Figure 7-10: Sensor Tree B Winter 2009 to Spring 2011 184
Figure 7-11: Sensor Tree at Site 4 193
Figure 7-12: Sensor Trees at Site 4 194
Figure 7-13: Rain Event Site 4A Fall 2008 194
Figure 7-14: Rain Event Site 4B Fall 2008 195
Figure 7-15: Rain Event Site 4C Fall 2008 196
Figure 7-16: Permeability of Site 4 October 2008 197
Figure 7-17: Site 4A Following Maintenance May 2010 198
Figure 7-18: Site 4B Following Maintenance May 2010 198
Figure 7-19: Site 4C Following Maintenance May 2010 199
Figure 7-20: Sensor Tree Site 5 206
Figure 7-21: Moisture Gauges in Fall and Spring Seasons 2009 - 2011 208

LIST OF TABLES

Table 2-1: Granular Gradations for Storage Base Layer (AASHTO, 2008) 33
Table 3-1: Field Site Matrix 70
Table 4-1: t-test Application 79
Table 4-2: Mix Design Details of Each Site 81
Table 4-3: Field Site Pavement Structures 89
Table 4-4: Comparison of Structural Condition of Field Site Over Time 93
Table 4-5: Differences of PFWD Means at Field Sites Over Time 94
Table 4-6: Evaluation of Pavement Structures at Comparable Ages 97
Table 5-1: Void Content Differences Throughout Cores 104
Table 5-2: Permeability Renewable Maintenance Method Results 122
Table 5-3: Differences in Permeability Rate with Full Scale Maintenance 124
Table 5-4: Ranking of Permeability Renewal Maintenance Methods Results 128
Table 6-1: Sample Type and Associated Factors 131
Table 6-2: Void Content of Samples 133
Table 6-3: Permeability of Heavy and Moderate Sand Loaded Slab Groups 141
Table 6-4: Permeability of Slabs and Precipitation Loading 141
Table 6-5: HSHP and HSMP Compared to HSaltHP 143
Table 6-6: t-tests Comparing Moderate Sand Loaded Slabs with HSaltHP Slabs 143
Table 6-7: Comparison of Mean Permeability Rates of HSaltHP and HP 144
Table 6-8: Slab Surface Condition 147
Table 6-9: Permeability ANOVA Table Results for Field Sites 150
Table 6-10: Field Sites 1, 2 and 3 Surface Conditions 153
Table 6-11: Field Sites 4 and 5 Surface Conditions 154
Table 6-12: t-test of Mean Permeability Rates at 20 Months and 24 Months of Age at Site 4 158
Table 6-13: Statistical Analysis of Permeability from Field Sites and Laboratory Slabs at Various Ages 160
Table 6-14: Confidence Intervals of Mean Permeability Rates Between Field Sites and Laboratory Cast Slabs 162
Table 6-15: Core and Slab Comparison 164
Table 6-16: Comparison of Cores and Slabs Permeability 165
Table 7-1: Fall 2009 Site 3 Moisture Readings 180
Table 7-2: Moisture Movement Site 3 188
Table 7-3: Site 3 Rain Events 190
Table 7-4: Moisture Movement at Site 4 201
Table 7-5: Site 4 Rain Events 202
Table 7-6: Moisture Gauge SWP Results July 5, 2010 to July 16, 2010 209
Table 7-7: Moisture Gauge SWP Results September 29, 2010 to October 7, 2010 209
Table 7-8: Summary of Water Movement through Pervious Concrete Pavement Structure 212
Table 7-9: Site 5 Rain Events 213

1.0 INTRODUCTION

A well-known, early reference to sustainable development and sustainability is presented in the Brundtland report in 1987 where the term sustainable development is defined as "development that meets the needs of the present without compromising the ability of future generations to meet their own needs. The concept of sustainable development does imply limits – not absolute limits but limitations" (Brundtland, 1987). At that time the term sustainability was not a common theme for industries and educational environments. In 2012 and recent years however, sustainability has become a large factor in decisions and activities around the world. A community can be described as having three aspects; society, economy and environment, also referred to as the triple bottom line. Often, each of these three groups works individually and in return mitigates problems related to their particular focus. However, it may not be possible to solve challenges without the incorporation of the other two aspects and the associated groups (Marsalek, Kok, & Colas, 2004). To evolve sustainability in the future development of a community, the three parts need to work together to achieve solutions. Figure 1-1A shows the illustration that is commonly used to present the concept of sustainability. Figure 1-1B portrays Hart's description of these three groups and their interaction (Hart, 1999).

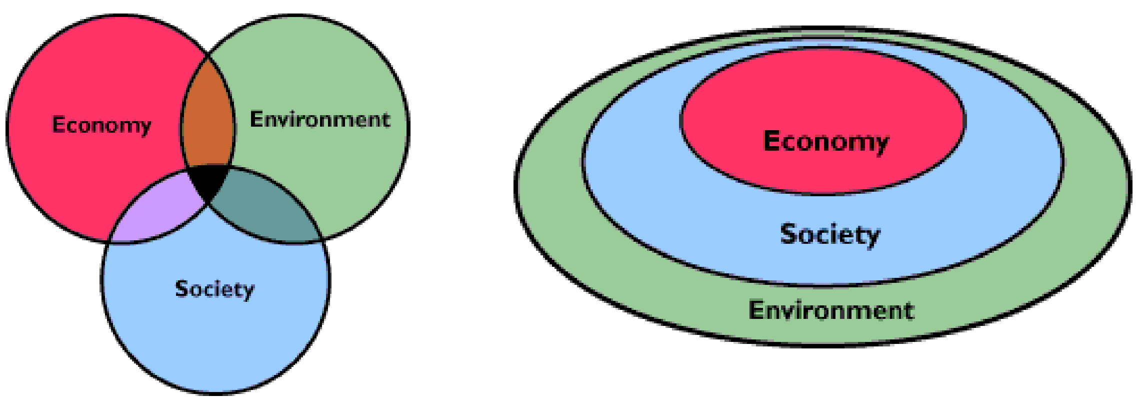


Figure 1-1A Figure 1-1B

Figure 1-1: Descriptions of Sustainability (Hart, 1999)

Figure 1-1A presents a Venn diagram that describes sustainability and the interaction between the economy, environment and society. Ideally solutions to a challenge would contain components of these elements and fall within the black area in the center, sustainability. Figure 1-1B presents a different description of sustainability and the associated interaction of these three groups. Hart explains that in Figure 1-1B the economy is based on society and exists within society. However, society includes more than just the economy. Other aspects of society include friends, family, art and so on which may involve the economy in some aspects but not entirely. All aspects of society are within the environment and society depends on the products of the environment such as food and water to exist. One of the significant challenges of sustainable development is that sustainability is a broad concept that involves many aspects which all contribute to achieving optimum solutions.

When sustainability is considered in the context of infrastructure, the definition remains similar and is described by Community Research Connections as "…the designing, building, and operating of these structural elements in ways that do not diminish the social, economic and ecological processes required to maintain human equity, diversity and the functionality of natural systems." (CRC, 2009). A key factor in developing sustainable infrastructure is the consideration of all aspects of the life cycle of the asset from design to operation. Community Research Connections describes sustainable infrastructure as having five focus areas: energy; transportation; waste management; land use planning; and governance. Similar to the definition of sustainable development, it is apparent that in order to achieve sustainability in these five areas, connections need to be made between them. This concept is included in efforts to improve streetscapes from a sustainable perspective. The City of Chicago has compiled a Green Alley Handbook, the Region of York is developing Complete Streets, the Ontario Professional Planners Institute has published Planning by Design: A Healthy Communities Handbook with the Ministry of Municipal Affairs and the City of Vancouver is promoting Green Design for streets (Daley, 2008; City of Vancouver, 2009; York Region, 2012; Ontario Professional Planners Institute, 2009). Similar documents are available from other cities and municipalities as well; the list above is a small example.

Greenberg has carried out work to assess the current practice of sustainable street design and development. Within this work, Greenberg describes sustainable streets as "…multimodal rights of way designed and operated to create benefits relating to mobility, ecology, and community that together support a broad sustainability agenda embracing the three E's of environment, equity, and economy" (Greenberg, 2009). The definition of sustainable streets highlights that although a particular infrastructure asset is being considered, streets, the intent is to optimize several aspects, which would consider at least some if not all of the focus areas mentioned by the Community Research Connection. In efforts to heighten the sustainability of streetscapes it is important to ensure that the quality of each asset is not compromised.

Pervious concrete pavement is an alternative to conventional impermeable pavements. Pervious concrete pavement is a type of permeable pavement, others include porous asphalt and permeable interlocking concrete pavers. Pervious concrete is currently primarily considered for low volume, low speed applications. Low volume, low speed traffic is describing personal vehicles in applications where the traffic is moving at the speed of residential neighbourhoods or less (50 km/hour maximum).The characteristics of pervious concrete pavement make it a sustainable alternative and a Low Impact Development (LID). Pervious concrete has a high rate of permeability due to the open void structure of the material. Water is able to drain from the surface and infiltrate into the groundwater therefore not altering the natural hydrological cycle or increasing the demand on the local stormwater management system.

The introduction of pervious concrete pavement to cold weather climates, specifically Canada, was initiated by the sustainable benefits it offers. Figure 1-2 demonstrates how pervious concrete pavement can be involved in sustainable development and sustainable streets.

Pervious concrete pavement has been used in other areas of North America; however there has been caution in the pavement industry to implement it in climates that experience freeze-thaw cycles (NRMCA, Freeze-Thaw Resistance of Pervious Concrete, 2004). The presence of sustainability driven decisions in Canada is currently notable throughout communities and industries. Pervious concrete has potential to provide a unique balance between the three areas (society, environment and economy) shown in the illustrations in Figure 1-1. In order to experience the potential benefits in each of these areas from using pervious concrete, it is critical that the pavement perform and function to the expectations of the owners and users. The work presented in this thesis is intended to assist in the development of quality pervious concrete pavement applications in Canada.

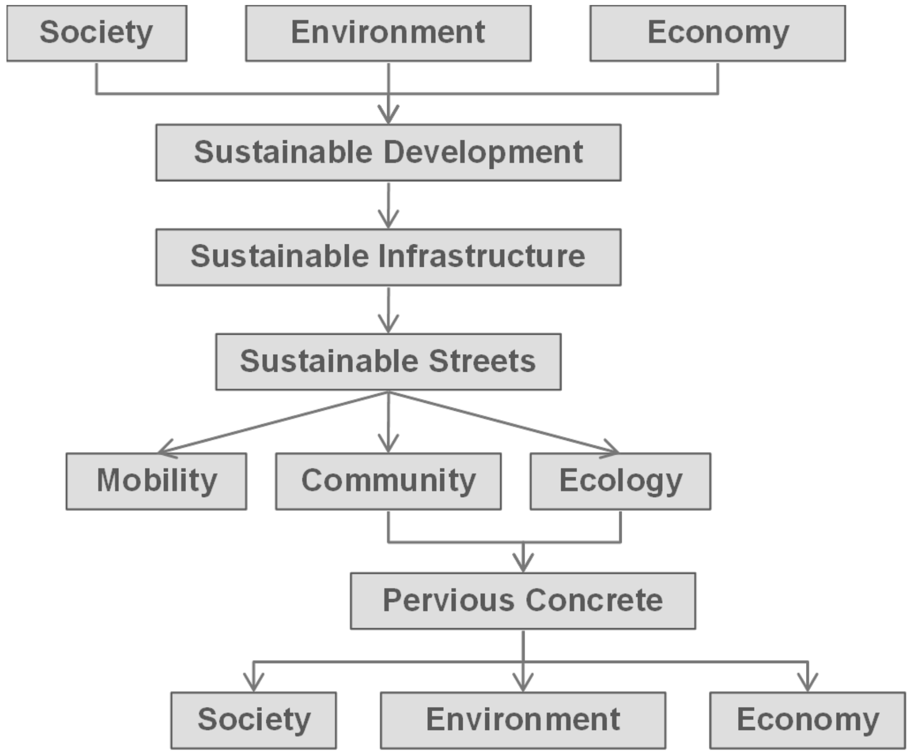


Figure 1-2: Pervious Concrete Pavement Sustainability Flow Chart (Henderson & Tighe, 2011)

1.1 Pervious Concrete Pavement

Pervious concrete pavement is a sustainable solution to a challenge that often exists in urban areas. The challenge is handling the large amount of existing impervious area in urban environments and additional area created during urban growth and development. The challenge of impermeable areas is that runoff is produced and it requires an infrastructure system for adequate control (Walker, 2009). Pervious concrete pavement has two functions: a paved surface available for low volume, low speed applications; and a stormwater management alternative. The rigid concrete creates a paved surface for various uses while the open voids in the concrete allow water to drain from the surface. Research has been done to develop pervious concrete mixes for high volume applications (PCA, 2009). In this work, only low volume and low speed applications were considered. The low volume, low speed applications that are generally considered for pervious concrete pavement are:

- Sidewalks;

- Paths;
- Driveways;
- Parking lots;
- Shoulders; and
- Residential streets.

Pervious concrete can be used in active or passive drainage applications and therefore reduces the demand to develop a high strength mix at this time. Active drainage occurs in scenarios when runoff from surrounding impermeable areas is drained through the pervious concrete pavement structure in addition to the precipitation occurring directly on the pervious concrete pavement. Passive drainage occurs when only precipitation falling directly on the pervious concrete pavement system is being drained through the pavement structure (PCA, 2012). Figure 1-3 shows a sample of pervious concrete pavement from a parking lot in southwestern Ontario.

Figure 1-3: Pervious Concrete Pavement

As shown in Figure 1-3, the consistency of pervious concrete includes several interconnected voids, generally ranging from 15% to 30%. These voids and the connectivity of them allow moisture to be drained from the surface. The moisture drained from the surface can then infiltrate into the groundwater or be directed through a pipe network.

1.2 Benefits of Using Pervious Concrete Pavement

Pervious concrete pavement offers many benefits when integrated into urban areas. The ability for pervious concrete to drain water from the surface offers benefits which span various sectors including the environment, society and economy. Figure 1-2 described the relationship between sustainable development and pervious concrete pavement. The relationship demonstrated in Figure 1-2 is unique to pervious concrete pavement as not all pavement or stormwater management alternatives would fulfil the presented features.In all three aspects of sustainability, the benefits that can be achieved from the use of pervious concrete pavement are often immediate as well as long term and connected.

1.2.1 Environmental Benefits

The low impact of using pervious concrete pavement is one of the main factors attributing to the environmental benefits of this material. The drainage capabilities of pervious concrete pavement lead to developments that have minimal to no effect on the natural water cycle in the area. The reduction or often elimination of runoff in areas paved with pervious concrete pavement, in comparison to the use of an impermeable pavement, is a tremendous environmental gain. Permeable pavements are commonly referenced as Best Management Practices (BMP) for source control measures of urban stormwater management.

The City of Toronto’s 2003 Wet Weather Flow Management Master Plan includes porous pavement as primarily a source control method for stormwater management but also conveyance as an additional application. It notes that porous pavements reduce erosion impacts and re-establish natural hydrological processes (City of Toronto, 2003). The 2006 Halifax Regional Municipality Stormwater Management Guidelines reference permeable pavement as a common BMP in literature (Devereauz & Lorant, 2006). Permeable pavement is referenced by many cities and municipalities throughout Canada as being a stormwater management alternative. This is discussed further in Section 2.1.

The City of Calgary's stormwater strategy goals includes reducing both rates and volumes of stormwater runoff to protect the health of the watershed. The solution to these goals is noted as being the development and implementation of innovative stormwater alternatives in new and redeveloped areas. Sustainable streetscapes and permeable pavements are included as source control solutions (Bozic, Deong, & Fesko, 2007).

The United States Environmental Protection Agency (USEPA) has acknowledged two requirements in the recent development of the National Pollutant Discharge Elimination System (NPDES). These requirements are: private and public land owners reduce the amount of stormwater runoff on their property; and reduce the contaminants in the runoff water to near pre-development levels (EPA, 2008). These reductions can be achieved by detention ponds and vegetative buffers; however, pervious concrete pavement has also been recognized as an effective tool in reducing stormwater runoff and initially treating stormwater (WERF, 2005). The USEPA has recognized the abilities of pervious concrete pavement to reduce stormwater and act as a management tool by including it as a Best Management Practice. The benefit of eliminating runoff and maintaining the local, natural hydrological cycle is substantial to the surrounding environment.

The additional benefit of pervious concrete pavement, similar to any concrete pavement, is that if there is runoff over the surface, it will not become heated. Surfaces that are dark in colour such as roofs and asphalt parking lots can cause runoff that is much higher in temperature than the rainfall naturally was. Heated runoff is detrimental to aquatic life if it reaches streams and waterways (Roa-Espinosa, Norman , Wilson, & Johnson, 2003).

Projects have been carried out by many groups to evaluate the ability of pervious concrete pavement to remove particulate from water that moves through the pavement structure. Pollutants that would otherwise remain in runoff and contaminate runoff have been shown to be removed during filtering through the voids in a pervious concrete pavement (Tennis, Leming, & Akers, 2004).

1.2.2 Societal Benefits

Often the environmental benefits of sustainable infrastructure alternatives are more apparent to the user than those that improve society. Pervious concrete has several attributes that can improve the quality of life for the local community. Many of the societal benefits that pervious concrete exhibits are shared by conventional concrete pavements as well. These include minimizing heat islands and increased reflectivity. Both of these benefits are attributed to the colour of concrete, light grey, in comparison to asphalt pavement which is black in colour. The urban heat island effect occurs in urban areas with many dark surfaces that lead to the localized air temperature being higher than it would be on a comparable day in a rural environment. Research indicates that the air temperature in urban areas can be up to 4°C degrees higher than it would be in a rural setting (Pon, 2000). The effect of this increased temperature, is a higher frequency of heat related illness, especially in the susceptible population: children; and seniors (EPA (. P., 2009). While the effects of urban heat islands on humans are of great concern to communities and cities, reductions of urban heat islands also benefit the environment and economy. By maintaining the natural air temperature and not increasing it unnecessarily, due to dark areas, the demand for resources is not accelerated. Therefore additional resources and funds are not spent cooling buildings because of increased outdoor temperature (Pon, 2000). Pervious concrete allows for the development of paved surfaces without creating dark, hot areas.

The second benefit of the light colour of pervious concrete is that it is more reflective and therefore brighter during the evening and night than dark pavements. The lighter colour of the surface has the potential of reducing the amount of lighting required. In conventional, impermeable concrete pavements, the light colour has been shown to reduce the lighting requirement by 30 % (Cement Association of Canada, 2011). Similar to the reduction in heat islands, the high albedo of pervious concrete is beneficial to not only pavement users but also the environment and economy.

The use of pervious concrete pavement minimizes or eliminates stormwater runoff. Where stormwater runoff exists it is drained to a stormwater management pond in some scenarios. Stormwater management ponds use valuable land and are a safety hazards as drowning continues to be the cause of a significant number of deaths in Canada and the

United States. In addition to being safety hazards, stormwater management ponds are large areas of standing water which provide opportunities for disease development, such as West Nile (SKC, 2007; Heron, Hoyert, Murphy, Xu, Kochanek, & Tejada-Vera, 2009; Beatty, 2007).

A benefit that is easily recognizable by users is the dry surface that is constant on pervious concrete pavement surfaces. The high drainage rates of pervious concrete pavement lead to immediate movement of stormwater off the surface, therefore resulting in splash and puddle free areas. In addition the potential to harvest this rainwater for landscaping is attractive. While this benefit of a continually dry surface, or at least puddle free, may seem minimal, it is a reflection of the local area and presents an inviting environment to the clients of the surrounding businesses. The benefits that pervious concrete provides to the local community are significant and a key reason why this material can be used in applications to achieve sustainable results.

1.2.3 Economic

Pervious concrete pavements have the potential to exhibit the same low life cycle costs as conventional concrete pavements. The life cycle cost of conventional concrete pavement is low because concrete has a longer life time than other paving materials and can require less maintenance throughout the life cycle. Pervious concrete that is well designed and constructed should also exhibit similar life cycle performance (NRMCA, Economic Benefits, 2010).

Pervious concrete pavement can offer economical benefits to individuals using it on their own property as it maintains the natural water cycle. Since pervious concrete is a low impact development it ensures that surrounding vegetation, such as gardens and lawns, receive natural moisture. This limits expenses for the home owner related to watering. In the case of both private and commercial properties pervious concrete can be used in a water harvesting system which reduces the water demand.

In order to accommodate stormwater management systems, additional land is often required for stormwater retention ponds. Stormwater management systems also require infrastructure such as pipe networks which adds to the expense of a project. The use of pervious concrete pavement can eliminate the need for land and infrastructure to support stormwater management systems (NRMCA, 2010).

Dark areas in urban centres result in urban heat islands which are harmful to the health of the community but also result in an increased expense related to cooling surrounding buildings (Pon, 2000). If urban heat islands can be mitigated the cooling costs of the surrounding community will not be increased and therefore less resources and funding will be required. The light colour of concrete reduces the amount of lighting infrastructure required to create the desired brightness of parking lots and paths during evening and night use. It is suspected that the light colour of pervious concrete pavement will have a similar benefit. In addition to less funding being spent on lighting infrastructure, less funding is required during the life cycle of the infrastructure as there are fewer assets to supply power for. Achievement of economical benefits with the use of pervious concrete contributes to the sustainability of the material.

1.3 Challenges in Using Pervious Concrete Pavement

Pervious concrete has been used in climate without regular freeze-thaw cycles for decades; however, use in freeze-thaw climates such as Canada has been limited (ConcreteNetwork, 2010). The climate in Canada presents additional challenges for optimizing pervious concrete performance. However, some challenges to using pervious concrete pavement are present in all climates. These include: ensuring appropriate site design; achieving adequate mix performance; and complete compliance by involved parties to use educated staff for construction. All three of these elements are essential in achieving a quality pervious concrete pavement in any climate (Delatte, Miller, & Mrkajic, 2007).

While pervious concrete offers exceptional drainage capabilities, industry members are conscientious of the limitations of this material. Land uses such as commercial nurseries, automobile recycling facilities, gas stations and outdoor liquid container storage areas should not include pervious concrete pavement as these are stormwater hot spots. These areas create runoff with high levels of contamination and pervious concrete can provide a path for the contaminated runoff to enter groundwater (ACPA, 2006).

A pervious concrete application should be considered in terms of traffic loading. The high drainage capability of pervious concrete allows for use in active drainage applications. These are locations where the pervious concrete pavement handles drainage from adjacent, impervious surfaces. This design alternative allows for higher strength materials to be used in heavily loaded areas without creating a need for a stormwater management system as the runoff can be captured by the adjacent pervious concrete (NRMCA, Hydrological Design Considerations, 2010). Similar to all paving materials, when pervious concrete is placed in a location where it experiences more loading than the design included, the result is surface distress development and poor performance for the owner.

The mix design is critical in achieving quality pervious concrete pavement and can present challenges. It is necessary to achieve the correct balance in the mix design in order to have adequate durability and permeability. Admixtures, fine aggregate and fibres have all been used to improve the performance of pervious concrete mixes (Kevern J. , Wang, Suleiman, & Schaefer, 2005; Wang, Schaefer, Suleiman, & Kevern, 2006).

As pervious concrete is unique from conventional concrete it is essential that the construction crew be familiar and educated about the material. Over compaction can result in sealing of the surface which eliminates the permeability characteristics. Slight compaction of the surface is necessary to develop strength in the material and minimize future ravelling. In addition, the mix should be handled minimally and efficiently in order to achieve optimum quality (NRMCA, 2010).

The greatest challenge of implementing pervious concrete pavement throughout Canada specifically is the concern related to the durability of the material in freeze-thaw conditions. Work being carried out in the northern United States has indicated with recent results that freeze-thaw cycles are not necessarily damaging to pervious concrete (Kevern J. , Wang, Suleiman, & Schaefer, 2005; Wang, Schaefer, Suleiman, & Kevern, 2006; Delatte, Miller, & Mrkajic, 2007). Distresses caused by freeze-thaw cycling could occur if water was trapped in the voids when these changes in temperature occurred. The high permeability rate of pervious concrete reduces the opportunity for water to remain in the voids. The suggested pavement structure for use in freeze-thaw climates is designed with the intention that water moves quickly through the pervious concrete and accumulates in the reservoir layer. The water then infiltrates into the natural subgrade.

1.4 Research Scope and Hypotheses

The scope of this project was to evaluate the performance of pervious concrete pavement in the Canadian climate. Figure 1-4 shows a flow chart of the activities involved in the scope of this project. The two research hypotheses of this research were the following:

1. Pervious concrete pavement can be successfully planned, designed, constructed and maintained in Canada for successful performance based on surface evaluations of permeability rate and surface condition.
2. Verification that the subsurface drainage capabilities of pervious concrete pavement are as described in literature and can be quantified using instrumentation.

This research involved the planning, design, construction and maintenance of pervious concrete pavement field sites throughout Canada as well as in the controlled laboratory setting at the Centre for Pavement and Transportation Technology (CPATT) laboratory. The inclusion of multiple field sites was instrumental as the monitoring started in the planning phase and continued with performance evaluations for multiple years. Each field site allowed for growth from the previous in terms of techniques and methods and provided information for the next. Field sites were monitored from the time of construction to a maximum of four years of use.

Scenarios were created in the laboratory that replicated the Canadian climate by freeze-thaw cycling samples in a walk-in freezer. Different winter maintenance techniques were applied to the samples that underwent several years of accelerated freeze-thaw cycling to monitor the effect of each. The laboratory study included accelerated testing such that samples were exposed to five to eight years of freeze-thaw climatic conditions. The inclusion of the field sites and laboratory testing provided opportunities to fully evaluate the behavior of this innovative material in the Canadian environment. Although the research did not span the anticipated life of a pervious concrete pavement (20 to 30 years), the results of the work are anticipated to be useful in determining if and how this life span can be achieved.

The project was initiated in 2007 as there was an interest from the Cement Association of Canada (CAC) and industry members to investigate the use of pervious concrete pavement in Canada. The CAC, CPATT at the University of Waterloo and industry members started this integrated laboratory and field study in 2007. The final work that will be completed for this research will involve the development of a Design, Construction and Maintenance Guide for the use of pervious concrete pavement in Canada. The guide will be intended for use by researchers and practitioners and will be based on the research findings of this doctoral thesis.

1.5 Research Objectives

The main focus of this work was to evaluate if pervious concrete pavement could be successfully used in Canada. In assessing if pervious concrete pavement could be successfully used in Canada, two areas of performance were considered; permeability rate and surface condition. The drainage behaviour of pervious concrete pavement below the surface was observed. The objectives of this research were the following:

1. Determination of pervious concrete applications, pavement structure designs, mix designs and construction methods that are needed for a successful pervious concrete pavement in Canada.
2. Evaluation of permeability renewal maintenance methods to determine suitable methods for use on pervious concrete pavement in Canada.
3. Assessment of the effect of freeze-thaw climate factors including freeze-thaw cycles, winter maintenance and vehicular and pedestrian traffic on the performance (permeability and surface condition) of pervious concrete pavement in Canada.
4. Description of the movement of moisture below the surface in a pervious concrete pavement structure and comparison to the assumed drainage characteristics.

1.6 Research Contribution

The basis of this thesis and the overall intended contribution is to identify the actions from the planning to the permeability renewal maintenance stages in a pervious concrete pavement life cycle that were effective as well as those that were not in producing a quality product. In terms of pervious concrete pavement, a quality product is deemed to be a pavement that remains sufficiently permeable throughout the design life. It is recognized that in order to remain sufficiently permeable, permeability renewal maintenance may need to be performed. The second aspect of a quality pervious concrete pavement product is maintaining a surface

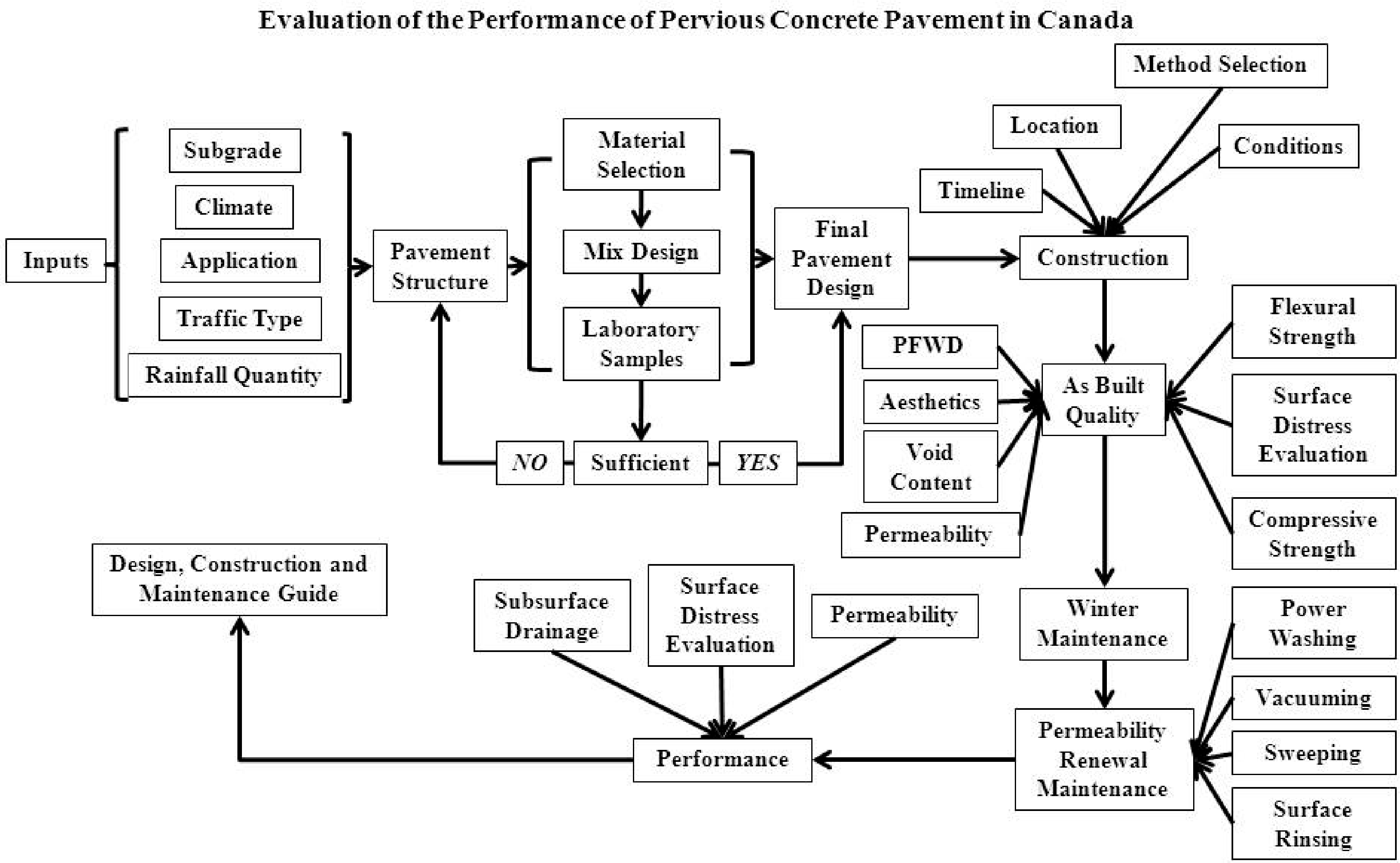


Figure 1-4: Flow Chart of Research Scope

condition that allows for the pavement to be functional for the user. At the end of the design life it is likely that distresses will have developed on the surface. However, the extent of these distresses should not prohibit the user from easily using the pavement.

The findings of this research are anticipated to be immediately applicable at the industry level. In order to provide results that are useful in industry following their analysis, it was paramount to use test methods throughout the research which were current and state of the practice. This was the case in all laboratory and field work with the intention of providing meaningful and realistic solutions for those involved in the use of pervious concrete pavement. The incorporation of both laboratory and field work was intended to provide a complete picture of the behavior of pervious concrete pavement. The controlled environment of the laboratory allowed for the effect of slight differences in conditions to be evaluated. In comparison, the field sites provided opportunities for full scale in-situ evaluations and monitoring.

As the project included several partners across the country, there were opportunities to construct various field sites in several locations. The benefit of this was that each field site could be an advancement, based on the findings at the previous. Additionally, each field site was unique and therefore provided different contributions to the overall project, while confirming others. A substantial benefit of having the opportunity to construct multiple field sites was the ability to evaluate different, full scale construction methods. Various aspects of a pavement life cycle can be modeled in the laboratory through testing. However, it is often not possible to fully depict how a construction process will occur in laboratory simulation. By constructing full scale field sites it was possible to monitor the efficiency, effectiveness and ease provided by different construction methods. This information is provided to industry members through this project. The construction of the field sites contributed data that combined with the performance data offered useful information as to the long term behavior of pervious concrete pavement in the Canadian climate.

The work carried out in this research was both practical and innovative. The monitoring and evaluation of effective design, construction and maintenance factors in pervious concrete pavement projects is critical to the growth of the industry. The innovative instrumentation that was included in many of the field sites provided data that could verify if the assumed behaviour of pervious concrete pavement was occurring. This information is not known to be included in other work. This knowledge will allow for pervious concrete pavement to be more effectively designed and understood for future projects.

The field sites and laboratory test results have indicated promising possibilities for pervious concrete pavement use in Canada. It is anticipated that the work presented in this thesis will be beneficial to a variety of industry members who are working on initial pervious concrete projects, including owners, contractors and consultants. In addition to those working with pervious concrete pavement on initial projects, the thesis also provides information that would be helpful in improving current practices. Finally, many of the conclusions and recommendations are not only applicable to Canada but other freeze-thaw climates and pervious concrete pavements in general.

1.7 Organization of Thesis

This thesis consists of eight chapters. A brief description of each chapter is included below.

Chapter One: An introduction to the topic of pervious concrete pavement as well as the scope, objectives and hypotheses of this work.

Chapter Two: A compilation of the knowledge available in literature related to this research is presented. This chapter discusses the state of the practice in terms of pervious concrete pavement and demonstrates the need for the research that was performed in this project.

Chapter Three: The sources of data for this project are described in this chapter. This includes a description of the field sites, instrumentation and testing that was performed in both the laboratory and field.

Chapter Four: The planning and design aspects of a pervious concrete pavement application are discussed in this chapter. The results and effectiveness of these phases of the field sites in this research are evaluated.

Chapter 5: The construction methods used at the field sites in this project are compared and the outcomes of each are discussed. Permeability renewal maintenance methods were used at the field sites and the results of the various methods are analyzed and compared in this chapter.

Chapter 6: The behaviour of pervious concrete pavement in the laboratory and at the field sites that could be measured or visually evaluated on the surface is included in this chapter. The results include data from regions with regular freeze-thaw cycles and those that have fewer freeze-thaw cycles. Data and information gathered from the laboratory samples and field sites was used together to assess the response of pervious concrete pavement to freeze-thaw cycles and activities associated with a freeze-thaw climate.

Chapter 7: The movement of moisture through the pavement structure of the pervious concrete field sites was monitored with instrumentation installed during construction. This chapter compares the subsurface findings from the three instrumented field sites.

Chapter 8: The conclusions drawn throughout the analysis of data in this project are presented in this chapter. Recommendations for future research related to the use of pervious concrete pavement in freeze-thaw climates, specifically Canada, are presented.

2.0 PERVIOUS CONCRETE PAVEMENT IN COLD CLIMATES

Canada offers large volumes of several natural resources, such as having 9 to 20 percent of the world's fresh water, depending on the definition(CBC, 2004). The Canadian population is not high in comparison to other countries, however in 2006, approximately 80% lived in urban areas and 98% lived in southern Canada and all relied on the surrounding systems to provide water (Statistics Canada, 2009; Statistics Canada, 2010). Canadians rank only second behind Americans in the most water consumption daily in the world, 350 litre per person(CBC, 2004). Figure 2-1 shows the decline in water resources in Canada since 1971. The urban regions need to make sustainable decisions in order to maintain the resources that are needed to support the residences locally. Initiatives such as creating sustainable communities are a step towards preserving resources for future generations in Canada.

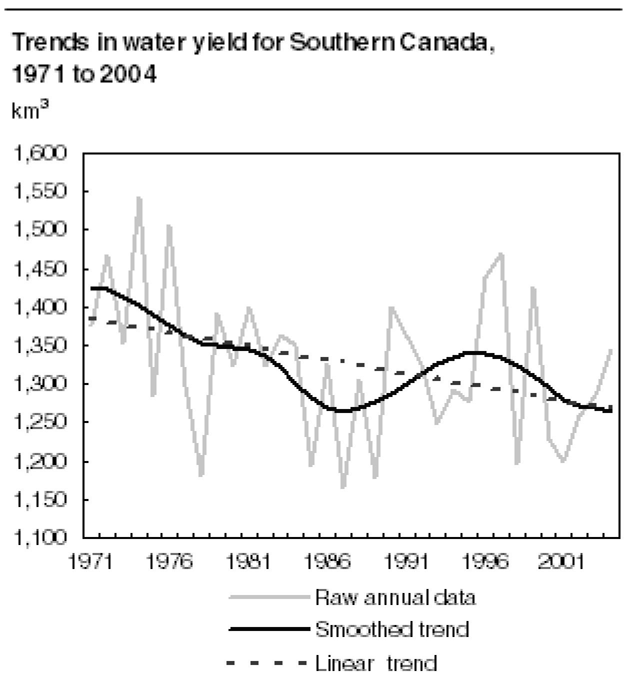


Figure 2-1: Trends in Water Yield in Southern Canada (Statistics Canada, 2010)

It is possible to implement sustainable practices within one type of infrastructure or in one industry, however, Engle-Yan describes that sustainable communities are only fully achieved when there is integration of various types of industry, infrastructure and practices

(Engle-Yan, Kennedy, Saiz, & Pressnail, 2005). Pervious concrete pavement lends itself to integration of industries naturally as it encompasses two types of infrastructure: pavement and stormwater management. The ability for pervious concrete pavement to be used as an effective stormwater management alternative is discussed in this chapter. The current academic knowledge and industry practices related to the use of pervious concrete pavement in cold weather climates are presented in this chapter. Where possible, cold weather climate practices were referenced from Canadian experiences. However, at this time substantially more literature has been generated from work on pervious concrete pavement in the cold weather regions of the United States.

2.1 Stormwater Management and Pervious Concrete

Many stormwater management guidelines throughout Canada reference permeable pavement as a method and often BMP for stormwater management. These references include the City of Hamilton, Toronto and Region Conservation,Credit Valley Conservation, and Niagara Peninsula Conservation Authority (Aquafor Beech Limited, 2007; Toronto and Region Conservation, 2010; AECOM, 2010). The inclusion of pervious concrete pavement in a development can offer four benefits to the stormwater management in the area (Hood, Clausen, & Warner, 2007; Guillette, 2010):

- Peak flow (reducing peak size);
- Volume (water into ground);
- Hydrograph timing; and
- Duration.

When pervious concrete pavement is used, the peak flow during storm events is less than it would be for an impermeable pavement. Since water moves through at least a portion of the pavement structure it is slowed down by having to negotiate a path through the voids (Hood, Clausen, & Warner, 2007). Depending on the site specifics all of the water can drain into the groundwater or a pipe can be included in the bottom of the aggregate storage layer to accommodate some drainage. In some scenarios a pipe is also included at the top of the aggregate storage layer to handle drainage.

The volume of water flowing into a storm outlet is reduced with the use of pervious concrete pavement. The volume for a specific storm is reduced when at least a portion of a rain event is permitted to infiltrate into the groundwater.

The hydrograph timing is shifted when pervious concrete pavements are used in comparison to conventional, impermeable pavements. The peak flow is less and occurs later than it would with impermeable pavement. The lag time is dependent on the size of storm but can be 45 minutes to 145 minutes (Fassman & Blackbourn, 2010).

The duration of drainage related to a storm event when pervious concrete pavement is used is decreased as less water needs to be drained.

The benefits noted above have led to pervious concrete pavement being recognized as a stormwater management Best Management Practice (BMP).

The United States Green Building Councils Leadership in Energy and Environmental Design (LEED) Green Building Rating System has assigned a point to the development of a sustainable site. The use of pervious concrete pavement is considered a step towards a sustainable site in the LEED program. In addition to the sustainable site credit, the use of pervious concrete maybe eligible to receive the same LEED credits as conventional concrete which are: reduction of heat island effect; use of recycled material; and use of regional materials (Tennis, Leming, & Akers, 2004).

The Ontario Ministry of Transportation (MTO) has developed a program called GreenPave which allows owners to compare the sustainability of different project alternatives. Within GreenPave there are credits awarded for permeable pavements which pervious concrete pavement would fall within. There are also additional credits directed towards concrete pavement that pervious concrete pavement maybe eligible for (MTO MERO, 2012).

As a development is being considered for an area, environmental management plans and stormwater management plans are produced. These plans reflect the goals of the stakeholders in the project. This stage of a project allows for comparison of the stormwater management practices that could be appropriate as well as where they should be located. This planning stage should identify techniques that will maintain available natural areas and minimize the generation of stormwater runoff (MOE, 2003). The inclusion of pervious concrete pavement in this stage can ensure that it is used in effective locations for both pavement performance and stormwater runoff reduction.

Stormwater management treatment trains are part of a stormwater management plan and the goals for the area. The steps of a treatment train are shown in Figure 2-2, as well as the areas where pervious concrete pavement can be incorporated effectively.

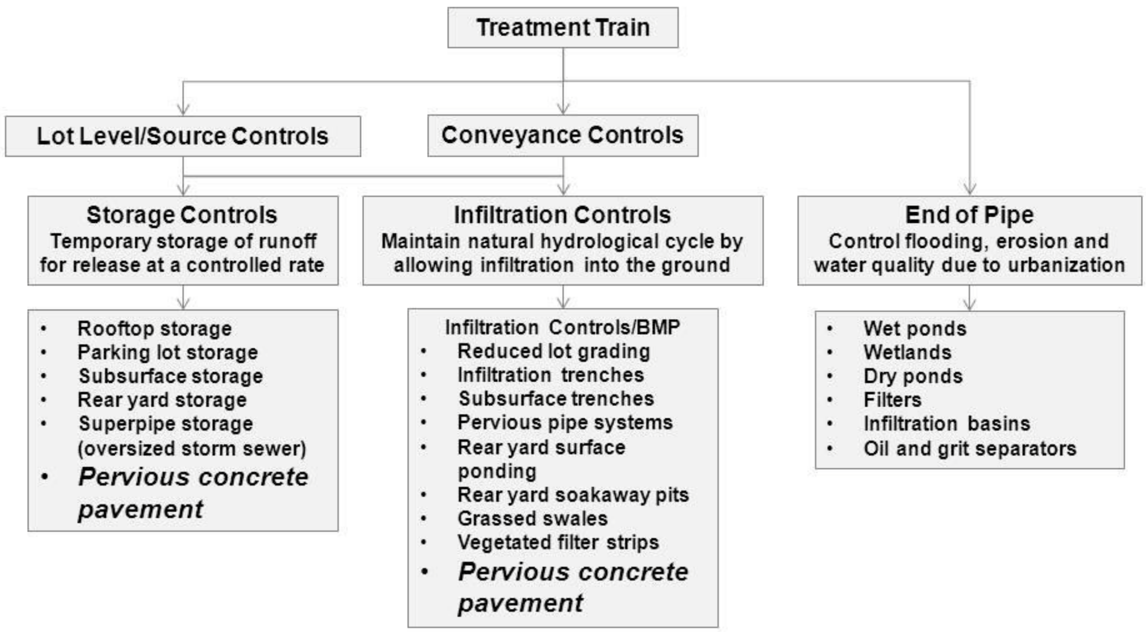


Figure 2-2: Treatment Train for Stormwater and the Inclusion of Pervious Concrete Pavement (MOE, 2003; Walker, 2009)

Figure 2-2 identifies that pervious concrete can be included in the development as either a storage control or infiltration control. The two functions that pervious concrete pavement offers: a parking surface; and a drainage path make it useful in urban settings. Infrastructure used by cars and otherwise referred to as “car habitats” are noted as being areas where reductions would be beneficial in reducing stormwater runoff generation. Reductions to car habitats such as residential street width, minimizing the use of cul-de-sacs, smaller parking lots and increased efficiency in parking lot designs are recommended by the Ontario Ministry of the Environment (MOE, 2003). Regular inclusion of pervious concrete pavement in car habitats would provide additional reductions in stormwater runoff volumes.

2.2 Pervious Concrete Pavement

Pervious concrete pavement assists in optimizing land use during new development and rehabilitation of existing development. The amount of land and resources that would generally be required to enhance stormwater management systems for new developments are decreased when pervious concrete is incorporated (Paine, 1992). Pervious concrete pavement is a sustainable paving material, as it is a LID and therefore does not alter the natural hydrological cycle. The natural flow of water through the pavement structure and into the groundwater ensures that surrounding vegetation receives an adequate water supply, as would occur in the natural setting (AECOM, 2010).

Recently the interest to use pervious concrete pavement in cold weather climates has increased. Pervious concrete pavement has been used in warm climates, such as the southern states of the United States for a long period of time (Huffman, 2007). Prior to the time that this work was initiated there had been limited research and use of pervious concrete pavement in Canada. Currently, there are many provinces, cities, municipalities and industry organizations throughout Canada that have expressed interest in using pervious concrete pavement. Some of these organizations have placed trial sections and now have developed specifications for future projects. Other groups are currently working on constructing their initial test site projects.

Currently there are a few test procedures specifically for pervious concrete. In some scenarios conventional concrete testing methods can be used to compare pervious concrete mixes however these should not be used to specifically describe the characteristics of a pervious concrete pavement mix. The test procedures currently available and specific to pervious concrete are the following:

- ASTM C1688
- OPSS 356

Within OPSS 356 it is required that testing be performed for compressive strength and void content (OPSS, 2010; ASTM, 2010). Both of these tests are to be performed on core samples. Core samples are required as they represent the pervious concrete pavement in place and there is currently not a standard procedure for casting pervious concrete samples that consistently produce the material placed in the field applications. Research has been done to demonstrate that alternatives to conventional methods for sample preparation are needed (Rizvi, Tighe, Henderson, & Norris, 2009).

Laboratory prepared samples can be used for relative comparison in ranking various mixes; however, should not be used to predict field site conditions. Samples cast in the laboratory, all 2:1 in height to diameter ratio, with varying diameter sizes, 75 mm to 150 mm, show that the measured characteristics are dependent on the sample size. Permeability and compressive strength both showed a wide range of results when samples of different sizes were tested for the same mix (McCain & Dewoolkar, 2010).

Laboratory cast mixes have shown that samples with less than 20% void content resulted in compressive strengths of 20 Mpa. These samples generally had a permeability of 0.2 cm/sec(Wu, Shu, Dong, Shrum, Jared, & Wu, 2010).

While pervious concrete pavement has been in use in warm climates for decades use in freeze-thaw climates such as Canada and the northern United States has been limited (Huffman, 2007). The performance and characteristics of pervious concrete pavement in warm climates has been adequate with limited surface distress development and sufficient surface drainage while carrying out minimal to no maintenance (Chopra, Wanielista, Ballock, & Spence, 2007). The knowledge that is available related to the use of pervious concrete pavement in warm climates is helpful in predicting the behaviour of this pavement in the Canadian freeze-thaw climate. It is anticipated that distresses common in warm climates will occur in freeze-thaw climates as well as others caused by the changes in temperature. The permeability performance and clogging characteristics of pervious concrete pavement in warm climates can also be used in initial estimates of performance in Canada.

2.2.1 Cold Climate Experience

The use of pervious concrete pavement in Canada and similar climates has been cautioned by the concern that if the voids become saturated and then freeze, severe distresses such as spalling, cracking and ravelling could occur (NRMCA, Freeze-Thaw Resistance of Pervious Concrete, 2004). The following suggestions have been proposed in literature to avoid failures of pervious concrete pavement when used in freeze-thaw climates (NRMCA, Freeze-Thaw

Resistance of Pervious Concrete, 2004; Wang, Schaefer, Suleiman, & Kevern, 2006; Tennis, Leming, & Akers, 2004).:

- Use of air entrainment in the mix design;
- Use of latex in the mix design;
- Use of fine aggregate in the mix design;
- Use of a high void content granular reservoir base layer (clear stone); and
- Ensuring that the groundwater table is a minimum of 1 m from the bottom of the pervious concrete layer.

In addition to the potential for failure due to freeze-thaw cycling, there is also a concern that winter maintenance will damage the pavement surface and clog the voids. Information in the literature suggests that snow should be removed with a plastic snow plow blade versus a conventional, metal highway grade snow plow. There is data indicating that the removal of snow with a metal blade can in some cases cause abrasion to the pavement surface or accelerate and increase ravelling (Henderson, Banfield, Bergsma, & Malleck, 2008). Regardless of the snow removal technique both in cold and warm climates it is common for ravelling to occur on the surface immediately following construction (Tennis, Leming, & Akers, 2004). After this time, ravelling often decreases and is minimal. This trend in surface distress development was shown at the pervious concrete test areas at MnRoad in Minnesota, USA (Rohne & Izevbekhai, 2009).

The recommendations for developing and constructing quality pervious concrete pavement in warm climates are applicable to cold weather climates. In addition to the practices used in warm climates however, it is necessary to develop and construct pervious concrete pavement that is durable in freeze-thaw conditions. The following sections will discuss the current findings regarding achieving quality pervious concrete pavement in cold weather climates. The current literature has been presented based on the following topics: mix design; pavement structure; construction; and maintenance.

2.2.2 Mix Design

A pervious concrete mix design needs to provide a void structure that allows water to drain from the surface. It also has to provide a pavement with adequate durability for the intended users, to minimize the development of surface distresses. The durability of a pervious concrete mix can be jeopardized if there are too many voids and not enough strength is present. The ability to continue to perform under exposure to freeze-thaw cycling and regular traffic is critical in the Canadian climate. The traffic that is in consideration in this work includes traffic that is present in low speed and low volume applications.

A pervious concrete mix design generally includes coarse aggregate, cement and water. Fine aggregate can be included and the quantity is limited when it is included. Various supplementary cementitious materials, admixtures and fibres are also included in some cases (Kevern J. , Wang, Suleiman, & Schaefer, 2005). The type and size of coarse aggregate used in a pervious concrete mix can alter the properties of the pavement.

Wang et al. carried out a project to develop a functional and durable pervious concrete mix design for cold climates (Wang, Schaefer, Suleiman, & Kevern, 2006). The conclusion of the study was that the following criteria should be met for a performance based mix design in a cold weather climate:

- Permeability exceeding 0.1 cm/sec;
- Compressive strength at 28 days exceeding 20 MPa; and
- ASTM C 666 testing results of less than 5% weight loss after 300 freeze-thaw cycles.

2.2.2.1 Aggregate

A comparison of two pervious concrete mixes in the laboratory demonstrated that the size of aggregate can result in different void structures developing and therefore different characteristics. Pervious concrete mixes were prepared in the laboratory, one with smaller sized aggregate, 8 mm – 10 mm and the other with larger, 16 mm – 20 mm. The aggregate used in both mixes was gravel. Using x-ray scanning it was identified that the smaller aggregate mix had a higher percentage of voids and mortar than the larger aggregate mix. The hydraulic permeability was found to be lower in the smaller aggregate mix when measured, than in the larger aggregate mix, even though the smaller aggregate mix had more voids. The x-ray scans showed that although there were more voids in the smaller aggregate mix, the voids were smaller and more distributed throughout the sample than in the larger aggregate mix. The larger aggregate mix had larger voids and visibly small contact points between aggregates. Smaller aggregates have more surface area when a volume of space is considered than larger aggregates. Therefore there is more area for mortar coverage and opportunity for contact and bond development (Kringos, Vassilikou, Kotsovos, & Scarpas, 2011).

Pervious concrete samples with the same void content were identified to provide different permeability rates depending on the characteristics of the voids themselves. Using binary images, x-ray microtomography and computational analysis, the characteristics of the voids in three laboratory prepared pervious concrete mixes were compared. Each mix contained a different size of aggregate. Although the void content of the three mixes was similar, the permeability increased as the aggregate size increased (2.36 mm to 12.7 mm). The images and computational analysis showed that when larger aggregate was used in the pervious concrete mix the void size increased. The permeability results suggest that larger sized voids lead to higher permeability and the void content does not necessarily describe this (Neithalath, Bentz, & Sumanasooriya, May 2010).

The permeability of a pervious concrete pavement structure is not necessarily the determining factor as to whether the pavement performs adequately. Permeability rates have been found to increase exponentially as the void content of a pervious concrete mix increases. It is important to find a balance between void content, durability and permeability rate in a pervious concrete mix design (Schaefer, Wang, Suleiman, & Kevern, 2006; Delatte & Cleary, 2006).

2.2.2.2 Cement Content

The cement content of a pervious concrete mix was evaluated in terms of its effect on freeze-thaw durability with samples that were prepared in the laboratory. With a higher cement content, the samples showed better performance in freeze-thaw cycling. This freeze-thaw cycling was performed while the samples were continually submerged. The inclusion of more cement is predicted to lead to a thicker paste and therefore more bond development (Yang Z. , 2011). Ranges of water to cement ratios were evaluated in laboratory prepared pervious concrete mixes. A water cement ratio of 0.25 showed less freeze-thaw durability in saturated conditions than a ratio of 0.35. It is suspected that below a water cement ratio of 0.35 there may have been excessive drying and shrinkage cracking occurred (Yang Z. , 2011).

Mixes prepared in the laboratory were compared for compressive strength and permeability. Samples with a higher water cement ratio, 0.33, showed the highest compressive strength results. The mix with a water cement ratio of 0.25 had the lowest compressive strength and a mix with a water cement ratio of 0.29 had compressive strength results in between. A clear relationship between density of the sample and water cement ratio and compressive strength result was present. Samples within a specific mix with higher densities had higher compressive strength results. The permeability test results did not show as clear a correlation, however a trend was present. The higher water cement ratio mixes consistently had lower permeability rates (McCain & Dewoolkar, 2010).

2.2.2.3 Admixtures and Additional Cementitious Materials

Laboratory prepared samples with various cement contents, silica fume, polypropylene fibres or both silica fume and fibres were exposed to saturated freeze-thaw cycling to evaluate the effect of each of these mix variations on performance. The addition of silica fume at 5 %, fibres at 1.78 kg/m^3 or both 5 % silica fume and 1.78 kg/m^3 fibres improved the freeze-thaw durability in each scenario. The best freeze-thaw durability was experienced in the mix with the highest cement content, 5 % silica fume and 1.78 kg/m^3 fibres. The presence of silica fume alone in the mix increased the freeze-thaw durability and minimal improvements were noted with high cement content. The inclusion of fibres, silica fume or both increased the freeze-thaw durability from the control mix for all cement contents that were evaluated (Yang Z. , 2011).

The effect of air entrainment and a high range water reducer on compressive strength and permeability were evaluated on laboratory prepared samples. The largest compressive strength results were achieved when no air entrainment was included, only high range water reducer. The mix containing only air entrainment and no high range water reducer had the lowest compressive strength. The compressive strength of all three variations was deemed to be adequate, greater than 17 MPa. Permeability results were similar for all three mix variations. The mix with only high range water reducer had some results that were lower than the other two mixes (McCain & Dewoolkar, 2010).

Two pervious concrete mixes were prepared in the laboratory and their performance during exposure to de-icing chemicals was evaluated. The two mixes that were compared were the same except one contained a viscosity modifying agent and the other contained latex polymer. The pervious concrete was either submerged in de-icing chemicals or the de-icing chemicals were allowed to drain through the pervious concrete. Less mass loss was present in all samples when the de-icer was able to drain through the pervious concrete, in comparison to the samples being submerged. Samples including latex polymer had more mass loss than the mix with no latex polymer. The difference in compressive strength of the two mixes after freeze-thaw cycling was less apparent than the difference in mass loss of the samples (Cutler, Wang, Schaefer, & Kevern, 2010).

Latex and fibres were added to a pervious concrete mix in the laboratory. Four different mixes were prepared in the lab: control mix; control mix plus latex; control mix plus fibres; and control mix plus latex and fibres. Samples containing latex and those with latex and fibres had the lowest void content. The four mixes were prepared with #7 aggregate (nominal sieve size 12.5 mm to 4.75 mm) and again with #89 aggregate (nominal sieve size 9.5 mm to 1.18 mm). Larger void content results were measured for the mixes with #7 aggregate than those with #89. The permeability of the samples had a similar trend to the void content results. Samples with latex or fibre and latex had low permeability results. Samples containing #7 aggregate had higher permeability values than those containing #89 aggregate (Wu, Shu, Dong, Shrum, Jared, & Wu, 2010; ASTM, 2009).

Compressive strength and splitting tensile strength were measured on samples from 16 laboratory prepared mixes. The compressive strength and splitting tensile strength results showed the same trend. Samples containing latex and #89 aggregate had higher results than the other mix combinations. The addition of fibres to the mix did not change the results substantially. Samples with #7 aggregate consistently had lower results than those containing # 89 aggregate (Wu, Shu, Dong, Shrum, Jared, & Wu, 2010).

Laboratory testing was performed to assess the ravelling tendency of various pervious concrete mixes. The testing involved cycling samples for 300 cycles in an LA Abrasion machine, without the conventional steel balls. In all scenarios, samples containing #89 aggregate performed better than those with #7 aggregate. The addition of latex to the mixes improved the ravelling resistance the most. Including fibres in the mix also improved the performance, however not as much as latex (Wu, Shu, Dong, Shrum, Jared, & Wu, 2010).

The resistance of pervious concrete mixes to abrasion from traffic was evaluated in the laboratory on several mixes using as asphalt pavement analyzer. The results of all mix variations showed minimal aggregate loss from the vehicle tire scenario. Samples containing #89 aggregate performed better than those with #7 aggregate. Samples with granite had better results than those with limestone. The inclusion of latex in the mixes produced samples with the best resistance to vehicle wheel abrasion. The inclusion of fibres improved the mixes' performance however not as much as the inclusion of latex (Wu, Shu, Dong, Shrum, Jared, & Wu, 2010).

Test method ASTM C 666 is used for freeze-thaw testing of conventional concrete. In this test concrete samples are saturated and exposed to freeze-thaw cycles. This loading is not representative of pervious concrete pavement in the field. These conditions would exist only if the voids have minimal connectivity and the permeability rate of the pervious concrete is very low. ASTM C 666 is a severe test for pervious concrete however not representative of field performance. Wang et al. compared multiple mixes by carrying out ASTM C 666 testing. The mix containing 7% sand and river gravel performed the best, with only 2.1% weight loss at 300 cycles. A mix containing 7% sand, river gravel and fibers also performed well, as did the mix containing 7% sand, river gravel and a latex admixture [Wang, Schaefer, Kevern and Suleiman, 2006].

2.2.3 Pavement Structure

The pavement structures used for pervious concrete pavement must meet the demands of the two functional purposes that it is designed for. The functional purposes include providing a rigid pavement surface and a stormwater management solution. The general pavement structure used with pervious concrete pavements to achieve both functions is shown in Figure 2-3. Figure 2-3 shows the pervious concrete surface, a choker course, a granular storage base layer, a geotextile and the existing subgrade.

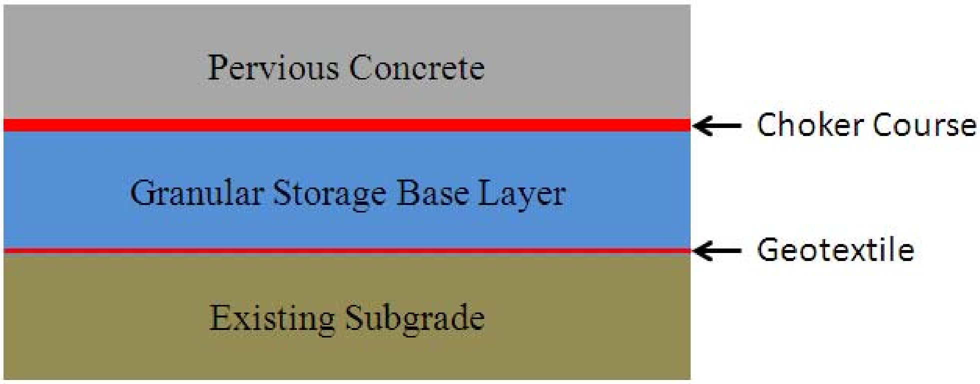


Figure 2-3: Pervious Concrete Pavement Structure

The pavement structure shown in Figure 2-3 can be altered for site specific requirements. The requirements for the pavement structure need to be considered and optimized for both purposes of the pavement. Both the choker course and geotextile are not used in all scenarios. The granular base layer is recommended in freeze-thaw climates in all scenarios (NRMCA, 2004). The granular storage base layer is intended to act as a reservoir, allowing water to accumulate and infiltrate into the existing subgrade and groundwater. In 2006 the typical pavement structure of pervious concrete pavement in the United States was 100 mm to 150 mm of pervious concrete, and up to 450 mm of permeable base on a permeable subgrade (Schaefer, Wang, Suleiman, & Kevern, 2006). The typical pavement structure for low volume pervious concrete applications in the State of Vermont is 150 mm pervious concrete, 50 mm of AASHTO #57 stone choker course, minimum 850 mm AASHTO #2 stone granular storage base layer and geotextile on the existing subgrade (McCain & Dewoolkar, 2010; AASHTO, 2008).

2.2.3.1 Subgrade

The characteristics of the subgrade beneath a pervious concrete pavement structure will determine whether water that drains through the structure can naturally infiltrate into the subgrade in an adequate amount of time or if a drainage system needs to be included. The National Ready-Mixed Concrete Association (NRMCA) suggests that subgrades with percolation rates of 12 mm/hour or more are suitable for pervious concrete pavement structures. Therefore, a subgrade with a percolation rate of less than 12 mm/hour would require water to be drained out of the pavement structure. The determination of the percolation rate of the subgrade is important in developing the design of the entire pavement structure (NRMCA, 2010).

Pervious concrete pavement structures can be used on soils with low permeability rates by incorporating a drainage system. The drainage system would include an underdrain in the reservoir layer of the pavement structure. Figure 2-4 shows an example of how an underdrain could be included. The inclusion of the underdrain would provide drainage of the water that was not able to dissipate into the existing subgrade or would be too slow. The positive benefits of using pervious concrete as a stormwater management tool would still exist as runoff would be mitigated and drainage from the underdrain would not include substantial peaks. Figure 2-4 shows a simple example of the inclusion of an underdrain, there are several other possible structure layouts (ACPA, 2009).

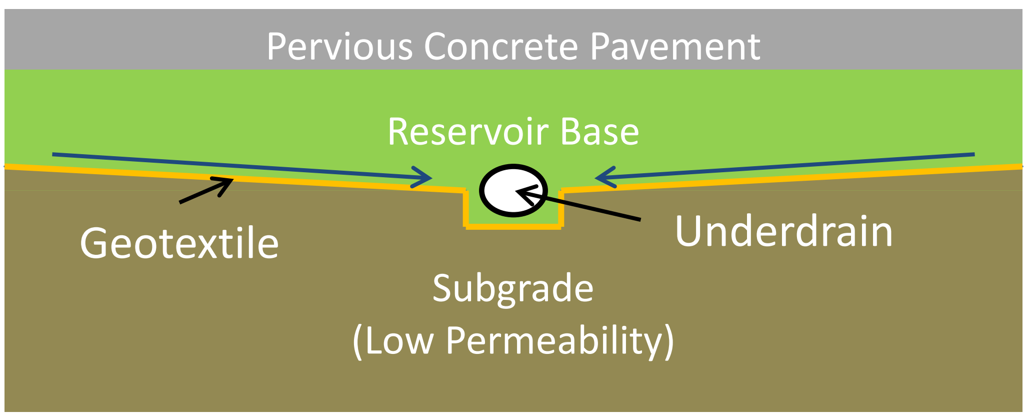


Figure 2-4: Underdrain Pipe for Low Permeability Subgrade Soil

As Figure 2-4 shows, the underdrain pipe would be below the base storage layer and positive drainage to the underdrain should be provided. The storage base layer would continue to provide a reservoir as water drained into the underdrain (ACPA, 2009).

A subgrade with good drainage characteristics is ideal. However, it has been demonstrated that subgrades with limited drainage capabilities can be improved and become effective in pervious concrete pavement structures. Pervious concrete test plots were constructed on predominantly clay soil to evaluate if different enhancements to the soil were effective in increasing the infiltration capabilities. Four scenarios were compared: control;

boreholes; ripped; and trenches. The details of the methods used in each of the three modifications are available in Tyner et al's 2009 (Tyner, Wright, & Dobbs, 2009). The average infiltration rate from the stone base layer was the following for each of the clay soil scenarios: control 8 mm/day; borehole 46 mm/day; ripped 100 mm/day; and trenched 258 mm/day. This work indicates that it is possible to use pervious concrete pavement on subgrades with low infiltration rates with success (Tyner, Wright, & Dobbs, 2009).

The strength of the subgrade is important in determining the support that will be required by the base layers in the pavement structure. The K value (modulus of subgrade reaction) of a subgrade material describes the support it will provide to a pavement structure. When possible a measurement should be taken from the natural subgrade that the pavement structure will be constructed on as it will provide the most reliable information for the design. Relationships are available between K values, CBR results and R values (Tennis, Leming, & Akers, 2004).

A geotextile should be included between the base layer and the subgrade. The base layer is intended to hold moisture as a storage area continually throughout the year. The constant moisture in this area is anticipated to lead to the top of the existing subgrade being in a moist condition as well. Without the inclusion of a geotextile, it is likely that the fines from the subgrade soil will migrate into the storage layer, therefore reducing the amount of storage capacity available over time and reducing soil support capacity (NRMCA, 2005).

2.2.3.2 Granular Base

The granular base layer of a pervious concrete pavement structure are intended to provide structural support to the pavement, as would be the case in a conventional concrete pavement. Additionally, base layers need to be at least free draining and also incorporate an area where storage of water can occur in order for a pervious concrete pavement structure to behave as a stormwater management solution. The suggested material for the granular base layer of a pervious concrete pavement structure is an open-graded clean stone with 20% - 40% void space or porosity (PI, 2010; ACPA, 2006; NRMCA, Hydrological Design Considerations, 2010). Table 2-1 provides gradations recommended for the granular base layer by different users of pervious concrete throughout North America.

Table 2-1: Granular Gradations for Storage Base Layer (AASHTO, 2008)

Sieve Size	Types of Materials					
	AASHTO No. 2	**AASHTO No. 3 (30% Porosity)**	**AASHTO No. 4 (30% Porosity)**	**AASHTO No. 57 (40% Porosity)**	**Granular O (OPSS 1010)**	**AASHTO No. 67 (40% Porosity)**
	Percent Passing Sieve (%)					
75mm	100					
63mm	90-100	100				
50mm	35-70	90-100	100			
37.5mm	0-15	35-70	90 – 100	100	100	
25.0mm (26.5mm*)		0-15	20 – 55	95 – 100	95-100	100
19.0mm	0-5		0 – 15		80-95	90 – 100
12.5mm (13.2mm*)		0-5		25 – 60	60-80	
9.5mm			0 - 5		50-70	20 – 55
4.75mm				0 – 10	20-45	0 – 10
2.36mm				0 – 5		0 – 5
1.18mm					0-15	
300μm						
150 μm						
75 μm					0-5	
* Used in Granular O gradation only						

NRMCA suggests the use of AASHTO No. 67 for the base layer of pervious concrete pavement structures, noting that a porosity of 40% can be expected. The use of AASHTO No. 3 or AASHTO No. 4 stone is suggested by the Urban Drainage and Flood Control District (UDFCD) in Colorado for the base layer of pervious concrete pavement structures and anticipates 30% porosity (UDFCD, 2008). The New Jersey Stormwater Best Management Practice Manual includes AASHTO No. 2 as an appropriate gradation for the base layer (Blick, Kelly, Skupien, 2004). The Iowa Stormwater Management Manual recommends the use of AASHTO No. 57 for the base layer and a porosity of close to 40% is common (IT, 2009). The Ready Mixed Concrete Association of Ontario (RMCAO) suggests Granular O or a 19 mm clear stone for the base layer. Table 2-1 shows that Granular O has more material passing the 4.75 mm sieve than the other suggested materials. It is a free draining material used as a free draining granular base in a pavement structure and its water storage capacity maybe limited (RMCAO, 2009; OPSS, 2003).

The choker course is included in a pervious concrete pavement structure to provide a construction platform. The clear stone material in the reservoir base layer is generally not stable enough for loading from construction equipment and concrete trucks. To create a surface that provides support for construction equipment a choker layer can be placed on top of the clear stone reservoir base. The choker layer should be free draining but have less voids than the reservoir layer. The intent is for water to drain through the thin choker course and be stored in the voids in the reservoir layer (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008)

2.2.3.3 Pervious Concrete Surface

The top layer, the pervious concrete layer is designed to handle the load of the anticipated traffic. Using the characteristics of the pervious concrete mix, scenarios can be run in concrete pavement design programs to determine the required thickness. The Ready Mixed Concrete Association of Ontario suggests a minimum pervious concrete layer thickness of 150 mm (RMCAO, 2009). Delatte identified that 150 mm of pervious concrete pavement is roughly equivalent to 100 mm of conventional concrete pavement that would be used in a parking lot application (Delatte & Cleary, 2006). This relationship was developed using Streetpave concrete pavement design fatigue curves and assuming that pervious concrete had a flexural strength of 2.1 MPa to 2.8 MPa.

2.2.4 Construction

The construction method used with pervious concrete pavement is an important factor in the performance of the pavement in the long-term. The NRMCA provides training and certification for contractors in the appropriate methods to handle and construct pervious concrete pavement. The OPSS 356 requires that at least one person from the contractor who is constructing the pervious concrete pavement be certified by the NRMCA Pervious Concrete Contractor Certification program (OPSS, 2010). This is a common requirement by owners who are having pervious concrete pavement constructed as it is unique from conventional concrete pavement (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008).

2.2.4.1 Site Preparation and Material Placement

Successful construction of pervious concrete pavement starts with the subgrade which should be even and uniform. Compaction of the subgrade should be to 90% to 95% of the Standard Proctor Maximum Dry Density (SPMDD) in order to provide consistent support. Further compaction of the subgrade surface will generally lead to a reduction in percolation ability which is not desired for a pervious concrete pavement structure. Following compaction of the subgrade the geotextile can be placed on the surface (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008; NRMCA, 2010)

The base layer material and the construction method need to be compatible. A choker course can be included to provide a construction platform while maintaining drainage through the pavement structure (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008) The granular base layers should be rolled.

Kevern et al summarized the common pervious concrete placement method in 2006. Pervious concrete is placed from the chute of a concrete truck directly on to the base material. Due to the stiff nature of pervious concrete it is often necessary to pull the material down the chute and distribute it throughout the area being paved. Movement of the pervious concrete should be limited when possible. Forms are used and the pervious is filled 20 mm to 25 mm higher than the desired final thickness, using shims on top of the forms. A vibratory screed is then moved across the top of the shims, striking off the pervious concrete. The shims are then removed and a manual weighted roller or a hydraulic vibratory roller is rolled on the forms to compact the pervious concrete. This method produces a level surface (Kevern J. , Wang, Suleiman, & Schaefer, 2006).

Pervious concrete was placed in an area of Iowa that did not have access to a weighted roller or vibratory roller and an alternate system was developed. Pervious concrete was placed to the top of 20 mm shims (on top of forms). The pervious concrete was striked off to the top of the shims using a hand held vibratory screed. The shims were then removed and sheets of plywood were placed on the pervious concrete surface. A vibratory plate compactor was used on top of the plywood to compact the pervious concrete. This site was evaluated after one winter and no ravelling was evident (Kevern J. , Wang, Suleiman, & Schaefer, 2006).

The level of compaction during the construction of pervious concrete pavement is anticipated to have an effect on the durability of the pavement when exposed to freeze-thaw cycling. In a laboratory experiment, compaction of cast cylinder samples was evaluated at two levels using a vibratory table. The cylindrical samples were freeze-thaw cycled and the failure method was observed. Testing following ASTM C666, Procedure A, was carried out and included the samples being saturated. Failure was deemed to have occurred when specimens lost 15% mass and testing was complete after 300 cycles. Both of the specimen that were less compacted and those with more compaction failed before the 300 cycles were completed. Specimens that had received more compaction failed at 153 and 196 cycles and those having had less compaction failed at 110 cycles in both cases (Suleiman, Kevern, Schaefer, & Wang, 2008). The failure of all samples before the 300 cycles is not an indication of general poor performance of pervious concrete as this test method is not representative of a pavement in good condition in the field. A pervious concrete pavement in good condition would drain the water quickly and not be saturated. This test did provide a method for comparing the resistance to deterioration of the mixes however. A mix exhibiting more general strength is anticipated to be able to withstand pressure of expanding, freezing water for a longer time. In a true application these pressures or forces may come from traffic rather than freezing water. The specimens that had experienced more compaction demonstrated more general strength (Suleiman, Kevern, Schaefer, & Wang, 2008).

A roller or roller screed can be used in the construction of pervious concrete however it has been noted that the surface may not be as consistent as when mechanical equipment such as a paver is used. Pervious concrete pavement was placed with a roller screed and efforts were made to ensure consistency was achieved across the site by carrying out the following steps: overfilling concrete in forms; rolling a short 1 m to 3 m section then returning the roller to the initial location; adding a thin layer of pervious concrete to the previously rolled surface; and rolling the surface again. The intent of the process was to eliminate areas with visibly low density or an insufficient amount of material. A surface evaluation at seven months of age showed that low density areas were visible in many portions of the pervious concrete pavement site even after the efforts to avoid this (Schaefer, Kevern, Izevbekhai, Wang, Culter, & Wiegand, 2010).

2.2.4.2 Curing

Curing of pervious concrete pavement is crucial in achieving a quality product. Curing generally includes covering the pervious concrete pavement with a plastic sheet that is 4 mm to 6 mm in thickness (ORMCA, 2009; Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008). The curing should begin as soon as practically possible during construction and it is generally required that the plastic sheet be on the pervious concrete surface within 20 minutes of placement from the truck (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008). In Ohio it is required that the plastic sheet be placed on the surface within 10 minutes of placing the pervious concrete (ORMCA, 2009). Some specifications require that the pervious concrete surface be sprayed with a curing compound prior to placing the plastic sheeting for curing (ORMCA, 2009; Kevern J. , Wang, Suleiman, & Schaefer, 2006; Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008)

The effect of curing on freeze-thaw durability resistance was evaluated in a laboratory study. Samples were cast in the laboratory and then covered in a plastic sheet for seven days. After the seven days some samples were placed in a 22°C water bath for 60 days and the remainder were placed in an environmental chamber for 60 days, at 22°C to continue curing. Freeze-thaw testing involving the samples being submerged in tap water showed the same performance for a variety of mixes. The specimens cured in water for 60 days experienced more freeze-thaw cycles than those cured in air, before failing. The difference in exposure before failure ranged from 10 to 95 freeze-thaw cycles (Yang Z. , 2011).

Samples exposed to freeze-thaw cycling while being continually submerged were compared to those that rotated between being submerged and being in air. Following laboratory casting of the samples some had been cured in air and others in a water bath as noted above. The rotation between air and submersion cycling extended the failure time of the air cured samples by approximately 10 cycles. The samples cured in water were exposed to 85 to 95 more freeze-thaw cycles before failure than the samples in air. This was the case for samples submerged and those rotated in and out of water during freeze-thaw cycling (Yang Z. , 2011).

2.2.4.3 Joints

Pervious concrete pavements can include joints similar to a conventional concrete pavement to control cracking. The joints should be one quarter of the thickness of the slab (Tennis, Leming, & Akers, 2004). The joint spacing can be larger than in conventional concrete as pervious concrete is considered to shrink less than conventional concrete. Joints spacing of 6 m is recommended however wider spacings, up to 13 m, have been used without uncontrolled cracking appearing. It is recommended that joints meet those of adjacent pavements to prevent uncontrolled cracking occurring (Tennis, Leming, & Akers, 2004).

Joints can be constructed using a joint former, also known as a pizza cutter. When joints are formed in this manner it has been found that long term performance is better if the joint former is passed across the fresh pervious concrete only once. It is intuitive to bring the joint former back to its initial position, given the mechanics of the tool. However, more than one pass provides the opportunity for the joint former to take a different path, thus disturbing aggregate and promoting ravelling (Schaefer, Kevern, Izevbekhai, Wang, Culter, & Wiegand, 2010).

Joint construction using a joint former is often recommended over saw cutting. However, saw cutting is generally included as another option. The use of a joint former can lead to an opportunity for ravelling as demonstrated in the previously discussed project. Kevern suggests saw cutting joints or forming them using a joint former. If joints are saw cut, this should be carried out as early as possible after the pervious concrete has become hard enough that it will not ravell, this is usually after 24 hours (Kevern J. , 2010; Tennis, Leming, & Akers, 2004).

The Ohio Ready Mixed Concrete Association’s (ORMCA) Specifier’s Guide for Pervious Concrete Pavement suggests the use of a joint former for forming joints into plastic pervious concrete or saw cutting joints into hardened pervious concrete pavement. Caution is included regarding using a joint former as de-consolidation of the adjacent 50 mm of pervious concrete pavement can occur if the joint former is not used properly and cleaned. When saw cutting is performed it should be done after sufficient hardening of the pervious concrete pavement has occurred such that ravelling will not occur. Experience in Ohio has demonstrated that saw cutting of joints can be performed after seven days of curing with minimal to no cracking developing(ORMCA, 2009).

Pervious concrete pavements do not always include joints. Without the inclusion of joints, cracking will develop. The appearance of pervious concrete makes the cracks typically unnoticeable to the general public. Development of shrinkage cracks due to not including joints in the pervious concrete pavement surface is considered to not affect the structural integrity of the pavement (Tennis, Leming, & Akers, 2004).

2.2.4.4 Opening to Traffic

Guidelines for constructing pervious concrete pavement routinely reference that no vehicle traffic should be allowed on the pavement until seven days after construction (Kevern J. , 2010; Tennis, Leming, & Akers, 2004). The Ontario Ministry of Transportation (MTO) currently requires that pervious concrete not be opened to traffic until a core with a compressive strength of 15 MPa is attained from the site. The minimum curing requirement is seven days. The OPSS 356 specifications do not allow vehicles weighing 20,000 kg or more on pervious concrete at any time (OPSS, 2010).

The ORMCA requires that no truck traffic use a pervious concrete pavement until 14 days after construction (ORMCA, 2009). The Colorado Ready Mixed Concrete Association (CRMCA) requires that pervious concrete pavement not be opened to the traffic until the pavement has reached the equivalent maturity that would have been experienced after 14 days of curing at 21°C at 95% relative humidity (Bush, Cawley, Durham, MacKenzie, Rottman, & Thomas, 2008).

2.2.5 Surface Distresses

The following distresses have been noticed in pervious concrete pavements:

- Slab ravelling
- Joint ravelling
- Cracking
- Sealing

A description of each distress is included in the following sections, as well as the experiences in industry with these distresses. As the list above indicates, the distresses that have been experienced in pervious concrete pavements have been limited in terms of various types. The low speed and low volume traffic is considered to be one reason for there having been a small number of distresses noticed in pervious concrete pavement. The distresses that have been noticed are thought to be due to materials, construction methods, traffic and environmental conditions.

2.2.5.1 Slab Ravelling

Ravelling is one of the most noted distresses that develop in pervious concrete pavements. It is common for a small amount of ravelling to occur following construction and this is referenced in literature (Kevern J. , 2010). It is anticipated that a few aggregate pieces will be loosely attached to the pervious concrete following construction. These aggregate pieces will ravel and then in a quality pervious concrete pavement, no further ravelling will occur (Kevern J. , 2010). Ravelling can be quantified by measuring the depth of the ravelled area and determining how many layers of aggregate has been lost, based on the size of aggregate in the mix design. The MTO describes five severities of ravelling based on visual observation in (Chong & Wrong, 1995). These descriptions are for an impermeable concrete pavement but can be interpreted for pervious concrete pavement as well. The descriptions are based on visual observations. Ravelling can become a problem if areas are created on the pavement surface that lead to debris accumulation due to be a lower. As well, if the ravelling creates potholes that lead the dangerous or uncomfortable conditions for the users.

Surface condition evaluations were performed on 29 pervious concrete pavements in various freeze-thaw climate locations. Of the 29 pavements, 20 showed ravelling of the top one to three aggregates. In some cases the ravelling was localized while others had ravelled areas throughout the pavement surface. The ravelling was felt to have occurred due to turning vehicle tires, possibly freeze-thaw damage, drying shrinkage (poor curing), snow plow use, temperatures during construction and over finishing of the surface (Vancura, Khazanovich, & MacDonald, 2010).

Deep ravelling has been characterized by Vancura as areas where more than three layers of aggregate have separated from the pervious concrete layer and loose aggregate is often present. In an evaluation of 29 pervious concrete sites, 12 exhibited deep ravelling. The deep ravelled areas were typically at a discontinuity such as a joint, crack or edge. The discontinuities were not deemed to be the reason for the deep ravelling but did promote further ravelling (Vancura, Khazanovich, & MacDonald, 2010).

In 2008 numerous pervious concrete pavements in Denver, Colorado were experiencing an unacceptable amount of ravelling. The Urban Drainage and Flood Control District (UDFCD) removed pervious concrete pavement from the Best Management Practices (BMP) list for stormwater management. Work was carried out in partnership with the University of Colorado, CRMCA and CTL Thompson to develop guidelines for producing pervious concrete that would perform adequately in Denver. A Specifier’s Guide was developed and use of pervious concrete was resumed in 2009(MacKenzie, 2009). Prior to the removal of pervious concrete pavement from the BMP list in 2008 cement paste had been settling to the bottom of the pervious concrete, construction had been carried out during weather conditions deemed to be too hot or too cold and measures were not taken to aid in curing. All three of these factors were found to result is ravelling occurring. The specifier’s guide included the following requirements and it was recommended that pervious concrete be used again as long as the guide requirements were met.

- Minimum 6 % sand in mix design;
- Construction to be carried out only when current air temperature and predicted air temperature during curing was between 4 °C and 32 °C; and

During curing pervious concrete must be covered with plastic and fogged with water to prevent the surface from drying out.

2.2.5.2 Joint Ravelling

The mechanism of material ravelling at the joints of a pervious concrete pavement is consistent with that of ravelling within a slab (Section 2.2.5.1). The cause of ravelling at the joint however can be a result of the construction method and it may not occur throughout the remainder of the pervious concrete pavement. Schaefer et al. noted in 2010 that improper use of a joint former can lead to the development of joint ravelling in the future (Schaefer, Kevern, Izevbekhai, Wang, Culter, & Wiegand, 2010). Both Kevern and the ORMCA identified that joints can successfully be saw cut into a pervious concrete pavement although it is critical to find a balance between the strength and extent of curing to ensure that cracks have not developed and the joints will be effective (ORMCA, 2009; Kevern J. , 2010)

2.2.5.3 Cracking

Cracking has been observed to develop in pervious concrete pavement due to shrinkage and lack of structural capacity.

A field site in Indiana that was constructed without the inclusion of construction joints led to cracking developing (Delatte, Mrkajic, & Miller, 2009). Cracking of the pervious concrete surface is anticipated to develop if no joints are included. In some cases these cracks are not visible to the general public and therefore not a concern of the owners (Tennis, Leming, & Akers, 2004).

Similar to other types of infrastructure, pavements will show deterioration at an increased rate when exposed to loading beyond the intended design. Two sites were identified in work by Delatte, Mrkajic and Miller in 2009 that had developed cracking due to being heavily loaded. One site included the location of a fire hydrant. The regular loading of the fire trucks and additional delivery trucks lead to the pervious concrete cracking. The second site was used to store concrete form work. The concrete forms were loaded and unloaded onto trucks using various types of heavy equipment daily (Delatte, Mrkajic, & Miller, 2009).

2.2.5.4 Sealing

A pervious concrete surface that exhibits sealing is essentially impermeable in the localized area and does not allow drainage of any water. While clogging of the pervious concrete pavement may occur over the long term, sealing is typically a result of the mix design or construction method or a combination of both. An unsuitable mix design can result in a pervious concrete being impermeable or sealed immediately following construction. Over compaction of the surface during construction can also lead to the pervious concrete surface being sealed. Sealed surfaces at field sites deemed to be a result of over compaction or too much moisture in the mix design were identified by Delatte, Mrkajic and Miller in their 2009 work (Delatte, Mrkajic, & Miller, 2009). The use of finishing tools during the construction of pervious concrete can lead to sealing of the surface, even with generally suitable mixes (Tennis, Leming, & Akers, 2004).

2.2.6 Maintenance

Winter maintenance, surface maintenance and permeability renewal maintenance should be considered in applications including pervious concrete pavement structures. Winter maintenance includes the removal of snow from the pervious concrete pavement and the efforts to increase friction and remove ice from the surface. Surface maintenance involves cleaning of the pervious concrete surface to remove debris that has potential to fill the voids and decrease the permeability of the surface. These efforts are referenced as permeability renewal maintenance in this document. Surface maintenance also includes repairing surface distresses that have developed, such as cracks or deeply ravelled areas. This is referenced as distress maintenance in this document.

2.2.6.1 Winter Maintenance

Snow removal can be performed using conventional equipment as demonstrated in research by Henderson in 2009 (Henderson, Tighe, & Norris, 2009). The open void structure of pervious concrete allows moisture to be drained from the surface efficiently. Therefore once snow is removed from the surface any remaining snow or ice can drain through the pervious concrete surface after it melts. In some applications minimal to no sand or anti-icing solutions are required to maintain the friction of the surface. When friction of the surface is a concern the application of sand or anti-icing solutions should be limited to avoid unnecessary clogging of voids or deterioration of the pavement (Cement Association of Canada, 2011).

Using salt or sand to increase friction or melt ice on a pervious concrete surface has the potential to clog voids and make the surface impermeable. The University of Vermont evaluated the changes in permeability due to the application of a salt and sand mixture on pervious concrete samples in the laboratory. Samples were loaded with a 2:1 ratio of sand to salt as this is the common practice for pavements in Vermont. The loading was applied at a rate of 0.12 g/cm^2, this loading visually covered the samples' surfaces. The rate of loading currently in practice in Vermont was unknown at the time of testing. A single application of the salt and sand mixture resulted in a decrease of permeability of 15.6 % on average. After loading the samples with the mixture the permeability rate remained more than adequate for anticipated rainfall events in Vermont, measuring 0.87 cm/sec on average. This project did not include freeze-thaw cycling (McCain & Dewoolkar, 2009).

Laboratory cast samples were freeze-thaw cycled while submerged in water containing 2 % salt. Various mixes were exposed to the cycling, some with no admixtures, others with silica fume or fibres or silica fume and fibres. The cement content was varied in samples within each mix. The best freeze-thaw durability was found in mixes with silica fume and high cement content. It is anticipated that a thicker paste is generated with higher cement contents and silica fume decreases the permeability of the paste. The thicker paste likely allows for better bond development. The inclusion of fibres and the inclusion of silica and fibres, at all cement contents, did not increase the freeze-thaw durability in comparison to the control mix (Yang Z. , 2011).

Samples were exposed to three de-icing solutions: sodium chloride (NaCl), calcium chloride ($CaCl_2$), Calcium Magnesium Acetate (CMA). Control samples were exposed to distilled water. The de-icing solutions were at a 9% concentration by weight in each case. Freeze-thaw cycling was carried out while the samples were exposed to the de-icing solutions. In terms of mass loss, the performance of pervious concrete was poorest with NaCl and the best with CMA. Samples exposed to distilled water during freeze-thaw cycling had very little mass loss in comparison to the others (Cutler, Wang, Schaefer, & Kevern, 2010).

Samples cast in the laboratory were clogged and then freeze-thaw cycled. Unclogged control samples were also freeze-thaw cycled. The clogged samples were clogged manually with soil in the laboratory in the upper portion of the samples only, as this was noticed to be common in samples collected in the field. Unclogged samples showed gradual deterioration, loosing aggregates off the sides of the samples overtime. The unclogged samples that allowed water to drain through them did not generally fail in less than 300 freeze-thaw cycles. Unclogged samples that were saturated in water during the freeze-thaw cycling failed before 300 cycles (at approximately 96). In comparison, the samples that were clogged did not show deterioration but rather failed entirely by breaking in half. The clogged samples that drained water failed after more freeze-thaw cycles (approximately 122) than those that were saturated in water (approximately 64) (Guthrie, DeMille, & Eggett, 2010).

2.2.6.2 Permeability Renewal Maintenance

During adjacent construction, the condition of the pervious concrete surface should be monitored to ensure that soils and other debris are not accumulating (Tennis, Leming, & Akers, 2004).

Practice has demonstrated that the permeability of pervious concrete applications can often be maintained with two permeability renewal maintenance activities being performed annually (Kevern J. , 2010).

A standard street-sweeper has been used to renew the permeability to pervious concrete pavements in Kansas. When using this equipment, Kevern suggests that the pervious concrete be wetted, such that all soil debris in the pervious concrete surface voids is saturated. The surface should then be swept using the street sweeper at a slow speed (Kevern J. , 2010).

McCain and Dewoolkar researched permeability testing on saturated pervious concrete specimens. Laboratory specimens were saturated with soil material and tested for permeability. In a laboratory prepared saturated condition permeability rate results ranged from 0.76 cm/sec to 0.98 cm/sec (McCain & Dewoolkar, 2010)

2.2.6.3 Distress Maintenance

As noted, ravelling is the most commonly identified distress in pervious concrete pavements. The presence of surface ravelling within the first couple aggregate layers does not indicate a structural or functional deficiency of the pavement but does present an aesthetic issue. To repair ravelling in pervious concrete pavement, whether it is within a slab or at a joint, the surface should initially be swept to remove loose aggregate. This will allow the depth of the ravelling to be examined as it is sometimes unclear when loose aggregate is present. After sweeping the loose aggregate off the surface, the pavement can be monitored to determine if the ravelling is ongoing or if it has stopped (Kevern J. , 2010).

When surface ravelling is ongoing or deep enough that a repair is requested then Kevern suggests that a complete removal and replacement of the pervious concrete be performed or localized areas can be milled and an overlay placed (Kevern J. , 2010). Ravelling at joints maybe repaired by cleaning the area and sealing the joint (Kevern J. , 2010). This method is likely simpler than repairing a ravelled area in a slab. The joint sealant would be impermeable. However, the impermeable area would be small in relation to the entire project and therefore should not be an issue in overall performance. Examples outlining the effectiveness of milling the pervious concrete and placing an overlay and using joint sealant were not included in Kevern's work.

Where surface sealing is present there may not be a need for repair if the sealing is localized. In this scenario the water will runoff to the surrounding permeable pervious concrete pavement and drain through that area. If sealed areas do require remediation to create better permeability, one alternative is to remove cores and fill the core holes with permeable material. Alternatively the sealed area can be milled. In some cases pervious concrete may not be sealed throughout the entire slab thickness, only in the top portion where more compaction was present during construction (Kevern J. , 2010). No examples were included in Kevern's work demonstrating the extent to which permeability is improved by milling sealed areas.

2.3 Summary

The information presented in current literature demonstrates the current state of the art practices in pervious concrete pavement. In Canada there is interest to use pervious concrete pavement and this is identified by the specifications available in Ontario and the references to permeable pavement in sustainable development and stormwater management documents within the country. Limited research is available on pervious concrete pavement use in Canada at this time.

Many of the various literature sources presented similar trends in findings:

- Aggregate size effects the shape of the voids but not necessarily the quantity of voids. Using smaller aggregates can generate as many voids as larger aggregates but has potential to create a more durable hardened concrete material.
- The water to cement ratio is important in pervious concrete pavement mixtures as the areas that offer support to the entire material can be small contact points. A paste with a higher water cement ratio tends to exhibit more strength characteristics than one with a lower water cement ratio.
- The use of additional cementitious materials can improve the resistance of pervious concrete pavement to deterioration during freeze-thaw cycling.
- Pervious concrete mixes can continue to perform adequately under freeze-thaw cycling. Exposure to salt solutions which could be used in winter maintenance applications can have a negative effect on the performance of pervious concrete samples. Pervious concrete samples that are saturated with water during freeze-thaw cycling perform poorer than those that are able to have water drain during freeze-thaw cycling.
- Solutions to challenges in constructing pervious concrete pavement have been developed, such as including a choker course in the pavement structure that provides a construction platform.
- Drainage alternatives for use of pervious concrete pavement on existing subgrades with low permeability are suggested in literature and some research has been performed that indicates the results that can be anticipated.
- Ravelling commonly occurs on the surface of pervious concrete pavement structures. However, ravelling is expected to occur only briefly after construction and not be extensive. When ravelling is more extensive it has been found to be a result of turning vehicles, temperatures during construction and over finishing during construction, among other causes.
- Cracking was identified when adequate joints were not included in the pervious concrete pavement during construction or when heavy traffic was continually using the pervious concrete pavement.

- The presence of sealing on the surface of a pervious concrete pavement is deemed to be a result of over finishing the surface during construction or too much moisture in the mix design.

The majority of the research that is available in the literature is from laboratory based research. This type of research is needed to advance technology and examine behavior in a controlled setting. Transitioning findings in the laboratory to application in the field is often not direct or simple. There are generally adjustments that are required in order to achieve success on full scale projects. Literature has been included that looks at full scale field research projects. In full scale projects it is a larger task to evaluate several scenarios as the requirements of each are larger than in the laboratory. The field literature findings include useful information which can be used to improve future projects. This information is often short term and does not follow the performance of the pervious concrete or effects of a scenario for multiple years. It takes a substantial amount of time to be able to report on full scale project performance and behavior over multiple years. This information though is critical as it provides more detail and understanding into the life time performance that maybe possible for a pervious concrete pavement. The information available regarding pavement structure, construction practices, maintenance activities and distress maintenance is generally in guidelines and does not provide details of research done to achieve the guidelines. With the popularity of pervious concrete increasing substantially in recent years there has not been the opportunity to monitor the performance of these guideline suggestions in the field for a long term basis as they are very recent. The guideline suggestions for these various activities need to be monitored in a long term scenario, which could occur in the field over several years or through accelerated testing in the laboratory.

3.0 DATA SOURCES AND TESTING MATERIALS

The data used in the analysis in this research has come from several sources, which are outlined in this chapter. The project involved an integration of laboratory and field work across Canada. The combination of laboratory and field research has led to several benefits and challenges, which are noted below:

- Controlled conditions in the laboratory;
- "Real-time" conditions at field sites; and
- Ability to estimate future performance based on accelerated laboratory testing.

The methods in which data has been collected in both field work and laboratory research will be outlined in Chapter 3. Many of the tests were carried out in both the field and laboratory work. The standards that were followed for all testing will be outlined in the following section.

3.1 Concrete Testing Procedures

Throughout the research, conventional concrete tests were performed in addition to carrying out testing specific to pervious concrete pavement. As previously noted, the results of this work are intended for use by industry members throughout Canada. In order to make the results useful initially to the community, they needed to be practical and applicable. Most of the testing outlined in Section 3.1 is conventional concrete testing with the exception of a few tests that are specific to pervious concrete pavement.

3.1.1 Slump Test

The result of performing a slump test on a batch of fresh concrete provides information as to the consistency of the mix. It also provides a result that can be used to compare different loads of concrete, concrete samples or mixes. Slump testing was performed throughout the project following CSA A23.2 – 5C "Slump of Concrete" (CSA, 1996). Slump testing was performed in this research during the construction of field sites and when pervious concrete was prepared in the CPATT laboratory for casting of samples both cylinders and slabs.

Figure 3-1 shows slump testing being performed in the laboratory at the University of Waterloo.

Figure 3-1: Slump Testing

Following the procedure in CSA A23.2-5C the slump cone or mould is moistened and placed on a flat non-absorbent surface (CSA, 1996). The cone is then filled in three layers, with each layer consisting of enough concrete mix to fill approximately one third of the volume of the mould. Each layer is rodded 25 times, with the rodding being distributed evenly across the layer. The final layer should be filled over the top of the cone, so that after the 25 rods are complete there is still excess material and it can be rolled off with the rod and the concrete is left flush with the top of the cone. The cone is then lifted off the concrete, at a steady rate, that is performed in close to five seconds. The cone can then be placed, upside down beside the concrete and the difference in height between the concrete and mould can be measured. This measured value is the “slump” of the mix. As Figure 3-2 shows, pervious concrete generally has a low slump, often 0 mm.

Figure 3-2: Measuring the Slump of Pervious Concrete

3.1.2 Air Content

Concrete intended for outdoor use in freeze-thaw climates such as Canada is often air entrained. The addition of air entrainment admixtures creates microscopic bubbles in the concrete mix that provide relief for pressure that is created when moisture in the voids of conventional concrete freezes. Many of the pervious concrete mixes placed at the field sites include air entrainment as well as the mixes prepared in the laboratory.

The percentage of air entrainment present in a pervious concrete mix was tested following CSA A23.2 – 4C "Air Content of Plastic Concrete by the Pressure Method" (CSA, 1996). This method is intended for conventional concrete mixes and has potential to provide inaccurate results due to the consistency of pervious concrete.

The test involves filling the bowl of a pressure tight container with fresh concrete in three layers. Each layer should be equal in volume. The size of bowl required is determined based on the nominal maximum size of aggregate in the mix. In this work a bowl with a capacity of seven to fifteen litres was acceptable in all cases.

The procedure requires that mixes with a slump less than 40 mm be vibrated, while those with a slump greater than 40 mm can be rodded. In this project, pervious concrete samples were always rodded to avoid filling intended voids in the mixes. Each layer is placed in the bowl then rodded 25 times. The outside of the bowl is then tapped smartly using a rubber mallet to achieve further consolidation. The final layer should slightly overfill the bowl, by 3 mm is optimum. The top of the bowl is then striked off using a strike off bar. The exterior of the filled bowl is then cleaned with a damp cloth to remove any excess material. The lid to the air meter can then be attached. Figure 3-3 shows an air meter bucket with the lid attached.

Figure 3-3: Air Meter Bucket

At this point the air meter should be filled with water through the holes in the lid of the air meter until water is flowing out and is clear. The holes are then closed and using the air valve, the air meter should be pumped with air to the calibration value. The air meter pressure is then released and the air entrainment content can be read off the dial.

3.1.3 Density

When this project was initiated in 2007 no pervious concrete test methods were available through ASTM or CSA. As previously noted, test methods for conventional concrete were used for comparison purposes for industry between conventional concrete and pervious concrete as well as for comparing various pervious concrete mixes within the project. In October 2008 an ASTM standard was released for density and void content measurements of pervious concrete pavement, ASTM C1688 “Standard Method for Density and Void Content of Freshly Mixed Pervious Concrete". Moving forward from the fall of 2008 both the conventional concrete test method for density, CSA A23.2-6C "Density, Yield, and Cement Materials Factor of Plastic Concrete" and the new pervious concrete method were used. Both methods are described in this section.

3.1.3.1 CSA A23.2-6C "Density, Yield and Cementing Materials Factor of Plastic Concrete"

The density of the fresh pervious concrete mixes was measured following CSA A23.2-6C, "Density, Yield and Cementing Materials Factor of Plastic Concrete" (CSA, 1996). To perform this test the bucket from the air meter was used. In general density measurements were performed and then the sample was tested for air entrainment content. The same method is used in both tests to fill the air meter bucket. The method for filling the air meter bucket is described in further detail in Section 3.1.2.

Prior to filling the air meter bucket, the weight of the air meter bucket is recorded or the scale is torn to that value. Once the bucket has been filled, the bucket and material are weighed. Using Equation 3-1, the density of the pervious concrete mix can be determined.

$$D = \frac{M}{V} \qquad \text{Equation 3-1}$$

Where,

D is the density of the pervious concrete mix, kg/m^3,

M is the mass of the mix in the air meter bucket, kg,

V is the volume of the air meter bucket, m^3.

3.1.3.2 ASTM C1688 "Standard Test Method for Density and Void Content of Freshly Mixed Pervious Concrete"

In the fall of 2008, ASTM C1688 "Standard Test Method for Density and Void Content of Freshly Mixed Pervious Concrete" was published (ASTM, 2010). This test method is solely for pervious concrete. In comparison, prior to 2008 test methods were generally for concrete and not pervious concrete specifically. The main difference between this method and CSA

A23.2-6C is the manner in which the pervious concrete is compacted prior to measuring the density.

In this method a cylindrical container, such as an air meter bucket, with a diameter to height ratio of 0.75 to 1.25 should be used. Pervious concrete should be placed in the container in two layers, with the layers being approximately equal in volume. When each layer is placed in the container the pervious concrete should be distributed throughout the bowl. Each layer is then compacted using a Standard Proctor Hammer. The Proctor Hammer should be dropped 20 times per layer with the drops being distributed across the layer. Figure 3-4 shows the Proctor Hammer being used to compact the pervious concrete during density testing.

Figure 3-4: Proctor Hammer Compacting Pervious Concrete for Density Testing

After compacting the second layer the pervious concrete should be overflowing the container, ideally by 3 mm. If after ten Proctor Hammer drops on the second layer it does not appear that the pervious concrete will be overflowing following the total 20 drops then more pervious concrete should be added. After the 20 drops are completed the excess pervious concrete should be removed by striking off the surface. The pervious concrete should be level with the top of the container. All excess pervious concrete on the exterior of the container should be removed with a damp sponge. The pervious concrete and container can then be weighed. Using Equation 3-1 the density of the fresh pervious concrete mix can be determined.

3.1.4 *Temperature*

The temperature of the fresh pervious concrete mixes was monitored during construction and batching in the laboratory. These measurements were taken following ASTM C1064 “Standard Test Method for Temperature of Freshly Mixed Portland-Cement Concrete” (CSA, 1996). Pervious concrete samples were collected from the concrete truck chute on site in a wheel barrow or in the laboratory from the mixer into a wheel barrow. Following procedure, the thermometer was then inserted into the fresh concrete, ensuring 75 mm of concrete cover on all sides. The concrete was then gently pressed against the thermometer to ensure the concrete was measured and not the air temperature. The reading was taken after at least two minutes and when the thermometer had stabilized.

3.1.5 *Compressive Strength*

Compressive strength testing was carried out on cores extracted from the field sites and laboratory prepared slabs and cylinders cast during the construction of field sites and laboratory projects. The cylinders were cast following CSA A23.2-3C “Making and Curing Concrete Compression and Flexural Test Specimens” (CSA, 1996). Additionally a complimentary project at the University of Waterloo was carried out to assess the effect of different compressive strength cylinder preparation methods (Rizvi, Tighe, Henderson, & Norris, 2009). The results of this project indicated that when pervious concrete samples were compacted with a Proctor Hammer rather than rodding, the results were more consistent. Following the completion of the project with Rizvi, samples were compacted with the conventional method of rodding and others with the Proctor Hammer for further comparison. Based on the recently released standard, ASTM C1688 “Standard Test Method for Density and Void Content of Freshly Mixed Pervious Concrete”, it is evident that industry members involved in pervious concrete are also acknowledging that the use of the Proctor Hammer for compaction may be more applicable.

Both 100 mm and 150 mm diameter cylinders were cast for compressive strength testing following CSA A23.2-3C. In this case the moulds were filled in three layers which were approximately equal in volume. When 100 mm diameter cylinders were prepared each layer was rodded 20 times and for 150 mm diameter cylinders each layer was rodded 25 times with the rodding being evenly distributed throughout the layer. The sides of the moulds were not tapped as is described in the procedure because of the design of the pervious concrete mix and interest to replicate the condition of the material placed in the field.

The procedure describes that concrete mixes with low slump values, which is the case of pervious concrete, should be consolidated in the mould using a vibrator. Similar to the reasoning behind not tapping the outside of the mould the objective was to replicate pervious concrete being placed in the field in terms of characteristics. Therefore if a vibrator was used for the compaction it was anticipated that it would create cylinders differing in characteristics than field placed pervious concrete. The mould was overfilled with the third layer and then striked off to be flush. A lid was secured on the cylinder. Cylinders were removed from the moulds 20 +/- 4 hours after casting and placed in a moisture room for curing until testing.

In general the cylinders were tested for compressive strength at 7 and 28 days of age. The ends of the cylinders and cores were prepared for compressive strength testing using an end grinder which is shown in Figure 3-5. In Rizvi 2009 end preparation techniques including end grinding and sulfur capping were evaluated (Rizvi, Tighe, Henderson, & Norris, 2009). The results proved end grinding produced samples that provided consistent data and the preparation was much easier to carry out without damaging the samples.

Figure 3-5: Cylinder Sample End Preparation by End Grinding

Compressive strength testing was carried out following CSA A23.2-9C "Compressive Strength of Cylindrical Concrete Specimens" (CSA, 1996). The cylinder was centered on the platen of the testing equipment. The cylinder was then loaded at a rate between 0.15 MPa/sec and 0.35 MPa/sec with the maximum load before failure being recorded.

3.1.6 *Flexural Strength*

Flexural strength testing was performed on beams cast during the construction of field sites. All beams were 605 mm by 155 mm by 155 mm in size and were cast in accordance with CSA A23.2-3C "Making and Curing Concrete Compression and Flexural Test Specimens" (CSA, 1996). As the beams were less than 200 mm in height the pervious concrete was compacted in two layers. Using Equation 3-2 it was calculated that 94 rods were required per layer.

$$R_n = \frac{A}{10cm^2} \qquad \text{Equation 3-2}$$

Where,

R_n is the number of rods required per layer,

A is the cross sectional surface area of the beam being cast, cm^2

After each layer was rodded then a trowel was inserted along the inside perimeter of the mould. At this point the sides of the mould were not tapped, as to not create beams that were different in characteristics than the pervious concrete in the field. The second layer overfilled the beam mould so that the material could be striked off level with the top of the mould. The beams were then covered in plastic to maintain moisture during initial curing. The beams were removed from the moulds at 20 hours +/- 4 hours and then moved to the moisture room to continue curing.

The beams were tested for flexural strength at 28 days following ASTM C78 "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) (ASTM, 2008). The beam was placed in the testing frame as shown in Figure

3-6, with the bottom support blocks being L width apart. In the case of the beams prepared in this project L was at minimum 465 mm as the height of the beams was 155 mm. The load applying blocks were one third of L apart and centered over the beam. The beam was then loaded until rupture occurred.

Figure 3-6: Flexural Beam in Testing Frame

3.1.7 Void Content

The void content of cores and cylinders was measured using one of two methods: Corelok ® vacuum; or T-040R. The test method was selected based on the size of the sample. The Corelok ® vacuum could only effectively seal smaller samples. If the sample was to large to be sealed in the vacuum then the T-040R method was used. The cores could have been cut and tested in the vacuum. In general the cores were not cut as the intent was to measure the entire sample and/or to use the complete sample for additional testing. Typically the samples that could not be vacuumed were 150 mm in diameter and more than 150 mm in length, approximately (Montes, Valavala, & Haselbach, 2009; Instrotek®, Incorporated, 2011).

3.1.7.1 Corelok ®

Void content measurements using the Corelok® vacuum system follow the instructions provided in the operating manual(Instrotek®, Incorporated, 2011). The dry weight of a sample is taken. The weight of the appropriate sized bag is measured. The sample is then sealed in the bag using the bag using the vacuum. The sealed bag and sample are weighed. The sealed sample and bag are then submerged in a water bath and the weight is recorded. The bag is then cut open and the sample becomes fully saturated in the water bath and the weight is recorded. Finally, using each of these measurements the void content is determined (Instrotek®, Incorporated, 2011).

3.1.7.2 T-040R

This test method was developed in order to measure the void content of pervious concrete without using a vacuum system. This method can be used on any size of sample. The sample is weighed initially when it is dry. It is then placed in a water bath and the weight is taken again. These measurements are used to calculate the void content (Montes, Valavala, & Haselbach, 2009).

3.1.8 Freeze-Thaw Durability

Freeze-thaw durability testing was carried out on cores following a modified version of ASTM C 666 (ASTM, 2008). In this testing cores were freeze-thaw cycled in a chamber between approximately 15 °C and – 12 °C. Cores were placed in trays during the freeze-thaw cycling such that the cores lay horizontally but were raised up off their side. Every 10 to 20 cycles, cores were removed from the chamber and weighed. Following being weighed, the cores were submerged in a water bath for approximately 10 minutes. The cores were then placed back in the trays and freeze-thaw cycling was resumed. Since the cores were raised in the trays, any draining water accumulated in the bottom of the tray and the cores were not submerged in the water (ASTM, 2008).

3.1.9 Permeability

Similar to the density measurement methods used in this work, at the initiation of the project there was not a standard published for measuring the permeability of pervious concrete pavement. At this point a method that had recently been sourced for another project was implemented for use in this work. This method is described in Section 3.1.7.1. This method has been used since the initiation of the project and therefore was continued to be used throughout the project for consistency even though an ASTM standard was developed. The standard, ASTM C1701 "Standard Test Method for Infiltration Rate of In Place Pervious Concrete" was published in September 2009 (ASTM, 2009). This method was not included in this research.

3.1.9.1 Permeability with Gilson Permeameter

The permeability of the pervious concrete field sites has been measured throughout the project using a Gilson Field Permeameter. This permeameter was also used in the laboratory for measuring the permeability of cores, cylinders, beams and slabs. An operating instructions manual was provided with the permeameter from Gilson Company Inc. (Gilson Company Inc., 2007). The process that has been used to measure permeability using the permeameter follows the guidelines in the operating instructions manual with a few modifications. Figure 3-7 shows the permeameter in use at one of the field sites.

The permeameter is sealed to the pervious concrete surface. This is done by applying a thick (approximately 25 mm) ring of plumber's putty to the bottom of the permeameter. The permeameter is then placed on the surface and pushed into the surface to move the putty into the pervious concrete voids around the edge. By sealing the permeameter to the surface the water is then forced to enter the pervious concrete during testing and not drain along the surface. After sealing the permeameter to the surface, four weights are placed along the base to ensure that leaks do not develop due to the pressure of the water if the permeability rate is low.

Figure 3-7: Gilson Permeameter

One evaluator lies flat on the ground so that their eyes will be level with the water in the permeameter. While the second evaluator then pours water into the permeameter. The person who is lying on the ground will record the time required for the water level to drop from one mark to another. Figure 3-8 shows the graduated marks on the permeameter. Both the initial and final points need to be within one tier.

Figure 3-8: Graduated Measurement Markings on Permeameter

Depending on the drainage capabilities of the pervious concrete the water can be poured into the permeameter from a jug or garden hose or has to be pumped through a larger hose in order to achieve a rate that creates a point when there is a level of water for long enough that it can be recorded. The test is repeated three times on each location. When possible the water is recorded between the same two points all three times that testing is repeated. This is simply to make comparisons easier. However, often the permeability decreases with each test and it may not be efficient to continue testing the same range. At least 25 points are evaluated at each site and if more than one mix is at a site then there are at least five points tested within each mix. The testing is always repeated on the same locations at a site and the test locations are distributed across the sites.

When slabs are tested in the laboratory the procedure is the same as it is at the field sites. Figure 3-9 shows a pervious concrete slab being tested in the laboratory.

Figure 3-9: Permeability Testing of a Slab in the Laboratory

The same permeameter is used for testing 100 mm and 150 mm cores and cylinders that is used for the slabs and field sites. The cores and cylinders are wrapped in a latex sleeve to minimize and ideally eliminate drainage out of the sides of the samples. In the field it is anticipated that drainage occurs both horizontally and vertically however, it is not predicted that there are any channels that provide as little resistance as the edges of a cylinder does. Therefore the latex sleeve is used to provide pressure to the open voids and reduce free flowing water out of those voids. The industrial putty is used along the rim of the permeameter and pressed into the top of the cylinders to seal the testing equipment. Figure 3-10 shows a 100 mm sample and a 150 mm sample being tested for permeability.

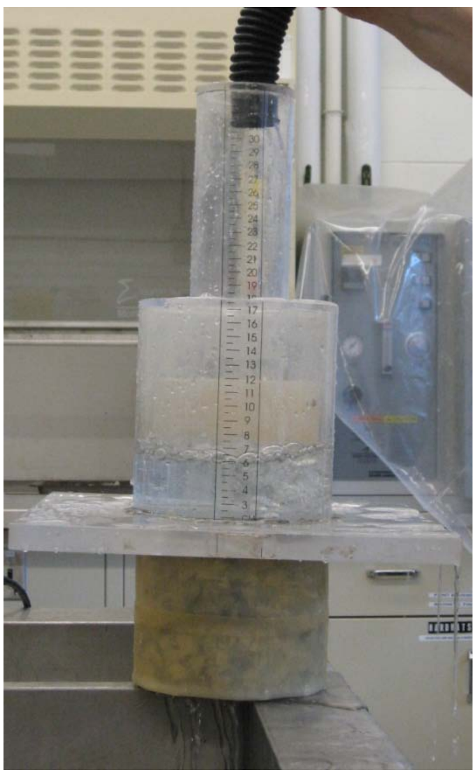


Figure 3-10A – 150mm Diameter Core

Figure 3-10B – 100mm Diameter Core

Figure 3-10: Permeability of Cores and Cylinders in the Laboratory

As Figure 3-10B shows, the 100 mm samples fit inside the larger tier and are sealed to the smaller top tier. The cores and cylinders are then tested in the same manner as the field sites and slabs with the permeameter being filled with water and timed between two points. In all cases, Equation 3-3 is used to determine the permeability of the pervious concrete.

$$K = \frac{aL}{At} ln\left(\frac{h_1}{h_2}\right)$$ Equation 3-3

Where,

K is the coefficient of permeability, cm/sec

a is the inside cross-sectional area of the permeameter, 167.53 cm^2 for the larger, bottom tier or 38.32 cm^2 for the smaller, upper tier

L is the length of the sample, the thickness of the core or pervious concrete layer, cm

A is the cross-sectional area of the drainage of the permeameter, cm^2

t is the elapsed time between h_1 and h_2, seconds

h_1 is the initial head, cm

h_2 is the final head, cm

The value of A is intended to represent the horizontal flow of water, as it is anticipated that not all water stays within the permeameter circumference once moving into the pervious concrete. In the case of the cores and cylinders all the water remains within the circumference as this is the goal of the latex sleeve. At the field sites and within the slabs it is challenging to predict how far water flows horizontally. The value of 214 cm^2 is provided by Gilson in the operating instructions manual and has been used in this work (Gilson Company Inc., 2007). The purpose of the permeability testing is to evaluate changes and relatively compare sites and different mixes and areas within sites. By using the constant value of 214 cm^2 for A this is possible.

The permeability rate results were compared to a maximum rainfall rate. If the permeability results were less than the maximum rainfall rate then the permeability of the pavement was deemed to be inadequate. The maximum rainfall rate was determined from intensity duration frequency curves for the area surrounding each field site (Environment Canada, 2007). The maximum rainfall rate was similar for each of the five field sites and the highest was selected, 0.0083 cm/sec. Given that the test method used for permeability testing in this research using the falling head principal and a much larger head than would exist at a field site, it was deemed appropriate to use the maximum rainfall rate value for comparison.

3.2 Surface Distress Evaluation Form

During the early stages of this project a surface distress evaluation form was developed for use during surface distress evaluations of both field sites and laboratory slabs. A literature review was carried out to compile a list of distresses that had developed in pervious concrete pavement in the United States at this time. Pervious concrete pavements throughout the United States were included as it was anticipated that distresses present in continually warm climates may also occur in Canada in addition to those generated by conditions in cooler climates. The details of this research can be found in Henderson, 2009 (Henderson, Tighe, & Norris, 2009). The form is included in Appendix A. The form includes areas to identify the type of distress present, severity of the distress and the density.

The severity and density ranges used on the form were driven by those used by the Ontario Ministry of Transportation (MTO) in their surface distress evaluations(Chong & Wrong, 1995). For the purposes of this research it was deemed that manual surveys of the surface conditions would be adequate. As is noted throughout this project, pervious concrete pavement is intended for low speed, low volume areas and thus manual surveys could be carried out safely. All surface distress evaluations were carried out by the same person, Vimy Henderson, in an effort to minimize bias. While this data collection is qualitative in many respects, efforts were made to ensure the quality and usefulness of the information. These efforts included taking photos during evaluations, measurements when applicable (ie. crack width, ravelling depth) and as previously mentioned, ensuring all evaluations were carried out by the same individual.

3.3 Instrumentation

Several of the field sites and laboratory samples included in this project were instrumented during construction or casting. Each type of instrumentation that is included in this project is described in Section 3.2.

3.3.1 Weather Data

Each field site includes a weather station that records air temperature and the quantity of rainfall in each event. An example of a weather station is shown in Figure 3-11.

Figure 3-11: Weather Station at a Field Site

Each of the weather stations includes a PVC tipping bucket rain gauge and a HOBO Pendant Event and Temperature Logger. The logger records rain events in the tipping bucket as well as temperature readings of the air. The temperature readings are taken from within the white housing shown in Figure 3-11, which is a solar radiation shield. The weather stations provide information regarding the weather activities occurring at each field site. It was intended that the data collected from these weather stations would be used to compliment and better understand other data collected in this project. Examples of such are the timeline of moisture movement through the pavement structure and the temperature of pervious concrete pavement structures in comparison to the air.

Environment Canada weather stations are located throughout Canada and fortunately provide public data, including various weather elements at several collection rates (Environment Canada, 2010). Data from one weather station, Egbert was included in the analysis of this project. The data retrieved from the Egbert weather station in Ontario was also supported by the radar images available for the King City station (Environment Canada, 2011; Environment Canada, 2011).

3.3.2 Moisture Gauges

Moisture sensors were included in three of the field sites. In all cases Watermark Moisture Gauges were used. The moisture gauges were attached to sensor trees and placed in the pavement structures during the construction phase. Figure 3-12 shows a sensor tree that included three Watermark Moisture Gauges.

When the sensor tree is within the pavement structure, the moisture gauges are at different heights, ranging from the pervious concrete surface to within the natural subgrade. The intent of the moisture gauges on the sensor trees is to understand the behavior of moisture after leaving the surface. As is noted in this work, the drainage rate from the surface can be easily monitored using a permeameter, however the movement after this point is unknown. By placing the moisture gauges throughout the depth of the pavement structure, it is anticipated that moisture movement can be followed.

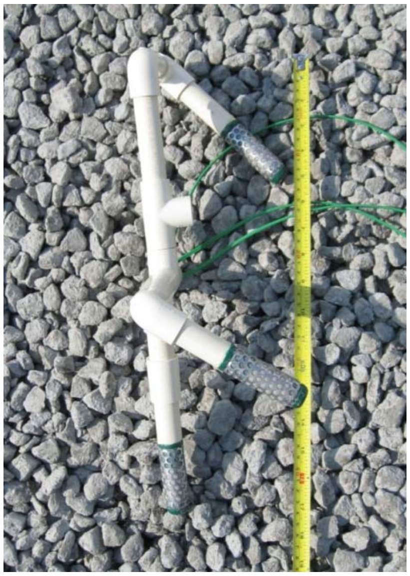


Figure 3-12: Sensor Tree including three Moisture Gauges

3.3.3 Maturity Sensors

A maturity sensor is a small type of instrumentation that includes a datalogger, temperature sensor and time stamping device. They are used in concrete applications to monitor the curing of placed concrete immediately following construction. Generally the interest is to detect when the concrete has reached an adequate strength to continue further construction in that area. The temperature and time correlation relates to the properties of a particular concrete mixture. In this work, the raw data generated by the maturity sensors in laboratory prepared slabs was used: temperature; and date and time. Maturity sensors were placed within several of the laboratory cast slabs and were used to determine the time required to cycle the slabs from freezing to thawing. Figure 3-13 shows the small, yellow, maturity sensor being placed in a slab during casting.

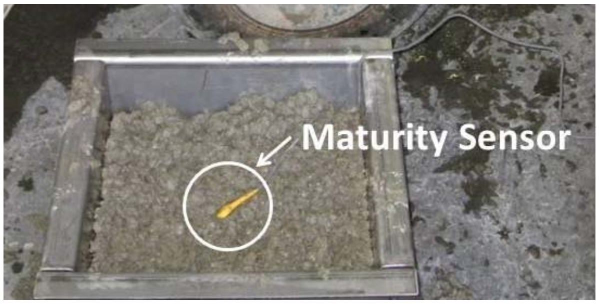


Figure 3-13: Maturity sensor being placed in a slab during casting

3.3.4 Temperature Sensors

Temperature probes, model 109B, supplied by Campbell Scientific, were included in the subgrade of some of the field sites. The inclusion of this instrumentation was a necessity in order to interpret data recorded by the moisture gauges. Figure 3-14 shows a temperature probe during installation at the construction of a field site.

Figure 3-14: Temperature Probes Being Installed

3.4 Field Sites

This research has involved the monitoring of the performance of five field sites throughout Canada. The design and construction of the five sites contributed to the research as well. The location of the five field sites is shown in Appendix B, Figure 3-15 and listed below:

- Site 1, Georgetown, ON;
- Site 2, Campbellville, ON;
- Site 3, Maple Ridge, BC;
- Site 4, Barrie, ON; and
- Site 5, Laval, QC.

Figure 3-15: Field Site Locations

By including these sites in this project the goal was to achieve a variety of scenarios in terms of materials, climate, traffic and construction methods. With the contributions of industry partners this was achieved and a matrix is shown describing the site features in Table 3-1.

Table 3-1: Field Site Matrix

Site	Location (Climate Zone)	Construction Date	Construction Method	Size (m^2)	Application (Traffic)	Pavement Structure	Comments
1	Georgetown, Ontario (Dfa)	Summer 2007	Automated Vibrating Roller	630	Employee Parking at Concrete Plant (Personal Vehicles)	Pervious Concrete (300 mm) Clear Stone Base (600 mm) Impermeable Slab (100 mm) Existing Subgrade	Impermeable slab included to collect water for testing of characteristics.
2	Campbellville, Ontario (Dfa)	Fall 2007	a. Bidwell Bridgedeck Paver b. Razorback Paver	1800	Public Car Pool Lot (Personal Vehicles and Personal Vehicles with Trailers)	Pervious Concrete (240 mm) Granular O (OPSS) Base (300 mm) Subdrain across width of site Silty Sand Subgrade	Two methods used for construction
3	Maple Ridge, British Columbia (Cfb)	Spring 2008	Manual Roller Compacted	60	Entrance and Exit Driveway of Concrete Plant (Concrete, Aggregate Trucks, Personal Vehicles)	Pervious Concrete (250 mm) Clear Stone Base (200 mm) Silty Sand, Sandy Silt with Gravel and Cobbles Subgrade	Two variations of mix design (3A, 3B) Instrumented with moisture gauges and static strain gauges
4	Barrie, Ontario (Dfb)	Fall 2008	Asphalt Paver and Vibratory Plate Compactor	750	Employee Parking at Concrete Plant (Personal Vehicles)	Pervious Concrete (175 mm) Clear Stone Base (350 mm) Silty Sand Subgrade	Three vairations of mix design (4A, 4B, 4C) Each mix instrumented with moisture gauges
5	Laval, Quebec (Dfb)	Summer 2009	Automated Vibrating Roller and Vibratory Plate Compactor	255	Employee Parking at Concrete Yard (Personal Vehicles)	Pervious Concrete (200 mm) Cement Stabilized Clear Stone Base (Popcorn Concrete) (200 mm) Existing Subgrade	Three variation of mix design (5A, 5B, 5C) Two variations of construction method (5B, 5D) Instrumented with moisture gauges (5B, 5C)

W. Köppen Climatic Classifications (Schultz, 2004)		
	Dfa	Cold winter, hot summer, adequate moisture throughout the year
	Dfb	Cold winter, warm summer, adequate moisture throughout the year
	Cfb	Mild wet winter, short warm moist summer

Each of the field sites has been monitored throughout this project. All sites underwent the same types of testing which are described in the following sections. Characteristics of each site are presented in the following sections. A photo of each site describing the layout is included in Appendix B.

3.4.1 Site 1

Site 1 is an employee parking lot at a concrete ready mix plant. Within the parking lot there is a centre area where vehicles drive to access the surrounding parking spots. Vehicles are turning in all areas of the parking lot, both driving and parking areas. The pavement surrounding the parking lot is generally sand. Substantial amounts of sand and debris from the plant is brought on to the parking lot. The sand and debris create a scenario that is not ideal for pervious concrete pavement. The conditions at Site 1 are generally more aggressive in terms of loading from debris than would be recommended for the use of pervious concrete pavement. Snow is removed throughout the winter using a front end loader. Sand is applied to the surface in the winter for traction. Snow removal and application of sand to the surface would be required several times throughout the winter, often multiple occasions weekly.

3.4.2 Site 2

Site 2 is a public car pool lot along Highway 401, west of Toronto. The lot is used by personal vehicles 24 hours a day. Some larger vehicles use the lot for turning around, these would generally consist of personal vehicles towing trailers such as campers and boats and small trucks such as delivery trucks. Transport trucks and buses would not use the facility. There are areas at the site where vehicles park and other areas that are driving lanes to access the parking spots. On all portions of the site vehicles would be turning at some times. There is a berm on the north side of the site that includes a variety of natural vegetation. Debris maybe washed down the berm during rain events. However, very limited debris of this nature was observed at the site. The site is plowed throughout the winter seasons with highway grade snow removal equipment. Salt was applied to the parking lot as required during storms. The frequency that salt was applied is not known.

3.4.3 Site 3

Site 3 includes two areas at a concrete ready mix plant: 3A; and 3B. Site 3A is a 1 m wide strip that is in the centre of the entrance driveway to the concrete plant. Often vehicles would straddle the pervious concrete strip. While some vehicles may drive on the pervious concrete pavement for the length of the driveway, most would just be on the pervious concrete for a short length if they turned off the main driveway. Loading from vehicles includes aggregate trucks, concrete trucks and personal vehicles. Site 3B is 1 m wide and runs the width of the exit driveway of the concrete plant. Given that this section spans the entire width of the driveway, all vehicles that are on site cross over the pervious concrete. The concrete ready mix plant includes about five concrete trucks that each take at a maximum a few loads daily. Snow removal is not generally carried out or required at this site. As well, no winter maintenance such as sand or salt is applied to the pervious concrete pavement. The plant has some dust and debris from the aggregates and ongoing activities but generally this material is not tracked onto the pervious concrete pavement due to the layout of the site.

3.4.4 Site 4

Site 4 is an employee parking lot at a concrete ready mix plant. The traffic is personal vehicles which are only parked on the pervious concrete. The site includes three different mixes: 4A; 4B; and 4C. The vehicles drive across all three mixes and park primarily on mixes 5A and 5B. The pavement surrounding the pervious concrete is a combination of sand and gravel. Similar to Site 1, a substantial amount of the sand and gravel is tracked on to the pervious concrete pavement. On the north side of the parking lot there is a small berm that is covered in grass. Some soil and vegetation from the berm may be washed on to the pervious concrete pavement during rain events. During the winter the pervious concrete and remainder of the plant is plowed with a front end loader. The snow that is plowed is piled on the pervious concrete and with this comes a large amount of sand and gravel. Some sand is applied to the pervious concrete surface during winter storms, as required. The pervious concrete parking lot is higher on the west side and slopes down to the right.

3.4.5 Site 5

Site 5 is at a yard for mobile/volumetric concrete mixers. Although the concrete is not batched at this facility, all aggregates and cement are stored on site. There is debris from the yard activities on all of the pervious concrete sections. The amount of debris that is present is more than would be generally recommended for areas with pervious concrete pavement. There are three mix designs and two pervious concrete areas. Sites 5A, 5B and 5C are all unique mix designs and in the same part of the site. This area is an employee parking lot. The traffic includes personal vehicles which are only parked on the pervious concrete and limited turning occurs when vehicles are arriving and departing. The second area is 5D, which is the same mix design as 5B. Site 5D is a small paved area at the corner of a building that receives only foot traffic if any traffic at all. All of Site 5 is plowed in the winter using a front end loader. Limited winter maintenance is applied to the site and when any is used it is sand. To the east of sections 5A, 5B and 5C is a large grass berm. Debris from the berm washes onto the pervious concrete during rain events.

3.4.6 Surface Distress Evaluations

As noted in Section 3.2, a surface distress evaluation form was developed in the early stages of this project. Using this form each field site was monitored and manually evaluated for distress development since the completion of construction.

3.4.7 Permeability

Understanding the permeability of the pervious concrete field sites was a key aspect in this work as it is an important measure of the functional performance. Section 3.1.9.1 describes how a Gilson permeameter was used for testing at each site. The testing was performed at a minimum of 25 points within each field site, the points being distributed throughout the site. The testing was repeated three times on each point to achieve an average. By repeating the testing three times it was also intended to evaluate if trends occurred in permeability, however, it should be noted that the quantity of water that is often used for just one test could be received during several hours of a rain event.

3.4.8 Portable Falling Weight Deflectometer

Portable Falling Weight Deflectometer (PFWD) testing was carried out several times at the field sites through the course of the project. PFWD testing is modeled after the Falling Weight Deflectometer (FWD) which is conventionally used on a pavement to gain information about the structural integrity. The PFWD is the same concept, with a weight being dropped on the surface and the deflection of the surface being measured. PFWD uses a much smaller weight, therefore, making it a portable device that can be operated manually by one or two individuals. PFWD testing is recommended for use on subgrade and granular materials. A PFWD was available at CPATT during this research and was trialed on the pervious concrete pavement. The results of the PFWD will be used to assess if it is capable of capturing changes in the pervious concrete pavement structures. If the PFWD presents results that are deemed to describe the pavement structure then it is anticipated that the consistency in material characteristics throughout a particular site can be assessed. As well, the results could be used in comparing the characteristics of the five field sites.

3.4.9 Permeability Renewal Maintenance Methods

The ability to renew the permeability of the pervious concrete pavement is necessary if it is found to have deteriorated to an inadequate level. Several renewal methods were evaluated within the scope of this project. Evaluation of each method was carried out at a full scale level, meaning an entire field site or a sample scale level, being a few points within a field site. Whenever, possible, the field site was tested for permeability then the maintenance was carried out. One of the considerations when carrying out renewal techniques or initiating them, was to consider the feasibility of the method being implemented on future commercial or residential projects. Key aspects being simplicity, equipment availability and efficiency in addition to the determining factor, effectiveness.

3.5 Summary

Testing was performed throughout the life cycle of the pervious concrete pavement as was deemed feasible. Methods currently used for testing fresh and hardened properties of pervious concrete pavement were used. When available and applicable, pervious concrete specific test methods were also included. A surface condition form was developed in this research and used throughout. Permeability testing was carried out in the laboratory and at field sites. The permeability testing method remained generally consistent on various types and sizes of samples for comparison purposes. PFWD testing was performed at the field sites throughout the performance monitoring period. Some field sites were instrumented with moisture gauges during construction. Each field site also had a seasonal weather station constructed and placed on site.

4.0 PLANNING AND DESIGN OF PERVIOUS CONCRETE PAVEMENT

Infrastructure projects generally have multiple phases: planning; design; construction; maintenance; and decommissioning. In this chapter the first two phases (Planning and Design) are discussed for pervious concrete pavement. In Chapter 5.0 Construction and Maintenance are discussed. The fifth phase; decommissioning or reconstruction was not covered in this research. Within each phase the areas that were researched were focused on pervious concrete pavement as a pavement, and stormwater management research was deemed to be outside the scope of this project.

The required behavior for a pervious concrete pavement to be deemed to be functioning adequately is different from that of most pavements, specifically those designed to be impermeable. Pervious concrete is generally used in applications where there is a common demand and that is to drain moisture from the surface. The additional requirement of pervious concrete is often to act as a pavement. However, the loading from users can vary between pedestrians only, to moving, lightly loaded vehicles. In some cases the pervious concrete may not receive any regular traffic if it is used on an embankment or other location where only drainage is required. To assess if a pervious concrete pavement is functioning the ability for it to drain moisture from the surface must be determined. If a pervious concrete pavement does not drain water at an adequate rate then it is not functioning for its intended purpose. The degree to which a pervious concrete pavement functions is related to the portion of the site that drains water at the required rate as well as the site design. If areas that do not drain slope so that water runs off to areas that do drain then the pervious concrete still functions; however, not as well as intended.

The rate at which a pervious concrete pavement needs to drain to be adequate is dependent on a couple of factors. These factors include rainfall rate of the area and site design. It is anticipated that owners and designers intend for pervious concrete pavements to function so that puddles do not develop on the surface. In order to avoid puddles developing on the surface a pervious concrete pavement must drain at a rate that is equivalent or greater to that of the maximum rainfall rate experienced in the area.

The permeability of a pervious concrete pavement is related to several factors. These factors include characteristics of the following:

- Application;
- Mix design;
- Pavement structure;
- Construction method; and
- Maintenance.

An evaluation of the effects of each of the factors noted above was carried out and is described within the sections of this chapter and the following chapter. In general, each factor was considered in terms of the two primary performance measures used in this research: permeability; and surface distress development. In evaluating some of the factors, additional measures were also used.

4.1 Planning

The characteristics of pervious concrete pavement provide the ability to perform as a pavement and as a stormwater management alternative. Conventionally two types of infrastructure would be required to adequately meet the needs of a pavement and a stormwater management alternative. In order for pervious concrete to function sufficiently for both of these uses it is important that the objectives of both uses be understood and appropriate planning be performed.

When pervious concrete is used as a pavement it is important to consider the types of traffic that will be using the pavement. Given the high permeability rate of pervious concrete pavement, it can handle active drainage and therefore does not need to be used throughout an entire area in order to provide drainage to the entire area. Using pervious concrete pavement for active drainage can allow for a variety of site layouts and minimize the need for heavy traffic loading on the pervious concrete pavement.

4.1.1 Application

At Sites 1 and 2 the parking spots and the driving portions are clearly defined. At the other field sites, areas are used for both parking and driving or the pervious concrete is only used for one application, such as driving at Site 3. Figure 4-1 shows the permeability of the pervious concrete pavement of Sites 1 and 2 in the two applications.

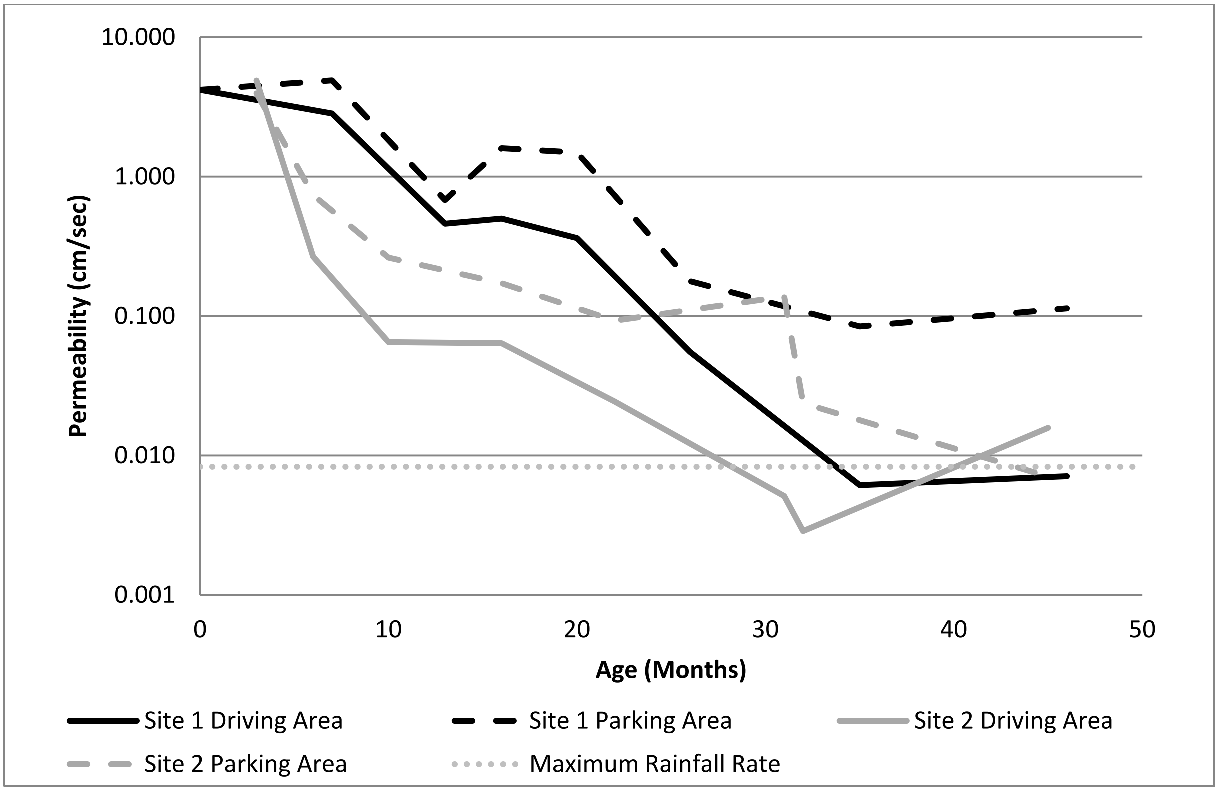


Figure 4-1: Permeability Based on Application

Figure 4-1 shows that the permeability at Site 1 is consistently greater in the parking areas than in the driving areas. At Site 2 the parking area had a higher permeability in general however recently, after three years of age (36 months) the permeability of the driving areas became greater than the parking areas. At this time and before this time the permeability of both areas at Site 2 dropped below the maximum rainfall rate. Average permeability rates below the maximum rainfall rate are deemed to not be adequate as the pervious concrete does not have the capacity to functionally drain the storms in the area.

A t-test was performed with each set of test results to assess if at 95 % confidence the means of the permeability rates from the parking and driving areas were statistically different. Table 4-1 shows the results of the t-tests.

The results shown in Table 4-1 indicate that the presence of a statistical difference in mean permeability between the parking and driving areas fluctuates over time. Initially at Site 1 there is a statistical difference in permeability between the areas used for parking and driving. Following the initial testing, seven months after construction, a statistical difference in the mean permeability of the two areas no longer exists. At Site 2 however there is more fluctuation between a statistical difference in mean permeability rates existing and not existing.

Table 4-1: t-test Application

Site	Age (Months)	T_{calc}	T_{crit}	Outcome
2	3	-0.929	2.080	Statistically the Same
2	6	0.378	2.101	Statistically the Same
1	7	2.164	2.131	Statistically Different
2	10	2.673	2.228	Statistically Different
1	13	0.635	2.074	Statistically the Same
1	15	1.162	2.201	Statistically the Same
2	16	1.427	2.365	Statistically the Same
1	19	0.946	2.228	Statistically the Same
2	22	2.232	2.228	Statistically Different
1	26	1.319	2.306	Statistically the Same
2	31	3.074	2.365	Statistically Different
2	32	2.869	2.306	Statistically Different
1	35	1.338	2.365	Statistically the Same
2	45	-0.693	2.120	Statistically the Same
1	46	1.980	2.447	Statistically the Same

The analysis carried out on pervious concrete in two different applications at two field sites provided an example of how the application can affect the permeability of the pervious concrete. It is possible to achieve both performance and financial benefits by effectively planning the use of pervious concrete within a project. During the planning phase of a project considerations should be made as to the objectives of using pervious concrete pavement, in terms of stormwater management and pavement infrastructure.

4.2 Design

The design phase of a pervious concrete pavement project includes the mix design of the pervious concrete, the characteristics of the granular base storage layer and the entire pavement structure. The mix designs that were used in this project will be compared, the pavement structure that was developed for each field site will be discussed, and the structural performance of the pavement structures will be analyzed in this chapter.

4.2.1 Mix Design

This project was a partnership with industry members across Canada. The development of the mix design for each of the field sites was led by the associated industry partner. The available literature related to pervious concrete pavement mix designs and experiences from previous field sites in the research was provided to the field site partners. The material was discussed and interpreted with the University of Waterloo and the concrete supplier as it pertained to the specific field site. The concrete supplier then developed a mix design that was deemed to be suitable for their local climate, materials and field site application. Table 4-2 shows a summary of the mix designs that were included in this project.

The performance of each mix has been monitored at the field sites throughout this research. In this chapter the characteristics of each mix based on the features of the mix design are compared. During the construction of each field site fresh concrete testing was carried out to monitor changes between loads.

Core samples were taken from each field site. The core samples were tested for various characteristics in the laboratory, including: void content; compressive strength; and density. Samples were also cast during construction. However, there is not yet a reliable method for casting pervious concrete samples that reflects the pervious concrete that is constructed in the field (Rizvi, Tighe, Henderson, & Norris, 2009). The characteristics of the cored samples are used in the analysis in this chapter.

Table 4-2: Mix Design Details of Each Site

Site Section	**Aggregate**		**Fine Aggregate (%)**	**Water to Cement Ratio (W/C)**	**Air Entrainment**	**Fibres**	**Other Admixtures**
	Type	**Size (mm)**					
1	Gravel	13.2	0	0.244	Yes (29.8 ml/m^3)	Yes	Super Plasticizer (350 ml/m^3) Retarder (100 ml/m^3)
2	Pea Gravel	10	0	0.231	Yes (24.9 ml/m^3)	Yes (0.6 kg/m^3)	Super Plasticizer (375 ml/m^3) Retarder (100 ml/m^3)
3A	Felsic/Mafic Volcanics	14	0	0.29	No	No	N/A
3B	Felsic/Mafic Volcanics	14	0	> 0.29	No	No	N/A
4A	Limestone	14	Yes	N/A	No	No	N/A
4B	Gravel	20	Yes	N/A	No	No	N/A
4C	Gravel and Limestone	20	Yes	N/A	No	No	N/A
5A	Granite	14	Yes (7.6 %)	0.286	Yes (250 ml/m^3)	Yes	Super Plasticizer (1.5 l/m^3) Viscosity Modifier (1.5 l/m^3)
5B/5D	Granite	14	Yes (8.2 %)	0.252	Yes (250 ml/m^3)	No	Latex (Styrene Butadene) (34.0 l/m^3) Super Plasticizer (1.5 l/m^3) Viscosity Modifier (1.5 l/m^3)
5C	Granite	14	Yes (7.6 %)	0.286	Yes (250 ml/m^3)	No	Super Plasticizer (1.5 l/m^3) Viscosity Modifier (1.5 l/m^3)
N/A – Not Available							

The test results of the various core samples will be compared in the following section to evaluate where relationships exist based on characteristics of the mix designs. Figure 4-2 compares the void content and hard density relationship of all the core samples.

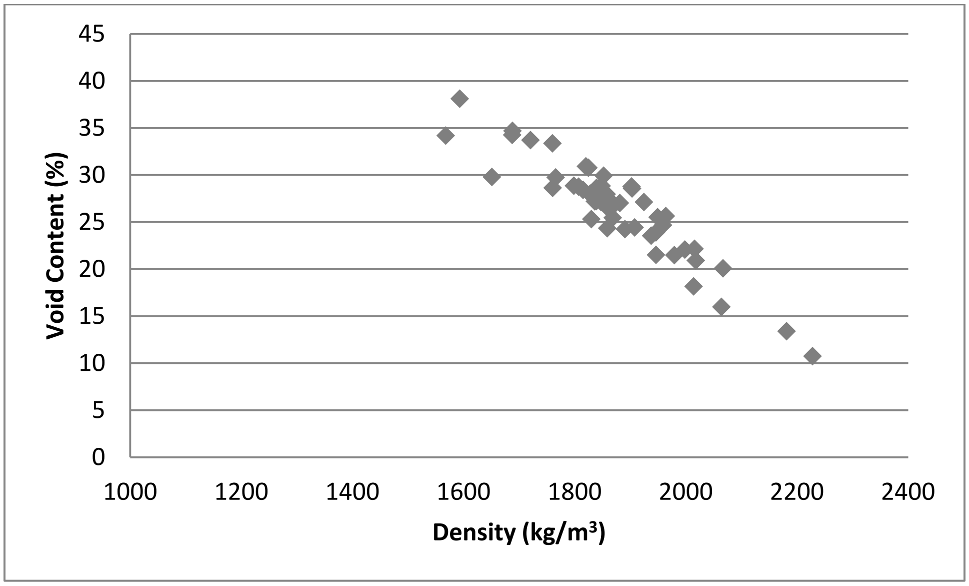


Figure 4-2: Void Content and Density of Cores

Figure 4-2 shows that a relationship exists between the void content of core samples and the hardened density of the core sample. The results presented in Figure 4-2 include the cores from all of the field sites. The data in Figure 4-2 suggests that the trend is applicable to pervious concrete in general and variables such as construction method, mix components and mix ratios do not substantially change the relationship.

4.2.1.1 Fine Aggregate

Initial pervious concrete mix designs did not include fine aggregate. The elimination of all fine aggregate led to a high void content and limited surface area and contact points in the mix. The high void content created a pervious concrete pavement that was more permeable than generally required. To increase the contact area and long term durability of pervious concrete mixes, a portion of fine aggregate was introduced back into the mix design. Figure 4-3 shows the void content and density of the cores both with and without fine aggregate.

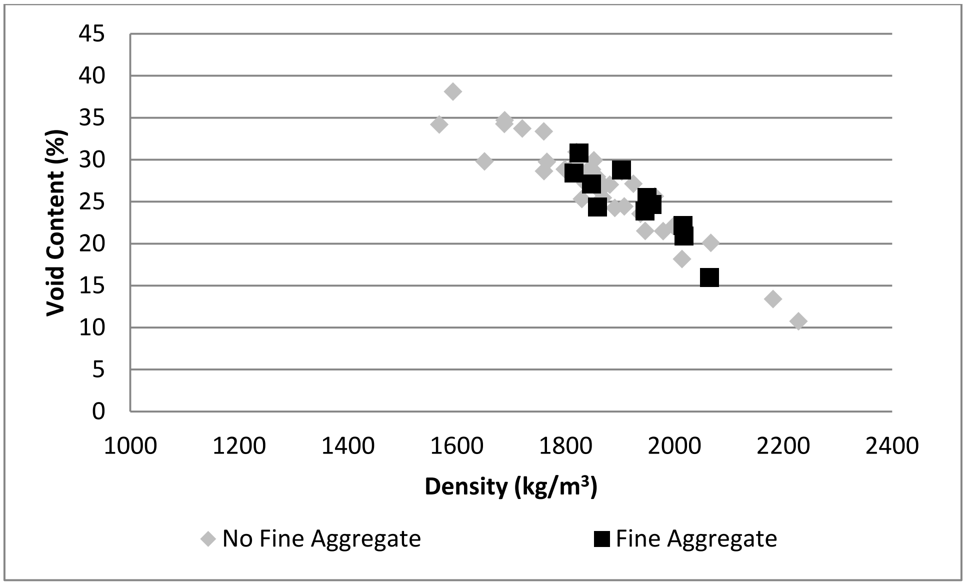


Figure 4-3: Void Content and Density of Cores with and without Fine Aggregate

Figure 4-3 shows that the cores containing fine aggregate were not towards one end of the range of void contents and densities but were within the middle of the range for all the core samples. This result indicates that the inclusion of fine aggregate in pervious concrete mixes can be achieved without the mix having different void content and density characteristics than pervious concrete mixes were no fine aggregate included. It is anticipated that by having fine aggregate in a pervious concrete mix design the durability would be increased as there is more surface area and points of contact.

4.2.1.2 Water Cement Ratio

The water cement ratio (W/C) of a concrete mix affects the characteristics of the mix and the performance. The mix designs used in this project had various W/C, ranging from 0.23 to more than 0.29. The W/C is an important descriptor of the paste in a concrete mix design. The structure of a pervious concrete mix makes the paste characteristics very important. Figure 4-4 shows the relationship between the W/C of pervious concrete cores and the measured void content.

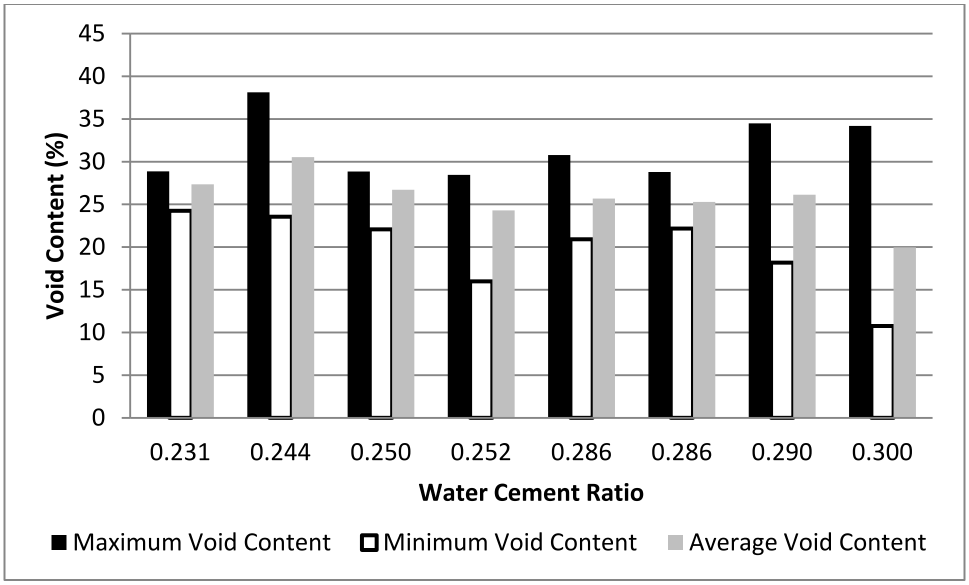


Figure 4-4: Pervious Concrete Water Cement Ratio and Void Content

Figure 4-4 does not show a visible trend in the average void content of the core samples based on the W/C of the mix design. As the W/C increases in the mix design lower void contents are shown to be possible (minimum void content results) however not occurring on average. When the W/C increases the consistency of the paste in a mix is more liquid and therefore can have the tendency to flow throughout the mix rather than coat the aggregate. In cases where the paste flows throughout the mix it is likely that some voids would be lost and filled with paste.

4.2.1.3 Aggregate Type

The mix designs included in this project used three types of aggregates, from various sources. The types of aggregates that were used were felsic/mafic volcanics, crushed limestone, natural gravel and crushed granite. In general the mix included only one type of aggregate. However, one mix design included both crushed limestone and gravel. Crushed materials generally provide more structural integrity than round aggregate. Pervious concrete pavement is not intended for heavy loading however the structure of the material does rely substantially on the characteristics of the aggregate as the void content is significant. Figure 4-5 compares the void content and density of pervious concrete containing various types of aggregate. The aggregate type is considering only the coarse aggregate and not any fine aggregate in the mix.

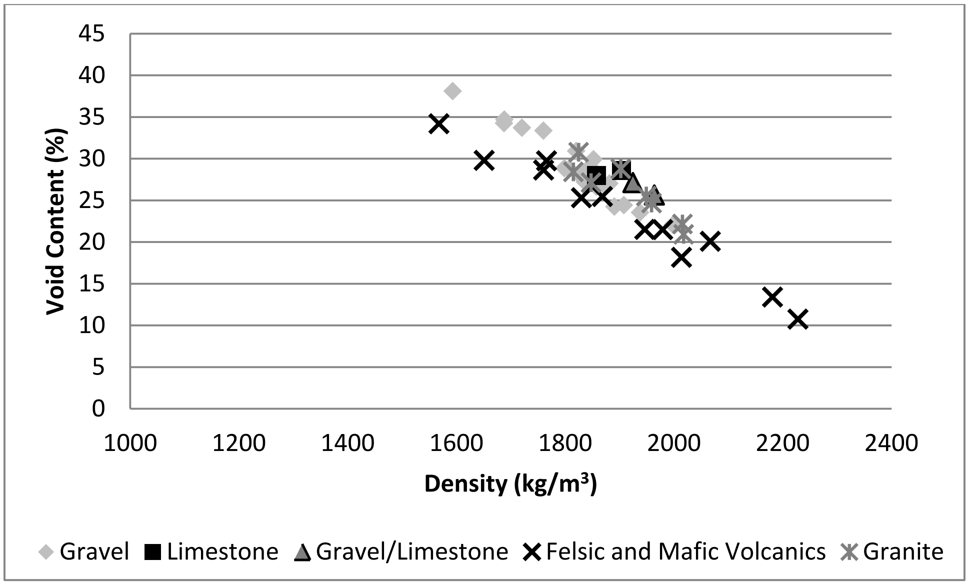


Figure 4-5: Void Content and Density of Cores with Different Aggregate Types

In Figure 4-5 there are some visible differences between the mixes containing different types of aggregate. The cores containing felsic and mafic volcanics have the largest range in void contents and densities. The relationship between void content and density for this type of aggregate appears linear over the entire range. The cores containing gravel have a smaller range of void contents and densities and it is generally linear. The linear relationship of the samples containing gravel is visually slightly different from that of those containing volcanics. The samples containing limestone, gravel and limestone, and granite are all generally along the linear relationship of both the volcanics and the gravel mixes.

4.2.1.4 Aggregate Size

The size of aggregate used in pervious concrete mixes has been observed to affect the characteristics of the voids in the hardened pervious concrete structure, as discussed in Chapter 2.0. Within this project aggregates ranged in size from 10 mm to 20 mm. Figure 4-6 shows the void content and density of the cores based on the size of aggregate used in the mix design.

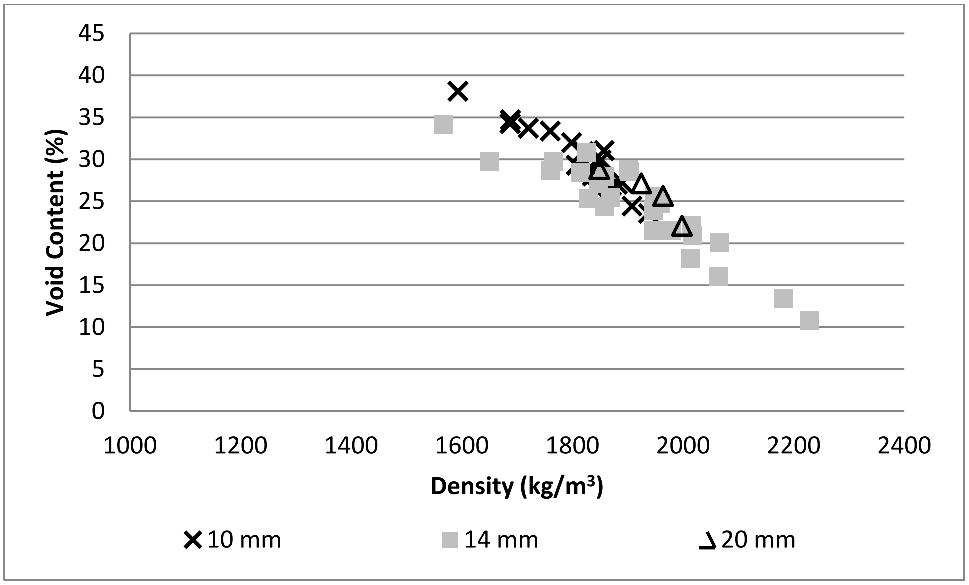
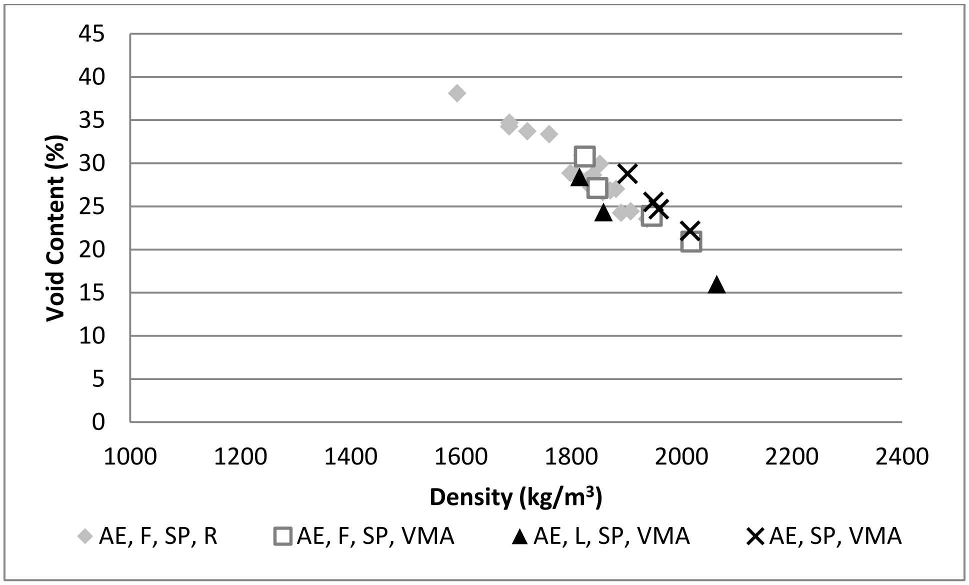


Figure 4-6: Void Content and Density of Cores with Different Sizes of Aggregate

Figure 4-6 shows that each size of aggregate presents a linear relationship between the void content and density of the cores. The relationship appears to be dependent on the aggregate size however. The linear relationship of the 10 mm aggregate visually appears to be slightly different than that of the 14 mm aggregate. There are only a few cores with 20 mm aggregate and therefore it is not as clear as to the characteristics of the linear relationship.

4.2.1.5 Admixtures

Admixtures were used in most of the mix designs and included some of the following: air entrainment; super plasticizer; retarder; fibres; latex (Styrene Butadene); and viscosity modifier. The concrete suppliers included various admixtures as were deemed necessary to create a pervious concrete mix design that was feasible for construction and would have adequate long term performance. The admixtures have potential to alter the characteristics of the mix design such as the void content. Figure 4-7 shows the void content and densities of the mixes containing admixtures.



Note: AE – Air Entrainment; F – Fibres; L – Latex (Styrene Butadene); SP – Super Plasticizer; R – Retarder; VMA – Viscosity Modifer Admixture

Figure 4-7: Void Content and Density of Cores with Admixtures

Figure 4-7 shows a consistent linear relationship between the void content and density of the cores. The combination of admixtures does not appear to affect the relationship. Cores containing a VMA had a tendency to have a higher density and lower void content than those without VMA.

4.2.2 Mix Design Summary

The mix designs used at the field sites were compared on the basis of the various aspects of the mix designs. The relationship between the void content and density for most aspects of the mix designs were compared. The relationship between the void content and hardened density of the pervious concrete cores was linear with none of the cores being visually identified as outliers. The void content and density of the cores was compared in terms of: inclusion of fine aggregate; water cement ratio; aggregate type; aggregate size; and inclusion of admixtures.

When the W/C increased in a mix design the minimum measured void content of the cores decreased. Therefore when the W/C was higher there was a possibility of having a lower void content in the core. The average void content of the cores did not show this trend with changes in W/C.

The type of aggregate in the mix design showed differences in the relationship between the void content and density of the cores. The crushed aggregate (limestone) showed more variability and a less consistent relationship between the void content and density than the other types of aggregate. This is deemed to be due to the increased ability to compact crushed material over natural round material. Depending on the amount of compaction effort during construction, the pervious concrete containing crushed aggregate may have a range in void contents and density characteristics.

The size of aggregate in a pervious concrete mix was found to determine the relationship between the void content and density of the material. Cores with different sizes of aggregate in the mix design had visually different linear relationships between the void content and hardened density.

The other aspects of the mix design that were compared: inclusion of fine aggregate; aggregate type; and inclusion of admixtures did not have specific trends and the relationship between void content and density was uniform.

4.2.3 Pavement Structure

The pavement structure for each field site was developed in collaboration between the University of Waterloo and the associated industry members. The pavement structure for a pervious concrete pavement must meet the demands of both the pavement and stormwater management applications. Pervious concrete pavement structures generally consist of a pervious concrete surface layer; a granular base storage layer; and the existing subgrade.

The surface of the pavement structure, the pervious concrete pavement layer, handles most of the structural loading. In this work, the thickness of this layer was determined using Streetpave, a rigid pavement design software (ACPA, 2012). The characteristics of the pervious concrete pavement were input into Streetpave and different traffic loading scenarios were analyzed.

The granular base storage layer is designed to have the capacity to store the water from recent rain events while it infiltrates into the existing subgrade. The thickness requirement of the granular base layer is therefore a function of the drainage abilities of the existing subgrade and the volume of the rain events common to the local area.

In some pervious concrete projects a geotextile is included in between the granular base storage layer and the existing subgrade. Geotextiles were not used in this project. However, it is a recommendation for future projects. The function of the granular base storage layer is to hold water as it drains into the existing subgrade. The lower portion of this layer is therefore anticipated to be moist. This would also lead to the top of the existing subgrade generally remaining moist. A geotextile would prevent the fines from the subgrade migrating into the granular base layer and the granular material mixing into the subgrade. The mixing of the subgrade material and granular base layer could decrease the structural integrity of the pavement structure and reduce the storage capacity of the base layer.

The pavement structure should not only be effective in terms of pavement performance and stormwater management but should also be designed such that construction is feasible. Alternatives in the pavement structure that were included in this project to make the construction of a field site feasible are discussed later, in Chapter 5.0.

4.2.3.1 Pavement Structure Designs

A sensitivity analysis was carried out during the design of the pavement structures for each of the field sites. Table 4-3 shows a summary of the final pavement structures that were constructed at each field site.

Table 4-3: Field Site Pavement Structures

Material Layer	**Layer Thickness (mm)**				
	Site 1	**Site 2**	**Site 3**	**Site 4**	**Site 5**
Pervious Concrete	300	240	250	175	200
Granular Reservoir Base	600 (100 concrete mudslab)	300	200	350	200 (Popcorn Concrete)
Total	*1000*	*540*	*450*	*525*	*400*

In general the mix design at each field site was developed with the intent of handling a similar amount of traffic. Based on the pavement structures in Table 4-3 it is anticipated that under comparable loading there could be varied structural performances at the field sites. The pervious concrete layer is intended to handle most of the structural loading and between the five field sites the thickness of the pervious concrete layer ranges from 175 mm to 300 mm. The thickness of the granular reservoir base layer is primarily determined by the anticipated rain events of the location and the characteristics of the subgrade. The base layer can additionally provide some structural support to the pavement structure. The following section compares the structural performance of the five field sites.

4.2.3.2 Structural Integrity of Field Sites

As noted earlier, the pervious concrete pavement in this project was not designed to carry heavy or high volume traffic but rather low speed, low volume traffic. Although the demands from the traffic are not substantial in comparison to other types of pavements, it is important that the pervious concrete pavement remain structurally sound throughout the design life. Changes in the structural condition of the pavement at each field site were monitored with Portable Falling Weight Deflectometer (PFWD) testing. PFWD testing is described earlier in Chapter 3.0. The general function of PFWD testing includes dropping a 20 kg weight on the pavement and a sensor measures the deflection of the pavement directly below the loading. PFWD testing was carried out multiple times at each field site throughout this project. Figure 4-8 shows an example of the PFWD testing results from Site 5. The testing is repeated at least 20 times at each site. The test locations are recorded or marked on the pavement surface so that testing can be repeated in the same locations. A figure for each of the five field sites, similar to that shown in Figure 4-8 is included in Appendix C.

PFWD testing is repeated at least six times at each test point in order to achieve a representative result. For each of the test points the data was checked for outliers. Outliers were identified using the 25th and 75th quartile. Using Equation 4-1 and Equation 4-2 outliers were identified and removed from the dataset.

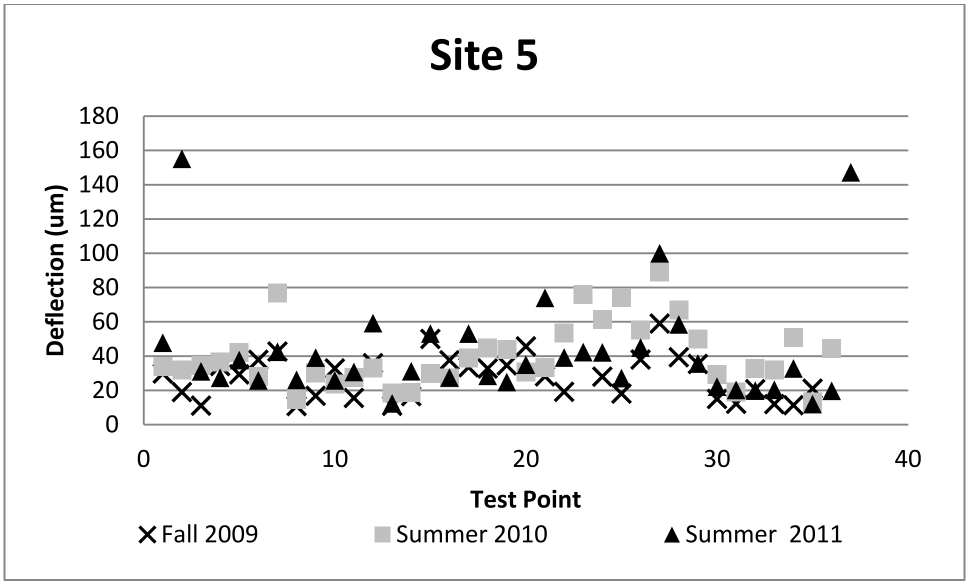


Figure 4-8: PFWD Results at Site 5

$$L_L = Q_1 - 1.5(Q_3 - Q_1) \quad \text{Equation 4-1}$$

$$U_L = Q_1 + 1.5(Q_3 - Q_1) \quad \text{Equation 4-2}$$

Where,

L_L is the lower limit;

U_L is the upper limit;

Q_1 is the 25th percentile; and

Q_3 is the 75th percentile;

Data points less than the L_L value or greater than the U_L value for a particular test point were deemed to be outliers and were removed from the data set. The cause of the outlier values is generally a result of the testing procedure itself. The PFWD may not have been entirely flush on the pavement surface or have moved as the load was dropped. Readings can be taken by the PFWD when it is being aligned to begin a new test point and this data is not representative of the pavement in that location. The plots in Appendix C show the average test point deflection during each session of testing after otliers were removed.

The PFWD testing results were used to assess the following three areas:

1. Did the structural capacity of a field site change over time?

2. Did the field sites have different structural capacities?
3. Did localized areas within a field site change in structural capacity over time?

Using statistics, each of the three questions noted above were assessed from the PFWD data. The first question was initially considered by visually interpreting the five plots shown in Appendix C, which are all the same format as Figure 4-8. Visually distinct trends were not apparent however some observations were noted. At Site 1, in Fall 2008, Spring and Summer 2009 the deflections were similar. In Spring 2011 deflections in many of the areas had increased from the previous sessions of testing.

At Site 2 large changes were visually visible in the figure shown in Appendix C. Portions of the field site sounded hollow in the Spring and Summer of 2011 when a small load was applied to the surface, even a load less than the PFWD weight. This observation is assessed further in the analysis of the third question. Overall across Site 2, the deflections measured in Summer 2011 were higher than those measured in Summer 2009. It anticipated that the hollow noise was coming from the movement of the pervious concrete when a small load was applied to it. The pervious concrete may have been able to move and sound hollow due to loss of support underneath it. The loss of support could be due to the pervious concrete at the bottom of the slab breaking down or the granular base layers settling due to a loss of subgrade material.

At Site 3 the deflections have become more consistent across the site overtime. In both testing sessions in 2009 there were some areas with higher deflections than the rest of the site. The results of the testing in 2010 and 2011 were more consistent across the site and were similar in value to the areas with less deflection in 2009.

At Site 4 there is a consistent pavement structure throughout the site. However, there are three different mixes. Section 4A corresponds to test points 1 through 10, 4B corresponds to test points 11 through 20 and the third area, 4C corresponds to test points 21 through 29. In the area 4A the test results were consistent during all of the testing sessions. In the other two areas there is a larger range in the deflection measurements, even within one testing session. In Summer 2009 and Spring 2011 the deflections were the largest. In the Summer 2009 testing there were several areas with higher deflection and in Spring 2011 only a few areas had higher deflections. The testing in Fall 2009 and Fall 2010 did not have any deflections as high as in the other two testing sessions.

Site 5 is similar to Site 4 in that there are multiple mixes present but the pavement structure is consistent. Sections 5A, 5B and 5D have consistent deflection results from all of the testing sessions. Area 5C, test points 21 to 30, has a wider range in deflection results, with the highest deflections occurring during the Summer 2010 testing.

The Kruskal-Wallis (KW) Test was used to statistically compare the data sets for each field site. The data from each of the testing sessions was compared to evaluate if the data was all from the same population or different populations. The null hypothesis of the test was that the data from each testing session for a particular field site was all from the same population, therefore suggesting no difference in the structural condition of the field site over time. If the null hypothesis was rejected then the data sets were deemed to not be from the same population.

Table 4-4 presents the results of the KW test which was performed for each field site and twice for Site 2. At Site 2 the analysis was carried out on the data from the entire site (entire site) and then on the data from the areas that appeared to have changed structurally (structurally different areas). The calculated value from the KW test (H_{calc}) is compared to the critical value from the Chi-Squared distribution (H_{crit}) for 95% confidence and the associated degrees of freedom.

Table 4-4: Comparison of Structural Condition of Field Site Over Time

Site	H_{calc}	H_{crit}	Outcome
1	3.625	5.991	Statistically Same
2 (Entire Site)	5.255	11.070	Statistically Same
2 (Structurally Different Areas)	6.555	9.488	Statistically Same
3	5.668	7.815	Statistically Same
4	64.656	7.815	Statistically Different
5	8.567	5.991	Statistically Different

Table 4-4 shows that the KW test results proved that at Sites 4 and 5 the structural condition of the field site based on the collected data was changing between the different testing sessions as the H_{calc} was larger than the H_{crit}. The results of Sites 1, 2 and 3 failed to prove that the structural condition of the field sites was different between different testing sessions.

The mean of each testing session for a particular field site was compared to evaluate if they were changing over time. This analysis would identify changes in terms of either decrease or increases in deflection. An ANOVA table was calculated for each field site to assess whether the means of the various testing sessions were significantly different. This was carried out at a 95 % confidence level. At field sites that demonstrated significant differences in PFWD mean test results over time, the 95 % confidence interval was calculated for pairings of testing sessions. This comparison considered if two testing sessions had consistent results or if this was the time in which change in structural capacity was likely to have occurred, either improvement or deterioration of structural condition. Table 4-5 shows the F tests from the ANOVA tables and the confidence intervals for each applicable pair of test sessions for a field site. The ANOVA table for each field site is in Appendix C.

Table 4-5: Differences of PFWD Means at Field Sites Over Time

Site	**All Testing Sessions**		**Outcome**	**Session Comparison (Age)**	**Confidence Interval**		**Outcome**
	F_{Calc}	**F_{Crit}**					
1	3.569	3.117	Significantly Different	46 Months and 26 Months	2.19	29.08	46 Months of Age Likely More Deflection
				26 Months and 17 Months	-12.59	14.60	No Difference Includes 0
2*	-2.93 (t_{calc})	2.056 (t_{crit})	Significantly Different	* 2009 results compared to 2011 results with a t test at 95 % confidence			
3	2.307	2.732	Statistically Same	Not Applicable			
4	6.988	2.696	Significantly Different	31 Months and 24 Months	-2.34	29.71	No Difference, 31 Months may have more deflection
				24 Months and 13 Months	-8.38	23.66	No Difference, 24 Months may have more deflection
				13 Months and 8 Months	-51.10	-18.77	More deflection at 8 Months of Age Possible
5	3.406	3.111	Significantly Different	22 Months and 12 Months	-11.55	10.87	No Difference Includes 0
				12 Months and 1 Month	1.78	24.83	More deflection at 12 Months of age than 1 Month

Table 4-5 shows that the ANOVA results indicated Sites 1, 2, 4 and 5 all changed in structural condition in between the testing sessions. When changes in structural condition changed at a field site it was an increase in deflection with time. An increase in deflection indicates a decrease in structural capacity. It is anticipated that structural capacity would decrease if there is a lack of support in the pavement structure, either due to a loss of material such as subgrade of cracking or ravelling occurring in the pervious concrete slab. Without taking cores or boreholes it was not possible to know which of these scenarios was occurring. Site 1 showed a change in deflection between the testing at 26 months and 46 months. The confidence interval at 95% indicates that the deflection results were greater at 46 months than at 26 which indicates a deterioration in the structural capacity of the pavement. This trend is logical, assuming that the pavement deteriorates over time.

Sites 4 and 5 show a different pattern than Site 1. However, both have more test results from earlier ages and are not as old as Site 1. Changes in deflection occurred between 8 months of age and 13 months of age at Site 4, with larger deflections occurring at 8 months of age based on the 95 % confidence interval. This conclusion could be a result of the testing conditions but this is not deemed to be the reason as the 8 month testing was done during summer months which are not generally the worst conditions, the spring is anticipated to be. Compaction of the pavement structure may have been ongoing at 8 months of age and this lead to higher deflection measurements. Between 13 and 24 months of age and 24 and 31 months of age the 95 % confidence interval does not suggest that difference in deflection measurements exist. Both intervals include zero, indicating that there may not be any difference between testing sessions. Most of the interval is positive though, therefore deflections may have increased from the previous testing sessions.

At Site 5, between 1 month and 12 months after construction the deflection measurements increased. Between 12 months and 22 months the 95 % confidence interval does not indicate that the deflection measurements changed. These two conclusions suggest that the same behaviour may be occurring at Site 5 and Site 1 but with the given data sets it is not possible to confirm this. This behaviour is realistic however, suggesting ongoing compaction shortly after construction and then a stabilization in the pavement structure. The testing sessions were not done at the same times at Site 4 as Site 5. However, the same behaviour is apparent, larger deflections initially following construction and then more consistent results later, suggesting that the pavement structure has stabilized.

The data presented in Table 4-5 suggests that some trends may be present between the field sites in terms of the structural behavior of the pervious concrete pavement structures. The following analysis addresses the second question, “is the structural capacity different between the field sites?”. The PFWD data for all field sites was divided into five groups based on the age at which the pavement structure was tested. The five groups are the following and include the test results from the noted field sites:

1. Eight Months After Construction;
    - Site 3 at eight months; and
    - Site 4 at eight months.
2. 12 to 18 Months After Construction;
    - Site 1 at 17 months;
    - Site 3 at 18 months;
    - Site 4 at 13 months; and
    - Site 5 at 12 months.
3. 22 to 24 Months After Construction;
    - Site 2 at 22 months;
    - Site 4 at 24 months; and
    - Site 5 at 22 months.
4. 26 to 31 Months After Construction; and
    - Site 1 at 26 months;
    - Site 3 at 28 months; and
    - Site 4 at 31 months.
5. 37 to 48 Months After Construction.
    - Site 1 at 46 months;
    - Site 2 at 48 months; and
    - Site 3 at 37 months.

The purpose of dividing the test results by age was to further assess if all the field sites were following a similar trend in terms of structural performance. For each of the five groups the PFWD deflection data was plotted and compared statistically. Figure 4-9 shows a plot of group 3, ages from 22 to 24 months and the results from the three testing sessions that were carried out.

The comparable figures to Figure 4-9 for each age group are included in Appendix C. The three sets of data presented in Figure 4-9 each visually have slightly different characteristics. The statistical analysis including ANOVA tables for each age group assisted in assessing if each field site was structurally different at similar ages. The results of statistical analysis are shown in

Table 4-6.

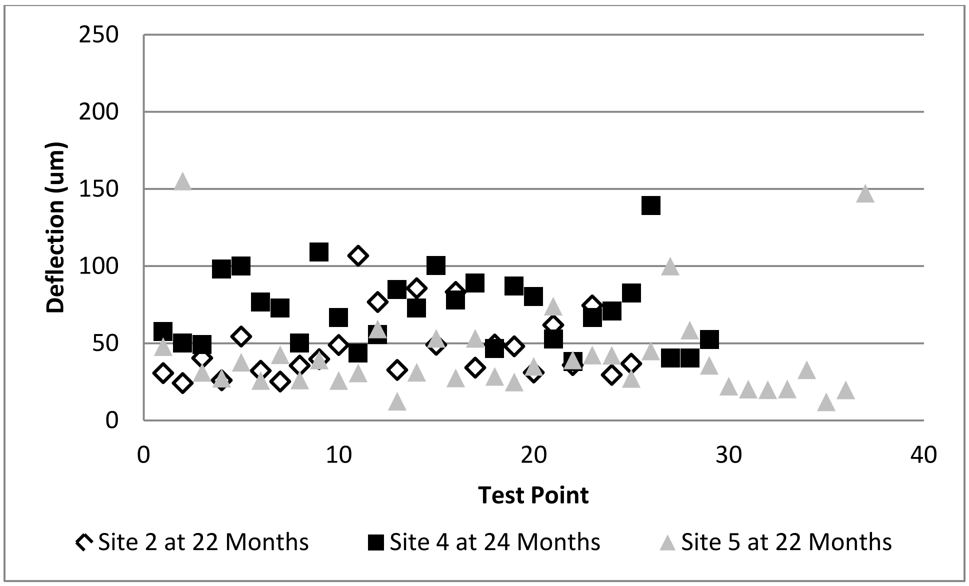


Figure 4-9: PFWD Results from Sites 2, 4 and 5 at 22 to 24 Months

Table 4-6: Evaluation of Pavement Structures at Comparable Ages

Testing Group	All Testing Sessions		Outcome
	F_{Calc}	F_{Crit}	
1 (8 Months)	0.069	3.209	Statistically Same
2 (12 to 18 Months)	6.572	2.696	Significantly Different
3 (22 to 24 Months)	9.663	3.100	Significantly Different
4 (26 to 31 Months)	17.631	3.119	Significantly Different
5 (37 to 48 Months)	3.191	3.134	Statistically Different

Table 4-6 shows that the test result means at eight months of age were statistically the same. However, the results in each of the other four groups were significantly different. The outcome of the statistical tests suggests that at the same age or similar ages the field sites do not necessarily have the same deflection response from PFWD testing and therefore likely have different structural capacities.

Finally the condition at Site 2 was investigated to assess the changes and differences in structural condition at Site 2, question three, “did localized changes occur within the field site?”. As previously noted, it was identified during a surface condition evaluation that portions of Site 2 appeared hollow in the Winter of 2011. The areas that sounded hollow under loading from a light weight also appeared to be surrounded by a crack or abnormality in the surface. The cracks or abnormalities were visible and were a line of ravelled material. It is anticipated that this area cracked and ravelled material from the cracking had a tendency to gather on the cracks. The hollow areas and cracks were not consistent in shape but tended to be random. They were located in two areas of Site 2, the northwest corner of the site, this area had a small hollow area, and over the drainage pipe that runs the width for the field site. The hollow areas fluctuate around the location of the drainage pipe for the entire width of the site.

Testing was carried out to monitor the changes of the hollow areas following the identification of them. Changes were monitored visually and by carrying out PFWD testing. All of the PFWD testing results for Site 2 are shown in Figure 4-10.

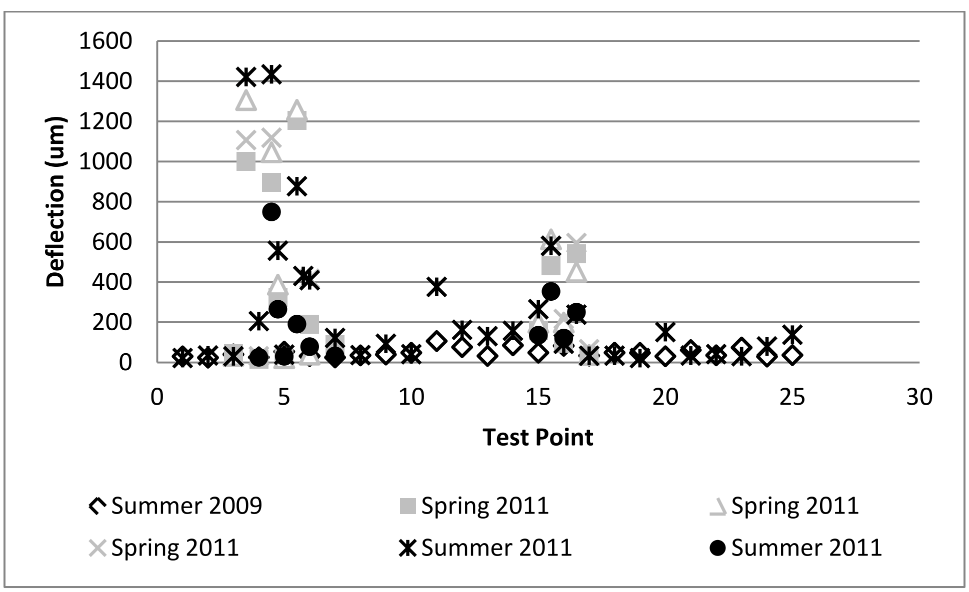


Figure 4-10: PFWD Results at Site 2

In Figure 4-10 the data appears to be divided into three groups. The consistent results below a deflection of 200 µm, data around test point five that ranges up to 1400 µm and data around point 15 that goes to 600 µm. Test point five and test point 15 are the locations where the pavement structure was identified as sounding hollow during site visits. The PFWD testing results also identify those areas as having different behavior than the rest of the site

4.3 Summary

Each field site included a different mix design, some of which had multiple mix designs. The relationship between the void content and hardened density of the pervious concrete cores was linear with none of the cores being visually identified as outliers. The void content and density of the cores were compared in terms of: inclusion of fine aggregate; water cement ratio; aggregate type; aggregate size; and inclusion of admixtures. When the W/C increased in a mix design the minimum measured void content of the cores decreased. Therefore when the W/C was higher there was a possibility of having a lower void content in the core. The average void content of the cores did not show this trend with changes in W/C. The type of aggregate in the mix design showed differences in the relationship between the void content and density of the cores. The size of aggregate in a pervious concrete mix was found to determine the relationship between the void content and density of the material. Cores with different sizes of aggregate in the mix design had visually different linear relationships between the void content and hardened density. The other aspects of the mix design that were compared: inclusion of fine aggregate; aggregate type; and inclusion of admixtures did not have specific trends and the relationship between void content and density was uniform.

The general pavement structure at the field sites was similar with very slight variations in the layer thicknesses. Substantial deterioration in pavement structural performance was identified at one site, Site 2. At Site 2, localized areas of reduced structural capacity developed in the spring of 2011, approximately 2.5 years of age. It is anticipated that these areas developed due to loss of supporting material in the pavement structure. Sites 1, 4 and 5 showed changes in structural capacity over the monitoring timeline. However, no locations of substantial decreases in structural capacity were identified. When the structural capacity of various field sites was compared based on age it was found that field sites of the same age generally had statistically different structural capacities. The results demonstrated that even under personal vehicle loading the structural design of a pervious concrete structure should be considered as differences can exist.

5.0 CONSTRUCTION

The field sites included in this project were constructed with a variety of methods. This section analyzes the pervious concrete pavement characteristics that resulted from each construction method. Many aspects of construction were considered in this work and include the following:

- Pervious concrete source;
- Construction method; and
- Joint construction method;

Alternatives for curing the pervious concrete were not covered in this work. All of the field sites were cured with the same method, placement of a plastic sheet on the surface for at least seven days with only loading from pedestrians during this time. Loading from pedestrians was kept to a minimum and generally only occurred during the construction of the adjacent pavement. Although only one method of curing was included in this work, some comments related to the curing have been included within this chapter.

5.1 Source

Pervious concrete pavement was supplied to the construction site in two methods in this project: common concrete truck with a drum mixer; and a volumetric or mobile mixer. The common concrete truck was used at Sites 1 through 4 and the volumetric mixer was used at Site 5. The common concrete truck was loaded at the plant and delivered the ready mixed concrete to the site. The volumetric mixer arrived on site with all of the components of the pervious concrete mix and mixed the required quantity on demand. Both types of trucks were effective and adequately supported the construction of the pervious concrete pavement. Some challenges arose and were overcome, those are discussed below.

Given the properties of pervious concrete pavement it discharges slowly out of either type of concrete truck. In many cases the crew assists in moving it down the truck chute to make the process more efficient. At Site 4, where an asphalt paver was used to construct the pavement (discussed further in following sections) the pervious concrete mix did not flow out of the concrete truck at a high enough rate to keep the necessary quantity of mix in the paver. This was overcome by having trucks feed into a front end loader and the loader then placed the pervious concrete mix into the asphalt paver. This unfortunately presents an additional step in the construction process and the requirement of additional equipment but was effective in the construction of Site 4. The slower supply of pervious concrete mix from either type of concrete truck is not an issue when the construction is being carried out with a roller.

Another challenge with both types of concrete truck is accessing the construction site. The granular base storage layer generally does not provide substantial structural support for loading of concrete trucks. The stabilized open graded concrete base at Site 5 was a suitable alternative to the granular base storage layer and had no challenges in adequately supporting the load of the volumetric mixers. At Site 3 the pervious concrete portion was only 1 m in width and therefore it was not necessary that the trucks drive on the base layer. At Sites 1, 2 and 4 there was concern about trucks getting stuck in the granular base layer. In the end the construction of Site 1 and 2 was completed without the trucks being stuck and at Site 4 the system was changed to the front end loader. Since the construction of these field sites, other pervious concrete sites have been constructed using conveyors, therefore eliminating the need for the concrete trucks to go on the granular base layer.

5.2 Method

The pervious concrete in this project was constructed using various construction methods. In all cases the pervious concrete was initially placed from the chute of the concrete truck and then leveled or compacted. As noted in the previous section, the process of unloading the pervious concrete at Site 4 was revised part way through construction. At Sites 1, 3 and 5 the concrete was compacted using a roller. At Sites 1 and 3 the concrete was leveled to approximately 12 mm above the desired final height and then compacted with the roller. At Site 5 the roller was on tracks, therefore ensuring the desired height was achieved and was followed by a vibratory plate compactor.

At Site 2, two methods were used for the construction of the pervious concrete. In one area of Site 2 the concrete was compacted using a Razorback paver while in the other area it was only leveled, which was done with a Bidwell bridgedeck paver.

At Site 4 an ABG asphalt paver was used to place the pervious concrete and was followed by a vibratory plate compactor. The observations from each of the field sites based on the construction method used are described below. Finally, the performance and characteristics of the pervious concrete pavement related to the various construction methods is presented.

It has been noticed in some cores extracted from the field sites that the top of the core appears denser, therefore visually appearing to have fewer voids than the bottom of the core. This is anticipated to be a function of the mix design and the construction method. Some of the cores were cut into slices in order to measure the voids in these various areas. Void contents of cores from Sites 1 and 3 are provided in Table 5-1. Cores from Sites 2 and 4 did not visually show variable densities. Cores from Site 5 also showed varying densities. However, were not sliced for testing as they were used in other types of analysis.. Figure 5-1 shows an example of a core from Site 5 and the visible differences in density.

Figure 5-1: Varying Density in a Core from Site 5

Table 5-1 shows that the differences in void content within one core ranged from 2.7 % to 20.8 %. The cores from Site 3 labeled 1U# are from the entrance portion of the field site (3A) and those labeled 2U# are from the exit portion (3B).

Table 5-1: Void Content Differences Throughout Cores

Field Site	Core	Slice	Void Content (%)	Maximum Void Content Different Throughout Core (%)
Site 1	GT4	Top	29.9	4.8
		Bottom	34.7	
	GT24	Top	29.3	2.7
		Bottom	32.0	
	GT1	Top	30.9	3.3
		Upper Middle	33.4	
		Lower Middle	34.3	
		Bottom	33.7	
Site 3	1U1	Top	28.6	1.2
		Bottom	29.8	
	1U2	Top	25.5	9.0
		Middle	34.5	
		Bottom	29.8	
	1U3	Top	20.1	5.2
		Middle	21.5	
		Bottom	25.3	
	1U4	Top	18.2	10.4
		Bottom	28.6	
	2U1	Top	10.8	10.7
		Bottom	21.5	
	2U2	Top	13.4	20.8
		Bottom	34.2	

Figure 5-2: Core 2U1 from Site 3

Figure 5-2 shows a photo of one of the cores from Site 3. Unfortunately the permeability of neither portion of Site 3 has remained adequate and it is anticipated that the top portion of the top half of the cores has even fewer voids, in some cases is sealed. It is critical that the surface have voids available for water to drain through.

5.3 Joints

Concrete pavements generally include joints, similar to many applications that use concrete. The joints provide a location for cracks to develop in the concrete. Cracks are likely to occur as the concrete shrinks. In the case of pervious concrete in this project, joints were handled with three techniques:

1. Formed;
2. Saw cut; and
3. No joints.

Joints that were formed were done so using a joint former, also referenced as a "pizza cutter". Figure 5-3 shows a joint former in use.

Figure 5-3: Joint Former

At Sites 1, 2 and 3 the joints were constructed using a joint former. The joints are formed within minutes of the pervious concrete being placed.

At Site 5 and at some locations at Site 2, the joints were saw cut into the pervious concrete. This was performed within 24 hours of the placement of the pervious concrete. At Site 5 concrete was placed and then compacted with a roller and then a vibratory plate compactor. Plastic was placed on the surface immediately following the compaction by the vibratory plate compactor. The plastic was removed to saw cut the joints and then placed back on the surface.

At Site 4 there were no joints formed or cut into the pervious concrete. The only joints were construction joints which were not designed to behave as joints for crack locations.

5.3.1 Performance

The joints provide a functional purpose in that it is intended that cracks will develop at them instead of randomly across the pavement. As pervious concrete in this project is for low volume, low speed applications, the performance of the joints is part of the general performance of the pavement, rather than the functionality. When concrete pavements are used on highways or higher speed facilities, the deterioration of a joint affects the ride quality experienced by the user as a large bump could occur. In low speed applications, such as the pervious concrete field sites, it is not anticipated that users would be travelling at high speeds and thus a mildly deteriorated joint would not be as disruptive to the user.

The visible surface distress is therefore the problem with poor joint performance in pervious concrete. Examples of visible surface distresses are ravelling or cracks developing off of the joints and can create a surface that is not in the condition desired by the owner. If ravelling continues and becomes extensive it can create deep potholes which could be damaging to vehicles and an inconvenience to users. It is not likely that these potholes would only be in the area of the joint; however, they would likely occur throughout the pavement surface, at least encompassing areas beyond the joint. The following sections present photos of the performance of joints constructed with the various previously noted methods as well as cracks that have been noted in pervious concrete pavements with and without joints.

5.3.1.1 Formed Joints

Figure 5-4 to Figure 5-7 show the performance over multiple years of joints that were formed during construction at Site 1.

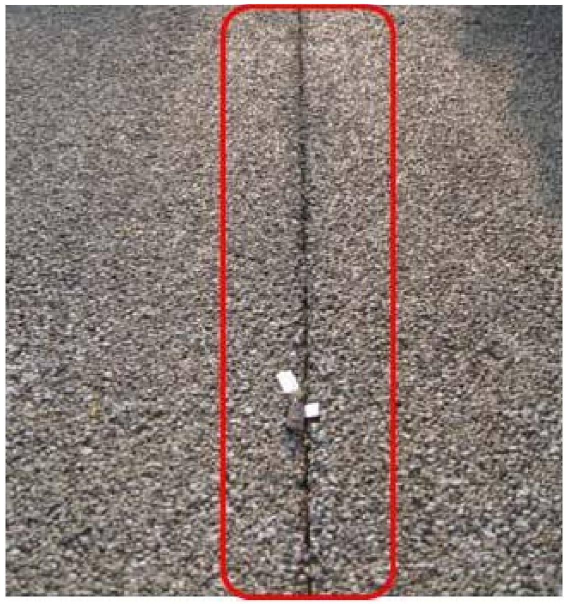


Figure 5-4: Formed Joint after Construction Site 1 October 2008 (16 Months)

Figure 5-5: Formed Joint August 2009 Site 1 (26 Months)

Figure 5-6: Formed Joint March 2010 (33 Months) Site 1

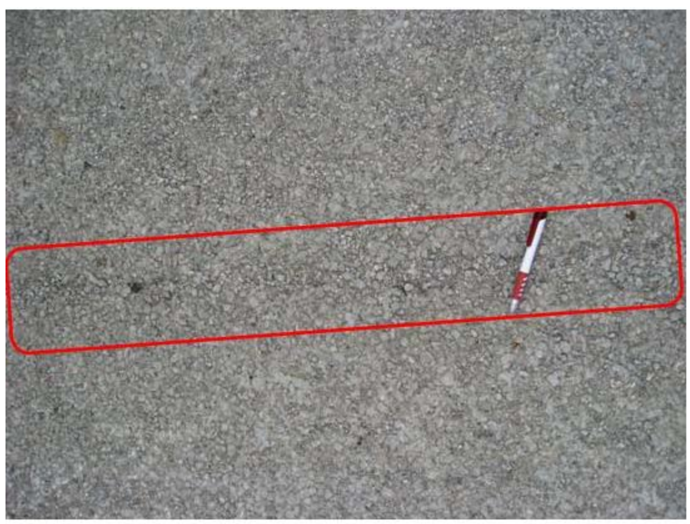


Figure 5-7: Formed Joint April 2011 (46 Months) Site 1

In general, the figures show deterioration at the joints over time. Deterioration is seen in the form of ravelling for the most part. As the ravelling occurs the loose aggregate collects in the joint area, making it challenging to see the joint itself, such as in Figure 5-7. Although the joint has ravelled and the area surrounding the joint has also ravelled, debris from the surrounding environment and users has collected on the pavement. The joint provides a location for debris both from ravelling and adjacent areas to collect. The debris collected in the joint may lead to further deterioration of the joint however. Generally joints in concrete pavements need to be clean in order to provide the needed space for the pavement to move under changing climatic conditions. If the joint is filled with debris then there is no space for the pavement to move and this may lead to additional ravelling and distress development.

The process of forming the joint is one of the primary reasons for the ravelling being initiated. A mix that is more prone to ravelling would also attribute to the ravelling of the joint. In some cases the forming of the joints leads to the poor performance. The joints are formed after the pervious concrete has been compacted. By forming a joint, aggregate is being moved from the location it was in following compaction. Therefore bonds that were initiated with the compaction have the potential to be broken, resulting in low strength bonds between aggregate. The presence of low strength bonds can result in aggregate ravelling earlier than it would have had it not been disturbed or it may not have ravelled at all had it not been disturbed.

5.3.1.2 Saw Cut

When a joint is saw cut the aggregate is cut with the blade of the saw to create an open space. Disturbance is experienced by the aggregate and bond between surrounding aggregates due to the forces created from the saw blade. However, no aggregates are moved as is done when the joint is formed. In order to minimize damage to the pervious concrete the joints are not cut until the pervious concrete has had adequate time to cure and gain strength. It is important though, to sawcut the joints before cracks develop in the pervious concrete due to shrinkage. If cracks have already initiated development then the joints will not serve their intended purpose, to provide a location for cracking. Joints were saw cut at Site 5 and at some locations at Site 2. Figure 5-8 to Figure 5-10 show the performance of the saw cut joints at Site 5.

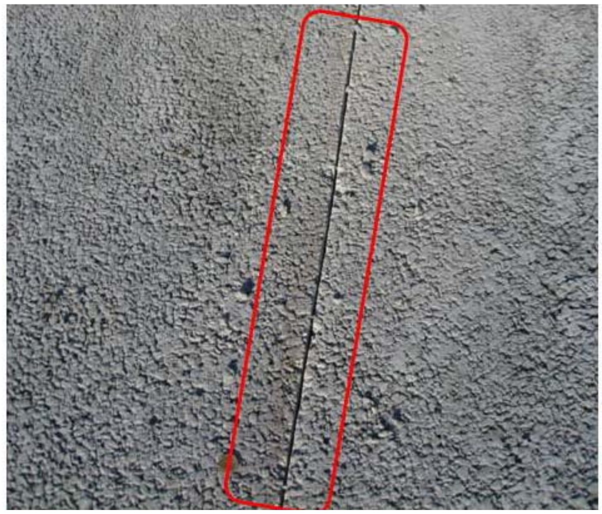


Figure 5-8: Saw Cut Joint After Construction Site 4 August 2009 (10 Months)

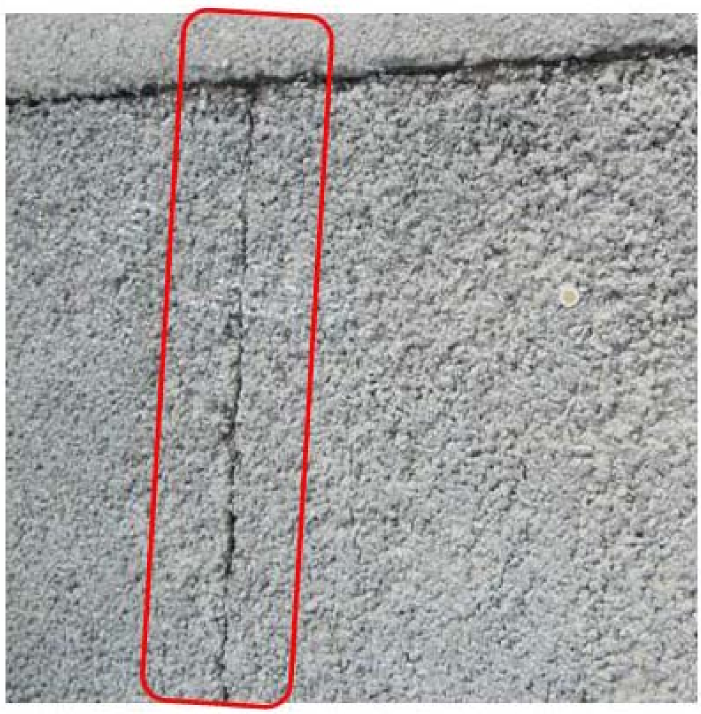


Figure 5-9: Saw Cut Joint Site 5 August 2010 (22 Months)

Figure 5-10: Saw Cut Joint Site 5 June 2011 (32 Months)

Figure 5-8 to Figure 5-10 show the development of some ravelling at the joints. In Figure 5-9 there is not only ravelling at the joint but also in the adjacent slabs. In Figure 5-10 it is challenging to assess the amount of ravelling due to the debris in the joint. The saw cut joints remain more clearly visible in the figures than the formed joints. This is likely a combination of the amount of debris at the site and also the ravelling at the joints. When less ravelling occurs at the joints it is anticipated to remain more visible than if a lot of ravelling occurs, adding to the debris and loose aggregate.

5.3.2 Construction Joints

Construction joints are joint locations that occur for constructability purposes. They can occur at the location where work ends on one day and is resumed the next day or when the pervious concrete is constructed in adjacent sections, either in one day or over several days. In some cases no construction joints are needed within a pervious concrete project. Sites 3 and 5 are examples of this scenario. No construction joints were required as at both sites the pervious concrete was placed in one continual pass and on one day. The number of passes required to place the pervious concrete is dependent on the width of the area being constructed and the equipment being used in the construction.

In the case when only one pass is required to construct the pervious concrete then a joint is formed between the pervious concrete and the adjacent materials, which could be asphalt, concrete curb, concrete pavement or other materials. Pervious concrete can be constructed with the adjacent material being soil with vegetation, such as grass.

Construction joints can be handled using various methods with the key factors for consideration including:

- Complete construction method is carried out to the end of the paved section;
- Pervious concrete pavement edges are provided adequate support to maintain shape during curing; and
- Sufficient time is allowed for pervious concrete to cure and gain strength before being loaded or disturbed by adjacent construction.

At Sites 1, 2 and 4 multiple passes were required to complete the pervious concrete paving. Sites 1 and 2 were paved over multiple days for constructability. This provided the edges of the pervious concrete and what would become construction joints, an opportunity to cure and gain strength before being used as forms for the adjacent sections on the following days. At Sites 1 and 2 no visible difference can be seen in the performance of joints which were construction joints and joints formed for the purpose of creating a location for crack development. Site 4 however, shows considerable distress development at the construction joints. Site 4 was constructed in three passes, with each pass using a different pervious concrete mix, 4A, 4B and 4C. The construction joint between 4A and 4B is noted as A/B and the other, between 4B and 4C is noted as B/C. Construction included the use of an ABG asphalt paver, followed by a vibratory plate compactor. Upon the completion of one pass the adjacent pass was started immediately. Since an asphalt paver was used, there was no need for forms along the edges of each pass. The paver provides support and forms edges as the pavement is placed.

By placing the adjacent pervious concrete pass before allowing for curing the pervious concrete from the previous pass was weak and vulnerable to disturbance. Disturbance to the edge of the prior pass was inevitable as the intent was to place the next pass flush against the edge. This scenario was repeated at the construction joint B/C as well.

Figure 5-11 shows the construction of Site 4 and Figure 5-12 to Figure 5-14 show the performance of the construction joints.

Figure 5-11: Construction of Site 4

Figure 5-12: Construction Joint A/B at Site 4 November 2008 (1 Month)

Figure 5-13: Construction Joint A/B at Site 4 June 2010 (20 Months)

Figure 5-14: Construction Joint A/B at Site 4 March 2011 (29 Months)

As is seen in Figure 5-12 to Figure 5-14 the performance of the construction joints continues to deteriorate with time. While it is possible that only the upper most portion of the pervious concrete layer was disturbed by the adjacent pass being constructed it is evident from the results in Section 5.2 that uniform compaction is not always achieved throughout the depth of the pervious concrete pavement layer. Therefore, as deeper portions of the pervious concrete layer become exposed they are more likely to ravell as the strength of the bonds between the aggregates is lower due to less compaction during construction. The cores did not show a visible difference in density between the top and bottom, so the compaction at Site 4 could be uniform throughout the pervious concrete layer.

If future projects require construction in the same manner, adjacent passes being placed immediately, then the following method could be used and is anticipated to perform better aesthetically and structurally; however, may have limited permeability. The limited permeability would be localized and thus not a problem overall in the functionality of the pavement. The solution would include placing pass A including any compaction that was part of the construction method. The adjacent pass would then be placed using the same method, including compaction. As soon as space was available behind the paving operation of the second pass, B, then a vibratory plate compactor would be used over the construction joint area, therefore recompacting any material that had been disturbed. If desired, a joint could later be saw cut into the construction joint area.

As soon as any pervious concrete had been placed and compacted, with the construction compaction regime, it should be covered in plastic to assist in retaining moisture for curing. This plastic could then be removed from the area receiving secondary compaction over the construction joint. The plastic would be replaced when the secondary compaction was completed. This is one solution to this challenge and it is anticipated that others exist as well. It is important that the construction staging and timing be considered before the initiation of construction to avoid conditions such as those at Site 4.

5.3.3 No Joints and Ineffective Joints

In conventional concrete pavements cracks will develop to accommodate movement of the concrete due to changing climatic conditions if no joints are included. When joints are included then the cracks develop at the joints. Cracks may develop to accommodate movement even if joints are included because the joints are spaced too far apart or do not match joints in adjacent pavement. Site 4 was constructed without joints to assess the behaviour of pervious concrete pavement cracking. The open void structure of pervious concrete pavement was considered to potentially handle the movement of the concrete in changing climatic conditions and therefore eliminate the development of cracks.

Cracks have developed at Sites 3, 4 and 5. In all cases the cracks are not as easily noticeable as they are in a conventional concrete pavement. The cracks are likely not clearly visible due to them being low in severity and ravelled material accumulating on them. At Site 3 cracks developed within slabs and parallel to the joints. The slabs at Site 3 are rectangular and the pervious concrete is surrounded by conventional concrete. The cracks are anticipated to have occurred due to the joint spacing being too wide for the width of the slabs.

At Site 4 the two cracks both run across the entire pervious concrete area, therefore through all three mixes and parallel to the shorter side of the rectangular parking lot. As there are no joints in the pervious concrete it is anticipated that the cracks developed to accommodate movement in changing climatic conditions. The two cracks are fairly evenly spaced in the parking lot which would also indicate that they are accommodating movement of the pervious concrete.

At Site 5 cracks developed adjacent to the saw cut joints, in half circle patterns, connecting to the joints. The joints may have been cut too late and the cracking had already developed. At Site 5 there is also a crack joining a joint in the adjacent concrete pavement to the joint in the pervious concrete pavement. This occurs in conventional concrete pavement and should be considered in the design phases to ensure that joints align adequately.

At Site 2 cracks have developed where the structural capacity of the pavement structure has decreased. Cracks appear to form the perimeter around the area that has weakened. These cracks are suspected to be a result of movement in the weakened area of the pervious concrete structure. The cracks appear as lines of ravelled material in the surface. The cracks are random in direction and generally not straight. The ravelled material is anticipated to gather in the cracked area due to a low spot being created. Additionally the material surrounding the crack is suspected to also be ravelling.

Cracks have developed in some of the field sites. All of the cracks that developed were low in severity and density and not pronounced or visible to the general user. The cracks have generally developed due to the lack of joints or ineffective joints. When joints were not included cracks developed. When joints were spaced widely, cracks developed. When joints did not align with joints in adjacent concrete pavement, cracks developed. Cracks were not observed to develop due to heavy loading.It may not be necessary to include joints in pervious concrete pavements and still maintain adequate surface condition. The presence of cracks in the surface presents an additional opportunity for ravelling or deterioration to initiate. The use of joints and the method in which they are constructed or not including joints should be considered and planned prior to construction.

5.3.4 Curing

At the beginning of this chapter it was noted that curing was not included in the research aspect of this project. All of the field sites were cured in the same manner, that being with a plastic sheet being placed on the pervious concrete efficiently after any compaction and joint forming had been completed. The plastic sheeting was left on the pervious concrete for at least seven days and only pedestrian traffic was allowed on the pervious concrete surface. Pedestrian traffic was generally limited and only occurred during the construction of adjacent pavement.

Keeping the plastic sheeting secured on the pervious concrete surface proved to be a challenge. The appropriate method for keeping it secured is highly dependent on the particular site conditions. In some cases the sheeting was secured to the wood forms using staples which was effective until the wood forms had to be removed. In other cases no particular plan had been made and available cobbles or stones were placed on the sheeting on the pervious concrete surface. Regardless of the method used to keep the sheeting secured it was evident that over the seven days or the time that the plastic was to stay on the surface for curing it was important to monitor the site. Often wind would get under the plastic and lift it off the pervious concrete, therefore making the plastic ineffective in that area.Curing is deemed to be a critical aspect in the construction of pervious concrete pavement should be planned in advance of the placement of the pavement.

5.4 Maintenance

Maintenance was considered in two forms in this project: winter maintenance for the safety of users; and permeability renewal maintenance. Winter maintenance and its effect on the performance of pervious concrete pavement is presented in Chapter 6.0. Permeability renewal maintenance will be discussed in the following section.

5.4.1 Permeability Renewal Maintenance

The permeability of a pervious concrete pavement overtime is dependent on various factors such as the use, surrounding conditions, initial permeability and climate. The initial permeability is important because if the pervious concrete has minimal ability to drain water immediately following construction then there is less deterioration in permeability that can occur before the pervious concrete is no longer functioning at an adequate rate. Permeability renewal maintenance methods are anticipated to be needed to keep a pervious concrete pavement functioning for several years.

Maintenance techniques for pervious concrete pavement that are currently considered in practice and referenced in literature include the following (NRMCA, Inspection and Maintenance, 2009):

- Vacuuming;
- Sweeping;
- Power washing;
- Power blowing; and
- Rinsing.

5.4.1.1 Permeability Renewal Maintenance Methods

The following five maintenance methods were evaluated at the field sites in this project: vacuuming; sweeping; power washing; garden hose rinsing; and fire hose washing. In some cases the maintenance was evaluated on a small scale (trials) for comparison purposes and other work has been done to renew the permeability of an entire site (full scale).

Vacuuming has been carried out using a wet/dry shop vacuum with a peak of 5.0 horsepower. Trials have been performed by vacuuming the surface and by sweeping the surface and then vacuuming. The wet/dry vacuum that was used is smaller in scale than equipment that would be intended for regular use on pervious concrete pavement areas. Industrial size equipment would also likely have higher power capabilities. The wet/dry shop vacuum was an accessible tool for this work and was intended to act as an estimate as to whether vacuuming was a viable alternative for renewing permeability to a pervious concrete surface.

Sweeping of the surface has been performed on both trial and full scale evaluations. At the trial sized evaluation, 1 m by 1 m areas were swept using a stiff household broom. Although this was carried out at a trial sized evaluation for the purposes of this research, this method could be useful to homeowners with pervious concrete driveways or sidewalks. This method requires minimal effort and uses equipment that is readily available. Full scale sweeping was carried out at three of the sites using highway grade sweepers.

Power washing was evaluated at both trial and full scale applications. The trial evaluation used a 100 psi cold water washer. The nozzle was approximately 500 mm above the pervious concrete and at a 45° angle to the surface. In the trial scale evaluation power washing was carried out on 1 m by 1 m locations for about one minute each. As noted with sweeping, power washing may be a simple maintenance method for some small commercial or residential owners. The full scale evaluation was carried out at Site 4 using commercial equipment. The complete details of the full scale equipment were not available; however, it is anticipated that the spray diameter and pressure was between that of the trial scale method and the large hose method described below.

Rinsing the surface using a garden hose has been evaluated on a trial application. A garden hose was used in the trial evaluation to rinse debris off the surface. The garden hose was running at the full available pressure and the operator had their finger over a portion of the hose to create more pressure and also reach a larger area. The water was directed at the surface at a 45° angle and the end of the hose was approximately 450 mm above the pervious concrete. No sprayer nozzle was used on the end of the hose. Rinsing the surface with a garden hose or collected rain water would be a simple maintenance technique for many homeowners. If a full scale rinsing was desired, such as a parking lot, this could be achieved using a water truck.

At Sites 3 and 5 the pervious concrete pavement was washed using a large hose, similar to a fire hose. At one site the inner diameter of the hose was 25 mm and at the other it was 38 mm. This size of hose provided more force than a garden hose but was not as aggressive or direct as a power washing system. At both field sites the entire area was washed. Additionally, a portion of Site 5 was washed for a second time, nine months after the first washing.

5.4.2 Results

Table 5-2 shows the results of each maintenance method used at each site. The initial permeability of the site (following construction) is noted in the construction permeability column. The permeability before maintenance, permeability after maintenance, change in permeability and a relative ranking are presented in Table 5-2. The permeability of each site after maintenance is described as a percentage of the initial permeability of the site and these values are ranked.

The ranking for permeability after maintenance ranges from 1 to 5. A ranking of 1 indicates the most substantial change in permeability and 5 is the least. The rankings are defined below:

1. More than 1 cm/sec change in permeability or permeability is renewed to almost initial value;
2. Less than 1 cm/sec change in permeability and more than 0.09 cm/sec change;
3. Change shown but less than 0.09 cm/sec;
4. Permeability remained constant; and
5. Permeability decreased.

Rankings 4 and 5 are often equivalent as there may have been decreases in permeability only due to repeated testing and in subsequent days the permeability returns to the same rate as prior to maintenance.

The ranking of percentage of original permeability ranges from 1 to 3 and 1 represents permeability rates most similar to the initial permeability and 3 is poorest. The results shaded grey in Table 5-2 should be highlighted in assessing the results. When this maintenance was performed the permeability was generally much higher than at the other field sites, therefore maintenance may not have been required but was done for research purposes and to remove debris from the winter from the surface. Since the permeability was initially high, less change in permeability was experienced, however there was still improvement for the most part.

Table 5-2 shows that in some cases the maintenance method decreased the average permeability of a site rather than improving it. These results are interpreted as not improving the permeability of the pervious concrete at the site. The permeability testing was generally repeated three times at each test point for all permeability testing in this project. When the testing was repeated three times the permeability rate routinely decreased. This finding is anticipated as voids that are not interconnected become filled with water and the options for water to drain decrease.

The quantity of water used to do permeability testing is generally much larger than occurs in a rain event and therefore this was not deemed to be a concern. The change in permeability typically small. The permeability renewal maintenance methods were often evaluated within one day or over two days, therefore repeating permeability testing multiple times in a short time frame. Some permeability renewal maintenance methods also involved the use of water.

The permeability of Site 3, especially Site 3B was low immediately following construction. At least portions of Site 3A and Site 3B were sealed or impermeable from the time of construction. In general the maintenance activities have not been able to renew the site to original permeability. The inability to bring pervious concrete back to original permeability is not unexpected, especially for sites that initially had a very high permeability, such as 4 cm/sec and these results are shaded in Table 5-2. However, since initial permeability cannot generally be renewed it is challenging to improve sites that start with very low permeability rates, such as Site 3.

Table 5-2: Permeability Renewable Maintenance Method Results

Site	Maintenance Method	Age at Maintenance (Months)	Permeability (cm/sec)					Percentage of Original Permeability	
			Construction	Pre-Maintenance	Post Maintenance	Change	Rank	(%)	Rank
1	Vacuum (trial)	11	4	0.04	1.41	1.37	1	35	1
2	Sweeping (full scale)	31	4.5	0.05	0.01	-0.04	5	0	3
3A	Sweeping and Vacuum (trial)	7	1.26	0	0.01	0.01	3	1	3
3B	Sweeping and Power Wash (trial)	7	0.05	0.01	0	-0.01	5	0	3
3A	Sweeping and Vacuum (trial)	18	1.26	0	0	0	4	0	3
3B	Sweeping and Vacuum (trial)	18	0.05	0	0	0	4	0	3
3A	Large Hose (full scale)	28	1.26	0	0.02	0.02	3	2	2
4A	Sweeping (trial)	8	4.7	0.24	2.33	2.09	1	50	1
4B	Rinsing with Garden Hose (trial)	8	4.1	0.03	1.31	1.28	1	32	1
4C	Sweeping (trial)	8	3.7	0.1	0.57	0.47	2	15	2
4C	Vacuum (trial)	8	3.7	0.15	0.15	0	4	4	3
4A	Sweeping and Power Wash (full scale)	19	4.7	0.52	0.5	-0.02	5	11	3
4B	Sweeping and Power Wash (full scale)	19	4.1	0.16	0.22	0.06	3	5	2
4C	Sweeping and Power Wash (full scale)	19	3.7	0.07	0.33	0.26	2	9	2
5A	Sweeping (full scale)	12	0.35	0.01	0	-0.01	5	0	3
5B	Sweeping (full scale)	12	0.44	0	0	0	4	0	3
5C	Sweeping (full scale)	12	0.45	0.01	0.02	0.01	3	4	2
5A	Large Hose (full scale)	12	0.35	0	0.33	0.33	1	94	1
5B	Large Hose (full scale)	12	0.44	0	0.05	0.05	3	11	2
5C	Large Hose (full scale)	12	0.45	0.02	0.4	0.38	1	89	1

Shaded Results: Initial permeability was very high therefore making renewal results low.

The trial sized maintenance methods shown in Table 5-2 consistently show greater improvement than the others. The locations selected to carry out the various trials of the methods were random, it is apparent that when the entire site is maintained the improvements may appear to be less. This trend may be a function of more detail being paid to small trial areas. Small trial areas are also carried out with smaller equipment which could be more effective. There are also possibly portions of a site that show more improvement in permeability than others. Some areas may not have any improvement in permeability and show a decrease in permeability. It is possible that the average of these values is less than what it would have been if the method had only been evaluated at a trial scale.

Figure 5-15 compares the effectiveness of each permeability renewal method that was carried out on a full scale. The effectiveness is being described by how much of the area of a site showed an improvement when the renewal method was carried out. Each vertical bar represents a unique full scale evaluation of the type of permeability renewal maintenance method.

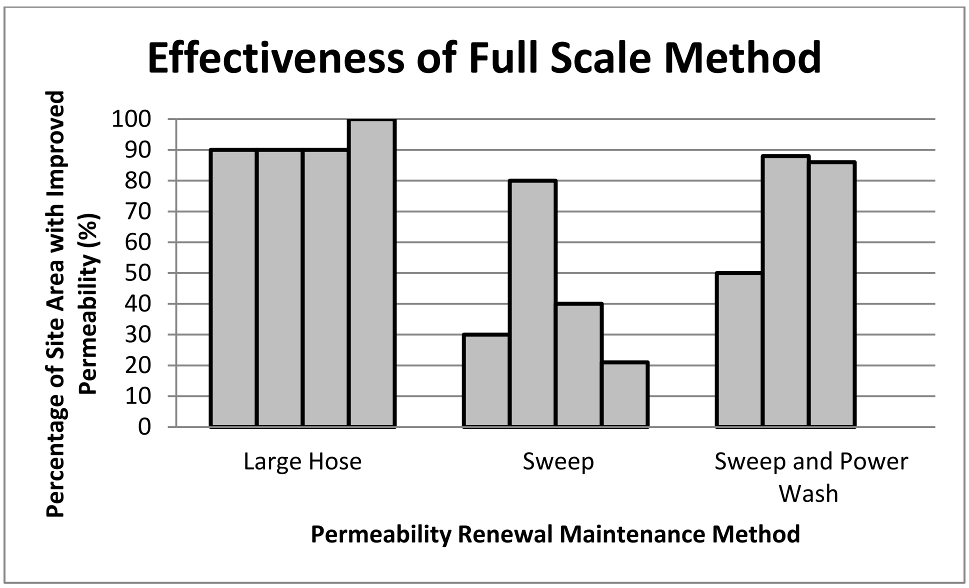


Figure 5-15: Effectiveness of Permeability Renewal Maintenance Methods

Figure 5-15 shows the percentage of the total surface area of a field site that showed an improvement in permeability following a maintenance method being performed. Each bar in the graph represents a particular event when a maintenance technique was carried out at an entire field site. In order to determine the percentage of the field site that showed an improvement, the permeability was measured throughout the site prior to maintenance. Following maintenance the permeability was measured again at the same points. The difference between each prior and post maintenance value was determined and used to assess how much of the site had been improved. If no change was measured at a test location then it was not included as an "improved" area. Ranking the effectiveness of the three methods shown in Figure 5-15, it is apparent that using a large hose such as a fire hose was most effective in all cases. Sweeping followed with power washing was the next most effective and finally only sweeping was least effective, improving 20 %- 80 % of a field site.

Although permeability readings may have shown a minimal change in permeability following maintenance of the surface, it is important to understand whether the change is significant for the particular field site. To determine if the maintenance method provided a significant difference, the difference between the two sample means, prior and post maintenance, was assessed. All permeability measurements from prior and post maintenance were used, including areas that increased, decreased and those that remained constant. A t-test was performed on each of the full scale maintenance treatments at 95 % confidence.

Table 5-3 describes which of the maintenance methods showed a significant difference in permeability when the t-test was used.

Table 5-3: Differences in Permeability Rate with Full Scale Maintenance

Method	T_{calc}	T_{crit}	Difference	Rank
Site 2: Sweep	1.97	2.06	Significantly Same	2
Site 3A: Wash with Large Hose	-2.46	2.31	Significantly Different	1
Site 4A: Sweep & Power Wash	0.09	2.11	Significantly Same	2
Site 4B: Sweep & Power Wash	-0.55	2.14	Significantly Same	2
Site 4C: Sweep & Power Wash	-1.47	2.23	Significantly Same	2
Site 5A: Sweep	0.93	2.14	Significantly Same	2
Site 5B: Sweep	-0.58	2.18	Significantly Same	2
Site 5C: Sweep	-0.97	2.31	Significantly Same	2
Site 5A: Wash with Large Hose	-2.43	2.26	Significantly Different	1
Site 5B: Wash with Large Hose	-2.47	2.26	Significantly Different	1
Site 5C: Wash with Large Hose	-2.60	2.26	Significantly Different	1

Washing the surface with a large hose showed significant differences in the mean permeability rates of the sites before and after maintenance was performed. Sweeping or sweeping and power washing the field site did not show a significant difference in mean permeability before and after maintenance.

Figure 5-16 shows the permeability of the field sites that were used in the full scale evaluation. The permeability results are presented from the time of construction up until spring of 2011.

The dates at which permeability renewal maintenance methods were performed in full scale on field sites have been identified in Figure 5-16 with markers in the shape of stars. In some cases the permeability was measured before maintenance was performed on the day of the maintenance method application. In other cases the permeability was not measured the day of maintenance. When the permeability was not measured the day of the maintenance then the positive slope is a result of the two data points being connects, not the permeability necessarily gradually increasing during that time period

5.4.3 Analysis

The rankings of the results from the four evaluation parameters: change in permeability; renewal to initial permeability; effectiveness of method; and statistical change in permeability, are shown in Table 5-4.

Table 5-4 shows the use of a large hose to rinse or clean the surface as being the best option when compared in terms of all four parameters for some of the scenarios. The maintenance method of only sweeping the surface consistently showed poor performance in terms of improving the permeability of the pervious concrete. Power washing the surface after sweeping it, demonstrated that this could be used to achieve some benefits in terms of increasing the permeability of the surface. Rinsing the surface with the large hose however, remained a more effective alternative in many cases.

The information presented in Figure 5-16 should be combined with the interpretation of the analysis in Table 5-4. Figure 5-16 demonstrates the permeability of the field sites during testing sessions multiple months and in some cases more than a year after the permeability renewal maintenance method was performed. All three mixes at Site 4 show the best long term benefit from the maintenance being performed. Sites 3 and 5 both show a decrease in permeability to pre-maintenance levels during the next testing session. Site 2 showed a lower

.

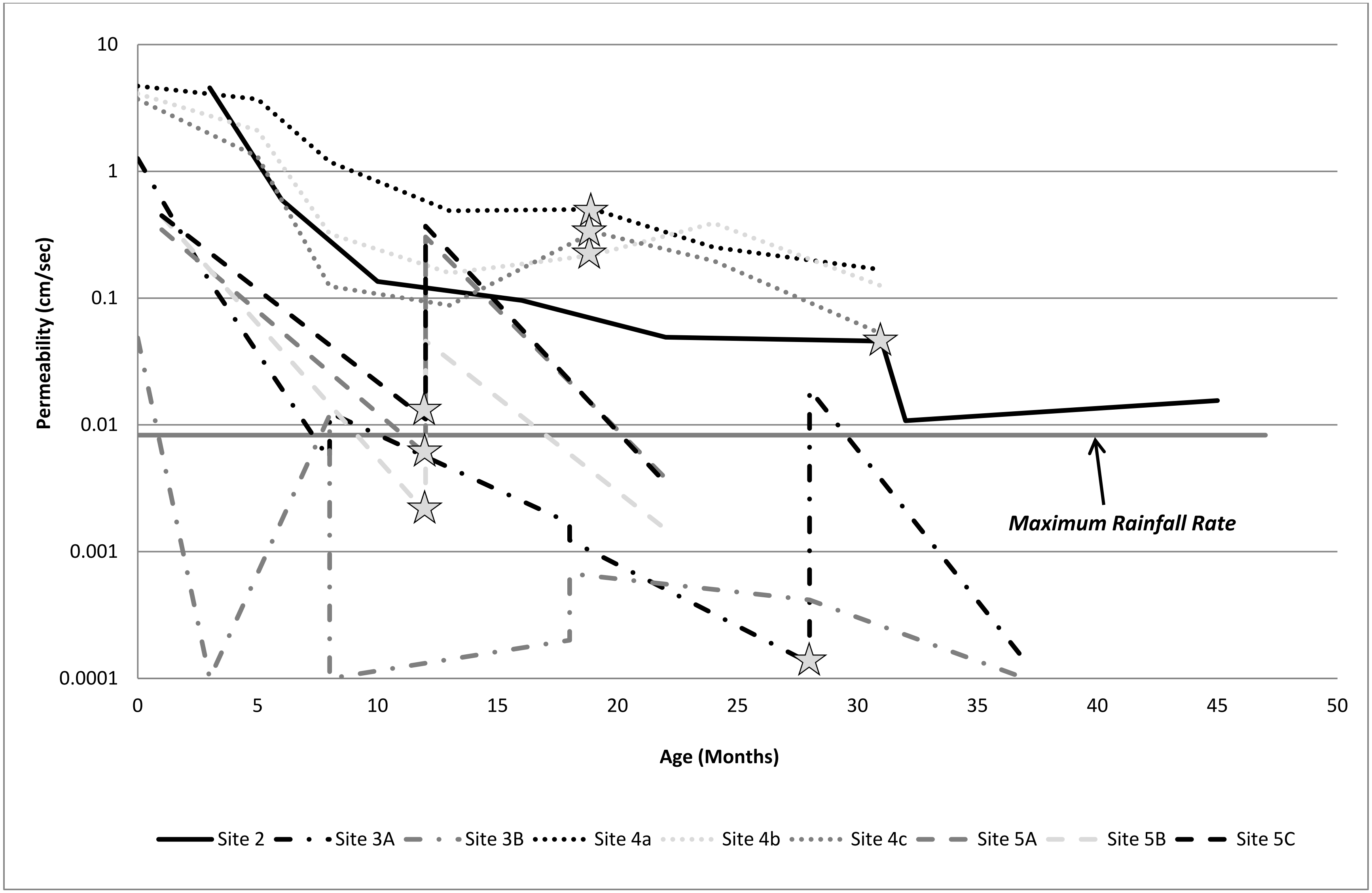


Figure 5-16: Average Permeability of Field Sites

mean permeability after maintenance but some increase in permeability has since occurred. The results in Figure 5-16 are somewhat contradictory to the analysis of Table 5-4. It is anticipated that the success at Site 4 is a combination of several factors: mix design; construction and site conditions. In comparison, at Sites 2, 3 and 5 there maybe attributes in these factors that are leading to less long term success from the permeability renewal maintenance methods. The analysis of Table 5-4 and Figure 5-16 together suggest that both methods are viable alternatives for full scale permeability renewal maintenance.

Analysis completed in Chapter 7.0 show that when maintenance was performed to the entire site at Site 4 the volume of water saturated the pavement structure for four weeks. Instrumentation that is discussed in Chapter 7.0 is located in the clear stone base and subgrade layers of the pavement structure. The data collected from the instrumentation identifies that the amount of water used in power washing the pervious concrete pavement surface resulted in the pavement structure being saturated for four weeks.

The methods that were performed at a trial scale provided valuable information. Sweeping was carried out at more than one site and had different results at both. Vacuuming was also done at more than one site and had varied results. As previously noted Site 3 initially had a low average permeability rate. In comparison, Sites 1 and 2 started with high permeability rates. The sites that had higher permeability rates generally showed more changes when maintenance was performed than those with lower permeability.

Sweeping in trial scale evaluations showed improvements. However, when used on a larger scale did not improve the average permeability of the sites and was not effective on most of the site area. The poor performance in the full scale application could be attributed to different equipment or different scenarios and conditions. The different equipment being used is anticipated to be a substantial factor. When an area was swept on a trial scale, a stiff, industrial broom was used and a significant attempt was made to agitate debris deep in the voids. In comparison, the full scale equipment is typically used for street sweeping and likely does not apply as much force or detail.

Power washing the surface in general was not an effective technique. In the small scale application a decrease in permeability was experienced and in the full scale applications the results were not as substantial as the washing with the large hose technique. The results show

Table 5-4: Ranking of Permeability Renewal Maintenance Methods Results

Site	Maintenance Method	Ranking			
		Permeability Improvement	Renewal to Original Permeability	Effectiveness	Statistical Change
1	Vacuum (trial)	1	1		
2	Sweeping (full scale)	5	3	3	2
3A	Sweeping and Vacuum (trial)	3	3		
3B	Sweeping and Power Wash (trial)	5	3		
3A	Sweeping and Vacuum (trial)	4	3		
3B	Sweeping and Vacuum (trial)	4	3		
3A	Large Hose (full scale)	3	2	1	1
4A	Sweeping (trial)	1	1		
4B	Rinsing with Garden Hose (trial)	1	1		
4C	Sweeping (trial)	2	2		
4C	Vacuum (trial)	4	3		
4A	Sweeping and Power Wash (full scale)	5	3	2	2
4B	Sweeping and Power Wash (full scale)	3	2	2	2
4C	Sweeping and Power Wash (full scale)	2	2	2	2
5A	Sweeping (full scale)	5	3	3	2
5B	Sweeping (full scale)	4	3	3	2
5C	Sweeping (full scale)	3	2	3	2
5A	Large Hose (full scale)	1	1	1	1
5B	Large Hose (full scale)	3	2	1	1
5C	Large Hose (full scale)	1	1	1	1

Shaded Results: Initial permeability was very high therefore making renewal results low.

that neither sweeping alone nor power washing and sweeping the surface lead to as much improvement in permeability as washing the surface with a large hose does. The reasoning behind these results is possibly that the power washing drives debris deeper into the voids of the pavement structure rather than removing them out of the pavement structure. The sweeping removes excess debris from the surface but does not dislodge deeper debris.

The main outcome of all of these evaluations is that the usefulness of the maintenance is generally site specific. Sites that have shown large decreases in permeability will require more aggressive maintenance such as rinsing with a large hose. Sites that appear to have debris gathering on the surface but it has not yet been compacted into voids may benefit from sweeping and washing with a small, garden hose.

The following additional points of interest have been observed during this research:

- Improvements in permeability can occur due to events in nature. Site 1 demonstrated this, as an improvement in permeability occurred during approximately 15 months of age which would have been late summer and early fall. During this time there were several aggressive thunderstorms which proved to be enough to rinse the pervious concrete pavement structure and improve permeability.
- Site 4B is showing a similar trend as Site 1 did in that following maintenance the permeability is not only staying above pre-maintenance values but is increasing. The increasing permeability is likely caused by natural effects such as heavy rain events.

5.5 Summary

The field sites constructed in this research included a few variables in terms of the construction method. The variables included: staging; concrete truck type; placement method; compaction effort; and inclusion of joints. The construction of Site 4 demonstrated that it is necessary to provide edges of a pervious concrete pavement with the opportunity to cure prior to adjacent construction being carried out. Concrete was successfully supplied by both volumetric mobile mixers and conventional ready mixed concrete trucks in this research. Both methods adequately supplied the concrete. However, the slow discharge of pervious concrete in general led to challenges in maintaining sufficient material for paving with an asphalt paver. Pervious concrete was placed successfully using the following methods: manually from the chute of a concrete truck; using an asphalt paver; using a Bidwell Bridgedeck paver; and using a Razorback paver. No substantial challenges were experienced using any of these methods. The pervious concrete at the field sites was compacted using manual rollers, vibrating automated rollers, vibratory plate compactors and a Razorback paver. Surface performance of the sites demonstrated that compaction during construction is necessary. The Bidwell Bridgedeck paver did not provide compaction and lead to poor surface performance. Both saw cutting and forming the joints had similar performance during the monitoring period. The selection of one method over another is project specific and either can be used effectively. When joints were not included, cracks developed. Curing was identified to be a step of construction that was often not well planned. Resources for curing (plastic sheeting and a securing method) should be readily available before construction begins.

It is apparent that in order for pervious concrete pavement to be used throughout Canada as a stormwater management tool and as a pavement, maintenance requirements need to be understood. In general the test sites included in this project have performed well with minimum demand for maintenance in order to maintain adequate performance. Maintenance methods have been trialed for research purposes and also to improve the performance in some cases. The following findings have been made:

- The initial permeability of the pervious concrete pavement can influence future performance.
- Power washing using personal sized equipment can push debris deeper into voids and decrease permeability rather than improve it.
- Sweeping of the surface can be effective in removing debris only off the surface and not from deep voids, therefore not necessarily improving permeability.
- Washing the surface with a large diameter hose can dislodge debris deep in voids and renew permeability, in some cases, to near initial permeability values.

It is important to plan construction of areas including pervious concrete so that unnecessary debris, such as landscaping material, is not placed on the surface. A pervious concrete pavement that is constructed with an adequate permeability rate, suitable mix design and quality construction practices is anticipated to be able to be maintained using one of or a few of the techniques outlined in this section.

6.0 PERVIOUS CONCRETE PAVEMENT SURFACE PERFORMANCE

The intent of this chapter is to evaluate factors that may affect the performance of pervious concrete pavement when it is used in freeze-thaw climates. The factors that have been considered in this research include: exposure to freeze-thaw cycles, winter maintenance activities, and vehicular traffic and pedestrian traffic. The effect that each factor had on the performance of pervious concrete pavement was assessed by the changes in permeability and surface condition as a function of exposure. Pervious concrete pavements were evaluated in both laboratory and field evaluations. Cores from Site 5, laboratory prepared slabs and field sites were evaluated.
Table 6-1 outlines the factors that each scenario has been exposed to.

Table 6-1: Sample Type and Associated Factors

Sample	Freeze-Thaw Cycles	Winter Maintenance	Vehicle and Pedestrian Traffic
Cores	X		
Slabs	X	X	
Field Sites 1, 2, 4, 5	X	X	X
Field Site 3	X		X

The cores, slabs and field sites and the frequency and extent of their exposure to the factors is described later in this chapter. The purpose of this analysis was to highlight the conditions of freeze-thaw climates that adversely affect the performance of pervious concrete pavement, as well as those that do not.

6.1 Samples and Field Sites in Analysis

The analysis in this chapter used data from all five field sites.
Table 6-1 highlights the differences between Site 3 and the other four sites. Site 3 is located in Maple Ridge, British Columbia, and due to the climate requires winter maintenance very rarely. Site 3 however, exposed to freeze-thaw cycles throughout the year. The other four sites are located in Ontario and Quebec and each one receives winter maintenance involving at least snow removal.

The six cores included in the results and analyses in this chapter are from Site 5. The cores were extracted from across the field site, two from Mix 5A, two from Mix 5B and two from Mix 5C. The cores were extracted for material characterization and were included in the data presented in Chapter 4.0. The primary characteristics being evaluated were void content, compressive strength and freeze-thaw durability. The freeze-thaw durability results will be the focus of this chapter.

Freeze-thaw durability of the cores was evaluated following a modified version of ASTM C666 as described in Chapter 3.0 (ASTM, 2008). Freeze-thaw cycling of the cores was performed initially using a freeze-thaw chamber which automatically cycled the temperature. The chamber was used for approximately the initial 120 cycles. After this point, the chamber was required for other projects and the cores were exposed to freeze-thaw cycling in the same manner as the slabs. The freeze-thaw cycling was a very similar range in temperatures to that of the chamber and was achieved by placing the cores in a large walk in freezer and then moving them back to a room temperature environment manually.

Approximately every 20 freeze-thaw cycles, the cores were weighed and then submerged in water. The cores were then lifted out of the water bath, at which point most water drained away, and they were then placed in trays. The cores were freeze-thaw cycled in trays such that the cores lay horizontal and were raised off the bottom of the tray to allow for water to drain out of the core without leaving the core partially submerged. When the cores were weighed they were not dried to a constant mass first using an oven but were weighed in their current state which in many instances included moisture on the surface due to being recently removed from the freezer.

Slabs were cast in the CPATT laboratory in 2009 using the mix design and aggregate sources used to construct Site 1. There were 20 slabs cast in total, all with surface dimensions of 300 mm by 300 mm. Of the 20 slabs, six were 300 mm in height and the other 14 were 200 mm in height. The slabs were prepared in order to evaluate the effects of winter maintenance on pervious concrete pavement.

Two of the slabs were cast for material characterization and the remaining 18 were divided into six groups of three. Each group experienced a different winter maintenance scenario. The six winter maintenance scenarios are listed below:

1. Heavy sand loading, heavy precipitation (HSHP);
2. Moderate sand loading, moderate precipitation (MSMP);
3. Heavy sand loading, moderate precipitation (HSMP);
4. Moderate sand loading, heavy precipitation (MSHP);
5. Heavy salt loading, heavy precipitation (HSaltHP); and

6. No salt or sand loading, heavy precipitation (HP).

The slabs were freeze-thaw cycled for 255 cycles, the equivalent of five years of freeze-thaw exposure in Toronto, ON, on average (Ho & Gough, 2006). During each cycle precipitation was applied to the slabs as well as the winter maintenance treatment for the particular scenario: heavy sand; moderate sand; salt solution; or no treatment.

Freeze-thaw cycling of the slabs involved moving them in and out of a walk-in freezer. Instrumentation was included in six of the slabs during casting. The instrumentation was used to monitor the temperature in the centre of the slab during freeze-thaw cycling. The data collected by the instrumentation was used to determine how long each portion of the freeze-thaw cycle should be in order to achieve a consistent temperature throughout the slabs. It was determined that 14 hours were required to achieve a complete cycle between approximately 8°C to -13°C (Henderson & Tighe, 2010).

The cores were extracted from a field site which was constructed with full scale equipment, Site 5, and therefore represents pervious concrete pavement in place. The field sites were all constructed for vehicular traffic and pedestrian traffic. In addition, the structural and hydrological designs were in accordance with the local specifications. Cores were also extracted from all of the field sites.

The slabs were compacted in two lifts following the method outlined by the Canadian Standards Association (CSA) for flexural beam casting (CSA, 1996). Cores were extracted from the material characterization slabs for void content measurement. The void content of each of the sample types is shown in
Table 6-2.

Table 6-2: Void Content of Samples

Sample Source	**Average Void Content (%)**
Laboratory Cast Slabs	29.1
Site 1	29.4
Site 2	27.4
Site 4	26.7
Site 5*	25.1
* Cores Used in Freeze-Thaw Cycling	

As

Table 6-2 shows, the void content of the slabs, field sites and cores ranged from 25 % to 29 %.

6.2 Factors

Pervious concrete pavement that is exposed to freeze-thaw cycling while moisture is present has the potential for the voids to be filled with frozen water. Water getting trapped and frozen in the voids could not only limit permeability but also possibly be destructive to the pervious concrete itself. All three types of samples being evaluated in this study were exposed to freeze-thaw cycling. The ranges in temperature used for the freeze-thaw cycling in the lab were aggressive; however, not uncommon for many regions of Canada. The laboratory freeze-thaw cycles for the slabs were from 20°C to -12°C. In the Canadian climate pervious concrete would be exposed to temperatures as low as -12°C and lower, although not during each freeze-thaw cycle. Similarly, the cores were cycled from approximately 12°C to -10°C and the low end of the range is not uncommon in Canada but it does not occur in every freeze-thaw cycle necessarily.

Winter maintenance generally involves the removal of snow from the pavement surface. In this project snow removal was carried out using highway grade equipment or equipment routinely used on parking lots in Canada. Following the removal of snow from the surface, there is often sand applied to the surface. An alternative to sand is a salt solution which is applied to the surface of a pavement prior to a snow event. Both the sand and salt solution are used to create a pavement surface that is not slippery for vehicular and pedestrian users. The sand provides traction on top of ice while the salt solution minimizes the potential of water freezing on the surface and ice developing. Salt is also sometimes applied to the surface after a winter storm to melt any ice that maybe present. Sand could fill the voids in the pervious concrete and lead to a reduction in permeability. Application of solid salt could also fill voids. A salt solution was deemed to not likely reduce the permeability of the pervious concrete as much as the other two maintenance options.

In this project sand and salt solutions were evaluated as winter maintenance techniques. Sand was applied to the laboratory slabs at two rates. The first rate was referenced as the moderate rate and is equivalent to that used by local contractors and public agencies in the southwestern Ontario region. This rate was also found to be a common rate in many areas throughout North America. The second rate of sand application, which some of the laboratory slabs received was heavy sanding, meaning that sand was applied at 1.5 times the moderate rate.

Sand was applied at both the moderate and heavy rates to evaluate if this difference in debris on the surface was reflected in the permeability. Similarly, the salt solution was prepared and applied by the same rates as are used in the Region of Waterloo. The moderate application rate of the sand was 0.02 kg/m^3 and the salt was proportioned at 23% by weight and applied at 0.02 kg/m^3.

The anticipated effect of traffic is varied as in some scenarios traffic is anticipated to help clean the voids and thus keep the permeability rate higher. In other scenarios no difference in permeability between areas with different traffic types has been identified. Finally, results of this project have identified that in some scenarios, permeability remains higher where traffic is stationary, in comparison to moving traffic. None the less, it is apparent that traffic has the potential to move debris that is on the pervious concrete pavement surface. Traffic is also a source of debris, as vehicles can bring material on to the pervious concrete surface from surrounding areas. This occurs at some of the field sites where vehicles that are moving on to the pervious concrete are coming off of the surrounding sand areas or dirty paved areas. In addition to tracking debris on to the pervious concrete pavement, traffic also has the potential to compact debris into the voids of the pavement.

The natural environment that the pervious concrete is set within may include a variety of debris sources. Examples of these sources are lawn trimmings, landscape soil, leaves and construction materials, amongst other items. The surrounding conditions should be considered when pervious concrete pavement is intended to be used.

6.3 Testing

All three types of samples were monitored for changes in two characteristics: permeability and surface condition. Similar techniques were used to measure both of these characteristics on each sample type, as is described in Chapter 3.0. The condition of the pervious concrete pavement surface of all samples was monitored visually by the same individual, throughout the project. The cores were also visually monitored to identify if portions of the cores were breaking off. Photos were taken during the surface evaluations of cores, slabs and field sites. The cores and slabs were both weighed during surface condition evaluations. Changes in weight could be the result of pieces falling off the samples due to ravelling. The sand being applied to the slab samples continually increased the weight until maintenance was carried out. However, it is known how much sand was added to each sample and was considered in the evaluation by recording changes in weight.

6.4 Results

6.4.1 Permeability of Cores

The cores from Site 5 were tested for permeability before freeze-thaw cycling began. Permeability testing was also repeated throughout the completion of the 250 freeze-thaw cycles. One year at Site 5 includes approximately 28 freeze-thaw cycles. The change in permeability over almost nine years of freeze-thaw cycling is shown in Figure 6-1 and Figure 6-2. There were two cores for each mix (5A – Fibre, 5B – Latex, 5C - Control) at the beginning of the freeze-thaw cycling and each core is represented by a separate line in Figure 6-1 and Figure 6-2. The cores were tested for permeability in both directions. Figure 6-1 shows the first direction of testing, as they would be in place in the field, with water draining from the surface to the bottom. Figure 6-2 shows the permeability of the cores when they were tested such that water drained from the bottom to the surface, opposite of the orientation in the field. The cores were tested in both directions because some of the cores showed a visible difference in density between the portion of the core closer to the surface and the portion closer to the bottom. When a difference was visible, the surface always appeared denser, therefore appearing to have less voids, than the bottom.

Both Figure 6-1 and Figure 6-2 show only the initial reading for one of the Mix 5B cores. This core failed prior to the next round of testing at 100 freeze-thaw cycles. The failure of this core will be described later in this chapter, in more detail.

The mean permeability of the cores at each testing session were compared using an ANOVA table to evaluate if the permeability of the cores was different over time. The null hypothesis was that the mean permeability of all the cores was not statistically different between different testing sessions and therefore after different amounts of freeze-thaw cycles. The F_{calc} value at 95 % confidence was 0.611 and the F_{crit} was 3.197, therefore the null hypothesis was not rejected and the mean permeability rates of the various testing sessions were statistically the same. The ANOVA table is included in Appendix D.

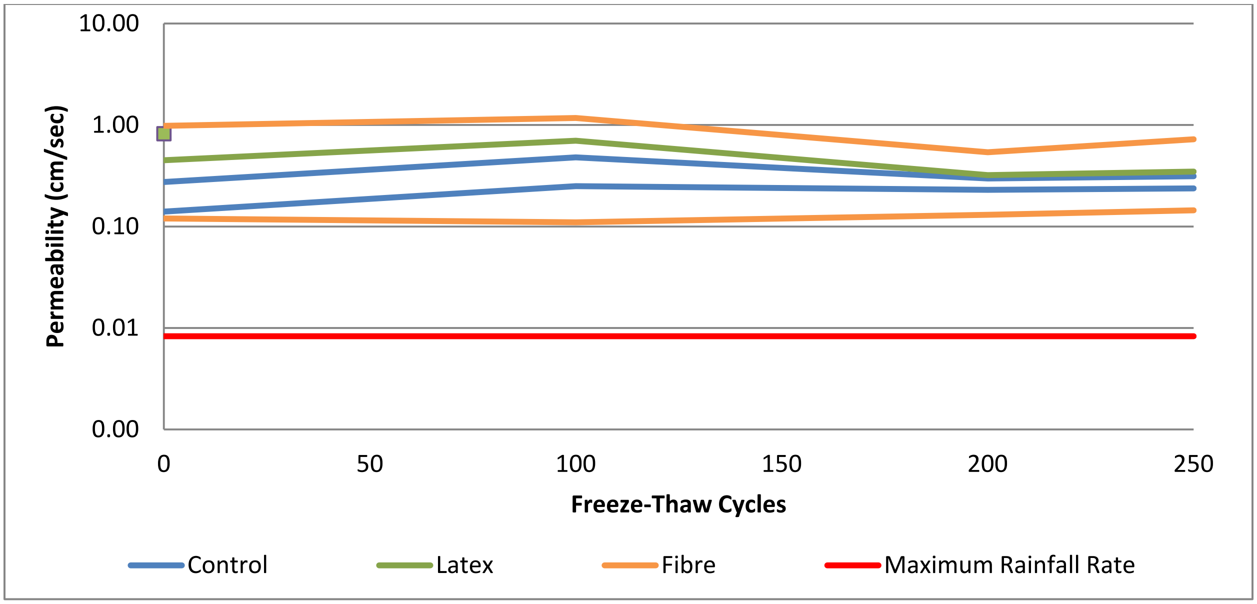


Figure 6-1: Core Permeability (Surface to Bottom)

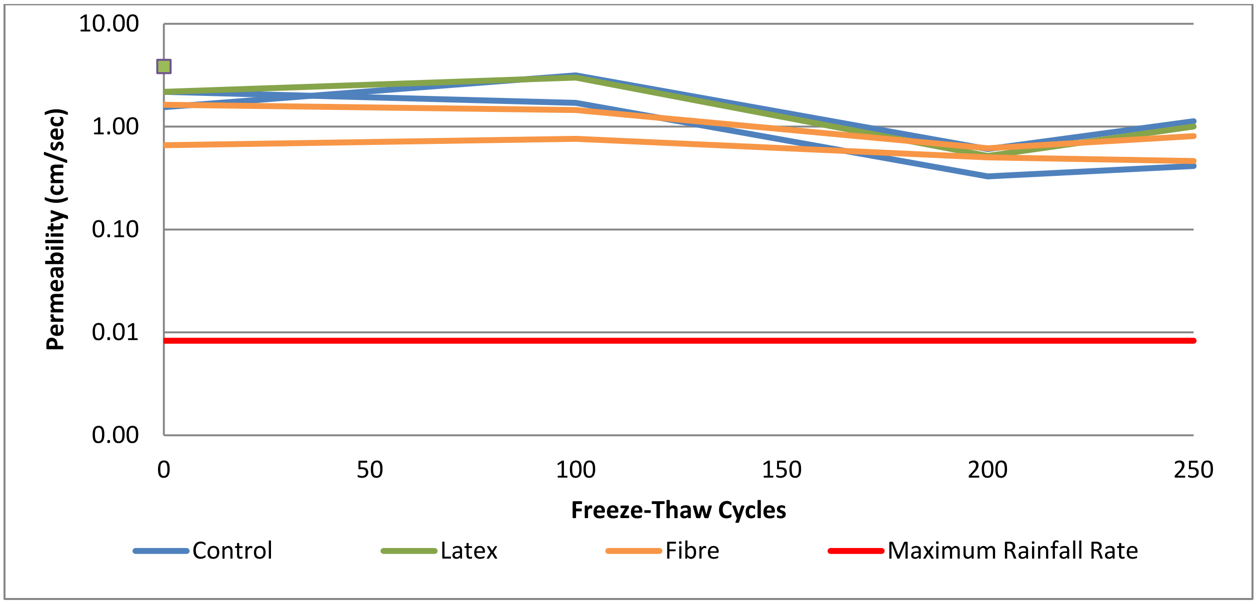


Figure 6-2: Core Permeability (Bottom to Surface)

6.4.2 Surface Condition of Cores

Surface condition of the cores was monitored throughout the freeze-thaw cycling. The cores were weighed regularly during the freeze-thaw cycles and photos were taken. The weight of the cores throughout the equivalence of almost nine years of freeze-thaw cycling is shown in Figure 6-3.

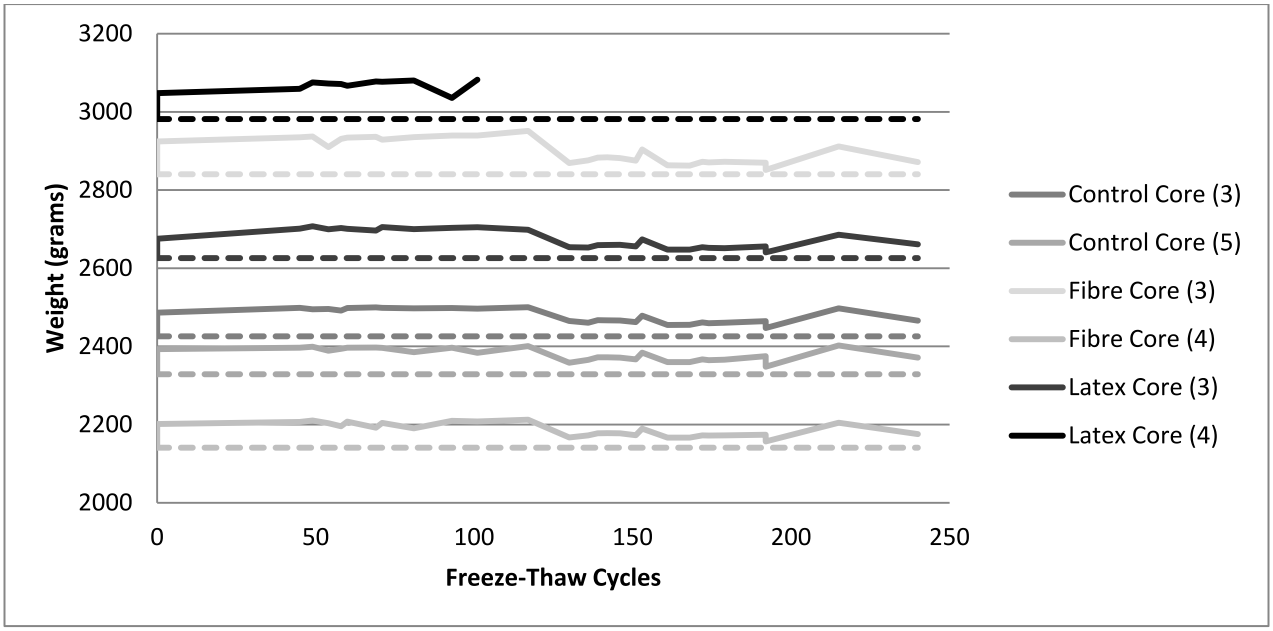
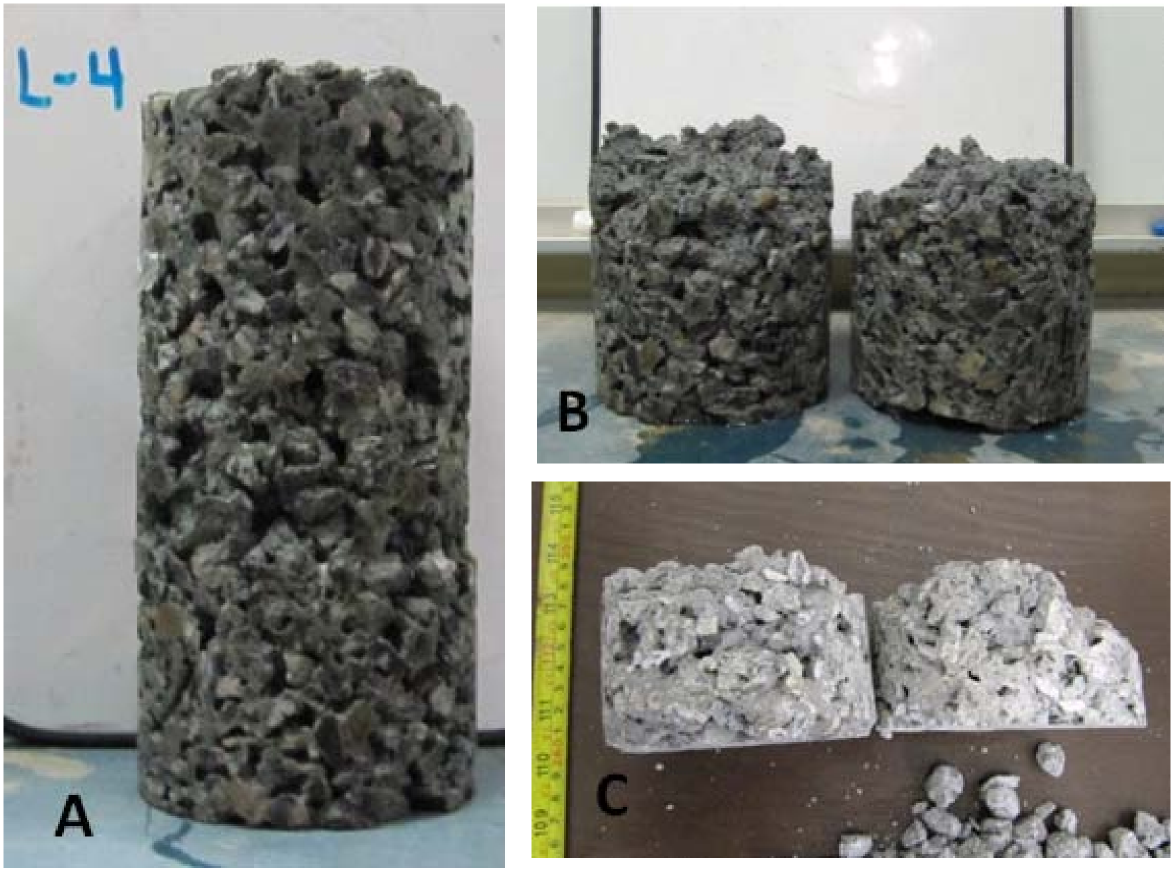


Figure 6-3: Weight of Cores During Freeze-Thaw Cycling

Figure 6-3 shows two lines for each core, a dashed line and a solid line. The dashed line shows the weight of the core prior to starting freeze-thaw cycling. The cores had been dried to a constant mass before freeze-thaw cycling was started. The cores then increased in mass when exposed to water during the routine submersion. The mass of the cores never dropped below the initial mass, indicating that material was not ravelling off the cores. Decreases in weight are due to cores having been dried for testing recently or exposed to air with no saturation for multiple days, such as over multiple day holidays.

Figure 6-3 shows that the measurements stopped for Mix 5B Core (4) at 100 freeze-thaw cycles. Following being weighed at 100 cycles the core was submerged in water, as was done throughout the testing. As the core was removed from the water it broke in half, horizontally. Visual inspection showed that the paste throughout the inside of the core had crumbled or broken down. Figure 6-4 shows Mix 5B Core (4) after it failed.



A. After 69 cycles, B. Failure at 101 cycles, C. Remaining intact portion of core
Figure 6-4: Failure Progression of Mix 5B Core

The remaining five cores do not show evidence of failure occurring. However, Mix 5B Core (4) did not have an indication of failure prior to completely breaking into two pieces. Each core has lost a couple of small pieces from the sides of the core. Generally the pieces that have come off the cores have only been one piece of aggregate in size.

6.4.3 *Permeability of Slabs*

The permeability of the slabs has been evaluated several times throughout the five year freeze-thaw cycling period. Maintenance was also performed on several of the slabs during the freeze-thaw cycling. Figure 6-5 shows the average permeability of each of the four groups of slabs that had sand loaded on the surface. Vertical lines in Figure 6-5 indicate when maintenance was performed to the slabs. The maintenance included sweeping sand off the slabs and then vacuuming them with a wet/dry vacuum.

Figure 6-5 shows the permeability of the slabs starting at 12 freeze-thaw cycles. The slabs were tested prior to being loaded with any sand and the permeability rates were observed to be very high. The data presented in Figure 6-5 is from 12 freeze-thaw cycles onward so that differences between the groups of slabs can be seen. All four slab groups appeared to have similar permeability rates throughout the five years of freeze-thaw cycling, as is shown in Figure 6-5.

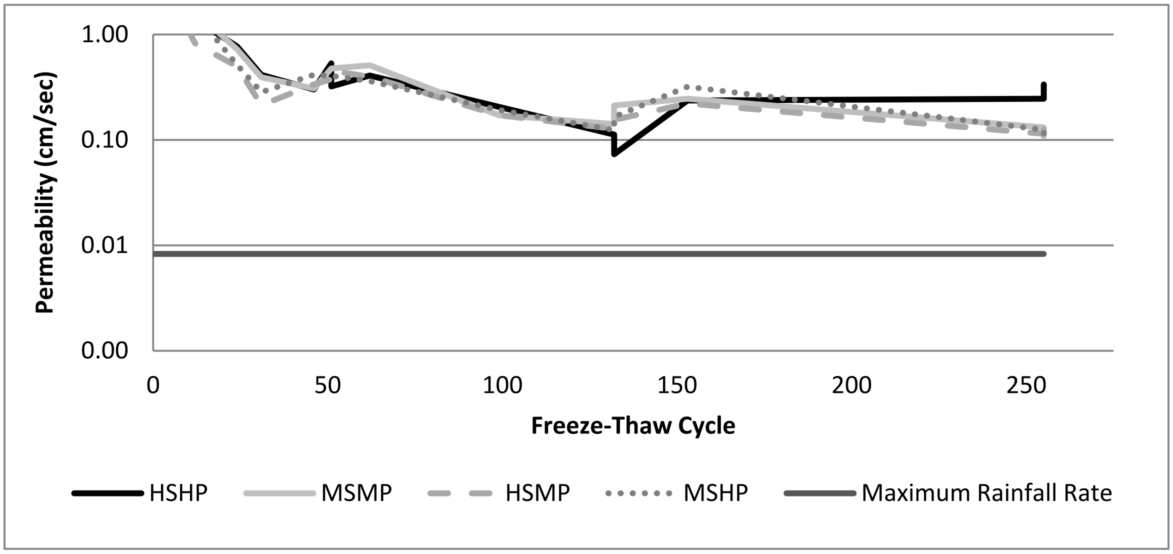


Figure 6-5: Permeability of Slabs Loaded with Sand

To further evaluate if differences in permeability were significant between the slab groups, they were compared using the t test. The permeability of the two groups of slabs loaded heavily with sand was compared to the permeability of the two groups of slabs that were loaded with less sand (moderate). Table 6-3 shows the results of this comparison at a 95% confidence using a one sided t-test. The null hypothesis of each test was that the mean permeability of the slabs loaded heavily with sand was equal to the mean permeability of the slabs loaded with a moderate amount of sand. It is anticipated that when there is debris on the pavement surface, then its ability to drain water would be decreased. The permeability of the groups was compared five times and the approximate corresponding age of the slabs is noted in brackets: 24 cycles (0.5 years); 51 cycles (1 year); 99 cycles (2 years); 153 cycles (3 years); and 250 cycles (5 years). If the null hypothesis is rejected then the slabs loaded with a moderate amount of sand would have a higher permeability rate than those loaded with an average amount of sand, there making the permeability rates of the two groups of slabs statistically different.

Table 6-3: Permeability of Heavy and Moderate Sand Loaded Slab Groups

Freeze-Thaw Cycles (Equivalent Time in Years)	T_{calc}	T_{crit}	**Outcome**
24 Cycles (0.5 Years)	0.109	-1.833	Statistically Same
51 Cycles (1 Year)	0.221	-1.833	Statistically Same
99 Cycles (2 Years)	0.267	-1.833	Statistically Same
153 Cycles (3 Years)	-1.795	-1.895	Statistically Same
250 Cycles (5 Years)	1.418	-2.015	Statistically Same

Table 6-3 shows that the null hypothesis was true for all of the tests and therefore it was shown that the slabs being loaded heavily with sand had statistically the same permeability rate as those loaded with a moderate amount of sand.

The slabs that were exposed to heavy precipitation were then compared to those loaded with a moderate amount of precipitation, at 95% confidence, using a t-test. The two groups were compared five times and the approximate corresponding age of the slabs is noted in brackets: 24 cycles (0.5 years); 51 cycles (1 year); 99 cycles (2 years); 153 cycles (3 years); and 250 cycles (5 years). The null hypothesis of each t-test was that the slabs receiving the average amount of precipitation had a mean permeability that was equal to those that received the heavier amount of precipitation. From observations at the field sites it was anticipated that precipitation may assist in keeping the pervious concrete clean and debris out of the voids and therefore the precipitation at the site may be reflected in the permeability. The results of the two sided t-test at 95 % confidence are shown in Table 6-4.

Table 6-4: Permeability of Slabs and Precipitation Loading

Freeze-Thaw Cycles (Equivalent Time in Years)	T_{calc}	T_{crit}	**Outcome**
24 Cycles (0.5 Years)	0.249	2.228	Statistically Same
51 Cycles (1 Year)	0.674	2.228	Statistically Same
99 Cycles (2 Years)	1.212	2.306	Statistically Same
153 Cycles (3 Years)	1.404	2.262	Statistically Same
250 Cycles (5 Years)	1.781	2.447	Statistically Same

All of the t-tests failed to reject the null hypothesis and demonstrated that the pervious concrete slabs loaded with more precipitation had statistically the same permeability rates to those loaded with moderate precipitation. Figure 6-6 shows the average permeability of Groups HSaltHP and HP.

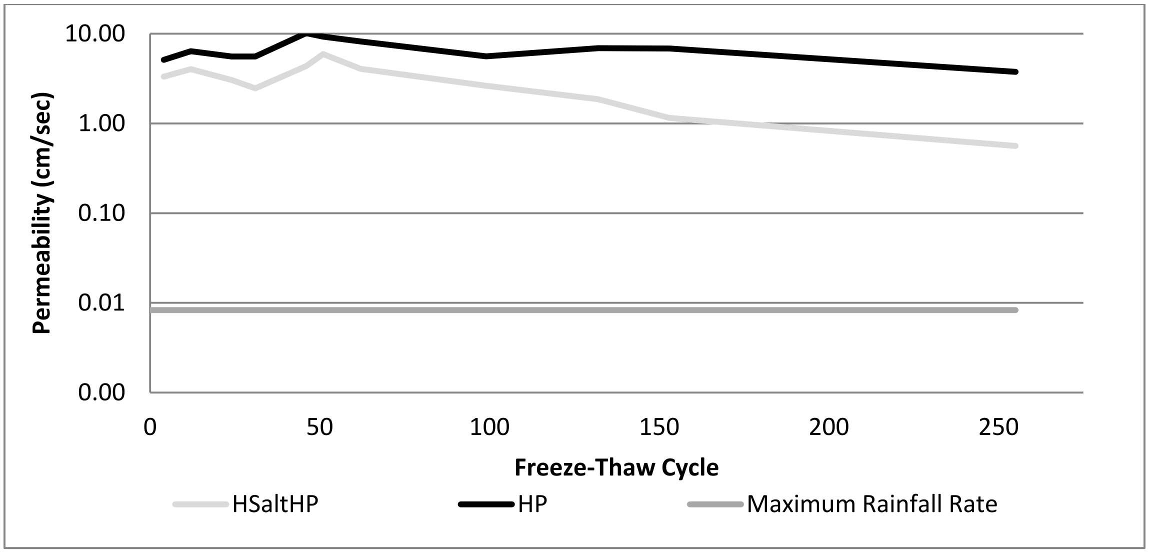


Figure 6-6: Permeability of HSaltHP and HP Slab Groups

Figure 6-6 shows that the permeability of the slabs loaded with the salt solution decreased over time while the control slabs, HP that were only loaded with precipitation, did not show a substantial change in permeability over the equivalent of five years. The permeability of the HSaltHP was compared to the permeability of the slabs receiving heavy loadings of sand and moderate loadings of sand to evaluate if the salt solution decreased the permeability as much as either of the sand treatments. First the permeability of the HSaltHP slabs was compared to the permeability of the slabs heavily loaded with sand. A two sided t-test was used for this evaluation and the null hypothesis was that the mean permeability rate of the slabs loaded heavily with sand was equivalent to the mean permeability of the HSaltHP slabs. The t-test was completed at 95 % confidence.

Table 6-5 shows the results of the t-tests, which were carried out at four different ages of the slabs: 24 cycles (0.5 years); 51 cycles (1 year); 99 cycles (2 years); and 153 cycles (3 years). The permeability rates were not compared at 250 cycles (5 years) as only one HSaltHP slab was still intact at this time, the other two had disintegrated.

Table 6-5: HSHP and HSMP Compared to HSaltHP

Age	T_{calc}	T_{crit}	Outcome
24 Cycles (0.5 Years)	-8.617	4.303	Significantly Different
51 Cycles (1 Year)	-59.508	3.182	Significantly Different
99 Cycles (2 Years)	-17.313	4.303	Significantly Different
153 Cycles (3 Years)	-17.078	4.303	Significantly Different

The t-tests showed that the permeability rates of the two types of slabs were significantly different at each age. This outcome is somewhat anticipated. However, the HSaltHP slabs did show some change in permeability over time and the significance of this in comparison to the other slabs was of interest. The t-tests were repeated for the mean permeability rates of the slabs receiving moderate loading of sand and the HSaltHP slabs. Similarly to

Table 6-5, the slabs were compared four times as at 250 cycles or five years, three of the HSaltHP slabs had deteriorated.
Table 6-6 shows the results of the two sided t-tests at 95 % confidence.

Table 6-6: t-tests Comparing Moderate Sand Loaded Slabs with HSaltHP Slabs

Age	T_{calc}	T_{crit}	Outcome
24 Cycles (0.5 Years)	-8.427	3.182	Significantly Different
51 Cycles (1 Year)	-56.268	2.776	Significantly Different
99 Cycles (2 Years)	-17.425	4.303	Significantly Different
153 Cycles (3 Years)	-14.695	3.182	Significantly Different

Table 6-6 shows that each of the t-tests demonstrated that the mean permeability rates of the two groups of slabs were significantly different. This outcome is anticipated as all the slabs loaded with sand, both a moderate amount and the heavier loading were continually covered in sand, such that it was often not possible to see the pervious concrete surface. The slabs loaded with the salt solution did not have a visible substantial collection of debris or material on the surface. However, it was visible that the salt was accumulating on the slabs and within the slabs. The slabs loaded with salt were darker in colour than the other slabs which is discussed in the following section. After many cycles the white salt was also visibly accumulating on the slabs.

The mean permeability rates of the HSaltHP and HP groups were compared using a two sided t-test to evaluate if they were statistically different. The null hypothesis was that the mean permeability rates of the two slab groups after a particular number of freeze-thaw cycles were equal. Table 6-7 shows the results of the t-tests at 95 % confidence.

Table 6-7: Comparison of Mean Permeability Rates of HSaltHP and HP

Number of Freeze-Thaw Cycles	Slab Group	Permeability Rate Mean (cm/sec)	Permeability Rate Standard Deviation (cm/sec)	T_{calc}	T_{crit}	Outcome
0	HSaltHP	2.6	0.3	-1.008	4.303	Statistically Same
	HP	4.2	2.6			
4	HSaltHP	3.3	0.2	-1.198	4.303	Statistically Same
	HP	5.1	2.6			
12	HSaltHP	4.0	0.3	-1.452	4.303	Statistically Same
	HP	6.4	2.8			
24	HSaltHP	3.0	0.5	-2.132	4.303	Statistically Same
	HP	5.6	2.0			
31	HSaltHP	2.5	0.1	-3.862	4.303	Statistically Same
	HP	5.6	1.4			
46	HSaltHP	4.4	0.2	-1.774	4.303	Statistically Same
	HP	10.1	5.6			
51	HSaltHP	5.9	0.1	-2.072	4.303	Statistically Same
	HP	9.3	2.8			
62	HSaltHP	4.1	0.8	-3.771	4.303	Significantly Same
	HP	8.8	2.0			
99	HSaltHP	2.6	0.2	-2.387	4.303	Statistically Same
	HP	7.1	3.2			
132	HSaltHP	1.9	0.5	-4.310	4.303	Significantly Different
	HP	6.9	2.0			
153	HSaltHP	1.2	0.1	-4.388	4.303	Significantly Different
	HP	6.9	2.2			
250	HSaltHP	0.6	N/A	N/A		
	HP	3.8	0.7			

Table 6-7 shows that prior to 132 freeze-thaw cycles the slabs loaded with salt and the control slabs had statistically the same mean permeability rates. At 132 freeze-thaw cycles and beyond enough salt had accumulated in the HSaltHP slabs that the permeability was affected. At 132 and 153 freeze-thaw cycles the null hypothesis was rejected however the T_{calc} and T_{crit} values were very similar. At 250 freeze-thaw cycles the t-test was not completed. Of the three initial slabs in the HSaltHP group, two had failed to a point that permeability testing could not be carried out.

6.4.4 Surface Condition of Slabs

The surface distresses that developed on the slab samples were generally limited and included a small amount of ravelling, some paste loss and fracturing of very few aggregates. Table 6-8 shows the surface distress present on each slab throughout the simulation of five years of freeze-thaw cycling in the laboratory. One group of slabs showed more surface condition deterioration than the others, HSaltHP. Slab Group HSaltHP, which experiences winter maintenance including application of a salt solution, had cracks develop throughout the slabs, substantial portions of the slabs fell off and at the end of five years, two of the three slabs fell apart entirely.

The surface distress of the slabs showed a common trend in many of the slab groups. This trend was the initial presence of paste loss and at a later point the presence of ravelling. This tendency suggests that ravelling maybe a result of the paste characteristics. The integrity of the paste to withstand exposure to freeze-thaw cycling may have a large effect on the amount of ravelling experienced by a pervious concrete pavement. Figure 6-7 and Figure 6-8 show the weight of the slabs throughout the five years of freeze-thaw cycling.

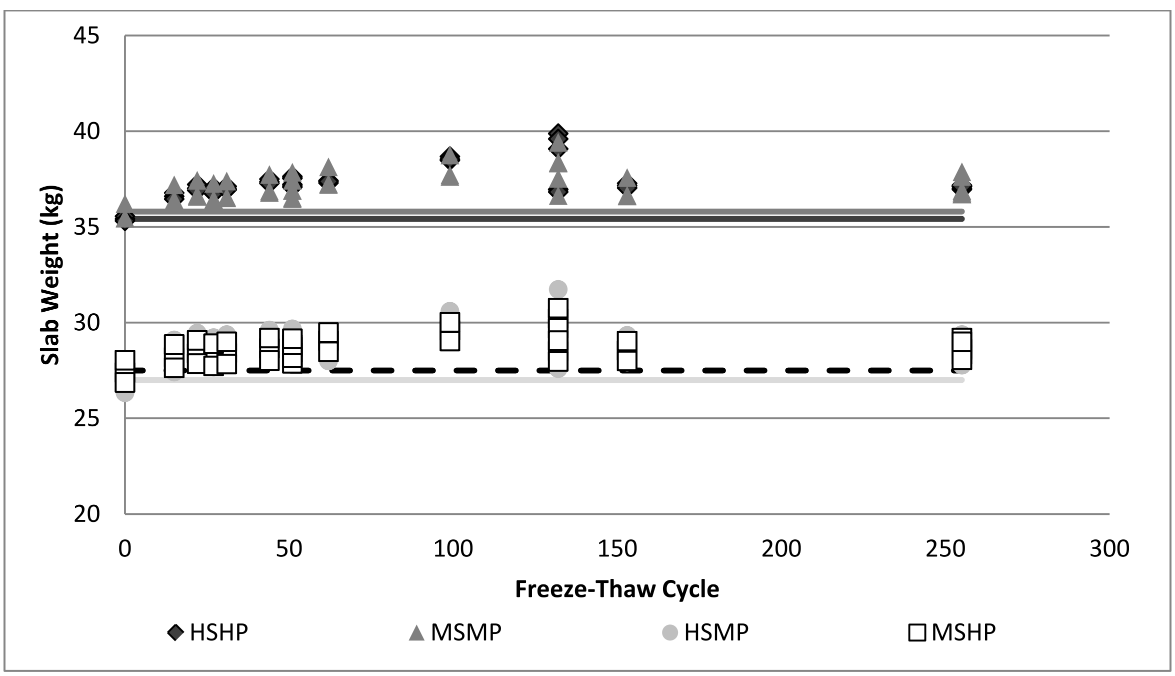


Figure 6-7: Weight of Slabs throughout Freeze-Thaw Cycles

In Figure 6-7 the horizontal line indicates the initial average weight of the slabs in each of the four slab groups. Similar to the cores, the weights never fell below the initial weight, therefore suggesting that extensive material loss was not occurring.

Table 6-8: Slab Surface Condition

Slab	**Age (Years)**		
	1.25 Years	2.5 Years	5 Years
HSHP - A	Very Slight Paste Loss	Slight Paste Loss, Slight Ravelling	Slight ravelling
HSHP – B	Fractured Aggregate, Slight Paste Loss	Slight Paste Loss, Slight Ravelling	Slight Ravelling
HSHP – C	Fractured Aggregate, Very Slight Paste Loss	Fractured Aggregate, Slight Ravelling	Slight Ravelling, Very Slight Paste Loss, Fractured Aggregate
MSMP – A	Moderate Paste Loss	Slight Ravelling, Slight Paste Loss	Slight ravell, Slight Paste Loss
MSMP – B	Moderate Paste loss	Slight Ravelling, Slight Paste Loss	Slight Ravelling, Slight Paste Loss
MSMP – C	Moderate Paste Loss	Fractured Aggregate, Slight Ravelling	Slight Ravell, Slight Paste Loss
HSMP – A	Very Slight Paste Loss	Slight Ravelling	Slight Ravelling
HSMP – B	Fractured Aggregate, Slight Paste Loss	Fractured Aggregate, Slight Ravelling	Fractured Aggregate, Slight Ravelling, Slight Paste Loss
HSMP – C	Very Slight Paste Loss	Fractured Aggregate, Slight Ravelling	Slight Ravelling, Slight Paste Loss
MSHP – A	Very Slight Paste Loss	Fractured Aggregate, Very Slight Paste Loss, Slight Ravelling	Slight Ravelling
MSHP – B	Very Slight Paste Loss	Very Slight Paste Loss, Fractured Aggregate, Slight Ravelling	Slight Ravelling
MSHP – C	Slight Paste Loss, Fractured Aggregate	Fractured Aggregate, Slight Paste Loss, Very Slight Ravelling	Very Slight Ravelling
HSaltHP - A	Fractured Aggregate, Moderate Paste Loss	Extensive Paste Loss, Fractured Aggregate	n/a
HSaltHP – B	Fractured Aggregate, Moderate Paste Loss	Moderate Paste Loss, Fractured Aggregate, Salt Visible on Surface	n/a
HSaltHP – C	Fractured Aggregate, Moderate Paste loss	Extensive Paste Loss, Fractured Aggregate, Salt Visible on Surface	Surface Colour has Darkened, Extensive Material Loss on Bottom of Slab, Fractured Aggregate, Extensive Paste Loss
HP – A	Fractured Aggregate, Slight Paste Loss	Very Slight Ravelling	Very Slight Ravelling
HP – B	Fractured Aggregate, Slight Paste Loss	Fractured Aggregate, Slight Ravelling, Very Slight Paste Loss	Slight Ravelling
HP – C	Fractured Aggregate, Slight Paste Loss	Very Slight Paste Loss, Very Slight Ravelling	Very Slight Ravelling

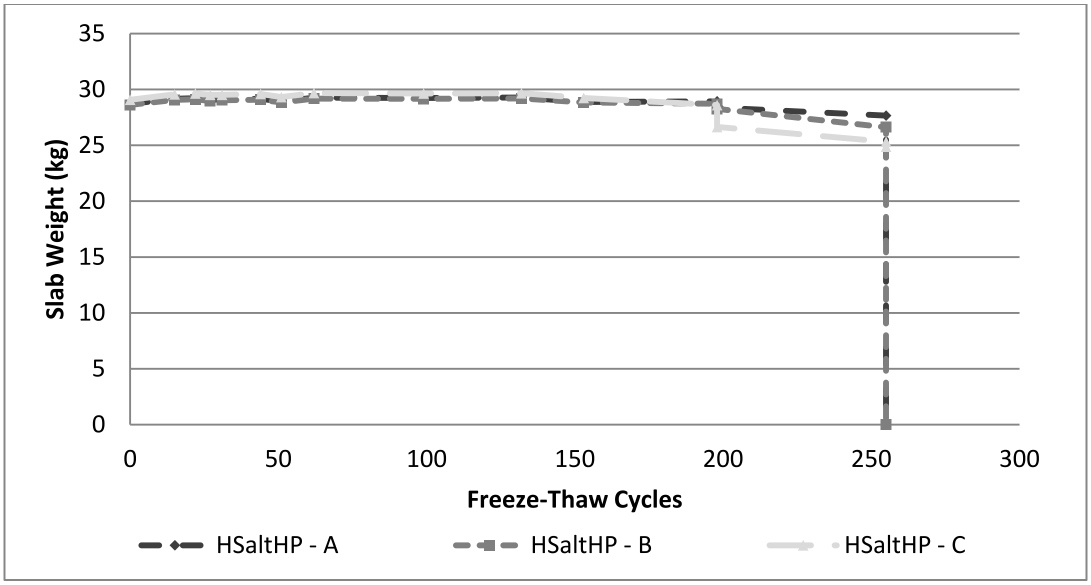


Figure 6-8: Weight of Slab Group HSaltHP throughout Freeze-Thaw Cycling

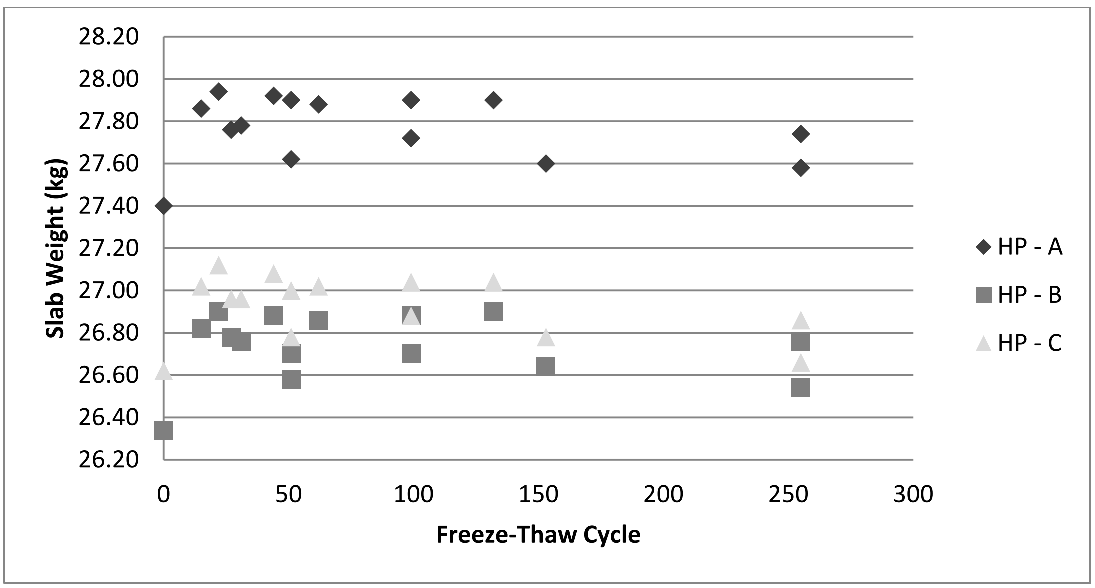


Figure 6-9: Weight of Slab Group HP

Figure 6-8 shows the weight of the slabs in Group HSaltHP during the five years of freeze-thaw cycling. In all three slabs, material loss became visible at approximately 150 cycles (after three years of exposure).

Figure 6-9 shows the weight of the slabs in Group HP during the five years of freeze-thaw cycling. The three slabs in this group were only loaded with precipitation and no winter maintenance materials. They remained within approximately 500 g of their initial weight throughout the five years of freeze-thaw cycling.

6.4.5 Permeability of Field Sites

The five field sites included in this analysis each experience a different combination of traffic and environment, as is described in Chapter 3.0. Figure 6-10 shows the average permeability of the five field sites since construction.

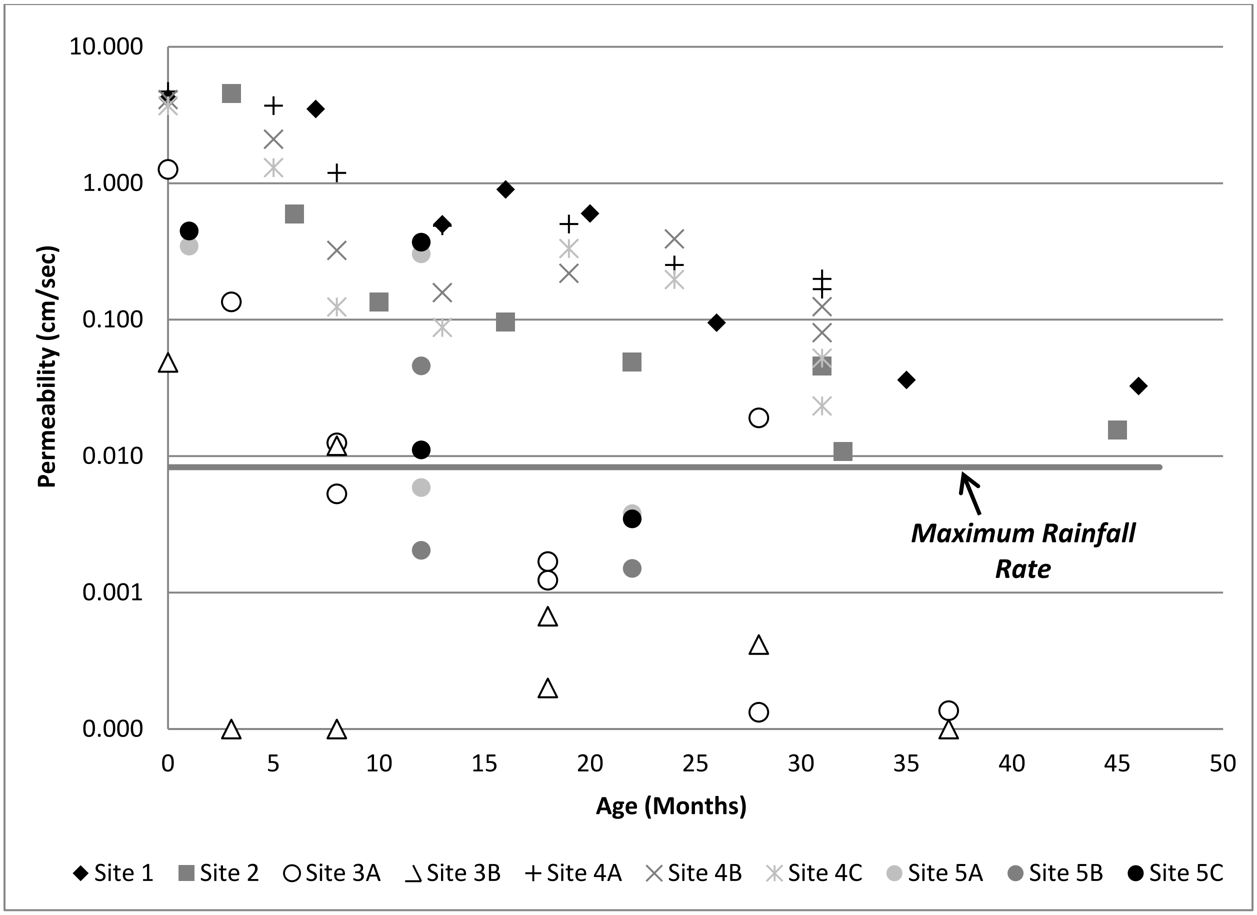


Figure 6-10: Field Site Permeability

As Figure 6-10 shows, Sites 1, 2, and 4 all had high permeability rates initially (more than 3 cm/sec). In comparison Sites 3 and 5 had lower permeability rates; however, still more than adequate given it was much higher than the "maximum rainfall rate" (Environment Canada, 2007). The maximum rainfall rate is discussed in Chapter 3.0. The maximum rainfall intensity of a rain event at each of the field sites was determined from intensity frequency duration plots. The value was similar for all sites across the country and the highest was selected 0.0083 cm/sec. The maximum rainfall rate was deemed to be a guide in this research as to whether a field site was draining adequately. This rate was verified to be suitable through observations in field site visits. Test results meeting or exceeding 0.0083 cm/sec were visually much slower than most of the areas that appeared to be effective. Additionally, the reported permeability rate of a field site is the average of the entire site and therefore does not indicate that an entire site or even half of the site is draining at this rate. Figure 6-10 shows that the sites have all decreased in permeability but have generally remained above the maximum rainfall rate, therefore remaining functional. The data shown in Figure 6-10 is the average of the measurements taken across the sites and at each site there are areas with higher and lower permeability rates than that presented.

The permeability results for each field site were compared further using ANOVA tables. At 95% confidence all of the testing sessions for a specific field site were evaluated to determine if there was a significant difference amongst them. Table 6-9 shows a summary of the ANOVA tables and the complete ANOVA table for each site is included in Appendix D.

Table 6-9: Permeability ANOVA Table Results for Field Sites

Field Site	**F_{calc}**	**F_{crit}**	**Outcome**
1	15.947	2.191	Significantly Different
2	45.408	2.103	Significantly Different
3A	4.877	2.032	Significantly Different
3B	1.606	2.359	Statistically Same
4A	23.467	2.145	Significantly Different
4B	29.355	2.15	Significantly Different
4C	34.891	2.172	Significantly Different
5A	4.293	2.29	Significantly Different
5B	7.015	2.413	Significantly Different
5C	4.575	2.579	Significantly Different
5D	0.982	3.682	Statistically Same

The ANOVA results showed similar results to those visually observed in Figure 6-10. The permeability at each of the field sites has changed over time. The only field sites that did not show a change over time was 3B and 5D. Site 3B is small in area and had a low permeability rate immediately following construction. The result of the ANOVA test is expected since the permeability rate was already low after construction. The 5D portion of Site 5 is a small area, with six test points. A portion of test points drain well and the others have limited to no permeability. The mean permeability of 5D likely has not changed substantially since construction in 2009.

6.4.6 Surface Condition of Field Sites

The surface condition of each of the five field sites is summarized in Table 6-10 and Table 6-11. Table 6-10 and Table 6-11 show the surface distresses that have developed over time at each of the five field sites. Some of the surface distresses were primarily site specific while others were present to some extent at each site. Ravelling within slabs and at joints was present at all sites. Sites 3, 4 and 5 had limited cracking develop. In each case it was a result of not including any joints or the joints not matching the location of those in adjacent concrete pavement.

Ravelling of the construction joint was site specific to Site 4. Fractured aggregate was also seen primarily at Site 4. It was not present to the same extent at the other field sites.

Localized, deep, very severe ravelling only developed at Site 2. The presence of cracks that were ravelled on the edges also only developed at Site 2. These cracks are suspected to be a structural deficiency or pavement structure failure as discussed in Chapter 4.0.

Surface sealing was present more at Site 3 than any of the other sites. At Site 3 impermeable areas where the surface was sealed were present immediately following construction. This surface sealing was determined to be due to the mix design. At Site 5 one area was sealed immediately following construction as well, 5D. In this area the vibratory plate compactor was passed over the pervious concrete multiple times. This was the first area that was constructed and the crew was still becoming familiar with pervious concrete pavement. Due to multiple passes of the vibratory plate compactor, the surface was sealed. At some locations within the other sites, such as Sites 2 and 5, the permeability dropped substantially over time. However, this is suspected to be due to debris accumulation rather than the mix sealing on the surface during construction.

6.5 Analysis

6.5.1 Permeability

The cores and slab group HP were each exposed to similar conditions. All of the slabs cast in this project appeared consistent in density and void distribution throughout. As is noted earlier, the cores tended to appear denser at the top (surface) and to have more voids at the bottom. Both the cores and slab group HP showed fluctuations in permeability but did not show a continual, constant trend in terms of change in permeability. The changes in permeability of these samples is suspected to be related to two conditions; changing internal structure of the pervious concrete sample; and variability in the test procedure.

Changes in the internal structure of the pervious concrete samples are believed to occur based on two occurrences in this research. The first occurrence being the failure of the Latex Core (4) from Site 5B. As noted in Chapter 4.0, when the Latex Core (4) failed the visible interior of the core appeared as though the paste had completely broken apart or fractured. This fracturing of the paste is likely occurring in the other cores as well and as the fine particles from the fractured paste move within the core, the permeability is changing. The fine particles moving within the core would close or partially close existing voids but also create new voids. The cores have not all failed likely due to the variability in the void structure in each core. It is unknown as to whether the paste was thinner in the core that did fail. However, this seems possible as the permeability of both the latex cores were some of the highest in comparison to the other cores.

It is anticipated that some of the fluctuation in permeability in slab group HP is also due to changes in the internal structure of the slab. The characteristics of the paste in a pervious concrete pavement could be reflected in the amount of change that is seen in the internal structure of the pervious concrete pavement.

The method used to test the rate of permeability of the pervious concrete in this project inevitably allows for variation. Variation can result from differences in head in the permeameter between various testing sessions. The equation used to measure permeability includes the points on the permeameter that the change in head is measured between. However, it has been observed that this effects the calculated permeability. Additionally, the intent is to measure permeability at

Table 6-10: Field Sites 1, 2 and 3 Surface Conditions

Site	Surface Condition (Age - months)										
	1 – 4	**4 – 8**	**8 - 12**	**12 – 16**	**16 – 24**	**24 – 28**	**28 – 32**	**32 – 36**	**36 – 40**	**40 – 44**	**44 – 48**
1	Slight ravelling in slabs, 10 – 20%		Moderate ravelling in slabs and joints, 20 – 50%,			Moderate ravelling 50 – 80%, slight aggregate fracturing 10 – 20%,moderate joint ravelling 20 – 50%		Moderate joint and slab ravelling 50 – 80%, <10% surface abrasion, severe ravelling <10%			80% - 100% Slight Ravelling, <10% Very Severe Ravelling, <10% Surface Abrasion, Fractured Aggregate, 50% - 80% Slight Joint Ravelling
2	Slight ravelling in slabs and joints, 20 – 50%	Moderate and severe ravelling in slabs and joints, 20%, slight surface abrasion, 10%	Moderate and severe ravelling in slabs and joints, 70%,	Slight ravelling in slabs and joints, 80%	Moderate ravelling in slabs and joints, 80%	Moderate ravelling in slabs and joints 80%, slight surface abrasion <10%				80% - 100% Moderate Ravelling, 80% - 100% Moderate Joint Ravelling, Potholes 4.5cm in Depth (Very Severe Ravelling), <10% Cracks with Ravelling	80% - 100% Moderate Ravelling, <10% Potholes (Very Severe Ravelling), 80% - 100% Moderate Joint Ravelling, 80% - 100% Paste Loss, <10% Cracks with Ravelling
3A	Surface sealing, 20%, slight ravelling, <10%		Slight cracks, <10%, moderate ravelling, 20% - 50%		Moderate ravelling in slabs and joints, 50% - 70%	Moderate ravelling in slabs and joints, 50% - 70%, Surface sealing					
3B	Surface sealing, 20%		Surface sealing, 50%			Surface sealing, 80%					

Table 6-11: Field Sites 4 and 5 Surface Conditions

Site	Surface Condition (Age - months)								
	1 – 4	**4 – 8**	**8 – 12**	**12 – 16**	**16 - 24**	**24 – 28**	**28 – 32**	**32 – 36**	**36 – 40**
4A	Construction joint slight to severe ravelling, 40%	Aggregate failure, 10%		Moderate joint ravelling 20-50%, moderate slab ravelling <10%, aggregate failure <10%		Severe joint ravelling 80%, moderate surface abrasion <10%, aggregate failure 20-50% - *Under wheels of vehicles*, very slight crack		10% - 20% Severe Ravelling, <10% Surface Abrasion, 20% - 50% Fractured Aggregate, 80% - 100% Severe Joint Ravelling	10% - 20% Moderate Ravelling, <10% Very Severe Ravelling <10% Surface Abrasion, 80% - 100% Moderate Joint Ravelling, 20% - 50% Fractured Aggregate, Two Cracks One is Ravelling and Other is Not
4B		Aggregate failure, 20% - 50%, Moderate ravelling, 20% - 50%		Moderate joint ravelling 80-100%, moderate slab ravelling <10%, slight surface abrasion <10%		Moderate joint ravelling 80-100%, moderate slab ravelling <10%, slight surface abrasion <10%		10% - 20% Moderate Ravelling, <10% Surface Abrasion, Fractured Aggregate, 20% - 50% Moderate Joint Ravelling, 1 Crack, 10% - 20% Paste Loss	10% - 20% Moderate Ravelling, Fractured Aggregate, One Crack, 20% - 50% Moderate Joint Ravelling
4C		Slight ravelling, 20%		Moderate joint ravelling 20 – 50%, aggregate fracturing <10%, slight surface abrasion <10%		Moderate joint ravelling 20 – 50%, slight surface abrasion 10-20%, aggregate fracturing <10%		10% - 20% Slight Ravell, Surface Abrasion, <10% Fractured Aggregate, 20% - 50% Moderate Ravelling Adjacent to Joint, One Crack	10% - 20% Surface Abrasion, <10% Fractured Aggregate, Two Cracks One is Ravelling Other is not, 20% - 50% Moderate Ravelling adjacent to Joint
5A	Slight crack, <10%, Very slight ravelling, <10%	10% - 20% Moderate Ravelling, 20% - 50% Moderate Joint Ravelling		10% - 20% Moderate Ravelling, Sand on Surface		Not Applicable			
5B	Very slight ravelling, <10%	<10% Slight Ravelling, 10% - 20% Slight Joint Ravelling		10% - 20% Slight Ravelling, Sand on Surface					
5C	Very slight to slight ravelling, 10% - 20%	20% - 50% Moderate Ravelling, 20% - 50% Moderate Joint Ravelling		Meandering Crack Adjacent to Joint, 20% - 50% Moderate Ravelling					
5D	Uneven surface	<10% Moderate Ravelling, 10% - 20% Joint Ravelling		<10% Surface Abrasion (many areas were ground down and therefore prior ravelling is not present)					

the same location on samples during each testing session. However, if permeability is measured at a slightly different location on the surface this could also lead to differences between measurements.

Slab group HSaltHP was initially anticipated to behave in a similar manner as the cores and the HP slabs given that it was receiving winter maintenance in the form of a liquid. Initially the HSaltHP slabs exhibited the same permeability trend as the HP slabs. These differences were noted but did not result in any apparent continual decrease in permeability. After approximately 100 freeze-thaw cycles (ie. 100 loadings with the salt solution) the permeability of the HSaltHP slabs began to decrease noticeably in comparison to the HP slabs. A constant decrease in permeability was identified. It is anticipated that although the salt was being loaded onto the slabs in a solution state, the following two activities were occurring:

1. Salt solution was becoming trapped in the pervious concrete. The solution has drained into a void that is not connected to other voids in the pervious concrete. The water in the solution evaporates over time and the salt remains in the void. The salt may build up in the void over time and block voids that did previously allow for drainage through the pervious concrete. It is anticipated that moisture from a rain event would also become trapped in these voids. However, it would not be an issue in the long term as the moisture would eventually evaporate entirely.
2. All of the salt solution is not flowing through the pervious concrete structure. It is anticipated that this is occurring to some degree. The surface of the HSaltHP slabs all changed in colour from that of the other slabs. The slab surfaces became darker and eventually salt accumulations were visible and were white. The salt solution was applied at the same rate it would be on a parking lot and without continual additional water not all of the solution moved into and through the pervious concrete. Salt solution that did move further than the surface of the pervious concrete slab may not have drained all the way through the pervious concrete simply due to the limited amount of solution that was applied with each winter maintenance loading.

The slabs loaded with sand as a winter maintenance technique showed more change in permeability than the control slab group, HP and the cores. The decrease in permeability of the slabs loaded with sand was not so extensive as to bring the pervious concrete close to being inadequate though. The pervious concrete is deemed to not be functional when the permeability is below the maximum rainfall rate. The application of sand to the surface alone did not prove to make the pervious concrete incapable of draining water at an adequate rate.

The field sites demonstrated more changes in permeability than the slabs in the laboratory based on visual observation from Figure 6-10. The most change in permeability was seen at the field sites. Visually the slabs in the laboratory had a thicker layer of sand on them on many occasions than the field sites did. The activities existing from full scale daily use on the pervious concrete field sites therefore appear to have a factor in the change of permeability over the long term. During the time that the 10 field site sections were monitored in this research, five dropped below the sufficient level of permeability. The five that dropped below this level were the sections that all had lower initial permeability rates. These sites were also only visited annually. If pervious concrete has a low permeability rate immediately following construction and is routinely exposed to debris, it is possible that maintenance will be required more regularly to maintain adequate permeability. Sites 3 and 5, similar to 1 and 4, were constructed in extreme locations. Typically it would not be recommended that pervious concrete be included in these locations without routine maintenance due to the large amount of debris from surrounding activities and pavement. In general the changes in permeability rates experienced at the field sites in this research is not a substantial concern. It is necessary to monitor the condition of the pervious concrete pavement permeability with testing. This monitoring can be carried out by simply pouring water on the pavement surface and visually monitoring changes in permeability rates over time.

To further assess if exposure to traffic and routine activities affected the permeability of pervious concrete, the permeability of the field sites and the laboratory slabs were compared statistically. Table 6-3 and Table 6-4 and Figure 6-5 showed that the amount of sand and the amount of permeability loaded on the slabs did not affect the permeability of the slabs. Given this finding, the results of slab groups HSHP, MSMP, HSMP and MSHP were combined to form one data set for comparison to the field sites. The testing ages of the field sites and slabs were compared and divided into five groups. The five groups are noted below:

1. 6 to 8 Months
    - Site 1 at 7 Months
    - Site 2 at 6 Months

- Site 3 at 8 Months
- Site 4 at 8 Months
- Slabs at 6 or 7 Months

2. 11 to 13 Months
   - Site 1 at 13 Months
   - Site 2 at 11 Months
   - Site 4 at 13 Months
   - Site 5 at 12 Months
   - Slabs at 12 Months
3. 17 to 24 Months
   - Site 1 at 20 Months
   - Site 2 at 22 Months
   - Site 3 at 17 Months
   - Site 4 at 20 or 24 Months
   - Site 5 at 22 Months
   - Slabs at 22 Months
4. 28 to 31 Months
   - Site 2 at 31 Months
   - Site 3 at 28 Months
   - Site 4 at 31 Months
   - Slabs at 31 Months
5. 45 to 60 Months
   - Site 1 at 46 Months
   - Site 2 at 45 Months
   - Slabs at 60 Months

In age groups 1 and 3 there is one group of data (Slabs at 6 or 7 Months and Site 4 at 20 or 24 Months) that includes two dates. These data sets had two sets of test results within the age range being considered in the analysis. Before selecting one set over another for the slabs at 6 and 7 months of age, a t-test was done to evaluate if the data sets were not equivalent. The null hypothesis for the t-test was that the mean permeability of one data set was equal to the mean permeability of the other. The T_{calc} result was 4.33 and the T_{crit} was 2.12. At 95 % confidence, the two sided t-test demonstrated that the permeability of the slabs at 6 and 7 months of age were statistically different. The results from 7 months of age were used as the other data sets being used were from 6, 7 and 8 months of age. By using the slab data from 7 months of age, the data set was falling in the middle of the range of ages being considered and therefore not intentionally altering the data towards one side of the range of ages.

Site 4 has three sections, 4A, 4B and 4C. Prior to comparing the values at 20 months of age to those from 24 months of age, the results from the same testing period were compared. An ANOVA table was used to assess if differences were present between the results of mixes 4A, 4B and 4C at the same age. At 95 % confidence the F_{calc} value was 13 and the F_{crit} was 3.369. Therefore the means of the three mixes at 20 months of age had differences between at least two of the mixes. The complete ANOVA table is included in Appendix D. Since the results demonstrated that at least two of the mixes had mean permeability rates which were statistically different it was determined that the results of each mix would be compared individually in the comparison with other sites and slabs, rather than considering the entire site as one data set.

To continue evaluating if data from the age of 20 months or 24 months would be used in the third age grouping, three t-tests were carried out. The three t-tests were done to attempt to identify if one mix had a statistically different mean permeability rate between the ages of 20 months and 24 months. The t-tests were two sided, at 95 % confidence and with a null hypothesis assuming that the mean permeability rates from each testing age were equal. Table 6-12 shows the results of the t-test for each of the three mixes.

Table 6-12: t-test of Mean Permeability Rates at 20 Months and 24 Months of Age at Site 4

Mix	**T_{calc}**	**T_{crit}**	**Outcome**
Mix 4A	1.662	2.160	Statistically Same
Mix 4B	-0.852	2.201	Statistically Same
Mix 4C	0.818	2.228	Statistically Same

Table 6-12 shows that for all three mixes at Site 4 the t-test failed to reject the null hypothesis and therefore the mean permeability rates at ages 20 and 24 months were statistically the same. Considering this result, the permeability data from both 20 months of age and 24 months of age will be used in the third age group as the other four data sets range in age from 20 to 24 months.

Site 5 also has multiple sections, four in total and therefore a brief analysis was carried out to evaluate if the data from each section could be combined for the comparisons or if it should be considered individually. An ANOVA table was used to compare the mean permeability rate of each of the four sections at 12 months of age. Some sections have multiple days of testing at 12 months of age because permeability renewal maintenance methods were used on the pavement. For all sections the initial permeability at a particular age was used in this analysis. The four sections were compared at 95 % confidence assuming a null hypothesis that the mean permeability rates of all sections of the site were equal. The F_{calc} value was 10.000 and the F_{crit} value was 2.922, therefore rejecting the null hypothesis and indicating that at least two of the sections have significantly different mean permeability rates. The ANOVA was repeated using only sections 5A, 5B, and 5C. Section 5D was not included as it is fairly unique from the other pavement sections because it receives essentially no traffic. The construction of section 5D also led to the majority of the section being impermeable immediately following placement and the remainder of the site being of very good quality and highly permeable. Section 5D was constructed initially during the construction of Site 5 and is a small area. The comparison of the mean permeability rates of the other three sections at a 95 % confidence interval using an ANOVA table and assuming a null hypothesis that all mean permeability rates were equal led to a F_{calc} of 12.500 and an F_{crit} of 3.354. The null hypothesis was therefore rejected and the mean permeability rates of at least two of the sections of the site were found to be significantly different. Section 5D was deemed to be an outlier of the dataset, based on the reasons previously discussed and was not included in further analysis. The other three sections of Site 5, 5A, 5B and 5C were all included in the analysis and were considered individually. Both ANOVA tables are included in Appendix D.

Site 3 has two sections, 3A and 3B. The mean permeability rates of the sections were compared for each testing session using a t-test. The t-test was two sided and carried out at a confidence level of 95 %. There were six testing sessions, 0, 3, 8, 17, 28 and 37 months of age. All of the t-tests failed to reject the null hypothesis therefore proving that the mean permeability rates were statistically the same except at 17 months of age. At 17 months of age the mean permeability rates of the two sections of Site 3 were found to be statistically different.

Considering these results, the permeability rate results of Site 3 from 8 and 28 months of age were combined. For the analysis at 17 months of age, only the results from Site 3A were used. Section 3B is small and has only five test points.

An ANOVA table was used for each of the five age groups. The complete ANOVA tables are included in Appendix D and a summary of the results are shown in Table 6-13. The analysis was repeated for age groups 1, 3 and 5. Site 3 is unique from the other sites as it does not receive winter maintenance. To evaluate the effect of this the groups including data from Site 3 were repeated, once including Site 3 and once without Site 3. The null hypothesis for each comparison was that the mean permeability of all the field sites and the slabs was statistically equal within a particular age range, at 95 % confidence.

Table 6-13: Statistical Analysis of Permeability from Field Sites and Laboratory Slabs at Various Ages

Age Group	Fcalc	Fcrit	Outcome
1 (6 to 8 Months)*	13.342	2.317	Statistically Different
1 (6 to 8 Months)	27.961	2.191	Statistically Different
2 (11 to 13 Months)	2.977	2.032	Statistically Different
3 (20 to 24 Months)*	0.865	2.032	Statistically Same
3 (17 to 24 Months)	0.958	1.850	Statistically Same
4 (31 Months)*	1.904	2.525	Statistically Same
4 (28 to 31 Months)	3.686	2.330	Statistically Different
5 (45 to 60 Months)	9.764	2.540	Statistically Different
* Site 3 not included			

Table 6-13 shows the results of the statistical comparison of the mean permeability rates for each field site and the slabs during a particular age range. The purpose of this statistical comparison was to evaluate if there were differences in permeability between the field sites and the slabs at the same age. Most of the field sites and all the slabs included in the analysis were exposed to freeze-thaw cycles and winter maintenance however only the field sites experienced regular loading and use by vehicles and pedestrians. Site 3 did not experience winter maintenance and this is why it was evaluated separately. The results show that there is a statistical difference between at least two of the mean permeability rates in age groups 1, 2 and 5. When Site 3 was included in group 4 there was also a statistical difference in the mean permeability rates. Although it may have been expected that Site 3 would have better permeability due to not having winter maintenance applied, the results in Figure 6-10 suggest that this is not the case. It is anticipated that when Site 3 is included in the analysis the mean permeability rates are proven to be statistically different because Site 3 is lower than the others. The null hypothesis was not rejected for age groups 3 and 4 (when Site 3 was excluded) therefore indicating that no difference occurred between any two mean permeability rates at this age and therefore the permeability rates of all sites were statistically the same.

Although it is known that a statistical difference existed between the mean permeability rates of two of the data sets in age groups 1, 2, 4 (when Site 3 was included) and 5, it is unknown if the difference was between two field sites or between a field site and the slabs. For the purpose of this evaluation it is of interest to know if the mean permeability of a field site was statistically different than that of the slabs. To assess this, confidence intervals at 95 % confidence were calculated for age groups 1, 3 and 5. Confidence intervals were not calculated for age group 4, because when Site 3 was not included the mean permeability rates were statistically the same. Therefore Site 3 has a different mean permeability rate than the slabs and other field sites and this is supported by the results shown in Figure 6-10. For each age group a confidence interval between each field site and the slab group was determined. The calculated confidence intervals for the three age groups are shown in Table 6-14.

Table 6-14 shows the confidence interval for the difference in permeability between the field site and the slabs at 95 % confidence. The confidence intervals that do not include zero have been shaded for identification purposes. If the confidence interval does not include zero then at 95 % confidence the mean permeability rates of the particular field site and the slabs are not equivalent. In all scenarios the comparison was carried out such that an entirely positive confidence interval would represent the mean permeability of slabs being greater than that of the field site.

Many of the confidence intervals shown in Table 6-14 include zero, therefore not indicating a difference between the mean permeability of the particular field site and the slabs. Within the first age group, only the comparison of Site 1 and the slabs showed a confidence interval that indicated the mean permeability rates of the two sample types were not equal. The confidence interval was entirely negative and given the assumptions in the confidence interval calculation, this indicates that the mean permeability of Site 1 was greater than that of the slabs at 95 % confidence. This finding reflects the high permeability rate of Site 1 after construction and following months.

Table 6-14: Confidence Intervals of Mean Permeability Rates Between Field Sites and Laboratory Cast Slabs

Age Group	Comparison	Low	High
1	Slabs to Site 1	-4.286	-2.064
	Slabs to Site 2	-1.395	0.856
	Slabs to Site 4A	-2.276	0.549
	Slabs to Site 4B	-1.408	1.417
	Slabs to Site 4C	-1.260	1.665
2	Slabs to Site 1	-0.431	0.245
	Slabs to Site 2	-0.035	0.660
	Slabs to Site 4A	-0.494	0.353
	Slabs to Site 4B	-0.162	0.742
	Slabs to Site 4C	-0.111	0.831
	Slabs to Site 5A	0.018	0.866
	Slabs to Site 5B	-0.006	0.897
	Slabs to Site 5C	0.013	0.871
5	Slabs to Site 1	0.076	0.167
	Slabs to Site 2	0.108	0.201
Shaded cells are confidence intervals that do not include zero, therefore indicating a statistically difference in these results.			

In the second age group of testing Sites 5A and 5C were found to have lower mean permeability rates at 95 % confidence than the slabs. The other field sites all had confidence intervals that suggested that the mean permeability rates of the sites and the slabs were statistically the same at 95 % confidence. All three sections of Site 5 included in this comparison had lower initial permeability rates than the other field sites. The lower initial permeability is possibly the reason for sites having a lower mean permeability rate than the slabs at 95 % confidence at 12 months of age.

The final age comparison group had a large range in sample ages. Both field sites were between 45 and 46 months in age while the slabs were 60 months in age. The confidence intervals of the mean permeability rates between each site and the slabs indicated that in both scenarios the slabs had a greater mean permeability at 95 % confidence. The slabs are more than one year older than the field sites in these comparisons.

The results of the ANOVA tables, summarized in Table 6-13 and the results shown in Table 6-14 present the following trend in terms of the effect of regular, daily use of pervious concrete pavement by vehicles and pedestrians on the permeability rate.

- After approximately six months of exposure and use by the public the pervious concrete field sites were performing statistically the same or better than the slabs, in terms of permeability.
- At one year of age many field sites are performing statistically the same as the slabs. However, field sites with lower initial permeability rates did show lower permeability rates than the slabs that were not exposed to daily use by the public.
- At 1.5 to 2.5 years of age the field sites and slabs were performing similarly in terms of permeability. The winter maintenance applied to the slabs had caused the permeability of the slabs to remain comparable to that of the field sites which are exposed to varying winter maintenance and public use. However, the field site that was exposed to no winter maintenance but did initially have a low permeability rate had a lower permeability rate at 2.5 years of age than all other samples, both slabs and field sites.
- When the field sites had reached nearly four years of age the permeability rates were lower than those of the slabs which were five years in age. Regular vehicular and pedestrian traffic exposure led to lower permeability rates at the field sites as compared to slabs which had only been loaded with winter maintenance.

The core samples included in this analysis were the most controlled, being exposed to only freeze-thaw cycles. The permeability of the cores were measured four times throughout the completion of 250 freeze-thaw cycles. An ANOVA table was used to evaluate if the mean permeability of the cores was different throughout the freeze-thaw cycles. The null hypothesis was that the mean permeability of the cores after 0, 100, 200 and 250 freeze-thaw cycles were equal. At 95 % confidence the test ANOVA results failed to reject the null hypothesis as the F_{calc} result was 0.611 and the F_{crit} was 3.197. Therefore the mean permeability rates of the cores were statistically the same after different quantities of freeze-thaw cycles. The complete ANOVA table is provided in Appendix D.

A comparison was carried out to assess if the application of winter maintenance to the slabs led to a difference in permeability between the slabs and the cores throughout the testing period. The mean permeability rate of the cores and slabs were compared using a t-test. The comparisons shown in Table 6-15 were made with a two sided t-test, at 95 % confidence. The null hypothesis of each t-test was that the mean permeability rate of the cores and slabs was equal. The HSaltHP and HP slabs were considered in one group for the t-test comparisons, based on the results shown in Table 6-7. All of the slab groups receiving winter maintenance in the form of sand were combined and compared to the cores, based on the results of Table 6-3 and Table 6-4. The outcome of these comparisons is shown in Table 6-16.

Initially the permeability rates of the slabs were shown to be statistically different from the cores. This corresponds to the data presented in **Error! Reference source not found.** and Figure 6-5. The mix design of the cores created a pervious concrete pavement with lower permeability rates than that of the slabs' mix design. After 100 freeze-thaw cycles and loading of sand for winter maintenance, the permeability rates of the cores and the slabs loaded with sand were statistically the same. The slabs loaded with salt and the cores were shown to be significantly different from each other after 99 and 132 freeze-thaw cycles than the cores. The salt was not shown to decrease the permeability to a level comparable to the cores.

Table 6-15: Core and Slab Comparison

Core Freeze-Thaw Cycles	Slab Freeze-Thaw Cycles	Slab Groups Compared to Cores
0	0	HSHP, MSMP, HSMP, MSHP
0	0	HSaltHP, HP
100	99	HSHP, MSMP, HSMP, MSHP
100	99	HSaltHP, HP
100	132	HSHP, MSMP, HSMP, MSHP
100	132	HSaltHP, HP
200	153	HSHP, MSMP, HSMP, MSHP
200	153	HSaltHP, HP
250	250	HSHP, MSMP, HSMP, MSHP
250	250	HSaltHP, HP

After 200 freeze-thaw cycles of the cores and 153 of the slabs the same trend remained. The sand loaded slabs and cores had mean permeability rates that were statistically the same. The cores and slabs loaded with salt however did show that they were statistically different in terms of mean permeability rates at 95 % confidence. After 250 freeze-thaw cycles however, neither the cores and sand loaded slabs nor the cores and salt loaded slabs were identified as having significantly different mean permeability rates. This comparison indicates that the application of sand to the pervious concrete pavement slabs changed the permeability over time. The application of salt solution to the slabs also changed the permeability, slower than the sand but did lead to a change.

Table 6-16: Comparison of Cores and Slabs Permeability

Freeze-Thaw Cycles - Cores	Slab Type	Freeze-Thaw Cycles - Slabs	T_{calc}	T_{crit}	Outcome
0	Sand	0	-5.773	2.201	Significantly Different
0	Salt/Control	0	-3.793	2.571	Significantly Different
100	Sand	99	1.909	2.776	Statistically Same
100	Salt/Control	99	-3.296	2.571	Significantly Different
100	Sand	132	2.114	2.776	Statistically Same
100	Salt/Control	132	-3.058	2.571	Significantly Different
200	Sand	153	0.687	2.776	Statistically Same
200	Salt/Control	153	-2.638	2.571	Significantly Different
250	Sand	250	1.969	2.776	Statistically Same
250	Salt/Control	250	-3.057	3.182	Statistically Same

6.5.2 Surface Condition

As noted earlier in the analysis, the development of some surface distresses in pervious concrete pavement does not necessarily impact the ability of the pervious concrete pavement to function adequately. Functioning adequately includes sufficient drainage capabilities and the ability for the user to use the pavement as intended. Development of surface distresses leads to poor performance from the owner and user perspective as the appearance of the pavement can deteriorate. If surface distresses develop to a sufficient extent in both severity and density then they may result in the pavement being less serviceable by users.

The cores, in general, and the control slabs, HP, showed very little surface distress development with exposure to moisture and freeze-thaw cycling. Both the cores and control slab have shown a small amount of ravelling. In the slabs there was initially paste loss visible and then ravelling. At five years of freeze thaw cycling the slabs do not continue to have paste loss. Since the presence of paste loss has ended it is anticipated that no further ravelling will occur on these slabs. Based on the condition observed on the other slabs and these slabs it is anticipated that in a controlled setting such as the lab, it is possible to notice paste loss on aggregates before ravelling occurs. Therefore the slabs suggest that under no winter maintenance and no traffic, minimum ravelling may occur initially but is not anticipated to continue. It is likely that where ravelling did occur, the paste did not cure sufficiently during casting and was weaker in these areas and therefore susceptible to deterioration.

Paste loss has not been visible in the cores but ravelling of individual aggregates off the edges of the cores has been noticed. Paste loss is likely not visible here as the aggregate on the outer surface of the core is not covered in paste. Since the core was removed from existing pavement, the aggregates on the edges on the core were cut directly through. Paste loss is anticipated to be occurring on the internal edge of these aggregates. Ravelling off the driving surface of the cores has not been identified and this is what would be visible in the field site. Ravelling from the edges of the cores could be a result of the paste being weakened or damaged slightly during coring. The latex core did however fail. The failure of the latex core appeared to be throughout and is not likely due to damage during coring.

The slabs loaded with salt solution, HSaltHP, showed the most severe surface distress of any of the samples in this project. All three slabs deteriorated to the extent that they were no longer holding their initial shape. The deterioration was in the form of cracks developing throughout the thickness of the slabs and aggregate ravelling off the bottom on the slabs in large handfuls. This ravelling is different than what was identified in the cores or field sites. In the cores or at the field sites ravelling appeared to occur at a rate of individual aggregates. In the HSaltHP slabs large chunks were separating from the slab. Given that the aggregate was separating in large chunks it is likely that salt had been building up along the area where the chunk fractured away from the slab. The believed reasons for salt buildup are described in the earlier in this chapter. The development of the cracks through the thickness of the slabs also suggests that salt was building up in the pervious concrete pavement. The easiest path initially for the water drainage through the pervious concrete is likely to have been along the path where the cracks developed and the salt was therefore deposited in this area. This collection of salt is anticipated to have caused the crack development by exceeding the capacity of the structure.

The slabs that were loaded with sand showed more surface distress development than the HP slab group but generally less than the field sites. The ravelling is generally a bit more severe on the slabs loaded with sand than the HP slabs. The other difference is that at five years of age, paste loss is visible on the slabs that were loaded with sand and the HP slabs did not show any paste loss at that age. This suggests that more ravelling may occur in the future on the slabs loaded with sand. It is anticipated that this additional ravelling is not entirely a result of the loading from sand but more so the frequency on the cleaning of the surface. During many freeze-thaw cycles and slab loadings there was a thick layer of sand on the surface of the slabs, thick enough that the pervious concrete was not visible. This thick layer, although permeable is less permeable than the pervious concrete pavement below. Water loaded onto the slabs during the precipitation loading may move slowly through the sand and in some cases still be in the sand when the slab is moved back into the freezer to initiate the freezing portion of the cycle. This water would have expanded and put pressure on the pervious concrete and paste, leading to the paste breaking down and fracturing off the aggregate. As the paste around an aggregate breaks down and fractures away from the aggregate, the bond keeping the aggregate secured to the remainder of the pervious concrete is lost and therefore ravelling occurs.

The field sites showed a range of surface distresses, more so than the samples in the laboratory. The development of more surface distresses at the field sites is expected as there are more variables in these scenarios. None of the field sites are loaded with salt solution or salt on a regular basis although it is known that salt is applied to Site 2 periodically during the winter season. With none of the field sites having been exposed to the same amount of salt or salt solution as the HSaltHP slabs it is expected that the deterioration and cracking distress were not identified.

All of the field sites have shown some ravelling. At Site 1 the ravelling has been observed to be comparable to the ravelling of the slabs loaded with sand. Over time, ravelling has occurred across the site and prior to the ravelling across the site being noticed, the site was covered in a layer of sand from the surrounding area.

The ravelling at Site 2 has become very severe in some areas with deep ravelled areas developing. Given the other findings in the this project, it is not anticipated that deep ravelled areas are occurring due to the Canadian climate but rather are a function of the conditions during construction and the method used for the construction.

Site 3 has shown limited surface distress development. Cracking has occurred which is anticipated to be a result of the slabs being long in length. Cracking has also developed to match the joint locations of adjacent concrete pavements. Some ravelling has developed within the slabs. However, in general the surface has remained in good condition.

Site 4 has demonstrated the best resistance to ravelling. Ravelling has occurred along the construction joints and this is believed to be due to the lack of time that the pervious concrete was given to cure during the construction phasing. Most of the area at Site 4 that is not adjacent to the construction joints has performed well with minimal ravelling. This performance suggests that when all three factors are present (freeze-thaw cycling, winter maintenance, and full scale traffic) extensive ravelling does not necessarily occur.

Site 5 has developed some ravelling but not extensively. The ravelling is likely a combination of paste that had not cured amply during construction and then some water being trapped in debris in localized areas. Immediately following construction some portions of the site were unfortunately loaded with some landscaping material.

The cracking development at Sites 3, 4 and 5 are not predicted to be due to the climate or winter maintenance. At Site 4 there were no joints included and minor cracks have developed as would occur in any type of concrete without joints to control the cracking. At Sites 3 and 5 there is conventional concrete adjacent to the pervious concrete. The joints in the conventional concrete do not match the location of the joints in the pervious concrete and this has led to cracks developing to join the two joint locations.

6.6 Summary

Pervious concrete pavement behaviour was evaluated under the presence of three factors: freeze-thaw cycles and moisture; winter maintenance; and pedestrian and vehicle traffic. Three types of samples were included in the evaluation: cores taken from field sites; slabs cast in the laboratory; and full scale field sites. The behaviour of the pervious concrete was assessed in terms of permeability and surface condition.

Freeze-thaw cycling and moisture were found to alter the internal structure of pervious concrete. The internal structure is altered when the paste in the pervious concrete deteriorates due to the freeze-thaw cycling. This deterioration creates small particles which can move throughout the pervious concrete and close previously available drainage paths. The extent to which this occurs is anticipated to be highly dependent on the characteristics of the paste in the pervious concrete.

The type of winter maintenance that was used on the pervious concrete affected the permeability and surface condition. The application of sand as the winter maintenance method decreased the permeability but not to an unacceptable level. When a salt solution was used the permeability also decreased. However, the decreased permeability was not as large as compared to when sand was applied. The surface conditions of the HSaltHP slabs were worse than any other samples in the project. All three slabs deteriorated to a point where the original size and shape of the slabs was not apparent.

Exposure to daily use conditions including pedestrian and vehicle traffic lead to decreases in permeability at all the field sites. In half the site sections the permeability remained greater than the maximum rainfall rate and therefore the pervious concrete pavement continued to drain adequately. The five field sites where the permeability rate did not remain adequate had lower permeability rates than the other sites immediately following construction. Two of these sections did not receive any winter maintenance. The initial permeability of the field sites proved to be important and although some sites start with what appeared to be very high permeability rates, these sites were successful in the multiple year evaluation in maintaining adequate permeability rates.

Surface distresses developed at each field site. The types of distresses that had developed in the cores and slabs in the laboratory were generally not substantially worse at the field sites, suggesting that pedestrian and vehicle traffic do not necessarily escalate distresses caused by the Canadian climate and corresponding winter activities. Additional distresses were observed on the surface of the field sites and these could be related to the mix designs, construction method or the features of the field site layout.

The testing carried out on the three types of samples in this project, in the four scenarios, identified some behaviours of pervious concrete that appear to be specific to the Canadian climate condition and associated activities, such as failure of the core and failure of slabs loaded with salt solution. In general though, it was identified that adequate surface and permeability performance could be achieved with pervious concrete pavement in Canada during the length of this research period. Adequate surface performance was deemed to occur when there was limited surface distress development. The permeability was found to be adequate when it remained greater than the maximum rainfall rate. The results in this chapter are considered to the applicable across Canada given the broad assessment that was carried out. More specific details for particular areas of Canada could certainly be considered in future research.

The evaluation of field sites was a maximum of four years in length and the cores were exposed to the equivalent of more than eight years of freeze-thaw cycles in the lab. Winter maintenance was applied to the slabs during each freeze-thaw cycle which is not felt to be entirely representative of real scenarios. It is anticipated that winter maintenance is only applied to a pavement once every two or three freeze-thaw cycles. This would suggest that the slabs were exposed to the equivalent of 15 years of freeze-thaw cycles. The scenarios in which the field sites were constructed in resulted in large quantities of debris on the pavements. The findings of this chapter suggest that a 15 year or more design life of a pervious concrete pavement would be possible if placed in a suitable scenario.

7.0 SUBSURFACE DRAINAGE BEHAVIOUR OF PERVIOUS CONCRETE PAVEMENT

Drainage of moisture from the surface of a pervious concrete pavement can be observed visually and quantified using equipment, as was described in Section 3.4.7. Quantifying and assessing drainage of moisture below the surface in a pervious concrete pavement structure is not straight forward. In this research, instrumentation was included in three of the field sites to attempt to monitor the movement of moisture below the surface. The instrumentation that was included in the field sites is described in Section 3.3.2. The data collected from the instrumentation is presented in this chapter as well as an analysis of the results.

A pervious concrete pavement structure is designed with the intention that moisture moves quickly through the pervious concrete layer. Moisture then moves through the granular storage base layer. The moisture accumulates in the bottom of the granular storage base layer until it naturally infiltrates into the existing subgrade. The analysis of the data collected from the instrumentation is expected to describe the subsurface behaviour of drainage of moisture through the pavement structure.

7.1 Moisture Gauge Data Computation

The moisture gauges described in Section 3.3.2 and included in the field sites are intended for use in agricultural applications. They were selected for use in this project as they can record measurements when not entirely surrounded by soil. This is important as both pervious concrete and clear stone have several voids which could lead to air being adjacent to the moisture gauges in some locations.

A program was developed in this research to record moisture measurements of the material surrounding each Watermark moisture gauge. The moisture gauges are sent an excitation voltage from the CR1000 datalogger (CR1000) and after a pause of three milliseconds the voltage is measured. The measured voltage divided by the excitation voltage is recorded by the CR1000. These readings are converted to resistance and then adjusted to 21°C using Equation 7-1, results were adjusted to 21 °C for comparison purposes.

$$R_{21} = \frac{R_m}{1 - [0.018(T_m - 21)]} \qquad \text{Equation 7-1}$$

Where,

R_{21} is the resistance adjusted to 21°C.

R_m is the resistance reading determined from the raw data collected by the CR1000, in kOhms.

T_m is the temperature of the soil surrounding the moisture gauge in °C.

The resistance at 21°C, R_{21}, is then converted to Soil Water Potential (SWP) in centibars (cb) using Equation 7-2.

$$SWP = 7.407\ x\ R_{21} - 3.704 \qquad \text{Equation 7-2}$$

Where,

SWP is the soil water potential measured in centibars (cb).

The unit of centibars is equivalent to kPa and therefore equal to kN/m^2. The data in cb represents water tension or SWP and in agricultural applications describes the availability of water for a plant root. When there is minimal available water the centibar values are higher. When water is readily available, the cb values are lower. The moisture gauges are calibrated for 0 to 200 cb as this range is typical in agricultural applications. Although the moisture gauges are calibrated to a range up to 200 cb, Irrometer Co. Inc., the producers and suppliers of Watermark moisture gauges predict that results beyond 200 cb are reasonably accurate and can be used in analysis (Penning, 2011).

Temperature sensors were included at each of the three field sites to provide subsurface data for use in analyzing the moisture gauge data. The temperature sensors are also connected to the CR1000 at each site and measurements are taken by the temperature sensors and moisture gauges at the same time. The temperature sensors must be surrounded by material in order to take an accurate reading. For this reason the temperature sensors were located close to the top of the existing subgrade at each site.

Both the temperature sensors and moisture gauges collect data hourly. This data was averaged over 24 hours to provide a daily value. The cb readings from the pervious concrete pavement structures are interpreted by comparing the slope between readings. Figure 7-1 presents an example of SWP data generated from moisture gauges over seven days.

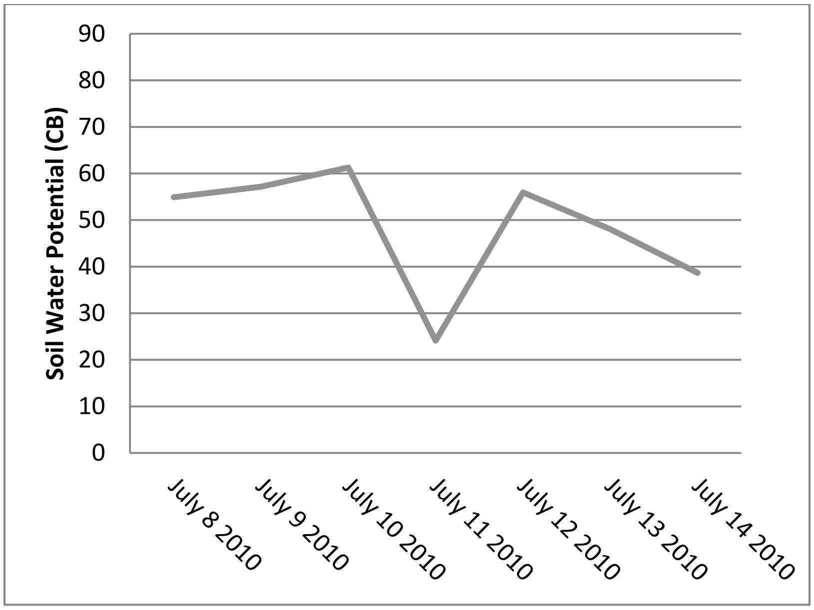


Figure 7-1: Example of Moisture Gauge Data

Figure 7-1 shows that the slope changes at July 10 2010, July 11 2010 and July 12 2010 in the plot. Prior to July 10 2010 the slope was positive, it then changed to negative, then positive and finally back to negative. Lower centibar values indicate that more water is present. The negative slope is caused by increased amounts of water moving into the instrumented area as compared to the amount of water draining away. The slope between July 11 2010 and July 12 2010 is positive, therefore water is draining away from the instrumentation at a higher rate than it is coming into the area. Thus, the pavement layer is drying out. In Figure 7-1 the soil is drying on July 8, 9 and 10 2010. On July 11, the soil becomes wet and starts to dry out until July 12 2010. On July 13 it becomes wet again. Coarser aggregate materials have higher SWP values when saturated as compared to finer grained materials. Saturation values of the pervious concrete and clear stone base layer should be greater than the values observed when the existing subgrade is saturated. The information that is of the most interest from the moisture gauge data is: whether water is travelling through the pavement structure; how quickly it is moving into each area of the pavement structure; and how quickly it drains away from the particular portion of the pavement structure. When the material surrounding the moisture gauge is frozen it is anticipated that a spike in readings would occur.

The results are primarily presented in tabular form. Within the results of each moisture gauge, the minimum value is highlighted. If no minimum is observed, then there are no highlighted values. This value corresponds to the July 11 2010 data shown in Figure 7-1, the point at which more moisture begins to drain away from the instrumented material than toward the material. In the tabular presentation of the results, it is possible to identify at which point moisture begins to move into the instrumented material as well. This occurs at the point in which the results start decreasing in value. Tables showing the daily moisture gauge results are included in Appendix E and summary tables have been presented in this chapter.

7.2 Weather Data

As described in Section 3.3.1 each field site includes a weather station that measures rainfall and temperature. Each weather station includes the same instrumentation: a tipping bucket; and an air temperature sensor in a radiation shield. The tipping bucket measures rainfall accumulation in 0.25 mm quantities. The temperature sensors were programmed to measure air temperature hourly. Both the temperature data and rainfall quantities are recorded on a HOBO event logger.

The weather stations do not record snowfall accumulation. With this in mind it was anticipated that weather data from the winter seasons would have to be collected from another source. Given the location of Sites 3 and 5 the weather stations were left outdoors all year long. In reviewing the data from these two weather stations it was deemed that the information collected in the winter seasons could be used in this project. At Site 4, the weather station was brought indoors in the winter season. Following recommendations from the Department of Environment and Resources Studies Environment Canada liaison at the University of Waterloo, data was downloaded from the Egbert weather station (Mills, 2011). This weather station was recommended based on its location and elevation. Daily data was downloaded for dates when the weather station was not on site.

The weather station data was used to identify dates that should be explored in the moisture gauge data. The weather data was divided into four groups annually, to generally represent the seasons. The months throughout the year were divided in the following way:

- Winter - December, January, February
- Spring – March, April, May
- Summer – June, July, August

- Fall – September, October, November

The rain is recorded in the tipping bucket in 0.25 mm intervals. The daily rain accumulation was determined from the tipping bucket data for each site. A continual summation was used to identify the largest quantity of rain over five days in each season. The rain event was then examined in the data. Ideally a rain event from each season would be used that was large in quantity and had minimal to no rain before and after the rain event for several days. It was anticipated that with a scenario such as this it would be easiest to follow moisture through the pavement structure. Some rain events were identified that met this description. However, in many cases this was not possible. The rain events that occurred in each season were evaluated and at least one was selected for analysis in this research. This was performed for each of the three instrumented field sites.

In the case of Site 4 where the weather station was not onsite all year the data from the Egbert weather station was used (Environment Canada, 2011). Maximum quantities of precipitation over five days were identified. Radar data from the King City, ON location was then used to confirm that precipitation events had occurred in the area of Site 4 on the highlighted dates (Environment Canada, 2011).

For the analysis of the two sites that had the weather stations on site year round, the weather station data was used in all seasons to highlight large storm events. Although the tipping bucket does not identify snow volumes it was deemed that activity in the tipping bucket during the winter seasons would directly relate to the presence of rain or melted snow attempting to drain through the pavement structure.

7.3 Site 3

Site 3 was instrumented with 15 moisture gauges during construction. Sensor trees were constructed for the moisture gauges, four in total, two in section 3A and two in section 3B. The dimensions of the sensor trees are shown in Figure 7-2 to Figure 7-5. Sensor Tree A and Sensor Tree B are in section 3A and Sensor Tree C and Sensor Tree D are in section 3B.

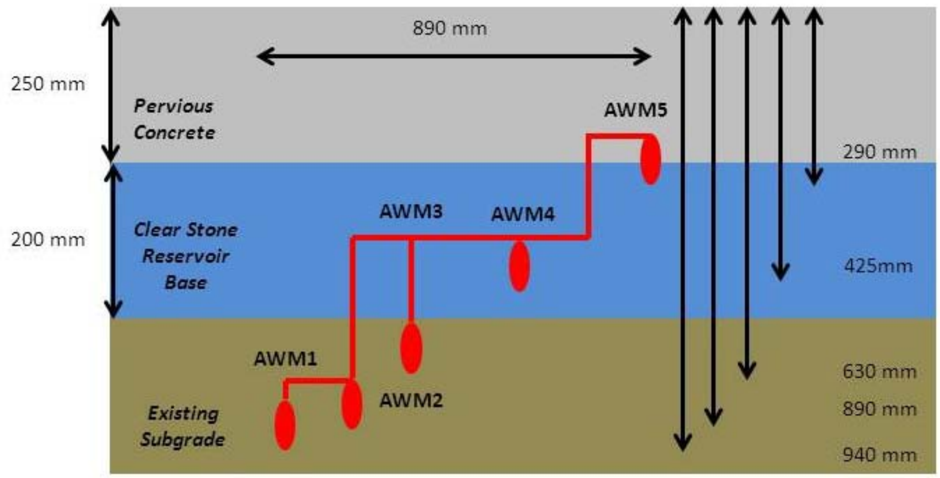


Figure 7-2: Moisture Gauge Sensor Tree A at Site 3A

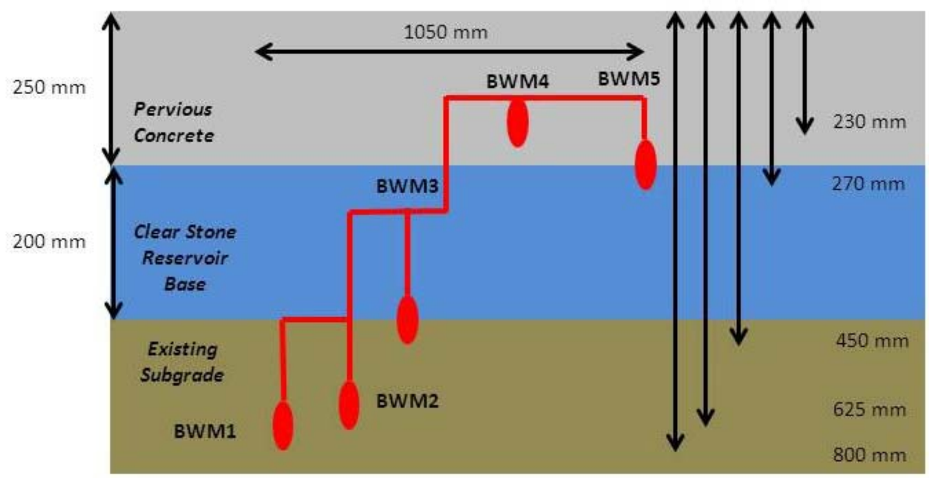


Figure 7-3: Moisture Gauge Sensor Tree B at Site 3A

As Figure 7-2 to Figure 7-5 demonstrate, the sensor trees were constructed with the moisture gauges being located at various depths and horizontal locations throughout the pavement structure. Figure 7-6 summarizes the sensor trees at Site 3.

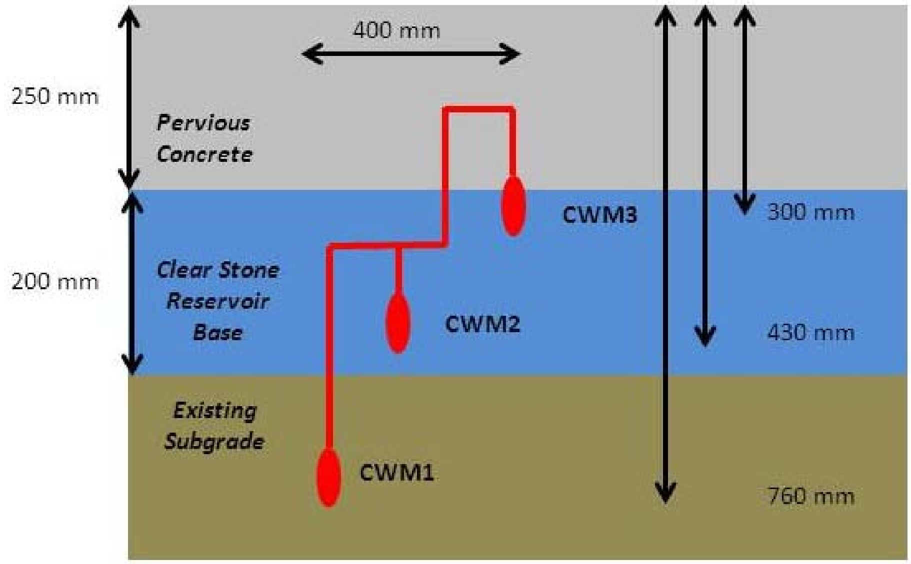


Figure 7-4: Moisture Gauge Sensor Tree C at Site 3B

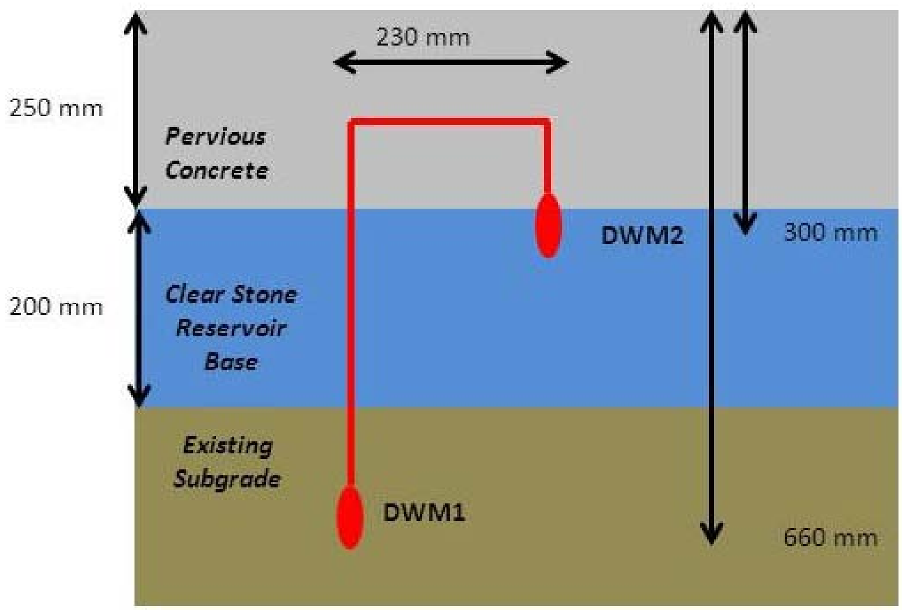


Figure 7-5: Moisture Gauge Sensor Tree D at Site 3B

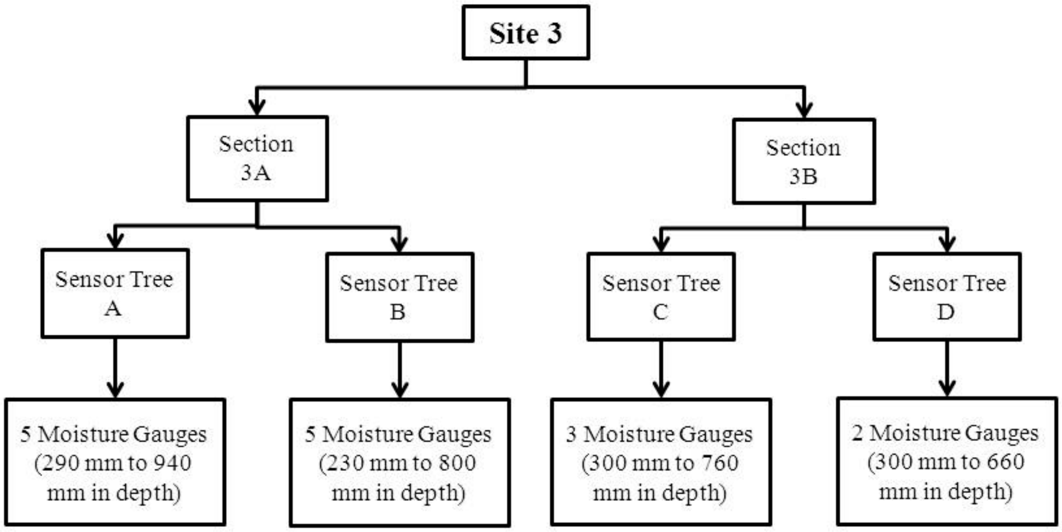


Figure 7-6: Sensor Trees at Site 3

7.3.1 Instrumentation Functionality Verification

The moisture gauge data was matched to the corresponding rain event dates. Table 7-1 shows an example of the results that were noticed in many of the rain events from Site 3. The trend that was identified in all of these results is a lack of response from the moisture gauges in Sensor Tree A and Sensor Tree B to rain events. The moisture gauges appear to not be responding to rain events as the minimum values for all gauges is on the same day. In the previous section, the method for calculating the SWP from the measured data was described and it was noted that the temperature is used in the calculation. The same temperature reading is used for the moisture gauges in the same portion of the site, 3A or 3B. The temperature changes the SWP value of a moisture gauge. If no changes occur in the material surrounding various moisture gauges, then the readings of the gauges would change with the change in temperature. If the instrumentation was not recording measurements at all, then an error would be recorded or a consistent value. The data recorded during these rain events was not a constant value or an error notification. However, the values were within a small range.

In Table 7-1 AWM1, AWM2, AWM3, AWM4, BWM1, BWM2 and BWM3 all show the same trend and the minimum value on the same date. The other three moisture gauges in Sensor Tree A and Sensor Tree B have a different trend but again a consistent one. The moisture gauge readings continually increase. This suggests that the material around the gauges is becoming dryer. Moisture gauge BWM5 generally shows the trend of drying however does have a minimum reading at the same point as many of the other sensors.

The trend noticed in the majority of the moisture gauges in Sensor Tree A and Sensor Tree B indicated that the instrumentation may not be reading the surrounding conditions and only be changing with the temperature changes.

Sensor Tree C and Sensor Tree D had results during this and most rain events that suggested a reaction to the current event as the trends were not identical between all sensors. Moisture gauges CWM3 and DWM2, are both located closest to the surface of the pavement structure in the particular sensor trees. They are at approximately the same vertical location which is the interface between the pervious concrete and the clear stone base. In the Fall 2009 rain event shown in Table 7-1 the data shows similar responses for these two gauges. The moisture appears to reach the instrumentation on the same day that it occurs. This was identified because on a day of multiple millimeters of rainfall occurring, the SWP of these two gauges either dropped to a minimum value or was already decreasing and continued to decrease.

The other three moisture gauges at Sensor Tree C and Sensor Tree D also show results indicating that moisture is moving into and away from these areas. The results indicate that this is occurring as minimum values are present and do not occur on consistent dates for all gauges.

The results of most of the moisture gauges in Sensor Tree A and Sensor Tree B presented concern that the moisture gauges may not be functioning properly. The data was evaluated to verify whether the results generated from these gauges was truly a representation of the condition of the surrounding material. It is possible that the material in the instrumented area remains in a consistent condition over multiple days even if rain events do occur. This would occur if moisture was not moving vertically through the pavement structure but draining into the instrumented areas from adjacent material. The values in the data for these moisture gauges suggests that the pavement layers are moist most of the time as the values are low in comparison to those noted at other field sites. The geometric layout of the field site also suggests that the instrumented material is consistently moist as the location is at a low point of the site.

Table 7-1: Fall 2009 Site 3 Moisture Readings

Date	Rain Fall (mm)	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	CWM1	CWM2	CWM3	DWM1	DWM2
10/20/2009	0.00	5.31	3.01	4.18	1.77	0.78	4.64	5.79	2.77	0.18	0.93	5.59	11.81	2.85	6.61	3.00
10/21/2009	8.25	5.31	2.93	4.09	1.68	0.79	4.69	5.85	2.70	0.21	0.97	5.47	11.78	2.83	6.66	2.99
10/22/2009	0.25	5.22	2.65	4.02	1.65	0.82	4.60	5.74	2.76	0.24	0.99	5.59	11.72	2.87	6.55	3.02
10/23/2009	26.25	5.22	2.46	4.03	1.63	0.83	4.63	5.86	2.79	0.25	1.01	5.71	11.66	2.85	6.51	2.95
10/24/2009	0.25	5.18	2.42	4.01	1.65	0.85	4.60	5.81	2.85	0.28	1.04	5.79	11.66	2.83	6.37	2.97
10/25/2009	13.00	4.95	2.41	3.79	1.53	0.88	4.40	5.55	2.71	0.33	1.03	5.65	11.23	2.76	6.17	2.92
10/26/2009	37.25	5.15	2.63	4.04	1.66	0.93	4.77	6.09	2.94	0.35	1.11	6.17	11.16	2.72	6.29	2.81
10/27/2009	0.00	5.06	2.59	3.91	1.59	0.93	4.57	5.79	2.89	0.40	1.09	5.98	11.13	2.78	5.84	2.88
10/28/2009	1.75	5.01	2.56	3.88	1.58	0.96	4.48	5.68	2.87	0.44	1.10	5.83	11.18	2.80	6.02	2.86
10/29/2009	13.50	5.11	2.59	3.94	1.66	1.02	4.60	5.89	3.02	0.49	1.14	6.25	11.43	2.82	6.23	2.81
10/30/2009	16.00	5.43	2.83	4.21	1.86	1.08	4.88	6.13	3.21	0.59	1.20	6.47	11.61	2.80	6.45	2.80
10/31/2009	6.00	5.65	2.96	4.27	1.91	1.11	5.03	6.21	3.25	0.66	1.18	6.32	11.37	2.75	6.47	2.69
11/1/2009	0.00	5.42	2.85	4.17	1.81	1.19	4.78	5.97	3.14	0.74	1.15	6.17	11.37	2.74	6.34	2.71
11/2/2009	4.50	5.34	2.79	4.03	1.74	1.26	4.75	5.86	3.06	0.80	1.15	6.04	11.27	2.71	6.40	2.68
11/3/2009	1.50	5.40	2.84	4.09	1.80	1.36	4.84	5.98	3.20	0.83	1.19	6.13	11.41	2.71	6.50	2.70
11/4/2009	0.00	5.25	2.76	3.98	1.73	1.41	4.69	5.77	3.11	0.86	1.17	5.88	11.20	2.72	6.32	2.63

To verify that the gauges were measuring the surrounding material, two aspects of the data were analyzed. First the data from the winter months was examined to determine if there were reactions from the instrumentation to the surrounding material freezing. Secondly, the complete data set was analyzed to evaluate if the values were changing throughout the seasons and showing patterns of drainage that would be assumed to occur with changing seasons.

Figure 7-7 shows the air temperature and the response of the material instrumented by Sensor Tree B. Figure 7-8 shows the same data as Figure 7-7 however the primary Y axis is a smaller scale in order to see the data points of BWM1, BWM2, and BWM3 more clearly. Moisture gauges BWM4 and BWM5 show larger values during the cold temperatures as shown in Figure 7-7, which indicates freezing in the surrounding material. The largest response to temperature changes are shown in BWM4 and BWM5 which is anticipated, as these moisture gauges are closest to the surface.

In Figure 7-8 the moisture gauge BWM3 had increased values in the Winter of 2009 and Winter of 2011. In Winter 2011 the data from BWM3 suggests that the surrounding material is frozen for a long period of time. This observation in the data follows the observed air temperature results. The material around BWM3 was frozen in Winter 2009 based on the increased values in the data and in early January 2011 onward.

Figures for Sensor Tree A showing the same information as Figure 7-7 and Figure 7-8 are included in Appendix E. The same figures were also produced for Sensor Tree C and Sensor Tree D for comparison purposes and they are included in Appendix E as well.

Of the five moisture gauges on Sensor Tree A, only AWM5, the moisture gauge closest to the surface, responded to the freezing temperatures. The next moisture gauge, AWM4, is deeper than BWM3 and BWM4 so it is possible that the material at this depth did not freeze during the winter seasons.

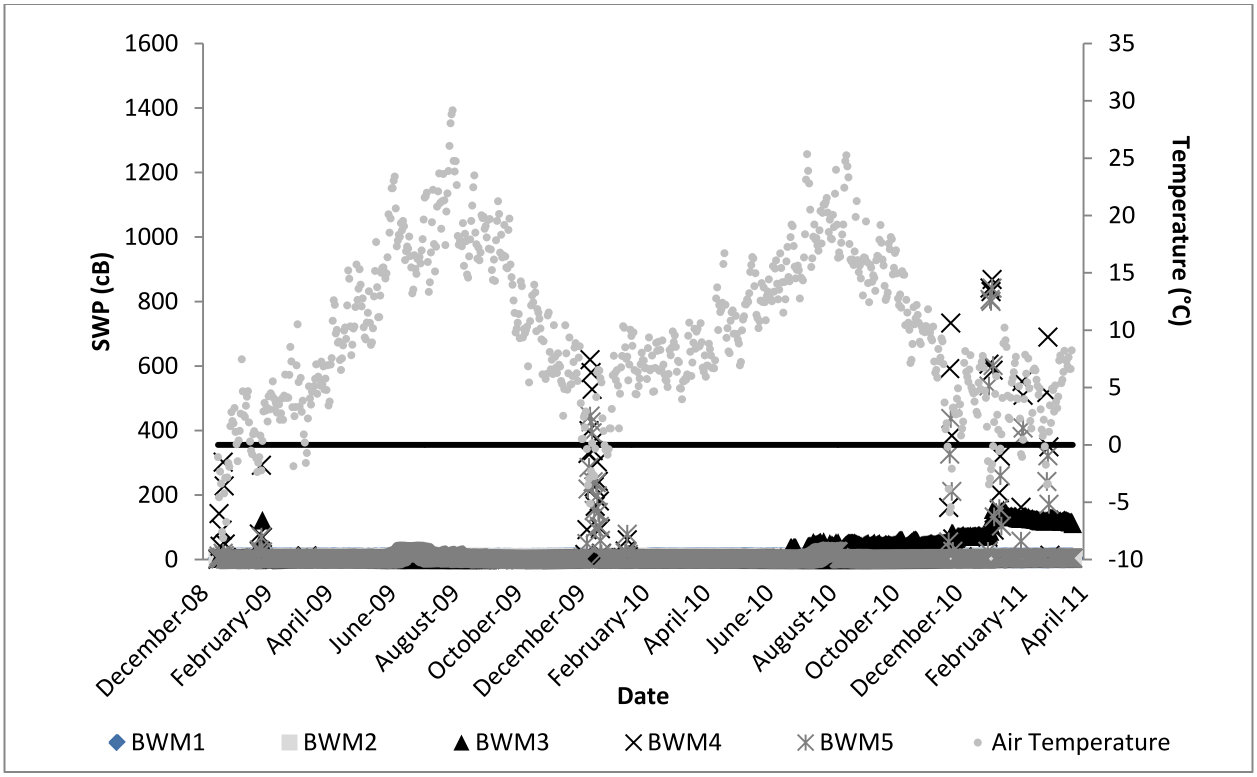


Figure 7-7: Sensor Tree B Response to Freezing Temperatures at Site 3A

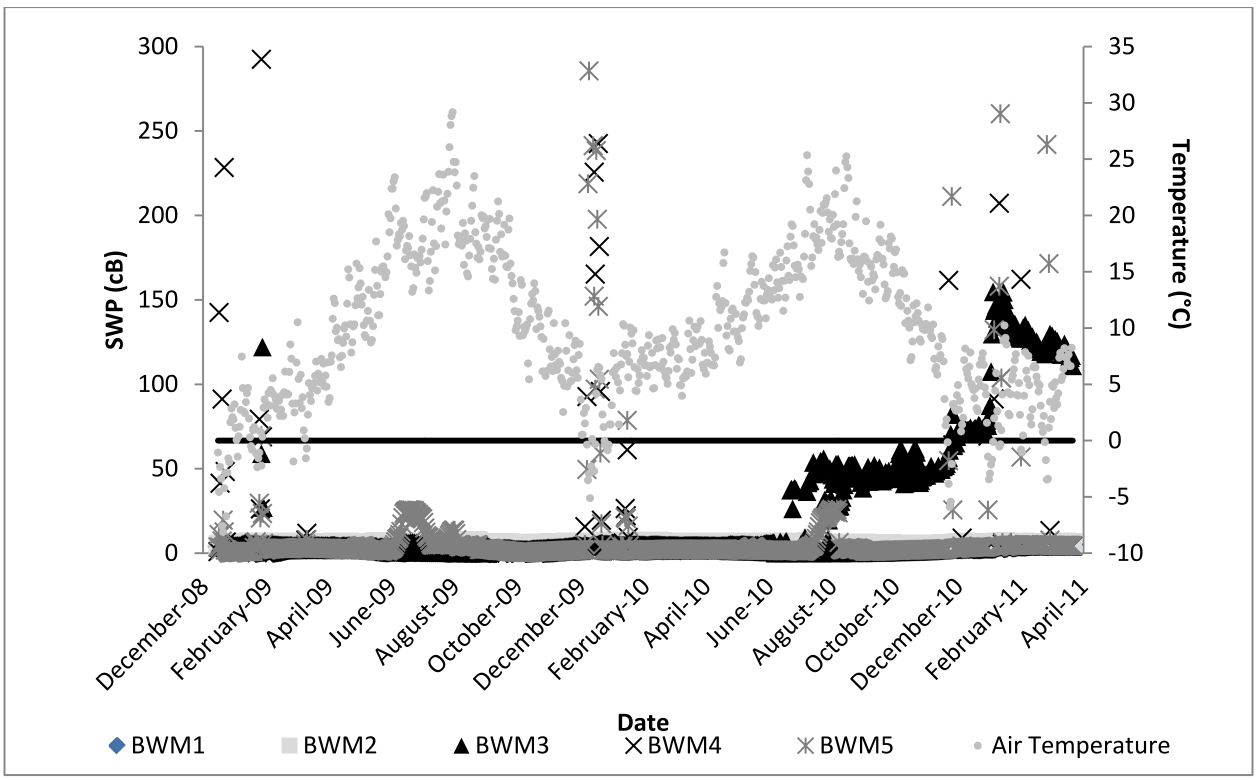


Figure 7-8: Sensor Tree B Response to Freezing Temperatures (Smaller Scale) at Site 3A

In the exit driveway CWM2 and CWM3 had increased values in the data during freezing temperatures. Moisture gauges CWM2 and CWM3 are at similar depths in the pavement structure as BWM3 and BWM5 respectively. On Sensor Tree D, DWM2 is at a comparable location to CWM3 and BWM5 and showed increased values during freezing temperatures.

The comparison of the moisture gauge responses to air temperatures below zero degree celcius suggests that many of them are measuring the condition of the surrounding material. This comparison was primarily applicable to the moisture gauges located closer to the pavement surface and down to depths that freeze during winter conditions. From this comparison, the pervious concrete pavement structure was regularly freezing to a depth of 300 mm and in some cases as deep as 430 mm. The material at depths below 430 mm from the surface was not found to freeze when the air temperature dropped below zero degrees celcius.

The second analysis that was carried out to verify that the moisture gauges were recording changes in the surrounding material was done by looking at the overall changes in moisture over a complete year, four seasons. It is anticipated that the moisture in the instrumented material would change with the seasons, even if water was not moving vertically through the pavement structure. The moisture gauge data for each sensor tree was plotted for the entire time period that data was collected to evaluate if gradual changes in overall moisture in surrounding material were noticeable. Figure 7-9 shows Sensor Tree A from Winter 2009 to Spring 2011 and Figure 7-10 shows Sensor Tree B for the same time period. The same figures for Sensor Tree C and Sensor Tree D are included in Appendix E. Not all of the data points for each moisture gauge are shown in Figure 7-9 and Figure 7-10 as the Y axis has been reduced to show the majority of the data points.

Figure 7-9 and Figure 7-10 show similar trends for instrumentation at comparable depths in the pavement structure. Both AWM5 and BWM5 visually show the clearest changes in SWP and these sensors are each located at the interface between the pervious concrete pavement and the clear stone. Moisture gauges CWM3 and DWM2 are also at this approximate location in the pavement structure. Both of these gauges showed a clear trend within a similar range of values (less than 20 cb). However, in the summer of 2010, the material surrounding these gauges became dryer and the readings were up to approximately 50 cb. In the Winter of 2011

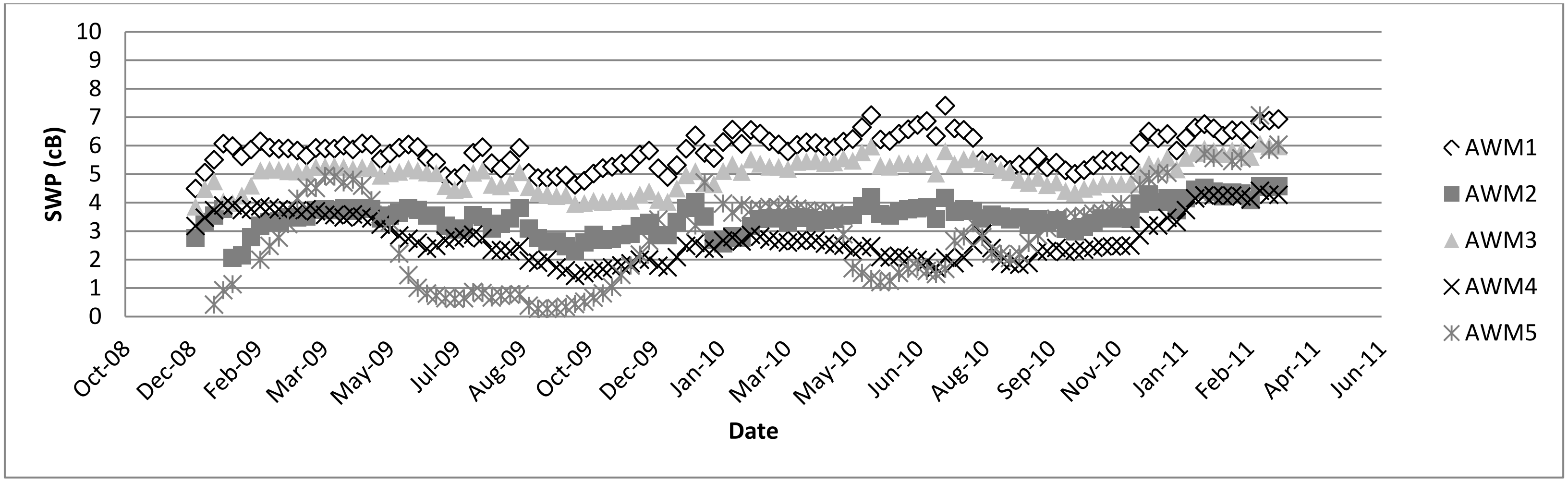


Figure 7-9: Sensor Tree A Winter 2009 to Spring 2011

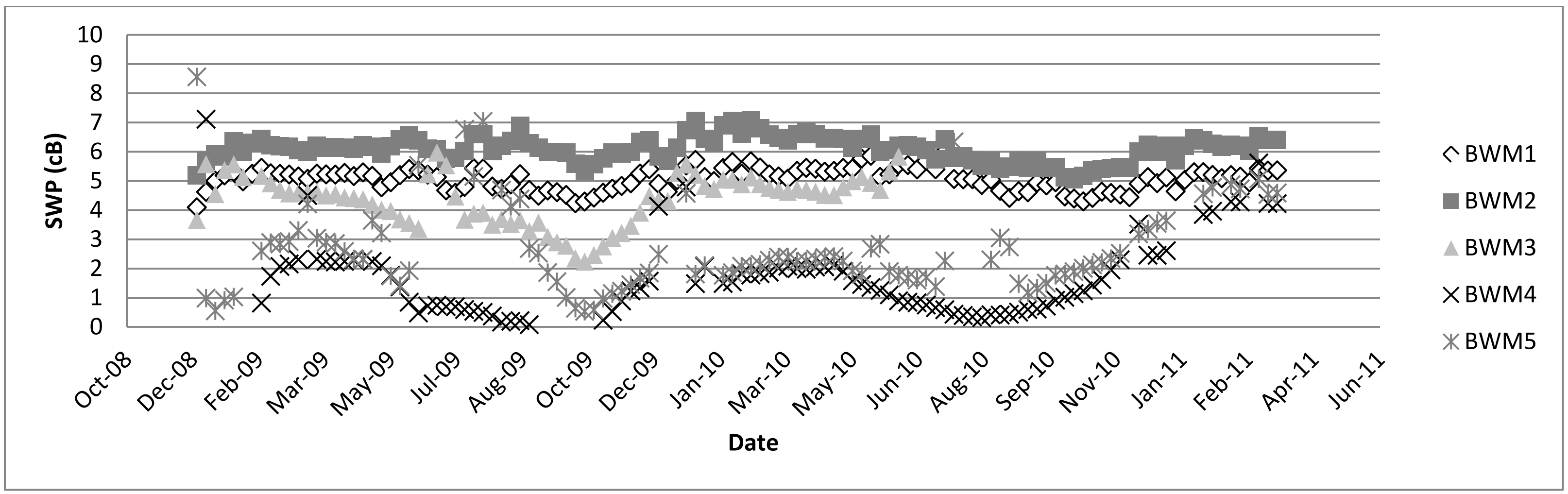


Figure 7-10: Sensor Tree B Winter 2009 to Spring 2011

(December 2010 to February 2011) the sensor at this location, interface of pervious concrete and clear stone base layer showed larger spikes and more spikes due to freezing conditions than in the other winter seasons. All of the observations noted above verified that the moisture gauges at the pervious concrete and clear stone interface are reacting to the surrounding conditions and are functional.

Moisture gauge BWM4 is the closest of all the gauges to the pavement surface, being located in the pervious concrete layer. It showed similar behaviour to BWM5 which is anticipated as they are close in vertical and horizontal location. As noted earlier, there are more voids in the pervious concrete at the bottom of the pavement layer as compared to the top. Therefore the bottom of the pervious layer is anticipated to be creating similar drainage characteristics to the clear stone base layer below.

The moisture gauges located deeper in the pavement structure showed less fluctuation with seasons. However, changes over multiple months are visible for all four sensor trees. In Sensor Tree A, AWM1, AWM2, AWM3 and AWM4 all showed similar trends in terms of increases and decreases in SWP. In comparison to AWM5, the changes in the other four sensors were less and in some cases delayed. Sensor AWM5 started increasing in SWP continually from August 2010. However, the other four sensors did not start increasing until September 2010. This trend matches the locations of the sensors, as AWM5 is closest to the surface and water would drain away from it initially and into the clear stone base layer.

Moisture gauges BWM1 and BWM2 have similar results, less fluctuation than BWM4 and BWM5 but some changes over time. These deep sensors showed the same trend as the deeper sensors in Sensor Tree A. The instrumentation closer to the surface showed an increase in SWP from August 2010 onwards. However, the deeper instrumentation did not start to show this trend until October 2010.

Sensor BWM3 is located at the interface between the clear stone base and the existing subgrade. There are no other sensors located at this location in the other sensor trees. The SWP results from this sensor show bigger fluctuations than those from deeper instrumentation. The fluctuation range of SWP for BWM3 is similar to the two sensors closer to the surface. In June 2010 however, the readings from BWM3 increase more than any of the others had over the testing period to date, not considering freezing temperatures. These readings are not within the scale shown in Figure 7-10 and increase to values of 50 cb during the fall season and then 140 cb during the winter season. Peak readings in January 2011 include 140 cb and then decreased to 120 cb by March 2011. These results from BWM3 led to three observations:

- During June 2010 and dates following this, no moisture was moving into the area instrumented by BWM3;
- The location of BWM3 was not freezing in the winter season; and
- Moisture gauge BWM3 is located at the interface of the clear stone base layer and the existing subgrade. Although it is anticipated that BWM3 is monitoring more of the activity in the clear stone layer it may actually be taking readings from the subgrade material. It is possible that the subgrade material and clear stone base material have mixed somewhat and the material that was previously subgrade is now a coarser material. If the consistency of the material has become more coarse this may lead to higher moisture gauge readings as were observed in the Fall of 2010 onwards.

The SWP was decreasing from January 2011 onwards at BWM3, indicating moisture moving into the instrumented area. It is not clear as to how much the SWP would decrease over the longer term.

Sensor CWM2 showed changes in SWP over time. This sensor is located in the clear stone base layer in the 3B area and showed a dryer period during both Summer 2009 and 2010. This trend was not shown in the other sensor trees and suggests that water is draining through the clear stone base in this area. Sensor CWM1 is in the existing subgrade and had consistent readings up to October 2009. During October 2009, the SWP began to increase in the material surrounding this sensor. Until March 2010 the SWP gradually increased. Following March 2010 and until October 2010, the SWP of the material increased at a higher rate, to a peak of 180 cb. During this time the SWP was increasing on average. However, the increase was not continual as there were spikes and drops between readings. Following October 2010 and to March 2011, the SWP fluctuated between a range of 190 cb and 120 cb. The results of Sensor Tree C indicate that water is moving through the clear stone base layer. However, since October 2009 there has been limited moisture moving into the existing subgrade.

Sensor DWM2 in Section 3B showed similar results to CWM2, although it is located at a different depth. The material surrounding DWM2 became dryer in the summer season of 2009 and 2010. The material surrounding DWM1, existing subgrade, showed changes in SWP between seasons and an overall increase in SWP from December 2008 to March 2011.

The visual review of the results shown in the Figure 7-9, Figure 7-10 and Appendix E, verify that all of the moisture gauges have been functioning throughout the data collection period from December 2008 to March 2011. The results are therefore assumed to be representative of the surrounding material in each instrumented location.

7.3.2 Analysis

The response of each moisture gauge to a large rain event or snow melt was evaluated. The data for each moisture gauge during a particular rain event is provided in Appendix E. The data for all the moisture gauges was reduced further and is presented in Table 7-2. Information about the rain events is included in Table 7-3. As noted earlier, rain over five days was initially considered. In some cases though, substantial quantities of rain would occur over more than five days. These events were included when deemed appropriate. The column entitled "entering area" refers to the days in which a negative slope in SWP was shown, as was described in Figure 7-1. Therefore more moisture was draining into the instrumented area than out of it. The column entitled "draining area" refers to the SWP results showing a positive slope, as shown in Figure 7-1. The day that the rain event started is numbered 0. The values in the "entering area" and "draining area" represent the day(s) after the initiation of the rain event that the behaviour was noticed in the SWP results.

In the Summer of 2009, a rain event of 38 mm occurred on days 0 and 1. In Section 3A moisture was present on the same day as the rain event occurred in all instrumentation on Sensor Tree A and Sensor Tree B. At most depths of instrumentation the presence of moisture entering and draining from the adjacent material alternated each day. The alternating of moisture draining into and away from the instrumented areas does not describe the anticipated behaviour of pervious concrete pavement. These results could be describing small amounts of water slowly draining into and away from the instrumented material. The small amounts are not anticipated to be moving vertically through the pavement structure as they are arriving at each depth of the pavement structure at approximately the same time. Moisture is observed to be draining into the instrumented areas until eight days after the rain event.

Table 7-2: Moisture Movement Site 3

Moisture Gauge Location	Date	Time from Rain Event to Instrumented Area (Days): Entering Area	Time from Rain Event to Instrumented Area (Days): Draining Area
Pervious Concrete Layer (230 mm) BWM4	Winter 2009	3 9	4-8 10-11
	Spring 2009	2-3 5 7-10	4 6 11-12
	Summer 2009	1 3 5 7	2 4 6 8
	Fall 2009	1-2 5 15 19 21 26-28	3-4 6-14 16-18 20 22-25
	Fall 2009	Always Dry	
	Winter 2010	2 4 6 8-10 12-13	3 5 7 11
	Spring 2010	1-7 9	8 10
	Summer 2010	0	1-7
	Fall 2010	0 4 10-11	1-3 5-9
	Winter 2011	0-2	3-6

Moisture Gauge Location	Moisture Gauges	Entering: AWM5	Entering: BWM5	Entering: CWM3	Entering: DWM2	Draining: AWM5	Draining: BWM5	Draining: CWM3	Draining: DWM2
Pervious Concrete and Clear Stone Interface (270 mm – 300 mm) AWM5 BWM5 CWM3 DWM2	Winter 2009	3 7-8	3 7-9	2 4 6-10	6 8	4-6 9-11	4-6 10-11	3 5 11	7 9-11
	Spring 2009	3 5 8-10 12	3 5 8-12	2-3 8 10	2-3 5 7-9	4 6-7 11	4 6-7	4-7 9 11-12	4 6 10-12
	Summer 2009	1 3 7	1 3-5 7	0-8	0-3 5-8	2 4-6 8	2 6 8		4
	Fall 2009	1 15 19 21 27-28	15-16 21 26-28	1-2 4 6-7 9-11 13 15-17 19-20 22 26-27	1-2 4-11 13-21 26-27	2-14 16-18 20 22-26	17-20 22-25	3 5 8 12 14 18 21 23-25 28	3 12 22-25 28
	Fall 2009	6	4 6 10-12 14	2-5 9-13	2 4-5 7-10 12 14	7-14	5 7-9 13	6-8 14	3 6 11 13
	Winter 2010	2 4 8-12	2 4 6 8-10 12-13	2 5-6 8 10-11	2 4-6 8 11-12	3 5-7 13	3 5 7 11	3-4 7 9 12-13	3 7 9-10 13
	Spring 2010	2 5 7 9	1-3 5 7 9	2 5-10	1-3 5-9	3-4 6 8 10	4 6 8 10	3-4	4 10
	Summer 2010	0-1	0-5	1 3 6-7	0-7	2-7	6-7	2 4-5	
	Fall 2010	0-2 4-5 7 11	2 5 7 11	4-11	0-11	3 6 8-10	3-4 6 8-10		
	Winter 2011	0-4	0-1	0-4	0-1 3-4	5-6	2-6	5-6	2 5-6

Moisture Gauge Location	Moisture Gauge	Entering: AWM4	Entering: CWM2	Draining: AWM4	Draining: CWM2
Clear Stone (425 mm) AWM4 CWM2	Winter 2009	3-4 7-11	2-3 5-6 10	5-6	4 7-9 11
	Spring 2009	1 3 5-6 11-12	1 3 5 8-11	2 4 7-10	2 4 6-7 12
	Summer 2009	1 3-5 7	0-7	2 6 8	8
	Fall 2009	2-3 6 10 12-13 16-17 20 22-23 25	2 6 8 10-11 15-17 20 22 27-28	4-5 7-9 11 14-15 18-19 21 24 26-28	3-5 7 9 12-14 18-19 21 23-26
	Fall 2009	1-2 4 6-7 11-12 14	1-2 4-6 10 12 14	3 5 8-10 13	3 7-9 11 13
	Winter 2010	1 4-6 11	2-5 8 11 13	2-3 7-10 12-13	6-7 9-10 12
	Spring 2010	2-3 5 7 9	2-3 5-7 9	4 6 8 10	4 8 10
	Summer 2010	0-1 3-5	0-6	2 6-7	7
	Fall 2010	2 5 7-8 11	1-2 4-7 11	3-4 6 9-10	3 8-10
	Winter 2011	3-4	0-5	5-6	6

Moisture Gauge Location	Date	Entering Area	Draining Area
Clear Stone and Subgrade Interface (450 mm) BWM3	Winter 2009	6 8-11	7
	Spring 2009	1 3 5-6 11-12	2 4 7-10
	Summer 2009	1 3 5 7	2 4 68
	Fall 2009	2-3 6 12 15-17 20 22-26	4-5 7-11 13-14 18-19 21 27-28
	Fall 2009	4 6-7 11-12 14	5 8-10 13
	Winter 2010	1-6	7-13
	Spring 2010	5 7 9	6 8 10
	Summer 2010	1	2-7
	Fall 2010	0-1	2-11
	Winter 2011	0 3-4	1-2 5-6

Moisture Gauge Location	Moisture Gauge	Entering: AWM3	Entering: BWM2	Entering: DWM1	Draining: AWM3	Draining: BWM2	Draining: DWM1
Subgrade (630 mm - 660 mm) AWM3 BWM2 DWM1	Winter 2009	2-3 8-9 11	3 8-11	1-2 8-11	4-7 10	4-7	3-7
	Spring 2009	1 3 5-6 11-12	1 3 -56 11-12	1 3 5-6 11-12	2 4 7-10	2 4 7-10	2 4 7-10
	Summer 2009	1-5 7	1 3 5 7	0-1 3-4	6 8	2 4 6 8	2 5-8
	Fall 2009	3-4 6-7 12-13 16-17 20 22-25	2-3 6-7 12-13 16-17 20 22-23 25	2-3 6-7 10 12 16-17 20 22-23	5 8-11 14-15 18-19 21 26-28	4-5 8-11 14-15 18-19 21 24 26-28	4-5 8-9 11 13-15 18-19 21 24-28
	Fall 2009	1 3-4 6-7 11-12 14	1 3-5 6-7 11-12 14	1-4 6 11 14	2 5 8-10 13	2 5 8-10 13	5 7-10 12-13
	Winter 2010	1-3 5-6 10	1 3-6 12	5-8 11-13	4 7-9 11-13	2 7-12	9-10
	Spring 2010	2-3 7 9	2-3 7 9	1-3 5 7-9	4-6 8 10	4-6 8 10	4 6 10
	Summer 2010	0-1	0-1	0-1	2-7	2-7	2-7
	Fall 2010	2 5 7 11	2 4-5 7 11	0-2 4 7	3-4 6 8-10	3 6 8-10	3 5-6 8-11
	Winter 2011	3-5	3-5	1 3-5	6	6	2 6

Moisture Gauge Location	Moisture Gauge	Entering: CWM1	Entering: BWM1	Draining: CWM1	Draining: BWM1
Subgrade (760 mm – 800 mm) CWM1 BWM1	Winter 2009	3 8-10	3-4 8-11	4-7 11	7-May
	Spring 2009	1 3 5-6 11-12	1 3 5-6 12	2 4 7-10	2 4 7-11
	Summer 2009	0 3-4	1 35 7	1-2 5-8	2 6 8
	Fall 2009	2 5-7 12 15-18 20 22-23 28	2-3 6-7 12-13 16-18 20 22-23 25	3-4 8-11 13-14 19 21 24-27	4-5 8-11 14-15 19 21 24 26-28
	Fall 2009	4 6-7 10-12 14	1 3-4 6-7 11-12 14	5 8-9 13	2 5 8-10 13
	Winter 2010	3 5 8 11-13	1 4-6	4 6-7 9-10	2-3 7-13
	Spring 2010	4-5 9-10	2 5 7 9	6-8	3-4 6 8 10
	Summer 2010	0 3-4	0-1	1-2 5-7	2-7

	Fall 2010	2 4 7-8 10-11	2 4-9 11	3 5-6 9	3 10
	Winter 2011	3-4 6	3-5	5	6
Subgrade (890 mm) AWM2	Winter 2009	3-4 6-8 10-11		5 9	
	Spring 2009	1 3 5-6 12		2 4 7-11	
	Summer 2009	1-5 7		6 8	
	Fall 2009	3 6-7 12-13 16-17 20 22-24		4-5 8-11 14-15 18-19 21 25-28	
	Fall 2009	1-4 6-7 11-12 14		5 8-10 13	
	Winter 2010	1 3-6 8-9		2 7 10-13	
	Spring 2010	2-3 7		4-6 8-10	
	Summer 2010	0-1		2-7	
	Fall 2010	2 5-7 11		3-4 8-10	
	Winter 2011	3-5		6	
Subgrade (940 mm) AWM1	Winter 2009	3-4 8-11		5-7	
	Spring 2009	1 3 5-6 12		2 4 7-11	
	Summer 2009	1 3-5 7		2 6 8	
	Fall 2009	3-4 6-7 12-13 16-18 20 22-23 25		5 8-11 14-15 19 21 24 26-28	
	Fall 2009	1-4 6-7 11-12 14		5 8-10 13	
	Winter 2010	1 4 6		2-3 5 7-13	
	Spring 2010	2-3 7		4-6 8-10	
	Summer 2010	0-1		2-7	
	Fall 2010	2 4-711		3 8-10	
	Winter 2011	3-4		6	

Table 7-3: Site 3 Rain Events

Season	**Winter**			**Spring**			**Summer**			**Fall**			**Fall**		
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
	1/6/2009	0	65	3/27/2009	0	9	8/10/2009	0	24	11/15/2009	0	25	10/21/2009	0	8
	1/7/2009	1	49	3/28/2009	1	8	8/11/2009	1	14	11/16/2009	1	31	10/22/2009	1	0
	1/8/2009	2	22	3/29/2009	2	2	*Total Rainfall (mm)*		38	11/17/2009	2	9	10/23/2009	2	26
	1/9/2009	3	1	3/30/2009	3	7				11/18/2009	3	16	10/24/2009	3	0
	1/10/2009	4	32	3/31/2009	4	18				11/19/2009	4	23	10/25/2009	4	13
	1/11/2009	5	6	4/1/2009	5	13				11/20/2009	5	13	10/26/2009	5	37
	1/12/2009	6	8	4/2/2009	6	17				11/21/2009	6	13	10/27/2009	6	0
	Total Rainfall (mm)		183	*Total Rainfall (mm)*		74				11/22/2009	7	25	10/28/2009	7	2
2009										11/23/2009	8	6	10/29/2009	8	14
										11/24/2009	9	3	10/30/2009	9	16
										11/25/2009	10	34	10/31/2009	10	6
										11/26/2009	11	23	11/1/2009	11	0
										11/27/2009	12	0	11/2/2009	12	5
										11/28/2009	13	23	*Total Rainfall (mm)*		127
										11/29/2009	14	12			
										11/30/2009	15	5			
										Total Rainfall (mm)		261			
	Date	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**			
	1/8/2010	0	23	5/26/2010	0	5	8/7/2010	0	11	10/9/2010	0	25			
	1/9/2010	1	4	5/27/2010	1	7	8/8/2010	1	5	10/10/2010	1	16			
	1/10/2010	2	0	5/28/2010	2	18	*Total Rainfall (mm)*		16	*Total Rainfall (mm)*		41			
	1/11/2010	3	44	5/29/2010	3	19									
2010	1/12/2010	4	23	5/30/2010	4	0									
	1/13/2010	5	5	5/31/2010	5	22									
	1/14/2010	6	21	6/1/2010	6	2									
	1/15/2010	7	30	6/2/2010	7	29									
	1/16/2010	8	0	*Total Rainfall (mm)*		102									
	1/17/2010	9	7												
	Total Rainfall (mm)		157												
	Date	**Day**	**Rainfall (mm)**												
	1/5/2011	0	34												
2011	1/6/2011	1	42												
	1/7/2011	2	27												
	1/8/2011	3	6												
	Total Rainfall (mm)		109												

Sensor Tree C and Sensor Tree D in Section 3B of Site 3 showed different results than the sensor trees in section 3A although not indicating direct drainage of moisture through the pervious concrete pavement structure. The two moisture gauges of Sensor Tree C that are closer to the surface, CWM3 and CWM2 are continually increasing in moisture and the deepest moisture gauge, CWM1 is increasing in moisture on days 0 and 3 to 4. The two moisture gauges on Sensor Tree D are showing changes in moisture in the surrounding material. However, the changes do not reflect the assumed behaviour of pervious concrete pavement. The higher moisture gauge DWM2, is becoming wetter on days 0 to 3 and 5 to 8. The deeper moisture gauge, DWM1, is increasing in moisture on days 0 to 1 and 3 to 4. These dates do not align to suggest that moisture is moving through the pavement structure from one moisture gauge to another.

Changes in moisture content of the material surrounding Sensor Trees A, B, C and D in the Summer of 2009 do not show results to suggest that moisture is draining vertically through the pervious concrete pavement structure.

In the Fall of 2010 more movement of moisture through the pavement structure can be identified than in the Summer of 2009 results. In the Fall of 2010 a rain event on days 0 and 1 occurred included 41 mm of rain. The two moisture gauges closest to the surface in Sensor Tree B, BWM5 and BWM4 and the closest to the surface in Sensor Tree A, AWM5 all showed results identifying the presence of moisture on days 4 and/or 5. In the pervious concrete layer, BWM4, no further moisture was identified to be draining into the area until days 10 and 11. At the interface of the pervious concrete and the clear stone base moisture was observed in the data on days 7 and 11. In moisture gauge AWM4, also in the clear stone base moisture was present on days 7 to 8 and 11. Moisture gauge BWM3 at the interface between the clear stone base and subgrade did not show any presence of moisture related to the rain event, moisture was only identified as draining into the area on days 0 and 1. Moisture gauges AWM3 and BWM2 in the subgrade showed the presence of moisture on days 2, 5, 7 and 11. If any of the moisture is assumed to be draining vertically between the moisture gauges then the days of moisture draining into the instrumented areas that are plausible are 7 and 11. Finally the deepest moisture gauge, BWM1 shows moisture being present in the surrounding material on days 7 and 9. Either of these could possibly be a result of moisture draining vertically from the pavement structure layers above.

In the Fall of 2010 rain event moisture gauges CWM3 and DWM2 both show moisture moving into the instrumented material for multiple days. These moisture gauges are located at the interface of the pervious concrete and the clear stone. Moisture is shown to be continuing to move into the area instrumented by CWM3 from day 4 to 11 and the area instrumented by DWM2 from day 0 to day 11. Moisture gauge CWM2 located in the clear stone shows moisture moving into the surrounding material from days 4 to 7 and on day 11. This moisture could be draining vertically from CWM3. The subgrade material instrumented with DWM1 shows moisture being present on days 0 to 2, 4 and 7. Some of this moisture could be draining from the area of DWM2. The days when moisture is present at DWM1 does not cover all days of moisture at DWM2 though. Finally the deepest subgrade material, surrounding CWM1 identifies moisture being present on days 4 to 5 and 9 to 10. The moisture present on the latter days could be draining vertically through the pavement structure.

Analysis of the moisture gauges at Site 3 verifies the functionality of them. The discussion above indicates that it is not possible to identify continual clear vertical drainage through the pavement structure during or following a rain event. It is anticipated that continual vertical drainage from the surface to the subgrade is not occurring due to the low permeability rate of the surface. There is no means for moisture to enter into the pavement structure from the surface. Any limited permeability that is possible on the surface may not be occurring if moisture is evaporating prior to drain through the pavement. If moisture is moving into the pervious concrete slowly and remaining close to the surface for a length of time then it is anticipated that some evaporation could occur. However, the moisture gauge results indicate that moisture is moving through various portions of the pavement structure. Therefore it is anticipated that moisture is draining into the clear stone base and subgrade layers of the pavement structure from adjacent sources. At Site 3 the pervious concrete pavement is not performing as intended and is not demonstrating the desired drainage abilities of pervious concrete pavement.

7.4 Site 4

At Site 4 twelve moisture gauges were used on three sensor trees. The sensor trees were constructed and the dimensions of the three trees are consistent. The sensor trees were then placed in the pavement structure when the subgrade layer was exposed. One sensor tree is located in each of the three sections of the site, 4A, 4B and 4C. Figure 7-11 shows the sensor trees that were installed at the field site.

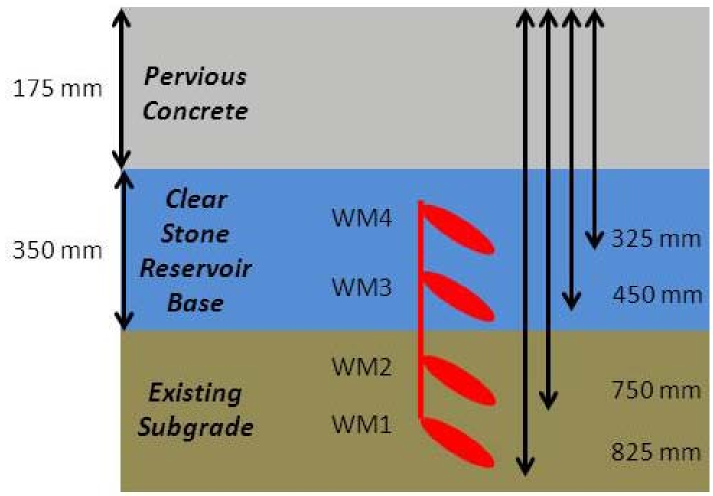


Figure 7-11: Sensor Tree at Site 4

As Figure 7-11 shows, there are four moisture gauges in each sensor tree at the field site. In Figure 7-11 the moisture gauges labeled as they are in each of the three areas. The letter describing the area of the site (A, B or C) is added to the beginning of the moisture gauge label, such as AWM4 in area 4A. The reference name for each moisture gauge refers to the mix it is in, A, B or C and then the depth of the moisture gauge, with moisture gauge "1" being the deepest and "4" being closest to the surface. The sensor trees are at the same depths within each mix and are located in the same area of each mix. The site slopes and the sensor trees are all at the same portion of the slope, therefore anticipating that horizontal movement due to the slope would be consistent. Figure 7-12 shows a summary of the sensor trees at Site 4.

7.4.1 Instrumentation Functionality Verification

One of the initial large rain events at Site 4 after construction was plotted for each sensor tree. The plots demonstrate the movement of water through the pavement structure. Figure 7-13 to Figure 7-15 show the response of the instrumentation in each of the three mixes during a rain event of 41 mm over five days in Fall 2008. The plot shows the instrumentation readings and rain from November 6, 2008 to November 20, 2008. The site was constructed October 15, 2008 and opened to traffic October 27, 2008.

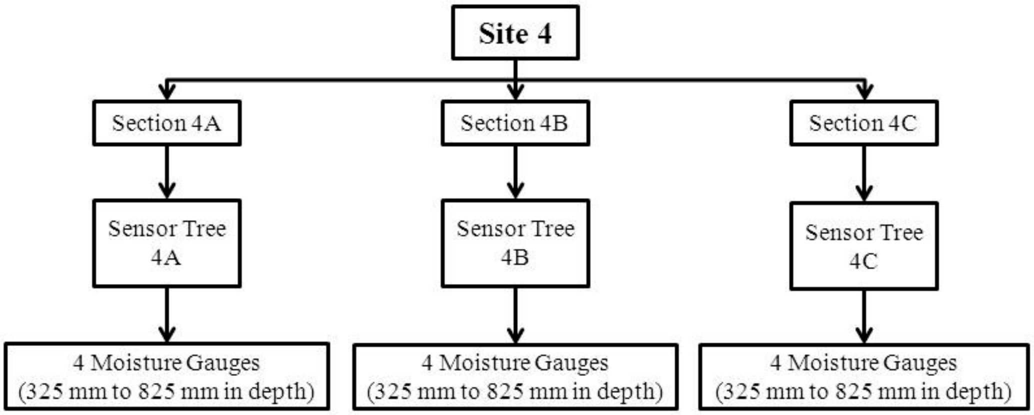


Figure 7-12: Sensor Trees at Site 4

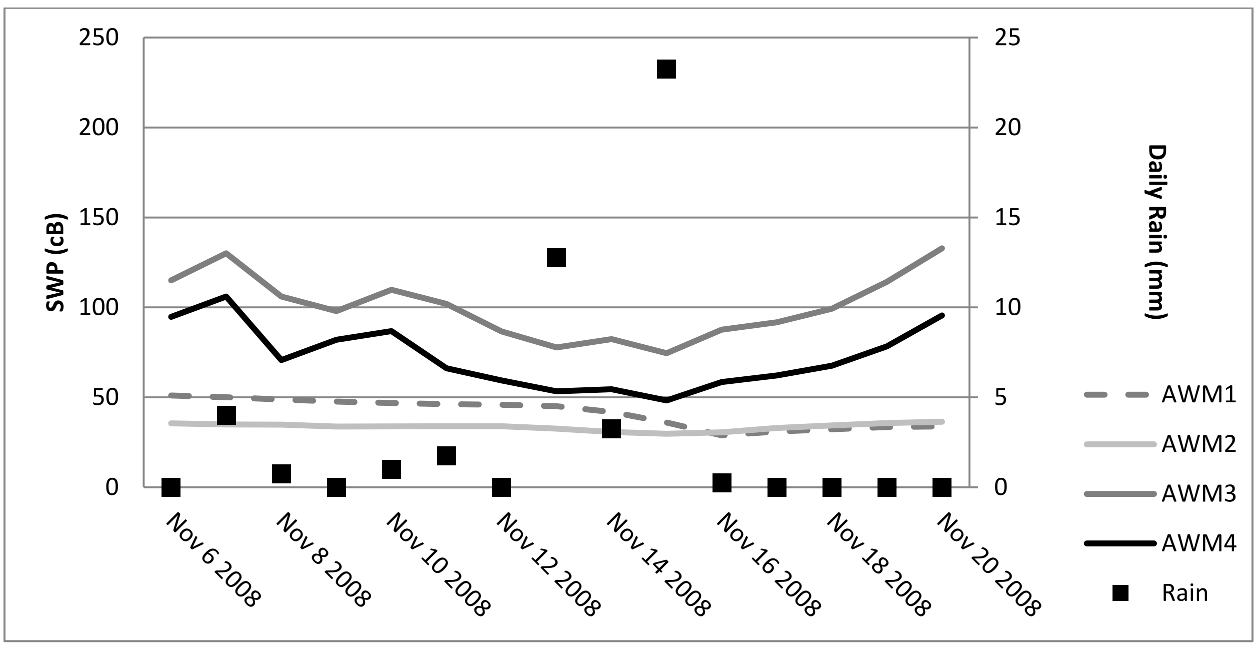


Figure 7-13: Rain Event Site 4A Fall 2008

Sections 4A and 4B show similar behavior in Figure 7-13 and Figure 7-14. The upper two moisture gauges, WM3 and WM4, are dryer and showing more fluctuation while the two moisture gauges in the subgrade, WM1 and WM2, are more consistent and wetter. In 4A the two largest rainfalls, 13 mm on November 13, 2008 and 23 mm on November 15, 2008 are identifiable by moisture gauges AWM3 and AWM4. On the same day that the rain events occurrs, moisture was present at the instrumented areas and it was drained by the next day, November 14, 2008 and November 16, 2008, where positive slopes are notable in both cases. The deepest moisture gauge, AWM1, shows a continual, small negative slope from November 6, 2008 to November 17, 2008. Following November 17, 2008 a slight positive slope is present, however. The higher moisture gauge in the subgrade, AWM2, shows less fluctuation at this scale. However, slight changes in slope are visible with the rain events.

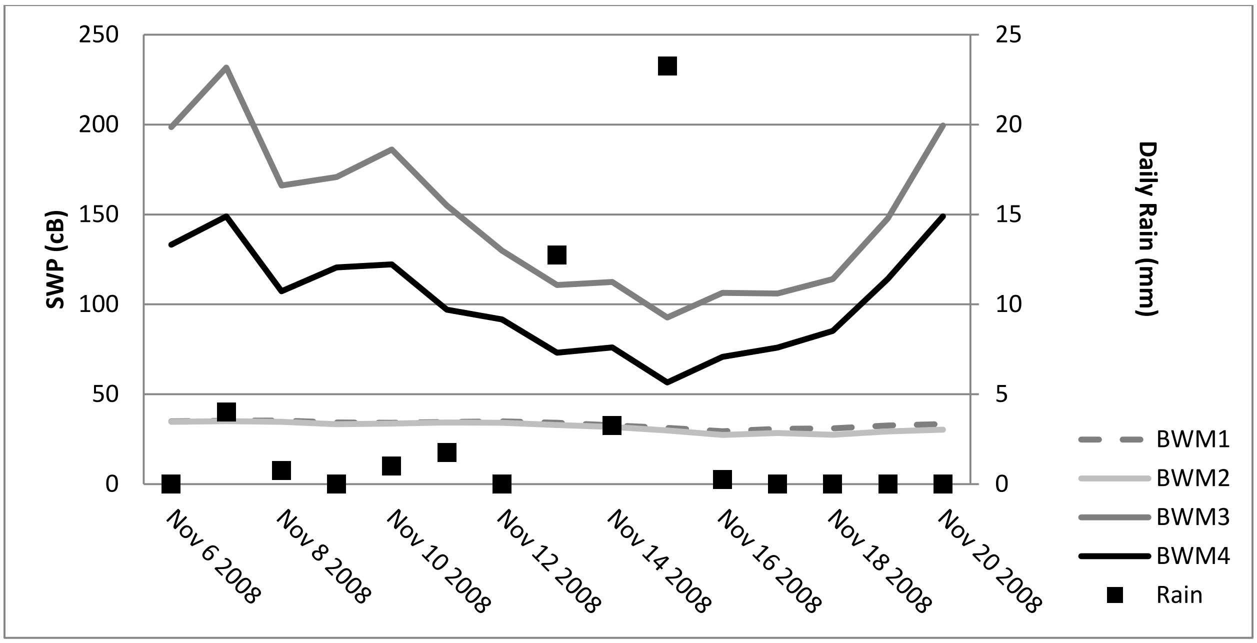


Figure 7-14: Rain Event Site 4B Fall 2008

In comparison to 4A, the two highest moisture gauges in 4B, BWM4 and BWM3, show a larger reaction to the rain events. Therefore the water is anticipated to be moving faster through the area and also is draining through the pervious concrete layer faster as well. The same trend is seen in BWM3 and BWM4 as AWM3 and AWM4. The changes in SWP are larger in the case of both moisture gauges in section 4B. The deeper moisture gauges in section 4B, BWM1 and BWM2, show more consistent readings similar to those in AWM1 and AWM2.

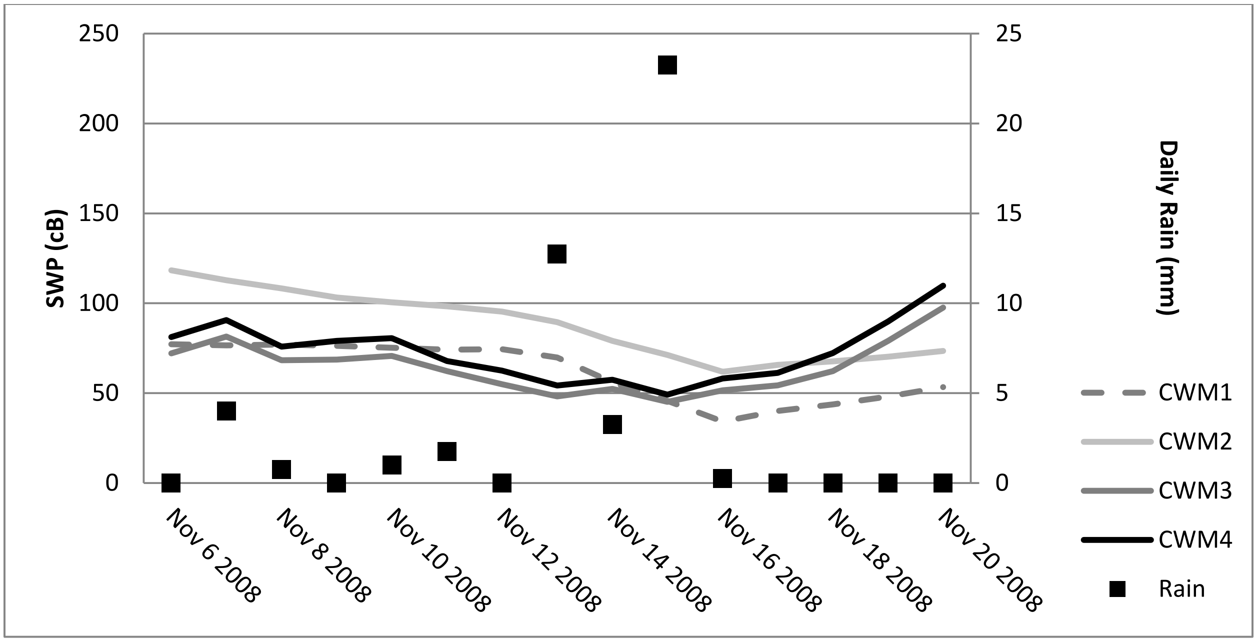


Figure 7-15: Rain Event Site 4C Fall 2008

All four moisture gauges in Sensor Tree C showed moisture from the rain event entering the instrumented areas and draining away from them. The deeper moisture gauges, CWM2 and CWM1 showed more fluctuation with the rain event than the comparable moisture gauge in the other two sensor trees. In comparison, the two high moisture gauges showed less change with the rain event than the other two sensor trees. Overall, a substantial, visible change in SWP in the instrumented area was clear in all moisture gauges at Site 4.

On October 27, 2008, permeability measurements were taken across the field site. The permeability values of the entire site are shown in Figure 7-16. The points on the plot represent the individual locations within a section which were each tested three times and averaged. The solid lines represent the average permeability for a particular mix.

As Figure 7-16 shows, the permeability of each mix was very high following construction. The horizontal line shows the average permeability rate of that mix. The maximum rainfall intensity for a storm in this area is less than 0.01 cm/sec (Environment Canada, 2007). The results of Section 4C show the lowest average permeability of the three. The lower permeability may lead to the moisture moving to the instrumented area in a continual slower volume rather than a large volume over a shorter period of time. Section 4B does not have a higher permeability on average than 4A. However, 4B does contain a larger aggregate than 4A.

The larger aggregate may create a pervious concrete layer with a different void structure as discussed in Section 2.2. The difference between the SWP change in the two upper moisture gauges in Sections 4A and 4B could be due to the mix designs. The use of smaller aggregate had been identified to create many small voids whereas when larger aggregates are used there are fewer voids but they are larger in size. From Figure 7-13 to Figure 7-15 it can be verified that all moisture gauges at Site 4 were functioning following construction.

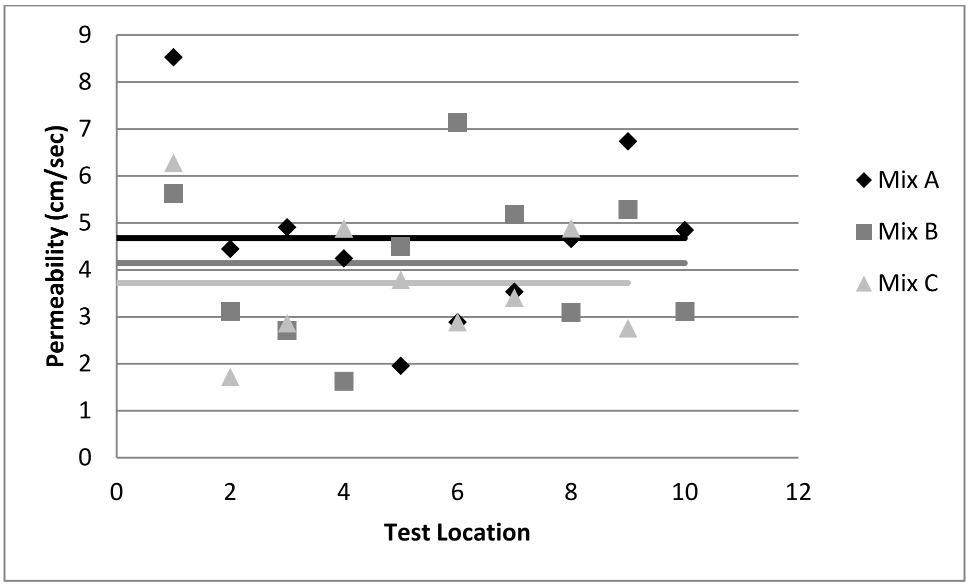


Figure 7-16: Permeability of Site 4 October 2008

7.4.2 Maintenance at Site 4 in Spring 2010

Maintenance was performed to clean the surface of the pervious concrete at Site 4 in May 10, 2010. The maintenance involved sweeping loose debris off the surface and then washing the surface with a power washing truck. This permeability renewal maintenance method was discussed in Section 5.4.

It was anticipated that the volume of water used to wash the pervious concrete would be noticeable in the instrumentation data. A change in SWP from the surface washing was evident in all three mixes and is shown in Figure 7-17 to Figure 7-19.

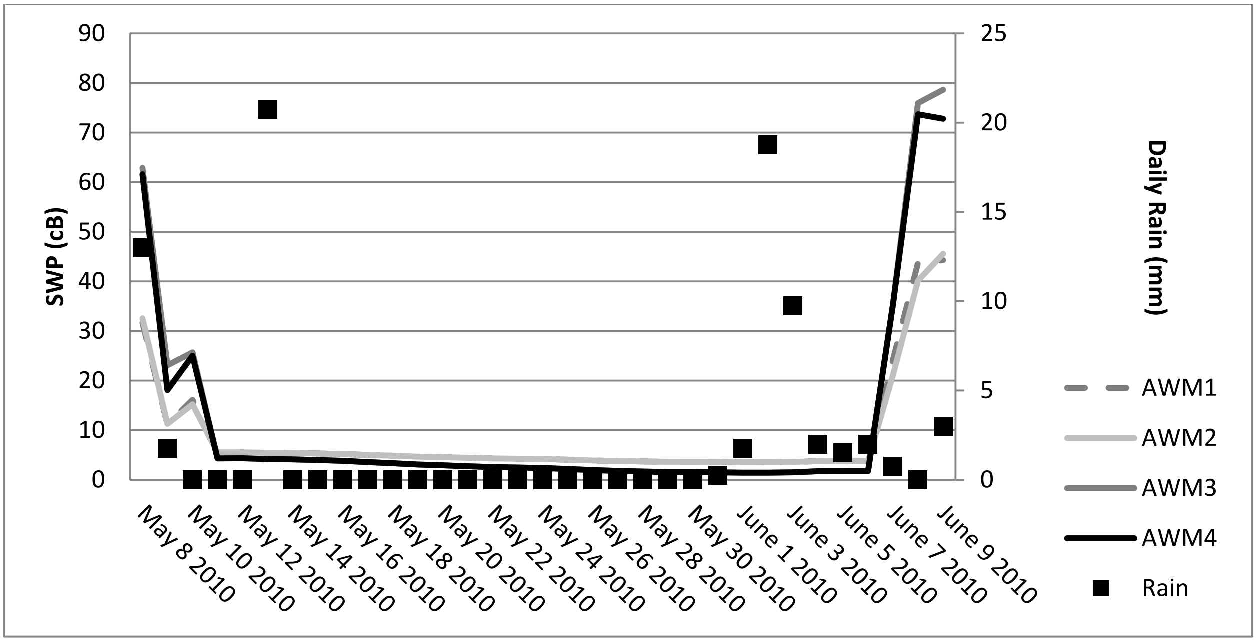


Figure 7-17: Site 4A Following Maintenance May 2010

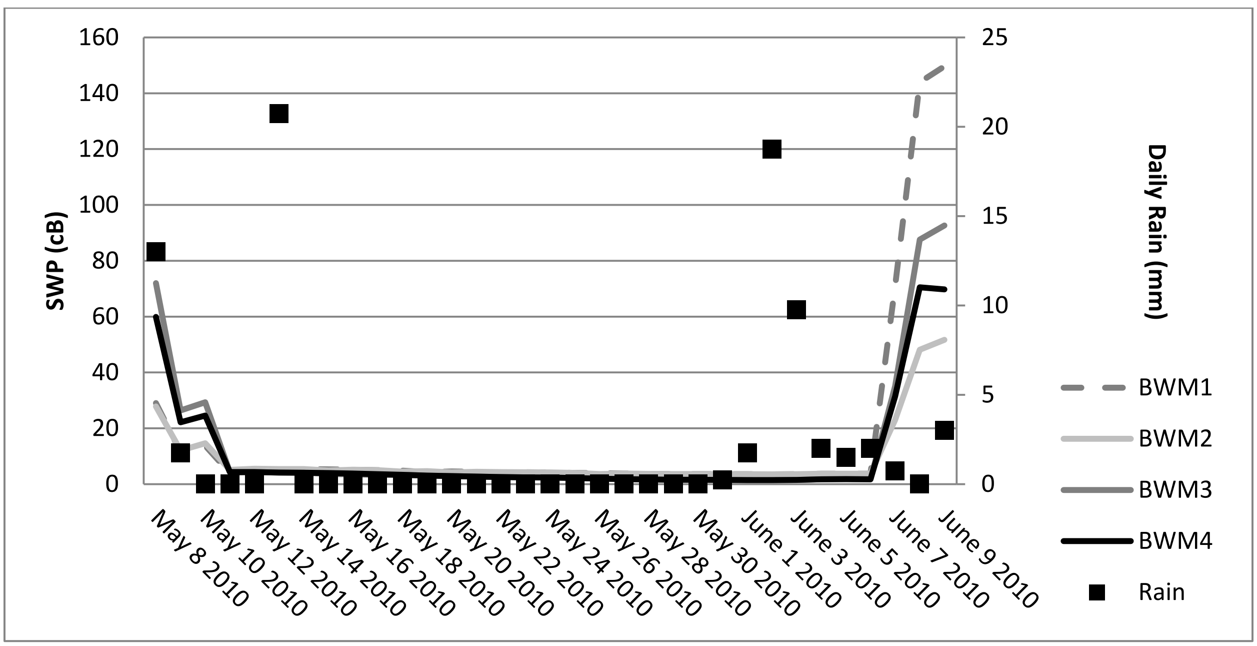


Figure 7-18: Site 4B Following Maintenance May 2010

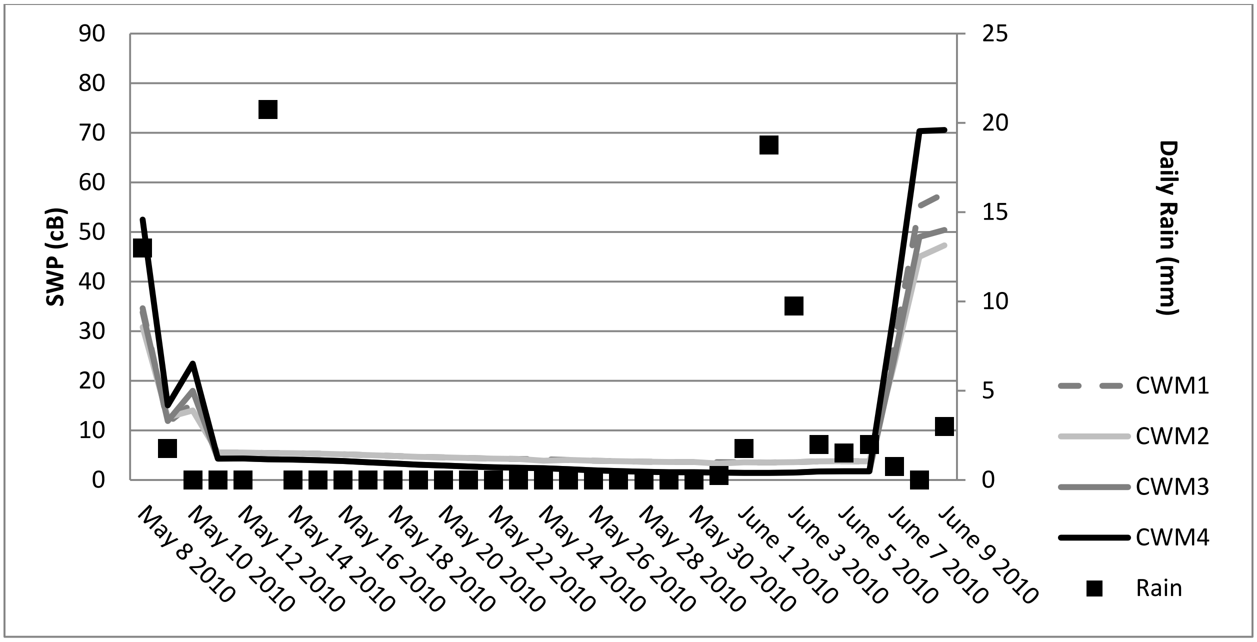


Figure 7-19: Site 4C Following Maintenance May 2010

The surface was washed at the beginning of May 2010 and Figure 7-17 to Figure 7-19 demonstrate, the water from this event was still present in the base and subgrade of the pavement structure until early June. Additionally, the three figures show that very little rain fell during this time and the water present in the pavement structure was a result of the surface washing. In all three mixes, by June 7, 2010, almost a month after the instrumented layers had become saturated, the base and subgrade were draining at a higher rate than water was moving into these materials.

In all three mixes the same trend occurred, in that SWP increased in the highest moisture gauge, “4”, to around the same level, 70 cb, on June 8, 2010, suggesting that the water level in the pavement structure was lowering in general. In Section 4B the behaviour of BWM1 indicates the subgrade around it draining water faster than the same material in the pavement structures of the two adjacent mixes. Moisture gauge BWM3 also shows faster drainage than the comparable moisture gauge “3” in each of the other two mixes.

Given that the sensor readings were consistent for almost four continuous weeks without any rain events, it is anticipated that the portions of the pavement structure that are instrumented: clear stone base; and the subgrade, were saturated during this time. From this event, values representing saturation of the representative materials were determined. The SWP that describes material being saturated is material specific and understood to be unique between similar materials at each field site. Saturation of the clear stone layer, moisture gauges 3 and 4, was 1.6 cb and saturation of the instrumentation in the subgrade was 3.7 cb. It was not suspected that the water would remain in the pavement structure for such a long period of time. The data indicates that the pavement structure became saturated during the permeability renewal maintenance. Although the permeability renewal maintenance method was identified to be a viable alternative in Chapter 5.0 it is evident that the amount of water used in this method should be considered.

7.4.3 Analysis

A large rain event was identified from the onsite weather station data or Egbert Environment Canada weather station for each season between Fall 2008 and Winter 2011. Each of the rain events and the associated moisture gauge data are presented in Appendix E. Table 7-4 shows a summary of the moisture movement through the pavement structure for each rain event. The rain events are summarized in Table 7-5.

The movement of moisture through the pavement structure over time was assessed in Table 7-4. Table 7-4 also compares the movement of moisture through the different sections at Site 4. When interpreting the results presented in Table 7-4 it is important to include the rain event information provided in Table 7-5. For all rain events a large amount of rainfall occurred on day 0. However, more rain may have occurred on any of the following days over approximately the next week. In the Fall of 2009 the rain event included 40 mm of rain, occurring on days 0 and 2 primarily.

The Sensor Tree A results from the Fall of 2009 indicate that the moisture from the rain event moved through the entire pavement structure on the same day as the rain event occurred, day 0. In the clear stone base layers the moisture drained away from the instrumentation on the same day as the rain event, day 0. Moisture was again present on day 2 of the rain event in the two clear stone base layer moisture gauges. The two deeper moisture gauges showed moisture present on day 0, as previously noted and that moisture continued to drain into the area until day 2 in the area of AWM2 and day 3 in AWM1. The moisture gauge data was observed until day 7.

Table 7-4: Moisture Movement at Site 4

Moisture Gauge Location in Pavement Structure (Depth from Surface)	**Date**	**Time from Rain Event to Instrumented Area (Days)**					
		Entering Area			**Draining Area**		
		4A	4B	4C	4A	4B	4C
Clear Stone Base (325 mm) WM4	Fall 2008	0 2	0 2	0 2	1 3-7	1 3-7	1 3-7
	Winter 2009	0-1	0-2	0-1	2-6	3-6	2-6
	Spring 2009	0 3 5 7-8	0 3 5 7-8	0 3-4 6	1-2 4 6 9	1-2 4 6 9	1-2 5 7-9
	Summer 2009	0 2 6 9	0-2 6 8	0 2 6	1 3-5 7-8	3-5 7 9	1 3-4 7-9
	Fall 2009	0 4 6 9	4-6 9	0-1 4-5 7 9	1-3 5 7-8	7-8	2-3 6 8
	Winter 2010	0-1	1 5	0-1	2-5	2-4	2-5
	Spring 2010	2 4 7 8	4 7-8	0 2 4 7-8	3 5-6 9	5-6 9	1 3 5-6 9
	Summer 2010	0 2 4 6-8	0 4 6-9	0 2 4 6-10	1 3 5 9	1-3 5 10-11	1 3 5 11
	Summer 2010	2 4-7 9	2 4-7 9	2 4-7 9	3 8	3 8	3 8
	Fall 2010	0-1 6-7	1 6-7 12	0-1 7	2-5 8-13	2-5 8-11 13	2-6 8-13
	Winter 2011	4-9	11-12	7	10-12	No drying occurred	8-12
Clear Stone Base (450 mm) WM3	Fall 2008	0 2	0 2 4	0 2	1 3-7	1 3 5-7	1 3-7
	Winter 2009	0-1	0-1	0-1	2-6	2-6	2-6
	Spring 2009	0-1 3-5	0 3 8	0 3-4 6	2 6-9	1-2 4-7 9	1-2 5 7-9
	Summer 2009	0-2 6	0-2 6 9	0-2 5-6	3-4 7-9	3-5 7-8	3-4 7-9
	Fall 2009	0-1 4-7 9	0 4 6 9	0-1 4-5 7	2-3 8	1-3 5 7-8	2-3 6 8-9
	Winter 2010	1	1	1	2-5	2-5	2-5
	Spring 2010	2 5 7-8	4 7-8	1-2 4 6-8	3-4 6 9	5-6 9	3 5 9
	Summer 2010	0 4 8-12	0 2 4 6 8-10	0 2 4 6 8-9	1-3 5-7 13-14	1 3 5 7 11	1 3 5 7 10-11
	Summer 2010	2 5-7 9	2 4-7 9	2 4-7 9	3-4 8	3 8	3 8
	Fall 2010	0-1 7 9	1 7 13	1 3 7 9	2-6 8 10-13	2-6 8-12	2 4-6 8 10-13
	Winter 2011	9-11	7	7	12	8-12	8-12
Subgrade (750 mm) WM2	Fall 2008	0-2	0-3 5	0-3	3-7	4 6-7	4-7
	Winter 2009	0-1	0-5	0-1	2-9	6-9	2-9
	Spring 2009	0-1 5-6	0-6	0-1 5	2-4 7-9	7-9	2-4 6-9
	Summer 2009	0-1 3 6	0-4 6	0-1 3 7	2 4-5 7-9	5 7-9	2 4-6 8-9
	Fall 2009	0-1 6-7	0 4-5 7 9	1-5 7-8	2-5 8-9	1-3 6 8	6 9
	Winter 2010	1-2	1	1-2	3-5	2-5	3-5
	Spring 2010	2 5 7-8	5 7-8	2-3 5 7-8	3-4 6 9	6 9	4 6 9
	Summer 2010	0-2 4-5 7-8 11 13	0 4-5 7 10-12	0-1 4-5 11-13	3 6 9-10 12 14	1-3 6 8-9 13-14	2-3 6-10 14-15
	Summer 2010	2 4-7 9	2 4-7 9	4-7 9	3 8	3 8	3 8
	Fall 2010	1 5 7 9	1 6-7 10 12	1-2 7 8 11	2-4 6 10-13	2-5 9 11 13	3-6 9-10 12-13
	Winter 2011	DRY	11-12	DRY	DRY	No drying occurred	DRY
Subgrade (825 mm) WM1	Fall 2008	0-3	0-3	0-3	4-7	4-7	4-7
	Winter 2009	0-1	0-6	0-1	2-9	7-9	2-9
	Spring 2009	0-1 6-7	0-6 8	0-1 5	2-5 8-9	7 9	2-4 6-9
	Summer 2009	0-1 3-4 7	1-3 6	0-1 3-4	2 5-6 8-9	4-5 7-9	2 5-9
	Fall 2009	0-2 6 8	0-1 3-5 7-9	1-3 6 8	3-5 7 9	2 6	4-5 7 9
	Winter 2010	1-2	1	1-2	3-5	2-5	3-5
	Spring 2010	2-3 5 7-8	5 7-8	2-3 5 7-8	4 6 9	6 9	4 6 9
	Summer 2010	3 5-6 10-14	0-1 4-5 10-11	0-1 4-5 11-13	2 4 7-9 15	2-3 6-9 12-15	2-3 6-10 14-15
	Summer 2010	0-2 4-7 9	2 4-7 9	4-7 9	3 8	3 8	3 8
	Fall 2010	1-2 5 7-8	0-1 5 7-8 10 13	1-2 7-8	3-4 6 9-13	2-4 6 9 11-12	3-6 9-13
	Winter 2011	10 12	8 11-12	DRY	11	9-10	DRY

Table 7-5: Site 4 Rain Events

Season	**Winter**			**Spring**			**Summer**			**Fall**		
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2008										Nov 13 2008	0	13
										Nov 14 2008	1	3
										Nov 15 2008	2	23
										Nov 16 2008	3	0
										Total Rainfall (mm)		29
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2009	2/11/2009	0	20	April 3 2009	0	32	July 23 2009	0	37	September 28 2009	0	27
	2/12/2009	1	15	April 4 2009	1	2	July 24 2009	1	3	September 29 2009	1	9
	Total Rainfall (mm)		35	April 5 2009	2	0	July 25 2009	2	16	Total Rainfall (mm)		36
				April 6 2009	3	18	July 26 2009	3	16			
				Total Rainfall (mm)		52	July 27 2009	4	1			
							July 28 2009	5	10			
							July 29 2009	6	3			
							Total Rainfall (mm)		86			
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2010	Dec 2 2009	0	9	May 1 2010	0	10	June 12 2010	0	16	Sept 21 2010	0	6
	Dec 3 2009	1	3	May 2 2010	1	14	June 13 2010	1	0	Sept 22 2010	1	15
	Total Rainfall (mm)		12	May 3 2010	2	12	June 14 2010	2	0	Sept 23 2010	2	0
				May 4 2010	3	3	June 15 2010	3	0	Sept 24 2010	3	0
				May 5 2010	4	12	June 16 2010	4	12	Sept 25 2010	4	2
				May 6 2010	5	0	*Total Rainfall (mm)*		28	Sept 26 2010	5	0
				May 7 2010	6	14				Sept 27 2010	6	7
				May 8 2010	7	13	July 9 2010	0	32	Sept 28 2010	7	23
				Total Rainfall (mm)		78	July 10 2010	1	0	*Total Rainfall (mm)*		53
							July 11 2010	2	6			
							July 12 2010	3	1			
							July 13 2010	4	16			
							Total Rainfall (mm)		55			
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2011	Dec 5 2010	0	22									
	Dec 6 2010	1	2									
	Dec 7 2010	2	11									
	Dec 8 2010	3	6									
	Total Rainfall (mm)		41									

The moisture gauges in Sensor Tree B during the Fall of 2009 rain event show moisture moving through the entire pavement structure on the same day as the rain event, day 0. Moisture drained away from the two instrumented areas in the clear stone base layer on the same day that the rain event occurred. Moisture again moved into and away from these two areas on the second day of rain in the rain event, day 2. Moisture drained into the deeper clear stone base instrumented area (BWM3) on day 4 as well. The two moisture gauges in the subgrade material showed moisture moving into the area on the same day as the rain event and continuing to drain into the area until day 3 of the rain event. Moisture gauge BWM2 also showed moisture moving into the instrumented area on day 5, possibly the moisture from BWM3 on day 4. All four moisture gauges showed continual drainage of moisture away from the instrumented area until the end of the observation, day 7.

In Sensor Tree C, moisture moved into and away from the clear stone material instrumented by CWM3 and CWM4 on the same day as the rain event occurred, day 0 and day 2. Moisture moved continually toward the instrumented areas in the subgrade material during days 0 to 3 of the rain event. All four moisture gauges indicated that moisture was draining away from the instrumented material throughout the remainder of the observation period, day 7.

The Fall 2009 rain event that was described in the above discussion occurred shortly after the construction of Site 4 and presents the initial drainage characteristics of the three pavement structures at the site. All three sections, 4A, 4B and 4C show similar drainage abilities, with moisture travelling throughout the depth of the pavement structure on the same day that the rain event occurred, day 0. Two days after the end of the rain event, day 4, the deepest instrumented material, 825 mm from the surface, subgrade, was demonstrating that the surrounding material was draining and no further moisture was moving into this area from the rain event.

The rain event in the Fall of 2010 included 53 mm of rain, dispersed between days 0 and 1 and 6 and 7 primarily. The larger quantities of rain occurred on days 1 and 7. It is unknown what quantity of rain generally moves throughout the depth of a pervious concrete pavement structure. It is assumed that small quantities of rain would not travel through the entire depth of the pavement structure as some would evaporate and some would eventually remain in a location where there is not an interconnected void, either in the pervious concrete layer or the clear stone base material. Assuming that a sufficient number of routes exist through the pavement structure, it is not anticipated that a small quantity of water becoming trapped in non-connected void areas is an issue.

In the area instrumented by Sensor Tree A, the moisture gauges in the clear stone base, AWM4 and AWM3 both indicated the presence of moisture on the same days as the rain events occurring, days 0, 1, 6 and 7. The higher moisture gauge in the subgrade material, AWM2 shows that only the moisture from the heavier rain falls drained to this area and it moves into the area on the same day as it occurred. Additionally moisture drains into this area and away from it on day 5 and 9 as well, 4 and 2 days after the heavier rainfalls respectively. The deepest moisture gauge AWM1 shows moisture draining into the area continually on days 1 to 2 and 7 to 8. Therefore on the same day as the heavier rain event and continuing for one following day. Again moisture moved into and away from the deepest instrumented area on day 5 as well. The behaviour of the material in the instrumented areas was observed until day 13. Each moisture gauge demonstrates a clear time span of three to five days between the two periods of rain when moisture in continually draining away from the areas. Therefore both rain events had completely drained through a depth in the pavement structure of 825 mm two to three days after occurring.

In the area of Sensor Tree B, the moisture gauges in the clear stone base layer both identified the presence of moisture on the same day as the rain event, days 1 and 7. Moisture is also present on days 12 and 13 in the areas of moisture gauges BWM4 and BWM3 respectively. No rain occurred at this time. It is possible that this is moisture draining through layers of the pervious concrete pavement structure from another source. Moisture gauge BWM2, the higher of the two in the subgrade material shows moisture on the same days as the rain events and then draining away. Moisture is again present on day 10 and 12, likely due to an outside source. The deepest moisture gauge, BWM1, shows the presence of moisture on the same day as the rain event and during the second rain event, day 7, the moisture continues to drain into the instrumented area for an additional day, until day 8. In all instrumented locations, drying of the material began the day after the rain event, day 2. The material at each level in the pavement structure was continually drying until the next rain event occurred, on day 6. Following the rain on days 6 and 7 the drying behaviour is not as defined due to the presence on moisture from another source.

The material instrumented by Sensor Tree C, shows the presence of the rain event on the same day as it occurred in each location, days 1 and 7. In the area of CWM4, the moisture moved into and away from the material generally on the same day. In the location of CWM3 the moisture also moved into and away from the material on the same day as the rain event occurred. However, moisture also moved into and away from the material on days 3 and 9 as well. It is anticipated that on both these days this was moisture that had drained through the entire pavement structure prior to this location at a slower rate. Perhaps some of the drainage routes that are available have become slower or others have become obstructed. Both of the moisture gauges in the subgrade material, CWM2 and CWM1 show the moisture moving into the associated material on the day of the rain event and the following day. Moisture also moves into and away from CWM2 on day 11. This could be the moisture that was present at CWM3 on day 9. The moisture is not identified at CWM1 at any time and there is anticipated to be a small quantity. A draining period is present at each moisture gauge two to three days after the occurrence of the rain event.

The pavement structure at Site 4 has a consistent thickness throughout. Each of the three pervious concrete mix designs at Site 4 are the same, the difference is only in the size and type of aggregate that was used. In the Fall of 2009 moisture from one rain event, occurring on days 0 and 2 moved through the entire pavement structure surrounding Sensor Tree A by day 3. Following day 3, all material was identified as drying. In the Fall of 2010, the same performance was occurring in the material surrounding Sensor Tree A. All moisture from a rain event on days 0 and 1 had moved throughout the pavement structure by day 2 or 3. In the Fall of 2009, moisture moved through the entire pavement structure around Sensor Tree B by day 3. There was some moisture present in the material surrounding AWM3 and AWM2 on days 4 and 5 which did demonstrate moving to AWM1. In the Fall of 2010 a rain event moved through the entire pavement structure at Sensor Tree B by day 2, after rain occurring on days 0 and 1. At Sensor Tree B the performance identified in the Fall of 2009 was still present in Fall of 2010. Finally, in the Fall of 2009 rain events moved through the pavement structure instrumented with Sensor Tree C one day after the rain event, day 3. In the Fall of 2010, the rain event moved through the pavement structure in Section 4C fully by two days after the end of the rain event, day 3.

The results presented for Site 4 indicate that each pavement structure is providing similar moisture behaviour. This finding is anticipated as the pavement structures are identical other than the pervious concrete layer. As was identified earlier in Figure 7-16 and in Chapter 6.0 the permeability rates of the three pervious concrete mixes at Site 4 are similar. Given that the pavement structures are consistent and the surface performance of the three mixes has been observed to be similar it is anticipated that the subsurface behaviour of the three sections would also be similar.

7.5 Site 5

At Site 5 two areas were instrumented during construction. Each instrumented area includes a sensor tree, constructed from three Watermark moisture gauges. Figure 7-20 shows a diagram of a sensor tree, at Site 5. Both of the sensor trees had the same dimensions. The data was measured from the moisture gauges using a CR1000 datalogger installed in the office at Site 5.

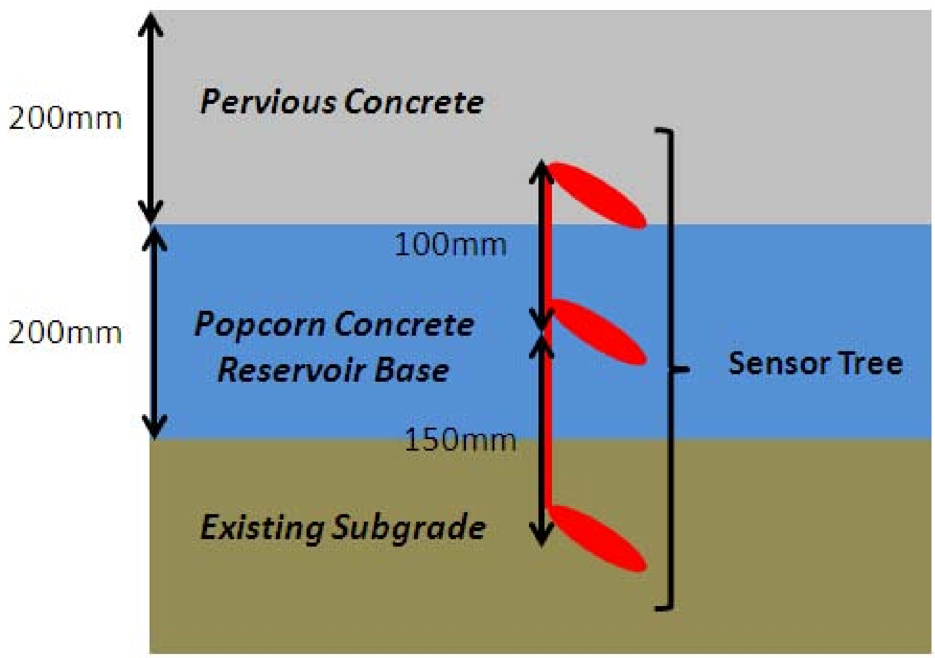


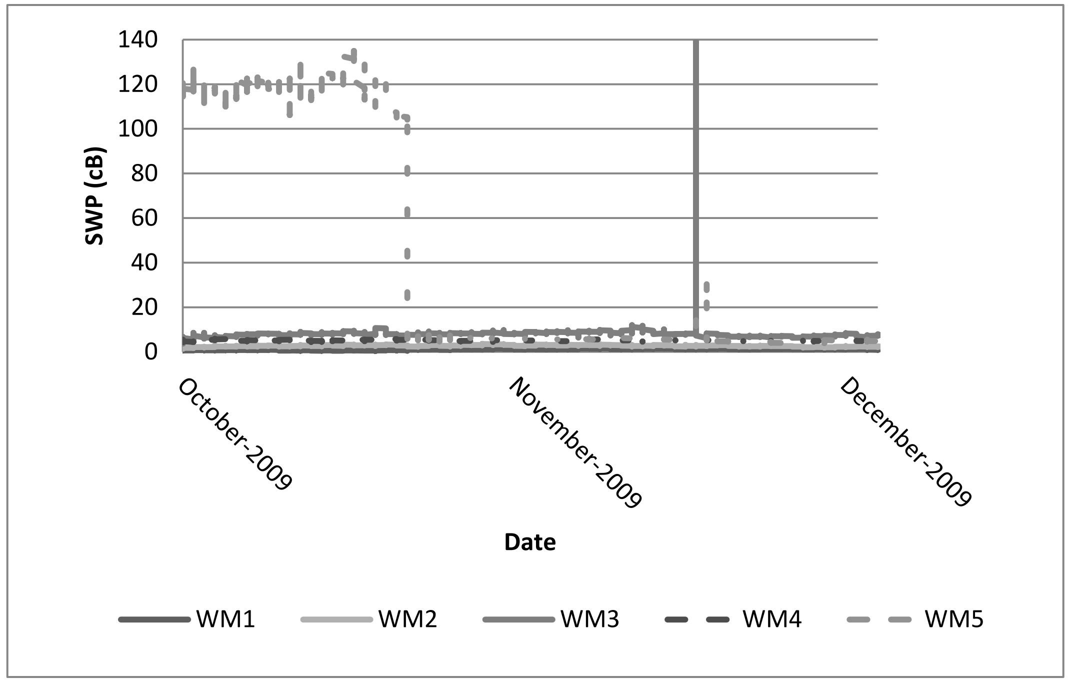
Figure 7-20: Sensor Tree Site 5

7.5.1 Instrumentation Functionality Verification

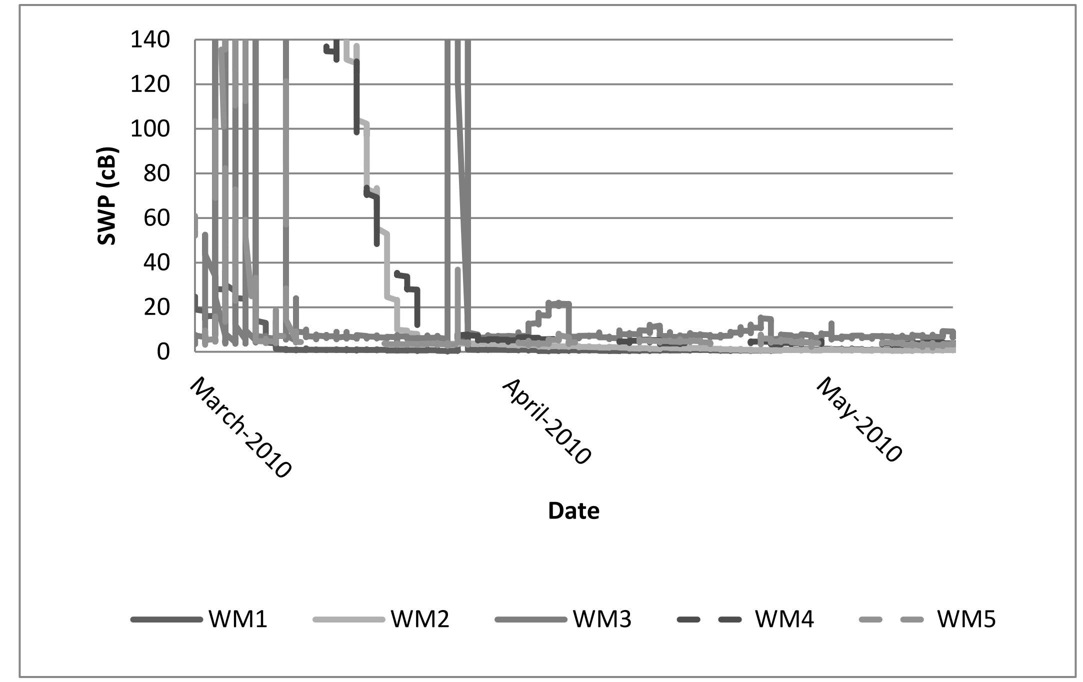
At Site 5, two sensor trees were prepared and installed during construction. During construction of the adjacent roller compacted concrete the wires connecting the moisture gauges to the datalogger were cut. The wires were reattached and five of the initial six moisture gauges were deemed to have the potential to function normally. However, due to the wires being cut it was unknown as to which wire connected to which moisture gauge. Efforts were made to evaluate which wire connected to which moisture gauge. These efforts included pouring water on the general area of each sensor tree. The data collected from these efforts was not conclusive as to which wire connected to each sensor. The data did not show noticeable differences in tracking water movement from the top of sensor trees to the bottom. Possibly if more water had been poured on the sensor trees and with a larger time period between each sensor tree this would have been more effective. Data from all sensors prior to this evaluation and after was low. The SWP was less than 5 cb. These low values suggest that the area surrounding the instrumentation was saturated. To determine which data set is from a particular sensor all fall and spring seasons from the collection period were analyzed, October 2009 to June 2011. Figure 7-21 shows moisture gauge data from Fall 2009, Spring 2010, Fall 2010 and Spring 2011.

From analysis of the moisture gauge data shown in Figure 7-21 it was determined that the two moisture gauges in the pervious concrete layer are both collecting data. The two moisture gauges that are in the pervious concrete layer were identified as they reacted to freezing temperatures and the other moisture gauges did not. When freezing occurs in a material there is a large spike in the SWP. Figure 7-21 shows two moisture gauges, WM3 and WM5, reacting to freezing and thawing conditions while the other three do not show spiking. Figure 7-21 shows the moisture gauge results from the Fall of 2009 and 2010 and the Spring seasons of 2010 and 2011. In both seasons the WM3 and WM5 moisture gauges are reacting to surrounding changes by having extreme maximum and minimum values which often exceeded 140 cb. In the fall, the other three sensors appear saturated as the readings are consistently very low. In the spring season the other three moisture gauges are again surrounded by moist material.

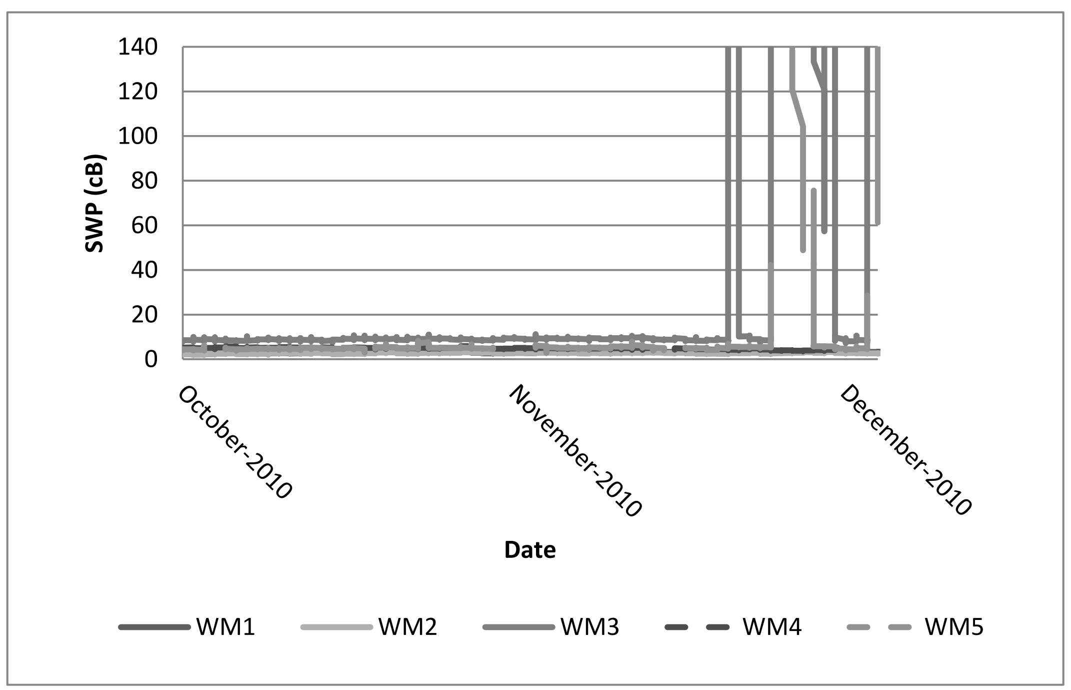
The location of the other three sensors within the pavement structure was evaluated using rain event data. Table 7-6 shows a rain event in July 2010 after an eight day long period of no rain. The highlighted cells track the point when the moisture gauge readings are the lowest. The slope between moisture gauge readings describes the behaviour in the surrounding material as described in Section 7.1.



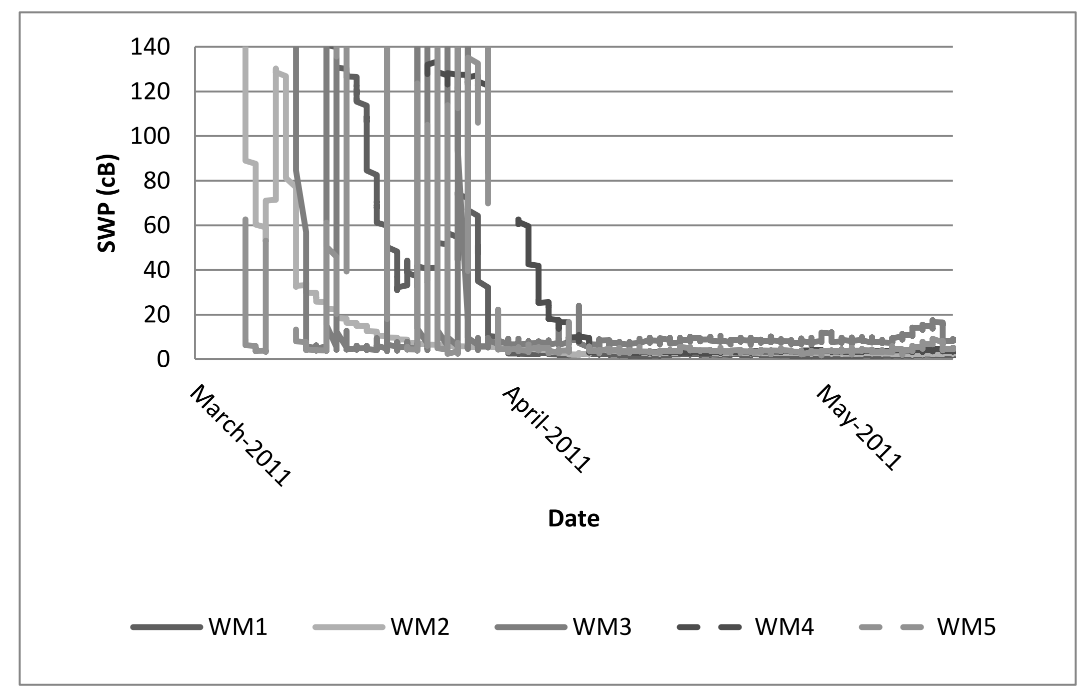
Fall 2009



Spring 2010



Fall 2010



Spring 2011

Figure 7-21: Moisture Gauges in Fall and Spring Seasons 2009 - 2011

Table 7-6: Moisture Gauge SWP Results July 5, 2010 to July 16, 2010

Date	Rain (mm)	Daily Average Soil Water Potential (cb)				
		WM1	WM2	WM3	WM4	WM5
July 5 2010	0	2.4	2.1	8.2	4.6	3.5
July 6 2010	0	2.6	2.2	9.4	4.9	3.6
July 7 2010	0	2.6	2.3	11.8	4.8	3.6
July 8 2010	0	2.6	2.2	14.4	4.9	3.7
July 9 2010	48	2.4	2.0	13.0	4.5	3.8
July 10 2010	1	2.6	2.3	6.7	4.5	3.9
July 11 2010	0	2.7	2.4	6.5	4.7	3.7
July 12 2010	0	2.6	2.3	6.7	4.6	3.6
July 13 2010	2	2.6	2.2	6.9	4.5	3.7
July 14 2010	0	2.9	2.5	7.4	4.7	3.8
July 15 2010	0	3.1	2.6	7.6	4.9	4.0
July 16 2010	0	3.1	2.7	7.3	4.8	3.9

Table 7-7 shows another rain event after a period of dry weather. The focus of Table 7-7 is that there was minimal rain following the large quantity of 75 mm on September 30, 2010. Since there was minimal rain following this event, the analysis was focused on monitoring how the moisture moved away from the instrumented areas as it drained through the pavement structure.

Table 7-7: Moisture Gauge SWP Results September 29, 2010 to October 7, 2010

Date	Rain (mm)	Daily Average Soil Water Potential (cb)				
		WM1	WM2	WM3	WM4	WM5
September 29 2010	0	2.5	2.7	7.9	5.1	4.2
September 30 2010	75	2.4	2.5	8.0	5.0	4.2
October 1 2010	5	2.5	2.4	8.3	5.1	4.3
October 2 2010	0	2.3	2.1	8.5	4.7	4.2
October 3 2010	0	2.4	2.2	8.7	4.7	4.4
October 4 2010	0	2.6	2.4	8.7	4.9	4.7
October 5 2010	0	2.8	2.6	8.3	5.2	4.5
October 6 2010	11	2.6	2.6	8.2	5.0	4.3
October 7 2010	7	2.8	2.4	8.7	5.1	4.6

The data in Table 7-6 and Table 7-7 and Figure 7-21 have been compiled to draw the following conclusions regarding the location of each moisture gauge.

- WM3 and WM5 are in the pervious concrete layer based on the data during freezing temperatures and in Table 7-6 and Table 7-7.
  - In Table 7-6 there is a large change in SWP in WM3 the day after the rain event. The large change suggests a large amount of water moved into the instrumented area at the same time. The flow of water is anticipated to slow down as it travels deeper into the pavement structure because there are more possible routes through the voids of the highly permeable concrete, some of which may or may not connect to other interconnected voids. In Table 7-7 the moisture is generally flowing away from WM3 at a higher rate than it is arriving at the instrumentation. This is likely a combination of the high permeability characteristics of the popcorn concrete and water moving slower through the top of the pervious concrete layer due to presence of debris.
  - Moisture gauge WM5 shows similar results in both tables to the results in Table 7-6 and Table 7-7 for WM3. The results suggest that moisture is moving away from the instrumentation at nearly the same rate it is travelling to the instrumented area. The SWP values are low suggesting that there is likely always some moisture present in the area, perhaps trapped in voids that are not interconnected. The high drainage rate of the popcorn concrete reservoir base allows water to continue to drain away from the area. The permeability results in Chapter 6 identify limited permeability through the pervious concrete pavement. Similar to Site 3, it is possible that while moisture remains close to the surface as it drains slowly through the pervious concrete layer, a portion is evaporating.
  - Moisture gauge WM3 is anticipated to be in the Section 5C (control mix) and it is predicted that WM5 is in the Section 5B (latex mix). Analysis of the cores discussed in Section 6.4.2 from these two sections have lead to this prediction. The 5B cores showed the mix breaking down within the core, in one case leading to the core splitting in half. This has not been noticed in the cores from 5C. The breaking down of the paste in mix 5B could lead to some voids being filled and some water remaining in the pavement structure for a longer time before draining. This behaviour is suggested by the data collected from WM5.

- Moisture gauge WM4 is anticipated to be located in the popcorn concrete reservoir base layer. In the case of the rain events shown in Table 7-6 and Table 7-7 water is moving into the instrumented area and then draining away from the area within a few days of the rain event occurring. It is unknown as to which sensor tree the WM4 moisture gauge is on however it is probable that it is on the sensor tree in Section 5C. The water moves more quickly through the pervious concrete layer in Section 5C, rather than remaining there as is shown in Section 5B. The results suggest the water moves quickly through the popcorn concrete reservoir base layer where WM4 is located and thus would have to feed into that area quickly, therefore likely from Section 5C.
- Finally moisture gauges WM1 and WM2 are assumed to be in the subgrade. The data collected from both the moisture gauges shows the last change in readings of all the sensors, suggesting that this is the deepest location. The two moisture gauges also have similar, low SWP readings which could relate to a finer material.

7.5.2 Analysis

One of the largest storm events for each of the winter, spring, summer and fall seasons has been identified in the weather station data and the corresponding collected moisture gauge data is shown in Appendix E. Table 7-8 shows a summary of the moisture gauge data from each of the rain events. Table 7-9 shows a summary of the rain events at Site 5 between Fall 2009 and Spring 2011.

All of the rain events presented in Table 7-8 indicate that water is moving through the pavement structure. Given the circumstances of the instrumentation at Site 5 the data was presented slightly differently in Table 7-8 than in Table 7-2 and Table 7-4. The days presented in Table 7-8 represent the day in which moisture is initially identified as reaching a moisture gauge or beginning to drain away from one. A range is presented if two moisture gauges are deemed to be at the same vertical location in a pavement structure and moisture reached these locations or started draining away from them at different times. The range also describes up to how many days a behaviour of either movement into or away from an instrumented material was observed.

In the Fall of 2009 a rain event of 42 mm occurred, with the rain fall being observed on days 0 and 2. Moisture reached the instrumented areas of the pervious concrete layer on the same day or the following day to that of the rain event occurring. Moisture from the rain event drained into the instrumented portion of the base layer from days 0 to 2, therefore continuing to move into this area up to two days after the rain event occurred. Finally moisture from the Fall of 2009 rain event reached the existing subgrade one to two days after the rain event had occurred. The time required for moisture to move through the entire pavement structure is up to two days after the end of the rain event and therefore day 4 in the rain event during the Fall of 2009.

Table 7-8: Summary of Water Movement through Pervious Concrete Pavement Structure

Pavement Structure Layer Instrumentation Location	**Date**	**Time from Rain Event to Instrumented Area Activity (Days)**	
		Entering Area	**Draining From Area**
Pervious Concrete (Bottom of Layer)	Fall 2009	0 – 1	1 – 3
	Winter 2010	0 – 2	2 +
	Spring 2010	0 – 1	1 – 4
	Summer 2010	0 – 1	3
	Fall 2010	3 – 5	7
	Winter 2011	0 – 2	3 +
	Spring 2011	2 – 4	1 – 5
Popcorn Concrete Reservoir Base (Middle of Layer)	Fall 2009	0 – 2	1 – 3
	Winter 2010	0 – 3	4 +
	Spring 2010	0	1
	Summer 2010	0	3
	Fall 2010	2	4
	Winter 2011	2 – 5	6 +
	Spring 2011	0 – 1	1 – 2
Existing Subgrade	Fall 2009	1 – 2	2 – 3
	Winter 2010	0 – 3	2 +
	Spring 2010	3	4
	Summer 2010	2	3
	Fall 2010	2	3
	Winter 2011	2 – 5	6 – 7
	Spring 2011	3	5

Table 7-9: Site 5 Rain Events

Season	Winter			Spring			Summer			Fall		
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2009										10/22/2009	0	14
										10/23/2009	1	0
										10/24/2009	2	28
										Total Rainfall (mm)		42
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2010	1/25/2010	0	49	4/27/2010	0	11	7/17/2010	0	45	9/30/2010	0	75
	Total Rainfall (mm)		49	4/28/2010	1	24	7/18/2010	1	0	10/1/2010	1	5
				Total Rainfall (mm)		35	7/19/2010	2	11	10/2/2010	2	
							7/20/2010	3	0	10/3/2010	3	
							7/21/2010	4	7	10/4/2010	4	
							7/22/2010	5	11	10/5/2010	5	
							Total Rainfall (mm)		74	10/6/2010	6	11
										10/7/2010	7	7
										Total Rainfall (mm)		98
Year	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**	**Date**	**Day**	**Rainfall (mm)**
2011	11/30/2010	0	11	4/16/2011	0	21						
	12/1/2010	1	44	4/17/2011	1	22						
	12/2/2010	2	2	4/18/2011	2	0						
	Total Rainfall (mm)		57	4/19/2011	3	0						
				4/20/2011	4	25						
				Total Rainfall (mm)		668						

In the Spring of 2011 it takes one to four days for moisture from the rain event to reach the instrumentation in the pervious concrete layer. Comparably moisture reaches the moisture gauges in the base layer on the same day as the rain occurs or the following days. Moisture is shown to generally reach the moisture gauge in the subgrade three days after the rain event occurs. The slow movement of moisture to the moisture gauge in the pervious concrete matches the permeability data shown in Figure 6-10. At Site 5 the permeability of sections 5A, 5B, and 5C have dropped below the level deemed to be adequate. It is therefore anticipated that any moisture that does drain through the pavement structure would do so slowly. The moisture that is identified in the reservoir base layer is likely moving into the instrumented area from a surrounding source. To the west of the pervious concrete there is a substantial embankment and moisture moving through this area may drain through the reservoir layer. The reservoir layer is also continual outside of the pervious concrete area. It extends under the adjacent roller compacted concrete. some drainage may occur past the instrumentation from the adjacent roller compacted concrete. The moisture moving into the subgrade is anticipated to have come from the moisture in the reservoir layer rather than any moisture draining from the surface, vertically through the pervious concrete pavement. Water that does drain through the pervious concrete pavement is anticipated to continue moving vertically. However, it is suspected that it does not drain into the next two instrumented areas in a time span or volume that is identified in this analysis.

7.6 Findings

The data collected and reviewed from the moisture gauges at each field site indicates that the permeability rate of the pervious concrete surface layer is critical as to whether moisture will drain vertically through the pervious concrete pavement structure. The permeability rate of each field site since construction is presented in Figure 6-10. The subsurface drainage performance of the field sites relates to the permeability rates shown in Figure 6-10. Vertical drainage from the pavement surface to the subgrade is observed in the data from Site 4. The data from Site 3 does not provide any results demonstrating vertical movement through the entire pavement structure. Vertical movement within the pavement structure at Site 3 is also limited. The permeability rate of the pervious concrete surface at Site 3 is less than the maximum rainfall rate and is therefore inadequate. At Site 5 there is also limited permeability on the surface of the pervious concrete pavement. Moisture can be identified in the instrumented portion of the pervious concrete layer after a rain event though. In the Fall of 2009, immediately following construction, moisture drained into the instrumented portion of the pervious concrete layer on the same day or following day as the rain event occurring. By the Spring of 2011 moisture reaches the instrumented portion of the pervious concrete pavement two to four days after the rain event. This delay reflects the decreased permeability rate of the pervious concrete pavement surface. At Site 5 however, it is possible to track moisture moving through the reservoir base layer and into the subgrade during rain events. The drainage of the reservoir base layer continues to be very functional and effective.

At all three sites, the moisture gauge data from the winter seasons show some different trends than those from the other three seasons. It is anticipated that given the analysis method not all winter activity was captured or fully described. In this research the results from the winter seasons were primarily of interest to evaluate if the possibility of drainage during this season existed in pervious concrete pavement structures. At all three field sites minimum values were observed in the data during the winter seasons that represented the presence of moisture and were comparable to the other seasons. It is therefore evident that moisture is moving through the pavement structures throughout the winter seasons.

The three field sites that were instrumented, each showed clearly different performance in terms on subsurface drainage. Comparisons regarding the total time required for a rain event to drain through the pervious concrete pavement structure are not relevant as only Site 4 clearly demonstrated this ability and continued to demonstrate this. However, even within Site 4 there were fluctuations between seasons and years regarding the number of days for a rain event to move through the depths of the pavement structure. Some of the reasons for changes in time between rain events and water reaching the instrumentation are the following:

- Previous recent rain events and associated moisture still being present in some areas of the pavement structure;
- No rain events recently and area is dry and thus all drainage routes are available so water drains easily and efficiently;
- Horizontal, downhill flow from surrounding area: pervious concrete; vegetated berm; and roller compacter concrete and;
- Debris in voids at surface of pervious concrete, slowing down initial drainage of water;

The moisture gauges installed in the three field sites during construction provided a unique descriptive tool regarding the subsurface behaviour of pervious concrete pavement. The three field sites that were instrumented all proved to have adequate pavement structures for drainage purposes below the pervious concrete layer. It was not possible to fully examine the true drainage capabilities of these layers without the pervious concrete layers providing vertical drainage capabilities. The conclusions that could be drawn were that the unbound clear stone base layer at Site 4 was effective in providing vertical drainage. The subgrade material at Site 4 is silty sand and therefore the instrumentation did not demonstrate continual saturation of the clear stone base layer. The drainage through the pavement structure at Site 4 did not appear to have any locations leading to delays or water being held for a substantial period of time. This performance of the base layer and subgrade is ideal and superior to the anticipated average pervious concrete pavement scenario. At Site 5 the bound, popcorn concrete reservoir base, demonstrated the capacity to handle efficient drainage from outside moisture sources. Similar to Site 4, storage of moisture in the reservoir base layer was not experienced in the analysis of the moisture gauge results. Therefore, the pervious concrete pavement structure at Site 4 surpassed the expected drainage capabilities of the system.

7.7 Summary

The three field sites that were instrumented with moisture gauges demonstrated that it was possible to successfully include this type of sensor in a pervious concrete pavement structure. The verification of the functionality of all the instrumentation was successful at each field site. Through the verification of the moisture gauge functionality other observations were also concluded. At Site 3 the pervious concrete pavement structure routinely froze to a depth of 300 mm throughout the winter season. During extended periods of cold weather freezing would occur to a depth of 430 mm. Permeability renewal maintenance proved to have the potential to saturate the pervious concrete pavement structure for a four week period. As is discussed below, the storage capabilities of the clear stone base layer were only tested following permeability renewal maintenance involving washing of the surface. This maintenance was performed using street grade equipment. The volume of water used in this maintenance lead to the pervious concrete pavement structure from a depth of 325 mm to 825 mm being saturated for four weeks.

The water may have been higher in the pavement structure but the moisture gauge closest to the surface was at a depth of 325 mm.

The subsurface drainage that was quantified by the instrumentation confirmed observations from the surface of the pavement and exceeded other expectations. Sites 3 and 5 had limited drainage capabilities on the surface of the pervious concrete pavement, Site 3 shortly after construction, Site 5 within a year following construction. The subsurface analysis quantified and confirmed that moisture was not able to drain completely vertically through the pavement structures at these two sites due to the limited access in the pervious concrete pavement surface. In comparison, the subsurface drainage at Site 4 surpassed the assumed behaviour of pervious concrete pavement structures. The pavement structure in general at Site 4 was highly permeable and this was identified as moisture was not found to be holding in the bottom of the storage base layer at any time or for any period of time. The successful overall drainage performance of Site 4 demonstrates the ability to effectively use pervious concrete pavement in southern Canada.

8.0 CONCLUSIONS AND RECOMMENDATIONS

In the process of evaluating the performance of pervious concrete pavement in Canada, the research included many phases of the life cycle of a pavement. This research was the first in Canada to consider the use of pervious concrete pavement. It provides preliminary results for use of pervious concrete pavement in Canada. Additional research into each aspect of the life cycle of a pervious concrete pavement will be required in the long term for ongoing development. Experiences throughout this research in the planning, design, construction and maintenance phases were evaluated objectively and quantified when feasible. The inclusion of laboratory testing and full scale field sites provided multiple resources for assessing the behaviour and performance of pervious concrete pavement in the Canadian climate.

The objective of this research was to consider multiple applications, designs and construction methods of pervious concrete pavement and assess which, if any, were suitable for use in the Canadian climate. The applications that were included in this research were areas loaded in static mode by parked vehicles, areas loaded from slow moving vehicles, parking lots in general used by public vehicular and pedestrian traffic and driveways loaded by public vehicles and heavier loaded trucks. When compared on the basis of permeability, the areas loaded with static traffic demonstrated better permeability rates at some times. In terms of surface distress development, a relationship between the application and the surface condition was not evident. Traffic within this project was generally low volume and low speed. Had a larger range in traffic been evaluated it is anticipated that surface condition changes related to traffic may have been noticed. A larger range in traffic was not included in this research as at this time pervious concrete pavement was being considered for low speed low volume applications only.

Design considerations included both the pavement structural design and mix design. The general pavement structure at the field sites was similar with very slight variations in the layer thicknesses. Substantial deterioration in pavement structural performance was identified at one site, Site 2. At Site 2, localized areas of limited structural capacity developed in the spring of 2011, approximately 2.5 years of age. It is anticipated that these areas developed due to loss of supporting material in the pavement structure. The areas were primarily located along a drainage pipe that was included in the pavement structure. It is possible that granular base material settled into the subgrade or that the granular base material disintegrated over time. The structural capacity decreased in one other area of the site although not to the same extent. Similarly it is suspected that supporting material in the pavement structure was lost. Sites 1, 4 and 5 showed changes in structural capacity over the monitoring timeline however no locations of substantial decreases in structural capacity were identified. When the structural capacity of various field sites was compared based on age it was found that field sites of the same age generally had statistically different structural capacities. At eight months of age there were no statistical differences in structural capacity identified between the multiple field sites. However, between 12 to 18 months, 22 to 24 months, 26 to 31 months and 37 to 48 months, statistical differences in structural capacity between sites. These results demonstrate that even under personal vehicle loading the structural design of a pervious concrete structure should be considered as differences can exist.

Each field site included a different mix design, some of which had multiple mix designs. The relationship between the void content and hardened density of the pervious concrete cores was linear with no cores being visually identified as outliers. The void content and density of the cores were compared in terms of: inclusion of fine aggregate; water cement ratio; aggregate type; aggregate size; and inclusion of admixtures. When the W/C ratio increased in a mix design, the minimum measured void content of the cores decreased. Therefore when the W/C ratio was higher, there was a possibility of having a lower void content in the core. The average void content of the cores did not show this trend with changes in W/C. The type of aggregate in the mix design showed differences in the relationship between the void content and density of the cores. The size of aggregate in a pervious concrete mix was found to determine the relationship between the void content and density of the material. Cores with different sizes of aggregate in the mix design had visually different linear relationships between the void content and hardened density. The other aspects of the mix design that were compared: inclusion of fine aggregate; aggregate type; and inclusion of admixtures did not have specific trends and the relationship between void content and density was uniform.

The field sites constructed in this research included a few variables in terms of the construction method. The variables included: staging; concrete truck type; placement method; compaction effort; and inclusion of joints. The options available for staging a pervious concrete project are certainly dependent on the project. However, in general there are often many alternatives. The construction of Site 4 demonstrated that it is necessary to provide edges of a pervious concrete pavement with the opportunity to cure prior to adjacent construction being carried out. At Site 1, pervious concrete was left untouched for 12 to 24 hours before pervious concrete was constructed adjacent to it. The performance of the construction joints at Site1 was good. Concrete was successfully supplied by both volumetric mobile mixers and conventional ready mixed concrete trucks in this research. Both methods adequately supplied the concrete however the slow discharge of pervious concrete in general led to challenges in maintaining sufficient material for paving with an asphalt paver. Pervious concrete was placed successfully manually from the chute of concrete trucks, using an asphalt paver, using a Bidwell Bridgedeck paver and using a Razorback paver. No substantial challenges were experienced using any of these methods. The pervious concrete at the field sites was compacted using manual rollers, vibrating automated rollers, vibratory plate compactors and a Razorback paver. In the location that was placed with the Bidwell Bridgedeck paver at Site 2, no compactive effort was applied to the surface. This area has shown to exhibit the highest amount of ravelling of any of the sites. The other sites were all compacted during construction and have generally performed adequately in terms of permeability and better than the Bidwell Bridgedeck paver section of Site 2 in regards to surface condition. At Site 3, minimal compaction was applied to the surface during construction. A small manual roller was only used at this location. The low permeability of this site is attributed to the mix design characteristics rather than the compaction in the construction. Joints were handled at the field sites in one of three ways: saw cut; formed with a joint former; or not constructed. Both saw cutting and forming the joints had similar long term performance. The selection of one method over another is project specific and either can be used effectively. When joints were not included, cracks developed. However, the cracks were not observed to have a large impact on pavement serviceability. In some cases these cracks are not anticipated to be identifiable by the general user.

Permeability renewal maintenance methods were evaluated in this research, sometimes due to poor performance of the field site but also for research purposes. The following findings have been made:

- The initial permeability of the pervious concrete pavement can influence future performance.

- Power washing using personal sized equipment can push debris deeper into voids and decrease permeability rather than improve it.
- Sweeping of the surface can be effective in removing debris off the surface but not from deeper voids, therefore not necessarily improving permeability.
- Washing the surface with a large diameter hose can dislodge debris deep in voids and renew permeability, in some cases, to near initial permeability values.
- Full scale power washing trucks can be less abrasive on the pavement surface and prove to be more suitable than personal sized power washing equipment.
- Increases in permeability at field sites were observed without carrying out permeability renewal maintenance. It is anticipated that the increases in permeability were a result of intense rain events in the late summer season.

Pervious concrete pavement behaviour was evaluated under the presence of three factors: freeze-thaw cycles and moisture; winter maintenance; and pedestrian and vehicle traffic. Three types of samples were included in the evaluation: cores taken from field sites; slabs cast in the laboratory; and full scale field sites. The behaviour of the pervious concrete was assessed in terms of permeability and surface condition.

Freeze-thaw cycling and moisture were found to alter the internal structure of pervious concrete. The internal structure is altered when the paste in the pervious concrete deteriorates due to the freeze-thaw cycling. This deterioration creates small particles which can move throughout the pervious concrete and close previously available drainage paths. The extent to which this occurs is anticipated to be highly dependent on the characteristics of the paste in the pervious concrete.

The type of winter maintenance that was used on the pervious concrete affected the permeability and surface condition. The application of sand as a winter maintenance method decreased the permeability but not to an unacceptable level. When a salt solution was used, the permeability also decreased but at a much lower rate than when compared to sand being applied. However, it is notable that the surface condition of the HSaltHP slabs was observed to be worse than any other samples in the project. In short, these slabs failed completely by the end of the five year equivalency of freeze-thaw cycles. All three slabs deteriorated to a point of complete failure.

Exposure to daily use conditions including pedestrian and vehicle traffic lead to decreases in permeability at all the field sites. In half the site sections the permeability remained greater than the maximum rainfall rate and therefore the pervious concrete pavement continued to drain adequately. The five field sites where the permeability rate did not remain adequate had lower permeability rates than the other sites immediately following construction. Two of these sections did not receive any winter maintenance. The initial permeability of the field sites proved to be important. Some sites directly after construction, had very high permeability rates, and remained adequate after time in service over the multiple year evaluation period.

Surface distresses developed at each field site. The types of distresses that had developed in the cores and slabs in the laboratory and were generally not substantially worse at the field sites, suggesting that pedestrian and vehicle traffic do not necessarily escalate distresses caused by the Canadian climate and corresponding winter activities. Additional distresses were observed on the surface of the field sites and these could be related to the mix designs, construction method or the features of the field site layout.

The three field sites that were instrumented with moisture gauges demonstrated that it was possible to successfully include this type of sensor in a pervious concrete pavement structure. The verification of the functionality of all the instrumentation was successful at each field site. Through the verification of the moisture functionality other observations were also concluded. At Site 3 the pervious concrete pavement structure routinely froze to a depth of 300 mm throughout the winter season. During extended periods of cold weather freezing would occur to a depth of 430 mm. Permeability renewal maintenance proved to have the potential to saturate the pervious concrete pavement structure for a four week period. As is discussed below, the storage capabilities of the clear stone base layer were only tested following permeability renewal maintenance involving washing of the surface. This maintenance was performed using street grade equipment. The volume of water used in this maintenance lead to the pervious concrete pavement structure from a depth of 325 mm to 825 mm being saturated for four weeks. The water may have been higher in the pavement structure but the moisture gauge closest to the surface was at a depth of 325 mm.

The subsurface drainage that was quantified by the instrumentation confirmed observations from the surface of the pavement and exceeded other expectations. Sites 3 and 5 had limited drainage capabilities on the surface of the pervious concrete pavement, Site 3 shortly after construction, Site 5 within a year following construction. The subsurface analysis quantified and confirmed that moisture was not able to drain completely vertically through the pavement structures at these two sites due to the limited access in the pervious concrete pavement surface. In comparison, the subsurface drainage at Site 4 surpassed the assumed behaviour of pervious concrete pavement structures. The pavement structure in general at Site 4 was highly permeable and this was identified as moisture was not identified to be holding in the bottom of the storage base layer at any time or for any period of time. The successful overall drainage performance of Site 4 demonstrates the ability to effectively use pervious concrete pavement in Canada.

The two research hypotheses of this research were the following:

1. Pervious concrete pavement can be successfully planned, designed, constructed and maintained in Canada for successful performance based on surface evaluations of permeability rate and surface condition.
2. Verification that the subsurface drainage capabilities of pervious concrete pavement are as described in literature and by industry can be quantified using instrumentation.

The findings presented in this Chapter conclude that both of the research hypotheses were demonstrated in this research. The four phases of a pervious concrete pavement life cycle were evaluated in both the laboratory and field and critical aspects to achieving performance in terms of permeability and surface condition were identified and investigated. The use of moisture gauges at some of the field sites allowed for monitoring of the movement of moisture through the pervious concrete pavement structures. One field site clearly displayed drainage of moisture throughout the entire pavement structure. The other two sites had limited surface permeability and thus prohibited moisture from directly accessing the remainder of the pavement structure. The site that had an adequate surface permeability rate demonstrated highly effective drainage throughout. The anticipated need for a storage facility in the pavement structure was not demonstrated by this field site. This field site displayed better than anticipated subsurface drainage performance. Further research is necessary to develop thresholds relating to pavement structure materials that use the storage capacity and to what extent and those that do not.

8.1 Recommendations to Practioners for the Use of Pervious Concrete Pavement

This research has shown promising results for the future use of pervious concrete pavement in Canada. Many lessons were learned at each field site and none of the field sites represent the ideal pervious concrete pavement or scenario. However, the successful aspects of the various field sites, when combined, suggest the ability to effectively plan, design and construct pervious concrete pavement in Canada. From the results generated during the monitoring period of the field sites and the laboratory testing it is anticipated that pervious concrete pavement in Canada can achieve a design life of 15 years, when used in a suitable application.

It is encouraged that the application of pervious concrete and the geometry of the site be considered during the planning. The objectives of the site should be identified and then an assessment should be carried out regarding the potential debris sources and commitment to maintenance by the owner. Pervious concrete is not encouraged for use in industrial applications that have substantial amounts of debris such as concrete plants or stormwater hot spots, such as gas stations, where there is high probably of a contaminant spills. It is important for this planning phase to be a joint effort with stormwater management representatives to ensure that the interests of both parties are achieved.

The pavement structure design should reflect the traffic loading and stormwater requirements. Traffic loading should be low speed, low volume. The anticipated traffic should be discussed to ensure that all anticipated vehicles are accounted for. Snow plows are considered occasional loading and it is anticipated that load damage will not occur from their use. Snow plows operated by private contractors are often comparable to personal vehicles and the weight of these would be less than that of a highway grade plow. The mix design would be handled by the concrete supplier.

Multiple methods are available for successful construction, as demonstrated in this research. The important considerations include the constructability of the pavement, ensuring some compaction of the surface and being prepared for curing. It is recommended that a test strip be placed ahead of time using the complete system that will be used for the project. This will provide an opportunity to evaluate the effectiveness of the methods as well as the hardened material that is produced.

Maintenance should be considered from a proactive perspective and it is anticipated to be one of the key factors in achieving performance of pervious concrete pavement in Canada. It is apparent though, that if the pavement is not planned, designed and constructed well then maintenance will not change the performance characteristics. At a minimum, maintenance to remove debris from the surface should be performed in the fall and spring seasons.

Owners should be educated about winter maintenance. Snow removal with conventional plowing equipment is deemed to be suitable for pervious concrete pavement. The ideal winter maintenance scenario would involve the removal of snow and allow any remaining ice or snow to melt and drain through the pervious concrete pavement. It is acknowledged that this is not always a suitable alternative. If necessary snow should be removed and limited sand should be applied to the surface. Salt or salt solutions should not be used. The sand will not lead to the pervious concrete becoming clogged, assuming that maintenance is performed in the spring season. The application of salt solution has been shown in this research to lead to failure of the pervious concrete pavement. Should the use of salt or salt solution be required, it should be limited.

8.2 Recommendations for Future Research

This research represents novel work in Canada related to pervious concrete pavement. Research has been carried out throughout the United States, including the northern United States regarding the use of pervious concrete pavement in cold weather climates. This Canadian research initiated, highlighted and brought attention to pervious concrete pavement for a variety of applications. The broad scope of this work created the opportunity to have pervious concrete field sites developed across the country with simultaneous laboratory work being carried out. Recommendations for future work related to the topic of pervious concrete pavement use in Canada are the following: mix design enhancements; relationship between construction method and hardened concrete void content; quality control and quality assurance test methods; routine permeability renewal maintenance program; hourly analysis of SWP data; and ongoing analysis of SWP data.

- Mix Design Enhancements - The pervious concrete pavement constructed in this research was some of the initial work related to pervious concrete pavement by most of the industry members involved. Moving forward with the use of pervious concrete pavement in Canada there are opportunities to improve on the mix designs that were used in this research. Although it was not presented in this research, some of the industry partners have already been working towards developing a revised pervious concrete pavement mix design, from the one that was used in the field site. In some cases this revised mix design is already available and is being used on projects. Similar to any paving material, opportunities for advancements will always be present.
- Relationship Between Construction Method and Hardened Concrete Void Content - An area of further research that would be useful to industry members and immediately applicable would be an assessment of the relationship between compaction during construction and void concrete of the hardened pervious concrete. The void content of a pervious concrete mix is certainly a result of the mix design. It is anticipated though that the amount of compaction during construction also attributes to the void content of the hardened concrete. Ideally a method of preparing samples in the laboratory would be related to various compaction efforts in construction. Prior to constructing a pervious concrete pavement an evaluation could be carried out in the laboratory to understand the effect that the compaction of the chosen construction method would have on the final product. This work would continue the work carried out by Rizvi in 2009 (Rizvi, Tighe, Henderson, & Norris, 2009).
- Quality Control and Quality Assurance Test Methods – Currently there are only a couple test methods available that are specifically for testing pervious concrete pavement. Municipalities require test methods that are reliable to include in specifications in order to use pervious concrete pavement. Quality Control (QC) and Quality Assurance (QA) testing is required by both the supplier and contractor and the owner to ensure that the pervious concrete pavement being supplied and constructed meets the requested characteristics. In order for broader inclusion of pervious concrete pavement it is necessary that conventional concrete test methods be evaluated and guidelines be developed for owner, suppliers and contractors, discussing the testing expectations.

- Routine Permeability Renewal Maintenance Program - In this research permeability renewal maintenance methods were generally performed on entire field sites when deemed necessary due to decreases in permeability or substantial quantities of debris on the surface. This approach is reactive in nature and unfortunately often the common approach taken to maintenance of pavements in general. Further research in the laboratory or at field sites would involve proactive permeability renewal maintenance. Methods such as sweeping may become effective when repeated regularly at a field site. This research would be immediately beneficial to both designers and planners as well as owners who are deciding whether to incorporate pervious concrete pavement but are questioning the permeability renewal maintenance requirements.
- Hourly Analysis of SWP Data - In this research the SWP data was analyzed daily. By averaging the data over a 24 hour period it was possible to compile and interpret the large volume of data. Moving forward, it would be beneficial to further evaluate the SWP results from the three field sites and looking at average over a smaller time span or hourly. In many examples drainage was noticed within the same day that the rain event occurred. By examining data on a smaller time scale it would be possible to assess within how many hours of a rain event the moisture was draining into a particular area. This research would strengthen the understanding of moisture movement through pervious concrete pavement structures and allow both the pavement and stormwater management industries the knowledge to make appropriate adjustments in the future.
- Ongoing Analysis of SWP Data - At this time SWP data from as early as fall 2008 was analyzed until the spring of 2011. The instrumentation included in the field sites is anticipated to have a serviceable life beyond this time period. Continued compilation and analysis of the SWP data would be very beneficial. Often limited data is available on a project after the initial few years of the project as the researchers have for various reasons advanced to other projects. Longer term interpretation of the SWP results would provide industry members with an understanding of the performance that can be expected from pervious concrete pavement structures. Analysis of results for several more years would provide information to assess if changes have occurred in the functionality of the pavement structures. In addition to ongoing analysis of the existing instrumentation, laboratory specimens could be developed which pushed the boundaries of the pervious

concrete pavement structure. The performance of subsurface drainage at Site 4 was better than the anticipated prediction. In the laboratory it would be useful to create full depth pavement structures using various materials for the base and subgrade and evaluate how the subsurface drainage characteristics change.

REFERENCES

AASHTO. (2008). Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes M145 - 91. In AASHTO, *AASHTO Standard Specifications* (pp. 145-1 to 145-8). Washington, DC: American Association of State Highway and Transportation Officials.

ACPA. (2006). *Stormwater Management with Pervious Concrete Pavement.* American Concrete Pavement Association Concrete Paving Technology.

ACPA. (2009). *Stormwater Management with Pervious Concrete Pavement.* Skokie, IL: American Concrete Pavement Association.

ACPA. (2012). *StreetPave*. Retrieved February 27, 2012, from American Concrete Pavement Association: http://www.acpa.org/StreetPave/

AECOM. (2010). *Niagara Peninsula Conservation Authority Stormwater Management Guidelines.* Kitchener, ON: AECOM.

Aquafor Beech Limited. (2007). *City of Hamilton Stormwater Master Plan - Class Environmental Assessment Report (City - Wide).* Brampton, ON: Aquafor Beech Limited.

ASTM. (2009). C 33/C 33M - 08 Standard Specification for Concrete Aggregates. In A. S. Materials, *Annual Book of ASTM Standards.* West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. (2010). *C1688/C1688M-10 Standard Test Method for Density and Void Content of Freshly Mixed Pervious Concrete.* West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. (2009). *C1701/C1701M Standard Test Method for Infiltration Rate of In Place Pervious Concrete.* West Conshohocken, PA: American Society for Testing and Materials International.

ASTM. (2008). *C666 Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing.* West Conshohocken, PN: American Society for Testing and Materials International.

ASTM. (2008). *C78-08 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading).* West Conshohocken, PA: American Society for Testing and Materials International.

Beatty, D. (2007). *Report - 2007-04-18 - Drowning Prevention in Young Children.* Perth County: Board of Health.

Bozic, L., Deong, Y., & Fesko, P. (2007). Stormwater Management Strategy for Calgary. *Innovative Stormwater Management Conference.* Vancouver, BC.

Brundtland, G. (1987). *Report of the World Commission on Environment and Development, Our Common Future.* Oxford University Press.

Bush, E., Cawley, B., Durham, S., MacKenzie, K., Rottman, J., & Thomas, D. (2008). *Specifier's Guide for Pervious Concrete Pavement Design Version 1.2.* Centennial, CO: Colorado Ready Mixed Concrete Association.

CBC. (2004, August 25). *Selling Canada's Water*. Retrieved February 18, 2012, from CBC News: http://www.cbc.ca/news/background/water/

Cement Association of Canada. (2011). *Building Sustainable Highways in Canada*. Retrieved February 18, 2012, from Cement Association of Canada: http://www.cement.ca/en/Highways/Building-Sustainable-Highways-in-Canada.html

Cement Association of Canada. (2011). *Pervious Concrete*. Retrieved February 27, 2012, from Cement Association of Canada: http://www.cement.ca/en/Parking-Lots/Pervious-Concrete.html

Chong, C. J., & Wrong, G. A. (1995). *Manual for Condition Rating of Rigid Pavements - Concrete Surface and Composite Distress Manifestations.* Downsview, ON: Research & Development Branch, MTO.

Chopra, M., Wanielista, M., Ballock, C., & Spence, J. (2007). *Construction and Maintenance Assessment of Pervious Concrete Pavements.* Orlando, FL: Stormwater Management Academy.

City of Toronto. (2003). *Wet - Weather Flow Management Master Plan.* Toronto, ON: City of Toronto.

City of Vancouver. (2009). *Green Design*. Retrieved May 2010, from City of Vancouver: vancouver.ca/engsvcs/streets/design/green.htm

ConcreteNetwork. (2010). *Where Pervious Concrete is Being Used*. Retrieved May 2010, from Concrete Network: www.concretenetwork.com/pervious/being_used.html

CRC. (2009). *Sustainable Infrastructure*. Retrieved April 2010, from Community Research Connections Research: http://www.crcresearch.org/sustainable-infrastructure/sustainable-infrastructure

CSA. (1996). *Technician Workbook For Training of Concrete Fiel Testing Technicians - Concrete Materials and Methods of Concrete Construction/Methods of Test for Concrete CSA A23.1/A23.2-94.* Rexdale, Ontario: Canadian Standards Association.

Cutler, H. E., Wang, K., Schaefer, V. R., & Kevern, J. T. (2010). Resistance of Portland Cement Pervious Concrete to Deicing Chemicals. *Transportation Research Board Annual Meeting.* Washington, DC.

Daley, R. (2008). *The Chicago Green Alley Handbook.*

Delatte, N., & Cleary, J. (2006). Developing a Structural Design Method for Pervious Concrete Pavement. *International Center for Aggregates Research (ICAR) 14th Annual Symposium.*

Delatte, N., Miller, D., & Mrkajic, A. (2007). *Portland Cement Pervious Concrete Pavement: Field Performance Investigation on Parking Lot and Roadway Pavements, Final Report.* Cleveland, Ohio: Cleveland State University.

Delatte, N., Mrkajic, A., & Miller, D. (2009). Field and Laboratory Evaluation of Pervious Concrete Pavements. *Transportation Research Record NO 2113* , 132 - 139.

Devereaux, S., & Lorant, F. I. (2006). *Stormwater Management Guidelines.* Halifax, NS: Dillion Consulting Ltd.

Engle-Yan, J., Kennedy, C., Saiz, S., & Pressnail, K. (2005). Toward Sustainable Neighbourhoods: The Need to Consider Infrastructure Interactions. *Canadian Journal of Civil Engineering* , 45 - 57.

Environment Canada. (2011). *Canadian Historical Weather Radar King City (near Toronto)*. Retrieved January 16, 2012, from National Climatic Data and Information Archive: http://www.climate.weatheroffice.gc.ca/radar/index_e.html?RadarSite=WKR&sYear=2011&sMonth=1&sDay=11&sHour=09&sMin=00&Duration=2&ImageType=Default

Environment Canada. (2011). *Daily Data Report Egbert CS Ontario*. Retrieved January 15, 2012, from National Climate Data and Information Archive: http://www.climate.weatheroffice.gc.ca/climateData/dailydata_e.html?timeframe=2&Prov=ONT&StationID=27604&dlyRange=2000-08-17|2012-02-24&Year=2011&Month=1&Day=01

Environment Canada. (2007). *FTP Directory Version 1.00.* Retrieved 2008, from Environment Canada Short Duration Rainfall Intensity Duration Frequency Tables and Graphs: ftp://arcdm20.tor.ec.gc.ca/pub/dist/IDF/archive/IDF_v1.00/IDF_Documentation_2007-12-11_e.pdf, ftp://arcdm20.tor.ec.gc.ca/pub/dist/IDF/archive/IDF_v1.00/

Environment Canada. (2010, September 28). *National Climate Data and Information Archive*. Retrieved March 4, 2010, from Climate Data Online: http://www.climate.weatheroffice.gc.ca/climateData/canada_e.html

EPA. (2008). *Effluent Limitations Guidelines and New Source Performance Standards for the Construction and Development Category.* Environmental Protection Agency.

Fassman, E. A., & Blackbourn, S. (2010). Urban Runoff Mitigation by a Permeable Pavement System over Impermeable Soils. *Journal of Hydrologic Engineering* , 475 - 485.

Gilson Company Inc. (2007). *AP-1B Asphalt Field Permeameter Operating Instructions.* Lewis Center, OH: Gilson Company Inc.

Greenberg, E. (2009). Sustainable Streets: Foundation for an Emerging Practice. *Transportation Research Board Annual Meetings.* Washington, DC: TRB.

Guillette, A. (2010, October 18). *Low Impact Development Technologies*. Retrieved February 27, 2012, from Whole Building Design Guide - National Institute of Building Sciences: www.wbdg.org/resources/lidtech.php

Guthrie, W. S., DeMille, C. B., & Eggett, D. L. (2010). Effects of Soil Clogging and Water Saturation on Freeze-Thaw Durability of Pervious Concrete. *Transportation Research Board Annual Meeting.* Washington, DC.

Hart, M. (1999). *Guide to Sustainable Community Indicators, 2nd Edition.*

Henderson, V., & Tighe, S. L. (2011). Pervious Concrete Pavement - A Sustainable Alternative. *Transportation Research Board Annual Meeting.* Washington, DC.

Henderson, V., & Tighe, S. L. (2010). Pervious Concrete Pavement Performance in Field Applications and Laboratory Testing. *Transportation Association of Canada Annual Conference* (p. Permeable Pavement Design and Technology). Halifax, NS: TAC.

Henderson, V., Banfield, N., Bergsma, J., & Malleck, B. (2008). *Pervious Pavement: The Green Solution.* Waterloo, ON: University of Waterloo CivE 401.

Henderson, V., Tighe, S. L., & Norris, J. (2009). Pervious Concrete Pavement Integrated Laboratory and Field Study. *Journal of the Transportation Research Record No. 2113* , 13-21.

Heron, M. P., Hoyert, D. L., Murphy, S. L., Xu, J. Q., Kochanek, K. D., & Tejada-Vera, B. (2009). *Deaths: Final Data for 2006, National Vital Statistics Reports.* Hyattsville, MD: National Center for Health Statistics.

Ho, E., & Gough, W. A. (2006). Freeze thaw cycles in Toronto. *Canada in a changing climate, Earth and Environmental Science* , Vol 83, Numbers 1 - 4.

Hood, M. J., Clausen, J. C., & Warner, G. S. (2007). Comparison of Stormwater Lag Times for Low Impact and Tranditional Residential Development. *Journal of the American Water Resources Association* , Vol. 43, No. 4, 1036 - 1046.

Huffman, D. (2007, May 10). *GREEN Transporation Infrastructure: Challenges to Access and Implementation.* Retrieved July 2009, from Committee on Science - U.S. House of Representatives: http://commdocs.house.gov/committees/science/hsy34909.000/hsy34909_0.htm

Instrotek®, Incorporated. (2011, February). Corelok (R) Operator's Guide. Raleigh, North Carolina, USA.

Kevern, J. (2010). Maintenance and Repair Options for Pervious Concrete. *Low Impact Development 2010: Redefining Water in the City* (pp. 113 - 125). San Francisco, CA: American Society of Civil Engineers.

Kevern, J., Wang, K., Suleiman, M. T., & Schaefer, V. (2006). Pervious Concrete Construction: Methods and Quality Control. *NRMCA Concrete Technology Forum: Focus on Pervious Concrete.* Nashville, TN: National Ready Mixed Concrete Association.

Kevern, J., Wang, K., Suleiman, M., & Schaefer, V. (2005). Mix Design Development for Pervious Concrete in Cold Climates. *Mid-Continent Transportation Research Symposium.* Ames, IA.

Kringos, N., Vassilikou, F., Kotsovos, M., & Scarpas, A. (2011). Application of Pervious Concrete for Sustainable Pavements: A Micr-Mechanical Investigation. *Transportation Research Board Annual Meetings.* Washington, DC.

MacKenzie, K. (2009, August 21). UDFCD Temporary Moratorium on Pervious Concrete Pavement Now Lifted. Denver, Colorado, USA.

Marsalek, J., Kok, S., & Colas, H. (2004). Managing Urban Wet-Weather Flows: On the Road to Sustainability. *Water Quality Research Journal of Canada* , vi - viii.

McCain, G. N., & Dewoolkar, M. M. (2010). Porous Concrete Pavements: Mechanical and Hydraulic Properties. *Transportation Research Board Annual Meetings.* Washington, DC.

McCain, G. N., & Dewoolkar, M. M. (2009). Strength and Permeability Characteristics of Porous Concrete Pavements. *Transportation Research Board Annual Meetings.* Washington, DC.

Mills, B. (2011, August 10). Adaptation and Impacts Research, Environment Canada. (V. Henderson, Interviewer)

MOE. (2003). *Understanding Stormwater Management: An Introduction to Stormwater Management Planning and Design.* Ministry of the Environment, Queen's Printer for Ontario.

Montes, F., Valavala, S., & Haselbach, L. (2009). *Test Method for Porosity Measurements of Portland Cement Pervious Concrete.* Mississauga, ON: RMCAO.

MTO MERO. (2012). *Greenpave Green Pavement Design Rating System Version 1.0.* January: Ontario Ministry of Transportation Materials Engineering and Research Office.

Neithalath, N., Bentz, D. P., & Sumanasooriya, M. S. (May 2010). Predicting the Permeability of Pervious Concrete. *Concrete International* , 35-40.

NRMCA. (2010). *Construction*. Retrieved July 13, 2011, from National Ready Mixed Concrete Association Pervious Concrete: http://www.perviouspavement.org/construction.htm

NRMCA. (2010). *Economic Benefits*. Retrieved July 13, 2011, from National Ready Mixed Concrete Association Pervious Concrete: http://www.perviouspavement.org/benefits_economic.htm

NRMCA. (2004). *Freeze-Thaw Resistance of Pervious Concrete.* Silver Spring, Maryland: National Ready Mixed Concrete Association.

NRMCA. (2010). *Hydrological Design Considerations*. Retrieved July 13, 2011, from National Ready Mixed Concrete Association Pervious Concrete: http://www.perviouspavement.org/hydrological_design.htm

NRMCA. (2009). *Inspection and Maintenance*. Retrieved 2009, from National Ready Mixed Concrete Association Pervious Concrete When it Rains ... it Drains.: http://www.perviouspavement.org/inspection_and_maintenance.htm

NRMCA. (2005). *Pervious Concrete When it Rains it Drains.* Retrieved February 27, 2012, from Green Concrete National Ready Mixed Concrete Association: www.greenconcrete.info

Ontario Professional Planners Institute. (2009). *Planning by Design: A Healthy Communities Handbook.* Toronto, Ontario: Ministry of Municipal Affairs and Ontario Professional Planners Institue.

OPSS. (2010). *Construction Specification for Pervious Concrete Pavement for Low-Volume Traffic Applications, OPSS 356.* Ontario Provincial Standard Specifications.

OPSS. (2003). *Material Specification for Aggregates - Base, Subbase, Select Subgrade and Backfill material, OPSS 1010.* Ontario Provincial Standard Specifications.

ORMCA. (2009). *Specifier's Guide for Pervious Concrete Pavement with Detention, PCP 2795.* Ohio Ready Mixed Concrete Association.

Paine, J. (1992). *Portland Cement Pervious Pavement Construction.* Retrieved July 2009, from EBuild: ftp://ftp.ebuild.com/woc/C920655.PDF

PCA. (2009, July 6). *Heavy Duty Pervious Concrete Breaks the Mold in New Jersey - Executive Report.* Retrieved July 6, 2009, from Portland Cement Associatrion: http://www.cement.org/exec2/07-06-09.htm

PCA. (2012). *Pervious Concrete*. Retrieved February 18, 2012, from Portland Cement Association: http://www.cement.org/tech/cct_con_design_pervious_hydro.asp

Penning, T. (2011, July 11). (V. Henderson, Interviewer)

PI. (2010, September). *Permeable Pavements*. Retrieved July 2011, from Pavement Interactive: http://pavementinteractive.org/index.php?title=Permeable_Pavements#Porous_Concrete

Pomerantz, M. (1999). *Benefits of Cooler Pavements.* Lawrence Berkely National Laboratory.

Pon, B. (2000). *Heat Island Group*. Retrieved May 2010, from High Temperatures: http://eetd.lbl.gov/heatisland/hightemps/

Rizvi, R., Tighe, S. L., Henderson, V., & Norris, J. (2009). Laboratory Sample Preparation Techniques for Pervious Concrete. *Transportation Research Board Annual Meeting.* Washington, DC.

RMCAO. (2009, May). *Pervious Concrete Specifier's Guidelines.* Retrieved February 19, 2012, from Ready Mixed Concrete Association of Ontario:

http://www.rmcao.org/sites/default/files/RMCAO%20Pervious%20Concrete%20Specifiers%20Guidelines,%20May%202009.pdf

Roa-Espinosa, A., Norman , J., Wilson, T. B., & Johnson, K. (2003). Predicting the Impact of Urban Development on Stream Temperature Using a Thermal Urban Runoff Model (TURM). *National Conference on Urban Stormwater: Enhancing Programs at the Local Level*, (pp. 369-389). Chicago, IL.

Rohne, R. J., & Izevbekhai, B. I. (2009). Early Performance of Pervious Concrete. *Transportation Research Board Annual Meetings.* Washington, DC.

Schaefer, V. R., Kevern, J. T., Izevbekhai, B., Wang, K., Culter, H. E., & Wiegand, P. (2010). Construction and Performance of the Pervious Concrete Overlay at MnRoad. *Transportation Research Board Annual Meeting.* Washington, DC.

Schaefer, V., Wang, K., Suleiman, M., & Kevern, J. (2006). *Mix Design Development for Pervious Concrete in Cold Weather Climates.* Ames, IA: Center for Transportation Research and Education, Iowa State University.

Schultz, R. B. (2004). *Koppen Climate Classification.* Retrieved February 27, 2012, from Geography 101: http://www.elmhurst.edu/~richs/EC/101/
http://www.elmhurst.edu/~richs/EC/101/KoppenClimateClassification.pdf

SKC. (2007). *Child and Youth: Unitentional Injury: 1994 - 2003 10 Years in Review.* Sick Kids Canada.

Statistics Canada. (2009, September 22). *Population Urban and Rural*. Retrieved October 2011, from Statistics Canada: http://www40.statcan.gc.ca/l01/cst01/demo62a-eng.htm

Statistics Canada. (2010, September 13). *Study: Freshwater Supply and Demand in Canada*. Retrieved February 18, 2012, from Statistics Canada: http://www.statcan.gc.ca/daily-quotidien/100913/dq100913b-eng.htm

Suleiman, M. T., Kevern, J., Schaefer, V. R., & Wang, K. (2008, November). *Effect of Compaction Energy on Pervious Concrete Properties.* Retrieved July 2011, from Pervious Concrete Research Compilation: Past, Present and Future: http://www.rmc-foundation.org/images/PCRC%20Files/Construction%20Techniques/Effect%20of%20Compaction%20Energy%20on%20Pervious%20Concrete%20Properties.pdf

Tennis, P. D., Leming, M. L., & Akers, D. J. (2004). *Pervious Concrete Pavements.* Skokie, IL: Portland Cement Association.

Toronto and Region Conservation. (2010, July 9). Permeable Pavements. Toronto, Ontario, Canada.

Tyner, J. S., Wright, W. C., & Dobbs, P. A. (2009). Increasing exfiltration from pervious concrete and temperature monitoring. *Journal of Environmental Management* , 2636-2641.

UDFCD. (2008). *BMP Monitoring Sites.* Retrieved August 2011, from Urban Drainage and Flood Control District: www.udfcd.org/sw_quality/SQMP.pdf

Vancura, M., Khazanovich, L., & MacDonald, K. (2010). *Performance of In-Service Pervious Concrete Pavements in Cold Weather.* Silver Spring, MD: Ready Mixed Concrete Research and Education Foundation .

Walker, J. (2009, December 21). Sustainable Stormwater Management. *Water Canada* .

Wang, K., Schaefer, V. R., Suleiman, M. T., & Kevern, J. T. (2006). *Mix Design Development for Pervious Concrete Pavement in Cold Weather Climates.* National Concrete Pavement Technology Center.

WERF. (2005). International Stormwater Best Management Practices Database - Water Environment Research Foundation. Alexandria, Virginia, USA.

Wu, H., Shu, X., Dong, Q., Shrum, E., Jared, D., & Wu, P. (2010). Laboratroy Evaluation of Latex-Modified Pervious Concrete. *Transportation Research Board Annual Meeting.* Washington, DC.

Yang, Z. (2011). Freezing-and-Thawing Durability of Pervious Concrete using Simulated Field Conditions. *ACI Materials Journal* , 187-195.

Yang, Z., Brown, H., & Cheney, A. (2008, June). *Influence of Moisture Conditions on Freeze and Thaw Durability of Portland.* Retrieved July 2008, from Pervious Concrete Research Compilation: http://www.rmc-foundation.org/images/PCRC%20Files/Durability%20&%20Maintenance/Influence%20of%20Moisture%20Conditions%20on%20Freeze%20and%20Thaw%20Durabiltity%20of%20Portland%20Cement%20Pervious%20Concrete.pdf

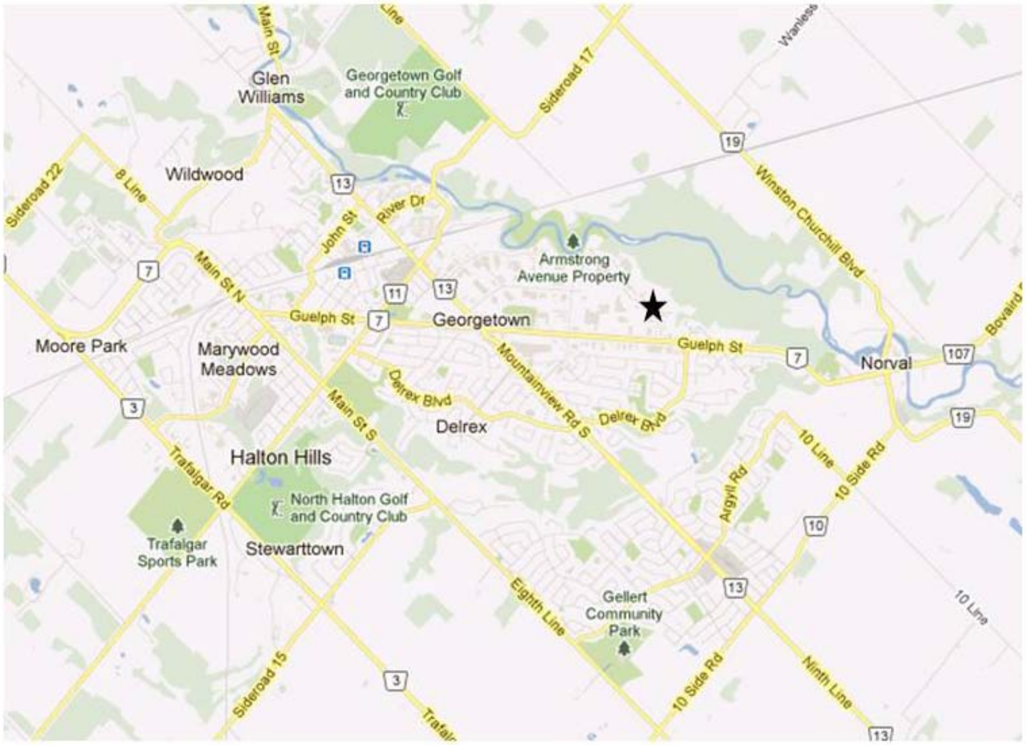
York Region. (2012). *Creating Complete Streets*. Retrieved February 18, 2012, from York Region: http://www.york.ca/Departments/Transportation+Services/Creating+Complete+Streets/Creating+Complete+Streets.htm

APPENDIX A - SURFACE DISTRESS EVALUATION FORM

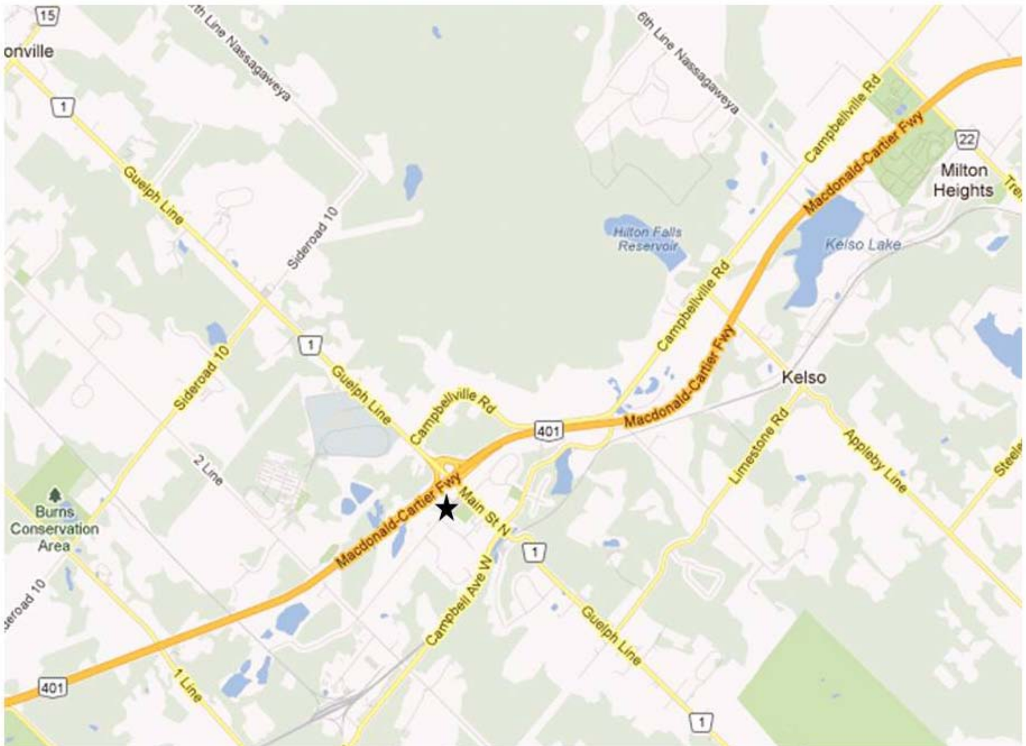
Pervious Concrete Surface Distress Evaluation Form										
Date:	______	Use:	______							
Location:	______									
Pavement Distress	Severity of Distress					Density of Distress				
	Very Slight	Slight	Moderate	Severe	Very Severe	<10%	10-20%	20-50%	50-80%	80-100%
Raveling and Aggregate Loss										
Surface Abrasion										
Polishing										
Scaling										
Joint Raveling										
Joint Separation										
Distortion										
Faulting										
Cracking										
Ponding/Clogging										
Maintenance										
Winter:										
Other:										
Comments:										
Evaluated by:										

APPENDIX B - FIELD SITE LOCATIONS

Site 1 – 301 Armstrong Ave., Georgetown, ON

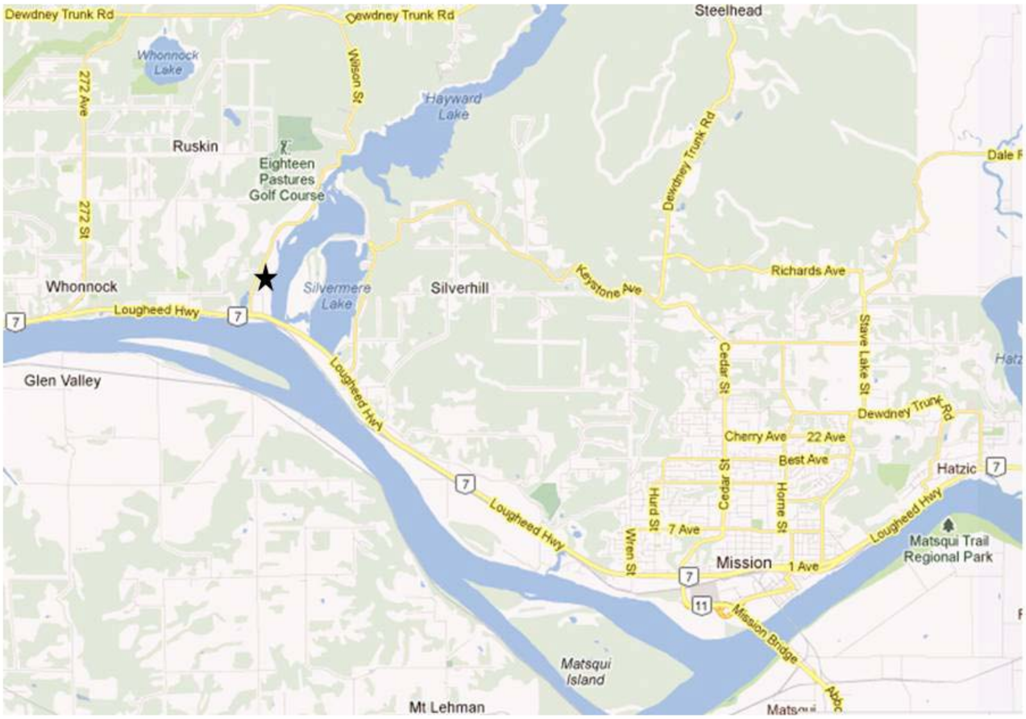


Site 2 – Hwy 401 and Guelph Line, southwest portion of Interchange 312, Campbellville, ON




No Compaction (Bidwell
Bridgedeck Paver)
Compaction (Razorback Paver)

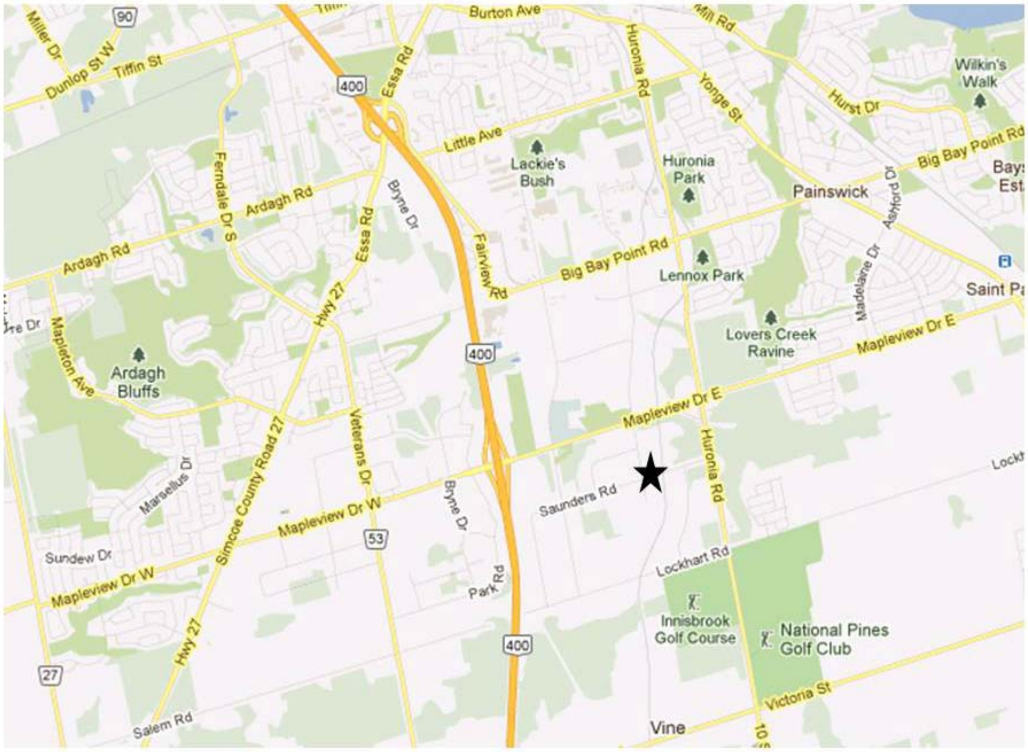
Site 3 – 9670 287 St, Maple Ridge, BC

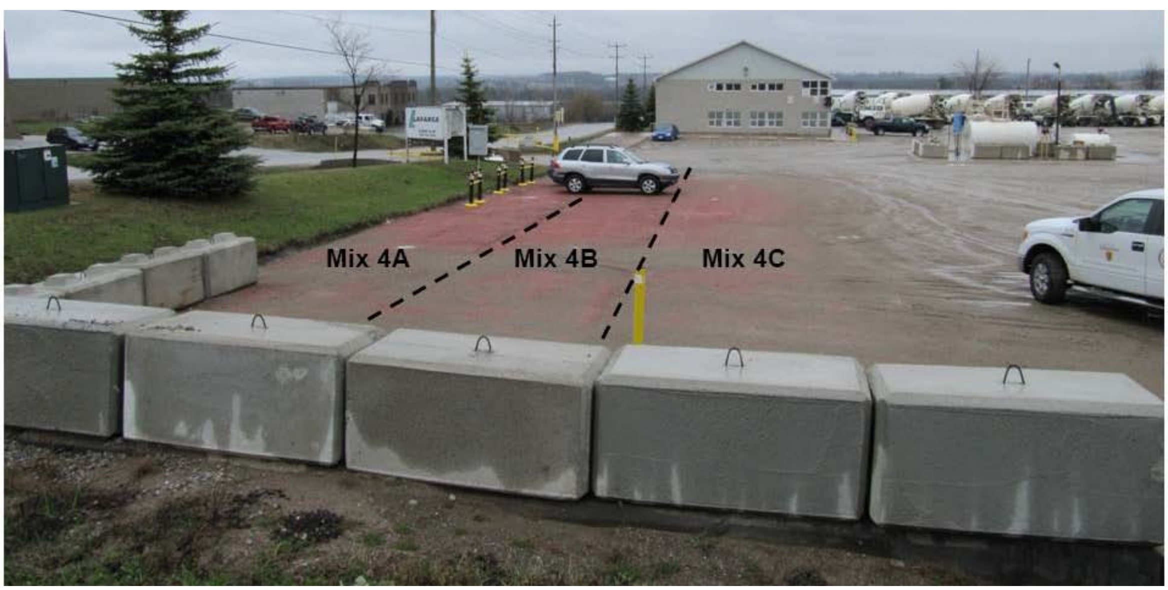



Conventional
Concrete
(Driveway -
Driving Traffic)
Conventional
Concrete
(Driveway -
Driving Traffic)
Mix 3A
Pervious Concrete
(Centre of Driveway -
Driving Traffic)


Conventional Concrete (Exit Driveway)
Mix 3B Pervious Concrete (Driving Traffic on Exit Driveway)
Conventional Concrete (Exit Driveway)

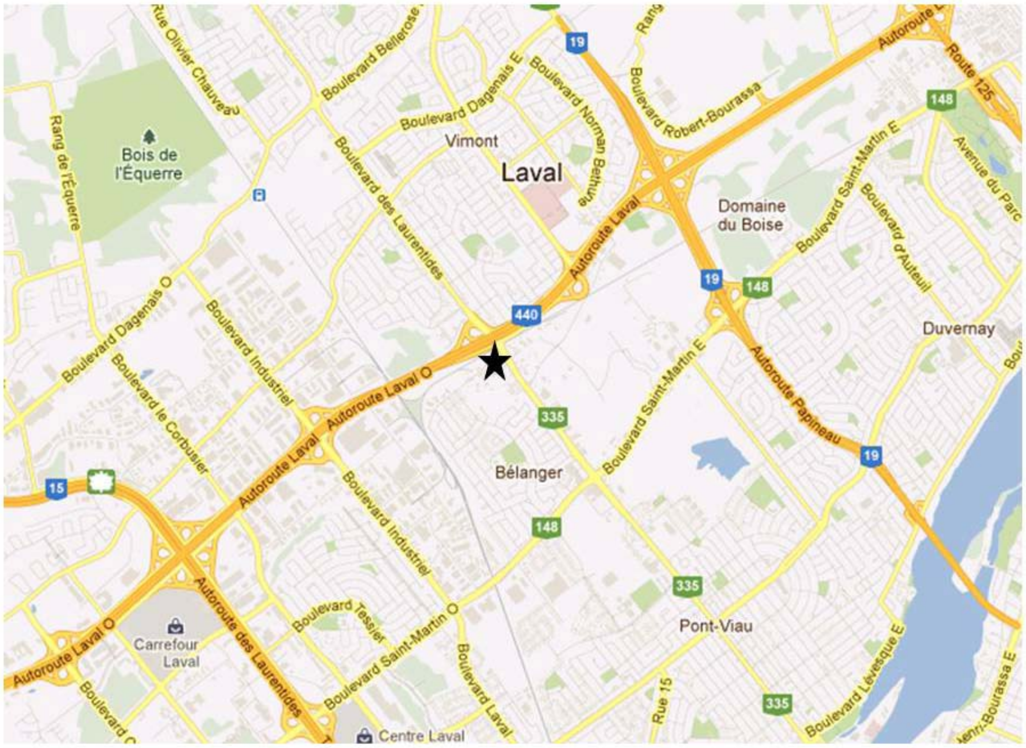
Site 4 – 275 Saunders Rd, Barrie, ON

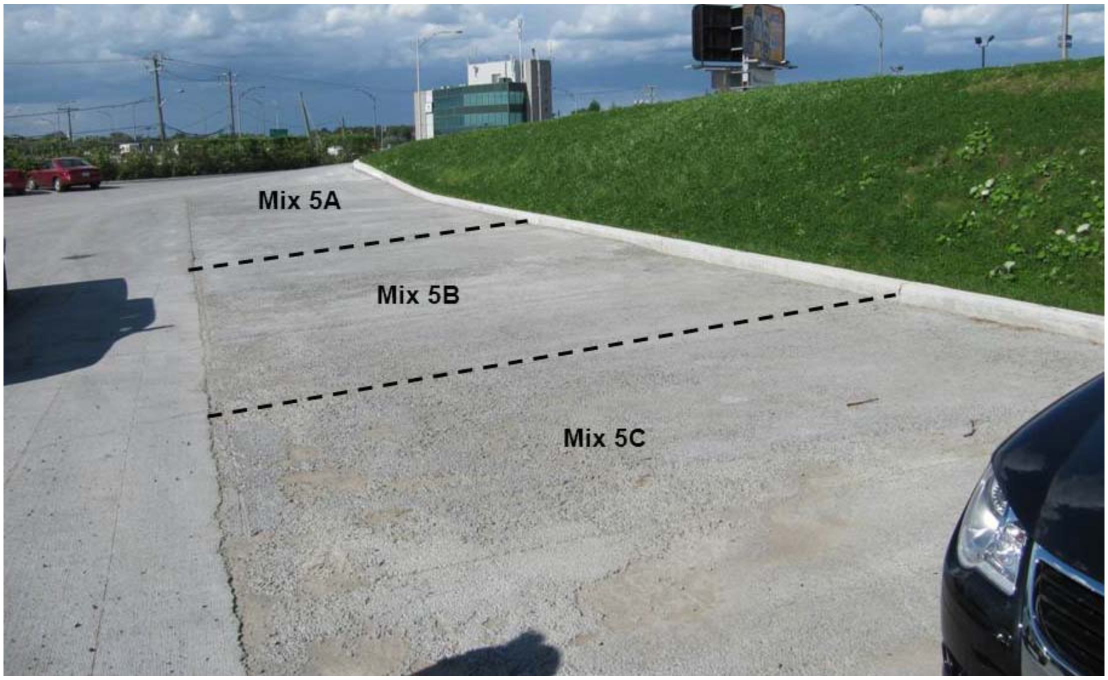





Sand/Gravel
(Driving Traffic)
Pervious Concrete
(Parked Traffic)

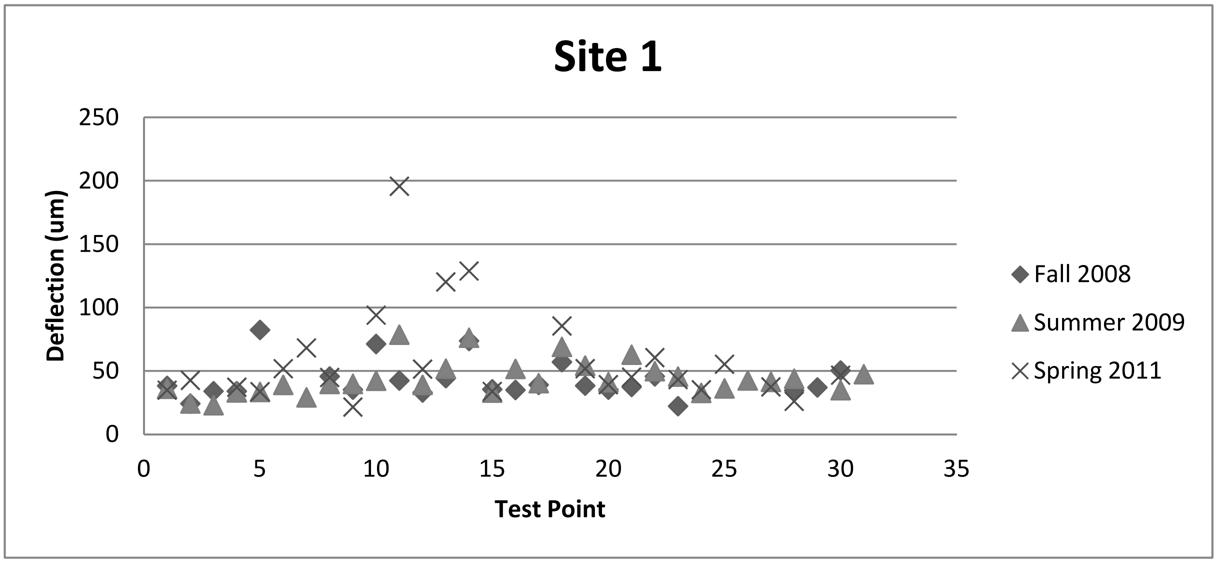
Site 5 – 1522 Blvd. des Laurentides, Laval, QC

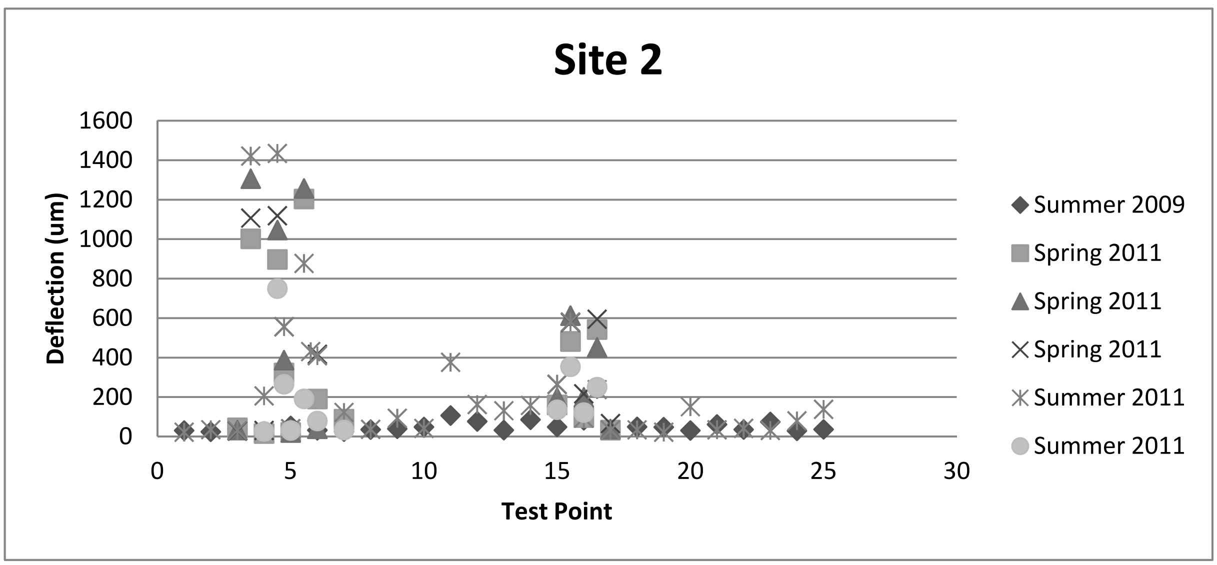


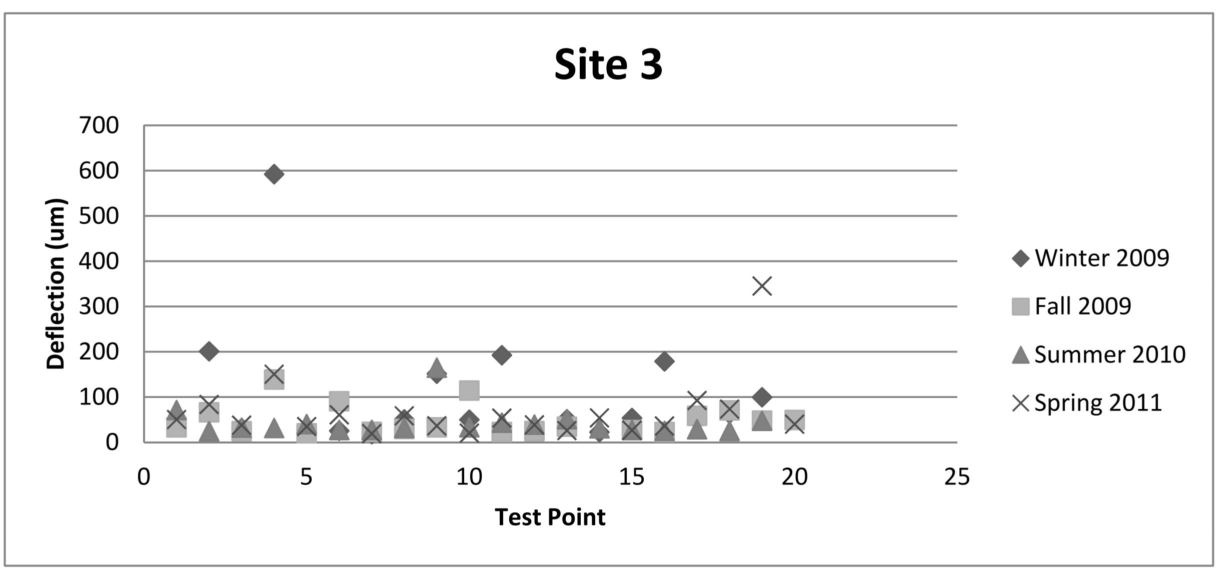

Mix 5A
Mix 5B
Mix 5C

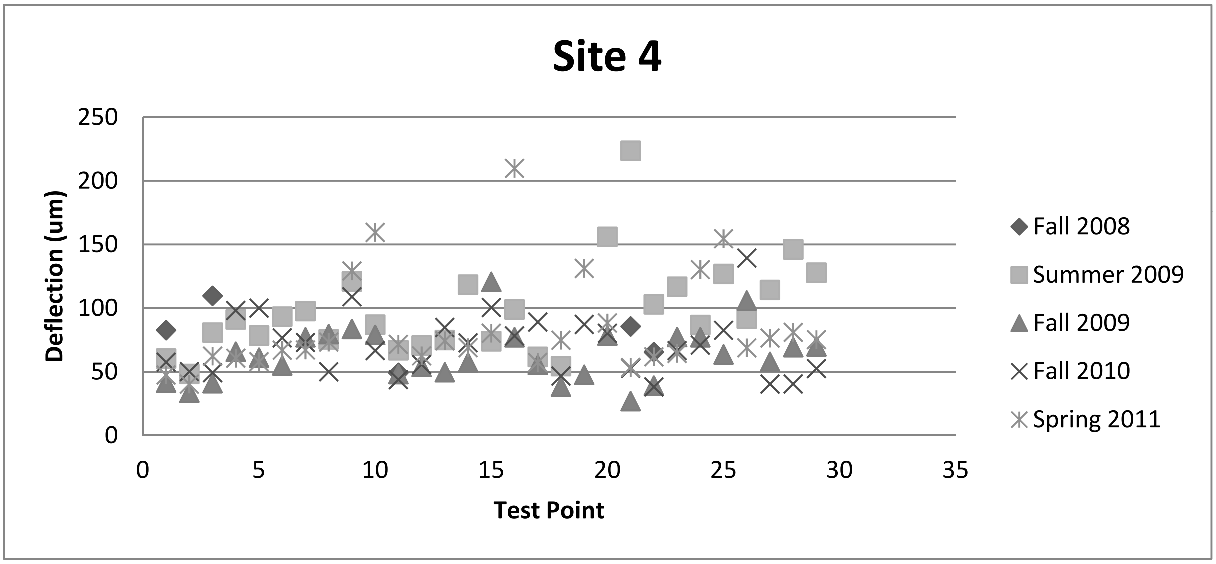

Pervious Concrete
(Parked Traffic)
Roller Compacted
Concrete
(Driving Traffic)

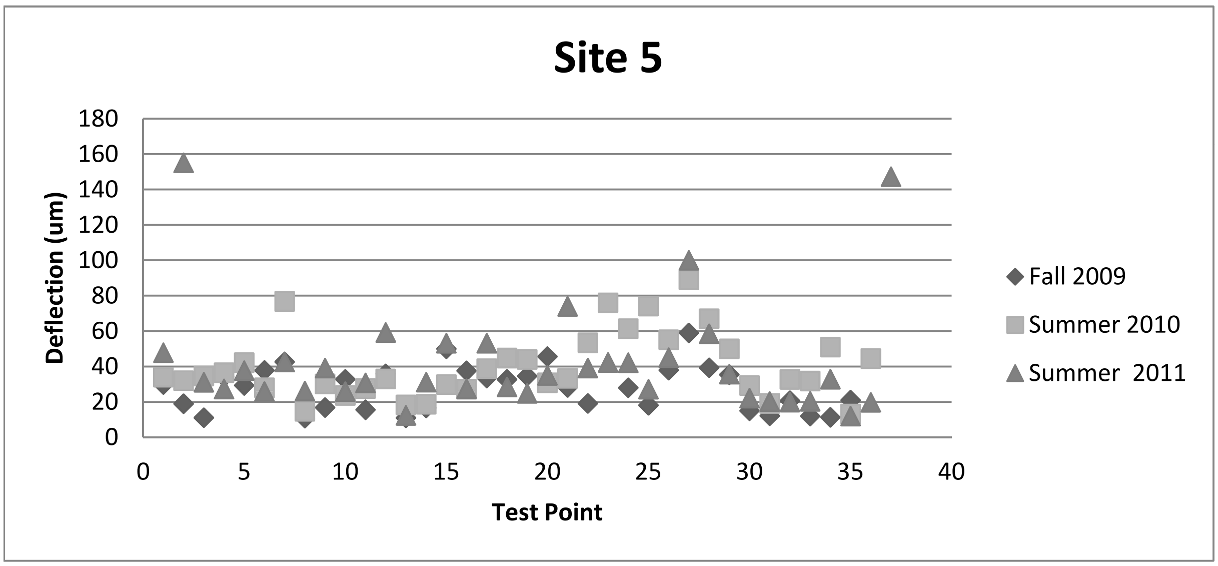
APPENDIX C - PORTABLE FALLING WEIGHT DEFLECTOMETER RESULTS


Site 1
250
200
150
100
50
0
Deflection (um)
0
5
10
15
20
25
30
35
Test Point
Fall 2008
Summer 2009
Spring 2011


Site 2
1600
1400
1200
1000
800
600
400
200
0
Deflection (um)
0
5
10
15
20
25
30
Test Point
Summer 2009
Spring 2011
Spring 2011
Spring 2011
Summer 2011
Summer 2011


Site 3
700
600
500
400
300
200
100
0
Deflection (um)
0
5
10
15
20
25
Test Point
Winter 2009
Fall 2009
Summer 2010
Spring 2011


Site 4
Deflection (um)
250
200
150
100
50
0
0
5
10
15
20
25
30
35
Test Point
Fall 2008
Summer 2009
Fall 2009
Fall 2010
Spring 2011


Site 5
Deflection (um)
180
160
140
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
40
Test Point
Fall 2009
Summer 2010
Summer 2011

ANOVA - Change in Structural Condition Over Time at a Specific Site

Site 1

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	4434.08	2	2217.04	3.569	3.117
Error	47206.5	76	621.138		*Reject*
Total	51640.6	78			

Site 2

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	29874.1	1	29874.1	6.705	4.113
Error	160403	36	4455.64		*Reject*
Total	190277	37			

Site 3

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	43620.5	3	14540.2	2.307	2.732
Error	453778	72	6302.47		*Fail to Reject*
Total	497398	75			

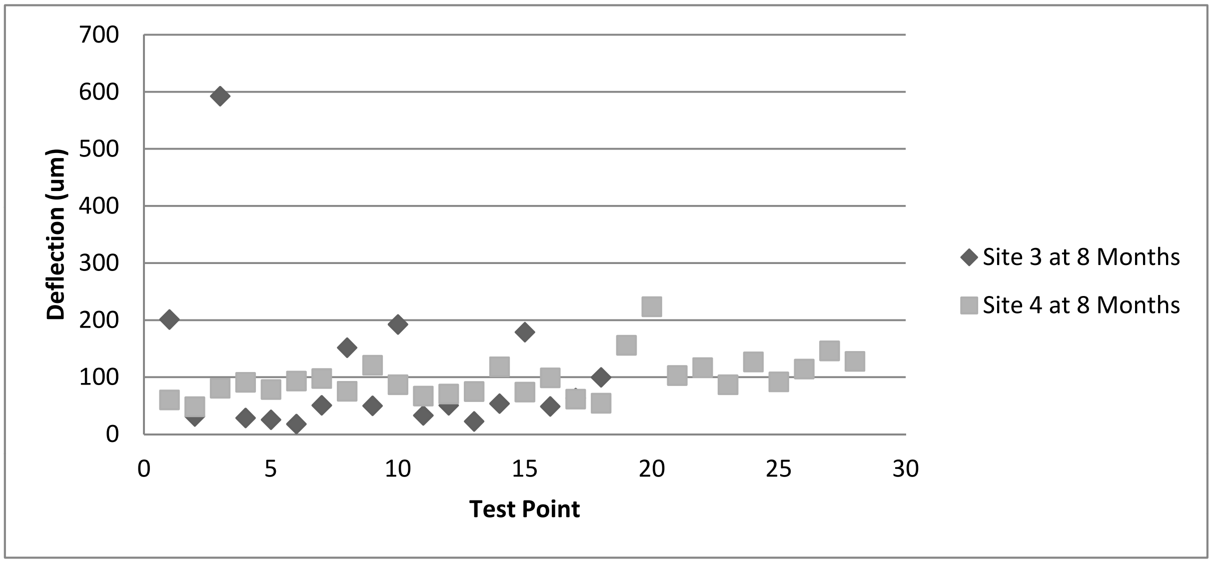
Site 4

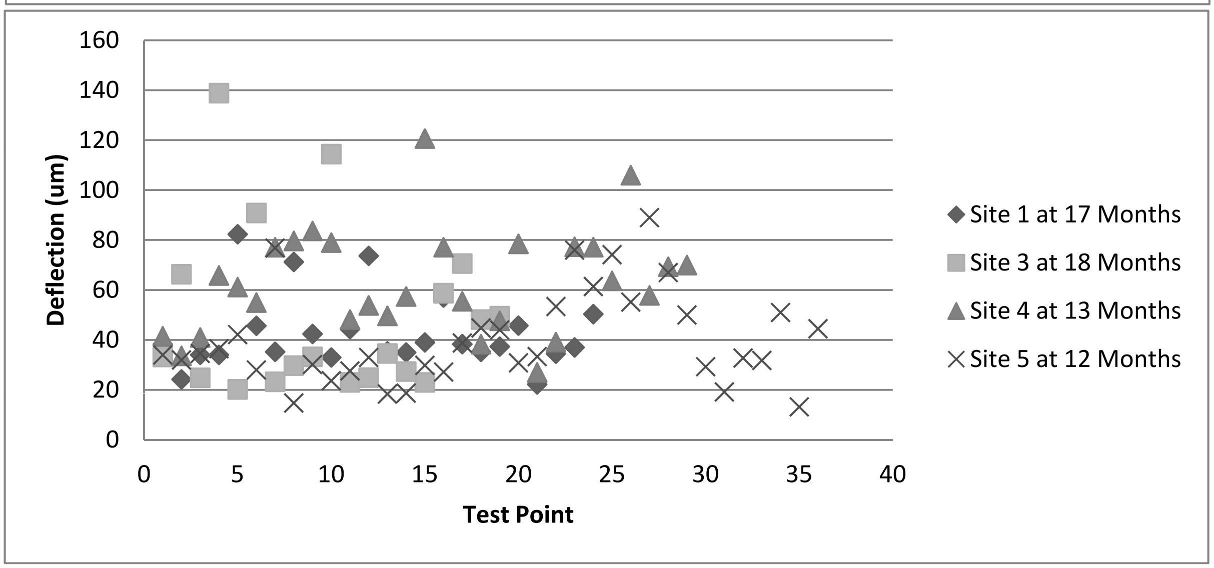
Source	SS	DF	MS	Fcalc	Fcrit
Treatment	20310	3	6769.99	6.988	2.696
Error	107545	111	968.871		*Reject*
Total	127855	114			

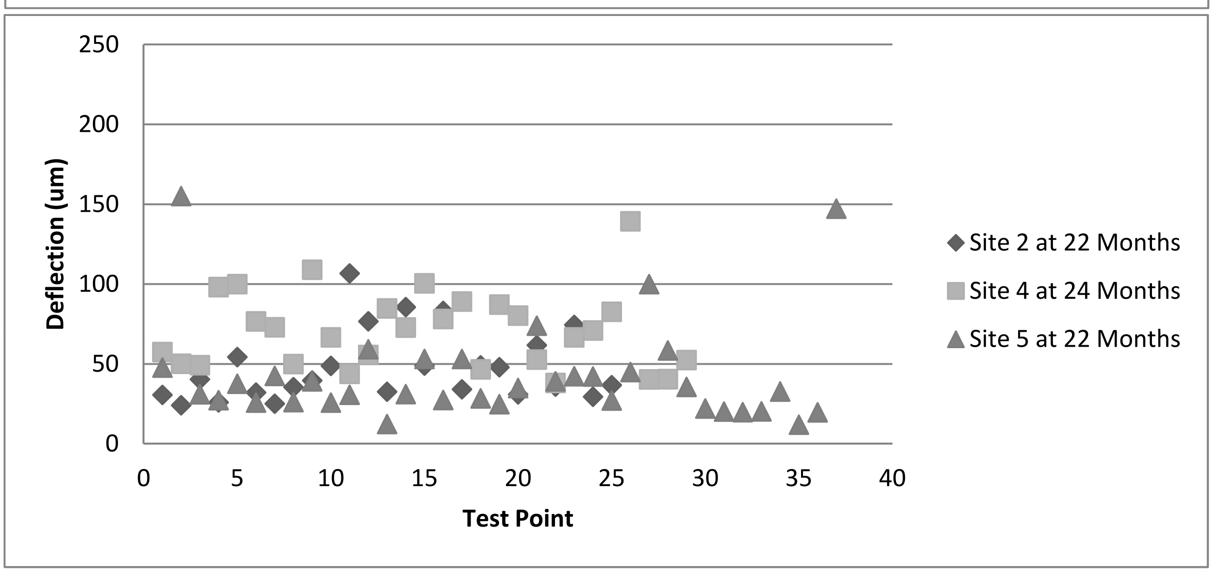
Site 5

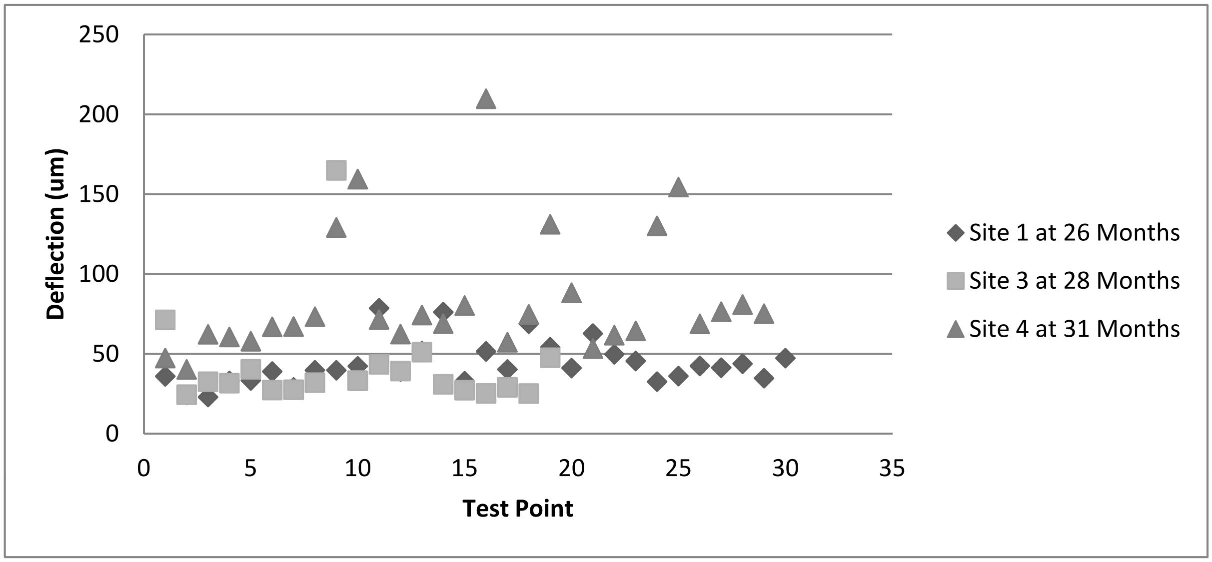
Source	SS	DF	MS	Fcalc	Fcrit
Treatment	3079.2	2	1539.6	3.406	3.111
Error	36166.3	80	452.079		*Reject*
Total	39245.5	82			

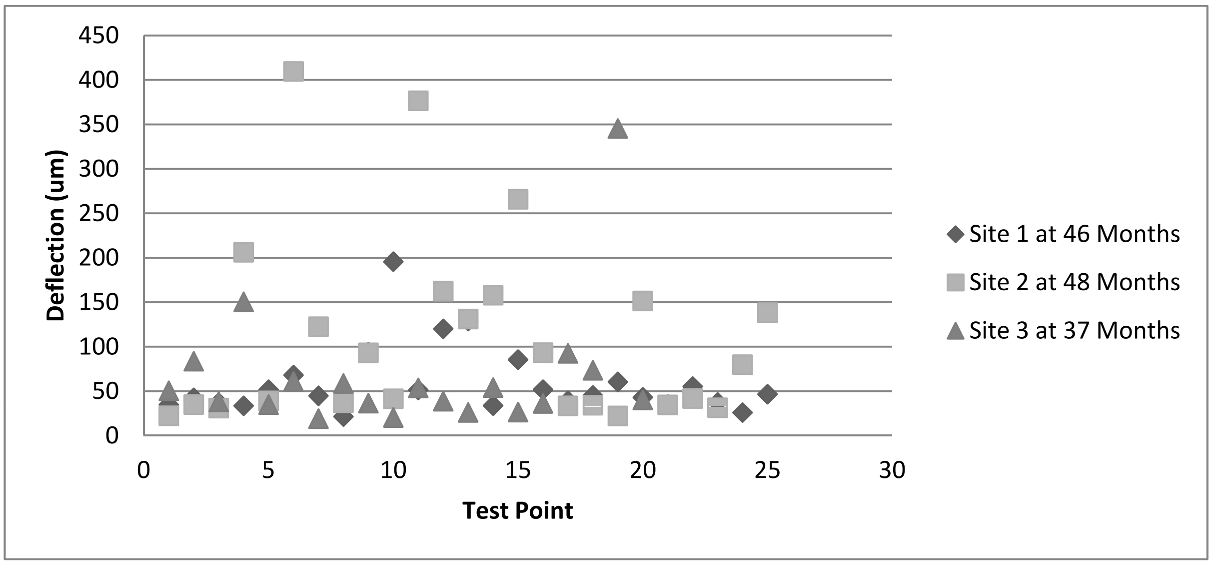
Comparison of Pavement Structures Based on Age








250
200
150
100
50
0
Deflection (um)
0
5
10
15
20
25
30
35
Test Point
Site 1 at 26 Months
Site 3 at 28 Months
Site 4 at 31 Months


450
400
350
300
250
200
150
100
50
0
Deflection (um)
0
5
10
15
20
25
30
Test Point
Site 1 at 46 Months
Site 2 at 48 Months
Site 3 at 37 Months

ANOVA - Comparison of Pavement Structures Based on Age

Age Group 1 (8 Months)

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	551.604	1	551.604	0.0690	3.209
Error	351959	44	7999.08		*Fail to Reject*
Total	352511	45			

Age Group 2 (12 to 18 Months)

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	9538.47	3	3179.49	6.572	2.696
Error	50312.8	104	483.777		*Reject*
Total	59851.3	107			

Age Group 3 (22 to 24 Months)

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	13990.3	2	6995.13	9.663	3.100
Error	63701.9	88	723.885		*Reject*
Total	77692.1	90			

Age Group 4 (26 to 31 Months)

Source	SS	DF	MS	F Calc	F Crit
Treatment	31070	2	15535	17.631	3.119
Error	66083.7	75	881.116		*Reject*
Total	97153.7	77			

Age Group 5 (37 to 48 Months)

Source	SS	DF	MS	F Calc	F Crit
Treatment	39104.5	2	19552.2	3.191	3.134
Error	410478	67	6126.54		*Reject*
Total	449583	69			

APPENDIX D - STATISTICAL ANALAYSIS OF CHAPTER 6.0

ANOVA - Permeability Rate of Cores At Various Ages

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.17629	3	0.05876499	0.611	3.197
Error	1.63547	17	0.09620436		*Fail to Reject*
Total	1.81177	20			

ANOVA - Permeability Rate of Each Field Site Section Over Time

Site 1

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	242.405	6	40.4009096	15.947	2.191
Error	471.216	186	2.53341735		*Reject*
Total	713.621	192			

Site 2

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	430.7	7	61.5285976	45.408	2.103
Error	253.39	187	1.3550274		*Reject*
Total	684.09	194			

Site 3A

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	8.39422	8	1.04927771	4.877	2.032
Error	23.6668	110	0.21515268		*Reject*
Total	32.061	118			

Site 3B

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.38294	7	0.0547061	1.606	2.359
Error	0.95384	28	0.03406583		*Fail to Reject*
Total	1.33679	35			

Site 4A

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	206.486	7	29.4980518	23.467	2.145
Error	86.7332	69	1.25700269		*Reject*
Total	293.22	76			

Site 4B

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	143.613	7	20.5161078	29.355	2.15
Error	46.8258	67	0.69889235		*Reject*
Total	190.439	74			

Site 4C

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	100.889	7	14.412767	34.891	2.172
Error	23.9587	58	0.41308091		*Reject*
Total	124.848	65			

Site 5A

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	1.29182	5	0.25836312	4.293	2.29
Error	2.94876	49	0.06017887		*Reject*
Total	4.24058	54			

Site 5B

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	1.53368	5	0.30673662	7.015	2.413
Error	2.05517	47	0.04372697		*Reject*
Total	3.58885	52			

Site 5C

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	1.78446	4	0.44611521	4.575	2.579
Error	4.38755	45	0.09750119		*Reject*
Total	6.17201	49			

Site 5D

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.07234	2	0.03617176	0.982	3.682
Error	0.55234	15	0.03682234		*Fail to Reject*
Total	0.62468	17			

ANOVA - Sections 4A, 4B and 4C at 20 Months of Age

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.40353	2	0.20176611	13.000	3.369
Error	2.13471	26	0.01552047		*Reject*
Total	2.53825	28			

ANOVA - Sections 5A, 5B, 5C and 5D at 12 Months of Age

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.01988	3	0.00662633	10.000	2.922
Error	0.13027	30	0.00066263		*Reject*
Total	0.15015	33			

ANOVA - Sections 5A, 5B and 5C at 12 Months of Age

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.00037	2	0.00018726	12.500	3.354
Error	0.00092	25	0.0000150		*Reject*
Total	0.0013	27			

ANOVA - Comparison of Field Site Permeability Based on Age

Age Group 1 (6 to 8 Months) Site 3 Excluded

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	173.074	5	34.6147104	13.342	2.317
Error	217.924	84	2.59433074		*Reject*
Total	390.997	89			

Age Group 1 (6 to 8 Months) Site 3 Included

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	354.958	6	59.1596812	27.961	2.191
Error	217.929	103	2.11581207		*Reject*
Total	572.887	109			

Age Group 2 (11 to 13 Months)

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	5.92486	8	0.74060775	2.977	2.032
Error	27.6167	111	0.24879891		*Reject*
Total	33.5415	119			

Age Group 3 (20 to 24 Months) Site 3 Excluded

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	10.9301	11	0.99364781	0.865	2.032
Error	163.146	142	1.1489145		*Fail to Reject*
Total	174.076	153			

Age Group 3 (20 to 24 Months) Site 3 Included

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	12.0185	12	1.001542		1.85
Error	163.146	156	1.045807		*o Reject*
Total	175.164	168			

Age Group 4 (31 Months) Site 3 Exclud

Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.16785	4	0.0419618	1.904	2.525
Error	1.32261	60	0.02204345		*Fail to Reject*
Total	1.49045	64			

Age Group 4 (28 to 31 Months) Site 3 Included

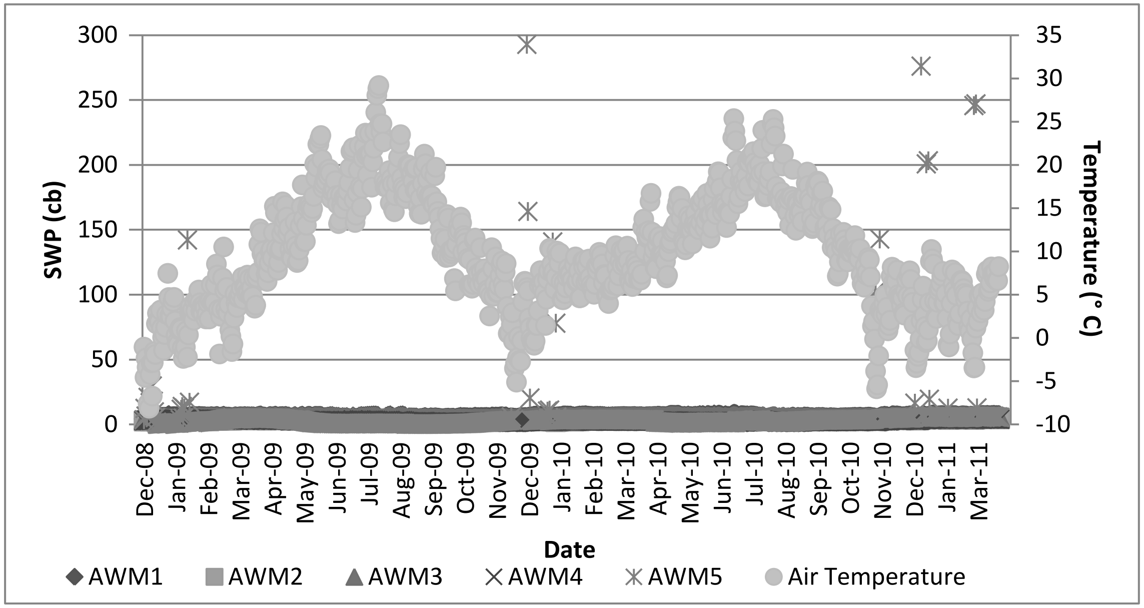
Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.3085	5	0.0617	3.686	
Error	1.3226	79	0.0167		
Total	1.6311	84			

Age Group 5 (45 to 60 Months)

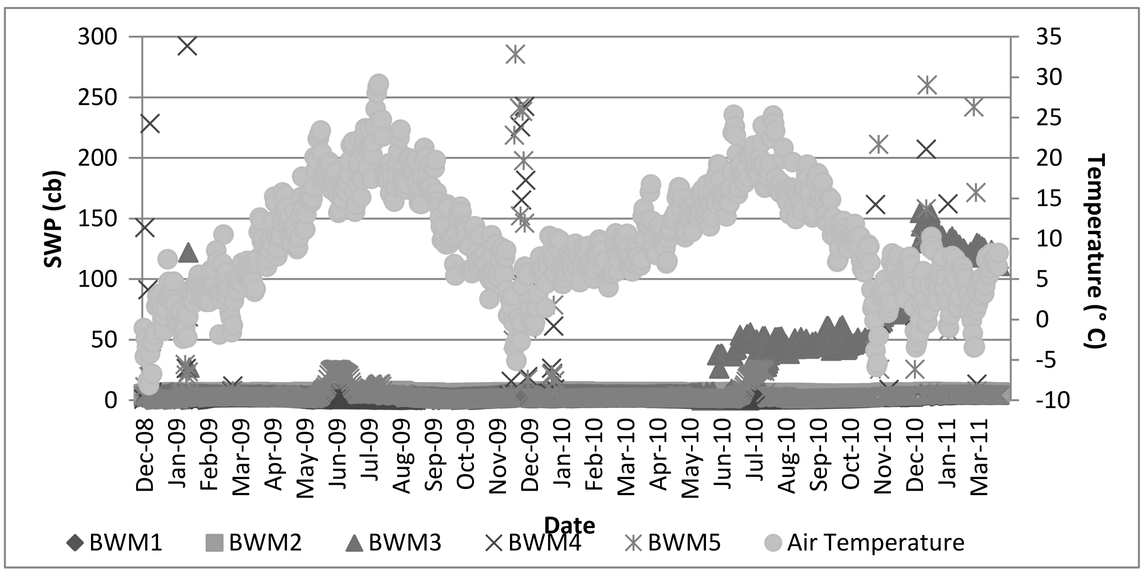
Source	SS	DF	MS	Fcalc	Fcrit
Treatment	0.16521	4	0.04130149	9.764	2.54
Error	0.23266	55	0.00423012		*Reject*
Total	0.39786	59			

APPENDIX E - CHAPTER 7 RAIN EVENTS

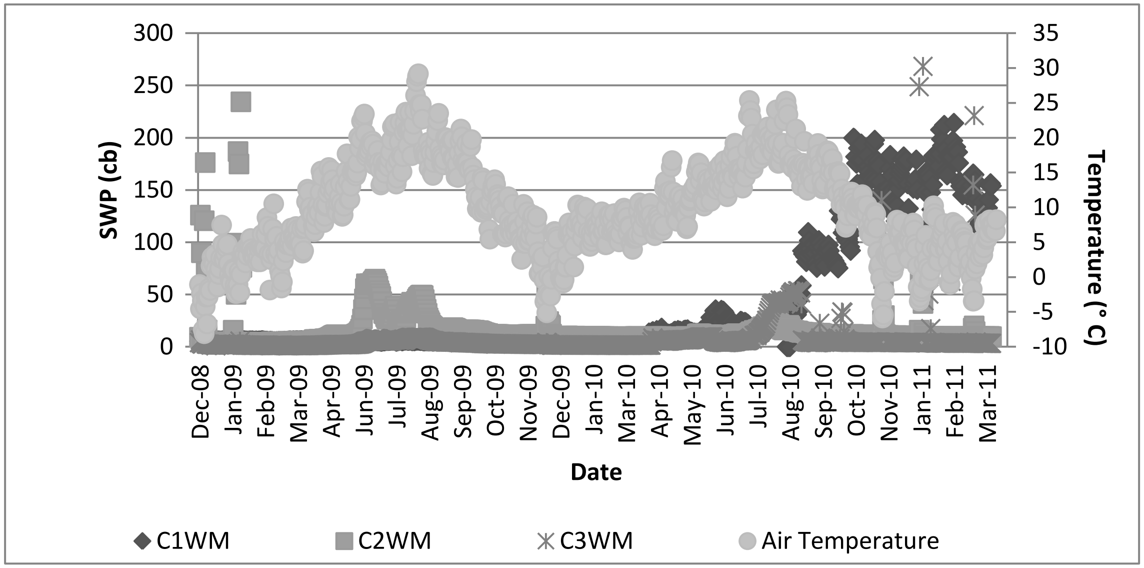
Site 3



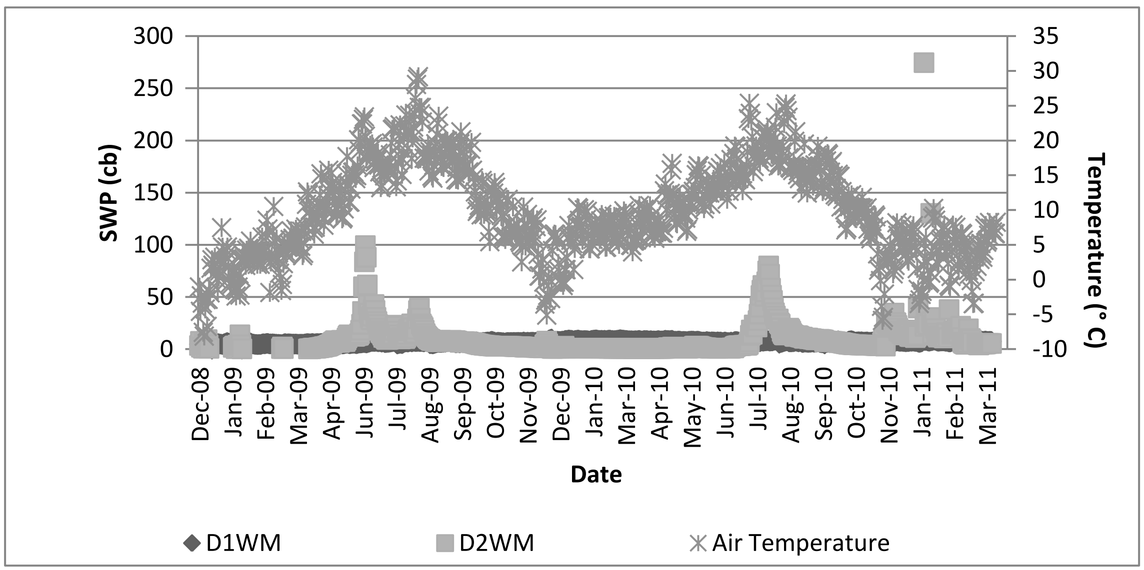
SWP (cb)
Temperature (° C)
Date
300
250
200
150
100
50
0
35
30
25
20
15
10
5
0
-5
-10
Dec-08 Jan-09 Feb-09 Mar-09 Apr-09 May-09 Jun-09 Jul-09 Aug-09 Sep-09 Oct-09 Nov-09 Dec-09 Jan-10 Feb-10 Mar-10 Apr-10 May-10 Jun-10 Jul-10 Aug-10 Sep-10 Oct-10 Nov-10 Dec-10 Jan-11 Mar-11
AWM1 AWM2 AWM3 AWM4 AWM5 Air Temperature



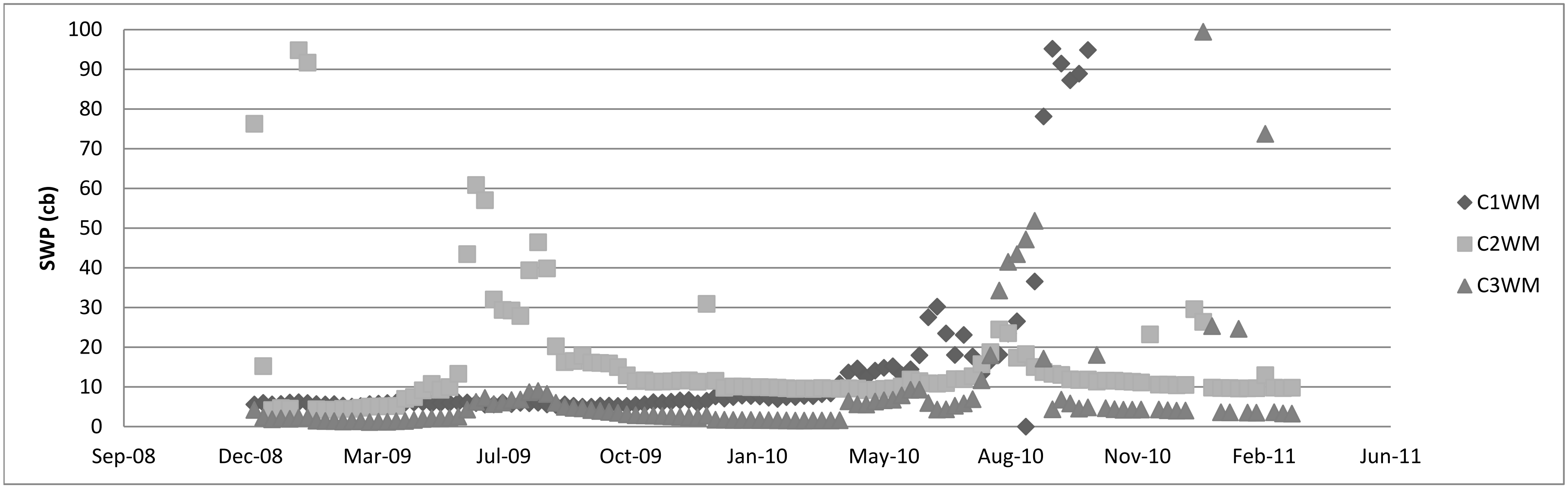
SWP (cb)
Temperature (° C)
Date
300
250
200
150
100
50
0
35
30
25
20
15
10
5
0
-5
-10
Dec-08 Jan-09 Feb-09 Mar-09 Apr-09 May-09 Jun-09 Jul-09 Aug-09 Sep-09 Oct-09 Nov-09 Dec-09 Jan-10 Feb-10 Mar-10 Apr-10 May-10 Jun-10 Jul-10 Aug-10 Sep-10 Oct-10 Nov-10 Dec-10 Jan-11 Mar-11
BWM1 BWM2 BWM3 BWM4 BWM5 Air Temperature

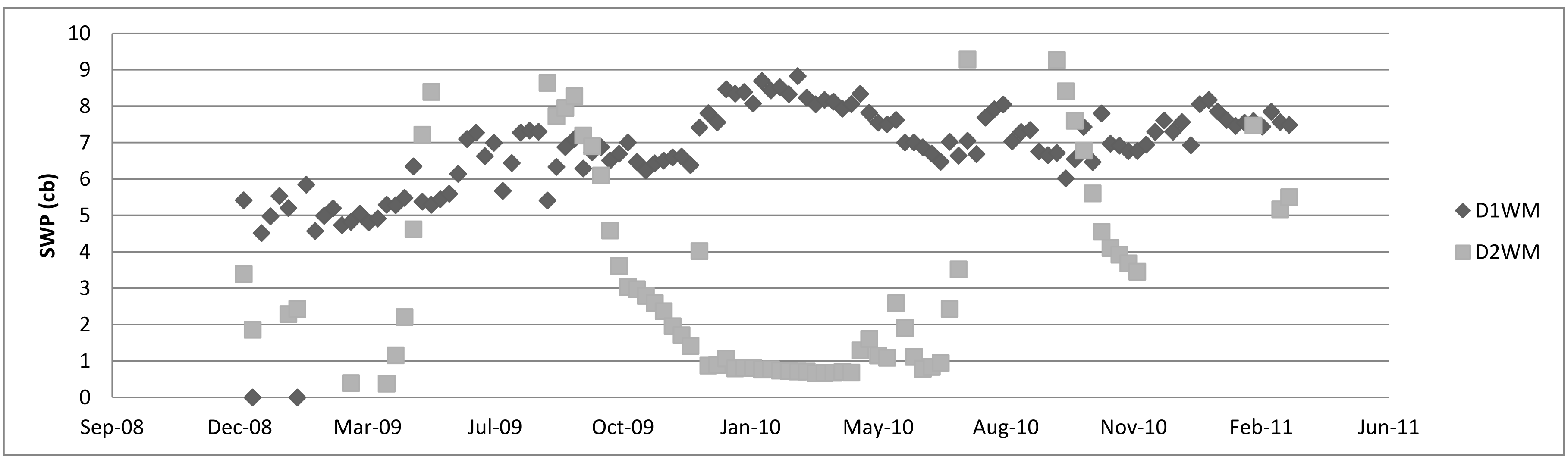


SWP (cb)
Temperature (° C)
Date
Dec-08 Jan-09 Feb-09 Mar-09 Apr-09 Jun-09 Jul-09 Aug-09 Sep-09 Oct-09 Nov-09 Dec-09 Jan-10 Mar-10 Apr-10 May-10 Jun-10 Jul-10 Aug-10 Sep-10 Oct-10 Nov-10 Jan-11 Feb-11 Mar-11
C1WM
C2WM
C3WM
Air Temperature



SWP (cb)
Temperature (° C)
Date
Dec-08 Jan-09 Feb-09 Mar-09 Apr-09 Jun-09 Jul-09 Aug-09 Sep-09 Oct-09 Nov-09 Dec-09 Jan-10 Mar-10 Apr-10 May-10 Jun-10 Jul-10 Aug-10 Sep-10 Oct-10 Nov-10 Jan-11 Feb-11 Mar-11
D1WM
D2WM
Air Temperature


SWP (cb)
100
90
80
70
60
50
40
30
20
10
0
Sep-08
Dec-08
Mar-09
Jul-09
Oct-09
Jan-10
May-10
Aug-10
Nov-10
Feb-11
Jun-11
C1WM
C2WM
C3WM


SWP (cb)
10
9
8
7
6
5
4
3
2
1
0
Sep-08
Dec-08
Mar-09
Jul-09
Oct-09
Jan-10
May-10
Aug-10
Nov-10
Feb-11
Jun-11
D1WM
D2WM

Site 3

Winter 2009	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
0	1/6/2009	65.25	5.6	3.7	4.3	3.8	0.7	5.1	5.2	4.6		0.8	5.6	4.7	2.0	4.8	
1	1/7/2009	49.25	6.5	3.9	4.4	4.0	0.9	5.3	5.9	5.1		0.9	5.6	4.9	2.1	4.7	
2	1/8/2009	22	6.9	3.9	4.0	4.0	1.0	5.4	6.3	5.4		1.0	5.8	4.8	2.1	4.5	
3	1/9/2009	0.5	6.1	3.8	3.6	4.0	1.0	5.1	6.2	5.4		0.9	5.7	4.5	2.1	5.0	
4	1/10/2009	32.25	5.9	3.7	3.6	3.8	1.1	4.9	6.3	5.9		0.9	5.9	4.7	2.1	5.4	
5	1/11/2009	6.25	6.0	3.8	3.7	4.0	1.1	5.3	6.4	6.6		1.0	5.9	4.6	2.1	5.6	
6	1/12/2009	8	6.1	2.5	3.8	4.1	1.2	5.4	6.5	5.8		1.0	6.0	4.4	2.0	6.2	
7	1/13/2009	0	6.1	2.1	3.9	4.0	1.1	5.4	6.5	5.8		1.0	6.2	4.5	2.0	6.3	
8	1/14/2009	0	6.1	2.1	3.9	4.0	1.1	5.4	6.5	5.7		1.0	6.1	4.6	1.9	5.9	
9	1/15/2009	0.25	6.0	2.1	3.8	3.9	1.1	5.3	6.4	5.6		1.0	6.1	4.6	1.9	5.6	
10	1/16/2009	0	6.0	1.9	4.2	3.9	1.2	5.3	6.3	5.5		1.0	6.1	4.5	1.9	4.9	
11	1/17/2009	0	5.9	1.9	4.1	3.8	1.2	5.2	6.2	5.4		1.0	6.1	4.5	2.0	4.9	
Spring 2009	**Date**	**Rain**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**AWM5**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**BWM5**	**C1WM**	**C2WM**	**C3WM**	**D1WM**	**D2WM**
	3/26/2009	0	5.9	3.8	5.3	3.6	4.9	5.2	6.1	4.5	2.2	2.9	5.9	5.1	1.3	4.8	
0	3/27/2009	8.5	5.9	3.8	5.3	3.5	4.9	5.2	6.2	4.5	2.2	2.8	5.8	5.1	1.2	4.8	
1	3/28/2009	7.75	5.7	3.6	5.1	3.4	5.0	5.1	6.0	4.4	2.3	2.9	5.7	5.1	1.3	4.7	
2	3/29/2009	2	6.0	3.8	5.4	3.7	5.0	5.3	6.3	4.6	2.3	2.9	5.9	5.1	1.3	4.8	
3	3/30/2009	6.5	5.8	3.7	5.2	3.5	4.8	5.1	6.0	4.4	2.2	2.8	5.8	4.9	1.2	4.8	
4	3/31/2009	17.5	5.9	3.7	5.3	3.6	5.0	5.2	6.2	4.6	2.2	2.9	5.9	5.2	1.2	4.8	
5	4/1/2009	13	5.7	3.6	5.0	3.4	4.9	5.0	5.9	4.3	2.2	2.8	5.7	5.1	1.2	4.7	
6	4/2/2009	16.5	5.5	3.4	4.9	3.3	5.0	4.8	5.8	4.3	2.3	2.9	5.7	5.3	1.3	4.7	
7	4/3/2009	0.25	5.9	3.7	5.4	3.7	5.0	5.2	6.2	4.7	2.2	2.9	6.0	5.4	1.3	5.0	
8	4/4/2009	0	6.1	3.9	5.5	3.7	4.9	5.5	6.4	4.7	2.2	2.8	6.1	5.1	1.2	5.1	
9	4/5/2009	0	6.4	4.1	5.6	3.8	4.8	5.7	6.5	4.8	2.2	2.8	6.4	5.1	1.3	5.3	
10	4/6/2009	0	6.6	4.2	5.8	3.8	4.7	5.8	6.7	4.8	2.1	2.7	6.4	4.9	1.3	5.4	
11	4/7/2009	0	6.6	4.3	5.7	3.7	4.7	5.8	6.6	4.8	2.2	2.7	6.3	4.8	1.3	5.3	
12	4/8/2009	0	6.1	3.9	5.3	3.3	4.6	5.3	6.1	4.4	2.3	2.6	6.1	4.9	1.3	5.2	

Summer 2009	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
	8/7/2009	0	5.2	3.2	4.5	2.3	0.7	4.7	6.1	3.6	0.2	3.7	5.6	35.9	7.9	7.5	12.9
	8/8/2009	0	6.4	4.2	5.6	3.0	0.9	5.8	7.4	4.4	0.3	4.0	5.5	32.0	7.5	7.3	11.8
	8/9/2009	0	5.6	3.5	4.8	2.4	0.7	5.0	6.4	3.7	0.2	3.9	5.8	27.6	7.1	7.6	10.7
0	8/10/2009	24.25	6.7	4.4	5.7	3.0	0.9	6.0	7.6	4.4	0.3	4.4	5.5	25.1	6.6	6.2	9.8
1	8/11/2009	14	5.7	3.6	4.8	2.3	0.7	5.0	6.5	3.6	0.2	4.4	5.6	23.1	6.5	5.2	9.5
2	8/12/2009	0	5.8	3.6	4.8	2.4	0.8	5.1	6.5	3.6	0.2	4.4	5.6	20.7	6.2	5.4	8.6
3	8/13/2009	1.75	5.1	3.1	4.3	2.0	0.7	4.4	5.8	3.1	0.1	4.1	5.4	19.8	5.9	5.2	8.2
4	8/14/2009	1.5	5.0	3.0	4.3	2.0	0.7	4.3	5.8	3.2	0.1	3.9	5.2	18.9	5.8	5.0	8.4
5	8/15/2009	0	4.7	2.9	4.2	2.0	0.7	4.2	5.7	3.1	0.1	3.7	5.3	17.5	5.7	5.3	8.3
6	8/16/2009	0	5.5	3.5	4.9	2.4	0.9	5.0	6.6	3.6	0.2	3.9	5.6	16.4	5.4	5.6	7.7
7	8/17/2009	0	5.2	3.3	4.5	2.2	0.8	4.6	6.3	3.4	0.2	3.7	5.7	15.4	5.1	5.9	7.1
8	8/18/2009	0	6.2	4.1	5.5	2.8	0.9	5.5	7.4	4.1	0.2	4.0	5.8	15.4	4.9	6.2	7.1

Fall 2009	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
	11/14/2009	0	5.2	2.7	3.9	1.8	1.9	4.8	5.7	3.4	1.4	1.5	6.0	11.2	2.6	6.4	2.3
0	11/15/2009	25	5.3	2.9	4.1	1.9	2.0	5.0	6.2	3.6	1.4	1.6	6.4	11.4	2.5	6.6	2.2
1	11/16/2009	30.5	5.8	3.2	4.4	2.1	2.0	5.4	6.5	3.8	1.3	1.6	6.7	11.4	2.5	6.9	2.1
2	11/17/2009	8.5	5.8	3.3	4.4	2.1	2.0	5.3	6.4	3.8	1.3	1.6	6.6	11.3	2.5	6.6	2.0
3	11/18/2009	15.75	5.6	3.1	4.2	1.9	2.1	5.1	6.2	3.7	1.3	1.6	6.6	11.5	2.5	6.4	2.0
4	11/19/2009	23.25	5.6	3.1	4.2	1.9	2.2	5.1	6.2	3.9	1.3	1.7	6.8	11.7	2.5	6.5	2.0
5	11/20/2009	12.5	5.8	3.3	4.4	2.0	2.3	5.3	6.4	4.0	1.3	1.7	6.8	11.8	2.5	6.7	1.9
6	11/21/2009	12.75	5.7	3.2	4.3	1.9	2.3	5.3	6.3	4.0	1.4	1.7	6.5	11.7	2.4	6.6	1.9
7	11/22/2009	25	5.7	3.1	4.2	2.0	2.4	5.2	6.3	4.1	1.4	1.8	6.3	11.9	2.4	6.5	1.9
8	11/23/2009	6	5.7	3.2	4.3	2.0	2.5	5.3	6.4	4.2	1.4	1.8	6.6	11.7	2.4	6.6	1.8
9	11/24/2009	3.25	5.9	3.3	4.3	2.0	2.6	5.4	6.4	4.4	1.5	1.8	6.7	11.7	2.4	6.7	1.8
10	11/25/2009	33.75	5.9	3.4	4.5	2.0	2.6	5.5	6.5	4.5	1.5	1.8	6.8	11.7	2.4	6.7	1.8
11	11/26/2009	22.5	6.0	3.4	4.5	2.1	2.6	5.5	6.5	4.6	1.5	1.8	6.8	11.5	2.3	6.7	1.7
12	11/27/2009	0	5.8	3.3	4.4	2.0	2.7	5.4	6.3	4.5	1.6	1.8	6.5	11.6	2.4	6.5	1.7
13	11/28/2009	22.75	5.7	3.2	4.3	1.9	2.8	5.2	6.3	4.5	1.7	1.9	6.7	11.6	2.4	6.6	1.7
14	11/29/2009	12.25	5.8	3.3	4.4	2.0	2.8	5.4	6.4	4.7	1.7	1.9	6.9	11.7	2.4	6.6	1.6
15	11/30/2009	4.75	5.9	3.3	4.5	2.1	2.8	5.5	6.4	4.6	1.7	1.9	6.7	11.6	2.4	6.7	1.6
16	12/1/2009	0	5.5	3.0	4.2	1.8	2.9	5.1	6.0	4.4	1.7	1.9	6.1	11.4	2.4	6.3	1.6
17	12/2/2009	0.25	5.0	2.6	3.8	1.6	3.0	4.7	5.5	4.1	1.9	2.0	5.6	11.1	2.3	6.1	1.5
18	12/3/2009	0	4.9	2.7	3.9	1.7	3.1	4.6	5.6	4.2	3.9	2.1	5.6	11.2	2.4	6.2	1.5
19	12/4/2009	0	5.1	2.8	4.1	1.8	3.1	4.9	5.8	4.3	2.0	2.1	5.8	11.4	2.3	6.4	1.3
20	12/5/2009	0.25	5.0	2.7	4.0	1.7	5.8	4.7	5.7	4.2	15.7	5.3	5.6	11.2	2.3	6.3	1.3
21	12/6/2009	0	5.1	2.9	4.2	1.8	3.1	4.8	5.8	4.5	2.1	2.1	5.6	11.3	2.3	6.6	1.2
22	12/7/2009	0	5.0	2.8	4.0	1.7	81.5	4.7	5.7	4.4	92.7	49.6	5.5	11.1	2.2	6.5	1.4
23	12/8/2009	0	4.8	2.7	3.9	1.6	352.0	4.5	5.5	4.2	329.0	218.8	5.4	12.2	2.4	6.4	4.1
24	12/9/2009	0	4.8	2.7	3.9	1.7	478.5	4.5	5.6	4.2	399.7	285.6	5.9	19.8	2.5	6.7	5.0
25	12/10/2009	0	4.7	2.7	3.8	1.6	816.2	4.4	5.5	4.0	619.7	445.0	6.7	57.3	4.1	6.9	7.7
26	12/11/2009	0	4.8	2.8	4.0	1.7	851.5	4.6	5.6	4.0	579.8	426.8	7.1	70.7	3.6	7.6	5.2
27	12/12/2009	0	5.0	3.0	4.2	1.8	834.4	4.7	5.8	4.3	527.8	393.0	7.6	24.1	2.0	8.7	1.8
28	12/13/2009	0	5.2	3.2	4.3	1.9	572.1	4.9	6.0	5.0	337.9	241.3	7.5	21.2	2.5	9.0	2.8

Fall 2009	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
	10/20/2009	0	5.3	3.0	4.2	1.8	0.8	4.6	5.8	2.8	0.2	0.9	5.6	11.8	2.9	6.6	3.0
0	10/21/2009	8.25	5.3	2.9	4.1	1.7	0.8	4.7	5.9	2.7	0.2	1.0	5.5	11.8	2.8	6.7	3.0
1	10/22/2009	0.25	5.2	2.7	4.0	1.6	0.8	4.6	5.7	2.8	0.2	1.0	5.6	11.7	2.9	6.6	3.0
2	10/23/2009	26.25	5.2	2.5	4.0	1.6	0.8	4.6	5.9	2.8	0.3	1.0	5.7	11.7	2.9	6.5	2.9
3	10/24/2009	0.25	5.2	2.4	4.0	1.6	0.9	4.6	5.8	2.8	0.3	1.0	5.8	11.7	2.8	6.4	3.0
4	10/25/2009	13	5.0	2.4	3.8	1.5	0.9	4.4	5.5	2.7	0.3	1.0	5.6	11.2	2.8	6.2	2.9
5	10/26/2009	37.25	5.1	2.6	4.0	1.7	0.9	4.8	6.1	2.9	0.3	1.1	6.2	11.2	2.7	6.3	2.8
6	10/27/2009	0	5.1	2.6	3.9	1.6	0.9	4.6	5.8	2.9	0.4	1.1	6.0	11.1	2.8	5.8	2.9
7	10/28/2009	1.75	5.0	2.6	3.9	1.6	1.0	4.5	5.7	2.9	0.4	1.1	5.8	11.2	2.8	6.0	2.9
8	10/29/2009	13.5	5.1	2.6	3.9	1.7	1.0	4.6	5.9	3.0	0.5	1.1	6.3	11.4	2.8	6.2	2.8
9	10/30/2009	16	5.4	2.8	4.2	1.9	1.1	4.9	6.1	3.2	0.6	1.2	6.5	11.6	2.8	6.5	2.8
10	10/31/2009	6	5.7	3.0	4.3	1.9	1.1	5.0	6.2	3.2	0.7	1.2	6.3	11.4	2.8	6.5	2.7
11	11/1/2009	0	5.4	2.9	4.2	1.8	1.2	4.8	6.0	3.1	0.7	1.2	6.2	11.4	2.7	6.3	2.7
12	11/2/2009	4.5	5.3	2.8	4.0	1.7	1.3	4.8	5.9	3.1	0.8	1.1	6.0	11.3	2.7	6.4	2.7
13	11/3/2009	1.5	5.4	2.8	4.1	1.8	1.4	4.8	6.0	3.2	0.8	1.2	6.1	11.4	2.7	6.5	2.7
14	11/4/2009	0	5.2	2.8	4.0	1.7	1.4	4.7	5.8	3.1	0.9	1.2	5.9	11.2	2.7	6.3	2.6
Winter 2010	**Date**	**Rain**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**AWM5**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**BWM5**	**C1WM**	**C2WM**	**C3WM**	**D1WM**	**D2WM**
	1/7/2010	0	5.7	3.5	4.7	2.3	3.7	5.1	6.4	4.8	1.5	1.8	7.4	10.0	1.6	7.9	0.8
0	1/8/2010	22.75	5.7	3.4	4.6	2.4	3.6	5.1	6.4	4.8	1.5	1.8	7.8	10.2	1.7	8.3	0.8
1	1/9/2010	4	5.5	3.4	4.6	2.3	10.3	5.0	6.3	4.7	5.7	3.8	7.9	10.2	1.7	8.5	0.8
2	1/10/2010	0.25	5.6	3.4	4.6	2.4	4.8	5.0	6.3	4.7	1.5	1.9	8.1	10.2	1.7	8.5	0.8
3	1/11/2010	43.75	5.7	3.4	4.6	2.4	11.1	5.0	6.3	4.7	6.1	4.0	7.5	10.0	1.7	8.7	0.8
4	1/12/2010	22.75	5.5	3.0	4.7	2.3	4.5	5.0	6.3	4.7	1.8	1.7	7.6	10.0	1.7	8.7	0.8
5	1/13/2010	4.75	5.5	2.7	4.5	2.3	109.8	4.9	6.2	4.6	26.5	21.2	7.6	9.9	1.7	8.4	0.8
6	1/14/2010	21.25	5.4	2.6	4.5	2.3	140.6	4.8	6.1	4.6	22.2	20.6	7.9	9.9	1.7	8.3	0.8
7	1/15/2010	29.5	5.5	2.6	4.6	2.3	329.8	4.9	6.2	4.6	61.2	78.6	8.1	10.0	1.7	8.2	0.8
8	1/16/2010	0	5.7	2.4	4.8	2.5	128.9	5.1	6.4	4.9	9.4	16.1	7.5	9.9	1.7	8.1	0.8
9	1/17/2010	7.25	5.7	2.1	4.9	2.5	78.0	5.2	6.5	4.9	3.8	4.3	7.8	10.1	1.7	8.2	0.8
10	1/18/2010	1.5	5.9	2.4	4.9	2.5	5.8	5.2	6.6	4.9	1.5	1.8	7.9	10.1	1.7	8.3	0.9
11	1/19/2010	0	5.9	2.4	4.9	2.5	3.7	5.2	6.6	4.9	1.5	1.8	7.7	10.0	1.7	8.3	0.8
12	1/20/2010	0	6.0	2.4	5.0	2.6	3.7	5.4	6.9	5.0	1.5	1.7	7.7	10.0	1.7	8.2	0.8
13	1/21/2010	0	6.1	2.6	5.1	2.6	3.7	5.4	6.9	5.0	1.5	1.7	7.7	10.0	1.7	8.2	0.8

Spring/ Summer 2010	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
	5/25/2010	0	6.6	3.8	5.6	2.2	1.3	5.5	6.3	5.0	1.2	2.0	12.9	11.6	9.3	7.3	1.2
0	5/26/2010	4.5	6.1	3.5	5.2	2.0	1.2	5.0	5.9	4.9	1.1	1.9	11.6	11.4	9.3	7.0	1.1
1	5/27/2010	7	6.1	3.5	5.3	2.1	1.2	5.0	6.1	5.3	1.1	1.9	13.3	11.5	9.3	7.0	1.1
2	5/28/2010	18	6.1	3.5	5.1	2.0	1.2	5.0	6.0	5.5	1.1	1.8	21.3	11.4	9.3	6.9	1.0
3	5/29/2010	18.5	5.8	3.3	4.9	1.8	1.2	5.2	5.8	5.6	1.0	1.7	28.1	11.3	9.3	6.6	0.9
4	5/30/2010	0	6.2	3.5	5.3	2.2	1.4	5.6	6.1	5.9	1.0	1.8	25.4	11.5	9.4	7.1	1.0
5	5/31/2010	22	6.2	3.6	5.3	2.1	1.4	5.5	6.1	5.6	0.9	1.8	25.4	11.1	9.4	7.0	0.9
6	6/1/2010	2.25	6.4	3.6	5.3	2.2	1.5	5.5	6.2	5.7	0.9	1.8	28.2	10.9	6.5	7.0	0.9
7	6/2/2010	28.5	6.1	3.5	5.1	2.0	1.5	5.3	5.9	5.6	0.9	1.7	34.4	10.8	5.6	6.8	0.8
8	6/3/2010	0.25	6.4	3.7	5.4	2.2	1.6	5.5	6.2	5.8	0.9	1.8	35.5	10.9	5.4	6.7	0.8
9	6/4/2010	0.5	6.4	3.7	5.3	2.1	1.5	5.5	6.1	5.5	0.9	1.7	26.1	10.8	5.1	6.7	0.7
10	6/5/2010	0	6.9	4.0	5.8	2.3	1.7	6.0	6.6	6.3	0.9	1.9	23.0	11.2	5.0	7.0	0.8
Summer 2010	**Date**	**Rain**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**AWM5**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**BWM5**	**C1WM**	**C2WM**	**C3WM**	**D1WM**	**D2WM**
	8/1/2010	0	5.7	3.4	5.2	2.6	3.4	4.7	5.3	46.3	0.3	26.0	23.5	24.5	41.5	7.9	46.8
	8/2/2010	0	5.8	3.5	5.5	2.9	3.5	4.9	5.5	43.7	0.3	24.9	25.5	23.5	41.7	8.3	39.6
	8/3/2010	1	5.6	3.4	5.3	2.8	3.4	4.8	5.4	41.2	0.3	25.3	23.6	23.2	41.6	8.5	34.9
	8/4/2010	0	5.7	3.5	5.6	3.0	2.7	4.9	5.7	47.2	0.4	24.5	23.1	22.9	41.4	8.6	31.8
	8/5/2010	0	6.0	3.8	5.9	3.2	2.7	5.3	6.0	52.7	0.4	24.5	24.5	23.6	40.8	9.0	28.5
	8/6/2010	0	5.7	3.7	5.7	3.1	2.8	5.1	5.7	44.8	0.3	25.9	22.8	25.1	41.7	8.7	29.0
0	8/7/2010	11	5.0	3.2	5.0	2.7	2.5	4.6	5.2	52.9	0.2	23.9	19.4	25.1	42.3	7.6	28.9
1	8/8/2010	5	4.3	2.7	4.4	2.5	1.9	4.0	4.7	27.6	0.3	5.8	25.1	21.3	41.0	5.7	25.2
2	8/9/2010	0.5	4.7	3.0	4.8	2.6	2.0	4.4	5.1	27.9	0.3	2.9	26.1	20.4	44.7	6.2	24.0
3	8/10/2010	0	5.0	3.2	5.0	2.5	2.1	4.8	5.4	30.4	0.4	2.3	25.8	18.3	43.5	6.4	20.3
4	8/11/2010	0	5.2	3.4	5.1	2.4	2.1	4.9	5.5	32.9	0.4	2.0	25.8	16.8	43.6	6.7	16.9
5	8/12/2010	0	5.4	3.6	5.4	2.3	2.1	5.1	5.7	37.4	0.4	2.0	26.9	16.4	44.5	7.0	15.0
6	8/13/2010	0	5.7	3.8	5.5	2.3	2.2	5.3	5.9	40.5	0.5	2.1	27.2	16.0	43.2	7.3	13.8
7	8/14/2010	0	5.9	4.0	5.7	2.4	2.4	5.4	6.1	44.3	0.5	2.3	27.2	16.3	42.0	7.6	13.5

Fall 2010	Date	Rain	AWM1	AWM2	AWM3	AWM4	AWM5	BWM1	BWM2	BWM3	BWM4	BWM5	C1WM	C2WM	C3WM	D1WM	D2WM
	10/5/2010	0	5.1	3.0	4.4	2.2	3.4	4.4	5.0	58.9	1.0	1.8	79.9	11.9	4.5	7.1	7.2
	10/6/2010	0	5.1	3.1	4.4	2.2	3.4	4.3	5.0	61.1	1.0	1.8	78.8	11.7	4.4	7.6	7.0
	10/7/2010	0	4.9	2.9	4.3	2.1	3.4	4.2	4.9	61.3	1.0	1.8	76.2	11.7	4.3	7.8	6.7
	10/8/2010	0.5	5.1	3.0	4.4	2.3	3.5	4.5	5.1	60.3	1.1	1.9	75.2	12.0	4.3	8.1	6.5
0	10/9/2010	25	5.2	3.1	4.4	2.3	3.5	4.6	5.3	51.9	1.1	1.9	130.1	12.1	4.3	7.6	6.4
1	10/10/2010	16	5.4	3.2	4.5	2.4	3.5	4.7	5.4	41.0	1.1	1.9	141.6	11.8	7.8	6.7	6.3
2	10/11/2010	1.5	5.1	3.0	4.3	2.3	3.5	4.5	5.1	41.5	1.1	1.8	122.0	11.4	27.1	6.3	6.2
3	10/12/2010	0.75	5.1	3.1	4.4	2.4	3.5	4.5	5.2	42.0	1.1	1.9	123.6	11.5	33.3	6.4	6.0
4	10/13/2010	0	5.1	3.1	4.4	2.4	3.5	4.5	5.1	43.6	1.1	1.9	109.0	11.4	32.5	6.4	5.8
5	10/14/2010	0.25	5.0	3.0	4.3	2.3	3.5	4.4	5.0	45.5	1.1	1.9	109.3	11.3	14.6	6.4	5.6
6	10/15/2010	0	5.0	3.0	4.4	2.3	3.5	4.4	5.0	48.8	1.2	1.9	112.3	11.3	7.5	6.5	5.4
7	10/16/2010	0	4.9	2.9	4.3	2.2	3.5	4.2	4.9	51.4	1.2	1.9	103.8	11.1	6.0	6.5	5.2
8	10/17/2010	0	4.9	2.9	4.3	2.2	3.5	4.2	4.9	54.8	1.2	1.9	102.2	11.3	5.4	6.8	5.1
9	10/18/2010	0	5.0	2.9	4.3	2.3	3.5	4.2	5.0	56.1	1.2	1.9	103.0	11.4	5.1	7.3	4.9
10	10/19/2010	0	5.1	3.1	4.5	2.4	3.5	4.3	5.2	60.4	1.2	2.0	99.9	11.5	4.9	7.8	4.7
11	10/20/2010	0	5.1	3.1	4.5	2.3	3.5	4.3	5.1	60.6	1.2	2.0	95.6	11.4	4.7	7.9	4.6
Winter 2011	**Date**	**Rain**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**AWM5**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**BWM5**	**C1WM**	**C2WM**	**C3WM**	**D1WM**	**D2WM**
	1/1/2011	0.25	5.3	3.4	4.8	3.2	1210.7	4.3	5.3	87.3	841.9	806.5	100.0	67.6	383.0	6.6	974.2
	1/2/2011	0	5.4	3.4	4.9	3.2	1268.7	4.3	5.4	107.5	830.7	800.4	101.2	82.6	367.0	6.8	1001.7
	1/3/2011	0	5.3	3.4	4.8	3.2	1360.8	4.3	5.4	129.7	868.4	841.5	100.3	90.4	368.5	7.1	1070.5
	1/4/2011	0.25	5.7	3.7	5.2	3.4	983.3	4.6	5.7	154.7	587.2	601.8	102.8	41.4	268.3	8.2	824.2
0	1/5/2011	33.5	6.2	4.1	5.6	3.7	276.1	5.0	6.2	143.6	91.3	132.3	111.2	13.2	44.0	8.4	274.6
1	1/6/2011	42.25	6.7	4.5	6.0	4.0	7.2	5.4	6.7	145.2	3.0	3.8	147.9	10.1	3.9	8.2	21.9
2	1/7/2011	26.75	7.0	4.7	6.1	4.0	6.0	5.5	6.8	147.0	3.0	3.9	166.9	10.0	3.9	8.4	29.7
3	1/8/2011	5.75	6.7	4.5	5.8	3.9	5.8	5.3	6.4	139.1	3.2	3.9	166.6	9.9	3.8	8.2	25.2
4	1/9/2011	0	6.3	4.2	5.5	3.8	5.8	5.0	6.1	134.3	3.3	4.1	162.7	9.7	3.7	7.8	23.7
5	1/10/2011	0	6.1	4.1	5.3	3.8	200.8	4.9	6.0	134.9	207.2	158.1	165.4	9.6	50.4	7.6	382.9
6	1/11/2011	0	6.2	4.1	5.5	3.9	416.2	5.0	6.1	140.9	320.5	260.2	161.3	9.7	93.6	7.7	442.3

Site 4

Fall 2008	Date	Rainfall	AWM1	AWM2	AWM3	AWM4	BWM1	BWM2	BWM3	BWM4	CWM1	CWM2	CWM3	CWM4
	Nov 9 2008	0	47.7	33.8	98.0	82.0	34.3	33.3	170.8	120.5	76.3	103.2	68.6	79.0
	Nov 10 2008	1	46.9	33.8	109.8	86.8	34.2	33.7	186.2	122.2	75.2	100.5	70.7	80.6
	Nov 11 2008	1.75	46.2	34.0	102.0	66.2	34.5	34.3	155.0	97.0	74.2	98.3	62.3	67.9
	Nov 12 2008	0	45.9	33.9	86.6	59.4	35.0	34.1	129.9	91.7	74.5	95.4	55.0	62.5
0	Nov 13 2008	12.75	45.1	32.6	77.7	53.4	34.1	32.9	110.8	73.1	69.8	89.5	48.2	54.3
1	Nov 14 2008	3.25	41.9	30.7	82.4	54.5	32.7	31.8	112.5	76.0	56.0	79.1	52.4	57.5
2	Nov 15 2008	23.25	36.0	29.7	74.6	48.3	31.2	29.8	92.7	56.6	45.7	71.2	45.1	49.2
3	Nov 16 2008	0.25	28.9	30.5	87.6	58.5	29.4	27.3	106.4	70.8	34.2	61.9	51.5	58.1
4	Nov 17 2008	0	31.0	33.0	91.7	62.2	30.7	28.4	106.0	76.0	40.0	65.7	54.3	61.3
5	Nov 18 2008	0	32.2	34.4	99.2	67.6	31.0	27.5	114.1	85.2	43.8	67.7	62.2	72.3
Winter 2009	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	2/8/2009	0	72.8	96.2	1299.9	1861.9	1913.2	2547.0	4977.0	2945.4	5533.6	6831.3	5431.4	4334.6
	2/9/2009	0.7	72.9	93.7	739.9	951.7	1562.1	2022.6	3487.5	1960.3	4487.3	4632.9	3646.2	2866.6
	2/10/2009	0.6	70.8	90.6	520.1	554.7	1320.5	1679.3	2757.2	1409.3	2489.8	2495.8	2270.8	1782.3
0	2/11/2009	20.1	47.0	52.4	177.6	97.0	1082.8	1281.8	1199.8	437.4	70.3	63.3	225.1	169.5
1	2/12/2009	14.8	43.6	46.9	140.7	65.7	153.5	145.4	117.6	107.6	33.8	28.4	66.0	89.7
2	2/13/2009	0	50.0	53.3	142.7	94.1	112.2	110.5	123.6	103.8	46.8	46.1	85.1	100.7
3	2/14/2009	0	53.4	56.5	253.6	1756.3	96.1	90.1	213.6	480.5	56.9	56.2	131.3	481.5
4	2/15/2009	0.8	55.2	57.2	1340.1	3138.0	84.1	78.5	680.7	1119.0	64.2	63.2	789.1	1759.6
5	2/16/2009	0	56.1	59.2	2039.3	3754.5	81.5	75.0	1007.1	1329.1	70.4	69.8	1391.3	2166.8
6	2/17/2009	0.8	57.5	61.0	2587.3	4413.4	81.1	75.1	1214.3	1509.5	72.6	73.6	1729.4	2485.9
7	2/18/2009	4.2	57.5	61.3	2470.0	3577.4	84.3	76.3	1194.0	1340.7	73.9	74.5	1712.2	2139.8
8	2/19/2009	1.4	58.9	62.7	2071.9	2580.7	89.2	79.5	1105.3	1114.2	78.5	79.9	1550.0	1829.3
9	2/20/2009	0.7	60.3	63.9	2094.4	2774.1	95.5	86.9	1148.4	1170.1	84.7	90.9	1804.6	2182.4

Spring 2009	Date	Rainfall	AWM1	AWM2	AWM3	AWM4	BWM1	BWM2	BWM3	BWM4	CWM1	CWM2	CWM3	CWM4
	April 1 2009	1.9	42.8	43.1	106.4	60.2	159.0	105.8	80.8	50.2	43.3	54.3	75.4	85.7
	April 2 2009	0	43.1	43.5	97.7	59.7	148.8	97.3	70.9	49.0	44.0	55.6	70.3	83.8
0	April 3 2009	31.6	40.9	41.1	78.6	50.6	141.1	89.5	62.2	43.6	40.7	48.9	56.1	65.0
1	April 4 2009	2.3	36.6	37.5	77.7	60.5	124.3	66.4	75.7	56.4	38.1	45.4	61.7	78.5
2	April 5 2009	0	40.6	39.9	96.8	71.8	94.4	48.7	98.2	69.1	43.4	50.6	76.7	99.8
3	April 6 2009	17.9	43.2	41.9	91.7	61.3	67.6	39.7	84.1	60.7	47.1	55.7	70.4	87.4
4	April 7 2009	0	43.9	42.0	85.6	65.7	35.3	26.4	87.8	68.5	49.0	58.7	63.9	86.8
5	April 8 2009	0	44.0	41.9	84.8	65.6	17.1	17.1	91.2	68.2	48.4	58.6	72.1	98.4
6	April 9 2009	0.3	41.9	39.6	93.7	68.3	17.1	17.1	103.7	75.3	50.2	61.0	71.8	96.8
7	April 10 2009	0	41.0	40.0	102.5	67.4	18.0	17.8	106.8	73.2	53.3	62.1	76.1	99.5
8	April 11 2009	0	42.3	40.8	105.7	65.8	18.0	18.7	104.9	70.8	54.5	63.9	79.5	100.5
9	April 12 2009	0	43.1	41.7	112.0	69.2	18.5	19.2	107.9	72.1	55.4	65.0	82.2	103.8
Summer 2009	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	July 22 2009	0	43.4	38.9	59.0	40.9	59.4	27.3	73.2	53.9	48.8	41.7	36.4	49.1
0	July 23 2009	37.25	39.9	36.3	51.2	39.0	60.1	27.1	71.0	49.8	41.3	35.5	32.3	44.6
1	July 24 2009	2.75	37.6	35.8	49.7	40.5	54.5	26.3	53.5	44.9	36.8	31.8	31.3	47.7
2	July 25 2009	16	38.7	36.2	45.4	38.8	54.5	25.6	48.1	42.7	40.1	33.9	30.6	45.3
3	July 26 2009	15.75	36.3	33.8	47.5	41.5	48.6	23.7	56.5	49.6	36.9	30.6	32.5	48.0
4	July 27 2009	0.5	36.2	33.8	50.0	42.4	51.5	23.5	62.2	54.1	36.8	30.6	33.9	49.9
5	July 28 2009	9.75	38.8	36.2	52.5	42.5	62.1	24.6	66.5	54.4	41.3	34.2	33.7	50.5
6	July 29 2009	3	39.7	35.5	48.4	40.9	54.8	23.7	61.5	49.8	43.0	35.2	30.8	45.3
7	July 30 2009	0	39.6	35.6	52.4	42.6	61.6	25.7	75.4	55.6	43.3	35.0	32.5	49.3
8	July 31 2009	0	40.7	36.8	55.2	43.3	67.5	27.0	78.7	55.6	44.3	36.3	33.4	49.5
Fall 2009	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	Sept 24 2009	0	54.7	49.4	67.0	48.1	158.5	33.6	85.3	58.8	65.5	53.5	48.6	61.6
	Sept 25 2009	0	57.5	50.6	68.4	49.4	155.0	33.6	82.6	59.1	67.7	55.5	48.9	59.9
	Sept 26 2009	0.75	54.3	49.2	73.2	50.6	152.0	34.0	85.7	62.5	66.2	56.0	49.6	61.6
	Sept 27 2009	0.25	57.1	49.6	70.4	50.3	146.1	33.8	91.7	62.0	67.2	54.2	48.9	61.2
0	Sept 28 2009	26.5	55.7	46.6	57.5	45.5	121.7	32.7	90.6	63.4	67.4	56.5	40.5	54.2
1	Sept 29 2009	8.75	44.3	38.0	55.7	51.6	100.5	34.1	98.7	68.3	63.6	54.2	34.7	51.3
2	Sept 30 2009	1	43.6	41.7	65.7	56.8	110.4	35.3	114.4	74.2	57.9	52.1	40.5	61.4
3	October 1 2009	0.5	43.9	43.4	69.1	62.0	109.4	35.6	117.4	77.0	54.5	50.0	43.4	62.5
4	October 2 2009	0.75	45.0	43.7	67.3	57.3	108.9	33.8	107.5	74.8	54.8	49.7	42.3	62.4

Winter 2010	Date	Rainfall	AWM1	AWM2	AWM3	AWM4	BWM1	BWM2	BWM3	BWM4	CWM1	CWM2	CWM3	CWM4
	Nov 27 2009	0.4	48.3	46.9	93.8	82.1	179.0	43.0	147.2	97.6	59.4	45.3	60.3	89.2
	Nov 28 2009	0	48.2	47.1	90.2	80.0	185.4	44.3	137.3	92.3	59.1	45.1	59.0	85.3
	Nov 29 2009	4.7	48.4	47.2	91.0	83.7	185.4	43.7	138.8	95.1	58.7	44.9	60.1	88.0
	Nov 30 2009	0.3	46.7	45.1	93.0	85.5	179.6	41.6	146.8	98.7	56.6	42.2	61.4	91.6
	Dec 1 2009	0.6	45.6	45.4	99.7	91.1	187.1	43.0	163.4	106.3	53.2	39.9	64.4	97.7
0	Dec 2 2009	9.4	46.6	46.2	101.9	89.4	191.8	44.3	169.3	107.3	53.3	40.7	65.1	97.0
1	Dec 3 2009	3.1	45.4	44.6	91.8	82.9	181.6	40.9	140.9	97.4	50.8	38.3	62.2	90.9
2	Dec 4 2009	0	44.8	44.5	99.4	91.2	188.3	41.5	165.9	108.9	47.8	36.9	66.7	100.9
3	Dec 5 2009	0.6	46.0	45.9	105.4	97.7	194.9	43.2	171.7	115.8	49.9	39.2	69.9	107.0
4	Dec 6 2009	0	47.0	47.1	111.4	102.6	198.8	44.5	184.9	121.4	52.2	41.2	73.2	111.7
5	Dec 7 2009	0	48.2	48.2	115.4	103.5	203.3	46.1	195.8	115.6	54.5	43.2	76.6	114.7
Spring 2010	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	April 30 2010	0	33.4	33.5	72.8	54.3	33.0	30.3	60.0	50.6	50.1	42.0	44.9	54.1
0	May 1 2010	10	40.7	41.3	69.4	57.3	40.7	37.5	60.0	51.7	60.5	50.5	47.3	53.9
1	May 2 2010	13.5	47.4	46.6	71.9	66.0	43.2	38.8	68.3	58.2	70.6	59.2	42.6	57.4
2	May 3 2010	11.5	40.5	39.6	64.3	61.4	33.6	31.9	68.9	58.6	56.0	51.4	32.4	54.6
3	May 4 2010	2.5	39.2	41.2	68.1	64.3	35.5	34.5	72.2	59.6	45.4	41.4	43.2	57.7
4	May 5 2010	12.25	40.3	42.0	70.4	63.0	36.7	35.0	67.6	58.4	46.4	42.0	39.7	57.7
5	May 6 2010	0	38.9	39.9	69.8	65.8	34.8	33.7	71.2	62.0	45.6	41.4	42.2	58.4
6	May 7 2010	14	40.1	40.7	75.4	70.1	35.8	34.0	74.9	65.5	46.0	41.8	42.0	60.8
7	May 8 2010	13	31.7	32.6	62.9	61.6	29.1	27.8	72.0	59.9	34.6	30.8	33.8	52.5
8	May 9 2010	1.75	11.2	11.3	23.1	18.1	11.5	12.0	26.4	22.1	11.5	12.6	11.9	15.0
9	May 10 2010	0	16.1	15.2	25.7	25.0	13.8	14.6	29.3	24.6	15.6	14.0	18.0	23.5
Summer 2010	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	August 5 2010	0	7.7	7.7	9.9	7.9	20.1	8.7	13.0	15.2	10.6	9.3	7.7	8.6
	August 6 2010	0	11.6	11.5	17.9	14.4	38.4	14.6	24.5	29.1	18.2	15.8	14.2	16.3
0	August 7 2010	17.75	3.3	3.3	1.0	1.0	3.3	3.3	1.0	3.3	3.3	3.3	1.0	1.0
1	August 8 2010	5	3.3	3.3	1.2	1.2	3.3	3.3	1.2	3.3	3.3	3.3	1.2	1.2
2	August 9 2010	0	6.4	8.7	10.6	10.1	29.3	9.7	16.9	23.4	10.9	11.1	8.6	10.7
3	August 10 2010	0	5.3	3.3	6.7	4.0	22.4	7.3	13.6	17.8	5.9	8.3	5.2	4.2
4	August 11 2010	0	18.0	17.8	23.2	19.3	64.1	22.0	30.4	48.7	23.1	20.2	16.9	20.6
5	August 12 2010	0	28.7	28.3	45.4	37.7	109.2	35.8	59.5	82.9	37.8	32.8	32.9	39.5
6	August 13 2010	0	13.8	10.7	20.4	16.5	53.9	17.6	28.1	40.3	18.3	16.8	15.3	17.8

Fall 2010	Date	Rainfall	AWM1	AWM2	AWM3	AWM4	BWM1	BWM2	BWM3	BWM4	CWM1	CWM2	CWM3	CWM4
	Sept 24 2010	0	39.7	39.8	78.6	76.3	160.6	49.7	115.0	99.3	51.8	46.8	59.7	72.6
	Sept 25 2010	2	40.1	41.2	80.7	76.8	168.4	52.6	118.5	101.9	56.2	49.5	60.4	74.4
	Sept 26 2010	0	39.6	41.0	85.0	83.0	167.6	55.0	122.2	106.1	57.0	52.8	65.1	79.6
0	Sept 27 2010	7.25	43.5	42.0	89.1	81.1	170.0	52.2	126.1	102.9	61.0	55.9	69.8	83.7
1	Sept 28 2010	23	37.2	35.1	74.9	72.8	159.6	49.3	96.8	90.6	55.3	49.4	57.1	70.3
2	Sept 29 2010	0	36.6	40.2	79.7	77.8	151.5	47.3	112.9	101.0	48.7	43.7	64.2	75.3
3	Sept 30 2010	0	40.1	38.4	78.8	78.6	158.9	50.6	121.5	107.7	54.3	47.1	63.3	75.6
4	October 1 2010	0	41.6	41.9	86.3	80.5	153.6	49.0	121.9	108.2	55.7	52.0	66.7	78.7
5	October 2 2010	0	41.8	42.7	91.2	84.1	161.8	51.9	128.9	116.1	56.3	51.9	68.9	83.2
Winter 2011	**Date**	**Rainfall**	**AWM1**	**AWM2**	**AWM3**	**AWM4**	**BWM1**	**BWM2**	**BWM3**	**BWM4**	**CWM1**	**CWM2**	**CWM3**	**CWM4**
	Dec 2 2010	0	48.2	45.8	125.0	119.3	186.7	90.7	173.4	154.9	61.0	51.1	107.7	124.0
	Dec 3 2010	0.3	49.8	47.4	128.6	124.9	193.1	93.8	182.4	162.2	64.4	55.0	111.3	130.8
	Dec 4 2010	0	51.2	49.0	134.5	128.9	193.2	96.6	192.3	168.1	67.6	58.4	111.7	134.6
0	Dec 5 2010	22	52.4	50.0	141.9	142.6	194.6	99.0	203.6	175.4	70.5	61.1	115.8	139.7
1	Dec 6 2010	1.8	53.8	51.2	149.0	170.2	196.8	101.3	214.4	182.6	73.5	63.7	120.2	143.6
2	Dec 7 2010	10.5	54.9	52.2	155.9	520.2	199.7	104.0	224.3	188.9	76.4	66.2	125.5	148.6
3	Dec 8 2010	5.8	56.2	53.2	163.3	571.9	200.5	105.9	238.1	197.0	79.3	68.7	131.1	154.5
4	Dec 9 2010	0	57.0	53.8	166.3	203.5	205.1	107.6	248.4	219.8	82.3	70.8	137.1	161.8
5	Dec 10 2010	0	57.7	54.3	167.6	167.9	209.1	109.6	249.7	265.9	84.8	72.7	143.0	181.2
6	Dec 11 2010	0	58.0	54.7	168.3	158.4	211.2	110.8	251.1	302.1	86.9	74.6	149.7	206.2
7	Dec 12 2010	9.8	58.6	55.1	168.7	150.2	212.3	111.7	240.7	303.9	88.5	75.5	149.7	180.7
8	Dec 13 2010	0	59.3	55.5	168.9	143.0	212.3	111.7	245.3	308.1	90.1	76.6	153.8	182.2
9	Dec 14 2010	3.3	60.0	56.1	168.3	136.1	216.8	114.1	256.2	2583.3	92.3	78.7	161.2	817.4
10	Dec 15 2010	4	59.9	56.3	164.9	138.5	222.7	116.5	567.3	6230.5	95.1	81.1	236.4	5373.8
11	Dec 16 2010	1.7	60.9	56.8	160.7	144.2	222.4	110.9	2576.9	6177.3	97.9	83.6	1535.1	6213.4
12	Dec 17 2010	0	60.6	57.2	161.3	148.0	221.5	110.1	2729.0	5618.5	99.8	85.5	2265.7	6227.7

Site 5

Fall 2009	Date	Rainfall	WM1	WM2	WM3	WM4	WM5
	10/17/2009	0	0.7	3.0	8.4	5.1	124.1
	10/18/2009	0	0.7	3.1	8.1	5.0	117.5
	10/19/2009	0	0.6	3.0	8.5	5.1	118.1
	10/20/2009	0	0.8	3.1	7.8	5.5	111.5
	10/21/2009	0.5	0.7	3.0	7.4	5.3	104.7
0	10/22/2009	14	0.8	2.6	7.4	4.8	65.0
1	10/23/2009	0	1.0	2.5	8.0	5.2	6.0
2	10/24/2009	27.75	1.1	3.1	8.2	5.1	6.1
3	10/25/2009	0	1.1	3.0	8.0	4.9	5.8
4	10/26/2009	0	1.1	2.7	8.0	4.6	6.1
5	10/27/2009	0	1.2	3.1	8.1	4.8	6.1
6	10/28/2009	0	1.1	3.1	8.1	4.8	6.2
7	10/29/2009	0	1.1	3.3	8.2	4.8	6.0
Winter 2010	**Date**	**Rainfall**	**WM1**	**WM2**	**WM3**	**WM4**	**WM5**
	1/20/2010	0	272.9	424.8	995.1	545.5	739.1
	1/21/2010	0	262.4	407.1	1366.7	513.2	903.4
	1/22/2010	0	337.1	450.4	2451.3	589.8	1530.8
	1/23/2010	0	408.2	507.8	3457.0	698.6	2166.1
	1/24/2010	0	462.4	561.1	3489.6	805.8	2115.5
0	1/25/2010	49.25	288.4	510.8	1306.2	708.6	952.1
1	1/26/2010	0.5	21.3	316.1	166.5	357.8	310.5
2	1/27/2010	0	35.7	302.2	288.3	324.1	278.1
3	1/28/2010	0	58.5	289.4	823.9	311.5	524.1
4	1/29/2010	0	193.1	333.0	2817.0	387.8	1445.7
5	1/30/2010	0	440.2	533.0	5035.3	753.7	3090.7
6	1/31/2010	0	549.7	715.0	4887.1	997.9	3178.0
Spring 2010	**Date**	**Rainfall**	**WM1**	**WM2**	**WM3**	**WM4**	**WM5**
	4/24/2010	0	0.8	1.1	9.7	4.2	4.5
	4/25/2010	0	0.7	0.9	11.2	4.3	4.2
	4/26/2010	0	0.7	0.9	14.3	4.3	5.6
0	4/27/2010	10.5	0.5	0.7	8.5	3.6	4.8
1	4/28/2010	24	0.9	0.8	7.0	3.6	5.0
2	4/29/2010	0	1.1	0.9	6.5	3.7	4.4
3	4/30/2010	0	1.2	0.9	6.9	3.8	4.2
4	1/5/2010	0.5	1.2	0.8	6.6	3.8	4.1
5	2/5/2010	0	1.3	1.0	8.2	4.2	4.5

Summer 2010	Date	Rainfall	WM1	WM2	WM3	WM4	WM5
	7/15/2010	0	3.1	2.6	7.6	4.9	4.0
	7/16/2010	0	3.1	2.7	7.3	4.8	3.9
0	7/17/2010	44.75	2.9	2.5	7.1	4.6	3.7
1	7/18/2010	0	2.9	2.6	7.1	4.5	3.6
2	7/19/2010	11.25	2.7	2.4	6.8	4.3	3.5
3	7/20/2010	0	2.8	2.6	7.4	4.6	3.7
4	7/21/2010	6.5	2.7	2.5	7.2	4.5	3.7
5	7/22/2010	10.75	2.7	2.7	7.4	4.6	3.8
6	7/23/2010	0	2.5	2.6	7.2	4.5	3.6
7	7/24/2010	0.5	2.4	2.6	7.3	4.6	3.8
Fall 2010	**Date**	**Rainfall**	**WM1**	**WM2**	**WM3**	**WM4**	**WM5**
	9/29/2010	0	2.5	2.7	7.9	5.1	4.2
0	9/30/2010	74.75	2.4	2.5	8.0	5.0	4.2
1	10/1/2010	5	2.5	2.4	8.3	5.1	4.3
2	10/2/2010	0	2.3	2.1	8.5	4.7	4.2
3	10/3/2010	0	2.4	2.2	8.7	4.7	4.4
4	10/4/2010	0	2.6	2.4	8.7	4.9	4.7
5	10/5/2010	0	2.8	2.6	8.3	5.2	4.5
6	10/6/2010	11	2.6	2.6	8.2	5.0	4.3
7	10/7/2010	6.75	2.8	2.4	8.7	5.1	4.6
Winter 2011	**Date**	**Rainfall**	**WM1**	**WM2**	**WM3**	**WM4**	**WM5**
	11/25/2010	0	3.1	2.5	687.0	3.9	434.9
	11/26/2010	0.5	3.4	2.8	524.4	3.9	372.9
	11/27/2010	0	3.4	2.9	400.9	3.8	159.3
	11/28/2010	1	3.4	3.0	379.4	3.8	105.7
	11/29/2010	0	3.7	3.1	167.3	4.0	13.0
0	11/30/2010	10.75	3.7	3.0	156.0	4.0	5.5
1	12/1/2010	44	3.8	3.2	8.8	4.5	4.9
2	12/2/2010	2	3.4	2.7	8.3	4.4	4.6
3	12/3/2010	0	3.4	2.7	9.1	4.4	4.7
4	12/4/2010	0	3.4	2.7	145.5	4.2	16.9
5	12/5/2010	0	3.3	2.6	739.6	4.1	363.2
6	12/6/2010	0	3.4	2.8	852.8	4.2	500.4
7	12/7/2010	0	3.4	2.8	861.3	4.1	440.2
8	12/8/2010	0	3.3	2.7	968.6	4.1	517.6
Spring 2011	**Date**	**Rainfall**	**WM1**	**WM2**	**WM3**	**WM4**	**WM5**
	4/14/2011	1	1.6	3.1	7.9	3.0	3.6
	4/15/2011	0	1.6	2.7	8.3	2.9	3.2
0	4/16/2011	20.5	1.7	2.7	8.4	3.0	3.5
1	4/17/2011	21.75	2.2	2.9	8.2	3.0	3.9
2	4/18/2011	0	2.2	3.0	8.5	2.9	4.9
3	4/19/2011	0	2.3	3.1	8.9	3.0	4.0
4	4/20/2011	25	2.1	2.8	8.5	2.9	3.9
5	4/21/2011	0	2.1	2.8	8.9	3.0	3.9
6	4/22/2011	0	2.2	2.8	8.5	3.2	3.7
7	4/23/2011	3.5	2.1	2.8	7.8	3.1	3.7