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MEASUREMENT OF CONCRETE PERMEABILITY

By

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A Doctoral Thesis Submitted in partial fulfilment of the requirements for the award of Doctor of Philosophy of the Loughborough University of Technology

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DECLARATION

No portion of the research referred to in this thesis has been submitted in support of an application for another degree or qualification at this or any other University or other institution of learning.
SYNOPSIS

A comparison was made between a number of laboratory and in-situ concrete permeability test methods. The laboratory tests used measured air, water, and water vapour permeability, whilst the in-situ tests used were the Initial surface absorption test, the Figg air and water tests, and a new in-situ method called the Egg test; a non-destructive surface test which measures air permeability properties.

An initial set of tests were carried out on six concrete mixes with water:cement (w/c) ratios between 0.3 and 0.8. These tests showed that problems existed with both the laboratory and in-situ test methods. Some of these problems arose from the preparation of specimens or test procedures and these were overcome with practice or by modifying the test methods. However, it was found that a major problem is moisture in the concrete which decreases it's measured permeability.

Further tests were carried out on a second set of concrete specimens with the same mix proportions as the first and a set of mortar specimens with w/c ratios ranging from 0.3 to 1.1 and cement:sand (c:s) ratios from 1:1 to 1:5. Results from tests on oven dry specimens were used to compare the different methods and showed that few simple relationships existed between the different methods. Comparing the test results with the mix proportions showed that in the majority of cases, the measured permeability values increased as the w/c ratio increased, but the relationships between the tests results and c:s ratio were more complicated.

After these tests had been completed, specimens from twenty six of the mixes were retested after being conditioned to various different moisture contents. The results of these tests showed that in most cases there was a rapid increase in measured permeability as the specimens dried, followed by a slower increase (in some cases a decrease) as the specimens approached an oven dry condition.

To complement this study a number of methods were examined for measuring in-situ moisture content. The most promising of these was a non-destructive method which operated
by measuring the electrical permittivity of the material it was placed against. Because the electrical permittivity varies with the amount of water in the concrete, it is largely independent of the type material being tested. Results from this test showed a shallow linear drop from saturated to approximately half of the saturated moisture content, followed by a steep drop towards the oven dry condition.
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## CONTENTS

Title page
SYNOPSIS
ACKNOWLEDGEMENTS
Contents
List of Tables
List of Figures
List of Plates

1 INTRODUCTION
   1.1 Thesis Plan

2 CONCRETE DETERIORATION
   2.1 Background to Concrete Deterioration
   2.2 Causes of Deterioration
      2.2.1 Chemical Deterioration
         2.2.1.1 Sulphate Attack
         2.2.1.2 Acid Attack
         2.2.1.3 Chloride Attack
         2.2.1.4 Other Causes
      2.2.2 Physical Deterioration
         2.2.2.1 Abrasion and Erosion
         2.2.2.2 Fire Damage
         2.2.2.3 Freezing
         2.2.2.4 Alkali Aggregate Reactions
         2.2.2.5 Shock Waves
         2.2.2.6 Other Causes
      2.2.3 Reinforcement Corrosion
         2.2.3.1 Carbonation
         2.2.3.2 Chlorides
         2.2.3.3 Other Causes
   2.3 Discussion of Deterioration
      2.3.1 Extent of Deterioration
      2.3.2 Attitudes to Deterioration
      2.3.3 Effects of Deterioration

3 CONCRETE REPAIR
   3.1 Historical Background
   3.2 Materials and Methods of Repair
      3.2.1 Crack Repairs
      3.2.2 Repairs to Spalling and Disintegrating Concrete
         3.2.2.1 Repairs to Concrete Damaged by
         Reinforcement Corrosion
         3.2.2.2 Protective Coatings
   3.2.3 Repairs to Fire Damaged Concrete
3.3 Discussion of Concrete Repair

3.3.1 Current Extent of the Concrete Repair Industry

3.3.2 Points of Dispute Relating to Concrete Repair

3.3.2.1 Materials

3.3.2.2 Base Preparation

3.3.2.3 Steel Preparation

3.3.3 Cost of Repair

4 PREVENTING DETERIORATION IN CONCRETE

4.1 Current Practice

4.1.1 Prevention at the Design Stage

4.1.2 Prevention on Site

4.2 Improvements in Materials

4.2.1 Cements and Cement Replacements

4.2.2 Aggregates and Fillers

4.2.3 Admixtures and Additives

4.2.4 Reinforcement

4.2.4.1 Stainless Steel

4.2.4.2 Galvanised Steel

4.2.4.3 Epoxy Coated Steel

4.2.4.4 Cathodic Protection

4.3 Improvements in Preventive Measures

4.3.1 Improved Codes of Practice

4.3.2 Improvements in Site Awareness

4.3.3 Improvements in Design Office Awareness

4.3.4 Improvements in Materials

4.4 Cost Effectiveness of Prevention

4.4.1 Prevention versus Repair

4.4.2 Better Materials versus Better Site Practice

4.4.3 Better Design Awareness

5 TESTING CONCRETE FOR DETERIORATION

5.1 Testing for Deterioration in the Laboratory

5.1.1 Long Term Testing

5.1.2 Accelerated Testing

5.1.3 Permeability Testing

5.1.4 Chemical Analysis

5.1.5 Petrographic Analysis

5.2 Testing for Deterioration In Situ

5.2.1 Assessment of the Environment

5.2.2 Inspection of Records

5.2.3 Permeability Tests

5.2.4 Visual Tests

5.2.5 Determining Depth of Cover to Reinforcement

5.2.6 Testing for Carbonation
5.2.7 Testing for Chlorides 62
5.2.8 Electrical Methods 62

6 TEST METHODS EMPLOYED IN THIS RESEARCH

6.1 Philosophy of Testing 64
6.2 Laboratory Test Methods 64
   6.2.1 Permeability Measurement 64
      6.2.1.1 Liquid Permeability Tests 66
      6.2.1.2 Gas Permeability Tests 68
      6.2.1.3 Vapour Permeability Tests 70
   6.2.2 Capillary Intrusion 70
   6.2.3 Strength 71
   6.2.4 Salt Immersion and Oven Drying 71
6.3 In-situ Test Methods 72
   6.3.1 Initial Surface Absorption Test 74
   6.3.2 Figg Air Test 76
   6.3.3 Figg Water Test 78
   6.3.4 Germinns Gas Test 79
   6.3.5 The Egg Test 81
   6.3.6 Carbonation Testing 81
   6.3.7 Moisture Measurement 82
      6.3.7.1 Nuclear Methods 83
      6.3.7.2 Weighing Methods 84
      6.3.7.3 Electrical methods 85

7 DETAILS OF TEST PROGRAMME

7.1 General 101
7.2 Details of Concrete Mixes 1 & 2 101
   7.2.1 Mixing Procedure used for Concrete Mixes 102
   7.2.2 Storage of Specimens Prior to Testing 102
   7.2.3 Specimen Preparation 103
      7.2.3.1 Figg Tests Hole Preparation 103
      7.2.3.2 Cutting of Concrete Cylinders 103
      7.2.3.3 Preparation of Permeability Specimens 104
      7.2.3.4 Conditioning Specimens to Different Moisture Contents 105
   7.2.4 Testing of Concrete Specimens 106
      7.2.4.1 Laboratory Permeability Methods 106
      7.2.4.2 In Situ Permeability Test Methods 106
7.3 Details of Mortar Mixes 108
   7.3.1 Mixing Procedure used for Mortar Mixes 109
   7.3.2 Storage of Specimens Prior to Testing 109
   7.3.3 Specimen Preparation 109
   7.3.4 Testing Mortar Specimens 110
8 DISCUSSION AND INTERPRETATION OF RESULTS OF LABORATORY TEST METHODS

8.1 Water Permeability Tests
8.1.1 Results from C1 Concrete Mixes
8.1.2 Results from Mortar Mixes
8.1.3 Results from C2 Concrete Mixes

8.2 Air Permeability Tests
8.2.1 Tests on Concrete
8.2.2 Tests on Mortar
8.2.2.1 Tests on Oven Dry Specimens
8.2.2.2 Effects of Different Moisture Content

8.3 Vapour Permeability Tests
8.3.1 Tests on Mortar
8.3.2 Tests on Concrete

8.4 Strength Tests
8.4.1 Concrete Strengths
8.4.2 Mortar Strengths

8.5 Accelerated Deterioration Tests
8.5.1 Results from Concrete Cubes
8.5.2 Results from Mortar Cubes

9 DISCUSSION AND INTERPRETATION OF RESULTS FROM IN-SITU TEST METHODS

9.1 Initial Surface Absorption Test
9.1.1 Tests on Site Concrete
9.1.2 Initial Tests on Laboratory Concrete Mixes
9.1.3 Initial Tests on Mortar Mixes
9.1.4 Effects of Moisture Content

9.2 Figg Air Test
9.2.1 Initial Tests on Laboratory Concrete Mixes
9.2.2 Initial Tests on Mortar Mixes
9.2.3 Effects of Moisture Content

9.3 Figg Water Test
9.3.1 Initial Tests on Laboratory Concrete Mixes
9.3.2 Initial Tests on Mortar Mixes
9.3.3 Effects of Moisture Content

9.4 The Egg Test
9.4.1 Tests on Site Concrete
9.4.2 Initial Tests on Laboratory Concrete Mixes
9.4.3 Initial Tests on Mortar Mixes
9.4.4 Effect of Moisture Content

9.5 Moisture Measurement

9.6 Carbonation Tests
10 COMPARISON OF TESTS

10.1 Laboratory Tests

10.1.1 Permeability Tests

10.1.2 Strength and Permeability

10.2 In-situ Test Methods

10.2.1 Figg Tests

10.2.1.1 Tests on Concrete

10.2.1.2 Tests on Mortar

10.2.2 Surface Tests

10.2.2.1 Tests on Concrete

10.2.2.2 Tests on Mortar

10.2.3 Water Tests

10.2.3.1 Tests on Concrete

10.2.3.2 Tests on Mortar

10.2.4 Air Tests

10.2.4.1 Tests on Concrete

10.2.4.2 Tests on Mortar

10.2.4.3 Effects of Different Moisture Contents

10.2.5 Carbonation Tests

10.3 Comparison of Laboratory and In-situ Test Methods

10.3.1 Comparison of Permeability Tests

10.3.2 Comparison of Non-permeability Tests

11 CONCLUSIONS AND RECOMMENDATIONS

11.1 Conclusions

11.2 Recommendations

12 REFERENCES

APPENDICES
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>Concrete Mix Details</td>
<td>111</td>
</tr>
<tr>
<td>7.2</td>
<td>Mortar Mix Details</td>
<td>112</td>
</tr>
<tr>
<td>8.1</td>
<td>Water Permeability Results from Cl Concrete Mixes</td>
<td>129</td>
</tr>
<tr>
<td>8.2</td>
<td>Air Permeability Results from Cl Concrete Mixes</td>
<td>130</td>
</tr>
<tr>
<td>8.3</td>
<td>Percentage Dry Flows and Percentage Saturations from Mortar Specimens</td>
<td>131</td>
</tr>
<tr>
<td>8.4</td>
<td>Compressive Strengths of Concrete Specimens</td>
<td>133</td>
</tr>
<tr>
<td>8.5</td>
<td>Differences Between Calculated and Achieved Strengths from Cl Concrete Mixes</td>
<td>133</td>
</tr>
<tr>
<td>8.6</td>
<td>Compressive Strengths of Mortar Specimens</td>
<td>134</td>
</tr>
<tr>
<td>8.7</td>
<td>Details of Concrete Specimens after One Year of Repeated Salt Immersion and Oven Drying</td>
<td>135</td>
</tr>
<tr>
<td>8.8</td>
<td>Details of Mortar Specimens after One Year of Repeated Salt Immersion and Oven Drying</td>
<td>135</td>
</tr>
<tr>
<td>9.1</td>
<td>Results from Initial Surface Absorption Tests on a Laboratory Column</td>
<td>168</td>
</tr>
<tr>
<td>9.2</td>
<td>Average Results from In-situ Permeability Tests on Air Dried Concrete Specimens</td>
<td>169</td>
</tr>
<tr>
<td>9.3</td>
<td>Initial Surface Absorptions from Mortar Specimens</td>
<td>170</td>
</tr>
<tr>
<td>9.4</td>
<td>Variations in the Initial Surface Absorptions from Mortar Specimens</td>
<td>171</td>
</tr>
<tr>
<td>9.5</td>
<td>Figg Air Times from C2 Concrete Beams</td>
<td>172</td>
</tr>
<tr>
<td>9.6</td>
<td>Figg Air Times from C2 Concrete Cubes</td>
<td>172</td>
</tr>
<tr>
<td>9.7</td>
<td>Figg Air Times from Mortar Specimens</td>
<td>173</td>
</tr>
<tr>
<td>9.8</td>
<td>Figg Water Times from Mortar Specimens</td>
<td>174</td>
</tr>
<tr>
<td>9.9</td>
<td>Egg Test Times from Mortar Beams</td>
<td>175</td>
</tr>
<tr>
<td>9.10</td>
<td>Egg Test Times from Mortar Cubes</td>
<td>176</td>
</tr>
<tr>
<td>9.11</td>
<td>Carbonation Test Results at One Year from Concrete and Mortar Specimens</td>
<td>177</td>
</tr>
<tr>
<td>10.1</td>
<td>Comparison of In-situ Permeability Test Methods Employed in this Research</td>
<td>239</td>
</tr>
</tbody>
</table>

## APPENDIX I

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Water Permeability Test Details from Mortar Mixes</td>
<td>287</td>
</tr>
<tr>
<td>2</td>
<td>Water Permeability Test Details from C2 Concrete Mixes</td>
<td>288</td>
</tr>
<tr>
<td>3</td>
<td>Air Permeability Test Details of Mortar Specimens</td>
<td>289</td>
</tr>
<tr>
<td>4</td>
<td>Vapour Permeability Test Details of Mortar Specimens</td>
<td>291</td>
</tr>
<tr>
<td>5</td>
<td>Vapour Permeability Results from Mortar Specimens</td>
<td>292</td>
</tr>
<tr>
<td>6</td>
<td>Vapour Permeability Results from C2 Concrete Mixes</td>
<td>293</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

6.1a Effects of Drying out 1:2:4 mix (After Glanville(216)) 88
6.1b Effect of Reversing Specimens and Wire Brushing Surface (after Glanville(216)) 88
6.2 Cutaway Drawing of Water Permeability Cell 89
6.3 Relationship Between Pressure and Time (Figg Air Test) 90
6.4 Relationship Between Pressure and Time (Egg Test) 90
6.5 Relationship Between Pressure change and Time for Germanns Gas Test 91
8.1 Relationship Between Water Permeability and Time for Mortar Mix M124 136
8.2 Corrected Relationship Between Water Permeability and Time for Mortar Mix M124 136
8.3 Relationship Between $L_n$ Water Permeability and $L_n$ Time for Mortar Mixes M134/5/6/7/8 137
8.4 Relationship Between Water Permeability and W/C Ratio for Mortar Mixes 138
8.5 Relationship Between $L_n$ Water Permeability and W/C Ratio for C2 Concrete Mixes 139
8.6 Relationship Between $L_n$ Water Permeability and $L_n$ Time for C2 Concrete Mixes 140
8.7 Relationship Between $L_n$ Water Permeability and $L_n$ Time for C1 Concrete Mixes 141
8.8 Relationship Between Air Permeability and W/C Ratio for Mortar Mixes 142
8.9 Relationship Between Air Permeability and Percentage Sand for Mortar Mixes 143
8.10 Relationship Between Air Flow and Percentage Saturation for Mortar Mixes 144
8.11 Relationship Between Weight Change and Time for Vapour Permeability Tests on Mortar Mix M137 145
8.12 Relationship Between Vapour Permeability and W/C Ratio for Mortar Mixes 146
8.13 Relationship Between Vapour Permeability and W/C Ratio for C2 Concrete Mixes 146
8.14 Relationship Between Compressive Strength and W/C Ratio for Mortar Mixes 147
8.15 Relationship Between Compressive Strength and Percentage Sand for Mortar Mixes 148
9.1 Relationship Between Initial Surface Absorption and Time for Laboratory Column 178
9.2 Relationship Between Initial Surface Absorption and W/C Ratio for Cl Concrete Mixes in an Air Dried Condition 179
9.3 Relationship Between Initial Surface Absorption and W/C Ratio for Cl Concrete Mixes after Oven Drying 179
9.4 Relationship Between Initial Surface Absorption and W/C Ratio for Concrete Beams 180
9.5 Relationship Between Initial Surface Absorption and W/C Ratio for Cubes from C2 Mixes 180
9.6 Relationship Between Initial Surface Absorption and W/C Ratio for Mortar Beams 181
9.7 Relationship Between Initial Surface Absorption and W/C Ratio for Mortar Cubes 181
9.8 Relationship Between Initial Surface Absorption and Percentage Sand for Mortar Mixes 182
9.9 Relationship Between Initial Surface Absorption and Saturation for Cl Concrete Mixes 183
9.10 Relationship Between Initial Surface Absorption and W/C Ratio for Retests on Cl Concrete Beams 184
9.11 Relationship Between Initial Surface Absorption and Saturation for Mortar Mixes M113/4/5/6 185
9.12 Relationship Between Initial Surface Absorption and W/C Ratio for Tests on Mortar Beams in an Oven Dry Condition 186
9.13 Relationship Between Initial Surface Absorption and Saturation for Mixes with a W/C Ratio of 0.6 187
9.14 Relationship Between Figg Air Time and W/C Ratio for Cl Concrete Mixes in an Air Dried Condition 188
9.15 Relationship Between Figg Air Time and W/C Ratio for Cl Concrete Mixes After Oven Drying 189
9.16a Relationship Between Figg Air Time and W/C Ratio for Concrete Beams 190
9.16b Relationship Between Figg Air Time and W/C Ratio for Cubes from C2 Mixes 190
9.17 Relationship Between Figg Air Time and W/C Ratio for Mortar Beams 191
9.18 Relationship Between Figg Air Time and W/C Ratio for Mortar Cubes 191
9.19 Relationship Between Figg Air Time and Percentage Sand for Mortar Mixes 192
9.20 Relationship Between Figg Air Time and W/C Ratio for Tests on C1 Mixes in an Oven Dry Condition 193
9.21 Relationship Between Figg Air Time and W/C Ratio for Mortar Beams in an Oven Dry Condition 194
9.22 Relationship Between Figg Air Time and Saturation for Mortar Mix M113 195
9.23 Relationship Between Figg Air Time and Saturation for Concrete Mix C105 195
9.24 Relationship Between Figg Air Time and Saturation for Mortar Mix M114 196
9.25 Relationship Between Figg Air Time and Saturation for Concrete Mix C106 196
9.26 Relationship Between Figg Air Time and Saturation for Concrete Mix C107 197
9.27 Relationship Between Figg Air Time and Saturation for Mortar Mixes M157/8/9 197
9.28 Relationship Between Figg Air Time and Saturation for Mortar Mixes M133/4/5/6 198
9.29 Relationship Between Figg Air Time and Saturation for Mortar Mixes M146/7/8/9 198
9.30 Relationship Between Figg Air Time and Saturation for Mixes with a W/C Ratio of 0.6 199
9.31 Relationship Between Figg Water Time and W/C Ratio for C1 Concrete Mixes in Air Dried Condition 200
9.32 Relationship Between Figg Water Time and W/C Ratio for Concrete Beams 201
9.33 Relationship Between Figg Water Time and W/C Ratio for Cubes from C2 Mixes 201
9.34 Relationship Between Figg Water Time and W/C Ratio for Mortar Beams 202
9.35 Relationship Between Figg Water Time and W/C Ratio for Mortar Cubes 202
9.36 Relationship Between Figg Water Time and W/C Ratio for C1 Concrete Mixes in an Oven Dry Condition 203
9.37 Relationship Between Figg Water Time and W/C Ratio for Mortar Mixes in an Oven Dry Condition 204
9.38 Relationship Between Figg Water Time and Saturation for Concrete Mixes 205
9.39 Relationship Between Figg Water Time and Saturation for Mortar Mixes M113/4/5/6 206
9.40 Relationship Between Figg Water Time and Saturation

xii
9.41 Relationship Between Figg Water Time and Saturation for Concrete Mix C103
9.42 Relationship Between Figg Water Time and Saturation for Concrete Mix C106
9.43 Relationship Between Figg Water Time and Saturation for Concrete Mix C107
9.44 Relationship Between Figg Water Time and Saturation for Mixes with a W/C Ratio of 0.6
9.45 Relationship Between Egg Test Time and W/C Ratio for Concrete Mixes in an Air Dried Condition
9.46 Relationship Between Egg Test Time and W/C Ratio for Cubes from C2 Mixes
9.47 Relationship Between Egg Test Time and W/C Ratio for Mortar Beams
9.48 Relationship Between Egg Test Time and W/C Ratio for Mortar Cubes
9.49 Relationship Between Egg Test Time and Percentage Sand for Mortar Mixes
9.50 Relationship Between Egg Test Time and W/C Ratio for Cl Concrete Mixes Tested in an Oven Dry Condition
9.51 Relationship Between Egg Test Time and W/C Ratio for Mortar Mixes Tested in an Oven Dry Condition
9.52 Relationship Between Egg Test Time and Saturation for Cl Concrete Mixes
9.53 Relationship Between Egg Test Time and Saturation for Concrete Mix C105
9.54 Relationship Between Egg Test Time and Saturation for Concrete Mix C107
9.55 Relationship Between Egg Test Time and Saturation for Mortar Mixes M123/4/5/6
9.56 Relationship Between Egg Test Time and Saturation for Mortar Mixes M113/4/5/6
9.57 Relationship Between Egg Test Time and Saturation for Mortar Mixes M157/8/9
9.58 Relationship Between Egg Test Time and Saturation for Mortar Mixes M146/7/8/9
9.59 Relationship Between Egg Test Time and Saturation for Mortar Mixes M134/5/6/7/8
9.60 Relationship Between Egg Test Time and Saturation for Mortar Mix M113
9.61 Relationship Between Egg Test Time and Saturation for Mixes with a W/C Ratio of 0.6

9.62 Relationship Between Moisture Monitor Reading and Saturation for All Specimens

9.63 Relationship Between Moisture Monitor Reading and Saturation for Selected Specimens

9.64 Relationship Between Moisture Monitor Reading and Saturation for Concrete Mix C103

9.65 Relationship Between Moisture Monitor Reading and Saturation for Mortar Mix M159

9.66 Relationship Between Carbonation Depth and W/C Ratio for Concrete Cubes

9.67 Relationship Between Carbonation Depth and W/C Ratio for Mortar Cubes Stored Outside

9.68 Relationship Between Carbonation Depth and W/C Ratio for Mortar Mixes M113/4/5/6 and M156/7/8/9/0/1

10.1 Relationship Between Ln Water Permeability and Strength for C2 Concrete Mixes

10.2 Relationship Between Ln Water Permeability and Strength for Mortar Mixes

10.3 Relationship Between Ln Water Permeability and Strength for Mortar Mixes with Relationship Proposed by Bamforth

10.4 Relationship Between $\log_{10}$ Intrinsic Permeability and Strength for Mortar Mixes

10.5 Relationship Between Vapour Permeability and Strength for Mortar Mixes

10.6 Relationship Between Vapour Permeability and Strength for C2 Concrete Mixes

10.7 Relationship Between Figg Air Time and Figg Water Time for C1 Concrete Mixes

10.8 Relationship Between Figg Air Time and Figg Water Time for C2 Concrete Mixes

10.9 Relationship Between Figg Air Time and Figg Water Time for Mortar Mixes

10.10a Relationship Between Figg Air Time and Saturation for Mortar Mix M114

10.10b Relationship Between Figg Water Time and Saturation for Mortar Mix M114

10.11 Relationship Between Initial Surface Absorption and
10.12 Relationship Between Initial Surface Absorption and Egg Test Time for Mortar Mixes 248
10.13a Relationship Between Egg Test Time and Saturation for Mortar Mixes M146/7/8/9 249
10.13b Relationship Between Initial Surface Absorption and Saturation for Mortar Mixes M146/7/8/9 249
10.14 Relationship Between Figg Water Time and Initial Surface Absorption for Mortar Mixes 250
10.15 Relationship Between Figg Water Time and Initial Surface Absorption for Retests on Mortar Mixes 251
10.16 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Tests on C1 Concrete Mixes 252
10.17 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Retests on C1 Concrete Mixes 252
10.18 Relationship Between Figg Air and Egg Test Times and W/C Ratio for C2 Concrete Beams 253
10.19 Relationship Between Figg Air and Egg Test Times and W/C Ratio for C2 Concrete Cubes 253
10.20 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Retests on Mortar Mixes 254
10.21 Relationship Between Figg Air and Egg Test Times and Saturation for C1 Concrete Mixes 255
10.22 Relationship Between Figg Air and Egg Test Times and Saturation for Mixes with a W/C Ratio of 0.6 256
10.23 Relationship Between initial Surface Absorption and Strength for Mortar Mixes 257
10.24 Relationship Between Vapour Permeability and W/C Ratio for Mortar Mixes with C:S ratios of 1:3, 1:4 and 1:5 258
10.25 Relationship Between carbonation Depth and W/C Ratio for Mortar Mixes with C:S ratios of 1:3, 1:4 and 1:5 258
10.26 Relationship Between Carbonation Depth and Vapour Permeability for Mortar Mixes with C:S ratios of 1:3, 1:4 and 1:5 259
LIST OF PLATES

6.1 Water Permeability Apparatus 92
6.2a Air Permeability Apparatus 93
6.2b Air Permeability Specimen and Sleeve 93
6.3 Vapour Permeability Specimens 94
6.4 Mortar Specimens During Immersion in Salt Bath 94
6.5a ISAT Cap for Inclined Surfaces 95
6.5b ISAT Cap for Horizontal Surfaces, with Apparatus 95
6.6a Figg Test Hole Detail 96
6.6b Figg Air Test Apparatus 96
6.7 Figg Water Test Apparatus 97
6.8 Germanns Gas Test Apparatus 97
6.9 Egg and Metal Former 98
6.10 Carbonation Test using Phenolphthalein Indicator Solution 98
6.11 Protimeter Surveymaster 99
6.12 Bouyoucos Moisture Meter 99
6.13 Moisture Monitor M49 100
7.1a 'Clipper' Masonry Saw, with Diamond Tipped Blade 113
7.1b Jig to Hold Cylindrical Specimens 113
7.2a Water Permeability Specimens (Isopon sealant-left polyester resin sealant-right) 114
7.2b Preparation of Water Permeability Specimens 114
9.1 Initial Surface Absorption Test in use at Budbrook Water Tower 227
9.2 The Egg Test in use at Budbrook Water Tower 227
Concrete is an extremely versatile construction material, which sees considerable use in construction worldwide\(^{(1)}\). In the majority of cases, it is entirely satisfactory. Unfortunately, in a small proportion of concrete, problems occur, either because the concrete is not adequate for the particular conditions of use, or because it does not achieve its full potential\(^{(2)}\).

Concrete deterioration is not a new problem\(^{(3)}\), but it has become more prevalent in recent years\(^{(4)}\). More extensive use of concrete, changes in material properties, construction techniques, and design approaches have resulted in a large amount of concrete of uncertain durability\(^{(5)}\).

A byproduct of this era is the concrete repair industry, which has grown to meet the demand caused by more widespread deterioration, and increased maintenance costs. This industry has been faced with the problem of producing cost effective durable repairs, rather than merely an aesthetic patching up of deteriorated concrete\(^{(6)}\). This has resulted in the development of new materials and improved techniques to meet the demand, but unfortunately little evidence is yet available to support the claims that these new materials and methods can be used to produce durable repairs\(^{(7,8)}\).

As a result of the increased deterioration there has also been an increase in the use of preventative measures, either in the form of concrete additives or protective coatings, which are applied to the concrete to try and avoid deterioration, or stop it progressing to a critical condition\(^{(9)}\).

This in itself has introduced a new problem of how to assess concrete, to determine the likelihood of deterioration occurring\(^{(10)}\). Two approaches are available, for assessing this. The first relies on having an understanding of the deterioration processes which are occurring, so that it is possible to determine the rate of deterioration, by periodic measurement\(^{(6,11)}\). This is rather inexact, and requires that the deterioration occurs at a known rate. It also relies on the deterioration taking place, which may not be desirable for
aesthetic reasons. A preferable approach is to measure a concrete property or properties that can be related to durability.

The surface is frequently seen as the weakest part of the concrete because it is here that aggressive agents attack\(^{12}\). The surface also has different properties from the bulk of the concrete, which makes it more vulnerable\(^{2,13,14}\). The surface is also important for aesthetic reasons. Nevertheless it is still necessary to recognise that not all concrete deterioration is the result of external attack; for example faulty or unsuitable materials may be included during the mixing process, either deliberately, or by accident.

Because virtually every form of deterioration needs water either as a transport mechanism, or as a reaction medium, excluding water is assumed to be the simplest method of preventing deterioration. Producing impermeable concrete, or treating concrete to make it impermeable is probably one of the best ways of achieving durability\(^{15-17}\).

As a result of this a great deal of emphasis has been placed on measuring permeability, because there is a widely held belief that because this affects the ingress and movement of fluids and ions, it must be related to durability\(^{18}\). Laboratory measurements of concrete permeability have been carried out for many years using samples either cast in a laboratory, or recovered from site. These suffer from the problem that they are not always representative of the concrete under attack, and in the latter case obtaining the samples is a destructive process\(^{19}\).

Efforts have been made in recent years to introduce non-destructive in-situ permeability tests which can be used for assessing durability. These have the advantage, as well as being non-destructive, that they assess the surface properties of the concrete\(^{20}\). A number of tests have been developed, which test either at the surface, or just below it. These tests do not give values of permeability, but rather times, rates of absorption, or rates of flow, none of which can be easily converted to a true permeability value, but which it is hoped can be related to durability\(^{20}\).
Although some of these tests have been available for many years\(^{(21)}\), little or no data is available to demonstrate their suitability for in-situ use. One major problem which has led to a general reluctance to develop these tests further, is that moisture content affects the permeability of the concrete by the action of pore blocking as the degree of saturation increases\(^{(22)}\). As a result, data relating to these tests are either from specimens conditioned by oven drying in the laboratory, or from site concrete with no allowance made for the moisture in the concrete\(^{(20,23)}\).

Some limited relationships have been obtained for oven dried concrete and some limits have been proposed for relating results to durability, or quality of concrete\(^{(24,25)}\). Even so, problems arise due to variations in drying techniques, with different researchers using different drying temperatures and different definitions of 'oven dry'. This has meant that it is difficult to compare the tests on the basis of laboratory results, and harder still to consider how to use them on site. Until these problems are tackled, it will not be possible to assess durability using these techniques. This means that more conventional permeability methods will have to be used, or that deterioration will have to be assessed as it happens.

None of the available techniques can yet be seen as an end in themselves, because it is still essential to consider the environment that the concrete is exposed to, and to recognise that concrete is heterogeneous. A highly skilled approach is necessary to assess durability, considering all of the factors present\(^{(6)}\). The tests can however provide the information about the concrete which can be used to assess its potential, after which further investigations need to be carried out to determine whether this potential is adequate\(^{(18)}\).

1.1 Thesis Plan

This thesis is divided into three sections, which are intended to guide the reader through the reasoning behind the direction of the research. The first section consisting of chapters 2 to 5 examines the background of the subject, the second section (chapters 6 and 7) examines the experimental method which was adopted, and the third (chapters 8 to 10) examines the results.
of the experimental work. The following gives brief details of the content of the thesis on a chapter to chapter basis.

Chapter 2 gives a general overview of the problems of concrete deterioration, specifically related to chemical and physical deterioration, and reinforcement corrosion. The chapter also offers some views on the possible future problems that may be experienced within this industry.

Chapter 3 examines the concrete repair industry, both the methods and materials that are in use and the possible future of this part of the construction industry. Areas of debate relating to concrete repair are also considered and discussed.

Chapter 4 examines the prevention of concrete deterioration as a means of avoiding defects in the future. It considers the different methods and materials that are available today to ensure that tomorrow's concrete will not suffer from the same problems as that of today. Consideration is also given to the cost of using these preventative measures, and the need to educate engineers and clients alike to the fact that long term costs of deterioration may well exceed the short term cost of preventative measures.

Chapter 5 examines the problems of testing concrete both in the laboratory and in situ. Consideration is given to two categories of tests, those that can be used as a direct measure of performance, and those that can be used as indicators of likely performance. Permeability testing is introduced, and falls into the latter category.

Chapter 6 details the tests that were used in the laboratory investigation. These include three laboratory permeability tests, and four in-situ permeability tests, one of which was devised during this research. This chapter also examines the different methods that are available for measuring the in-situ moisture content of concrete.

Chapter 7 gives details of the specimens that were used for the tests, including how they were mixed, stored, conditioned and prepared for the tests.

Chapter 8 gives the discussion and interpretation of the
results of the individual laboratory tests.

Chapter 9 gives the discussion and interpretation of the results of the in-situ permeability methods from experiments carried out on specimens in the laboratory and tests carried out on site. The effects of partial saturation are examined and also the results of the moisture measurements carried out in parallel with the tests in the latter part of the test program.

Chapter 10 gives a comparison of the results of the laboratory and in-situ tests. It also examines similarities between the tests both where these have been expected, and where they were not expected.

Chapter 11 gives the conclusions and recommendations of the work.
2 CONCRETE DETERIORATION

2.1 Background to Deterioration

Deterioration is not a new problem. All construction materials suffer some degree of deterioration with time. The problems occur when deterioration occurs to an unsatisfactory degree. In such cases the concrete is not durable enough for the environment in which it is placed. It is important to realise that most concrete is durable, and that only a small amount suffers from problems. However this small amount is of sufficient proportion to be of concern, as the costs of 'making good' the deterioration increase(26).

Under normal circumstances materials such as steel will deteriorate. This is expected, so measures are taken to reduce the deterioration as much as possible if not completely(27). The same is not true of concrete, which may well exhibit excellent qualities in most environments(28,29). However in some instances deterioration occurs, and when it does it is necessary to understand the processes which cause it, in order to take remedial action and avoid the same problem in future(30,31).

Concrete deterioration has been reported since the 19th century(3), but in recent years it has become of major concern. This has prompted greater research into the causes and ways of assessing deterioration, as well as into new materials to make more durable concrete.

Inclusion of steel reinforcement adds another element to concrete and in many instances this has proved to be the weak link in the durability chain(32). Even so, frequently, no single process can be pinpointed as the sole cause of deterioration.

2.2 Causes of Deterioration

There are many causes of concrete deterioration. They are usually of a 'complex physico-chemical nature' (11,33), and most processes are water dependent, which means that without water present, the deterioration cannot occur(28). Deterioration may
be conveniently subdivided into processes which are of a chemical nature and those which are physical in nature. The processes which affect steel reinforcement combine both physical and chemical processes so these will be dealt with independently.

2.2.1 Chemical Deterioration

Most forms of chemical attack are well documented and in most cases concrete can be specified to resist them. Problems may occur if a particular feature of the environment has been overlooked.

2.2.1.1 Sulphate Attack

This is a problem suffered especially by concrete underground in sulphate bearing ground water\(^{(34)}\), or in open water containing dissolved sulphates for example seawater\(^{(35)}\). In addition to this, sulphates inside the concrete can have disastrous effects for example in the Middle East\(^{(36)}\).

This form of attack has been recognised and understood since the 1930s\(^{(3)}\) and is typified by a softening and swelling of the concrete eventually leading to disintegration. It has been shown that cement content and w/c ratio are important for sulphate resistance\(^{(37)}\). The British Standard, BS8110\(^{(38)}\), gives recommendations for concrete exposed to sulphate attack. It suggests five classes of exposure for sulphate contents in soil ranging from 0.2 to 2.0\% SO\(_3\). To resist these conditions requires increases in the minimum cement content (including any cement replacement material) from 330 to 370kg/m\(^3\), and a reduction in the maximum w/c ratio from 0.55 to 0.45. In addition to this it is recommended that sulphate resisting cements are used, and for the class 5 conditions (the most aggressive) it is recommended that the concrete is painted.

2.2.1.2 Acid Attack

This can occur in a number of ways, either from natural or man-made acids. Naturally occurring acids are often mild and usually affect large areas for example acidic ground water or acidic rainwater. In such cases the attack is aggravated by a continual replacement of the acid and removal of deteriorated
material. This is particularly likely to occur where the problem is caused by flowing acidic ground water.

Man-made acids are usually concentrated, being related to handling or processing of acidic chemicals, or to processes which may result indirectly in acid formation such as lactic acid in dairies. As such, the problem is usually confined to storage vessels and processing areas. The attack takes the form of etching away of the cement leaving exposed aggregate which then becomes dislodged and exposes more concrete to attack.

There are two solutions to this which can help to improve the resistance of the concrete:

i) use soluble aggregate such as limestone. Soluble aggregates will slow down the rate of attack to a more acceptable level, by being dissolved with the cement so reducing the attack on the cement, and neutralising the acid; or,

ii) use acid resisting coatings, such as epoxy resin, or water glass. These prevent the acid from reaching the concrete, so prevent the attack altogether.

Alternatively, where acid tends to collect and concentrate in one place, a good maintenance and cleaning scheme should be implemented, using frequent applications of fresh water, to dilute the acid and flush it away.

2.2.1.3 Chloride attack

This occurs naturally in coastal regions or in areas with large concentrations of salt in the ground such as The Middle East. Chloride attack can also occur in chemical plants if precautions are not taken to prevent it. In the last 20 years it has become a problem with concrete subjected to de-icing salts. This includes problems on structures such as motorway bridges, which were designed before the introduction of de-icing salts and were thus not designed to resist this attack. Many of the problems associated with chlorides in these conditions result from reinforcement corrosion, which will be dealt with in Section 2.2.3.2.

Chlorides have a number of other effects on concrete: reduced sulphate resistance; disintegration caused by salts crystallising out within the pores; efflorescence; expansion;
increased alkali metal content; reduced resistance to alkali aggregate reactions; as well as more complex chemical reactions which are not yet fully understood\(^{36}\). In addition to these long term effects, chlorides cause accelerated hardening which can lead to micro-cracking due to the more rapid shrinkage and heat generation\(^{44}\).

Physical disruption due to salt crystallising out within the concrete pores is especially prevalent in areas where the concrete is subjected to wetting and drying whilst in salty conditions such as in the splash zone in coastal exposures\(^{35}\).

2.2.1.4 Other causes

Many forms of chemical attack are not of major concern from a structural point of view, but they can affect the aesthetics\(^{45}\) and they may also be the first indicators of more serious problems\(^{46}\). They include: leaching; the formation of stalactites and stalagmites; staining; and growths of algae or lichen. These are all dependent on and caused by water on the surface of the concrete or seeping through it. The easiest solution to these problems is a program of regular inspection and the prevention or stoppage of the flow of water\(^{6}\).

2.2.2 Physical Deterioration

Physical deterioration can be either natural or man-made. It can affect the concrete by wearing away the surface, or by disrupting the concrete from the inside. Some chemical processes do not directly affect the concrete chemistry, but cause disruption, so they have been included here.

2.2.2.1 Abrasion and Erosion

These are processes which result in removal of the concrete at the surface. They are caused by movement across the concrete surface, or by impacts on it. Flowing water is a frequent cause and is a greater threat when it contains abrasive material\(^{6,33}\). In structural concrete, especially in industrial environments, vehicles can cause damage either by wear from the wheels or by impact with the concrete.

Where damage due to abrasion is likely to be problem, a
good quality concrete wearing course is usually the best option. In special circumstances such as factory floors, a high quality screed may be used instead. If large areas of concrete are at risk such as roads, or sea defenses, then a degree of deterioration may be acceptable when combined with a planned programme of maintenance\(^{(47)}\).

In addition to causing damage, these processes will take advantage of any weakness caused by other deterioration processes, so accelerating the deterioration\(^{(46)}\).

2.2.2.2 Fire Damage

Fire damage is a special case, which has always to be treated individually\(^{(48)}\). Damage can be merely discolouration, or it may be disruption of the structural elements of the concrete\(^{(49)}\). Fire can cause damage directly by the effects of differential expansion and contraction of the constituents, as well as a pressure build up of steam. In addition the heat may lead to a breakdown of the cement hydrates, and in reinforced concrete it may lead to a loss of strength of the steel.

2.2.2.3 Freezing

Freezing in itself may not be detrimental to concrete, however a combination of freezing and thawing will produce scaling or general deterioration\(^{(28)}\). This is a problem that is not limited to concrete, and many materials suffer from it. As a result, certain aggregates are susceptible as well\(^{(50)}\). The problem is caused by the expansion of water as it freezes, followed by repeated expansion as the ice melts, and the water refreezes. This has the effect of breaking down the pore walls and enlarging any cracks or defects, until the material crumbles.

Many factors affects a concrete's ability to resist freeze-thaw action, these include aggregate type, curing conditions, and the time of year that the concrete was cast\(^{(51,52)}\). The problem can be exaggerated by the use of de-icing chemicals, which reduce the freezing point of the solution, so increasing the number of freezing and thawing cycles\(^{(53)}\). The problem can be combattted in concrete by the use of air entraining agents. The actual mechanism of this form of protection is not yet fully understood, but it is presumed to
2.2.2.4 Alkali Aggregate Reactions

This takes two forms, Alkali Silica Reaction (ASR) and Alkali Carbonate Reaction (ACR)\(^{(55)}\). The latter appears to be less common, and no cases have been reported in the UK\(^{(56)}\). ASR is essentially a chemical reaction between the alkali metals in the cement compounds, principally the sodium and potassium, with certain types of reactive silica which may be present as aggregate within the mix. The reaction can only take place in the presence of water and leads to the formation of a gel which expands and so disrupts the concrete. It is possible that the presence of free chloride increases the risk of this occurring\(^{(57)}\). ACR is a reaction between certain types of carbonate aggregates such as dolomites and the cement compounds\(^{(56)}\).

ASR has been recognised for a number of years\(^{(11)}\). It was first recognised in the United States in 1940, but was not discovered in the UK until 1970\(^{(58)}\). However, its extent is difficult to define as the chemical reaction is a complex one and requires a very fine balance of materials in order to occur\(^{(55)}\), so frequently it is not diagnosed. Fortunately, it is not believed to be a major problem in the UK, although the number of confirmed cases is rising\(^{(59,60)}\).

There is no known way of dealing with the problem once it has occurred, although preventing the ingress of water and allowing the concrete to dry out can prevent further damage. Reducing the quantity of the alkali metals in the cement\(^{(61)}\), and avoiding reactive aggregates wherever possible are the best ways of avoiding the problem\(^{(62)}\). Partial cement replacements also show a lot of potential to reduce AAR, by improving the concrete generally and by reducing the amount of water soluble alkali in the hardened cement paste\(^{(63-66)}\).

2.2.2.5 Shock Waves

The heterogeneous nature of concrete makes it prone to damage from shock waves, because shock induced forces cannot be transmitted through the material adequately\(^{(6)}\). Common shock forces are pile driving, and impacts of ships or ice on coastal
structures. Experience from earthquakes and blasts shows that the best way to protect concrete against shock loads is through extensive use of reinforcement, which helps to resist the loads and should the concrete fail, can prevent a catastrophic collapse of the structure. Fibre reinforced concrete may be useful in such circumstances, as it has very good post cracking properties. Pile forces can be reduced by jetting, or some other means to prepare the hole. Docks and other coastal structures can be protected by the use of fenders. Blasts cannot always be foreseen, but where there are obvious risks such as storage areas containing explosive materials, then extra consideration should be given to providing extra protection.

2.2.2.6 Other Causes

Many concrete defects may occur before the concrete sets. These include: settlement of the subgrade; formwork movement; vibration; setting shrinkage; and, premature support removal\(^{(6)}\). These can all lead to defects which could be avoided by good site practice. Other defects may be the result of poor design detailing, which again could be avoided such as rapid changes of section, rigid joints, excessive deflections, leaking joints, poorly designed drips, poor drainage, inadequate movement joints, thermal stresses, and incompatible materials\(^{(6,45)}\).

Many of these defects can produce cracks, which can be exploited by the other forms of deterioration detailed in the previous sections.

2.2.3 Reinforcement Corrosion

Reinforcement corrosion is one of the major cause of concrete deterioration in the UK\(^{(67)}\), and it is considered to be the main cause of bridge deterioration\(^{(68)}\).

Reinforcement corrosion has been closely related to the process of carbonation, and to the presence of chlorides in the concrete. Both have different effects on the concrete and so affect the reinforcement in different ways. There are other causes, but they are limited in extent and not a major problem.
2.2.3.1 Carbonation

Carbonation is considered to be the main cause of reinforcement corrosion in the UK\(^{(69)}\). It is a chemical process in which atmospheric carbon dioxide\((\text{CO}_2)\), dissolved in water\(\text{(H}_2\text{O)}\), reacts with the calcium hydroxide\((\text{Ca(OH)}_2)\) produced by the cement hydration to form calcium carbonate\((\text{CaCO}_3)\). In a simple form this is as follows:

\[
\text{CO}_2 + \text{H}_2\text{O} + \text{Ca(OH)}_2 \rightarrow \text{CaCO}_3 + 2\text{H}_2\text{O}
\]

This process in itself is not harmful to concrete or steel, and it may in some instances be beneficial. It has been shown that in impermeable concrete the carbonation actually improves the surface properties\(^{(70)}\), increasing the strength and forming a protective layer on the concrete surface\(^{(71)}\). This can help to reduce sulphate and corrosive attacks\(^{(28)}\), and reduces leaching\(^{(33)}\). In more permeable concrete the carbonation can move a long way into the concrete. This creates a problem when steel reinforcement is present.

Under normal circumstances, steel in concrete is protected by the alkaline environment, with pH values in excess of 13 frequently being observed in Portland cement paste\(^{(72)}\). This leads to the formation of a passive layer of magnetite (ferric oxide) on the surface of the steel which prevents corrosion\(^{(73)}\). If the concrete becomes carbonated, then the pH drops to below 10 and the passive layer breaks down leaving the steel unprotected. Rusting can then commence if water and oxygen are present\(^{(74-6)}\). The effects of this are not immediately evident, but because rust occupies a greater volume than the steel from which it is derived, the reinforcement expands, producing bursting pressures, which eventually lead to cracking or scaling of the concrete. Once this occurs then rusting can proceed unabated along the exposed steel.

Evidence suggests that a good quality concrete will only carbonate a few millimetres in its life time, so the steel will be protected\(^{(77)}\). Only if the concrete is too permeable, or if the cover is not thick enough will protection be required to avoid this problem\(^{(78)}\). As a result, reinforcement corrosion due to carbonation can be avoided by good practice at the design
and construction stages. This is discussed in more detail in Chapter 4. Deterioration from this cause can also be dealt with successfully by using appropriate repair techniques which are discussed in Chapter 3.

2.2.3.2 Chlorides

Chlorides have three effects which influence reinforcement corrosion:

i) the chlorides attack the steel by breaking down the passive layer which leads to corrosion of the unprotected steel; 

ii) the presence of chloride concentrations results in the formation of electrical differentials within the steel which produces anodes and cathodes leading to removal of steel from isolated spots, causing pitting; 

iii) the chlorides affect the cement hydrates leading to micro-cracking (see Section 2.2.3.2), which allows carbonation to reach the steel directly and so corrosion can occur.

Chlorides can occur in three forms in concrete. They can be chemically bound into the tricalcium aluminate hydrates (C₃A), as calcium chloroaluminates, they can be bound into the calcium silicate hydrates, or they can occur as free chlorides in the pore solution. Upto 90% of any chloride added at the mixing stage becomes bound, and it is evident that the C₃A content of the cement is important in reducing the concentration of free chloride. There is however, a state of equilibrium between the bound and the free chlorides, such that if some of the free chloride is removed by leaching, then some of the bound chloride will be released into solution. In addition, the chloroaluminates are only stable at high pH, so if the pH falls due to carbonation for example, then the chloride is released as calcium chlorides, so increasing the risk of corrosion both from the low alkalinity and the chlorides. This does have the advantage that it reduces the risk of pitting, although it results in more widespread corrosion.

It has been reported that corrosion resulting from externally applied chlorides is much worse than that resulting from chlorides added at the mixing stage, and it is possible that this is a result of this binding.
Chloride induced corrosion is a much more serious problem than that due to carbonation, because it does not automatically result in cracking, but may merely reduce the cross-sectional area of the reinforcement to a dangerous level by pitting. Pitting is influenced by several properties of the concrete including, low electrical resistivity, high alkali content, and oxygen access, which is dependent upon the concrete being partially dry\(^8\). As chloride induced corrosion is largely water based\(^7\), drying out increases the electrical resistivity\(^8\), so dry concrete contaminated with chlorides is unlikely to exhibit any serious steel corrosion\(^8\). As mentioned earlier, carbonation also reduces the risk of pitting.

Dealing with chloride induced corrosion depends very much on the source. Chlorides may be added at the mixing stage either deliberately in additives, (for example calcium chloride being used as an accelerator) or indirectly in the constituents (for example mix water, or aggregates contaminated with salts\(^8\)). Alternatively they may originate from outside the concrete from sources such as seawater, or de-icing salts\(^8\). Many of the current problems are due either to the use of calcium chloride which is now prohibited in the U.K., or due to the introduction of road salt as a de-icing agent in winter. The problems due to de-icing salt illustrates one of the problems of concrete durability. Today's concrete will only be designed with today's hazards in mind, but it may have to last for 50 years or more. Much of the concrete which is suffering today from the effects of de-icing salt was not designed to resist this particular form of attack\(^5,28\). This demonstrates the inherent resilience of concrete as a material, because so much of it that was not designed to resist these conditions is still sound\(^8\).

Dealing with chloride induced corrosion is not easy. Repair techniques usually involve the removal of all the affected concrete, otherwise the problem can reoccur\(^43\). Alternatively, repairs can be carried out as needed to try and control the problem rather than solve it\(^8\). A number of methods have been tried to reduce the problem, these include cathodic protection of the steel, and electrochemical removal of the chlorides. These are discussed in more detail in Section 4.2.4.
As with carbonation, it is better to avoid the problem at the design stage, by taking special precautions if chlorides are present.

2.2.3.3 Other Causes

Reinforcement corrosion may occur as a direct result of another form of concrete deterioration, which exposes the steel directly to water and oxygen, or weakens the concrete sufficiently to allow these in. In addition poor design and site practice can result in poor concrete in the cover zone which is then unable to provide protection to the steel\textsuperscript{(12,33,76)}. This may be caused by:
i) inadequate cover, due to poor design, or bad steel fixing;
ii) plastic cracking, or settlement cracking, due to inadequate compaction;
iii) honeycombing, or permeable cover due to poor mix design, or poor concrete handling on site; or,
iv) permeable cover due to inadequate curing.

In addition small voids, cracks, water pockets, and similar small features may cause localised corrosion, though this is usually insignificant as a sound concrete has the ability to keep these controlled as local events\textsuperscript{(79)}.

Alternatively, corrosion may be induced by electrical currents flowing in the concrete. This is termed 'forced' corrosion, which may be caused by stray electrical currents induced by external electrical appliances (electricity generating stations or substations) or may result from differences within the concrete such as pore fluid, different corrosion levels on the steel, or the type of steel reinforcement. If stainless steel is in contact with carbon steel, then the carbon steel will corrode\textsuperscript{(28,90-91)}.

2.3 Discussion of Deterioration

Deterioration is a subject which is very easy to consider in purely academic terms. However the implications of deterioration go deeper than the processes that cause it. What is its extent, what attitudes are there to concrete at design, construction, and during its lifetime, will the problem of
deterioration get worse, or will it be reduced by current knowledge, and what options are available in the light of the knowledge we already have?

2.3.1 Extent of Deterioration

Deterioration in one or more forms is likely to occur whenever anything is built, no matter what materials are used. The geographical extent of the deterioration is not really important; of real importance is whether this is a crisis that is growing to epidemic proportions, or whether it is merely an over reaction to a problem which until recently received very little publicity. Most deterioration can be attributed to a particular set of circumstances. In the past these have been such things as (5):

i) the introduction of salt as a de-icing agent on roads;
ii) the boom in construction in The Middle East during the 1970s, without enough thought being given to the aggressive conditions and materials;
iii) the use of high alumina cement without enough knowledge of its stability in certain service conditions;
iv) the post war boom in precast housing using inferior materials;
v) the motorway construction boom of the 1960s; and,
v) the use of calcium chloride as an accelerator, without recognising the potential for reinforcement corrosion.

All of these particular sets of circumstances have particular problems which are now recognised and can be avoided, but unfortunately there is still a legacy of concrete suffering from these mistakes. The future of concrete is uncertain, the same problems may reoccur or new problems may appear.

Alkali aggregate reaction is by no means a new problem, and in the light of current knowledge, the likelihood of it occurring should be decreasing. New materials and techniques always pose a problem, because the required life is long and long-term testing is difficult to accomplish. Caution needs to be exercised when implementing new ideas.

'Improvements' in design methods, materials, and construction techniques, result in what must be realised are
new materials that need researching thoroughly. Ideally, the best way to do this is to monitor new structures over a period of time so that any defects can be identified at an early stage. This has to be a calculated risk with any new idea, but it must be better than merely throwing a new product at an unsuspecting world. A good example is the use of calcium chloride as an accelerator, which was accepted in good faith by many people(26). Risks are an essential part of progress. Unfortunately innovation does not usually get the publicity it deserves, unless it goes wrong, which means that there is then an overemphasis on the problems rather than the successes.

2.3.2 Attitudes to Deterioration

'Deterioration should not occur'(4,29) and 'deterioration is an unavoidable fact of life'(5,11,90) are two different attitudes, and both to some extent represent different aspects of the real problems. Concrete should not deteriorate, but in a real world, the factors controlling whether it does or not are frequently nothing to do with the material properties. The view that deterioration is unavoidable recognises that ageing of concrete is a natural process which cannot be stopped; the problem then is to define the design life or service life that we can expect or want from the concrete(91); should it be acceptable, either now, or in the future, and what if anything can be done to change the situation? It has been stated(30) that there is a known reason for every failure of concrete, and it is a needless reflection on the concrete industry when problems are repeated. It is however unfortunate that even with the available knowledge, it is not always economic, using locally available materials, to carry out the procedures to ensure that the concrete is durable(72).

With the right attitude, future concrete should not suffer from the problems of the past, provided everyone involved in the concrete manufacture accepts their responsibility(4). However the same has been said in the past, and the problems are greater now than they have ever been. To understand why these problems still occur, it is necessary to consider the people behind the material.

On the construction site, the person responsible for making and placing the concrete is not concerned about whether
the concrete will last for 50 years, because that is not his responsibility. The engineer in charge who is responsible in the short term, may be more concerned about meeting deadlines, and senior management want to ensure that the company makes a profit. Similarly in the design office, the consultants want to make a profit. Unfortunately, the client who has to pay may want the 'least cost option', which generally translates to the least cost 'now' option. This invariably leads to cost cutting measures in materials and design, resulting in a reduction of the inherent durability.

It must be realised that good design is essential for a low initial cost and, combined with good detailing and careful construction will result in low maintenance costs. Conversely, poor design and construction may profit the builder, but the owner will have high maintenance and repair costs\(^{(26)}\). Economy cannot be measured purely on initial expenditure, but on the total costs of keeping a structure usable\(^{(3)}\).

2.3.3 Effects of Deterioration

What are the effects of concrete deterioration? The physico-chemical effects are well understood and are easily analysed, but what about the socio-economic effects. A structure does not have to collapse and fall down to be a failure\(^{(30)}\). The main rival to concrete in the construction industry is steel, even though it is recognised that unprotected steel deteriorates under normal exposure conditions, whereas good concrete doesn't\(^{(28)}\). This has led to steel being specified with exceptional measures being used to protect it\(^{(92)}\). Both vary in popularity, due to changes in design style and cost. Deterioration does not appear to have affected the image of concrete, beyond making people more aware of the problems, so paying more attention to avoiding them in future.

The financial impacts of deterioration on clients are hopefully leading to greater initial investment, and more attention to regular inspections and maintenance to identify and stop the problems at an early stage. The latter also applies to existing concrete which may be neglected if it is not realised that the cheap option is to check it now rather than have to repair it later\(^{(6)}\).
3 CONCRETE REPAIR

3.1 Historical Background

Concrete repairs have been carried out for many years. Records of repair date back to the 1930s when guides were published for specific areas of repair\(^{(93-4)}\). As the problems of concrete deterioration have become more well known, so approaches to repair have improved by using this increased understanding and improvements in materials.

In the early days concrete or mortar were used exclusively for repairs\(^{(95)}\), and even up to the 1960s they were still the dominant materials\(^{(6)}\). However, since the early 1950s, it has been known that certain materials could be used to improve the properties of concrete repair materials\(^{(95)}\). This has led to the introduction of more advanced solutions for concrete repairs, with the use of alternative materials, to give more resilient repairs.

Unfortunately very little has been reported of success or failure of repairs, so there is a lack of information in this area\(^{(96)}\). The need to record successes and failures is not new\(^{(45)}\), but in recent years it has grown to meet the growing repair industry. Many reports have been published in the last decade, covering case studies of repairs involving different materials and methods\(^{(97-105)}\). These can serve to complement and confirm some of the recommendations made in some of the more general reports on the subject\(^{(8,31,46,106-16)}\), which are considered by some to be too general for detailed recommendations.

Even so, the different materials and methods still require long periods of time to gain general approval, as repairs have to prolong the life of the structure. This creates a reluctance to readily accept any new methods\(^{(7,8)}\), and introduces problems of knowing what to specify, and having to experiment with various systems to assess which are the most appropriate in different situations\(^{(117)}\). This combined with various disagreements about the virtues of different methods and materials makes the subject one with a great deal of
potential for future research\(^{(118)}\).

### 3.2 Materials and Methods of Repair

Various methods and materials are available for carrying out repairs, depending upon the type of deterioration. Johnson\(^{(6)}\) has considered repairs in three categories as follows: crack repairs; repairs to spalling and disintegrating concrete; and repairs to floors and pavements. Deterioration of floors and pavements frequently consists of spalling, so this will not be discussed independently. Repairs to cracks, and repairs to spalling and disintegrating concrete will be discussed in more detail with reference to modern techniques, as well as repairs to fire damage. It is important to note that in practice, more than one type of repair may be necessary\(^{(96,119)}\).

Whatever the types of repair employed, they must be both physically and chemically compatible with the concrete and with the basic concepts of the structure\(^{(43)}\). To do this they must fulfill certain requirements\(^{(120-1)}\):

i) they must be thoroughly and permanently bonded to the existing concrete;

ii) they must prevent moisture reaching the existing concrete;

iii) they must be free of shrinkage cracks;

iv) they must be frost resistant, where this is a factor; and,

v) they must be of good matching appearance to the surrounding concrete.

In addition, where the deterioration was caused by reinforcement corrosion, it will be necessary to restore protection to the steel either by the use of a suitable thickness of the repair material, or by a protective coating\(^{(122)}\).

The steps involved in carrying out a repair are essentially the same regardless of the actual type of repair. These steps have been described by Higgins\(^{(89)}\), and can be summarised as follows:

i) remove all deteriorated concrete;

ii) clean reinforcement and add additional reinforcement as necessary;

iii) prepare the surface to receive the repair;

...
iv) apply the repair; and,
v) apply protective coatings as necessary.

The different types of repair do however involve different approaches to these steps, and may involve different materials. It is necessary therefore to examine these repair methods individually, because the repair must not only be designed around the cause of the deterioration, but it must also take into account the design and construction of the structure[8].

3.2.1 Crack Repairs

The cause of any cracking should be established before repairs are carried out[8]. Cracks can only be successfully repaired if they are not active. If a crack is still active then sealing it will only result in cracking appearing elsewhere to relieve the stresses which caused the crack originally. If cracking is due to an internal cause (for example reinforcement corrosion), then sealing the cracks will merely allow the real problem to become worse unseen[112].

A number of alternatives are available for repairing cracks. Some can be used to restore the structural integrity of the cracked concrete if this is a problem, whilst others can be used to restore the water tightness or the aesthetics of the structure. Higgins[123] has suggested the use of resin injection, vacuum impregnation, and stitching to restore structural integrity, with some coatings as a means of improving appearance. In addition various suggestions are made for preventing moisture ingress to reinforcement, and for restoring water tightness; these include resin injection, vacuum impregnation, polymer emulsions, bandaging, and some types of surface coatings.

The Concrete Society[110] recommends the use of low viscosity epoxy resins for structural crack repair, while acrylic emulsions and resins can be used for crack sealing. The different materials and their uses have been described by Shaw[95].

Cracks are commonly sealed using epoxy resin injection. The standard method used involves a number of stages[112,119].
The concrete surface is cleaned of all dirt and dust and the cracks are blown out using compressed air. The crack is then sealed at the surface using an epoxy mortar, with injection ports stuck over the cracks at intervals. Once the sealant has set, resin is injected into the lowest port on the crack, until it overflows from the next highest. The lower port is then closed off, and the process continues from the next port until the whole crack has been sealed. Once the resin has set, the mortar and injection ports can be removed from the surface to give a smooth finish. The surface can then be given a uniform appearance by applying a coating.

Repair of active cracks has to take into account any movement that may take place. This can be achieved by making the crack into a movement joint, particularly if the original cause of cracking was due to inadequate movement joints; alternatively, the crack can be repaired by bandaging or by the use of flexible surface coatings\(^{6,123}\). A number of commercially surface coatings are available which are claimed to seal and waterproof concrete without the provision of movement joints\(^{124-5}\).

3.2.2 Repairs to Spalling and Disintegrating Concrete

Repairs to spalling and disintegrating concrete have been carried out in a variety of ways, depending upon the cause and severity of the deterioration. These have been dealt with in some detail by Johnson\(^6\). Much of the recent literature on the subject is related to deterioration due to reinforcement corrosion, so this method of repair will be described in some detail. The method can be applied to other causes of deterioration, but as in all cases of deterioration, the specific cause of the deterioration must be established first.

3.2.2.1 Repairs to Concrete Damaged by Reinforcement Corrosion

These repairs can be carried out using concrete or mortar depending upon the size of the area to be repaired. The repair material can be either a conventional cementitious mortar, a modified cementitious mortar, or a non-cementitious mortar. There is some debate about the relative merits of these different materials; this is discussed in more detail in Section 3.3.2.1.
As the cause of damage is reinforcement corrosion, it is essential to check and clean the reinforcement and replace it if necessary. Additional reinforcement can be spliced, welded, or grouted into place\(^{126}\). It may in some situations be more economic to use non-steel reinforcement such as plastic mesh or fibres, to give extra strength whilst reducing cracking and weight, and avoiding the need to add extra cover\(^{105}\). Preparation of the steel is also an area of some debate, this is discussed in more detail in Section 3.3.2.3.

As with crack repairs, the most commonly reported method of repair involves a number of stages. The first step is the preparation of the subbase to provide a firm foundation for the repair. This requires the removal of all loose material, cutting back to sound concrete, leaving an uneven surface to provide a good key for the repair material. In addition, the edges of the affected area must be cut back to a depth of at least 10mm to avoid feather edging of the repair material\(^{117,119,120}\). Cutting back can be achieved either using percussion tools, or by high pressure water jetting. This is discussed further in Section 3.3.2.2.

Once the subbase has been prepared, the repair material can be applied. The initial application may be a bond coat to improve the bond of the repair material to the substrate. This is an area of some debate which is also discussed in more detail in Section 3.3.2.2. Most commercially available systems are 'multilayer systems', which recommend the use of bond coats and also recommend that the repair material is applied in several layers, allowing each layer to become partially dry before applying the next\(^{127-9}\), although it is thought that this may increase the risk of bond failures due to the extra interfaces within the repair\(^{117}\).

On completion of the repair, a surface coating can be applied to improve appearance, and prevent further deterioration. With most repair systems, this is an integral part of the repair\(^{127-9}\).

If large volume repairs are required, it is generally recommended that the repair material should be concrete cast in situ against the existing concrete\(^{112}\). The same rules apply to preparation as for mortar repairs. In general, it is
recommended that thicknesses in excess of 75mm should be repaired with concrete\(^{(95)}\), although with good mix design and the use of plasticisers, concrete can be used to repair thicknesses as low as 40mm\(^{(113)}\).

With large volume repairs it will be necessary to consider the structural significance of removing large volumes of concrete and supporting large volumes of wet concrete and formwork. Where it is considered necessary, propping of structural members needs to be provided to give additional support while the work is being carried out\(^{(89)}\). Where the repaired member has to carry some load, then it will be necessary to relieve the load on the structure until the repair is complete, otherwise none of the existing load will be carried by the repair\(^{(126)}\).

Formwork needs to be designed in such a way that the concrete can be poured and easily compacted without too much vibration, as this may be difficult and could affect the seal of the formwork against the existing concrete. 'Flowing' concrete has proved to be advantageous in such cases\(^{(99)}\).

This method of repair can also be adapted to suit the larger areas of damage which may occur for example in slabs and walls of structures where delamination, or abrasion are common forms of deterioration. Preparation is the same as for patch repairs, and repairs can be carried out using mortar or concrete. A popular method of application for these types of repairs is pneumatically applied mortar or concrete\(^{(117)}\) (variously referred to as sprayed concrete, shotcrete or gunite). Examples of this include the Camsley Lane Viaduct repair\(^{(105)}\), and the repair and strengthening of cooling towers\(^{(130)}\). Sprayed concrete or mortar may be applied in one layer, and may incorporate additional reinforcement either as mesh fixed to the subbase, or as fibres included in the sprayed on mix\(^{(96,131)}\). This is one case where bond coats are not recommended\(^{(31)}\), and indeed may not survive the rigours of the application method\(^{(98)}\).

3.2.2.2 Protective Coatings

Most repairs are given coatings both for aesthetic reasons and for protection. The protection can be extended to the whole
structure, even if repairs have been localised. Repaired concrete is visually improved by the application of coatings or renders\(^{(132)}\), but the aesthetic requirements vary depending upon the position of the repair and the type of structure\(^{(119)}\).

Coatings used for protection must block the deterioration processes which caused the deterioration originally, and they must themselves be resistant to deterioration\(^{(119)}\). Conventional methods for protecting hardened concrete have been discussed in some detail by Johnson\(^{(6)}\), and Bicsok\(^{(33)}\). These techniques deal largely with protection of the concrete from chemical and physical attack, but may also be useful in protecting the reinforcement. Recent work has concentrated on reducing the ingress of moisture and acidic gases, in order to counter the effects of reinforcement corrosion brought about by the loss of the passive protection to the steel \(^{(9,76,133)}\).

As already mentioned, commercial repair systems usually include the application of a protective coating as an integral part of the repair process\(^{(128)}\), and they can be specified as part of a repair\(^{(119)}\). Coatings vary depending upon their viscosity and the manner in which they provide protection. They can be classified as penetrants\(^{(125)}\), sealers\(^{(128)}\), coatings\(^{(124)}\), or renders\(^{(124)}\), with increasing viscosity. These have been discussed in some detail by Leeming\(^{(9)}\). The most important factor, as with repairs is that the surface to be treated must be free of all contaminants and laitance, otherwise the coating may not adhere properly to the surface\(^{(9)}\).

### 3.2.3 Repairs to Fire Damaged Concrete

Fire damage of concrete is similar in many respects to other types of deterioration. As a result the same procedures can be applied to preparation and repair of fire damaged concrete as to other types of concrete repair\(^{(48)}\). However, there are several important factors to bear in mind when assessing the damage, as fire damage may be more substantial than it appears, and the loss of structural integrity may be a problem. Reinforcing steel may look sound, but it may have been affected by the heat. If this is the case then additional reinforcement must be incorporated into the repair. Concrete must be cut back to a sound base that was not affected by the fire. An initial idea of the extent of the damage may be obtained by 'sounding'
the concrete with a hammer, but a more thorough investigation must still be carried out. This may include using materials which give an indication of the temperatures that members were subjected to during the fire. This may prove useful for assessing the potential extent of unseen damage. Repairing fire damaged structures is still a very specialised part of a specialised industry, and each case must be considered separately.

3.3 Discussion of Concrete Repair

3.3.1 Current Extent of the Concrete Repair Industry

Concrete repair is a growing part of the construction industry. There are two likely explanations for this.

Firstly there is a greater amount of deterioration, because there is much more concrete, and the amount is growing. The problems are amplified by changing properties of materials and methods, especially in concrete construction. Unfortunately, much concrete was made in the belief that it is a durable material, even though the factors which influence concrete durability have been appreciated for some time. In essence this is only if the design, detailing, material selection and construction standards are high.

Changes in cement properties and construction techniques may also have contributed to the problem by removing some of the passive protection which was provided by the 'over design' necessary with the materials originally.

Knowledge of current problems and attempting to foresee future ones should prevent a recurrence of present problems, but this cannot change the existing concrete which is already deteriorating, or which could deteriorate. It is fortunate that a lot of poorly made concrete still survives in a reasonable condition. Unfortunately, many of the problems suffered today could not be foreseen at the time, so the structures could not be designed to resist them; so this does not bode too well for the future.

A second explanation for the growth in the concrete
repair industry is the high cost of construction which now makes repair and renovation a viable alternative to demolition and reconstruction\(^{139}\). This fact is gradually being more widely acknowledged, together with the realisation that repair costs can be reduced by correcting problems early, so minimising the amount of repair that has to be carried out. As Johnson states, 'problems of deterioration are not different from medical problems, if you catch them at an early stage, they are easier and cheaper to fix, and there is room for the exercise of ingenuity'\(^6\).

Whatever the case, current and future users will have to consider the costs to determine the best option available to them. They will also have to consider the cost of maintenance as part of the running cost of a building and see it as an integral part of their investment rather than something to be avoided\(^3\).

3.3.2 Points of Dispute Relating to Concrete Repair

As the repair industry has grown, so have opinions grown about particular issues particularly if little or no serious research information is available. These issues need to be resolved in order for reliable methods and materials to be used to the full to give reliable repairs. Three major areas of debate have been highlighted from an examination of the relevant literature. These relate to the most suitable materials, the most appropriate methods of base preparation, and the most appropriate method of steel preparation.

3.3.2.1 Materials

Some debate exists about which are the most suitable materials for repairs. Opinions differ over the choice of conventional cement mixes, modified cement mixes or non-cementitious materials.

Conventional materials, mortar or concrete, are self explanatory. They have their advocates who suggest a repair 'like with like' policy should be employed\(^{104}\). They have the advantage that they can easily be made like the original material\(^6\), and have proven qualities\(^{109}\).
Modified cementitious materials use cement as a binder, but are modified by the use of additives or special cements to improve their properties, making them more suitable for repairs. Most commercial repair materials use these, and have additives to reduce absorption and increase chemical resistance, as well as to reduce the setting time. In addition, they have low slumps to enable them to be placed on vertical surfaces and soffits without the use of formwork. Cements modified by the addition of polymers have improved compressive and tensile strengths, water absorption, freeze-thaw resistance, and resistance to acids and abrasion, but unfortunately insufficient data is available on their use. Different types of polymers can be used to obtain different properties. These have been discussed by Shaw. Rapid strength gain for repairs can also be obtained by using rapid hardening cements. Ultra Rapid Hardening Cement (Jet cement) has been used to great advantage in some situations.

Non-cementitious materials are chiefly resin based, although some success has been reported for mortars and concretes using sulphur as a binding material. The most common use of resin systems is for hard wearing floor screeds, cracks and small patch repairs. There may be situations where resins are more suitable than conventional materials, for example if the cover to the reinforcement is inadequate, and it is not possible to increase the it, then the resin will protect the steel by encapsulating it rather than by chemical passivation. The resin industry suggests that it may be able to offer solutions to many repair problems and some effort has been made to prove their suitability and ability to perform as repair materials.

The principal problem with all of these alternatives is that they are more expensive than conventional materials. Jet cement may cost five times more than normal cement, but it has been suggested that the extra cost can be overlooked as material costs are only a fraction of the total repair cost.

More importantly, the new materials have not been tried and tested for long enough to prove that they can work successfully over long periods; further research is...
necessary in this area. At present there is little evidence in favour of the new materials and those who are responsible for the decisions to use a new product are the engineers who have to use or specify them\(^{117}\). Evidence to date suggests that repairs made with modified and non-cementitious materials are not durable, but those made with conventional material are\(^{148}\).

It is probable that modified materials will eventually be accepted, once enough evidence has been produced to show that they are a reliable alternative to conventional materials. Cement based materials are more likely to be accepted in preference to resin based alternatives because of the cost and because of the thermal and physical compatibility with the base concrete\(^{138}\).

### 3.3.2.2 Base Preparation

Base preparation is considered to be the most important single factor contributing to the success or failure of a repair\(^{95,117,149}\). Differences in opinion relating to base preparation are in regard to cutting methods and use of bond coats.

Concrete can be cut using percussion tools or by high pressure water jetting. Both methods can remove deteriorated concrete, leaving a sound surface for the repair material to be attached to\(^{115,117}\).

High pressure water jetting is increasing in popularity even though it is rather specialised, whereas percussion tools are widely available and can be used by most site operatives. Water jetting has much in its favour, especially for concrete repairs, because it is quiet, clean, and safe. Although it requires high water pressures (3000-10000psi), the quantities of water used are low (around 50 l/min). The pressure can be adjusted to suit the particular concrete, so only loose or weak concrete is removed, and experiments have been carried out using low pressures to see how effective these are\(^{150}\). The water removes any deposits from the steel leaving it clean\(^{115}\) and the concrete, once cut, is clean and damp ready for the repairs to be applied. For cutting sound concrete and reinforcement, abrasives can be added to the water at the jet nozzle\(^{151}\), and this can be used to make sharp accurate cuts at
the edge of the repair area\(^{152}\).

In contrast percussion tools are noisy and produce a lot of dust. They can damage sound concrete if it is not very strong, the sensitivity of the operator being the controlling factor. The steel requires additional cleaning once the concrete has been cut away, and the subbase is left dry and dusty, so it needs separate cleaning as well.

Arguments against water jetting are based on its specialist nature and greater cost. There is some concern about the problem of reaction forces that operators must resist\(^{117}\), although these can be overcome by the use of a suitable restraint for the equipment. Another foreseeable problem is that of flying debris which could make an enclosed working area essential for normal site operations\(^{152}\). This may make it impractical in practice, by restricting other operations, so slowing down the complete repair operation\(^{105}\).

A widespread use of high pressure water jetting in the future is unlikely, as general contractors carrying out repairs are still likely to favour using percussion tools. However, specialist repair contractors will want to use the best equipment available for the job, and high pressure water jetting may well prove to be the most suitable for repair work.

The second area of debate, the use of bond coats, is not as clear cut as the arguments for and against the different methods of cutting the concrete. As yet there appears to be very little evidence to support the general use of bond coats, even though most technical reports recommend them\(^{107,110,111}\). Very little research work has been reported, and much of the work which has been done suggests that there is little or no benefit from their use\(^{153}\), although they may be useful as barriers to prevent chlorides from the base concrete migrating into the repair\(^{117}\). However, there is some evidence that certain bond coats do aid adhesion of the repair material, and that they can provide long lasting repairs\(^{121,154}\).

The main argument against the use of bond coats is that they are an additional process which, if it goes wrong, can affect the entire repair. The alternative is to have a damp surface to apply the repair material to (though some bond coats
have to be applied to a damp surface anyway\(^{(129)}\), which has the advantage that if it dries out it can always be rewetted. If a bond coat dries out however, it may have to be removed, and a new coat applied, although there are some bond coats which can be allowed to dry out\(^{(132)}\). Until more information about the subject is available it will remain a subject of dispute. A likely outcome will be the introduction of more reliable and easier to use bond coats.

3.3.2.3 Steel Preparation

The main areas of debate centre around whether to expose the reinforcement completely, and whether to use rust inhibitors on the cleaned steel\(^{(96)}\).

The first of these appears to be a matter of judgement rather than scientific debate. In some instances, the concrete behind the steel may be sound, whilst in others it will have to be cut away. There may be good cause for removal of concrete from around the reinforcement, to expose hidden corrosion for example, or to provide a better key for the repair. Commercial repair systems recommend it\(^{(127-9)}\), although they have a vested interest in the use of more of their product to fill the hole. It must also be remembered that the concrete within the reinforcement cage may be sound, and may require a great deal of extra effort to remove it\(^{(117)}\).

There is also some debate about how to clean the steel after it has been exposed. Some recommend that it should be cleaned to a shiny finish\(^{(155)}\), while others only recommend the removal of loose rust and scale\(^{(127-9)}\). It is generally recognised that if a mechanical method is used to clean the steel (grit or shot blasting, or needle gunning), then the finish will be satisfactory\(^{(89)}\). It has also been recognised that water jetting is very effective in cleaning the steel\(^{(115,117)}\).

The second area of debate concerns the use of rust inhibitors, though this only of concern if an unmodified repair material is going to be used. Arguments in favour of rust inhibitors suggest that they will prevent further corrosion which could otherwise stop the repair from working. This may be especially true in chloride contaminated concrete\(^{(89,100)}\). It has
been suggested that inhibitors prevent problems arising from inadequate cleaning of the steel, as some rust inhibitors can be applied to steel that has only been lightly cleaned\(^{132}\).

Arguments against inhibitors include the suggestion that they increase the likelihood of shoddy workmanship\(^{89}\), and may also result in an increase in corrosion in other parts of the reinforcement by setting up potential differences\(^2\). They may also result in a loss of bond with the repair material if they are not compatible with it.

3.3.3 Cost of Repair

It is very difficult to quantify costs of repairs, as each repair is usually costed individually. Publicised contracts have ranged in value from £80-200K, many of these are for both assessment and repair\(^{99,102-3}\). It is true to say that the main cost can be attributed to gaining access to carry out repairs, scaffolding and lane closures\(^{117}\). The cost of the materials may be only about 1% of the cost of a medium sized repair contract\(^{26,147}\).

Since the total extent of repair work is usually difficult to assess, many contracts are priced on a 'cost per item' (labour, access, materials, etc.) This gives the client the option of stopping the work once a certain cost limit has been reached\(^{117}\) although it has the disadvantage that the work may be incomplete when this limit has been reached. The Concrete Society\(^{110}\) has recommended this approach to tendering and some contractors agree with this method\(^{149}\), although some prefer to give a price after assessing the deterioration and designing a repair to suit\(^{96}\). One of the problems with this second method is that different methods and materials cannot always be easily compared, so it may be difficult to assess the relative merits of different tenders\(^89\).

Repair costs will continue to rise. Four years ago, about 40% of annual expenditure in the construction industry was on repair and maintenance, but this was expected to rise to about 60%\(^{26}\). As a result, anyone who is responsible for a concrete structure must consider regular inspection and maintenance as worthwhile expenditure, since leaving a structure until a problem reveals itself may result in much larger repair bills.
Ultimately, the cost of repair should be considered in the cost of construction, at which time avoiding the need for future repair may well prove to be cheaper than cutting corners to reduce the initial cost. Prevention of deterioration is discussed further in the next chapter.
4 PREVENTING DETERIORATION IN CONCRETE

4.1 Current Practice

An assessment of most problems of concrete deterioration would lead to the conclusion that prevention is better than cure and prevention is possible with foresight. Practically there are two ways of preventing problems in concrete, by better design, and by better site practice. This knowledge is by no means new, but deserves repeating, as it is frequently overlooked. At all stages it is essential to recognise that concrete is not merely a material, but a complete process, and one which is becoming more important in construction (156).

To avoid deterioration, it is necessary to ensure that the specification is correct, and adhered to, and that every part of the industry recognises its responsibilities in this.

4.1.1 Prevention at the Design Stage

Concrete and concrete structures are designed in accordance with relevant codes and accepted practice, which unfortunately place very little emphasis on the long term durability of the material. Emphasis at the design stage is on strength (157-8), with little or no reference to durability. Even modern approaches to specifying for more durable concrete concentrate on the cube strength as an indicator of durability (159), although if this approach is used sensibly, it could prove quite useful (160).

Consideration is only usually given to durability if an aggressive environment is present, and then codes of practice do have special recommendations for protection (158). Concrete can be designed to resist many conditions without deteriorating (28), though only in recent years has a greater awareness of the lack of durability resulted in efforts to produce more durable concrete. This has been done by implementing more severe guidelines in the codes of practice (38), but there is still some doubt about the adequacies of the modern codes (161).
In addition to improving design procedures, there are many alternatives to traditional concrete which have better characteristics and give improved performance. These need to be publicised more in design offices and greater use needs to be made of them. These include the use of cement replacements, alternative aggregates and fillers, and chemical admixtures and additives. These are discussed in more detail in Section 4.2.

4.1.2 Prevention on Site

This is one area of great concern, and certainly one which requires some change.

Because of the nature of the construction industry, most contractors organisations have to operate on very low profit margins. Unfortunately this may be reflected in the site practice, because as far as possible savings will be made by cutting corners, and reducing expenditure. This can affect concrete quality quite considerably. Concrete is cheaper if less cement is used, adding more water makes it more workable, so placing can be made easier. By stripping formwork early and cutting curing times, work can proceed faster on the next section of the project. Care in fixing reinforcement also takes time and commercial spacers cost money.

All of these procedures are detrimental to concrete and though it is unrealistic to suggest that they occur on every site, carrying out just one of the short cuts mentioned may be enough to cause significant reductions in the concrete durability. Curing is considered to be essential to reduce the permeability of the concrete\(^{162-3}\), this has been related to rates of carbonation\(^{164}\), especially in arid climates\(^{165}\). The w/c ratio also affects the permeability and determines whether or not the capillaries in the concrete will be sealed if it is cured properly\(^{166}\). Mixing, batching, transporting, placing, finishing, and curing have all been shown to affect the ability of the concrete to resist the effects of freezing and thawing\(^{52}\).

Unfortunately contractors can rarely be made liable for material failures, unless they occur soon after construction. It must be the site engineer or his staff, who are responsible for ensuring that good practice is used and this can only
happen if the engineer is aware of what is good practice.

4.2 Improvements in Materials

Concrete materials are being continually developed to meet different requirements and economic pressures. In recent years, significant developments have taken place in the field of cement and cement replacements, which in some cases can also be used as aggregates, or fillers. New admixtures and additives have opened up the possibilities for new concretes, extending the range of practical mix designs and improving the material properties. Other developments to enhance resistance to reinforcement corrosion are alternatives to conventional carbon steel.

4.2.1 Cements and Cement Replacements

The basic constituents of Ordinary Portland Cement (OPC) have not changed much since it was first introduced in the 18th century\(^{166}\). However, improvements in the manufacturing process have led to cements with higher early strength gains that are more refined\(^{167}\). Other types of cements have been developed from OPC, or been developed independently such as rapid hardening cement, sulphate resisting cement, and high alumina cement. The main differences in these being the proportions of the reactive constituents.

Some of the modern deterioration processes result from the use of modern cements. The high early strength gain and higher ultimate strength than their predecessors means that modern cements can be used in smaller quantities, and at higher w/c ratios than was previously possible, to achieve the same strength. In addition formwork stripping times are reduced so work can proceed faster. This is advantageous, but also has some drawbacks. Less cement means less resistance to processes which attack cement. More water means more capillaries when the concrete dries out. Early stripping times means early exposure of the concrete to the elements, and possibly less attention given to curing the concrete after stripping.

This does not mean that modern cements are not suitable for making concrete, rather that they need to be used with
greater forethought, and better site practice, to ensure a good end product. Concrete can be improved further and can often be made more economically by the use of replacements. These can be considered as alternative binders or partial cement replacements.

Alternative binders include sulphur\(^{142}\), epoxy resins, and polymers. Polymers can be included in the concrete in three ways, as polymer concrete, polymer portland cement concrete, or polymer impregnated concrete\(^{146}\). These have different properties and different applications, and are discussed in some detail in the Concrete Society's report on polymer concrete\(^{168}\). These are usually too expensive for normal structural use, but find applications in special situations where normal cements are inadequate such as repairs and protective screeds.

Some partial cement replacements are pozzolanic which means that they are 'sileceous, or sileceous and aluminous materials, which in themselves possesses little or no cementitious value, but will in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties'\(^{169}\).

Natural pozzolans were used in Roman times because of their cementitious properties, but only recently has their use as partial cement replacements begun to be appreciated. The immediate advantage of pozzolans, as well as being cheaper than OPC, is that they hydrate slowly, thus there is low heat development, which is important in mass construction\(^{170}\).

Common pozzolans in use today are Silica Fume, a by-product of the silicon metal industry, and Pulverised Fuel Ash (PFA) or Fly Ash, a by-product of the electricity generating industry. Both of these materials exhibit cementitious properties in the presence of free lime, and they are also by-products of carefully controlled processes, which means that the quality of the materials is very consistent. The result of using these in an OPC mix is that much of the free lime is converted to stable calcium silicate hydrates. This reduces the problem of leaching and it also improves the properties of the concrete with time, improving the strength,
reducing the permeability, and, because the reaction is slower than OPC, reducing the heat of hydration.

Another industrial byproduct which possesses cementitious properties is ground granulated blastfurnace slag, which is a product of the steel industry. This is formed by cooling the molten slag rapidly to form glassy granules which are then ground to produce a cement like powder\(^{171}\). This is also the product of a carefully controlled process, so as with microsilica and PFA, it is a high quality product. It is not a pozzolan and although it has very similar properties to OPC, it cannot be used as a cement on its own because it has to be activated. This can be done using OPC, and its use in Portland blastfurnace cement is well known\(^{172}\).

It has been shown that all of these materials can improve the resistance of concrete to deterioration in a number of ways. Microsilica in a slurry form acts as a lubricant in the wet mix, so it can be used to give good workability in concretes with low w/c ratios\(^{173}\). It reduces the possibility of attack from alkali aggregate reactions\(^{174}\), and has numerous other benefits resulting from its very fine particle size\(^{175}\). PFA, and ground granulated blastfurnace slag both reduce the ingress of chlorides from de-icing salts\(^ {176}\), as well as having many other beneficial properties\(^ {177-79}\). In addition, by using very large cement replacements, mass construction where strength is less important than durability can be carried out as if a lower strength cement was used\(^ {180}\).

Use of these materials requires careful design and good quality control on site, to ensure a good end product which can compete with conventional concrete. As a result, use of these materials to date has resulted in a high quality product.

4.2.2 Aggregates and Fillers

Aggregates and fillers used in conventional concrete are usually natural sand and gravel deposits, or graded crushed rock. Natural fillers such as rock flour or ground sand can be added to increase the amount of fines and reduce permeability\(^ {33}\). Finely divided powders resulting from industrial by-products are frequently used as cement replacements, these have been discussed in the previous
section. It is possible that the beneficial effects from these materials may result partly from the fact that they act as fillers, as well as having cementitious properties.

Anything can be used as an aggregate, as long as it is stable in the environment it is used in, and is readily available in the required quantities. Numerous alternative aggregates are available in different circumstances, which can be both lightweight and economic for example broken concrete or brick, cork, coconut piths, rice husks, rice husk ash, or even broken glass\(^{181}\).

Materials which have shown some potential are lightweight aggregates such as granulated blastfurnace slag, which gives concrete without the weight limitations of conventional concrete. If great strength is not required, expanded polystyrene balls may be an alternative\(^{182}\), or foamed concrete using air as the aggregate\(^{183}\).

4.2.3 Admixtures and Additives

An admixture is defined by the Cement Admixtures Association\(^{184}\) as 'a material other than water, aggregate or Portland cement, which is added to a batch of concrete, mortar or grout during or immediately before mixing, in order to extend the properties of the concrete and/or make it more economical'.

These can be used to improve a concrete's resistance to deterioration. It has been shown that using superplasticisers can reduce permeability and chloride diffusion coefficients, by reducing the w/c ratio for a given slump, or by improving compaction at a given w/c ratio\(^{185}\), (although this may produce some reduction in strength). Additives can also be employed to reduce water ingress into the hardened concrete\(^{186}\), or to eliminate it altogether\(^{187}\). They can also provide protection against reinforcement corrosion\(^{188}\), and some are claimed to improve resistance to most types of deterioration\(^{189}\).

4.2.4 Reinforcement

As reinforcement corrosion is one of the major causes of deterioration, it is only natural to consider alternatives to
carbon steel, or ways of reducing corrosion in carbon steel. Carbon steel is usually adequate for normal structural concrete. The principal disadvantage of carbon steel in corrosive environments is that the rate of corrosion is higher than other metals, because the magnetite produced in the corrosion process is a good conductor, so corrosion is not stopped by it\(^{(73)}\). Special types of reinforcing steel are available for extreme conditions, and they may be used as an added precaution against reinforcement corrosion in normal exposures. Alternatives to carbon steel, are stainless steel, galvanised steel, and epoxy coated steel. These all have advantages over conventional reinforcement, but are not without their limitations. Alternatively, the steel may be protected against corrosion by electrical methods such as cathodic protection, or by electrochemical removal of chlorides from the concrete.

4.2.4.1 Stainless Steel

Stainless steel with a chromium content of more than 11.5% will not corrode in any normal exposure where concrete would be used, and any damage is self healing\(^{(73)}\). Its use as rebar does have some disadvantages when compared with carbon steel. It is more expensive, about six times the price of normal rebar, although it can be argued that the steel only makes up a small proportion of the cost of a structure. This may be considered to be an acceptable extra cost, in view of the added protection\(^{(90)}\). Stainless steel in contact with other metals, including carbon steel, will create electrical corrosion cells\(^{(91)}\). This will apply to ties, links, and other fittings made from other metals, so this must be considered if stainless steel is used as the main reinforcement.

4.2.4.2 Galvanised Steel

Galvanised steel is also more expensive than carbon steel. It has very good corrosion resistance and it has been shown that in carbonated concrete and in chloride contaminated concrete with chloride levels below 1%, there are tangible benefits when using it\(^{(190)}\). Studies in Bermuda have demonstrated the advantages over long periods of immersion in sea water, where untreated steel would be expected to show severe corrosion\(^{(191)}\). The coating is self healing as long as it is not penetrated.
completely. However once damage occurs microcells in the vicinity of the damage will result in a pit forming in the underlying steel, which in turn leads to further damage of the coating\(^{(192)}\).

4.2.4.3 Epoxy Coated Steel

Epoxy coated reinforcement is a relatively new idea. It was introduced in North America in the early 1970s as a precaution against de-icing salt induced corrosion\(^{(193)}\). Problems with adhesion of the coating, and reduced bond with the concrete have been overcome, by using improved formulas. The current method used is fusion bonding, although there is still some doubt about the bond if the bars are bent after coating, and no reliable method has been found to control or measure the coating thickness. Any damage to the coating can lead to a total failure of the protection system, due to the coating peeling off, so in this respect great care needs to be exercised when handling it\(^{(194)}\).

Epoxy coated steel overcomes the problem of reinforcement corrosion\(^{(192)}\), and production and testing is described in the American standard ASTM D3963-86\(^{(195)}\). The cost of epoxy coated rebar is greater than that for plain carbon steel, with coated reinforcement at 1986 prices costing £700/tonne, including shipping costs, while uncoated steel cost £260/tonne\(^{(196)}\). However, recently manufacture has started in the U.K., so this should lead to a reduction in the price, and greater usage\(^{(197-8)}\).

The principal reason for a lack of use will still remain however, which is the different construction methods employed in this country and Europe from those used in North America. Use of bridge deck waterproofing in this country for instance makes this sort of protection unnecessary in these areas. However, it could find uses in unprotected areas such as bridge crossheads\(^{(199)}\). The possibility of cheaper epoxy coated steel in the future may make it a viable alternative to uncoated steel with a waterproof membrane, if it can be shown to be a reliable form of corrosion protection.
4.2.4.4 Cathodic Protection

An alternative approach to the use of different steels to avoid reinforcement corrosion is the use of cathodic protection to prevent and possibly reverse the corrosion effects. The principles are similar to those employed in steel structures, either as passive protection using a sacrificial anode or an impressed current to change the electro-potential of the steel so that the corrosion is reduced or stopped\(^{200}\). The application of cathodic protection has a dual effect; it protects the steel firstly by removing the oxygen from the steel, and then by increasing the pH to restore its passivity\(^{80}\). Because the method is still very much in its infancy for use in concrete structures, research is still being carried out to determine how useful it may be\(^{201-2}\). A number of attempts have been made in this country to apply it to repaired structures, using different impressed current methods, and employing careful monitoring to assess the effects it has on the steel\(^{203-6}\).

Cathodic protection has been criticised on economic grounds\(^{207}\), and there is concern that it may create problems such as a loss of bond between the steel and the concrete\(^{199}\).

4.2.4.5 Electrochemical Removal of Chlorides

Another preventive measure which has been suggested for chloride contaminated concrete is the electrochemical removal of chlorides\(^{208}\). In this system, a large direct current potential is applied between the steel and the concrete surface. The chlorides in the concrete are driven towards the concrete surface by the current where they can be immobilised. It has been found that not only are the chlorides removed by the method, but that some of the passive protection is restored to the steel. This method offers a cost effective way of extending the life of concrete affected by chlorides, and preventing corrosion in sound concrete.

4.3 Improvements in Preventive Measures

Prevention of concrete deterioration can be achieved in a number of ways. Successful preventive measures require a good
understanding of the problems. If the problems are understood, then most deterioration can be avoided. This understanding of the problems needs to be used to improve codes of practice and training at site and design office level; only then can better materials be employed to give the high quality concrete that the industry is expected to produce.

4.3.1 Improved Codes of Practice

Codes of practice can be used to aid improvements in the design process. In the U.K., the developments in reinforced concrete design have evolved to give the current BS8110. Durability is an increasing concern for the code writers, but continuing code changes and the lack of examples of durable concrete from existing codes makes it difficult to see if and where improvements have been achieved. Specifying concrete to prevent deterioration also requires a strict imposition of the recommendations of the codes. Unfortunately, to specify something is easy, to implement it may not be. The most appropriate changes that can be made to give more durable concrete are recommended minimum cement requirements, minimum cover, minimum curing periods and maximum w/c ratios, for particular environments.

Somerville has illustrated the various curing requirements that different codes from different countries recommend, for different exposure or durability requirements. These suggest broad categories of exposure and appropriate curing for different concretes under these conditions. Despite the differences and difficulties in comparison between the different codes, Somerville concluded that of those considered, BS8110 had the most stringent durability requirements.

4.3.2 Improvements in Site Awareness

Patterson has reported that over 50% of defects in construction occur at the construction stage. It is largely the responsibility of the construction industry and professional bodies connected with training to ensure that this is corrected, and that the training given is appropriate with the technology employed. As Neville states, 'Bad concrete, often of the consistence of soup, hardening into a honeycombed, non-homogeneous mass, is made simply by mixing cement, aggregate
and water. Surprisingly, the ingredients of a good concrete are exactly the same (170).

The current scare over concrete durability reflects a lack of understanding that good concrete requires good practice at site level. It only requires one step in the processes involved in making concrete to be done inadequately, for the concrete to suffer (2). It is unfortunate that some construction firms have no real interest in the long term durability of what they build. It must be seen as the responsibility of all concerned with concrete mixing, fixing, placing, and curing, that a good concrete is dependent upon them (4). Dewar (2) has highlighted the four 'C's as essential for durable concrete; these are constituents, cover, compaction, and curing. These must be implemented correctly at site level otherwise the concrete will not achieve the properties required to fulfill its specified purpose.

4.3.3 Improvements in Design Office Awareness

An awareness of shortfalls in concrete procedures on site should lead to better design office practice, which should eventually lead to better codes of practice. Knowing that cover may be lacking, extra cover can be specified. A good mix design, with good compaction properties can help a great deal. This should include consideration for compaction around reinforcement, and in complicated details. Complicated details should be avoided wherever possible. Curing is frequently overlooked, so a minimum period should be specified by the designer, and more thought should be given to programming to avoid the need to cut corners.

Special precautions can be taken where aggressive conditions are expected. The best approach would be to design with deterioration in mind. Allied to this, close cooperation and feedback from site personnel needs to be taken seriously, as well as good maintenance procedures laid down as part of a specification, to ensure that the concrete remains in good condition.

Other improvements can be made to prevent problems, by avoiding undesirable design features in the structure. Architectural details which may prove to be weaknesses include
spandrels and exposed aggregate finishes. Expansion joints and crossheads may provide places for aggressive agents to collect and poor drainage may make the situation worse, so special precautions need to be taken with these.

Finally the mistakes of the past must be taken seriously, to produce better structures today and better practices in the future.

4.3.4 Improvements in Materials

Many things can be added to concrete to make it more resistant to deterioration; some of these have been discussed elsewhere. It is important, however, not to see these as a means of avoiding good site practice. Materials that result in high quality concrete often demand a high standard of workmanship for example microsilica. All new materials need to be treated with respect, and recognition needs to be given to the fact that weaknesses are usually the result of improper use or specification\(^{210}\). The same remains true for conventional concrete. Hopefully the introduction of better materials will see a revolution in higher quality concrete, with possible uses beyond the capabilities of conventional concrete.

4.4 Cost Effectiveness of Prevention

It may be argued that a policy of prevention may not be cost effective, bearing in mind the unpredictability of failure and the long time scale that may be involved. After all, there are many structures which have so far shown no signs of durability problems. For new construction, it appears that within the industry there is still an element of the 'least cost philosophy', but most engineers would consider this attitude to be unacceptable\(^{26}\).

As well as comparing the relative costs of repair, it is also necessary to consider the relative merits of better materials with improved site practice, and the effects of greater awareness at design office level. These can all improve the image of concrete as well as the the long term properties, but in the short term the financial penalties may be restrictive.
4.4.1 Prevention versus Repair

In financial terms, it is undoubtedly cheaper to prevent deterioration, either at the design stage, or by planned maintenance, rather than to have to repair deterioration if it occurs. This assumes of course that it will occur. Inevitably situations could arise where an overcautious approach from the designer results in extra cost to avoid deterioration which would not occur anyway. It may be necessary to find a compromise between prevention and repair, and to determine how much usable life is required of a structure before repairs are necessary. In existing structures, effective repairs can be seen as a means of prolonging the useful life, so this may be considered as an acceptable alternative, for future structures as well.

Unfortunately, concrete is the loser in the public eye if it cannot be relied upon to be durable. The concrete industry will suffer in the long term unless it does all it can to restore public confidence in concrete as a material. Repair must be seen as the final solution when all else has failed. Preventative maintenance has to be a better alternative than large scale repairs. Without doubt, the image of concrete could be improved dramatically without excessive cost, by the use of preventative measures as detailed earlier.

4.4.2 Better Materials versus Better Site Practice

Again financial cost is difficult to quantify, although the use of both better materials and site practice will result in increased costs. To implement better materials could remove the need for better site practice and vice versa. From the industries viewpoint, implementing both can only be of benefit, if it results in better concrete. New materials show a lot of promise, and could improve the image of concrete by restoring the public's confidence in it as a structural and architectural material. Improvements in site practice will lead to improvements in concrete quality, but it will be a benefit that will be largely unseen, and could easily produce dramatic increases in cost by requiring constant supervision and quality checks on new materials.
4.4.3 Better Design Awareness

Improvements at design stage should not result in any increases in financial costs although undoubtedly they will. Indeed it is at this level that clients need to be informed of the measures being taken to protect their investment. This may result in a willingness to spend more money for a better end result, or a reluctance to spend money on unseen benefits. This will result in a better client awareness with recognition from the outset of the need to invest more on the basic structure. This must also be of benefit in the long term.
Designing a concrete mix, consists of selecting the correct proportions of cement, fine and coarse aggregate, and water to produce concrete having the specified properties\(^{157}\). Testing is the only means of assessing the concrete to see if it has the specified properties. Traditionally, testing has concentrated on both the wet mix, and the hardened concrete, as both can give indicators of whether the mix proportions are correct, and whether the concrete will perform as required. A lot of emphasis has been placed on cube strength as an indicator of concrete strength, this is clearly essential for design purposes.

Testing is still as important today as it was in the past, but the particular concerns of this generations' concrete technologists are different from their predecessors. Cube strength is still important but it has been realised that strength is only one property of concrete, and one which gives no indication of the concrete's resistance to deterioration. Of greater importance are the properties which relate to the deterioration processes which affect the concrete.

In addition to this, it has been recognised in recent years that samples taken from the wet mix, and treated differently from the concrete in the structure, may bear little or no resemblance to the concrete they are supposed to represent\(^{(2)}\). This has created a move towards testing samples taken from a structure, or actually carrying out tests on the structure. It should be remembered of course that numerous methods of measuring strength in situ have been reported\(^{(212-15)}\), but all too often, these have been disappointing when compared with the cube strength\(^{(213)}\).

5.1 Testing for Deterioration in the Laboratory

Laboratory tests can be carried out on specimens prepared in the laboratory, or specimens obtained from structures. The former are valuable for assessing the behaviour of different mixes, while the latter are of use when assessing deterioration in particular structures. Tests may be considered as those
which attempt to predict performance, and those which attempt to assess deterioration once it has occurred.

For predicting performance, a number of methods are frequently used. Long term tests and accelerated tests are used to assess performance in particular environments, and permeability tests are used to obtain a measure of a concretes resistance to deterioration, as it has long been felt that permeability affects the ability of the concrete to resist the ingress of aggressive agents\(^{216}\). To assess deterioration when it has occurred, chemical analysis and petrographic examination can be used to determine the cause of deterioration if this is unknown, and the current condition of the concrete.

5.1.1 Long_Term_Testing

Traditionally, where environmental conditions have been considered a problem, long term exposure tests have been carried out to gather information about the behaviour of different concretes under these conditions, for example sulphates in the ground\(^{217}\), or exposure to seawater\(^{218}\). The nature of these tests means that they must be carried out in the environment that is a problem, but the test pieces are usually made in the laboratory, and returned to the laboratory for assessment when the test is deemed to have finished.

These long term tests have a number of disadvantages. Aside from the length of time involved, which means that results are unavailable for years or even decades, there is also the problem that in that time, materials and methods will change, so relating the data obtained to new technology and practice will be very difficult\(^{219}\). This then makes accelerated testing a more valuable tool, if it can be applied to the problem.

5.1.2 Accelerated_Testing

Accelerated tests may be used to simulate the environmental conditions that a concrete will be subjected to, but without the time penalties\(^{220}\). The important feature of these tests is that they concentrate the important mechanisms involved in the deterioration process that is of concern. They can be applied to physical or chemical deterioration, and may be used to
assess the behaviour of a particular concrete in an environment, or a number of different concretes to determine which is the most suitable.

Physical tests include assessing the freeze-thaw resistance, by rapid freezing and thawing under very extreme conditions\(^{(221)}\). Abrasion resistance can be determined by repeated passing of weighted wheels over a test piece, or the surface in question. This can be useful for assessing surface treatments for concrete\(^{(222)}\). Problems with alkali aggregate reactions can be avoided by accelerated testing to see if suspect aggregates are reactive\(^{(223)}\).

Chemical attack can be tested by increasing the concentration of the chemicals involved (acids\(^{(40)}\), seawater\(^{(218)}\), or sulphates\(^{(224)}\)). Susceptibility of a concrete to carbonation can be assessed by using high concentrations of CO\(_2\) in a controlled environment\(^{(225)}\), and tests have been carried out to accelerate reinforcement corrosion for the same purposes\(^{(226)}\).

Accelerated tests are not without problems. Results obtained may be difficult to interpret, and extrapolating results may produce errors if the mechanisms are interpreted wrongly\(^{(227)}\). Ideally, accelerated tests could be used for predicting the service life of a structure, but in practice, the results are difficult to apply. The American standard ASTM E632-82\(^{(228)}\) gives recommendations for accelerated testing, and this suggests that tests must consider the particular attack mechanism, and that making accurate predictions from the results of the tests can only be done when data is available from long term tests. Tests using higher w/c ratios, or smaller test specimens to accelerate deterioration have shown that comparison of these results with the actual concrete in question is difficult\(^{(229-30)}\). In general accelerated tests cannot give an indication of service life, but can aid in the choice of suitable materials and mix proportions to resist a particular environment\(^{(220)}\).

5.1.3 Permeability Tests

Permeability tests have been reported for nearly 100 years. An extensive investigation was carried out by Glanville in the
1920s\cite{216}, and in the 1950s, Powers et al. carried out work on cement pastes\cite{231}. This and work by others has shown that permeability is affected by a large number of variables, including w/c ratio, cement type, cement content, cement replacements, aggregate type, aggregate content, aggregate grading, additives and admixtures, curing, time and moisture content\cite{23,193,148}. Much of this information can be useful for designing concrete mixes to achieve durable concretes\cite{15}.

Many different test methods have been developed to measure this property in the laboratory, but no standard has yet been introduced. A survey of different test methods has been published by the Concrete Society\cite{20}, and this shows the wide variety of methods which have been utilised by various researchers. All of the different methods operate by forcing fluids or ions through a concrete specimen using a differential fluid or ion pressure.

Some of these tests have been used to assess the suitability of concrete for containment structures; such tests use the fluid concerned: air\cite{232}, water\cite{234}, oil\cite{235}, liquid natural gas\cite{236}, or liquid nitrogen\cite{237}. Tests for permeability as a means of assessing durability have tended to use water, or air (occasionally oxygen\cite{23}) for fluid tests, and ion tests being carried out using the appropriate ions such as chlorides\cite{238}. Lawrence\cite{18} has reviewed the methods available for assessing the permeability of specimens taken from structures.

Unfortunately, very little work has been carried to try to relate durability and deterioration to permeability\cite{20}, although some attempts have been made to relate permeability to rates of carbonation\cite{239}. Buenfield\cite{240} has used permeability measurements to demonstrate the effects of seawater on submerged concrete. Work has also been carried out by Nyame et al, to try and relate permeability of cement pastes\cite{241} and normal and lightweight mortars\cite{242} to porosity, as the type and distribution of pores is also considered to relate to durability\cite{45}. Schwiete et al\cite{243} have obtained values of specific permeability using gas diffusion measurements, and these have been related to porosity, strength, and rates of carbonation.
Other methods are available for measuring properties related to permeability, such as absorption and capillarity. There is a British Standard test for measuring water absorption of concrete\textsuperscript{244} and Wainwright\textsuperscript{245} has attempted to relate this to durability and other parameters. The Reunion International des Laboratoires d'Essai de Recherches sur les Materiaux et les Construction (RILEM) have produced recommendations for measurement of water absorption by immersion\textsuperscript{246}, by capillarity\textsuperscript{247} and under a vacuum\textsuperscript{248}.

Methods have been proposed by Valenta\textsuperscript{15} for calculating permeability by measuring the capillary rise in a specimen, and Vuorinen\textsuperscript{249} has detailed methods for calculating permeability from the depth of water penetration into a specimen in a given time. A RILEM test also uses this as a means of indicating permeability\textsuperscript{250}, and Scales\textsuperscript{251} has used this with other methods to assess concrete durability. Tests have also been proposed by RILEM and others for measuring the vapour permeability of concrete\textsuperscript{252-4}.

5.1.4 Chemical Analysis

Concrete is the product of an ongoing chemical process\textsuperscript{2} and as a result problems are frequently related to unwanted chemistry occurring, changing the balance of the cement hydrates. The stability of the cement paste is dependent upon its existence in a saturated calcium hydroxide solution\textsuperscript{33}, and if this is upset, then the cement paste breaks down. This happens naturally with ageing, but is accelerated by chemical attack\textsuperscript{11}.

When problems do occur, chemical analysis can be used to identify the agents causing the problem, and thus help a solution to be formulated, either to prevent the problem reoccurring, or to reduce its effects\textsuperscript{6}. It may also be useful for analysing and comparing specimens from long term exposure sites\textsuperscript{217}. Chemical analysis can be used to determine the constituents of a concrete mix, including the cement content, and w/c ratio, as long as information is available about the aggregates in the mix\textsuperscript{255}.

Concern over reinforcement corrosion has led to the development of tests to assess chloride levels and carbonation
depths in concretes\(^{(12)}\). These can be carried out using either cores or drill dust taken from a structure\(^{(10, 18)}\). Chlorides are currently the main concern, and the Building Research Establishment has published guidelines for obtaining specimens and testing for chlorides in hardened concrete\(^{(256-7)}\). Test methods available for measuring chlorides in concrete have been reviewed by Buenfield\(^{(81)}\). Assessment of carbonation depths is readily carried out using phenolphthalein as an indicator of alkalinity. This has been shown to be sufficiently accurate for laboratory and site use\(^{(258)}\).

In general chemical analysis of concrete is a valuable tool, which can be useful for assessing deterioration processes. It also shows potential as a tool for predicting the risk of deterioration and assessing the time before problems such as reinforcement corrosion will start.

5.1.5 Petrographic Analysis

This is a specialised testing technique, which can be used to identify a large number of characteristics of the concrete at a microscopic level. A large amount of information can be obtained about the condition of the constituents, and the concrete itself using this method. The American Standard ASTM C856-83\(^{(259)}\) gives detailed information on the petrographic examination of hardened concrete and ASTM C457-82\(^{(260)}\) gives information on the assessment of air void properties of hardened concrete by microscopy.

Idorn\(^{(11)}\) has shown the usefulness of petrographic analysis for assessing field deterioration, and has shown how complex the deterioration processes are at this level. Patel\(^{(164)}\) has shown that the microstructure of cement paste varies with curing and depth into the structure. Petrographic analysis is the only way of properly diagnosing ASR\(^{(60)}\), and Idorn recommends a petrographic check of suspect aggregates to determine their suitability.

Petrography can be more useful than chemical analysis for diagnosing deterioration, because it is possible to visually identify problems such as ASR\(^{(261)}\). The technique has also been used to assess the effects of chlorides on the cement paste\(^{(44)}\) and is certainly a method that shows a great deal of potential.
for the future for assessing deterioration and providing information for predicting concrete service life.

5.2 Testing for Deterioration In Situ

In-situ testing has seen a rapid growth in recent years, as engineers have begun to recognise the need to test the actual concrete in a structure, and to move away from the cube for specification purposes\(^{262}\). This move towards in-situ testing has been complemented by new codes of practice, which publicise the available tests\(^{263-4}\), and by other publications doing the same\(^{265}\).

Appreciation of the differences between concrete in the cube and that in the structure has led to the introduction of terms such as 'labcrete' and 'sitecrete', to imply that they are indeed two different concretes. The arguments against labcrete (concrete made and tested under specified conditions in a laboratory) is that it is not the same as sitecrete (concrete used in a structure and subjected to less stringent controls).

In addition, it has been recognised that the surface of the concrete, the 'covercrete', has different properties from the bulk of the concrete, the 'heartcrete'\(^{(2)}\). This is important because the cover provides protection to the rest of the concrete and any reinforcing steel that is present. The difference between the concrete 'skin' and the bulk properties have been demonstrated by Kreijger\(^{266}\), and the surface has been shown to be important for the durability of concrete\(^{267}\). Accordingly, what is required is in-situ surface tests which for aesthetic reasons are non-destructive.

Available in-situ test methods may be classified as destructive or non-destructive tests, although some of the so-called non-destructive tests are slightly destructive. In all cases different amounts of information can be obtained about the state of the concrete. Non-destructive testing is obviously preferable, with the object of gaining as much information as possible, whilst doing as little damage as possible\(^{268}\). Destructive tests have little place for in-situ testing, except for obtaining samples for laboratory tests, or
to test a concrete to destruction for other information.

As with the laboratory tests, the in-situ tests may be considered as tests which assess the level of deterioration (which is especially important for assessing the need for repair and the extent of the required repairs), or those which attempt to predict a concrete's performance using particular parameters. Within these two categories, a large amount of work has concentrated on the problems of reinforcement corrosion.

For assessing the likely performance of the concrete, it is possible to consider the environment, as well as examining records of design, construction and maintenance. In addition, permeability tests may be used to determine the likelihood of aggressive agents entering the concrete. Assessing the deterioration in progress can be achieved by visual and physical examination of the concrete and structure. The likelihood of reinforcement corrosion occurring can be assessed by measuring the cover, carbonation depths, and chloride concentrations, while rates of corrosion can be assessed using electrical methods.

5.2.1 Assessment of the Environment

An assessment of environmental factors such as precipitation, temperature variations, chemical contaminants, can be useful for determining the likelihood of deterioration, and, if it is present, the rate at which it is occurring. Different exposure conditions can be defined, and are used by codes of practice to control the durability of the concrete placed. BS8110\(^{(38)}\) specifies five different exposure conditions from 'mild' to 'extreme', and gives recommendations for concrete designed to resist them.

Changes in environmental conditions may be responsible for new or increased deterioration (for example the introduction of de-icing salt onto roads\(^{(5)}\)), as may a change of use, or change of format of a building\(^{(180)}\). Some deterioration problems are the result of using concrete which is considered adequate for one environment in a new environment. In the Middle East for example, temperature differences and high salt concentrations are normal environmental conditions, so a concrete which was adequate in a
temperate climate may deteriorate very rapidly\(^{(1)}\).

5.2.2 Inspection of Records

Design and construction records may provide useful information about the concrete; such records can be used to reproduce a good concrete in future, or to avoid a repeat of the defects which may be responsible for deterioration in existing concrete. The importance of maintenance cannot be overlooked\(^{(269)}\) and regular maintenance records may provide information about concrete performance or failure. Details of the reinforcement, especially the positions of the bars, and the depth of cover may be of great use when trying to locate the bars, especially if reinforcement corrosion is suspected.

Records may also reveal the likelihood of localised or extensive deterioration, resulting from variations at the mixing or placing stage. Sources of materials (aggregates and cements) may become suspect after construction, or be shown to be defective\(^{(117)}\). Particular materials or practices used at only one place in the structure may show up quality variations. Unplanned changes in the material properties (such as adding extra water to a mix to improve workability) need to be recorded, in case a problem is revealed at a later date. The weather and season at the time of placing may influence the concrete quality, or the morale of the workforce, which again could lead to variations in quality\(^{(6,52)}\).

5.2.3 Permeability Testing

Permeability, as mentioned already, is widely believed to be a good indicator of potential durability, especially for normal environmental exposure conditions. The Concrete Society report on in-situ permeability testing of concrete\(^{(20)}\), details the available tests with their advantages and disadvantages, and emphasises that the tests employed must still be chosen because they are appropriate to the deterioration mechanisms which are present.

These different tests are at different stages of development. All are slightly destructive, and may be classified as 'surface tests', or 'subsurface tests'. The surface tests usually require fixing and sealing to the
concrete surface, which results in marking or damage to the concrete, whilst the subsurface tests require holes to be drilled or cast into the concrete.

The British Standards Institution's guide to non-destructive tests for concrete draws attention to the Initial Surface Absorption Test (ISAT), which is a surface test described in BS1881:Pt.5 and the Figg air test, which is a subsurface test, and has been described elsewhere. An important feature of these tests, which is mentioned in this standard, is that the results are affected by variations in the moisture content of the concrete.

This fact has also been reported by users of these and other permeability tests, who have found this to be a problem, or envisaged that it would be a problem for in-situ testing. Unfortunately, most research on these tests has been carried out on laboratory conditioned concrete, but Dhir et al have illustrated the problems of different conditioning, as well as comparing different test methods.

Another problem with these tests is that all too often the results are not related to any particular property; high values of permeability have been assumed to relate to non-durable concrete and low values to durable concrete. Attempts have been made to relate some of these to particular problems or concrete properties.

Levitt has shown a relationship between initial surface absorption and freeze-thaw resistance, as well as showing that there is a predictable relationship for ISA on oven dried laboratory specimens. Figg has shown relationships between air and water permeability indices measured using Figg air and water permeability methods, for different aggregates, w/c ratios, and compressive strengths. Wainwright et al have used the ISAT and Figg air test to assess the effects of different mix constituents and curing conditions, and Scales has also used the ISAT to assess the effects of curing conditions. Chantree has used a modification of the Figg water test to assess the effects on concrete of long term exposure to sulphates.

The ISAT has also been used as a specification compliance
in a number of contracts\(^{(20)}\), and proposals have been made for its use to indicate the time to the onset of reinforcement corrosion due to carbonation\(^{(25)}\). A series of subsurface tests have been reported by Petersen and Poulsen\(^{(280)}\) and Hansen et al\(^{(281)}\) for measuring the gas permeability of the surface layers of the concrete, and also for assessing surface cracking and likely carbonation rates.

Pihilajavaara and Parol\(^{(282)}\) have attempted to relate Figg air and water test indices to compressive strength, and a surface test reported by Montgomery\(^{(283)}\) also shows a relationship between permeability and strength. Work by Postacioglu\(^{(284)}\) has shown relationships between in-situ strength tests and porosity.

Newman and Whiteside\(^{(285)}\) have shown the benefits of waterproofing masonry walls, using an air permeability device attached to a wall using a vacuum. Schonlin and Hilsdorf\(^{(286)}\) have also reported a device using this principal, which has been used to assess the effectiveness of curing on different concretes.

These tests show a great deal of potential for measuring durability related properties, and can be used to highlight areas of permeable concrete, but until more information is available on their relationships, their in-situ use will be limited\(^{(81)}\).

5.2.4 Visual Tests

A visual assessment of a structure is completely non-destructive, and may be one of the most useful non-destructive tests available to anyone conducting a survey\(^{(88)}\). It consists of a visual examination of the concrete to see if any defects are visible and to determine their extent. Visible defects can range from staining and discolouration to spalling and disintegration\(^{(46)}\). Minor defects may be outward signs of more serious problems, and could indicate the need for a more thorough survey. A one off survey may not reveal much, but repeated checks at intervals could show changes in the concrete, and the progress of any deterioration\(^{(33)}\).

Guides for making condition surveys of concrete have been
published\(^{287-8}\), with the aim of standardising terminology and technique and making it easier to carry out regular inspections as part of regular maintenance checks. Even with these there is always a problem of individual interpretation of the concrete conditions and of the survey results. Survey reports are best accompanied by photographs and drawings to illustrate the deterioration, and to aid later comparison. Surveys also need to be carried out by experienced personnel, because correct interpretation is essential in these investigations\(^{33}\).

Visual testing can include surface crack monitoring. Fookes et al\(^{289}\) have used a process of crack ‘mapping’ to assess the extent and progression of deterioration, which has resulted in a detailed approach to interpreting the problems. Surface cracks can be easily seen if the concrete surface is wetted, then allowed to dry out. Simple measurements can be made to establish the size and extent of cracking. To assess movement, simple ‘telltales’ can be used, which drop out of the crack, or deform if any movement takes place\(^6\). If a crack is growing, then the rate of crack growth can be determined by marking the end of the crack, then measuring crack growth at intervals. For more accurate measurement, crack width gauges or crack measuring microscopes are commercially available\(^{290}\). More accurate measurement of crack movement can be assessed using graduated telltales, or by taking measurements with calipers between fixed points either side of the crack\(^{291}\).

More detailed physical investigations are required to determine the extent and nature of the cracking below the surface and also to locate hidden cracks such as delaminations. These methods are not entirely non-destructive, but may still be carried out as part of an initial survey of a structure. Methods vary from the very simple, such as ringing the concrete with a hammer to detect delaminations\(^{117}\), to much more sophisticated methods such as ultrasonic pulse velocity\(^{292}\), infra-red thermography\(^{293}\), and acoustic emission\(^{294}\). All of these methods require very careful interpretation in order to be of value.

5.2.5 Determining Depth of Cover to Reinforcement

To protect steel in concrete, there must be an adequate quality and quantity of the covercrete. Assessing the quality may not
be easy. However, measuring the depth of cover to reinforcement can be carried out relatively easily by using a covermeter, or by removing some of the cover to expose the reinforcement.

Measuring cover using electromagnetic covermeters is a method that has been available for many years, and is described in BS4408 pt.1\(^{(295)}\). However, experience is required to interpret the results and the bar sizes must be known\(^{(296)}\). This information can be obtained either by removing some of the cover (which is useful if reinforcement corrosion is suspected as the condition of the reinforcement can be established visually), or by using records if they are available. Covermeter surveys can be carried out non-destructively over large areas to assess overall cover and establish any areas that have inadequate cover and they can also be used as a quality control method on site.

5.2.6 Testing for Carbonation

Testing for carbonation can be done by testing the concrete with a phenolphthalein indicator solution\(^{(258)}\). This uses a mixture of phenolphthalein and alcohol dissolved in water, which turns uncarbonated alkaline concrete pink. This solution can be sprayed onto a fresh break in the concrete to reveal the depth of carbonation, or it can be sprayed onto drill dust as a hole is being drilled\(^{(89)}\). The depth of the drill when the colour change takes place can be used as an indicator of the carbonation depth. This method is only slightly destructive, so it is useful for carrying out surveys over large areas. A grid can be used to produce a contour plot or three dimensional image showing depths of carbonation\(^{(10)}\).

The rate of advance of the carbonation front can be estimated knowing the age of the building, and assuming that the rate of carbonation is linear with time. From this the time for the carbonation front to reach the steel can be estimated\(^{(10,12)}\).

An accelerated carbonation test has been reported by Hansen et al\(^{(281)}\), using high pressure CO\(_2\) forced into the concrete, followed by an indicator test on the concrete after removal from the structure. This test determines the degree to which the concrete may carbonate, rather than the amount of
carbonation that has taken place. Hansen et al have also reported a carbonation indicator. This is marketed by 'Germann In Situ Testing aps' (297) as Rainbow Indicator which measures pH and can be used as an alternative to the phenolphthalein solution.

5.2.7 Testing for Chlorides

Analysis for chlorides can be carried out in situ, but not with any great accuracy (81). Laboratory tests must be carried out for an accurate determination of chloride content (see Section 5.1.4). These can be done on solid specimens or on drill dust obtained from different depths in the structure. The results of the laboratory tests can then be plotted to give chloride concentration profiles through the structure (10,12).

Concentrations of free chloride are important (85), and localised variations can be found by using a contour plot of the tested area (12).

5.2.8 Electrical Methods

Electrical currents are produced by corrosion cells on the steel, so corrosion can theoretically be monitored by using current measuring devices (298). A number of electrical methods are available, which can be used as a means of assessing reinforcement corrosion or the likelihood of it occurring. The most commonly used are measurements of electrical resistance and half cell potential measurements (10). Other corrosion measuring devices such as electrical resistance probes (299) have to be implanted into the concrete to monitor the steel directly, so they are only really suitable for new concrete, or repairs (85).

Half cell potential methods measure the difference in potential of the steel in the concrete against a known potential, for example a copper/copper sulphate half cell. Some Authors suggest that this is not suitable for concrete, and that the calomel (mercury/mercury chloride) half cell is more suitable (72). This method is well documented and is described in ASTM C876-87 (300). Modern commercial equipment allows a rapid contour plot of the potential to be plotted (301).

Half cell potential methods do not indicate the rate of
any corrosion, but highlight areas at risk from corrosion. Limits have been proposed by Van Deveer\(^{(302)}\), suggesting that areas of high potential (less than \(-350\text{mV}\)) are at high risk from corrosion, above \(-200\text{mV}\) is a low risk, and \(-200\) to \(-350\text{mV}\) are areas which may require further investigation. The technique can be used to locate areas of localised corrosion, or general corrosion\(^{(303)}\), but is best suited for detecting isolated corrosion\(^{(85)}\). Half cell potential measurements have also been used to monitor the effectiveness of corrosion prevention measures such as cathodic protection\(^{(207)}\), or corrosion inhibiting admixtures\(^{(188)}\).

Resistivity methods measure the ability of the concrete to conduct a current. The higher the resistance the lower the risk of electrolytic corrosion, because the corrosion rate is proportional to the resistivity of the concrete\(^{(85)}\). Unfortunately, resistance is also moisture dependent, so dry concrete with high concentrations of chloride may not suffer from reinforcement corrosion\(^{(12)}\). Resistance can also be affected by carbonation, so measurements made on dry or carbonated concrete may give misleading results if these factors are not taken into account\(^{(10)}\).
6 TEST METHODS EMPLOYED IN THIS RESEARCH

6.1 Philosophy of Testing

The initial aim of the experimental work conducted in this research was to assess a number of different in-situ and laboratory permeability tests with respect to various concrete mixes to determine the effects of different mix parameters such as w/c ratio, cement content and slump on the measured permeability properties. This information would then be used in tests on site to relate concrete of known durability to concrete of unknown durability by comparing the relative permeabilities of the two.

After initial tests had been carried out, it became clear that the moisture content influenced the permeability measurements significantly. As a result of this discovery the direction of the research was changed so that this problem could be examined in greater detail. The principal objective of the research was now to examine the relationships between the permeability measurements and concrete moisture content and to find a method of measuring moisture content which could be used on in-situ concrete.

6.2 Laboratory Test Methods

6.2.1 Permeability Measurement

True permeability is the property of a material which governs the flow of fluid through the material under a pressure head\(^{(20)}\). A detailed explanation of the permeability of porous media has been given by Klinkenberg\(^{(303a)}\). The Concrete Society\(^{(20)}\) has given a more simplified explanation, the relevant parts of which are detailed here.

The permeability of porous materials including concrete have been defined in a number of ways by different researchers. Values are frequently quoted as the coefficient of permeability\((K)\), defined by the Darcy equation\(^{(304)}\):
\[
K = \frac{Q \cdot dl}{(A \cdot dh)}, \quad (6.1)
\]

where \(Q\) is the flow rate (m\(^3\)/s), \(A\) is the area of the specimen perpendicular to the direction of flow (m\(^2\)), and \(dh\) is the head drop (m) across a specimen thickness of \(dl\) (m). This gives \(K\) in units of m/s.

Intrinsic permeability (\(k\)) has been related to \(K\) by Muskat\(^{(305)}\) by including fluid viscosity (\(\mu\)) and specific weight (\(\gamma\)) in the formula such that:

\[
k = K \cdot \frac{\mu}{\gamma}, \quad (6.2)
\]

Because this equation takes the fluid properties into account \(k\) is a characteristic of the concrete, which is independent of the fluid being used. This relationship gives the coefficient \(k\) in units of area (m\(^2\)).

These equations are generally used to calculate liquid permeabilities. Calculating gas permeability cannot be done using the equations in the given form because of the compressibility of the gas\(^{(306)}\). As a result of this, calculations must be based upon mass flow rate, rather than volume flow rate, using the average pressure in the system. An expansion of equation 6.2 is proposed by the Concrete Society Working Party\(^{(20)}\) to give a value of intrinsic permeability, defined by:

\[
k = 2 \cdot Q_o \cdot P_o \cdot \mu \cdot l/A \cdot (P_i^2 - P_o^2), \quad (6.3)
\]

where \(Q_o\) is the outflow (m\(^3\)/s), \(P_o\) is the outlet pressure (N/mm\(^2\)), \(P_i\) is the inlet pressure (N/mm\(^2\)), \(\mu\) is the viscosity of air (sPa), \(l\) is the specimen thickness (m), and \(A\) is the cross sectional area (m\(^2\)). A derivation of this equation for gas flow has been shown by Nagataki and Ujike\(^{(306)}\) giving a value for the coefficient of permeability.

Vapour permeability can be measured by using a high vapour pressure difference across a specimen either with a saturated vapour sealed into a cap with the specimen exposed to
it one side, and another vapour pressure on the other side, or alternatively having the specimen enclosing a cap containing a desiccant, situated in a high relative humidity. In both cases the vapour permeability \( K_d \) is determined by measuring the change in weight of the specimen over a period of time. This corresponds to the weight of vapour flowing through the specimen under the given conditions.

The value of \( K_d \) can be calculated using a modification of the Darcy equation (equation 6.1). A formula is proposed in the RILEM recommendation No.33:252:

\[
K_d = \frac{G}{A} \cdot \frac{d}{t} \cdot \Delta \rho,
\]

where \( G \) is the weight gain (grammes) in time \( t \) (hours), \( d \) is the specimen thickness (m), \( A \) is the cross sectional area of the specimen (m²), and \( \Delta \rho \) is the vapour pressure difference (Pa) across the specimen. This gives \( K_d \) in units of m/hour.

6.2.1.1 Liquid Permeability Tests

The Darcy formula was chosen for evaluating water permeability because work had already been carried out using it with a permeability apparatus that was available in the laboratory.40 This apparatus was not entirely satisfactory, so an investigation was carried out to find a better design for it.

The major problem with the apparatus was sealing the specimen so that no leakage occurred. The method in the existing apparatus was unsatisfactory, because it required a very high quality of finish on each test piece, and a long period of preparation, cumulating in sealing the specimens into the apparatus. This final operation was deemed to be totally unsatisfactory, as it required a 24 hour setting period for the sealant before the test could be started.

Various methods have been reported by other workers to seal specimens. These include:

i) sealing the specimen with wax or bitumen within a ring set inside the apparatus;307

ii) coating the specimen in epoxy or polyester resin, then
sealing this into the apparatus\(^{(234,308-9)}\) (similar to that in the existing apparatus);

iii) sealing the specimen on cast or machined faces using grease\(^{(231)}\);

iv) coating the specimen in neat cement paste and sealing against this\(^{(216)}\); or,

v) using flexible seals pressed against the concrete surface designed to accommodate any irregularities in the surface\(^{(237,253)}\).

The main problem with most of these methods is the use of multiple seals, which only serve to increase the possibility of having leaks. It was decided that the simplest option was to seal the outside of the specimen with resin, and at the same time form a smooth surface on which a gasket could seal. Having attempted some of the other options, this was adopted and the test cell was modified, so that a simple sealing arrangement was arrived at using a single 'O' ring sealing against the resin surface.

Another problem which was encountered by other workers was accurate measurement of the inflow or outflow. This had already been overcome on the existing apparatus by the use of a graduated measuring column on the inlet side of the apparatus. This was similar to the method employed by Glanville\(^{(216)}\). This was employed in the modified cell, as measuring outflow was not considered practical with this particular cell. Others have employed methods which measure penetration\(^{(249)}\) or outflow. The best of these would appear to be those that use cryogenic liquids, which can be measured as gas volumes at room temperature and atmospheric pressure\(^{(237)}\). Even so there still appears to be problems with measuring flow, as no standard method is currently available.

Another problem which was not easily tackled was the effect of high pressure water on the concrete, particularly cement paste, since water at high pressure is likely to cause further hydration. Results of other research have always shown a drop in the permeability values with time as the tests progress, some of which is not recovered by drying and retesting (Figs 6.1a & b). Some of this has been attributed to silting and leaching of soluble compounds at the high pressure side of the specimen, which are then deposited at the low
Consideration was given to the use of non-aqueous liquids to avoid these problems. Hydrocarbons were suggested, such as paraffin, but this was considered unsuitable because it would leave deposits in the specimens. As another alternative, alcohol was considered. The principle problem with this was thought to be evaporation, but it was later suggested that alcohols would also react with cement to form complex hydration products so this was not considered further.

It was concluded from this that to avoid unforeseeable complications, water would be the most appropriate liquid to use. It was considered that this would make it easier to compare the results of this work with those from similar tests used by other workers in the field and it was assumed that it would make comparison between this and the in-situ water permeability tests easier.

The final design for the apparatus is shown in Figure 6.2, and the apparatus is shown in use in Plate 6.1. The apparatus was run at pressures up to 100 psi (0.69 N/mm²), depending on the permeability and strength of the specimens. Low strength specimens were run at pressures as low as 10 psi. Flow was measured by recording the change in level in the measuring column every 24 hours. Specimens were tested for a period of 14 days, and values of the coefficient of permeability, K, were calculated using equation 6.1.

6.2.1.2 Gas Permeability Tests

Initially it was planned to use the water permeability cell to measure gas permeability using air instead of water. A few tests were carried out using this method. On the first concrete mix, air flow was measured by recording the time for a volume of air to collect in a measuring cylinder over water. Values of time were recorded for a given volume of air typically ten consecutive times for 0-100 ml, to give an average time for 10 ml. Intrinsic permeabilities were calculated using equation 6.3.

However this method proved unsatisfactory because it was necessary to measure the air flow out of the specimen, and this
was inaccurate because no provision had been made to seal the outlet side of the specimens.

Accordingly, an alternative method was sought for these tests. An air permeability apparatus was available in the laboratory for testing rock specimens, so it was decided to try this using concrete or mortar specimens. The apparatus was based on equipment used by Lovelock\(^3\), which used a 'Hassler type' permeability cell\(^2\). This consists of a plastic sleeve squeezed against the specimen using high pressure air to seal the specimen, while lower pressure air is used to produce a flow through the specimens. The air flow out of the specimens was measured using either a bubble flow meter or rotating float flow meters. The apparatus and sleeve with a specimen are shown in Plates 6.2a & 6.2b.

Two sizes of cells were available for testing specimens: 21mm diameter and 50mm long; and, 50mm diameter and 100mm long. Manufacture of a larger cell for testing 100mm diameter specimens was contemplated, but this was discounted due to cost. The smaller cell was chosen for testing mortar specimens, as this allowed a large number of specimens to be tested. (Because of potential problems from large aggregates affecting the flow it was not considered practical to test concrete in this apparatus).

The specimens used were 21mm diameter cores cut to length using a diamond saw. Each was washed to remove any sludge produced by the cutting and then dried to constant weight (≤0.01g in 24 hours) in an oven at 50±5°C. For the tests the specimens were sealed in the apparatus using a pressure of 400psi, then flows were measured over a 10 minute period using a driving pressure of 50psi. The specimens were then reversed, and the test was repeated, to give two sets of results.

To assess the effect of moisture content on the air permeability, it was decided to condition the specimens to different moisture contents calculated as a percentage of the weight of water in the saturated specimen, rather than using the percentage of the wet or dry weight of the specimen. This was so that the different specimens could be compared easily, as the method employed was independent of the specimen weight.
To achieve this the specimens were first saturated by standing them in water and gradually increasing the depth. Each was then tested in this saturated condition. They were then dried in an oven at 50±5°C until the weight of water in the specimen was approximately 80% of the saturated water weight. (This was estimated using the saturated weight and the dry weight which had been achieved initially). They were then tested in this condition. The procedure was repeated to give weights of approximately 60, 40, and 20% of the saturated value. In all cases the flow rate was recorded after 10 minutes, and the intrinsic permeability, $k$, was calculated from equation 6.3 using the flow rate at 10 minutes.

6.2.1.3 Vapour Permeability Tests

This test was based on RILEM recommendation No.33(252) and the 'dry cup' test reported by the Concrete Society Working Party(20).

For this test, discs approximately 10mm thick were cut from 100mm diameter cylinders. Four discs from each mix were tested at any one time. These were sealed onto a petri dish containing dehydrated silica gel using Faraday wax (Plate 6.3), then placed in a cabinet at 25±2°C and 98-100% relative humidity. Each specimen was then weighed every day over a period of two weeks.

The weight change with time was estimated by plotting the results and selecting the most linear portion of the resulting curves. The vapour pressure across the specimen at this temperature and relative humidity difference is approximately 2340Pa(313). The value of vapour permeability, $K_d$, was calculated using equation 6.4.

6.2.2 Capillary Intrusion

A method for measuring capillary intrusion has been proposed by Valenta(15). This involves taking discs of concrete and treating the flat surfaces with a clear impermeable coating. The specimens are then immersed in water at a known depth, and the advance of the water front into the concrete through the curved surface can be seen and recorded at intervals, or over a given period. It was decided not to carry out these tests after
difficulties were encountered cutting the discs, and finding an impermeable coating which did not make the concrete look wet.

6.2.3 Strength

Concrete strength can be readily measured and specified, so it was reasonable to consider this as a control test. It was also considered essential to demonstrate any relationship between strength and permeability, in order to show that strength cannot be relied upon as an indicator of durability.

The compressive strengths of the concretes were determined initially by crushing three air dry cubes from each mix. This was later changed to two cubes being tested after they had been stored under water for 48 hours prior to testing. This was so that they would be wet, as recommended in BS1881 Pt.115(314).

6.2.4 Salt Immersion and Oven Drying

From the outset of the research it was hoped that some long and short term tests could be carried out to assess the durability of the test mixes in particular environments.

As explained earlier (Section 5.1.2), accelerated tests cannot always be relied on to give a true indication of a concrete's ability to resist deterioration so the possibility of long term exposure tests was considered. However, it was decided that this would be impractical due to the lack of time available. A period of decades rather than years was considered necessary for a measurable deterioration under normal exposure conditions. Accordingly, it was decided to use an accelerated test, but with due consideration given to its limitations.

The most straightforward accelerated test that could be conceived was immersing the specimens in a concentrated salt solution, followed by periods of drying to allow the salt to crystallise out within the concrete pores, thus disrupting the concrete. It was considered that this test could reflect any relationship between durability and permeability, as the more permeable specimens would absorb more solution to a greater depth, so the disruption would be more noticeable. To accelerate the drying, it was decided to place the specimens in
an oven at 105±5°C between periods of immersion.

A salt bath was prepared, suitable for six cubes at once. A concentrated solution was prepared by dissolving salt (cooking salt) in boiling water until no more could be dissolved. The cubes were placed in the salt bath on a wooden tray with wire handles, so that it could be easily lifted out. The salt solution was then poured in until it reached a depth sufficient to leave 10mm of each cube exposed. The cubes were placed so that the exposed face was one of the side faces. This arrangement is shown in Plate 6.4, with four mortar cubes.

After 24 hours, the cubes were removed from the salt bath and placed in the oven for 24 hours to dry. After this time they were returned to the salt bath. Any salt deposits and loose material were removed and the salt solution was topped up to the required level. Periodically the salt solution in the tank was replaced to remove all debris from the bottom of the tank, and to maintain a concentrated solution.

Initially six concrete cubes were used for these tests then later four mortar cubes were also tested. When the mortar cubes were being tested two additional cubes were used to maintain the level of the solution. The mortar cubes alternated with the concrete cubes in the solution and in the oven. After some time with these two sets of cubes, it became apparent that the deterioration was very slow, so no further specimens were included in this test.

During the course of the experiment, notes were made of the progress of the deterioration in each specimen. To assess deterioration ultimately, each specimen was removed, all loose material was removed, and the remaining material was placed in a measuring vessel of known volume. A measured quantity of water was then added to fill the vessel. The volume of water remaining was taken to be the final volume of the material.

6.3 In-Situ Test Methods

Four in-situ permeability methods were readily available. These were the ISAT, the Figg air and water tests, and the Germann gas test. The ISAT apparatus was made in the laboratory, whilst
the other three sets of test apparatus were purchased from commercial organisations. The Figg tests were purchased from Arup Research and Development \(^{(315)}\), and the Germann gas test was purchased from Germann In-situ Testing aps \(^{(297)}\). An additional test was made based on the Figg air test. It was initially referred to as the modified Figg test, but was ultimately named 'the Egg Test', because of its resemblance to a fried egg.

The Egg test method is similar to the test used by Schonlin and Hilsdorf \(^{(286)}\), and the method reported by Lydon and Al Odaallah \(^{(316)}\), which both use air below atmospheric pressure applied at the surface of the concrete. These three methods all have the advantage that they are non-destructive, and can be easily used on site concrete, although the method proposed by Schonlin and Hilsdorf uses an electrically powered vacuum pump to reduce the pressure for the test, which has its limitations for site use, and the method proposed by Lydon and Al Odaallah requires measurements to be taken over several hours, because it measures volume change rather than pressure change.

All five methods were assessed initially for their ease of use. The Germann gas test was abandoned at an early stage because of problems with its use and interpretation of results (Section 6.3.4). The other four tests were found to be practical for site use despite having certain drawbacks. They were subsequently assessed by testing against various concrete and mortar mixes. After initial testing on oven dried specimens, the tests were carried out on specimens at different saturations, to assess the effect of moisture content.

On completion of all of the other tests it was decided to determine the carbonation depths of the different mixes, as an indicator of the durability, with the hope of relating carbonation depth to permeability.

In addition to these tests, and because of the realisation that moisture content was a problem, various in-situ methods of measuring moisture content were examined to see if they were suitable and reliable enough for use with the chosen permeability tests.
6.3.1 Initial Surface Absorption Test

This is the only test detailed in a British Standard, and has been in use for much longer than any of the other tests. It was first proposed in the early 1960s by Levitt(21), and was developed from the British Standard test for water absorption in roofing tiles(317). It uses a low pressure head of water of 200±20mm which is equivalent to heavy wind blown rain.

The apparatus consists of a cap, a graduated capillary tube and a reservoir funnel connected with flexible tubing (Plate 6.5a). The cap, with a known contact area, is sealed against the concrete, and the water head is applied by water in the funnel acting both as a reservoir and to fill the cap initially. The inflow to the concrete is measured using the graduated capillary tube at a given time after the water first makes contact with the concrete. The times recommended by the Standard are 10 minutes, 30 minutes, 1 hour, and 2 hours, with the proviso that for in-situ tests only 10 minutes may be practical.

As far as possible, the recommendations of the British Standard were adhered to. Two types of cap are recommended, a square shape for testing horizontal surfaces, and a 'Vierendel frame' shape for testing vertical surfaces. Both types of cap were made, and two sets of apparatus were constructed for them (Plates 6.5a & 6.5b).

To carry out the experiment with the square cap, a greased gasket was used to provide the seal against the concrete. On the vertical cap a putty like substance was used which also helped to keep the cap in position. This sealant consisted of Arboseal(318) (a commercial pipe sealant), mixed with Water Pump Grease, or vaseline to give it the right consistency.

Two capillary tubes were calibrated using the method recommended in the British Standard, giving one for each cap. The capillary for the horizontal cap was permanently mounted on a board whilst the one for the vertical cap was fitted to a board which was fixed to a clamp stand, with a spirit level attached to the top to make it easier to use on site.
The horizontal cap was found to be easy to use, because it could be readily clamped into position, with the gasket providing a good seal. Difficulties were experienced initially with the vertical cap until the right consistency for the sealant was arrived at and an improved method suggested by Levitt\(^{(319)}\) was adopted. Initial attempts with the vertical cap included using it on a concrete repair contract at Junction 29 of the M1 Motorway.

Some trial tests were conducted on a concrete column in the Civil Engineering Department laboratory. These were carried out to assess the ease with which the test could be carried out on a piece of in-situ concrete, in the laboratory, without any means of clamping the cap. In this case the cap had to be supported by propping with a steel bar, as it was considered that this would be an appropriate method for site use. Repeatability was determined by repeating the test, allowing at least 48 hours to dry between tests, as recommended in the British Standard. Readings were taken at all four recommended times, but instead of recording the flow over 30 seconds, 1 minute, or 2 minutes, the flow was recorded every 15 seconds for several minutes to assess variation in readings during the time they were being taken.

Problems were encountered initially with filling the cap. This was because with the capillary tube connected to the cap, any water in the capillary formed an airlock. This was a problem, as it could take several minutes to fill the cap. This problem was avoided in later tests by disconnecting the outlet tube from the cap until it was full, then reconnecting it.

Another problem which occurred was filling the capillary tube prior to taking readings. Again, if air locks formed then the capillary would not fill up. This problem was overcome either by sucking the open end of the capillary tube to draw water into it, or by repeatedly squeezing the outlet tube which forced water into the capillary.

The main test program used both horizontal and vertical caps. Cubes were tested using the horizontal cap, and beams were tested using the vertical cap. Except for very poor specimens, no problems were encountered with these tests.
One improvement was made with the sealing method for the vertical cap. When it was being used frequently, it was found that by applying a layer of grease to the sealant contact area, a very quick and reliable seal was obtained. This reduced preparation time from approximately ten minutes to approximately two, which meant that the tests could be carried out more rapidly. This has obvious advantages for site applications.

6.3.2 Figg Air Test

Initial reports of this test were very promising\(^{(24,75)}\), and although it is slightly destructive, it was decided that this is an acceptable degree of damage for an in-situ test. The test requires a small hole to be drilled in the concrete, 10\(\text{mm}\) diameter and 40\(\text{mm}\) deep. This is plugged at a depth of 17\(\text{mm}\) using catalysed silicone rubber which is poured into the hole and allowed to cure. Once the rubber has cured, a hypodermic needle is inserted through it into the void below (Plate 6.6a).

The apparatus, consists essentially of a manometer connected to a hand vacuum pump and the hypodermic needle. In addition to these there is a high vacuum stopcock to isolate the vacuum pump, and a filter to protect the manometer. This is all connected using polythene manometer tubing with Luer push fit joints (Plate 6.6b). The equipment supplied is complete for carrying out the test, but requires a hammer drill for making the test holes. To carry out the test as described by Cather et al\(^{(24)}\) the vacuum pump is used to reduce the air pressure in the system to well below 45\(\text{kPa}\) absolute, then the time for the pressure to rise from 45 to 50\(\text{kPa}\) is recorded. This is the Figg air time. Although the method was relatively clear it was felt that the term 'well below' was imprecise and needed to be defined more accurately.

Accordingly initial tests were carried out to assess the method and compare the relationship between pressure change and time. This shows a rapid pressure change at the start of the test after which the rate of pressure change becomes almost linear (Figure 6.3).

These initial tests demonstrated a number of problems. The first few tests gave very consistent results, but the
magnitude of the results was indicative of a poor quality concrete, which was not the case. It was discovered by using a sealed test tube as the 'test hole' that the numerous Luer joints in the apparatus were leaking. This problem was solved by applying a small amount of high vacuum grease to the connecting surfaces. After this was done, a small but consistent pressure change of 0.2-0.3kPa in 5 minutes was recorded using the test tube. It was suspected that this was due to temperature change in the system caused by the initial pressure change. This was subsequently deducted from all the experimental results.

Another problem which arose was with the silicone rubber seals. It was found on some occasions, that the rate of pressure increase changed quite noticeably if the hypodermic needle was knocked. It was presumed that this was due to leakage past the seal between the rubber and the needle, so, as a precaution, grease was smeared around the test area on all subsequent experiments. This also ensured that any leakage between the silicone rubber and the sides of the test hole was prevented.

Once serious testing commenced, it was found that the time for a pressure change of 5kPa, as recommended, was frequently well outside the suggested values for normal concrete. (This demonstrated another problem with the test method, in that if the pressure was reduced to well below 45kPa then it would result in an excessive time period before 45kPa was reached and timing started). It was decided to alter the method of taking readings to take account of this. Initially the time for a random pressure rise was recorded and the time for a 5kPa rise was estimated from this. This gave very inconsistent results, so to standardise the method, two options were adopted.

If the time for a 5kPa rise was less than 5 minutes, then this was recorded as normal. Alternatively, if the time exceeded 5 minutes, then the pressure rise at 5 minutes was recorded, and the time for a 5kPa rise was estimated. In addition, rather than start timing when the recorded pressure reached 45kPa absolute, the system was evacuated to near to this value and timing started straight away. This took the initial rapid pressure change into account and reduced the
time required to carry out the test. In addition, because many times were based on pressure changes of 1kPa or less, it was decided to record Figg air times as the time for a 1kPa rise rather than for a 5kPa rise, as then fewer results had to be extrapolated.

One foreseeable drawback of the test is the hole preparation, which has to be done in advance, so that the silicone rubber plugs can be made and allowed to cure. Special precautions would have to be taken on site if the test hole is inverted or on a steep incline, to prevent the liquid rubber from running out. This problem has been tackled and overcome by at least one user\(^{(273)}\). An alternative method of sealing the test hole using a greased rubber bung has been proposed and tested\(^{(272)}\), and this may prove to be a simple and practical alternative to the silicone rubber plug.

Another possible drawback with this test method is the need to drill holes. It is considered by some authors that the drilling operation may well be detrimental to the concrete, causing localised cracking which could affect the test results\(^{(283)}\). In addition, for use in isolated locations, a portable power supply would be necessary. This mars the otherwise highly portable nature of this test.

Correspondence with other users has revealed a very limited use of the test, and where results had been obtained there was no way of relating them to concrete properties, because insufficient data was available. This has resulted in a reluctance to use the equipment for site testing by some people\(^{(271,273,320)}\).

6.3.3 Figg Water Test

This test was tried experimentally and found to work reasonably well. The apparatus consists of a fine canula connected to a syringe, and inserted via an adaptor block into a manometer tube. The adaptor block is fitted to an aluminium frame, with a graduated 0.1ml pipette fixed to the side of it (Plate 6.7).

To carry out the test, the two tubes are inserted into the test hole through a 21 gauge hypodermic needle. Water is then forced into the hole from the syringe via the canula. Once
the test hole is full, the water overflows through the manometer tube to the adaptor block, and then along the pipette. Having filled the pipette, and one minute after water first touches the concrete, the water level in the pipette is brought to the zero line, and timing is started. The time for 0.1ml of water to flow into the concrete is the Figg water permeability time\(^{321}\).

The main problem encountered with the test was ensuring that all the air had been forced out of the test hole. It was found that if this did not happen during initial filling, an airlock could form inside the hole and no flow would be recorded. In some cases, a negative flow was recorded. This was largely overcome by a technique of pumping water in and out of the hole during the first minute, after which the meniscus was zeroed.

As with the Figg air test, it was found that this test frequently took a long time. In some cases the time for a volume less than 0.1ml was recorded and then this time was extrapolated to give the time for 0.1ml. To standardise the method, a limiting time of 10 minutes was chosen. If the time for 0.1ml was less than 10 minutes, then this was recorded. If the time was greater than 10 minutes, then the volume was recorded at 10 minutes, and the time for 0.1ml estimated from it. This method was used throughout when testing specimens at different moisture contents.

Foreseeable difficulties when using this test on site concrete include hole preparation (section 6.3.2) and the need to maintain the apparatus at a head of 100mm for the duration of the test. In addition, difficulties will be encountered when filling the test hole with water if the test surface is inclined past the vertical.

6.3.4 Germann Gas Test

The Germann gas test is considered as a development of the Figg air test\(^{(20)}\), and was first reported in 1984\(^{(281)}\). It involves pressurising an area of the concrete surface with air, and measuring the pressure rise in a hole drilled at an angle below the surface.
The apparatus consists of a jig, which is bolted to the concrete surface using expanding fixings placed in holes drilled in the concrete. This provides a sealed area of 60mm diameter on the surface and also provides a guide for a core drill to go under that area. A pressure sensor with expanding seals is used to measure the pressure rise in the hole. Pressure is applied to the sealed area from a compressed air bottle attached to the jig (Plate 6.8).

A number of tests were carried out with this apparatus. The following problems were noted:

i) very rapid pressure changes were seen, but these varied, even on the same test area;

ii) getting a good seal at the surface was difficult. In some cases air could be seen bubbling through water next to the gasket;

iii) readings were taken every 15 seconds until the sensor scale was exceeded (less than 4 minutes), instead of every second for one hour as recommended\(^{(322)}\);

iv) it was very difficult to keep grease off the test area, because the jig tended to move about when it was being placed;

v) the theory provided to calculate the air permeability is difficult to interpret and includes some unspecified variables;

vi) the test was considered to be too destructive because of the three holes left in the concrete;

vii) no provision was made for replenishing the air supply so special arrangements had to be made;

viii) seals were difficult to maintain and to check;

ix) concrete which appeared to be good when checking the pressure gauge, proved too permeable to test;

x) repeated tests showed a marked variation in the rate of pressure rise in the same test areas on different days (Figure 6.5); and,

xi) interpreting the results suggested a poor quality concrete, whilst testing the same piece of concrete with the ISAT suggested the contrary.

This test was deemed to be unsatisfactory, both as a laboratory test and as a non-destructive in-situ test, so no further testing was carried out with it.
6.3.5 The Egg test

The Egg test was originally conceived as a non-destructive version of the Figg air test. The initial design was a small dome sealed at the edges in contact with the concrete, and held in position by external air pressure. This hole produced above the surface was to be tested in the same manner as the Figg air test. To ensure a good seal around the test area it was decided to incorporate a flexible 'skirt' to cover the concrete around the test area, sealed onto the concrete using grease.

The Egg was made by pouring de-aired catalysed silicone rubber onto a 100mm glass dish. Once it had set this gave a smooth convex surface which was reversed to make it slightly concave. By pressing a metal former into the the liquid rubber, a hole was formed in the skirt with a small rubber disc in the middle. This small disc was then used to support a hypodermic needle while more liquid rubber was poured on. Once this was set, more rubber was added to build up the thickness of the rubber supporting the needle, until it was strong enough to be self supporting. An improved version of this used a metal former with the needle already fixed into it (Plate 6.9).

To carry out the test, the skirt is coated with a thin layer of grease, and the Egg is stuck onto the concrete. The Figg air test apparatus is connected and the test is then conducted in the same manner as the Figg air test (Section 6.3.2). The pressure change with time is similar (Figure 6.4).

In the early stages it was found when removing the egg that an even grease layer had not always been obtained, usually due to the presence of dust or dampness. This problem was overcome by applying grease to both the concrete and the egg, to ensure that the grease was sticking to the whole of the contact area.

6.3.6 Carbonation Testing

Carbonation tests were carried out on concrete and mortar cubes after all of the other tests had been completed. Some of the cubes had been stored in the laboratory since casting, and had not been tested in any way. The other cubes had been tested, and then stored outside. In addition some cubes made from
cement paste were tested. (These were from a mix which had been prepared but had not been tested with the mortar mixes due to severe cracking when they were being dried out). These had also been stored outside. All the specimens tested were between one and two years old.

The cubes were rinsed in water to remove dust and dirt, then left for 24 hours in air to dry. Each cube was split using a concrete strength testing machine, then phenolphthalein indicator solution was applied to the freshly split surfaces. The depth of carbonation was determined, by measuring the depth of uncoloured material around the edges (Plate 6.10). Depths of carbonation were recorded from the top, bottom and two sides, of each cube, and indicator solution was also applied to the area around the Figg test holes, to see if any carbonation had occurred here. A note was made of whether an Egg test had been carried out on the sides of the cubes, as it was believed that this could influence the depth of carbonation, due to the presence of the grease which was applied for the test. In addition, tests were carried out on concrete cubes which had been cured in water for both four and seven days to determine what effect this had on the carbonation depth.

An average carbonation depth was calculated from the carbonation depths on the two sides, and allowance was made for variations due to the Egg tests. For comparison, results were interpolated to give an estimate of carbonation at one year assuming a linear ingress of the carbonation front.

Cracks were seen as areas of deeper carbonation, which were especially noticeable on the grout mixes, as spikes in the carbonation front. These variations were not included in the depth measurement. On a small number of specimens, Germann rainbow indicator solution was tried for comparison.

6.3.7 Moisture Measurement

The fact that moisture content affects permeability results has been recognised by many authors, but none to date have attempted to quantify it. Schonlin and Hilsdorf attempted to overcome the problems of moisture by drying the concrete for several minutes with a hair drier, but is doubtful whether this
would have any significant effect on the concrete moisture condition, as it has been shown that drying in an oven for several days at high temperatures (>100°C) is required to drive off most of the free water in a normal concrete.(275)

Several methods of measuring concrete moisture content are available, all of which are non-destructive, but some do require holes to be drilled into the concrete. Methods may be categorised as nuclear, weighing, or electrical. A chemical method of indicating moisture content has been proposed by Germann in-situ testing aps(297) but this was considered to be inappropriate for the type of tests that were being carried out in this study.

6.3.7.1 Nuclear Methods

These have been available for a number of years, and were originally introduced for measuring soil densities and moisture contents(215). Initial tests in the 1950s suggested that the method is accurate and independent of the type of material. The method operates by using a fast neutron source, which is either placed on the surface (backscatter method) or in a hole (direct method)(215,265). Neutrons from the source are fast high energy neutrons, but when they collide with elements of low atomic mass such as hydrogen atoms in water, they are converted to slow thermal neutrons, which are detected and counted by the device.

However, a number of sources of potential error have been highlighted(215,263,265):

i) the count rate is affected by moisture gradients near to the surface;

ii) hydrogen atoms in other compounds will reflect the fast neutrons as well as those in water (although this is unlikely to be a problem in concrete); and,

iii) the operating radius is between 6 and 18 inches, so results from members smaller than this will be affected by the surrounding material. It is claimed that modern equipment using the backscatter method will measure the moisture content of a few millimetres at the surface, but calibration is not straightforward and accuracy is poor at low moisture contents.

Commercial equipment is available for measuring density
as well as moisture content which uses Americium-241:Be as the fast neutron source\textsuperscript{(324-5)}. Use of these devices is specialised however, because they are expensive (in excess of £6000\textsuperscript{(325)}), and because they contain a radioactive source, a radioactive or byproduct material license is required to possess and operate the equipment.

6.3.7.2 Weighing Methods

Two methods of assessing in-situ moisture concrete by weighing have been investigated at the Building Research Establishment. These are the drilling method\textsuperscript{(326)} and the independant core method\textsuperscript{(327)}.

The drilling method consists of drilling a hole into the concrete (or other building material) and collecting the dust as it comes out of the hole. The drill is kept cool by periodically dipping it into a liquid coolant such as methylated spirit. Once a large enough specimen has been collected, the moisture content can be determined using a calcium carbide meter, or it can be taken to a laboratory and weighed, then oven dried. This gives results as a percentage of dry weight.

The second method involves taking a small core, which is dried in the laboratory, then returned to the parent hole and sealed to leave an annular space between it and the parent material. After a period of time it is assumed that the moisture content of the core has reached that of the surrounding concrete so it can be removed and reweighed. The moisture content as a percentage of dry weight can be calculated from this, or alternatively, the core can be saturated, and the moisture content as a percentage saturation can be calculated.

Both of these methods are reported to give accurate and unambiguous readings, but values of moisture content calculated as a percentage of dry weight are difficult to relate to concretes as two different concretes may have the same moisture content calculated as a percentage of dry weight, whilst containing different amounts of water. The core method is also only considered suitable for conditions where moisture movement is perpendicular to the face of the structure. Because of these
reasons, these methods were not considered suitable for this study. In addition, using them would have required drilling extra holes into the specimens which was impractical. It is however worth considering that the drilling method could be used when preparing Figg test holes.

6.3.7.3 Electrical Methods

Three electrical methods were investigated: two of which measure resistance, and a third which measures the dielectric constant of the material. A fourth method was also considered, which measures microwave absorption, but this was not examined experimentally.

The first method measures the resistance to ionic flow between a pair of electrodes placed on the concrete surface, the resistance being dependent upon the amount of water in the concrete. Unfortunately, it is also dependent on the properties of the pore fluid. This means that each concrete must be calibrated, to take into account these variables. This method was tried experimentally using a commercial instrument called the Protimeter Surveymaster\(^{328}\) (Plate 6.11). This was only calibrated to record whether a concrete had a problem of dampness or not, and it was not claimed to be suitable for measuring concrete moisture content. It was found that on surface dry concrete, no reading was obtained until the surface was wetted to provide a good electrical contact. This device is more suited to soft materials such as plaster where the electrodes can be pushed into the surface\(^{328}\) or timber as described in ASTM D4444-84\(^{329}\).

The second method uses a similar principal to the first, except that the electrodes are contained within a block which is either cast into the concrete, or cast into a hole prepared in the concrete\(^{330}\). By using a substance with known pore fluid properties the variations of the concrete pore fluid are eliminated. These types of probe can be calibrated prior to installation, so this method has more potential than the other. For in-situ use, it would have to be in position in the concrete for some time in order to reach an equilibrium moisture condition. The method has been used by a number of researchers to show increases in maturity of both normal and air entrained concrete\(^{331-2}\), as it has been shown that in a
saturated specimen, the resistivity changes with age\textsuperscript{(330)}.

The method was examined experimentally using a device called Bouyoucos Moisture Meter\textsuperscript{(333)} (Plate 6.12), which is a commercial device for assessing soil moisture, as a means of determining optimum irrigation times. The electrodes supplied with the device are made from stainless steel, cast into gypsum blocks. Unfortunately, these did not work, because the electrodes were not sensitive enough to measure the low moisture contents found in concrete. Blocks manufactured in the laboratory using copper electrodes cast into plaster did not work either because the plaster was not mature so the measured resistance changed, even though they were kept saturated. This method was considered to be unsuitable for the tests carried out because the electrodes could not be easily implanted into the specimens being used, but the method may be useful if a long term investigation of a concrete structure is contemplated.

The third method involved measuring the dielectric properties of the material. The method used the fact that the dielectric constant of water is significantly different from that of concrete, so any water present affects the measured value of the dielectric constant. The dielectric constant can be determined by assessing the change in an electric field which different dielectric constants produce. A method is described in ASTM D4444-84\textsuperscript{(329)} for calibration and use of these devices for measuring moisture content in timber.

A commercial device using this principal was obtained for this research. The device, a Moisture Monitor M49\textsuperscript{(334)} (Plate 6.13), is completely non-destructive and is simply held against the concrete. It has a digital scale which registers up to 20\% relative humidity. It has three different sensitivities for high, medium, and low density materials, the high density position is recommended for testing concrete. According to the manufacturers instructions it requires no calibration and readings are independent of the type of concrete being tested.

This device was used for most of the tests carried out on specimens at different moisture contents, and was also calibrated against a number of specimens as they were air dried from a saturated condition. The only real problem encountered
with this device was that it can only be operated successfully on smooth surfaces.

Another method of moisture content measurement which has been proposed utilises the microwave absorption of a material. It has been suggested that a linear relationship exists between microwave absorption and moisture content (272). The method is well established in the paper and textile industries (335), but problems exist for applying it to large volume solids, although some work has been carried out on particulate solids (336). This method is also dependent upon the dielectric constant of the material, but no information is available for using it on concrete. A device for measuring moisture content using this technique has been marketed by 'Physical and Electronic Laboratories Ltd (see ref. 334), but it is not considered to be any better than the M49 for use on concrete (337).
Figure 6.1a Effects of drying out 1:2:4 mix (after Glanville (216))

FIGURE 6.1b Effect of Reversing Specimens and Wire Brushing Surface (After Glanville (216))
FIGURE 6.2 Cut-Away Drawing of Water Permeability Cell
FIGURE 6.3 Relationship Between Pressure and Time (Figg Air Test)

FIGURE 6.4 Relationship Between Pressure and Time (Egg Test)
FIGURE 6.5 Relationship between Pressure Change and Time for Germainns Gas Test

N.B. Numbers are No. of days after first test (0).
PLATE 6.1 Water Permeability Apparatus
PLATE 6.2a Air Permeability Apparatus

PLATE 6.2b Air Permeability Specimen and Sleeve
PLATE 6.3 Vapour Permeability Specimens

PLATE 6.4 Mortar Specimens During Immersion in Salt Bath
Plate 6.5a ISAT Cap for Horizontal Surfaces with Apparatus

Plate 6.5b ISAT Cap for Inclined Surfaces
PLATE 6.6a Figg Test Hole Details

PLATE 6.6b Figg Air Test Apparatus
PLATE 6.7 Figg Water Test Apparatus

PLATE 6.8 Germanns Gas Test Apparatus
PLATE 6.9 Egg and Metal Former

PLATE 6.10 Carbonation Test using Phenolphthalein Indicator Solution

UNCARBONATED MORTAR
PLATE 6.11 Protimeter Surveymaster

PLATE 6.12 Bouyoucos Moisture Meter
PLATE 6.13 Moisture Monitor M49

The initial site design was carried out using the “Weather of Normal Concrete” method. It was planned to use the four specified slump values: 0 to 2 in., 2 to 4 in., and 4 to 6 in. and the available pumpeters to obtain a range of values. In addition, it was shown in the scheme how the values of the mixture between values of 0 and 4.5 were obtained. Later these figures, an injection method of 0 to 4.5 mix was designed, using 0 and 2.5 mix. The pumpeters were fabricated; however, the plan for...
7 DETAILS OF TEST PROGRAMME

7.1 General

In order to assess the in-situ and laboratory test methods chosen for this research, a series of concrete mixes was planned to give a range of mixes with varying properties.

The sizes of specimens chosen for these tests were 100mm cubes, 100x100x500mm beams, and 200mm long x100mm diameter cylinders to give the different sizes and shapes required for the planned tests (Chapter 6).

After initial problems with specimen preparation, and difficulties in comparing the initial set of concrete mixes it was decided to try mortar as a test material. It was hoped that this would make specimen preparation easier and that a more homogeneous material would result, which would make comparing the surface and subsurface tests easier. An added benefit of this decision was that smaller specimens could be used for air and vapour permeability tests because of the smaller aggregate size.

After the series of mortar mixes had been completed, a second set of concrete specimens was made using the same mix proportions as the first. This was to determine the repeatability of the tests, and also to obtain results from the water permeability test, which had not been very successful with the initial mix.

7.2 Details of Concrete Mixes 1 and 2

The initial mix design was carried out using the 'Design of Normal Concrete Mixes'\(^{(157)}\). It was planned to use the four specified slump zones (0-10, 10-30, 30-60, and 60-180mm) and the available aggregates to obtain a range of mixes. In addition it was planned to vary the w/c ratios of the mixes between values of 0.3 and 0.8 in increments of 0.1. Using these figures, an initial series of forty eight mixes was designed, using 10 and 20mm maximum aggregate sizes, however due to
problems with the initial tests, only six mixes were made.

The coarse aggregate used was Trent valley river gravel with gradings 5-10mm and 10-20mm. Fine aggregate was zone 2 river sand with maximum aggregate size of 5mm. The cement was Ordinary Portland Cement manufactured to BS 12(338), and the mix water was tap water at laboratory temperatures.

7.2.1 Mixing Procedure used for the Concrete Mixes

The first set of six concrete mixes (C1) was designed to give slumps in the range 60-180mm. The volume of concrete made was 0.015m³, to make one beam, six cubes, and one cylinder. The mix proportions used are shown in Table 7.1.

The dry constituents were placed in a mixer, then the water was added and the contents were mixed for two minutes. Once mixing was complete, the concrete was placed into the moulds on a vibrating table, and each mould was half filled before vibrating commenced. The moulds were then filled while the vibration was in progress, and struck off using a float to achieve a smooth level surface. Once all the moulds were filled and struck off, they were covered with polythene, and left for 24 hours to harden. The specimens were then demoulded, marked with a mix identification and placed in a curing tank.

The second set of mixes (C2) was made using the same mix proportions as the first for comparative purposes. The mixing procedure was the same except for a slight increase in the mix proportions. This was because two cylinders were made instead of one, so the quantity of concrete was increased to 0.017 m³. Mix details for the C2 mixes are also given in Table 7.1.

7.2.2 Storage of Specimens Prior to Testing

All specimens from the C1 mixes were stored in a curing tank at 20±2°C for a period of 6 days after which they were removed and placed in the laboratory air for 21 days. This brought the test date to 28 days after casting.

Specimens from the C2 mixes were also placed in a curing tank after 24 hours, but to assess the effects of different curing times, two cubes were removed from the curing tanks

102
after only three days and left in the laboratory air, while the other specimens remained in the curing tank for six days. All specimens were then air dried up to an age of twenty one days, after which they were placed in an oven to dry prior to testing.

7.2.3 Specimen Preparation

The Figg tests, and the laboratory permeability tests required some preparation to be done in advance. The Figg tests required holes to be drilled and plugged, and the laboratory permeability test specimens had to be cut from cylinders and sealed on the curved surface.

7.2.3.1 Figg Test Hole Preparation

The Figg test holes were prepared in the following manner. Each cube had one hole drilled in the centre of the trowelled face, and each beam had five holes drilled at 100mm centres in the trowelled face. After the holes had been drilled, any dust clinging to the sides of the hole was dislodged using the gun cleaning brush provided and then blown out using a compressed air line. Once the holes were clear, a 3mm thick foam plastic plug of 11mm diameter was inserted into each hole at a depth of 20mm. De-aired catalysed silicone rubber, was then poured into the holes and left to cure.

7.2.3.2 Cutting of Concrete Cylinders

Specimens for the laboratory permeability tests were cut from the concrete cylinders using a masonry saw. It was planned to cut eight slices from each cylinder, four 20mm thick slices, two for water and two for air permeability tests, and four 10mm thick slices, two for vapour permeability, and two for capillary intrusion tests.

However, difficulties were encountered in cutting 20mm thick slices from cylinders of mixes C103 and C104. Only one 10mm slice was obtained from mix C103, and none were obtained from mix C104. It was decided therefore to only attempt to cut two 25mm slices from the other four cylinders (mixes C105-8).

There were two main reasons for the problems with the
cutting. Firstly an abrasive disc was used in the saw. This did not cut the concrete very well, and the result was that the slices broke, or the aggregates broke out before the cuts were complete. Secondly, the lack of a positive method of holding the cylinders while they were being cut, meant that they could twist whilst being cut so increasing the likelihood of breakages.

The cylinders from C2 mixes were cut using the same saw, but fitted with a diamond tipped blade (Plate 7.1a). In addition, the cylinders were held firmly in position using a specially made jig (Plate 7.1b). It was planned to cut three 25mm thick slices and three 10mm slices from the cylinders. No problems were encountered with this method and six slices were successfully obtained from each cylinder as planned. The thicknesses of the slices varied slightly as it was not easy to position the specimen and allow for the cutting width of the saw. Thicknesses varied from 23-27mm for the 25mm slices, and 9.5-12.0mm for the 10mm slices.

7.2.3.3 Preparation of Permeability Specimens

Once the slices had been cut and marked, they were placed in an oven to dry prior to further preparation. When the slices were dry, they were taken out and allowed to cool.

To seal the edges of the water and air permeability specimens, the slices from the C1 mixes were coated with David's Isopon car body repair paste. The flat surface for the 'O'ring seal was made by spreading the paste around the top edge of the specimen, then placing it face down on a smooth surface. This was not entirely satisfactory because it was not easy and did not always give a smooth sealing surface for the 'O'ring. In addition, it was difficult to assess whether a good seal had been obtained between the paste and the concrete (Plate 7.2a).

Water permeability specimens from the C2 mixes were sealed using a two part liquid polyester resin. To do this, an 'O'ring was 'tacked' onto the top face of the specimen, using putty, to give an enclosed area of approximately 90mm diameter. The specimen was then placed face down onto a smooth greased surface, with a piece of split plastic pipe of
approximately 110mm internal diameter placed around it (Plate 7.2b). The polyester resin was mixed, then poured into the annular gap around the specimen, and allowed to harden. Once the resin was hard, the specimen was removed by taking off the section of pipe, which was split to facilitate removal. The specimen was then cleaned up, using a file to remove all rough and sharp edges, and made ready for the test (Plate 7.2a).

Using this method it was possible to see whether a good seal had been obtained with the concrete, because the resin was translucent, so any areas of poor bond showed up as light coloured areas within the resin. The specimen shown in Plate 7.2a has an area of poor bond, which can be easily seen.

Vapour permeability specimens were coated with a thin layer of molten wax, and then placed in the test environment to condition them ready for the test. When it was time to test the specimens, they were removed, any surface water was dried off, and then the petri dishes containing silica gel were attached using wax (Plate 6.3)

7.2.3.4 Conditioning Specimens to Different Moisture Contents

After initial testing of the cubes and beams using the in-situ tests, the beams were conditioned to assess the effects of different moisture contents. Initially it was intended to achieve this by placing them in a climatic cabinet at constant temperature and humidity. However, it was found that this did not work, so in later experiments the beams were saturated and then progressively dried out, tests being carried out as different weights were achieved.

Drying was carried out in stages, firstly in air, then in an oven. Each beam was dried for a maximum of one week, then sealed in a polythene bag and left for at least two weeks, prior to testing, to allow the moisture in the specimens to reach equilibrium between the surface and the interior. The oven temperature was initially 50±5°C, but this was increased to 105±5°C to speed up the drying process as the specimens became drier. The target 'saturations' for these tests were 100, 80, 60, 40, 20, and 0% of the water weight in the saturated specimens. Generally, it was not possible to achieve these saturations precisely, but in the majority of cases they
were achieved to within ±5%.

7.2.4 Testing of Concrete Specimens

Testing was carried out in accordance with the methods described earlier. Wherever possible, more than one test was conducted at one time in order to obtain the greatest number of results from each specimen in the available time. Initial tests on the first concrete mix were carried out at an age of 28 days, but after analysis of the results, it was concluded that it would be necessary to oven dry the specimens, in order to obtain more meaningful results.

7.2.4.1 Laboratory Permeability Methods

The water and air permeability tests on the C1 mixes were carried out on the same day as the in-situ tests, 28 days after casting. An attempt was made to saturate the water permeability specimens prior to the test by immersing them in water, leaving the top face clear. This was so that the measured flow from the start of the test would be through a saturated specimen. During the tests values of inflow and outflow were recorded at different times. The air permeability tests were conducted on dry specimens, but problems were encountered measuring the outflow from the test cell because it was difficult to get a good air tight seal with the sealing method employed at that time.

Water permeability tests on C2 mixes were carried out on oven dry specimens. Each test took fourteen days, so it was not considered necessary to saturate the specimens before the tests were started. No air permeability tests were carried out on the C2 mixes. Vapour permeability tests were carried out on two specimens from each mix, with each test taking one week.

7.2.4.2 In-situ Permeability Test Methods

It was not possible to carry out all of the in-situ tests planned on the C1 mixes, because only one day was time-tabled for testing each mix, and many of the tests took longer than expected to complete. Each cube had seven tests carried out on it. Figg air and water tests were carried out consecutively in each test hole. ISATs were carried out on opposite sides of the
cube, while Egg tests were carried out on the other two sides, and the bottom of each cube.

Five Figg air and water tests were carried out on each beam in holes numbered 1 to 5, starting from the end of the beam which had been uppermost when the beams were stored during air drying. Two Egg tests were carried out on the beam ends, numbered I and II, where end I had been the upper end during air drying. Six ISATs were carried out on the beam sides, three on each side; these were lettered U, V, W on one side and X, Y, Z on the other.

After the beam from mix C103 had been oven dried and retested, it was realised that moisture content affects the permeability to a greater extent than had previously been thought. To avoid inconsistency it was decided to oven dry and retest all of the beams from the C1 mixes. The beams were subsequently retested after they had been oven dried to give a weight change of less than 10g in 24 hours. This took longer than four weeks in all cases, which was an unforeseen problem in the preparation of the specimens.

A change in the test positions was implemented at this time. The number of Egg tests on the beam was increased to four, by carrying out Egg tests in positions V and X on the beam sides, as well as position I and II on the beam ends. The numbers of ISATs was reduced accordingly. It was hoped that by carrying out Egg tests on the sides of the beams that it would be possible to make a better comparison between the ISAT and Egg test results.

The beams were retested a second time after having been conditioned in a climatic cabinet for a period of time. The test positions were the same as on the previous retests. All subsequent retests at different moisture contents were carried out in the same manner. The cubes were not retested at any stage.

Specimens from the C2 mixes were oven dried to 1g change in 24 hours prior to testing. Tests were carried out on the beams in the same positions as the retests on the C1 beams. Cubes were tested with Figg tests remaining unchanged, but the ISAT and Egg tests were carried out on adjacent rather than
opposite sides. No tests were carried out on the bases of the cubes, and none of the specimens were retested at any time.

Carbonation tests were carried out on some of the cubes once all of the other tests had been completed and they were no longer needed.

7.3 Details of Mortar Mixes

After initial tests on concrete gave a large variation in results, it was decided to carry out the permeability tests on a number of different mortar mixes to see if these gave less variable results when used on mortar.

Mortar was considered to have a number of advantages over concrete:

i) easier comparison between different mixes, because there were less variables;

ii) a better comparison between surface and core results;

iii) a more consistent permeable matrix, uninterrupted by coarse aggregates;

iv) easier drilling and cutting of specimens; and,

v) the possibility of using small specimens for air permeability tests.

The original plan for these mixes was to use w/c ratios of 0.3-0.8 at increments of 0.1, with c:s ratios of 1:2, 1:3, 1:4, and 1:5.

Once mixing commenced, it became clear that not all of the proposed mixes were suitable, either being too wet, or too dry. Accordingly the range of mixes was changed to include the most suitable w/c ratios at each c:s ratio. This meant that less mixes were made at lower w/c ratios (0.3 and 0.4), but additional w/c ratios were included, up to 1.1 with a c:s ratio of 1:5.

In addition to changing the w/c ratios, it was decided to further extend the range of mixes by including a mix with a c:s ratio of 1:1, and also a neat cement mix (c:s ratio of 1:0), in order to get a better spread of results when comparing mix proportions in terms of the percentage of sand (0-80% sand by
weight instead of 66-80% as originally planned). This eventually resulted in a total of twenty five different mixes used in these tests, which are summarised in Table 7.2.

7.3.1 Mixing Procedure used for the Mortar Mixes

All of the mortar was mixed in a pan mixer and the neat cement paste was mixed in a commercial dough mixer because the pan mixer was found to be unsuitable. When mixing the mortar, the water was added gradually to the dry constituents with the mixer pan in rotation. With the cement mixes it was found that all of the water had to be added at once to avoid the formation of lumps, which the mixer could not easily break up.

7.3.2 Storage of Specimens Prior to Testing

Specimens were demoulded after 24 hours, then placed in curing tanks at 20±2°C for six days after demoulding. The specimens were then left to air dry for fourteen days in the laboratory air, after which they were oven dried prior to testing.

7.3.3 Specimen Preparation

After fourteen days all holes were drilled and plugged in the same manner as for the concrete specimens, and discs were cut prior to placing in an oven at 50±5°C to dry out. Initially drying was carried out to a weight change of less than 10g in 24 hours. When a more accurate balance became available, this was reduced to 2g for a cube and 5g for a beam.

The specimens used for the air permeability tests were cores taken from the off-cuts left after cutting the discs from the cylinders. In all but two cases, two specimens were obtained for each mix. These were oven dried to give a weight change of less than 0.01g in 24 hours. Further testing of these specimens required saturating them and then progressively drying and retesting them (Section 6.2.1.2).

Preparation of the beams at different moisture contents was carried out in the same manner as for the concrete mixes (Section 7.2.3.4), although only beams from mixes M113-6, M123-6, M134-8, M146-9, and M157-9 were retested. In some cases, the dry weights of the beams when they were retested

109
were lower than on the initial tests, which was thought to be a result of using higher drying temperatures and a longer drying period when drying the specimens for the retests.

7.3.4 Testing Mortar Specimens

The in-situ permeability methods were carried out in the same positions on the cubes and beams as on the C2 concrete mixes. Water permeability tests were the same as the C2 mixes, but vapour permeability tests were carried out on four specimens from each mix over a period of two weeks. In addition air permeability tests were carried out using the rock permeability apparatus.
Mix design based on: 'Design of Normal Concrete Mixes', Teychené et al;\(^{(157)}\)
Target slump: 60-180mm;
Cement: Ordinary Portland Cement to BS 12;\(^{(338)}\)
Fine aggregate: Zone 2 River Sand, 5mm maximum size;
Coarse aggregate: Trent River gravel; 20mm Maximum size;
split 2 parts 20-10mm to 1 part 10-5mm by weight.
N.B. All aggregates dry.

**TABLE 7.1 Concrete Mix Details**
<table>
<thead>
<tr>
<th>Mix*</th>
<th>c:s ratio</th>
<th>w/c ratio</th>
<th>Cement (kg/m³)</th>
<th>Sand (kg/m³)</th>
<th>Water (kg/m³)</th>
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<td>998</td>
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<td>907</td>
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<td>832</td>
<td>416</td>
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<td>768</td>
<td>461</td>
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<tr>
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<tr>
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<td></td>
<td>1.1</td>
<td>296</td>
<td>1497</td>
<td>329</td>
</tr>
</tbody>
</table>

*Mix identification as follows: M113 has c:s=1:1, w/c=0.3, etc.
Cement: Ordinary Portland Cement to BS 12;
Sand: zone 2 river sand, 5mm maximum size.

TABLE 7.2 Mortar Mix Details

112
PLATE 7.1a Masonary Saw with Diamond Tipped Blade

PLATE 7.1b Jig to Hold Cylindrical Specimens
PLATE 7.2a Water Permeability Specimens ('Isopon' sealant -left, polyester resin sealant-right)

PLATE 7.2b Preparation of Water Permeability Specimens
8 DISCUSSION AND INTERPRETATION OF RESULTS OF LABORATORY TEST METHODS

8.1 Water Permeability Tests

Difficulties were encountered initially with the water permeability test method, but once the problems had been overcome, it was found to work well. Specimen preparation appears to be a problem in most tests of this sort, and it is believed that the method adopted (Section 6.2.1.1) is probably better than any other that has been reported in literature. This is important because the large spread of results from various authors, reported by Lawrence(22), suggests that the test method must have an influence on the results.

The test method could be further improved, to give a seal on the outlet side of the specimen as well as the inlet side, although this would require modification in the method of supporting the specimen. The method of measuring flow could also be improved if measures were taken to remove the air from solution, or to avoid problems from evaporation, or condensation. On some occasions water had to be removed from the compressed air supply, although this was not considered to be a problem because a condensation trap was used in the compressed air supply.

The initial tests on the Cl concrete mixes were only of short duration, longer duration tests were used on the mortar and C2 concrete mixes. Accordingly, the results are recorded in a different manner for the two types of test.

8.1.1 Results from Cl Concrete Mixes

Results were recorded as the volume of water flowing in and out of the specimens at random times after the start of the test. The cross sectional areas of the specimens could not be accurately measured, because of the sealant used, so it was assumed that the flow took place across the whole of the concrete section. This gave a cross sectional area of $7.85 \times 10^{-3} \text{m}^3$, based on a specimen diameter of 100mm.
The tests were only of a short duration, with the longest test time under six hours. Initially it was intended to carry out the tests at a pressure of 80psi, but this resulted in some of the specimens breaking, so the pressures used for most of the tests were lower. On some of the tests, pressures were progressively reduced as the tests were being carried out if the flow rates were high, typically 40 to 20 to 10psi, although when low flows occurred the pressure was kept constant for the duration of the test. Results are summarised in Table 8.1.

8.1.2 Results from Mortar Mixes

Initial tests on the mortar mixes were carried out over a period of several weeks to determine the most suitable test period. Plotting values of $K$ against time shows a drop in the calculated values of $K$ with time, with $K$ becoming almost constant after 14 days (Figure 8.1). Because of this a maximum time of 14 days was chosen for these tests and values of $K$ at 14 days are used for making the comparisons between the different mixes.

Steps in the curves were initially thought to be due to variations in the results, but it was later realised that they were the result of a calculation error. The error occurred when calculating $K$ over a 24 hour period. When the results were plotted, they were plotted as the value of $K$ at the end of the 24 hour period (the total time, rather than the average time over which the value of $K$ had been calculated). This was not significant when calculating the value of $K$ at 14 days, but an improvement in the plotted curves was achieved by plotting the average value of $K$ at the average time between two readings (Figure 8.2).

Because the relationship between $K$ and time (Figure 8.1) exhibits what appears to be a logarithmic relationship, it was decided to plot the values on a logarithmic scale to see if a straight line relationship existed. Results from several specimens plotted using a logarithmic scale are illustrated in Figure 8.3. Plotting results on a logarithmic scale has the advantage that all of the different values of $K$ can be illustrated on a single plot and any variations in the relationship between permeability and time can be clearly seen as variations in gradient.
It can be seen from Figure 8.3 that the lines show fluctuations in the gradients at different points although in general, the gradients of the lines are similar. Unfortunately, this trend is not exhibited by all of the results, so it cannot be used to predict the long term permeability of any specimen from short term results. The maximum test time of 14 days \((t=1209600\text{s})\) gave a value of \(\text{Ln}t=14.0\) which was a useful reference when plotting the graphs.

Initial tests on mixes M123-6 used several specimens, in order to obtain an average value for \(K\), but because these tests took a long time, only one specimen was tested from each of the other mixes. In addition to this some problems were encountered with weaker specimens breaking (specimens from mix M151 broke even at 10psi, so no results were obtained from them). Values of \(K\) and test details are summarised in Table 1 (Appendix I).

The results do however show an unexpected relationship, in that there is little separation between the different \(c:s\) ratios at each \(w/c\) ratio when \(K\) is plotted against \(w/c\) ratio (Figure 8.4). This relationship is similar to those shown in literature, with \(K\) increasing quite rapidly beyond \(w/c\) ratios of 0.5. This confirms that the method is satisfactory, although it gives no real indication of what can be classed as a good or bad mix.

It had been assumed that, at each \(w/c\) ratio, if the amount of sand was increased then there would be a lower proportion of cement paste of similar permeability, so the bulk permeability would be lower. In addition, because there was more sand, more water would be absorbed by the sand, so the free \(w/c\) ratio of the mix would be lower than that specified, which would result in a lower permeability. This behaviour in mortars has been reported by Nyame\(^{242}\).

It can be seen from the Figure 8.4, that this is not significant for \(w/c\) ratios below 0.6, and, with \(c:s\) ratio 1:5 the permeabilities are higher for a given \(w/c\) ratio than those at \(c:s\) ratios 1:3 and 1:4, except at a \(w/c\) ratio of 0.7. This may be the result of different compaction on the different mixes.
8.1.3 Results from C2 Concrete Mixes

Tests on the C2 mixes were carried out in the same manner as the tests on the mortar. Results and specimen details are given in Table 2 (Appendix I). Comparing values of K with w/c ratio for concrete shows an unexpected relationship (Figure 8.5). Rather than K increasing as the w/c ratio increases, it decreases from 0.3 to 0.5, then increases from 0.5 to 0.8 in the expected fashion. This is probably due to the different mixes being used, with different proportions of the solid constituents as well as different w/c ratios. Taylor(16) has shown the variation in K as the aggregate size changes.

The relationship between LnK and Lnt for the C2 mixes is shown in Figure 8.6. This relationship is similar to that obtained from the mortar specimens. Unfortunately, the results from the C2 mixes cannot be easily compared with the results from the C1 mixes because the test procedures used were different. However plotting the results from the C1 mixes (Figure 8.7) and projecting the lines from the C2 results backwards does show a good intersection between the two sets of results, although the gradients of the lines differ in some cases.

8.2 Air Permeability Tests

8.2.1 Tests on Concrete

Only limited air permeability tests were carried out on specimens from C1 mixes and these were not considered to be very successful. Although repeatable results were obtained, in all cases they were very much higher than expected which may be the result of leakage on the outlet side of the apparatus which was not easy to detect.

Proposed values of intrinsic permeability for a good concrete are less than $10^{-17}m^2$, and greater than $10^{-19}m^2$ for a poor concrete(20). Based on these values, all of the concrete tested was poor by several orders of magnitude. Only results from mixes C104 and 106 came near the proposed values. Results from different specimens from each mix show some variations between specimens. The most comparable results were from mix
C108, with results of $1.85 \times 10^{-14} \text{m}^2$ and $1.11 \times 10^{-14} \text{m}^2$ for specimens 1 and 2 respectively. Details of the test specimens and results are shown in Table 8.2.

This test method may have been more successful if the specimens were sealed using polyester resin as used on the water permeability tests on the mortar and C2 mixes. If this method had been used, then it would have been necessary to provide an outlet seal on the specimen.

It is important to note that no efforts were made to condition the specimens to a constant moisture content in these initial tests, as the effects of moisture content were not appreciated when these tests were performed. As a result, it is difficult to relate the values obtained to any property, or to results published by other workers in this field.

No tests were carried out on specimens from the C2 mixes.

8.2.2 Tests on Mortar

As described earlier (Section 6.2.1.2), this test was based upon an existing method for measuring air permeability of rock specimens, so there were no foreseeable problems. The method appears to work very well, and is easy to carry out, principally because the necessary seals are provided within the apparatus, so no preparation of the specimens is necessary. The pressure used in these tests for sealing the specimens was 400 psi which was considered adequate as it was well in excess of that employed by Lawrence (308) who reported no leakage on specimens sealed at 3 bar (44 psi). This was confirmed in practice by the zero flow rate recorded on some of the saturated specimens.

Installation and removal of specimens is very easy with this apparatus, as it only requires a single component to be unscrewed, compared with the water permeability apparatus, which has six winged nuts to hold the apparatus together. The small specimen size is not suitable for concrete as it is considered that a specimen size in excess of the maximum aggregate size is essential to prevent the aggregate from influencing the result. A cell for testing larger specimens (50 mm diameter) was available if it had been required.
The only problem with the apparatus is accurately measuring high flows through permeable specimens. High flows have to be measured using a rotameter instead of collecting the air over water, but it was found that flows measured using the rotameters on this apparatus did not compare very well with flows measured by collecting the air over water.

For simplicity, no alternative to air as the flow gas was considered. Some researchers have used oxygen or nitrogen to avoid problems from the CO$_2$ in the air carbonating the specimens$^{(239,341)}$. Because of the short duration of the tests and low flows measured generally, carbonation of the specimens in this manner was not considered to be a problem. It is probable that if flows are measured over a longer period of time, then this may become more significant.

8.2.2.1 Tests on Oven Dry Specimens

Details of the test specimens and the dry permeabilities are given in Table 3 (Appendix I). It can be seen that the values of k (calculated using equation 6.3) range from $8.87 \times 10^{-12}$ to $7.10 \times 10^{-16}\text{m}^2$, although the majority of results are below $2.00 \times 10^{-14}\text{m}^2$. A comparison with values obtained from the literature$^{(20)}$ suggests that the values obtained are high. However, values quoted in literature are generally from specimens of undefined moisture content, and often relate to specimens kept in constant conditions for a period of time rather than being oven dried.

The relationship between air permeability and w/c ratio is illustrated in Figure 8.8. The pattern of results is very similar to that obtained for the water permeability (Figure 8.4) although there is a sharp rise with c:s ratio of 1:1 up to a w/c ratio of 0.6, whilst the majority of results from the c:s ratios of 1:2, 1:3, and 1:4 lie close together. The increase in results for c:s ratio of 1:4 flattens off at a w/c ratio of 0.9, behaviour which can also be seen with the water permeability results. It is possible that this 'flattening' of the curve is a result of settlement of the solids during compaction, allowing the water to rise to the surface. This would have had the effect of reducing the actual w/c ratio of the hardened cement paste, thereby reducing the apparent permeability of the mix. The potential for this was suspected.
when casting the specimens, because of the large amount of water which appeared at the surface of the wet mortar.

The relationship between k values and percentage sand is shown in Figure 8.9. This clearly shows that the lowest k values are achieved with low w/c ratios and low sand contents. As the sand content increases, so higher w/c ratios are required to obtain the optimum values of k. This suggests that k is a function of workability as well as w/c and c:s ratios.

Variations between results from the two specimens from each mix are quite large, which means that the results are rather unsatisfactory (Appendix I: Table 3). It is possible that less variation would have occurred if the specimens had been tested for a longer period of time. Some variation was also recorded in dry flow rates for each specimen on the retests after they were oven dried for the second time. These variations lead to the conclusion that there are problems with the test method or the method of conditioning the specimens which have not been established.

8.2.2.2 Effects of Different Moisture Contents

Carrying out the tests on the specimens at different moisture contents presented some problems, because no work of this sort had previously been reported. There was no way of ensuring that the specimens were in an equilibrium condition when they were dried to a given weight, so to try and standardise the method, specimens were sealed in plastic bags after drying to a given weight, and then left for a minimum period of 24 hours prior to testing.

Another problem which is reported in some literature is the effects of hysteresis, which causes the degree of saturation of a porous material to vary dependent upon how that saturation was achieved\(^{(342)}\). If the saturation is reached by drying a wet specimen, then the permeability can be different from the permeability if the moisture condition is reached by wetting a dry specimen. It was decided that this would not be a problem when carrying out these tests, as a constant method was used. However, for in-situ tests, the effects of this could affect permeability measurement and make accurate interpretation difficult.
Another foreseeable problem was that of comparing the results of different mixes with different dry flow rates. To overcome this it was decided to compare results at different moisture contents as a percentage of the dry flow rate. Results for these tests are given in this form in Table 8.3.

Plotting results of percentage dry flow against percentage saturation gives the relationship shown in Figure 8.10. This is similar to the relationships that have been obtained between oil and water in permeable rocks for oil extraction purposes\(^{(343)}\). Only a few specimens give results outside of this pattern, and these are the very permeable specimens tested at lower pressures.

It can be seen from the Figure that at high saturations, low flows occur and at low saturations high flows occur. For example above a 50% saturation the flow rates are less than 20% dry flow rate, whilst below 50% saturation the flow rates increase quite rapidly towards the dry flow rate.

This introduces a new factor to be considered when measuring permeability, which is the degree of dryness of a 'dry' specimen. It has long been recognised that the moisture content affects the permeability (especially the gas permeability) but the extent of this variation is rarely considered. Work reported by Bamforth\(^{(344)}\) has shown that for a well cured concrete there can be a variation of as much as 75% between saturated and oven dry gas permeabilities.

For comparative purposes, it should be adequate to condition specimens to a predefined constant weight, or to leave them in a constant drying environment for a long period of time. The latter method would hopefully exaggerate differences between mixes, as the more permeable mixes would be expected to dry out faster than the less permeable ones, although the time period may influence this as well. Clearly this method is not adequate for measuring the 'true' value of intrinsic permeability. In order to do this, it is necessary to test the specimens in a totally dry condition.

Figure 8.10 can be used to illustrate the problems of different definitions of 'oven dry'. In some of the tests if the definition of oven dry was less severe than that used here,
then the results could be quite different. (The ASTM Standard for water absorption\textsuperscript{(345)}, specifies drying for not less than 24 hours at 100-105°C, while the RILEM recommendation for Vapour Permeability\textsuperscript{(252)} recommends drying at 50°C until there is less than 0.1% change in two weighings over 4 hours). Considering results from mix M113, the initial specimen weight was 39.69g. If the accuracy employed had been 0.1g change in 24 hours, then the drying would have stopped at a weight of 37.55g. This corresponds to 15% saturation based on the results obtained. Estimating the flow for this saturation gives a flow rate of approximately 50% of the dry flow rate actually achieved.

The implication of this, is that if gas flow rates are used to calculate \( k \), then some error must be expected, unless the specimens can be completely dried. Observing the rate at which the percentage flow rate increases at these low saturation values, it is reasonable to assume that significant errors can be made, and it is possible that the results quoted here are not absolute values of \( k \).

8.3 Vapour Permeability Tests

Although the test method employed was based upon proposals for this type of test, there is no recognised test method available, and so no limits have been proposed for good or bad concrete.

The method used was easy to perform. The discs were first measured at several points on the circumference to determine the average thickness, then they were sealed onto the petri dishes with hot wax. Sealing the specimens with wax was straightforward and in general it is believed that good seals resulted. In a small number of cases, bubbling of the wax occurred soon after the tests were started. To avoid potential problems from leakage, these specimens were removed, resealed, and the tests restarted. After some trial and error with the sealing method it was found that a better seal was obtained by warming the specimens and petri dishes before the hot wax was applied. This prevented the wax from cooling too quickly and cracking off the petri dishes.

The specimens were weighed on a balance accurate to
0.01g, unless the specimens weighed more than 306.50g (the upper limit of the balance at this accuracy). Above this weight, specimens had to be weighed to an accuracy of 0.1g.

It was frequently found that the weight increases recorded varied between specimens from the same mix. Sometimes this was just at the start of the test, and on other occasions it was for the duration of the test. On some occasions, with some of the less permeable specimens, the weight at the start of the test dropped. No reason could be found for this behaviour. In the majority of cases, an initial variable weight increase was followed by fairly linear behaviour until the test was stopped (Figure 8.11).

Because of the variations in weight increase with time, it was necessary to plot each set of results, and then estimate the gradient of the most linear portion of the curve. This was adequate for most specimens, but in a few cases, it was very difficult to select a good line. The lines were then used to give values of $Q/t$ which were used in equation 6.4 to calculate the values of vapour permeability. The cross-sectional areas used in the calculations depended upon the size of the petri dish used (either 91 or 97mm).

With the most permeable mixes, the time for the desiccant to become saturated was just a few days, after which condensation appeared inside the petri dish. Results for these tests were calculated from the gradients at the early stages of the test. It was assumed that this was partly a result of the specimens being only approximately 10mm thick. If more time had been available, then thicker specimens could have been used. (The RILEM recommendation\(^{(252)}\) specifies specimens 40mm thick, but these take from 10 to 15 days to achieve steady flow conditions).

8.3.1 Tests on Mortar

Test details and results for the mortar specimens are given in Tables 4 and 5 (Appendix I). Comparing the values of $K_d$ with w/c ratio (Figure 8.12), shows that the results are quite variable. It can be seen from this Figure there is a broad increase in $K_d$ as the w/c ratio increases, although there is
some variation from this with the different c:s ratios. It can be seen that there is no consistent variation in values of $K_d$ as the c:s ratios change. For example at w/c ratios of 0.3 and 0.4, $K_d$ increases with c:s ratio in the order 1:1, 1:2, 1:3, whereas at a w/c ratio of 0.5 the order is 1:1, 1:3, 1:2. Similarly at a w/c ratio of 0.7, $K_d$ increases with c:s ratio in the order 1:3, 1:4, 1:5, whilst at a w/c ratio of 0.8 it is 1:5, 1:3, 1:4.

From these results, it must be concluded that the test is not accurate enough to distinguish between these different mixes. It is possible that errors were a result of the thin specimens, and the problems of determining some of the weight increases with time.

8.3.2 Tests on Concrete

Vapour permeability tests were not carried out on specimens from C1 mixes. Only two discs from each C2 mix were tested, and these tests only lasted for a period of one week. Results and specimen details are summarised in Table 6 (Appendix I).

The vapour permeabilities of the C2 mixes are much more consistent than those from the mortar and the values of $K_d$ increase gradually as the w/c ratio increases (Figure 8.13). The small variation in results may be a result of only two specimens from each mix, or possibly, because of the coarse aggregates in the concrete. Comparing the values of $K_d$ from the mortar mixes with those from the concrete, it can be seen that the values are similar at low w/c ratios, but at higher values, the concrete has noticeably lower values.

8.4 Strength Tests

8.4.1 Concrete Strengths

For the C1 mixes the average compressive strengths were calculated from failure loads of three cubes. Average values for C2 mixes were calculated from two cube results. Results are given in Table 8.4.
As stated already, comparison of results from the concrete mixes using w/c ratios is difficult, because the other mix proportions are different. This means that any relationship between the results could not be assumed to apply to concrete generally. However because the mixes were designed, the expected strength for the mixes can be predicted. The actual strengths recorded were higher than expected, which was thought to be because the free water content was lower than the designed value, because the aggregate was dry, and not saturated surface dry, so it absorbed some of the free water and, the achieved strengths were higher than the expected strength in all cases, because the expected strengths were the minimum specified strength (Table 8.5).

In addition, the cubes from the C1 mixes were air dry when they were tested (contrary to the British Standard[334]) so the strengths were probably also higher as a result of this. Comparing the strengths from the C1 mixes with those from the C2 mixes (Table 8.4), shows the C2 mixes to have slightly lower strengths, which could be attributed to the fact that these cubes were tested in a wet condition.

8.4.2 Mortar Strengths

The average compressive strengths of each mortar mix were calculated from the compressive strengths of two cubes (Table 8.6).

Comparing strengths with w/c ratios (Figure 8.14), it can be seen that all values lie on a broad curve, but results from each c:s ratio lie on straight lines, with increasing gradient as c:s ratio increases. The only major deviation from this is mix M133, which was a poorly compacted specimen, and a small deviation also occurs with mix M145 probably for the same reason. Of interest is the similarity in results at certain w/c ratios for example mixes M116/56, M149/59, and M138/48.

Comparing the strengths with percentage sand (Figure 8.15), shows a smooth relationship between different w/c ratios, with the lower w/c ratio in each mix having a higher strength (except for mix M133). It also shows that with most mixes the strengths peak between c:s ratio 1:3 and 1:4. The only exception to this are those with a w/c ratio of 0.3 which
shows no obvious peak although presumably it must lie below a c:s ratio of 1:3

8.5 Accelerated Deterioration Tests

Accelerated tests consisted of immersing cubes in a concentrated salt solution for a period of 24 hours, then placing them in an oven at 105±5°C for the next 24 hours (see Section 6.2.4). Cubes were tested from mixes C103-8 and M123-6.

8.5.1 Results from Concrete Cubes

Initial deterioration on the concrete cubes was surface pitting after one week, with cement particles falling off. This gradually became worse, and after two weeks, small sand particles started to fall off the cube from mix C108. After several weeks, cracks started to appear across the corners of the C107 and C108 cubes, and across the surfaces of the other cubes. These cracks gradually widened and deepened until fragments started to break off the specimens. At the same time, larger aggregate particles were becoming exposed and were falling off. The cube from mix C103 stayed relatively intact during the test, but showed a marked swelling on the top face.

After one year most of the surface concrete on all of the specimens was weak and crumbly, but all except the cube from mix C104 had a solid core of material. Volumes of material lost from the concrete measured after one year are between 0.15 and 0.38 litres, (15 to 38%). Results are summarised in Table 8.7.

8.5.2 Results from Mortar Cubes

As with the concrete, the first sign of deterioration in the mortar cubes was a loss of cement and small sand particles after one to two weeks, resulting in pitting of the surface. Visible disruption caused by swelling was seen in mix M126 after three weeks. After four weeks, flaking was visible on all specimens and cracks started to appear. After two months the cracking became quite severe in the cubes from mixes M124, 5 and 6. After six months, very severe cracking across the corners of the M126 cube led to the corners breaking off, and after seven months, a crack across the middle of the M124 cube.
caused it to split in half. The volumes of material lost from the mortar cubes after one year were up to 0.28 litres. Results are summarised in Table 8.8.
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<th>Elapsed Time (minutes)</th>
<th>K (m/s)</th>
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* Specimen identification: -C103/1 Mix Cl, w/c=0.3, specimen No.1 (specimen No.2 is /2)
† Specimens broke when pressure was applied

**TABLE 8.1 Water Permeability Results from Cl Concrete Mixes**
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* Mix identification: C103/1, Mix Cl, w/c=0.3, /1=specimen No.1

TABLE 8.2 Air Permeability Results from C1 Concrete Mixes
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*Mix designation M113A: c:s ratio=1:1, w/c ratio=0.3, specimen A
%DF=Percentage of Dry Flow; %S=Percentage Saturation

**TABLE 8.3 Percentage Dry Flows and Percentage Saturations from Mortar Specimens**

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<td>41/81</td>
<td>41/65</td>
<td>41/44</td>
<td>76/23</td>
</tr>
<tr>
<td>M147A</td>
<td>9/81</td>
<td>18/60</td>
<td>27/41</td>
<td>45/23</td>
</tr>
<tr>
<td>M147B</td>
<td>13/77</td>
<td>22/60</td>
<td>26/43</td>
<td>43/23</td>
</tr>
<tr>
<td>M148A</td>
<td>12/81</td>
<td>22/58</td>
<td>26/39</td>
<td>49/20</td>
</tr>
<tr>
<td>M148B</td>
<td>10/77</td>
<td>20/55</td>
<td>28/39</td>
<td>48/21</td>
</tr>
<tr>
<td>M149A</td>
<td>10/79</td>
<td>17/60</td>
<td>27/39</td>
<td>42/21</td>
</tr>
<tr>
<td>M149B</td>
<td>13/77</td>
<td>19/58</td>
<td>25/40</td>
<td>40/19</td>
</tr>
<tr>
<td>M156A</td>
<td>23/76</td>
<td>25/59</td>
<td>27/46</td>
<td>39/20</td>
</tr>
<tr>
<td>M157A</td>
<td>29/79</td>
<td>32/58</td>
<td>29/42</td>
<td>68/22</td>
</tr>
<tr>
<td>M157B</td>
<td>44/78</td>
<td>21/54</td>
<td>39/35</td>
<td>86/86</td>
</tr>
<tr>
<td>M158A</td>
<td>26/80</td>
<td>18/57</td>
<td>28/37</td>
<td>59/18</td>
</tr>
<tr>
<td>M159B</td>
<td>17/74</td>
<td>38/54</td>
<td>43/40</td>
<td>81/19</td>
</tr>
<tr>
<td>M159A</td>
<td>17/79</td>
<td>0/62</td>
<td>21/23</td>
<td>58/20</td>
</tr>
<tr>
<td>M150A</td>
<td>9/77</td>
<td>9/60</td>
<td>20/39</td>
<td>42/17</td>
</tr>
<tr>
<td>M150B</td>
<td>15/77</td>
<td>55/60</td>
<td>23/39</td>
<td>32/18</td>
</tr>
<tr>
<td>M151A</td>
<td>11/79</td>
<td>18/62</td>
<td>25/44</td>
<td>38/22</td>
</tr>
<tr>
<td>M151B</td>
<td>11/72</td>
<td>11/58</td>
<td>11/40</td>
<td>18/19</td>
</tr>
</tbody>
</table>

*Mix designation M146A: c:s ratio=1:1, w/c ratio=0.3, specimen A
%DF=Percentage of Dry Flow rate; %S=Percentage of Saturation.

**TABLE 8.3 Percentage Dry Flows and Percentage Saturations from Mortar Specimens (cntd)
### TABLE 8.4 Compressive Strengths of Concrete Specimens

<table>
<thead>
<tr>
<th>Mix*</th>
<th>$f_{c1}$</th>
<th>$f_{c2}$</th>
<th>$f_{c3}$</th>
<th>$\bar{f}_{c}(N/mm^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C103</td>
<td>81.4</td>
<td>77.8</td>
<td>81.1</td>
<td>79.9</td>
</tr>
<tr>
<td>C104</td>
<td>65.8</td>
<td>69.6</td>
<td>70.2</td>
<td>68.3</td>
</tr>
<tr>
<td>C105</td>
<td>58.2</td>
<td>58.2</td>
<td>55.8</td>
<td>57.2</td>
</tr>
<tr>
<td>C106</td>
<td>46.6</td>
<td>46.6</td>
<td>48.5</td>
<td>47.1</td>
</tr>
<tr>
<td>C107</td>
<td>36.6</td>
<td>34.6</td>
<td>36.4</td>
<td>35.8</td>
</tr>
<tr>
<td>C108</td>
<td>28.4</td>
<td>29.0</td>
<td>27.7</td>
<td>28.3</td>
</tr>
<tr>
<td>C203</td>
<td>66.4</td>
<td>68.1</td>
<td>-</td>
<td>67.0</td>
</tr>
<tr>
<td>C204</td>
<td>58.5</td>
<td>64.4</td>
<td>-</td>
<td>61.3</td>
</tr>
<tr>
<td>C205</td>
<td>49.2</td>
<td>49.3</td>
<td>-</td>
<td>49.1</td>
</tr>
<tr>
<td>C206</td>
<td>37.6</td>
<td>40.7</td>
<td>-</td>
<td>39.0</td>
</tr>
<tr>
<td>C207</td>
<td>30.9</td>
<td>30.3</td>
<td>-</td>
<td>30.5</td>
</tr>
<tr>
<td>C208</td>
<td>24.9</td>
<td>24.2</td>
<td>-</td>
<td>24.5</td>
</tr>
</tbody>
</table>

*Mix designation C103: concrete mix Cl; w/c ratio=0.3.
fc1=failure stress of cube No.1. fc=mean cube stress

### TABLE 8.5 Differences Between Calculated and Achieved Strengths from Cl Concrete Mixes

<table>
<thead>
<tr>
<th>Mix*</th>
<th>$f_{c}$</th>
<th>Margin</th>
<th>$f_{t}$</th>
<th>$f_{a}$</th>
<th>(f$<em>{t}$-f$</em>{a}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C103</td>
<td>69.0</td>
<td>13.0</td>
<td>82.0</td>
<td>79.9</td>
<td>+2.1</td>
</tr>
<tr>
<td>C104</td>
<td>53.0</td>
<td>13.0</td>
<td>66.0</td>
<td>68.3</td>
<td>-2.3</td>
</tr>
<tr>
<td>C105</td>
<td>40.0</td>
<td>13.0</td>
<td>53.0</td>
<td>57.2</td>
<td>-4.2</td>
</tr>
<tr>
<td>C106</td>
<td>30.0</td>
<td>13.0</td>
<td>43.0</td>
<td>47.1</td>
<td>-4.1</td>
</tr>
<tr>
<td>C107</td>
<td>23.0</td>
<td>13.0</td>
<td>36.0</td>
<td>35.8</td>
<td>+0.2</td>
</tr>
<tr>
<td>C108</td>
<td>17.0</td>
<td>11.5</td>
<td>28.5</td>
<td>28.3</td>
<td>+0.2</td>
</tr>
</tbody>
</table>

N.B. All strengths in N/mm$^2$. fc=characteristic strength; ft=target strength; fa=achieved strength; Margin for 5% defective

*Mix designation C103: concrete mix Cl with w/c ratio=0.3
<table>
<thead>
<tr>
<th>Mix*</th>
<th>$f_{c1}$</th>
<th>$f_{c2}$</th>
<th>$\bar{f}_c$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M113</td>
<td>70.4</td>
<td>61.9</td>
<td>66.2</td>
</tr>
<tr>
<td>M114</td>
<td>58.0</td>
<td>54.6</td>
<td>56.1</td>
</tr>
<tr>
<td>M115</td>
<td>32.0</td>
<td>32.6</td>
<td>32.2</td>
</tr>
<tr>
<td>M116</td>
<td>24.3</td>
<td>24.5</td>
<td>24.3</td>
</tr>
<tr>
<td>M123</td>
<td>61.8</td>
<td>70.8</td>
<td>66.1</td>
</tr>
<tr>
<td>M124</td>
<td>56.9</td>
<td>58.8</td>
<td>57.7</td>
</tr>
<tr>
<td>M125</td>
<td>45.4</td>
<td>44.4</td>
<td>44.8</td>
</tr>
<tr>
<td>M126</td>
<td>33.4</td>
<td>34.1</td>
<td>33.6</td>
</tr>
<tr>
<td>M133</td>
<td>43.0</td>
<td>36.8</td>
<td>39.8</td>
</tr>
<tr>
<td>M134</td>
<td>56.4</td>
<td>62.8</td>
<td>60.9</td>
</tr>
<tr>
<td>M135</td>
<td>50.2</td>
<td>51.3</td>
<td>50.6</td>
</tr>
<tr>
<td>M136</td>
<td>40.6</td>
<td>41.6</td>
<td>41.0</td>
</tr>
<tr>
<td>M137</td>
<td>30.8</td>
<td>31.9</td>
<td>31.4</td>
</tr>
<tr>
<td>M138</td>
<td>20.8</td>
<td>20.0</td>
<td>20.3</td>
</tr>
<tr>
<td>M145</td>
<td>41.5</td>
<td>37.1</td>
<td>39.2</td>
</tr>
<tr>
<td>M146</td>
<td>38.4</td>
<td>35.5</td>
<td>36.9</td>
</tr>
<tr>
<td>M147</td>
<td>27.8</td>
<td>30.0</td>
<td>28.8</td>
</tr>
<tr>
<td>M148</td>
<td>20.9</td>
<td>21.4</td>
<td>21.1</td>
</tr>
<tr>
<td>M149</td>
<td>14.4</td>
<td>15.4</td>
<td>14.9</td>
</tr>
<tr>
<td>M156</td>
<td>28.5</td>
<td>23.9</td>
<td>26.1</td>
</tr>
<tr>
<td>M157</td>
<td>22.1</td>
<td>18.8</td>
<td>20.3</td>
</tr>
<tr>
<td>M158</td>
<td>17.2</td>
<td>17.7</td>
<td>17.4</td>
</tr>
<tr>
<td>M159</td>
<td>14.4</td>
<td>14.5</td>
<td>14.4</td>
</tr>
<tr>
<td>M150</td>
<td>7.8</td>
<td>7.6</td>
<td>7.7</td>
</tr>
<tr>
<td>M151</td>
<td>7.4</td>
<td>7.5</td>
<td>7.4</td>
</tr>
</tbody>
</table>

*Mix designation for M113: c:s ratio=1:1; w/c ratio=0.3
$f_c$=mean stress for each mix

TABLE 8.6 Compressive Strengths of Mortar Specimens
### TABLE 8.7 Details of Concrete Specimens after One Year of Repeated Salt Immersion and Oven Drying

<table>
<thead>
<tr>
<th>Mix*</th>
<th>Volume Remaining (l)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>C103</td>
<td>0.75</td>
<td>Largely fragmented, only the top part intact, although badly cracked and distorted due to the expansion.</td>
</tr>
<tr>
<td>C106</td>
<td>0.62</td>
<td>Sample completely fragmented, no pieces larger than 30mm across.</td>
</tr>
<tr>
<td>C105</td>
<td>0.85</td>
<td>Bottom corners gone, but top of sample intact. Cement etched away and coarse sand exposed on the sides.</td>
</tr>
<tr>
<td>C106</td>
<td>0.70</td>
<td>All corners gone, but core still sound. Aggregates exposed as in C105.</td>
</tr>
<tr>
<td>C107</td>
<td>0.85</td>
<td>Bottom of cube crumbled, but top 3/4 still sound. Corners rounded, but not cracked off. Coarse aggregate exposed.</td>
</tr>
<tr>
<td>C108</td>
<td>0.80</td>
<td>Most of bottom half crumbled, but top half still sound. Coarse aggregate exposed.</td>
</tr>
</tbody>
</table>

*Mix designation C103: Concrete C1 w/c ratio=0.3

### TABLE 8.8 Details of Mortar Specimens after One Year of Repeated Salt Immersion and Oven Drying

<table>
<thead>
<tr>
<th>Mix*</th>
<th>Volume Remaining (l)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>M123</td>
<td>1.00</td>
<td>Cube still intact, only the surface cement etched away exposing sand grains.</td>
</tr>
<tr>
<td>M124</td>
<td>0.97</td>
<td>Cube split into two pieces, but otherwise intact, surface as for M123.</td>
</tr>
<tr>
<td>M125</td>
<td>0.95</td>
<td>Intact, but badly cracked, broke into two pieces when knocked. Surfaces as for M123.</td>
</tr>
<tr>
<td>M126</td>
<td>0.72</td>
<td>Three corners missing, and badly cracked. Easily broken into pieces using hand pressure, but individual pieces still sound. Surfaces as for M123.</td>
</tr>
</tbody>
</table>

*Mix designation M123: c:s ratio=1:2; w/c ratio=0.3
FIGURE 8.1 Relationship Between Water Permeability and Time for Mortar Mix M124

FIGURE 8.2 Corrected Relationship Between Permeability Coefficient and Time for Mortar Mix M124
FIGURE 8.3 Relationship Between Ln Water Permeability and Ln Time for Mortar Mixes M134/5/6/7/8
FIGURE 8.4 Relationship Between Water Permeability and W/C Ratio for Mortar Mixes
FIGURE 8.5 Relationship Between ln Water Permeability and W/C Ratio for C2 Concrete Mixes
FIGURE 8.6 Relationship Between Ln Water Permeability and Ln Time for C2 Concrete Mixes
FIGURE 8.7 Relationship Between Ln Water Permeability and Ln Time for C1 Concrete Mixes

N.B. Broken lines are extensions of lines from C2 mixes.
FIGURE 8.8 Relationship Between Air Permeability and W/C Ratio for Mortar Mixes
FIGURE 8.9 Relationship Between Air Permeability and Percentage Sand for Mortar Mixes
FIGURE 8.10 Relationship Between Air Flow and Percentage Saturation for Mortar Mixes
FIGURE 8.11 Relationship Between Weight Change and Time from Vapour Permeability Tests on Mortar Mix M137.
FIGURE 8.12 Relationship Between Vapour Permeability and W/C Ratio for Mortar Mixes

FIGURE 8.13 Relationship Between Vapour Permeability and W/C Ratio for C2 Concrete Mixes
FIGURE 8.14 Relationship Between Compressive Strength and W/C Ratio for Mortar Mixes
FIGURE 8.15 Relationship between Compressive Strength and Percentage Sand for Mortar Mixes
9 DISCUSSION AND INTERPRETATION OF RESULTS
OBTAINED FROM IN-SITU TESTING TECHNIQUES

9.1 Initial Surface Absorption Test

During this research, the ISAT was used on site concrete as well as the specimens cast in the laboratory. As discussed in section 6.3.1, various problems were experienced with the ISAT when it was first used, but these were overcome.

9.1.1 Tests on Site Concrete

Site tests were carried out using the ISAT at three different locations. These were:

i) Junction 29 of the M1 motorway, to assess the condition of concrete and repairs on the overbridge crossheads;

ii) a precast structural column in the Civil Engineering Laboratory, specifically to assess the ease with which the test can be carried out; and,

iii) Budbrook water tower, to assess different sprayed fibre concrete repairs.

In each of these three cases, a different surface was encountered. At location i) there was a smooth cast face, at location ii) there was a trowelled face, and at location iii) there was a rough sprayed finish. In case i) the tests were unsuccessful, because the sealing method had not been perfected, but results were obtained from locations ii) and iii).

The tests performed on the laboratory column were principally to establish the feasibility of the test for site use. They were also used initially to try and establish a relationship between ISA and time, and to assess the repeatability of the test if it was carried out in the same position on several occasions. Levitt\(^{277}\) has stated that the test cannot be carried out in the same location more than once because the concrete is altered by the water under pressure.

All of these tests show decreasing ISA on each retest at a particular test time (Table 9.1). This is assumed to be the
result of the concrete being wetter on each retest.

Flows were recorded at intervals during each test period, so it was possible to examine the short term trends of the results. It can be seen that the relationship between ISA and time at each test time fluctuates (Figure 9.1), although the general trend of ISA is downwards with time. The same trend was observed by Tan (346) who measured ISA over a one minute period, every two minutes after the start of the test.

A possible explanation for these short term variations in the ISA is that the time period over which they were measured was short (15 seconds), and the flows recorded were low (<0.05 ml/m²/s), so small variations in results appear as large relative variations. In addition, these tests were carried out at an early stage in the research when problems were still being experienced with this test method.

All of the results recorded indicate a low permeability concrete. This may be the case, although it was only later realised that the moisture content of the concrete has a significant effect on the ISA.

At Budbrook water tower three tests were performed. Despite the rough surface, it was still possible to get a seal against the concrete. The results obtained are 0.08, 0.21 and 0.07 ml/m²/s. The highest value is from a panel which showed leakage along small cracks which were not visible when they were dry. These cracks are the probable reason why one result is three times as high as the other two. The other two results were obtained from separate panels, and suggest a low permeability.

Unfortunately the moisture content could not be assessed for these tests because the rough surface prevented the M49 Moisture Monitor from being used. However, because the tests were all carried out on the same side of the building, it was assumed that the conditions for all of the test areas were the same, so the tests were valid for comparative purposes. It was encouraging to see the test in operation on site, despite the poor quality of the finish (Plate 9.1).

In all of the above tests the cap was supported by
propping with a length of wood or steel. This was found to be adequate under the given conditions, though some users of this apparatus have reported using bolts to fix the apparatus to the concrete (347).

Although these tests were limited, the ISAT appears to be a valid site test which should make a useful tool for site permeability measurement when the in-situ moisture content can be measured.

9.1.2 Initial Tests on Laboratory Concrete Mixes

Initial tests on air dry concrete gave results ranging from 0.058 to 0.297 ml/m²/s for the beams and 0.026 to 0.335 ml/m²/s for the cubes (Results are summarised in Table 9.2). Only three tests were unsuccessful due to leaks. The relationship between ISA and w/c ratio is as expected with ISA increasing as w/c ratio increases (Figure 9.2). Unfortunately as stated earlier, this is not very useful because the w/c ratio was not the only variable in the mix. Results for the beams and cubes are similar, with little variation between different specimens.

Results from the oven dried beams show a different relationship when compared with w/c ratio (Figure 9.3). Apart from the ISA being higher generally, it no longer increases with w/c ratio as before. This demonstrates a serious problem with the test, namely that in-situ concrete will be air dry, so concrete with a low ISA in situ will not necessarily be a low permeability concrete. In addition comparing different concrete mixes, air dried under similar conditions, does not necessarily give the same relationship that occurs between the same mixes if they are oven dried. For example, the average ISA from mix C103 is lower than that from mix C106 when the specimens are air dried (Figure 9.2), but the situation is reversed when the same specimens are oven dried (Figure 9.3).

Another problem which is illustrated is the wide range of results recorded for each mix. This is a problem when considering the number of tests that are required to obtain a good statistical average for site use.

The pattern of results for beams from the C2 mixes is slightly different from the C1 mixes, although there is a
reasonable comparison (Figure 9.4). It is important to note that these two mixes were not cured and dried under the same conditions, so this may have contributed to the variations.

There are two sets of results from the cubes, relating to the curing periods of four and seven days. In all cases, the longer curing period gives lower values of ISA, although the differences are not very large (Figure 9.5). It is worth noting that except for cubes from mix C203, the ISAs of the cubes were lower than those of the beams. The reason for the large variation in the C203 results is not known.

9.1.3 Initial Tests on Mortar Mixes

Results from the oven dried mortar specimens show an interesting relationship, which is similar to that expected for the laboratory permeability tests. The results show increasing ISA with increasing w/c ratio, and decreasing ISA with increasing c:s ratio for both beams and cubes (Figures 9.6 & 9.7). The implications of this are that by increasing both the w/c and c:s ratios, it is possible to reduce the ISA of the mix. This is thought to be a result of the decrease in the permeable matrix.

The relationship between percentage sand and ISA (Figure 9.8) shows a trend towards an optimum c:s ratio of 1:4. This is presumed to be a limit, above which the w/c ratio becomes so high that there are always open capillary channels once the water has evaporated, and below which the permeable matrix increases in volume, so allowing a larger flow. The only exceptions to this are at high w/c and c:s ratios. This is thought to be a result of segregation in the mix, while the mixes were wet, leading to a reduction in the planned w/c ratio as the solids settled and free water rose to the surface.

In the majority of cases the ISA of the cubes is lower than the ISA of the beams (Table 9.3). Variations in the results from individual cubes are sometimes less than 0.05ml/m²/s, but with some of the poorer specimens the variation is as great as 0.50ml/m²/s (for example cube No.3 from mix M151). Similarly with the beams, variation in some of the results is quite high in some mixes, and low in others. Some of these variations may be a result of surface cracking.
(mix M116) and some may have been caused by variations between the specimens for example mix M125, where the variation in results from each cube is less than 0.05ml/m²/s, but the maximum variation between the cubes is 0.15ml/m²/s.

These variations amount to differences in some instances of more than ±10%, from the average of each set of beam results (Table 9.4). The results from the cubes vary by more than ±5% from the averages in some cases, but the majority vary by less than this. This is one aspect of this method which should aid its' acceptance for site use, although it must be remembered that these results were obtained from oven dry beams, and it is possible that greater variations would occur on air dry in-situ concrete.

One source of error in the results from the beams may be variations in the contact area of the cap. The contact area used for the calculations is that within the cap boundary, but the actual wetted area on the beams depends upon the sealant boundary. The cap area is 5850mm², with an edge length of approximately 320mm. If the sealant boundary is 1mm either side of the edge of the cap, then the error in area is 320mm², which is approximately 5% of the assumed contact area. In some cases it was noted that the sealant was up to 5mm outside of the correct contact area. In addition, if the sealant does not seal properly, then water can seep underneath it and increase the contact area further. The contact area of the cubes is defined by the rubber sealing gasket, so this cannot be considered to be a source of error on these tests.

Surface cracking may also be responsible for some variation in results leading to high ISAT values in what are otherwise low permeability mixes. These occurred in the beams or cubes, probably as a result of shrinkage during oven drying, drilling the Figg test holes, or both. They were not always visible (unless the material is wetted) so were not always detected until the tests were underway, or until after they had been completed.

9.1.4 Effects of Moisture Content

The relationship between the percentage saturation and the ISA of the concrete (Figure 9.9), shows that as the specimens dry
out (hence the saturation is reduced), there is a gradual increase in the ISA. This Figure shows a slow increase in ISAs upto 60-70% saturation after which they increase more rapidly to peak between 0-20% saturation, and then they fall. It is notable that the values of ISA are different on the retests from the original values, probably because the condition of the concrete was not the same in the retests. Comparing the retest results with the original results, shows that the ISA is lower for all mixes, except for mix C104, even though the concrete was drier in that case (Figure 9.10). Although mixes C107 and C108, were dried to lower weights for the retests, the ISA of these mixes at the 1% saturation values is still a lot lower than the original values.

There are two likely explanations for the drop in results at low saturations:

i) the concrete is less permeable, due to continuing hydration or related processes each time the concrete is wetted then dried; or

ii) the recorded ISA is lower because the contact area was reduced by the improvements in the sealing method.

This will require investigation in any future research, because the effects of this would be significant for comparing different sets of results from different concrete mixes. It is important to realise that in-situ concrete will not dry out to this extent under normal conditions (When calibrating the M49, it was found that after 35 days of air drying, the beam from mix C103 had only reached 70% saturation). Figures quoted by Dhir et al(275) suggest that concrete only loses a small percentage of its total free water when air drying.

Comparing the results from the mortar specimens with percentage saturation shows similar behaviour to the concrete, with an increase in ISA up to a peak, followed by a drop (Figure 9.11). In the majority of cases, the dry weights achieved in the retests were less than those obtained in the initial tests, although as with the concrete, the values of ISA obtained in the retests are lower than the original values (Figure 9.12). The only exceptions to this are mixes M115 and M159 which did not achieve lower weights on retesting. It is important to note that in some cases the differences in the two sets of results is quite large.
The relationship between ISA and percentage saturation for the mixes with a w/c ratio of 0.6, shows mixes M116 and M126 separated from the rest of the results from the onset of drying whereas mixes M136, M146 and C106 are all very close together until the saturation is below 60% saturation, after which they diverge (Figure 9.13). The relative order of the results from the mortar shows that ISA increases as the cement content increases. The concrete has a lower ISA down to 20% saturation, then it rises sharply and cuts across the two lower mortar curves. Considering the mortar mixes, it is clear that mixes M116 and M126 would show different readings even if they were tested at a high percentage saturation, but mixes M136 and M146 would have to drop to below 60% saturation before it would be possible to differentiate between them.

9.2 Figg Air Test

The Figg air test was only used on laboratory specimens during this research. As explained in Section 6.3.2, the test method and the method of recording results was modified slightly, so comparison with other work must take this into consideration.

9.2.1 Initial Tests on Laboratory Concrete Mixes

Figg air times for air dried concrete were much higher than expected (Table 9.2). It was a result of these very high test times that it was decided to modify the method of recording results. Little indication is given in the literature relating to this test that long test times are a problem, although the report by Cather et al\(^{(24)}\), suggests that increased moisture content increases the test times, and the categories proposed in that report for different concrete qualities are for oven dried specimens.

As with the ISAT, no real comparison can be made between the different mixes. The only apparent relationship between the Figg air time and the w/c ratio is an overall decrease in Figg air time with increasing w/c ratio, although there is a very large spread of results (Figure 9.14).

Tests carried out on oven dry concrete all took less than 5 minutes, except for mixes C104 and C105, with mix C104 having
some test times higher than 1000s (for a 5kPa pressure change). The categories proposed by Cather et al(24), suggest that a time in excess of 1000 seconds is consistent with polymer modified concretes. Plotting the test times against w/c ratio shows an increase in test time as w/c ratio increases from 0.3 to 0.4, followed by a decrease in test time as the w/c ratio increases from 0.4 to 0.8 (Figure 9.15). It can be seen that the results from the C2 beams are lower than those from the C1 beams (Figure 9.16a), with correspondingly less difference between results from different w/c ratios. This difference in results between C1 and C2 mixes may be a result of drying the C2 specimens to a lower weight than the C1 specimens.

It is important to note that the variation in results from each of the C2 beams (Table 9.5) is still large in percentage terms even though the recorded test times are low, with the worst case being mix 204. This has serious implications for in-situ testing, where more consistent results would be required for an accurate assessment of the concrete, but as shown earlier, the tests on concrete in an air dry condition gave an even greater variation in results.

Average results for the cubes compare quite well with the beam results, although individual cube results vary considerably in some instances (Table 9.6). Comparing the test times from the cubes cured for four and seven days with w/c ratio (Figure 9.16b), it can be seen that for mixes C204-208 the seven day curing period gives longer test times than the four days curing period. The results from mix C203 vary a great deal, so it was assumed that there was an error when recording these.

9.2.2 Initial Tests on Mortar Mixes

In the majority of cases, no problems were experienced when testing the mortar specimens, although some results are higher than the expected values proposed by Cather et al(24). In some instances (for example mixes with c:s ratio of 1:1) the mortar was cracked so it was either not possible to reduce the pressure to start the test, or very short test times were recorded. In other cases (mixes M133 and M145), it was not possible to reduce the pressure to 45kPa absolute because the specimens were so permeable. In cases were the starting
pressure could not be reached the test times were recorded as zero seconds.

The relationship between Figg air time and w/c ratio for the beams and cubes shows that the recorded times drop rapidly towards a w/c ratio of 0.6 (Figures 9.17 & 9.18). The hierarchy of mixes above a w/c ratio of 0.6 is of increasing cement content corresponding to increasing Figg air time, which is the reverse of the trend shown by the ISAT. Comparing the Percentage of sand with Figg air time shows a good relationship for mixes with c:s ratios above 1:3, but a large variation in the mixes below this (Figure 9.19).

The variation in results from each mix is quite large except at high w/c ratios where the test times are all low (Table 9.7), and it can be seen that in the majority of cases the average test times from the cubes are higher than the test times from the beams.

It is possible that this variation in results is the result of variations in the test holes, resulting from the drilling, or errors when positioning the foam rubber plugs. Considering the small hole size, a 1mm error when placing the foam rubber plug would result in a 5% change in the area of the side of the hole. Micro-cracking may also influence the condition of the mortar in the test area, especially in the weaker mixes. This problem has been highlighted by Montgomery(283) and others(20).

The variation in the results suggests that if the Figg air times are consistently low, then the specimen probably has a w/c ratio higher than 0.6, whereas if the Figg air times for a mix vary between low and high values, then the w/c ratio is likely to be below 0.6. This could be a useful indicator of potential durability as a w/c ratio of 0.6 is considered by some to be the upper limit for a durable concrete(77).

9.2.3 Effects of Moisture Content

As with the ISAT, it was found that the test times on the retests were different from the initial tests. In Figure 9.20 it can be seen that the test times from the C1 mixes are all lower on the retests, although they are much closer to the
initial results from the C2 mixes. Again it is clear that variations in 'oven dry' condition are significant, and in this case they also result in a change in the relative positions of the different mixes in terms of which give the highest test times. The differences in the two sets of results from the mortar are even more significant (Figure 9.21). It is clearly difficult to compare the two sets of results because some of the test times from the different mixes vary by such a large amount. With concrete and mortar specimens, the results from the retests are all less than 20s/kPa, which for the concrete is below the values proposed by Cather et al\(^{(24)}\).

Comparing the test times with different saturations, the test times are initially high for all mixes, but fall rapidly as the percentage saturation falls (Figures 9.22-26). It is interesting to note the results from individual test holes, as the rates at which the test times drop varies widely within mixes. In some specimens (for example mixes M113 and C105) there is a large difference between individual holes (Figures 9.22 & 9.23). In others (mixes M114 and C106) all of the results are close together, with the exception of one hole (Figures 9.24 & 9.25), whilst in others, notably as the w/c ratio gets higher, all of the holes exhibit similar results (Figure 9.26).

It is not entirely clear what causes this behaviour but it is possible that in the more permeable mixes (high w/c ratios) a single factor (such as the material permeability) controls the Figg air time, whereas in the less permeable mixes (low w/c ratios) there may be two or more factors (such as material permeability and cracking) which results in variations in the Figg air times depending upon which factor is dominant. This may be the explanation for the wide variation in test times at low w/c ratios in the tests on oven dry specimens. Whatever the explanation, this will presents problems of interpretation if this method is used in situ.

Considering the effect of c:s ratio on the Figg air time as the specimens dry out, it appears that the rate at which the Figg air time falls is more rapid as the amount of sand increases. The rich mixes with low w/c ratio (such as M113 and 4) only give low test times when they have reached fairly low saturations (Figures 9.22 & 9.24), whereas leaner wetter mixes
(such as M157, 8 and 9) give low test times at high saturations (Figure 9.27). It is noticeable that in some of the mortar mixes similar results are obtained from different w/c ratios at particular c:s ratios for example mixes M157 and 9, M136 and 7, and M148 and 9 (Figures 9.27-29).

Comparing the test times from the specimens with a w/c ratio of 0.6 with percentage saturation (Figure 9.30), there is no clear distinction between the mortar mixes at low saturations. Above 50% saturation however, some distinction can be made between the mixes. It is important to note that the order the specimens in terms of increasing Figg air times is not the same as when the specimens were oven dry. The order of increasing Figg air time with c:s ratio on the retests is 1:1, 1:2, 1:3, 1:4, whereas at 50% saturation it is 1:4, 1:1, 1:3, 1:2. This again illustrates the problem of interpreting results from in-situ tests, or specifying categories using results obtained from oven dry specimens.

9.3 Figg Water Test

These tests were carried out in the same test holes as the Figg air tests. No site tests were carried out. As described earlier (Section 6.3.3), a number of practical problems were encountered with this test, and this may account for some of the variations in the results.

9.3.1 Initial Tests on Laboratory Concrete Mixes

As with the Figg air tests, the tests on the air dried concrete gave very high test times, with large variations within each mix. Not all of the Figg water tests attempted were successful, and there was insufficient time to carry out all of the tests planned, as the time for 0.1ml to flow into the concrete was frequently in excess of one hour. As with the other tests, this apparent 'low' permeability is attributed to the fact that the concrete was only air dry when it was tested. The relationship between Figg water time and w/c ratio (Figure 9.31) shows a similar relationship to that exhibited by the Figg air tests (Figure 9.14), with test times increasing from a w/c ratio of 0.3 to 0.4, and decreasing from w/c ratio of 0.4 to 0.8. Results are summarised in Table 9.2

159
The relationship between test times and w/c ratio for tests on the oven dry concrete (Figure 9.32), shows much lower test times generally, although some of the test times from the C1 mixes are still in excess of 1800s. The highest average values occur with mix C104 with the lowest from mix C108. With the exception of mix C103, these results are as expected, with the test time decreasing as the w/c ratio increases. The C2 mixes also have maximum test times from mix C204, but there is little difference between the mixes with w/c ratios above 0.5. All of the times recorded for C2 mixes are lower than those for the equivalent C1 mixes except for C208, which is slightly higher.

The relationships between the C2 cube test times and w/c ratio does not follow as clear a pattern as that of the beams (Figure 9.33). Comparing the different curing regimes, the results are as expected for mixes C205-8, but the results from mixes C203 and C204 are not considered to be very reliable due to problems experienced with the test on these mixes.

9.3.2 Initial Tests on Mortar Mixes

The relationships between the Figg water test times and w/c ratios for mortar beams and cubes (Figures 9.34 & 9.35) show similar patterns to those obtained from the Figg air test, with low test times for w/c ratios above 0.6. Comparing the different test times from the different mixes (Table 9.8), it can be seen that some of the test times are very low (5s from mix M150), and some are high (3861s from mix M125). However, most mixes exhibit some results below 600s (10 minutes) which justifies the choice of 10 minutes as a standard test time. As with the Figg air test, the test times vary widely within each mix. With the low w/c ratios, the average test times vary from low to high, whilst at w/c greater than 0.6, the average test times are all less than 200 seconds.

It has been suggested already that at low w/c ratios, more than one mode of fluid ingress occurs (Section 9.2.3). This has also been reported by Chantree(279), who concluded that at early stages in the test small cracks produce a high flow rate, whilst later in the test (once these cracks have been filled), the permeability of the bulk of the material becomes the dominant factor.
The mixes with w/c ratios in excess of 0.6, have relatively low strengths, so cracks may occur when drilling the test holes and, as the mixes are more permeable generally, the results will indicate a high permeability material from both causes. Below w/c ratios of 0.6, the mixes are stronger, so there could be no cracks present, and as these are low permeability mixes, the results would indicate a low permeability. If a few cracks are present in these mixes, then these will give a high initial flow before they fill up with water, and so give high test times.

9.3.2 Effects of Moisture Content

Figg water times for the retests were different from the initial tests. For the concrete mixes the difference in results is high, especially for mixes C103-6 with the retest times being much lower than the initial test times (Figure 9.36). In this case the results from the C2 mixes are only similar for w/c ratios above 0.5. The retest times from the mortar specimens however show a different relationship when compared with w/c ratio (Figure 9.37). At the high w/c ratios, the retest times are much higher in some cases than the initial test times, for example the average retest time from mix M149, are in excess of all of the others except M125. Only a few of the retest times such as those from mixes M115, M125 and M126 are close to the initial test times. It is important to note however that the times recorded are not necessarily the lowest values obtained.

Comparing the change in Figg water times with saturation, the initial drop in test times is very rapid, and most mixes appear to reach a shallow and fairly constant gradient around 70% saturation (Figure 9.38). It was found in some tests that at saturations below 30%, the trend of the results changes, and test times start to increase (Figure 9.39). As with the Figg air test, it can be seen that the results vary between the test holes in individual mixes (Figures 9.40-42).

Comparing the mixes with a w/c ratio of 0.6 (figure 9.43), there is a clear separation between the different mixes, until a low saturation is reached. At 50% saturation for example the concrete has the highest test times, while the mortar mixes have increasing test times for c:s ratios of 1:1,
1:4, 1:3 and 1:2 respectively, which is the same order achieved with the initial dry tests and with the Figg air tests at 50% saturation.

9.4 The Egg Test

The Egg test was used on one site location (Budbrook water tower) as well as on the laboratory specimens. It proved to be the easiest of the four in-situ permeability tests to perform, and the small amount of preparation that it requires makes it ideal as an in-situ test.

9.4.1 Tests on Site Concrete

The test was used in situ with the ISAT on the Budbrook Water Tower (Section 9.1.1). Due to the nature of the sprayed concrete surface it was necessary to alter the sealing method. A layer of the ISAT sealant was first applied to the concrete to produce a smooth surface and then the Egg was affixed to it. Because the moisture content could not be measured, the tests were only useful for comparative purposes. Results varied, the fibre reinforced concrete giving better results than normally reinforced concrete, and panels with a high percentage of fibres giving worse results than those with less fibres. In addition, in one position, a crack was detected after the starting pressure could not be obtained.

Despite the difficulties with the concrete surface, the test was found to be quite satisfactory, and was easier to perform than the ISAT under the same site conditions. Plate 9.2 shows the test set up in situ.

9.4.2 Initial Tests on Laboratory Concrete Mixes

As with the other tests, the test times from air dried concrete were high (Table 9.2). The relationship between Egg test time and w/c ratio (Figure 9.44) is similar to that shown by the Figg air test (Figure 9.14), with higher average test times from the lower w/c ratios. Comparing the results from the beam ends, it can be seen that in all cases the test times from beam end I are lower than those from beam end II. This is presumably because the beam ends I were drier having been open to the air.
whilst drying whilst the ends II were not.

The tests on oven dried concrete gave much lower test times and the relationships between Egg test time and w/c ratio for both sets of concrete beams are similar (Figure 9.45). The relationship between the C2 cube results and w/c ratio (Figure 9.46), shows that at w/c ratios of 0.3 and 0.4, the four day curing gives better times than the seven day curing, whereas for w/c ratios of 0.5-0.8, the seven day curing gives better results, although at a w/c ratio of 0.5, the difference between the two is only small. The cause of the discrepancy at w/c ratios of 0.3 and 0.4 is not known, although results from the other tests also revealed some discrepancies with the two curing regimes at the low w/c ratios.

9.4.3 Initial Tests on Mortar Mixes

Results from the mortar beams generally show a drop in Egg test time with increasing w/c ratio (Figure 9.47). The main exception to this is the small rise in the mixes with a c:s ratio of 1:5 up to a w/c ratio of 0.8, followed by a drop as the w/c ratio increases further. Above a w/c ratio of 0.7 the test times tend towards all low values, whilst below 0.7, there is a large variation between the different c:s ratios. Test times for mixes M113-6 are rather low; this is assumed to be a result of the surface cracks seen on these specimens. As with the Figg air tests it was not possible to reduce the pressure to 45 kPa absolute with some of the more permeable mixes. Comparison of results from beam ends I and II shows that beam ends I generally have higher test times, although in the majority of cases the differences are only small (Table 9.9). There are no obvious differences between results from the beam ends and the beam sides.

The test times from the cubes have a similar relationship to that exhibited by the beams (Figure 9.48). In the majority of cases, the results from individual cubes do not vary much although there is some variation between the cubes in each mix (Table 9.10). This is most noticeable with the specimens with high test times. In a few cases such as mix M113, because of the cracks the test times were only 1 or 2 seconds in some of the test positions these test times were not used for calculating the averages.
Comparing Egg test times with the percentage sand shows an increase in test times as the percentage of sand increases, up to a peak after which the times drop sharply (Figure 9.49). Each w/c ratio appears to have a peak time at a different c:s ratio from the others.

9.4.4 Effects of Moisture Content

As with the other tests, it can be seen that the test times from the retests on the concrete are different from those from the initial tests (Figure 9.50). The concrete test times increase from w/c ratios of 0.3 to 0.6, and then drop from 0.6 to 0.8. The results from the tests on the C2 mixes do not compare well with the results from the retests.

For the mortar beams, the main variations on retesting were lower times from mixes M123-6 and M134-6 (Figure 9.51). These mixes were only dried to give a 10g weight change in 24 hours for the initial tests, instead of 5g in 24 hours. The other mixes have average test times which lie very close to those obtained in the initial tests. Results from individual test positions on some of the beams are also similar in both tests. For example in positions I, II, V, and X on mix M147, the test times from the initial test are 40, 24, 37, and 38s respectively, whereas in the retest, they are 43, 29, 35, and 40s respectively. This shows that differences in results may be due to variations in the material, rather than the test.

When considering the effects of different saturations, as with the other tests, the Egg test times for the concrete get worse as the saturation falls (Figure 9.52), with the test times falling rapidly to between 90 and 70% saturation, after which they fall at a much slower rate to 0%. The relative order of the different mixes varies, which demonstrates that as with the Figg air test different concretes do not give the same order of results at any saturation. The results also vary widely with some mixes (Figure 9.53), or are close together with one position giving outlying results (Figure 9.54).

The test times for the mortar specimens vary a great deal, with some mixes having high test times at low saturations such as mixes M123, M124 and M125 (Figure 9.55). The more permeable mixes give more rapid drops in test times for example
mixes with c:s ratios of 1:1 and 1:5 show rapid drops in the test times from 100-70% saturation (Figures 9.56 & 9.57). This may be the result of water filled cracks and large capillaries near the surfaces of the specimens drying out rapidly. A similar pattern is shown by mixes M148 and M149, but not with M146 and M147 (Figure 9.58). This presents the possibility of more than one process influencing the air permeability, as has been suggested for results from the other tests. A similar pattern can be seen with mixes with c:s ratio of 1:3, with mix M138 giving low results from 70-0% saturation (Figure 9.59).

As with the results from the concrete, test times for the different positions on the specimens vary, and in some cases outliers relate to areas which had cracks in them such as position II on the beam from mix M113 (Figure 9.60).

Comparing the mixes with a w/c ratio of 0.6 (Figure 9.61), it can be seen that below 30% saturation, the order of results for the mortar is the same as for the dry specimens (Test times increasing with the mixes in the order 1:1, 1:2, 1:4, 1:3, concrete). Above 30% saturation, the positions of mixes M136 and M146 are reversed, with mix M146 giving the highest results.

9.5 Moisture Measurement

No indication of the expected trend of results was supplied with the M49. Initial tests were carried out during the testing of the different beams at different moisture contents. Readings were taken at the eight test positions on the sides and the two ends of the specimens.

Plotting the results from all specimens against percentage saturation gives the general relationship for concrete and mortar shown in Figure 9.62. This shows an initial shallow gradient to approximately 50% saturation, followed by a steep drop down to 0% saturation. From these results a broad band of values can be traced from 100-0% saturation. Unfortunately, this band is wide and accurate measurement of moisture content is clearly not practical using this relationship.
The M49 was used to test eight beams to try and obtain a more accurate relationship from these specimens as they were being dried out over a period of time. These beams were saturated then left in a climatic cabinet at 25°C and 50% R.H. to dry out for 48 hours prior to retesting. They were then tested at 7 day intervals for a total of 35 days while being stored in the climatic cabinet under the same conditions.

Unfortunately, the results from these tests also show a similar broad trend to that exhibited by the earlier test results (Figure 9.63). It was noted during the course of this second experiment that although the general trend of the results was downward, the average values from each beam fluctuate with time, and results from individual positions also fluctuate (Figures 9.64 & 9.65).

It is interesting to note that lower results relate to areas which had already had Egg tests carried out on them. This is thought to be due to a thin layer of grease which was left after the test had been performed. This prevented the specimens from becoming totally saturated when they were immersed in water, as a result these positions gave lower results. Also of interest was the rate at which drying took place in an 'impermeable' concrete, and a 'permeable' mortar. It can be seen that after 35 days air drying the concrete had only reached 72% saturation (Figure 9.64), whereas the mortar had reached 28% saturation (figure 9.65). However, the drop in the moisture monitor results for the concrete is more rapid, so the apparent values are still similar.

This test clearly requires further evaluation, which regrettably could not be carried out as part of this research due to a shortage of time.

9.6 Carbonation Measurement

Carbonation depths were measured on cubes from all mixes except mixes C104, 6, 7, and 8. The test is easy to perform, and the only problems encountered were with some of the concrete specimens which did not show very clear carbonation fronts. Results are summarised in Table 9.11.
Comparing the carbonation depths with w/c ratio for the concrete specimens (Figure 9.66), it can be seen that the carbonation depths vary between the different mixes and w/c ratios. It is interesting to note that mix C103 has the lowest carbonation depths even though this was the oldest concrete. These results may have been influenced by the fact that the Cl specimens were not oven dried prior to testing. However, the same is not true for mix C105 which has higher carbonation depths than mix C205, even though mix C205 was oven dried. This discrepancy may be a result of differences in the mixes, or it may indicate that the assumption that carbonation occurs at steady rate is not true. This presents problems for service life prediction.

The different curing times show the expected results for all mixes except C206, with the cubes which were cured for a longer period having lower carbonation depths. In addition, cubes from mix C205 show no significant difference between results from the two curing regimes. A major shortcoming with this experiment was the small number of specimens available, which meant that only one specimen from each mix could be tested.

The relationship between carbonation depth and w/c ratio for the mortar specimens shows a general increase in carbonation depth as w/c ratio increases (Figure 9.67). Untested cubes stored in the laboratory conditions have greater carbonation depths than the cubes from the same mixes which were tested then stored outside (Figure 9.68). It is clear that the carbonation depths are influenced by moisture content, with 'wet' specimens carbonating less than dry ones. This has also been reported by other investigators (9,78,348).

In all specimens there was no visible carbonation around the Figg test holes, which shows that the silicone rubber plugs afford some protection to the concrete after the test has been carried out. Tests using the Germann indicator solution did not show any marked difference in the measured carbonation depths.
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**TABLE 9.1** Results from Initial Surface Absorption Tests on a Laboratory Column
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<th>Type of Specimen</th>
<th>ISAT (ml/m²/s)</th>
<th>FIGG TESTS</th>
<th>Egg Test (s/kPa)</th>
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<td>Water (s/0.1ml)</td>
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*Mix designation: C103 is concrete mix C1 with w/c ratio=0.3

TABLE 9.2 Average Results from In-situ Permeability Tests on Air Dried Concrete Specimens
<table>
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<tr>
<th>Mix*</th>
<th>Cube Nos.</th>
<th>Beams</th>
<th>Cube Nos.</th>
<th>All cubes</th>
<th>Beams</th>
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<td></td>
<td>1   2   3</td>
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<td></td>
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<td>0.15 0.04 0.01</td>
<td>0.15 0.08</td>
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<tr>
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<td>0.14 0.01 0.29</td>
<td>0.28 0.15</td>
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<td>0.00 0.13 0.03</td>
<td>0.33 0.24</td>
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<td>0.32</td>
<td>- - -</td>
<td>-</td>
<td>0.00</td>
</tr>
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<td>0.12 0.13</td>
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<td>0.23 0.05 0.00</td>
<td>0.23 0.20</td>
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<td>-</td>
<td>-</td>
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<tr>
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<td>0.26</td>
<td>0.00 - 0.01</td>
<td>0.03 0.08</td>
<td></td>
</tr>
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<td>0.50</td>
<td>0.02 0.01 0.06</td>
<td>0.08 0.12</td>
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<td>0.09 0.07 0.06</td>
<td>0.11 0.10</td>
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</tr>
<tr>
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<td>0.96</td>
<td>0.00 0.03 0.07</td>
<td>0.07 0.12</td>
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<tr>
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<td>0.26 0.46</td>
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<td>- - -</td>
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<td>0.30 0.24 0.29</td>
<td>0.32</td>
<td>0.09 0.09 0.08</td>
<td>0.15 0.04</td>
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<td>0.82</td>
<td>0.08 0.09 0.07</td>
<td>0.16 0.17</td>
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<td>0.20 0.43</td>
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<td>0.52</td>
<td>0.09 - -</td>
<td>- 0.09</td>
<td>0.07</td>
</tr>
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<td>0.03 0.00 0.00</td>
<td>0.03 0.08</td>
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<td>0.33 0.43</td>
<td></td>
</tr>
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<td>0.73 0.03 -</td>
<td>- 0.99</td>
<td>0.12</td>
</tr>
<tr>
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<td>4.92</td>
<td>0.20 0.00 0.65</td>
<td>1.18 1.05</td>
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*Mix designations M113: c:s ratio=1:1; w/c ratio=0.3.

**TABLE 9.3 Initial Surface Absorptions from Mortar Specimens**
<table>
<thead>
<tr>
<th>Mix*</th>
<th>Cube Nos.</th>
<th>Beams</th>
</tr>
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<tbody>
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<td>1</td>
<td>2</td>
</tr>
<tr>
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<td>20.3</td>
<td>6.5</td>
</tr>
<tr>
<td>M114</td>
<td>6.2</td>
<td>0.5</td>
</tr>
<tr>
<td>M115</td>
<td>1.2</td>
<td>6.9</td>
</tr>
<tr>
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<td>4.2</td>
</tr>
<tr>
<td>M123</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M124</td>
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<td>13.2</td>
</tr>
<tr>
<td>M125</td>
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<td>11.5</td>
<td>2.5</td>
</tr>
<tr>
<td>M133</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M134</td>
<td>0.0</td>
<td>-</td>
</tr>
<tr>
<td>M135</td>
<td>2.2</td>
<td>1.2</td>
</tr>
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<td>M136</td>
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<td>6.0</td>
</tr>
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</tr>
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<td>3.0</td>
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<td>M146</td>
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*Mix designation M113: c:s ratio=1:1; w/c ratio=0.3

**TABLE 9.4 Variations in the Initial Surface Absorptions from Mortar Specimens**
### TABLE 9.5 Figg Air Times from C2 Concrete Beams

<table>
<thead>
<tr>
<th>Mix</th>
<th>Max</th>
<th>Min</th>
<th>Avge</th>
<th>%var.</th>
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</thead>
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<td>14.6</td>
<td>1.2</td>
<td>12.5</td>
<td>+298/-90</td>
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<td>21.7</td>
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<td>18.9</td>
<td>+15/-17</td>
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<tr>
<td>C206</td>
<td>15.4</td>
<td>11.2</td>
<td>12.9</td>
<td>+19/-13</td>
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<tr>
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<td>4.4</td>
<td>6.5</td>
<td>+42/-32</td>
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<td>+12/-8</td>
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*%mix designation C203: Mix C2; w/c ratio=0.3.*

† percentage variation about the mean

### TABLE 9.6 Figg Air Times from C2 Concrete Cubes

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<th>Mix</th>
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<th>Cured for 7 days</th>
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<td>2.4</td>
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*%mix designation C203: Mix C2; w/c ratio=0.3.*

† percentage variation about the mean

172
<table>
<thead>
<tr>
<th>Mix*</th>
<th>Cubes</th>
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<th>Beams</th>
<th></th>
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<tbody>
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<td></td>
<td>Max</td>
<td>Min</td>
<td>Avge</td>
<td>Max</td>
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<td>300.0</td>
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<td>9.5</td>
<td>7.6</td>
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*Mix designation M113: c:s ratio=1:1; w/c ratio=0.3

**TABLE 9.7 Figg Air Times from Mortar Specimens**
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<th>Mix*</th>
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Mix designation M113: c:s ratio=1:1; w/c ratio=0.3.

TABLE 9.8 Figg Water Times from Mortar Specimens
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*Mix designation M113: c:s ratio=1:1; w/c ratio=0.3
†=Specimens cracked in test area, so values not used to calculate averages

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*Mix designation M113: c:s ratio=1:1; w/c ratio=0.3
†specimens cracked in test area, so times not used for calculating averages

**TABLE 9.10 Egg Test Times from Mortar Cubes**
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<td>M148</td>
<td>74</td>
<td>10.</td>
<td>2.0</td>
<td>8.0</td>
<td>9.0</td>
<td>9.0</td>
<td>V</td>
<td>9.0</td>
<td>9.0</td>
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<tr>
<td>M149</td>
<td>71</td>
<td>3.0</td>
<td>1.0</td>
<td>6.0</td>
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<td>7.0</td>
<td>V</td>
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<td>M155</td>
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<td>1.0</td>
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<td>4.0</td>
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<td>M156</td>
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<td>M158</td>
<td>67</td>
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<td>2.0</td>
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N.B. ages in weeks; V=variations, where 4=4 day cured specimens, L=specimens stored in the laboratory and 2=second specimen in same conditions first. e=position of Egg test.

*Mix designations: C103=concrete C1, w/c ratio=0.3; C203=concrete C2, w/c ratio=0.3; M113=mortar c:s ratio=1:1, w/c ratio=0.3; G3=cement paste with w/c ratio=0.3

**TABLE 9.11 Carbonation Test Results at One Year from Concrete and Mortar Specimens**
FIGURE 9.1 Relationship Between Initial Surface Absorption and Time for Laboratory Column
FIGURE 9.2 Relationship Between Initial Surface Absorption and W/C Ratio for CI Concrete Mixes in and Air Dried Condition

FIGURE 9.3 Relationship Between Initial Surface Absorption and W/C Ratio for CI Mixes after Oven Drying
FIGURE 9.4 Relationship Between Initial Surface Sorption and W/C Ratio for Concrete Beams

Key
* = average result from C1 beams
o = average result from C2 beams

FIGURE 9.5 Relationship Between Initial Surface Absorption and W/C Ratio for Cubes from C2 Mixes

Key
+ = Cured for seven days
o = Cured for four days
FIGURE 9.6 Relationship Between Initial Surface Absorption and W/C Ratio for Mortar Beams

FIGURE 9.7 Relationship Between Initial Surface Absorption and W/C Ratio for Mortar Cubes
FIGURE 9.8 Relationship Between Initial Surface Absorption and Percentage Sand for Mortar Mixes
FIGURE 9.9 Relationship Between Initial Surface Absorption and Saturation for Cl Concrete Mixes
FIGURE 9.10 Relationships Between Initial Surface Absorption and W/C Ratio for Tests on C1 Beams in an Oven Dry Condition
FIGURE 9.11 Relationship Between Initial Surface Absorption and Saturation for Mortar Mixes M13/4/5/6
FIGURE 9.12 Relationships Between Initial Surface Absorption and W/C Ratio for Tests on Mortar Beams in an Oven Dry Condition
FIGURE 9.13 Relationship Between Initial Surface Absorption and Saturation for Mixes with a W/C Ratio of 0.6

N.B. Values in excess of 1.0 ml/m²/s omitted for clarity.
FIGURE 9.14 Relationship Between Figg Air Time and W/C Ratio for C1 Concrete Mixes in an Air Dried Condition
FIGURE 9.15 Relationship Between Figg Air Time and W/C Ratio for Cl Mixes after Oven Drying
FIGURE 9.16a Relationship Between Figg Air Time and W/C Ratio for Concrete Beams

FIGURE 9.16b Relationship Between Figg Air Time and W/C Ratio for Cubes from C2 Mixes
FIGURE 9.17 Relationship Between Figg Air Time and W/C Ratio for Mortar Beams

FIGURE 9.18 Relationship Between Figg Air Time and W/C Ratio for Mortar Cubes
N.B. Numbers refer to w/c ratio, ratios are cement:sand ratios

FIGURE 9.19 Relationship Between Figg Air Time and Percentage Sand for Mortar Mixes
FIGURE 9.20 Relationship Between Figg Air Time and W/C Ratio for Tests on C1 Mixes in Oven Dry Condition

Key
+=averages from initial tests
=averages from retests
*=averages from C2 mixes
FIGURE 9.21 Relationship Between Figg Air Time and W/C Ratio for Mortar Beams in Oven Dry Condition
FIGURE 9.22 Relationship Between Figg Air Time and Saturation for Mortar Mix M113

FIGURE 9.23 Relationships Between Figg Air Time and Saturation for Concrete Mix C105
FIGURE 9.24 Relationship Between Figg Air Time and Saturation for Mortar Mix M114

FIGURE 9.25 Relationship Between Figg Air Time and Saturation for Concrete Mix C106
FIGURE 9.26 Relationship Between Figg Air Time and Saturation for Concrete Mix C107

FIGURE 9.27 Relationship Between Figg Air Time and Saturation for Mortar Mixes M157/8/9
FIGURE 9.28 Relationship Between Figg Air Time and Saturation for Mortar Mixes M133/4/5/6

FIGURE 9.29 Relationship Between Figg Air Time and Saturation for Mortar Mixes M146/7/8/9
FIGURE 9.30 Relationship Between Figg Air Time and Saturation for Mixes with a W/C Ratio of 0.6

N.B. Values in excess of 300s omitted for clarity.
FIGURE 9.31 Relationship Between Figg Water Time and W/C Ratio for CI Concrete Mixes in an Air Dried Condition

Key
b = results from beams
c = results from cubes
N.B. results in excess of 6000s omitted for clarity
FIGURE 9.32 Relationship Between Figg Water Time and W/C Ratio for Concrete Beams

FIGURE 9.33 Relationship Between Figg Water Time and W/C Ratio for Cubes from C2 Mixes
FIGURE 9.34 Relationship Between Figg Water Time and W/C Ratio for Mortar Beams

FIGURE 9.35 Relationship Between Figg Water Time and W/C Ratio for Mortar Cubes

N.B. values in excess of 2700s omitted for clarity
FIGURE 9.36 Relationship Between Figg Water Time and W/C Ratio for C1 Concrete Mixes in an Oven Dry Condition
FIGURE 9.37 Relationship Between Figg Water Time and W/C Ratio for Mortar Mixes in an Oven Dry Condition
FIGURE 9.38 Relationship Between Figg Water Time and Saturation for Concrete Mixes
FIGURE 9.39 Relationship Between Figg Water Time and Saturation for Mortar Mixes M113/4/5/6

FIGURE 9.40 Relationship Between Figg Water Time and Saturation for Concrete Mix C103
FIGURE 9.41 Relationship Between Figg Water Time and Saturation for Concrete Mix C106

FIGURE 9.42 Relationship Between Figg Water Time and Saturation for Concrete Mix C107
FIGURE 9.43 Relationship Between Figg Water Time and Saturation for Mixes with a W/C Ratio of 0.6

N.B. Times in excess of 600s omitted for clarity
FIGURE 9.44 Relationship Between Egg Test Time and W/C Ratio for C1 Concrete Mixes in an Air Dried Condition
FIGURE 9.45 Relationship Between Egg Test Time and W/C Ratio for Concrete Beams

FIGURE 9.46 Relationship Between Egg Test Time and W/C Ratio for Cubes from C2 Mixes
FIGURE 9.47 Relationship Between Egg Test Time and W/C Ratio for Mortar Beams

FIGURE 9.48 Relationship Between Egg Test Time and W/C Ratio for Mortar Cubes
FIGURE 9.49 Relationship Between Egg Test Time and Percentage Sand for Mortar Mixes
FIGURE 9.50 Relationship Between Egg Test Time and W/C Ratio for C1 Concrete Mixes Tested in Oven Dry Condition
FIGURE 9.51 Relationship Between Egg Test Time and W/C Ratio for Mortar Mixes Tested in an Oven Dry Condition
FIGURE 9.52 Relationship Between Egg Test Time and Saturation for Cl Concrete Mixes
FIGURE 9.53 Relationship Between Egg Test Time and Saturation for Concrete Mix C105

FIGURE 9.54 Relationship Between Egg Test Time and Saturation for Concrete Mix C107
FIGURE 9.55 Relationship Between Egg Test Time and Saturation for Mortar Mixes M123/4/5/6

N.B. Test times in excess of 500s omitted for clarity.
FIGURE 9.56 Relationship Between Egg Test Time and Saturation for Mortar Mixes M113/4/5/6

FIGURE 9.57 Relationship Between Egg Test Time and Saturation for Mortar Mixes M157/8/9

N.B. Values in excess of 300s omitted for clarity
FIGURE 9.58 Relationship Between Egg Test Time and Saturation for Mortar Mixes M146/7/8/9

FIGURE 9.59 Relationship Between Egg Test Time and Saturation for Mortar Mixes M134/5/6/7/8
FIGURE 9.60 Relationship Between Egg Test Time and Saturation for Mortar Mix M113

FIGURE 9.61 Relationship Between Egg Test Time and Saturation for Mixes with a W/C ratio of 0.6
FIGURE 9.62 Relationship Between Moisture Monitor Reading and Saturation for All Specimens
FIGURE 9.63 Relationship Between Moisture Monitor Reading and Saturation for Selected Specimens
FIGURE 9.64 Relationship Between Moisture Monitor Reading and Saturation for Concrete Mix C103
FIGURE 9.65 Relationship Between Moisture Monitor Reading and Saturation for Mortar Mix M159

Key
+=average result
Letters represent positions on beam
FIGURE 9.66 Relationship Between Carbonation Depth and W/C Ratio for Concrete Cubes

FIGURE 9.67 Relationship Between Carbonation Depth and W/C Ratio for Mortar Cubes Stored Outside
FIGURE 9.68 Relationship Between Carbonation Depth and W/C Ratio for Mortar Mixes M113/4/5/6 and M156/7/8/9/0/1
PLATE 9.1 Initial Surface Absorption Test in use at Budbrook Water Tower

PLATE 9.2 The Egg Test in use at Budbrook Water Tower
10 COMPARISON OF TESTS

10.1 Laboratory Tests

10.1.1 Permeability Tests

Direct comparison between the air and water permeability test results from the concrete mixes is difficult, because the nature of the two tests is different and only a small number of results are available. The relationship between vapour permeability and w/c ratio for the concrete (Figure 8.13) shows $K_d$ increasing as the w/c ratio increases, a relationship that is not shown by the air and water tests. As a result there is no obvious relationship between the vapour permeabilities and the results from the other permeability tests on the concrete mixes. This may be a result of the test method used, or a property of the material.

All of the laboratory permeability tests on mortar specimens show increasing permeability with increasing w/c ratio. The water and air tests show very similar trends even though the test methods are different (Figs 8.4 & 8.8). Variations from the general trends are quite noticeable when they occur for example with a c:s ratio of 1:4 and a w/c ratio of 0.9. It has been shown\(^{(344)}\) that results of air permeability tests are moisture dependent, so this makes comparison with other work difficult, because results quoted by other researchers do not necessarily relate to similar moisture conditions. The large variation in vapour permeability results also made comparison between these and the other tests difficult.

10.1.2 Strength and Permeability

A number of authors have tried to find relationships between concrete strengths and permeability. Lawrence\(^{(308)}\) has quoted results which suggest that a linear relationship exists between concrete strength and the logarithm of oxygen permeability, but the spread of results leads to doubts about the validity of this conclusion. Recent work by Bamforth\(^{(236)}\) suggests that a linear relationship exists between the logarithm of the water
permeability coefficient and compressive strength. Although few results are available, the relationship proposed is linear, the only significant variations being in results from air entrained concretes.

Comparing the logarithm of water permeability with strength for the C2 concrete mixes does not show any simple relationship (Figure 10.1). However results from the mortar mixes do show a linear relationship (Figure 10.2). Comparison of these results with those proposed by Bamforth is not easy because Bamforths' results are recorded over a maximum period of 24 hours. Taking values of LnK at 24 hours for the mortar mixes and comparing these with Bamforths' results gives a much better comparison (Figure 10.3). The water permeabilities of the mortar mixes are lower than those reported by Bamforth. This is thought to be a result of differences in the test methods and the method of calculating K. Bamforth calculated K from the outflow, or from the water penetration after 24 hours. Theories has been proposed by Valenta\textsuperscript{(15)}, and Vuorinen\textsuperscript{(249)}, to relate penetration to permeability, and this method is also proposed by some standard test methods for measuring permeability\textsuperscript{(349-50)}.

Although a linear relationship is seen with these results, it is difficult to see how these relationships can be utilised, as it is clear from the results from the C2 concrete mixes that strength and permeability cannot always be so readily compared. Unfortunately, there is insufficient data available to try and define these relationships more precisely. Another problem which is clear from these results is that the time after the start of the test at which the values of K are calculated creates differences in results, even if the gradients of the lines are similar.

A similar comparison can be made with the air permeability results. As mentioned already, work by Lawrence\textsuperscript{(308)} suggests that linear behaviour exists between the logarithm of oxygen permeability and strength, but the results were obtained from specimens which were not dry and so are not conclusive. Plotting values of strength against the logarithm of air permeability gives the relationship shown in Figure 10.4. It can be seen that the relationship is not linear, but slightly curved. This actually corresponds quite well with the
results quoted by Lawrence, although the mortar permeabilities are higher, possibly because the results are from oven dried specimens, whereas the concrete tested by Lawrence was only air dried.

No simple relationship can be seen between the vapour permeabilities and strengths of the mortar mixes (Figure 10.5), although the results from the concrete mixes lie close to a straight line (Figure 10.6).

It is considered that if linear or non-linear relationships exist for certain concrete or mortar mixes, these relationships will be purely academic. Experience has shown that strength does not equate to durability, and the variety of results obtained suggests that any relationships between strength and permeability are the result of factors which are common to the particular mixes in question, and not a general trend for all mixes.

10.2 In-Situ Test Methods

Comparison of the in-situ test methods may be of great value for evaluating site concrete, as results from one test may not be sufficient to accurately assess the concrete. Comparison is made between results from the permeability tests with similar test positions (surface or sub-surface), and similar test fluids (water or air). In addition, comparison is made between different test positions and specimens from each mix where this is appropriate, to ascertain whether differences in the results are the result of test or specimen variations. The results from the tests on air dry concrete specimens have not been included in these comparisons. A comparison of the relative merits and features of the different tests employed is given in Table 10.1.

10.2.1 Figg Tests

Because the Figg tests are both carried out in the same test position, comparison of results from individual mixes and individual test pieces is relatively straightforward, although no relationship is believed to exist between the two tests(272).
10.2.1.1 Tests on Concrete

Although there are similarities in the relationships between Figg air time and w/c ratio and Figg water time and w/c ratio (Figs 9.16a & 9.32), it is not easy to compare individual sets of results. Plotting Figg air time against Figg water time for the C1 mixes gives a pattern of results which spreads out as the test times increase (Figure 10.7). The results from the high w/c ratio mixes are fairly close together, but those from mixes C104 and C105 are widely spaced, although this may be a result of these test times being much larger. A different pattern of results can be seen with the C2 mixes, with the majority of results close to either one of the two axes (Figure 10.8). It is unfortunate that the results from the two concrete mixes differ so much, as this prevents any overall comparison between the two mixes. Only at w/c ratios of 0.7 and 0.8 are the results of similar magnitude.

Considering results from specific test holes in the C1 mixes, there are some notable variations in results from particular holes. For example in mix C106, high results occur from both tests at different saturations, as well as in an oven dry condition (Figures 9.25 & 9.41). The variation in both cases was from the same test hole, so it must be attributed to a variation in the test holes. No similarities between test holes were seen with the results from the C2 mixes.

10.2.1.1 Tests on Mortar

As with the concrete the relationship between the Figg air and water times for the mortar mixes spread outwards as the air and water test times increase, with results from the higher w/c ratios bunched together (Figure 10.9). This relationship may be useful for applying limits to these tests. If both tests exhibit low test times from one mix (for example air times <50s and water times <250s), then this indicates a test mix with a w/c ratio above 0.5, or 0.6, which as discussed earlier (Section 9.2.2) may be a useful indicator of potential durability.

Considering the results from specific test holes on the beams although variations are shown in some results from one test, they are not always shown by the other test, for example
mix M114 (Figures 10.10a & b). In such cases it is assumed that the variation in results is related to the tests rather than the material.

Considering results from the different cubes (Tables 9.7 & 9.8), some cubes have consistent variations in results for both tests, for example results from mixes M123 and M124 both have high maximum and minimum values. However some cubes have very inconsistent results for example the cubes from mixes M133 and M145 have zero Figg air times (because the test pressure could not be achieved), but the maximum water times are nearly 200s, which in other mixes, such as mix M146, correspond to a Figg air time of about 40s.

Because of these inconsistencies, it is difficult to compare the different times from these tests and Figg(272) has stated that there is unlikely to be any relationship between these two tests, because the mechanisms involved in each are different. The lack of any relationship between these two tests may be advantageous, because it means that they can be used to give two different parameters for assessing the permeability of in-situ concrete, using the same test position. In addition, it may be possible to define more accurate limits for categories of concrete based on two parameters rather than one.

10.2.2 Surface Tests

The ISAT and the Egg test both measure surface properties, although the test fluids and direction of flow are different in each case. As each individual test position is different, comparison can only be made between the average results from each specimen.

10.2.2.1 Tests on Concrete

Plotting the test results from the ISAT and the Egg test for the oven dry concrete produces no obvious relationship (Figure 10.11). The expected trend of results is increasing Egg test times with decreasing ISA, as both would indicate improvements in the surface permeability properties. However since this is not the case, it must be assumed that as with the Figg tests, these two tests measure different parameters.
10.2.2.2 Tests on Mortar

Plotting test results from the mortar specimens (Figure 10.12) does show a relationship which is broadly that which is expected, with decreasing ISA as Egg time increases. From this curve, it may be possible to propose limits for different categories of concrete, for example: a 'Good' mix could have an ISA less than 0.3ml/m^2/s, and an Egg test time greater than 100s; 'Average' could have an ISA less than 0.6ml/m^2/s, and an Egg test time greater than 25s; and, 'Poor' could have an ISA greater than 0.6ml/m^2/s, and an Egg test time less than 25s.

No obvious relationships can be seen between results from the two tests at different saturations. Considering the mixes with a c:s ratio of 1:4, there is a noticeable difference between the Egg test times for mixes M146 and M147, and mixes M148 and M149 above a saturation of 50% (Figure 10.13a), but there is no obvious difference with the ISA from these mixes (Figure 10.13b).

10.2.3 Water Tests

The ISAT and Figg water test both use water as the permeating fluid. However, as well as testing different parts of the specimens, different methods of recording results are used. This means that it is unlikely that any simple relationship exists between the results from these tests.

10.2.3.1 Tests on Concrete

Considering the relationships between the two tests and w/c ratio for the C1 concrete mixes (Figures 9.10 & 9.36), it can be seen that for the initial tests the relative positions of mixes C103, C106, C107, and C108 are the same for both tests (remembering that the low ISA is better than high ISA, but the reverse is true for the Figg water test). However, the maximum values are from different mixes, with the maximum Figg water time from mix C104, and the lowest ISA from mix C106. The results from the retests, are harder to compare because the spread of Figg water times is very low although the highest average time occurs with mix C105.

Results from the tests on the C2 mixes do not show any
obvious relationships, mixes C205-8 have low Figg water times and mixes C203 and C204 have high test times (Figure 9.32). The ISAT gives more gradual increases in values from w/c ratio of 0.4 upwards (Figure 9.4). The only point of comparison is the best value which is from mix C204 for both tests.

10.2.3.2 Tests on Mortar

As with the concrete specimens, no obvious comparison can be made between the results of the two tests. Plotting ISA against Figg water time shows that low ISA does not always correspond to high Figg water time (Figure 10.14). From the retest results even though the ISA changes very little, the Figg water times change dramatically in some cases (Figure 10.15), so again high ISA occurs with high Figg water times.

At different saturations, more differences occur. For the mixes with a w/c ratio of 0.6, the hierarchy and spread of results is different (Figs 9.13 & 9.43). Although the concrete has the best results from both tests, the ISA improves with the c:s ratio in the order 1:1, 1:2, 1:3, 1:4, whilst the Figg water test improves with c:s ratio in the order 1:1, 1:4, 1:3, 1:2. The only obvious similarity is that both tests show an improvement in results at low saturations. This suggests that this behaviour is the result of using water instead of air as the test fluid, as this behaviour is not seen in the results from the air tests.

10.2.4 Air Tests

As with the water tests, the test positions used for the Figg air test and the Egg Test were different. However, the method of measurement is the same in both cases, so any relationship will be independent of this. Although the volumes of the two test apparatus' are the same, the contact areas differ. The Figg test hole has an interior surface area of approximately 700mm², whereas the area of the Egg is only 350mm², so at least a two fold difference can be expected from similar materials.

10.2.4.1 Tests on Concrete

Results from the tests on the C1 mixes are not conclusive, although the differences between the mixes is more marked with
the Figg air test. Plotting test times from both tests against w/c ratio shows the Egg test times to be approximately double the Figg air times for mixes C107 and C108 (Figure 10.16). It is important to note that if the concrete is being dried in an oven and is not fully dry when it is tested, then the interior will be wetter than the surface, so the Figg air times will be affected more than the Egg test times, making them higher. Plotting the results from the retests against w/c ratio, a closer relationship can be seen between the two tests (Figure 10.17), in this case the Egg test times are between 2 and 4.3 times higher than the equivalent Figg air time.

Plotting the results from the C2 beams against w/c ratio shows that at the higher w/c ratios the Egg test times do not decrease as much as the Figg air times (Figure 10.18). Egg test times for mixes C203 and C204 are approximately double the Figg air times, but that from mix C205 is only 1.4 times higher and from mixes C206-8 are 2.3, 3.4, and 5.0 times greater respectively. However, the results from the cubes do show a number of similarities (Figure 10.19), notably at the higher w/c ratios.

10.2.4.2 Tests on Mortar

The relationships between Figg air times and Egg test times and w/c ratio (Figs 9.17 & 9.47) both show high test times at low w/c ratios, decreasing as the w/c ratio increases. However, the Figg air times show a peaks from the mixes with c:s ratios of 1:1, 1:2, and 1:3, which are not shown by the Egg test, so there is a clear difference in the results. In addition, the Egg test times are generally lower that the Figg air times, unlike the results from the concrete mixes. This may be a result of the surface being drier than the core of the specimens, so giving lower test times, especially in the earlier tests (mixes with c:s ratios of 1:2 and 1:3). In addition, the surfaces of the specimens from the mixes with a c:s ratio of 1:1 all had cracks which may account for the low Egg test times from these specimens.

The retests times for the Figg air tests were much lower than the initial test times, the Egg test times are virtually unchanged, as a result, the Egg test times are noticeably
greater in some cases. For example the Egg test time for mix M123 is 12.3 times greater than the Figg air time (Figure 10.20).

10.2.4.3 Effects of Different Moisture Contents

The patterns of results shown by the two tests when plotted against percentage saturations for the concrete mixes show similar trends, although the order of results is different (Figure 10.21). Clearly, it is not easy to separate the two sets of results, although it can be seen that at high saturations the Figg air times are higher than the Egg test times at a given saturation, whilst at lower saturations the Egg test times are higher than the Figg air times (The actual crossover points vary from about 60% saturation for results from mix C108 to about 20% saturation for mix C104).

The results from the mortar mixes with a w/c ratio of 0.6 (Figure 10.22) show that the Figg air test results lie between the sets of results from the Egg tests on mixes with c:s ratios of 1:1 and 1:2, and 1:3 and 1:4.

10.2.5 Carbonation Tests

Comparison of the carbonation depth from the concrete mixes with the results of the in-situ permeability tests does not reveal any obvious relationships. The general trend of results is increasing carbonation depth with increasing w/c ratio (Figure 9.66). This pattern of results is not shown by the permeability tests. In these mixes, the amount of cement paste may influence the rate of carbonation as well as the permeability properties. This demonstrates the weakness of trying to use one test method to assess durability, and also the difficulties of relating 'permeability' to durability.

The trends of results from the tests on the mortar mixes do show similarities in that there are improvements as the w/c ratio gets lower, within the c:s ratios. Unfortunately, the carbonation depths fluctuate quite a lot between different w/c ratios, so it is difficult to draw any specific conclusions about these tests from these results.
10.3 Comparison of Laboratory and In-Situ Test Methods

It was expected that some comparison could be made between the different permeability tests, but as stated earlier, it was assumed that any other relationships would be merely the effects of common factors in particular mixes.

10.3.1 Comparison of Permeability Tests

For the C2 concrete mixes, some comparison can be made between the results of the water permeability tests and the in-situ permeability tests. Figure 8.5 shows reducing water permeability as the w/c ratio increases to 0.5, followed by increasing permeability as it increases further. A similar trend can be seen with the Figg air test results (Figure 9.16a), but not with the ISAT (Figure 9.4), the Figg water test (Figure 9.32) or the Egg test (Figure 9.45), all of which 'peak' at different w/c ratios.

Comparing the results from the mortar mixes, the most obvious similarities are between the relationship between ISA and w/c ratio, and water and air permeability and w/c ratio (Figs. 9.6, 8.4 & 8.8). Any similarity with the other tests is less obvious, because improvements in results are represented by increases in test times. For the two Figg tests it was noted that a w/c ratio of 0.6 is the upper limit for good results. Considering a w/c ratio of 0.6 for the laboratory test results, it can be seen that the permeabilities increase quite rapidly above this value, although this may be a function of the vertical scale used on these graphs. Considering the mixes with a c:s ratio of 1:1 both water and air permeability tests show increased permeabilities at w/c ratios of 0.5 and 0.6. Poor results can also be seen in all of the in-situ permeability tests, although it is more significant in the air permeability results, and can be seen clearly in the Figg air test and Egg Test (Figs 9.17 & 9.47).

It is clear from these results that some similarities do exist between the in-situ and laboratory water and air permeability tests. Vapour permeability test results are rather scattered so it is difficult to compare these test with results from the in-situ tests.
10.3.2 Comparison of Non-Permeability Tests

Initially no similarities were expected between the non-permeability tests and the permeability tests. However, examination of the results shows that a very good relationship exists between the ISA and strength of the mortar mixes (Figure 10.23). This relationship may be useful for specification compliances, if the cube strength of a particular mix is known, and the c:s ratio is known, then it may be possible to predict the ISA, and then test it in situ to confirm the result.

Philajavaara and Paroll\(^{(282)}\) have tried to find a relationship between both Figg tests and strength, but without success. No general relationships can be seen with these test results, although there is some similarity in some individual results, for example mixes M116 and M156, and mixes M113 and M123 (Figures 8.14 & 9.17).

The most striking similarity which can be seen, is between the vapour permeability and carbonation depths of the mortar specimens. The results from these two tests plotted against w/c ratio show a number of similarities (Figs 10.24 & 25). This relationship suggests that the rate of carbonation may be influenced by the diffusion properties of the material rather than the permeability. A similar relationship was not seen with the concrete.

As discussed earlier (Section 5.1.3), Schwiete et al\(^{(243)}\) have proposed relationships for carbonation and specific permeability, which is a function of the diffusion coefficients of the concrete. No comparison can be drawn between these results and those obtained in this work because the type of permeability was different in each case. However, the general trend of results is the same, with increasing depth of carbonation as permeability increases (Figure 10.26).
<table>
<thead>
<tr>
<th></th>
<th>Initial Surface Absorption test</th>
<th>Figg Air Test</th>
<th>Figg Water Test</th>
<th>Egg Test</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of test</strong></td>
<td>Surface water</td>
<td>Sub-surface air</td>
<td>Sub-surface water</td>
<td>Surface air</td>
</tr>
<tr>
<td><strong>Test time</strong></td>
<td>10 minutes</td>
<td>5 minutes*</td>
<td>10 minutes*</td>
<td>5 minutes</td>
</tr>
<tr>
<td></td>
<td><em>(test times used in these experiments)</em></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Preparation time</strong></td>
<td>5 minutes</td>
<td>Hole preparation time~30 minutes</td>
<td>+1 minute for test</td>
<td>+2 minutes for test</td>
</tr>
<tr>
<td><strong>Cost of apparatus†</strong></td>
<td>£400</td>
<td>£500</td>
<td>£450</td>
<td>£500*</td>
</tr>
<tr>
<td></td>
<td><em>(Figg air test apparatus)</em></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Relevant codes</strong></td>
<td>BS1881 pt5 1970</td>
<td>BS1881 pt 201 1985</td>
<td>No relevant code</td>
<td>No relevant code</td>
</tr>
</tbody>
</table>

**All tests affected by moisture content of the test concrete**

| Comments |
|-----------------|-------------------------------------------------|-----------------|-----------------|-----------------|
| Initial Surface Absorption test | It marks the concrete. The sealing method is not always reliable and needs practice to make it work. It is necessary to support the apparatus at a set level above the test area, and it is also necessary to hold the test cap in position by either propping it or fixing it to the concrete. |
| Figg Air Test | Hole preparation in situ could be a problem and may lead to microcracking which could affect the results. Problems may arise from leaks in the connecting tubes. |
| Figg Water Test | The apparatus has to be supported at a given height above the test hole. Filling the test hole is difficult even under laboratory conditions. Problems may arise from leaks in the connecting tubes. Hard to carry out on rough surfaces. |
| Egg Test | It marks the concrete in the test area. The test area is only small so can be affected by local variations such as cracks. |

† Costs of apparatus if bought commercially. Similar equipment can be manufactured 'in house' at much lower costs.

**TABLE 10.1 Comparison of In-situ Permeability Test Methods Employed in this Research**
FIGURE 10.1 Relationship Between Ln Water Permeability and Strength for C2 Concrete Mixes

FIGURE 10.2 Relationship Between Ln Water Permeability and Strength for Mortar Mixes
FIGURE 10.3 Relationship Between Ln Water Permeability and Strength for Mortar Mixes with Relationship Proposed by Bamforth
FIGURE 10.4 Relationship Between $\log_{10}$ of Intrinsic Permeability and Strength for Mortar
FIGURE 10.5 Relationship Between Vapour Permeability and Strength for Mortar Mixes

N.B. Numbers refer to w/c ratios e.g. 3 is a w/c ratio of 0.3

FIGURE 10.6 Relationship Between Vapour Permeability and Strength for C2 Concrete Mixes
FIGURE 10.7 Relationship Between Figg Air Time and Figg Water Time for C1 Concrete Mixes

FIGURE 10.8 Relationship Between Figg Air Time and Figg Water Time for C2 Concrete Mixes
N.B. Figg water times in excess of 2500s and Figg air times in excess of 300s omitted for clarity. Numbers represent w/c ratios e.g. 3 is a w/c ratio of 0.3

FIGURE 10.9 Relationship Between Figg Air Time and Figg Water Time for Mortar Mixes
N.B. times in excess of 500s omitted for clarity

FIGURE 10.10a Relationship Between Figg Air Time and Saturation for Mortar Mix M114

N.B. Times in excess of 5000s omitted for clarity

FIGURE 10.10b Relationship Between Figg Water Time and Saturation for Mortar Mix M114
Numbers indicate w/c ratios. Suffixes are C1 Beams (1), C2 Beams (2) and C2 cubes (c). e.g. 61 is the C1 beam with w/c ratio = 0.6.

FIGURE 10.11 Relationship Between Initial Surface Absorption and Egg Test Time for C1 and C2 Concrete Mixes
Numbers identify c:s and w/c ratios of mixes e.g. 34 indicates a c:s ratio of 1:3 and a w/c ratio of 0.4.

FIGURE 10.12 Relationship Between Initial Surface Absorption and Egg Test Time for Mortar Mixes
FIGURE 10.13a Relationship Between Egg Test Time and Saturation for Mortar Mixes M146/7/8/9

FIGURE 10.13b Relationship Between Initial Surface Absorption and Saturation for Mortar Mixes M146/7/8/9
N.B. ratios are c:s ratios, numbers are w/c ratios i.e. 3 is w/c ratio of 0.3

FIGURE 10.14 Relationship Between Initial Surface Absorption and Figg Water Time for Mortar Mixes
N.B. Figures indicate w/c ratios e.g. 3 is a w/c ratio of 0.3. Ratios are c:s ratios

FIGURE 10.15 Relationship Between Initial Surface Absorption (ml/m²/s) and Figg Water Time for Retests on Mortar Mixes
FIGURE 10.16 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Tests on C1 Concrete Mixes

FIGURE 10.17 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Retests on C1 Concrete Mixes
FIGURE 10.18 Relationship Between Figg Air and Egg Test Times and W/C Ratio for C2 Concrete Beams

FIGURE 10.19 Relationship Between Figg Air and Egg Test Times and W/C Ratio for C2 Concrete Cubes
FIGURE 10.20 Relationship Between Figg Air and Egg Test Times and W/C Ratio for Retests on Mortar Mixes

c=Egg test results
+=Figg air test results
Ratios are c:s ratios
FIGURE 10.21 Relationship Between Figg Air and Egg Test Times and Saturation for C1 Concrete Mixes
FIGURE 10.22 Relationship Between Figg Air and Egg Test Times and Saturation for Mixes with W/C ratio of 0.6
FIGURE 10.23 Relationship Between Initial Surface Absorption and Strength for Mortar Mixes
FIGURE 10.24 Relationship Between Vapour Permeability and W/C Ratio for Mortar Mixes with C:S Ratios of 1:3, 1:4, and 1:5

FIGURE 10.25 Relationship Between Carbonation Depth and W/C Ratio for Mortar Mixes with C:S Ratios of 1:3, 1:4 and 1:5
Key

Numbers indicate c:s and w/c ratios
e.g. 35 has a c:s ratio of 1:3 and a
w/c ratio of 0.5.
Bold numbers are results from
specimens stored inside

FIGURE 10.26 Relationship Between Carbonation Depth and Vapour Permeability for Mortar Mixes with C:S Ratios of 1:3, 1:4, and 1:5
11 CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

11.1 Conclusions

i) Air, water and water vapour permeability tests can be carried out in a laboratory using apparatus which are easy to manufacture and use.

ii) The results of the laboratory air and water permeability tests compared well with those found in literature with permeability increasing as w/c ratio increased, and permeability decreasing as the aggregate:cement ratio increased. Results from the water vapour permeability tests on concrete showed increasing permeability with increasing w/c ratio and results from tests on mortar specimens compared well with results of carbonation tests.

iii) Four in-situ permeability tests were examined in detail; the initial surface absorption test; the Figg air test; the Figg water test; and, a new test called the Egg test. All of these tests measure properties that relate to permeability, but none measure true permeability and none of the tests gives results that can be easily related to true permeability.

iv) All of the tests are affected by the moisture content of the concrete, with the measured permeability properties decreasing as the moisture content increases.

v) The initial surface absorption test is relatively easy to perform and gave acceptable results for the concrete and mortar mixes that were tested. There was a good relationship between initial surface absorption and strength for mortar.

vi) The Figg tests were carried out in holes drilled into the concrete and then plugged with a catalysed silicone rubber seal. This was seen as a major problem with these two tests for in-situ use.

vii) The Figg air test is easy to use but for practical reasons a modified method for recording results is necessary in normal usage. The results showed large variations, particularly with less permeable mixes. It is concluded that this is a result of microcracking affecting the permeability.

viii) The Figg water test is difficult to carry out and can give variable results which are believed to result in part from
these difficulties. As with the Figg air test, it is thought that microcracking also affects the results.

ix) A new test, the Egg test, has been developed. This uses the same principles as the Figg air test, but measures the permeability properties of the surface of the concrete instead of those in a hole drilled into the surface. It is non-destructive, easy to use and quicker than any of the other tests employed. Results were less variable than the Figg air test possibly because there was no drilling in the concrete which might cause microcracking.

x) Comparison of the test results from oven dried specimens with the mix proportions shows that as a general rule permeability increases as w/c ratio increases. Comparing different aggregate:cement ratios showed some trends towards optimum values for specific w/c ratios.

xi) Comparison of the different tests is not easy, because the tests relate to different test areas or test fluids (air or water). Some tests compare well on some test pieces, but not on others and in some cases similarities in the variation in results occur in more than one test. As a result, these comparisons are inconclusive.

xii) The relationship obtained between air permeability and moisture content of mortar specimens compares well with the relationships published in literature. This illustrates the problems that are experienced when trying to compare results from tests carried out on specimens with unknown moisture contents.

xiii) The relationships between in-situ permeability test results and moisture content, show that a wide variation in results can be obtained from any specimen if it is tested at a number of different moisture contents. This is considered to be the main problem with these tests for in-situ use.

xiv) A variety of methods are available to measure in-situ moisture content of concrete, but the majority of methods require some calibration before use because results are dependent upon the actual concrete being tested.

xv) A method of determining moisture content by measuring the electrical permittivity of the concrete was examined and was felt to have a great deal of potential. A consistent relationship was found, although there is a wide spread of results.

xvi) Results of limited accelerated tests were inconclusive.

xvii) Tests on concrete specimens cured for different lengths of
time showed variations in the results of all of the in-situ
tests, but only the initial surface absorption test showed
the expected relationships for all of the mixes tested.

11.2 Recommendations

i) Work should be carried out in two stages; firstly a
continuation of the laboratory investigation; secondly a site
investigation.

ii) Laboratory work should examine further the effects of
moisture content on the different permeability tests, and the
different methods of measuring in-situ moisture content.

iii) To avoid any influence that oven drying might have on the
results, tests should be conducted on specimens that are air
dried.

iv) Tests should be carried out to assess the effects of wetting
and drying on concrete to simulate normal environmental
conditions. Specimens tested in this condition should then be
more representative of in-situ concrete.

v) Tests should be carried out on site to verify the suitability
of the different test methods.

vi) In-situ permeability of concrete should be measured relative
to its moisture content using the available tests.

vii) Results of in-situ permeability measurements should be
related to site concrete with proven properties to confirm
whether or not they can be used as a measure of the
cement's ability to resist deterioration under the existing
environmental conditions.

viii) In-situ permeability tests should be carried out on new
and existing concrete structures to determine whether or not
they will require protection to avoid deterioration.
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APPENDIX I
<table>
<thead>
<tr>
<th>Mix*</th>
<th>Thickness (mm)</th>
<th>Pressure (psi)</th>
<th>K(m/s)</th>
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<td>1.48x10^{-12}</td>
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<td>3.92x10^{-12}</td>
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<td>2.32x10^{-11}</td>
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<td>2.77x10^{-12}</td>
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* Specimen identification: M113/1, c:s=1:1, w/c=0.3, /1=specimen No.1
§Only tested for 7 days; †Tested for 1 hour only; $Only tested for 11 days

TABLE 1 Water Permeability Test Results from Mortar Mixes
<table>
<thead>
<tr>
<th>Mix*</th>
<th>Thickness (mm)</th>
<th>Pressure (psi)</th>
<th>( K \times 10^{12} \text{m/s} )</th>
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<td>5.97</td>
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<td>50</td>
<td>2.51</td>
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<td>C206</td>
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<td>2.97</td>
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<td>C207</td>
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<td>40</td>
<td>3.51</td>
</tr>
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<td>C208</td>
<td>27</td>
<td>20</td>
<td>10.10</td>
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*Mix identification C203 is mix C2, w/c=0.3 etc.,

**TABLE 2 Water Permeability Results for C2 Concrete Mixes**
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<tr>
<th>Mix*</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>Dry Flow (cc/min)</th>
<th>k (m²)</th>
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<td>21.6</td>
<td>0.6</td>
<td>1.43x10⁻¹⁵</td>
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<tr>
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<td>21.6</td>
<td>0.4</td>
<td>9.52x10⁻¹⁶</td>
</tr>
<tr>
<td>M114A</td>
<td>49.1</td>
<td>21.6</td>
<td>0.7</td>
<td>1.66x10⁻¹⁵</td>
</tr>
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<td>M114B</td>
<td>49.0</td>
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<td>0.6</td>
<td>1.42x10⁻¹⁵</td>
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<td>21.6</td>
<td>3.4</td>
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<td>21.5</td>
<td>0.7</td>
<td>1.68x10⁻¹⁵</td>
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<td>21.5</td>
<td>0.6</td>
<td>1.44x10⁻¹⁵</td>
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<td>3.60x10⁻¹⁵</td>
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<tr>
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<td>21.6</td>
<td>370.0#</td>
<td>8.87x10⁻¹²</td>
</tr>
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</tbody>
</table>

* Mix identification: M113A c:s=1:1, w/c=0.3, specimen A. #=Inlet Pressure 10psi

**TABLE 3** Air Permeability Test Details for Mortar Specimens

*cntd...*
...cntd.

<table>
<thead>
<tr>
<th>Mix*</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>Dry Flow (cc/min)</th>
<th>k (m²)</th>
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<td>21.6</td>
<td>1.7</td>
<td>4.06×10⁻¹⁵</td>
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<td>49.1</td>
<td>21.6</td>
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<td>5.23×10⁻¹⁵</td>
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* Mix identification: M145A; c:s=1:4; w/c=0.5; Specimen A
† Inlet Pressure 10psi; † Inlet Pressure 20psi

TABLE 3 Air Permeability Test Details for Mortar Specimens
<table>
<thead>
<tr>
<th>Mix*</th>
<th>d (mm)</th>
<th>G (g)</th>
<th>t (hrs)</th>
<th>d (mm)</th>
<th>G (g)</th>
<th>t (hrs)</th>
<th>d (mm)</th>
<th>G (g)</th>
<th>t (hrs)</th>
<th>d (mm)</th>
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*Mix designation M113: c:s ratio=1:1; w/c ratio=0.3. † Mix M116 not tested.

**TABLE 4** Details of Vapour Permeability Tests for Mortar
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<th>$K_d 1$</th>
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*Mix designatio M113: c:s ratio=1:1; w/c ratio=0.3. #Values not used when calculating mean. t-mix M116 not tested. Kd1=vapour permeability from specimen No.1.

**TABLE 5** Vapour Permeability Results from Mortar Specimens
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<th>d (mm)</th>
<th>G (g)</th>
<th>t (hrs)</th>
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<th>$\bar{K}_d$ ($10^{-5}$ m/hr)</th>
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*Mix identification C203/1 is mix C2, w/c=0.3, specimen 1, etc.

**TABLE 6 Vapour Permeability Results from C2 Concrete Mixes**