School of Civil and Mechanical Engineering
Department of Civil Engineering

An Investigation into Opportunities for Improvement of Surface Mine Haul Road Functional Design, Construction and Maintenance

Jarrad P Coffey

This Thesis is presented for the Degree of
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Declaration

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

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Date: ………………………...

Project Documentation

Thesis Title: An Investigation into Opportunities for Improvement of Surface Mine Haul Road Functional Design, Construction and Maintenance

Author: Jarrad P Coffey

Date: March, 2012

Supervisors: Professor Hamid Nikraz
Mr Colin Leek

ABSTRACT:

Haul road pavement condition has long been considered as having a significant influence on the efficiency of haulage in surface mining. However limited literature exists relating wearing course condition with performance, resulting in mine operators not maximising the potential value of their haul road assets. This project focuses on defining current issues associated with the functional performance of haul roads, through a case study involving three iron ore mines in the Pilbara region of Western Australia. Consequently attempts are then made to relate material properties to performance so that existing criteria can be verified or refined. Cementitious stabilisation and naturally occurring gravels found adjacent to the mines involved in the case study are then trialled via laboratory testing, show promise and lead to a recommendation of subsequent field trials. Pavement lifetime costing is finally completed with the Pilbara environment as the basis, utilising the most appropriate models currently available for pavement condition and vehicle operating and maintenance costs. This resulted in a lack of variability due to material properties, with maintenance variables having a greater effect. Lastly it is found that potential production loss values (net reduction in ore hauled) outweigh expenditure on improved maintenance practice or even material treatments.

Indexing Terms: Pavement design, pavement maintenance, pavement management, haul road, wearing course, stabilisation, gravel selection, load and haul, haul truck, surface mining, pavement lifetime modelling, pavement defect modelling, pavement construction, Pilbara, iron ore.

TABLE OF CONTENTS

Table of Contents i
List of Figures ix
List of Tables xiii
List of Equations xvi
List of Symbols and Abbreviations xvii
Glossary xviii
1. Introduction 1
1.1. Functional Design of Haul Road Pavements 1
1.2. Objective of Project 1
1.3. Scope of Project 2
2. Background 7
2.1. Functional Design 7
2.1.1. Explanation of wearing course 7
2.1.2. Theory of Roughness – Origins, Meaning & Methods to Quantify 9
2.1.2.1. Historical development 9
2.1.2.2. Contributing Factors 11
2.1.3. Effects on VOC and RR 14
2.2. Functional defects 19
2.2.1. Dust 19
2.2.2. Gravel loss 20
2.2.3. Ravelling 21
2.2.4. Corrugations 22
2.2.5. Potholes 23
2.2.6. Ruts 24

2.2.7. Low Skid Resistance – wet and dry ... 24
2.2.8. Stoniness ... 25
2.2.9. Cracks ... 25
2.3. Traditional Provision/Design Considerations ... 26
2.3.1. Testing ... 26
2.3.2. Structural Requirements ... 26
2.3.3. Functional Requirements ... 30
2.4. Unsealed Road Construction ... 36
2.4.1. Preparation of Sub-Grade ... 37
2.4.2. Optimum Moisture and Control ... 37
2.4.3. Compaction and Maximum Dry Density ... 39
2.4.4. Deleterious Materials ... 40
2.5. Maintenance ... 41
2.5.1. Reasoning for Maintenance ... 41
2.5.2. Methods of Maintenance ... 43
2.5.2.1. Light Grader Blading ... 44
2.5.2.2. Re-Shaping/Heavy Grader Blading ... 44
2.5.2.3. Re-Sheeting ... 46
2.5.2.4. Patching ... 46
2.5.2.5. Impact Rollers ... 47
2.5.3. Economics of Haul Road Maintenance ... 48
2.5.4. Maintenance Expenditure ... 48
2.6. Defect Scoring ... 48
2.7. Dust Suppression ... 50
2.8. Stabilisation of the Wearing Course ... 55
2.8.1. Initial Selection of Stabilisers ... 58
2.8.2. Overview of Common Stabilisers ... 60
2.8.2.1. Mechanical ... 60
2.8.2.2. Cementitious ... 61

2.8.2.3. Cement ... 61
2.8.2.4. Lime ... 64
2.8.2.5. Fly Ash ... 68
2.8.3. Ground Granulated Blast Furnace Slag ... 70
2.8.4. Fatigue Failure of Stabilised Pavement Layers ... 72
2.8.5. Non-Cementitious ... 75
2.8.5.1. Polymers ... 75
2.8.5.2. Chemical ... 78
2.8.6. Testing of Stabilised Materials ... 79
2.9. Materials in Western Australia ... 80
2.9.1. Favourable Unbound Granular Material Properties ... 80
2.9.2. Occurrence and Investigation of Natural Gravels ... 80
2.9.3. Commonly Occurring Natural Gravels in Western Australia and their Use in Pavements ... 81
2.9.3.1. Lateritic Gravels ... 82
2.9.3.2. Calcerous Gravels ... 86
2.9.3.3. Scree Gravels ... 87
2.9.3.4. Cohesive Soils ... 88
3. Procedure ... 90
3.1. Laboratory testing ... 91
3.1.1. Particle Size Distribution (PSD) ... 92
3.1.2. Compaction Testing ... 93
3.1.3. Liquid Limit ... 95
3.1.4. Linear Shrinkage ... 97
3.1.5. Plastic Limit ... 99
3.1.6. Capillary Rise and Swell ... 99
3.1.7. Strength Testing ... 100
3.1.7.1. California Bearing Ratio (CBR) ... 100
3.1.7.2. Resilient Modulus (M_R) ... 101

3.1.7.3. Unconfined Compressive Strength (UCS) Testing 102
3.2. Material Treatment Trials 103
3.2.1. Borrow Material 104
3.2.2. Mechanical Stabilisation 104
3.2.3. Cementitious Stabilisers 104
3.2.4. Chemical Stabilisers/Dust Suppressants 105
3.3. Comparison of Materials 106
3.3.1. Material Properties - South African Model 106
3.3.2. Further Changes possible – Relating to Practical Use 108
3.3.2.1. Additional Considerations for Comparison 109
3.3.2.2. Strength 109
3.3.2.3. Permeability 110
3.3.2.4. Cohesion and Consistency Limits 110
3.3.3. Recommendation of optimal material for Pilbara region 111
3.4. Pavement Defect Modelling 111
3.4.1. Dust 112
3.4.2. Gravel Loss 114
3.4.3. Roughness 116
3.4.4. Rolling Resistance 119
3.5. Defect Costing Model 119
3.5.1. Production Impact of Material Type 120
3.5.2. Maintenance Cost 122
3.5.3. Material Construction Treatment Costing 130
3.5.4. Vehicle Operating Cost (VOC) 133
3.5.4.1. Fuel Consumption 133
3.5.4.2. Tyre Usage 142
3.5.4.3. Maintenance Costs – Parts and Labour 145
3.6. Defect Progression Score 148
3.6.1. Relation to material type 151

3.6.2. Relation to Defect Progression 151
3.6.3. Relation to Remediation Effort 152
3.6.4. Relation to Geometry 152
3.6.5. Modification for avoidance of likely defects from material testing 153
3.7. Construction/Design advice for use of waste material 153
4. Results 154
4.1. Laboratory Testing 154
4.1.1. Definition of Materials Currently Employed 154
4.1.2. Treated Materials 156
4.2. Comparison for Functional Design 160
4.3. Pavement Lifetime Modelling 164
4.3.1. Materials Currently Employed 164
4.3.1.1. Total Costs 164
4.3.2. Treated Materials 167
4.3.2.1. Total Costs 167
4.3.3. Borrow Materials 170
4.3.4. Material Construction Treatment Costing 174
4.3.5. Variation with Blading Interval (D) and Traffic (ADT) 176
4.3.6. Effect of Watering 179
4.3.7. Material and Maintenance Impact on VOC Components 181
4.3.7.1. Tyre Life 181
4.3.7.2. Truck Maintenance 182
4.3.7.3. Fuel Consumption 183
4.4. Defect Progression Score 184
4.4.1. Overall Weighting of Categories 184
4.4.2. Defect Progression 185
4.4.3. Geometric Effect 186
4.4.4. Remediation Cost 187
4.4.5. Material Properties 188

5. Discussion 189
5.1. Laboratory Testing – Definition of Materials Employed 189
5.1.1. Introduction 189
5.1.2. Index Tests 189
5.1.2.1. Particle Size Distribution 190
5.1.2.2. Compaction Testing 191
5.1.2.3. Liquid Limit 192
5.1.2.4. Plastic Limit/Plasticity Index 193
5.1.2.5. Linear Shrinkage 194
5.1.3. Capillary Rise and Swell 194
5.1.4. Strength Tests 195
5.1.4.1. California Bearing Ratio (CBR) 196
5.1.4.2. Resilient Modulus 197
5.1.4.3. Fatigue Assessment 198
5.2. Laboratory Testing - Treated Materials 199
5.2.1. Index Tests 199
5.2.1.1. Particle Size Distribution 199
5.2.1.2. Liquid limit 200
5.2.1.3. Plastic Limit/Plasticity Index 201
5.2.1.4. Linear Shrinkage 201
5.2.1.5. Capillary Rise and Swell 202
5.2.2. Strength Tests 203
5.2.2.1. Unconfined Compressive Strength (UCS) 203
5.2.3. Treated Materials Performance 204
5.3. Comparison for Functional Design 204
5.3.1. Materials Currently Employed 204
5.3.2. Treated Materials 205
5.3.3. Borrow Materials 207
5.3.4. Suitability of Selection Criteria 207

5.4. Pavement Lifetime Modelling ... 208
5.4.1. Materials Currently Employed ... 209
5.4.1.1. Potential Production Losses ... 209
5.4.1.2. Vehicle Operating Costs ... 212
5.4.1.3. Maintenance Costs ... 212
5.4.1.4. Treatment Costing ... 213
5.4.2. Treated Materials ... 214
5.4.2.1. Potential Production Losses ... 214
5.4.2.2. Vehicle Operating Costs ... 215
5.4.2.3. Maintenance Costs ... 216
5.4.3. Borrow Materials ... 217
5.4.4. Variation with Blading Interval (D) and Traffic (ADT) ... 217
5.4.5. Effect of Watering ... 218
5.4.6. Material and Maintenance Impacts on VOC Components ... 219
5.4.6.1. Tyre Life ... 219
5.4.6.2. Truck Maintenance ... 220
5.4.6.3. Fuel Consumption ... 221
5.5. Defect Progression Score ... 221
5.5.1. Defect Progression ... 222
5.5.2. Geometric Effect ... 222
5.5.3. Remediation Cost ... 222
5.5.4. Material Properties ... 223
6. Conclusions ... 224
7. Recommendations ... 227
References ... 229
Appendices ... 235
Appendix A – Particle Size Distribution Results ... 236
A1 – Mine A PSD Results – Non Treated within Project ... 236
A2 – Mine A PSD Results – Treated within Project ... 238

A3 – Mine A PSD Results – Treated Materials (Mechanical Stabilisation, including added material – 'Surface Fines') 239

A4 – Mine A PSD Results – Treated Materials (Lime/Fly Ash and Cement Stabilisation) 240

A5 – Mine C PSD Results 242

Appendix B – Plots for Compaction Testing 244

B1 – Compaction Plots for Mine A 244

B2 – Compaction Plots for Mine B 247

B3 – Compaction Plots for Mine C 247

Appendix C – Liquid Limit Results 249

C 1 – Liquid Limit Results for Mine A – Untreated Materials 249

C2 – Liquid Limit Results for Mine A: Treated Materials 250

C3 – Liquid Limit Results for Mine B 256

C4 – Liquid Limit Results for Mine C 256

Appendix D – CBR Plots for Calculation 258

D1 – CBR Plots for Mine A 258

D2 – CBR Plots for Mine B 259

D3 – CBR Plots for Mine C 260

Appendix E – K-Theta Plots for Resilient Modulus Calculation 262

E1 – K-Theta Plots for Mine A 262

E2 – K-Theta Plots for Mine B 263

E3 – K-Theta Plots for Mine C 264

Appendix F – UCS Plots 265

Appendix G – Capillary Rise and Swell Results 270

G1 – Summary of Results 270

G2 – Photos Examples of Treatments 271

Appendix H – Defect Scoring 274

7.1.1. H1 – Layout form 274

7.1.2. H2 - Defect Scoring advice 275

LIST OF FIGURES

Figure 1.1: Flow chart of project scope 5
Figure 2.1: The relationship of the three elements of unsealed pavement provision (Thompson and Visser, 2000) 8
Figure 2.2: Visualisation of the RTRRMS model (Gillespie, 1992) 10
Figure 2.3: Illustration of the quarter-car model (Gillespie, 1992) 11
Figure 2.4: Effect of speed on apparent IRI (Mclean et al, 1998) 13
Figure 2.5: Roughness related to structural deterioration shown as rutt depth (Martin et al, 2004) 14
Figure 2.6: Energy distribution of potential energy in fuel delivered to engine (Beuving et al, 2004) 15
Figure 2.7: Relationship of percentage increase due to increased texture depth (Mclean et al, 1998) 17
Figure 2.8: A typical illustration of the inverse relationship between gravel loss and blading (Jones et al, 2001) 20
Figure 2.9: Visual explanation of the forced oscillation theory of corrugation formation (Paige-Green et al, 1990) 22
Figure 2.10 : Original TRH20 wearing course material specification (1990) 32
Figure 2.11: Modified wearing course material specifications due to reflect use of cone penetration method of liquid limit quantification (Paige-Green, 2007) 33
Figure 2.12 : modified wearing course specification (Thompson and Visser, 2000) 35
Figure 2.13: Total cost of ongoing unsealed pavement provision broken into vehicle operating costs and road maintenance costs showing the existence of an optimal point (Thompson and Visser, 2003) 42
Figure 2.14: Passage of Defect Score/Roughness with time and the effect of pavement maintenance (Thompson and Visser, 2000) 43
Figure 2.15: The effect of impact rolling in terms of decreasing DCP penetration (increasing CBR) through a pavement or sub-grade 47
Figure 2.16: Accident potential scores for each defect type, derived from surveying South African coal mines (Thompson and Visser, 2000) 49

Figure 2.17: Functional defect ranking as a product of accident potential and impact score, derived from surveying South African coal mines (Thompson and Visser, 2000) 50
Figure 2.18: Mine haul road integrated design components, showing the inclusion of dust palliatives (Thompson and Visser, 2007) 52
Figure 2.19: Initial selection matrix of various stabilisers (Vorobieff et al, 2009) 58
Figure 2.20: Design procedure for stabilised design of pavement base layers (Auststab, 2011) 60
Figure 2.21: Flowchart for design of cementitious stabilised pavements (Austroads, 2002) 64
Figure 2.22: Interim mix design process for lime stabilised materials (Austroads, 2002) 68
Figure 2.23: Typical blast furnace slag production process (Australasian Slag Association, 2002) 71
Figure 2.24: Long term behaviour of lightly cemented material (Theyse et al, 1996) ... 72
Figure 2.25: Observed cracking in unstabilised shoulder adjacent to polymer stabilised pavement during drought period in New South Wales (Lacey, 2004) 76
Figure 2.26: Recommended grading of parent material for application of insoluble dry powdered polymer stabilisers (Lacey, 2006) 78
Figure 2.27: Occurrence of Lateritic gravels around Western Australia (Butkus, 2003)83
Figure 2.28: Common appearance of a coarse laterite in the Northern portion of Western Australia (Butkus, 2001) 84
Figure 2.29: Occurrence of Calcretes within Western Australia (Butkus, 2003) 87
Figure 2.30: Scree gravel sampled from near Newman, in the Pilbara region of Western Australia (Butkus, 2001) 88
Figure 3.1: Flow chart of methodology utilised for this project to determine optimal practices for provision of functional haul roads 91
Figure 3.2: Photograph of mechanical agitator used for large sieves 93
Figure 3.3: Cone Penetrometer dimensions (WA 120.2, MRWA) 96
Figure 3.4: Photograph of cone penetrometer setup 97
Figure 3.5: Photograph of dried moulds for linear shrinkage test 98
Figure 3.6: Photograph of the apparatus and setup for CBR penetration testing 101
Figure 3.7: Photograph of tri-axial apparatus used for resilient modulus testing 102
Figure 3.8: Modified model utilised for comparison of materials for functional design (Paige-Green, 2007) 106

Figure 3.9: Modified wearing course specification for avoidance of most common/costly haul road defects (Thompson and Visser, 2000) 107
Figure 3.10: Optimal range of grading coefficient against shrinkage product 108
Figure 3.11: Expected roughness values for varying quality pavements (Austroads, 2011) 118
Figure 3.12: Caterpillar 793C Gradeability Curve (Caterpillar, 1998) 121
Figure 4.1: Changes to Liquid Limit due to addition of stabilisers 158
Figure 4.2: Changes to Plasticity Index due to addition of stabilisers 158
Figure 4.3: Changes to Linear Shrinkage due to addition of stabilisers 159
Figure 4.4: Changes to Shrinkage Product due to addition of stabilisers 159
Figure 4.5: Changes to Grading Coefficient due to addition of stabilisers 160
Figure 4.6: All untreated sample results plotted within functional design criteria 161
Figure 4.7: All treated samples results plotted within functional design criteria 162
Figure 4.8: All potential borrow area results plotted within functional design criteria . 163
Figure 4.9: Potential production losses for sampled untreated materials 165
Figure 4.10: VOC for sampled untreated materials 165
Figure 4.11: Maintenance costs for sampled untreated materials – no watering 166
Figure 4.12: Maintenance costs for sampled untreated materials with watering applied 166
Figure 4.13: 90 day roughness progression for each raw and treated materials 167
Figure 4.14: Estimated maximum daily production cost for respective raw and treated materials 168
Figure 4.15: Total averaged VOC costs for raw and treated materials 168
Figure 4.16: Total averaged maintenance costs for raw and treated materials – non-watered 169
Figure 4.17: Total averaged maintenance costs for raw and treated materials – watered 169
Figure 4.18: Borrow material comparison – potential production losses 171
Figure 4.19: Borrow material comparison – VOC 172
Figure 4.20: Borrow material comparison – maintenance costs (note all non-watered material types have identical curves) 173
Figure 4.21: Total running costs for untreated materials for ADT of 100 and 300 (production impact shown as daily) 176
Figure 4.22: Variation in production losses against blading interval 177
Figure 4.23: Variation of VOC with major inputs 177
Figure 4.24: Maintenance costs variation with major inputs 178

Figure 4.25: Production impact due to water sprays, ADT 300 179
Figure 4.26: VOC impact due to water sprays, ADT 300 180
Figure 4.27: Remediation cost impact due to water sprays, ADT 300 180
Figure 4.28: VOC – Tyre cost variation with blading interval (D) and traffic (ADT) 181
Figure 4.29: VOC – Truck maintenance costs variation with blading interval (D) and Traffic (ADT) 182
Figure 4.30: VOC- Fuel cost variation with blading interval (D) and traffic (ADT) 183

LIST OF TABLES

Table 2.1: Factors effecting rolling resistance (Beuving et al, 2004)............................ 16
Table 2.2: Empirical findings from rolling resistance research (Mclean and Foley,1998) 18
Table 2.3: MRWA recommended particle size distribution for gravel used in sub-base (MRWA, 2010).. 27
Table 2.4: Other Limits for acceptability of natural gravels for use in sub-base (MRWA, 2010) 27
Table 2.5: Particle size distribution for gravel base course (MRWA, 2010) 28
Table 2.6: Other acceptance limits for gravel as base course (MRWA, 2010)............. 29
Table 2.7: Properties of unbound granular material types (ARRB, 2000) 31
Table 2.8: Paige-Green specifications beyond gradin coefficient vs. shrinkage product, to include maximum size and soaked CBR (Paige-Green, 2007)............................... 34
Table 2.9: Recommended parameter ranges for mine haul road wearing course material selection, after modification observing most costly and common defect (Thompson and Visser, 2000)... 36
Table 2.10: Typical assessment moisture control for each drainage unit (Butkus, 2003) 38
Table 2.11: Compaction equipment commonly available and where best to employ each (adapted from Caterpillar, 1989)... 40
Table 2.12: Palliative product selection matrix for mine haul road applications (Thompson and Visser, 2007)... 53
Table 2.13: Actions of chemical binders describing different types available (Vorobieff, 2004) 55
Table 2.14: Definition of various states of stabilised pavement materials (Vorobieff et al, 2009) 57
Table 2.15: Effects and uses of common stabilisers (Austroads, 2002) 59
Table 2.16: General properties of lime treated soils (Auststab, 2008) 66
Table 2.17: Example of effects of polymer stabiliser (Polyroad PR21L) applied to gravels of moderate PI (Rodway, 2001).. 77
Table 2.18: Recommended testing regime for stabilised material for the purpose of selection and design with any stabiliser (Vorobieff et al, 2006) 79

Table 2.19: Recommended selection criteria for laterite for use in heavy duty pavements (Butkus, 2003) 85
Table 3.1: Parameters for Standard Compaction Testing (WA 132.1, MRWA) 94
Table 3.2: Parameter values for dust modelling constants 113
Table 3.3: Percentage reduction in production for defect severity exceedance (adapted from Thompson, 2000) 122
Table 3.4: Individual defects and associated parameters for costing of remediation . 123
Table 3.5: Equipment and productivity rates for categories of remediation 124
Table 3.6: Plant, labour and fuel usage rates used for estimation of remediation costs 125
Table 3.7: Bulldozer fuel consumption rates (Caterpillar, 1998) 126
Table 3.8: Grader fuel consumption rates (Caterpillar, 1998) 127
Table 3.9: Loader fuel consumption rates (Caterpillar, 1998) 128
Table 3.10: Compactor fuel consumption rates (Caterpillar, 1998) 129
Table 3.11: Production rates of Caterpillar stabilizer (Caterpillar, 1998) 131
Table 3.12: Machine usage costs used for construction costing 132
Table 3.13: Assessment of Mechanistic Fuel Consumption Models (Greenwood and Bennett, 2003) 133
Table 3.14: All functions constituting the ARFCOM fuel consumption model used 135
Table 3.15: Constants and values used in ARFCOM model 136
Table 3.16: Drive train efficiency coefficient (α) (Caterpillar, 2007) 137
Table 3.17: Aerodynamic Drag Coefficient (CD) (Caterpillar, 2007) 137
Table 3.18: Rolling resistance constants (Greenwood and Bennett, 2003) 138
Table 3.19: Engine modelling constants in ARFCOM model (Greenwood and Bennett, 2003) 139
Table 3.20: Engine accessory load constants (Greenwood and Bennett, 2003) 140
Table 3.21: Engine cooling fan load constant (Greenwood and Bennett, 2003) 140
Table 3.22: Hourly fuel consumption manufacturer data – Caterpillar 793C dump truck (Caterpillar, 1998) 141
Table 3.23: Variables employed in tyre consumption model (Austroads, 2011) 143
Table 3.24: Variables and constants for modelling Caterpillar 793C tyre usage (Austroads, 2011) 143
Table 3.25: Default values for constants within tyre usage model (Carpenter et al, 1999) 144
Table 3.26: Constants and variables employed to estimate parts maintenance costs 146

Table 3.27: Constants and variables employed to estimate labour maintenance costs 147
Table 3.28: Recommended parts and labour model parameters (Bennett, 1998)...... 148
Table 3.29: Relative scores to be applied for Accident Potential (from Thompson and Visser, 2000) 150
Table 3.30: Relative scores to be applied for production impact (from Thompson and Visser, 2000) 151
Table 4.1: Untreated materials testing summary 155
Table 4.2: Treated materials testing summary 157
Table 4.3: Overall material treatment pavement construction costs 175
Table 4.4: Relative scores given to each of the six categories within the defect scoring model........ 184
Table 4.5: Relative scores to be applied for defect progression 185
Table 4.6: Relative scores to be applied for geometric type 186
Table 4.7: Relative scores to be applied for remediation cost 187
Table 4.8: Relative scores to be applied for material properties 188
Table 5.1: Comparison of CBR and resilient modulus values........ 198

LIST OF EQUATIONS

Equation 2.1: Annual gravel loss function (Paige-Green, 1990) 21
Equation 2.2: Grading coefficient function (Paige-Green, 1990).................................. 32
Equation 2.3: Shrinkage product function (Thompson and Visser, 2000) 32
Equation 2.4: Definition of oversize index (Paige-Green, 1990) 32
Equation 2.5: Austroads method of calculating ESA's for an axle group (Austroads, 2011) 73
Equation 2.6: Austroads fatigue relationship (Austroads, 2004) 73
Equation 2.7: Estimation of resilient modulus from UCS (Sukumaran, 2002) 74
Equation 2.8: Fatigue relationship for cemented layer with less than 100 mm cover (Austroads, 2008) 74
Equation 3.1: Linear shrinkage for cone penetrometer test from Casagrande method results (Paige-Green, 2007)... 107
Equation 3.2: Dust generation function (Jones, 2000).. 112
Equation 3.3: Annual gravel loss function (Paige-Green, 1990) 114
Equation 3.4: Definition of Weinert N value (Foley et al, 1996) 115
Equation 3.5: Natural logarithm of roughness (Paige-Green, 1990) 117
Equation 3.6: Quantification of roughness at a given time (Paige-Green, 1990)........ 117
Equation 3.7: Maximum roughness (Paige-Green, 1990)... 117
Equation 3.8: Product for use in roughness calculation (Paige-Green, 1990)............ 118
Equation 3.9: Instantaneous fuel consumption function (Greenwood and Bennett, 2003) 134
Equation 3.10: Tyre consumption model (Greenwood and Bennett, 2003)................ 142
Equation 3.11: Function for maximum circumferential force (Lach, 1996) 144
Equation 3.12: Parts cost estimation (Bennett, 1998) 145
Equation 3.13: Labour component of maintenance costs (Bennett, 1998)................. 146
Equation 3.14: Proposed defect score 149

LIST OF SYMBOLS AND ABBREVIATIONS

ADT	Average Daily Traffic
AGL	Annual Gravel Loss
ARFCOM	ARRB Fuel Consumption Model
ARRB	Australian Roads Research Board
Austroads	Association of Australian and New Zealand Road Transport and Traffic Authorities
CBR	Californian Bearing Ratio
HDM-4	Highway Development and Management 4
IRI	International Roughness Index
MDD	Maximum Dry Density
MRWA	Main Roads Western Australia
OMC	Optimum Moisture Content
UCS	Unconfined Compressive Strength

GLOSSARY

Base:

Section of pavement between the top of the sub-grade and bottom of the wearing course.

Off-Highway Dump Truck:

Large, generally rigid dump trucks used for hauling waste and ore materials, often referred to as a ‘haul pack’ or ‘haul truck’.

Overburden:

Material over-lying ore of surface mine, generally removed in pre-strip or selected as waste during the mining process.

Production:

Throughout the thesis this term is used to describe the amount of ore that is outputted by the mine each year. Alternatively it refers to the amount of material able to be hauled on a given road in a set time (generally tonne/km/day).

Sub-Grade:

Natural or somewhat improved layer of natural soil underlying a pavement.

Wearing Course:

Surface course of the pavement chiefly responsible for the functional design and performance of an unsealed road.

1. INTRODUCTION

1.1. Functional Design of Haul Road Pavements

This project is focussed on the functional design of haul roads, which refers to the provision of a sound wearing course material to ensure a competent and smooth running surface. Included within is a case study considering haul roads in the Pilbara region of Western Australia. There currently exists only a small amount of literature on this topic and thus it is hoped that through the modelling completed herein awareness of the potential cost saving benefits of providing improved haul roads may be realised. The models utilised are simplistic in order to efficiently demonstrate the requirements of an effective unsealed pavement.

1.2. Objective of Project

The main objectives of this project are as follows:

i. Define the range and nature of mine waste materials currently employed for haul road construction, including their respective running cost to determine if the design model proposed is sufficient for use.
ii. Ascertain if stabilisation of the wearing course through mechanical or cementitious means could improve performance whilst remaining economic.
iii. Examine if select borrow material could be sourced close enough to the mines examined in order to be used as or within the wearing course.
iv. Model the major cost inputs associated with the use of large off-highway dump trucks in the load and haul process in order to try and define areas where most

improvement can be made and if any recommendations can be made with regard to the management of haul roads.

1.3. Scope of Project

Haulage is one of the major costs for any mine operation, which is magnified when sub-standard and worn haul roads are utilised. Within the Pilbara region of Western Australia blasted mine waste is often employed for this purpose. Although some selection is made from the available materials, defects and excessive roughness of the pavement surface have been reported by practitioners to be common occurrences. This is likely due to the less than optimal material combined with poor construction practices. Long-term roads linking to key infrastructure are often constructed under contract in the development phase of the mines and are generally above the standard of those built as part of the ongoing mine development. This provides a clear example of the effect that proper construction involving planned compaction means and lift thicknesses could have. It is without doubt that a poor unsealed surface affects the large haul trucks in terms of fuel consumption, tyre wear and usage and general wear on truck components (Thompson and Visser, 2000). This project aims to quantify pavement running costs to identify potential areas for improvement and also demonstrate the effect of insufficient construction and maintenance practices.

The mine wastes currently used have been observed to generally be too coarse poorly graded or at least contain large over-sized particles that are detrimental to the pavements performance. This is an obvious deficiency, however a wider investigation into the physical properties of the materials currently employed has been undertaken in order to define the range of materials utilised. Once this had been completed, possible methods of improvement could be established. Testing initially focused on the classification of the materials with index and strength tests (CBR, resilient modulus) being completed. A focus is the Atterberg or consistency limit tests employed in order to utilise the functional design model/criteria described later. Although these tests cannot perfectly define the materials performance in service they do allow deficiencies to be identified so that methods of improvement can be trialled. For comparison of materials, initially the TRH20 model (via Paige-Green, 1990) will be modified to reflect the use of the cone-penetrometer method for determining the liquid limit of soils and then employed for comparison of materials. This is followed by the modelling of the major cost inputs for the haulage process as well as potential losses due to roads in an exceedingly poor condition. Further discussion of this work is described later.

Modelling the total user costs is relevant to a mine operation due to the inherent balance that must be struck between revenue and environmental impact. The clearest example of this conflict is with the example of water sprays being used for the limitation of dust generation, hereby an interesting paradigm is reached as excessive water usage not only conflicts legislation in most mining regions but is also a socially sensitive issue. This often results due to the potential of acid rock drainage issues due to leaching of active metal compounds from waste or ore stockpiles with excessive water being applied. However if nothing is done the surrounding flora can be smothered and therefore impacted in the long term, affecting a significant modification of the local ecosystem. This situation also initiates confusion with mine operators. If excessive water is applied the unsealed pavement surface will generally allow wetting of the base layer (depending on permeability of the pavement the sub-grade may also become moistened), which is always associated with structural deterioration if the point of saturation is approached. Thus optimisation of the use of water resources on haul roads is a topic of high importance and is addressed in this project through the modelling included. Pavement stabilisation is not a new idea, however it is only in recent times that such treatments have been trialled for use within unsealed wearing courses by the Pavement Recycling and Stabilisation Association (Auststab). Their results have pointed to very smooth and hard wearing surfaces that have outperformed their previous configuration in basically all facets of assessment. This provides the impetus for cementitious additives to be used in a laboratory trial of the parent materials sampled in an attempt to ascertain the potential benefits they may present. Although laboratory results showing an improvement in material deficiencies would be promising it is also acknowledged that such additives must be applied in practice with use of pavement stabiliser machines (White et al, 2010). This obviously involves a significant increase in the construction cost of any pavement and hence the financial saving would need to be significant for a recommendation to be made. Treating the wearing surface in this manner also calls into question the maintenance practices employed. Certainly dry blading of the surface will no longer be acceptable and water sprays will still be required for means of dust suppression so appropriate road cross-fall or camber will be essential. Thus although treating the pavement might seem simple it is of little benefit unless the whole methodology surrounding haul road construction and management is modified accordingly.

Also investigated is the possibility of utilising other naturally occurring materials for either mixing within or use as a wearing course. It is thought to be quite possible that superior materials are common around the mine sites or within the overburden, but are not employed due to a knowledge gap allowing their identification or a perceived excessive cost impact of recovering and transporting them to the point of use. By quantifying the potential benefits of using such materials it is hoped that this attitude can be changed so as to allow for improved materials to at least be stockpiled for use as sheeting material. Mechanical modification of unbound granular materials is a simple process once index tests have been carried out on the parent material, as one simply aims to add a material that provides for the deficiencies identified. The construction of such treatments is not so simple, as it involves thorough mixing-in of the secondary material. Although the costs appear to be potentially inhibitive, a close comparison with other alternatives is to be completed to identify where naturally sourced materials may be best employed.

Figure 1.1 shows a visual representation of the project scope:

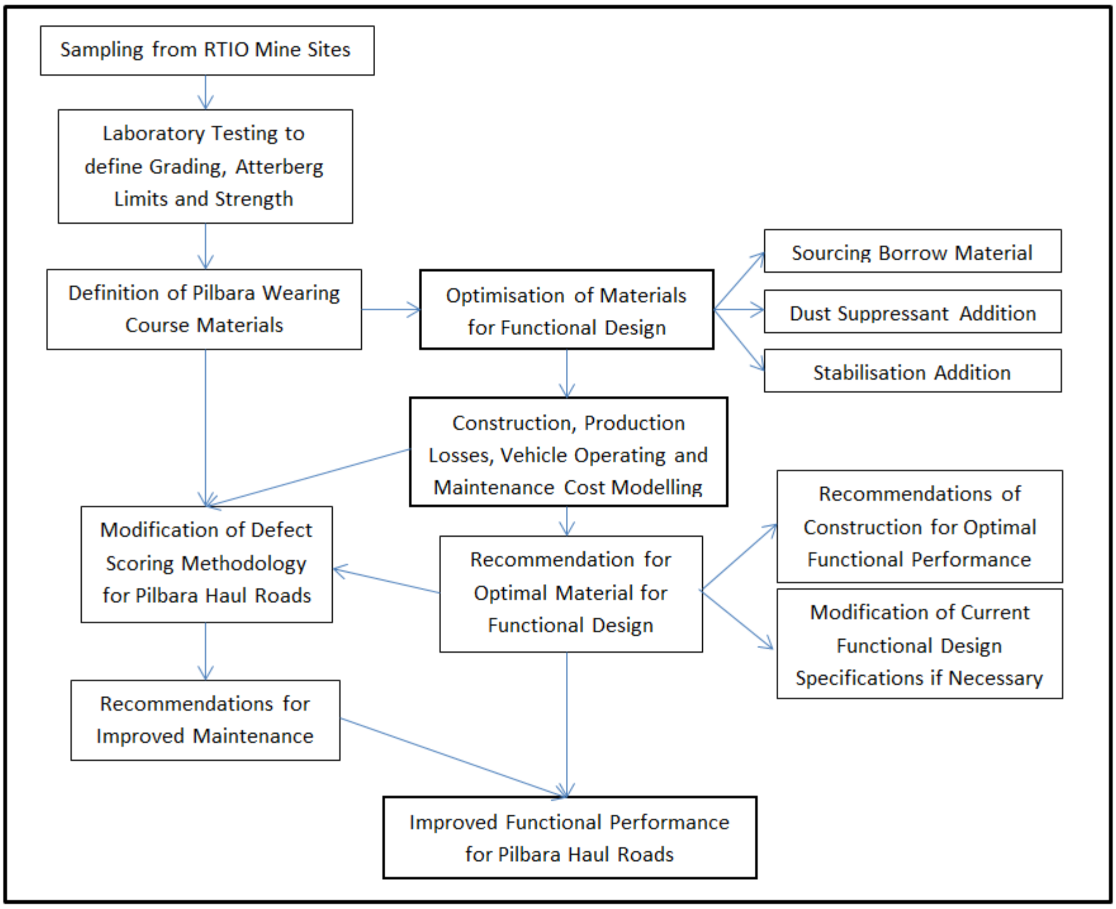


Figure 1.1: Flow chart of project scope

As can be seen in Figure 1.1 the outcome of this project hinges on the definition and assessment of the use of an effective design criteria and assessment methodology. For this the aforementioned TRH20 model is employed when looking at the materials in isolation. It is anticipated that it could require further refinement if all other evidence suggests that the range of materials it identifies as being within an optimal range are not such under Australian climatic conditions or indeed under non-standard traffic. Also possible is the potential miss-representation this model could provide for stabilised materials. For example although cement addition certainly increases cohesion between particles, it may not be shown within the linear shrinkage test and hence not reflected in superior performance within the limits defined.

In order to model the costs for the process of ore and waste haulage the Australian Roads Research Board Road Fuel Consumption Model (ARFCOM) has been utilised along with the other models employed within the Highway Development and Management 4 (HDM-4) modelling software for the quantification of vehicle maintenance and tyre wear costs. Obviously these were not developed specifically for large off-highway dump trucks, however their selection is based on their mechanistic nature allowing many inputs to be estimated by observation of smaller heavy-diesel vehicles. Also modelled are the anticipated remediation costs of each defect and the potential impact to production of each defect and its relative severity. The latter is completed by observation of research completed by Thompson and Visser (2000) where observation and operator surveys were employed to identify those defects most impacting the coal mining industry in South Africa. It is hoped this modelling can be further utilised to help define the parameters of a defect condition scoring methodology following on from the same research as above in order to quantify all the different cost impacts associated with maintenance of haul roads.

2. BACKGROUND

2.1. Functional Design

2.1.1. Explanation of wearing course

The running surface of an unsealed road generally consists of a natural gravel or crushed rock material. As such there are unique challenges related to the provision of an efficient and safe surface (Paige-Green, 1990). Thompson and Visser (2000) state that the functional design of a pavement is concerned with selection of appropriate wearing course materials which optimise the trafficability of a pavement. This view may be somewhat simplified in that some geometric type design factors greatly affect the performance of a wearing course; an example of this is the construction and maintenance of appropriate longitudinal drainage in order to prevent surface material from becoming saturated (Paige-Green, 1990). This is one such relationship that exists in what is ultimately a three element system, each as critical as the last for the establishment of an efficient and safe unsealed road pavements. Thompson and Visser describe the relationship as in Figure 2.1.

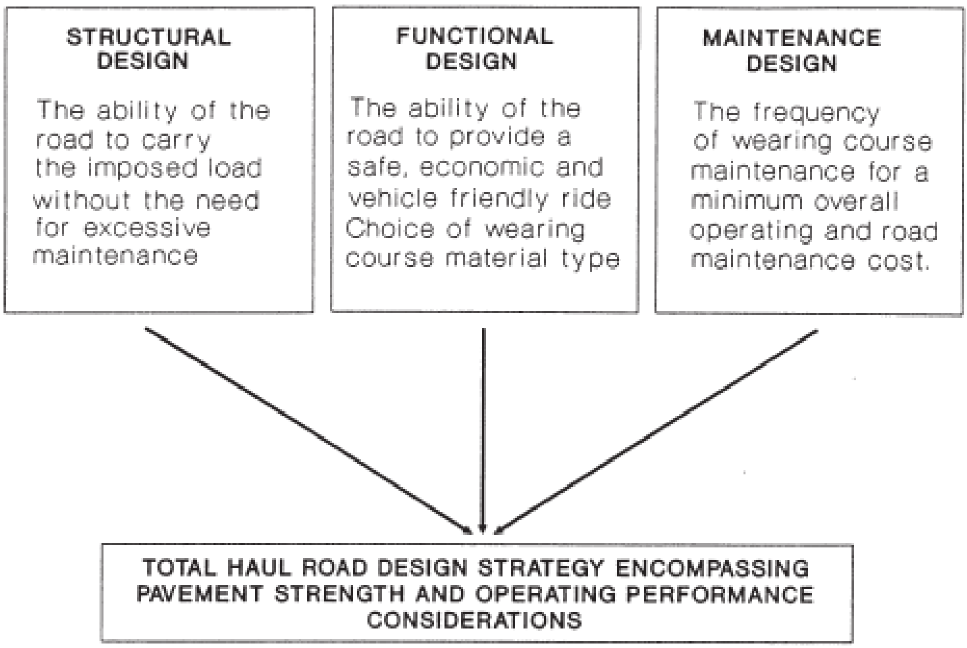


Figure 2.1: The relationship of the three elements of unsealed pavement provision (Thompson and Visser, 2000)

From consideration of this model it is clear that failures associated with maintenance or structural design will also impact the functional performance. Paige-green (1990) recommends that the wearing course thickness not be included within the structural design of a pavement, due to it being eroded over time due to surface water and traffic actions such as the loss of fines in the form of dust. Jones (2001) also notes that the generation of dust is not just a nuisance but also contributes to the deterioration of the surface material, leading to a loose gravel surface due to the lack of binding material which poses threat to windscreens, headlights and damage to bodywork. Ultimately Jones (2001) recommends that the dust control should be considered part of the asset management and road maintenance programme. Hence this defect that is generally grouped within the functional portion of unsealed pavements also undoubtedly requires consideration within the structural and maintenance elements.

The largest contributing factors of wearing course performance are that of material selection, construction technique and maintenance, each of which are described in the proceeding discussion.

2.1.2. Theory of Roughness – Origins, Meaning & Methods to Quantify

2.1.2.1. Historical development

A definition of roughness should first be provided, as some measurements have been developed that actually measure varied ranges of pavement texture. Roughness is defined as the surface irregularities with a wavelength of 0.5 to 50 meters (which is the frequency range that induces relative motion in vehicle suspensions over a working range of speeds (Tan et al, 2011). Texture defines a range of surface irregularities that are smaller in wavelength than this range and define pavement/tyre interaction such as traction (Hengmon, 1979).

Until the 20th Century consideration of pavement roughness was purely considered via rider comfort (Gillespie, 1992). The basis remains the same, only the measurements are being ever more refined and are in more recent times being studied in relation to the effect on vehicle operating costs. Early devices were quite simple and generally consisted of a straightedge with a device able to move vertically to record the total amount of undulation in the surface over a defined length. Figure 2.3 below explains this exceptionally well, where a total deviation can be derived by equating the total deviation from a datum level.

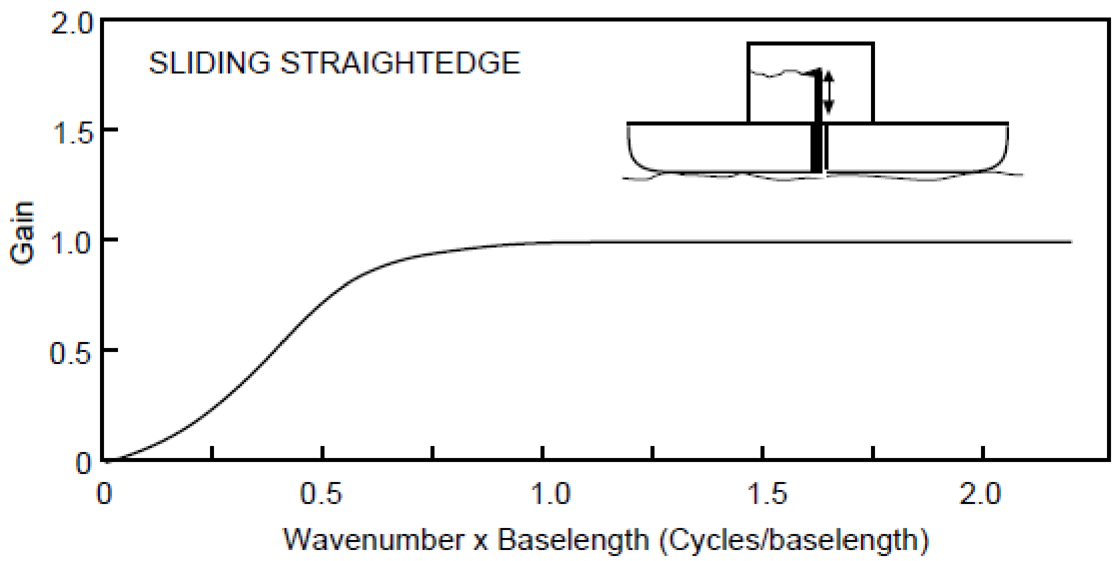


Figure 2.2: Response of sliding straightedge road roughness measuring device (Gillespie, 1992)

Devices slowly evolved allowing more data to be accumulated, in most cases this was done by adding wheels to the straightedge (Gillespie, 1992) allowing greater distances to be efficiently covered and the response to be better reflective of that experienced by a vehicles suspension, and hence how this may be interpreted by the passenger. Developments then focussed on allowing a more detailed quantification by ultimately measuring actual tyre response in some degree of isolation, whilst also taking a specific interest in vibration. Ultimately this lead to the Response-Type Road Roughness Measurement Systems (RTRRMS) model being derived, see Figure 2.3.

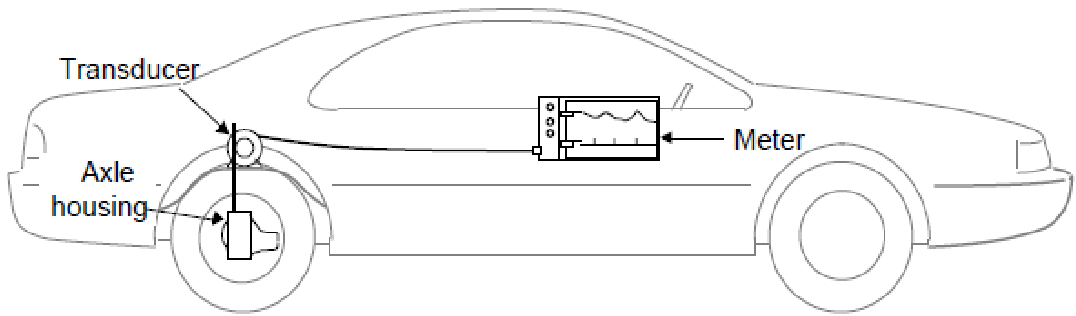


Figure 2.3: Visualisation of the RTRRMS model (Gillespie, 1992)

Finally it was recognised that the differing measures and methods should be consolidated into one theory that facilitated more ranging comparisons, which resulted in the development of the International Roughness Index (IRI) (Gillespie, 1992). This is a scale for roughness based on response of a generic or standard vehicles response to pavement roughness of a road surface when traversing at 80 km/h (Sun, 2001), it utilises an algorithm to interpret and standardise data collected to indicate the accumulated suspension displacements per kilometre of travel that would occur on the idealised quarter-car model (Austroads, 2007). Definition of this ‘reference vehicle’ is done by the quarter-car model, which is a simple dynamic model where each wheel is idealised as a sprung mass sitting on a suspension system with stiffness and damping (Gillespie, 1992), refer to Figure 2.4.

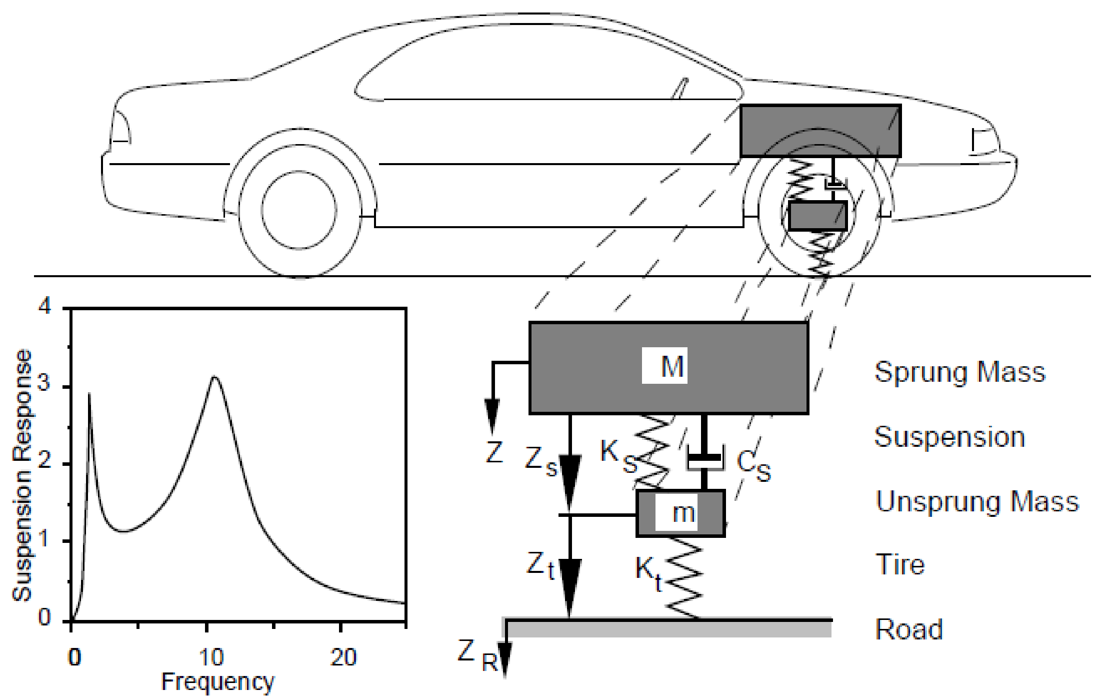


Figure 2.4: Illustration of the quarter-car model (Gillespie, 1992)

It is worth noting that the benefit of the quarter-car model is that whether roughness is defined as deviation in elevation, slope or change of slope, the response occurs in a defined manner (Gillespie, 1992).

Usefully the World Bank has validated a large range of equipment that are able to be used to define the IRI, each of which has a defined procedure that can be followed to produce repeatable and comparable results (Gillespie, 1992). Hence despite the fact that other measures are still available and being used today in different locations around the world, the suite of possible testing methods now means the IRI can be used by highway engineers across institutional boundaries (Gillespie, 1992).

2.1.2.2. Contributing Factors

There are many factors that will contribute to the roughness of a pavement. In some cases certain measurements cannot account for some contributing factors but still need to be considered throughout the life of any pavement. Most research surrounds quantifying the condition of the pavement that describe the vertical undulation of the surface into defined groups and subsequently attempting to prove a relationship exist between the roughness and each level (Beuving et al, 2004). Although this simplifies the problem it does not however reflect that any tyre/pavement interaction is inherently dynamic. This needs to be considered with each of these two elements in isolation, which themselves are constantly undergoing variations in their condition due to any one of a range of independent factors. Each of these effects are discussed in turn.

Differing definitions are available for the ranges of texture, however each includes definition of micro- through to megatexture with a further class often defining the most extreme case of unevenness (Beuving et al, 2004). The texture classes are linked to the wavelength of undulations of the surface plane up to a limit (500 mm in the above noted text) with unevenness being quantified by wavelengths exceeding this value. The correlation between the level of texture and roughness when using a measure such as the IRI is quite arbitrary, all that needs definition is the percentage of the surface has what level of texture. A simple integration can then define the total summed amount of vertical deviation per unit length (hence the IRI). However it is the definition of the magnitude of such a measurement and the energy required for a vehicle to pass over the pavement in question that is ultimately being investigated.

This somewhat over-simplifies the theory of roughness, it cannot simply be considered the sum total of vertical anomalies due to the fact that differences in the tyre/pavement interface vary the effect this has on forward movement of each vehicle. For example it is noted that the effects of texture decrease with an increase in tyre radius or inflation pressure (Mclean et al, 1998). To compound this it is also noted that mega texture can induce additional energy losses due to secondary effects within the suspension during the response of wheel hop (Mclean et al, 1998). Additionally the apparent roughness for each driver is varied owing to the speed of the vehicle they are driving. Clearly the rate of change of vertical displacements of the vehicle will vary with the speed at which the undulation is passed, once again generating secondary effects. Modelling this situation is extremely difficult, however ARRB present the curves in Figure 2.5 to define the change in apparent IRI.

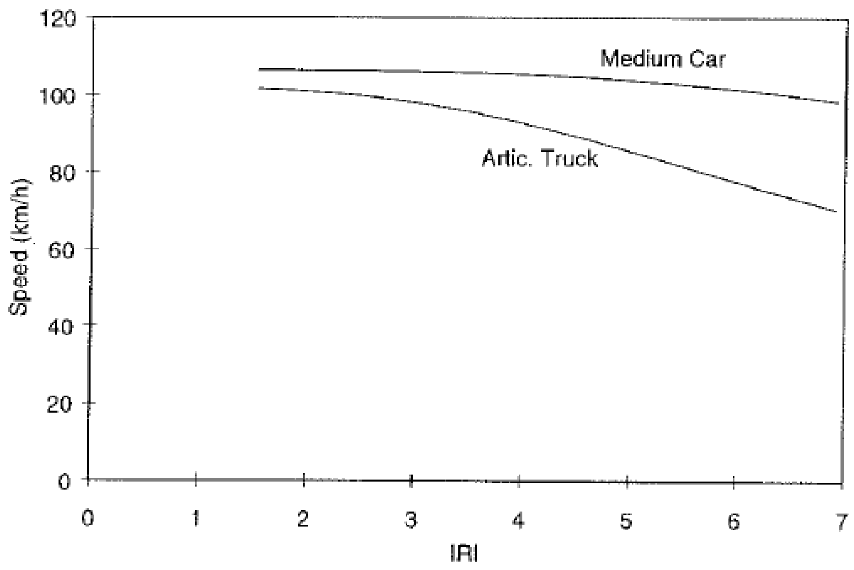


Figure 2.5: Effect of speed on apparent IRI (Mclean et al, 1998)

Beyond the dynamic interactions of pavement and tyres, the isolated effects of both elements also require consideration (Sandberg, 2011). Behaviour of a pavement under load is considered by the structural behaviour, which is an immensely complex system that is still being researched at length today in an attempt to develop an appropriate mechanical model that can be applied universally to any pavement. Ultimately if one extrapolates from the many systems at play within the structure in question and simply attempts to look at the progression of roughness with a decrease in structural integrity, generally quantified in the form of rut depth (accumulation of permanent vertical compressive strains), then some conclusions may be possible. Martin et al, (2004) attempt to model this relationship for asphalt pavements in Figure 2.6. It is clear from this figure that the solid markers which were associated with the observation of rapid deterioration do show that pavements that undergo consistent surface movements do indeed present a greater propensity for a high IRI. It is acknowledged that rutted pavements may indeed often be found to have a low roughness score. It must be noted that real pavements rarely experience rapid structural deterioration and thus the relationship shown is largely impractical and should only be applied sporadically at most (Martin et al, 2004). However it does still show that the roughness of a pavement will often be dependent on its structural condition also.

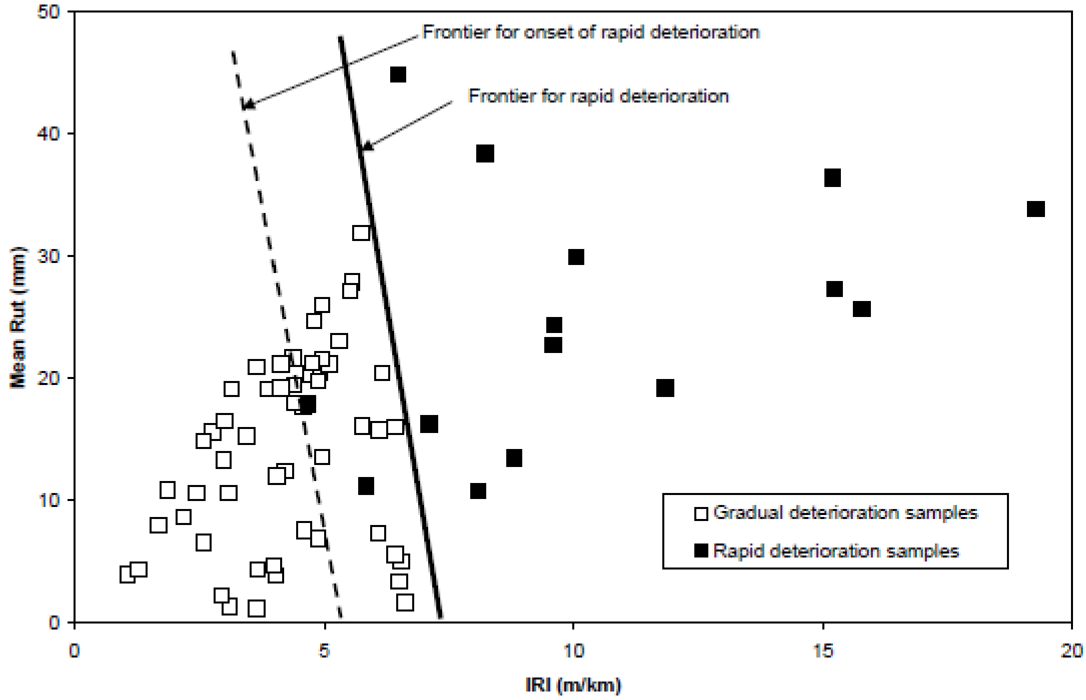


Figure 2.6: Roughness related to structural deterioration shown as rutt depth (Martin et al, 2004)

The other isolated element of the system is tyre response to vertical deviations, which can be further related to suspension, tyre and indeed whole vehicle response to traversing different textures or unevenness at varying speeds.

2.1.3. Effects on VOC and RR

If a surface provides a means for transport without adversely affecting the wellbeing of the driver and vehicle then the consideration of roughness relates directly with operating costs (thus energy or fuel consumption), represented by fuel consumption which is often modelled via the theory of rolling resistance. Thompson and Visser (2003) report that in the case of unsealed mine haul road the rolling resistance is derived purely from the roughness of the pavement. This view does have some merit in that it provides a convenient simplification but is at odds with much of the other literature available. For example ARRB (1998) reports that in one such study conducted in New Zealand it is found that pavement type/strength dominates pavement effects on a trucks fuel consumption, not roughness as is often accepted. A neat summation on the matter is provided by the EABA and Eurobitume fuel efficiency report (Beuving et al, 2004) which notes that the type of road pavement and its surface character influence rolling resistance, whereby different surface characteristics (texture) contribute to rolling resistance significantly as does the structural behaviour.

An overall context for the use of energy by a truck and in particular this in relation to the surface condition of the pavement is provided in Figure 2.7. It can be seen here that rolling loss represents approximately 10 % of energy use.

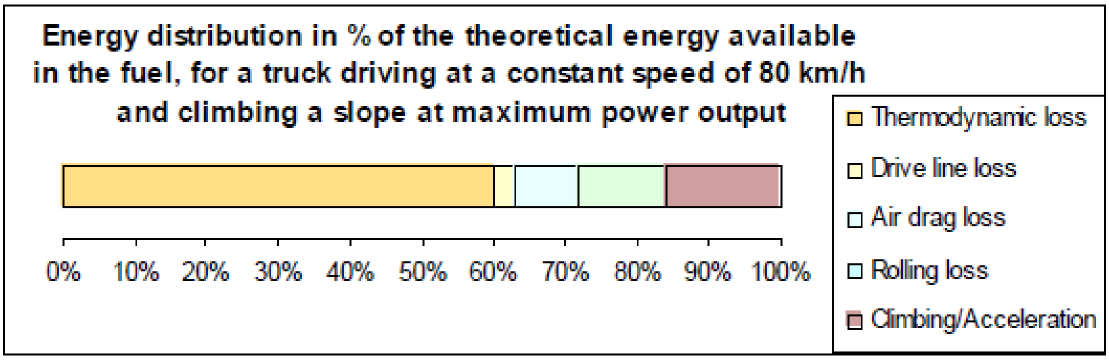


Figure 2.7: Energy distribution of potential energy in fuel delivered to engine (Beuving et al, 2004)

A more detailed definition of the factors contributing to vehicle operating costs with respect to rolling resistance is provided in Table 2.1, however each category is not quantified. The sheer amount of factors noted within this table explain how complex any modelling activity would need to be to accurately define the energy dissipations of a rolling wheel on a given pavement.

Table 2.1: Factors effecting rolling resistance (Beuving et al, 2004)

Tyre characteristics	Tyre Operating Conditions	Environmental Conditions	Road Surface Characteristics
Construction: - cross ply - bias-belted - radial Tread: - compound - pattern - depth - fragmentation	Inflation pressure Load Speed Slip angle Camber angle Driving/braking force Wheel/axle configuration	Temperature Water Snow Ice	Micro-texture Macro-texture Mega-texture Unevenness

To relate the effect of roughness to fuel consumption (which is complicated by but should technically include a quantification of rolling resistance) the ARRB has completed some research which summarises what information is available on the topic. Figure 2.8 shows four author's findings relating texture depth and increased rolling resistance. This removes any over-complication by inclusion of any measure such as the IRI which would add to the number of variables in the relation (as has been discussed above these measures themselves have many complex idealisations within them that are not easily defined). It is shown in Table 2.1 that larger texture depth relates to larger increases in rolling resistance, which could be true but this is also largely dependent on the speed and weight of the vehicle (Sandberg, 2011).

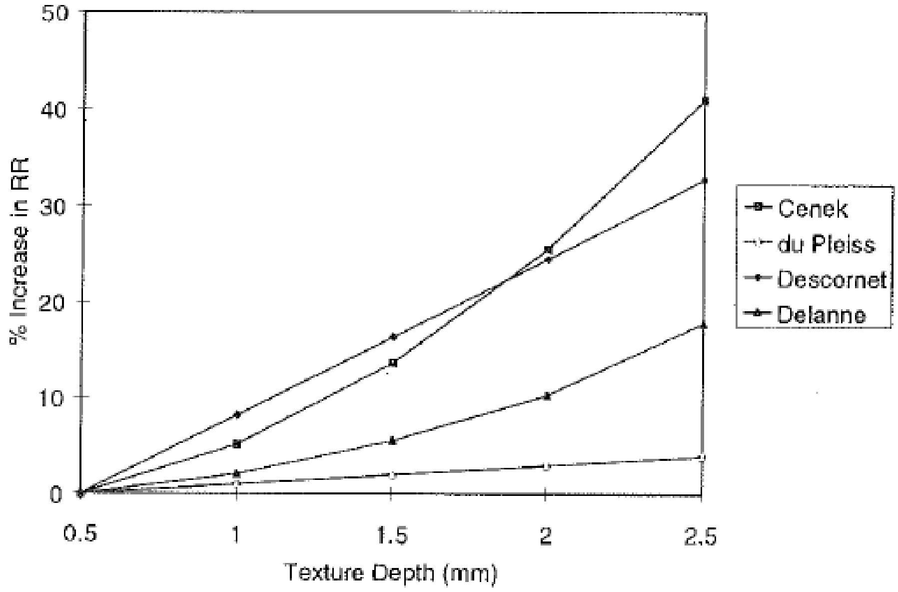


Figure 2.8: Relationship of percentage increase due to increased texture depth (Mclean et al, 1998)

This relation shows a general trend reported by several authors to date, in fact within the same report from which Figure 2.8 is taken is presented an even more comprehensive summary of the sensitivity of rolling resistance and fuel consumption to IRI (Table 2.2). This shows that there is generally a large variance found in the sensitivity of fuel consumption. There exists many variables in terms of independent environmental factors within this set of research, which are only compounded by the many different methods of defining the ultimate fuel consumption. This in turn can be noted by the range of measurement methods employed. It does appear a more tangible trend can be identified if those examples utilising the same methods of definition of fuel consumption are isolated over similar IRI ranges and vehicles.

Table 2.2: Empirical findings from rolling resistance research (Mclean and Foley,1998)

Source	Method	IRI* Range	Vehicle Type	% Change per Unit of IRI: Rolling Resistance	% Change per Unit of IRI: Fuel consumption
Young (1988)	Coast down - artificial roughness	1.3 to 4.0	Truck		4.1
	Direct fuel measurement - artificail roughness	3.3 to 5.6	Car		3.1
	Direct fuel measurement - vehicles side by side	2.3 to 4.4	Car		3.6
	Direct fuel measurement - range of surfaces	1.7 to 5.4	Car		0.8
Ross (1982)	Direct fuel measurement - range of surfaces	0.5 to 3.7	Car		0.4
Bester (1984)	Rolling resistance - range of surfaces	1.4 to 5.5	Car	2.6	0.5
Descornet (1990)	Rolling resistance - range of surfaces	0.8 to 7.7	Car	4.0	0.8
Laganier and Lucas (1990)	Rolling resistance - range of surfaces	1 to 6**	Car	6.0	1.2
Sandberg (1990)	Direct fuel measurement - range of surfaces	1 to 6*	Car		1.7
du Plessis et al (1990)	Rolling resistance - range of surfaces	1.2 to 15	Car	3.4	0.7
			Truck	4.4	1.1
Watanatada et al (1987)	Rolling resistance - range of surfaces	2 to 14	Car	2.5	0.5
			Truck	1.8	0.5

* 1 unit of IRI = 26 NRC

** Estimated range

2.2. Functional defects

An understanding of the development and impact of the full spectrum of functional defects is critical in order to develop optimisation practices in material selection, design, construction or throughout the pavements life with respect to maintenance. The proceeding discussion attempts to provide an overview of common functional defects, some related purely to material properties and others that are the direct result of traffic actions or deficiencies in geometric or structural design. In most cases a combination of these factors typically occurs.

2.2.1. Dust

Dust is the result of fine particles being released from the surface of the road and becoming airborne. In most cases dust is predominantly made up of silt sized particles, those ranging in size from 2 to 75 micrometers (Paige-Green, 1990). The amount of dust generated is a function of aerodynamic shape and speed of passing vehicles along with surface material properties (Paige-Green, 1990). Alternatively the number and size of wheels along with the passing speed significantly affects the quantity of dust generated as evidenced by Equation 3.2 (Jones, 2000). This would appear to have a larger affect in practice, with aerodynamic shape a secondary effect influencing the behaviour of airborne dust once it is generated. Adverse impacts of dust are primarily related to safety concerns due to reduced visibility, as reflected in the study completed by Thompson and Visser (2000) where it was found to have the greatest impact of all functional defects on South African surface mine haul roads, taking account of both safety and operational impacts. Additionally there are concerns held for the health impact of dust on those exposed, especially if constituents such as asbestos or silica are present (Paige-Green, 1990). Finally the effect on vehicle condition/wear and surrounding vegetation and thus animal populations should also be given a much higher priority than currently allotted.

2.2.2. Gravel loss

The consequence of gravel loss as a function of dust emissions from a pavement surface is coming to be understood. The research completed by Jones et al (2001) has assisted considerably in understanding the functional and economic impact of not controlling gravel loss through fugitive dust emissions. Figure 2.9 illustrates the exaggerated maintenance (blading) requirements if gravel loss is not controlled. Furthermore an appreciation is necessary of the effects to material degradation, due to a lack of binding fines. This has the lead on effect of accelerated roughness progression and decrease in structural adequacy through a weaker wearing course material and also decrease in total cover thickness of the sub-grade (Jones et al, 2001).

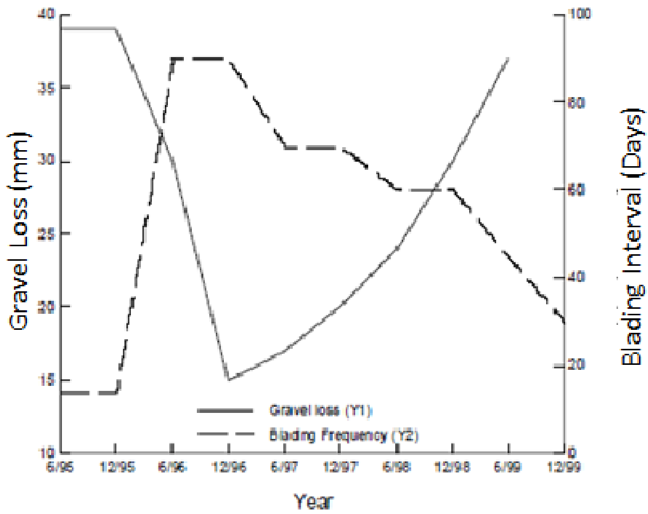


Figure 2.9: A typical illustration of the inverse relationship between gravel loss and blading (Jones et al, 2001)

Both Paige-Green et al (1990) and Jones et al (2001) produce the following equation (Equation 2.1) for prediction of gravel loss due to many factors . Most important to note is that loss prediction is a function of traffic volume, material composition (namely fines component) and climate through consideration of Weinert N-value. The latter is especially relevant and begins to show how important in-situ moisture content is to unsealed pavements, especially those containing a high percentage of silt or clay fines, which coincidentally are those particularly prone to dust when in the dry state.

Equation 2.1: Annual gravel loss function (Paige-Green, 1990)

$$G_{LA} = 3.65[(ADT(0.059 + 0.0027.N - 0.0006.P_{26}) - 0.367.N - 0.0017.PF + 0.0474.P_{26}]$$

Where**:**

G_{LA} = annual gravel loss (mm)

ADT = average daily traffic in both directions

N = Weinert N-value (climate index value)

PF = plastic factor (plastic limit * % passing 0.075 mm sieve)

P_{26} = % passing 26.5 mm sieve

2.2.3. Ravelling

Ravelling is the generation of loose material under traffic. It is a function of the fine material within the wearing course, mainly concerned with a lack of cohesion, poor grading (gap-graded materials or those with a deficiency of gravel sized particles) or poor compaction (Paige-Green et al, 1990). The effect of loose material is largely concerned with a decrease in the safety of the surface to passing traffic, as shown by Thompson and Visser (2000). Within this research project it was ranked the third most severe defect impact on safety when considering the effects of poor skid resistance in a dry state or the fifth most severe safety impact for just the presence of loose material that may form windrows on the surface of the alignment which can be hazardous to cross. Unbound material may also damage vehicles, especially windscreens hence in combination presenting a significant safety hazard (Paige-Green, 1990). Finally it must be acknowledged that rolling resistance is likely to increase, especially when ascending longitudinal curves due to an inherent decrease in traction as a result of loose wearing course materials.

2.2.4. Corrugations

The formation of corrugations has long been debated and the general consensus currently is the 'forced oscillation theory'. Explanation of depressions are related to the migration of fine constituents of wearing course material under the action of a tyre moving vertically (possibly as a function of suspension stiffness) whilst rolling horizontally over a pavements surface (Paige-Green, 1990). This theory is explained best diagrammatically below.

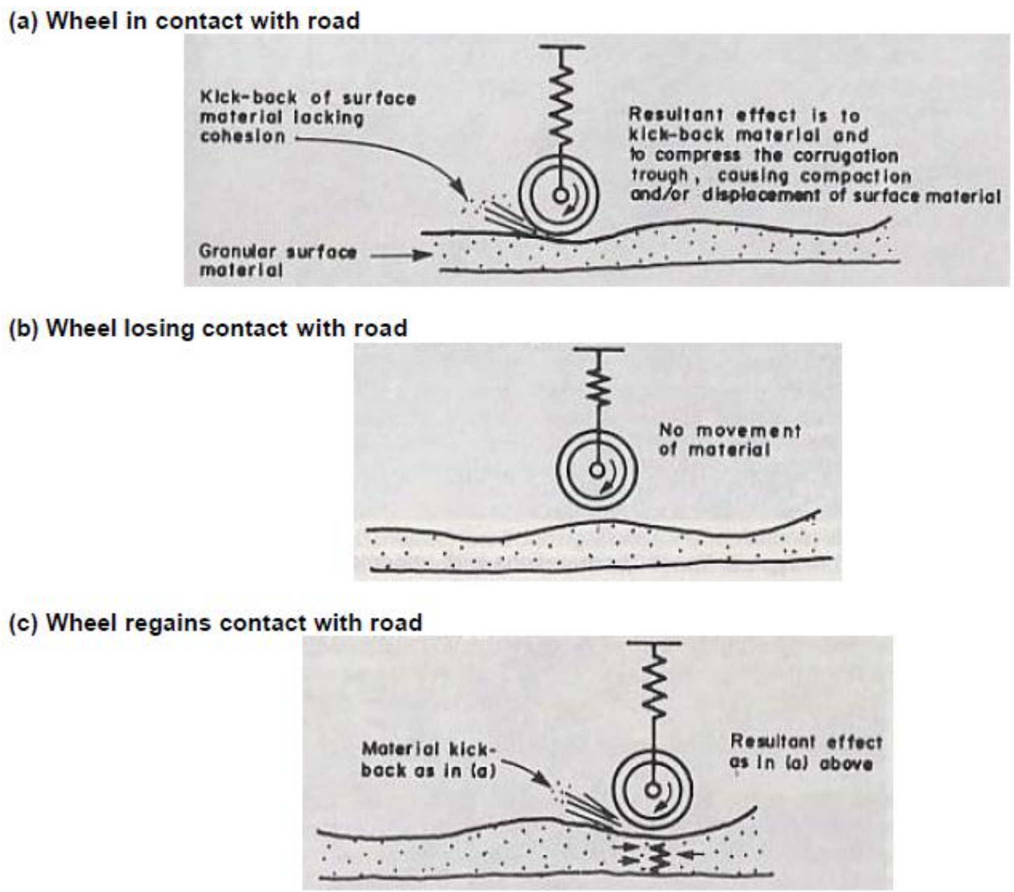


Figure 2.10: Visual explanation of the forced oscillation theory of corrugation formation (Paige-Green et al, 1990)

Corrugations can be termed 'loose' when they generally consist of parallel crests of loose, fine-sandy material perpendicular to direction of travel or 'fixed' which comprise compacted, parallel crests of hard fine-sandy material (Paige-Green, 1990). Consideration of the two formations is relevant due to the fact that loose corrugations are easily removed by blading, whereas fixed corrugations usually require tyning by grader and cutting to below the bottom of depressions for reinstatement of a smooth surface (Paige-Green, 1990).

From a material point of view corrugations generally occur with wearing course materials of low plasticity and a large percentage of sand or fine-gravel fraction (Paige-Green, 1990). These effects are exacerbated by a deficiency in binder or cohesion, as it is rare for corrugations to form during periods of high in-situ moisture content (Paige-Green, 1990). Lastly it should be noted that slow speed traffic such as that on haul roads are observed to be less likely to corrugate (Paige-Green, 1990).

Obviously the main effect of corrugations is to increase the roughness of a pavement surface, if it is observed that a typical corrugation may have a vertical deviation of say 50 mm and a wave length of one meter, then over say 20% of one kilometre travelled (200 meters total of corrugations), the total deviation would be 10 meters per kilometre, assuming the other 80 % (800 meters) of the alignment were perfectly smooth. No matter which roughness measure is used this is undoubtedly a significant result. The realms of this example are very much possible.

2.2.5. Potholes

Potholes are a common and easily identified defect on unsealed roads, with those having a diameter between 250 and 1500 mm and depth exceeding 50 mm having the greatest effect on vehicles (Paige-Green, 1990). In Thompson and Visser's (2000) quantification of severity of functional defects a significantly low rating was given to potholes. This result may be surprising but also could be the result of subjective research in that if the roads surveyed had sound reactive maintenance practices in place then potholes are unlikely to have occurred on a significant percentage of the alignment and to a significant severity (as noted above). Furthermore decreased haulage productivity could have resulted due to a lesser impedance of potholes to a large haul truck tyre, especially when considering a similar grading is specified as roads used with normal highway traffic which should be somewhat proportionate with pothole diameter. Hence it is perhaps that potholes should be considered a highly severe defect but more likely to occur only on very poorly maintained roads. Even if a pothole is to form it can often easily be avoided by the evasive action of an attentive driver, this is especially the case in slow moving haul trucks, where drivers are afforded the benefit of a high vantage point.

Repair of potholes can be problematic with the only successful way being by enlarging the hole and overfilling it with moist gravel and ensuring sound compaction throughout all of the placed fill (Paige-Green, 1990). The success of this remedial action is critical as Paige-Green et al (1990) also note it is very common to observe potholes occurring in the same location over many years probably due to susceptible material being present.

2.2.6. Ruts

Rutting is the development of a depression parallel to the direction travel and may form as a result of compaction of the sub-grade or compaction of the wearing course or base layers (all structural effects) or through gravel loss of the surface (Paige-Green, 1990). Thompson and Visser (2000) describe the impact in both safety and overall performance of rutting in comparable terms to potholing, in that it is not particularly significant to the performance of a haul road. It is implied by Paige-Green et al (1990) that the passing of vehicles repeatedly on the same lateral lane location only acts to increase the occurrence of rutting. This reinforces that it is primarily a sign of structural failure or distress rather than that of a functional defect. The exception to this may occur (like stated above) when the depression is due to material loss or could also result in particularly soft materials under a low number of passes either due to very weak material which often may be a result of it becoming saturated.

2.2.7. Low Skid Resistance – wet and dry

Low skid resistance was identified by Thompson and Visser (2000) as the single defect with the greatest overall impact on Southern African haul roads. The wet condition was rated first (attributed to a 15 % loss in production if present on any given haul road) with the dry condition ranking third as a separate defect. A lack of skid resistance is shown in this research to present a significant safety risk, subsequently it is worth considering the difficultly of traversing such surface conditions which is shown to potentially have a significant impact on production of any haul road, which is the product of the material hauled and at what speed over a given time. Quite simply a lack of skid resistance when the surface material is in a wet condition is due to an excess of predominantly cohesive clay or to a lesser extent silty fines. In the dry condition, as mentioned above (in the ravelling discussion), a lack of skid resistance is often due to a lack of such cohesive fines and also moisture (Paige-Green et al, 1990). Hence the design of the pavement involves a delicate balance between wet and dry season or the rate of application of dust suppressing water sprays and material selection.

2.2.8. Stoniness

According to Paige-Green et al (1990) stoniness is the inclusion of large particles considered out of the optimum grading (usually those greater than 37.5 mm). Failure to remove such particles may lead to rough roads, difficulty with grader maintenance, poor compaction and development of defects such as corrugations or loose material (Paige-Green, 1990). Thompson and Visser (2000) consider two forms of stoniness that being fixed or free (able to move about the surface under traffic). Fixed stoniness is considered to contribute directly to roughness by presenting a large particle that is not likely to be able to be compacted by repeated traffic at the surface.

2.2.9. Cracks

Cracks are not a defect on their own, hence they do not likely act to increase roughness or decrease the safety performance of a pavement, but rather are undesirable due to the possibility of them under the action of water and traffic to lead to potholes or erosion of the surface. They are most significantly an indication of a cohesive material that has become too dry and is likely to soon produce an undesirable amount of dust or a pavement that has poor structural integrity and hence may be subject to settlement or rutting under traffic, exacerbating the development of functional defects (Paige-Green, 1990).

2.3. Traditional Provision/Design Considerations

2.3.1. Testing

Unsealed roads are generally made of marginal locally available and naturally occurring materials. Mine haul roads particularly are constructed of mine waste materials (Thompson and Visser, 2000) hence material specifications are wide and involve relatively simple testing. In its simplest form, testing should include determination of the PSD as well as a strength test to provide an indication of likely pavement performance. The addition of Atterberg limit testing to these specifications allows one to obtain near the full suite of testing recommended by Paige-Green, which is the most widely accepted set of material specifications, as described in Section 2.3.3.

2.3.2. Structural Requirements

Structural requirements mirror that of any flexible pavement, however consideration of the effect of saturation or ingress of water needs to be heightened due to the lack of a seal (Paige-Green, 1990). Minimum mechanistic specifications do not currently exist for the specific purpose of haul roads or unsealed roads and therefore if required should be taken from flexible road design, if materials are able to fit into the material specifications for the techniques developed to date (Coffey, 2010). Hence any existing minimum material strengths relate to static bearing capacity and more likely the wholly empirical California Bearing Ratio (CBR).

Main Roads Western Australia (2010) make recommendations for natural gravels for use in sub-base in Table 2.3 and Table 2.4 below:

Table 2.3: MRWA recommended particle size distribution for gravel used in sub-base (MRWA, 2010)

AS Sieve Size (mm)	% Passing by mass Minimum and Maximum Limits
75.0	100
37.5	80 - 100
19.0	50 - 100
9.5	36 - 81
4.75	25 - 66
2.36	18 - 53
1.18	13 - 43
0.425	8 - 32
0.075	3 - 19

Table 2.4: Other Limits for acceptability of natural gravels for use in sub-base (MRWA, 2010)

Test	Limits	Test Method
Liquid limit	30.0% Maximum	WA 120.2
Plasticity Index	10.0% Maximum	WA 122.1
Linear Shrinkage	4.0% Maximum	WA 123.1
California Bearing Ratio (Soaked 4 days) at 94% of MDD and 100% of OMC	30% Minimum	WA 141.1

These specifications are not particularly difficult to meet with common pedogenic gravels are commonly found in most areas in the Pilbara region. The combination of these specifications means that to be marginal the material would be classified as Clayey Gravels with a large percentage of fines (GC-CL) according to the Unified Soil

Classification (AS 1726 – 1993). As such a CBR of 30 % is reasonable and likely achievable if proper compaction practices are employed.

Main Road Western Australia also provide specifications for natural gravels as base course within its Pavement Specification, as shown in Table 2.5 and Table 2.6 below. These limits need to be observed with caution as the intended use of these specifications is materials with a sealed surface and trafficked by standard axle loads. They are included for comparison purposes and should not be used in specifications for haul road construction.

Table 2.5: Particle size distribution for gravel base course (MRWA, 2010)

As Sieve Size (mm)	% Passing by Mass Target Grading	% Passing by Mass Minimum and Maximum Limits
37.5	100	100
19.0	80	72 – 100
9.5	57	50 - 78
4.75	43	36 - 58
2.36	31	25 - 44
1.18	23	18 - 35
0.600	18	13 – 28
0.425	15	11 - 25
0.300	13	9 – 22
0.150	9	6 – 17
0.075	7	4 - 13
0.0135	4	2 - 9

Table 2.6: Other acceptance limits for gravel as base course (MRWA, 2010)

Test	Limits	Test Method
Liquid limit	25.0% Maximum	WA 120.2
Linear Shrinkage	2.0% Maximum	WA 123.1
Maximum Dry Compressive Strength	2.3MPa Minimum	WA140.1
California Bearing Ratio (Soaked 4 days) at 96% of MDD and 100% of OMC	80% Minimum	WA 141.1

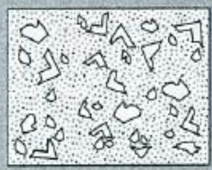
The tightened maximum particle size combined with lower maximum Atterberg limits mean that additional strength should be gained. However a consistent material with a CBR of 80% may be difficult to find in large quantities necessary for thick haul road pavement construction, especially when considering the test is to be carried out in a four day soaked state. The specification for minimum dry compressive strength also may mean that some stabilisation or modification, as is more commonly employed in Western Australia may be required. Note that a minimum is not necessarily a specification without qualification, as often the maximum strength is limited also for performance relating to fatigue for example, see Section 2.8.4. Another example is the specification of resilient modulus that better describes the loading/unloading performance of the pavement over time. Ultimately these specifications describe a material that is not likely to be excessively active with the addition of water (important on haul roads due to water sprays being employed for dust suppression) and also quite resistant to shear failure or progressive rutting. The unified soil classification of these gravels is still possibly the same, however due to the stringent control on Atterberg limits they are more likely to fit into the Silty or Silty to Clayey Gravels (GM-ML or GC) in accordance with AS1726 – 1993.

Specifications for material strength within the literature that focuses on unsealed roads specifically is also available. These do differ quite greatly to that presented above, most likely to facilitate the fact the majority of haul roads are constructed of mine waste material that often only undergoes processing in the form of blasting. Paige-Green (1990) states a minimum CBR of 10 % for general trafficked unsealed roads and a minimum CBR of 15% for use within any pavement layer for haul roads, whereas both Thompson and Visser (2000) only state the specification developed for wearing courses derived by Jones and Paige-Green. A minimum strength of 80% is recommended, which is the same specification as MRWA (2010) state for use in base course. This is therefore a logical specification in that the wearing course should indeed just be an extension of the base using the most stringent selection criteria discussed here.

2.3.3. Functional Requirements

Functional requirements are often seen as the more important aspect of unsealed pavement design, as defects such as those discussed in Section 2.2 above are more likely to develop due to functional short-comings and exacerbated by structural inadequacy. The most simple of specifications is provided by the Australian Road Research Board's 'Unsealed Roads Manual' (2000). These basic specifications are shown in Table 2.7 and include a holistic guide to material selection including provision for structural stability. Table 2.7 could be easily applied for construction of short-term roads not involved in the load and haul process, as extremely heavy axle loads would likely inflict large amounts of damage in a limited number of passes if the only selection criteria used was that in Table 2.7 in combination with lax construction techniques.

Table 2.7: Properties of unbound granular material types (ARRB, 2000)

Diagram			
Type of mixture	**Coarse stone low fines**	**Well graded coarse to fine**	**Excess fines**
Compaction	difficult	moderate	easy
Flexibility	realtively stiff after compaction	moderate	realtively easy
Stability	variable	good	fair
Frost	not affected	susceptible	very susceptible
Drainage	good	low	variable
Effect of water saturation on strength	not much	moderate	very significant strength loss
Chemical stabilisation	not very suitable	suitable	very suitable
Dust	low	moderate	high
Roughness	high	moderate	variable
Capillary effects	very low	beneficial suction	high suction potential instability

A much more detailed specification has been developed by Paige-Green (1990) through many years of observation and testing of unsealed roads in South Africa. These specifications are still being updated as more data is gained. The original specification is presented in Figure 2.11.

Note that for using the proceeding figures, the following equations should be adopted:

Grading Coefficient:

Equation 2.2: Grading coefficient function (Paige-Green, 1990)

$$Gc = \frac{(\%\ Passing\ 26.5\ mm\ Sieve - \%\ Passing\ 2.0\ mm\ Sieve) * \%\ Passing\ 4.75\ mm\ Sieve}{100}$$

Shrinkage Product:

Equation 2.3: Shrinkage product function (Thompson and Visser, 2000)

$$SP = Linear\ Shrinkage * Percent\ Passing\ 0.425\ mm\ Sieve$$

Oversize Index:

Equation 2.4: Definition of oversize index (Paige-Green, 1990)

$$Io = \%\ Material\ retained\ on\ 37.5\ mm\ Sieve$$

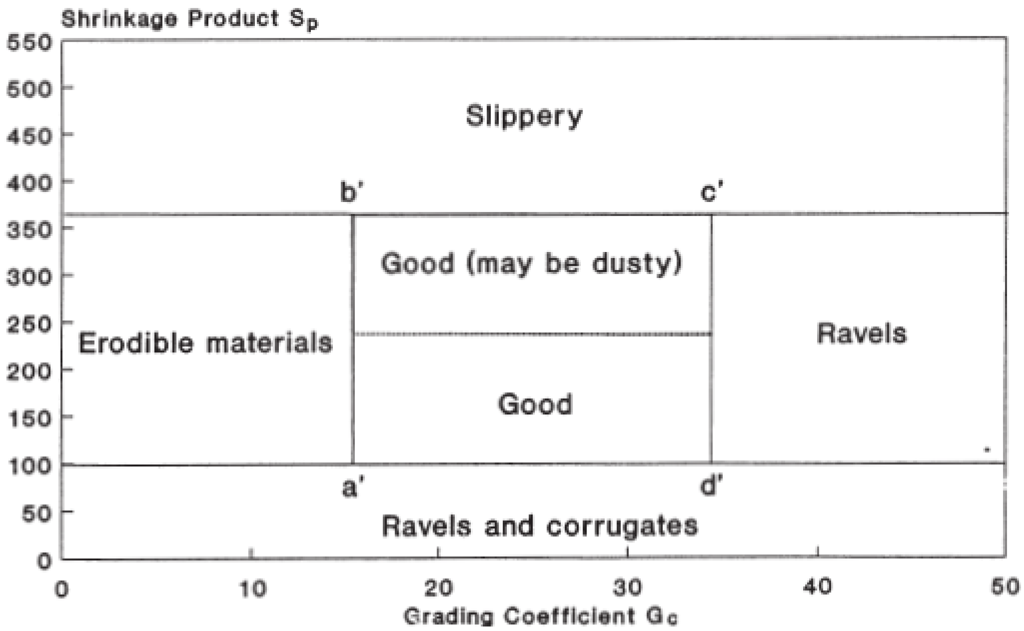


Figure 2.11 : Original TRH20 wearing course material specification (1990)

This specification has been modified most recently in 2007, to reflect use of the British Standard method of Liquid Limit quantification using the cone penetration method and not the Casagrande method. This has resulted in the movement of the limits as shown in Figure 2.12.

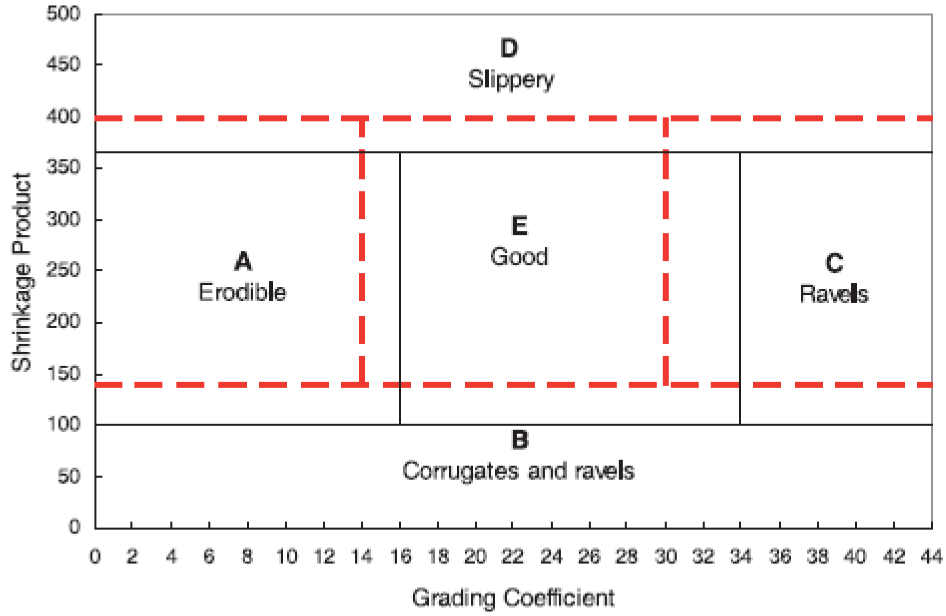


Figure 2.12: Modified wearing course material specifications due to reflect use of cone penetration method of liquid limit quantification (Paige-Green, 2007).

These specifications are reasonably simple to use if a typical regime of laboratory testing is employed as discussed above. The guidance provided by labelling of the segments from A to E also makes definition of likely deficiencies easy to communicate.

To provide further additional guidance Paige-Green (1990) provides further specification that is effectively an amalgamation of all that above in numeric form, as seen in Table 2.8.

Table 2.8: Paige-Green specifications beyond gradin coefficient vs. shrinkage product, to include maximum size and soaked CBR (Paige-Green, 2007)

Property	Value
Maximum size (mm)	37.5
Maximum oversize index (I_o)	5%
Shrinkage product (S_p)	100–365 (maximum of 240 preferable)
Grading coefficient (G_c)	16–34
Soaked CBR (at 95% modified AASHTO compaction)	>15%
Treton impact value (%)	20–65

Special note should be made that Paige-Green's specification (1990) allow a maximum particle size of 75 to 100 mm to allow for the fact that heavy haulage vehicles have larger tyres with lower inflation pressures and usually travel at lower speeds than vehicles travelling on a rural road (Paige-Green, 1990). Interpretation of this specification reveals that it is being inferred that heavy haulage vehicles vehicles are less sensitive to stoniness and thus a larger maximum particle size has been allowed. This specification is likely out-dated due to the fact that inflation pressures on haul trucks far exceed that of road vehicles and often reach pressures of 850 kPa (Caterpillar, 1998) and as such stoniness now presents an increased risk to expensive large off road tyres used on haul trucks. In fact due to recent global shortages of off-the-road-tyres this is now considered one of the most adverse of consequences of poor haul roads. This is reflected by Paige-Green (2007) himself, where a maximum size of 37.5 mm is specified for wearing course materials.

Thompson and Visser (2000) have modified the above specification to reflect a large amount of observations and testing on South African coal mines. The premise of this research was to refine the limits to avoid the occurrence of those defects evaluated to have the greatest impact on the haulage process within the industry. This has meant that the specification shown below attempts to provide for material with reduced risk to poor wet skid resistance primarily, with range 1 being preferable to range 2.

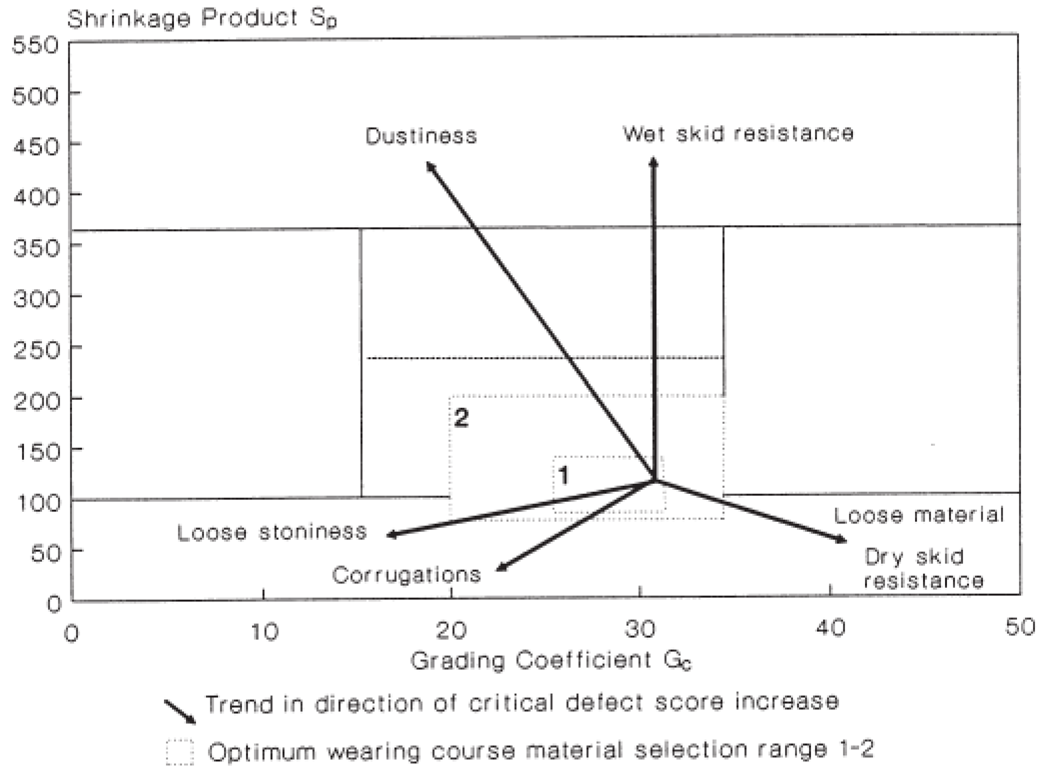


Figure 2.13 : modified wearing course specification (Thompson and Visser, 2000)

This specification has also been further refined by Paige-Green with presentation of Table 2.9. These specifications represent a considerable tightening of the range of materials meeting optimal specifications including the CBR specified which has been increased to a much increased value of 80% compared to that suggested by Paige-Green (2007) of greater than 15%, which would usually indicate only a sub-grade standard material. Hence adoption of these specifications could likely mean a much greater effort would likely be required to source sufficient amounts of material around any mine site. It would also likely mean mixing of materials might well be required increasing the amount of material handling required significantly and therefore subsequent increases in construction time and cost increases.

Table 2.9: Recommended parameter ranges for mine haul road wearing course material selection, after modification observing most costly and common defect (Thompson and Visser, 2000)

Material Parameter	Range Min	Range Max	Impact on Functionality
Shrinkage Product	85	200	Reduce slipperiness but prone to ravelling and corrugation
Grading Coefficient	20	35	Reduce erodibility of fine materials, but induces tendency to ravel
Dust Ratio	0,4	0,6	Reduce dust generation but induces ravelling
Liquid Limit (%)	17	24	Reduce slipperiness but prone to dustiness
Plastic Limit (%)	12	17	Reduce slipperiness but prone to dustiness
Plasticity Index	4	8	Reduce slipperiness but prone to dustiness and ravelling
CBR at 98% Mod AASHTO	80		Resistance to ersosion, rutting and improved trafficability
Maximum Particle size (mm)		40	Ease of maintenance, vehicle-friendly ride and no tyre damage

2.4. Unsealed Road Construction

As with construction of any pavement, haul roads require a certain degree of planning and quality assurance throughout the construction phase, whether following an approved design or optimising what materials and limited equipment is available. As such the following provides advice as to how best maximize the structural integrity and functional performance of an unsealed pavement intended for use as a haul road, however these comments are equally applicable to any lighter trafficked pavement.

2.4.1. Preparation of Sub-Grade

Prior to the commencement of construction activities the sub-grade should be cleared and grubbed with all plant matter and top-soil removed and in most situations stockpiled for future rehabilitation (Paige-Green, 1990). Once these activities are satisfactorily completed, preparation of the sub-grade can begin. At this point it is important to note that a road surface placed over any surface that does not possess the notional bearing capacity required will result in excessive rutting, sinking and accelerated deterioration and thus require a high degree of maintenance over its lifetime (Kaufman and Ault, 1977). Therefore it is strongly recommended that a degree of compaction is specified by way of either relevant on-site testing (commonly moisture ratio and density) or method, generally number of passes by a certain roller (Paige-Green, 1990).

2.4.2. Optimum Moisture and Control

The importance of the inclusion of adequate moisture during construction of pavements is described in Paige-Green (1990) where it is noted that a road built with a high degree of moist compaction resulted in lower roughness than on roads compacted at a less than optimum moisture content (dry condition). Thus the importance of the correct moisture content is to allow the maximum dry density and therefore strength to be mobilised by the material employed (within any layer). The determination of Optimum Moisture Content/Maximum Dry Density is very well known and easy to carry out within an adequately equipped laboratory, however some additional guidance can be provided to assist the practical use of such results. MRWA recommend that compaction is never completed with a moisture content less than 85% of OMC (Butkus, 2003).

Moisture content also extends beyond the construction phase over the pavements lifetime and therefore it is prudent to assess the materials stability at a moisture content representative of the most likely worst environmental conditions it will be subjected to over its lifetime. For assistance with this determination MRWA have provided the following four classes of drainage units (Butkus, 2003):

- Unit 1 (Well Drained) - Deep water table and

- Good external drainage and
- Moisture deficit climate and
- Permeability of base less than permeability of underlying layers

• Unit 2 (Permeability Inversion) - Deep water table and

- Good external drainage and
- Moisture deficit climate and
- Permeability of base greater than permeability of underlying layer

• Unit 3 (High Water Table) - High water table or moisture surplus climate

- Good external drainage
- Permeability of base less than permeability of underlying layer

• Unit 4 (Pavement Saturation) - Pavement subject to inundation or

- Very high water table or
- Poor external drainage or
- Moisture surplus climate

Practical application of these units is through the condition under which testing should be completed, which will be likely representative of the worst moisture conditions likely to be faced, as explained previously. For quantification of these observe below which lists the moisture content that material within each unit should be assessed for strength.

Table 2.10: Typical assessment moisture control for each drainage unit (Butkus, 2003)

Drainage Unit	Typical Assessment Moisture Content *
Unit 1	85% of OMC
Unit 2	110% of OMC
Unit 3	100% of OMC
Unit 4	120% of OMC (soaked)

A final reference should also be made to the technique of ‘drying-back’ during construction. This involves the allowing of a layer to dry to a certain extent (MRWA generally specify 85% OMC for the base course) prior to placement of the subsequent layers or in the case of highway pavements, sealing. This specification for base-course can also be extended for sub-base and sub-grade which when comprised of crushed rock (or natural gravel) should be dried back to at least 60% OMC (Emery et al, 2006). The physical reasoning behind it is that this allows a negative pore pressure to be developed and maintained giving materials a degree of cohesion they otherwise would not develop (Emery et al, 2006). In the case of ‘Pindan’ soils, which occur extensively in the northern regions of Western Australia, self-cementing has been shown to occur far more significantly after drying back has taken place (Jones and Emery, 2003). This is aided by the fact that the least porous material should be placed highest within the pavement, so as to limit the moisture reaching lower layers that would lose the strength gained if they were to become saturated once more or even wetted back to optimum. The confusion with this concept is often that the optimum moisture content is that for compaction/density generation, whereas the maximum strength may be at a lower moisture content once this density has been achieved and employed during compaction.

2.4.3. Compaction and Maximum Dry Density

Good compaction produces a tightly bound surface with optimum particle interlock, minimum permeability and porosity and significantly increased strength (Paige-Green, 1990). A poor degree of compaction results in a low density, permeable material which ravels easily and is highly moisture sensitive (Paige-Green, 1990). From this explanation it can be seen that favourable materials only provide a suitable pavement when they are placed correctly and subject to a closely followed construction methodology and quality control methods. To achieve a good degree of compaction a suitable moisture content is required as described in the preceding in combination with good compaction practise where each lift is of an appropriate thickness for the method of compaction employed (see Table 2.11 below).

One point of note for roads constructed using or over fine materials is that laminations can develop between placed layers, and as such should be placed in a single layer where possible (Butkus, 2003).

Some common compaction equipment and where each is best employed are listed in Table 2.11.

Table 2.11: Compaction equipment commonly available and where best to employ each (adapted from Caterpillar, 1989)

Equipment Type	Material Type	Depth of Penetration (mm)	Comments
Sheepsfoot	Clay, silt, gravel with >20% fines	200-300	Doesn't compact top 50mm
Grid	Rocky (lumpy) material	Less than 250	Breaks lumps
Smooth Drum - Vibrating	Non cohesive sand or gravel	100-500	Very useful for waste rock compaction
Tyre	Gravel surfacing and asphaltic concrete	Less than 250	Unlikely to be particularly useful for mine haul roads

2.4.4. Deleterious Materials

When a material that is considered unsuitable due to either lacking strength or being too moisture sensitive it should be removed and replaced or alternatively modified (Paige-Green, 1990). However this action has to be economically balanced with the alternative of constructing a thicker pavement over less than optimal material. Other such material that should be avoided is oversize particles, which introduces uncertainty in the determination of density (Butkus, 2003). Also as noted before this makes the final trimming of any surface significantly more difficult (Thompson, 2000).

2.5. Maintenance

2.5.1. Reasoning for Maintenance

Maintenance of unsealed pavements is a necessity borne by the fact that no matter how meticulously the design and construction the surface will still deflect to some degree (Tannant and Regensburg, 2001) which will be the basis of distress. The provision of maintenance cannot be considered as an extra element to consider after structural and functional design, but rather is mutually exclusive with both (Thompson and Visser, 2003).

To demonstrate this further consider that Kaufman and Ault (1977) note that a lack of rigidity of the bearing material will permit excessive rutting, sinkage and overall deterioration of the surface and thus necessitate a large expenditure on maintenance to keep the road in a passable state. Furthermore note Figure 2.11 presented in Section 2.3.3, where having exceedingly low or high values of either Shrinkage Product (a function of the activity of the material) or Grading Coefficient (predominantly a function of the percentage of gravel sized particles) lead to the defects noted, thus meaning the only way to keep the pavement serviceable is to employ a large amount of maintenance. Subsequently the benefits of utilising sound material is easily offset by employing less than optimal construction techniques (Kaufman and Ault, 1977), whereby the road surface is ultimately poorly compacted or hampered by geometrical deficiencies such as poor drainage owing to a lack of adequate cross-fall.

Supplied below is a summary of Section 2.2, comprising a comprehensive listing of the major causes of deterioration, the elimination or control of which will alleviate or reduce the need for maintenance. This list is summarised below:

1. Failure of pavement due to inadequate structural or functional design.
2. Dry weather resulting in loss of fines (as fugitive dust emissions) and hence ravelling of the pavement surface.
3. Heavy rain washing away fines leading ravelling of surface material.
4. Heavy rain saturating the surface material.
5. Heavy rain or over watering leading to flooding of the pavement and causing soft areas in pavement or even sub-grade failure.
6. Spillage of either ore or overburden material.
7. Corrugations of the surface due to migration of fines.

8. Pot holing due to rapid progression of defects often under the action of surface water.
9. Damage due to frost heave and consequent loss of density.
10. Damage by tracked equipment (for example Bulldozers of Excavators).

A balance must be found between all the elements of unsealed pavement provision. This is best displayed below in Figure 2.14 where total cost is shown broken into Vehicle Operating Costs (VOC) or rolling resistance and maintenance frequency or road maintenance costs. Observation of the curve showing summation of the two classes of expenditure reveals that an optimum point exists where costs reach a minimum.

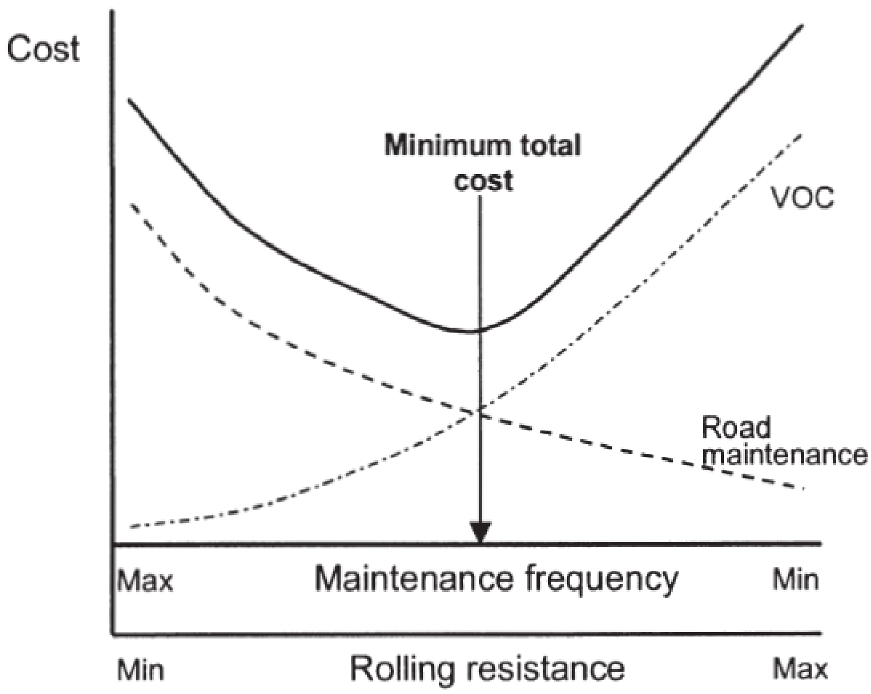


Figure 2.14: Total cost of ongoing unsealed pavement provision broken into vehicle operating costs and road maintenance costs showing the existence of an optimal point (Thompson and Visser, 2003)

When observing this model in the context of a mine haul road one should be reminded that a balance ultimately still needs to be found with the revenue or cash flow resultant from haul truck cycle time, which can be significantly impacted by the state of the pavements they pass.

2.5.2. Methods of Maintenance

There are some principal methods of unsealed road maintenance commonly employed, such as blading, watering, re-gravelling and rehabilitation. The balance on effect of employing such techniques can be shown visually in Figure 2.15, where a nominal defect score which is primarily a function of roughness is plotted against progression of time (Days since last maintenance). It is evident that a theoretical point of minimum roughness or defect score must exist, at which time the newly shaped and compacted surface likely experiences its maximum in-situ density due to compaction by traffic and any consolidation due to drainage of construction water, prior to the aggregation of wear.

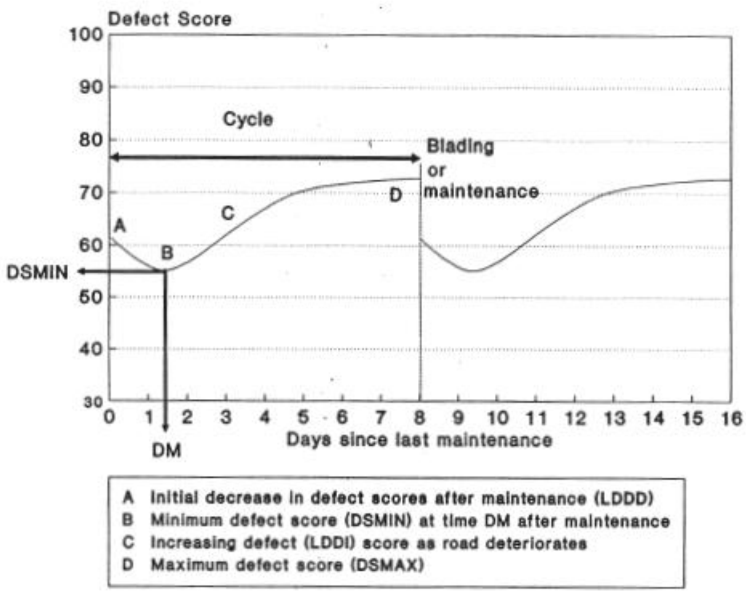


Figure 2.15: Passage of Defect Score/Roughness with time and the effect of pavement maintenance (Thompson and Visser, 2000)

2.5.2.1. Light Grader Blading

Light grader blading is completed by action of a grader running it's blade across the surface of the pavement in such a way to remove all defects such as potholes, ruts or corrugations, whilst reinstating adequate cross-fall for drainage, generally considered to be less than 5 % (Paige-Green, 1990). This is summarised in three steps (McRobert et al, 2000):

1. Heavy cut and winning/recovering of material.
2. Remove all surface defects.
3. Mixing and spreading material over the road surface, avoiding formation of windrows on verge or widening pavement.

It is also worth noting that any sub-standard material, such as that recovered from drains be removed and not re-used (see Section 2.4.4 for comment on the treatment of deleterious material). The justification for this being that after only a light blading the crests will likely not be cut back to even the bottom of the troughs, and any such material used to fill this area will likely not be compacted to a density of that underlying the fill, often leading to the rapid reoccurrence of corrugations.

The timing of maintenance can be critical, for example blading after heavy rain to remove scours from the surface (Paige-Green, 1990). It is also common for a programming of maintenance to be developed, although it must be noted that roads in a poor condition can be expensive. For example one case study found that nearly three quarters of tyre failures were due to cuts or impacts that ultimately could be attributed to poor road condition (Tannant and Regensburg, 2001). Thus blading also must be reactive to reinstate the riding quality.

Lastly, it should be noted that blading work is best carried out in times of average material moisture. Providing sound compaction to meet the same standard as the original construction is also highly important to alleviate the risk of rapid deterioration (Paige-Green, 1990), which ironically could potentially cost more than leaving the surface in its pre-maintenance works condition.

2.5.2.2. Re-Shaping/Heavy Grader Blading

Heavy blading should be carried out when inspections indicate severe defects present over a significant proportion of the alignment, where there is no risk of mixing material into the base course that could compromise its stability (McRobert et al, 2000). The wearing course depth, or at least 75 mm, should be scarified to ensure all defects are removed to their full depth allowing the road to be re-shaped and compacted (Paige-Green, 1990) which should involve the addition of construction water (McRobert et al, 2000). Avoidance of this type of blading may be appropriate in areas where large stones are present near the surface or where a hard crust or 'blad' has formed due to self-cementing materials being employed (Paige-Green, 1990). Once again, water should be added if necessary to allow compaction to the same level as that expected for a newly constructed pavement. It may be necessary to also add and blade in some fines, as these can be lost over time through dust emissions (see Section 2.7 for discussion of dust losses and palliation).

A detailed explanation of such work is described by McRobert et al (2000) and summarised in the following:

Blading:

- Pre-wet existing surface prior to scarifying.
- Additional passes of the grader should be required to ensure desired camber is made, with consideration of settlement adjacent to road crown due to compaction. Complete blading on one side of the road at a time if traffic flow is required.
- Initial passes cut to bottom of defects with material deposited in a windrow just beyond the centreline.
- Windrow is spread back across the surface to correct camber.
- Mix material (above reference describes how this is best achieved by use of a grader – reader is advised to view if more detail desired).
- Second application of water to ensure material is at optimum for compaction.

Compaction:

- Compaction plant should follow close behind grader if traffic passing, compaction to commence only once blading is completed. Generally 8 passes of roller is required, working towards centre of road, treating shoulders as part of running surface. Minimum depth of lift should be 2.5 times the nominal particle size.

Drainage:

- Side drains must be cleaned prior to blading of surface commencing. Material from drains should not be mixed into surface unless inspection determines they are fines sufficient to replace those the surface may have been/become deficient in over time.

2.5.2.3. Re-Sheeting

If the design thickness has been found to be insufficient or gravel loss has been such that more thickness is required to prevent further rapid deterioration, re-sheeting should be employed (Paige-Green, 1990). The re-sheeting process should involve the same construction standard and therefore practice as the original construction and should be blended with the existing surface by means of scarifying so as to avoid a lamination between the existing and new surfaces developing (Paige-Green, 1990). Studies carried out within Australia have suggested re-sheeting be considered once the wearing course has halved in thickness (Andrews, 2001). Thus if Equation 2.1 is employed to calculate the duration for gravel loss equal to half thickness (or whatever the critical value used) then such activity can be programmed into the operating budget of the asset manager. Additionally if an assumption is made that all loose sub 0.425 mm material is eventually lost in the form of dust emissions then an indication of the amount of loose material on the surface is provided by the estimate of gravel loss depth, which is heavily dependent on the grading of the material (Andrews, 2001). Again once a decision of the maximum amount of loose material is made (in the above paper 5 kilograms per square meter is suggested) an estimate of the re-sheeting interval can be made.

2.5.2.4. Patching

Any patches due to persistent or particularly severe defects should be made by way of first excavating the entire defect and surrounding material, if it is suspected there may be a lack of stability. This should then be back filled with use of moist wearing course standard material to approximately one centimetre above the existing surface, whilst being compacted, by hand if necessary, to the same density of the surrounding pavement layers (Paige-Green, 1990). For the specific application of patching it may be useful to have some wearing course standard material stockpiled in the borrow pit (Paige-Green, 1990).

2.5.2.5. Impact Rollers

In recent times a trend toward the use of a ‘square’ roller to break down oversize and achieve a deeper penetration when compacting has been employed on many mine sites (Avalle, 2006). It is thought that this form of rolling allows an impact wave to penetrate deep into the sub-grade in most cases by way of breaking up surface layers, which then leaves the surface ready for successive sheeting (Avalle, 2006). Figure 2.16 displays the improvement possible through a pavement and sub-grade showing that superior compaction is achieved at depths of 300 to 700 mm (Avalle, 2006), when comparing to static or even dynamic (vibrating) rolling.

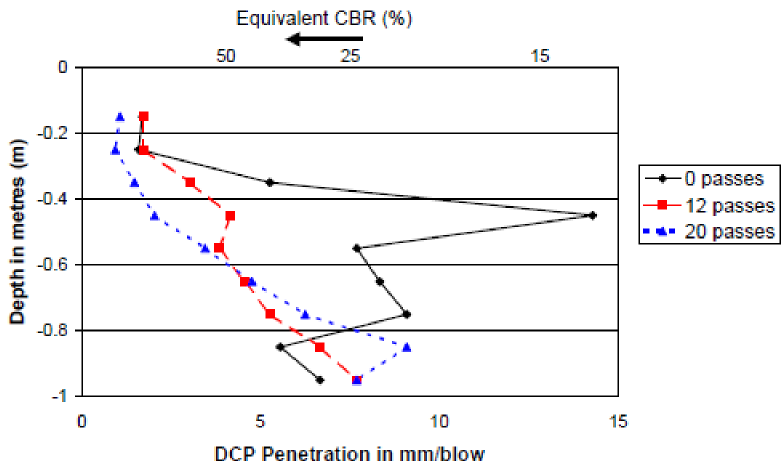


Figure 2.16: The effect of impact rolling in terms of decreasing DCP penetration (increasing CBR) through a pavement or sub-grade

It can be argued that if the sub-grade and subsequent pavement were compacted in appropriate layers of less than 300 mm, then impact rolling would have limited benefits. However as it stands it could provide a cost effective means of maximising the effectiveness of remedial works. A cautionary note is appropriate also, as there is an inherent risk of soil liquefaction occurring under such action (Sanjay et al, 2010).

2.5.3. Economics of Haul Road Maintenance

As mentioned previously the minimum amount of maintenance is largely determined by the effort expended on both categories of design. If the original design and construction are of a high standard there is a decreased likelihood of defects developing and therefore requiring remedial works to improve or maintain the surface. It is useful to have a vehicle operating cost model for use such as the Highway Design and Maintenance Model's various revisions (Paige-Green, 1990). However there doesn't exist such a model for haul trucks, which have the added variable of ore delivered to the processing plant. Thus the expenditure of maintenance becomes a balance of:

1. Design input
2. Material costs
3. Construction cost
4. Vehicle operating costs (maintenance, repairs, fuel consumption)
5. Mine revenue (haulage rate – number of truck passes * mass of ore in each truck)

2.5.4. Maintenance Expenditure

To assist in a decision regarding maintenance at any point of the haul road's life, a cost benefit model such as a Net Present Value (NPV) formula may be employed (Tannant and Regensburg, 2001). Definition of each element above can be very difficult as they each consist of many individual items. Further complicating the decision is the fact that safety should be inherent in each element but is very hard to define (Paige-Green, 1990); a good illustration of this is when the status quo is a rough surface where a decision can be made to maintain the haulage rate at the expense of safety and also road and truck repair/maintenance expenditure.

2.6. Defect Scoring

Defect scores have been developed and revised for sealed highways for some time now, however the programming of maintenance for unsealed pavements is more complex due to it being required on a more regular basis (Paige-Green, 1990). It also should be noted that providing more maintenance effort than is required will likely significantly increase expenditure. Thus it is beneficial to have developed a model that defines when maintenance should be undertaken and even helps to predict when it may be required next.

The best documented system for such for haul roads has been developed by Thompson and Visser (2000), where a ‘Defect Score’ was defined as the product of the summations of ‘Impact Score’, ‘Accident Potential’, and ‘Extent of Occurrence’. An extensive survey was carried out over a 12 month period on a large number of South African coal mines in order to sample a data set in order to rank both accident potential (Figure 2.17) and impact score of a range of common/possible defects. These are then used to produce a ranking of all the types of defect (Figure 2.18), which interestingly shows issues commonly attributed to materials with large fines content or poor cohesion.

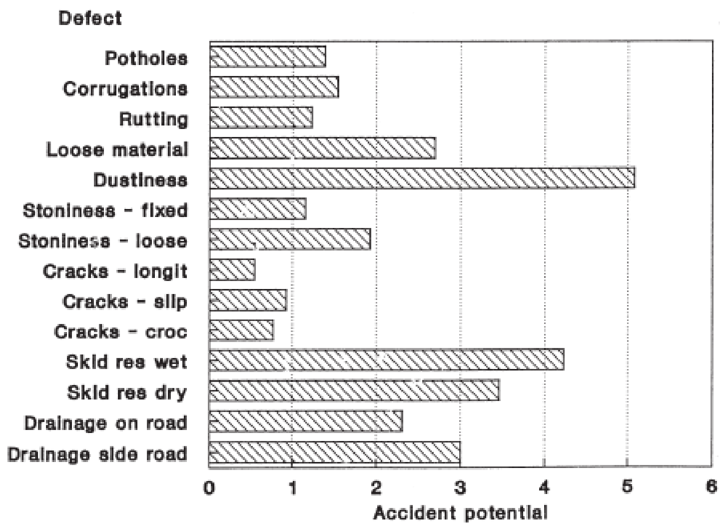


Figure 2.17: Accident potential scores for each defect type, derived from surveying South African coal mines (Thompson and Visser, 2000)

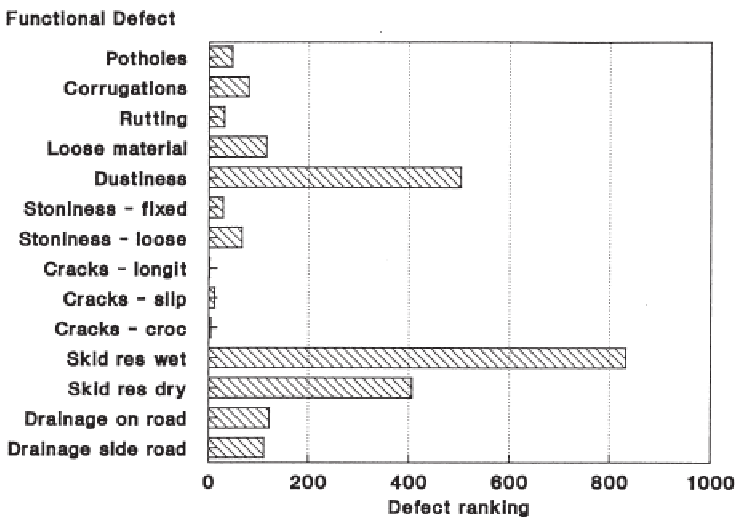


Figure 2.18: Functional defect ranking as a product of accident potential and impact score, derived from surveying South African coal mines (Thompson and Visser, 2000)

Hence these ranking scores then only need be determined for each defect along an alignment to define the defect score. This can then be analysed with a function of defect progression, which is considered to appear as in Figure 2.15. Plotting the defect scores for each haul road observed in terms of material parameters (Shrinkage Product and Grading Coefficient) allowed derivation of Figure 2.13. This allows further planning of the haul roads at the design/construction phases in order to avoid the development of common defects on South African coal mines, alleviating the need for large expenditure on maintenance effort (Thompson and VIsser, 2000).

2.7. Dust Suppression

Dust is primarily made up of silt sized particles that in a dry state become airborne under the passage of traffic (Paige-Green, 1990). As can be noted from Figure 2.18, dust has been found to be the single biggest functional defect that haul roads experience. The following list outlines the undesirable characteristics of an excessively dusty road surface:

- Potential Engine Damage (Thompson and Visser, 2007).

- Reduced visibility to the point of being a significant safety hazard (Paige-Green, 1990).
- It is an indicator of a material that will lack skid resistance when wet due to a high degree of fines, and the potential to develop poor skid resistance when dry due to the development of loose gravel sized particles once fines have been lost in the form of dust (Thompson and Visser, 2007).
- Increased vehicle operating costs through decreased efficiency and increased maintenance and repair costs (Thompson and Visser, 2007).
- Increased wear and structural degradation through gravel loss, leading to a compounding and rapid progression of defects (Jones et al, 2001).
- Smothering of surrounding flora (Paige-Green, 1990).
- Can lead to health issues to workers exposed to air with unacceptable amounts of dust (Paige-Green, 1990)

Hence it is desirable to control emissions. In the most simple of terms this can be achieved by providing a wearing course material that has a dust ratio (percentage of particles finer than 425 micron divided by percentage of particles finer than 75 micron) between o.4 and 0.6, or with a liquid limit in excess of 24 or plasticity index of greater than 8 (Thompson and Visser, 2007). Therefore it may be possible to reduce dustiness of a road by way of mechanically stabilising the wearing course to include a favourable amount of cohesive (clay) fines. This may be an onerous task or such material may not be available, in which case it may be useful to look to the wide range of dust suppressants available.

The use of dust suppressants (or palliatives) needs to be included within the design process mentioned before, as it will also affect the maintenance and structural performance of the road also. Therefore it must feed into the functional and maintenance design as suggested in Figure 2.19 below. Observation will reveal that palliative use is a decision made to enhance sub-standard wearing courses, however it is part of the maintenance design rather than a separate element. Note that for some palliatives such as lingo-sulphonate dry blading is not possible due to the formation of a durable crust over time. This means that any rework or significant damage caused by spillage from haulage trucks is likely to be time-consuming and costly.

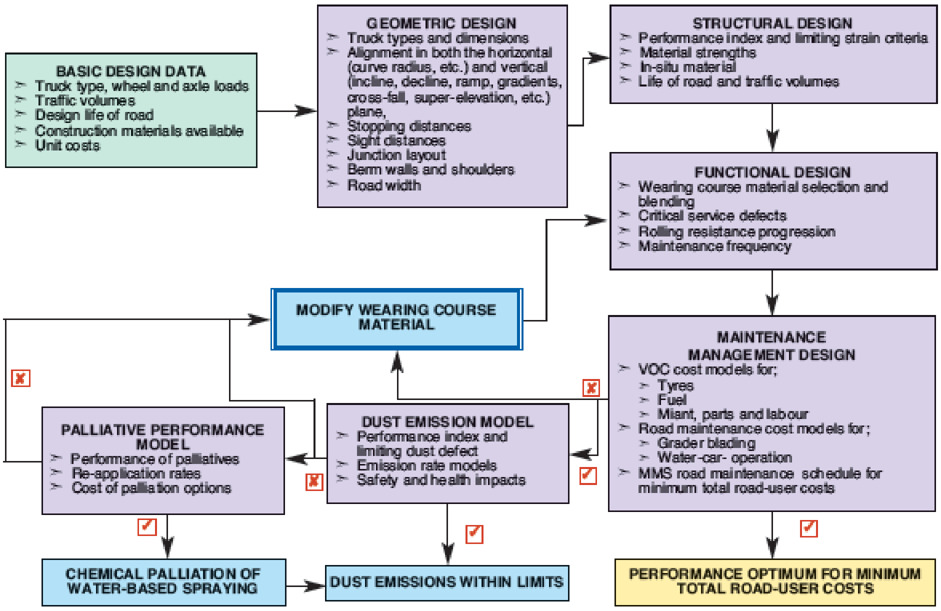


Figure 2.19: Mine haul road integrated design components, showing the inclusion of dust palliatives (Thompson and Visser, 2007)

Table 2.12 outlines the products available and the remedies they provide to any wearing course material. Simply wetting the surface via sprays is employed in many cases and has been noted to be effective, however over-watering can be an issue (Thompson and Visser, 2000) that exacerbates defect progression such as potholing.

Table 2.12: Palliative product selection matrix for mine haul road applications (Thompson and Visser, 2007)

	High PI (>10)	Medium PI (>10)	Sand	Wet weather trafficability	Ramp roads	Heavy traffic	Short-term	Long-term	Spray-on	Mix-in	Maintainable
Wetting agents		√			√		√		√I/R		√
Hygroscopic salts		√				√	√		√R	√I	√M
Lignosulphonates	√	√	√				√	√		√I/R	√SO
Petroleum emulsions	√	√	√	√	√	√	√		√I	√I	
Polymer emulsions	√	√		√	√	√		√		√I	√SO
Tar/ bitumen emulsions		√		√	√	√		√	√R	√I	√SR

Notes
I—Initial establishment application
R—Follow-on rejuvenation applications
M—Maintain when moist or lightly watered
SO—Maintain with spray-on reapplication
SR—Maintain with spot repairs

Analysis of the above table reveals that the most durable of palliatives is petroleum emulsions, which are also shown to be suitable for any short-term application. However such treatments have been legislated against in some parts of the world for their potential harms to the surrounding environment should any leach out from the pavement. Further note should be made of rainfall and climatic conditions, whereby high rainfall areas will obviously be exposed to a larger risk of leaching (Edvardsson and Magnusson, 2011) and also make application hard as many suppressants require to be placed in a dry environment. Interestingly some suppressants are restored at the surface via capillary rise in the pavement soil, with chloride solutions having been shown to be superior to lingo-sulphonates (Edvardsson and Magnusson, 2011).

General comment on the usefulness of each palliative product is difficult and therefore each case should be analysed independently and a decision made based on all available literature, practical experience/advice and field trials if possible. The following discusses the results of some trials completed in Australia, incorporating past experiences with the treatments noted.

There exist many products available commercially, all are differing versions of one of the groups listed in Table 2.13. It is important to note that distinguishing between dust suppressants and stabilisers is difficult, however additives primarily providing one of the actions below are generally defined as suppressants (Vorobieff, 2004). In a trial completed in South Australia it was found that significant variation was found to exist in structural and functional characteristics of treated with several popular products (Andrews, 2001). For this reason any consideration of applying dust suppressants should include practical trials, as the material characteristics will impact performance greatly, thus individual assessment is strongly recommended in each case.

Table 2.13: Actions of chemical binders describing different types available (Vorobieff, 2004)

Action	Description
Adhesion	Act as a glue in bonding particles
Adsorption:	To attract atmospheric moisture to reduce dust emission
Dilatant	To dispel water when compacted under vibration.
Dispersant	Separates fine particles from each other
Ionic	Bonding from a reversing of the electrostatic charge on some soil platelets
Surfactant	To reduce surface tension.

2.8. Stabilisation of the Wearing Course

Although usually reserved for base and sub-grade layers, stabilisation can also be employed to provide a sounder, more cohesive wearing course (Vorobieff, 1998). Mechanical stabilisation may be possible for the purpose of dust suppression by simply providing additional cohesive fines to a material with a sufficiently low dust ratio. Adding additional fines to a material already having a high dust ratio (and therefore high proportion of fines) is risky due to the increase in ratio this addition would affect. Hence in many cases it may be more appropriate to look to other additives. In Western Australia lime is commonly used with plastic materials in order to reduce plasticity, although it can also be used in a similar way as cement to increase strength (Butkus et al, 2003). Austroads recommend that if employing cementitious stabilisation (hence should be applied to lime stabilisation) the material must not exhibit:

- 7-day unconfined compressive strength (UCS) of greater than 1.0 MPa
- 28-day UCS of greater than 1.5 MPa
- Vertical modulus of greater than 1500 MPa (MRWA, 2010)

This is to prevent premature failure due to fatigue cracking (Butkus et al, 2003), owing to a material that is too rigid. Utilising this method within a wearing course material would make maintenance prohibitively difficult due to the hard crust that would be formed, making dry blading impossible and any blading difficult. This could be aided somewhat by the fact that it is common practice in Western Australia to use less than 2 % lime or cement (Butkus et al, 2003), which technically represents a modified and not stabilised material.

The definitions in Table 2.14 are important to understand, as they are based on UCS results and need to be applied correctly to properly describe stabilisation practices within pavement layers:

Table 2.14: Definition of various states of stabilised pavement materials (Vorobieff et al, 2009)

Category of stabilisation	Indicative laboratory strength after stabilisation	Common binders adopted	Anticipated performance attributes
Subgrade	$CBR^1 > 5\%$ (subgrades and formations)	Addition of lime Addition of chemical binders	Vertical deformation Shear failure Seasonal heave & shrinkage
Granular	$40\% < CBR^1 < +100\%$ (subbase and basecourse)	Blending other granular materials which are classified as binders in the context of this Guide.	Flexible pavement subject to shear failure within pavement layers and/or subgrade deformation
Modified	$0.7\ MPa < UCS^2 < 1.5\ MPa$ (basecourse)	Addition of small quantities of cementitious binders Addition of lime Addition of chemical binders	Flexible pavement subject to shear failure within pavement layers and/or subgrade deformation. Can also be subject to erosion by water penetration through cracks. Increased volumetric stability
Bound	$UCS^2 > 1.5\ MPa$ (basecourse)	Addition of higher quantities of cementitious binder Addition of a combination of cementitious and bituminous binders	Bound pavement subject to tensile fatigue cracking and transverse shrinkage cracking. Low binder contents may be subjected to erosion through pumping through cracks
Notes: 1. Four day soaked CBR. 2.Values determined from test specimens stabilised with GP cement and prepared using standard compactive effort, normal curing for a minimum 28 days and 4 hour soak conditioning.			

Other stabilisers do exist, such as bitumen and electrochemical or even microbial binders. However it is difficult to ascertain if any of these additives would currently be economical on a mine site unless the road alignment was planned for long-term use, thus any decision should be made subjectively. The following information is provided to assist.

2.8.1. Initial Selection of Stabilisers

Auststab provide sound advice on stabilisation of both new and recycled low traffic pavements. This organisation is currently carrying out trials of stabilisation of wearing course layers of unsealed pavements. An initial selection matrix is shown in Figure 2.20, it should be used to identify in broad terms which stabiliser may be effective on any typical alignment. Note that some laboratory testing is required in order to effectively utilise such advice, especially when determining application rates.

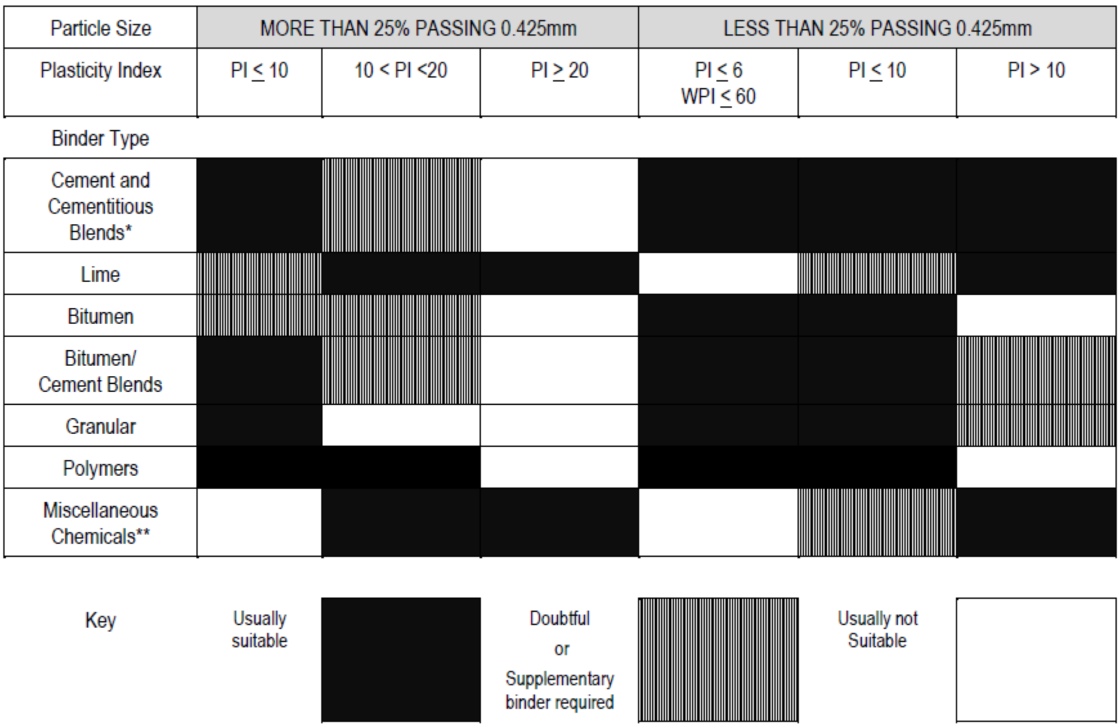


Figure 2.20: Initial selection matrix of various stabilisers (Vorobieff et al, 2009)

Selection should also be based on some degree of qualitative input, as the purpose of stabilisation is varied. As such Austroads have provided the Table 2.15 to describe the effects and use of the most common stabilisation practices.

Table 2.15: Effects and uses of common stabilisers (Austroads, 2002)

Stabilisation Binder	Process	Effects	Applicable Soil Types
Cement	Cementitious interparticle bonds are developed	(a) Low additive content (<2%); Decreases susceptibility to moisture changes resulting in modified or bound materials. (b) High additive content; Increases modulus and tensile strength significantly resulting in bound materials.	Not limited apart from deleterious components (organics, sulphates etc which retard cement reactions). Suitable for granular soils but inefficient in predominantly one sized materials and heavy clays.
Lime (including hydrated lime and quicklime)	Cementitious interparticle bonds are developed but rate of development slow compared to cement. Reactions are temperature dependent.	Improves handling properties of cohesive materials. (a) Low additive content (<2%): Decreases susceptibility to moisture changes, improves strength resulting in modified or bound materials. (b) High additive content. Increases modulus and tensile strength resulting in bound materials.	Suitable for cohesive soils. Requires clay components in the soil that will react with lime. Organic materials will retard reactions.
Blended slow setting binders such as slag/lime, fly ash/ lime and slag/lime/ fly ash blends.	Lime and pozzolan modifies particle size distribution and develops cementitious bonds.	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability. Generally reduces shrinkage cracking problems.	As for cement stabilisation, can be used where soils are not reactive to lime.
Bitumen (including foamed and bitumen emulsion)	Agglomeration of fine particles	Decreases permeability and improves cohesive strength.	Applicable to granular low cohesion, low plasticity materials.
Granular Material	Mixing two or more materials to achieve planned particle size distribution.	Some changes to soil strength, permeability, volume stability and compactibility. Materials remain granular.	Poorly graded soils, granular soils with a deficiency in some size(s) of the particle size distribution.
Chemicals	Agglomeration of fine particles and/or chemical bonding. (See trade literature)	Typically increased dry strength, changes in permeability and volume stability.	Typically poorly graded soils.

This input information should be combined with usual site investigation and testing for purposes of design and construction. Auststab have provided a summarising flowchart to effectively describe this process as shown in Figure 2.21:

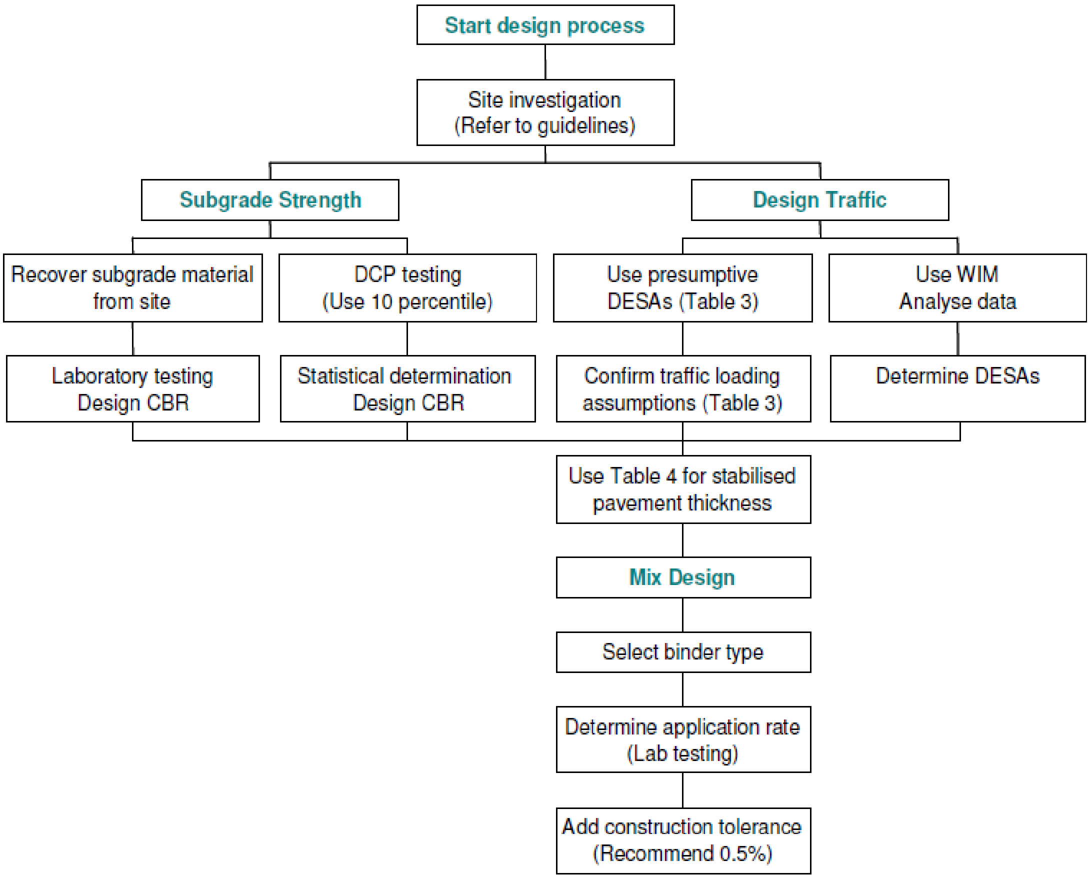


Figure 2.21: Design procedure for stabilised design of pavement base layers (Auststab, 2011)

2.8.2. Overview of Common Stabilisers

2.8.2.1. Mechanical

Often material meeting all design parameters may not be available locally, but this issue may be overcome by mixing two select samples (Henning et al, 2008). In this case blending of the materials is required which can be done either in the borrow pit or along the alignment, which is usually the most effective method. Regardless of which method is chosen it is important to control the quantity of water added preferably within the borrow pit (Paige-Green, 1990). For the sake of practicality the following should be observed when considering the utilisation of mechanical stabilisation (Henning et al, 2008):

- Avoid complicated ratios of mixing that construction equipment will not be able to achieve.
- Correction of grading below 0.075 mm is not feasible due to difficulties in obtaining such materials in the field.
- Extensive quality assurance is required to ensure adequate mixing.

Note that granular modification (mechanical stabilisation) is often required in order to meet an optimal grading prior to mixing in other stabilisers (Vorobieff et al, 2009).

2.8.2.2. Cementitious

Cementitious stabilisers are actually a class of materials extending beyond variants of portland cement, rather the term is used to describe materials whose addition results in chemical reactions (Wilmot, 1994). Each of the three common classes is discussed in the following. In all cases a major consideration is the setting or curing time of the resultant mix, which requires a clear distinction to be made. ‘Workability’ refers to the mixture of host and stabiliser, whereas ‘initial set’ is a function of the additive alone (Wilmot, 1994).

2.8.2.3. Cement

Portland cement comprises calcium-silicates and calcium-aluminates that hydrate when subject to the addition of water forming cementing compounds of calcium-silicate-hydrate and calcium hydroxide (lime) (Little et al, 2000). Working time of cement-treated soils is typically low, however mixtures containing equal amounts of cement and slag potentially have times of up to three hours, with other blends including higher fly ash/slag could reaching 8 hours (Wilmot, 1994). It is commonly accepted that these two components do result in improved working times compared to cement alone (White et al, 2010). Application rates, for mix design, can be determined by UCS testing with the rate producing a result of 1.5 MPa after 28 days curing often being implemented (Vorobieff et al, 2009). Materials that may be suitable for this treatment range a great deal, especially if blending with other stabilisers is an option and so selection and specification of optimum gradings should be determined through laboratory investigation for each individual project. However as a broad guide for addition without blending, a material with 100 % passing the 45 mm sieve and 60 % passing the 4.75 mm sieve is recommended (Little et al, 2000).

Benefits of the addition of Portland cement can be variable, indeed it is an incredibly flexible additive especially when blended with lime, fly ash or even both. Design is ultimately dictated by the desired effect. The following are noted possible effects (Little et al, 2000):

- Reducing the Plasticity Index.
- Increasing the Shrinkage Limit.
- Reducing volume change of the soil.
- Reducing clay/silt sized particles.
- Improved strength and resilient modulus.

Figure 2.22 has been produced by Austroads (2002) and is included to provide some guidance on the design process for cement modified pavements:

Some note needs to be made with regard to cracking induced in any cement treated layer as this can aid performance if used in the form of controlled micro-cracking. Alternatively the negative effect of utilising a rate of cement too high is an overly-stiff layer that results in shrinkage cracking. This can also be a function of soil, traffic, degree of compaction, curing, temperature and moisture changes (Adaska and Luhr, 2004). Hence control of pre and post-construction is highly important if a high percentage of cement is employed (that producing unconfined compressive strengths exceeding 1.5 MPa), these pavements are especially susceptible to drying shrinkage in the stabilised layer. Testing for an indication of potential of shrinkage cracking can be done via the linear shrinkage test, with a desirable upper limit being 1.5 % (Foley, 2002). Interesting to note is that cement addition generally will decrease the drying shrinkage of active soils (Adaska and Luhr, 2004) and therefore in some cases it is possible improvements are actually possible when compared to untreated soils. The most commonly prescribed method of eliminating adverse shrinkage cracking is to employ induced micro-cracking in a controlled manner. This can be completed by the passing of a vibratory roller during curing (24 to 48 hours after final compaction) to induce many hairline or ‘micro’ cracks that then are somewhat healed during further cement hydration (Adaska and Luhr, 2004) and therefore the overall structural performance of the pavement is not jeopardized. The result is a pavement layer relieved of shrinkage stress in early stages of curing.

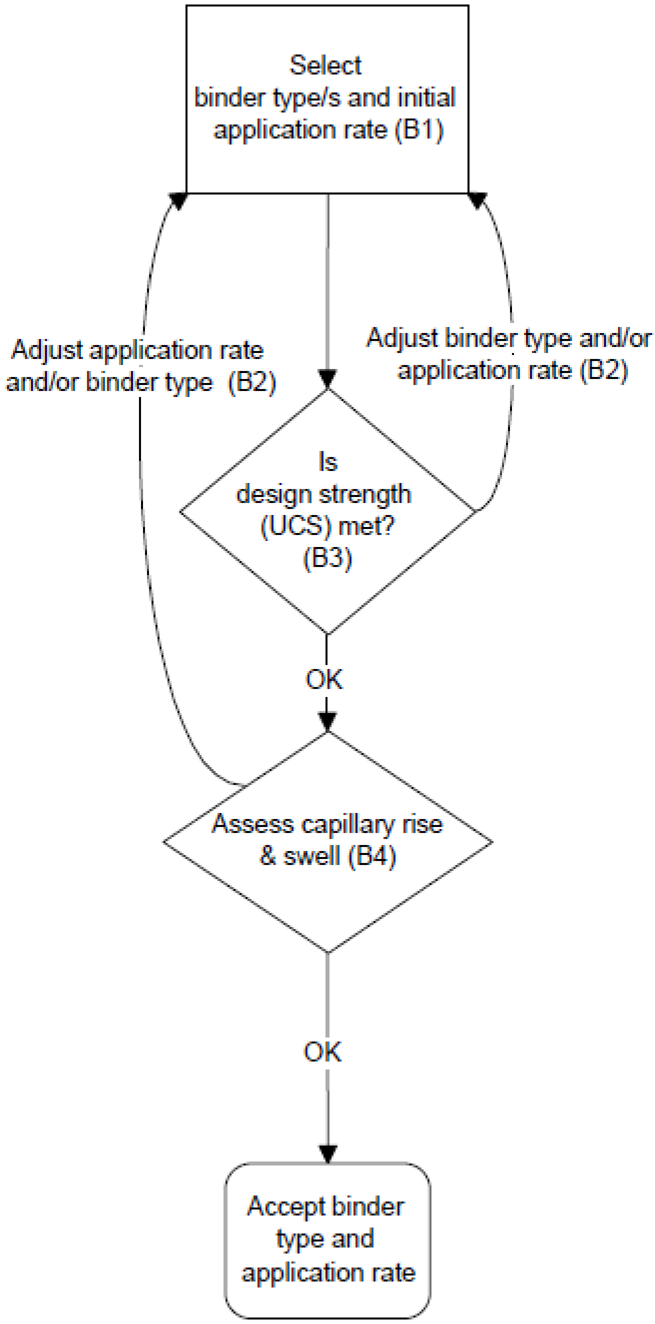


Figure 2.22: Flowchart for design of cementitious stabilised pavements (Austroads, 2002)

2.8.2.4. Lime

Soil-lime reactions are a relatively complex and sustained slow pozzolanic reaction between lime (calcium hydroxide) and soil silica and alumina which are released in a high pH environment (Little et al, 2000). Note that these reactions are dependent on time and temperature and as such should be considered and even exploited in mix design. In order to increase reaction time it is common to utilise quicklime which has a different content of calcium hydroxide. It is common that for safety reasons laboratory testing is completed with normal lime with quicklime being utilised in practice to decrease the amount of material transport and handling required, in this case the application rate determined should be factored by 0.76 (White et al, 2010).

The effects of lime addition are usually for modification of consistency limits rather than for control of volumetric changes and strength gain, which can often be manipulated by utilising blends. This means that lime is the most common treatment of materials with high or active clay contents (Vorobieff et al, 2009). Note there is no limit placed on the plasticity index of the parent material. Individual assessment should be used through lime demand and CBR testing, and if no strength gain occurs and/or the reduction in volumetric instability is minimal then the use of lime should be reassessed (Austroads, 2002). Common effects of use are provided by Auststab (2011):

- Increase sub-grade stiffness.
- Reduce Plasticity Index of in-situ pavement material.
- Enhance volumetric stability.
- Modify sub-base layers to improve stiffness of pavement.

These effects can be explained by Table 2.16Figure 2.16 noted effects of lime addition to soil samples:

Table 2.16: General properties of lime treated soils (Auststab, 2008)

Property	Description
Plasticity	The plasticity index decreases, as much as four times in some circumstances. This is due to the liquid limit decreasing and the plastic limit increasing.
Moisture density relationship	The result of immediate reactions between lime and the clay soil is a substantial change in the moisture density relationship. The moisture density changes reflect the new nature of the soil and are evidence of the physical property changes occurring in the soil upon lime treatment.
Swell potential	Soil swell potential and swelling pressures are normally significantly reduced by lime treatment.
Drying	Lime (particularly quicklime) aids the immediate drying of wet clay soils. This allows compaction to proceed more quickly.
Strength properties	Both the Unconfined Compressive Strength (UCS) and CBR increase considerably with the addition of lime. These values can be further increased by a follow up treatment of cement after the initial lime treatment. Experience has shown increases of CBR's from 3 up to 20 with lime only treatment and as high as CBR 50 with a follow up cement treatment. This gain in strength is often used in the design of pavements in order to reduce the depth of pavement material required.
Water resistance	The lime stabilised layer forms a water resistant barrier by impeding penetration of moisture from above and below. Thus, the layer becomes a working platform shedding water and allowing construction to proceed unaffected by weather. Experience in Victoria is that a second treatment with cement is required to achieve long-term waterproofing of the clay-stabilised layer unless the stabilised layer is covered by another pavement layer as quickly as possible.

Additional benefits include a noted decrease in roughness when added to cement based additives in trials completed in New South Wales (Wilmot, 1994). A unique effect that could be of a large benefit for unsealed pavements is the ‘mellowing’ of treated soil, in which extended pozzolanic reactions are able to repair small cracks in the soil under the action of water (Little et al, 2000). Note should be made that lime content of stabilised pavements are recommended to be increased by 2 % at intersections, steep longitudinal grades or curved alignments (Vorobieff et al, 2009).

Common testing of lime stabilised soils is recommended to commence with determining optimum lime content through (Vorobieff et al, 2009):

- Determination of Available Lime Index (ALI) of the lime.
- Lime demand test.

Testing should then focus on the physical effects of the lime addition (Vorobieff et al, 2009). Austroads recommend as an initial guide that best performance is generally with lime additions two to four percent greater than that indicated by the lime demand test (Austroads, 2002):

- Determination of CBR for sub-grade materials.
- UCS for pavement materials.
- Capillary rise and swell potential where wet sub-grades and/or sensitive soils are evident.

As with other cementitious stabilisers an interim mix design process has been developed for lime stabilisation by Austroads as depicted in Figure 2.23:

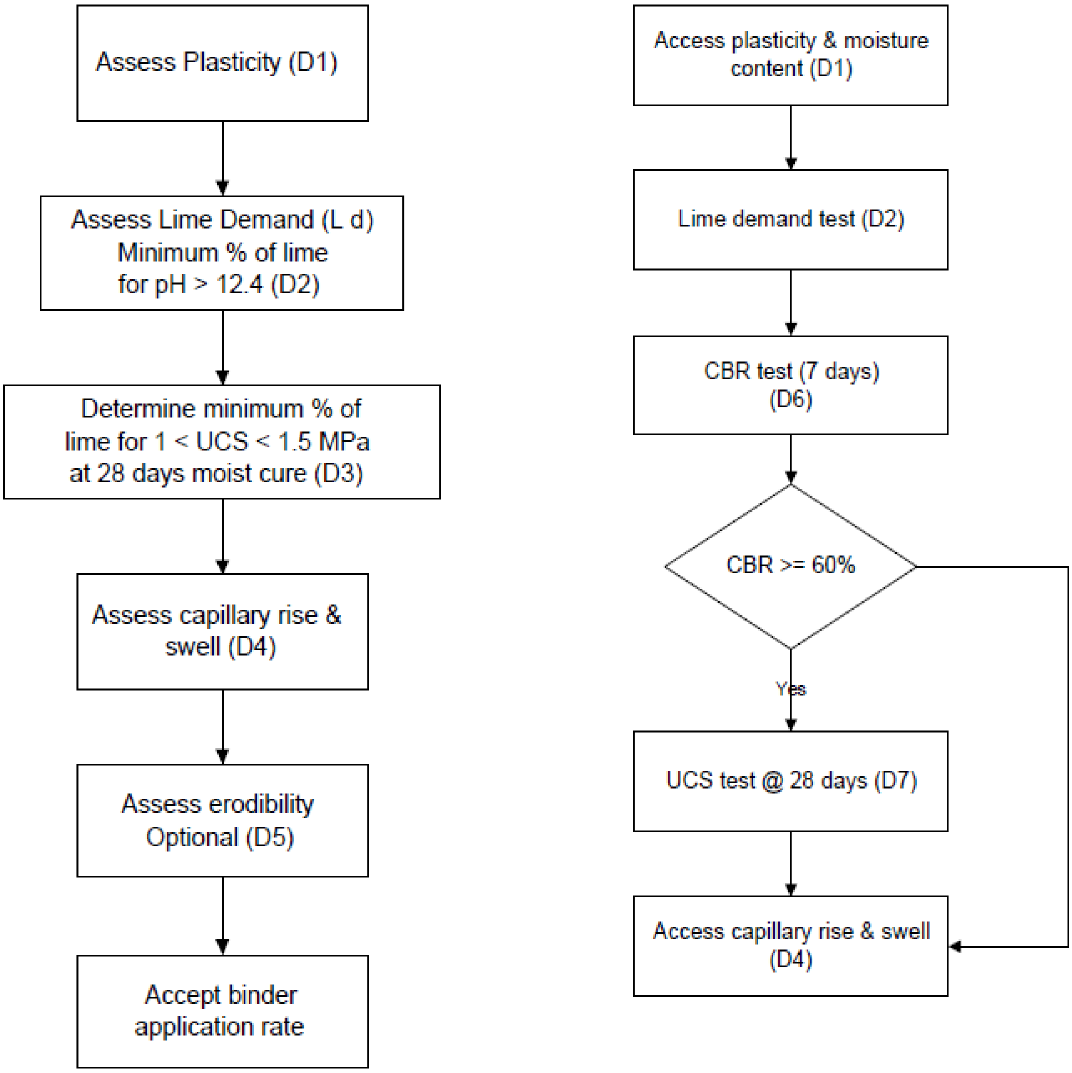


Figure 2.23: Interim mix design process for lime stabilised materials (Austroads, 2002)

2.8.2.5. Fly Ash

Fly ash is a by-product of burning coal in power generating stations and thus provides a viable recycling method for what else would be a bulk waste and land-filled (Edil and Benson, 2007). Since it is not manufactured but rather recycled large is a large inherent environmental benefit associated with its use due to a vastly lower net energy consumption (and greenhouse gas emissions) when compared with manufactured cement products (Mehta, 2004), which also means there are also possible cost savings. Although often fly ash is used within blended stabilisers so the benefit may not be quite as large as might be indicated initially. This is particularly common when only

Class F non-cementitious fly ash is available, where it is beneficial to add cement or lime to act as an activator, with previous research showing mixing such material with lime can be engineered for satisfactory long term performance with a cost saving of up to 50 % when compared to Portland cement stabilisation (Beeghly, 2003). There exist two types of fly ash to consider that being non- and self-cementing, which are derived from burning of bituminous and sub-bituminous coals respectively (Edil et al, 2006). Self-cementing or Class C fly ashes came about with the passing of the Clean Air Act in the USA in the 1970's which lead to the burning of lower-sulphur coal resulting in concentrations of 20 to 30 % of calcium oxide which provides for self-cementing (Little et al, 2000).

Materials suitable for fly-ash stabilisation depend on the type of fly ash used and any other additives. For example lower plasticity soils (PI<20) typically respond well to cement-fly ash stabilisation. However it has often been found that lime-fly ash mixes have shown to perform significantly better than lime alone for medium plasticity soils (10<PI<20) due to the pozzolanic reaction of lime and fly ash (Beeghly, 2003). Thus for materials with high fines content (25-75% passing 74 micron sieve) but only a medium plasticity due to non-expansive clay or high silty fines lime-fly ash may present significant advantages over lime addition in performance and cost (Beeghly, 2003).

In terms of performance the addition of fly ash can have many varied benefits that are similar to the effects of the other cementitious stabilisers, hence addition is focussed on providing a sound base (Little et al, 2000):

- CBR increases with fly ash content (Edil et al, 2006).
- Fine grained soils, especially those compacted wet of optimum typically have higher resilient modulus (Edil et al, 2006).

It is noteworthy that fly ash is commonly used as a second additive due to its financial and sustainability advantages over other stabilisers and as such the effect it has on the soil is generally reflective of the primary additive.

Mix design involving fly ash is a somewhat complex process that is often developed through vast experience. It is common to use Class F fly ash with lime in proportions of 1:2 to 1:5, lime:fly ash (in terms of total material often 3 to 5 % lime and 8 to 15 % fly ash) with 0.5 to 1.5 % portland cement being mixed in for early strength gain (Little et al, 2000). This mix is commonly referred to as a PSM (Pozzolanic Stabilised Mixture). An inherent benefit of such a mix is autogenous healing – the ability for un-reacted lime to close any cracks that open, this can be manipulated in mix design by aiming for slow pozzolanic reactions. Class C, self-cementing fly ash is commonly used in concentrations of 20 to 30 % with effects being similar to portland cement (Little et al, 2000).

In terms of construction two considerations must be well understood as explained by Little et al (2000):

- The effect of compaction delay time, in higher layers of pavement final compaction should be completed within an hour of hydration beginning.
- Maximum strength is typically achieved with moisture contents typically 7 to 8 % below that producing maximum density, with strength losses of 50 % observed in the past with moisture contents exceeding this recommended limit by 4 to 6 %.

2.8.3. Ground Granulated Blast Furnace Slag

Blast furnace slag is a by-product of the iron and steel making process and hence can provide a usage for one of the unintended outputs of the iron production process (Australasian Slag association, 2002). It is a result of the process undertaken within a blast furnace, which sees slag floating on the top of molten iron at the completion of the process, which is shown illustratively below. This can then be extracted and treated in many ways, such as allowed to slowly cool to produce rock or aggregate slag or cooled rapidly with large amounts of air to create a powdered product. It is this material that is then crushed or ground to produce a self-cementing product that can be utilised as a pavement stabiliser commonly referred to in short as ‘slag’ (Australasian Slag association, 2002)

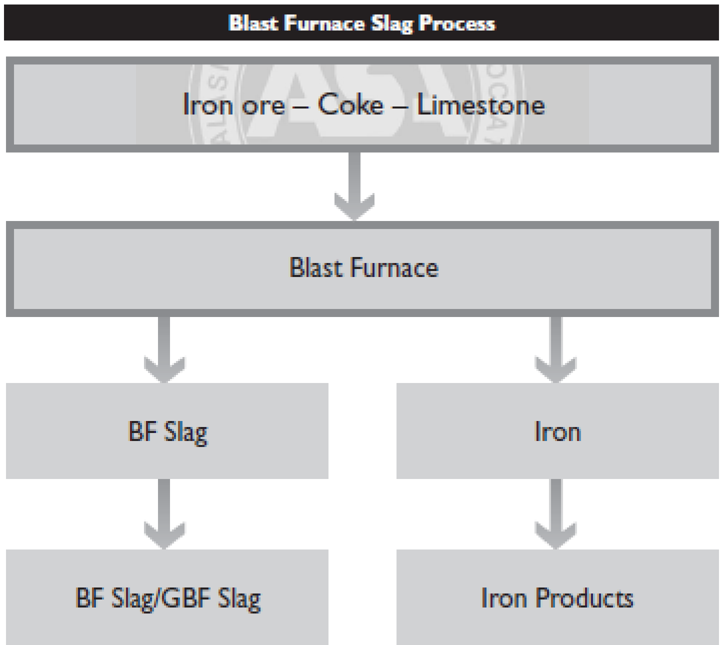


Figure 2.24: Typical blast furnace slag production process (Australasian Slag Association, 2002)

The benefits of slag are commonly expressed in comparison to Portland cement as a stabilising material. In summary the main differences are (Australasian Slag Association, 2002):

- Slower setting allowing reworking up to two days after initial mixing.
- Improved shrinkage properties, in particular the formation of much finer shrinkage cracks which do not adversely affect pavement structural performance.

Often slag is utilised as a binder with lime often used as an activator, or even in combination with lime and fly ash. It should also be noted that strength has been observed to change significantly with the fineness of the slag used, with large gains noted in a trial applying UCS testing to a material treated with three slags of varied fineness showing the finer the slag particles the stronger the resultant pavement material (Australasian Slag Association, 2002).

2.8.4. Fatigue Failure of Stabilised Pavement Layers

One possible adverse effect of stabilised pavements is that the induced rigidity can lead to fatigue behaviour and indeed failure inhibiting the effective working life, especially when layers fit into the stabilised and not modified classification (see Table 2.14 above). Theyse et al (1996) provided a neat summation of the behaviour stages of a cement treated pavement layer:

- Pre-cracked phase.
- Effective fatigue life phase.
- Equivalent granular phase.

These stages are shown in Figure 2.25 below. In considering the fatigue life and performance of treated layers it becomes clear that an additional consideration for the pavement engineer has arisen.

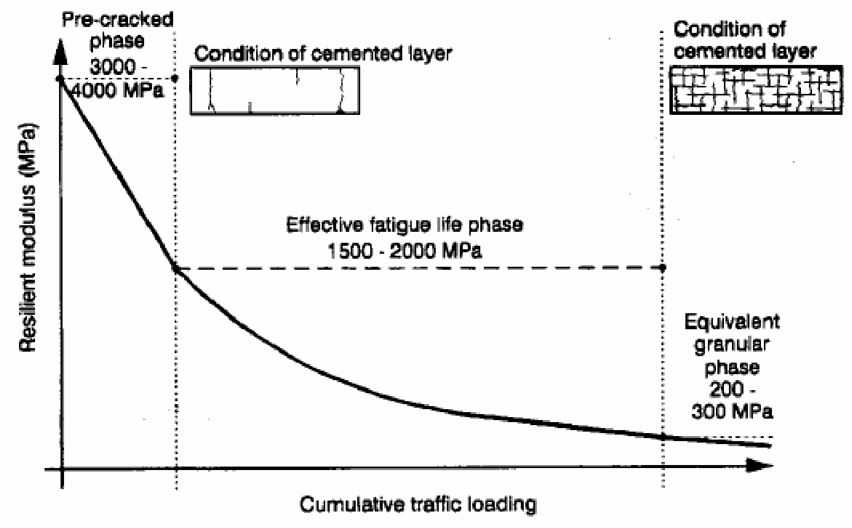


Figure 2.25: Long term behaviour of lightly cemented material (Theyse et al, 1996)

Firstly, it is useful to consider the relative magnitude of the loading to be placed on the treated pavement. Empirically this can be achieved by looking at the following model provided by Austroads (2011). Here ‘L’ represents the total load of the axle group and 'n' the number of axles in the group. This is a significant consideration of fatigue design as it is for general pavement design. Generally very large loads will lead to high strains induced in the pavement and a much reduced life.

Equation 2.5: Austroads method of calculating ESA's for an axle group (Austroads, 2011)

$$1\ \text{ESA} = n \times \left(\frac{L/n}{80} \right)^4$$

There have been many studies attempting to define a model for predicting allowable repetitions for cemented pavement materials. That of Equation 2.6 is currently the most developed and best suited to the application of wearing course stabilisation.

Equation 2.6: Austroads fatigue relationship (Austroads, 2004)

$$N = RF \left[\frac{\left(\frac{113000}{E^{0.804}} \right) + 191}{\varepsilon_t} \right]^{12}$$

Where:

N is the number of allowable repetitions within effective fatigue life phase.

RF is the reliability factor (=2 for 90% reliability, =1 for 95% reliability, =0.5 for 97.5% reliability).

E is the Elastic Modulus of the cemented material.

ε_t is the induced strain in the stabilised layer.

For the purposes of estimating the Elastic Modulus of a cemented natural gravel, allowing pavement life predictions using Equation 2.7, the following equation has been included (Vorobieff, 1998):

Equation 2.7: Estimation of resilient modulus from UCS (Sukumaran, 2002)

$$E = 2240.\mathrm{UCS}^{0.88} + 1100$$

Another model that may prove interesting for comparison sake is that noted by Austroads for materials in layers with less than 100 mm cover. Failure is defined as one meter of cracking present within one square meter on over 10 % of trafficked area:

Equation 2.8: Fatigue relationship for cemented layer with less than 100 mm cover (Austroads, 2008)

$$N = \left[\frac{28{,}400}{\varepsilon_t E^{0.41}}\right]^{7.1}$$

A recent study conducted by Austroads has found that single axle groups cause less damage than that of multiple axle groups and therefore load shape has a great influence on fatigue performance (Austroads, 2011). Within this study initial peak induced strains of 90 to 100 microstrain resulted in very short fatigue lives (approximately 10 000 cycles) and not surprisingly had little reliance on load shape (Austroads, 2011). This particular research involved cemented materials from existing un-trafficked pavements with flexural modulus of 4 000 to 30 000 MPa, with majority falling between 15 000 and 22 000 MPa thus the vast majority were bound materials. This however does show that heavy axle loads have the potential to vastly reduce the fatigue life of cemented or even simple stabilised pavement layers if a degree of rigidity is introduced which as described above is the primary reason for their use.

2.8.5. Non-Cementitious

2.8.5.1. Polymers

The action of polymer stabilisers is quite different to cementitious additives in that they do not act to simply provide binding of fine particles with the final result an increase in stiffness. Rather they act to provide internal and external 'waterproofing' of the material through creation of a hydrophobic soil matrix between larger particles reducing permeability and limiting water ingress (Rodway, 2001). This acts to prevent fine particles becoming 'greasy' with addition of water and thereby preventing permanent deformation. The Australian Stabilisation Industry Association (Auststab) define dry powdered polymers (the form products are commonly procured in) as 'a dry powdered road stabilising binder consisting of an insoluble polymer thermally bound to a very fine carrier such as fly ash' and is provided to avoid confusion with water-soluble binders sometimes called polymers (Lacey, 2004). Fly-ash is typically used as the carrier of fine material, which is often then also blended with hydrated lime which does not react with the blend but rather acts to flocculate clay particles to prepare them for adhesion to the polymer (Rodway, 2001). In turn it is therefore inappropriate to directly compare results of polymer stabilised materials with that of cementitious additives that provide a binding action through pozzolanic reactions (Auststab, 2003). Polymers therefore provide for a truly flexible pavement that is both less sensitive to plastic deformation by the addition of moisture but also protects the sub-grade from ingress and therefore further pavement rutting (Lacey, 2004). The latter effect is due to the decreased capillary rise or permeability agglomeration of fine particles (Wilmot, 1994).

Therefore the use of polymers is best suited to situations (Auststab, 2003):

- In flood prone areas.
- Where sub-grades have a high Plasticity Index (PI).
- In irrigated areas (including levee banks).
- Where there is a scarcity of local gravels and the existing road is a thin layer of base course.

Another potentially valuable property of pavements utilising such stabilisation is the control of severe dry back in periods of drought (Lacey, 2004). In such cases cracking is observed in the surrounding batters and even in shoulders (as in Figure 2.26) which are not stabilised but are adjacent to pavement sections that are.

Figure 2.26: Observed cracking in unstabilised shoulder adjacent to polymer stabilised pavement during drought period in New South Wales (Lacey, 2004)

Polymers can be used within mixes with hydrated lime, typically for heavy clays (PI>20%) with several different products available for gravels with differing composition with other additives (Lacey, 2004). An example of the effects of polymer stabilisation trialled with differing gravels of PI varying from 5 to 7 % is shown Table 2.17. For optimal performance it is also recommended that a minimum of 35 % of particles by mass pass the 2.36 mm sieve (Lacey, 2004), with an overall grading shown in Figure 2.27.

Table 2.17: Example of effects of polymer stabiliser (Polyroad PR21L) applied to gravels of moderate PI (Rodway, 2001)

Material	Additive	PI	AS 1289.5.2.1 & 1289.6.1.1*				AS 1141.53**		
			Density Ratio (% Modified)	OMC (%)	CBR (%)	Swell (%)	Capillary Rise (%)	Water Absorption (%)	Swell (%)
Gravelly Sand	Raw	7	98	6	180	0.0	100	9	13
	1.5% Polyroad#		98	6.5	230	0.0	32	2	0
Crushed River Gravel	Raw	5	98	5.5	150	0.0	100	6	2
	1.5% Polyroad		98	5.5	170	0.0	18	2	0
Soft Sandstone	Raw	5	98	8	110	0.0	100	16	19
	1.5% Polyroad		98	8.5	190	-0.1	74	8	2
Ripped & Crushed Mudstone/ Sandstone	Raw	6	98	10	60	1.1	100	20	11
	1.5% Polyroad		98	11	100	0.1	86	11	2

Samples cured for 3 days after the addition of Polyroad prior to compaction. Compacted samples cured for a further 3 days prior to 4 day soak.

* AS 1289.5.2.1 – Dry density/moisture content relation of a soil – Modified compactive effort
AS 1289.6.1.1 – CBR for a remoulded specimen

** AS 1141.53 – Absorption, swell and capillary rise of compacted materials.

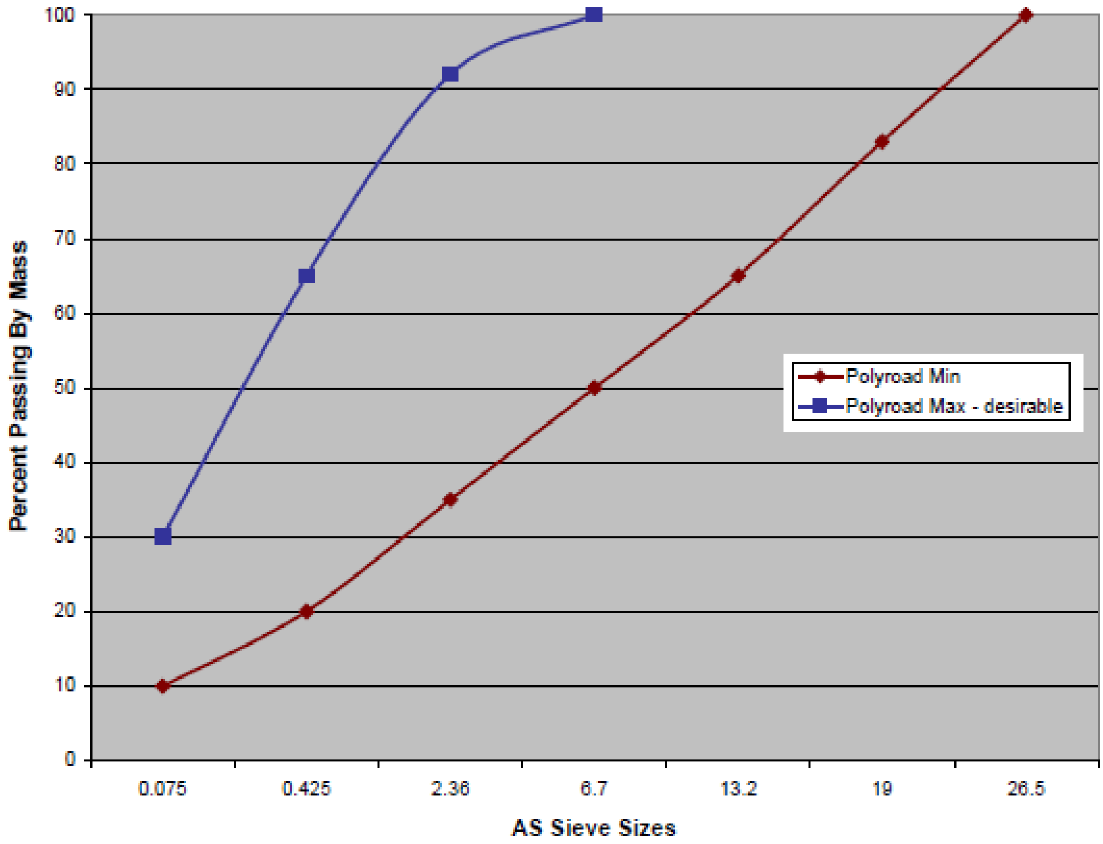


Figure 2.27: Recommended grading of parent material for application of insoluble dry powdered polymer stabilisers (Lacey, 2006)

Application rates for polymers is typically lower than for the production of bound pavements using a cementitious binder, with 1% by weight being suitable to produce desired waterproofing effects (Rodway, 2001). Additionally a similar amount of fly-ash is generally needed, but should be assessed for each particular project considering application.

2.8.5.2. Chemical

These tend to refer to products that are the same or very similar to that referred to in this project as a dust suppressant or palliative, thus see Section 2.7 above dedicated to this topic.

2.8.6. Testing of Stabilised Materials

Once an initial sampling and testing regime has been completed laboratory trials of one or more stabiliser can be trialled in order to determine optimal application rates (Austroads, 2002). The testing outlined in Table 2.18 should be used as a guide to what testing is likely to be of use in the selection and design process.

Table 2.18: Recommended testing regime for stabilised material for the purpose of selection and design with any stabiliser (Vorobieff et al, 2006)

Test	Test Method
Mechanical	
CBR	AS 1289.6.1.1
Unconfined compressive strength	AS 1141.51
Resilient modulus & deformation	AS 2891.13.1
Moisture	
Capillary rise and swell	AS 1141.53
Permeability	AS 1289.6.7.1
Vertical saturation	TSA[B]
Durability	
Erosion[A]	T133[C]

Notes:
A Not common test
B. Test method in draft format and not published.
C. RTA Test method – erosion by brushing (RTA, 2001a)

2.9. Materials in Western Australia

There exists a large accumulation of knowledge relating to naturally occurring gravels in Western Australia due to the high percentage of pavements that have been constructed with use of such materials within flexible pavement designs. This section is included in an attempt to draw on such knowledge to assist in the understanding of the impact material type and properties have on pavement performance. Focus has been placed on the Pilbara region, as the location of the case study included within this investigation.

2.9.1. Favourable Unbound Granular Material Properties

The characteristics that make a naturally occurring gravel favourable for use are those satisfying the above specifications for use in pavements. However some additional comment can be made to assist in field evaluation.

Apart from possessing a suitable particle size distribution it is also important to quantify the amount of fines within a prospective soil. Some degree of clay is favourable, due to the cohesion it provides, however a large amount are prohibitive due to such materials rapidly losing strength under the application of water (Butkus, 2001). Silt is also undesirable as it allows positive pore water pressure to develop at low moisture contents by way of the predominance of small voids, which causes a reduction in the frictional component of the soils shear strength which is obviously undesirable (Butkus, 2001). Particles should also be durable to avoid break-down and also ideally be angular allowing significant amounts of particle interlock (Butkus, 2001).

2.9.2. Occurrence and Investigation of Natural Gravels

Specific guidance for each type of common naturally occurring pavement materials is provided in the proceeding discussion, this section is focussed on outlining the specific steps that may assist in seeking and sourcing natural gravels in Western Australia. In broad terms suitable gravels often occur at flanks and low ridges of capped hills, with rocky ridges usually being unsuitable along with hollows and low ground due to the depositing of fine, highly plastic material over geologic time (Butkus, 2001). Note that the development of natural gravels is often the product of pedogenesis where massive or granular materials are formed by precipitation from groundwater of minerals which have been leached from in-situ soils in areas of high rainfall and evaporation rates (Butkus, 2003). This explanation most commonly relates to the development of laterites (most commonly used in Western Australia being ferricretes), although many other similar materials have been utilised in Western Australia within road pavements, as is outlined below.

Searching for gravels is best done firstly on a large scale from within a vehicle capable of handling the terrain. In the case of the North-West of Western Australia, the presence of Minerichie scrub often indicates gravels (Butkus, 2001). Once some potential deposits have been identified they should be investigated by way of ground breaking investigation, consisting of test pits dug to a suspected depth of the material or to refusal (Butkus, 2001), common practice is to begin such an investigation in a grid pattern with approximately 100 meter spacing. Early indications of suitability for use is mainly experience based, however a good grading can often be noted visually with particles not being able to be broken with firm hand pressure (one should still investigate the nature of fines before making note). This can be done by wetting a fine fraction (particles approximately less than 1 mm), this should feel gritty when rubbed through fingers and a presence of clay should discolour the hands, however if the material sticks or feels greasy the clay content will be too great (Butkus, 2001).

Once deposits have been identified they should be sampled and tested to the specifications provided earlier.

2.9.3. Commonly Occurring Natural Gravels in Western Australia and their Use in Pavements

The following is provided as a guide to the common gravel deposits that may occur adjacent or within Pilbara Iron Ore mines, the location of the subject case study within the associated research to this paper. Interpretation and use of the information contained within should be approached with caution and note that local experience is invaluable when deciding the use of natural gravels, especially when attempting to summarise an area as large as the Pilbara or indeed as varied and as vast as Western Australia.

2.9.3.1. Lateritic Gravels

Laterites are common in many areas within Western Australia and are in fact the most successfully employed of all natural soils. It is worth noting that the term laterite does not have a settled definition, however most uses indicate that the soil will be rich in aluminium and iron oxides (sesquioxides) with content exceeding 10 % in most cases in Western Australia (Butkus, 2003). Occurrence of laterites is as Figure 2.28 indicates below, however it is worth noting that those in the South of the state often lack cohesion and therefore tensile strength and those in the North are often found to have large plastic fines content (Butkus, 2001).

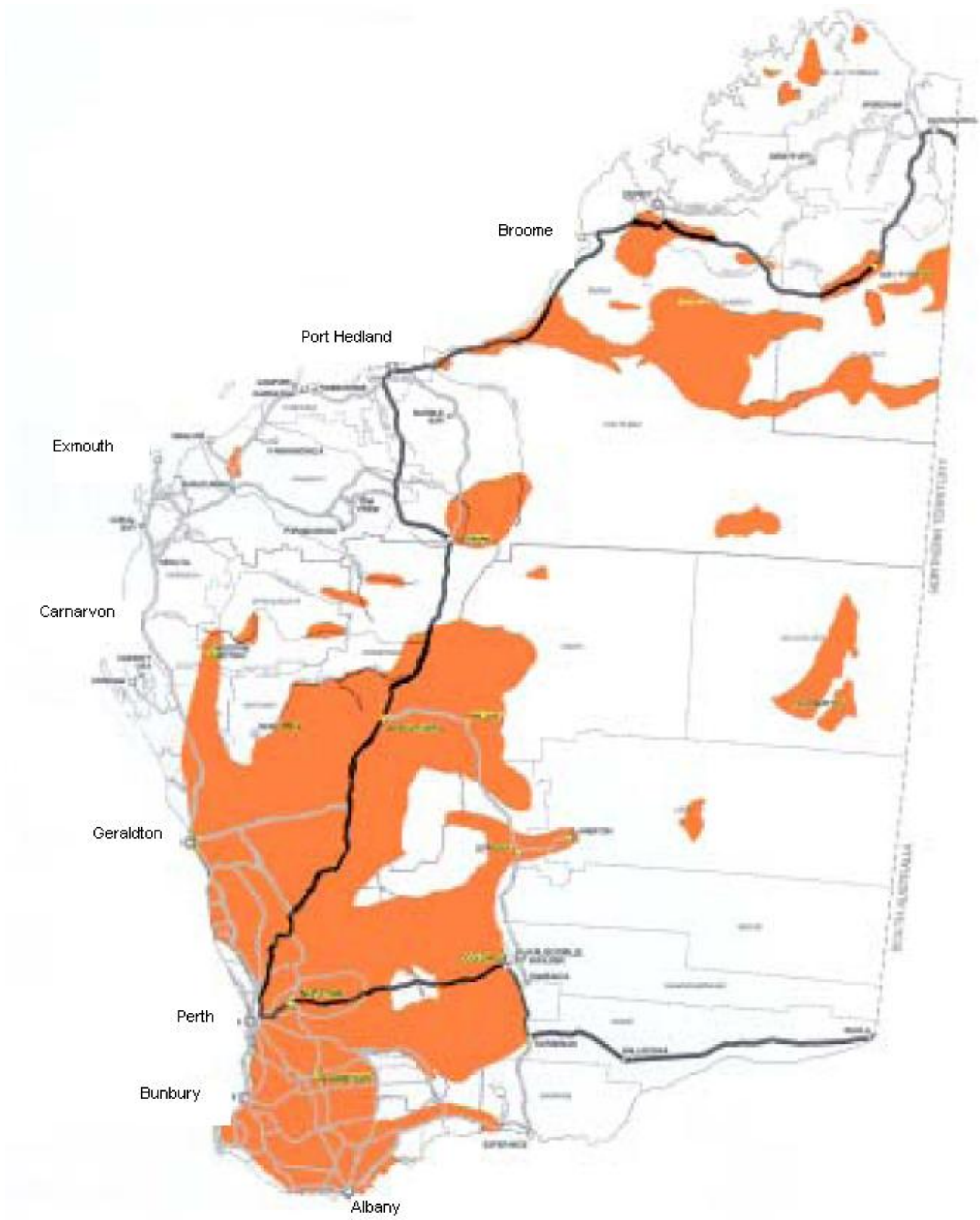


Figure 2.28: Occurrence of Lateritic gravels around Western Australia (Butkus, 2003)

The appearance of common laterites can range from colours from very pale yellow to dark brown (Butkus, 2001). A common coarse sample is shown in Figure 2.29.

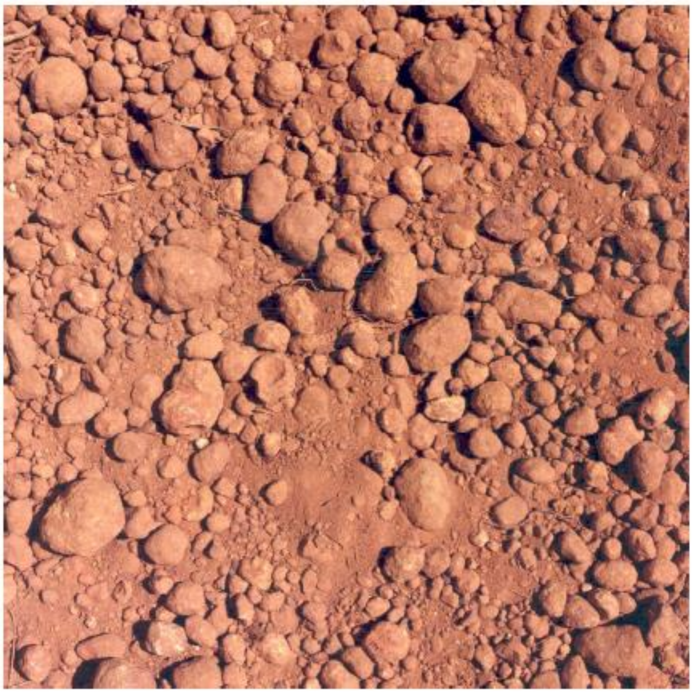


Figure 2.29: Common appearance of a coarse laterite in the Northern portion of Western Australia (Butkus, 2001)

Selection criteria for use of laterites in pavements is largely based on the nature of the climate and also the content and activity of fines. Advice on the use for different climate types present within Western Australia is available in Butkus et al, 2003. Table 2.19 is taken from the same reference that outlines the use of laterites in heavy duty pavements such as that of haul roads. Note that materials fitting this description may possibly present less cohesion than what has been considered ideal for use in a wearing course layer elsewhere. However this assessment is based on observation of the linear shrinkage and liquid limit specifications utilised within this specification in preference to the plasticity index, which is adopted by the other specifications discussed within this paper.

Table 2.19: Recommended selection criteria for laterite for use in heavy duty pavements (Butkus, 2003)

Lateritic Gravels for Heavy Duty Pavements		Target Grading(2)	Range
	Sieve Size mm	% Passing	% Passing
Grading (3)	37.5	100	100
	19.0	80	72-100
	9.50	57	50-78
	4.75	43	36-58
	2.36	31	25-44
	1.18	23	18-35
	0.600	18	13-28
	0.425	15	11-25
	0.300	13	9-22
	0.150	9	6-17
	0.075	7	4-13
	0.0135	4	2-9
Liquid Limit (4) %		≤25	
Linear Shrinkage %		≤2	
MDCS (5) kPa		≥2300	
Soaked CBR (6) %		≥80	
Dust Ratio(7)		0.3-0.7	
Dryback (8)%		≤85	

Notes:

(1) Selection criteria applies to basecourse roads with a thin bituminous surfacing with a 20 year design traffic loading of up to 1×10^7 ESA. For higher traffic loadings, specialist advice should be sought.

(2) The particle size distribution must conform as closely as possible to the target grading. Gap graded materials should not be used.

(3) Dry Sieving and Decantation, Test Method WA 115.1.

(4) Liquid Limit, (using the cone apparatus) and Linear Shrinkage tests on samples air-dried at 50°C.

(5) Maximum Dry Compressive Strength, Test Method WA 140.1

(6) California Bearing Ratio in accordance with Test Method WA 141.1. Specimen for soaking prepared at 96% of MDD and 100% of OMC

(7) Dust Ratio = $\frac{P_{0.075}}{P_{0.425}}$

(8) Basecourse should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing

2.9.3.2. Calcerous Gravels

Discussion of gravels composed of weathered sedimentary rock is very difficult and selection should be treated individually each time as such soils are variable across the state of Western Australia. Hence included in this paper is general discussion and a brief outline of calcrete type soils common in the Pilbara region, therefore most likely occurring near to the Iron Ore mines used as a case study within the associated research. Note once again the term ‘calcrete’ or ‘limestone’ is widely used interchangeably with other terms and does not have a settled scientific definition.

In general these soils exhibiting some degree of cementation by way of carbonate (Butkus, 2003). Strength varies greatly as there exists varied properties exhibited due to carbonate content. Occurrence is extremely wide-spread and varied owing to the wide ranging definition of the term, as Figure 2.30 would indicate.

Figure 2.30: Occurrence of Calcretes within Western Australia (Butkus, 2003)

Selection criteria are available in the previously recommended references, however they are largely based on the particular soil being used. Note that in broad terms selection criteria should include a requirement for dust ratio to lie between 0.3 and 0.7 for workability generally and low permeability in the case of nodular calcretes common in the Gascoyne region(Butkus, 2003), particularly favourable would be soils lying in the high portion of this measure for use in the specific application of a wearing course.

2.9.3.3. Scree Gravels

Scree gravels are formed by accumulation of material after undergoing mechanical weathering and gravitational transport on the slopes of hills (Butkus, 2001). In the context of the Pilbara they commonly occur higher on slopes than lateritic gravels. Suitability is largely dependent on parent rock, from which deposits are often not far removed, therefore it is unsurprising deposits often contain large amounts of oversize particles which need to be removed prior to use (Butkus, 2001). Note that parent rock such as ironstone, chert and quartz produce useful scree whereas igneous rocks tend to produce materials with high plasticity (Butkus, 2001). An example of the appearance of scree gravels present in the Pilbara region is presented in Figure 2.31, however note the gravels themselves are a function of the geologic process and are therefore as variable in appearance and mechanical properties as their parent rocks.

Figure 2.31: Scree gravel sampled from near Newman, in the Pilbara region of Western Australia (Butkus, 2001)

2.9.3.4. Cohesive Soils

Sand clays and collapsible red clayey/silty sands ('pindan') are common in Northern parts of Western Australia (Emery et al, 2010). They exhibit a degree of self-cementation upon drying-back and as such can provide sufficient strength for use in pavements if they are not exposed to a high degree of moisture (Emery et al, 2010). They are not typically suitable for use as a wearing course and thus little discussion is provided within. However it is worth noting that they may provide a convenient constituent to mixing or mechanical stabilisation due to the high degree of cohesion and ease of access (as surface soils) they present.

3. PROCEDURE

This project is comprised of two major elements ultimately facilitating recommendations for the provision of safe and efficient haul roads. Iron Ore mines operated by Rio Tinto in the Pilbara region of Western Australia provided the case study for practical testing. Firstly a literature study was conducted to identify current knowledge on the topic of functional design, construction and maintenance of unsealed pavements. The results of this study provided the impetus to sample materials from the above mentioned mines which were subjected to a laboratory testing regime that defined physical characteristics. This allowed a comparison of the suitability of various materials for the surface or wearing course layer for unsealed haul road pavements. A further desk top study was then conducted to classify the likely operating impact of various improvement techniques with those favoured being trialled in further laboratory testing. This regime is displayed visually in Figure 3.1. Finally the results from testing were used as inputs for modelling the construction, vehicle operating and maintenance costs in addition to an estimate of lost production due to poor wearing surface condition.

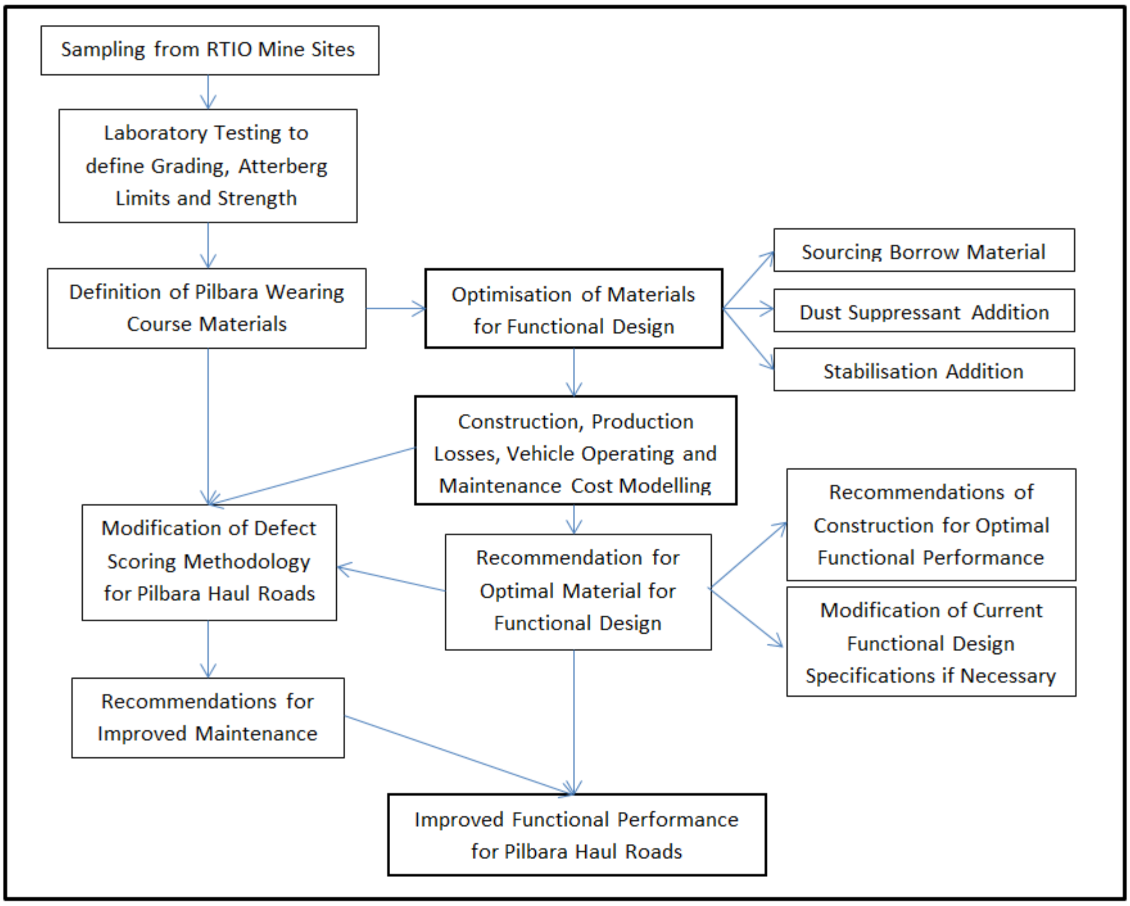


Figure 3.1: Flow chart of methodology utilised for this project to determine optimal practices for provision of functional haul roads

3.1. Laboratory testing

A laboratory testing methodology representative of typical pavement or bulk earthworks projects was chosen and refined to provide focus on the most significant properties for functional performance. The majority of tests are common index tests that should be able to be carried out by any geomechanics laboratory, which is important in allowing any recommendations made from the results of the tests to be applied in practice.

For all testing discussed herein the specification or criteria supplied should only be applied in the instance noted. Thus it is important to observe test results with the material type in mind and hence not attempt to extrapolate a specification derived for a particular material type to apply it arbitrarily.

3.1.1. Particle Size Distribution (PSD)

PSD testing is paramount to an understanding of the nature of the material being observed. Any significant changes to percentages of particles of certain sizes can greatly alter the performance of a pavement. It is necessary to complete additional index tests to effectively describe the physical nature of the material. Also note that limits are placed on the amount of particles allowed to exceed 19mm in the largest dimension (no greater than 20%, WA 132.2).

Procedures followed were that of MRWA standard WA 115.1, which is slightly less verbose but very similar to AS 1289.3.6.1-2009. Additional note was given to the process discussed in WA 115.1 with regard to sieving coarse grained materials. Although this discussion was duly noted, stringent compliance was not possible because of the practical restraints mentioned above. Hence washing of the material was not included in the process, instead only oven drying was completed prior to sieving. Observation and the final distributions of the materials transported to the laboratory show this process to be adequate.

For the sieving of larger particles a mechanical agitator was used as illustrated below, with an equivalent device used for the purpose of sieving fine particles.

Figure 3.2: Photograph of mechanical agitator used for large sieves

3.1.2. Compaction Testing

Accurate determination of the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of each material is important in facilitating further testing, due to strength testing typically being completed at a known percentage of, or at the maximum density. It indicates the total amount of water that should be added to a given volume of soil for the practice of compaction to achieve the maximum dry density.

The procedure used for this project is outlined by MRWA standard WA 132.1, which uses the following parameters of the standard version of the test:

Table 3.1: Parameters for Standard Compaction Testing (WA 132.1, MRWA)

Apparatus	Value	Working Tolerance
MOULD		
Individual internal diameter, mm	105.0	± 1.0
Average internal diameter, mm	105.0	± 0.5*
Height, mm	115.5	± 0.5*
Calculated volume, cm^3	1 000	± 15*
RAMMER		
Diameter, mm	50.0	± 0.4
Drop, mm	300.0	± 2.0†
Mass, kg	2.70	± 0.01†
Energy delivered per blow, J	7.94	± 0.08
Number of layers	3	-
Number of blows per layer	25	-
Energy input, kJ/m^3	596	± 14

* Either but not both of the tolerances may be exceeded provided that the appropriate tolerance of volume is not exceeded.
† Either but not both of the tolerances may be exceeded provided that the appropriate tolerance of energy per blow is not exceeded.

Note should be given to MRWA standard WA 132.2, which outlines testing of coarse materials. The definition of a coarse material in this standard is that having a total of more than 20% of material by weight retained on the 19 mm sieve. As discussed later no materials involved in this series of testing contain quite this quantity of large particles and hence do not require use of WA 132.2. If this had been the case a modified rammer and mould detailed in WA 132.3 would have been used, the parameters of which can be found in the document outlining the test. The modified rammer was used in the case of some of the CBR tests, in order to save time achieving the desired density inside the mould, testing of the density after testing verified there was no discernible difference experienced.

3.1.3. Liquid Limit

Quantifying the liquid limit provides an understanding of the moisture sensitivity of each material. This test is necessary as materials often may be observed to be very similar (even the texture felt in the hand of an observer) prior to testing, but are found to have vastly different modified physical properties once wetted. The liquid limit ultimately indicates when a soil begins acting with the characteristics of a liquid, as opposed to a solid (MRWA, 1987). The test utilised in this instance is one of an empirical nature, which quantifies the liquid limit as the moisture content which allows a standard cone to penetrate the soil 20 mm. Curing of treated materials is generally of high significance to test results , especially in this case as the soil is cured whilst sealed and for an arbitrary amount of time of 24 hours. Any duration could have been utilised as the aim was to gain only an indication of likely in service performance.

The procedure used for this testing followed MRWA standard WA 120.2. The cone used had the following parameters detailed in Figure 3.3.

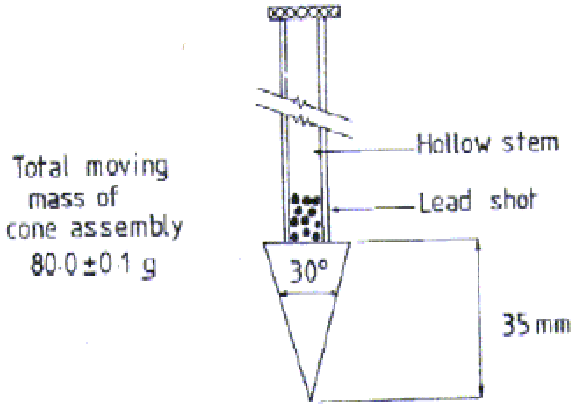


FIGURE 1

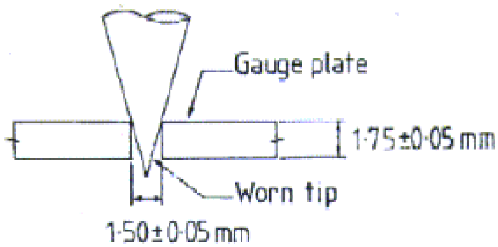


Figure 3.3: Cone Penetrometer dimensions (WA 120.2, MRWA)

The setup of the cone followed that detailed in WA 120.2, which in practice appears as in Figure 3.4.

Figure 3.4: Photograph of cone penetrometer setup

3.1.4. Linear Shrinkage

An understanding of the relative activity of a soil is paramount for selection as a construction material for use in a pavement. When completing Atterberg testing generally a plastic limit test is completed as well as liquid limit and linear shrinkage. However due to constraints on freight there not sufficient material to complete all these tests in some cases and thus the linear shrinkage results were used to estimate the plastic limit. This test gives an indication of the behaviour of the soil when moisture is added, as an active soil exhibits a large amount of volume change with the addition of water. Ultimately the shrinkage properties are determined from this test when the material contains a moisture content close to the liquid limit (considered giving cone penetration of 18 to 22 mm).

The testing procedure followed was that of WA 123.1, with modification for drying being completed via the samples being left out of the oven, in the mould overnight and then being oven dried at 105 degrees Celsius for at least one hour (which was considered adequate by the technical staff). The dried moulds for Mines A and B are shown below (from the top of shot downwards, ‘Alluvial’ from Mine B and ‘Detridal’, ‘Windrow 1’ and ‘Windrow 2’ from Mine A).

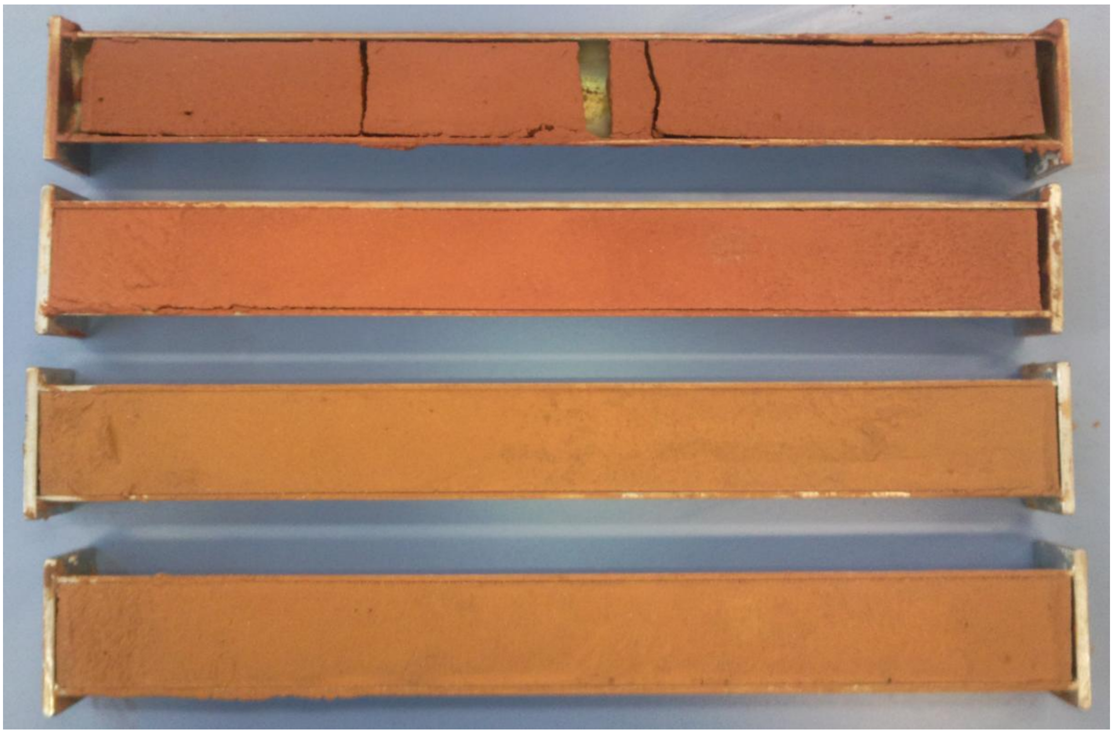


Figure 3.5: Photograph of dried moulds for linear shrinkage test

3.1.5. Plastic Limit

Similar to the liquid limit, the plastic limit provides an indication of the moisture content at which the soil transitions into behaving plastically. Hence in the context of a pavement layer, the plastic limit is the moisture content where a significant amount of plastic deformation is caused by the passing of a single vehicle. Testing is usually completed in conjunction with the liquid limit in order for calculation of the plasticity index, which allows material classification (within the unified soil classification system). Testing was completed in accordance with MRWA standard WA 121.1, with observation that three samples must be obtained within a moisture content variation of only 2 %.

3.1.6. Capillary Rise and Swell

This test indicates the capillary rise, swell and absorption properties of a soil. These measures are important to understand the behaviour of the material when water is added or indeed abundant. In this project these tests have been completed following Australian Standard 5101.5-2008. Originally it was hoped to complete permeability tests with the tri-axial apparatus, however the material was found to fail under only very low back-pressures meaning an equilibrium condition could not be reached and testing could not occur. As a result this test was implemented as an alternative to give an approximate indication the propensity of the soils to allow the passing of water through them and hence the likely protection they may offer base layers underneath the wearing coarse. Note that due to a lack of material being left available for testing and compaction along with 7-day curing having already been completed, 100 mm diameter by 200 mm tall cylinders that would usually be used for tri-axial testing were adopted. The other deviation to the standard testing procedure was the reduction of time in the water bath from 72 to 48 hours.

Although these changes have been made the relative effects between materials is expected to still be revealed by conducting the test. A degree of water absorption is a positive property of a wearing course material, provided it does not show excessive swell, as this means a denser surface that is less likely to produce dust emissions. The capillary rise provides an idea of the strength of the attraction for water the material has, which will indicate the likely penetration through the pavement. Similarly, a degree of penetration is a helpful characteristic, but if water is able to completely penetrate the wearing course the base may become saturated, which will likely result in shear failure of the pavement.

3.1.7. Strength Testing

Strength testing for this project was of three types, one relating to empirical pavement design, another for mechanistic pavement design and finally a static compressive strength measurement. The tests are vastly different in both procedure and the final results they produce, yet all provide an indication of pavement performance to some degree.

3.1.7.1. California Bearing Ratio (CBR)

This method of quantifying the strength of materials is relatively simple and somewhat representative of the shear strength of the sample. A constantly increasing force is applied and displacements of the centrally placed plunger are measured (MRWA, 2006). Results are used as an input to design cover curves derived by various authors for an array of vehicles. Note that current haul truck axle loads are not represented within the derived range.

The test method followed for CBR testing was MRWA standard WA 141.1 and AS 1289.6.1.1-1998. All specimens were soaked for approximately four days prior to penetration testing. Apparatus used was a Leonard-Farnell Manual Triaxial Test Machine, as shown in the photograph of the testing setup Figure 3.6.

Figure 3.6: Photograph of the apparatus and setup for CBR penetration testing

3.1.7.2. Resilient Modulus (M_R)

Resilient modulus is a measure used for the input of strength in mechanistic analyses of pavements. It is an approximation of the elastic modulus under repeated loadings, estimating the resilient response of the material. Testing is carried out in this project Is aimed at being completed at OMC/MDD conditions.

Repeated loading tri-axial testing is completed using a GCTS STX-300 Dynamic Stress-Path Soil Triaxial System run through a PCP Pressure Control Panel connected to a SCON-1500 Digital System Controller. The tests were completed in accordance with the procedure outlined in ‘AG:PT/T053 Determination of permanent deformation and resilient modulus characteristics of unbound granular materials under drained conditions’ (Austroads, 2007). A photograph of the apparatus completing one such test is presented in Figure 3.7.

Figure 3.7: Photograph of tri-axial apparatus used for resilient modulus testing

After all the test data was acquired, the commonly employed k-theta model (Jitsangiam et al, 2007) was employed to calculate the resilient modulus of each material. This technique determines a trend line in power form for the relationship of bulk stress against the axial modulus for each testing sequence. This function is then used to calculate the effective axial modulus (resilient modulus) at a central figure of bulk stress readings.

3.1.7.3. Unconfined Compressive Strength (UCS) Testing

The Unconfined Compressive Strength (UCS) test is a simple ultimate shear strength test for any (usually stabilised) soil. It is conducted with the same apparatus as the resilient modulus testing discussed above, however with a defined stress or strain rate applied until such a time as the sample has been observed to have mobilised its total shear capacity and failed. As such it has been used to provide an indication of other measures of strength that too rely on the shear capacity of a sample, such as CBR and resilient modulus. There does exist many relations that have been derived for this purpose, however since no material that was tested both raw and modified were also tested for either of the other measures none are included for reference. For testing in this project ‘Australian Standard 5101.4 – 2008: Methods for preparation and testing of stabilized materials - Unconfined compressive strength of compacted materials’ has been utilised, which is very similar to the equivalent MRWA standard.

Within this project UCS testing has been utilised as simple means to ascertain if any increase in strength has occurred as a result of stabilisation. As the project focuses on the functional design and performance of unsealed pavements the strength is one of only many indicators of performance and therefore should be investigated but not relied upon and thus it is critical that the other properties tested are taken in context with the results UCS testing. Especially consider the years of empirical evidence accumulated with use of CBR tests, which may be understood by local practitioners for example.

The UCS and resilient modulus testing will then be utilised to carry out a fatigue assessment of raw and treated materials. Consideration is included to firstly indicate likely performance of stabilised (or rather modified) mine waste under such large wheel loads and secondly to provide some indication of the usefulness of such empirical relationships with such a large extrapolation.

3.2. Material Treatment Trials

As has been discussed previously the material suitability is assessed primarily via the guidelines defined by Paige-Green (1990) and displayed previously in Figure 2.11. These values have been modified to specify material properties falling within a grading coefficient of 14 to 30 and a shrinkage product of 126 to 247. Various methods for the stabilisation or modification of materials are to be trialled to better meet this specification firstly and others outlined above, if applicable. Each method is discussed in some detail below.

3.2.1. Borrow Material

The obvious method to improve a deficient material is to simply replace it with a superior alternative. For this reason and the vast amounts of stripped overburden and waste material available within mine operations, the possibility of utilising select materials for sheeting or even the full depth of pavement construction is investigated. Recommendations are made from assessment of geotechnical reports from areas adjacent to two of the mines discussed above, which were completed by qualified geotechnical engineers with testing carried out in certified laboratories. In both cases these investigations were completed as part of mine expansions and as such constitute an opportunity for existing mines to make use of select fill that represent a surplus to construction needs. Alternatively borrow pits could be established to provide appropriate material with the small haulage distances meaning an economic assessment is warranted for sheeting works.

3.2.2. Mechanical Stabilisation

Similar to the utilisation of borrow material, mechanical stabilisation employs select natural material to improve the in-situ (or available material prior to construction) material to meet the specification. Within the case study of Pilbara mine haul roads this has meant the inclusion of cohesive fines to increase shrinkage product and decrease grading coefficient. Again this material has been sourced from adjacent to the mine providing the greatest amount of data in the testing regime.

3.2.3. Cementitious Stabilisers

The benefit of cementitious stabilisers has been outlined in the literature review above. Undoubtedly the resultant increase in stiffness as a result of cement binder addition is a benefit to pavement performance and is well understood, however in this project it is applied in an attempt to act as a binder to the surface. In this project, for this action to be represented in the modelling applied an increase in the linear shrinkage and therefore shrinkage product would need to result and hence improve materials subject to corrugation and ravelling due to deficiencies in these properties.

A modification of the representative material is to be made to determine if any tangible benefit is present. It must be noted that the chemical reactions forming matrices between particles may indeed assist in binding a pavement surface without showing any change within the Atterberg Limit tests, as these have been derived to quantify the activity of a soil and hence show response to clay content and activity.

A Portland cement (General Purpose) modification is trialled to determine any of the aforementioned benefits. From a desk-top study it is clear that lime addition is unlikely to be of significant benefit due to the low amount of fines and linear shrinkage of the materials sampled. Lime/fly ash presents an interesting combination that warrants further trials with similar parameters.

3.2.4. Chemical Stabilisers/Dust Suppressants

This title is a very broad term that requires some refinement. In reality the sheer number of chemical stabilisers or suppressants has meant that this category of products has been omitted from laboratory investigation. It is suggested that any study of these products be carried out with a wide focus so as to quantify the many various types and include an analysis of chemical composition. Ultimately this would culminate in extensive field trials over a long period of time, as it is unlikely that the laboratory tests used here would define well enough the effectiveness of the large number of treatments. Thus instead a desk-top study is to be completed to define specific product types probable to be effective for the type of material sampled and tested for this project.

A major advantage noted by many producers and suppliers of chemical treatments is the economic advantage compared to re-sheeting or reconstructing the pavement wearing course in the case of stabilisers or compared regular passes of a water cart for the case of dust suppression. This can be difficult to quantify, as each situation will involve a somewhat unique degree of climate, material type and traffic composition and hence extensive field trials would be required to provide a definitive assessment.

3.3. Comparison of Materials

3.3.1. Material Properties - South African Model

Modelling materials for the consideration of functional design is best done using the modified version of the TRH20 model, comparing grading coefficient against shrinkage product, developed by Paige-Green (1990). This model is shown in Figure 2.12 and again Figure 3.8.

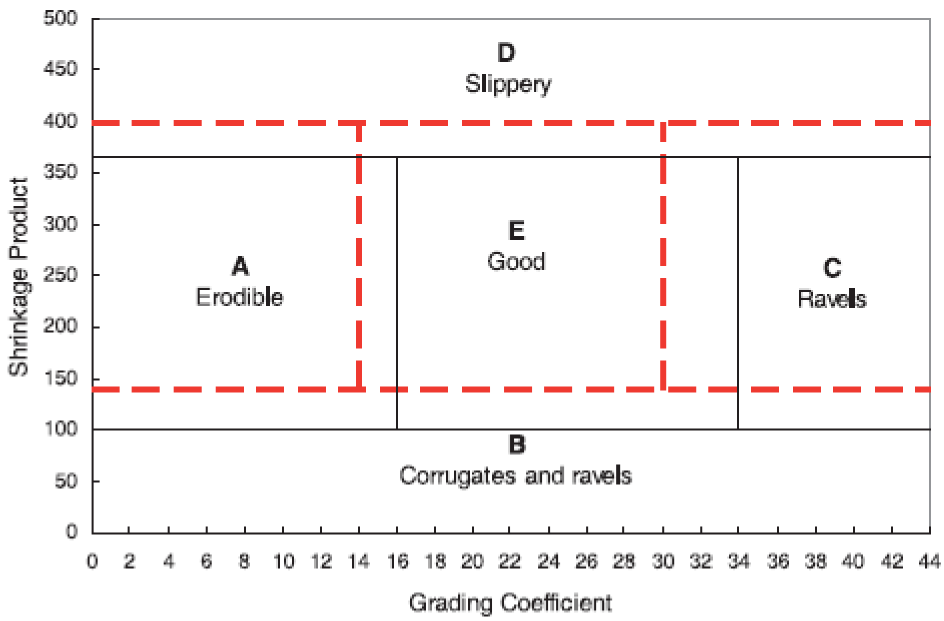


Figure 3.8: Modified model utilised for comparison of materials for functional design (Paige-Green, 2007)

This model has been modified by Thompson and Visser in the past to reflect survey results collected from the South African coal mining industry. Visual inspections and performance assessments were completed in parallel with definition on the model below to identify if the operable limits could be extended. Such a modification was deemed appropriate due to the original model being developed for provision of unsealed pavements on public roads and not for mine haul roads where maintenance and traffic composition could be vastly different. In this figure zone 1 indicates the preferable range with zone 2 representing the modified optimal range.

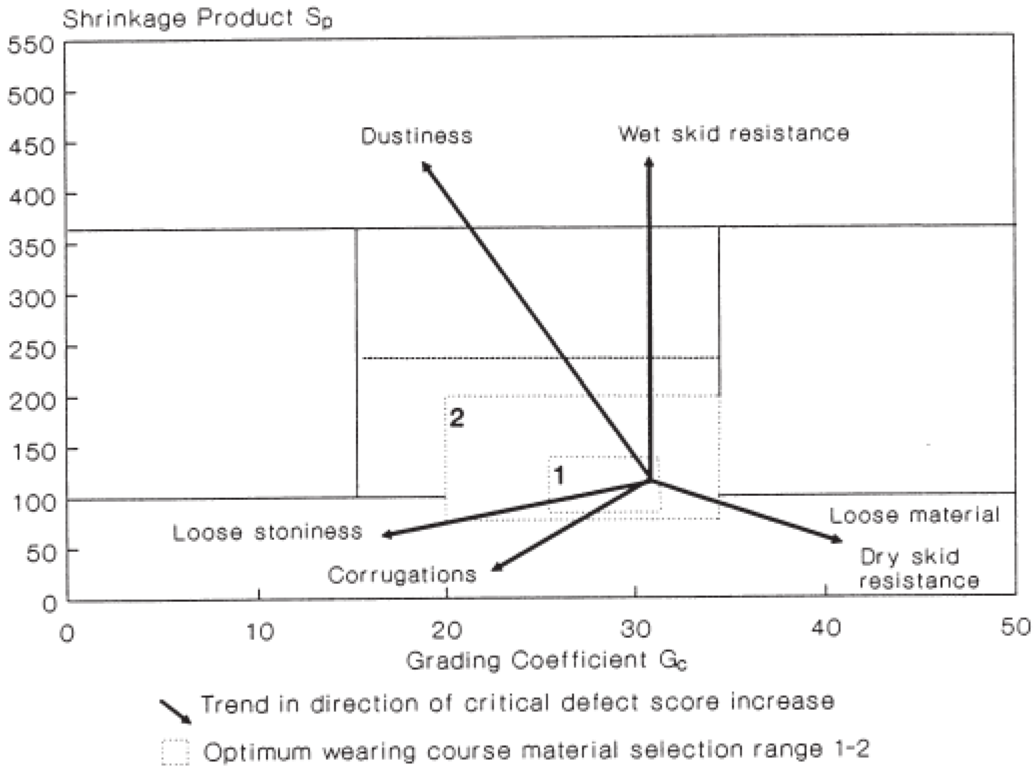


Figure 3.9: Modified wearing course specification for avoidance of most common/costly haul road defects (Thompson and Visser, 2000)

These specifications need to be changed due to a different linear shrinkage test being typically used in Australia (British Standard test utilising the cone penetrometer) than that in South Africa (Casagrande method). Paige-Green (2007) actually provides an equation for estimating the variance which was indeed applied originally to Figure 3.8 above (marked in red) but not in Thompson's modification in Figure 3.9.

Equation 3.1: Linear shrinkage for cone penetrometer test from Casagrande method results (Paige-Green, 2007)

$$\boldsymbol{BSLS = 1.0482 * SALS + 37.07}$$

Where: BSLS is Linear Shrinkage from use of British Standards test.

SALS is linear shrinkage form use of South African standard.

The result of applying this is that the optimal range for linear shrinkage now spans values of 126 to 247, with grading coefficient maintaining the same change as discussed above to 14 to 30, shown visually below.

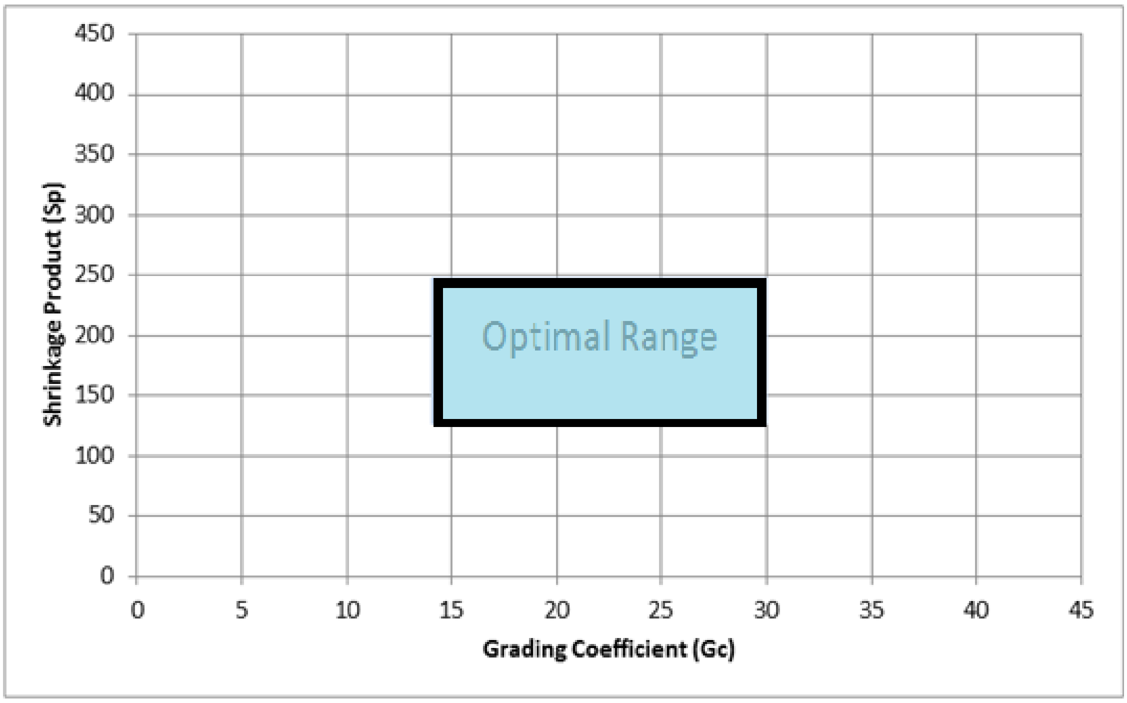


Figure 3.10: Optimal range of grading coefficient against shrinkage product

3.3.2. Further Changes possible – Relating to Practical Use

It is proposed within this project that due to all haul roads observed being regularly watered the climate is somewhat more constant than in the case of lowly trafficked public roads. Thus an avoidance of materials showing high shrinkage product would be prudent to avoid reduced shear strength and skid resistance due to the wearing course becoming overly wet. This is one example of implementing knowledge from mine personnel experience and qualitative assessment to better explain functional performance. In this case it has been highlighted to the author that rutting occurs very rapidly with highly plastic soils, with the added danger of low traction, especially after rain periods. Hence investigating if materials with a shrinkage product value in the upper reaches of the optimal range actually perform in this adverse way in practice would improve the selection criteria for the specific case of Pilbara haul roads. Therefore it is proposed that through material quantification the model can be further extended to consider:

- The observed pace of progression of certain defects.
- The ease of remediation for each defect type.
- The susceptibility of material to individual defect types from observation and interpretation of the existing model.

For the majority of the qualitative assessment described a decision can be made from searching the literature available, with additional consultation with experienced practitioners. This then allows real-time monitoring and assessment of pavements to improve maintenance practices. Such a pavement focussed approach can possibly be combined with real time monitoring systems now being installed into trucks whereby progression of roughness and defects can be identified using instrumentation such as accelerometers installed onto components such as suspension struts.

3.3.2.1. Additional Considerations for Comparison

Grading coefficient and shrinkage product are quite broad measures that are a simple capture of material qualities. However the typical pavement engineer will possess further understanding that can be used to better define performance in service and therefore consider a more complex approach to design. Further properties worth considering are discussed in the following.

3.3.2.2. Strength

It is common place for pavement materials to be compared by way of various measures of strength. The wearing course layer of unsealed pavements is one that is unique, in that strength is only one factor determining performance. It is worth noting that CBR and even resilient modulus are estimations (although modelling different characteristics) of vertical shear strength. Without doubt high shear resistance over any duration of loading is a valuable property of a wearing course (Paige-Green, 1990), however being able to perform satisfactorily under different climatic and loading conditions is just as critical and hence the measure of performance discussed above.

3.3.2.3. Permeability

Permeability is worth some consideration, as saturation is of course detrimental to nearly all pavement materials strength. A wearing course should maintain a low level of roughness but also protect the integrity of the base and sub-grade layers below it from ingress of moisture. Furthermore the importance of moisture sealing will vary depending on the activity of the materials constituting the underlying pavement layers. Therefore permeability is due some consideration when considering geometric parameters such at cross-fall and of course longitudinal drainage. Although complicating this definition of performance; it also may be beneficial for the wearing course layer to exhibit some degree of permeability in order to allow a sufficient degree of wetting for effective dust suppression, if water sprays are being employed. Hence comparison should consider a small amount of permeability beneficial.

3.3.2.4. Cohesion and Consistency Limits

Note of cohesion is included here as it provides (Paige-Green, 1990):

- Resistance to ravelling.
- Ensures low permeability.
- Provides a more stable surface in dry conditions.

Of course an excessive cohesive fines fraction presents a problem in itself as low skid resistance is likely in both dry and wet conditions. Shrinkage product in the above model provides a reasonable estimation of the cohesiveness of the soil however a high degree can be derived from a small amount of very active clay content, whereas a small shrinkage product can result from a high degree of non-plastic silty fines. Hence it is worth investigating the consistency limits of any material beyond simply utilising the most simple of the tests, being the linear shrinkage. In this case it is considered beneficial for a material to firstly present with some plasticity (Plasticity Index > 0) but not be excessive.

Furthermore it is beneficial to consider the absolute value of the plastic limit, especially if water sprays are being utilised (always the case in the case studies presented here). The likelihood of surface breakdown (development of defects leading to an increase in roughness) is only increased when a material has a low plastic limit, and hence will deform more easily with addition of less water. Hence no absolute value is defined but suffice to say if one material is completely identical in all other characteristics but for plastic limit, that with the higher limit should be selected for use, with a maximum value of 8 (Thompson and Visser, 2000) for the plasticity index the limiting parameter.

3.3.3. Recommendation of optimal material for Pilbara region

As part of the investigation into material used for Pilbara haul roads it may be possible that satisfactory performance is found with materials outside the range recommended in the literature and discussed at length above. For this reason it is suggested that the range could be extended so that what could be considered by some to be sub-optimal materials can be used in service. Use within an extended range of grading coefficient against shrinkage product will be with an understanding that increased running costs are likely to be associated as a consequence of likely increased roughness and rolling resistance.

3.4. Pavement Defect Modelling

The ability to predict pavement defect progression has been chosen as the basis of economic comparison of materials available for haul road construction. Such quantification allows estimates of adverse cost relating to vehicle operating costs, maintenance and production to be made which ultimately provides insight into the worth of expending ones budget on improved haul road design and construction. In the following each defect category modelled is explained in detail.

3.4.1. Dust

The modelling of dust generated by passing of a vehicle is of critical importance to the modelling of haul roads. As Thompson (2000) has shown the material associated with excessive dust generation accounts for the two defects with the highest perceived impact – dust and low wet skid resistance. In the case of the Pilbara region it is likely that dust is a much more likely effect on account of the dry hot climate for the majority of the year. Modelling dust emissions successfully is also significant due to the implications of gravel loss.

The United States Environmental Protection Agency has developed a model, AP-42[2], to predict dust emissions (USEPA, 1978). Jones (2000) has adopted this model for predicting total dust generated on South African unsealed roads. Its success with this application is the justification for its use within this project. It appears as:

Equation 3.2: Dust generation function (Jones, 2000)

$$E = k\left(\frac{s}{12}\right)\left(\frac{S}{48}\right)\left(\frac{W}{2.7}\right)^{0.7}\left(\frac{w}{4}\right)^{0.5}\left(\frac{365-\rho}{365}\right)$$

Where:

E = emission factor in kilograms/vehicle kilometer

k = particle size multiplier, values assigned as (Ono et al, 2004)

1.0 - particles < 30 micrometers (fugitive dust)

0.6 – particles <15 micrometers

0.5 – particles <10 micrometers (particulate matter – inhalable)

0.2 – particles <2.5 micrometers

s = silt content (<0.075 mm of surface material in %)

S = average vehicle speed (kilometres per hour)

W = average vehicle weight (tonnes)

w = average number of wheels

ρ = mean annual days with rainfall greater than 0.25 mm

Representative quantities used for modelling purposes are shown in Table 3.2:

Table 3.2: Parameter values for dust modelling constants

Parameter	Value Used
k	1.0
S	30 km/hr
W	390.5 tonnes
w	6 wheels
ρ	59 - No Watering 329 - Watering

The silt content was determined from laboratory testing of each material. Particle size multiplier was set at 1.0 to capture all dust emissions, as it is noted that any dust is unacceptable due to the safety implications of reduced driver sight and the environmental impact of smothering vegetation (Foley et al, 1996).

The above model is used to model roads with no water supplied from water carts for the purpose of dust suppression and also with such maintenance applied. In this case the mean number of days per annum with rainfall exceeding 0.25 mm is used. The Bureau of Meteorology in Western Australia is able to provide the number of days exceeding one millimetre of rainfall per annum as 29.5. Subsequently it is assumed twice as many days experience rainfall exceeding one-quarter of this quantity – or 59 days per annum (Bureau of Meteorology Western Australia Website – Tom Price Climate Data). When water sprays are provided it is assumed that this figure is exceeded 90% of the time. Beyond this the maintenance cost can be determined by the number of water carts required to achieve this coverage. It has been assumed that the small 6-wheeel water cart used for modelling purposes in this project can water 20 km of 30 m wide haul road per day if emission are between 5 and 10 kg/vehicle.km and 10 km if exceeding this range.

Capturing the increased operating costs for haulage equipment due to air-borne dust was found to be very cumbersome and approximate at best and as such has not been included within the modelling. However an assumption of lost production is made based on research carried out in South Africa as discussed in the proceeding.

3.4.2. Gravel Loss

Gravel loss is the result of dust emissions and as such is modelled to show the structural and functional effect beyond the direct impacts of air-borne particles. It has been estimated in the South African climate that seven mm of gravel loss will occur annually due to rain and wind erosion. This figure is captured within the model adopted which has been developed by Paige-Green (1990):

Equation 3.3: Annual gravel loss function (Paige-Green, 1990)

$$AGL: 3.65[ADT(0.059 + 0.0027N - 0.0006P_{26}) - 0.367N - 0.0014PF + 0.0474P_{26}]$$

Where:

AGL = Annual Gravel Loss (millimetres)

ADT = Average Daily Traffic

N = Weinert N Value (representative of climate)

P26 = Percentage of surface material passing 26.5 mm sieve

PF = Plastic Factor (Plastic Limit * percentage passing 0.425 mm sieve)

In order to utilise Equation 3.3 it is required that a Weinert N value be developed for the Pilbara in both dry conditions and also for the wet season/with water sprays applied for dust suppression. The ARRB (Foley et al, 1996) provides the detail behind calculating this value, however it must be noted this approximation could be further refined.

Equation 3.4: Definition of Weinert N value (Foley et al, 1996)

$$N = \frac{12Ej}{Pa}$$

Where:

N = Weinert N-value

Ej = evaporation in the warmest month (millimetres)

Pa = annual precipitation

Once again the Bureau of Meteorology of Western Australia was consulted for rainfall data for Tom Price, the average annual precipitation being 404.8 milimeters (Bureau of Meteorology Western Australia Website – Tom Price Climate Data). Evaporation is highest in the month of December with lower estimates being 299 mm (Luke et al, 1987). The net effect of using this value is that the climate is assumed to be favourable as possible for the consideration of annual gravel loss. Using these quantities a representative value of 8.86 was calculated. To allow representation of water sprays an assumed 1 mm of water was assumed applied to the road surface for 90 % of days in the year (as used above in dust modelling), making effective annual precipitation approximately 734 mm. This modification resulted in an effective Weinert N value of 4.89.

3.4.3. Roughness

Roughness captures the net effect of all defects outside of dust emissions and skid resistance on VOC and also impacts on production. Its modelling is critical to the success of the modelling within this project. It has the largest and most variable bearing on VOC and also has an undeniable negative impact on a mines production once it reaches an arbitrary level. In contrast to the vast importance of accurate modelling there are very few available models to predict the progression of roughness of an unsealed pavement. Roughness is a function of three main groups of inputs (Jones, 2001):

1. Material Properties
2. Maintenance
3. Traffic Composition

Accordingly a model was sought that captured all three factors, which is best achieved by the model presented by Jones (2001):

Firstly some qualifications are required to effectively apply the model in practice:

- Equation 3.5 should be applied for situations of less than 400 vehicles passes per day and a blading interval of less than 90 days, this is the South African Unsealed Road prediction model.
- Equation 3.6 should be used for situation of less than 400 vehicle passes per day and a blading interval greater than 90 days, this is the South African Steady State model.
- Equation 3.7 should be applied in the situation of greater than 400 vehicle passes a day, with the roughness calculated assumed to be reached 7 to 14 days after blading.

Note that this model was developed to output roughness in the unit of QI counts per kilometre which results in quantities 13 times larger than International Roughness Index (IRI), the adopted measure for this project. Appropriate modifications to the model have been made and are presented herein.

Equation 3.5: Natural logarithm of roughness (Paige-Green, 1990)

$$lnR = \left(\frac{1}{13}\right)D[-13.8 + 0.00022PF + 0.064S1 + 0.137P26 + 0.0003NADT + GM(6.42 - 0.0063P26)]$$

Equation 3.6: Quantification of roughness at a given time (Paige-Green, 1990)

$$RG_t = \left(\frac{1}{13}\right)(RG_{max} - \rho[RG_{max} - RG_a])$$

Equation 3.7: Maximum roughness (Paige-Green, 1990)

$$RG_{max} = \left(\frac{1}{13}\right)(-30.09 + 0.03PF + 294.7GM + 3.556P_{26})$$

Where:

lnR = natural logarithm of change of roughness with time

D = blading interval (days/100)

PF = Plastic Factor (Plastic Limit * % passing 0.075 mm sieve)

S1 = season dummy (1 for dry season and 0 for wet season)

P_{26} = % passing 26.5 mm sieve

N = Weinert N value

ADT = average daily traffic (if lanes separated, just considered for one way)

GM = Grading Modulus [(300-P_{20}-P_{425}-P_{075})/100]

RG_t = roughness at time ‘t’ (IRI – mm/m)

RG_{max} = as in Equation 3.7, maximum roughness of pavement for given physical properties

ρ = product involving traffic and maintenance data:

Equation 3.8: Product for use in roughness calculation (Paige-Green, 1990)

$$\rho = e^{\{-0.016+[0.001(D)(-0.17-0.000067ADT-0.00019NADT)]\}}$$

RG_a = roughness after blading – IRI (millimetres per meter)

For application in this project Weinert N values were assigned as discussed in the preceding section. The minimum roughness after blading was decided as 4.6 IRI, which is a combination of the value this model suggests when using the first equation for blading intervals of under 90 days, which was suggested by the original developer. Further justification is provided in Figure 3.11 below, showing this figure is within the lower reaches of expected roughness for an unsealed but maintained pavement.

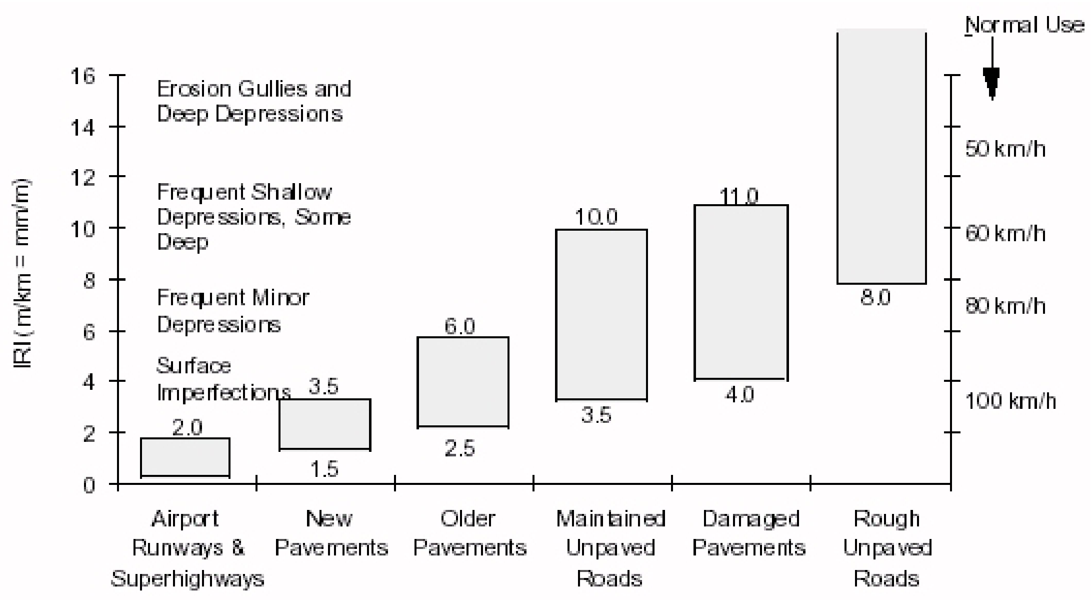


Figure 3.11: Expected roughness values for varying quality pavements (Austroads, 2011)

This model has been observed to match well over the intersection of the first two equations, at 90 days blading interval. For this reason sound continuity has been recorded when considering the effects of varying the blading interval around this critical 90 day figure.

Special note should be made of the dependence this model has on blading interval and average daily traffic when calculating roughness. Inspection of the equations reveals that they are by far the most influential variables in any of the three equations. The traffic composition was also intended to be applied for vehicles smaller than large haul trucks employed in mining. However since the roughness of the road surface is developed by such large trucks it is anticipated the relative roughness would also be equivalent in its effects on the vehicle considered.

3.4.4. Rolling Resistance

Estimating rolling resistance values is not a simple exercise as several relevant references exist with little agreement between them. For this reason it was decided an adaptation of the model proposed by Thompson (2003) be employed to determine if the range of roughness predicted would lead to an exceedance of the advised value of gradeability (maximum effective grade able to be traversed without slowing) due to increased rolling resistance. Of course this would involve a vastly higher fuel consumption rate than on lesser grades. Employing this model revealed a minimum rolling resistance of 1.8 % for a speed of 30 km per hour. This was assumed to occur at the minimum roughness of 4.6 ITI (m/km). The progression of rolling resistance to roughness appeared to be approximately 0.16 % per unit increase in IRI (mm/m). This figure does appear to output values lower than experience in some research would suggest, however it has been accepted and utilised in order to present what is expected to be a conservative estimate of rolling resistance in practice.

3.5. Defect Costing Model

As discussed in the previous section, the modelling of defect progression facilitates estimates of the major costs associated with haul road condition. The categories modelled include production losses and road maintenance costs. Additionally estimates have been made for treatment of haul road pavements as trialled in the laboratory testing component of this project, allowing life cycle assessments to be made, assisting the decision making process.

3.5.1. Production Impact of Material Type

Impacts to production of a haul road is assumed to be intrinsically linked to rolling resistance, therefore the above model has been implemented to assist in estimating losses of mine production due to road condition.

If a blading interval of one year was applied to the rolling resistance model developed above and a prediction is made of the roughness for all materials sampled, a maximum roughness of approximately 18 mm per meter is revealed which results in a rolling resistance of approximately 4.1 %. If this is compared to the gradeability curve for the design vehicle of a Caterpillar 793C off-highway dump truck (Figure 3.12), it can be seen that the gradeability of approximately 5 % is not exceeded for a speed of 30 km per hour when fully laden.

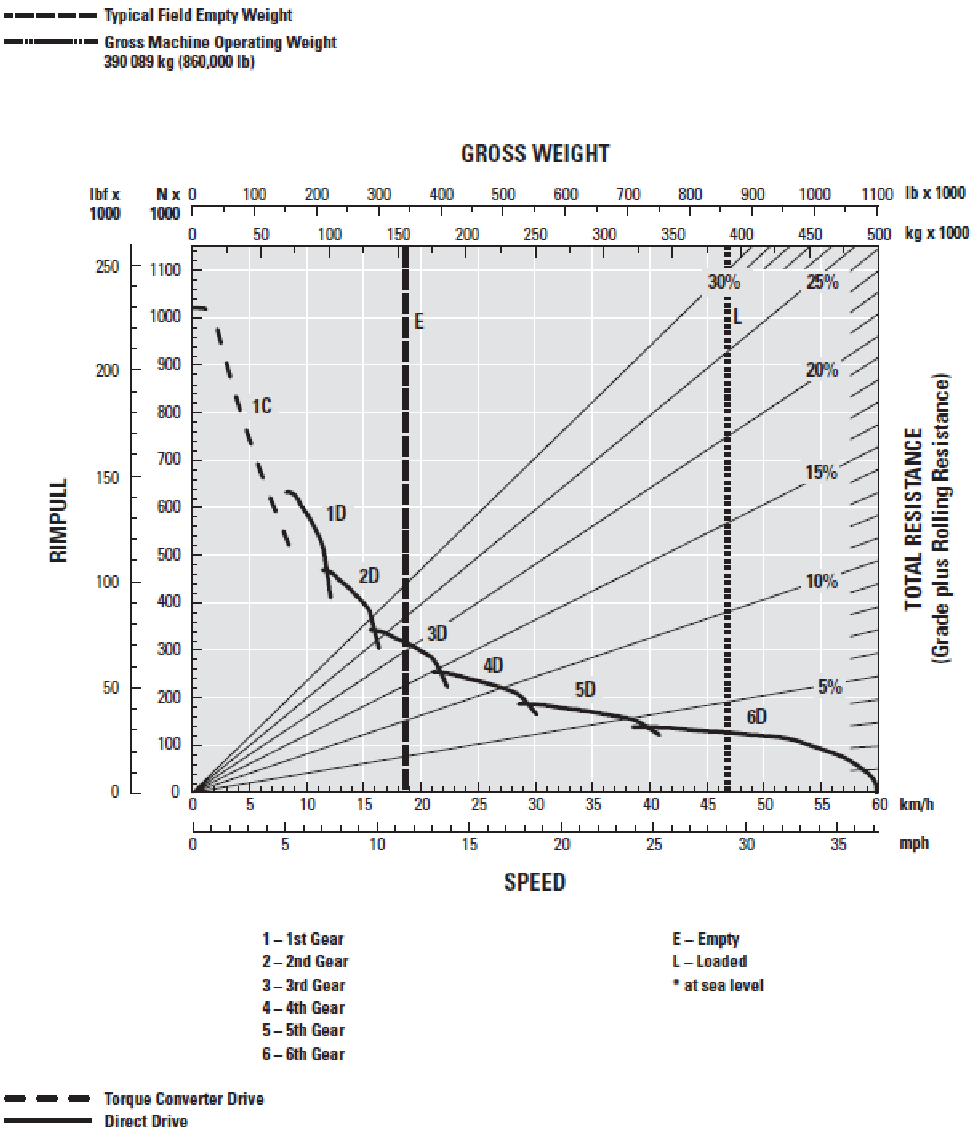


Figure 3.12: Caterpillar 793C Gradeability Curve (Caterpillar, 1998)

Although it therefore appears no slowing of haul trucks should be necessary in normal conditions (VOC increases but the design speed is achieved under all conditions), it is still acknowledged that a tangible impact must be applied to the mines production. This has been captured by considering the results of a survey and surveillance of South African coal mines by Thompson (2000). Once a certain defect (identified by probability of developing from inspection of material properties of Grading Coefficient and Shrinkage Product, as discussed above) exceeds a roughness of 8 mm per meter it is decided that the percentage reductions found in this study are implemented. The exception to this rule is that of dust, where half the reduction is assumed for emissions exceeding 5 kg per vehicle kilometre and the full percentage reduction when exceeding

10 kg per vehicle kilometre. The values used for each respective defect are presented in Table 3.3.

Table 3.3: Percentage reduction in production for defect severity exceedance (adapted from Thompson, 2000)

Corrugations	3 %
Rutting	2 %
Loose Material - Ravelling	11 %
Dustiness	6 %

Implementing such losses in this way is somewhat crude, however until the rolling resistance for haul trucks is properly determined and correctly related to roughness it is very difficult to predict slowing of the haulage process due to poor road surface condition in any other way. Additionally it will always be necessary to include acknowledgement of a human component whilst driving is conducted by a driver and not computerised, as there is an acceptance that drivers slow when driving on roads of higher roughness values.

3.5.2. Maintenance Cost

The decision of when to deploy maintenance is made through the setting of critical limits of each defect modelled. Material properties are then used to define which type of maintenance is required (as per which defect type is likely) as well as the base amount of equipment required to return the pavement to an assumed optimal roughness of 4.6 meters/kilometre. The levels of light and heavy maintenance and the equipment utilised, as long as assumed productivity rates used are those presented below in Table 3.4 and Table 3.5. The predominant defect is listed, so as to indicate that often if one defect is remediated it will also likely repair any other defect present, with the exception of dustiness which requires constant application of water. Ravelling and corrugations are costed together as the main causes of roughness, however a distinction is made for the purpose of differentiating two similar but in extreme cases very different materials associated with them. Gravel loss is defined as requiring remediation once a certain loss occurs and hence is in general an annualised type cost for re-sheeting. Rutting has not been determined from structural and settlement properties, from which it is actually caused. This decision is made due to the complexities and relative uncertainties with estimating the ratio of permanent/resilient deformation that will be ever changing with gradual compaction and vertical movements in an unbound and unsealed pavement. Similar can also be said for the ever-changing moisture content of the pavement. Rather rutting is reserved for materials that fall within the optimal range for functional design presented and discussed at length previously to incorporate the effects of roughness progression due to gradual wear of what may be a pavement with an optimised functional design and construction.

Table 3.4: Individual defects and associated parameters for costing of remediation

		Critical Limit	
Predominant Defect	**Material Parameter**	**Light**	**Heavy**
Corrugations	SP<126	IRI>8	IRI>11
Ravelling	Gc>30	IRI>8	IRI>11
Dustiness	-	E>5	E>10
Gravel Loss	-	AGL>50	
Rutting	126<SP247, 14<Gc<30	IRI>8	

Table 3.5: Assumed equipment and productivity rates for categories of remediation

Remediation	Equipment	Productivity
Light Blade	Grader 140C, 6-Wheel Water Cart	500m/12 hours
Heavy Blade	2* Grader 140H, 6-Wheel Water Cart, 12 tonne vibrating roller	250m/12 hours
Light Water	Semi-Water Cart	20 km/day
Heavy Water	Semi-Water Cart	10 km/day
Re-sheet	Grader 140H, 6-Wheel Water Cart, 12 tonne vibrating roller, Loader 980	250m/12 hours

The effect of watering also needs to be discussed, as it has been incorporated within the progression of all defects and therefore their remediation. However there has not been an automatic function built-in that initiates watering, rather the user of the model must indicate that water sprays are to be applied which is then added to the costing of dust suppression. This cost is calculated automatically due to the severity of emissions once the watering 'dummy' has been modified to indicate watering is to be considered.

The costing for equipment and operators is shown in Table 3.6. These values have been sourced from observation of market rates for work in the Pilbara and discussion with plant hire suppliers. Note that the costs largely reflect likely day-hire rates (reflecting only a short term hire would be required for maintenance) and hence for the case of plant ownership a saving of 10 to 15 % can be assumed. Fuel costs are calculated once the unit price (AUS$/L) for diesel is also inputted into the model. An assumed AUS$1.20/L has been used for generation of results for this projects specific case study. Note also that all costs are estimated upwards from hourly costs, all maintenance/remediation work is assumed to be completed over 12 hours per day, except for dust suppression by way of water carts which is assumed to be carried out 24 hours per day due to the imminent threat to driver safety at night resulting from further impaired visibility.

Table 3.6: Plant, labour and fuel usage rates used for estimation of remediation costs

Equipment	Hourly Cost (AUS$)	Estimated Hourly Fuel (AUS$)	Operator - Hourly Cost (AUS$)
Roller - Smooth Vibrating 12 tonne	80	20.4	110
Roller - Pads Foot	100	20.4	110
Roller - Multi-tyre	80	20.4	110
Grader - 140H	140	23.4	110
Grader - 160G	190	27.6	110
Water Cart - 6 Wheel	150	18	110
Water Cart - Semi	180	30	110
Dozer D8	250	39.6	110
Loader - 980	170	39.6	110
Loader - 992	300	104.4	110

The fuel consumption rates were assumed to be moderate and sourced from the Caterpillar Performance Handbook (Edition 29, 1998). The relevant tables are presented below (Table 3.7 through Table 3.10).

Table 3.7: Bulldozer fuel consumption rates (Caterpillar, 1998)

FUEL CONSUMPTION TABLES & LOAD FACTOR GUIDES

TRACK-TYPE TRACTORS

Model	Low		Medium		High	
	liter	U.S. gal	liter	U.S. gal	liter	U.S. gal
D3C & LGP Series III	4-7½	1-2	7½-11	2-3	9½-13	2½-3½
D4C & LGP Series III	5½-9½	1½-2½	9½-13	2½-3½	11-15	3-4
D5C & LGP Series III	5½-9½	1½-2½	9½-13	2½-3½	13-17	3½-4½
D4E	5½-9½	1½-2½	9½-13	2½-3½	11-15	3-4
D5M XL & LGP	6-10½	1½-3	10½-14½	3-4	12½-17	3½-4½
D5B	9½-13	2½-3½	11-17	3-4½	15-21	4-5½
D6M XL & LGP	11-15	3-4	12½-19½	3½-5	17-24	4½-6½
D6G	11-20½	3½-5	15½-21	4-6	23-28½	6-7½
D6R XL, XR & LGP	13-22½	3½-6	17½-25	4½-6½	25-30½	6½-8½
D7G Series II*	19-25	5-6½	26-34	7-9	32-40	8½-10½
D7R XR & LGP	19-23	5-6	25-28	6½-7½	32-36	8½-10
D8R & LGP	23-28	6-7½	28-38	7½-10	38-51	10-13½
D9R	36-47	9½-12½	47-58	12½-15½	60-76	16-20
D10R	44-59	11½-15½	59-76	15½-20	76-93	20-24½
D11R	62-87	16½-23	87-112	23-29½	112-134	29½-35½

*D7G fuel consumption data is based on a precombustion chamber equipped engine. Fuel consumption for a direct injection equipped D7G should be approximately 10% less.

Table 3.8: Grader fuel consumption rates (Caterpillar, 1998)

MOTOR GRADERS

Model	Low		Medium		High	
	liter	U.S. gal	liter	U.S. gal	liter	U.S. gal
120H*	9-13	2.4-3.4	13-17	3.4-4.5	17-21	4.5-5.5
135H*	10-14	2.6-3.7	14-18	3.7-4.8	18-22	4.8-5.9
12H	11-16	2.9-4.2	16-21	4.2-5.5	21-26	5.5-6.7
140H*	12-17	3.1-4.4	17-22	4.4-5.7	22-26	5.7-7.0
143H**	12-17	3.2-4.6	17-22	4.6-5.9	22-28	5.9-7.3
160H*	14-20	3.7-5.3	20-26	5.3-6.8	26-32	6.8-8.4
163H**	14-21	3.8-5.4	20-27	5.4-7.0	27-33	7.0-8.6
14H	15-22	4.0-5.8	22-28	5.8-7.5	28-35	7.5-9.2
16H	19-27	5.0-7.1	27-35	7.1-9.2	35-43	9.2-11.3
24H	32-46	8.6-12.2	46-60	12.2-15.8	60-74	15.8-19.4

*Multiply consumption by 1.10 when equipped with Variable Horsepower or Engine Power Management.
**Multiply consumption by 1.15 when operating in All Wheel Drive.

LOAD FACTOR GUIDE

High: Ditching, fill spreading, spreading base material, ripping, heavy road maintenance, snow plowing.
Medium: Average road maintenance, road mix work, scarifying, snow plowing.
Low: Finish grading, light maintenance, road travel.

Table 3.9: Loader fuel consumption rates (Caterpillar, 1998)

WHEEL LOADERS & INTEGRATED TOOLCARRIERS

Model	Low		Medium		High	
	liter	U.S. gal	liter	U.S. gal	liter	U.S. gal
902	*	*	*	*	*	*
906	*	*	*	*	*	*
914G, IT14G	5-6½	1-2	8-10½	2-2¾	11½-13	3-3½
924F, IT24F	5½-7½	1½-2	9½-12	2½-3	13-15	3½-4
928G, IT28G	7½-11	2-3	11-15	3-4	15-19	4-5
938G, IT38G	9-12½	2-3	13-17	3½-4½	18-22	4¾-5¾
950G	11-15	3-4	17-21	4½-5½	23-28	6-7½
962G, IT62G	12-16	3-4	18-22	5-6	24-29	6½-8
966F Series II	17-21	4½-5½	23-28	6-7½	32-38	8½-10
970F	19-23	5-6	25-30	6½-8	35-41	9-11
980G	23-26	6-7	30-36	8-9½	42-47	11-12½
988F Series II	32-38	8½-10	44-49	11½-13	60-66	16-17½
990 Series II	45-53	12-14	60½-68	16-18	79½-87	21-23
992G	58-66	15-17	83-91	22-24	116-125	30-33
994	102-109½	27-29	129-144	34-38	163-178	43-47

*Insufficient data.

LOAD FACTOR GUIDE

High: Steady cycling on basic loader cycle.
Medium: Steady cycling but over haul distances or work on basic loader cycle with frequent periods at idle.
Low: Light utility work. Considerable idling.

Table 3.10: Compactor fuel consumption rates (Caterpillar, 1998)

COMPACTION EQUIPMENT

Model	Low		Medium		High	
	liter	U.S. gal	liter	U.S. gal	liter	U.S. gal
CS-323C	8-11	2-3	11-13	3-3½	11-15	3-4
CS-431C	8-11	2-3	11-13	3-3½	11-15	3-4
CS-433C	11	3	11-13	3-3½	13-15	3½-4
CS-563C	13	3½	13-15	3½-4	15-21	4-5½
CS-573	13	3½	13-15	3½-4	15-21	4-5½
CS-583C	15-17	4-4½	17-19	4½-5	19-23	5-6
CP-323C	9-13	2½-3½	13-15	3½-4	15-19	4-5
CP-433C	13	3½	15-17	4-4½	17-19	4½-5
CP-533C	15	4	17-19	4½-5	19-25	5-6½
CP-563C	15	4	17-19	4½-5	21-25	5½-6½
CB-214C	6-8	1½-2	8-9	2-2½	9-13	2½-3½
CB-224C	8	2	9-11	2½-3	11-15	3-4
CB-434C	11-13	3-3½	13-17	3½-4½	17-19	4½-5
CB-534C	13	3½	15-17	4-4½	17-23	4½-6
CB-535B	13	3½	15-17	4-4½	17-23	4½-6
CB-544	11-13	3-3½	13-17	3½-4½	17-19	4½-5
CB-545	11-13	3-3½	13-17	3½-4½	17-19	4½-5
CB-634C	13-15	3½-4	15-19	4-5	19-21	5-5½
PF-300B	13	3½	15-17	4-4½	17-23	4½-6
PS-300B	13	3½	15-17	4-4½	17-23	4½-6
PS-500	13-15	3½-4	15-19	4-5	19-21	5-5½

LOAD FACTOR GUIDE

High: Vibration 80-100%, heavy cohesive soil, 305 mm **(12")** lifts or more.
Medium: Vibration 50-80%, granular soil, 100 mm-305 mm **(4"-12")** lifts.
Low: Vibration 30-50%, asphalt mix, 51 mm-305 mm **(2"-4")** lifts.

3.5.3. Material Construction Treatment Costing

To allow a complete comparison it was also required to complete estimates for unit construction of the various pavement treatments considered. This involved an additional material cost for any stabilisers along with a Caterpillar RM-350B stabiliser/reclaimer to mix an assumed constant 300 mm depth. To calculate the production rate possible within a day.

has been referenced giving production rates for this machine and ultimately showing half a kilometre per day (assumed 30 meter wide pavement) to be a reasonable estimate. Outside of the differing material costs the two cementitious stabiliser options also considered this equipment cost. In contrast the mechanically stabilised pavement involved only plant and labour costs, including representation of the haulage of materials from the borrow area where they are sourced. In this case five Caterpillar 740 articulated dump trucks were assumed sufficient, which are commonly used on construction projects and can haul longer distances on lesser surface conditions than if for example the much larger design vehicles for the mine haul roads considered in this project were used. However it should be highlighted that these vehicles could indeed be used for this purpose, although it would make the spreading and mixing process more time consuming. This is due to the Caterpillar 740's ability to place materials in consistent windrows, the dimensions of which a talented operator has good control over. For a 20 % in-situ blend to a depth of 300 mm the surface would need to be well scarified and hence a bulldozer may well be required (in the event the surface cannot be scarified by grader alone), with graders present to windrow the material into the borrow material for mixing. A similar process is in fact required in the case of all of the treatments and so a comparison can be omitted. All plant usage costs are presented in Table 3.12.

Table 3.11: Production rates of Caterpillar stabilizer (Caterpillar, 1998)

	PRODUCTION RATES																	
		m³/Minute																
		Cutting Depth — mm																
Travel Speed m/min	m²/ min	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500
3	7.3	0.73	0.9	1.1	1.3	1.5	1.6	1.8	2.0	2.2	2.4	2.6	2.7	2.9	3.1	3.3	3.5	3.7
6	14.6	1.46	1.8	2.2	2.6	2.9	3.3	3.7	4.0	4.4	4.8	5.1	5.5	5.9	6.2	6.6	6.9	7.3
9	21.9	2.2	2.7	3.3	3.8	4.4	4.9	5.5	6.0	6.6	7.1	7.7	8.2	8.8	9.3	9.9	10.4	11.0
12	29.3	2.9	3.7	4.4	5.1	5.9	6.6	7.3	8.0	8.8	9.5	10.2	11.0	11.7	12.4	13.2	13.9	14.6
15	36.6	3.6	4.6	5.5	6.4	7.3	8.2	9.1	10.0	11.0	11.9	12.8	13.7	14.6	15.5	16.5	17.4	18.3
18	43.9	4.4	5.5	6.6	7.7	8.8	9.9	11.0	12.1	13.2	14.3	15.4	16.5	17.6	18.7	19.7	20.8	21.9
21	51.2	5.1	6.4	7.7	9.0	10.2	11.5	12.8	14.1	15.4	16.6	17.9	19.2	20.5	21.8	23.0	24.3	25.6
24	58.5	5.9	7.3	8.8	10.2	11.7	13.2	14.6	16.1	17.6	19.0	20.5	21.9	23.4	24.9	26.3	27.8	29.3
27	65.8	6.6	8.2	9.9	11.5	13.2	14.8	16.4	18.1	19.7	21.4	23.0	24.7	26.3	28.0	29.6	31.3	32.9

Table 3.12: Machine usage costs used for construction costing

Equipment	Hourly Cost (AUS$)	Estimated Hourly Fuel (AUS$)	Operator - Hourly Cost (AUS$)
Roller - Smooth Vibrating 12 tonne	80	20.4	110
Roller - Pads Foot	100	20.4	110
Roller - Multi-tyre	80	20.4	110
Grader - 140H	140	23.4	110
Grader - 160G	190	27.6	110
Water Cart - 6 Wheel	150	18	110
Water Cart - Semi	180	30	110
6-Wheel Dump Truck	150	18	110
CAT 740 - 40 tonne Dump Truck	250	30	110
Dozer D8	250	39.6	110
Loader - 980	170	39.6	110
Loader - 992	300	104.4	110
Stabilizer - CAT RM-350B	200	25	110

3.5.4. Vehicle Operating Cost (VOC)

Modelling of VOC is a complex and well researched topic for passenger and even long-haul transportation vehicles. However it has not been addressed in the available literature for large off-highway dump trucks. Consequently existing models have been investigated with a view to modify them for use in predicting diesel consumption, maintenance and tyre wear rates for haul trucks.

3.5.4.1. Fuel Consumption

For definition of fuel consumption mechanistic models were sought that are governed by a defined vehicle traversing a defined terrain, such that they might be modified to represent the mine haulage process. The NIMPAC model was investigated but has been noted to over-estimate fuel use for heavy trucks due to being largely developed in the 1970's and gains having been made in haulage efficiency since then (Tsolakis et al, 2011). Additionally it lacks flexibility that may allow it to be extrapolated to large haul trucks. Additionally a South African model based on total energy requirements has also been developed but was not able to be detailed to an extent that would allow modification. Ultimately it was decided that the ARRB Road Fuel Consumption Model (ARFCOM) developed by the Australian Roads Research Board would be best suited due to its inherent flexibility (see Table 3.13) and complexity, limiting the amount of empirical functions that would need to be extended. It has also been adopted in the World Bank's Highway Development and Management 4 (HDM-4) model.

Table 3.13: Assessment of Mechanistic Fuel Consumption Models (Greenwood and Bennett, 2003)

Fuel Model	Forces Opposing Motion	Internal Vehicle Forces	Engine Speed Effects	Appropriate for Acceleratio n Fuel	Transferabl e to Different Vehicles
HDM-III	•		•		
South African	•			•	•
ARFCOM	•	•	•	•	•

The model itself is quite complex, but calculates fuel consumption in proportion to total power requirements. The overall function is presented below followed by a table of input equations, variables and constants (assuming a constant design vehicle of a Caterpillar 793C within this project). Following these inputs are tables describing their variance should an alternative vehicle be considered.

Equation 3.9: Instantaneous fuel consumption function (Greenwood and Bennett, 2003)

$$IFC = \max\left(FC_{min}, \beta . P_{tot}\right)$$

All the functions making up the model and details that were kept constant throughout the analysis, as relating to the Caterpillar 793C design vehicle or are defined by the model itself, are presented in Table 3.14.

Table 3.14: All functions constituting the ARFCOM fuel consumption model used

Parameter	Description	Function
ceng	speed independent engine drag parameter	0.017*Pmax
beng	speed dependent engine drag parameter	0.7+0.026*Pmax
α	Engine idle fuel consumption (mL/s)	β*(Peng+Paccs)
RPM	operating engine speed	$a0+a1*v+a2*v^{2}$
Pa	power to overcome air resistance (kW)	$0.7331*C*D*AF*v^{3}$
Pr	power to overcome rolling resistance (kW)	$M*g*v*(Co+Cv*v^{cm})$
Pg	power to overcome gradients (kW)	M*G*g
Pc	power to overcome horizontal curvature (kW)	Ff*tanϕ
Pi	power to overcome inertial effects (kW)	M'*a
Ptr	Power to overcome tractive resistance	Pa+Pr+Pg+Pc+Pi
Paccs	Power to run accessories	$EALC*(RPM/TRPM)+ECFLC*Pmax*(RPM/TRPM)^{2.5}$
Peng	Power to overcome internal engine drag	$ceng+beng*(RPM/1000)^{2}$
Ptot	Total power requirements (kW)	Ptr/edt+Paccs+Peng
IFC	Instantaneous Fuel Consumption (mL/s)	max(Fcmin,β*Ptot)
Co	static coefficient of rolling resistance	0.0041+0.00043*IRI
TRPM	load governed maximum engine speed (rev/min)	$4600-18*Pmax+0.03*Pmax^{2}$

Table 3.15: Constants and values used in ARFCOM model

Input Variable	Description	Modelling Value
G	Gradient (m/m)	-
a	acceleration (ms^{-2})	-
IRI	Roughness (m/km)	-
Constant variables		
Cd	Aerodynamic drag coefficient	0.9
AF	Projected frontal area (m^2)	42.2
g	acceleration due to gravity (m/s^2)	9.81
M	Vehicle Mass (kg)	390,500
v	Velocity (m/s)	8.33
M'	Effective Vehicle Mass (kg) (EMRAT*M)	0
Fcmin	min fuel consumption (mL/s)	1,560,356
β	fuel to power efficiency factor (mL/kW/s)	0.07
EALC	accessory load constant (kW)	10
ECFLC	cooling fan constant	0.05
Pmax	maximum rated engine power (kW)	1976
RPM	operating engine speed (RPM)	1100
Cv	dynamic coefficient of rolling resistance	25
cm	an exponent	1
edt	drive-train efficiency factor	0.95

The above values have been sourced from the following literature. Table 3.16 and Table 3.17 have been sourced from Caterpillar's performance handbook which is commonly used in compiling earthworks machine production rates. Table 3.18 present the original recommended values for rolling resistance constants and engine modelling constants derived in a New Zealand study by Greenwood and Bennett (2003). Additionally

Table 3.19, Table 3.20 and Table 3.21 provide the engine constants including accessory and cooling fan loads also developed by Greenwood and Bennett (2003).

Table 3.16: Drive train efficiency coefficient (α) (Caterpillar, 2007)

Component	Efficiency (%)
Transmission - DD	99
Transmission - OD	98
Drive Axle - Tandem	90
Drive Axle - Single	95

Table 3.17: Aerodynamic Drag Coefficient (CD) (Caterpillar, 2007)

Configuration	Factor
HD Tractor - Full Aero / Van Trailer full aero	0.42
HD Tractor - Full Aero / Van Trailer Typical	0.48
MD VanTruck - Full Aero	0.50
HD Tractor - Full Aero / Van Trailer some aero	0.54
HD Tractor - Full Aero / Tank Trailer Insulated	0.55
HD Tractor - Full Aero/Flat Trailer some aero(Smooth Load)	0.55
HD Tractor - Full Aero / Van Trailer no aero	0.80
HD Tractor - Full Aero/Flat Trailer some aero(Rough Load)	0.80
HD Tractor - Full Aero/Tank Trailer Non Insulated	0.80
HD Tractor - No Aero / Van Trailer no aero	0.80
MD VanTruck - No Aero	0.80
HD Dump	0.90
MD Dump - No Aero	0.90
HD Tractor - Car Hauler	1.00
HD Tractor - No Aero / Flat trailer some aero	1.00

Table 3.18: Rolling resistance constants (Greenwood and Bennett, 2003)

Vehicle	Co	Cv ($\times 10^{-6}$)	Cm
Light vehicles - radials	0.0124+0.00043 IRI $Tdsc^2$	6.99+0.64 IRI	2
Light vehicles - bias	1.30 (0.0124+0.00043 IRI $Tdsc^2$)	1.30 (6.99+0.64 IRI)	2
Trucks and buses - radials	0.0041 + 0.00043 IRI	25	1
Trucks and buses - bias	0.0066 + 0.00043 IRI	29	1

Table 3.19: Engine modelling constants in ARFCOM model (Greenwood and Bennett, 2003)

Vehicle Class	Engine Speed Model Parameters				Idle RPM rev/min	α mL/s	β mL/kJ	edt	TRPM rev/min	EALC kW	ECFLC
	a0 RPM	a1 RPM/(m/s)	a2 RPM/(m/s)^2	a3 m/s							
Motorcycle	2790	94.0	2.83	31	800	0.12	0.14	0.95	10000	0.75	0.00
Passenger Cars - Small/Medium	2280	17	0.83	42	800	0.48	0.10	0.90	6200	3.0	0.10
Passenger Cars - Large	1709	7.16	0.99	42	800	0.60	0.10	0.90	4000	3.0	0.10
Light Delivery	2490	-30.4	2.25	34	800	0.48	0.10	0.90	5000	1.5	0.10
Light Goods	2574	-27.8	2.46	32	800	0.37	0.10	0.90	5000	1.5	0.10
Light Truck	1214	17.6	2.32	22	500	0.37	0.07	0.86	3500	1.5	0.10
Medium Truck	1214	17.6	2.32	22	500	0.37	0.07	0.86	3500	3.0	0.05
Heavy Truck	1167	-24.0	1.76	22	500	1.12	0.07	0.86	2100	3.0	0.05
Articulated Truck	1167	-24.0	1.76	22	500	1.12	0.07	0.86	2800	3.0	0.05
Light Bus	2490	-30.4	2.25	34	800	0.48	0.10	0.90	5000	3.0	0.05
Medium Bus	1214	17.6	2.32	22	500	0.37	0.07	0.86	2800	15.0	0.05
Heavy Bus	1167	-24.0	1.76	22	500	0.37	0.07	0.86	2800	15.0	0.05

Table 3.20: Engine accessory load constants (Greenwood and Bennett, 2003)

Accessories	EALC (kW)
Alternator (Alt) only	0.75
Alt & Air Compressor (A-Comp)	1.50
Alt + (A-Comp) + Power Steering (P-S)	3.00
Alt + (A-Comp) + (P-S) + Cabin Air Conditioning	6.00
Alt + (A-Comp) + (P-S) + Coach Air Conditioning	15.00

Table 3.21: Engine cooling fan load constant (Greenwood and Bennett, 2003)

Accessory	ECFLC
Full time fan (front engine HD diesel)	0.050
Full time fan (front engine med. duty diesel & petrol engines)	0.100
Full time fan (rear transverse bus - diesel)	0.100
Thermo Controlled Fan (front eng. HD diesel)	0.002
Thermo Controlled Fan (front end med. duty diesel & petrol)	0.004
Thermo Controlled Fan (rear transverse bus radiator - diesel)	0.010

The above model was tested extensively through this project and appears to calculate fuel consumptions that align with Caterpillar's own published data as indicated in Table 3.22. Additionally, the model has been observed to produce relative movements that are in line with expectations (relative to roughness progression) and so the model was employed for analysis of pavement materials.

Table 3.22: Hourly fuel consumption manufacturer data – Caterpillar 793C dump truck (Caterpillar, 1998)

CONSTRUCTION & MINING TRUCKS & TRACTORS

Model	Low		Medium		High	
	liter	U.S. gal	liter	U.S. gal	liter	U.S. gal
769D	20.8-30.3	5½-8	30.3-40	8-10½	40-68	10½-14
771D	22.7-32.2	6-8½	32.2-41.6	8½-11	41.6-55	11-14½
773D	24.5-36	6½-9½	36-53	9½-14	53-68	14-18
775D	30.3-41.6	8-11	41.6-56.8	11-15	56.8-73.8	15-19½
776D	53.0-73.8	14-19½	73.8-96.5	19½-25½	96.5-117.3	25½-31
777D	36.0-53.0	9½-14	53.0-73.8	14-19½	73.8-96.5	19½-25½
784C/785C	53.0-79.5	14-21	79.5-109.8	21-29	100.8-145.7	29-38½
789C	68.1-102.2	18-27	102.2-141.9	27-37½	141.9-185.5	37½-49
793C	86-129	23-34	129-172	34-45½	172-215	45½-57

3.5.4.2. Tyre Usage

Modelling tyre usage costs has also been completed by way of the model adopted within HDM-4, which estimates the number of tyres used to the end of their serviceable lives per 1000 km travelled by the vehicle. It was originally included in HDM-3 for heavy vehicles only, despite this it does not extend to off the road (OTR) tyres nor large off highway dump trucks such as a Caterpillar 793C. For this reason the most appropriate representative values have been selected in order to model differences in wear due to differing calculated pavement roughness. As with the other VOC models the HDM-4 model does not display a particularly high sensitivity to roughness, which can be attributed to its development for sealed pavements and hence a much smaller range than that probable to occur on a haul road. Caution needs to be exercised in applying the tyre life estimates produced for large off-highway dump truck tyres, as the original development of the model was for asphalt pavements which will provide different wear characteristics. The surface aggregate alone will likely affect the rate of wear, furthermore the puncture rate is certain to be higher on unsealed pavements and therefore would ideally be represented within the modelling. Unfortunately a suitable tyre consumption model developed for off road driving could not be found for adaptation.

The overall function is shown in Equation 3.10.

Equation 3.10: Tyre consumption model (Greenwood and Bennett, 2003)

$$TC = NT * \left[\frac{(1 + 0.01 * RREC * NR) * \left(C_{0tc} + C_{tcte} + \frac{CF^2}{L}\right)}{(1 + NR) * VOL} + 0.0027\right]$$

The functions and constants, along with their sources, are laid out in Table 3.23, Table 3.24 and Table 3.25.

Table 3.23: Variables employed in tyre consumption model (Austroads, 2011)

Parameter	Description	Function
NR	Predicted number of retreads per tyre	NR_0*exp(-0.03224*IRI-0.00118C)-1

Table 3.24: Variables and constants for modelling Caterpillar 793C tyre usage (Austroads, 2011)

Input Variable	Description	Modelling Value
IRI	Roughness (m/km)	-
Constant Variables		
NT	Number of tyres per vehicle	6
RREC	Percentage cost of one re-tread to cost of new tyre	0.33
NR_0	Base number of recaps (default=1.3)	1.3
C	Average horizontal curvature in degrees/kilometer	0
C_{otc}	Constant term of the tread wear model	0.001
C_{tcte}	Wear coefficient	0.003
CF^2	Average circumferential force per tyre, tangent to road surface, squared (kN)	142189
L	Average force per tyre perpendicular to road surface (kN)	638.5
VOL	Average wearable rubber volume per tyre (dm^3)	5.44

Table 3.25: Default values for constants within tyre usage model (Carpenter et al, 1999)

Vehicle Number	Vehicle Type	Ctcte dm^3/MNm	C0tc $dm^3/1000km$
1	Motorcycle	0.0009	0.001
2	Small Car	0.0005	0.001
3	Medium Car	0.0005	0.001
4	Large Car	0.0005	0.001
5	Light Delivery Vehicle	0.0005	0.001
6	Light Goods Vehicle	0.0005	0.001
7	Four Wheel Drive	0.0005	0.001
8	Light Truck	0.0003	0.001
9	Medium Truck	0.0003	0.001
10	Heavy Truck	0.0003	0.001
11	Articulated Truck	0.0003	0.001
12	Mini-Bus	0.0003	0.001
13	Light Bus	0.0003	0.001
14	Medium Bus	0.0003	0.001
15	Heavy Bus	0.0003	0.001
16	Coach	0.0003	0.001

The volume of wearable tread for the 40.00R57 tyre size that is specified as the preferred tyre size for a Caterpillar 793C was found to be poorly quoted between manufacturers and so an estimated value of 0.0055 cubic meters (5.5 dm^3) of wearable tread was utilised. This is an estimated 30 mm of tread depth over all tread surface area.

The circumferential force was calculated using an estimated soil cohesion of 10 kPa and an internal friction angle of 30 degrees. The maximum circumferential force was adapted from Lach (1996) and appears is shown in Equation 3.11.

Equation 3.11: Function for maximum circumferential force (Lach, 1996)

$$U_{max} = A * c + F_z * tan\,\varphi$$

Where:

U_{max} = maximum circumferential force (kN)

A = contact area between tyre and soil (m^2)

c = soil cohesion (kPa)

F_z = wheel load (kN)

φ = soil internal friction angle (degrees)

An estimate was also made for the cost of re-treading a used tyre being one third the purchase price of a replacement. On appearances this may appear to be quite high, however a general preference to avoid re-treading OTR tyres is present in the mining industry currently, where labour costs are also in some cases prohibitively high. For these reasons a relatively high percentage was chosen. The equation also utilises the default value for base number of recaps of 1.3. Attributed to the reluctance for tyre restoration against replacement this figure has been maintained to reflect the fact that it is indeed possible and costs can be reduced, if the assumptions surrounding this cost estimate are accurate.

3.5.4.3. Maintenance Costs – Parts and Labour

Maintenance costs for the design vehicle is completed via adaptation of the HDM-4 model, as above. Some extrapolations are again required which results in a warning of caution when using the absolute values. The total vehicle maintenance costs are broken into parts and labour costs.

Parts costs, as a fraction of the new vehicle price per 1000 km are estimated by Equation 3.12.

Equation 3.12: Parts cost estimation (Bennett, 1998)

$$PC = K0pc * [CKM^{kp} * (a0 + a1 * IRI) + K1pc] * (1 - CPCON * dFUEL)$$

Labour maintenance costs, as a fraction of the new vehicle purchase price per 1000 km are estimated by Equation 3.13.

Equation 3.13: Labour component of maintenance costs (Bennett, 1998)

$$LH = K0lh * (a0 * PC^{a1}) + K1lh$$

All constants and variables adopted in the model are described and quantified in Table 3.26 and Table 3.27.

Table 3.26: Constants and variables employed to estimate parts maintenance costs

Input Variable	Description	Modelling Value
IRI	Roughness (m/km)	-
Constant Variables		
CKM	Vehicle cumulative kilometerage	400,000
kp	Age exponent	0.371
CPCON	Incremental change in parts due to congestion	0.1
dFUEL	Additional fuel due to congestion as a fraction	0
Kopc	Rotation calibration factor (default=1)	1
K1pc	Translation calibration factor (default=0)	0
a0	A constant	$11.58*10^{-6}$
a1	A constant	$2.96*10^{-6}$

Table 3.27: Constants and variables employed to estimate labour maintenance costs

Input Variable	Description	Modelling Value
PC	Parts consumption as fraction of new vehicle price per 1000km	-
Constants		
K0lh	Rotation calibration factor (default=1)	1
K1lh	Translation calibration factor (default=0)	0
a0	A constant	301.46
a1	A constant	0.519

Obviously the raw value estimates are predicated on the original vehicle purchase price. A figure of two million Australian dollars was used, although it is thought a new Caterpillar 793C may actually be more expensive at the current time. Any higher and the model seemingly over-estimates the total parts costs, to the extent that over 50 % of running costs are made up of this expense alone. The recommended cumulative kilometerage can be observed as being 602,000 in Table 3.28 (for 'HT' – Heavy Trucks) but has been reduced to 400,000 to reflect the more probable ages of the trucks involved in the case study of the project. Definition of variables for other vehicle types are defined herein also.

Table 3.28: Recommended parts and labour model parameters (Bennett, 1998)

Vehicle	Description	Parts Consumption Model				Labour Model	
		CKM	kp	a0 x 10^{-6}	a1 x 10^{-6}	a0	a1
1	MC	50,000	0.308	9.23	6.20	77.14	0.547
2	PC	115,000	0.308	36.94	6.20	77.14	0.547
3	PC	115,000	0.308	36.94	6.20	77.14	0.547
4	PC	115,000	0.308	36.94	6.20	77.14	0.547
5	LDV	120,000	0.308	36.94	6.20	77.14	0.547
6	LGV	120,000	0.308	36.94	6.20	77.14	0.547
7	4WD	120,000	0.371	7.29	2.96	77.14	0.547
8	LT	120,000	0.371	7.29	2.96	242.03	0.519
9	MT	240,000	0.371	11.58	2.96	242.03	0.519
10	HT	602,000	0.371	11.58	2.96	301.46	0.519
11	AT	602,000	0.371	13.58	2.96	301.46	0.519
12	MNB	120,000	0.308	36.76	6.20	77.14	0.547
13	LB	136,000	0.371	10.14	1.97	242.03	0.519
14	MB	245,000	0.483	0.57	0.49	293.44	0.517
15	HB	420,000	0.483	0.65	0.46	293.44	0.517
16	COACH	420,000	0.483	0.64	0.46	293.44	0.517

The model itself does appear to over-estimate parts costs in relation to labour costs, even with the relatively high estimate of 130 Australian dollars per hour for a heavy diesel fitter to perform service in the Pilbara region. The overall maintenance costs tend to account for at least 50 % of the VOC elements considered in this projects modelling, which is considered a realistic estimate.

3.6. Defect Progression Score

Defect scoring systems have been developed previously by other authors. The methodology discussed at length in Section 2.6 involves influence scoring of safety and impact. This reflects the effect on production each defect has and also the perceived effect on the safety with which the pavement can be traversed. Modification of this methodology has been completed to consider the additional information of wearing course material, relative defect progression, road geometry and difficulty of remediation. The first three can be changed for each individual alignment and pavement with remediation being unique. This then allows those responsible for maintenance or even design to better describe the condition of the surface and likely future deterioration. The expansion of the system is detailed such that it can be modified for any particular set of conditions, however it is developed specifically for the case study of Pilbara haul roads in the proceeding.

Note that a decision needs to be made on how to weight the six categories within the score. These need to be made based on potential impact to the overall production of the mine and therefore profitability. The final weightings are shown in Equation 3.14 via the relative value of the constant applied to each variable or are available in Table 4.4.

The output from modelling of pavement performance and unit costing is to be utilised to provide insight into the effect of the defect progression score to provide practical advice for the programming of maintenance. The premise of such development is to provide a simple tool for assessment, especially for practitioners with limited experience with unsealed pavements, or to provide consistency of maintenance effort by provision of a standardised scoring/assessment method.

The defect score proposed is calculated via Equation 3.14.

Equation 3.14: Proposed defect score

$$DS = \sum[CS * (5*AP+5*IS+2*DP+2*GE+3*RC+MP)]$$

Where:

DS = Defect Score

CS = Condition Score for each defect

AP = Accident Potential

IS = Impact Score

DP = Defect Progression

GE = Geometric Effect

RC = Remediation Cost

MP = Material Properties

It is recommended this be used in matrix form to simplify the calculations, any spread sheet application such as Microsoft Excel can calculate the defect score instantaneously with only a few minutes required to setup a suitable calculation sheet. The matrices for Accident Potential and Impact Score have been adapted from the results reported by Thompson and Visser (2000) and are shown in Table 3.29.

Table 3.29: Relative scores to be applied for Accident Potential (from Thompson and Visser, 2000)

Defect Type	Relative Score
Potholes	1.4
Corrugations	1.5
Rutting	1.3
Loose Material – Ravelling	2.7
Dustiness	5.1
Excessive Clay – wet skid resistance	3.5

Table 3.30: Relative scores to be applied for production impact (from Thompson and Visser, 2000)

Defect Type	Relative Score
Potholes	0.3
Corrugations	1
Rutting	0.3
Loose Material – Ravelling	4
Dustiness	2
Excessive Clay – wet skid resistance	5

3.6.1. Relation to material type

The ability to predict performance at the design stage of pavement construction has been explained at length in the preceding, utilising Figure 2.12. With this is mind the defect progression scoring methodology can be modified to reflect the additional knowledge of predicted performance. For example if the material is very coarse and prone to ravelling this particular defect is given a higher weighting so as to emphasise the need of maintenance once the defect begins to show.

3.6.2. Relation to Defect Progression

Different defects progress at a different pace, for example a pothole in the middle of winter is likely to progress to a point of requiring attention much quicker than a surface drying out between rain in summer months and consequently producing dust emissions. This is to be reflected within the scoring methodology by inputting a 'dummy' to indicate if water sprays are to be applied to the pavement as this will alter the behaviour of the surface a great deal.

3.6.3. Relation to Remediation Effort

Remediation of defects or maintenance is an expensive provision for haul roads, with additional cost to the business being incurred due to delays in production as a result of disruption to traffic flow. In most cases it is generally accepted that the remediation effort required to return a wearing course to an acceptable level increases somewhat exponentially to a point where the driver seeks areas of less roughness (Thompson, 2000), obviously slowing vehicles and therefore mine output. Hence maintenance should be programmed at an optimal time, prior to the effort required to return the surface to a high level of serviceability being prohibitively high. There is some literature available on the topic of remediation, however a simple approach has been taken here with some consultation from those in the mining industry to define the defects which are hardest to remediate, and giving them a higher weighting in this category. The amount of equipment and labour is also considered, as the cost of repair/rehabilitation is also a significant factor in both performance and programming of work. The cost of remediation considered in this projects' pavement modelling is to be utilised to develop weightings for each defect type within this category.

3.6.4. Relation to Geometry

The geometry of the road should have some bearing on how it is treated when utilising results of a defect score. In this case the following classes have been considered:

- Ramp: haulage routes climbing out of the pit.
- Short-term alignment: in-pit roads.
- Long-term alignment: predominantly ex-pit roads to processing plant or waste areas.

This separation of classes is significant in order to describe the worth of maintenance expenditure and perceived risk of a decrease in safety due to any given defect. For example the consequence of a haul truck having poor skid resistance and therefore a larger stopping distance due to excessive clay content in the wearing course is likely higher on a ramp than on a flat and straight long-term alignment.

Potential influence of road geometry is included into the scoring system by way of a 'dummy' to indicate the type of road alignment. The appropriate matrix with weighting for each defect is then used to determine the condition score for the pavement.

3.6.5. Modification for avoidance of likely defects from material testing

Once modelling for Pilbara haul roads has been completed for this project it may be possible to modify the bounds of optimal materials to improve selection for functional design purposes. This can then be used in conjunction with the defect progression score to assist in the decision of when to program maintenance activities.

3.7. Construction/Design advice for use of waste material

A search of the available literature suggests there are some measures available that may assist with the construction of pavements with less than ideal materials. As such some recommendations for the construction of haul road pavements in general but specifically wearing course layers is to be included. This is especially significant if a choice to apply what can be expensive treatments is made, or indeed if large amounts of capital expenditure is invested in sourcing borrow material involving large amounts of haulage and/or specialist equipment.

4. RESULTS

Within the project there were a range of results that combined provide a thorough assessment of the performance of an unsealed pavement, with all calibrated toward a theoretical haul road one kilometre long and of zero vertical gradient. The results for each element of the investigation shown in Figure 3.1 are presented in the proceeding.

4.1. Laboratory Testing

The regime described in Section 2.1 has been carried out in such a way so as to define both a range of untreated materials from 3 mines and also then explore the improvements that could be made by way of material improvements.

4.1.1. Definition of Materials Currently Employed

A summary of all final testing completed on untreated materials is displayed in Table 4.1. Results involving plotting and calculation can be found in Appendices A through G.

Table 4.1: Untreated materials testing summary

Mine	Material	PSD Result ***	Gc	Sp	OMC (%)	MDD ($tonne/m^3$)	LL (%)	PL (%)	PI (%)	LS (%)	UCS (kPa)	CBR (%)	% MDD Compact	% MDD - After Soak	M_R (MPa)	% MDD^^
Mine A	Windrow 1	NO	25.0	12.0	8.4	2.8	19.7			0.4		163.1	104	114	628	102
	Windrow 2	YES	24.0	10.0	9.2	2.5	24.3			0.6		109.0	96	149	555	100
	Detridal	NO	37.0	124.0	7.3	3.0	27.2			7.5		121.0	99	102	654	97
Mine A 2011	Cohesive	NO	32.6	129.6	11.8	2.5	23.0	15.5	7.5	7.2	65.0					
	ROM Tip Head	NO	22.4	37.6	8.8	2.5	19.4	13.2	6.2	6.0	55.0					
	HV Access	YES	31.2	88.0	8.8	2.6	17.7	11.4	6.3	4.3	40.0					
	Surface Fines	NO	-	327.2			21.7	12.4	9.3	5.6						
Mine B	Alluvial	NO	23.0	32.0	13.6	2.1	35.3			1.0		42.8	110	-	581	103
Mine C	Top	No	31.0	85.0	9.8	2.9	24.3			5.0		149.0	113	123	489	95
	Middle	YES^	32.0	126.0	9.2	2.7	26.3			7.4		12.1	107	119	787	92
	Subsoil	YES			10.5	2.5	*			*		5.1	104	127	**	

*CBR and Resilient Modulus testing revealed to be inappropriate base material so not tested for these parameters
**Too weak to register a value in testing
*** Noting if MRWA base material grading met
^ Had one sample passing rrequirement and one out, ultimatley if combined should be okay so grading passed as okay, especially Middle South which has been considered a little anomylous
^^ Average of moisture content before and after testing

4.1.2. Treated Materials

A summary of all treated materials is shown in

Table 4.2. All plots and raw data for these tests can be found in Appendices A through G.

Table 4.2: Treated materials testing summary

Mine	Material	PSD Result ***	Gc	Sp	OMC (%)	MDD (tonne/m^3)	LL (%)	PL (%)	PI (%)	LS (%)	UCS (kPa)
Cohesive Treated	2% Lime/4% Fly Ash	As per Parent Material	31.7	45.3	Inferred from Parent Material Results		26.6			2.0	640.0
	2% Cement		31.7	154.0			22.6	17.0	5.6	6.8	690.0
Rom Tip Head Treated	Mechanically		31.1	133.5			20.9	17.2	3.7	5.2	
	2% Lime/4% Fly Ash		23.8	18.5			23.8			1.6	150.0
	2% Cement		23.8	69.5			20.8	13.8	7.0	6.0	610.0
Mine A Hv Access Treated	Mechanically		30.3	106.9			16.4	11.1	5.3	4.7	41.0
	2% Lime/4% Fly Ash		30.7	39.9			17.7			1.6	515.0
	2% Cement		31.1	61.6			16.0			2.8	744.0

The testing most impacting functional design is that of the Atterberg tests. For this reason the modifications of these limits due to addition of stabilisers is quantified in the figures below. Figure 4.1 and Figure 4.2 show the modification of the liquid limit and plasticity index due the addition of the stabilisers noted in Section 3.

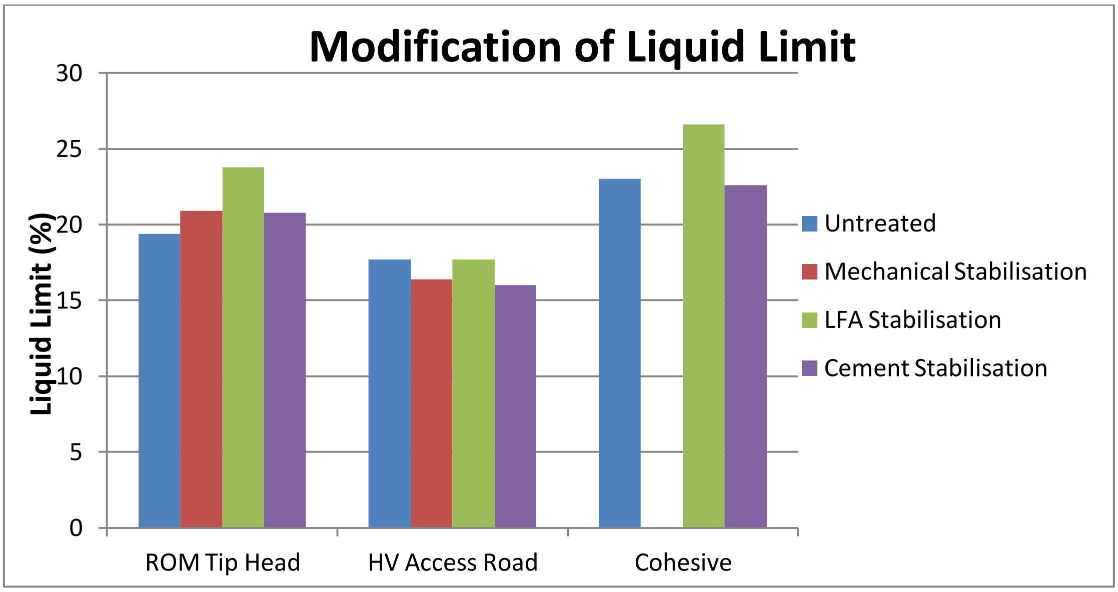


Figure 4.1: Changes to Liquid Limit due to addition of stabilisers

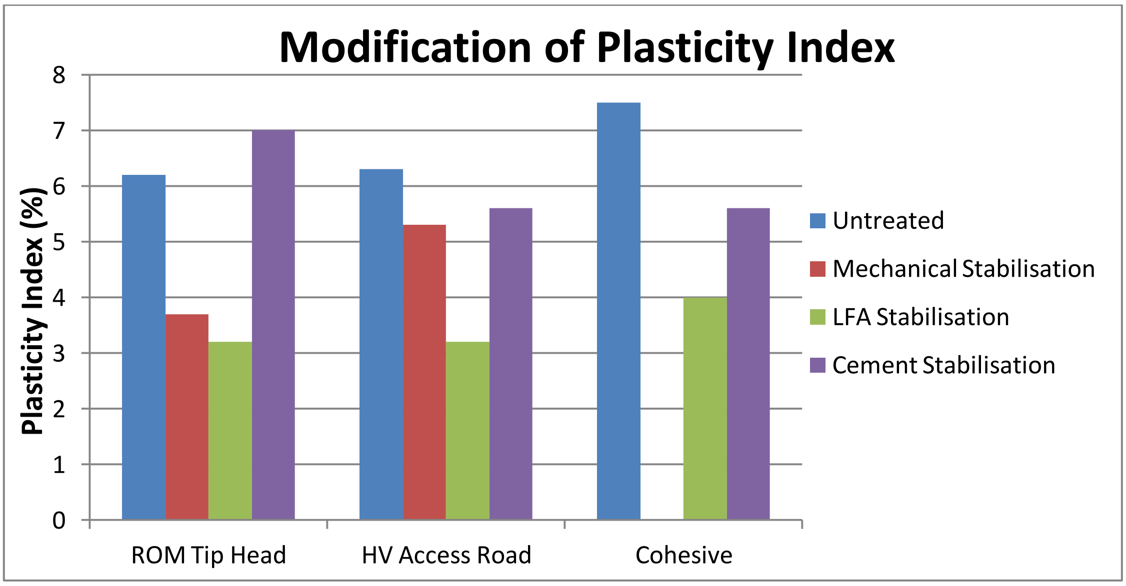


Figure 4.2: Changes to Plasticity Index due to addition of stabilisers

In addition to the consistency limits the linear shrinkage was tested, with the results being shown in Figure 4.3, which allows calculation of shrinkage product values, as presented in Figure 4.4. It is the changes in the shrinkage product values that are directly applied in the modelling discussed in Section 3.4.

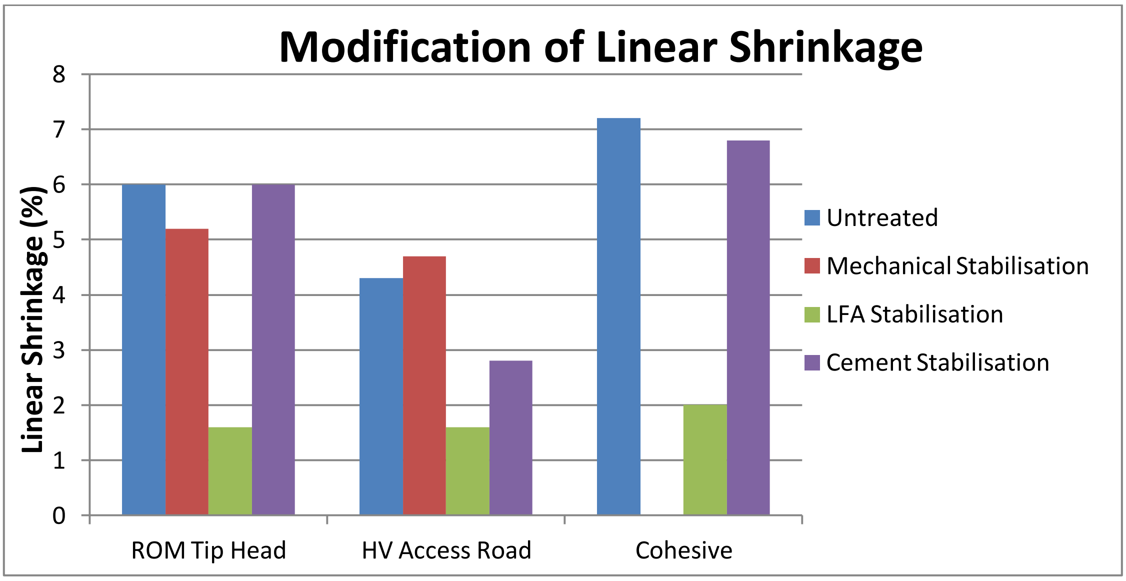


Figure 4.3: Changes to Linear Shrinkage due to addition of stabilisers

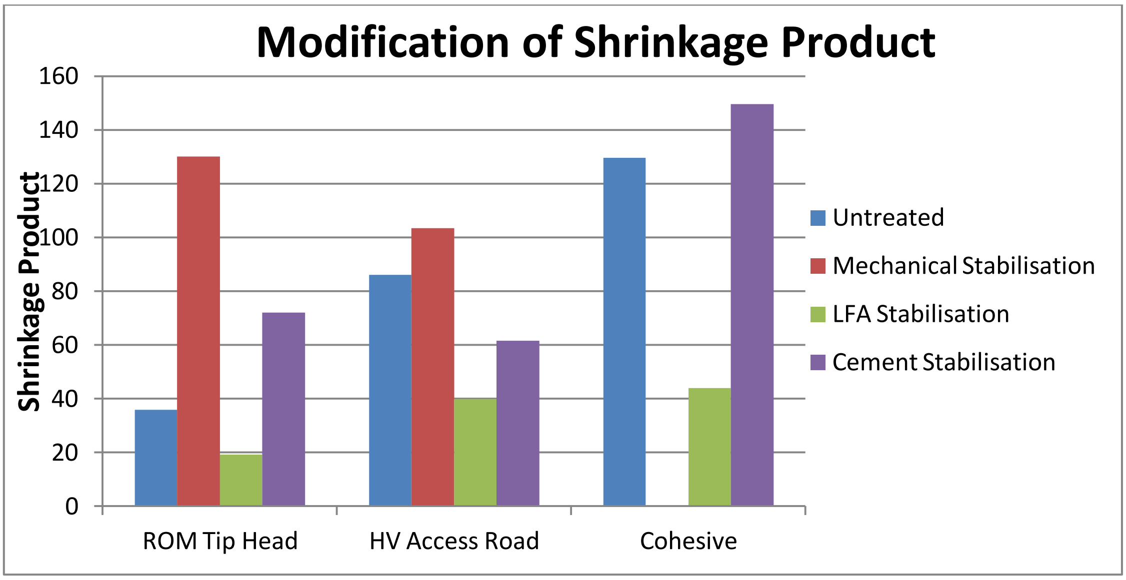


Figure 4.4: Changes to Shrinkage Product due to addition of stabilisers

Finally, due to the need for the calculation of Grading Coefficient for use in assessing adherence to the functional design criteria the modification to this parameter due to the various improvement techniques is displayed in Figure 4.5.

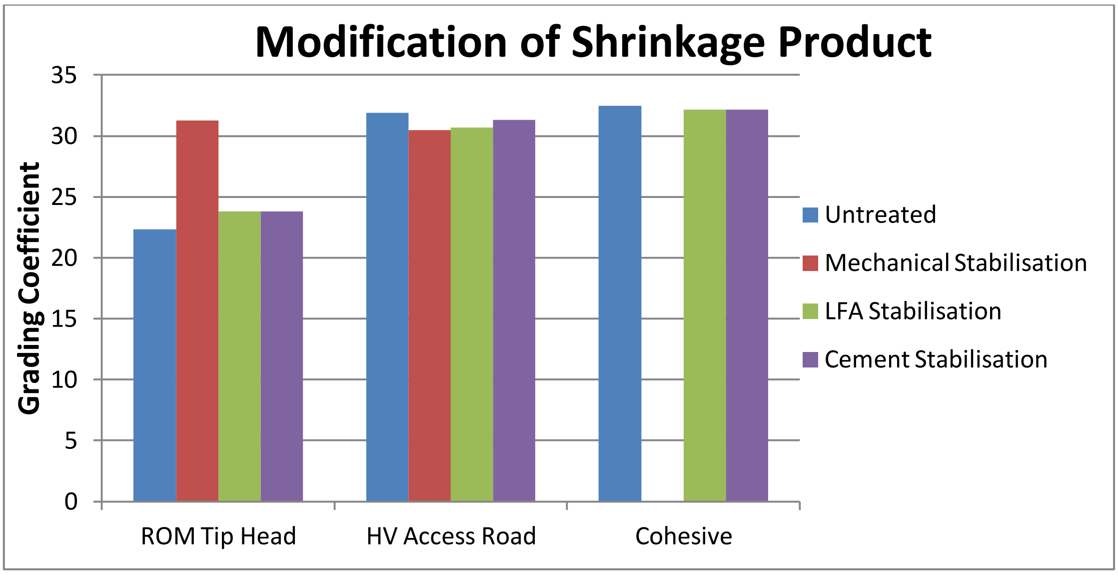


Figure 4.5: Changes to Grading Coefficient due to addition of stabilisers

4.2. Comparison for Functional Design

Utilising Figure 3.8 displayed and discussed previously all materials tested have been plotted. Figure 4.6 shows all untreated samples, Figure 4.7 presents the treated samples and Figure 4.8 displays all potential borrow samples. This presents the most basic comparison in terms of in-service performance, as discussed in Section 3.3.

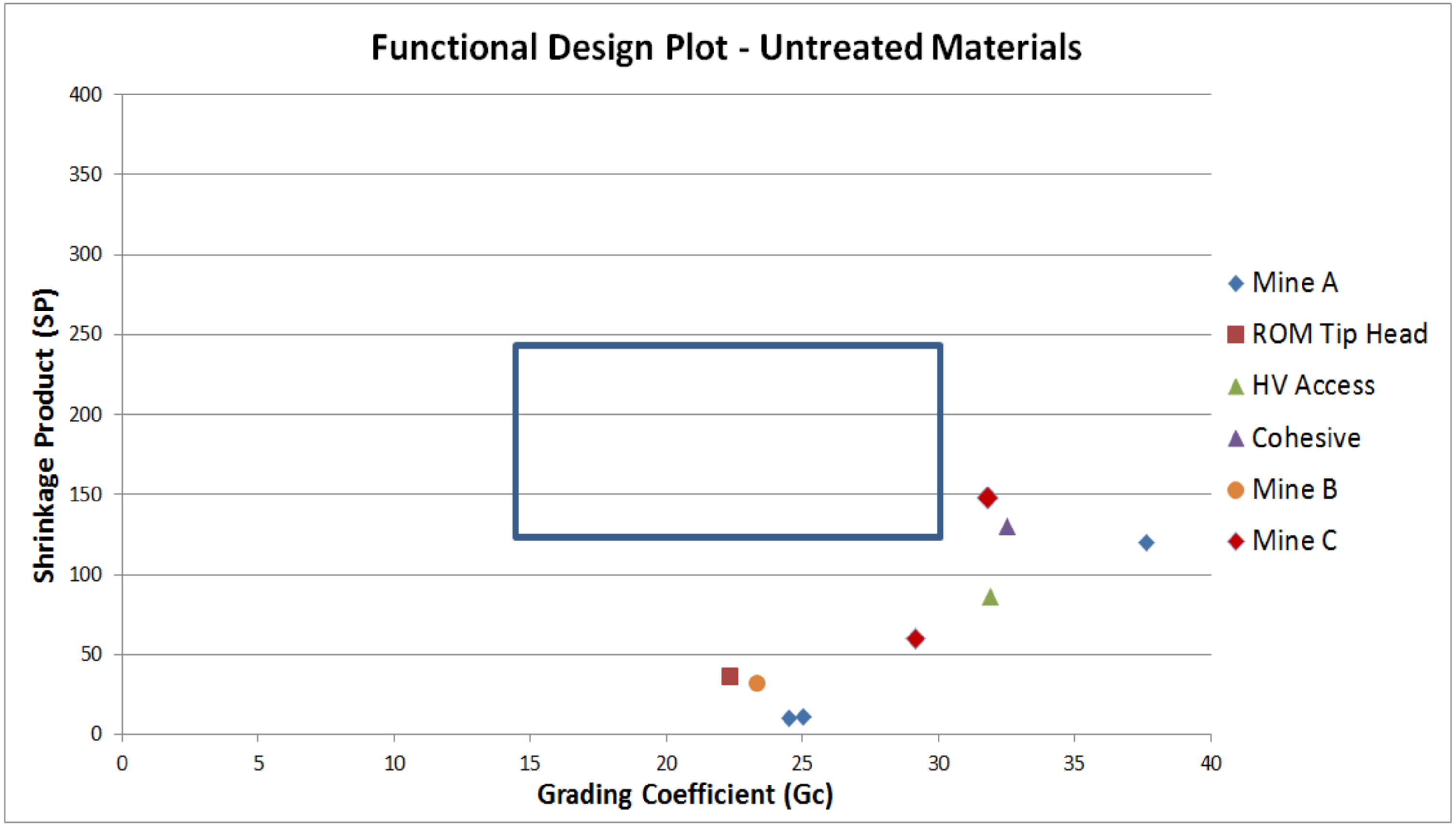


Figure 4.6: All untreated sample results plotted within functional design criteria

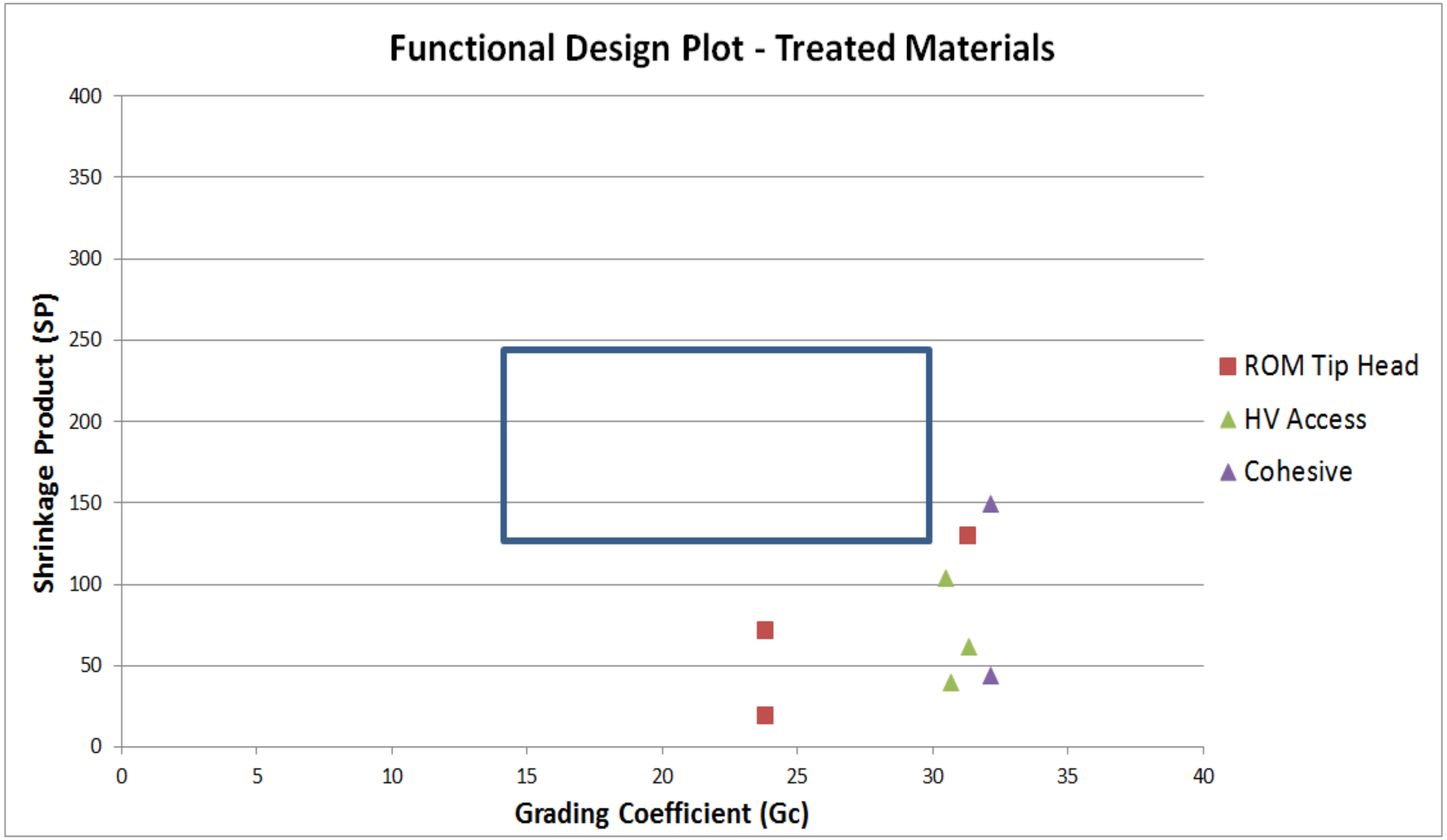


Figure 4.7: All treated samples results plotted within functional design criteria

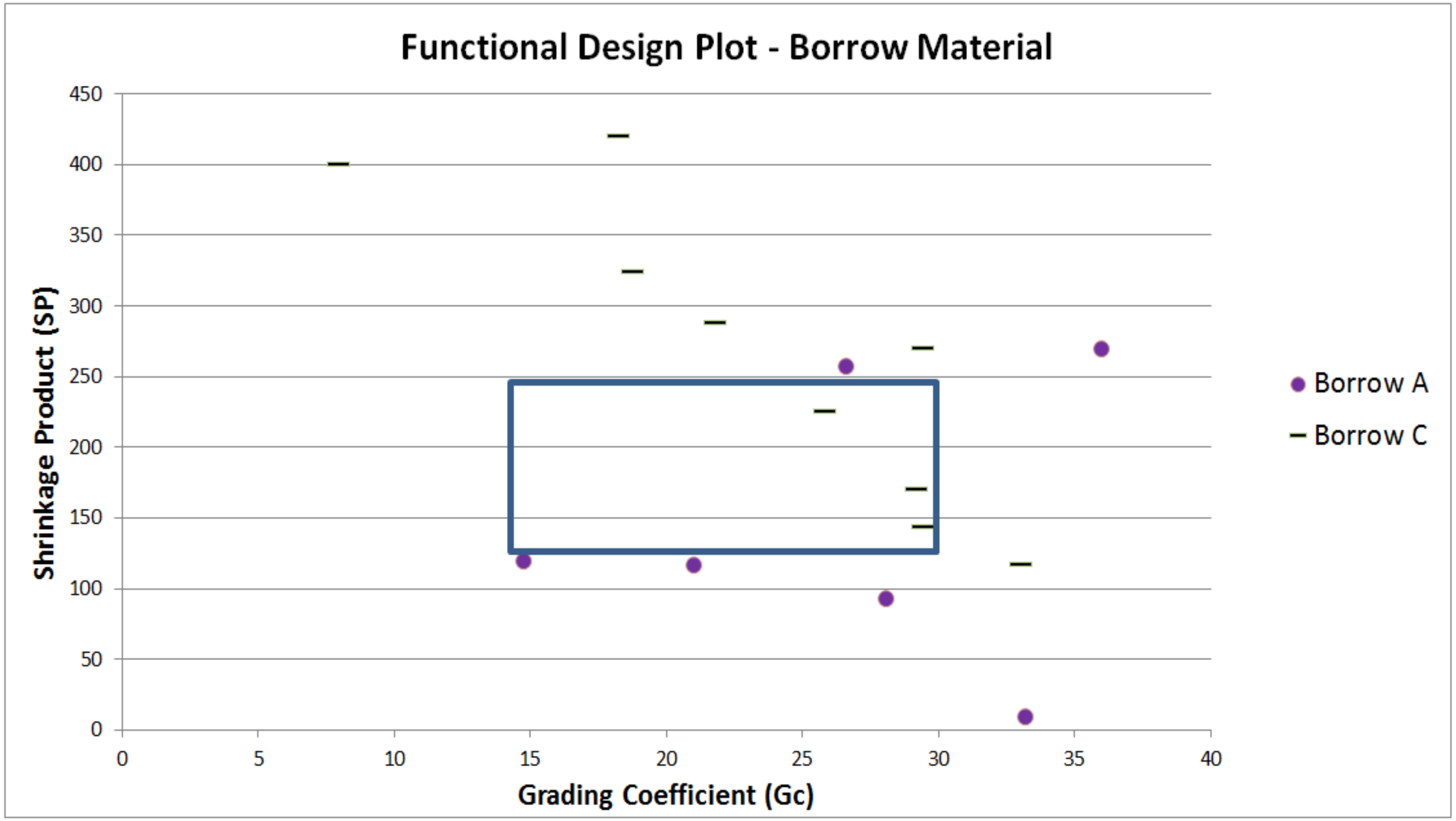


Figure 4.8: All potential borrow area results plotted within functional design criteria

4.3. Pavement Lifetime Modelling

4.3.1. Materials Currently Employed

4.3.1.1. Total Costs

In the following Figure 4.9 presents the potential production losses with Figure 4.10, Figure 4.11 and Figure 4.12 presenting the vehicle operating costs with no watering applied and the maintenance costs with and without watering applied, respectively. All values have been calculated assuming an average daily traffic of 240 vehicles, with no watering applied (except where noted) and a blading interval of 90 days.

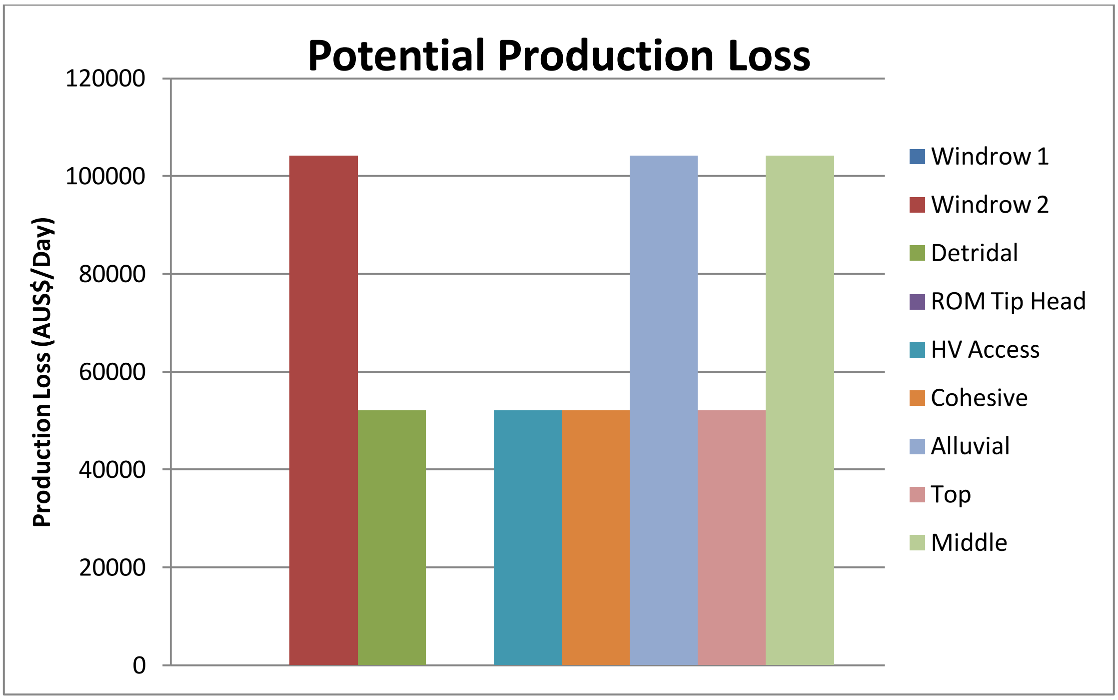


Figure 4.9: Potential production losses for sampled untreated materials

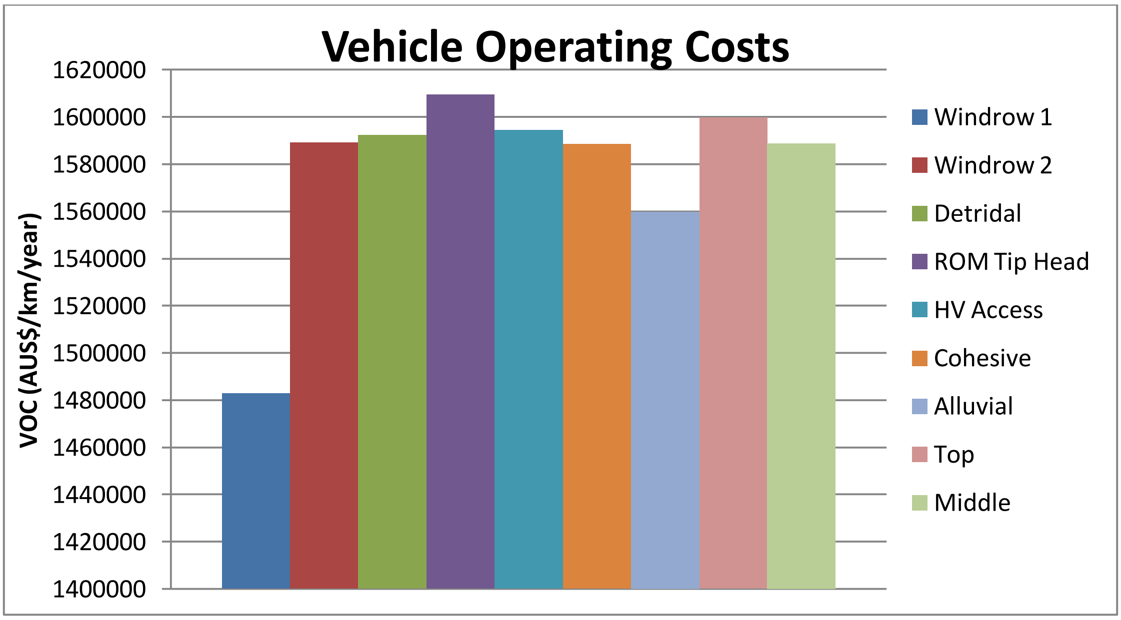


Figure 4.10: VOC for sampled untreated materials

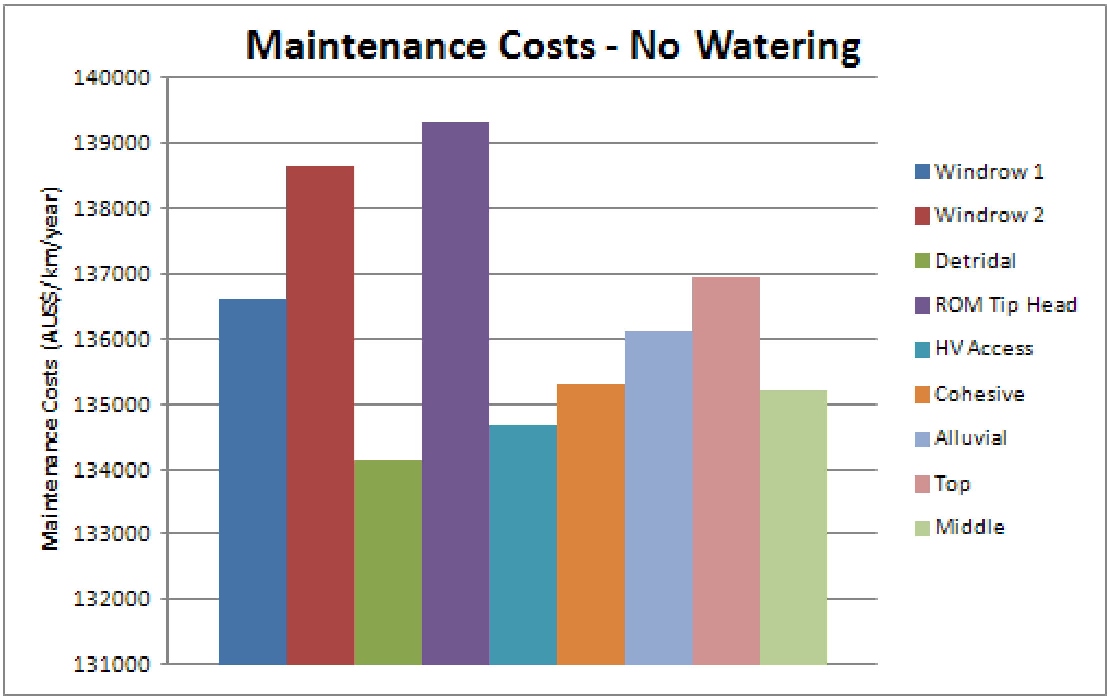


Figure 4.11: Maintenance costs for sampled untreated materials – no watering

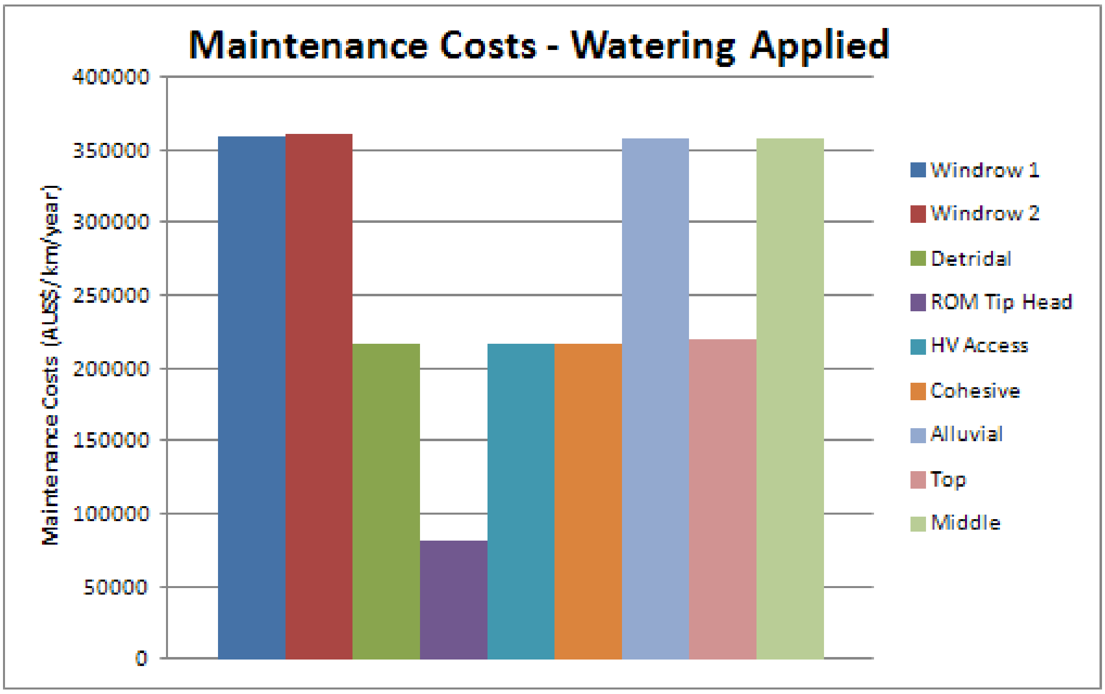


Figure 4.12: Maintenance costs for sampled untreated materials with watering applied

4.3.2. Treated Materials

4.3.2.1. Total Costs

The following Figures present annual and daily (as deemed appropriate for data type) roughness progression (Figure 4.13), production losses (Figure 4.14), VOC (Figure 4.15). Additionally the maintenance costs are presented for a scenario of 240 vehicle passes per day with a blading interval of 90 days and no water sprays applied in Figure 4.16 and also with watering applied in Figure 4.17.

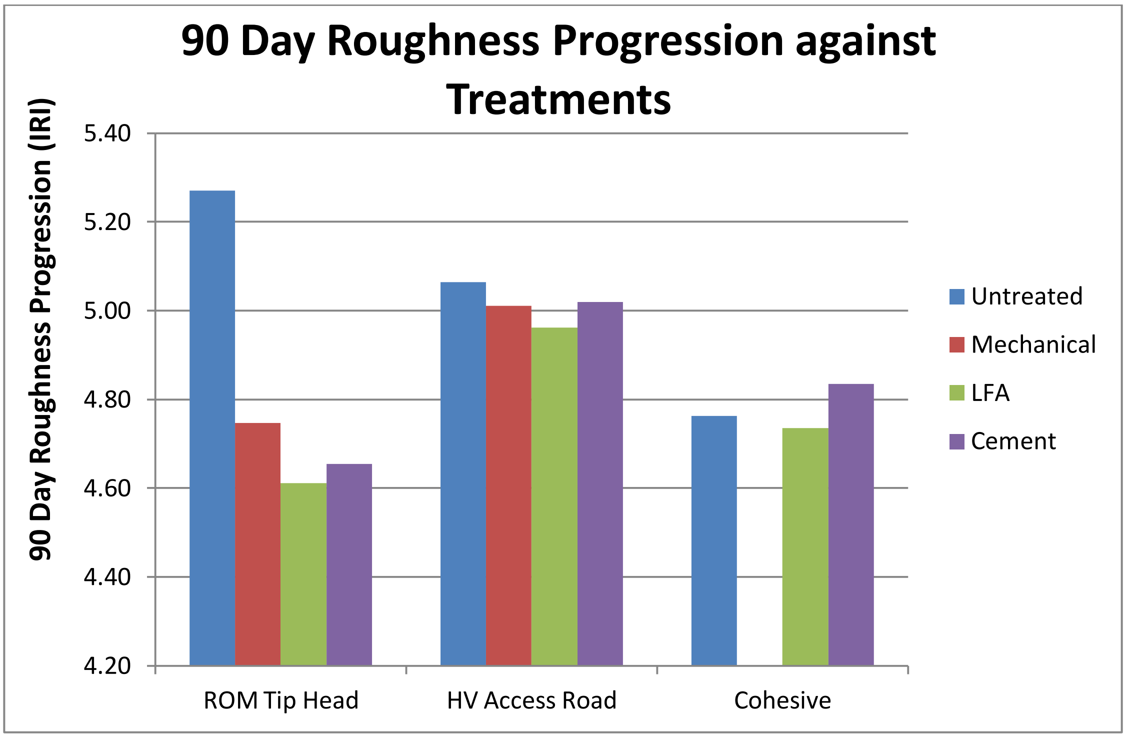


Figure 4.13: 90 day roughness progression for each raw and treated materials

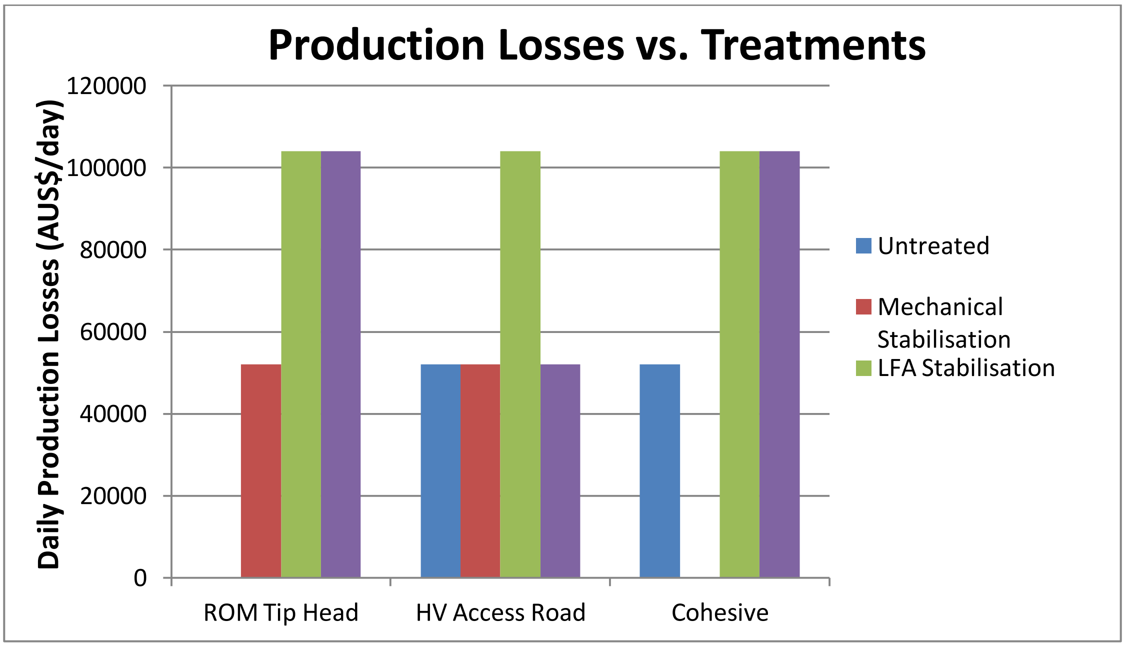


Figure 4.14: Estimated maximum daily production cost for respective raw and treated materials

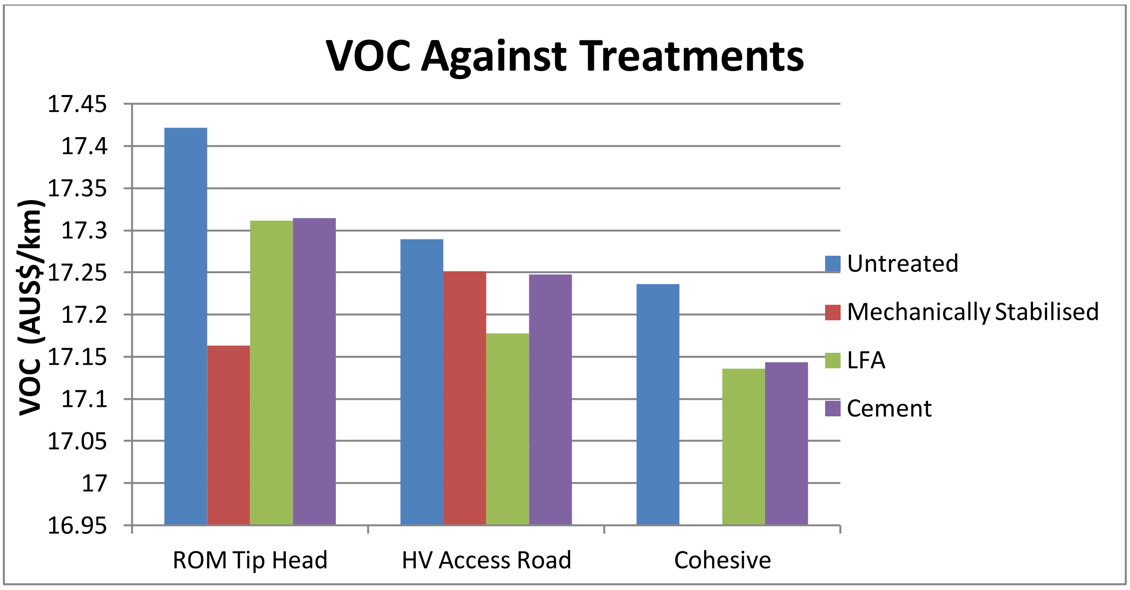


Figure 4.15: Total averaged VOC costs for raw and treated materials

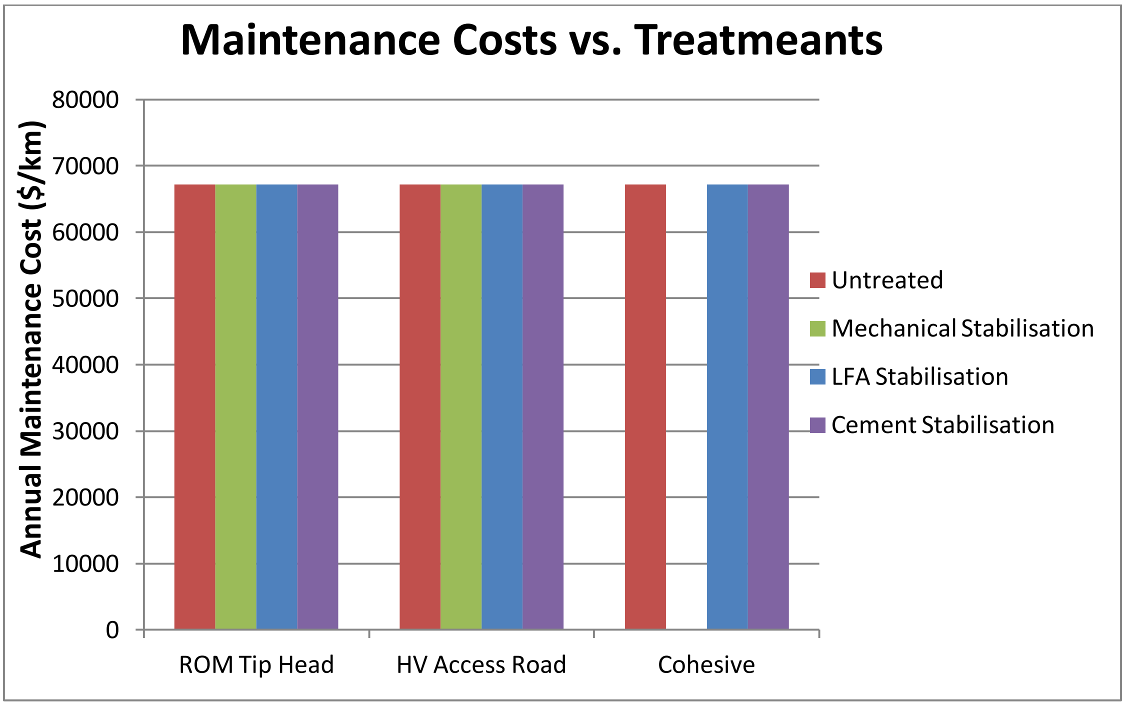


Figure 4.16: Total averaged maintenance costs for raw and treated materials – non-watered

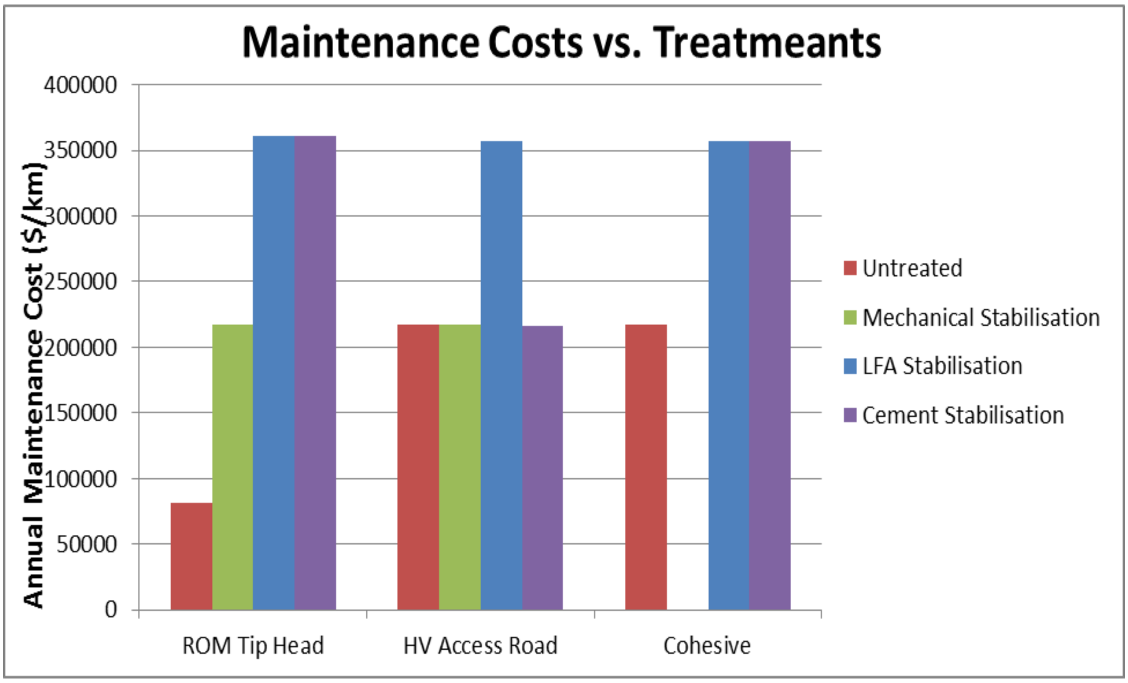


Figure 4.17: Total averaged maintenance costs for raw and treated materials – watered

4.3.3. Borrow Materials

The following presents the variances in production losses, VOC and maintenance costs due to differing borrow materials considering varied maintenance inputs and an average daily traffic of 240 vehicles. Figure 4.18 (potential production losses) and Figure 4.20 (maintenance costs) are presented with logarithmic regression functions plotted, whereas Figure 4.19 is a smoothed scatter plot.

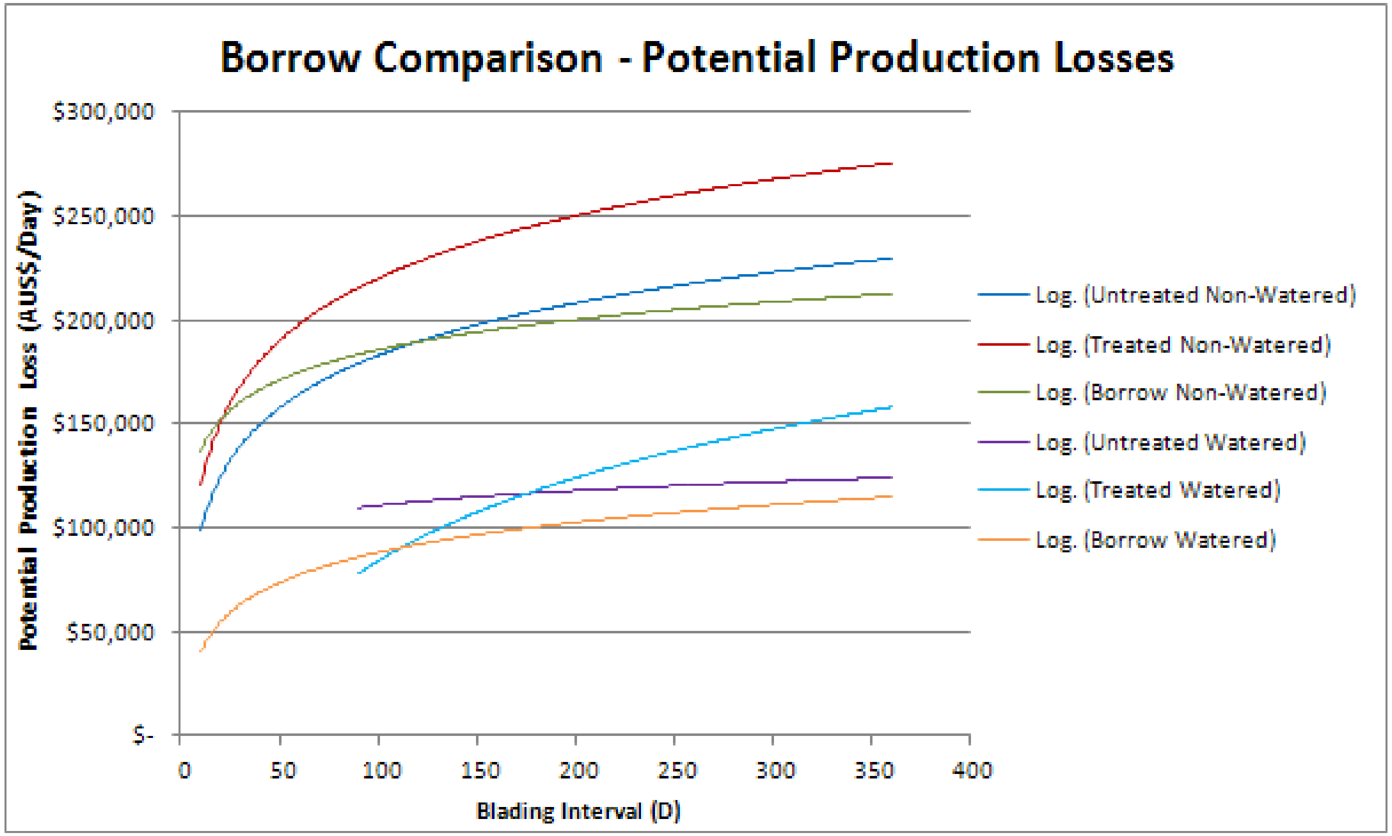


Figure 4.18: Borrow material comparison – potential production losses

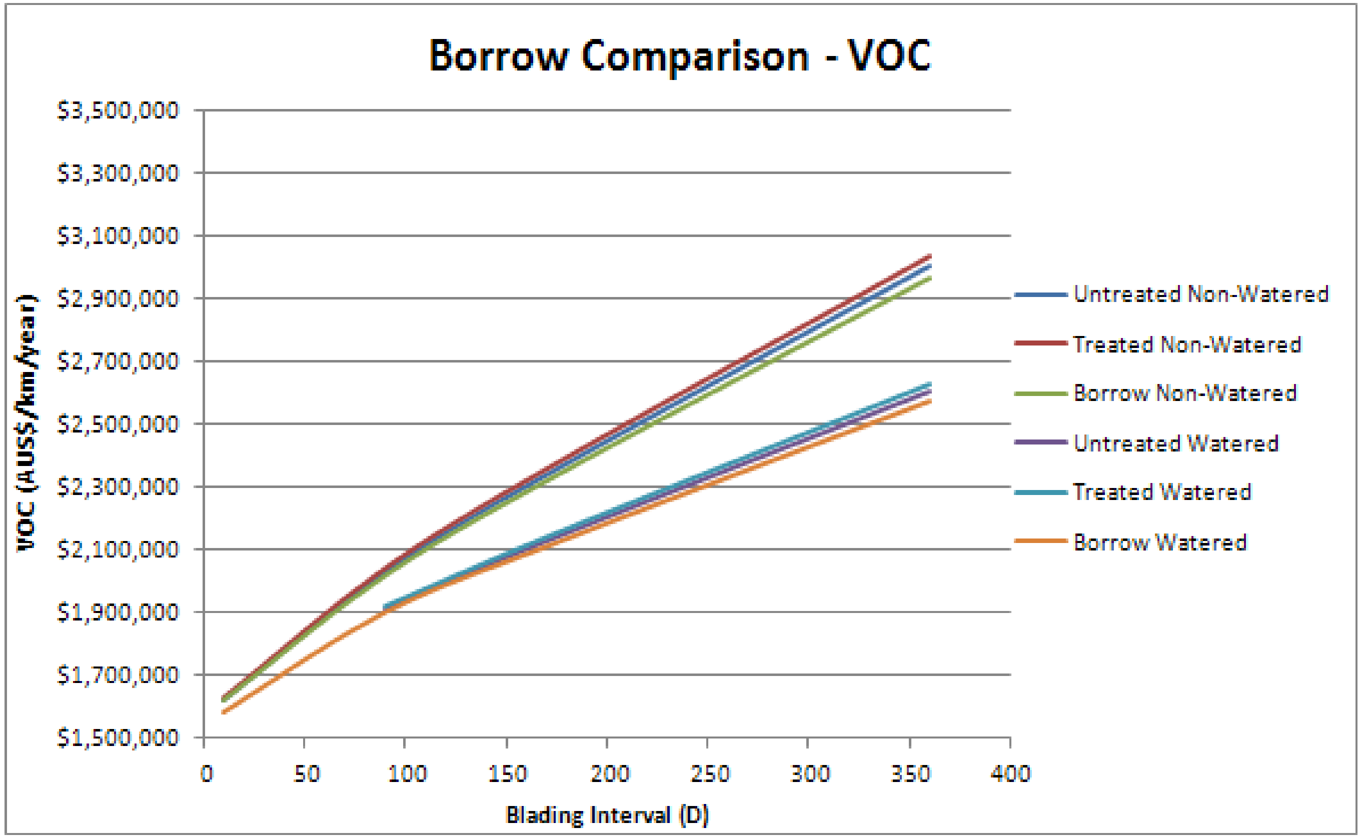


Figure 4.19: Borrow material comparison – VOC

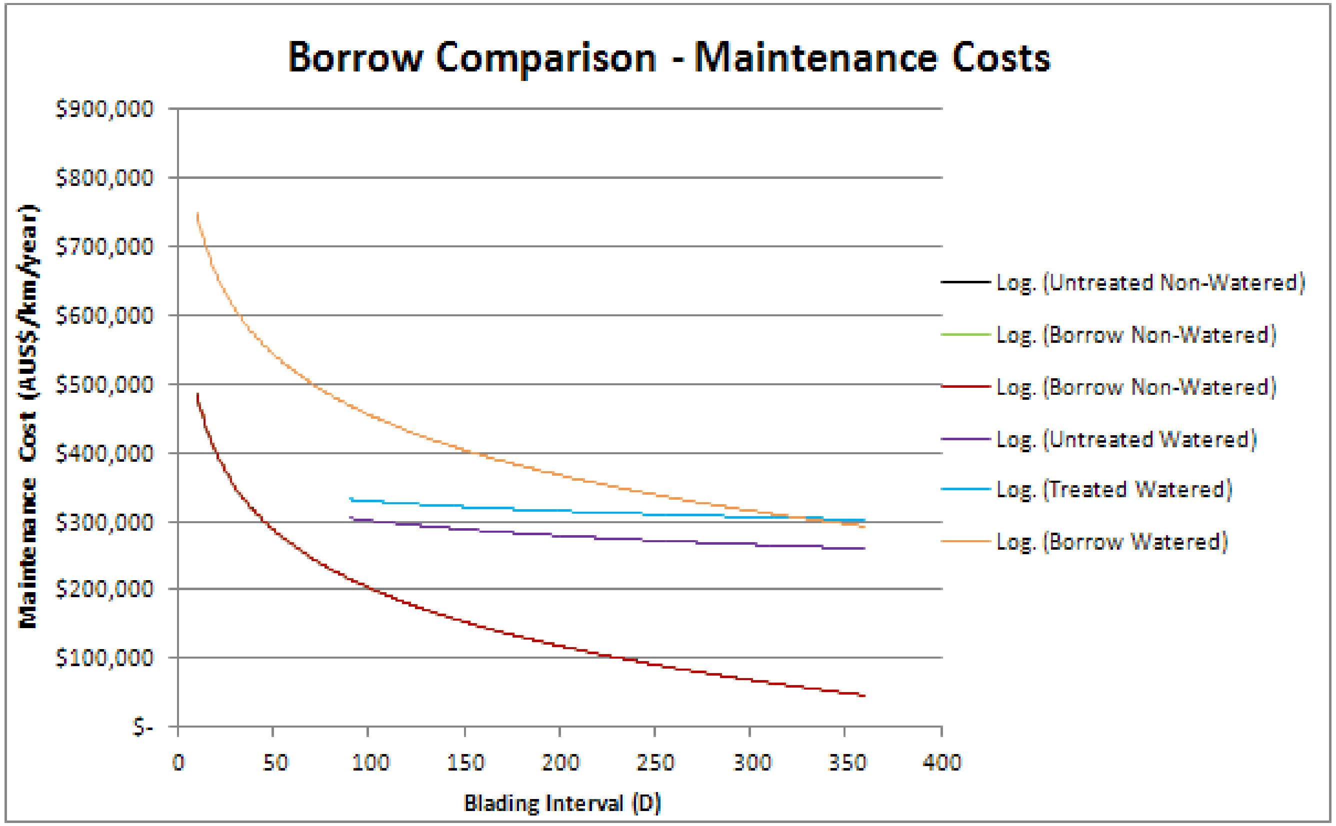


Figure 4.20: Borrow material comparison – maintenance costs (note all non-watered material types have identical curves)

4.3.4. Material Construction Treatment Costing

The total cost of application for each treatment per kilometre is outlined in Table 4.3.

Table 4.3: Overall material treatment pavement construction costs

	Additive Details			Construction Equipment										
Additive	**Unit Cost (AUS$/tonne)**	**Depth (m)**	**Cost / km****	**Stabilizer**	**Grader**	**Water Cart**	**12T Vibrating Roller**	**12T Multi-tyre Roller**	**12T Pads Foot**	**Dump Truck**	**Loader**	**Plant Cost (AUS$/km)**	**Productivity*** (m/day)**	**Cost (AUS$/km)**
Lime	$ 300	0.3	$ 118,800	1	1	1	1	1	-	1	1	$ 45,715	500	$ 402,115
Fly Ash	$ 300	0.3	$ 237,600											
Cement	$ 300	0.3	$ 118,800	1	1	1	1	1	-	1	1	$ 45,715	500	$ 164,515
Borrow****	*	0.3		-	2	1	1	-	-	5	1	$ 92,405	500	$ 92,405

* No cost for sourcing borrow for mixing, simply additional equipmet cost for load and haul, assumed 5 km haul

** Utilising the percentages trialled in laboratory (2:4% Lime:Fly Ash, 2% Cement, 20% borrow)

*** Assuming a 30 meter wide haul road

**** Requires moving 900 m^3 of material a day

4.3.5. Variation with Blading Interval (D) and Traffic (ADT)

Presented below is the variation in VOC (Figure 4.21) and maintenance costs (Figure 4.22), averaged for raw and treated materials, including variation to watering and also Average Daily Traffic. Also shown in Figure 4.23 is the effect of blading interval for Average Daily Traffic on the total running costs for the untreated materials (average taken across all those sampled and tested). Maximum production losses for a fixed Average Daily Traffic of 300 are also shown in Figure 4.24 to highlight the variations observed due to materials differences and also watering applied for dust suppression.

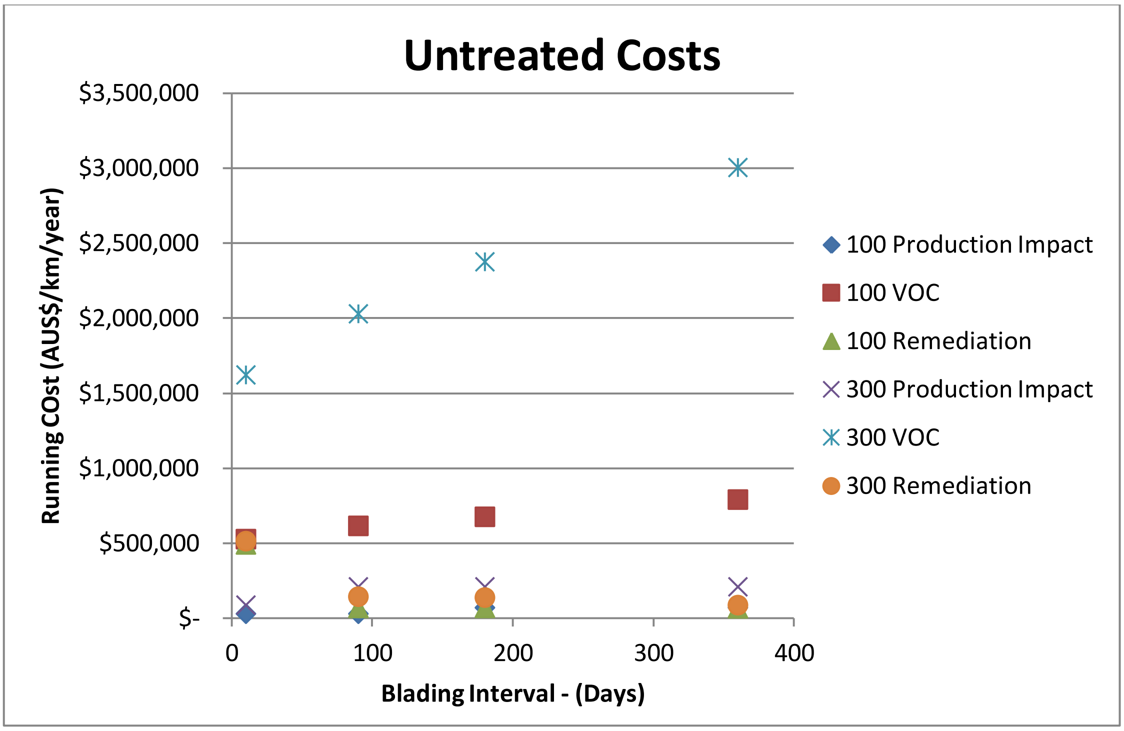


Figure 4.21: Total running costs for untreated materials for ADT of 100 and 300 (production impact shown as daily)

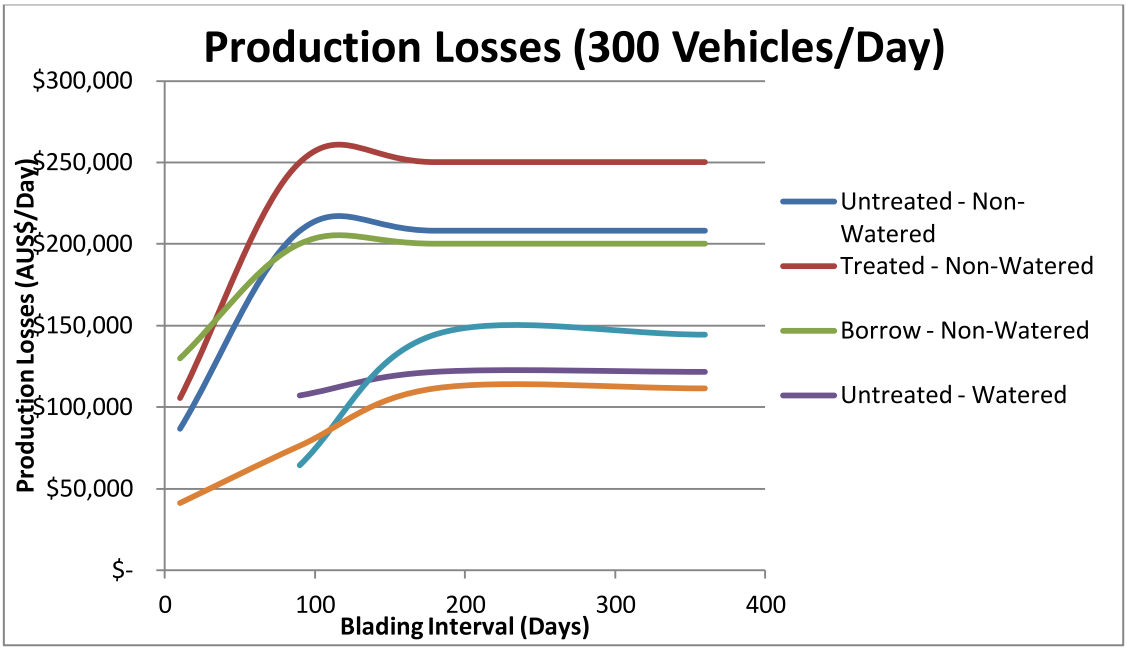


Figure 4.22: Variation in production losses against blading interval

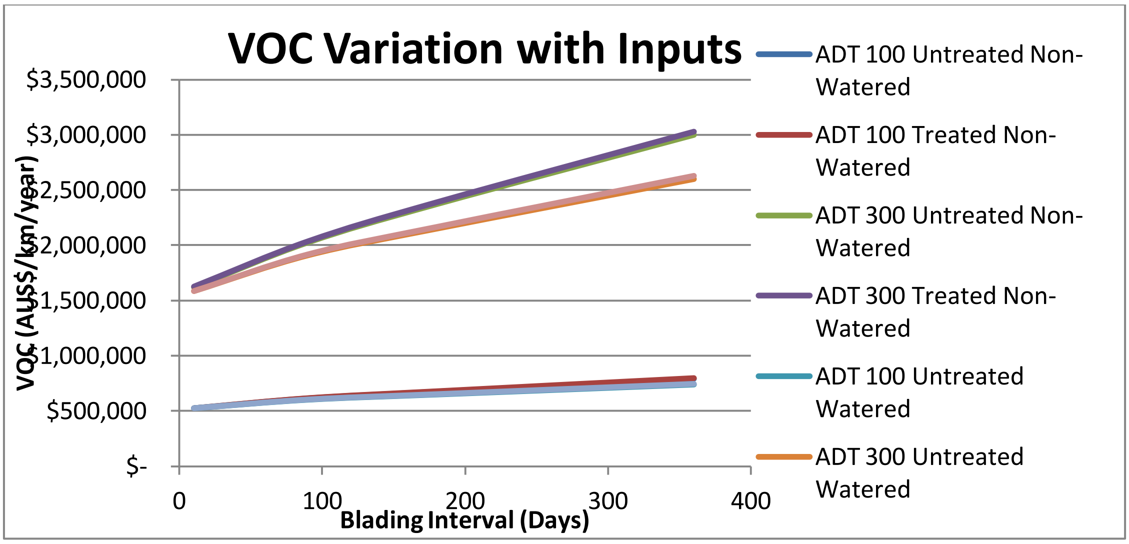


Figure 4.23: Variation of VOC with major inputs

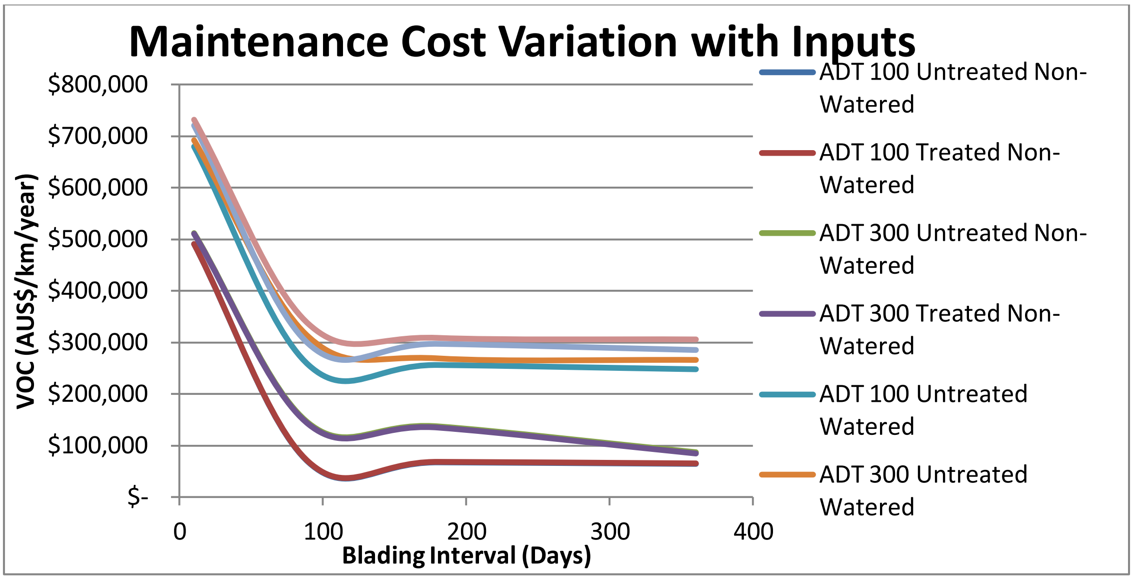


Figure 4.24: Maintenance costs variation with major inputs

4.3.6. Effect of Watering

The effect on the major categories of cost due to watering of the pavements surface is shown in the following Figure 4.25 presents potential production impacts, Figure 4.26 the affect of watering sprays on VOC and finally the variations in pavement maintenance or remediation costs are shown in Figure 4.27.

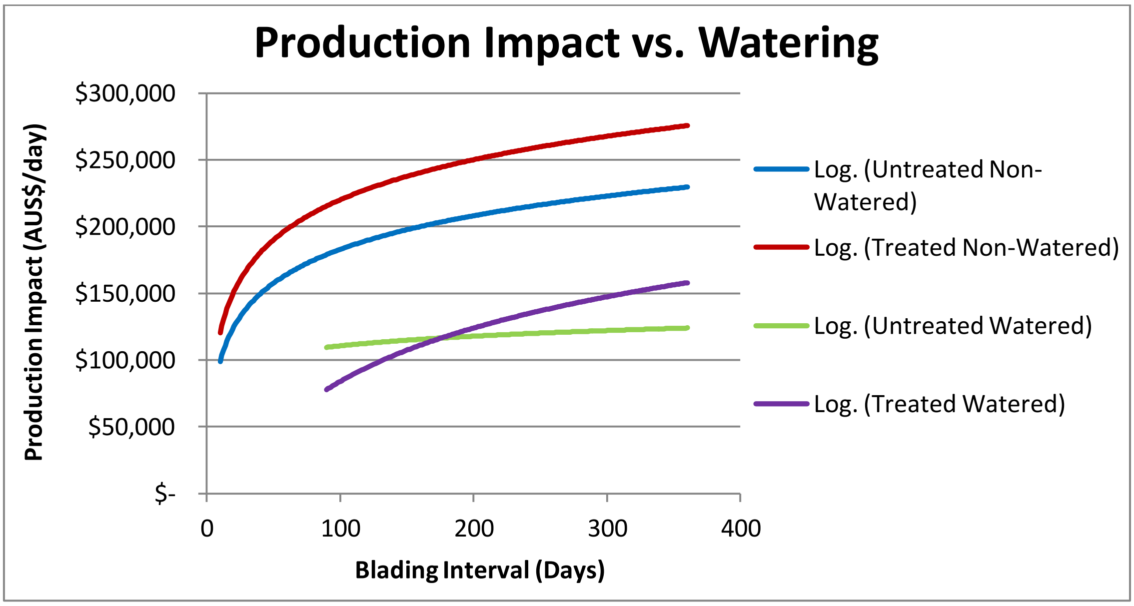


Figure 4.25: Production impact due to water sprays, ADT 300

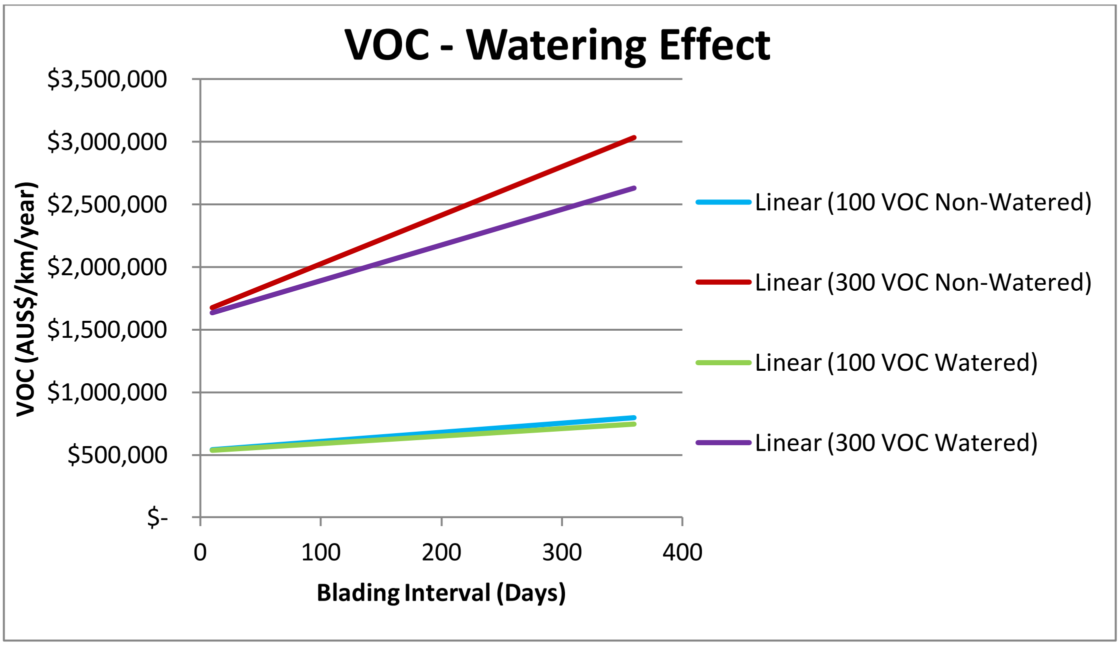


Figure 4.26: VOC impact due to water sprays, ADT 300

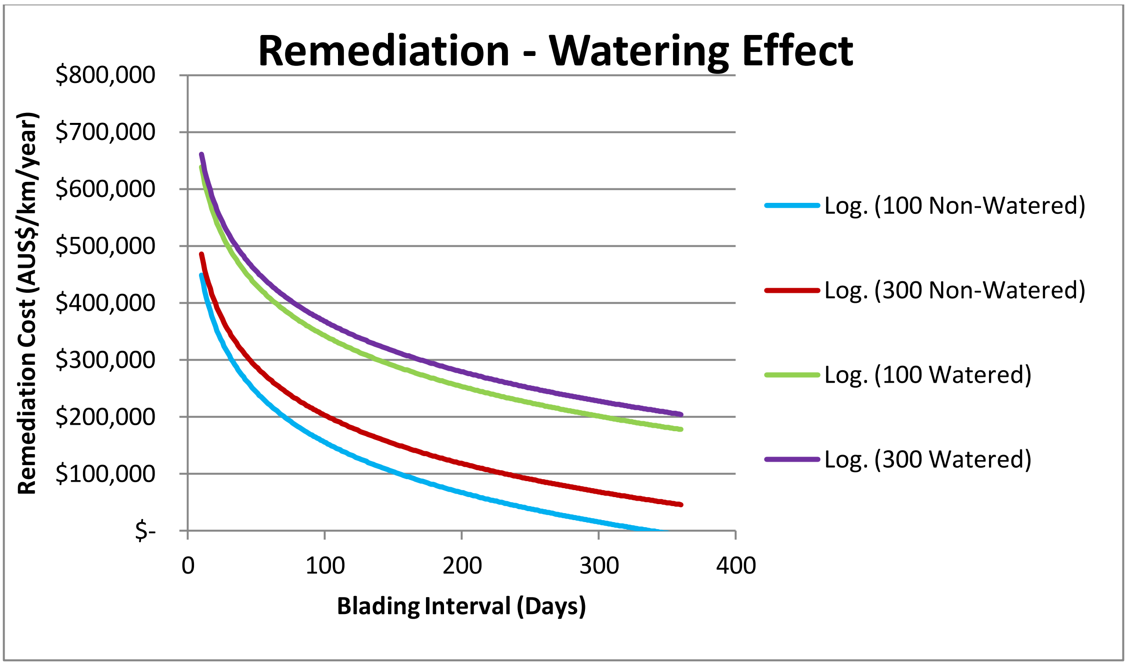


Figure 4.27: Remediation cost impact due to water sprays, ADT 300

4.3.7. Material and Maintenance Impact on VOC Components

The effect of material type and maintenance inputs (blading interval and water sprays) for the three major VOC components is quantified in the proceeding.

4.3.7.1. Tyre Life

The tyre consumption costs as impacted by variation in blading interval and the application of water sprays are illustrated in Figure 4.28.

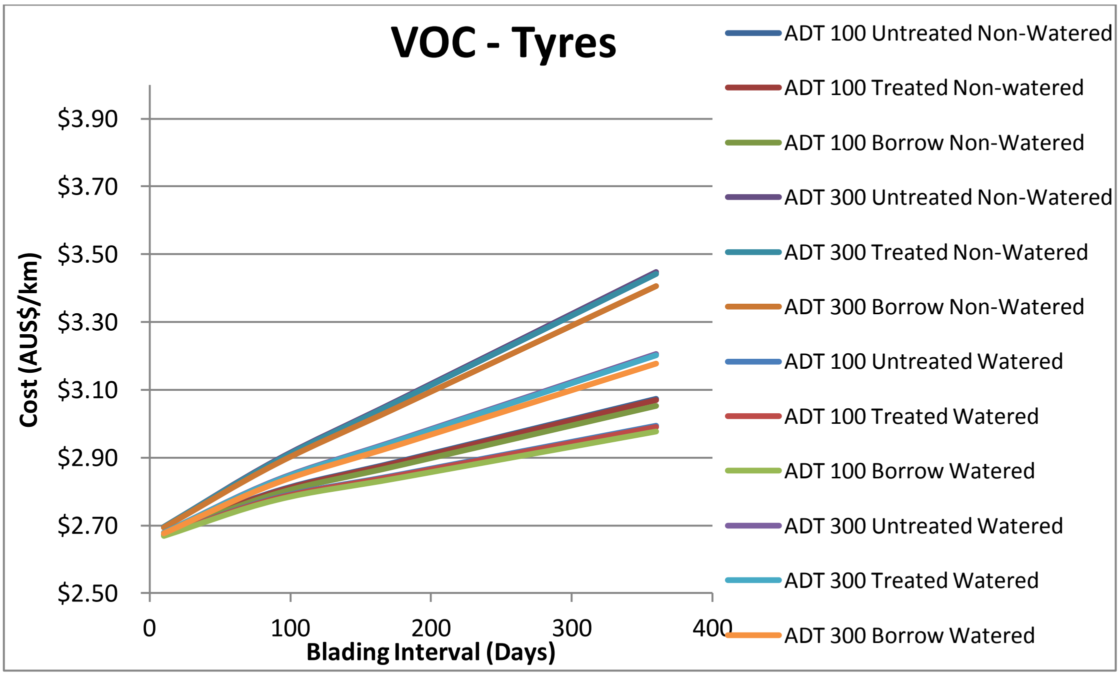


Figure 4.28: VOC – Tyre cost variation with blading interval (D) and traffic (ADT)

4.3.7.2. Truck Maintenance

The truck maintenance costs as impacted by variation in blading interval and the application of water sprays is shown in Figure 4.29.

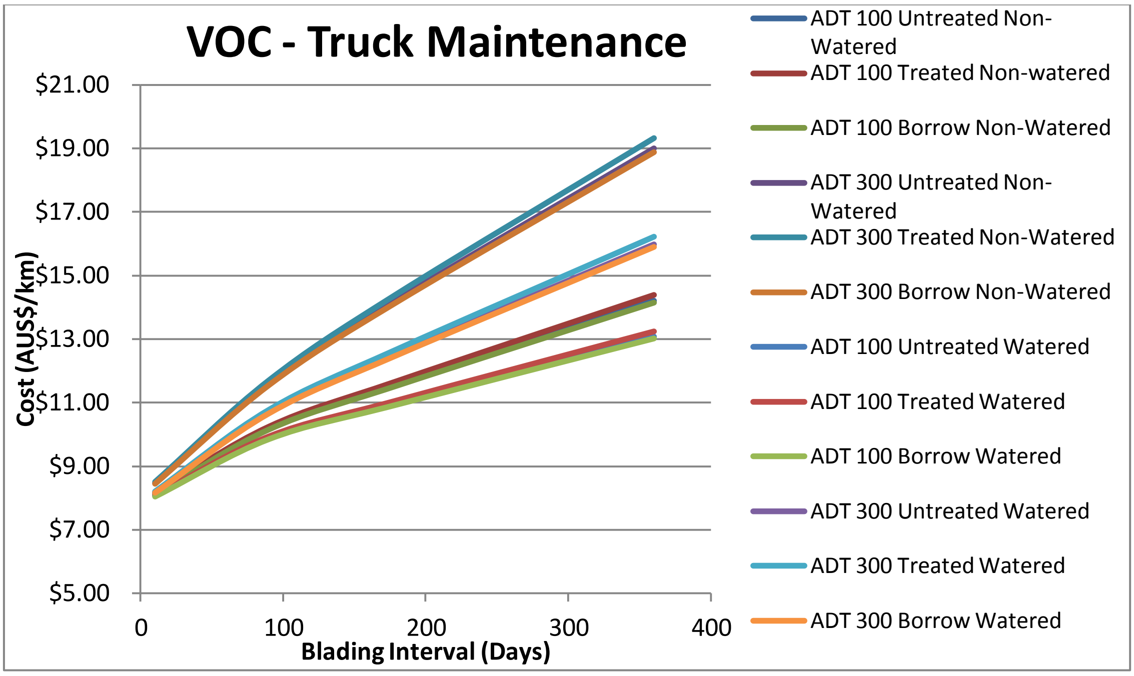


Figure 4.29: VOC – Truck maintenance costs variation with blading interval (D) and Traffic (ADT)

4.3.7.3. Fuel Consumption

The fuel consumption costs as impacted by variation in blading interval and the application of water sprays is presented in Figure 4.29.

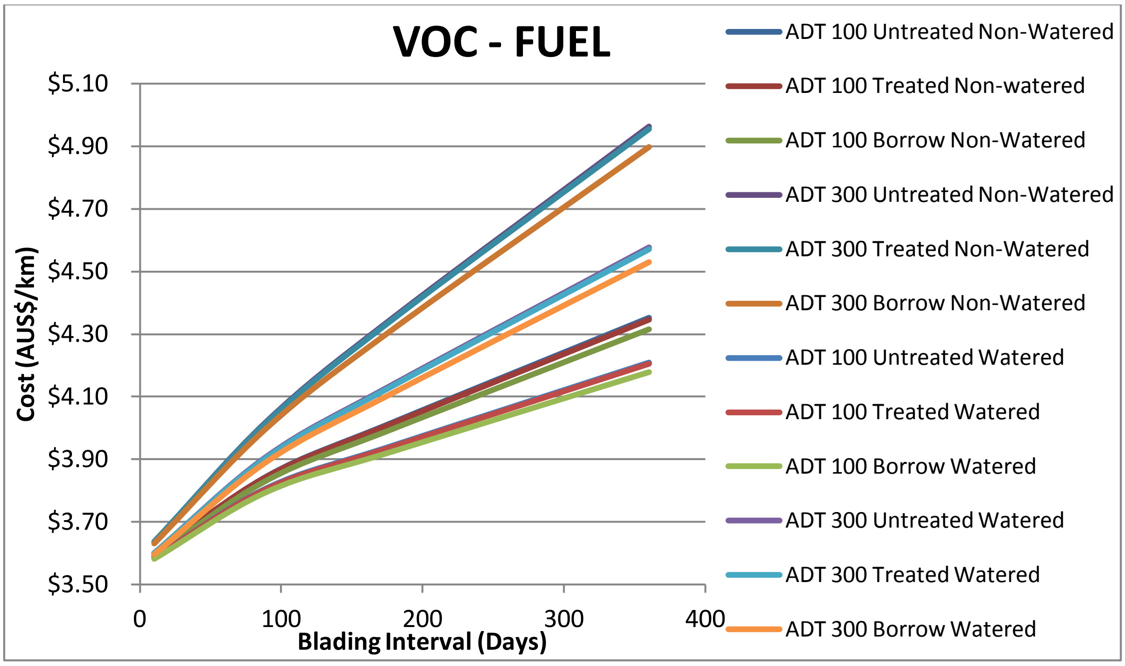


Figure 4.30: VOC- Fuel cost variation with blading interval (D) and traffic (ADT)

4.4. Defect Progression Score

The following presents the additional considerations to the current model and the matrices to be applied as described.

4.4.1. Overall Weighting of Categories

The weightings decided upon for each category are presented in Table 4.4. These have been derived as discussed in Section 3.6 and reflect the relative potential impact of each category.

Table 4.4: Relative scores given to each of the six categories within the defect scoring model

Assessment Category	**Relative Score**
Accident Potential	5
Impact Score	5
Defect Progression	2
Geometric Effect	2
Remediation Cost	3
Material Properties	1

4.4.2. Defect Progression

The progression potential for each defect category has been determined through the pavement lifetime modelling applied. The relative weightings, with and without watering sprays applied is presented in Table 4.5.

Table 4.5: Relative scores to be applied for defect progression

Defect Type	**Relative Score – No Watering**	**Relative Score - Watering**
Potholes	3	5
Corrugations	5	2
Rutting	2	1
Loose Material – Ravelling	5	4
Dustiness	5	1
Excessive Clay – wet skid resistance	2	5

4.4.3. Geometric Effect

The effect of road geometry has been discussed in Section 3.6.4, the relative impact for the three classes of road considered; long-term, short-term and ramp roads, is shown in Table 4.6.

Table 4.6: Relative scores to be applied for geometric type

Defect Type	Relative Score: Long-Term	Relative Score: Short-Term	Relative Score: Ramp
Potholes	3	2	4
Corrugations	5	3	4
Rutting	4	2	5
Loose Material – Ravelling	5	3	5
Dustiness	4	1	4
Excessive Clay – wet skid resistance	4	3	5

4.4.4. Remediation Cost

The relative pavement modelling has resulted in the relative scores for remediation of each defect category in Table 4.7.

Table 4.7: Relative scores to be applied for remediation cost

Defect Type	Relative Score
Potholes	2
Corrugations	5
Rutting	5
Loose Material – Ravelling	4
Dustiness	4
Excessive Clay – wet skid resistance	3

4.4.5. Material Properties

From observation of the properties of materials available for use within haul road wearing courses in the Pilbara region, the following weightings presented in Table 4.8 are suggested to reflect the likelihood of each defect type occurring.

Table 4.8: Relative scores to be applied for material properties

Defect Type	**Relative Score**
Potholes	1
Corrugations	3
Rutting	1
Loose Material – Ravelling	3
Dustiness	3
Excessive Clay – wet skid resistance	2

5. DISCUSSION

5.1. Laboratory Testing – Definition of Materials Employed

5.1.1. Introduction

The laboratory testing of materials for this project was carried out to define the range of materials available for haul road construction on Pilbara iron ore mine sites. There was no published data for the materials used on mines within this region prior. Accordingly a comprehensive set of tests were employed in order for boundaries to be set for the design criteria and pavement life modelling discussed later. The suitability of the materials for road pavement construction is discussed in the following with a focus on the impact each property has on the likely performance of a haul roads wearing course.

This project is focussed on the functional design and construction of haul roads, which has a large bearing on the overall performance on efficiency of any mines haulage process. The tests have been chosen and conducted in such a way so as to define the usefulness of different materials.

5.1.2. Index Tests

The index tests carried out for this project provide a sound illustration of the materials considered. They are critical in distinguishing between the worth of each material to be used as or within a surface course, due to the chosen criteria primarily relying on simple Atterberg tests. In the following each result will be compared to relevant standards or recommended values that exist for general road pavement or haul road pavement design. This has been completed to define the quality of the materials sampled in relative terms to those commonly encountered within general pavement construction.

5.1.2.1. Particle Size Distribution

Strong indicators of the suitability of a material for road construction can be gained from observation of the particles size distribution (PSD). This is especially true within this project due to the coarse nature of most materials, providing an explanation for potential ravelling or corrugation defects that may be observed. Six of the thirteen samples tested were deemed to meet Main Roads Western Australia (MRWA) standard for a base course. Early observations of the materials (particularly on-site) raised doubt as to whether many materials would register a distribution within these bounds due to the variable nature present, especially considering the inclusion of some over-sized particles common to blasted mine waste.

Although providing a good basis for future construction of haul roads on the sites involved, this result must also be criticised to some degree due to transportation restrictions meaning larger particles had to be excluded during the material sampling process. Generally this enforced the exclusion of particles that appeared over approximately 200 mm in the largest dimension. This complication is further compounded by particles over 19 mm not able to be included in testing procedures used in this project. There are some allowances made with MRWA placing an acceptable exceedance value of 20% on this restraint, although some doubt existed as to whether even this criteria would be met by some samples. Ultimately all did although note that the majority of materials were also observed to include much larger particles that were avoided in sampling for practical purposes, and hence the PSD results noting compliance with this requirement may be somewhat invalid. A decision was made that this situation occurred only rarely and was too hard to gauge and so testing continued. Regardless these particles could be easily removed during construction and as such would not affect performance.

The on-site observations made earlier provide evidence that it was widely accepted by those involved in the project that in some cases such large particles would alter the final result and compliance of the material. Knowledge of the dangers of including such materials in a pavement wearing course may be one of the most important points to be raised within this project, especially when considering tyre life estimates. It has been the experience of the author that this is generally the case with those over-seeing the construction of such roads but the lack of funding available and shortage of time for construction of all but the longest term haul roads means it is not possible to remove all over-sized particles. The nature of such bulk earthworks inherent in haul road construction is such that some large materials are occasionally placed too high in the pavement, often resulting in pot-holing and associated defects in a localised area.

5.1.2.2. Compaction Testing

This test is completed largely as a precursor to the strength tests, as it is both quite obvious and accepted that the maximum strength a soil can achieve is when it is at maximum dry density, and hence has been compacted at its optimum moisture content.

The optimum moisture content is also of high importance when it comes to construction. It is noted by the author and also by others involved in both construction and maintenance of the haul roads observed during the data collection for this project that often no significant watering effort is made during construction. This lack of care is not consistent with the practices accepted with construction of pavements when completed by engagement of a contractor and hence should be employed not just when constructing long-term roads during mine establishment but also during shorter-term roads and also during maintenance and rework. The effects on performance of the pavements due to this is compounded when it is noted that water sprays (applied via water cart), intended for dust suppression often involve such high application rates that the economic viability of the road surface is called into question. These facts further increase the likelihood of the most common defects (corrugations and ravelling) occurring. Although watering alone can have a marked effect on reducing corrugating, it would appear much roughness has resulted due to the poor integrity of the wearing course which is exacerbated by watering.

Many of the roads in this study experienced pot-holing, which can be attributed to the surfacing layer being periodically wet to above optimum and also poorly compacted. This is most obvious when observing a truck round a corner that has recently been passed by a water cart. Here it is clear to see the surface is often breaking apart and spreading under the tyres. Unfortunately it may be that the application rate of water for dust suppression cannot be reduced a great deal without modifying the material that has been utilised and so this adverse effect may be unavoidable due to the environmental implications of excessive dust emissions. This is an example of the impetus for the effects of pavement material modification to be considered.

5.1.2.3. Liquid Limit

The liquid limit is a strong indicator of the nature and cohesiveness of a soil, since a higher presence of clay will preserve the material performing as a solid at far higher moisture contents. This is the primary indicator of such behaviour when classifying a material under the Unified Soil Classification system. Any assertion from this value alone should be compared with the plastic limit also to gain a more complete view of the plastic characteristics of any soil, which is definitely important when considering the likely performance of a wearing course material. However it is worth noting that most of the soils tested in the laboratory did present reasonably high activity for the fraction of fines (particles passing the 425 micrometre sieve as per WA 120.2), with 5 samples meeting the boundaries set by Thompson (2009) for the liquid limit of 17 to 24 %. Four of the materials exceed the upper bounds by only a few percent; however the ‘Alluvial’ sample taken from Mine B far exceeded the maximum liquid limit value of 24 %, with a reading of 35.3 %. These results were not unexpected as the materials had been noted to have significant fines content during earlier testing and preparation for this test.

Overall the liquid limit should not have a huge bearing on material performance whilst the value is below an arbitrary figure. As noted above Thompson has set this at 24%; however it is possible that materials showing a liquid limit up to 35 % may perform satisfactorily. This is because only the fine fraction is tested and the actual amount of fines by weight could be quite low, meaning that depending on the actual grading of the material sound performance could be observed. Ultimately as the functional design criteria adopted in this project depicts, more information is required and hence plastic limit and linear shrinkage tests were completed and are discussed below.

5.1.2.4. Plastic Limit/Plasticity Index

Plastic limit tests were not completed for every material within this project, as the realisation of the impact this value had to material definition was only realised mid-way through laboratory testing. Overall the plastic limit was found to be relatively consistent, with values ranging from 11.4 to 15.5 %. The tests that were completed do show a good spread among the whole sample group and as such it can be expected that the others will not differ a great deal, with the exception of the ‘Alluvial’ material described above due to its noted high activity. The effect of the plastic limit of a material in isolation is usually largely ignored; however in this case it is quite possible that the water applied for dust suppression results in the plastic limit being exceeded within the immediate surface of the pavement. This means that regardless of the loading permanent deformation is certain to occur, which is only accelerated under further water sprays and trafficking.

More significant in the definition of materials is the plasticity index, which is generally considered with the liquid limit to provide for classification and character of the soil being investigated. Within the testing of raw materials in this project a range of 6.2 to 9.3 % is found. MRWA considers a maximum value of 10 % for use within a base layer (see Table 2.4), which means all sampled materials pass, however note that this is for sealed pavement layers that are not typically exposed to regular moisture applications through rain events or watering for dust suppression. Table 2.9 presents a recommended maximum of 8 % which is consistent with the fact the derivation of this guideline was for unsealed pavements and hence some reduced activity is recommended to preserve wearing course strength and skid resistance. Overall it is fair to say that by the criteria shown in Table 2.9 the materials tested were within an acceptable range, however the linear shrinkage should be closely observed to provide an idea of the activity of the fines present.

5.1.2.5. Linear Shrinkage

This measure is the most critical in defining the functional performance of the materials sampled, due to it being one of only two products within the calculation of the shrinkage product. To compound this importance the amount of material passing the 425 micrometer sieve is somewhat consistent between the materials sampled. Thus for the functional design criteria being used as the only means of comparison this measure is critical. It also plays a big part in roughness estimation in the pavement life modelling completed. For this reason the definition of an arbitrary range for linear shrinkage values is somewhat pointless, as the test result is far more complex than a simple pass or fail and is interlinked with other material properties within the overall modelling involved in the project.

There is also a lack of specifications for an acceptable value for linear shrinkage, Table 2.6 indicates just 2 % should be used as the limit for design purposes, however this has been sourced from MRWA specifications for sealed base layers and so has little meaning for the consideration of a material for use as an unsealed wearing course. Hence rather than comment on the value of linear shrinkage in isolation the reader is encouraged to read the discussion on pavement life modelling keeping in mind that the linear shrinkage is one of the key inputs.

5.1.3. Capillary Rise and Swell

Specific details of the Capillary Rise and Swell testing can be viewed in Appendix G1 with photo example of the treatments and in Appendix G2. What these results show is that the materials have become somewhat resistant to water and swelling when treated with cementitious additives. The capillary rise reduced significantly with all treated materials, which is an encouraging result in general, resulting in a reduction of the penetration of water into the pavement. These results show particular promise when combined with the absorption results that were seen to increase. In practice this means more water would be drawn into the wearing course, but yet the material will not swell (all treated materials experienced zero swell) which reduces breaks in the surface by way of improved shear resistance, in turn improving functional performance.

Overall the results from this test strongly indicate the positive aspects of stabilising or modifying a pavement wearing course. In addition to the expected reduction in roughness progression due to modification the pavement can also be expected to produce less dust, due to the absorption of the water sprays applied.

5.1.4. Strength Tests

The two pavement specific tests (CBR and Resilient Modulus) carried out in this project are vastly different and as discussed in section 2.4.5 are very difficult to relate numerically. As has been shown earlier, the materials tested exhibited strengths far exceeding those included in Table 2.8 providing further evidence of the difficulty of relating the two measures of strength.

These estimations must be used as an approximation only and should not be used for final design purposes, unless there is a complete lack of any additional suitable detail. If this is the case the following contents of the report explaining the relationship of CBR designs with the most suitable mechanistic design technique needs to be noted to be able to design the pavement with both methods and for a final assessment to be made effectively.

The nature of the strength tests employed within this project is quite varied. It is for this reason that some general comments are provided preceding detailed discussion. The materials tested showed quite promising results for both the CBR and resilient modulus, unfortunately the UCS results did not mirror these results. No samples that were tested for UCS were also tested for either CBR or resilient modulus so it is unclear what correlation exists for the mine wastes sampled. This deficiency is an area for further research in the future. For the purpose of this project it is justifiable to say the strength of the mine waste materials are predominantly suitable for use in terms of functional design with significant improvements possible if stabilisation of the materials is employed, however further investigation is required if used for an investigation into structural adequacy.

5.1.4.1. California Bearing Ratio (CBR)

As discussed above this test is well understood by pavement engineers with results being easily applied for design in all cases except that of the axle loads presented by modern mine haul trucks. Decision of the layer thickness usually involves the consideration of curves that have been empirically generated to assign a value considering axle load, and hence need to be extrapolated to a large extent for application to haul roads. Whether pavements constructed of poor materials (with a sufficient layer thickness as per the design curves) are economical is another consideration to be explored by the designer, but the fact remains that a pavement providing adequate structural performance can be made from even very weak materials (adequate functional performance would be questionable). Accordingly there is no reason that a low CBR value alone can indicate likely poor functional performance either. Paige-Green (1990) suggests a very low minimum value of 15 % for the wearing course on a haul road, thus the base and sub-base can have an even lower minimum value if such a wearing course was to be provided with sufficient depth. However the structural adequacy of such a pavement is called into question especially when trafficked by such heavy vehicles, in isolation it would seem likely that shear failure early in the pavement life will occur with deep rutting the result in practice.

Not all materials sampled were tested, as the focus shifted to UCS testing once material modification was considered. If the above limitation is accepted then only two materials that were tested are insufficient for haul road construction (see Table 4.1). One is the sub-soil sample from Mine C, taken as an example of an in-pit sub-grade and therefore is of no concern to pavement functional design. The other is the ‘Middle’ sample, also taken from Mine C and is a cause for concern due to it being taken from a similar material to the ‘Top’ sample and not far from it along the alignment of the same haul road. It is a plastic soil and hence the four day soaking condition had a large affect, contributing to the poor performance which was not shown when tested for resilient modulus at optimum moisture content. Outside of these the results are quite promising, with five materials exceeding a reading of 100 % which is often considered a good road building material for any application. Note also that the large gains shown in UCS testing was not able to be shown for CBR testing as no treated materials were tested, however the two tests both rely on shear strength of the material and hence any changes in strength are likely to be somewhat proportional.

Ultimately CBR testing should be employed only as an indicator of performance. It has little significance when considering mechanistic modelling or design and for this reason needs to be better correlated with extremely heavy wheel loads prior to being used extensively with haul roads. Since the design methods that have been developed utilising this measure of strength are empirical and have not been properly extended to the wheel loads typically employed on modern mine sites it is recommended that a mechanistic measure be adopted in preference to CBR for representing strength. This allows for analysis to be completed via several techniques and software platforms that are commercially available, facilitating a greater depth of available information and thus a greater understanding of performance.

5.1.4.2. Resilient Modulus

The measure of resilient modulus for defining the strength of pavement materials is not new, however its use has lagged behind that of the CBR method. This provides the basis for inclusion of Equation 2.7.

The results of resilient modulus tests are hard to compare to anything else, since it truly is a mechanical entity. There are no recommendations as to what a minimum value should be, but rather the bounds are placed on mechanistic structural design by way of observation of vertical compressive strain limits in most cases. Consequently full modelling and analysis is the intended use for any test results, rather than simplistic minimum value specifications.

Caution must be exercised when considering the resilient modulus results within this project. Due to only 15 stress repetitions being used in the testing it can be difficult to decide upon a ‘mean’ bulk stress value to use in calculations. This is no more evident than with the ‘Windrow 1’ sample from Mine A. The test did not complete through all 15 points, resulting in not all bulk stress states having a corresponding resilient modulus reading. Thus selecting which bulk stress value to apply is made very difficult, with a value of 150 kPa ultimately being selected. However an argument for a much higher value can be made (to reflect the commonly used 450 kPa if all testing had completed), resulting in a resilient modulus of closer to 900 MPa. Most other readings were much simpler, commonly with the recommended value of 450 kPa able to be used.

A comparison with the CBR readings for the same materials is presented below to emphasise the possible differences in strength in the optimum and soaked moisture conditions. In general only the 'Alluvial' and 'Middle' materials show greatly reduced strength in a soaked state for CBR testing, which is attributable to the consistency limits of these materials.

Table 5.1: Comparison of CBR and resilient modulus values

Mine	Material	CBR (%)	M_R (MPa)
Mine A	Windrow 1	163.1	628
	Windrow 2	109.0	555
	Detridal	121.0	654
Mine B	Alluvial	42.8	581
Mine C	Top	149.0	489
	Middle	12.1	787
	Subsoil	5.1	**

Over time the resilient modulus of the base layers of the pavement will have large bearing on the functional performance. This is due to the effects of permanent deformation (vertical compressive strains) accumulating leading to less consistent performance and likely compression of the base leading to increased progression of functional defects.

5.1.4.3. Fatigue Assessment

A fatigue assessment revealed likely poor performance of the cementitious stabilised materials. Although it appears through observation of Equation 2.5 that Equation 2.6

and Equation 2.7 are likely not suitable for use in the case of off-highway haul trucks. Consider that an equivalent standard axle group has a weight of eight tonnes and a Caterpillar 793 has 32.5 (260 tonnes) times more when fully laden. The above mentioned equations for calculating fatigue life are indeed empirical and thus the situation considered in this project is outside the bounds of intended application. Despite this fact the results are able to provide an estimate that has shown a reduction of fatigue life from hundreds of repetitions to less than one. Thus the issue should be considered via a classic static shear strength or shallow foundation for performance estimates. Only once a design technique is derived to specifically address the above criteria could a specific performance model for fatigue performance be developed.

5.2. Laboratory Testing - Treated Materials

5.2.1. Index Tests

As with untreated materials, index tests were completed to determine the nature of the materials after modification by mechanical or cementitious means. Each test holds its own varied significance, as would be expected as most simply show very slight alterations to that of the parent material.

5.2.1.1. Particle Size Distribution

The results for the modified materials PSD’s can be seen in Appendix

A3 – Mine A PSD Results – Treated Materials (Mechanical Stabilisation, including added material – ‘Surface Fines’). It is clear from these plots that only a small addition of fines has resulted from addition of the stabilising materials, as would be expected. The mechanical stabilisation of the ‘ROM Tip Head’ is the most notable as this material had 20 % by weight of the ‘Surface Fines’ material mixed in. As will be discussed in the proceeding, this resultant material showed the most significant change in properties. This proves that even with a material containing silty surface fines in a moderate proportion that does not show an exceedingly high degree of activity the mine wastes can show large improvements by modification of their grading.

In the case of the mechanically stabilised samples the improvement in grading has lead to curves that actually lack gravel sized particles by MRWA standards. This is not considered an issue due to the fact that, as with the low limit of linear shrinkage noted above, this specification has been developed for sealed pavements where the sole performance consideration is the structural performance over a long period of time, which is quite different to the situation for an unsealed haul road. This result emphasises the presence of additional fine material which is deemed appropriate to provide a binding action for the provision of a sound surface.

5.2.1.2. Liquid limit

Overall the liquid limits of the treated samples did not show much modification, and if anything a slight upward trend. This is expected with the cementitious stabilisers, as they are not added for modification of consistency limits in the case of the cement and to lessen the plasticity index in the case of lime. Also of note is the testing methodology employed usually not being specified for treated materials. The 24 hour curing of samples in sealed conditions will have meant that a significant amount of cement will have undergone hydration by the time of testing. However a notable water demand will still exist, thus altering the physical properties of the material when compared to the untreated soil with the same moisture content, in essence decreasing activity against moisture content. Furthermore the cementitious additives will undoubtedly impact upon the flow characteristics of the material which are essentially being tested by means of the liquid limit test. It is not clear in the literature or from the tests completed which effect is greater and therefore it is difficult to determine if such modification can indeed assist in optimising functional performance. Consequently the significant result in this category is that of the mechanically stabilised samples. Obviously once again there is a strong dependence on the amount and nature of material added, but overall there has been shown to be little movement. The likely reason for this effect, which is significant due to the quantity of added fine particles, is that the silty surface material added is very similar to the parent material’s fine fraction. This is supported by the results of the raw materials testing, except for exhibiting a slightly higher plasticity index.

Ideally the liquid limit would be increased somewhat with a plasticity index reduced to a moderate level (perhaps around 5 %). This would provide for a wearing course that would be able to have a relatively high amount of water applied to it for dust suppression but would not be so likely to experience accelerated roughness progression.

5.2.1.3. Plastic Limit/Plasticity Index

The plasticity index of treated materials was in all cases but one lower than the virgin material. The exception was that of the ‘ROM Tip Head’ cement stabilised sample, which was deficient in fines and thus is not a surprising result. As would be expected the lime/fly ash materials decreased the most with near 50 % reductions in all cases. Once again the mechanically stabilised materials require special note. As they had a more active material added, with a similar plastic limit to the raw materials, it is surprising to see the plasticity index reduce as a result, since the ‘Surface Fines’ material added was observed to be more active. Although somewhat anomalous this is a positive result for the prediction of pavement performance.

An added effect of the modification of the plastic limit for the materials is the impact it has on the plastic factor. The reduction has lead to an increased prediction of roughness progression (see Equation 3.5 through Equation 3.7). This result is significant as it has lead to additional VOC and remediation costs in the pavement life modelling discussed later. To compound the situation the materials are also predicted to produce more dust due to modification, in simple terms owing to the addition of more fine material. In both cases it is expected that the models’ limitations have produced the result and field trials should be implemented to define the true effect on these defects.

5.2.1.4. Linear Shrinkage

The addition of all stabilising materials has resulted in a general reduction in linear shrinkage values. As the activity has seemingly been decreased (as evidenced by the reduction in plasticity index values) this is to be expected. However when observing the importance of the linear shrinkage to functional design criteria and the desired increase in shrinkage product it is concerning that the materials should behave in this way. Furthermore the mechanically modified materials show similar results as the parent material, which has been discussed in the sections above outlining the linear shrinkage and plasticity index. There exists the potential that if the test was extended over a longer period of time with a slowed curing process utilised the shrinkage would be found to be far greater. This would result in the materials being shown to be closer to the optimal range in Figure 3.10. However this effect would not be well represented within the pavement life modelling as the plastic factor (function of plastic limit and percentage of particles passing 0.075 mm sieve) is utilised in both dust generation and roughness progression modelling. Thus it is likely that by application of the accepted selection criteria, over time the treated materials would be found to be better suited than being tested after only 24 hours of curing time, in the case of the linear shrinkage test. This is not assessable in practice as the materials are unlikely to be malleable enough to allow liquid limit and thus linear shrinkage testing after being cured for 24 hours.

5.2.1.5. Capillary Rise and Swell

The capillary rise and swell results show great promise for material modification. In general capillary rise and swell reduce significantly with absorption results also appearing neutral for the raw samples with a greatly increased result observed with the treated samples. The two untreated samples tested showed a capillary rise of 100 %. This figure reduced to between 40 and 55 % for treatments, the exception being the ‘HV Access Road ‘ sample which appears to likely have a much lower value even in the untreated state as both treated samples show a result of 16 %. When viewed in conjunction with this result the elimination of swell is most promising for the functional performance of a wearing course. Practically this means that any water applied does not so easily penetrate the surface and more importantly the material does not undergo any volume change, allowing the surface to remain intact which preserves stability. The absorption results can be viewed as positive and negative. This is due to the fact that some absorption is good for the binding action a small amount of moisture can provide to the wearing course, however if the layer is permeable it can result in the base and sub-grade becoming moist and in the worst case saturated. Undoubtedly in the most extreme case this would result in poor structural performance. Ultimately further research into the level of moisture content providing the best performance for differing material types should be carried out, but it is likely that in this case the magnitude of 40 to 50 % observed for both treatments is a cause for concern. However it should be noted that these samples had cured for only 7 days and in only a sealed condition and not in a moist environment and hence should be expected to absorb a greater proportion of water. It may be more appropriate to complete the tests again employing steam curing and also in both cases after at least 28 days to determine the more likely final effect.

5.2.2. Strength Tests

The only strength testing completed for the treated materials was the UCS. The untreated materials were tested utilising CBR and resilient modulus methods and thus there is only so much comment that can be made about the predicted performance for the same materials post treatment, from observation of the UCS testing. Ultimately both test the shear strength of the sample, but in a varying and dynamic way that has to date not been able to be related to the static shear strength found in the UCS tests in any reliable way.

5.2.2.1. Unconfined Compressive Strength (UCS)

The UCS testing has shown very significant static strength gains are possible with additives. In many cases an increase in excess of 1000 % occurs. The untreated samples were quite comparable, however the somewhat gap-graded ROM Tip Head material showed a lesser strength when treated. This effect is not surprising and does not necessarily exclude such materials from consideration of stabilisation, however in this case the lack of finer materials has resulted in a relatively poor result for the lime/fly ash modified sample. Overall the results hold more promise for structural considerations with an obvious increase in shear strength evident.

5.2.3. Treated Materials Performance

As a result of the decreased activity of the materials especially due to addition of cementitious additives, there is a lack of results supporting the addition of such materials. The use of other stabilisers, such as polymers or those primarily aimed at dust suppression is worth due consideration on the modifications witnessed in the consistency limits for the addition of the binders involved in this project alone. Chapter 2 can be reviewed for discussion of these additives, which demonstrates that there exists a disproportionate amount of literature available compared to the number of products commercially available and thus it is strongly recommended that a specific assessment is undertaken to best define product will provide optimal performance. The pavement life modelling should be referenced for a more comprehensive assessment.

5.3. Comparison for Functional Design

5.3.1. Materials Currently Employed

Testing of the materials currently employed for use of the design criteria shown in Figure 3.8 has revealed that they are not likely to provide optimal performance. Overall the most significant shortcoming is a general lack of cohesion, as represented by low shrinkage product values. As discussed previously this failing is expected to result in corrugations and ravelling, which is consistent with observation of the defects occurring on the three mine sites. Remedying a lack of cohesion has inherent risks, since Thompson on numerous occasions has indicated that defects arising from materials containing too larger fraction of fine particles have been found to produce the largest cost to the mine operator in South Africa (Thompson and Visser, 2000). Thus if a fine material is added in an effort to improve performance the amount must be carefully controlled and well mixed with the parent material. The activity of the fines then also calls into question the watering of the road for dust suppression. It makes an interesting juxtaposition; if water is applied to the road skid resistance may be severely reduced however if no water is applied the fugitive dust emission are likely to be excessive leading to ravelling, loose material and lower skid resistance. It is for this reason that chemical additives have been considered. Despite being expensive and difficult to maintain in comparison to an untreated road surface, it is also anticipated they decrease the progression of potentially severe defects when adding to raw mine waste material. The grading coefficients are shown to be within the preferred range in most cases, however if one is to mix with other materials there is the risk this figure moves to a sub-optimal range, with the limited evidence that exists suggesting that the surface is then more susceptible to defects and associated increases in roughness. It also should be noted that the measure of grading coefficient is not perfect, as it focuses on gravel and sand sized particles only.

Figure 4.6 does indeed reveal that mine wastes do exist that are close to fitting the design criteria. It may be possible in the first instance for those involved in the design and planning of a new haul road to become knowledgeable in the material types that are favourable and even order material testing. After several materials have been tested on a given site and the results recorded it is more than likely there would be sufficient knowledge and empirical evidence gained that would improve the material selection process in the future. There is little doubt that this approach would yield improvements in functional performance at only a small additional cost.

5.3.2. Treated Materials

The treated material results in Figure 4.7 show a modification of shrinkage product as the most common effect. Addition of cementitious stabilisers shows a small decrease in grading coefficient but not such a magnitude to be considered significant. In the case of the mine wastes sampled the cement stabilisation shows some promise using the chosen design criteria as it presents movements generally toward the optimal range. In contrast the lime/fly ash mixes showed decreased shrinkage product, as one would expect with an addition of 2 % hydrated lime. Although these results do not show an immediate solution for improved functional design they do hold promise as there are added benefits in the way of structural effects associated with a large increase in the stiffness of the pavement. Through handling the stabilised materials in wet and dry conditions while completing laboratory testing for this project it was noted that, as expected, the materials showed vastly increased shear strength, to the extent that the increased resistance could be observed just by applying hand pressure to an uncompacted sample. Unfortunately compaction tests could not be completed for the treated materials due to a shortage of the volume sampled, however it is suspected the optimal density increased with cementitious stabilisation, as the cylinders for tri-axial testing were noted to be heavier with the same moisture and compactive effort as those left untreated. The net effect of a stabilised wearing course could be a very sound and long life pavement, as long as spillage from haul trucks is kept to a minimum and also removed with care so as to limit damage of the modified surface. The fact that the shrinkage product values also remain very low suggests that the amount of dust generated may be quite low also, reducing the need for a large amount of water sprays which is consistent with the discussion in Section 2.8. Great promise is especially held for lime/fly ash addition for any roads that may be found to have a high shrinkage product, as strength gain was also found to be similar to cement addition.

Consideration of non-cementitious stabilisers also needs to be given, as they present surfaces that in some cases can continue to be bladed and also may prove more effective as dust suppressants in dry conditions. It would appear from observing Figure 2.20 that there exist several options for the material type to be modified in this study. However it is unlikely the effort required to utilise bitumen stabilisation would be adopted, leaving polymers as the only remaining non-cementitious material to consider, unless one considers that dust suppressants such as lignosulphonate or chloride salts are likely to improve roughness progression also. This would need to be substantiated through field trials as such performance is often claimed by manufacturers but with little empirical evidence in support. As with those considered above, polymers will require the use of a pavement stabiliser machine for application although it is likely that a total of 500 meters could be covered per 12 hour day, as found and utilised in the pavement modelling (see Section 3.5.3). However once again the depth of detail in the design criteria proposed is called into question and thus its ability to model any material property changes in a way that would mirror performance in practice.

Overall the consideration of modified materials within the design criteria noted does not provide a great deal of insight as to their actual performance in the field. It is therefore recommended that further trials be carried out at full scale in order to quantify if a modified surface can provide the longevity required to provide economic justification for stabilisation.

5.3.3. Borrow Materials

The borrow materials considered show great variability as would be expected when considering two different geographic areas and a large amount of sampled sub-areas. Potential for materials to be sourced as wearing courses in their raw form or as additives to improve any deficiencies in a mine waste material is undeniable. Unlike with the raw materials there exist within the data collected for this project several samples within the optimal range and also many with a high shrinkage product, suggesting they would make perfect additions to the coarse and relatively inactive mine wastes currently employed. Thus if a mechanical stabilisation is to be employed a careful examination of the parent materials deficiencies should first be made so that a targeted search for a favourable additive material can be initiated. As mentioned below this would be made far easier with the engagement of professional geologists.

A decision to employ this improvement technique is likely to be significantly influenced by the haul distance and associated cost and pace of construction. Also required would be engagement of engineering geologist consultants if the search area was to be large, as sourcing a particular material is difficult without specialist knowledge. The modelling of construction costs in this projects case study considers a haul of five kilometres to reflect the assumption that haul road sheeting is likely to be sourced from outside of the current mine site, and as Table 4.3 shows, any increase quickly calls into question the economics involved in such a treatment. Hence the use of borrow material has great potential, especially if considered from the start of mine design allowing optimal materials encountered during the removal of mine over-burden to be stockpiled for later use. However it would involve an increased knowledge set within a mines personnel and also a slowing of haul road construction for quality assurance purposes.

5.3.4. Suitability of Selection Criteria

The use of Figure 3.10 has been examined as part of this project. It appears that materials fitting within the optimal range displayed here will indeed provide superior performance. However a comprehensive assessment is very difficult without an extensive monitoring regime to compare with the material properties. Many material with a shrinkage product lower then 126 (lower bounds of optimal range) appear to perform well in service, however the absence of monitoring over time makes a conclusion difficult to generate as the maintenance applied to the road needs to be considered also. Coarser materials would seem to provide serviceable wearing courses if the grading is sufficient, with the exception of those with particularly low shrinkage products, as they are prone to ravelling despite the application of water. The UCS results discussed in Section 5.2.2.1 show that similar performance is seen in all materials tested, however it is generally acknowledged that a coarser material presenting the same strength as a fine material in the dry state is likely preferable for performance in a pavement structure. This comment applies to haul roads, especially when considering that water is likely to be applied for dust suppression, which will affect structural and functional performance less with the coarse material and also will require less water to be applied due to less dust generation being likely. Hence an optimal material for the Pilbara region could extend to those with lower shrinkage products than 126 but this definition would require an extensive monitoring and testing program.

5.4. Pavement Lifetime Modelling

The modelling carried out for this project was intended to provide insight into the real impact of differing and modifying materials for provision of a wearing course. Outside of mining and perhaps large construction projects, unsealed roads are provided by a government entity where the sole purpose is to provide access to private property or minor services. In contrast, large proportions of what are in some cases vast mine operating budgets are spent on the load and haul process. For this reason it is prudent to provide detail on the cost and production benefits a haul road could provide if its functional design was to be optimized. Through carrying out this analysis it was found the maintenance inputs are also huge contributors to the operating cost and so analysis and discussion of their variation has been included. Although relatively comprehensive it must be remembered that the models used were largely developed for the aforementioned local low volume roads and in some cases for cars and long-haul type trucks usually present on highways. However there are only a few of such models available and it is for that reason some assumptions and extrapolations had to be made.

What has been modelled is a theoretical kilometre of haul road that is 30 meters wide and perfectly flat (longitudinally), however the model does allow for a longitudinal gradient to be included. There exist three variables within the model, that of Blading Interval (D), Average Daily Traffic (ADT) and a watering ‘dummy’ that denotes whether watering sprays are to be utilised, which are then increased once a certain dust emission volume is predicted. The impact of these variables is also discussed but the focus is to isolate the pavement wearing course material in order to examine what an optimal haul road pavement is in terms of functionality. In theory this model would be best applied by comparing it to the ‘status quo’, which in practice would be the current condition and expenditure of the haul roads around any one site. This comparison is not made for any particular road but instead comparing the production losses purely as any lost production due to the condition of the haul roads being rougher than 4.6 IRI (meters/kilometre). Maintenance is very similar although it considers any additional expenditure that is required for a road with a roughness of 4.6 IRI. Vehicle Operating Costs are calculated to the same criteria and a more detailed breakdown is also presented in order to reveal any potential specific savings that could be made.

5.4.1. Materials Currently Employed

The materials currently employed are somewhat variable in all three categories. Within each section is a detailed discussion of the causes and effects of the results.

5.4.1.1. Potential Production Losses

There exists 600 % variability in results for potential production losses for currently employed materials. This raw figure is largely due to one material, ‘ROM Tip Head’, not presenting any impact as a result of low dust emissions primarily but also a lack of any modelled loose material. It must be remembered that once the critical value is reached for these defects the production impact increases rapidly. Therefore although the results in Figure 4.9 capture a typical scenario, it is possible much less variability may occur if the full range of realistic inputs was considered as loose material and ravelling have been considered to have twice the production impact of dust or corrugations and more for the others considered. Thus once the critical value for either is reached the ‘ROM Tip Head’ material will present a rapidly increasing production impact. Although this must be tempered with the fact that the criteria for ravelling impacting production is considered to be an exceedance of a roughness of 8 IRI when grading coefficient is above 30, whereas the dust level need only exceed five kilograms per vehicle kilometre which occurs very easily without water applied.

These potential losses have a huge impact on the functioning of a mine. For example with the ‘Middle’ material at its worst condition, after only 90 days since last blading the road could be costing the business up to AUS$400,000 per day (for a mine producing 30 million tonnes of iron ore per year with a profit margin of AUS$50 per tonne), assuming this condition is present over the whole haulage distance. The other two categories do not present the financial saving opportunities as the loss in production does. It is believed this is a true representation of the actual situation in the Pilbara; for example if after a long period using the same alignment the road ravels a great deal and then a seasonal tropical rain storm passes, the road could become impassable very quickly potentially slowing the rate of ore being delivered to the processing plant. Subsequently if the mine produces 30 million tonne of iron ore per year, with a profit margin of AUS$50 per tonne and production is decreased 10 per cent for just one week due to poor haul road condition the resultant loss in profit is approximately equal to the purchase price of a new haul truck (AUS$2.8 million). A situation of this amount of production being lost is not inconceivable and hence why modelling the production impact of the materials currently employed is so important. See below for a more detailed example.

Note that maximum potential production impact is outputted. Since roughness progression is logarithmic, it is safe to assume that some production loss is likely to have occurred some days before the end of the blading interval. Consider how much production could be lost if a road is not bladed more than once a year. It is somewhat unrealistic that a road be used for such a long time without any maintenance, as the road would surely be in or approaching a state of impassability. But the modelling does indeed show the worth of slowing production for a short time by diverting traffic to maintain the surface, as it is clear to see what a large magnitude of production (and therefore revenue) a mine operation could potentially forego.

For example consider the situation of a one kilometre haul for both waste and ore, with five trucks (Caterpillar 793C) doing one hourly cycle or 24 cycles a day. In total this notionally means that approximately 26,500 tonnes of material can be hauled in a day, if one is to make the assumption that half of the cycles are dedicated for ore (in this example iron ore with a marginal profit of AUS$50 per tonne) then a profit of approximately AUS$660,000 could be realised in one day's work. If the road is considered to have reached an especially poor condition it is feasible that production could be reduced by 10 %, representing a AUS$66,000 loss for each day the road is in a poor condition. If the assumptions shown in Section 3.5.2 are adopted the plant, labour and fuel costs to repair this road would total approximately AUS$27,500 notwithstanding the reduced production over the two days the maintenance would take to carry out due to obstruction of haulage vehicles. Even if it was assumed that maintenance practices meant that haul trucks now sat idle it would only take 20 days to recoup the total losses alone (due to 10 % increase in daily production after maintenance). This is assuming truck drivers are able to be utilised for other tasks during the days of maintenance. This situation should be unlikely to occur if diligent maintenance planning is put in place, whereby the haul trucks not able to pass the section of road being maintained are diverted to other work. Hence in this example the cost of maintenance can possibly be recaptured in less than a day of optimal production, depending on the impedance to production during maintenance. It is worth noting that a profit margin of AUS$50 is at the current time a realistic estimate, but even if a lower margin is available for the ore being mined the cost of maintenance is in general much smaller than the profit that the mine can experience from efficient production. Furthermore one should observe that this situation applies especially to mining operations with bulk handling requirements and if for example the loads hauled are much smaller the justification for maintenance explained above is reduced.

Lastly the materials currently employed should be considered when being utilised on ramp roads. In this situation the gradeability of a Caterpillar 793C at 30 km per hour of approximately 9 % is called into question, as if the vertical grade exceeds 4 % the rolling resistance (shown to likely have a maximum value of around 5 % on a poor surface) could mean the gradeability is exceeded and the truck would be slowed. Ramps as steep as 10 % are common and hence for every extra percentage of relative grade a truck must pass the potential losses are compounded. Figure 3.12 should be considered for such analysis and a slower ramp design speed.

5.4.1.2. Vehicle Operating Costs

In contrast to the above result, the 'ROM Tip Head' material displays the highest yearly operating costs for the average traffic of 240 daily passes. It exceeds the best performed pavement, that of 'Windrow 1' by approximately 8 %. The VOC costs are very closely linked to the roughness progression and hence this material shows the largest standardised roughness over the blading interval.

It is reasonable to expect these costs to not vary a great deal when considering a constant set of maintenance inputs, as the materials are all reasonably similar. The bigger difference comes when considering variable blading intervals or water sprays. Furthermore a balance needs to be struck to make the most efficient use of the mine waste materials. This comment should be compared with the fact that with the same inputs, but a fixed theoretical roughness of 4.6 IRI (minimum possible roughness) the VOC's only total AUS$935,195 per kilometre per year, this is only approximately 59 % of the average cost shown in Figure 4.10. Thus the balance of blading interval and VOC should be considered, especially observing the cost of a grader and operator for constant coverage. The traffic and therefore size of the mine will also have a sizeable impact, with the material type also playing a part but only significantly as blading interval increase to a point that could see road surface roughness reach such a level that could be considered unsafe.

5.4.1.3. Maintenance Costs

The maintenance costs show a variance of only approximately 3.8 % with the 'ROM Tip Head' material once again being the worst performed and the 'Detridal' the best. The variances in cost in this particular situation are due to the cost of the re-sheeting required due to a higher rate of gravel loss. The small differences in the condition are not surprising, with an average AUS$136,326 per kilometre per year being required.

The more realistic situation to model in this category is the impact to the total maintenance cost of applying water, although discussed later in Section 5.4.5 it is worthwhile to include in the discussion here particularly. Figure 4.12 shows a remarkable change in relative costs, with the ‘Alluvial’ material requiring AUS$358,371 per kilometre per year, which is approximately 341 % higher than the ‘ROM Tip Head’ material. This can be attributed to the high degree of fines within the grading of the ‘Alluvial’ material and hence the large amount of watering required to maintain it in a serviceable condition. It is also clear from observing these results that the cost of applying water sprays has a large bearing on the total cost of maintenance for any given pavement. For this reason an avoidance of materials containing a higher degree of fines in understandable, however the reader is referred again to the large effect of lost production and the effect that small variances in certain defect types have on the potential loss of income.

5.4.1.4. Treatment Costing

Treatment costing shown in Table 4.3 reveals that lime/fly ash treatments are likely not an economic solution considering that they do not show a vastly superior performance to justify the extra cost of application. When compounded with the approximately 300 % greater cost in comparison to cement addition it is very likely that such treatments are very unlikely to be selected in practice, purely due to the prohibitive initial cost. The one exception may be in the case of particularly active soils where control on the volume change is desired. Ultimately material improvement then becomes a balance of applying cement stabilisation at the extra cost of a stabiliser to apply in-situ or a significantly slowed production process to allow for the mixing of cement by way of graders. The alternative is the sourcing of superior borrow material which is often highly dependent on whether such material has already been identified, as a targeted search is usually best completed by a specialist and the total haulage costs required to utilise any chosen material. Hence there is no holistic solution available but rather a detailed investigation needs to be carried out leading to an informed decision that is probable to lead to differing optimal solutions in each specific situation.

5.4.2. Treated Materials

The treated materials considered within the project show variability mirroring that of the untreated parent material. For a true comparison Section 4.3.4 should also be referenced to compare the potential benefits shown herein with additional capital expenditure to provide a modified wearing course.

5.4.2.1. Potential Production Losses

The potential production losses are in all cases higher or equal to that of the parent material. Within the modelling completed this is primarily due to the added fine material resulting in dust generation being much more likely. In the case of the 'ROM Tip Head' material the increase is most marked due to the coarse nature of the untreated material that also caused extreme cost fluctuations when compared to the other untreated materials in the above. Other attributable reasons for such poor performance include the fact that none of the materials had a modified shrinkage product above 126 and so were still considered susceptible to corrugations (via Figure 3.10), and the grading coefficient was increased making ravelling more likely. Hence in this case it may be that the model cannot reflect the true performance in practice or the test methods employed are not suitable for treated materials and may require modification to better define the treated material properties. For this reason it is recommended that field trials be carried out for such treatments, as if a negative impact on production is shown then there is no basis for considering that specific pavement material modification any further.

The cost of treating the materials could potentially be offset in one good day of production, assuming a reasonably poor road surface condition originally and a sufficiently high amount of ore being hauled daily, if such treatments were effective. This alone provides the impetus for investigating the addition of stabilisers to haul road wearing courses.

Of the three different treatments the mechanical stabilisation and cement additives were the best performed. This is due to the higher degree of dust generation shown by the lime/fly ash treatment, which is a result of the increased amount of fines with 6 % of fine material being added as compared to the two % of cement. This results in Equation 3.2 predicting a higher volume of dust. This is a possible shortcoming of the model as the binding action applied to the material as a result of the cementitious additions is not reflected within the consistency limit testing nor is it given any recognition in the dust generation model chosen for use. It is difficult to assess that any of the treatments is superior in this category. The borrow option does appear worth considering first due to its lower cost, however the cost is also much more variable as it relies on material being sourced within a sufficiently short haulage distance.

5.4.2.2. Vehicle Operating Costs

Before noting the effect of VOC of each treatment the reader is referred to Figure 4.13. This shows the improved performance in terms of roughness that is possible by way of material modification. This presents a promising result, particularly for coarse materials such as the ‘ROM Tip Head’ that typically does not exhibit a high degree of dust generation but does progress in roughness most readily. Additionally it can be seen that when considering all treatments it shows the lowest roughness progression in all cases. The other two materials that were treated also show a general slight improvement.

It is not surprising then that the VOC’s mirror the results just discussed, with the coarse ‘ROM Tip Head’ continuing to present the highest costs. Although one should note that the total variance of the results is a minimal 2.1 % difference. The maintenance inputs once again have the largest bearing, most notably the blading interval due to the large effect the roughness has on the total cost. The total VOC costs in some materials can be seen to almost double when the blading interval changes from 10 to 180 days – this is clearly a potentially large additional cost to the business with each vehicle kilometre costing approximately an additional AUS$10. To justify the modification of the pavement purely due to VOC savings would require approximately one million tonne-kilometers of haulage to be completed in the case of a mechanical stabilisation (with the associated assumptions relating to mine output discussed earlier),with the remaining treatments requiring even more. Note that this comment has been estimated utilising an assumed blading interval of 90 days.

5.4.2.3. Maintenance Costs

The maintenance costs do not show much change from that of the parent material, with the exception of the mechanically stabilised ‘ROM Tip Head’ material which was to be expected due to the previously noted large modification it underwent. Outside of this result the variations in maintenance costs were not particularly notable except for the effect of watering. Due to the aforementioned increase in dust generation associated with the modification of the materials, the cost of maintenance increases over 100 % in some cases. This result suggests that watering would most certainly be required for the pavement to remain serviceable.

These results need to be considered in conjunction with the changes to maintenance that would likely be required due to material modification. Water sprays would still be possible but the quantity of water applied would have to be closely monitored to prevent leaching of the cementitious or borrowed fines, which leads to poor and unpredictable pavement performance and also possible environmental issues. For this reason it is recommended that the camber of the road be carefully considered and maintained should modified wearing courses be used. Additionally a prediction of the pumping of fines is required prior to use as it could prove expensive should the additive be lost in a short time. The blading of the surface once it reaches a poor condition is also questionable. It is possible that when wetted some blading would be possible, but the pozzolanic reactions formed during cement hydration will likely have completed and as such the wearing course cannot be expected to form a bound surface after blading. These considerations call into question the use of stabilisers since the requirement of some blading is assured to remove spillage (material fallen from haul trucks). However if over-filling is restricted and spillage removal procedures changed to perhaps use a smaller dump truck and wheel loader perhaps the impact could be reduced. Overall it is clear that if a bound or even modified pavement wearing course was instituted, the maintenance would have to be monitored much more closely.

5.4.3. Borrow Materials

The borrow materials examined show superior performance in all categories, however do require more watering due to the higher shrinkage product many have resulting in a high degree of dust emissions. Potential production losses are less than that of the untreated waste materials in both watered and non-watered conditions. The VOC costs mirror this where treated materials are the highest in both states, the total variance is consistent for both with borrow material having an associated positive cost impact.

These results appear very promising, especially considering the materials were found near two of the three mines considered and not even in a considered search for material specific to the design parameters. However the use of borrow material for either sheeting or mixing is always going to depend on how economically the material can be hauled to the point of application, and indeed if it can even be sourced. The comparatively lower cost of construction for an assumed five kilometre haul is promising, but the cost would rise swiftly to be equal to that of the cementitious treatments. However the superior performance in all categories of the borrow material would suggest that it may be the chosen improvement option even if it did cost slightly more. It is very likely that pockets of suitable material will exist within the mine over-burden and could be stockpiled once reached. Such a practice could prohibit production for the length of time the stockpiling was being completed and so is not necessarily always possible but should at least be considered, especially considering the magnitude of production costs that could be saved as shown by this modelling.

5.4.4. Variation with Blading Interval (D) and Traffic (ADT)

The effect of variation in the blading interval is quite simple for production impact and also for VOC, where there is a linear trend upward with a similar gradient for curves considering differing traffic volumes. The Maintenance costs however are not linear within approximately the first 180 days, as the initial costs are high due to the regular blading. A balance point exists where the minimum cost exists before the cost rises slightly and then remains somewhat constant.

The effect of average daily traffic is also quite simple. All categories can be seen to nearly double in cost between 100 and 300 vehicle passes per day. This impact is quite straightforward, as the defects all progress at a much accelerated rate. Hence the maintenance required must rise in line to meet requirements.

Overall it is clear that these two inputs have a huge bearing on the performance of a haul road and as such should be in parallel with the material employed. For example if there is a lack of good material available it may well be cheaper to accept a large amount of maintenance as required against the treatment of the material. Short-term roads (those that are ironically only constructed for direct haulage and thus are critical to mine production) certainly apply to this rule. Since they are realigned and moved constantly one must consider programming their maintenance, as the production losses associated with irregular blading and compaction have been shown to be great.

5.4.5. Effect of Watering

The effect of watering varies greatly between the three categories costed. The maintenance costs obviously rise quite significantly as can be seen in Figure 4.27 as more water sprays are applied to the surface. However this effect needs to be balanced with the saving in production losses and also VOC. Once again the large absolute value of potential losses means that this may be considered the primary concern, with only a few days of improved production paying for a year’s supply of an additional water cart and operator. Figure 4.26 shows the huge impact that water sprays have on VOC, which is perhaps the simplest justification for their use. The water applied slows the defects that the Pilbara haul roads are most susceptible to, that of corrugations, ravelling and dust and hence the much improved cost of hauling. Watering is assumed to be very effective for dust control and has a big impact on the roughness progression function shown in Equation 3.5, Equation 3.6 and Equation 3.7. The surprising outcome is the magnitude of the saving on VOC, approximately a 60 to 70 % saving on the non-watered equivalents. Another notable effect is the similarity of the VOC with varied traffic volumes, which is primarily due to the elimination of dust alone as roughness progression is still approximately proportional.

Overall the modelling in this project has shown the impact of watering to be very beneficial. However this comment needs to be qualified with note of the compaction and camber of the pavement surface. Should either of these be sub-standard, which has been observed to be common on short-term roads especially, the effect of watering may well be negative as pothole progression leads to largely increased VOC costs and undoubtable losses in production.

5.4.6. Material and Maintenance Impacts on VOC Components

The breakdown of VOC costs and their sensitivity to differing maintenance impacts has been provided due to its regular discussion within the mining industry. For example tyre life has been magnified in the recent past as large off-the-road (OTR) tyres experienced a global shortage in supply. Also worth considering in isolation is the fuel consumption due to road condition as the purchase price of diesel is ever increasing. Lastly the truck maintenance is significant due to the need for trucks to last the planned life of mine duration as replacements are often not able to be sourced on short notice. Also needing to be considered is the high labour cost of maintenance in the Pilbara.

5.4.6.1. Tyre Life

The tyre life parallels the relationships of VOC against blading interval and traffic volume quite closely. Most significant is the decreased tyre cost per vehicle kilometre when watering is applied. Tyre savings can be made by reducing traffic (not a realistic action as it would decrease mine production proportionately), reducing the roughness progression of the roads and so minimise the roughness at the time of blading, or alternatively by shortening the blading interval and finally through watering.

The relationship with blading interval is approximately linear with an increase of between approximately 9 to 27 % between the intervals of 10 and 360 days. Worth noting also is that the costs are similar for a blading interval of 10 days, also taking into consideration the traffic. Tyre costs are also decreased 5 to 10 % with watering being applied. These results are somewhat significant considering the cost of new tyres and difficulty in sourcing them in global mining booms, especially when considering average tyre costs per haul truck in this model average approximately AUS$400,000. To put this figure into perspective if the mine has 20 such trucks operating for a 10 year mine life, resulting in a potential saving of approximately AUS$7.8 million dollars if a 10 % improvement is made. This value may be insignificant when considering the production losses should there once again be a chronic shortage of such large off-highway tyres.

The effect not mentioned yet is the impact damage to tyres due to extremely poor haul roads or via contact with spillage, which is difficult to quantify. However one assumption that can be made is that haul roads with increased roughness only serve to increase the likelihood of both potholes and spillage and therefore the true cost of poor pavements is likely higher than indicated above. Once again this does not even consider the effect of a sudden unavailability of a truck on the mines production. Of course the lesser number of trucks present the greater the impact, so this is especially critical to mines that have decided to make use of large trucks as opposed to having a large fleet of smaller trucks.

5.4.6.2. Truck Maintenance

Within Figure 4.29Figure 4.29: VOC – Truck maintenance costs variation with blading interval (D) and Traffic (ADT) there is four distinct groups, they are from the top; ADT 300 non-watered, ADT 300 watered, ADT 100 non-watered and ADT 100 watered. The total cost is far larger than that of tyres and fuel and so this cost should be considered most significant in VOC. Once again the costs show far less variability at short blading intervals than at long intervals, with variability at 360 days of 53 to 123 % the original average cost. This cost is made up by the labour and parts necessary to keep a haul truck serviceable, which has the added significance that a truck requiring more maintenance is likely to be out of operation for longer, further impacting the mine through additional lost production.

Without question there are large potential improvements to be made from the reduction in roughness that the application of water sprays supplies. However the cost not quantified in this model is that of maintenance impact of the additional dust, which is important due to the fact that pavement not showing large amounts of roughness due to a low shrinkage product are indeed more likely to be dusty and therefore require increased maintenance in this way. Also worth noting is the effect of breakdowns on the production of the mine and also on the maintenance cost of the truck, as replacement parts are sourced and repairs completed. This effect surely will be proportional to the roughness of the road surface also.

5.4.6.3. Fuel Consumption

Figure 4.30 has four distinct groups as in Figure 4.29 with the same order, due to the reliance of both truck maintenance and fuel modelling on roughness prediction. Variance from the average cost (AUS$/km) at a blading interval of 10 days to 360 days is approximately 15 to 34 %. This cost is similar in magnitude to tyre consumption with the sum of them approximately half of the truck maintenance costs. Most significant regarding the estimation of fuel consumption and cost is the variable nature of fuel prices globally. Hence an estimation assuming a very high price must be made when commencing planning for any mine, due to the potential of any large price shocks possibly rendering the operation in question uneconomic.

5.5. Defect Progression Score

The defect progression scoring methodology has been developed to reflect the trends found within the pavement lifetime modelling completed for this project. It is to be used to provide guidance when comparing a mines haul roads and deciding where any maintenance effort should be spent. For a simple layout for scoring and advice on applying this methodology see Appendix H – Defect Scoring. The overall weightings were given due to the relative cost impacts in the modelling presented above, which were then attributed to each category.

5.5.1. Defect Progression

Relative scores have been assigned to reflect the rate of progression of each defect type. For example it has been observed in the modelling that roughness in the form of corrugations and ravelling requires remediation quite rapidly, whereas rutting is more so associated with long-term structural distress of the pavement. Dust also develops quickly when the pavement is not watered, however when watering is applied the progression is considered minimal. Watering also has a large impact on the progression of roughness as it helps prevent the migration of fine material that leads to corrugations and ravelling. The effect is reversed for potholing however, where the action of water only serves to accelerate the progression of this defect.

5.5.2. Geometric Effect

The effect of road geometry has been included to reflect the different impact respective defects have with varied alignment types, due to speed and anticipated pavement life. Short-term roads are scored low due to the fact they are only to be in place for a short amount of time, whereas the long-term and ramp roads are scored similarly. The weightings is high for ramps roads to reflect the increased potential for an accident should haul trucks have to negotiate tight switch-backs and ascending steep ramps on a pavement that is exceedingly rough or has poor skid resistance. Additionally a high weighting is also applied to long-term alignments for their potential to be utilised in a poor and costly state if not remediated in a timely manner.

5.5.3. Remediation Cost

The cost to remediate the respective defects is directly related to the costs implemented in the pavement life modelling for the same purpose. However they were observed over a range of conditions and periods of time to determine the most appropriate weightings to be applied. Rutting therefore is the most critical due to the requirement to construct a new pavement to properly remediate. Corrugations are also given the maximum rating to reflect the fact that blading to remove them only off-sets the defect for a short time and it is indeed a material deficiency that is primarily leading to their formation.

5.5.4. Material Properties

The relative scores due to material properties were allocated to indicate the potential that the Pilbara wearing course materials sampled for this project have of developing each respective defect. Hence dustiness, corrugations and ravelling are each given the maximum score. However the maximum weighting of five is not used here as the materials are not so alike so as to provide certainty for such a high weighting for one particular defect; however if this system was refined for a particular mine and this matrix modified then perhaps the materials would be consistent enough in one particular deficiency to highlight one particular defect in these weightings. It may be possible for the particular case of Pilbara haul roads that the methodology could be modified to indicate that materials with higher shrinkage product do perform better, as long as the watering for dust suppression is not excessive.

6. CONCLUSIONS

The following conclusions have been made at the completion of this investigation:

- The materials currently employed for provision of wearing courses range from being far too coarse with little fines and low activity to near-optimal materials. However all materials have reasonably good grading curves, with those assessed non-conformant to MRWA specifications being only marginal at worst.
- Stabilisation of materials holds great promise due to evidence in the literature, strength and moisture sensitivity testing. In contrast, pavement life modelling completed within this project does not necessarily indicate superiority of stabilised materials when compared to carefully selected mine waste material.
- The use of borrow material appears to be the most effective method of improving haul road functional performance, although this is heavily weighted on such material being available within a sufficient distance to the mine.
- The materials currently employed mostly display a relatively high strength for unbound granular materials. However some with higher fines fractions tend to lose shear strength rapidly when soaked.
- The modified TRH20 model used for functional design appears to be quite appropriate when observing the results of pavement lifetime modelling, with the exception of the defect of fugitive dust emissions where a large relative remediation cost means some materials in the optimal range via laboratory testing are shown to be sub-optimal in such modelling.
- The models used to describe general unsealed pavements and vehicle user costs appear to lend themselves well to modification for application with haul roads. However close field monitoring would be required to verify the values calculated.
- Pavement modelling reveals that the materials sampled are prone to being excessively dusty, especially within the predominantly dry conditions of the Pilbara region.

- Water sprays may be the single most effective method of pavement maintenance beyond watering for dust suppression. This effect is related to the increased density and therefore strength associated with moist materials that results in a slower rate of roughness progression, estimates indicate that significant reductions in blading maintenance costs are possible due to watering.
- There exists a critical limit near to the plastic limit of the materials that when reached the breakup of the road surface is accelerated in the form of potholes and ravelling.
- Production loss is the largest financial impact of poor road conditions, and presents follow on effects such a breakdown of haul trucks. The modelling used for this effect within this project was only simple but did highlight the need for the relationship to be more closely studied as it may be inhibiting production significantly.
- Modelling shows that wearing courses treated with cementitious stabilisers would require more regular water sprays than the same untreated materials. However roughness progression is reduced as desired. An additional consideration is then the drainage employed (lateral and longitudinal) and also leaching of the binder over time reducing its effectiveness.
- Watering reduces roughness progression, however the cost of watering the road regularly is the largest of pavement maintenance costs and hence a decision balancing this cost with vehicle and production cost reductions as well as other road maintenance savings must be made.
- Prediction of rolling resistance from roughness is difficult and requires further research. The modelling employed in this project shows low sensitivity in this relationship. Due to this it appears unlikely the gradeability of haul trucks is reached except when climbing ramps. Trucks will however be slowed if particularly severe defects occur in isolated areas, due to avoidance being necessary for safety implications.
- Road Maintenance costs are largely dominated by the use of water sprays for dust suppression, due to the regularity of application as the dustiness defect develops much quicker than roughness for the materials observed and therefore consistent expenditure required. The blading costs required are also decreased with this practice but in general have been shown to be relatively less.

- Vehicle Operating Costs show a high dependence on the vehicle maintenance costs primarily, but are also influenced road maintenance in the case of dust mitigation. This is exacerbated by the modelling that utilises a finite level of emissions to employ mitigating water sprays.
- Fuel consumption does not show much variation over the range of materials sampled and tested in this project, when utilising the ARFCOM model.
- Tyre consumption does not present much variation over the range of materials tested, it is possible the greater effect to be considered is the elimination of particles or very poor road surfaces that could cause punctures.
- Truck maintenance has been shown to make up a large proportion of vehicle operating costs, the labour costs associated with repairs and maintenance representing a significant amount.
- The defect scoring methodology that has been developed through use of the results of financial analyses could be employed to provide guidance with or without including further detail such that each category of construction, usage and maintenance is detailed.
- This project has sought to:
    - Determine if currently utilised mine waste materials are suitable for haul road wearing courses and if they can be improved through defect (Section 3.4) and VOC (Section 3.5) modelling available currently with modification for the specific application of the Pilbara region of Western Australia and large off-highway dump trucks respectively.
    - Assess if treated or better selected naturally occurring materials can provide superior performance.
- This investigation has resulted in:
    - The conclusion that potential impacts to a mines production and therefore reduced revenue due to poor haul rod condition is significant when compared to the cost associated with improving or better maintaining haul road wearing courses.
    - A modified defect scoring methodology that takes economic impacts into account.

7. RECOMMENDATIONS

The following recommendations apply to the conclusions noted above:

- The materials currently employed could be improved by replacement with select borrow, mechanical or cementitious stabilisation. Other means of stabilisation such as polymer additives should also be investigated.
- Field trials of material treatments should be conducted to fully define the impact and potential benefits.
- The TRH20 model, modified appropriately, could be employed for definition of a design criteria for materials employed for provision of haul road functionality.
- The maintenance inputs of watering and blading need to be better understood and leveraged to the mine operations biggest benefit.
- The potential impacts to production (quantity of ore produced) arising from the initiation and development of defects or general pavement roughness and dust emissions should be quantified. As the single biggest potential cost associated with haul roads for a mine operator this would provide a much improved definition of the financial advantages of such practices.
- Modelling of road maintenance could be furthered by way of field trials and logging of the associated inputs of road and truck maintenance, in addition to wearing course material properties.
- Vehicle Operating Cost modelling could be improved by a focussed study in to the definition of constants used in the ARFCOM and other HDM-4 models utilised in this project. Alternatively other models could be developed with a focus on the large off-highway dump trucks considered.
- An investigation into the relationship between fuel consumption and roughness is warranted. Alternatively a more accurate function for rolling resistance could be developed to describe the energy consumption in the hauling process.
- The link between tyre consumption and roughness of an unsealed road is required, especially with a function relating to inputs such as over-size index and degree of compaction to better predict the occurrence of punctures.

- True truck maintenance costs should be investigated via field trials or closer input from mine operations.
- An extension of the defect scoring methodology could be undertaken with inclusion of field trials would potentially greatly increase the benefits of its use.

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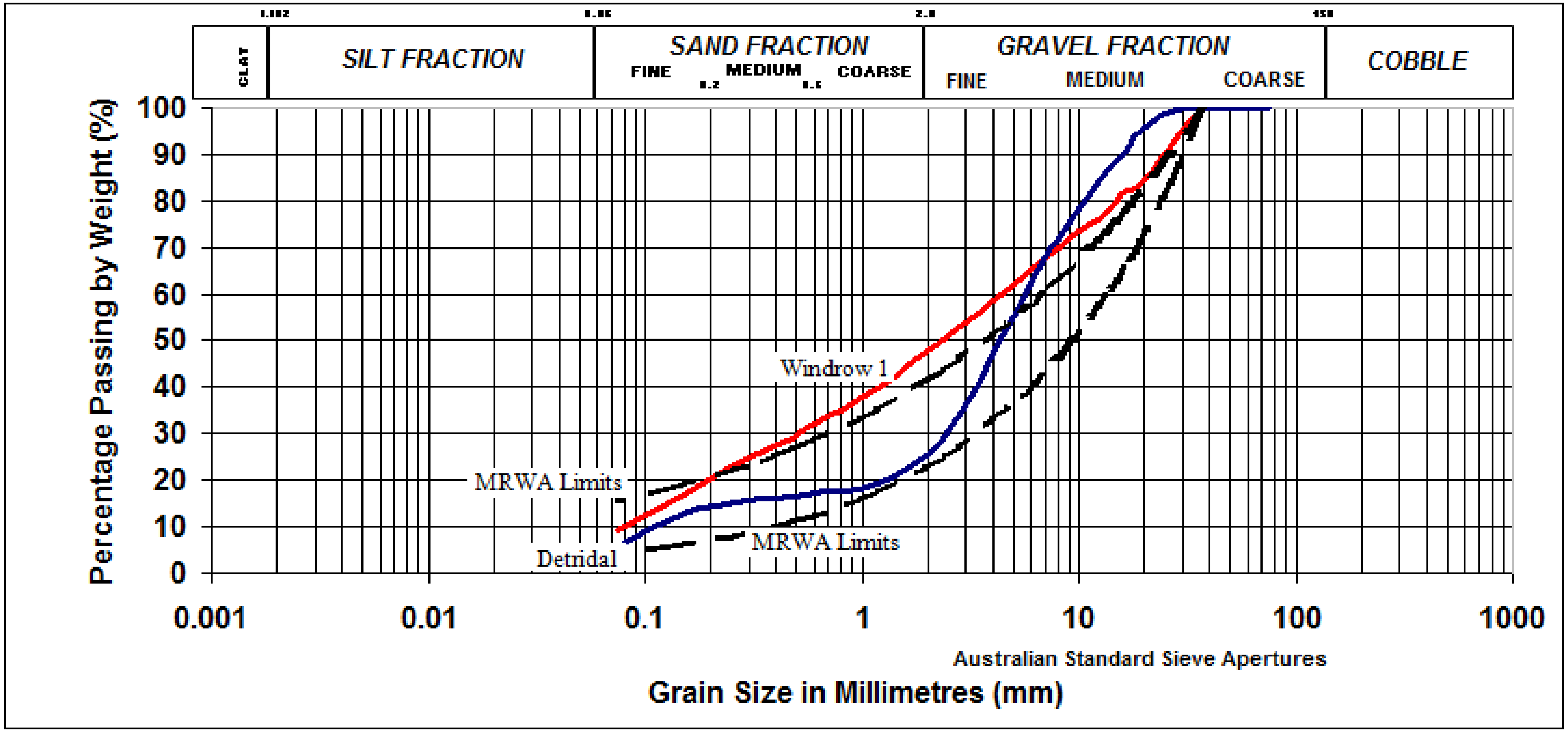
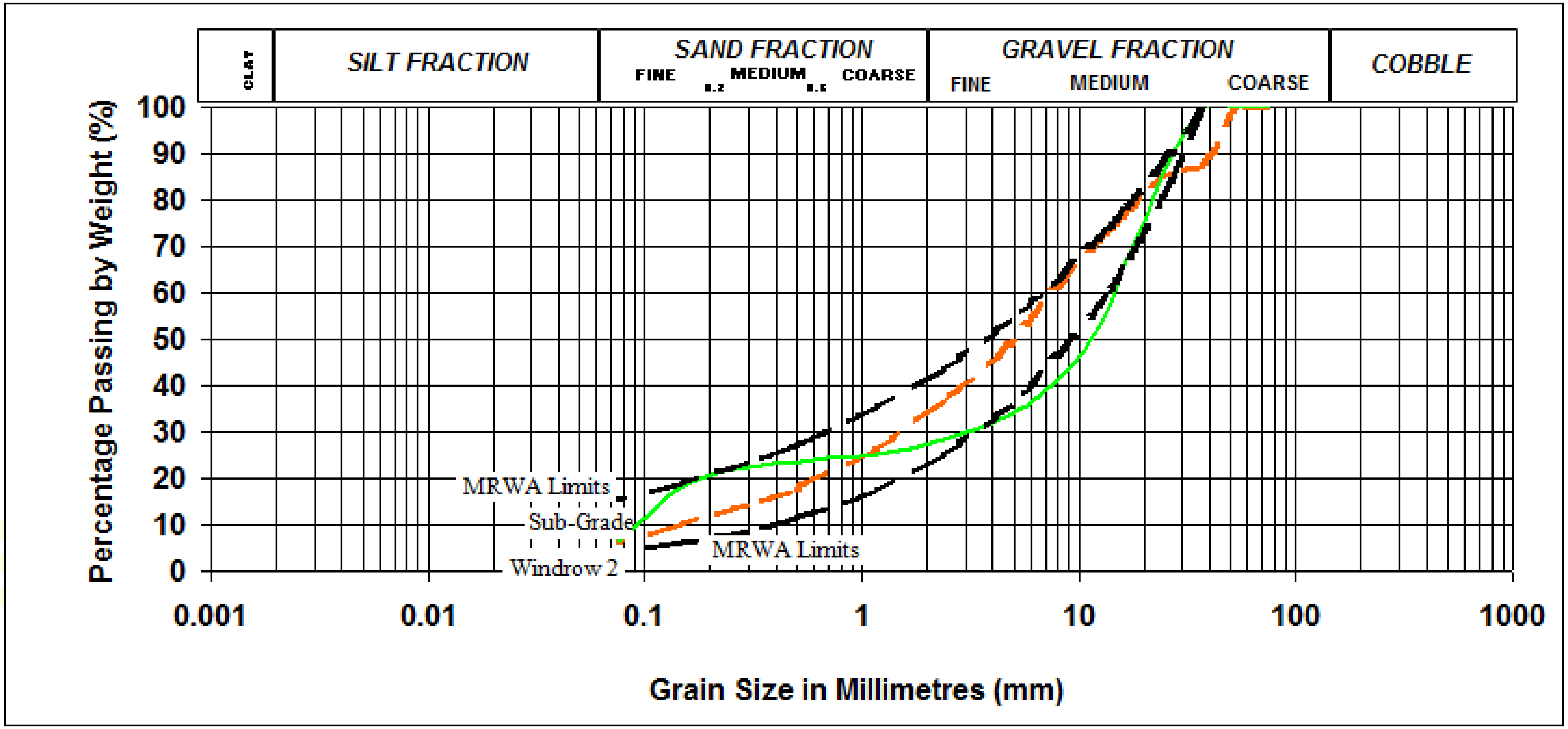
All of my friends and family that are so understanding of my irregular contact on weekends while I was hidden away writing this thesis.

APPENDICES

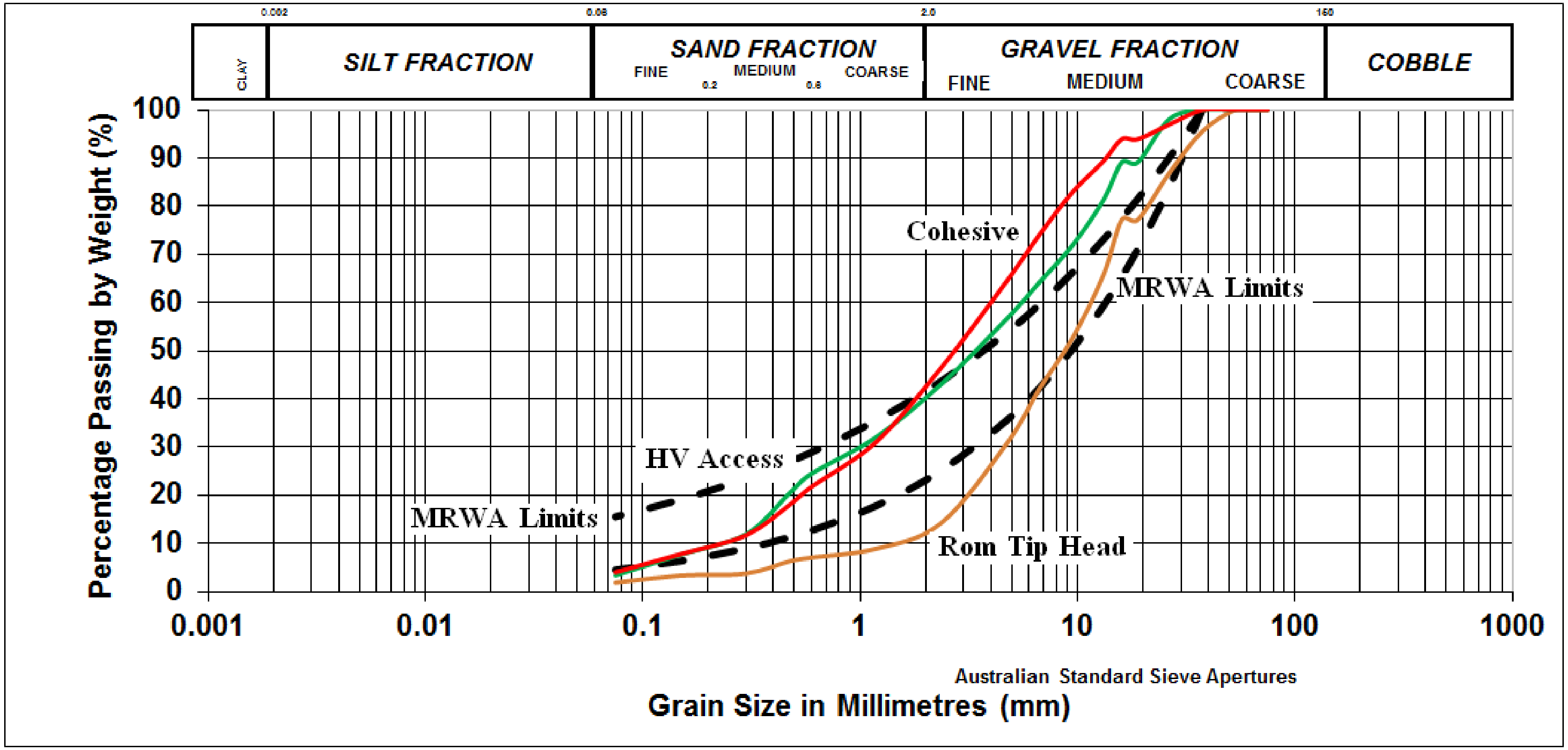
Appendix A – Particle Size Distribution Results

A1 – Mine A PSD Results – Non Treated within Project

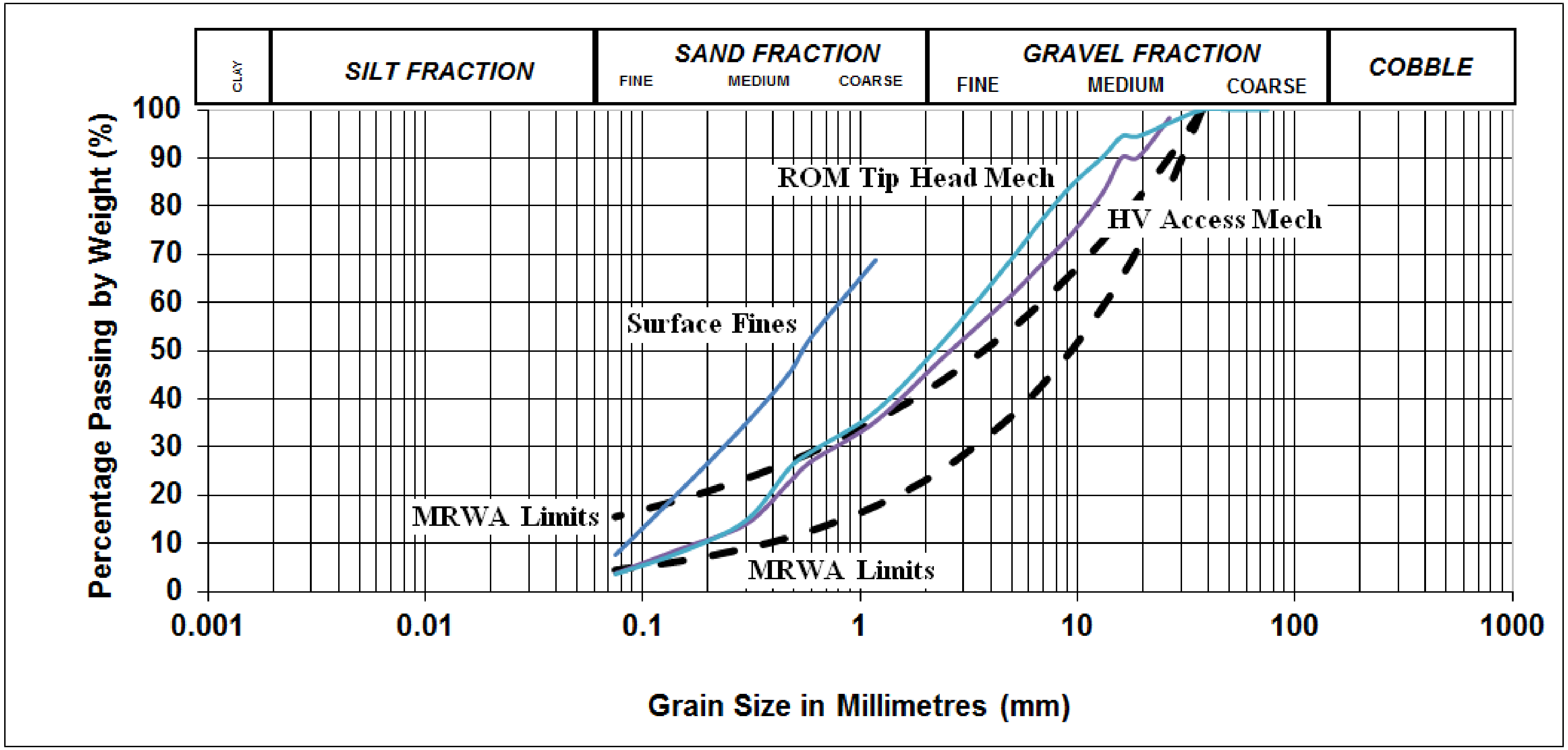



CLAY
SILT FRACTION
SAND FRACTION
FINE
MEDIUM
COARSE
GRAVEL FRACTION
FINE
MEDIUM
COARSE
COBBLE
Percentage Passing by Weight (%)
100
90
80
70
60
50
40
30
20
10
0
MRWA Limits
Sub-Grade
Windrow 2
MRWA Limits
0.001
0.01
0.1
1
10
100
1000
Grain Size in Millimetres (mm)

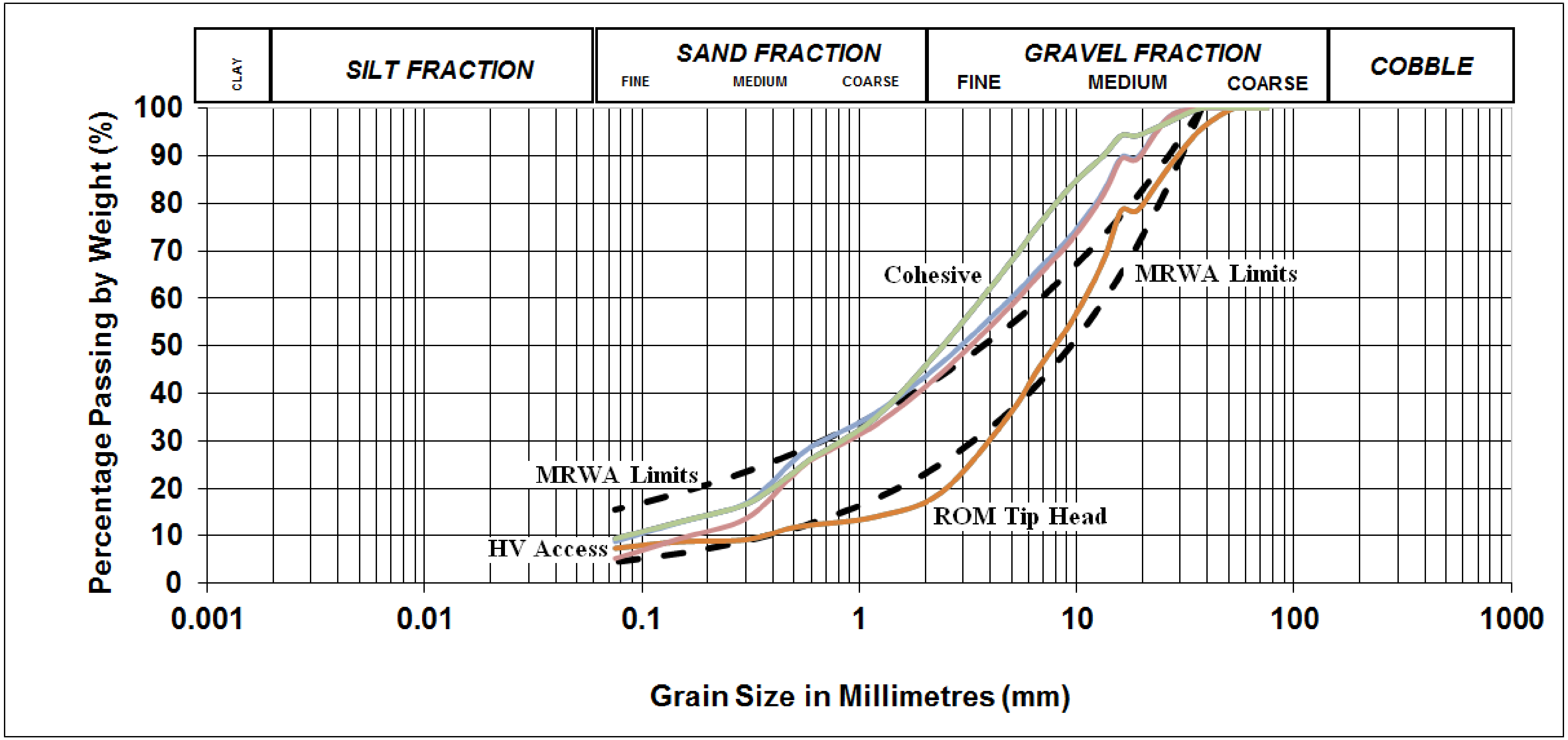
A2 – Mine A PSD Results – Treated within Project

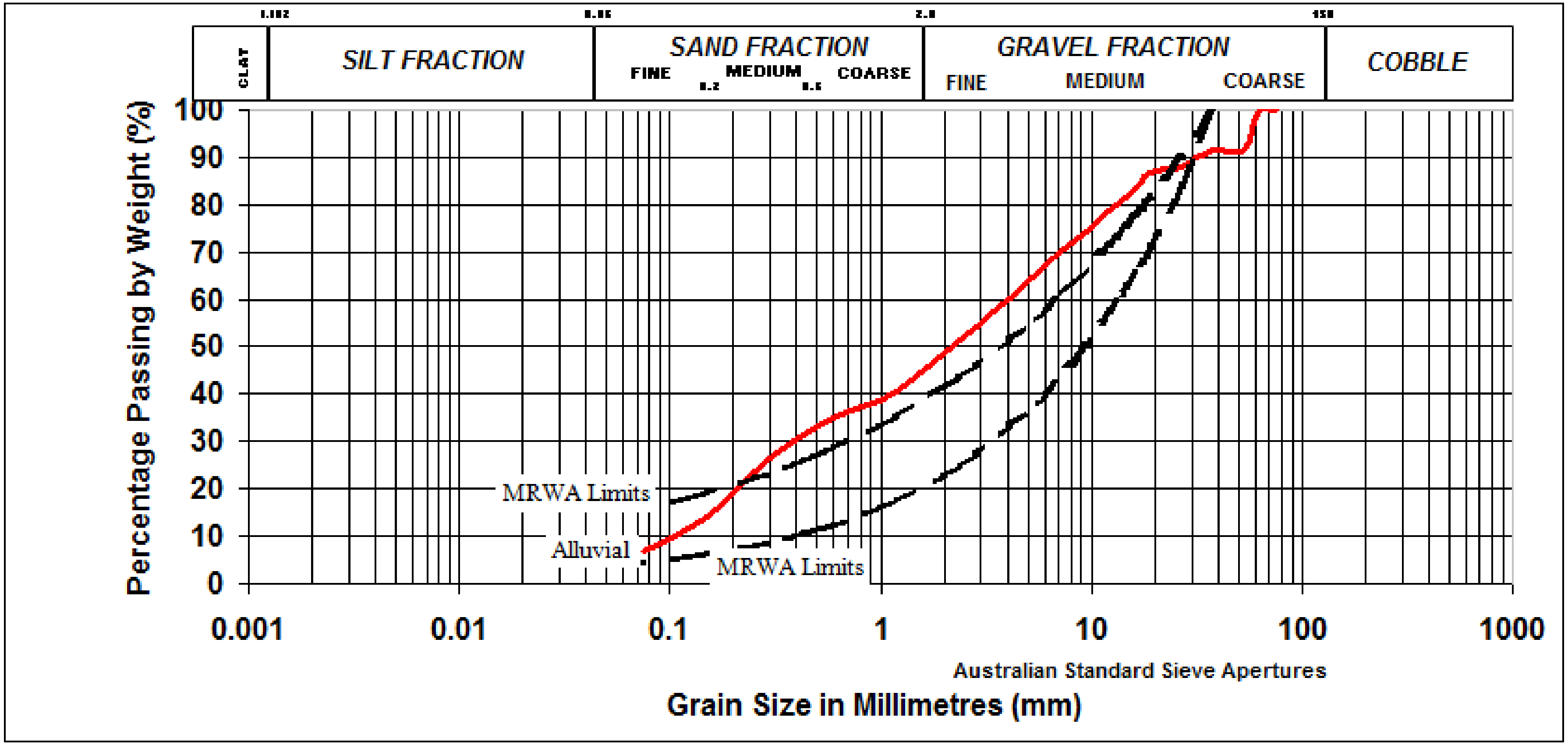


A3 – Mine A PSD Results – Treated Materials (Mechanical Stabilisation, including added material – 'Surface Fines')

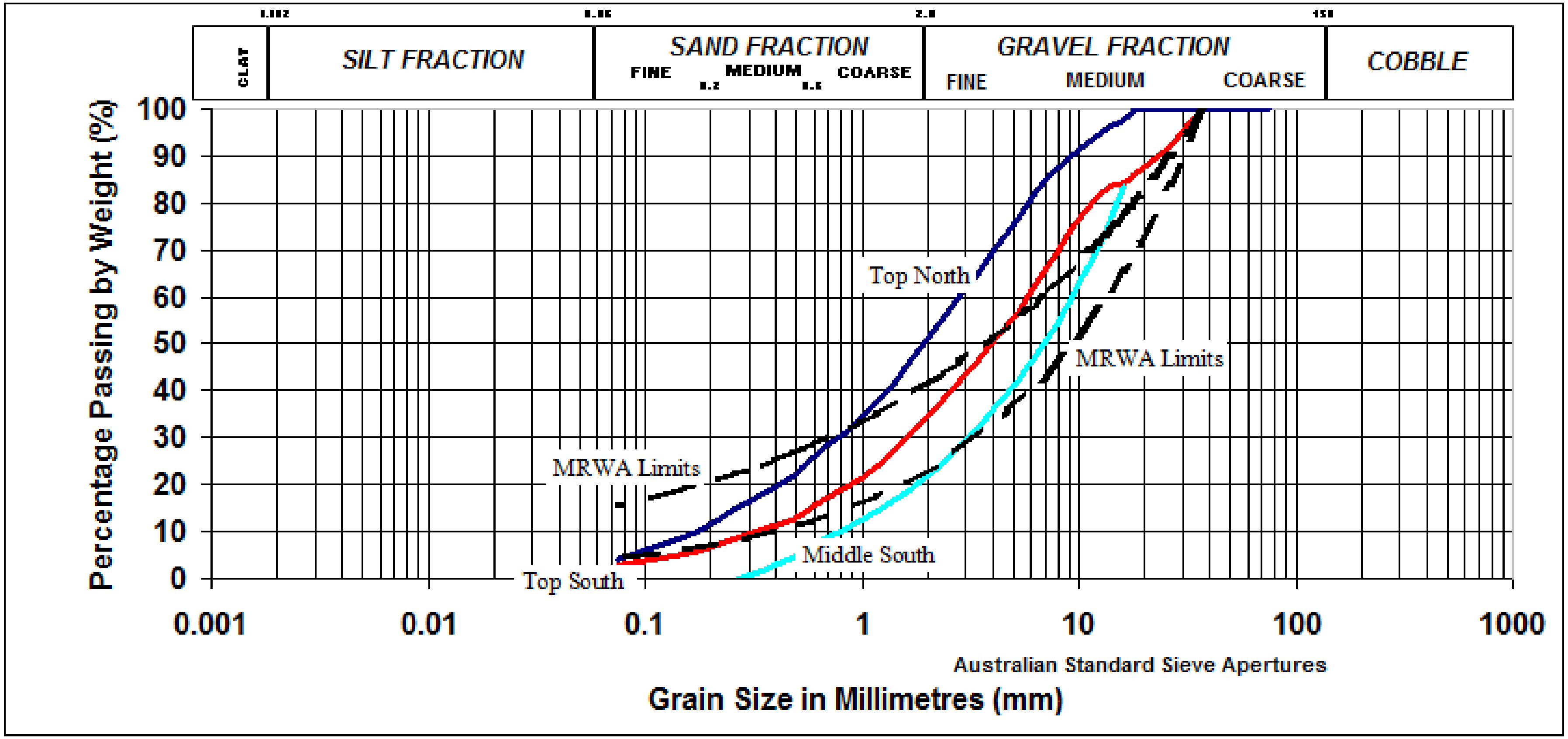


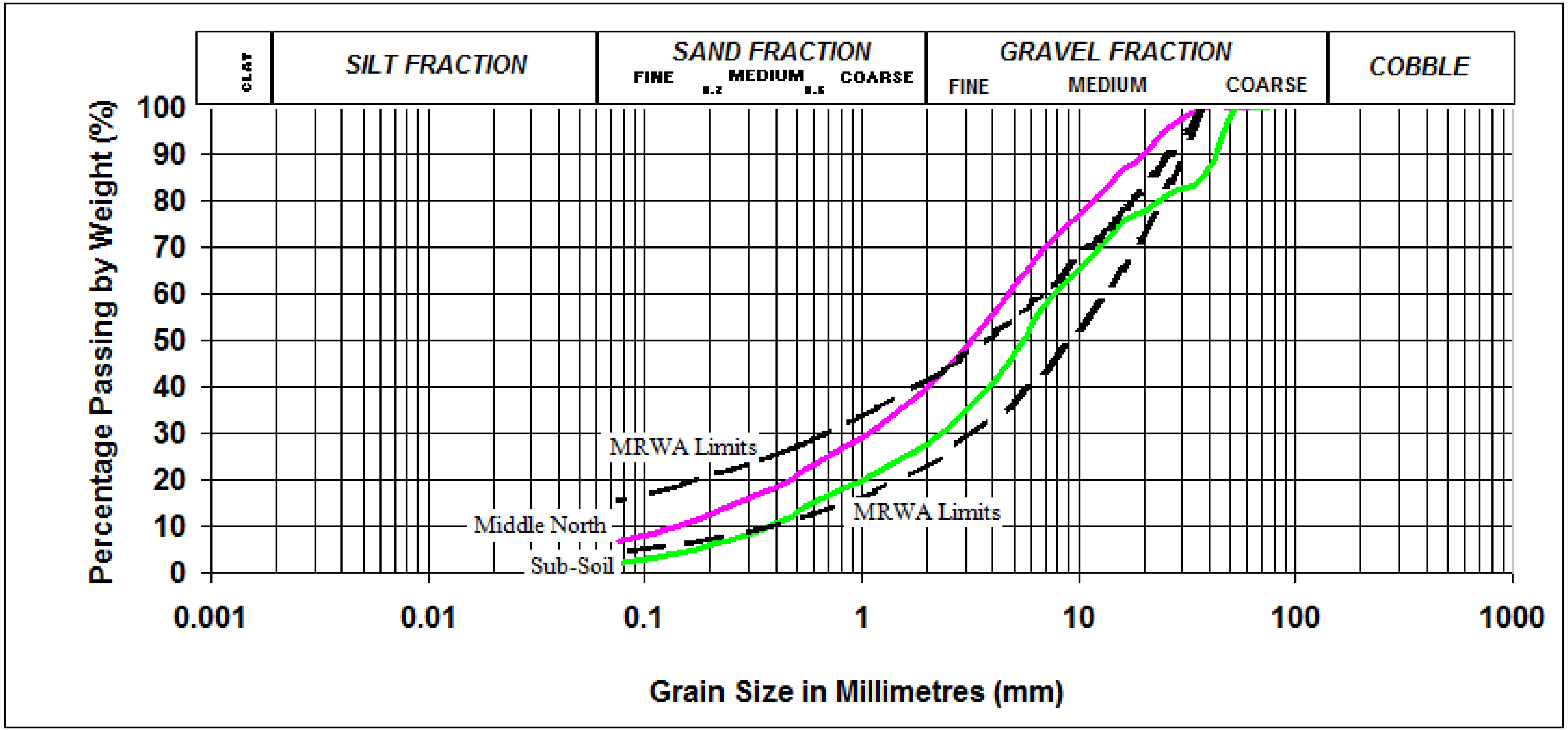
A4 – Mine A PSD Results – Treated Materials (Lime/Fly Ash and Cement Stabilisation)




SILT FRACTION
SAND FRACTION
FINE
MEDIUM
COARSE
GRAVEL FRACTION
FINE
MEDIUM
COARSE
COBBLE
Percentage Passing by Weight (%)
100
90
80
70
60
50
40
30
20
10
0
MRWA Limits
Alluvial
MRWA Limits
0.001
0.01
0.1
1
10
100
1000
Australian Standard Sieve Apertures
Grain Size in Millimetres (mm)

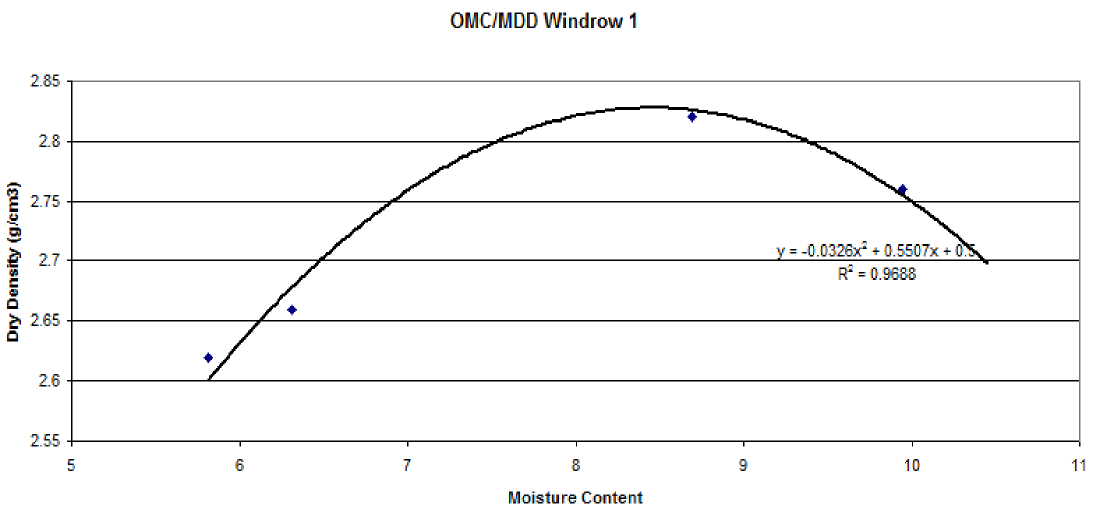
A5 – Mine C PSD Results

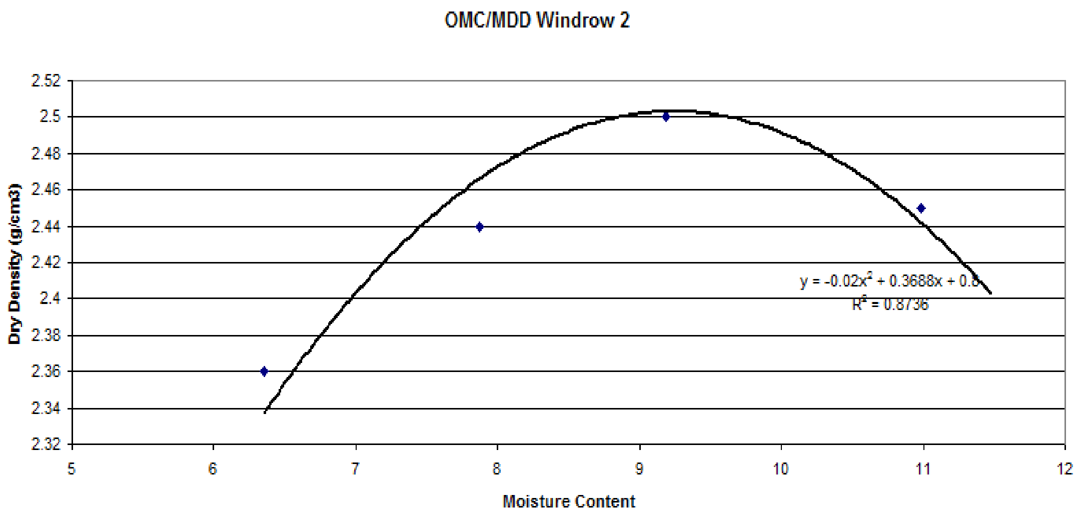


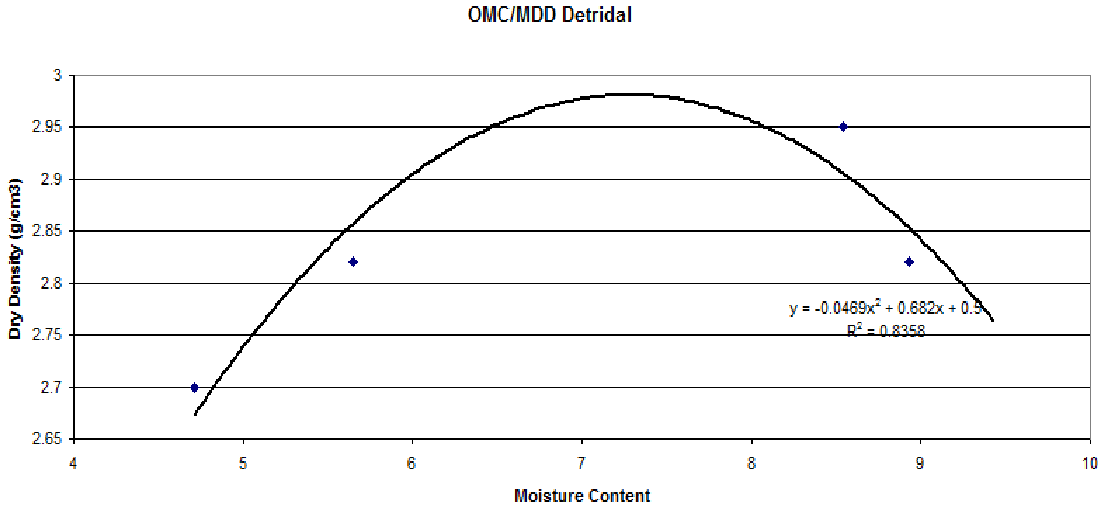

CLAY
SILT FRACTION
SAND FRACTION
FINE
MEDIUM
COARSE
GRAVEL FRACTION
FINE
MEDIUM
COARSE
COBBLE
Percentage Passing by Weight (%)
100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
1000
Grain Size in Millimetres (mm)
MRWA Limits
Middle North
Sub-Soil
MRWA Limits

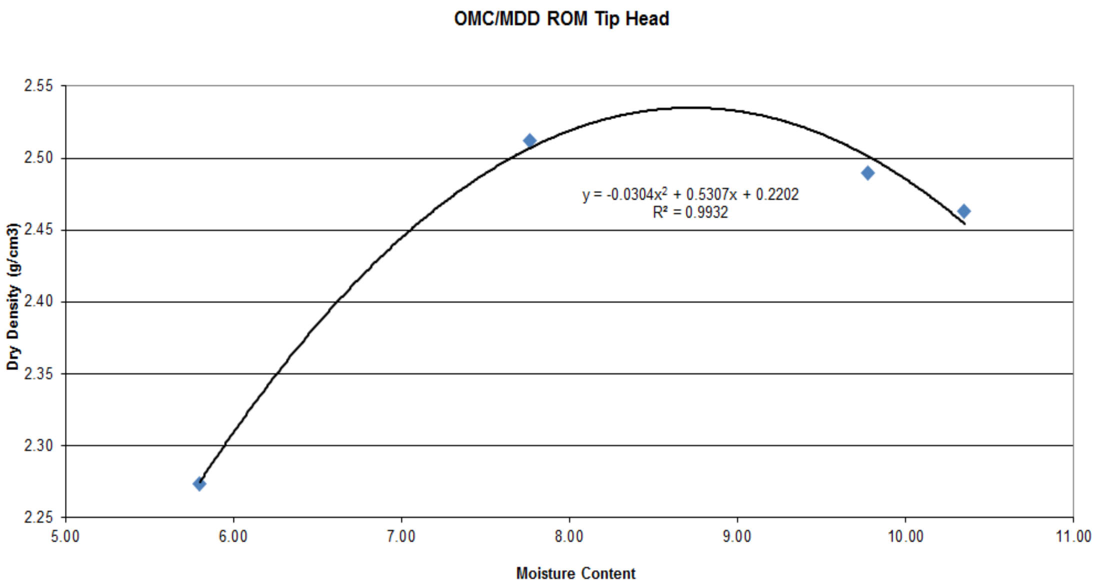
Appendix B – Plots for Compaction Testing

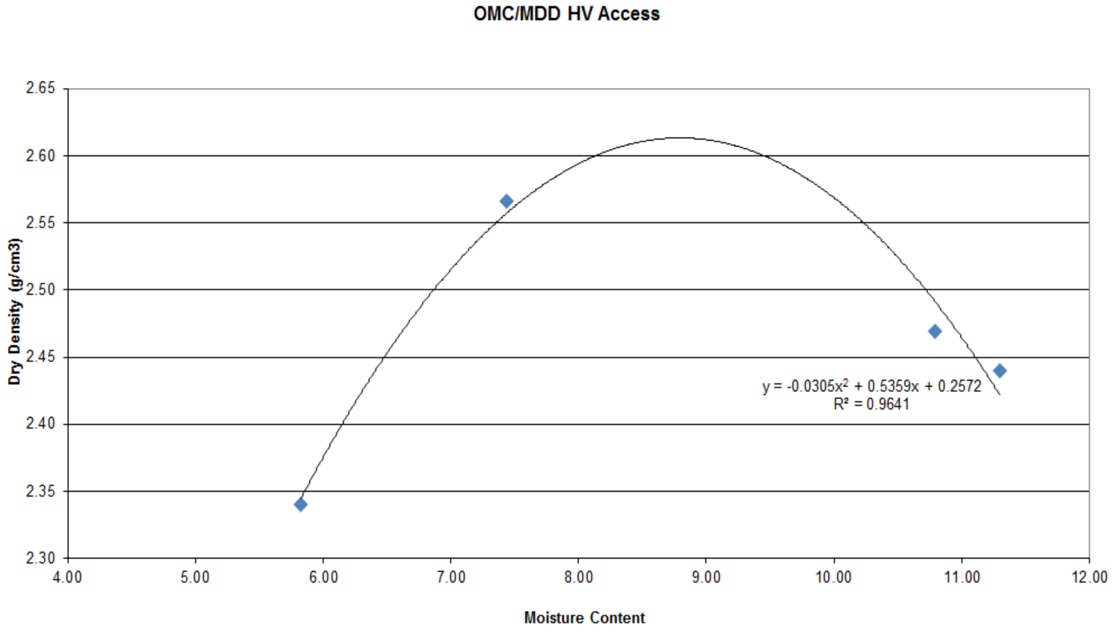
B1 – Compaction Plots for Mine A

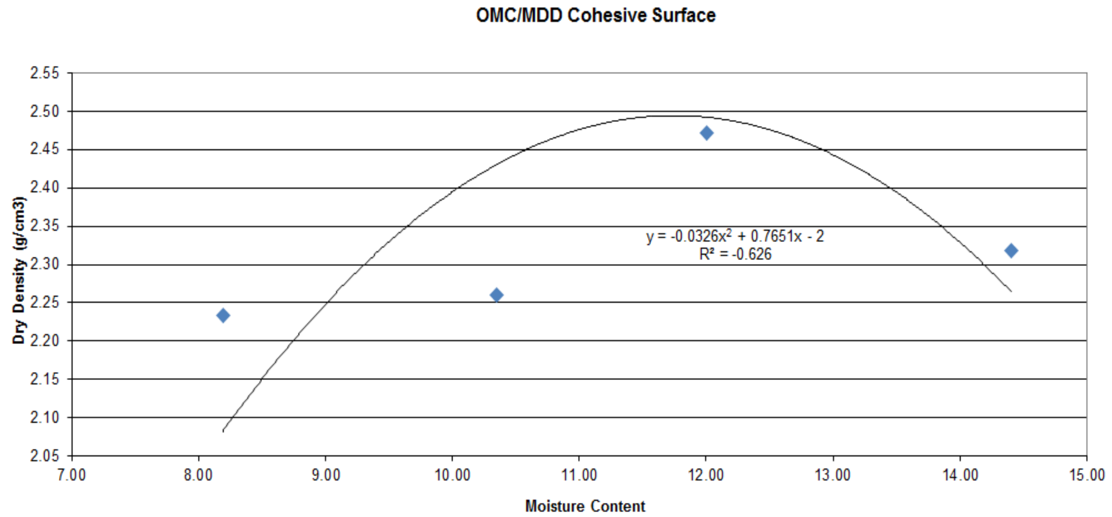




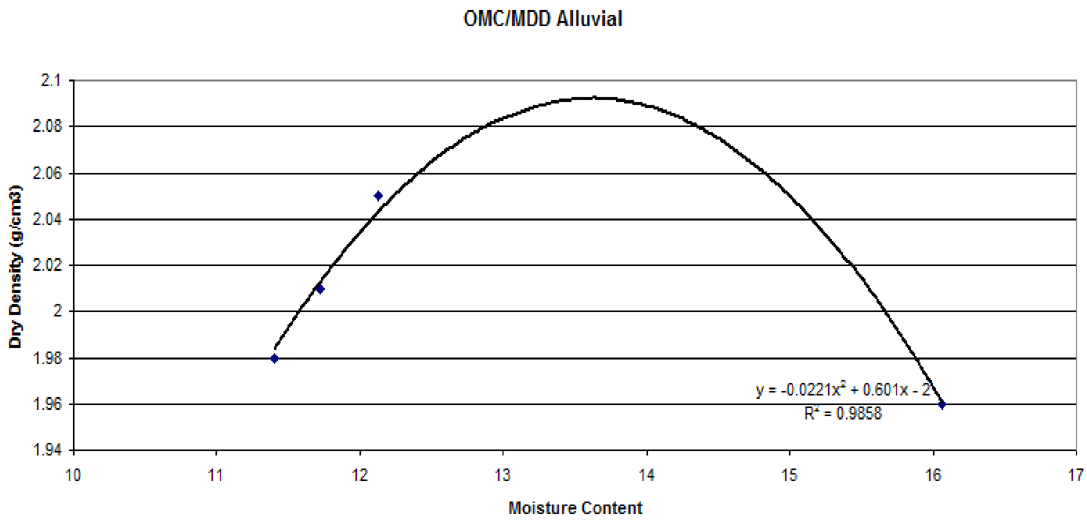

OMC/MDD Detridal
Dry Density (g/cm3)
3
2.95
2.9
2.85
2.8
2.75
2.7
2.65
4
5
6
7
8
9
10
Moisture Content
y = -0.0469x² + 0.682x + 0.5
R² = 0.8358


OMC/MDD ROM Tip Head
Dry Density (g/cm3)
2.55
2.50
2.45
2.40
2.35
2.30
2.25
5.00
6.00
7.00
8.00
9.00
10.00
11.00
Moisture Content
y = -0.0304x² + 0.5307x + 0.2202
R² = 0.9932

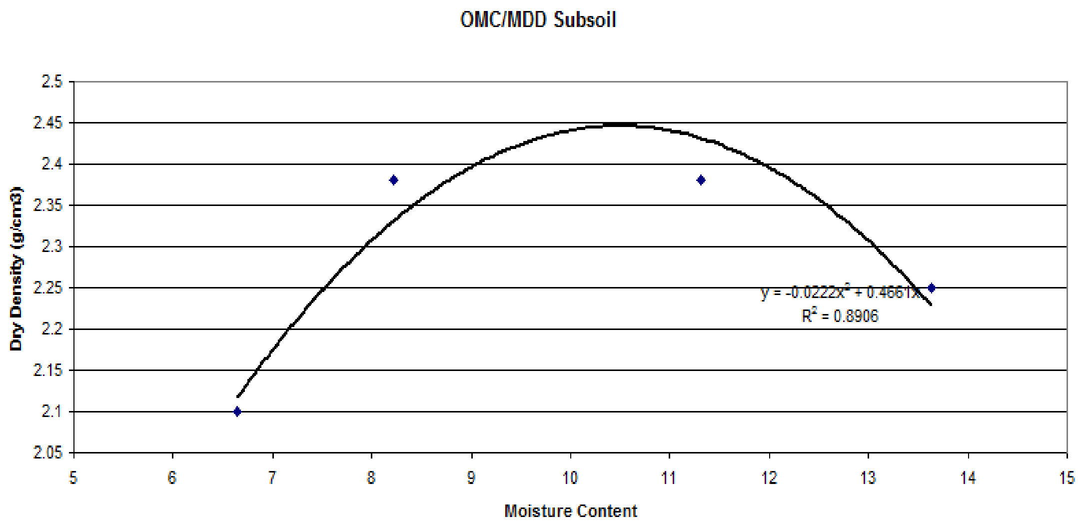

OMC/MDD HV Access
Dry Density (g/cm3)
2.65
2.60
2.55
2.50
2.45
2.40
2.35
2.30
4.00
5.00
6.00
7.00
8.00
9.00
10.00
11.00
12.00
Moisture Content
y = -0.0305x² + 0.5359x + 0.2572
R² = 0.9641

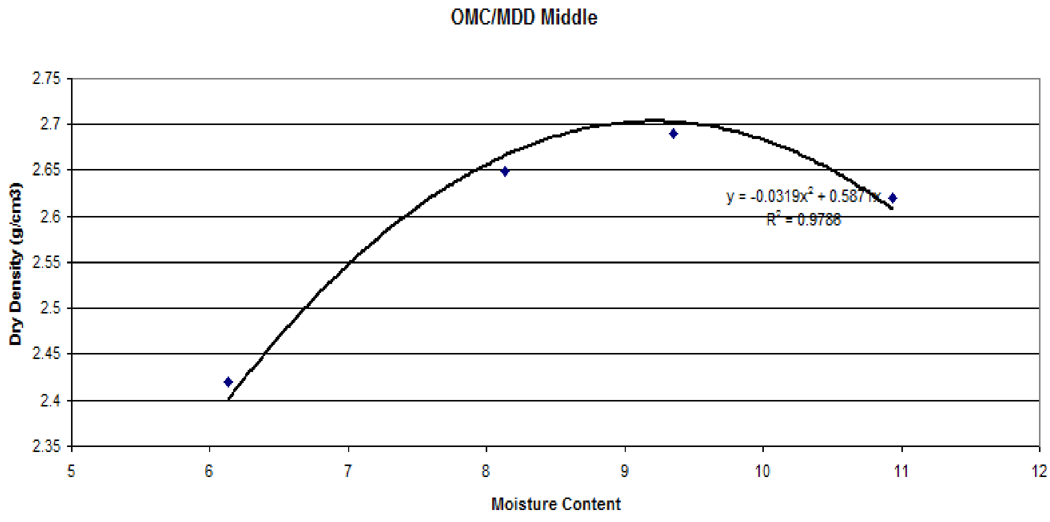

OMC/MDD Cohesive Surface
Dry Density (g/cm3)
2.55
2.50
2.45
2.40
2.35
2.30
2.25
2.20
2.15
2.10
2.05
7.00
8.00
9.00
10.00
11.00
12.00
13.00
14.00
15.00
Moisture Content
y = -0.0326x² + 0.7651x - 2
R² = -0.626

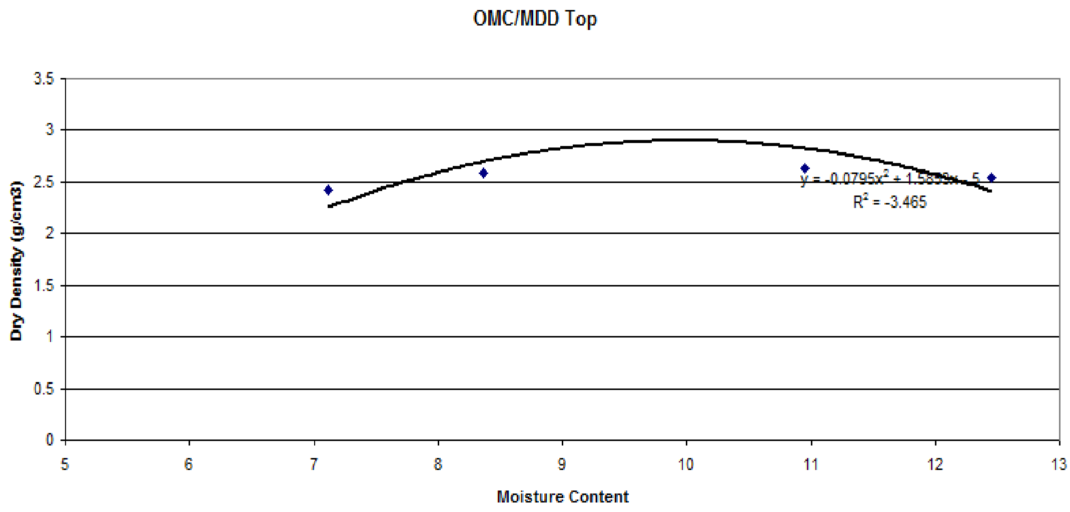
B2 – Compaction Plots for Mine B



B3 – Compaction Plots for Mine C

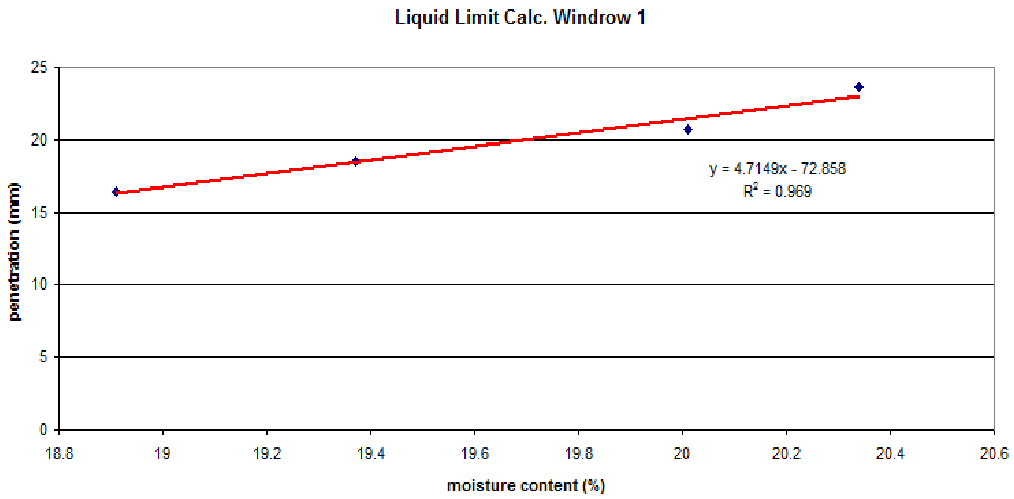


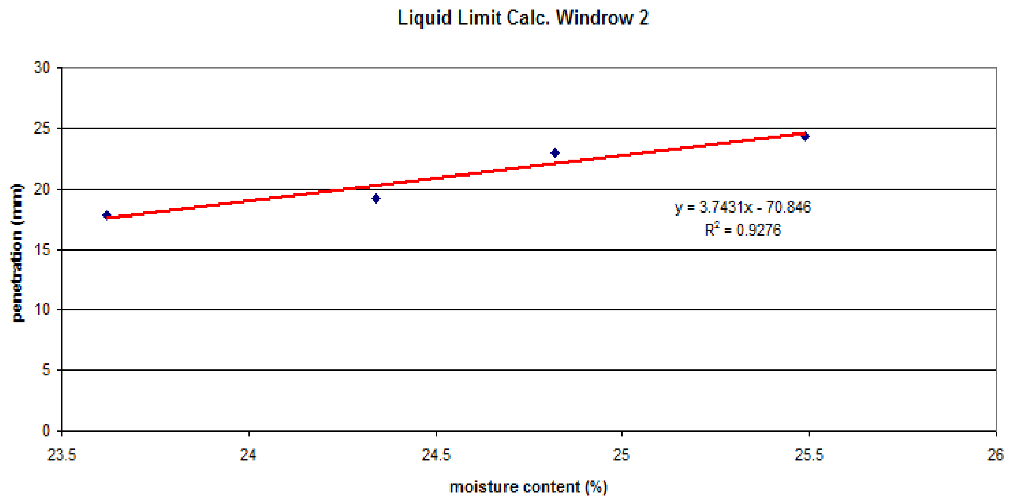

OMC/MDD Middle
Dry Density (g/cm3)
Moisture Content
y = -0.0319x² + 0.5871x
R² = 0.9788

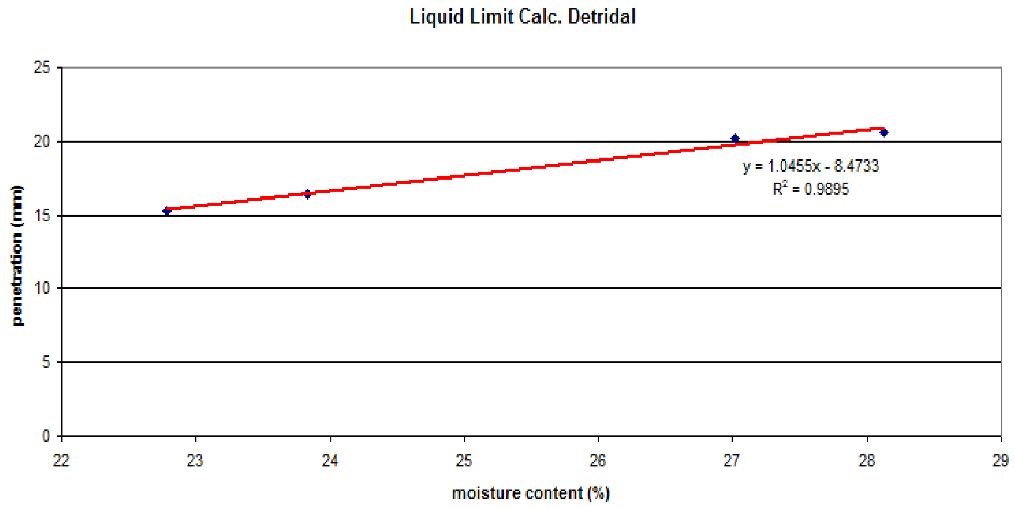

OMC/MDD Top
Dry Density (g/cm3)
Moisture Content
R² = -3.465

Appendix C – Liquid Limit Results

C 1 – Liquid Limit Results for Mine A – Untreated Materials

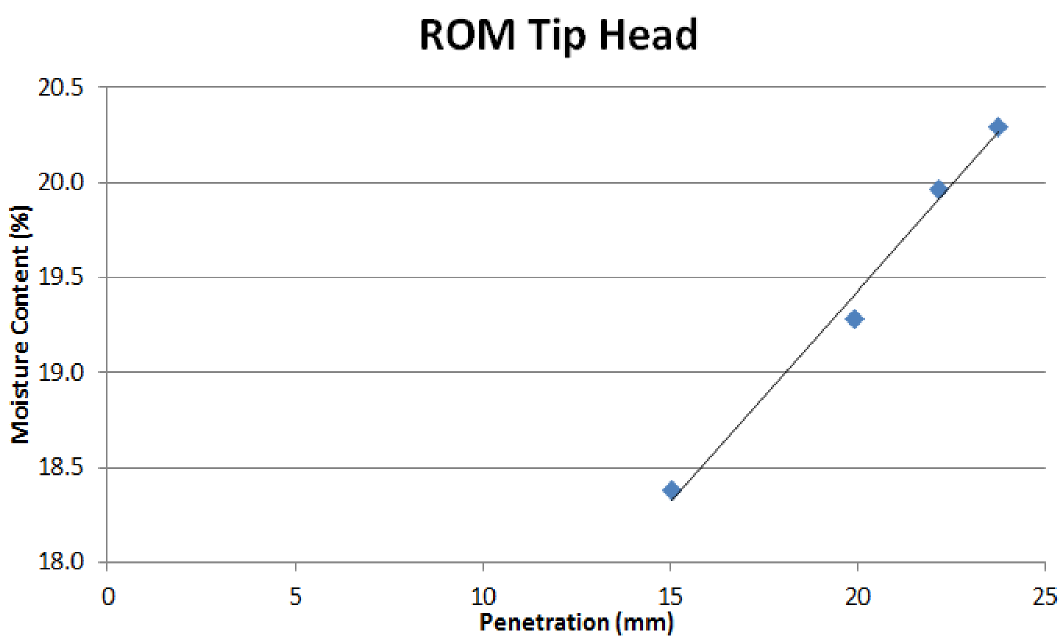


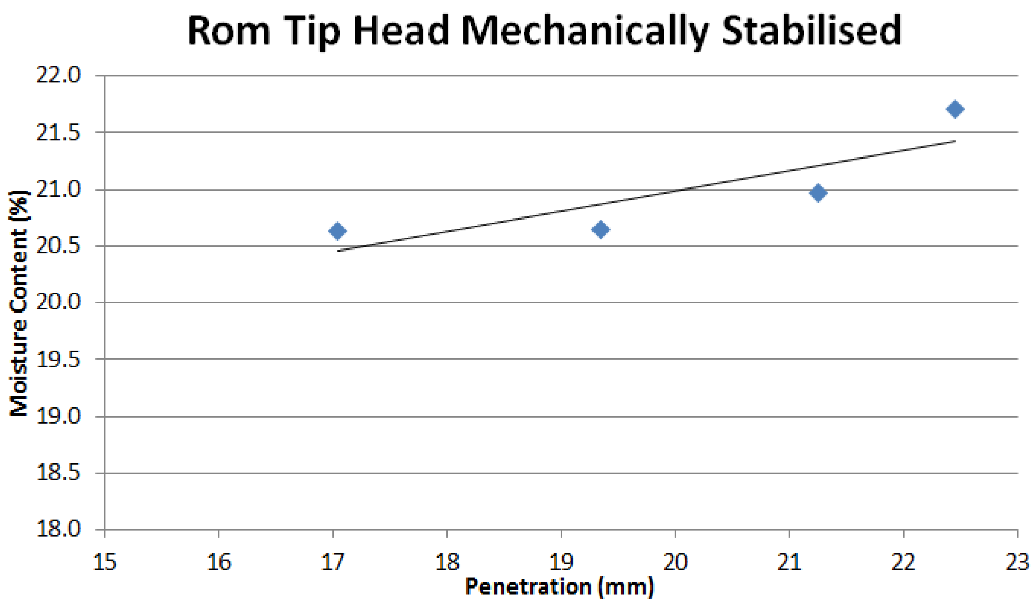


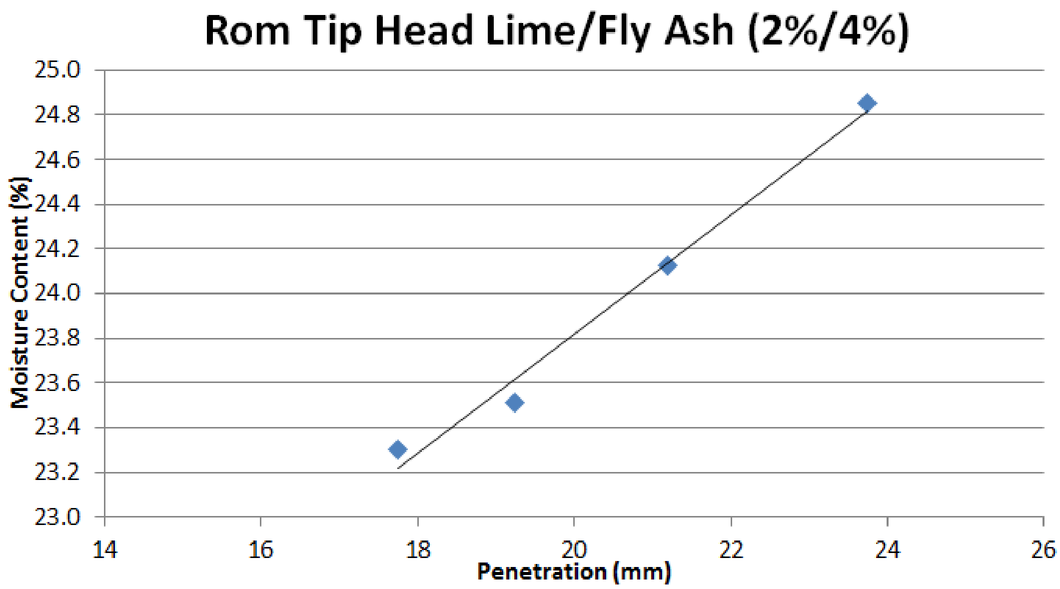


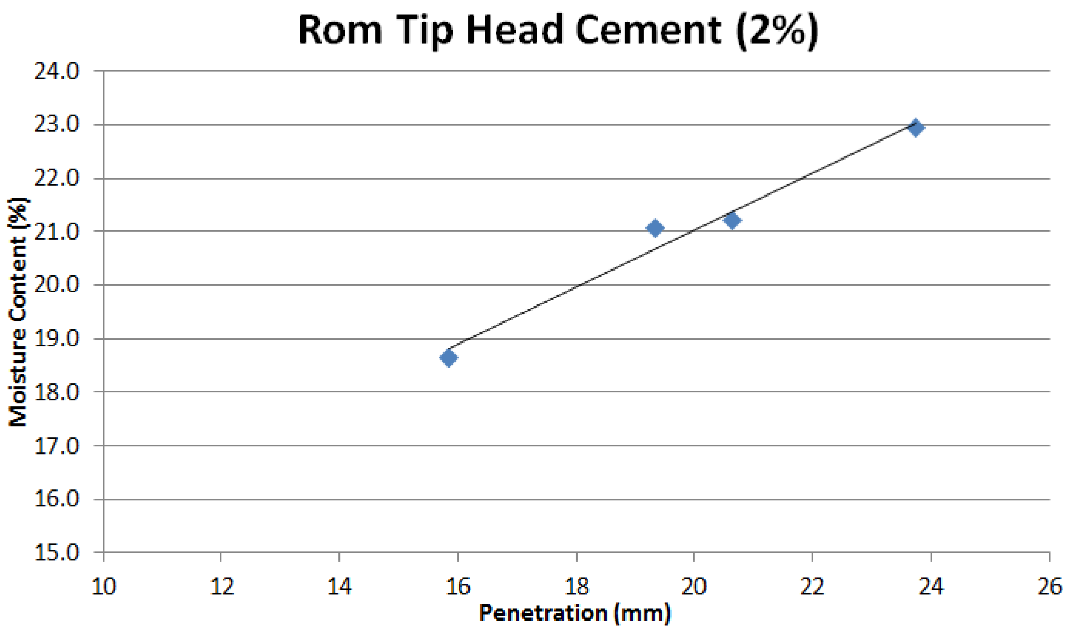
C2 – Liquid Limit Results for Mine A: Treated Materials

ROM Tip Head

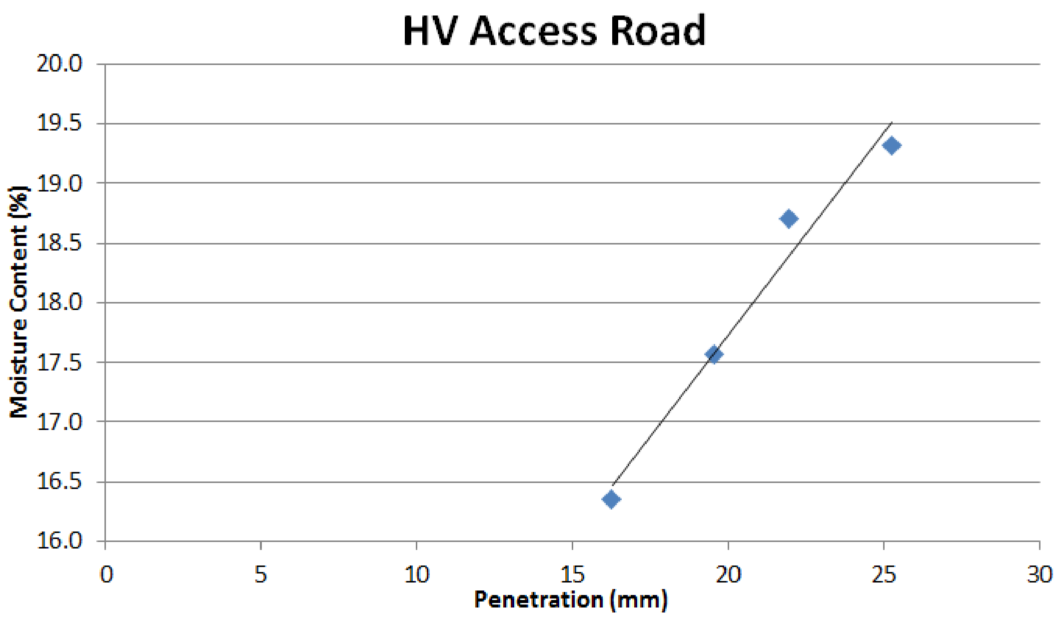


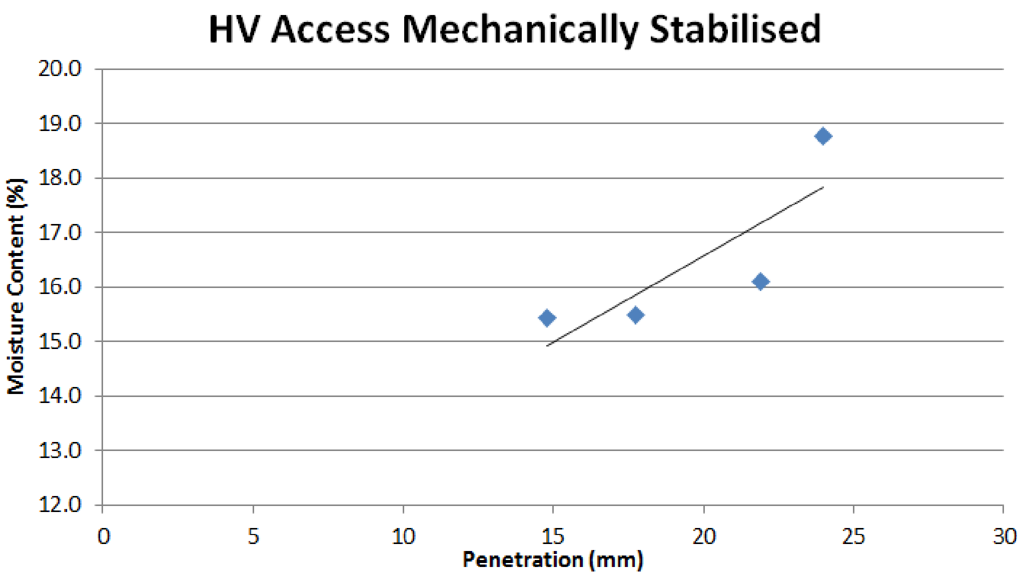

Rom Tip Head Mechanically Stabilised
Moisture Content (%)
22.0
21.5
21.0
20.5
20.0
19.5
19.0
18.5
18.0
15
16
17
18
19
20
21
22
23
Penetration (mm)

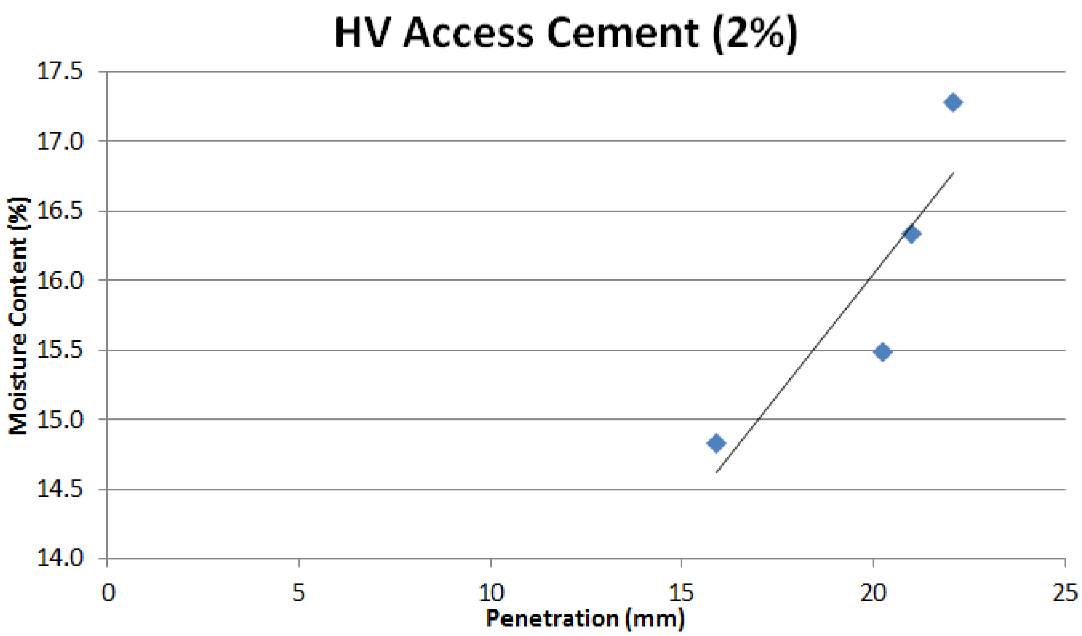

Rom Tip Head Lime/Fly Ash (2%/4%)
Moisture Content (%)
25.0
24.8
24.6
24.4
24.2
24.0
23.8
23.6
23.4
23.2
23.0
14
16
18
20
22
24
26
Penetration (mm)



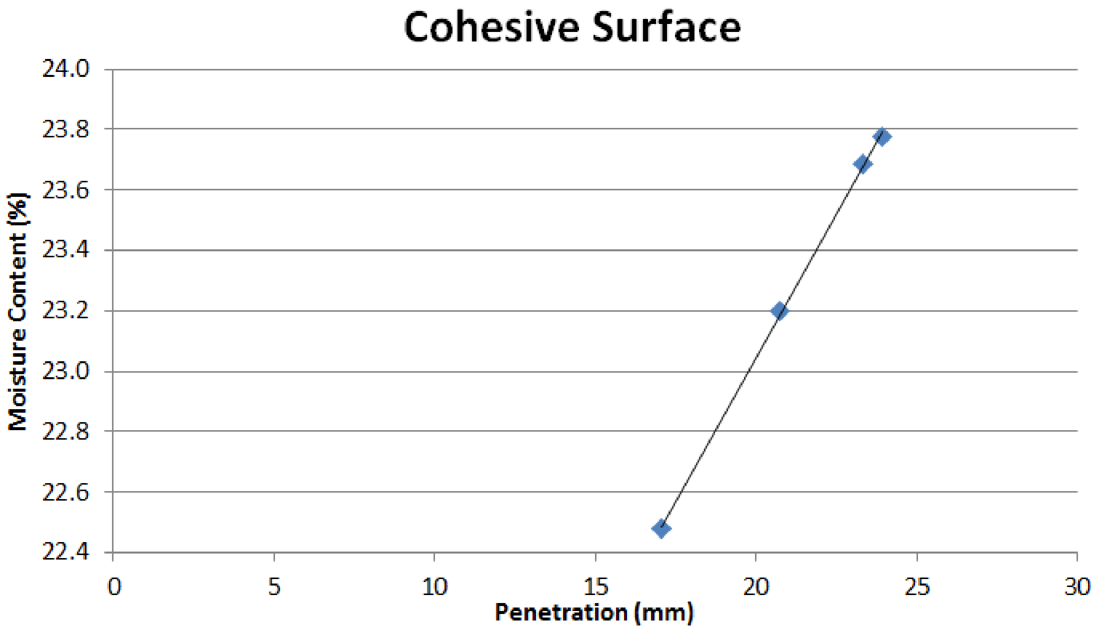
HV Access Road

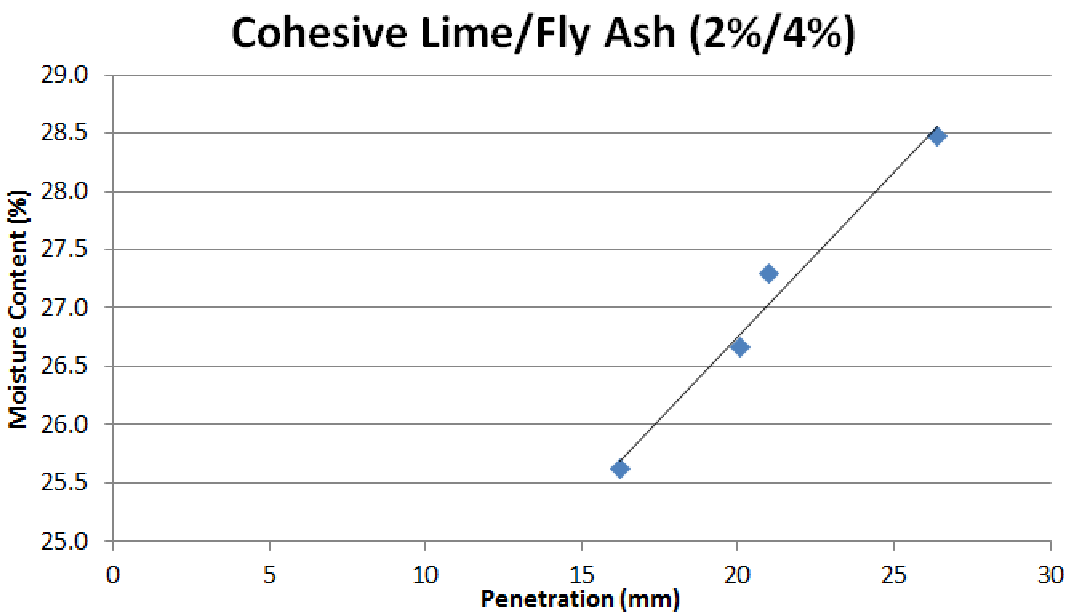


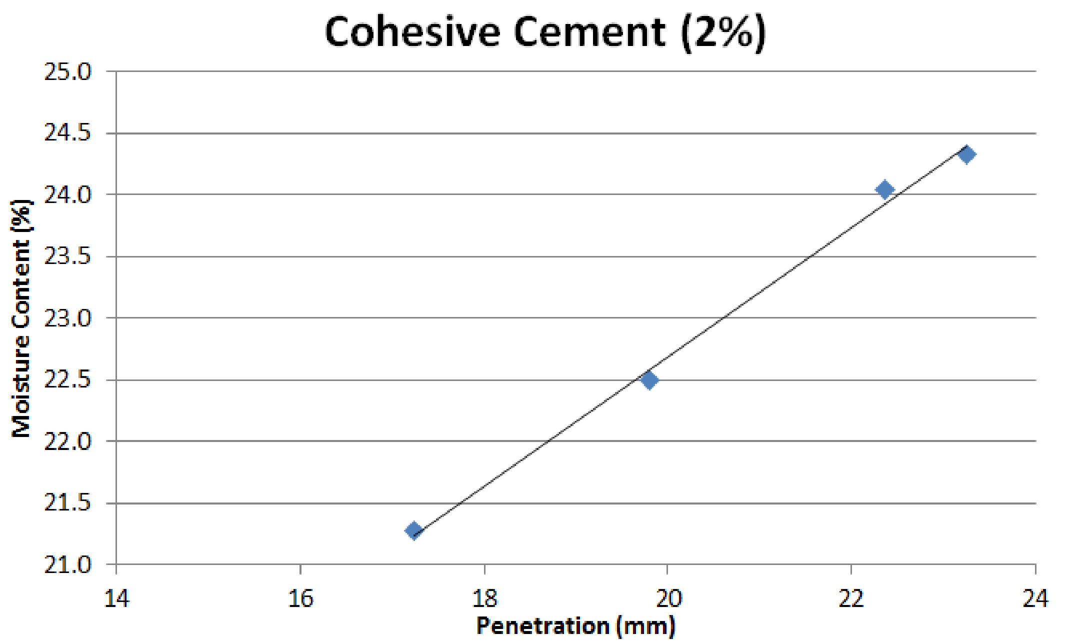

HV Access Mechanically Stabilised
Moisture Content (%)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
0
5
10
15
20
25
30
Penetration (mm)


HV Access Cement (2%)
Moisture Content (%)
17.5
17.0
16.5
16.0
15.5
15.0
14.5
14.0
0
5
10
15
20
25
Penetration (mm)

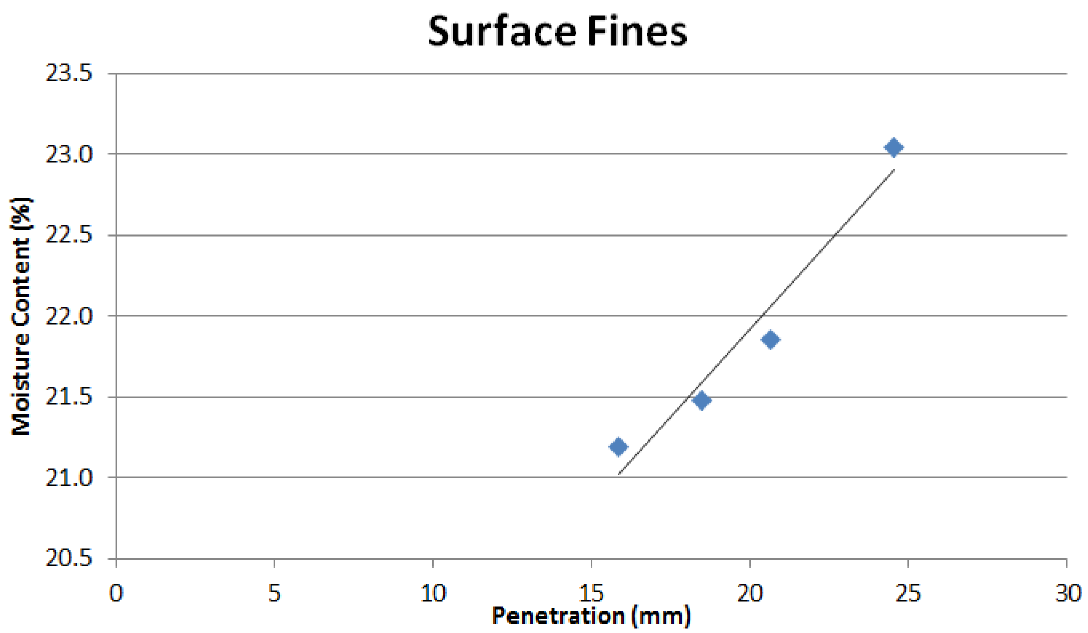
Cohesive Surface



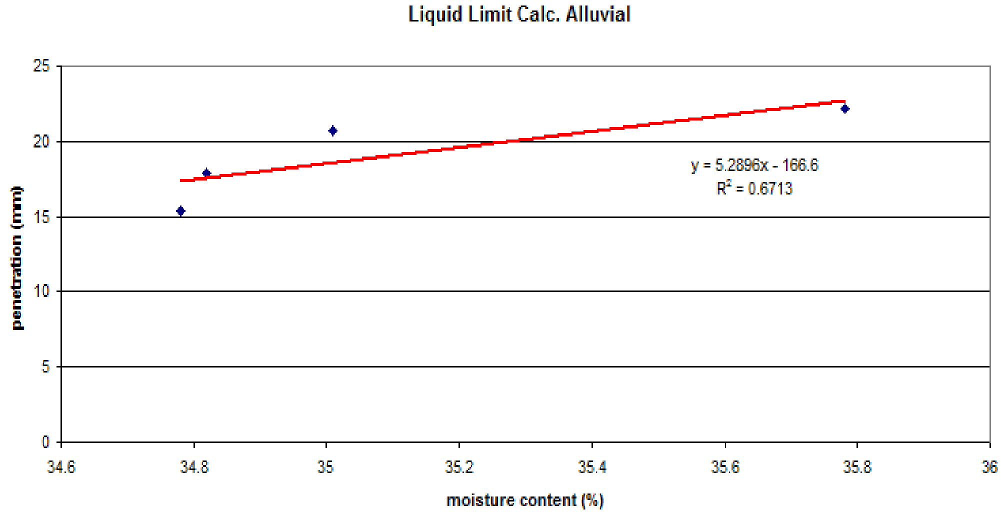




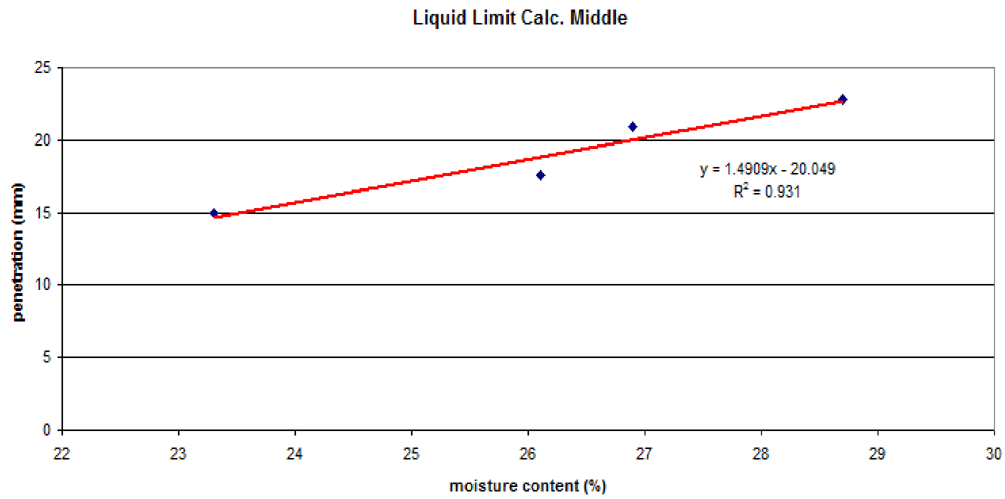
Surface Fines – Used as additive for mechanical stabilisations

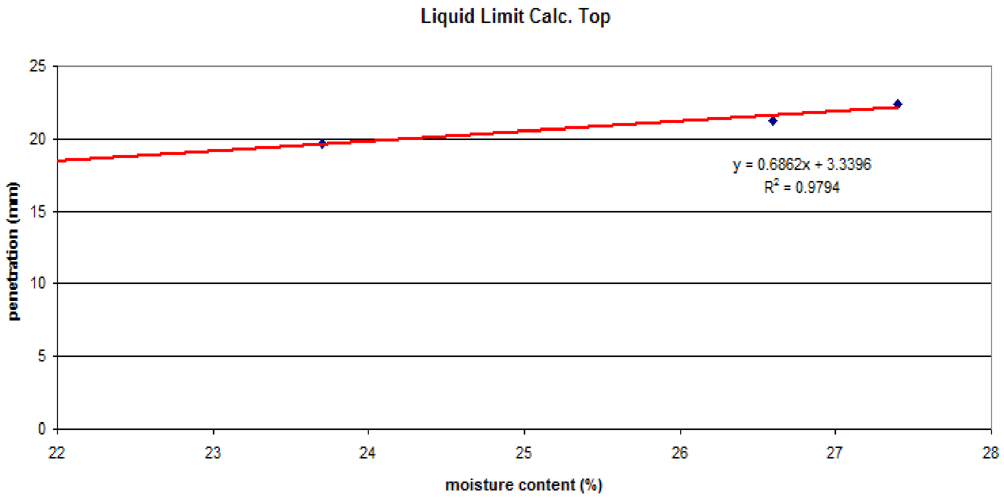


C3 – Liquid Limit Results for Mine B



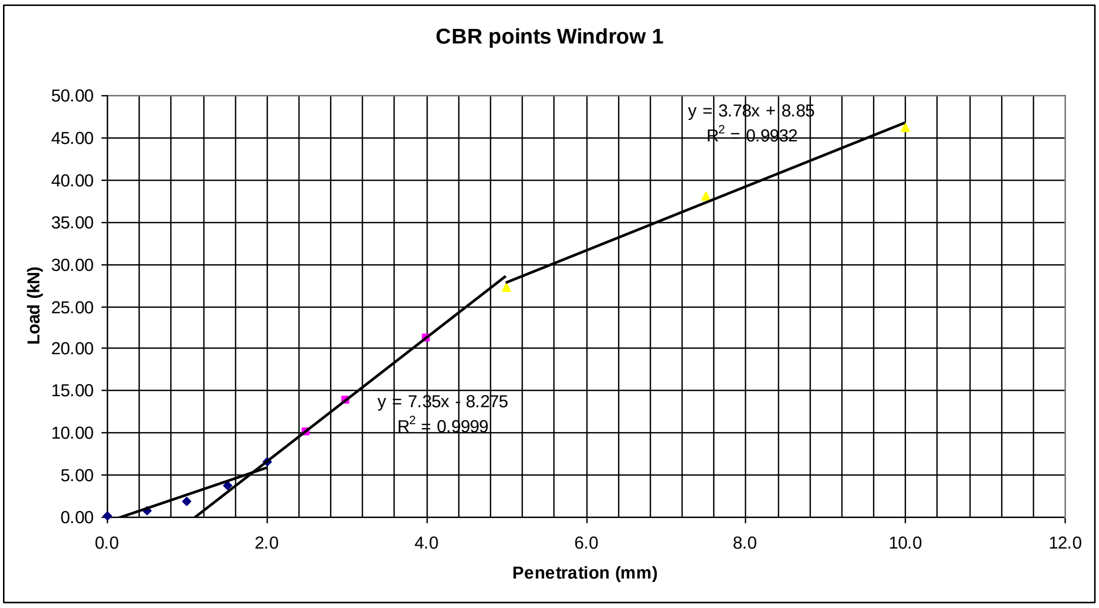
C4 – Liquid Limit Results for Mine C

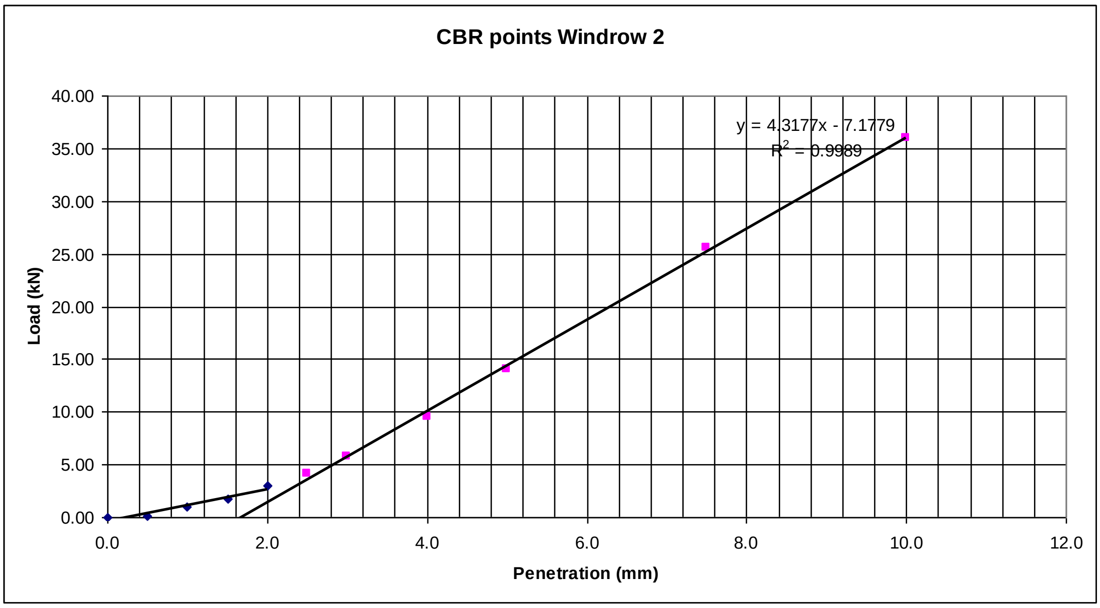


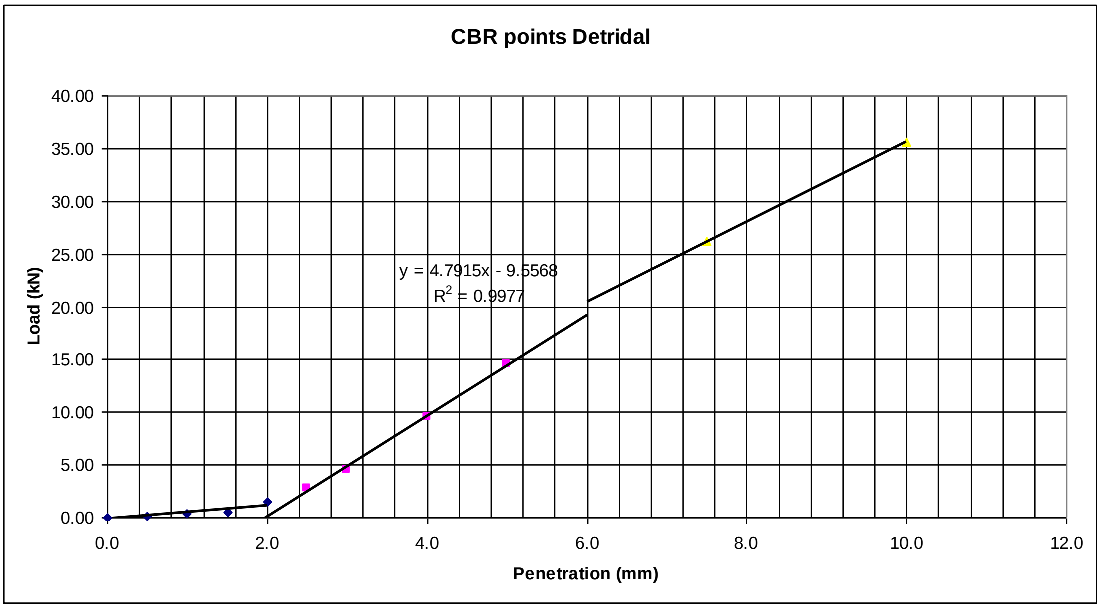

Liquid Limit Calc. Top
y = 0.6862x + 3.3396
R² = 0.9794
penetration (mm)
moisture content (%)
25
20
15
10
5
0
22
23
24
25
26
27
28

Appendix D – CBR Plots for Calculation

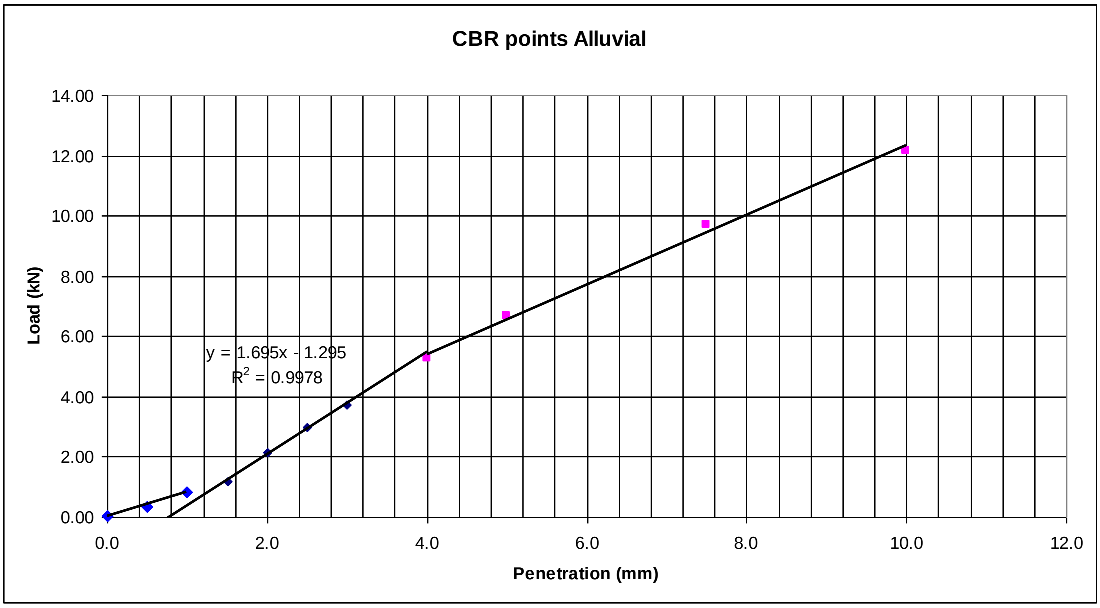
D1 – CBR Plots for Mine A



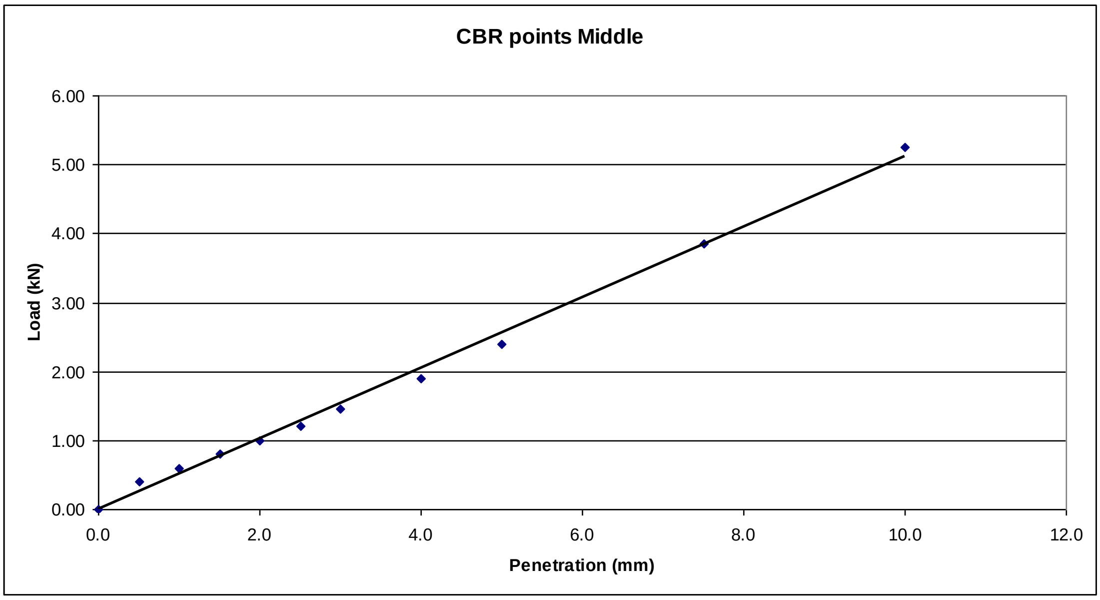


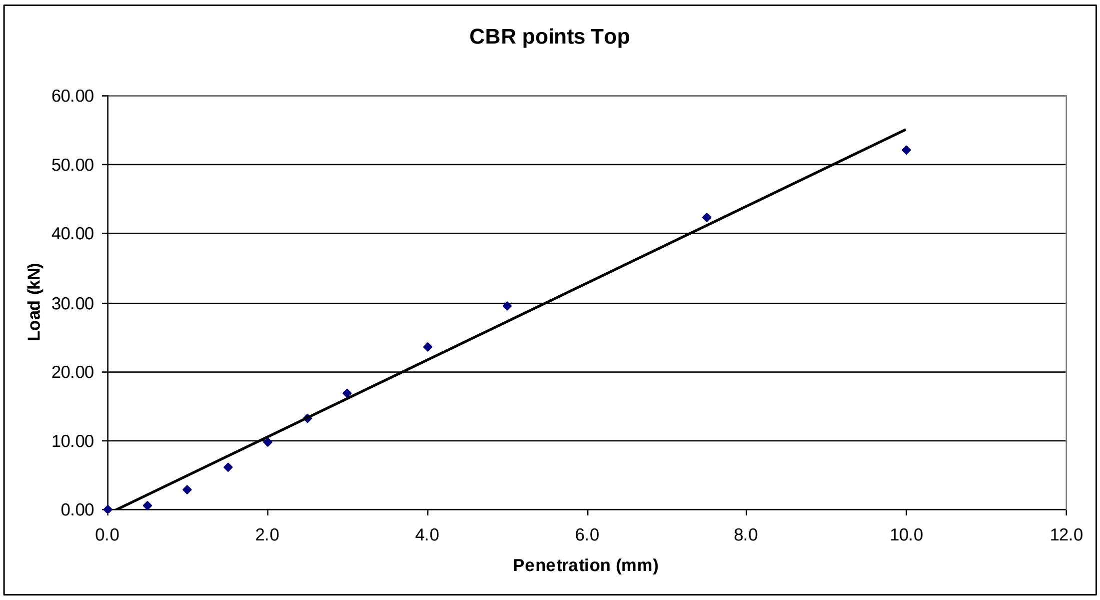


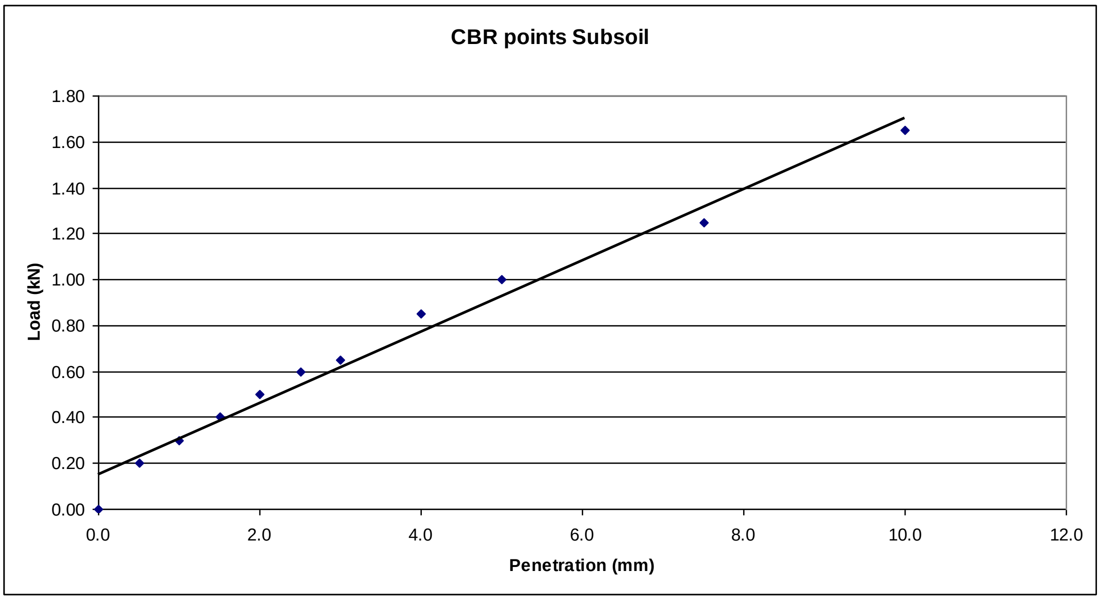
D2 – CBR Plots for Mine B



D3 – CBR Plots for Mine C

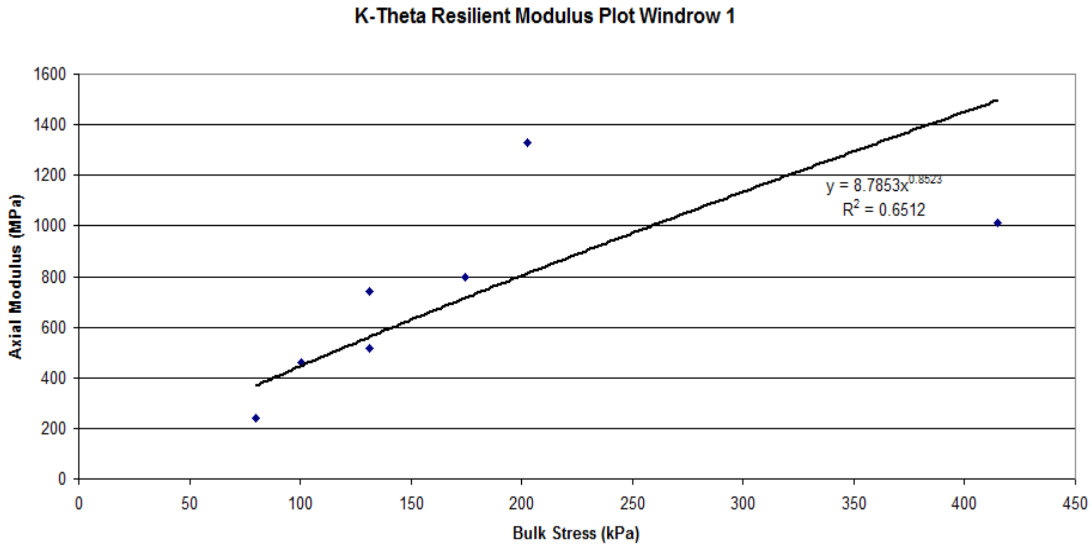


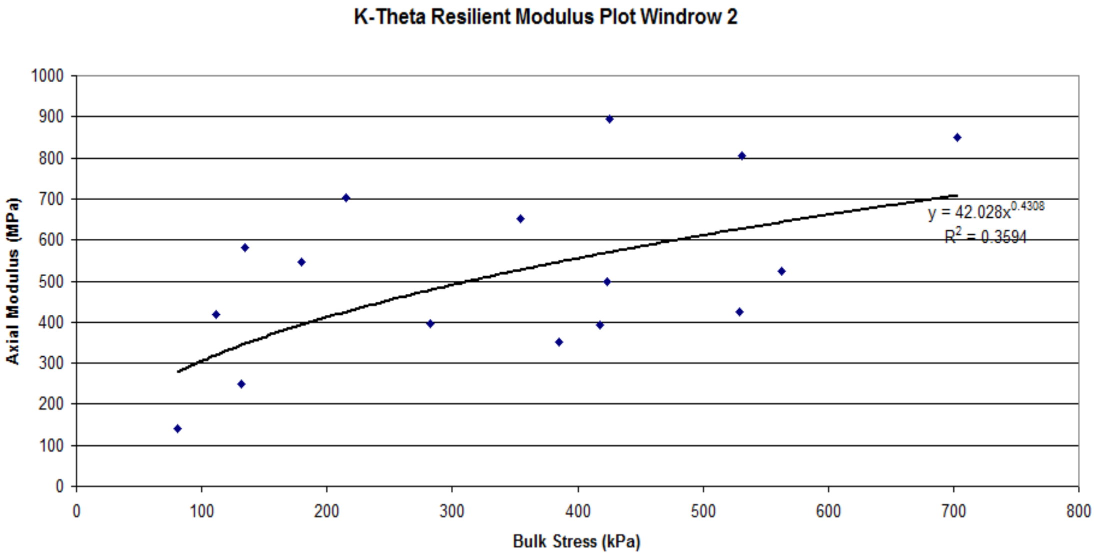


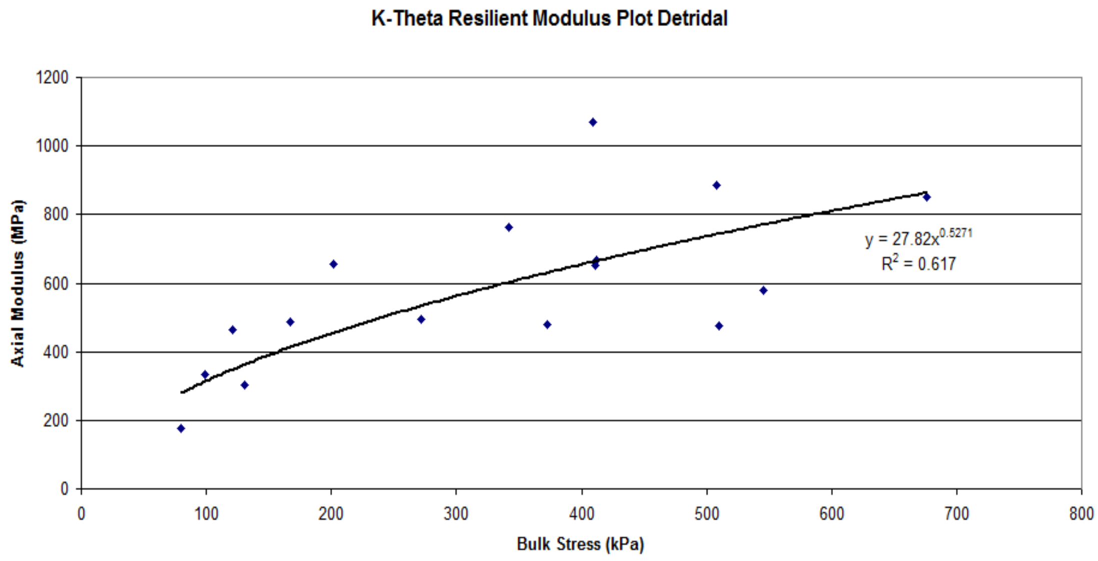

CBR points Subsoil
Load (kN)
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Penetration (mm)

Appendix E – K-Theta Plots for Resilient Modulus Calculation

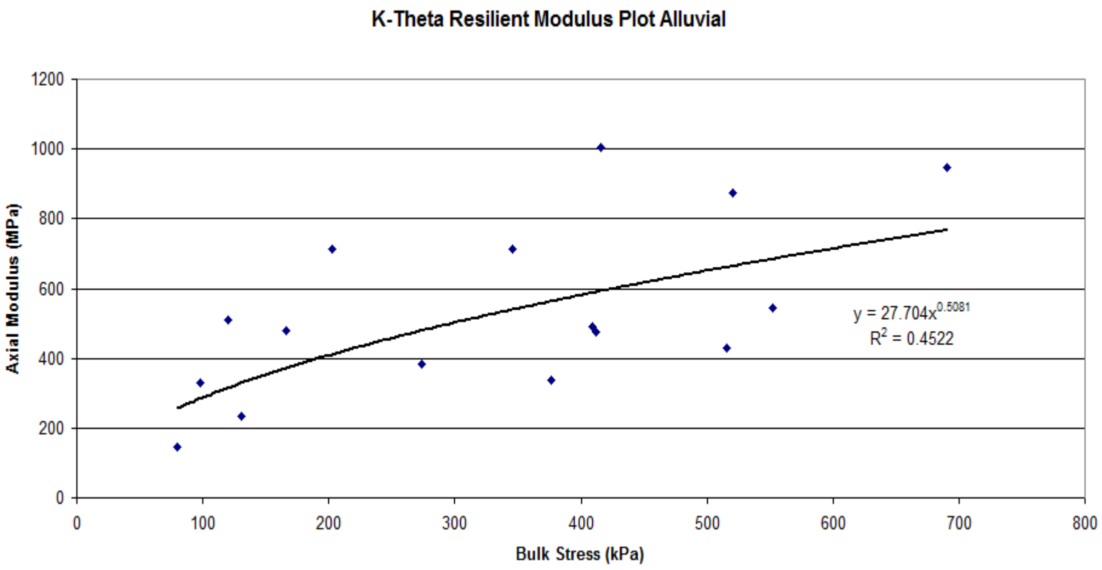
E1 – K-Theta Plots for Mine A



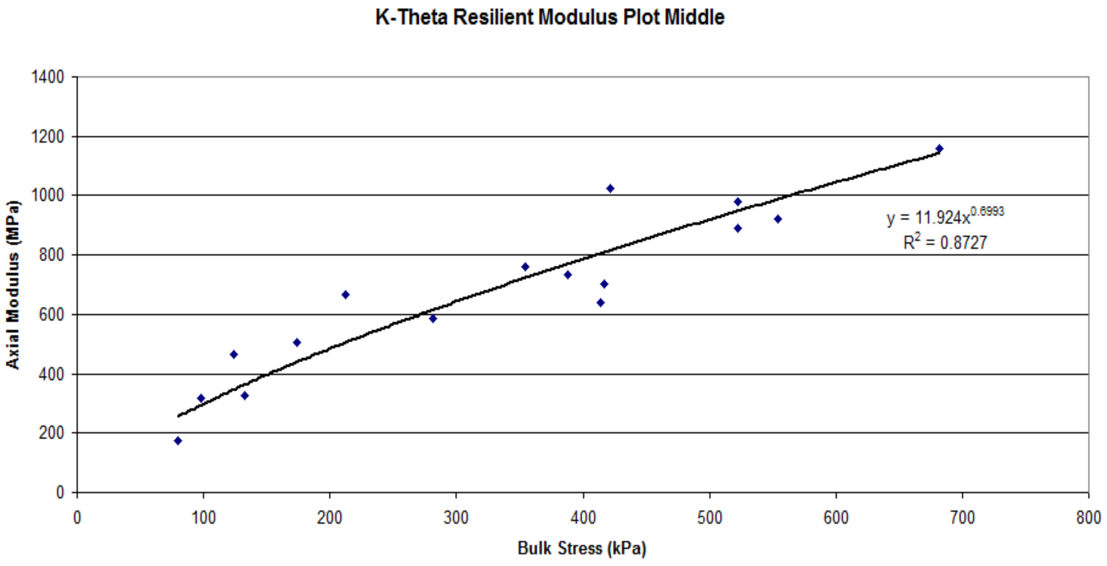


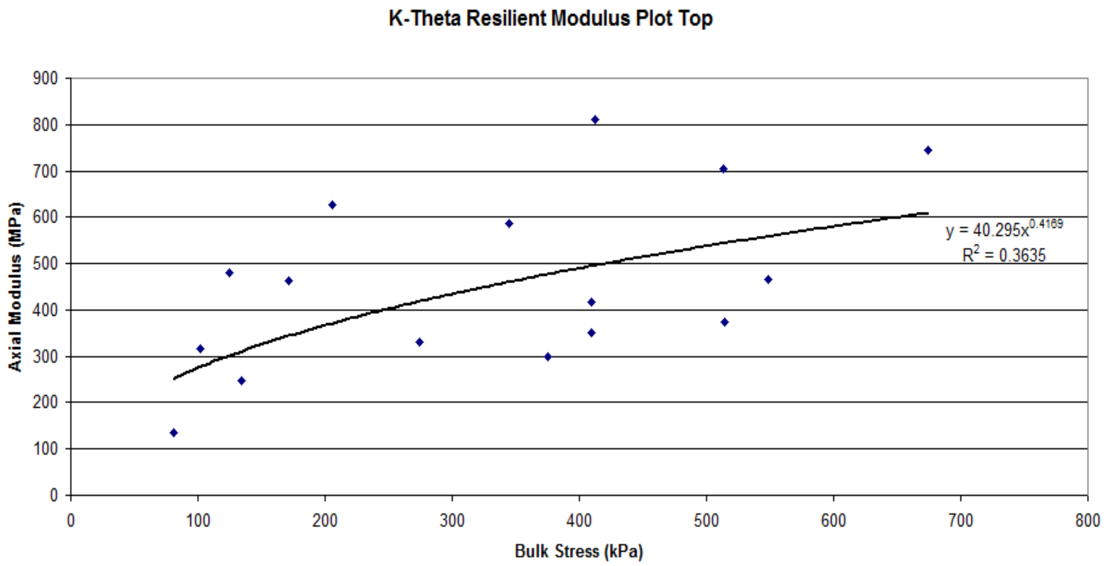


E2 – K-Theta Plots for Mine B



E3 – K-Theta Plots for Mine C

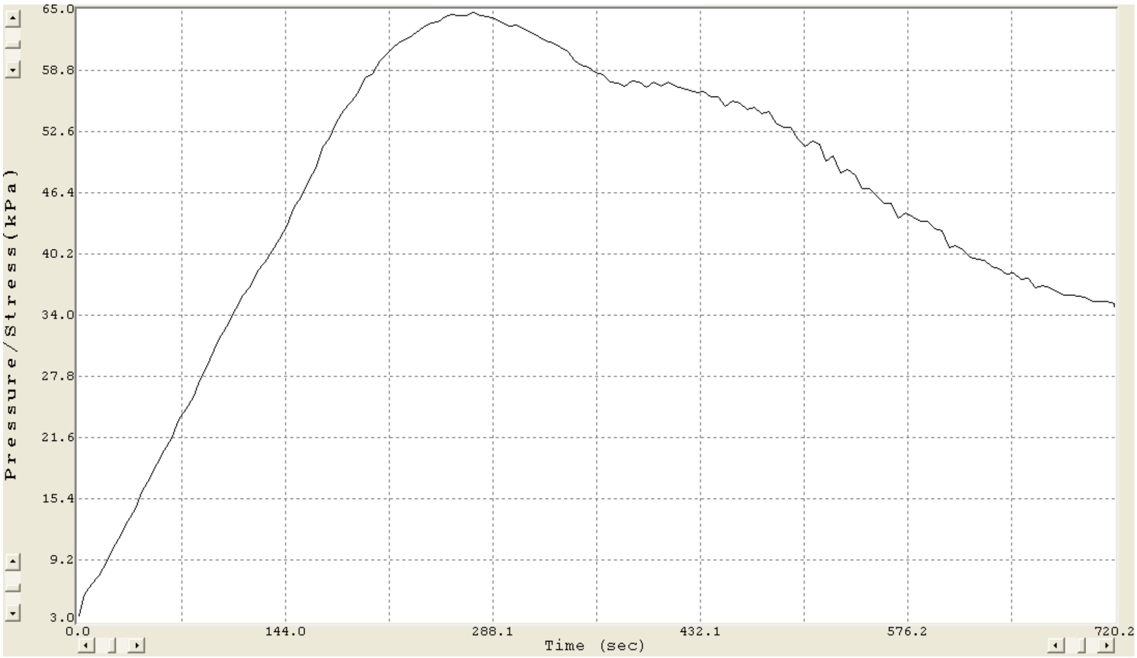




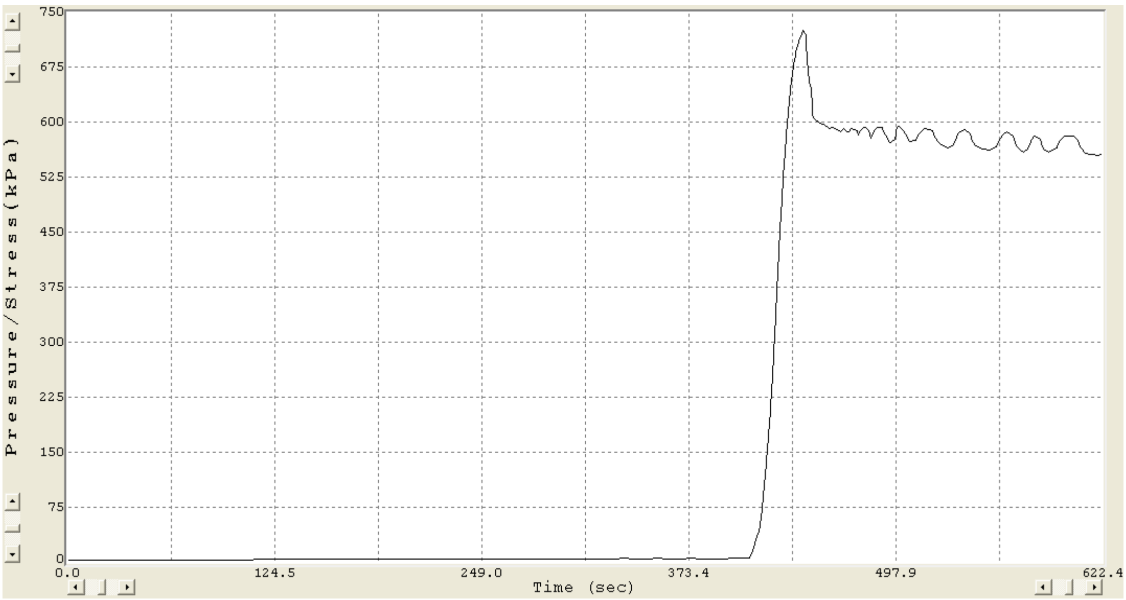
Appendix F – UCS Plots

Mine A

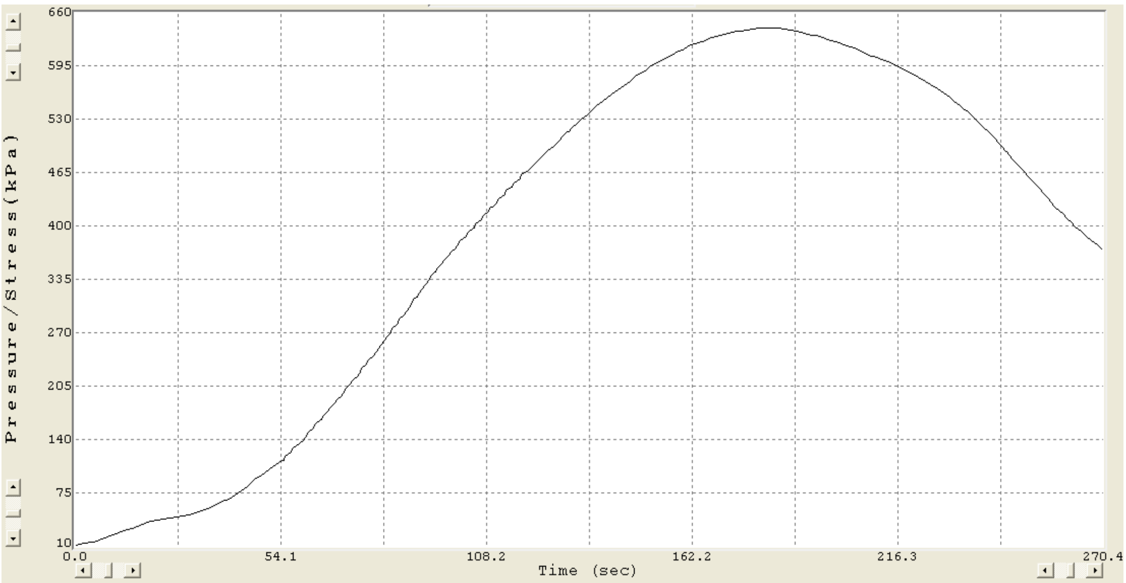
Cohesive Surface Untreated



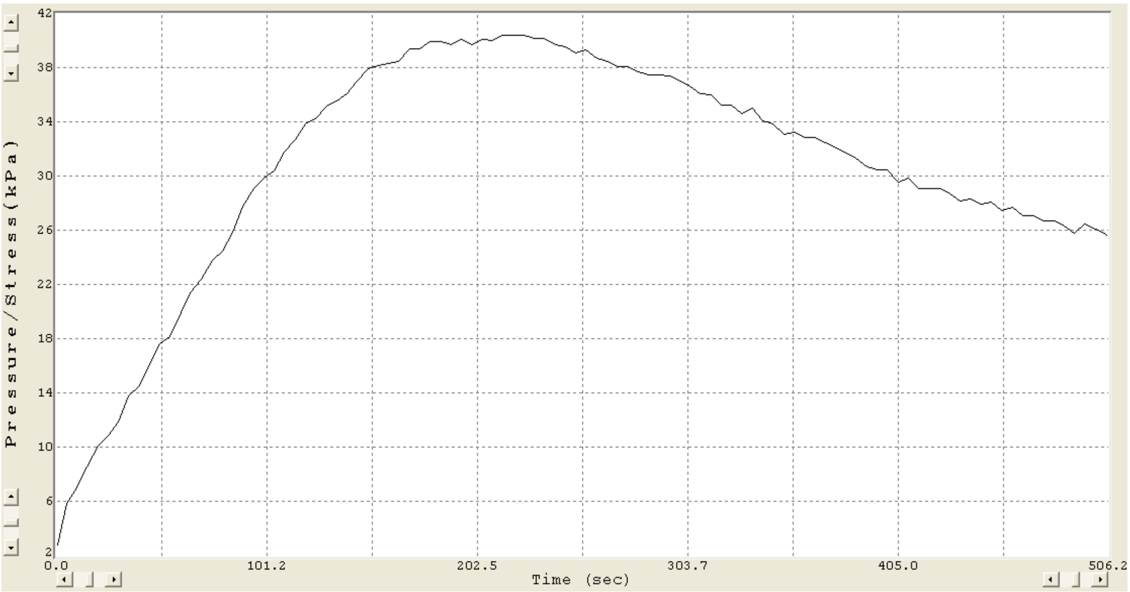
Cohesive Surface Cement Treated



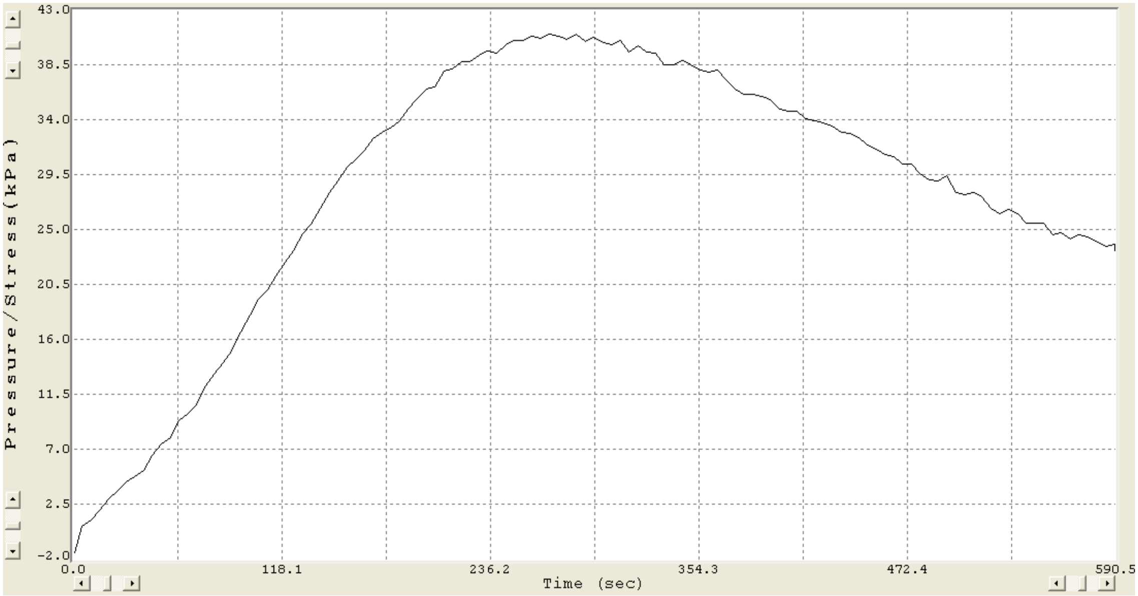
Cohesive Surface Lime/Fly Ash Treated



HV Access Road Untreated

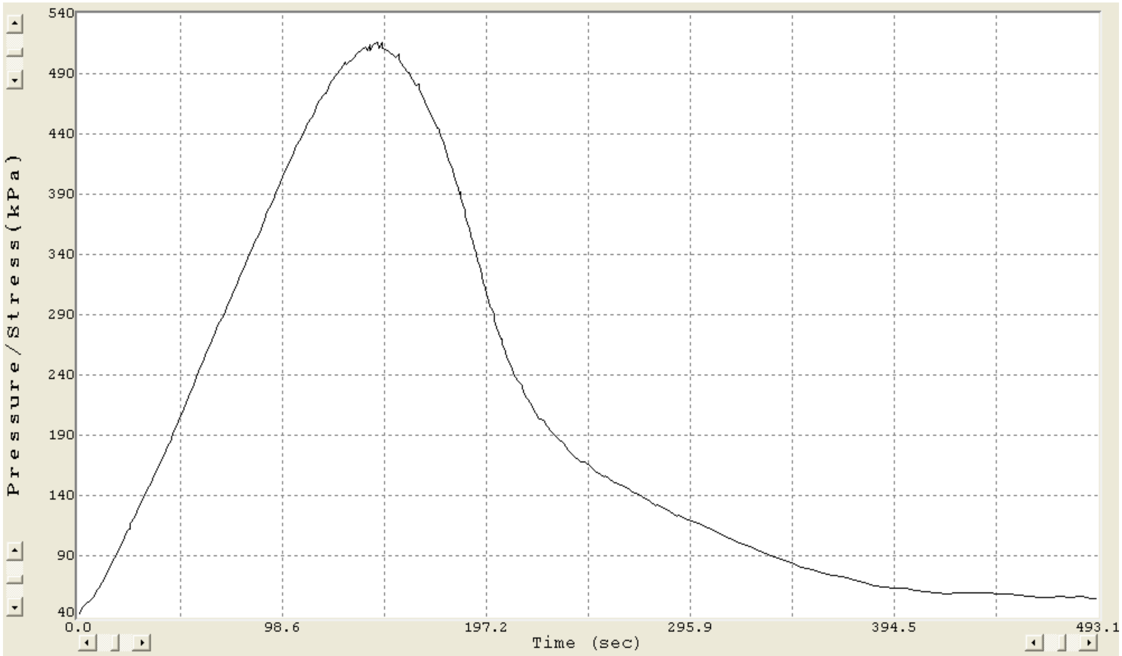


HV Access Mechanically Stabilised

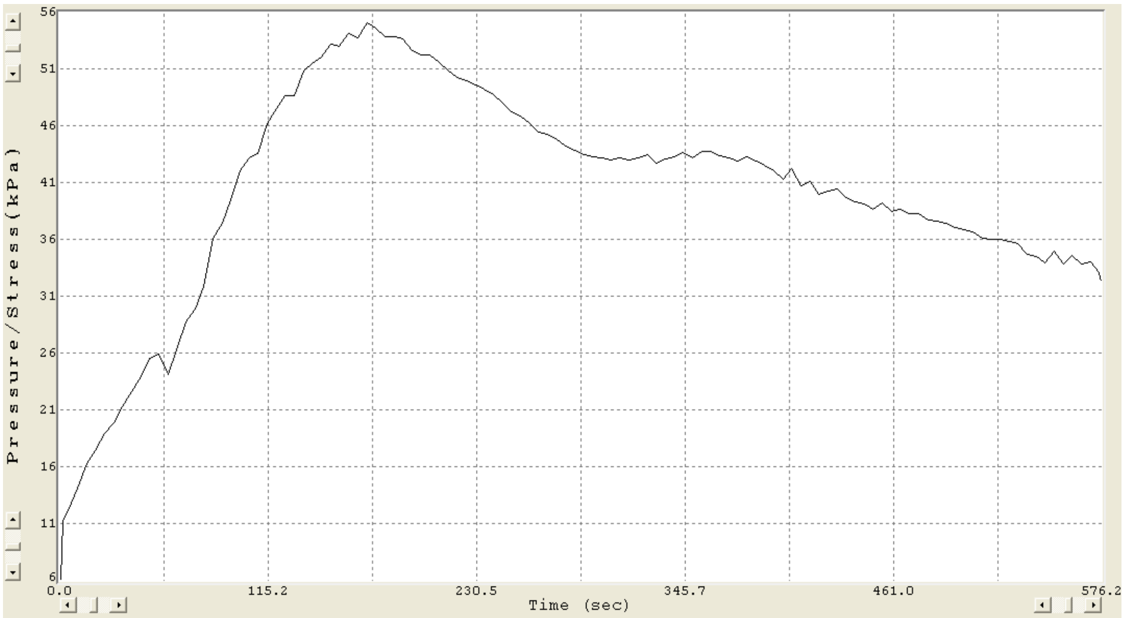


HV Access Cement Treated

HV Access Lime/Fly Ash Treated

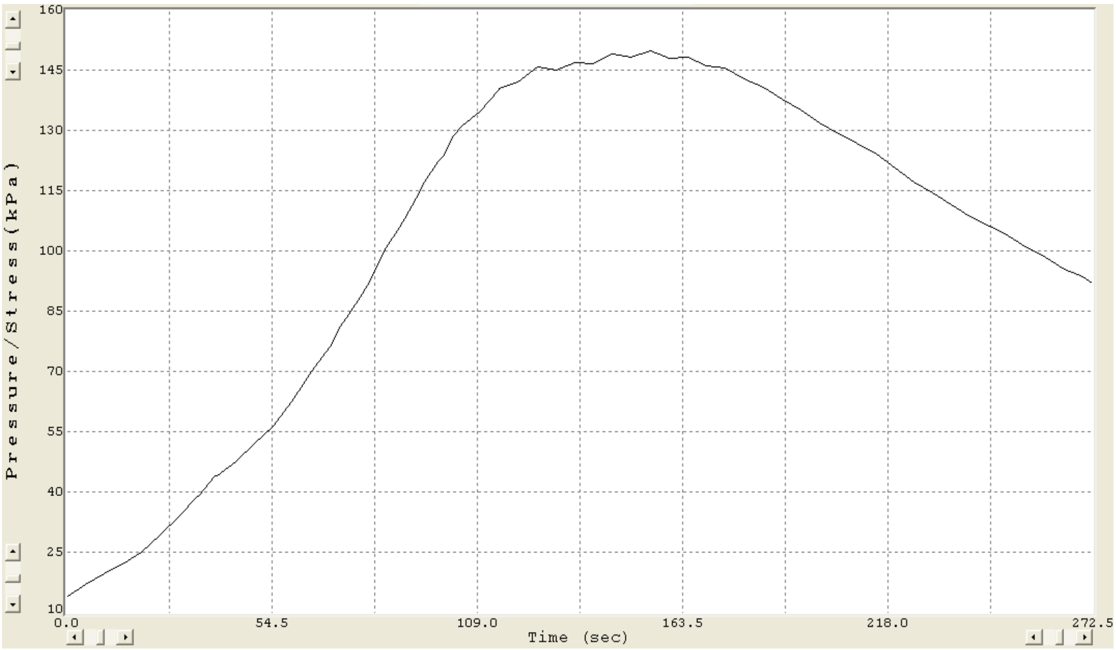


ROM Tip Head Untreated



ROM Tip Head Cement Treated

ROM Tip Head Lime/Fly Ash Treated



Appendix G – Capillary Rise and Swell Results

G1 – Summary of Results

	ROM Tip Head			HV Access		Cohesive		
	Untreated	LFA	Cement	LFA	Cement	Untreated	LFA	Cement
M_3	4026	3798	4063	4394	4354	4162	3967	4157
M_4	4027	3958	4090	4442	4358	4165	4127	4182
H_1	200	200	200	200	200	200	200	200
H_2	200	200	200	200	200	200	200	200
h	200	100	90	40	40	200	90	110
D_1	100	100	100	100	100	100	100	100
D_2	140	100	100	100	100	120	100	100
MC (%)	8.8	8.8	8.8	8.8	8.8	10.8	10.8	10.8
Dry Dens. (tonne/m^3)	2.5	2.5	2.5	2.6	2.6	2.5	2.5	2.5
CR (%)	100.00	47.37	42.11	15.79	15.79	100.00	42.11	52.63
M_5 (g)	360.98	360.98	360.98	375.42	375.42	354.47	354.47	354.47
Absorption (%)	0.28	44.32	7.48	12.79	1.07	0.85	45.14	7.05
Swell (%)	96.00	0.00	0.00	0.00	0.00	44.00	0.00	0.00

G2 – Photos Examples of Treatments

Cohesive Untreated:

Cohesive Lime/Fly Ash Treated:

Cohesive Cement Treated:

Appendix H – Defect Scoring

7.1.1. H1 – Layout form

		Assessment Category													
Defect	**Condition Score**	AP	Wt.	IS	Wt.	DP	Wt.	GE	Wt.	RC	Wt.	MP	Wt.	Product	**Total**
Potholes		1.4	5	0.3	5	3	2	3	2	2	3	1	1	27.5	
Corrugations		1.5		1		5		5		5		3		50.5	
Rutting		1.3		0.3		2		4		5		1		36	
Loose Material - Ravelling		2.7		4		5		5		4		3		68.5	
Dustiness		5.1		2		5		4		4		3		68.5	
Excessive clay - wet		3.5		5		2		4		3		2		65.5	
														Defect Score	

To complete: fill in the condition score for each defect and multiply it with the ‘Product’ to get the total score. Each defects total is then tallied to get the Defect Score for the haul road section. Note that each Assessment Category can be modified to those given in the matrixes for variation in Geometry and Remediation to properly reflect the situation being assessed (see Section 4.4).

7.1.2. H2 - Defect Scoring advice

Condition Scoring Categories (Adapted from Thompson, 2003):

Defect	Degree 1	Degree 2	Degree 3	Degree 4	Degree 5
Potholes	Surface is pock-marked <50mm in diameter	Potholes 50-100mm in diameter	Potholes 100-400mm in diameter	Potholes 400-800mm in diameter, influencing riding quality and obviously avoided by most vehicles	Potholes >800mm in diameter, influencing riding quality and require speed reduction or avoidance
Corrugations	Slight Corrugations, difficult to feel in Light Vehicle	Corrugations present and noticeable in light vehicles	Corrugations very visible and reduce riding quality noticeably	Corrugations noticeable in haul truck and causing driver to reduce speed	Corrugations noticeable in haul truck and causing driver to reduce speed significantly
Rutting	Difficult to discern unaided, <20mm	Just discernible with eye, 20-50mm	Discernible, 50-80mm	Obvious from moving vehicle, >80mm	Severe, affects direction stability of vehicle
Loose Material - Ravelling	Very loose material on road, <5mm depth	Small amount of loose material on road to a depth of 5-10mm	Lose material present on road to a depth of 10-20mm	Significant loose material on road of 20-40mm	
Dustiness	Dust just visible behind vehicle	Dust visible, no oncoming vehicle driver discomfort, good visibility	Notable amount of dust, windows closed in oncoming vehicle if they had	Potholes 400-800mm in diameter, influencing riding quality and obviously avoided by	Very dusty, surroundings obscured to a dangerous level

			been open, visibility just acceptable	most vehicles	
Excessive clay - wet	Wearing course material of good quality, road properly cambered	Wearing course strength and PI acceptable, road cambered	Wearing course strength low, PI fairly high, unsatisfactory camber	Wearing course strength low, PI high, water standing on surface when raining	Wearing course strength very low, PI very high, road very slippery when wet

Dust Category Photos (Thompson, 2007):