Liquefaction of Early Age Cemented Paste Backfill

by

Abdolreza Saebimoghaddam

A thesis submitted in conformity with the requirements for the degree of Doctor of Philosophy
Department of Civil Engineering
University of Toronto

© Copyright by Abdolreza Saebimoghaddam (2010)
Liquefaction of Early Age Cemented Paste Backfill

Abdolreza Saebimoghaddam

Doctor of Philosophy

Department of Civil Engineering
University of Toronto

2010

Abstract

Modern mines require systems that quickly deliver backfill to support the rock mass surrounding underground openings. Cemented Paste Backfill (CPB) is one such backfilling method, but concerns have been raised about CPB’s liquefaction susceptibility especially when the material has just been placed, and if it is exposed to earthquakes or large mining induced seismic events. Conventional geotechnical earthquake engineering for surface structures is now relatively advanced and well accepted, and so the objective of this thesis is to consider how that framework might be extended to assess the liquefaction potential of CPB.

Seismic records were analyzed for earthquakes and for large mining induced events. Important seismological trends were consistent for rockbursts and earthquakes when the signals were recorded at distances as proximate as one kilometre, suggesting that the conventional earthquake engineering approach might plausibly be adapted for such design situations. For production blasts and for more proximate locations to rockbursts, much higher frequencies dominate and therefore new design methods may be required.

Monotonic triaxial tests conducted on normally consolidated uncemented mine tailings demonstrated that the material is initially contractive up to a phase transition point, beyond which dilation occurs. Most importantly the material never exhibits unstable strain softening behaviour.
in compression, and only temporary or limited liquefaction in extension. The addition of 3% binder results in initial sample void ratios that are even higher than their uncemented counterparts, and yet the material friction is slightly enhanced when tested at 4 hours cure. These results suggest that the flow liquefaction phenomenon commonly associate with undrained loose sand fills will not occur with paste backfill.

Cyclic triaxial test results analyzed in terms of number of cycles to failure for a given cyclic stress ratio exhibited a trend consistent with previous tests on similar materials. However, the addition of 3% binder and testing at 4 hours cure resulted in an order of magnitude larger number of cycles to failure – a surprising and dramatic increase, suggesting good resistance of the material to cyclic mobility.

Future research is recommended to build on these results and develop more robust methods for liquefaction assessment of CPB.
Acknowledgments

I would like to express my sincere gratitude to my supervisor Professor Murray Grabinsky for his wisdom, guidance, patience and encouragement throughout this process and for giving me the opportunity to study at the University of Toronto.

I am appreciative to my advisors, Professor Mohanty, Professor Bawden and Professor Hooton for their constructive advices and guidance. I am also grateful to Dr. Robert Mercer from the Williams Mine and Mr. Norm Disley from the Kidd Mine for providing me with technical information.

I would like to extend my sincere appreciation to our research assistant, Dr. Dragana Simon for her support, kindness and friendship over the past four years. I am also thankful to our fellow post-doc, Dr. Ben Thompson and our lab manager, Dr. Farzin Nasseri for their helps and friendships.

I would also like to thank my fellow graduate students, Omid Khajeh Mahabadi, Abdullah Abedaal, Leonardo Trivino, Ryan Veenstra for their helps and persistent friendships and all the graduate students of GB313; I value all of our shared conversations.

I wish to deeply thank to my family for their unceasing love, support and encouragement.

Some part of the financial support received through the National Sciences and Engineering Research Council of Canada (NSERC) is gratefully appreciated. The cooperation of mining companies, Williams Mine and Kidd Mine, is also appreciated.
This thesis is dedicated to:

Shabnam, Sepid, Hamid and My Parents
# Table of Contents

Table of Contents ........................................................................................................................... vi

List of Tables ................................................................................................................................. xii

List of Figures ............................................................................................................................... xiv

List of Appendices ....................................................................................................................... xxii

CHAPTER 1 .................................................................................................................................... 1

1. Introduction ................................................................................................................................ 1

   1.1. Problem Statement ............................................................................................................... 1

   1.2. Objectives ............................................................................................................................ 2

   1.3. Thesis Organization ............................................................................................................. 3

CHAPTER 2 .................................................................................................................................... 5

2. Cemented Paste Backfill ............................................................................................................. 5

   2.1. Mine Tailings ....................................................................................................................... 6

   2.2. Binder Agents ...................................................................................................................... 8

       2.2.1. Portland Cement ......................................................................................................... 8

       2.2.2. Supplementary Cementing Materials ...................................................................... 10

           2.2.2.1. Fly Ash .............................................................................................................. 10

           2.2.2.2. Blast Furnace Slag ......................................................................................... 11

   2.3. Strength of Cemented Paste Backfill ................................................................................ 11

       2.3.1. Parameters Affecting the UCS of CPB ................................................................. 12

       2.3.2. Shear Strength Development ............................................................................... 15

       2.3.3. Long Term Strength ............................................................................................... 18

   2.4. Summary ............................................................................................................................ 19

CHAPTER 3 .................................................................................................................................. 21

3. Literature Review ...................................................................................................................... 21
3.1. Definition of Liquefaction ................................................................................................... 22
3.2. Liquefaction Susceptibility and Its Mechanisms .................................................................. 23
  3.2.1. State Criteria ............................................................................................................... 23
    3.2.1.1. Critical Void Ratio ............................................................................................ 23
    3.2.1.2. Steady State of Deformation .......................................................................... 24
  3.2.2. Flow Liquefaction Surface .......................................................................................... 26
    3.2.2.1. Flow Liquefaction ............................................................................................ 27
    3.2.2.2. Cyclic Mobility ................................................................................................. 27
3.3. Dynamic Loads .................................................................................................................... 29
  3.3.1. Earthquake-induced Stress Waves .............................................................................. 30
    3.3.1.1. Ground Motion Parameters .............................................................................. 31
    3.3.1.2. Cyclic Stress Approach .................................................................................... 34
  3.3.2. Rockburst-induced Stress Waves .............................................................................. 36
  3.3.3. Blast-induced Stress Waves ...................................................................................... 40
3.4. Mining Related Liquefaction Studies ................................................................................... 41
  3.4.1. Monotonic and Cyclic Loadings induced Liquefaction for Mine Tailings and CPB ......... 41
  3.4.2. Rockburst-induced Liquefaction .............................................................................. 42
  3.4.3. Blast-induced Liquefaction ...................................................................................... 44
3.5. Additional Material Parameters Affecting Liquefaction ..................................................... 48
  3.5.1. State of Saturation and Occluded Air .......................................................................... 49
  3.5.2. Relative Density and Particle Characteristics ............................................................ 49
  3.5.3. Effect of Plasticity Index ........................................................................................... 50
  3.5.4. Effect of Cement ...................................................................................................... 50
  3.5.5. Effective Confining Stress and Fines Content ............................................................ 52
3.6. Liquefaction Susceptibility Criteria for Fine Grained Soils .............................................. 53
3.6.1. Chinese Criteria .......................................................................................................55
3.6.2. Criteria for Silts .......................................................................................................56
3.7. Summary and Plan of Work ..............................................................................................58
  3.7.1 Summary ...................................................................................................................58
    3.7.1.1 Behaviour of fine grained soils .............................................................59
    3.7.1.2 Dynamic Loadings .................................................................................60
  3.7.2 Plan of Work .............................................................................................................61
CHAPTER 4 ..................................................................................................................................62
4. Characteristics of Seismic Events ...........................................................................................62
  4.1. Seismic Events in North Eastern Ontario .................................................................65
    4.1.1. Seismographs in North Eastern Ontario ..................................................65
    4.1.2. Natural Earthquakes in North Eastern Ontario ........................................67
    4.1.3. Mining Related Events in North Eastern Ontario ....................................69
    4.1.4. The Earthquakes versus the Rockbursts in Northern Ontario .............73
  4.2. Mining Seismic Events Recorded at Mines .............................................................81
    4.2.1. Rockbursts .................................................................................................83
    4.2.2. Production Blasts .......................................................................................88
    4.2.3. Seismic Events Recorded at the Mines versus the NRCan Events ........91
  4.3. Near-Field Blasting Events ....................................................................................95
    4.3.1. Overview and Instrumentation .................................................................95
    4.3.2. Blasting Program at Williams Mine ..........................................................97
    4.3.3. Acceleration-time Series ............................................................................98
    4.3.4. Data Processing .........................................................................................98
    4.3.5. Blasting Results .........................................................................................100
      4.3.5.1. Variation in PPA and PPV in Rock ..................................................100
      4.3.5.2. Variation in Frequency for PPA and PPV in Rock .........................100
4.3.5.3. Variation in PPA and PPV in CPB ................................................................. 100
4.3.5.4. Variation in Frequency for PPA and PPV in CPB ........................................ 100
4.3.6. Near-Field Blasting Events versus Far-Field Seismic Events ....................... 105
4.4. Conclusions ............................................................................................................. 107
4.4.1. Far-field NRCan Events ................................................................. 108
4.4.2. Far-field Mining Events ................................................................. 108
4.4.3. Near-field Production Blasts ................................................................. 109
4.4.4. Applicability of Geotechnical Earthquake Engineering Criteria .................... 109
CHAPTER 5 ............................................................................................................................ 111
5. Laboratory Study ........................................................................................................ 111
5.1. Material Tested ...................................................................................................... 111
5.1.1. Mine Tailings .............................................................................................. 111
5.1.2. CPB Set Time .............................................................................................. 114
5.2. Experimental Design and Procedure ..................................................................... 116
5.2.1. In Situ Properties and Stress Conditions ...................................................... 116
5.2.2. Monotonic Loading ...................................................................................... 117
5.2.3. Cyclic Loading ............................................................................................. 118
5.2.4. Applicability of Triaxial Tests for Rockburst and Blasting ......................... 119
5.3. Sample Preparation and Setup ........................................................................... 120
5.3.1. Mixing Method ............................................................................................ 120
5.3.2. Equipment ................................................................................................... 121
5.3.3. Sample Preparation ..................................................................................... 122
5.3.4. Sample Preparation Limitations ................................................................. 125
5.4. Triaxial Machine .................................................................................................. 126
CHAPTER 6 ............................................................................................................................ 129
6. Compression Characteristics of CPB ........................................................................ 129
6.1. Consolidation Tests .........................................................................................................129
6.2. Isotropic Consolidation Curves .......................................................................................131
6.3. Compressibility of Sand versus CPB ...............................................................................135
6.4. Specimen Anisotropy .......................................................................................................136
6.5. Summary and Conclusions ..............................................................................................137

CHAPTER 7 ................................................................................................................................138
7. Monotonic Undrained Test Results ........................................................................................138
7.1. Effect of Axial Strain Rate on Monotonic Behaviour of Uncemented Tailings..............139
   7.1.1. Monotonic Test Results at Different Strain Rates .........................................................140
   7.1.2. Discussion of Effect of Strain Rate ...............................................................................144
7.2. Monotonic Test Results for Uncemented Mine Tailings..................................................145
7.3. Discussion of Monotonic Behaviour of Uncemented Mine Tailings ..............................147
7.4. Monotonic Test Results for CPB .....................................................................................149
   7.4.1. Compression Test Results ..........................................................................................149
   7.4.2. Extension Test Results ..............................................................................................152
7.5. Discussion of Monotonic Behaviour of CPB .................................................................153
   7.5.1. CPB in Compression ..................................................................................................153
   7.5.2. CPB in Extension .....................................................................................................156
7.6. Summary and Conclusions ..............................................................................................158
   7.6.1. Uncemented Mine Tailings .......................................................................................158
   7.6.2. CPB ........................................................................................................................158
   7.6.3. Monotonic Liquefaction Susceptibility .................................................................159

CHAPTER 8 ................................................................................................................................160
8. Cyclic Test Results .............................................................................................................160
8.1. Uncemented Mine Tailings ..............................................................................................160
   8.1.1. Cyclic Test Results of Tailings..................................................................................160
8.1.2. Cyclic Shear Resistance of Tailings ................................................................. 165
8.1.3. Discussion of Cyclic Response of Uncemented Mine Tailings ..................... 166
8.2. Cemented Paste Backfill ....................................................................................... 167
  8.2.1. Cyclic Test Results of CPB ............................................................................. 167
  8.2.2. Cyclic Resistance of CPB .............................................................................. 173
  8.2.3. Discussion of Cyclic Response of CPB......................................................... 175
8.3. Applicability of Liquefaction Criteria for CPB ..................................................... 176
8.4. Summary and Conclusions ............................................................................... 178
CHAPTER 9 ................................................................................................................. 179
9. Conclusions and Recommendations ..................................................................... 179
  9.1. Conclusions ....................................................................................................... 179
    9.1.1. Consolidation Characteristics ...................................................................... 179
    9.1.2. Monotonic Response ................................................................................. 180
    9.1.3. Cyclic Response ......................................................................................... 182
    9.1.4. Seismic Events .......................................................................................... 183
  9.2. Recommendations and Future Works ............................................................... 185
  9.3. Main Contributions of the Thesis ..................................................................... 186
References ...................................................................................................................... 188
List of Tables

Table 2-1: Index of plasticity for tailings slimes (after Ishihara et al., 1980). ......................... 8

Table 2-2: Properties of major constituents of Portland cement clinker. ......................................... 9

Table 2-3: Chemical compositions of fly ash and ASTM C618 limits. ........................................ 10

Table 3-1: Characteristics of rockburst (Johnston, 1992)............................................................. 37

Table 3-2: Record of seismic events at a mine during an eight-year period (Kaiser et al., 1995). 38

Table 3-3: Blast-induced liquefaction cases (Bretz, 1989)............................................................ 45

Table 3-4: Empirical equations to predict peak and residual pore pressure, and peak particle
velocity (Al-Qasimi et al., 2005). .................................................................................................. 47

Table 3-5: Threshold peak particle velocity resulting in liquefaction............................................ 48

Table 3-6: Effects of PI on liquefaction resistance of silts............................................................... 50

Table 4-1: Available seismic events recorded by different stations............................................... 64

Table 4-2: Type and coordinates of the CNSN stations. ................................................................. 66

Table 4-3: Strong earthquakes in north eastern Ontario (from NRCan, 2009). ............................. 68

Table 4-4: Strong mining events in north eastern Ontario. ............................................................. 69

Table 4-5: PPA and the equivalent number of cycles for the strongest component of the NRC
events. ............................................................................................................................................... 76

Table 4-6: PPV and the corresponding frequency for all components of the NRC events. ............. 79

Table 4-7: Mining seismic events recorded at mines. .................................................................... 83

Table 4-8: Summary for PPV and frequency content of rockbursts recorded at the mines. .......... 86
Table 4-9: Summary for PPV and frequency content of the production blasts recorded at the Kidd Mine. ......................................................................................................................................................................................... 91

Table 4-10: Summary of dynamic parameters induced by different seismic events. .................... 107

Table 5-1: Chemical composition of Williams tailings and Portland cement. ............................ 113

Table 6-1: Properties of CPB specimens after dead-weight consolidation and isotropic consolidation stages of triaxial testing. ......................................................................................................................................................................................... 130

Table 7-1: CU monotonic triaxial testing program and key test parameters. ............................ 139

Table 7-2: Angle of the constant stress ratio line for each CPB specimen. ................................. 152

Table 8-1: CU cyclic triaxial testing program and key test parameters. ..................................... 161

Table 8-2: Properties of uncemented mine tailings and CPB for assessing the liquefaction criteria. ......................................................................................................................................................................................................................................................... 177
List of Figures

Figure 2-1: Basic configurations for CPB distribution systems and the schematic of a backfilled stope (after Belem and Benzaazoua, 2008). ................................................................. 6

Figure 2-2: Particle size distribution for tailings materials (after Garga and McKay, 1984). .... 7

Figure 2-3: Chemical composition of Portland cement in comparison with supplementary cementing materials ......................................................................................................................... 9

Figure 2-4: Variation of UCS with binder content at different curing times (after Belem and Benzaazoua, 2008). ....................................................................................................................... 13

Figure 2-5: Effect of tailings fineness on strength of CPB (after Fall et al., 2005) .................. 13

Figure 2-6: Effect of mixing water on the strength of CPB (after Belem and Benzaazoua, 2008). ....................................................................................................................................................... 14

Figure 2-7: Average UCS of specimens after 1, 2, 7, 28, and 56 days of curing at the paste plant and underground site (after Grabinsky et al., 2008) ................................................................. 15

Figure 2-8: Backscattered-electron image of the interstitial space in the 50%FA:50%PC CPB specimen (from Ramlochan et al., 2004) .............................................................................................................. 16

Figure 2-9: Backscattered-electron Image of the interstitial space in the 90%slag:10%PC CPB specimen (from Ramlochan et al., 2004) .............................................................................................................. 17

Figure 2-10: Typical BSE image of a Portland cement mortar (200 days old, w/c = 0.4), with its microstructural constituents (from Scrivener, 2004) .............................................................................................................. 17

Figure 2-11: Evolution of the shear wave velocity in various CPB specimens (after Klein and Simon, 2006). ................................................................................................................................. 18

Figure 2-12: The effect of binder agent on the strength of CPB with 7wt. % (a) binder B1; and (b) binder B2 (after Kesimal et al., 2005) ....................................................................................................... 20

Figure 3-1: The CVR line as a boundary between loose and dense states. ......................... 24
Figure 3-2: Stress-strain behaviour and liquefaction susceptibility soils at different initial states under monotonic loading (after Kramer, 1996). ................................................................. 25

Figure 3-3: State criteria for flow liquefaction susceptibility (after Kramer, 1996). ............... 25

Figure 3-4: The monotonic response of five isotropically consolidated specimens (after Kramer, 1996). ............................................................................................................. 26

Figure 3-5: Truncated flow liquefaction surface in stress path space (after Kramer, 1996). ........ 27

Figure 3-6: Zone of susceptibility to flow liquefaction (after Kramer, 1996). ......................... 28

Figure 3-7: Zone of susceptibility to cyclic mobility (after Kramer, 1996). ............................ 28

Figure 3-8: Classification of dynamic problems (Ishihara, 1996). .......................................... 29

Figure 3-9: Relationship between various magnitude scales (after Youd et al., 2001). .......... 31

Figure 3-10: Idealized shape of smoothed Fourier amplitude spectrum showing the corner frequency and cut-off frequency ......................................................................................... 33

Figure 3-11: (a) Stress and strain conditions imposed on element of soil below level ground by vertically propagating S waves at four different times; (b) orientations of principal stress axis; (c) stress path (from Kramer, 1996)........................................................................................................... 35

Figure 3-12: Equilibrium of forces near the surface for a column of soil. .............................. 35

Figure 3-13: Anticipated levels of maximum dynamic stress induced by rockburst (Kaiser et al. 1995). ................................................................................................................................. 43

Figure 3-14: Anticipated levels of ground motion induced by rockburst (Kaiser et al. 1995). .... 43

Figure 3-15: Permeability of early age 5% CPB (after le Roux, 2004) ..................................... 51

Figure 3-16: Typical behaviour of loose clean sands. ............................................................. 53

Figure 3-17: Response of loose sands with low silt content (after Yamamuro and Lade, 1998). 53
Figure 3-18: Response of loose sands with high silt content (after Yamamuro and Covert, 2001). .......................................................... 54

Figure 3-19: The original data which led to the development of the Chinese Criteria (after Bray and Sancio). .......................................................... 55

Figure 3-20: The Chinese criteria adapted to ASTM (after Perlea et al. 1999). .......................................................... 56

Figure 3-21: The fine-grained soil liquefaction susceptibility criteria proposed by Bray and Sancio (2006)........................................................................................................................................... 57

Figure 3-22: Applicability of the liquefaction susceptibility criteria proposed by Bray et al. for three mine tailings (Wijewickreme et al., 2005). ................................................................................................. 57

Figure 3-23: Representative values for each soil that exhibited clay-like, sand-like, or intermediate behaviour (after Boulanger and Idriss, 2006)........................................................................................................................................... 59

Figure 4-1: Schematic of a CPB system exposed to different dynamic loads and the available recording stations. (Note: distances shown for earthquake and rockburst represent distances from epicentre, and not the depth). ........................................................................................................ 63

Figure 4-2: Response curve of BHN for the KAPO station (from NRC, 2009). ......................... 66

Figure 4-3: North eastern Ontario seismic zone (from NRCan, 2009). ........................................ 67

Figure 4-4: 7 magnitude > 3.5 earthquakes in North eastern Ontario between 1985 and 2009.... 68

Figure 4-5: Three components of the trace of the 2006/12/07 earthquake with a magnitude of 4.2 Mn recorded at KAPO station. (a) Component N; (b) component E; (c) component Z. ........... 70

Figure 4-6: Frequency spectra of the acceleration for the 2006/12/07 earthquake with a magnitude of 4.2 Mn recorded at KAPO station. (a) Component N; (b) component E; (c) component Z. ... 71

Figure 4-7: Five magnitude > 3.5 mining events in north eastern Ontario between 1985 and 2009. ........................................................................................................................................... 72

xvi
Figure 4-8: Three components of the trace of the 2006/11/29 mining event with a magnitude of 4.1 m$_N$ recorded at SUNO station. (a) Component N; (b) component E; (c) component Z. 74

Figure 4-9: Frequency spectra of the acceleration for the 2006/11/29 mining event with a magnitude of 4.1 m$_N$ recorded at SUNO station. (a) Component N; (b) component E; (c) component Z. 75

Figure 4-10: PPA versus hypocentral distance for the all components of the NRCan events (curve fitting limited to the 3.5 m$_N$ events). 77

Figure 4-11: Frequency versus hypocentral distance for the all components of the NRCan events. 80

Figure 4-12: Magnitude versus the equivalent numbers of uniform stress cycles at 0.65$_{\tau_{\text{max}}}$ for strongest components of the NRCan events along with. 82

Figure 4-13: Three components of velocity time series of the rockburst event in January 2009 at the Williams Mine. 84

Figure 4-14: FFT of component SV for velocity time series of the rockburst event in January at the Williams Mine. 84

Figure 4-15: Three components of velocity time series of the rockburst event in January 6, 2009 at the Kidd Mine. 85

Figure 4-16: FFT of component SV for velocity time series of the rockburst event in January at the Kidd Mine. 85

Figure 4-17: Three components of velocity time series of the rockburst event in June 15, 2009 at the Kidd Mine. 87

Figure 4-18: FFT of component SV for velocity time series of the rockburst event in June at the Kidd Mine. 87

Figure 4-19: Levels of ground motion induced by the rockbursts at the Williams and Kidd Creek Mines. 88
Figure 4-20: Three components of velocity time series of the production blast in stope 70-867 at the Kidd Mine. ................................................................................................................................ 89

Figure 4-21: FFT of component E for velocity time series of the production blast in stope 70-867 at the Kidd Mine. ................................................................................................................................ 89

Figure 4-22: Three components of the velocity time series of the production blast in stope 71-862 at the Kidd Mine. ................................................................................................................................ 90

Figure 4-23: FFT of component N for velocity time series of the production blast in stope 71-862 at the Kidd Mine. ................................................................................................................................ 90

Figure 4-24: PPV versus distance for the seismic events recorded at the mines in comparison with the NRCan events. ......................................................................................................................... 92

Figure 4-25: Frequency versus distance for the seismic events recorded at the mines in comparison with the NRCan events. ......................................................................................................................... 93

Figure 4-26: Plan view of 9555 and 9415 levels in vicinity of stope # 55 (from Thompson et al., 2008). .............................................................................................................................................. 95

Figure 4-27: Schematic of layout of instrumentation within cage # 10 in stope # 55. .......... 96

Figure 4-28: The location of accelerometers in CPB and the stages of pouring for stope # 55 (after Thompson et al., 2008). ..................................................................................................................... 97

Figure 4-29: Plan view of blast holes pattern in a production ring. Numbers in parenthesis indicate delay numbers (×25 msec). ..................................................................................................................... 98

Figure 4-30: Acceleration time series for accelerometer # 9. ......................................................... 99

Figure 4-31: Acceleration time series for accelerometer # 10. ......................................................... 99

Figure 4-32: PPA for P and S waves versus scaled distance in rock (Mohanty and Trivino, 2009). .................................................................................................................. 101
Figure 4-33: PPV for P and S waves versus scaled distance in rock (Mohanty and Trivino, 2009).

Figure 4-34: Variation in frequency for acceleration in Rock (Mohanty and Trivino, 2009)....

Figure 4-35: Variation in frequency for velocity in Rock (Mohanty and Trivino, 2009)....

Figure 4-36: PPA versus scaled distance in CPB (Mohanty and Trivino, 2009).

Figure 4-37: PPV versus scaled distance in CPB (Mohanty and Trivino, 2009).

Figure 4-38: Variation in frequency for acceleration in CPB (Mohanty and Trivino, 2009)....

Figure 4-39: Variation in frequency for velocity in CPB (Mohanty and Trivino, 2009)....

Figure 4-40: Variation of PPV versus distance for all near-field and far-field events.

Figure 5-1: Particle size distribution of Williams tailings.

Figure 5-2: Mineral composition of The Williams Mine tailings.

Figure 5-3: Mineral composition of Williams Portland cement.

Figure 5-4: The initial and final set of 3% CPB specimens along with the electrical conductivity measurements.

Figure 5-5: Schematic of direct simple shear apparatus (after Kramer, 1996).

Figure 5-6: Membrane adjustment and CPB placement in a mould.

Figure 5-7: Dead-weight consolidation process.

Figure 5-8: Triaxial setup.

Figure 5-9: GCTS Hydraulic Power Supply (GCTS manual).

Figure 5-10: Servo valve and axial actuator mounted on the triaxial frame. The digital system controller SCON-1500 (black box on the left), the pressure control box (black box on the right), and the air/water transfer cells of the GCTS system.
Figure 6-1: Isotropic consolidation curves for CPB ................................................................. 132

Figure 6-2: Laboratory consolidation curves for the tailings and the CPB ...................... 133

Figure 6-3: Normalized isotropic consolidation curves for the CPB and the tailings ......... 134

Figure 6-4: Compressibility of Ottawa sand versus CPB from the Williams Mine (MT, moist
tamped; WP, water pluviated) ................................................................................................. 135

Figure 6-5: Volumetric strain versus axial strain of the uncemented mine tailings (after Khalili,
2009) ........................................................................................................................................ 136

Figure 7-1: Monotonic response of the uncemented mine tailings at different axial strain rates. a) Stress path, b) pore pressure ratio, c) deviator stress versus axial strain, and d) stress obliquity versus axial strain ................................................................. 142

Figure 7-2: a) Stress path of two tailings specimens (WMTM4 and WMTM7 at 2%/min and 0.1%/min axial strain rates, respectively), b) the shape of the uncemented tailings specimen (WMTM4-2%/min axial strain rate) after experiencing 25% axial strain .................... 143

Figure 7-3: Monotonic response of uncemented mine tailings at different effective confining stresses. a) Stress path for the uncemented mine tailings, b) pore pressure ratio versus axial strain, c) stress-strain behaviour, and d) stress obliquity versus axial strain ........................................................................... 146

Figure 7-4: The monotonic compression response of the 3% CPB cured for 4 hours. a) Stress path, b) pore pressure ratio versus axial strain, c) stress obliquity versus axial strain, and d) deviator stress versus axial strain ......................................................................................... 150

Figure 7-5: The effect of void ratio on the angle of the constant stress ratio line for CPB specimens ................................................................................................................................. 152

Figure 7-6: The monotonic extension response of 3% CPB cured for 4 hours. a) Stress path, b) pore pressure ratio versus axial strain, c) stress ratio versus axial strain, and d) deviator stress versus axial strain ......................................................................................... 154
Figure 8-1: Response of uncemented tailings specimen WMT-CY6 tested at CSR = 0.15 and \( \sigma_c' = 100 \text{ kPa} \). a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response. ........................................................................................................................................... 162

Figure 8-2: Response of uncemented tailings specimen WMT-CY1 tested at CSR = 0.24 and \( \sigma_c' = 50 \text{ kPa} \). a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response. ........................................................................................................................................... 164

Figure 8-3: CSR versus number of cycles to liquefaction for the uncemented mine tailings. .... 165

Figure 8-4: Response of CPB specimen WCPB-CY8 tested at CSR = 0.24 and \( \sigma_c' = 50 \text{ kPa} \). a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response. ........................................................................................................................................... 169

Figure 8-5: Response of CPB specimen WCPB-CY12 tested at the CSR of 0.24 and \( \sigma_c' = 100 \text{ kPa} \). a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response. ........................................................................................................................................... 171

Figure 8-6: Response of CPB specimen WCPB-CY4 tested at the CSR of 0.24 and \( \sigma_c' = 30 \text{ kPa} \). a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response. ........................................................................................................................................... 172

Figure 8-7: The response of CPB specimen WCPB-CY10 cured for 12 hours and tested at the CSR of 0.29 and \( \sigma_c' = 50 \text{ kPa} \). a) Stress path, b) variation in pore pressure ratio versus number of cycles. ........................................................................................................................................... 173

Figure 8-8: Potential of CPB to liquefaction due to cyclic loading. .............................................. 174

Figure 8-9: Application of Bray et al criteria for liquefaction assessment of the uncemented mine tailings and CPB. ........................................................................................................................................... 177
List of Appendices

Appendix A

A. Conversion of NRC seismic waveform data to actual velocity time series.........................204

Appendix B

B. Frequency spectra for NRCan seismic events.................................................................215

Appendix C

C. Cyclic Stress Results...........................................................................................................233
CHAPTER 1

INTRODUCTION

1. Introduction

1.1. Problem Statement

Cemented paste backfill (CPB) is a mixture of mine tailings, water, and binder agents used to fill previously mined underground openings (stopes). The application of backfill in a stope reduces the amount of mine tailings that need to be stored on the surface, and contributes to the stability of the mine. Although the long term stability is important, the short term stability of fresh CPB, including its resistance to liquefaction, is of concern.

The state of practice in paste technology is to add a small quantity of cementitious materials (i.e., binder agents) to mine tailings as backfill material in order to improve short term and long term strengths. The ‘rule of thumb’ used to consider backfill as liquefaction resistant is to achieve an unconfined compressive strength (UCS) of 100 kPa (Le Roux, 2004). This guideline has been adopted from the special case study on clean rounded cemented sand (Clough et al., 1989). It has been shown that the 100 kPa UCS can be achieved by adding a small quantity of binder to CPB in a short period of time (Aref, 1988; Pierce, 1997, Le Roux, 2004). However, there might still be a risk of liquefaction at early ages when the cement in CPB has not hydrated significantly.

Once CPB is prepared, it is then delivered through pipelines into a previously mined stope at a controlled filling rate. The filling rate depends on the consolidation and strength development of
CPB over time due to the hydration of cementitious binders. In a long narrow stope, the filling rate may be very slow to prevent a barricade failure at the bottom of the stope. Therefore, the ability of CPB to resist static liquefaction due to self-weight during the filling of a stope is important because of the safety and economic implications associated with the potential failure of the fills.

In addition to static liquefaction, the dynamic response of CPB to liquefaction induced by an earthquake is of concern. Furthermore, the resistance of freshly placed CPB to liquefaction due to blasting during the excavation of adjacent stopes or due to rockbursts as unexpected seismic events in hard rock mining is important. Although the characteristics of dynamic loads induced by blasting and rockbursts are different from earthquakes, the method used in conventional geotechnical earthquake engineering of surface structures might plausibly be adapted to the geomechanical design of CPB systems.

1.2. Objectives

The mechanical properties of mine tailings, including static and dynamic strengths, have been studied for a wide range of particle sizes and mineralogy. However, the effect of binder agents on the mechanical properties of fresh CPB including its static and dynamic strengths (i.e., resistance to liquefaction) is not well understood. Therefore, determining the laboratory responses of fresh CPB to monotonic and cyclic loadings are the main objectives in this thesis. To better understand the effect of cement on the response of CPB, the response of uncemented mine tailings to liquefaction will also be investigated for the same mine tailings. The cyclic stress approach used in geotechnical earthquake engineering will be applied to determine the dynamic behaviour of CPB.

In addition to earthquake-induced liquefaction, blast- and rockburst-induced liquefaction are two other phenomena that may occur in the vicinity of a backfilled stope with CPB. Determining the applicability of the cyclic stress approach used in geotechnical earthquake engineering for these two phenomena will be another objective of this thesis. For this purpose, the characteristics of far field and near field dynamic loads induced by rockburst and blasting should be determined and compared with that of typical earthquakes.
The liquefaction criteria for evaluating the susceptibility of fine grained soils such as silts and clays to earthquake-induced liquefaction has recently been proposed by Bray and Sancio (2006). Since the mine tailings used in this study are categorized as non-plastic silts, evaluation of applicability of these criteria for mine tailings and fresh CPB will be another objective of this thesis. The following section describes how the above objectives will be addressed in this thesis.

1.3. Thesis Organization

Chapter 2 describes CPB, its ingredients and the basic properties of mine tailings and CPB in a broad range of materials used in industry. The main concepts of cement hydration and microstructure development in a CPB similar to the material used in this study will be presented.

Chapter 3 presents the definition of liquefaction in the context of geotechnical earthquake engineering and the state of the art for evaluating the liquefaction susceptibility of soils. The general characteristics of dynamic loads, such as earthquake, rockburst, and blasting as well as ground motion parameters, such as amplitude, frequency and duration will be reviewed.

The characteristics of the typical earthquakes in northern Ontario will be presented in chapter 4. To determine the characteristics of rockbursts, two far-field sets of data recorded by Canadian national seismic networks and individual mines will be investigated. Two far-field blasting data sets recorded at The Kidd Creek Mine and a near field blasting data set recorded at Williams mine will be investigated to determine the characteristics of blasting loads. A comparison between the characteristics of rockbursts and blasting events and those of earthquakes will then be presented in this chapter.

Chapter 5 presents the basic properties of the mine tailing and cement used in this study. The experimental design, the sample preparation technique and equipment used are also discussed in this chapter.

Chapter 6 addresses the triaxial consolidation characteristics of CPB used in this study.

Chapter 7 presents the results of monotonic tests in an undrained condition on mine tailings and CPB at different effective confining stresses. The effect of strain rate on the monotonic response of mine tailings will also be investigated in this chapter.
Chapter 8 presents the results of cyclic tests for mine tailings and CPB at different cyclic stress ratios and curing times. The applicability of the Bray and Sancio (2006) liquefaction susceptibility criteria for fine grained materials is also considered in this chapter. The conclusions and recommended future work will be presented in Chapter 9.
CHAPTER 2

CEMENTED PASTE BACKFILL

2. Cemented Paste Backfill

Cemented paste backfill (CPB) is a mixture of mine tailings, water, and binder agents used to fill previously mined underground openings (stopes). Typically, CPB is produced in a paste plant by dewatering of fine tailings to a filter cake and then adding sufficient amount of water and binder agents to make the mixture pumpable. CPB is mixed using the “batch” or “continual” technique and then delivered into a previously mined stope in pipelines, predominantly by gravity transport (Hassani and Archibald, 1998). CPB discharges from the pipe at the end of the transportation and is poured into the stope from the top while the bottom side of the stope is closed by building a barricade. Figure 2-1 shows the basic configurations for CPB distribution systems and the schematic of a backfilled stope.

For pipeline design purposes, CPB can be considered as a non-Newtonian fluid material with viscoplastic behaviour (Saebimoghaddam, 2005). The rheological properties of CPB, such as apparent yield stress and viscosity, and the effect of index parameters on rheological characteristics of CPB have been studied previously and are beyond of the scope of this thesis (Moghaddam and Hassani, 2007; Simon, 2005; Crowder, 2004; Kwak, 2004).

For geotechnical design purposes, which are the main concern of this thesis, CPB can be considered as a cemented soil. In this regard, the characteristics of mine tailings that make the solid portion of CPB on one hand and the characteristics of binder agents on the other hand control the mechanical properties of CPB including its strength. Since the mechanical properties of CPB will change with time as the binders hydrate, two phases of strength gain can be
considered. First, a transient phase where CPB has low strength and might be susceptible to liquefaction due to static and dynamic loads. Second, a hardening phase where CPB has gained an adequate strength for underground support. The background information on the liquefaction potential of CPB during the transient phase will be discussed in Chapter 3, while some basic properties of the main constituents of CPB and its strength development will be briefly reviewed in the following sections.

![Diagram of CPB distribution systems and backfilled stope](image)

Figure 2-1: Basic configurations for CPB distribution systems and the schematic of a backfilled stope (after Belem and Benzaazoua, 2008).

### 2.1. Mine Tailings

Mine tailings are residual materials obtained as a by-product of mineral processing. The basic properties of mine tailings, such as particle size distribution, mineralogy, and chemical composition, might vary not only from mine to mine but also within a mine itself. It has been suggested that mine tailings in CPB should contain a minimum weight of 15% of the particles that are smaller than 20 microns. The role of fine particles (<20 microns) is to prevent the water/solids separation (Landriault, 1995, Cincilla et al., 1997; Tenbergen, 2000).

Particle size distribution influences several properties of mine tailings, such as bulk density, consistency, and effective surface area of particles. The bulk density of mine tailings mixtures changes with an increase in fines of the particles. The effective surface area of the particles
affects the rheological properties of mine tailings. The finer the particle size distribution, the more water exists surrounding the particle surface area. The more the water, the more is the “workability” or consistency (i.e., slump) of mine tailings. Furthermore, the finer the particles, the greater the effective surface that the cementitious materials must act on. The grain shapes might be another factor on cementation. It has been noted that the effects of cementitious materials are typically weaker in sands with rounded grains than those with angular grains (Clough et al., 1989).

Mine tailings that are typically used to make CPB in hard rock mining might be categorized as fine-grained soil with zero to low plasticity index. Mine tailings may generally be produced at various particle sizes with different plasticity indexes (PI). For example, Figure 2-2 shows the particle size distribution of tailings from different locations in the world. The tailings sands shown in this figure are non-plastic while silt-clay tailings might have low to high plasticity indexes, as shown in Table 2-1. Vick (1990) also presented the particle size distribution and PI of different mine tailings. Particle size distribution and PI affect the mechanical properties of CPB as will be discussed later in this chapter.
The mineralogy of mine tailings depends on the type of host rock and it mainly consists of quartz, feldspar, and some minor minerals for hard rock mines. The specific gravity of minerals controls the bulk density of mine tailings mixtures at a given volumetric water content (Brackebusch, 2002). Mineralogy is an important factor as it may influence the strength of CPB. For instance, the presence of sulphide minerals, such as pyrite, has been shown to have a negative effect on the long term strength of CPB (e.g., Benzaazoua et al. 2002).

Table 2-1: Index of plasticity for tailings slimes (after Ishihara et al., 1980).

<table>
<thead>
<tr>
<th>Name</th>
<th>Country</th>
<th>Mine Tailings</th>
<th>PI (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Cobre, old dyke</td>
<td>Chile</td>
<td>Copper tailings (slime)</td>
<td>~4.8</td>
</tr>
<tr>
<td>El Cobre, No4</td>
<td>Chile</td>
<td>Copper tailings (slime)</td>
<td>~11</td>
</tr>
<tr>
<td>Mochikoshi</td>
<td>Japan</td>
<td>Gold tailings (slime)</td>
<td>~10</td>
</tr>
<tr>
<td>Kosaka</td>
<td>Japan</td>
<td>Lead zinc tailings</td>
<td>~16</td>
</tr>
<tr>
<td>Furutobe</td>
<td>Japan</td>
<td>Tailings slimes</td>
<td>~28</td>
</tr>
</tbody>
</table>

2.2. Binder Agents

Binder agents, including Portland cement (PC) and supplementary cementing materials (SCM), such as blast furnace slag and fly ash, are added to mine tailings-water mixtures to improve the mechanical properties of CPB. Portland cement at various contents is the main constituent of the binder agents in CPB. The percentage of PC used in Canadian Mines is typically between 3 and 6.5 percent of the solid mass (Ouellet et al., 1998). A combination of PC-slag or PC-fly ash might also be used in CPB. In addition, chemical additives, such as flocculants, super plasticizers, and accelerators may be employed to improve the permeability or consolidation of fills or to increase the flowability of CPB. The main characteristics of Portland cement and SCM’s and their effects on CPB are briefly discussed in the following subsections.

2.2.1. Portland Cement

Portland cement clinker mainly contains four mineral components: tricalcium silicate or alite (3CaO·SiO₂ or C₃S), dicalcium silicate or belite (2CaO·SiO₂ or C₂S), tricalcium aluminate (3CaO·Al₂O₃ or C₃A), and tetracalcium alumina ferrite (4CaO·Al₂O₃·Fe₂O₃ or C₄AF). The properties of major components and minor components of Portland cement clinker are shown in Table 2-2 (Neville, 1995). Figure 2-3 shows the range of chemical compositions of Portland cement in a ternary diagram including CaO, SiO₂, and Al₂O₃.
Table 2-2: Properties of major constituents of Portland cement clinker.

<table>
<thead>
<tr>
<th>Major Components</th>
<th>C₃S</th>
<th>C₂S</th>
<th>C₃A</th>
<th>C₄AF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage of PC</td>
<td>50%</td>
<td>20%</td>
<td>3-8%</td>
<td>-</td>
</tr>
<tr>
<td>Grain Shape</td>
<td>Equidimensional</td>
<td>Rounded</td>
<td>Rectangular</td>
<td>-</td>
</tr>
<tr>
<td>Rate of Hydration</td>
<td>Rapid (hours)</td>
<td>Slow (days)</td>
<td>Instantaneous</td>
<td>Very Rapid (minute)</td>
</tr>
<tr>
<td>Heat of Hydration</td>
<td>M: ~500 J/g</td>
<td>L: ~259 J/g</td>
<td>VH: ~850 J/g</td>
<td>M: ~420 J/g</td>
</tr>
</tbody>
</table>

| Minor Components | MgO | TiO₂ | Mn₂O₃ | K₂O | Na₂O |

The reaction of Portland cement with water forms hydrates that produce a stiff mass after sufficient time. The typical hydration products are: calcium hydroxide (Ca(OH)₂ or CH), calcium-silicate-hydrate (C-S-H), and calcium sulfoaluminate known as ettringite (3CaO·Al₂O₃·3CaSO₄·31H₂O). To control the violent reaction between C₃A and water, gypsum is added to Portland cement during grinding.

Figure 2-3: Chemical composition of Portland cement in comparison with supplementary cementing materials.

The hydration process of Portland cement typically occurs in four stages, as follows:

i) Initial reactions (0-15 min.): initial dissolution of C₃A, C₃S and gypsum is the first chemical reaction to occur in the cement paste resulting in the release of Na, K, Ca, OH ions into the pore solution.

ii) Induction (dormant) period (1-3 hrs.): an amorphous, semi-permeable gel, rich in calcium, silica and alumina will be forming around the cement grains, along with ettringite and CH
during this period (Soroka, 1980). Further hydration might be retarded due to the formation of this gel and ettringite around C\textsubscript{3}S and C\textsubscript{3}A grains, respectively.

iii) Setting period (3-17 hrs.): the end of dormant period is associated with the rupture of gel formed around the cement grains due to excess pressure. This results in the formation of C-S-H gel due to the contact of the silica rich solution from the gel with the external solution. In this stage, large crystals of CH might form out of the supersaturated solution (Soroka, 1980).

iv) Hardening period (17 hrs. to 90 days): short fibres of C-S-H and massive deposition of CH continue to form at a very slow rate resulting in further decrease in paste porosity (Neville, 1995).

The strength of hardened cement pastes might be affected by many factors, but the ratio by weight of water-to-cementitious material (w/cm) is the most important factor. The lower the w/cm ratio, the stronger and less permeable the cement paste will be. The typical w/cm ratios of Portland cement concrete and mortar mixtures are less than 0.5. For comparison, w/cm ratios of about 2.5-7 are typical values for CPB.

2.2.2. Supplementary Cementing Materials

Supplementary cementing materials, typically by-product of other industries, are usually added to Portland cement to improve its characteristics. Characteristics of fly ash and blast-furnace slag that are two common SCM’s used in CPB will be discussed in the following subsections.

2.2.2.1. Fly Ash

Fly ash is the residue collected electrostatically from the exhaust gases of coal-fired power stations (Neville, 1995). Fly ash contains alumino-silicate glass which must be suitably activated to react with water; Portland cement is a suitable material for this activation. Based on chemical composition, two types of fly ash are available, class F and class C. Table 2-3 and Figure 2-3 show the typical chemical compositions of these two types (ASTM C618).

<table>
<thead>
<tr>
<th>Type</th>
<th>SiO\textsubscript{2} + Al\textsubscript{2}O\textsubscript{3} + Fe\textsubscript{2}O\textsubscript{3}, %</th>
<th>Lime, %</th>
<th>Alkalis, %</th>
<th>Carbon, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class F</td>
<td>80-90 (70 min.)</td>
<td>1-9</td>
<td>0.3-1.0</td>
<td>0.8-4.2 (6 max.)</td>
</tr>
<tr>
<td>Class C</td>
<td>48-74 (50 min.)</td>
<td>11-29</td>
<td>2.3-4.1</td>
<td>0.1-0.5 (6 max.)</td>
</tr>
</tbody>
</table>
Class F fly ash consists of a relatively high glass content. The glass phase consists primarily of ordered silicate and alumino-silicate phases. However, class C fly ash has a higher CaO content and relatively low glass content. The glass phase for class C consists primarily of substituted calcium aluminate and calcium aluminate silicate phases. The glass materials are only reactive when the pH of pore water is more than 12 (pH ≥ 12). Fly ash particles are spherical, ranging in size from 1 μm to 100 μm. The typical value of specific gravity of fly ash is 2.2-2.5 (Neville, 1995).

### 2.2.2.2. Blast Furnace Slag

Blast furnace slag is a by-product obtained during molten iron production. If it is cooled slowly from the molten state, it will almost completely crystallize into well-ordered minerals. However, if it is rapidly cooled, known as granulated slag, 90 to 100% glass will be formed. Granulated slag is latently hydraulic. In terms of chemical composition, slag is a mixture of lime, silica and alumina, that is, the same oxides that make up Portland cement but not in the same proportions (see Figure 2-3).

The hydration products of granulated slag are similar to those of Portland cement including C-S-H and various calcium aluminate and calcium alumino-silicate compounds. Although slag hydrates when mixed on its own with water, the process is very slow. Latent hydraulic materials, such as slag need the addition of an activator to promote significant hydration, as slag is more soluble in a basic medium (pH ≥ 12). Alkali and lime compounds are effective activators for these types of materials. The alkali compounds present in Portland cement and calcium hydroxide (CH) released during the hydration of cement are very effective activators of slag. The activators, first, ensure slag dissolution, second, lead to higher solution concentrations, and then precipitate hydration components. One of the most important factors that affects the ability of cement to sustain the activation of slag is the concentration of the alkalis in the total cementitious material. Generally, a better development of strength has been reported with finer cements and with cements that have high contents of C₃A and of the alkalis (Neville, 1995).

### 2.3. Strength of Cemented Paste Backfill

The required strength for CPB depends on the intended purpose. Belem and Benzaazoua (2008) reviewed the design unconfined compressive strength (UCS) required for different backfill
applications, such as vertical backfill support, development opening through backfill mass, pillar recovery, and ground support. Previous studies indicate that although the UCS of surrounding rock mass of a backfill in hard rock mining might vary from 5 MPa to 240 MPa, the UCS of backfill itself is commonly lower than 5 MPa ranging from 0.2 MPa to 4 MPa (e.g., Grice 1998; Revell 2000). In another instance, the UCS for backfill applications while experiencing no extra external loads can be lower than 1 MPa, as presented by Stone (1993) and Li et al. (2002). Grabinsky et al. (2008) also showed that the UCS of the CPB specimens (3wt%, 50:50-PC:FA) sampled from different locations of a narrow long Alimak stope at Williams mine reached a maximum value of 300 kPa after 9 months curing. Note that the strength gained at the time was adequate for developing an opening through the CPB mass in this case. More details on the properties of CPB used at Williams mine will be reviewed in Chapter 5 since the same material was obtained for the laboratory work in this thesis.

In the following subsection the main factors that affect the UCS of CPB, such as binding agents, tailings characteristics (specific gravity, mineralogy, particle size distribution), and finally, mixing water chemistry, will be reviewed.

2.3.1. Parameters Affecting the UCS of CPB

Several UCS test results show that the strength of CPB is proportional to binder content at a given curing time. For example, Figure 2-4 shows an increase in UCS as the binder content and curing time increase. However, the relationship between binder content and the strength is specific to each mine. In addition to binders, it has been shown that the precipitation of hydrated phases from the paste pore water affects the hardening process of CPB (Benzaazoua et al. 2004).

It has been shown that the proportion of fine particles (i.e., <20 μm) in tailings has a strong influence on the strength gain of CPB. For example, Fall et al. (2005) showed that the UCS of CPB decreases as the percentage of fine particles increases for different CPB specimens, as shown in Figure 2-5. In this figure, PCI and PCV are ASTM Portland cement type I and V (CSA types GU and HS), respectively. The PCI to PCV ratio was 50/50, while the PCI to slag ratio was 20/80 for the samples.

The tailings for this study were reprocessed to create several grain size classes corresponding to fine (20 μm particles >60 wt%), medium (35–60 wt% of 20 μm particles) and coarse (15–35 wt%
of 20 μm particles) tailings. Fall et al. (2005) concluded that the coarse and medium tailings are more favourable for CPB strength gain. The density of tailings also affects the strength of CPB. Fall et al. (2004) showed that the increase of tailings density generally results in an increase in the strength of CPB for the same binder proportion.

Figure 2-4: Variation of UCS with binder content at different curing times (after Belem and Benzaazoua, 2008).

Figure 2-5: Effect of tailings fineness on strength of CPB (after Fall et al., 2005).
The quality of water used in CPB is another concern to achieve an adequate strength. The pH and sulphate salts content of the water are two important factors. It has been shown that acidic water and sulphate salts result in strength loss in CPB (Benzaazoua et al. 2002, 2004). Figure 2-6 shows the effect of different mixing water chemistries on the UCS of CPB specimens for a given binder type and content. The hardening process of CPB in this case is slow for all three waters and the UCS’s are all the same at 14-day curing. However, the UCS of the specimens with sulphate-free waters (tap and lake water) is higher than that of the specimen with sulphate-rich mine-A process water at 28-day curing (Benzaazoua et al., 2002).

![Figure 2-6: Effect of mixing water on the strength of CPB (after Belem and Benzaazoua, 2008).](image)

Although it is possible to obtain the strength of CPB by making specimens in the laboratory, it must be noted that there can be a difference between field specimens and laboratory specimens. Grabinsky et al. (2008) recently showed that the average UCS values of the specimens obtained from the paste plant and underground during filling a stope at a mine were different, as shown in Figure 2-7. In addition, the average UCS of the 9 month-cured CPB samples obtained from behind the fill fence of the stope was about 310 kPa, which is less than the values for the paste plant and underground specimens cured for 2 months. These differences can be attributed to the heterogeneity of the in-situ samples, such as air voids and planes of weakness, whereas laboratory samples are more homogenous because of the mixing and rodding during casting.
2.3.2. Shear Strength Development

The study of microstructure evolution in CPB can provide insights into the strength development of CPB. Several studies have investigated the relationship between the microstructure and the strength of paste backfill using the scanning electronic microscope (SEM) technique (Ouellet et al., 1998; Benzaazoua, et al., 1999; Benzaazoua, et al., 2002; Mohamed et al., 2003). In these studies, fractured or cut surfaces of CPB specimens were investigated. However, those surfaces may not be necessarily representative of the microstructures since fractured surfaces are obtained from naturally weak planes in the material and cut surfaces are disturbed by the cutting process. Instead, Ramlochan et al. (2004) suggested the backscattered-electron imaging (BEI) of polished sections as a better method to reveal the microstructure of CPB. They investigated the microstructure of four mature CPB mixtures using three different mine tailings and two types of binder recipes. The first binder recipe consisted of 50% PC and 50% fly ash and the second binder recipe consisted of 10% PC and 90% blast furnace slag. Figure 2-8 shows the microstructure of the first recipe CPB specimen with the total binder content of 15% by mass. Some hydration products can be seen at the surface of tailings particles and sparsely infilling the interstitial space. The void space is filled with epoxy and appears near black in BEI.

Figure 2-7: Average UCS of specimens after 1, 2, 7, 28, and 56 days of curing at the paste plant and underground site (after Grabinsky et al., 2008).
The microstructure of the second recipe CPB specimen with the total binder content of 3.2% by weight is shown in Figure 2-9. The relative amount of hydration products in the interstices of this specimen appears to be greater than in the first specimen.

The unconfined compressive strength (UCS) measurements of the specimens confirmed that the second specimen had a greater compressive strength than mixtures of similar water-to-cementitious material ratios, which contained the PC and fly ash binder. Ramlochan et al. (2004) concluded that the PC-blast furnace slag binder would likely be more appropriate for CPB strength gain than the PC-fly ash binder.

It should be noted that in both cases, the hydration products of cementitious materials did not effectively fill the interstices separating the tailings particles, which resulted in relatively low strength mixtures. Ramlochan et al. (2004) also suggested that the increase of pH in the fluid phase with additions of lime or alkali hydroxides to the mixing water would facilitate the reaction of SCM’s.

To better understand the incomplete filled interstices between tailings particles in CPB, it is good to compare the microstructure of CPB with a Portland cement mortar. For example, Figure 2-10 shows the typical back scattered electron image of a Portland mortar with its microstructural constituents. Note that the mortar is cured for 200 days and its w/c ratio is 0.4. Although there are
still pores, the interstices between sand aggregates are completely filled with the hydration products, which results in a dense and strong matrix.

![Figure 2-9: Backscattered-electron Image of the interstitial space in the 90% slag:10% PC CPB specimen (from Ramlochan et al., 2004).](image)

In addition to microstructure analysis, Klein and Simon (2006) suggested shear wave velocity measurements as a useful tool to study the strength development in CPB. Since shear waves
propagate through the solid skeleton of mixtures, the skeletal development and hardening process in CPB due to the formation of hydration products can be monitored using this technique. Figure 2-11 shows the evolution of shear wave velocity in various CPB specimens measured using a bender element. It is also possible to see the effect of mixture composition on the strength development in these CPB specimens.

Figure 2-11: Evolution of the shear wave velocity in various CPB specimens (after Klein and Simon, 2006).

\[\Delta, 100\%\ MT\ (w = 24.0\%); \bigcirc, 100\%\ MT\ (w = 26.4\%); \Diamond, 100\%\ PC\ (w = 28.1\%); +, 95:5\ MT:PC\ (w = 27.5\%); \blacksquare, 95:5\ MT:PC\ with\ 0.185\%\ admixture\ PCA*\ (w = 23.5\%); \bullet, 95:5\ MT:PC\ with\ 0.5\ mol/L\ HCl\ (w = 27.3\%).\]

*PCA = Polycarboxylated acrylic acid polymer.

2.3.3. Long Term Strength

The mineralogy of mine tailings and cementitious binders controls the strength development of CPB. As compared to conventional concrete, CPB is very porous with very high water-to-cementitious material ratios and this condition may enhance the chemical reactions between ions in the pore solution resulting in strength instability of CPB. The most important concern regarding the “long term” strength stability of CPB might be the existence of sulphidic mine tailings. According to Fall and Benzaazoua (2005), sulphidic mine tailings oxidize in the presence of oxygen, producing sulphates. The effect of sulphates on the strength of CPB depends on several parameters, such as the sulphate concentration, the curing time, and the cementitious material composition and content. Several studies have shown that the strength of CPB decreases
with time due to an internal sulphate attack, which results from the chemical interactions of the sulphate ions with the Portland cement hydration products (Benzaazoua et al. 2002; Kesimal et al. 2004; Fall and Benzaazoua 2005). Ayore et al. (1998) proposed that porous systems might increase sulphide oxidation and the migration of calcium, aluminium, and sulphate ions, thus altering the hydration products and affecting the overall strength of the cement- and sulphide-containing pastes.

In contrast, it has been shown that the strength of sealed CPB specimens with high sulphur content has continuously shown an increase over a “long period” (Klein and Simon, 2006; Pierce et al., 1998). However, the oxidation of the specimens that have been removed from their sealed moulds after 28 days of curing results in a reduction in the 56 day strength of the CPB (Pierce et al., 1998). As explained by Klein and Simon (2006), the curing condition is an effective parameter that may change the strength behaviour of CPB even with the presence of sulphidic minerals.

Kesimal et al., (2005) showed the effect of binders on the “long term” strength stability of CPB using two mine tailings with high sulphur contents. As shown in Figure 2-12, over the same curing time, the deterioration in the stability of the CPB specimens seemed considerably less severe with only 14% and 9% losses in the strength for tailings T1 and T2, respectively, when binder B2 was used (Figure 2-12b). The difference in the long term performances of binder B1 and B2 can be attributed to their respective chemical composition and response to the chemical conditions within the CPB matrix. Although both binders are Portland cement based composite cements, binder B2 contains more pozzolanic material (29%) than binder B2 (14%) (Kesimal et al., 2005).

2.4. Summary

The characteristics of CPB main constituents (i.e., mine tailings, binder agents) were generally reviewed in this chapter. This provides necessary background information for comparison with the materials used in this study. Although the SCM’s will not be used, an overview of characteristics of typical SCM’s was required since most of the research works reviewed in this chapter included these materials.
The strength development of CPB and the effective parameters on the strength gain in CPB were also reviewed. The microstructure evaluation shows that CPB with high water-to-cementitious material ratio is a highly porous material even during its hardening phase. The low UCS values also show the low strength properties of CPB. Some of the parameters affecting long term strength of CPB might also affect the early strength of CPB (i.e., liquefaction potential of CPB in its transient phase), such as percentage of fine particles and binder content. The shear wave velocity measurement was also shown as a proper tool to investigate the strength development of CPB in its early or hardening stage.
CHAPTER 3

LITERATURE REVIEW

3. Literature Review

Based on the objectives of the thesis, this chapter aims to review i) monotonic and dynamic behaviours of fine grained soils, ii) characteristics of dynamic loadings that may induce liquefaction in the vicinity of mining activities including rockburst and production blasts, and iii) availability of liquefaction design criterion, that the designer can rely on for geomechanical design of CPB systems.

First, some basic concepts and definitions related to liquefaction phenomena will briefly be reviewed, noting that most of these concepts come from coarse grained materials (i.e., loose clean sand). Second, the loading characteristics of seismic events such as earthquakes, rockbursts and production blasts will be discussed with respect to the ground motion parameters that describe these events. Background information on liquefaction potential of silts, silt-sized mine tailings and CPB induced by these dynamic loads will then be reviewed. The effect of cement and other material parameters on the liquefaction potential of soils will also be reviewed. Finally, more recent liquefaction susceptibility criteria for fine grained soils will be reviewed. At the end of this chapter, this body of literature is summarized in terms of what it has to offer the designer of cemented paste backfill systems, and what shortcomings exist. The remainder of the thesis will address some of these shortcomings.
3.1. Definition of Liquefaction

The term liquefaction has been used in conjunction with a variety of phenomena that involves soil deformations caused by monotonic, transient, or cyclic loading of saturated soils under undrained conditions. For example, the Committee on Soil Dynamics of the Geotechnical Engineering Division, American Society of Civil Engineering (1978) has defined liquefaction as

\[...\text{the process of transforming any substance into a liquid.}\]
\[\text{In cohesionless soils, the transformation is from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress.}\]

According to this definition, the generation of excess porewater pressure is a key feature of liquefaction phenomena. Generally, when saturated soils are subjected to rapid loading under undrained conditions the tendency for contraction causes excess porewater pressures to increase and effective stresses to decrease. In other words, the generation of excess porewater pressure due to static or dynamic loading, might be sufficient to bring the soil to the steady state condition or a condition of zero effective stress leading to deformation. Depending on stress and state conditions at which liquefaction might occur and the resultant deformation, liquefaction can be divided into two main groups: flow liquefaction and cyclic mobility. Note that both flow liquefaction and cyclic mobility are generally referred to as “liquefaction” in geotechnical engineering practice.

Flow liquefaction can occur when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state (Kramer, 1996). Both monotonic and cyclic loading may bring the soil to an unstable state at which its strength drops sufficiently to allow the static stresses to produce the flow failure.

Cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil (Kramer, 1996). Dynamic loadings, such as earthquakes may cause cyclic mobility. A special case of cyclic mobility is called level-ground liquefaction where static horizontal shear stresses do not exist. Level-ground liquefaction can produce large, chaotic movement known as \textit{ground oscillation} during earthquake loading (Kramer, 1996). Flow liquefaction and cyclic mobility of sands will be reviewed in detail in the following sections.
3.2. Liquefaction Susceptibility and Its Mechanisms

Liquefaction susceptibility can be evaluated by several criteria while some of them are different for flow liquefaction and cyclic mobility. Generally, liquefaction susceptibility criteria for soils can be categorized as historical, geological, state and compositional criteria (Kramer, 1996). In this chapter, only state criteria and compositional criteria will be reviewed. To better understand the state criteria for soils and concept of liquefaction, the well known behaviour of clean sands will be reviewed in the following section. However, the compositional criteria for fine grained soils including silts and clays will be discussed separately at the end of this chapter (Section 3.6).

3.2.1. State Criteria

Liquefaction susceptibility depends on the initial state of the soil. The initial state of the soil refers to both initial density and stress conditions at the time of loading (e.g., earthquake). These two affect the generation of pore water pressure during the loading and consequently control the liquefaction susceptibility of the soil (Kramer, 1996). To express the methods for evaluating state criteria, some basic concepts of cohesionless soil behaviour will be reviewed in the following sections.

3.2.1.1. Critical Void Ratio

It has been shown that initially loose specimens contract during shearing and initially dense specimens dilate after a quick contraction at the beginning in a drained, strain-controlled monotonic triaxial test. At large strains, both loose and dense specimens approach the same density and continue to shear with constant shearing resistance. The void ratio, e, corresponding to this constant density is called the critical void ratio (CVR). It has also been shown that CVR is uniquely related to a specific effective confining pressure, $\sigma^\prime_{3c}$. Therefore, it is possible to define a CVR line by determining CVR at different effective confining pressures. Figure 3-1 shows the use of the CVR line as a boundary between loose (contractive) and dense (dilative) states.

Therefore, by defining the initial state of the soil in terms of void ratio and effective confining pressure, it is possible to evaluate the tendency of the soil to contraction or dilatation with respect to the CVR line. Generally, saturated soil with initial void ratios above the CVR line is considered susceptible to flow liquefaction, and those with void ratios below the CVR is considered non-susceptible to liquefaction.
3.2.1.2. Steady State of Deformation

The stress-strain behaviours of soil specimens at three different states under undrained monotonic loading are shown in Figure 3-2. Loose specimens (i.e., specimen A) exhibit stain softening behaviour with peak strength at a small shear strain and then collapse to follow to large strains. This behaviour is considered as flow liquefaction. Dense specimens (i.e., specimen B) exhibit strain hardening behaviour with an initial contraction and then dilation at large strains. At intermediate densities (i.e., specimen C) a peak strength at low strain is followed by a limited period of strain softening behaviour and then end with strain hardening behaviour at intermediate strain. The transformation between strain softening (contractive) and strain hardening (dilative) occurs at a point known as phase transformation point. This type of behaviour is called limited liquefaction.

The state in which the soil flows continuously under constant shear stress and constant effective confining pressure at constant volume is defined as the steady state of deformation (Castro and Poulos, 1977; Poulos 1981). Note that the steady state of deformation is reached at large strains. Specimens A and C show two examples of the steady state of deformation at large strains. Constant $\Delta u$ (excess pore water pressure) shows constant volume change and constant $q$ is corresponding to constant shear stress while straining in an undrained condition. Therefore, there is a unique relationship between void ratio and effective confining pressure at large strains. The locus of points describing this relationship in the steady state of deformation is called the steady state line (SSL). The SSL can also be expressed in terms of the steady state strength, $S_{su}$. The
SSL can be used to identify the susceptibility of a particular soil to flow liquefaction. As shown in Figure 3-3, a soil is not susceptible to flow liquefaction if its state lies below the SSL. On the other hand, a soil whose state lies above the SSL will be susceptible to flow liquefaction only if the static stress exceeds its steady state strength.

Figure 3-2: Stress-strain behaviour and liquefaction susceptibility soils at different initial states under monotonic loading (after Kramer, 1996).

Figure 3-3: State criteria for flow liquefaction susceptibility (after Kramer, 1996).
In contrast to flow liquefaction, cyclic mobility can occur in both loose (above the SSL) and dense (below the SSL) soils. Note that the identification of susceptibility to liquefaction for a given soil does not necessarily refer to occurrence of liquefaction in an earthquake. A strong disturbance for the initiation of liquefaction is required. In addition, the initiation of liquefaction might be different for flow liquefaction and cyclic mobility. To understand the initiation of liquefaction, the state of the soil when liquefaction is triggered must be identified for each type of liquefaction. Therefore some concepts will be reviewed in the following sections.

3.2.2. Flow Liquefaction Surface

A stress path can be used to demonstrate the effective stress conditions at which the initiation of flow liquefaction is triggered. The flow liquefaction surface, FLS, is a three dimensional surface determined in the stress path to describe the effective stress conditions at the initiation of flow liquefaction (Vaid and Chern, 1985). Since the initiation of flow liquefaction can be easily seen for the soils under monotonic loading, the concept of the FLS is described for this condition in this section.

The monotonic response of five specimens isotropically consolidated to the same initial void ratio at different effective confining pressures is shown in Figure 3-4. Specimens A and B are in dense conditions (below the SSL) while the other three (C, D and E) are in loose conditions.

![Figure 3-4: The monotonic response of five isotropically consolidated specimens (after Kramer, 1996).](image-url)
All these specimens will reach the same effective stress conditions at the steady state, but the stress paths are different for each of them. The dense specimens show dilative behaviour while the loose specimens show contractive behaviour. The later specimens (i.e., C, D and E) reach maximum points on the stress paths where flow liquefaction is initiated (points are marked with an × in the figure). It has been shown that the locus of points showing the initiation of flow liquefaction is a straight line that projects though the origin of the stress path, known as flow liquefaction surface (Kramer, 1996). Note that the FLS is truncated since flow liquefaction cannot be occur if the stress path is below the steady state point, as shown in Figure 3-5. The FLS can also be used to define the liquefaction susceptibility of soils. Note that the key factor to the initiation of liquefaction is the generation of excess pore water pressure.

![Flow liquefaction surface](image)

**Figure 3-5:** Truncated flow liquefaction surface in stress path space (after Kramer, 1996).

### 3.2.2.1. Flow Liquefaction

If initial stress conditions of a soil fall within the shaded zone of Figure 3-6, flow liquefaction will occur if a strong undrained disturbance brings the effective stress path from the initial conditions to the FLS. As mentioned earlier in section 3.1, both monotonic and cyclic loading may cause flow liquefaction.

### 3.2.2.2. Cyclic Mobility

In contrast, flow liquefaction cannot occur when the static shear stress is smaller than the steady state shear strength. However, cyclic mobility can occur in that case. Therefore, if initial stress states of a soil fall within the shaded zone of Figure 3-7, the soil is susceptible to cyclic mobility. The susceptibility of soil to cyclic mobility can be investigated by cyclic triaxial tests.
stress conditions and cyclic loading conditions that might develop cyclic mobility can be divided into three groups as follows: i) No stress reversal (τ_{static} > τ_{cyclic}) and the total stresses are less than steady state strength (τ_{static} + τ_{cyclic} < S_{su}); ii) No stress reversal (τ_{static} > τ_{cyclic}) and steady state strength is surpassed temporarily (τ_{static} + τ_{cyclic} > S_{su}); iii) stress reversal occurs (τ_{static} < τ_{cyclic}) and steady state strength is not surpassed (τ_{static} + τ_{cyclic} < S_{su}). In the later case, each time the effective stress path passes through the origin the specimen is in an instantaneous state of zero effective stress. However, soil still has its shear strength although this instantaneous state of zero effective stress is referred to as initial liquefaction by Seed and Lee (1966).

Figure 3-6: Zone of susceptibility to flow liquefaction (after Kramer, 1996).

Figure 3-7: Zone of susceptibility to cyclic mobility (after Kramer, 1996).
The initiation of liquefaction for the case of cyclic mobility is not a certain point as it is for the case of flow liquefaction. The permanent deformation produced by cyclic mobility depends on the static shear stress and the duration of the ground motion. The ground motion that results in cyclic mobility is usually produced by earthquakes. However, liquefaction can be induced by other type of dynamic loads. The following section addresses the dynamic-load-induced liquefaction phenomena.

### 3.3. Dynamic Loads

Classification of dynamic problems in geotechnical engineering is shown in Figure 3-8 (Ishihara, 1996). Among these dynamic problems, soil liquefaction induced by earthquakes on one hand and soil liquefaction induced by rockbursts and blasting on the other hand may be of concern, particularly for mining industry. In these three liquefaction categories (i.e., earthquake-, rockburst-, and blast-induced liquefactions), soil liquefaction occurs due to excess pore water pressure although the characteristics of dynamic loads are different. To study soil liquefaction induced by these loads, the characteristics of stress waves, the changes in initial stress conditions due to propagation of stress waves, and the initial states of soil are important.

![Figure 3-8: Classification of dynamic problems (Ishihara, 1996).](image)
Stress waves induced by dynamic loads are characterized by three parameters including amplitude, frequency and duration. In addition, the propagation of stress wave depends on the type of dynamic loads. In the following subsections, these parameters as well as stress conditions related to the propagation of stress waves induced by earthquakes, rockburst and blasting will be reviewed.

3.3.1. Earthquake-induced Stress Waves

When an earthquake occurs, different types of seismic waves are produced: body waves and surface waves. The definition of these types of seismic waves can be found in any Geotechnical Engineering text book. However, the definition of body waves only is given in this section. Body waves can be categorized into two types: P waves and S waves. i) P waves: the motion of an individual particle that the wave travels through is parallel to the direction of travel; P waves are also known as primary, compressional, or dilatational waves, ii) S waves: the motion of an individual particle that the wave travels through is perpendicular to the direction of travel; S waves are also known as secondary, shear, or transverse waves.

Generally, seismic waves radiated away from the source of an earthquake travel through the earth’s crust and produce shaking when they reach the ground surface. The strength and duration of shaking at a particular site depend on the magnitude and location of earthquake and on the characteristics of the site.

The size of earthquakes in one hand and the ground motions produced by earthquakes on the other hand are important. The size of earthquakes might be characterized by the magnitude, intensity or energy (Kramer 1996). For engineering proposes, the earthquake magnitude is more frequently used. The magnitude of earthquakes has been described in different ways, such as Richter local magnitude (ML), surface wave magnitude (Ms), body wave magnitude (mb), and moment magnitude (Mw). Figure 3-9 shows the relationship between the various magnitude scales. There are also some other magnitudes, such as Nuttli magnitude (mN) that is the most common catalogue magnitude for eastern North America (Nuttli, 1973). Nuttli magnitude is based on the amplitude of the Lg phase (multiply reflected and refracted shear waves). Nuttli magnitude can be expressed as a function of seismic moment (mN = log M0 [GN.m]-1±0.15 if there is no stress drop). Therefore, the difference between Nuttli magnitude and Richter local magnitude is 0.5 or mN - ML = 0.5 (Rockburst Handbook, 1996).
Most of the energy released due to an earthquake takes the form of stress waves. The amount of this energy and consequently the characteristics of the stress waves are strongly related to the magnitude of the earthquake. Therefore, the magnitude of an earthquake affects ground motion characteristics.

![Figure 3-9: Relationship between various magnitude scales (after Youd et al., 2001).](image)

The strong ground motions produced by earthquakes can be described by different ground motion parameters. For engineering proposes, three characteristics of earthquake motions are: i) amplitude, ii) frequency content, and iii) duration. The following section will present some of the characteristics of ground motion parameters in earthquake engineering.

### 3.3.1.1. Ground Motion Parameters

Ground motion parameters, such as amplitude, frequency content and duration describe the important characteristics of strong ground motion in a quantitative form.

**Amplitude Parameters**
A time series/history of acceleration, velocity or displacement is the common way of describing a ground motion. Typically, one of these quantities is measured directly while the others are computed by integration or differentiation. For example, a displacement and velocity time series can be computed by double and single integration of acceleration time series recorded at the time of an earthquake, respectively. Note that the integration procedure might produce a smoothing or filtering effect in the frequency domain. Therefore, the velocity time series obtained from an acceleration time series with relatively high frequency content might show less high-frequency motion (Kramer, 1996).

Peak horizontal acceleration (PHA), which is the largest vector sum of two orthogonal horizontal components of a triaxial accelerogram, is commonly used to describe ground motions because PHA are closely related to the largest dynamic forces induced in certain structures. The PHA can also be correlated to earthquake intensity.

The higher the PHA, the more destructive might be the ground motion. However, very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures. Since peak acceleration is sensitive to high frequency components of ground motion, it provides no useful information on the frequency content of the motion. Therefore, peak horizontal velocity (PHV) might be a good substitution for PHA to characterize ground motion amplitude at intermediate frequencies (Kramer, 1996).

**Frequency Content Parameters**

The dynamic response of compliant objects, such as slopes and soil deposit, is dependent on the frequency at which they are loaded. Complicated dynamic loading produced by earthquakes shows a broad range of frequency motions. The frequency content expresses the distribution of frequencies with respect to the amplitude of ground motion. Therefore, the characterization of frequency helps to understand the effects of ground motion induced by earthquakes.

The frequency content of a ground motion can be described by different types of spectra, such as Fourier spectra, power spectra and response spectra. In this thesis, Fourier amplitude spectrum, which is a plot of Fourier amplitude versus frequency, is used. A Fourier amplitude spectrum of a ground motion can be smoothed and plotted on logarithmic scales to define its characteristics shapes, as shown in Figure 3-10. Fourier acceleration amplitudes tend to be largest over an
intermediate range of frequencies bounded by the corner frequency, \( f_c \), on the low side and the cut-off frequency, \( f_{\text{max}} \), on the high side. Another useful parameter that might be a representation of the frequency content of a ground motion is the predominant frequency. Predominant frequency is defined as the frequency of ground motion corresponding to the maximum value of the Fourier amplitude spectrum.

![Idealized shape of smoothed Fourier amplitude spectrum showing the corner frequency and cut-off frequency.](image)

**Figure 3-10:** Idealized shape of smoothed Fourier amplitude spectrum showing the corner frequency and cut-off frequency.

It has been shown that the corner frequency is inversely proportional to the cube root of the seismic moment. Therefore, the large earthquakes produce greater low-frequency motions than do smaller earthquakes (Kramer, 1996). Note that the higher-frequency components of a seismic wave that travels away from its source are scattered and absorbed more rapidly than are the lower-frequency components. Therefore, the frequency content changes with distance (Kramer, 1996).

**Duration**

The duration of strong ground motion has a strong effect on earthquake damages. For example, the generation of pore water pressure in loose, saturated sands during an earthquake loading that might result in liquefaction is sensitive to the number of load or stress reversals. A motion with high amplitude and low duration may not produce enough load reversals to cause damage while a motion with moderate amplitude and long duration can produce enough load reversals to cause substantial damage. In general, the duration of strong motion increases as the magnitude of the earthquake increases (Kramer, 1996). The duration of earthquakes has been defined by different
approaches, such as bracketed duration, energy based definition, and the power spectral density concept. For earthquake engineering proposes, bracketed duration has been commonly used, which is the time between the first and last surpasses of threshold acceleration (i.e., 0.05g).

3.3.1.2. Cyclic Stress Approach

Cyclic stress approach is a common way of evaluating the liquefaction potential of soils. In this approach, the earthquake-induced loading is expressed in terms of cyclic shear stresses. This loading is then compared with the liquefaction resistance of the soil. If the loading exceeds the resistance of the soil, liquefaction occurs. In the following sections, the loading conditions due to earthquakes will be illustrated first and the simplified procedure will then be reviewed.

Loading Conditions

As explained by Ishihara (1996), the main part of the ground motion during an earthquake is due to the upward propagation of body waves from an underlying rock formation. The effect of surface waves is not usually considered although surface waves are also involved. In the case of level ground, body waves produce shear stress and compressional stress while the soil element is not allowed to deform in the horizontal direction. It is known that the horizontal normal stress induced by the propagation of the compressional wave is nearly equal to the value of vertical normal stress. Therefore, the component of deviator stress is practically equal to zero. Since there is no change in the effective stress induced by the compressional wave, effects of compressional wave are disregarded in evaluating the stability of the ground. Therefore, horizontal shear stress due to vertically propagating shear waves is the main component of stress that is to be considered in one-dimensional stability analysis of level ground during an earthquake.

For an element of soil beneath a level ground surface subjected to vertically propagating shear waves (Figure 3-11), the most realistic loading conditions involve simultaneous changes in horizontal and vertical stresses and the development of shear stresses on horizontal and vertical planes of a soil element (Kramer 1996). The most important characteristics of the loading induced by vertically propagating S-waves are shown in Figure 3-11-b and -c. The stress path (Figure 3-11-c) never shows isotropic stress conditions since it never reaches the p’-axis and the principal stress axes rotate continuously.
Figure 3-11: (a) Stress and strain conditions imposed on element of soil below level ground by vertically propagating S waves at four different times; (b) orientations of principal stress axis; (c) stress path (from Kramer, 1996).

**Simplified Procedure**

For practical purposes, the liquefaction potential of a soil induced by an earthquake can be estimated by the cyclic stress ratio (CSR) approach known as “simplified procedure” proposed by Seed and Idriss (1971). In this method, a soil column of unit width and length at level ground surface is considered (Figure 3-12). The soil column will move horizontally as a rigid body in response to the horizontal maximum acceleration, $a_{\text{max}}$, exerted by the earthquake. The maximum shear stress, $\tau_{\text{max}}$, is equal to the horizontal force, $F$, acting on the soil column:

Figure 3-12: Equilibrium of forces near the surface for a column of soil.
where \( m \) is the total mass of soil column, \( \sigma_{v0} \) is the total vertical stress at the bottom of the soil column, and \( z \) is the depth of the soil column. Dividing both sides of the equation by the vertical effective stress, \( \sigma^{'}_v \), and adding the depth reduction factor, \( r_d \), to the right side of the equation suggested by Seed and Idriss (1971) due to deformability of the soil column gives

\[
\frac{\tau_{\text{max}}}{\sigma_{v0}} = r_d \left( \frac{\sigma_{v0}}{\sigma_{v0}} \right) \left( \frac{a_{\text{max}}}{g} \right).
\]  

(3.2)

For the simplified procedure, Seed et al., (1975) converted the typical irregular earthquake record to an equivalent series of uniform shear stress cycles, \( \tau_{\text{cyc}} \), by adopting a weighting method commonly referred to as the Palmgern-Miner (P-M) cumulative damage hypothesis (Green and Terri, 2005). The following assumption is then considered:

\[
\tau_{\text{cyc}} = 0.65 \tau_{\text{max}}
\]  

(3.3)

By substituting equation 3.3 into equation 3.2, the cyclic stress ratio is obtained:

\[
CSR = \frac{\tau_{\text{cyc}}}{\sigma_{v0}} = 0.65r_d \left( \frac{\sigma_{v0}}{\sigma_{v0}} \right) \left( \frac{a_{\text{max}}}{g} \right).
\]  

(3.4)

This CSR induced by the earthquake can then be compared with the cyclic resistance ratio (CRR) of the soil that represents the liquefaction resistance of the in situ soil. The CRR of the soil can be determined using different techniques, such as the standard penetration test and the cone penetration test. If the CSR caused by the anticipated earthquake is greater than the CRR of the in situ soil, then liquefaction could occur during the earthquake. The latest innovations in evaluation of liquefaction resistance of soils are beyond the context of this chapter and can be found in Youd et al. (2001).

3.3.2. Rockburst-induced Stress Waves

The seismic activity in underground mines is of great importance due to safety and productivity implications associated with seismicity-induced damage. Rockburst is a seismic event that occurs when accumulated stresses due to mining activities rupture an intact rock. In other words,
rockbursts are violent failure of rock that result in damage to excavations while every seismic event, known as mine tremors, may not result in damage (Gibowicz and Kijko, 1994).

Three mechanisms of mine failure that produce rockbursts are: (i) breaking of intact rock ahead of an excavation from stress concentrations, (ii) cracking at the active face and at opposite end of excavation from stress concentration, (iii) pillar bursting (Johnston, 1992). There are three processes that occur during a rockburst. First, there is a fracturing of the rock near the surface of the excavation. Second, this fracturing is accompanied by the displacement of the fractured rock. And third, a violent ejection of this fractured rock might be possible as this material becomes detached from the wall of the excavation (Rockburst Handbook, 1995). The violent ejections of rock do not always occur, but it can still be considered as a rockburst if only rock fracturing occurs. If there is no rock fracturing, the occurrence is not classified as a rockburst (Rockburst Handbook, 1995). The mechanisms of mine failure that might produce seismic events but not particularly rockbursts can be categorized as (i) roof collapse, (ii) slippage along pre-existing faults, and (iii) ore extruded because of high vertical stress from overburden (Johnston, 1992). Rockbursts can be categorized into two groups, Type I and Type II. The main characteristics of these two types are illustrated in Table 3-1.

Table 3-1: Characteristics of rockburst (Johnston, 1992).

<table>
<thead>
<tr>
<th></th>
<th>Type I</th>
<th>Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rate of occurrence</td>
<td>is a function of mining activity.</td>
<td>Relationship with mining rates is not determined.</td>
</tr>
<tr>
<td>Location</td>
<td>is generally within 100m of mining face or some pre-existing zone of</td>
<td>Location is on some pre-existing fault surface that may be up to 3 km</td>
</tr>
<tr>
<td></td>
<td>weakness or geological discontinuity near the mine.</td>
<td>from the mine.</td>
</tr>
<tr>
<td>Intact rock</td>
<td>can be broken in the rupture when mining-induced stresses exceed the</td>
<td>All occur in pre-existing, possibly pre-stressed tectonic faults.</td>
</tr>
<tr>
<td></td>
<td>shear strength of the material.</td>
<td>Mining may simply “trigger” these events on faults of preferred</td>
</tr>
<tr>
<td></td>
<td></td>
<td>orientation.</td>
</tr>
<tr>
<td>High stress drops</td>
<td>Often high stress drops observed. Low to medium magnitudes.</td>
<td>Stress drops more similar to natural earthquakes. Potential for high</td>
</tr>
<tr>
<td></td>
<td></td>
<td>magnitudes.</td>
</tr>
</tbody>
</table>

Generally, the methods and techniques used to study seismicity in mines have been adopted from earthquake seismology. The intensity of a seismic event, such as a rockburst should be defined in a manner that describes the seismic energy radiated from the source because induced ground motion and dynamic stresses are proportional to the radiated energy. Similar to earthquake engineering, the magnitude of a seismic event on the local Richter, $M_L$, or the Nuttli scale, $m_N$,
can be used to define the intensity of a seismic event although the magnitude does not
differentiate between different source-failure mechanisms. There is an upper limit to the seismic
event magnitude for a specific mine. This upper limit is controlled by the physical size of the
volume of the rock disturbed by mining activities, the nature of the largest discontinuities (faults,
dykes or contacts), the elastic properties of the rock mass, and the mean stress acting within the
volume (Kaiser et al., 1995). Typical seismic event magnitudes and the corresponding number of
events for a large Canadian mine are shown in Table 3-2. According to this table, large
magnitude events are significantly less frequent than low magnitude events. Gibowicz and Kijko
(1994) also gave an overview on seismicity in underground Canadian mines.

Table 3-2: Record of seismic events at a mine during an eight-year period (Kaiser et al., 1995).

<table>
<thead>
<tr>
<th>Local Richter magnitude</th>
<th>Number of seismic events larger during monitoring period</th>
<th>Number of seismic events larger per annum</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>127</td>
<td>15.875</td>
</tr>
<tr>
<td>1.5</td>
<td>91</td>
<td>11.375</td>
</tr>
<tr>
<td>2</td>
<td>36</td>
<td>4.5</td>
</tr>
<tr>
<td>2.5</td>
<td>7</td>
<td>0.875</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>0.375</td>
</tr>
<tr>
<td>3.5</td>
<td>1</td>
<td>0.125</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Dynamic stress waves induced by rockbursts depend on the magnitude of the event that may add
an increment of stress to the \textit{in situ} stresses. Rockbursts produce both compressional (i.e., \textit{p}-
wave) and shear waves that propagate through the rock as one-dimensional plane waves while
shear waves are more effective (Kaiser et al., 1995).

To better understand the changes in the stress condition in a rock mass at a distance from the
source of rockburst, the propagation of a one-dimensional plane wave is usually considered. The
one dimensional equation (along \textit{x}- axis) of motion in a homogeneous and isotropic medium is
expressed as

\[
\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2},
\]

(3.5)

where \( u \) is particle displacement and \( c \) is either \( \alpha \) or \( \beta \). The velocity \( \alpha = [ (\lambda + 2\mu)/\rho ]^{0.5} \) is
associated with the component of displacement parallel to the direction of propagation and the
velocity $\beta = [\mu / \rho]^{0.5}$ is associated with the components in two mutually perpendicular directions normal to the propagation, where, $\rho$ is the density of the medium, $\mu$ and $\lambda$ are elastic constants (Lame constants).

The shear stress, $\sigma_s$, can then be expressed as a function of particle velocity, $\dot{u}$, by considering the strain-displacement and the stress-strain relations:

$$\sigma_s = \rho c \ddot{u} \quad (3.6)$$

Therefore, the maximum dynamic stress induced by a shear wave can be written as a function of peak particle velocity, PPV, and shear wave velocity, $V_s$:

$$\sigma_{\text{max}} = \rho V_s \cdot (\text{PPV})_s \quad (3.7)$$

Note that the S-wave may be horizontally (SH) or/and vertically (SV) polarized. The same equation can be derived for the P-wave.

As indicated by Vasak and Kaiser (1995), the PPV of the P-wave is normally much lower than the PPV of the S-wave in case of rockbursts. Therefore, to modify the in situ state of stress, only the shear wave is considered. According to equation 3.7, there is a linear relationship between PPV and dynamic stress in a medium with constant wave velocity and density. Therefore, the time history of the particle velocity might be a good representative for practical purposes.

The seismic energy associated with elastic motion can be described by the kinetic and potential energies of the vibrating medium or by the energy density. The energy density is the energy per unit volume in the medium. For a plane wave, the strain energy density is equal to the kinetic energy density. Therefore, it is possible to show that the amount of energy transmitted per unit time across a unit area normal to the direction of propagation is $\rho \alpha (\partial u / \partial t)^2$ for P waves and $\rho \beta (\partial u / \partial t)^2$ for S waves (Gibowicz and Kijko, 1994). In other words, the seismic energy, $E_s$, radiated from a seismic source can be defined as a function of PPV and distance, $r$, as described by Perret (1972):

$$E_s = 4\pi \rho C r^2 (\text{PPV})^2 \int f(t)dt \quad (3.8)$$
where $C$ stands for $\alpha$ or $\beta$ wave velocity, and $f(t)$ is a time function dominated by attenuation and radiation characterises. Therefore, the seismic energy is proportional to $r^2$. $(\text{PPV})^2$ or $\log E_s$ is proportional to $2 \log [r \times (\text{PPV})]$.

### 3.3.3. Blast-induced Stress Waves

The stress wave induced by an explosion can be categorized as shock wave. When energy is imparted to the ground by an explosion, some is converted into a radiating stress-strain field and some is utilized in producing local deformations of a particular site. When the stresses in the expanding wave no longer exceed the strength of the medium, the shock wave has become an elastic wave and from there on is propagated according to the laws of elastic wave theory (Heelan 1953).

The duration of the elastic wave induced by blasting is about milliseconds with a relatively high frequency. Explosives induce both compressional and shear stresses. However, the compressional component of the induced elastic waves is more significant. It has been shown that blast-induced soil liquefaction is induced by one or several cycles of compressive strain followed by shear strain. Explosives in water saturated soil produce a compressional wave of high intensity and short duration (Al-Qasimi et al., 2005).

As explained by Rinehart, (1975), a saw-tooth compressional shock induced by blasting rapidly develops tensile stresses and becomes oscillatory behind the wave front. The fronts of the waves move with the dilation wave velocity at the front of the wave decaying as $1/r$ and $1/r^{1/2}$ for a spherical wave and a cylindrical wave, respectively ($r$ is the distance from the source of the disturbance). These two cases are categorized as non-planar waves. Some factors such as the source of dynamic loads, the location and orientation of blast holes and the type of blasts might affect the propagation of stress waves. If the length of the explosives in a bore hole is relatively less than the distance from the source in a medium, it may be possible to apply the criteria of spherical propagation of stress waves from a cavity source. However, if the length of the explosives is relatively high enough, it may be possible to apply the criteria of cylindrical propagation of stress waves. However, the front of the wave can be treated as plane waves in many practical problems particularly at short distances from the source. The typical solutions for the case where a uniform pressure is generated on the surface of a spherical and cylindrical cavity
in an infinite, isotropic, homogeneous medium can be found in many textbooks (e.g., Rinehart, 1975).

3.4. Mining Related Liquefaction Studies

Earthquake-induced liquefaction of soils has been studied extensively. However, there exist a limited number of studies on the liquefaction potential of silts and cemented silts that is the main concern of this thesis. In addition, the liquefaction potential of soils induced by rockbursts and explosives is another concern. Therefore, the following subsections will provide background information on the dynamic-load-induced liquefaction on one hand, and the liquefaction studies on silts, mine tailings, and CPB on the other hand.

3.4.1. Monotonic and Cyclic Loadings induced Liquefaction for Mine Tailings and CPB

Several studies reported that mine tailings are vulnerable to earthquake-induced liquefaction (Al Tarhouni, 2008; Crowder, 2004; Dobry and Alvarez, 1967; Fourie and Papageorgiou, 2001; Garga and McKay, 1984; Ishihara et al., 1980; Okusa et al., 1980; Poulos et al., 1985). The mine tailings used in these studies range from silts-sandy-silts to silty sands-sands. In the most recent studies on silt-sized mine tailings, Crowder (2004) showed that the behaviour of non-plastic tailings in both monotonic and cyclic triaxial tests is consistent with the definition of cyclic liquefaction suggested by Robertson (1994). It is noteworthy that the mine tailings specimens (void ratios ~ 0.70-0.72) tested in his study show strain hardening behaviour under monotonic loading and are not susceptible to liquefaction. In contrast, it has been shown that the monotonic response of similar mine tailings in simple shear tests at void ratios higher than 0.66 may show contractive behaviour, which results in liquefaction (Al Tarhouni, 2008). The specimens tested by Al Tarhouni (2008) at the same range of void ratios showed that the tailings are susceptible to cyclic mobility while zero effective stress was not achieved. He used the shear strain ($\gamma$) of 3.75% to estimate the number of cycles to liquefaction. Likewise, Wijewickreme et al. (2005) previously observed that the cyclic mobility type stress-strain response of clayey silt and silt mine tailings under simple shear loading are similar to the behaviour of natural clayey soils and natural silts, respectively. In addition to mine tailings, the monotonic behaviour of silt made of limestone was investigated under compression and extension triaxial tests. In these two types of triaxial test, Hyde et al. (2006) showed that silt specimens experience both contractive and dilative
behaviours, respectively while the dilative behaviour continued until an ultimate effective stress line was reached. This indicates that the silts are not susceptible to liquefaction under monotonic loading. It is noteworthy that the angle of the ultimate line in a compression test is higher than that of the ultimate line in an extension test. There are also some studies on the liquefaction susceptibility of natural silts and fine-grained soils, which will be reviewed in section 3.6 of this thesis.

A few studies also exist on the liquefaction potential of CPB. Aref (1989) showed that the response of CPB to monotonic loading in triaxial tests is dilative and CPB is not susceptible to liquefaction. Been et al. (2002) also investigated the response of cured CPB using undrained compression triaxial tests. The CPB specimens showed dilative behaviour with no significant pore pressure development during monotonic loading. More recently, le Roux et al. (2004) showed that the cyclic response of Golden Giant CPB with 5% binder content and cured for 3 hours is similar to that of non-plastic silts. They also showed that the 12 hour-cured CPB are not susceptible to cyclic liquefaction although the 3 hour-cured CPB specimens are susceptible to cyclic liquefaction. Le Roux, (2005) also showed that the response of CPB to monotonic loading in triaxial tests is similar to silts. In addition, she showed the evolution of cohesion and the angle of the phase transformation line due to the hydration of cement in CPB.

3.4.2. Rockburst-induced Liquefaction

Unlike earthquake-induced liquefaction, there is no study on rockburst-induced liquefaction for CPB and mine tailings. However, Kaiser et al. (1995) provided general guidance regarding rockburst-induced ground motion in Canadian hard rock mines. As shown in Figure 3-13, the anticipated levels of dynamic stress increment induced at the wall of an underground opening, which may be backfilled, vary based on the distance from the source and the magnitude of rockburst. It is important to note that for highly stressed or meta-stable underground openings even a small dynamic stress might be significant in terms of ground motion. Figure 3-14 also indicates the levels of maximum ground motion based on PPV as a function of distance from seismic source and its magnitude. Although these parameters are well characterized, there is no data on the frequency of stress waves induced by rockbursts in hard rock mining.
Figure 3-13: Anticipated levels of maximum dynamic stress induced by rockburst (Kaiser et al. 1995).

Figure 3-14: Anticipated levels of ground motion induced by rockburst (Kaiser et al. 1995).
Johnson et al. (2007) investigated the response of CPB to dynamic loads based on rockburst observations in the Galena mine and Split Hopkinson Pressure Bar tests (SHPB). The Galena mine uses an underhand mining technique with silty sand CPB. This mine has experienced rockbursts as large as magnitude 3.5. The idea of using SHPB tests was to simulate the field condition in the laboratory since the configuration of the wall rock and backfill in the mine was similar to the test method. Although the test method was not appropriate for evaluating the liquefaction of CPB, it was possible to determine the dynamic strength of the cured specimen. The dynamic compressive strength measured for 28 hour cured CPB specimens was about twice the unconfined compressive strength (UCS). The results also showed that 95% of the initial energy was reflected away from the CPB specimen and only 5% of the energy was absorbed.

Rockburst-induced ground motion may also be of interest for engineers with respect to surface intensity that causes structural vibrations. Rockbursts in this regards are classified after underground nuclear explosions, but before surface mine explosions and construction blasts. Zembaty (2004) compared strong ground motion parameters induced by rockburst events with low intensity earthquakes. In his study, the ground motions during rockbursts in the vicinity of a copper mine were recorded. The depth of mine activity was 1000 m and the instruments were installed at the foundations of some buildings. The typical time history record of acceleration showed that the duration of events is short (~2-4 seconds) in comparison with earthquakes. The Fourier spectra of acceleration also show that the dominating parts have relatively high frequencies (~10-20 Hz). The peak ground velocities and displacements induced by rockbursts were much lower than those induced by earthquakes with the same peak ground accelerations.

### 3.4.3. Blast-induced Liquefaction

Several cases of blast-induced liquefaction have been reported as shown in Table 3-3. The description of these events can be found in Bretz (1989). Blast-induced liquefaction studies have been of interest in many countries since the 1960’s. Some background theory used in blast-induced liquefaction has usually been adopted from earthquake-induced liquefaction science. For example, the determination of excess pore water pressure, threshold strain, and threshold particle velocity has been a common interest in both categories. Studer and Kok (1980) categorized the pore water pressure responses before, during and after blasting into three groups: (i) hydrostatic, (ii) dynamic pore water pressure, which is related to the amplitude of the stress waves, and (iii)
residual pore water pressure, which is controlled by the intensity of loading and soil conditions. Charlie and Veyera (1985) also categorized the dynamic pore water pressure into transient response during loading and a short-term residual response after passage of the stress waves. Based on some case studies, Charlie et al. (1985a) indicated that the compressive strains of less than 10-2 percent should be in the elastic range of soil, thus residual pore water pressure would not be induced after passage of a stress wave. Having a threshold for compressive strain, it is possible to calculate the threshold particle velocity by using the relationship between strain and particle velocity for a plain wave.

**Table 3-3: Blast-induced liquefaction cases (Bretz, 1989).**

<table>
<thead>
<tr>
<th>Event</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calaveras Dam, California</td>
<td>1918</td>
</tr>
<tr>
<td>Swir III Dam, Russia</td>
<td>1935</td>
</tr>
<tr>
<td>Pacific Attolls</td>
<td>1950’s</td>
</tr>
<tr>
<td>Snowball Event, Canada</td>
<td>1964</td>
</tr>
<tr>
<td>Prairie Flat Event, Canada</td>
<td>1968</td>
</tr>
<tr>
<td>Dial Pack Event, Canada</td>
<td>1970</td>
</tr>
<tr>
<td>Pre-Dice Throw, New Mexico</td>
<td>1975</td>
</tr>
<tr>
<td>Haymon Igloo Test, Utah</td>
<td>1988</td>
</tr>
</tbody>
</table>

The blast-induced liquefaction mechanism is based on the assumption that the stress-strain behaviour of the pore water phase in a saturated soil is linearly elastic while the soil skeleton exhibits non-linear or plastic behaviour (Rischbieter, 1977; Shaepermeier, 1978; Charlie et al., 1985b). In other words, the dynamic strain in the skeleton is equal to the overall strain of the total soil-water mixture and a large hysteresis in stress-strain path occurs in the soil skeleton as the compression stress wave passes. Therefore, it is possible to reach zero effective stress while the pore pressure has a positive value, which results in liquefaction. Fragaszy and Voss (1981) verified the blast-induced liquefaction mechanism for quasi-static, isotropic loading by performing a series of high pressure undrained isotropic compression triaxial tests on Eniwetok Beach sand and Ottawa sand. In addition to quasi-static tests, three other laboratory tests have been performed to investigate blast-induced liquefaction: shock tube tests, impact tests, and small scale blast tests.

Veyera (1985) and Veyera and Charlie (1990), described laboratory experiments in which fine, uniform sands were placed in a shock tube and subjected to one-dimensional, high amplitude, short duration compression loads. According to Veyera (1985), liquefaction was generally
observed at strains greater than 0.1% for Monterey sands. Veyera and Charlie (1990) also mentioned that saturated sands at low relative densities ($D_r = 10\%$) and an initial effective stress of 86 kPa could be liquefied if the maximum compression strain exceeded 0.05%. Bolton et al. (1994) determined the relationships between quasi-static and shock tests on confined saturated Monterey sand, bonny silt and the mixture of the two soils. The results showed that there is no significant difference in pore pressure response and maximum strain between confined quasi-static and shock tests for Monterey sand. However, the maximum strain required to liquefy the silty sand mixture and bonny silt under confined quasi-static loading is about an order of magnitude smaller than the maximum strain required under shock loading.

Tanimoto (1967) presented results of impact tests on saturated, fine grained sand which was placed loosely in a container completely submerged in water. In this experiment, a free falling hammer was used to apply an impact loading. The excess pore water pressure was observed near the bottom of the sand after applying one impact. An approximately linear triangular distribution of settlement versus time occurred until 90% consolidation.

The compaction of Volga sand with surface, underwater, and buried detonation was investigated by Ivanov (1967). The experiments were conducted in a concrete container with a saturated sand height of about 1 m. excess pore water pressure were observed after detonation of 1.5 g explosive. Dowding and Hryciw (1986) investigated the effect of explosives on pore water pressure of loose saturated sand. The explosives were detonated in a 102 cm high tank of sand. The surface settlement and maximum water rise in a piezometer ranged from 0.8 to 1.7 cm and 152 to 178 cm, respectively.

The common practice to investigate blast-induced liquefaction in the field is to derive an empirical equation relating peak pore water pressures ($u_{peak}$), residual pore pressure ratio (PPR) and PPV versus scaled distance. The residual pore pressure ratio ($\Delta u_{res}$) is measured following passage of the blast induce stress wave and is defined as the ratio of the residual pore pressure increase divided by the initial effective vertical stress. Table 3-4 shows an abbreviated list of empirical equations developed from previous field explosive tests as well as laboratory high intensity compression wave tests conducted on sands.
Table 3-4: Empirical equations to predict peak and residual pore pressure, and peak particle velocity (Al-Qasimi et al., 2005).

<table>
<thead>
<tr>
<th>Empirical Equation</th>
<th>Best Fit</th>
<th>Explosive Type</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Peak Pore Pressure (kPa)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( u_{\text{peak}} = 59000 \frac{R}{M^{1/3}}^{-1.05} )</td>
<td>–</td>
<td>Unknown</td>
<td>Lyakhov (1961)</td>
</tr>
<tr>
<td>( u_{\text{peak}} = 50093 \frac{R}{M^{1/3}}^{-2.38} )</td>
<td>0.78</td>
<td>Tovex ( R )</td>
<td>Charlie et al. (1992)</td>
</tr>
<tr>
<td><strong>Peak Particle Velocity (m/s)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \text{PPV} = 5.6 \frac{R}{M^{1/3}}^{-1.50} )</td>
<td>–</td>
<td>TNT</td>
<td>Drake &amp; Little (1983)</td>
</tr>
<tr>
<td>( \text{PPV} = 8.75 \frac{R}{M^{1/3}}^{-2.06} )</td>
<td>0.91</td>
<td>Tovex ( R )</td>
<td>Charlie et al. (1992)</td>
</tr>
<tr>
<td>( \text{PPV} = 0.264 \frac{R}{M^{1/2}}^{-0.74} )</td>
<td>0.67</td>
<td>Several</td>
<td>Narin van Court (1997)</td>
</tr>
<tr>
<td>( \text{PPV} = 22 \frac{R}{M^{1/3}}^{-2.01} )</td>
<td>–</td>
<td>Geogel ( R )</td>
<td>Handford (1988)</td>
</tr>
<tr>
<td>( \text{PPV} = 1.7 \frac{R}{M^{1/3}}^{-1.36} )</td>
<td>–</td>
<td>Mix</td>
<td>Rollins et al. (2001)</td>
</tr>
<tr>
<td><strong>Pore Pressure Ratio</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \text{PPR} = 65 \frac{R}{M^{1/3}}^{-2.7} (\sigma'_{\text{i}})^{-0.33} )</td>
<td>0.9</td>
<td>Dynamite</td>
<td>Kummeneje &amp; Eide (1961)</td>
</tr>
<tr>
<td>( \text{PPR} = 1.65 + 0.64 \ln \left( \frac{M^{1/3}}{R} \right) )</td>
<td>–</td>
<td>TNT</td>
<td>Studer &amp; Kok (1980)</td>
</tr>
<tr>
<td>( \text{PPR} = 6.7 \left( \text{PPV}^{0.31} (\sigma'_{\text{i}})^{-0.31} (D_R)^{-0.179} \right) )</td>
<td>0.66</td>
<td>Impact</td>
<td>Veyera (1985)</td>
</tr>
</tbody>
</table>

*Note: R = vector distance in m; M = explosive mass in kg; DR = relative density in percent; \( \sigma'_{\text{i}} \) = initial effective stress in kPa; and \( R^2 \) = coefficient of determination.*

Most recently, Al-Qasimi et al. (2005) have developed the following empirical equations based on single- and multiple-blast *in situ* experiments on near surface mine tailings deposits as part of the Canadian Liquefaction Experiment (CANLEX):

\[
\text{PPV} = 47.64 \left[ \frac{R}{(M_{\text{TNT}})^{1/3}} \right]^{-2.34} \text{ m/s (r}^2 = 0.89) \quad (3.9)
\]

\[
\Delta u_{\text{peak}} = 143,000 \left[ \frac{R}{(M_{\text{TNT}})^{1/3}} \right]^{-2.34} \text{ kPa.} \quad (3.10)
\]

Table 3-5 shows the minimum PPV at which blast-induced stress wave results in increasing residual pore pressure and liquefaction. The criteria to evaluate cyclic liquefaction in these cases are defined as PPR=1. It is possible to see that the threshold PPV is different from site to site and depends on the type of deposit.
Table 3-5: Threshold peak particle velocity resulting in liquefaction.

<table>
<thead>
<tr>
<th>Site</th>
<th>PPV (m/s)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Syncrude tailings</td>
<td>&gt; 0.04</td>
<td>Handford (1988)</td>
</tr>
<tr>
<td>Levee and dams</td>
<td>0.025-0.1</td>
<td>Sanders (1982)</td>
</tr>
<tr>
<td>Natural sand &amp; small sand-fill dam</td>
<td>&gt; 0.015</td>
<td>Charlie et al. (1992) and (2001)</td>
</tr>
<tr>
<td>Syncrude J-Pit (Single detonation)</td>
<td>&gt; 0.65</td>
<td>Al-Qasimi et al. (2005)</td>
</tr>
<tr>
<td>Syncrude J-Pit (12 multiple detonations, millisecond delays)</td>
<td>&gt; 0.13</td>
<td>Al-Qasimi et al. (2005)</td>
</tr>
</tbody>
</table>

Al-Qasimi et al. (2005) also calculated the peak total stress increase induced by a compressive stress wave in Syncrude tailings with a density of 1988 kg/m³ and $V_p = 1608$ m/s based on equation 3.7 as follows:

$$\Delta\sigma_{\text{peak}} = 3200 \text{ (PPV) kPa.} \quad (3.11)$$

Although, a number of blast-induced liquefactions have been studied for near surface deposits, the investigations on blast-induced liquefaction of CPB in underground conditions are limited. Aref (1989) investigated the dynamic response of CPB at Dome Mines by recording the acceleration time series during blasting. Mohanty et al. (1995) also developed an empirical equation to show the relationship between PPV (mm/sec) and scaled distance in backfill during VCR blasting at Crean Hill Mine as follows

$$\text{PPV} = 745 \left(\frac{R}{M^{1/25}}\right)^{-1.22}, \quad (3.12)$$

where R is distance in ft and M is charge weight in lbs. The most important difference between blast-induced liquefaction results in CPB and those in soils is that the explosive source in the case of CPB is inside the surrounding rock while the explosive source is located in the soil in the case of surface deposit soils. In other words, the stress wave induced by blasting is traveling through two media in underground CPB response analysis studies.

### 3.5. Additional Material Parameters Affecting Liquefaction

The most important factors affecting the liquefaction of soils can be articulated as follows: degree of saturation, void ratio, cementation, plasticity index, and confining pressure. The effect of these factors will be reviewed in the following section.
3.5.1. State of Saturation and Occluded Air

The state of saturation can significantly affect the resistance of soils to liquefaction. Unsaturated conditions, which result in the existence of air or any type of gas in soils, may increase the liquefaction resistance of soils. Many case studies show that the liquefaction resistance of sands depends on the degree of saturation. For example, Yang (2004) has presented the results of four case studies and concluded that the cyclic stress ratio (CSR) increases with decreasing the degree of saturation at a specific number of cycles causing liquefaction. In addition, fully saturated sands have the least values of its resistance to liquefaction.

The flow and cyclic liquefaction of gassy sands, which have a large amount of gas dissolved in the pore fluid, have also been studied by Grozic et al. (1999) and Grozic et al. (2000), respectively. The results showed that the cyclic resistance of gassy sand increases with increasing amount of gas. It is noteworthy that the presence of air in gassy sand is different from that of unsaturated soil.

The presence of occluded air bubbles in tailings sand and CPB has been reported by Fourie et al. (2001) and le Roux (2004), respectively. Undisturbed samples of tailings sand showed that the mine tailings can be unsaturated even below the phreatic surface. The results of undrained loading tests revealed that the contractive behaviour of the materials was reduced by the presence of bubbles (Fourie et al., 2001). The air bubbles in CPB may be entrapped during mixing, transport and placement processes (Grabinsky and Simms, 2006). However, the amount of gas (i.e., air or mostly nitrogen) dissolved in pore water is not significant in CPB due to low solubility of air in water. The liquefaction potential of CPB in unsaturated conditions is not well studied.

3.5.2. Relative Density and Particle Characteristics

Relative density is one of the most important factors that may affect the liquefaction potential of soils (Lee and Seed, 1967). It is well known that loose sand is more susceptible to liquefaction than dense sand because of the tendency of loose sand to contraction. The gradation also affects the liquefaction susceptibility of soils. It has been noted that uniformly graded sands are most susceptible to liquefaction (Rischbieter, 1977; Prakash, 1981; Charlie et al., 1985a). Soil particle characteristics that influence liquefaction potential include roughness, roundness, and
crushability. It has been shown that a high crushable sand is more liquefiable than a less crushable sand. Angular to sub-angular sands are more stable than rounded sands (Fragaszy and Voss, 1981).

3.5.3. Effect of Plasticity Index

An addition of clays or fine-grained minerals with plasticity to non-plastic silts may change the resistance of silts against liquefaction (Guo and Prakash, 1999). It is noted that the liquefaction resistance of silts increases as PI increases if 5 < PI < 20 (El Hosri et al., 1984; Puri, 1990), while it decreases with increasing PI if 2 < PI < 4 (Sandoval, 1989; Prakash and Sandoval, 1992). In addition, Ishihara et al. (1980) described that the non-plastic tailings have much smaller cyclic strength than the tailings with a plasticity index of 15 to 20. Table 3-6 summarizes the results of the studies on the liquefaction resistance of silts with respect to plasticity index.

<table>
<thead>
<tr>
<th>Sample</th>
<th>PI</th>
<th>Liquefaction resistance</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt and clayey silt</td>
<td>2-4</td>
<td>decreases with increasing PI</td>
<td>Prakash &amp; Sandoval (1992)</td>
</tr>
<tr>
<td>Silt-clay</td>
<td>1.7-3.4</td>
<td>decreases with increasing PI</td>
<td>Sandoval (1989)</td>
</tr>
<tr>
<td>Silts and clay</td>
<td>10-20</td>
<td>increases with increasing PI</td>
<td>Puri (1984)</td>
</tr>
<tr>
<td>Silts - clayey silts</td>
<td>5-15</td>
<td>increases with increasing PI</td>
<td>El Hosri et al. (1984)</td>
</tr>
</tbody>
</table>

Prakash et al. (1998) also indicated that the cyclic stress ratio (CSR) for silty soil samples (undisturbed or reconstituted) starts decreasing until a critical value of PI is reached and then it increases with the increase of PI. The CSR of silty-clay mixtures with high plasticity index may be higher than the CSR for the non-plastic silt or sand-silt mixtures. Singh (1994) presented that the CSR of non-plastic silt is less than the CSR of sand (100%), while the CSR for the mixture of same silt and sand is less than the CSR of the non-plastic silt. More studies have recently been completed on the effect of PI on the liquefaction potential of silts and will be reviewed in section 3.6 of this chapter.

3.5.4. Effect of Cement

The monotonic and cyclic liquefaction potentials of cemented sands have been well understood (Clough et al., 1989; Clough et al., 1981; Dupas, 1979; Frydman et al., 1980; Rad and Clough, 1982; Saxena et al., 1989; Saxena and Lastrico, 1978; and Lo et al., 2003). Generally, cementing agents increase the effective cohesion of soils (Lo et al., 2003). Therefore, the increase of
liquefaction resistance is expected. Two aspects of dynamic response of cemented sands to liquefaction in saturated states can be summarized as follows: (1) the response of loose, cemented sand is similar to dense uncedemented sand; and (2) with an increase in cement, the resistance to liquefaction increases (Clough et al., 1989).

The monotonic response of CPB tested by Aref (1989) and Been et al. (2002) in compression triaxial tests was dilative and consequently resistant to liquefaction. However, they did not show the behaviour of un-cemented mixtures for comparison. Le Roux (2004) showed that the cyclic response of early age CPB was the same as non-plastic silt. In addition, CPB showed a less sensitive response to stress ratio due to the employment of cementing agent.

The properties of CPB including the coefficient of permeability change with time due to the hydration of cement. For example, le Roux (2004) showed that the coefficient of permeability for a silt-sized 5% CPB specimen significantly reduces at the early stage of cement hydration while stabilizing at about 5x10^-6 cm/s after a day, as shown in Figure 3-15. This reduction in permeability may affect the generation of pore water pressure and consequently the monotonic or cyclic behaviour of CPB. Therefore, timing is an important factor when determination of mechanical properties of CPB including its monotonic behaviour is desired.

![Figure 3-15: Permeability of early age 5% CPB (after le Roux, 2004).](image)

To determine the monotonic behaviour of CPB in the laboratory, specimens are subjected to an axial loading under constant strain or stress rate. The lower the strain rate is chosen for a test, the
longer the time is required to achieve an axial strain level. For example, Crowder (2004) used a low axial strain rate of 0.03%/min for uncemented tailings specimens. Therefore, the time required for the specimens to reach their 10% axial strains was about more than 5.5 hours. Although this strain rate is applicable for uncemented tailings, this might not be applicable for CPB since the mechanical properties of CPB may change during this time frame of the test (i.e., 5.5 hours). To minimize the effect of cement hydration on the monotonic behaviour of CPB during the time frame of the test, therefore, high axial strain rates may be used. However, the strain rate might affect the monotonic behaviour of the material. For example, Yamamuro and Lade (1999) in the undrained triaxial compression tests on Nevada sand with relatively low silt content showed that the dilative behaviour of the sand might increase as the axial strain rate increases. In other words, flow liquefaction might occur at a low axial strain rate while the dilatancy of Nevada sand increases as the axial strain rate increases at a specific effective confining stress. Therefore, the strain rate and the effect of cement during the monotonic test must be investigated for CPB.

3.5.5. Effective Confining Stress and Fines Content

The effective confining stress is known to be a factor affecting the liquefaction potential of soils. For example, the typical monotonic response of loose sands at different effective confining stresses in triaxial testing is shown in Figure 3-16. This figure indicates that loose sands are stable at a low effective confining stress while the instability increases as effective confining stresses increases (Yamamuro and Lade, 1998).

Yamamuro and Lade (1998) showed that the addition of low amount of silt to loose clean sand may change the monotonic response of the soil. As shown in Figure 3-17, the static liquefaction may be achieved at a low effective confining stress for sands with low silt content, which is different from the typical response. Yamamuro and Covert (2001) also observed a similar response for loose sands with high silt content. Static liquefaction (i.e., flow liquefaction) was achieved for the specimens of loose sands with high silt content at the effective stresses less than 50 kPa (Figure 3-18). They also interpreted the changes in behaviour of loose sand by defining a model, which is beyond the scope of this section.
3.6. Liquefaction Susceptibility Criteria for Fine Grained Soils

The evaluation of liquefaction susceptibility of fine grained soils including silts and clays has recently come to attention because these so-called non-liquefiable soils have shown the opposite behaviour. There are several cases where earthquake-induced ground failure in silty and clayey soils caused damage to buildings. The 1999 Kocaeli earthquake in Turkey (Yilmaz et al., 2004) and the 1999 Chi-Chi earthquake in Taiwan (Chu et al., 2004) are two examples.
Figure 3-18: Response of loose sands with high silt content (after Yamamuro and Covert, 2001).

The current state-of-the-practice for the evaluation of soil liquefaction is explained by Youd et al. (2001). It is generally believed that clay-rich soils are not susceptible to liquefaction. However, Youd et al. (2001) recommended that if a soil is classified as a clay-rich soil by using the Soil Classification Chart by Robertson and Wride (1998), a laboratory experiment would be required to check the liquefaction resistance. The use of liquefaction criteria, such as the Chinese criteria is also recommended to assess the liquefaction resistance of clayey soils in Youd et al. (2001). The same procedure is recommended for a soil with high silt content in Youd et al. report. However, it seems this report is not clear on the evaluation of liquefaction potential of these types of soils. Besides, direct experiments are preferred to the Chinese criteria. There are also a few studies indicating that there is no definite criterion for evaluating the cyclic liquefaction potential of silts and silt-clay mixtures (Guo and Prakash, 1999; Singh, 1994). In the following sections, some conventional and novel liquefaction susceptibility criteria for fine grained soils, which might be applicable for silts, mine tailings, and CPB, will be reviewed.
3.6.1. Chinese Criteria

According to the Chinese criteria developed by Wang (1979), fine-grained soils (i.e., clays (CL) or silty clays (ML-CL)) may be susceptible to liquefaction as a result of earthquake loading if they satisfy all the following conditions: (i) percent of particles finer than 0.005 mm <15%, (ii) liquid limit (LL) < 35%, and (iii) natural water content > 0.9LL (or w/LL >0.9). The original Wang (1979) data plotted on the Casagrande plasticity chart, which led to the development of the Chinese Criteria, are shown in Figure 3-19.

Figure 3-19: The original data which led to the development of the Chinese Criteria (after Bray and Sancio).

As described by Prakash and Puri (2003), the Chinese practice of determining the liquid limit, plastic limit, water content and clay fraction differs from the ASTM procedures. To eliminate this problem, Finn (1991, 1993) and Perlea et al. (1999) suggested the following adjustments of the index properties prior to applying the Chinese criteria: (i) decrease the fines content by 5%, (ii) increase the liquid limit by 1%, and (iii) increase the water content by 2%. Figure 3-20 illustrates the Chinese criteria modified in accordance with the above suggestions. In this figure, the soils that fall below the line defined by w = 0.87 LL and LL = 33.5 are considered as susceptible to liquefaction.

Observations from recent earthquakes, such as the 1999 Kocaeli earthquake in Adapazari (Turkey) and the results of cyclic tests on the silts of Adapazari indicate that the Chinese criteria are not reliable for determining the liquefaction susceptibility of fine-grained soils (Boulanger and Idriss, 2006; Bray and Sancio, 2006). One of the possible reasons is that the original data used to develop the Chinese criteria are based on clay data. In other words, there is no silt or low plasticity silt (ML) data in the Casagrande chart (Figure 3-19). Therefore, some modifications to the Chinese criteria would be required for evaluating the subspecialty of silts.
3.6.2. Criteria for Silts

Bray et al. (2004) proposed preliminary recommendations to modify the criteria. They used PI and wc/LL ratio to classify the soils of Adapazari region (Turkey) with respect to cyclic liquefaction potential. According to the further data provided using cyclic triaxial tests on the same type of soil, Bray and Sancio (2006) showed that a soil may be (i) susceptible to liquefaction when wc/LL >0.85 and PI < 12, (ii) moderately susceptible to liquefaction wc/LL >0.8 and 12 < PI < 18; (iii) liquefaction resistant when PI > 18. Figure 3-21 presents the fine-grained soil liquefaction susceptibility criteria proposed by Bray and Sancio (2006).

The data sets include: (i) the isotropically consolidated CTX tests; (ii) field observations and tests in Adapazari (Bray et al. 2004); (iii) the re-evaluation of the Bennett et al. (1998) field and index tests from Potrero Canyon for soils that liquefied during the 1994 Northridge earthquake; (iv) data in China from Wang (1979); and (v) some recent observations in Taiwan after the 1999 Chi-Chi earthquake from Chu et al. (2004).

It is possible to see that although the data obtained from the laboratory and field experiments in Adapazari (i.e., Figure 3-21a and b) show great agreement with the Bray et al. criteria, the other data presented in Figure 3-21 do not show good agreement with the criteria (Bray and Sancio, 2006). In another instance, Wijewickreme et al. (2005) investigated the applicability of the Bray et al. liquefaction susceptibility criteria on fine-grained mine tailings. As shown in Figure 3-22, the cyclic direct simple shear tests (DSS) for laterite tailings are in good agreement with the Bray et al. criteria. However, copper-gold tailings and copper-gold-zinc tailings show indecisive
behaviour, where some specimens are considered not susceptible to liquefaction, which is contrary to the results obtained from the cyclic DSS tests (Wijewickreme et al., 2005).

Figure 3-21: The fine-grained soil liquefaction susceptibility criteria proposed by Bray and Sancio (2006).

Figure 3-22: Applicability of the liquefaction susceptibility criteria proposed by Bray et al. for three mine tailings (Wijewickreme et al., 2005).

More recently, Sanin and Wijewickreme (2006) investigated the applicability of the Chinese criteria and the criteria proposed by Bray et al. for the channel-fill Fraser river delta (FRD) silt (PI = 4). According to the Chinese criteria, the #3-FRD silt is located in the category of
“potentially liquefiable”. On the other hand, the Bray et al. criteria classify the silt into the category of susceptible to “liquefaction or cyclic mobility”. Since the cyclic DSS test suggest that the #3-FRD silt is unlikely to experience flow failure, the Chinese criteria is slightly conservative while the Bray et al. criteria do not provide a clear distinction between the liquefaction associated with strength loss and “cyclic mobility”.

As an alternative to the Bray et al criteria, Boulanger and Idriss (2006) proposed other liquefaction susceptibility criteria for silts and clays subjected to earthquake loading. The details on the development of these criteria have been given in Boulanger and Idriss (2004 and 2005). The philosophy behind this approach is to answer the question, “What is the best way to estimate the potential for strength loss and large strains in different types of fine-grained soils?” instead of trying to answer the question, “what types of silts and clays are susceptible to liquefaction?” In this approach, therefore, the behaviour of fine-grained soils due to monotonic and cyclic undrained loading has been observed to distinguish between sand-like behaviour and clay-like behaviour over the range of Atterberg limits. In addition, there is also an intermediate behaviour between these two groups. As shown in Figure 3-23, the Atterberg limit chart demonstrates the representative values for these three groups of soils. It has been noted that for engineering practice, fine-grained soils can be considered as clay-like if PI > 7 and sand-like if PI < 7 (Boulanger and Idriss, 2006).

For fine-grained soils that behave like clays, the cyclic and monotonic undrained shear strengths are closely related and show relatively unique normalized stress-history behaviours. Cyclic strengths can then be evaluated based on information from in situ testing, laboratory testing, and empirical correlations, which has been called “shear-strength-based” procedures described in Boulanger and Idriss (2004). For fine-grained soils that behave like sands, the cyclic strengths may be more appropriately estimated within the framework of existing SPT and CPT based liquefaction correlations.

### 3.7. Summary and Plan of Work

#### 3.7.1 Summary

Based on the objectives of the thesis, the three aspects of the literature review that had to be investigated were: i) monotonic and dynamic behaviours of fine grained soils, ii) characteristics
of dynamic loadings, and iii) availability of liquefaction design criterion, that the designer can rely on for geomechanical design of CPB systems.

Figure 3-23: Representative values for each soil that exhibited clay-like, sand-like, or intermediate behaviour (after Boulanger and Idriss, 2006).

3.7.1.1 Behaviour of fine grained soils

Within the context of geotechnical earthquake engineering, there are four liquefaction susceptibility criteria for soils, including historical, geological, state and compositional criteria (Kramer, 1996). Among these criteria, state criteria and compositional criteria were reviewed for mainly fine grained soils (i.e., silt-sized mine tailings, silts and clays), which are the main concern of this thesis. The literature review revealed that the state criteria for silts and cemented silts are less well understood than for sands. Therefore, a detailed review of behaviour of clean sands was given to better understand the basic concepts of liquefaction potential.

It was then shown that there are different laboratory experiments on silt-sized mine tailings (e.g., Al Tarhouni, 2008; Wijewickreme et al., 2005; Crowder, 2004) and manufactured silts (e.g., Hyde et al.). However, the effect of cement at the early stage of hydration on mine tailings is not well understood. The laboratory study performed by le Roux (2004) just showed the monotonic behaviour of CPB at low axial strains (~ 3%) and the cyclic behaviour of CPB for limited specimens. In addition, the effect of binder content and binder type (i.e., fly ash and slag) on the monotonic and cyclic behaviours of CPB has yet to be investigated.
The state of practice in paste technology is to add a small quantity of cementitious materials (i.e., binder agents) to backfill in order to improve short term and long term strength. The ‘rule of thumb’ used to consider backfill as liquefaction resistant is to achieve an unconfined compressive strength (UCS) of 100 kPa (le Roux, 2004). This guideline has been adopted from the special case study on clean rounded cemented sand (Clough et al., 1989). However, this guideline might be conservative for the design of CPB systems and needs to be investigated.

In addition to cement content, other material parameters including occluded air, fines content and plasticity index that might affect the liquefaction potential of soils were reviewed. These studies were performed on the variety of soil types (e.g., sands, silty sands, sandy silts and clays). However, the effect these parameters on the liquefaction potential of silts is not well investigated.

In contrast to the state criteria, the compositional liquefaction susceptibility criteria for fine grained soils have been intensively studied. The two new criteria proposed by Bray et al. (2004) and Boulanger and Idriss (2004) were reviewed in this chapter. The applicability of the Bray et al. criteria has been investigated for fine-grained mine tailings (e.g., Wijewickreme et al., 2005) and natural silt deposits (e.g., Sanin and Wijewickreme, 2006). In most cases, there is a good agreement between the criteria and the laboratory behaviour of the materials. However, applicability of the Bray et al. criteria has not yet been investigated for cemented tailings (i.e., CPB).

### 3.7.1.2 Dynamic Loadings

Liquefaction (i.e., cyclic mobility) might be triggered by different dynamic loadings in the vicinity of a mine including earthquakes, rockbursts and blasting events. However, the liquefaction criteria are only available for earthquake-induced liquefaction. To investigate the applicability of these criteria for the other types of loading, the characteristics of the three dynamic loadings including earthquakes, rockbursts and blasting events were investigated in this chapter. Although earthquake-induced liquefaction is very well understood, rockburst- and blast-induced liquefaction phenomena are less well understood.

It was shown that the empirical equations derived based on the field data for evaluating blast-induced liquefaction depend on the site conditions. There are no well defined susceptibility criteria for evaluating the liquefaction potential of soils induced by blasting. On the other hand,
the ground response parameters including PPA, PPV and frequency content induced by blasting might be different in each case and not well determined. Most of the studies were performed in one medium which means that the seismic source was in the same medium in which its response to the seismic event is of interest. However, in an underground stope, the seismic source is located in one medium (i.e., rock) and the response of a different medium (i.e., CPB) adjacent to the medium of the source is of interest.

For the rockburst events, although there are some design criteria (for other geomechanical design problems) available in underground hard rock mining with respect to PPV in rock and the magnitude of the events, the frequency content of the seismic events are not determined. Note that there is no field data available showing the response of CPB due to a rockburst event.

3.7.2 Plan of Work

This thesis aims to understand the state criteria of CPB and the characteristics of rockbursts and blast loadings. Chapter 4 will focus on a comparison of loads for earthquakes versus mine-induced events (i.e., rockbursts and blasting).

The remainder of the thesis will examine the liquefaction potential of early age CPB under monotonic and cyclic loading conditions. In addition, the compression characteristics of CPB will be investigated. The new work will then be synthesized and recommendations made for design as well as for future research.
CHAPTER 4

CHARACTERISTICS OF SEISMIC EVENTS

4. Characteristics of Seismic Events

In addition to earthquake-induced liquefaction, blasting and rockburst are two other sources of dynamic loads that might occur in the vicinity of freshly placed CPB in a stope. Rockbursts and production blasts can be categorized as near-field or far-field seismic events depending on their distance to an underground CPB system. The stress wave induced by these dynamic loads might also result in the shear strength loss or liquefaction of CPB. Therefore, the response of CPB or the surrounding rock to these loads is of most importance for the purpose of geomechanical design.

To define an appropriate approach for evaluating the liquefaction potential of CPB in the field and the laboratory, the dynamic parameters of the stress waves, such as intensity, frequency and duration must be characterized. In other words, if a stope in proximity to a near-field production blast or a rockburst is exposed to dynamic loading similar (in terms of frequency, intensity, and duration) to conventional surface structures exposed to earthquake loading, then the general approach (i.e., simplified procedure) used in conventional geotechnical earthquake engineering might plausibly be adapted to geomechanical design of CPB systems.

To address the response of CPB systems or the surrounding rock to these dynamic loads, different data sets of seismic events were used in this study. Figure 4-1 schematically shows a backfilled stope with CPB exposed to a near-field production blast and a near-field rockburst as well as a far-field earthquake. However, the available recording stations could only provide
dynamic parameters for near-field production blasts as well as far-field rockbursts and earthquakes. These recording stations were the Canadian National Seismograph Network (CNSN) installed on the ground surface, the Strong Ground Motion (SGM) systems installed underground at the local mines in northern Ontario, and the accelerometers installed in a backfilled stope with CPB and its surrounding rock. Table 4-1 summarizes the type of seismic events recorded by these stations and used in this study. Although these data sets might provide remarkable information on the dynamic parameters of different seismic events, there is a lack of information on the near-field rockbursts, which cause the damage and instability in stopes. More details will be discussed in the following subsections.

Figure 4-1: Schematic of a CPB system exposed to different dynamic loads and the available recording stations. (Note: distances shown for earthquake and rockburst represent distances from epicentre, and not the depth).
Table 4-1: Available seismic events recorded by different stations.

<table>
<thead>
<tr>
<th>Stations</th>
<th>Natural Events Earthquake</th>
<th>Mining Events Rockburst</th>
<th>Mining Events Production Blast</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canadian National Seismograph Network</td>
<td>Far-field (100-400 km)</td>
<td>Far-field (22-400 km)</td>
<td>-</td>
</tr>
<tr>
<td>Strong Ground</td>
<td></td>
<td>Far-field (1-2 km)</td>
<td>Far-field</td>
</tr>
<tr>
<td>Motion System</td>
<td>-</td>
<td>-</td>
<td>Near-field (20-100 m)</td>
</tr>
<tr>
<td>Accelerometers</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Based on the available data sets, the starting point in this chapter is to investigate the characteristics of earthquakes in north eastern Ontario recorded by the CNSN. This provides a basis for comparing the dynamic parameters induced by earthquakes with those induced by production blasts and rockbursts. In addition, the “simplified procedure” practiced for evaluating the liquefaction resistance of soils induced by earthquakes does not explicitly account for the difference in the nature of earthquakes, as stated in Youd et al. (2001). The nature of earthquakes is quite different depending on the geo-structural situation and tectonics of the region. Therefore, the nature of earthquakes in this region will be discussed.

The characteristics of mining events (i.e., far-field rockbursts recorded by the CNSN) in this region will then be presented. Note that the waveforms of the earthquakes and mining events (i.e., far-field rockbursts) recorded by the CNSN are available in the National Waveform Archive (NWFA) on the Natural Resource Canada website (NRCan, 2009). All these earthquakes and rockburst events are named the NRCan events in this chapter.

The characteristics of far-field seismic events including production blasts and rockbursts recorded at two Ontario mines (i.e., the Williams Mine and the Kidd Mine) will then be compared with those of the NRCan events.

The characteristics of typical near-field production blasts at the Williams Mine will also be presented at the end of this chapter.

The dynamic parameters of far- and near-field seismic events provided in this chapter will help to assess the potential for extending the framework used for conventional geotechnical earthquake engineering of surface structures to the design of liquefaction assessment of CPB systems.
4.1. Seismic Events in North Eastern Ontario

To investigate the characteristics of earthquakes and mining events in north eastern Ontario, the waveforms of seismic events recorded by some stations of the CNSN were used. The specification of the seismograph and the stations used in this study will be presented in the following section followed by investigation of the characteristics of the events.

4.1.1. Seismographs in North Eastern Ontario

According to NRCan (2009), the CNSN contains over 100 high-gain instruments (i.e., seismographs) and over 60 low-gain or strong motion instruments (i.e., accelerographs). The high-gain instruments (i.e., seismographs) are used to record weak ground motion from natural earthquakes or mining events that are small or distant. However, low-gain accelerographs are used to record the strong ground motion from large earthquakes at nearby sites likely to cause damage. Table 4-2 presents the type of seismographs and the coordinates of those seismograph stations used in this study. The recording rate of the broadband seismographs is 40 (samples/sec) whereas that of high broadband and extremely short period seismographs is 100 (samples/sec). Note that the difference between high broadband and extremely short period seismographs is their seismometers, which are the part of the seismograph that sense ground movement. Seismometers convert ground motion vibrations into an electrical signal that in turn are converted into a digital signal (NRCan, 2009). It is worth noting that only the vertical component of a seismic wave is recorded at the extremely short period stations (i.e., EEO, GTO, SOLO, and TBO). All the seismographs are mounted on the bedrock; however, the bedrock might have different geological characteristics.

The most important specification of the seismographs is their response characteristics. The response information of each seismograph might be available in the form of “response curve” or “poles and zeros”. In either case, this response information is used to find a transfer function for converting the digital waveforms to an actual velocity time series. Figure 4-2 shows an example of the response curve for the vertical component of the KAPO station, showing the variation of the transfer function (based on counts/m/s) versus frequency (Hz). If the frequency of the digital waveform is within the flat part of the response curve, the corresponding transfer function can be directly used to convert the digital waveform to the actual velocity time series. However, if the frequency is not in the flat part of the response curve, corrections need to be made for the
conversion. More details on the response information of each station and the conversion of NRCan seismic waveform data to actual time series of velocity is presented in Appendix A.

Table 4-2: Type and coordinates of the CNSN stations.

<table>
<thead>
<tr>
<th>Station Code</th>
<th>Station Type</th>
<th>Recording Rate (samples/sec)</th>
<th>Seismometer</th>
<th>Latitude °N</th>
<th>Longitude °W</th>
<th>Elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EEO</td>
<td>Extremely Short Period-Vertical</td>
<td>100</td>
<td>S-13</td>
<td>46.64</td>
<td>-79.07</td>
<td>398</td>
</tr>
<tr>
<td>GTO</td>
<td>Extremely Short Period-Vertical</td>
<td>100</td>
<td>S-13</td>
<td>49.74</td>
<td>-86.96</td>
<td>350</td>
</tr>
<tr>
<td>KAPO</td>
<td>Broadband</td>
<td>40</td>
<td>3ESP</td>
<td>49.45</td>
<td>-82.51</td>
<td>210</td>
</tr>
<tr>
<td>KILO</td>
<td>High Broadband</td>
<td>100</td>
<td>3ESP</td>
<td>48.50</td>
<td>-79.72</td>
<td>314</td>
</tr>
<tr>
<td>SADO</td>
<td>Broadband</td>
<td>40</td>
<td>3ESP</td>
<td>44.77</td>
<td>-79.41</td>
<td>243</td>
</tr>
<tr>
<td>SOLO</td>
<td>Extremely Short Period-Vertical</td>
<td>100</td>
<td>S-13</td>
<td>50.02</td>
<td>-92.08</td>
<td>373</td>
</tr>
<tr>
<td>SUNO</td>
<td>High Broadband</td>
<td>100</td>
<td>3ESP</td>
<td>46.64</td>
<td>-81.34</td>
<td>343</td>
</tr>
<tr>
<td>TBO</td>
<td>Extremely Short Period-Vertical</td>
<td>100</td>
<td>S-13</td>
<td>48.65</td>
<td>-89.41</td>
<td>475</td>
</tr>
<tr>
<td>TIMO</td>
<td>High Broadband</td>
<td>100</td>
<td>3ESP</td>
<td>48.46</td>
<td>-81.30</td>
<td>392</td>
</tr>
</tbody>
</table>

Figure 4-2: Response curve of BHN for the KAPO station (from NRC, 2009).
Note that the frequency of the waveform data used in this study was within the flat part of the response curve of each station, indicating no correction was required.

4.1.2. Natural Earthquakes in North Eastern Ontario

North eastern Ontario has a very low level of seismic activity. According to NRCan, only 1 or 2 magnitude 2.5 or greater earthquakes have been recorded in this large area from 1970 to 1999 (Figure 4-3). The 1905 northern Michigan and the 1928 Kapuskasing earthquakes are two magnitude 5 events that occurred in this region. The data related to these events are not available. However, the NWFA of NRCan offers data sets of earthquakes in Canada since 1985.

![Map of North Eastern Ontario seismic zone](#)

**Figure 4-3: North eastern Ontario seismic zone (from NRCan, 2009).**

Figure 4-4 shows the location of seven magnitude 3.5 or greater earthquakes between 1985 and 2009. Among these events, the three largest earthquakes whose data were available for the specific stations in the NWFA will be investigated in this study. Table 4-3 presents the general information about these three earthquakes as well as the 1988 Saguenay earthquake (Quebec),
which is one of the historical events in eastern Canada. As presented in this table, the focus of earthquakes which occurred in this region is quite deep and the magnitudes are not very high. Note that the magnitude of earthquakes is usually given in moment magnitude while the magnitudes in this chapter are based on Nuttli magnitude, \( m_N \) (defined in Section 3.3.1).

**Figure 4-4:** 7 magnitude > 3.5 earthquakes in North eastern Ontario between 1985 and 2009.

**Table 4-3:** Strong earthquakes in north eastern Ontario (from NRCan, 2009).

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Latitude °N</th>
<th>Longitude °W</th>
<th>Depth (km)</th>
<th>Magnitude (m_N)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>2006/12/07</td>
<td>04:44:59</td>
<td>49.51</td>
<td>-81.53</td>
<td>16</td>
<td>4.2</td>
<td>62 km NW from Cochrane</td>
</tr>
<tr>
<td>2006/01/03</td>
<td>11:05:10</td>
<td>49.33</td>
<td>-81.09</td>
<td>22</td>
<td>3.7</td>
<td>30 km N from Cochrane</td>
</tr>
<tr>
<td>2005/10/20</td>
<td>21:16:28</td>
<td>44.68</td>
<td>-80.48</td>
<td>11</td>
<td>4.3</td>
<td>Felt in Owen Sound</td>
</tr>
<tr>
<td>1988/11/25</td>
<td>23:46:00</td>
<td>48.12</td>
<td>-71.18</td>
<td>29</td>
<td>6.4</td>
<td>34 km SW from LA BAIE</td>
</tr>
</tbody>
</table>
The acceleration time series of the 2006/12/07 earthquake with a relatively low 4.2 mN are shown in Figure 4-5 as the typical earthquakes in Northern Ontario. These traces are shown in a 30 second window for three orthogonal components north (N), east (E) and vertical (z) recorded at the KAPO station, 71 km away from the epicentre. The trace of each component shows both P-wave and the combination of S-wave and surface waves (i.e., Rayleigh wave) although it seems that the combination of S-wave and surface waves appears dominant. Note that the differentiation between S-wave and Rayleigh wave is not possible for these time series. Component N of this earthquake shows the strongest ground motion with an absolute value of 0.00137g as the peak particle acceleration. The frequency spectra of total acceleration time series (i.e., P- and S-waves between 0 and 30 sec time window) for these three components show the three maximum peaks at frequencies between 5 and 16 Hz, as shown in Figure 4-6.

4.1.3. Mining Related Events in North Eastern Ontario

The NWFA of NRCan also offers data sets of mining events (i.e., rockbursts) in Canada since 1985. Figure 4-7 shows the location of five magnitude 3.5 or greater earthquakes between 1985 and 2009 in north eastern Ontario. Among these events, the three largest mining related events whose data were available for the specific stations in the NWFA will be investigated in this study. Table 4-4 presents the general information about these three mining events. The magnitude of these events is between 3.5 mN and 4.1 mN, which is similar to that of typical natural earthquakes in this region. Unlike the earthquakes, these events are relatively close to the ground surface (1-2.5 km). However, the epicentral distances are far from the recording stations, as shown in Table 4-2. As previously mentioned, these events are categorized as far field events.

Table 4-4: Strong mining events in north eastern Ontario.

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Latitude °N</th>
<th>Longitude °W</th>
<th>Depth (km)</th>
<th>Magnitude (mN)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>2006/11/29</td>
<td>07:22:55</td>
<td>46.48</td>
<td>-81.18</td>
<td>2.5</td>
<td>4.1</td>
<td>Rockburst Sudbury</td>
</tr>
<tr>
<td>2004/07/15</td>
<td>06:29:38</td>
<td>46.49</td>
<td>-81.16</td>
<td>1</td>
<td>3.5</td>
<td>Rockburst Sudbury</td>
</tr>
<tr>
<td>2003/09/13</td>
<td>18:25:56</td>
<td>48.69</td>
<td>-85.90</td>
<td>1</td>
<td>3.5</td>
<td>Rockburst Williams Mine</td>
</tr>
</tbody>
</table>
Figure 4-5: Three components of the trace of the 2006/12/07 earthquake with a magnitude of 4.2 Mn recorded at KAPO station. (a) Component N; (b) component E; (c) component Z.
Figure 4-6: Frequency spectra of the acceleration for the 2006/12/07 earthquake with a magnitude of 4.2 Mn recorded at KAPO station. (a) Component N; (b) component E; (c) component Z.
Figure 4-7: Five magnitude > 3.5 mining events in north eastern Ontario between 1985 and 2009.

The acceleration time series of the 2006/11/29 mining event with a magnitude of 4.1 mN are shown in Figure 4-8 as the typical seismic event induced by a rockburst in this region. These traces are shown in a 10 second window for three orthogonal components north (N), east (E) and vertical (z) recorded at the SUNO station, 22.1 km away from the epicentre. Component E of this event shows the strongest ground motion with an absolute value of 0.0027g as the peak particle acceleration. Figure 4-9 shows the frequency spectra of total acceleration time series for these three components between 0 and 10 sec. There are two significant peak amplitudes for component N at 3 Hz and 9 Hz. However, the maximum amplitudes at ~3.5 Hz are dominant for components E and Z and the second peaks are corresponding to 10 Hz. Regardless of the range of frequency, there is a difference in frequency distribution between spectra of this event and those of the 2006/12/07 earthquake shown in Figure 4-6. The spectra of the 2006/12/07 are approximately normal distributions whereas the spectra of the 2006/11/29 mining event (i.e.,
rockburst) are positive skewed distributions. Note that the frequency range of the 2006/11/29 mining event is lower than that of the earthquake. This is opposite to the expectation since the frequency of mining events, which have shorter source length than earthquakes, should contain more high frequency content. The reason for this anomaly is not known at present.

To ensure the time windows used in this analysis have no effect on the spectra of these two events, the spectra of high amplitude sections (i.e., 8-13 sec window of Figure 4-5 for the 2006/12/07 earthquake and 3-6 sec window of Figure 4-8 for the 2006/11/29 mining event) of these events were also obtained. The result showed that the two time windows give almost identical spectra for the mining event. The comparison between the short and long time window for the earthquake records appear to have significant effect on the amplitude of the respective spectra, but nothing significant enough to change the frequency content. The frequency spectra of these two events with short time windows are presented at the end of Appendix B.

4.1.4. The Earthquakes versus the Rockbursts in Northern Ontario

Peak particle acceleration (PPA), peak particle velocity (PPV) and the corresponding frequency of the earthquakes and the rockbursts will be compared in this section. Table 4-5 presents the summary of the PPA of the NRCan seismic events including earthquakes and rockbursts with respect to the available components for the recording stations of the CNSN. Figure 4-10 shows the PPA versus the hypocentral distance of all the components of the seismic events. Note that the result of the 2005 earthquake is not presented in this figure because the PPA’s for this event are extremely low and cannot be presented in the same scale. Generally, PPA increases as the distance decreases. In addition, PPA increases as the magnitude of the events increases at a specific distance.

A good statistical fit to the two 3.5 mN mining events was obtained, as shown in this figure. This line shows the lower limit of PPA for these events. The PPA of the 1988 Saguenay earthquake shows the highest value corresponding to the high 6.4 mN event, which is significantly higher than the other events. This result indicates that the variation of PPA versus distance (22-470 km from the source) for the earthquakes within the range of 3.7 - 4.2 mN is approximately similar to that for the rockbursts with the magnitude ranging from 3.5 to 4.1 mN.
Figure 4-8: Three components of the trace of the 2006/11/29 mining event with a magnitude of 4.1 m$_N$ recorded at SUNO station. (a) Component N; (b) component E; (c) component Z.
Figure 4-9: Frequency spectra of the acceleration for the 2006/11/29 mining event with a magnitude of 4.1 mN recorded at SUNO station. (a) Component N; (b) component E; (c) component Z.
Table 4.5: PPA and the equivalent number of cycles for the strongest component of the NRC events.

<table>
<thead>
<tr>
<th>Event</th>
<th>Station</th>
<th>Component</th>
<th>Hypocentral Distance</th>
<th>PPA</th>
<th>Equivalent No. of Cycles</th>
<th>Depth</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>km</td>
<td>m/s²</td>
<td></td>
<td>km</td>
<td>m_N</td>
</tr>
<tr>
<td>Williams Rockburst</td>
<td>GTO</td>
<td>Z</td>
<td>140.3</td>
<td>2.26E-04</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td>20030913</td>
<td></td>
<td>E</td>
<td>261.0</td>
<td>1.51E-04</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>KAPO</td>
<td>Z</td>
<td>261.0</td>
<td>2.37E-04</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>SOLO</td>
<td>Z</td>
<td>471.1</td>
<td>2.82E-05</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>TBO</td>
<td>Z</td>
<td>257.5</td>
<td>7.82E-05</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td>Mining Event</td>
<td>EEO</td>
<td>Z</td>
<td>162.0</td>
<td>3.58E-03</td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
<tr>
<td>200601129</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.1</td>
</tr>
<tr>
<td>Time:07:19:54</td>
<td>SUNO</td>
<td>N</td>
<td>22.0</td>
<td>2.70E-02</td>
<td></td>
<td>6.3</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>22.0</td>
<td>2.69E-02</td>
<td></td>
<td></td>
<td>2.5</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>Z</td>
<td>22.0</td>
<td>3.31E-02</td>
<td></td>
<td></td>
<td>2.5</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>N</td>
<td>221.0</td>
<td>9.95E-04</td>
<td></td>
<td>4</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>221.0</td>
<td>1.03E-03</td>
<td></td>
<td>2.5</td>
<td>4.1</td>
</tr>
<tr>
<td>Mining Event</td>
<td>EEO</td>
<td>Z</td>
<td>160.3</td>
<td>6.40E-04</td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>20040715</td>
<td>SADO</td>
<td>N</td>
<td>247.3</td>
<td>3.45E-04</td>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>Time:06:29</td>
<td></td>
<td>E</td>
<td>247.3</td>
<td>2.11E-04</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>247.3</td>
<td>1.72E-04</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>N</td>
<td>22.1</td>
<td>1.62E-02</td>
<td></td>
<td>2.5</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>22.1</td>
<td>9.33E-03</td>
<td></td>
<td>1</td>
<td>3.5</td>
</tr>
<tr>
<td>Earthquake</td>
<td>KAPO</td>
<td>E</td>
<td>72.8</td>
<td>4.36E-03</td>
<td></td>
<td>16</td>
<td>4.2</td>
</tr>
<tr>
<td>20061207</td>
<td>N</td>
<td>72.8</td>
<td>1.34E-02</td>
<td></td>
<td>16</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td>Time:04:14:48</td>
<td>Z</td>
<td>72.8</td>
<td>1.24E-02</td>
<td></td>
<td>16</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>E</td>
<td>118.1</td>
<td>8.43E-03</td>
<td></td>
<td>16</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>118.1</td>
<td>9.36E-03</td>
<td></td>
<td>15.9</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>118.1</td>
<td>7.22E-03</td>
<td></td>
<td>16</td>
<td>4.2</td>
</tr>
<tr>
<td>Earthquake</td>
<td>KAPO</td>
<td>E</td>
<td>105.7</td>
<td>2.63E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
</tr>
<tr>
<td>20060103</td>
<td>N</td>
<td>105.7</td>
<td>6.01E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>Time:11:04</td>
<td>Z</td>
<td>105.7</td>
<td>4.65E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>KILO</td>
<td>E</td>
<td>137.9</td>
<td>1.21E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>137.9</td>
<td>2.50E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>137.9</td>
<td>8.91E-04</td>
<td></td>
<td>22</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>N</td>
<td>99.8</td>
<td>4.76E-03</td>
<td></td>
<td>26.3</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>99.8</td>
<td>2.86E-03</td>
<td></td>
<td>22</td>
<td>3.7</td>
</tr>
<tr>
<td>Earthquake</td>
<td>EEO</td>
<td>Z</td>
<td>244.0</td>
<td>1.61E-13</td>
<td></td>
<td>11</td>
<td>4.3</td>
</tr>
<tr>
<td>20050120</td>
<td></td>
<td>E</td>
<td>106.7</td>
<td>1.70E-12</td>
<td></td>
<td>11</td>
<td>4.3</td>
</tr>
<tr>
<td>Time:21:16:06</td>
<td></td>
<td>Z</td>
<td>106.7</td>
<td>1.45E-12</td>
<td></td>
<td>11</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td>SADO</td>
<td>N</td>
<td>228.6</td>
<td>4.67E-12</td>
<td></td>
<td>11</td>
<td>4.3</td>
</tr>
<tr>
<td>1988 Saguenay</td>
<td></td>
<td>Z</td>
<td>228.6</td>
<td>2.64E-12</td>
<td></td>
<td>11</td>
<td>4.3</td>
</tr>
<tr>
<td>Earthquake</td>
<td>S16</td>
<td>T</td>
<td>51.9</td>
<td>1.2753</td>
<td></td>
<td>16.2</td>
<td>29</td>
</tr>
<tr>
<td>1988 Saguenay</td>
<td></td>
<td>R</td>
<td>51.9</td>
<td>1.0448</td>
<td></td>
<td>19.2</td>
<td>29</td>
</tr>
<tr>
<td>Earthquake</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note: Depth and Magnitude values are approximate and may vary depending on the specific event.*
Figure 4-10: PPA versus hypocentral distance for the all components of the NRCan events (curve fitting limited to the 3.5 mN events).
Table 4-6 also presents the PPV and their corresponding frequency for the same NRCan events. In this table, there are two types of frequency, the frequency corresponding to the maximum amplitude of velocity and the frequency determined from FFT spectra obtained from velocity time series. Note that the frequency corresponding to the maximum amplitude is the same as the frequency used to obtain the transfer function from the response curve for each station, as explained in Appendix A. The frequency obtained from FFT spectra might have two or three significant peaks. Depending on the spectral distribution, an average frequency was determined for the peaks with amplitudes higher than 85% of the maximum amplitude. The first two average frequencies for velocity are shown in this table. The frequency spectra for all the NRCan events can be found in Appendix B.

Figure 4-11 shows the variation of the frequency corresponding to the maximum velocity and the frequency obtained from FFT versus the hypocentral distance of all the components of the same seismic events. Regardless of the magnitude of the events, the frequency corresponding to the maximum velocity of both the earthquakes and rockbursts show their dependency on distance. Generally, the frequency increases as the distance from the focus of the seismic event decreases. Frequency changes from a minimum of 3 Hz at 500 km to a maximum of 16 Hz at 22 km from the source. In contrast, the frequency obtained from FFT is more scattered although showing similar trend for the first peak. The correlation coefficient between the frequency corresponding to the maximum velocity and the first peak of FFT is about 0.8.

The frequency of the 1988 Saguenay earthquake with a high magnitude also is within the ballpark of the other events. This result also indicates that the frequency of the far-field rockbursts are within the same range as the frequency of the earthquakes in north eastern Ontario and does not exceed 20 Hz. This is partly due to intervening rock, and not due to difference in source between these two types of event.

Since the variation in frequency of the far-field rockbursts is similar to that of the earthquakes, the simplified procedure used in conventional geotechnical earthquake engineering might plausibly be applied to geomechanical design of CPB systems if the far-field seismic events are of concerns. Therefore, the equivalent numbers of uniform cycles corresponding to the seismic event magnitude can be obtained for suitably performing related laboratory experiments.
Table 4-6: PPV and the corresponding frequency for all components of the NRC events.

<table>
<thead>
<tr>
<th>Event</th>
<th>Station</th>
<th>Component</th>
<th>Hypocentral Distance</th>
<th>PPV</th>
<th>Average Frequency based on FFT Hz</th>
<th>Frequency Corresp. to Maximum Velocity Hz</th>
<th>First</th>
<th>Second</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Williams Rockburst 20030913</strong></td>
<td>GTO</td>
<td>Z</td>
<td>140.3</td>
<td>6.55E-06</td>
<td>5.5</td>
<td>4</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>KAPO</td>
<td>E</td>
<td>261.0</td>
<td>4.81E-06</td>
<td>5</td>
<td>3.5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>261.0</td>
<td>9.45E-06</td>
<td>4</td>
<td>4</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>261.0</td>
<td>5.76E-06</td>
<td>5</td>
<td>4.5</td>
<td></td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>SOLO</td>
<td>Z</td>
<td>471.1</td>
<td>1.44E-06</td>
<td>3.125</td>
<td>2.5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>TBO</td>
<td>Z</td>
<td>257.5</td>
<td>2.49E-06</td>
<td>5</td>
<td>2.7</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td><strong>Mining Event 20061129 Time:07:19:54</strong></td>
<td>EEO</td>
<td>Z</td>
<td>162.0</td>
<td>9.12E-05</td>
<td>6.25</td>
<td>5</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SUNO</td>
<td>E</td>
<td>22.0</td>
<td>4.30E-04</td>
<td>10</td>
<td>4.5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>22.0</td>
<td>2.55E-04</td>
<td>12.5</td>
<td>8.8</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>22.0</td>
<td>2.00E-04</td>
<td>12.4</td>
<td>4</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>N</td>
<td>221.0</td>
<td>2.88E-05</td>
<td>5.5</td>
<td>5</td>
<td>3</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>221.0</td>
<td>2.88E-05</td>
<td>6.25</td>
<td>5.5</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td><strong>Mining Event 20040715 Time:06:29</strong></td>
<td>EEO</td>
<td>Z</td>
<td>160.3</td>
<td>1.23E-05</td>
<td>8.3</td>
<td>5.5</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SADO</td>
<td>E</td>
<td>247.3</td>
<td>8.32E-06</td>
<td>6.6</td>
<td>3.5</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>247.3</td>
<td>5.09E-06</td>
<td>6.6</td>
<td>4</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>247.3</td>
<td>5.48E-06</td>
<td>6.6</td>
<td>3</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>SUNO</td>
<td>E</td>
<td>22.1</td>
<td>1.56E-04</td>
<td>16.5</td>
<td>15</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>22.1</td>
<td>7.90E-05</td>
<td>16.6</td>
<td>16</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>22.1</td>
<td>8.95E-05</td>
<td>16.4</td>
<td>15</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td><strong>Earthquake 20061207 Time:04:41:48</strong></td>
<td>KAPO</td>
<td>E</td>
<td>72.8</td>
<td>1.05E-04</td>
<td>6.6</td>
<td>12</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>72.8</td>
<td>2.19E-04</td>
<td>10</td>
<td>11</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>72.8</td>
<td>1.98E-04</td>
<td>10</td>
<td>11</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>E</td>
<td>118.1</td>
<td>8.09E-05</td>
<td>16.6</td>
<td>11</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>118.1</td>
<td>8.98E-05</td>
<td>16.6</td>
<td>10</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>118.1</td>
<td>4.37E-05</td>
<td>12.5</td>
<td>13.5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td><strong>Earthquake 20060103 Time:11:04</strong></td>
<td>KAPO</td>
<td>E</td>
<td>105.7</td>
<td>4.19E-05</td>
<td>10</td>
<td>9.5</td>
<td>6.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>105.7</td>
<td>9.57E-05</td>
<td>10</td>
<td>8.5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>105.7</td>
<td>7.40E-05</td>
<td>10</td>
<td>10</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>KILO</td>
<td>E</td>
<td>137.9</td>
<td>2.33E-05</td>
<td>8.3</td>
<td>6</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>137.9</td>
<td>2.39E-05</td>
<td>16.6</td>
<td>12.5</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>137.9</td>
<td>1.71E-05</td>
<td>8.3</td>
<td>8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TIMO</td>
<td>N</td>
<td>99.8</td>
<td>4.57E-05</td>
<td>16.6</td>
<td>12.5</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>99.8</td>
<td>2.74E-05</td>
<td>16.6</td>
<td>8.8</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td><strong>Earthquake 20051020 Time:21:16:06</strong></td>
<td>EEO</td>
<td>Z</td>
<td>244.0</td>
<td>3.28E-05</td>
<td>6.25</td>
<td>5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>SADO</td>
<td>E</td>
<td>106.7</td>
<td>6.23E-05</td>
<td>10</td>
<td>10.5</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>106.7</td>
<td>1.15E-04</td>
<td>10</td>
<td>10.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>106.7</td>
<td>5.33E-05</td>
<td>10</td>
<td>9</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SUNO</td>
<td>E</td>
<td>228.6</td>
<td>5.84E-05</td>
<td>10</td>
<td>7.5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>228.6</td>
<td>3.97E-05</td>
<td>10</td>
<td>7</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>228.6</td>
<td>3.99E-05</td>
<td>8.33</td>
<td>5</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td><strong>1988 Saguenay Earthquake</strong></td>
<td>S16</td>
<td>T</td>
<td>51.9</td>
<td>2.52E-02</td>
<td>9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>S16</td>
<td>R</td>
<td>51.9</td>
<td>1.0448</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 4-11: Frequency versus hypocentral distance for the all components of the NRCan events.
Figure 4-12 shows the equivalent numbers of uniform cycles at 0.65$\tau_{\text{max}}$ for the strongest horizontal component of these seismic events along with the original data set based on the procedure proposed by Seed et al. (1975). In this procedure, the irregular time series of velocity or stress is converted to uniform numbers of cycles by the weighting method. The equivalent numbers of cycles can then be determined at any percentage of maximum shear stress. 0.65$\tau_{\text{max}}$ is conventionally used in geotechnical earthquake engineering. Note that the strongest horizontal component is also used for determining the maximum shear stress, $\tau_{\text{max}}$, in the field, as explained in Youd et al. (2001).

According to Seed et al. (1975), the equivalent numbers of uniform cycles increase as magnitude increases. As shown in Figure 4-12, the original data points did not extend down to 3.5 m$\text{N}$, but the trend line flattened at a low end with magnitude 5.5. A comparison between data points obtained for the NRCan events and the original data points obtained by Seed et al. (1975) shows that the equivalent numbers of cycles for six NRCan records are within the range of the low end events in the original trend line (i.e., < 7 cycles). There are 5 other NRCan records including the 1988 Saguenay earthquake that show higher numbers of cycles with regards to their magnitudes. These data points might be considered as outliers. Generally, the data obtained by this procedure are scattered and do not show any trends. Note that the original data points are also scattered with a $R^2$ value of about 0.68. In addition, the weighting method suggested by Seed et al. (1975) is based on natural earthquakes occurred in the west of North America that might not be appropriate for earthquakes in eastern part due to geological and geo-structural differences.

### 4.2. Mining Seismic Events Recorded at Mines

The dynamic parameters including peak particle velocity and frequency content of seismic events recorded at the Williams Mine and the Kidd Mine will be investigated in this section. As presented in Table 4-7, the seismic events include rockbursts and production blasts with magnitudes ($m_{\text{N}}$) range between 2.7 and 3.8. The magnitudes of these events are similar to the NRCan events presented in the previous section. However, the distance of the recording stations from the source for these events is closer, ranging between 1.37- 2 km. Although the distances are closer to the source in comparison with the NRCan events, these events are still categorized as far-field events.
Figure 4-12: Magnitude versus the equivalent numbers of uniform stress cycles at $0.65\tau_{\text{max}}$ for strongest components of the NRCan events along with.
To record seismic events in these mines, a strong ground motion (SGM) system, installed in underground, is used. The SGM system consists of triaxial geophones sensitive in the range of 4.5-500Hz. The SGM system is capable of recording large seismic events that occur in the mine from moment magnitude around 0 up to 4-5 without clipping. Therefore the true low frequency ground velocities are measured that occurred from the large events. In the following sections the characteristics of rockburst at the Williams Mine and the Kidd Mine will be presented first. The characteristics of production blast at Kidd mine will then be presented. These events will then be compared with the NRCan events.

4.2.1. Rockbursts

Figure 4-13 shows the three rotated components of velocity time series for the January rockburst event at the Williams Mine. The original coordinate system was based on three orthogonal components E, N, and Z while the rotated coordinate system showing the velocity along the raypath, P wave, and two perpendicular axes to the raypath axis, SV and SH. SV and SH show the shear waves of the seismic event. Component SV for this event shows the strongest ground motion with an absolute PPV of 4.78 mm/sec. A frequency spectrum was obtained from the velocity time series for the strongest component, SV, between zero and 781 (msec) time window. The predominant frequency for this component ranges between 10 and 14 Hz, as shown in Figure 4-14 (see Appendix B.8 for the typical FFT of acceleration). The 3D component of this rockburst shows that the PPV equals to 5.03 mm/s. Figure 4-15 shows the three rotated components of velocity time series for the January rockburst event at the Kidd Mine. Component SH shows the strongest ground motion with an absolute PPV of 74 mm/sec. The absolute PPV for component SV of this event is 54 mm/sec. The predominant frequencies for components SH and SV obtained from the velocity time series between zero and 1916 msec time window are 7.3 -12.5 Hz, and 5.2-6.7 Hz, respectively. The frequency spectrum for component SV is shown in Figure 4-16.
Figure 4-13: Three components of velocity time series of the rockburst event in January 2009 at the Williams Mine.

Figure 4-14: FFT of component SV for velocity time series of the rockburst event in January at the Williams Mine.
Figure 4-15: Three components of velocity time series of the rockburst event in January 6, 2009 at the Kidd Mine.

Figure 4-16: FFT of component SV for velocity time series of the rockburst event in January at the Kidd Mine.
In another instance, the three rotated components of velocity time series of the June rockburst at the Kidd Mine is shown in Figure 4-17. The PPV is 33 mm/sec for the strongest component, SH and 18 mm/sec for component SV. The predominant frequency of velocity for component SV is 18.9 Hz, as shown in Figure 4-18. Other frequencies might also be identified for this component (~ 9 Hz and 26 Hz).

Generally, the frequency distribution for these three rockbursts is different from each other. On the other hand, the frequency content for velocity varies from 5 Hz and 25 Hz. Table 4-8 summarizes the PPV and frequency of each event in accordance with the components. A comparison between the frequencies of components SV and SH for the two rockbursts at the Kidd Mine with a similar average distance of 2 km from the source shows that the maximum frequency for the June rockburst (18.9 and 25 Hz) is more than 2 times of the maximum frequency for the January rockburst (6.7 and 12.5 Hz). This might be considered somewhat anomalies although the frequency for all these events remains less than 25 Hz.

Table 4-8: Summary for PPV and frequency content of rockbursts recorded at the mines.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Date</th>
<th>Component</th>
<th>PPV m/sec</th>
<th>Magnitude mN</th>
<th>Frequency for Velocity Hz</th>
<th>Distance m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Williams Mine</td>
<td>Jan-15</td>
<td>P</td>
<td>0.0029</td>
<td>3.1</td>
<td></td>
<td>1371</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV</td>
<td>0.00478</td>
<td>3.1</td>
<td>10.2-14</td>
<td>1371</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH</td>
<td>0.00129</td>
<td>3.1</td>
<td>-</td>
<td>1371</td>
</tr>
<tr>
<td>Kidd Mine</td>
<td>Jan-06</td>
<td>P</td>
<td>0.02</td>
<td>3.8</td>
<td>5.2-6.7</td>
<td>1965</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV</td>
<td>0.054</td>
<td>3.8</td>
<td>7.3-12.5</td>
<td>1965</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH</td>
<td>0.074</td>
<td>3.8</td>
<td>15-21.5</td>
<td>1965</td>
</tr>
<tr>
<td></td>
<td>Jun-15</td>
<td>P</td>
<td>0.016</td>
<td>3.1</td>
<td>9-18.9</td>
<td>2066</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV</td>
<td>0.018</td>
<td>3.1</td>
<td>21.5-25</td>
<td>2066</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH</td>
<td>0.033</td>
<td>3.1</td>
<td></td>
<td>2066</td>
</tr>
</tbody>
</table>

The correlation between the magnitude of each rockburst and its distance from the source with respect to the PPV of the strongest component shows that the results of the Kidd Mine are in good agreement with the anticipated levels of ground motion from data in the Rockburst Handbook (Kaiser et al. 1995). However, the PPV of the Williams rockburst is lower than the level of ground motion suggested in the handbook, as shown in Figure 4-19. According to this chart, these rockburst events are categorized as the moderate level of ground motion.
Figure 4-17: Three components of velocity time series of the rockburst event in June 15, 2009 at the Kidd Mine.

Figure 4-18: FFT of component SV for velocity time series of the rockburst event in June at the Kidd Mine.
4.2.2. Production Blasts

The dynamic parameters of two production blasts recorded about 2 km away from two stopes at the Kidd Mine will be presented in this section. The recording system is the same SGM system used for the rockbursts. Figure 4-20 shows the three un-rotated components (E, N, and Z) of the velocity time series for the production blast at stope 70-867. The S-waves are dominant for all components and component N shows the strongest ground motion with a PPV of 8.1 mm/sec. The predominant frequency for component E of the production blast at stope 70-867 is about 33.8 Hz, as shown in Figure 4-21. Note that the magnitude of this event is 2.7 mN.

For the production blast at stope 71-862 with a magnitude of 3.4 mN, the PPV of the strongest component, N, is about 1.8 m/sec. The three un-rotated components of velocity for this event are shown in Figure 4-22. The predominant frequency of the strongest component (i.e., N) for velocity is about 40.6 Hz, as shown in Figure 4-23. Table 4-9 summarizes the PPV for all the components and the frequency of these two events.
Figure 4-20: Three components of velocity time series of the production blast in stope 70-867 at the Kidd Mine.

Figure 4-21: FFT of component E for velocity time series of the production blast in stope 70-867 at the Kidd Mine.
Figure 4-22: Three components of the velocity time series of the production blast in stope 71-862 at the Kidd Mine.

Figure 4-23: FFT of component N for velocity time series of the production blast in stope 71-862 at the Kidd Mine.
Table 4-9: Summary for PPV and frequency content of the production blasts recorded at the Kidd Mine.

<table>
<thead>
<tr>
<th>Mine</th>
<th>Stope</th>
<th>Component</th>
<th>PPV m/s</th>
<th>Magnitude Mn</th>
<th>frequency for Velocity Hz</th>
<th>Distance km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kidd Creek</td>
<td>70-867</td>
<td>E</td>
<td>0.0056</td>
<td>2.7</td>
<td>33.8</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>0.0081</td>
<td>2.7</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>0.0005</td>
<td>2.7</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>71-862</td>
<td>E</td>
<td>0.0016</td>
<td>3.4</td>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>0.0018</td>
<td>3.4</td>
<td>40.6</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Z</td>
<td>0.0015</td>
<td>3.4</td>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

4.2.3. Seismic Events Recorded at the Mines versus the NRCan Events

In the preceding subsections, a number of far-field seismic events (rockbursts and blasting events) recorded at the mines were presented. The dynamic parameters of these events will be compared with those of the NRCan seismic events in this section.

The PPV versus distance of seismic events recorded at the mines and the NRCan events are shown in Figure 4-24. As previously shown for the acceleration, the PPV’s of the NRCan events are also consistent with the magnitude and distance of the events. A good statistical fit was obtained for two NRCan rockburst events with a low magnitude of 3.5 mN. Another statistical fit was obtained for two NRCan earthquakes with high magnitudes of 4.2 and 4.3 mN. These two lines provide a lower limit and an upper limit of PPV for the rockbursts and earthquakes within these magnitudes and the distances higher than 22 km, respectively.

In contrast, the PPV of the seismic events recorded at the mine sites are scattered. Note that all the components of these events are shown in this figure. The extension of upper and lower limit lines shows that the PPV of two rockbursts recorded at Kidd mine are beyond these two lines while the magnitude of these events are similar to the NRCan events. However, the PPV of the rockburst recorded at the Williams Mine is within the upper- and lower-limit of the NRCan events with respect to the distance. Note that the PPV of the rockburst at the Williams Mine is lower than the level of ground motion suggested by Rockburst Handbook, as shown in Figure 4-19.
Figure 4-24: PPV versus distance for the seismic events recorded at the mines in comparison with the NRCan events.
Figure 4-25: Frequency versus distance for the seismic events recorded at the mines in comparison with the NRCan events.
The 2.7 magnitude production blast has a higher PPV than the 3.4 magnitude production blast, while the distances from the source are the same (the magnitudes of these events have been determined by the mines). However, the PPV’s of these events are about the upper and lower limits of the NRCan events. Figure 4-25 shows the frequency content for velocity of the seismic events recorded at the mines and the NRCan events. Generally, the predominant frequencies of the strongest component for the rockbursts recorded at the mines are slightly higher than those of the NRCan events. This is due to a closer distance of the rockbursts recorded at the mines. However, the variation in frequency for all the components of these rockbursts (5-25 Hz) is similar to the variation in frequency for the NRCan events (3-16 Hz). In contrast, the production blasts show higher frequencies for velocity (33-40 Hz).

The results of the seismic events recorded at these two mines indicate that the PPV of the events with respect to their distances are similar to that of the earthquakes in northern Ontario. Note that the S-waves are dominant for the rockbursts recorded at the mines while both S-waves and surface waves are involved in the case of earthquakes and rockbursts recorded on the surface.

Although the maximum frequency of the rockburst events recorded at a distance of 1.3-2 km away from the source is slightly higher than that of the rockbursts and earthquakes at greater distances (Max 25 Hz vs. Max 16 Hz, respectively), the general variations in frequency are the same. Therefore, it is possible to use the conventional approach in geotechnical engineering when dealing with rockbursts at even a lower distance of ~1.3-2 km from source.

In contrast, the average frequency of the production blasts (33-40 Hz) recorded at a distance of 2 km away from the source is about twice the maximum frequency of the NRCan events. Since the effect of frequency within the range of earthquakes is considered negligible in geotechnical engineering and the frequency of the production blasts is significantly higher than this range, it is unlikely to apply the conventional approach for geomechanical design. Note that this conclusion is based on the data presented for these two mines and should not be generalized for all the cases.

The following section presents some near-field (less than 200 m) results of a production blasts at the Williams Mine to determine similarities and differences of the near-field blasting with far-field seismic events.
4.3. Near-Field Blasting Events

4.3.1. Overview and Instrumentation

The response of freshly placed CPB in a stope and its surrounding rocks to production blasts was investigated as part of the paste project at the Williams Mine. An Alimak stope with 149 m height (dipping 70 degree North), 5 m width (from hanging wall to footwall), and 30 m length was considered in this project. The featured stope was stope # 55 where its over-cut was in level 9555 and its under-cut was in level 9415, as shown in Figure 4-26.

Prior to filling the stope with CPB, the following instrumentation was placed inside the stope for different purposes: i) to monitor the vibration level during blasting two accelerometers, (i.e., #9 and #10 with ranges of ±50g and ± 5g, respectively) were used in CPB. Note that accelerometer # 9 includes only one component (L) while accelerometer # 10 includes all three components of vertical (V), longitudinal (L) and transversal (T); ii) to measure the vertical and horizontal stresses, four total earth pressure cells (TEPC) were installed inside the cage # 10; iii) the pore pressure in CPB was monitored by using a piezometer; iv) to monitor suction and electromagnetic properties, a heat dissipative sensor (HDS) and an EM probe were also used, respectively.

Figure 4-26: Plan view of 9555 and 9415 levels in vicinity of stope # 55 (from Thompson et al., 2008).
Figure 4-27 shows the schematic of layout of accelerometer #10 and TEPCs inside cage #10. More details on the other instruments can be found in Thompson et al. (2008). It is important to note that the black lines marking the TEPCs indicate the position of pressure cells rather than principle stress orientations, which are measured perpendicular to these lines. In other words, TEPC1 measures the vertical stress within the stope and TEPC2 and TEPC3 measure the horizontal stresses.

Figure 4-28 shows the geometry of stope #55 and the location of cage #10 and accelerometer #9 in the stope. A total volume of 10,000 m$^3$ of CPB was used to fill the stope. The CPB contained 3% binder (50% PC/50% fly ash) and its water content was about 38.9% at the paste plant. Stope #55 was poured in 5 stages, as shown in Figure 4-28. For this study, pouring stages #2 and #3 where the accelerometers are covered by CPB are important. In addition, two accelerometers were installed in rock by grouting them in two boreholes. These accelerometers were able to record a maximum amplitude of 100g with a frequency range between 0.5 Hz to 10 kHz (accuracy = ±5%).

Figure 4-27: Schematic of layout of instrumentation within cage #10 in stope #55.
4.3.2. Blasting Program at Williams Mine

The blasting program was divided into two groups: single-hole blasts and production blasts with multiple holes. The single-hole blasts were executed at different depths of a blast borehole along the stope. Note that the blast borehole and the boreholes for accelerometers are parallel and have the same inclination as the stope.

The production blasts began on March, 4th 2007 and finished on March, 26th, 2007 for stope # 52-9415 adjacent to the backfilled test stope # 55-9415. Note that accelerometer # 9 was covered by CPB on Feb 21st while accelerometer 10 was covered about 20 days later on March 12. Therefore, it was possible to monitor the response of CPB from the beginning of the production blasts by accelerometer # 9 although the curing time of CPB was about 12 days at the time (March 4th). Generally, a production blast consists of several single-hole blasts with specific delays. Depending on the design, the explosive in two or more blast holes is detonated at the same time. Figure 4-29 shows the pattern of blast holes in a production ring of the Alimak stope #52 before detonation.
4.3.3. Acceleration-time Series

Figure 4-30 and Figure 4-31 show typical signals recorded by accelerometers # 9 and # 10 for the same production blast, respectively. Some differences and similarities between these two signals can be expressed as follows. The maximum acceleration for accelerometer # 9L is about 2 times of that of accelerometer # 10L. Signals and the delay related to each blast in a production blast are more recognizable for accelerometer # 9 than those for accelerometer # 10. The lower distance of accelerometer # 9 from host rock might be responsible for these differences. In other words, the distance that waves travel through CPB for accelerometer # 9 is less than the distance for accelerometer # 10. In addition, it is most likely to have stiffer CPB around accelerometer # 9 in comparison with CPB around accelerometer # 10 because of the 20-day earlier pouring.

4.3.4. Data Processing

The aim of data processing was to determine the variations in PPA, PPV and the respective frequencies versus scaled-distance (i.e., $R/W^{1/2} = \text{distance}/(\text{charge weight})^{1/2}$). The velocity-time series were obtained by integrating acceleration-time series associated with a base line correction. Velocity-time series were then used to determine the frequency spectrum of velocity based on Fast Fourier Transform (FFT) algorithm. Note that the rotated spherical coordinate system was used for the analysis of signals in rock. All the data in these blasts represented P-wave events and lower amplitude S-waves as there were no significant surface waves.
Figure 4-30: Acceleration time series for accelerometer # 9.

Figure 4-31: Acceleration time series for accelerometer # 10.
4.3.5. Blasting Results

4.3.5.1. Variation in PPA and PPV in Rock

Figure 4-32 and Figure 4-33 show the variations in PPA and PPV versus scaled distance in rock, respectively. Generally, these two graphs show that the PPA and PPV increase as the distance decreases. However, the data set is scattered. For example, the PPA at a scaled distance of 5 m/kg$^{1/2}$ ranges between 60g and 300g while the PPA at a scaled distance of 30 m/kg$^{1/2}$ ranges between 5g and 40g. The variation in PPV is less scattered and the P- and S-wave velocities show similar values.

4.3.5.2. Variation in Frequency for PPA and PPV in Rock

Figure 4-34 shows the variation in frequency for acceleration of the same blasts versus distance in rock. Generally, frequency decreases as the distance increases although the data set is scattered. The distance of the explosive to the accelerometers is close (near-field) and ranging from 30 m to 100 metres. For such close distances, the frequency of the blasting changes from 1000 Hz to 10000 Hz in rock. The frequency of velocity for the same blasts in rock is more scattered and does not show any trend (Figure 4-35). The frequency of velocity ranges between 300 Hz and 5000 Hz and is lower than that of acceleration at a specific distance.

4.3.5.3. Variation in PPA and PPV in CPB

Figure 4-36 and Figure 4-37 show the variation in PPA and PPV versus scaled distance in CPB, respectively. Generally, the values are more scattered in the case of CPB and lower than the values in rock for both PPA and PPV at the same scaled distance. The main reason is that the propagation of wave has been through two different media with different stiffness and elastic parameters. The other effective parameter is the change in mechanical properties of CPB with time due to the hydration of cement.

4.3.5.4. Variation in Frequency for PPA and PPV in CPB

The variation in frequency for acceleration and velocity in CPB is also shown in Figure 4-38 and Figure 4-39, respectively. These values are significantly lower than similar values in rock and do not show any trend. Note that CPB around accelerometer 9 is more mature than CPB around accelerometer 10 at the time of each production blast.
Figure 4-32: PPA for P and S waves versus scaled distance in rock (Mohanty and Trivino, 2009).

Figure 4-33: PPV for P and S waves versus scaled distance in rock (Mohanty and Trivino, 2009).
Figure 4-34: Variation in frequency for acceleration in Rock (Mohanty and Trivino, 2009).

Figure 4-35: Variation in frequency for velocity in Rock (Mohanty and Trivino, 2009).
Figure 4-36: PPA versus scaled distance in CPB (Mohanty and Trivino, 2009).

Figure 4-37: PPV versus scaled distance in CPB (Mohanty and Trivino, 2009).
Figure 4-38: Variation in frequency for acceleration in CPB (Mohanty and Trivino, 2009).

Figure 4-39: Variation in frequency for velocity in CPB (Mohanty and Trivino, 2009).
4.3.6. Near-Field Blasting Events versus Far-Field Seismic Events

As shown in the previous section, the level of ground motion (i.e., PPV) is controlled by distance and charge weight. The worst case scenario would then be when the distance has its shortest value and the charge weight has its highest value. The general empirical equation of PPV for P-waves in rock developed based on the production blast results at the Williams mine was given as (Mohanty and Trivino, 2009):

$$PPV = 10^{(3.41+0.58)} \left( \frac{R}{W^{1/2}} \right),$$

(4.1)

where $R$ is distance in m and $W$ is charge weight in kg.

Based on the blasting patterns at the Williams Mine, the shortest distance between the accelerometer and the production blast boreholes in rock is about 30 m while the longest distance is about 100 m. The charge weights also range from 10 kg to 60 kg for the boreholes in a production blast. Therefore, the upper limit of PPV in rock based on equation 4.1 for the strongest charge weight ranges from 22.6 mm/sec to 209.9 mm/sec for the distances 100 m and 30 m, respectively. The lower limit of PPV ranges from 4.3 mm/sec to 40.0 mm/sec for the distances 100 m and 30 m, respectively. Figure 4-40 shows the upper and lower limit of PPV for P-waves recorded during production blasts at the Williams Mine along with the NRCan events and the seismic events recorded at the mines using the strong ground motion (SGM) system.

The lowest value for the PPV at a distance of 100 m from the blasting ring is about the PPV of the 1988 Saguenay earthquake (6.4 $m_N$) with a distance of 51 km from its hypocentre. On the other hand, the highest value for the PPV at a distance of 30 m from the blasting ring is about 2.8 times the PPV of the strongest component (SH) of the January rockburst (3.8 $m_N$) at the Kidd Mine at a distance of 2 km from its source. This result indicates that the trend of PPV versus distance of a typical near-field production blast as usually practiced in northern Ontario’s mines is not significantly different from that of the far-field seismic events occur in northern Ontario. Note that the NRCan far-field earthquakes and rockbursts recorded on the surface consist of surface waves while the near-field production blasts are for P-waves.
Figure 4-40: Variation of PPV versus distance for all near-field and far-field events.

In contrast, the frequency content of the typical production blast in rock is significantly higher than that of the far-field seismic events shown in this study. The stress waves induced by production blasts and recorded in near distances contained high frequency components, while the high frequency components of the far-field seismic events are scattered and absorbed due to long traveling distances. For example, the frequency of P-wave velocity for production blasts recorded at the closest distance in rock changes from 1000 Hz to 5000 Hz, which is significantly higher than typical frequencies of the far-field seismic events in particular and that of engineering problems in geotechnical engineering in general. Therefore, the conventional approach used in geotechnical engineering might not be applicable in the case of near-field blasting due to the effect of frequency.

In addition, the effect of P-waves is not considered as important for the design of surface structures in geotechnical engineering as only surface waves induced by earthquakes are considered for such designs. In contrast, the near-field blasting shows that both P- and S-waves
are involved. Therefore, the changes in stress condition due to blasting are more complicated since both compressional and shear stresses are involved.

4.4. Conclusions

Underground CPB systems may be subjected to different near- and far-field seismic events including earthquakes, rockbursts and production blasts. Earthquakes are considered as far-field events. In Northern Ontario, which has a very low level of seismic activity, earthquakes are weak and less frequent compared to the north western North America. Thus, the effect of earthquakes on the stability of CPB systems (i.e., liquefaction potential) is not as important as that of near-field seismic events. In other words, the stability of CPB systems subjected to near-field rockbursts and production blasts with the same magnitude as the far-field earthquakes is more of concern. Although the effect of a typical near-field production blast was analyzed, there were no waveforms available for the near-field rockbursts to study in this thesis. On the other hand, there is no definite criterion to evaluate the liquefaction potential of materials induced by near-field seismic events. Since the development of a new criterion requires understanding of ground motion parameters (i.e., PPA, PPV and frequency) as well as variations in stress conditions, the characterization of different near- and far-field seismic events was carried on in this chapter. Although the approach to this analysis was not intended to develop a new criterion, the results provided some basic information that can be used to investigate a potential for extending the criteria used in conventional geotechnical earthquake engineering. Table 4-10 summarises the dynamic parameters (magnitude, PPV, frequency) of the seismic events analyzed in this chapter. The main conclusions based on this analysis can be articulated as follows:

<table>
<thead>
<tr>
<th>Seismic Events</th>
<th>Type</th>
<th>Magnitude ( m_N )</th>
<th>Distance ( \text{km} )</th>
<th>PPV ( \text{m/s} )</th>
<th>Predominant Frequency ( \text{Hz} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake</td>
<td>Far-field</td>
<td>3.7-4.3</td>
<td>100-400</td>
<td>( 10^{-6}-10^{-3} )</td>
<td>3-16</td>
</tr>
<tr>
<td>Rockburst</td>
<td>Far-field</td>
<td>3.1-4.1</td>
<td>1-400</td>
<td>( 10^{6}-0.1 )</td>
<td>3-25</td>
</tr>
<tr>
<td></td>
<td>Near-field</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Production Blast</td>
<td>Far-field</td>
<td>2.7-3.4</td>
<td>1-2</td>
<td>0.001-0.01</td>
<td>33-40</td>
</tr>
<tr>
<td></td>
<td>Near-field</td>
<td>-</td>
<td>0.03-0.1</td>
<td>0.004-0.074</td>
<td>200-4000</td>
</tr>
</tbody>
</table>
4.4.1. Far-field NRCan Events

1. NRCan seismic events including far-field earthquakes and mining events showed good correlation between the dynamic parameters, such as PPA, PPV and their corresponding predominant frequencies versus distance. The PPA of the seismic events varied from 0.02 m/s² to 0.0009 m/s² when the distance from the source varied from 22 km to 250 km for the seismic events with $m_N$ 3.5-4.3, as described in Section 4.1.

2. There is a good consistency between the magnitude and PPA of the NRCan seismic events (i.e., earthquakes and rockbursts) at a specific distance. The higher the magnitude, the higher the PPA at a specific distance, as presented in Figure 4-10. Generally, the magnitudes of the earthquakes are relatively low due to the low seismic activity of the north eastern Ontario seismic zone.

3. The NRCan seismic events including earthquakes and rockbursts can be categorized as low frequency events (3-16 Hz), as shown in Figure 4-11. Therefore, the NRCan rockbursts can plausibly be treated the same as earthquakes for conventional geotechnical earthquake engineering design problems.

4.4.2. Far-field Mining Events

1. The mining events, including rockbursts and production blasts recorded at the mines but still at a relatively far distance (1-2 km) from the seismic sources, also showed similar trends (in comparison with NRCan events) in PPV and PPA with respect to their distance. However, the PPV of these events is more scattered with respect to their magnitudes, ranging from 0.008 m/s to 0.09 m/s when magnitude varied from $m_N$ 2.7 to $m_N$ 3.8 at 2 km from the source (Figure 4-24).

2. The predominant frequency of the rockbursts is similar to that of the NRCan seismic events, ranging between 5 Hz and 25 Hz. However, the production blasts show higher values of about 33-40 Hz for frequency (Figure 4-25). Conventional geotechnical earthquake engineering design problems rarely consider events with the frequencies characteristics of production blasts; therefore, CPB response to blasting remains an area of further research.
4.4.3. Near-field Production Blasts

1. The typical blasting practice at the Williams Mine shows that the PPV in rock induced by production blasts with different charge weights at a near distance (30-100m) from the source ranges from 0.004 to 0.2 m/s, which is not significantly higher than the upper limit of PPV (0.004-0.074 m/sec) for mining events recorded at ~1-2 km from the source (Figure 4-40).

2. In contrast, the predominant frequency arising from spectral analyses of velocity-time series for the production blasts recorded in rock at the Williams Mine ranged from 200 Hz to 4000 Hz, which is significantly higher than the predominant frequencies of the NRCan seismic events and those mining events recorded about 1-2 km from the source (i.e., 5-40 Hz, as shown in Figure 4-25). Therefore, production blasts occurring at close distance (30-100 m) might not be treated the same as the seismic events recorded at distances more than 1-2 km for geomechanical design problems.

3. The PPV induced by the same production blasts at the Williams Mine but recorded in CPB varied from 0.002 m/s to 0.04 m/s when the distance from the source was about 60-80 m. The frequency content for velocity in CPB varied from 120 Hz to 700 Hz depending on curing time, as described in Section 4.3.5. This indicates that the frequency in rock is significantly higher than the frequency in CPB for the same production blast.

4.4.4. Applicability of Geotechnical Earthquake Engineering Criteria

1. Far-field rockbursts (1-400 km) can plausibly be treated the same as earthquakes for conventional geotechnical earthquake engineering design problems. Therefore, the cyclic response of CPB, which will be presented in Chapter 8, can be used to evaluate the susceptibility of CPB to far-field rockbursts.

2. Near-field production blasts might not be treated the same as the earthquakes for conventional geotechnical earthquake engineering design problems. Therefore, determining the response of CPB to these higher loading remains a significant research challenge. However, in the case of the CPB system at the Williams Mine, there has been no sign of instability or significant excess pore water pressure in CPB within the range of
PPV and frequency (described in Section 4.3.5) of stress waves induced by the production blasts.
5. Laboratory Study

5.1. Material Tested

5.1.1. Mine Tailings

The mine tailings used to make CPB in this study were collected from the paste plant at the Williams Mine. Samples were poured into 20 litre buckets and covered with about 15 cm of water to prevent oxidation. The samples were then sealed and transported to the University of Toronto. The tailings samples were received in April 2008.

The tailings can be described as grey, angular fine-grained soil with an initial water content of 30-33% with no evidence of oxidation. It is necessary to mention that the water content used in this study is defined based on geotechnical engineering as $M_{\text{water}}/M_{\text{dry}}$ because the water content in the mining industry is considered as $M_{\text{water}}/M_{\text{total}}$. The index properties of mine tailings, such as liquid limit, plasticity index, particle size distribution and specific gravity, were then determined.

The particle size distribution of the tailings was determined using the standard sieve analysis method (ASTM C136-06), followed by the standard hydrometer test (ASTM D422-63) without addition of the deflocculating agent. The sieve analysis test was used to categorize the particles larger than 75 microns while the hydrometer test was used to determine the distribution of the particles smaller than 75 microns. As shown in Figure 5-1, the tailings mainly contain silt with 14% of fine sands and 4% of clay-sized particles. The percentage of fine particles appropriate for
CPB, which are less than 20 microns, is about 40%. The specific gravity of the tailings, determined using (ASTM D854-06), is 2.72.

To determine the liquid limit (LL) and plastic limit (PL) of the tailings, the standard test method (ASTM D4318 – 05) was used. The liquid limit of the tailings is 30% and the plastic limit is 26% resulting in a plasticity index (PI) of 4%. As recommended by Boulanger and Idriss (2004), fine grained soils are classified as “sand like” if PI < 7, “clay like” if PI ≥ 7. Therefore, the silt-sized tailings in this study would classify as “sand like” with low plasticity.

The chemical composition of the tailings, determined using the X-Ray Fluorescence Spectroscopy (XRF) method, is shown in Table 5-1. The major compounds are SiO₂ and Al₂O₃, and the minor compounds are CaCO₃, iron and alkalis. Note that Ca was not detected in the form of oxide. In addition to these major and minor compounds, the tailings have a low sulphur content (S = 1.081%) relative to other hard rock mine tailings, such as those reported in Benzaazoua et al. (2002). This table also shows the chemical composition of the Portland cement used in this study. The X-ray diffraction (XRD) analysis, used to identify the mineral components of the tailings, showed that quartz and albite are the major minerals, as shown in Figure 5-2. The
minor minerals can be identified as microcline and clinochlore. No sulphide mineral, such as Pyrite, was identified in the tailings, which is consistent with the low percentage of sulphur determined in the XRF test.

Table 5-1: Chemical composition of Williams tailings and Portland cement.

<table>
<thead>
<tr>
<th>Compound</th>
<th>SiO₂ %</th>
<th>Al₂O₃ %</th>
<th>CaCO₃ %</th>
<th>Fe₂O₃ %</th>
<th>MgO %</th>
<th>Na₂O %</th>
<th>K₂O %</th>
<th>MnO %</th>
<th>TiO₂ %</th>
<th>P₂O₅ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Williams</td>
<td>58.30</td>
<td>11.85</td>
<td>5.13</td>
<td>2.89</td>
<td>3.70</td>
<td>3.25</td>
<td>3.42</td>
<td>0.03</td>
<td>0.36</td>
<td>0.18</td>
</tr>
<tr>
<td>Portland</td>
<td>16.27</td>
<td>3.37</td>
<td>71.18</td>
<td>2.26</td>
<td>3.01</td>
<td>0.20</td>
<td>0.45</td>
<td>0.10</td>
<td>0.20</td>
<td>0.11</td>
</tr>
<tr>
<td>cement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-2: Mineral composition of The Williams Mine tailings.

The XRD analysis on Portland cement received from the Williams Mine revealed that the major minerals are C₃S and C₂S while other PC minerals were not detected. Figure 5-3 shows the major and minor mineral composition of the Portland cement used in this study.
### Table 5.1: Chemical Formulas of Portland Cement Components

<table>
<thead>
<tr>
<th>Ref. Code</th>
<th>Compound Name</th>
<th>Chemical Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>00-055-0740</td>
<td>Tri-Calcium Silicate</td>
<td>Ca$_3$SiO$_5$</td>
</tr>
<tr>
<td>00-024-0034</td>
<td>Di-Calcium Silicate</td>
<td>Ca$_2$SiO$_4$</td>
</tr>
<tr>
<td>00-049-1882</td>
<td>Calcium Aluminium Oxide Fluoride</td>
<td>11CaO · 7 Al$_2$O$_3$ · Ca F$_2$</td>
</tr>
<tr>
<td>00-036-0617</td>
<td>Bassanite, syn</td>
<td>CaSO$_4$ · 0.67 H$_2$O</td>
</tr>
</tbody>
</table>

Figure 5-3: Mineral composition of Williams Portland cement.

### 5.1.2. CPB Set Time

The CPB specimens tested in this study contain 3% Portland cement from the Williams Mine. The specimens were cured for 4 hours and 12 hours prior to the laboratory testing. To investigate the setting of the CPB during this period, a series of standard tests for the time of setting (ASTM C191-08) was conducted by using a Vicat needle apparatus. According to this test method, the initial set is the time required for the needle to reach a penetration depth of 25 mm. The final set occurs when the needle does not penetrate into the CPB specimen.

Figure 5-4 shows the results of the Vicat needle tests for 3% CPB specimens along with the electrical conductivity (EC) tested by Simon and Grabinsky (2007). The initial and final set for these specimens are approximately 630 min (10.5 hours) and 1680 min (28 hours), respectively. Note that the Vicat needle penetration readings started giving non-maximum values at 300 min (5 hours), which coincides with the appearance of the EC peak. Similar correlation between the
onset of the penetrative resistance (i.e., non-maximum values) and the maximum EC has been reported by Levita et al. (2000) for Portland cement mixtures in the early age of hydration. Furthermore, Hwang and Shen (1991) showed that the onset of the penetrative resistance (the equivalent of non-maximum in the Vicat needle test) corresponds well to the onset of the acceleration phase determined from the heat evolution curve, thus implying that the onset of penetrative resistance, as well as maximum EC, takes place during the acceleration phase of hydration process.

These results suggest that the CPB specimens cured for four hours have not set yet and the hydration products have not yet started forming a network between the tailings particles. However, the CPB specimens tested at 12 hours have already experienced the initial setting (the initial set = 10.5 hours), implying that the network of hydration products between the particles has started forming, thus contributing to specimen stiffness and strength. Therefore, differences in the mechanical behaviour of the specimens cured for four hours and those cured for 12 hours should be expected.

![Figure 5-4: The initial and final set of 3% CPB specimens along with the electrical conductivity measurements.](image)

Figure 5-4: The initial and final set of 3% CPB specimens along with the electrical conductivity measurements.
5.2. Experimental Design and Procedure

According to the objective of the thesis, two experimental components for laboratory work were suggested. The first laboratory component was to investigate the monotonic response of CPB with respect to the stress conditions in the field during the filling of the stope. The second laboratory component was to simulate the cyclic response of CPB induced by certain forms of dynamic loadings, in particular, large rockbursts.

Since the laboratory experiments intend to simulate the field conditions, the in situ properties of CPB in the test stope, such as water content, void ratio and degree of saturation, must be obtained. In addition, the initial field stress conditions must be determined.

5.2.1. In Situ Properties and Stress Conditions

The initial water content of CPB was about 39% with a slump of 20 cm at the Williams Mine paste plant. The sampling from CPB at the test stope # 55-9415 before pouring showed that the slump values might change depending on the water content of the samples. However, the samples at the test stope with the same water content as at the paste plant showed an increase in slump. The difference between the slumps at the paste plant and the test stope might be related to the shearing process during the transport of CPB in pipelines. The in-situ properties of CPB were then obtained for the test stope # 55-9415 after the placement of CPB. It has been shown that there is no significant change in water content, void ratio and degree of saturation during the first 9 months of curing paste in the stope. The water content, void ratio and degree of saturation were about 38-39%, 0.86-1.20 (an average of 1.02±0.05,) and 98± 2%, respectively (Grabinsky et al., 2008). These values can be used as index parameters for both components of the laboratory work in this study.

The stress measurements at the Williams Mine also showed that the vertical and horizontal effective stresses might change in a narrow long stope (i.e., stope # 55-9415) over time due the development of arching (Thompson et al., 2008). However, the effective stresses might be the same as hydrostatic pressures at the early stage of the pour due to the self-weight of freshly placed CPB. The minimum and maximum effective stresses obtained during the third stage of CPB pour in the test stope # 55-9415 were as low as 20 kPa and 60 kPa, respectively. However, for laboratory work, a wider range of effective stresses, between 20 kPa and 200 kPa, is...
considered to also cover stresses that might occur in wide stopes where arching is not so prevalent.

### 5.2.2. Monotonic Loading

The purposes of investigating the monotonic response of fresh CPB (i.e., 4 hours old) were (i) to determine whether or not the material was liquefaction resistant, and (ii) to determine the contractive or dilative behaviour of the material at different effective stresses similar to the field conditions. Therefore, the consolidated undrained (CU) triaxial compression test (ASTM D4767-04) was recommended. In this method, fully saturated CPB specimens at the desired effective confining stress were subjected to a monotonic load under a constant strain rate in an undrained condition. A relatively “high” axial strain rate of 2% /min has been considered for CPB because its mechanical properties depend on the time of hydration. The maximum axial strain during triaxial shearing was 20% and this took 10 minutes at an axial strain rate of 2%/min. Therefore, for the specimens cured for four hours at the start of shearing, the actual shearing phase took place between 240 minutes and 250 minutes. In this timeframe, the hydration process has not yet achieved a stage where any significant resistance to penetration of the Vicat needle can be detected (see Section 5.1.2), and thus the mechanical behaviour of the sample can be taken as essentially constant during the triaxial shearing phase.

The high axial strain rate reduces the duration of the experiment, but it might also affect the measured response of the material through unequal distribution of pore water pressure throughout the sample. According to ASTM D4767-04, the axial strain rate, $\varepsilon'$, for the drained triaxial testing can be estimated from equation (5.1) if failure occurs after 4% axial strain.

$$\varepsilon' = 4\%/10 \times t_{50},$$  \hspace{1cm} (5.1)

where $t_{50} = \text{time to achieve, on average, 50\% of primary consolidation}$

$$t_{50} = T_{50} (H_{dr})^2/C_v$$  \hspace{1cm} (5.2)

where

$$T_{50} = \pi(U_{av}/100)^2/4 \text{ (i.e., } T_{50} = \text{time factor and } U_{av} = \text{average degree of consolidation = 50\%)},$$

$H_{dr} = \text{length of drainage path, and}$
\( C_v = \) coefficient of consolidation.

The typical values of coefficient of consolidation for mine tailings similar to the one tested in this study might vary from \( C_v = 5 \times 10^{-3} \) to \( 5 \text{ cm}^2/\text{sec} \) (Le Roux, 2004). Furthermore, le Roux (2004) noted the difficulty in determining \( C_v \) from the consolidation stage of triaxial testing to better than an order of magnitude given the very short times needed to achieve essentially full consolidation. Therefore, taking the range \( C_v = 5 \times 10^{-3} \) to \( 5 \text{ cm}^2/\text{sec} \), the corresponding minimum and maximum axial strain rates can be calculated based on equations (5.1) and (5.2) for a specimen with single drainage (i.e., \( H_{dr} = 10 \text{ cm} \)) as \( \varepsilon' = 0.006 – 6 \%/\text{min} \). The axial strain rate used in this thesis (2\%/min) is within the range of axial strain rates calculated using equation (5.1); nevertheless, the effect of axial strain rates of 0.1 – 2\%/min on the measured monotonic behaviour of the tailings will be investigated in Chapter 7.

### 5.2.3. Cyclic Loading

The common practice to simulate earthquake loadings in the laboratory is to conduct cyclic tests using a simple direct shear or triaxial machine. The advantage of using the simple shear test over the triaxial test is to better represent the stress conditions due to vertically propagating S-waves induced by earthquakes. In other words, the specimen in the direct simple shear test is directly subjected to cyclic horizontal shear stresses, as shown in Figure 5-5. However, in the triaxial test, the specimen is subjected to a radial stress and an axial stress. By virtue of these boundary conditions, the principal stresses in the specimen are always vertical and horizontal. These conditions are not similar to the rotation of principal stresses due to vertically propagating S-waves. However, the cyclic triaxial tests have been used by many researchers to investigate the cyclic response of soils.

The input parameters for the cyclic tests include cyclic stress ratio (CSR) and the frequency of uniform load cycles. The CSR is defined as the ratio between the deviator stress, \( \Delta \sigma_d \), and twice the effective confining stress, \( \sigma'_c \).

\[
CSR = \frac{\Delta \sigma_d}{2 \sigma'_c} \tag{5.3}
\]
Therefore, at a specific CSR and effective confining stress, it is possible to calculate the deviator stress required in a triaxial test under the stress or load controlled mode. The effective confining stress in this study is similar to the anticipated field stress conditions (i.e., 20-200 kPa).

![Diagram of direct simple shear apparatus](image)

**Figure 5-5: Schematic of direct simple shear apparatus (after Kramer, 1996).**

In geotechnical earthquake engineering, the frequency of uniform load cycles is considered to have a small effect on liquefaction potential within the range of frequencies of engineering interest (e.g., Lee and Focht, 1975). The typical frequencies applied for the triaxial tests range from 0.1 to 2 Hz (e.g., Hyde et al., 2006).

To investigate the response of CPB to stress waves induced by earthquakes, the standard test method for load controlled cyclic triaxial strength of soil (ASTM D5311 – 92, 2004) were considered. The uniform sinusoidal stress cycles with constant frequency of 0.12 Hz at different CSR’s ranging from 0.15 to 0.3 were used in this study. Similar to monotonic loading, the cyclic response of mine tailings mixtures with no cement should also be investigated to better understand the effect of binder agents on the behaviour of the material.

### 5.2.4. Applicability of Triaxial Tests for Rockburst and Blasting

Instead of earthquake loading, a backfilled stope with CPB is more likely to experience loadings caused by blasting or rockbursts. As described in Chapter 3, compressional stress waves rather than shear stress waves are dominant in the case of blasting loads and rockbursts. This condition
can be simulated by applying uniform cyclic axial loads (i.e., compression-extension) during the triaxial tests. However, the main problem is the frequency content of cyclic uniform loads.

As described in Chapter 4, the near field blast monitoring during the blasting program at The Williams Mine showed that the frequency of particle velocities and consequently the stress wave in CPB is relatively high ranging from 200 Hz to 4000 Hz. This frequency is some three orders of magnitude higher than the typical frequency applied for simulating earthquakes in the triaxial tests. Unlike geotechnical earthquake engineering, the effect of frequency is plausibly not negligible in the case of blasting loads. In addition, this frequency is beyond the capability of the triaxial machine. However, there are other laboratory methods to investigate blast-induced liquefaction potential of the materials. For example, Veyera and Charlie (1990) suggested the shock loading method to investigate the liquefaction potential of sands (see Chapter 3). Therefore, the results of cyclic tests at low frequencies might not be useful to determine the liquefaction susceptibility of CPB to blasting.

In contrast, the frequency of stress waves induced by rockbursts in the far field is significantly less than that of blasting waves. As described in Chapter 4, the frequency of particle velocities in rock recorded at the mines is about 8-21 Hz. Since fresh CPB has a lower impedance than the surrounding rock, the frequency of the particle velocity might be less than these values in CPB for the same rockburst event. This frequency is about the range of the frequencies recorded for the earthquakes. Therefore, it is plausible to consider a negligible effect for frequency in the case of laboratory simulation of far field rockbursts using the triaxial tests.

5.3. Sample Preparation and Setup

There is no standard triaxial specimen preparation method for the paste material, which has a high water content and slump. However, Crowder (2004) and le Roux (2005) suggested the pre-consolidation method of preparation for creating a triaxial specimen of paste material. In this study, the same methodology was generally applied with some minor changes due to the changes in the apparatus and triaxial setups.

5.3.1. Mixing Method

As received from the mine, the tailings were mixed with process water using a paint mixing attachment on an electric drill to ensure the material was well blended. The tailings mixture was
stored in the same sealed bucket. Prior to sampling from the bucket, the tailings and water should be remixed. To create a CPB specimen, a 600 gram sample of tailings was collected from the bucket and its initial water content was measured one day in advance. Having the initial water content of tailings, it was possible to calculate the mass of cement required based on the mass of the solids. The 600 gram sample was then mixed using a hand mixer, with additional water to have a mixture with a total water content of 39% considering the amount of cement, and similar to the field water content. The binder agent was then mixed with the tailings mixture for an additional 10 minutes. The water content of the CPB sample was then measured prior to casting the triaxial specimen. For this study, 3% by weight Portland cement was used as the binder agent in CPB.

The method of mixing was intended to simulate the field conditions. However, there are some limitations during mixing the specimens. The transportation of CPB from the paste plant to the test stope takes about 20 minutes in pipelines at the Williams Mine. In addition, the transportation of CPB in pipelines is based on a plug flow mechanism where the bulk of the material flows as a core of interlocked and water saturated solid particles surrounded by a thin lubricating layer of a homogeneous mixture made up of water and very fine particles. CPB is sheared during this method of transportation and its flow properties including slump are different from un-sheared material (Cook, 2007). In contrast, the laboratory mixing procedure creates a homogenous mixture in less than 10 minutes, while the shearing process during mixing is different from shearing in pipelines. Therefore, in this study, a 10 minute mixing period was considered since the additional mixing period has no effect on the shearing process, meanwhile the placement of the CPB sample in the mould is more appropriate after 10 minutes. The same method of mixing was used for the triaxial testing of uncemented tailings mixtures without the step of addition of cement to the mixture.

5.3.2. Equipment

A GCTS triaxial cell, manufactured from stainless steel, was used in this study (GCTS manual). The top and bottom platens of the triaxial cell had a diameter of 50 mm and one drainage line. To create a stable sample, a split mould was used, which had a groove inside to allow for the vacuum to reach the entire area of the membrane. The mould has a larger diameter at the bottom to accommodate o-rings on the bottom platen. The mould is tall enough to build a 110 mm sample
plus two porous stones and allows the top platen to seat inside the mould 10 mm or more. The split mould is held together with two clamps.

5.3.3. Sample Preparation

The basic setup for a triaxial CPB sample involves placing the material between two porous stones in a latex membrane while the specimen stands on its own. Prior to placing the sample, two porous stones and the top and bottom platen lines should be saturated with water. One of the porous stones is placed on the bottom platen, and a filter paper is used to separate the porous stone from the sample. A latex membrane is only fixed to the bottom platen using two o-rings. The split mould is then brought together and is secured using a clamp. Two o-rings are temporarily placed onto the top of the split mould. The membrane is then flipped over the top of the split mould. To ensure the membrane lies flat against the insides of the split mould, a 10 kPa vacuum is applied through the groove of the mould (Figure 5-6).

Figure 5-6: Membrane adjustment and CPB placement in a mould.
Once the CPB sample is ready, it is poured into the mould by using about 4-5 scoops of a spoon. The mould is filled in three stages while each stage should be accompanied by removing any large voids using a 5 mm diameter glass rod to rod the sample about 20 times. When the mould is filled to the desired height, the top filter paper and porous stone are placed on top of the sample, followed by the top platen. The piston, which goes through the assembly guide mounted on the triaxial cell, must be attached to the top platen at this point.

To calculate the void ratio and degree of saturation of the specimen, the mass and volume (i.e., area × height) of the placed specimen are required. The mass of CPB specimen used for the triaxial testing can be calculated by subtracting the masses of bowl containing the CPB sample before and after pouring. The initial height of specimen is measured after placing the top platen. At this point, the specimen is ready for dead-weight consolidation while the latex membrane is not yet attached to the top platen.

The procedure of dead weight consolidation is to apply 2.5 kg (equivalent to about 12.5 kPa) on the top of the sample while the water is collected from the bottom drainage line and at the top between the membrane and the top platen (the top drainage valve is closed). Since the mass of the top platen and piston is 500 g, two additional 1 kg weights should be applied within a few minutes. The dead-weight consolidation phase takes less than an hour. Considering the duration of mixing and the dead-weight consolidation phase, the CPB specimen has cured for an hour at the end of this phase.

After dead-weight consolidation, the bottom drainage valve is closed, the final mass of water collected is recorded, and the membrane is then fixed onto the top platen with the o-rings. The piston, which is attached to the top platen, is then locked using a special mechanism on the assembly guide. The weights are then removed, followed by removing the vacuum and the split mould. The final height of the specimen is measured to calculate the final void ratio of the triaxial specimen. Figure 5-7 shows the dead-weight consolidation phase of sample preparation.

Once the triaxial sample stands on its own, the triaxial cell is assembled. Prior to filling the triaxial cell with de-aired water, it is placed in the middle of the triaxial frame in a way that the piston is aligned, and in contact with the load cell, as shown in Figure 5-8. The back saturation process begins after the triaxial cell is completely filled with water.
Figure 5-7: Dead-weight consolidation process.

Figure 5-8: Triaxial setup.
To achieve a fully saturated specimen, the back saturation process is followed in accordance with ASTM D4767-04. The process begins with applying a low cell pressure of 15 kPa, and back pressure of 5 kPa, while the piston is unlocked to ensure that the specimen experiences the isotropic pressure. Generally, a back pressure of more than 350 kPa was required to obtain a specimen with a B-value of more than 0.96. The cell pressure and back pressure are then gradually increased in such a way that the difference does not exceed the final effective consolidation stress. After achieving a fully saturated specimen, it is time to consolidate the specimen at a desired effective confining stress, $\sigma'$, by increasing the cell pressure while the back pressure remains constant. During the consolidation phase, the drainage is allowed at a bottom platen while the pore pressure is monitored from the top drainage line. After consolidation is complete, when the difference between cell pressure and pore pressure is equal to the desired effective confining stress, the bottom drainage valve is closed to maintain the undrained condition during the triaxial tests.

The minimum time required to prepare a fully saturated, consolidated CPB specimen was about 4 hours since adding the cement to the tailings mixture in this study. There are some limitations that will be discussed in the following section.

5.3.4. Sample Preparation Limitations

Generally, the initial void ratio of CPB specimens before dead-weight consolidation was around 1.000±0.005, which was similar to the field void ratio. However, the void ratio of the specimens after dead-weight consolidation decreased to 0.860±0.005 (further details will be discussed in Chapter 6). In addition, the final void ratio of the specimens after back saturation and triaxial consolidation phases ranged between 0.770 and 0.890 depending on the final effective confining stress. Therefore, the tested specimens in the triaxial tests have a lower void ratio than the field materials. The maximum void ratio achieved during the dead-weight consolidation was 0.890 while applying a minimum of 500 g weight on the specimen during dead-weight consolidation. However, it was difficult to reproduce this type of sample.

By using a single drainage dead-weight consolidation suggested by Le Roux (2004), a higher void ratio specimen of 1.200±0.005 can be produced after dead-weight consolidation. However, the distributions of void ratio in these specimens were not uniform. This was investigated by measuring the void ratio of the top, middle and bottom parts of a specimen after it is cured. In
contrast, the results for the double drainage specimens showed that the specimens were more uniform.

For the tailings mixtures with no cement, the 2.5 kg dead-weight consolidation yielded the specimens with a void ratio of 0.750±0.005. This shows that the void ratio of uncemented tailings specimens is generally lower than that of CPB specimens after dead-weight consolidation.

5.4. Triaxial Machine

A servo-hydraulic triaxial machine designed by Geotechnical Consulting and Testing Systems (GCTS) was used in this study. A hydraulic power supply provides hydraulic pressure for the triaxial machine with a flow rate of about 5 gpm. This power supply is fitted with the necessary filter to supply oil pressure to the servo valve mounted on the triaxial frame. The pump is operated in either low pressure or high pressure settings. The low pressure output is about 1MPa and the output pressure for the high pressure setting is about 21MPa. Figure 5-9 shows the top view of the hydraulic power supply. In this figure, the “pressure line” is the connection point for the high pressure supply hose to the system servo valve, and the “return line” is the connection point for the hydraulic return line from the servo valve.

Figure 5-10 shows the servo valve and axial actuator mounted on the triaxial frame, as well as the other parts of GCTS system such as the digital system controller (black box on the left), the pressure control box (black box on the right), and the air/water transfer and triaxial cells. The hydraulic power supply, the servo valve and all the sensors including the load cell, the linear variable differential transducer (LVDT), and regulators/pressure transducers are controlled by a digital system controller (GCTS SCON-1500) through the GCTS Computer Aided Testing Software (CATS). Using this digital system, optimization and calibration settings (e.g., gains and offsets) can be controlled by the software.

The two sets of regulator/pressure transducer are installed in a separate black box (i.e., pressure control box). The cell pressure required for a triaxial test is controlled by one set of regulator/pressure transducer while the second set is used for the back pressure supply. The pressure transducer in the second set can also be used for measuring the pore pressure. In this case, the ball-valve located between the regulator and pressure transducer should be closed. The ball-valve is controlled by the software as a digital output.
Figure 5-9: GCTS Hydraulic Power Supply (GCTS manual).

Figure 5-10: Servo valve and axial actuator mounted on the triaxial frame. The digital system controller SCON-1500 (black box on the left), the pressure control box (black box on the right), and the air/water transfer cells of the GCTS system.
The features of the sensors used for this study can be articulated as follows. The load cell has the capacity of $\pm 22000 \pm 2$ N. The LVDT has a range of $\pm 25.000 \pm 0.005$ mm displacement. The pore pressure and cell pressure sensors have $1000 \pm 1$ kPa limits. The machine is capable of applying a uniform wave form, such as sinusoid and square at different frequencies, or any user defined irregular wave forms. The maximum frequency is about 70 Hz. The different phases of a triaxial test, such as back saturation, consolidation, and loading can be run automatically or manually. For example, the cell and back pressure increases during the back saturation phase can be performed automatically by defining the time interval and the maximum back pressure in the software. For this study, the back saturation and consolidation phases were performed manually.
CHAPTER 6

COMPRESSSION CHARACTERISTICS OF CPB

6. Compression Characteristics of CPB

6.1. Consolidation Tests

As explained in Chapter 5, the sample preparation (i.e., dead-weight consolidation), the back pressure saturation stage and the consolidation stage of the triaxial testing are similar for all the monotonic and cyclic tests in this study. Therefore, it is possible to use the data obtained from these three stages to evaluate the compression characteristics of CPB. Note that the time frame for the triaxial consolidation stage is about 1 hour and CPB is about 3 hours old in the beginning of this stage. Therefore, the CPB specimens even at the end of this stage have still shown maximum values in the Vicat needle tests; the specimens have not yet set (initial set = 10.5 hours), as described in Section 5.1.2 of this thesis.

Table 6-1 shows the void ratio, water content and the degree of saturation of CPB specimens before and after the dead-weight consolidation stage. The void ratios after the dead-weight consolidation are the initial void ratios corresponding to 10 kPa effective vertical stress. The correlation coefficient between the void ratios before and after dead-weight consolidation is 0.7 indicating a reasonable consistency of the results in this stage. Note that the water content was determined by direct measurements, while the void ratio was determined using the mass and height of specimens. The absolute error for void ratio is ±0.005 and the absolute error for water content is ±0.001. The absolute errors were calculated based on the precision of the scale (±0.01 gram) and the calliper (i.e., 0.01 mm) that were used to determine the mass and the dimension of
specimens, respectively. The degree of saturation was calculated based on the void ratio and water content measured before and after the dead-weight consolidation stage. A degree of saturation larger than 1 is an artifact of the calculation processed used, and should not be literally taken as \( V_w/V_v \) (volume of water / volume of voids). The degree of saturation after dead-weight consolidation was lower than 100%.

### Table 6-1: Properties of CPB specimens after dead-weight consolidation and isotropic consolidation stages of triaxial testing.

<table>
<thead>
<tr>
<th>Group Test No</th>
<th>Dead-weight consolidation (10 kPa)</th>
<th>After triaxial consolidation stage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water Content (±0.001)</td>
<td>Void Ratio, e (±0.005)</td>
</tr>
<tr>
<td>Before</td>
<td>After/initial</td>
<td>Before</td>
</tr>
<tr>
<td>Group 1 1</td>
<td>39.179</td>
<td>30.685</td>
</tr>
<tr>
<td>2</td>
<td>38.427</td>
<td>30.822</td>
</tr>
<tr>
<td>3</td>
<td>39.373</td>
<td>30.753</td>
</tr>
<tr>
<td>4</td>
<td>38.870</td>
<td>30.907</td>
</tr>
<tr>
<td>5</td>
<td>38.313</td>
<td>30.157</td>
</tr>
<tr>
<td>Group 2 6</td>
<td>39.160</td>
<td>30.483</td>
</tr>
<tr>
<td>7</td>
<td>39.548</td>
<td>29.988</td>
</tr>
<tr>
<td>8</td>
<td>39.159</td>
<td>31.216</td>
</tr>
<tr>
<td>9</td>
<td>38.773</td>
<td>31.337</td>
</tr>
<tr>
<td>10</td>
<td>39.353</td>
<td>30.361</td>
</tr>
<tr>
<td>11</td>
<td>38.313</td>
<td>30.157</td>
</tr>
<tr>
<td>Group 3 12</td>
<td>39.587</td>
<td>31.544</td>
</tr>
<tr>
<td>13</td>
<td>39.256</td>
<td>31.337</td>
</tr>
<tr>
<td>14</td>
<td>39.626</td>
<td>31.492</td>
</tr>
<tr>
<td>15</td>
<td>39.762</td>
<td>31.718</td>
</tr>
<tr>
<td>Group 4 16</td>
<td>39.509</td>
<td>31.062</td>
</tr>
<tr>
<td>17</td>
<td>39.762</td>
<td>31.718</td>
</tr>
<tr>
<td>18</td>
<td>39.043</td>
<td>31.251</td>
</tr>
<tr>
<td>19</td>
<td>39.353</td>
<td>31.614</td>
</tr>
<tr>
<td>20</td>
<td>39.860</td>
<td>31.320</td>
</tr>
</tbody>
</table>

Following dead-weight consolidation, these specimens were back saturated and triaxially consolidated to specific effective confining stresses. A decrease in void ratio for the specimens can be expected due to the volume change at the triaxial consolidation stage and the final void ratio can be calculated by measuring the total volume of water extruded from the specimen. The final void ratios of CPB specimens and their corresponding effective confining stresses are also shown in this table. The last column of this table shows the type of test considered for each specimen. It should be noted that data is given for all specimens that were successfully triaxially
consolidated, although not all of these were successfully tested in triaxial shear. The results of monotonic and cyclic tests will be presented in the following chapters.

6.2. Isotropic Consolidation Curves

The data presented in Table 6-1 was used to obtain isotropic consolidation curves for CPB. Crowder (2004) and le Roux (2004) showed that the initial void ratio is sensitive to small amounts of entrained air resulting from the mixing process, and that this subsequently influences void ratios during consolidation. Le Roux (2004) chose to normalize all void ratios with respect to the void ratio at the start of the test. Here, instead the specimens were divided into four groups with regard to their initial void ratios (i.e., void ratios after dead-weight consolidation). The intervals of the initial void ratio for Groups 1, 2, 3 and 4 are 0.840-0.845, 0.850-0.855, 0.860-0.865, and 0.870-0.875, respectively. Therefore, Group 1 shows the relatively densest state, while Group 4 shows the relatively loosest state. The final void ratios of each group can then be plotted versus corresponding effective confining stress. Note that an average value for the final void ratios was used for those groups that have more than two specimens at the same effective confining stress. Figure 6-1 shows the relationship between the average final void ratio and effective confining stress for the isotropic consolidation of each CPB group. Since the maximum effective confining stress for the triaxial tests was about 150 kPa and limited to one specimen, the relationship beyond this value was not obtained. Note that the void ratio has been presented in an expanded scale in this figure in order to show the differences between these groups.

Generally, the void ratio decreases as the effective confining stress increases in each group. The average decrease in void ratio is about 0.060±0.005 for all groups as the effective confining stress increases from 30 kPa to 100 kPa. The logarithmic trend lines fit to these data points show a similar trend for all groups with high R² (coefficient of determination) values of more than 0.90 (Figure 6-1). The compression indexes, Cc, calculated based on these trend lines (i.e., the coefficients of trend line) for Groups 1-4 are 0.10, 0.11, 0.12, and 0.11, respectively. This suggests that the compression index is almost the same (0.11±0.01) for all groups within the range of the effective confining stresses used in this study.

Vick (1990) summarized typical values for the compression index of sand and slime tailings, determined in one-dimensional consolidation tests by different researchers. Generally, Cc for sand tailings (0.05-0.1) represented by Vick (1990) is lower than Cc for slime tailings (0.2-0.3).
Note that the initial void ratios of these materials (1-1.7) are mostly high. Since the CPB specimens tested in this study are generally in a dense state (0.840-0.875), the comparison between these results and the results presented by Vick (1990) might be difficult. However, the compression index of CPB determined in the triaxial consolidation test is within the range of the compression index of sand tailings with high void ratios and slimes tailings with low void ratios (e.g., \( C_c = 0.1 \) for Lead-zinc slimes, \( e = 0.7 \)).

![Graph showing isotropic consolidation curves for CPB.](image)

**Figure 6-1: Isotropic consolidation curves for CPB.**

In addition, Crowder (2004) showed the results of small-strain one-dimensional consolidation tests for four different non-plastic silt sized tailings pastes (i.e., similar to the tailings in this study in terms of particle size distribution and mineralogy) using the standard one-dimensional oedometer test (Figure 6-2). It is possible to see that the consolidation curves do not show the well-defined break between “recompression” and “virgin compression” parts of the loading curves, which is the typical response for over-consolidated soils. To make a comparison, the results presented in Figure 6-1 are also shown in Figure 6-2. The compression indexes, \( C_c \), calculated based on the trend lines for the high water content and low water content specimens.
are 0.12 and 0.10, respectively. This result suggests that the compression indexes of CPB (i.e., 0.11±0.01) triaxially tested in this study are similar to those of uncemented tailings tested in the one-dimensional consolidation test by Crowder (2004).

![Laboratory consolidation curves for the tailings and the CPB.](image)

**Figure 6-2: Laboratory consolidation curves for the tailings and the CPB.**

To draw a better comparison, it is possible to normalize the laboratory consolidation curves using “intrinsic” properties. Burland (1990) suggested intrinsic properties for normally consolidated clays, which are independent of their *in-situ* state. Burland (1990) defined the intrinsic compression index, $C_c^*$, as follows:

$$
C_c^* = \frac{e_{100}^* - e_{1000}^*}{\log(1000/100)} = e_{100}^* - e_{1000}^*,
$$

(6.1)

where $e_{100}^*$ and $e_{1000}^*$ are the intrinsic void ratios corresponding to effective stresses of 100 and 1000 kPa, respectively. The laboratory consolidation curves can then be normalized using the void index, $I_v$, defined as:
\[ I_v = \frac{e - e^{*}_{100}}{C^{*}_c}. \quad (6.2) \]

However, the intrinsic compression index cannot strictly be calculated in this study because the laboratory consolidation curves were not obtained at the effective stress of 1000 kPa. Instead, the compression index of CPB was used, as determined earlier in this section. The void indexes for the tailings were also obtained based on the intrinsic compression index, \( C^{*}_c \), rather than \( C_c \) because the data were available for the tailings at the effective stress of 1000 kPa. Figure 6-3 shows the relationship between the void index and effective stress for both CPB and the tailings in a logarithmic scale. As shown in this figure, there is a strong correlation between the four groups of CPB data as a unique curve with an \( R^2 \) value of 0.96. There is also a good agreement between the void index of these four groups and the tailings (correlation coefficient > 0.99). However, the slight difference between these two sets of data might be related to applying different compression indexes for calculating the void index.

Figure 6-3: Normalized isotropic consolidation curves for the CPB and the tailings.
6.3. Compressibility of Sand versus CPB

Ishihara (1993) showed the variations in void ratio for loose to dense Toyoura sand specimens prepared by different laboratory techniques. He showed that the different preparation techniques result in different initial void ratios for the specimens. Moist tamping might produce specimens with higher void ratios (i.e., e > 1 for Toyoura sand), while dry deposition and water pluviation techniques generate specimens with lower void ratios (e = 0.9-0.61). Generally, loose Toyoura sand shows higher compressibility than dense Toyoura sand. The void ratio of loose sand changed from an initial value of 1.040 at 10 kPa to 0.875 at 3000 kPa, while the void ratio of dense sand with an initial void ratio of 0.850 (i.e., similar to the initial void ratio of CPB specimens in this study) changed to 0.785 when the stress changed from 10 kPa to 3000 kPa. Note that the void ratio of Toyoura sand does not significantly change within the range of stresses tested for CPB in this study (20-200 kPa).

Similar behaviour has been shown for Ottawa sand. Sasitharan (1994) showed the e-log p plot of Ottawa sand specimens prepared by different techniques (Figure 6-4).

![Figure 6-4: Compressibility of Ottawa sand versus CPB from the Williams Mine (MT, moist tamped; WP, water pluviated.](image-url)
The moist tamping technique produced specimens with higher void ratios than the water pluviation technique. The specimens prepared by these techniques might have different initial void ratios. The Ottawa sand specimens with different initial void ratios have shown similar trends, as it was shown for CPB specimen groups. However, the compressibility of Ottawa sand is less than CPB, as shown in this figure. This might explain the differences in the monotonic behaviour of sands and silt-sized CPB. More details on the monotonic behaviour of CPB will be discussed in the following chapter.

6.4. Specimen Anisotropy

The initial anisotropy of the specimens during triaxial consolidation phase of triaxial testing can be investigated by determining the volumetric strain, $\varepsilon_v$, and axial strain, $\varepsilon_a$, responses of the material. A completely isotropic behaviour during hydrostatic loading results in a volumetric-to-axial strain ratio ($\Delta \varepsilon_v / \Delta \varepsilon_a$) of 3.0. However, the response of the uncemented mine tailings and CPB during the consolidation phase in this study suggested a higher ratio of between 3.0 and 3.5. Figure 6-5 shows the strain response of 2 mine tailings specimens tested in this study, which is comparable with that of rock paste tested by Khalili (2009).

![Figure 6-5: Volumetric strain versus axial strain of the uncemented mine tailings (after Khalili, 2009)](image-url)
6.5. Summary and Conclusions

The compression characteristics of CPB were evaluated by data obtained from the consolidation stage of triaxial tests. The compression index of CPB (0.11±0.01) cured for four hours in this study was about the same as uncemented silt sized tailings with similar void ratio or the same as sand tailings with higher void ratios. The results show that the initial void ratio is an effective parameter for determining the isotropic consolidation curves. The intrinsic compression index, which is originally used for clays, can be applicable for CPB and tailings for normalizing the consolidation curves.
7. Monotonic Undrained Test Results

A series of monotonic triaxial tests were conducted to investigate material behaviour under monotonic loads, and in particular the potential for flow liquefaction. Table 7-1 presents the detailed testing program and key test parameters for the consolidated undrained (CU) monotonic triaxial experiments. For all CPB specimens the percentage of Portland cement is 3% and the specimens were cured for 4 hours at the start of shearing. The experiments have been performed at different effective confining stresses (20-400 kPa) and void ratios (0.675-0.890) when the specimens were normally consolidated, as described in Chapter 6. This might help to understand the effect of these two parameters on the CPB response.

Since the axial strain rate used for the CPB (i.e., 2 %/min) is higher than the typical axial strain rates used for similar materials without cement, the effect of axial strain rate on the response of mine tailings will be investigated first in this chapter. The response of uncemented mine tailings will be presented at different effective confining stresses. The response of CPB to compression and extension monotonic loadings will then be presented. A comparison between the monotonic response of the uncemented tailings and that of CPB will be made in order to assess the effect of cementation after 4 hours on the behaviour of the material.
### Table 7-1: CU monotonic triaxial testing program and key test parameters.

<table>
<thead>
<tr>
<th>Material</th>
<th>Test No.</th>
<th>Effective Confining Stress (kPa)</th>
<th>Void Ratio at Start of Shearing (±0.005)</th>
<th>Axial Strain Rate</th>
<th>Type of Test</th>
<th>Curing time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncemented Mine Tailings</td>
<td>WMTM1</td>
<td>50</td>
<td>0.750</td>
<td>2% /min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM2</td>
<td>100</td>
<td>0.725</td>
<td>2% /min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM3</td>
<td>150</td>
<td>0.730</td>
<td>2% /min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM4</td>
<td>400</td>
<td>0.675</td>
<td>2% /min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM5</td>
<td>400</td>
<td>0.685</td>
<td>1% /min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM6</td>
<td>400</td>
<td>0.690</td>
<td>0.5%/min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>WMTM7</td>
<td>400</td>
<td>0.680</td>
<td>0.1%/min</td>
<td>Compression</td>
<td>-</td>
</tr>
<tr>
<td>Cemented Paste Backfill</td>
<td>WCPBM1</td>
<td>20</td>
<td>0.840</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td>cured for 4 hours</td>
<td>WCPBM2</td>
<td>30</td>
<td>0.805</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM3</td>
<td>50</td>
<td>0.840</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM4</td>
<td>50</td>
<td>0.890</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM5</td>
<td>100</td>
<td>0.805</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM6</td>
<td>200</td>
<td>0.775</td>
<td>2% /min</td>
<td>Compression</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM7</td>
<td>50</td>
<td>0.790</td>
<td>2% /min</td>
<td>Extension</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM8</td>
<td>100</td>
<td>0.770</td>
<td>2% /min</td>
<td>Extension</td>
<td>4 hours</td>
</tr>
<tr>
<td></td>
<td>WCPBM9</td>
<td>150</td>
<td>0.770</td>
<td>2% /min</td>
<td>Extension</td>
<td>4 hours</td>
</tr>
</tbody>
</table>

#### 7.1. Effect of Axial Strain Rate on Monotonic Behaviour of Uncemented Tailings

In this thesis, stress path results will be plotted in \((\sigma'_1+\sigma'_3)/2\) and \((\sigma'_1-\sigma'_3)/2\) space. The stress points based on these invariants can be interpreted as the top of a corresponding stress circle in Mohr stress space. State lines of inclination \(\alpha'\) are the locus of these stress invariants. In contrast, state lines of inclination \(\varphi'\) are tangent to the Mohr’s circle. The two angles can be related by \(\tan \alpha' = \sin \varphi'\). Some results are reported in the literature in terms of invariants of the stress tensor, \(p' = (\sigma'_1+2\sigma'_3)/3\) and \(q = (\sigma'_1-\sigma'_3)\), and the slopes of the corresponding state lines in this stress space conventionally denoted using the symbol \(M\). The stress invariant ratio \(M\) is obtained from \(M_c = 6 \sin \varphi'_c/(3-\sin \varphi'_c)\) in compression and \(M_e = 6 \sin \varphi'_e/(3+\sin \varphi'_e)\) in extension in which \(\varphi'_c\) and \(\varphi'_e\) are friction angles at the steady state in compression and extension, respectively. Hereafter, the conversions from \(\alpha'\) to \(\varphi'\) and from \(\alpha'\) or \(\varphi'\) to \(M\) will be given where appropriate.
7.1.1. Monotonic Test Results at Different Strain Rates

As described in Chapter 5, the triaxial monotonic tests on CPB are intended to be performed at a relatively high axial strain rate (2%/min) in this study to minimize the effect of cement hydration on the monotonic response of CPB during the time frame of the experiment. To investigate the effect of axial strain rate, a series of triaxial compression tests were performed on the uncemented tailings at an effective confining stress of 400 kPa. This stress level was chosen simply because it was possible to obtain specimens at relatively similar starting void ratios (~0.680±0.005).

Figure 7-1 (a-d) shows the monotonic response of four tailings specimens (WMTM4, WMTM5, WMTM6, and WMTM7) tested at different axial strain rates (2, 1, 0.5, and 0.1 %/min, respectively). Figure 7-1a shows the stress paths of the specimens. All the specimens exhibit both contractive and dilative behaviours. The contractive behaviour of the specimens is accompanied by increasing pore pressure with axial straining, resulting in deviation of the stress path from the hypothetical drained path. The contractive behaviour changes to a dilative behaviour at the maximum excess pore pressure (Figure 7-1b), corresponding to the phase transformation point (PT point) shown on the stress path. The stress state corresponding to maximum excess pore water pressure, Δu_{max}, is also plotted in Figure 7-1a. Beyond the PT point, the pore pressure ratio decreases with continued axial strain, indicating a dilative behaviour. Determination of Δu_{max}, and therefore the PT point, was unique for each of the datasets (i.e., Δu increased monotonically to Δu_{max}, and then decreased monotonically). A good statistical fit of the phase transformation line (PTL) to the PT data was obtained with a corresponding angle of α^{PT} = 32.2° in the stress space (Figure 7-1a). The corresponding angle in Mohr’s stress space is φ^{PT} = 39.0°.

Determining the “failure envelope” for this material is more problematic. As shown in Figure 7-1c, none of the deviator stresses of these specimens reaches either a peak or a steady state within the maximum axial strain range used in testing; nor does the excess pore water pressure, Δu, asymptotically approach zero. However, the stress paths coincide in a unique line at their dilative states, as shown in Figure 7-1a. This unique line corresponds to a temporary constant stress ratio around the maximum stress obliquity (i.e., maximum of (σ_{1}'-σ_{3}')/ (σ_{1}'+σ_{3}')), as shown in Figure 7-1d. To obtain this unique line, named as the constant stress ratio line (CSRL), a good statistical fit is applied to the data points at which the maximum stress obliquity is reached. Determination of the maximum stress obliquity point was unique for each of the data
sets, as demonstrated in Figure 7-1d. The CSRL fit to these points has an angle of $\alpha_{\text{CSRL}} = 33.3^\circ$ in the invariant stress space with an equivalent Mohr angle of $\varphi_{\text{CSRL}} = 41.1^\circ$.

Two of those four specimens (WMTM4 and WMTM7) shown in Figure 7-1 were tested to higher axial strains. It is possible to see that the stress-strain curves almost linearly increase after phase transformation points, particularly between 5% and 15% axial strain (Figure 7-1c). None of these have plateaus; they have not reached either a peak or a steady state. To better understand the behaviour of these two specimens at high axial strains, the stress paths are shown in Figure 7-2a. It is possible to see that the stress paths deviate from the constant stress ratio line at the axial strains higher than 7.6%. In other words, the stress obliquity reaches a peak value and temporary remains constant; then decreases as axial strain increases (Figure 7-1d). For example, the stress obliquity for specimen WMTM4 decreases about 0.04 (corresponding to an angle of $2.3^\circ$) from its peak value after experiencing 20% axial strain. Further details will be discussed in the following section.

To calculate the deviator stress in this study, uniform lateral deformation has been considered. In other words, it was considered that the area of the specimens increases uniformly along the axis of the specimen, as axial strain increases. Therefore, the calculated deviator stress is larger than the deviator stress related to the actual bulged shape deformation of the specimens. A semi-symmetrical bulge shape deformation of specimen WMTM4 tested at 2%/min axial strain after experiencing 25% axial strain is shown in Figure 7-2b. For this specimen, a sheared plane can be recognized with an angle of about $\theta = 66^\circ$ (determined graphically) from horizontal corresponding to a theoretical friction angle of $\varphi' = 42.0^\circ$ ($\theta = 45 + \varphi'/2$). The corresponding angle of “failure line” is $\alpha' = 33.8^\circ$ in the stress space, as shown in Figure 7-2a. The graphically determined $\alpha' = 33.8^\circ$ can be considered the same as the stress-path determined $\alpha_{\text{CSRL}} = 33.3^\circ$, to within experimental error.
Figure 7-1: Monotonic response of the uncemented mine tailings at different axial strain rates. a) Stress path, b) pore pressure ratio, c) deviator stress versus axial strain, and d) stress obliquity versus axial strain.
Figure 7-2: a) Stress path of two tailings specimens (WMTM4 and WMTM7 at 2%/min and 0.1%/min axial strain rates, respectively), b) the shape of the uncemented tailings specimen (WMTM4-2%/min axial strain rate) after experiencing 25% axial strain.
7.1.2. Discussion of Effect of Strain Rate

All the specimens tested at different axial strain rates approach the same constant stress ratio line after passing through the phase transformation point. In other words, the different axial strain rates tested (0.1-2%/min) have no effect on the Mohr friction angle ($\phi_{\text{CSRL}}$) and the angle at the phase transformation state ($\phi_{\text{PT}}$). Although each stress path follows a similar trend, the lower the axial strain rate is the lower the stress path is located in the stress space.

On the other hand, the stress paths for the specimens with low axial strain rates start deviating from the hypothetical drained path earlier than the specimen tested at high axial strain rates. Also, the highest pore pressure ratio belongs to the specimen tested at the lowest strain rate. This suggests that the generation of pore water pressure is fully developed in the specimens tested at lowest axial strain rates. The higher the axial strain rate, the lower the pore pressure ratio.

In addition, although the uncemented tailings specimens exhibit different stress paths, the strain rate has no effect on the monotonic behaviour of the uncemented tailings. In other words, all the specimens show strain hardening type of behaviour within limits of void ratio and strain rates used for these tests. The strain hardening behaviour of the uncemented tailings will be discussed more in the following section.

The constant stress ratio line was uniquely determined by fitting a line to the maximum stress obliquity points. The maximum stress obliquity for each test occurs at different axial strains and remains constant temporarily. The lower the axial strain rate that is used for the test, the lower the axial strain that is required to reach the maximum stress obliquity. The stress obliquity slightly decreases after the peak value as axial strain increases. However, the stress obliquity curves do not reach a steady state even at high axial strains. This can also be seen in the stress space. For example, the stress paths of the specimens tested at 2%/min and 0.1%/min approach the constant stress ratio line. However, these stress paths exhibit a deviation from the constant stress ratio line at high axial strains.

Although the stress and strain are non-uniform, the friction angle calculated using the angle of failure plane obtained from an actual sample, $\phi_{f}$, is consistent with the angle of the constant stress ratio line ($\phi_{\text{CSRL}}$). It was also possible to observe that $\phi_{\text{PT}}$ is close to $\phi_{\text{CSRL}}$ for these specimens.
Since the axial strain rate has no significant effect on the Mohr friction angle and the slope of phase transformation line, the axial strain of 2%/min is used in this study to minimize the effect of cement hydration on the response of the material during testing. Using 2%/min as the axial strain rate, the following sections present the results of triaxial monotonic tests for uncemented mine tailings and CPB at different effective confining stresses.

### 7.2. Monotonic Test Results for Uncemented Mine Tailings

The monotonic result of four tailings specimens (i.e., WMTM1, WMTM2, WMTM3, and WMTM4) at different effective confining stresses (50, 100, 150, and 400 kPa, respectively) is presented in this section. The void ratios of the specimens tested at 50-150 kPa range from 0.725 to 0.750, while the void ratio of the specimen tested at 400 kPa is 0.675. The stress path of these specimens is shown in Figure 7-3a, the pore pressure ratio versus axial strain is shown Figure 7-3b, the deviator stress versus axial strain is shown in Figure 7-3c, and the stress obliquity versus axial strain is shown in Figure 7-3d.

All the specimens show contractive and dilative behaviours within the range of void ratio and effective confining stresses tested. The contractive behaviour changes to a dilative behaviour at the phase transformation point on the stress path. At this point, the difference between the stress path and hypothetical undrained path becomes maximum (i.e., maximum excess pore pressure ratio $\Delta u_{\text{max}}$). This is shown for the case of the specimen tested at 150 kPa in Figure 7-3a. The phase transformation points can be determined uniquely from pore pressure ratio, as shown in Figure 7-3b (i.e., the maximum value of $\Delta u$). A good statistical fit of the phase transformation line (PTL) to the PT data was obtained with a corresponding angle of $\alpha_{\text{PT}} = 32.0^\circ$ in the stress space (Figure 7-3a). The corresponding angle in Mohr’s stress space at this state is $\phi_{\text{PT}} = 38.7^\circ$.

Beyond the phase transformation points, the stress paths approach a unique line. This line is called the constant stress ratio line although none of the deviator stresses reach a steady state, as shown in Figure 7-3c. A good statistical fit of the constant stress ratio line to the data points at which the maximum stress obliquity is reached was obtained. The maximum stress obliquity points are uniquely determined, as demonstrated in Figure 7-3d. The constant stress ratio line has an angle of $\alpha_{\text{CSRL}} = 33.3^\circ$ in the stress space with a corresponding Mohr friction angle of $\phi_{\text{CSRL}} = 41.1^\circ$. 
Figure 7-3: Monotonic response of uncemented mine tailings at different effective confining stresses. a) Stress path for the uncemented mine tailings, b) pore pressure ratio versus axial strain, c) stress-strain behaviour, and d) stress obliquity versus axial strain.
As shown in Figure 7-3c, the deviator stress increases monotonically as the axial strain increases. This increase indicates a strain hardening behaviour for the uncemented mine tailings. In other words, a peak and post-peak type of behaviour is never observed within the axial strain ranges of testing.

### 7.3. Discussion of Monotonic Behaviour of Uncemented Mine Tailings

Flow liquefaction was not achieved for the normally consolidated uncemented mine tailings in the triaxial consolidated undrained compression monotonic tests within the range of effective stresses (50 to 400 kPa) tested in this study. The compression monotonic response of the tailings showed that the PT line and the constant stress ratio line passes through the origin in the stress space, which is consistent with the purely frictional behaviour expected of a low plasticity index tailings.

The strain hardening type of response identified for the uncemented mine tailings might explain the resistance of the tailings to flow liquefaction in an undrained condition. The stress paths showed that the general behaviour of the tailings was similar at the different effective confining stresses tested in this study. Although there are both contractive and dilative behaviours, there is no sign of initiation of either flow liquefaction or “limited liquefaction”. In the context of liquefaction behaviour of sands (e.g., Kramer 1995), the stress path for the tailings is below any potential flow liquefaction surface (FLS) or steady state point if, in fact, these concepts are even applicable to the material under consideration. This can also be seen in Figure 7-3d where the deviator stress curves at different effective confining stresses do not intersect at a specific point (i.e., steady state point, as suggested by Ishihara (1996) for sands).

A comparison between stress-strain curves of tests at low effective confining stress (50 kPa) and those at high confining stress (400 kPa) shows that they are unlikely to intersect each other even at axial strains, as high as 10%. As described before, the deviator stress curves do not exhibit a peak and post peak response at low axial strains (< 10%). Hyde et al. (2006) presented the similar response for a silt-sized limestone at axial strains lower than 10%. However, the deviator stress curves reach peak values at an axial strain of about 10% depending on the effective confining stress, and subsequently decreased with larger axial strains in their studies.
Unlike the silt-sized tailings in this study, Ishihara (1993) showed that the medium-loose Toyoura sands (i.e., $e = 0.833$, similar to the void ratio of the uncemented tailings specimens) reach a steady state or critical state point at high axial strains (between 25% and 30%). The sand specimens showed fully dilative behaviour at a low effective confining stress of 100 kPa or fully contractive behaviour (flow liquefaction) at a high effective confining stress of 2000 kPa. However, the Toyoura sand specimens consolidated to a void ratio of 0.735 only exhibited dilative behaviour at different effective confining stresses (100-2000 kPa), while reaching a steady state point at a high axial strain of 20%. Ishihara (1993) stated that Toyoura sand consolidated to a void ratio of 0.930 or higher completely liquefies in an undrained triaxial test. Imam et al. (2005) predicted the same behaviour for Toyoura sand by developing a critical-state constitutive model. In another instance, Ishihara (1996) showed that the undrained behaviour of Tia Juana silty sand specimens prepared by the method of dry deposition and consolidated to void ratios between 0.840 and 0.890 can be categorized as strain softening with “limited liquefaction” followed by strain hardening behaviour. For these specimens tested at different effective confining stresses (50-200 kPa), no steady state was reached, similar to the tailings tested in this study.

Chu et al. (2003) showed the typical drained and undrained response of isotropically consolidated medium dense sand specimens with void ratios ranging from 0.643 to 0.695. The stress paths of the sand specimens in undrained conditions approached a constant stress ratio line while showing dilative behaviour similar to uncemented tailings tested in this study. They also showed that the CSRL lies between the critical state line and the failure line obtained from isotropically consolidated drained tests. The slope of the CSRL in the $p^\prime$-$q$ stress space for the sand specimens was $M_{CSRL} = 1.5$ with a corresponding $\phi_{CSRL}$ angle of $36.9^\circ$, which is lower than that for the uncemented tailings ($\phi_{CSRL} = 41.1^\circ$) tested in this study even though the void ratios of the uncemented tailings ($e = 0.675-0.750$) were higher than those of the sands.

Vick (1990) also represented the data from Wahler (1974), and showed that the slime tailings exhibit strain hardening behaviour without reaching a steady state point within the range of axial strain (i.e., 12%) and effective confining stresses tested in that study. As shown by Crowder (2004), three silt-sized mine tailings specimens with an average void ratio of 0.72 also exhibit similar strain hardening behaviour with no limited or flow liquefaction in the triaxial CU test.
In contrast, Al-Tarhuni (2008) has recently reported the flow liquefaction of silt-sized gold mine tailings using the direct simple shear test. He showed that the tailings specimens even with a low void ratio of 0.585±0.01 might be susceptible to flow liquefaction at the high effective confining stress of 400 kPa. In another instance, he also showed that the specimens with a void ratio of 0.660±0.01 experience flow liquefaction at the effective confining stress of 100 kPa. These results are opposite to the results presented in this study. However, the difference in monotonic behaviour of the tailings might be attributed to the differences in laboratory sample preparation techniques and the stress conditions due to the methods of shearing. Al-Tarhuni (2008) was concerned with tailings on surface that were desiccated and re-wetted. This may have influenced sample fabric and subsequent response to direct simple shear. In the case of underground field conditions, the laboratory pre-consolidation technique, described in Chapter 5, is believed to be the best technique to create a representative sample for CPB. In the case of the shearing method, the triaxial monotonic loading rather than direct simple shear test is believed to be more representative of the field stress conditions.

Although there exist few results that show flow liquefaction might occur for mine tailings using the direct simple shear test, the triaxial testing of the uncemented mine tailings showed strain hardening behaviour with no sign of flow liquefaction in this study. Based on this result, the effect of 3% Portland cement on the monotonic response of mine tailings will be investigated in the following section.

7.4. Monotonic Test Results for CPB

7.4.1. Compression Test Results

The monotonic compression test results for five 3% CPB specimens cured for 4 hours before shearing (i.e., WCPBM1, WCPBM2, WCPBM3, WCPBM5, and WCPBM6) at different effective confining stresses (i.e., 20, 30, 50, 100, and 200 kPa, respectively) are presented in this section. The stress paths of these specimens are shown in Figure 7-4a, the pore pressure ratio versus axial strain is shown in Figure 7-4b, the stress obliquity versus axial strain is shown in Figure 7-4c, and the deviator stress versus axial strain is shown in Figure 7-4d.
Figure 7-4: The monotonic compression response of the 3% CPB cured for 4 hours. a) Stress path, b) pore pressure ratio versus axial strain, c) stress obliquity versus axial strain, and d) deviator stress versus axial strain.
Generally, the specimens showed both contractive and dilative behaviours similar to the response of the uncemented mine tailings in this study. The maximum pore pressure ratio corresponding to the phase transformation point for the specimens was determined, as demonstrated in Figure 7-4b. At this point, the difference between the stress path and the hypothetical undrained path becomes maximum (e.g., \( \Delta u_{\text{max}} \) demonstrated for the case of the specimen tested at 100 kPa in Figure 7-4a). A good statistical fit of the phase transformation line (PTL) to the PT data was obtained with a corresponding angle of \( \alpha_{\text{PT}} = 32.7^\circ \) (\( \phi_{\text{PT}} = 39.9^\circ \)) in the stress space.

Beyond the phase transformation point at which the behaviour changes from contractive to dilative, the stress paths of the specimens approach the CSRL. To determine the CSRL, the maximum stress obliquity points were obtained, as demonstrated in Figure 7-4c. A good statistical fit of the constant stress ratio line to these data points was obtained with a corresponding angle of \( \alpha_{\text{CSRL}} = 33.8^\circ \). This angle is corresponding to a Mohr friction angle of \( \phi_{\text{CSRL}} = 42.0^\circ \).

Figure 7-4d also shows that the deviator stress increases monotonically as the axial strain increases, but no steady state is reached. The maximum tested axial strain ranges from 6% to 10% for these specimens. Although the void ratio of the specimens presented in Figure 7-4 varies from 0.775 to 0.840, a unique angle of the CSRL was determined for these specimens. Note that this line was obtained with a good statistical fit (\( R^2 = 0.9999 \)) to the data points at which the maximum stress obliquity is reached suggesting that the effect of void ratio is not significant. However, a more detailed examination suggests a suitable effect of void ratio on the angle of the constant stress ratio line.

For this purpose, a CSRL was separately determined for each specimen. Table 7-2 summarizes the angle of the CSRL and the corresponding void ratio for each specimen. Note that the results are related to the five specimens presented in Figure 7-4a, plus specimen WCPBM4 at 50 kPa effective confining stress and a high void ratio of 0.890. Figure 7-5 shows the angle of the constant stress ratio line versus the corresponding void ratio for each CPB specimen. A statistical fit to these data points except specimen WCPBM2 show a linear relationship between \( \alpha \) and void ratio with a high \( R^2 \) value (0.9319).
Table 7-2: Angle of the constant stress ratio line for each CPB specimen.

<table>
<thead>
<tr>
<th>Specimen, ((3%) CPB cured for 4 hours)</th>
<th>Effective confining stress, kPa</th>
<th>Void ratio, (e) ((\pm0.005))</th>
<th>(\alpha^\prime_{\text{CSRL}}) (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WCPBM1</td>
<td>20</td>
<td>0.840</td>
<td>33.4</td>
</tr>
<tr>
<td>WCPBM2</td>
<td>30</td>
<td>0.805</td>
<td>34.2</td>
</tr>
<tr>
<td>WCPBM3</td>
<td>50</td>
<td>0.840</td>
<td>33.6</td>
</tr>
<tr>
<td>WCPBM4</td>
<td>50</td>
<td>0.890</td>
<td>33.0</td>
</tr>
<tr>
<td>WCPBM5</td>
<td>100</td>
<td>0.805</td>
<td>33.7</td>
</tr>
<tr>
<td>WCPBM6</td>
<td>200</td>
<td>0.775</td>
<td>33.9</td>
</tr>
</tbody>
</table>

Figure 7-5: The effect of void ratio on the angle of the constant stress ratio line for CPB specimens.

Specimen WCPBM2 is considered as an outlier while the addition of this point to the others results in a similar trend with lower slope and \(R^2\) value. In general, this result indicates that the higher the void ratio is, the lower the angle of the constant stress ratio would be.

7.4.2. Extension Test Results

As described in Chapter 5, the subsequent cyclic triaxial testing will include portions of loading in extension. It is therefore important to capture extension loading behaviour of the CPB in monotonic loading as well. The monotonic extension results of three CPB specimens (i.e., WCPBM7, WCPBM8, WCPBM9) at different effective confining stresses (i.e., 50, 100 and 150 kPa, respectively) are presented in this section. The stress paths of the specimens are shown in Figure 7-6a, the pore pressure ratio versus axial strain is shown in Figure 7-6b, the stress obliquity versus axial stain is shown in Figure 7-6c, and the deviator stress versus axial strain is
shown in Figure 7-6d. Note that the void ratio of the specimens in these figures ranges from 0.770 to 0.790.

Generally, the stress paths show both contractive and dilative behaviours, while the contractive behaviour is accompanied by a temporary instability (TI) state. Both the phase transformation and temporary instability points are shown in Figure 7-6a. The phase transformation points are strictly determined as the maximum pore pressure ratio, as demonstrated in Figure 7-6b. The temporary instability points can be defined as the temporary peak values for deviator stress, as demonstrated in Figure 7-6d. In other words, this figure shows that the deviator stress exhibits a local peak value at the temporary instability point. The deviator stress then monotonically increases after passing the phase transformation point. Figure 7-6d also shows that none of the deviator stresses reach a steady state.

A good statistical fit of the PTL to the PT points was obtained with a corresponding angle of $\alpha'_{PT} = 27.6^\circ$ ($\phi'_{PT} = 31.5^\circ$) in the stress space. Similar fit of the temporary instability line (TIL) to the TI data was obtained with a corresponding angle of $\alpha'_{TI} = 22.1^\circ$, as shown in Figure 7-6. This figure also shows that the stress paths approach the CSRL after passing the phase transformation points by showing a dilative behaviour. To obtain the CSRL, the maximum stress obliquity points were determined, as demonstrated in Figure 7-6c. A good statistical fit of the constant stress ratio line to these data points on the stress path was obtained with a corresponding angle of $\alpha'_{CSRL} = 32.9^\circ$ ($\phi'_{CSRL} = 40.3^\circ$) for the CPB in extension tests.

7.5. Discussion of Monotonic Behaviour of CPB

7.5.1. CPB in Compression

Flow liquefaction was not achieved for the 3% CPB specimens cured for four hours in the undrained compression monotonic tests in this study. These normally consolidated CPB specimens showed both contractive and dilative behaviour at different void ratios (i.e., 0.805-0.890) and initial effective confining stresses (20-200 kPa) tested in this study. This result is consistent with the result of the same mine tailings without cement tested in this study.
Figure 7-6: The monotonic extension response of 3% CPB cured for 4 hours. a) Stress path, b) pore pressure ratio versus axial strain, c) stress ratio versus axial strain, and d) deviator stress versus axial strain.
The CSRL and the PTL determined for the 3% CPB cured for four hours passed through the origin of the stress path space. This indicates that the monotonic behaviour of CPB after four hours of curing has no cohesion, the same as for uncemented mine tailings or purely frictional soils. However, the angle of the CSRL and the PTL for the CPB is higher than that for the uncemented mine tailings. In other words, the addition of 3% cement and 4 hours cure would increase $\alpha_{\text{CSRL}}$ and $\alpha_{\text{PT}}$, but not cohesion. This result is consistent with the results of Vicat needle test and EC measurements presented in Section 5.2.1. As the electrical conductivity and Vicat needle results showed, the critical time is 5 hours after the addition of cement where the acceleration phase of hydration begins. Therefore, no significant amount of hydration products is formed at four hours where the CPB specimens tested, resulting in no development of cohesion. Nevertheless, the addition of 3% cement results in an increase in the percentage of fines in the CPB specimens in comparison with the uncemented mine tailings, and consequently the packing density of CPB increases. Therefore, the higher packing density might be responsible for having higher friction angle for CPB.

As mentioned previously, to obtain the CSRL, the data points at which the maximum stress obliquity is reached was used. The maximum stress obliquity is reached at different axial strains for the CPB specimens in compression. The lower the effective confining stress used for the tests, the lower the strain level required to reach the maximum value. For CPB, the strain level at the maximum stress obliquity ranges between 2-7% for the specimens tested between 20-200 kPa. In contrast, the maximum stress obliquity is reached between 6-9% for uncemented mine tailings. For both CPB and uncemented tailings the stress obliquity decreases slightly after the maximum value is obtained; but this is generally less than 0.01 (corresponding to $\Delta \alpha^{'} = 0.6^\circ$) up to 10% axial strain.

Similar to the uncemented mine tailings, the CPB specimens in compression monotonic tests also exhibit strain hardening behaviour, while no steady state is observed within the axial strain tested (i.e., $\sim 10\%$). As explained in the previous section for the uncemented mine tailings, the stress path for CPB in compression tests is also the below temporary instability line or flow liquefaction surface, if these concepts apply, since the CPB specimens do not exhibit temporary instability during the contractive phase.
7.5.2. CPB in Extension

Flow liquefaction was also not achieved for the CPB specimens in extension. However, the undrained extension response of CPB is slightly different from the compression response. Although the deviator stress curves do not reach a steady state in extension, they show a strain softening type of behaviour with temporary instability. This temporary instability type of behaviour is similar to the behaviour of loose clean sand suggested by Yamamuro and Covert (2001). Note that the temporary instability line is not a unique feature for contractive materials and might be also observed for dilative material (dense sand) under some conditions. For example, Vaid and Eliadorani (1998) observed that dilative sand with a strain hardening behaviour under undrained condition can be transformed into a contractive material if a small amount of drainage into the sample is allowed.

Similar to the behaviour of CPB in compression, the temporary instability line, the PTL and the CSRL passed through the origin of the stress path space in the extension tests. This suggests that the 3% CPB after 4 hours of curing in extension exhibits a purely frictional behaviour. Note that the PTL in compression is closer to the CSRL than in extension. The angle or slope of the CSRL and the PTL in extension is lower than those in compression. For example, the slopes \( \tan \alpha_{\text{CSRL}} \) of the CSRL for the CPB specimens are 0.671 and 0.647 in compression and extension, respectively (i.e., \( M_c = 1.73 \) and \( M_e = 1.06 \) in compression and extension in the \( p' - q \) stress space).

Hyodo et al. (1994) showed that the undrained behaviour of Toyoura sand in extension is different from compression. In their studies, although the sand specimen at 100 kPa exhibited stain softening behaviour with limited liquefaction in the compression test, the specimen showed flow liquefaction behaviour in the extension test. The difference between the behaviour of the specimens in compression and extension has been attributed to specimen anisotropy induced during sample preparation. Imam et al. (2005) also compared the undrained triaxial extension behaviour of medium loose Toyoura sand consolidated to a void ratio of 0.802 to 0.817 and tested at confining stresses between 50 kPa and 500 kPa with their critical-state constitutive model. They showed that the sand specimens exhibit both contractive and dilative behaviours similar to the CPB specimens in extension although there is a drop in stress ratio predicted by the model at high axial strains. Imam et al. (2005) also showed that complete liquefaction in undrained triaxial extension might occur for Toyoura sand consolidated to void ratios higher than
0.860 and tested at confining stresses between 50 kPa and 500 kPa. They noted that zero strength is reached at or before PT in triaxial extension, whereas zero strength is reached at critical state in triaxial compression. In addition, larger contraction takes place before the PT is reached in triaxial extension, compared with triaxial compression. Vaid and Thomas (1995) also showed that the deviator stress beyond the peak value in extension drops more significantly than in compression for Fraser sand specimens, which were formed by the water pluviation technique. They also attributed the difference in the undrained responses between extension and compression to anisotropy of the specimens due to the sample preparation technique.

Boukpeti et al. (2002) developed an elastoplastic model for predicting the undrained triaxial response of soils in extension and compression. This model also suggests a slope of $3M/(3+M)$ for the steady state line in the $p'-q$ stress space (i.e., different from the “stress invariant” type of stress space used in this chapter) in triaxial extension, which is lower than that in triaxial compression (i.e., $M = \text{slope of steady state line in compression}$). To examine the applicability of this model for the CPB, it is possible to compare the actual slope of the CSRL in extension with the calculated one using the actual slope of the line in compression based on this model.

Based on the test results for CPB, the slope of the CSRL in compression in the $p'-q$ space is $M_c = 1.73$ and in extension is $M_e = 1.06$. However, the slope of the CSRL calculated based on this model in extension is $3M/(3+M) = 1.09$ for an $M = 1.73$. The difference between the calculated value (i.e., 1.09) and the actual value ($M_e = 1.06$) for the CPB in extension is 0.03, which is not significant. In other words, the model slightly overestimates the equivalent friction angle in extension with a deviation of 1.7°.

As explained above, to interpret the difference between the behaviour for soils in extension and compression, the specimen anisotropy due to laboratory sample preparation technique has been noted. In other words, the response of soil in extension might not be an actual response since the natural deposition of soil is different from the laboratory sample preparation techniques. In the case of CPB, however, the sample preparation method has been developed based on the actual placement of CPB in the field; therefore, the response of CPB in an extension test is most likely to be the same as actual response in the field.
7.6. Summary and Conclusions

7.6.1. Uncemented Mine Tailings

1. Neither flow liquefaction nor temporary liquefaction was achieved for the silt-sized, normally consolidated, uncemented mine tailings tested at different effective confining stresses (50-400 kPa) in this study.

2. The uncemented mine tailings exhibit strain hardening behaviour in the undrained compression monotonic tests.

3. The deviator stress and pore pressure ratio curves do not reach a plateau even at high axial strains (i.e., 25%), as shown in Figure 7-1c and b, respectively, indicating a steady state has not been reached.

4. The angle of the shear plane appearing through the sample at high axial strain is in good agreement with the theoretical angle of conjugate shear planes calculated from stress path space (Figure 7-2).

5. Axial strain rates (i.e., 0.1-2%/min) have no significant effect on the state lines (i.e., PTL and constant stress ratio line), as shown in Figure 7-1.

7.6.2. CPB

1. Flow liquefaction was not achieved for the 3% CPB specimens cured for four hours at different effective confining stresses (i.e., 20-200 kPa) in this study.

2. The stress paths for the CPB specimens exhibit initial contractive behaviour followed by dilation. The response of CPB to the undrained monotonic loading is strain hardening in compression (Figure 7-4a) and strain softening with temporary instability followed by strain hardening in extension (Figure 7-6a).

3. None of the deviator stresses reached a steady state (Figure 7-4d). The stress paths approach a unique line, the constant stress ratio line, at their maximum stress ratio points.
4. The PTL appears to be very close to the constant stress ratio line. The PT points were uniquely picked from pore pressure ratio graphs (e.g., Figure 7-4b) and the PTL was obtained with good statistical fit to these points.

5. The angle of the constant stress ratio line depends slightly on the void ratio; the higher the void ratio of the specimen is, the lower is the friction angle (Figure 7-5).

6. The angle of the constant stress ratio line for CPB is slightly higher than that of the uncemented mine tailings at the same void ratio.

7. The angle of the constant stress ratio line in compression is higher than that in extension.

7.6.3. Monotonic Liquefaction Susceptibility

1. The 3% CPB specimens cured for four hours and uncemented mine tailings are not susceptible to flow liquefaction.

2. The addition of 3% Portland cement improved the state parameters of the material (i.e., \( \alpha'_{CSRL} \) and \( \alpha'_{PT} \)) at four hours, which is even before the onset of acceleration phase of hydration (i.e., 5 hours); resistance to liquefaction increases as the acceleration phase begins.
CHAPTER 8

CYCLIC TEST RESULTS

8. Cyclic Test Results

A total of 24 cyclic triaxial tests were successfully conducted under undrained conditions to investigate the liquefaction potential of both CPB and uncemented mine tailings. Table 8-1 presents the detailed testing program and key test parameters for the CU cyclic triaxial experiments. The normally consolidated specimens were tested at different cyclic stress ratios (CSR’s) ranging from 0.18 to 0.3 and three different effective confining stresses (i.e., 30, 50, and 100 kPa). Note that for all CPB specimens, the percentage of Portland cement is 3% and the CPB specimens were cured for 4 hours except the test number WCPB-CY10, which was cured for 12 hours. The cyclic test results of uncemented mine tailings will be presented first. The response of CPB to cyclic loading will then be presented. The cyclic resistance of materials will be evaluated and the overall liquefaction susceptibility will be investigated under cyclic loading. The applicability of liquefaction susceptibility criteria for silts described in Chapter 3 will be examined for the uncemented mine tailings and CPB at the end of this chapter. Note that all the specimens are isotropically consolidated. In other words, the effect of initial static shear stress bias on the cyclic response of the materials will not be addressed in this thesis.

8.1. Uncemented Mine Tailings

8.1.1. Cyclic Test Results of Tailings

The response of specimens WMT-CY6 at $\sigma_c^{'}=100$ kPa and CSR = 0.15, and WMT-CY1 at $\sigma_c^{'}=50$ kPa and CSR = 0.24 will be presented as two typical responses of the uncemented mine
tailings at relatively low and high CSR’s. The responses from the remaining test results of the un cemented mine tailings specimens are presented in Appendix C.

Table 8-1: CU cyclic triaxial testing program and key test parameters.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>CSR</th>
<th>Effective Confining Stress(kPa)</th>
<th>Void Ratio at start of test (±0.05)</th>
<th>Number of Cycles at 5% DA axial strain*</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMT-CY1</td>
<td>0.24</td>
<td>50</td>
<td>0.725</td>
<td>3</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY2</td>
<td>0.22</td>
<td>50</td>
<td>0.730</td>
<td>5</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY3</td>
<td>0.16</td>
<td>50</td>
<td>0.730</td>
<td>19</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY4</td>
<td>0.18</td>
<td>50</td>
<td>0.760</td>
<td>9</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY5</td>
<td>0.13</td>
<td>50</td>
<td>0.750</td>
<td>Not reached at 60</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY6</td>
<td>0.15</td>
<td>100</td>
<td>0.735</td>
<td>26</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY7</td>
<td>0.20</td>
<td>100</td>
<td>0.745</td>
<td>8</td>
<td>tailings</td>
</tr>
<tr>
<td>WMT-CY8</td>
<td>0.18</td>
<td>100</td>
<td>0.715</td>
<td>12</td>
<td>tailings</td>
</tr>
<tr>
<td>WCPB-CY1</td>
<td>0.21</td>
<td>30</td>
<td>0.840</td>
<td>62</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY2</td>
<td>0.25</td>
<td>30</td>
<td>0.840</td>
<td>15</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY3</td>
<td>0.25</td>
<td>30</td>
<td>0.795</td>
<td>17</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY4</td>
<td>0.24</td>
<td>30</td>
<td>0.840</td>
<td>30</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY5</td>
<td>0.29</td>
<td>50</td>
<td>0.835</td>
<td>5</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY6</td>
<td>0.24</td>
<td>50</td>
<td>0.840</td>
<td>35</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY7</td>
<td>0.21</td>
<td>50</td>
<td>0.835</td>
<td>67</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY8</td>
<td>0.24</td>
<td>50</td>
<td>0.835</td>
<td>34</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY9</td>
<td>0.26</td>
<td>50</td>
<td>0.840</td>
<td>14</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY10</td>
<td>0.29</td>
<td>50</td>
<td>0.835</td>
<td>Not reached at 120</td>
<td>12hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY11</td>
<td>0.18</td>
<td>50</td>
<td>0.835</td>
<td>Not reached at 120</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY12</td>
<td>0.24</td>
<td>100</td>
<td>0.750</td>
<td>41</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY13</td>
<td>0.26</td>
<td>100</td>
<td>0.745</td>
<td>16</td>
<td>4hrs.CPB</td>
</tr>
<tr>
<td>WCPB-CY14</td>
<td>0.30</td>
<td>100</td>
<td>0.735</td>
<td>5</td>
<td>4hrs.CPB</td>
</tr>
</tbody>
</table>

* 5% double amplitude axial strain

Figure 8-1a shows the stress path of the un cemented mine tailings for specimen WMT-CY6. The stress path from the compression monotonic test results, presented in Chapter 7, is also superposed on this stress path (no extension result is available for un cemented mine tailings). Note that the void ratios of the specimens are 0.735 and 0.725 for cyclic and monotonic tests, respectively. Generally, the stress path in this cyclic test moves from the initial effective stress (i.e., 100 kPa) towards the origin of the plot as a result of an increase in pore pressure ratio. Figure 8-1b shows that the pore pressure ratio increases in a progressive manner with number of cycles, while the variation in axial strain for this specimen does not significantly change up to 20 cycles, as shown in Figure 8-1c. However, relatively large axial strains occur after the 20th cycle. Note that the strain development in compression is higher than it is in extension in this cyclic test.
Figure 8-1: Response of uncemented tailings specimen WMT-CY6 tested at CSR = 0.15 and \( \sigma_c' = 100 \) kPa. a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress-strain response.
At the 26th cycle of uniform cyclic loading, the specimen reaches its 5% double amplitude (DA) axial strain, which is used as the criterion in this study to consider the initiation of liquefaction (Ishihara, 1996). The pore pressure ratio at this cycle is more than 0.95, which is slightly lower than 1 where the zero effective confining stress would be achieved. The residual effective stress at this cycle is about 5% of consolidation stress.

The stress strain response of the specimen shown in Figure 8-1d illustrates that Young’s modulus of the specimen decreases as number of cycles increases. This can be explained by calculating the secant Young’s modulus, $E_{\text{secant}}$, at each cycle, as demonstrated in this figure. For example, $E_{\text{secant}} = 8108$ kPa at the 22nd cycle, while $E_{\text{secant}}$ decreases to 1909 kPa in the next cycle.

Figure 8-2(a-d) presents the cyclic response of specimen WMT-CY1 tested at a CSR of 0.24 and an effective confining stress of 50 kPa. Figure 8-2a shows that the stress path moves from the initial effective stress (i.e., 50 kPa) towards the origin of the plot as a result of an increase in pore pressure ratio, but this movement occurs quickly. The stress path from the compression monotonic test results, presented in Chapter 7, is also superposed on this stress path. Figure 8-2b shows that the pore pressure significantly increases during the compression portion of cyclic loading and decreases during the extension.

The specimen reaches 5% DA axial strain after only three cycles under this relatively high CSR, as shown in Figure 8-2c. The maximum pore pressure ratio after 3 cycles is 0.95, which is close to 1 where zero effective stress would be achieved. The residual effective stress at this cycle is about 5% of consolidation stress. The relationship between stress and strain for this specimen is shown in Figure 8-2d. This figure indicates that the modulus of the material dramatically decreases in three cycles. This can also be explained by determining the secant Young’s modulus for each cycle, as described for the previous specimen.
Figure 8-2: Response of uncemented tailings specimen WMT-CY1 tested at CSR = 0.24 and $\sigma_c'=50$ kPa. a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress-strain response.
8.1.2. Cyclic Shear Resistance of Tailings

It is possible to obtain a liquefaction susceptibility chart by using CSR and corresponding number of cycles at 5%DA axial strain for each test presented in Table 8-1. Figure 8-3 shows the liquefaction susceptibility chart for the uncemented mine tailings tested at two different effective confining stresses in this study. Note that the CSR in this case is described as cyclic resistance ratio (CRR) by Youd et al. (2001) since it represents the capacity of the soil to resist cyclic loading. The void ratios of the specimens presented in this figure range between 0.715 and 0.760, as shown in Table 8-1. This figure shows that the number of cycles increases as the CSR decreases. It is noteworthy that the 5% DA axial strain was not achieved for specimen WMT-CY5 after 60 cycles at a CSR of 0.13 and effective confining stress of 50 kPa. In other words, more cycles would be required to achieve the desired axial strain in this case. This shows the high resistance of the specimen to cyclic loading at this CSR.

![Graph showing CSR versus number of cycles to liquefaction for the uncemented mine tailings.](image)

Figure 8-3: CSR versus number of cycles to liquefaction for the uncemented mine tailings.

Generally, the liquefaction susceptibility curves for uncemented mine tailings show the same trend at 50 kPa and 100 kPa effective confining stresses with a coefficient of correlation of 0.99. The specimens tested at 100 kPa more than those tested at 50 kPa show resistance to cyclic...
loading at the same CSR. However, the standard deviation between these two data sets (between 5 and 30 cycles) is not significant (i.e., 0.03).

The cyclic resistance of uncemented mine tailings tested in this study is similar to the cyclic resistance of comparable tailings. For example, the cyclic resistance of Bulyanhulu tailings tested at $\sigma_{c}' = 50$ kPa and $e = 0.65$ (Crowder, 2004) is lower than that of the uncemented tailings tested in this study. In contrast, the cyclic resistance of Old Dike Slime tested at $\sigma_{c}' = 100$ kPa and $e = 0.57$ (Ishihara et al., 1980) is higher than that of the uncemented tailings, as demonstrated in Figure 8-3.

8.1.3. Discussion of Cyclic Response of Uncemented Mine Tailings

The cyclic triaxial tests showed that the uncemented mine tailings exhibit a cyclic mobility type of response. Although the pore pressure develops with number of cycles, zero effective stress is unlikely to be achieved for the uncemented mine tailings specimens tested in this study. The cyclic mobility type of response identified for the uncemented mine tailings is similar to the behaviour of clayey silt and silt mine tailings reported by Wijewickreme et al. (2005). This result is also consistent with the cyclic response of natural fine-grained soils tested by Sanin and Wijewickreme (2006), and Bray and Sancio (2006). Note that none of these authors compared the cyclic results with the monotonic response of the material. The general assumption is that the cyclic and monotonic results should be consistent (Kramer, 1995). The consistency of these two results will be discussed in following paragraphs.

The results of tests on uncemented mine tailings in this study show that the cyclic stress path lies slightly above the constant stress ratio line in the compression test after reaching 5% DA axial strain. There are some studies that show this apparent discrepancy. For example, Boulanger and Truman (1996) showed the cyclic mobility type of response for a sand in a volumetric strain controlled triaxial test, while the specimen was anisotropically consolidated. They showed that the mobilized friction angle of the soil is higher than the critical state friction angle in both compression and extension. Zergoun and Vaid (1994) explained that the stress path of Cloverdale clay in a cyclic test becomes bounded by the compression monotonic loading failure envelope. However, the stress path in the cyclic test lies beyond the extension failure envelope, while the maximum residual effective stress is about 20% of consolidation stress. However, it should be
noted that the stress strain response of the clay to monotonic loading is similar in compression and extension and it is different from the mine tailings tested in this study.

In contrast, there are some studies showing that the cyclic stress path lies below monotonic loading failure lines and are eventually bounded by them. For example, Hyde et al. (2006) showed that the stress path in cyclic loading of a silt sized limestone lies below the failure envelope while the specimen shows similar strain softening type of behaviour in both compression and extension monotonic tests. They also showed that the dramatic changes in the stress path in cyclic loading start when the stress path first hits the initial phase transformation line determined in monotonic loading.

In another instance, Vaid and Sivathayalan (1998) presented the cyclic response of Fraser River sand in comparison with its monotonic response. They showed that the steady state line during cyclic loading is identical to that observed under monotonic loading. They also noted that the strain softening behaviour during cycling loading is responsible for having an identical steady state line. The criteria for exhibiting strain softening behaviour during cyclic loading can be summarized as follows: i) strain softening under monotonic loading; ii) maximum shear stress should exceed the steady state strength in compression or extension; iii) sufficient number of loading cycles is required.

The uncemented mine tailings tested in this study do not meet the above criteria since the material does not exhibit strain softening under compression monotonic loading. In other words, comparisons between the behaviour of sands and clays presented above and the uncemented mine tailings tested in this study show that the strain hardening behaviour of uncemented mine tailings in compression might be responsible for the apparent discrepancy between cyclic and monotonic results. Further details will be discussed after presenting the cyclic test results for CPB in the following section.

8.2. Cemented Paste Backfill

8.2.1. Cyclic Test Results of CPB

The response of CPB for three specimens (i.e., WCPB-CY8, WCPB-CY12, and WCPB-CY4) cured for four hours and one specimen (i.e., WCPB-CY10) cured for 12 hours will be presented in detail in this section. The remaining test results can be found in Appendix C). The first three
specimens were tested at the same CSR of 0.24, while the effective confining stress was 50, 100, and 30 kPa, respectively.

The stress path of specimen WCPB-CY8 with superposition of monotonic loading results is shown in Figure 8-4a. The stress path in this cyclic test moves from the initial effective stress (i.e., 50 kPa) towards the origin of the plot as a result of an increase in pore pressure ratio. The progressive increase in pore pressure ratio with number of cycles for this specimen is shown in Figure 8-4b. The maximum pore pressure ratio is more than 0.91 after 34 cycles, where 5% DA axial strain is reached, as shown in Figure 8-4c. In other words, zero effective stress is not achieved at this point. Note that the axial strain during extension is higher than the axial strain during compression in this cyclic test. Figure 8-4d shows the stress strain relationship for the specimen. This figure indicates a decrease in modulus as number of cycles increases.

In the stress path space (Figure 8-4a), the constant stress ratio, phase transformation and temporary instability lines for both compression and extension testes are also shown. As described in Chapter 7, no temporary instability line was recognized for monotonic compression tests. The stress path in the cyclic test is bounded with the constant stress ratio line in the compression test, while the stress path lies beyond the constant stress ratio line in the extension test after reaching its 5% DA axial strain. The maximum residual effective stress at this point is about 5 kPa, which is about 10% of consolidation stress.

Similar type of response can also be seen for specimen WCPB-CY12 tested at \( \sigma_c'=100 \) kPa. Figure 8-5a shows the stress path of this specimen with superposition of monotonic loading results. The stress path moves towards the origin of the plot as a result of excess pore water pressure. Figure 8-5b shows that the pore pressure ratio progressively changes with cycles and reaches a maximum value of 0.95 at 5% DA axial strain. Figure 8-5c shows that the specimen reaches 5% DA axial strain after 41 cycles. The stress strain relationship for this specimen is also shown in Figure 8-5d. Note that the axial strain during extension is higher than the axial strain during compression.
Figure 8-4: Response of CPB specimen WCPB-CY8 tested at CSR = 0.24 and $\sigma_c' = 50$ kPa. a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress-strain response.
The stress path of this specimen is almost bounded with the constant stress ratio line in compression after reaching its 5% DA axial strain, while the maximum residual effective stress is about 10% of the consolidation stress. However, the stress path lies beyond the constant stress ratio line in extension.

In contrast to the specimens tested at $\sigma_c' = 50$ kPa, and $\sigma_c' = 100$ kPa, the specimens tested at $\sigma_c' = 30$ kPa reached zero effective stress. For example, Figure 8-6 shows the response of specimen WCPB-CY4 tested at $\sigma_c' = 30$ kPa and CSR = 0.24. The stress path moves from the initial effective confining stress towards the origin of the plot and it reaches zero residual effective stress, as shown in Figure 8-6a. The pore pressure ratio at this point becomes unity indicating that zero effective stress is achieved (Figure 8-6b). In other words, the specimen is liquefied after 30 cycles due to an excess pore water pressure of 100%. Figure 8-6c shows that DA axial strain at 30th cycle is less than 3% in this case. The axial strain during extension is higher than the axial strain during compression. The stress strain relationship shown in Figure 8-6d indicates that the shear modulus decreases as the number of cycles or pore pressure ratio increases. The stress path of this specimen lies beyond the constant stress ratio line in both compression and extension after reaching zero effective stress.

Specimen WCPB-CY10 was cured for 12 hours and cyclically tested to investigate the effect of relatively long curing time on the liquefaction potential of CPB. The stress path of this specimen, tested at a relatively high CSR of 0.29, is shown in Figure 8-7a. The stress path does not move toward the origin of the plot as number of cycles increases. Figure 8-7b shows that there is no progressive excess pore water pressure after 120 cycles. The variation in axial strain is also not significant for this specimen and it is not shown in this section. This indicates the resistance of the specimen to cyclic loading.
Figure 8-5: Response of CPB specimen WCPB-CY12 tested at the CSR of 0.24 and $\sigma_c'=100$ kPa. a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response.
Figure 8-6: Response of CPB specimen WCPB-CY4 tested at the CSR of 0.24 and $\sigma_c'=30$ kPa. a) Stress path, b) pore pressure ratio, c) axial strain versus number of cycles, d) Stress- strain response.
Figure 8-7: The response of CPB specimen WCPB-CY10 cured for 12 hours and tested at the CSR of 0.29 and $\sigma_c' = 50$ kPa. a) Stress path, b) variation in pore pressure ratio versus number of cycles.

8.2.2. Cyclic Resistance of CPB

The relationship between CSR and number of cycles corresponding to 5% DA axial strain for the 3% CPB specimens cured for four hours and tested at two effective confining stresses (i.e., 50 and 100 kPa) in this study is shown in Figure 8-8. In addition, the relationship between CSR and number of cycles at zero effective stress for the 3% CPB specimens cured for four hours and tested at $\sigma_c' = 30$ kPa is also shown in this figure.
The results suggest that the number of cycles increases as the CSR decreases for all datasets. A good statistical fit of cyclic resistance curve to data points was obtained for each series of tests at specific effective confining stress. A comparison between the cyclic resistance curves (between 5-65 cycles) for the specimens tested at $\sigma_c'=50$ kPa and $\sigma_c'=100$ kPa indicates that the standard deviation is about 0.02 and the coefficient of correlation is 0.99. In other words, the effect of effective confining stress is not significant on the resistance of the CPB. A similar trend is observed for the CPB tested at $\sigma_c'=30$ kPa although different criterion (i.e., zero effective stress) was used to obtain the cyclic resistance curve.

Figure 8-8: Potential of CPB to liquefaction due to cyclic loading.

The 4 hour cured CPB specimen tested (i.e., WCPB-CY11) at a low CSR of 0.18 shows that 5% DA axial strain was not reached after 120 cycles for this specimen. In other words, liquefaction is unlikely to occur for the CPB specimens below this CSR within the number of cycles tested in this study.

A similar result is observed for specimen WCPB-CY10 tested at a high CSR of 0.29 but with a 12 hours curing time. As shown in Figure 8-8, 5% DA axial strain was not achieved for this specimen after 120 cycles, while the CPB specimen cured for four hours tested at the same CSR.
was liquefied after 6 cycles. This indicates that the curing time has significantly increased the liquefaction resistance of CPB.

To better understand the effect of 3% cement after 4 hours of hydration on the liquefaction susceptibility of materials, the CPB results are plotted along with the results for the uncemented mine tailings in Figure 8-8. The result indicates that the addition of cement to the mine tailings significantly improves the resistance of the material to cyclic loading. For example, at the same CSR of 0.24 and \( \sigma_c' = 50 \text{ kPa} \) the uncemented mine tailings is liquefied after 3 cycles, while the CPB needs more than 34 cycles to liquefy.

### 8.2.3. Discussion of Cyclic Response of CPB

The cyclic triaxial tests showed that the 4 hour cured CPB exhibited cyclic mobility type of response. Although the pore pressure develops with the number of cycles, zero effective stress is unlikely to be achieved for the 4 hour cured CPB specimens tested at \( \sigma_c' = 50 \text{ kPa} \) and \( \sigma_c' = 100 \text{ kPa} \). In contrast, zero effective stress is achieved for the specimens tested at \( \sigma_c' = 30 \text{ kPa} \). Note that the maximum pore pressure ratio for the specimens tested at \( \sigma_c' = 50 \text{ kPa} \) and \( \sigma_c' = 100 \text{ kPa} \) ranges between 0.92 and 0.95, which is close to unity where zero effective stress would be achieved. For the specimens tested at 30 kPa, the maximum DA axial strain ranges 1% to 3%. These values are significantly lower than 5%, which is reached for the specimens tested at 50 and 100 kPa. In other words, the axial deformation is less pronounced for the specimens tested at 30 kPa, while pore pressure is fully developed in these specimens.

The cyclic mobility type of response identified for the 4 hour cured CPB is similar to the behaviour of the uncemented mine tailings tested in this study. However, the CPB shows more resistance to cyclic loading than the uncemented mine tailings.

Like uncemented mine tailings, the cyclic test results on CPB in this study show that the cyclic stress path lies slightly above the constant stress ratio line in compression after reaching 5% DA axial strain. However, the cyclic stress path significantly lies beyond the constant stress ratio line in extension after reaching 5% DA axial strain. The critical distinction is that the response of CPB to monotonic loading in compression (i.e., strain hardening) is different from that in extension (i.e., strain softening). This makes the interpretation of the cyclic response more complicated. As explained for the uncemented mine tailings in the previous section, the strain...
hardening behaviour in compression might be responsible for the apparent discrepancy between cyclic and monotonic results.

The effect of strain hardening behaviour can also be explained by the model developed by Boukpeti et al. (2002). According to this model, the isotropic hardening-softening has been related to plastic deformation and distance to a reference yield curve. Previously, Boukpeti and Drescher (2000) discussed the isotropic hardening rule as the main element of this model. The hardening parameter in this model is a function of both the volumetric and the weighted total plastic strain increments. This hardening parameter allows the stress path to cross the steady state line in triaxial compression and extension. Note that the contribution of these increments vanishes by decreasing the distance above the critical state line between the actual yield curve and a reference yield fictitious curved introduced in this model. Therefore, the hardening parameter is an effective parameter to interpret the consistency of the cyclic loading with monotonic results.

As shown by Chu et al (2003), the failure line in the stress space lies above the constant stress ratio line when no steady state reaches for the dense sand. The behaviour of CPB is similar to that of the dense sand since the stress paths for CPB never reached a steady state (strain hardening) and approached the constant stress ratio line; the constant stress ratio lines are not the same as failure lines for CPB. Therefore, if the cyclic stress path crosses the constant stress ratio line, it does not imply that it passes the failure line too. The situation of steady state and failure lines for CPB must be better understood for more precise interpretation.

8.3. Applicability of Liquefaction Criteria for CPB

Based on the liquefaction criteria proposed by Bray et al. (2006) for silts explained in Chapter 3, the liquefaction susceptibility of uncemented mine tailings and CPB will be investigated in this section. The properties of material required for this investigation, including water content, liquid limit and plasticity index were determined in Chapter 5. Note that there is no standard test method for determining the liquid limit and plastic limit of cementing materials. Table 8-2 summarizes the main properties of uncemented mine tailings and 3% CPB cured for four hours. Note that the void ratio and liquid limit of the specimens are calculated after the dead-weight consolidation stage, while the water content was calculated based on the saturated state in this condition.
Table 8-2: Properties of uncemented mine tailings and CPB for assessing the liquefaction criteria.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Void ratio (±0.005)</th>
<th>Water Content (w_c) %</th>
<th>LL %</th>
<th>w_c/LL</th>
<th>PI (±0.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPB</td>
<td>0.860</td>
<td>31.62</td>
<td>35.49</td>
<td>0.89</td>
<td>6.5</td>
</tr>
<tr>
<td>MT</td>
<td>0.780</td>
<td>28.68</td>
<td>29.93</td>
<td>0.95</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 8-9 shows that both the uncemented mine tailings and 3% CPB cured for four hours are susceptible to cyclic mobility based on criteria proposed by Bray et al. (2006) for silts. This prediction is in agreement with the cyclic triaxial results shown in the preceding sections. Note that the CPB was tested before the onset of acceleration phase of hydration (i.e., 5 hours), as described in Section 5.1.2.

A comparison between the uncemented mine tailings and CPB illustrates the tendency of CPB towards the moderate liquefaction part of the chart and the uncemented mine tailings is slightly away from CPB and closer to the centre, which is consistent with the changes in properties of CPB by adding cement to the tailings. Figure 8-9 also shows the susceptibility of Fraser River Delta (FRD) silt to cyclic mobility tested by Sanin and Wijewickreme (2006). Apparently, the criteria might be applicable for assessing the cyclic liquefaction of slit sized material such as mine tailings and CPB tested in this study.

Figure 8-9: Application of Bray et al criteria for liquefaction assessment of the uncemented mine tailings and CPB.
8.4. Summary and Conclusions

1. The cyclic mobility type of response was identified in this study for both the uncemented mine tailings and CPB tested under cyclic triaxial loading.

2. The liquefaction under cyclic loading was achieved within the tested range of CSR’s (0.15-0.25 for uncemented mine tailings and 0.2-0.3 for CPB), as shown in Figure 8-8. The lower the CSR, the higher the number of cycles required to achieve 5% DA axial strain.

3. 5% DA axial strain was used as the liquefaction criterion for the specimens tested at 50 and 100 kPa, while the maximum pore pressure ratio at this point was about 0.95 (no zero effective stress achieved).

4. Zero effective stress was only achieved for the 3% CPB specimens cured for 4 hours and tested at 30 kPa (e.g., Figure 8-6).

5. A comparison between cyclic results for the uncemented mine tailings and CPB shows that for an equivalent CSR, the number of cycles to failure increases by an order of magnitude. This shows that the addition of 3% cement to the tailings and curing for four hours significantly increases resistance to liquefaction potential (Figure 8-8).

6. The 3% CPB specimens cured for 4 hours and tested at CSR’s of less than 0.2 were highly resistant to cyclic mobility (i.e., up to 120 cycles produced insignificant axial strain and excess pore water pressure).

7. The 3% CPB specimen (WCPB-CY10) cured for 12 hours and tested at a CSR of 0.29 was highly resistant to cyclic mobility (Figure 8-7).

8. The liquefaction criteria for silts proposed by Bray et al. (2006) were applicable for the CPB and uncemented mine tailings tested in this study (Figure 8-9). Some qualifications to this conclusion are that
   i) determining liquid limit for CPB must follow non-standard procedure.
   ii) the CPB was tested before the onset of acceleration phase of hydration; resistance increases significantly as the acceleration phase begins.
CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9. Conclusions and Recommendations

As stated in Chapter 1, the main objectives of this thesis were to: i) investigate the liquefaction potential of fresh CPB under compression and extension monotonic loading, and under cyclic loading in an undrained triaxial test, ii) characterize mining induced events, including rockbursts and production blasts, and compare them with naturally occurring earthquakes in northern Ontario, and iii) investigate the applicability of the “cyclic stress approach” used in geotechnical earthquake engineering for these seismic events, as well as other more recent liquefaction susceptibility criteria proposed for fine-grained soils.

The conclusions and recommendations are based on the experiments conducted and the material used in this study, and may only be applicable for similar material and type of experiments. However, the general framework of investigation can be applied to other mines’ materials and geomechanical parameters.

9.1. Conclusions

9.1.1. Consolidation Characteristics

1. The compression characteristics of CPB were evaluated by data obtained from the consolidation stage of triaxial tests, as described in Chapter 6. The compression index of CPB cured for four hours in this study was 0.11±0.01 showing similarities with that of uncemented
silt sized tailings tested by Crowder (2004), as shown in Figure 6-2. Note that the initial void ratio of 4 groups of CPB specimens ranges between 0.845 and 0.875 (±0.005) while the uncemented tailings specimens tested by Crowder (2004) have an initial void ratio of 0.930 for the high water content specimen and 0.705 for the low water content specimen. This indicates that the initial void ratio has no discernable effect on the compression index of these non-plastic fine grained materials. This may be true of other similar materials, although further testing should be carried out to confirm this hypothesis. The intrinsic compression index proposed by Burland (1990), which was originally used for clays, can be applicable for CPB and tailings for normalizing the consolidation curves (Figure 6-3).

2. A comparison between the silt-sized mine tailings in the form of CPB used in this study and typical sands (e.g., Ottawa or Toyoura sand) showed that the silt-sized mine tailings are more compressible than typical sands regardless of their initial void ratios and the techniques of sample preparation (Figure 6-4).

3. Pre-consolidation (i.e., dead-weight consolidation by applying ~12.5 kPa) type of sample preparation generates CPB specimens with initial void ratios ranging between 0.840-0.875 (±0.005), whereas the void ratios in the field range from 0.86 to 1.2 with an average value of 1.02±0.05, as described in Section 5.2.1. Note that the field vertical effective stress ranged from 10 to 30 kPa after the placement of CPB inside the stope (Thompson et al., 2008). The underlying reasons for the differences between field specimens and lab-prepared specimens require further investigation.

**9.1.2. Monotonic Response**

1. Neither flow liquefaction nor temporary liquefaction was achieved for the normally consolidated 3% CPB specimens (e = 0.775-0.840±0.005) cured for four hours at different effective confining stresses (i.e., 20-200 kPa) in the undrained monotonic compression tests (Figure 7-4). This result is identical to the response of uncemented mine tailings (e = 0.675-0.750) even though the void ratio of the CPB was higher than that of the uncemented tailings.

2. The response of the 3% fresh CPB to the undrained monotonic loading was strain hardening in compression and strain softening with temporary instability followed by strain hardening in extension.
3. The stress paths for the 3% fresh CPB specimens exhibited both contractive and dilative behaviours in compression and extension. The phase transformation (PT) points showing the transformation between contractive and dilative behaviours were identified at strain levels lower than 4% for CPB and uncemented tailings specimens. The angle of the phase transformation line (α'_PTL), which was obtained by a best-fit line to the PT points, was 32.7º for 3% CPB cured for 4 hours and 32.0º for the uncemented tailings.

4. None of the deviator stresses reached a true steady state even though axial straining was taken up to 20%. Larger strains are apparently required to achieve a steady state in undrained loading. However, the stress paths did approach a unique line (i.e., the constant stress ratio line) when reaching a maximum stress ratio value.

5. The angle of the constant stress ratio line for 3% CPB specimens cured for four hours (i.e., α'_CSRL = 33.8º, e = 0.775-0.840) is higher than that of the uncemented mine tailings (i.e., α'_CSRL = 33.3º, e = 0.675-0.750) even though the void ratios of CPB specimens were higher than those of uncemented mine tailings.

6. The difference between the angle of the constant stress ratio line (i.e., α'_CSRL = 33.8º) and that of the PTL (i.e., α'_PTL = 32.7º) for 3% CPB cured for four hours is 1.1º. This relatively small difference might suggest the application of the PTL instead of the constant stress ratio line for the geomechanical design of CPB systems.

7. The angle of the constant stress ratio line for 3% CPB specimens cured for four hours showed a modest dependence on the void ratio of the specimens. The experiments showed that when void ratio varied from 0.775 to 0.890 the angle of the constant stress ratio line (α'_CSRL) varied from 33.9º to 33.0º, respectively.

8. There is no generation of cohesion in four hours for CPB specimens. However, the addition of cement helps to increase the angle of the constant stress ratio line (or friction angle), as compared to uncemented tailings.

9. The angle of the constant stress ratio line in compression (i.e., α'_CSRL = 33.8º, e = 0.775-0.840) is higher than that in extension (i.e., α'_CSRL = 32.9º, e = 0.770-0.790). The reasons for
this relatively small difference are not known at present, although this result is consistent with
other published results on fine grained material (e.g., Hyde et al., 2006).

10. Axial strain rates in the range 0.1-2.0%/min have no discernible effect on the angles of the
corresponding state lines (i.e., PTL and constant stress ratio line).

9.1.3. Cyclic Response

1. The cyclic mobility type of response was identified in this study for both the un cemented
mine tailings and 3% CPB cured for 4 hours tested under cyclic triaxial loading, as described
in Chapter 8.

2. The liquefaction under cyclic loading was achieved within the tested range of CSR’s of 0.15-
0.25 for un cemented mine tailings and 0.2-0.3 for the 3% CPB cured for 4 hours (Figure 8-8).

3. The lower the CSR, the higher was the number of cycles required to achieve liquefaction. The
trend of CSR curves for CPB and un cemented tailings follows expected trends similar to le

4. 5% double amplitude (DA) axial strain was used as the liquefaction criterion for the
specimens tested at 50 and 100 kPa initial effective confining stress, while the maximum pore
pressure ratio at this point was about 0.95; no zero effective stress was achieved.

5. Zero effective stress was only achieved for the 3% CPB specimens cured for 4 hours and
tested at 30 kPa initial effective confining stress (e.g., Figure 8-6).

6. A comparison between cyclic results for the un cemented mine tailings and CPB tested at $\sigma'_c$
= 50 kPa initial effective confining stress shows that the cyclic resistance of CPB (e.g.,
CSR$_{20cycles}$) is about 0.25 while that of un cemented tailings is about 0.15 indicating a
quantifiable increase in resistance after adding 3% cement to the mine tailings and curing for
four hours (Figure 8-8). While such a difference would be important to the design of many
surface structures in geotechnical earthquake engineering, its importance to the design of
geomechanical systems in underground mining has yet to be investigated.
7. The 3% CPB specimens cured for 4 hours and tested at CSR’s of less than 0.2 were highly resistant to cyclic mobility (i.e., up to 120 cycles produced insignificant axial strain and excess pore water pressure).

8. The 3% CPB specimen cured for 12 hours (i.e., WCPB-CY10) and tested at a CSR of 0.29 was highly resistant to cyclic mobility (Figure 8-7). The 3% CPB cured for 4 hours liquefied in 5 cycles at a CSR of 0.29, whereas similar specimen cured for 12 hours did not liquefy in 120 cycles. This shows the significant effect of curing time on the resistance of CPB to liquefaction potential.

9. The liquefaction criteria for silts proposed by Bray et al. (2006) was applicable for the CPB cured for four hours and uncemented mine tailings tested in this study (Figure 8-9). A caveat to this conclusion is that standard test methods do not exist for determining Liquid Limit (LL) and Plastic Limit (PL) for hydrating materials; however, the adaptations of the test methods used in this thesis and applied to the CPB seemed reasonable for the material studied and should be investigated in future on other types of paste backfill.

9.1.4. Seismic Events

1. The NRCan seismic events including far-field earthquakes and far-field mining events showed good correlation between the dynamic parameters, such as PPA, PPV and their corresponding predominant frequencies versus distance. The PPA of the seismic events varied from 0.02 m/s² to 0.0009 m/s² when the distance from the source varied from 22 km to 250 km for the seismic events with mN 3.5-4.3 (Figure 4-10).

2. There is a good consistency between the magnitude and PPV or PPA of the NRCan seismic events (i.e., far-field earthquakes and mining events) at a specific distance. The higher the magnitude, the higher the PPV or PPA at a specific distance. Generally, the magnitudes of the earthquakes are relatively low due to the low seismic activity of the north eastern Ontario seismic zone.

3. The NRCan far-field seismic events (i.e., earthquakes and mining events) can be categorized as low frequency content events (3-16 Hz). Therefore, far-field mining related events might plausibly be treated the same as earthquakes for conventional geotechnical earthquake engineering design problems (Table 4-10).
4. The mining events including rockbursts and production blasts recorded at the mines but still at a relatively far distance (1-2 km) from the seismic sources also showed similar trend in PPV and PPA with respect to their distance. However, the PPV of these events is more scattered with respect to their magnitudes, ranging from 0.008 m/s to 0.09 m/s when magnitude varied from $m_N$ 2.7 to $m_N$ 3.8 at 2 km from source (Figure 4-40). The predominant frequency of the rockbursts is similar to that of the NRCan seismic events, ranging between 5 Hz and 25 Hz. However, the production blasts show higher values of about 33-40 Hz for frequency (Table 4-10). Conventional geotechnical earthquake engineering design problems rarely consider events with the frequencies characteristics of far-field production blasts; therefore CPB response to blasting remains an area of further research.

5. The typical blasting practice at The Williams Mine show that the PPV in rock induced by production blasts with different charge weights at a near distance (20-100m) from the source ranges from 0.004 to 0.2 m/s, which is not significantly higher than the upper limit of PPV of mining events recorded at ~1-2 km from the sources (Figure 4-40).

6. In contrast, the predominant frequency arising from spectral analysis of velocity-time series for the production blasts recorded in rock at The Williams Mine ranged from 200 Hz to 4000 Hz, which is significantly higher than the frequency content of the NRCan seismic events and those mining events recorded about 1-2 km from the source (i.e., 5-40 Hz, Table 4-10). Therefore, production blasts occurring at close distance (30-100 m) to the source might not be treated the same as the seismic events recorded at distances more than 1-2 km for geomechanical design problems.

7. The PPV induced by the same production blasts at The Williams Mine but recorded in CPB varied from 0.002 m/s to 0.04 m/s when the distance from the source was about 60-80 m. The frequency content for velocity in CPB varied from 120 Hz to 700 Hz depending on curing time, as described in Section 4.3.5. This indicates that the frequency in rock is significantly higher than the frequency in CPB for the same production blast.
9.2. Recommendations and Future Works

To apply the results and findings in this thesis to a variety of CPB design problems the following items are recommended:

1- The CPB specimens tested in this study have a void ratios ranging from 0.775 ($\sigma_c' = 200$ kPa) to 0.840 ($\sigma_c' = 50$ kPa) while the void ratios in the field range from 0.86-1.2 with an average value of 1.02±0.05, as described in Section 5.2.1. Therefore, the sample preparation technique should be improved to make a sample with higher void ratios.

2- The CPB specimens were cured for four hours, which was a practical limitation of the specimen preparation technique employed (i.e., testing at earlier times would not generally be possible). The response of CPB cured for other durations may also be of interest, depending on the particular design context. The monotonic tests were performed before the onset of acceleration phase of hydration, which is the worst case scenario in this study; more experiments may be required to investigate the effect of curing time on the monotonic response of CPB when the acceleration phase begins or the initial set is reached.

3- The binder agent used in this study was 3% PC while various binder agents and binder content may be used in different mines. Therefore, the effect of different binder types and contents deserves further investigation.

The effect of cyclic loading frequency on material response is typically considered negligible in conventional geotechnical earthquake engineering problems. However, the frequency of near field seismic events is significantly higher than those in conventional earthquake engineering (Table 4-10). Therefore, the conventional low frequency cyclic triaxial tests are possibly not appropriate for simulating those high frequency seismic events in the laboratory. To address the stress conditions and simulate the blast-induced liquefaction in the case of a backfilled stope with CPB, the following recommendations are presented:

4- More rockbursts and blasting data sets will be required to have a better perspective of amplitude and frequency content of such seismic events close to the source ranging between 20 m to 1000m.
5- Wave propagation in CPB design problems inherently involves multiple materials (i.e., the rock and the backfill) and therefore numerical analysis will generally be required to properly model this behaviour.

6- Based on the current findings, the triaxial machine is incapable of applying cyclic loads with high enough frequencies to simulate the blasting loads. One dimensional impact machines similar to the Split Hopkinson Bar but with low velocity impact might be useful for these types of studies. Note that a special cell should be designed for applying confining pressures and dynamic pressure transducers are required to monitor excess pore water pressure induced by loading.

9.3. Main Contributions of the Thesis

The monotonic triaxial testing conducted as part of this thesis provides a much more complete understanding of the liquefaction mechanisms in CPB than previously existed. In particular, both the uncemented tailings as well as the CPB specimens tested – all of which were normally consolidated – never exhibited the unstable form of liquefaction (i.e., flow liquefaction) commonly observed in loose sand specimens. Both uncemented and CPB specimens exhibited Mohr friction angles in excess of 40° over the range of confining stresses used (i.e., 20-400 kPa), which is considerably higher than typical values for loose sands (i.e., 28-32°). In general terms, this suggests that CPB can be expected to be considerably stronger and less susceptible to sudden strength loss than would be the case for an equivalent sand fill. This contribution confirms the suspicions of many practitioners that CPB is much more stable than current practice gives the material credit for.

The cyclic loading tests showed that both uncemented tailings and 3% CPB cured for four hours are susceptible to cyclic mobility within a tested range of CSR’s (0.15-0.3). The cyclic resistance of CPB is higher than that of uncemented tailings even though the void ratios of the CPB specimens are generally higher than those for uncemented tailings. In other words, the number of cycles to failure (i.e., 5% DA axial strain) for CPB tested at four hours is about an order of magnitude higher than that for uncemented tailings for an equivalent cyclic stress ratio. Note that the CPB cured for four hours was tested before the onset of acceleration phase of cement hydration. For the CPB cured for 12 hours (e.g., WCPB-CY10), the resistance to cyclic mobility increased significantly as the initial set was reached.
The analysis of naturally occurring earthquakes as well as mining events such as rockbursts and production blasts, as well as the results of cyclic triaxial testing of uncemented tailings and CPB suggests that the potential exists to extend the framework used for conventional geotechnical earthquake engineering of surface structures to the design of liquefaction assessment of paste backfill systems. Such analysis will require better characterization of the input loading arising from production blasts, as well as rockbursts when the backfill is in the near field of such an event, and this represents a significant research challenge. The conventional geotechnical earthquake engineering framework will also need to be modified to account for differences as compared to surface structures, including boundary conditions, the heterogeneity of a mined stope with multiple backfilled zones, the potential for larger pressure wave components, and the time-sensitive nature of a hydrating backfill material. Although much research remains to be done before such an analysis can be considered “routine” for CPB systems, this thesis has made founding contributions in terms of the nature of the input loading, the material properties under cyclic loading, and the potential applicability of existing susceptibility criteria for fine grained materials. Future research projects may now build on these contributions and work towards a more refined approach for assessing the liquefaction potential of cemented paste backfill systems.
References


Al-Tarhouni, M. 2008. Liquefaction and post-liquefaction behaviour of gold mine tailings under simple shear loading. Master’s Thesis, Department of Civil and Environmental Engineering, Carleton University, Ottawa, Ontario, Canada.


References

Abdolreza Saebi Moghaddam, Doctor of Philosophy


Johnson, J.C., Williams, T., and Pierce, P. 2007. The response of cemented backfill to dynamic loads from field observations and split Hopkinson pressure bar tests. Minefill 2007, paper # 2490, Montreal, Quebec, Canada.


Prakash, S., and Puri, V.K. 2003. Liquefaction of silts and silt-clay mixtures. Internal report, Department of Civil Engineering, University of Missouri, USA.


Shaepermeier, M.E. 1978. Liquefaction induced by compressional waves. Transcripts of the International Workshop on Blast-induced Liquefaction, United Kingdom, pp. 57-64.


Appendix A

Conversion of NRC Seismic Waveform Data to Actual Velocity Time Series

A. Conversion of NRC seismic waveform data to actual velocity time series

A.1. Waveforms

The National Waveform Archive (NWFA) contains digital seismic waveform data captured by regional telemetered seismic networks in Eastern and Western Canada (ECTN/WCTN) and local telemetered networks located in Charlevoix, Quebec (CLTN) and Sudbury, Ontario (SLTN) since 1975 (NRC, 2009). The datasets are available in different formats including ASCII integers. The ASCII files of the seismic events (seismogram) for each station of the Canadian National Seismograph Network (CNSN) contain a short header and a list of integer sample values. As shown in Figure A-1, the header includes the unique code of the station (usually 3 or 5 capital letters, e.g. “KAPO” for Kapuskasing, Ontario), the characteristics of the seismograph signal (e.g., “BHN” indicates a broadband, high gain seismograph that is oriented on the North axis), date and time of the event (Date: 2006/12/07, GTM: 04:58:30), data rate as sample per second (SPS = 40) and the number of integer samples (e.g., 14400) followed by integer sample values or digital counts of the amplitude of the waveform.
Figure A-1: ASCII file format (NRC, 2009).

Figure A-2 shows the waveform of the 2006/12/07 earthquake obtained from the ASCII file presented in Figure A-1. The amplitude of this waveform is digital counts and needs to be converted to actual velocity.

To determine the amplitude of the waveform in velocity a transfer function based on the instrument response must be defined. In deed, the amplitude of velocity in m/s is obtained by
dividing the amplitude of the waveform in counts over the transfer function (counts/m/s). The response information of each station might be available in different formats, such as “response curve” and “poles and zeros”. Using one of these two formats, the obtained transfer function should be the same and depends on the dominant frequency of the waveform.

The dominant frequency might be determined in different ways. The procedure that is used by the seismologists in Natural Resources Canada can be described as follows. The highest amplitude of a waveform is determined and considered as the representative of a wave with the most energy. The difference in time between the adjacent peak and trough near there is then determined. The period is defined by multiplying this difference by two (Woodgold, 2009). Therefore, the dominant frequency of the waveform for the 2006/12/07 earthquake is 10 Hz, as illustrated in Figure A-2.

A.2. Transfer Function for Response Curves

Response curve is a graph that shows the relation between the digital counts in the seismographs and ground motion in m/sec corresponding to the dominant frequency of a waveform. Figure A-3 shows an example of response curve for component N of KAPO station, showing the variation of transfer function (based on counts/m/s) versus frequency (Hz). If the frequency of the digital waveform is within the flat part of the response curve, the corresponding transfer function can be directly used to convert the digital waveform to actual velocity time series. For example, the dominant frequency of the waveform recorded at KAPO station is 10 Hz, the corresponding value on the response curve for KAPO is about 5E+9 counts/(m/s). Therefore, all the data points in the ASCII file should be divided by this number to obtain the velocity of the ground based on m/s.

A.3. Transfer Function for Poles and Zeros

The poles and zeros format can also be used to obtain a precise transfer function. The procedure involves arithmetic with pole and zero complex numbers available on the response information section of the CNSN stations. Figure A-4 shows the format of poles and zeros for KAPO station. The first and second rows indicate that this file must contain 3 zeros and 4 poles (total 7) while each pole or zero is a complex number. However, the total number of complex numbers is 4. According to NRC, if the number of poles or zeros given is fewer than the number it says there
are, then it is implied that there are some additional ones which are (0, 0) to fill out the number (Woodgold, 2009). In this case there are three (0, 0) complex numbers for zero and 4 poles, as shown in this figure. This file also contains a constant value relative to transfer function.

![Overall Response: [Counts/M/S]](image_url)

Figure A-3: Response curve of BHN for KAPO station (NRC 2009).

```
<table>
<thead>
<tr>
<th>Zeros</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poles</td>
<td>4</td>
</tr>
<tr>
<td>-0.031400</td>
<td>0.000000</td>
</tr>
<tr>
<td>-0.209000</td>
<td>0.000000</td>
</tr>
<tr>
<td>-222.111000</td>
<td>-222.178000</td>
</tr>
<tr>
<td>-222.111000</td>
<td>222.178000</td>
</tr>
<tr>
<td>Constant</td>
<td>4.937607e+14</td>
</tr>
</tbody>
</table>
```

Figure A-4: Poles and zeros data file for all components of KAPO station (NRC, 2009).
The transfer function $H$ for the given dominant frequency is defined as below:

$$H = \text{const}. \times \frac{(s - z_1)(s - z_2)...(s - z_j)}{(s - p_1)(s - p_2)...(s - p_k)},$$  \hspace{1cm} (A.1)

where $z_j$ is the jth zero, $p_k$ is the kth pole and $s$ is complex frequency:

$$s = 2\pi fi$$  \hspace{1cm} (A.2)

where $i = \sqrt{-1}$ and $f$ = dominant frequency.

The absolute value of $H$ provides the displacement response in counts per meter (Woodgold, 2009). To obtain the velocity response (in counts per m/s), the displacement response must be divided by the complex frequency. For example, the velocity response for KAPO station calculated based on poles and zeros at 10Hz is equal to 5.00002E9 counts/(m/s), which is consistent with the value derived from the response curve at the same frequency (see Figure A-3). Note that the acceleration response (in counts per m/s$^2$) can be obtained by dividing the velocity response by the complex frequency again.

**A.4. Velocity Time Series**

As stated before, the amplitude of velocity in m/s is obtained by dividing the amplitude of the waveform in counts over the transfer function (counts/m/s). Figure A-5 shows the velocity time series of the 2006/12/07 earthquake recorded at KAPO station. A comparison between this figure and Figure A-2 shows that the velocity time series is exactly the same as the raw digital waveform in terms of frequency.
Figure A-5: Velocity time series of the 2006/12/07 earthquake recorded at KAPO station.

A.5. Response Curves of the CNSN Stations

In the following section, the response curves of the CNSN stations used in this study will be presented. For those stations that the response curves are not available the velocity was determined using the poles and zeros format.
Appendix A Abdolreza Saebi Moghaddam, Doctor of Philosophy 210

CN: EEC: EH2

Analogue Response – (Poles & Zeros)

Digital Response – (FIR)
dB vs. Normalized Freq

Overall Response: [Counts/M/S]

Nominal Mid-Band Gain (G/M/S): 9.0000000e+09
Equivalent Sample Rate [Hz]: 100.00
Nominalization Frequency [Hz]: 1.00e+01
A0 Normalization Factor: 5.43768356e+15

Frequency [Hz]
Appendix A
Abdolreza Saebi Moghaddam, Doctor of Philosophy

CN.GTC.EHZ

Analogue Response – (Poles & Zeros)

Normalized

Velocity

Response

Phase

Response

(Degrees)

Frequency [Hz]

0.001 0.01 0.1 1 10 100

Digital Response – (FIR)

dB vs. Normalized Freq.

Overall Response: [Counts/M/S]

Nominal Mid-Band Gain [Gd/M/S]: 9.0000000e+09
Spikelet Sample Rate [Hz]: 100.00
Nomination Frequency [Hz]: 1.00 e 01
A0 Normalization Factor: 6.4378335e-15

Frequency [Hz]

0.001 0.01 0.1 1 10 100
Appendix A

Abdolreza Saebi Moghaddam, Doctor of Philosophy

CN.KAPO.BHZ

Analog Response – (Poles & Zeros)

Normalized Velocity Response

Phase Response (Degrees)

Frequency [Hz]

Digital Response – (FIR)

dB vs. Normalized Freq

Overall Response: [Counts/M/S]

Nominal Mid-Band Gain (D Buff): \(5.000000 \times 10^9\)

Filter Sample Rate [Hz]: \(40.00\)

Normalization Frequency [Hz]: \(1.00 \times 10^9\)

All Normalization Factor: \(9.972495 \times 10^{-4}\)
**Appendix A**

Abdolreza Saebi Moghaddam, Doctor of Philosophy

---

**CN.SADO.BHZ**

**Analogue Response – (Poles & Zeros)**

- Normalized Velocity Response
- Phase Response (Degrees)

**Frequency [Hz]**

---

**Digital Response – (FIR)**

- dB vs. Normalized Freq

**Overall Response: [Counts/M/S]**

- Nominal Mid-Band Gain [dB/dB]: $4.598\times10^{-9}$
- Filter Sample Rate [Hz]: 40.00
- Normalization Frequency [Hz]: 1.00e-06
- A0 Normalization Factor : $5.715\times10^{11}$

---

**Frequency [Hz]**
Appendix A
Abdolreza Saebi Moghaddam, Doctor of Philosophy

CN.TBO.EHZ

Analogue Response – (Poles & Zeros)

Normalized
Velocity
Response

Phase
Response
(Degrees)

Frequency [Hz]

0.001 0.01 0.1 1 10 100

Digital Response – (FIR)
dB vs. Normalized Freq

Overall Response: [Counts/M/S]

Nominal Mid-Band Gain [G/M/S]: 9.000000e+09
Spike/Scan Sample Rate [Hz]: 100.00
Normalization Frequency [Hz]: 1.00e-01
A0 Normalization Factor: 6.437835e-15
Appendix B

Frequency Spectra for NRCan Seismic Events

B. Frequency spectra for NRCan seismic events

In this appendix, the frequency spectra of the NRCan seismic events that have been presented in Table 4-6 are shown.
B.1. 2003/09/13 - Williams Rockburst - FFT-3.5 m

Counts-SQLO-EHZ

Counts-TBO-EHZ

Counts-GTO-EHZ
B.2. 2004/07/15 Rockburst – FFT - 3.5 mN
B.3. 2006/11/29 Rockburst – FFT – 4.1 mN
B.4. 2005/10/20 Earthquake – FFT – 4.3 m$_N$
B.5. 2006/01/03 Earthquake – FFT- 3.7 m_N

Counts-KILO-HHN

Counts-KILO-HHE

Counts-KILO-HHZ
Appendix B

Abdolreza Saebi Moghaddam, Doctor of Philosophy

Counts-KAPO-BHE

Counts-KAPO-BHN

Counts-KAPO-BHZ
Counts-TIMO-HHN

Counts-TIMO-HHZ

Frequency

Amplitude
B.6. 2006/12/07 Earthquake – FFT – 4.2 m$_N$
B.7. 2006/12/07 Earthquake vs. 2006/11/29 Mining Event

Spectra of 8-13 sec window versus 0-30 sec window of Figure 4-5.

Spectra of 3-6 sec window versus 0-10 sec window of Figure 4-8.
B.8. Typical Acceleration Time Series of Rockburst Events Recorded in Mines and Their Corresponding FFT’s

![Time Series Graph](image1)

![FFT Graph](image2)
Kidd Rockburst in June - SV Component

 FFT for Acceleration
 Kidd Rockburst in June - SV Component
Appendix C

Cyclic Stress Results
WMT-CY2
CSR = 0.22
5% DA Axial Strain = 5 cycles

\( \frac{(\sigma'_1 - \sigma'_3)}{2}, \text{kPa} \)

\( \frac{(\sigma'_1 + \sigma'_3)}{2}, \text{kPa} \)

Pore pressure ratio, \( \frac{\Delta u}{\sigma'_3} \)

Number of cycles

WMT-CY2
CSR = 0.22
5% DA Axial Strain = 5 cycles

Number of cycles
WMT-CY2
CSR = 0.22
5% DA Axial Strain = 5 cycles

Axial strain, %

Number of cycles

-3 -2 -1 0 1 2 3 4 5

-25 -20 -15 -10 -5 0 5 10 15 20 25

\( (\sigma^1 - \sigma^3), \text{kPa} \)
WMT-CY4
CSR = 0.175
5% DA Axial Strain = 9 cycles

\( \frac{\sigma'_1 + \sigma'_3}{2}, \text{kPa} \)

Pore pressure ratio, \( \frac{\Delta u}{\sigma'_3} \)
Appendix C
Abdolreza Saebi Moghadam, Doctor of Philosophy

WMT-CY4
CSR = 0.175
5% DA Axial Strain = 9 cycles

WMT-CY4
CSR = 0.175
5% DA Axial Strain = 9 cycles
\[
\frac{(\sigma_1 - \sigma_3)}{2}, \text{kPa} \quad \frac{(\sigma_1 + \sigma_3)}{2}, \text{kPa}
\]

WMT-CY7
CSR = 0.20
5% DA Axial Strain = 8 cycles

Pore pressure ratio, \(\frac{\Delta u}{\sigma_3}\)

WMT-CY7
CSR = 0.20
5% DA Axial Strain = 8 cycles
WMT-CY7
CSR = 0.20
5% DA Axial Strain = 8 cycles

Axial strain, %

(σ' - σ'₃), kPa

Number of cycles

Axial strain, %
WMT-CY8
CSR = 0.18
5% DA Axial Strain = 12 cycles

\[ \frac{\sigma'_1 - \sigma'_3}{2}, \text{kPa} \]

\[ \frac{\sigma'_1 + \sigma'_3}{2}, \text{kPa} \]

WMT-CY8
CSR = 0.18
5% DA Axial Strain = 12 cycles

Pore pressure ratio, \( \frac{\Delta u}{\sigma'_3} \)
WMT-CY8
CSR = 0.18
5% DA Axial Strain = 12 cycles

Axial strain, %

Number of cycles