ON THE STRENGTH OF SATURATED CEMENT-TREATED SOIL RECONSTITUTED BY WET-MIXING

by

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ABSTRACT

Cutter Soil Mixing (CSM) is a recently developed deep mixing technique that has grown to include the treatment of sandy and silty soils. This study seeks to investigate the influence of (i) sand-silt ratio, (ii) cement content, (iii) water content and (iv) time on the unconfined compressive strength of saturated cement-treated soil specimens. A new test device and method of specimen reconstitution were conceived in order to obtain a saturated mix of soil and cement. A comparison of results show strength increases non-linearly to decreasing total water-cement ratio, and that this trend is largely independent of sand-silt ratio. Furthermore, strength increases non-linearly with time and is independent of sand-silt ratio. Lastly, it is recommended that the strength be correlated with total water-cement ratio rather than cement content, in order to improve data reporting and provide design guidance to engineering practice.

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1 INTRODUCTION

The Deep Mixing Method (DMM) has become an increasingly popular in situ ground improvement technology in North America in recent years. DMM improves the engineering properties of soil by blending it with a wet, or alternatively, a dry binder (normally cement). Cutter Soil Mixing (Brunner et al., 2006) is a relatively new 'wet' DMM that has proved to be useful and cost effective. Golder Associates Innovative Applications Inc. (GAIA) has recently implemented two Cutter Soil Mixing (CSM) projects in British Columbia: the Vancouver Island Conference Centre at Nanaimo, and a low-rise commercial development at Still Creek Basin in Burnaby. Geotechnical considerations in these projects mainly included mitigation of material susceptible to liquefaction or large ground deformation during a significant seismic event. Mitigation of these sites was achieved by injecting cement slurry through the cutter-mixing tool, followed by subsequent blending with the in situ soil to create homogeneous cement-treated soil columns, as part of an inter-locking grid system. Improvement of liquefiable loose sands and silts using a matrix of singular cement-treated columns has been shown to prevent liquefaction-related damages during the 1999 Kocaeli M7.4 earthquake, Turkey (Martin et al., 2004). Therefore, it is of interest to examine the various factors that govern the strength of cement-treated soil strength properties, and to apply this knowledge to ground improvement projects in the inter-bedded saturated sands and silts of the Fraser River Delta that underlie much of the Vancouver area and its suburbs.

1.1 Purpose of the study

This study investigates the factors that influence the strength of cement-treated silty sand and sandy silt. Accordingly, a laboratory testing program was undertaken using soil specimens with four different percentage combinations of sand and silt, treated with varying proportions of water and cement. A new method of specimen reconstitution was developed to yield reproducible specimens that are homogeneous and fully saturated, whilst also allowing for an accurate measurement of the proportion by mass of sand, silt and cement. This new method of reconstitution is believed to replicate the fabric and condition of the CSM cement-soil mix at sites where the ground is saturated. The specimens were tested to establish unconfined compressive strength, at a number of different curing periods.

The need for guidance on economical mixing design parameters arose from a lack of published laboratory data pertaining to sandy and silty soils. In addition, little is known about the strength gain with time of wet-mixed cement-treated soil. Therefore, the influence of (i) sand-silt ratio, (ii) cement content, (iii) water content and (iv) time on the unconfined compressive strength of saturated cement-treated soil specimens is examined. Where appropriate, comparison is made to other laboratory studies that also tested specimens applicable to the 'wet-mixing' method of ground improvement.

The main objectives of the current study are:

- To provide guidance arising from a study of the influence of water and cement in four sand-silt mixtures believed to characterize the typical range encountered in engineering practice; and,
- As a means for decision support, to explore those trends with respect to time; and,
- To contrast the findings with other published data from laboratory studies.

1.2 Organisation of the thesis

In Chapter 2, a brief review of the literature is presented on methods of reconstitution, the factors governing cement-treated soil behaviour, and the peak unconfined compression strength of cement-treated soil. Experiences with existing reconstitution techniques, specimen homogeneity, curing environment and physicochemical processes are considered. Trends from other studies on soil type, cement content, water-cement ratio and time are also presented.

A description of materials used in testing is presented in Chapter 3. Chapter 4 outlines the configuration of the newly manufactured test device and compression system, followed by an extensive description of the method of specimen reconstitution. Finally, the main program of testing describes the range of variables examined in the laboratory investigation.

Chapter 5 presents results from commissioning of the test procedure, a selection of scanning electron microscope images from test specimens, and unconfined compression data from the main program of testing. In Chapter 6, the peak unconfined compressive strength of test specimens is analysed with respect to the influence of sand-silt ratio, cement content, time and total water-cement ratio. Data from the current study are then compared with data from other laboratory programs. The main conclusions and recommendations of the study are reported in Chapter 7.

Supplementary information regarding the chemical reactions that occur during cement hydration is found in Appendix A. Additional scanning electron microscope images acquired at a relatively lower magnification, and a comprehensive description of each test specimen, are found in Appendix B.

2 LITERATURE REVIEW

The deep mixing method (DMM), a soil improvement technology, improves the engineering properties of soil by introducing some form of binder, such as Ordinary Portland cement, which is subsequently mixed within the soil deposit. Many forms of this type of soil improvement have been used since the 1960's in Scandinavia, Japan, and North America (Bruce, 2000). The DMM generally fall into the 'wet' mixing (water and binder is added during mixing) or 'dry' mixing (binder only is added) category (Bruce and Bruce, 2003). A recently developed (2003) type of wet DMM is Cutter Soil Mixing (CSM), which is capable of creating homogeneous cement-treated soil columns up to a depth of approximately 35m below the ground surface. CSM implements this process by inserting two sets of cutting wheels that rotate around a horizontal axis and 'cut' into the ground, loosening the in situ soil (Fiorotto et al., 2005). The process is aided by the introduction of a fluidizing agent, such as water, through the cutting tool's injection system. Once the soil is sufficiently fluidized, the binder (cement) is injected into the soil producing a cement-treated soil mass, which gains strength with time and improves the engineering properties of the soil. Typical CSM column arrangements include: single elements, inter-locking panels, grids, and blocks (Brunner et al., 2006). CSM applications include: seismic stabilization, slope stability, excavation support and seepage control.

Golder Associates Innovation Applications Inc. (GAIA) implemented the first CSM project in North America (Environmental Services Association of Alberta, 2007). The award-winning project (CEBC, 2006) improved the variable ground conditions at the site of the newly built

Figure 2.1. Cutter Soil Mixing at Still Creek Basin, Burnaby, British Columbia

Vancouver Island Conference Centre, Nanaimo, British Columbia. The CSM technique improved sandy soil and other infill material, which was potentially liquefiable and susceptible to large settlement during a significant seismic event (Vancouver Island Conference Centre Foundation, 2007). GAIA also used the CSM technique at a second location on the Still Creek Basin site, Burnaby (a suburb of Vancouver), British Columbia (Figure 2.1). The soil stratigraphy consisted of fill, peat, organic silt, clayey silt underlain by silty sand. The CSM technique treated the soil in order to create an in situ cellular grid structure for seismic ground improvement.

Therefore, in response to the growing need for a quality controlled laboratory data set to evaluate strength relations of CSM cement-treated silts and sands of the saturated Fraser River Delta, a new method of reconstitution is required. Further, cost-effectively controlling the amount of cement used in CSM projects, whilst also meeting the project-specified unconfined compressive strength value, presents a further challenge to engineering consultants during the preliminary design stage.

2.1 Reconstitution techniques

Various researchers have reconstituted specimens for unconfined compression testing in order to study the engineering properties of cement-treated soil under controlled laboratory conditions, using a variety of methods. For the case of CSM and indeed most other wet DMM applications, the laboratory method should replicate a cement-treated soil that is saturated, homogeneous and mixed rather than compacted.

2.1.1 Existing methods of reconstitution

There are a number of existing techniques (ASTM D 1632-87, 1996; Jacobson, 2002; Hodges et al., 2004) that have been used in order to reconstitute cement-treated soil specimens in the laboratory. The most common technique used is to first mix all of the solid materials (finer grained soil generally has a significant natural water content, and this water is normally mixed in as well at this stage) and then add water for purposes of blending the materials. The mixing process may be done either by hand (using a trowel) or by using a mechanically operated mixing device. The cement-treated soil mixture is either poured or spooned into a mould, and then usually compacted. The cement-treated soil may be left inside the mould for the entire curing period (usually 7-90 days in most studies) or, alternatively, extruded after a sufficient hardening time before being returned to the curing environment. Specimens are cured in either a moist room or a water bath, which provides protection from fluctuating temperatures and humidity.

The advantage of these methods is that the specimen reconstitution is quick and relatively simple.

While the abovementioned, partially saturated reconstitution methods are appropriate to some applications such as highway sub-base construction (Horpibulsuk, 2006), none replicate significant aspects of the wet DMM process. In practice, the wet-mixed soil is blended with cement slurry under saturated conditions, and there is no compaction. Rather, self-weight consolidation occurs in the treated ground. Accordingly, a partially saturated reconstitution method, yielding air entrained within the cement-treated soil mix, is not believed representative of the improved ground and is expected to affect the magnitude and variability of the resulting unconfined compressive strength values obtained from test specimens.

2.1.2 Specimen homogeneity

In order to provide the best possible conditions for chemical reactions (i.e. hydration, discussed later in section 2.2.2.1) to take place, the cement particles must be uniformly dispersed throughout the treated soil (Larsson, 2005). A uniform distribution will then optimize the strength and deformation characteristics of the cement-treated soil and create a homogeneous specimen. Further, a uniformly-mixed homogeneous laboratory specimen is less likely to produce scatter when analysing the peak strength data. To ensure that the specimens are homogeneous, a certain degree of mixing energy is required.

Observations by Shen et al. (2004) conclude that mixing energy is related to a number of factors such as the cement content, rotation speed and water content. Furthermore, their data on duplicate unconfined compressive strength specimens (Figure 2.2) suggest that a threshold mixing energy exists. The data also indicate that once the threshold mixing energy is surpassed, there appears to be no significant increase in strength. Therefore, it is advisable to use a mixing energy that is very high for all specimens in order to discount the prospect of inadvertently introducing a mixing energy variable into the reconstitution of cement-treated soil specimens.

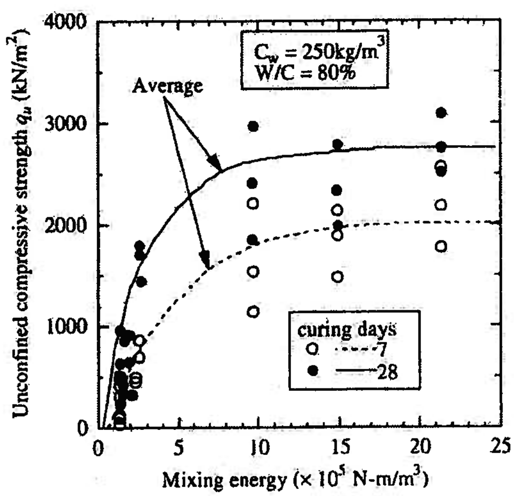


Figure 2.2. Relation between strength and mixing energy in cement-treated soil (Shen et al., 2004, with permission from TRB)

2.2 Factors governing cement-treated soil behaviour

A number of researchers have undertaken studies to analyse factors that control the engineering properties (such as strength) of cement-treated soil, given that it is uniformly mixed. The strength of cement-treated soil test specimens has proven challenging to reproduce with any great degree of repeatability. Therefore, a review of the various factors is reported in order to yield greater confidence in test results presented later in this study.

2.2.1 Curing environment

The curing environment will be different for laboratory versus field conditions. Several aspects of that environment are reviewed in order to select the most suitable and reasonable conditions for curing unconfined compression test specimens.

2.2.1.1 Laboratory techniques

Babasaki et al. (1997) presented some data to further understand the effect of curing temperature (0-30°C) on the peak unconfined compressive strength for cement-treated Chiba silt at five curing periods (1, 3, 7, 14 and 28 days). The data shown in Figure 2.3 clearly indicate a linear trend of higher peak strength with increasing temperature. At 20°C, the data suggest a 10°C increase in curing temperature yields a 20 to 25% increase in strength. The trend establishes the importance of maintaining constant temperature throughout the curing period, if indeed temperature can be controlled as a test parameter.

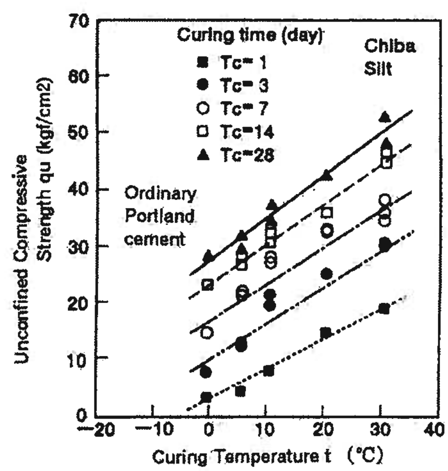


Figure 2.3. Relation between unconfined compressive strength (q_u) and curing temperature (Babasaki et al., 1997, with permission from Balkema)

In order to maintain the water content of a cement-treated soil, many researchers elect to wrap specimens in plastic film (Zhu et al., 2007) or alternatively leave the specimen in its mould (Hodges et al., 2004), before placing them into a moist room with a high relative humidity (90-100%). It is preferable to leave specimens in a sealed mould, however this may not be feasible in some methods of reconstitution. Whichever method is used, the loss of water due to evaporation should be negligible.

Depending on the type of mould used in the reconstitution method, the specimen may require some form of extrusion. If so, then as for any program of testing, the disturbance to the specimen should be minimal in order reduce inconsistencies in the unconfined compressive strength test data.

2.2.1.2 Field observations

In addition to the curing temperature, cement-treated soils will generate their own heat as cement hydration is primarily an exothermic reaction. In the field, heat generation in cement-treated soil is significant and increases the temperature of the treated soil. Heat generation is dependent upon factors such as the cement content and total volume of treated soil. Field measurements (Babasaki et al., 1997) of heat generation indicate in situ temperature increases quite significantly with time. However, the minimum length of heat dissipation path is small in laboratory unconfined compression test specimens in comparison to cement-treated soil columns in the field, suggesting the strength of a laboratory specimen will not significantly increase due to the effects of its own heat generation (discussed again in section 5.3).

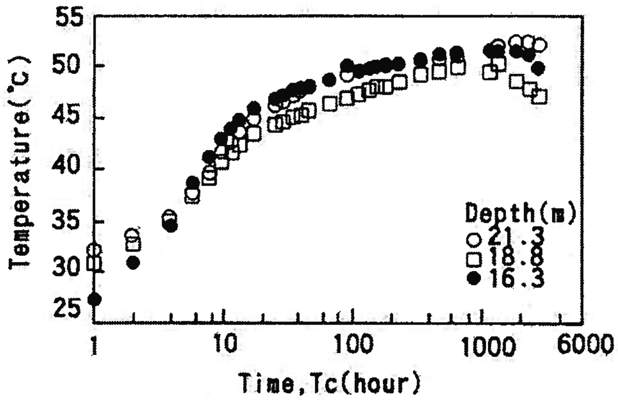


Figure 2.4. On-site measurements of hydration-generated heat in improved ground (Babasaki et al., 1997, with permission from Balkema)

Uchida et al. (1993) observed that there is a large scatter, which was attributed to the sampling procedure, in the unconfined compressive strength data of cored cement-treated soil (Figure 2.5). The peak strength (q_{max}) does not appear to increase significantly with increase in depth of treated soil (i.e. an increase in overburden stress). Rather, the data shown indicate an increase in strength when there is a reduction in the natural water content (w_n). This observation is evident below a depth of 40m, and less noticeably between depths of 30m and 40m. A reduction in water content will reduce the 'total water to cement ratio', provided the cement content is maintained constant. In wet DMM applications, experience suggests a lower total water to cement ratio normally increases the treated soil compressive strength (discussed further in section 2.3.3).

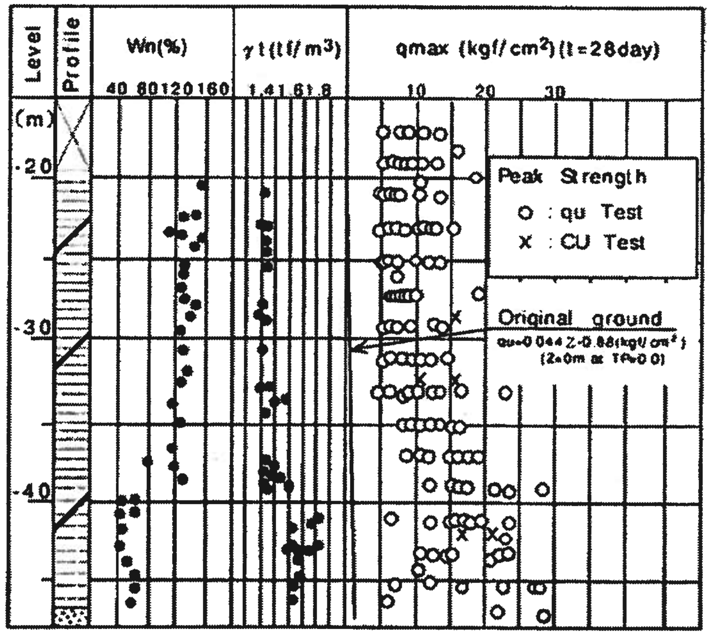


Figure 2.5. Comparison of unconfined compression strength and triaxial compression strength on field improved soil (Uchida et al., 1993, with permission from Balkema)

Uchida et al. (1993) also noted that the peak strength the cement-treated soil is found to be comparable in magnitude to the maximum triaxial (consolidated-undrained) compressive strength, suggesting that unconfined compression testing provides a reasonable index value of peak strength. However, this observation was based upon many scattered unconfined compressive strength data and few triaxial data, and may be considered somewhat speculative.

2.2.2 Physicochemical

The following sections describe several physical and chemical changes a soil undergoes during transformation to a hardened material, as a direct result of cement treatment.

2.2.2.1 Chemistry of Portland cement in soil

The strength gain in cement-treated soil is due to the formation of cementitious hydrates during primary (hydration) reactions and secondary (pozzolanic) reactions (Table A.1). The primary reactions combine Portland cement with water to form calcium silicate hydrate (C-S-H gel), calcium aluminate hydrate (C-A-H gel) and calcium hydroxide (Lea, 1998). Although the hydration reaction is independent of soil type, the cementitious C-S-H and C-A-H gels bind the soil particles together by filling the voids through a series of hydrate crystals. The pozzolanic reaction is caused by the calcium hydroxide reacting with clay minerals that may be present in the soil itself, creating more cementitious hydrates (Hausmann, 1990). The pozzolanic reaction is a much slower reaction (Lea, 1998), which may explain the ongoing strength increase of cement-treated soil over a number of years. Furthermore, soil type will influence the pozzolanic reaction, thus influencing the long-term strength increase. As clays tend to exhibit more pozzolanic reactivity (Hausmann, 1990), it is possible that long-term strength gain is more significant in finer grained materials in comparison to coarser grained, cohesionless soil.

2.2.2.2 Effect of cement content

Generally, an increase in cement content (for the typical wet DMM range of 100-500 kg/m^3 (Bruce and Bruce, 2003)) leads to a more brittle behaviour in cement-treated soil. Experience suggests lower cement content yield relatively large axial strains (2.0 ~ 4.0%) to failure and a ductile response to loading (Figure 2.6), whereas specimens with higher cement content have a clearly-defined peak stress associated with failure at smaller strains (0.5 ~ 1.0%) and exhibit a relatively brittle response (Zhu et al., 2007). However, the specimens tested were cement-treated high plasticity clay, tested in unconfined compression, and likely exhibit an 'insensitive' or 're-molded' clay stress-strain behaviour (Mitchell and Soga, 2005). In contrast, sandy soils treated at low cement content likely have similar stress-strain behaviour to 'dense sands' whereby a clearly-defined peak stress and brittle failure is expected to occur upon increasing strain (Mitchell and Soga, 2005).

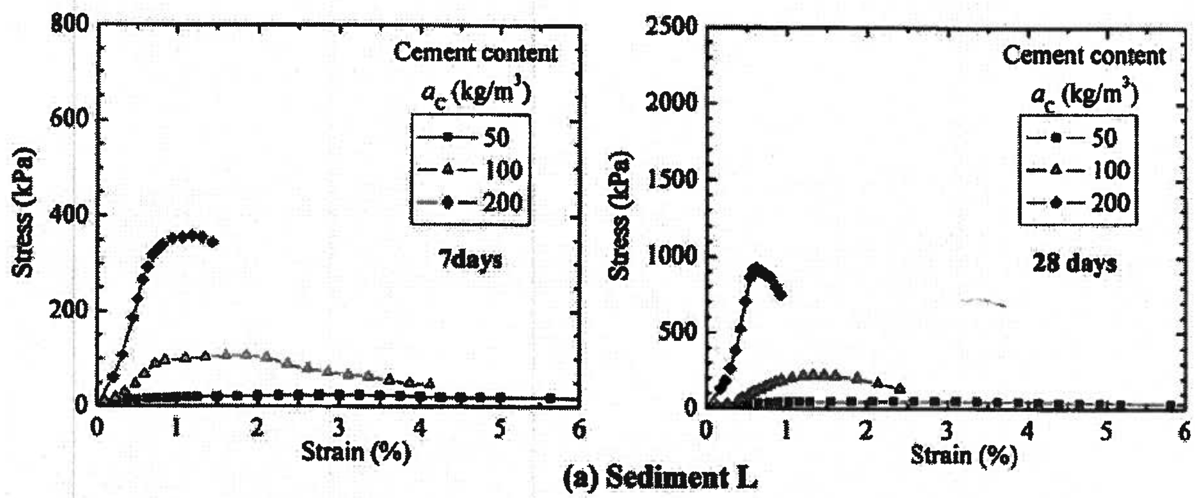


Figure 2.6. Typical stress-strain behaviour of cement-treated soil at low, moderate and high cement contents (Zhu et al., 2007, with permission from ASCE)

2.2.2.3 Effect of water content

At the beginning of curing, the cement-treated soil specimen consists of a percentage of water (w_p), soil and cement (Figure 2.7). Over time, the water content of the specimen will diminish (by w_h) due to the consumption of water and formation of new solids (hydrates) in cement hydration. As the cement content increases, the value of w_h increases. A small amount of water (w_e) will also evaporate (as a result of increased heat of hydration), and again, it is a function of the cement content. Zhu et al. (2007) illustrated this mechanism in a schematic diagram of soil-water transfer, which is modified in Figure 2.7. Loss of water due to evaporation is usually negligible, and varies depending on the mixing and curing procedures. Accordingly, the relation between water content at the beginning of the curing period, and water content taken at the time of failure (w_f), is given by:

$$w_p = w_e + w_h + w_f \quad 2.1$$

In addition, Zhu et al. (2007) showed a steeper reduction in 'PW content' (equivalent to w_f) associated with longer curing periods (Figure 2.8) upon an increase in cement content.

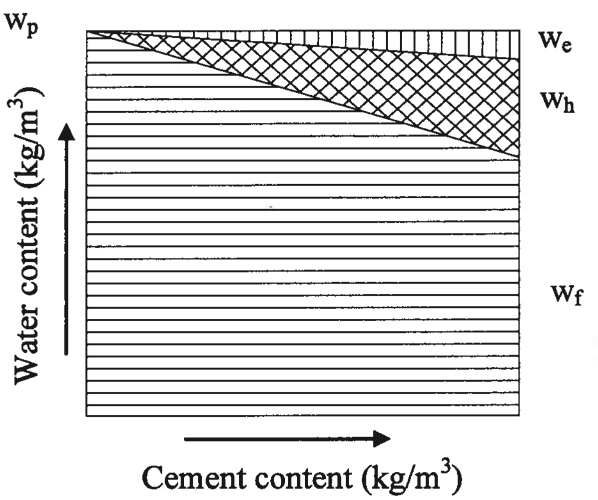


Figure 2.7. Soil-water transfer model (modified from Zhu et al., 2007)

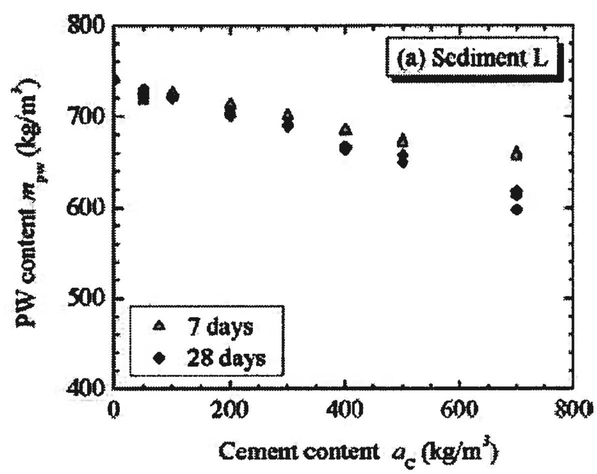


Figure 2.8. Effect of cement content on the water content (Zhu et al., 2007, with permission from ASCE)

2.3 Peak strength of cement-treated soil

The following sections discuss the influence of soil type, cement content, time and water-cement ratio on existing strength data of cement-treated soil. Although the method of specimen reconstitution and soil types examined in testing vary greatly, it is possible to interpret general trends as a basis for interpretation and discussion of results obtained in this study.

2.3.1 Soil type

The type of soil that is treated with cement exerts a very significant influence on peak strength. The particle size, fines content and mineralogy all influence the fabric of the cement-treated soil and therefore its mechanical resistance to loading. Taki and Yang (1991) presented wet DMM field data showing the peak unconfined compressive strength for a gravel, a sandy and a clayey soil. The recovery methods used were 'field wet' (a large block sample taken at the time of mixing which is then transferred into smaller moulds and cured) and 'cored' (from hardened treated soil) samples. Both methods of sampling produced significant scatter, however it was possible to discern trends. A clear distinction existed between the treated clayey soil and the treated sand and gravel, with the latter soils exhibiting significantly improved compressive strength. Another key point in their findings is that the increase of strength in cohesive soils due to an increase in cement content is minor in comparison to sandy and gravely soils. Laboratory data from Taki (2003) also suggest a significantly higher strength (approximately 1 to 2 MPa) in cement-treated silty sand compared with cement-treated clay, tested at equal cement content. Zhu et al. (2007) prepared laboratory specimens of cement-treated clays and a silt, at equal cement content and similar water content. The unconfined compressive strength data at a curing period of 28 days showed a lower strength (1.0 and 1.4 MPa) for the clays in comparison to the silt (2.1 MPa). This suggests that properties inherent in the original soil significantly affect the strength of the cement-treated soil. However, while the aforementioned work has largely been concerned with clay versus coarser grained material, the knowledge-base for the influence of silt in coarser grained treated soil strength remains much less studied.

2.3.2 Cement content

Cement content is arguably the most influential factor governing strength and stiffness of cement-treated soil, assuming an appropriate water/cement ratio is achieved with regard to the initial ground conditions and the type of DMM. Various authors have presented 28 day (most commonly reported curing period) peak unconfined compressive strength data of cement-treated soil in terms of the cement content, but in very different ways (see Table 2.1). Close inspection of the cement content is required to compare the trends of this commonly reported variable across laboratory studies. Much research has been done on clayey soils (Matsuo et al., 1996; Taki, 2003; Bergado and Lorenzo, 2005; Filz et al., 2005; Zhu et al., 2007 and Liu et al., 2008), although silts and sands remain much less studied with respect to the DMM. Taki (2003) and Filz et al. (2005) however have provided some silt/sand data, however, there is clearly a substantial shortfall in the number of specimens tested (to allow trends to be established),

Table 2.1. Comparison illustrating the effect of 'cement content' on the 28 day unconfined compressive strength of various soils between DMM laboratory studies

Study	Soil description	Cement per unit volume of wet soil (kg/m^3)	Cement per unit volume of specimen (kg/m^3)	Cement per unit volume of initial mix (kg/m^3)	Cement to dry soil (%)	Cement slurry to wet soil (%)	q_u (MPa)
Matsuo et al. (1996)	Marine clay	86 - 214		74 - 167	13 - 32	15 - 30*	1.4 - 7.4
Bergado and Lorenzo (1998)	Bangkok clay	58 - 113		29 - 61	10 & 15*		0.3 - 1.0
Taki et al. (2003)	Soft silty clay	200 - 300*					1.7 - 4.9
	Silty sand	200 - 300*					4.1 - 7.2
Filz et al. (2005)	Light Castle sand		160 - 440*	170 - 650*	11 - 40*		1.3 - 6.4
	Northern VA sandy lean clay		150 - 360*	190 - 970*	10 - 52*		2.5 - 6.7
	P2 silty and clayey sand		150 - 370*	180 - 940*	13 - 73*		0.6 - 5.0
	Vicksburg silt		150 - 360*	190 - 970*	12 - 61*		0.8 - 4.7
	Washed Yatesville silty sand		150 - 260*	170 - 350*	10 - 20*		0.6 - 3.7
Zhu et al. (2007)	Clay (lake)	50 - 700*		49 - 573	6 - 85		0.1 - 10
	Clay (marine)	50 - 700*		49 - 573	6 - 81		0.2 - 13
	High plasticity silt (river)	50 - 700*		49 - 573	6 - 82		0.1 - 15
Liu et al. (2008)	Lianyungang marine clay	530 ~ 540	170 - 270	150 - 226	20*		1.3 - 4.2

*Indicates cement content range specifically reported. Un-starred cement content ranges are calculated values based on additional information supplied by the authors. Missing cement content ranges are either not applicable or difficult to determine from the information given.

compared to that of clayey soils. Table 2.1 shows a compilation of 28 day unconfined compressive strength data from the aforementioned authors for various ranges of cement content. Taki (2003) showed that treated silty sands can be improved to at least a 7.2 MPa peak compressive strength, whereas Zhu et al. (2007) showed clayey soils can exhibit significantly higher strengths of about 15 MPa. However, the cement content attributed to the 15 MPa strength (Zhu et al.) was more than twice the cement content of the 7.2 MPa strength (Taki), which explains the significant strength increase. Other trends from the literature not apparent upon inspection of Table 2.1 show that it appears that silts and sands generally exhibit a steep non-linear increase in strength with an increase in cement content, whereas clays tend to show a more gradual increase in strength upon an increase in cement content. However, Filz et al. (2005) showed that the peak strength of 'P2 silty/clayey sand' clearly began decreasing upon surpassing a threshold cement content, suggesting in this case that there appears to be a limit to cement-treated sand strength improvement, which is not readily apparent from the trends in the literature pertaining to cement-treated clays.

To summarise, cement-treated clayey soils typically require a larger cement content in order to exhibit a significant improvement in their peak strength. In contrast, silts and sands generally exhibit significantly improved strength with a more modest cement content. However, due to the variability in soil type and its natural water content, the cement content does not accurately describe the coupled interaction of water and cement on the cement-treated soil strength, making it difficult to discern trends across laboratory studies.

2.3.3 Time effects

Many studies (such as Taki, 2003; Zhu et al., 2007) report data on the short-term strength of cement-treated soil, which illustrate a significant increase during the first 28 days of curing. However, long-term strength remains a largely unstudied phenomenon, especially in North America. Recognising that strength gains occur over curing periods of years, most studies simply do not have the resources to monitor comprehensively the effect of time. However, Kongsukprasert et al. (2007) have compiled some data from Japan with some from their own work to establish a useful overview of strength gain with time in various cement-treated soil mixes. Across these studies, there appears to be a significant influence of time on the unconfined compressive strength. Their own data also showed that the compressive strength of cement-treated sand could increase by a factor of three over a long period of time (3 years).

2.3.4 Total water-cement ratio

Saturated soils, which have been treated with cement, vary tremendously in terms of their natural water content. Sand deposits generally have lower natural water contents (typically an upper limit in the region of 30-35%) than silty deposits (typically in excess of 40%). Further, in order to facilitate the wet DMM mixing process, water is injected (Larsson, 2005) to fluidize the soil and also added when delivering the cement slurry before mixing. Accordingly, the total water content of the improved ground after mixing is governed by the natural water content and by decisions related to operation of the equipment on-site.

Table 2.2. Comparison illustrating the effect of 'total water-cement ratio' on the 28 day unconfined compressive strength of various soils between laboratory studies

Study	Soil description	Total water-cement ratio	q_u (MPa)
Zhu et al. (2007)	Clay (lake)	0.9 - 11.7*	0.1 - 10.0
Zhu et al. (2007)	Clay (marine)	0.9 - 12.1*	0.2 - 13.0
Zhu et al. (2007)	High plasticity silt (river)	0.9 - 12.3*	0.1 - 15.0
Filz et al. (2005)	Light Castle sand	1.0 - 2.7	1.3 - 6.4
Liu et al. (2008)	Lianyungang marine clay	1.5 - 3.6	1.3 - 4.2
Filz et al. (2005)	Northern VA sandy lean clay	1.5 - 3.2	2.5 - 6.7
Filz et al. (2005)	Vicksburg silt	1.6 - 3.5	0.8 - 4.7
Filz et al. (2005)	P2 silty and clayey sand	1.6 - 3.9	0.6 - 5.0
Filz et al. (2005)	Washed Yatesville silty sand	1.7 - 2.6	0.6 - 3.7
Kongsukprasert et al. (2007)	Aomori sand	3.6 - 6.1*	0.2 - 1.2
Matsuo et al. (1996)	Marine clay	4.3 - 10.0*	1.4 - 7.4
Bergado and Lorenzo (1998)	Bangkok clay	5.6 - 16.6*	0.3 - 1.0

*Indicates total water-cement range specifically reported. Un-starred cement total water-cement ratios are calculated values based on additional information supplied by the authors.

The total water-cement ratio describes the ratio, by mass, of total water in cement-treated soil to dry cement. Currently, few studies explicitly present the total water-cement ratio. However, Filz et al. (2005) and Liu et al. (2008) relate the total water-cement ratio to 28 day unconfined compressive strength, as shown in Table 2.2. Other studies (Matsuo et al., 1996; Bergado and Lorenzo, 1998; Zhu et al., 2007 and Liu et al., 2008) also provide enough information to enable calculation of the total water-cement ratio, also shown with the 28 day unconfined compressive strength in Table 2.2. High total water-cement ratios are typically associated with low peak strengths, whereas low ratios are attributed to high peak strengths. Filz et al. (2005) have shown that, for several soil types, the strength generally follows a decreasing non-linear trend with increasing total water-cement ratio. Using the total water-cement ratio appears to quantify elegantly the effect of water and cement on the cement-treated soil strength, across DMM laboratory studies.

2.4 Summary

There appears to be a fairly sizable knowledge-base on the wet deep mixing method (DMM), mostly from Japanese sources, which examine various contributions to the behaviour of cement-treated soil. Filz et al. (2005) reported unconfined compression data for 5 tests on a silt and 7 tests on a sand. Whilst the number of data was limited, it became apparent that their method of reporting strength against both water and cement as a single variable seemed a useful technique and one that will be further explored in the current study. Kongsukprasert et al. (2007) provide valuable data reporting the long-term strength increase of cement-treated soil with time. Such work is important to the current study, and thus will be given further consideration.

Several researchers have reported techniques to improve the method of reconstitution, and examined factors which govern the behaviour of cement-treated soil, with the intent of enabling progress towards a quality controlled laboratory based data set. In wet DMM applications, water and a binder (Portland cement) is injected into the soil and subsequently mixed in place under saturated conditions, without any compaction effort. At present there is no standard laboratory test method which replicates this process. Furthermore, there is limited unconfined compressive strength data, from laboratory testing, on silty/sandy soils pertaining to wet DMM. Lastly, strength to water/cement relations established from recent work is not universally defined, causing difficulty in discerning trends across studies. Accordingly, there is need for additional work in an attempt to provide a basis for standard laboratory unconfined compression testing of reconstituted wet DMM soil. Interpretation of arising test data will assist engineering consultants in decisions related to strength parameter inputs for wet-mixing applications.

3 MATERIALS

The specimens used in testing were reconstituted from a mix of sand, silt and cement. Properties of the two soils are reported with reference to index test data. Properties of the cement are reported from information provided by the commercial supplier. Scanning electron microscope (SEM) images of the test materials were captured on a Hitachi S-4700 Field Emission SEM at the University of British Columbia BioImaging Facility.

3.1 Sand

The sand was obtained from a stockpile created by industrial dredging of the Fraser River. The stockpile is located at the bottom of No. 3 Road, Richmond (suburb of Vancouver). Permission to access the stockpile was given by Mr. Rob Millar of Mainland Sand & Gravel Ltd. Fraser River sand was chosen for this study because it comprises a very high percentage of sand sized particles (approximately 99% in the 75 μm to 2.0 mm range, see Figure 3.1). It is a poorly-graded (D_{50} = 0.23 mm, C_u = 1.6) fine to medium sand. Inspection of an SEM image (see Figure 3.2) reveals the grains are angular to sub-angular in shape. The specific gravity was assumed to be 2.75 (Chillarige et al., 1997).

Sand from the stockpile was passed through a large sieve, with 2.0mm openings, in order to remove any coarse lumps or foreign objects. It was then stored at the laboratory in covered plastic buckets until it was needed for testing.

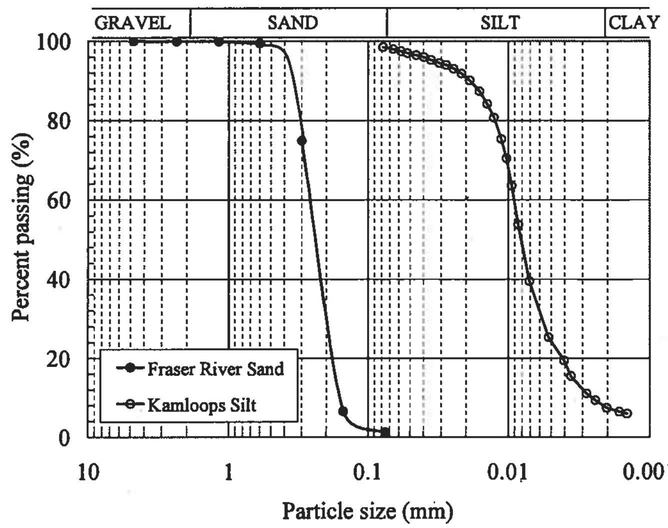


Figure 3.1. Particle size distribution of Fraser River sand and Kamloops silt

3.2 Silt

The silt was obtained from bluffs at Kamloops, British Columbia. The soil is a lacustrine deposit that contains a significant proportion of (approximately 90% in the 2 μm to 75 μm range, see Figure 3.1) silt-sized particles. It is a poorly-graded clayey silt (D_{50} = 8.1 μm, C_u = 3.4) of low plasticity, I_P = 8.4%, (Lum, 1979). The specific gravity was assumed to be 2.77 (Lum, 1979). A scanning electron microscope image is shown in Figure 3.2. It was unnecessarily time consuming to sieve the soil through a 75 μm mesh screen to remove all sand particles: instead it was sieved through a 250 μm mesh screen to remove coarse lumps and foreign objects. Knowing that some fine sand particles remain (about 4% in the 75 μm to 250 μm range), and knowing that the Fraser River sand and Kamloops silt would later be combined during specimen reconstitution, it was most practical simply to account for this small portion of fine sand in subsequent calculations. Kamloops silt has the attraction of having a very low natural water content of 2 to 3%, making it easy to handle and process during the specimen reconstitution procedure.

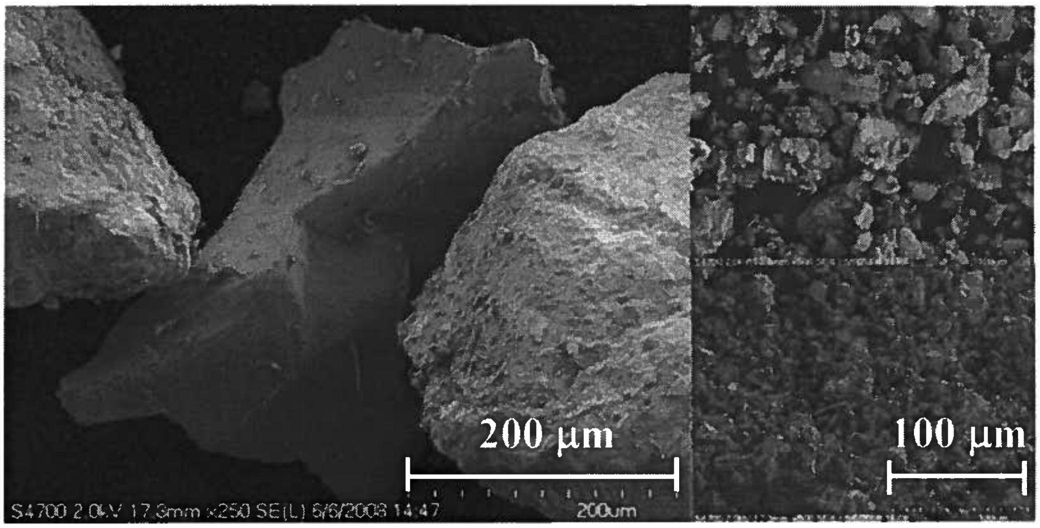


Figure 3.2. Scanning electron microscope images (to scale) of Fraser River sand (left), Kamloops silt (top right) and Portland cement (bottom right).

3.3 Cement

Cement was obtained from Lafarge North America Inc., at a Vancouver-based branch. It was a Type GU (or Type 1 in U.S. classification) Portland cement, supplied in 40 kg bags. The specific gravity was assumed to be 3.15 (Lafarge North America Inc., 2005). The cement meets all applicable chemical and physical requirements of ASTM C 150 and CSA A3000-03 (formerly A5-98). A scanning electron microscope image is shown in Figure 3.2.

4 APPARATUS AND TEST PROCEDURE

A new test device is described, together with a method of unconfined compression testing. A new method for reconstitution of a saturated cement-treated soil specimen is also described, which enables the mass proportion of sand, silt and cement to be controlled precisely.

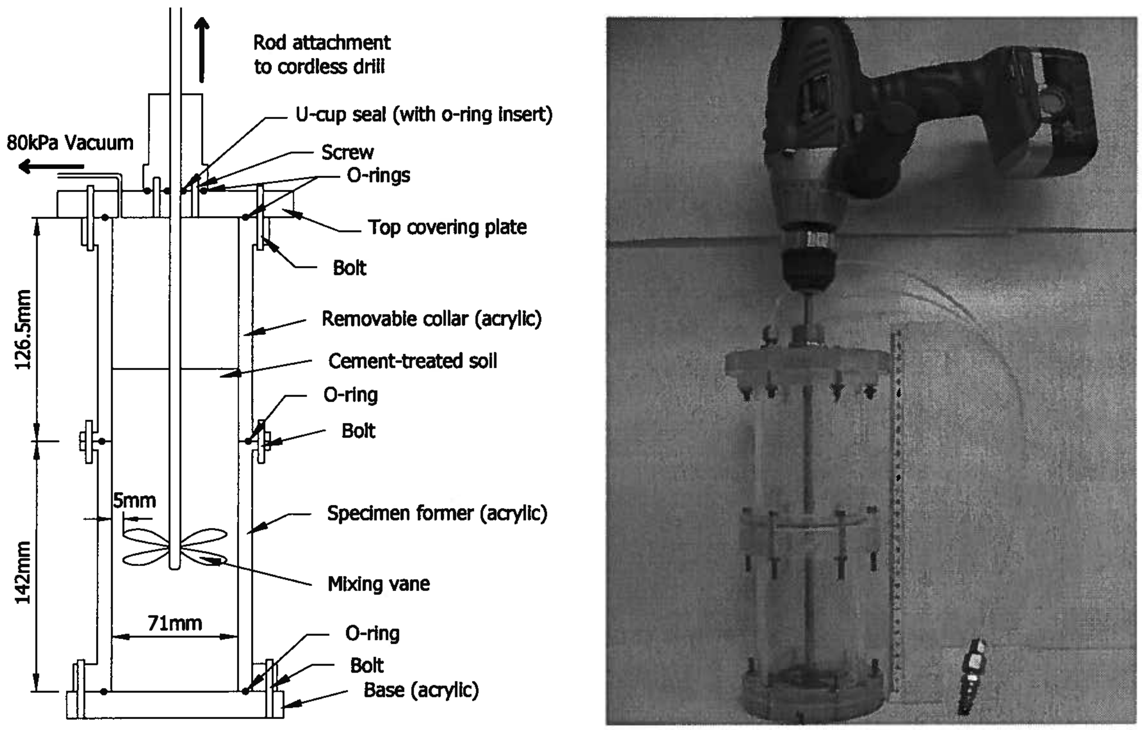
4.1 Test device

In order to develop a method for mixing a homogeneous and saturated cement-treated soil specimen there was a need to design and manufacture laboratory equipment. To provide specimens of good quality, the device had to meet the following conditions:

1. Mould a cement-treated soil specimen 71 mm (2.8") in diameter and 142 mm (5.6") in length, as described in ATSM D 1633-00.
2. Thoroughly blend the cement-treated soil at a high mixing energy.
3. Ensure that no air is entrained throughout the reconstitution procedure.

4.1.1 Configuration of test device

Acrylic was used for the base, specimen former, removable collar, and also the top covering plate of the mould assembly (Figure 4.1a) due to its ease of manufacture and the ability to assess, visually, the condition of the specimen. The cylindrical sections and top/bottom coverings were connected using a series of 18 bolts. O-rings were placed into grooves at the section/covering joints enabling the device to be sealed when the bolts are tightened and a vacuum applied.



(a) (b)

Figure 4.1. Mould assembly and vane (a) photograph and (b) schematic diagram

A metallic guide was attached, using screws, to the top covering plate. The guide provides alignment for the mixing vane, which passes through the centre of the plate. An additional groove on the underside of the guide allows a U-cup seal and O-rings to be positioned between the top covering plate and the guide. The U-cup seal (with O-ring inserted inside) sits tight against the rod attachment preventing entrainment of air during rotation. The top covering plate has an additional port that connects to a vacuum outlet during mixing. The fully assembled device is shown in Figure 4.1b together with a 14.4 V cordless drill used to rotate the mixing vane.

4.2 Compression system

The following section describes the unconfined compression frame (configured to the requirements of ASTM D 1633-00), and provides a brief summary of the data acquisition system.

4.2.1 Compression frame

A 49.8 kN capacity Wykeham-Farrance compression machine was modified in order to attach a load cell to the upper restraint (see Figures 4.2 and 4.6). The test specimen mounted between smooth top and bottom platens (88 mm in diameter, with bearing faces that are planar within 0.01 mm). The top platen was machined to allow for a ball bearing to be seated between it and the load cell.

4.2.2 Data acquisition system

Output voltages from the load cell (measuring axial force) and from a LVDT (measuring axial displacement) were recorded by a data acquisition system. This system comprises of a signal conditioning unit which amplifies the output signals, a 12 bit resolution DAS board with a digital input/output, a desktop computer and data acquisition software (LabTech Notebook by Laboratory Technologies Corporation). The software records the data at a rate of 20 Hz and stores an output file on the desktop computer.

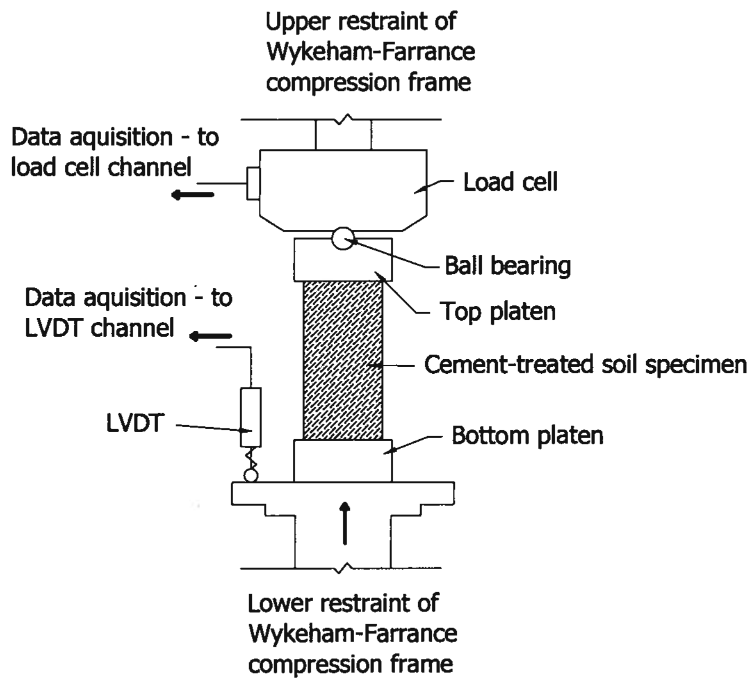


Figure 4.2. Compression system

4.3 Specimen reconstitution

Application of a new method of reconstitution is described with reference to unconfined compression testing of sand-silt-cement specimens.

4.3.1 Sand

A specified mass (M_S) of Fraser River sand was brought to a fully saturated state by boiling, in a conical flask of de-aired water, for a period of 30 min. Upon cooling to room temperature, the sand was 'pluviated' into an acrylic mould (Figure 4.1) containing de-aired water. This was done by filling the remainder of the flask with de-aired water, and plugging its opening with a rubber bung through which passes a glass tube. Filling the tube with de-aired water and sealing it by thumb enables the flask to be turned upside down, allowing immersion of the tube in the mould. Uncovering the tube causes sand particles to pluviate from the flask, yielding a loose saturated deposit in the mould (see, for example, Fannin et al. 1994). Manual agitation of the mould consolidates the deposit, reducing the void space (e) and hence the water content (w).

4.3.2 Silt

Concurrently, a specified mass (M_M) of Kamloops silt was also boiled in de-aired water, for a period of 1 h., in order to remove entrapped air. The beaker was then placed in a desiccator (while still at a temperature of approximately 100 °C) in order to accelerate the de-airing process by further boiling under a vacuum for a period of 15mins; as the elevated temperature of the slurry reduces to the ambient room temperature of 21 °C, imposing a slow increase in the vacuum pressure (to a final value of 80 kPa) maintains a gentle boiling action. The abovementioned steps are necessary because experience has shown the silt slurry does not release entrapped air with the same ease as the sand slurry. Upon cooling to room temperature, the silt was poured from the beaker into the mould containing saturated sand, with care taken to minimise any entrainment of air. Any residue of silt left in the beaker was weighed in order to correct the mass of silt deposited on top of the sand in the mould, and thereby determine the total mass of solids. Leaving the specimen for several hours allows self-weight consolidation of the silt layer to occur. Excess water is removed at this stage using a syringe (whilst leaving a film of water to cover the top of the silt).

4.3.3 Cement

A pre-determined mass of Type I Portland cement powder (M_C) was 'air-pluviated' onto the standing film of de-aired water above the silt layer (Figure 4.3). The quantity is selected to yield a target mass of cement per unit volume of total mix, (c_i kg/m^3). Thereafter, and knowing the total mass of soil and cement, de-aired water was added to the slurry by means of a graduated syringe to achieve a target mass of water (M_w) and thereby control the initial water content of the cement-treated soil mixture (w_i):

$$w_i = \frac{M_w}{M_S + M_M + M_C} \qquad 4.1$$

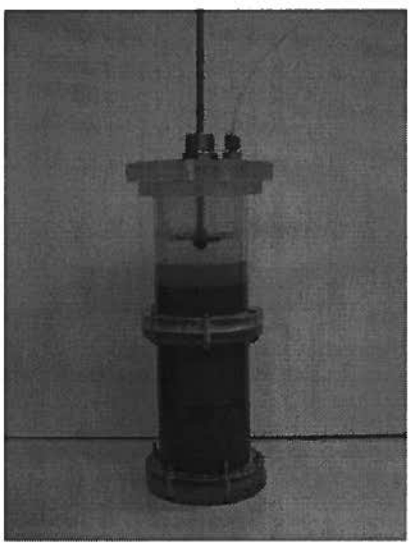


Figure 4.3. Initial cement-treated soil (just before mixing)

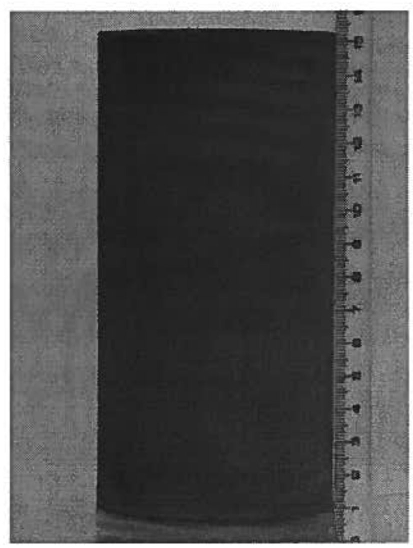


Figure 4.4. Cured specimen (just before testing)

4.3.4 Wet mixing

After securing the top covering plate of the mould with bolts (Figure 4.1), a vacuum pressure of 80kPa is applied and the mixing vane then lowered slowly (Figure 4.3). The vane is connected to a commercial ¾-inch drill, operated at a speed of approximately 1150 RPM. A 5 mm clearance exists between the tip of the vane and the inside wall of the mould. It is lowered and raised at a rate of approximately 3 secs per cycle. Mixing the materials under a vacuum ensures no air is entrained during the blending action. These actions impose a relatively high mixing energy per unit volume, which is believed to reduce specimen variability (Shen et al. 2004). The cement-treated soil is mixed until visual observations establish it is uniform in colour and the consistency is found similar across the depth of slurry, which typically requires a mixing period of 5 min. Thereafter the top plate is removed and the vane withdrawn, allowing the mould (with removable collar still attached) to be placed in a moist room for purposes of curing (with its exposed top surface covered by plastic film), at a very high relative humidity and a temperature of 23°C (±1°C).

4.3.5 Trimming and specimen extrusion

'Bleed' water (w_b) appears on the top surface of the specimen as a result of self-weight consolidation: it is removed and weighed once the cement-treated soil attains a strength comparable to that of soft clay (typically after 1 to 5 h., depending on the proportional mix of solids). Thereafter, the collar extension to the mould is removed, and excess material trimmed using a wire saw. This is done in order to obtain a specimen length equal to that of the rigid-walled former (Figure 4.1a), and also to smooth the exposed top surface of the reconstituted mixture flush with the top of the former. The resulting trimmings are used for determination of water content at the onset of curing (w_p) and the cement content at this time (c_p), which is assumed equal to that when tested to failure (c_f). Degree of saturation (S_r) is back-calculated from these data as a measure of quality control. Statistical analysis of a total of 147 test specimens established a mean value S_r = 100.4 % and a standard deviation of 0.7%, which provides valuable confidence in the wet-mixing method yielding a saturated cement-treated soil.

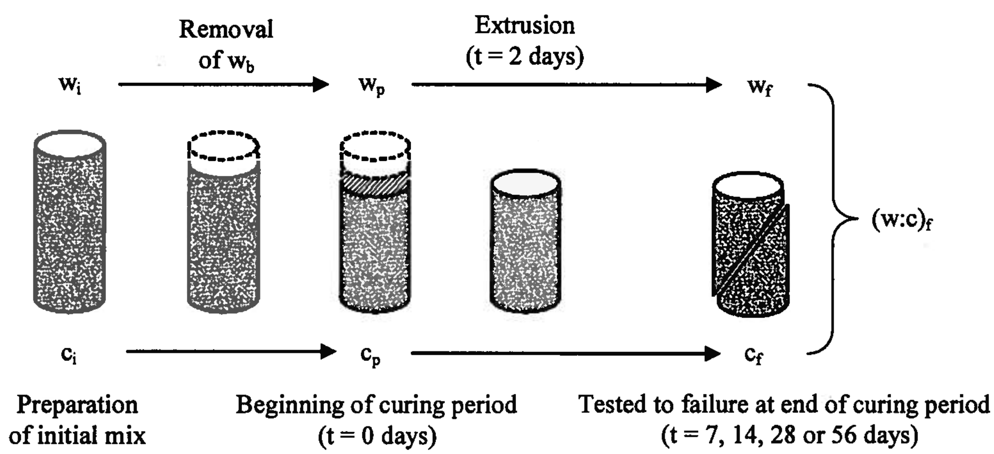


Figure 4.5. Schematic showing the water content (w_i, w_p, w_f) and associated cement content (c_i, c_p, c_f) of the cement-treated soil at various stages of specimen reconstitution

After trimming, the specimen is returned to the moist room (again with its exposed top surface covered by thin plastic film) for a period of 48 h., during which time it gains sufficient strength to enable extrusion. The base of the mould is first removed, taking care to avoid any damage to the bottom surface, and the specimen is then pushed out of the former in a vertical and upwards direction using a loading frame adapted for this purpose. The frame incorporates a plunger of diameter 0.1mm less than the inside diameter of the former, to ensure a uniformly applied pressure across the base of the specimen. After extrusion, the specimen is carefully wrapped in plastic film and returned to the moist room for the remainder of the intended curing period. The final dimensions of the reconstituted cement-treated soil specimen (Figure 4.4) are approximately 71 mm (2.8") in diameter and 142 mm (5.6") in length, as required by ASTM D 1633-00.

4.3.6 Compression testing

Upon completion of curing, if inspection reveals any slight unevenness on the top and bottom surface of a specimen, it is smoothed by gentle rubbing with medium sand-paper. Specimen mass, length and diameter are recorded prior to its being mounted it the loading frame. Unconfined compression testing is performed at a constant rate of displacement of 1 mm/min (0.7 %/min), with automated measurement of axial force and displacement.

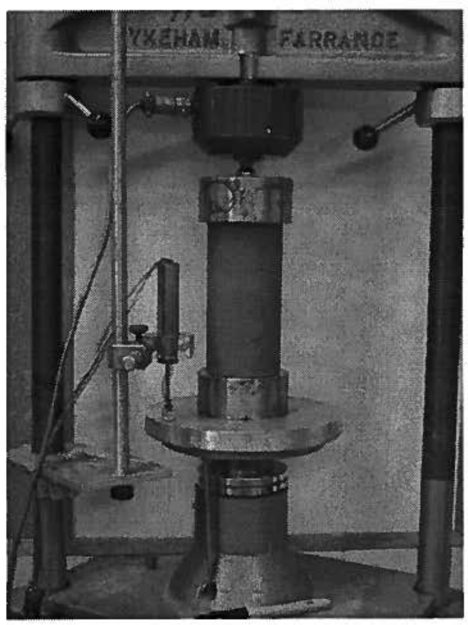


Figure 4.6. Loading of specimen in compression frame

The specimen is loaded to failure, with data recorded to an axial strain typically between 0.5 and 4.5%. Upon inspection of the resulting shear plane, the entire specimen is oven dried to establish the water content at failure (w_f) so that the water-cement ratio at failure, $(w{:}c)_f$, can be calculated

(Figure 4.5). Figure 4.6 shows a typical specimen mounted in the compression frame prior to testing.

4.4 Test Program

The program of testing was conceived to investigate the influence of sand-silt ratio (*S:M*), cement content (*c*) water content (*w*) and time (*t*), on the unconfined compressive strength (UCS) of cement-treated soil. Sand to silt ratios in the range 25S:75M to 90S:10M were investigated, at three values of cement content (c_L, c_M, and c_H) and three values of water content (w_L w_M, and w_H). Four different curing periods (7 to 56 days) were examined. The program of testing is summarised in Figure 4.7. The total number of reconstituted cement-treated soil specimens in the main program of testing was 144.

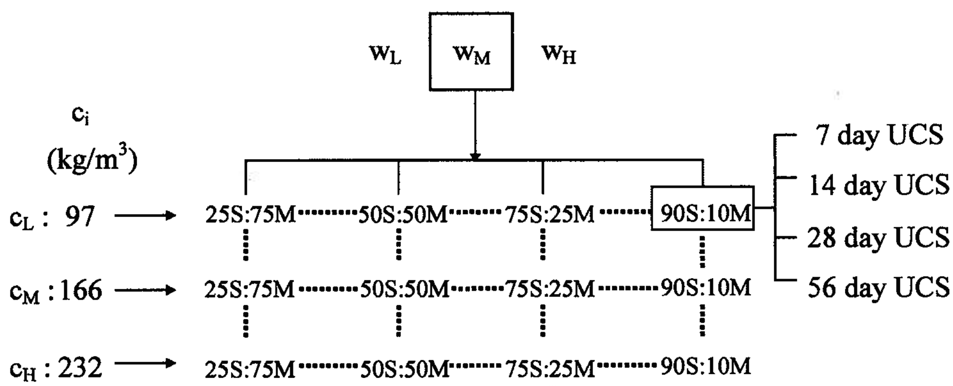


Figure 4.7. Test program matrix

The ranges used for the proportioning the initial mixes of cement-treated soil (*S:M*, *c*, *w* and *t*) were obtained upon consultation by the industry sponsor, Golder Associates Innovative Applications (GAIA). Sand-silt (*S:M*) ratios of 25S:75M to 90S:10M typically describe soil conditions encountered by GAIA in the Fraser River delta, British Columbia. Three different values of cement content (*c*) were examined such that a 'weak', an 'ideal' and a 'strong' specimen would be tested for strength in order to establish trends and optimize mixing design. Water content (w) was believed to be a significant variable influencing the strength of cement-treated soil, therefore three different values of water content were examined such that trends would be established. The range of water content was believed to be representative of cement-treated soil in the Fraser River delta. Specimens were tested at four different curing times (t) to allow for an appreciation of strength gain with time and comparison with other studies. A curing period of 56 days was the limit on what could be conveniently achieved, given the time frame of the current study.

5 TEST RESULTS

Results from commissioning of the apparatus and test procedure are first presented, followed by scanning electron microscope images of selected test specimens. The results of the main program of unconfined compression tests are then described.

5.1 Commissioning of test procedure

In order to verify the method of reconstitution is reproducible, four nominally identical specimens were prepared at a sand-silt ratio of 75S:25M, with a water content w_i = 38.5 % and a cement content c_i = 232 kg/m^3.

5.1.1 Reproducibility of specimens

The range in specimen water content at the beginning of curing, $34.9 \leq w_p \leq 35.6$ %, and the range in companion values of bulk density, $1.91 \leq \rho_p \leq 1.93$ Mg/m^3, are both relatively narrow (see Table 5.1). Bulk density reduces with increasing water content in a very consistent manner (Figure 5.1). Given the remarkably small variation in these two parameters, and the good

Table 5.1. Duplicate specimens cured for 7 days (75S:25M, w_i: 38.5 %, c_i: 232 kg/m^3 of total mix)

Specimen	w_p (%)	w_f (%)	ρ_p (Mg/m^3)	c_f (kg/m^3)	q_u (MPa)
#1	35.0	28.6	1.93	248	1.94
#2	35.6	29.2	1.91	245	1.88
#3	35.3	28.3	1.92	246	1.90
#4	34.9	27.9	1.92	247	1.92

trend in their correlation, the new method of reconstitution by wet-mixing of a saturated slurry is believed highly reproducible. More generally, the data also illustrate the importance of achieving a precise control of water content during reconstitution in order to obtain repeatable results.

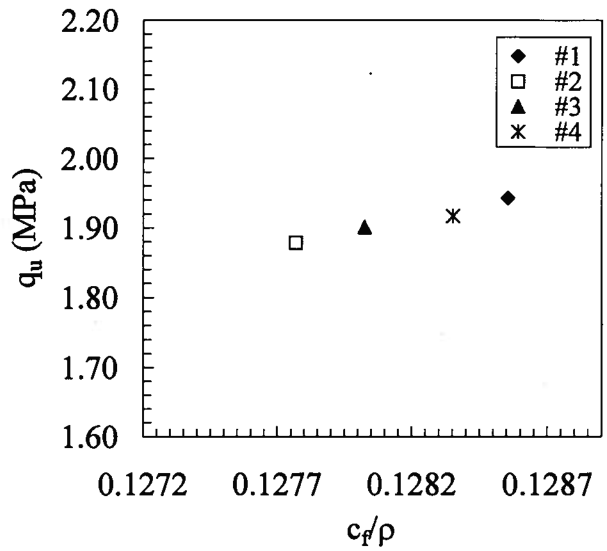


Figure 5.1. Bulk density vs. water content at beginning of curing

All four specimens were cured for a period of 7 days prior to testing, which yielded an unconfined compression strength $1.88 \leq q_u \leq 1.94$ MPa (see Table 5.1). The repeatability of these strength values is considered excellent. The unconfined compressive stress vs. axial strain plot (Figure 5.2) further illustrates a very similar response to loading in the four specimens up to, and indeed just beyond the peak strengths mobilized at an axial strain of approximately 0.55 %. The uniformly stiff response follows some non-linearity at small axial strains of less than 0.1 %. In small strain stiffness analysis, it is useful to determine Young's modulus (E) and Poisson's ratio (v), and experience shows (Kohata et al., 1997) that further local strain measurements may be needed to correct for a 'bedding error'. The scope of this study focuses on the peak strength of specimens comprising four variables (S:M, c, w and t), however it is acknowledged that for some geotechnical engineering applications, an analysis of deformation is more appropriate.

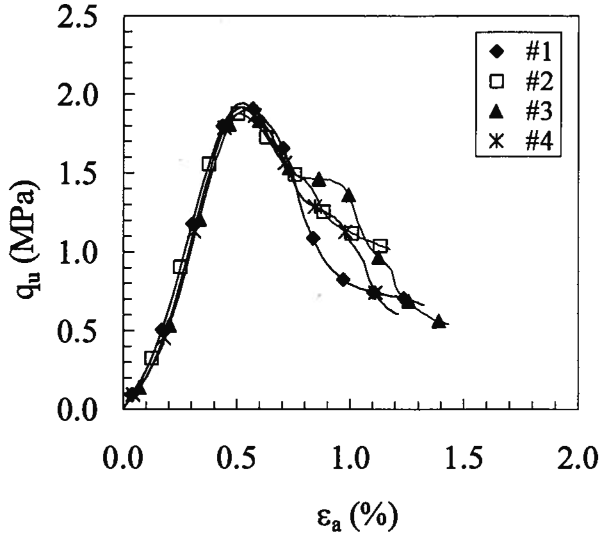


Figure 5.2. Unconfined compressive stress vs. axial strain

Inspection of Table 5.1 shows the water content at failure (w_f) is significantly less than that at the beginning of curing (w_p), a finding that is attributed primarily to the phenomenon of water consumption by the cement in the chemical reactions of hydration. A further key to understanding the utility of the reconstitution method is provided in the variation of q_u with normalized cement content (Figure 5.3), which shows the strength to increase with cement

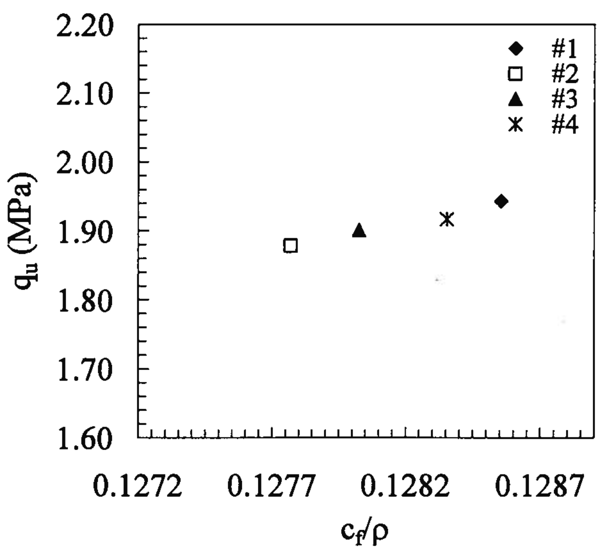


Figure 5.3. Unconfined compressive strength vs. normalized cement content

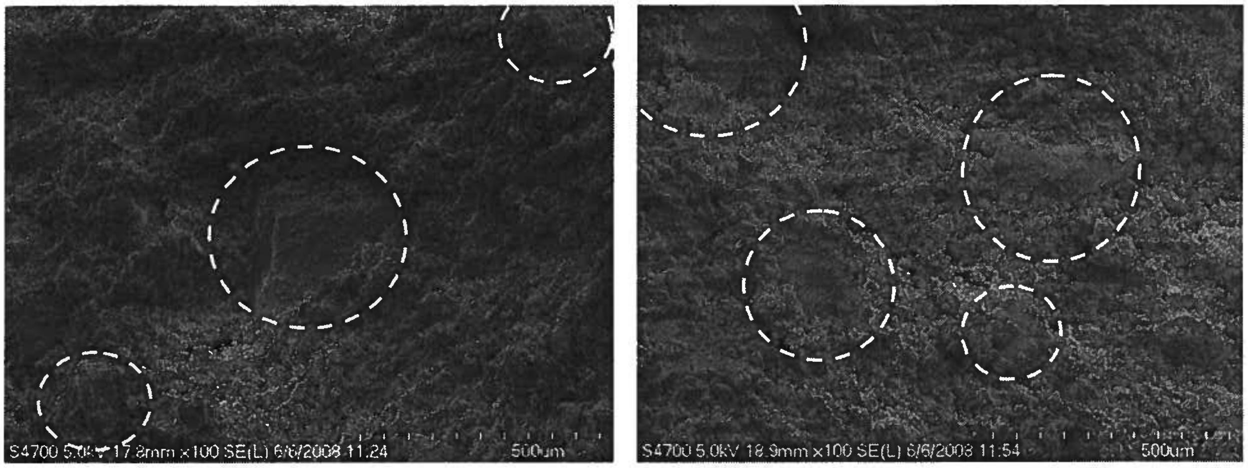
content. This subtle trend across a narrow range in parameters yields even greater confidence in the success of the proposed new method for reconstitution of saturated cement-treated soil by wet-mixing.

5.2 Scanning electron microscope images

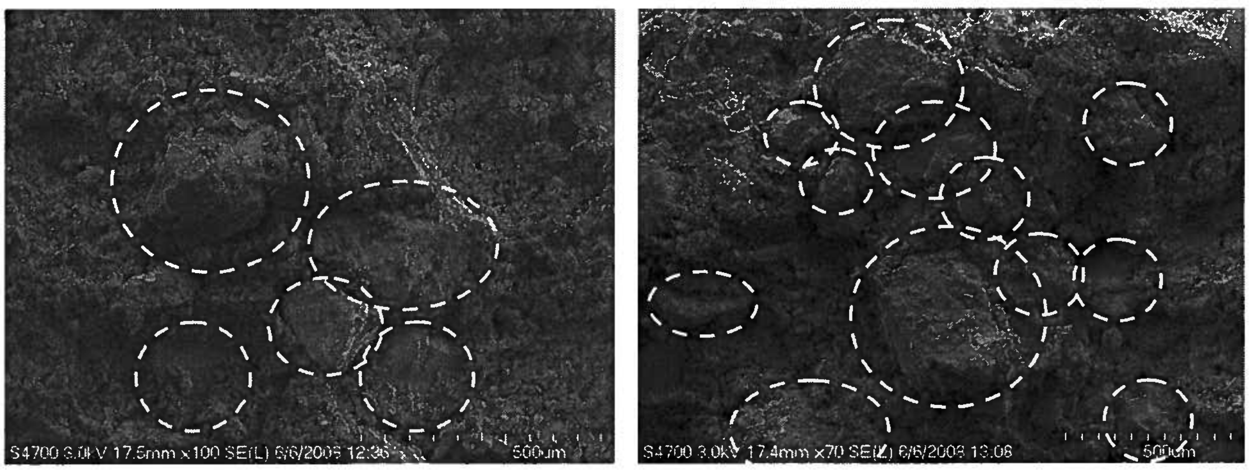
Scanning electron microscope (SEM) imaging was carried out in order to gain further understanding of the influence of soil type and cement content in cement-treated soil. These images were also captured (see Chapter 3) at the University of British Columbia BioImaging Facility.

5.2.1 Effect of sand-silt ratio

Four sand-silt mixes (25S:75M to 90S:10M) with equal cement contents (c_i) of 166 kg/m^3 (per unit volume of total initial mix) were analysed under the SEM, as shown in Figure 5.4. The ratio of sand to silt increases from 25 % to 90 % respectively. Recall the size of the silt grains is similar to that of the cement particles (see Figure 3.2). Inspection of the images suggests the sand grains in the 25S:75M, 50S:50M and 75S:25M specimens appear to be mostly separated by the finer silt/cement intra-assemblage. In contrast, the sand grains in the 90S:10M specimen appears to exhibit many more grain-to-grain contacts. Companion SEM images (Figure B.2), taken at a lower magnification, show a greater number of sand grains within the specimens. The broader image captures uniformly dispersed grains within the 25S:75M to 90S:10M sand-silt mixes, giving further assurance that the proposed new method of reconstitution yields specimens which are a homogeneous mix of blended material.



(a) 25S:75M (b) 50S:50M



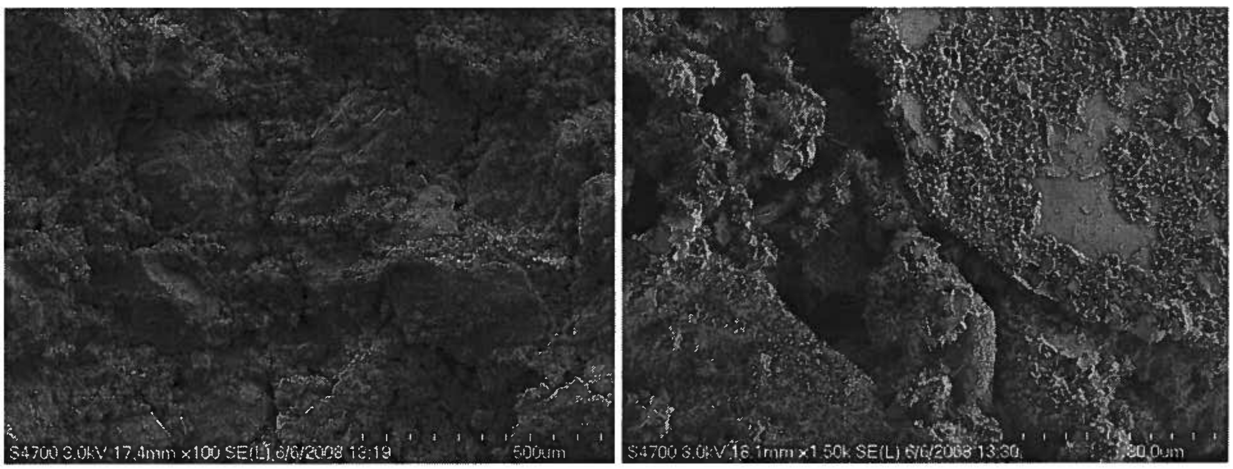
(c) 75S:25M (d) 90S:10M

Figure 5.4. SEM photographs of 25S:75M to 90S:10M specimens with cement content of (c_i) of 166 kg/m^3 (per unit volume of total initial mix): sand particles have been circled for ease of identification (see Figure B.1 for original images).

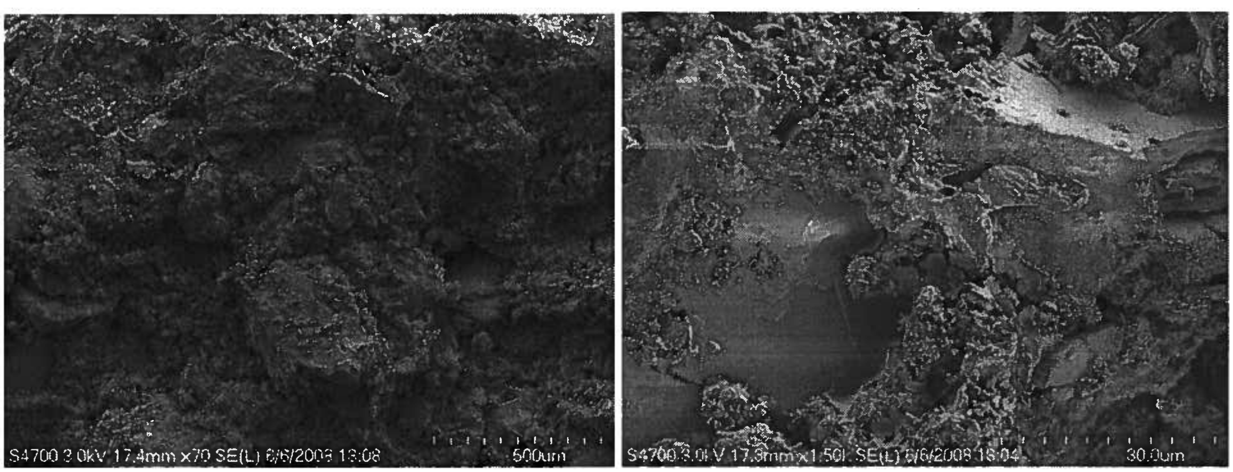
5.2.2 Effect of cement content

Specimens with a sand-silt ratio of 90S:10M and a low, moderate and high cement content (c_i) of 97 to 232 kg/m^3 (per unit volume of total initial mix) were analysed under the SEM, as shown in Figure 5.5. The images on the left hand side were taken at a lower magnification (x100), whilst the images on the right were taken at a much higher magnification (x800 to 1500).

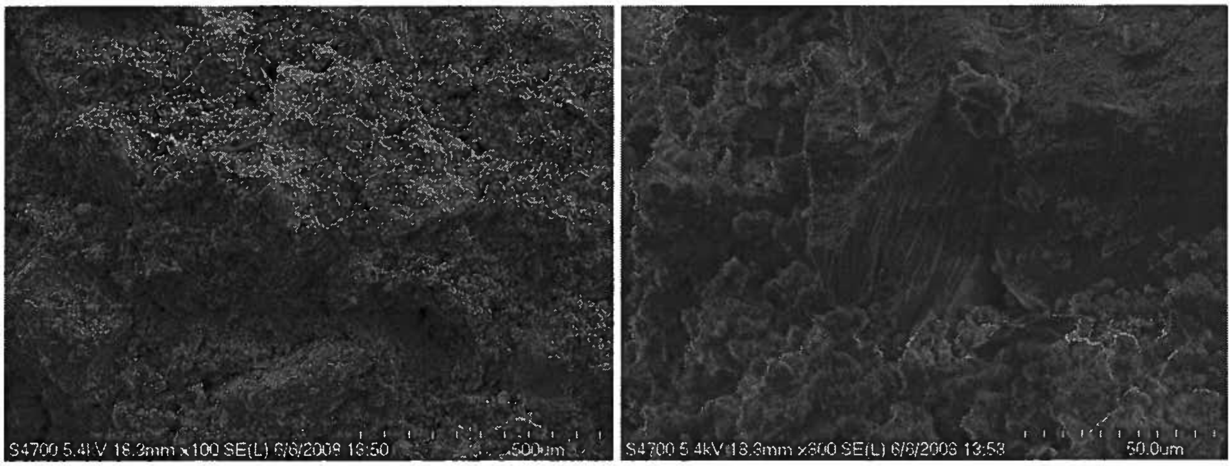
The differences between the low and moderate cement content appear to be subtle. However, the high cement content specimen seems to have more silt/cement intra-assemblage filling the voids between the sand grains, causing them to be less apparent through visual observation. The silt/cement filling is likely cement dominated at high cement content.



(a) Low cement content: $c_i = 97$ kg/m^3 (per unit volume of total initial mix)



(b) Moderate cement content: $c_i = 166$ kg/m^3 (per unit volume of total initial mix)



(c) High cement content: $c_i = 232$ kg/m^3 (per unit volume of total initial mix)

Figure 5.5. 90S:10M specimens with varying initial cement content (c_i)

High cement content of the filling between sand grains will produce an increased number of cemented bonds during the hydration reactions (Table A.1), thereby strengthening resistance to applied external force.

5.3 Unconfined compressive strength

Unconfined compressive strength (q_u) was calculated from the axial force acquired by the load cell divided by the cross-sectional area prior to testing (in accordance with ATSM D 1633-00). Axial strain (ε_a) was calculated as the displacement from the start of loading on the specimen divided by the original length (prior to testing). The unconfined compression test data for all 144 specimens of the main test program are presented as q_u against ε_a, in Figures 5.6 to 5.9. The four sand-silt ratios (25S:75M to 90S:10M), tested after 7 to 56 days of curing are presented (a total of 16 graphs). Each graph shows unconfined compressive stress plotted against axial strain for three values of water content (w_L, w_M, w_H) and three values of cement content (c_L, c_M, c_H), for a total of nine specimens. The subscripts represent low (L), moderate (M) and high

(H) values of water content and cement content, respectively. The specific values of water and cement content, along with further information on each test specimen, are provided in Tables B.1 to B.4. A summary of the peak unconfined compressive strength data for all specimens is given in Table 5.2.

Table 5.2. Peak unconfined compressive strength (q_u, MPa) of 144 cement-treated soil specimens

	Curing period	25S:75M			50S:50M			75S:25M			90S:10M		
	(days)	w_L	w_M	w_H	w_L	w_M	w_H	w_L	w_M	w_H	w_L	w_M	w_H
c_L	7	0.40	0.26	0.28	0.28	0.19	0.17	0.21	0.21	0.30	0.43	0.32	0.32
	14	0.43	0.52	0.49	0.17	0.25	0.25	0.32	0.30	0.30	0.47	0.41	0.29
	28	0.77	0.66	0.70	0.45	0.36	0.31	0.39	0.44	0.34	0.60	0.52	0.52
	56	0.70	0.64	0.72	0.39	0.35	0.40	0.42	0.38	0.38	0.73	0.70	0.42
c_M	7	0.98	0.93	0.69	0.68	0.71	0.72	0.94	0.92	0.84	1.52	1.72	1.40
	14	1.23	1.11	1.08	0.98	0.80	0.76	1.18	1.07	1.07	1.84	1.55	1.73
	28	1.75	1.67	1.20	1.08	1.05	0.97	1.27	1.31	1.27	2.38	1.96	2.22
	56	1.99	1.57	1.63	1.45	1.40	1.07	1.54	1.36	1.41	2.68	2.56	1.51
c_H	7	1.83	1.79	1.64	1.42	1.48	1.39	1.99	1.94	1.91	3.58	3.81	2.33
	14	2.24	2.02	1.89	1.96	2.03	1.76	2.56	2.51	2.45	3.87	3.83	3.64
	28	2.57	2.75	2.63	2.27	2.35	2.26	3.05	2.87	2.78	5.16	5.02	4.60
	56	3.34	3.12	2.85	3.06	2.37	2.57	3.44	2.11	2.89	5.28	5.48	4.99

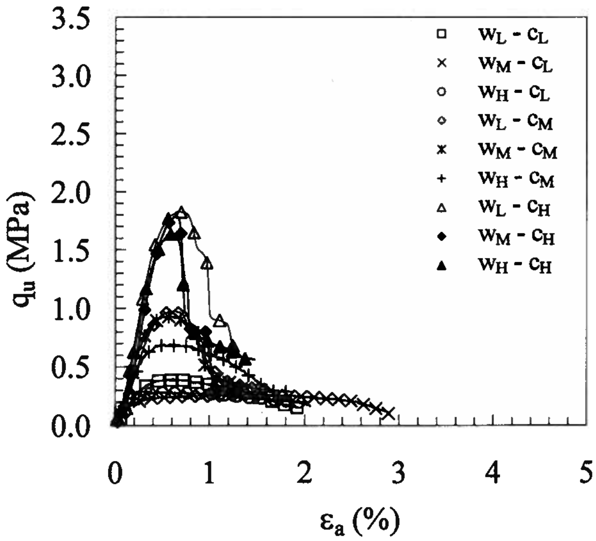
A dummy specimen, which was not part of the main test program, was prepared in order to characterize temperature change for curing and testing a typical specimen following wet-mixing in the laboratory. A sand-silt ratio of 25S:75M with relatively low water content (w_i: 46.8 %) and high cement content (c_i: 232 kg/m^3), was chosen in order to induce the largest increase in temperature that might be expected in the main test program. A maximum increase of +3.1°C above laboratory air temperature was observed a total of 8h. after mixing, which decreased slowly +1°C above air temperature after 24h., and returned to the laboratory air temperature within 48h. of mixing. Given the duration of the specimen curing time (7 to 56 days) and size of the cement-treated soil specimen, it is therefore believed that the minor temperature increase associated with specimen reconstitution had no impact on the temperature of specimens at the time of testing.

5.3.1 Influence of sand-silt ratio

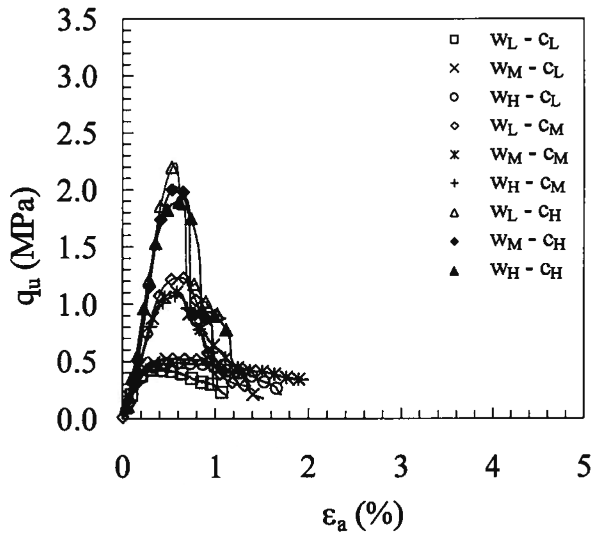
The influence of sand-silt ratio on strength is readily apparent from inspection of test data for the 56 day curing period (Figures 5.6d to 5.9d). More generally, these trends are also observed in data for curing periods of 7 to 28 days (discussed further in section 5.3.4), which suggests the findings are independent of elapsed time.

Comparison of the q_u vs. ε_a relations obtained after a 56 day curing period, in all sand-silt ratios (25S:75M to 90S:10M) reveal broad similarities in the shape and arrangement of the curves for different water-cement (w-c) contents. All curves exhibit a 'peak' strength that is mobilized at less than 1% axial strain. Generally, the three sand-silt ratios with lower sand content (25S:75M to 75S:25M) have peak strengths that lie with in a similar range, not exceeding 3.5 MPa. However, peak strengths observed for the most sandy specimens (90S:10M) are much higher, with values up to 5.5 MPa in some specimens (see Table 5.1).

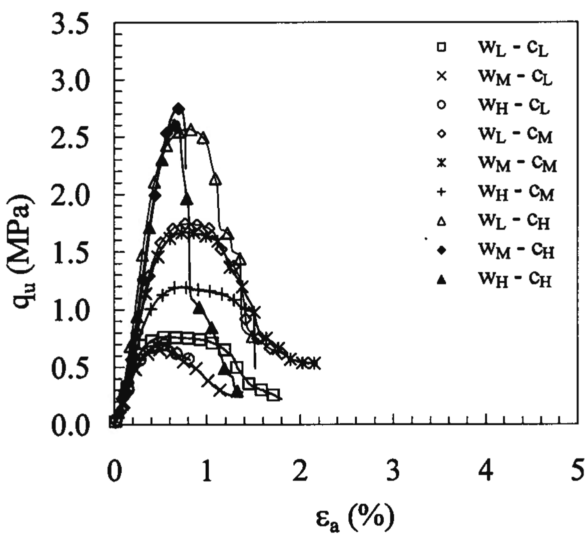
Notwithstanding this general trend in data for the 56 day curing period, there are some cases where specimens exhibit peak strengths at axial strains ranging well in excess of 1%. Consider, for example the curves of 7 to 28 day curing periods (Figures 5.7a to 5.7c) for the 50S:50M specimens with a relatively low cement content (denoted w_L-c_L, w_M-c_L and w_H-c_L): it is not easy to discern a 'peak' or maximum strength in the q_u vs. ε_a relations, in tests with axial strains of up to 2.25% (see Table B.2).



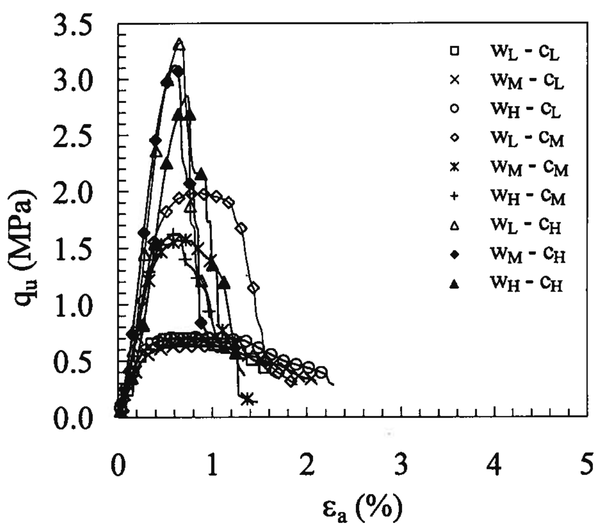
(a) Curing period of 7 days



(b) Curing period of 14 days

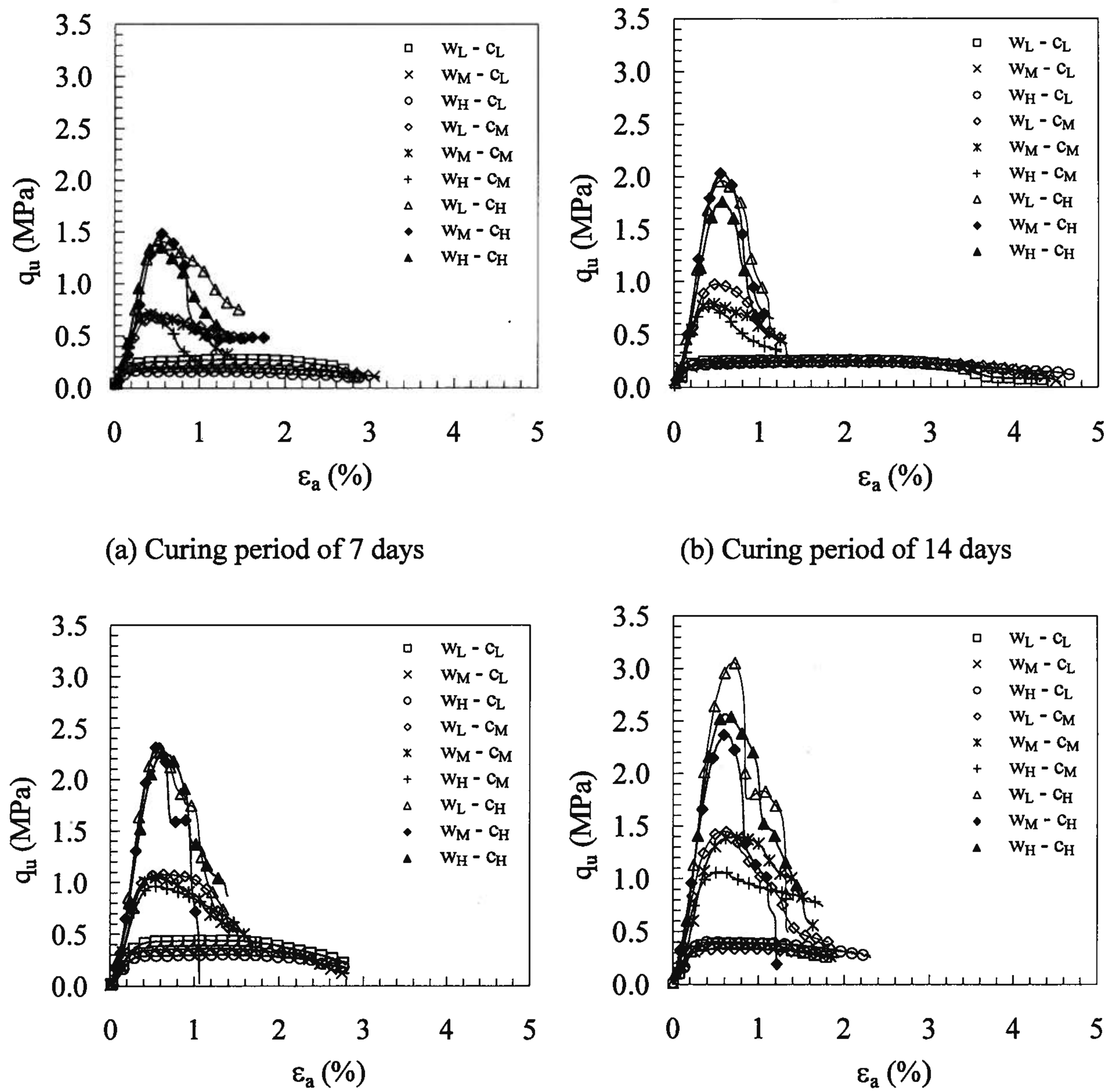


(c) Curing period of 28 days



(d) Curing period of 56 days

Figure 5.6. Unconfined compressive stress vs. axial strain for the 25S:75M specimens



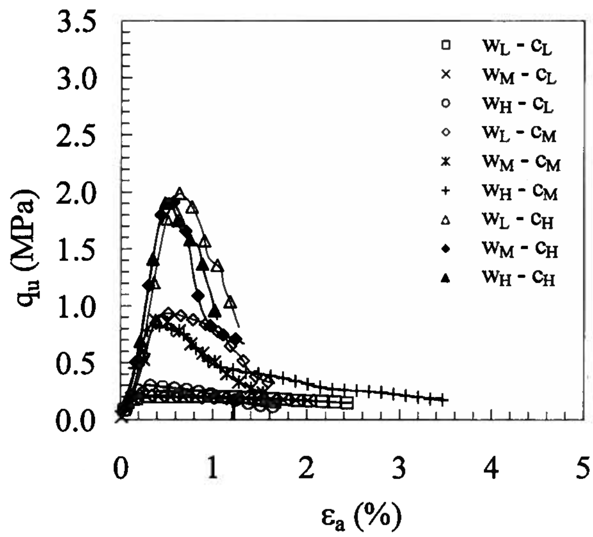
(a) Curing period of 7 days

(b) Curing period of 14 days

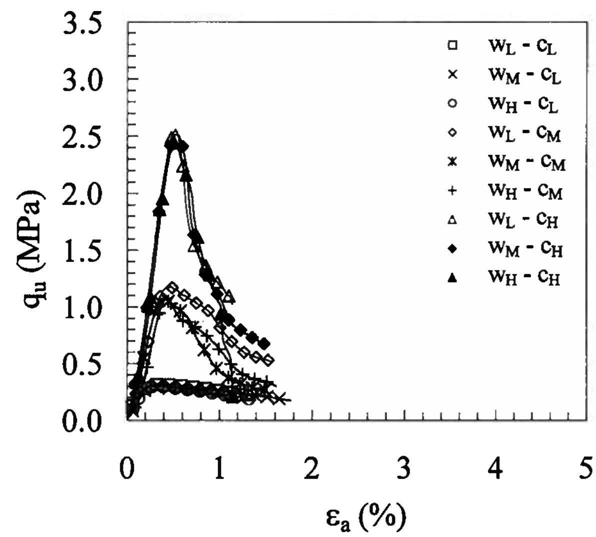
(c) Curing period of 28 days

(d) Curing period of 56 days

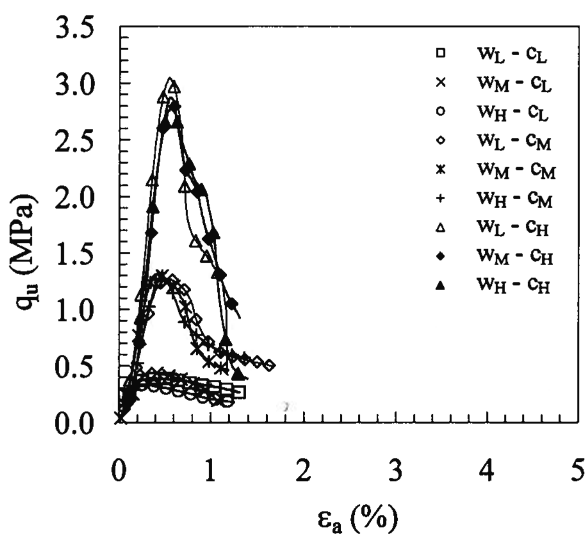
Figure 5.7. Unconfined compressive stress vs. axial strain for the 50S:50M specimens



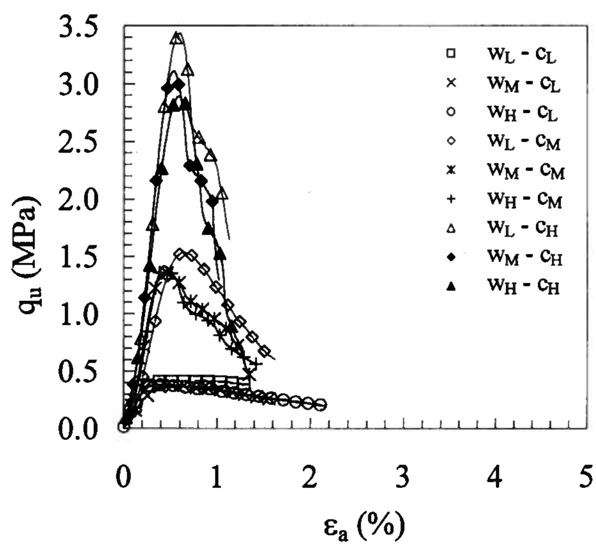
(a) Curing period of 7 days



(b) Curing period of 14 days

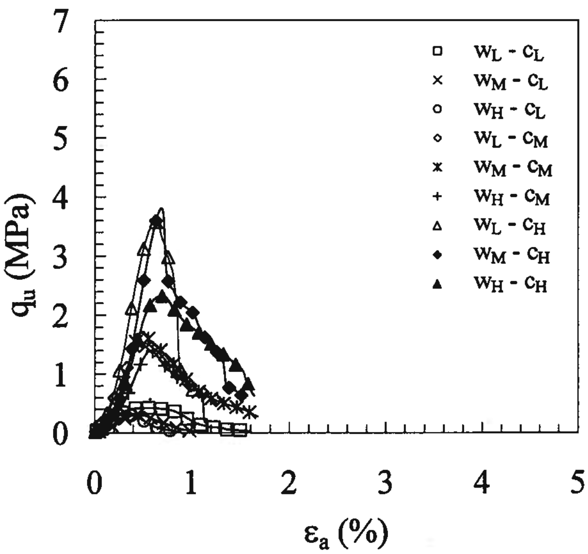


(c) Curing period of 28 days

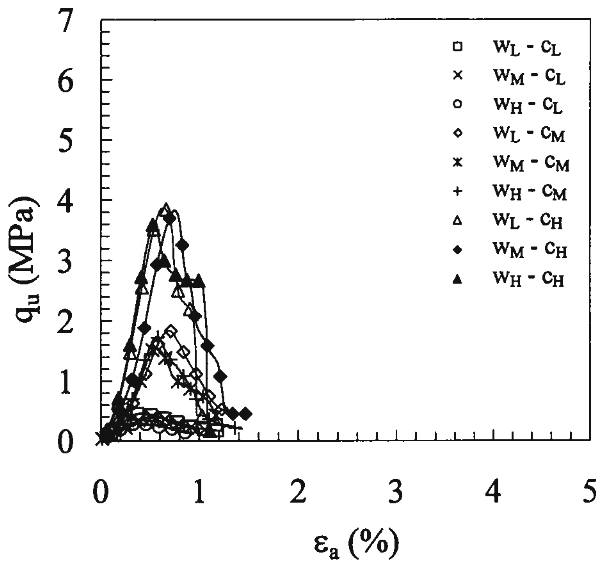


(d) Curing period of 56 days

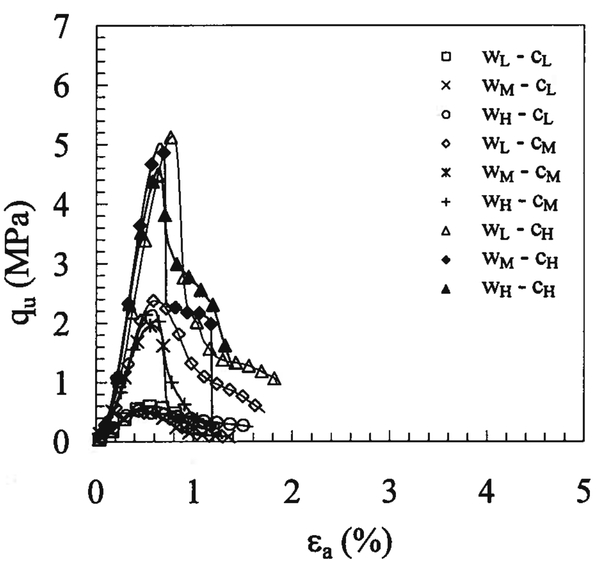
Figure 5.8. Unconfined compressive stress vs. axial strain for the 75S:25M specimens



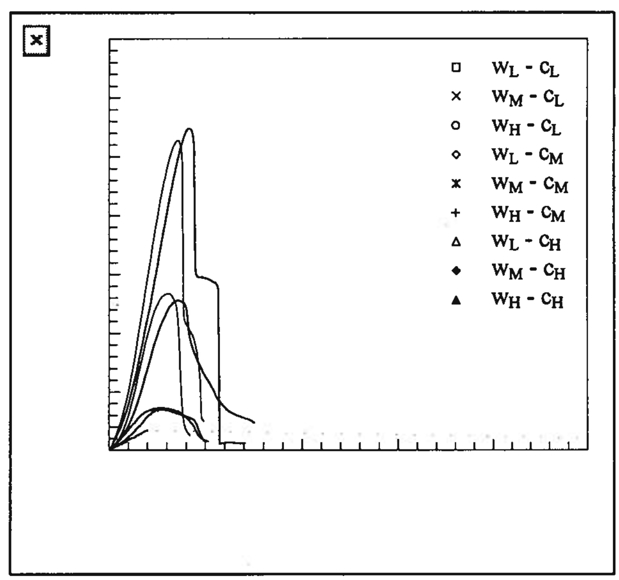
(a) Curing period of 7 days



(b) Curing period of 14 days



(c) Curing period of 28 days



(d) Curing period of 56 days

Figure 5.9. Unconfined compressive stress vs. axial strain for the 90S:10M specimens

Accordingly, the influence of sand content appears to be minimal in specimens with a sand-silt ratio of 25S:75M to 75S:25M , but of significance in 90S:10M specimens. The influence governs strength, but does not appear to govern axial strain to failure, although, a small number of 50S:50M specimens do show exceptions to this pattern.

5.3.2 Influence of cement content

The influence of cement content can be seen once again by first observing the data for a 56 day curing period (Figures 5.6d to 5.9d). These trends are also typically inherent in the curing periods of 7 to 28 days and are therefore considered independent of elapsed time.

The arrangement of the q_u vs. ε_a curves for the 56 day curing periods is such that all exhibit three distinct clusters of curves associated with the low (c_L), the moderate (c_M) and the high (c_H) cement contents, respectively. The low cement contents are generally associated with curves showing a 'ductile' response. Once loading is applied, a steep increase is observed in q_u followed by a relatively gradual decrease (once the peak has been surpassed) with increasing axial strain. In contrast, specimens with a high cement content yield a 'brittle' response whereby there is a very significant reduction in q_u with increasing axial strain after peak strength is exceeded.

Closer inspection of the 56 day curing period, shows that low cement content specimens exhibit peak strengths (see Table 5.2) in the range of $0.35 \leq q_u \leq 0.73$ MPa that are mobilised at $0.33 \leq \varepsilon_a \leq 0.74$ % (see Tables B.1 to B.4). Moderate and high cement content specimens exhibit peak strengths of $1.07 \leq q_u \leq 2.68$ MPa and $2.37 \leq q_u \leq 5.48$ MPa, that are mobilised at strains of $0.43 \leq \varepsilon_a \leq 0.84$ % and $0.53 \leq \varepsilon_a \leq 0.83$ % respectively.

As mentioned previously in section 5.3.1, the 7 to 28 day curing periods (Figures 5.7a to 5.7c) for 50S:50M sand-silt specimens (denoted w_L-c_L, w_M-c_L and w_H-c_L) provide an exception to the general trends. It is apparent that these anomalies are indeed associated with the 50S:50M specimens of low cement content.

To summarise the observations, a low cement content yields specimens with a ductile response to loading and is associated with specimens of low compressive strength. High cement content specimens have a clearly-defined peak stress, a relatively brittle response to loading and are associated with a relatively high compressive strength.

5.3.3 Influence of water content

Initial observations of the relations shown in Figures 5.6 to 5.9, do not yield a strong trend in the influence of low (w_L), moderate (w_M) and high (w_H) water contents. However, the peak strengths tabulated in Table 5.2 do show a decrease in strength from low to high water content in most cases, but not all. More specifically, 26 of the 48 w_L-w_M-w_H combinations yield a consistent reduction in strength with increasing water content. Further, it proves difficult to discern any trends between the 25S:75M to 90S:10M specimens because the absolute value of the initial water content is different for each sand-silt ratio (see Table 5.3). This is a consequence of the method of specimen reconstitution (see section 4.3) whereby minimum water contents could only be achieved through self-weight consolidation of the saturated sand-silt-water mixture.

For example, consider the set of twelve 25S:75M test specimens at moderate cement content (c_M) and a 7 to 56 day curing period. Inspection of Table 5.2 shows that increasing the water content (i.e. w_L to w_H) yields a decrease in peak strength (for example, the 28 day curing period shows a peak strength of 1.75 MPa decreasing to 1.20 MPa, with increasing water content). However, the 56 day curing period shows a peak strength of 1.99 MPa at w_L, 1.57 MPa at w_M and 1.63 MPa for w_H, which is not consistent with the general trend of the 7 to 28 day curing periods.

Table 5.3. Average initial water content (w_i) of 25S:75M to 90S:10M specimens

		Average w_i (%)	
(S:M)	w_L	w_M	w_H
25S:75M	45.8	47.6	49.1
50S:50M	43.6	45.5	47.0
75S:25M	36.3	38.4	40.4
90S:10M	35.0	36.9	39.0

In summary, the peak strength of specimens typically decreases with increasing water content. It appears that, for the range of parameters examined in testing, subtle differences in strength attributed to an increase in water content are much less significant than trends apparent in the influence of sand-silt ratio, cement content, or time (discussed in section 5.3.4).

5.3.4 Influence of time effects

The trends associated with time for the 25S:75M to 90S:10M specimens are apparent from inspection of the 7 to 56 day curing periods for the 25S:75M specimens (Figure 5.6), and are generally found independent of sand-silt ratio.

The peak strength of the 25S:75M curves typically exhibit a steady increase as the curing period increases from 7 to 56 days. It is evident that curves for the high cement content (c_H) show a greater increase in peak strength than those for the low cement content (c_L), over all the 7 to 56 day curing periods. The axial strain at which peak strength is mobilized, appears to increase slightly from the 7 day curing period to the 56 day period.

Upon inspection of the peak strengths (see Table 5.2) associated with high cement content the 25S:75M specimens yield a range of $1.64 \leq q_u \leq 1.83$ MPa at 7 days. A strength increment of approximately 0.2-0.8 MPa is evident each time the curing period is doubled (i.e. 14, 28 and 56 days of curing), with the final curing period of 56 days yielding a strength range of $2.85 \leq q_u \leq 3.34$ MPa. Similarly, the curves associated with low cement content and 7 day curing period have strengths in the range of $0.28 \leq q_u \leq 0.40$ MPa, and increase to $0.66 \leq q_u \leq 0.77$ MPa for 28 day curing periods. However, the 56 day curing period gives a range of $0.64 \leq q_u \leq 0.72$ MPa, and therefore provides some evidence that there does not always appear to be a significant strength increase (and in some cases a decrease is found) from the 28 to 56 day curing periods. The peak strength occurs at axial strains of $0.40 \leq \varepsilon_a \leq 0.67$ % and $0.61 \leq \varepsilon_a \leq 0.80$ % (see Tables B.1 to B.4) for specimens at tested at 7 and 56 day curing periods respectively.

In summary, at low cement content it appears the strength gain with time typically occurs during the first 28 days of curing, for the 25S:75M to 90S:10M specimens, with a modest strength gain taking place between 28 and 56 days. In contrast, the peak strength of the high cement content specimens shows a steady increase across all curing periods. In most specimens, there was a slight increase in axial strain with increasing curing period that is independent of sand-silt ratio.

6 ANALYSIS AND DISCUSSION

Consideration is first given to the influence of sand-silt ratio on the strength of cement-treated soil. Using the test data presented in Chapter 5, an analysis of the effect of cement content, time and water-cement ratio on peak unconfined compression strength is then made for the 25S:75M to 90S:10M specimens. From this an index of strength can be obtained, provided careful considerations are made as to the suitability of the data. Obvious caveats to the laboratory test data include the effect of confining pressure, ground temperature and quality control which are typically inherent in field wet-mixing. Factors influencing unconfined compressive strength are then described with respect to the Cutter Soil Mixing (CSM) technique; more generally, it is believed the trends observed can also be applied to other wet DMM applications that do not involve compaction and which produce saturated, homogeneous cement-treated sand-silt. Finally, a comparison is made with other work reported in the literature in order to provide further insight to the results obtained from this study.

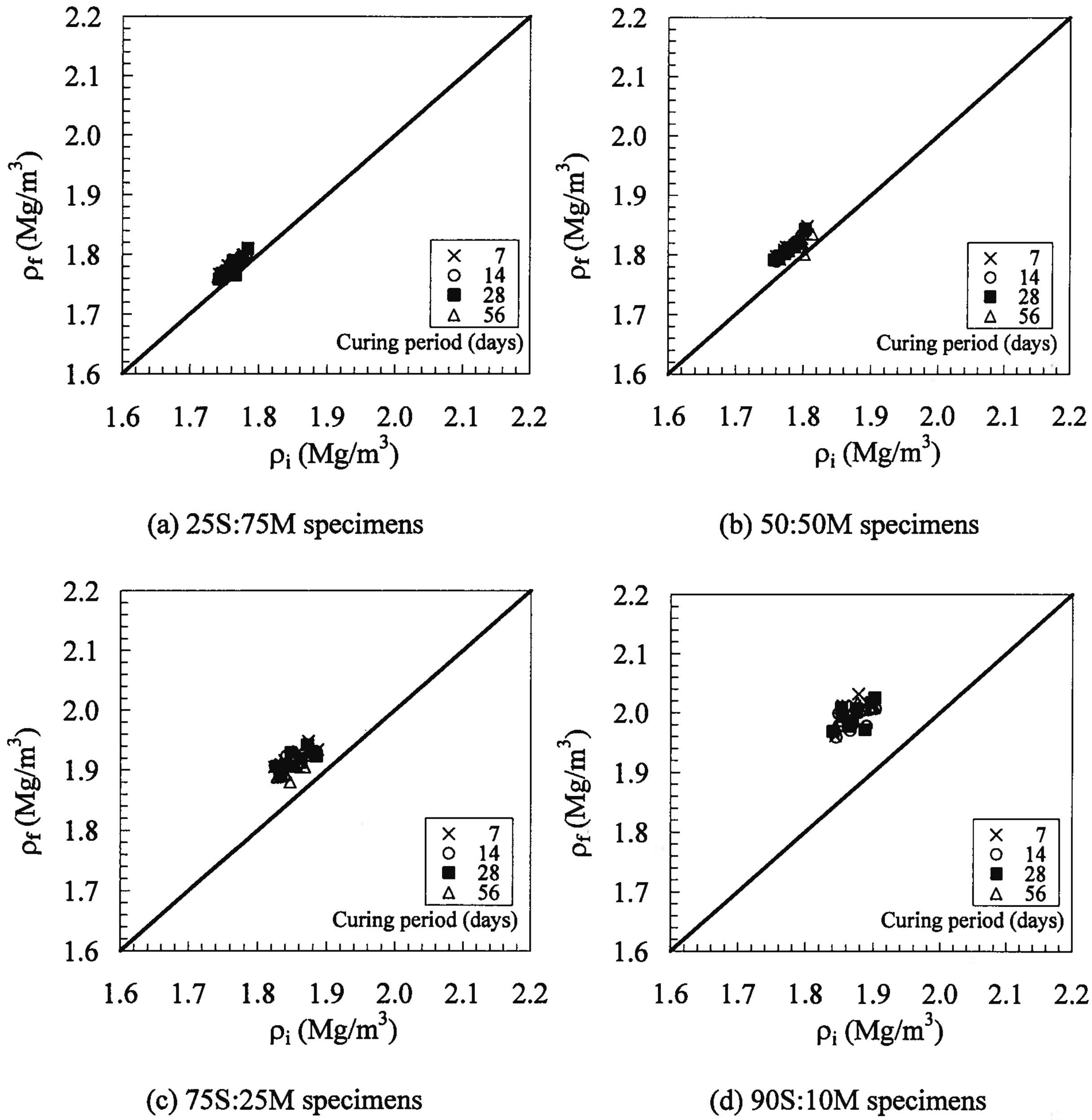
6.1 Sand-silt ratio

Results obtained in section 5.3.1 showed that sand-silt ratio influenced the peak unconfined compressive strength of test specimens. During the reconstitution method (outlined in section 4.3) some specimens were found to exude more water than others, yielding subtle variations in specimen density. Accordingly, density of the initial cement-treated mixture is first compared with density of the specimen prior to testing. An attempt is then made to characterise the change in water content as a consequence of this loss. The reduction in water content for each of the 144 laboratory specimens is reported in Tables B.1 to B.4.

6.1.1 Density

The value of specimen density prior to testing (ρ_f) is assumed equal to its density at the beginning of curing (ρ_p). This assumption was based on the evidence that change in mass and volume of the specimen from the beginning to end of curing was negligible. Density was related to strength in Figure 5.3, where an increase in the unconfined compressive strength of a specimen 'at failure' was attributed to an increase in density. Accordingly, Figure 6.1 shows the density at failure plotted against initial density (ρ_i) for the 25S:75M to 90S:10M specimens, for curing periods of 7 to 56 days. In the following analysis, the 25S:75M and 90S:10M sand-silt ratios are mainly described, because these specimens represent the lower and upper bound cases and capture sufficiently the trend of density at failure vs. initial density of the reconstituted mix.

The general trend suggests a linear relation between initial and final density for the ranges tested, with least data scatter in the 25S:75M specimens, and most scatter in the 90S:10M specimens of the tightly packed data. More specifically, the 25S:75M specimens show data on Figure 6.1a that produce a relatively precise relation. The 25S:75M specimens had an initial density in the range $1.74 \leq \rho_i \leq 1.78$ Mg/m^3 yielding a similar density at failure in the range $1.76 \leq \rho_f \leq 1.81$ Mg/m^3 (see Figure 6.1a). The increase is approximately 1 %. In contrast, the initial density of the 90S:10M specimens increased significantly, from values in the range $1.84 \leq \rho_i \leq 1.90$ Mg/m^3 to values in the range $1.96 \leq \rho_f \leq 2.03$ Mg/m^3 (see Figure 6.1d), a change of approximately 5%. From inspection of the ranges of density, it is apparent that specimens of higher sand content (S) experience a greater change in density of the final mix. From a review of the literature and due to the approach of reporting density during specimen reconstitution (outlined in section 4.3), no study currently provides details on the initial and final density for 'saturated' cement-treated soil specimens. Accordingly, no comparison with other laboratory data was possible. The following section provides further analysis in order understand the likely significance of these changes in density and water content, and more importantly, the ratio of water to cement in the treated ground.



(a) 25S:75M specimens

(b) 50:50M specimens

(c) 75S:25M specimens

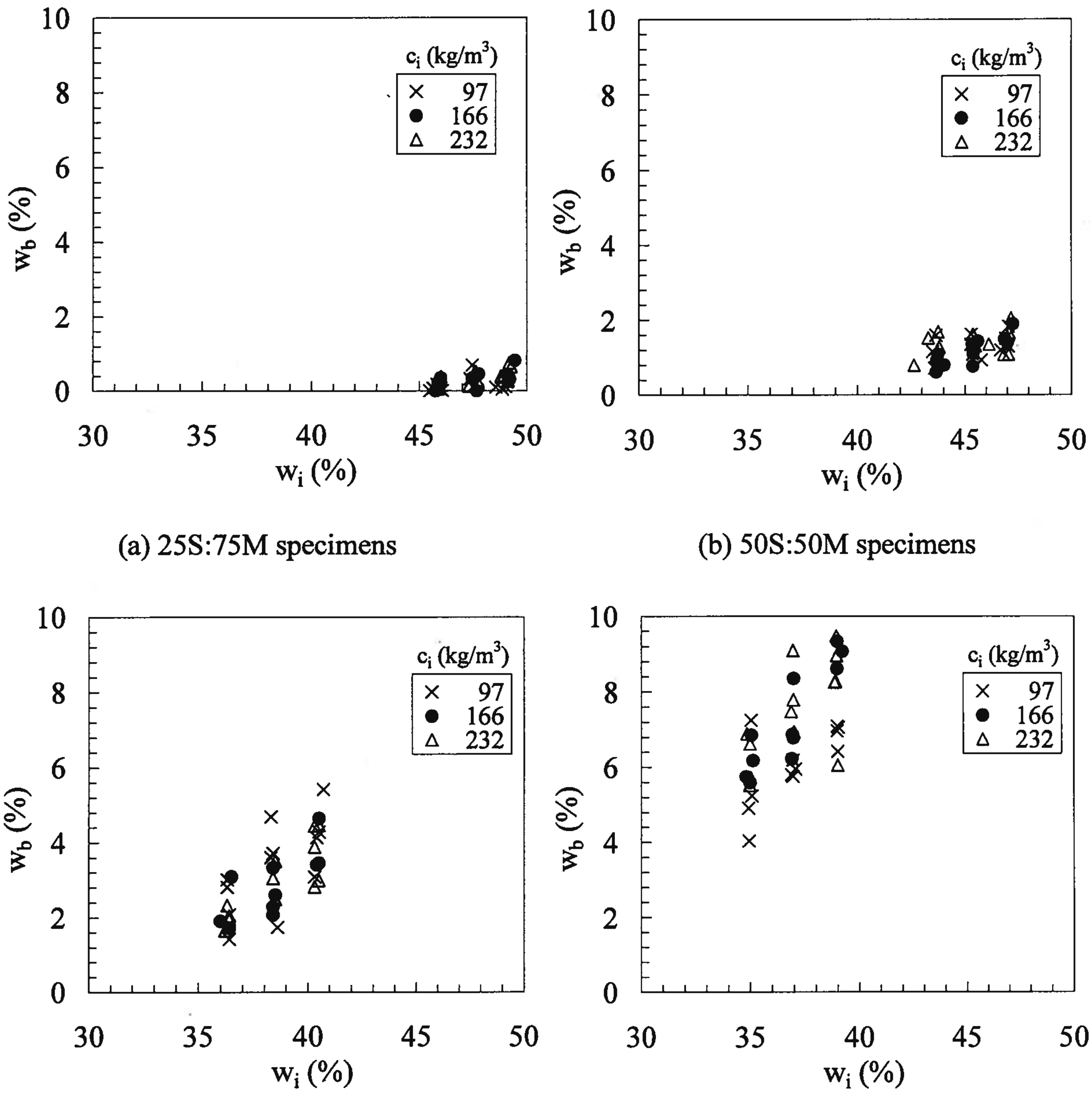
(d) 90S:10M specimens

Figure 6.1. Density of initial mix vs. density at failure

6.1.2 Bleed water

From observations during the wet-mixing process used in specimen reconstitution, it was apparent that many specimens exuded 'bleed' water (calculated by dividing the mass of bleed water by the mass of total solids) that percolated to the surface of the sand-silt-water-cement mixture, as a result of self-weight consolidation. In order to characterise this effect, the bleed water (w_b) is plotted against the initial water content (w_i) for the 25S:75M to 90S:10M sand-silt specimens in Figure 6.2. The data represent standing water that was removed from the top of specimens approximately one to five hours after mixing, as described in the specimen reconstitution method (section 4.3.5). As mentioned previously, the 25S:75M and 90S:10M sand-silt ratios are mainly described below, because these mixtures represent lower and upper bound characteristics of the phenomenon related to bleed water.

The percentage of bleed water arising in the 25S:75M sand-silt mixtures was relatively small: in all cases $w_b < 1.0$ % and in some cases it was zero (see Figure 6.2a and Table B.1). In contrast, the 90S:10M specimens yielded a much larger quantity of bleed water, typically in the range 5 to 10 %. The phenomenon appears to be independent of cement content for the range $97 \le c_i \le 232$ kg/m^3 examined in testing.



(a) 25S:75M specimens

(b) 50S:50M specimens

(c) 75S:25M specimens

(d) 90S:10M specimens

Figure 6.2. Initial water content vs. bleed water content

The general trend reveals that, with greater sand content (compare Figures 6.2a to 6.2d), there was an increased propensity for the soil-water-cement mixture to bleed water (w_b) under self-weight consolidation. Further, a steeper upward trend of bleed water was observed in mixtures with greater sand content, even though they were reconstituted at relatively lower water contents (w_i).

'Bleeding' (w_b) causes a reduction in the water content (from w_i to w_p, see Figure 4.5) of the saturated sand-silt-cement mix. The reduction in void space yields a closer packing of solid particles, and therefore a higher density (as shown in Figure 6.1). Additionally, the percent silt/cement in the sand grains is observed in the SEM images of section 5.2.1 for the 25S:75M to 90S:10M specimens. Whilst it is recognised that bleed water influences density, the consequence of a reduction in water content has much greater repercussions. Accordingly, upon first addressing an analysis on cement content (section 6.2) and time (section 6.3), the implications of this change in water content can be drawn from an analysis and a discussion of the water to cement ratio (section 6.4).

6.2 Cement content

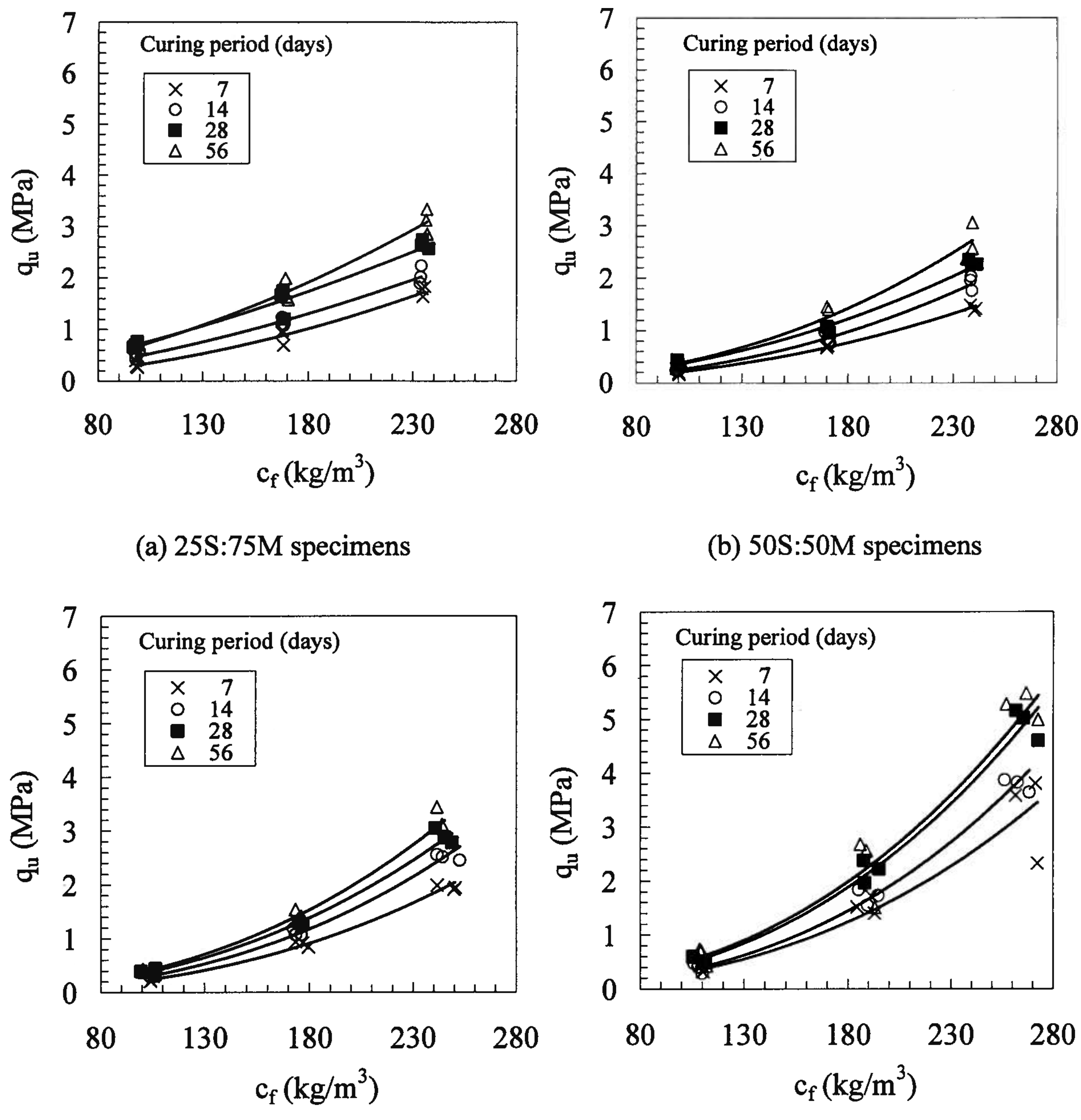
The following sections present an analysis of the cement content of test specimens, in order to establish trends between the low (c_L), moderate (c_M) and high (c_H) cement contents of each sand-silt ratio. Test results for the 25S:75M and the 90S:10M specimens are then compared to other laboratory data reported in the literature.

6.2.1 Test data of the current study

The change in mass and volume of the specimen from the beginning of the curing period to the time of testing was found negligible. Hence, the cement content (which is a mass per unit volume) at the time of testing (c_f) was assumed to equal the cement content at the beginning of curing (c_p). As noted previously, bleed water causes a volume reduction in the reconstituted mix, and a commensurate increase in the mass of cement per unit volume. This effect is apparent in Figure 6.3 where the range in c_f values is always greater than the range in three values of c_i used in reconstitution (namely 97, 166 and 232 kg/m^3). This phenomenon was seen most clearly by inspection of c_f in the 90S:10M specimens (see Figure 6.3d) when the value of c_i = 232 kg/m^3 increased to $255 \leq c_f \leq 272$ kg/m^3.

Figure 6.3 shows the peak unconfined compressive strengths of the 25S:75M to 90S:10M specimens plotted against cement content at failure, for 7 to 56 day curing periods. Independent of sand-silt ratio and curing time, there appears to be a non-linear increase in peak strength (q_u) with an increase in cement content (c_f). The 90S:10M specimens appear to have strengths that are much higher, but also more scattered, in comparison to the 25S:75M to 75S:25M specimens, at high cement content. These specimens also exhibit a slightly greater increase in strength with increment of cement content.

Inspection of the curves suggests the increment in strength over the 7 to 56 day curing period is less than 0.5 MPa, at $c_f \approx 100$ kg/m^3, and independent of sand-silt ratio. Similarly, the strength increment at $c_f \approx 235$ kg/m^3 is approximately 1.5 MPa, and also appears independent of sand-silt ratio. Accordingly, the range in strength over the duration of curing period appears to be much smaller at the relatively low cement content of $c_f \approx 100$ kg/m^3, in comparison to the relatively high cement content of $c_f \approx 235$ kg/m^3.



(a) 25S:75M specimens

(b) 50S:50M specimens

(c) 75S:25M specimens

(d) 90S:10M specimens

Figure 6.3. Peak unconfined compressive strength vs. cement content at failure

6.2.2 Comparison to other work

Filz et al. (2005) presented unconfined compressive strength data for specimens prepared using a wet-mixing technique described by Hodges et al. (2004). Two soils, 'Vicksburg silt' and

'Light Castle sand', were included in their program of testing. The results obtained are compared to data from this study.

Vicksburg silt is a soil of low plasticity (D_{50} = 20 μm, C_u = 5.6, I_P = 5.0 %, 92 % silt sized grains) that is similar to the Kamloops silt (described in section 3.2) used for specimen reconstitution in this study. Accordingly, it is expected to provide a good benchmark for comparison with the soil mix of the current study that contains the most silt (25S:75M specimens). 'Light Castle' sand (D_{50} = 0.32 mm, C_u = 2.6, 100 % in the 75 μm to 2.0 mm range) is very similar to the Fraser River sand (described in section 3.1) used for specimen reconstitution in this study, and is therefore expected to provide a suitable comparison with the most sandy test data (90S:10M specimens).

As always when making comparisons between laboratory studies, it is important to identify any significant differences in the respective test methods. In comparing data of the current study to that of Filz et al. (2005), the differences relate to their compacting of specimens during reconstitution, assuming 100% saturation of test soils, trimming specimens using a rock saw, testing with reference to a different test standard for unconfined compression testing (ASTM D 2166-91 - for cohesive undisturbed, remoulded or compacted soil cylinders), using a rate of strain approximately 1 mm per minute, and reconstituting specimens of approximately 50 mm (2") diameter and 90 mm (3.5") in length.

The peak unconfined compressive strengths of the 25S:75M test specimens are compared with the Vicksburg silt data, for a curing period of 28 days, in Figure 6.4. Inspection shows the

25S:75M specimens exhibit a greater strength than the Vicksburg silt, at the same cement content (definition of c_f is identical for both the current study and Filz et al. (2005)). However, a similar non-linear upward trend is apparent from comparison of the two data sets, providing further confidence in the results of the current study. Recognizing that plots of peak strength against cement content do not address the influence of water, it must be noted that further analysis (discussed in section 6.4) is needed in order to properly account for the effect of total water content of the specimen.

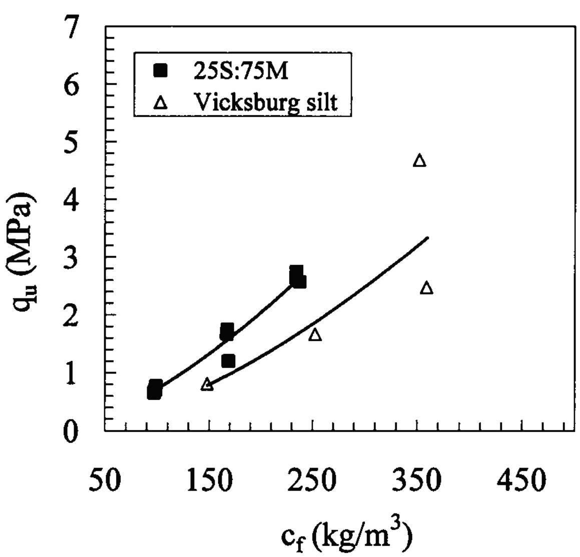


Figure 6.4. Unconfined compressive strength at 28 days vs. cement content data

Figure 6.5 shows the peak unconfined compressive strengths of the 90S:10M test specimens compared with the Light Castle sand data, again for a curing period of 28 days. Inspection shows the trend line of the 90S:10M specimens crosses that of the Light Castle sand. However, most of the Filz et al. (2005) data plot close to the 90S:10M trend line. Again, as plots of peak strength against cement content do not take into consideration of the influence of water, further analysis is appropriate (see section 6.4).

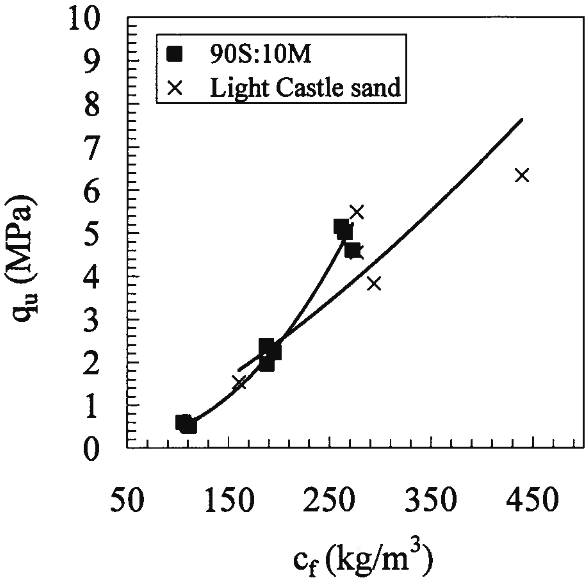


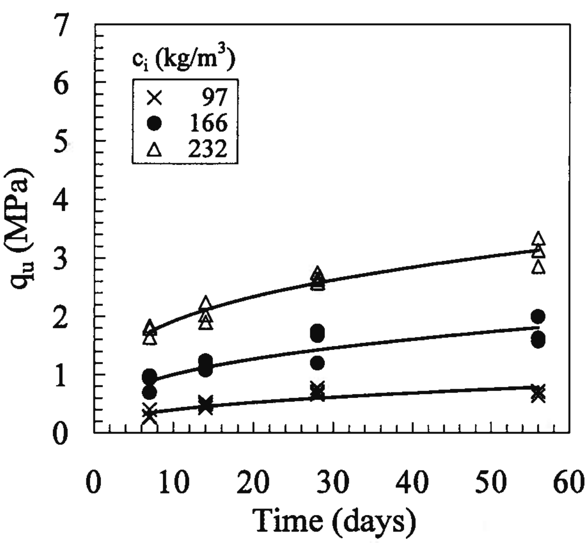
Figure 6.5. Unconfined compressive strength of 28 days vs. cement content data

6.3 Time effects

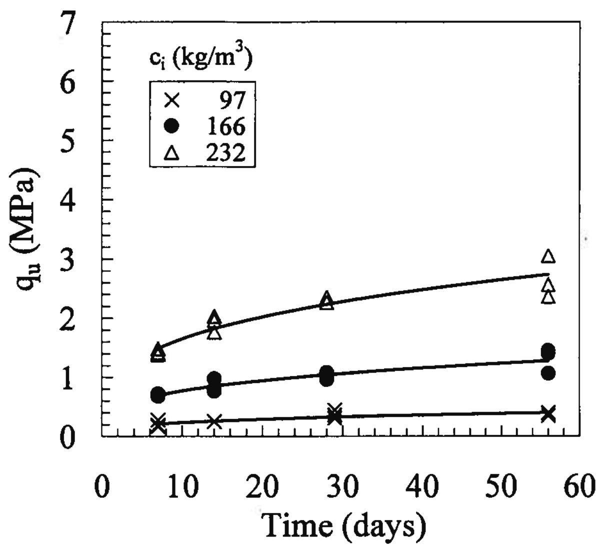
The following sections present an analysis of the curing period, in order to understand the influence of elapsed time on the peak unconfined compressive strength of different sand-silt ratios, and to account for varying cement content. Test results for the 25S:75M to 90S:10M specimens are then compared to other laboratory data reported in the literature.

6.3.1 Test data of the current study

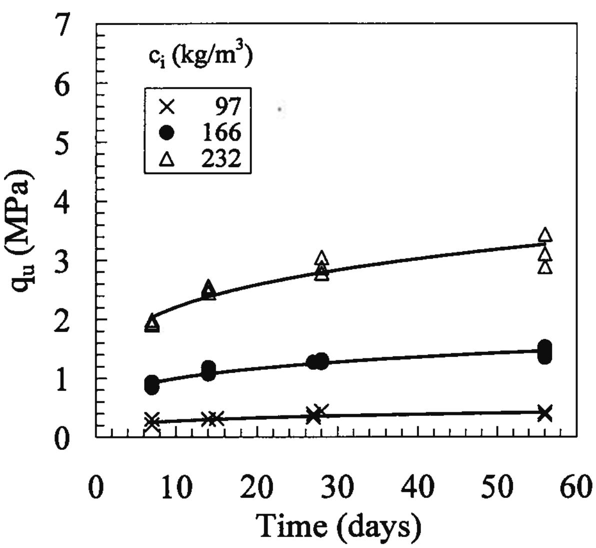
Peak unconfined compressive strengths of the 25S:75M to 90S:10M specimens are plotted against elapsed time in Figure 6.6. There appears to be a non-linear increase in peak strength (q_u) with greater duration of curing time (t). Typically, the strength increase observed in specimens of lower cement content occurs in the first 28 days, whereas the moderate to high



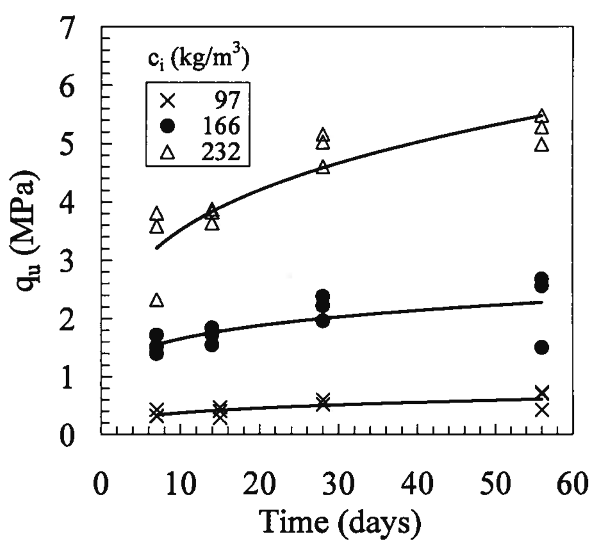
(a) 25S:75M specimens



(b) 50S:50M specimens



(c) 75S:25M specimens



(d) 90S:10M specimens

Figure 6.6. Peak unconfined compressive strength vs. curing time

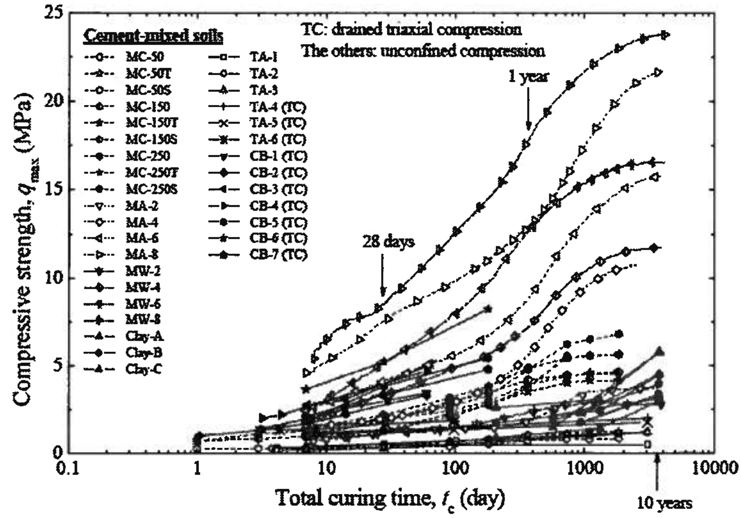
cement contents yield specimens that to continue to exhibit a gain in strength up to elapsed time of 56 days. Inspection of these latter curves for c_i = 232 kg/m^3 suggests there is potential for additional significant strength gain beyond 56 days.

Strength-time curves for the 25S:75M to 90S:10M specimens show a clear distinction between the relatively low (97 kg/m^3), moderate (166 kg/m^3) and high (232 kg/m^3) initial cement content. Further, although the 90S:10M specimens exhibit some scatter at c_i = 232 kg/m^3, the trend depicts a strength-time curve that is much stronger than the corresponding c_i contents of the three 25S:75M to 75S:25M specimens. The observed scatter in peak strength (q_u) values for specimens with identical initial cement content, curing period and sand-silt ratio (for example, the 90S:10M specimen, at c_i = 232 kg/m^3, after 7 days) are attributed to small differences in water content (w_L to w_H), which are found in Tables B.1 to B.4.

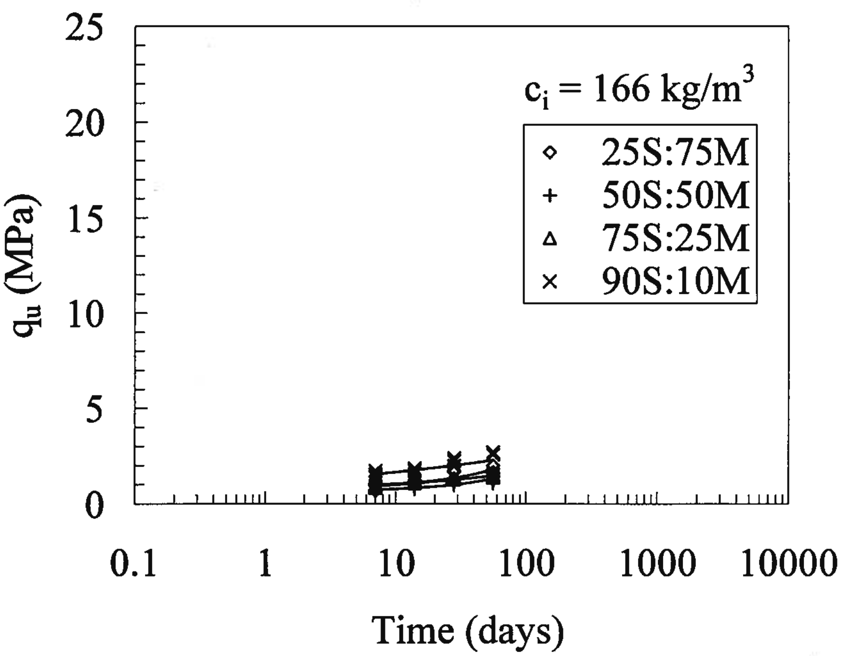
6.3.2 Comparison to other work

Kongsukprasert et al. (2007) compiled unconfined compressive strength data for specimens over a long-term curing period of up to 10 years. Evidence on the strength gain of cement-treated soil is shown in Figure 6.7a. The strength-time data, are for various methods of reconstitution, soil type, cement content and water content.

Figure 6.7b shows four sand-silt ratios (25S:75M to 90S:10M) and initial cement content c_i = 166 kg/m^3 from the current study. Inspection of the trends indicate a strength gain rate of approximately 0.2 MPa per week during the 7 to 14 day curing periods, in contrast to the strength gain rate of approximately 0.02 MPa per week during the 28 to 56 day curing periods. However, in general the average strength gain rate from 7 to the last curing period of 56 days, was approximately 0.1 MPa. The strength gain rate was largely independent of sand-silt ratio. Comparison of these trends (Figure 6.7b), indicate a general strength increase with time that is consistent with much of data presented in Figure 6.7a. Evidence is provided herein that strength increase over a very long time is likely; to a period of at least 10 years. This information provides guidance to engineers to enable prediction of the expected long-term strength of cement-treated soil, during the decision making process of wet-mixing design.



(a) Kongsukprasert et al. (2007), with permission of F. Tatsuoka



(b) 25S:75M to 90S:10M specimens

Figure 6.7. Peak unconfined compressive strength vs. time of cement-treated soils

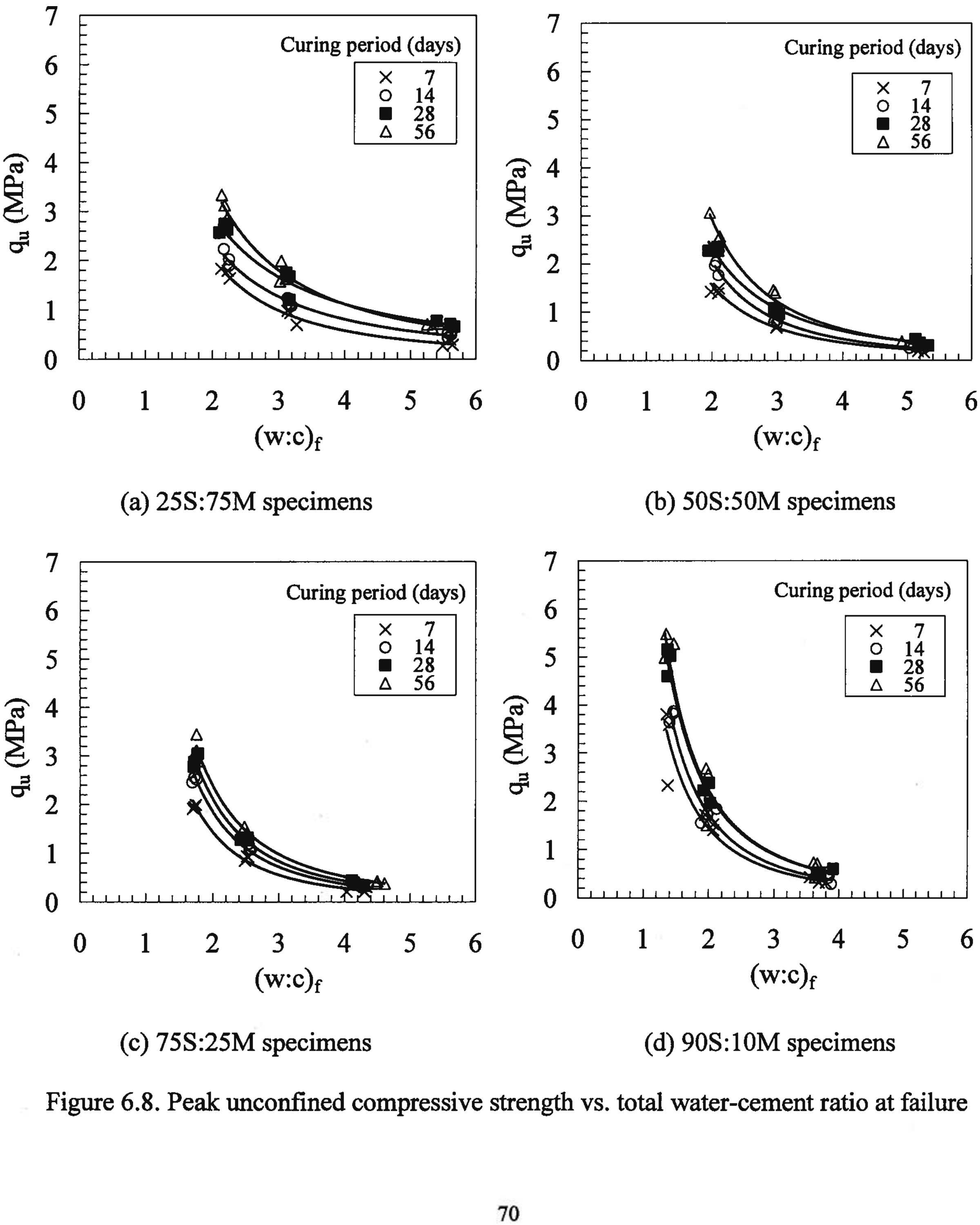
6.4 Total water-cement ratio

Natural ground conditions vary considerably from site to site, most notably, the water content of the soil. Therefore, in seeking to characterize factors influencing the strength of cement-treated soil, a method of quantifying the effect of both water and cement content is required. Accordingly, an analysis, is now made of peak strength against the total water-cement ratio. The findings of the current study are then compared to the data of another similar study.

6.4.1 Test data of the current study

The total water-cement ratio $(w{:}c)_f$, presented earlier in Figure 4.5, describes mass of water measured at the time of testing divided by the mass of cement in prior to testing. Values of $(w{:}c)_f$ for all specimens can be found in Tables B.1 to B.4. Plots of peak unconfined compressive strength against total water-cement ratio are given in Figure 6.8. The relations depict a non-linear variation of decreasing peak strength with increasing total water-cement ratio, for all four sand-silt ratios (25S:75M to 90S:10M). As noted previously, the trend lines also reveal an increase in strength with longer curing period. The increase is greatest at lower total water-cement ratios.

The 25S:75M specimens, with greatest silt content, exhibit a range of $2.10 \leq (w{:}c)_f \leq 5.66$, whereas the 90S:10M specimens, with greatest sand content, were in the range $1.33 \leq (w{:}c)_f \leq 3.91$. The lower water-cement ratio in specimens with higher sand content is attributed to two factors: (i) the process of specimen reconstitution (described in section 4.3) whereby the initial water content was assigned different values depending on the sand-silt ratio (see Tables B.1 to B.4), and (ii) loss of water due to bleeding (discussed in section 6.1). For wet-mixing applications for treatment of saturated silty to sandy soils, the range of water-cement ratio appears to be approximately $1.0 \leq (w{:}c)_f \leq 6.0$.



(a) 25S:75M specimens

(b) 50S:50M specimens

(c) 75S:25M specimens

(d) 90S:10M specimens

Figure 6.8. Peak unconfined compressive strength vs. total water-cement ratio at failure

Figure 6.9 illustrates the variation unconfined compressive strength with percentage of silt (to sand) in the 25S:75M to 90S:10M specimens, for a constant value of total water-cement ratio ($(w{:}c)_f$ = 3). It appears that there is no significant change in strength for a silt content less than 30 to 35 % silt. Interpretation of the trend lines suggests that a silt content in excess of approximately 35% yields an increase in strength.

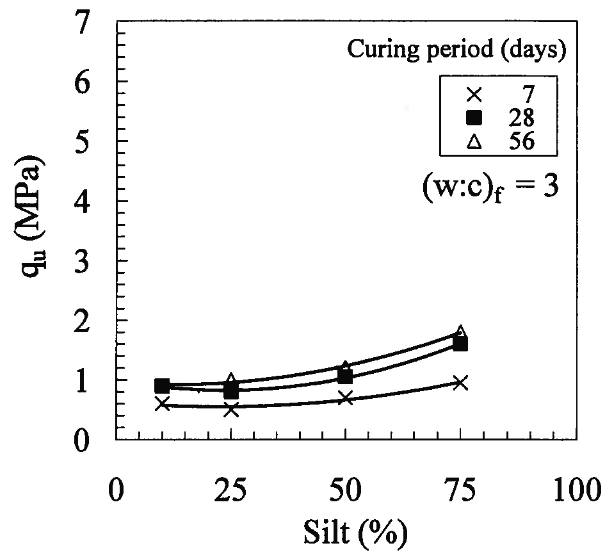


Figure 6.9. Influence of silt content on peak unconfined compressive strength at constant total water-cement ratio

6.4.2 Comparison to other work

Figure 6.10 shows a comparison of peak unconfined compressive strengths obtained in the current study with data reported by Filz et al. (2005) from their laboratory tests. More specifically, results for the 25S:75M to 90S:10M specimens are plotted with data for Vicksburg silt and Light Castle sand (first described in section 6.2.2), at a curing period of 28 days. Both sets of data show good general agreement.

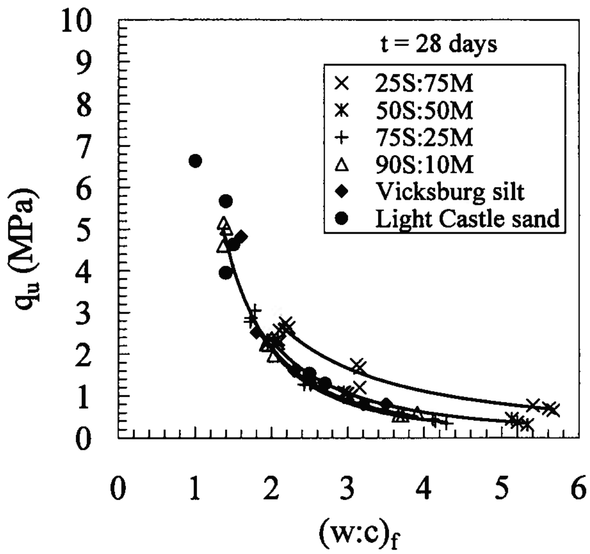


Figure 6.10. Peak unconfined compressive strength vs. total water-cement ratio

The good agreement has several implications. Firstly, it provides strong evidence that strength is governed by water-cement ratio rather than water and cement alone. The plotting method provides a useful tool to enable engineers to optimize preliminary mixture design on wet-mixing projects such as CSM, where there are requirements to obtain a minimum compressive strength. Ideally, in order to obtain this strength, a minimum cement content is preferred so that operating costs are reduced. Knowing the water content of treated soil, total water-cement ratio can give this guidance on the minimum cement content needed to fulfill the project-specified strength requirements. Secondly, the relations in Figure 6.10 suggest that for six different sand-silt mixtures, a very similar non-linear trend exists relating total water-cement ratio to strength. The trend does not appear to be greatly influenced by the exact ratio of the sand-silt mixtures. This implies the natural water content of the soil imparts much more influence on strength, compared to the sand-silt ratio. Lastly, the strength vs. total water-cement ratio curves of Figure 6.10 provide a very significant improvement in plotting method compared to the strength vs. cement content curves of Figures 6.4 and 6.5, as the data points follow a much clearer non-linear trend, due to the combination of water and cement as a single variable.

7 CONCLUSIONS AND RECOMMENDATIONS

This study investigated the factors that influence the strength of a range of cement-treated sand-silt specimens, reconstituted by wet-mixing with cement in the laboratory. A new method of specimen reconstitution was developed to yield reproducible laboratory specimens that were believed to replicate the fabric and condition of the CSM cement-soil, at sites where the ground is saturated. The new method of specimen reconstitution was believed to produce a set of high quality data that studied the influence of sand-silt ratio, cement content, water content and time. Further, a quality set of data enabled trends to be confidently established, in order to guide decisions on preliminary mixing design of CSM treated soil. The work of the current study was then contrasted to published data from other laboratory studies.

7.1 Conclusions

Considerable effort was applied in the method of specimen reconstitution with particular emphasis to ensure saturation, comprehensive mixing of cement-treated sand-silt mixes, precise accounting and correcting for mass quantity. Accordingly, the following conclusion is made:

1. Reproducibility of the method is demonstrated using four nominally identical specimens and homogeneity of the mix is evident from scanning electron microscope images of the specimen fabric, which together yield confidence in the trends obtained from the data.

The shape of the stress-strain curves were dominated by the initial cement contents (c_i): $c_L = 97$ kg/m^3, $c_M = 166$ kg/m^3 and $c_H = 232$ kg/m^3 and were independent of sand-silt ratio, leading to the following conclusion:

2. Low cement content (c_L) yielded specimens that were associated with a ductile loading response. The sand-rich (90S:10M) specimens exhibited a low cement content range at failure of $105 \leq c_f \leq 111$ kg/m^3, whereas the silt-rich (25S:75M) specimens exhibited a range of $97 \leq c_f \leq 99$ kg/m^3. High cement content (c_H) yielded specimens with a clearly-defined peak stress and a relatively brittle response to loading. The sand-rich (90S:10M) specimens exhibited a high cement content range at failure of $255 \leq c_f \leq 272$ kg/m^3, whereas the silt-rich (25S:75M) specimens exhibited a range of $233 \leq c_f \leq 235$ kg/m^3.

Conclusions are drawn for the relative influences of cement content, water content, water-cement ratio sand-silt ratio and time on strain controlled (displacement rate of 1 mm/min) unconfined compression tests of cement-treated soil specimens, as follows:

3. An increase in cement content lead to an increase in the unconfined compressive strength;
4. An increase in water content produced a general decrease in unconfined compressive strength;
5. However, the total water-cement ratio proved to be the key variable, relating strength to both water and cement. Peak unconfined compressive strength (q_u) increased with a non-linear decrease in water-cement ratio;
6. At a given total water-cement ratio, the unconfined compressive strength was largely independent of sand-silt ratio; and,
7. The trend in unconfined compressive strength gain was, on average, approximately 0.1 MPa per week, between the 7 to 56 day curing periods, and was largely independent of sand-silt ratio. In comparison to other laboratory studies, this rate appears reasonable.

7.2 Recommendations

Recommendations are given below with respect to the laboratory testing, stress-strain performance, data reporting and further study of CSM-treated soils:

1. In a fundamental study of mix proportions, the relative simplicity of the newly developed method, and the importance of ensuring total water-cement ratio in a saturated soil; it is recommended to eliminate degree of saturation as a variable. However, it is acknowledged that ensuring the full saturation of soil can prove a time consuming process and in engineering practice this may not be feasible. Accordingly, if the degree of saturation is proven to be of little significance to unconfined compressive strength and rate of strength gain with time, then there is opportunity for further adjustment of the test procedure;
2. If ductile soil response behaviour is important to the CSM application, then relatively low cement of less than 100 kg/m^3 may be used;
3. Recognising that cement and water both exert an influence on the unconfined compressive strength, it is recommended to plot strength against total water-cement for purposes of data reporting and as design guidance for ground improvement applications and,
4. Implementation of a study in order to verify laboratory findings in field practice.
5. It may be of interest to the reader to consider the effect of strain-rate on the relation of strength and stiffness, given the rate of displacement selected for this study (1 mm/min).

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APPENDIX A

Table A.1 Chemical reactions of Portland cement during cement-treatment of soils

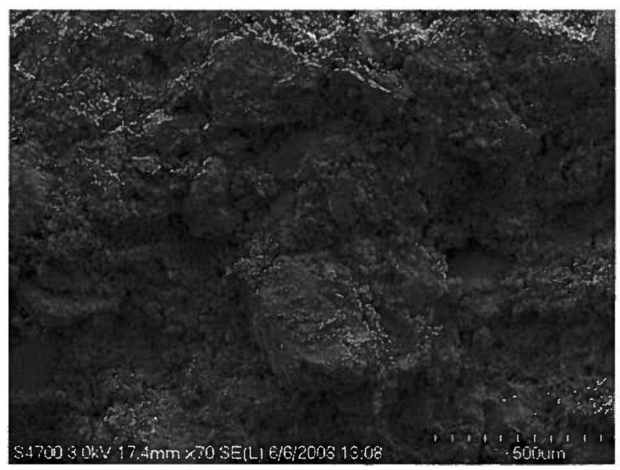
Hydration of Portland cement (Lea, 1998):		
$2(3CaO \cdot SiO_2) + 5.5H_2O$	→	$3CaO \cdot 2SiO_2 \cdot 2.5H_2O + 3Ca(OH)_2$
Tricalcium silicate + water		Calcium silicate hydrate (CSH) + calcium hydroxide
$2(2CaO \cdot SiO_2) + 3.5H_2O$	→	$3CaO \cdot 2SiO_2 \cdot 2.5H_2O + Ca(OH)_2$
Dicalcium silicate + water		Calcium silicate hydrate (CSH) + calcium hydroxide
$3CaO \cdot Al_2O_3 + 12H_2O + Ca(OH)_2$	→	$4CaO \cdot Al_2O_3 \cdot 13H_2O$
Tricalcium aluminate + water + calcium hydroxide		Calcium aluminate hydrate (CAH)
Pozzolanic (Zhu et al., 2007):		
$3Ca(OH)_2$	→	$Ca^{2+} + 2OH^-$
$Ca^{2+}+2OH^-+SiO_2$ (clay silica)	→	CSH
$Ca^{2+}+2OH^-+Al_2O_3$ (clay alumina)	→	CAH

APPENDIX B

(a) 25S:75M specimen

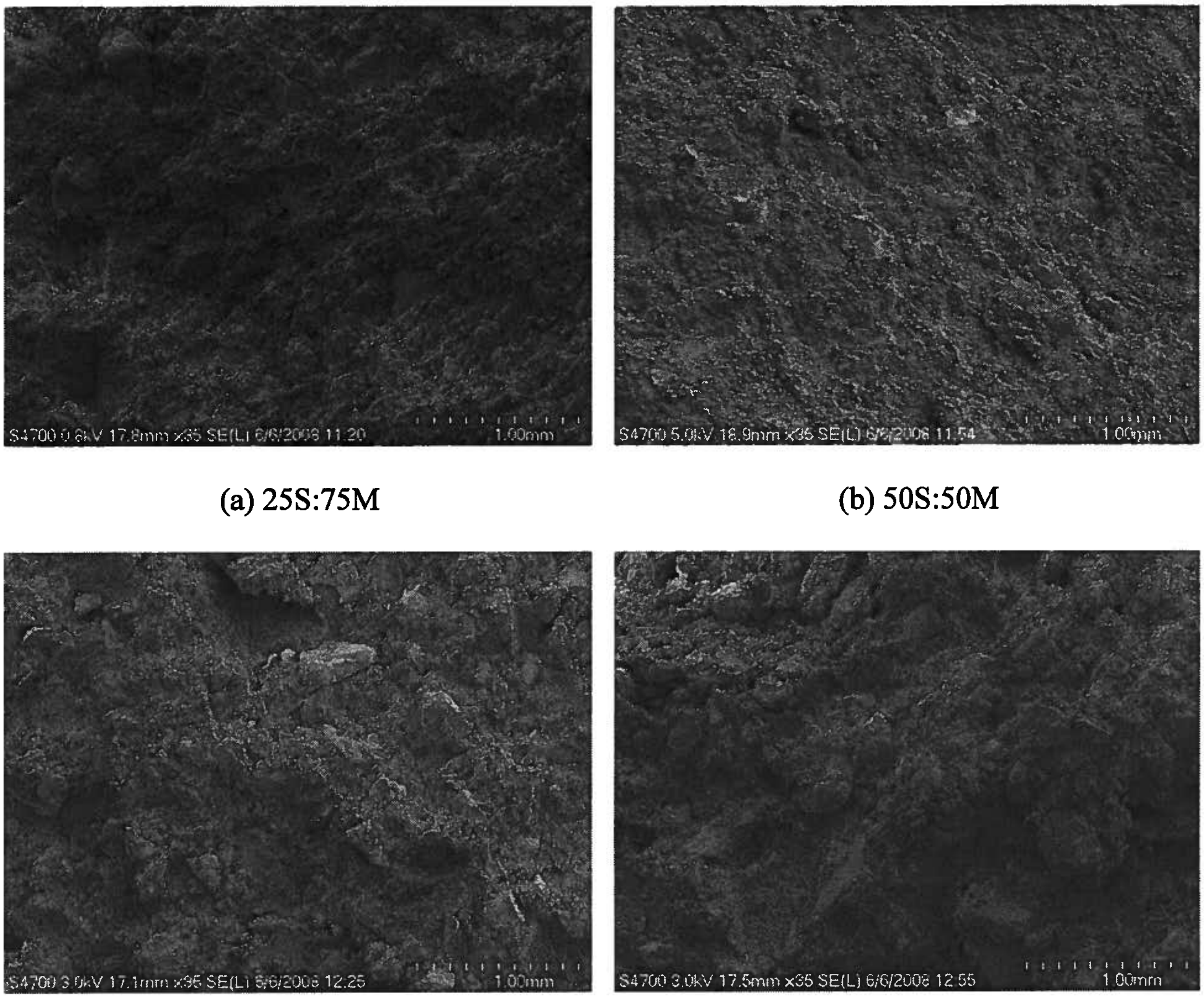
(b) 50S:50M specimen

(c) 75S:25M specimen



(d) 90S:10M specimen

Figure B.1. SEM photographs of 25S:75M to 90S:10M specimens with cement content of (c_i) of 166 kg/m^3 (per unit volume of total initial mix)



(a) 25S:75M

(b) 50S:50M

(c) 75S:25M

(d) 90S:10M

Figure B.2. SEM photographs of 25S:75M to 90S:10M specimens and moderate cement content (c_i) of 166 kg/m^3 (per unit volume of total mix) with 1mm scale shown

Table B.1. Test data for 25S:75M specimens

Plotting code w - c	Dry solids S (%)	Dry solids M (%)	Dry solids C (%)	Intial mix w_i (%)	Intial mix c_i (kg/m^3)	Intial mix ρ_i (Mg/m^3)	Bleeding w_b (%)	Beginning of curing w_p (%)	Beginning of curing c_p (kg/m^3)	Beginning of curing ρ_p (Mg/m^3)	At failure w_f (%)	At failure c_f (kg/m^3)	At failure $(w:c)_f$	At failure ρ_f (Mg/m^3)	At failure q_u (MPa)	At failure ε_a (%)	Curing period (days)
W_L-C_L	22.8	69.3	8.0	45.5	96.8	1.77	0.00	45.4	97.9	1.79	44.4	97.9	5.61	1.79	0.40	0.62	7
W_M-C_L	22.8	69.1	8.1	47.5	96.9	1.76	0.69	46.6	98.9	1.78	43.9	98.9	5.49	1.78	0.26	1.38	7
W_H-C_L	22.7	69.0	8.3	49.1	96.9	1.74	0.39	48.9	98.4	1.77	45.9	98.4	5.64	1.77	0.28	0.46	7
W_L-C_M	21.4	65.0	13.6	46.0	165.7	1.78	0.35	46.0	167.7	1.80	41.3	167.7	3.13	1.80	0.98	0.59	7
W_M-C_M	21.4	64.8	13.9	47.5	165.7	1.76	0.31	47.5	167.5	1.78	42.6	167.5	3.17	1.78	0.93	0.56	7
W_H-C_M	21.3	64.6	14.2	49.5	165.7	1.75	0.82	48.2	168.1	1.76	45.6	168.1	3.27	1.76	0.69	0.56	7
W_L-C_H	20.1	61.0	19.0	45.8	231.6	1.78	0.08	46.1	234.9	1.81	38.5	234.9	2.13	1.81	1.83	0.65	7
W_M-C_H	20.0	60.7	19.3	47.6	231.6	1.77	0.52	47.4	234.1	1.78	41.5	234.1	2.23	1.78	1.79	0.67	7
W_H-C_H	19.9	60.4	19.7	49.2	231.6	1.75	0.75	48.7	234.4	1.77	42.9	234.4	2.26	1.77	1.64	0.63	7
W_L-C_L	22.8	69.2	8.0	45.8	96.8	1.77	0.18	45.3	97.9	1.78	44.2	97.9	5.57	1.78	0.43	0.32	14
W_M-C_L	22.8	69.1	8.1	47.5	96.9	1.76	0.11	47.3	97.9	1.77	44.5	97.9	5.57	1.77	0.52	0.50	14
W_H-C_L	22.8	69.0	8.3	49.0	96.9	1.74	0.11	48.7	98.2	1.76	45.8	98.2	5.62	1.76	0.49	0.52	14
W_L-C_M	21.4	65.0	13.6	45.9	165.7	1.78	0.22	46.0	167.4	1.80	41.4	167.4	3.14	1.80	1.23	0.60	14
W_M-C_M	21.4	64.8	13.9	47.8	165.7	1.76	0.46	47.7	167.2	1.78	42.4	167.2	3.16	1.78	1.11	0.58	14
W_H-C_M	21.3	64.6	14.2	49.2	165.7	1.75	0.32	49.0	168.1	1.77	43.7	168.1	3.19	1.77	1.08	0.58	14
W_L-C_H	20.1	61.0	19.0	45.9	231.6	1.78	0.32	46.1	233.8	1.80	39.3	233.8	2.17	1.80	2.24	0.56	14
W_M-C_H	20.0	60.7	19.3	47.6	231.6	1.77	0.52	47.4	233.4	1.78	41.9	233.4	2.25	1.78	2.02	0.56	14
W_H-C_H	19.9	60.4	19.7	49.3	231.6	1.75	0.65	48.9	233.2	1.76	42.4	233.2	2.25	1.76	1.89	0.59	14
W_L-C_L	22.8	69.2	8.0	46.1	96.8	1.76	0.01	45.7	98.5	1.79	42.4	98.5	5.40	1.79	0.77	0.58	28
W_M-C_L	22.8	69.1	8.2	48.6	96.9	1.77	0.10	48.8	96.8	1.76	45.1	96.8	5.66	1.76	0.66	0.54	28
W_H-C_L	22.7	69.0	8.3	48.9	96.9	1.74	0.05	49.2	97.7	1.76	45.4	97.7	5.61	1.76	0.70	0.53	28
W_L-C_M	21.4	65.0	13.6	45.8	165.7	1.78	0.00	45.9	167.5	1.79	41.1	167.5	3.11	1.79	1.75	0.84	28
W_M-C_M	21.4	64.8	13.9	47.7	165.7	1.76	0.00	48.2	166.6	1.78	42.2	166.6	3.16	1.78	1.67	0.78	28
W_H-C_M	21.3	64.6	14.2	49.1	165.7	1.75	0.44	48.9	168.1	1.77	43.0	168.1	3.16	1.77	1.20	0.73	28
W_L-C_H	20.1	61.0	18.9	46.0	231.6	1.78	0.38	46.1	234.7	1.81	38.0	234.7	2.10	1.81	2.57	0.82	28
W_M-C_H	20.0	60.7	19.3	47.7	231.6	1.77	0.23	47.7	234.1	1.79	40.2	234.1	2.18	1.79	2.74	0.71	28
W_H-C_H	19.9	60.4	19.7	49.1	231.6	1.75	0.27	49.4	233.7	1.77	41.6	233.7	2.22	1.77	2.63	0.68	28
W_L-C_L	22.8	69.2	8.0	45.6	96.8	1.77	0.08	45.8	98.4	1.80	40.9	98.4	5.26	1.80	0.70	0.68	56
W_M-C_L	22.8	69.1	8.1	47.4	96.9	1.76	0.32	47.5	98.7	1.79	42.1	98.7	5.33	1.79	0.64	0.74	56
W_H-C_L	22.7	69.0	8.3	49.1	96.9	1.74	0.24	49.0	98.1	1.76	43.9	98.1	5.43	1.76	0.72	0.74	56
W_L-C_M	21.4	65.0	13.6	45.9	165.7	1.78	0.03	45.9	167.7	1.80	39.9	167.7	3.04	1.80	1.99	0.80	56
W_M-C_M	21.4	64.8	13.9	47.5	165.7	1.76	0.35	47.2	168.5	1.79	40.1	168.5	3.02	1.79	1.57	0.70	56
W_H-C_M	21.3	64.6	14.1	48.8	165.7	1.75	0.39	48.7	168.1	1.77	42.3	168.1	3.11	1.77	1.63	0.63	56
W_L-C_H	20.1	61.0	19.0	45.9	231.6	1.78	0.05	45.9	234.0	1.80	38.9	234.0	2.14	1.80	3.34	0.67	56
W_M-C_H	20.0	60.7	19.3	47.3	231.6	1.77	0.12	47.4	234.0	1.78	40.4	234.0	2.18	1.78	3.12	0.61	56
W_H-C_H	19.9	60.4	19.7	48.9	231.6	1.75	0.40	48.6	235.2	1.77	42.0	235.2	2.22	1.77	2.85	0.72	56

Notes: *S*, sand; *M*, silt; *C*, cement; *w*, water content; *c*, cement content; ρ, bulk density; q_u, peak strength; ε_a, strain at peak strength; subscripts: *L* (low), *M* (moderate), *H* (high).

Table B.2. Test data for 50S:50M specimens

Plotting code	Dry solids			Intial mix			Bleeding	Beginning of curing			At failure						Curing period
w - c	S (%)	M (%)	C (%)	w_i (%)	c_i (kg/m^3)	ρ_i (Mg/m^3)	w_b (%)	w_p (%)	c_p (kg/m^3)	ρ_p (Mg/m^3)	w_f (%)	c_f (kg/m^3)	$(w{:}c)_f$	ρ_f (Mg/m^3)	q_u (MPa)	ε_a (%)	(days)
W_L-C_L	45.9	46.4	7.8	43.6	97.1	1.79	0.72	42.8	98.9	1.82	39.5	98.9	5.20	1.82	0.28	1.49	7
W_M-C_L	45.8	46.3	7.9	45.3	96.8	1.77	1.62	43.4	100.3	1.81	40.0	100.3	5.17	1.81	0.19	0.95	7
W_H-C_L	45.7	46.2	8.1	47.1	96.8	1.76	1.83	45.0	100.3	1.80	41.5	100.3	5.26	1.80	0.17	0.25	7
W_L-C_M	43.1	43.6	13.2	43.7	165.7	1.80	1.09	42.6	169.8	1.83	38.3	169.8	2.98	1.83	0.68	0.50	7
W_M-C_M	43.0	43.5	13.5	45.3	165.6	1.78	1.35	43.9	170.4	1.81	39.2	170.4	2.99	1.81	0.71	0.42	7
W_H-C_M	42.9	43.4	13.8	46.9	165.6	1.77	1.49	45.4	170.0	1.79	39.8	170.0	3.00	1.79	0.72	0.36	7
W_L-C_H	40.6	41.0	18.4	43.8	231.6	1.81	1.69	41.9	240.1	1.85	34.9	240.1	1.98	1.85	1.42	0.51	7
W_M-C_H	40.4	40.8	18.8	45.4	231.6	1.79	1.62	43.6	238.3	1.82	38.2	238.3	2.11	1.82	1.48	0.56	7
W_H-C_H	40.2	40.6	19.2	47.2	231.6	1.77	2.05	44.9	239.9	1.81	38.4	239.9	2.09	1.81	1.39	0.49	7
W_L-C_L	45.9	46.4	7.8	43.6	97.1	1.80	1.59	41.6	100.1	1.82	38.2	100.1	5.03	1.82	0.26	1.93	14
W_M-C_L	45.8	46.3	7.9	45.8	96.8	1.78	0.93	44.6	98.7	1.80	40.3	98.7	5.23	1.80	0.25	2.08	14
W_H-C_L	45.7	46.2	8.1	46.7	96.8	1.76	1.19	45.0	99.7	1.79	41.6	99.7	5.26	1.79	0.25	2.25	14
W_L-C_M	43.1	43.6	13.3	44.0	165.7	1.79	0.80	43.2	169.0	1.82	38.1	169.0	2.97	1.82	0.98	0.48	14
W_M-C_M	43.0	43.5	13.6	45.6	165.6	1.78	1.43	43.9	170.7	1.81	38.6	170.7	2.95	1.81	0.80	0.41	14
W_H-C_M	42.9	43.3	13.8	47.3	165.6	1.76	1.89	45.1	171.5	1.80	39.9	171.5	2.99	1.80	0.76	0.39	14
W_L-C_H	40.6	41.0	18.5	43.8	231.6	1.80	1.27	42.6	238.0	1.84	36.0	238.0	2.04	1.84	1.96	0.58	14
W_M-C_H	40.4	40.8	18.8	45.3	231.7	1.79	1.37	44.0	238.2	1.82	37.0	238.2	2.06	1.82	2.03	0.55	14
W_H-C_H	40.2	40.6	19.2	47.1	231.6	1.77	1.66	45.6	238.6	1.81	38.3	238.6	2.10	1.81	1.76	0.54	14
W_L-C_L	45.9	46.4	7.8	43.7	97.1	1.79	0.78	42.9	99.2	1.82	38.9	99.2	5.13	1.82	0.44	1.15	29
W_M-C_L	45.8	46.3	7.9	45.3	96.8	1.77	1.08	44.1	99.5	1.81	40.3	99.5	5.20	1.81	0.36	1.24	29
W_H-C_L	45.7	46.2	8.1	47.1	96.8	1.76	1.29	45.6	99.7	1.79	42.3	99.7	5.33	1.79	0.31	1.16	29
W_L-C_M	43.1	43.6	13.3	43.7	165.7	1.79	0.92	42.7	169.2	1.82	37.9	169.2	2.95	1.82	1.08	0.69	28
W_M-C_M	43.0	43.5	13.5	45.4	165.6	1.78	1.10	44.3	170.2	1.81	39.2	170.2	2.99	1.81	1.05	0.51	28
W_H-C_M	42.9	43.3	13.8	47.0	165.6	1.76	1.39	45.5	170.6	1.80	40.4	170.6	3.02	1.80	0.97	0.53	28
W_L-C_H	40.6	41.0	18.4	43.3	231.6	1.80	1.52	42.5	238.6	1.84	34.0	238.6	1.95	1.84	2.27	0.58	28
W_M-C_H	40.4	40.8	18.9	45.5	231.7	1.79	1.31	44.5	236.8	1.81	37.4	236.8	2.08	1.81	2.35	0.58	28
W_H-C_H	40.2	40.6	19.2	47.0	231.6	1.77	1.55	45.4	238.0	1.80	38.2	238.0	2.09	1.80	2.26	0.65	28
W_L-C_L	45.9	46.4	7.8	43.5	97.1	1.79	1.15	42.3	100.0	1.83	37.1	100.0	4.91	1.83	0.39	0.57	56
W_M-C_L	45.8	46.3	7.9	45.3	96.8	1.77	1.34	43.8	99.8	1.81	39.2	99.8	5.08	1.81	0.35	0.74	56
W_H-C_L	45.7	46.2	8.1	46.9	96.8	1.76	1.51	45.2	100.0	1.79	40.9	100.0	5.18	1.79	0.40	0.41	56
W_L-C_M	43.2	43.6	13.2	43.7	165.7	1.80	0.61	42.9	168.7	1.82	37.6	168.7	2.93	1.82	1.45	0.57	56
W_M-C_M	43.0	43.5	13.5	45.4	165.6	1.78	0.76	44.4	169.5	1.81	38.7	169.5	2.96	1.81	1.40	0.70	56
W_H-C_M	42.9	43.4	13.8	47.1	165.6	1.76	1.08	46.0	169.5	1.79	40.0	169.5	3.01	1.79	1.07	0.57	56
W_L-C_H	40.7	41.1	18.3	42.6	231.7	1.81	0.79	41.9	236.2	1.83	34.2	236.2	1.96	1.83	3.06	0.70	56
W_M-C_H	40.4	40.8	18.8	46.1	231.6	1.80	1.35	44.6	234.2	1.80	37.9	234.2	2.10	1.80	2.37	0.61	56
W_H-C_H	40.2	40.6	19.2	46.8	231.6	1.77	1.07	45.7	238.0	1.81	38.9	238.0	2.12	1.81	2.57	0.60	56

Notes: *S*, sand; *M*, silt; *C*, cement; *w*, water content; *c*, cement content; ρ, bulk density; q_u, peak strength; ε_a, strain at peak strength; subscripts: *L* (low), *M* (moderate), *H* (high).

Table B.3. Test data for 75S:25M specimens

Plotting code	Dry solids			Intial mix			Bleeding	Beginning of curing			At failure						Curing period
	S	M	C	w_i	c_i	ρ_i	w_b	w_p	c_p	ρ_p	w_f	c_f	$(w{:}c)_f$	ρ_f	q_u	ε_a	
w - c	(%)	(%)	(%)	(%)	(kg/m^3)	(Mg/m^3)	(%)	(%)	(kg/m^3)	(Mg/m^3)	(%)	(kg/m^3)		(Mg/m^3)	(MPa)	(%)	(days)
W_L-C_L	69.5	23.4	7.1	36.3	96.9	1.87	3.02	32.7	103.7	1.95	27.4	103.7	4.03	1.95	0.21	0.96	7
W_M-C_L	69.3	23.4	7.2	38.3	96.9	1.85	3.61	34.7	103.8	1.93	30.2	103.8	4.30	1.93	0.21	0.28	7
W_H-C_L	69.2	23.3	7.5	40.7	96.8	1.82	5.42	34.8	105.3	1.91	31.5	105.3	4.33	1.91	0.30	0.29	7
W_L-C_M	65.8	22.2	12.0	36.0	165.7	1.88	1.90	34.1	173.3	1.93	29.3	173.3	2.52	1.93	0.94	0.55	7
W_M-C_M	65.6	22.1	12.4	38.4	165.7	1.85	3.33	34.7	176.6	1.92	30.5	176.6	2.54	1.92	0.92	0.42	7
W_H-C_M	65.3	22.0	12.7	40.5	165.7	1.83	4.65	35.0	179.3	1.91	30.6	179.3	2.49	1.91	0.84	0.36	7
W_L-C_H	62.3	21.0	16.8	36.4	231.9	1.89	2.04	34.2	241.6	1.93	28.2	241.6	1.75	1.93	1.99	0.63	7
W_M-C_H	61.9	20.9	17.3	38.5	231.6	1.86	3.51	35.0	249.0	1.93	28.6	249.0	1.73	1.93	1.94	0.52	7
W_H-C_H	61.6	20.7	17.7	40.3	231.6	1.84	4.45	35.5	249.7	1.92	28.7	249.7	1.71	1.92	1.91	0.51	7
W_L-C_L	69.5	23.4	7.1	36.3	96.9	1.87	2.82	33.1	102.8	1.94	29.5	102.8	4.28	1.94	0.32	0.35	15
W_M-C_L	69.3	23.4	7.2	38.4	96.9	1.85	3.72	34.7	103.5	1.92	29.9	103.5	4.26	1.92	0.30	0.33	14
W_H-C_L	69.2	23.3	7.5	40.5	96.9	1.83	4.29	35.1	105.2	1.91	30.8	105.2	4.26	1.91	0.30	0.31	14
W_L-C_M	65.8	22.2	12.0	36.4	165.7	1.88	1.71	34.7	172.4	1.93	29.4	172.4	2.54	1.93	1.18	0.47	14
W_M-C_M	65.6	22.1	12.4	38.4	165.7	1.86	2.29	36.0	173.3	1.91	30.8	173.3	2.58	1.91	1.07	0.37	14
W_H-C_M	65.3	22.0	12.7	40.5	165.7	1.83	3.46	36.7	175.8	1.89	31.4	175.8	2.56	1.89	1.07	0.43	14
W_L-C_H	62.3	21.0	16.8	36.3	231.6	1.89	2.33	34.0	241.3	1.93	27.7	241.3	1.73	1.93	2.56	0.52	14
W_M-C_H	61.9	20.9	17.2	38.4	231.6	1.86	3.06	35.2	244.0	1.91	29.0	244.0	1.76	1.91	2.51	0.53	14
W_H-C_H	61.6	20.7	17.7	40.5	231.6	1.84	4.47	35.6	251.2	1.92	28.6	251.2	1.70	1.92	2.45	0.52	14
W_L-C_L	69.5	23.4	7.1	36.4	96.9	1.87	2.08	32.6	98.5	1.94	28.1	98.5	4.14	1.94	0.39	0.40	27
W_M-C_L	69.3	23.4	7.3	38.3	96.9	1.85	4.69	32.9	105.4	1.93	29.2	105.4	4.11	1.93	0.44	0.35	28
W_H-C_L	69.2	23.3	7.5	40.4	96.9	1.83	4.13	35.4	104.9	1.91	31.2	104.9	4.29	1.91	0.34	0.26	27
W_L-C_M	65.8	22.2	12.1	36.5	165.8	1.88	3.09	32.5	175.5	1.93	28.6	175.5	2.43	1.93	1.27	0.54	27
W_M-C_M	65.6	22.1	12.4	38.5	165.7	1.85	2.60	35.5	174.7	1.91	30.1	174.7	2.53	1.91	1.31	0.40	28
W_H-C_M	65.3	22.0	12.7	40.4	165.7	1.83	3.41	36.6	176.1	1.89	30.5	176.1	2.51	1.89	1.27	0.46	28
W_L-C_H	62.3	21.0	16.8	36.4	231.6	1.89	1.95	34.4	240.1	1.92	28.6	240.1	1.78	1.92	3.05	0.54	28
W_M-C_H	61.9	20.9	17.2	38.5	231.6	1.86	2.49	35.3	244.4	1.92	28.4	244.4	1.73	1.92	2.87	0.55	28
W_H-C_H	61.6	20.7	17.7	40.3	231.6	1.84	3.89	36.0	247.7	1.91	28.9	247.7	1.72	1.91	2.78	0.56	28
W_L-C_L	69.5	23.4	7.1	36.4	96.9	1.87	1.42	34.8	100.1	1.91	31.1	100.1	4.50	1.91	0.42	0.47	56
W_M-C_L	69.4	23.4	7.3	38.6	96.9	1.85	1.74	36.4	100.3	1.88	32.7	100.3	4.61	1.88	0.38	0.47	56
W_H-C_L	69.2	23.3	7.4	40.3	96.9	1.83	3.08	36.7	102.9	1.89	32.4	102.9	4.48	1.89	0.38	0.34	56
W_L-C_M	65.8	22.2	12.0	36.4	165.7	1.88	1.73	34.6	171.8	1.93	28.7	171.8	2.48	1.93	1.54	0.65	56
W_M-C_M	65.6	22.1	12.4	38.4	165.7	1.86	2.07	35.9	173.6	1.91	30.0	173.6	2.53	1.91	1.36	0.44	56
W_H-C_M	65.3	22.0	12.7	40.5	165.7	1.83	2.99	36.9	175.1	1.89	30.6	175.1	2.52	1.89	1.41	0.43	56
W_L-C_H	62.2	21.0	16.8	36.2	231.6	1.88	1.65	34.5	240.1	1.93	28.2	240.1	1.76	1.93	3.44	0.60	56
W_M-C_H	61.9	20.9	17.2	38.4	231.6	1.86	2.19	36.1	242.8	1.91	28.7	242.8	1.76	1.91	3.11	0.54	56
W_H-C_H	61.6	20.8	17.7	40.3	231.6	1.84	2.82	37.2	243.3	1.89	30.2	243.3	1.79	1.89	2.89	0.58	56

Notes: *S*, sand; *M*, silt; *C*, cement; *w*, water content; *c*, cement content; ρ, bulk density; q_u, peak strength; ε_a, strain at peak strength; subscripts: *L* (low), *M* (moderate), *H* (high).

Table B.4. Test data for 90S:10M specimens

Plotting code	Dry solids			Intial mix			Bleeding	Beginning of curing			At failure						Curing period
	S	M	C	w_i	c_i	ρ_i	w_b	w_p	c_p	ρ_p	w_f	c_f	$(w:c)_f$	ρ_f	q_u	ε_a	
w - c	(%)	(%)	(%)	(%)	(kg/m^3)	(Mg/m^3)	(%)	(%)	(kg/m^3)	(Mg/m^3)	(%)	(kg/m^3)		(Mg/m^3)	(MPa)	(%)	(days)
W_L-C_L	83.7	9.4	6.9	35.0	96.9	1.89	7.25	27.3	109.5	2.02	24.0	109.5	3.55	2.02	0.43	0.52	7
W_M-C_L	83.5	9.4	7.1	36.9	96.9	1.87	6.19	28.3	110.1	1.99	25.7	110.1	3.69	1.99	0.32	0.44	7
W_H-C_L	83.3	9.4	7.3	39.0	96.9	1.84	7.06	29.9	110.2	1.96	27.2	110.2	3.80	1.96	0.32	0.31	7
W_L-C_M	79.3	8.9	11.8	35.1	165.8	1.90	6.18	28.6	184.3	2.01	23.5	184.3	2.07	2.01	1.52	0.53	7
W_M-C_M	79.0	8.9	12.1	37.0	165.7	1.87	8.35	27.6	191.1	2.01	22.8	191.1	1.95	2.01	1.72	0.49	7
W_H-C_M	78.7	8.8	12.4	39.2	165.7	1.85	9.07	28.6	192.6	1.99	25.2	192.6	2.07	1.99	1.40	0.58	7
W_L-C_H	75.1	8.4	16.4	34.8	231.5	1.90	6.89	27.2	261.0	2.02	22.0	261.0	1.40	2.02	3.58	0.62	7
W_M-C_H	74.7	8.4	16.9	36.9	231.5	1.88	9.10	26.5	270.9	2.03	22.1	270.9	1.35	2.03	3.81	0.69	7
W_H-C_H	74.3	8.3	17.3	38.8	231.5	1.85	8.26	28.2	271.9	2.01	22.8	271.9	1.37	2.01	2.33	0.69	7
W_L-C_L	83.7	9.4	6.9	34.9	96.9	1.89	4.91	29.3	105.8	1.98	26.1	105.8	3.85	1.98	0.47	0.44	15
W_M-C_L	83.5	9.4	7.1	36.9	96.9	1.87	5.75	29.9	107.9	1.97	26.7	107.9	3.83	1.97	0.41	0.53	15
W_H-C_L	83.3	9.4	7.3	39.0	96.9	1.85	6.41	30.3	109.7	1.96	27.9	109.7	3.88	1.96	0.29	0.41	15
W_L-C_M	79.3	8.9	11.8	35.0	165.7	1.89	5.59	28.1	184.9	2.01	24.4	184.9	2.12	2.01	1.84	0.69	14
W_M-C_M	79.0	8.9	12.1	36.9	165.7	1.87	6.22	28.1	189.2	2.00	21.8	189.2	1.88	2.00	1.55	0.54	14
W_H-C_M	78.7	8.8	12.5	39.0	165.7	1.85	8.62	28.1	194.2	2.00	24.0	194.2	1.98	2.00	1.73	0.57	14
W_L-C_H	75.1	8.4	16.4	35.0	231.5	1.90	5.73	29.3	254.8	2.01	22.9	254.8	1.46	2.01	3.87	0.64	14
W_M-C_H	74.7	8.4	16.9	37.0	231.5	1.88	6.94	29.4	260.9	2.00	23.5	260.9	1.45	2.00	3.83	0.75	14
W_H-C_H	74.3	8.3	17.3	38.9	231.5	1.86	8.28	29.5	267.0	2.00	23.1	267.0	1.40	2.00	3.64	0.55	14
W_L-C_L	83.7	9.4	6.9	34.9	96.9	1.89	4.04	29.9	105.1	1.97	26.5	105.1	3.91	1.97	0.60	0.56	28
W_M-C_L	83.5	9.4	7.1	37.0	96.9	1.86	5.95	29.0	109.3	1.98	25.6	109.3	3.66	1.98	0.52	0.46	28
W_H-C_L	83.3	9.4	7.3	39.0	96.9	1.84	7.09	30.8	110.2	1.97	26.5	110.2	3.72	1.97	0.52	0.44	28
W_L-C_M	79.3	8.9	11.8	35.0	165.7	1.90	6.85	27.2	187.0	2.02	22.9	187.0	2.00	2.02	2.38	0.60	28
W_M-C_M	79.0	8.9	12.1	37.0	165.7	1.87	6.78	29.5	186.3	1.99	23.6	186.3	2.02	1.99	1.96	0.57	28
W_H-C_M	78.7	8.8	12.4	39.0	165.7	1.85	9.33	28.3	193.0	1.99	23.1	193.0	1.92	1.99	2.22	0.58	28
W_L-C_H	75.1	8.4	16.4	35.0	231.5	1.90	6.62	27.5	260.8	2.03	21.5	260.8	1.37	2.03	5.16	0.78	28
W_M-C_H	74.7	8.4	16.9	37.0	231.5	1.88	7.79	28.3	264.4	2.01	22.9	264.4	1.41	2.01	5.02	0.66	28
W_H-C_H	74.3	8.3	17.4	38.9	231.5	1.85	8.96	28.2	272.0	2.01	22.8	272.0	1.36	2.01	4.60	0.64	28
W_L-C_L	83.7	9.4	6.9	35.1	96.9	1.89	5.23	28.7	108.1	2.01	24.3	108.1	3.61	2.01	0.73	0.54	56
W_M-C_L	83.5	9.4	7.1	36.9	96.9	1.87	5.80	29.5	108.9	1.98	25.3	108.9	3.67	1.98	0.70	0.55	56
W_H-C_L	83.3	9.4	7.3	39.0	96.9	1.84	6.97	29.6	111.4	1.98	25.9	111.4	3.63	1.98	0.42	0.58	56
W_L-C_M	79.3	8.9	11.8	34.8	165.7	1.90	5.75	28.2	184.7	2.01	22.2	184.7	1.96	2.01	2.68	0.61	56
W_M-C_M	79.0	8.9	12.1	36.9	165.7	1.87	6.87	29.3	187.4	2.00	23.1	187.4	1.99	2.00	2.56	0.72	56
W_H-C_M	78.7	8.8	12.4	39.0	165.7	1.85	6.06	29.1	191.5	1.99	23.7	191.5	1.97	1.99	1.51	0.64	56
W_L-C_H	75.1	8.4	16.4	35.0	231.5	1.90	5.51	29.1	255.6	2.01	23.1	255.6	1.47	2.01	5.28	0.71	56
W_M-C_H	74.7	8.4	16.9	36.9	231.5	1.88	7.48	28.5	265.1	2.02	21.7	265.1	1.35	2.02	5.48	0.83	56
W_H-C_H	74.3	8.3	17.3	38.9	231.5	1.85	9.48	28.7	271.0	2.01	21.9	271.0	1.33	2.01	4.99	0.73	56

Notes: *S*, sand; *M*, silt; *C*, cement; *w*, water content; *c*, cement content; ρ, bulk density; q_u, peak strength; ε_a, strain at peak strength; subscripts: *L* (low), *M* (moderate), *H* (high).