

2015

Engineering characteristics of compacted sand-bentonite mixtures

Ali Faisal Ghazi
Edith Cowan University

Follow this and additional works at: <https://ro.ecu.edu.au/theses>



Part of the [Civil and Environmental Engineering Commons](#)

Recommended Citation

Ghazi, A. F. (2015). *Engineering characteristics of compacted sand-bentonite mixtures*. Edith Cowan University. Retrieved from <https://ro.ecu.edu.au/theses/1615>

This Thesis is posted at Research Online.
<https://ro.ecu.edu.au/theses/1615>

Edith Cowan University

Copyright Warning

You may print or download ONE copy of this document for the purpose of your own research or study.

The University does not authorize you to copy, communicate or otherwise make available electronically to any other person any copyright material contained on this site.

You are reminded of the following:

- Copyright owners are entitled to take legal action against persons who infringe their copyright.
- A reproduction of material that is protected by copyright may be a copyright infringement. Where the reproduction of such material is done without attribution of authorship, with false attribution of authorship or the authorship is treated in a derogatory manner, this may be a breach of the author's moral rights contained in Part IX of the Copyright Act 1968 (Cth).
- Courts have the power to impose a wide range of civil and criminal sanctions for infringement of copyright, infringement of moral rights and other offences under the Copyright Act 1968 (Cth). Higher penalties may apply, and higher damages may be awarded, for offences and infringements involving the conversion of material into digital or electronic form.

Engineering Characteristics of Compacted Sand-Bentonite Mixtures

By
Ali Faisal Ghazi
(10245009)

Associate Professor Sanjay Kumar Shukla, Principal Supervisor
Dr Mehdi Khiadani, Associate Supervisor

This thesis is presented in fulfilment of the requirements for the
Degree of Master of Engineering Science (MEngSc)
May 2015



School of Engineering
Faculty of Health, Engineering and Science
Edith Cowan University

USE OF THESIS

The Use of Thesis statement is not included in this version of the thesis.

ABSTRACT

Sand-clay mixtures are commonly used as a liner/barrier material in various engineering applications, such as construction of hydraulic and waste containments. Permeability, compressibility and strength are important properties of sand-clay mixtures and are often required for the design of the liner/barrier of the containments.

In states like Western Australia, which is covered mostly by sandy soils, engineers face difficulties with economically sourcing clays for liner/barrier applications. Any reliable research finding that recommends an optimum clay content to be used with sandy soil can be of significant importance. Such findings for the local Perth sandy soil are rarely available in the literature. Sodium bentonite can be added to Perth sandy soil as active clay in an appropriate amount to create a cost-effective liner/barrier material, especially for landfill applications. Bentonite has been used for such applications in other parts of the world.

In this research, the permeability, compressibility and strength characteristics of Perth sand–bentonite mixtures are investigated to support recommendation for a cost-effective liner material with three different local soils. A series of standard compaction tests, a one-dimensional consolidation test for compressibility and permeability characteristics, and an unconfined compression test and direct shear tests for strength characteristics were conducted on nine different sand-bentonite mixtures. The mixtures were formed by mixing local soils, namely brickies sand, plaster sand, and river sand with 5, 10, and 20%, by dry weight, of sodium bentonite.

The test results show that soil permeability and compressibility are greatly affected by the type of soil used in the mixtures. The optimum amount of bentonite for brickies sand, plaster sand and river sand to achieve a permeability of less than 10^{-9} m/s, which is a liner design requirement, was found to be 5%, 10%, and 10%, respectively. The compression index increases linearly with the increasing bentonite content for any type of sand-bentonite mixture, but the rate of increase is relatively higher when bentonite is mixed with brickies sand. The results obtained from strength tests indicate that the unconfined compressive strength, the cohesion and the Young's modulus all increase with increasing bentonite content, while the angle of internal friction decreases.

Further, four possible methods, namely Casagrande logarithm of time fitting method, Taylor square root of time fitting method, analytical method and improved rectangular hyperbola fitting, are compared for estimating the coefficient of consolidation of sand-

bentonite mixture. The analysis shows that the improved rectangular hyperbola method is the most reliable method for calculating the coefficient of consolidation among the four methods.

ACKNOWLEDGEMENTS

First of all, I am grateful to the almighty God who helped me and supported me to complete this thesis.

I would like to express my sincere gratitude to my main supervisor **Associate Professor Sanjay Kumar Shukla** for his extensive knowledge, guidance, and support during the planning and development of this research work. Also, I would like to thank him for his advice, encouragement, and friendship.

I would like to express my great appreciation to my second supervisor **Dr. Mehdi Khiadani** and the head of school of engineering **Professor Daryoush Habibi** for their continued suggestions and support during my research work.

I thank **Dr. Mohamed Ismail** and **Adrian Styles** for their help in handling the experimental instruments in the geotechnical and concrete labs.

I also would like to thank **Simon Restall** and **Joe McCormick** of **Bentonite Resources Pty Ltd** and **Matt Vos** of **HIND'S Sand Supplies and Earthmoving** for their help in providing the materials used for the experimental part in the study.

Finally, I would like to thank **my parents, brothers, and sisters** for their support and encouragement, and also a special thanks to **my wife and children (Hasanin, Janna, Lujain, and Fatima)** for their love, patience, understanding, inspiration. I am so blessed to have you all in my life!

The declaration page
is not included in this version of the thesis

TABLE OF CONTENTS

USE OF THESIS	i
ABSTRACT	ii
ACKNOWLEDGEMENTS	iv
DECLARATION	v
TABLE OF CONTENTS	vi
LIST OF TABLES	ix
LIST OF FIGURES	x
LIST OF ABBREVIATIONS	xiv
CHAPTER ONE: INTRODUCTION	1
1.1 General	1
1.2 Applications of Sand-Bentonite Mixtures.....	1
1.2.1 Landfill.....	2
1.2.2 Slurry cutoff walls.....	2
1.2.3 Radioactive waste disposal facilities	4
1.3 Objectives and Scope of the Work.....	5
1.4 Publications based on the present work	5
1.5 Organisation of the Work.....	5
CHAPTER TWO: LITERATURE REVIEW.....	7
2.1 General	7
2.2 Basic Details of Sand-Bentonite Mixtures	7
2.3 Characteristics of Sand-bentonite Mixtures	10
2.3.1 Compaction characteristics	10
2.3.2 Compressibility characteristics	14
2.3.3 Permeability characteristics	17
2.3.4 Strength characteristics	23
2.4 Conclusions	25

CHAPTER THREE: MATERIALS AND METHODS26

3.1 General.....	26
3.2 Materials.....	26
3.2.1 Sands	26
3.2.2 Bentonite	28
3.3 Methods.....	30
3.3.1 Basic tests.....	30
3.3.2 Standard compaction tests.....	30
3.3.3 One-dimensional consolidation tests	30
3.3.4 Direct shear tests	31
3.3.5 Unconfined compression tests	32
3.4 Conclusions	33

CHAPTER FOUR: PERMEABILITY AND COMPRESSIBILITY

CHARACTERISTICS OF SAND-BENTONITE MIXTURES34

4.1 General.....	34
4.2 Compaction Test Results	34
4.3 One-Dimensional Consolidation Test Results	37
4.4 Conclusions.....	46

CHAPTER FIVE: STRENGTH CHARACTERISTICS OF SAND -

BENTONITE MIXTURE.....47

5.1 General.....	47
5.2 Strength Tests Results	47
5.2.1 Unconfined compressive strength tests results	47
5.2.2 Direct shear tests results.....	59
5.3 Conclusions.....	62

CHAPTER SIX: METHODS OF ESTIMATING COEFFICIENT OF

CONSOLIDATION (c_v).....63

6.1 General.....	63
6.2 Terzaghi's One-Dimensional Consolidation Theory	63

6.3	Methods of Estimating c_v	64
6.3.1	Casagrande logarithm of time fitting method	64
6.3.2	Taylor square root of time fitting method.....	66
6.3.3	Analytical method.....	67
6.3.4	Improved rectangular hyperbola fitting method	68
6.4	Results and Discussion.....	69
6.5	Conclusions.....	74
 CHAPTER SEVEN: SUMMARY AND CONCLUSIONS.....		75
7.1	Summary	75
7.2	Conclusions.....	75
7.2.1	Compaction behaviour	75
7.2.2	Compressibility and permeability behaviour	76
7.2.3	Strength behaviour	76
7.2.4	Methods for estimating c_v	77
7.3	Recommendations for Future Works	78
 REFERENCES.....		79

LIST OF TABLES

Table 2.1	Change in maximum dry unit weight and optimum moisture content with increasing bentonite content for different sand-clay mixtures presented by Howell et al. (1997)	11
Table 2.2	Results of the compaction tests reported by Akgün et al. (2006)	13
Table 2.3	Effects of coarse particles on the permeability of fine grained soil	22
Table 3.1	Percentage of fine, medium and coarse gradation of the three types of sands used in the study according to the Australian standard AS 1726-1993	27
Table 3.2	Physical properties of sands	28
Table 4.1	Maximum dry unit weight and optimum moisture content for soils	35
Table 5.1	Constants of the linear equation estimated for friction angle trend lines of three types of mixtures	61

LIST OF FIGURES

Figure 1.1	A typical cross section area of sand-bentonite liner (After Chapuis, 1990)	2
Figure 1.2	Slurry cutoff walls around landfill (After USEPA, 1995)	3
Figure 1.3	The construction process of slurry cutoff walls (After Barrier, 1995)	3
Figure 1.4	Diagram of disposal facility for high-level radioactive wastes in Japan (After Ogata et al. 1999)	4
Figure 2.1	Diagram of the montmorillonite structure (After Das, 2013)	8
Figure 2.2	Comparison between montmorillonite (M), hydrous mica ("illite") (H), and kaolinite (K) with regards to hydraulic conductivity and void ratio (After Pusch, 1992)	9
Figure 2.3	Influence of clay soil type and content on (a) optimum moisture content and (b) maximum dry unit weight versus clay soil content (After Howell, 1997)	11
Figure 2.4	Compaction curve reported by Chalermyanont and Arrykul (2005)	12
Figure 2.5	Variation of maximum dry unit weight versus bentonite content (After Amadi and Eberemu, 2012)	13
Figure 2.6	Variation of optimum moisture content versus bentonite content (After Amadi and Eberemu, 2012)	14
Figure 2.7	Time-deformation curve during consolidation for a given load (After Das, 2013)	15
Figure 2.8	Compression ratio versus fines content for different soil-bentonite mixtures (After D'Appolonia, 2000)	16
Figure 2.9	Hydraulic conductivity versus added bentonite for Sodium and Calcium sand- bentonite mixtures (After Gleason et al., 1997)	18
Figure 2.10	Hydraulic conductivities of sand-bentonite with different permeants (After Kenney, 1992)	20
Figure 2.11	Hydraulic conductivity versus clay void ratio of sand-bentonite mixtures (After Mollins et al., 1996)	21
Figure 3.1	The materials used in the study	27

Figure 3.2	Particle-size distributions of sands	28
Figure 3.3	Particle-size distribution of bentonite	29
Figure 3.4	Compaction curve of bentonite	29
Figure 3.5	The apparatus of direct shear test used in the study	32
Figure 3.6	The apparatus of unconfined compressive strength test used in the study	33
Figure 4.1	Compaction curves of brickies sand-bentonite mixtures	35
Figure 4.2	Compaction curves of plaster sand-bentonite mixtures	36
Figure 4.3	Compaction curves of river sand-bentonite mixtures	36
Figure 4.4	Bentonite content versus maximum dry unit weight for all mixtures	37
Figure 4.5	Bentonite content versus optimum moisture content for all mixtures	37
Figure 4.6	Settlement (δ) versus time (t) relationship for 5RS-B mixtures	38
Figure 4.7	Settlement (δ) versus time (t) relationship for 10RS-B mixtures	38
Figure 4.8	Settlement (δ) versus time (t) relationship for 20RS-B mixtures	39
Figure 4.9	Settlement (δ) versus time (t) relationship for 5PS-B mixtures	39
Figure 4.10	Settlement (δ) versus time (t) relationship for 10PS-B mixtures	40
Figure 4.11	Settlement (δ) versus time (t) relationship for 20PS-B mixtures	40
Figure 4.12	Settlement (δ) versus time (t) relationship for 5BrS-B mixtures	41
Figure 4.13	Settlement (δ) versus time (t) relationship for 10BrS-B mixtures	41
Figure 4.14	Settlement (δ) versus time (t) relationship for 20BrS-B mixtures	42
Figure 4.15	Void ratio (e) versus logarithm effective stress (σ'_v) relationship for all mixtures	44
Figure 4.16	Compression index (C_c) versus bentonite content (p) relationship of all mixtures	45
Figure 4.17	Permeability (k) versus effective stress (σ'_v) relationships of all mixtures	46
Figure 4.18	Permeability (k) versus void ratio (e) relationships of all mixtures	47
Figure 5.1	Unconfined compressive test results for river sand bentonite mixtures	49
Figure 5.2	Unconfined compressive test results for plaster sand bentonite mixtures	49

Figure 5.3	Unconfined compressive test results for brickies sand bentonite mixtures	50
Figure 5.4	Bentonite content versus compressive strength for all mixtures	51
Figure 5.5	Bentonite content versus maximum vertical strain for all mixtures	52
Figure 5.6	Bentonite content versus Young's modulus for all mixtures	53
Figure 5.7	Bentonite content versus angle of failure for all mixtures	54
Figure 5.8	Photographs of brickies sand specimens tested by unconfined compressive strength test	54
Figure 5.9	Photographs of 5BrS-B specimens tested by unconfined compressive strength test	55
Figure 5.10	Photographs of 10BrS-B specimens tested by unconfined compressive strength test	55
Figure 5.11	Photographs of 20BrS-B specimens tested by unconfined compressive strength test	56
Figure 5.12	Photographs of plaster sand specimens tested by unconfined compressive strength test	56
Figure 5.13	Photographs of 5PS-B specimens tested by unconfined compressive strength test	57
Figure 5.14	Photographs of 10PS-B specimens tested by unconfined compressive strength test	57
Figure 5.15	Photographs of 20PS-B specimens tested by unconfined compressive strength test	58
Figure 5.16	Photographs of river sand specimens tested by unconfined compressive strength test	58
Figure 5.17	Photographs of 5RS-B specimens tested by unconfined compressive strength test	59
Figure 5.18	Photographs of 10RS-B specimens tested by unconfined compressive strength test	59
Figure 5.19	Photographs of 20RS-B specimens tested by unconfined compressive strength test	60
Figure 5.20	Mohr-Coloumb failure envelopes for RSB mixtures	61
Figure 5.21	Mohr-Coloumb failure envelopes for PSB mixtures	61

Figure 5.22	Mohr-Coloumb failure envelopes for BrSB mixtures	62
Figure 5.23	Bentonite content versus friction angle for all mixtures	63
Figure 5.24	Bentonite content versus cohesion factor for all mixtures	64
Figure 6.1	Casagrande logarithm of time method for estimating c_v (After Taylor, 1948)	67
Figure 6.2	Taylor square root of time method for estimating c_v (After Taylor, 1948)	68
Figure 6.3	Theoretical T/U versus T relationship	70
Figure 6.4	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% river sand-bentonite mixture	71
Figure 6.5	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% river sand-bentonite mixture	72
Figure 6.6	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% river sand-bentonite mixture	72
Figure 6.7	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% plaster sand-bentonite mixture	73
Figure 6.8	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% plaster sand-bentonite mixture	73
Figure 6.9	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% plaster sand-bentonite mixture	74
Figure 6.10	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% brickies sand-bentonite mixture	74
Figure 6.11	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% brickies sand-bentonite mixture	75
Figure 6.12	Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% brickies sand-bentonite mixture	75

LIST OF ABBREVIATIONS

$(a), (b)$	constants
B_e	percent bentonite ($m_s^{\text{bentonite}} / m_s^{\text{sand}}$)
c	cohesion
C_c	compressibility index
c_v	coefficient of consolidation
e	void ratio
e_o	initial void ratio
e_{\min}	minimum void ratio
e_{\max}	maximum void ratio
H	maximum drainage distance
k	permeability
k_I	bentonite content per pore volume
$m_s^{\text{bentonite}}$	solid mass of bentonite
m_s^{sand}	solid mass of sand
T_1, T_2, T_3	time factors corresponding to δ_1, δ_2 , and δ_3 , respectively.
t	elapsed time
t_{50}	time for 50% primary consolidation
t_{90}	time for 90% primary consolidation
u	excess pore water pressure
z	sample depth
δ_1, δ_2	two reading in the early phase of consolidation corresponding to t_1 and t_2 in consolidation test, respectively.
δ_3	the third reading taking at t_3 in consolidation test.
δ_i, δ_f	initial and final readings in consolidation test, respectively.
ρ_m	density
γ_d	dry unit weight
$\gamma_{d\min}$	minimum dry unit weight

$\gamma_{d\max}$	maximum dry unit weight
θ°	angle of failure
$\rho_d^{mixture}$	dry density of the mixture
σ_v'	effective stress
τ_{\max}	maximum shear strength
ϕ	friction angle

CHAPTER ONE

INTRODUCTION

1.1 General

For the past several decades, mixing sand with an adequate amount of clay/bentonite has been a common practice for creating mixtures as construction materials used in a variety of engineering applications, such as hydraulic and waste containments. The combination of mixing sand and bentonite can be able to provide a very low permeability because of the ability of bentonite to swell and then fill the voids between sand particles. Another benefit of the mixture is low compressibility which is provided by sand framework. Furthermore, the mixture has less susceptible to frost damage comparing with natural clays (Dixon and Gray 1985) with low shrinkage potential in terms of wetting or drying processes (Kraus et al. 1997) which lead to better volume stability and higher strength. The sand-bentonite mixture seems to be an economical solution for the geoenvironmental applications in places which are covered mostly by sandy soils. For the design purposes, permeability and strength characteristics of the nominated materials should be examined in order to select the suitable and economical ratio which meets the requirements. In this chapter, the merits of sand-bentonite mixture and its geoenvironmental applications are introduced. Moreover, it provides the objectives and the scope of the present work. Finally, it describes how the work will be organized.

1.2 Applications of Sand-Bentonite Mixtures

Sand-clay mixtures have been utilized as a liner/barrier material in several engineering applications. These engineering applications include waste containments, such as landfill, cutoff walls, cores of earth dams, and buffer and backfill materials of radioactive nuclear waste containments, and also hydraulic containments, such as reservoirs. In the following sections, there are basic descriptions about the most common engineering applications which used sand-clay mixtures as a liner/barrier in their structures.

1.2.1 Landfill

In the past, the way of solid waste disposal was by open dumps. This way has affected the human health and the environment, because of the contaminants migrating to the underground soil and water; therefore, modern landfills with the isolated liners have been used. Various types of isolated liners have been suggested, such as compacted clay liners, geosynthetic clay liners, and soil-bentonite liners. Soil-bentonite liner is a way for controlling the effects of waste materials on underground soil and water by mixing soil such as sand with a low amount of bentonite and water as an insulation barrier. This kind of barrier has been presented in many research studies (Abeele 1986; Akgün et al. 2006; Chapuis 1990; Chapuis 1981; Sivapullaiah et al. 2000). In landfills, the typical cross section area of this liner should be consisted of layers namely: a sand-bentonite layer, two filter layers, and protective layer (Chapuis 1990), as shown in Figure 1.1. The thickness of these layers is usually between (15 - 20) cm.



Figure 1.1 A typical cross section area of soil-bentonite liner (After Chapuis 1990)

1.2.2 Slurry cutoff walls

Slurry cutoff walls are subsurface barriers established to isolate the existing landfills (Figure 1.2) and the contaminated soils, in order to prevent the spread of the resulted contaminants from the subsurface soil and water to the surrounding environment. Soil such as sand, amended with bentonite has been used as a slurry cutoff wall (D'Appolonia 1980; Ryan 1985; Evans 1993). The method used for the construction process is a slurry trench method as seen in Figure 1.3. This method includes excavating a narrow trench with a typical width of 2 to 5 feet (D'Appolonia 1980), and filling it later with a mixture of the excavated soils and bentonite-water slurry. The percentage of bentonite in the trench is typically 4% to 6% by weight (Barrier, 1995).

According to Barrier, the typical hydraulic conductivity of soil-bentonite is generally 10^{-7} to 10^{-8} cm/s.

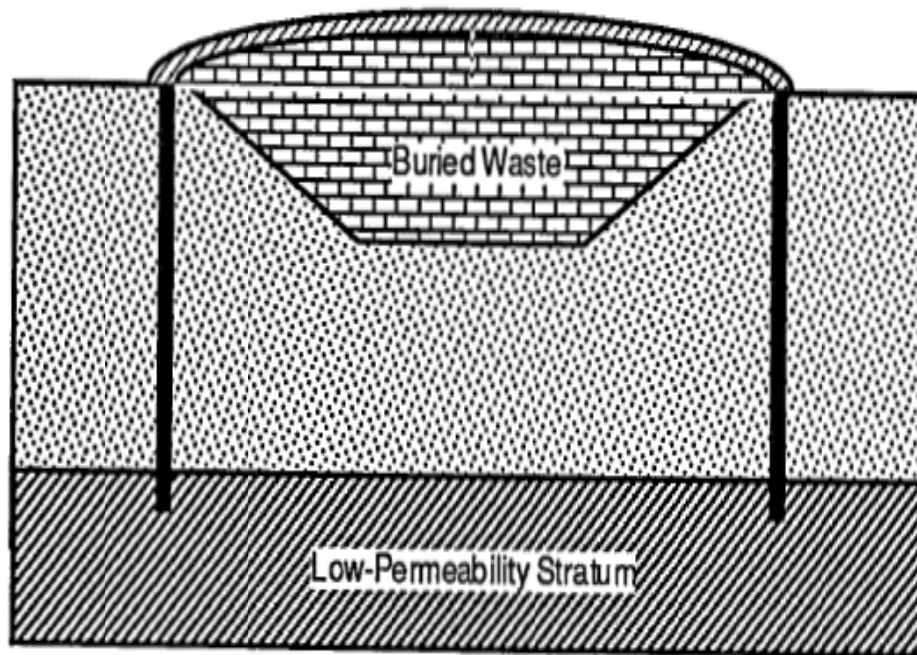


Figure 1.2 Slurry cutoff walls around landfill (After USEPA 1995)

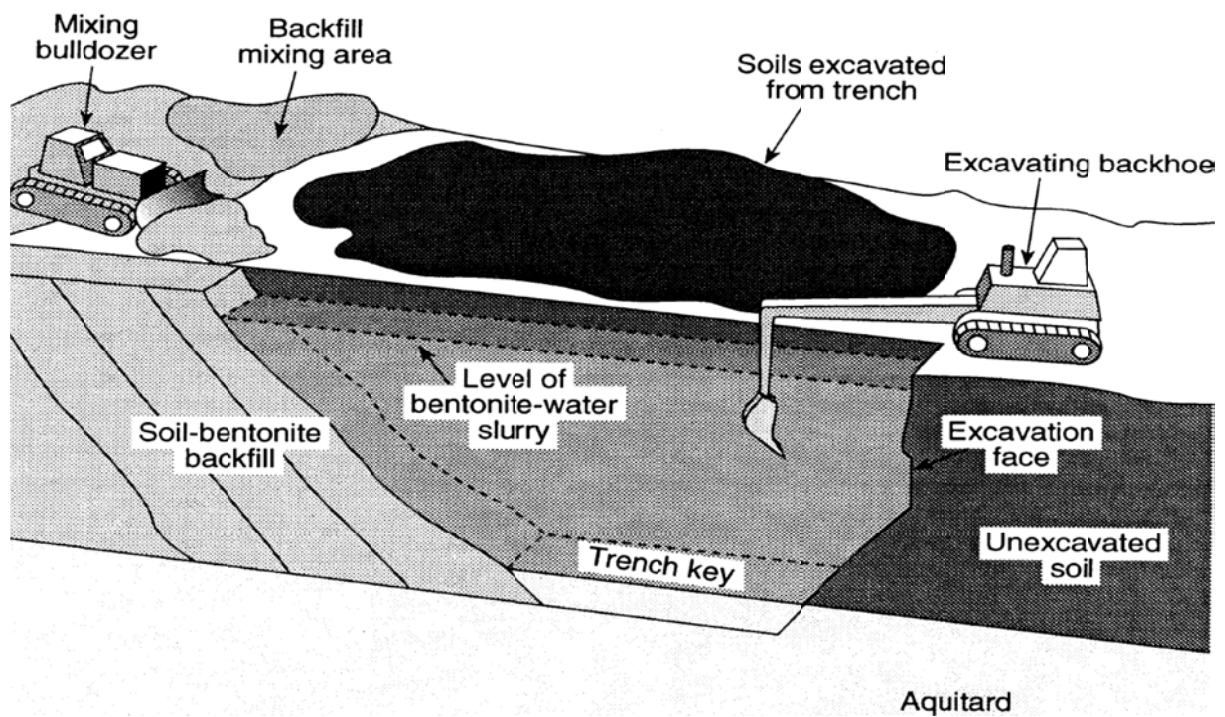


Figure 1.3 The construction process of slurry cutoff walls (After Barrier 1995)

1.2.3 Radioactive waste disposal facilities

Radioactive waste disposal facilities can be defined as repositories for different types of radioactive wastes which cause hazardous for human beings and the environment. The radioactive wastes were categorized into different levels and the way for classification differs from nation to another. These levels are very low-level wastes, low-level wastes, intermediate-level wastes, and high-level wastes. A sand-bentonite mixture has been received a great attention as buffer and backfill materials in the radioactive waste disposal facilities in order to protect the surrounding area from the migration of pollutants. Such a practice has been adopted in different countries, such as Japan (Ogata et al. 1999), Canada, Sweden (Komine 2004), Germany, Switzerland, and France (Akgün et al. 2006). Figure 1.4 shows the diagram of disposal facility for high-level radioactive wastes used in Japan. From the Figure, the buffer material is located around the container of high-level radioactive waste, while the backfill material is located in the access tunnel of disposal facility. According to the Japanese program, the hydraulic conductivity of backfill materials is required to be between 10^{-11} and 10^{-12} m/s (Japan Nuclear Cycle Development Institute 1999 a, b).

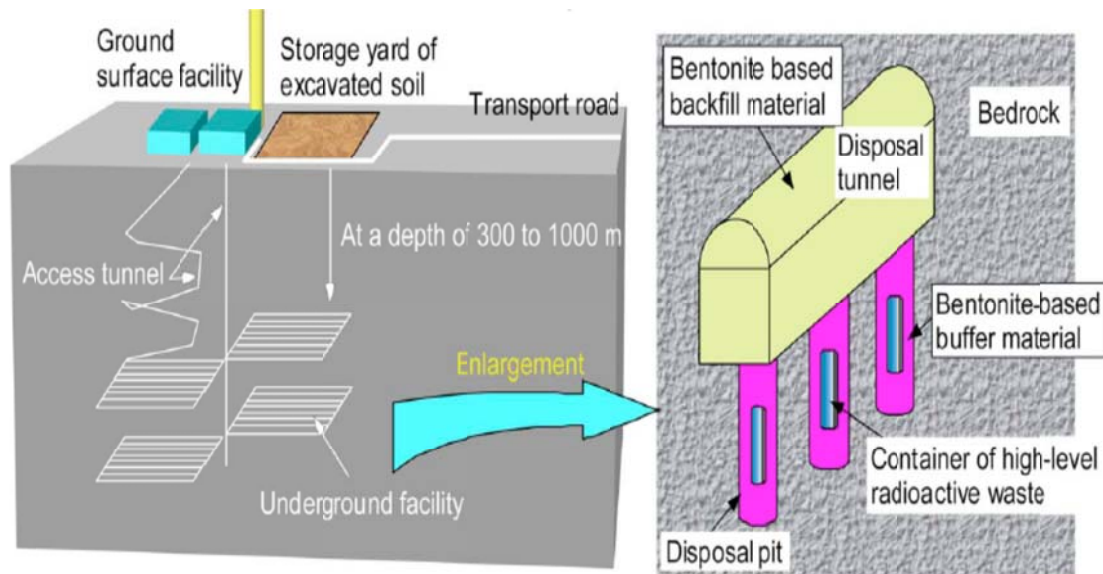


Figure 1.4 Diagram of disposal facility for high-level radioactive wastes in Japan (After Ogata et al. 1999)

1.3 Objectives and Scope of the Work

This research aims at investigating different Perth sand-bentonite mixtures for the purpose of hydraulic barriers in engineering applications such as hydraulic and waste barriers/liners. In order to cope with the research problem, which is mentioned previously, the major aims of this study are given below:

- Evaluating the effects of the variation in particle-size distribution of compacted sand-bentonite mixtures on their permeability (k) and compressibility (C_c) characteristics.
- Comparing different strength characteristics of sand-bentonite mixtures containing different particle-size distribution.
- Recommending a specific sand-bentonite mixture composition which can produce permeability to meet the hydraulic barrier/liner design requirements and suitable strength.
- Investigating four different methods for estimating the coefficient of consolidation (c_v) of sand-bentonite mixtures and a comparison is made to find out the most suitable method.

1.4 Publications based on the present work (Under Review and Preparation)

1. Ghazi, A.F., Shukla, S.K., and Khiadani, M., 2014. Permeability and compressibility characteristics of compacted Perth sand-bentonite mixtures. *International Journal of Geomechanics and Geoengineering* (under review)
2. Ghazi, A.F., Shukla, S.K., and Khiadani, M., 2015. Strength characteristics of compacted Perth sand-bentonite mixtures (under preparation)

1.5 Organization of the Work

The study presents seven different chapters. **Chapter 1** gives a general introduction about sand-bentonite mixture and its engineering applications, and the objectives and scope of the work. **Chapter 2** provides the previous literature review which covers different characteristics of sand-bentonite mixture. **Chapter 3** is about the materials used in the study and also the tests which are conducted to determine the characteristics of sand-bentonite mixture. In **Chapter 4**, the results of permeability and compressibility characteristics of sand-bentonite mixtures are introduced.

Chapter 5 describes the results of different strength aspects of sand-bentonite mixtures. In **Chapter 6**, a comparison is made among four different methods used for estimating the coefficient of consolidation. Finally, a summary, conclusions and recommendations for the future work regarding this aspect are given in **Chapter 7**.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

In the absence of impervious natural soils, a sand-bentonite mixture has been commonly suggested as an impermeable material for preventing and reducing migration of the contaminants in geoenvironmental engineering applications such as, landfill liners, cutoff walls, and buffers and backfills of radioactive waste disposal facilities. It is also used in some engineering applications as hydraulic barrier, such as reservoir. The engineering characteristics of sand-bentonite mixtures, such as compressibility, permeability, and strength were investigated by several research works. This chapter has focused on reporting the literature related to investigating the characteristics of sand-bentonite mixtures.

2.2 Basic Details of Sand-Bentonite Mixtures

Liners/barriers are low permeable materials in the structure of engineering applications such as landfills and other containing sites. The role of these liners is represented by protecting and preventing the mobility of contaminants based on having very low permeability. These liners can be created of different materials such as, compacted clay, bentonite and soil, geotextile, plastic geomembrane, and cement (Sällfors & Öberg-Högsta 2002). This protective system consists of one or more from these kinds of liners which have different behaviour in terms of contaminants movement. Many types of liners in the modern landfills were generated namely compacted clay liner, sand-bentonite liner, geosynthetic clay liner, geomembrane (GM). The system of liner should be designed in a suitable way for the specific waste conditions and the contaminated sites (Alther 1987). Therefore; Alther (1987) provided criteria for designing the liners which were: (1) facility type such as, landfills; (2) type and volume of waste; (3) approximate location of the site; and (4) predicted long life for the facility.

The sand-bentonite mixture is a combination of two different materials in terms of particle-size distribution and chemical activity to produce a material with low permeability, low compressibility, and appropriate strength. Sand is consisted of small particles of rock fragments

and mineral. The main component of sands is mineral quartz. The physical characteristics of bentonite are based on the characteristics of smectite minerals. These characteristics are: high swelling, large cation-exchange capacity, low hydraulic conductivity, and large specific surface area (Gleason et al. 1997). Bentonite is available in two major types; Sodium (Na) and Calcium (Ca) depending on the type of external cation. Sodium bentonite is more used in the engineering practices than Calcium bentonite because sodium bentonite has lower hydraulic conductivity and higher swelling (Alther 1982, 1987; Reschke and Haug 1991). Mesri and Olson (1971) stated that at the same void ratio, a calcium-dominated smectite was about 1,000 times more permeable than a sodium-dominated smectite. Bentonite is utilized in different engineering practices, such as barriers in landfill, geosynthetic clay liners, and vertical cutoff walls (Gleason et al. 1997). Smectite minerals in bentonite have mainly montmorillonite in its structure which is shown in Figure 2.1. The high percentage of montmorillonite is the reason for swelling property of bentonite.

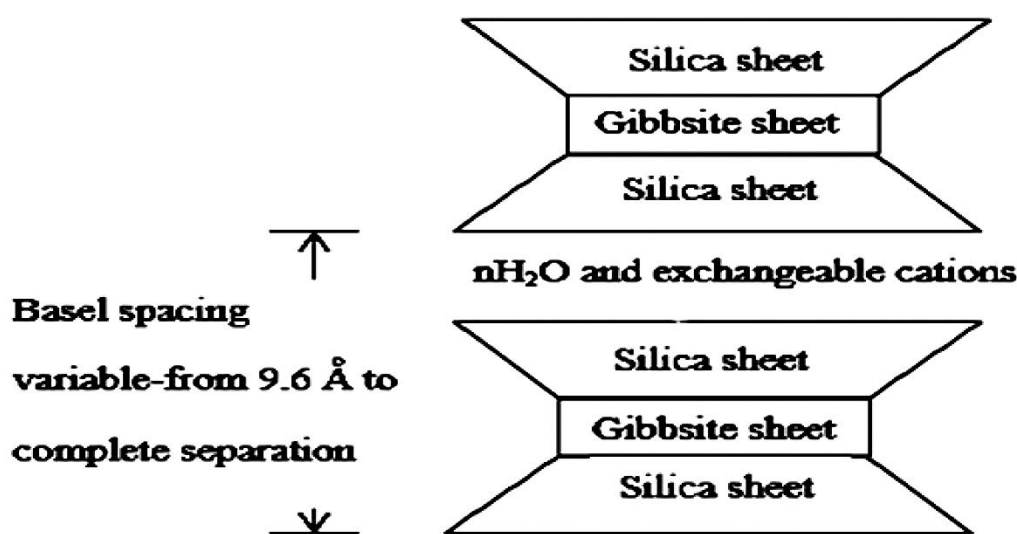


Figure 2.1 Diagram of the montmorillonite structure (After Das 2013)

Further, the hydraulic conductivity for montmorillonite (M) is higher than other Smectite groups, which are: illite (H), and kaolinite (K), at very low densities (Figure 2.2) (Pusch 1992).

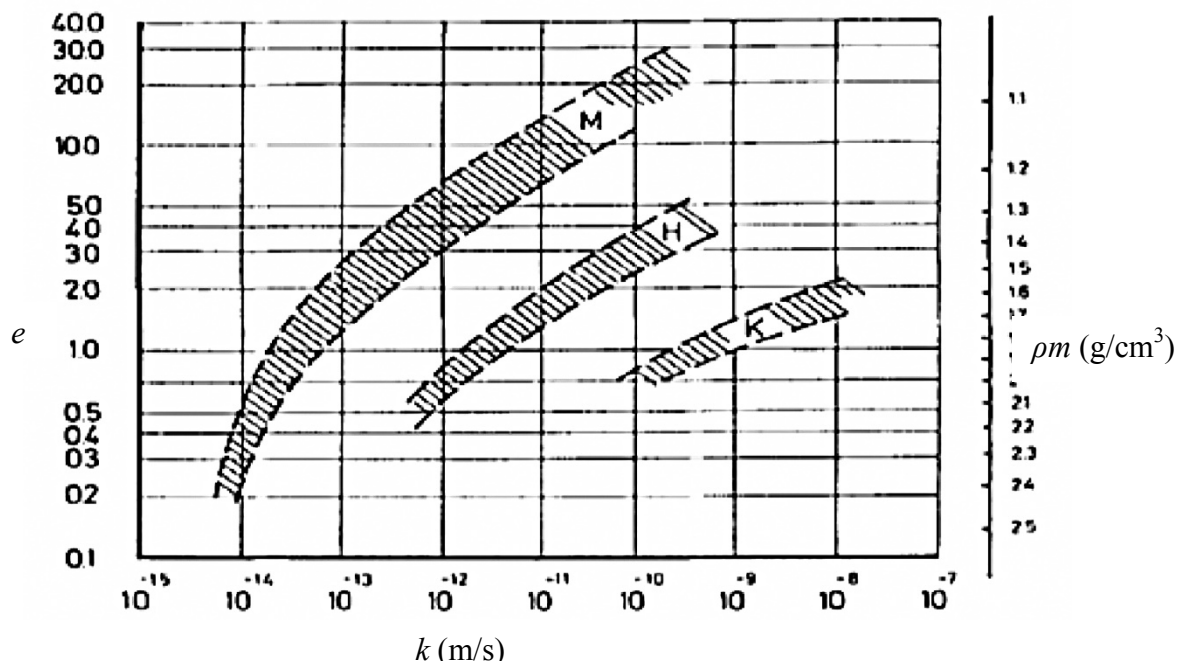


Figure 2.2 Comparison between montmorillonite (M), hydrous mica ("illite") (H), and kaolinite (K) with regards to hydraulic conductivity and void ratio (After Pusch 1992)

The efficiency of sand-bentonite mixtures used as barriers/liners in the hydraulic and waste containments should have some requirements. These requirements depend mainly on the hydraulic and mechanical characteristics of sand-bentonite mixtures. According to Gueddouda et al. (2008) who cited Chapuis (1990), Parker et al. (1993), and Thériault (2000), the requirements are mainly as follows:

- The typical thickness for sand-bentonite barriers should range between (15-30) cm.
- The range of permeability at saturated condition should be between (10^{-6} - 10^{-8}) cm/s.
- The exchange and adsorption properties are able to prevent some preferentially pollutants.
- The physical stability cannot be effected by water in the wet condition.
- Good ability to swell and interact with host rock to fill the cracks.
- The particle-size distribution of sand in the mixture should ensure the hydraulic stability and form the mixture skeleton.

2.3 Characteristics of Sand-Bentonite Mixtures

2.3.1 Compaction characteristics

Compaction of soil can be defined as the method of increasing soil density by applying a mechanical energy to reduce the voids between the soil particles. There was a significant amount of data presented on compaction of soil-bentonite mixture. These data examined the effect of bentonite content, curing periods, compactive efforts, and mixing procedures on maximum dry density (or weight) and optimum moisture content described in the following literature.

Kenney et al. (1992) carried out a series of standard compaction tests on bentonite–sand mixtures containing a bentonite content of 4%, 8%, 12%, 16%, and 22% considering using freshwater in the tests. Kenney et al. also examined using two mixing methods for the materials. The first method was mixing sand and bentonite in a dry condition before adding water. The second method was mixing wet sand with dry bentonite followed by adding more water. They stated that the addition of bentonite up to 20% caused an increase in the values of maximum dry density and a decrease in the maximum dry density. The value of maximum dry density were estimated to be 1.70 to 1.85 Mg/m³; while the corresponding values of optimum water content were from about 12% to 15%. Kenney et al. (1992) found that both of the mixing methods led to the same results.

Howell et al. (1997) examined the effects of type of processed clay soil, curing period, and mixing procedure on compaction behavior of sand-attapulgite clay (S-AC), sand-granular bentonite (S-GB), sand-powdery bentonite (S-PB), and sand-attapulgite clay-granular bentonite (S-AC-GB) mixtures. The percentages of total clay soil contents used for the mixtures were 10, 15, and 20%. Two curing periods were used, one day and seven days. Two mixing procedures were adopted (1) mixing dry sand with bentonite before adding water and (2) mixing dry sand with water before adding bentonite. The results of the compaction tests indicated that different trends in terms of the relationships between with increasing bentonite content as shown in Table 2.1 and Figure 2.3(a, b). According to the Howell et al. (1997), the reasons for that were: (1) attapulgite clay has the greater water sorptivity and lower swelling potential compared with other two clays, (2) the granular bentonite has larger particle sizes compared with powdery bentonite. Howell et al. (1997) indicated also that there was a small effect of the two curing periods on the maximum dry unit weight and optimum moisture content. Finally, they found that the first mixing procedure produced maximum dry unit weight and optimum moisture content greater than the second mixing procedure.

Table 2.1 Change in maximum dry unit weight and optimum moisture content with respect to increasing bentonite content for different sand-clay mixtures presented by Howell et al. (1997)

Type of mixture	Maximum dry unit weight with increasing bentonite	Optimum moisture content with increasing bentonite
Sand-attapulgite clay (S-AC)	Decreased	Increased
sand-powdery bentonite (S-PB)	Increased	Decreased
sand-granular bentonite (S-GB)	Relatively constant	Decreased

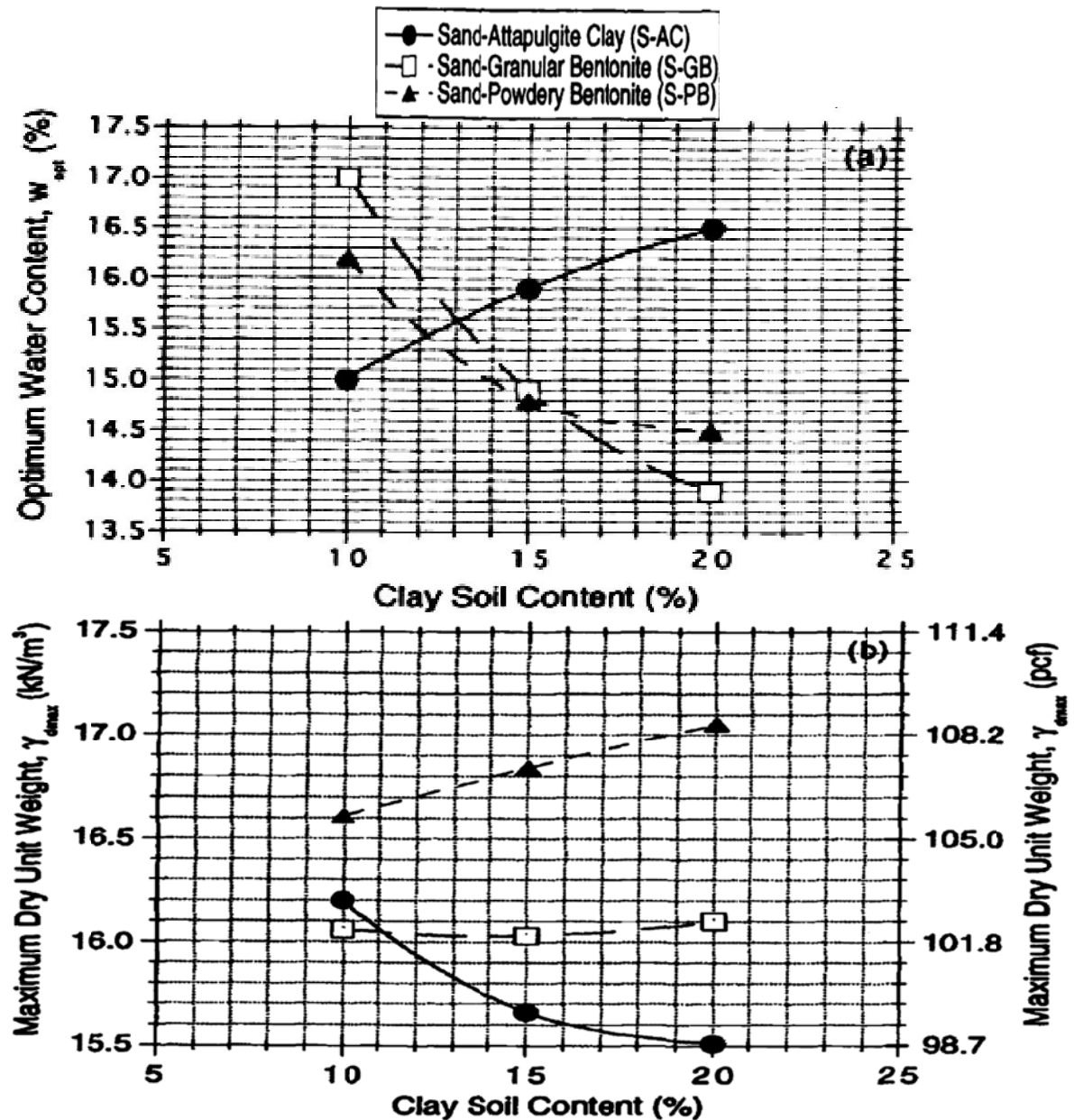


Figure 2.3 Influence of clay soil type and content on (a) optimum moisture content and (b) maximum dry unit weight versus clay soil content (After Howell, 1997)

Komine and Ogata (1999) conducted a series of standard compaction tests on several bentonite–sand mixtures. The bentonite contents in the mixtures were 5%, 10%, 20%, 30%, and 50%. The results of maximum dry density of these mixtures were found to be 1.61, 1.64, 1.68, 1.72, and 1.66 Mg/m³, respectively. The corresponding values of optimum water content were found to be 19.4, 17.6, 17.0, 14.6, and 17.5, respectively. From these results, it can be noticed that as bentonite content increases up to 30%, the maximum dry density increases and optimum water content decreases.

Chalermyanont and Arrykul (2005) performed standard compaction tests on 0, 3, 5, 7, and 9% bentonite-sand mixtures (Figure 2.4). They indicate that as bentonite content increases, the values of maximum dry density decreases and optimum moisture content increases. The values of maximum dry unit weight were 19.47, 19.35, 19.10, 18.68, and 18.56 kN/m³ and the corresponding optimum water contents were 9, 10, 10.5, 11.2, and 12%, respectively.

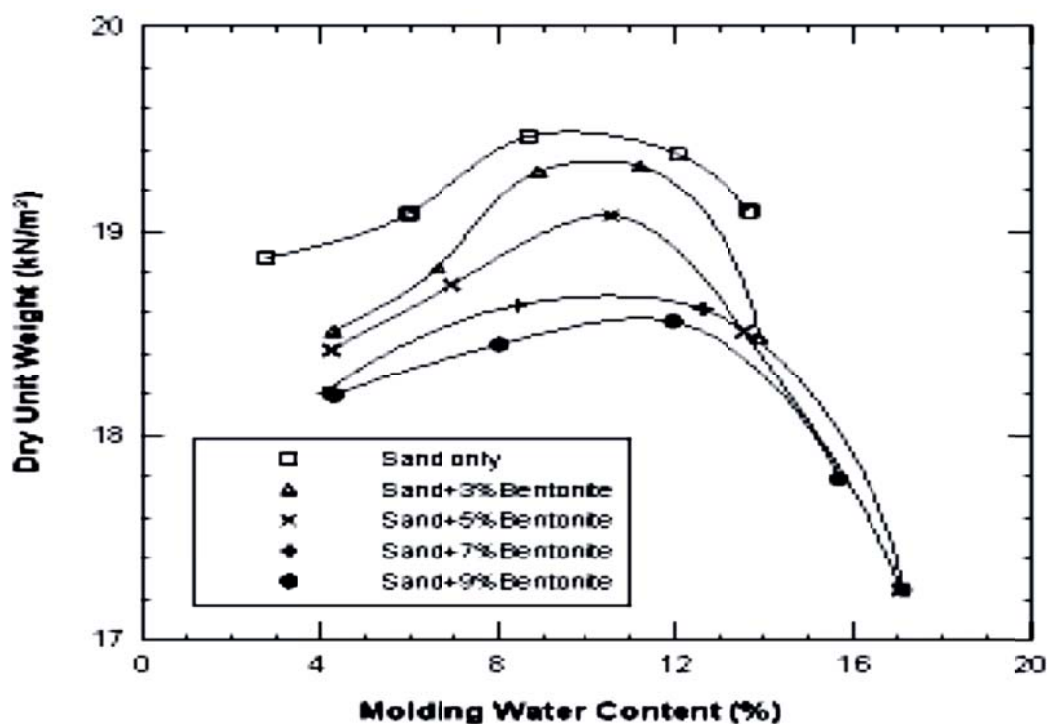


Figure 2.4 Compaction curve reported by Chalermyanont and Arrykul (2005)

Akgün et al. (2006) conducted compaction tests on three sand/bentonite mixtures consisting of 15, 17.5, and 20% bentonite content. The results of the compaction tests are shown in Table 2.2. It is clear that as bentonite increases, the maximum dry unit weight decreases and optimum moisture content increases.

Table 2.2 Results of the compaction tests reported by Akgün et al. (2006)

Bentonite content %	Maximum dry unit weight (kN/m ³)	Optimum moisture content %
15	16.01	15.82
17.5	15.41	16.24
20	15.10	16.29

In terms of examining compactive efforts, Amadi and Eberemu (2012) performed compaction tests using four different compactive efforts on mixtures containing 0, 2.5, 5, 7.5, and 10% bentonite and lateritic soil. The compactive efforts applied were; the reduced British Standard Light (RBSL), British Standard Light (BSL), West African Standard (WAS), and British Standard Heavy (BSH). The moisture contents used in the mixtures were dry of optimum, optimum, and wet of optimum. The results of all the compactive efforts indicated that the maximum dry unit weight decreased and the optimum moisture content increased with the increasing bentonite content as shown in Figures 2.5 and 2.6, respectively.

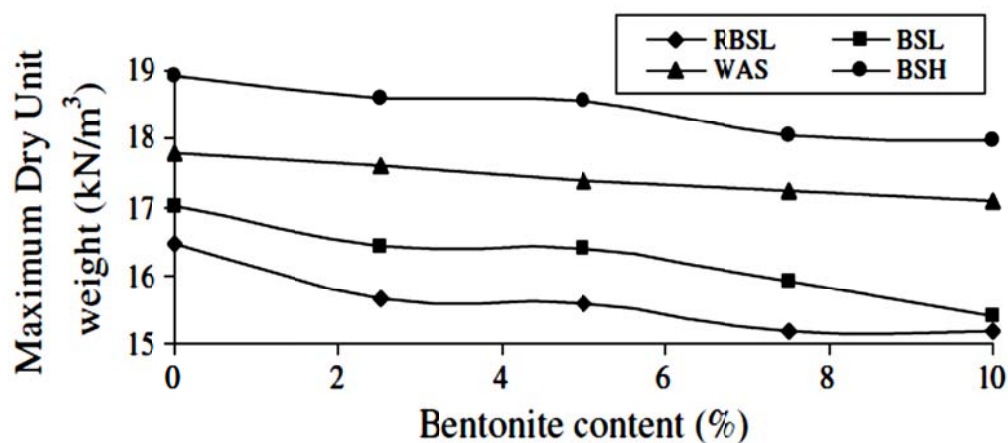


Figure 2.5 Variation of maximum dry unit weight versus bentonite content (After Amadi and Eberemu, 2012)

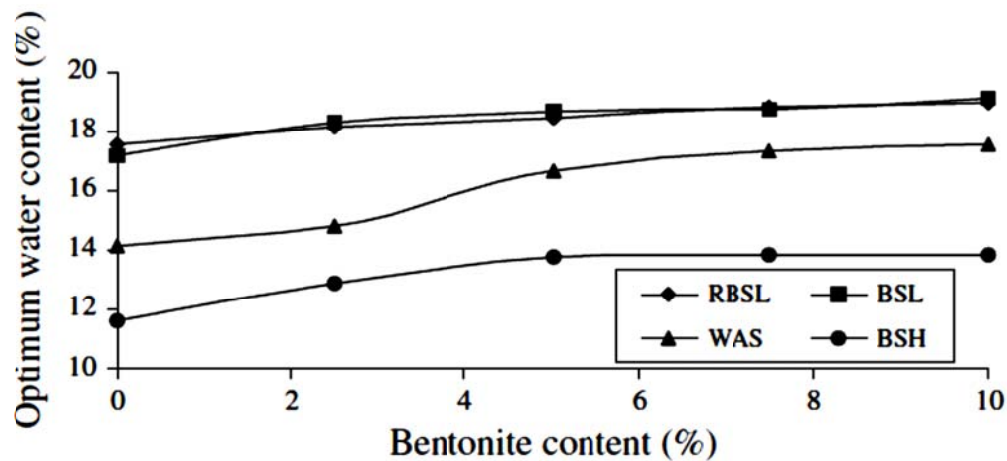


Figure 2.6 Variation of optimum moisture content versus bentonite content (After Amadi and Eberemu, 2012)

2.3.2 Compressibility Characteristics

Generally, the compressibility of a soil is the change in void ratio of the soil as a result of applying a load. The soil compressibility includes three different stages. These stages are initial compression, primary consolidation and secondary compression as seen in Figure 2.7 (Das 2013). The initial compression occurs immediately after a new load is applied but before drainage caused by the compression of gas inside the voids is started and also by the elastic compression of soil particles. The primary compression is time dependent settlement caused by expulsion of pore water which leads to transfer of excess pore water pressure into effective stress. The secondary compression is also time dependent settlement which caused by the plastic readjustment of soil fabric and occurs after the dissipation activity of the excess pore water pressure. The time required for water dissipation from the soil during consolidation process depends on the permeability of soils. In cohesionless soils, which have high permeability, the time required for water dissipation is less than that for cohesive soils.

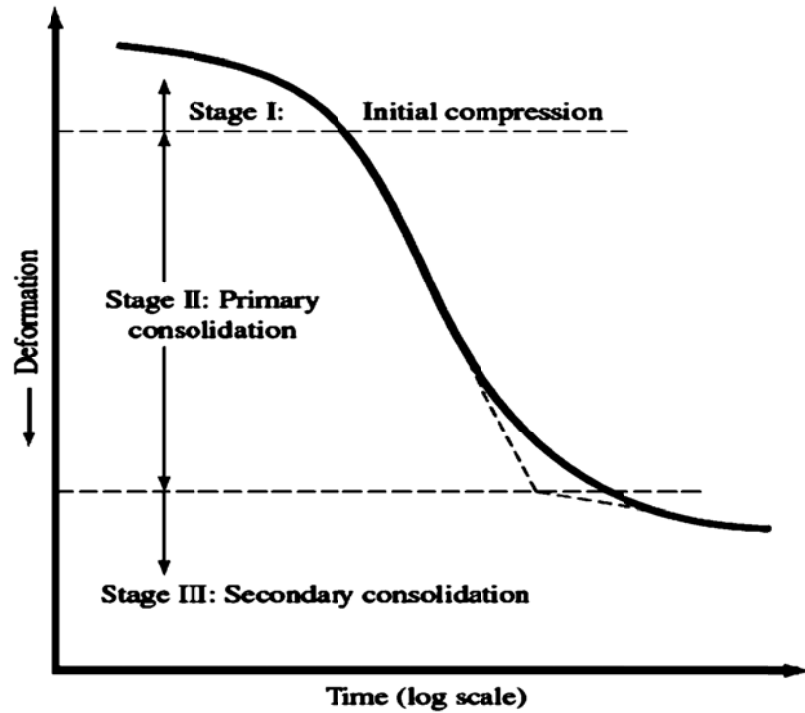


Figure 2.7 Time-deformation curve during consolidation for a given load (After Das 2013)

The compression of sand-bentonite mixture was investigated in different studies. Baxter (2000) cited D'Appolonia (1980) who plotted the relationship between the compression ratio and fine content of several soil-bentonite mixtures as shown in the following formula:

$$\text{Compression ratio} = \frac{C_c}{1 + e_o} \quad (2.1)$$

where: C_c = compression index

e_o = initial void ratio

Baxter (2000) cited D'Appolonia (1980) who has conducted one dimensional compression and isotropic compression tests on different soil-bentonite mixtures. The compression ratio was measured in the stress range of 1000 to 4000 psf. In the results it was indicated that the compressibility increases with increasing fines content as shown in Figure 2.8. It was also indicated that the compression ratio of the soil-bentonite which had 20% to 40% fines was between 0.02 and 0.07 for the stated stress range. Further, the compression ratio obtained from one-dimensional compression was higher than that obtained from isotropic compression. Additionally, the soil-bentonite mixtures having plastic fines were noticed to be more compressible than soil-bentonite mixtures having non-plastic fines.

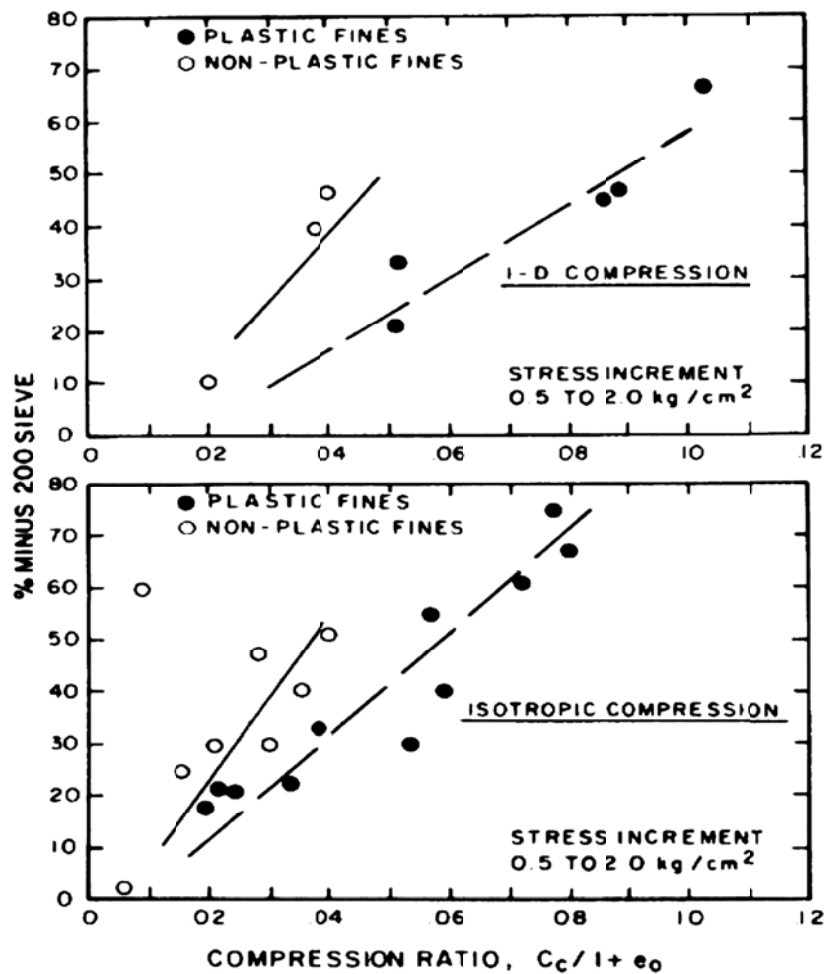


Figure 2.8 Compression ratios versus fines content for different soil-bentonite mixtures (After D'Appolonia, 2000)

Watabe et al. (2011) investigated the effect of sand and bentonite fractions on compressibility property of their mixtures. They performed a series of incremental loading oedometer tests and microscopic observations on different soils (sand-clay and sand-bentonite). They reported that the compressibility decreases by increasing the sand fraction when the sand particles are floating in the mixture. However, when the skeleton structure was formed by sand particles with a large sand fraction, the compressibility remained almost constant with increasing the additive fraction of sand.

Fan et al. (2014) conducted a series of one-dimensional consolidation tests for clayey soil (Kaolin)-bentonite and sand-bentonite mixtures to be used as a backfill in vertical cutoff walls. The aim of this study was to find out the effect of sand fraction and moisture content on the compressibility of soil-bentonite mixtures. The grain size of the sand was in the range of 0.075-1 mm. The bentonite contents used in the soil (Kaolin)-bentonite mixtures and sand-bentonite

mixtures were (5, 10, and 15%) and (3, 5, 8, and 11%), respectively. They reported that the compressibility were critically affected by the bentonite content and also by moisture content. However, there was insignificant effect of grain size of sand on compressibility.

2.3.3 Permeability Characteristics

Permeability is the dominant property in the design of liners/barriers, such as sand-bentonite mixtures in waste and hydraulic containments. Characteristics of the permeability of sand-bentonite mixtures were studied in several literatures. In these literatures, attempts were made to examine the permeability of sand-bentonite mixtures and also to investigate some factors which may affect the permeability. These factors were: type of bentonite (Mitchell and Soga 1976), bentonite content (Gueddouda et al. 2008; Chapuis 1990; Chalermyanont and Arrykul 2005), permeant type (Studds et al. 1998; Kenney et al. 1992), void ratio (Abeele 1986), compaction water content (Kenney et al. 1992), degree of saturation (Chapuis 1990), swelling behaviour (Studds et al. 1998; Shirazi et al. 2010; Komine 2008), size particle distribution (Sivapullaiah et al. 2000; Chapuis 1990). There were also some attempts to create models for predicting the permeability of sand-bentonite mixtures in order to avoid a considerable time and cost resulting from conducting the permeability tests.

Gleason et al. (1997) compared the effects of changes in bentonite type on the permeability of compacted sand-bentonite mixtures. Two types of air-dry bentonite were selected in the study, namely Sodium bentonite and Calcium bentonite. The Sodium bentonite was provided by Bentonite Corp. of Denver, Colo., while Calcium bentonite was provided by Vulcan materials Co., San Antonio, Tex. The Sodium bentonite had approximately 90% retained on the sieve No. 40 (U.S. Standard); while Calcium bentonite had approximately 90% passing the sieve No. 100 (U.S. Standard). The sand used were three groups, obtained from three locations. The three groups of sand were group A which was a medium, uniform, Ottawa sand sourced from Clemtex, Inc., Houston, Tex., group B which was a broadly graded sand provided by Vulcan, and group C which was a silty sand obtained from east Texas. The classifications of the three sands were SP, SW, and SM, respectively. The mixtures consisted of different contents of Sodium and Calcium bentonite and three sands. The percentages of added bentonite were 6, 12, 20, and 30%. The permeability tests were conducted on samples using compaction mould and then permeated with the permeant liquids. Two groups of permeability tests were conducted: (1) specimens permeated with tap water and then with 0.25 M Calcium chloride (CaCl_2); and (2) specimens permeated with only 0.25 M Calcium chloride (CaCl_2). The results were calculated

using Darcy's Law. The results of permeability indicated that the amount of calcium bentonite needed was three times more than sodium bentonite in order to decrease the permeability to less than 1×10^{-7} cm/s (Figure 2.9). They indicated also that the specimens which permeated with 0.25 M Calcium chloride (CaCl_2) had higher permeability than the specimens which permeated with tap water and then with 0.25 M Calcium chloride (CaCl_2)

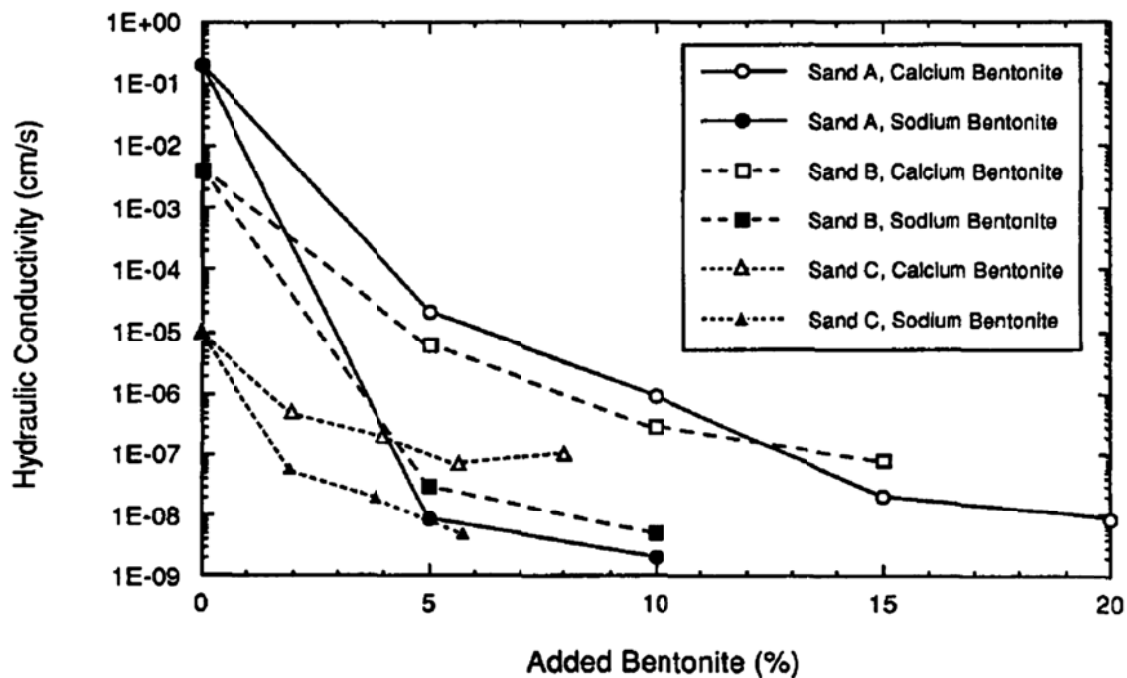


Figure 2.9 Hydraulic conductivity versus added bentonite for Sodium and Calcium sand-bentonite mixtures (After Gleason et al., 1997)

Borgesson et al. (2002) carried out several permeability tests on mixtures of 0–50% bentonite content with different ballast materials (crushed rocks). They indicated that the permeability of the bentonite with ballast materials mixtures was higher than the permeability of the bentonite alone. According to Borgesson et al. (2002), the reason for the strange data was uneven distribution of bentonite in the mixtures.

Sällfors, and Öberg-Högsta (2002) examined the permeability of sand-bentonite mixtures with regard to bentonite content, compaction, and degree of saturation in order to predict the permeability in early stages of design. Several falling head tests in a triaxial cell were conducted on the sand-bentonite mixtures, using gradient ranging from 10-20, confining pressure of 20 kPa, and no back pressure. The materials used were a medium sand, a coarse sand, and a high swelling Sodium bentonite. In the results, a new parameter k_1 was proposed, given in Equation (2.2), which represented the amount of bentonite content per pore volume. Sällfors, and Öberg-

Högsta (2002) indicted at $k_1 < 0.5$, bentonite content is less than 12%, and at $k_1 > 0.5$, bentonite content is rather high; however, these results were valid for fully saturated homogeneous mixture.

$$k_1 = \frac{B_e \rho_d^{mixture}}{(1 + B_e) - \frac{\rho_d^{mixture}}{G_s^{sand}}} \quad (2.2)$$

where:

k_1 = proposed parameter by Sällfors, and Öberg-Högsta

B_e = percent bentonite ($m_s^{bentonite} / m_s^{sand}$)

$\rho_d^{mixture}$ = dry density of the mixture

$m_s^{bentonite}$ = solid mass of bentonite

m_s^{sand} = solid mass of sand

Komine (2010) experimentally investigated the changes in the permeability of sand-bentonite mixtures before and after swelling activity of sand-bentonite mixture with different content of bentonite (10%, 20%, 30%, and, 50%) and then the results were evaluated and compared with the results obtained from theoretical equations which were already developed by Komine (2008). Komine (2010) suggested that the equations can be an applicable model for predicting the permeability of sand-bentonite mixtures. However, the model has some limitations because it had many chemical parameters which were estimated using sophisticated procedures and equipment (Tripathi, 2013).

Fan et al. (2014) conducted a series of one-dimensional consolidation tests on clayey soil (Kaolin)-bentonite and sand-bentonite mixtures in order to find out the influences of sand fraction and moisture content on the permeability property. The range of the grain size of the sand used was 0.075-1 mm. The bentonite contents used in the soil (Kaolin)-bentonite mixtures and sand-bentonite mixtures were (5, 10, and 15%) and (3, 5, 8, and 11%), respectively. They indicated that the permeability was greatly controlled by the bentonite content.

There were also some attempts to predict the permeability of sand-bentonite mixtures by different researchers. Kenney et al. (1992) presented a model in which sand-bentonite mixture was assumed to be ideal. They conducted falling-head permeability tests using a rigid wall consolidometer apparatus. The samples used were sand having hydraulic conductivity of 10^{-2}

cm/s, sodium bentonite of up to 20% having high swelling property, two types of permeant: distilled water and saline solution. The aim of using two permeants in the tests was to examine two situations of swelling that can be happened: high-swell bentonite and low-swell bentonite respectively. They stated that for the sand-bentonite mixtures having bentonite content of up to 20%, the materials create a barrier to prevent the seepage and the specific role of each material include: sand for stability and bentonite for filling the voids. Also, they stated that in order to gain a proper distribution and a suitable compaction, the best water content of the mixture should be equal or more than the optimum water content (Figure 2.11).

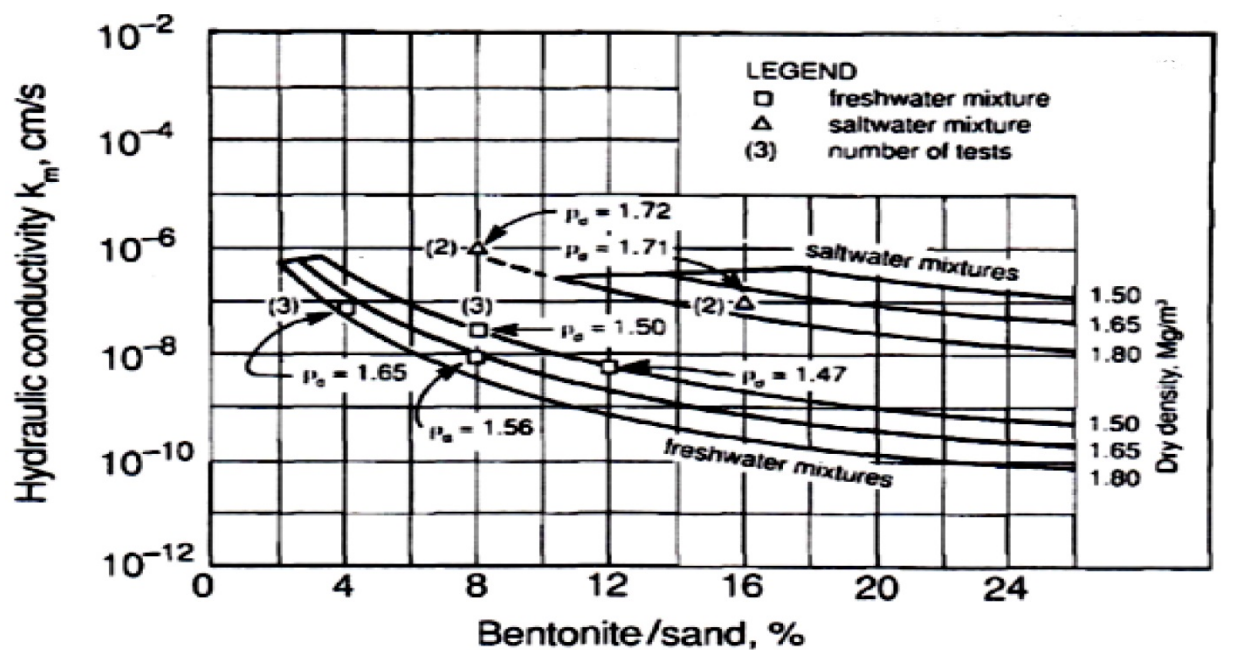


Figure 2.10 Hydraulic conductivities of sand-bentonite with different permeants (After Kenney, 1992)

Mollins et al. (1996) conducted four different methods for estimating the permeability of sand-bentonite mixtures. These methods were Rowe cell constant head tests, falling head tests, standard compaction permeameter, and consolidation tests. Distilled water has been used as a permeant in the tests. The materials selected were Conquest grade Wyoming bentonite and Knapton Quarry sand (silty fine angular quartz sand). The mixtures used having bentonite contents of 5, 10 and 20% by dry weight. In the results, they indicated that there was a linear relationship between the void ratio and logarithm of vertical effective stress for different bentonite contents at a particular effective stress (Figure 2.11). They also indicated that using very low bentonite content causes uneven distribution of bentonite in the mixture.

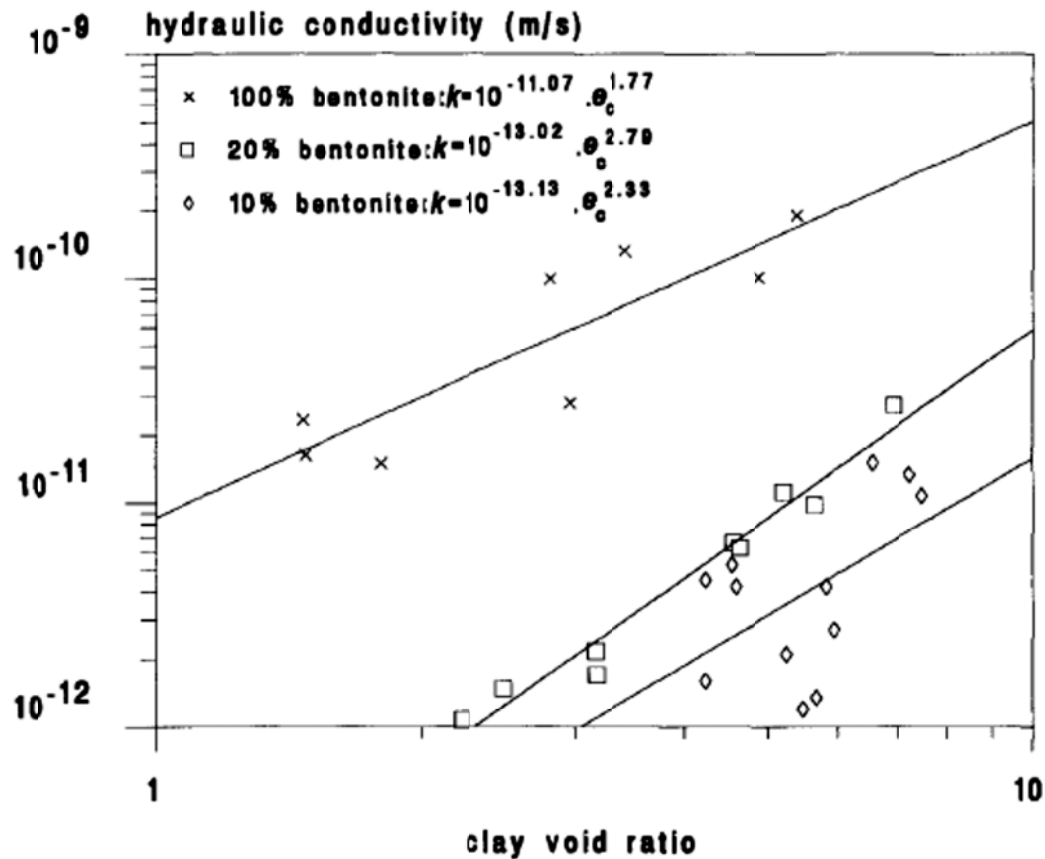


Figure 2.11 Hydraulic conductivity versus clay void ratio of sand-bentonite mixtures (After Mollins et al., 1996)

These results can be used for other similar mixtures after estimating the swelling behaviour for the materials of that mixture. However, this model is applicable in case of an ideal sand-bentonite mixture. The term (ideal sand-bentonite mixture) referred to the case in which bentonite particles hydrate uniformly and fill all the voids between sand particles in the mixtures (Tripathi, 2013).

In terms of examining the effects of coarse particles content on the permeability of fine grained soil, several research studies were presented. Mollins (1996) cited relevant literature describing using various mixtures having different coarse and fine contents in order to explain the effects (Table 2.3).

Table 2.3 Effects of coarse particles on the permeability of fine grained soil (Mollins, 1996)

Author (year)	Type of tested mixtures	coarse particles content	Outcome
Jones (1954)	Sand-gravel mixtures	Gravel content <65%	Less permeability comparing with sand alone
Holtz and Lowitz (1957)	Gravel-clay mixtures	gravel content \geq two-thirds of the total amount	Affecting filling all the voids in the mixtures
Dixon et al. (1985)	Sand-bentonite mixtures	Increasing sand up to 50%	Reducing the effective porosity available for flow
Daniel (1990)	Clay-gravel mixtures	Gravel content <10-20%	A suitable ratio for liner materials
Shakoor and Cook (1990)	Silty clay-gravel mixtures	gravel content >50%	Increasing permeability significantly
Chapuis (1990)	Sand-bentonite mixtures	***	Increasing total fine content leading to a decrease in the permeability of sand

Sivapullaiah et al. (2000) also investigated the effects of the coarser fraction size on the permeability of sand-bentonite mixtures. For calculating permeability, Sivapullaiah et al. (2000) carried out a series of consolidation tests on several sand-bentonite mixtures. The materials used were bentonite, two silt, two fine sand, and two coarse sand. The bentonite was obtained from Kolar district in Karnataka State, India. Two groups of sands were obtained from locally available river sand which was washed and sieved into two groups; (1) fine sand (rounded) with a range of 0.425-212 mm; and (2) coarse sand (rounded) with a range of 1.4-1 mm. In the same way, another two groups were obtained from the quarry dust to produce (1) fine sand (angular) with a range of 0.425-212 mm, coarse sand (angular) with a range of 1.4-1 mm. The last two groups were silt 1 obtained from quarry dust and silt 2 from Kaolinitic clay. These mixtures were consisted of several percentages of bentonite ranged from 0% to 100% mixed with one type of

the sands. They indicated that for any void ratio, increasing the size of the coarser fraction leads to an increase in the permeability of these mixtures. Sivapullaiah et al. (2000) also indicated that for any type of mixture, the logarithm of hydraulic conductivity changed linearly with the void ratio. They produced four methods to estimate the permeability of the sand-bentonite mixture, these equations had statistical correlations and were valid in case of using water as a permeant (Tripathi, 2013).

Watabe et al. (2011) investigated the effect of sand-bentonite fractions on permeability by conducting several incremental loading oedometer tests on different soils (sand/clay and sand-bentonite). It was indicated that there is no change in the permeability in case of sand particles were independent in the clay matrix. However, the permeability significantly increased with increasing the additive fraction of sand when the skeleton structure was formed by sand particles with a large sand fraction.

2.3.4 Strength Characteristics

In recent decades, mixing sand with an adequate amount of active clay/bentonite has become a common practice for providing mixtures as construction materials for geoenvironmental engineering applications, such as hydraulic barriers in landfills (Alther 1982) and cutoff walls (D'Appolonia 1980) cores in earth dams (Alkaya and Esener 2011), and buffer and backfill materials in radioactive nuclear waste containments (Dixon et al. 1985). The main advantages of sand-bentonite mixtures are low permeability and high mechanical stability for its applications despite the difference in particle-size distribution and chemical activity in these materials. The mechanical behaviour of sand-bentonite mixtures has become an important research topic in geotechnical engineering because of the need for long-term integrity structures.

The strength behaviour of clean sand was investigated first by Coulomb in the 18th century (Das 1983); while the strength behaviour of pure clays was investigated approximately 150 years later (Wasti and Alyanak 1968). The strength of sand-clay mixtures has been examined in the literature considering different characteristics. Miller and Sowers (1957) studied the effect of clay and sand contents on the shear strength of sand-clay mixtures by performing unconsolidated undrained triaxial tests. Wasti and Alyanak (1968) produced a relationship between Atterberg limits and clay content of sand-clay mixtures; they showed that the behaviour of the mixtures changes from sand to clay when there are sufficient clay particles to fill the voids. Cho et al. (2002) reviewed some literatures related to investigating the unconfined compressive strength and Young's modulus of elasticity of compacted sand-bentonite mixtures

as a function of sand content, dry density, and water content. The mixtures were tested to be used as buffer materials in high-level radioactive waste repositories. They found that the unconfined compressive strength and Young's modulus decreased as the sand content increased. They also noted that the logarithm of compressive strength and Young's modulus increased linearly with the increase in dry density. Chalermyanont and Arrykul (2005) conducted direct shear tests on sand-bentonite mixtures having bentonite content of 0, 3, 5, 7, and 9% in order to estimate its shear strength parameters (i.e., friction angle and cohesion). They stated that the friction angle of the sand-bentonite mixtures decreased and cohesion increased as the bentonite content increased. Ölmez (2008) examined the shear strength properties and stress-strain characteristics of sand-kaolin mixtures, and found that these properties and characteristics changed remarkably at a kaolin content of 20%. Gueddouda et al. (2008) conducted an unconsolidated undrained direct shear test on saturated and unsaturated dune sand-bentonite mixtures with bentonite contents of 3, 5, 10, 12, and 15%. They noticed that, contrary to the case of the internal friction, the values of cohesion in the unsaturated case exceeded the values in the saturated case. Chen and Meehan (2011) carried out a series of unconsolidated-undrained triaxial tests on remoulded sand-bentonite mixtures (bentonite content 15, 25 and 50% by dry weight) using three compactive efforts. They investigated the influence of bentonite/sand mix proportion, compaction energy, compaction moisture content, and confining pressure on the stress-strain and shear strength behaviour of compacted sand-bentonite mixtures. Chen and Meehan (2011) found that at the same bentonite content, the optimum water content decreased and the dry unit weight increased with increasing the compaction energy, while at the same energy level, the optimum water content increased and the dry unit weight decreased with increasing the bentonite content in the mixture. They also found that the undrained strength increased as the compactive effort and confining pressure were increased and decreased as the water content was increased. Pakbaz and Moqaddam (2012) investigated the effect of clay content and sand gradation on the shear strength properties and the overconsolidation ratio, OCR exponent (m) of over-consolidated sand-clay mixtures. The sieves ranges of sand gradation numbers were 10-200, 30-200, and 50-200, while the clay content were 15, 20, 30 and 40%. They stated that at a particular sand gradation, the shear strength and m decreased with the increase of clay content; however, at particular clay content, as the sand gradation decreased, the shear strength and m also decreased. Elkady et al. (2014) conducted direct shear test on the compacted sand-attapulgitic clay mixtures and cement (bentonite range of 0-60%) in order to find the most economical mixture to be used for clay core of earth fill dams and liners of solid waste containments. The main investigation was to determine the influence of clay content,

initial moulding conditions, normal stress, and wetting conditions on the shear strength behaviour. They indicated that the sand-clay mixture which included bentonite of 10% and sand of 90% was the most economical mixture. The literature mentioned described the importance of optimising the sand-bentonite mixture and investigating its strength characteristics, for structure integrity; however, the outcomes may depend on the type of sand (gradation) and bentonite used.

In the states like Western Australia which predominantly have sandy soils, engineers face cost and physical integrity challenges in geoenvironmental projects. Investigating the strength behaviour of combinations of three types of local sands with different amounts of bentonite should assist with these challenges. Such findings have rarely been reported for sandy soils in this state. This study aims to provide a comprehensive understanding of the changes in strength characteristics with regards to bentonite content and sand gradation.

2.4 Conclusions

Design and investigating the behaviour of compacted sand/bentonite mixture in some geoenvironmental applications need to examine the following properties: permeability, compressibility, and strength of these mixtures, which require a considerable time and effort. Therefore, many researchers studied this mixture taking into account some factors that have a considerable effect on the economical sand-bentonite mixture with low permeability and appropriate strength. These factors are the bentonite content, bentonite type, particle-size distribution, and permeated liquid. In this study, the permeability, compressibility, and strength properties of mixtures consisting of bentonite and three types of sands have been examined. Further, the effect of two factors, namely particle-size distribution and bentonite content on these properties have been considered.

CHAPTER THREE

MATERIALS AND METHODS

3.1 General

The materials used in this study were three local Perth sands and sodium powder bentonite. A series of tests were carried out on these materials namely: standard Proctor compaction test, one-dimensional consolidation tests, unconfined compressive strength test, and direct shear test. In this chapter, a brief description about these materials, their properties and the tests are presented.

3.2 Materials

3.2.1 Sands

The sands used in this study were obtained from regional area around Perth, the state's capital city and major population centre. Three types of sands were used, namely brickies, river, and plaster sands as shown in Figure 3.1. Brickies sand was sourced from a quarry site about 40 km north of Perth region, and used for different construction projects. Plaster sand was sourced from Carramar city, which is located at 30 km north of Perth, and commonly used for rendering and paving work. River sand was provided from Mundaring city, which is located in 34 km east of Perth. All types of sands were classified as poorly graded sand (SP) according to the unified soil classification system (USCS). The physical properties of sands are reported in Table 3.1. The Percentages of fine, medium and coarse gradation of the three types of sands are reported in Table 3.2. The Particle-size distributions of sands are shown in Figure 3.2.



Figure 3.1 The soils used in the study

Table 3.1 Physical properties of sands

Property	Types of sand		
	River sand	Plaster sand	Brickies sand
Permeability (As compacted samples) (m/s)	$9.67E^{-07}$	$1.11E^{-07}$	$1.09E^{-07}$
Specific gravity	2.65	2.65	2.65
γ_{dmin} (kN/m ³)	13.6	15.3	14.3
γ_{dmax} (kN/m ³)	15.8	17.5	17.3
e_{min}	0.53	0.51	0.95
e_{max}	0.86	0.74	0.68
Effective Diameter (D_{10}) (mm)	0.24	0.23	0.17
D_{30} (mm)	0.44	0.33	0.32
D_{60} (mm)	0.74	0.53	0.46
Coefficient of uniformity	3.08	2.30	2.71
Coefficient of curvature	1.09	0.89	1.31
Classification as per the USCS	SP	SP	SP

Table 3.2 Percentage of fine, medium and coarse gradation of the three types of sands used in the study according to the Australian standard AS 1726-1993

Gradation of sand	River sand %	Plaster sand %	Brickies sand %
Coarse sand (0.600-2.36) mm	50	29	18
Medium sand (0.212-0.600) mm	45	67	52
Fine sand (0.075-0.212) mm	4.5	3.5	25
Fines <0.075 mm	0.5	0.5	5

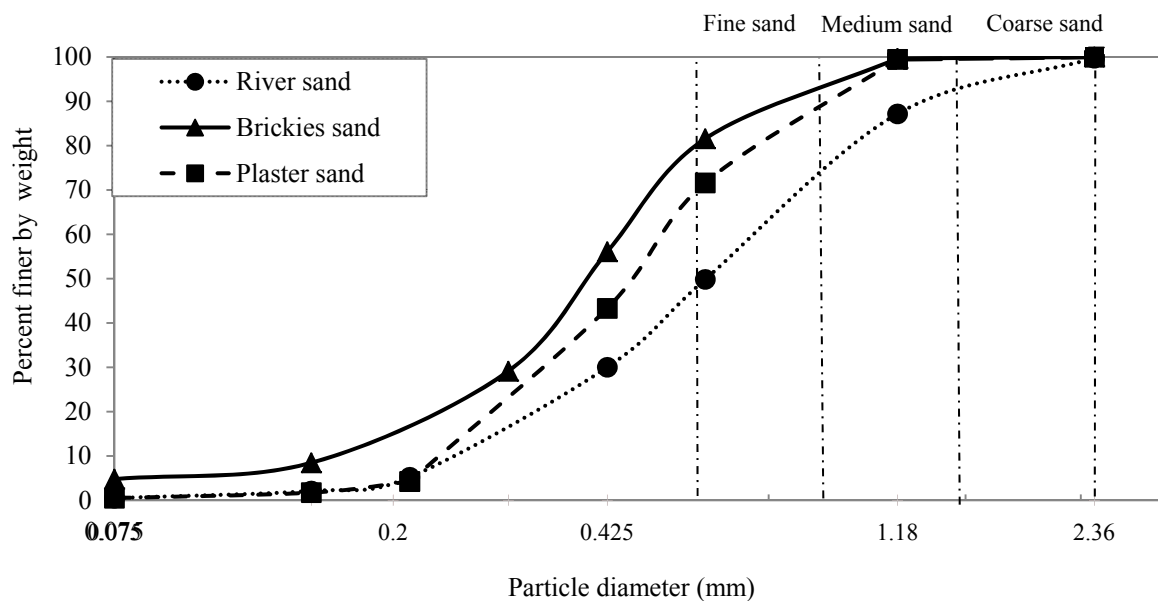


Figure 3.2 Particle-size distributions of sands

3.2.2 Bentonite

The bentonite used in this study was a powdered sodium bentonite, called the Ebenezer supplied by Bentonite Products Pty Ltd from the Ebenezer mine site in Queensland, Australia, as shown in Figure 3.1. The bentonite consisted of 81% montmorillonite with other components such as silica, feldspar and carbonates. The particle-size distribution of bentonite was estimated using hydrometer test as per (AS 1289.3.6.3-2003) as shown in Figure 3.3. From the test, it was indicated that more than 85% of bentonite particles are less than 75 μm . The values of the specific gravity, liquid limit, and plastic limit of bentonite are 2.67, 310%, and 56%, respectively.

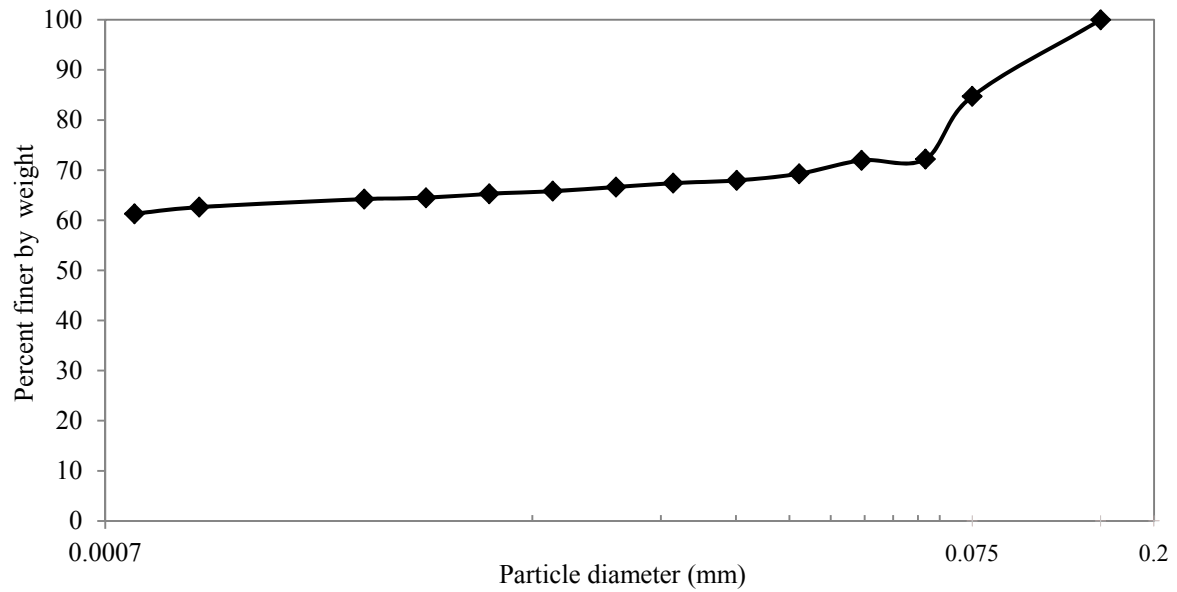


Figure 3.3 Particle-size distribution of bentonite

The compaction curve of bentonite was also evaluated and presented in Figure 3.4. It can be seen that the maximum dry unit weight is 12.18 kN/m^3 and the optimum moisture content is 37.20%.

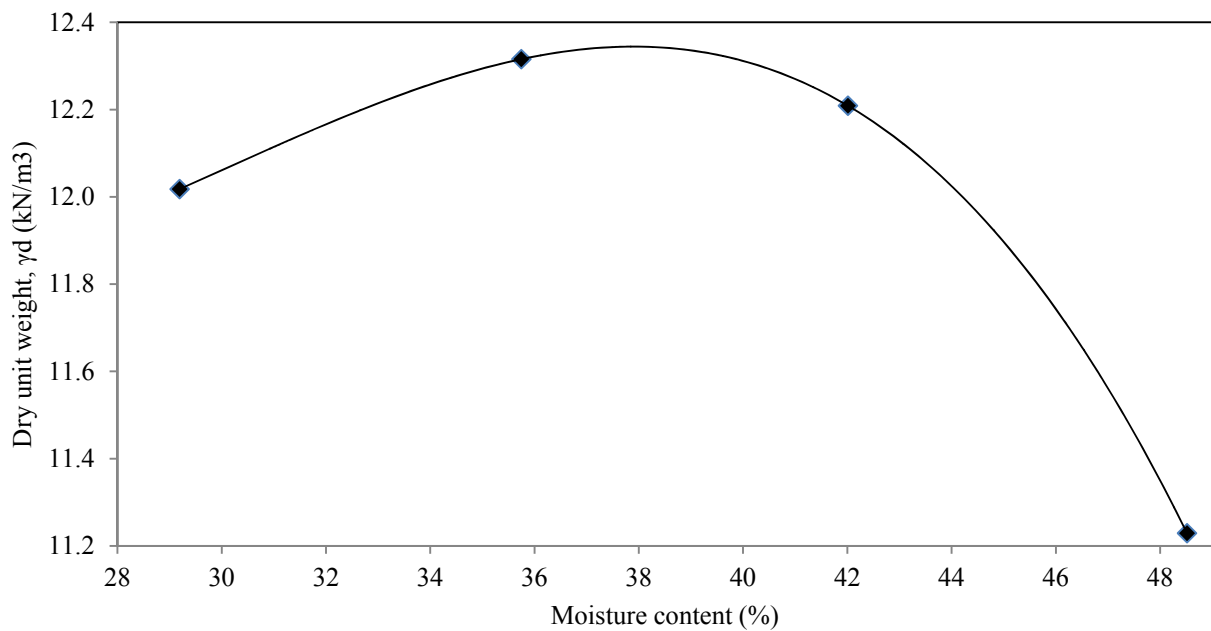


Figure 3.4 Compaction curve of bentonite

3.3 Methods

3.3.1 Basic tests

Several tests were carried out on sands and bentonite to estimate their basic properties according to the Australian standards as listed in AS 1289.0. These tests are: sieve analysis, specific gravity, minimum and maximum relative densities, constant head permeability, hydrometer test, liquid limit (cone penetrometer method) and plastic limit tests.

3.3.2 Standard compaction tests

Compaction tests were conducted for bentonite, three types of sands, and nine sand-bentonite mixtures in order to determine the optimum moisture contents (OMC) and the maximum dry unit weights (γ_{dmax}) for these materials. The tests were carried out using standard Proctor compaction method according to the Australian standard AS 1289.5.1.1. The equipment used in this test consisted of a mould with a diameter of 105 mm and a height of 115 mm, and a 2.5 kg rammer with a drop height of 300 mm. The procedures of the test started with mixing sand and bentonite in a dry condition to ensure homogeneity as suggested by Gleason et al. 1997. Distilled water was then added to the dry soils (sands and mixtures) at different moisture contents and sufficiently mixed to ensure the proper distribution of water into the soil particles before adding the distilled water. After adding the distilled water, the materials were sufficiently mixed to ensure a good distribution of water to all particles. These samples were stored in plastic bags and cured for 48 hours prior to the compaction. After the curing, the samples were compacted in the mould in three layers using 25 blows for each layer. Finally, the moisture contents and dry unit weights were calculated, and then plotted to have the compaction curve, which provides the optimum moisture content and maximum dry unit weight.

3.3.3 One-dimensional consolidation tests

The principle of consolidation test is measuring the settlement rate of a saturated cohesive soil subjected to a constant vertical load due to squeezing out the water from the voids. In this study, the consolidation tests were conducted according to the procedures given in Head (1982). The apparatus used in the test consisted of 75 mm fixed ring consolidation cell, loading frame with 9:1 lever arm, and an automatic dial gauge connected to a computer. The samples were first prepared by compacting them in the same way mentioned in the previous section. The amount of distilled water used for the compaction was 2% wetter than the optimum moisture content in order to achieve efficient compaction (Haug and Wong, 1992) with less permeability (Gleason et

al. 1997). The ring with the cutting edge was attached to the mould and pushed gently to the desired depth. After trimming the soil outside the ring, the ring was assembled to the cell. Porous stones and filter papers were placed in the bottom and top of the ring. The loads applied in the test were 100, 200, 400, 800 kPa and each load step was maintained for 24 hours. After applying the maximum load, the specimens were unloaded to 400 and 200 kPa, respectively. The dial reading and time rate of settlement data of the tested samples were recorded automatically, arranged and plotted in graphs in order to estimate the coefficient of consolidation.

3.3.4 Direct shear tests

Direct shear tests were conducted on the samples in order to estimate shear strength parameters of the compacted soils, namely friction angle (ϕ) and cohesion (c). The test was conducted based on the procedures based on Head (1982), which considered as a quick test, using ShearTrac-II apparatus (Geocomp Company, United States) as shown in Figure 3.5. The water content used for the mixtures were (OMC+2%) (Gleason et al. 1997; Daniel 1994). After the curing procedures, the samples were compacted using the standard Proctor compaction method. The compacted soils were then extruded from the standard mould to create three samples (60 mm length, 60 mm width, 20 mm height), using a sharp cutting tool. These samples were immediately sheared by applying three normal stresses (100, 200, and 300 kPa). A strain rate of 1 mm/min was applied for shearing all the samples.



Figure 3.5 The apparatus of direct shear test used in the study

3.3.5 Unconfined compression tests

Unconfined compression tests were conducted to estimate the compressive strength of the compacted soils as per ASTM (D2166/D2166M-13). The apparatus used was a LoadTrac-II load frame machine (Geocomp Company, United States) as shown in Figure 3.6. The samples were prepared in the same manner as for the compaction test using water contents of OMC+2% for the mixtures. A split mould (height 100.6 mm, diameter 50.5 mm) was selected to prepare the compacted samples. Three samples of each soil were tested using a strain rate of 1 mm/min.

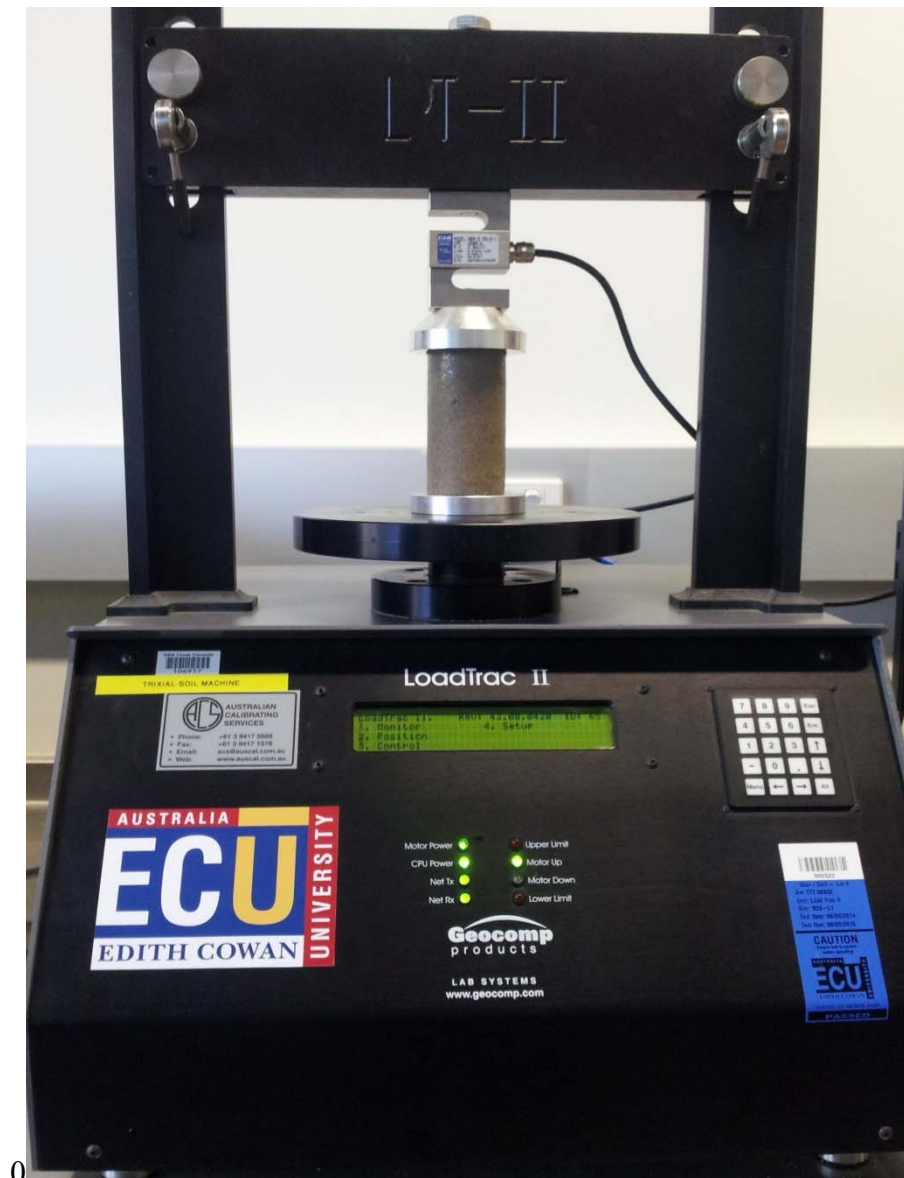


Figure 3.6 The apparatus of unconfined compressive strength test used in the study

3.4 Conclusion

A series of the following tests: standard compaction test, one-dimensional consolidation test, direct shear test, and unconfined compression test was conducted on sand/bentonite mixtures in order to investigate their permeability, compressibility, strength characteristics. The mixtures consisted of three different Perth sands, namely brickies sand, plaster sand, and river sand mixed with 5, 10, 20% of sodium bentonite from Queensland, Australia.

CHAPTER FOUR

PERMEABILITY AND COMPRESSIBILITY CHARACTERISTICS OF SAND-BENTONITE MIXTURES

4.1 General

In states like Western Australia which is covered mostly by sandy soils, the engineers find difficulties in having clays economically for geoenvironmental projects. Any research finding that investigates the strength behaviour of the combinations of three types of Perth sands with different content of powder bentonite can be of significant importance. Such findings are rarely reported in the literature for local Perth sandy soil. The main investigation in this study is to give a full understanding about the trends of the change in the permeability and compressibility characteristics with regards to bentonite content and sand gradation. In this chapter, the effect of bentonite content and sand gradation in the sand-bentonite mixtures on different aspects regarding permeability and compressibility are examined. These aspects are compaction, coefficient of settlement, compression index, and permeability.

4.2 Compaction Test Results

Standard compaction tests were carried out on sands, bentonite, and sand-bentonite mixtures. From these tests, curves of dry unit weight-moisture content relationship were plotted as shown in Figures 4.1, 4.2, and 4.3. The maximum dry unit weight and optimum moisture content of each soil were evaluated by appointing the peak of compaction curves as reported in Table 4.1 and shown in Figures 4.4 and 4.5, respectively. Figure 4.4 reveals that the value of the maximum dry unit weight of all mixtures for any type of sand increases with increasing the amount of added bentonite. This finding is in agreement with the result of the previous studies (Abdelrahman and Shahien 2004, Dixon and Gray 1985, Howell et al. 1997, Kenney et al. 1992, Mollins 1996, Seed and Chan 1959). However, in terms of optimum moisture content, the relationship becomes inversely with the bentonite content after 5%. It is noticed in all mixtures that the maximum dry unit weight increases with increasing the percentage of bentonite. This is because, the added bentonite filled the air voids within the sand particles and that led to an increase in the amount of compacted soil to be more than the case without bentonite or with less

amount of bentonite.

Table 4.1 Maximum dry unit weight and optimum moisture content for soils

No.	Type of sample	Abbreviation	Maximum dry unit weight (kN/m ³)	Optimum moisture content %
1	Brickies sand	Br	17.37	12.15
2	95% Brickies sand + 5% Bentonite	BrS-5%B	17.82	13.19
3	90% Brickies sand + 10% Bentonite	BrS-10%B	17.93	12.54
4	80% Brickies sand + 20% Bentonite	BrS-20%B	17.98	12.52
5	River sand	R	14.65	13.50
6	95% River sand + 5% Bentonite	RS-5%B	15.30	15.95
7	90% River sand + 10% Bentonite	RS-10%B	15.42	15.20
8	80% River sand + 20% Bentonite	RS-20%B	16.06	14.00
9	Plaster sand	P	16.92	14.29
10	95% Plaster sand + 5% Bentonite	PS-5%B	17.33	15.50
11	90% Plaster sand + 10% Bentonite	PS-10%B	17.56	13.40
12	80% Plaster sand + 20% Bentonite	PS-20%B	17.69	11.62
13	Bentonite	B	12.18	37.20

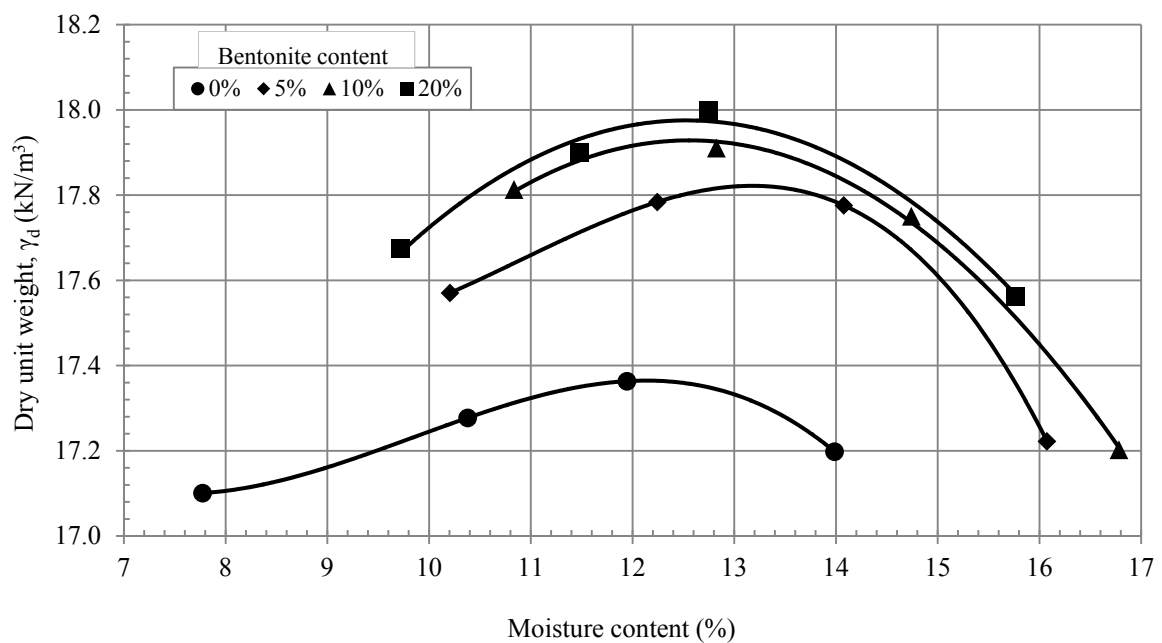


Figure 4.1 Compaction curves of brickies sand-bentonite mixtures

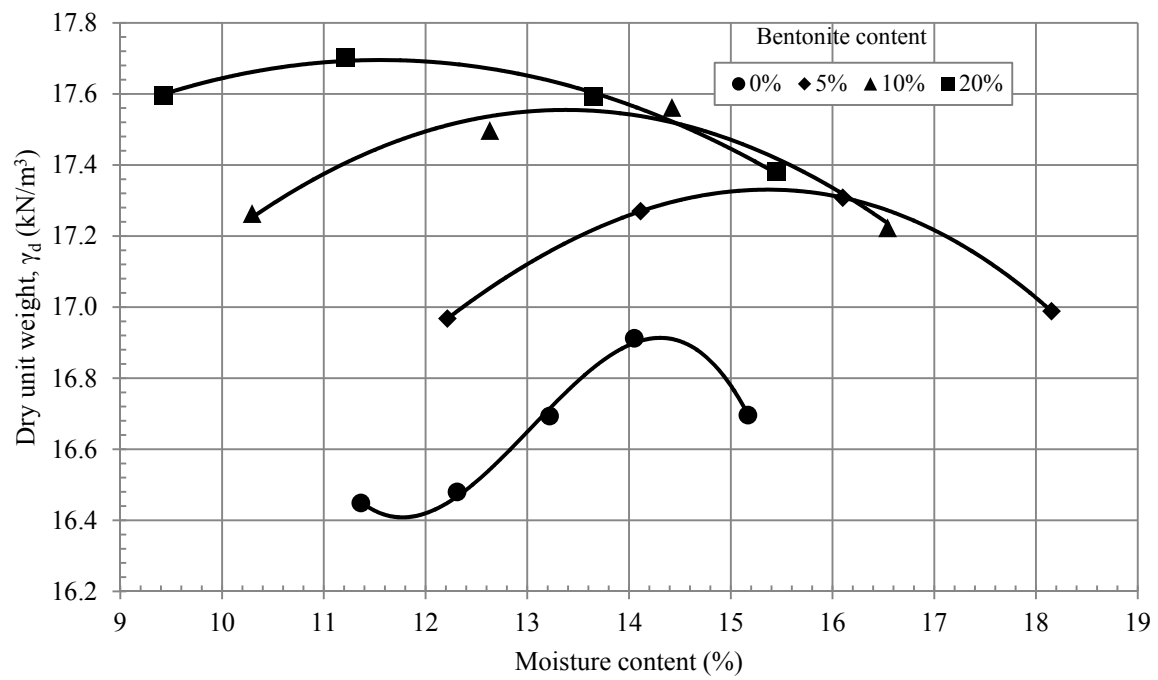


Figure 4.2 Compaction curves of plaster sand-bentonite mixtures

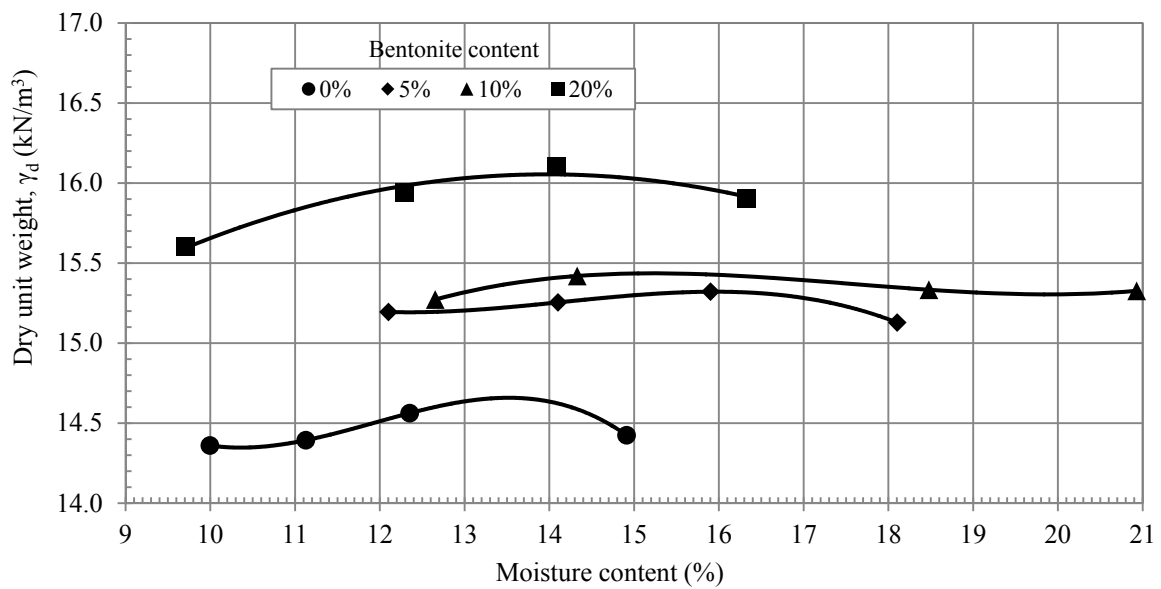


Figure 4.3 Compaction curves of river sand-bentonite mixtures

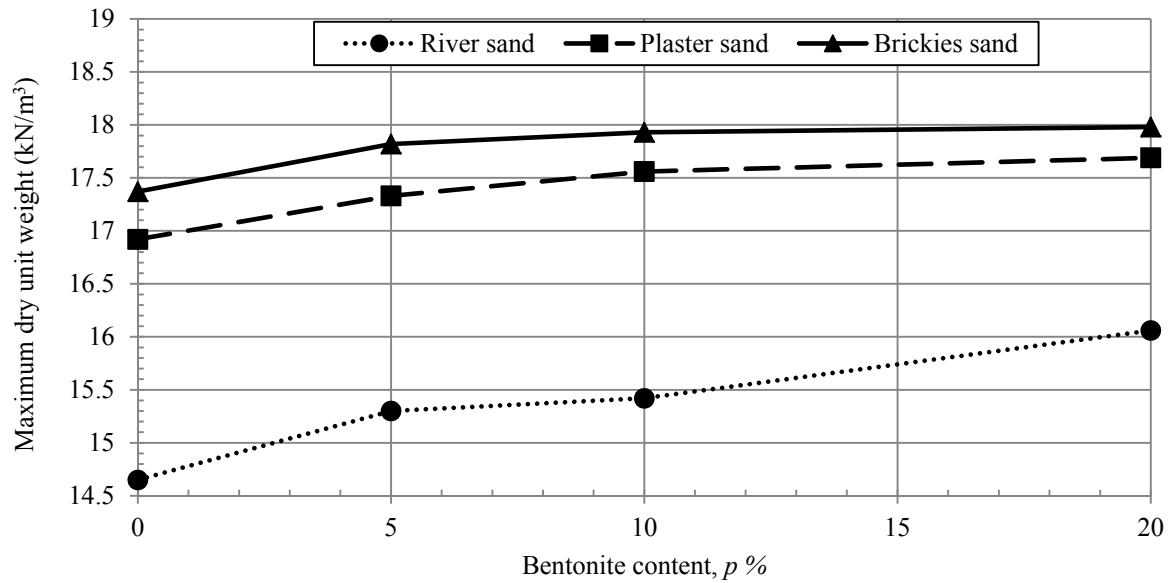


Figure 4.4 Bentonite content versus maximum dry unit weight for all mixtures

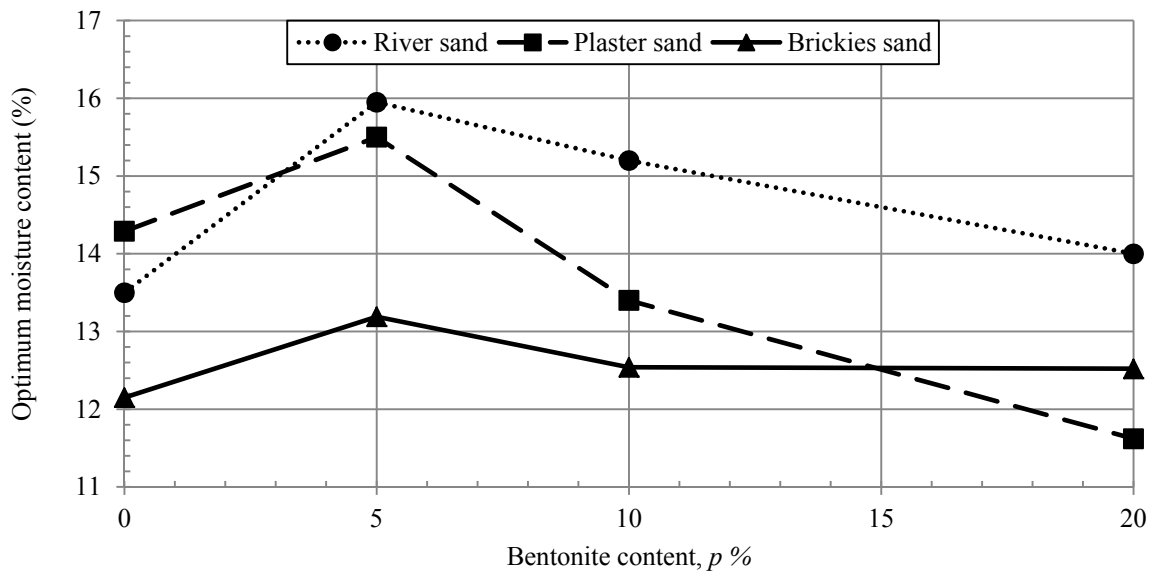


Figure 4.5 Bentonite content versus optimum moisture content for all mixture

4.3 One-Dimensional Consolidation Test Results

Data of the rate of settlement (δ) versus time factor (t) for sand-bentonite mixtures were produced by conducting one-dimensional consolidation tests as shown in Figures 4.6 - 4.14. From these data, curves were drawn and the coefficients of consolidation, compressibility, volume compressibility, and permeability were estimated.

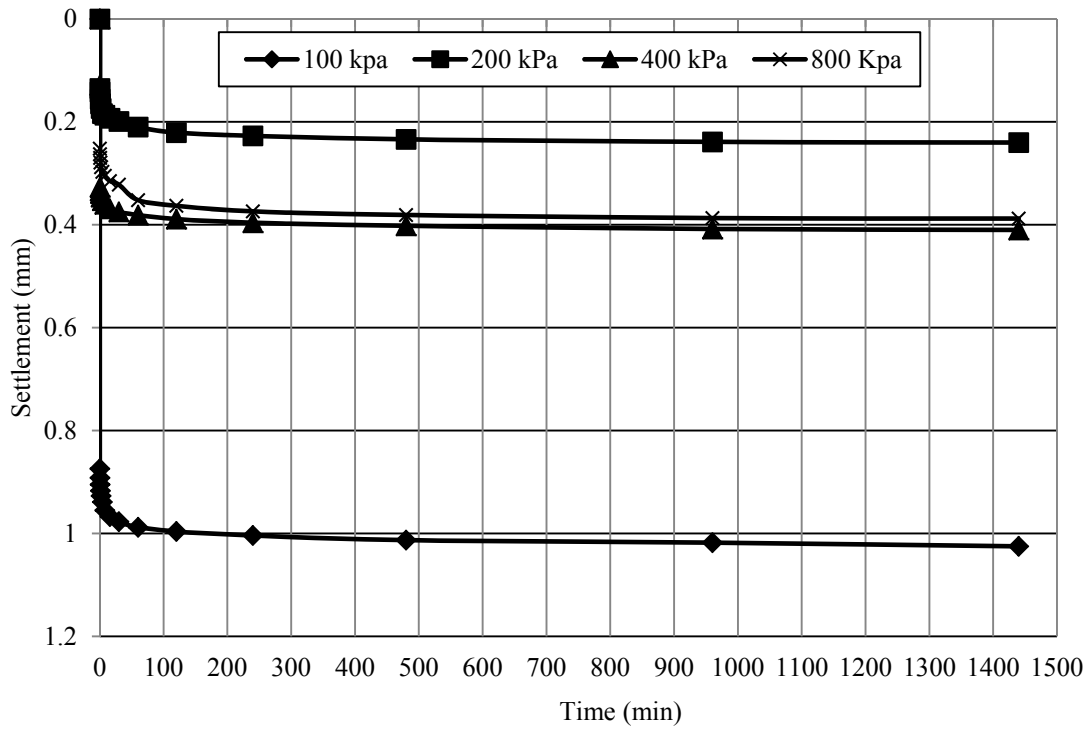


Figure 4.6 Settlement (δ) versus time (t) relationship for 5RS-B mixtures

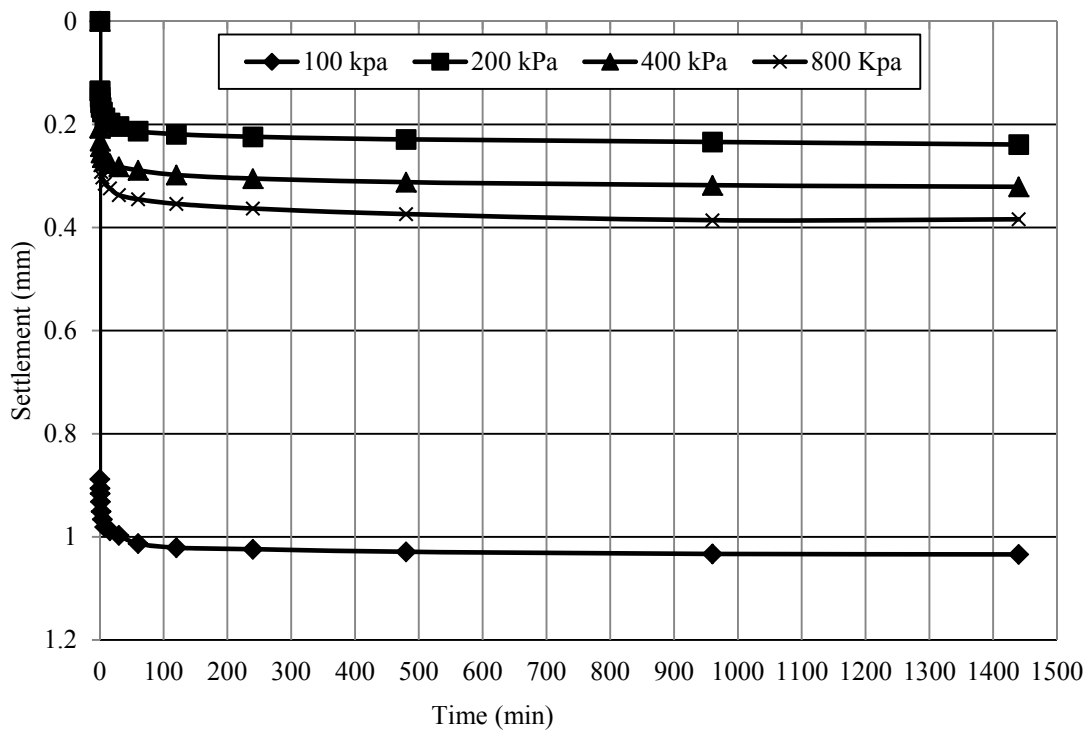


Figure 4.7 Settlement (δ) versus time (t) relationship for 10RS-B mixtures

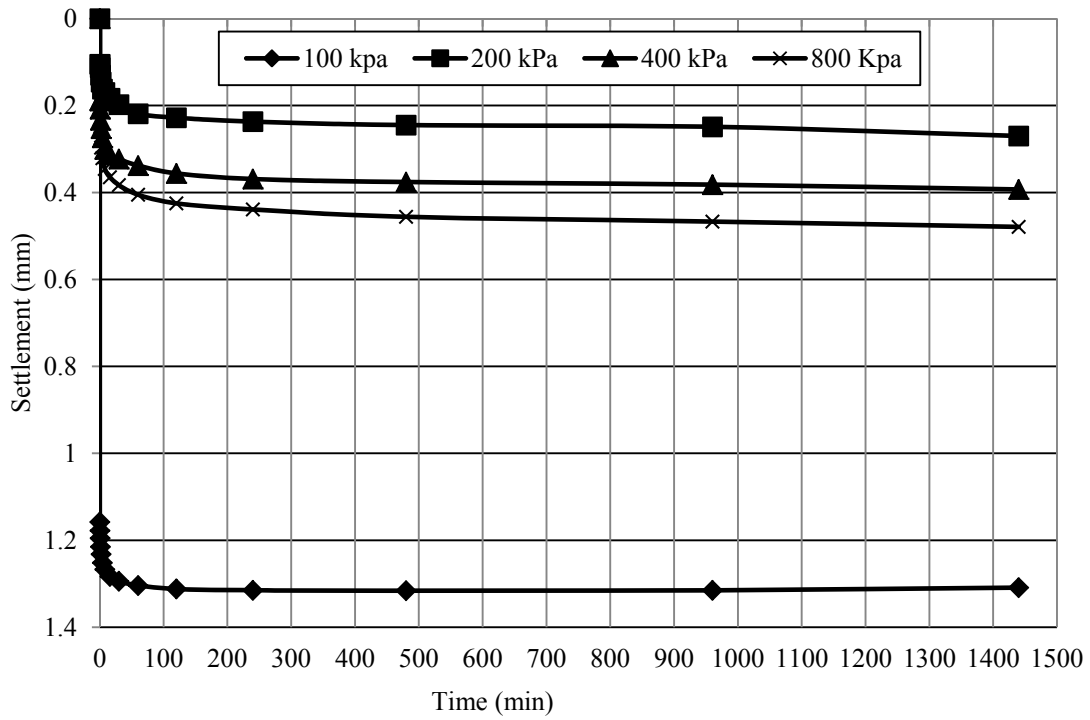


Figure 4.8 Settlement (δ) versus time (t) relationship for 20RS-B mixtures

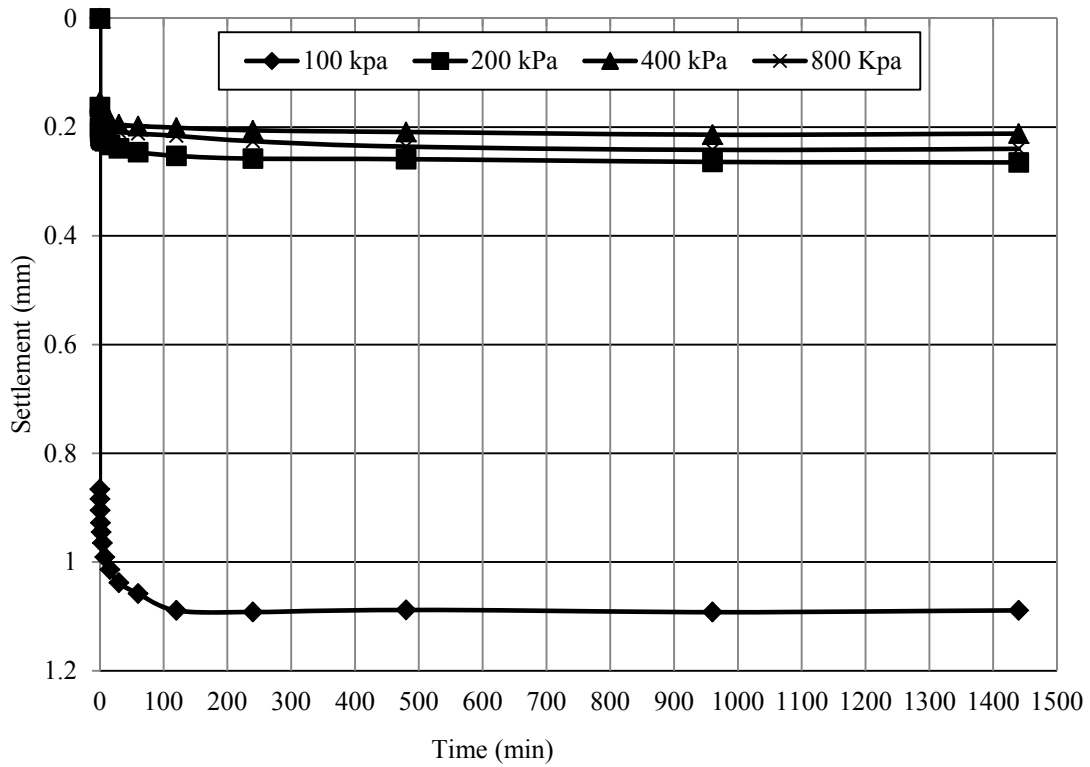


Figure 4.9 Settlement (δ) versus time (t) relationship for 5PS-B mixtures

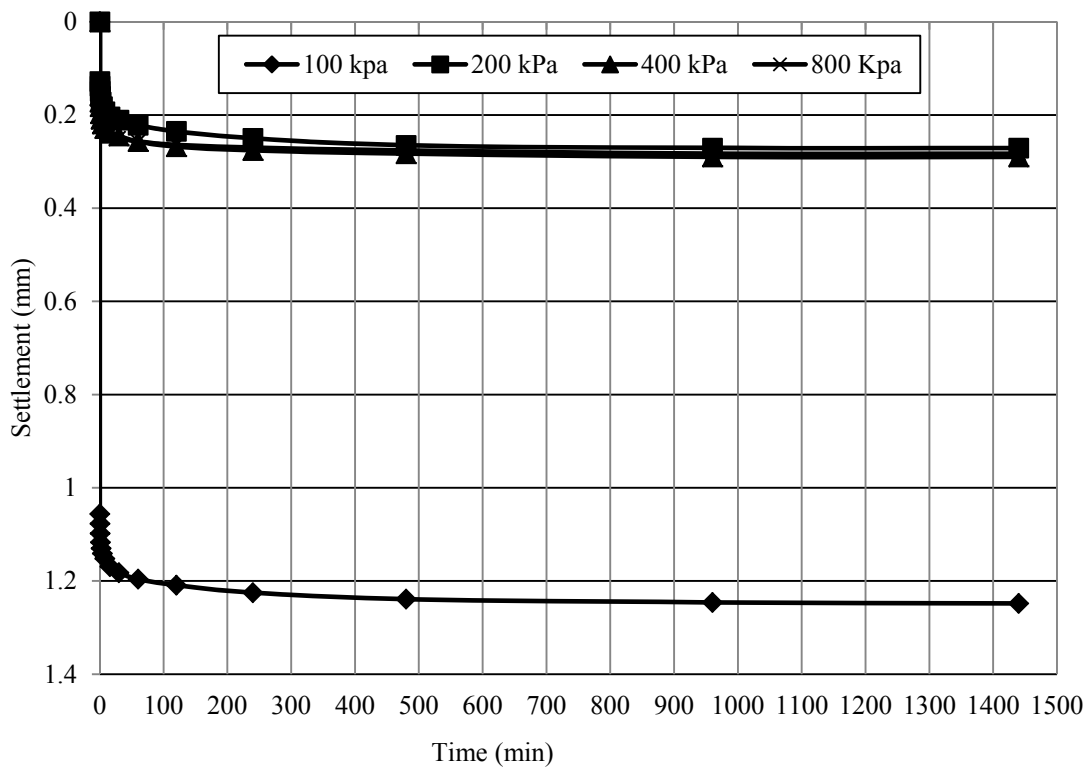


Figure 4.10 Settlement (δ) versus time (t) relationship for 10PS-B mixtures

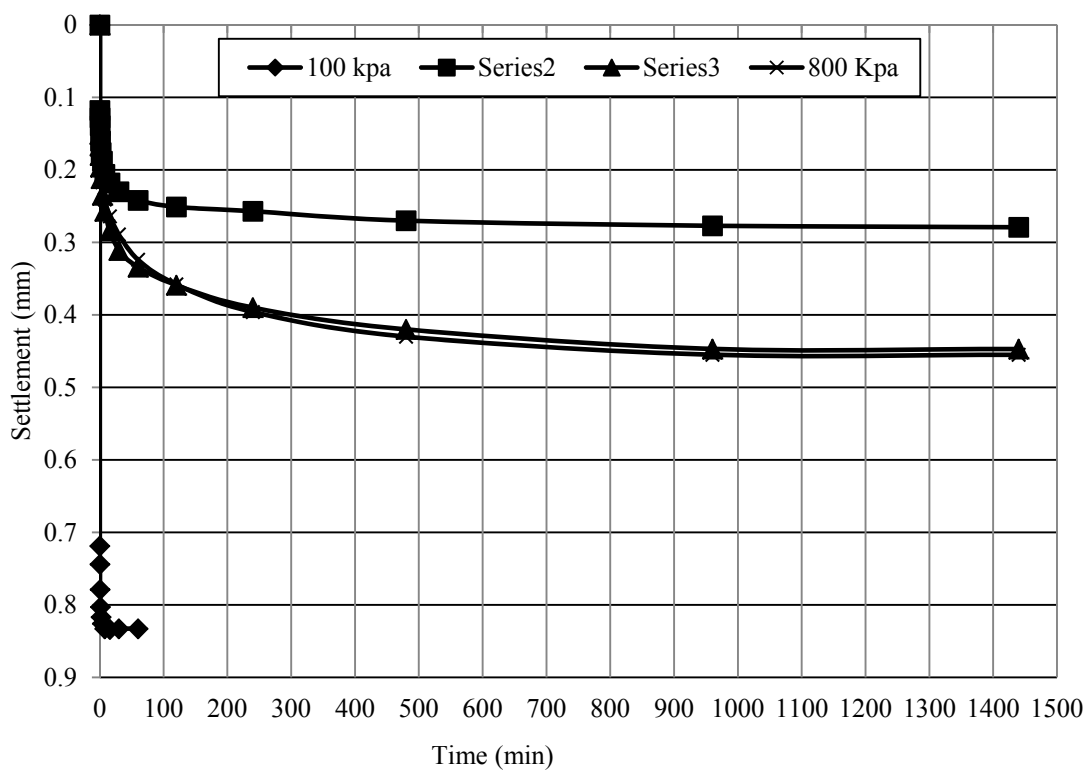


Figure 4.11 Settlement (δ) versus time (t) relationship for 20PS-B mixtures

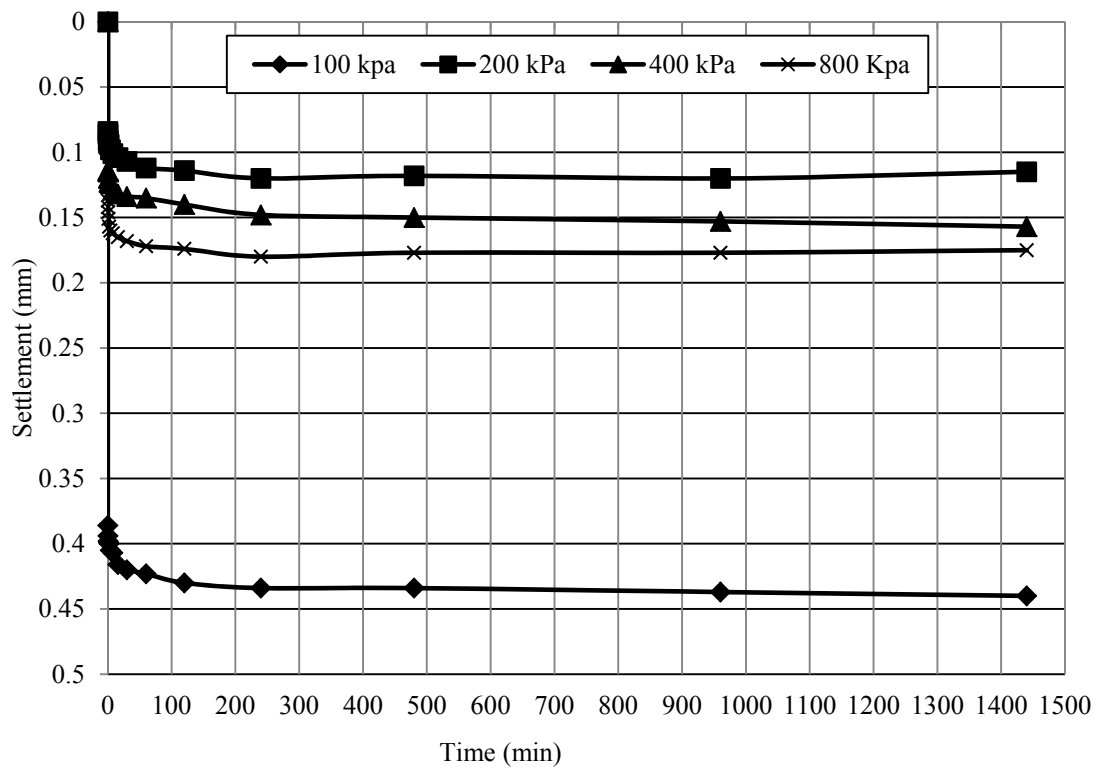


Figure 4.12 Settlement (δ) versus time (t) relationship for 5BrS-B mixtures

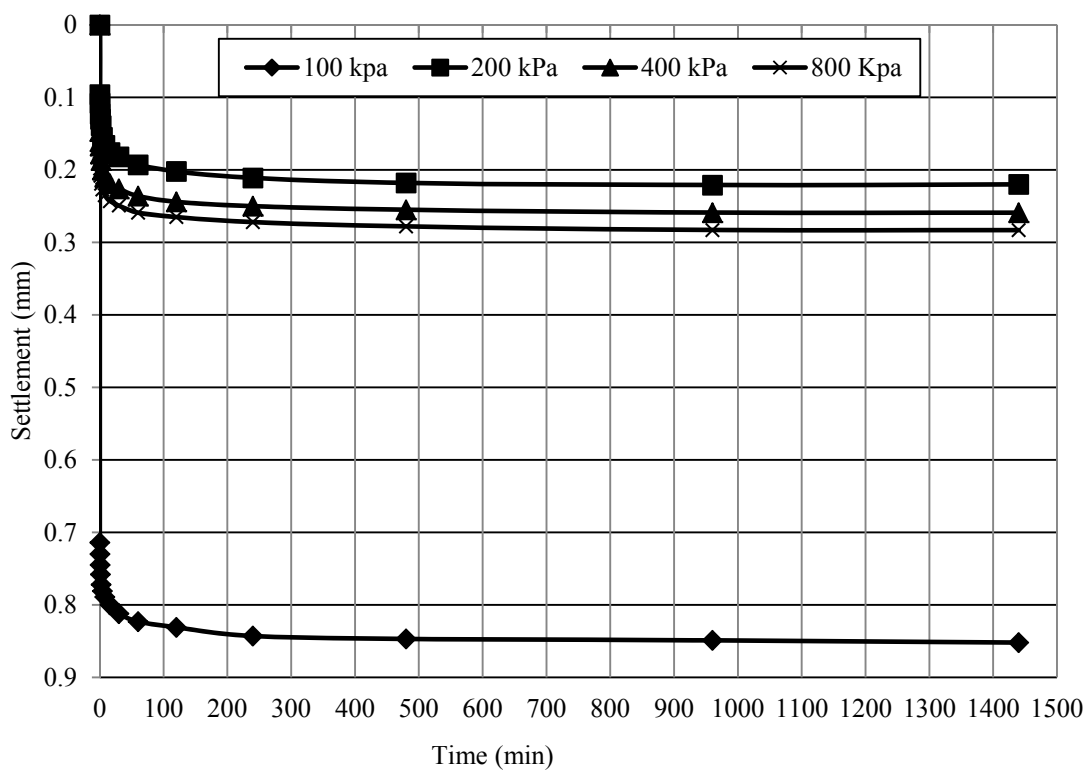


Figure 4.13 Settlement (δ) versus time (t) relationship for 10BrS-B mixtures

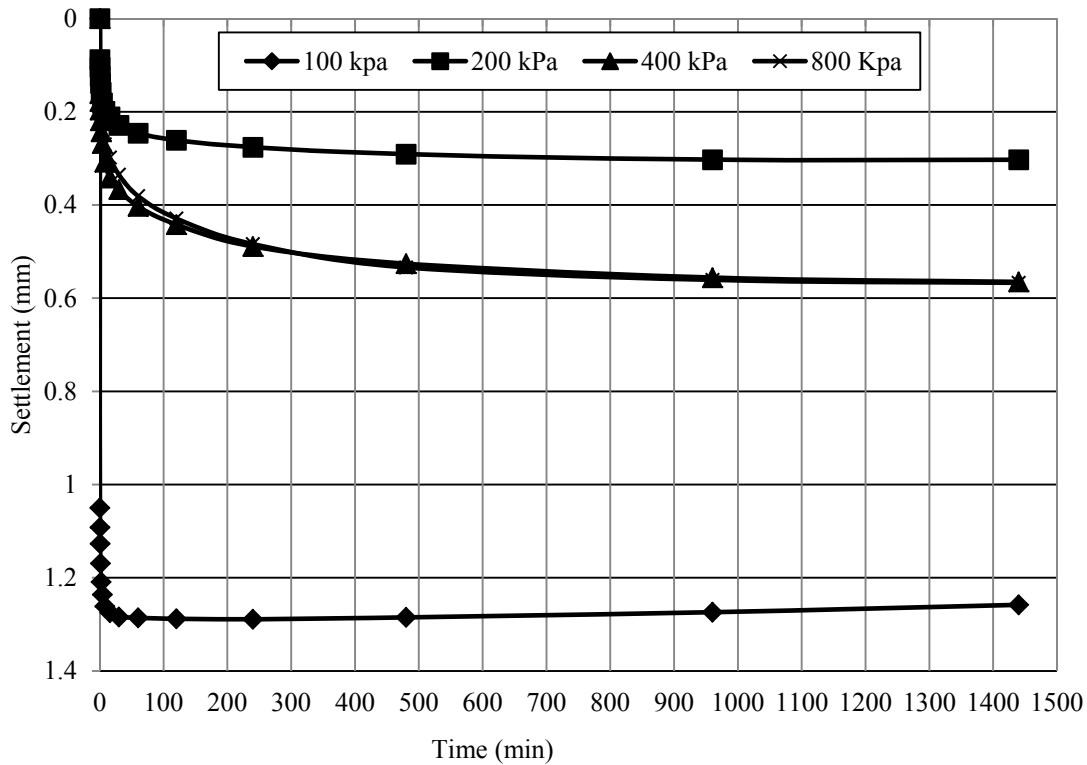


Figure 4.14 Settlement (δ) versus time (t) relationship for 20BrS-B mixtures

The relationships between the void ratio and logarithm of effective stress for all mixtures are shown in Figure 4.15. It can be seen that for any type of mixture, the value of void ratio decreases with the increasing bentonite content. It can also be seen that the highest range of void ratio is found in RS-B mixtures to be about 0.52 to 0.64 with an average difference in the void ratio between the mixtures of about 0.025. In the case of PS-B mixtures, the void ratio ranges from about 0.35 to 0.49. In these mixtures, as the bentonite content increases from 5% to 10%, there is a significant decrease in the void ratio of about 0.08. However, the decrease in the void ratio becomes very low of about 0.006 in the case of 10% to 20%. Figure 4.15 also shows that BrS-B mixtures have the lowest range of void ratio of about 0.14 to 0.43. As the bentonite content increases from 5% to 10% in BrS-B mixtures, there is a dramatic decrease in the void ratio of about 0.084, and this value becomes double in terms of bentonite contents of 10% to 20%.

The compression index (C_c) is the ratio of the change in the void ratio to the change in the logarithm of effective stress of cohesive soils, which means the slope of the virgin part in the compression curves at the loading stage. Figure 4.16 explains the relationship between the compression index and the bentonite content, which is estimated from Figure 4.15. From Figure 4.16, it can be seen that the compression index increases linearly with increasing bentonite

content for any type of sand-bentonite mixture. This pattern was also achieved by some previous researchers (De Magistris et al. 1998, Mishra et al. 2010, Watabe et al. 2011). However, the change in the compressibility index with the bentonite content differs and depends on the type of the sand used in the mixture. This difference is clearly explained in the slope of the trendlines of the mixtures in Figure 4.16. The descending order of the slopes of the mixtures is as follows: BrS-B, PS-B, and RS-B, and the values are: 0.0057, 0.0035, and 0.0013, respectively. This could be because of the effects of the variation in the particle-size distribution of the mixtures. For example, the percentage of coarse particles in RS-B mixtures is about double of the percentage of coarse particles in other mixtures, which leads the RS-B mixtures to have the least compressibility comparing with others. Therefore, it can be said that a mixture which contains a higher amount of coarse particles has the least compressibility. Further, in terms of BrS-B mixtures, having about six times more fine gradation than other mixtures, it would be the reason for the dramatic decrease in the void ratio from mixture to another, and the highest slope of compressibility index trendline.

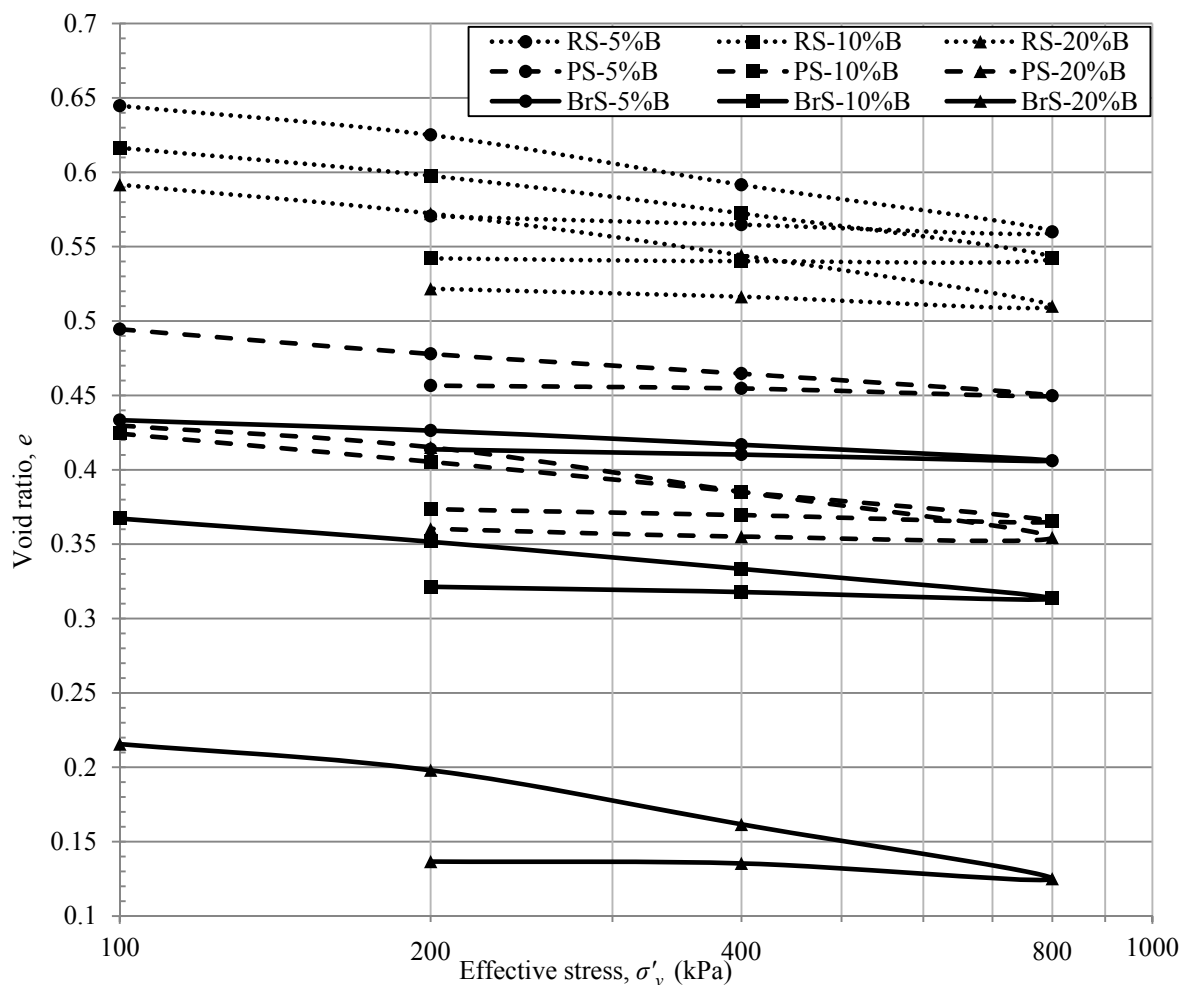


Figure 4.15 Void ratio (e) versus logarithm effective stress (σ'_v) relationship for all mixtures

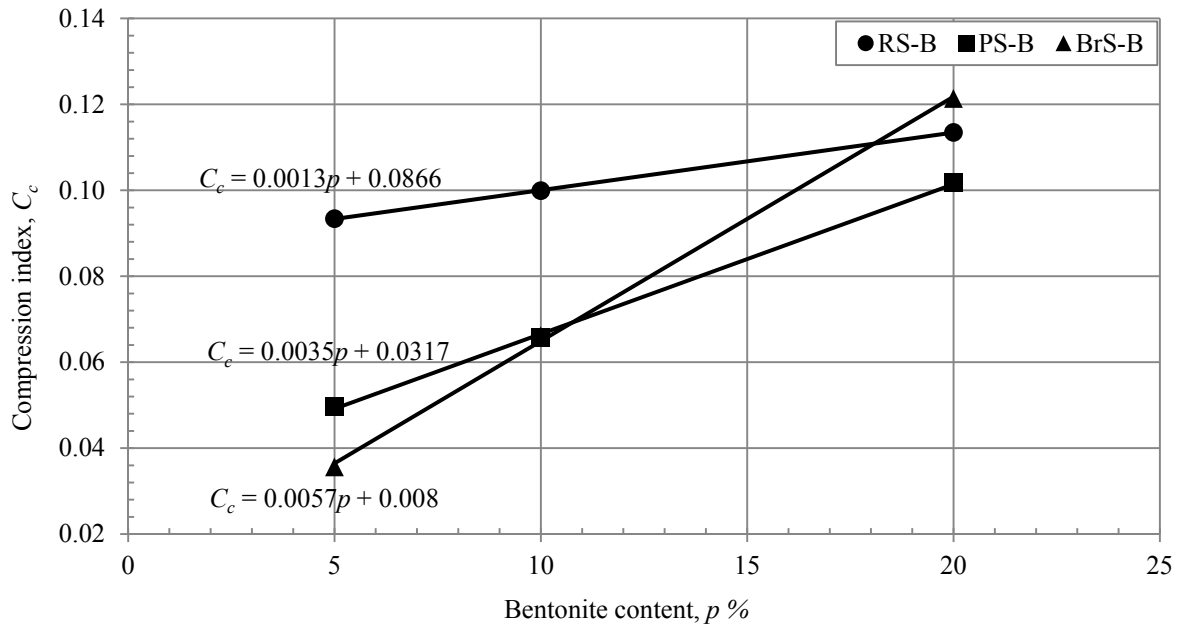


Figure 4.16 Compression index (C_c) versus bentonite content (p) relationship for all mixtures

Based on the data of the coefficients of consolidation evaluated using improved rectangular hyperbola method, the coefficients of permeability (k) were estimated. The coefficients of permeability were plotted with respect to the effective stress as shown in Figure 4.17. From the results of permeability, it can be seen that there is a significant improvement in the permeability of most mixtures compared with the permeability of sands alone (as reported in Table 3.2). Figure 4.17 reveals that for all mixtures, the permeability decreases with the increased effective stress; and for any type of mixture, the permeability also decreases with the increase of bentonite content. It is also noticed that the highest permeability is achieved by RS-5%B mixture, which is in the range of 4.04×10^{-7} to 1.17×10^{-7} m/s, whereas the lowest permeability is achieved by BrS-20%B mixture, which is in the range of 2.38×10^{-9} - 2.78×10^{-10} m/s. In Figure 4.17, it can also be observed that, as the bentonite content increases, there is a considerable change in the permeability of RS-B and PS-B mixtures. However, in case of BrS-B mixtures, the change in the permeability is approximately similar after adding bentonite for all loads. It may be stated that the different structural forms (voids and particles) of the mixtures reflect the variation in the permeability behaviour. Another outcome is that all the sand-bentonite mixtures except RS-5%B and PS-5%B meet the criteria of the hydraulic barrier, which means that their permeability are less than 10^{-9} m/s. Therefore, the optimum amount of bentonite which should be added to the sand as a hydraulic barrier is as follows: 5% for Brickies sand, and 10% for River and Plaster sand.

Figure 4.18 explains the variation of permeability versus the void ratio of all mixtures. It is observed that the permeability decreases with the decrease in the void ratio for all mixtures. This result has been presented in some previous research (e.g. Sivapullaiah et al. 2000, Watabe et al. 2011).

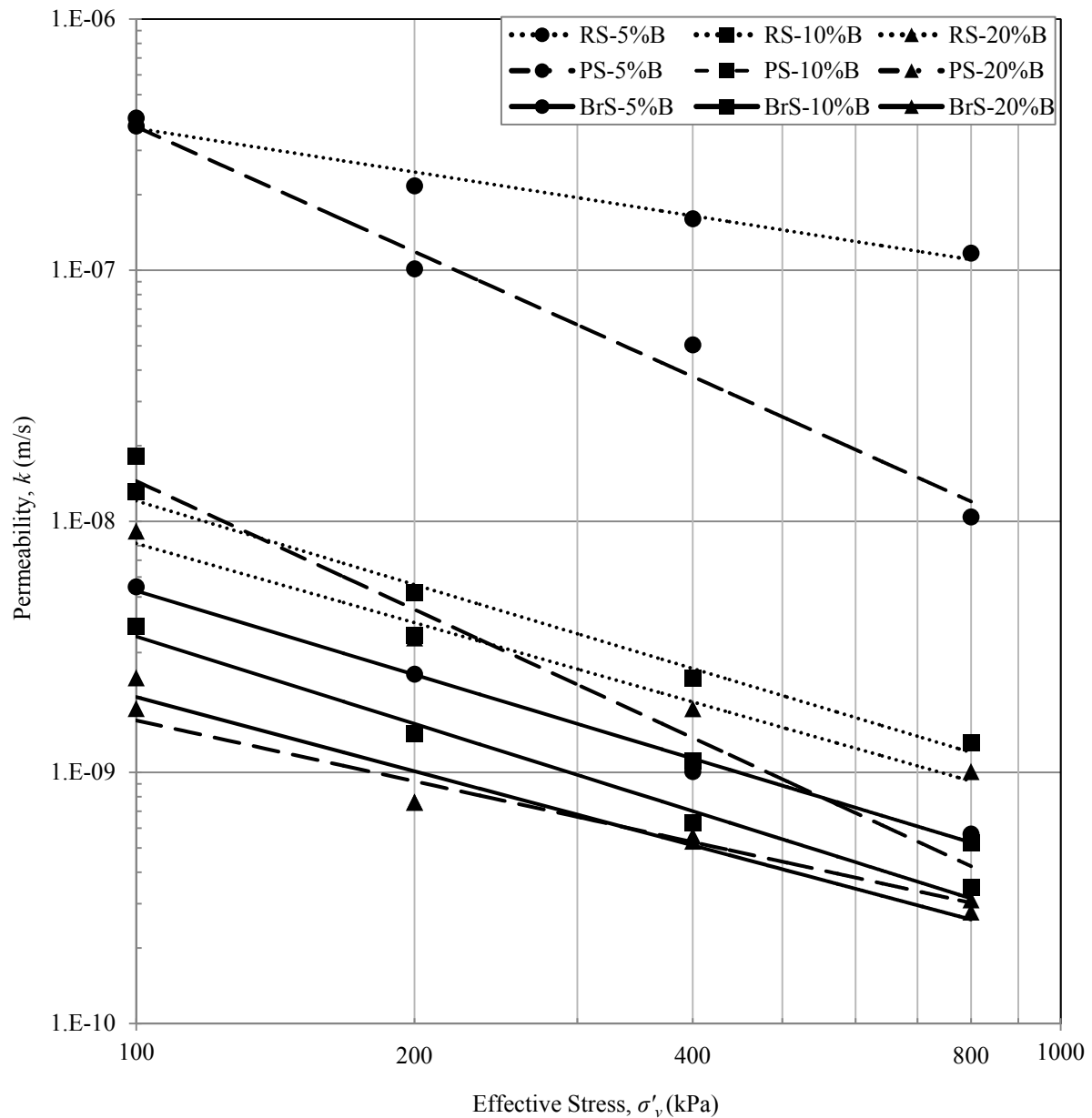


Figure 4.17 Permeability (k) versus effective stress (σ'_v) relationships of all mixtures

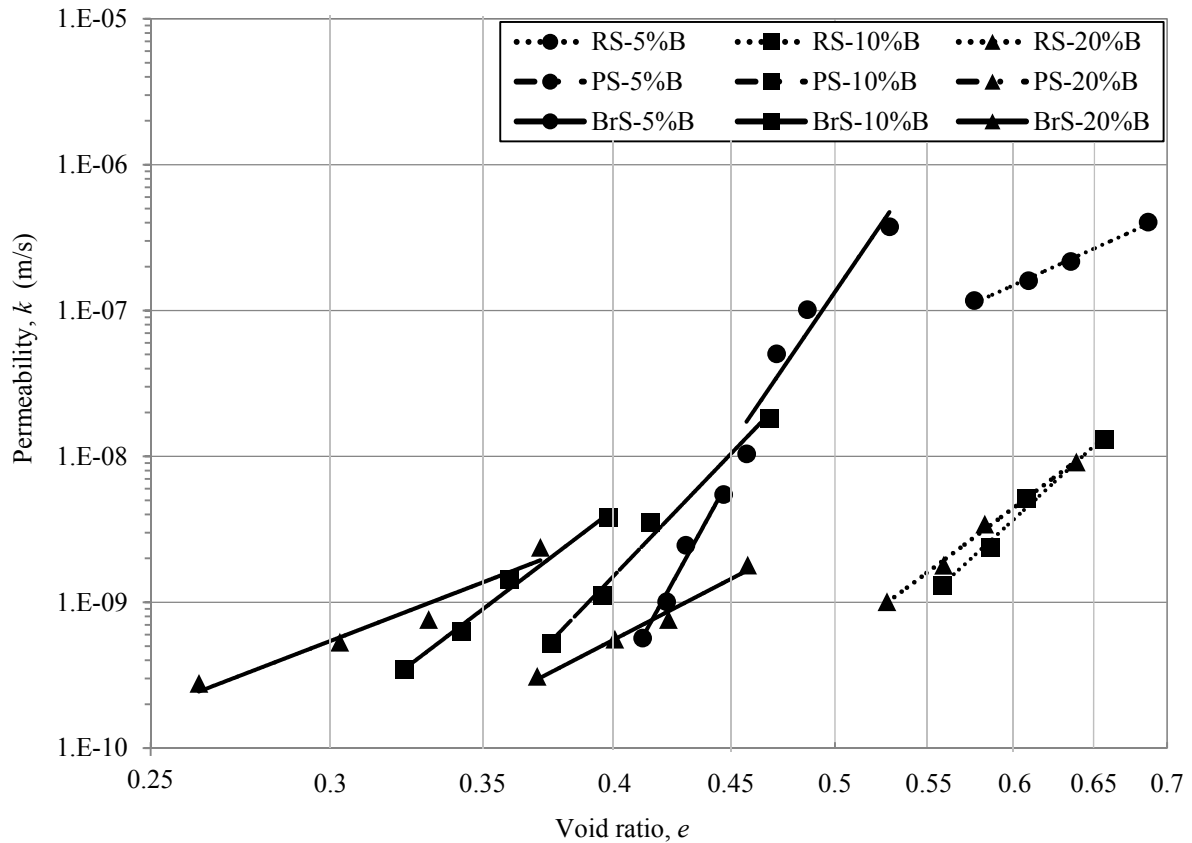


Figure 4.18 Permeability (k) versus void ratio (e) relationships of all mixtures

4.4 Conclusions

A series of one-dimensional consolidation tests were conducted on different sand-bentonite mixtures prepared from three types of sands, namely brickies sand (BrS), river sand (RS) and plaster sand (PS), and three percentages of bentonite (B) (5%, 10%, and 20%), in order to investigate the effect of the variation in particle-size distribution on the compressibility and permeability behaviour of the mixtures. Four different methods for estimating the coefficients of consolidation were used and examined in this study. Based on the results and discussion, it can be noticed that the permeability and compressibility are greatly affected by the type of soil used in the mixtures. The optimum amount of bentonite for brickies sand, plaster sand and river sand to achieve a permeability of less than 10^{-9} m/s, which is a liner design requirement, was found to be 5%, 10%, and 10%, respectively. The compression index increases linearly with the increasing bentonite content for any type of sand-bentonite mixture, but the rate of increase is relatively higher for the mixtures of brickies sand and bentonite.

CHAPTER FIVE

STRENGTH PROPERTIES OF SAND-BENTONITE MIXTURES

5.1 General

The patterns of the change in the strength characteristics with respect to bentonite content and sand gradation are investigated. This chapter describes the results of unconfined compression tests and direct shear tests conducted on the sand-bentonite mixtures considering the effects of bentonite content and sand gradation on the strength characteristics of these mixtures. These characteristics are shear-strain behaviour, unconfined compressive strength, maximum vertical strain, and Young's modulus. In addition to these characteristics, the failure plane of the mixtures is also examined in this chapter.

5.2 Strength Test Results

Two different types of tests were conducted on the sand-bentonite mixtures, namely unconfined compressive strength tests and direct shear tests. The results of the two tests are described in the following sections.

5.2.1 Unconfined compressive strength results

Stress-strain curve for all mixtures of the three types of sand was estimated by taking the average result of the three tested samples, as shown in Figures 5.1 - 5.3. From these curves, four aspects were evaluated namely: unconfined compressive strength, maximum vertical strain, angle of failure, and Young's modulus (i.e., the ratio of stress to strain), which are shown in Figures 5.4 - 5.7, respectively.

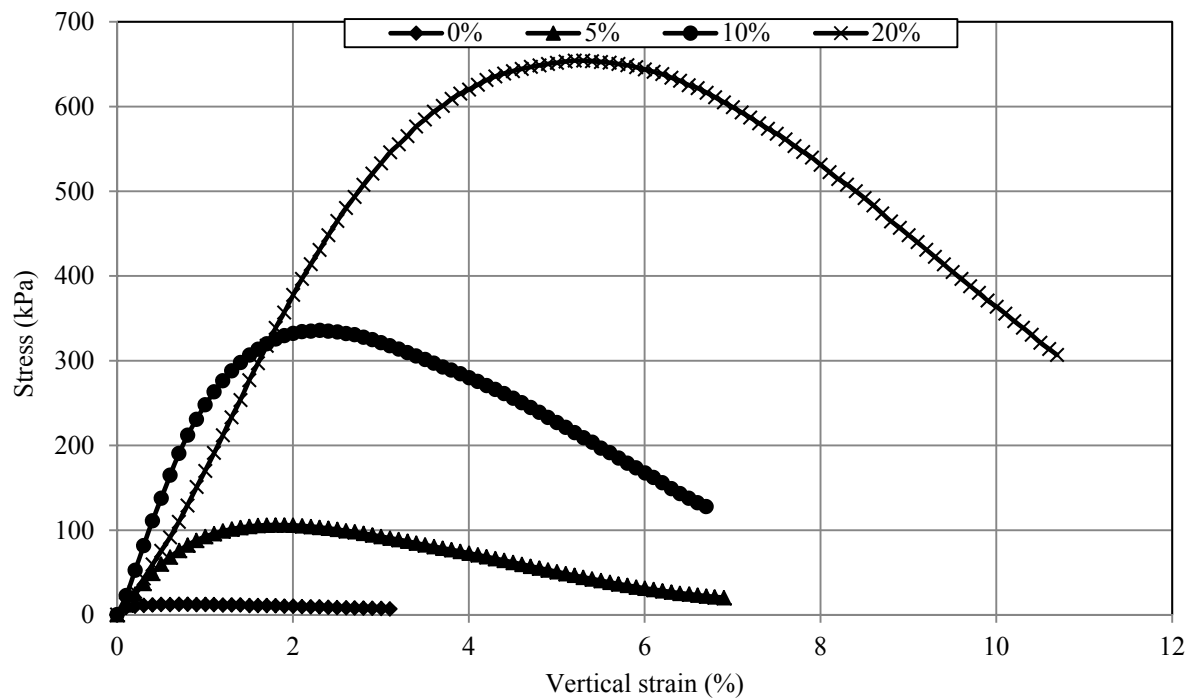


Figure 5.1 Unconfined compressive test results for river sand bentonite mixtures

Note: Each point in the curve is the average result of testing three samples.

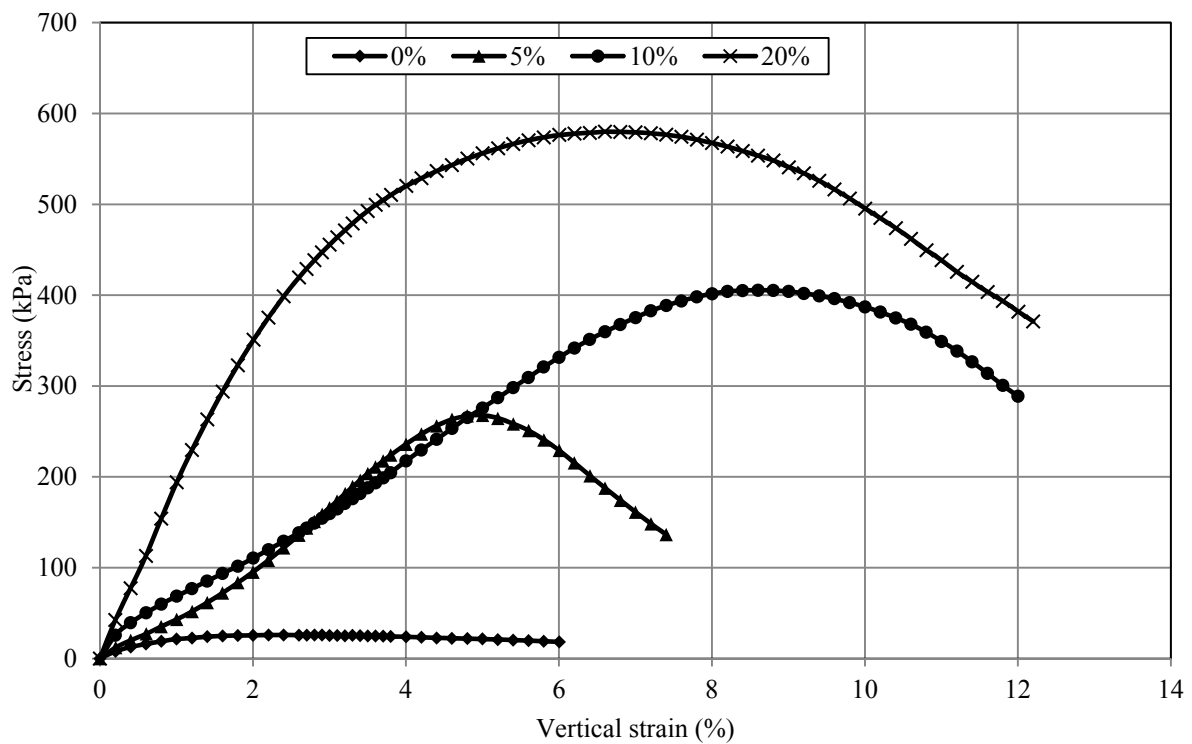


Figure 5.2 Unconfined compressive test results for plaster sand bentonite mixtures

Note: Each point in the curve is the average result of testing three samples.

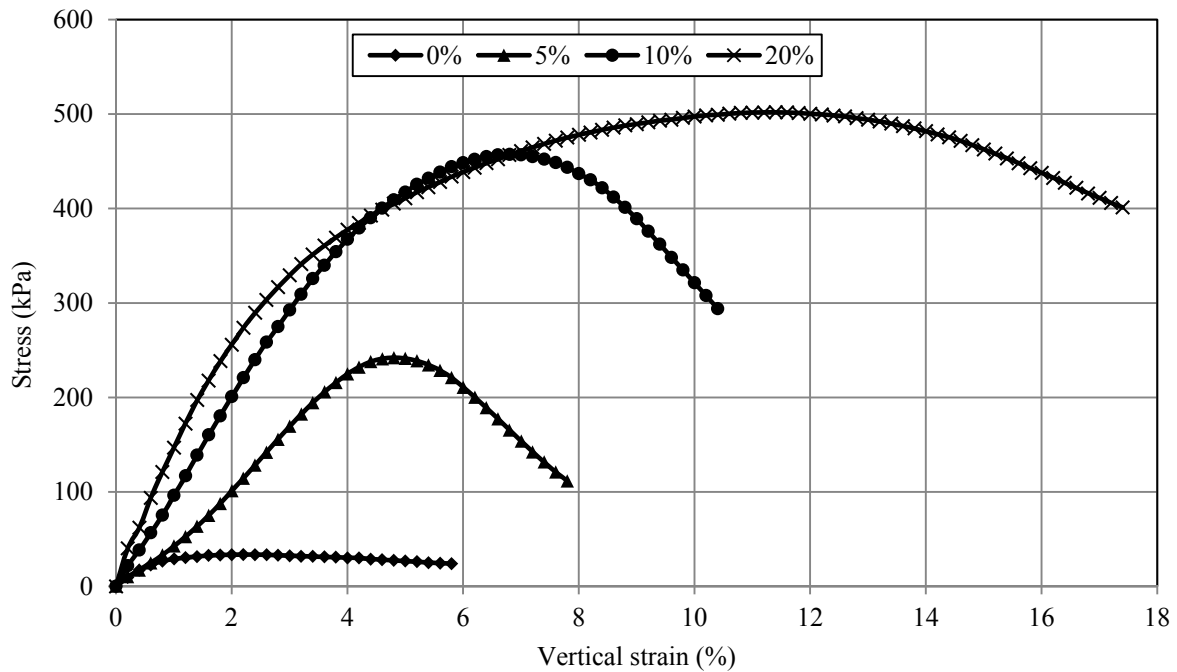


Figure 5.3 Unconfined compressive test results for brickies sand bentonite mixtures

Note: Each point in the curve is the average result of testing three samples.

Figure 5.4 shows that the unconfined compressive strength q_u values for PS-B and RS-B increase with the bentonite content as reported by Cho et al. (2002). The rates of increase are 33 % and 26 % for PS-B and RS-B, respectively. However, the rate of increase in BrS-B mixtures up to a bentonite content of 10% is 42%; beyond 10% it rises slowly, possibly because of the higher content of fine particles in 20BrS-B mixture. It means BrS-B mixtures do not show a significant improvement in the strength as the bentonite content exceeds 10%. Figure 5.4 also shows that the highest values of maximum compressive strength were recorded for 20RS-B, 20PS-B, and 20BrS-B at 654.28 kPa, 579.95 kPa, and 502.05 kPa, respectively.

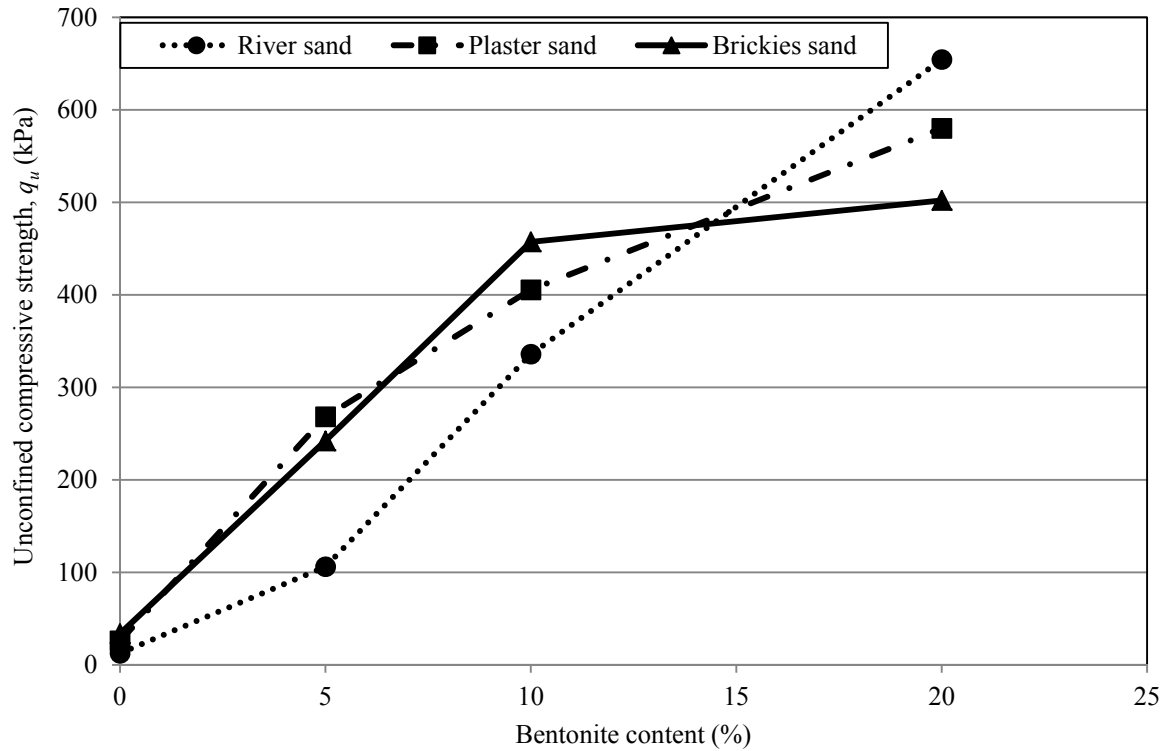


Figure 5.4 Bentonite content versus compressive strength for all mixtures

The maximum vertical strain increases linearly with increasing bentonite content for all mixtures except for PS-B (Figure 5.5). The slope for BrS-B and RS-B mixtures were estimated to be 0.46 and 0.22, respectively. In the case of PS-B mixtures, the maximum vertical strain increases rapidly at a rate of 0.62; however, the strain rate decreases from 8.6% to 6.6% when the bentonite content exceeds 10%.

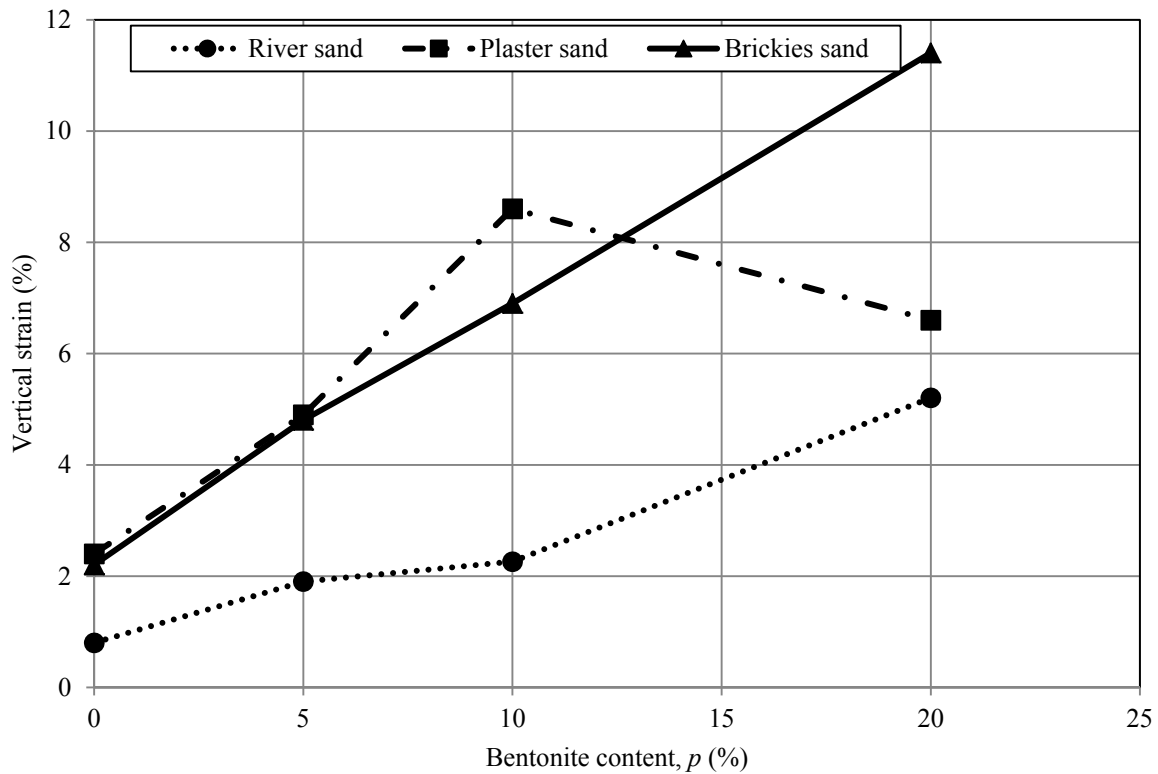


Figure 5.5 Bentonite content versus maximum vertical strain for all mixtures

Young's modulus of elasticity was also estimated from the stress-strain curves (Figure 5.6); as the bentonite content increases Young's modulus also increases, as also reported by Cho et al. (2002), except for 20BrS-B and 20RS-B, at which they decrease. The highest values of Young's modulus of elasticity i.e., the stiffest materials, are 14.86, and 12.57 MPa, for RS-B with 10 and 20% bentonite content, respectively.

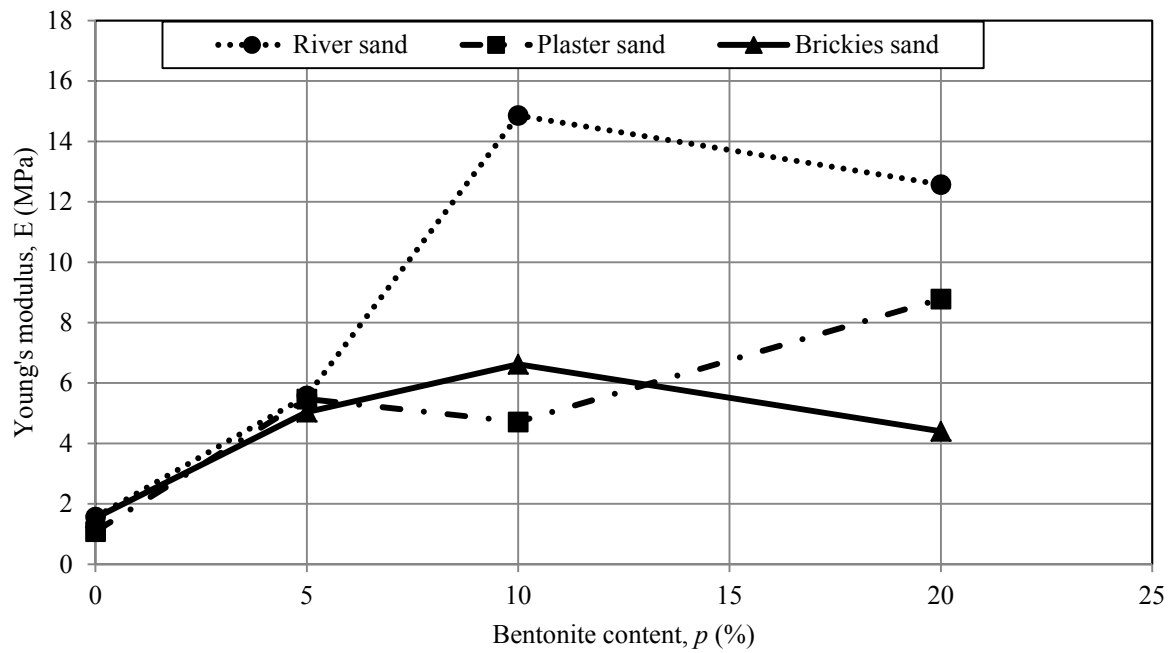


Figure 5.6 Bentonite content versus Young's modulus for all mixtures

Photographs for the failure plane of all sand-bentonite mixtures were captured and listed in Figures 5.8 - 5.19. The angles of failure for the tested sample of mixtures were also estimated and drawn in Figure 5.7. It can be seen that the angle of failure decreases approximately linearly with the increase of bentonite content. However, for RS-B and PS-B mixtures after the bentonite contents exceed 10%, the change in the angle of failure becomes marginal. The rate of decrease for RS-B, BrS-B, and PS-B are 0.702, 0.544, and 0.466, respectively.

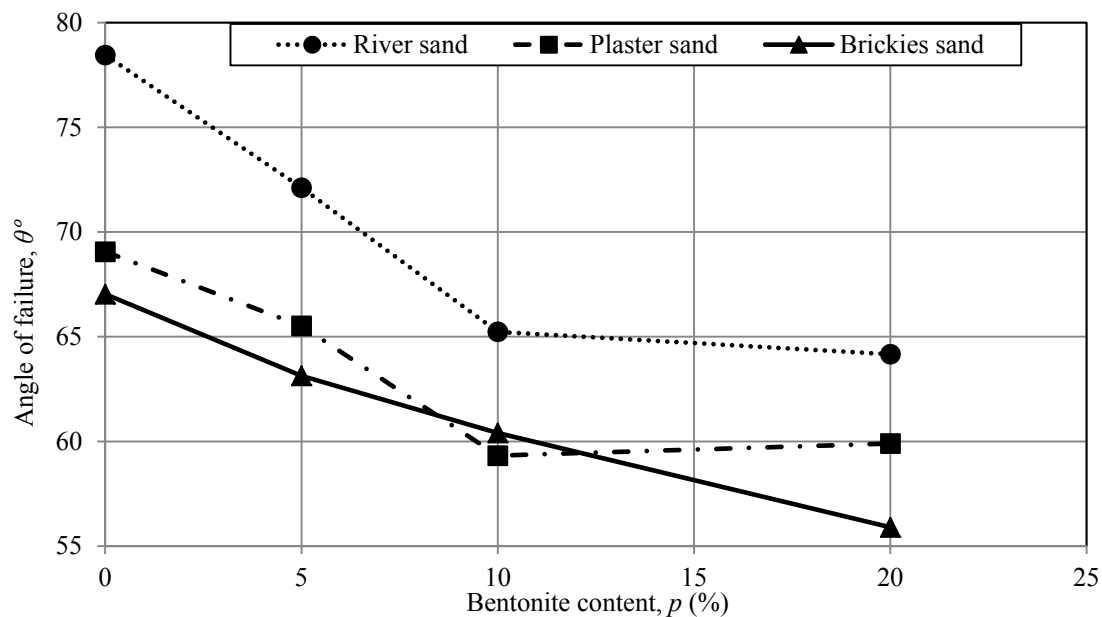


Figure 5.7 Bentonite content versus angle of failure for all mixtures

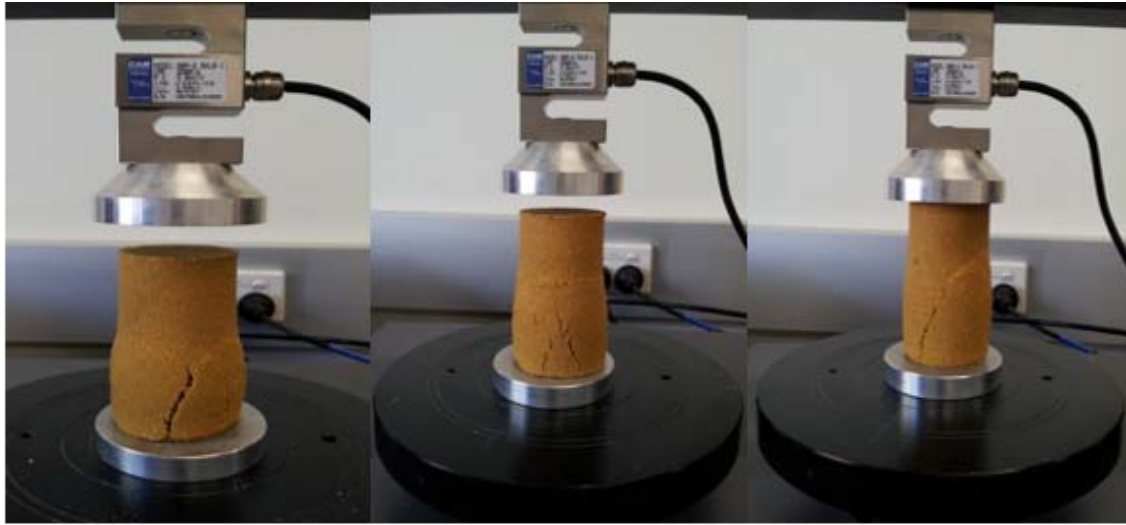


Figure 5.8 Photographs of brickies sand specimens tested by unconfined compressive strength test

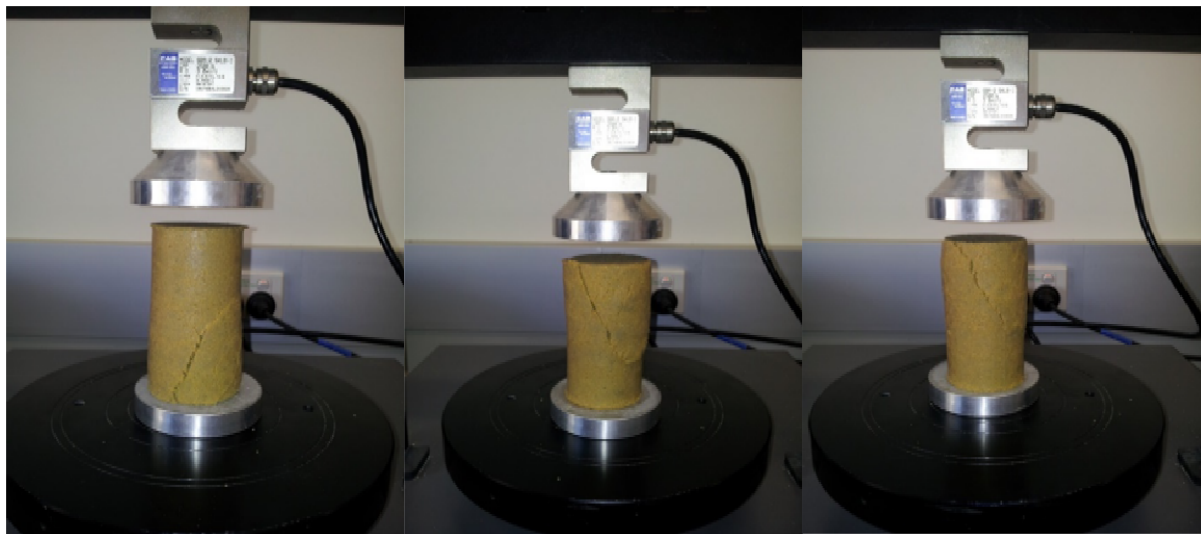


Figure 5.9 Photographs of 5BrS-B specimens tested by unconfined compressive strength test

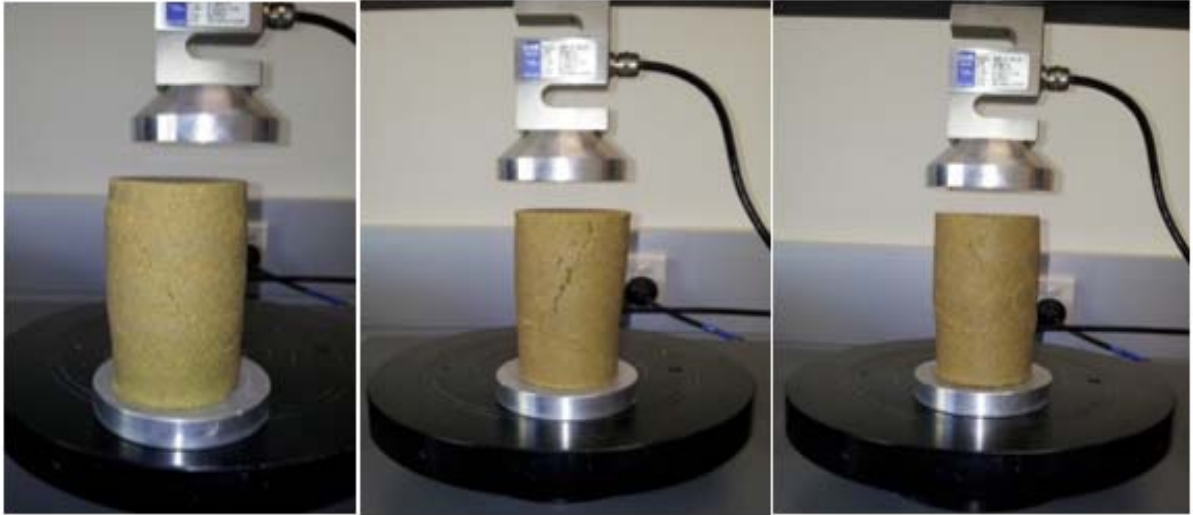


Figure 5.10 Photographs of 10BrS-B specimens tested by unconfined compressive strength test



Figure 5.11 Photographs of 20BrS-B specimens tested by unconfined compressive strength test

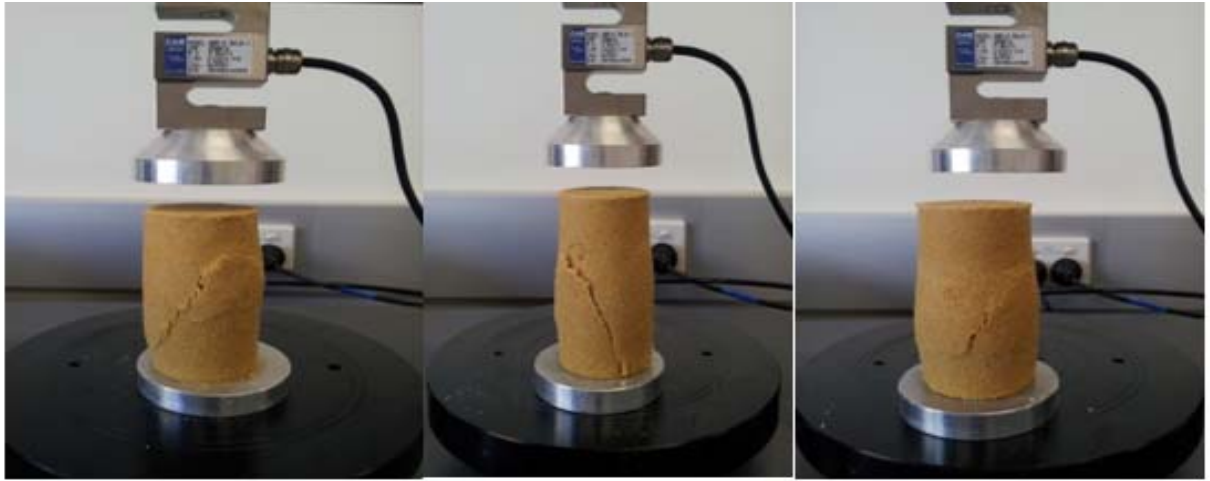


Figure 5.12 Photographs of plaster sand specimens tested by unconfined compressive strength test

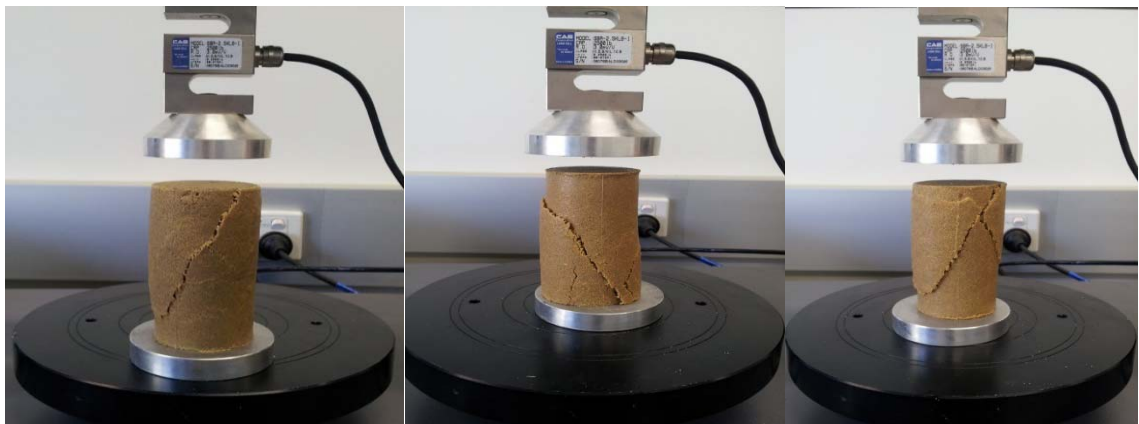


Figure 5.13 Photographs of 5PS-B specimens tested by unconfined compressive strength test

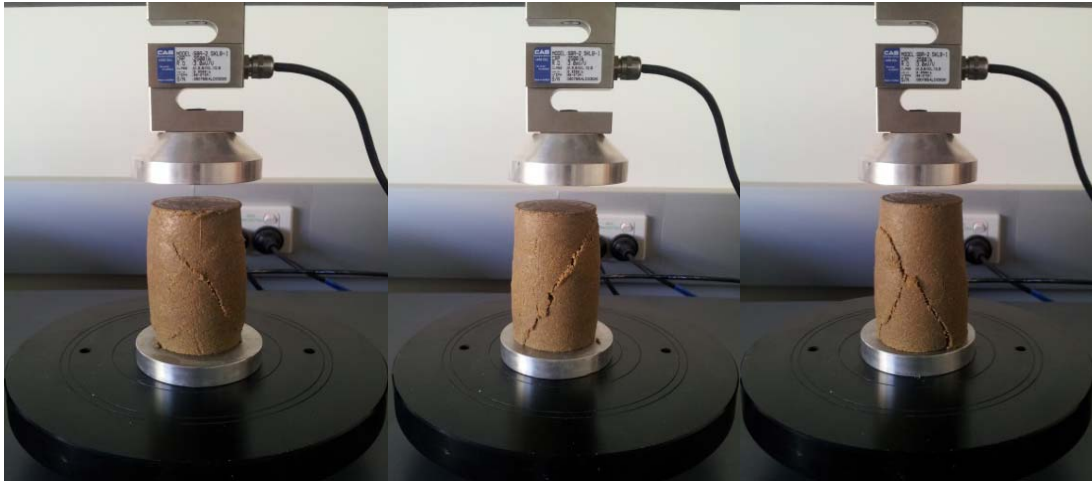


Figure 5.14 Photographs of 10PS-B specimens tested by unconfined compressive strength test

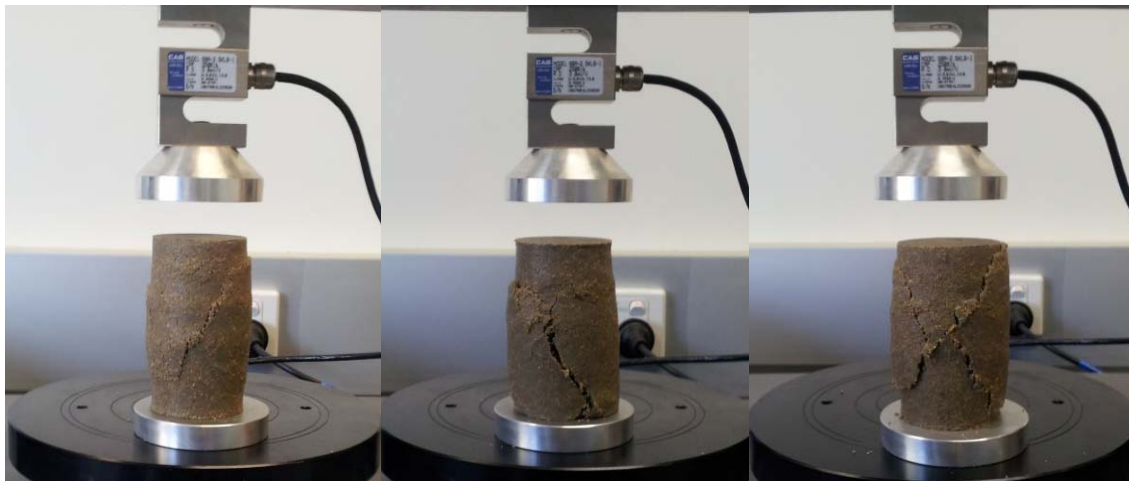


Figure 5.15 Photographs of 20PS-B specimens tested by unconfined compressive strength test

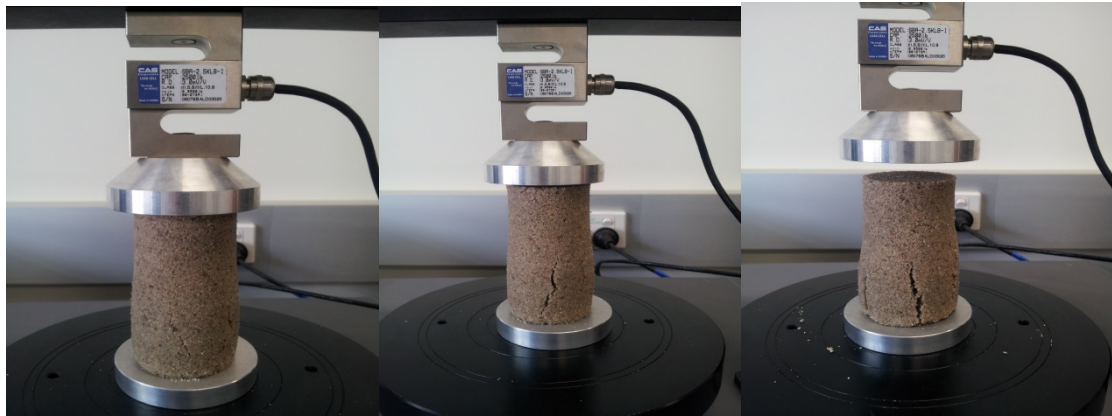


Figure 5.16 Photographs of river sand specimens tested by unconfined compressive strength test

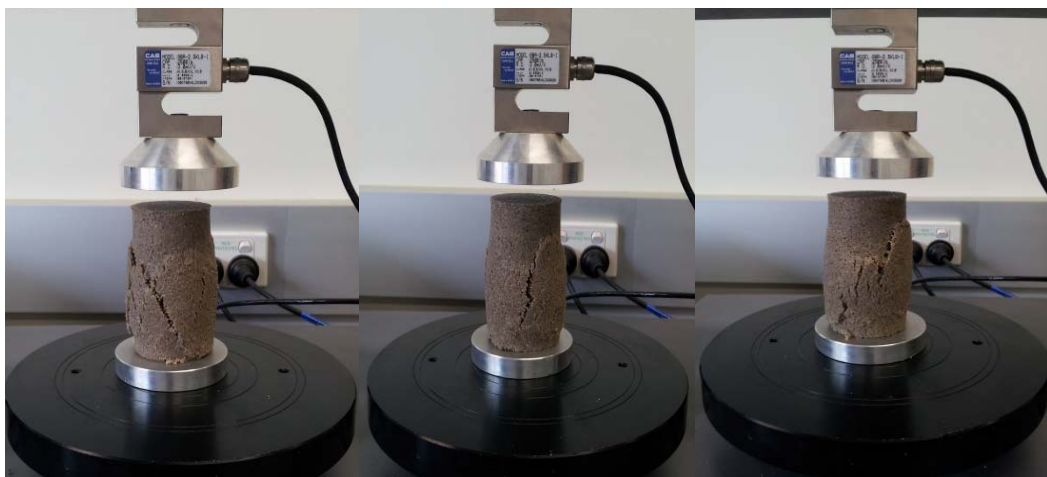


Figure 5.17 Photographs of 5RS-B specimens tested by unconfined compressive strength test

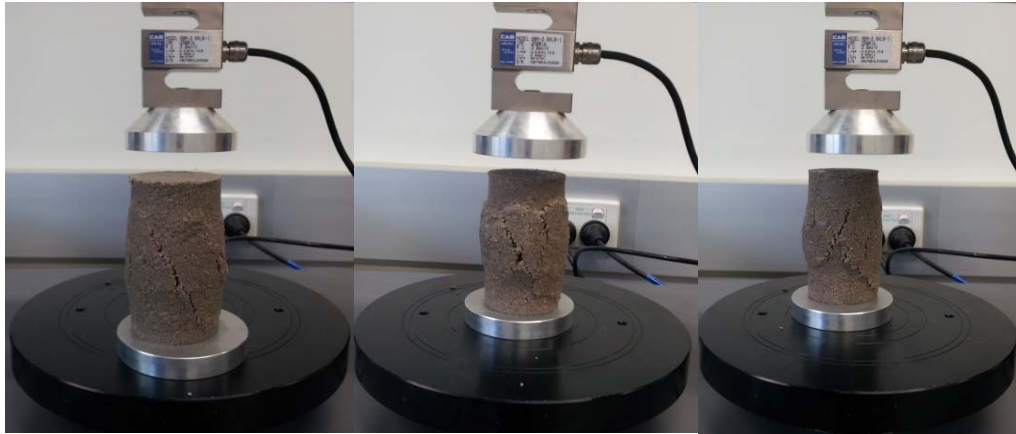


Figure 5.18 Photographs of 10RS-B specimens tested by unconfined compressive strength test

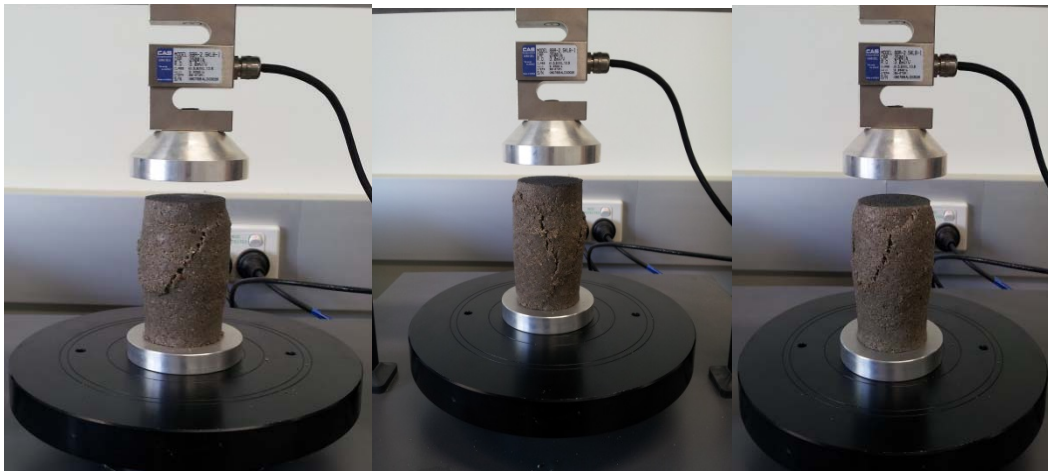


Figure 5.19 Photographs of 20RS-B specimens tested by unconfined compressive strength test

5.2.2 Direct shear tests results

Direct shear tests were performed with three normal stresses of 100, 200, and 300 kPa for three sands and nine sand-bentonite mixtures in the range 5-20% bentonite content, in order to estimate their Mohr-Coulomb failure envelope (Figures 5.20 – 5.22) and hence, their strength parameters i.e., angle of internal friction and cohesion (Figures 5.23 – 5.24). From Figures 5.20 – 5.22, it is clear that the maximum shear strength (τ_{\max}) increases linearly with normal stresses for the sand and bentonite mixtures. A similar result was also noted by Chalermyanont and Arrykul (2005). The values of τ_{\max} were also analysed for the three normal loads. For the low normal load considered in this study, the τ_{\max} tends to merge regardless of the case examined. At this load, the average value of τ_{\max} for all bentonite levels for BrS-B, PS-B, and RS-P mixtures were determined to be 107.18, 110, and 118.76 kPa, respectively. However, at the maximum normal load, as the bentonite content increases, the τ_{\max} decreases as reported by Gueddouda et al. (2008). The range of values determined for τ_{\max} with bentonite content for BrS-B, PS-B, and RS-P mixtures are 266.7-184.6 kPa, 302.3-250.9 kPa, and 313.4-210.8 kPa, respectively. It is also indicated that at normal stress of 200 kPa, the rate of decrease is not consistent for all types of the sands tested in this study.

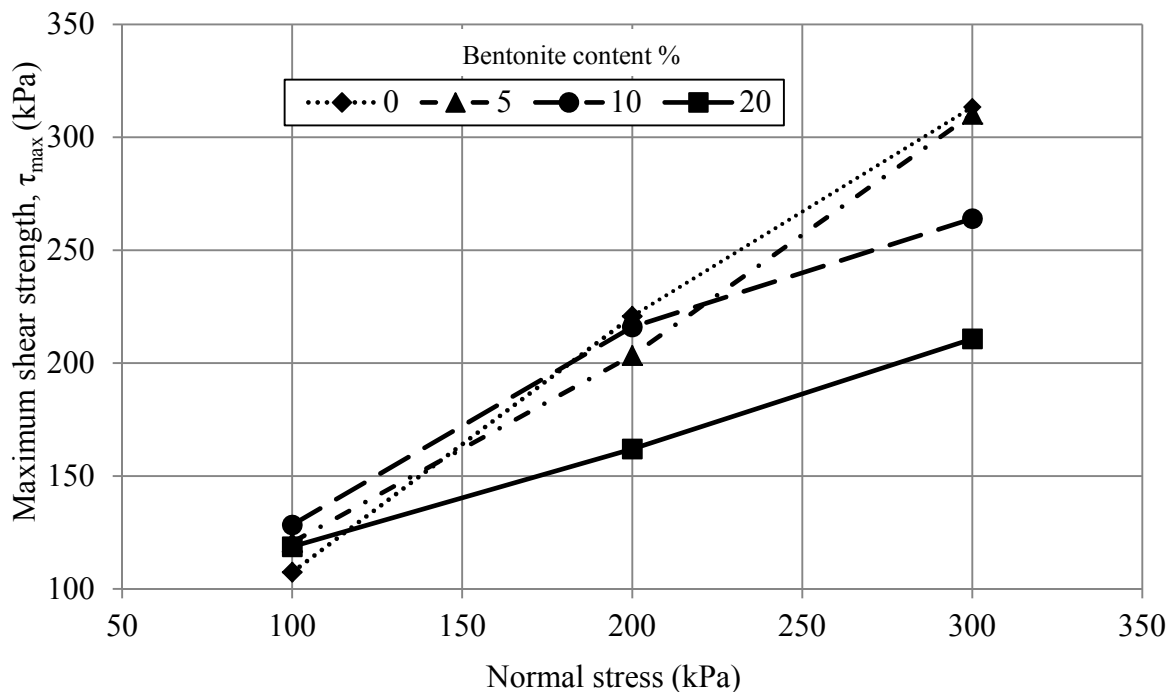


Figure 5.20 Mohr-Coloumb failure envelopes for RSB mixtures

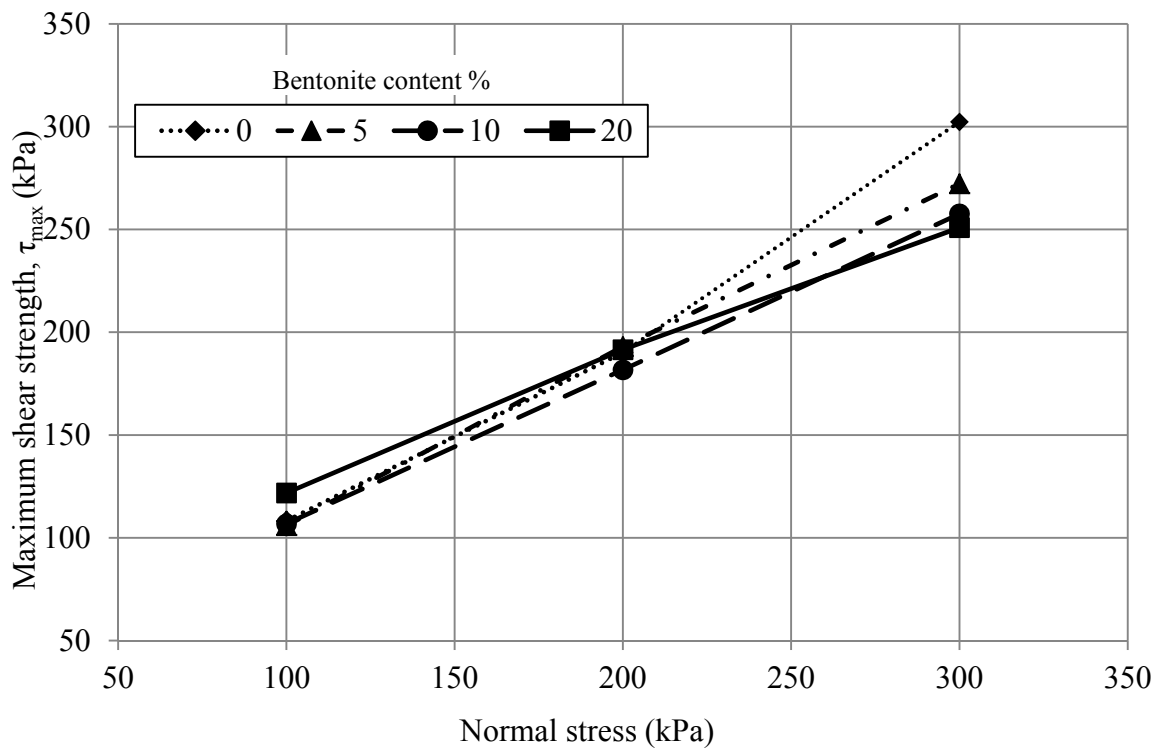


Figure 5.21 Mohr-Coloumb failure envelopes for PSB mixtures

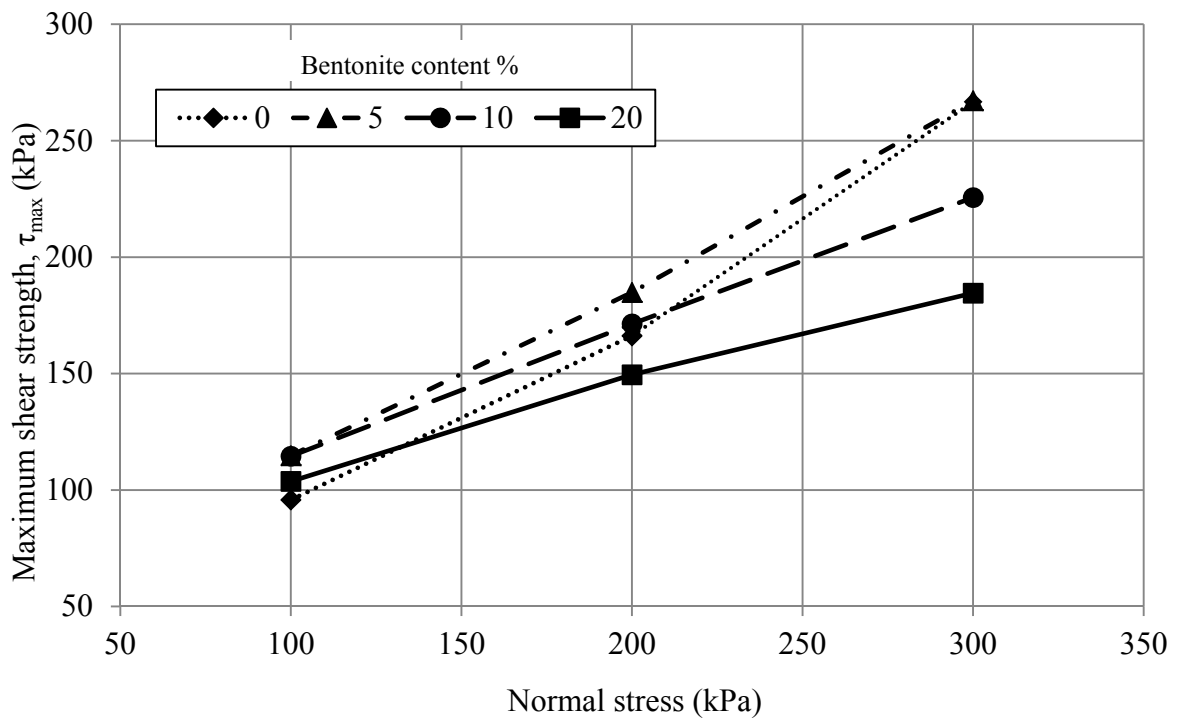


Figure 5.22 Mohr-Coloumb failure envelopes for BrSB mixtures

The angle of internal friction decreases with increasing bentonite content for all three sand types (Figure 5.23). This is in agreement with the results reported by Chalermyanont and Arrykul (2005) and Gueddouda et al. (2008). For each type of mixture, a linear equation, was fitted to the data to describe the corresponding pattern. For this equation, a and b are given in Table 5.1 and represent the constants specified for any type of mixture. A high rate of decrease can be found in RS-B and BrS-B mixtures at about 1.1 and 0.96, respectively; whereas, PS-B mixture has the low rate of decrease at about 0.55.

Table 5.1 Constants of the linear equation estimated for friction angle trend lines of three types of mixtures

No.	Type of sample	a	b	R^2
1	BrSB	-0.96	40.7	0.97
2	RSB	-1.1	46.7	0.97
3	PSB	-0.55	43.2	0.97

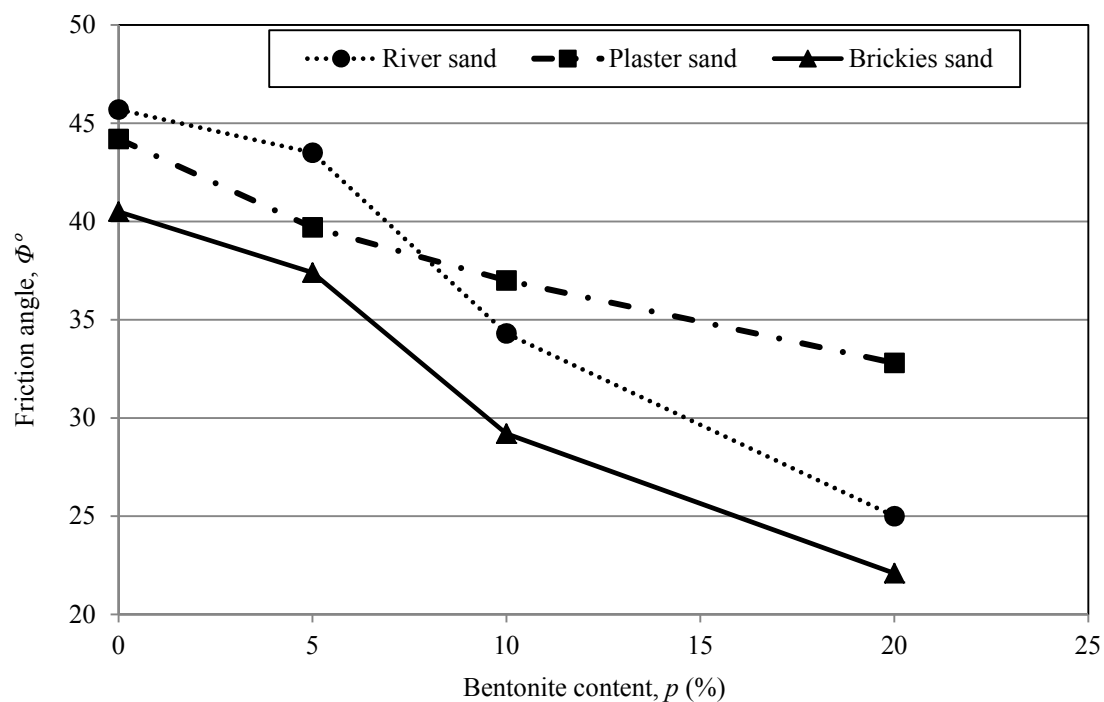


Figure 5.23 Bentonite content versus friction angle for all mixtures

Contrary to the angle of internal friction results, Figure 5.24 indicates that the cohesion increases linearly as the bentonite content increases for all cases, as reported by Chalermyanont and Arrykul (2005) and Gueddouda et al. (2008). Notably, PS-B mixture has the lowest value and the rate of increase changes linearly in contrast to the other mixtures. The values of cohesion for PSB mixture are as follows: 3.35, 23.4, 30.5, and 59.2 kPa, for BrS-B and PS-B, the cohesion increases rapidly with increasing bentonite content from 0 to 10%, after this the rate of increase becomes low and ranges from 58.3 to 64.5 kPa for 0%, and 65.4 to 69.4 kPa for 10%.

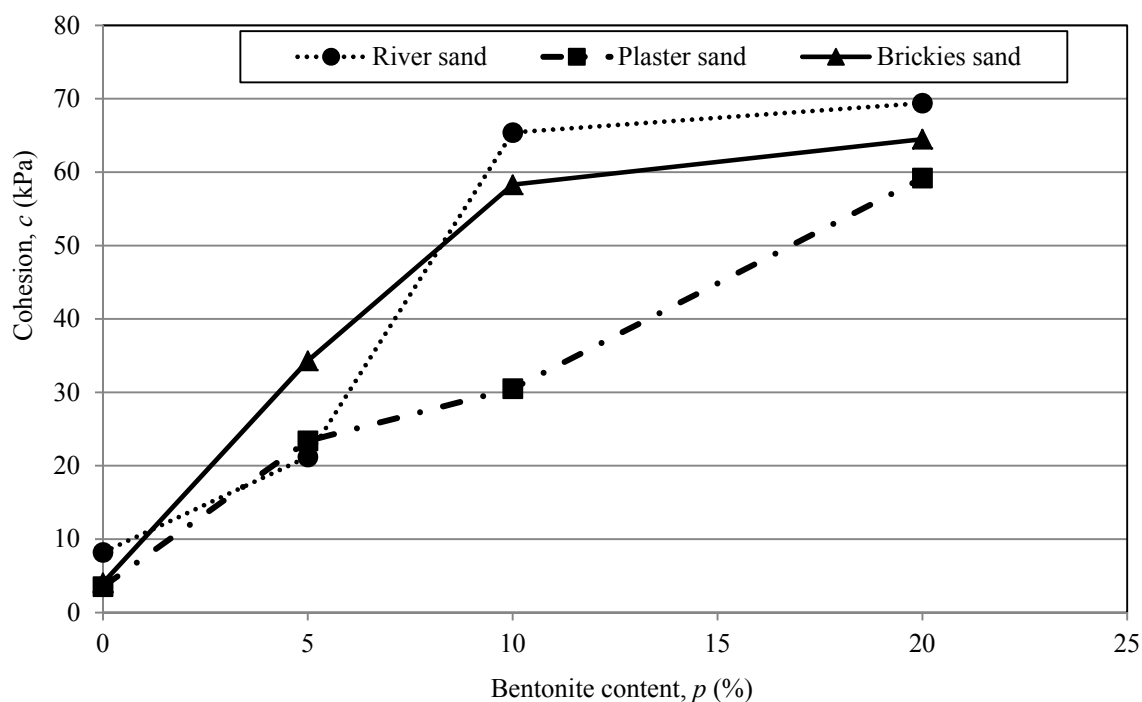


Figure 5.24 Bentonite content versus cohesion factor for all mixtures

5.9 Conclusions

A series of the following tests: unconfined compressive test and direct shear test were carried out on different sand-bentonite mixtures containing three types of Perth sands, namely brickies sand (BrS), river sand (RS) and plaster sand (PS), and three percentages of bentonite (B) (5%, 10%, and 20%). The results obtained from strength tests indicate that the unconfined compressive strength, the cohesion and the Young's modulus increase with the increase in bentonite content, while the angle of internal friction decreases.

CHAPTER SIX

METHODS OF ESTIMATING COEFFICIENT OF CONSOLIDATION

6.1 General

The assessment of settlement and permeability behaviour of a soil under pressure applied in increments requires evaluating the coefficient of consolidation (c_v) using one-dimensional consolidation test, which produces data of compression versus time. Different analytical methods have been suggested in the literature for estimating c_v . Most of these methods are graphical and based on Terzaghi's one-dimensional consolidation theory. Two of these methods are standards and widely used namely: Casagrande logarithm of time fitting method (Casagrande and Fadum 1940, Taylor 1948) and Taylor square root of time fitting method (Taylor 1948). Other methods are analytical method (Sivaram and Swamee 1977), improved rectangular hyperbola fitting (Sridharan and Prakash 1985), velocity (Perkin, 1978), inflection point (Cour, 1971), and revised logarithm of time fitting (Robinson and Allam, 1996). As the values of c_v produced by these methods vary broadly; it is hard for engineers to select the most reasonable method. In this chapter, an attempt is made to compare between the first four methods, which were used in Chapter 6 to calculate the permeability for sand/bentonite mixtures, to select the most reliable method. Other methods were disregarded because they are required settlement-time relationships having S-shape curves to be applicable (Shukla et al. 2009) which have not been obtained for all mixtures.

6.2 Terzaghi's One-Dimensional Consolidation Theory

Terzaghi (1925) presented the first theory regarding the time rate of one-dimensional consolidation of a saturated cohesive soil under the applied load (Das, 2013). Terzaghi expressed the theory in the form of a diffusion equation which is shown in Eq. (6.1). The equation describes the rate of change in the excess pore water pressure to the sample depth; it is applicable to a cohesive soil which is laterally confined and subjected to a sustained load.

$$c_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} \quad (6.1)$$

where:

u = excess pore water pressure,

z = depth,

t = time

Equation (6.1) was simplified to describe the relationship between average degree of consolidation (U) and time factor (T) as shown in Eq. (6.2).

$$U = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp(-M^2 T) \quad (6.2)$$

where:

$$M = \frac{\pi}{2}(2m + 1)$$

$$T = \frac{c_v t}{H^2}$$

U = average degree of consolidation (It is defined as a ratio of the compression at any time to the ultimate compression, the compression at the end of primary consolidation)

T = time factor

c_v = coefficient of consolidation

H = maximum drainage distance

6.3 Methods of Estimating (c_v)

6.3.1 Casagrande Logarithm of Time Fitting Method

This method was derived from Terzaghi's one-dimensional consolidation theory by Casagrande and Fadum (1940). The method is used in most standards; it relies on the plots of the relationship between compression dial readings (δ) and logarithm of time (t) of consolidation curve. The procedures of Casagrande method are described below and shown in Figure 6.1 (Head, 1982):

1. Plot the compression (δ) versus Log t curve for each load increment.
2. Find the compression which represents 0% primary consolidation.
3. Draw a tangent to the inflection point and a tangent to the steepest part in the secondary consolidation part.
4. Find the compression which represents 100% consolidation from the secondary compression part by plotting the intersection between the two tangents.
5. Estimate the time for 50% primary consolidation.
6. Calculate the coefficient of consolidation using Eq. (6.2).

$$c_v = \frac{0.197H^2}{t_{50}} \quad (6.2)$$

where:

t_{50} = time at 50% primary consolidation

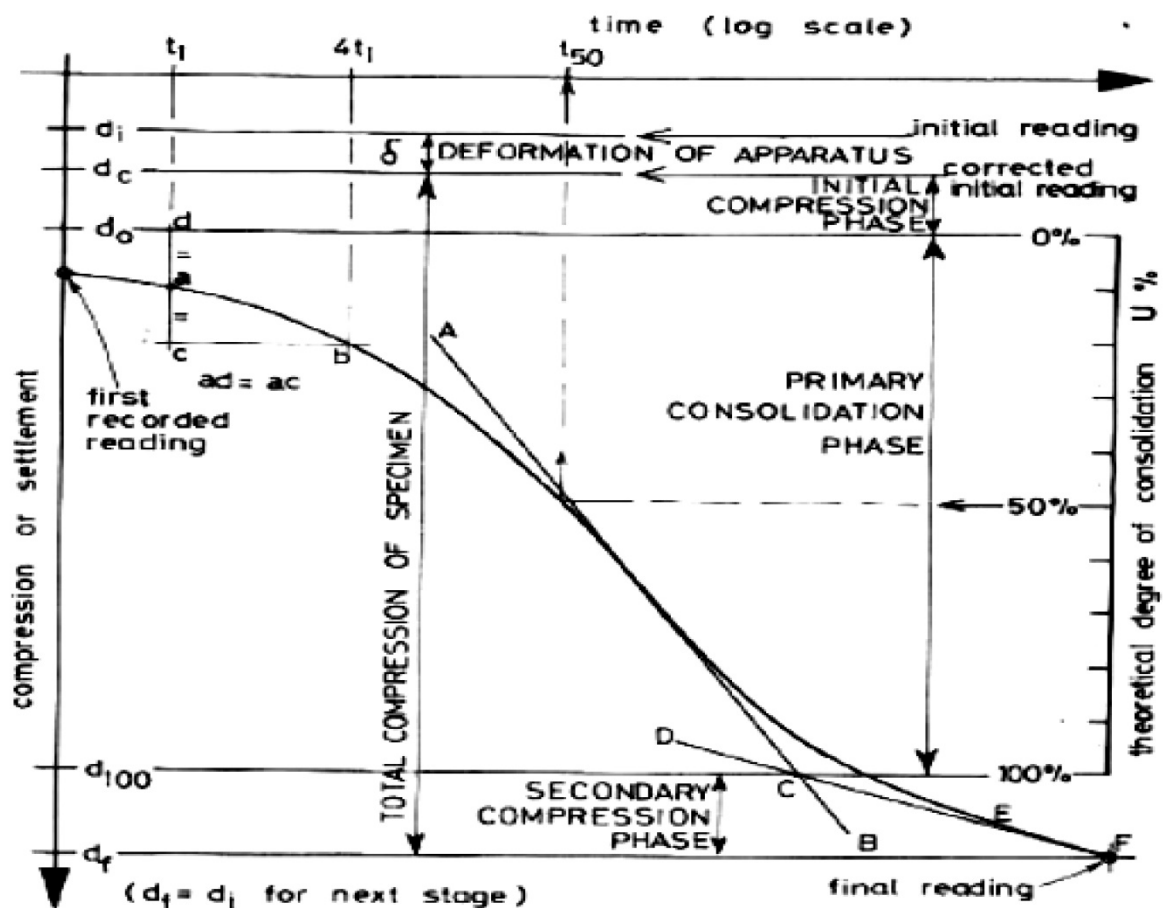


Figure 6.1 Casagrande logarithm of time method for estimating c_v (After Head, 1982)

6.3.2 Taylor Square Root of Time Fitting Method

This method has been proposed by Taylor (1948); the procedures of this method are reported in below and shown in Figure 6.2:

1. Plot the compression (δ) versus square root of time (t) curve for each load increment.
2. Draw a line through the initial straight portion of the compression curve; the intersection of the line with the compression-axis represents the compression at 0% primary consolidation and with the abscissa represents A .
3. Estimate a point such as B on the abscissa that indicates a distance of $(1.15\sqrt{t_A})$
4. Draw another line between the 0% primary consolidation point and point B ; the intersection with the compression curve indicates the compression at 90% primary consolidation.
5. Estimate the time for 90% primary consolidation.
6. Calculate the coefficient of consolidation using Eq. (6.3).

$$c_v = \frac{0.848H^2}{t_{90}} \quad (6.3)$$

where:

t_{90} = time at 90% primary consolidation

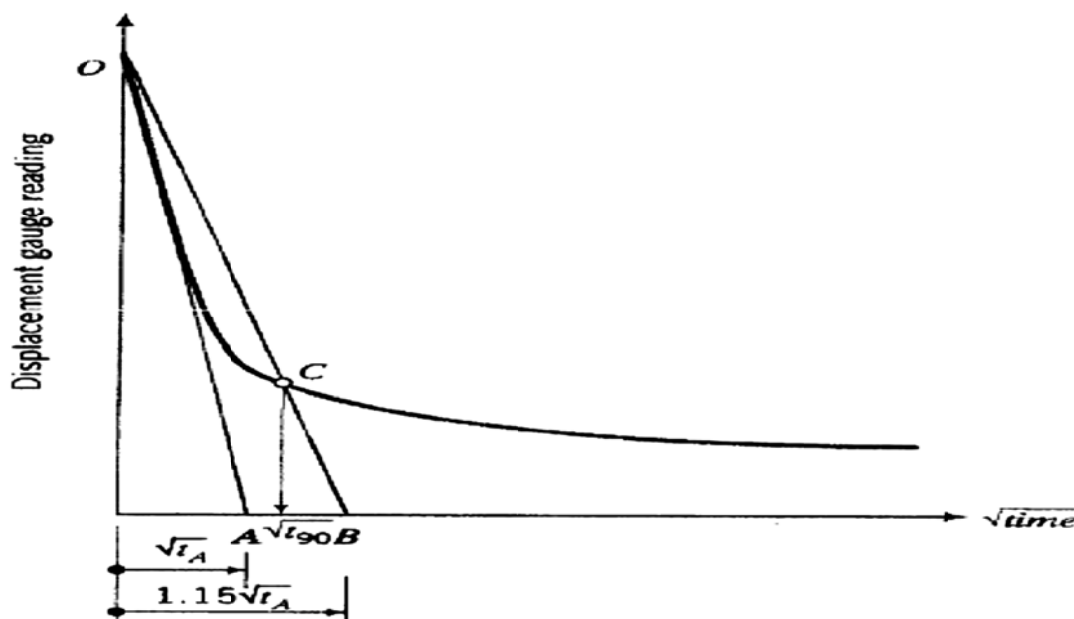


Figure 6.2 Taylor square root of time fitting method for estimating c_v (After Taylor, 1948)

6.3.3 Analytical Method

The analytical method used for calculating c_v was proposed by Sivaram and Swamee (1977). The method utilizes three incremental loading readings resulted from the consolidation test. The required equations for this method are reported in the following Equations (Eqs 6.4 - 6.9):

$$T_1 = \frac{c_v t_1}{H^2} = \frac{\pi}{4} \left(\frac{\delta_1 - \delta_i}{\delta_f - \delta_i} \right)^2 \quad (6.4)$$

$$T_2 = \frac{c_v t_2}{H^2} = \frac{\pi}{4} \left(\frac{\delta_2 - \delta_i}{\delta_f - \delta_i} \right)^2 \quad (6.5)$$

$$T_3 = \frac{c_v t_3}{H^2} = \frac{\pi}{4} \frac{\left(\frac{\delta_3 - \delta_i}{\delta_f - \delta_i} \right)^2}{\left[1 - \left(\frac{\delta_3 - \delta_i}{\delta_f - \delta_i} \right)^{5.6} \right]^{0.357}} \quad (6.6)$$

$$\delta_i = \frac{\delta_1 - \delta_2 \sqrt{\frac{t_1}{t_2}}}{1 - \sqrt{\frac{t_1}{t_2}}} \quad (6.7)$$

$$\delta_f = \delta_i - \frac{\delta_i - \delta_3}{\left[1 - \left\{ \frac{(\delta_3 - \delta_i)(\sqrt{t_2} - \sqrt{t_1})}{(\delta_2 - \delta_1)\sqrt{t_3}} \right\}^{5.6} \right]^{0.179}} \quad (6.8)$$

$$c_v = \frac{\pi}{4} \left[\left(\frac{\delta_2 - \delta_1}{\delta_f - \delta_i} \right) \left(\frac{H}{\sqrt{t_2} - \sqrt{t_1}} \right) \right]^2 \quad (6.9)$$

where:

δ_1, δ_2 = two reading in the early phase of consolidation, corresponding to t_1 and t_2 , respectively.

δ_3 = the third reading taking at t_3 .

δ_i, δ_f = initial and final readings, respectively.

T_1, T_2, T_3 = time factors corresponding to δ_1, δ_2 and δ_3 respectively.

6.3.4 Improved Rectangular Hyperbola Fitting Method

The fourth method was suggested by Sridharan and Prakash (1985). They improved and simplified the rectangular hyperbola method proposed by Sridharan and Rao (1981). Sridharan and Rao assumed that the relationship between the average degree of consolidation and theoretical time factor, as suggested by Terzaghi theory, to be a rectangular hyperbola. The improved method includes the following procedures (Figure 6.3):

1. Plot the t versus t/δ curve.
2. Draw a tangent line for the curve.
3. Find the equation for the tangent line.
4. Estimate D and m required for calculating c_v
5. Estimate c_v from Eq. (6.10).

$$c_v = \frac{0.24mH^2}{c} \quad (6.10)$$

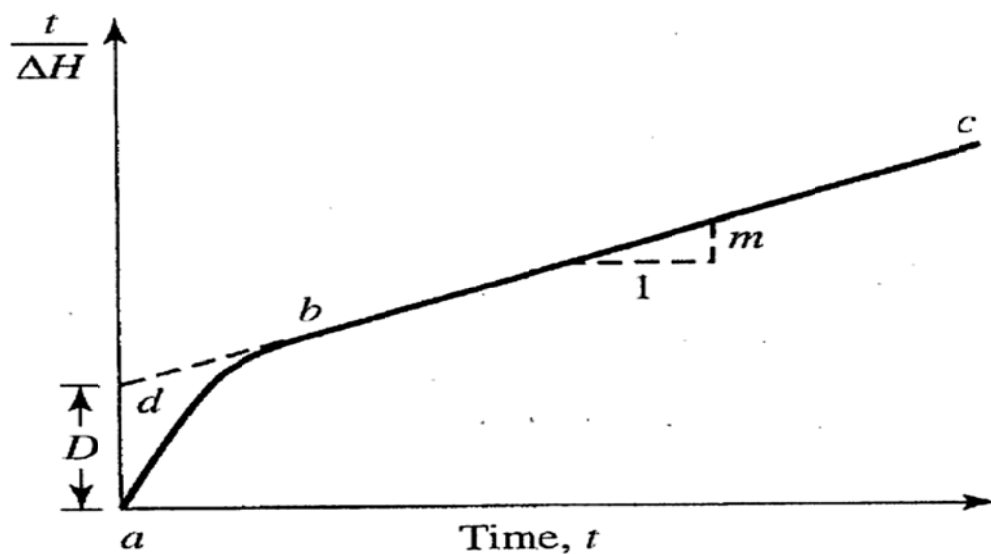


Figure 6.3 Improved rectangular hyperbola method for determining coefficient of consolidation (Das, 2002)

6.4 Results and Discussion

The coefficients of consolidation versus the logarithm effective stress for 5RS-B, 10RS-B, 20RS-B 5BrS-B, 10BrS-B, 20BrS-B, 5PS-B, 10PS-B, and 20PS-B, mixtures were estimated using the four methods and plotted in Figures 6.1 - 6.9, respectively. The results of coefficients of consolidation show that most of the coefficients decrease with the increase in the effective stress for any mixture. This result is in agreement with some previous research (Ameta and Wayal 2008). Further, the descending order of the methods in terms of the value of coefficient of consolidation is as follows: improved rectangular hyperbola fitting, Taylor square root of time fitting method, Casagrande logarithm of time fitting method, and analytical method. Therefore, in this study the improved rectangular hyperbola method was selected and considered as a conservative and safe method for calculating the coefficient of consolidation which was used later on for estimating the coefficients of permeability.

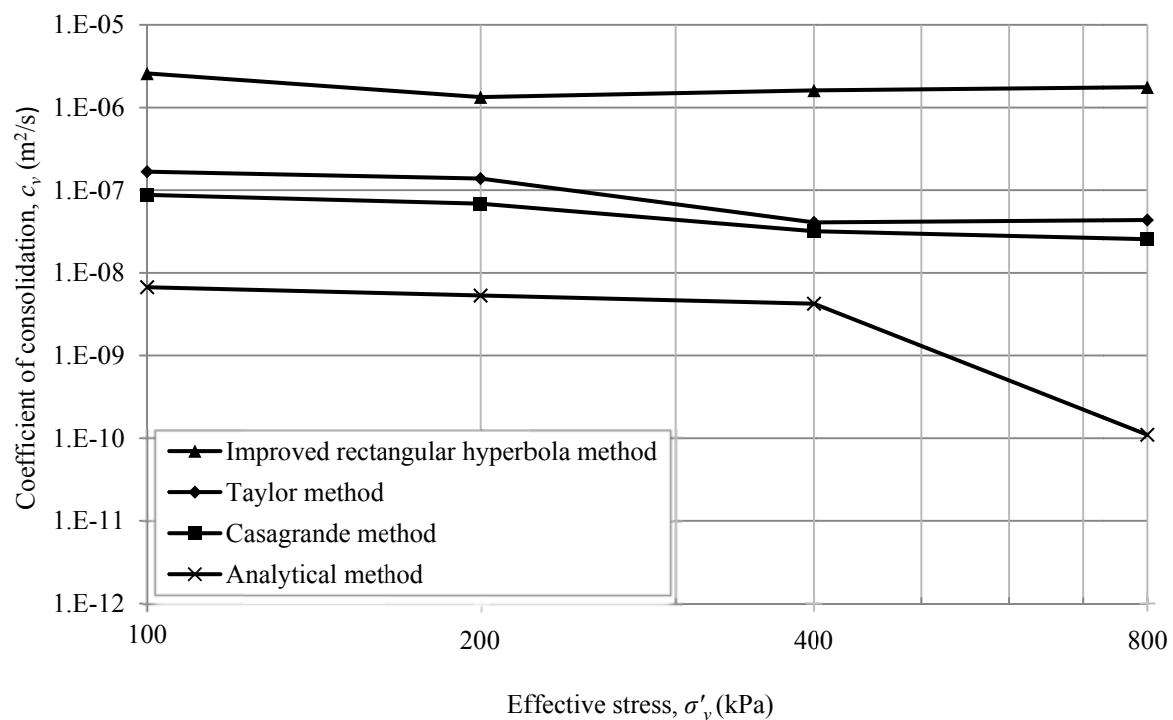


Figure 6.4 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% river sand-bentonite mixture

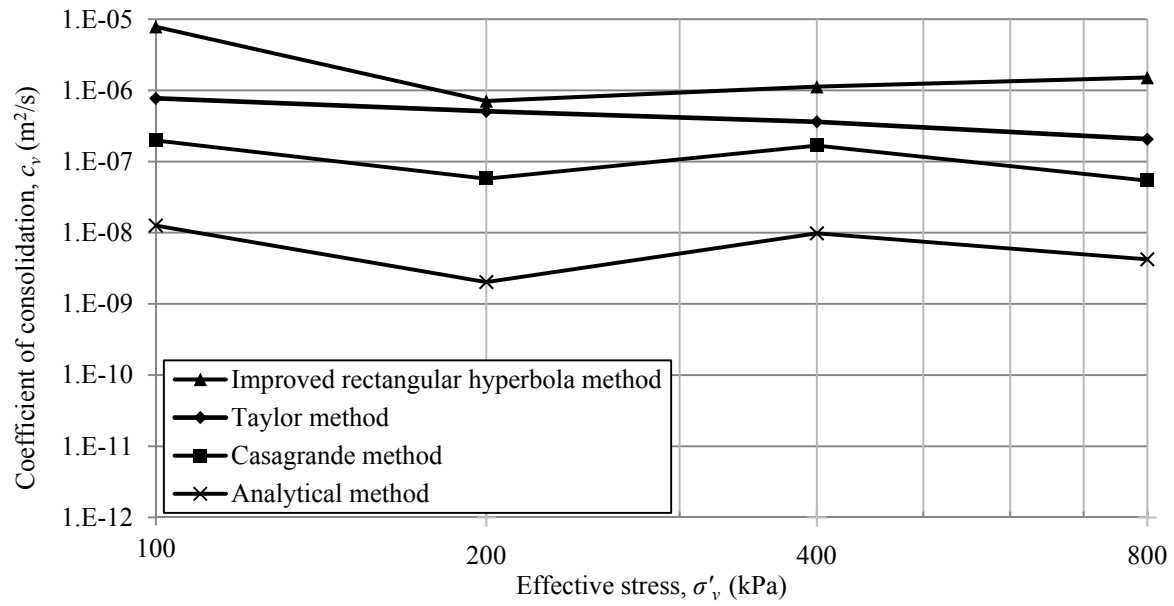


Figure 6.5 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% river sand-bentonite mixture

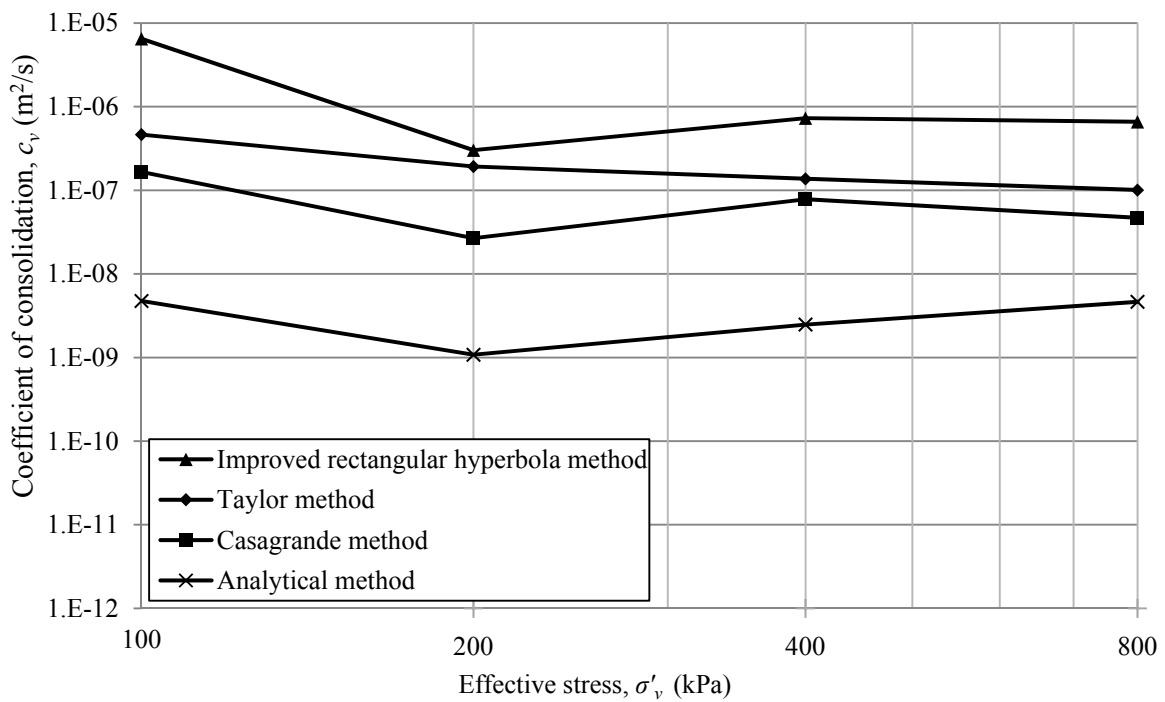


Figure 6.6 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% river sand-bentonite mixture

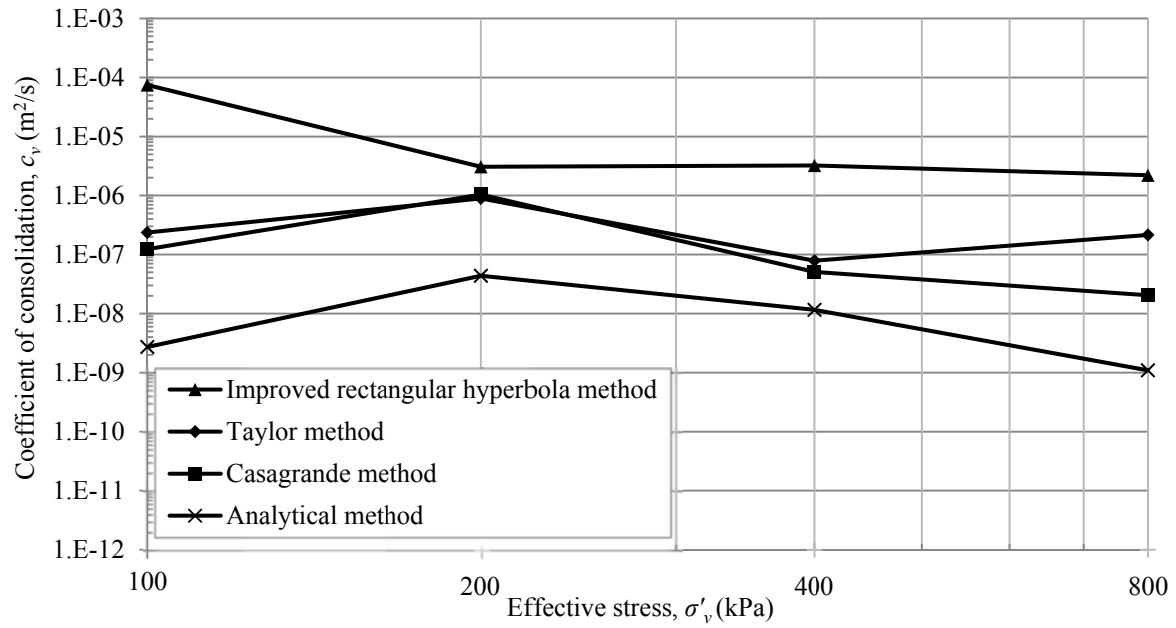


Figure 6.7 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% plaster sand-bentonite mixture

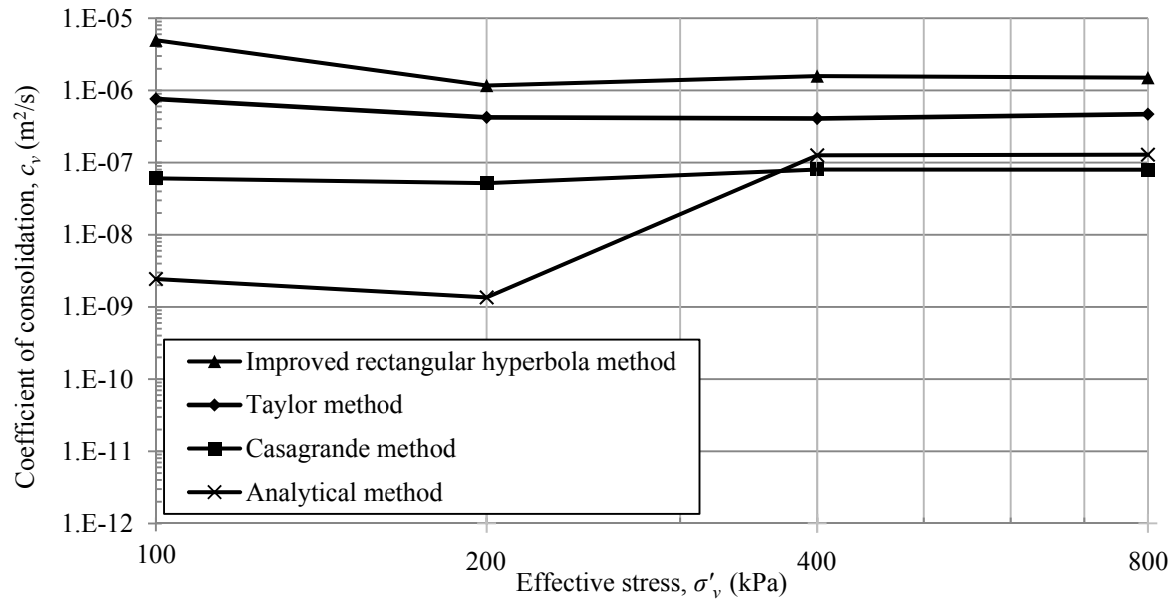


Figure 6.8 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% plaster sand-bentonite mixture

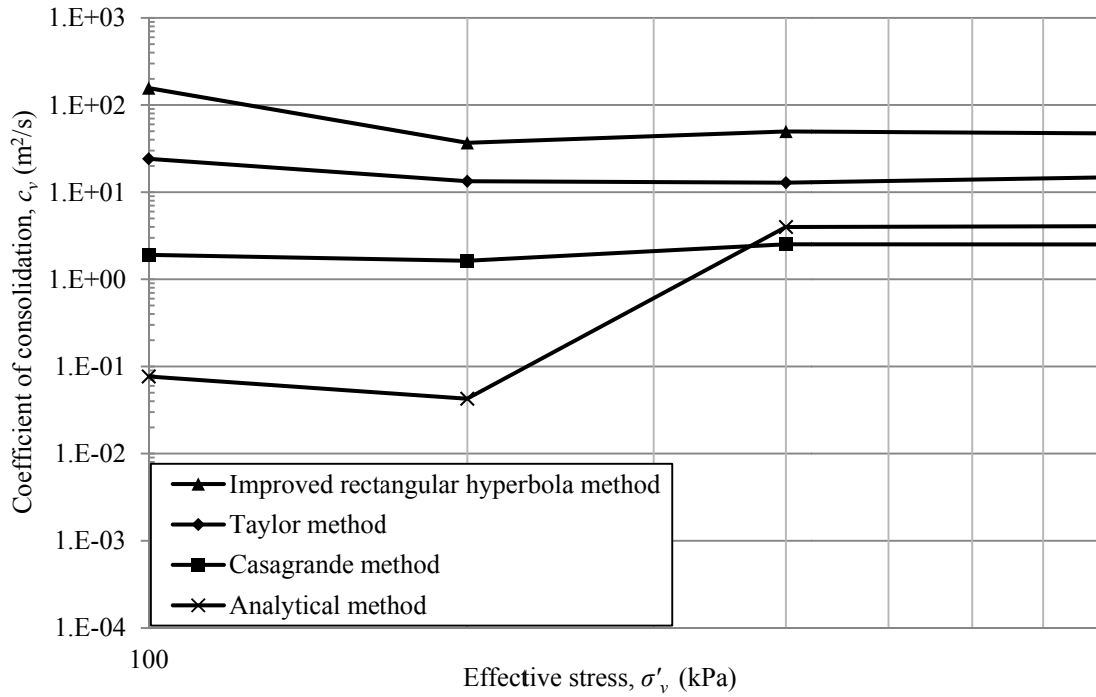


Figure 6.9 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% plaster sand-bentonite mixture

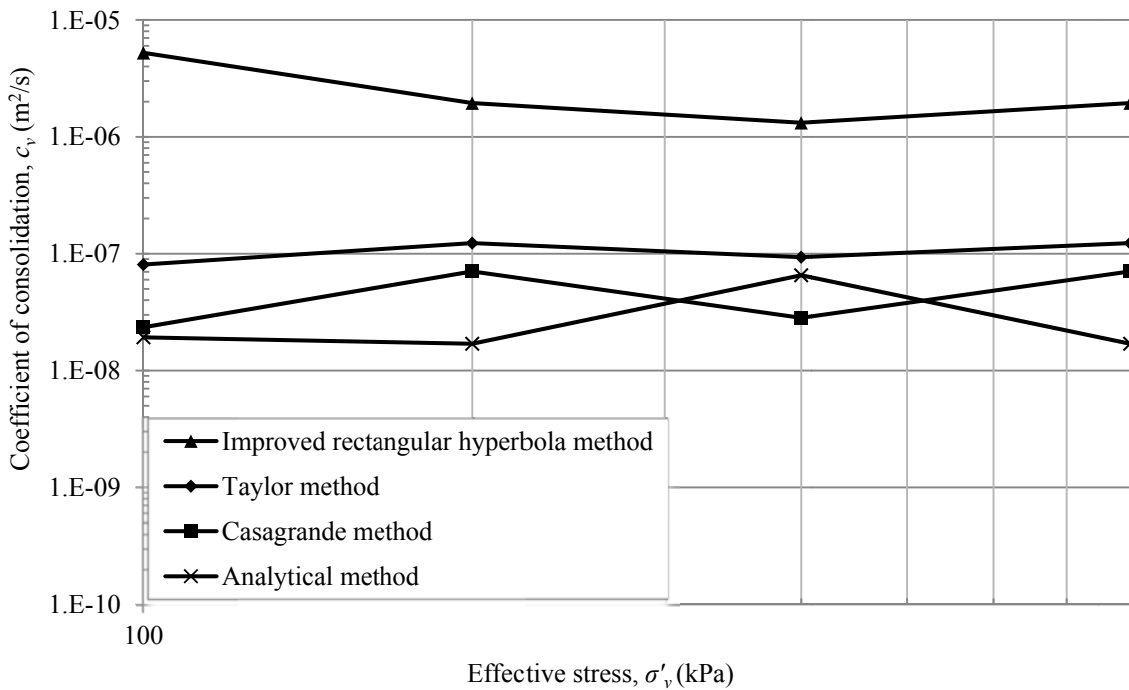


Figure 6.10 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 5% brickies sand-bentonite mixture

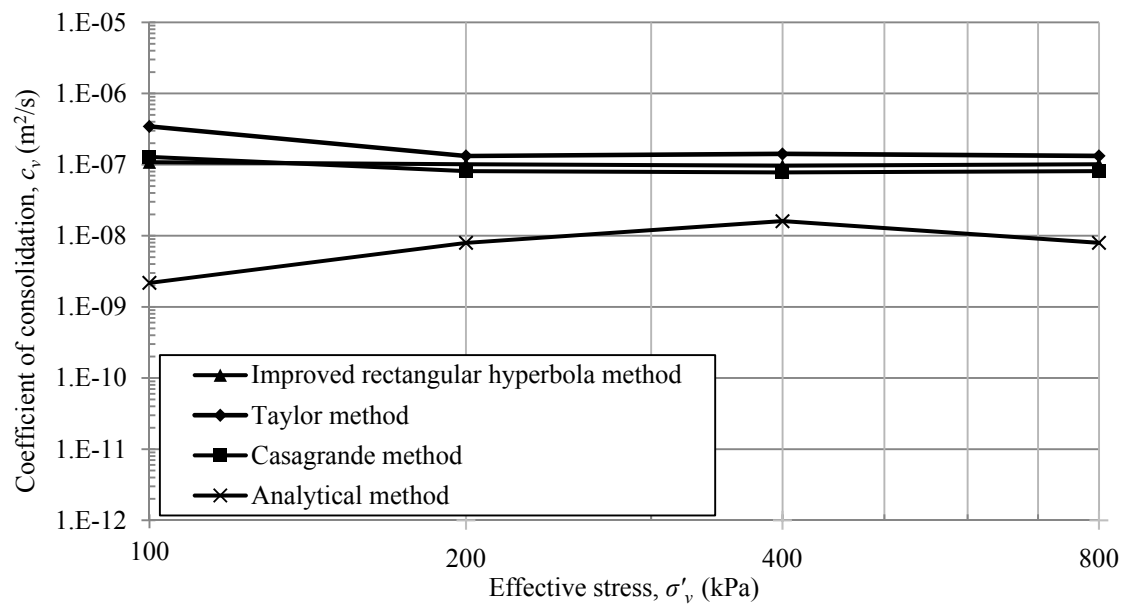


Figure 6.11 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 10% brickies sand-bentonite mixture

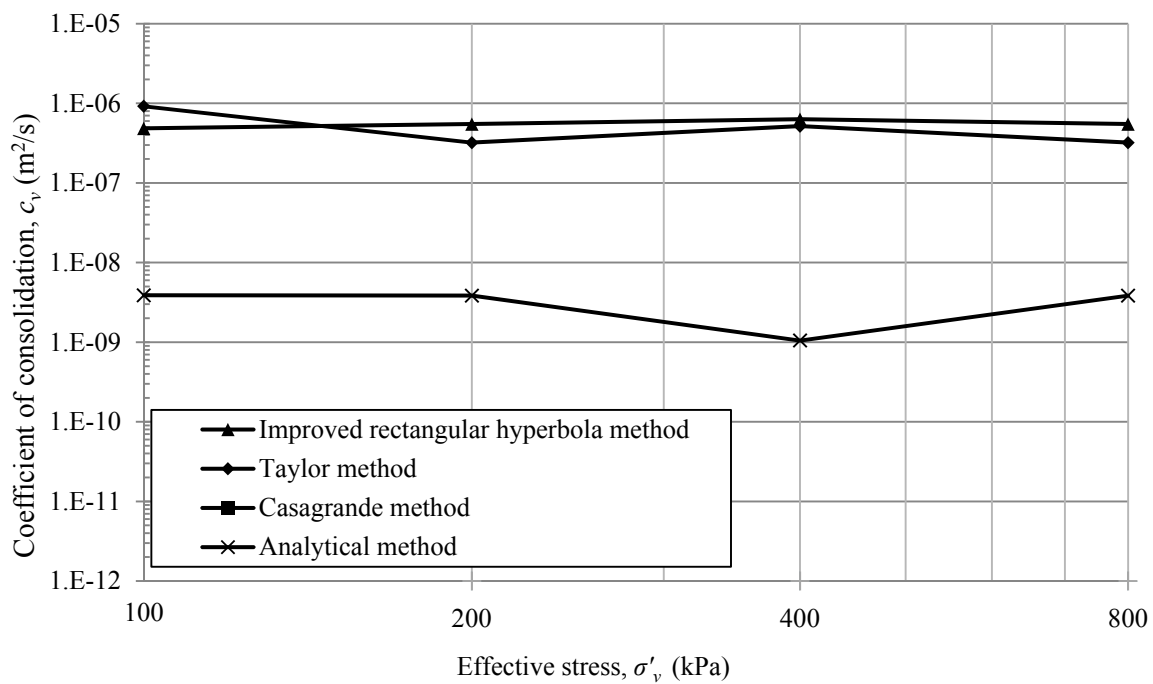


Figure 6.12 Coefficient of consolidation (c_v) estimated by using four methods versus logarithm effective stress (σ'_v) for 20% brickies sand-bentonite mixture

6.5 Conclusions

Four methods, namely Casagrande logarithm of time fitting method, Taylor square root of time fitting method, analytical method and improved rectangular hyperbola fitting, are compared for estimating the coefficient of consolidation of sand-bentonite mixture. The analysis shows that the improved rectangular hyperbola method is the most reliable method for calculating the coefficient of consolidation among the four methods.

CHAPTER SEVEN

SUMMARY AND CONCLUSIONS

7.1 Summary

A series of the following tests: one-dimensional consolidation tests, unconfined compressive test and direct shear test were carried out on different sand-bentonite mixtures containing three types of Perth sands, namely brickies sand (BrS), river sand (RS) and plaster sand (PS), and three percentages of bentonite (B) (5%, 10%, and 20%), in order to investigate the effect of the variation in particle-size distribution on the compressibility, permeability, and strength characteristics behaviour of sand/bentonite mixtures and also to describe the trends of change in these characteristics with regards to bentonite content. Four different methods for estimating the coefficients of consolidation were used and examined in this study.

7.2 Conclusions

Based on the results and discussion obtained from standard compaction tests, one-dimensional consolidation tests, unconfined compression strength tests, and direct shear tests the following conclusions are made:

7.2.1 Compaction behaviour

1. For all types of the mixtures studied, the maximum dry unit weight increases as the bentonite content increases. Also, for a particular bentonite content, the maximum dry unit weight increases as the fine particles in the sands increase.
2. The optimum moisture content increases with initial bentonite content of 5%, and then decreases with further increase.

7.2.2 Compressibility and permeability behaviour

1. The void ratio of the sand-bentonite mixture decreases with increasing bentonite content for all three types of sand used in this study.
2. The compressibility behaviour of the brickies sand-bentonite (BrS-B) mixtures changed significantly with increasing bentonite content more than the change in the other mixtures.
3. The highest range of void ratio can be found in RS-B mixtures is about 0.64 to 0.52, however, the lowest range of void ratio is about 0.43 to 0.14 in the BrS-B mixtures and finally, in the case of PS-B mixtures, the range becomes from about 0.49 to 0.35.
4. The compression index increases linearly with increasing bentonite content for any type of sand-bentonite mixture and the slope of the increase in the compression index of BrS-B mixtures is about four times more than the compression index of RS-B and 1.5 times more than the compression index of PS-B .
5. The permeability decreases with increasing effective stress in any mixture, and also decreases with an increase in the bentonite content.
6. All the sand-bentonite mixtures except RS-B and PS-B mixtures which have 5% bentonite content can meet the requirements of the hydraulic barrier in terms of permeability property. Further, the optimum amount of bentonite which should be added to the sand as a hydraulic barrier is as follows: 5% for brickies sand, and 10% for river sand, and 10% plaster sand.

7.2.3 Strength behaviour

7.2.3.1 unconfined compressive strength

1. The unconfined compressive strength increases approximately linearly with the bentonite content for the three types of mixtures. However, for the BrS-B mixture, after the bentonite content exceeds 10%, the increase is much smaller.
2. The maximum vertical strain increases linearly with bentonite content for all mixtures except for PS-B mixture for which with bentonite content of 10%, it decreases significantly.
3. The slope of the failure planes under unconfined loading decreases approximately linearly with the bentonite content; however, there is no change after the bentonite exceeds 10% in RS-B and PS-B mixtures.
4. As the bentonite content increases, Young's modulus of elasticity also increases except for 20BrS-B and 20RS-B mixtures. The RS-B mixture has the highest Young's modulus of elasticity.

7.2.3.2 direct shear

1. For all types of the mixtures in this study, the maximum shear strength (τ_{\max}) increases linearly with the normal stresses; however, the distribution of τ_{\max} with respect to the bentonite content is different from one load to another as follows: almost merging at 100 kPa, no consistency at 200 kPa, and decreasing with bentonite content at 300 kPa.
2. The angle of internal friction decreases linearly with increasing bentonite content for all three cases. The RS-B has the highest rate of decrease.
3. For all mixture types, the cohesion increases linearly as the bentonite content increases.

7.2.4 Methods of estimating c_v

Four different methods for estimating the coefficient of consolidation (c_v) of sand-bentonite mixtures were compared to find the most suitable method among them. The result produced a descending order in terms of the values of c_v , as follow: improved rectangular hyperbola fitting method, Taylor square root of time fitting method, Casagrande logarithm of time fitting method, and analytical method. It can be noticed also that the improved rectangular hyperbola fitting method produced the most consistent values of c_v . Therefore, the improved rectangular hyperbola method is the most reliable method for calculating the coefficient of consolidation and hence the permeability among the four methods.

7.3 Recommendations for Future Works

The objective of this study is to investigate the engineering characteristics of Perth sand-bentonite mixtures as liners/barriers in some engineering applications. Investigating the engineering characteristics, such as permeability and strength helps to support recommendation for a cost-effective liner/barrier material with three different local soils. Some recommendations for future works can be presented as follows:

- Investigating the permeability for the sand-bentonite mixtures using another methods, such as falling head using a compaction permeameter and standard triaxial cell; and drawing a comparison between the new results with the results presented in this study.
- Investigating the swelling characteristics of the sand-bentonite mixtures by conducting swelling tests.
- Investigating the permeability and strength characteristics of coastal Perth sands, which is very fine sand, with bentonite as liners/barriers for the engineering applications.

REFERENCES

- Abdelrahman, G.E. and Shahien M.M., 2004. Swelling Treatment by Using Sand for Tamia Swelling Soil. *Forth International Engineering Conference (4th IEC)*, Egypt: Sharm El Sheikh.
- Abeele, W.V., 1986. The influence of bentonite on the permeability of sandy silts. *Nuclear And Chemical Waste Management*, 6(1), 81-88.
- Akgün, H., Koçkar, M. K., and Aktürk, Ö., 2006. Evaluation of a compacted bentonite/sand seal for underground waste repository isolation. *Environmental Geology*, 50(3), 331-337.
- Alkaya, D. and Esener, A.B., 2011. Usability of sand-bentonite-cement mixture in the construction of unpermeable layer. *Scientific Research And Essays*, 6(21), 4492-4503.
- Alther, G., 1982. The role of bentonite in soil sealing applications. *Bulletin of the Association of Engineering Geologists*, 18(4).
- Alther, G.R., 1987. The qualifications of bentonite as a soil sealant. *Engineering Geology*, 23(3), 177-191.
- Amadi, A.A. and Eberemu, A.O., 2012. Delineation of compaction criteria for acceptable hydraulic conductivity of lateritic soil-bentonite mixtures designed as landfill liners. *Environmental Earth Sciences*, 67(4), 999-1006.
- American Society for Testing and Materials, 2013. ASTM D2166/D2166M - 13, Standard test method for unconfined compressive strength of cohesive soil.
- Ameta, N. and Wayal, A.S., 2008. Effect of bentonite on permeability of dune sand. *Electronic Journal of Geotechnical Engineering*, 13, 1-7.
- Australian Standards, 2000. Australian Standard: Methods of Testing Soils for Engineering Purposes – General Requirements and List of Methods (AS1289.0-2000), Sydney, Australia.
- Barrier Containment Technologies for Environmental Remedial Applications, 1995. John Wiley & Sons, Inc., R. R. Rumer and M. E. Ryan, eds., New York.
- Baxter, D.Y., 2000. *Mechanical behavior of soil-bentonite cutoff walls*. Thesis (PhD). Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
- Borgesson, L., Johannesson, L. E., and Gunnarsson, D., 2002. Influence of soil structural inhomogeneities on the behaviour of backfill materials based on mixtures of bentonite and crushed rock. *In Workshop on Clay Microstructure and its Importance to Soil Behaviour*, Lund, Sweden, October 15th to 17th.
- Casagrande, A. and Fadum, R.E., 1940. Notes on soil testing for engineering purposes.
- Chalermyanont, T. and Arrykul, S., 2005. Compacted sand-bentonite mixtures for hydraulic

- containment liners. *Songklanakarin Journal of Science and Technology*, 27(2), 313-323.
- Chapuis, R.P., 1981. Permeability testing of soil-bentonite mixtures. *Proceeding, 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, vol. 4, pp. 744-745.
- Chapuis, R.P., 1990. Sand-bentonite liners: predicting permeability from laboratory tests. *Canadian Geotechnical Journal*, 27(1), 47-57.
- Council Directive 99/31/EC of 1999 on Landfill of waste.
- Chen, Y. and Meehan, C.L., 2011. Undrained strength characteristics of compacted bentonite/sand mixtures. *Geo-Frontiers 2011: Advances in Geotechnical Engineering. ASCE*, pp 2699–2708.
- Cho, W.J., Lee, J.O., and Kang, C.H., 2002. A compilation and evaluation of thermal and mechanical properties of bentonite-based buffer materials for a high-level waste repository. *Journal-Korean Nuclear Society*, 34(1), 90-103.
- Daniel, D.E., 1990. Summary review of construction quality control for compacted soil liners. Waste containment system: construction, regulation, and performance. *ASCE*, pp. 175-189.
- Daniel, D.E., 1994. State of the art: Laboratory hydraulic conductivity tests for saturated soils, *ASTM Special Technical Publication*, 1142, 30-30.
- Das, B.M., 1983. Advanced soil mechanics. Hemisphere Publishing Corp., McGraw Hill, London.
- Das, B.M., 2002. Principles of geotechnical engineering, 5th Edition, Thomson Brooks/Cole.
- Das, B.M., 2013. Fundamentals of geotechnical engineering. Global Engineering: Christopher M. Shortt Senior. McGraw Hill, USA.
- D'Appolonia, D.J., 1980. Soil-bentonite slurry trench cutoffs. *Journal of the Geotechnical Engineering Division*, 106(4), 399-417.
- De Magistris, F.S., Silvestri, F., and Vinale, F., 1998. Physical and mechanical properties of a compacted silty sand with low bentonite fraction. *Canadian geotechnical journal*, 35(6), 909-925.
- Department of environmental, 2005. Siting, design, operation and rehabilitation of landfills. Australia: Government of Western Australia.
- Dixon, D.A. and Gray, M.N., 1985. The engineering properties of buffer material—research at Whiteshell Nuclear Research Establishment. *Proceedings of the 19th Information Meeting of The Nuclear Waste Management Program*. Paper presented at the Atomic Energy of Canada Limited Technical Report., Toronto.
- Dixon, D.A., Gray, M.N., and Thomas, A.W., 1985. A study of the compaction properties of

- potential clay-sand buffer mixtures for use in nuclear fuel waste disposal, *Engineering Geology*, 21, p. 247–255.
- Elkady, T.Y., Shaker, A.A., and Dhowain, A.W., 2014. Shear strengths and volume changes of sand–attapulgite clay mixtures. *Bulletin of Engineering Geology and the Environment*, 1–15.
- Evans, J. C., 1993. Vertical cutoff walls. *Geotechnical Practice for Waste Disposal*, Chapter 17, D. E. Daniel, ed., Chapman & Hall, London, 430-454. Chapter 17, D. E. Daniel, ed., Chapman & Hall, London, 430-454.
- Fan, R., Du, Y., Liu, S., and Chen, Z., 2014. Compressibility and hydraulic conductivity of sand/clay-bentonite backfills. *geoenvironmental engineering*, 21-30.
- Garlanger, J.E., Cheung, F.K., and Tannous, B.S., 1987. Quality control testing for a sand-bentonite liner. *Paper presented at the Geotechnical practice for waste disposal '87*.
- Gleason, M.H., Daniel, D.E., and Eykholt, G.R., 1997. Calcium and sodium bentonite for hydraulic containment applications. *Journal of Geotechnical and Geoenvironmental Engineering*, 123(5), 438-445.
- Grim, R., and Guven, N., 1978. Bentonites: geology, mineralogy, properties, and uses. *Elsevier Science Publishing Co., Inc.*, New York, N.Y.
- Gueddouda, M.K., Lamara, M.d., Aboubaker, N., and Taibi, S., 2008. Hydraulic conductivity and shear strength of dune sand-bentonite mixtures. *Electronic Journal of Geotechnical Engineering*, 13, 1-15.
- Haug, M.D., and Wong, L.C., 1992. Impact of molding water content on hydraulic conductivity of compacted sand-bentonite. *Canadian Geotechnical Journal*, 29(2), 253-262.
- Head, K., 1982. *Permeability, shear strength and compressibility tests*. London: Pentech Press. Vol. 2.
- Holtz, W.G., and Lowitz, C.W., Compaction Characteristics of Gravelly Soils. Special Technical Publication No. 232. ASTM, Philadelphia, pp. 70-83.
- Howell, J.L., Shackelford, C.D., Amer, N.H., and Stern, R.T., 1997. Compaction of sand-processed clay soil mixtures. *ASTM geotechnical testing journal*, 20(4), 443-458.
- Japan Nuclear Cycle Development Institute, 1999a. H12: Project to establish the scientific and technical basis for HLW disposal in Japan, Project overview report.
- Japan Nuclear Cycle Development Institute, 1999b. H12: Project to establish the scientific and technical basis for HLW disposal in Japan, Supporting report 2. Repository design and engineering technology.
- Jones, C.W., 1954. The permeability and settlement of laboratory specimen's of sand and sand

- gravel mixtures. *Philadelphia American Society for Testing and Materials*. STP 163, pp 6878.
- Kenney, T.C., Veen, W.A.V., Swallow, M.A., and Sungaila, M.A., 1992. Hydraulic conductivity of compacted bentonite-sand mixtures. *Canadian Geotechnical Journal*, 29(3), 364-374.
- Komine, H., 2004. Simplified evaluation on hydraulic conductivities of sand–bentonite mixture backfill. *Applied Clay Science*, 26(1), 13-19.
- Komine, H., 2008. Theoretical equations on hydraulic conductivities of bentonite-based buffer and backfill for underground disposal of radioactive wastes. *Journal of geotechnical and geoenvironmental engineering*, 134(4), 497-508.
- Komine, H., 2010. Predicting hydraulic conductivity of sand–bentonite mixture backfill before and after swelling deformation for underground disposal of radioactive wastes. *Engineering Geology*, 114(3), 123-134.
- Komine, H. and Ogata, N., 1999. Experimental study on swelling characteristics of sand-bentonite mixture for nuclear waste disposal. *Soils and Foundations*, 39(2), 83-97.
- Kraus, J.F., Benson, C.H., Erickson, A.E., and Chamberlain, E.J., 1997. Freeze-thaw cycling and hydraulic conductivity of bentonitic barriers. *Journal of geotechnical and geoenvironmental engineering*, 123(3), 229-238.
- Mesri, G., and Olson, R. E., 1971. Mechanisms controlling the permeability of clays. *Clays and Clay Minerals*, 19, 151-158.
- Miller, E.A., and Sower, G.F., 1957. Strength characteristics of soil-aggregate mixtures. *Highway Research Board Bulletin*, 183:16-23.
- Mishra, A.K., Ohtsubo, M., Li, L.Y., and Higashi, T., 2010. Influence of the bentonite on the consolidation behaviour of soil–bentonite mixtures. *Carbonates and evaporites*, 25(1), 43-49.
- Mitchell, J. K. and Soga, K., 1976. *Fundamentals of soil behavior*. New York: Wiley.
- Mollins, L.H., Stewart, D.I. and Cousens, T.W., 1996. Predicting the properties of bentonite-sand mixtures. *Clay Minerals*, 31(2), 243-252.
- Mollins, L. H., 1996. *The design of bentonite-sand mixtures*. Thesis (PhD). University of Leeds, England.
- Ölmez, M.S., 2008. *Shear strength behavior of sand–clay mixtures*. MSc Thesis. Middle East Technical University, Ankara, Turkey.
- Olson, R.E. and Daniel, D.E., 1981. Measurement of the hydraulic conductivity of fine-grained soils. In: T.F. Zimmie, ed. Permeability and groundwater contaminant transport. *ASTM STP*, 746, 18-64.

- Ogata, N., Kosaki, A., Ueda, H., Asano, H., and Takao, H., 1999. Execution techniques for high level radioactive waste disposal. 4. Design and manufacturing procedure of engineered barriers.
- Pakbaz, M.S., and Moqaddam, A.S., 2012. Effect of sand gradation on the behavior of sand-clay mixtures. *International Journal Of GEOMATE: Geotechnique, Construction Materials And Environment*, 3(1), 325-331.
- Parker, R.J., Bateman S., and Williams, D., 1993. Design and management of landfills Geotechnical management of waste and contamination, Fell, Phillips & Gerard (eds), Balkema, Rotterdam, pp.209-252.
- Pusch, R., 1979. Highly compacted sodium bentonite for isolating rock-deposited radioactive waste products. *Nuclear Technology*, 45(2), 153-157.
- Pusch, R., 1992. Use of bentonite for isolation of radioactive waste products. *Clay Miner*, 27(3), 353-361.
- Reschke, A.E., and Haug, M.D., 1991. The physico-chemical properties of bentonites and the performance of sand-bentonite mixtures. *In Proc. 44th Canadian Geotechnical Conf* (pp. 62r1-62r10).
- Ryan, C.R., 1985. Slurry cutoff walls: applications in the control of hazardous wastes. Hydraulic Barriers in Soil and Rock, ASTM STP 874, A. I. Johnson, R. K. Frobels, N. J. Cavalli, and C. B. Pettersson, eds., ASTM, Philadelphia, Pa, 9-23.
- Sällfors, G. and Öberg-Högsta, A.L., 2002. Determination of hydraulic conductivity of sand-bentonite mixtures for engineering purposes. *Geotechnical & Geological Engineering*, 20(1), 65-80.
- Seed, H.B. and Chan, C.K., 1959. Structure and strength characteristics of compacted clays. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 85(5), 87-128.
- Shakoor, A. and Cook, B.D., 1990. The effect of stone content, size, and shape on the engineering properties of a compacted silty clay: *Assoc. Eng. Geol. Bull.*, v. XXVII, pp. 245-253.
- Shirazi, S.M., Kazama, H., Salman, F.A., Othman, F. and Akib, S., 2010. Permeability and swelling characteristics of bentonite. *International Journal of Physical Sciences*, 5(11), 1647-1659.
- Shukla, S., Sivakugan, N., and Das, B., 2009. Methods for determination of the coefficient of consolidation and field observations of time rate of settlement—an overview. *International Journal of Geotechnical Engineering*, 3(1), 89-108.
- Sivapullaiah, P.V., Sridharan, A., and Stalin, V.K., 2000. Hydraulic conductivity of bentonite-

- sand mixtures. *Canadian Geotechnical Journal*, 37(2), 406-413.
- Sivaram, B. and Swamee, P.K., 1977. A computational method for consolidation-coefficient. *The Japanese Geotechnical Society*, NII-Electronic Library Service 17(2), 48-52.
- Sridharan, A., and Rao, A. S., 1981. Rectangular hyperbola fitting method for one dimensional consolidation. *Geotechnical Testing Journal*, 4(4), 161-168.
- Sridharan, A. and Prakash, K., 1985. Improved rectangular hyperbola method for the determination of coefficient of consolidation. *Geotechnical Testing Journal*, 8(1), 37-40.
- Studds, P.G., Stewart, D.I. and Cousens, T.W., 1998, The effects of salt solutions on the properties of bentonite-sand mixtures. *Clay Minerals*, 33(4), 651-660.
- Taylor, D.W., 1948. *Fundamentals of soil mechanics*. New York: Wiley, 66(2), 161.
- Thériault P., (2000). *Etude de l'influence des métaux lourds sur la conductivité hydraulique de couches sable/bentonite*. Mémoire pour l'obtention du grade de maître èssciences (M.Sc.). Canada. Mai 2000. 119p.
- U.S. EPA., Resource Conservation and Recovery Act 1976, Regulations Subtitle D.
- U.S. EPA., 1995. Quality assurance and quality control for waste containment facilities. National Risk Management Research Laboratory, Cincinnati, OH.
- Wasti, Y. and Alyanak, I., 1968. kil muhtevasının zeminin davranışına tesiri. *İnşaat Mühendisleri Odası, Türkiye İnşaat Mühendisliği, Teknik Kongresi*. Ankara, Turkey.
- Watabe, Y., Yamada, K., and Saitoh, K., 2011. Hydraulic conductivity and compressibility of mixtures of Nagoya clay with sand or bentonite. *Géotechnique*, 61(3), 211-219.
- Yong, R.N., Boonsinsuk, P., and Wong, G., 1986. Formulation of backfill material for a nuclear fuel waste disposal vault. *Canadian Geotechnical Journal*, 23(2), 216-228.