Cyclic Shear Strength of Artificially Prepared Cementitious Sands and Their Liquefaction Susceptibility

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by

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It is hereby declared that this thesis or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Signature of the Candidate

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ABSTRACT

The present experimental investigation involves the study of liquefaction susceptibility of sands of varying coarseness collected from three selected locations of Bangladesh and the samples were designated as Soil A (FM = 3.17, D_{50} = 0.98 mm and F_{200} = 0%), Soil B (FM = 1.66, D_{50} = 0.37 mm and F_{200} = 3.4%) and Soil C (FM = 0.96, D_{50} = 0.21 mm and F_{200} = 15.15%). Collected sand samples were treated with lime, fly ash plus lime, rice husk ash plus lime, clay plus lime, bentonite and cement to generate cementitious properties in the sand samples. The samples were prepared by dry mixing and compacting them at 10% moisture content in a standard compaction mold of 71 mm diameter and 142 mm height at a constant relative density of 50% by moist tamping method. A series of isotropically consolidated cyclic and monotonic triaxial tests were performed on the soil specimens both in untreated and treated conditions. All the tests were conducted at an effective confining pressure of 50 kPa and frequency of 1 Hz with 10% double amplitude axial strain for cyclic test, and axial strain rate of 0.1 percent per minute for monotonic test. The tests were carried out on lime stabilized samples at 7 and 28 days with different cyclic stress ratio, whereas, with fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement stabilized samples they were carried out at 7 days with a cyclic stress ratio of 0.40. The testing procedure involved saturation, isotropic consolidation and a uniform sinusoidal compressive loading.

Experimental results revealed that the Soil A of fineness modulus 3.17 and mean particle size of 0.98 mm were more susceptible to liquefaction as compared to the Soil B and Soil C of fineness modulus of 1.66 and 0.96 and mean particle size of 0.37 mm and 0.21 mm. The cyclic shear resistance of lime stabilized sand specimens continues to increase with increasing lime percentages and curing periods. The number of cycles required to cause liquefaction were greater for cement, lime, fly ash-lime, rice husk ash-lime and clay-lime stabilized specimens whereas the bentonite stabilized sands have no significant resistance to liquefaction compared to untreated sands. From monotonic triaxial tests of 5 percent lime and 10 percent lime, fly ash-lime and rice husk-lime treated sands were found dense specimens where 0 and 2 percent lime treated sands behaves like loose specimens. The compaction effort was found greater for Soil B and Soil C than for Soil A. The compaction energy were decreases with lime, fly ash-lime, rice husk ash-lime, clay-lime and bentonite mixed sands than for untreated sands. The mixture of lime, cement, fly ash-lime, rice husk ash-lime and clay-lime with sands and water reacts and binds the sand grains together thus reduce soils permeability and improve soils pH and their liquefaction strength.
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## Glossary

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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>$a_{max}$</td>
<td>Peak ground horizontal acceleration</td>
</tr>
<tr>
<td>$B$</td>
<td>Skempton’s pore pressure coefficient</td>
</tr>
<tr>
<td>BNBC</td>
<td>Bangladesh National Building Code</td>
</tr>
<tr>
<td>$c$</td>
<td>Cohesion of soil</td>
</tr>
<tr>
<td>CAH</td>
<td>Calcium aluminate hydrate</td>
</tr>
<tr>
<td>$C_c$</td>
<td>Coefficient of curvature in relation to grain size curve</td>
</tr>
<tr>
<td>CD or S</td>
<td>Consolidated Drained test or Slow test</td>
</tr>
<tr>
<td>CEC</td>
<td>Cation Exchange Capacity</td>
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<tr>
<td>$CO_2$</td>
<td>Carbon dioxide</td>
</tr>
<tr>
<td>CRR</td>
<td>Cyclic Resistance Ratio</td>
</tr>
<tr>
<td>CSH</td>
<td>Calcium silicate hydrate</td>
</tr>
<tr>
<td>CSR</td>
<td>Cyclic Stress Ratio</td>
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<tr>
<td>$C_u$</td>
<td>Uniformity coefficient in relation to grain size curve</td>
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<tr>
<td>CU or R</td>
<td>Consolidated Undrained test</td>
</tr>
<tr>
<td>DA</td>
<td>Double Amplitude</td>
</tr>
<tr>
<td>$D_r$</td>
<td>Relative density of sand</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>Mean particle diameter (size)</td>
</tr>
<tr>
<td>$e$</td>
<td>Actual void ratio</td>
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<tr>
<td>EDS</td>
<td>Energy Dispersive Spectrum</td>
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<tr>
<td>$e_{max}$</td>
<td>Maximum void ratio</td>
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<tr>
<td>$e_{min}$</td>
<td>Minimum void ratio</td>
</tr>
<tr>
<td>$f$</td>
<td>Frequency</td>
</tr>
<tr>
<td>$F_L$</td>
<td>Factor of safety against liquefaction</td>
</tr>
<tr>
<td>FM</td>
<td>Fineness Modulus</td>
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<tr>
<td>$g$</td>
<td>Gravitational acceleration</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Specific gravity of soil solids</td>
</tr>
<tr>
<td>HVDM</td>
<td>High Vacuum Densification Method</td>
</tr>
<tr>
<td>$K_o$</td>
<td>Coefficient of lateral earth pressure at rest condition</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid Limit</td>
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<tr>
<td>MIT</td>
<td>Massachusetts Institute of Technology</td>
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<tr>
<td>Term</td>
<td>Definition</td>
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<td>------------------------------------------------</td>
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<tr>
<td>$M_b$</td>
<td>Body wave magnitude</td>
</tr>
<tr>
<td>$M_w$</td>
<td>Moment magnitude</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of cycle of loading</td>
</tr>
<tr>
<td>$pH$</td>
<td>Potential of Hydrogen</td>
</tr>
<tr>
<td>$PI$ or $I_P$</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>$PL$</td>
<td>Plastic Limit</td>
</tr>
<tr>
<td>PVD</td>
<td>Prefabricated Vertical Drain</td>
</tr>
<tr>
<td>$p'$</td>
<td>Effective mean stress</td>
</tr>
<tr>
<td>$q$</td>
<td>Deviator stress</td>
</tr>
<tr>
<td>RHA</td>
<td>Rice Husk Ash</td>
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<tr>
<td>SCP</td>
<td>Sand Compaction Pile</td>
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<tr>
<td>SEM</td>
<td>Scanning Electron Microscope</td>
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<tr>
<td>SL</td>
<td>Shrinkage Limit</td>
</tr>
<tr>
<td>SW - SM</td>
<td>Well graded silty sand</td>
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<tr>
<td>Soil A</td>
<td>Coarse sand</td>
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<td>Soil B</td>
<td>Medium sand</td>
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<tr>
<td>Soil C</td>
<td>Fine sand</td>
</tr>
<tr>
<td>SP</td>
<td>Poorly graded sand</td>
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<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
</tr>
<tr>
<td>$u$</td>
<td>Pore water pressure</td>
</tr>
<tr>
<td>USGS</td>
<td>U.S. Geological Survey</td>
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<tr>
<td>USCS</td>
<td>Unified Soil Classification System</td>
</tr>
<tr>
<td>XRD</td>
<td>X-Ray Diffraction</td>
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<tr>
<td>$\sigma$</td>
<td>Total vertical overburden stress</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>Major Principal Stress</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Chamber pressure</td>
</tr>
<tr>
<td>$\sigma_\delta$</td>
<td>Effective confining pressure</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>Effective stress</td>
</tr>
<tr>
<td>$\sigma_v'$</td>
<td>Effective vertical stress</td>
</tr>
<tr>
<td>$p'$</td>
<td>Mean effective stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Angle of internal friction</td>
</tr>
<tr>
<td>$\gamma_t$</td>
<td>Unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>Stress reduction coefficient</td>
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CHAPTER ONE
INTRODUCTION

1.1 General

Techniques to evaluate the stability of soil deposits subjected to earthquakes include an evaluation of the cyclic strength of the underlying soil (often called its liquefaction potential). At present the most widely used laboratory procedure to evaluate these strength characteristics is the stress-controlled cyclic triaxial strength test. Recently several investigators used cyclic triaxial test to evaluate liquefaction potential of soils. Existence of cyclic shear strength has been shown in the literature for uncemented sands. Similar cyclic strength for cemented sands, if developed, could be useful for practicing engineers. The purpose of the stress controlled cyclic triaxial testing program was to determine the cyclic strength of cementitious sands and their liquefaction resistance induced by an earthquake that meets the essential performance criteria for liquefaction potential of reclaiming landfill areas of Bangladesh to the geotechnical engineers. The goal of this research is to determine the liquefaction resistance of artificially prepared cementitious sands where lime be used for their stabilization. Other environmental friendly and cost effective waste materials will also be used as cementitious material to compare the effectiveness of liquefaction susceptibility with that of lime treated soils.

1.2 Background of the Present Research

Liquefaction is one of the most important, interesting, complex and controversial topics in geotechnical earthquake engineering. Liquefaction phenomena have been recorded and developed in many parts of the world that has been one of the major concerns for geotechnical engineers working in the moderate to major earthquakes active regions. Liquefaction can occur at moderate to major earthquakes, which can cause severe damage to structures. In liquefaction phenomena, ground shaking is frequent and soils consist of loose fine sand under water table.

Historical seismic data and recent seismic activities in Bangladesh and adjoining areas indicate that Bangladesh is seismically moderate country and is at seismic risk. Bangladesh covers one of the largest deltas and one of the thickest sedimentary basins in the world. Subsoil of Bangladesh including capital city Dhaka is largely an alluvial plain
consisting of loose fine sand and silt deposits (BNBC, 1993; Islam et al., 2010; Sharfuddin, 2001 and Seed et al., 1971) with shallow ground water table in most places.

Bangladesh is the world’s most densely populated country. Over the last three decades Dhaka city has experienced a rapid growth of urban population and it will continue in the future due to several unavoidable reasons. Unfortunately, most parts of the capital city Dhaka have already been occupied. As a result, many reclaimed areas are being developed in and around the cities by both government and private agencies. These reclaimed areas are being developed by filling lowlands (3~12 m) using dredge materials of adjacent river bed which is silty sand and the sand layers of the landfill areas are being developed on loose sand (Rahman, 2010) and that are not well compacted in many instances. Development procedures and characteristics of the dredged fill material indicate that the reclaimed areas are susceptible to liquefaction if an earthquake of sufficient energy occurs. According to Sonmez (2003), those loose filling type soil have a high to very high susceptibility to liquefaction during earthquake, when the soil comes into saturation. A few researches have been conducted to estimate liquefaction possibilities at local levels in Bangladesh. Researchers carried out studies to estimate soil liquefaction potentials and possibilities at various zones of Bangladesh (Ansary and Rashid, 2000; Khan, 1988; Rashid, 2000 and Shah, 2005) and found that these loosely filled sandy soils have a high to very high susceptibility to liquefaction due to earthquake (Islam and Ahamed, 2005 and Sonmez, 2003). Khan (1988) studied the soils liquefaction possibilities in Bangladesh. It was observed that some parts of the reclaimed areas are susceptible to liquefaction. Islam and Ahamed (2005) conducted preliminary evaluation of liquefaction potential of some selected reclaimed areas of Dhaka city. Clearly liquefaction is a serious component of the earthquake hazard in certain parts of Dhaka as indicated by the researchers (Ansary and Rashid, 2000) and needs to be considered. Rashid (2000) developed seismic microzonation map of Dhaka city based on site amplification and liquefaction. Ansary and Rashid (2000) generated liquefaction potential map for Dhaka city. Rahman (2004) updated the seismic microzonation maps for liquefaction as well as site amplification due to earthquake. Saha (2005) developed liquefaction potential map for Rangpur Town. Islam (2005) estimated the seismic losses especially due to the anticipated liquefaction in Sylhet city. Based on Seed-Idriss procedures, Sherpur-Maulivibazar areas were found to be vulnerable to liquefaction.
Soil stabilization introduced the use of technique to add a binder to the soil in order to improve the geotechnical performance of soil. The practice of treating soil with different additives is an attractive or interesting technique to geotechnical engineers to improve the geotechnical performance of soil. Over time, popular and traditionally extensively used soil improvement techniques for both cohesive and cohesionless soils are using lime and cement. Lime and Portland cement have traditionally been used to bind cohesive and cohesionless soil. These two materials have rapidly increased in price due to the sharp increase in the cost of energy (Neville, 2000). Recently a lot of researches have been done to find possible alternative soil stabilizing materials, especially those that are locally available, less costly and environment friendly. To reduce stabilization costs and use of waste material for environmental aspects, replacement of the lime and cement with locally available waste materials, such as fly ash, rice husk ash and natural pozzolona clay has been widely applied in practice. Rice husk ash is an agricultural waste obtained from rice mills and fly ash is a waste product of heat and power plants. The mixture of lime individually with fly ash (class F fly ash), rice husk ash and clay can produce cementations materials which enhanced pozzolanic reaction of soil and lime. Some studies are reported that, different additives such as cement, lime, fly ash, rice husk ash, bentonite and clay have been used for chemical stabilization of soils. The improved characteristics of compacted soils resulting from fly ash–lime utilization may be very important to some infrastructure projects such as foundations and sub-grades of roads and railways platforms, engineered fills, to strengthen slopes in slope stability problems, and to prevent sand liquefaction to stabilize backfill to reduce lateral earth pressures and to stabilize embankments to improve slope stability and in mitigating liquefaction and expansibility problems (Brown, 1996; Thome et al., 2005; Buhler and Cerato, 2007; Ghosh and Subbarao, 2007; Kumar et al., 2007; Daniels and Das, 2008; Consoli et al., 2009a and Rao and Asha 2012).

The pozzolan stabilized mixtures improve the compressive and shearing strength of soils and most importantly should be economical and environmentally friendly. Recently a lot of researches have been illustrated that adding the additives, enhanced the pozzolanic reaction of soil and lime, leads to generate long-term strength of soil. Compacting in-situ soil mixed with cement slurry is an extensively used soil improvement technique for problematic soil that is in relatively a dry state. An advantage of this technique is that adequate strength can be achieved in a short time. To reduce stabilization costs,
replacement of the cement with waste materials, such as fly ash, rice husk ash and biomass ash has been widely applied in practice. The effects of some influential factors such as water content, cement content, curing condition, replacement ratio and compaction energy on the microstructure and engineering characteristics of cement-stabilized soils have been extensively researched (Terashi et al., 1979; Terashi et al., 1980; Tatsuoka and Kobayashi, 1983; Kamon and Bergado, 1992; Nagaraj et al., 1997; Yin and Lai, 1998; Consoli et al., 2000; Kasama et al., 2000; Miura et al., 2001; Horpibulsuk and Miura, 2001; Horpibulsuk et al., 2006; Horpibulsuk et al., 2009; Horpibulsuk et al., 2010; Horpibulsuk et al., 2011; Horpibulsuk et al., 2012; Zillur Rabbi et al., 2011 and Gens and Nova, 1993).

The liquefaction resistance of soil is estimated using cyclic shear strength of soil. Cyclic triaxial tests have been used for seismic liquefaction evaluation of saturated sands since the pioneering work of Seed and Lee (1966) on isotropically consolidated specimens. Field and laboratory observations have shown that the resistance to liquefaction tends to increase with decreasing particle size (Ishihara, 1985). The liquefaction resistance increases as the degree of cementation increases and after a certain level of cementation is reached, the soil is not liquefiable at all. Saxena et al., (1988) found that even a small amount of cement significantly increases the cyclic strength compared to uncemented sands. Clough et al., (1989) and Saxena et al., (1988) noted that cemented loose sands behave similarly to dense uncemented sands. Clough et al., (1989) considered the effect of weak cementation on the liquefaction resistance of sand. Clough et al., (1989) concluded that the liquefaction resistance increases as the degree of cementation increases. Additionally, if there are weak lenses in cemented sand, these lenses lower the liquefaction resistance. This effect is more pronounced at higher stress levels. And finally, after a certain level of cementation is reached, the soil is essentially not liquefiable. Maher et al., (1994a) found that treatment with micro fine cement grout and tested at cyclic stress ratios between 0.3 and 1.0 did not liquefy at all.

The specific goal of this research is to determine the sustainable liquefaction resistance or cyclic strength of liquefaction susceptible soil stabilizing with environmental friendly and cost effective waste materials as additives for long term performance. For soil stabilization at reclaiming landfill areas this investigation may be environmental friendly, economical and efficient method for soil treatment for long term performance.
1.3 Objectives

The aim of this study is to find out the cyclic strength of cementitious sands and thus their liquefaction resistance. The specific objectives of this study are outlined as under:

(i) To determine the cyclic shear strength of sand treated with different lime contents.

(ii) To compare the cyclic strength of coarse, medium and fine sands treated with lime, fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement.

(iii) To evaluate the static shear strength of stabilized sands after the failure of the soil specimens at cyclic loading.

(iv) To compare the variation of compaction effort for lime, fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement stabilized sands.

(v) To study the bonding behavior of treated and untreated sands.

1.4 The Research Schemes

In order to achieve the above objectives, investigations will be carried out on sandy soils; the whole researches have been conducted in accordance with the following scheme:

(a) Sand samples (coarse, medium and fine sands) were collected from three selected rivers (the Surma river, the Jamuna river and the Turag river).

(b) Different cementitious materials (lime, fly ash, rice husk ash, clay, bentonite and cement) were collected.

(c) Some index properties such as the grain size analysis, specific gravity, maximum and minimum densities, maximum dry density and optimum moisture content, consistency limits etc. were determined for the selected samples and cementitious additives materials.

(d) The specimens were prepared properly for all experiments. Specimens of 71 mm in diameter and 142 mm in height have been used which were prepared by moist tamping method. Mixing of soil samples have been done with different stabilizing materials with varying percentages. The stabilizing materials prepared samples will be cured with water moisture at different ages.
(e) Cyclic and monotonous triaxial tests were conducted on reconstituted untreated and treated sand samples using Cyclic Triaxial Testing Machine.

(f) Static triaxial tests were carried out to get static shear strength of the soil after the failure of the specimen at cyclic loading.

(g) Analysis of laboratory data to determine the liquefaction resistance capacity of treated soil samples.

(h) Study the bonding behavior of treated sands using scanning electro-microscope techniques were carried out to identify the reaction products that are responsible for the modification of the soil properties.

1.5 Organization of Thesis

The study conducted for achieving the stated objectives is presented in several chapter of this thesis. The dissertation has been presented in five distinct chapters comprising different aspects of this study.

Chapter 1 concern about the introduction of the study with its background and the objectives; also include the research schemes and structure of the dissertation.

Chapter 2 contains a literature review summarizing what is currently understood about the effects of earthquake liquefaction behavior on natural and artificial cementitious sands and previous research works on liquefaction analysis.

Chapter 3 discusses the materials and methods that were used in the laboratory testing stage, specifically the sample preparation procedure, mixing procedure, cyclic and monotonic triaxial test techniques.

Chapter 4 shows graphically the results from cyclic and monotonic triaxial tests that were conducted on untreated and treated specimens; also describes the bonding behavior of untreated and treated sands from scanning electro-microscope (SEM) images and energy dispersive spectrum (EDS) analysis.

Chapter 5 presents the overall conclusions from this research study, along with recommendations for future research on liquefaction potential analysis for improvement of soil properties.
2.1 General

In spite of so much advancement of science and technology, earthquakes are still the cause of heavy destruction of life and property. Liquefaction of saturated soils during earthquakes often had been a major cause of damage to structures. Liquefaction is one of the most important, interesting, complex and controversial topics in geotechnical earthquake engineering (Kramer, 1996). Liquefaction and related phenomena have been responsible for tremendous amounts of damage in historical earthquake around the world. Especially, it’s devastating effects sprang to the attention of geotechnical engineers in a three-month period in 1964 when the Niigata earthquake in Japan followed the Good Friday earthquake in Alaska. Evaluation of liquefaction resistance of soils is an important step in many geotechnical investigations in earthquake-liable region. From the geotechnical engineering point of view, ground improvement may be defined as to increase shear strength, reduce compressibility and permeability of the soil depending on the specific needs for a given project at a given site. Engineering behavior and performance of the soil are effectively improved by various ground improvement techniques developed over the last century. Ground improvement techniques reduce the hazards associated with an earthquake, particularly those associated with liquefaction in earthquake prone areas. The associated environmental risk for many cementing agents encourages the development of alternative soil improvement method that is more environmentally friendly and sustainable.

2.2 Geology of Bangladesh

Understanding the geological setting of Bangladesh is important for assessment of earthquake hazards. Geological evolution of Bangladesh is basically related to the uplift of the Himalayan Mountains and outbuilding of deltaic landmass by major river systems. A process of deltaic sedimentation, into a slowly and continuously subsiding tectonic basin has developed the geology of Bangladesh. Bangladesh occupies major part of the Bengal Delta, one of the largest deltas and one of the thickest sedimentary basins in the world. Bangladesh is a riverside country. The river system can be divided into four major networks, (i) Brahmaputra-Jamuna river system, (ii) Ganges-Padma river system,
(iii) Surma-Meghna river system and (iv) Chittagong Region river system. Two Himalayan rivers, the Ganges and the Brahmaputra, which drain to the Bay of Bengal as a combined river, carry the largest sediment load. These two rivers together with another non-Himalayan river, the Meghna, have built the Ganges-Brahmaputra delta or the Bengal Delta. This is covered with about 100m of sandy, silty and clayey sediment of Bengal alluvium of recent age. Moreover, the enormous volume of sediments that the rivers carry to the Bay of Bengal each year (approximately 2.4 billion tons) builds new land along the sea front. Only the eastern part of Bangladesh (Chittagong Hill tract area) has been uplifted into hilly landform incorporating itself into the frontal belt of the Indo-Burman Range lying to the east.

Geological and geotechnical characteristics of soils of different regions of Bangladesh have been studied by many researches. Morgan and McIntire (1959) and Hunt (1976) had investigated geological characteristics of soils of different regions of Bangladesh. Ameen (1985) and Bashar (2000) investigated the geotechnical characteristics of the Dhaka clay. Islam (1999) and Hoque and Islam (2000) reported the anisotropy of Dhaka clay. Serajuddin et al., (2001) reported characteristics of uplifted Pleistocene deposits in Dhaka city. Some research works have been performed on coastal and regional soils, such as Serajuddin (1969), Amin et al., (1987). Mollah (1993) reported the geotechnical characteristics of the sub-soil of the deltaic alluvial plains of Bangladesh to provide a useful guideline for development planning. A few works have been reported in the literatures about the soft soils of Bangladesh, such as Kabir et al., (1992 and 2000); Razzaque and Alamgir (1999); Siddique et al., (2002) and Islam et al., (2003). Apart from Bangladesh soils, a lot of works are available in the literature which dealt with the characterization of soft clays and soft organic clays (Parry and Nadarajha, 1974; Mesri and Godlewski, 1977; Cox, 1981 and 1985; Patrick et al., 1992; Allman and Atkinson, 1992; Siddique and Clayton, 1999; Diaz-Rodriguez and Santamarina, 2001 and Chai and Miura, 2002).

Bangladesh can be divided into three major physiographic units namely, (i) the tertiary hill formations, (ii) the Pleistocene terrace, and (iii) the recent flood plains. According to the study of Morgan and McIntire (1959), there are two major areas of higher jungle composed of old alluvium of Pleistocene sediments, commonly known as the Modhupur tract and Barind tract. The Madhupur block lies between the Jamuna and Old
Brahmaputra channels and 6 to 30 meters above mean sea level. All or part of the clay is depositional. Patches of residuum also overlie gently dipping Tertiary units in the Fold Belt, including the Lalmai Hills, Comilla area. Dhaka is situated on the southern tip of a Pleistocene Terrace, called the Madhupur Tract. Two characteristics units cover the city and surroundings are Madhupur clay of Pleistocene age and alluvial deposits of recent age. Nearly 85 percent of Bangladesh is underlain by quaternary sediments consisting of deltaic and alluvial deposits of the Ganges, Brahmaputra and Meghna rivers and their numerous tributaries (Figure 2.1). The deltaic deposits are sediments that are deposited on the active delta, which is defined as the area south of the Ganges River and mostly west of the Meghna estuary. Most of the delta is less than 15 meters above mean sea level. Nearly 85 percent of Bangladesh is underlain by quaternary sediments consisting of deltaic and alluvial deposits of the Ganges, Brahmaputra and Meghna rivers and their numerous tributaries (Figure 2.1). The deltaic deposits are sediments that are deposited on the active delta, which is defined as the area south of the Ganges River and mostly west of the Meghna estuary. Most of the delta is less than 15 meters above mean sea level. From the Geological Map of Bangladesh in Figure 2.1 (Alam et al., 1990), different types of soil is found, which is shown in different colored and identified in the map. Sand deposits are found in many areas of the country. In the Geological Map of Bangladesh (Figure 2.1) it has shown that the country is mainly covered largely an alluvial plain consisting of sand and silt deposits. The coastal deposit (csd) are formed by beach and dune sand that contain light to grey colored medium to fine sand. Under the deltaic deposit, mangrove swamp deposit (dsw), tidal mud (dm), tidal deltaic deposits (dt), estuarine deposit (de), deltaic silt (dsl) are mainly silt and clay soils. Under the alluvial deposit, alluvial sand (asd) are of coarse to fine along the Brahmaputra river channel and medium to fine along the Padma and the Meghna river. In alluvial fan deposits along the Tista River, young and old gravelly sand (afy and afo) are mainly coarse to silty fine sand. The average grain size of the sand generally decreases from north to south and away from the channels. Most areas of Bangladesh are flooded annually. Included in this unit are thin veneers of sand spread by episodic large floods over flood-plain silts.

2.3 Status of Earthquakes in Bangladesh

Because of geographical, topographical and meteorological conditions of Bangladesh and the adjoining surrounding areas, Bangladesh and the north eastern Indian states have long been one of the seismically active regions of the world, and have experienced numerous large earthquakes during the past 200 years. The record of approximately 150 years shows that Bangladesh and the surrounding regions experienced several major earthquakes (with \( M_b = 7 \)). The earthquake record suggests that since 1900 more than 100 moderate to large earthquakes occurred in Bangladesh, out of which more than 65 events
occurred after 1960. This brings to light an increased frequency of earthquakes in the last 30 years. In the recent past, a number of tremors of moderate to severe intensity had already taken place in and around Bangladesh (Choudhury, 2005).

Historical seismic data and recent seismic activities in Bangladesh and adjoining areas indicate that Bangladesh is seismically moderate country and is at seismic risk. The country is situated in the moderate seismic zone in world seismic map. The country is located close to the junction of two subduction zones created by two active tectonic plates: the Indian plate and the Eurasian plate. Moreover, Bangladesh is surrounded by the regions of high seismicity which include the Himalayan Arc and Shillong Plateau in the north, the Burmese Arc, Arakan Yoma anticliniorium in the east and complex Naga-Disang-Jaflong thrust zones in the northeast. It is also the site of the Dauki Fault system of eastern Sylhet along with numerous subsurface active faults and a flexure zone called Hinge Zone. In the generalized tectonic map of Bangladesh the distribution of epicenters is found to be linear along the Dauki Fault system and random in other regions of Bangladesh (Figure 2.4 and Figure 2.5). Lots of seismic-tectonic studies have been undertaken on the area comprising the Indo-Burman ranges and their western extension and in the northern India. Major active fault zones of the country have been delineated through geological trenching and dating methods (Haque, 1990).

The first seismic zoning map of the subcontinent was compiled by the Geological Survey of India in 1935. The Bangladesh Meteorological Department adopted a seismic zoning map in 1972. In 1977, the Bangladesh government prepared a national seismic zoning map, and outlined building codes for earthquake resistant structural designs. The code provides a seismic zoning map that divides Bangladesh into three zones: Zone-I, Zone-II and Zone-III with Zone-III the most and Zone-I the least vulnerable to seismic risks (BNBC, 1993) shown in Figure 2.2. Table 2.1 includes a description of the three seismic zones. Figure 2.2 presents the present seismic zoning map of Bangladesh showing the boundaries of the three zones. Zone-III comprising the northern and eastern regions of Bangladesh with the presence of the Dauki Fault system of eastern Sylhet and the deep seated Sylhet Fault, and proximity to the highly disturbed southeastern Assam region with the Jaflong thrust, Naga thrust and Disang thrust, is a zone of the high seismic risk. Northern Bangladesh comprising greater Rangpur and Dinajpur districts is also a region of high seismicity (Figure 2.4).
Figure 2.1: Geological map of Bangladesh (after Alam et al., 1990).
Figure 2.2: Seismic zoning map of Bangladesh (BNBC, 2006).
The Chittagong-Tripura Folded Belt experiences frequent earthquakes, as just to its east is the Burmese Arc where a large number of shallow depth earthquakes originate. Zone-II comprising the central part of Bangladesh represents the regions of recent uplifted Pleistocene blocks of the Barind and Madhupur Tracts, and the western extension of the folded belt (Figure 2.4). The Zone-I comprising the southwestern part of Bangladesh is seismically quiet.

Table 2.1: Description of present seismic zones (BNBC, 1993)

<table>
<thead>
<tr>
<th>Seismic Zone</th>
<th>Location</th>
<th>Seismic Intensity</th>
<th>Seismic Zone Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Southwestern part including Barisal, Khulna, Jessore, Rajshahi, Sundarbans</td>
<td>Low</td>
<td>0.075</td>
</tr>
<tr>
<td>II</td>
<td>Central and Northwestern part including Dinajpur, Dhaka, Comilla as well as Souththeastern part including Chittagong, Cox’s Bazar, Chittagong Hill Tracks</td>
<td>Moderate</td>
<td>0.15</td>
</tr>
<tr>
<td>III</td>
<td>Northern and eastern regions of Bangladesh including Sylhet, Mymenshingh, Rangpur, Bogra, Kurigram</td>
<td>Very Severe</td>
<td>0.25</td>
</tr>
</tbody>
</table>

However, the seismic zones presented in the Bangladesh National Building Code (BNBC, 1993) are not based on the analytical assessment of seismic hazard and are mainly based on the location of historical data. In 2012, the Government of Bangladesh constituted a Committee of Experts to examine the seismic problem and make appropriate recommendations. The Committee proposed a new seismic zoning map of Bangladesh. The intent of the recommended new seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country based on extensive researches and studies and on the analytical assessment of seismic hazard. From the recommended seismic zoning map of Bangladesh, the country has been divided into four seismic zones with different levels of ground motion. Table 2.2 includes a description of the four seismic zones. Figure 2.3 presents the proposed seismic zoning map of Bangladesh showing the boundaries of the four zones. The most severe earthquake prone zone is Zone 4 and the least vulnerable to seismic risks zone is Zone 1.
Figure 2.3: Proposed seismic zoning map of Bangladesh (Proposed BNBC, 2016).
Table 2.2: Description of proposed seismic zones

<table>
<thead>
<tr>
<th>Seismic Zone</th>
<th>Location</th>
<th>Seismic Intensity</th>
<th>Seismic Zone Coefficient, Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Southwestern part including Barisal, Khulna, Jessore, Rajshahi</td>
<td>Low</td>
<td>0.12</td>
</tr>
<tr>
<td>2</td>
<td>Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans</td>
<td>Moderate</td>
<td>0.20</td>
</tr>
<tr>
<td>3</td>
<td>Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur</td>
<td>Severe</td>
<td>0.28</td>
</tr>
<tr>
<td>4</td>
<td>Northeastern part including Sylhet, Mymensingh, Kurigram</td>
<td>Very Severe</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Figure 2.4: Generalized tectonic map of Bangladesh and adjoining areas (after Alam et al., 1990).
2.4 Liquefaction in Context of Bangladesh

Bangladesh covers one of the largest deltas and one of the thickest sedimentary basins in the world. Subsoil of Bangladesh is largely an alluvial plain consisting of loose fine sand and silt deposits (BNBC, 1993; Islam et al., 2010 and Sharfuddin, 2001) with shallow ground water table in most places. Due to rapid growth of urban population most parts of the cities have already been occupied. As a result, many reclaimed areas are being developed in and around the cities. These reclaimed areas are being developed by filling lowlands (3~12 m) using dredge materials of adjacent river bed which is silty sand and the sand layers of the landfill areas are being developed on loose sand (Rahman, 2010)
and that are not well compacted in many instances. Development procedures and characteristics of the dredged fill material indicate that the reclaimed areas are susceptible to liquefaction if an earthquake of sufficient energy occurs (Sonmez, 2003).

In Bangladesh both natural soil and environmental factors are conducive for the occurrence of liquefaction. Bangladesh is covered for a greater part by alluvium deposited by three mighty rivers and their innumerable tributaries. These river systems bring a huge volume of new silt and medium to coarse-grained sands to replenish the land. Most of the land area in Bangladesh has been built up of alluvial deposits brought down by some major rivers. The soil condition is loose silty sand deposit and most of it remains in saturated condition round the year, thereby vulnerable to liquefaction in case of strong ground motions. A few researches have been conducted to estimate liquefaction possibilities at local levels in Bangladesh. It is expected that in near future a liquefaction potential map of Bangladesh will be developed and difficulties raised in finalizing design of many important structures will be over. Researchers carried out studies to estimate soil liquefaction potentials and possibilities at various zones of Bangladesh (Ansary and Rashid, 2000; Khan, 1988; Rashid, 2000 and Shah, 2005) and found that loosely filled sandy soils have a high to very high susceptibility to liquefaction due to earthquake (Islam and Ahamed, 2005 and Sonmez, 2003). Khan (1988) studied the soils liquefaction possibilities in Bangladesh. It was observed that some parts of the reclaimed areas are susceptible to liquefaction. Islam and Ahamed (2005) conducted preliminary evaluation of liquefaction potential of some selected reclaimed areas of Dhaka city. Clearly liquefaction is a serious component of the earthquake hazard in certain parts of Dhaka as indicated by the researchers (Ansary and Rashid, 2000) and needs to be considered. Rashid (2000) developed seismic microzonation map of Dhaka city based on site amplification and liquefaction. Ansary and Rashid (2000) generated liquefaction potential map for Dhaka city. Rahman (2004) updated the seismic microzonation maps for liquefaction as well as site amplification due to earthquake. Most of the parts of Chittagong city consisting of fine sand and silt deposits are susceptible to liquefaction (Bhuiyan et al., 2006). Saha (2005) developed liquefaction potential map for Rangpur Town. Islam (2005) estimated the seismic losses especially due to the anticipated liquefaction in Sylhet city. Based on Seed-Idriss procedures, Sherpur-Maulivibazar areas were found to be vulnerable to liquefaction.
2.5  Seismic Liquefaction

Liquefaction is a phenomena when there is loss of strength in saturated cohesionless soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. In this phenomenon, the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. According to Seed and Idriss (1982), “If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume. If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore pressure. If the pore water pressure builds up to the point at which it is equal to the overburden pressure, then the effective stress become zero, and the sand loses its strength completely, hence liquefaction occur.” A more precise definition as given by Sladen et al., (1985) states that “Liquefaction is a phenomena wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shocking loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance”.

Liquefaction phenomena can be divided into two main groups: flow liquefaction and cyclic softening (Robertson and Fear, 1996). In the field, flow liquefaction occurs much less frequently than cyclic softening but its effects are usually far more severe. Cyclic mobility can occur under a much broader range of soil and site conditions than flow liquefaction; its effects can range from insignificant to highly damaging.

2.5.1  Flow Liquefaction

Flow liquefaction can occur when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state. The undrained flow of saturated, contractive soil when subjected to cyclic or monotonic shear loading as the static shear stress exceeds the residual strength of the soil. Once triggered the large deformations produced by flow liquefaction are actually driven by static shear stresses.

2.5.2  Cyclic Softening

Cyclic softening is another phenomenon that can produce unacceptably large permanent deformations during earthquake shaking. Large deformations occurring during cyclic shear due to increase in pore water pressure tend to dilate in undrained monotonic shear. Cyclic softening can be further classified as, cyclic liquefaction and cyclic mobility.
2.5.2.1 Cyclic Liquefaction

It occurs when the initial, static shear stress is exceeded by the cyclic shear stresses to produce a stress reversal. This may help in attaining a condition of zero effective stress during which large deformations may develop.

2.5.2.2 Cyclic Mobility

Cyclic loads do not result in a reversal of shear stress and condition of zero effective stress does not occur. Deformations accumulate in each cycle of shear stress. Cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil. The deformations produced by cyclic mobility failures develop incrementally during earthquake shaking.

2.5.3 Pre-requisite Conditions for Liquefaction to Occur

Liquefaction can only occur if a soil deposit is granular soil such as silty sand or sandy silt with or without fine contents. The most important ones are the following:

(i) The ground is loose sandy deposit i.e., the soil deposit is loose cohesionless soil.

(ii) The ground water table is high and the ground is saturated i.e., the soil deposit is fully saturated.

(iii) The earthquake intensity is sufficiently large and the duration of shaking is sufficiently long i.e., a strong shaking strikes the soil at a high rate.

(iv) Loading rate should be high enough to establish undrain condition in the cohesionless soil mass. Load application should be repeated or cyclic in nature so that pore pressure can be build up gradually.

2.5.4 Causes of Liquefaction

Soil liquefaction is a phenomenon resulting when the pore pressure increases, thereby reducing the effective confining stress and hence the strength of the soil. Liquefaction occurs in saturated soils and saturated soils are the soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles. The water pressure is however relatively low before the occurrence of earthquake. But earthquake shaking can cause the water pressure to increase to the point
at which the soil particles can readily move with respect to one another. When rapid loading occurs on saturated sand under undrained conditions, the tendency for densification causes excess pore pressure to increase and effective stresses to decrease. In general, the higher the intensity of the vibration, the greater the potential for an increase in pore water pressure. Seed and Idriss (1982) present the qualitative explanation of soil liquefaction is that if a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume. If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore water pressure. When the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress become zero, the sand loses its strength completely, and it develops a liquefied state. Mathematically, shear strength ($\tau$) is expressed as:

$$\tau = c' + \sigma' \tan \phi' = c' + (\sigma - u) \tan \phi'$$  \hspace{1cm} (2.1)

Where, $\sigma'$ is effective stress, $\sigma$ is overburden pressure, $u$ is pore water pressure, $c'$ is effective cohesion and $\phi'$ is effective angle of internal friction of soil. In case of pure cohesionless or sandy soil, $c' = 0$.

$$\tau = (\sigma - u) \tan \phi'$$  \hspace{1cm} (2.2)

When liquefaction occurs, then, $\sigma = u$; thus in sandy soil, $\tau = 0$.

In case of cohesive soil, $c \neq 0$ and $\sigma = u$ such that $\tau = c$.

However, liquefaction occurs in cohesionless soils that are sandy and silty soil only. Liquefaction of non-plastic silts has been observed (Ishihara, 1984 and 1985) in the laboratory and the filed. Clays with appreciable plasticity are usually not prone to pore water pressure generation. Therefore, clays remain non-susceptible to liquefaction.

### 2.5.5 Mechanism of Liquefaction

It is important to recognize the conditions that exist in a soil deposit before an earthquake in order to understand liquefaction. Soil is basically assemblages of many individual soil particles which stay in contact with many neighboring particles [Figure 2.6(a)]. The contact forces produced by the weight of the overlying particles holds individual soil particle in its place and provide strength of the soil [Figure 2.6(b)]. The height of the blue column to the right represents the level of pore water pressure in the soil. The lengths of the arrows represent the size of the contact forces between individual soil grains.
When dry sand is subjected to shear stress, it changes its volume depending upon the density; the looser sands contract and densely packed sands increase in volume. This phenomenon is commonly known as dilatancy. If the looser sand is saturated with water and subjected to the same shear stress in an undrained condition, no volume change can occur during the shearing. Under this condition, the soil skeleton with a tendency toward contraction transfers some of its load to the water, and the water pressure thus produced reacts in turn to prevent the volume change from occurring. Occurrence of liquefaction is the result of rapid load application and break down of the loose and saturated sand and the loosely-packed individual soil particles tries to move into a denser configuration. However, there is not enough time for the pore-water of the soil to be squeezed out in case of earthquake. Instead, the water is trapped and prevents the soil particles from moving closer together. This is accompanied by an increase in water pressure which reduces the contact forces between the individual soil particles, thereby causing softening and weakening the soil deposit. In dynamic loading conditions shear stresses are cycled, excess pore pressure develops, thereby decreasing the effective confining stress initially applied to the soil elements. Under the appropriate conditions, the developed pore pressure eventually becomes equal to the effective confining pressure. The generation of excess pore pressure under undrained loading conditions is a hallmark of all liquefaction phenomena. In this state, without effective confining stress, the sand loses all its strength. Each particle of sand is separated and thrown into the surrounding water. In such cases, the sand mass deposit is transformed into a state of suspension and behaves like a viscous liquid than a solid. This state is named liquefaction or more commonly, quicksand.
2.5.6  Liquefaction Analysis

Liquefaction can be analyzed by a simple comparison of the seismically induced shear stress with the similarly expressed shear stress required to cause initial liquefaction or whatever level of shear strain amplitude is deemed intolerable in design. Usually, the occurrence of 5 or 10% double amplitude ($DA$) axial strain is adopted to define the cyclic strength consistent with 100% pore water pressure build-up. The corresponding strength ($CRR$) can be obtained by several procedures.

Thus, the liquefaction potential of a sand deposit is evaluated in terms of factor of safety, $FL$, defined as in Equation (2.3). The externally applied cyclic stress ratio ($CSR$) can be evaluated by Equations (2.4), (2.5) and (2.6).

$$F_L = \frac{CRR}{CSR}$$  \hspace{1cm} (2.3)

Using a factor of safety less than 1.00 against liquefaction is not considered a sound engineering practice. This is because a factor of safety less than 1.00 indicates failure is likely to occur. If the factor of safety $FL$ is less than 1.00, liquefaction is said to take place. Otherwise, liquefaction does not occur. The factor of safety obtained in this way is generally used to identify the depth to which liquefaction is expected to occur in a future earthquake. This information is necessary if some countermeasure is to be implemented in an in situ deposit of sands.

2.5.6.1 Cyclic Stress Ratio ($CSR$)

The cyclic shear stress induced at any point in level ground during an earthquake due to the upward propagation of shear waves can be assessed by means of a simple procedure proposed. The cyclic stress ratio ($CSR$) depends on the shear stress imposed by the earthquake as a function of the initial effective overburden pressure. This therefore depends primarily on the magnitude and distance of the earthquake, the depth of the liquefiable soil, the depth of the water table and the characteristics of the soil layer. If a soil column to a depth $z$ is assumed to move horizontally and if the peak horizontal acceleration on the ground surface is $a_{max}$, the maximum shear stress $\tau_{max}$ acting at the bottom of the soil column is given by

$$\tau_{max} = a_{max} y_d \gamma (z/g)$$  \hspace{1cm} (2.4)
Where, $\gamma_t$ is unit weight of the soil, $g$ is the gravitational acceleration and $\gamma_d$ is a stress reduction coefficient to allow for the deformability of the soil column ($\gamma_d < 1$). $\gamma_d$ is the rigidity of soil to include the effect of more flexible soil at depth, hence lower shear stresses. Generally, $\gamma_d$ is a function of the depth to the liquefiable layer and varies from 1 at the surface to 0.9 at 10m depth, for instance it is recommended (Liao and Whitman 1986) to use the empirical formula given in Equation (2.5) to compute stress reduction coefficient $\gamma_d$, where $z$ is depth to point of measurement or liquefiable soil in meters.

Division of both sides of Equation (2.4) by the effective vertical stress $\sigma'_v$ gives

$$C_{SR} = \frac{\tau_{max}}{\sigma'_v} = \frac{a_{max}}{g} \gamma_d \frac{\sigma_v}{\sigma'_v} = 0.65 a_{max} \gamma_d \frac{g z}{\sigma_v}$$

(2.6)

Where, $\sigma_v = \gamma_t z$ is the total vertical overburden stress at the level of the liquefiable layer and $\sigma'_v$ is the effective overburden pressure at the level of the liquefiable layer. $\sigma_v/\sigma'_v$ takes the position of the water table into account. The peak ground horizontal acceleration at site ($a_{max}$) can be evaluated from an attenuation relationship based on a maximum, $M_{w_{max}}$.

$$\log(a_{max}) = -1.02 + 0.24 M_{w_{max}} - \log(r)$$

(2.7)

Where, $r^2 = R_f^2 + 53.3$ (Joyner and Boore, 1981).

It is to be noted that as with most attenuation relationships, $a_{max}$ is the numerical fraction of $g$ and 0.65 of $a_{max}$ represents 95% energy of the record. Equation (2.6) has been used widely to assess the magnitude of shear stress induced in a soil element during an earthquake. One of the advantages of Equation (2.6) is that all the vast amount of information on the horizontal accelerations that has ever been recorded on the ground surface can be used directly to assess the shear stress induced by seismic shaking in the horizontal plane within the ground.

**2.5.6.2 Cyclic Resistance Ratio (CRR)**

The second step is to determine the cyclic resistance ratio (CRR) of the in situ soil. Cyclic resistance ratio is one of the fundamental parameters in the prediction of liquefaction phenomenon, can be determined using cyclic tests on the undisturbed or reconstituted...
laboratory specimens. The cyclic resistance ratio represents the liquefaction resistance of the in situ soil. The $CRR$ value depends on the quality of the soil and the depth of the liquefiable layer. This is defined as the cyclic stress required causing liquefaction in a given number of cycles expressed as a fraction of the initial effective confining pressure. $CRR$ is the required $CSR$ for liquefaction. The $CRR$ values are usually determined for liquefaction at 15 cycles of loading representing a 7.5 magnitude earthquake. Since the number of cycles is a function of the duration of the earthquake, which in turn depends on the magnitude, $CRR$ is dependent on the magnitude of the earthquake as well. The factors affecting the $CRR$ value are void ratio and relative density, over consolidation ratio, fines content and plasticity of fines. $CRR$ can be determined from shear box and triaxial tests, but field measurements are preferable. The most commonly used method for determining the liquefaction resistance is to use the data obtained from the standard penetration test. A cyclic triaxial test may also be used to estimate $CRR$ more accurately.

### 2.5.7 Simulation of Field Stress Conditions in Laboratory Tests

Prior to shaking by an earthquake, an element of saturated soil under level ground has undergone a long-term consolidation process under $K_o$ conditions. For the horizontal soil deposits, the triaxial specimen is consolidated isotropically, and the cyclic shear stresses generated by an earthquake are simulated in the laboratory by cycling shear stresses along the 45-deg plane of a triaxial compression specimen under undrained conditions. This soil element is subjected undrained to a sequence of shear stress cycles during an earthquake as illustrated in Figure 2.7(a) and 2.7(b). The cyclic shear stress application is executed in such a way that lateral deformation is prohibited because the flat ground surface is assumed to extend infinitely in the horizontal direction. In the case of sloping ground, a soil element is considered to have been anisotropically consolidated approximately under $K_o$ conditions with additional shear stress acting on the horizontal plane. For sloping ground surfaces, the triaxial specimen is consolidated anisotropically. Hence, the test is comparable to an R test with pore pressure measurements with the notable exception that the load is cyclically applied to the specimen such that the specimen is subjected to alternating cycles of vertical compression and extension about some ambient stress state, which produce corresponding cyclic shear stresses on the 45-deg plane [Figure 2.7(c)]. Cyclic shear stresses in the test on anisotropically consolidated specimens are depicted in
Figure 2.8(b). During an earthquake, a sequence of shear stress cycles is applied to the soil element in undrained conditions shown in Figure 2.8(a).

When attempting to investigate behavior of soils in the laboratory test, in situ stress conditions can be most fittingly reproduced in the sample tested in a cyclic triaxial apparatus. The principle and procedures for laboratory testing will be illustrated by referring to the stress conditions that can be produced in the cyclic triaxial apparatus. In a typical test scheme, samples of saturated sand are consolidated first under $K_0$ conditions and then subjected to a sequence of cyclic triaxial stress cycles under undrained conditions. The consequence of cyclic loading under undrained conditions is generally an increase in the pore water pressure, which causes the effective stress to decrease and the cyclic deformations of the specimen to increase. In this test, initial liquefaction is defined to occur when the pore water pressure first equals the confining pressure, i.e., a condition where the effective stress is zero, with failure defined in terms of a limiting cyclic strain, generally either 5 or 10 percent from peak to peak. Peak-to-peak strain is usually referred to as double amplitude strain and is the total strain that the specimen undergoes on consecutive peaks of the strain-time trace. For isotropically consolidated specimens, these strains will be alternatively compressional and extensional; however, in the case of anisotropically consolidated specimens, these strains may be alternative compression-extension or permanent compression strain only.

The applied cyclic axial stress, $(\sigma_1 - \sigma_3)$ or $P/A$ [Figure 2.7(c)] is termed the cyclic deviator stress, $\sigma_{dc}$, which is alternatively positive and negative about some ambient stress state. During shear, the major principal stress, $\sigma_1$ is equal to the applied axial stress ($P/A$) plus the chamber pressure; $\sigma_3$ [Figure 2.7(c)]. The applied axial stress, $(\sigma_1 - \sigma_3)$, is termed the deviator stress. For isotropically consolidated specimens, test results are expressed in terms of the cyclic stress ratio, $\sigma_{dc}/2\sigma_{3c}$, which is the cyclic shear stress, $\sigma_{dc}/2$ normalized by the confining pressure $\sigma_{3c}$. For anisotropically consolidated specimens, test results may be expressed in terms $\tau_{cyc}/\sigma_{fc}$, where $\tau_{cyc}$ the cyclic shear stress on the failure plane is $(45^\circ + \varphi'/2)$ and $\sigma_{fc}$ is the normal stress on this plane during consolidation. The value of $\varphi'$ can be estimated from static tests. Research has shown that the cyclic strength depends on density, confining pressure, applied cyclic shear stress, stress history, specimen preparation procedure, and uniformity and shape of cyclic wave form; hence, close attention must be given to testing details and equipment.
Figure 2.7: Stress conditions in the field (a) level ground and (b) sloping ground prior to and during seismic shaking and (c) stresses during triaxial compression test at laboratory test.

Figure 2.8: Mohr’s circle of total stress representation for a cyclic triaxial strength test for (a) an isotropically consolidated specimen (after Silver, 1976) and (b) an anisotropically consolidated specimen (after Silver et al., 1976).
2.6 Typical Effects of Liquefaction

Soil liquefaction has been responsible for extensive damage to buildings, bridges, pipelines and other critical infrastructure in many past earthquakes. According to USGS (2006), typical effects of liquefaction are described in the following sections.

2.6.1 Loss of Bearing Capacity

The ground can liquefy and lose its ability to support structures. The Niigata earthquake of 1964 provided the first well-documented modern example of the detrimental effects of liquefaction in an urban environment. It is seen in the Figure 2.9 of the overturned apartment complex buildings in Niigata in 1964 that the structure was all right but the soil failed to bear the load due to liquefaction.

![Figure 2.9: Damage of apartment buildings resulting from liquefaction during 1964 Niigata, Japan earthquake.](image)

2.6.2 Sand Boiling

Sand boils occur when water under pressure wells up through a bed of sand. Sand-laden water can be ejected from a buried liquefied layer and erupt at the surface to form sand volcanoes; the surrounding ground often fractures and settles (Figure 2.10). The numerous sand boils that were observed in the earthquake, affected area provided indisputable evidence of the occurrence of liquefaction.

2.6.3 Land Sliding

Earth structures such as embankments, dams and dikes or levees and other earth retaining structures constructed of liquefiable materials are also vulnerable to this behavior.
Increased water pressure can also trigger landslides and causes the collapse of dams (Figure 2.11). Liquefaction in sand layers, and in sand and silt seams in the clayey soils beneath Anchorage, caused many of the destructive landslides that occurred during the earthquake. The near failure of the Lower San Fernando Dam (Figure 2.11) during the San Fernando earthquake of 1971 offered an excellent case history of the seismic performance of embankment dams constructed on and of liquefiable materials. In fact, the most common problem leading to the instability of embankment dams in a seismic environment is the presence of liquefiable soils in the dams themselves or in the foundations on which they rest (Marcuson et al., 1996).

Figure 2.10: Sand boils (a) around the Mississippi and New Madrid Fault regions that erupted during the 2011 Christchurch earthquake and (b) along a fissure near the Pajaro River, 1989 Loma Prieta earthquake.

Figure 2.11: (a) Lower San Fernando Dam suffered an underwater slide due to liquefaction during the 1971 San Fernando California earthquake and (b) Bitter spring landslide, Coconino County, Arizona, 2013.
2.6.4 Floating of Structure

Light structures that are buried in the ground (like pipelines, sewers, nearly empty fuel tanks and utilities) can float to the surface when they are surrounded by liquefied soil. The manholes and sewer channels underneath streets may rise up due to buyout force caused by water during liquefaction (Figure 2.12).

Figure 2.12: Liquefaction in uplifted buried structures (sewer line) after Niigata-ken Chuetsu, 2004 Japan earthquake.

2.6.5 Settlement

When liquefied ground re-consolidates following an earthquake, the ground surface may settle or subside as shaking decreases and the underlying liquefied soil becomes denser (Figure 2.13). In many cases the weight of a structure will not be great enough to cause the large settlements associated with soil bearing capacity failures. However, smaller settlements may occur as soil pore-water pressures dissipate and the soil consolidates after the earthquake. These settlements may be damaging, although they would tend to be much less so then the large movements accompanying flow failures, lateral spreading, and bearing capacity failures. The sand boils or liquefied sand is a common manifestation of liquefaction that can also lead to localized differential settlements (Youd, 1992).

Figure 2.13: (a) Settlement and disruption of ground and pavement over filled ground, Dore Street, 1906 San Francisco earthquake; (b) Damage of building for differential settlement due to liquefaction at Adapazari, 1999 Turkey earthquake.
2.6.6 Flow Failures

Flow liquefaction produces the most dramatic effects of all the liquefaction-related phenomena—tremendous instabilities known as flow failures. Earth moves down in steep slope with large displacement and much internal disruption of material due to liquefaction (Figure 2.14).

Figure 2.14: (a) Flow failure in highway fills at Lake Merced in San Francisco during the 1957 Daly City Earthquake and (b) Failure of a span of the Nishinomiya Bridge during the 1995 Kobe, Japan earthquake.

2.6.7 Ground Oscillation

Ground failure and permanent deformations due to liquefaction are pervasive forms of damage during earthquakes. Where the ground is flat or the slope is too gentle to allow lateral displacement, liquefaction at depth may decouple overlaying soil layers from the underlying ground, allowing the upper soil to oscillate back and forth and up and down in the form of ground waves. The surface layer, riding on a buried liquefied layer, is thrown back and forth by the shaking and can be severely deformed (Figure 2.15).

Figure 2.15: (a) Walkway and pavement buckled by ground oscillation, Marina District of San Francisco, 1989 Loma Prieta earthquake and (b) highways were to be closed down, Florence.
2.6.8 Increased Lateral Pressure on Retaining Walls

If the soil behind a retaining wall liquefies, the lateral pressures on the wall may greatly increase. As a result, retaining walls may be laterally displaced, tilt, or structurally fail, as has been observed for waterfront walls retaining loose saturated sand in a number of earthquakes (Youd, 1992).

![Concrete gravity retaining wall rotated outward due to failure of the slope above during the 1999 Chi-Chi earthquake in Taiwan and quay wall failure due to liquefaction, 1995 Kobe, Japan earthquake.]

2.6.9 Lateral Spreading

Lateral spreading often occurs where loose, saturated sandy soils are commonly encountered at shallow depths as pore water pressure build up in the soil resulting from earthquake shaking reduces the stiffness and strength of the soil. The ground can slide down very gentle slopes or toward stream banks, riding on a buried liquefied layer can make big cracks on the ground (Figure 2.17). Structures supported on shallow foundations, pavements, and buried pipelines are susceptible to damage from lateral spreading.

![Burning gas main ruptured by lateral movement at Balboa Blvd in Granada Hills, 1994 Northridge earthquake and lateral spreads at Hebgen Lake near West Yellowstone, 1959 Hebgen Lake earthquake caused liquefaction of sediments beneath the road.]

(a) 
(b)
2.7 Mitigation of Liquefaction

Soil liquefaction and associated ground failures have been a major source of damage during the past earthquakes. During earthquakes, severe damage to buildings and other structures occurred due to extensive liquefaction of sands (e.g., Alaska earthquake of 1964 and Niigata earthquake of 1969). Remediation methods against liquefaction have been developed and applied since Niigata earthquake. The risk of liquefaction and associated ground deformation can be reduced by various ground-improvement methods. There are three possibilities to reduce liquefaction hazards when designing and constructing new buildings or other structures as bridges, tunnels, roads etc. The efforts for liquefaction mitigations include the following three strategies,

2.7.1 Avoiding Liquefaction Susceptible Soils for Building Infrastructures

The first possibility is to avoid locating construction of buildings and infrastructure in areas with high liquefaction susceptible soils. By characterizing the soil at a particular building site one can decide if the site is susceptible to liquefaction and therefore unsuitable for the desired structure.

2.7.2 Building Liquefaction Resistant Structures

If it is necessary to construct on liquefaction susceptible soil because of space restrictions, favorable location, or other reasons, it may be possible to make the structure liquefaction resistant by designing the foundation elements to resist the effects of liquefaction. A structure that possesses ductility, has the ability to accommodate large deformations, adjustable supports for correction of differential settlements, and having foundation design that can span soft spots can decrease the amount of damage a structure may suffer in case of liquefaction. To achieve these features in a building there are various aspects to consider.

2.7.2.1 Design Considerations for Shallow Foundation

It is important that all foundation elements in a shallow foundation is tied together to make the foundation move or settle uniformly, thus decreasing the shear forces resting upon the foundation. A stiff foundation mat is a good type of shallow foundation, which can transfer loads from locally liquefied zones to adjacent stronger ground. Buried
utilities, such as sewage and water pipes, should have ductile connections to the structure to accommodate the large movements and settlements that can occur due to liquefaction.

### 2.7.2.2 Design Considerations for Deep Foundation

Liquefaction can cause large lateral loads on pile foundations. Piles driven through a weak, potentially liquefiable, soil layer to a stronger layer not only have to carry vertical loads from the superstructure, but must also be able to resist horizontal loads and bending moments induced by lateral movements if the weak layer liquefies. Sufficient resistance can be achieved by piles of larger dimensions and more reinforcement. It is important that the piles are connected to the cap in a ductile manner that allows some rotation to occur without a failure of the connection.

### 2.7.3 Improving the Soil

Ground improvement is mainly involves to improve the load bearing capacity of in-situ soils and fills, to reduce differential settlements of non-homogeneous soils and to mitigate the liquefaction hazards by improving the strength, density, and/or drainage characteristics of saturated sands. Improve the existing soil formation by changing the soil properties by mechanical or chemical treatment. This can be done using a variety of soil improvement techniques.

#### 2.7.3.1 Stone Columns

Stone columns are columns of gravel constructed in the ground. Stone Columns are formed by inserting a vibrating probe to incorporate graded stone aggregate into the resulting cavity. Stone columns are installed by boring holes in the ground up to specified depth preferably in an equilateral triangular or square pattern and filling the holes with well graded stone aggregate. Relatively rapid consolidation is achieved through excess pore water pressures being readily dissipated by the stone columns. Stone columns help to limit the amount and consequences of future liquefaction by densifying the soil through vibration and introducing stone into the soil and reinforcing the soil creating a stiff composite soil mass [Figure 2.18(a)]. The soft soil is strengthened by replacing a certain percentage of the weak soil with well graded stone aggregate and the aggregate column thus formed will act as a pile as well as drainage channel to release the pore water within the sub soil during loading. By achieving this, the non-liquefying soil crust is thickened.
and stiffened to reduce the likelihood of undulations, tilt and uneven ground surface subsidence from liquefaction of the underlying soil layers, therefore reducing damage to the foundations. In addition, stone columns may sometimes provide the soil with an increased drainage path to help reduce excess pore water pressure that can lead to liquefaction, so the columns can reduce the consequences of liquefaction when this occurs.

### 2.7.3.2 Compaction Grouting

Compaction grouting is a technique whereby a very stiff, low slump (25-75mm) mortar-type slow-flowing grout mix (normally sand-cement grouts, sometimes with additives) is injected under relatively high pressure into a granular soil to displace and compact soils in place. The grout forms a bulb that displaces and hence densifies the surrounding soil [Figure 2.18(b)]. The method provides a low cost, low risk solution for underpinning, liquefaction mitigation and re-leveling of buildings back to their original elevation. It can be used on large projects using high capacity plant as well as with hand tools, allowing improvement to be carried out in the most restricted locations. Most effective in cohesionless soils but can also be effective in the short term in finer grained soils where disturbance has occurred.

### 2.7.3.3 Dynamic Compaction

Dynamic compaction is a ground improvement technique that densifies granular soils and fills materials by using a drop weight. Densification by dynamic compaction is performed by dropping a heavy weight of steel or concrete weighing 15 to 40 tons in a grid pattern from heights of 10 to 40 meters. The energy from the falling weight is transferred by propagation of Rayleigh (surface) waves and Volumic (shear and compression) waves [Figure 2.18(c)]. In cohesionless soils, these waves create liquefaction that is followed by the compaction of the soil. Treated granular soils and fills have increased density, friction angle and stiffness. The technique has been used to increase bearing capacity, and decrease settlement and liquefaction potential for planned structures.

### 2.7.3.4 Drainage Techniques

Vertical Drains are used to accelerate the consolidation of fine grain soils. Liquefaction hazards can be reduced by increasing the drainage ability of the soil. Drainage techniques
include installation of drains of gravel, sand or synthetic materials. This approach has resulted in a number of techniques involving (i) pre-compression or pre-loading, (ii) sand drains, (iii) pre-fabricated vertical drains (PVD), (iv) vacuum consolidation, and (v) high vacuum densification method (HVDM) embankment on clay. Sand drains are vertical drains installed by boring holes at specified intervals in the ground to the required depth (preferably not more than 25m) in triangular or square pattern by displacement or non-displacement method and filling the hole with coarse sand. Prefabricated Vertical Drain (PVD) is a modern technique of installing a vertical drain of polyester for improving the soil properties to strengthen the soil in reclamation bed, coastal areas and marshy area where soft clay is encountered at much more depth [Figure 2.18(d)]. Vacuum consolidation method is a technique of applying vacuum suction to an isolated soil mass to reduce the atmospheric pressure in it, thus by the way of reducing the pore water pressure in the soil the effective stress is increased without changing the total stress.

2.7.3.5 Vibroflotation

The vibroflotation is a specialist deep foundation technique used for ground improvement to considerable depth (usually 30 to 75 meters). It involves the introduction of granular soil to form interlocking columns with surrounding soil. Vibroflotation uses a vibrating probe that can penetrate granular soil to strengthen the soil by deep compaction to make the soil suitable to support proposed imposed load [Figure 2.18(e)]. The vibrations of the probe cause the grain structure to collapse thereby densifying the soil surrounding the probe. The technique is used to improve bearing capacity and reduce the possibility of differential settlements that might be allowed for the proposed loads. The risk of liquefaction in an earthquake prone area is also drastically reduced.
2.7.3.6 Sand Compaction Piles

Sand Compaction Pile (SCP) technology has been widely applied to various structures on both clayey and sandy grounds (as liquefaction mitigation). Compaction piles have gained wide acceptance for strengthening and reducing the compressibility of loose sands. It combines such fundamental principles of ground improvement as densification and drainage. The volume of loose sand replaced by injected sand is one of the most important factors in improving weak ground using compaction piles. Sand compaction piles (SCP) of soil improvement techniques act as drains and under favorable conditions can significantly decrease the time for primary consolidation to occur. Furthermore, the sand pile reduces the build-up in pore pressure in granular layers, and hence decreases the soil potential to liquefy during an earthquake. The sand compaction pile method is the most useful countermeasure against liquefaction.

2.8 Ground Improvement by Soil Stabilization

Artificial cementation of poor sandy deposits can mitigate such earthquake-induced damage considerably so that important structures can be protected. Even though there are many studies examining the complex phenomenon of liquefaction-cyclic mobility of uncemented sands, there is only limited information regarding such behavior among cemented sands. As of today, the available information dealing with naturally cemented sands is rather scarce and inconclusive mainly due to non-uniform cementation and disturbances induced during sampling, usually associated with such materials (Salomone et al., 1978 and Frydman et al., 1980). As far as artificially cemented sands are concerned, the existing studies are confined to their immediate interests (Rad and Clough, 1982; Dupas and Pecker, 1979 and Sitar et al., 1980).

The objective of soil stabilization is to increase strength, reduce deformability, reduce permeability, increase durability, and to provide volume stability of the soil. The main goal of ground improvement techniques that are commonly used depends on the specific needs for a given project at a given site, but it can typically be classified into one of the following general categories, (i) increasing the shear strength of the soil to guard against catastrophic failure and (ii) reducing the compressibility of the soil to prevent excessive ground movements, or reducing the permeability of the soil to reduce the rate of water seepage (common for earth dam or environmental applications).
2.8.1 Stabilizing Materials

Cementing agents such as lime, Portland cement, fly ash, rice husk ash, bentonite, clay, gypsum, calcite, wax resins and bitumen, calcium chloride or others, can be used as stabilizing material for soil stabilization.

2.8.1.1 Cement

Portland cement is the most commonly used and effective additive for soil stabilization. It has both adhesive and cohesive properties, enabling it to bind mineral fragments into a solid mass, i.e., those that can set and harden in the presence of water and so it is termed as Hydraulic cement. Cement consists of numerous minerals and is manufactured by combining cement clinker (a sintered material of limestone and clay) with gypsum. It is a fine grey powder produced by grinding Portland cement clinker, a limited amount of calcium sulphate and up to 5% minor constituents. The main chemical constituents of clinker are CaO, SiO₂, Al₂O₃ and Fe₃O₃. The usual hydraulic cement used is known as Portland cement. ASTM C150 defined Portland cement as hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, usually containing one or more of the forms of calcium sulphate as an underground addition. Type I Portland cement is the most widely used in soil stabilization. Typical composition of ordinary Portland cement is presented in Table 2.3.

Table 2.3: Typical composition of ordinary Portland cement (Mindess and Young, 1981)

<table>
<thead>
<tr>
<th>Chemical Name</th>
<th>Chemical Formula</th>
<th>Weight (per cent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium Silicate (C₃S)</td>
<td>3CaO·SiO₂</td>
<td>45-55</td>
</tr>
<tr>
<td>Dicalcium Silicate (C₂S)</td>
<td>2CaO·SiO₂</td>
<td>20-30</td>
</tr>
<tr>
<td>Tricalcium Aluminate (C₃A)</td>
<td>3CaO·Al₂O₃</td>
<td>9-13</td>
</tr>
<tr>
<td>Tetracalcium Aluminofenite (C₄AF)</td>
<td>4CaO·Al₂O₃·Fe₂O₃</td>
<td>8-20</td>
</tr>
<tr>
<td>Calcium Sulphate Dihydrate (Gypsum)</td>
<td>CaSO₄·2H₂O</td>
<td>2-6</td>
</tr>
<tr>
<td>Other Compounds</td>
<td></td>
<td>2-8</td>
</tr>
</tbody>
</table>

2.8.1.2 Lime

Soil stabilization by the addition of quicklime or hydrated lime is an ancient art, successfully adapted in modern times by perceptive engineers and now widely practiced. Lime is calcium containing inorganic material in which carbonates, oxides and hydroxides predominate (Figure B.5). Lime are obtained in the form of powder or limestone. The scanning electro-microscope image of limestone are shown in Figure 2.19. In this study, lime means quicklime, hydrated lime, or hydrated lime slurry.
Lime, refers to hydrated or slaked lime (calcium hydroxide), quicklime (calcium oxide), or dolomitic limes (calcium/magnesium oxide), that is, the highly alkaline \(pH > 12.3\) lime products. Lime in the form of quicklime (calcium oxide, CaO), hydrated lime (calcium hydroxide, Ca(OH)\(_2\)), or lime slurry can be used to treat soils. Quick lime is generally more effective than hydrate lime (Kezdi, 1979). Quicklime is manufactured by chemically transforming calcium carbonate (limestone, CaCO\(_3\)) into calcium oxide. The chemical name of quick lime or lime is calcium oxide. The chemical formula of quicklime is CaO. The raw material used for the quicklime is calcium carbonate (CaCO\(_3\)). Hydrated lime or Ca(OH)\(_2\) is in the form of fine powder. The hydrated lime comes in the form of a dry, very fine powder or as slurry. Hydrated lime is created when quicklime chemically reacts with water. Hydrated lime reacts with soil particles and permanently transforms them into a strong cementitious matrix. Calcium hydroxide (slaked lime) is most widely used for stabilization. Calcium oxide (quick lime) may be more effective in some cases, however. Most lime used for soil treatment is high calcium lime, which contains no more than 5 percent magnesium oxide or hydroxide. On some occasions, however, dolomitic lime is used. Dolomitic lime is usually not as effective as calcium lime (i.e., hydrated or slaked lime and quicklime). Dolomitic lime contains 35 to 46 percent magnesium oxide or hydroxide. Dolomitic lime can perform well in soil stabilization, although the magnesium fraction reacts more slowly than the calcium fraction. Sometimes the term lime is used to describe agricultural lime which is generally finely ground limestone, a useful soil amendment but not chemically active enough to lead to soil stabilization. Agricultural lime (calcium carbonate) is not suitable for stabilization.

![Figure 2.191: Scanning Electron Microscope (SEM) images of lime.](image)
Table 2.4: Properties of lime (NAASRA, 1986)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Hydrated Lime</th>
<th>Quick Lime</th>
<th>Slurry Lime</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composition</td>
<td>Ca(OH)$_2$</td>
<td>CaO</td>
<td>Ca(OH)$_2$</td>
</tr>
<tr>
<td>Form</td>
<td>Fine</td>
<td>Powder</td>
<td>Granular Slurry</td>
</tr>
<tr>
<td>Equivalent Ca(OH)$_2$/Unit Mass</td>
<td>1.00</td>
<td>1.32</td>
<td>0.56 to 0.33</td>
</tr>
<tr>
<td>Bulk Density (kg/m$^3$)</td>
<td>450 to 560</td>
<td>1050</td>
<td>1250</td>
</tr>
</tbody>
</table>

### 2.8.1.3 Water

There is no precise measure of the quality of water required, it being generally regarded that potable water is satisfactory. Potable water is preferred for lime stabilization. Acidic (organic) water should be avoided. However, highly organic water or water containing high concentration of sulphates (above 0.05 per cent) may cause problems and should be avoided. The amount of water used in lime stabilization is governed by the requirements of compaction. However, if quicklime is used then extra water may be required in soils having less than 50 percent moisture content to provide for very rapid hydration process. Most importantly, the quantity of water added to cement-treated mix is determined by the requirements of the maximum dry density and not that needed for cement hydration.

### 2.8.2 Pozzolonic Materials

A pozzolan is a siliceous or aluminous material, which in itself possesses little or no cementitious value but in finely divided form with moisture, it will react chemically with calcium hydroxide (lime) at ordinary temperatures to form compounds possessing cementitious properties (ASTM, 1950). The broad definition of a pozzolan imparts no bearing on the origin of the material, only on its capability of reacting with calcium hydroxide and water. Although the principal products of the reaction between a pozzolan and calcium hydroxide are considered to be calcium silicates and aluminates, there is some evidence that more complex compounds are formed.

#### 2.8.2.1 Fly Ash

Fly ash, a material with good pozzolanic properties, is frequently used in conjunction with lime and cement treatment. Fly ash, also known as flue-ash, is one of the residues generated in combustion, and comprises the fine particles that rise with the flue gases. Fly ash is a byproduct from burning pulverized coal in electric power generating plants. Fly
ash is obtained as a residue from power stations and heating plants fuelled with pulverized fuel. Fly ash is generally stored at coal power plants or placed in landfills. Fly ash in soil stabilization has three major effects, long term reaction, use of waste materials and environment friendly nature i.e., reduces carbon content. The recycling of fly ash has become an increasing concern in recent years due to increasing landfill costs and current interest in sustainable development. Other environmental benefits to recycling fly ash include reducing the demand for virgin materials such as cement and lime.

Fly ash particles are generally spherical in shape and range in size from 0.5 µm to 300 µm. In consequence, fly ash is a heterogeneous material (Figure 2.20). SiO₂, Al₂O₃, Fe₂O₃ and occasionally CaO are the main chemical components present in fly ashes (Figure B.7). The mineralogy of fly ashes is very diverse.

Figure 2.20: Scanning Electron Microscope (SEM) images of fly ash.

Two classes of fly ash are defined by ASTM C618, Class F fly ash and Class C fly ash. (i) Class F fly ash, the burning of harder, older anthracite and bituminous coal typically produces Class F fly ash. This fly ash is pozzolanic in nature and contains less than 20% lime (CaO). (ii) Class C fly ash, fly ash produced from the burning of younger lignite or sub-bituminous coal, in addition to having pozzolanic properties, also has some self-cementing properties. In the presence of water, Class C fly ash hardens and gets stronger over time. Class C fly ash generally contains more than 20% lime (CaO) and cementitious and hydraulic in nature. Unlike Class F, self-cementing Class C fly ash does not require an activator. Alkali and sulfate (SO₄) contents are generally higher in Class C fly ashes. The chief difference between these classes is the amount of calcium, silica, alumina, and iron content in the ash shown in Table 2.5.
Table 2.5: Physical and chemical requirements of class C and class F fly ash (ASTM C618 and AASHTO M 295)

<table>
<thead>
<tr>
<th>Chemical requirements:</th>
<th>Class F Fly Ash</th>
<th>Class C Fly Ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon dioxide (SiO₂) plus aluminum oxide (Al₂O₃) plus iron oxide (Fe₂O₃), min, %</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>Sulfur trioxide (SO₃), max %</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Moisture content, max, %</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Loss on Ignition, max, %</td>
<td>6 to 12</td>
<td>6</td>
</tr>
</tbody>
</table>

| Physical requirements:                                                                 |                  |                  |
| Fineness retained on No. 325 sieve, max, %                                            | 34               | 34               |
| Strength activity index with Portland cement at 7 and 28 days, min, %                  | 75               | 75               |
| Water requirement on soundness control, max, %                                         | 105              | 105              |
| Density, max variation from average, %                                                 | 5                | 5                |
| % retained on No. 325 sieve, max variation from average, %                            | 5                | 5                |

2.8.2.2 Rice Husk Ash

Rice husk ash (RHA) is by-product material produced from the process of manufacturing puffed rice, contains large amount of iron oxide and silicate. It is chemically stable. Rice husk ash are generally plate like irregular in shape (Figure 2.21). Rice husk ash is a pozzolanic material. When rice husk was allowed to burn under controlled temperature, higher pozzolanic properties (than other leaf plants) were observed. During the burning process, the components that can evaporate are evaporated and the only components left are the silicates. The silicates are the components that gives the pozzolanic reactivity capacity for rice husk ash. Rice milling industry generates a lot of rice husk during milling of paddy which comes from the fields. Rice husk is also used as a fuel for power generation. Rice husk ash is about 25% by weight of rice husk when burnt in boilers. It is estimated that about 70 million tons of rice husk ash is produced annually worldwide. This rice husk ash is a great environment threat causing damage to the land and the surrounding area in which it is dumped. The rice husk ash possesses a chemical composition similar to many of the organic fibers. The silicates are one of the primary components of the rice husk ash. Rice husk ash consists of the cellulose (C₅H₁₀O₅), lignin (C₇H₁₀O₃), hemicellulose, silica (SiO₂) and holocellulose (Figure B.9). The constituents of rice husk ash are listed in Table 2.6.
Table 2.6: Composition of rice husk ash

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Composition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>67.3</td>
</tr>
<tr>
<td>Al₃O₃</td>
<td>4.9</td>
</tr>
<tr>
<td>Fe₃O₅</td>
<td>0.95</td>
</tr>
<tr>
<td>CaO</td>
<td>1.36</td>
</tr>
<tr>
<td>MgO</td>
<td>1.81</td>
</tr>
<tr>
<td>Loss on Ignition (LOI)</td>
<td>17.78</td>
</tr>
</tbody>
</table>

During milling of paddy about 78% of weight is received as rice, broken rice and bran. Rest 22% of the weight of paddy is received as husk. This husk is used as fuel in the rice mills to generate steam for the parboiling process. This husk contains about 75% organic volatile matter and the balance 25% of the weight of this husk is converted into ash during the firing process, is known as rice husk ash. This rice husk ash in turn contains around 85% - 90% amorphous silica. So for every 1000 kg of paddy milled, about 220 kg (22%) of husk is produced and when this husk is burnt in the boilers, about 55 kg (25%) of rice husk ash is generated.

Rice husk ash is a carbon neutral green product. Lots of ways are being thought of for disposing them by making commercial use of this rice husk ash. Rice husk ash is a good super-pozzolan. This super-pozzolan can be used in a big way to make special concrete mixes and soil stabilization. There is a growing demand for fine amorphous silica in the production of special cement and concrete mixes, high performance concrete, high strength, low permeability concrete, for use in bridges, marine environments, nuclear power plants etc.
2.8.2.3 Clay

Soil is an uncemented or weakly cemented accumulation of mineral grains or particles formed by the weathering of rocks, the void space between the particles, containing water and/or air. Clay soil are also bound to the chemical variability of its constituents. It is shown in Figure 2.22. Soil contains almost all type of elements, the most important being oxygen, silicon, aluminium, calcium, sulfur, potassium, magnesium and carbon shown in appendix B (Figure B.10).

![Figure 2.22: Scanning Electron Microscope (SEM) images of clay.](image)

Natural clays contain structure (combination of fabric and bonding), which influences their properties (Burland, 1990). Clay is normally understood to mean a clay soil whose grains are predominantly composed of clay minerals and which has plasticity and cohesion. The ingredients necessary to give cohesion to a soil deposit are clay minerals. Clay minerals are very active electrochemically and the presence of even a small amount of clay mineral can appreciably alter the engineering properties of a soil mass.

2.8.2.4 Bentonite

Bentonite is montmorillonite clay derived from volcanic ash. Bentonite is one of the impure clay containing montmorillonite as its principal constituent. Molamorillonite (Al$_2$O$_3$.4SiO$_2$.6H$_2$O), also called smectite, is a three layer clay mineral composed of two silica sheets and one alumina (gibbsite) sheet (Figure 4.23), a 2: 1 mineral (three layer sheet). The octahedral sheet is sandwiched between two silica sheets with the tips of the tetrahedrons combining with the hydroxyls of the octahedral sheet to form a single layer. The thickness of this layer is about 9.6Å and the dimensions in the other two directions are indefinite.
Bentonite clay is composed of aged volcanic ash. Bentonite usually forms from weathering of volcanic ash, most often in the presence of water. Bentonite has a high concentration of minerals including silica, calcium, magnesium, sodium, iron, and potassium. For industrial purposes, two main classes of bentonite exist: sodium and calcium bentonite. Bentonite is swelling clay. When it becomes mixed with water it rapidly swells open like a highly porous sponge (Figure 2.24). Bentonite is unique clay due to the ability to produce an electrical charge when hydrated. The main uses of bentonite are for drilling mud, binder (e.g. foundry-sand bond, iron ore pelletizer), purifier, absorbent (e.g. pet litter), and as a groundwater barrier. A good quality bentonite should be a grey/cream color and anything bordering pure white is suspect.
2.9 Mechanism of Soil-Lime Reactions for Stabilization

Lime is added to soils either in the form of quicklime (CaO) or as hydrated lime [Ca(OH)₂]. It is recognized that lime has an immediate effect on soils, improving its granulation and handling properties. The lime and the soil system with water pass through several different reactions such as hydration, cation exchange and pozzolanic reactions. The basic mechanisms of soil-lime interactions have been described by IRC (1973) and Hausmann (1990). Eades and Grims (1960); Thompson (1966); Compendium (1987) and other researchers worked with the mechanism of lime stabilization. They observed that mechanism of lime stabilization can be classified into four phases. These mechanisms are briefly presented in the following sections.

(a) Cation exchange/ Base exchange
(b) Flocculation/ Agglomeration
(c) Carbonation
(d) Pozzolanic reaction/ Cementation

2.9.1 Cation Exchange/ Base Exchange

Soil particles are usually negatively charged and they contain adsorbed exchangeable cations of sodium (Na⁺), calcium (Ca²⁺), magnesium (Mg²⁺), aluminum (Al³⁺), potassium (K⁺) or hydrogen on the surface (Thompson, 1966). Soils which have high amounts of organic matter and moderately weathered clays tend to have a net negative charge and high Cation Exchange Capacities (CEC). Sandy soils generally have lower CEC values. Pure sand has a very low CEC and the CEC of the sand and silt size fractions of most soils is negligible. Thus, these negatively-charged soil particles will attract and hold positively-charged particles, much like the opposite poles of a magnet attract each other. By the same token, they will repel other negatively-charged particles, as like poles of a magnet repel each other. Eades and Grims (1960) indicated to the formation of new crystalline phases in the soil lime electrolyte system due to the addition of lime to the soil in presence of water. The reaction of lime and sand begin by the replacement of existing cations. Many natural soils are largely cation saturated. Addition of lime to the soil supplies to an excess of Ca²⁺ and the cation exchange will occur, with Ca²⁺ replacing dissimilar cations from the exchange complex of the soil. In some cases the exchange complex is particularly Ca²⁺ saturated before the lime addition but cation exchange may still take place because the CEC will increase as the pH of the soil increase.
2.9.2 Flocculation/Agglomeration

Flocculation of the soil particles occurs due to the mixing of soil with lime in presence of water. After cation exchange of soil and lime take place, agglomeration of the flocculated particles occurs. Diamond and Kinter (1965) suggested that the rapid formation of calcium aluminate hydrate cementing materials is significant in the development of flocculation agglomeration tendencies in the soil lime mixture.

Thompson (1966) indicated that flocculation and agglomeration are primarily responsible for the change in plasticity, shrinkage, and workability characteristics of soil lime mixture. Mitchell and Hopper (1961) demonstrated that the introduction of lime in moist soil increase the pH of the soil and release Ca\(^{2+}\) ions into the pore water. This phenomenon reduces the forces of repulsion and encourages compression of the diffused double layers. Mitchell (1981) stated that increased thickness of double layer creates less tendency of repulsion for particles in double layer resulting in a flocculated structure. These reactions results to an apparent change in texture the clay particles clumping into larger sized aggregate.

2.9.3 Cementation/Pozzolanic Reaction

Cementation is the main contributor to the strength of the stabilized soil. Long term chemical reaction of lime with certain non cementitious minerals (silicate and aluminate) of soil in presence of water is referred to pozzolanic reaction in lime stabilization. The minerals that react with lime to produce a cementing material are known as pozzolans (Khan, 1989). If lime is added in excess of the lime fixation point, complex chemical reactions similar to pozzolanic reactions are known to take place between lime and the soil minerals. Lime, water, soil silica and alumina react to form various cementitious compounds. The reaction of non cementitious aluminous and siliceous materials with calcium hydroxide in the presence of water to form cementitious compounds according to the following equations:

\[
Ca^{2+} + 2(OH^-) + SiO_2 \text{ (soil silica)} = CSH \tag{2.8}
\]

\[
Ca^{2+} + 2(OH^-) + Al_2O_3 \text{ (soil alumina)} = CAH \tag{2.9}
\]
In Equations (2.8) and (2.9), $C_{SH}$ and $C_{AH}$ are cementitious products. The above reactions represented by Equations (2.8) and (2.9) are slow and long-term in nature, measured in weeks or months. Moreover, these are more effective when the soil-lime mixture is adequately compacted. Cementation is limited by the availability of silica. Increasing the quantity of lime added will increase strength only up to the point where all the silica of the soil is used up; adding too much lime can actually be counterproductive.

Addition of lime to the soil in presence of water causes instantaneous rise in pH of the molding water due to dissociation of $\text{Ca(OH)}_2$ in water. Eades and Grims (1960) showed that high pH cause’s silica and alumina to be dissolved out of the structure of the soil minerals and it combines with calcium to form calcium silicate and calcium aluminates. The calcium ions combine with reactive hydrous silica and alumina and form gradually hardening cementitious material. This reaction will continue as long as $\text{Ca(OH)}_2$ exist in the soil and available silica. This mechanism may be referred as "Through-Solution".

Herzog and Mitchell (1963) indicated that soil lime pozzolanic reaction usually does not appear until after long curing period and then only in cases where a high percentage of lime was added. Pozzolanic materials (silicious or aluminous) possess little or no cementitious value, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. The lime-soil pozzolonic reaction mechanism was studied by Boardman et al., (2001). The pozzolanic reaction, affecting the long-term changes, took place after 7 days of curing. Asserson et al., (1974) worked with red tropical soils suggested that after the initial 7 days of curing, strength increases are the result of hydration and increase in crystallizing of reaction products rather than from the continued formation of additional pozzolanic compounds. Ramle (1987) indicated that surface chemical reaction can occur and new phase may nucleate directly on the surface of soil particles while conducting research concerning the adsorption of lime by kaolinite and montmorillonite. They mentioned that it is also possible that the reactions may occur by a combination of through solution (solution-precipitation) and surface chemical (hydration-crystallization) process. Kezdi (1979) stated the dissociation of hydrated lime into $\text{Ca}^{2+}$ and $\text{OH}^{-}$ causes’ loss of its crystalline structure and assume an amorphous form and flocculation of soil particles occurs, causing improvement of soil texture, rendering the soil more workable.
2.9.4 Carbonation

Lime reacts with carbon dioxide in atmosphere or in the soil to form relatively weak cementing agents such as calcium carbonate or magnesium carbonate. As lime absorbs carbon dioxide from the air, calcium carbonate (CaCO$_3$) is formed. These carbonates are relatively weak cementing agent (Hausmann, 1990). This reaction is the slowest of all the reactions involved in a soil-lime system and as in pozzolanic reaction, requires that the mixture must be thoroughly compacted. Carbonation may be beneficial where lime is plentiful; the CaCO$_3$ formed will not react any further with the soil. The reaction of calcium hydroxide with carbon dioxide is faster, producing a mortar that hardens more quickly.

\[
Ca(OH)_2 (s) + CO_2 (g) \rightarrow CaCO_3 (s) + H_2O (l)
\] (2.10)

Yu Kuen (1975) stated that carbonation is normally confined to the surface exposed to the air and involve the conversion of lime to the Calcium carbonate by carbon dioxide absorbed from the air. Eades et al., (1962) demonstrated that although carbonation does take place, the strength gain is said to be occur by virtue of cementation of soil grains with calcium carbonate is negligible. Diamond and Kinter (1965) indicated that the carbonation is probably deteriorate rather than a helpful phenomenon in soil stabilization. The strength of calcium carbonates which are formed by this process can generally be discounted.

2.10 Effect of Lime Treatment on the Geotechnical Properties of Soil

The drying of wet soil and the increase in soil workability is attributed to the immediate treatment, whereas the increase in the strength, durability and compressibility of the soil are associated with the long-term treatment (Locat et al., 1990; Wild et al., 1996; Mallela et al., 2004 and Geiman, 2005). The following applications and benefits can be accomplished by lime-treated soil.

2.10.1 Water Content-Density Relationship

When lime is used as soil treatment additive, soil particles became large-sized clusters, resulting in texture change (Terrei et al., 1984). This flocculation-agglomeration process results in floc formation. The enlarged particle size causes the void ratio to increase
(Kinuthia et al., 1999). This increase in void ratio reflects the decrease in maximum dry density. The moisture content for the soil-lime mixture compaction increased. Thus, the required density can be easily achieved for a broad range of water content, thereby conserving time, effort and energy (Thompson, 1965; Tabatabi, 1997 and Mallela et al., 2004).

2.10.2 Increase in Soil Strength

Several researchers have used various methodologies to evaluate the evolution of uncured and cured soil strength (determined in the laboratory) with respect to lime content. Thompson (1965) and Beubauer Jr. and Thompson (1972) stated that uncured improved specimen achieve in instantaneous strength gains and that the strength increased up to 60% because of pozzolanic reaction after curing for 28 days. The researchers demonstrated that using lime as additive to treat fine-grained soils yields a significant increase in soil cohesion and a slight improvement of the internal friction angle. The factors that affect the maximum strength gain are the percentage of added lime and the soil mineralogy (Eades and Grim, 1966). The curing environment, curing period, soil mineralogy and amount of added lime significantly affect the strength gain (Doty and Alexander, 1968).

2.10.3 Decreased Plasticity Index

Most plastic soils show significant reduction in plasticity index. This reduction results from the decrease in liquid limit and the increase in plastic limit (Little et al., 1995 and Mallela et al., 2004). Moreover, a number of high plasticity soils can be modified into non-plastic soils through lime addition (Holtz, 1969). The reduction in the plasticity is attributed to the change in soil nature (granular nature after flocculation and agglomeration) and the modified soil is as crumbly as silt soil (Osinubi, 1995).

2.10.4 Increase in Fatigue Strength

The number of load cycles that a material can tolerate at a constant stress level reflects the fatigue strength of that material (Mallela et al., 2004). Swanson and Thompson (1967) studied that the number of cyclic loading increases adversely to the ratio of applied stress to static strength. In addition, Mallela et al., (2004) reported that the strength developed over time reduces the stress-to-strength ratio, thereby increasing fatigue strength.
2.10.5 Increased Durability

Durability is the capability of lime-treated soil to resist the adverse effects of the wet-dry and freeze-thaw cycles resulting from the changes in environmental conditions during a year. This is to assure the sustainability of strength gain achieved by soil treatment (Al-Amoudi et al., 2010). Thompson and Dempsey (1969) demonstrated the ability of the lime-soil mixture to cure provided that the pozzolanic reaction persists.

2.10.6 Decreased Swell Potential and Volume Change

Soils are considered problematic because of their swell potential and volume change, which apply uplift pressure and cause substantial damage to the structures (particularly for the light-weight structure). Mallela et al., (2004) defined the percent of swell as the volume change that the soil has endured when the moisture content approaches saturation level. Little et al., (1995) stated that a significant reduction in swell potential and swell pressure can be achieved in lime treated soil. This reduction in swell potential is associated with the decrease in plasticity index caused by lime treatment. In addition, curing and pozzolanic reaction provide additional reduction in swelling during the long-term treatment (Dempsey and Thompson, 1968; Thompson, 1969 and Little et al., 1995).

2.10.7 Effect on Permeability

The literature does not provide information on the precise effect of lime treatment on soil permeability. A number of studies found that the hydraulic conductivity increases when the soil is mixed with lime. Townsend and Klyn (1970) stated that the permeability of the soil increase due to the addition of lime to the soil. Broms and Boman (1977) and Brandl (1981) stated that the addition of lime usually increases the permeability of soil. The increase in permeability is associated with flocculation, where larger pore between the flocks enable the fluid to flow more readily in between the soil and corresponding change in grain size distribution. However, other studies reported that soil permeability significantly decreases when lime content is increased. Nalbantoglu and Tuncer (2001) found that higher permeability was obtained from lime soil mixture because of soil aggregation and flocculation. Tedesco (2006) stated that although the grain size distribution was modified toward the sand fraction, the hydraulic conductivity decreased when soil was treated with lime. Onitsuka et al., (2001) concluded that although permeability is a function of pore space, the hydraulic conductivity decreased because of
the contraction of the pore space when the cement products were formed. Milburn and Parsons (2004) showed that the lime-treated samples had reduced hydraulic conductivity; this reduction was attributed to the formation of bonds between the soil particles. Nonetheless, a group of researchers (Locat *et al*., 1996; Kassim and Chow, 2000; De Brito Galvao *et al*., 2004; Onitsuka *et al*., 2001; Milburn and Parsons, 2004 and Alhassan, 2008) believe that hydraulic conductivity increases with increasing lime content until a specified percentage or a certain age is reached; then, the hydraulic conductivity declines. Kassim and Chow (2000) indicated that the coefficient of permeability was higher in stabilized soil than that in non-stabilized soil at the early stage. As the mixture aged, the permeability decreased because of the formation of cementitious gel. Alhassan (2008) believed that the differences in soil behavior with respect to permeability are attributed to soil mineralogy.

### 2.10.8 Effect on Compressibility

Similar to permeability studies, limited studies have dealt with the effect of lime on soil compressibility (Rajasekaran and Rao, 1997; Tremblay *et al*., 2001 and Rao and Shivananda, 2005). Kassim and Chow (2000) demonstrated that as the curing period progressed, the lime-treated soil modified the compression index and reduced the coefficient of compressibility settlement. Tedesco (2006) pointed out that the lime-treated soil samples exhibited lower compressibility. Moreover, he found that the lime addition had no effect on curing time. The remarkable decrease in compressibility related to the lime addition was associated with short-term reaction. Further, the pozzolanic reaction had limited influence.

### 2.11 Compaction Effort

Compaction is a process that brings about an increase in soil density or unit weight, accompanied by a decrease in air volume. There is usually no change in water content. Compaction is the application of mechanical energy to a soil to rearrange the particles and reduce the void ratio. As the layers of sand build up during compaction, the pressure on the lower layers increases. The layers are squeezed together and any water mixed in with the solid grains is forced out. In compaction process the sand particle is squeezed out and the size of the pore space between the solid grains is reduced by the weight and the pressure of the overlying layers. This squashes the grains together more tightly. At the same time the sand particles begin to stick to each other - they are cemented together by
clay, or by minerals like silica or calcite. For coarse soils, the material is unsaturated and derives strength from suction in pore water which collects at grain contacts. As the water content increases, suctions, and hence effective stresses decrease. The soil weakens, and is therefore more easily compacted.

The degree of compaction is measured by dry unit weight and depends on the water content and compactive effort (weight of hammer, number of impacts, weight of roller, and number of passes). For a given compactive effort, the maximum dry unit weight occurs at optimum water content. The compaction effort in this sample preparation method or compactive energy per volume is defined as:

\[
Compaction \text{ Effort} = \frac{W \times \text{No of Layers} \times \text{Heigh of Drops} \times \text{No of blows per layer}}{\text{Volume of Mold}}
\]  

(2.11)

In the above equation 2.11, the volume of mold, weight of hammer, height of hammer drop and number of layer is fixed for all specimens. For a constant relative density the compaction effort is directly proportional to the number of blow. That is the compaction effort varies with in the variation of number of blows. If the number of blows increases then the compaction effort is increased.

The variation in compaction with water content and compactive effort is first established in the laboratory. Target values are then specified for the dry density and/or air-voids content to be achieved on site. The compactive effort will be greater when using a heavier roller on site or a heavier rammer in the laboratory.

Compaction on the lime stabilized soil as well as untreated soil can be done by different methods in the field and in the laboratory. The amounts of compaction energy greatly affect the maximum dry density and the optimum moisture content of the soil. The effect of increasing the compactive effort results in an increase in the maximum dry density and decrease in optimum moisture content as found by Singh and Punmia (1965). Herrin and Mitchel (1961) pointed that a minimum amount of energy is essential for properly stabilize soil with lime. Without a certain minimum amount of energy, a very low strength of the lime stabilized soil is observed. They also mentioned that the required compactive effort depends upon the soil type. For the fine grained soil required compactive effort is lower than the other types of soil.
2.12 Cyclic Resistance of Reconstituted Clean Sand

The potential for liquefaction of saturated sands under seismic loading conditions has been extensively investigated by many researchers by means of cyclic triaxial tests, cyclic simple shear tests and cyclic torsional tests (Yoshimi et al., 1977, Seed, 1979 and Finn, 1981). The outcome of these studies has generally confirmed that the resistance to liquefaction of the samples of clean sand reconstituted in the laboratory is influenced primarily by the factors such as initial confining stress, intensity of shaking as represented by the cyclic shear stress, the number of cyclic stress application and void ratio or relative density. As a result of comprehensive laboratory studies, it has been recognized as reasonable and become customary to consider the combined effect of cyclic shear stress and initial confining stress in terms of the cyclic stress ratio which is defined as ($\sigma_d/2\sigma_0'$) for the triaxial loading condition. Where, $\sigma_d$ is single amplitude of cyclic axial stress and $2\sigma_0'$ is the initial confining stress. Thus it has become a routine practice to take the cyclic stress ratio required to cause 5% double amplitude (D.A.) axial strain under 20 load cycles as a factor quantifying the liquefaction resistance of sand under a given state of packing as represented by void ratio or relative density. This cyclic stress is represented by $[\sigma_d/2\sigma_0']_{20}$ and is referred to as the cyclic strength.

2.13 Cyclic Resistance of in Situ Deposits of Sands

In view of the diversity of cyclic strength of sand samples reconstituted by different methods of preparation, it has been recognized that deposits of sands in the field might exhibit varying resistance to seismic load application. Thus there has been an increasing demand for any effort to recover as perfectly undisturbed samples as possible from in situ sand deposits and to test them in the laboratory under conditions representative of those prevailing in the field.

A series of cyclic triaxial tests were conducted on the undisturbed sample obtained by ground freezing method (Yoshimi et al., 1989). The outcome of the test program is demonstrated in Figure 2.25, where it can be seen that, for clean sands with relative densities of about 50% and 80%, the cyclic resistance of undisturbed samples from the in situ deposit is about twice as great as the cyclic resistance of the samples from the newly deposited sand fill. The cyclic strength of in situ deposits is thus considered to vary greatly depending upon aging and the inherent fabric structure of sands created under different depositional conditions. For this reason, the test results on reconstituted samples
should be considered generally not to reflect true behavior of in situ sands, and tests on high-quality undisturbed samples are always needed to precisely evaluate performances of in situ deposits of sands during earthquakes.

Figure 2.25: Cyclic strength of undisturbed samples of sand (after Yoshimi et al., 1989).

2.14 Cyclic Resistance of Silty Sands

Liquefaction is a state of particle suspension resulting from release of contacts between particles of sands constituting a deposit. Therefore the type of soil most susceptible to liquefaction is the one in which the resistance to deformation is mobilized by friction between particles under the influence of confining pressures. When the soil is fine grained or contains some amount of fines, cohesion or adhesion tends to develop between fine particles thereby making it difficult for them to be separated from each other. Consequently, a greater resistance to liquefaction is generally exhibited by the sand containing some fines. However this tendency depends on the nature of the fines contained in the sand. If the fines comprise minerals with a dry surface texture free from adhesion, individual particles will separate readily, therefore the sand containing such fines will exhibit as great a potential to liquefaction as does the clean sand.

The degree of liquefiability of the sand containing more or less cohesive fines such as those found in the fluvial deposits has been investigated in the laboratory tests by Ishihara et al., (1978). It was seen that, with increasing content of fines, the cyclic resistance of sand tends to increase to a certain extent. This tendency may be taken for granted because adhesion between fine particles tends to prevent separation of individual particles when
the sand is about to liquefy. Thus the sand containing such plastic fines generally exhibits a higher resistance to liquefaction. However, since the effects of fines are manifested variously depending upon the nature of the fines themselves, of prime importance among other things would be to seek a key parameter which is capable of more specifically quantifying the liquefaction characteristics of fines-containing sands.

It was also seen that if the fines content is greater than about 50% the cyclic stress depends on plasticity index where the relative density fails to be an appropriate index parameter. Compilation of several series of laboratory test data has shown that the most important index property influencing the cyclic strength is the plasticity index of the fines contained in the sand (Ishihara and Koseki, 1989). This is clearly demonstrated in the results summarized in Figure 2.26, where the cyclic strength changes with plasticity index. The cyclic strength does not change much for the low plasticity range, but increases thereafter with increasing plasticity index.

Figure 2.26: Effects of plasticity index on the cyclic strength of fines-containing sand (after Ishihara and Koseki, 1989).

2.15 Cyclic Resistance of Stabilized Sands

Cyclic strength testing of stabilized sands is to determine if the stabilized sands will perform adequately under the proposed earthquake loading. Cyclic laboratory tests cannot reproduce the stress conditions that exist in actual seismic events, but can give us an idea of how the stabilized formation might behave if it were subjected to an earthquake
loading. Earthquakes have a random loading pattern that applies stresses in three dimensions. The loading experienced by a soil formation in the field depends on many factors. It is not practical to reproduce an actual earthquake loading in the laboratory. Even if it were, earthquake events have unique time histories that are not likely to be representative of the next earthquake loading. Therefore, standardized tests are performed in the laboratory under controlled conditions, where the components of the load applied to the sample and the boundary conditions of the test are known. Predictions of the behavior in the field are extrapolated from the test results.

The cyclic triaxial test is one method that can be used to determine the liquefaction resistance of a soil. One way to determine if treatment is successful is to develop a cyclic resistance curve for untreated sands and stabilized sands to compare the improvement due to treatment. If the stabilized sands require more cycles to liquefy, then the treatment may be considered successful. Treatment could also be considered successful if the strain that develops during cyclic loading is limited. Loose sands and dense sands behave differently during cyclic loading. Loose strains accumulate very little strain until just prior to liquefaction. At the onset of liquefaction, strain accumulates very quickly and can cause rapid collapse of the sample. In contrast, dense sands accumulate a small amount of strain early in the cyclic loading, but that level of strain increases very slowly or not at all.

There have been many liquefaction studies done on untreated sands, but very few have been done on sands treated with cement or chemical grouts. Some dynamic testing studies have been done on sands treated with chemical grouts to determine dynamic properties for design of machine foundations (Maher et al., 1994b; Rosenfarb and Hackman, 1981 and Vipulanandan et al., 2000). Several liquefaction studies have been done on naturally and artificially cemented sands (Clough et al., 1989 and Saxena et al., 1988) and silicate-grouted sands (Maher et al., 1994a).

Clough et al., (1989) considered the effect of weak cementation on the liquefaction resistance of sand and following conclusions relating to liquefaction resistance are suggested. First, the behavior of loose cemented sand is similar to the behavior of denser, uncemented sand. Second, the liquefaction resistance increases as the degree of cementation increases. If there are weak lenses in cemented sand, these lenses lower the liquefaction resistance. This effect is more pronounced at higher stress levels. Finally, after a certain level of cementation is reached, the soil is essentially not liquefiable.
Saxena et al., (1988) found that even a small amount of cement significantly increases the cyclic strength compared to uncemented sands. Like Clough et al., (1989), Saxena et al., (1988) noted that cemented loose sands behave similarly to dense uncemented sands. They also noted that when cemented sands were tested cyclically, the axial strains were “often more asymmetrical about the zero strain axis” than for untreated sands. In addition, they found that the cyclic stress of treated sands increased with relative density and with curing period.

Maher et al., (1994a) did monotonic and cyclic testing on Ottawa sand treated with sodium silicate, acrylate polymer, or microfine cement grout. The sands were at relative densities of 41 and percent. The sands were saturated prior to testing using the method of vacuum saturation (Rad and Clough 1984). Maher et al., (1994a) found that treatment with sodium silicate in concentrations between 20 and 60 percent increased the liquefaction resistance of sand from three to six times over untreated sand. Similarly, treatment with acrylate polymer grout increased the liquefaction resistance from two to four times over untreated sands. Finally, sands treated with micro fine cement grout and tested at cyclic stress ratios between 0.3 and 1.0 did not liquefy at all.

2.16 Factors Affecting the Liquefaction Susceptibility

Cyclic shear strength and dynamic modulus of soils depend on the same parameters such as density, strain, soil structure, stress history, and others (Hardin and Drnevich, 1972; Townsend, 1977 and Saxena and Reddy, 1987). Using the cyclic triaxial test, (Seed and Lee, 1966) obtained data which influence the liquefaction potential of saturated sand. The susceptibility of soil deposits to liquefaction is determined by a combination of various factors to which they may be subjected, such as soil properties, geological conditions and ground motion characteristics. Not all soils are susceptible to liquefaction; consequently, the first step in a liquefaction hazard evaluation is usually the evaluation of liquefaction susceptibility. If the soil at a particular site is not susceptible, liquefaction hazards do not exist and the liquefaction hazard evaluation can be ended. If the soil is susceptible, however, the matters of liquefaction initiation and effects must be addressed. The initial liquefaction failure criterion could be defined in terms of the peak-to-peak strain (five percent or ten percent) that a specimen undergoes during cyclic loading, the initial liquefaction criterion was adopted because the observed patterns concerning important parameters such as excess pore water pressure development for cemented specimens,
could be compared easily with similarly obtained, well-established patterns of uncemented specimens (Seed et al., 1977). For cemented sands, Rad and Clough (1982) have demonstrated that there was no significant difference in the cyclic triaxial test results for different peak-to-peak strain failure criteria adopted. There are several criteria by which liquefaction susceptibility can be judged, and some are different for flow liquefaction and cyclic mobility. Based on the results of laboratory tests as well as field observations and studies, liquefaction characteristics of cohesionless soils are affected by a number of factors. The most important factors that govern liquefaction are as follows:

2.16.1 Seismic Stress-Strain History

A great deal of information on liquefaction behavior has come from post-earthquake field investigations, which have shown that liquefaction often recurs at the same location when soil and groundwater conditions have remained unchanged (Youd, 1984a). Thus liquefaction case histories can be used to identify specific sites, or more general site conditions, that may be susceptible to liquefaction in future earthquakes. Youd (1991) described a number of instances where historical evidence of liquefaction has been used to map liquefaction susceptibility.

It has been demonstrated from laboratory test results that prior seismic strain history can significantly affect the resistance of soils to liquefaction (Finn et al., 1970; Seed et al., 1977 and Singh et al., 1980). Low levels of prior seismic strain history, as a result of a series of previous shakings producing low levels of excess pore pressure, can significantly increase soil resistance to pore pressure buildup during subsequent cyclic loading. This increased resistance may result from uniform densification of the soil or from better interlocking of the particles in the original structure due to elimination of small local instabilities at the contact points without any general structural rearrangement taking place. Large strains, however, associated with large pore pressure generation and conditions of full liquefaction can develop weak zones in the soil due to uneven densification and redistribution of water content (National Research Council, 1985 and Whitman, 1985), and thus lower the resistance of the soil to pore pressure generation during subsequent cyclic loading. Stress history is also crucial in determining the liquefaction resistance of a soil. For example, soil deposits with an initial static shear stress \( i.e. \), anisotropic consolidation conditions are generally more resistant to pore water
pressure generation (Seed, 1979) although static shear stresses may result in greater deformations since liquefaction gets initiated.

Post-earthquake field investigations have also shown that liquefaction effects have historically been confined to a zone within a particular distance of the seismic source. Ambraseys (1988) compiled worldwide data from shallow earthquakes to estimate a limiting epicentral distance beyond which liquefaction has not been observed in earthquakes of different magnitudes (Figure 2.27). The distance to which liquefaction can be expected increases dramatically with increasing magnitude. While relationships of the type shown in Figure 2.27 offer no guarantee that liquefaction cannot occur at greater distances, they are helpful for estimation of regional liquefaction hazard scenarios.

![Figure 2.27: Relationship between limiting epicentral distance of sites at which liquefaction has been observed and moment magnitude for shallow earthquakes. Deep earthquakes (focal depths > 50 km) have produced liquefaction at greater distances (after Ambraseys, 1988).](image)

2.16.2 Effects on Age and Geological Origin of the Soils

Soil deposits that are susceptible to liquefaction are formed within a relatively narrow range of geological environments (Youd, 1991). The depositional environment, hydrological environment, and age of a soil deposit all contribute to its liquefaction susceptibility (Youd and Hoose, 1977). Soil deposits can have in-situ weathering of rock (residual soil), river or alluvial deposit, lake or lacustrine deposit, marine (beach and sea)
deposit, organic deposits, wind or aeolian deposit etc. of the geological origins. Soils from different geological origin behave differently. Natural deposits of alluvial and fluvial origins generally have soil grains in the state of loose packing. These deposits are young, weak and free from added strength due to cementation and aging. Geologic processes that sort soils into uniform grain size distribution and deposit them in loose produce soil deposits with high liquefaction susceptibility. Consequently, fluvial deposits, and colluvial and neolian deposits when saturated, are likely to be susceptible to liquefaction. Liquefaction has also been observed in alluvial-fan, alluvial-plain, beach, terrace, playa, and estuarine deposits, but not as consistently as in those listed previously. The susceptibility of older soil deposits to liquefaction is generally lower than that of newer deposits. Soils of Holocene age are more susceptible than soils of Pleistocene age, although susceptibility decreases with age within the Holocene. Youd and Hoose (1977) stated that, as a rule of thumb, alluvial deposits older than late Pleistocene (10,000-130,000 years) are unlikely to liquefy except under severe earthquake loading conditions, while late Holocene deposits (1,000 years or less) are most likely to liquefy, and earlier Holocene (1,000-10,000 years) deposits are moderately liquefiable. Liquefaction of pre-Pleistocene deposits is rare.

Liquefaction occurs only in saturated soils, so the depth to groundwater (either free of perched) influences liquefaction susceptibility. Liquefaction susceptibility decreases with increasing groundwater depth; the effect of liquefaction are most commonly observed at sites where groundwater is within a few meters of the ground surface. At sites where groundwater levels fluctuate significantly, liquefaction hazards may also fluctuate.

Human-made soils deposited also deserve attention. Loose fills, such as those placed without compaction, are very likely to be susceptible to liquefaction. Stability of hydraulic fill dams and mine tailings piles, in which soil particles are loosely deposited by settling through water, remain an important contemporary seismic hazard. Well-compacted fills, on the other hand, are unlikely to satisfy state criteria for liquefaction susceptibility.

### 2.16.3 Effects on Soil Characteristics

Since liquefaction requires the development of excess pore pressure, liquefaction susceptibility is influenced by the soil characteristics that influence volume change
behavior. Characteristics of the soil grains like distribution of shapes, particle sizes and shape, gradation, composition etc. associated with high volume change potential influence the susceptibility of a soil to liquefy (Seed, 1979). While sands or silts are most commonly observed to liquefy, gravelly soils have also been known to have liquefied. Compositional characteristics associated with high volume change potential tend to be associated with high liquefaction susceptibility.

Particle shape can also influence liquefaction susceptibility. Soils with rounded particle shapes are known to density more easily than soils with angular grains. Consequently, they are usually more susceptible to liquefaction than angular grained soils. Rounded soil particles of uniform size are mostly susceptible to liquefaction (Poulos et al., 1985). Particle rounding frequently occurs in the fluvial and alluvial environments where loosely deposited saturated soils are frequently found, and liquefaction susceptibility is often high in those areas. Sand particles behave apparently different from clay particles. In an assortment of sand particles under a magnifying glass, some would be recognized as particles of mica, some of quartz, some of feldspar and so on. Sands particles are made up of rock minerals. They have the same composition as that of big boulder from a rock mass. The process of weathering reduces boulders to cobbles, cobbles to gravel, gravel to sand, sand to silt and even silt to rock dust which have particles of clay size. Particles of rock minerals are electrically neutral; they are acted upon by the gravitational force. There is, as such, no mechanism to induce them to stick to each other or to anything else. Except for rock dust, particles of clay size are different. Whereas sand, silt or rock dust particles are usually more or less equi-dimensional in the three orthogonal directions, clay particles are usually flaky or plate like, or needle like in shape (Figure 2.28).

![Different shapes of soil particles.](image)

For many years, liquefaction related phenomena were thought to be limited to sands. Finer-grained soils were considered incapable of generating the high pore pressure commonly associated with liquefaction and coarser-grained soils were considered too permeable to sustain any generated pore pressure long enough for liquefaction to develop.
More recently, the bounds on gradation criteria for liquefaction susceptibility have broadened. Well graded soils, due to their stable inter-locking configuration, are less prone to liquefaction. Natural silty sands tend to be deposited in a looser state, and hence are more likely to display contractive shear behavior, than clear sands. At the other end of the grain size spectrum, liquefaction of gravels has been observed in the field (Coulter and Migliaccio, 1966; Chang, 1978; Wong, 1984; Youd et al., 1985 and Yegian et al., 1994) and in the laboratory (Wong et al., 1975 and Evans and Seed, 1987). When pore pressure dissipation is impeded by the presence of impermeable layers so that truly un-drained conditions exist, gravelly soils can also be susceptible to liquefaction. Liquefaction of non-plastic silts has been observed (Ishihara, 1984 and 1985) in the laboratory and the filed indicating that plasticity characteristics rather than grain size along influence the liquefaction susceptibility of fine-grained soils. Coarse silts with bulky particle shape, which are non-plastic and cohesion-less are fully susceptible to liquefaction (Ishihara, 1993); finer silts with flaky or plate like particles generally exhibit sufficient cohesion to inhibit liquefaction. Clays remain non-susceptible to liquefaction; although sensitive clays can exhibit strain softening behavior similar to that of liquefied soil.

2.16.4 Effects of Grain Size Distribution

Liquefaction susceptibility is influenced by gradation. Chang and Ko (1982) concluded that the liquefaction potential of clean sands is strongly affected by their gradation characteristics. The resistance to liquefaction decreases with decreasing mean grain size. Uniform fine sands were found to be most vulnerable to liquefaction. Clean, medium to coarse soils are subject to potentially serious membrane penetration effects and the liquefaction resistance of these soils may be overestimated if the membrane penetration effect is not accounted for. Well-graded soils are generally has susceptible to liquefaction than poorly graded soils; the filling of voids between larger particles by smaller particles in a well-graded soil results in lower volume change potential under drained conditions and, consequently, lower excess pore pressures under un-drained conditions. Field evidence indicates that most liquefaction failures have involved uniformly graded soils. Uniformly and gap graded sands tend to form more unstable particle arrangements and are more susceptible to liquefaction than well-graded soils. Some researches on field evidence indicate that most liquefaction failures have involved uniformly graded granular
soils. Well-graded soils will also have small particles that fill in the void spaces between the large particles. This tends to reduce the potential contraction of the soil, resulting in less excess pore water pressures being generated during the earthquake. There is also ample evidence to show that uniformly graded materials, generally having a uniformity coefficient smaller than five, are more susceptible to liquefaction than well-graded materials (Ross et al., 1969 and Lee and Fitton, 1969) and that for uniformly graded soils, fine sands tend to liquefy more easily than coarse sands, gravelly soils, silts or clay. Field and laboratory observations also have shown that the resistance to liquefaction tends to increase with decreasing particle size (Ishihara, 1985). Fine-grained soils may not be pore pressure rise up to overburden pressure but high residual pore water pressure may develop. As the result of liquefaction, excessive ground settlement and sometimes flow failures involving extremely large movement of soil masses occur.

The type of soil most susceptible to liquefaction is one in which the resistance to deformation is mobilized by friction between particles. If other factors such as grain shape, uniformity coefficient and relative density are equal, the frictional resistance of cohesionless soil decreases as the grain size of soils becomes smaller.

Figure 2.29: Limits in the gradation curves separating liquefiable and non-liquefiable soils (after Tsuchida, 1970).

Tsuchida (1970) summarized the results of sieve analyses performed on a number of alluvial and diluvia soils that were known to have liquefied or not to have liquefied during earthquakes as shown in Figure 2.29. The area within the two inner curves in the figure represents sands and silty sands, the soils with the lowest resistance to liquefaction.
A soil with a gradation curve falling in the zones between the outer and inner curves is less likely to liquefy. Soils with a higher percentage of gravels tend to mobilize higher strength during shearing, and to dissipate excess pore pressures more rapidly than sands. However, there are case histories indicating that liquefaction has occurred in loose gravelly soils (Seed, 1968; Ishihara, 1985 and Andrus et al., 1991) during severe ground shaking or when the gravel layer is confined by an impervious layer. The space between the two curves farthest to the left reflects the influence of fines in decreasing the tendency of sands to densify during seismic shearing. Fines with cohesion and cementation tend to make sand particles more difficult to liquefy or to seek denser arrangements. However, non-plastic fines such as rock flour, silt and tailing slimes may not have as much of this restraining effect. Ishihara (1985) stated that clay or silt size materials having a low plasticity index value will exhibit physical characteristics resembling those of cohesionless soils, and thus have a high degree of potential for liquefaction. Walker and Steward (1989), based on their extensive dynamic tests on silts, have also concluded that non plastic and low plasticity silts, despite having their grain size distribution curves outside of Tsuchida's boundaries for soils susceptible to liquefaction, have a potential for liquefaction similar to that of sands and that increased plasticity will reduce the level of pore pressure response in silts.

A comparison of the liquefaction resistance curves obtained for the two samples of Pumice A sands considered as clean sand (negligible fines content) and Pumice C samples have fines content of 52% is shown in Figure 2.30. It is obvious that there is a reduction in liquefaction resistance as the fines content increases. However, recent studies (Matsumoto et al., 1999) have shown that the difference in effect can be attributed to the activity of fines; i.e., non-plastic fines decrease the liquefaction resistance of sands while plastic fines increase the resistance.

2.16.5 Thickness of Sand Layer

In order to induce extensive damage at level ground surface from liquefaction, the liquefied soil layer must be thick enough so that the resulting uplift pressure and amount of water expelled from the liquefied layer can result in ground rupture such as sand boiling and fissuring (Ishihara, 1985 and Dobry, 1989). If the liquefied sand layer is thin and buried within a soil profile, the presence of a non-liquefiable surface layer may prevent the effects of the at-depth liquefaction from reaching the surface. Ishihara (1985) has set up a criterion to stipulate a threshold value for the thickness of a non-liquefiable
surface layer to avoid ground damage due to liquefaction, as shown in Figure 2.31. Although this figure is believed to be speculative and should not be used for design purposes, it provides initial guidance in this matter for sites having a buried liquefiable sand layer with a standard penetration resistance of less than 10 blows per foot (0.3 m). It should also be noted that even though the thickness of a non-liquefiable surface layer exceeds the threshold thickness shown in the Figure 2.31, the ground surface may still experience some settlement which may be undesirable for certain settlement-sensitive structures. Like all of the empirical curves shown in this report, this figure, based on just three case histories, may need to be modified as more data become available.

Figure 2.30: Comparison of liquefaction resistance curves for reconstituted sands (after Matsumoto et al., 1999).

Figure 2.31: Proposed boundary curves for site identification of liquefaction-induced damage (after Ishirhara, 1985).
2.16.6 Vertical Effective Stress and Over Consolidation

Liquefaction resistance of a soil deposit increases with depth as overburden pressure increases. It is well known that an increase in the effective vertical stress increases the bearing capacity and shear strength of soil, and thereby increases the shear stress required to cause liquefaction and decreases the potential for liquefaction. From field observations it has been concluded by a number of investigators that saturated sands located deeper than 50 to 60 feet (15 to 18 m) are not likely to liquefy. That is why soil deposits deeper than about 15m are rarely found to have liquefied (Krinitzky et al., 1993). These depths are in general agreement with Kishida (1969) who states that a saturated sandy soil is not liquefiable if the value of the effective overburden pressure exceeds 2 tsf. Over consolidated soils (i.e., the soils that have been subjected to greater static pressures in the past) are more resistant to particle rearrangement and hence liquefaction as the soil grains tends to be in a more stable arrangement. Both theory and experimental data show that for a given soil a higher over consolidation ratio leads to higher lateral earth pressure at rest and thereby increases the shear stress ratio required to cause liquefaction.

2.16.7 Earthquake Loading Characteristics

The vulnerability of any cohesionless soil to liquefaction during an earthquake depends on the magnitude and number of cycles of stresses or strains induced in it by the earthquake shaking. These in turn are related to the intensity, predominant frequency, and duration of ground shaking.

2.16.8 Effects of Sample Preparation

Cyclic shear strength is different even when the relative density is kept constant but the method of sample preparation is different (Ladd, 1974; Mulilis et al., 1977 and Tatsuoka et al., 1986a). So it is important to specify a method of sample preparation if cyclic tests are to be run on reconstituted samples. As shown in Figure 2.32 are the results of cyclic triaxial tests which were performed by Mulilis et al., (1977) on saturated samples of Monterey sand prepared by two different methods air pluviation and moist tamping. Among several other methods employed by Mulilis et al., (1977), the above two methods were shown to produce the samples having the lowest and greatest resistance to liquefaction.
2.16.9 Effects of Sample Diameter

In Figure 2.33 the test data for different sample sizes from five laboratories are indicated by different signs. It may be seen that the data from several sources fall within a relatively narrow band, indicating a reasonable degree of coincidence among the values of cyclic strength from various laboratories. There is a tendency, however, for the smaller-size specimens to show slightly greater resistance to cyclic softening as compared to the large samples. This difference is attributed to the effects of system compliance arising from membrane penetration, sample seating, or tubing in the test apparatus.

Figure 2.33: Results of the co-operative tests in Japan on the cyclic strength of sand (after Toki et al., 1986).
2.16.10 Effects of Saturation

Liquefaction will not occur in dry soils. Available laboratory test results (Sherif et al., 1977) show liquefaction resistance for soils increases with decreasing degree of saturation, and that sand samples with low degree of saturation can become liquefied only under severe and long duration of earthquake shaking. Saturation of sand specimens in laboratory is necessary to provide reliable measurements of volume change and pore pressure. It is often necessary to saturate soil specimens in laboratory investigations to prevent the volume change and provide reliable measurements of pore pressure response during static or cyclic dynamic undrained loading, which is especially important for evaluation of liquefaction potential (Chaney, 1978 and Sheriff et al., 1977). A number of methods have been used for soil saturation (Lowe and Johnson, 1960). Conventional methods for saturation of soil specimens utilize back pressuring (Bishop and Henkel, 1962; Lowe and Johnson, 1960 and Wissa, 1969) or back pressing with supplemental use of carbon dioxide to produce complete saturation in various types of laboratory test specimens. For sands, the back pressures for saturation have been lowered by supplemental use of carbon dioxide. If no carbon dioxide is used, back pressures can easily become large and requiring extra time to achieve the objective. On the other hand, the percolation of carbon dioxide through the sand can lead to unwanted reactions between the CO$_3$H$_2$ and some soil components. For saturating cementing sands, application of CO$_2$ to assist saturation was ruled out because of possible component reactions (Clough, 1981 and Rad, 1982). The concept of vacuum saturation procedure can be examining best justification by developing a theoretical and experimental basis to overcome these problems.

Figure 2.34: Schematic relationship between air, water and solids in soils before and after application of back pressure.
From the experimental study of Arab et al., (2010) indicate that a lower soil saturation level results in the increased resistance of the sand to liquefaction. A decrease in the Skempton coefficient (B=Δu/Δσ) leads to a large increase in the number of cycles necessary for sand liquefaction.

The phenomenon of soil liquefaction has been observed in response to moderate and large earthquakes (Loma, 1989; Luzon, 1990; Manjil, 1990; Kobe, 1995; Manzanillo, 1995; Chi-Chi, 1999; Kocaeli, 1999; Bhuj, 2001 and Bird et al., 2004). Understanding the mechanism of liquefaction in water saturated soils and developing methods to evaluate their liquefaction potential have been the subjects of intense research (Seed and Idriss, 1971 and Seed, 1979). However, in geotechnical engineering partially saturated soils are frequently encountered, especially above the water table. Further, due to the presence of small air bubbles, soils below the water table could also be partially saturated and this issue has been explored in both experimental and analytical analyses. Thus, Mulilis et al., (1978) reported that a change in Skempton’s coefficient (B) from 0.91 to 0.97 moderately affected the liquefaction of Monterrey sand. Chaney (1978) specified that coefficient B has to be higher than 0.96 so that the soil is well saturated and Sherif et al., (1977) showed that a fine or clayey sand could be considered saturated if the value of B exceeded 0.8. More recently, laboratory test results have indicated that the liquefaction resistance of sands increases with a decreasing degree of saturation (Ishihara et al., 2001 and 2004; Yang, 2002; Yang et al., 2004; Atigh and Byrne, 2004 and Bouferra et al., 2007). As an in situ measure of the level of water saturation of a soil, Berryman et al., (1988) and Santos et al., (1990) reported the use of compression wave speed. Recently, a constitutive relation based on the multiphase approach was used to address the liquefaction of partially saturated sand (Bian and Shahrour, 2009).

The cyclic triaxial apparatus is used for investigating the effects that cyclic loading will have on a soil sample. These effects will primarily have an impact on pore pressure. It is therefore important that all test samples that are used in the cyclic triaxial apparatus are completely saturated. The criterion for saturation of the test is given by the Skempton’s constant, B, which is 1 for a fully saturated specimen. A degree of saturation of 99 % will be considered as sufficient, and the dependency of Skempton’s B as a function of pore pressure is given in Figure 2.35.
2.16.11 Effects of Number of Cycles

The magnitude and the number of cycles is the greater the cyclic stress and the number of cycles, the easier the development of liquefaction. The resistance to liquefaction may be conveniently evaluated by using the stress ratio. The effect of the number of cycles depends on the inversion of the sign of strain or stress rate (Ishihara, 1996); it is also referred to as stress/strain reversal effect. The increasing number of cycles may induce stiffness degradation, or cyclic softening of the soil and as well as energy dissipation, ultimately by heat. The two major consequences of the effect of the number of cycles, even at very low frequencies, are (i) the generation of resilient deformations under stress controlled cycles, and (ii) excess pore water pressure increase in undrained conditions (Pecker, 1984 and O'Reilly and Brown, 1991). When a very large amount of cycles occur, it may induce cyclic effects that can be referred to as fatigue. Wichtmann et al., (2005) studied the effect of thousands to millions of low amplitude cycles on strain accumulation in sand. Li and Yang (1998) studied the effect of vibration loading history on shear modulus and damping ratio of dry sand. Equivalent numbers of stress cycles of either 15 or 20 have been proposed for magnitude 7.5 earthquakes (Seed et al., 1983; Ishihara, 1993 and Idriss, 1999). Therefore, as recommended by Seed et al., (1983), 15 cycles was used to represent a MW 7.5 event.
2.16.12 Effects of Confining Pressure

Initial effective confining pressure is the lower the confining pressure the more easily liquefaction will develop. As to the initial confining pressure, it was found that the magnitude of shear stress required to cause liquefaction under a given number of cycles is approximately in direct proportion to the initial confining pressure.

To investigate the confining pressure dependency of liquefaction resistance, the curves for the three samples are illustrated in Figure 2.36. During triaxial testing, sand specimens are typically subjected to an effective confining pressure equivalent to the effective overburden pressure it is subjected to in-situ. Thus, the influence of effective confining pressure on the liquefaction resistance of reconstituted pumice sands was investigated (Figure 2.36). It is clear from the plots that the specimen subjected to higher $\sigma_c'$ requires less number of cyclic loading for liquefaction to occur (zero effective confining pressure) while the specimen subjected to low $\sigma_c'$ did not even liquefy after hundreds of cyclic load applications. This indicates that the effective confining pressure has significant effect on the development of excess pore water pressure during cyclic loading. It can be seen that the curves are almost parallel to each other, with the liquefaction resistance increasing as the confining pressure decreases, consistent with the observations made on natural sands (Rollins and Seed, 1988 and Seed and Harder, 1990). This figure is similar to those reported for reconstituted natural sands (Ishihara 1996 and Boulanger and Idriss, 2004).

![Figure 2.36: Comparison of liquefaction resistance curves for reconstituted dense pumice sands under different effective confining pressures (after Orense and Pender, 2012).](image)

Figure 2.36: Comparison of liquefaction resistance curves for reconstituted dense pumice sands under different effective confining pressures (after Orense and Pender, 2012).
2.16.13 Effects of Pore Water Pressure Response

Figure 2.34 shows that the excess pore water pressure develops rapidly in the uncemented specimen. The pore pressure ratio reaches initial liquefaction in the second cycle. For the case of one percent stabilization, the pore pressure ratio increases. When two percent cement was added to the specimen, the increase in the pore pressure ratio during cycle loading was relatively much slower, particularly in the early stages. It is of interest to note in Figure 2.37 that the maximum increase in the excess pore water pressure during loading is inversely proportional to the cement content. These results clearly demonstrate the beneficial increase of the cyclic strength of loose sand by cement stabilization. At higher cement content levels, five or eight percent, the specimens become much stronger and the applied cyclic loading does not break the strong cementation bonds, as may be concluded from the negligible axial strains that result even after more than 500 loading repetitions (Figure 2.37).

![Figure 2.37: Effect of cement content on pore pressure ratio and double amplitude axial strain for loose sand (after Saxena et al., 1998).](image)

2.16.14 Effects of Curing Period

The stability of lime, fly ash, and soil mixtures depends on the curing time and on such curing conditions as the temperature and relative humidity during curing. The environmental conditions under which curing takes place have significant influences on
the extent to which a soil may be stabilized with cement. The cyclic strength of soil-cement mix increases with the increase in the curing age. Soil-cement must be moist cured during the initial stages of its life so that moisture sufficient to meet the hydration needs of the cement can be maintained in the mixtures. Curing in the laboratory moist room meets the requirements of humidity and temperature. But in field a loose material such as straw, foliage, reed, earth etc. must cover the fresh surface. Figure 2.38 shows a consistent increase of the cyclic strength with curing period and may be attributed to the nature of the cement hydration. This increase is greater for the lower range of relative densities. Furthermore, at low relative densities, there being more voids, the exposed surface of the cement particles is much larger; therefore, the cement hydrates faster than at higher relative densities, which have less exposed surface for a given period of time. Also, at the early stages of hydration the process is faster.

![Figure 2.38: Effects of curing period on liquefaction resistance (after Saxena et al., 1998).](image)

**2.16.15 Effects of Relative Density**

Laboratory test results and field case histories indicate that, for a given soil, initial void ratio or relative density is one of the most important factors controlling liquefaction for cohesionless soils as strength for cohesive soils. It can be found by comparing the soil’s actual void ratio with the range in void ratio from loose to dense for that soil. Mathematically it can be calculated using either of the following two formulae.

\[ D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% \]  
(2.12)

\[ D_r = \frac{\gamma_d - \gamma_{dmin}}{\gamma_d - \gamma_{dmax} - \gamma_{dmin}} \times 100\% \]  
(2.13)
The relative density of sandy soil may be approximated from simple field tests as mentioned in Table 2.7. Relative density is also related to SPT value (N-value) and angle of internal friction of sandy soils.

Table 2.7: Consistency, relative density and angle of internal friction of sandy soils

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Relative Density (%)</th>
<th>Friction angle, $\varphi$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>0 – 15</td>
<td>28</td>
</tr>
<tr>
<td>Loose</td>
<td>15 – 35</td>
<td>28-30</td>
</tr>
<tr>
<td>Medium</td>
<td>35 – 65</td>
<td>30-36</td>
</tr>
<tr>
<td>Dense</td>
<td>65 – 85</td>
<td>36-41</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt; 85</td>
<td>41</td>
</tr>
</tbody>
</table>

Relative density is a measure of how the current state of sand compares with its loosest and densest states. The higher the void ratio or relative density of the sand tends to easier the inducement of liquefaction. It has also been observed that the resistance to liquefaction tends to increase with an increase in relative density at which the sample is prepared for the laboratory tests.

After collecting a number of triaxial test data, Tatsuoka et al., (1986) has shown that the resistance to liquefaction, as represented by the stress ratio, is correlated with the relative density (Figure 2.39). The effects of density were presented by Tatsuoka et al., (1986b) as shown in Figure 2.39. It may be seen that up to a relative density of 70%, the cyclic stress ratio tends to increase linearly with the relative density, but for the density in excess of 70%, the cyclic strength goes up sharply.
Figure 2.40: Effect of density on liquefaction resistance for untreated sand (after Saxena et al., 1998).

Figure 2.41: Effect of density on liquefaction resistance for cemented sand with (a) cement content (CC=1%) and (b) cement content (CC=2%) (after Saxena et al., 1998).
Liquefaction occurs principally in saturated clean sands and silty sands having a relative density less than 50%. For dense sands, however, their tendency to dilate during cyclic shearing will generate negative pore water pressures and increase their resistance to shear stress. The lower limit of relative density beyond which liquefaction will not occur is about 75%. From the research of Surendra et al., 1989; Figure 2.40 shows the variation of cyclic strength of uncemented sands with relative density. The number of cycles required to cause initial liquefaction for a given stress ratio increases as the relative density increases. Figure 2.41(a) and 2.41(b) show the effect of increase of cement content on the cyclic strength of sands of these mentioned densities.

2.16.16 Effects of Permeability

Permeability is the property of the soil which permits the passage of water through its interconnecting void. Permeability also plays a significant role in liquefaction. When movement of pore water within the soil is retarded by low permeability, pore water pressures are likely to generate during the cyclic loading. Soils with large non-plastic fines content are more likely to get liquefied because the fines inhibit drainage of excess pore pressures. The permeability of surrounding soils also affects the vulnerability of the soil deposit. Less pervious soils such as clay can prevent the rapid dissipation of excess pore water pressures that may have generated in the adjacent saturated sand deposit. Sufficient drainage above or below a saturated deposit may inhibit the accumulation of excess pore water pressure and hence liquefaction. Gravelly soils are less prone to liquefaction due to a relatively high permeability unless pore water drainage is impeded by less pervious, adjoining deposits.

2.16.17 Effects of Cement Content

There is a consistent increase in cyclic strength with cement content. Figure 2.42 compares the strengths of cemented and uncemented sands for a relative density of 43% and curing period of 15 days. This conclusion is more obvious if the stress ratio required to cause initial liquefaction in a specific number of cycles is plotted with relative density ($D_r$) at various cement contents (CC) and a given curing period (CP), as shown in Figure 2.43(a) and Figure 2.43(b).
Figure 2.42: Effect of cement content on liquefaction resistance (after Saxena et al., 1998).

Figure 2.43: Effects of cement content and density on stress ratio to cause liquefaction after (a) 10 cycles and (b) 100 cycles (after Saxena et al., 1998).

2.16.18 Effects of Lime Content

The strength of soil-lime mix is determined to a great extent by the quantity of lime added. Small quantities of lime 1 to 2 percent help in the immediate effects caused by the Base Exchange and flocculation. The tangible effect of soil-lime stabilization in increasing the strength of the mixture begins to be felt as the lime content is further increased and this is due to pozzolanic reactions resulting in the production of cementitious compounds. It is also observed that this strength gain is time-dependent and efficiencies in strength gain due to varying lime percentages are more marked for longer curing periods. Ingles and Metcalf (1972) suggested that the addition of up to 3% of lime
would modify well graded clay gravels, while 2% to 4% was required for the stabilization of silty clay, and 3% to 8% was proposed for stabilization of heavy and very heavy clays. Ingles and Metcalf (1972) further suggested that a useful guide is to allow 1% of lime (by weight of dry soil) for each 10% of clay in the soil.

2.16.19 Effects of Fine Content

The effect of confining pressure on the cyclic resistance can be mentioned confining pressure increases, the liquefaction resistance of silty sands decreases and with increasing relative density, the cyclic resistance ratio (CRR) was increased. The effect of non-plastic fine content on the liquefaction behavior of sand-silt mixtures is more complex than the effect of other variables. Several researchers have claimed that sands deposits with silt content are much more susceptible to liquefaction than clean sand. The effect of non-plastic fines on the cyclic strength of sands has also been studied in the laboratory. The majority of laboratory tests show that at similar void ratio, the presence of fines (up to at least about 30%) of the total weight decreases the cyclic strength. In most of the correlations epitomized above, the effects of the presence of fines are allowed for in such a way that the penetration resistance becomes smaller with increasing amount of fines if soils were to possess equal cyclic strength. In fact, with equal magnitude of penetration resistance, soils are observed to have increasing cyclic strength with increasing fines content as schematically illustrated in Figure 2.44, 2.45 and 2.46. This characteristic tendency is incorporated into the above mentioned correlations in terms of a parameter associated with grading, such as the fines content or the average diameter, $D_{50}$.

![Figure 2.44: Effect of fines content on the liquefaction resistance of sand non plastic fines mixtures for constant values of global void ratio and CSR = 0.23 (after Xenaki and Athanasopoulos 2003).](image-url)
2.16.20 Effects on Soil Disturbance

The cyclic tests on the undisturbed soil samples indicated higher liquefaction resistance than specimens reconstituted to the same density. In addition, undisturbed specimens exhibit higher cyclic resistance than reconstituted specimens, although their void ratio is higher than that of reconstituted specimens. This is because the soil structure (fabric, stress history, cementation, etc.) was totally erased when the reconstitution was made. The undisturbed soil samples taken at Mikkelsen Rd site have higher liquefaction resistance than specimens reconstituted to the same density.
2.17 Concluding Remark

Literature review shows that various ground improvement techniques were used as remedies of liquefaction in different types of sands in and around the world. Review of literature in the preceding sections reveals that lime, cement, fly ash-lime, rice husk ash-lime, clay-lime and bentonite stabilized soil cementation technique is a useful, more environment friendly and novel approach for the improvement of strength and deformation behavior of in-situ soil. Literature review of the above mentioned research work suggested improved liquefaction behavior of lime and Portland cement treated soil as compared to untreated soil. Fly ash-lime, rice husk ash-lime, clay-lime and bentonite stabilized soil improvement technique is a new and more environment friendly approach to determine the liquefaction potential of in-situ soil. No research work has yet been reported that investigates the effect of treatment of liquefaction susceptible different types of sand deposits in Bangladesh. As such, these factors point was felt urgent necessity to carry out a research in an attempt to improve Bangladesh soils by lime, cement, fly ash-lime, rice husk ash-lime, clay-lime and bentonite stabilizing materials to determine the liquefaction potential of liquefaction susceptible reclaiming areas of Bangladesh. This treatment method may play a vital role in mitigating seismic liquefaction.
3.1 General

The experimental investigations carried out on the treated and untreated samples have been described in this chapter. Thompson procedure was used as a guide in preparing the sample and evaluating the properties of soil lime mixtures. Cyclic and monotonic triaxial test were conducted on the prepared specimens. Cyclic and monotonous triaxial tests will be conducted in order to study the relative increase in strength of treated sand samples as affected by curing time and molding conditions. Moreover, the test results were analyzed to evaluate the cyclic shear strength of these soils and their liquefaction susceptibility.

3.2 Research Strategy

Preliminary site selection for sample collection was made to collect three different types of sands. The soil samples were collected from three selected sites and the sand samples were carried out into the laboratory for performing the cyclic triaxial test to determine the liquefaction resistance capacity of the soils. The stabilizing materials were collected from the different locations and local markets. Laboratory tests were performed on the collected soils and stabilizing additives to determine the basic index properties.

![Outline of the experimental programmed.](image)

Figure 3.1: Outline of the experimental programmed.
The specimens were prepared properly for all experiments. For preparation of treated specimens, mixing of soil samples with different stabilizing materials with varying percentages was done. Cyclic and monotonous triaxial tests will be conducted on reconstituted untreated and treated sand samples using Cyclic Triaxial Testing Machine. Static triaxial test will be carried out to get static shear strength of the soil after the failure of the specimen at cyclic loading. Analysis of laboratory data to determine the liquefaction resistance capacity of treated soil samples.

3.3 Collection of Soil Samples and Stabilizing Materials

Three different types of sandy soil (coarse, medium and fine sands) were used in this research work. The soil samples were collected from three different selected sites the Surma River, the Jamuna River and the Turag River shown in Table 3.1. Care was taken to remove any coarse aggregates, debris and vegetation. All samples were packed in large polythene bags covered by gunny bags and transported to the Geotechnical Laboratory of Bangladesh University of Engineering and Technology (BUET). The soil samples are named as follows:

<table>
<thead>
<tr>
<th>Soil</th>
<th>Type</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil A</td>
<td>Coarse (Sylhet) Sand</td>
<td>Surma River, Sylhet</td>
</tr>
<tr>
<td>Soil B</td>
<td>Medium Sand</td>
<td>Jamuna River, Bhuapur, Tangail</td>
</tr>
<tr>
<td>Soil C</td>
<td>Fine Sand</td>
<td>Turag River, Diabadi, Dhaka</td>
</tr>
</tbody>
</table>

Different cementitious materials (lime, fly ash, rice husk ash, clay, bentonite and cement) were collected from different locations and local markets. The lime (origin: Sylhet) was collected from open market near Lalbagh Fort, Dhaka and it was kept sealed to prevent carbonation until immediately before use. The pozzolana fly ash was collected from fly ash producing supplier from Kolkata, India and it was packed in large polythene bags covered by gunny bags. The rice husk ash was collected from a local rice mill (Narayanganj) and it was burned in laboratory about 300 to 450ºC in a chimney to remove excess carbon content and plant leaf. The bentonite was collected from open market (imported by China). The red clay used for this study was collected from Birampur, Dinajpur district. The cement (Brand: Super Create) was collected from open market and it was kept sealed to prevent hydration until immediately before use.
3.4 **Laboratory Test on Base Soils**

The sands collected from the fields were naturally moisture and the sand samples were carried out oven dried and air dried into the laboratory for performing the following test to determine the physical properties of untreated soil (Table 3.2).

3.5 **Laboratory Test on Cementitious Additives**

The stabilizing materials collected from the different locations were air dried. The collected clay sample was dried in open air. The clay lumps were broken down with a wooden mallet. The lime stone were broken down with a steel mallet. The rice husk ash was burned about 300 to 450°C in a chimney to remove excess carbon content. The following tests were performed in determining the physical properties of pozzolonic additives (Table 3.3).

<table>
<thead>
<tr>
<th>Table 3.2: Laboratory test standards to determine the properties of soils</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Grain size analysis</strong></td>
</tr>
<tr>
<td><strong>Specific gravity</strong></td>
</tr>
<tr>
<td><strong>Maximum and minimum densities</strong></td>
</tr>
<tr>
<td><strong>Max. dry density and Opt. moisture content</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3.3: Laboratory test standards to determine the properties of additives</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Liquid, plastic and shrinkage limits</strong></td>
</tr>
<tr>
<td><strong>Hydrometer analysis</strong></td>
</tr>
<tr>
<td><strong>Specific gravity</strong></td>
</tr>
<tr>
<td><strong>Maximum and minimum densities</strong></td>
</tr>
<tr>
<td><strong>Max. dry density &amp; Opt. moisture content</strong></td>
</tr>
<tr>
<td><strong>pH test</strong></td>
</tr>
</tbody>
</table>

The chemical properties investigated are the soil pH. The soil pH was determined by using pH meter with glass electrode (Metrohm 691) from extraction of soil water ratio is 1:2.5. Details of pH determination are given in Appendix A.

To study the bonding behavior of treated sands, scanning electro-microscope for mineralogical study were carried out at Bangladesh University of Engineering and Technology (BUET) Glass and Ceramics laboratories. The scanning electro-microscope
was carried out to identify the reaction products that are responsible for the modification of the bonding behavior of the soil properties. The procedure involved in the behavior of sands and additives’ and results using scanning electro-microscope is described briefly in Appendix B.

3.6 Mixing of Soil Samples and Stabilizing Materials

Mixing of soil samples were done with different stabilizing materials. The samples were prepared in a standard compaction mold of 71 mm diameter and 142 mm height by tamping the samples in four layers using a tamping rod of weight 1 kg. Samples were prepared at a constant relative density of 50%. The sands and additives mixtures were prepared by dry mixing the proper amount of dry sand and dry additives in batches. Before mixing, the water content of the air-dry soil was determined. The mixing was done thoroughly in a steel pan. The sand-additive was thoroughly mixed by hand until homogeneity was reached. Then required quantity of water was gradually added in to the soil mass and was blended. Mixtures were prepared at or near optimum moisture content as described in Section 3.7.

3.6.1 Mixing of Soil Samples and Lime

To investigate the effect of lime on geotechnical properties of these soils, lime was added to each specimen at room temperature, in the order of 2, 5 and 10 percent by weight. The soil-lime mixtures were prepared by dry mixing the proper amount of soil and lime in batches to ensure soil–lime pozzolanic reactions. Before mixing, the water content of the air-dry soil was determined. The mixing was done thoroughly in a steel pan. The lime was thoroughly mixed by hand until homogeneity was reached. Then required quantity of water was added in to the soil mass and was blended. Mixtures were prepared at or near optimum moisture content as described in section 3.7. The flow chart for the preparation of soil-lime stabilized specimens for cyclic shear strength test is shown in Figure

3.6.2 Mixing of Pozzolonic Additives-Lime for Samples Preparation

For preparation of each type of treated specimens, several combinations of lime and additives concentrations were considered. The different combinations of lime and additives concentrations are shown in Table 3.4.
Figure 3.2: Flow chart for different stages for the preparation of lime treated specimens for cyclic triaxial test.

Table 3.4: Mixing portion of fly ash-lime, rice husk ash-lime, clay-lime for samples preparation

<table>
<thead>
<tr>
<th>Type of Stabilizing Materials</th>
<th>Lime Percentages (Weight Basis)</th>
<th>Fly Ash, Rice Husk Ash and Clay Percentages (Weight Basis)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fly ash-lime (Class F fly ash)</td>
<td>15</td>
<td>85</td>
</tr>
<tr>
<td>Rice husk ash-lime</td>
<td>15</td>
<td>85</td>
</tr>
<tr>
<td>Clay-lime</td>
<td>20</td>
<td>80</td>
</tr>
</tbody>
</table>

Class F fly ash is designated in ASTM C618. Class F is generally low in lime, usually under 15%, and contains a greater combination of silica, alumina and iron (greater than 70%). Normally class F fly ash was used in soil stabilization. For use of waste material rice husk ash and natural pozzolan clay there was no standard mixing proportions with lime for soil stabilization. In this study the above mixing portion was used for rice husk ash-lime and clay-lime combinations.
3.6.3 Mixing of Stabilizing Materials and Soils

For preparation of each type of treated specimens, the following amounts of stabilizing materials and soils were considered shown in Table 3.5.

<table>
<thead>
<tr>
<th>Name of the Specimens</th>
<th>Stabilizing Materials (Weight %)</th>
<th>Sands (Weight %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fly ash-lime treated specimen</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>Rice husk ash-lime treated specimen</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>Clay-lime stabilized specimen</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>Bentonite treated specimen</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>Cement treated specimen</td>
<td>2</td>
<td>98</td>
</tr>
<tr>
<td>Lime treated specimen</td>
<td>2</td>
<td>98</td>
</tr>
<tr>
<td>Lime treated specimen</td>
<td>5</td>
<td>95</td>
</tr>
<tr>
<td>Lime treated specimen</td>
<td>10</td>
<td>90</td>
</tr>
</tbody>
</table>

3.7 Compaction Tests on Stabilized Soils

The compaction characteristics of treated soils were studied in the laboratory using Modified Proctor test based on the ASTM D1557. Mixing of soil samples will be done with different stabilizing materials with varying percentages. Compaction tests were performed on lime-soil, fly ash-lime-soil, rice husk ash-lime-soil, clay-lime-soil and bentonite-soil mixture with different lime content and compaction energy. Cyclic triaxial tests were performed at different moisture curing days. The optimum moisture content and maximum dry density of the lime, lime-fly ash, lime-rice husk ash, lime-clay, cement and bentonite treated soils were determined by the Modified Proctor Compaction Test (ASTM D1557) method. These laboratory compaction test methods used to determine the relationship between molding water content and dry unit weight of soils (compaction curve) compacted in a 4inch (101.6 mm) inside diameter and 4.60 inch height cylindrical mold with a 10.00 lbf (44.48 N) rammer dropped from a height of 18.00 inch (457.2 mm) producing a compactive effort of 56 000 ft-lb/ft³ (2700 kN-m/m³).

Air-dried soils passing through No. 4 sieve were mixed with desired quantity of stabilizing additives. The mixture was blended until it formed a uniform color. The water was then added and it was thoroughly mixed to ensure even distribution. After applying...
the first increment of water, the samples were compacted. A series of soil-lime mixture of varying moisture contents were prepared. These samples were compacted in 5 (five) layers of approximately equal height. Each layer was compacted by 25 blows with the rammer, falling from a free height of 18 inch. The amount of used soil-lime was such that the fifth compacted layer was slightly above the top of mold by not exceeding 6 mm. Moisture content and dry density were determined for each compacted sample. Compaction tests to determine the effect of lime on maximum dry density and optimum moisture content were carried out on soils after mixing with 2%, 5%, and 10% lime by weight. The optimum moisture content and maximum dry density were determined.

3.8 Preparation of Stabilized Specimens

Soil specimens used in this study were of 71 mm in diameter and 142 mm in height. The specimens were formed by using wet tamping method in a split mold. In order to prepare test samples for cyclic strength test of this research work a compaction mold and a tamping rod were fabricated.

3.8.1 Preparation of Specimen Compaction Mold

The mold used for compacting untreated soil and treated soils was fabricated using mild steel seamless pipe. The mold is 71 mm inner diameter and 142 mm height composed of two longitudinal halves joined with nut-bolts. The removable collar and base plate are similar to that of used for Modified Proctor Test (ASTMD1557). A tamping rod was fabricated using mild steel and its height controller casing was a seamless stainless steel pipe. The tamping rod weighs 1kg, and has a drop of 6 inches. The mold and tamping rod are shown in Figure 3.3.

3.8.2 Preparation of Test Specimen

Tests were carried out on reconstituted untreated and treated prismatic sand samples prepared at the particular relative density of 50%. Cylindrical soils specimens of 71 mm diameter and 142 mm height were used in this study (Figure 3.5). In order to obtain a uniform density throughout the reconstituted sand specimen for determination of optimum cyclic strength, the more consistent compaction method of specimen preparation suggested by Ladd (1978) was used. The air-dried weighted amount of soil and stabilizing material was mixed and blended until it formed a uniform color. The water (10 percent)
was then added and it was thoroughly mixed to ensure even distribution and then compacted in 4 (four) equal layers by a 1 kg weight tamping rod (Figure 3.3) falling from a height of 6 inch that delivers a number of blows to each layer to achieve the target relative density. In this research, number of blows per layer (Figure 3.4) was determined by trial to achieve target relative density. After preparing the specimen, it was encased by a thin rubber membrane and griped top and bottom with O-ring after that placed inside a plastic cylindrical chamber that was filled with water (Figure 3.4).

Figure 3.3: Sampling split mold with collar and tamping rod for preparing test sample.

Figure 3.4: Rammer pattern for compaction of specimen preparation.
Figure 3.5: Standard cylindrical test specimen of 71 mm diameter and 142 mm height used in this study.

3.9 Curing of Specimen

To study the long term cyclic strength of stabilized soil, the prepared samples were cured with water moisture at 7 days and 28 days. The stabilized samples were kept in airtight buckets and then sealed by rope bags, geotextile and cloths at top for a period of 7 days and 28 days at a constant room temperature of 25°C for moisture curing. Cyclic triaxial tests were performed at different moisture curing days.

3.10 Cyclic Triaxial Tests

This kind of test is widely used for the characterization of cyclic behavior of fine-grained and cohesionless soil. In a typical cyclic triaxial testing device (Figure 3.9), the cylindrical sample, which is under a constant hydrostatic pressure, is cyclically sheared by the application of a vertical load. Pore water pressure or the volumetric variation of water inside the sample must be recorded, as well as vertical displacement and force.

After the desired curing period the cyclic triaxial and monotonic triaxial tests were carried out to study the cyclic shear strength and post-liquefaction behavior of untreated and treated samples. The tests were conducted according to ASTM D 3999 test procedure. The cyclic triaxial and monotonic triaxial testing programmed for untreated and different stabilizing material, indicated in Table 3.5, treated Soil A, Soil B and Soil C specimens were presented in Table 3.6(a), 3.6(b) and 3.6(c) respectively.
The prepared specimen was placed on a loading platform of GEOCOMP Corp. cyclic triaxial machine immediately after removing from the curing chamber. The specimen was carefully aligned in the cyclic triaxial machine and the loading device was adjusted carefully such that the upper platoon just makes contact with the specimen.

When the preparation of the required size test specimen was complete, the specimen was placed between top and bottom loading patterns. After assembling, the specimen was surrounded by a thin latex rubber membrane and positioning O-rings around the base pedestal and top cap after placing inside a plastic cylindrical chamber filled with water.

Table 3.6(a): Programme of cyclic and monotonic triaxial test experiments for Soil A

<table>
<thead>
<tr>
<th>Test No</th>
<th>Source Soil</th>
<th>Preparing Date</th>
<th>Initial Wt.(gm)</th>
<th>Testing Date</th>
<th>Wt. on Curing (gm)</th>
<th>Curing Age (Days)</th>
<th>CSR</th>
<th>Static Triaxial Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil A</td>
<td>04.10.14</td>
<td>944.4</td>
<td>04.10.14</td>
<td>944.8</td>
<td>0</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Soil A</td>
<td>18.10.14</td>
<td>958.6</td>
<td>18.10.14</td>
<td>958.6</td>
<td>0</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Soil A</td>
<td>28.05.14</td>
<td>959</td>
<td>28.05.14</td>
<td>959</td>
<td>0</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Soil A</td>
<td>22.05.14</td>
<td>950.9</td>
<td>22.05.14</td>
<td>950.3</td>
<td>0</td>
<td>0.17</td>
<td></td>
</tr>
</tbody>
</table>

**SOIL A + 10% Bentonite**

| 5       | Soil A      | 13.10.14       | 987.3          | 20.10.14     | 987.3             | 7                 | 0.4 |                     |

**SOIL A + 10% Lime**

| 6       | Soil A      | 19.10.14       | 968            | 26.10.14     | 979.8             | 7                 | 0.40|                     |

**SOIL A + 10% Fly Ash-Lime**

| 7       | Soil A      | 21.10.14       | 948.6          | 28.10.14     | 987               | 7                 | 0.40| *S-U                |

**SOIL A + 10% Clay-Lime**

| 8       | Soil A      | 22.10.14       | 942.9          | 29.10.14     | 975.6             | 7                 | 0.40|                     |

**SOIL A + 10% Rice Husk Ash-Lime**

| 9       | Soil A      | 20.10.14       | 967            | 27.10.14     | 975.4             | 7                 | 0.40| S-U                 |

**SOIL A + 2% Cement**

| 10      | Soil A      | 21.05.14       | 960.6          | 28.05.14     | 962.6             | 7                 | 0.40|                     |

Note: * S-D = Static Drain Compression Test  
** S-U = Static Undrain Compression Test  
† E-U = Static Undrain Extension Test  
++ E-D = Static Drain Extension Test
Table 3.6(b): Programme of cyclic and monotonic triaxial test experiments for Soil B

<table>
<thead>
<tr>
<th>Test No</th>
<th>Source Soil</th>
<th>Preparing Date</th>
<th>Initial Wt.(gm)</th>
<th>Testing Date</th>
<th>Wt. on Curing (gm)</th>
<th>Curing Age (Days)</th>
<th>CSR Static Triaxial Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil B</td>
<td>31.05.14</td>
<td>949</td>
<td>31.05.14</td>
<td>949.0</td>
<td>0</td>
<td>0.20 *S-D</td>
</tr>
<tr>
<td>2</td>
<td>Soil B</td>
<td>31.05.14</td>
<td>955</td>
<td>01.06.14</td>
<td>955.5</td>
<td>1</td>
<td>0.30 *E-U</td>
</tr>
<tr>
<td>3</td>
<td>Soil B</td>
<td>01.06.14</td>
<td>950</td>
<td>02.06.14</td>
<td>945.3</td>
<td>1</td>
<td>0.15</td>
</tr>
<tr>
<td>4</td>
<td>Soil B</td>
<td>02.06.14</td>
<td>955</td>
<td>03.06.14</td>
<td>951.1</td>
<td>1</td>
<td>0.25 **S-U</td>
</tr>
<tr>
<td>5</td>
<td>Soil B</td>
<td>16.07.14</td>
<td>963.6</td>
<td>17.07.14</td>
<td>962.8</td>
<td>1</td>
<td>0.35 S-D</td>
</tr>
<tr>
<td>5</td>
<td>Soil B</td>
<td>29.06.14</td>
<td>949.7</td>
<td>16.07.14</td>
<td>950.5</td>
<td>17</td>
<td>0.40 **E-D</td>
</tr>
<tr>
<td>6</td>
<td>Soil B</td>
<td>05.08.14</td>
<td>931</td>
<td>06.08.14</td>
<td>931.0</td>
<td>1</td>
<td>0.28 S-U</td>
</tr>
<tr>
<td>7</td>
<td>Soil B</td>
<td>06.08.14</td>
<td>935</td>
<td>06.08.14</td>
<td>935.0</td>
<td>0</td>
<td>0.22 S-U</td>
</tr>
<tr>
<td>8</td>
<td>Soil B</td>
<td>28.05.14</td>
<td>969</td>
<td>04.06.14</td>
<td>978</td>
<td>7</td>
<td>0.25</td>
</tr>
<tr>
<td>9</td>
<td>Soil B</td>
<td>03.06.14</td>
<td>968</td>
<td>10.06.14</td>
<td>971</td>
<td>7</td>
<td>0.30 S-U</td>
</tr>
<tr>
<td>10</td>
<td>Soil B</td>
<td>27.06.14</td>
<td>954.6</td>
<td>03.07.14</td>
<td>982.7</td>
<td>7</td>
<td>0.35 S-D</td>
</tr>
<tr>
<td>11</td>
<td>Soil B</td>
<td>29.06.14</td>
<td>956.8</td>
<td>05.07.14</td>
<td>959.7</td>
<td>7</td>
<td>0.40 E-D</td>
</tr>
<tr>
<td>12</td>
<td>Soil B</td>
<td>12.07.14</td>
<td>932</td>
<td>20.07.14</td>
<td>932.7</td>
<td>7</td>
<td>0.30 S-U</td>
</tr>
<tr>
<td>13</td>
<td>Soil B</td>
<td>13.07.14</td>
<td>941.7</td>
<td>21.07.14</td>
<td>941</td>
<td>7</td>
<td>0.35 S-D</td>
</tr>
<tr>
<td>14</td>
<td>Soil B</td>
<td>14.07.14</td>
<td>937.5</td>
<td>22.07.14</td>
<td>937</td>
<td>7</td>
<td>0.40 E-D</td>
</tr>
<tr>
<td>15</td>
<td>Soil B</td>
<td>16.07.14</td>
<td>941.7</td>
<td>23.07.14</td>
<td>941.1</td>
<td>7</td>
<td>0.45</td>
</tr>
<tr>
<td>16</td>
<td>Soil B</td>
<td>05.08.14</td>
<td>957</td>
<td>12.08.14</td>
<td>1000.5</td>
<td>7</td>
<td>0.45 E-D</td>
</tr>
<tr>
<td>17</td>
<td>Soil B</td>
<td>05.08.14</td>
<td>952</td>
<td>13.08.14</td>
<td>1019.1</td>
<td>7</td>
<td>0.50 E-U</td>
</tr>
<tr>
<td>18</td>
<td>Soil B</td>
<td>12.08.14</td>
<td>953.4</td>
<td>19.08.14</td>
<td>988.9</td>
<td>7</td>
<td>0.45</td>
</tr>
<tr>
<td>19</td>
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<td>21.06.14</td>
<td>958</td>
<td>19.07.14</td>
<td>967.5</td>
<td>28</td>
<td>0.35</td>
</tr>
<tr>
<td>20</td>
<td>Soil B</td>
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<td>15.08.14</td>
<td>950</td>
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<td>0.50 E-U</td>
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<td>17.08.14</td>
<td>974.8</td>
<td>28</td>
<td>0.40 E-D</td>
</tr>
<tr>
<td>22</td>
<td>Soil B</td>
<td>13.07.14</td>
<td>946</td>
<td>16.08.14</td>
<td>948</td>
<td>28</td>
<td>0.50 E-U</td>
</tr>
<tr>
<td>23</td>
<td>Soil B</td>
<td>16.07.14</td>
<td>960</td>
<td>18.08.14</td>
<td>969.3</td>
<td>28</td>
<td>0.75 S-U</td>
</tr>
<tr>
<td>24</td>
<td>Soil B</td>
<td>17.07.14</td>
<td>945.5</td>
<td>19.08.14</td>
<td>981.4</td>
<td>28</td>
<td>0.80 E-D</td>
</tr>
<tr>
<td>25</td>
<td>Soil B</td>
<td>22.07.14</td>
<td>950.9</td>
<td>24.08.14</td>
<td>964.3</td>
<td>28</td>
<td>0.70 S-D</td>
</tr>
<tr>
<td>26</td>
<td>Soil B</td>
<td>21.07.14</td>
<td>945.3</td>
<td>25.08.14</td>
<td>1045</td>
<td>28</td>
<td>0.90 S-D</td>
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<tr>
<td>27</td>
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<td>949.8</td>
<td>28</td>
<td>0.80 S-U</td>
</tr>
<tr>
<td>28</td>
<td>Soil B</td>
<td>22.07.14</td>
<td>931.9</td>
<td>27.08.14</td>
<td>1006.5</td>
<td>28</td>
<td>0.75 S-D</td>
</tr>
<tr>
<td>29</td>
<td>Soil B</td>
<td>31.08.14</td>
<td>958</td>
<td>25.10.14</td>
<td>1029</td>
<td>7</td>
<td>0.40 S-D</td>
</tr>
<tr>
<td>30</td>
<td>Soil B</td>
<td>12.10.14</td>
<td>944.4</td>
<td>19.10.14</td>
<td>944.4</td>
<td>7</td>
<td>0.40 S-D</td>
</tr>
<tr>
<td>31</td>
<td>Soil B</td>
<td>18.10.14</td>
<td>944.4</td>
<td>26.10.14</td>
<td>951</td>
<td>7</td>
<td>0.40 S-D</td>
</tr>
<tr>
<td>32</td>
<td>Soil B</td>
<td>22.08.14</td>
<td>956</td>
<td>04.11.14</td>
<td>999.8</td>
<td>7</td>
<td>0.40</td>
</tr>
<tr>
<td>33</td>
<td>Soil B</td>
<td>18.10.14</td>
<td>944.4</td>
<td>26.10.14</td>
<td>951</td>
<td>7</td>
<td>0.40 S-D</td>
</tr>
<tr>
<td>34</td>
<td>Soil B</td>
<td>28.05.14</td>
<td>960</td>
<td>04.06.14</td>
<td>973.2</td>
<td>7</td>
<td>0.40</td>
</tr>
</tbody>
</table>
Table 3.6(c): Programme of cyclic and monotonic triaxial test experiments for Soil C

<table>
<thead>
<tr>
<th>Test No</th>
<th>Source Soil</th>
<th>Preparing Date</th>
<th>Initial Wt.(gm)</th>
<th>Testing Date</th>
<th>Wt. on Curing (gm)</th>
<th>Curing Age (Days)</th>
<th>CSR</th>
<th>Static Triaxial Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Soil C</td>
<td>29.05.14</td>
<td>959</td>
<td>31.05.14</td>
<td>967.0</td>
<td>2</td>
<td>0.30</td>
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</tr>
<tr>
<td>2</td>
<td>Soil C</td>
<td>26.08.14</td>
<td>951.7</td>
<td>29.08.14</td>
<td>951.7</td>
<td>3</td>
<td>0.25</td>
<td>E-D</td>
</tr>
<tr>
<td>3</td>
<td>Soil C</td>
<td>12.10.14</td>
<td>988</td>
<td>12.10.14</td>
<td>1031.0</td>
<td>0</td>
<td>0.27</td>
<td>S-D</td>
</tr>
<tr>
<td>4</td>
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<td>30.11.14</td>
<td>959</td>
<td>30.11.14</td>
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<td>0.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SOIL C + 10% Bentonite</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Soil C</td>
<td>07.10.14</td>
<td>965</td>
<td>14.10.14</td>
<td>975</td>
<td>7</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Soil C</td>
<td>14.10.14</td>
<td>1015</td>
<td>15.10.14</td>
<td>1025</td>
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<td>0.40</td>
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</tr>
<tr>
<td></td>
<td>SOIL C + 10% Lime</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Soil C</td>
<td>14.10.14</td>
<td>1003</td>
<td>21.10.14</td>
<td>1003</td>
<td>7</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SOIL C + 10% Fly Ash-Lime</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Soil C</td>
<td>25.10.14</td>
<td>972</td>
<td>01.11.14</td>
<td>971.7</td>
<td>7</td>
<td>0.40</td>
<td>E-D</td>
</tr>
<tr>
<td></td>
<td>SOIL C + 10% Clay-Lime</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Soil C</td>
<td>26.10.14</td>
<td>986.13</td>
<td>02.11.14</td>
<td>996.5</td>
<td>7</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SOIL C + 10% Rice Husk Ash-Lime</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Soil C</td>
<td>27.10.14</td>
<td>964.7</td>
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<td>972.5</td>
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<td>0.4</td>
<td>E-D</td>
</tr>
<tr>
<td></td>
<td>SOIL C + 2% Cement</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>Soil C</td>
<td>26.05.14</td>
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<td>03.06.14</td>
<td>962.5</td>
<td>7</td>
<td>0.40</td>
<td></td>
</tr>
</tbody>
</table>

3.10.1 Indirect Determination of Permeability of the Specimens

The permeability tests have done by an indirect method of Variable or Falling Head method. In this method the diameter of the specimen was 71 mm and height 142 mm. The soil is mixed with 10 percent of water amounts of the total sample and then compacted in four equal layers by a tamping rod to obtain 50% relative density specified in Section 3.8.2. Water was fallen from a 75 mm diameter water supply bottle at a falling height of 62 mm. The top of water level in the bottle was not constant throughout the test. After that the water was passing through the sample and collected at a constant head chamber of 100 ml volume. The water was passing through the specimen and collected by a 2 mm inner diameter stand pipe. The time was recorded and coefficients of permeability (k) have been estimated by using the modified Darcy’s law on untreated and treated specimens. The results of permeability are listed in Appendix C.
3.10.2 Saturation Stage

The objective of the saturation phase of the test is to fill all voids in the specimen with water without undesirable pre stressing of the specimen or allowing the specimen to swell. After the cell is filled with water, the test specimen has to be fully saturated. Before the sample is saturated with water it first needs to be saturated with carbon dioxide (CO₂) because CO₂ is easily dissolved as it is heavier than other atmospheric air components. When CO₂ let into the soil from the lower pressure head the atmospheric air will be driven out. After assembling and filling the triaxial chamber with water, the optional vacuum source carbon dioxide gas (CO₂) is permitted about one hour to pass through the specimen prior to seepage saturation for obtaining complete saturation (Ishihara et al., 1978). Next the soil sample has to be saturated with water. This is done by letting water from the small water container (top in the left column) through the lower pressure head. The water has to pass through the sample and out through the upper pressure head over into the small water container (bottom of the left column) shown in Figure 3.7. After that the distilled water was allowed slowly to seep up through the specimen from the bottom by chamber pressure of 5 kPa with all drainage valves closed. Allow a minimum of 30 min for stabilization of the specimen pore water pressure. It is mentionable that, at the beginning 20 kPa confining pressure was permitted by compression of the fluid in the chamber. At the end of this process the machine was switched on. Saturation is accomplished by applying a back pressure to the specimen pore water to drive remaining air into solution. Sufficient Back pressure is applied to the specimen in steps and evaluation of the degree of saturation is done at appropriate intervals by measuring Skempton’s pore pressure parameter B and a value of B greater than 0.95 is ensured (Figure 3.6). In Holtz et al. (2011) an example of very dense sand is given, which is shown in Table 3.7. The volume change during saturation is measured using automatic digitally controlled volume changed apparatus.

Table 3.7: Skemptons B as a function of degree of saturation, after Holtz et al. (2011)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>S = 100 %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft, normally consolidated clays</td>
<td>0.9998</td>
</tr>
<tr>
<td>Compacted silts and clays; lightly over consolidated clays</td>
<td>0.9988</td>
</tr>
<tr>
<td>Over consolidated stiff clays; sands at most densities</td>
<td>0.9877</td>
</tr>
<tr>
<td>Very dense sands</td>
<td>0.9130</td>
</tr>
</tbody>
</table>
Figure 3.6: Pressure to attain various degree of saturation (after Lowe and Johnson, 1960).

Figure 3.7: Mechanism of carbon dioxide gas (CO₂) and de-aired water seep up through the specimen from the bottom.
3.10.3 Consolidation Stage

After ensuring saturation, the sample was allowed to consolidate isotropically under the required effective confining stress by compression of the fluid in the chamber, which corresponds to normally consolidated state of soil in the field. The duration of the consolidation process was varied from about 30 minutes to 12 hours. The consolidation volume and height of the specimen are obtained using the software through the data acquisition system. The void ratio after consolidation is estimated and found that the post consolidated relative density is only slightly different from the initial relative density. It is due to fact that the test are carried out at relatively higher densities. Similar observations are also reported by Ishihara (1996). Therefore, in the present study, the test results are interpreted through initial relative density of sand. All relative densities reported here are post consolidation relative densities.

Same sample preparation, saturation, and consolidation techniques were adopted for both cyclic and monotonic triaxial test.

3.10.4 Cyclic Triaxial Testing Stage

After consolidation of the soil sample cyclic loading is started. The parameters for cyclic loading are chosen by the operator. The controlling and data acquisition software provides certain predefined testing options according to respectively standard ASTM D3999 and standard ASTM D5311. Following the standards the cyclic loading is to be performed under undrained conditions. Thus only consolidation stress conditions, loading frequency and amplitude are to be defined by the operator. During cyclic loading the cell pressure is kept constant. The cyclic load is to be applied with a sinusoidal wave shape following several recommendations according to the mentioned standards.

The test methods fit for undisturbed as well as for reconstituted soil samples of both fine-grained and coarse-grained material. The amplitude of the cyclic load can be given either by the stress or the strain level to be applied to the specimen. After standard ASTM D3999 the specimen is loaded cyclically with 40 cycles and the first half cycle in compression.

Standard ASTM D5311 describes a test method for the investigation of the load controlled cyclic triaxial strength (also called the liquefaction potential) of soil using the cyclic triaxial testing apparatus. This test method fits for undisturbed as well as for
reconstituted soil samples and is generally applicable for testing cohesionless free draining soils of relatively high permeability. After standard ASTM D5311 the specimen is loaded cyclically with the first cycle applied in compression. The load is to be cycled until either cyclic double amplitude vertical strain exceeds 20%, the single amplitude strain in either extension or compression exceeds 20%, 500 load cycles are exceeded, or the load wave form deteriorates beyond acceptable values.

Beside these predefined loading options, user defined settings can be made using the options of the control software of the testing equipment. In this case it is possible to use several wave shapes for the cyclic loading. Triangular wave shapes as well as pulses can be applied. Even self-programmed wave shapes are applicable. The operator can choose between drained or undrained loading, number of cycles, and termination stress or strain. The load can be applied alternatively in compression and extension, compression only, or extension only.

Isotropically consolidated undrained cyclic triaxial tests were performed on three different sand samples in this study. The testing apparatus used in this study is capable of conducting static as well as cyclic loadings. The triaxial system used is shown in Figure 3.9. The automation of a cyclic triaxial test requires a Load Trac-II load frame, two Flow Trac-II units, a Cyclic-RM unit, a cyclic triaxial test cell and a computer.

The cyclic triaxial testing system is manufactured by Geocomp. The system is based on the Geocomp monotonic triaxial system with additional components for cyclic loading. The system consists of the Load Trac II load frame and two piston pumps to apply and measure cell and sample pressures. All static vertical loading is applied by the stepper motor at the base of the load frame. Cyclic loading is performed by a hydraulic cylinder coupled with a servo control valve and a hydraulic actuator driven by a 2.5 gpm/3000 psi hydraulic pump. The hydraulic cylinder applies the load from above of the cell. The cylinder is attached to the top of the loading frame. A second set of sample and cell pressure transducers that connect directly to the cell are used during the cyclic phase of testing (Geocomp, 2007).

Following consolidation a cyclic stress is applied under stress controlled conditions. The cyclic phase is started by first powering the hydraulic pump. The initial position of the hydraulic cylinder is fully retracted. Because the sample is loaded in compression and extension during the cyclic phase of testing the cylinder must be able to act in both
directions. The system centers the cylinder by lowering the base as the servo controls the hydraulic cylinder to maintain a constant load on the sample. After the hydraulic cylinder is centered, cyclic loading commences. After a minimum of 10% accumulated axial strain is achieved the test is terminated.

The cyclic triaxial software fully automates a cyclic triaxial test that is run on a Load Trac-II/Flow Trac-II/Cyclic-RM system (Figure 3.9). Once a soil specimen is in place and the test conditions selected, the software will run all phases of the test up to the cyclic phase. The program will then prompt to make some adjustments before start the cyclic phase. The software will then run the cyclic phase (vertical cyclic, horizontal cyclic or combined). Test data will be stored in a file for subsequent reduction and plotting by way of the report capability built into the software. The configuration for any test can be used to define the initial conditions of a subsequent test. This capability allows an operator familiar with cyclic triaxial to set up and start a test within a few minutes after the specimen has been prepared.

![Cyclic Triaxial System](image)

**Figure 3.8**: Cyclic-RM system installation diagram.
Figure 3.9: Cyclic triaxial testing system (Geocomp Corporation) used for the present study.
3.10.5 Properties Sheet

The Properties Sheet is used to enter general information about the test, the physical characteristics of the specimen, and the parameters that are used for the test. The Properties Sheet is composed of ten Property Pages.

3.10.5.1 Project Page

The Project page (Figure 3.10) is used to enter general information concerning the project and the specimen to be tested. A test does not use any of the information entered on this page.

![Figure 3.10: Project page of cyclic triaxial software.](image)

3.10.5.2 Specimen Page

The Specimen page is used to enter the physical parameters that describe the specimen to be tested (Figure 3.11). The initial diameter is used by cyclic triaxial to determine the applied stress during consolidation and so needs to be an accurate value. The other parameters are used in the data reduction and reporting feature of cyclic triaxial.

3.10.5.3 Water Content Page

The Water Content page is used to enter data related to specimen water content before the test and after the test (Figure 3.12). The water content information is not used to control the test.
3.10.5.4 Read Table Page

The Read Table page (Figure 3.13) is used to control when sensor output values are read and stored during a test.
3.10.5.5 Test Parameters Page

The Test Parameters page (Figure 3.14) is used to choose the Area Correction type and to enter two types of Correction Factor. It is used to choose the Start Phase for a test. The cyclic triaxial software controls five phases of a cyclic triaxial test.

![Figure 3.14: Test parameters page of cyclic triaxial software.](image)

3.10.5.6 Initialization

The initialization phase of a cyclic triaxial test (Figure 3.15) applies a small horizontal stress (cell pressure or confining stress) and vertical stress (confining stress plus the calculated stress applied by the piston) along with pore pressure (sample pressure) to the specimen for a specific time while the system is checked for possible leaks. The pore pressure, and should always be maintained at a value less than the cell pressure. The three stresses define the total and effective stress that will be applied to the specimen during the initialization phase of the test.

![Figure 3.15: Initialization page of cyclic triaxial software.](image)
3.10.5.7 Consolidation/A Table

Two independent consolidation phases and a saturation phase are provided by cyclic triaxial. The Consolidation/A Table page (Figure 3.16) is used to enter parameters that control a consolidation phase prior to the saturation phase. In this stage consolidates specimen in steps to specified horizontal and vertical stresses before saturation.

![Figure 3.16: Consolidation/A table page without K0 consolidation of cyclic triaxial software.](image)

3.10.5.8 Saturation

Saturates specimen to specified conditions by incrementally was increasing the cell pressure and pore pressure while the system is checked for possible leaks. The saturation phase (Figure 3.17) consists of incremental increases in cell pressure and pore pressure in order to achieve maximum saturation while a constant effective stress (total stress minus pore pressure) on the specimen is maintained. As the increase in cell pressure is applied, the system will monitor the increase in pore pressure. The ratio of the pore pressure increase to the cell pressure increase is the saturation ratio or B value. Before the next cell pressure increment is applied, the pore pressure is raised and then held constant so that the effective stress on the specimen is maintained. This will cause air in the specimen to be absorbed and will result in a higher B value. The B value will be close to 1 when the specimen is fully saturated.
The system will continue to incrementally increase the cell pressure and pore pressure until the B value is equal to or exceeds the value entered in the Minimum Saturation Ratio text field or the cell pressure reaches the value entered in the Maximum Cell Pressure text field. The Minimum Saturation Ratio (B value) is typically 0.95.

If the cell pressure reaches the Maximum Cell Pressure value and the Minimum Saturation Ratio has not been reached, the system will begin a cyclic process of decreasing and increasing the cell pressure (at constant effective stress) while continuing to monitor the B value. Each cycle begins with both the cell pressure and pore pressure being reduced. Then, after a period of time, the cell pressure is increased, the B value monitored, and the pore pressure increased. This cyclic process will continue until either the Minimum Saturation Ratio is attained or the Maximum Number of Cycles has been reached. If the specified B value is attained, the test proceeds immediately to the next phase. If the Maximum Number of Cycles is reached before the specified B value is attained, the system waits a period of time before proceeding to the next phase.

**T1:** is the duration for which the applied cell pressure is maintained before the pore pressure is increased. During this time the B value (the Minimum Saturation Ratio) is continuously updated and checked against the target value (typically 0.95). This duration depends on the soil and the initial degree of saturation. Common T1 values are from 2 to 30 minutes, the latter for heavily consolidated clays with low permeability.

**T2:** is the duration for which the incremented pore pressure is maintained before the cell pressure increased. During this time, there is no checking of the B value. This duration
depends on the soil and the initial degree of saturation. Common T2 values are from 3 to 30 minutes, the latter for heavily consolidated clays with low permeability.

**T3:** is the length of time, at the beginning of each cycle, between the decrease of cell and pore pressure and the increase of cell pressure. This length of time depends on the soil and the initial degree of saturation. Common T3 values are from 10 to 60 minutes. A value of 0 in the T3 text field will cause the system to skip the cycling feature of the saturation phase.

**T4:** comes into play only if the Maximum Number of Cycles has been reached but the minimum B value has not been attained. T4 is the time the program waits after the Maximum Number of Cycles has been reached before beginning the next phase of the cyclic triaxial test. The value for T4 depends on the B value could not be reached. Typically, T4 is set to a long interval such as 3000 minutes.

### 3.10.5.9 Consolidation/B

In this stage consolidates specimen in steps to specified horizontal and vertical stresses after saturation. The Consolidation/B (Figure 3.18) is used to control a consolidation phase after the saturation phase. The $K_0$ Consolidation option has been selected.

![Consolidation/B table page with $K_0$ consolidation of cyclic triaxial software.](image)

The $K$ used in a $K_0$ consolidation is the coefficient of earth pressure and is defined by the Equation (3.1).

$$K_0 = \frac{\sigma_h}{\sigma_v}$$ (3.1)
Where, the numerator and denominator are the horizontal earth pressure and vertical earth pressure respectively. \( K_0 \) is the coefficient of earth pressure at rest (i.e., when there is no lateral movement). For this situation, \( K_0 = 1 - \sin \varphi \) (Jaky's Solution, 1944) where the angle \( \varphi \) is the angle of shear resistance (the effective internal friction angle). \( K_{\text{max}} \) is the maximum value of the coefficient of earth pressure. \( K_{\text{max}} \) is known also as the coefficient of passive earth pressure, \( K_p \) (about 3.0). \( K_{\text{min}} \) is the minimum value of the coefficient of earth pressure. \( K_{\text{min}} \) is known also as the coefficient of active earth pressure \( K_a \) (about 0.3).

The cyclic triaxial program runs a \( K_0 \) consolidation by forcing zero lateral strain (i.e., keeping the area constant) throughout the consolidation phase. This is done by equating the vertical strain to the volumetric strain (Figure 3.19).

![Figure 3.19: K_0 consolidation graph.](image)

The duration of a step depends on several factors, one of which is the time it takes to reach the T100 condition (the condition of 100% primary consolidation). The basis on which cyclic triaxial determines the T100 condition can be either the change in specimen height (Displacement) or the change in overall specimen size (Volume). In either case, cyclic triaxial determines T100 by utilizing a graph for which the horizontal axis is the square root of time (Figure 3.20).

Once the program has constructed the (rather dim) green line, it draws the blue line starting at the same place as the green line and with a slope equal to the green-line slope...
divided by 1.15. Note that the starting point of the two lines is not the same as the starting point of data recording. This is due to the initial compression. The intercept of the blue line and the data curve indicates T90. The red horizontal line is drawn through this interception point. T100 is found by multiplying T90 by 1.5 and the first vertical yellow line indicates the result. The second vertical yellow line indicates the end of T100 offset. The recommended values for the maximum and minimum durations are shown in the following Table 3.8.

![Real-time T100 graph with (blue) labels added.](image)

**Table 3.8: Recommended duration settings for a consolidation phase**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Recommended Max. Duration</th>
<th>Recommended Min. Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy Silt</td>
<td>24 hours</td>
<td>30 min</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>24 hours</td>
<td>60 min</td>
</tr>
<tr>
<td>Plastic Clay</td>
<td>24 hours</td>
<td>120 min</td>
</tr>
</tbody>
</table>

### 3.10.5.10 Cyclic Phase

Subjects sample to cyclic stress. The Cyclic page (Figure 3.21) is used to enter information that will control the cyclic phase of a test.

All cyclic triaxial tests were conducted at a cyclic loading frequency of 1 Hz and before starting the cyclic deviator load the triaxial cell was needed to make quarter inch air cushion using pump. The specimen was then loaded with a sinusoidal deviator stress until liquefied or 10% double amplitude axial strain (± 5% axial strain) occurred. Tests were
run until the pore pressure become equal to the effective confining pressure or axial strain reaches 5% double amplitude or cumulative axial strain of 10% or 10 cycles of loading. If the sample withstood 10 cycles of loading, the sample was immediately subjected to another series of loading. This was repeated till failure was reached.

After completion of cyclic triaxial tests samples were removed and water content was determined. The tests were conducted at different cyclic stress ratios (CSR). Following cyclic testing, the specimen should be carefully removed from the triaxial cell in order that no particles are lost, then dried and weighed for dry unit weight calculations.

![Figure 3.21: Cyclic phase of cyclic triaxial software.](image)

### 3.11 Triaxial Test Stage

Monotonic triaxial testing was performed using the Geocomp Load Trac II/Flow Trac II system. This is a fully automatic apparatus used for performing all types of monotonic triaxial tests. All tests were run at the highest recommended rate of strain of 0.1% min. The shearing was terminated at 25% strain or when the maximum load was reached on the load cell. The procedure of monotonic triaxial test is as like as cyclic triaxial test only the different is in the last phase the monotonic test the actuator gives a static axial stress which is called deviatory stress. All tests were conducted at an axial strain rate 0.05 % per minutes, maximum axial strain 15% (ASTM 4258-09).

#### 3.11.1 Triaxial Properties Sheet

The Properties Sheet is used to enter general information about the test, the physical characteristics of the specimen, and the parameters which are used for the test. The
Properties Sheet is composed of ten Property Pages (shown in Figure 3.22), except shear table the remaining nine property pages are same as described in cyclic triaxial test above.

<table>
<thead>
<tr>
<th>Consolidation A Table</th>
<th>Saturation</th>
<th>Consolidation B Table</th>
<th>Shear Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project</td>
<td>Specimen</td>
<td>Water Content</td>
<td>Read Table</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test Parameters</td>
<td>Initialization</td>
</tr>
</tbody>
</table>

Figure 3.22: Properties sheet page tabs for static triaxial software.

3.11.2 Shear Table Page

Shear opens a window that displays the shear phase test results in tabular form. The Shear Table page (Figure 3.23) is used to enter parameters that control the shear phase of a triaxial test. Triaxial can perform the shear phase by utilizing up to 32 different stress path steps. Each path is defined on a row of the Shear Table. Four examples of the shear phase for the most commonly run triaxial tests are presented in the following Sections.

Figure 3.23: Shear table page of static triaxial software.

3.11.3 CU (or R) Shear Phase for Triaxial Undrained Compression Test

For a typical triaxial test where the specimen is sheared undrained by increasing the vertical stress, the Shear Table might consist of one row of values as shown below in Figure 3.24.

Figure 3.24: CU test shear table of static triaxial software.
The vertical stress can be a very large value such as 10,000 kPa (1400 psi). It should be equal to at least the confining stress plus twice the anticipated undrained shear strength. These parameters will shear the specimen undrained (the sample flow pump valve will be closed) at 0.02% strain/min under strain control until the axial strain equals a maximum value of 15%. The system will then close the cell flow pump valve and stop the test. During the phase, data will be collected at the times specified in the Strain column of the Read Table.

### 3.11.4 CD (or S) Shear Phase for Triaxial Drained Compression Test

For a typical triaxial test where the specimen is sheared drained by increasing the vertical stress, the Shear Table might consist of one row of values as shown in Figure 3.25 below.

![Figure 3.25: CD test shear table of static triaxial software.](image)

The vertical stress can be a very large value such as 10,000 kPa (1400 psi). It should be equal to at least the confining stress plus twice the anticipated drained shear strength. These parameters will shear the specimen drained (the sample flow pump valve will be open) at 0.01% strain/min under strain control until the axial strain equals a maximum value of 15%. The system will then close all valves and stop the test. During the phase, data will be collected at the times specified in the Strain column of the Read Table. This will cause data to be taken in 0.025% increments and result in 600 data points for a maximum strain is 15%.

### 3.11.5 Triaxial Extension Test

For an extension triaxial test where the specimen is sheared undrained by increasing the vertical stress, the Shear Table might consist of one row of values as shown in Figure 3.26 below.
Figure 3.26: CU triaxial extension test shear table of static triaxial software.

For extension triaxial test where the specimen is sheared drained by increasing the vertical stress, the Shear Table might consist of one row of values as shown in Figure 3.27 below.

Figure 3.27: CD triaxial extension test shear table of static triaxial software.
CHAPTER FOUR
RESULTS AND DISCUSSIONS

4.1 General

This chapter deals with the findings of the experimental results and discussions from the tests conducted on the characteristics of untreated/base soils and stabilized samples of the three reclaimed soils. The properties of the soil collected from the three different areas of Bangladesh were determined at the laboratory to identify the soil and to ascertain its different characteristics. The properties of the base soils used in the study are first presented, it include the physical, chemical and mineralogical properties. The effect of lime treatment on the properties of soil is presented next. Lime stabilized soil specimen was prepared and the variables, which are considered, are soil, different additives, lime content, compaction energy/effort, compaction delay (mellowing time) and curing time, stage mixing was studied. These results demonstrate the effect of additives, e.g., cement and lime on the physical and engineering properties of the samples investigated. Chemical analysis of lime was done in the laboratory. To evaluate the cyclic strength (sometimes called the liquefaction potential) of stabilized saturated soils, the load-controlled cyclic triaxial test was performed on the prepared specimens. The results are used for evaluating the ability of a soil to resist the shear stresses induced in a soil mass due to earthquake or other cyclic loading. All the test results and analytical investigations are presented and discussed in the following sections of this chapter.

4.2 Properties of the Base Soils

The properties of the three types of soil are presented in Table 4.1. It is seen from the table that the fineness modulus of the Soil A, B and C are 3.17, 1.66 and 0.96 and the maximum weight retained on the ASTM standard sieves are Sieve No.30 (0.60mm), No.50 (0.30mm) and No.100 (0.150mm). The soil A contains 0.09% fine size particle and soil 'B' contains 3.36% fine size particle. Whereas soil C contains 15.15% fine size particle. The effective size of D_{50} of the Soil A, B and C are 0.98, 0.37 and 0.21. The coefficient of curvature or gradation of the Soil A, B and C are 1.01, 1.07 and 2.44 and coefficient of uniformity are 3.16, 2.87 and 7.75 respectively. Specific gravity of the soil A, B and C have been found to be 2.70, 2.85 and 2.79 respectively.
Table 4.1: Characteristic values of the physical properties of the base soils

<table>
<thead>
<tr>
<th>Properties of the soil</th>
<th>Characteristic Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SOIL A</td>
</tr>
<tr>
<td><strong>Grain size distribution:</strong></td>
<td></td>
</tr>
<tr>
<td>Fineness Modulus (FM)</td>
<td>3.17</td>
</tr>
<tr>
<td>Fine fraction, F_{200}</td>
<td>0.09</td>
</tr>
<tr>
<td>Coarse fraction, R_{200}</td>
<td>99.91</td>
</tr>
<tr>
<td>Coefficient of curvature/gradation, C_c</td>
<td>1.01</td>
</tr>
<tr>
<td>Coefficient of uniformity, C_u</td>
<td>3.16</td>
</tr>
<tr>
<td>Mean Particle size, D_{50} (mm)</td>
<td>0.98</td>
</tr>
<tr>
<td>Color</td>
<td>Brown</td>
</tr>
<tr>
<td>Field moisture content (%)</td>
<td>3.03</td>
</tr>
<tr>
<td>Minimum dry unit weight, (kN/m³)</td>
<td>14.421</td>
</tr>
<tr>
<td>Maximum dry unit weight, (kN/m³)</td>
<td>17.364</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.70</td>
</tr>
<tr>
<td>Density (loosest stage), gm/cc</td>
<td>1.47</td>
</tr>
<tr>
<td>Density (densest stage), gm/cc</td>
<td>1.77</td>
</tr>
<tr>
<td>Dry Unit weight with Relative density=50%</td>
<td>1.6</td>
</tr>
<tr>
<td><strong>Classification:</strong></td>
<td></td>
</tr>
<tr>
<td>USCS (ASTM D 2487)</td>
<td>SP</td>
</tr>
<tr>
<td>AASHTO (M 145)</td>
<td>A-1-a</td>
</tr>
<tr>
<td>MIT and U.S. Bureau</td>
<td>Coarse sand</td>
</tr>
<tr>
<td><strong>Chemical properties:</strong></td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>6.9</td>
</tr>
</tbody>
</table>

According to Unified Soil Classification System (USCS), SOIL A, SOIL B and SOIL C are divided in a major category of coarse-grained soils that are naturally sandy soil. In Unified Soil Classification System (ASTM D 2487), based on the obtained results of particle size and plasticity characteristics, Soil A and Soil B are SP (poorly graded sand) type and Soil C is SW-SM (well graded silty sand) type. According to American Association of State Highway and Transportation Officials, (AASHTO M145) Soil A is A-1-a type, Soil B is A-3 type and Soil C is A-1-b type. According to Massachusetts Institute of Technology (MIT) and U.S. Bureau of soil classification system Soil A, Soil B and Soil C, are coarse, medium and fine sand type respectively.
Sand specimens were viewed under Scanning Electron Microscope (SEM) (Appendix B) to see the shape and size of particles. Figure B.1 shows the SEM image of sands. From these SEM images it is clearly seen that the sand particles are angular and rough. That means sands are same granular material with different particle sizes.

4.2.1 Particle Size Analysis

Two methods are used to find the particle-size distribution of three soils, are sieve analysis (sands) and Hydrometer analysis (>15% passing through No. 200 sieves) in accordance with the standards depicted in ASTM D 421 and 422. The grain size analysis reports are shown in Figure 4.1. The gradation curves for Soil A, Soil B and Soil C were found to be poor graded sands. The purpose of this analysis was to determine the gradations of the material and to classify them. When the results of this experiment were analyzed, it was seen that Soil A, B and C contain percent clay and silt respectively.

Figure 4.2 shows the grain size distribution curves proposed by (Tsuchida, 1970) for the liquefaction susceptible soils and the curves for the Soil A, Soil B and Soil C. It is observed that the grain size curves of all the three sands lie within the boundaries of most liquefiable soils where Soil A also lie on the boundaries of potentially liquefiable soils.

![Figure 4.1: Grain size distribution curves of Soil A, Soil B and Soil C.](image-url)
4.2.2 Determination of $pH$

The $pH$ experiments were conducted by using the method of Eades and Grim (1963) in order to determine the alkalinity of the soils and amount of lime. The $pH$ tests were performed on the samples by mixing lime and 10 percent lime-pozzolona with the natural soils. The results of this analysis are given in Table A.1 (Appendix A). The $pH$ values of Soil A, Soil B and Soil C were found to be 6.9, 5.6 and 6.1 respectively.

4.3 Properties of Stabilizing Materials Used in Soil Stabilization

Additives materials such as lime, Portland cement, fly ash and rice husk ash, bentonite and clay, can be used as stabilizing material for soil stabilization which differs from natural cements. The interaction of soil with cementing agent follows by transfer and exchange of ions, changes in pH and temperature of the environment and precipitation of cement. Additives materials is normally siliceous or aluminous material, which in itself possesses little or no cementitious value but in finely divided form with moisture, it will react chemically with calcium hydroxide (lime) at ordinary temperatures to form compounds possessing cementitious properties (ASTM, 1950).
4.3.1 Physical and Chemical Properties of Lime

The physical and chemical properties of lime used are given in Table 4.2. The physical appearance of the used lime is lime stone, which was grind to make powder form before used in soil stabilization. The used lime is aesthetically white in color. The specific gravity of lime corresponds to 1.35. Chemical composition of lime shows that the pH value of lime corresponds to 12.7 that are below 7 and alkaline in nature.

Table 4.2: Characteristic values of the physical and chemical properties of the lime

<table>
<thead>
<tr>
<th>Properties of the lime</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical appearance</td>
<td>Dry white powder</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>1.35</td>
</tr>
<tr>
<td>Color</td>
<td>White</td>
</tr>
<tr>
<td>pH</td>
<td>12.7</td>
</tr>
</tbody>
</table>

4.3.2 Fly Ash Characterization

The physical and chemical properties of fly ash are presented in Table 4.3. The used fly ash is aesthetically dark gray in color. A hydrometer analysis was performed for fly ash in accordance with the standards depicted in ASTM D 422. The hydrometer analysis reports for fly ash are shown in Figure 4.3. The fly ash specimen is primarily composed of silt sized particles (76%); the remaining fraction is composed of fine sand sized fraction (18%) and clay (6%) sized fraction. The specific gravity of fly ash corresponds to 2.11. Chemical composition of fly ash shows that the pH of fly ash corresponds to 8.6 that are below 7.0 and alkaline in nature.

Table 4.3: Characteristic values of the physical and chemical properties of the fly ash

<table>
<thead>
<tr>
<th>Properties of the fly ash</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.11</td>
</tr>
<tr>
<td>Sand fraction (4.75-0.075mm)</td>
<td>18</td>
</tr>
<tr>
<td>Silt fraction (0.075-0.002mm)</td>
<td>76</td>
</tr>
<tr>
<td>Clay fraction (&lt;0.002mm)</td>
<td>6</td>
</tr>
<tr>
<td>Color</td>
<td>Dark gray</td>
</tr>
<tr>
<td>pH</td>
<td>8.6</td>
</tr>
</tbody>
</table>
4.3.3 Rice Husk Ash Characterization

The physical and chemical properties of rice husk ash are presented in Table 4.4. The rice husk ash is black in color. A hydrometer analysis was performed for rice husk ash in accordance with the standards depicted in ASTM D 422. The hydrometer analysis reports are shown in Figure 4.4. The rice husk ash specimen is primarily composed of silt sized particles (86%); the remaining fraction is composed of fine sand sized fraction (11%) and clay (3%) sized fraction. The specific gravity of rice husk ash corresponds to 2.15.

Table 4.4: Characteristic values of the physical and chemical properties of the rice husk ash

<table>
<thead>
<tr>
<th>Properties of the rice husk ash</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.15</td>
</tr>
<tr>
<td>Particle Size Distribution (%):</td>
<td></td>
</tr>
<tr>
<td>Sand fraction (4.75-0.075mm)</td>
<td>11</td>
</tr>
<tr>
<td>Silt fraction (0.075-0.002mm)</td>
<td>86</td>
</tr>
<tr>
<td>Clay fraction (&lt;0.002mm)</td>
<td>3</td>
</tr>
<tr>
<td>Color</td>
<td>Black</td>
</tr>
</tbody>
</table>
4.3.4 Bentonite Characterization

The Atterberg limits were conducted on the additives bentonite clay. The experiments were conducted in accordance with the ASTM D 4318 standard. The results of this analysis are given in Figure 4.5. The physical properties of bentonite used are given in Table 4.5. The color of bentonite is brown. The specific gravity of bentonite corresponds to 2.15. Chemical composition of bentonite shows that the pH of bentonite corresponds to 5.6 that are above 7.0 and acidic in nature.

Figure 4.4: Grain size distribution curve for rice husk ash.

Figure 4.5: Flow curve for bentonite clay.
Table 4.5: Characteristic values of the physical and chemical properties of the bentonite

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit, LL (%)</td>
<td>144</td>
</tr>
<tr>
<td>Plastic limit, PL (%)</td>
<td>21</td>
</tr>
<tr>
<td>Shrinkage Limit, SL (%)</td>
<td>12</td>
</tr>
<tr>
<td>Plasticity Index, PI (%)</td>
<td>123</td>
</tr>
<tr>
<td>Flow Index (%)</td>
<td>0.4546</td>
</tr>
<tr>
<td>Toughness Index</td>
<td>277.167</td>
</tr>
<tr>
<td>Color</td>
<td>Brown</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.60</td>
</tr>
<tr>
<td>pH</td>
<td>5.6</td>
</tr>
</tbody>
</table>

4.3.5 Clay Characterization

The Atterberg limits were conducted on the red clays. The experiments were conducted in accordance with the ASTM D 4318 standard. The results of this analysis are given in Figure 4.6. The physical properties of red clay used are given in Table 4.6. The specific gravity of clay corresponds to 2.65. The pH tests revealed that the natural pH values of red clays were found to be 7.7 that are near 7.0 and neutral in nature. It was also observed that lime increased the pH value of the clay increases. For clay-lime mixture, pH value was found to be 12.4.

Table 4.6: Characteristic values of the physical and chemical properties of the red clay

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit, LL (%)</td>
<td>53</td>
</tr>
<tr>
<td>Plastic limit, PL (%)</td>
<td>18</td>
</tr>
<tr>
<td>Shrinkage Limit, SL (%)</td>
<td>80</td>
</tr>
<tr>
<td>Plasticity Index, PI (%)</td>
<td>35</td>
</tr>
<tr>
<td>Flow Index (%)</td>
<td>-0.5962</td>
</tr>
<tr>
<td>Toughness Index</td>
<td>58.705</td>
</tr>
<tr>
<td>Plasticity index for 20% lime content</td>
<td>0</td>
</tr>
<tr>
<td>Color</td>
<td>Red</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
</tr>
<tr>
<td>pH (Pure Red Clay)</td>
<td>7.7</td>
</tr>
<tr>
<td>pH (20% lime content)</td>
<td>12.4</td>
</tr>
</tbody>
</table>
In cases where lime was used, the experiments were performed after clay-lime mixtures were rested for one hour allowing the first reactions to occur. Based on the experiments conducted, it was seen that as the amount of lime in the mixture was increased, substantial reductions in PI values were observed. Accordingly, the PI value of the natural clay is 35 without lime, which has reduced to zero when mixed with 20% lime. Hydrometer analysis was performed on clay and the results were shown in Figure 4.7.

Figure 4.6: Flow curve for red clay.

Figure 4.7: Grain size distribution curve for red clay.
4.3.6 Cement Characterization

The physical properties of used Portland cement (Brand Name: Supercrete - Lafarge Surma Cement Ltd.) are given in table 4.7. The color of cement is gray. The specific gravity of clay corresponds to 3.15.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Characteristic Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clinker, %</td>
<td>65-79</td>
</tr>
<tr>
<td>Limestone, %</td>
<td>21-35</td>
</tr>
<tr>
<td>Gypsum, %</td>
<td>0-5</td>
</tr>
<tr>
<td>Color</td>
<td>Gray</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>3.15</td>
</tr>
</tbody>
</table>

4.4 Discussions of Cyclic Triaxial Test Results

Stress-controlled isotropically consolidated undrained cyclic triaxial tests were performed for both remolded untreated and treated sand specimens to identify the effect of liquefaction behavior. These tests have been done on three different categories sand specimens of size 71 mm diameters and 142 mm height at a constant relative density of 50 kPa. All specimens were manually saturated with CO₂ and distilled water under a confining pressure of 20 kPa for 30 minutes and 2 hours respectively. After manual saturation, triaxial machine was started with initialization step. The initialization step continued for 15 minutes with confining pressure of 20 kPa and sample pressure of 10 kPa (effective confining pressure was 10 kPa). After initialization phase, backpressure saturation of each specimen was performed by simultaneously increasing cell and pore pressure until the saturation ratio became 0.95. During the backpressure saturation phase, the effective confining pressure was kept at a constant value of 10 kPa. After completion of backpressure saturation, each specimen was isotropically consolidated to an effective confining pressure of 50 kPa. During both saturation and consolidation, the pressure was increased very slowly at a rate of 0.167kPa/sec. The consolidation phase was maintained for a minimum of 60 minutes and a maximum of 100 minutes. After completion of consolidation, cyclic loading was applied. Several untreated specimens were tested with cyclic stress ratio of 0.15, 0.20, 0.22, 0.25, 0.27, 0.28, 0.30, 0.35 and 0.40 and cycle period of 1 second up to peak-peak axial strain of 10%. In the entire test program,
harmonic loading was applied using sine wave with a constant frequency of 1.0 Hz. During each cyclic loading test axial deformation, cell pressure, cyclic load, deviator stress, pore pressure and sample pore water pressure were monitored using a computer built-in data acquisition system. All data were stored and analyzed using the software provided with the machine. The data recorded during the test program is used to calculate the dynamic properties of the tested soil such as initial liquefaction, cyclic strength ratio. From the tests of untreated specimens, three sets of data were taken. Similarly lime, fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement treated specimens were tested with the same cyclic stress ratios, cycle period and peak-peak axial strain as untreated specimens.

The analysis of test results on the reconstituted untreated and treated sand samples under the cyclic undrained tests have been presented in the following sections. The cyclic strength curves generated by conducting cyclic undrained tests which gives the variation of applied deviator stress, axial strain and excess pore pressure ratio with number of cycles required to cause initiation of liquefaction. The initial liquefaction is defined as the number of cycles required where the excess pore water pressure reached the value just equal to the effective confining pressure and 5% double amplitude axial strain whichever is earlier (Ishihara, 1993). For a constant relative density (50%) the effect of different lime content, pore pressure response and cyclic strength was observed. The rate of generation of excess pore water pressure with respect to cycles of loading was found to initially increase with increase in lime its trend when the specimens were tested at a constant relative density. The cyclic resistance behavior was observed to be just opposite to the pore pressure response. Permeability and static shear strength after failure at cyclic loading decreased.

4.4.1 Effects of Sand Types on Cyclic Strength

The cyclic strength of the soil is specified in terms of magnitude of cyclic stress ratio (CSR). The cyclic triaxial tests were conducted on three different categories sands at different cyclic stress ratios (CSR). The tests were conducted with the cyclic stress ratios equal of 0.25, 0.20 and 0.15 for Soil A specimens, 0.40, 0.30, 0.25, 0.22, and 0.20 for Soil B specimens and 0.35, 0.27, 0.25, and 0.20 for Soil C specimens with relative density 50% were used. Figures 4.8 to 4.10 provide the results (plot of cyclic stress ratio verses number of cycles to cause 10% double amplitude axial strain) obtained in tests performed at the different cyclic stress ratio (CSR). The corresponding cyclic stress
ratio at 15 cycles, an earthquake magnitude of 7.5 (De Alba et al., 1975; Seed and Idriss, 1982; Ishihara, 1993 and Stark and Olson, 1995), of uniform load application is presented in the figures.

From the Figure 4.8 to 4.10, it was seen that the cyclic stress ratio at 15 cycles for Soil A was 0.185, whereas Soil B and Soil C shows a marked increase in strength of 0.32 and 0.35 respectively. The results presented herein indicate that the coarse sand was highly sensitive to liquefaction than fine sand although tested under similar conditions. Sand specimens with higher fines contents tend to have better particle packing, i.e., lower void ratios and for coarser sand specimens have higher void ratios and have lower cyclic stress ratios. The cyclic resistance generally decreases as the grain size of the sand particles increases until a minimum cyclic resistance is reached (Chang and Ko, 1982). As the fine content continues to increase from Soil A to Soil C, so the cyclic resistance increases.

Figure 4.11(a) shows the liquefaction potential curves of Soil A, Soil B and Soil C. The slope of the curve for Soil B and Soil C was found to gentle when compared to that of Soil A, with the latter having higher cyclic resistance. It was seen that the curves of Soil B and Soil C are almost parallel to each other, with the liquefaction resistance increasing as the grain size decreases.

The liquefaction susceptibility of saturated sand samples is evaluated by cyclic undrained triaxial tests that were carried out on reconstituted specimens of three Algerian sands at a density index of 0.5 and an initial confining pressure, $\sigma'_c$ of 100 kPa. For Figure 4.11(b) Chlef sand (specific gravity of 2.65, $D_{50}$ of 0.39mm, $c_u$ and $c_c$ are of 2.72 and 1.18 respectively and maximum and minimum densities are 1.5 and 1.8 gm/cm$^3$), Rass sand ($Gs=2.67$, $D_{50}=0.35mm$, $c_u =1.9and c_c=1.01$ and maximum and minimum densities are 1.5 and 1.8 gm/cm$^3$) and Zemmouri sand ($Gs=2.69$, $D_{50}=0.33mm$, $c_u =1.66and c_c=0.97$ and maximum and minimum densities are 1.4 and 1.7 gm/cm$^3$) of cylindrical samples of 70 mm diameter and 140 mm height was used at a relative density of 50%. The method of dry funnel deposition has been used in which the sample is poured into the mold gently in seven layers, each one is 2 cm thick, using a funnel with a drop height of zero and slightly compacted with a tamper. Saturation by carbon dioxide and de-aerated water and the application of a back pressure of 200 kPa gives the coefficient of Skempton, B, greater or equal to 0.98 in most of the considered tests. The test were conducted at 5% double amplitude and a frequency of 0.50 Hz. From the Figure 4.11(b) it was shown that all the
liquefaction potential curves for all sands were in a similar in decreasing trend of increase in number of cycles with decrease in cyclic stress ratios.

To investigate the effect of deviator stress, induced axial deformation and excess pore water pressure response, effective stress path and stress-strain behavior for Soil A, Soil B and Soil C typical curve obtained at cyclic stress ratio 0.25 are shown in Figure 4.12, 4.13 and 4.14. Other variations were shown in Appendix D. In the Figure 4.12, 4.13 and 4.14 deviator stresses, excess pore pressure and axial strain are each plotted against the number of cycles of loading. Two aspects of soil liquefaction criteria, the accumulated axial strain and pore pressure increase with number of loading cycles are plotted in Figure 4.12(b), 4.12(c), 4.13(b), 4.13(c), 4.14(b) and 4.14(c) respectively, for distinct numbers of loading cycles. As shown in Figure 4.12(a), 4.12(b), 4.13(a), 4.13(b), 4.14(a) and 4.14(b) for Soil A, B and C specimens, tested with a cyclic stress ratio of 0.25, the deviator stress and axial strain remain relatively constant until the excess pore water pressure is below 40 kPa. When the excess pore water pressure reaches around 40kPa, axial strain were beings to increase at an accelerated rate. For the test with larger cyclic stress ratio, axial strains after liquefaction are higher than with the lower cyclic stress ratio.

Figure 4.12(a), 4.13(a) and 4.14(a) shows the variations of deviator stress versus axial deformation during the tests. From Figure 4.12(a), 4.13(a) and 4.14(a) the specimens were loaded to a maximum peak deviator stress of approximately 20 kPa as all tests were performed at an initial confining stress of 50 kPa. It may be seen in these figures that the deviator stress remained altered till the end of the test. Figure 4.12(a), 4.13(a) and 4.14(a) represents the exponential decay of deviator stress with the increasing number of cycles due to deformation of soil structure; while, the increase in pore pressure generation is portrayed in Figure 4.12(c), 4.13(c) and 4.14(c) respectively. The rate of degradation is comparatively more pronounced for specimens with the higher granular material content.

Figure 4.12(b), 4.13(b) and 4.14(b) show that axial strains developments on specimens remain relatively very low at initial cycles of loading but it drastically increased towards the end. This drastic increase in axial strain started corresponding to around 80% excess pore water pressure generation. Axial strains start to increase at an excess pore water pressure around 40kPa. The constant cyclic load applied to the specimen till ±10% axial strain was developed as shown in Figure 4.12(b), 4.13(b) and 4.14(b). When axial load start to increases the specimen has been softened by cyclic loading. At this CSR of 0.25 and low initial effective confining stress of 50 kPa, soil A specimen quickly generates
negative pore pressures and reaches an axial strain of -8% in compression in 8 cycles of loading whereas, Soil B and Soil C generate positive pore pressures and reaches an axial strain of 10% and 9% in extension with 31 and 24 cycles respectively.

Figure 4.12(c), 4.13(c) and 4.14(c) illustrate that for Soil A, B and C; water pore-pressure increased regularly and reached the value of the initial effective confining pressure (50 kPa) after only a few cycles. The variation of pore pressure increases as loading proceeds. However, the trend of pore pressure generation changes at a few cycles prior to liquefaction. This change in pore pressure variation which hereafter is called splitting and is an indication of the dilative behavior. The Soil A, B and C specimens reached splitting at the end of the cycles 03, 17 and 15 respectively. This state corresponds to liquefaction of the sand through cancellation of the effective confining pressure (i.e., zero effective confining pressure). The results from the test shown in Figure 4.12(c), 4.13(c) and 4.14(c) indicate that excess pore pressures gradually accumulate within the sample indicating initial liquefaction i.e., zero effective stress, in about 8, 31 and 24 cycles for Soil A, B and C.

Figure 4.8: Variation of cyclic stress ratio with the number of cycles for Soil A.
Figure 4.9: Variation of cyclic stress ratio with the number of cycles for Soil B.

Figure 4.10: Variation of cyclic stress ratio with the number of cycles for Soil C.
Figure 4.11(a): Liquefaction potential curves of Soil A, Soil B and Soil C.

Figure 4.11(b): Liquefaction potential curves of different sands.
It is to be noted here that the specimen achieved nearly 100% excess pore water pressure at the 8, 31 and 24 cycles of uniform loading in the test. After initial liquefaction was reached, the double amplitude strains increased more rapidly. However, because the sample remained dilative, as indicated by the downward spikes or splitting in the pore pressure data, the samples have limited potential for large deformations after initial liquefaction. The contractive nature of the reconstituted sample suggests that it may be susceptible to flow liquefaction, having the potential for large liquefaction deformations.

From Figure 4.12(c), 4.13(c) and 4.14(c) it has shown that fine grain soils were considered incapable of generating the high pore pressure commonly associated with liquefaction and coarse grained soils were considered too permeable to sustain any generated pore pressure long enough for liquefaction to develop. It can be noticed from the figure that the pore pressure generation rate for the Soil A is a little faster at the beginning compared to Soil B and Soil C. This is due to the fact that the presence of fine content decreases the speed of drainage within the specimen. It is obvious that the coarse sand is much more sensitive to liquefaction with a higher void ratio. The number of cycles needed to reach liquefaction (pore pressure ratio or strain) is low in that case. The number of cycles needed for liquefaction substantially increases in a dense state of material. For coarse sand with cyclic axial stresses (CSR) 0.20 and 0.25 for Soil B the pore water pressure builds up fast causing drastic reduction in the effective stress thereby, inducing liquefaction in reduced cycles (15 and 8 respectively) of uniform load applications, as indicated in Figure 4.14(c) and Figure D.1(c). However, when the Soil B is subjected to higher cyclic axial stresses (CSR = 0.30 and 0.40 for Soil B), the pore water pressure builds up fast causing drastic reduction in the effective stress (Appendix D) thereby, inducing liquefaction in reduced cycles (10 and 13 respectively) of uniform load applications, as indicated in Figure D.3(c) and D.4(c).

It is clear from the above discussion that the pore water pressure builds up steadily and gradually and reaches initially applied confining stress, depending upon the magnitude of cyclic load as well as the density of soil. At higher cyclic stress ratio, the pore water pressure builds up fast and liquefaction is triggered at reduced stress or lower cycles of uniform load applications.
Figures 4.12(d), 4.13(d) and 4.14(d) reflect and enlighten the effective stress path in response to cyclic axial stress of three sands. Figures 4.12(d), 4.13(d) and 4.14(d) show that the effective stress path during the application of constant amplitude of cyclic deviator stress. The stress path will either move up the line (dilative tendency) or down the line (contractive tendency). The extent to which this behavior occurs defines the difference between flow liquefaction (total strength loss) and cyclic mobility (some strength gain). There is a significant tendency for dilation, reducing pore-pressure. For Soil A, the typical shape of double wings, characteristic of the alternation of the increase and decrease of excess pore pressure, appears from the beginning of the tests. For Soil B and C, there is first an accumulation of pore pressure which induces regular shift of the stress path to the left. There is a permanent loss in shear strength as the stress path moves to the left with each additional cycle of constant amplitude cyclic deviator stress. It is evident from Figures 4.12(d), 4.13(d) and 4.14(d) that the Soil A, B and C liquefies in 8, 31 and 24 cycles of deviator stress application, at which the mean effective stress finally reaches zero. The degradation of dissipated energy with increase in the number of loading cycles can be observed from Figures 4.12(d), 4.13(d) and 4.14(d) where it is noted that the area of hysteresis loop is decreasing with increasing number of loading cycles. It has also been observed from Figure 4.12(d), 4.13(d) and 4.14(d), that both mean effective stress and deviator stress reduces due to increase in pore water pressure. The effective stress path of the specimens where it can be seen that the specimen were loses all its strength and stiffness corresponding to nearly 100% excess pore water pressure generation. Similar outputs have been observed at others shear strain level shown in Figure 4.12(e), 4.13(e) and 4.14(e).

Plot of the axial strain versus the applied deviator stress showing the hysteresis stress-strain loops during cyclic loading measured over the duration of the test are shown in Figure 4.12(e), 4.13(e) and 4.14(e). All hysteresis curves normally show larger axial strains in the tension side than in compression side. Positive values indicate compression and negative tension. The fact that the average stress was takes negative values. The shape of the stress-strain relation of Soils A, B and C decays because the stress amplitude decreases. This is due to the fact that the sand stiffness decreases so suddenly that the regulation system cannot impose the prescribed force. The hysteresis loops shown in Figure 4.12(e), 4.13(e) and 4.14(e) reveal that the loops are relatively steep during the
early cycle of loading. So the specimens are behaving in a relatively stiff manner. As
shown in figures, none of the samples have perfect elliptical shaped hysteresis loops. As
the test progresses, the loop flatten as the specimen soften. However, at larger strains near
the end of the tests, the stress-strain loops began to exhibit a banana like shape or inverted
s-shaped for all sands, which is an indication of strain hardening. In the plots in the Figure
4.12(e), 4.13(e) and 4.14(e), the banana shaped hysteresis loops appear more definitively
for all sands. At smaller strain within the cycle the specimen act like a contractive
material, so the slope of the stress-strain curve is relatively flat. The progressive increase
in pore water pressure with strain cycles causes reduction in shear stiffness of the soil,
which causes the hysteresis loop to become flatter, experimentally observed to represent a
liquefaction state. But strain increases within the cycle, the behavior transitions from
contractive to dilative. All of the specimens eventually undergo dilation during shearing,
leading to the development of banana loops. Referring to Figure 4.12(e), 4.13(e) and
4.14(e), dilative behavior is seen for all specimens. The stress path follows typical
behavior; initial tendency for compression is followed by a tendency for dilation. It can
be shown that the dilative behavior, in general, begins at axial strains from 3 to 6% and
continues to the end of the tests.

Upon inspection, some differences can be identified between each of the stress-strain
curves. All soil specimens develop higher strains in extension during the first cycle of
loading. It is also shown in Figures 4.12(e), 4.13(e) and 4.14(e) that as the test progresses
the values of axial strain for soil A become more negative which indicates that the sample
is becoming extended throughout the test. And for Soil B and Soil C axial strain become
positive which indicates that the samples of B and C are becoming compressed
throughout the test. The increase of deviator strain arises predominantly in the side of
extension for soil A and compressed for Soil B and C. As seen before, the rate of increase
of the deviator strain is faster in Soil B and Soil C than in Soil A. The loops are larger
below the x-axis, showing that more energy is being dissipated during the extension
portion of the cycle (Simcock et al., 1983).
Figure 4.12: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of coarse sand (Soil A) specimen at $CSR=0.25$, relative density $(D_r)=50\%$, effective confining pressure $(\sigma'_3)=50\text{kPa}$, $f=1\text{Hz}$ and 10% Double Amplitude $(D.A)$. 
Figure 4.13: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of medium sand (Soil B) specimen at CSR= 0.25, relative density ($D_r$) = 50%, effective confining pressure ($\sigma'_3$) = 50kPa, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure 4.14: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of fine sand (Soil C) specimen at CSR= 0.25, relative density =50%, effective confining pressure ($\sigma'_{3}$) =50kPa, $f$ = 1Hz and 10% Double Amplitude (D.A).
4.4.2 Effect of Lime Contents on Cyclic Strength

The test samples were prepared with the Soil B (medium sand) and it was stabilized with 2%, 5% and 10% lime and at optimum moisture content of soil-lime mix. The cyclic shear strength of the lime stabilized sands cured for 7 and 28 days are presented graphically in Figures 4.15 to 4.22. Figures 4.15 to 4.22 provide the results obtained in tests performed at the different cyclic stress ratio (CSR). The cyclic triaxial tests were conducted on Soil B stabilized with 2%, 5% and 10% lime content at different cyclic stress ratios (CSR). The tests were conducted with the cyclic stress ratios equal of 0.25, 0.30 and 0.35 for 2% lime content, 0.30, 0.35, 0.40, and 0.45 for 5% lime content and 0.40, 0.50 and 0.55 for 10% lime content after 7 days and 0.30, 0.40 and 0.50 for 2% lime content, 0.70, 0.75 and 0.80 for 5% lime content and 0.75, 0.80 and 0.90 for 10% lime content after 28 days. Figures 4.15 to 4.22 provide the results (plot of cyclic stress ratio verses number of cycles to cause 10% double amplitude axial strain) obtained in tests performed at the different cyclic stress ratio (CSR). The corresponding cyclic stress ratio at 15 cycles of uniform load application is presented in the figures.

From the Figures 4.15 to 4.22, it was seen that the cyclic stress ratio at 15 cycles for 2%, 5% and 10% lime stabilized sand after 7 days curing were 0.37, 0.41 and 0.46 respectively and for 28 days curing were 0.53, 0.85 and 0.88 respectively. From the Figure 4.15 to 4.22, it is seen that the cyclic stress ratio of the Soil B stabilized with 5% and 10% lime content is nearly 3 times greater for 28 days curing times, 1.5 times for curing period 7 days and 1.5 times for 2% lime content at curing age 28 days that of the base soil B. It was also seen from the figures that the increase in cyclic strength of Soil B are negligible when it is stabilized with 2% lime at curing age 7 days.

Figure 4.18 and Figure 4.22 show the liquefaction potential curves of Soil B treated with different lime contents at 7 days and 28 days curing. The slope of the curves for both 7 and 28 days was found to gentle, with the latter having higher cyclic resistance. It was seen that the curves of 7 days and 28 days Soil B specimens stabilized with 2, 5 and 10 percent lime content were almost parallel to each other, with the liquefaction resistance increasing as the increase in lime percentages. For 5 and 10 percent lime contents and at 28 days the curves were shown the almost equal magnitudes.
To investigate the effect of deviator stress, induced axial deformation and excess pore water pressure response, effective stress path and stress-strain behavior for 5% and 10% lime content typical curve obtained at cyclic stress ratio 0.40 are shown in Figure 4.18 to 4.22. In the Figures 4.23 (a), 4.23(b), 4.23(c), 4.24(a), 4.24(b), 4.24(c), 4.25(a), 4.25(b), 4.25(c), 4.26(a), 4.26(b) and 4.26(c) deviator stresses, excess pore pressure and axial strain are each plotted against the number of cycles of loading.

Two aspects of soil liquefaction criteria, the accumulated axial strain and pore pressure increase with number of loading cycles are plotted in Figures 4.23(a), 4.23(c), 4.24(a), 4.24(c), 4.25(a), 4.25(c), 4.26(a) and 4.26(c) respectively, for distinct numbers of loading cycles. Figure 4.23(c), 4.24(c), 4.25(c) and 4.26(c) indicates the variation of pore pressure with cyclic loading for the soil specimen (with lime content 5% and 10%) at relative density 50% for cyclic stress ratios of 0.40 and 0.80 after curing times 7 days and 28 days respectively. As shown in Figures 4.23(a) and 4.23(b), 4.24(a) and 4.24(b) for 5% and 10% lime stabilized Soil B specimen, tested with a cyclic stress ratio of 0.40, the deviator stress and axial strain remain relatively constant until the excess pore water pressure is below 40 kPa. When excess pore water pressure reaches around 40kPa, axial strain were beings to increase at an accelerated rate. For tests with larger cyclic stress ratio, axial strains after liquefaction are higher as compared to lower cyclic stress ratio.

Figures 4.23(a), 4.24(a), 4.25(a) and 4.26(a) show the variations of deviator stress versus axial deformation during the tests. From Figures 4.23(a), 4.24(a), 4.25(a) and 4.26(a) the specimens were loaded to a maximum peak deviator stress of approximately 25 kPa as all tests were performed at an initial confining stress of 50 kPa. It may be seen in these figures that the deviator stress remained altered till the end of the test. Figures 4.23(a), 4.24(a), 4.25(a) and 4.26(a) represents the exponential decay of deviator stress with the increasing number of cycles due to deformation of soil structure; while, the increase in pore pressure generation is portrayed in Figures 4.23(a), 4.24(a), 4.25(a) and 4.26(a) respectively. The rate of degradation is comparatively more pronounced for the specimens with the higher lime content.

It was shown in Figures 4.23(b), 4.24(b), 4.25(b) and 4.26(b) that axial strains developments on the specimens remain relatively very low at initial cycles of loading but it drastically increased towards the end. This drastic increase in axial strain started corresponding to around 80% excess pore water pressure generation. Axial strains start to
Figure 4.15: Variation of cyclic stress ratio with the number of cycles for 2% Lime + Soil B at 7 days curing age.

Figure 4.16: Variation of cyclic stress ratio with the number of cycles for 5% Lime + Soil B at 7 days curing age.
Figure 4.17: Variation of cyclic stress ratio with the number of cycles for 10% Lime + Soil B at 7 days curing age.

Figure 4.18: Liquefaction potential curves of 2% Lime + Soil B, 5% Lime + Soil B and 10% Lime + Soil B at 7 days curing age.
Figure 4.19: Variation of cyclic stress ratio with number of cycles for 2% Lime + Soil B at 28 days curing age.

Figure 4.20: Variation of cyclic stress ratio with number of cycles for 5% Lime + Soil B at 28 days curing age.
Figure 4.21: Variation of cyclic stress ratio with number of cycles for 10% Lime + Soil B at 28 days curing age.

Figure 4.22: Liquefaction potential curves of 2% Lime + Soil B, 5% Lime + Soil B and 10% Lime + Soil B at 28 days curing age.
increase at an excess pore water pressure around 40kPa. The constant cyclic load applied to the specimen till ±10% axial strain was developed as shown in Figures 4.23(b), 4.24(b), 4.25(b) and 4.26(b). When axial load start to increases the specimen has been softened by cyclic loading. At this CSR of 0.40 and low initial effective confining stress of 50 kPa, 5% and 10% lime stabilized Soil B specimen quickly generate pore pressures and reached an axial strain of ±5% in 22 and 39 cycles of loading at 7 days curing age whereas 5% and 10% lime stabilized Soil B specimen at 28 days curing age generates pore pressures and reaches an axial strain of +3% to -6% in 22 and 28 cycles of loading.

Figure 4.23(c), 4.24(c), 4.25(c) and 4.26(c) illustrate that for 5% and 10% lime stabilized soil B specimen; water pore-pressure increased regularly and reached the value of the initial effective confining pressure (50 kPa) after only a few cycles. The variation of pore pressure increases as loading proceeds. However, the trend of pore pressure generation changes at a few cycles prior to liquefaction. This change in pore pressure variation which hereafter is called splitting and is an indication of the dilative behavior. The 5% and 10% lime stabilized Soil B specimens after 7 days curing reached splitting at the end of the cycles 12 and 25 respectively and 5% and 10% lime stabilized Soil B specimens after 28 days curing reached splitting at the end of the cycles 4 and 12 respectively. This state corresponds to liquefaction of the sand through cancellation of the effective confining pressure (i.e., zero effective confining pressure). The results from the test shown in Figure 4.23(c), 4.24(c), 4.25(c) and 4.26(c) indicate that excess pore pressures gradually accumulate within the sample indicating initial liquefaction i.e., zero effective stress, in 22 and 39 cycles of loading at 7 days curing age for 5% and 10% lime stabilized Soil B with CSR 0.40 and 22 and 28 cycles of loading at 7 days curing age for 5% and 10% lime stabilized Soil B with CSR 0.80. It is to be noted here that the specimens achieved nearly 100% excess pore water pressure. After initial liquefaction was reached, the DA strains increased more rapidly. However, because the sample remained dilative, as indicated by the downward spikes or splitting in the pore pressure data, the samples have limited potential for large deformations after initial liquefaction. The contractive nature of the reconstituted sample suggests that it may be susceptible to flow liquefaction, having the potential for large liquefaction deformations. From Figure 4.23(c), 4.24(c), 4.25(c) and 4.26(c) it has shown from the figure that the pore pressure generation rate for the 5% and 10% lime stabilized Soil B specimens after 28 days curing is a little faster at
the beginning compared to 5% and 10% lime stabilized Soil B specimens after 7 days curing. This is due to the fact that the lower cyclic stress ratio decreases the speed of drainage within the specimens. It is obvious that the increase in lime content and curing period is much more resistance to liquefaction. The number of cycles needed for liquefaction substantially increases in lime content and curing period. It is clear that the pore water pressure builds up steadily and gradually and reaches initially applied confining stress, depending upon the magnitude of cyclic load as well as the lime content and curing period. At higher cyclic stress ratio, the pore water pressure builds up fast and liquefaction is triggered at reduced stress or lower cycles of uniform load applications. It has shown that with lower cyclic stress ratio 5% and 10% lime stabilized Soil B specimens after 7 days curing were considered incapable of generating the high pore pressure commonly associated with liquefaction and with higher cyclic stress ratio 5% and 10% lime stabilized Soil B specimens after 28 days curing were considered too permeable to sustain any generated pore pressure long enough for liquefaction to develop.

Figures 4.23(d), 4.24(d), 4.25(d) and 4.26(d) reflect and enlighten the effective stress path in response to cyclic axial stress of 5% and 10% lime stabilized Soil B specimens after 7 and 28 days curing. The stress path will either move up the line (dilative tendency) or down the line (contractive tendency). The extent to which this behavior occurs defines the difference between flow liquefaction (total strength loss) and cyclic mobility (some strength gain). There is a significant tendency for dilation, reducing pore-pressure. For 5% and 10% lime stabilized Soil B specimens after 7 and 28 days curing, the typical shape of double wings, characteristic of the alternation of the increase and decrease of excess pore pressure, appears from the beginning of the tests. There is first an accumulation of pore pressure which induces regular shift of the stress path to the left. There is a permanent loss in shear strength as the stress path moves to the left with each additional cycle of constant amplitude cyclic deviator stress. It is evident from Figures 4.23(d), 4.24(d), 4.25(d) and 4.26(d) that the 5% and 10% lime stabilized Soil B specimens after 7 and 28 days curing liquefies in respectively 22 and 39 and 22 and 29 cycles of deviator stress application, at which the mean effective stress finally reaches zero. The degradation of dissipated energy with increase in the number of loading cycles can be observed from Figures 4.23(d), 4.24(d), 4.25(d) and 4.26(d), where it is noted that the area of hysteresis loop is decreasing with increasing number of loading cycles. It has been observed from Figure 4.23(d), 4.24(d), 4.25(d) and 4.26(d) that both mean effective
stress and deviator stress reduces due to increase in pore water pressure. In the effective stress path where the specimen loses all its strength and stiffness were corresponding to nearly 100% excess pore water pressure generation. Similar outputs have been observed at others shear strain level shown in Figures 4.23(e), 4.24(e), 4.25(e) and 4.26(e).

Plot of the axial strain versus the applied deviator stress showing the hysteresis stress-strain loops during cyclic loading were shown in Figure 4.23(e), 4.24(e), 4.25(e) and 4.26(e). All hysteresis curves normally show larger axial strains in the tension side (negative) than in compression side (positive). The shape of the stress-strain relation of 5% and 10% lime stabilized Soil B specimens after 7 and 28 days curing decays because the stress amplitude decreases. The shape of the stress-strain curve appears to be more symmetrical as the lime content of the soil increases. The hysteresis loops shown in Figure 4.23(e), 4.24(e), 4.25(e) and 4.26(e) reveal that the loops are relatively steep during the early cycle of loading. So the specimens are behaving in a relatively stiff manner. As shown in figures, none of the samples have perfect elliptical shaped hysteresis loops. As the test progresses, the loop flatten as the specimen soften. However, at larger strains near the end of the tests, the stress-strain loops began to exhibit a banana like shape or inverted s-shaped for all samples, which is an indication of strain hardening.

In the plots in the Figure 4.23(e), 4.24(e), 4.25(e) and 4.26(e), the banana shaped hysteresis loops appear more definitively for all sands. At smaller strain within the cycle the specimen act like a contractive material, so the slope of the stress-strain curve is relatively flat. The progressive increase in pore water pressure with strain cycles causes reduction in shear stiffness of the soil, which causes the hysteresis loop to become flatter, experimentally observed to represent a liquefaction state. All of the specimens eventually undergo dilation during shearing, leading to the development of banana loops. Referring to Figures 4.23(e), 4.24(e), 4.25(e) and 4.26(e), dilative behavior is seen for all specimens. It can be shown that the dilative behavior, in general, begins at axial strains from 3 to 6% and continues to the end of the tests. In Figures 4.23(e), 4.24(e), 4.25(e) and 4.26(e), as the test progresses the values of axial strain for 5% and 10% lime stabilized Soil B specimens after 28 days curing become more negative which indicates that the sample is becoming extended throughout the test with higher cyclic stress ratio. And for soil 5% and 10% lime stabilized Soil B specimens after 7 days curing axial strain become positive which indicates that the samples are becoming compressed throughout the test with higher cyclic stress ratio.
Figure 4.23: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% lime + 95% Soil B specimen for curing times 7 days and at relative density \( (D_r) = 50\% \), effective confining pressure \( (\sigma'_c) = 50\text{kPa} \), cyclic stress ratio \( (CSR = 0.40) \), \( f = 1\text{Hz} \) and 10% Double Amplitude (D.A).
Figure 4.24: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% lime + 90% Soil B specimen for curing times 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR=0.40$), $f = 1$ Hz and 10% Double Amplitude (D.A).
Figure 4.25: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% lime + 95% Soil B specimen for curing times 28 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio (CSR= 0.80), $f = 1$ Hz and 10% Double Amplitude (D.A).
Figure 4.26: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% lime + 90% Soil B specimen for curing times 28 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR=0.80$), $f=1$ Hz and 10% Double Amplitude (D.A.).
4.5 Comparisons of Different Stabilizing Materials Treated Sands

For fixed fly ash-lime, rice husk ash-lime, clay-lime and bentonite content a marked difference in cyclic strength is observed for the three types of sands. The tests were conducted with a constant cyclic stress ratio of 0.40 at a constant relative density of 50% and a constant effective confining pressure of 50 kPa with frequency 1 Hz and 10% Double Amplitude (D.A). The tests were conducted at a curing age of 7 days.

Figures 4.27 show typical varying of number of cycles with different additives stabilized soils at 7 days curing and cyclic stress ratio of 0.40. From the Figure 4.27, the number of cycles for Soil A stabilized with lime, fly ash-lime, rice husk ash-lime, clay-lime and bentonite at a constant cyclic stress ratio of 0.40 for 7 days curing were 0, 6, 9, 16, 15 and 31 respectively and for Soil B the corresponding values were 10, 10, 19, 23, 27 and 39 and that of Soil C were 9, 10, 22, 28, 31 and 40 respectively. It was clear from the Figure 4.23 that for lime stabilized sands the number of cycles required to cause liquefaction were more. The numbers of cycles for purely lime treated sands at same content were greater compare to other pozzolonic additives. The numbers of cycles at cyclic stress ratio of 0.40 for three different sands stabilized with 10% percent of fly ash-lime and rice husk ash-lime were almost same.

At 10% fly ash-lime, rice husk ash-lime and clay-lime mixture, the amount of lime present was 2% and remaining 8% was fly ash or rice husk ash or clay. From Figure 4.15, the required number of cycles for cyclic stress ratio of 0.40 with 2% lime stabilized Soil B for 7 days curing at a constant relative density of 50% and a constant effective confining pressure of 50 kPa with frequency 1 Hz and 10% Double Amplitude were found 10. Adding the pozzolonic materials fly ash, rice husk ash and clay with 2% lime and stabilizing the same Soil B, the numbers of cycles required for cyclic stress ratio of 0.40 were 24, 28 and 19 respectively where the tests were run at the same conditions. From this observation, it was seen that the cyclic stress ratio for the Soil B stabilized with 2% lime content and 8% waste materials is enhanced the cyclic stress ratio nearly 2 to 3 times greater than that of the 2% lime stabilized Soil B for curing period 7 days. From this investigation it has concluded that fly ash and rice husk ash of acceptable chemical and physical composition can be used as a partial replacement of lime to make a better environment friendly and cost effective solution. It has also concluded that the fly ash and rice husk ash were most beneficial in soil stabilization for liquefaction resistance.
The numbers of cycles for bentonite treated sands are almost same to base sands. Which conclude that bentonite stabilized sands has no significant resistance to liquefaction. The numbers of cycles at cyclic stress ratio of 0.40 for Soil A stabilized with 2 percent cement at 7 days was 330. Soil B and Soil C were not liquefied at this cyclic stress ratio (0.40). From the Appendix D, Figure D.27 and D.28, it was shown that for 2% cement treated Soil B and Soil C, the specimen were not liquefied at all that means the soils were not liquefied any means of failure like generation of excess pore pressure development, exponential decay of deviator stress, increase in axial strain at a accelerate rate etc.

![Figure 4.27](image)

**Figure 4.27:** Variation of required number of cycles to liquefaction with different stabilizing materials stabilized soils at 7 days curing and cyclic stress ratio of 0.40.

The effect of deviator stress, axial deformation and excess pore water pressure response against the number of cycles of loading, effective stress path and stress-strain behavior for fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement treated Soil A, Soil B and Soil C typical curve were shown in Appendix D at Figure D.12 to D.28 at cyclic stress ratio 0.40. The plots are similar in nature (except for 2% cement stabilized Soil B and Soil C) as described in Sections 4.4.1 and 4.4.2.

From the Figure 4.27 it is shown that the number of cycles for lime, fly ash-lime, rice husk ash-lime and clay-lime stabilized coarse grained soil specimens was lower compare to similar additives stabilized fine grained soils.
4.6 Discussions of Monotonic Triaxial Test Results

To observe post-liquefaction behavior of the treated and untreated specimens, once the specimen liquefied at cyclic loading, the static loading phase was terminated and volume change and pore pressure dissipation were measured immediately. Triaxial shear strength tests were performed because they simulate in situ states of stress and because resultant parameters were difficult to obtain by other means. Stress–strain curves, strength characteristics, and deformation moduli for both cemented and uncemented samples were obtained from monotonic triaxial tests. All the monotonic tests were performed at a strain rate of 0.1%/min. To evaluate the static shear strength of stabilized sands after the failure of the soil specimens at cyclic loading strain-controlled monotonic triaxial tests were carried out on lime, fly ash-lime and rice husk ash-lime treated sand specimen of size 71 mm diameter and 142 mm height at a constant relative density at same isotropic effective confining pressure of 50 kPa. Six drained triaxial compression and six drained triaxial extension and six undrained triaxial compression and six undrain triaxial extension shear tests were conducted on treated and untreated sand at a pre consolidation density of 50 percent. The results of 11 undrained and 10 drained strain controlled triaxial test are reported. Interesting trends were apparent from the engineering plots of triaxial test data. This results (stress-strain relationship, volumetric strain verses axial strain, excess pore pressure response, stress ratio verses axial strain and effective stress path) were shown in Figure 4.28 to Figure 4.34. Although the sample have been disturbed because triaxial test are performed after the failure of the soil specimens at cyclic loading the specimen behaved as a "normally consolidated" sample representative of in situ conditions. The stress-strain response of lime, fly ash-lime and rice husk ash-lime treated specimens for triaxial compression test and triaxial extension test for both drained and undrained conditions are presented in Figures 4.28 to 4.34.

4.6.1 Drain Triaxial Compression Test Results for Lime Treated Sand

Figure 4.28 illustrates stress-strain behavior, volumetric strain changes with axial strain, excess pore pressure response, stress ratio verses axial strain and effective stress path graphs for consolidated drain triaxial compression test for Soil B with 0%, 2%, 5% and 10% lime content after the failure of specimens at cyclic loading of 10% double amplitude at a constant relative density and confining pressure of 50% and 50kPa respectively. From stress-strain relationship, the shear stress tends to increase with shear strain and after failure the shear stress tends to decrease with increasing shear strain.
Figure 4.28: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) excess pore pressure response, (d) stress ratio verses axial strain and (e) effective stress path graphs for consolidated drain triaxial compression test for Soil B specimen with lime contents of 0 to 10 percent after the failure of specimens at cyclic loading with CSR 0.35 (10% D.A) of 7 days at $D_r=50\%$ and $\sigma''_3=50kPa$. 
For increasing the lime content the deviator stress is also increased although all the specimens were failed at cyclic loading. The origin of 10% lime treated sand started from zero, but 0%, 2% and 5% lime treated sands a deviation was observed. It has occurred as the specimens were fully liquefied at cyclic loading for 0%, 2% and 5% lime treated sands. For clean sand to 10% lime stabilized sand the deviator stress tends to increase about 1.5 times (Figure 4.28(a)). The volumetric strain is decreasing with the increase in vertical strain shown in Figure 4.28(b). The excess pore pressure response and stress ratio verses axial strain of lime treated sand with 0 to 10 percent lime content is shown in Figure 4.28(c) and 4.28(d). The excess pore water pressure and stress ratio fluctuate along with the axial strain. The effective stress path of lime treated sand at 50 percent relative density is shown in Figure 4.28(e). The effective stress paths of clean sand and 10 percent lime content sample are seen to deviate upwards from phase transformation point increasing the strength which is termed as stable behavior. The effective failure angle for 0, 2, 5 and 10 percent lime content are 80.5, 81.4, 87.3 and 82.7 degree respectively.

4.6.2 Undrain Triaxial Compression Test Results for Lime Treated Sand

Figure 4.29 shows the monotonic stress-strain response of 0, 2, 5 and 10 percent lime treated sand in isotropically consolidated undrained triaxial compression tests. The stress-strain response of these specimens is presented in Figure 4.29(a). Figure 4.29(a) illustrates types of stress-strain behavior obtained from undrained shear tests on saturated and consolidated samples of lime treated sand at 50% relative density. When it is clean sand, the sample tends to exhibit strain-hardening behavior where the shear stress always goes up with increasing shear strain. For increasing the lime content the deviator stress is also increased although all the specimens are failed at cyclic loading. For clean sand to 10% lime stabilized sand the deviator stress tends to increase. The volumetric strain is zero with the increase in vertical strain as the test is performed in undrained condition shown in Figure 4.29(b). The stress ratio increases with increasing axial strain up to 2 to 7 percent axial strain, after that it becomes steady (Figure 4.29(d)).

The excess pore pressure response of 0 to 10 percent lime treated sand is shown in Figure 4.29(c). For clean sand and 10 percent lime content, the excess pore water pressure decreases at increasing axial strain. The rate of decreasing is higher for 10 percent lime content sand than clean sand specimen. It means that increasing lime content increases the rate of generation of negative excess pore water pressure. Excess pore pressure generation was found to be maximum for 10 percent lime treated sand.
Figure 4.29: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) excess pore pressure response, (d) stress ratio verses axial strain and (e) effective stress path graphs for consolidated undrain triaxial compression test for Soil B specimen with lime contents of 0 to 10 percent after the failure of specimens at cyclic loading with CSR 0.30 (10% D.A) of 7 days at $D_r=50\%$ and $\sigma'_3=50\text{kPa}$. 
The effective stress path of lime treated sand at 50 percent relative density is shown in Figure 4.29(e). The effective stress paths of clean sand and 10 percent lime content sample are seen to deviate upwards from phase transformation point increasing the strength which is termed as stable behavior. The effective failure angle for 0, 2, 5 and 10 percent lime content are 80, 43.4, 76.2 and 79.3 degree.

The corresponding effective stress paths are shown in Figure 4.29(a). In all tests, the applied shear stress first increased and then dropped rapidly to a constant steady state or residual strength value; the pore pressure, shown in Figure 4.29(c), always increased until steady state flow, when it remained constant. Thus, during steady state deformation, the effective normal and shear stresses remained constant.

### 4.6.3 Drain Triaxial Extension Test Results for Lime Treated Sand

Figure 4.30(a) illustrates types of stress-strain behavior obtained from drained extension shear tests on saturated and consolidated samples of lime treated sand at 50% relative density. As shown below, the failure seemed to appear in the form of cracking, which is seen circulated as a sudden dip in axial stresses in Figure 4.30(a). Stress-strain curves for extension triaxial testing showed a continuing decrease in deviator stress after reaching failure of axial strain. The stress-strain curves are not as even as the curves for compression triaxial testing, this is especially noticeable for samples that were subjected to axial extension.

As the sample was extended the pore pressure decreased. As can be seen in Figure 4.30(c), the pore pressure showed the same behavior as in the drained compression shearing test.

The volumetric strain decreases with increasing axial strain up and the stress ratio is zero with the increase in vertical strain as the test is performed in drained condition shown in Figure 4.30(b) and 4.30(d).

Stress paths for extension triaxial tests yielded similar shear stress regardless of whether the specimen is vertically extended or radially compressed. The main difference visible was sudden decreases in $q$ and effective mean stress ($p’$) for tests subjugated to vertical extension, Figure 4.30(e). This effect was suspected to be the result of cracking of the stabilized sand sample. The effective failure angle for 0, 2 and 10 percent lime content are 9.9, -0.3 and 0.7 degree.
Figure 4.30: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) Excess pore pressure response, (d) stress ratio verses axial strain and (e) effective stress path graphs for consolidated drain triaxial extension test for soil B specimen with lime content of 0 to 10 percent after the failure of specimens at cyclic loading with CSR 0.40 (10% D.A) of 7 days at $D_r=50\%$ and $\sigma'_3=50\text{kPa}$. 
4.6.4 Undrain Triaxial Extension Test Results for Lime Treated Sand

The stress-strain response of these specimens is presented in Figure 4.31(a). Figure 4.31(a) illustrates types of stress-strain behavior at cracking, which is seen circulated as a sudden dip in axial stresses. Stress-strain curves for extension triaxial testing showed a continuing decrease in deviator stress after reaching failure of axial strain the deviator stress is increased.

The volumetric strain is zero with the increase in vertical strain as the test is performed in undrained condition shown in Figure 4.31(b). The stress ratio increases with increasing axial strain up to -6 to -12 percent axial strain, after that it becomes steady (Fig. 4.31(d)).

Excess pore pressure response of 0 to 10 percent lime treated sand is shown in Figure 4.31(c). For clean sand and 10 percent lime content, the excess pore water pressure decreases at increasing axial strain. The rate of decreasing is higher for clean sand than 10 percent lime content specimen. It means that increasing lime content decreases the rate of generation of negative pore water pressure on failure of the specimens at cyclic loading.

Stress paths for extension triaxial tests yielded similar shear stress regardless of whether the specimen is vertically extended or radially compressed. The main difference visible is sudden decreases in $q$ and effective mean stress ($p'$) for tests subjugated to vertical extension, Figure 4.31(e). This effect was suspected to be the result of cracking of the stabilized sand sample. The effective failure angle for 0, 2, 5 and 10 percent lime content are -73.9, -84.1, -75 and -82.5 degree.

4.6.5 Drain Triaxial Compression Test Results for Fly Ash-Lime and Rice Husk Ash-Lime Treated Sand

Figure 4.32(a) illustrates types of stress-strain behavior obtained from drained shear tests on saturated and consolidated samples of fly ash-lime and rice husk ash-lime treated sand (Soil B) at 50% relative density. From stress-strain relationship, the shear stress tends to increase with shear strain and after failure the shear stress tends to decrease with increasing shear strain. For fly ash-lime treated specimen the deviator stress is slightly higher than rice husk ash-lime treated specimen. The volumetric strain is decreasing with the increase in vertical strain shown in Figure 4.32(b). The excess pore pressure response of 10% fly ash-lime and 10% rice husk ash-lime treated sand specimen is shown in Figure 4.32(c). The excess pore pressure and stress ratio Figure 4.32(d) fluctuate along with the axial strain.
Figure 4.31: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) excess pore pressure response, (d) Stress ratio verses axial strain and (e) effective stress path graphs for consolidated undrain triaxial extension test for Soil B specimen with lime content of 0 to 10 percent after the failure of specimens of 28 days at cyclic loading with CSR 0.50 (10% D.A) of 28 days at $D_r=50\%$ and $\sigma'_3=50\text{kPa}$. 
Figure 4.32: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) excess pore pressure response, (d) stress ratio verses axial strain and (e) effective stress path graphs for consolidated drain triaxial compression test for Soil B with 10% fly ash-lime and 10% rice husk ash-lime after the failure of specimens at cyclic loading with CSR 0.40 (10% D.A) of 7 days at $D_r=50\%$ and $\sigma'_3=50$ kPa.
The effective stress path of 10% fly ash-lime and 10% rice husk ash-lime treated sand specimen at 50 percent relative density is shown in Figure 4.32(e). The effective stress paths of fly ash-lime and rice husk ash-lime treated samples are seen to deviate upwards from phase transformation point increasing the strength which is termed as stable behavior. The effective failure angles for fly ash-lime and rice husk ash-lime treated samples are 83.1 and 82.4 degree.

4.6.6 Undrain Triaxial Compression Test Results for Fly Ash-Lime and Rice Husk Ash-Lime Treated Sand

The stress-strain response of Soil A specimens is presented in Figure 4.33(a). Figure 4.33(a) illustrates types of stress-strain behavior obtained from undrained shear tests on saturated and consolidated samples of lime treated sand at 50% relative density. From stress-strain relationship, the shear stress tends to increase with shear strain and after failure the shear stress tends to decrease with increasing shear strain. For fly ash-lime treated specimen the deviator stress is lower than rice husk ash-lime treated specimen.

The volumetric strain is zero with the increase in vertical strain as the test is performed in undrained condition shown in Figure 4.33(b). The stress ratio increases with increasing axial strain up to 0 to 4 percent axial strain, after that it becomes steady (Figure 4.33(d)). The excess pore pressure response of 10% fly ash-lime and 10% rice husk ash-lime treated sand specimen is shown in Figure 4.33(c). For both 10% fly ash-lime and 10% rice husk ash-lime treated samples, the excess pore water pressure decreases at increasing axial strain. The rate of decreasing is higher for rice husk ash-lime treated sample than fly ash-lime treated sand specimen.

The effective stress path of fly 10% ash-lime and 10% rice husk ash-lime treated sand at 50 percent relative density is shown in Figure 4.33(e). The effective stress paths of fly ash-lime and rice husk ash-lime treated sand sample are seen to deviate upwards from phase transformation point increasing the strength which is termed as stable behavior. The effective failure angle for fly ash-lime and rice husk ash-lime treated sand samples are 82.4 and 83.1 degree.
Figure 4.33: (a) Stress-strain relationship, (b) volumetric strain verses axial strain, (c) Excess pore pressure response, (d) stress ratio verses axial strain and (e) effective stress path graphs for consolidated undrain triaxial compression test for Soil A with 10% fly ash-lime and 10% rice husk ash-lime after the failure of specimens at cyclic loading with CSR 0.40 (10% D.A) of 7 days at Dr=50% and σ’3=50kPa.
### 4.6.7 Drain Triaxial Extension Test Results for Fly Ash-Lime and Rice Husk-Lime Treated Sand

Stress-strain relationship, volumetric strain verses axial strain, excess pore pressure response with axial strain, stress ratio verses axial strain and effective stress path graphs for consolidated drain triaxial extension test for Soil C with 10% fly ash-lime and 10% rice husk ash-lime after the failure of specimens at cyclic loading with CSR 0.40 (10% D.A) of 7 days at a density of 50% and effective confining pressure of 50kPa were presented in Figure 4.34.

The stress-strain response of Soil C specimens is presented in Figure 4.34(a). Figure 4.34(a) illustrates types of stress-strain behavior at cracking, which is seen circulated as a sudden dip in axial stresses. Stress-strain curves for extension triaxial testing showed a continuing decrease in deviator stress after reaching failure of axial strain. As the sample was extended the pore pressure decreased. As can be seen in Figure 4.34(c), the pore pressure showed the same behavior as in the drained compression shearing test. The volumetric strain decreases with increasing axial strain up and the stress ratio is zero with the increase in vertical strain as the test is performed in drained condition shown in Figure 4.34(b) and 4.34(d) respectively.

The effective stress path of 10% fly ash-lime and 10% rice husk ash-lime treated sand at 50 percent relative density is shown in Figure 4.34(e). Stress paths for extension triaxial tests yielded similar shear stress regardless of whether the specimen is vertically extended or radially compressed. The main difference visible was sudden decreases in $q$ and effective mean stress ($p'$) for tests subjugated to vertical extension, Figure 4.34(e). The effective stress paths of fly ash-lime and rice husk ash-lime treated sand sample are seen to deviate upwards from phase transformation point increasing the strength which is termed as stable behavior. This effect was suspected to be the result of cracking of the stabilized sand sample. The effective failure angle for fly ash-lime and rice husk ash-lime treated sand samples are -56.1 and 45.7 degree.
Figure 4.34: (a) Stress-strain relationship, (b) volumetric strain versus axial strain, (c) Excess pore pressure response, (d) stress ratio versus axial strain and (e) effective stress path graphs for consolidated drain triaxial extension test for Soil C with 10% fly ash-lime and 10% rice husk ash-lime after the failure of specimens at cyclic loading with CSR 0.40 (10% D.A) of 7 days at $D_1=50\%$ and $\sigma'_3=50\text{kPa}$. 
4.7 Variation of Compaction Efforts

In this research, all the treated and untreated sand samples were prepared by compacting five layers of moist samples into the split mold of 71 mm inner diameter and 142 mm height (Figure 3.1) at a constant 10% moisture content with 01 kg weighted hammer at a free fall height of 6 inch. The amounts of compactive energy used for a constant relative density forever layers are shown in Table E.1 and Table E.2 (Appendix E). To compare the variation of compaction effort, different additives stabilized sands versus total number of blows was plotted in a plain graph shown in Figure 4.35 and 4.36.

Figure 4.35 shows the variation of compaction effort of lime fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement treated sands with constant additives content (10%). Figure 4.35 shows that the required number of blows for Soil A was 45 and for Soil B and Soil C were 90 and 91 respectively to obtain a constant density of 50%. That is the required number of blows for Soil A was less than (about half) that of Soil B and Soil C. So, the compaction effort was greater for finer sand particles than for coarser sand particles at a constant density. The compaction effort is depended on grain size distribution, size and shape of soil grains, specific gravity of soil solids etc.

For 10% fly ash-lime and 10% rice husk ash-lime treated sands the number of blows for Soil A was around 25 and for Soil B and Soil C this values become near 75, which has about three times greater. For 10% lime, 10% clay-lime and 10% bentonite treated sand samples the number of blows was slightly higher for Soil B and Soil C than for Soil A. For 2% cement treated sands this values increases in the same order according to the Soil A, Soil B and Soil C were 38, 78 and 74 respectively. This far reaching result makes fine and medium sand qualitatively different from coarse sand.

In Figure 4.35 it was also shown that the number of blows i.e., the compaction effort for normal Soil A, Soil B and Soil C was higher compare to different additives stabilized sand specimens. The compaction energy decreases with the addition of additive content. For Soil A the number of blow was slightly deferrable for different additives stabilized sand specimens. But for Soil B and Soil C this variation was remarkable for 10% lime, 10% clay-lime and 10% bentonite treated sand samples than for normal sand. And for 10% fly ash-lime, 10% rice husk ash-lime and 2% cement treated sand specimens these values were nearly equal.
Figure 4.35: Variation of compaction effort for sands stabilized with different stabilizing materials.

Figure 4.36: Variation of compaction effort of sands stabilized with different lime contents.

Figure 4.36 shows the variation of compaction effort of 0, 2, 5 and 10 percent lime content sand samples. In Figure 4.36 the required number of blows for Soil A was less than that of Soil B and Soil C. The number of blows for Soil B and Soil C was slightly variable. It was also shown in Figure 4.36 that the number of blows i.e., the compaction effort for normal Soil A, B and C was higher compare to 2, 5 and 10 percent lime stabilized sand specimens. For Soil A, Soil B and Soil C the number of blows decreases with the increase of lime contents. For fine sand this variation was larger compare to the coarser sand. That is the compaction effort is greater for finer sand particles than for coarser sand particles at a constant density with different lime contents. So for increase in additives content the required compaction energy is decreased.
4.8 Bonding Behavior of Untreated Sands

The excessive large deformation for the granular sandy soil occurs due to sliding, rolling or rearrangements of the particles within a soil mass. In order to establish a relationship between the smaller and coarser sand particles it has investigated the particles in Scanning Electro-microscope analysis (Figure 4.37 and 4.38). From Figure 4.37 it was shown that Soil A, Soil B and Soil C are granular materials that consist of individual particles. From energy dispersive spectrum (EDS) analysis in Figure B.4 it was shown that the soils contain mainly O (>50%) and Si (around 30%) and a small amount of Al, Mg, Fe, and Ca and so on.

When soils were wetted at 10 percent moisture they behave quite differently to dry or saturated sand. Under normal conditions, water molecules were strongly adsorbed on soil particle surface. When particles are fine of FM 1.66 to 0.96, magnitude of soil-water interaction forces are larger than coarser particles of FM 3.17. A bonding between the particles occurs in presence of water which helps to generate various chemical bonding among the particles. The resistances to liquefaction of soils are the result of friction and interlocking of particles, and cementation or bonding at particle contacts. The controlling factor was degree of compaction. In this research a constant relative density of 50 percent was used. In a compacted state, granular soils contain a granular skeleton (Figure 4.37) giving them good volume stability and friction properties. Cohesionless soils possess negligible strength without effective confinement. The overall strength of these untreated sands depends on particle interaction and this particle interaction forces depends on the effective confining stress. In most cases, natural cementation occurs when the soil is under confinement. In this study 50 kPa confining pressure was applied. When loading applied at cyclic stages, Soil A expands its volume more rapidly and the shear strength and density of particles were decreased.

When two particles were brought into contact there exists forces namely body or gravitational force, electrostatic force and Van der Waals force. The property of fine grained soils Soil B and Soil C of FM 0.96 to 1.66 were governed by electrical charges and the Van der Walls force and for coarse grained Soil A of FM 3.17 and above the body force has influencial effect as the mass of the bodies are predominant. For these above reasons, compaction effort is higher for smaller sized sands than for larger sized sands.
From this SEM experiment results it is clear that Soil A has increasing surface roughness (Figure 4.38(a)) that decreases the real area of contact between solid surfaces, thus reduced the strength. For Soil B and Soil C have decrease in surface roughness (Figure 4.38(b)) that increased adhesion/cohesion leads to increase in liquefaction strength. In case of Soil B and Soil C particles the specific surface area (area per unit mass) was significantly higher as compared to Soil A. As the electrical charges and hence the Van der Walls force are directly proportional to surface charge, the property of fine grained soil was governed by cohesion of the soil.

4.9 Bonding Behavior of Treated Sands

The strength of an artificially cemented soil mass such as cement or lime, fly ash-lime, rice husk ash-lime, clay-lime and bentonite was highly dependent on the intimacy of grain to grain contact. The controlling factor was degree of compaction. In this research a 50%
relative density was used. Lime, cement and fly ash-lime, rice husk ash-lime, clay-lime and bentonite depend on their chemical reactions with the soil elements in the presence of water. From EDS analysis in Figure B.4 to B.12 it was shown that, the main component of lime was Ca (Figure B.5); cement was Ca, Si, Al and Fe (Figure B.12); fly ash was C, Al, Si, Fe and Ti (Figure B.7); rice husk ash was C and Si (Figure B.9); clay was C, Si and Al (Figure B.10) and bentonite was Si (Figure B.11).

When lime was mixed with water and Soil A, B and C, the result was initially a stiff paste, which has used in soil stabilization. It gradually hardened, cementing the sand particles together. At room temperature, the reaction of lime with sand grain was speeded by mixing with water. When lime was mixed with water, it forms calcium hydroxide. Even with the increased reaction speed, lime sand mixture requires long time for complete reaction to occur as the liquefaction strength increases at 28 days compare to 7 days with 2, 5 and 10 percent lime content. The mechanism of soil-lime treatment involves cation exchange, flocculation, carbonation and pozzolanic reaction. After the application of lime to the soil, the pH of soil lime component was increased to 12.9 indicated in Table A.1. This high pH environment then causes a pozzolanic reaction between the free Ca$^{2+}$ cations and the dissolved silica and alumina.

From Figure B.6 of EDS analysis of lime treated Soil B of FM 1.66 show that after reaction take place the lime treated Soil B compound contain mainly Ca. From these investigation it may be conclude that lime particles coat the soil particles and bind the soil particles together, resulting in a decrease in soil porosity and permeability (Table C.1).

Figure 4.39 and Figure 4.40 shows the SEM images of lime stabilized Soil A and Soil B. Lime bounded the soil particles in both Soils but Soil A have porous spaces on the surface where soil B has smooth polishing surfaces. The bonding of higher FM soil was relatively weak than for lower FM based soils.
The interaction between Soils and lime, fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement was investigated by scanning electron microscope. The results of scanning electron microscope on these stabilizing material treated soil are presented in Figure 4.39 to Figure 4.45. No differences could be ascertained by the scanning electron microscope images on the nature of the reaction products formed on the surface on lime, fly ash-lime, rice husk ash-lime and cement treated Soil B. Clay-lime and bentonite treated Soil B have not a smooth surface. It seems that a non-crystalline reaction product was formed in clay and bentonite stabilization.

Lime were used for cementing materials for soil stabilization. Fly ash, rice husk ash and clay may act as a catalyst, as an inhibitor, or as a component of the pozzolanic reaction,
and also may go into side reactions other than the pozzolanic reaction and produce either cementing or inert materials. Lime reacts with fly ash, rice husk ash and clay to form a bonded calcium-silicate hydrate (CSH) gel \([\text{Ca} (\text{SiO}_3)]\) with soil particles. This reaction is also water insoluble. The gel proceeds immediately to coat and bind the soil particles and to block off the soil voids. Calcium silicate hydrate gel has a strongly cementing effect that binds the soil together and increases high strength of the compounds.

The main components of fly ash and rice husk ash were C, Si and Al (Figure B.7 and B.9). After the stabilization reaction the stabilized material also show the height component is carbon (Figure B.8). It may conclude that siliceous and aluminous fly ash and rice husk ash material in the presence of lime and water would become cementitious.

The lime, fly ash-lime, rice husk ash-lime, clay-lime and sandy soils mix improve the permeability coefficient and pH values. The pH of fly ash-lime, rice husk ash-lime and clay-lime stabilized sand were 13.5, 13.3 and 13.0 respectively shown in Table A.1 i.e., the treated sand was highly alkaline. It may indicate that the free bases were entering into a chemical reaction that bond the sand grains. Also the pH value of fly ash-lime, rice husk ash-lime and clay-lime stabilized sand were greater than that of lime treated sand. So the pozzolanic reaction of fly ash-lime, rice husk ash-lime and clay-lime stabilized sand would be greater than that of lime treated sand.

The permeability coefficient of fly ash-lime, rice husk ash-lime and clay-lime stabilized Soil B at 7 days were 0.049, 0.047 and 0.053 m/sec respectively. At 10% fly ash-lime, rice husk ash-lime and clay-lime stabilized Soils the lime percentages was less than 2 percent but the permeability coefficient for 10% fly ash-lime, rice husk ash-lime and clay-lime stabilized Soils were about 15 times less than that of 2 percent lime stabilized soils. This may conclude that fly ash, rice husk ash and clay have a greater bonding capability to bind the sand particles.

From Figure 4.45 and Figure 4.40 it was shown that lime stabilized sand is more porous than cement stabilized sand. Therefore, cement has a greater bonding behavior that leads to strength the soil which will not liquefied at all.
Figure 4.41: Scanning Electron Microscope (SEM) images of fly ash-lime treated Soil B.

Figure 4.42: Scanning Electron Microscope (SEM) images of rice husk ash-lime treated Soil B.

Figure 4.43: Scanning Electron Microscope (SEM) images of clay-lime treated Soil B.
Figure 4.44: Scanning Electron Microscope (SEM) images of bentonite treated Soil B.

Figure 4.45: Scanning Electron Microscope (SEM) images of cement treated Soil B.
CHAPTER FIVE
CONCLUSIONS AND RECOMMENDATIONS

5.1 General

A comprehensive test program was undertaken on treated and untreated sands to define the cyclic strength characteristics, often called liquefaction. By performing a series of cyclic triaxial tests and drained and undrained monotonic strain control triaxial compression and extension tests has been done on three different coarseness sand specimens stabilized with different stabilizing materials with constant relative density of 50% and effective confining pressure of 50 kPa. From the present investigation it was observed that the sand samples obtained from different locations and varying coarseness were susceptible to liquefaction. Liquefaction susceptibility of these sand specimens could be reduced greatly by adding different types and amounts of stabilizing materials. The resistance increases with the increase in degree of cementation of the sands. However, adding a small amount of stabilizing materials like cement, lime, fly ash-lime, rice husk ash-lime and clay-lime increases significantly the cyclic resistance of these uncemented sand samples.

5.2 Conclusions

The present study was carried out in order to fulfill five objectives as outlined in Chapter One. The conclusions of the study are objective wise as follows.

Concerning the first objective, the cyclic shear strength of uncemented sands are increases significantly due to the increase in lime percentages. The findings of the investigation are the following:

(i) The untreated reconstituted sands were found to easily liquefy during cyclic loading. The sands of fineness modulus (FM) 3.17 and mean grain sized particles of 0.98 mm were more susceptible to liquefaction. The rate of generation of excess pore water pressure was found to initially increase with this sand.

(ii) The cyclic shear resistance of lime stabilized sand specimens continues to increase with increasing lime percentages and curing periods.

(iii) Axial strains developments on all the specimens remain relatively very low at initial cycles of loading but it drastically increased corresponding to around 80% (around
40kPa) excess pore water pressure generation that leads to initiation of liquefaction. The specimens lose all its strength and stiffness corresponding to nearly 100% excess pore water pressure generation (50 kPa).

(iv) The cyclic resistance behavior was observed to be just opposite to the pore pressure response. At higher cyclic stress ratio, the pore water pressure builds up fast and liquefaction was triggered at reduced stress or lower cycles of uniform load applications.

(v) The cyclic strength and the pore pressure development curves corresponding to lime treated sands were similar in nature to those for untreated sands. The deviator stress altered and at the initiation of liquefaction it continues to decay.

(vi) The number of cycle required to failure decreases with increasing cyclic stress ratio.

Regarding the second objectives, the following conclusions can be drawn-

(i) The number of cycles required to cause liquefaction were found to be almost 1.5 to 3 times greater for 10 percent lime stabilized sands compare to 10 percent fly ash-lime, 10 percent rice husk ash-lime, 10 percent clay-lime and 10 percent bentonite stabilized sands.

(ii) The number of cycles were found 2 to 3 times greater for 2 percent lime plus 8 percent fly ash, 8 percent rice husk ash and 8 percent clay stabilized sands than that of 2 percent pure lime stabilized sands.

(iii) The cyclic resistance ratio for 10 percent bentonite stabilized sands almost same that of untreated sands.

(iv) The soils of FM 0.96 to 1.66 stabilized with 2 percent cement were not liquefied with CSR of 0.40 at 7 days.

With reference to the third objectives, it was observed that there has static shear strength after failure at cyclic loading. In the light of the experimental evidences, the following conclusions can be drawn:

(i) In drained triaxial test the peak shear strength reached a maximum value and goes towards a constant ultimate value for 5 and 10 percent lime treated sands that behaves like dense specimens. For 0 and 2 percent lime treated sands the deviator
stress increases with no distinct peak towards an ultimate value that behaves like loose specimens.

(ii) The peak shear stress was found higher for 10 percent fly ash-lime and rice husk ash-lime treated sands that behaves like dense specimens.

(iii) The effective stress paths of 0, 2, 5 and 10 percent lime content samples were found to increasing the strength which has termed as stable behavior.

**The conclusions of the fourth objective are-**

(i) The compaction effort was found greater for sands with FM 1.66 to 0.96 than for sands of FM 3.17.

(ii) The compaction energy were decreases with 10 percent lime, 10 percent fly ash-lime, 10 percent rice husk ash-lime, 10 percent clay-lime and 10 percent bentonite treated sands than for untreated sands.

**With reference to the fifth objectives** lime, fly ash-lime, rice husk ash-lime, clay-lime and cement mixed with water reacts and forms calcium silicate hydrate (CSH) gel and calcium hydroxide which have strong cementing effect that binds the sand particles together and improves their strength. In the light of the experimental evidences, the summarized following conclusions can be drawn from this study:

(i) Untreated sands with fineness modulus (FM) 0.96 to 1.66 were brought into contact and bonded toghether by electrostatic force and Van der Waals force where for Soils of FM 3.17 the body force has influencial effect.

(ii) For lime treated sands, lime particles coat the soil particles and binding them together.

(iii) The mixture of lime, cement, fly ash-lime, rice husk ash-lime and clay-lime and sand bind the sand grains together that improve the soil pH and reduce soil permeability.

(iv) Cement has a greater bonding behavior that leads to greater shear strength gain of the soil.
5.3 Recommendations for Further Study

The relationship between liquefaction strength for different categories treated and untreated sands and their static shear strength at failure of cyclic loading; the variation of compaction effort for different additives treated sands and the bonding behavior of sand-stabilizing additives materials has been the motivation for the further researches. Based on this research context, a few future recommendations to extend the research can be suggested.

(i) The present study was carried out on samples collected from the three selected rivers in Bangladesh. Similar investigations may be carried out with soils collected from other rivers and also reclaimed sites of any region of Bangladesh and the results may be compared with those obtained in the present investigation.

(ii) In this research work, a moist tamping technique was chosen for sample preparation ensures uniformity among the specimens and the determination of compaction effort; however it cannot be inferred that this is the proper specimen preparation technique to properly model the field behavior. So, different laboratory specimen preparation techniques (such as wet tamping technique, air pluviation method, water pluviation method, rodding dry soil and several vibration techniques) could be used to understand the best models the field performance of sand-additives mixtures.

(iii) This study only considered the constant relative density and constant confining pressure approach. So the effect on the cyclic resistance of different confining pressures and relative densities on treated and untreated sands at different approach may also be investigated.

(iv) In this investigation, lime, fly ash-lime, rice husk ash-lime, clay-lime, bentonite and cement have been used as additives for sand stabilization. Investigations on the liquefaction resistance properties could be carried out by stabilizing the soils studied with other additives and different additives combinations in order to assess the most suitable type of additive for stabilizing these liquefaction susceptible sands.

(v) In this research work, three different sized samples were stabilized with a lime content of 2%, 5% and 10%, and lime-natural pozzolonas of 5% for investigating their liquefaction susceptibility for maximum curing times of 28 days. The scope of the present work could be extended by stabilization of soil samples with different
percentages of cement, lime and cement or lime-natural additives and laboratory experiments will be conducted after long term curing ages to evaluate the optimum additive content for satisfactory liquefaction resistance of the stabilized soil samples.

(vi) Based on the result of this research work, it was shown that coarse sand was recognized as a highly sensitive material for liquefaction susceptibility, so further investigation may be carried out on coarser sands with high lime content for avoiding liquefaction triggering.

(vii) In this research work, monotonic triaxial test were performed on the specimens after the failure at cyclic loading. Laboratory experiments may be carried out on the reconstituted specimens for analyzing the monotonic stress-strain response controls to a large extent the effect of initial shear stress on liquefaction resistance.

(viii) In this investigation, permeability characteristics of the untreated and treated samples have been determined in an indirect measurement. So, detailed investigation can be performed on the permeability characteristics of the untreated and treated samples.

(ix) In this research work, limited investigations have been conducted to determine whether pH variations will affect properties of lime-stabilized soils. Additional studies are therefore necessary to explain the erosion mechanism of lime-stabilized soils due to pH variations.

(x) In this investigation, scanning electron microscope (SEM) analysis was performed in order to establish a general indication of fine and coarser sand particles and their bonding behavior with different additives. This required a more detailed research to identify the geometry of particles, the molecular structure and the minerals present in sand samples, the lime-pozzolona reaction (particularly about products formed), crystalline structure and properties of stabilizing materials used X-Ray diffraction technique (XRD) and Differential Thermal Analysis (DTA) should be done.
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APPENDIX A
Determination of soil pH

Soil pH was determined by using pH meter with glass electrode (Metrohm 691) from extraction of soil: water 1:2.5. In this process, 10 g of soil was mixed with 25 ml of water in a small beaker and it was stirred frequently for 50 minutes. After that the mixture was left for 10 minutes without stirring. Then the pH meter electrode was placed in the suspension and waited until the reading become steady. The pH meter was calibrated by three buffer solutions (pH 4.01, pH 6.87 and pH 9.18).

Table A.1: pH values of different additives treated sand

<table>
<thead>
<tr>
<th>Samples</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lime + soil B</td>
<td>12.9</td>
</tr>
<tr>
<td>Rice husk ash-lime + soil B</td>
<td>13.3</td>
</tr>
<tr>
<td>Fly ash-lime + soil B</td>
<td>13.5</td>
</tr>
<tr>
<td>Clay-lime + soil B</td>
<td>13.0</td>
</tr>
</tbody>
</table>
APPENDIX B
SCANNING ELECTRON MICROSCOPE IMAGES

Scanning electron microscope (SEM) is a non-destructive electron microscope method that produces images of a sample by scanning the surface with a focused beam of electrons in the backscattered mode. The electrons interact with atoms in the sample, producing various signals that contain information about the sample's surface topography (morphological) and compositions of the elemental profiles of individual particles. The electron beam is scanned in a raster scan pattern, and the beam's position is combined with the detected signal to produce an image. The analysis is performed in an automated fashion where some amount of the analysis is controlled by manually. The instrument is capable of operating automated the energy dispersive spectrum (EDS) system software and collect a spectrum from each detected particle. The results are described below.

Figure B.1: SEM images of soil A particles.
Figure B.2: SEM images of soil B particles.

Figure B.3: SEM images of soil C particles.
Figure B.4: SEM image of soil B with EDS analysis of corresponding areas.
Figure B.5: SEM image of lime with EDS analysis of corresponding areas.
Figure B.6: SEM image of lime treated soil B with EDS analysis of corresponding areas.
Figure B.7: SEM image of fly ash with EDS analysis of corresponding areas.
Figure B.8: SEM image of fly ash-lime treated soil B with EDS analysis of corresponding areas.
Figure B.9: SEM image of rice husk ash with EDS analysis of corresponding areas.
Figure B.10: SEM image of clay with EDS analysis of corresponding areas.
Figure B.11: SEM image of bentonite with EDS analysis of corresponding areas.
Figure B.12: SEM image of cement with EDS analysis of corresponding areas.
APPENDIX C
INDIRECT DETERMINATION OF PERMEABILITY COEFFICIENT

Soil permeability have determined by using indirect measurement of variable (falling) head test. In this method, a 71 mm diameter and 142 mm height specimen of at 50% relative density was used. The specimen was wetted at 10% moisture content and compacted in four equal layers by a tamping rod to obtain specified 50% relative density. Water was fallen from a 75 mm diameter water supply bottle at a falling height of 62 mm. The top of water level in the bottle was not constant throughout the test. After that the water was passing through the sample and collected at a constant head chamber of 100 ml volume. The water was passing through the specimen and collected by a 2 mm inner diameter stand pipe. The time was recorded and coefficients of permeability (k) have been estimated by using the modified Darcy’s law on untreated and treated specimens. The results are listed in the following Table C.1.

Table C.1: Hydraulic conductivity for seeping water through the specimens

<table>
<thead>
<tr>
<th>Samples</th>
<th>Time (sec)</th>
<th>Hydraulic conductivity, k (mm/sec)</th>
<th>Curing Period (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil A</td>
<td>162</td>
<td>0.000305</td>
<td>0</td>
</tr>
<tr>
<td>Soil B</td>
<td>477</td>
<td>0.000104</td>
<td>0</td>
</tr>
<tr>
<td>Soil C</td>
<td>479</td>
<td>0.000103</td>
<td>0</td>
</tr>
<tr>
<td>2% lime + soil B</td>
<td>487</td>
<td>0.000101</td>
<td>07</td>
</tr>
<tr>
<td>2% lime + soil B</td>
<td>489</td>
<td>0.000101</td>
<td>28</td>
</tr>
<tr>
<td>5% lime + soil B</td>
<td>590</td>
<td>8.37E-05</td>
<td>07</td>
</tr>
<tr>
<td>5% lime + soil B</td>
<td>620</td>
<td>7.97E-05</td>
<td>28</td>
</tr>
<tr>
<td>10% lime + soil B</td>
<td>1512</td>
<td>3.27E-05</td>
<td>07</td>
</tr>
<tr>
<td>10% lime + soil B</td>
<td>1680</td>
<td>2.94E-05</td>
<td>28</td>
</tr>
<tr>
<td>2% cement + soil B</td>
<td>885</td>
<td>5.58E-05</td>
<td>07</td>
</tr>
<tr>
<td>10% clay-lime + soil B</td>
<td>928</td>
<td>5.32E-05</td>
<td>07</td>
</tr>
<tr>
<td>10% Bentonite + soil B</td>
<td>472</td>
<td>0.000105</td>
<td>07</td>
</tr>
<tr>
<td>10% fly ash-lime + soil B</td>
<td>997</td>
<td>4.95E-05</td>
<td>07</td>
</tr>
<tr>
<td>10% rice husk ash-lime + soil B</td>
<td>1041</td>
<td>4.74E-05</td>
<td>07</td>
</tr>
</tbody>
</table>
APPENDIX D
CYCLIC TRIAXIAL TEST RESULTS

Figure D.1: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil A specimen at relative density \((D_r) = 50\%\), effective confining pressure \((\sigma_3') = 50\text{kPa}\), cyclic stress ratio \((CSR) = 0.20\), \(f = 1\text{Hz}\) and 10% Double Amplitude (D.A).
Figure D.2: (a) Deviator stress versus number of cycles, (b) axial strain versus number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress versus axial strain of Soil B specimen at relative density ($D_r$) = 50%, effective confining pressure ($\sigma'_3$) = 50kPa, cyclic stress ratio (CSR) = 0.22, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.3: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil B specimen at relative density ($D_r$) =50%, effective confining pressure ($\sigma'_3$) =50kPa, cyclic stress ratio ($CSR$)= 0.30, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.4: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil B specimen at relative density ($D_r$) = 50%, effective confining pressure ($\sigma'_3$) = 50 kPa, cyclic stress ratio ($CSR$) = 0.40, $f$ = 1 Hz and 10% Double Amplitude (D.A).
Figure D.5: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil C specimen at relative density ($D_r$) =50%, effective confining pressure ($\sigma'_3$) =50kPa, cyclic stress ratio ($CSR$) = 0.20, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.6: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil C specimen at relative density \( (D_r) = 50\% \), effective confining pressure \( (\sigma'_3) = 50 \text{kPa} \), cyclic stress ratio \( (CSR) = 0.27 \), \( f = 1 \text{Hz} \) and 10% Double Amplitude (D.A).
Figure D.7: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil B specimen at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=1000kPa, cyclic stress ratio ($CSR$)= 0.25, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.8: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of Soil A specimen at relative density \(D_r = 50\%\), effective confining pressure \(\sigma'_{3} = 100\text{kPa}\), cyclic stress ratio \(CSR = 0.25\), \(f = 1\text{Hz}\) and 10% Double Amplitude (D.A).
Figure D.9: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% Lime + 95% Soil B specimen for curing period 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.30, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.10: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% Lime + 95% Soil B specimen for curing period 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.35, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.11: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% lime + 95% Soil B specimen for curing period 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.45, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.12: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% lime + 90% Soil B specimen for curing period 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)=0.50, $f =$ 1Hz and 10% Double Amplitude (D.A).
Figure D.13: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% Lime + 95% Soil B specimen for curing period 28 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'$)=50kPa, cyclic stress ratio ($CSR$)= 0.70, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.14: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 5% lime + 95% Soil B specimen for curing period 28 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'$)=50kPa, cyclic stress ratio ($CSR$)= 0.75, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.15: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% lime + 90% Soil B specimen for curing period 28 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.90, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.16: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% bentonite + 90% Soil A specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.40, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.17: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% bentonite + 90% Soil B specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.40, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.18: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% bentonite + 90% Soil C specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio (CSR) = 0.40, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.19: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% clay-lime + 90% Soil A specimen for curing period of 7 days at relative density \((D_r)=50\%\), effective confining pressure \((\sigma'_3)=50kPa\), cyclic stress ratio \((CSR)= 0.40\), \(f = 1Hz\) and 10% Double Amplitude (D.A).
Figure D.20: (a) Deviator stress versus number of cycles, (b) axial strain versus number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress versus axial strain of 10\% clay-lime + 90\% Soil C specimen for curing period of 7 days at relative density $D_r=50\%$, effective confining pressure $\sigma'_3=50kPa$, cyclic stress ratio $CSR=0.40$, $f=1Hz$ and 10\% Double Amplitude (D.A).
Figure D.21: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% fly ash-lime + 90% Soil A specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.40, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.22: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% fly ash-lime + 90% Soil B specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_{3}$)=50kPa, cyclic stress ratio ($CSR$)= 0.40, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.23: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% fly ash-lime + 90% Soil C specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio ($CSR$)= 0.40, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.24: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% Rice Husk Ash-lime + 90% Soil A specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio (CSR)= 0.40, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.25: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 10% Rice Husk Ash-lime + 90% Soil B specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'_3$)=50kPa, cyclic stress ratio (CSR)= 0.40, $f = 1$Hz and 10% Double Amplitude (D.A).
Figure D.26: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 2% cement + 98% Soil A specimen for curing period of 7 days at relative density ($D_r$)=50\%, effective confining pressure ($\sigma'$)=50kPa, cyclic stress ratio (CSR)=0.40, $f$= 1Hz and 10\% Double Amplitude (D.A).
Figure D.27: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 2% cement + 98% Soil B specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'$)=50kPa, cyclic stress ratio (CSR)=0.40, $f$ = 1Hz and 10% Double Amplitude (D.A).
Figure D.28: (a) Deviator stress verses number of cycles, (b) axial strain verses number of cycles, (c) excess pore water pressure response, (d) effective stress path and (e) shear stress verses axial strain of 2% cement + 98% Soil C specimen for curing period of 7 days at relative density ($D_r$)=50%, effective confining pressure ($\sigma'$)=50kPa, cyclic stress ratio (CSR)= 0.40, $f = 1$Hz and 10% Double Amplitude (D.A).
### APPENDIX E

**COMPACTION EFFORT**

Table E.1: Compaction effort at every layer of sands stabilized with different additives for a constant relative density.

<table>
<thead>
<tr>
<th>Sand stabilized with</th>
<th>Soil A</th>
<th>Soil B</th>
<th>Soil C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base sand</td>
<td>8,10,12,15 = 45</td>
<td>15,20,25,30 = 90</td>
<td>17,21,26,27 = 91</td>
</tr>
<tr>
<td>10% lime</td>
<td>5,6,7,7 = 25</td>
<td>6,7,8,8 = 29</td>
<td>7,7,8,8 = 30</td>
</tr>
<tr>
<td>10% fly ash-lime</td>
<td>6,6,6,6 = 24</td>
<td>11,16,21,26 = 74</td>
<td>12,17,21,25 = 75</td>
</tr>
<tr>
<td>10% rice husk ash-lime</td>
<td>7,8,8,8 = 31</td>
<td>12,17,22,27 = 78</td>
<td>12,17,22,27 = 78</td>
</tr>
<tr>
<td>10% clay-lime</td>
<td>6,6,6,6 = 24</td>
<td>6,7,8,8 = 29</td>
<td>8,8,8,8 = 32</td>
</tr>
<tr>
<td>10% bentonite</td>
<td>6,7,8,8 = 29</td>
<td>9,10,12,12 = 43</td>
<td>8,10,12,12 = 42</td>
</tr>
<tr>
<td>2% cement</td>
<td>8,9,10,11 = 38</td>
<td>12,17,22,27 = 78</td>
<td>11,16,21,26 = 74</td>
</tr>
</tbody>
</table>

Table E.2: Compaction effort at every layer of sands stabilized with various lime contents for a constant relative density.

<table>
<thead>
<tr>
<th>Sand stabilized with</th>
<th>Soil A</th>
<th>Soil B</th>
<th>Soil C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base sand</td>
<td>8,10,12,15 = 45</td>
<td>15,20,25,30 = 90</td>
<td>17,21,26,27 = 91</td>
</tr>
<tr>
<td>2% lime</td>
<td>8,9,10,12 = 39</td>
<td>10,14,18,22 = 64</td>
<td>10,15,20,25 = 70</td>
</tr>
<tr>
<td>5% lime</td>
<td>7,8,9,11 = 35</td>
<td>8,9,10,12 = 40</td>
<td>8,10,12,15 = 45</td>
</tr>
<tr>
<td>10% lime</td>
<td>5,6,7,7 = 25</td>
<td>6,7,8,8 = 29</td>
<td>7,7,8,8 = 30</td>
</tr>
</tbody>
</table>