UTILIZATION OF BUILDING DEBRIS AS AGGREGATES IN STONE COLUMN CONSTRUCTION IN SRI LANKA

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Thesis submitted to University of Moratuwa in partial fulfillment of the requirements for the Degree of Master of Engineering in Foundation Engineering and Earth Retaining Systems

by

Kongaha Waththage Dilan Sanjeewa

February 2017

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ABSTRACT

Due to rapid development and population growth, construction industry has emerged with few recent problems. The major problem faced by the construction industry is the scarcity of construction material and disposal of construction waste because of high disposal cost and inadequate land fill area.

Due to the remnants of 30 year civil war happened in Sri Lanka, huge amount of building debris are to be disposed during new infrastructure constructions. To curtail the amount of building debris, the possibility of using them (concrete, brick and plaster) for civil engineering applications can provide an attractive way to reduce the wastes to be disposed of and it may also provide fiscal benefits. In this study, the scope for using building debris as the traditional rock aggregate for stone columns was investigated.

Experiments were conducted using building debris(concrete, brick and plaster)and stone aggregate passing through a 14 mm and retained on a 10mm British standard (BS) sieves. Where experimental studies were carried out to determine the engineering properties (Durability, Shear strength & Compressive strength) of the recycled construction material and compared with conventional road construction material (aggregates).

AIV, ACV and LAAV tests and slake durability index test were carried on selected building debris to find out the suitability to be used in stone columns construction.And uniaxial compressive strength testwas carried out to find the resistance to impact and crushing under loads. Improvement in shear strength was tested using vane shear in radially as well as with depth in several laboratory models with a centred stone column made up of different building debris.

It was observed that the model done using concrete wastes exhibited a similar capacity of traditional rock aggregates of same size. Other materials did exhibit the same behavior though their results from slake durability tests were relatively low.

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NOTATIONS

LL	-	Liquid Limit
PL	-	Plastic Limit
PI	-	Plastic Index
MDD	-	Maximum Dry Density
OMC	-	Optimum Moisture Content
SSCM	-	Standard Specification of Construction & Maintenance of Roads & Bridges
BS	-	British Standards
ASTM	-	American Standards for Testing and Materials
AASHTO	-	American Association of State Highways and Technical Official
GOSL	-	Government of Sri Lanka
RDA	-	Road Development Authority
ICTAD	-	Institute of Construction Training and Development

AIV -Aggregate Impact Value

ACV -Aggregate Crushing Value

LAAV- Los Angele Abrasive Value

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CHAPTER 1

INTRODUCTION

Due to rapid development of infrastructure and population in Sri Lanka, construction industry has become very dynamic. The problems anticipated by the Government of Sri Lanka are scarcity of construction material and disposal of construction and demolition waste because of unavailability of land fill area to be used as dumping sites.

Most of the new roads have been designed through the paddy fields, marshy lands, barren lands etc. to reduce disturbances to the present livelihood. Therefore, deep deposits of soft soil coupled with construction of high embankments could result in potentially large post construction settlements, which may exceed the post construction settlement limits. Therefore, in the long term (operational stage), soft ground treatment (SGT) is required to control the embankment settlement and stability whilst ensuring an adequate pavement and structural performance is maintained during service life.

Where soft cohesive soil thickness is greater than 5m and stability and stringent post construction settlement requirements cannot be fulfilled using conventional ground treatment methods such as remove/replace, preloading and preloading with wick drains, ground improvement techniques involving vibro-displacement can be positively considered. Columns formed using the vibro-displacement method is often referred to as stone columns.

Construction of stone columns consumes a huge quantities of different materials, and, the major consumption of materials is stone or rock aggregates.

Figure 1.1: Aggregates Usage in Stone Columns in Road Construction (www.waikogroup.com)

However, recently encountered problem in acquiring sufficient road construction material (especially aggregates) for road projects is due to natural geological reasons as well as protests after environmental awareness among the public. In addition to that, disposal of huge quantities of building debris and other construction wastes is also a serious concern to the authorities because of limited landfill area and huge cost of transportation.

Ex:After 30 years of war, there are enough debris all around in North and Eastern provinces and those building debris consist large amount of concrete, brick and plastermasonry units.

Figure 1.2: Building Debris(Kankesanthurai, Jaffna)

Sri Lanka is practicing open dumping at lowlands in most of the waste dumping situations, nevertheless, disposing of industrial waste in these lands will cause flooding, environmental degradation and could affect the livelihood of residents. Hence, it is regarded as one of the best solutions to reuse the construction waste and building debris in construction projects.

The purpose of this study is to identify suitable building debris and utilizing them as aggregates in stone columns for improving unsuitable grounds in road construction projects. This would curtail the construction cost as well as will help overcome the problem of disposal of building debris.

Concrete, plaster and block masonry units consist of burnt brick pieces could be identified as mostly available and selected to find out the suitability as aggregates in stone columns construction work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The road sector (especially expressways) is booming due to urbanization and infrastructure development in Sri Lanka and of course it leads to seek huge quantities of construction materials. Therefore, significant volumes of demolition debris are generated in Sri Lanka and ends up in municipal solid waste landfills or incinerators. Government of Sri Lanka (GOSL) is facing difficulties in finding solid waste landfills and costly disposal of demolition waste especially in urban area. Hence, GOSL is continuing to work in order to divert this waste away from land disposal by promoting the reuse and recycling of demolition debris and reducing its generation through green building.

Some states enforce local regulations & policies to limit disposal of demolition debris and promoting to reduce, reuse and recycle waste (United States Environmental Protection Agency, 2004). Reducing and recycling of Construction and Demolition(C&D) materials conserves landfill space, reduces the environmental impact of producing new materials, creates jobs, and can reduce overall building project expenses through avoided purchase/disposal costs (United States Environmental Protection Agency, 2004).

Nowadays, old traditional buildings are demolished and built new multistory buildings for upgrading living standard of people according to the current trend and availability of limited lands.With the limited supply of landfill sites and great demand for waste disposal, the cost of dumping of waste has been increased in recent times. Therefore, if construction debris such as concrete, plaster and bricks can be reused as a stone column construction material instead aggregates, it would be an attractive way to reduce the wastes to be disposed of and it may also provide fiscal benefits.

In the advancement of ground improvement technique, stone columnsconstruction are broadly used in the poorgrounded soil to increase the bearing capacity. The stone column technique is a very efficient method ofimproving the strength parameters and reducing primary consolidation settlement. It offers a mucheconomical and sustainable alternative to piling and deep foundation solutions. Before finding information of previous researchstudiesrelated to the use of alternative material to replace stone aggregate either fully or partially for the construction of stone columns, it is expected to find the details about stone column construction only related to our study from theliteratures.

2.2 Stone Columns

Among the various techniques for improving in situ ground conditions, reinforcing the ground with stone columns or granular piles is one of the most versatile and cost-effective methods. Load bearing columns of well compacted coarse aggregates are installed in the ground to serve various purposes such as reinforcement, densification and drainage. Stone columns provide the primary functions of reinforcement and drainage by improving the strength and deformation properties of the soft soil (Black et al., 2007; Madhav and Miura, 1994).

Stone columns are most effective in clayey soils with untrained shear strength in the range 7-50 kPa [Indian Standard(IS) 15284; Bureau of Indian standards (BIS), 2003]. The most common material used for stone column construction is crushed stone aggregates of sizes varying in the range 12-100mm.(Mitchell and Huber, 1985; Murugesan and Rajagopal, 2007).Due to high angle of internal friction and stiffness of stone columns when compared to that of in-situ weak soil,majority of applied load is transferred to stone column. As a result, fewer loads are transferred to surrounding weak soil which leads to reduction in settlement.

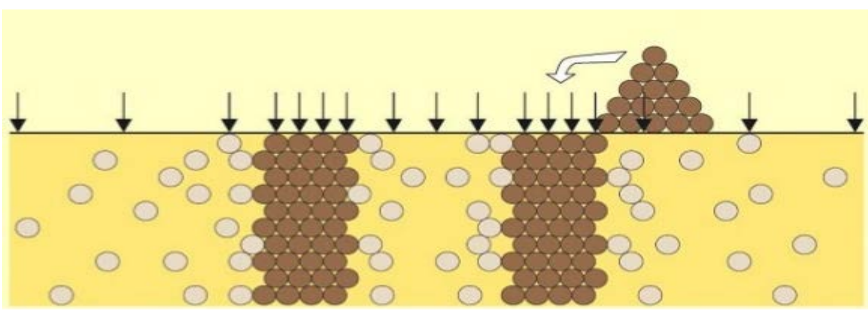
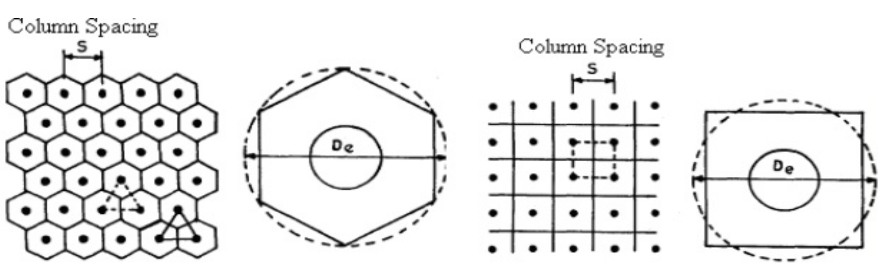


Figure 2.1: Construction Procedure of Stone Column



D_e = 1.05 S (a)D_e = 1.13 S (b)

Figure 2.2: Installation Patterns of Stone Columns (a) Triangular arrangement (b) Squarearrangement[S.V. Abhishek& V. Tarachand (Department of Civil Engineering, College of Engineering, Andhra University, Visakhapatnam.), 2007].

2.2.1 Load Carrying Mechanism of Stone Column

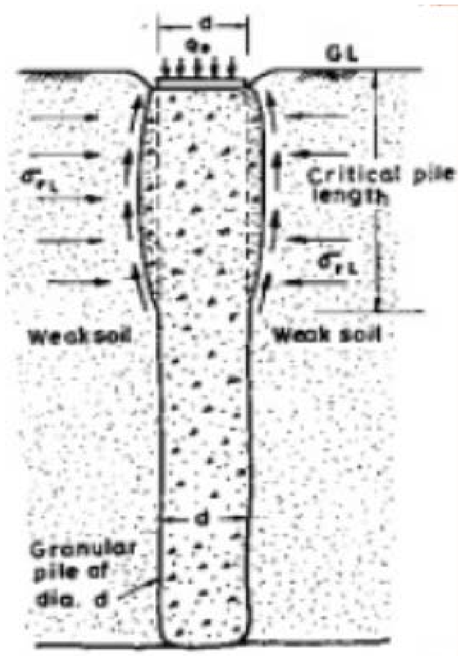


Figure 2.3:Load Carrying Mechanism of a Single Stone Column

[S.V. Abhishek& V. Tarachand (Department of Civil Engineering, College of Engineering, Andhra University, Visakhapatnam.), 2007].

The stone columns derive their load carrying capacity from thelateral earth pressure/radial confining stress against bulging from surrounding soil (Hughes and Withers, 1974; Hughes et al.1976). However, when stone columns are installed in extremely soft soils having undrained shear strength less than 7 kPa, the radial confinement/restraint offered by the surrounding soil is inadequate, resulting in excessive lateral displacement of stone into the surrounding soil. In such circumstances, the load carrying capacity of the stone column can be improved by encasing the column in a suitable geosynthetic. Figure 2.3 depicts the load carrying mechanism of a single, isolated stone column in compression. The length of stone columns over which bulging takes place is known as critical length (about 4 times the diameter of the column).

Apart from the bulging, stone columns derive their load carrying capacity through surface resistance or frictional resistance developed between the column material and surrounding weak soil acting upwards within the critical length, and also from the passive resistance mobilized by the column material. The portion of the stone column below the critical length does not participate in load transfer but functions similar to a vertical drain and accelerates the consolidation of the surrounding soft soil.

2.2.2 Estimation of Load Carrying Capacity of Stone Column

Load carrying capacity of stone columns can be between 100 to 400 kN and end bearing is not considered in the estimation of load carrying capacity because load carrying mechanism is due to local perimeter shear. According toHughes and Withers, (1974) estimation, load carrying capacity of a single stone column can be done by using following equation-01 where assuming that foundation loads are carried only by the stone columns with no contribution from the intermediate ground.

$$q_a = \frac{K_p(4c + \sigma_r')}{F.S.} \qquad \longleftarrow \text{Equation -01}$$

Where;

q_a = allowable bearing capacity of stone column

K_p = coefficient of passive earth pressure

= $\tan^2(45+\varphi/2)$

c = cohesion of soil

σ_r' = average effective radial stress over a depth of '4d' where 'd' is the diameter of the column

F.S. = factor of safety = 1.5 to 3.0

Load carrying capacity of stone columns is generated by the top section of the column which extends to about 4 times the diameter of the stone column. The length below 4xdiameter allows for radial drainage and acceleration of settlements. To retain continuity of drainage path, it is necessary to provide a 150 mm thick drainage blanket on top of the stone columns. This drainage blanket will reduce the drainage path by changing the situation to doubly drained case. Stone columns should extend through weak soil to harder firm strata to control settlements. Provision of stone columns does not reduce the entire consolidation settlement. The reduction depends on the spacing of stone columns (generally 2.0 to 3.0 m c/c over the site). Maximum percentage reduction of settlement is 75%.

2.2.3 Failure Mechanisms of Stone Columns

Stone columns are often constructed through soft soils fully penetrating to an end bearing stratum. However, they may be constructed as floating piles and the tips ending within the soft layer but at depths where the strength of soil is adequate. Stone columns may fail individually or as a group. The failure mechanisms(IS: 15284 Part 1 – 2003) for a single, isolated stone column in compression are illustrated in Figure 2.4 indicating respectively, the possible failures as a) bulging failure, b) shear failure and c) sliding /punching failure.

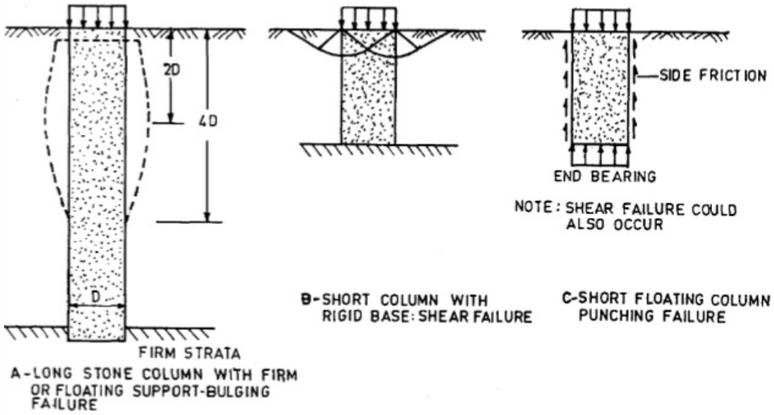


Figure 2.4: Failures of a Single Stone Column

[S.V. Abhishek& V. Tarachand (Department of Civil Engineering, College of Engineering, Andhra University, Visakhapatnam.), 2007].

For single, isolated stone columns, the most probable failure mechanisms are bulging or punching. Punching failure mechanism controls the ultimate load for short columns resting on soft to medium stiff bearing layer(the top of the column is floating in the soft soil) while bulging failure is most likely for a long stone column irrespective of end bearing or floating(Madhav et al. 1994). A long stone column is one whose length is greater than its critical length (about 4 times the diameter of the column). Practically since most stone columns are installed up to depths of 10 - 15 m preferably into stiff end bearing stratum, lateral bulging of the column into the surrounding weak soil is the pre-dominant load transfer mechanism.

2.2.4 Stone Column Construction

Natural rock aggregate are the basic materials used in stone column constructions. Whatever installation techniques used, engineering properties and behavior of stone column construction materials (aggregates) will vary considerably. As such, the testing of these materials is very essential to ensure the quality and durability of the stone columns constructed. The desirable properties of aggregates relevant to stone columns construction are listed below.(Testing of Road Construction Materials, 2006)

i. Resistance to impact and crushing under traffic loads and construction equipment (Compressive strength).
ii. Resistance to shear failure (Shear strength).
iii. Resistance to weathering (Durability).

2.3Previous Studies Related to The Use of Alternative Material for Stone Column Construction

Although there have been a lot of studies to improve stone column performance by encasing them or by inserting steel rebar, there has been few studies carried out on the use of alternative material to replace stone aggregate either fully or partially for the construction of stone columns.

V.Sivakumar and D.Glynn (2004) carried out experimental studies by considering four recycled waste materials such as freshly quarried basalt(for comparison purpose), quarry waste, building debris and crushed concrete and these materials were examined under various test conditions; namely dry, wet and mixed with 10% and 20% clay slurry. Tests were also carried out to examine how the performance of these recycled waste materials could be enhanced using geogirds. Laboratory tests were carried out in a 305 mm x 305 mm direct shear box. The results showed the performance of the recycled materials were not significantly different to that of freshly quarried basalt, although the conditions under which the products were tested (ie. dry, wet and smeared with clay slurry) appeared to influence their performances in a significant way.

A case study (Serridge, 2005) was reported in which spent railway track ballast and crushed concrete were used as aggregate for stone columns in UK. Joe Persichetti (2010) did a similar study with crushed concrete to check whether that could be used as a sustainable alternative to quarried crushed stone in vibro stone columns and used crushed concrete generated from the onsite demolition of a large slab and concrete columns. The allowable bearing pressure observed at foundations had been 6 ksf or 290 kPa (column loads over 2,000 kips or 900 kN) and the settlement tolerance had been 1 inch (25.4 mm) total and 0.5 inch (12.7 mm) differential.

Ramanathan and Sasikala (2014) had carried out experimental studies to measure the behavior of single stone columns made of both tyre chips and stone aggregate. Where fifteen model experiments had been carried out on stone columns made of different mix proportions of stone aggregate (S) and tyre chips (T), in a kaolinite clay bed of uniform consistency. They had found that the mixtures of 60%T + 40%S and 40%T + 60%S provided a load-carrying capacity in stone columns similar to that of stone columns made using stone aggregate (100%S). This concludes that waste tyre chips could be used as a partial replacement for stone aggregate up to about 40-60% in the construction of stone columns.

Having gone through the literature, it is expected to do experimental study to find out the suitability of engineering properties of various kinds of building debris such as concrete, plaster and bricks for Stone

Columns Construction as aggregatesfor improving unsuitable grounds in road construction projects in Sri Lanka.

CHAPTER 3

METHODOLOGY

This research is an experimental study to find out the suitability of engineering properties of various kinds of building debris for Stone Column Constructions where samples of plaster, concrete and block masonry samples were collected from Jaffna District, Northern Province of Sri Lanka and the standard tests mentioned in Table 1 were carried out for each made debris sample and stone column modelsseparately.

Table 3.1: Standard Tests

Tests	**Standards**	**For what**
AIV Test	BS 812, part 112	each debris sample separately
ACV Test	BS 812, part 110	each debris sample separately
LAAV Test	ASTM C131	each debris sample separately
Slake durability index test	ASTM D 4644 - 87	each debris sample separately
Vane shear test	ASTM D2573 / D2573M - 15	clay surrounding the stone column model made of each debris material separately
Uniaxial compressive strength test	ASTM D 2166 / D2166M - 16	stone column model made of each debris material separately

AIV, ACV& LAAV tests and Slake durability index test were carried on selected building debris to find out the suitability to be used in stone columns construction.

Where soft soils are identified as problematic in the construction area, Stone columns are most effective and those provide the primary functions of reinforcement and drainage by improving the strength and deformation properties of the soft soil. Improvements in shear strength was tested using Vane shear in radially as well as with depth in several model studies done on stone columnmodels of different building debris.

Uniaxial Compressive Strength test was done to point out the capacity of made stone column models with each building debrisseparately to withstand loads tending to reduce size, as opposed to tensile strength.

Then analyzed the results of above test properties with conventional material(aggregate) properties used for Stone Columns construction published by ICTAD publication No. SCA/5, Second Edition, June 2009; 'Standard Specification for Construction and Maintenance of Roads and bridges (SSCM)' issued under the authority of the Director General of the Road Development Authority.

Finally, Conclusions and recommendations are laid down based on these compared results.

CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 Characterization of Materials

4.1.1 Clay

Koalinite clay found in the region was collected in the studyfor the surrounding of each stone column. The various index and engineering properties of clay are measured as per procedures recommended by ASTM and British standard (BS) codes of practice and these test and values are presented in below.

4.1.2 Building debris

The scaling factor used for the size of building debris chips was 1/10 (Murugesan and Rajagopal,2007; wood, 2004).Building debris as the particles passing through a 14 mm sieve and retained on a 10 mm sieve were used as aggregates in the present study.Tests and Properties obtained for building debris are presented in next pages.

4.1.3 Stone aggregates

To compare the behavior of building debris with that of stone aggregate, the size of the stone aggregate should be similar to that of the building debris. Crushed stone aggregates passing through a 14 mm sieve and retained on a 10 mm sieve were chosen for the study.

4.2Tests on Clay Sample

Kaolinite clay is used for the surrounding of each stone column and various index and engineering properties of kaolinite clay are measured as per procedures recommended by ASTMD 854,D 4318, D 2216, D 421&422, D 698-00a andD 2974Standards and BS 1377:Part5:1990:3Standards. According to that Specific gravity test, Atterberg limit test, Moisture content test,Hydrometer analysis test,Proctor compaction testandConsolidationtest in clayey samples were done and each test observation and specimen calculationis given in Appendix 1-7.

4.3Tests on Building DebrisSamples

4.3.1 Aggregate Impact value (AIV)

The aggregate Impact Value gives a relative measure of the resistance of an aggregate to sudden shock of impact. The test is carried out according to BS 812, part 112. Thus it will be seen that higher the aggregate impact value weaker is the aggregate. Generally, aggregate whose aggregate impact value is greater than 30% is not used in stone column construction.

4.3.2Aggregate Crushing Value (ACV)

The aggregate crushing value is a measure of the resistance of an aggregate to crushing under a gradually applied compressive load. As per BS 812 part 110, the test is carried out to determine ACV value. The aggregate with a low aggregate crushing value is stronger than an aggregate which has a high value. Generally, aggregate whose aggregate impact value is greater than 35% is not used in stone column construction.

4.3.3 Los Angeles Abrasion Value Test (LAAV)

This test attempts to measure the deterioration of aggregate particles subjected to attrition. ASTM method test is done to determine LAAV value (ASTM C131). A low value of LAAV reflects and aggregate which is more resistant to abrasion. The aggregate with LAAV greater than 40 are too soft for stone column construction.

Above tests observationsand specimen calculations are given in Appendix 8.

4.3.4Slake Durability Index Testand Test Procedure(ASTM D 4644-87)

Slake durability index test for durability testing of building debris was done and test procedure can be given as follows.The slake-durability test is intended to assess the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles of drying and wetting.

- Rock samples were put into an apparatus that comprises two sets of drums of the length of 100 mm and the diameter of 140 mm.
- The two drums rotated in water that had a level of about 20 mm below the drum axis.
- The rotation was driven by a motor capable of rotating the drums at a speed of 20 rpm, which was held constant for a period of 10 minutes.
- Ten rock lumps, each had a mass of 40-60 g, were placed in the drums.

- After slaking for the period of 10 minutes, these rock samples were then dried in an oven at a temperature of 105 degree centigrade for up to 24 hrs.
- Finally, the mass of dried samples was weighted to obtain the first cycle. The test was conducted over two cycles, in which the weight of particles of 10 rock lumps retained in these wet-dry cycling tests was therefore determined.

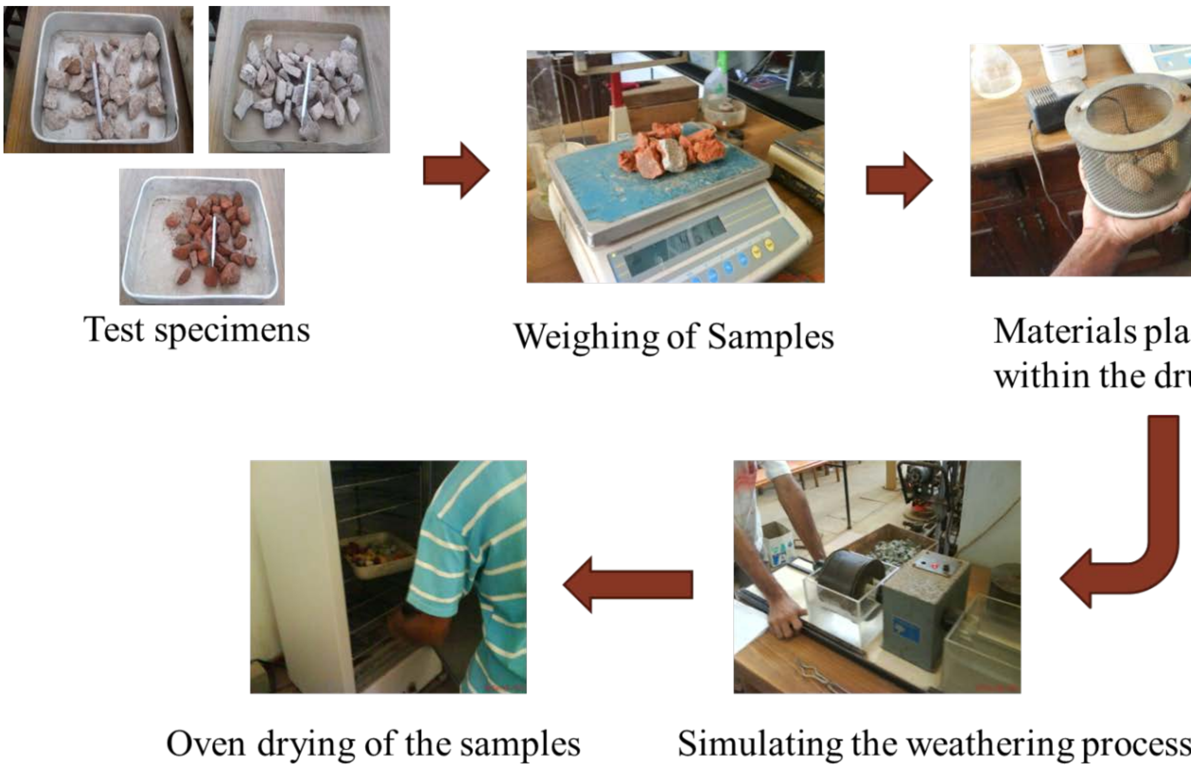


Figure 4.1: Slake Durability Test Procedure

Test results are expressed as a slake durability index for each particular materials. The slake-durability test is regarded as a simple test for assessing the influence of weathering on rock and its disintegration. Where building debris was subjected into slake durability test machine and tested samples initially, after one and two months respectively.Samples subjected to testmachine are finally classified according to Gamble classification(Goodman,1980) which gives group names for materials according to percentage of materials retained after first and second cycles of durability test.Hence comparing the values of the gamble's table, thesample can be found to be very or less durable.This test observationand specimen calculations are given in Appendix 9.

4.4Tests on Stone ColumnModels

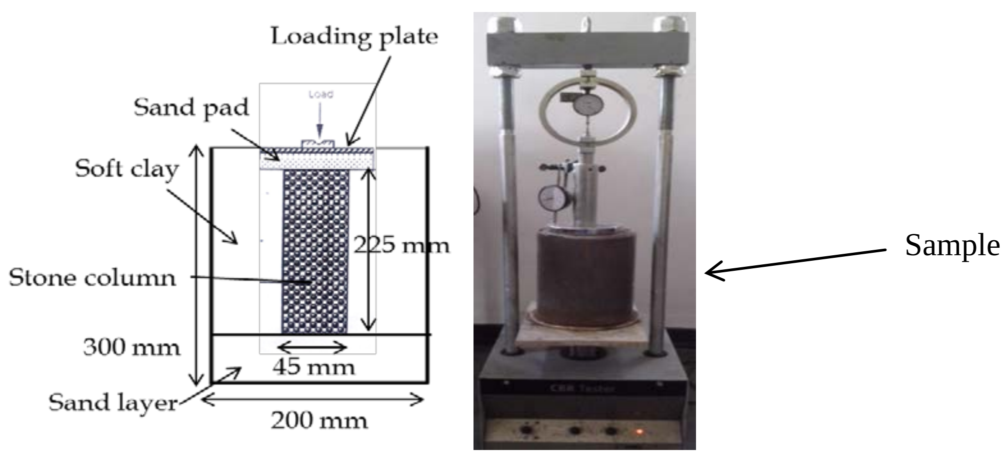
4.4.1 Test set-up and Loading Arrangement

An acrylic cylindrical tank 300mm height was used as the model tank (Figure 2.7).The diameter of the tank was 200mm, with a wall thickness of 5mm. The scaling factor used for the stone column model was 1/12 (Murugeshan and Rajagopal, 2007; Wood, 2004).

The stone column diameter used for the test was 45mm. In the tank, clay was placed to a height of 225mm, and the stone column was installed at the centre (process discussed in 2.3.3.1.3 section). The height of the stone column was 225mm. This was chosen in a such a way that the ratio of the length of the column to the diameter of the column (L/d) was a minimum of 5, which was required to develop the full limiting axial stress on the column (McKelvey at al., 2004).

The total height of the clay bed placed in the tank was five times the diameter of the column. The initial gap between the bottom of the stone column and the bottom of the tank base was 50 mm and which was filled with sand(density of 1951 kgm^{-3}).

Another sand layer of 20mm thickness was placed at the top of the stone column, with a diameter equal to 100mmand acted as a sand pad. A vertical load was applied over a diameter equal to thecircular influence area that is more than two times the diameter of the stone column. The load was applied through a proving ring at a constant strain rate of 0.25 mm/min. The typical loading arrangement used in the present study is shown in Figure 4.2(a) and Figure 4.2(b).



(a)(b)

Figure 4.2: TypicalLoading Arrangement of Stone Column:

(a) Sketch; (b) Sample within the test machine

4.4.2 Preparation of the Clay Bed

Sufficient amount of clay was collected with minimum disturbance from a place where a highway construction is proposed.The shear strength of the clay varies with the water content, so it was necessary to conduct all the experiments with the same water content to compare the performance of the stone column.

The clay was then covered with wet cloths until it was placed in the made PVC column. Silicon grease was applied to the PVC column wall to reduce any friction between the clay and the column wall.

The test column was inserted into sample collected basket to take column sample by hand, which was collected, spread and patted evenly to avoid entrapment of air.Care was taken to ensure that no air was entrapped within clayey soil as they were inserted into columns or between the soil and the column wall. The variation of water content with different heights of the clay layer (top, middle and bottom)for each trial of model test was same, and proves that the clay bed was prepared homogeneously.

4.4.3 Construction of the Stone Columnand Compressive Strength Test Procedure(ASTM D 2166 / D2166M - 16)

The column was constructed by the replacement method. The centre of the cylindrical tank was accurately marked. A 45mm outer diameter, thin, open-ended and seamless polyvinyl chloride (PVC) pipe was pushed into the clay at its centre up to 225mm. A thin coat of grease (less than 1mm thick) was applied both the inner and outer surfaces of the pipe for easy withdrawal without any disturbance to the surrounding soil. A small helical auger was used for scooping out the soil within the casing pipe.

One of building debris sample 10-14 mm, in size were charged into the hole in layers of 30mm, giving a light uniform compaction to each layer. The casing pipe was raised in stages, ensuring a minimum of 5mm penetration below the gravel placed. This process was carried out for other building debris samples separately. It was also observed that, after removing the casing pipe, the grease coat remained on the pipe only. Hence, this grease coat did not affect the smearing and hydraulic conductivity of the interface between the stone column and the surrounding soil.

Finally stone column with one of building debris within clay surrounding was made. The same procedure was adopted for other column test samples as well.Construction procedure of the stone column for each building debris sample is shown in Figure 4.3.

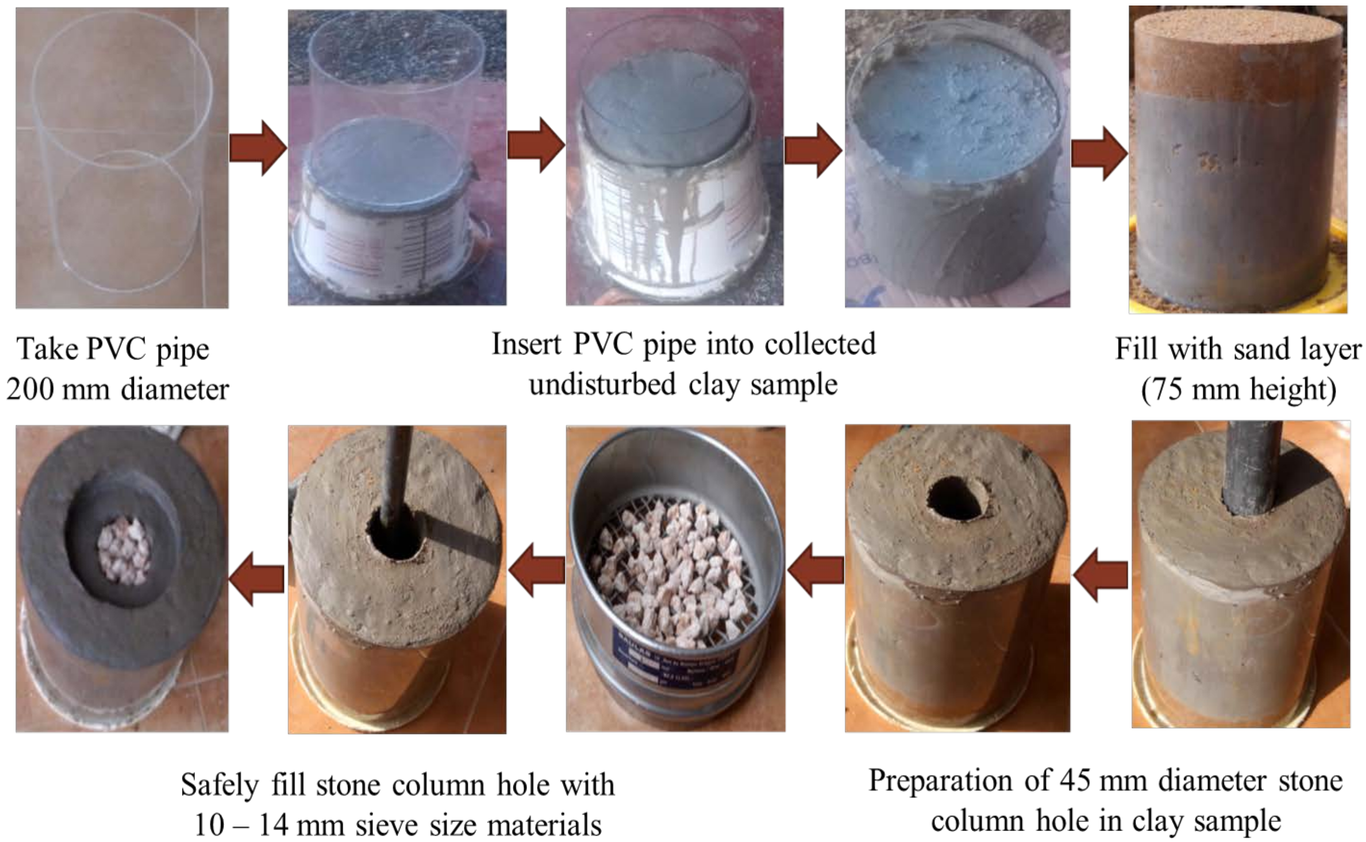


Figure 4.3: Constructionprocess of the Stone Column

- After preparing the stone column up to the soil level, sand passing through a 4.75mm sieve was placed as a 20mm thick blanket over the prepared stone column.

- Finally, a loading cap 100mm in diameter, which was equal to the diameter of the circular influence area and 12 mm thick, was placed over the stone column to load evenly at the centre.

- The load deformation behavior of the column was studied by loading it in a loading machine frame at a strain rate of 0.25 mm/min.

- The load was observed for equal intervals of 0.125mm displacement. Since the loading was rapid, it was essentially undrained loading which simulates loading immediately after construction. Arrangement of the stone column prior to fix into the compressive test machine is shown in Figure 4.4.

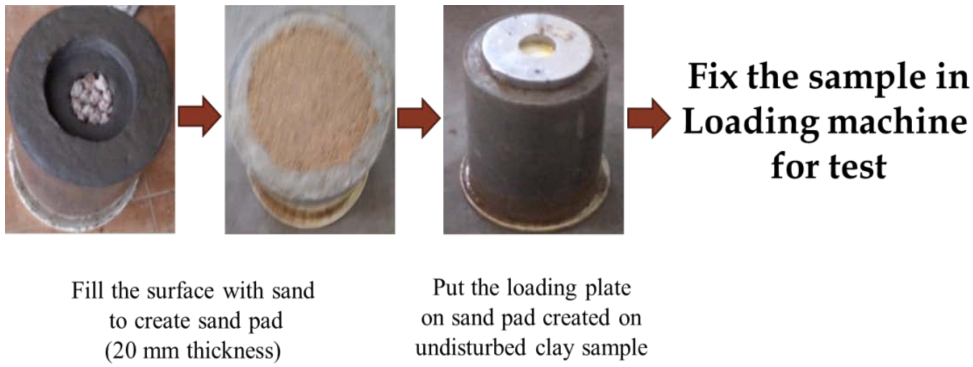


Figure 4.4: Arrangement of the Stone Column Prior to Compressive Test

Calculation of actual load values (kN) corresponding to the dial reading of proving ring values were taken directly from compressive strength machine calibration chart. From compressive strength machine calibration chart, calculation of actual load values (kN) corresponding to the dial reading of proving ring values with displacement was done for concrete block, bricks, cement plasterdebris separately. Also compressive strength values of clay and aggregates with displacement are taken to compare compressive strength with building debris separately.Observationsand calculated compressive strength values for these samples are tabulated in Appendix 10&11.

4.4.4Vane Shear Testand Test procedure(ASTM D2573 / D2573M - 15)

In situ vane shear tests in the made test columns were carried out using a pocket torvane shear apparatus for verifying the drained shear strength characteristics of the clay surroundingof each columns with time and depth.Vane sheartest on each model made of different type of building debris was conducted initiallywithout load and 07 days & one month soaked periods with 8 kg load at top (below 33mm of blade height) and 80 mm (without blade height) depth separately at locations as shown in Figure 4.5.

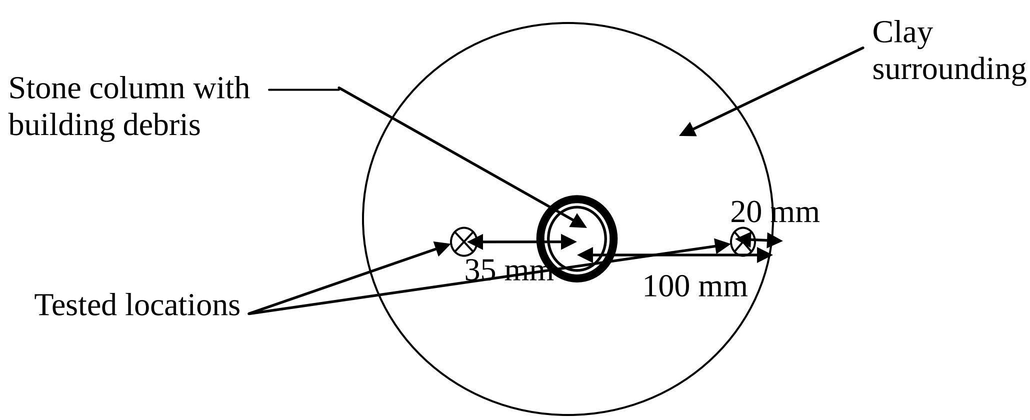


Figure 4.5: Testing Locations of Vane Shear Test

- The apparatus was cleanedthoroughly and apply grease to the lead screw.
- Sampling tube was mountedwith sample.
- The maximum pointer was brought into contact with the strain indicating pointer and noted down the initial reading of these pointers on the circular graduated scale.
- The bracket was lowereduntil the shear vanes went into the soil sample to their full length.
- The torque applicator was operated and handled until the specimen failed, which was indicated by the return of the strain-indicating pointer or rotation of drum.
- Readings of the maximum pointerwere noted down.
- The difference between the two readings (initial & final) gave the angle of torque.
- Above steps were repeated on othermodels to obtain the average shear strength of the sample.
- This process was done for each column model at top (below 33mm of blade height) and 80 mm depth(without blade height) at above mentionedtested locations separately.

Figure 4.6: TestingArrangement of Vane Shear Test

Shear strength values (kPa) of clay surrounding corresponding to the apparatus divisions can be directly obtained by following calibration chart. These observations, shear strength values and calibration chart are tabulated in Appendix 12,13& 14.

CHAPTER 6

ANALYSIS

6.1Analysis of Clay Sample

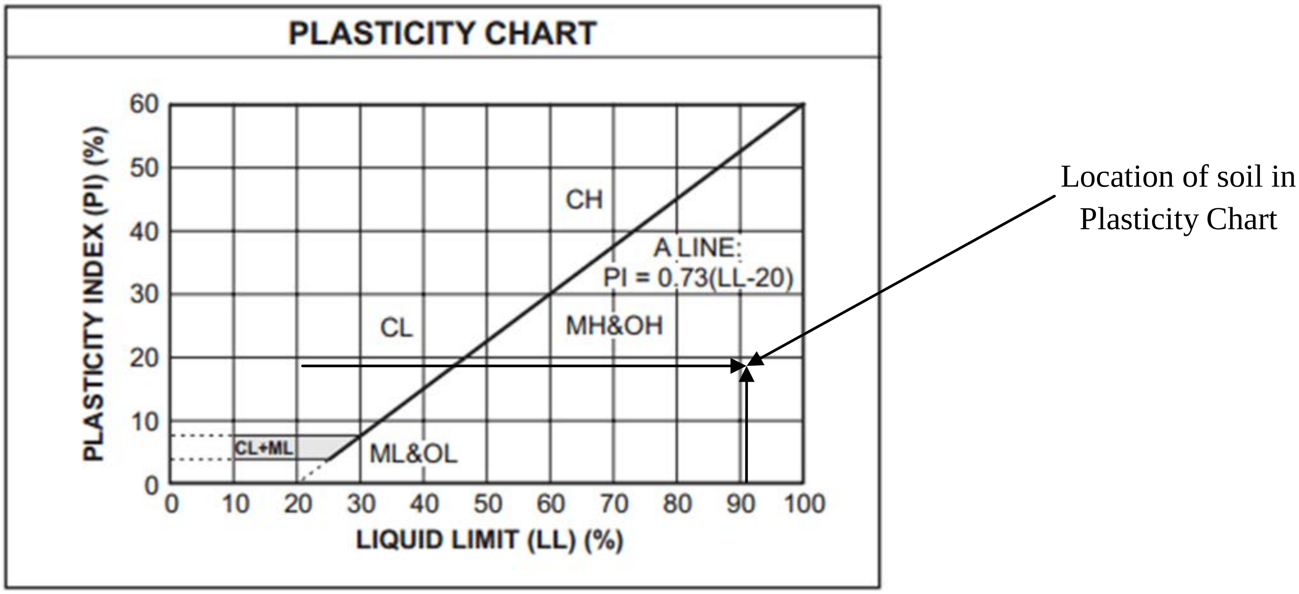


Figure 6.1: Plasticity Chart

According to Unified Soil Classification System (USCS) soil classification and based on the plasticity characteristics of the clay, it is classified as a **High PlasticitySilt Soil** (**MH**). Where Activity of the soil (A) was0.36which is less than 0.75, therefore it is considered as **Inactive** soil.

According to American Association of State Highways and Technical Official (AASHTO) soil classification, this soil sample belongs to **A-7-5** group and it is classified as **Very Poor Clayey Soil.**

Then, average coefficient of permeability of our sample was 2.5E-10 m/s and it means that soil has poor drainage property. And based on consolidation test results, this clay sample has high settlement characteristics therefore stone columns are most effective in its' property improvements. Then it is clearly observed that our collected clay sample for surrounding is suitable forstudy of behavior of stone column models made with different building debris under this research.

6.2 Analysis of Building Debris Tests

6.2.1 Slake Durability Test

Using the results of slake durability test, slake durability index groups for different types of building debris can be analyzed according to Gamble classification (Goodman, 1980). Accordingly, slake durability index groups for different types of building debris can be proposed as presented in Table 6.1.

Table 6.1: Slake Durability Index Groups of Different Types of Debris

Type of waste material	Durability		
	Initial	Soaked for one month	Soaked for two months
Concrete	Medium High	Medium High	Medium High
Plaster	Medium	Medium High	Medium High
Brick	Low	Low	Low
Rubble	Very High	Very High	Very High

It was clearly observed that the concrete is categorized as medium high durability in comparison to the other tested materials. This is still lower than the standard stone column fill material of rock aggregates. However, when aggregates' durability values are concerned, crushed concrete could be better utilized compared to other debris in stone column construction work instead rock aggregates subject to further testing.

6.2.2 AIV, ACV & LAAV Tests

Table 6.2: Summary of AIV, ACV & LAAV Tests

Sample	AIV	ACV	LAAV
Concrete Debris	31	35	42
Aggregates	25	25	31
Brick	unable to prepare a sample		
Plaster	unable to prepare a sample		

As per the ICTAD Publication for Standard Specifications for Construction and Maintenance of Roads and Bridges (SSCM), June 2009 stone column material should have following properties,

Table 6.3:RDA Criteria for Stone Column Material

Test	Properties
AIV (%)	< 30
ACV (%)	< 35
LAAV (%)	< 40

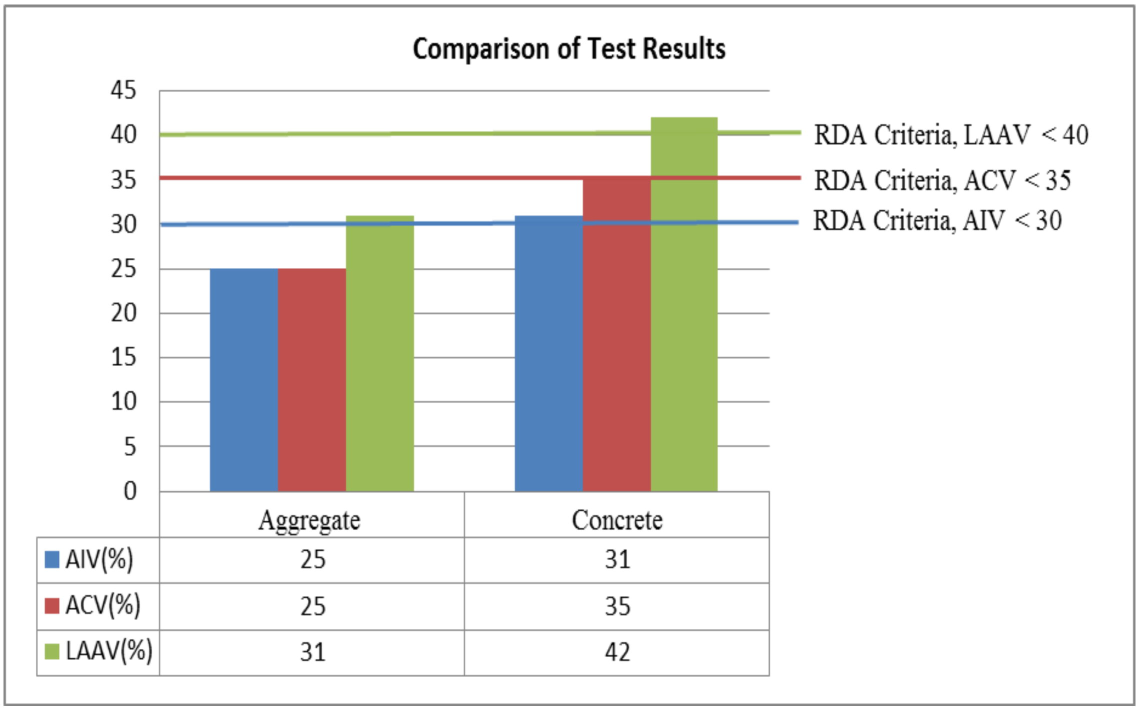


Figure 6.2 Comparison of AIV, ACV & LAAV Tests

Considering RDA criteria given in Table 6.3 and Figure 6.2 for stone column material, aggregates can only be directly used for stone column construction. Only concrete debris, among other building debris fulfills the above requirements marginally. Therefore, crushed concrete instead aggregates could be a feasible solution especially in situations where excessive concrete demolition waste is available.

6.3Analysis of Stone Column Model Tests

6.3.1Compressive Strength Test

According to the results of compressive strength test for different types of building debris, graphs shown in Figure 6.3 are plotted between compressive load and penetration. Subsequently the ultimate load for building debris was obtained.

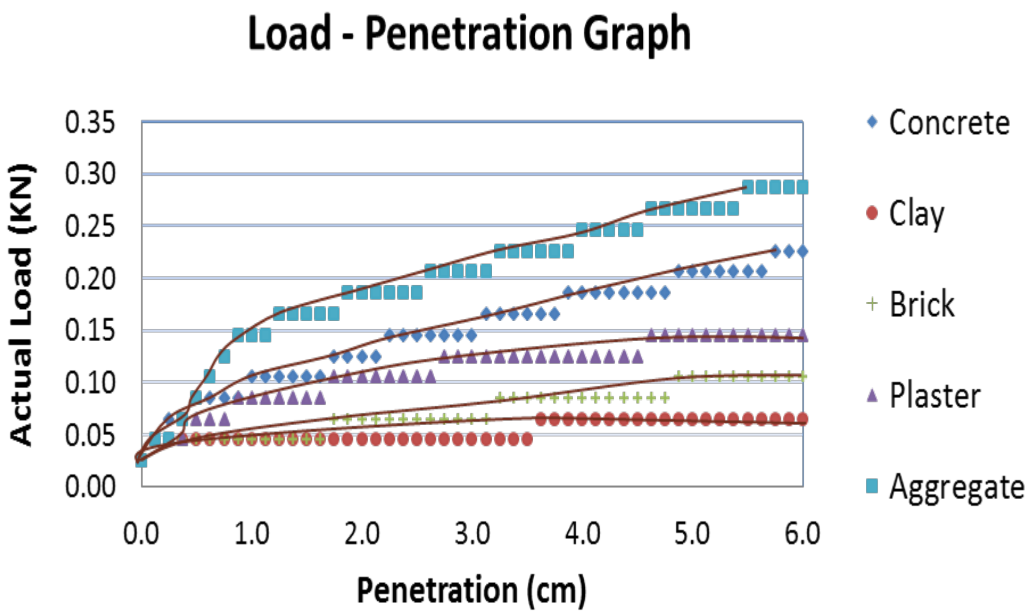


Figure 6.3:Variation between compressive load and penetration for different type of building debris

Figure 6.3 clearly shows that the load-settlement behavior is non-linear for all materials tested. Aggregates had higher ultimate compressive load capacity (250 N) and not surprisingly it can be seen that the stone column made of only clay shows the lowest ultimate carrying capacity (40N).Efficiency of building debris compared to rock aggregates can be tabulated as presented in Table 6.4.

Table 6.4:Ultimate load efficiency of building debris related to aggregate

Description	Q ult : N	Efficiency % related to Aggregate
Aggregate	250	100
Concrete	180	72
Plaster	120	48
Brick	80	32

Accordingly, concrete had a higher efficiency in comparison with aggregate than other tested materials. For the same settlement, the difference in load–carrying capacity between the two materials is only marginal (30N) at small settlements. However, it is significant (80N) at high settlement that is at the ultimate stage. A similar trend was observed for both trial tests conducted. Although there is a comparable difference between the stone columns made of aggregates and crushed concrete, concrete debris can be used as an aggregate for the construction of stone column under working loads.

6.3.2Vane Shear Test

It was found that the measured undrained shear strength in the model ranged from 5 to 13 kN/m^{-2}, thus proving the accuracy of the clay bed preparation work and stone column has been most effective in this type of clayey soils.

According to the results of shear strength of surrounding clay of these stone column models, it was observed that the shear strength has increased near stone column area compared to the other distances. Thus, high shear strength can be observed at high depth of surrounding clay and it has become lower with decreasing depth of surrounding clay. Then, graphical representation of shear strength of stone columns made of different building debris in clay surrounding area for a distance of 80mm and 35mm from the center is given in Figure 6.4 and 6.5 respectively.

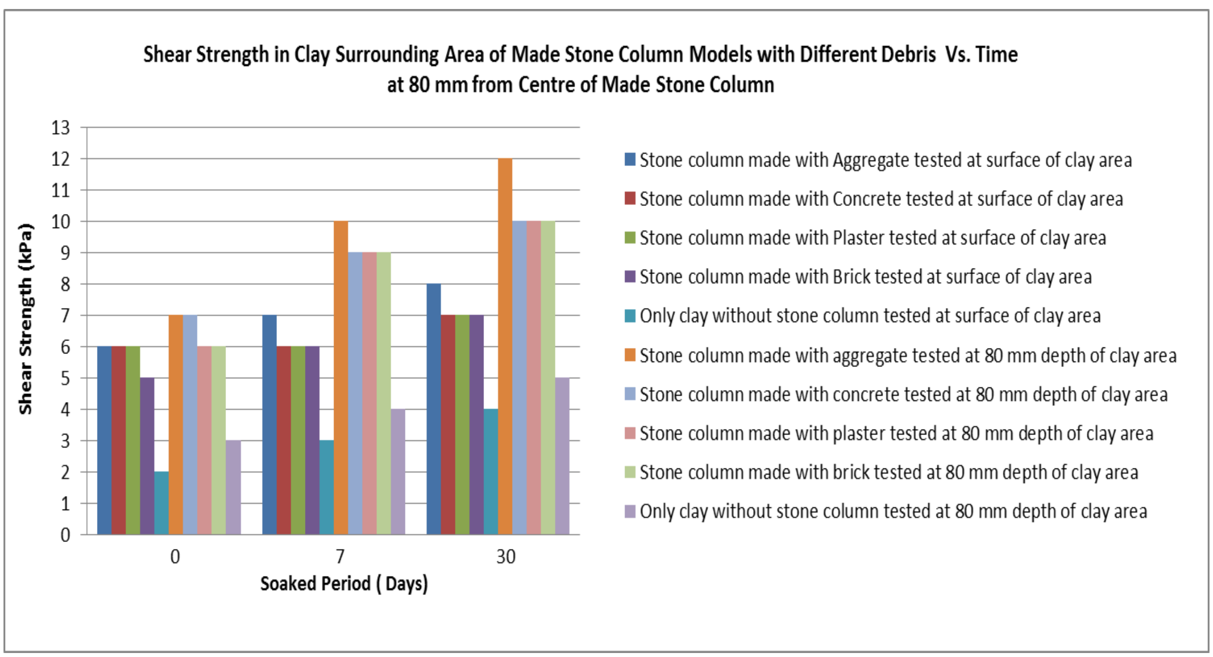


Figure 6.4: Graphical representation of undrained shear strength in clay surrounding of different stone column models and time relationship at 80 mm distance from the centre of stone column

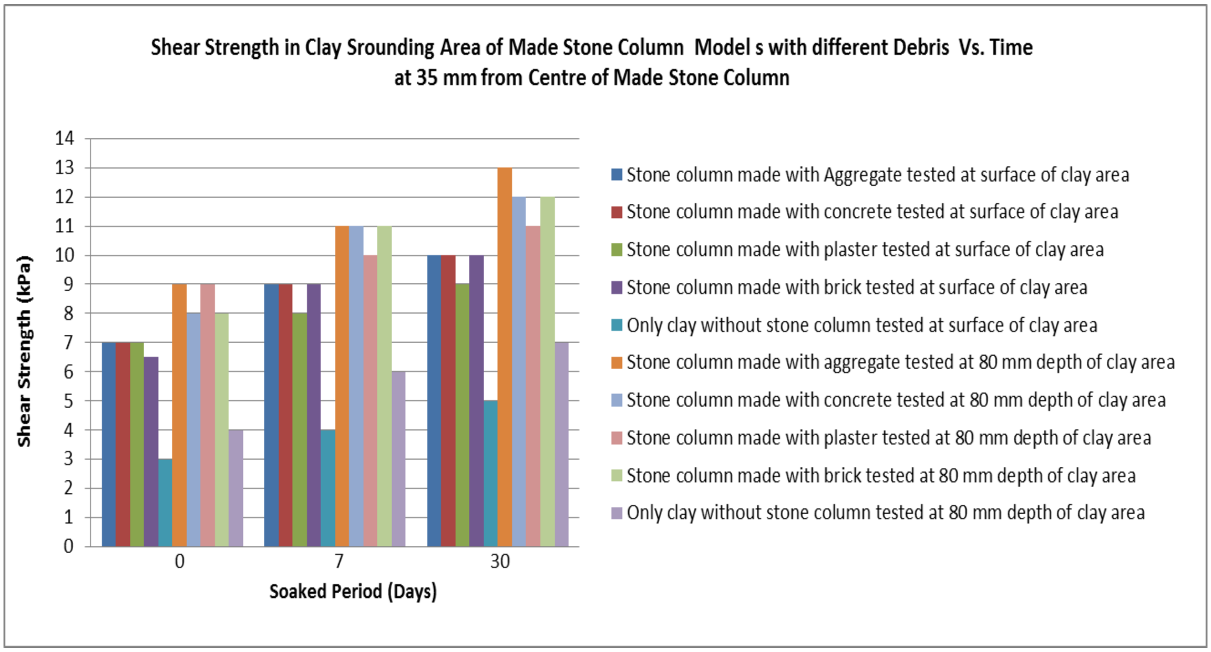
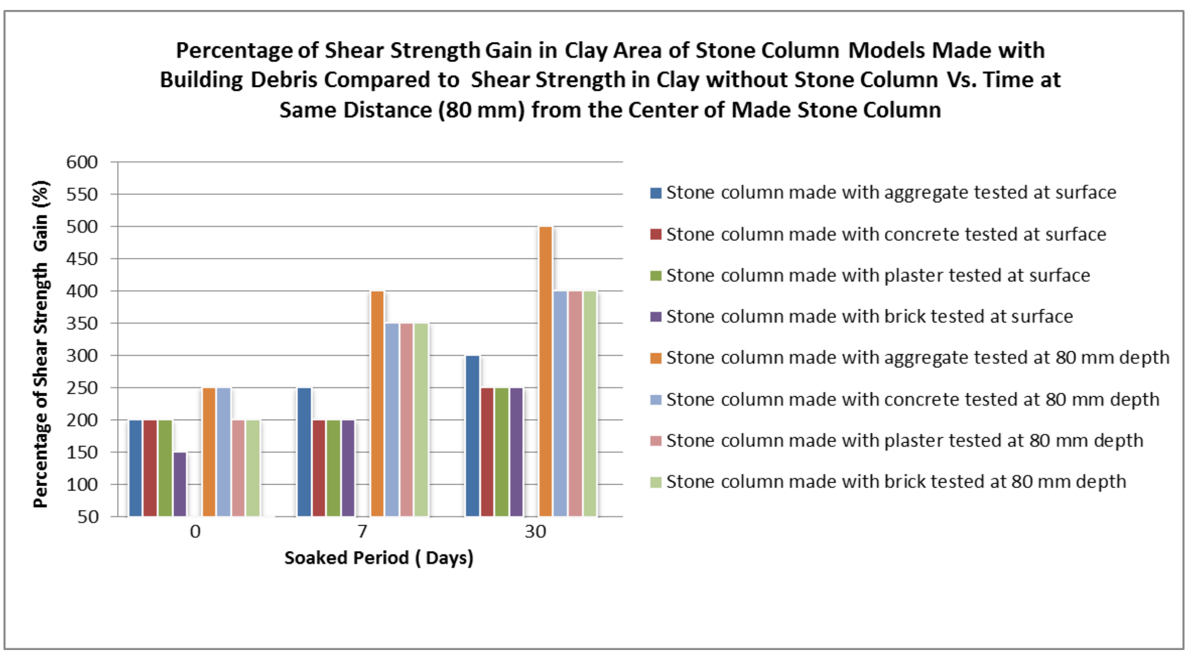
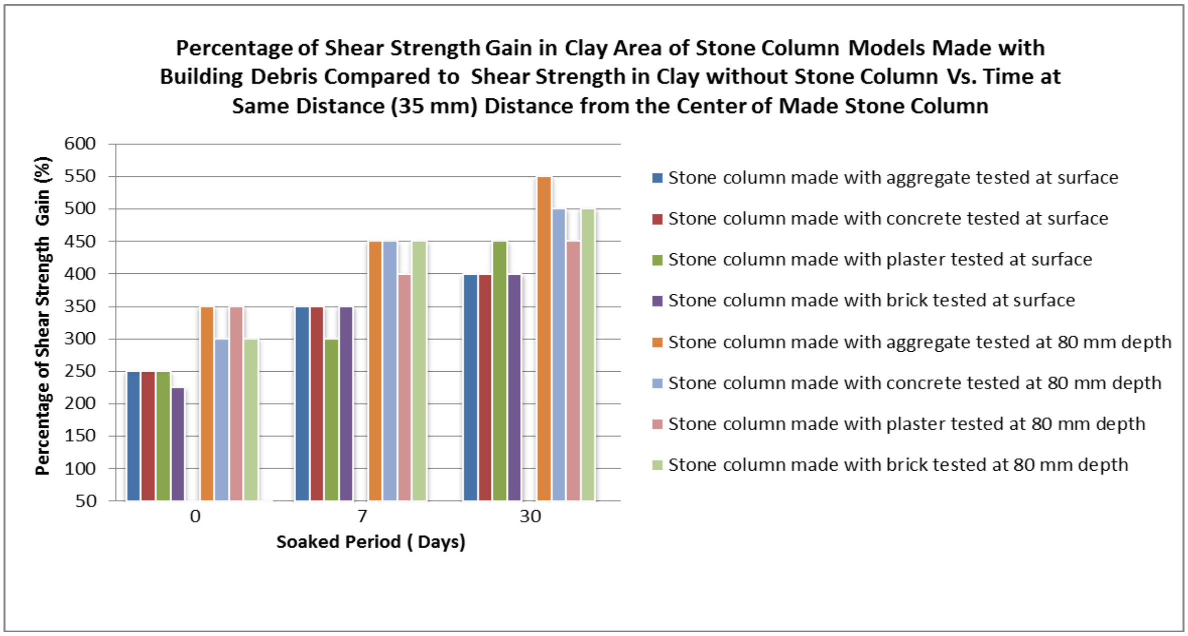


Figure 6.5:Graphical representation of undrained shear strength in clay surrounding of different stone column models and time relationship at 35 mm distance from the centre of stone column

Then, graphical representationsof percentage of shear strength gain of building debris compared to shear strength of clay Vs. time at same distance (80 mm & 35 mm) from the center of made stone columnbut different tested depth are given in Figure 6.6 (a) and (b).



(a)

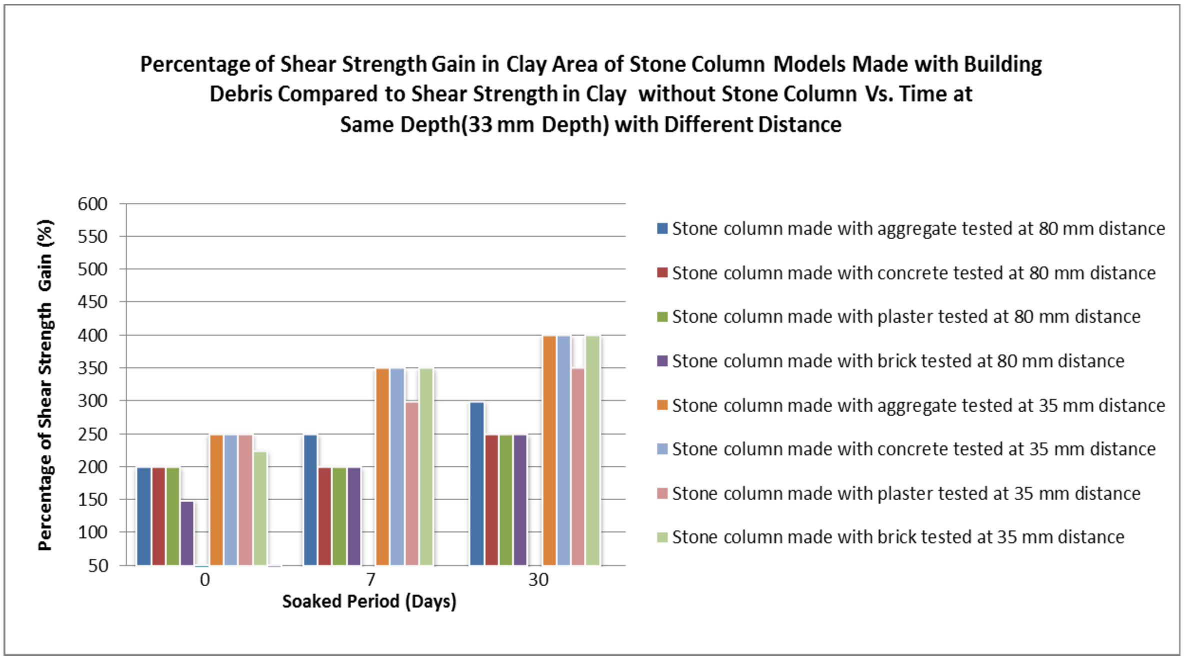


(b)

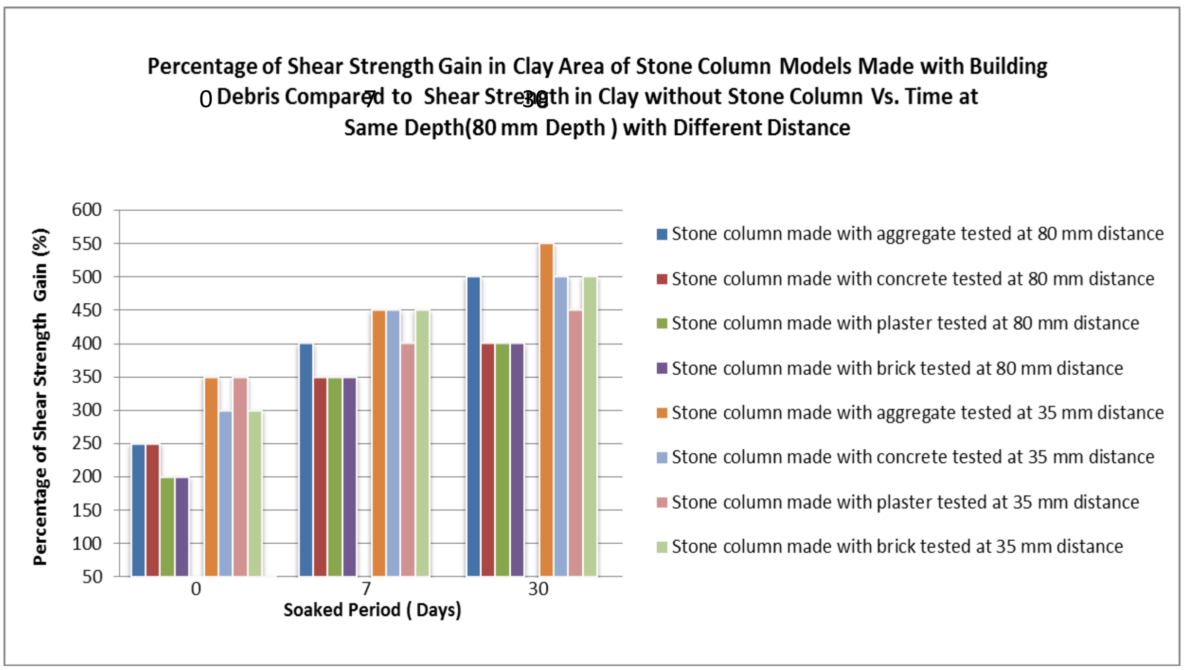
Figure 6.6:Graphical representation of percentage of shear strength gain in clay surrounding of stone column models made with different building debris compared to shear strength in clay without stone column Vs. time at same distance from the centre of made stone column models

(a) From 80 mm Distance; (b) From 35 mm Distance

Also, graphical representationsof percentage of shear strength gain of building debris compared to shear strength of clay Vs. time at same tested depth (33 mm & 80 mm without blade height) from the surface of made stone columnbut different tested distance from the center of made stone column are given in Figure 6.7(a) and (b).



(a)



(b)

Figure 6.7:Graphical representation of percentage of shear strength gain in clay surrounding of stone column models made with different building debris compared to shear strength in clay without stone column Vs. time at same depth level from the surface of made stone column models

(a) From 33 mm Depth; (b) From 80 mm Depth

Finally it is observed that the undrained shear strength of clay without stone column is very low and it is clearly visible that the shear strength has increased more than 50% of unimproved clay with stone column construction in clay area. Thus, shear strength at high depth and closer to stone column with time has become higher values than others because particles are much closer in that area due to horizontal and vertical load from made stone column. And also shear strength of surrounding clay of made stone column models has improved with time because of acceleration of consolidation due to top and bottom drains of sand layers. Therefore a strong recommendation could be made to use crushed concrete chips in stone column construction instead aggregates.

CHAPTER 7

CONCLUSION

Stone columns improve the bearing capacity of soft clayey soils by introducing a good drainage path and bearing the load through friction. The hardness and porosity of compacted stones make it ideal as granular column material. Since building debris (concrete, plaster and bricks) are not costly, easily available and possess similar properties as stone, the utility of building debris as replacement of stone in granular columns was studied in this work.

Based on the requirements for aggregate material for stone column construction as per Standard Specification of Construction & Maintenance (SSCM) of Roads & Bridges and Gamble Durability Classification, (Goodman, 1980) for durability of construction material, following conclusions were drawn from this study.

- Bricks and bricks with mortar were weak in durability and very low compressive strength(0.08 kN at 35 mm settlement), as well as low shear strength value compared to aggregates' values, thus not suitable as a stone column fill material.
- Plaster showed medium durability and medium-low in compressive strength(0.12 kN at 35 mm settlement), as well as medium shear strength value compared to aggregates' values, thus cannot be used when in high load bearing situations.
- Concrete showed medium high durability and high compressive strength (0.18 kN at 35 mm settlement) in comparison to the other tested materials. Still the values were lower than standard stone column fill material (i.e. aggregates) and also clearly observed that model stone columns with aggregates and concrete particles separately, both provided relatively same results in shear strength at different distance and depth. Also concrete debris provided more than 70% of efficiency related to ultimate load carrying capacity of aggregate.
- Thus, shear strength of clay had improved with construction of stone column with concrete and it had become higher values with higher depth and closer to the stone column.

It was also clearly observed that model stone columns with aggregates and concrete particles separately, both provided relatively same results in shear strength, at different distance and depth and satisfied criteria for durability, AIV,ACV & LAAV and more than 70% of efficiency related to ultimate load carrying capacity of aggregate. Therefore, it is proposed to utilize concrete particles in stone column construction in place of aggregates especially in situations where excessive concrete demolition waste is available in Sri Lanka.

Although stones columns have been used in Katunayake Expressway Project in Sri Lanka, it is a relatively new method and detail analysis has to be done on the size and percentage of concrete building debris depending on the properties of construction debris, dimension of stone column, type of clay surrounding, types of stone column and type of usage in road construction. The required laboratory and field tests are proposed based on ICTAD publication, SSCM of Roads and Bridges to carry out prior to site construction in order to ensure the quality of works.

A reduction in the amount of stone aggregate used in the stone columns will lead to a reduction in the cost of stone columns as well as better utilization of waste material and preservation of natural resources. Thus, use of particles of concrete debris has proved to offer a cost-effective and environmentally friendly technique. However, the above results are obtained from model tests conducted in the laboratory; hence the results and recommendations need to be verified based on full-scale tests conducted in-situ, to develop design guidelines before implementing the use of concrete debris in the field.

According to the results obtained from the laboratory tests, construction debris shall be prepared at the site. Field trial test shall be carried out before the construction work to determine the effective thickness of compaction layer, dimensions of stone column, size of debris particles and suitability according to surrounding clay materials. Site construction shall be carried out as per the results of field trial test. Additionally, the long-term durability and sustainability of stone columns made of concrete debris in different ground water conditions need to be investigated, before using them in real applications.

Since stone column construction is relatively new method in Sri Lanka, it is proposed to mix only different percentage of each building debris separately and also aggregate with different percentage of each building debris and carryout above tests methodology and comparethe results with them for future reference.

CHAPTER 8

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Appendix1

Observations of Specific Gravity Test

Table A1.1:Specific Gravity Test of Clayey Sample

No	Description	Sample 1	Sample 2
1	Temperature in ^0C	31	31
2	Weight of bottle (W_1) in g	18.57	18.50
3	Weight of bottle + Dry clayey soil (W_2) in g	28.57	28.50
4	Weight of bottle + clayey soil + water (W_3) in g	90.88	90.20
5	Weight of bottle + Water (W_4) in g	84.74	84.12

Specimen Calculations of Specific Gravity Test

From Table A1.1, set of readings for sample 1;

Specific gravity (G) of the clayey soil = $(W_2 – W_1) / [(W_4 – W_1) - (W_3 - W_2)]$

= (28.57 – 18.57) / [(84.74 – 18.57) – (90.88 – 28.57)

= 2.59

Similarly;

From Table A1.1, set of readings for sample 2;

Specific gravity (G) of the clayey soil = $(W_2 – W_1) / [(W_4 – W_1) - (W_3 - W_2)]$

= (28.50 – 18.50) / [(84.12 – 18.50) – (90.20 – 28.50)

= 2.55

Average specific gravity (G) of the clayey soil = (2.59 + 2.55)/ 2

= 2.57

Appendix2

Observations of Atterberg Limit Test

Table A2.1:Liquid Limit Test (Penetration method) of Clayey Sample

Container No.	CP4	B2	P6	9
Penetration (mm)	10.6	18.2	23.5	28.6
Weight of Container (g)	7.00	5.07	20.25	9.21
Weight of Water +Container(g)	22.37	17.73	32.06	21.15
Weight of Dry Clayey Soil+ Container (g)	16.44	12.50	27.09	16.04

Table A2.2:Plastic Limit Test of Clayey Sample

Container No.	24F	T
Weight of Wet Clayey Soil + Container (g)	24.73	20.65
Weight of Dry Clayey Soil + Container (g)	19.86	15.35
Weight of Container (g)	10.26	5.07

Specimen Calculations of Atterberg Limit Test

From Table A2.1, first set of readings for sample CP4;

Weight of Wet Clayey Soil + Container (g) = 22.37

Weight of Dry Clayey Soil + Container (g) = 16.44

Weight of Container (g) = 7.00

Weight of Water (g) = 5.93

Weight of Dry Clayey Soil (g) = 9.44

Moisture Content (%) = (5.93 / 9.44) x 100%

= 62.8 %

From Table A2.2, first set of readings for sample 24F;

Weight of Wet Clayey Soil + Container (g) = 24.73

Weight of Dry Clayey Soil + Container (g) = 19.86

Weight of Container (g) = 10.26

Weight of Water (g) = 4.87

Weight of Dry Clayey Soil (g) = 9.6

Moisture Content (%) = (4.87/9.6) x 100%

= 50.7%

Similarly, from Table A2.2, calculation for moisture content of B2, P6, 9 and 18samples can be done and tabulated below;

Table A2.3: Moisture Content Results of Liquid Limit Test

Container No.	CP4	B2	P6	9
Weight of Water (g)	5.93	5.23	4.97	5.11
Weight of Dry Clayey Soil (g)	9.44	7.43	6.84	6.83
Moisture Content (%)	62.8	70.5	72.6	74.8

Penetration method for obtaining liquid limit of sample (Moisture content Vs. Penetration) is shown graphically in Figure A2.1.

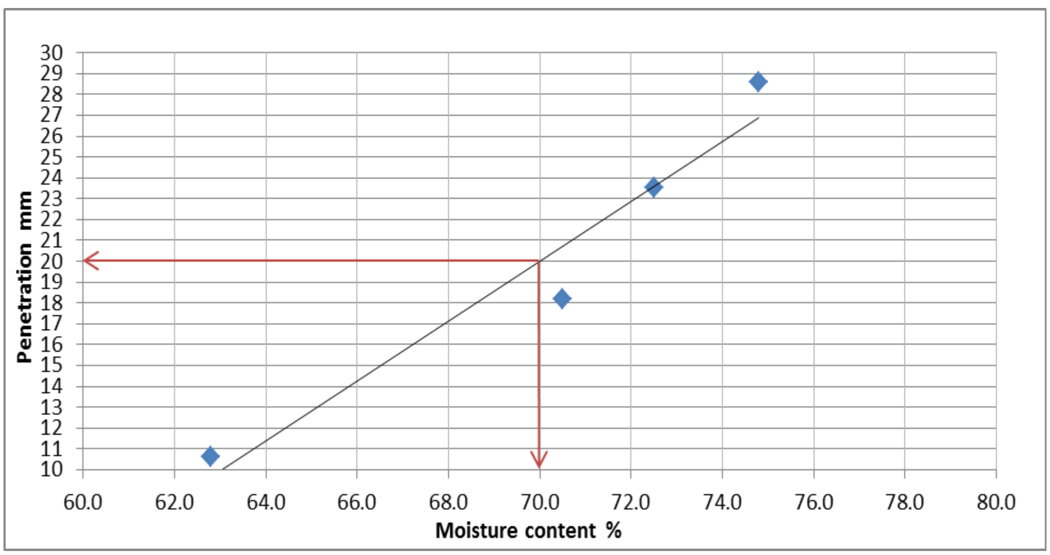

Figure A2.1: Graph of Moisture Content vs. Penetration

Then;

Liquid Limit of clayey sample (%) = 70

Table A2.4: Moisture Content Results of Plastic Limit Test

Container No.	24F	T
Weight of Water (g)	4.87	5.3
Weight of Dry Clayey Soil (g)	9.6	10.28
Moisture Content (%)	50.7	51.6

Then;

Water content of first sample (%) = 50.7

Water content of second sample (%) =51.6

Plastic Limit of Clayey sample (%) = (50.7+ 51.6)/2

= 51.2

Therefore;

Plasticity Indexof Clayey sample(%) = 70.0 - 51.2

= 18.8

Appendix3

Observations of Moisture Content Test

Table A3.1:Moisture Content Test of Clayey Model

No	Description	Top	Middle	Bottom
1	Weight of empty container (W_1) in g	5.07	7.00	20.25
2	Weight of container+ wet clayey soil (W_2) in g	24.13	23.47	39.23
3	Weight of container+ dry clayey soil (W_3) in g	16.98	17.27	32.13

Specimen Calculations of Moisture Content Test

From Table A3.1, set of readings for top sample of clayey model;

Moisture content (w) of topsample of clayey model = $[W_2 - W_3] / [W_3 - W_1] * 100\%$

$= [24.13 - 16.98] / [16.98 - 5.07] * 100\%$

$= 60\%$

Similarly, from Table A3.1, calculation of moisture content of middle and bottom samples of clayey model can be done and results can be as follows;

- Moisture content (w) of middle sample of clayey model = 60.3%
- Moisture content (w) of bottom sample of clayey model = 59.8%
    - Average moisture content (w) of clayey model = (60 + 60.3 + 59.8)/ 3

      = 60.03%

Appendix4

Observations of Hydrometer Analysis Test

Table A4.1: Hydrometer Analysis Test of Clayey Sample

Weight of Sample (g) = 50 Meniscus Correction C_m = 0.5 Dispersing Agent Correction C_d = 2.0		G_s = 2.57 K = 0.01246 a = 1.015
Temperature (^{0}C)	Time (min)	$R_H^{'}$
30	0.5	46.9
30	1	45.2
30	2	39.7
30	4	34.4
30	8	31.8
30	15	30.7
30	30	28.3
30	60	26.6
30	120	26.0
30	240	25.4

Specimen Calculations of Hydrometer Analysis Test

From Table A4.1, % Finer and Diameter of particle after 0.5 minute can be calculated as follows;

Hydrometer reading $R_H^{'}$ = 46.9

After Meniscus correction R_H = 46.9 + 0.5 (C_m = 0.5 g/l)

= 47.4

Value of L from table provided with the hydrometer = 8.5

Diameter of a particle from Stroke's law D $= K\sqrt{\frac{L}{t}}$

(K = 0.01246 at 30^{0}C temperature and 2.57 specific gravity of clay particles according to table provided with the hydrometer)

$$= 0.01246\sqrt{\frac{8.5}{0.5}}$$

$$= 0.05137 \text{ mm} \rightarrow 0.051 \text{ mm}$$

After dispersing agent correction $R = R_H - C_d = R_H^{'} - (C_d - C_m)$

$= 47.4 – 2$ ($C_d = 2$ g/l)

$= 45.4$

Percentage of soil remaining in suspension $P = \frac{Ra}{W} \times 100\,\%$

(a= 1.015 at 2.57 specific gravity of soil particles and taken as 1.0)

$$= \frac{45.4 \times 1.0}{50} \times 100\%$$

% Finer = 90.7 %

Similarly, from Table A4.1, calculation for % Finer and Diameter of particle can be obtained after 1 min, 2 min, 4 min, 8 min, 15 min, 30 min, 1 hr, 2 hrs and 4 hrs respectively and tabulated below;

Table A4.2:Results of Hydrometer Analysis Test

Time (min)	$R_H^{'}$	$R_H = R_H^{'} + C_m$	L (cm)	**L/t**	D (mm)	$R = R_H - C_d$	% Finer $P = \frac{Ra}{W} \times 100$
0.5	46.9	47.4	8.5	17	0.051	45.4	90.7
1	45.2	45.7	8.8	8.8	0.044	43.7	87.4
2	39.7	40.2	9.7	4.85	0.032	38.2	76.4
4	34.4	34.9	10.6	2.65	0.022	32.9	65.8
8	31.8	32.3	11.1	1.3875	0.015	30.3	60.5
15	30.7	31.2	11.2	0.7467	0.011	29.2	58.4
30	29.9	29.4	11.5	0.3833	0.008	27.4	54.8
60	29.3	28.8	11.9	0.1983	0.006	26.8	53.5
120	29.0	28.5	11.95	0.099583	0.004	26.5	52.9
240	29.2	28.2	12.0	0.05	0.002	26.2	52.3

Finally, Graph of Percentage of Finer vs. Particle Size (Diameter of particle) can be drawn.

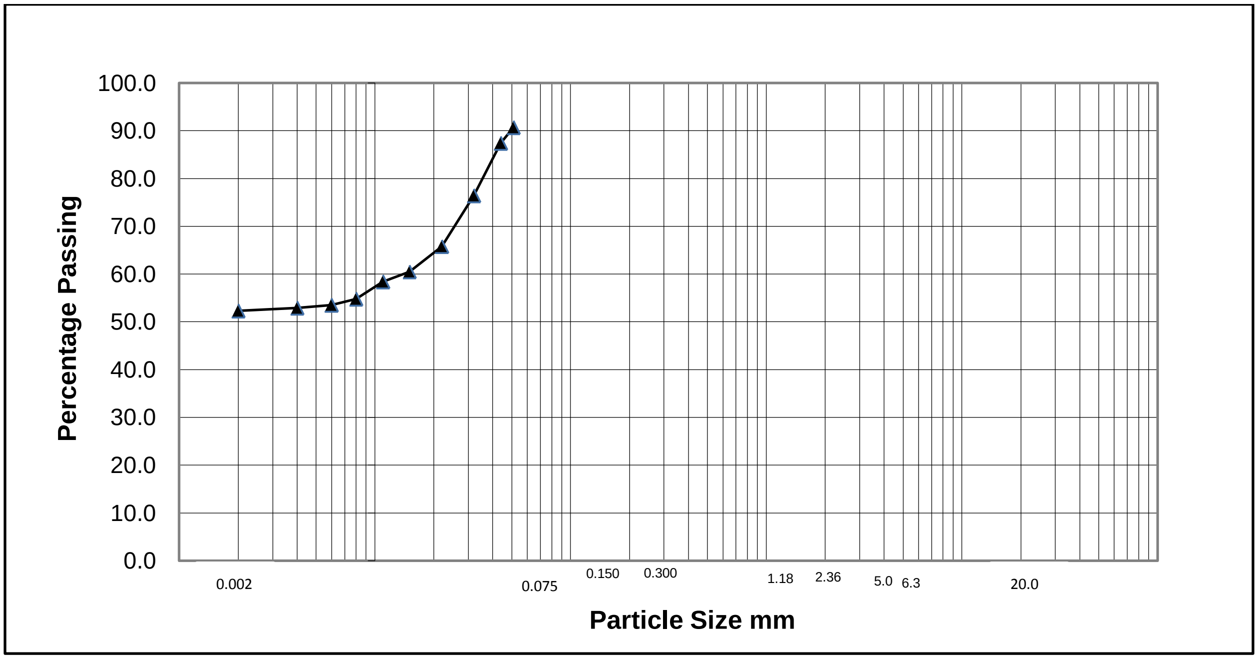

Figure A4.1:Graph of Percentage of Finer vs. Particle Size (Diameter of particle)

From above details of hydrometer analysis test, activity of soil(A) can be calculated as follows;

Activity of soil(A) = Plasticity Index / Percent of clay-sized particles (less than 2 µm)

= 18.8 / 52.3

= 0.36

Appendix 5

Observations of Standard Proctor Compaction Test

- Mass of the mould = 1.954 kg
- Volume of the mould = 944* 10^{-6} kg m^{-3}

Table A5.1: Standard Proctor Compaction Test of Clayey Sample

No	Mass of the mould + soil (g)	Mass of empty can (g)		Mass of wet soil + can (g)	
		Top sample	Bottom sample	Top sample	Bottom sample
1	3.493	9.83	9.17	141.35	165.04
2	3.550	9.54	9.34	142.70	167.45
3	3.648	10.24	9.80	150.80	158.50
4	3.749	10.58	10.50	134.36	140.60
5	3.757	9.56	10.80	144.56	145.70
6	3.751	11.78	9.45	157.80	160.46
7	3.671	10.70	11.96	147.30	150.40

Specimen Calculations of Standard Proctor Compaction Test

For first sample;

Mass of the soil = (Mass of the mould + soil) – (Mass of the mould)

= 3.493 kg -1.954 kg

= 1.539 kg

Bulk density = (Mass of the soil) / (Volume of the mould)

= 1.539 / 944* 10^{-6} kg m^{-3}

= 1629.98 kg m-3

Moisture content for top sample = (Mass of the wet soil + can) – (Mass of the dry soil + can) *100 / (Mass of the dry soil + can – Mass of the can)

= (141.35 - 124.93) * 100%

(124.93 – 9.83)

= 14.27 %

$$\text{Moisture content for bottom sample} = \frac{(165.04 - 145.47)}{(145.47 - 9.17)} * 100$$

$$= 14.36\%$$

$$\text{Average moisture content} = \frac{14.27 + 14.36}{2}$$

$$= 14.315\%$$

$$\text{Dry density} = \frac{\text{Bulk density}}{(1 + \text{moisture content})}$$

$$= \frac{1629.98}{1+0.14315}$$

$$= 1425.87 \text{ kg m}^{-3}$$

Similarly, from Table A5.1, calculation for Dry Density and Moisture Content of other samples can be obtained and tabulated below;

Table A5.2:Dry Density and Moisture Contentof Other Samples

Dry Density(kg m^{-3})	Moisture Content(%)
1425.87	14.315
1455.10	16.155
1513.62	18.555
1576.71	20.610
1556.45	22.730
1543.55	23.315
1437.24	26.585

Calculation for Dry density at 100% saturation (theoretical) to plot zero air void line can be done using equation - 02;

$$\rho_{d,\max} = \frac{G\rho_w}{1 + wG} \quad \text{Equation - 02} \longleftarrow$$

Where;

Specific Gravity (G) = 2.57

Density of Water (P_W) =1000 kg m^{-3}

w = Moisture Content

For sample 01,

$$\text{Dry density} = \frac{2.57 \times 1000}{(1 + 0.14315 \times 2.57)}$$

$$= 1878.79 \text{ kg m}^{-3}$$

Similarly values of dry density at 100% saturation can be calculated at different moisture content and tabulated below.

Table A5.3:Dry DensityValuesat 100% Saturation

Sample No	Moisture Content (%)	Dry Density (kg m^{-3})
01	14.315	1878.79
02	16.155	1815.99
03	18.555	1740.13
04	20.610	1680.07
05	22.730	1622.27
06	23.315	1607.05
07	26.585	1526.85

Appendix 6

Observations of Consolidation Test

Table A6.1:Clayey Sample Details

Test method		**BS 1377 : Part5 :1990 :3**		Date	30-Oct-16
		Particle density		2.57	Mg/m^3
DIMENSIONS		Initial specimen	Overall Change	Final Specimen	Specimen preparation method
Diameter	D mm	50.00		50.00	
Area	A mm^2	1963.50		1963.50	
Height	H mm	H_0 20.00	0.000	17.668	
Volume	V cm^3	39.27	0.00	39.27	

Table A6.2:Consolidation Test of Clayey Sample

Time(min)	Settlement Corresponding to the Different Loads			
	First Day 25 kPa	Second Day 50 kPa	Third Day 75 kPa	Fourth Day 100 kPa
0	0.000	1.160	1.666	2.020
0.25	0.212	1.202	1.690	2.040
0.5	0.236	1.210	1.698	2.044
1	0.262	1.218	1.712	2.050
2	0.298	1.232	1.721	2.058
4	0.360	1.252	1.742	2.068
8	0.436	1.286	1.764	2.082
15	0.502	1.328	1.790	2.100
30	0.626	1.392	1.822	2.128
60	0.772	1.462	1.858	2.162
120	0.938	1.532	1.898	2.208
240	1.052	1.586	1.936	2.240
1440	1.160	1.666	2.020	2.332

Specimen Calculations of Consolidation Test

Table A6.3:SpecimensCalculation

WEIGHINGS		Initial specimen				Final specimen	
			(a)		(b)		(c)
Wet soil + ~~ring~~ + tray	g		50.80		142.22		67.92
Dry soil + ~~ring~~ +tray	g		38.92		122.85		53.82
Ring + tray	g		19.23		90.73		30.53
Wet soil	g	m_o	31.57	m_o	51.49		37.39
Dry soil	g	m_d	19.69	m_d	32.12	m_d	23.3
Water	g		11.88		19.37		14.10
Moisture content (measured)	%		60.3		60.3		61.0
Moisture content (from trimmings)	%	w_o			60.3		
Density	Mg/m^3				1.31		0.95
Dry density	Mg/m^3				0.82		0.59
Voids ratio		e_o			2.143		3.35
Degree of saturation	%	S_o			72.4		46.9
Height of solids	H_S mm				6.36		4.07

Table A6.4:Consolidation TestCalculation

VOIDS RATIO					COMPRESSIBILITY			COEFFICIENT OF CONSOLIDATION			
Increment No.	Pressure P	Cumulative Compression (ΔH-Y)	Consolidated height H = Ho-(ΔH-Y)	Voids ratio $e = \frac{H - H_S}{H_S}$	Incremental height change $\delta H = H_1 - H_2$	Incremental pressure change δp	$m_v = \frac{\delta H}{\bar{H}_1} \cdot \frac{1000}{\delta p}$	t_{90}	$\bar{H} = 1/2(H_1+H_2)$	$c_V = \frac{0.111\bar{H}^2}{t_{90}}$	k $= c_v * m_v * \gamma_w$ $*10^{-7}$mm/s (Clay range)
N	kPa				mm	kPa	m^2/MN	min	mm	m^2/Year	
0	0	0	20.000	2.143							
L1	25.000	1.160	18.840	1.960	1.160	25.00	2.320	43.560	19.420	0.961	7
L2	50.000	1.666	18.334	1.881	0.506	25.00	1.074	88.360	18.587	0.434	1
L3	75.000	2.020	17.980	1.825	0.354	25.00	0.772	100.000	18.157	0.366	1
L4	100.000	2.332	17.668	1.776	0.312	25.00	0.694	141.610	17.824	0.249	1

According to above calculations, Consolidation test graphs of clayey sample can be drawn as follow.

Consolidation Test Graphs

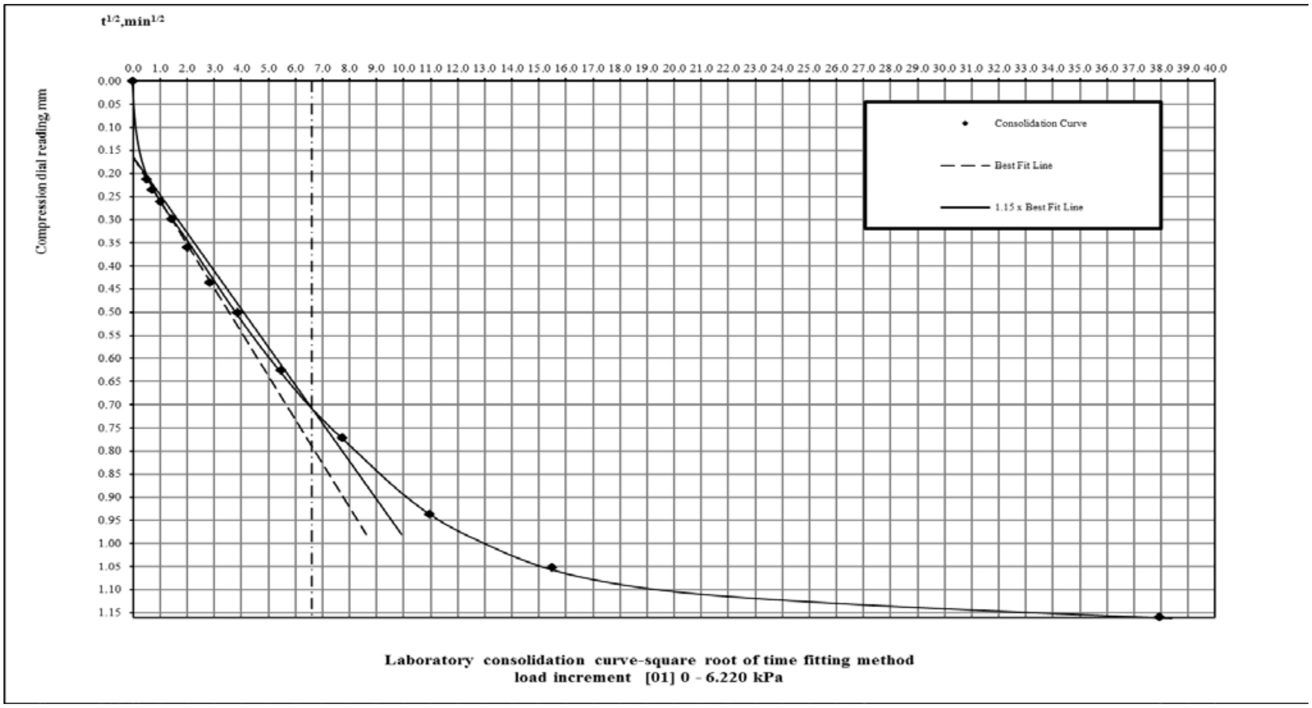

Figure A6.1: Consolidation Graph for 25 kPa Load

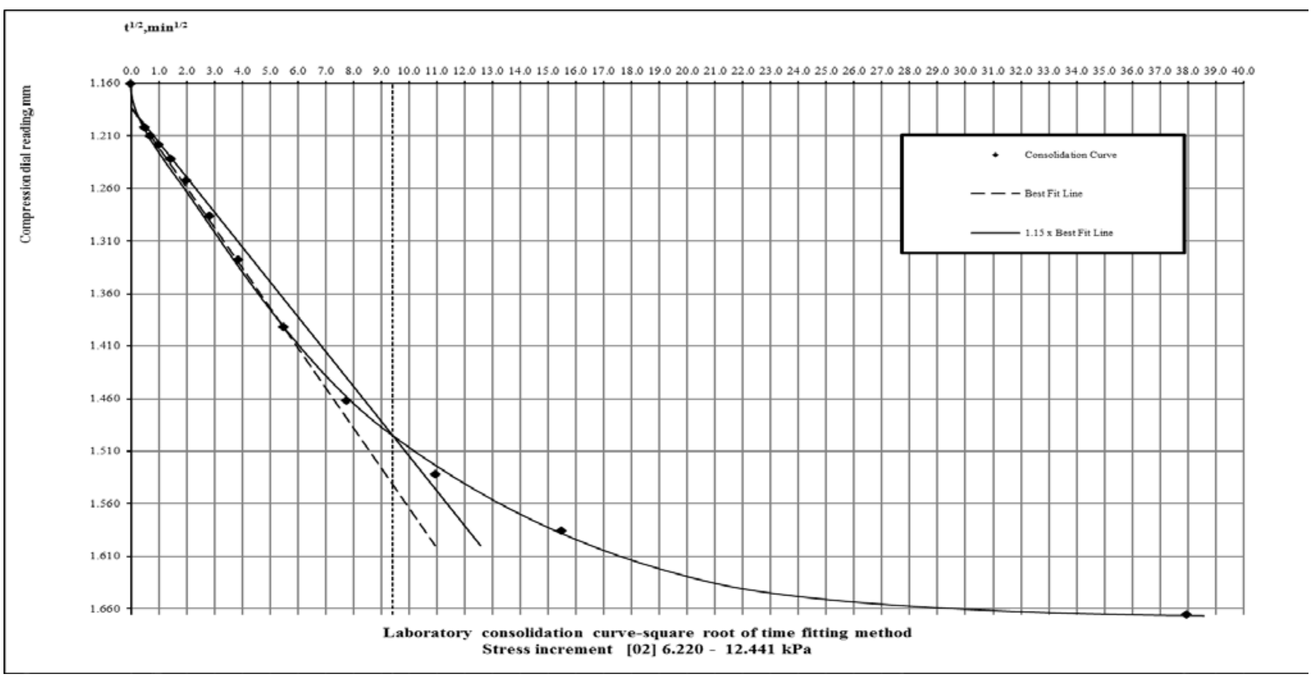

Figure A6.2: Consolidation Graph for 50 kPa Load

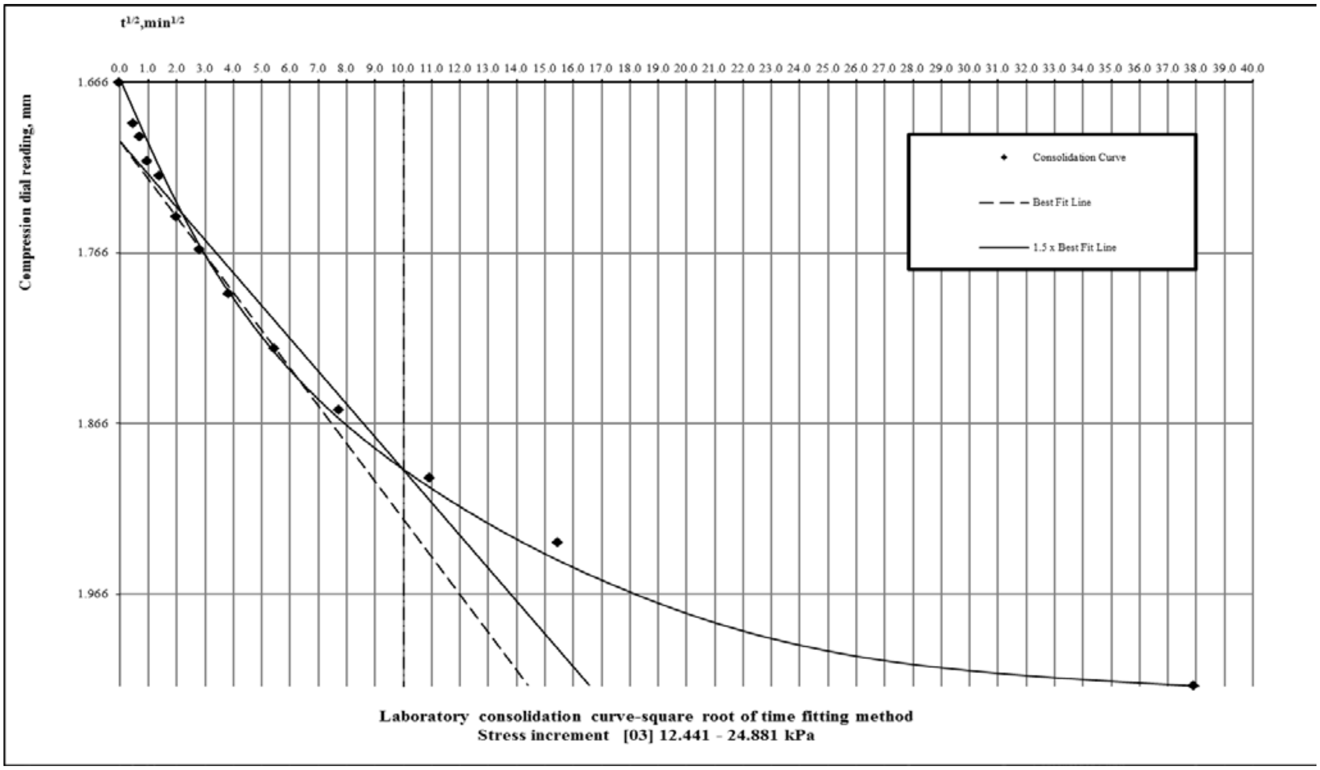

Figure A6.3: Consolidation Graph for 75 kPa Load

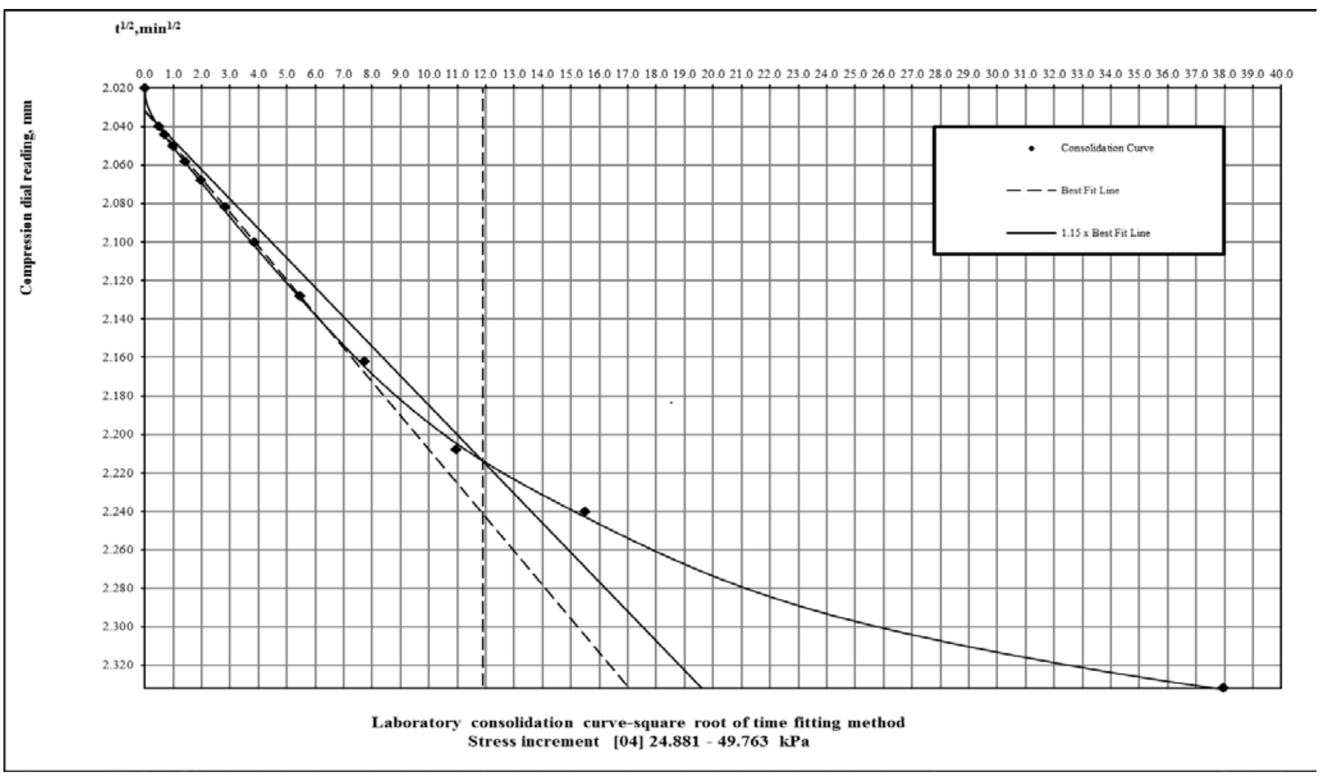

Figure A6.4: Consolidation Graph for 100 kPa Load

Appendix 7

Observations of Organic Content Test

- The mass of an empty, clean, and dry porcelain dish (M_P)(g)= 40.32
- The mass of the dish and soil specimen (M_{PDS})(g)= 49.73
- The mass of the dish containing the ash (burned soil) (M_{PA})(g) = 48.60

Specimen Calculations of Organic Content Test

The mass of the dry soil (M_D) = M_{PDS} - M_P
= 49.73 – 40.32
=9.41g

The mass of the ashed (burned) soil (M_A) = M_{PA} - M_P
= 48.60 - 40.32
= 8.28 g

The mass of organic matter(M_O) = M_D – M_A
= 9.41- 8.28
= 1.13 g

The organic matter (content) (*OM*) = (M_O/M_D)*100
= 1.13 / 9.41
= 12%

The organic matter (content) of given clayey sample is 12%.

Appendix 8

Observations of AIV, ACV and LAAV Tests

Table A8.1: Aggregate Impact Value Test of Concrete Debris

Test No.	1	2
Weight of sample (g)	248.6	248.2
Weight of sample passing 2.36 mm sieve after test (g)	78.8	75.2
Weight of sample retained on 2.36 mm sieve after test (g)	169.8	173.0

Table A8.2: Aggregate Impact Value Test of Aggregates

Test No.	1	2
Weight of sample (g)	303.4	303.4
Weight of sample passing 2.36 mm sieve after test (g)	74.7	77.0
Weight of sample retained on 2.36 mm sieve after test (g)	228.4	226.5

Specimen Calculation

From Table A8.1, 2nd set of readings,

Weight of sample in standard measure = 248.2 g

Weight of sample passing 2.36 mm sieve after test = 75.2 g

Weight of sample retained on 2.36 mm sieve after test = 158.3 g

Hence, Aggregate Impact Value of concrete debris = 30.3 %

Similarly,

From 1st set of readings, Aggregate Impact Value = 31.7 %

Hence, Average Aggregate Impact Value of concrete debris = 31 %

Note: Plaster debris sample was not be prepared for Aggregate Impact Value Test as it was broken into small particles when preparing sample.

Table A8.3: Aggregate Crushing Value Test of Concrete Debris

Test No.	1	2
Weight of sample in standard measure (g)	2181	2185
Weight of sample passing 2.36 mm sieve after test (g)	761	767
Weight of sample retained on 2.36 mm sieve after test (g)	1414	1410

Table A8.4: Aggregate Crushing Value Test of Aggregates

Test No.	1	2
Weight of sample in standard measure (g)	2185	2189
Weight of sample passing 2.36 mm sieve after test (g)	546	556
Weight of sample retained on 2.36 mm sieve after test (g)	1634	1629

Specimen Calculation

From Table A8.3, 2nd set of readings,

Weight of sample in standard measure = 2185 g

Weight of sample passing 2.36 mm sieve after test = 767 g

Weight of sample retained on 2.36 mm sieve after test = 1410 g

Hence, Aggregate Crushing Value of concrete debris = 767/2185 x 100

= 35.1 %

Similarly,

From 1st set of readings, Aggregate Crushing Value = 34.9 %

Hence, Average Aggregate Crushing Value of concrete debris = 35 %

Note: Plaster debris sample was not be prepared for Aggregate Crushing Value Test as it broken to small particles when preparing sample

Table A8.5: Los Angeles Abrasive Value Test of Concrete Debris

Test No.	**1**	**2**
Total Weight of sample (g)	5000	5000
Weight of sample passing 1.7 mm sieve after test (g)	2130	2070
Weight of sample retained on 1.7 mm sieve after test (g)	2870	2930

Table A8.6: Los Angeles Abrasive Value Test of Aggregates

Test No.	**1**	**2**
Total Weight of sample (g)	5000	5000
Weight of sample passing 1.7 mm sieve after test (g)	1596	1544
Weight of sample retained on 1.7 mm sieve after test (g)	3400	3449

Specimen Calculation

From Table A8.5, 2nd set of readings,

Total weight of sample (W1 g) = 5000 g

Weight retained on 1.7 mm sieve after rotation (W2 g) = 2699 g

Weight passing 1.7 mm sieve after rotation (W2 g) = 2271 g

Hence, Los Angeles Abrasion Value = 41.4 %

Similarly,

From 1st set of readings, Los Angeles Abrasion Value = 42.6 %

Hence, Average Los Angeles Abrasion Value of concrete debris = 42 %

Note: Plaster debris sample was not be prepared for Los Angeles Abrasive Value Test as it broken to small particles when preparing sample.

Appendix 9

Observations of Slake Durability Test

Table A9.1: Slake Durability Test of Brick Debris

Test date	Initial	After one month	After two months
Weight of sample, dry weight basis(g)	500	500	500
Weight retained after 1st cycle, dry weight basis(g)	416	413	412
Weight retained after 2nd cycle, dry weight basis(g)	357	353	346

Table A9.2: Slake Durability Test of Concrete Debris

Test date	Initial	After one month	After two months
Weight of sample, dry weight basis(g)	500	500	500
Weight retained after 1st cycle, dry weight basis(g)	479	481	485
Weight retained after 2nd cycle, dry weight basis(g)	464	470	477

Table A9.3: Slake Durability Test of Plaster Debris

Test date	Initial	After one month	After two months
Weight of sample, dry weight basis(g)	500	500	500
Weight retained after 1st cycle, dry weight basis(g)	460	481	479
Weight retained after 2nd cycle, dry weight basis(g)	459	465	467

Specimen Calculations of Slake Durability Test

From Table A9.1, Initial set of readings;

Weight of sample,dry weight basis (g) = 500 g

Weight retained after 1st cycle, dry weight basis (g) = 416 g

Hence, Slake Indexof Brick debris after 1st cycle =(416/500)x100 %

= 83.2 %

Weight retained after 2ndcycle, dry weight basis (g) = 357 g

Hence, Slake Index of Brick debris after 2ndcycle = (357/500)x100%

=71.4 %

Similarly;

From after one month set of readings,

Slake Index of Brick debrisafter 1st cycle=82.6 %
Slake Index of Brick debris after 2nd cycle = 70.6 %

From after two months set of readings,

Slake Index of Brick debrisafter 1st cycle= 82.4 %
Slake Index of Brick debris after 2nd cycle = 69.2 %

Similarly, calculation was done for concrete block debris and cement plaster separately and values are tabulated below;

Table A9.4: Initial Values of Slake Durability Test of Different Type of Debris

Sample name	Initial weight,dry weight basis (g)	Weight retained after 1st cycle, dry weight basis (g)	% Weight retained after 1st cycle, dry weight basis	Weight retained after 2nd cycle, dry weight basis (g)	% Weight retained after 2nd cycle, dry weight basis
Concrete	500	479	95.8	464	92.8
Plaster	500	460	92.0	459	91.8
Brick	500	416	83.2	357	71.4

Table A9.5:Values of Slake Durability Test of Different Type of Debris after One Month

Sample name	Initial weight,dry weight basis (g)	Weight retained after 1st cycle, dry weight basis (g)	% Weight retained after 1st cycle, dry weight basis	Weight retained after 2nd cycle, dry weight basis (g)	% Weight retained after 2nd cycle, dry weight basis
Concrete	500	481	96.2	470	94.0
Plaster	500	481	96.2	465	93.0
Brick	500	413	82.6	353	70.6

Table A9.6:Values of Slake Durability Test of Different Type of Debrisafter Two Months

Sample name	Initial weight,dry weight basis (g)	Weight retained after 1st cycle, dry weight basis (g)	% Weight retained after 1st cycle, dry weight basis	Weight retained after 2nd cycle, dry weight basis (g)	% Weight retained after 2nd cycle, dry weight basis
Concrete	500	485	97.0	477	95.4
Plaster	500	479	95.8	467	93.4
Brick	500	412	82.4	346	69.2

These valuescan be shown graphically in Figure A9.1, FigureA9.2 and FigureA9.3.

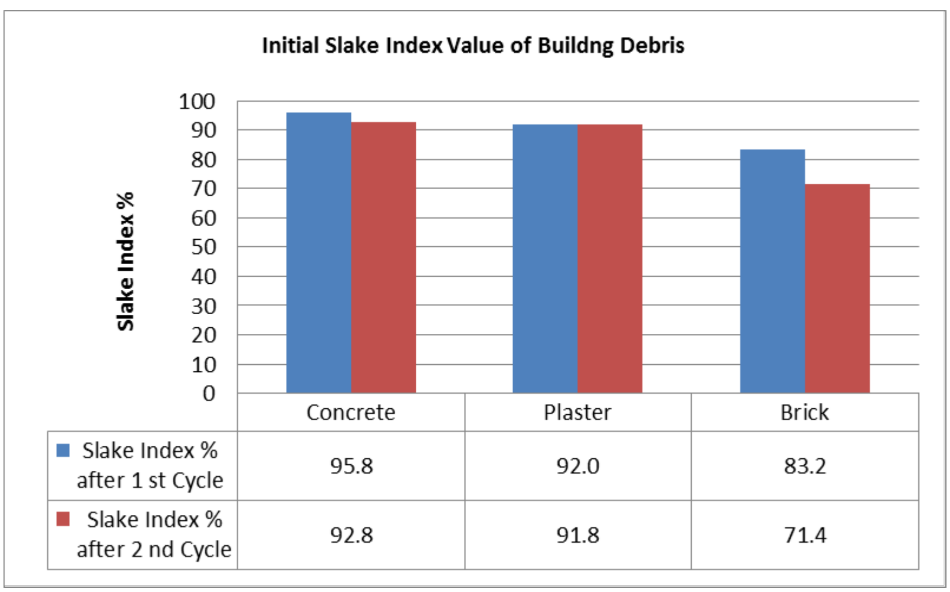

Figure A9.1:Initial Slake Index Values of Different Type of Debris

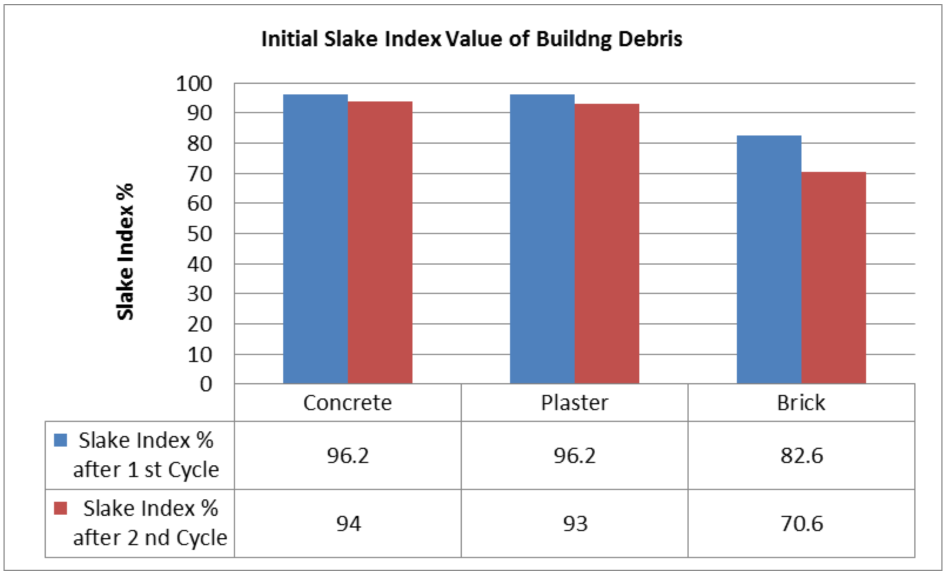

Figure A9.2:Slake Index Values of Different Type of Debris after One Month

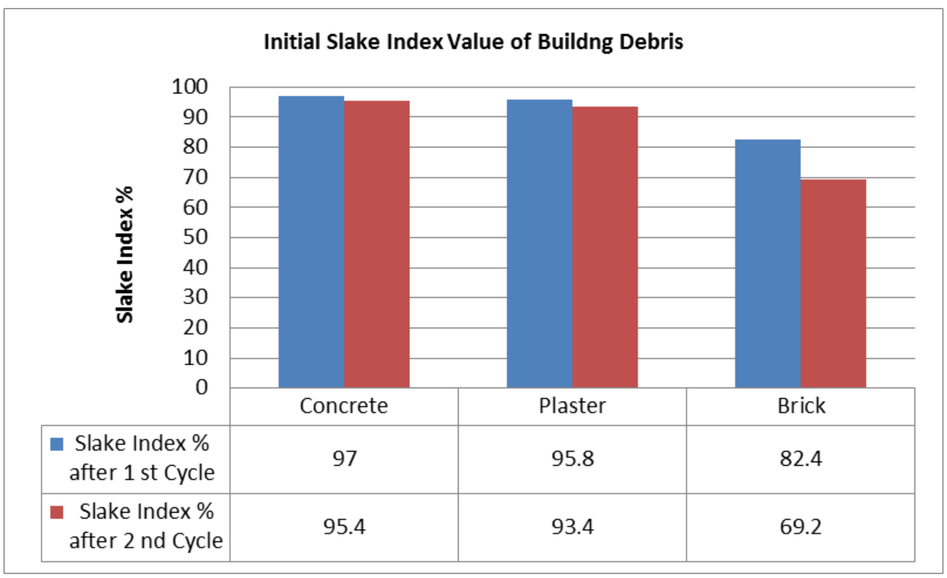

Figure A9.3:Slake Index Values of Different Type of Debris after Two Months

Appendix 10

Observations of Compressive Strength Test

Table A10.1:Compressive Strength Test of Concrete

Penetration of Plunger (cm)	Dial Reading of proving ring
0	0
0.125	1
0.25	2
0.375	2
0.5	3
0.625	3
0.75	3
0.875	3
1	4
1.125	4
1.25	4
1.375	4
1.5	4
1.625	4
1.75	5
1.875	5
2	5
2.125	5
2.25	5
2.375	5
2.5	5
2.625	5
2.875	6
3	6
3.125	6
3.25	6
3.375	6
3.5	7
3.625	7
3.75	7
3.875	7
4	7
4.125	7
4.25	8
4.375	8
4.5	8
4.625	8
4.75	8
4.875	9
5	9
5.125	9
5.25	9
5.375	10
5.5	10
5.625	10
5.75	10
5.875	10
6	10

Table A10.2:Compressive Strength Test of Aggregate

Penetration of Plunger (cm)	Dial Reading of proving ring
0	0
0.125	1
0.25	1
0.375	2
0.5	3
0.625	4
0.75	5
0.875	6
1	6
1.125	6
1.25	7
1.375	7
1.5	7
1.625	7
1.75	7
1.875	8
2	8
2.125	8
2.25	8
2.375	8
2.5	8
2.625	9
2.75	9
2.875	9
3	9
3.125	9
3.25	10
3.375	10
3.5	10
3.625	10
3.75	10
3.875	10
4	11
4.125	11
4.25	11
4.375	11
4.5	11
4.625	12
4.75	12
4.875	12
5	12
5.125	12
5.25	12
5.375	12
5.5	13
5.625	13
5.75	13
5.875	13
6	13

Table A10.3:Compressive Strength Test of Brick

Penetration of Plunger (cm)	Dial Reading of proving ring
0	0
0.125	1
0.25	1
0.375	1
0.5	1
0.625	1
0.75	1
0.875	1
1	1
1.125	1
1.25	1
1.375	1
1.5	1
1.625	1
1.75	2
1.875	2
2	2
2.125	2
2.25	2
2.375	2
2.5	2
2.625	2
2.75	2
2.875	2
3	2
3.125	2
3.25	3
3.375	3
3.5	3
3.625	3
3.75	3
3.875	3
4	3
4.125	3
4.25	3
4.375	3
4.5	3
4.625	3
4.75	3
4.875	4
5	4
5.125	4
5.25	4
5.375	4
5.5	4
5.625	4
5.75	4
5.875	4
6	4

Table A10.4:Compressive Strength Test of Plaster

Penetration of Plunger (cm)	Dial Reading of proving ring
0	0
0.125	1
0.25	1
0.375	1
0.5	2
0.625	2
0.75	2
0.875	3
1	3
1.125	3
1.25	3
1.375	3
1.5	3
1.625	3
1.75	4
1.875	4
2	4
2.125	4
2.25	4
2.375	4
2.5	4
2.625	4
2.75	5
2.875	5
3	5
3.125	5
3.25	5
3.375	5
3.5	5
3.625	5
3.75	5
3.875	5
4	5
4.125	5
4.25	5
4.375	5
4.5	5
4.625	6
4.75	6
4.875	6
5	6
5.125	6
5.25	6
5.375	6
5.5	6
5.625	6
5.75	6
5.875	6
6	6

Table A10.5:Compressive Strength Test of Clay

Penetration of Plunger (cm)	Dial Reading of proving ring
0	0
0.125	1
0.25	2
0.375	2
0.5	3
0.625	3
0.75	3
0.875	3
1	4
1.125	4
1.25	4
1.375	4
1.5	4
1.625	4
1.75	5
1.875	5
2	5
2.125	5
2.25	6
2.375	6
2.5	6
2.625	6
2.75	6
2.875	6
3	6
3.125	7
3.25	7
3.375	7
3.5	7
3.625	7
3.75	7
3.875	8
4	8
4.125	8
4.25	8
4.375	8
4.5	8
4.625	8
4.75	8
4.875	9
5	9
5.125	9
5.25	9
5.375	9
5.5	9
5.625	9
5.75	10
5.875	10
6	10

Appendix 11

Values of Compressive Strength Test

Table A11.1: Compressive Strength Values of Concrete

Penetration of Plunger (cm)	Dial Reading of proving ring	Actual Load (kN)
0	0	0.03
0.125	1	0.05
0.25	2	0.07
0.375	2	0.07
0.5	3	0.09
0.625	3	0.09
0.75	3	0.09
0.875	3	0.09
1	4	0.11
1.125	4	0.11
1.25	4	0.11
1.375	4	0.11
1.5	4	0.11
1.625	4	0.11
1.75	5	0.13
1.875	5	0.13
2	5	0.13
2.125	5	0.13
2.25	5	0.13
2.375	5	0.13
2.5	5	0.13
2.625	5	0.13
2.875	6	0.15
3	6	0.15
3.125	6	0.15
3.25	6	0.15
3.375	6	0.15
3.5	7	0.17
3.625	7	0.17
3.75	7	0.17
3.875	7	0.17
4	7	0.17
4.125	7	0.17
4.25	8	0.19
4.375	8	0.19
4.5	8	0.19
4.625	8	0.19
4.75	8	0.19
4.875	9	0.21
5	9	0.21
5.125	9	0.21
5.25	9	0.21
5.375	10	0.23
5.5	10	0.23
5.625	10	0.23
5.75	10	0.23
5.875	10	0.23
6	10	0.23

Table A11.2: Compressive Strength Values of Aggregate

Penetration of Plunger (cm)	Dial Reading of proving ring	Actual Load KN
0	0	0.03
0.125	1	0.05
0.25	1	0.05
0.375	2	0.07
0.5	3	0.09
0.625	4	0.11
0.75	5	0.13
0.875	6	0.15
1	6	0.15
1.125	6	0.15
1.25	7	0.17
1.375	7	0.17
1.5	7	0.17
1.625	7	0.17
1.75	7	0.17
1.875	8	0.19
2	8	0.19
2.125	8	0.19
2.25	8	0.19
2.375	8	0.19
2.5	8	0.19
2.625	9	0.21
2.75	9	0.21
2.875	9	0.21
3	9	0.21
3.125	9	0.21
3.25	10	0.23
3.375	10	0.23
3.5	10	0.23
3.625	10	0.23
3.75	10	0.23
3.875	10	0.23
4	11	0.25
4.125	11	0.25
4.25	11	0.25
4.375	11	0.25
4.5	11	0.25
4.625	12	0.27
4.75	12	0.27
4.875	12	0.27
5	12	0.27
5.125	12	0.27
5.25	12	0.27
5.375	12	0.27
5.5	13	0.29
5.625	13	0.29
5.75	13	0.29
5.875	13	0.29
6	13	0.29

Table A11.3: Compressive Strength Values of Brick

Penetration of Plunger (cm)	Dial Reading of proving ring	Actual Load KN
0	0	0.03
0.125	1	0.05
0.25	1	0.05
0.375	1	0.05
0.5	1	0.05
0.625	1	0.05
0.75	1	0.05
0.875	1	0.05
1	1	0.05
1.125	1	0.05
1.25	1	0.05
1.375	1	0.05
1.5	1	0.05
1.625	1	0.05
1.75	2	0.07
1.875	2	0.07
2	2	0.07
2.125	2	0.07
2.25	2	0.07
2.375	2	0.07
2.5	2	0.07
2.625	2	0.07
2.75	2	0.07
2.875	2	0.07
3	2	0.07
3.125	2	0.07
3.25	3	0.09
3.375	3	0.09
3.5	3	0.09
3.625	3	0.09
3.75	3	0.09
3.875	3	0.09
4	3	0.09
4.125	3	0.09
4.25	3	0.09
4.375	3	0.09
4.5	3	0.09
4.625	3	0.09
4.75	3	0.09
4.875	4	0.11
5	4	0.11
5.125	4	0.11
5.25	4	0.11
5.375	4	0.11
5.5	4	0.11
5.625	4	0.11
5.75	4	0.11
5.875	4	0.11
6	4	0.11

Table A11.4:Compressive Strength Values of Plaster

Penetration of Plunger (cm)	Dial Reading of proving ring	Actual Load KN
0	0	0.03
0.125	1	0.05
0.25	1	0.05
0.375	1	0.05
0.5	2	0.07
0.625	2	0.07
0.75	2	0.07
0.875	3	0.09
1	3	0.09
1.125	3	0.09
1.25	3	0.09
1.375	3	0.09
1.5	3	0.09
1.625	3	0.09
1.75	4	0.11
1.875	4	0.11
2	4	0.11
2.125	4	0.11
2.25	4	0.11
2.375	4	0.11
2.5	4	0.11
2.625	4	0.11
2.75	5	0.13
2.875	5	0.13
3	5	0.13
3.125	5	0.13
3.25	5	0.13
3.375	5	0.13
3.5	5	0.13
3.625	5	0.13
3.75	5	0.13
3.875	5	0.13
4	5	0.13
4.125	5	0.13
4.25	5	0.13
4.375	5	0.13
4.5	5	0.13
4.625	6	0.15
4.75	6	0.15
4.875	6	0.15
5	6	0.15
5.125	6	0.15
5.25	6	0.15
5.375	6	0.15
5.5	6	0.15
5.625	6	0.15
5.75	6	0.15
5.875	6	0.15
6	6	0.15

Table A11.5:Compressive Strength Values of Clay

Penetration of Plunger (cm)	Dial Reading of proving ring	Actual Load KN
0	0	0.03
0.125	1	0.05
0.25	2	0.07
0.375	2	0.07
0.5	3	0.09
0.625	3	0.09
0.75	3	0.09
0.875	3	0.09
1	4	0.11
1.125	4	0.11
1.25	4	0.11
1.375	4	0.11
1.5	4	0.11
1.625	4	0.11
1.75	5	0.13
1.875	5	0.13
2	5	0.13
2.125	5	0.13
2.25	6	0.15
2.375	6	0.15
2.5	6	0.15
2.625	6	0.15
2.75	6	0.15
2.875	6	0.15
3	6	0.15
3.125	7	0.17
3.25	7	0.17
3.375	7	0.17
3.5	7	0.17
3.625	7	0.17
3.75	7	0.17
3.875	8	0.19
4	8	0.19
4.125	8	0.19
4.25	8	0.19
4.375	8	0.19
4.5	8	0.19
4.625	8	0.19
4.75	8	0.19
4.875	9	0.21
5	9	0.21
5.125	9	0.21
5.25	9	0.21
5.375	9	0.21
5.5	9	0.21
5.625	9	0.21
5.75	10	0.23
5.875	10	0.23
6	10	0.23

Appendix 12

Observations of Vane Shear Test

Table A12.1: Vane Shear Test at 80 mm Distancefrom Center of Model Initially

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
80 mm from center of model	0	20	20	21	19	7.5
80 mm from center of model	80	26	24	23	22	11

Table A12.2:Vane Shear Test at 35 mm Distancefrom Center of Model Initially

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	24	24	24	23.5	12
35 mm from center of model	80	32	30	31	29	16

Table A12.3:Vane Shear Test at 80 mm Distancefrom Center of Model with Load after 07 Days Soaked Period

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
80 mm from center of model	0	26	21	22	20	10
80 mm from center of model	80	36	32	34	34	13

Table A12.4:Vane Shear Test at 35 mm Distancefrom Center of Model with Load after 07 Days Soaked Period

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	34	31	30	32	14
35 mm from center of model	80	40	39	37	38	22

Table A12.5:Vane Shear Test at 80 mm Distancefrom Center of Model with Load after One Month Soaked Period

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
80 mm from center of model	0	31	30	29	29	13
80 mm from center of model	80	42	37	36	36	17

Table A12.6:Vane Shear Test at 35 mm Distancefrom Center of Model with Load after One Month Soaked Period

Test Location	Test Depth (mm)	Shear Strength(divisions) fromApparatus for Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	37	36	34	36	17
35 mm from center of model	80	45	43	39	42	24

Appendix 13

Shear Strength Values of Clay Surrounding

Table A13.1:Shear Strength Valuesat 80 mm Distancefrom Center of Model Initially

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
80 mm from center of model	0	6	6	6	5	2
80 mm from center of model	80	7	7	6	6	3

Table A13.2:Shear Strength Valuesat 35 mm Distancefrom Center of Model Initially

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	7	7	7	6.5	3
35 mm from center of model	80	9	8	9	8	4

Table A13.3:Shear Strength Valuesat 80 mm Distancefrom Center of Model with Load after 07 Days Soaked Period

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding with load(8 kg) after 07 days				
		Aggregate	Concrete	Plaster	Brick	Clay
20 mm from edge of model	0	7	6	6	6	3
20 mm from edge of model	80	10	9	9	9	4

Table A13.4:Shear Strength Valuesat 35 mm Distancefrom Center of Model with Load after 07 Days Soaked Period

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding with load(8 kg) after 07 days				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	9	9	8	9	4
35 mm from center of model	80	11	11	10	11	6

Table A13.5:Shear Strength Valuesat 80 mm Distancefrom Center of Model with Load after One Month Soaked Period

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding with load(8 kg) after One Month				
		Aggregate	Concrete	Plaster	Brick	Clay
20 mm from edge of model	0	9	8	8	8	4
20 mm from edge of model	80	12	10	10	10	5

Table A13.6:Shear Strength Valuesat 35 mm Distancefrom Center of Model with Load after One Month Soaked Period

Test Location	Test Depth (mm)	Shear Strength(kPa) of Clay Surrounding with load(8 kg) after One Month				
		Aggregate	Concrete	Plaster	Brick	Clay
35 mm from center of model	0	10	10	9	10	5
35 mm from center of model	80	13	12	11	12	7

Appendix 14

Table A14.1: Shear Strength Calibration Chart for the 33 mm Blade

Divisions	kPa	Divisions	kPa	Divisions	kPa	Divisions	kPa
1	0	36	10	71	20	106	30
2	1	37	10	72	20	107	30
3	1	38	11	73	20	108	30
4	1	39	11	74	21	109	30
5	1	40	11	75	21	110	31
6	2	41	11	76	21	111	31
7	2	42	12	77	21	112	31
8	2	43	12	78	22	113	31
9	3	44	12	79	22	114	32
10	3	45	13	80	22	115	32
11	3	46	13	81	23	116	32
12	3	47	13	82	23	117	33
13	4	48	13	83	23	118	33
14	4	49	14	84	23	119	33
15	4	50	14	85	24	120	33
16	4	51	14	86	24	121	34
17	5	52	14	87	24	122	34
18	5	53	15	88	25	123	34
19	5	54	15	89	25	124	35
20	6	55	15	90	25	125	35
21	6	56	16	91	25	126	35
22	6	57	16	92	26	127	35
23	6	58	16	93	26	128	36
24	7	59	16	94	26	129	36
25	7	60	17	95	26	130	36
26	7	61	17	96	27	131	36
27	8	62	17	97	27	132	37
28	8	63	18	98	27	133	37
29	8	64	18	99	28	134	37
30	8	65	18	100	28	135	38
31	9	66	18	101	28	136	38
32	9	67	19	102	28	137	38
33	9	68	19	103	29	138	38
34	9	69	19	104	29	139	39
35	10	70	20	105	29	140	39

Appendix 15

Figure A15.1: Construction Debris

Figure A15.2: Concrete Debris

Figure A15.3: Brick Debris

Figure A15.4: Cement Plaster Debris