IDENTIFICATION AND TREATMENT OF EXPANSIVE SOIL IN SRI LANKA

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R.A.I. Senarathne
July 2008

DECLARATION

I hereby declare that this submission is my own work and that, to the best of my knowledge and behalf, it contains no material previously published or written by another person nor material, which to substantial extent, has been accepted for the award of any other academic qualification of an university or institute of higher learning except where acknowledgment is made in the text.

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Chapter 1

Introduction

1.1 Expansive Soil

Soils that swell excessively with the absorption of water are referred to as expansive soils. It should be also recognized that these soils shrink considerably with the loss of moisture. The deformations in the process of swelling and shrinkage are significantly grater than that predicted by classical elastic and plastic theories. An expansive soil is generally very stiff and hard during dry weather, but becomes soft when moist. Ground surface shows cracks during dry seasons.

Movements are usually in an uneven pattern and of magnitudes that will cause extensive damage to structures resting on them. Typical distress patterns resulting from heave of expansive soils is presented in Figure1.1.

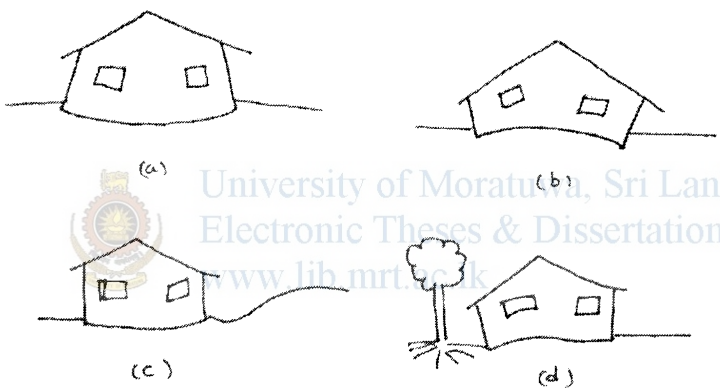


Figure.1.1 Typical distress patterns resulting from heave of expansive soils as indicated in foundations on expansive soils (a) adage lift, (b) centre lift, (c)Localized heave due to drainage problems, (d) Localized shrinkage caused by aggressive tree. (After Jones and Holtz,1973)

In areas where expansive soils are present, road bases, building foundations or walls can become vulnerable to uplift, and also retaining walls constructed on this situation may experience huge problems due to additional horizontal pressure. The roads constructed over such sub grade indicate unsatisfactory performance due to expansiveness. Swell movement can exert enough pressure to cause cracks in sidewalks, driveways, basement floors, pipelines and even foundations.

The expansiveness of soil is a considerable challenge to engineers in the construction industry to design safe and economic foundations for building structures. It is reported that expansive soils caused more damage to lightly loaded buildings and pavements than any other natural hazard, including floods and earthquakes in USA. (Jones & Holtz 1973)

1.2 Occurrence of Expansive Soils in Sri Lanka and Recorded Problems

Chemical weatherings of materials such as feldspars, micas and limestones can form clay minerals. The three most common clay minerals are Kaolinite, Illite and Montmorrilonite. The particular mineral formed depends on the make up of the parent rock, topography, climate, neighburing vegetation, duration of weathering and several other factors. Expansiveness is particularly noted in Montmorrilonitic clays.

Montmorillonitic clays are often formed as a result of the weathering of ferromagnesian minerals, Calcic feldspars, and volcanic materials. They are most likely to from in an alkaline environment with a supply of magnesium ions and a lack of leaching. Such conditions would most likely be present in semi and arid regions.

The swelling mechanism of clay mineral with water is presented in Figure1.2. Cracking of expansive soil during shrinking is shown in Figure 1.3.

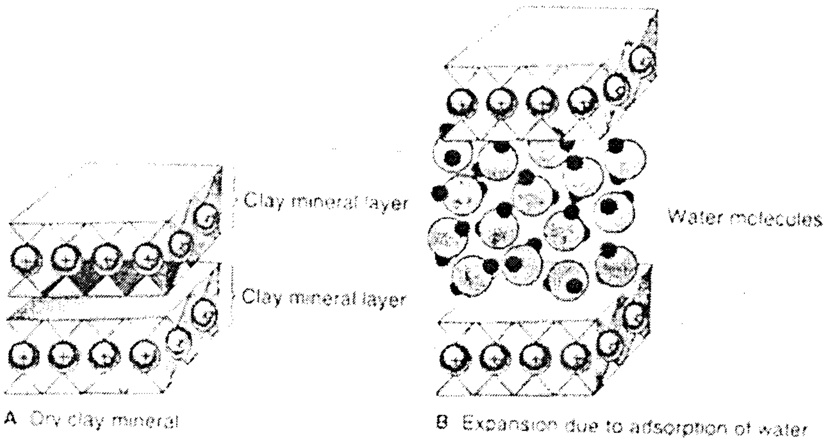


Figure 1.2 Swelling mechanism of clay mineral layer (After http://www.origins.rpi.edu/claycatalyzed.html.)

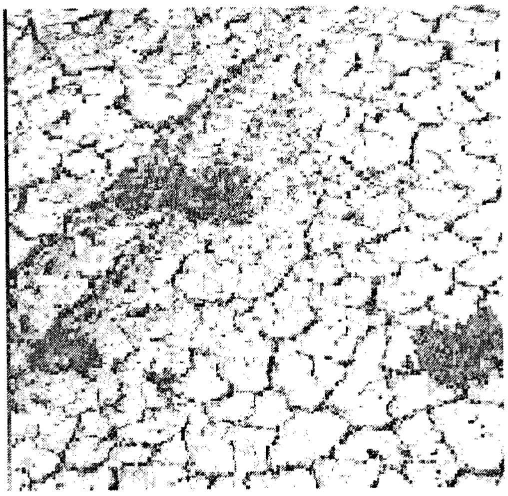


Figure 1.3.Expansive soil showing cracks. (After http://www.surevoid.com/ surevoidweb/soils/ xpansive_cons.html)

Expansive soils are common in USA, Canada, Israel, South Africa, Australia, Morocco, India, Sudan, Peru, Spain, Sri Lanka and in many other countries. Black cotton soil of India is a well known expansive soil where intensive studies have been carried out.

Shrinking and swelling of a potentially expansive soil will occur with significant moisture content changes. In humid climate, the soil is moist or wet and tends to remain so throughout the year and very little shrinkage or swelling will occur.

Most of the problems with expansive soils occur in arid, semi-arid and monsoonal areas. In such areas, seasonal distribution of precipitation and evaporation/ transpiration cause wide fluctuation in the soil moisture content.

Presence of expansive soil in Sri Lanka was first identified in Dambulla Gam Udawa project in 1986. Prior to this the structural distress in the buildings were attributed to poor workmanship. Since then many structural distress shown in lightly loaded building in regions such as Anuradhapura and Hambanthota were identified to be a result of the expansiveness of the underlying soil.

1.3 Identification of Expansive Soils

To be expansive a soil must have a significantly high clay content, probably falling within unified classification symbols CL or CH. Occasionally soils with symbols ML, MH and SC have also expansive. The expansive soils will have a high LL and PL. It will also have high activity. Some correlations are presented in Table1.1. The degree / level of expansiveness is expressed by swell pressure and free swell which can be determined in the laboratory. Some correlations with common soils are shown in Table 1.2. Unequal high peeks in X-ray diffraction pattern are indicated in expansive soils.

Table 1.1. Correlations with common soil tests (After Jones and Holtz 1973)

Percent colloids	Plasticity index (%)	Shrinkage limit (%)	Liquid limit (%)	Swelling potential
<15	<18	<15	<39	Low
13-23	15-28	10-16	39-50	Medium
20-30	25-41	7-12	50-63	High
>28	>35	>11	>63	Very high

Table 1.2. Correlations with common soil tests (After Jones and Holtz 1973)

Laboratory and field data			Degree of expansiveness		Swelling potential
			Probable expansion (%)	Swelling pressure	
Percent passing #200sieve	Liquid limit (%)	SPT N value		kPa	
<30	<30	<10	<1	50	Low
30-60	30-40	10-20	1-5	150-250	Medium
60-95	40-60	20-30	3-10	250-1000	High
>95	>60	>30	>10	>1000	Very high

1.4 Approaches for Handling Sites With Expansive Soils

Swelling and shrinking of a soil will take place with the fluctuation of the water content. If that fluctuation can be kept at a minimum by providing suitable means of drainage the problem can be minimized. Vertical moisture barriers could be placed adjacent to pavements or around the perimeter of foundations down to the maximum depth of moisture changes to maintain uniform soil moisture content within the barrier with that differential movement are minimized. (Jones and Holtz 1973)

The other option is to transfer the structural loads to a level below the active zone of expansive soils.

The expansiveness of a soil could be reduced by the use of various additives such as lime. If the critical region of soil in the active zone can be modified with the use of an additive, the problem can be solved to same extent. (Jones and Holtz 1973)

The depth of moisture change is often limited to around 8 feet. Hence removal of expansive soil and replacement with modified nonexpansive materials can be done over this depth. (Jones and Holtz 1973)

Lime injected or mixed with expansive soil can reduce potential of heave by reducing the mass permeability, thereby reducing the amount of water seeping into the soil, by cementation and exchange of sodium for calcium ions. Fissures should exist in situ to promote penetration of lime-injected slurry. Lime may be detrimental in soils containing sulfates. (Jones and Holtz 1973)

Postassium solutions injected into expansive soil can cause a base exchange, to increase the soil permeability, and can effectively reduce the potential for swelling.

Free water is added by ponding to bring soil to the estimated final water content prior to construction. Vertical sand drains may promote wetting of subsurface soil. On the other hand, by enhancing the stiffness of the structure and the foundation system, it could be made to behave as a one rigid unit with minimum differential settlements. Thus it may not experience much structural distress due to the expansiveness of the soil. (Jones and Holtz 1973)

1.5 Scope of the Present Study and the Thesis

The study was carried out on potentially expansive soils from several identified sites in Anuradhapura and Hambanthota. A Number of single storied building in above sites have experienced extensive structures distress. Soils obtained from these sites were subjected to laboratory tests such as liquid limit, plastic limit, activity, free swell and swell pressure.

The specific sites used in the study are;

(1). Post Harvesting Institute – Anuradhapura

Several buildings used for accommodation in this site have a lot of cracks. In the construction, rubble masonry strip foundations, load bearing brick walls and asbestos roof with wooden skeleton had been used. Severe cracking and distress could be seen on the walls, pavement and floor.

(2).Aurvedic Hospital- Anuradhapura

This is a two-storied building having a length 202 feet and breath 71 feet. For the construction, concrete pad foundation for columns and rubble strip foundations for masonry walls were used. A 4 inch thick slab had been used. Severe cracking and distress could be seen on the walls.

(3).Sanasa Child care center –Saliyapura

Several buildings used for accommodation in this institute have a lot of cracks. For the construction, rubble masonry wall foundation, brick wall and asbestos roof with wooden skeleton had been used.

(4).Maithreegama Tsunami Housing Project-Hambanthota

The site is situated at Siribopura, 1 ½ Km away from Hambanthota town and close to Hambanthota- Gannoruwa road. Under this project, two thousand single storied houses were constructed. For the construction, rubble masonry walls, cement block walls and tile roofs with wooden skeletons had been used. Considerable mount of cracks could be seen o walls. As a consequence, inhabitants of this area had to face significant economic losses and severe hardships. With the rapid increase of the development activities presently taking place in this area, more and more problems associated with expansive soils can be expected in the future.

After studying the characteristics indicative of expansiveness in the natural soils found in these sites, the possibility of reducing the expansiveness with the use of additives was studied.

For the process to be economically viable paddy hush ash was selected as the additive. Various percentages of the paddy hush ash were added to the natural soil and the effects on the basis characterization were studied.

Thereafter, using SAP 2000, Structural analysis was done to assess the effect of the foundation and structural stiffness on walls. Different type of foundations and walls with or without doors and windows were used. The swell pressure was applied at the bottom of the foundation. The SAP 2000 is a finite element analysis package which can be used to analyze buildings, bridges, walls, foundations and water retaining structures very easily. In this analysis, grade 30 concrete, block work, and rubble masonry are the major materials that were represented by using specific material properties. The stresses developed in the walls were obtained in the FE Programme and critical tensile stress zones were identified.

1.6. Outline of the Thesis

Chapter 01 gives Introduction to expansive soils and Chapter 02 of this report discusses the reasons for expansiveness in soils. Identification of expensive soils in Sri Lanka is presented in Chapter 03. Chapter 04 discuses Laboratory studies on collected samples from sites in Auradhapura and Hambathota. Laboratory studies on collected samples are presented in Chapter 05. Chapter 06- discusses numerical simulation to indicate reduction of effect of swell pressure generated by expansive soils by Enhancement of Foundation Stiffness (using SAP2000). Sammary and conclusions are presented in Chapter 07.

Chapter 2

Reasons for Expansiveness in Soils

2.1 Formation of Clay Minerals

Chemical weathering of materials such as feldspars, micas and limestone can from clay minerals. Clay minerals are complex aluminum silicates composed of two basic units, silica tetrahedra and alumina octahedron. The combination of tetrahedral silica units gives a silica sheet and the combination of the octahedron aluminium hydroxyl units gives an octahedral sheet (or gibbsite sheet). When magnesium replaces the aluminium atoms in the octahedral units it is refereed to as a brucite sheet. (Figure.2.1)

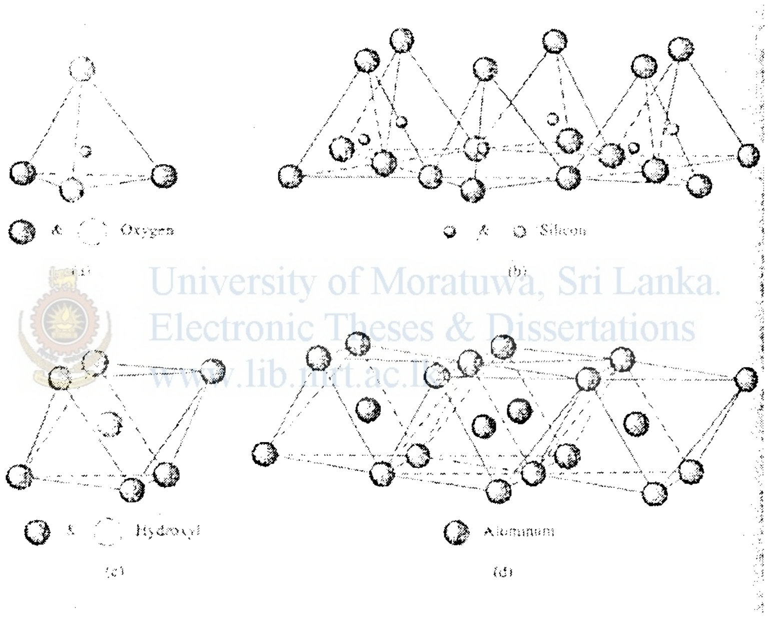


Figure 2.1 (a) silica tetrahedron (b) silica sheet (c) aluminum octahedron (d) octahedral (gibbsite) sheet (After Das 1998)

Several different clay minerals of different structural configuration occur in nature. The three most common clay minerals are kaolinite, illite and montmorillonite. Montmorillonites are a part of the smectite group. Clay minerals are very different from gravels, sands and soils due to the very small particle size and due to fact that they are usually plate shaped. The different structural configurations of the three most common clay minerals are presented in Figure 2.2.

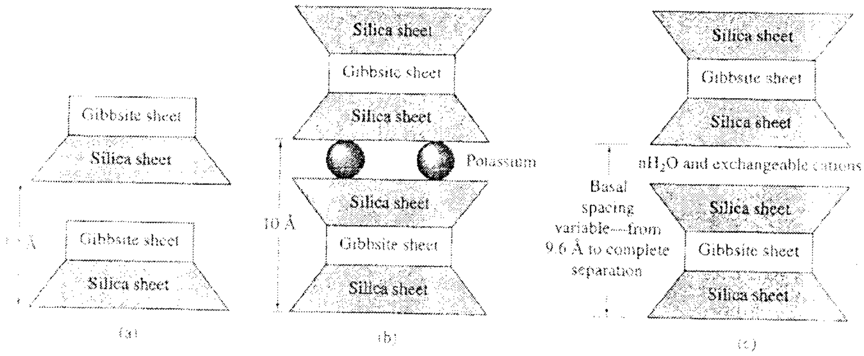


Figure 2.2. Diagram of the structures of (a)Kaolinite (b)Illite (c) Montmorillonite (After Das 1998)

The plate shaped clay minerals have a negative static electric surface charge and a very large specific area. The engineering properties of clays are strongly influenced by the very small particle size, large surface area and their inherent electrical charges.

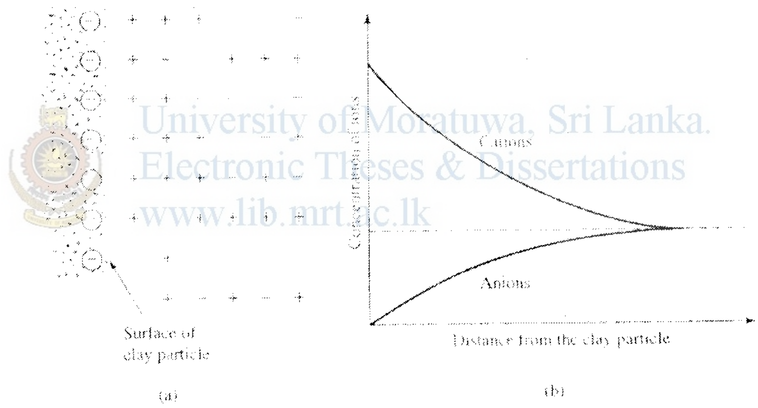


Figure 2.3. Diffuse double layer (After Braja M. Das)

In a dry clay, the negative charge is balanced by exchangeable cations like Ca^{2+},Mg^{2+} , Na^+ and K^+ surrounding the particles that are held by electrostatic attraction. When water is added dipolar water particles are also attached to the negatively charge clay surface (Figure 2.3). All the water held to the clay particle by force of attraction is known as "Double layer of water". The innermost of double layer water, which is very strongly bonded, is known as "Adsorbed water". Typical double layer configuration of Montmorrilonite and Kaolinite particles are illustrated in Figure 2.4.

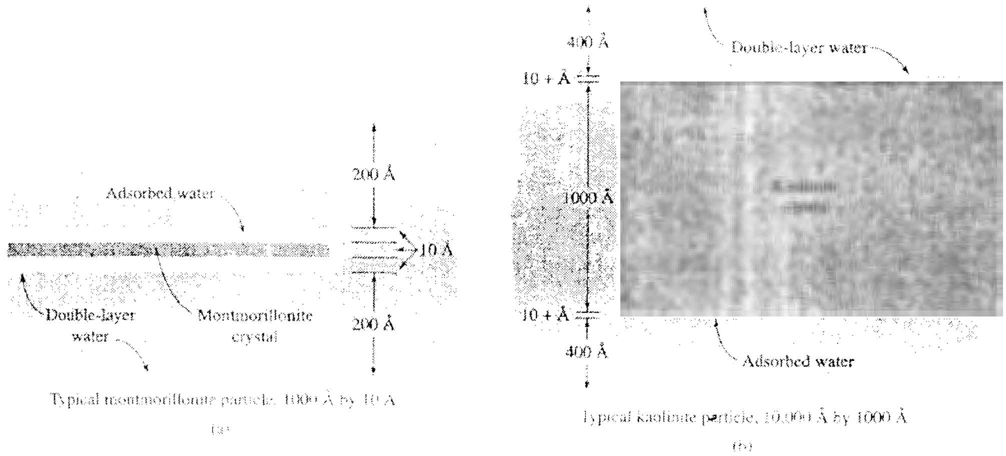


Figure 2.4.Clay water (After Das 1998)

2.2 Formation of Expansive Clays

Swelling occurs when water infiltrates between and within the clay particles, causing them to separate. Kaolinte is essentially non expansive because of the presence of strong hydrogen bond that hold the individual clay particles together, and Illite contains weaker potassium bonds that allow limited expansion. Montmorrillonite particles are only weakly linked and water can easily flow in to montmorillonite clays and separate the particles.

The high affinity to water exhibited by montmorillonite clays are illustrated by the increased thickness of the adsorbed water layer (Figure 2.4). There are two types of montmorillonite clay; calcium montmorillonite and sodium montmorillonite (bentonite). The latter is much more expansive but less common.

The difference in swell potential of different clay minerals under different surcharge loads is illustrated in Table 2.1.

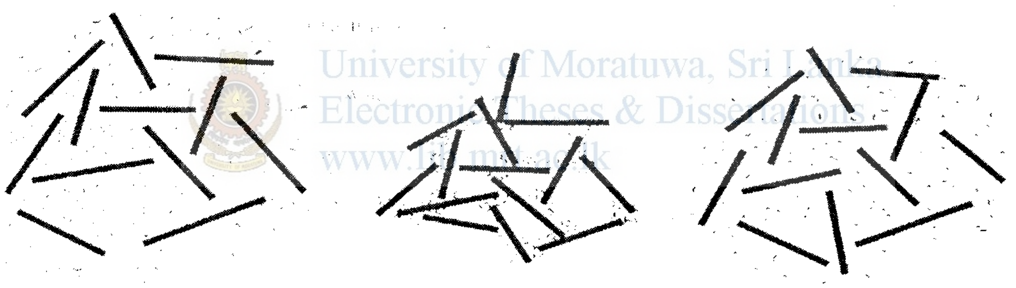
Table 2.1 swell potential of pure clay minerals (After Jones and Holtz 1973)

Surcharge load	Swell potential %		
kPa	Kaolinite	Illite	montmorillonite
9.6	Negligible	350	1500
19.1	Negligible	150	350

2.3 Shrinking and Swelling Process of Expansive Clays.

The process of shrinking and swelling in an expansive soil is illustrated in Figure 2.5. The structure of saturated clay is presented in Figure 2.5 (a). When this soil dries the remaining moisture congregates near the particle interface, forming menisci, as shown in Figure 2.5. (b). The resulting surface tension forces will pull the particles closer together causing the soil to shrink. This stage of the soil could be compared to a compressed spring.

The soil in this state will have a high affinity for water and will draw in available water using osmosis. Then it is referred to have a high "Soil suction". If water balance is available the suction will draw it into the space between the particles and the soil will swell. This will result in the stage given in Figure 2.5.(c). The compressed spring has how been released.



(a) (b) (c)

Figure 2.5 (a)expansive clay (b) shrinkage (c) swelling(After Jones & Holtz 1973)

2.4 Expansive Potential

The potential expansion of a soil will depend on the percentage of expansive clays in the soil. A pure Montmorillonite could swell more than fifteen times its original volume, but clay minerals are rarely found in such a pare form. Usually, the expansive clay minerals are mixed with more stable clays and with sands or sills.

A typical "Montmorillonite" would probably not expand more than 30% to 50% even under the worst laboratory conditions. The swell in the field would be much less.

Two of the most common variables to consider in relation to swelling are the initial moisture content and the surcharge pressure. If the soil is initially moist then there is much less potential to additional expansion than if it were dry. Similarly even a moderate surcharge pressure restrains much of the swell potential.
Figure 2.6 illustrate a typical relationship between swell potential, initial moisture content and surcharge pressure. This relationship demonstrates why pavement and slabs or grade are so susceptible to damage from expansive soils.

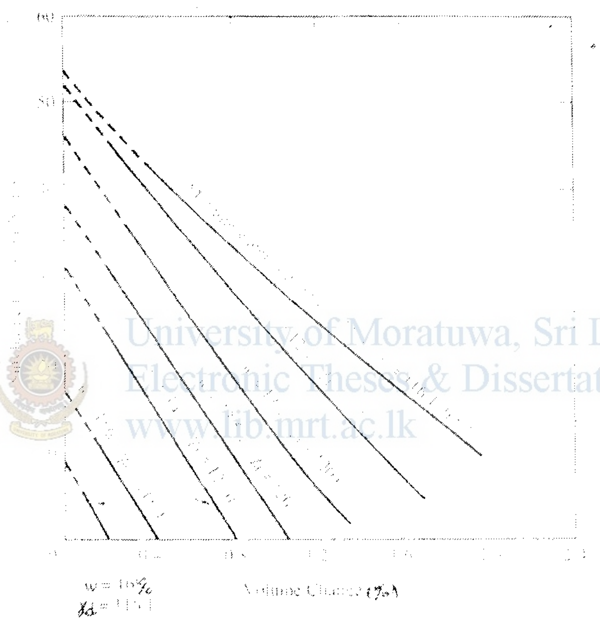


Figure 2.6 swell potential as a function of initial moisture content and surcharge load (After Jones & Holtz 1969)

Remolding a soil into a compacted fill may make it more expansive (O' Neill and Poormoayed 1980) probably because main process breaks up cementation in the soil and produce high negative pure water pressures that later dissipate. Factors such as the method used in the compaction process (Static or kneading) the as compacted moisture content and dry unit weight influences the swelling potential. Kneading compaction has been shown to create dispersed structural with lower swell potential than soil statically compacted.

As illustrated by Figure 2.7 a soil compacted wet of optimum will have a reduced swell potential. It also illustrates that a soil compacted to a lower dry density will have a lower swell potential.

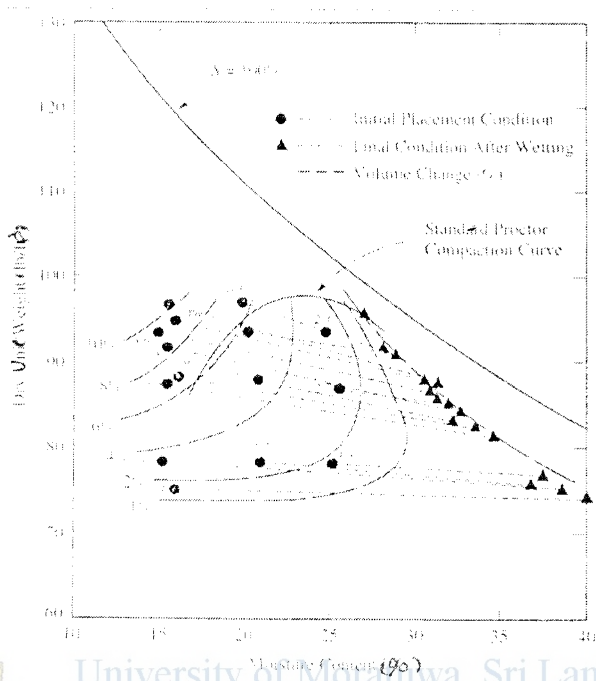


Figure 2.7 swell potential to compacted clay (After Jones and Holtz 1969)

Chapter 3

Identification of Expansive Soils in Sri Lanka

3.1 Soil Types in Sri Lanka

Sri Lanka is an island situated in Indian Ocean. Normally the country consists of soils such as Residual soils, Organic soils and Peat etc. Residual soils contain materials which originate from the in situ parent rocks by mechanical and chemical weathering (decomposition), and are found above the yet-to-be weathered strata. The properties of residual soils depend strongly on weathering conditions and features of the parent rocks. The unique formation history of residual soils potentially leads to different engineering properties compared with sedimentary (transported) soils such as sands and clays. Nevertheless, the knowledge of "classical" geotechnical engineering is mostly based on the behavior of sedimentary (transported) soils. The understanding of residual soils is, in general, insufficient, and cannot satisfy the need generated by the extensive development in those areas covered by residual soils.

Peat is an organic material that forms in the waterlogged areas. Peat can be found in bogs and fens under acidic condition. These conditions favor the growth of mosses, especially sphagnum. As plants die, they do not decompose. Instead, the organic matter is laid down, and slowly accumulates as peat because of the lack of oxygen in the bog.

Peat bogs contribute to the welfare of all living things by 'locking up' carbon that would otherwise increase the greenhouse effect. Carbon, removed from the atmosphere over thousands of years, is released when bogs are drained and peat starts to decompose.

3.2 Expansive soils in Sri Lanka-Identified Locations

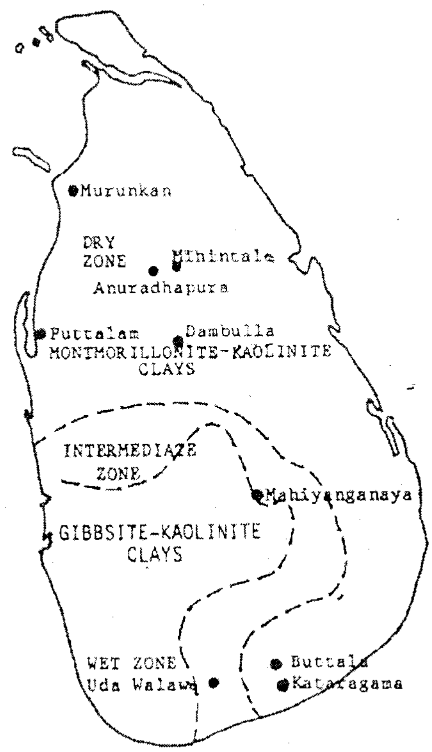


Figure 3.1. Distribution of reported locations of expansive soils in Sri Lanka (After Herath.1993)

Sri Lanka has been divided into three clay mineral zones based on composition of clay minerals, namely; wet zone clay minerals province, dry zone clay minerals province and intermediate zone clay mineral province.(Figure 3.1- Herath 1993)

It is further revealed that the dry zone clay minerals province mainly consists of kaolinite- montmorrilonite clays with calcareous material while the intermediate zone clay mineral province consists of kaolinite clays with a low proportion of gibbsite and montmorrilonite. No montmorrilonite was found in the wet zone clay mineral province. These clay mineral zones closely follow the main climatic zones of wet, dry and intermediate in Sri Lanka, showing that the climate has played a major role in the development of clay minerals in the country. (Herath 1993)

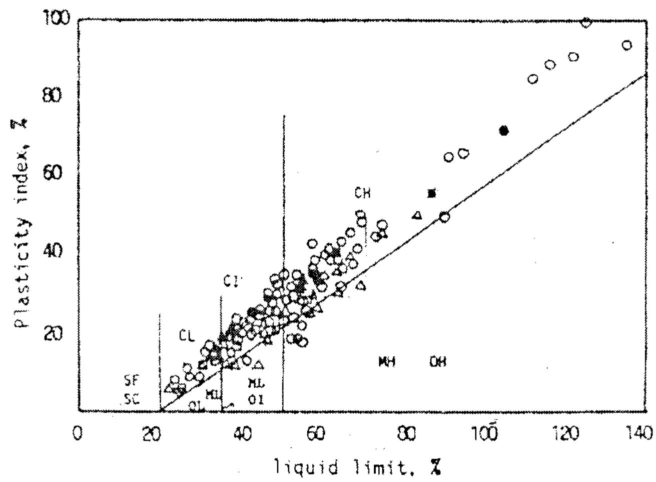


Figure 3.2. The graph of plasticity index vs liquid limit. (After Herath1993)

Herath (1993) summarized the results of large number of soil investigation projects carried out in Sri Lanka buy Central Soil Testing Laboratory (CSTL) and the National Building Research Organization (NBRO) in the decade of 1980. It was reported that expansive soil deposites were found in places such as Anuradhapura, Butthala, Dambulla, Kataragama, Mahiyanganaya, Mihintalaya, Murukan (Mannar), Puttalam and Udawalawa.

A large volume of data were from the soil samples obtained from numerous investigation projects conducted in these areas. Atterberg limit tests, swell pressure tests and tests for free swell were conducted on these samples. Figure 3.2 (Herath 1993) presents a plot of plasticity index vs. liquid limit for these samples, done on the Casagrande's plasticity chart.

It can be seen that almost all the values lie above that 'A' line with a few exceptions. Majority of the soils tested fall in the zones of low to high plasticity inorganic clays. It must be noted here that a considerable proportion of clayey sand (SC) samples also exhibited both swell pressures and high free swell values. However such soils (SC) are not indicated in the plasticity chart here as practical difficulties encountered in performing Atterberg limits on them.

The plot of plasticity index verses percentage of clay size particles was presented in Figure 3.3. This illustrates the influences of clay content on plasticity index Van Der Merwe's (1964) classification boundaries were also superimposed in the figure. It could be seen that a large number of soil samples tested lie in the moderately expansive and highly expansive zones. A few results fall in the very high expansive zones.

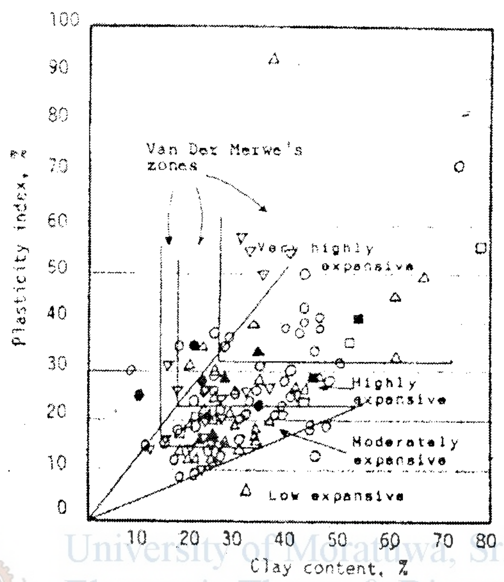


Figure 3.3. The graph of plasticity index vs. clay content (After Herath 1993)

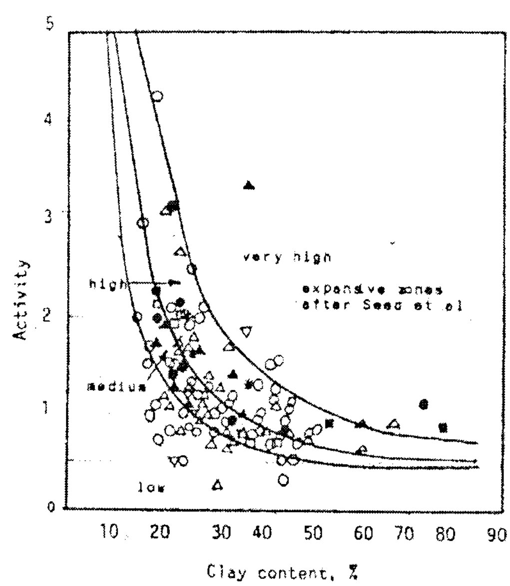


Figure 3.4 the graph of activity vs clay content (After Herath 1993)

Activity Vs clay size particles (percent minus 0.002) were plotted and compared with the zones developed by seed at all for expansiveness. It appears from the figure that almost all the samples tested fall in the medium to high expansive category. (After N. W. Herath 1993)

3.3 Sites Studied in this Project

Three sites in Anuradhapura district and one site from Hambanthota district were studied in this project. In all four sites, lightly loaded buildings constructed had been subjected to considerable structural distress. These sites considered are;

(1) Post Harvesting Institute – Anuradhapura

(2) Aurvedic Hospital- Anuradhapura

(3) Sanasa Child care center –Saliyapura

(4) Maithreegama Tsunami Housing Project-Hambanthota

3.3.1 Post Harvesting Institute – Anuradhapura

Several buildings used for accommodation at this institution showed considerable amount of cracks. These buildings were constructed with rubble masonry foundations, brick walls and asbestos roofs with wooden skeleton.

Lot of cracks was seen in number of buildings spreaded around the site. There were cracks on the floor, under the window sills, pavements, etc. as illustrated in Figures.3.5 to 3.7.

Figure 3.5 Appearance of the ground surface cracked in dry season

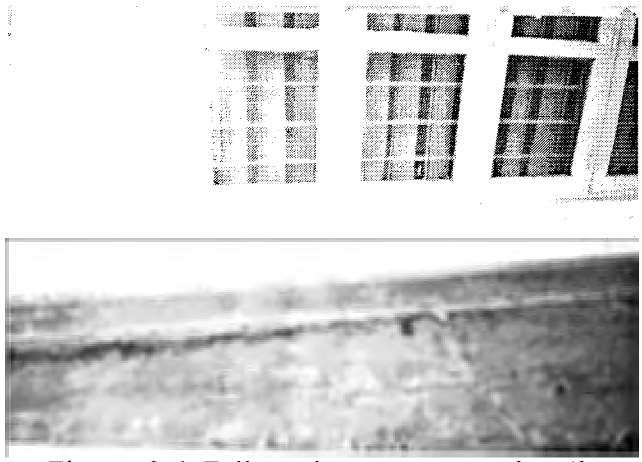


Figure. 3.6. Failures in pavement and walls

Figure 3.7 Failures in pavement

Figure 3.8 New building constructed according to NERDC construction technology with foundation constructed on sand bedding

A new building constructed on the bed on sand according to new technology developed by NERDC does not have any cracks. It was constructed on 4" x 4" pre stressed and pre cast concrete columns with individual pad footings. All columns were combined on top of the ground using reinforced concrete beams. Walls were constructed by slipfom method with cement and quarry dust. Thereafter, reinforced concrete beams were laid on top of the walls combining all columns. Concrete door and window frames were fixed at all openings.

3.2 2.Aurvedic Hospital- Anuradhapura

This is a two-storied building having a length of 202 feet and width of 71 feet. This building consists of reinforced concrete columns supported on isolated pad foundations, masonry walls supported on rubble foundation and a 4" thick concrete slab. The cracks are shown in Figure 3.9 to Figure 3.12 in the ground floor.

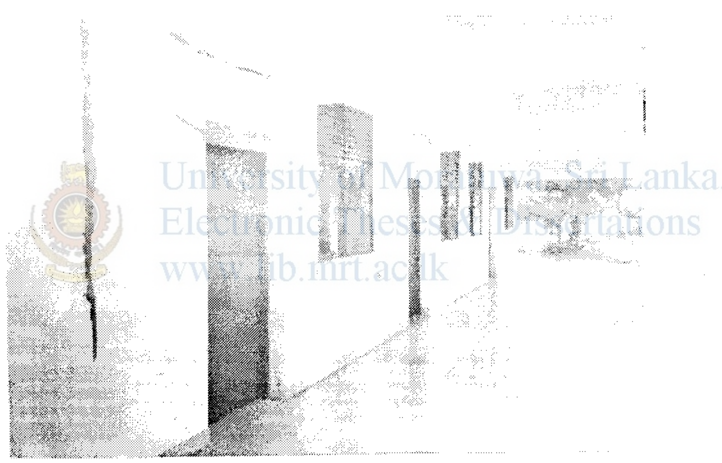


Figure 3.9 Cracks appearing at the wall

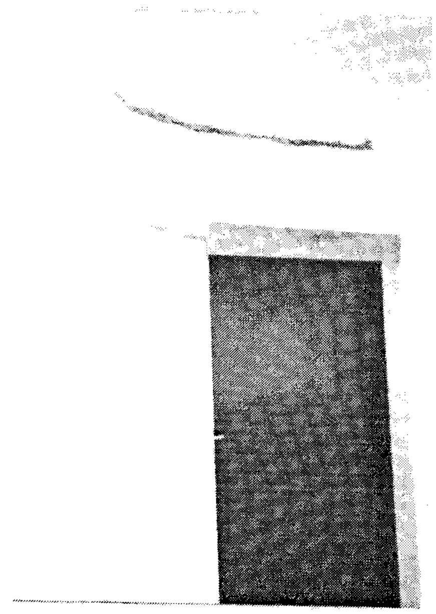


Figure 3.10 Cracks appearing at corner of door

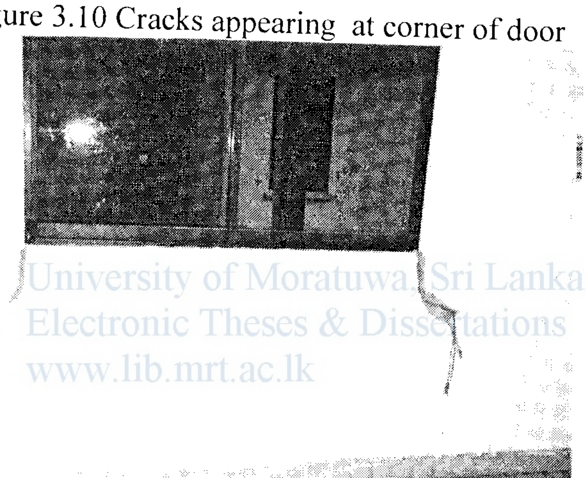


Figure 3.11 cracks appearing at corner of window

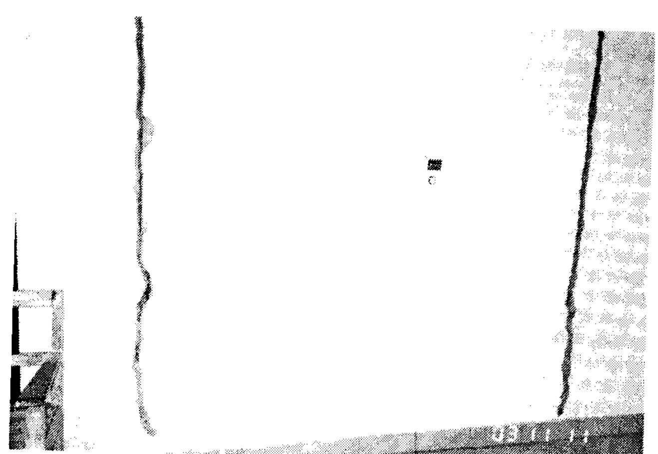


Figure 3.12. Vertical cracks along the column and wall

3.3.3. Sahana child care center- Saliyapura

This building consists of masonry walls supported on rubble foundation and tile roof with wooden skeleton. The cracks are shown in Figure 3.13 to Figure 3.14.

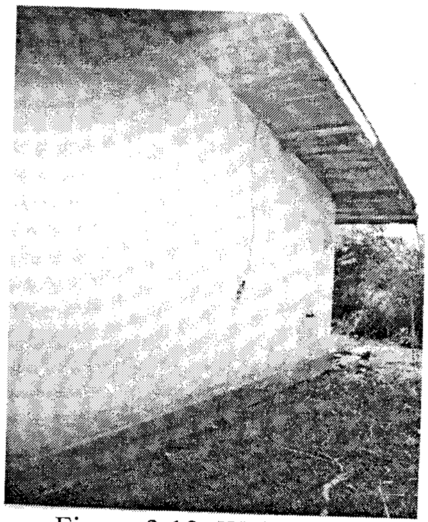


Figure 3.13. Wall cracks

Figure 3.14. Wall cracks along the window

3.3.4. Maithreegama Tsunami Housing Project-Hambanthota

The site is situated at Siribopura, one and half kilo metres away from Hambanthota town close to Hambanthota- Gannoruwa road. Under this project, two hundred single storied houses were constructed. For the construction, rubble masonry foundation, block wall and tile roof with wooden skeleton were used. Lot of cracks were observed in a number of buildings.

3.4 Investigations, Laboratory Study Program

Soil sample were collected from the above mentioned sites where structural distresses were observed. Attempts were taken to obtain undisturbed samples wherever possible. When it was not possible disturbed sample were obtained. Figure 3.15 illustrates collection of a soil sample at foundation level at the site of Post Harvesting Institute in Anuradhapura.

Figure 3.15 Soil sample collecting at foundation level

All samples were in a very stiff state and following standard tests were conducted in the laboratory.

- Attaberg limits; Liquid limit and Plastic limit have done, and then plasticity index was calculated.
- Particle size distribution, wet sieve analysis and hydrometer analysis were carried out. Activity (the ratio of plasticity index to percentage of particles finer than 0.002mm) was calculated.
- Using oedometer apparatus, amount of free swell and swelling pressure was obtained. These are important parameters used in the analysis of expansive soils.

Chapter 4

Laboratory Studies on Collected Samples

4.1. Introduction

Observations of existing buildings in the four selected sites indicated that single storied buildings or light structures have undergone severe distress, especially in the cases of brick or cement block walls. Width of cracks on walls were varied from hair line cracks to a width of about 25 mm. Physical characteristics such as dry hard lumps of soils and appearance of shrinkage cracks during dry seasons were observed in all four sites. Disturbed soil samples from the four sites are presented in Figure 4.1

The routine laboratory tests such as Atterberg limits, grain size distribution, swell pressure and free swell were conducted on the samples obtained.

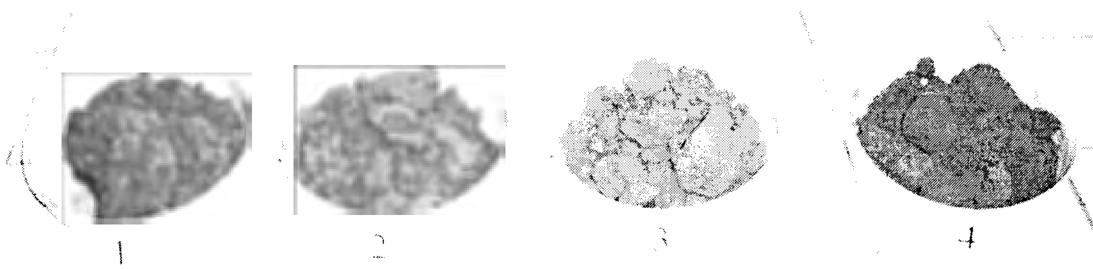


Figure 4.1. Disturbed soil samples collected for the laboratory tests (1).Maithreegama Tsunami Housing Project- Hambanthota. (2).Post Harvesting Institute – Anuradhapura (3).Aurvedic Hospital- Anuradhapura (4). Sanasa Child care center – Saliyapura

4.2. Basic laboratory tests

Atterberg Limits; the liquid and plastic limits of the soils are dependent on the amount and type of clay in soil. This also makes the basis for the soil classification system for cohesive soils. High liquid and high plastic limit are indicative of expansiveness in soils.

Activity is defined as the ratio of plasticity index and percentage of particles finer than 0.002 mm in clay fraction. Expansive soils are typical in having a high activity.

Particle size distribution tests were conducted on soil samples obtained from the four sites. As the fine fractions were quite high, hydrometer analysis was also conducted.

4.3. Swell pressure and free swell tests

Swell pressure test and the free swell tests were done initially in the conventional consolidation apparatus. Subsequently some tests were conducted using the CBR mould.

4.3.1. Swell pressure test

After a seating pressure is applied on for 24 hours to the specimen in a oedometer, the specimen was inundated with water and allowed to swell vertically until primary swell is completed. The specimen is loaded following primary swell until its initial void ratio/height is obtained. The total pressure required to reduce the specimen height to the original height prior to inundation is defined as the swell pressure. High swell pressures are observed in expansive soil.

Swell pressure test were conducted on sample remoulded in the consolidation ring to a predetermined remoulded density. Density was achieved by trail and error. The mass of soil required to give the necessary density was placed and compacted inside the consolidation ring in three stages. Thereafter two filter papers were kept on top and bottom of the remoulded sample.

4.3.2. Free swell test

Free swell is also measured in the conventional consolidation apparatus. CBR mould can also be used to determine the free swell (Figure4.2). Sample preparation was same as in the swell pressure test.

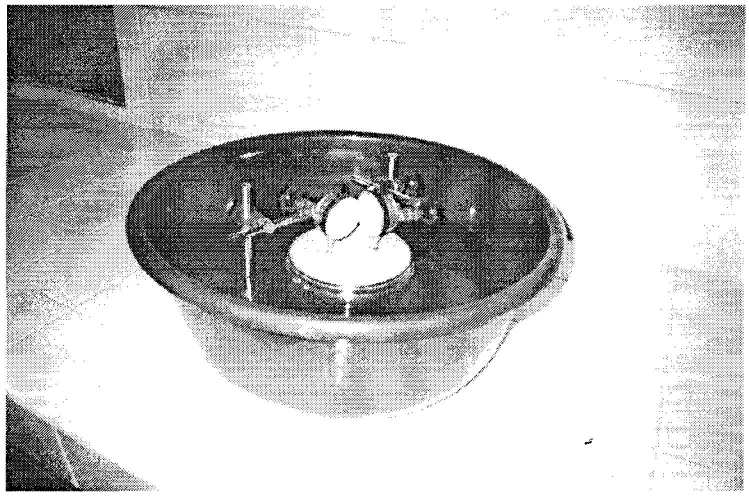


Figure 4.2. Use of CBR mould to identify the free swell.

4.4. Test Results.

The results of the particle size distribution tests on all four samples are presented in Figure 4.3 The result of the particle size analysis test (Figure 4.3), Atterberg limit text, swell pressure test and pre swell test are summarized in Table 4.1, above with the computed quantities of plasticity index, and activity.

Table 4.1 Laboratory tests done to identify the expansiveness

	Maithreegama housing project Hambantota	Aurvedic hospital Anuradhapura	Sahana Chidran home Anuradhapura	Post harvesting institute Anuradhapura
Dry density of sample-- g/cm^3	1.57	1.58	1.53	1.57
Liquid limit --%	120.88	89.19	77.90	79.34
plastic limit-- %	38.86	35.8	29.1	34.6
Plasticity index	82.02	53.4	48.8	44.7
% finer <0.075mm	55	55	74	74
Classification symbol	CH	CH	CH	CH
% finer than 0.002mm (Clay content.)	47	53	69.9	73
Activity	1.75	1.00	0.70	0.61
Swell pressure (kN/m^2)	80.4	49.12	27.2	19.24
Free swell (mm)	15	6.9	3.1	2.5

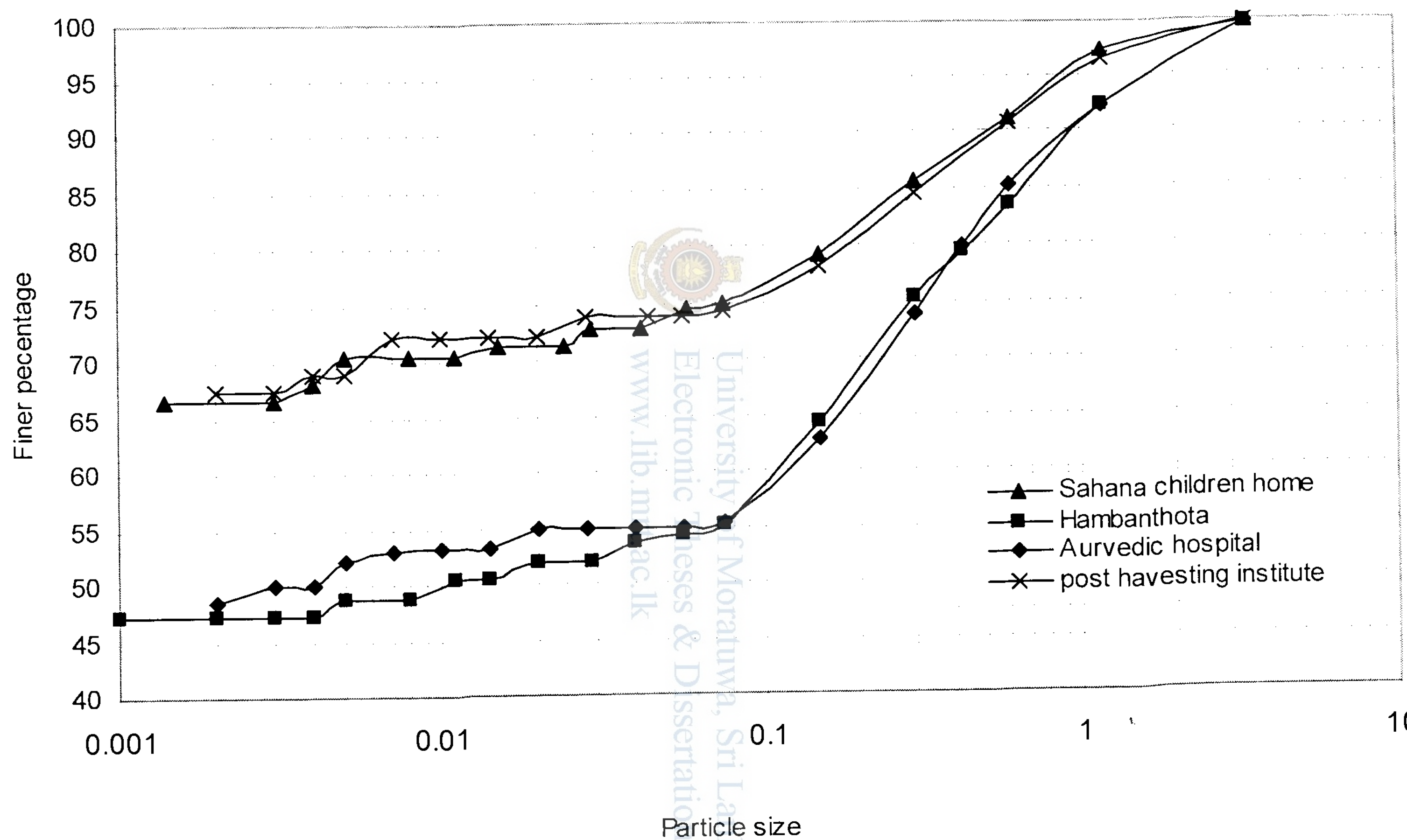
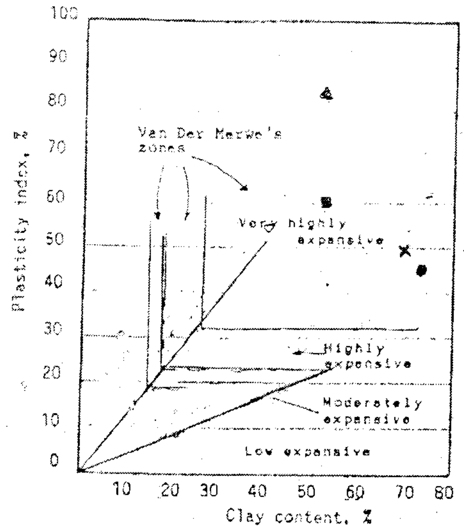


Figure 4.3. The graph of finer percentage to particle size for four sites.

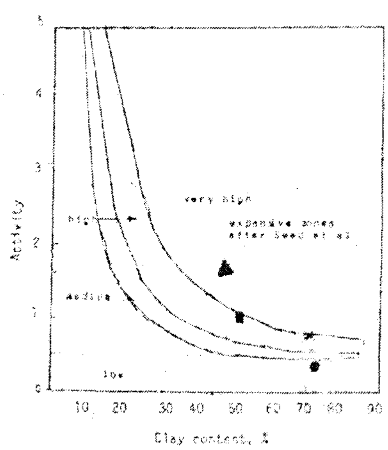
The results indicated that the soils encountered in this research, were laid in the high expansive zoon of "Van Der Merwe's Zones" chart (Fiure.4.4). The results are also plotted in the chart in Figure.4.5. Maithreegama housing project in Hambanthota is the most expansive.



▲ ..Housing project Hambanthota X..Aurvedic hospital

■..sanasa children home ●..post havesting institute

Figure 4.4. Variation of plasticity vs clay content (After Herath 1993)



▲ ..Housing project Hambanthota X..Aurvedic hospital

■..sanasa children home ●..Post harvesting institute

Figure 4.5. Variation of activity vs clay content (After Herath 1993)

Chapter 5

Reduction of expansiveness of a soil by the use of additives

5.1. Current practices

Treatment procedures that are available for stabilizing expansive soils can be listed as

- adding chemicals
- Pre wetting
- Soil replacement with compaction control
- Moisture loading
- Thermal methods

The successful applications of soil stabilization procedure require considerable experience and judgment regarding the soil on site, consideration of limitations of the method to be chosen and correct implementation procedures.

Lime stabilization had been used successfully in many projects with expansive soils. When hydrated lime is mixed with an expansive clay, chemical reactions take place and the swelling potential is decreased. The soil plasticity, workability and shear strength is also improved.

The lime can be mechanically mixed with the soil at a rate of about 2% to 8% by weight. The special equipment is needed to assure adequate mixing and the process generally limited to shallow depths of around 300mm. The chemical theory involved in the lime reaction is complex. The primary reactions include; cation exchange, flocculation-agglomeration, lime carbonation and pozzolanic reaction. The strength characteristic of a lime treated soil depend primarily on soil type, lime type, lime percentage and curing conditions.

Another method of treating soil with lime is to inject it in a slurry form using a technique known as pressure injected lime (PIL). The lime slurry is forced in to the soil under high pressure. This method is capable of mixing soils to depth of up to 2.5m. The PIL technique is most effective in highly fissured clays as the fissures provide path ways for the slurry to disperse. In addition to chemical effects the filling of fissures retard moisture migration in the soil.

Postassium solutions injected into expansive soil can cause a base exchange, increase the soil permeability, and effectively reduce the potential for swell. Free water is added by ponding to bring soil to the estimated final water content prior to construction. Vertical sand drains may be used for wetting of subsurface soil.

Other less common construction treatments include mixing cement with the soil or adding salts like sodium chloride or calcium chloride. These treatments are not recommended. They create severe challenges for gardeners by either cementing soils or causing them to become sodic or saline. It is better to avoid normal remedies for leaching salts out of these soils, because the soils will become more expansive (Jones and Holtz 1969)

Studies were done in India (Kate 1998, Kumaretal 1998) on the reduction of the expansiveness of "black cotton soil" with mixing of fly ash, a byproduct of thermal power plants. In India is was reported that an area of approximately 0.65 Million square kilometers are covered with expansive soils. Fly ash also possesses pozzolanic properties as lime, but the advantage is that it is available at almost no cost as it is an industrial by product.

It was found that with the introduction of about 15% of fly ash, pre swell index reduced from around 80% to 30-20 %, when the addition of about 12% of fly ash has reportedly caused reduction of swell pressure from around 120kN/m^2 to 90kN/ m^2.

5.2. Studies done in this project

In this project attempts were made to mix the expansive soils with different percentages of paddy husk ash. Paddy husk is freely available in large quantities in areas where expansive soils are encountered in Sri Lanka. As such, it would be a cost effective solution. Studies were done for the soils from Mithreegama Project in Hambanthota, which was found to be the most expansive.

5.3. Results of the test

Samples were prepared by mixing of burned paddy husks with soils, maintaining mix proposion as 5%, 10%, and 15% of soil weight. For easy identification of swell pressure reduction, three types of soil sample were taken for the test. Natural soil means the soil taken straight away from the site. Natural soil dried 24 hours in an oven maintaining constant temperature 105 C^0 was called Oven dry soil sample. Powder natural soil passing through 425mm sieve was taken for the test as sieved soil.

In the laboratory, powdered soil sample and burnt paddy husk were mixed together using an electrical mixture. Natural powdered soil sample was put into the Oedometer ring to one third of height of specimen and was well compacted. The complete thickness was achieved in three stages. Attempts were made to achieve a density close to the insitu density in the field. The remoulded sample was prepared also following the same procedure and two filter papers were kept on top and bottom of the remoulded sample and it was placed into Oedometer. Same procedure was followed to prepare all three types of sample. Thereafter swell pressure and free swell tests were carried out for all soil samples. The results of the tests are illustrated in Table 5.1.

The Figure 5.1 to Figure 5.3 illustrated the variation of swell pressure with time for the soil samples with different mix proportion of burned paddy husk. When the percentage of added paddy husk was increased, swell pressure decreased. With the addition of increased amounts of paddy husk ash the density of the soil sample also reduced. Table.5.1 presented the ultimate values of swell pressures.

For three types of soil sample were given nearly equal pressure reduction with burned paddy husk. Therefore, an effect of different methods of sample preparation has not had a major influence on the swell pressure reduction.

Table 5.1 Variations of swell pressure and densities for natural soil sample with different mix proportion of paddy husk ash.

Soil type	Adding % of paddy husk ash	Max. Swell pressure (kN/m^2)	Density of the sample (kN/m^3)	Swell Pressure reduction (%)
Natural soil	-	82.57	16.91	-
	5	47.75	16.17	42.17
	10	42.00	13.4	49.13
	15	30.55	12.12	63.00
Oven dry soil	-	82.39	16.41	-
	5	47.10	15.88	42.83
	10	40.60	13.24	50.72
	15	29.85	11.88	63.77
Sieved soil	-	83.76	16.99	-
	5	48.10	16.3	42.57
	10	42.80	13.24	48.90
	15	30.35	10.97	63.76

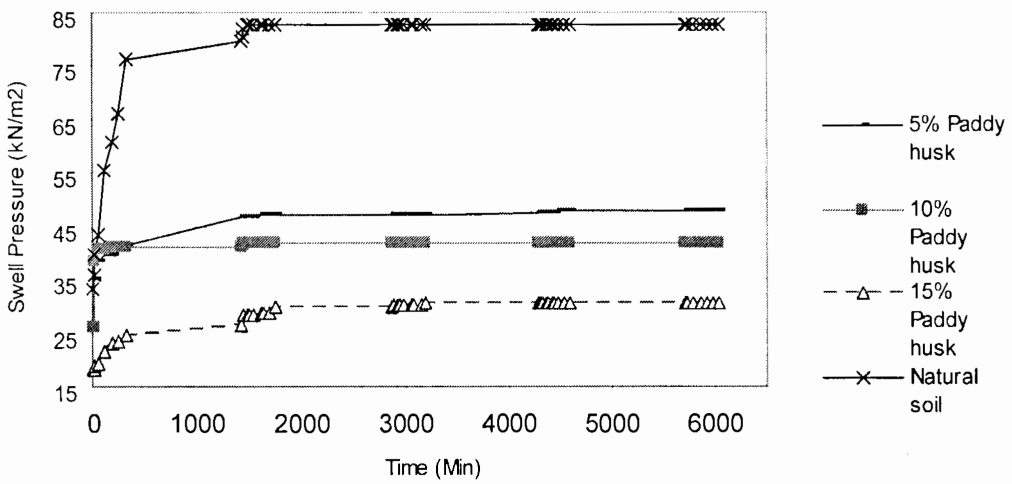


Figure 5.1. Variations of swell pressure with time for natural soil sample with different mix proportion of paddy husk ash.

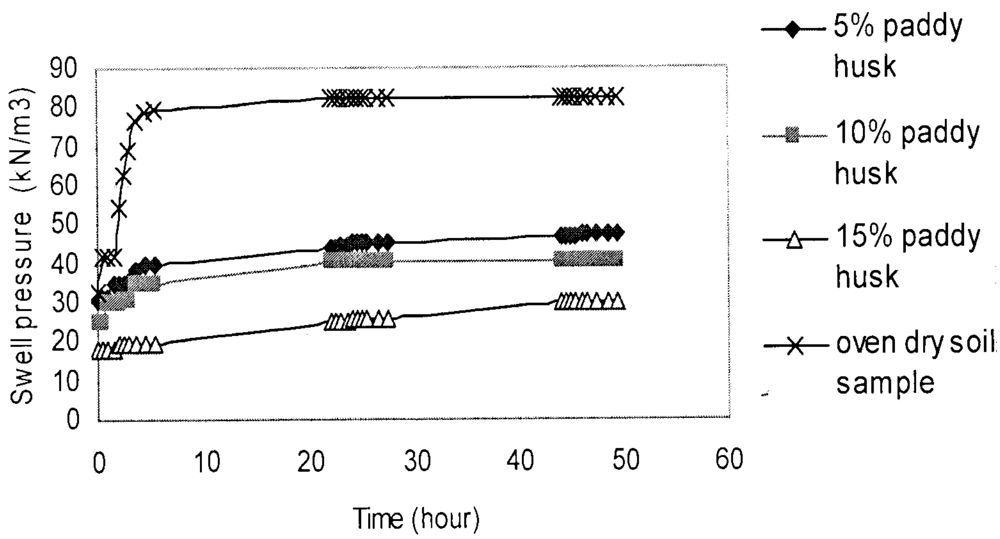


Figure 5.3. Variations of swell pressure with time for oven dry soil sample with different mix proportion of paddy husk ash.

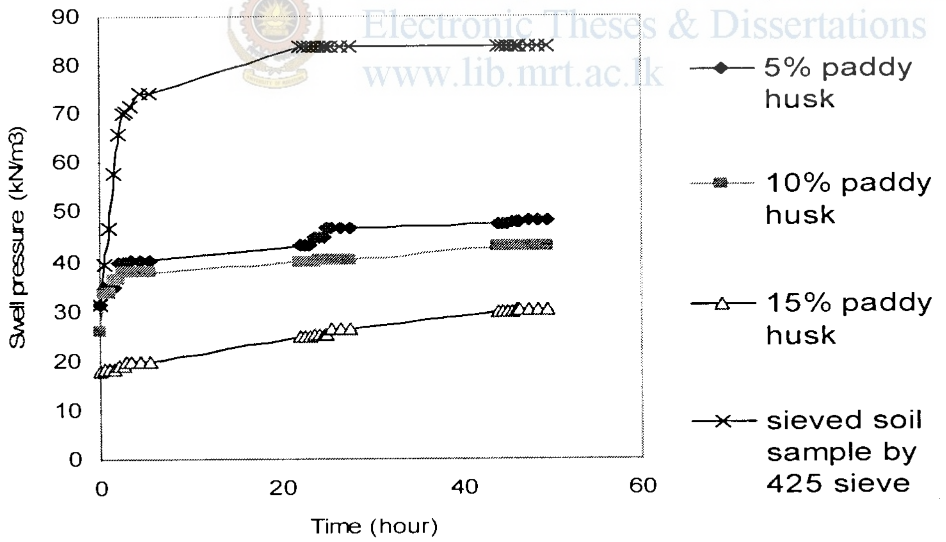


Figure 5.4. Variations of swell pressure with time for sieved soil sample by 425 mm sieve with different mix proportion of paddy husk ash.

Chapter 6

Reduction of the effect on structures by enhancement of Structural stiffness

6.1. Introduction

It would be possible to minimize structural distress shown by formation of cracks etc in lightly loaded structures constructed on expansive soils, by enhancing the structural stiffness. This will cause the structure to deform as one unit and differential deformations (Uplift) could be minimized. This would be less costly than the other structural solution of transferring the loads to an underlaying stratum that is not expansive. In this project the effect of enhanced foundation stiffness was studied with the use of the SAP 2000 computer package. SAP 2000 performs a plane stress analysis.

6.2. Basic frame work of the Finite Element analysis of SAP 2000

The SAP 2000 is a finite element analysis package which can be used to analyze buildings, bridges, walls, foundations and water retaining structures etc. very easily. This chapter presents the critical forces / stresses developed in a wall panel due to the application of the different swell pressures on the foundation, while varying the foundation type. Swell pressures of 80 kN/m^2, 150kN/m^2 and 250 kN/m^2, were considered in the study.

In the finite element mesh is used to create the wall panel and rubble foundation. The reinforced concrete strip and inverted tee foundation were idealized as beam elements of appropriate stiffness.

The nodes at the left and right boundaries which are connected to the cross walls were taken as pin jointed.

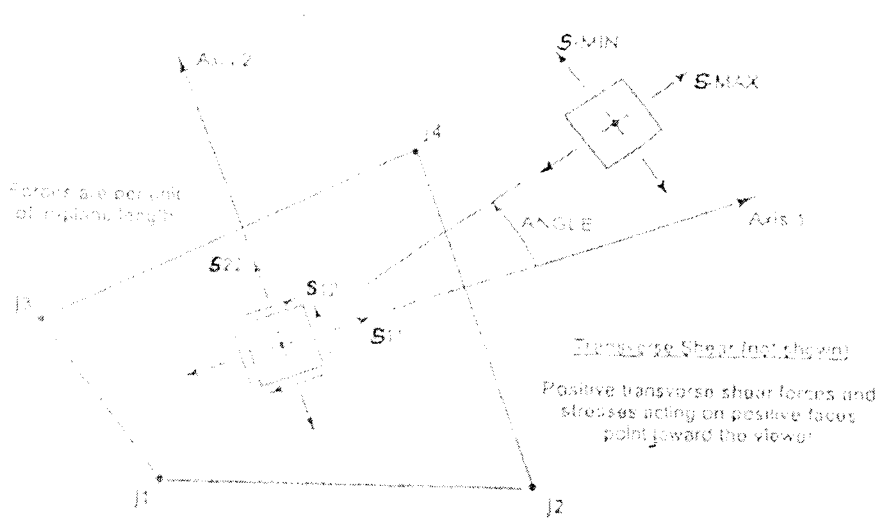


Figure 6.1 Stresses defined in the sap2000

The stresses computed in this analysis are defined in Figure 6.1. The stresses are evaluated at the standard two by two Gauss integration points of the element and extrapolated to the joints. Although they are reported at the joints, the stresses exist throughout the element. Following different foundation options were taken into analysis using above programme.

1. Rubble foundation and brick wall
2. Rubble foundation on a reinforced concrete strip footing and brick wall
3. Inverted Tee Reinforced concrete foundation and brick wall

SAP 2000; Structural finite element analysis was done for the selected types of foundations on a wall panel with a door and window. Swell pressure of the order of 80 kN/m^2, 150 kN/m^2 and 250 kN/m^2 were applied at the base of the foundation. In this analysis, grade 30 concrete, brick work, and ruble masonry are the major materials and they were represented by the use of appropriate material properties. Material properties chosen are given in Table 6.1. Graphic object can be used to represent geometry, boundary and loads. All the materials were assumed to be liner elastic.

Figure 6.2 coordinate systems defined in SAP2000

All coordinate systems are three dimensional, right handed, rectangular (Cartesian systems). SAP 2000 always assumed that Z is the vertical axis with +Z being upward, X-Y plane is horizontal. For the tension limit, it must be Zero to positive value.

Table 6.1. Selective material properties used in above analysis (After Reynold 1993)

	Concrete	Rubble	Brick
Mass per unite volume $*10^3$ - kg/m^3	2.4	2.6	1.48
Weight per unite volume --kN/m^3	23.56	25.5	18.05
Modulus of elasticity--kN/m^2	$24.8x10^6$	$19.0x10^6$	$14.0x10^6$
Poison's ratio	0.3	0.2	0.35

The three types of foundation systems taken for the analysis are depicted in Figure 6.3. On each foundation, a 3m height brick wall with 150mm wall thickness was with a widow opening of 1200mm x 1500mm and a door opening of 900mm x 2100mm considered.

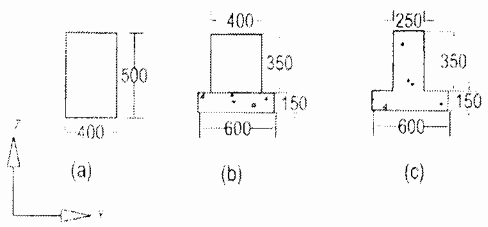


Figure 6.3 Three types of Foundation systems, (a) Rubble masonry wall foundation (b) Rubble masonry wall foundation with reinforced concrete (c) Inverted tee concrete foundation.

6.3. Results of Finite Element analysis of SAP 2000

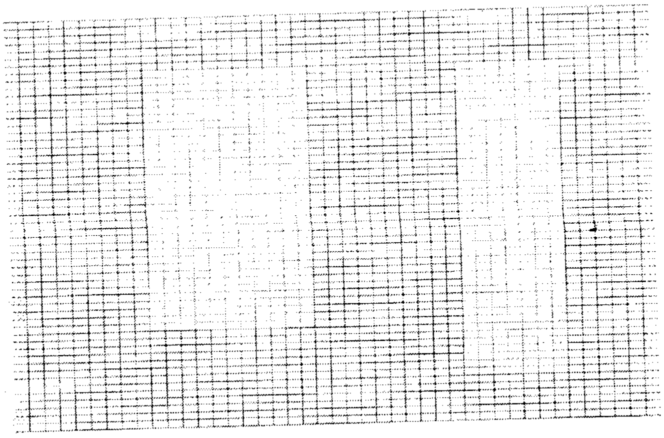


Figure 6.4. mesh elements X-Z plane defined for the sap 2000 Analysis.

First mesh elements were defined to be of required size of 50mm x100mm. Thus the material properties, area sections, groups of areas and load cases were also defined. Subsequently, all defined properties, joints, joint loads and areas assigned to the mesh and the case was analyzed. Details of the walls panel analyzed are presented in Figure 6.4. The notation of stresses in an element is presented in Figure 6.5. The out puts of SAP 2000 are presented in Figure 6.6 to Figure 6.14.

Tensile stresses on wall for each foundation systems are presented from Figure 6.6 to Figure 6.14. These tensile stress zones confirms well with the observed zones of cracks in this single storied building considered in this project.

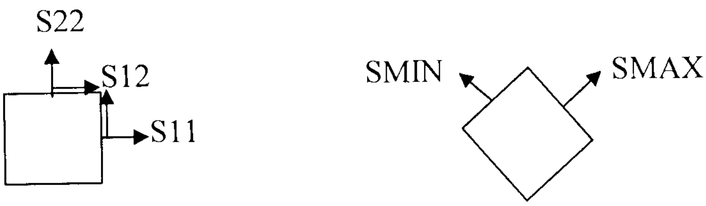


Figure 6.5. Horizontal stress (S11), vertical stress (S22), minimum principal stress (SMIN) and maximum principal stress (SMAX) as indicated in SAP2000.

Variations of tension Stress distribution (kN/m^2) in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX) and minimum principal stress (SMIN) are plotted in Figure 6.6, to Figure 6.8 for foundation type (a), (b) and (c) respectively, when swell pressure is 80 kN/m^2.

For swell pressure 150 kN/m^2, Variations of tension Stress distribution (kN/m^2) in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX) and minimum principal stress (SMIN) are plotted in Figure 6.9, to Figure 6.11 for foundation type (a), (b) and (c) respectively.

Figure 6.12, Figure 6.13 and Figure 6.14 for same foundation types are illustrated Variations of tension Stress distribution (kN/m^2) in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX) and minimum principal stress (SMIN) for swell pressure 250 kN/m^2 respectively.

In selected problematic building in Hambanthota and Anuradhapura considerable cracks could be observed at corner and parallel to vertical edge of doors and windows. Thus the regions of higher computed tensile stresses corresponds with the zones where cracks were observed in the structure. Therefore, above exercise has proved the common characteristic of problematic buildings in Hambanthota and Anuradhapura.

The results indicated that the zones of tensile stresses would reduce significantly with the introduction of inverted tee type foundation. There was not much of a reduction with only a reinforced concrete strip foundation at the base. With the increase of the applied swell pressure, tensile stresses extended over a larger region.

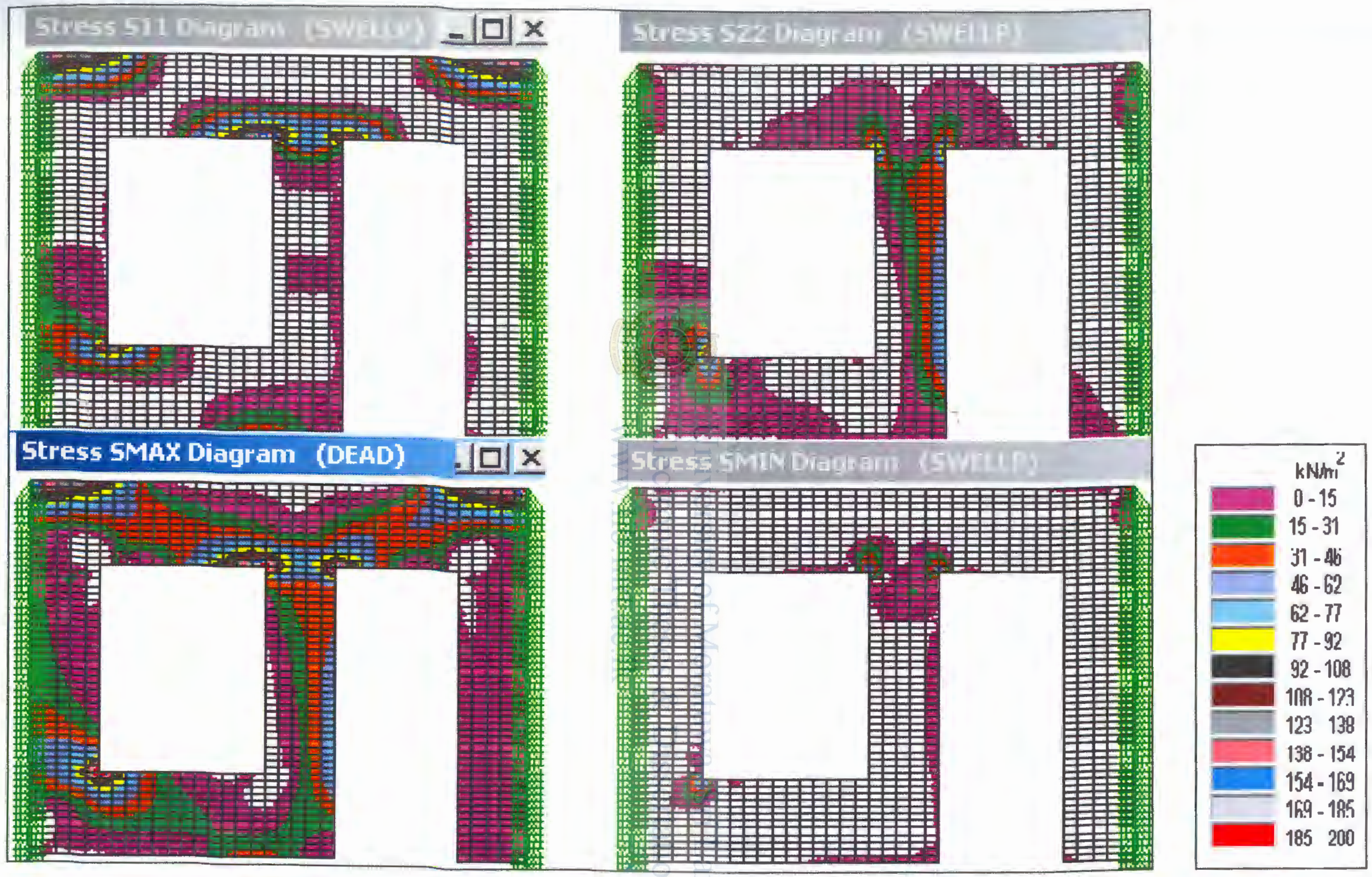


Figure 6.6. Stress distribution in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX) and minimum principal stress (SMIN) for rubble foundation with brick wall when swell pressure is 80 kN/m^2.

Considered only Tensile Stresses.

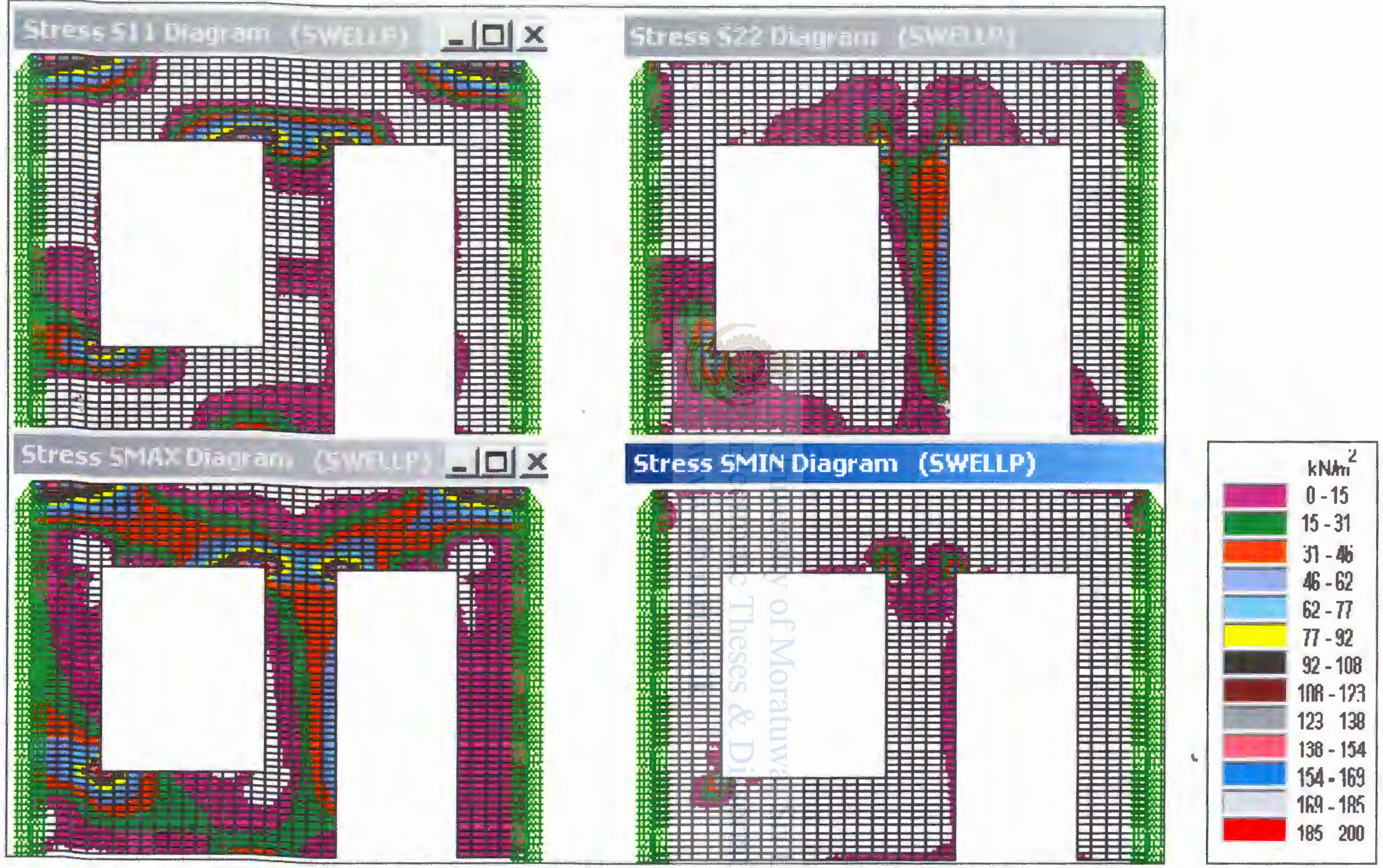


Figure 6.7 Stress distribution in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX) and minimum principal stress (SMIN) for rubble masonry wall foundation on reinforced concrete base with brick wall when swell pressure is 80 kN/m^2.

Considered only Tensile Stresses.

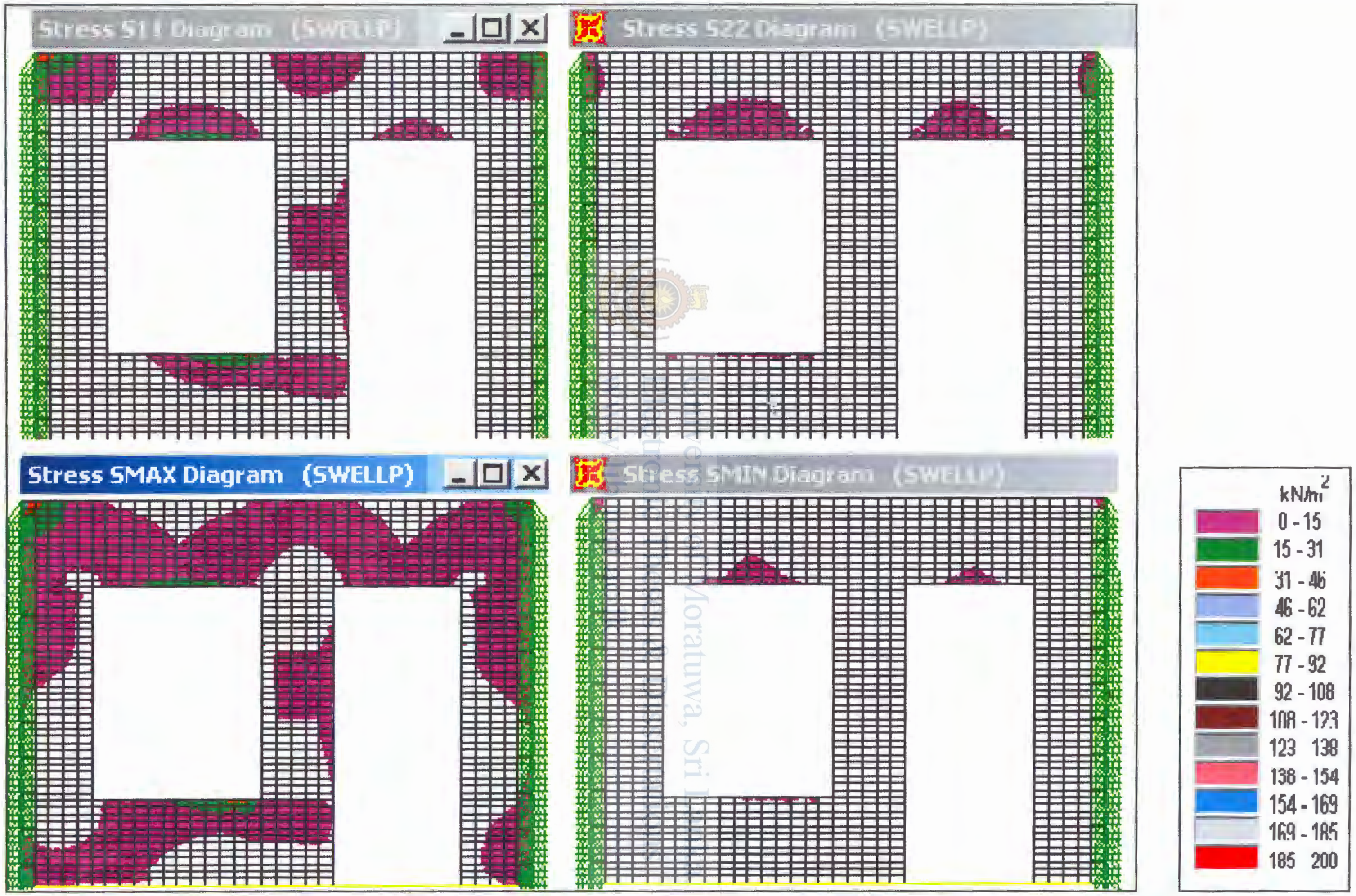


Figure 6.8. Stress distribution in horizontal direction (S11), vertical direction (S22), maximum principal stress (SMAX)and minimum principal stress (SMIN) for inverted tee reinforced concrete strip footing with brick wall when swell pressure is 80 kN/m^2.

Considered only Tensile Stresses.

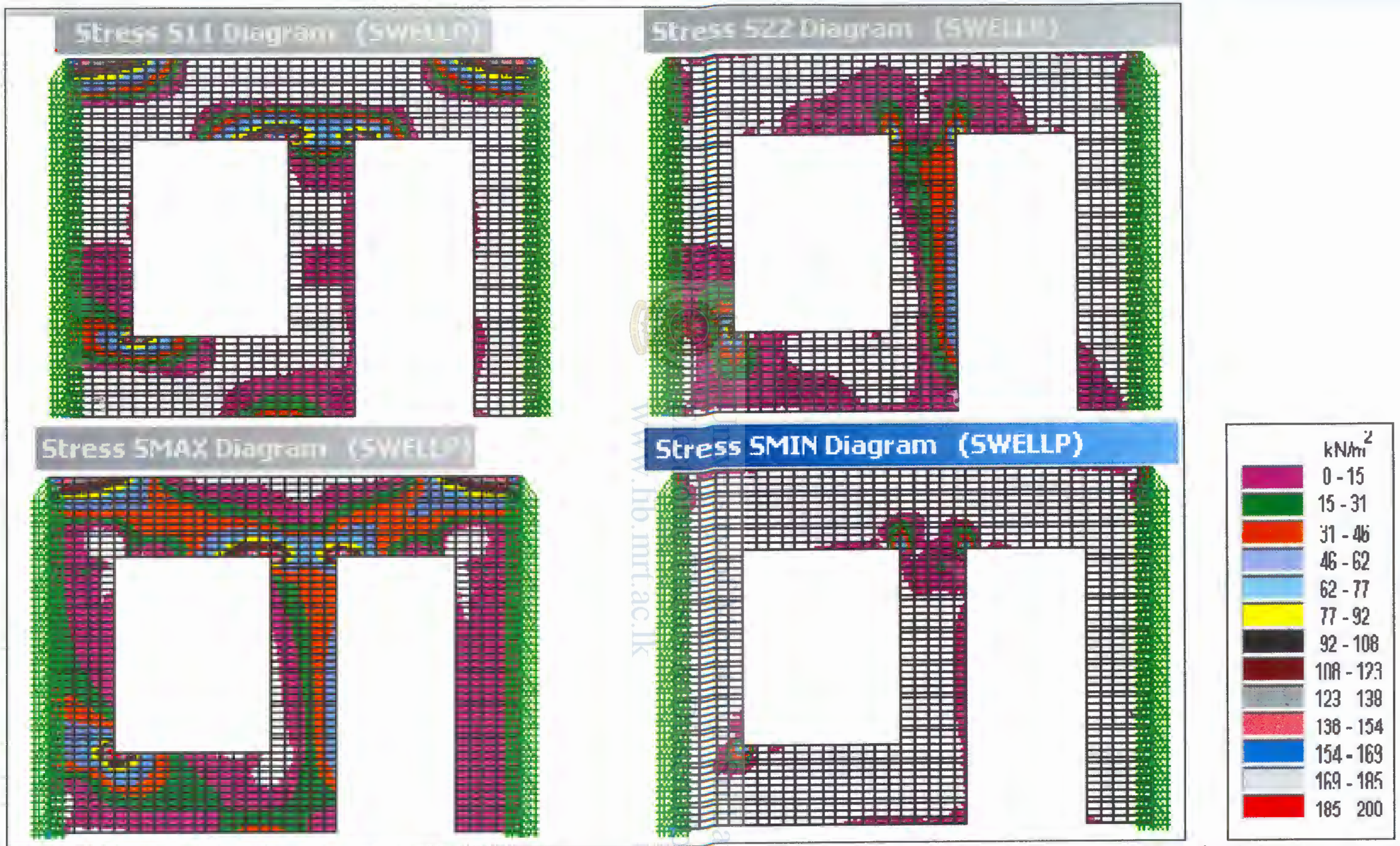


Figure 6.9.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX) and minimum stress (SMIN) for rubble foundation with brick wall when swell pressure is 150 kN/m^2.

Considered only Tensile Stresses.

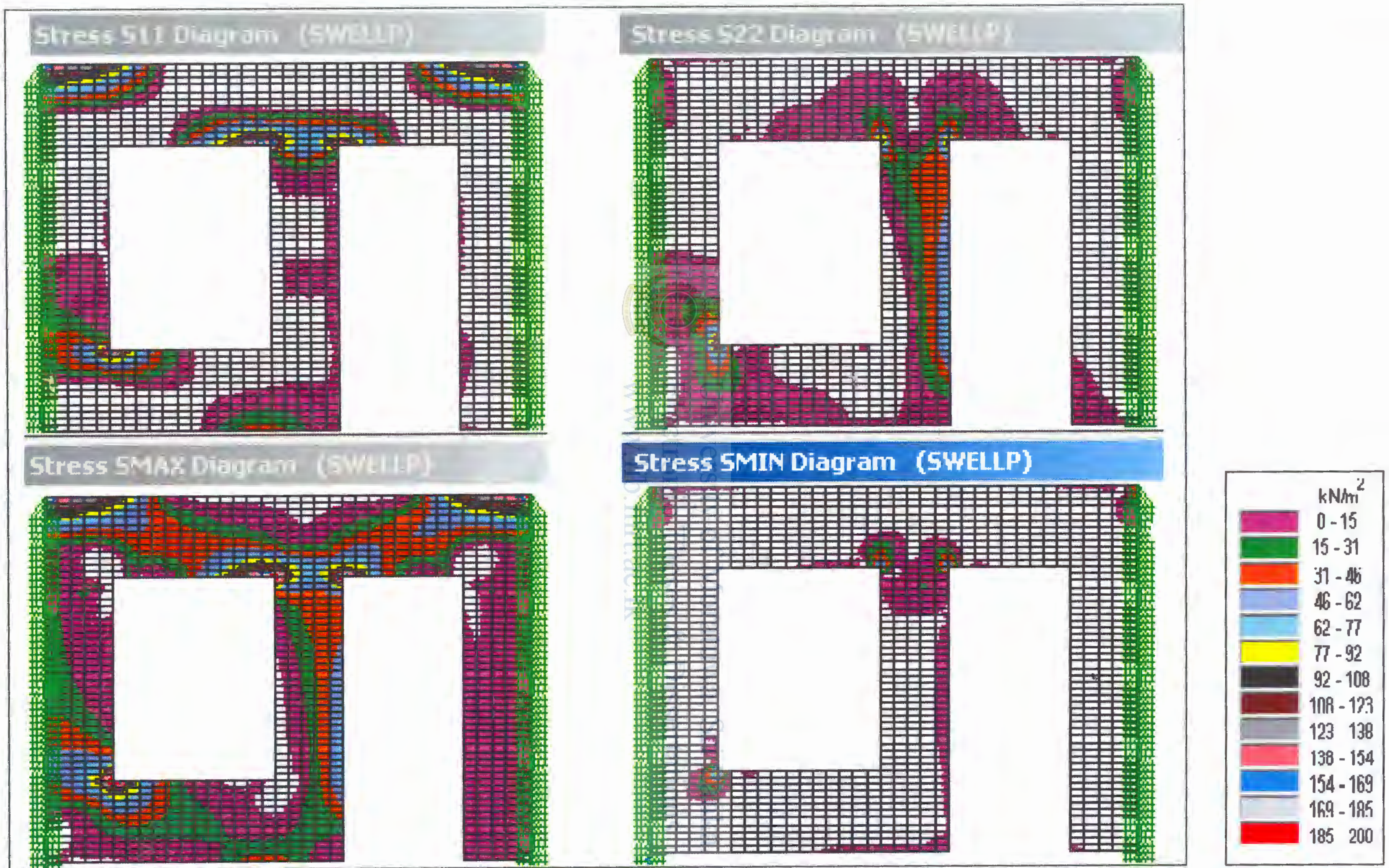


Figure 6.10.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX) and minimum stress (SMIN)for rubble masonry wall foundation on reinforced concrete base with brick wall when swell pressure is 150 kN/m^2.

Considered only Tensile Stresses.

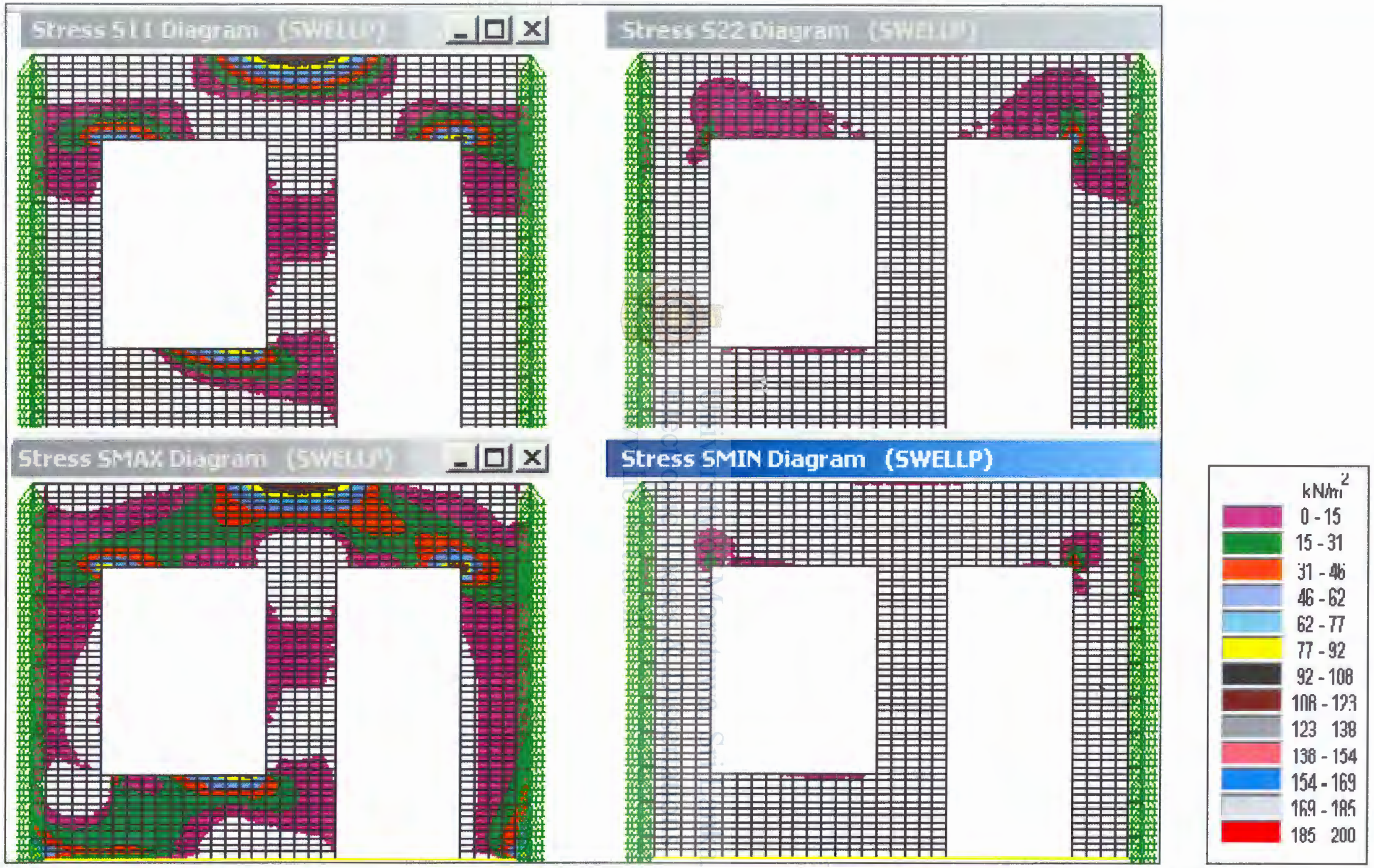


Figure 6.11.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX)and minimum stress (SMIN)for inverted tee reinforced concrete strip footing with brick wall when swell pressure is 150 kN/m^2.

Considered only Tensile Stresses.

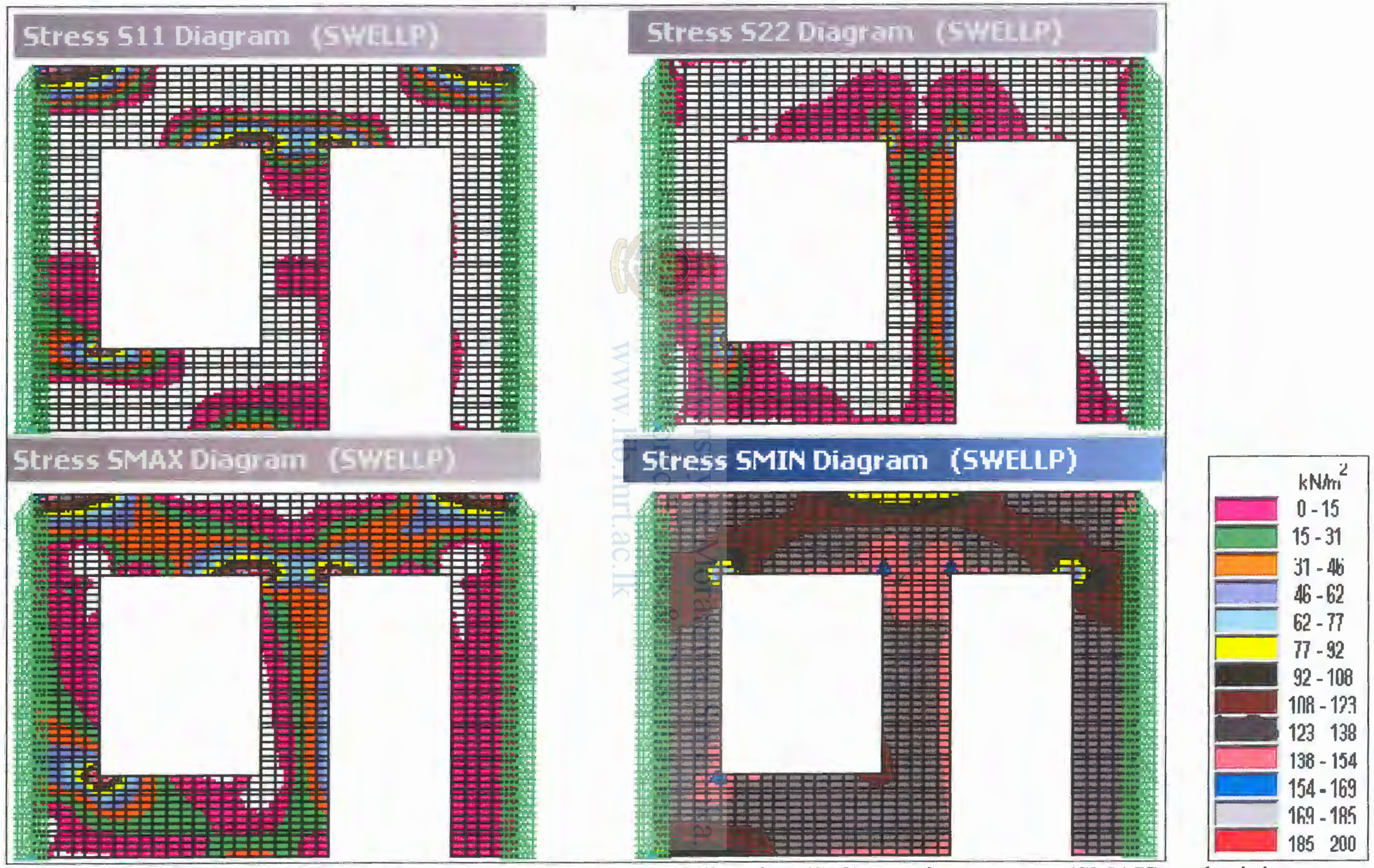


Figure 6.12.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX) and minimum stress (SMIN) for rubble foundation with brick wall when swell pressure is 250 kN/m^2.

Considered only Tensile Stresses.

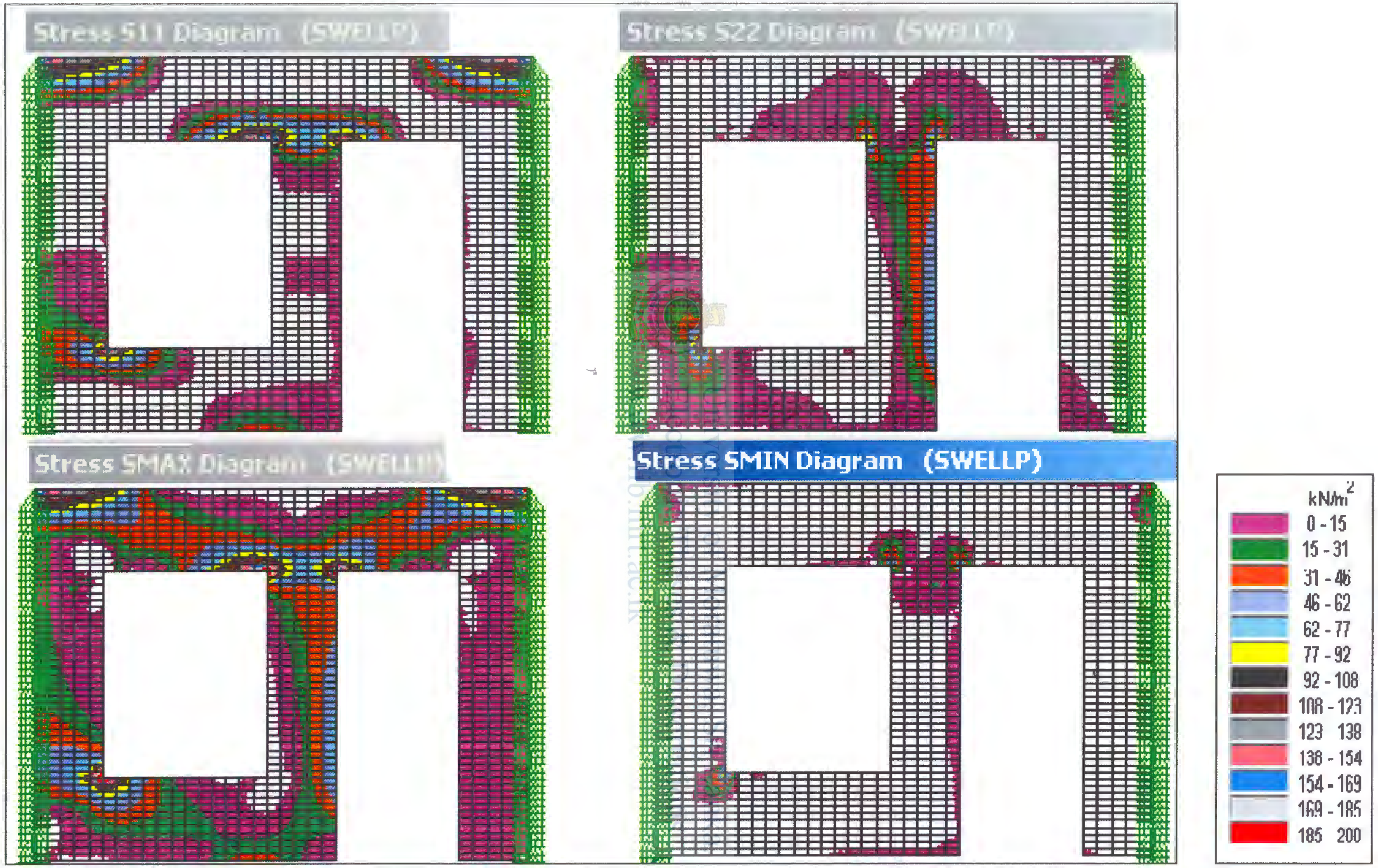


Figure 6.13.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX) and minimum stress (SMIN) for rubble masonry wall foundation on reinforced concrete base with brick wall when swell pressure is 250 kN/m^2.

Considered only Tensile Stresses.

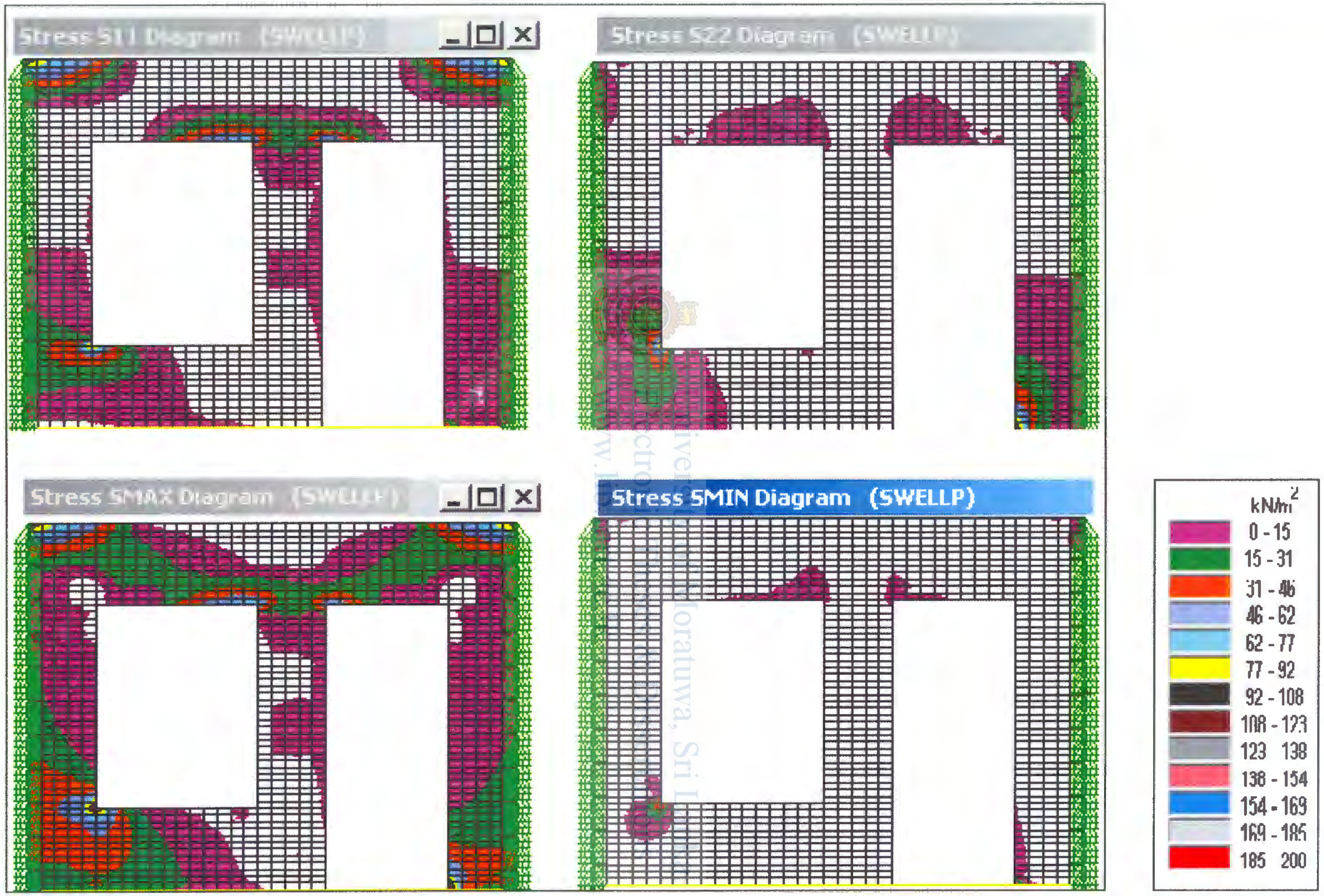


Figure 6.14.stress distribution in horizontal direction (S11), vertical direction (S22), maximum stress (SMAX) and minimum stress (SMIN) for inverted tee reinforced concrete strip footing with brick wall when swell pressure is 250 kN/m^2.

Considered only Tensile Stresses.

Brick elements will possess some tensile strength, which could be quite small. Any details of the tensile stresses are not available. However, tensile stresses goes to zero when swell pressure is equal to the total weight of foundation, wall and roof. It is illustrated in Table 6.2

Table 6.2. Pressure under foundation due to self weight of wall, foundation and roof.

Foundation type	Pressure under foundation kN/m^2
(a)	24.0
(b)	17.5
(c)	15.0

Summary and Conclusions

7.1.Summary

Hambanthota and Anuradhapura are considered to be two districts in arid zones of Sri Lanka. There are number of reported cases of building with structural distress in these districts. Study of samples obtained from four sites in these districts indicated that the soils in these sites are expansive. The soil from the Hambanthota site was the most expansive.

Expansive soils are characterized by high liquid limits and high plasticity index. Expansive soils remain plastic over very high range water content and have a high affinity for water. They also possess a very high activity.

Liquid limit, plastic limit and activity are three parameters that could be obtained by simple laboratory tests conducted on disturbed samples of soils. Swell pressure test and free swell tests are two tests that could directly indicate the effect of expansiveness. Ideally these tests should be performed on undisturbed samples. However, in this project these tests were performed on samples remoulded to the estimated field density.

7.2.Conclusions

Many different approaches can be adopted to minimize the effects of expansive soils in the structure. In this project the effects of the use of locally available inexpensive additive paddy husk ash, and the improvement of structural stiffness was studied.

It was found that mixing of paddy husk ash of about 15% caused a significant reduction of the swell pressure. In this application, natural soil below the foundation should be removed to some depth and replace with compacted soil/paddy husk ash mix.

Analysis with SAP 2000 showed that the use of a more stiff foundation system can minimize the structural distress in lightly loaded structures, where the problem is quite critical.

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A.1 Laboratory testing

A.1.1. Swell pressure

	Load come back to constant reading (g)			
Time (min)	Mithreegama Tsunami Housing project Hambanthota	Aurvedic hospital Anuradhapura	Sahana Chidran home Anuradhapura	Post harvesting institute Anuradhapura
0	0	0	0	0
1	170	120	18	35
4	206	218	128	159
8	-	201	130	76
15	229.9	200.8	160	121
30	-	474	-	100
45	-	-	179	11
60	235	319	42	6
90	110	58.7	39	4
210	446	20.9	119.5	31
270	49	79	2	10
330	80	165	39	20
360	82	93	15	3
Total (g)	1607.9	982.4	876.5	5 76
Swell pressure kN/m^2	80.4	49.12	43.83	28.8
Density g/cm^3	1.56	1.57	1.59	1.55

Table. A.1.1. swell pressures in all sites

Particle Size Analysis			Wet Sieve Analysis		
Total Mass taken (g) = 500 g			Project: Mithree gama tsunami houses Hambanthota		
Sieve Number	Sieve Size mm	Mass of Soil (g)	Cumulative mass (g)	Percent retained %	Percent finer %
2.36mm	2.36	0.88	0.88	0.18	99.82
1.18mm	1.18	37.98	38.86	7.77	92.23
600um	0.6	44.38	83.24	16.65	83.35
425um	0.425	20.08	103.32	20.66	79.34
300um	0.3	20.58	123.9	24.78	75.22
150um	0.15	54.88	178.78	35.76	64.24
75um	0.075	45.48	224.26	44.85	55.15
Total Mass taken (g) = 500 g			Project: Aurvedic Hospital Anuradhapura		
Sieve Number	Sieve Size mm	Mass of Soil (g)	Cumulative mass (g)	Percent retained %	Percent finer %
2.36mm	2.36	0.4	0.4	0.2	99.8
1.18mm	1.18	38.5	38.9	7.78	92.22
600um	0.6	40.12	79.02	15.80	89.20
425um	0.425	21.3	100.32	20.06	79.94
300um	0.3	30.7	131.02	26.20	73.8
150um	0.15	55.3	186.32	37.26	62.74
75um	0.075	49.76	223.03	44.6	55.4
Total Mass taken (g) = 500 g			Project: post harvesting institute Anuradhapura		
Sieve Number	Sieve Size mm	Mass of Soil (g)	Cumulative mass (g)	Percent retained %	Percent finer %
2.36mm	2.36	0.9	0.9	0.18	99.82
1.18mm	1.18	12.71	13.61	2.72	97.28
600um	0.6	30.56	44.17	8.83	91.17
300um	0.3	28.42	72.59	14.52	85.48
150um	0.15	31.3	103.89	20.78	79.22
75um	0.075	21.4	125.29	25.06	74.94

Table is cont...

Total Mass taken (g) = 500 g			Project: Sahana children home Anuradhapura		
Sieve Number	Sieve Size mm	Mass of Soil (g)	Cumulative mass (g)	Percent retained %	Percent finer %
2.36mm	2.36	0.4	0.4	0.08	99.92
1.18mm	1.18	14.3	18.3	3.66	96.34
600um	0.6	28.5	46.8	9.36	90.64
300um	0.3	30.9	77.7	15.54	84.46
150um	0.15	32.1	109.8	21.96	78.04
75um	0.075	18.5	128.31	25.66	74.34

Table A. 1.2.wet sieve analysis for selective sites

University of Moratuwa – Department of Civil Engineering

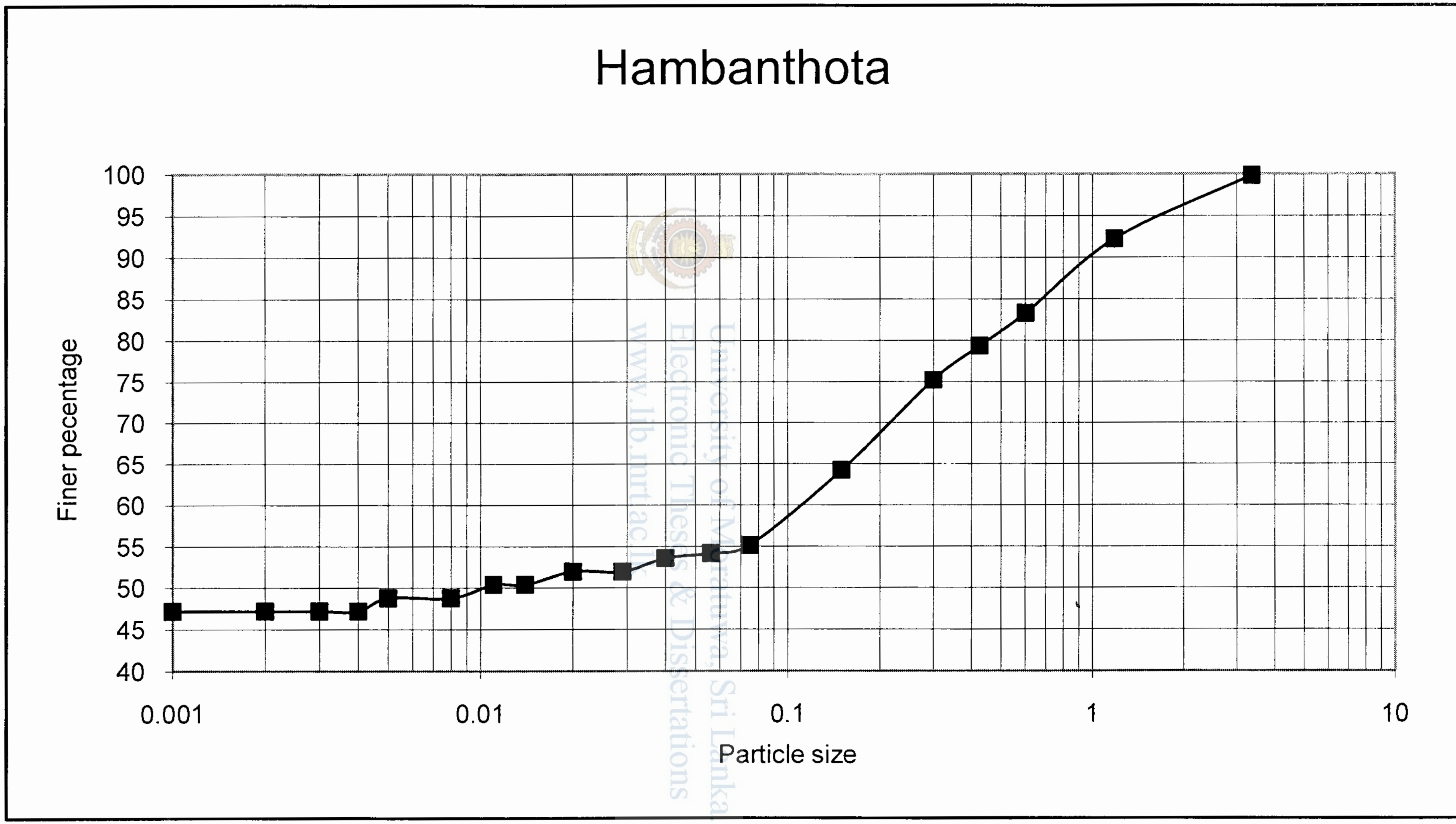
Hydrometer Analysis of Soils

Project: Hambanthota

Weight of Sample (g) = 50g	T(C)=30^o
Hydrometer Test :	Gs=2.67
Hydrometer No:	K=0.01218
Meniscus Correction C_m =+0.5	‘a =0.995
Dispersing Agent Correction =+2	Cylinder No=

Bore Hole No : = | Depth of Sample = | Date =

Day	Time	Time after start	Time (min)	Temp. C	R_H	RH= $R^1_H+C_m$	L	√ L/t	D (mm)	R= R_H-C_D	%Finer P= (R/w)xax 100%	Modified %
14.12.6			0.15	30	35	35.5	10.5	8.37	0.102	33.5	66.67	53.34
			0.25	30	34	34.5	10.6	6.51	0.079	32.5	64.68	51.74
			0.5	30	33	33.5	10.7	4.63	0.056	31.5	62.69	50.15
			1	30	32	32.5	11	3.32	0.04	30.5	60.7	48.56
			2	30	31	31.5	11.2	2.37	0.029	29.5	58.71	46.97
			4	30	31	31.5	11.2	1.67	0.02	29.5	58.71	46.97
			8	30	30	30.5	11.3	1.19	0.014	28.5	56.72	45.38
			15	30	30	30.5	11.3	0.87	0.011	28.5	56.72	45.38
			30	30	29	29.5	11.5	0.62	0.008	27.5	54.725	43.78
			60	30	29	29.5	11.5	1.44	0.005	27.5	54.725	43.78
			120	30	28	28.5	11.6	0.31	0.004	26.5	52.735	42.19
			240	30	28	28.5	11.6	0.22	0.003	26.5	52.735	42.19
			360	30	28	28.5	11.6	0.18	0.002	26.5	52.735	42.19
			1005	30	28	28.5	11.6	0.11	0.001	26.5	52.735	42.19
			1210	30	27	27.5	11.8	0.1	0.001	25.5	50.745	40.60
			1440	30	26	26.5	11.95	0.09	0.0009	24.5	50.745	40.60
			2490	30	26	26.5	11.95	0.07	0.0008	24.5	48.755	39.00


Hambanthota
100
95
90
85
80
75
70
65
60
55
50
45
40
Finer pecentage
0.001
0.01
0.1
1
10
Particle size

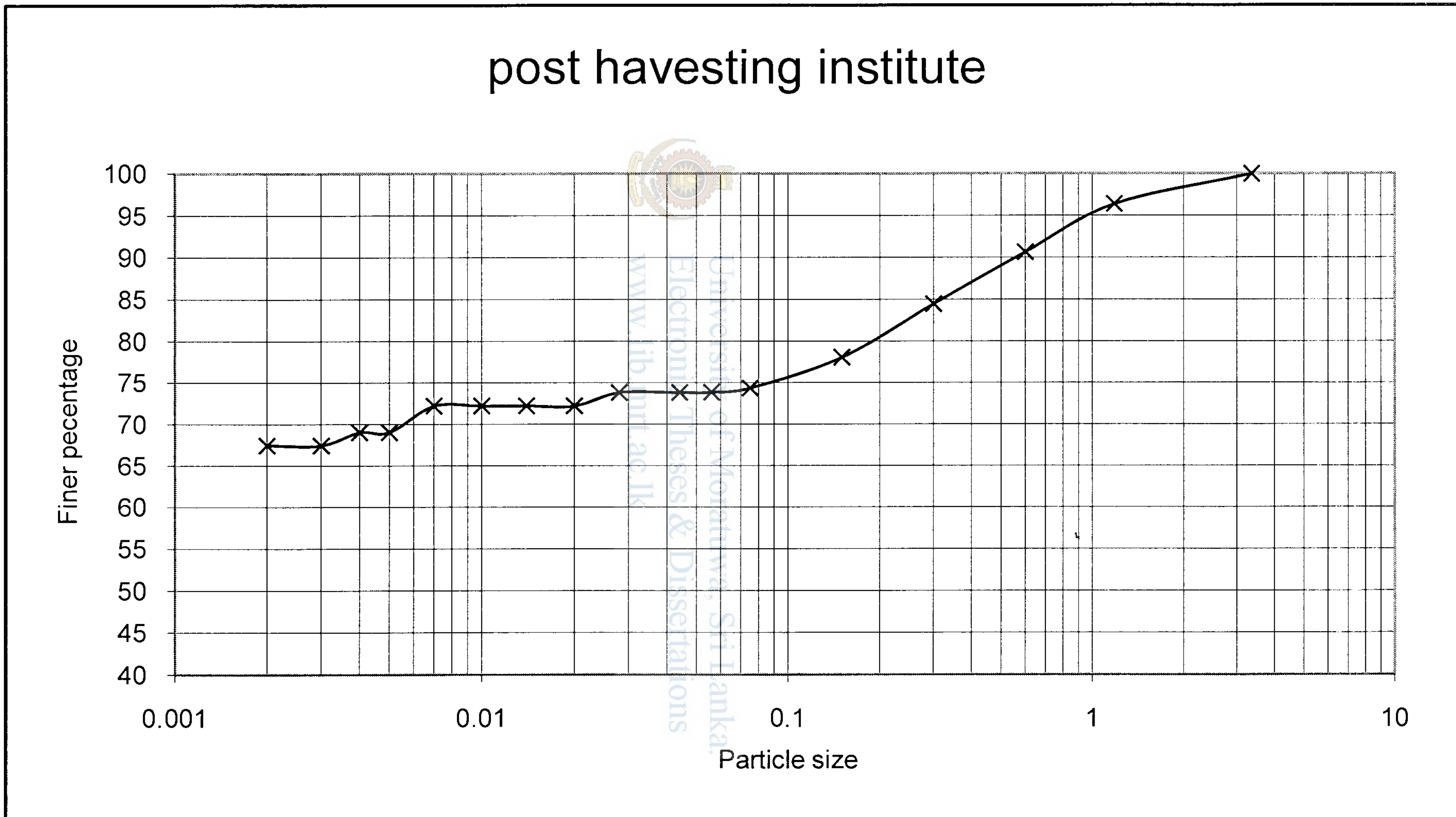
University of Moratuwa – Department of Civil Engineering

Hydrometer Analysis of Soils

Project: Post Havesting Institute Anuradhapura.

Weight of Sample (g) = 50g	T(C)=30^o
Hydrometer Test :	Gs=2.67
Hydrometer No:	K=0.01218
Meniscus Correction C_m =+0.5	‘a =0.995
Dispersing Agent Correction =+2	Cylinder No=
Bore Hole No : = Depth of Sample =	Date =

Day	Time	Time after start	Time (min)	Temp. C	R_H	RH= $R^1_H+C_m$	L	√ L/t	D (mm)	R= R_H-C_D	%Finer P= (R/w)xax 100%	Modified %
14.12.6			0.15	28	34	34.5	10.6	8.4	0.106	34.3	68.06	54.45
			0.25	28	34	34.5	10.6	5.9	0.075	34.3	68.06	54.45
			0.5	28	34	34.5	10.6	4.6	0.058	34.3	68.06	54.45
			1	28	33	33.5	10.7	3.3	0.042	33.3	66.07	52.86
			2	28	33	33.5	10.7	2.3	0.029	33.3	66.07	52.86
			4	28	32	32.5	11	1.7	0.024	32.3	64.09	51.27
			8	28	32	32.5	11	1.2	0.015	32.3	64.09	51.27
			15	28	31	31.5	11.2	0.9	0.011	31.3	63.11	50.49
			30	28	31	31.5	11.2	0.6	0.008	31.3	63.11	50.49
			60	28	31	31.5	11.2	0.4	0.005	31.3	63.11	50.49
			120	28	30	30.5	11.3	0.3	0.004	30.3	60.12	48.10
			240	28	29	29.5	11.5	0.2	0.003	29.3	58.14	46.51
			360	28	29	29.5	11.5	0.2	0.003	29.3	58.14	46.51
			1005	28	29	29.5	11.5	0.1	0.0014	29.3	58.14	46.51
			1210	28	28	28.5	11.6	0.1	0.0014	28.3	56.15	44.92
			1440	28	26	26.5	11.95	0.09	0.0013	26.3	52.18	41.74
			2490	28	26	26.5	11.95	0.06	0.0013	26.3	52.18	41.74



post havesting institute
Finer pecentage
100
95
90
85
80
75
70
65
60
55
50
45
40
0.001
0.01
0.1
1
10
Particle size

University of Moratuwa – Department of Civil Engineering

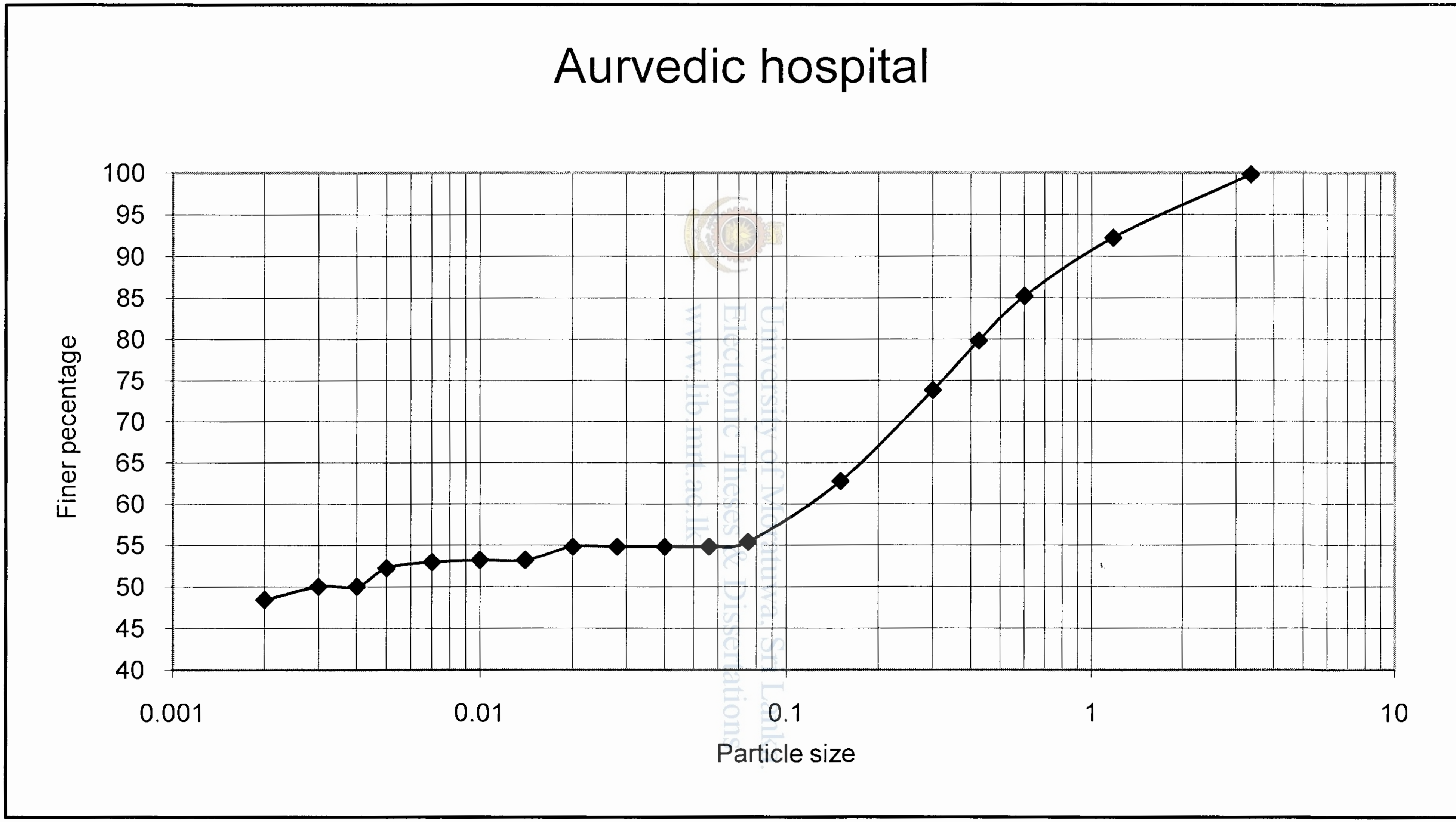
Hydrometer Analysis of Soils

Project: Aurvedic Hospital Anuradhapura

Weight of Sample (g) = 50g	$T(C)=30^{o}$
Hydrometer Test :	Gs=2.67
Hydrometer No:	K=0.01218
Meniscus Correction C_m =+0.5	‘a =0.995
Dispersing Agent Correction =+2	Cylinder No=

Bore Hole No : =   Depth of Sample =   Date =

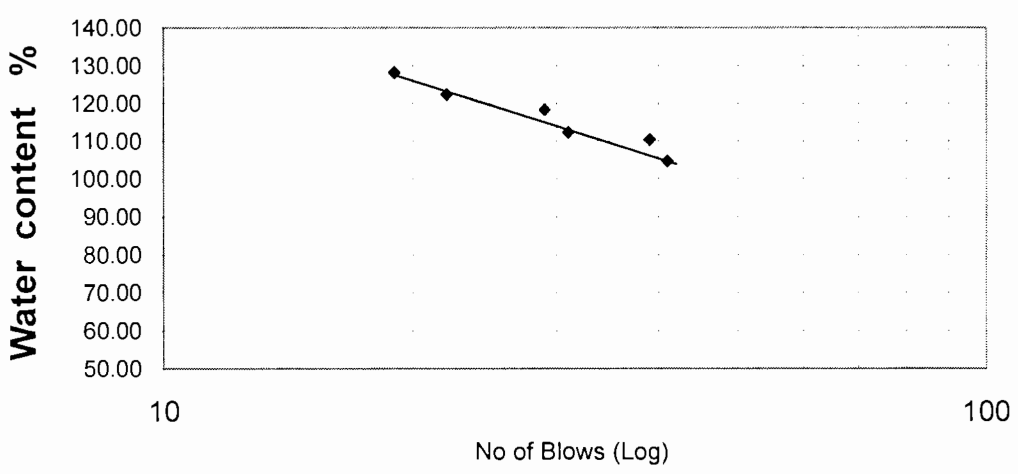
Day	Time	Time after start	Time (min)	Temp. C	R_H	RH= $R^1_H+C_m$	L	√ L/t	D (mm)	R= R_H-C_D	%Finer P= (R/w)xax 100%	Modified %
18.12.6			0.15	30	34	34.5	10.6	4.6	0.056	32.5	64.79	51.83
			1	30	34	34.5	10.6	3.26	0.04	32.5	64.75	51.80
			2	30	34	34.5	10.6	2.3	0.028	32.5	64.75	51.80
			4	30	33	33.5	10.7	1.64	0.02	31.5	62.75	50.20
			8	30	33	33.5	10.7	1.16	0.014	31.5	62.75	50.20
			15	30	33	33.5	10.7	0.84	0.01	31.5	62.75	50.20
			30	30	33	33.5	10.7	0.6	0.007	31.5	62.75	50.20
			60	30	31	31.5	11.2	0.43	0.005	29.5	58.76	47.01
			120	30	31	31.5	11.2	0.31	0.004	29.5	58.76	47.01
			240	30	30	30.5	11.3	0.22	0.003	28.5	56.77	45.42
			360	30	30	30.5	11.3	0.18	0.002	28.5	56.77	45.42
			900	30	28	28.5	11.6	0.11	0.0013	26.5	52.79	42.23
			1440	30	26	26.5	11.96	0.09	0.001	24.5	48.8	39.04
			2880	30	26	26.5	11.96	0.06	0.0007	24.5	48.8	39.04


Aurvedic hospital
100
95
90
85
80
75
70
65
60
55
50
45
40
Finer pecentage
0.001
0.01
0.1
1
10
Particle size

Soil Mechanics Laboratory - University of Moratuwa						
Atterberg Limit Test Results						
Client :						
Project :						
SampleDetails : Hambanthota				Depth :		
Liquid Limit Test						
Trial Number	Number of Blows	Moisture Can No	Mass of wet soil+ can (g)	Mass of dry soil + can (g)	Mass of can (g)	mc %
1	41	B1	29.81	18.31	7.32	104.64
2	39	B2	18.44	12.76	7.61	110.29
3	31	B8	22.71	14.51	7.21	112.33
4	29	B7	14.38	10.37	6.98	118.29
5	22	B9	20.38	12.78	6.57	122.38
6	19	B3	23.62	14.46	7.31	128.11
tic limit Test Data :						
		C7	10.59	9.72	7.51	39.37
		C9	8.94	8.51	7.4	38.74
		C21	12.06	10.74	7.31	38.48

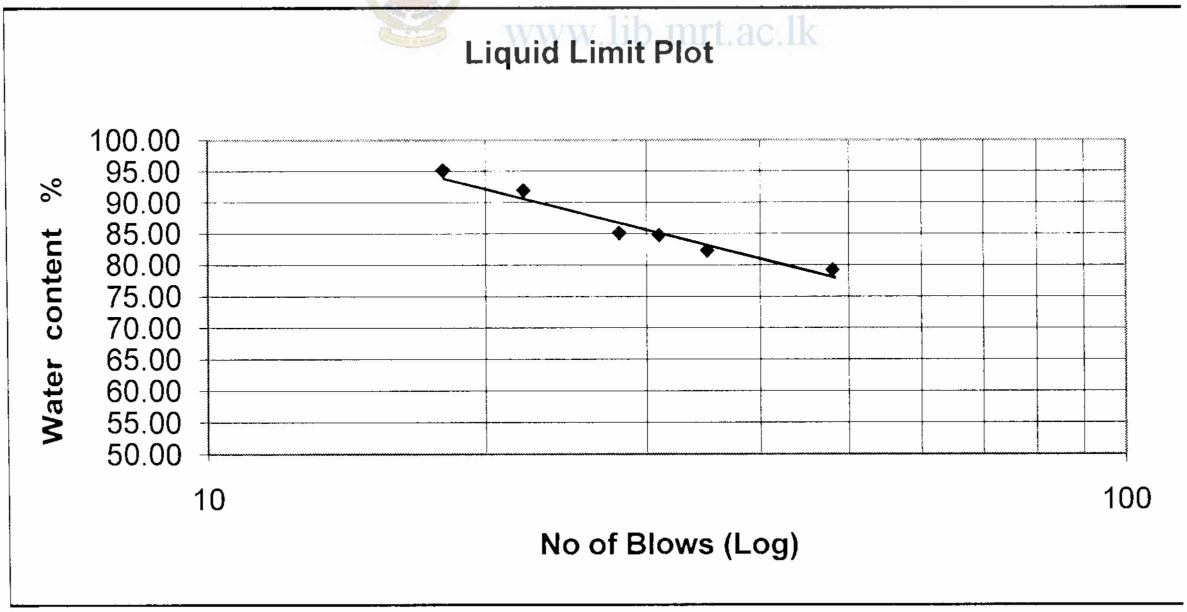
L L % = 120.88 P L % = 38.86 P I % = 82.02

Liquid Limit Plot



Soil Mechanics Laboratory - University of Moratuwa						
Atterberg Limit Test Results						
Client :						
Project :						
SampleDetails : Aurvedic Hospital Anuradhapura				Depth :		
Liquid Limit Test						
Trial Number	Number of Blows	Moisture Can No	Mass of wet soil+ can (g)	Mass of dry soil + can (g)	Mass of can (g)	mc %
1	48	C11	28.21	18.59	6.43	79.11
2	35	C16	11.62	9.63	7.21	82.23
3	31	C20	26.15	17.18	6.59	84.70
4	28	C31	33.96	21.71	7.31	85.07
5	22	C18	31.12	19.57	7	91.89
6	18	C10	21.06	14.31	7.21	95.07
ic limit Test Data :						
		C12	25.99	21.21	7.61	35.15
		C14	20.11	16.72	7.26	35.84
		C16	11.14	10.12	7.31	36.30

L L % = 89.19　　P L % = 35.8　　P I % = 53.4



Soil Mechanics Laboratory - University of Moratuwa

Atterberg Limit Test Results

Client

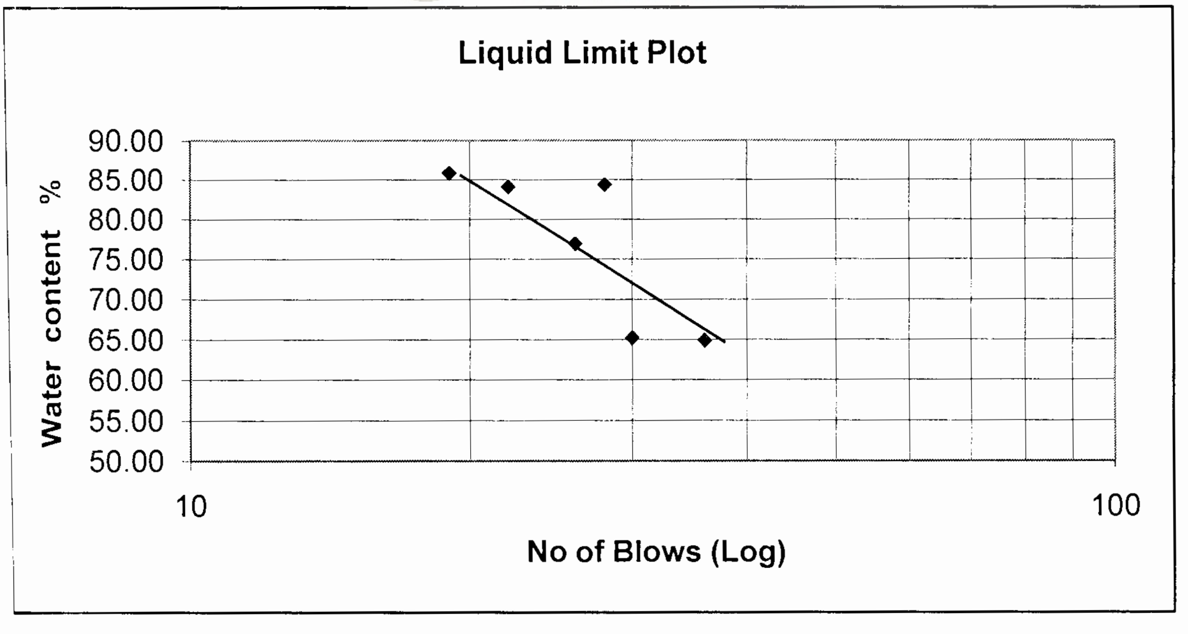
Project

SampleDetails : Post Havesting Inst. Anuradhapura   Depth :

Liquid Limit Test

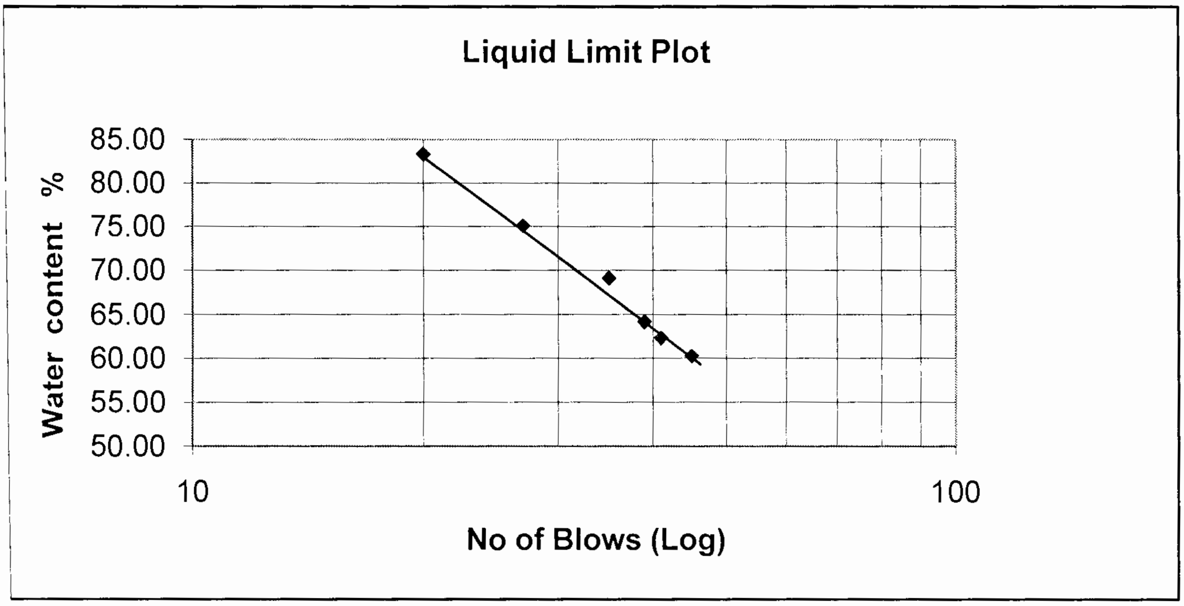
Trial Number	Number of Blows	Moisture Can No	Mass of wet soil+ can (g)	Mass of dry soil + can (g)	Mass of can (g)	mc %
1	36	B1	17.17	13.16	6.98	64.89
2	30	B7	16.18	12.51	6.88	65.19
3	28	B2	17.79	12.89	7.08	84.34
4	26	B8	27.38	18.52	7	76.91
5	22	B9	24.58	16.45	6.78	84.07
6	19	B3	28.91	18.78	6.98	85.85
tic limit Test Data :						
		A1	11.18	10.1	7.08	35.76
		A8	10.18	9.31	6.88	35.80
		A12	12.9	11.5	7.17	32.33

L L % = 79.34   P L % = 34.6   P I % = 44.7



Soil Mechanics Laboratory - University of Moratuwa						
Atterberg Limit Test Results						
Client :						
Project :						
SampleDetails : Sahana Children Home Anuradhapura				Depth :		
Liquid Limit Test						
Trial Number	**Number of Blows**	**Moisture Can No**	**Mass of wet soil+ can (g)**	**Mass of dry soil + can (g)**	**Mass of can (g)**	**mc %**
1	45	B1	18.52	14.31	7.32	60.23
2	41	B7	23.55	17.19	6.98	62.29
3	39	B2	14.22	11.52	7.31	64.13
4	35	B8	17.76	13.45	7.21	69.07
5	27	B9	22.84	16.18	7.31	75.08
6	20	B3	17.2	12.71	7.32	83.30
ic limit Test Data :						
		A1	14.36	12.75	7.31	29.60
		A8	13.55	12.1	7.1	29.00
		A12	18.76	16.18	7.17	28.63

L L % = 77.90 P L % = 29.1 P I % = 48.8



A.2. *Soil improvements using paddy husk.*

A.2.1. Observations of swell pressure of Natural soil sample with burn paddy husk

A.2.1. 1. Mix proportion 5% (soil : burn paddy husk)

Oidometer No: 01
Sample weight 63.49 g
Sample density 16.17 kN/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
9.7.07	8	42	-	630	35.00	0	0	
	9	55	-	630	35.00	0	13	
	10	0	-	769	38.45	0	18	
1st Day	10	40	-	769	38.45	0	58	
	11	40	-	789	39.45	1	58	
	12	10	-	789	39.45	3	8	
	13	00	-	831	41.55	4	8	
	14	10	-	832	41.55	5	18	
10.7.7	8	35	-	937	46.85	23	43	
	8	55	-	937	46.85	24	03	
	9	00	-	937	46.85	24	08	
	9	30	-	938	46.90	24	38	
2nd Day	10	0	-	938	46.90	25	08	
	10	40	-	938	46.90	25	48	
	11	40	-	941	47.05	26	48	
	11	10	-	941	47.05	27	08	
	13	05	-	941	47.05	28	12	
	14	05	-	941	47.05	29	12	
11.7.7	8	40	-	945	47.25	47	47	
	9	5	-	945	47.25	48	12	
	9	30	-	945	47.25	48	37	
3rd Day	10	0	-	945	47.25	49	7	
	10	35	-	945	47.25	49	42	
	11	45	-	945	47.25	50	52	
	12	10	-	947	47.35	51	17	
	13	25	-	947	47.35	52	32	
	14	10	-	947	47.35	53	17	

Table is cont...

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
12.7.7	8	30	-	953	47.65	71	37	
	8	35	-	953	47.65	71	42	
	8	45	-	953	47.65	71	52	
	9	00	-	953	47.65	72	07	
4th Day	9	30	-	953	47.65	72	37	
	10	00	-	953	47.65	73	07	
	10	30	-	953	47.65	73	37	
	11	30	-	953	47.65	74	07	
	12	00	-	954	47.70	74	37	
	13	00	-	954	47.70	75	37	
	14	00	-	954	47.70	76	37	
13.7.7	8	30	-	955	47.75	95	07	
	9	0	-	955	47.75	95	37	
	10	0	-	955	47.75	96	37	
5th Day	11	0	-	955	47.75	97	37	
	12	0	-	955	47.75	98	37	
	13	0	-	955	47.75	99	37	
	14	0	-	955	47.75	100	37	

Table. A.2.1. 1. Measurements of swell pressure for natural soil sample with burn paddy husk [Mix proportion 5% (soil: burn paddy husk)]

A.2.1.2. Mix proportion 10% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 52.63 g
Sample density 13.4 kN/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
8.10.7	8	49	-	518	25.90	0	0	
	8	55	-	518	25.90	0	6	
	9	5	-	768	38.40	0	16	
1st Day	9	30	-	817	40.85	0	41	
	10	0	-	817	40.85	1	11	
	10	40	-	820	41.00	1	51	
	11	40	-	820	41.00	2	51	
	12	0	-	830	41.50	3	11	
	13	10	-	832	41.60	4	21	
	14	10	-	832	14.60	5	21	
9.10.7	8	35	-	837	41.85	23	46	
	9	0	-	837	41.85	24	11	
	9	30	-	837	41.85	24	41	
	10	0	-	837	41.85	25	11	
2nd Day	10	30	-	837	41.85	25	41	
	11	40	-	837	41.85	26	51	
	12	40	-	837	41.85	27	51	
	13	10	-	837	41.85	28	21	
	14	10	-	837	41.85	29	21	
10.10.7	8	40	-	837	41.85	47	51	
	9	5	-	837	41.85	48	16	
	9	30	-	837	41.85	48	41	
	10	00	-	839	41.95	49	11	
3rd Day	10	40	-	839	41.95	49	51	
	11	30	-	839	41.95	50	41	
	12	10	-	839	41.95	51	21	
	13	10	-	839	41.95	52	21	
	14	10	-	839	41.95	53	21	

Table is cont...

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
11.10.7	8	40	-	840	42.0	71	51	
	9	5	-	840	42.0	72	16	
	9	30	-	840	42.0	72	41	
4th Day	10	0	-	840	42.0	73	11	
	10	30	-	840	42.0	73	41	
	11	00	-	840	42.0	74	11	
	12	10	-	840	42.0	75	21	
	13	00	-	840	42.0	76	21	
	14	00	-	840	42.0	77	21	
12.10.7	8	55	-	840	42.0	96	16	
	9	30	-	840	42.0	96	51	
5th Day	10	0	-	840	42.0	97	21	
	10	35	-	840	42.0	97	56	
	11	00	-	840	42.0	98	21	
	12	00	-	840	42.0	99	21	
	13	00	-	840	42.0	100	21	
	14	10	-	840	42.0	101	31	

Table. A.2.1. 2. Measurements of swell pressure for natural soil sample with burn paddy husk [Mix proportion 10% (soil: burn paddy husk)]

A.2.1.3. Mix proportion 15% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 47.62 g
Sample density 12.12 Kn/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
15.10.07	8	40	-	359	17.95	0	0	
	9	0	-	363	18.15	0	20	
	9	10	-	378	18.90	0	30	
	9	30	-	384	19.2	1	0	
1st Day	10	00	-	430	21.5	1	30	
	11	00	-	460	23.0	2	30	
	12	00	-	469	23.45	3	30	
	13	00	-	492	24.6	4	30	
	14	10	-	501	25.05	5	30	
16.10.07	8	30	-	567	28.35	23	55	
	8	55	-	567	28.35	24	20	
	9	15	-	567	28.35	24	40	
	9	30	-	567	28.35	24	55	
2nd Day	10	0	-	567	28.35	25	25	
	10	30	-	573	28.65	25	55	
	11	30	-	573	28.65	26	25	
	12	20	-	573	28.65	27	15	
	13	00	-	587	29.63	27	55	
	14	00	-	587	29.63	28	55	
17.10.07	8	35	-	602	30.1	46	35	
	9	00	-	602	30.1	47	00	
	10	00	-	602	30.1	48	00	
3rd Day	11	00	-	602	30.1	49	00	
	12	00	-	603	30.15	50	00	
	13	00	-	603	30.15	51	00	
	14	00	-	603	30.15	52	00	

Table is cont…

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
18.10.07	8	35	-	609	30.45	70	35	
	9	00	-	609	30.45	71	00	
	9	30	-	609	30.45	71	30	
	10	00	-	609	30.45	72	00	
4th Day	10	30	-	609	30.45	72	30	
	11	00	-	609	30.45	73	00	
	12	00	-	609	30.45	74	00	
	13	00	-	609	30.45	75	00	
	14	00	-	609	30.45	76	00	
19.10.07	8	30	-	611	30.55	94	30	
	9	00	-	611	30.55	95	00	
	9	30	-	611	30.55	95	30	
5th Day	10	0	-	611	30.55	96	00	
	11	0	-	611	30.55	97	00	
	12	0	-	611	30.55	98	00	
	13	0	-	611	30.55	99	00	
	14	0	-	611	30.55	100	00	

Table. A.2.1. 3. Measurements of swell pressure for natural soil sample with burn paddy husk [Mix proportion 15% (soil: burn paddy husk)]

A.2.2. Observations of swell pressure of Oven dry soil sample with burn paddy husk

A.2.2.1.Mix proportion 5% (soil :burn paddy husk)

Oidometer No: 02
Sample weight 62.37 g
Sample density 15.88 kN/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
5.11.07	8	30	-	610	30.5	0	0	
	9	0	-	627	31.35	0	30	
	9	30	-	627	31.35	1	0	
1st Day	10	0	-	692	34.6	1	30	
	10	30	-	692	34.6	2	0	
	11	0	-	692	34.6	2	30	
	12	0	-	711	35.55	3	30	
	13	0	-	765	38.25	4	30	
	14	0		795	39.75	5	30	
6.11.07	8	35	-	882	44.1	22	0	
	9	0	-	882	44.1	22	30	
2nd Day	9	30	-	882	44.1	23	0	
	10	0	-	899	44.95	23	30	
	10	30	-	899	44.95	24	0	
	11	0	-	899	44.95	24	30	
	11	30	-	908	45.4	25	0	
	12	0	-	908	45.4	25	30	
	13	0	-	908	45.4	26	30	
	14	0	-	908	45.4	27	30	
7.11.7	8	30	-	930	46.5	44	0	
	9	0	-	930	46.5	44	30	
	9	30	-	930	46.5	45	0	
	10	0	-	930	46.5	45	30	
3rd Day	10	30	-	942	47.1	46	0	
	11	0	-	942	47.1	46	30	
	12	0	-	942	47.1	47	30	
	13	0	-	942	47.1	48	30	
	14	0	-	942	47.1	49	30	

Table. A.2.2. 1. Measurements of swell pressure for oven dry soil sample with burn paddy husk [Mix proportion 5% (soil: burn paddy husk)]

A.2.2.2.Mix proportion 10% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 51.97 g
Sample density 13.24 Kn/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
12.11.07	8	30	-	505	25.25	0	0	
	9	0	-	505	25.25	0	30	
	9	30	-	597	29.85	1	0	
	10	0	-	597	29.85	1	30	
1st Day	10	30	-	600	30.00	2	0	
	11	0	-	613	30.68	2	30	
	12	0	-	701	35.05	3	30	
	13	0	-	701	35.05	4	30	
	14	0	-	701	35.05	5	30	
13.11.07	8	35	-	811	40.55	22	0	
	9	0	-	811	40.55	22	30	
	9	30	-	811	40.55	23	0	
	10	0	-	811	40.55	23	30	
2nd Day	10	30	-	811	40.55	24	0	
	11	0	-	811	40.55	24	30	
	12	0	-	811	40.55	25	30	
	13	0	-	811	40.55	26	30	
	14	0	-	811	40.55	27	30	
14.11.07	8	35	-	812	40.6	44	0	
	9	0	-	812	40.6	44	30	
	9	30	-	812	40.6	45	0	
	10	0	-	812	40.6	45	30	
3rd Day	10	30	-	812	40.6	46	0	
	11	0	-	812	40.6	46	30	
	12	0	-	812	40.6	47	30	
	13	0	-	812	40.6	48	30	
	14	0	-	812	40.6	49	30	

Table. A.2.2. 1. Measurements of swell pressure for oven dry soil sample with burn paddy husk [Mix proportion 10% (soil: burn paddy husk)]

A.2.2.3.Mix proportion 15% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 46.66 g
Sample density 11.88 Kn/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
19.11.07	8	30	-	359	17.95	0	0	
	9	0	-	359	17.95	0	30	
	9	30	-	359	17.95	1	0	
1st Day	10	0	-	359	17.95	1	30	
	10	30	-	393	19.65	2	0	
	11	0	-	393	19.65	2	30	
	12	0	-	393	19.65	3	30	
	13	0	-	393	19.65	4	30	
	14	0	-	393	19.65	5	30	
20.11.07	8	30	-	497	24.85	22	0	
	9	0	-	497	24.85	22	30	
2nd Day	9	30	-	497	24.85	23	0	
	10	0	-	497	24.85	23	30	
	10	30	-	497	24.85	24	0	
	11	0	-	521	26.05	24	30	
	12	0	-	521	26.05	25	30	
	13	0	-	521	26.05	26	30	
	14	0	-	521	26.05	27	30	
17.11.07	8	30	-	597	29.85	44	0	
	9	0	-	597	29.85	44	30	
	9	30	-	597	29.85	45	0	
	10	0	-	597	29.85	45	30	
3rd Day	10	30	-	597	29.85	46	0	
	11	0	-	597	29.85	46	30	
	12	0	-	597	29.85	47	30	
	13	0	-	597	29.85	48	30	
	14	0	-	597	29.85	49	30	

Table. A.2.2. 1. Measurements of swell pressure for oven dry soil sample with burn paddy husk [Mix proportion 15% (soil: burn paddy husk)]

A.2.3. Observations of swell pressure of sieved soil sample by 425mm sieve with burn paddy husk

A.2.3.1.Mix proportion 5% (soil :burn paddy husk)

Oidometer No: 01
Sample weight 63.99 g
Sample density 16.3 Kn/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
10.9.07	8	30	-	631	31.55	0	0	
	9	0	-	698	34.9	0	30	
	9	30	-	698	34.9	1	0	
1st Day	10	0	-	698	34.9	1	30	
	10	30	-	797	39.85	2	0	
	11	0	-	797	39.85	2	30	
	12	0	-	797	39.85	3	30	
	13	0	-	802	40.1	4	30	
	14	0	-	802	40.1	5	30	
11.9.07	8	35	-	867	43.35	22	0	
	9	0	-	867	43.35	22	30	
2nd Day	9	30	-	867	43.35	23	0	
	10	0	-	867	43.35	23	30	
	10	30	-	897	44.85	24	0	
	11	0	-	897	44.85	24	30	
	11	30	-	897	44.85	25	0	
	12	0	-	938	46.9	25	30	
	13	0	-	938	46.9	26	30	
	14	0	-	938	46.9	27	30	
12.9.07	8	30	-	951	47.55	44	0	
	9	0	-	951	47.55	44	30	
	9	30	-	951	47.55	45	0	
	10	0	-	960	48.0	45	30	
3rd Day	10	30	-	960	48.0	46	0	
	11	0	-	960	48.0	46	30	
	12	0	-	962	48.1	47	30	
	13	0	-	962	48.1	48	30	
	14	0	-	962	48.1	49	30	

Table. A.2.3. 1. Measurements of swell pressure for sieved soil sample by 425mm sieve with burn paddy husk [Mix proportion 15% (soil: burn paddy husk)]

A.2.3.2.Mix proportion 10% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 51.97 g
Sample density 13.24 kN/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
12.11.07	8	30	-	521	26.05	0	0	
	9	0	-	677	33.85	0	30	
	9	30	-	677	33.85	1	0	
	10	0	-	731	36.55	1	30	
1st Day	10	30	-	731	36.55	2	0	
	11	0	-	755	37.75	2	30	
	12	0	-	755	37.75	3	30	
	13	0	-	755	37.75	4	30	
	14	0	-	755	37.75	5	30	
13.11.07	8	35	-	795	39.75	22	0	
	9	0	-	795	39.75	22	30	
	9	30	-	795	39.75	23	0	
	10	0	-	798	39.9	23	30	
2nd Day	10	30	-	801	40.05	24	0	
	11	0	-	801	40.05	24	30	
	12	0	-	801	40.05	25	30	
	13	0	-	801	40.05	26	30	
	14	0	-	801	40.05	27	30	
14.11.07	8	35	-	856	42.8	44	0	
	9	0	-	856	42.8	44	30	
	9	30	-	856	42.8	45	0	
	10	0	-	856	42.8	45	30	
3rd Day	10	30	-	856	42.8	46	0	
	11	0	-	856	42.8	46	30	
	12	0	-	856	42.8	47	30	
	13	0	-	856	42.8	48	30	
	14	0	-	856	42.8	49	30	

Table. A.2.3. 2. Measurements of swell pressure for sieved soil sample by 425mm sieve with burn paddy husk [Mix proportion 10% (soil: burn paddy husk)]

A.2.3.3.Mix proportion 15% (soil : burn paddy husk)

Oidometer No: 03
Sample weight 46.66 g
Sample density 11.88 kN/m^3

Date	Clock Time			Weight (g)	Pressure (kN/m^2)	Time		Remarks
	Hour	Mn.	Sec.			Hour	Mn.	
19.11.07	8	30	-	362	18.1	0	0	
	9	0	-	369	18.45	0	30	
	9	30	-	369	18.45	1	0	
1st Day	10	0	-	369	18.45	1	30	
	10	30	-	386	19.3	2	0	
	11	0	-	386	19.3	2	30	
	12	0	-	395	19.75	3	30	
	13	0	-	395	19.75	4	30	
	14	0	-	395	19.75	5	30	
20.11.07	8	30	-	498	24.9	22	0	
	9	0	-	498	24.9	22	30	
2nd Day	9	30	-	498	24.9	23	0	
	10	0	-	502	25.1	23	30	
	10	30	-	502	25.1	24	0	
	11	0	-	502	25.1	24	30	
	12	0	-	531	26.55	25	30	
	13	0	-	531	26.55	26	30	
	14	0	-	531	26.55	27	30	
17.10.07	8	30	-	598	29.9	44	0	
	9	0	-	598	29.9	44	30	
	9	30	-	598	29.9	45	0	
	10	0	-	598	29.9	45	30	
3rd Day	10	30	-	602	30.1	46	0	
	11	0	-	602	30.1	46	30	
	12	0	-	607	30.35	47	30	
	13	0	-	607	30.35	48	30	
	14	0	-	607	30.35	49	30	

Table. A.2.3. 3. Measurements of swell pressure for sieved soil sample by 425mm sieve with burn paddy husk [Mix proportion 15% (soil: burn paddy husk)]