Chapter I

Introduction

1.1 General

Ground settlement can damage man-made structures such as foundations, pavements, concrete, utilities and irrigation work. Founding structures/services on collapsible soil is one of the reasons of these settlements. According to study of United States Department of Agriculture, one half of the homes in United States are built on collapsible type of soils, and the American Society of Civil Engineers estimate that a quarter of all homes have suffered damage caused by these soils. Collapsible soils are moisture sensitive soils that show large reduction in volume upon wetting. These soils are found all over the world; however, arid to semi arid climatic conditions favour their formation (Al Rawas 2000). Urbanization of arid /semi arid regions increase the possibility of soil being exposed to water.

Risalpur small cantonment in the Nowshera region lies in the semi-arid region, where structural damage due to settlement and cracking of building is a common phenomenon. It is imperative that the cause of structural damage in the region be investigated to plan remedial measures.

1.2 Problem statement

Structural damage to buildings causes an increase in maintenance cost and lowering of serviceability level of the building. The recent of floods in 2010 resulted in failure of structures due to excessive settlement indicating the susceptibility of loss in soil strength due to flooding. It was reported that the allowable bearing capacity in Nowshera region reduced from 1tsf to less than 0.2 tsf. Interestingly, the soil regained its strength on drying. Thus it is important to investigate the cause of sudden loss of soil strength and resulting settlement due to flooding in the area.

1.3 Objectives

The objectives of this project are:-

(a) Characterization of Risalpur soil up to zone of influence of shallow foundations.
(b) Evaluation of collapse potential within the zone of influence of shallow foundations.

1.4 Scope

1.4.1 Limitations

- Characterize soil up to a depth of 20feet
- Carry out all necessary test for which facilities are available in the college laboratory.
- Total load in case of plate load test was restricted due to availability of limited reaction loads.

1.4.2 Scope

- Chapter 1 describes the problem statement, objective, limitation and scope of project
- Chapter 2 describes the literature review of past studies and familiarizes us with the background of collapsible soils and the experimental procedure
- Chapter 3 describes the methodology of test procedures followed
- Chapter 4 discusses the result of the test performed

1.5 Methodology

A general outline of the study is shown in figure 1.1

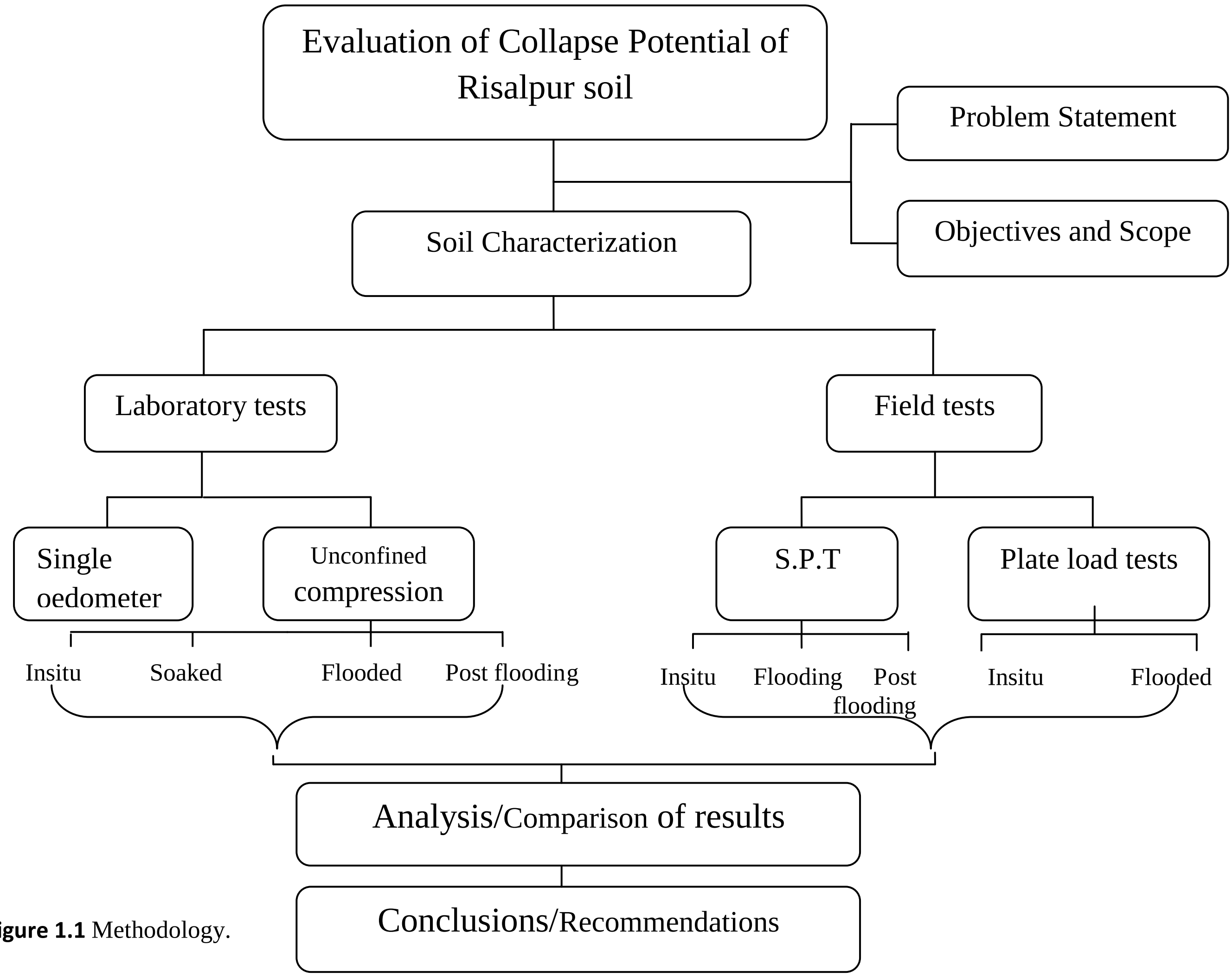


Figure 1.1 Methodology.

Chapter II

Literature Review

2.1 Introduction

Collapsible soils are characterized by loose and open structure with particle sizes ranging from silt to fine sand. The soils mostly exist in dry condition and derive their strength due to bonding at the particle contacts provided by some cementing agents. The cementing agents are carbonates, salts or even dried clays. As the moisture content increases, the bond tends to loose their strength, thus increasing the possibility of collapse.

2.2 Types of collapsible soils

Collapsible soils are found all over the World, however the arid and semi-arid region favors the formation of collapsible soils (Al Rawas (2000)).

Most common types of collapsible soils are:-

2.2.1 Aeolian (windblown) deposits

They have a loose open, metastructurebinded by cementing agents which upon wetting, become weak and may dissolve causing collapse. These soils are composed primarily of quartz along with feldspar and clay minerals. Bell and Bruyn (1997) reported that increasing the clay mineral content decreases the likelihood of collapse.

2.2.2 Water deposits

They contain alluvial fans, mud’s flows and flash flood deposits. At the time of deposition the soil is in saturated; however, with the passage of time the soil becomes hard at relatively low density the water dries out. Bonding of soil particles due to cementing agents leaves the structure open and porous

2.2.3 Residual soils

These soils cover a wide range of sizes, which start from the clayey size upto the gravel range. The collapse structure is developed as a result of the washing off of the soluble and colloidal (suspension matter somewhere b/w size of a molecule and a grain of sand) matter from the residual soil. This leaching effect of the soluble and fine materials results in porous and unstable structure

2.2.4 Colluvial deposits

These are loose bodies of sediment that have been or built up at bottom of a low grade slope or against a barrier on that slope, transported by gravity. These soils are deposited with the passage of time primarily through the action of gravitational force as in landslide.

2.3 The collapse mechanism

Collapsible soils are primarily composed of silt sized particle these particles are loosely arranged in a cemented honey combed structured, they have large spaces between adjacent particles. The loose structure is held together by small amounts of water softening or water soluble cementing agents such as clay minerals and $CaCO_3$as shown in Fig 2.1 (a). The introduction of water dissolves the soluble colloids or softens the bonds between the particles and allows them to take a denser packing under any type of compressive loading, theses particles can be clay minerals, fine silt or colloids which can be easily washed away by water if the soil in inundated as shown in Fig 2.1.

At natural moisture content, these soils compress slightly as a result of increase in overburden pressures; however the load brings the soil structure to a metastable condition. When the loaded soil is exposed to moisture, and certain critical moisture content is exceeded, binding agent providing the cementatiotion softens up or gets dissolved into water eventually reaching a stage where the soil structure can no longer resist deformation thereby causing collapse.

(a) (b)

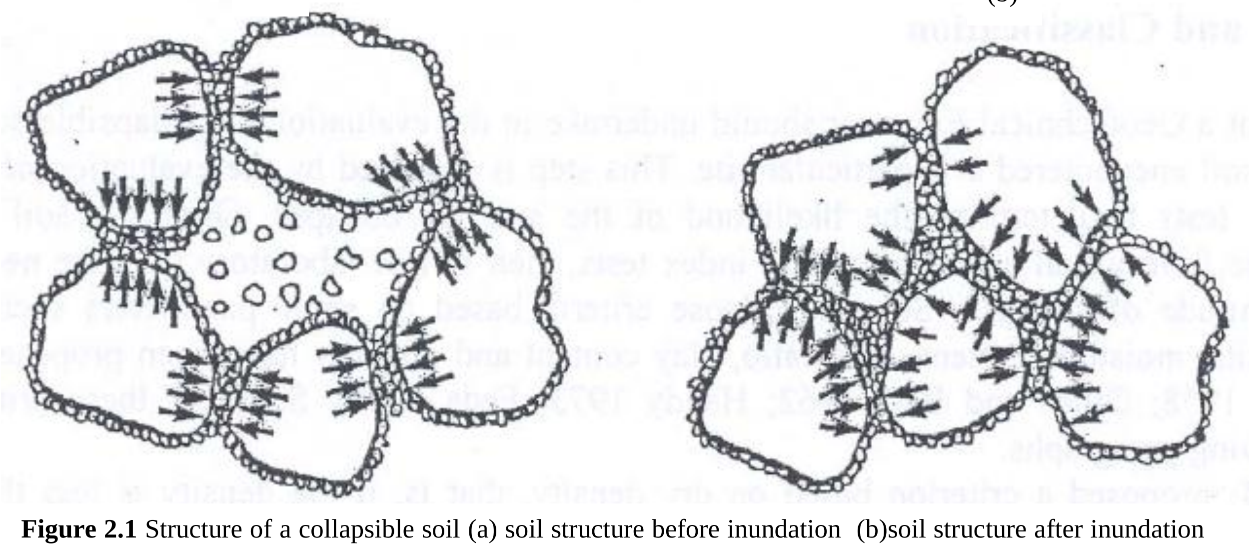


Figure 2.1 Structure of a collapsible soil (a) soil structure before inundation (b)soil structure after inundation (after Al Rawas ;2000).

Mitchell (2005) proposed three conditions required to trigger collapse

- An open, partially unstable and partially saturated soil fabric.
- A sufficiently high applied stress to bring the soil structure to metastable condition.
- A strong binder to stabilize the soil structure in dry condition.

2.4 Identification and classification

It is important to identify and classify collapsible soils to avoid likelihood of potential collapse of foundation soils after the structure has been built. Several criterions based on index properties of soils are reported in literature and given in Table 2.1. The use of these criterions can help classify the collapsible soil as well as to make a comprehensive investigation plan and select suitable remedial measure.

Table 2.1 Existing criterion for identifying collapse soils (after Ayadat 2011).

S/no	Criterion/Correlation	Reference	Remarks
1.	$K=\frac{eL}{eo}$	T. Ayadat and A.M. Hanna (Denisov)	K = 0.5 – 0.75 highly collapsing soils K = 1.0 non collapsible loams K = 1.5 – 2.0 non collapsible soils
2.	$\frac{Wl}{\frac{\gamma w}{\gamma d}-\frac{1}{Gs}}$	T. Ayadat and A.M. Hanna (Gibbs and Bara)	< 1.0 collapse occur
3.	$\alpha=(e_o-e_l)/(1+e_o)$	T. Ayadat and A.M. Hanna (Markin)	α < - 0.3 prone to swelling α > - 0.1 and S0 < 60% susceptible to collapse
4.	$\alpha=(e_o-e_l)/(1+e_o)$	T. Ayadat and A.M. Hanna (Minheev (1969))	S_o < 0.6 and α > - 0.1 susceptible to collapse (this criterion is known as the new soviet building code)
5.	$\alpha=\frac{eo-eL}{1+eo}$	T. Ayadat and A.M. Hanna (Markin)	α < - 0.3 prone to swelling α > - 0.1 and S0 < 60% susceptible to collapse

Table 2.1 continued.

6.	$K_L = \frac{(wo/So)-wp}{Ip}$	T. Ayadat and A.M. Hanna (Feda)	For S0 < 60% KL > 0.85 collapsible soils
7.	I'_p 14.69S/9	T. Ayadat and A.M. Hanna (Salas et al.)	Gypsum soils of low IP are in many respect similar to loess soils, although they exhibit greater collapse and compressibility
8.	$n_o > 40\%$	T. Ayadat and A.M. Hanna (Feda)	Soil is susceptible to collapse
9.	$\alpha = \gamma_{od}/\gamma_{ld}$	T. Ayadat and A.M. Hanna (Markin)	α > 1.3 prone to swelling α < 1.1 prone to collapse
10.	$\gamma_{0d} < 1.28 g/cm^3$, $\gamma_{0d} > 1.44 g/cm^3$	T. Ayadat and A.M. Hanna (Clevenger)	Settlement will be large, Settlement will be small
11.	$K_{d=}\ w_l - w_o/I_p$	T. Ayadat and A.M. Hanna (Priklonskij)	Kd < 0 highly collapsing soils, Kd > 0.5 non collapsing soils, Kd > 1.0 swelling soils.
12.	Lowa loess with clay < 0.002 mm contents	T. Ayadat and A.M. Hanna (Handy)	< 16% high probability of collapse, 16 to 24 % probably collapsible , 24 to 32 % less than 50% probability > 32% usually safe from collapse
13.	*Cu< 4 4<Cu<12 Cu> 12	T. Ayadat and A.M. Hanna (Ayadat and Belouahri)	Safe from collapse Transition interval (collapse may occur) Soil is collapsible
14.	Graphical method based on the work of Kenney and Lau (1985)	T. Ayadat and A.M. Hanna (Ayadat et al.)	Collapse occur if the equivalent grain size curve of the soil is situated above or cut the line H = 1.3 F
15.	$I_p < 20$, $15 < w_L < 35$	T. Ayadat and A.M. Hanna (Ayadat and Ouali)	Collapse is susceptible

Table 2.1 continued.

16.	$CP=\alpha(\gamma_d-15.27)+bw_o+17$ $a=-0.036C_u-1.379$ $b=0.0006C_u^2-.089C_u+1.3$	Ayadat and Hanna	CP < 1 collapse will not take place CP > 1 collapse is susceptible

2.4.1 Classification based on dry unit weight and L.L

Al-Rawas (2000) reported method proposed by Gibbs & Bara (1962) which relates the dry density of soil with its liquid limit. The classification chart proposed by Gibbs & Bara (1962) is shown in Fig 2.2. A similar criterion proposed by Lutenegger and Saber (1988) as reported by Mansour et.al (2008) and shown in Fig 2.3 it can be seen in Fig 2.2 and 2.3 that the soil susceptible to collapse, have low dry density and a low liquid limit, i-e for a given dry density, increase in moisture content is likely to reduce collapse susceptibility of soil.

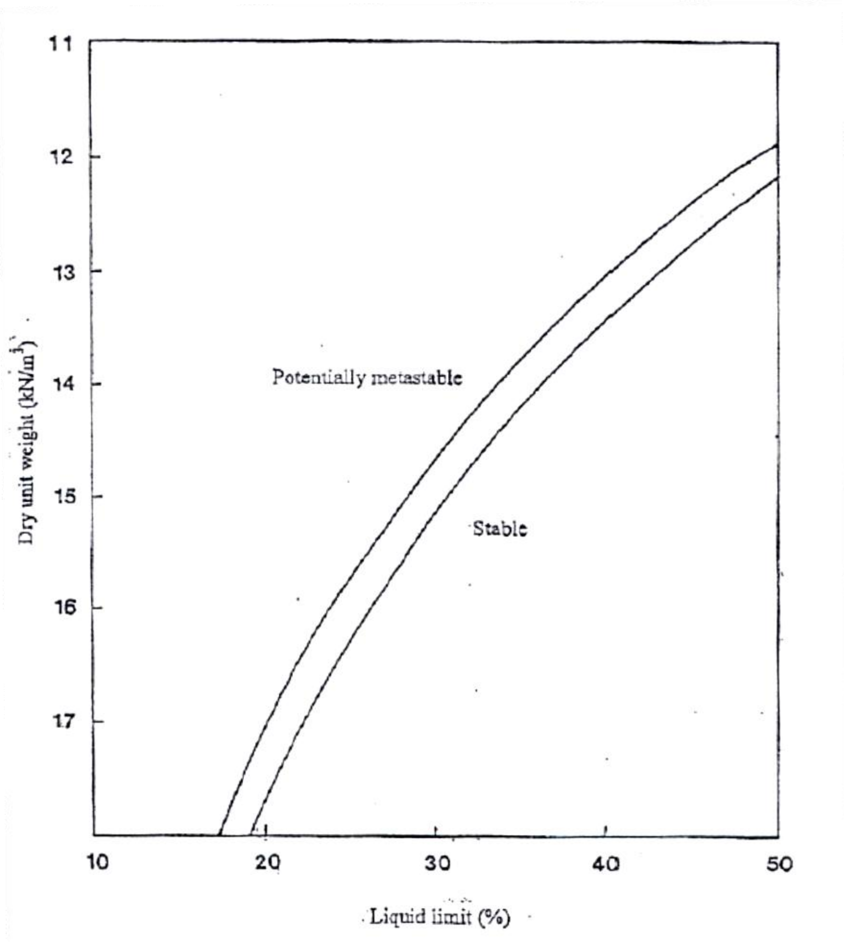


Figure 2.2 Classification of collapsing soil (after Gibbs and Bara 1962).

2.4.2 Classification Based on consistency limits & moisture content.

Feda (1996) proposed a collapse index *(Ic)* as defined by equation 2.1

$$i_c = \frac{\frac{W_c}{Sr} - PL}{LL - PL} \tag{2.1}$$

Where i_c = collapse index

W_c = natural moisture content

PL = plastic limit

LL = liquid limit

Sr = degree of saturation

A value of i_c greater than 0.85 indicates collapsible soils.

2.4.3 Classification based on void ratio & plasticity index

For loess, Reznik (1989) correlated plasticityI_p index of soils to a collapse index CI defined in terms of different void ratios as follows:-

$$CI = \frac{e_L - e}{1 + e} \tag{2.2}$$

Where e_L= in situ void ratio

e = void ratio corresponding to liquid limit

Table 2.2 shows the indicator of CI for collapsible soil for different ranges of I_p

Table *2.2* Limits for the indicator CI values for different loessial soils (After Reznik 1989).

Soil plasticity index, Ip	**Indicator CI**
1% ≤ Ip<10%	0.10
10% ≤ Ip<14%	0.17
14% ≤ Ip<22%	0.24

2.5 Evaluation of collapse potential

2.5.1 Laboratory tests

Oedometer test

Conventional oedometer used for One-dimensional compression can also be used to evaluate the collapse potential. Both single and double oedometer test can be used to evaluate the collapse potential.

2.5.1.1 Single oedometer test

As the name implies, the single oedometer test uses only a single soil specimen. The standard procedure is given by ASTM D 5333-03. In this test an undisturbed sample is placed in the oedometer at its natural (dry) moisture content. A small seating load is applied to the specimen. The specimen is then gradually loaded to the anticipated field loading conditions. At this stress level, the sample is then inundated with water and allowed to saturate. The resulting hydro collapse is then observed. Loading of the specimen is then continued with consolidation permitted.

The collapse potential is then expressed as.

$$C_P = \frac{\Delta e_c}{1+e_o} \quad (2.3)$$

In which Δe_c = change in void ratio, e_o= natural void ratio.

The collapse potential is also defined as.

$$C_{P=\frac{\Delta H_O}{H_C}} \tag{2.4}$$

In which, ΔH_o= change in height upon wetting, H_c= initial height.

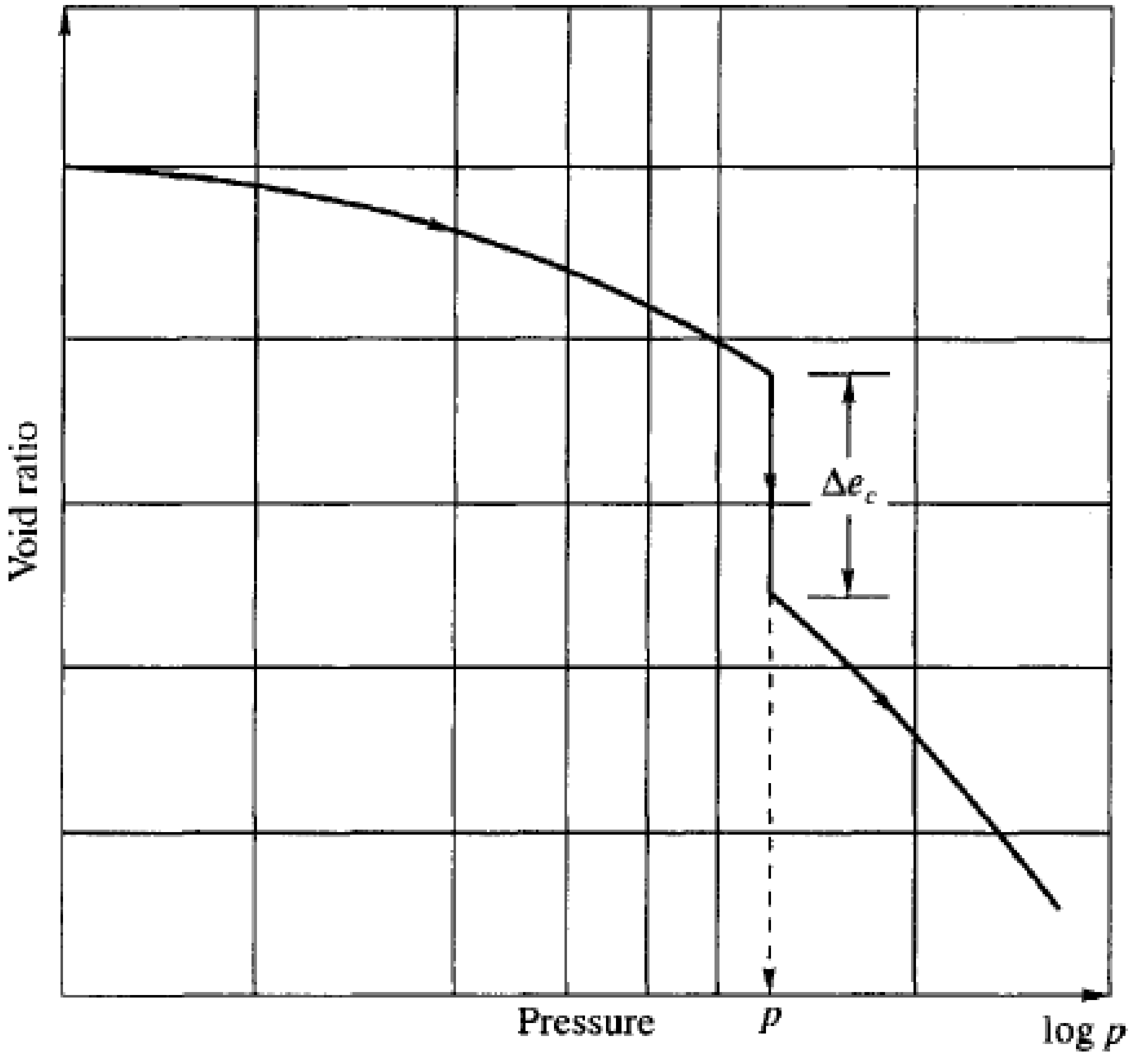


Figure 2.3. Typical results from a single oedometer test on a collapsible soil specimen.

Table 2.3 shows the different severity of collapse related to the collapse index of the soil according to ASTM D 5333-03

Table 2.3 ASTM D 5333-03 classification of collapse index.

Degree of collapse of specimen	Collapse index
None	o
Slight	0.1 to 2.0
Moderate	2.1 to 6.0
Moderately severe	6.1 to 10.0
Severe	>10

2.5.1.2 Double oedometer test

The procedure was proposed by Jennings and Knight (1975). In this test, two identical soil specimens are placed in two oedometers and subjected to consolidation tests. One of the specimens is tested in natural water content, which is generally quite low. The other specimen is fully saturated before the test begins, and then subjected to an identical consolidation test. Two stress versus strain p curves will be generated, one for the dry soil and one for the saturated soil. If the soil is strongly hydro collapsible, the stress-strain response for the saturated curve will be significantly different than that of the dry soil. For a given applied stress σ’n, the strain offset εw between the two curves is called the hydro-collapse strain for that stress level.

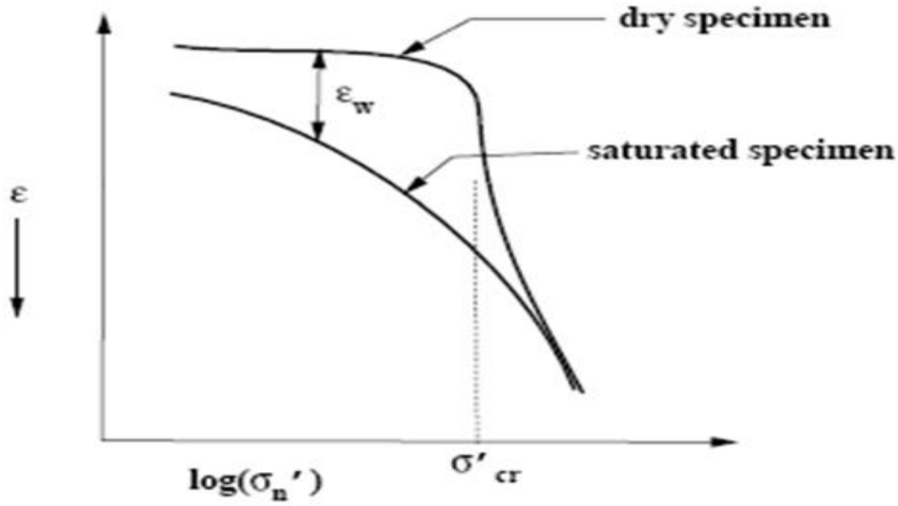


Figure 2.4. Results of a double-oedometer test on a hydro-collapsible soil.

2.5.2 Field tests

2.5.2.1 Plate load test

These tests are normally conducted near the ground surface. Standard procedure is given in ASTM D-1195. In this test, the water is introduced to the loaded soil and the resultant displacement due to wetting is recorded. The settlement can be co-related with the collapse potential of the soil. The difference in settlement from dry soil to flooded soil gives a broader perspective to the collapsing dynamics of the soil.

The advantages of plate load test include the minimum disturbance of soil sample, larger volume of soil being tested, and the test followed the actual site conditions

2.5.2.2 Sausage test

It is a very simple field test introduced by Clemence and Finbarr 1981. In this test a block of soil of about 500 $cm^3$is taken from the test trial pit and broken into two pieces, and each is trimmed until they are approximately equal in volume. One specimen is then wetted and molded by hands to form a damp ball. The volume of this ball is then compared with the volume of the undisturbed specimen. If the wetted ball has significantly smaller volume, then collapse may be suspected. This test only shows us as to whether the soil can collapse or not.

2.6 Geological conditions of the test site

The sediment deposition environment of Nowshera is traced back to Devonian period. It lies in a semi-arid region more specifically the hot semi-arid region.

2.6.1 Devonian Period

The Devonian is the geological period of the Paleozoic era spanners from the end of siluvian period about 416+-20.8 million to the beginning of carboniferous period about 359.2+-25 million.

Devonian sedimentary rocks are continued to the outer corps of relatively small extent in Khyber Pass and Nowshera.

2.6.2 Semi arid regions

The semiarid regions of the world are defined as transition zones between arid and sub-humid belts. Semiarid regions are also defined as areas where precipitation is less than potential evaporation. Figure 2.5 shows the semi-arid regions of Pakistan.

Hot semi-arid climates tend to be located in the tropics and subtropics. These climates tend to have hot, sometimes extremely hot, summers and mild to warm winters. Snow rarely (if ever) falls in these regions. Hot semi-arid climates are most commonly found around the fringes of subtropical deserts.

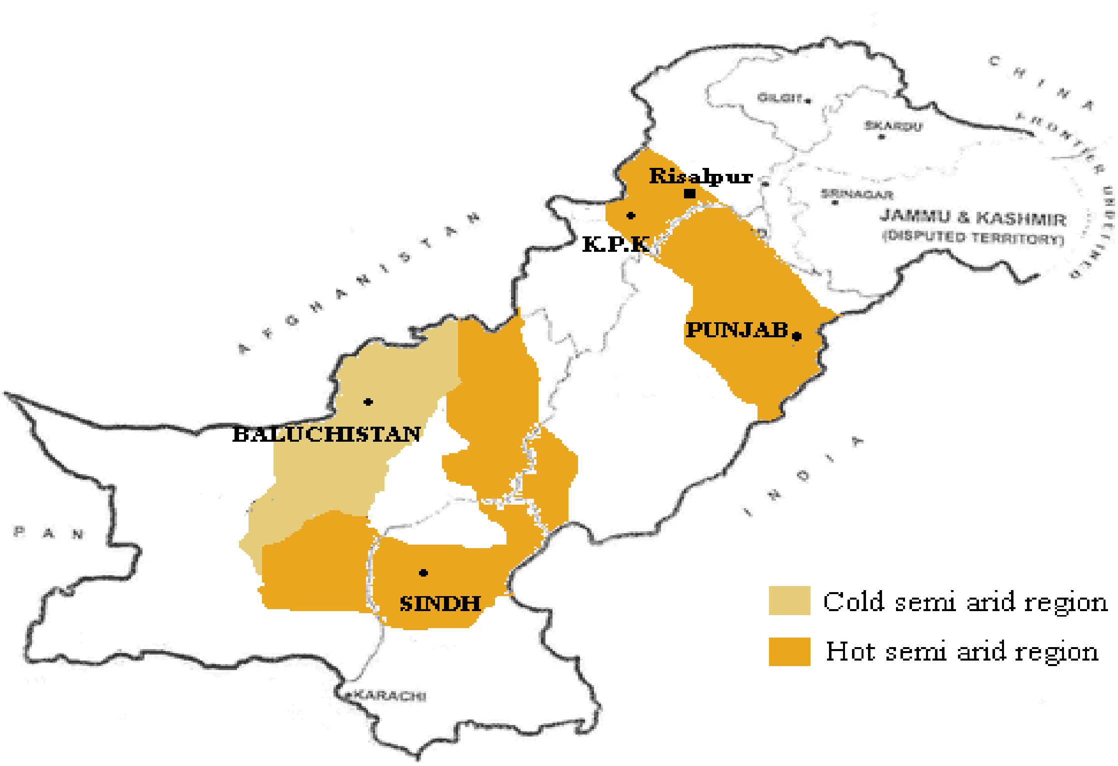


Figure 2.5.semi-arid regions of Pakistan (ww.wikipedia.com).

Table 2.4 Comparison of Semi-Arid Region Characteristics With Risalpur.

Characteristics	**Semi-Arid region**	**Risalpur**
Precipitation sum/annum	250 to 500 mm (9.84 to 19.6 in)	13.6 in (345.44mm)
Highest temp	high temperatures (30-45^{o}C) (86-113°F)	122 °F (50°C)
Average no of rainy days	25-35	27.2
Rainfall	700mm per annum	711.2 mm

Table2.5 annual weather reports of Risalpur and surrounding (www.weatherreports.com).

	Units	**Year**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**June**	**July**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Average temperature over 60 years	°F	73	51	55	62	73	82	91	91	87	82	73	62	53
Average high temperature over 60 years	°F	84	62	66	75	84	96	105	102	98	95	87	77	66
Average low temperature over 60 years	°F	60	39	44	51	60	69	77	80	78	71	60	48	41
Highest recorded temperature over 60 years	°F	122	77	86	98	107	118	120	122	118	109	100	91	82
Lowest recorded temperature over 60 years	°F	21	26	28	21	41	51	62	57	66	57	34	32	28
Average number of rainy days	days	27.2	3.1	3.2	4.5	3.6	1.8	0.6	2.2	3.5	1.7	0.6	0.8	1.6

Table2.5 continued.

over 60 years														
Average number of days above 90°F over 60 years	days	-	-	-	-	6.5	29.1	30	31	30.2	25.2	12.6	-	-
Average precipitation over 60 years	in	13.6	1.4	1.5	2.4	1.8	0.8	0.3	1.3	2.0	0.8	0.2	0.3	0.7
Average relative humidity over 10 years	%	57	66	66	64	56	37	32	55	64	65	55	60	68
Average dew point over 43 years	°F	53	39	42	48	53	51	53	71	73	69	55	46	42
Average number of days below 32°F over 60 years	days	1.1	0.5	0.2	0.1	-	-	-	-	-	-	-	-	0.3
Most recorded rainfall over 129 years	in	28.0	5.3	5.1	10.8	7.4	5.2	3.9	8.4	17.8	7.0	2.8	8.5	5.7
least recorded rainfall over 129 years	in	4.2	-	-	-	-	-	-	-	-	-	-	-	-

2.6.3 Soil of Nowshera

Risalpur lies in the semi arid region of Pakistan, which is more specifically Hot semi-arid region. the soil of Risalpur mainly loamy and clayey part non-calcareous soil of alluvial or **<u>loess</u>** plain. It has also some unconsolidated surfacial deposits of silt, sand and gravel, to the south it has Cambrian rocks which are foliated clay, slate, grey wacke, and lime stone. Figure 2.6 and 2.7 show the distribution of rock and soil in Risalpur.

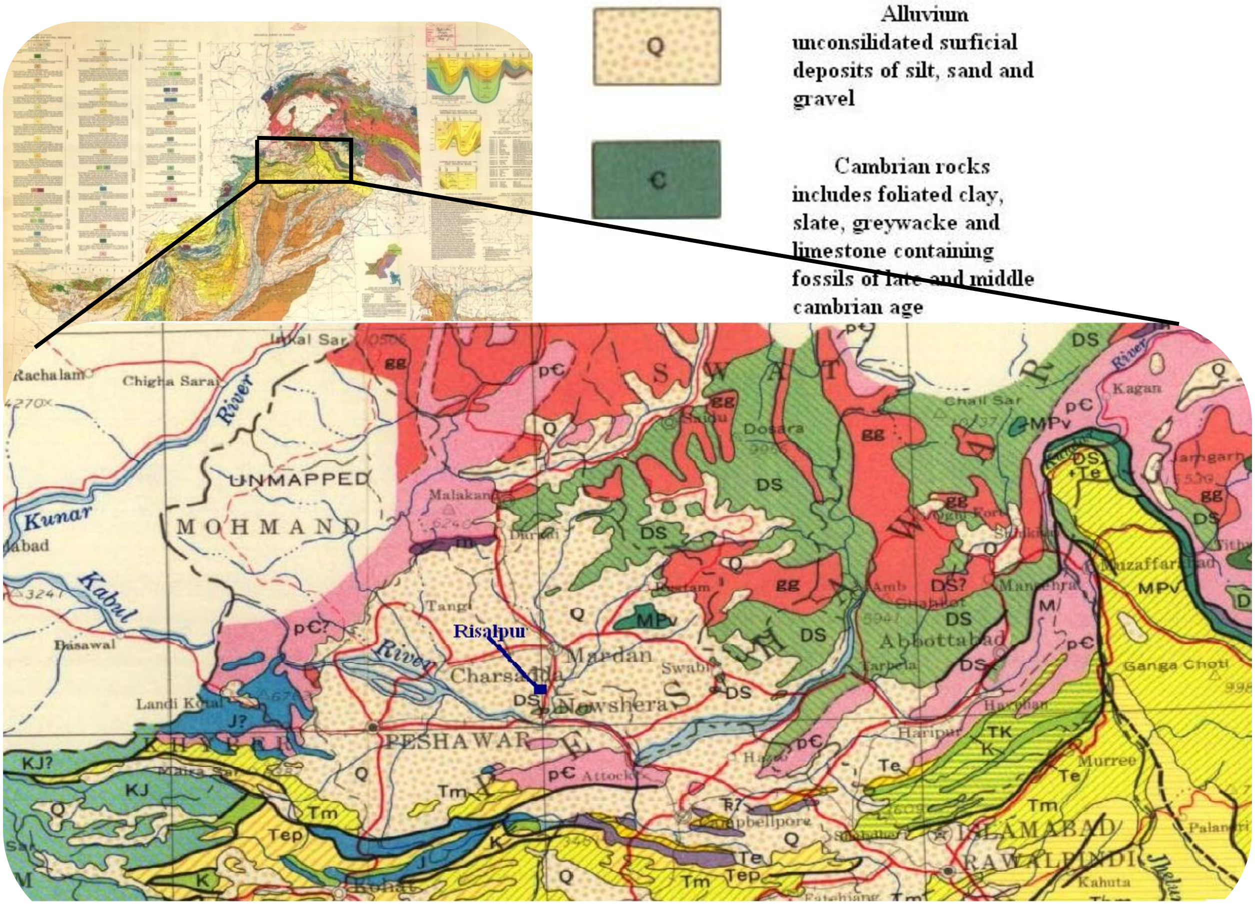


Figure 2.6 detailed geological soil map of Nowshera region.

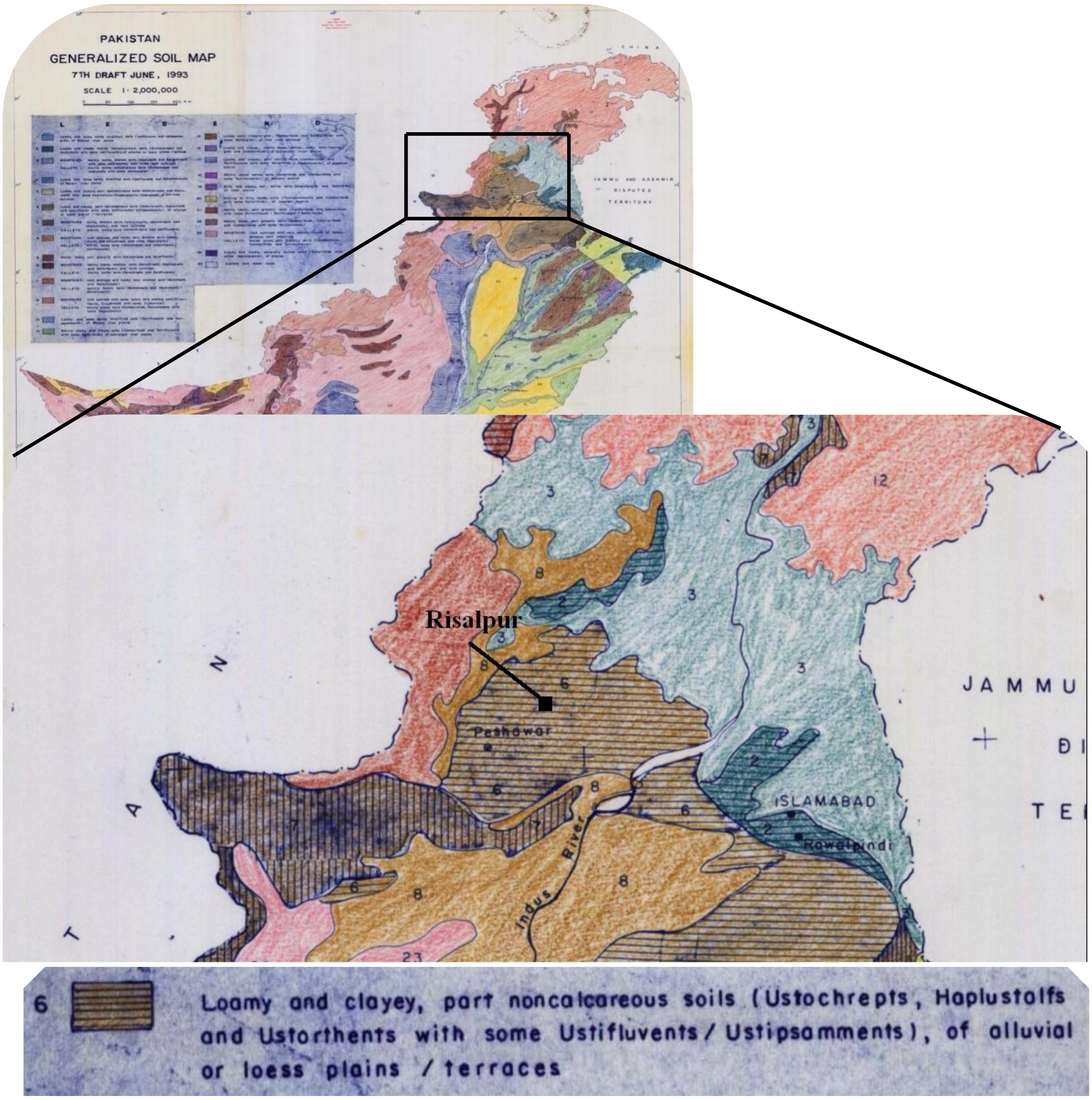


Figure 2.7 generalized soil map of Nowshera region.

2.7 Mitigation and site improvement

Mitigation measures may be broadly defined as any actions or designs that lessen or solve collapsing soil problems. In this context, soil improvement techniques are a subset of mitigation measures generally, although soil improvement is needed and used for many problems other than collapsing soils.

The best technique or combination of mitigation methods for a collapsible soil site depends on several factors, including (a) the timing of the discovery of soil collapsibility-whether during investigation, during construction, or after structures are in place (b) the primary source of the driving stress-overburden or structural load (c) the depth range of the collapsibility problem-usually deposits are simply classified as shallow or deep (d) the probable source(s) of wetting, and (e) mitigation cost-including, particularly in USA, the cost of risk/liability.

Mitigation measures have been summarized and described by several authors (Beckwith,1995, Clemence and Finbarr, 1981, Evstatiev, 1995, Houston and Houston, 1989, pengelly, et al., 1997, Rollins and Rogers, 1994, and Turnbull, 1968).

The following list shows the categories into which these techniques may be placed.

- Removal of volume moisture-sensitive soil
- Removal and replacement or compaction
- Avoidance of wetting
- Prewetting
- Controlled wetting
- Dynamic wetting compaction
- Pile or pier foundation
- Differential settlement resistant foundations

2.8 Damages in housing sector

Collapsible soils generate large & often settlements .This can yield disastrous consequences for structures unconditionally built upon such type of soils.

Collapsible soils can be considered a constructional hazard due to the settlement of structures. In this phenomenon in the foundation can cause differential settlement in the structures and cracks can appear.

2.8.1 Risalpur Cantt

Certain buildings in the Risalpur cantonment have been observed to have differential settlement and cracks. The phenomenon of collapsible soils can be a possible cause of this damage.

2.8.2 Nowshera Region (2010 Floods)

Nowshera and Risalpur are observed to be in the same geographical belt therefore they have the same soil deposits therefore collapse was observed to occur in the Nowshera CMH in the recent 2010 floods. This was due to the fault that the foundation was built upon collapsible soil

Therefore before laying the foundation on such type of soil the geotechnical engineer must look upon the ground improvement techniques to improve the foundation for safe structural foundation.

Chapter III

Methodology

3.1 General

Characterization and evaluation of collapse potential of Risalpur soil was carried out by excavating 3 x test pits to a depth of 4 feet.In test pit 1 S.P.Ts were carried out in natural condition. Disturbed and undisturbed samples were also taken up to a depth of 20 feet at an interval of 4 feet. Test pit 2 was flooded with water for three weeks. S.P.T was carried out in the wet condition, and samples were taken for tests after inundation. Test pit 3 was used for the plate load tests, which were carried in insitu condition as well as after installation of a stone column.

3.2 Sampling

Disturbed samples were collected during S.P.T whereas undisturbed samples were obtained using 18 inch long and 4 inch dia, thin walled shelby tubes. Sampling was carried out in the following stages. In all cases samples were collected to a depth of 20 feet at every 4 feet interval.

3.2.1 Stage 1

In this stage the disturbed/undisturbed samples were collected from the soil in its natural state from test pit 1. The purpose was to characterize soil, evaluate the unconfined compression strength and the collapse potential of the soil in its natural condition, the collection of disturbed samples as shown in Fig 3.1

Figure 3.1 A view of test pit #1 and collection of disturbed samples using split spoon samples.

3.2.2 Stage 2

In this stage undisturbed samples were collected from test pit 2 which was flooded with water for three weeks. These samples were collected to evaluate the unconfined compressive strength of soil after a the wetting period. Shelby tubes were driven to desired depth using S.P.T hammer and a specially manufactured connector which can be fixed with S.P.T extension rods. The shelby tube and connector is shown in Fig 3.2.

(a)

(b)

Figure 3.2 (a) Shelby tube used for extraction of samples from different depths; (b)connector for fixing shelby tube with S.P.T extension rod.

3.2.3 Stage 3

In this stage the samples were collected from test pit 2, four weeks after the wetting period to evaluate the gain in strength using unconfined compressive strength test.

Figure 3.3 The undisturbed sample were later ejected carefully from the Shelby tubes using hydraulic jacks and carefully trimmed sample were cut for different tests.

3.2.4 Handling of Samples

During all the stages, samples were immediately sealed after extraction using hot wax, plastic sheet and a plastic cap to preserve the insitu moisture. The samples were then carefully stacked in the Geotechnical laborotary to avoid any disturbance due to shocks/movement etc. all samples were carefully ejected using a hydraulic jack. (Fig 3.3) specimens for different tests were trimmed from ejected samples.

3.3 Soil Characterization

Soil characterization included all basic tests including grain size distribution. Atterberg limits, insitu moisture content etc. These parameters were also used to estimate the collapsibility of soil using criterion mentioned in section 2.4.2

3.3.1 Index properties

Tests to estimate index properties of soil were carried using ASTM standards as shown in Table 3.1. These tests were carried out at each sampling depth to estimate the vertical profile of these properties.

Table 3.1: Basic tests to estimate index properties of soil.

S.No	Test	Purpose	Relevant standard
1	Atterberg limits	To obtain general information about the soil, its classification and interaction with water.	ASTM D-4318
2	Moisture Content	Water effect the density, shear strength, bulking and swelling. It incorporates its effect on the performance of any structure	ASTM D-2216
3	Specific gravity	Together with other soil parameters it is used to compute other useful soil parameters	ASTM D-854

3.3.2 Gradation analysis

Both sieve analysis and hydrometer analysis were carried out to establish the grain size distribution of the soil. These tests were also performed at each sampling depth to establish the variation in particle size with depth. Table 3.2 shows the relevant standard according to which these tests were conducted.

Table 3.2.

S.No	Test	Purpose	Relevant standard
1	Sieve analysis	The size distribution of gravel and sand particles	ASTM D-6913-04
2	Hydrometer analysis	To determine the grain size distribution of fine grained soils.	ASTM D-421

3.4 Evaluation of collapse potential

3.4.1 Laboratory tests

3.4.1.1 Single oedometer tests

This test was carried out at every 4 feet depth in accordance with **ASTM 5333-03.** The specimen was trimmed into a 2.5 inch diameter and 19mm high oedometer steel ring. Using samples obtained from the shelby tubes. Load was then applied in increments of 25, 50, 100, 200, 400, and 800kPa and the corresponding settlements were recorded. At 200 kPa the sample was flooded and the collapse was noted down. The sample was kept flooded for 24 hrs after which the load was increased to 800 kPa in increments.

Figure 3.4 Oedometer testing.

3.4.1.2 Unconfined Compression (U.C.C) tests

U.C.C test were carried out on specimen trimmed from sample obtained from shelby tubes. The test was carried out according to the procedure specified by **ASTM D 2166-66**. The specimen were trimmed to 1.5 inch dia and 3 inch height and sheared at a strain rate of 1.2mm/min. The tests were performed on specimen obtained from soil samples in natural/dry condition, wet condition and post wetting condition Figure 3.5 shows the testing of an insitu specimen obtained from a depth of 4 feet.

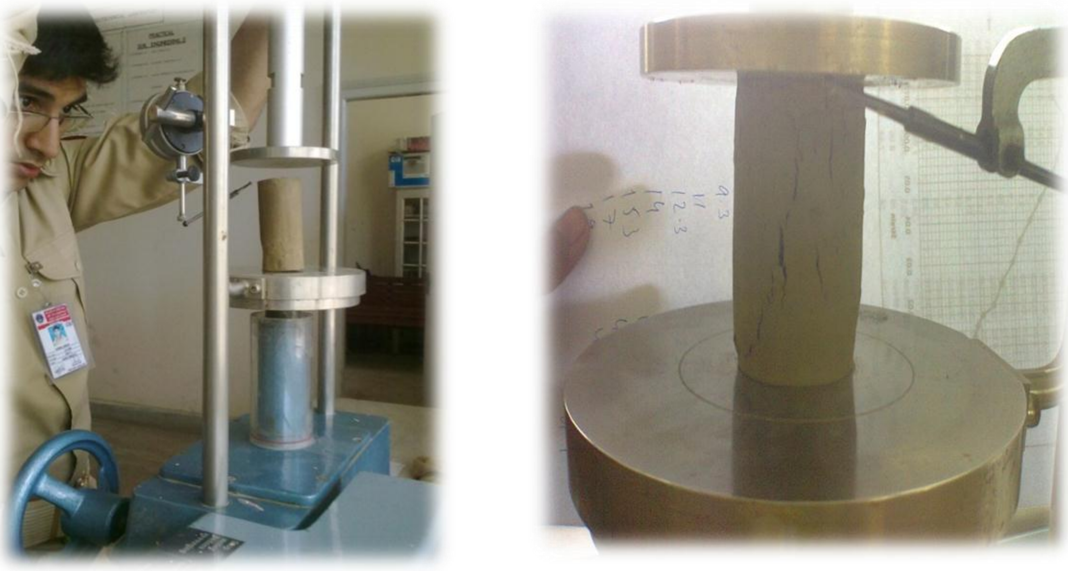


Figure 3.5 UCC at unsoaked condition.

At each depth the U.C.C specimens were to be tested in both soaked and unsoaked conditions.

For soaking the specimen was wrapped in a porous fabric and placed on top of a porous stone in contact with water surface in the water bath. The specimens were not directly in contact with water. The soaking arrangement is shown in Fig 3.6.

Figure 3.6 UCC samples before and after 24hrs for soaking.

3.4.2 Field tests

Field tests; frequently used to identify collapsible soil, are more realistc as these are performed in actual field conditions and do not carry the effect of sample disturbance or remoulding. Plate load tests and S.P.T were used in the field to evaluate collapse potential.

3.4.2.1 Plate Load Test

Plate load test was carried out in accordance with **ASTM D1194-94**. The soil was loaded in increments of 60, 120 and 249 kPa. The limitation of reaction load did not allow any further loading of the soil. After testing the soil in dry/insitu condition, the test pit was flooded to observe soil behavior under sustained loading. The pit was kept flooded for overnight and the resulting settlement was recorded.

Plate load test in dry and flooded condition were also carried out after installing 6 inch diameter and 4 feet deep stone column compacted in 4 layers of 1 foot each using a 14 lb hammer 15 number of blows were applied on each layer at a drop height of 2 feet. Modified hammer was used. The gradation of the gravel was according to AASHTO T27, alternate number 4.

Figure 3.7 Plate load test.

3.4.2.2 Standard Penetration Test (S.P.T)

S.P.T was performed in accordance with **ASTM D-1586**. In test pit # 1 S.P.T was conducted in dry condition, whereas in test pit # 2 S.P.Ts were conducted three weeks after flooding and four weeks after the flooding was stopped and soil was allowed to dry up. Figure 3.8 shows the conduct of S.P.T in test pit #1.

Figure 3.8 conduct of SPT test in test pit# 1.

CHAPTER IV

TEST RESULTS AND DISCUSSIONS

4.1 GENERAL

Collapse potential of Risalpur soil was evaluated by different laboratory and field tests. Existing criterion for collapse potential evaluation, laboratory and field tests indicate that the soil up to the depth of investigation (20 feet) is collapsible. Laboratory and field tests indicated soil layer from 4 to 8 feet depth; which is the general depth of foundations in the project area; is most susceptible to collapse. Increase in depth showed reduction in the collapse potential till 16 feet after which an increase in collapse potential was observed till 20 feet. Effort to mitigate collapse potential of soil was made by installing a stone column which resulted in a significant reduction in settlement under flooding. The soil also exhibited a regain in strength when dried after flooding

4.2 SOIL CHARACTERIZATION

Laboratory tests were carried out as explained in section 3.3.1 and 3.3.2 to find the grain size distribution, consistency limits, unit weight, specific gravity and degree of saturation etc. Insitu strength parameters were evaluated as described in section 3.4.2.2 for SPT and 3.4.1.2 for UCS. .Results from these tests are briefly discussed in the following sections.

4.2.1 Geotechnical Properties

Grain size distribution at different depths is shown in Fig 4.1. The figure shows that (a) bulk of the soil has a particle size of silt to clay (b) soil at 12 feet depth has slightly coarser size particles as compared to soil at other depths (c) soil at 16 feet depth has finer particle size (d) within the available range of particle sizes, the soil is well graded at all depths. Variation of clay size fraction with depth is shown in Fig 4.1 (b) which shows that the clay size fraction is increasing with depth.

Vertical profile of different soil properties is shown in Fig 4.2. Following is evident from this figure

- Unit weight of soil is constant up to a depth of 12 feet after which it increases with depth as shown in Fig 4.2 (a).

- Plasticity Index of soil varies between 7 and 3 at different depths (Fig 4.2 b) which indicate the non plastic nature of fine content.
- Figure 4.2 b also clearly indicates that the insitu moisture content at all depths is significantly lower than the plastic limits. Insitu moisture content at 16 feet depth is lowest indicating the susceptibility to collapse.
- Specific gravity generally ranges between 2.67 to 2.73 which lies within the typical range of silts and clays.
- The degree of saturation; an important parameter governing behavior of collapsing soils, increases with depth except at 16 feet where it is lowest(Fig 4.2 d).
- In situ void ratio decreases with depth except at 16 feet as shown in Fig 4.2 e

Index properties of soil show that (a) the soil is extremely dry and, (b) a distinct sub layer is present at a depth of 16 feet.

Based on the consistency limits, gradation and using Unified Soil Classification System, the soil is classified as low plastic clayey silt (CL-ML) at all depths except at 16 feet where it is low plastic silt (ML).

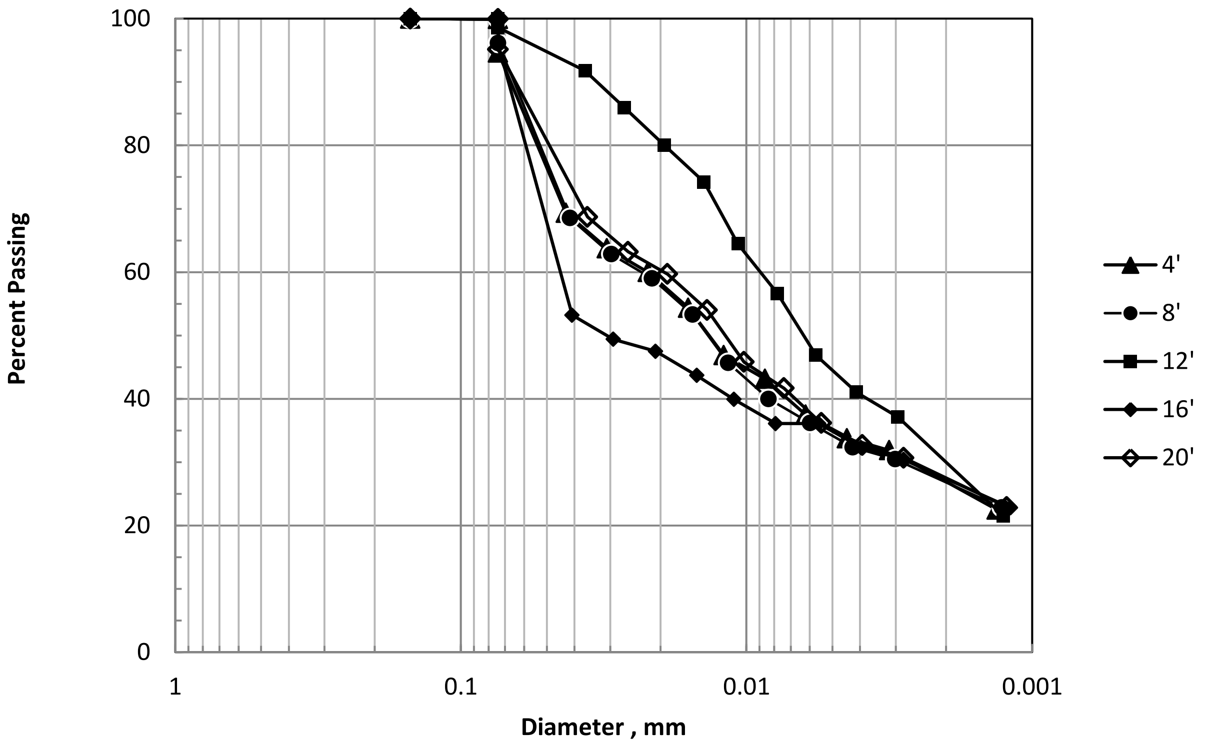


Figure 4.1 (a) Grain size distribution at different depths.

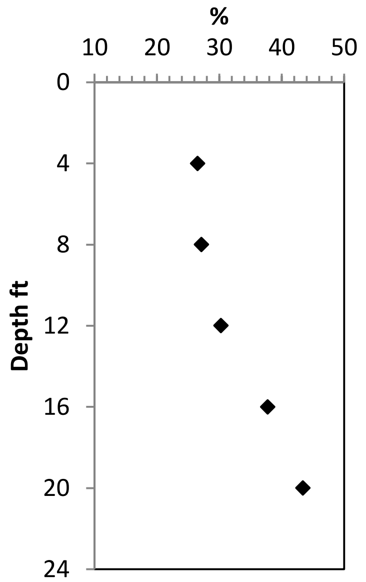
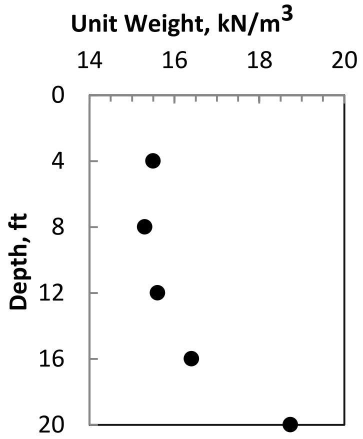
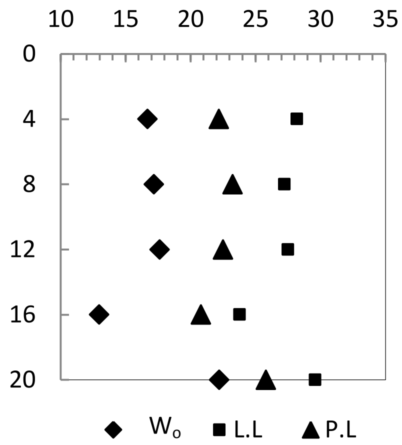


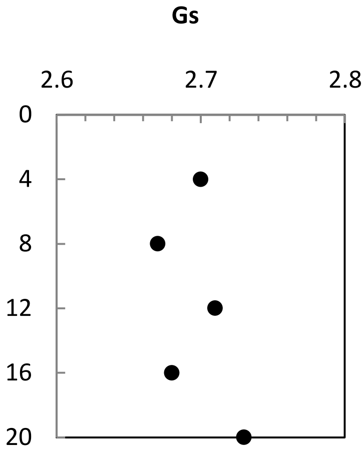
Figure 4.1 (b) Clay size fraction at different depths.



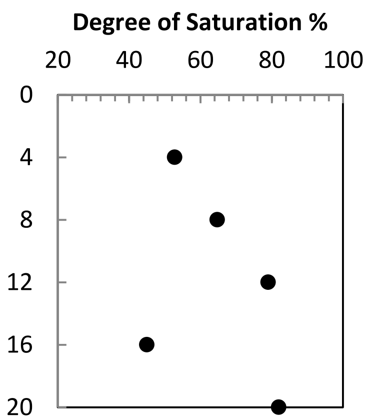
(a) Unit Weight



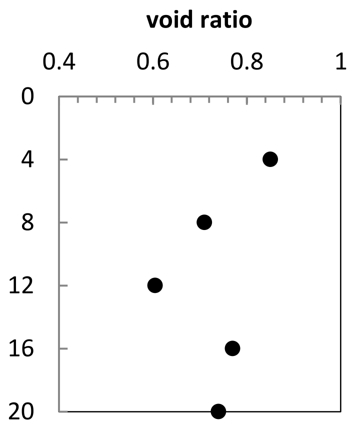
(b) Consistency Limits



(c) Specific Gravity



(d) Degree of Saturation

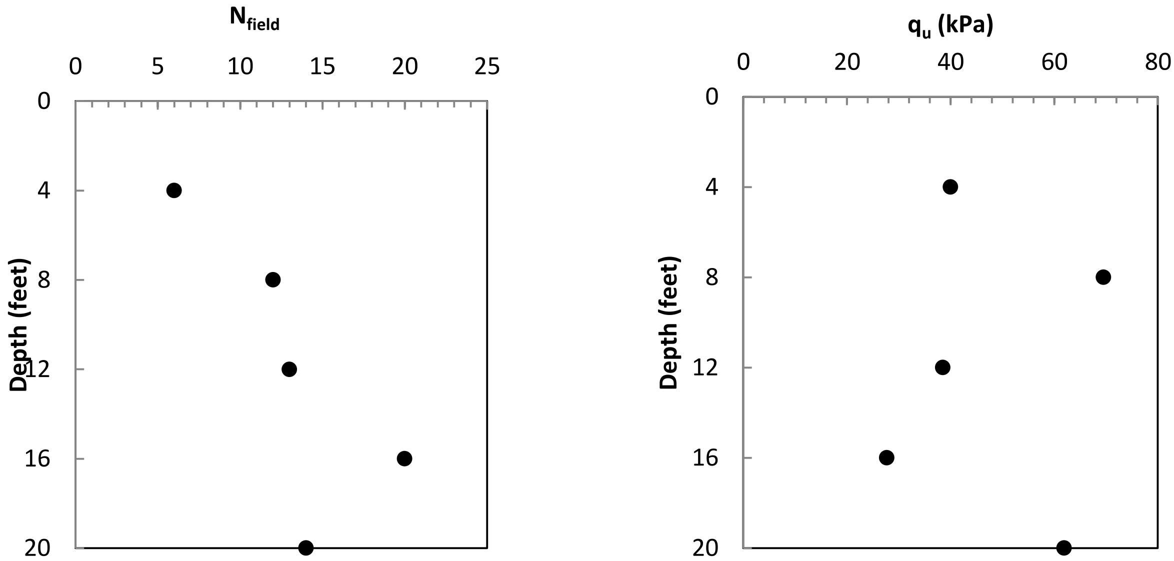


(e) Void Ratio

Figure 4.2 Vertical profile of soil properties.

4.3 In situ Soil Strength

In situ soil strength was estimated by Standard Penetration Test (SPT) and Unconfined compression test (UCS) using procedures described in sections 3.4.2.2 and 3.4.1.2 respectively. The results of these tests are shown in Fig 4.3

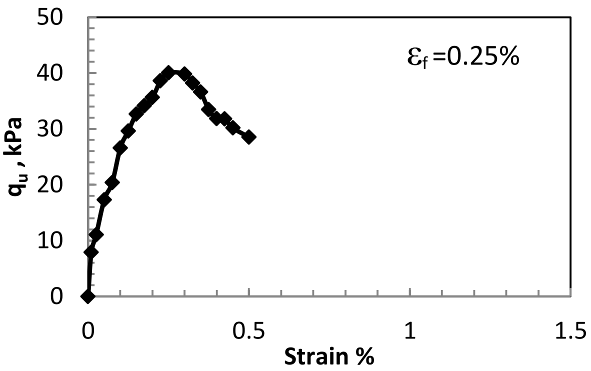


(a) Insitu S.P.T blows (b) Insitu Unconfined Compression Test

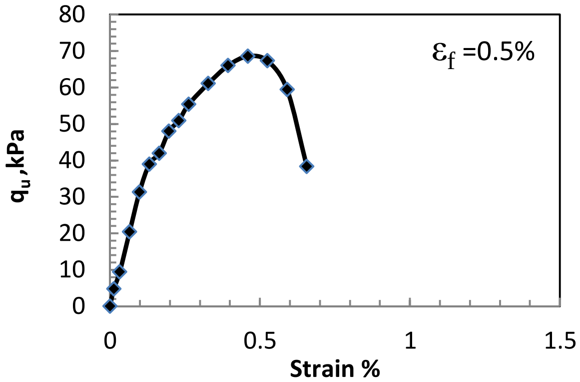
Figure 4.3 Insitu soil strength.

The results of SPT and UCS show different patterns of soil strength with depth especially at 16 feet. Figure 4.3 a shows that the SPT blow counts increase from 6 at 4 feet to 11 at 8 feet, remains constant till 12 feet, increases sharply to 20 at 16 feet and reduces abruptly to 13 at 20 feet depth; whereas the UCS results shown in Fig 4.3 b indicates a continuous and significant decrease in strength from 8 to 16 feet depth. Both tests indicate that the soil is sufficiently strong at shallow depths.

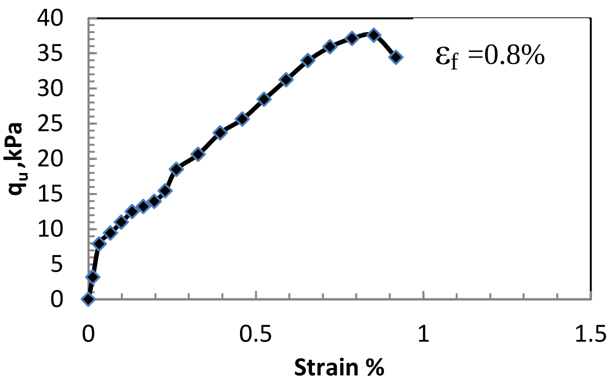
Figure 4.4 shows the Stress-Strain relation at different depths obtained from UCS. The figure show that soil at all depths fails at around 0.5% strain; moreover; the soil exhibits a brittle behavior at all depths, indicating the possibility of a sudden failure.



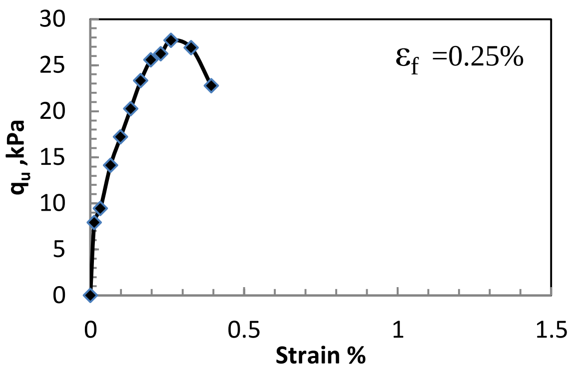
(a) 4 feet



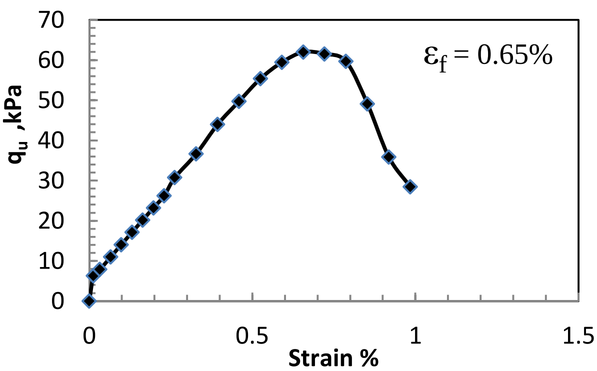
(b) 8 feet



(c) 12 feet



(d) 16 feet



(e) 20 feet

Figure 4.4 Unconfined compression test at different depths.

4.4 Evaluation of Collapse potential

Existing criterions as well as laboratory and field tests were conducted to evaluate the collapse potential to a depth of 20 feet. The results are briefly discussed in following sections.

4.4.1 Collapse Potential Evaluation Using Existing Criterions

A preliminary analysis based on index properties and different criterion proposed in literature and discussed in section 2.4 is given in Table 4.1.

Table 4.1 Preliminary analysis based on existing criterions of soil at different depths.

S/no	Criteria/Correlation	Depth (ft)					Remarks
		4	8	12	16	20	
1	$K=\frac{eL}{eo}$	0.89	1.01	1.24	0.83	1.08	Non collapsing
2	$\frac{Wl}{\frac{\gamma w}{\gamma d}-\frac{1}{Gs}}$	0.76	0.72	0.74	0.78	1.08	Collapsible except at 20 feet
3	$\alpha=\frac{eo-eL}{1+eo}$	0.05	@	@	-0.074	@	Soil at 4 feet is susceptible to collapse soil at 16 feet is not
4	$K_L=\frac{(wo/So)-wp}{Ip}$	0.015	@	@	0.03	@	Non-collapsing
5	$n_o>40\%$	46	42	38	43	43	Collapsible
6	*Cu< 4a (safe) 4<Cu<12 (transition) Cu> 12 (collapsible)	25	25	8.5	45	20	collapsible except at 12 feet depth
7	$I_p<20$, $15<w_L<35$	6, 28.2	4, 27.25	5, 27.5	3, 23.8	3.8, 29.6	Collapsible

@ ; S_o>60 (criterion applicable for S_o <60%)

*C_u - Coefficient of uniformity

Table 4.1 indicates a probability of collapse at all depths using different criterions. It is important to note that different results are predicted by different criterions making it necessary to carry out detailed investigations.

Al-Rawas (2000) reported method proposed by Gibbs & Bara (1962) which relates the dry density and liquid limit with collapsible soils. The values for the different depths are plotted on the correlation proposed by Gibbs and Bara (1962) and shown in Fig 4.5. It can be seen from this figure soil at 4, 8, 12 and 16 feet depth lie in the meta-stable zone of and hence are collapsible. At 20 feet depth, the soil lies in the stable zone.

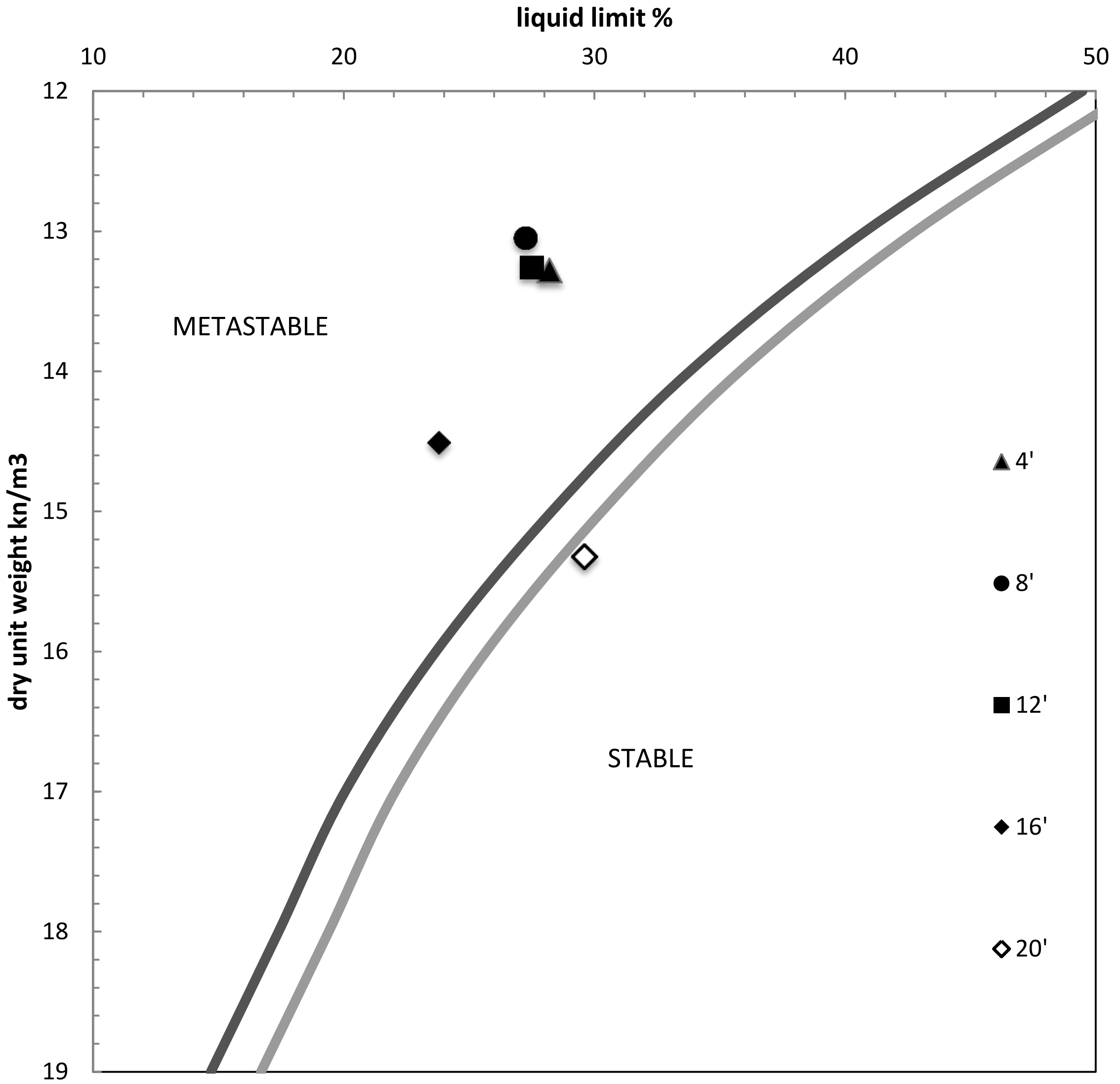


Figure 4.5 Collapse potential at different depths using correlation proposed by Gibbs and Bara (1962).

4.4.2 Collapse Potential Evaluation Using Laboratory Tests

4.4.2.1 Single Oedometer Test

The results from single oedometer test at different depths are shown in fig 4.6. Percent collapse at different depths is also shown in Fig 4.6 which shows that collapse potential

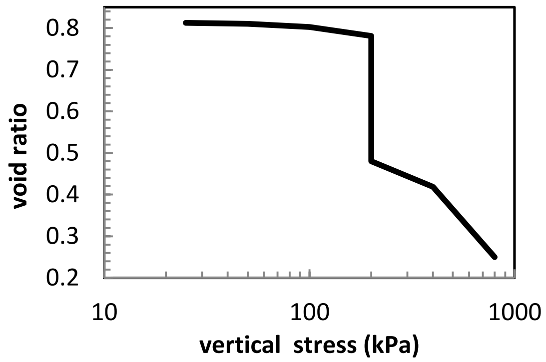
- Is maximum at 4 feet depth.
- Reduce significantly with depth up till 16 feet, and
- From 16 to 20 feet the collapse potential increases again; which is contradictory to results shown in Fig 4.5.

Using the severity of collapse as proposed by ASTM 5333-03, the degree of collapse at different depths is shown in Table 4.2

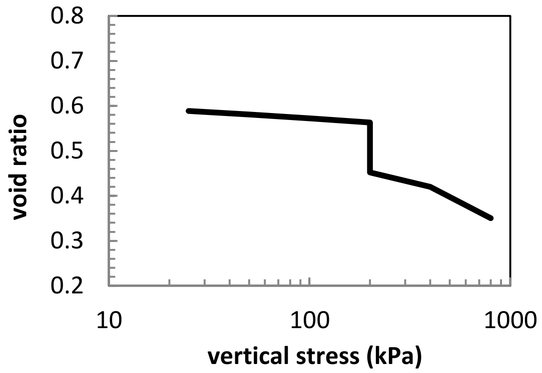
Table 4.2 Degree of collapse at different depths.

Depth (feet)	Degree Of Collapse
4	Severe
8	Moderately Severe
12	Moderate
16	Moderate
20	Moderately severe

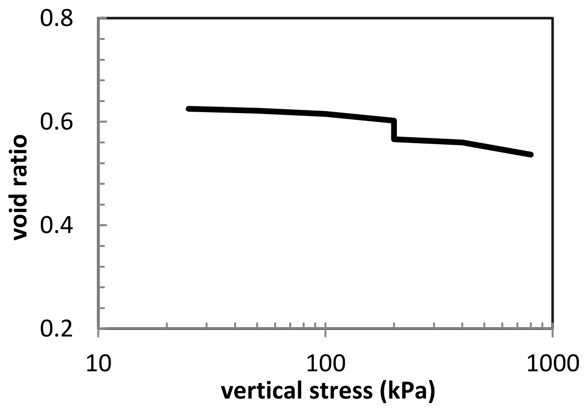
The typical foundation depth in the project area is 3 to 5 feet which corresponds to the soil layer with “severe” degree of collapse.



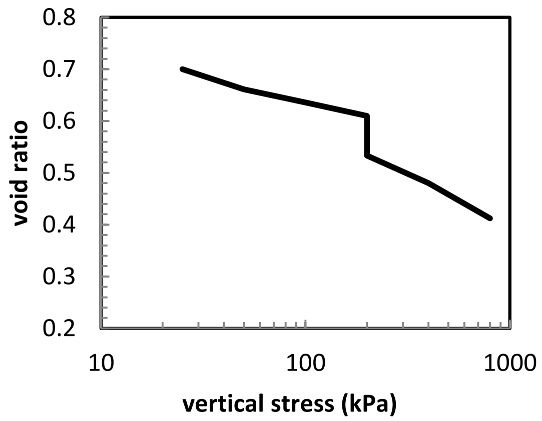
(a) **4 feet**



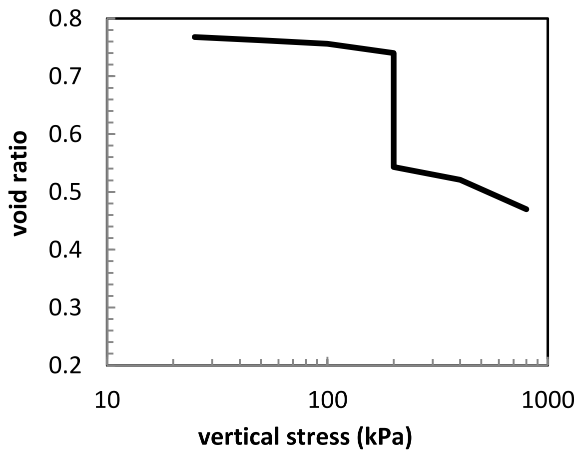
(b) **8 feet**



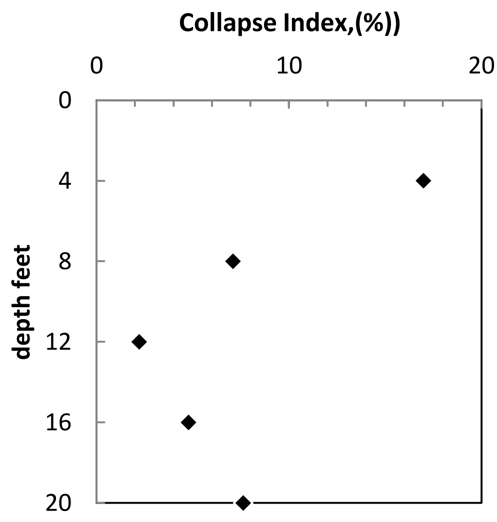
(c) **12 feet**



(d) **16 feet**



(e) **20 feet**



(f) **Collapse Index**

Figure 4.6 (a) to (e) change in void ratio at different depths (f) collapse index at different depths.

4.4.2.2 Unconfined Compression Test (UCC)

The use of unconfined compression test (UCC) as a measure to indicate/evaluate collapse potential has not been reported in the reviewed literature. Comparison of UCC test using undisturbed soil specimen (i.e., in dry condition) and after a predefined wetting period or after controlled soaking can give a fair idea about the strength which can be correlated with collapse potential.

In addition to insitu conditions, UCC tests were to be carried out on soaked specimens and samples obtained after flooding. The soaked specimens collapsed/deformed excessively under self weight (Fig 4.7) and thus UCC test could not be performed on those specimens.

Figure 4.7 Specimens kept for soaking.

Figure 4.8 show the UCC test results of (a) dry/insitu conditions (b) after wetting of three weeks and (c) after four weeks of drying following the wetting period. The figure clearly shows a significant drop in UCS at all depths except at 16 feet where the loss in strength is minimum. Figure 4.9 and 4.10 respectively show the comparison of insitu and wetted UCC and strength ratio of wetted and dry conditions at different depths. A strength ratio closer to 1 indicates a stable soil whereas a much smaller strength ratio is expected for collapsing soils. The results clearly reflect a strength loss of more than 50% except at depth of 16 feet where it is minimum.

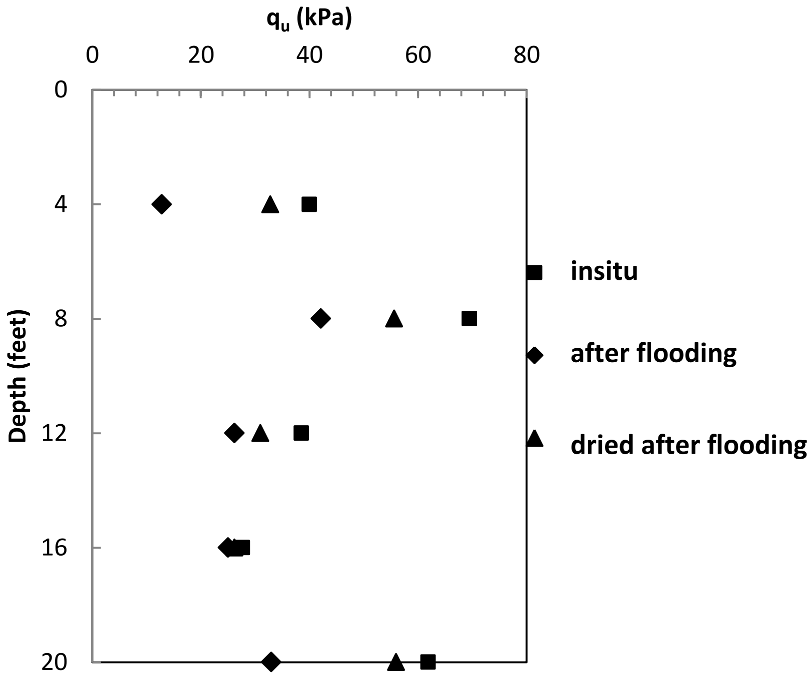


Figure 4.8 UCC test results at different moisture contents.

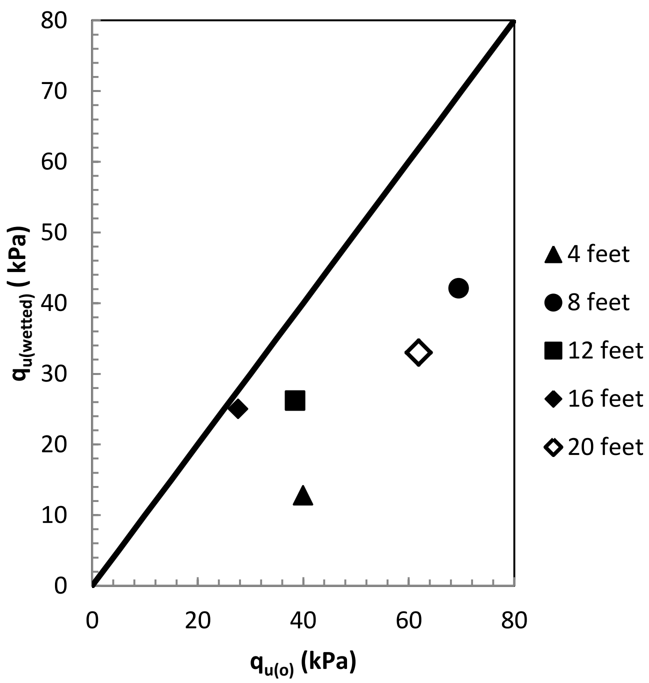


Figure 4.9

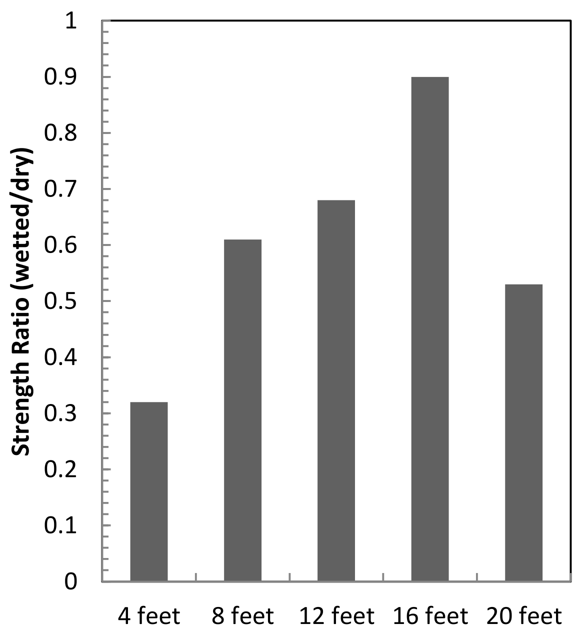


Figure 4.10

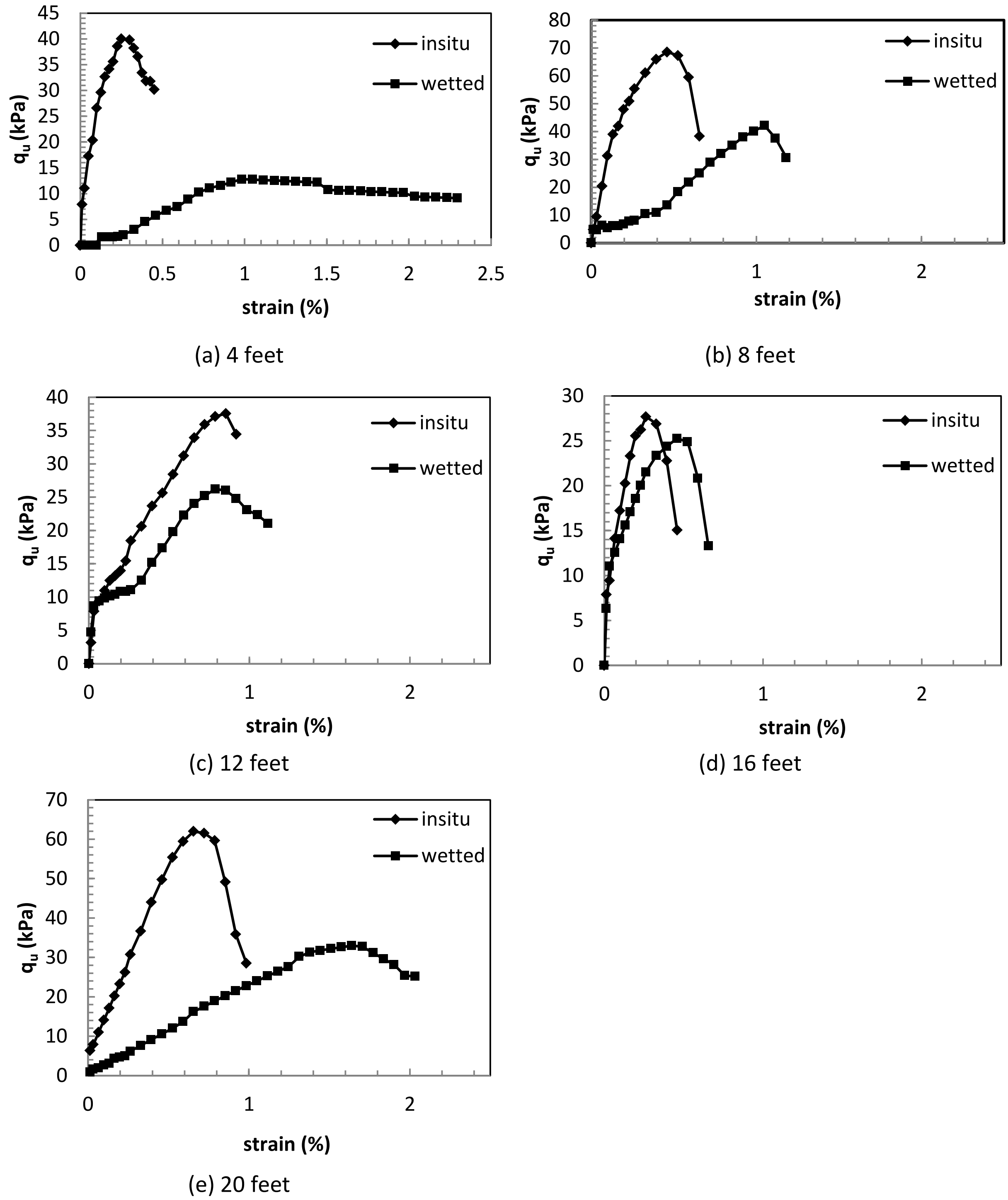


Figure 4.11 comparison of UCS of insitu and wetted soil at different depths.

Figure 4.11 compare UCC test results before and after flooding of test pit #2. It can be seen that there is a more dramatic strength loss at shallow depths. Moreover the failure strain at 4, 8 and 20 feet depths are significantly increased.

4.4.3 Field Tests

4.4.3.1 Plate Load Test

4.4.3.1.1 Insitu condition

Plate load test was carried out in accordance with ASTM as described in section 3.4.2.1, 18 inch diameter plate was used and a load of 240 kPa was applied in three increments i.e., 60kPa, 120kPa, 240kPa. The final load of 240 kPa was left in place for approximately 2 hours after which the pit was flooded. Figure 4.12 show the time versus settlement relations obtained from this test. It can be seen from Fig 4.12 that (a) settlement is mostly instantaneous, i.e., bulk of the settlement occurs with the application of load, (b) settlement is completed with 60 to 90 minutes of load application, and (c) flooding increase the rate of settlement and resulted in the settlement of 2.26cm within 30 minutes of flooding.

The Plate Load assembly was left overnight and it was ensured that the test pit remained saturated. After a lapse of 10 hrs it was observed that the load had dropped from 240 kPa to 90kPa (Fig 4.13) with a recorded settlement of 2.304 cm. An attempt to load the soil to 240 kPa was made but it failed. The soil showed excessive settlement (the bearing plate was observed to be punching into the ground) but could not take more than 120 kPa of load. The load was maintained at 120 kPa and a final settlement of 5.3 cm was recorded after a lapse of 5 hrs. The load settlement relation from the test is shown in Fig 4.14.

The inability of soil to take the applied load after saturation indicates a serious collapse hazard to foundations in the project area. It is probably due to this reason that even lightly loaded (single storey) structures experience severe distresses resulting the structure unserviceable or increasing the maintenance cost.

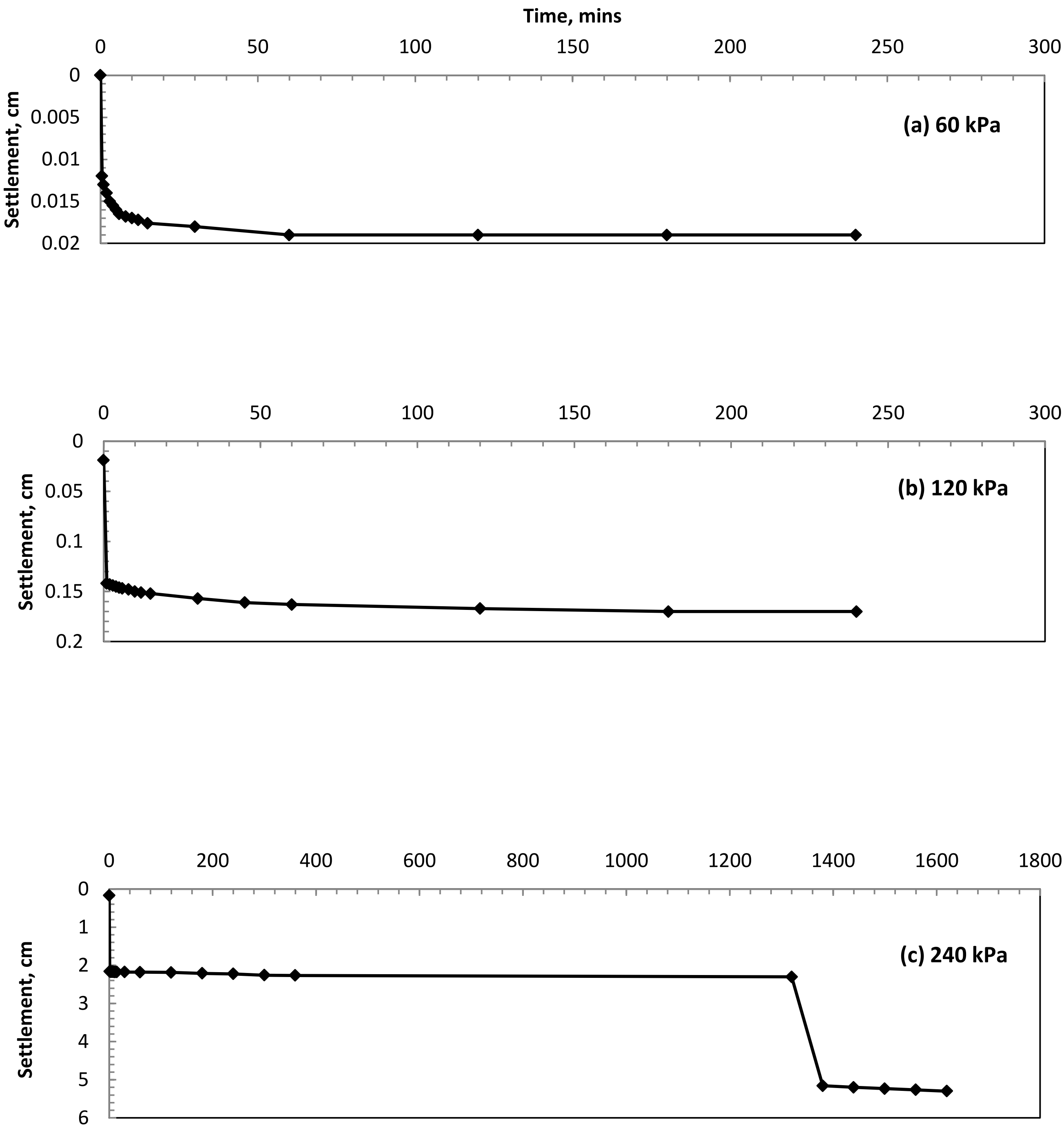


Figure 4.12 Time v/s Settlement relation at different applied loads.

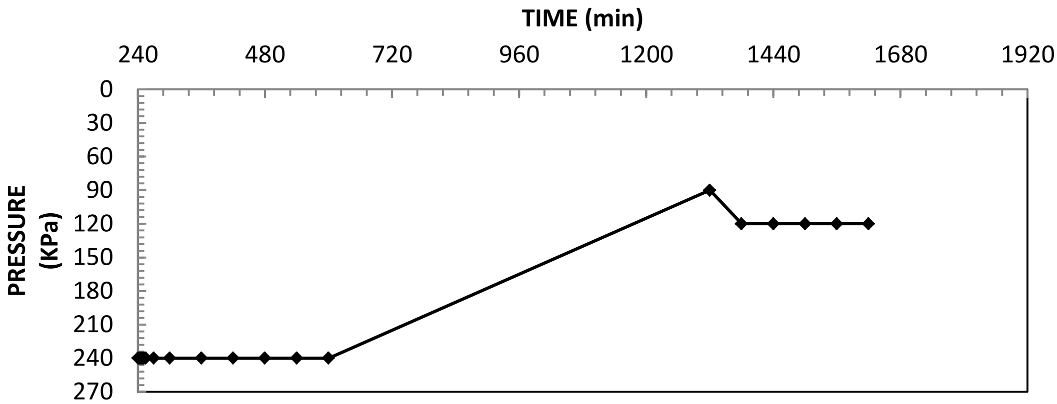


Figure 4.13 Graphs showing settlement with varying pressure at 240 kPa load

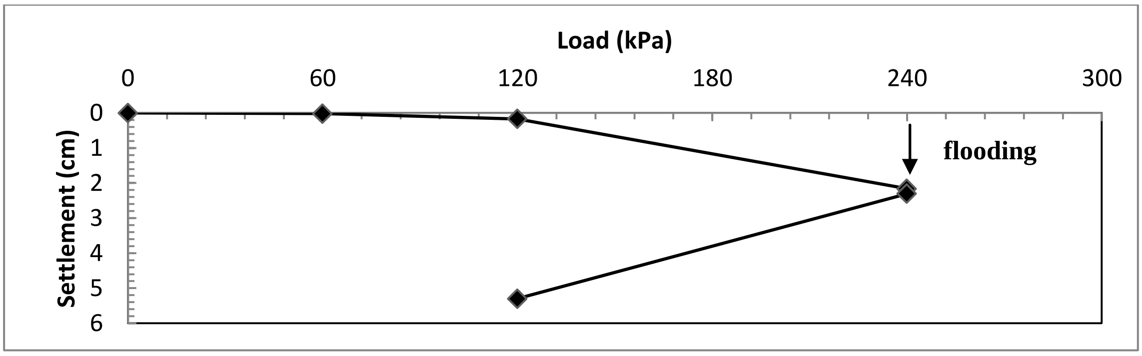


Figure 4.14 Load settlement relation.

4.4.3.2 SPT

Though SPT is not reported to be used for predicting collapse in the reviewed literature; the blow count before and after flooding can serve as an indicator of collapse.

SPT blow counts were recorded after a predefined wetting period in addition to insitu dry condition. SPT blow counts were also recorded after a drying period followed by wetting. Figure 4.15 shows the SPT blow counts in, (a) dry/insitu conditions, (b) after wetting of four weeks, and (c) after four weeks of drying following the wetting period. The figure clearly shows a significant drop in SPT blows at all depths except at 16 feet where the percentage drop in blows is minimum. The trend is quite similar to strength loss predicted by UCC test; i.e., the loss in blow counts is maximum at shallow depth and relatively smaller at depths of 12 and 16 feet.

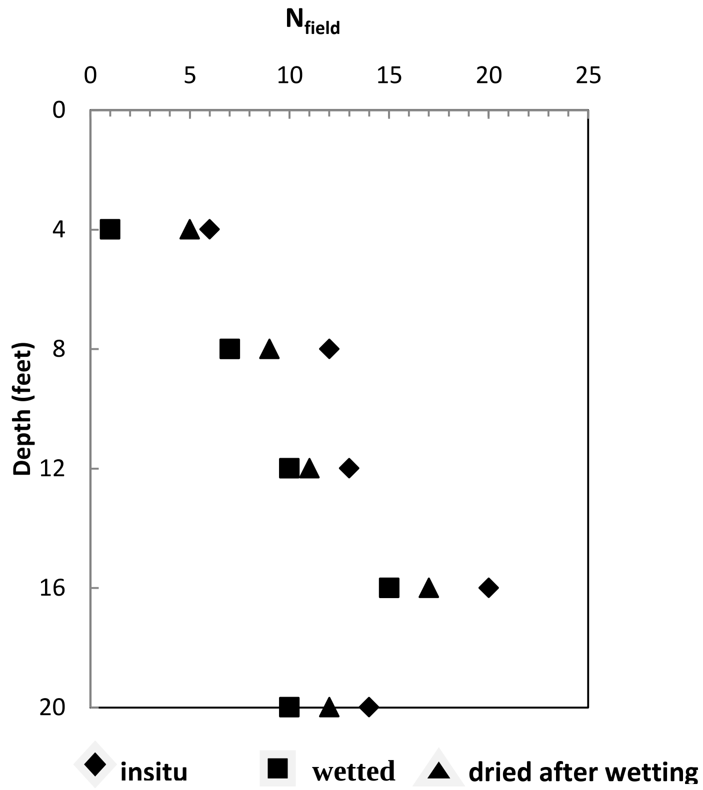


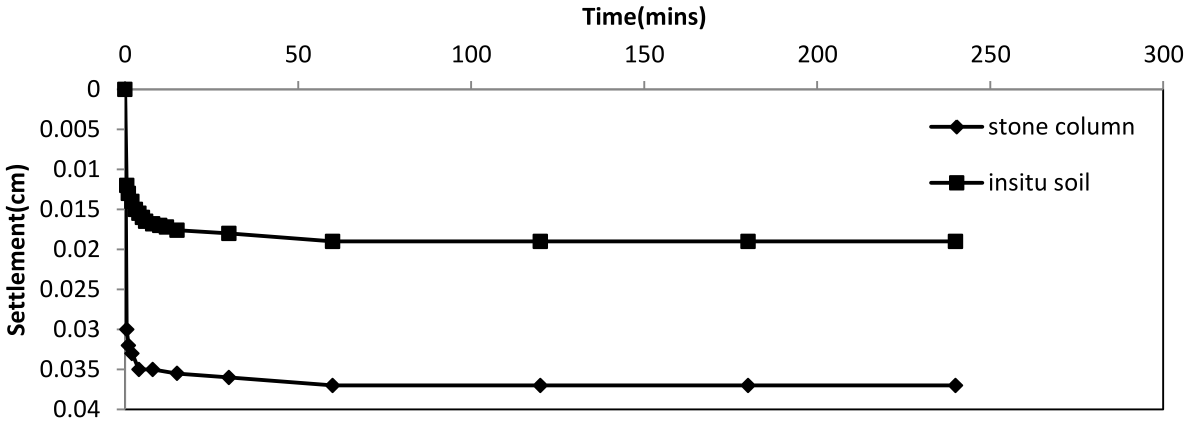
Figure 4.15 Variation in SPT at insitu, wetted and dried after wetting condition.

4.5 Collapse Mitigation

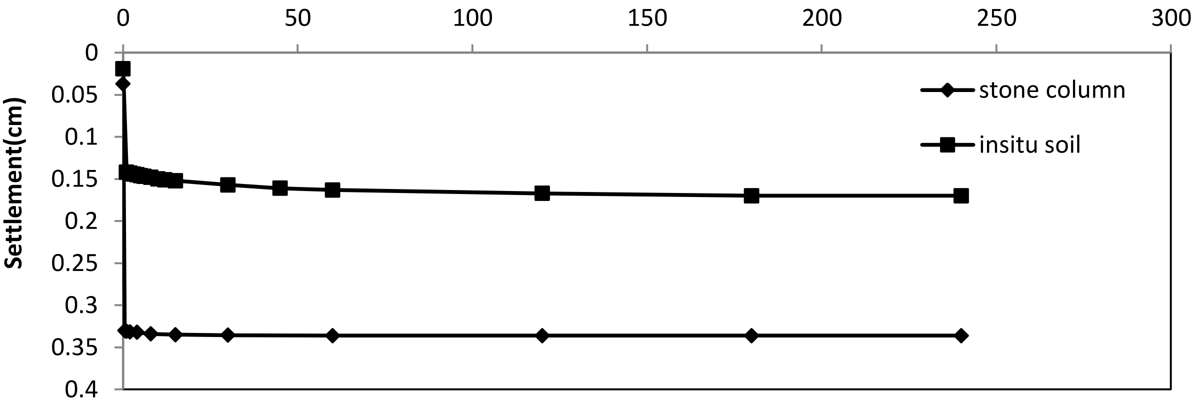
Several methods are available for mitigating the collapsible soil as discussed in chapter 2. Mitigation was not the scope of this project, however, a preliminary study was carried out using a single stone column (with an area ratio of 11%) and plate load test as described in section 3.4.2.2. Results from load tests along with stone column are shown in Fig 4.16. The plate load test on the stone column resulted in a total settlement of 3.343cm. Similar to insitu soil, the treated soil was loaded to 240 kPa in increments. It can be seen from Fig 4.16 that the initial increments with stone column show a greater settlement than the natural soil; however, under the final increment, the settlement is reduced both before and after flooding. The reason for the behavior could be:-

- Stone column derive its strength by mobilizing the passive resistance of soil i.e., a stone column will take load after it has deformed.
- The compactive effort in this case was much smaller (12 lb hammer with a drop height of 2 feet) as compared to convention vibrator weighing 5-7 tons.
- Thus during the initial load increments, a greater settlement was observed as the soil was still weak and stone column was not contributing , however, with the increase in load the stone column deformed, which mobilized its strength and hence a significant reduction in collapse potential.

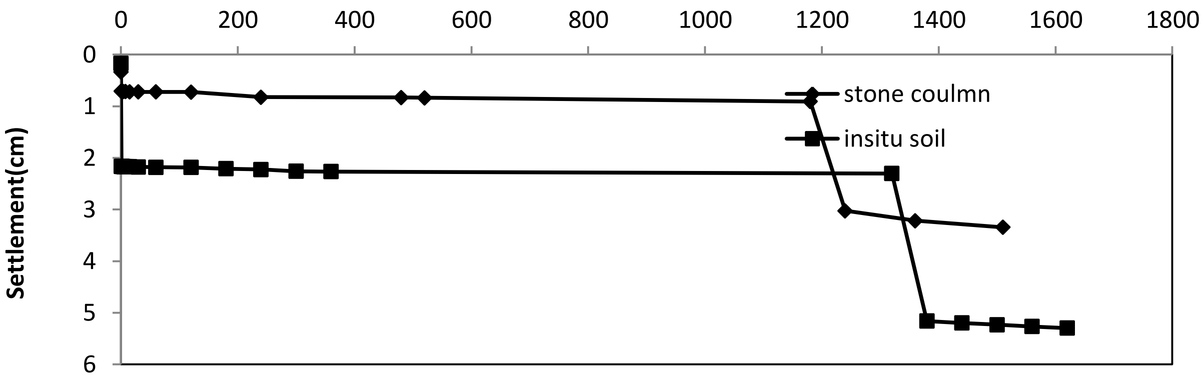
As in previous case the flooding resulted in a loss of pressure which reduced to 40 kPa as shown in Fig4.17. However in this case the soil could take the full load of 240 kPa which shows a significant improvement in soil strength. Figure 4.18 compare the load settlement relation for treated and untreated soil. It can be seen that stone column resulted in a significantly less settlement at a greater pressure as compared to untreated soil. Thus stone columns can be effectively used as a mitigation measure against collapsible soil of Risalpur.



(a) 60 kPa



(b) 120 kPa



(c) 240 kPa

Figure 4.16 Time Settlement relation at various applied loads.

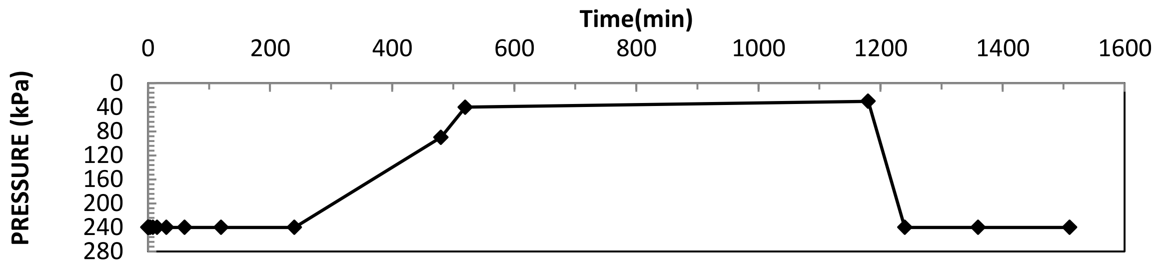


Figure 4.17 Graph showing settlement at varying pressure at 240 kPa load.

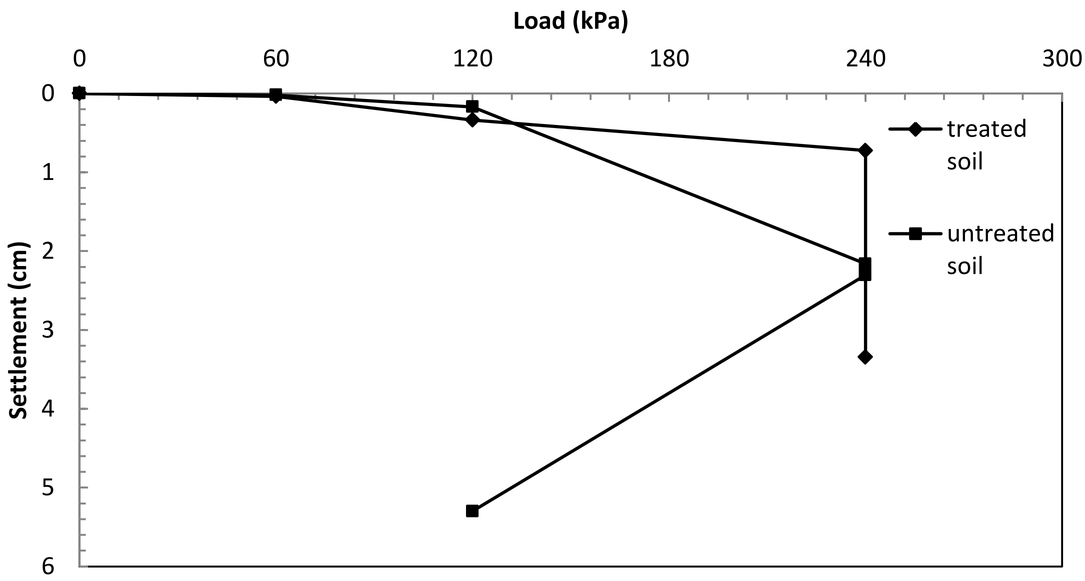


Figure 4.18 Load settlement relation for treated and untreated soil.

Chapter V

Conclusions and Recommendations

5.1 Conclusions

Following major conclusions can be drawn from this study

- Existing criteria are useful to identify collapsible soil; however, laboratory and field tests are essential to evaluate/quantify collapse potential of soil.
- Risalpur soil upto the depth of investigation is collapsible; however, the magnitude of collapse generally decreases with depth.
- Collapse is "severe" at 4 feet depth which is the typical depth of shallow foundation.
- Excessive deformation of UCC specimen due to soaking indicates that foundations can fail quickly when exposed to water. Thus major structural damages are expected if foundation get inundated.
- Soil regains its strength when it is dried after flooding; this indicates the presence of cementing agents which re-precipitate at particle contacts on drying.
- Stone column is an effective measure to mitigate the collapse; however, a detailed study is required to find out optimum area ratio and other parameters thus stone columns may be used under foundations to control settlements.

5.2 Recommendations

- Soil is found collapsible within the zone of influence of shallow foundation; there is need to investigate the extent/thickness of collapsible layer to ascertain whether it will affect deep foundations.
- Different methods of treating collapsible soil at Risalpur may be investigated to find out most economical solution.
- Use of stone has shown promising results; however, a more elaborate study is required for meaningful conclusions.
- Soil improvement may be considered as an integral part of future construction.
- Study may be undertaken to suggest retrofitting of foundations of damaged structure before undertaking structural repair.

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