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Evaluation of Some Geo Mechanical Parameters of the Soil Samples from Ganakbari Area, Dhaka, Bangladesh

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ABSTRACT

Geo mechanical parameters are evaluated here in. The unconfined compressive strength (q_u) value ranges from o.38 kg/cm^2 to 1.60 kg/cm^2 and the average value is 1.06 kg/cm^2. The analyzed soil is 'medium to stiff' based on 'q_u '. The UCS is closer to the typical strength for kaolinite and illite. The soil is brittle to ductile in nature. The values of cohesion (C) vary from 0.1 kg/cm^2 to 0.6 kg/cm^2 in the Madhupur Clay. The values of the angle of internal friction (ϕ) vary from 7º to 34º. The values of cohesion decrease with increasing the values of moisture content. The obtained values of 'ϕ' are closer to the typical values for illite and kaolinite. The shear stress increases linearly with increasing normal stress. The compression index (C_c) varies from 0.16 to 0.20 and the average value is 0.18. The soil samples are medium plastic based on compression index values. The compression index (C_c) increases with increasing the value of void ratios and moisture content. The soil might be normally consolidated to over consolidated in nature.

Keyword: *Geomechanical parameters, unconfined compressive strength, shear strength,* compression index.

1. INTRODUCTION

The aim of evaluation of geomechanical parameters is to determine the subsurface condition and soil strength that helps to develop structure or foundation in an area. A geomechanical investigation has been carried out at Ganakbari, Savar, Dhaka by the detailed subsurface investigation programme which includes nine borings, execution of the Standard Penetration Test (SPT), collection of the soil samples and performance of laboratory tests. Dhaka, the capital of Bangladesh, is expanding rapidly. Dhaka is characterized by tropical, humid climate conditions. The Madhupur Clay soil of the area consists mainly of silt and clay, which is sticky when wet and shrinks when dry.

Ganakbari and its adjoining areas are the recently developed part of Dhaka city. The investigated area is situated on the yellowish brown to grey, mottled, sticky clay. It is important to know the geomechanical parameters of the area in order to use the result in the design and analysis the geo-engineering problems in the region. The study area (Ganakbari) comprises the northern extremity of Dhaka District and located within longitude 90º13´E to 90º18´E and latitude 23º55´N to 23º58´N (Fig. 1). Because of rapid urbanization and huge population and nearest to the export processing zone (EPZ), the study area is very much important and familiar to the people. This paper deals with the behaviour of the soil samples of the investigated area which include the geomechanical parameters such as the unconfined compressive strength, the shear strength and consolidation parameters of the soil.

2. MATERIALS AND METHODS

In the investigated area, nine (9) borings each extending 65 feet depth, have been selected for this research work. Both disturbed and undisturbed samples were collected in the field. The disturbed soil samples have been collected by using split spoon sampler with the performance of the standard penetration test (SPT). These soil samples have been extracted from every 5 feet depth up to the investigation in case of all the boreholes. The undisturbed soil samples were collected from the cohesive layers by hydraulic rotary method with the help of thin open Shelby tubes. A thin walled shelby tube having 76mm diameter was penetrated into the undisturbed soil formation at the bottom of the borehole by applying rapid but continuous force. The samples covered within the shelby tubes were sealed immediately with aluminum foil and then sealed with paraffin wax in order to measure UCS, the shear strength and consolidation parameters.

The geotechnical parameters were determined in the soil test laboratory in the Housing and Building Research Institute (HBRI), according to standard practice [1]. The UCS and the shear strength parameters were determined from graphs but the compression index was determined according to the following equation:

$$C_C = \frac{e_1 - e}{\log_{10} P - \log_{10} P_1}$$

Where, C_c = Compression index, e_1 =initial void ratio, e= final void ratio, P_1= initial pressure, P= final pressure.

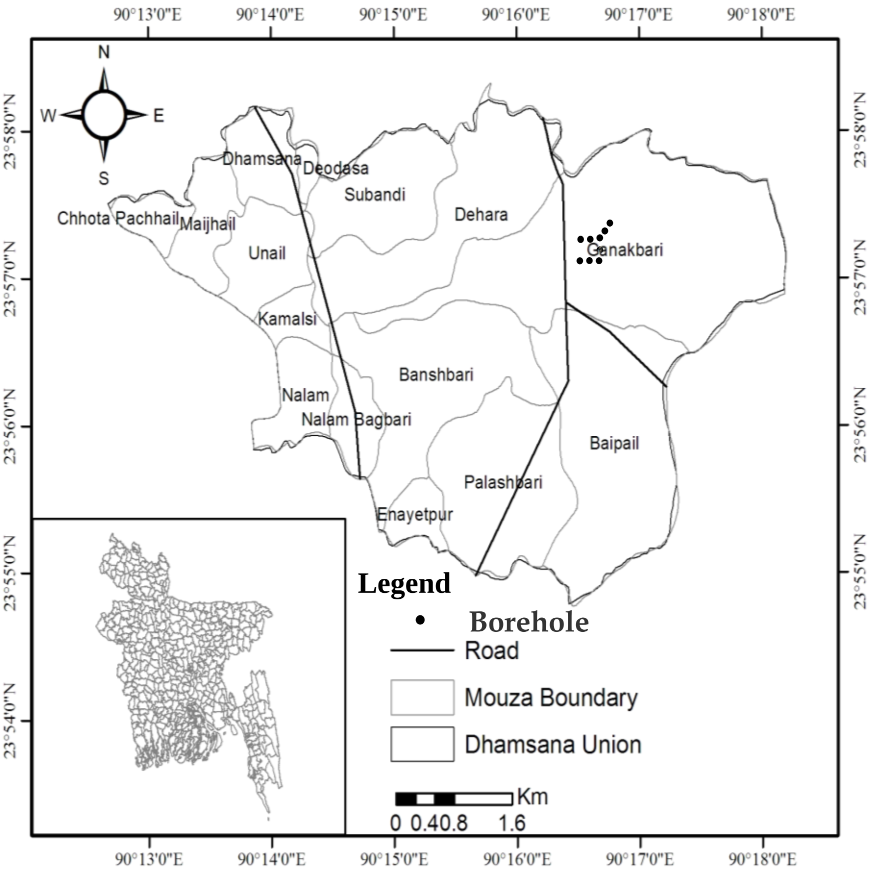

Fig 1: Location map of the study area

3. BOREHOLE LITHOLOGY

The litho logy of the study area has been revealed from the soils/rocks encountered in the wells bored (table 1). The variety of soils has been developed because of difference in the degree of weathering in different boreholes of the study area. Three units namely A, B and C have been identified. Individual soil/rock unit has different litho logy in the study area. Unit A is mainly a non cohesive sandy unit. This unit is light reddish yellow or pale reddish yellow in colour, dense to very dense in nature, and fine to very fine grained sand. Some or little silt is also present in this unit.

The thickness is around 15 feet and depth ranges from 50 to 65 feet in all the bore holes. Unit B is mainly a silty sand unit. Unit B overlies the unit A in this area. This unit is mottled, light reddish yellow or pale reddish yellow in colour and medium dense to dense in nature. The thickness is around 22 feet and depth ranges from 28 to 50 feet in all the bore holes. Unit C is mainly a clay unit of 28 feet thickness which overlies the unit B in this area. This unit is pale grayish to light reddish yellow or pale yellowish in colour, mainly medium to stiff in nature and show medium plasticity. Some or little silt is also present in this unit.

Table 1: The litho logic description of the study area from bore log data.

Unit	Soil/Rock Type	Litho logic Description	Depth (ft)	Thickness (ft)
C	Clay	Pale grayish ash to light reddish, yellow ash or pale yellowish ash to light reddish yellow ash, mottled, medium plastic and medium stiff to stiff clay.	0-28	28
B	Silty sand	Light reddish yellow ash, mottled low plastic, stiff silt, little fine sand or pale reddish yellow, medium dense to dense, silty fine sand.	28-50	22
A	Sand	Light reddish yellow, dense to very dense, fine sand or pale reddish yellow and yellow, dense fine sand, with some/ little silt.	50-65	15

4. RESULTS AND DISCUSSION

4.1 Unit Weight

The unit weight (dry and wet) values are shown in table 2. The dry unit weight varies from 1.47 to 1.96 and the average value is 1.74. It varies at different depths and at different boreholes. Generally the results are higher at comparatively higher depth (table 2). The wet unit weight varies from 1.92 to 2.66 and the average value is 2.15. It varies at different depths and at different boreholes. Generally the wet unit weight values are also higher at comparatively higher depth (table 2). The analytical results revealed that the unit weight is variable with depth and boreholes and it generally increases with increasing depth in the analyzed samples (table 2).

4.2 Unconfined Compressive Strength (UCS)

The unconfined compressive strength (q_u) value ranges from o.38 kg/cm^2 to 1.60 kg/cm^2 and the average value is 1.06 kg/cm^2 (table 2) . The results are variable at different depths and at different boreholes and are comparatively low in BH-3 (depth 8.5 feet) and in BH-4 (depths 8.5 feet and 13.5 feet). These variations may be due to the effect of mechanical action during sample collection and preparation, trimming, grain size, mineral content and change in moisture content. The strain at failure point ranges from 5% to 14% in all the samples. The strain percentages at failure are much closer and consistent in most of the samples. A few sample in BH-3 and BH-4 show lower strain in comparison with others.

The UCS of Madhupur Clay soils of Mirpur area and mentioned that the unconfined compressive strength ranges from 10.72 psi (0.75 kg/cm^2) to 21.65 psi (1.52 kg/cm^2) and the average value is 18psi (1.27 kg/cm^2) [2]. The obtained results are very much closer to the recommended value for 'medium to stiff' soil based on 'q_u' [3].

Table 2: Geomechanical parameters of some soil samples of Ganakbari, Savar.

BH No.	Sample No.	Depth (ft)	Unit Weight		Unconfined Compression Test		Consolidation Test		Direct Shear Test	
			Wet unit weight (gm/cc)	Dry unit weight (gm/cc)	Compressive Strength (kg/cm^2)	Strain (%)	Natural void ratio	Compression Index (Cc)	Cohesion (kg/cm^2)	Angle of Internal friction (°)
1	UD-1	8.5	2.14	1.76	0.85	9	0.64	0.18	0.37	9
	UD-2	13.5	2.18	1.828	1.05	12				
2	UD-1	8.5	2.14	1.78	1.134	10				
	UD-2	13.5	1.92	1.47	1.20	12			0.5	12
	D-7	35							0.05	22
3	UD-1	8.5	2.66	1.789	0.65	6	0.75	0.2		
	UD-2	13.5	2.07	1.67	0.939	8			0.4	10
	D-12	60							00	29
4	UD-1	8.5	2.02	1.59	0.75	6			0.30	7
	UD-2	13.5	2.00	1.586	0.38	5				
	D-13	65							00	34
5	UD-1	8.5	2.12	1.727	1.00	9	0.68	0.176	0.40	10
	UD-2	13.5	2.13	1.76	1.15	10				
	D-8	40							0.05	21
6	UD-1	8.5	2.12	1.758	1.25	10				
	UD-2	13.5	2.18	1.83	1.40	12	0.58	0.16	0.55	12
	D-5	25							0.15	17
7	UD-1	8.5	2.11	1.70	0.95	10				
	UD-2	13.5	2.31	1.96	1.20	12			0.50	11
	D-10	50							0.05	28
8	UD-1	8.5	2.09	1.67	1.00	10			0.40	10
	UD-2	13.5	2.28	1.92	1.22	11				
	D-8	40							0.05	22
9	UD-1	8.5	2.14	1.74	1.513	12	0.69	0.16	0.55	10
	UD-2	13.5	2.16	1.80	1.60	14			0.60	12
	D-11	55							00	31

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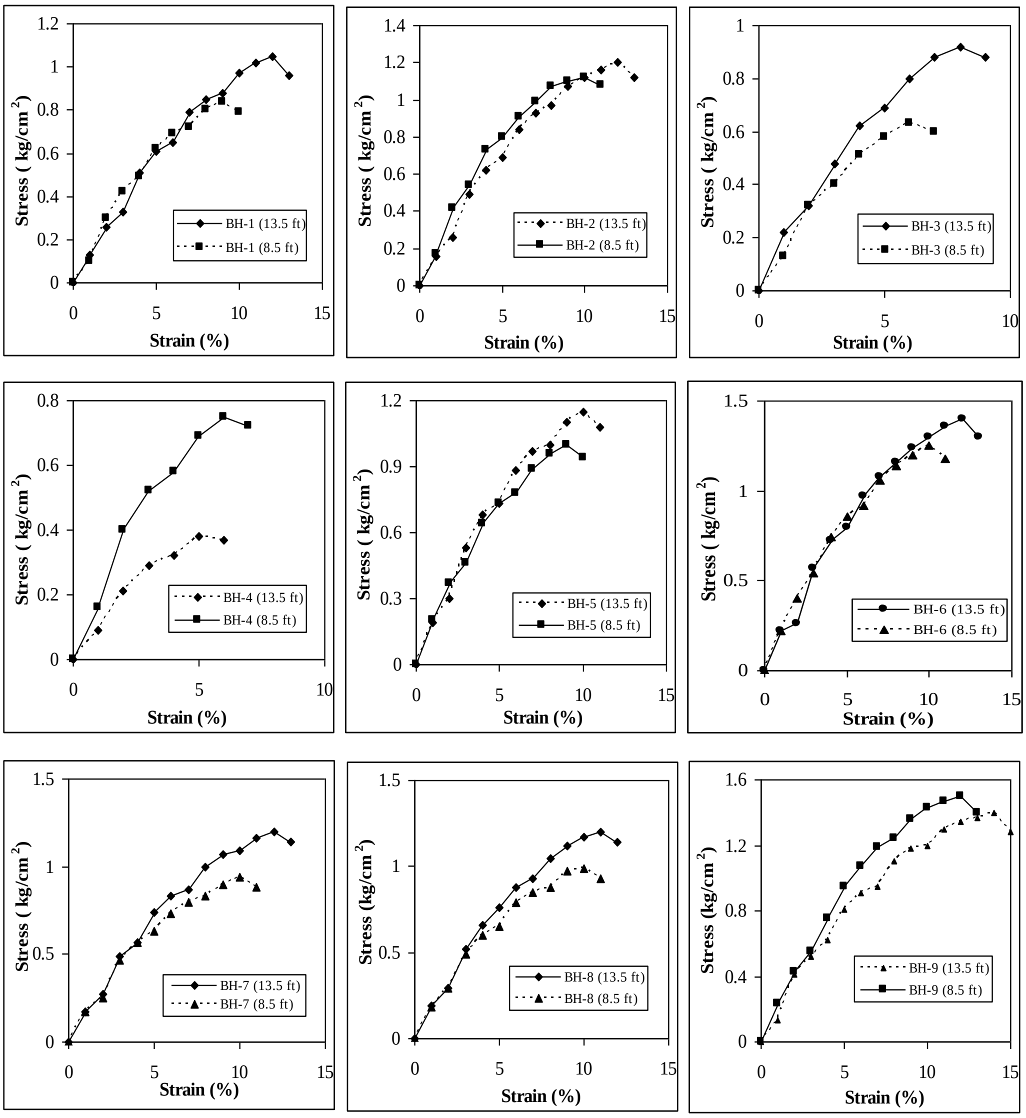

Fig 2: Relationship between stress and corresponding strain in UCS tests at different boreholes.

The montmorillonite yields lower strength than kaolinite when not mixed with sand but yields higher strength than kaolinite when mixed with sands. For clay minerals, the strength would increase with increasing percentages of clay minerals in the following order: montmorillonite, illite, kaolinite [3]. The Madhupur Clay is mainly kaolinitic and Illitic in composition [2, 4 & 5]. So, its strengths should lie closer to the strength for kaolinite and illite.

The relationship between stress and corresponding strain in UCS tests at different boreholes are shown in figure 2. The stress-strain curves (for UCS) are almost linear in nature up to the soil failure. From the failure point, the graphs decline sharply in almost all the cases, and in a few cases the curvature is smooth. It emphasizes the fact that the soil is brittle to ductile in nature. The unconfined compressive strength increases with decreasing the moisture content in the analyzed samples (figure 3).

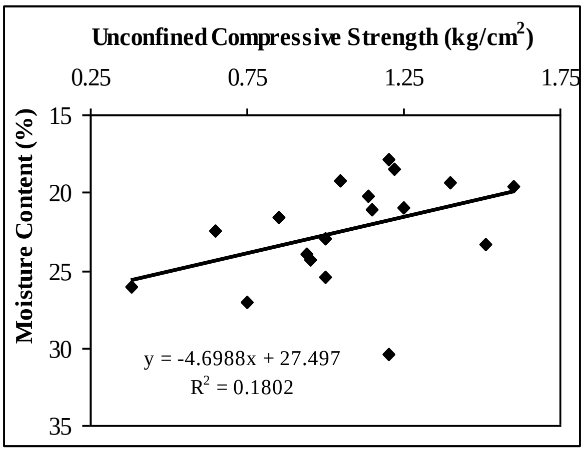

Fig 3: Relationship between unconfined compressive strength and moisture content (%).

4.3 Shear Strength

The shear strength parameters obtained from direct shear test are listed in the table 2. The values of cohesion (C) vary from 0.1 kg/cm^2 to 0.6 kg/cm^2 in the cohesive soil (Madhupur Clay). A few samples bellow the cohesive layer has been analyzed which show that the value of cohesion is zero or almost zero. The values of the angle of internal friction (ϕ) vary from 7º to 34º.

The Madhupur Clay soils of Dhaka and Savar area especially from JU campus, Mirpur and Curzon Hall area and mentioned that the value of cohesion lies between 3.5 psi (0.25 kg/cm^2) and 6.5 psi (0.46 kg/cm^2) and the angle of internal friction (ϕ) ranges from 11º to 36º [2]. The analyzed results are very much consistent with Hossain & Haque (1995) [2]. The obtained values of 'ϕ' are closer to the typical values for illite and kaolinite [3] and also consistent with clay mineralogy of Madhupur Clay [2, 4].

There is little variation of the cohesion and the angle of internal friction in different boreholes and in different depths (table 2). This variation of the shear strength parameters may be due to the disturbance during sampling, trimming and cutting, the degree of weathering, water content and clay fraction [6].

The results also suggest that the cohesion is relatively higher in borehole 9 which may be due to the high percentage of clay content. The angle of internal friction is higher in BH-3, BH-4 and in BH-7 at greater depth due to the abundance of cohesionless soil (sand) in those particular depths in the boreholes. In the case of direct shear test, the value of cohesion slightly decreases with increasing depth in different boreholes (table 2).

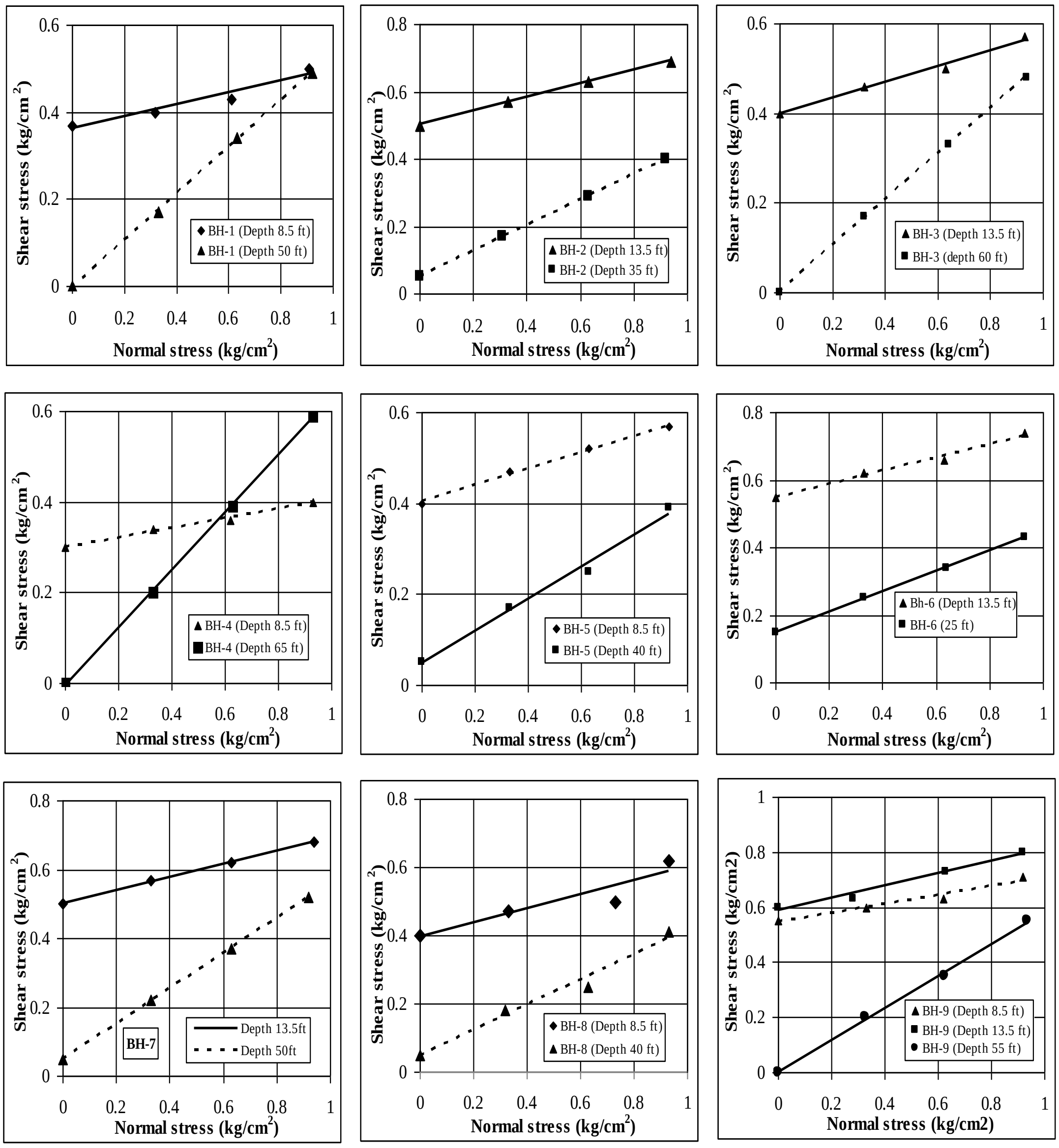

Fig 4: Relationship between normal stress and corresponding shear stress in direct shear tests at different boreholes.

The relationship between normal stress and shear stress (in direct shear test) of different samples are shown in figure 4. The shear stress increases linearly with increasing normal stress. The straight line intersects with the shear stress axis in case of cohesive soil giving rise to a value of cohesion in the analyzed samples. Almost all the obtained points touch the shear strength envelope. Only a few points in figure 4 lie bellow the strength envelopes which are not harmful for engineering works.

It is also evident that the cohesion decreases with increasing the angle of internal friction in the analyzed soil (figure 5). The cohesion is higher in the cohesive soil where as it is lower or tends to zero in granular or non cohesive soil. That is why, this type of relationship between cohesion and internal friction is found. It is also found that the values of cohesion decrease with increasing moisture content (figure 6).

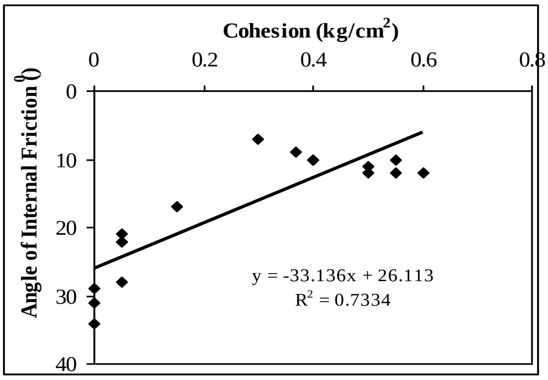

Fig 5: Relationship between cohesion (C) and angle of internal friction (ϕ).

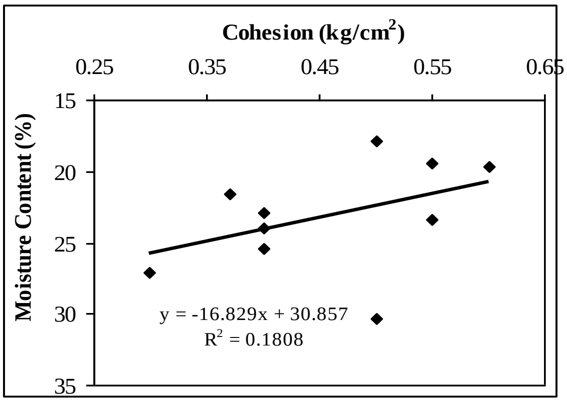

Fig 6: Relationship between cohesion (C) and moisture content (%).

4.4 Consolidation Behaviors

The consolidation parameters such as compression index as well as natural void ratio value are listed in table 2. The compression index (C_c) varies from 0.16 to 0.20 and the average value is 0.18. The natural void ratio ranges from 0.58 to 0.75 and the average value is 0.67. The obtained values are very much close to the values quoted by several authors for Madhupur Clay [2, 5, 7-8].

The compression index values slightly vary in different boreholes and in different depths and the value of compression index (C_c) increases with increasing the value of void ratios (e) and also with the moisture content of the soil (figures 7 & 8). The variation of compression index values may be due to the disturbance during sampling, due to the effect of sample preparation and trimming. The relationship between void ratio (e) and applied pressure on soil samples of different boreholes are shown the in figure 9 and suggest that the soil is normally consolidated to slightly over consolidated in nature. The void ratio decreases continuously with the increase in applied pressure in all the samples and followed a nearly smooth curve. In case of lowering the applied pressure, the void ratio again increases in a regular pattern following a smooth curve in all the samples, indicating the ideal relationship between void ratio and applied pressure in soil mechanics.

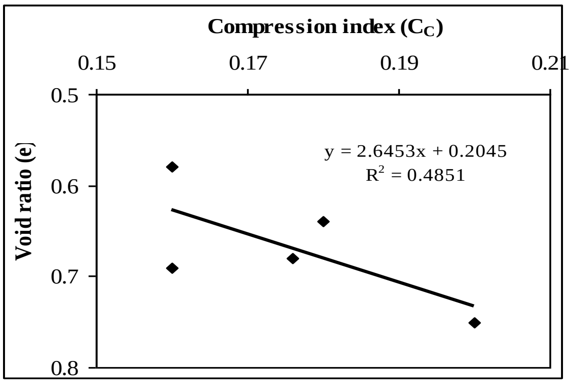

Fig 7: Relationship between void ratio (e) and compression index (C_C).

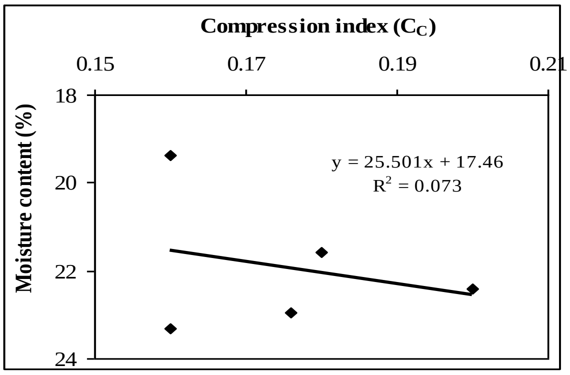

Fig 8: Relationship between moisture content (%) and compression index (C_C).

The inorganic soil having compression index (C_c) value from 0.2 to 0.8 are medium plastic and soil having compression index (C_c) value up to 2.0 are high plastic [9]. And different clay minerals have variable C_c values i.e. the values range from 0.5 to 1.10 for illite, from 0.19 to 0.28 for kaolinite and 1.0 to 2.6 for montmorillonite in different ionic forms [10]. The analyzed results suggest that the studied samples are kaolinitic composition and medium plastic nature.

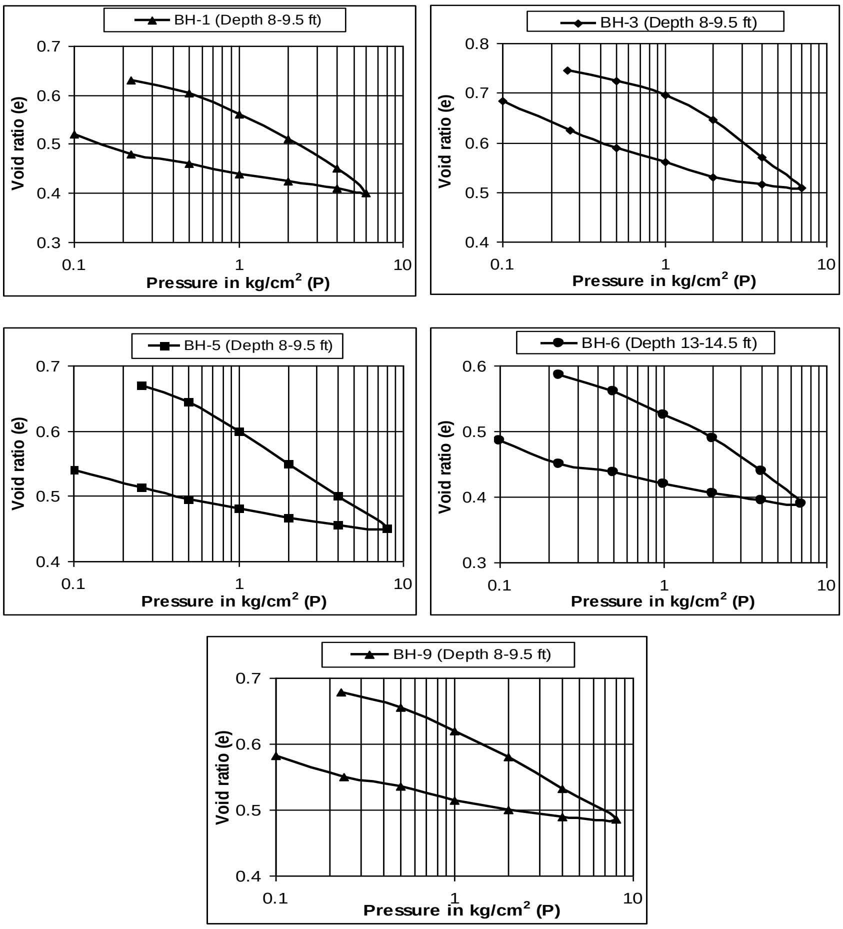

Fig 9: Void ratio (e) versus log pressure (log P) curves for the samples of different boreholes.

5. CONCLUSION

This paper deals with some geomechanical behaviour of the soil samples of Ganakbari, Savar. From the analytical results the following conclusion can be drawn. The unconfined compressive strength (q_u) value ranges from o.38 kg/cm^2 to 1.60 kg/cm^2 and the average value is 1.06 kg/cm^2. The results are variable at different depths and at different boreholes but much closer to one another. The analyzed soil is 'medium to stiff' based on 'q_u'. The UCS is closer to the typical strength for kaolinite and illite. The soil is brittle to ductile in nature. The unconfined compressive strength increases with decreasing the moisture content. The values of cohesion (C) vary from 0.1 kg/cm^2 to 0.6 kg/cm^2 in the Madhupur Clay. The value of cohesion is zero or almost zero in noncohesive soil. The values of the angle of internal friction (ϕ) vary from 7º to 34º. The value of cohesion slightly decreases with increasing depth in different boreholes. The cohesion also decreases with increasing the angle of internal friction in the analyzed soil. It is also found that the values of cohesion decrease with increasing the values of moisture content. The obtained values of 'ϕ' are closer to the typical values for illite and kaolinite. The shear stress increases linearly with increasing normal stress. The compression index (C_c) varies from 0.16 to 0.20 and the average value is 0.18. The natural void ratio ranges from 0.58 to 0.75 and the average value is 0.67. All the samples show a narrow range of compression index values. The soil samples are medium plastic based on compression index values. The compression index (C_c) increases with increasing void ratios and moisture content. The e-log P graph indicates the ideal relationship between void ratio and applied pressure in soil mechanics and also suggests that the soil is normally consolidated to slightly over consolidated in nature.

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