CHAPTER- VI

SHEAR STRENGTH PARAMETERS OF HOMOGENEOUS AND STRATIFIED SOIL-ASH DEPOSIT

6.1 INTRODUCTION

One of the biggest difficulties facing the globe right now is finding solutions and alternatives to the issue of trash generation and management. The most economical and plentiful source of electricity in India is coal, and in 2017–18, 721.66 million tonnes of coal were produced (Energy statistics 2019). Fly ash makes up 70–80% of the coal combustion residue in natural coal, which has an average ash concentration of 30–50% (Chikkatur et al. 2009; Sadasivan and Negi 1991; Singh et al. 2012). In addition to being utilized as a building material in cement mortar, fired bricks, polymer concrete, self-compacting concrete, reclaimed asphalt pavement, nano-silica, and cement stabilized macadam, fly ash is also used for soil stabilization and the removal of components like phosphorus and phenol (Chindaprasirt et al. 2004; Lingling et al. 2005; Ram and Mohanty 2022; Turk and Karatas 2011; Ghazy et al. 2016; Yan et al. 2019; Bijen 1986; Sun and Guo 2015; Sharma and Swain 2012; Ugurlu and Salman 1998; Sarkar and Acharya 2006).

Most of the research is focuses on homogeneous or stabilized/mixed soils, even though the aforementioned wastes are typically dumped in layers at sites or always placed in layers if they are utilized as a building or geomaterial, particularly in the case of pavements or embankments. For the building of embankments and pavements, several materials must be used due to environmental restrictions and the uneven availability of acceptable geomaterials. New researchers are driven to create innovative solutions to the problem of industrial wastes and byproducts in order to reduce and utilize the waste on a larger scale. Each layer of a layered soil or soil-waste system deforms differently and exhibits different elastic moduli under various stress conditions. Layers interacting within a layered matrix alter the stress distribution, which impacts the overall elastic and plastic strains produced inside each material and, as a result, their reaction to those stresses. Because of the built-up pressures in the layers, using these materials in the pavement in layered form could permanently distort the deposit (Fleming and Rogers 1995).

The shear strength behaviour of waste or soils as a homogenous material has been the subject of extensive research (Kamata et al. 2009; Pandian 2004; Ramamurthy 2001; Xu et al. 2018; Mohanty and Patra 2015). Studies that consider the behaviour of the layered system's strength are quite limited for layered soils. Under three point, triaxial, and unconfined compression loading conditions, Shroff et al. (2004)'s investigation on the soil-fly ash composite mass failure mechanism on cylindrical specimens discovered that the soil-fly ash layered system can increase the cohesion (0 kPa to 69 kPa) and percent failure strain value (9.5%). Murthy et al. (1980) found that the effective stress parameters (c' and ϕ'), as well as two primary orientations, were different and independent of the stress path for layered soil created in the laboratory. The values of ϕ', however, were 6-7 times higher in the extension studies than in compression testing. In several investigations, layered soil or soil-waste systems were examined in order to calculate the parameters related to permeability (Sridharan and

Prakash 2002; Prakash and Sridharan 2013), consolidation (Barden and Younan 1969; Nixon 1973), and bearing capacity/settlement (Misir and Laman 2017; Consoli et al. 2008). Studies that describe the shear strength parameters of layered soil systems and specifically layered soil-waste systems while taking varied field conditions into account are scarce.

In the present study, the investigation of homogeneous and stratified soil-ash deposit has been made. This chapter is precisely focused on the strength and stiffness evaluation of the homogeneous fly ash, local soil & its combinations, determined under various loading and lateral confinement conditions in triaxial test. The effect of layering of soil has also been examined in terms of failure pattern of the samples. The optimum theoretical approach for forecasting the composite system's modulus of elasticity (E) is examined using the layered soil-ash deposit. The main goal is to determine how well the current formulations anticipate the maximum bearing capacity of a layered deposit.

6.2 SIGNIFICANCE OF SHEAR STRENGTH

For the building of earthen dams, road pavements, and liners, the compacted soils are typically laid out in layers, making it imperative to investigate each layer from a geotechnical standpoint. The emphasis now is on finding ways to reuse industrial wastes or byproducts in ways that benefit both the environment and the economy. A thorough shear strength analysis of these wastes both as an individual and as a layered system makes more sense because using any form of waste on a wide scale in the field involves extensive research. In order to analyze the shear strength characteristics and failure behaviour, the study considers all field conditions, including drainage, saturation, and consolidation under compressive triaxial tests. Also, this aids in forecasting the total layered system's bearing capacity, settling, and stability.

6.3 TRIAXIAL TEST TECHNIQUES

The static triaxial assembly was incorporated to conduct compressive strength test of fly ash, local soil, and stratified soil-ash deposits under different confining and saturation conditions. The complete prepared sample of both the arrangements has been shown in Fig. 6.1. The sampling, loading (saturation and confinement), and shearing stage are the three basic phases of the triaxial test. Once Skemton's pore pressure parameter (B) reached 0.99 or higher, the samples were assumed to be saturated with de-aired water (Skempton 1954). Once the samples had reached saturation, they were isotopically consolidated at the necessary effective confining pressure and stopped when there was no change in the extracted volume of water. The samples were then sheared under three distinct controlled loading circumstances, namely the unconsolidated undrained (UU), consolidated undrained (CU), and consolidated drained (CD) states. While the consolidation stage is skipped in the UU test and drainage is prohibited during the shearing phase, these restrictions are not present in the CU and CD tests following consolidation. Instead, drainage is either prohibited or permitted during the shearing phase in these tests. The CD test simulates the actual field circumstances, such as excavation/slope stability or the long-term stability of continuous seepage in the case of an embankment dam. The CU strength is useful for stability analysis when the soil is fully saturated, consolidated, and additional stresses are applied quickly without drainage, such as when embankment dams, canal slopes, or reservoir slopes experience a rapid drawdown. The UU strength is beneficial in crucial design situations where loading is anticipated to happen so quickly that there is no time for consolidation or to dissipate the pore pressure, like when building an embankment or footing over a soft clay deposit quickly.

Fig. 6.1. Prepared cylindrical specimen of homogeneous and stratified soil-ash deposit.

6.4 EXPERIMENTAL PLAN

There are three types of soil samples considered in the present study that have been evaluated for shear strength parameter investigation. The samples were subjected to three different confining pressure for the determination of particular cohesion and angle of friction. Chandra et al. (2016) state that the confining pressure for an embankment/ash tailing made up of fly/bottom ash up to 3 to 5 m height is roughly 10 to 50 kPa. The confining pressure between 50 to 150 kPa has been taken into account in order to access the strength of the composite material at deeper depths since the higher height of tailings exerts a greater confining pressure at the bottom. The detailed testing plan of the present study has been illustrated in the Table 6.1.

Table 6.1 Layout of laboratory experiments for shear strength determination.

	No. of Samples	Type of Test	Confining pressure, σ_c (kPa)
	FA (Fly ash)	Unconsolidated Undrained (UU)	
	LA (Local Soil)	Consolidated Undrained (CU)	50,100,150
	2-Layers (FA+LS)	Consolidated Drained (CD)	
Total	3	3 Each case	3
Total No. of tests			27

6.5 RESULTS AND DISCUSSION

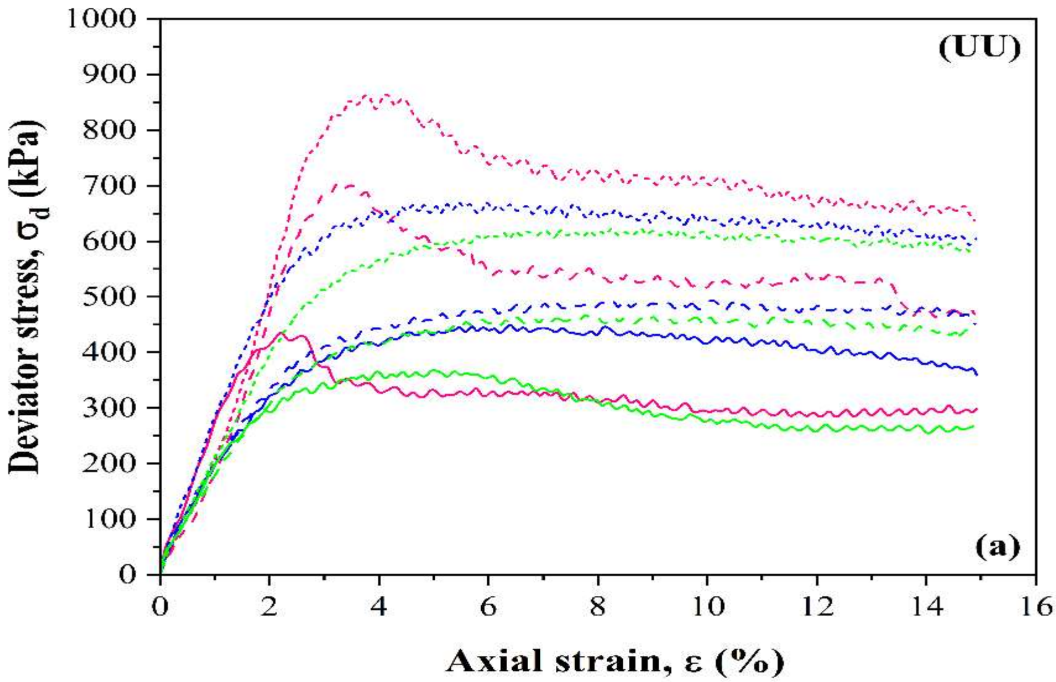
The experimental investigation of triaxial testing plan has been subdivided into multiple headings. Such that along with the strength aspect, the failure mechanism, elastic properties, and the application can be explored for better understanding. The brief explanations of these subheadings were formalized in the subsequent sections.

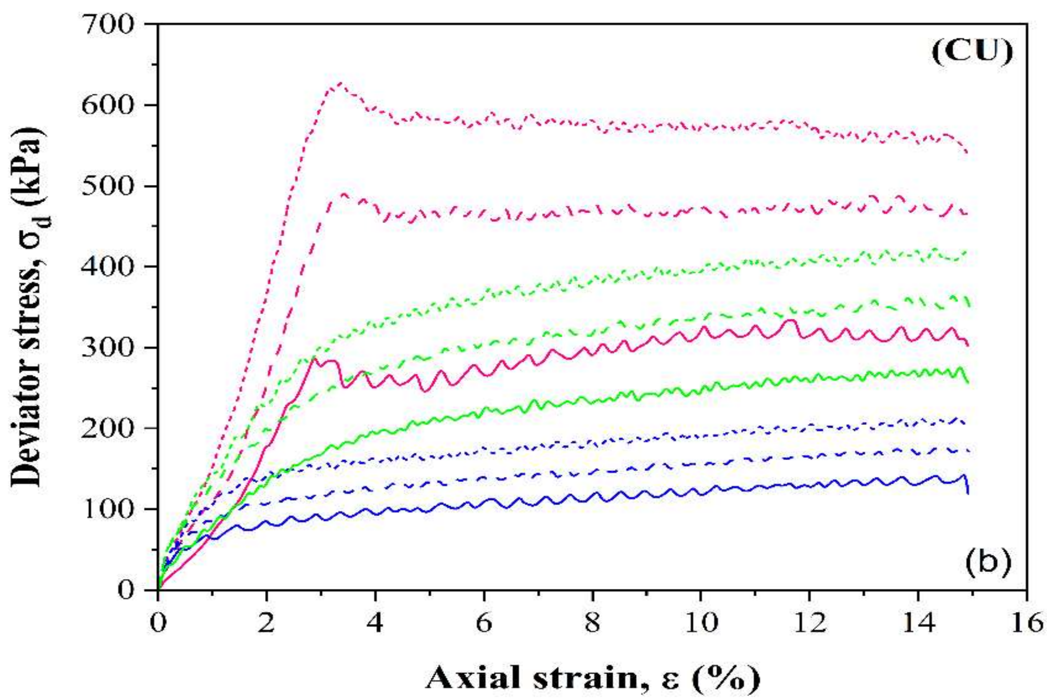
6.5.1 Evaluation of Cohesion and Angle of Internal Friction

The deviator stress versus axial strain plots of all the soil combinations subjected to different confining pressure were presented in Fig. 6.2. It can be inferred from the figure that for higher confinement higher deviator stress has been observed. This is due to the fact that higher confining pressure resembles with the field situation at higher depths which develops lateral movement restriction, hence required higher load for failure. Fly ash (FA) outperforms all other materials taken into consideration for each test condition, with the maximum stress values obtained in a 2-4% strain range. Under both CU and CD scenarios, the samples fail below the deviator stress value of 200 kPa, indicating that the performance of local soil (LS) diminishes with saturation regardless of the drainage state. This is due to the high influence of clayey particles (approx. 16%) in the LS. In general, fine-grained soils have higher surface areas than the coarse-grained soils (which are non-cohesive), which makes them more likely to absorb larger amounts of water when they are saturated. The pore water serves as a lubricant for the soil particles and lowers the soil's ability to withstand loads, which eventually affects the shear strength of the soil. The 2 layered system performs more or less similar as compared with the LS under UU condition. However, under the CU & CD condition the 2 layered soil performs even better than the LS.

From the deviator stress versus axial strain plots, the peak deviator stress (σ_d) has been determined under different loading conditions. These peak deviator stress helps in the estimation of major principal stress, which is required for the determination of shear strength parameters (c & ϕ). The bar chart of the cohesion and angle of internal friction under UU, CU, and CD conditions has been shown in Fig. 6.3. With the exception of the 2-layered sample, all samples had the highest cohesion (c) value achieved under the UU condition, although the cohesion, or c′ (effective cohesion), decreases with saturation and drainage conditions. Although while the difference between c and c′ in the case of the 2-layered sample is not very large, this could be because of the combined material behaviour of the fly ash and local soil. When samples are allowed to drain during shearing under CD condition, the cohesion values are lower. Nearly all of the samples taken into consideration had lower values for the ϕ' (effective friction angle) measured by CU tests. Due to the small effective contact area between the soil grains under saturated undrained circumstances, the friction angle is reduced. As a result, differences in the pore water pressure have no impact on the intergranular forces since they operate equally throughout the soil grains. In contrast to soil structure, water is incompressible, therefore any change in applied pressure is totally carried by the pore water. Unless drainage conditions allow for a volumetric shift, there are no changes in the stresses in the soil structure (Bishop and Eldin 1950; Terzaghi 1936). When drainage circumstances, such as CU or CD conditions are present, there are no significant changes in the shear strength characteristics of the FA and 2-layered (FA and LS) samples.

The particle size, shape, heterogeneity, degree of compaction, location of the origin, method of processing, saturation level, and testing conditions all affect the shear strength properties of the material. For better understanding of the strength parameter of the present soil, it has been compared with the past reported results in Table 6.2. It is clear from comparing the findings of the current study with the previously published findings that the layered system provides stronger strength parameters than that of the homogeneous system.





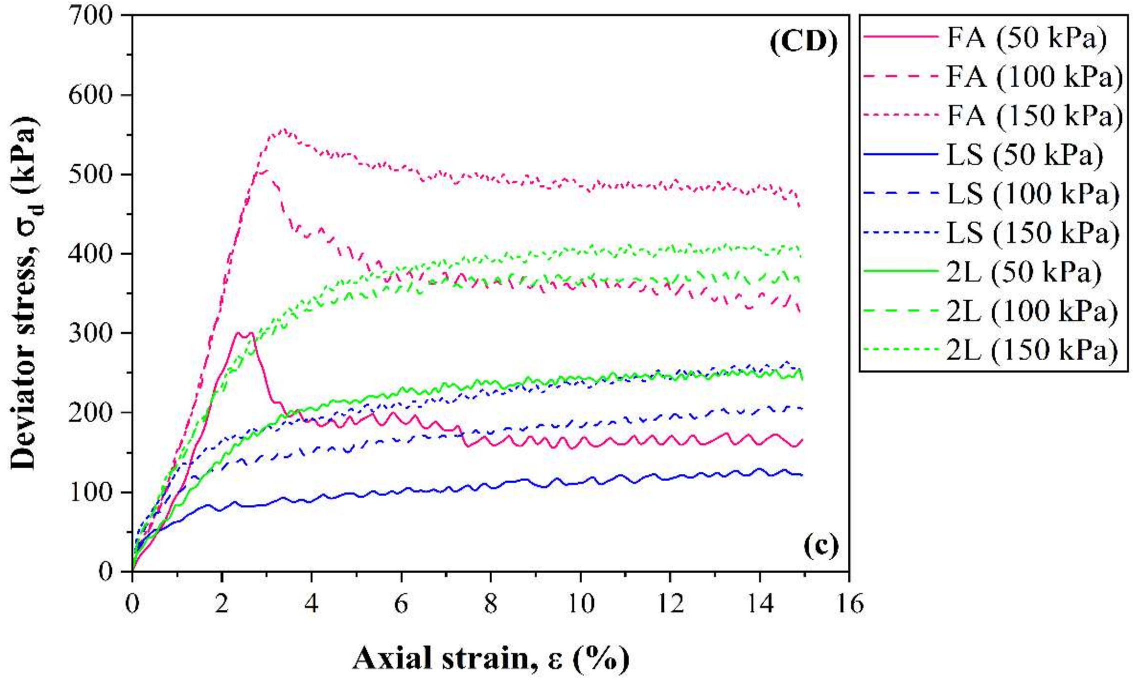


Fig. 6.2. Deviator stress versus axial strain plot of fly ash (FA), local soil (LS), and 2 Layered soil-ash deposit (2L) under (a) Unconsolidated undrained, (b) Consolidated undrained, and (c) Consolidated drained conditions subjected to different confining pressure.

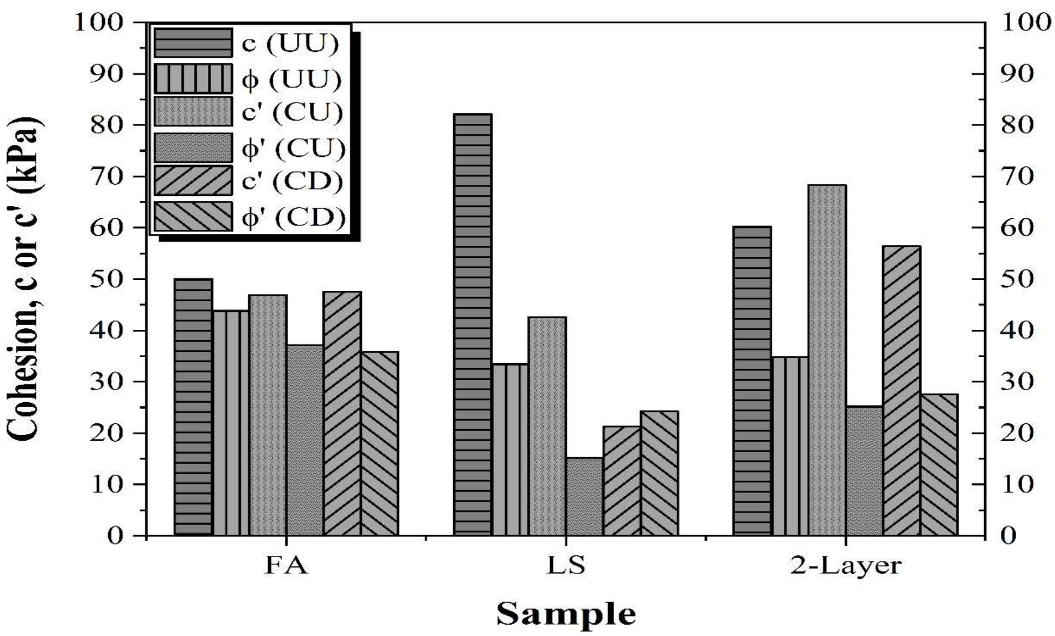


Fig. 6.3. Graphical representation of cohesion and angle of internal friction of fly ash (FA), local soil (LS), and 2 layered soil-ash deposit (2L) under UU, CU, and CD conditions.

Table 6.2 Comparisons of the shear strength parameters of the present study with the past reported studies.

Material	Cohesion (c) (kPa)	Angle of friction (φ) (°)	Reference
Fly ash	23; 0	34; 33	Sridharan et al. 1998 (Compacted and saturated condition)
Pond ash	16; 0	31; 30	
Bottom ash	17; 0	32; 31	
Fly ash (CU)	0	20-41	Pandian 2004
Pond ash (CU)	0-56	25-34	
Bottom ash (CU)	0-27	24-35	
Fly ash (CD)	16-96	26-39	
Pond ash (CD)	28-101	28-36	
Bottom ash (CD)	28-55	24-35	
Badarpur Pond ash (CU)	0	22.3-35.4	Jakka et al. 2010
Indraprastha Pond ash (CU)	0	24.2-38.6	
Yamuna sand (CU)	0	31.9-36.2	
Badarpur Pond ash (CD)	0	32.9-40.1	
Indraprastha Pond ash (CD)	0	33.1-41.7	
Yamuna sand (CD)	0	33.1-37.2	
MSWI (UU)	52.4	44.5	Zekkos et al. 2013
MSW (Delhi landfill)	20	35	Havangi et al. 2017
MSW (CU) (< 9.5 mm)	16.8	34	Gabr 1995
Recommended for MSW	25	35	Eid et al. 2000
Recommended for MSW (effective parameters)	10-18	15-25	Fassett et al. 1994; Kolsch 1995; Cowland et al. 1993
Kaolinite with pulp fiber (CU)	0	80.4	Andersland et al. 1981
Kaolinite with pulp fiber (CD)	0	31	
Sandy silty clay (UU)	60.9	28.8	Xu et al. 2018
Sandy silty clay (CU)	8.5	29.3	
FA	50; 46.9; 47.5	43.8; 37.2; 35.8	Present study (Range for UU, CU, and CD test)
LS	82.2; 42.6; 21.3	33.4; 15.1; 24.3	
2-layers	60.3; 68.3; 56.4	34.9; 25.2; 27.6	

6.5.2 Failure Mechanism Assessment

The failure pattern response of the soil specimen exhibits the behaviour of material subjected to external loads. The shear failure and bulging failure are the two common type of failure, in which shear failure signifies the brittle nature of the material whereas bulging failure signifies ductile failure of the material. The current homogeneous and layered soil-ash specimen fails in a number of ways, including single shear, double shear (X-shaped), shear with bulging, and bulging. These failures have been highlighted in the pictorial form depicted in Fig. 6.4. In addition, detailed description of the height of bulging from the top and maximum diameter at that position for all soil combinations under UU, CU, and CD conditions have been tabulated in Table 6.3. In fly ash, a double shear failure surface (X shape) with several shear planes subtended at angles ranging from 61.38° to 63.43° is depicted, in which the angle is roughly comparable to $45° + \frac{\phi}{2}$. The majority of the authors have described the deformation feature that results in the creation of a hard end zone. According to Haythornthwaite's (1960) theoretical analysis of this rigid zone, the angle between the cone's surface and base will be identical to $45° + \frac{\phi}{2}$ in compression and $45° - \frac{\phi}{2}$ in extension. Balasubramanian (1976) examined the failure behaviour of the saturated clay and, in contrast to Haythornthwaite's research, discovered no conical zone in the extension and a smaller conical zone under compression loading. Alshibli et al. (2003) in silica/Ottawa sand and Chen et al. (2017) in sandstone both recorded identical double shear failure. Similar to the previously mentioned local soil, shear failure is seen at low confinement and follows the $(45° + \frac{\phi}{2})$ failure plane. However, when confinement rises, this failure shifts to bulging failure. Typically, micro-cracks begin to form at the beginning of the loading process. When the pressure increases, the micro-cracks spread and cause numerous shear failure planes (Zhang et al. 2021). Similar to this, shear failure occurs when a sample is subjected to deviator stress. This is because some cracks originate on the weakest plane and then propagate diagonally. The layered structure shows hybrid (shear + bulging) failure (Fig. 6.4 (g, h, i)), which spreads from the middle of the FA layer to the bottom of the LS layer. Moreover, as the confining pressure rises, the bulging failure moves from the interface to the LS layer. With a stiff conical zone at the top that has moved to the bottom as the confining pressure has increased, the FA displays a hybrid (shear +

bulging) failure under the CD condition. Regardless of the kind of material, the studied specimens' failure profiles when in shear essentially follow $45° + \frac{\phi}{2}$. Wu et al. (2014) and Xu et al. (2020) both observed a similar shear failure trend for fiber-reinforced silty clay and cemented tailings, respectively. Furthermore, the failure surfaces according to Yang et al. (2020) were linear, smooth/rough, non-linear rough, and bench failure surfaces. For coal and sandstone samples, Liu et al. (2020) and Zhao et al. (2021) revealed failure types of single/double shear mode, conjugate shear mode, and T mode. Because the samples are saturated, all other samples aside from fly ash tend to bulge under CU and CD conditions (Table 6.3). When a sample is saturated, the void spaces are typically filled with water. This water tends to leak out during loading, densifying the sample and giving it a bulging appearance. The shear failures are more common in non-plastic soils and plastic soil at low confining pressure, according to the experimental findings of the current study. Moreover, all the samples other than homogeneous fly ash samples have switched from shear failure to bulging failure with an increase in confining pressure.

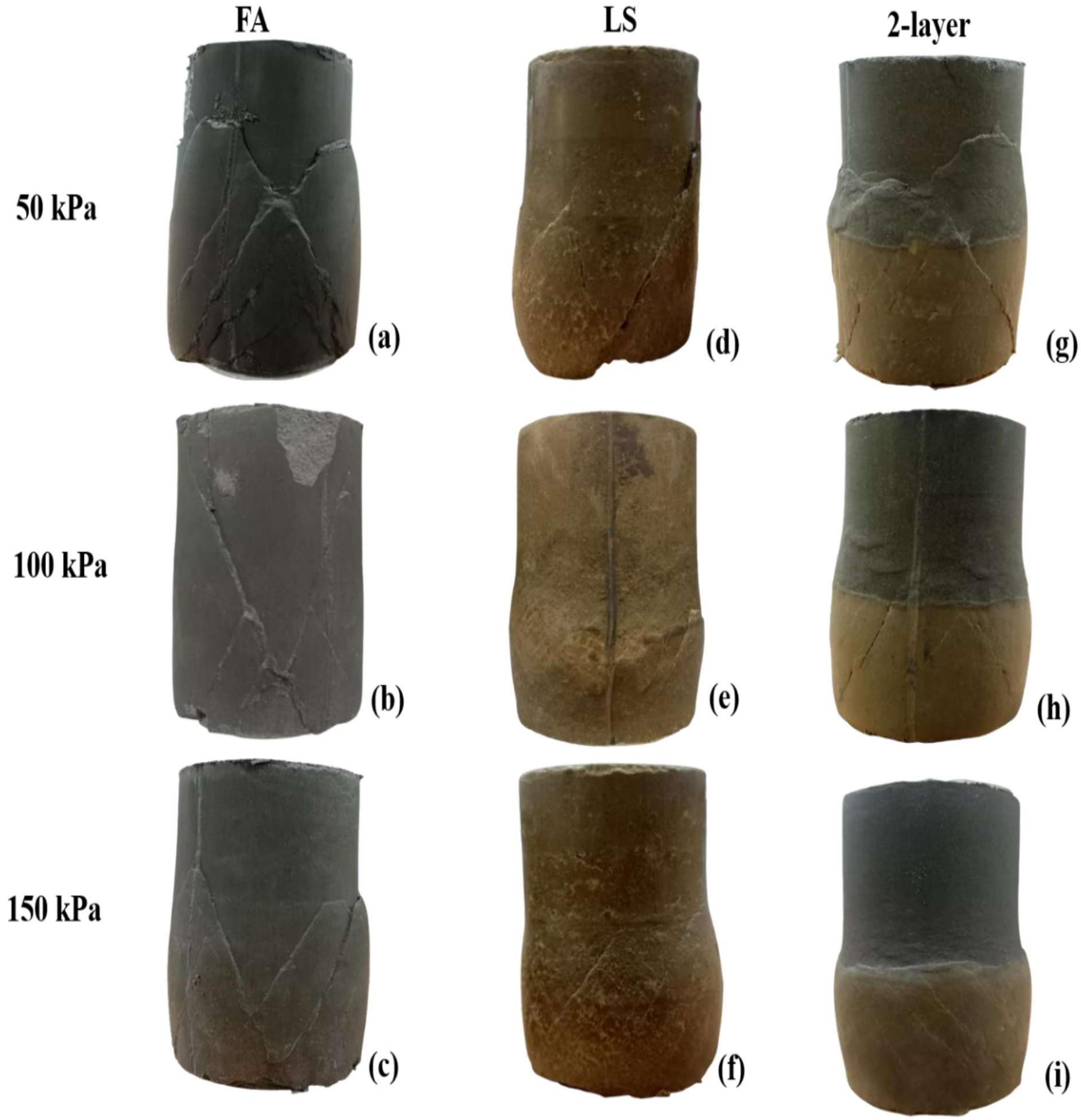


Fig. 6.4. Failure pattern analysis of unconsolidated undrained triaxial test of all the specimen combinations.

Table 6.3 Failure description of all the samples under different triaxial conditions (UU, CU, and CD).

Sample	σ_c (kPa)	UU Test				CU Test				CD Test			
		Bulging diameter (cm)	Bulging height (H_T) (cm)	Failure Type	Remarks	Bulging diameter (cm)	Bulging height (H_T) (cm)	Failure Type	Remarks	Bulging diameter (cm)	Bulging height (H_T) (cm)	Failure Type	Remarks
FA	50	5.9	5.7	Double shear	Middle to Bottom	6.1	2.9	Shear +Bulging	Top with wide cracks	5.9	2.4	Shear +Bulging	Top
	100	Nil	Nil	Double shear	Along the specimen length	6	5.5	Bulging	Bottom with wide cracks	6.1	2.9	Shear +Bulging	Top
	150	6	6.6	Double shear with bulging	Middle to Bottom	5.9	4.8	Bulging	Bottom with wide cracks	5.9	6.4	Shear +Bulging	Bottom
LS	50	5.9	6.2	Shear	Bottom	6.1	6.3	Bulging	Bottom	6.2	6.2	Bulging	Bottom
	100	5.8	6.3	Shear	Bottom	6.1	6.2	Bulging	Bottom	6.2	5.7	Bulging	Bottom
	150	5.9	6.7	Shear	Bottom	6.8	6.2	Bulging	Bottom	6.1	4.6	Bulging	Bottom
2-Layers	50	6.2	5.6	Shear +Bulging	Middle to Bottom	6	6.4	Bulging	Bottom	6.1	$3.4_T/1.8_B$	Double Bulging	Top/Bottom
	100	6.15	6.1	Shear +Bulging	Bottom	6.15	6.6	Bulging	Bottom	5.9	6.2	Bulging	Bottom
	150	6.3	5.9	Bulging	Bottom	6.1	6	Bulging	Bottom	6.2	6.6	Bulging	Bottom

6.5.3 Assessment of Initial Tangent Modulus and Secant Modulus

The elastic properties of the soils are estimated using the deviator versus axial strain plot of the homogeneous and stratified soil-ash deposit. The elastic properties including the initial tangent modulus (E_i) and secant modulus (E_s) have been investigated in the present study which is shown in Table 6.4. The initial tangent modulus was determined from the initial part of the stress-strain plot whereas the secant modulus was determined at 2% of the axial strain. The measured moduli exhibit a greater magnitude under the UU condition than that of the CD and CU triaxial conditions. The E_s has been considered for further study since it has a more stable modulus than E_i. By examining the secant modulus (E_s), it can be seen that the fly ash has the largest modulus range, followed by two layered systems and local soil. The stiffness of the material is directly impacted by the Young's modulus, meaning that a large stress will be needed to accommodate a given strain. Also, high confining stress raises the load carrying capacity, which raises the Young's modulus.

A comparison between the analytical formulas of the equivalent modulus (E_{eq}) for multi-layered soils and the experimentally measured modulus of the two-layered specimens was also made. The weighted arithmetic mean (M1), weighted harmonic mean (M2) (Brahma and Mukherjee 2010), and weighted average (M3) (Budhu 2010) are the formulations that have been employed for the equivalent modulus determination. While examining the settlement of a shallow foundation lying on a multilayered deposit, Brahma and Mukherjee (2010) used the equivalent modulus calculated from M1 and M2. It was discovered that the arithmetic mean technique (M1) overestimates the E_{eq}, which in turn lowers the settlement and may lead to an overestimate of foundation bearing capacity. Similar findings have been made with M1, i.e., overstated values when compared to the other approaches. According to the comparisons between variations of E_{eq} from the current experimental data and the E_{eq} computed using various expressions, the weighted harmonic mean approach (M2) is the expression that fits the layered soil-ash deposit the best.

Table 6.4 Variation of elastic modulus of all the samples represented by initial tangent and secant modulus.

Type of Test	σ_c (kPa)	FA		LS		2-Layers	
		E_i (kPa)	E_S (kPa)	E_i (kPa)	E_S (kPa)	E_i (kPa)	E_S (kPa)
UU	50	232.69	204.60	209.73	159.05	162.23	141.18
	100	246.20	238.11	218.76	168.24	183.28	152.93
	150	283.81	256.42	254.69	245.67	290.19	198.65
CU	50	83.74	86.62	121.20	39.88	98.47	70.56
	100	147.74	134.75	112.02	52.09	113.23	94.69
	150	202.09	183.09	140.97	71.96	133.23	112.97
CD	50	135.72	124.30	143.35	39.10	102.35	74.36
	100	178.17	168.33	179.46	64.69	166.07	109.92
	150	277.17	174.91	324.58	82.68	208.27	117.11

6.5.4 Interpretation of Ultimate Bearing Capacity of Strip Footing

The strength parameters evaluated in the present study have been incorporated to get an idea about the ultimate bearing capacity (q_u) of a strip footing resting on the homogeneous and stratified soil-ash deposit. The considered strip footing of depth 1 m and width of 1.5 m has been shown in the Fig. 6.5. As a result of its applicability to both local and global shear failure, Meyerhof's methodology has been used to predict the ultimate bearing capacity. The universal bearing capacity statement for shallow foundations proposed by Meyerhof (1963) is illustrated in Eq. (6.1).

$$q_u = cN_cS_cd_ci_c + qN_qS_qd_qi_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma \tag{6.1}$$

where, q_u is the ultimate bearing capacity, N_c, N_q, N_γ are the bearing capacity factors, s_c, s_q, s_γ are the shape factors, d_c, d_q, d_γ are the depth factors, i_c, i_q, i_γ are the inclination factor, B is the width of the footing, D is the depth of the footing and c is the cohesion. For the current strip footing problem, all the shape factors used in Eq. (6.1) can be taken as 1. Because of the vertical nature of the loading, the inclination factors can also be taken as 1.

The outcomes of the ultimate bearing capacity estimated for all the soil combinations have been shown in Table 6.5. In addition, the ultimate bearing capacity estimated from the weighted average values of cohesion and angle of internal friction for the stratified soil-ash deposit has also been added in the table. By comparing the findings of the homogenous materials under UU conditions to CU and CD conditions, it can be seen that for local soil, saturation causes the bearing capacity to drop by more than 85% under both the testing conditions. As fly ash's q_u is at its highest under UU conditions, it performs the best out of all the samples taken into consideration. Notwithstanding the fact that under both CU and CD test circumstances, the reduction in q_u is greater than 60%. When compared to UU conditions, the q_u value in the two-layered soil-ash deposit was reduced by about 50% in both CU and CD situations. Regardless of the drainage conditions, the considered LS exhibits poor performance in terms of q_u for the current investigation under saturation. As shown by the outcomes of the layered systems, stratification can enhance the bearing performance of the LS. The values obtained under UU conditions cannot be taken into consideration for the design as the geotechnical structures are often constructed for extreme loading conditions (minimum shear strength/bearing capacity). Therefore, it is more common and feasible to accept CU and CD circumstances.

For the layered systems, the weighted average technique significantly overestimates the q_u values under UU conditions compared to q_u predicted using experimentally computed c-ϕ, by 1.7 times. However, for the other two situations, the weighted average method's findings can be trusted because the discrepancy in the q_u values' percentage is less than 15%.

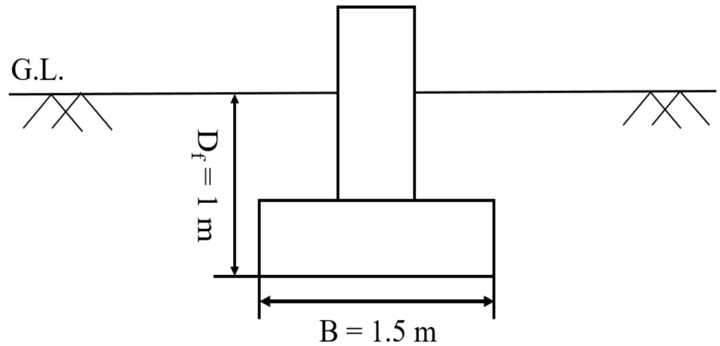


Fig. 6.5. Strip footing used for the assessment of ultimate bearing capacity.

Table 6.5 Ultimate bearing capacity of strip foundation resting on the considered soil and soil-ash deposit under all triaxial conditions.

Sample	c	φ	N_c	N_q	N_γ	d_c	d_q	d_γ	γ_d	q	q_u
	(kPa)	(°)							(kN/m^3)	(kN/m^2)	(kN/m^2)
UU											
Fly ash	50.04	43.79	114.74	110.92	200.63	1.31	1.15	1.15	12.36	12.36	11269.3
Local soil	82.16	33.41	39.88	27.29	27.96	1.24	1.12	1.12	16.97	16.97	5008.35
2-Layers	60.27	34.85	45.29	32.51	35.96	1.25	1.12	1.12	12.36	12.36	4254.74
2-Layers*	66.10	38.98	67.38	55.49	76.53	1.27	1.13	1.13	12.36	12.36	7287.73
CU											
Fly ash	46.87	37.16	56.21	43.58	54.50	1.26	1.13	1.13	12.36	12.36	4525.44
Local soil	42.62	15.13	11.02	3.98	1.15	1.17	1.08	1.08	16.97	16.97	641.269
2-Layers	68.33	25.18	20.91	10.82	6.94	1.20	1.10	1.10	12.36	12.36	1947.75
2-Layers*	44.75	27.21	24.23	13.45	9.75	1.21	1.10	1.10	12.36	12.36	1605.71
CD											
Fly ash	47.54	35.76	49.24	36.44	42.31	1.26	1.13	1.13	12.36	12.36	3902.07
Local soil	21.27	24.26	19.61	9.83	5.95	1.20	1.10	1.10	16.97	16.97	770.8
2-Layers	56.38	27.58	24.90	14.00	10.37	1.21	1.10	1.10	12.36	12.36	2011.83
2-Layers*	34.40	30.35	30.86	19.06	16.53	1.23	1.11	1.11	12.36	12.36	1742.78

Note: - q_u: Ultimate bearing capacity, (N_c, N_q, N_γ): Bearing capacity factors, (d_c, d_q, d_γ): Depth factor, γ_d: Maximum dry density, $q = \gamma * D_f$, *: In this case weighted average values of c & φ has been estimated using homogeneous soil.

6.6 SUMMARY

The biggest need in the world now is waste utilization. The majority of the research is devoted to finding ways to lessen and reuse this waste while also considering the project's social, environmental, and economic implications. The purpose of the current study was to determine whether fly ash, a waste product, could be used to combine locally available soil in a layered system. From the 27 triaxial tests conducted under various controlled loading conditions (UU, CU, and CD), the following conclusions can be drawn:

The outcomes of the study on the shear strength properties of the homogeneous and stratified soil-ash deposits reveal several important findings. Initially, the shear strength parameters obtained from the UU tests show overestimated values, indicating the good performance of the samples. However, under saturated conditions, i.e., CU and CD tests, significant changes are observed in the effective cohesion and effective friction angle. In the same way, regardless of the drainage circumstances, the fly ash and stratified soil-ash samples exhibit no significant changes in the strength parameters, while local soil samples show a decrease in the strength with saturation. Shear failures are observed in UU tests for fly ash and local soil samples, but the failure pattern changes to bulging with saturation and layering. A mathematical formulation of the equivalent modulus (E_{eq}) for multi-layered soil-ash deposits is developed, and it is compared with the experimentally measured modulus of two-layered specimens. The study concludes that the weighted harmonic mean approach is the most appropriate method for the investigation. Overall, this comprehensive investigation of the shear strength properties aids in selecting suitable soil replacements, while also improving the shear strength and bearing capacity in homogeneous and stratified soil-ash deposits.