CHAPTER 5 - MECHANICAL CONSOLIDATION TECHNIQUES

5.1. General

This chapter details a research study that explores the outcome of conducting laboratory consolidation tests using incremental loading (IL) and indigenously developed CRS methods using modified consolidation cells. The consolidation experiments on four distinct geomaterials, namely Marine soil, black cotton soil, red mud, and Varanasi local soil using IL and CRS approaches from the experiments were assessed. The tests were carried out using conventional and modified oedometer as per the ASTM standards ASTM D2435-11 and ASTM D4186-12, respectively. The study discusses the data collected based on the test results and then analyzes it to determine the consolidation parameters of the samples.

5.2. Experimental Programme

Firstly, IL consolidation testing under stress-controlled conditions using conventional and modified consolidation cells was performed for comparing and validating the results due to the size effect considered in the modified cell. Later the consolidation test results of the IL method using the modified consolidation cell were also compared with the indigenously developed CRS testing method with strain-controlled conditions. The stages of sample preparation, testing procedure and the results obtained from the tests are discussed in the subsequent

The conventional method used in the laboratory is mentioned in the (ASTM International, 2003). For CRS data analysis, the consolidation parameters were analysed as per ASTM Standard D4186-89 (1999). When the steady state factor, F_n> 0.4; the consolidation test results are considered reliable and valid for further analysis and interpretation.

In the present study, two techniques were employed to prepare the reconstituted test samples. The first technique, the slurry consolidating technique, was used to produce homogeneous samples of marine soil (CRSMS1, CRSMS2, CRSMS3, CRSMS4, ILMS-C and ILMS-M) and black cotton soil (CRSBCS1, CRSBCS2, CRSBCS3, CRSBCS4, ILBCS-C and ILBCS-M). The second technique, the conventional consolidation technique, was followed to prepare the samples of red mud (CRSRM1, CRSRM2, CRSRM3, CRSRM4, ILRM-C and ILRM-M) and Varanasi local soil (CRSLS1, CRSLS2, CRSLS3, CRSLS4, ILLS-C and ILLS-M). Moreover, significant parameters such as initial water content, initial void ratio, and final void ratio were comprehensively determined for each sample through a sampling programme. Table 5.1 presents a summary of the mechanical consolidation testing series using IL and CRS methods.

In the present experimental programme, an indigenously developed strain-controlled triaxial loading frame was used to apply the loading on the specimen for the required deformation rate and was unloaded after reaching the required total stress. In triaxial testing, the preference for expressing strain rate in millimeters per minute (mm/min) rather than in percentage per minute (%/min) can be attributed to the desire for a more tangible and direct representation of the physical movement of the loading apparatus. The use of mm/min provides a clearer correspondence between the rate of deformation and the actual displacement of the loading platen, enhancing the intuitive understanding of the experimental conditions. This choice of units facilitates a more straightforward translation of strain rate values into the real-world context of equipment motion, contributing to better clarity and interpretation of results. While both units are valid and serve the same fundamental purpose of specifying deformation rate, the adoption of mm/min offers a pragmatic approach to aligning experimental parameters with the observable mechanics of the triaxial testing apparatus (Mustefa Teha, 2016; Foriero, 2022).

The formula (Eq. 5.1) to convert strain rate from millimeters per minute (mm/min) to percentage per minute (%/min) with initial height of sample equal to 40mm is:

$$Strain\,rate\;(\%/min) = \frac{Strain\,rate\;(mm/min)}{Initial\;Height(mm)} X100 \qquad \text{(Eq. 5.1)}$$

Where:

- Strain Rate (%/*min*) is the strain rate in percentage per minute.
- Strain Rate (*mm*/*min*) is the strain rate in millimeters per minute.
- Initial Height (*mm*) is the initial height or length (40mm) of the specimen being tested in millimeters.

For conducting such type of strain-controlled consolidation tests on geomaterials having varying plasticity characteristics the criteria to fix the suitable strain rate was followed based on the pore pressure ratio (PPR) defined as the ratio of excess pore water pressure (u_b) at the base of the sample to total stress (σ_v) within the range of 0.03-0.15. The detailed discussion on PPR is discussed in the following sections

Table 5.1 Summary of mechanical consolidation tests

Type of Soil	Sample Type	Strain rate (mm/min)*	Strain rate (%/min)	Max. Loading Intensity upto (kg/cm^2)
(a) Indigenously developed CRS consolidation tests using modified oedometer cell				
Marine Soil (MS)	CRSMS1	0.002	0.005	--
	CRSMS2	0.003	0.0075	--
	CRSMS3	0.004	0.01	--
	CRSMS4	0.005	0.0125	--
Black Cotton Soil (BCS)	CRSBCS1	0.0025	0.00625	--
	CRSBCS2	0.003	0.0075	--
	CRSBCS3	0.005	0.0125	--
	CRSBCS4	0.0075	0.01875	--
Red Mud (RM)	CRSRM1	0.100	0.25	--
	CRSRM2	0.125	0.3125	--
	CRSRM3	0.25	0.625	--
	CRSRM4	0.50	1.25	--
Varanasi Local Soil (LS)	CRSLS1	0.05	0.125	--
	CRSLS2	0.075	0.1875	--
	CRSLS3	0.1	0.25	--
	CRSLS4	0.125	0.3125	--
(b) IL consolidation tests using conventional oedometer cell				
Marine Soil (MS)	ILMS-C	--	--	4.0
Black Cotton Soil (BCS)	ILBCS-C	--	--	4.0
Red Mud (RM)	ILRM-C	--	--	4.0
Varanasi Local Soil (LS)	ILLS-C	--	--	4.0
(c) IL consolidation tests using modified oedometer cell				
Marine Soil (MS)	ILMS-M	--	--	4.0
Black Cotton Soil (BCS)	ILBCS-M	--	--	4.0
Red Mud (RM)	ILRM-M	--	--	4.0
Varanasi Local Soil (LS)	ILLS-M	--	--	4.0

**Note- ***When the initial height of the specimen is 40mm

5.3. Pore pressure ratio

The CRS test is a better alternative to the conventional IL system and has the advantage of generating accurate compression curves within a shorter duration (Maleksaeedi et al., 2018;

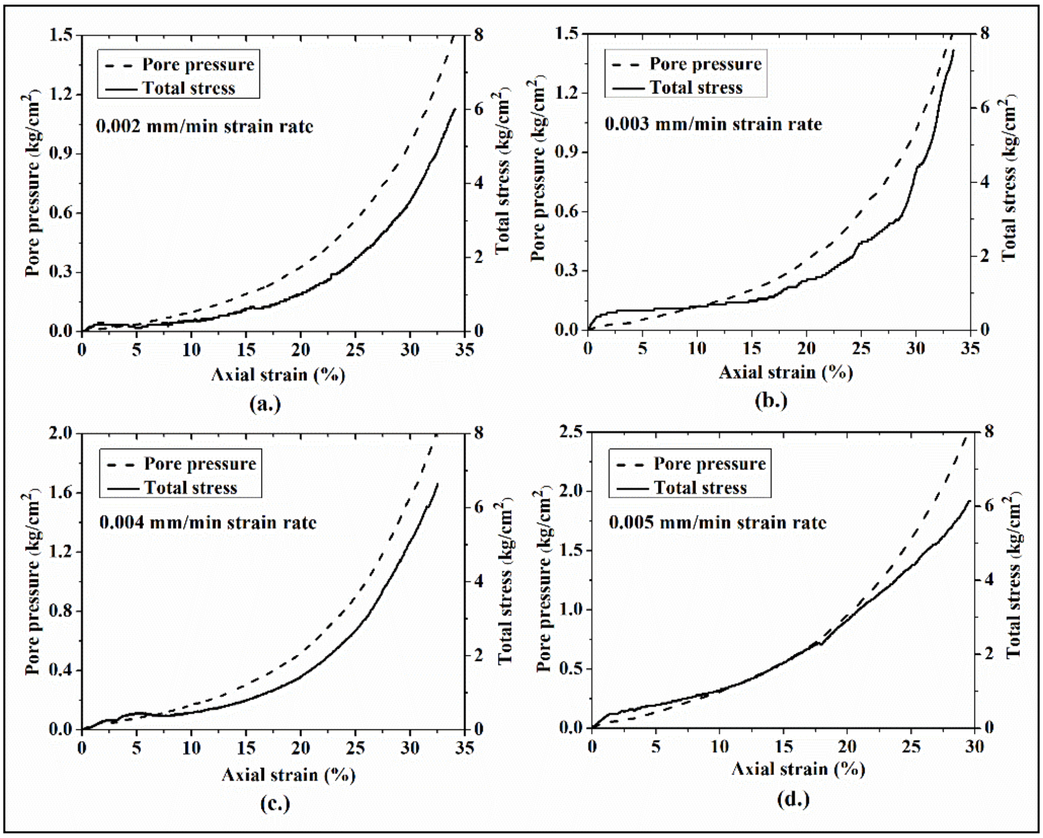
Galeano-parra et al., 2019; Moozhikkal et al., 2019; Raheena et al., 2019). The primary focus of the present chapter is on the validity of the indigenously developed CRS technique, specifically the pore-water pressure ratio (PPR). This ratio is defined as the excess pore water pressure divided by the applied total vertical stress. To determine acceptable PPR values, literature was reviewed and suggested PPR data were identified based on reported values of allowable PPR. Smith and Wahls (1969) reported an allowable PPR of 0.5 in their studies on Kaolinite, Ca-Montmorillonite, and Messara clay. Wissa (1971) considered 0.05 as PPR in CRS studies on Boston blue clay. Sallfors (1975) used a range of PPR values from 0.10 to 0.50 for Bakebol clay, and Gorman et al. (1978) considered a range of 0.30-0.50 during their studies on Kentucky region clays. Lee et al. (1993) reported a PPR value of 0.15 in their studies on Singapore Marine soils. Furthermore, ASTM D4186-89 (1998) and ASTM D4186-12 (2012) recommend PPR values in the range of 0.03-0.3 and 0.03 − 0.15 respectively. Sheahan and Watters (1997) and Gonzalez (2000) observed PPR values of 0.70 and 0.15, respectively. In this study, the consolidation process is investigated through the CRS approach in saturated geomaterials using the modified consolidometer cell as per ASTM D4186-12. The cell has a D/H ratio of 2.5. Strain-controlled loading tests were performed on reconstituted saturated samples using the conventional triaxial compression testing system without any back pressure arrangement. During the experiment, the data, such as axial load, axial deformation, and base pore water pressure, were continuously monitored and recorded and discussed in the following paragraphs. Finally, the obtained data were evaluated to determine the consolidation parameters of the soil samples.

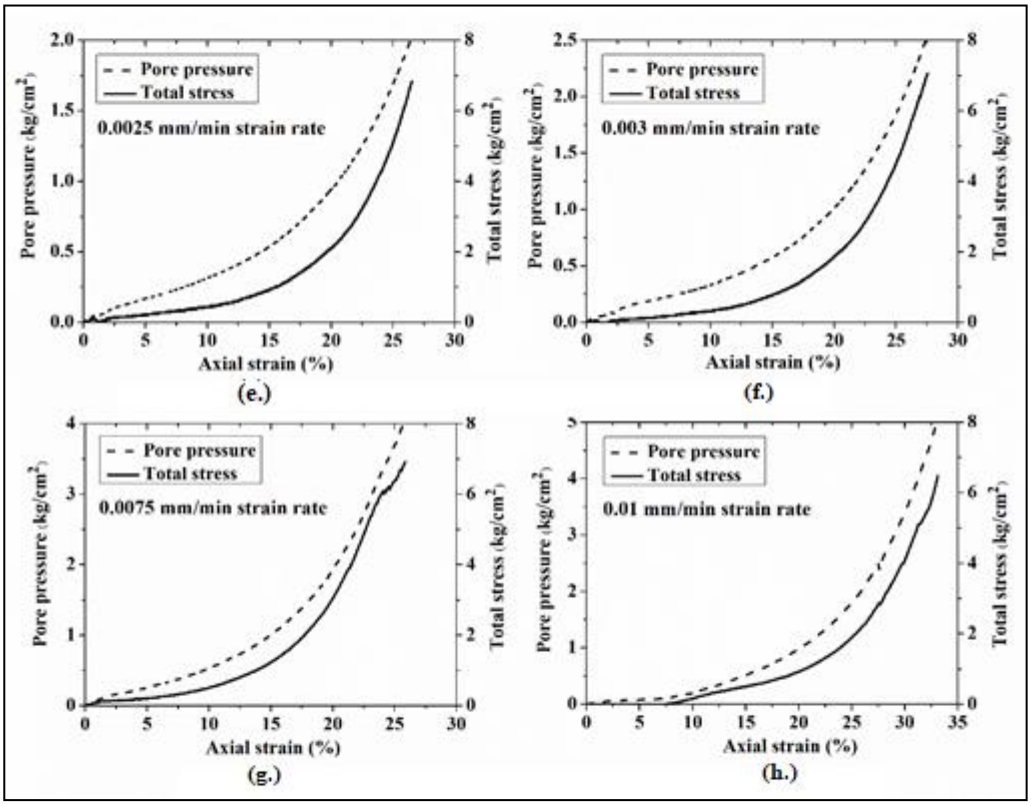
5.4. Interpretation of Consolidation Test Data

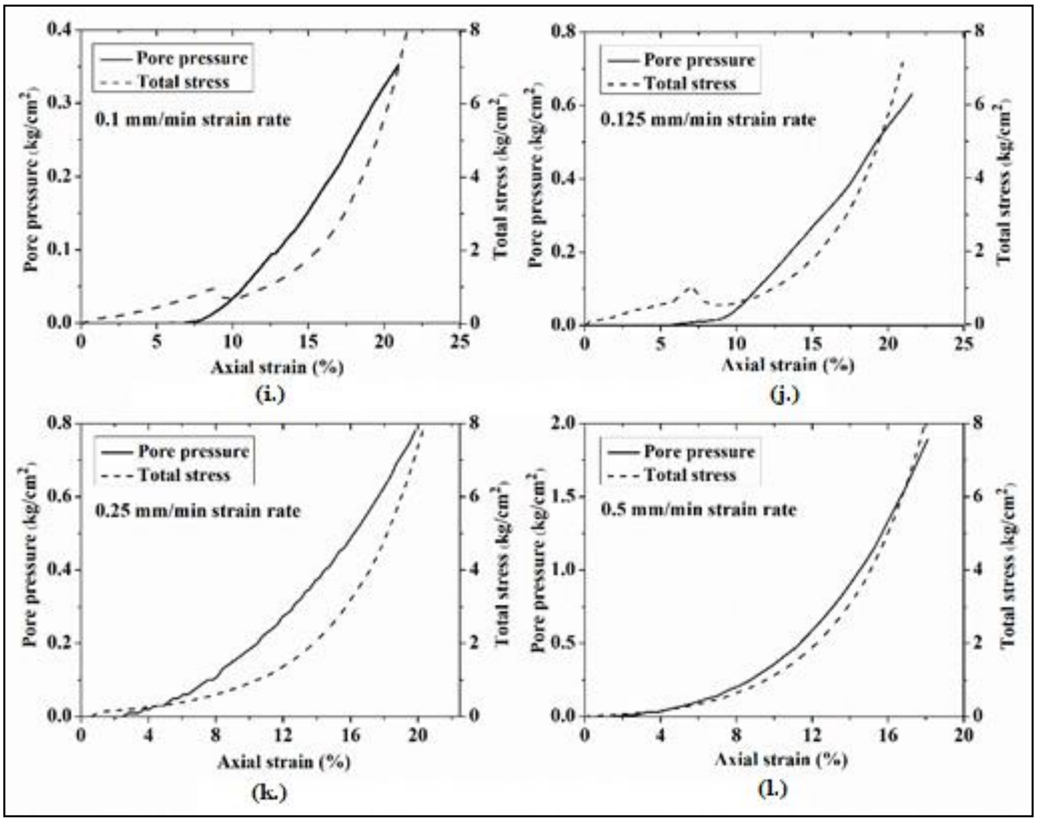
The IL tests using both conventional and modified oedometer consolidation rings incremental surcharge loading is applied up to 4.0 kg/cm^2. Whereas, in CRS testing, appropriate strain rate selection based on the PPR criterion is crucial for the accurate determination of consolidation parameters. The saturated soil sample undergoes compression at a constant strain rate, leading to changes in the sample's void ratio, porosity, and permeability during the test. As axial load is applied at a constant strain rate, the pore pressure and hydraulic gradient gradually increase and reach a maximum value before pore pressure dissipation starts. The study shows that the trends of pore pressure at the bottom have a significant dependence on the strain rate. High strain rates generate high pore pressure ratios (u_b/σ_v), leading to an unstable condition due to insufficient time for pore water dissipation. Conversely, slow strain rates cause insufficient pore water pressure due to enough time for pore water dissipation, which also affects the consolidation parameters (Ahmadi et al., 2014). The void ratio and effective stress data were used to calculate the primary compression index (c_c) and recompression index (c_r). Additionally, other parameters such as the coefficient of consolidation (c_v), coefficient of volume compressibility (m_v), and permeability (k) were determined following ASTM D4186-12 guidelines.

5.4.1. Pore pressure (u_b)/Total stress (σ_v) versus Axial Strain (ε)

The graphs in Figure 5.1. (a)-(p) depict the relationship between pore pressure, total stress and axial strain for reconstituted marine soil (CRSMS1, CRSMS2, CRSMS3, CRSMS4), black cotton soil (CRSBCS1, CRSBCS2, CRSBCS3, CRSBCS4), red mud (CRSRM1, CRSRM2, CRSRM3, CRSRM4) and Varanasi local soil (CRSLS1, CRSLS2, CRSLS3, CRSLS4) samples. Pre and post-test parameters, including moisture content, pore pressure, initial void ratio (e_o), final void ratio (e_f), and bulk density (γ_b), were analysed and are presented in Table 5.2. LVDT, pore pressure and load cell sensors provided a continuous set of data points for analysis. The graphs illustrate that as total stress increases, pore water pressure develops at the base of the sample. The CRS technique provides multiple data points, which facilitates the analysis and interpretation of consolidation parameters. The steady-state factor (F), a ratio of the changes in applied vertical effective stress at the top boundary of the specimen to the excess pore pressure measured at the base of the specimen, was evaluated based on the data analysis using ASTMD 4186-12. The value of F should be higher than 0.4, but it slightly decreased throughout the test, implying a decreased transient flow condition and increased steady condition during the test.




Pore pressure
Total stress
0.0025 mm/min strain rate
Pore pressure (kg/cm²)
Total stress (kg/cm²)
Axial strain (%)
(e.)
0.003 mm/min strain rate
(f.)
0.0075 mm/min strain rate
(g.)
0.01 mm/min strain rate
(h.)


Pore pressure
Total stress
0.1 mm/min strain rate
Pore pressure (kg/cm²)
Total stress (kg/cm²)
Axial strain (%)
(i.)
0.125 mm/min strain rate
(j.)
0.25 mm/min strain rate
(k.)
0.5 mm/min strain rate
(l.)

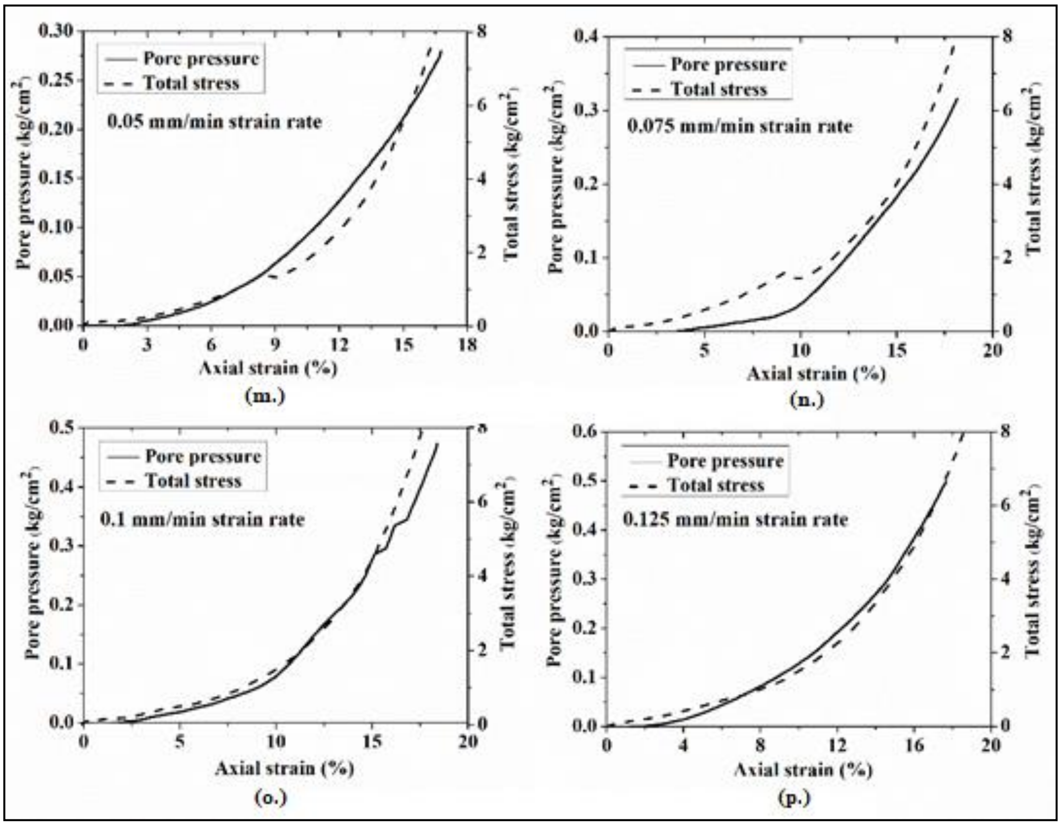


Figure 5.1 Pore pressure, Total stress vs Axial strain of reconstituted samples at different strain rates (mm/min): Marine soil at (a) 0.002, (b) 0.003, (c) 0.004, (d) 0.005; Black cotton soil at (e) 0.0025, (f) 0.003, (g) 0.005, (h) 0.0075; Red mud at (i) 0.10, (j) 0.125, (k) 0.25, (l) 0.50; Varanasi local soil at (m) 0.05, (n) 0.075, (o) 0.10, (p) 0.125 (Conversion 1kg/cm^2= 100kPa)

5.4.2. Pore Pressure Ratio (PPR) (u_b/σ_v) vs Effective Stress (σ_v')

Figure 5.2. (a-d) shows the variation of pore pressure ratio (u_b/σ_v) versus effective stress (σ_v') of all reconstituted samples of geomaterials. This graph facilitates the selection of strain rate for further determination of consolidation parameters and its comparison with IL tests with conventional and modified oedometer. The PPR is dependent on the pore pressure generated and applied total stress and affects the void ratio and effective stress parameter through the sample.

The PPR is determined by the pore pressure generated and applied total stress, which affects the void ratio and effective stress parameters of the sample. At low strain rates, insufficient pore water pressure is generated due to the presence of cavities in the soil matrix, which prevents pore water pressure generation. At high strain rates, high pore water pressure is generated at the bottom, possibly due to cavity blockage, which results in an unsatisfactory PPR criterion (Maleksaeedi et al., 2019). The graph in Figure 5.2 shows that as the applied stress increases, the pore water pressure also increases due to the expulsion of pore water from the voids of the sample. This study shows that for low plastic geomaterials such as red mud and Varanasi local soil, the PPR value varies less with increasing strain rates, and the PPR curve versus effective stress follows a trend of reaching a gradual peak and then dropping. For high plastic soils such as Marine soil and black cotton soils, the PPR is within the range of 0.03 to 0.15 and exhibits minimal time for the buildup and dissipation of pore water pressure. Marine soils have a faster and steeper slope with a sharp drop in the PPR curve as the effective stress increases.

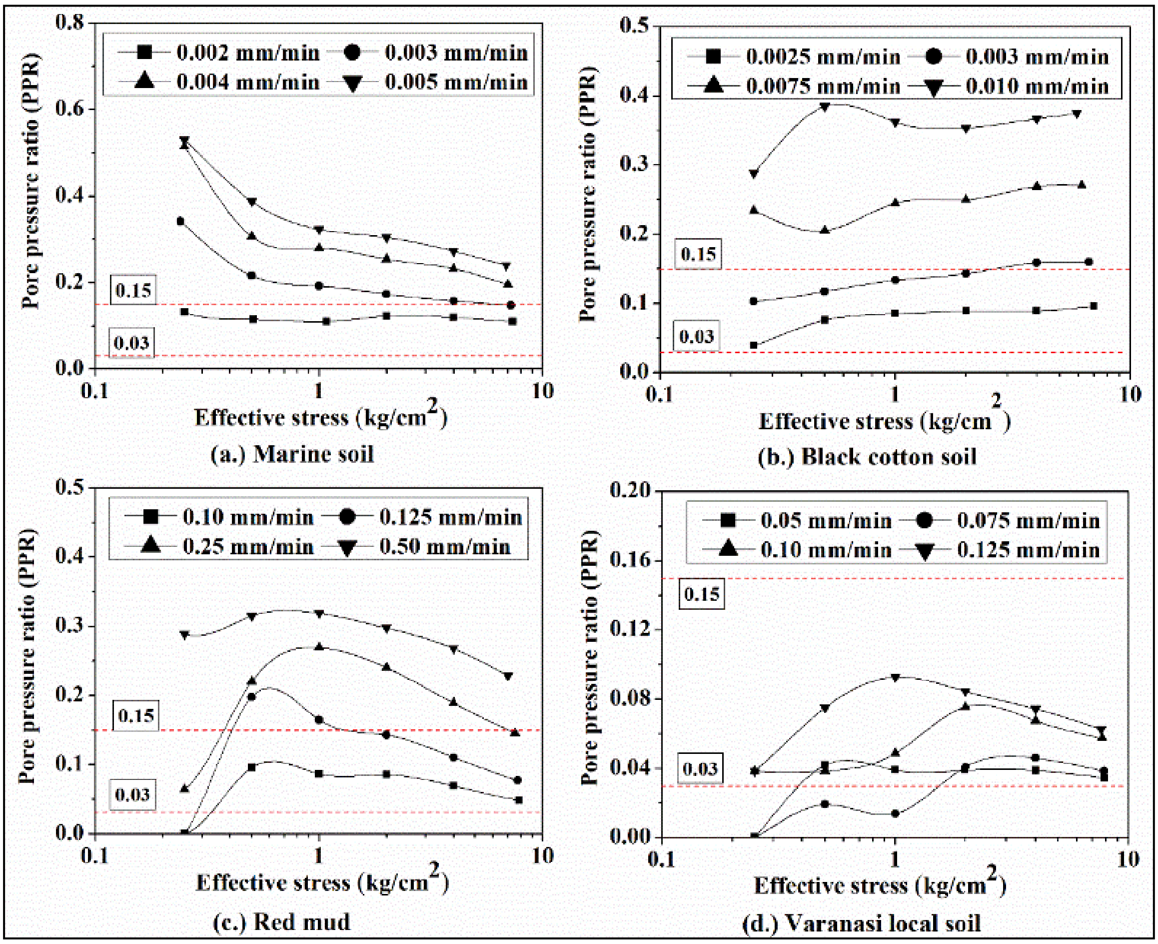


Figure 5.2 Pore pressure ratio versus Effective stress curves of reconstituted soil samples at different strain rates (Conversion 1kg/cm^2= 100kPa)

5.4.3. Void ratio (e) versus Effective stress (σ_v')

Figure 5.3. (a-d) displays the comparative compression behaviour of four soils, namely reconstituted marine soil (CRSMS1), black cotton soil (CRSBCS1), red mud (CRSRM1), and Varanasi local soil (CRSLS1), for selected strain rates of 0.002, 0.0025, 0.1, and 0.05 mm/min respectively in the CRS technique and IL tests with conventional and modified cells. These graphs depict the variation in the void ratio (e) with effective stress (σ_v') for different loading conditions. At the start of the experiment, all soils had a high void ratio due to their high-water content and softness. In the primary consolidation stage, the curve for marine soil and black cotton soil showed minimal or negligible creep for all loading conditions, which later decreased exponentially with an increase in effective stress. For red mud and Varanasi local soil, the creep was also negligible initially and then decreased exponentially with a steeper slope. The change in the void ratio (Δe) observed for all soils followed the trend: Red mud < Varanasi local soil < Black cotton soil < Marine soil. A similar trend was observed in both conventional and modified IL tests for all four samples. Table 5.2 presents the initial and final void ratio values obtained from the collected data.

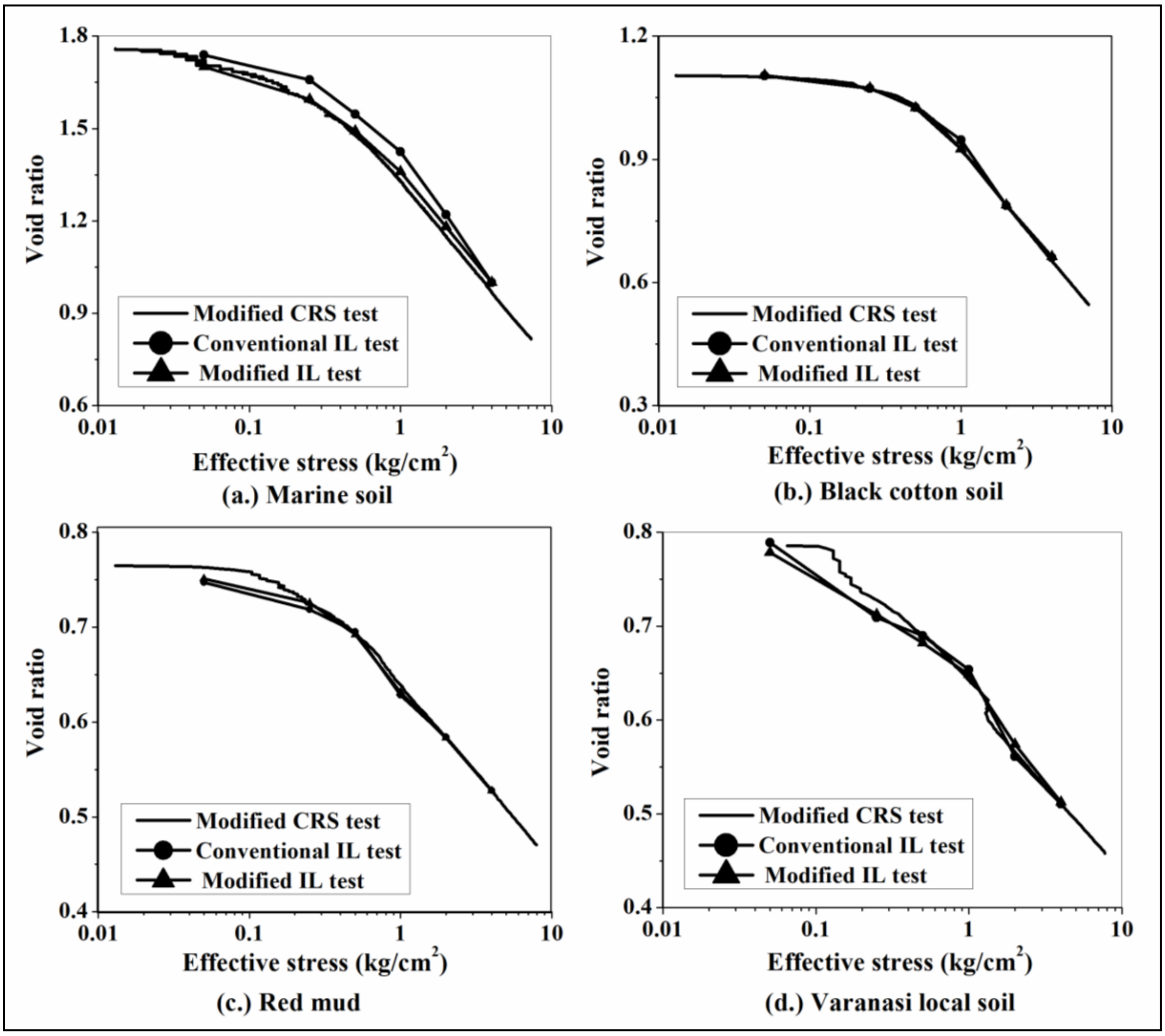


Figure 5.3 Void ratio (e) versus Effective Stress (log σ'_v) of geomaterials (Conversion 1kg/cm^2= 100kPa)

5.4.4. Coefficient of consolidation (c_v) vs Effective stress (σ'_v)

Figure 5.4. (a-d) illustrates the relationship between the coefficient of consolidation (c_v) and effective stress (σv') for all four reconstituted soil samples. The samples were selected based on the pore pressure ratio (PPR) criterion mentioned in ASTM D4186-12 and were later compared with the results obtained from conventional and modified IL tests using the Taylor square root time fitting method. The data collected from the tests were used to perform calculations, and the results obtained were found to agree with the general trends observed in previous studies (Reddy et al., 2018).

The c_v curves exhibit a convergence at the end of the curve when the pore water pressure has reduced, indicating a steady-state condition. The values obtained for cv at different loading conditions were found to be more or less close to each other, indicating satisfactory results. Based on the comparison of the c_v values, the rate of compressibility for the samples can be ranked as red mud > Varanasi local soil > black cotton soil > marine soil. The values of c_v obtained from Figure 5.4 have been reported in Table 5.3. The results provide insights into the compression behaviour of the soils under different loading conditions and can aid in designing structures that require a thorough understanding of the consolidation behaviour of soils.

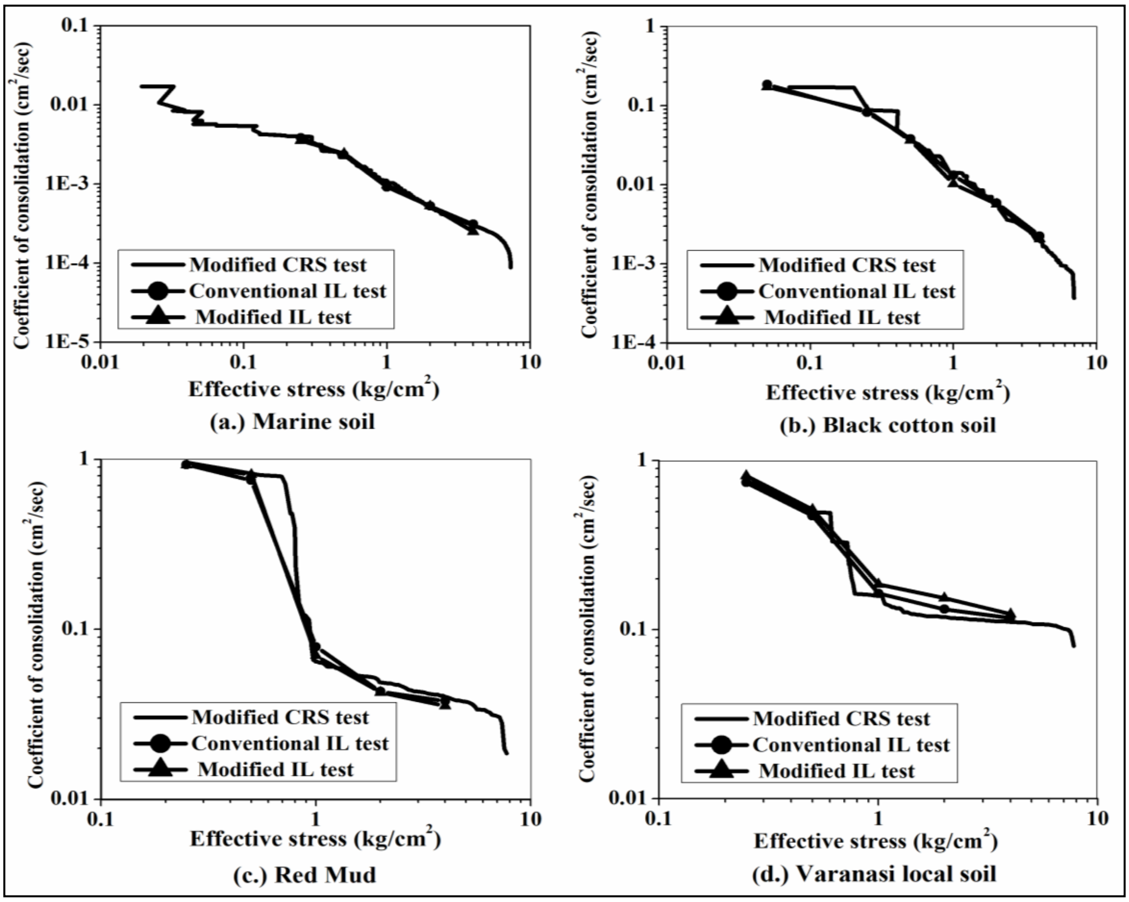


Figure 5.4 Coefficient of Consolidation versus Effective Stress (log σ'_v) for all geomaterials (Conversion 1kg/cm^2= 100kPa)

5.4.5. Permeability (k) versus effective stress (σ_v')

The permeability of soil is an important parameter that affects the consolidation behavior of soil. Figure 5.5 illustrates the variation of permeability (k) to effective stress (σ_v') for the four reconstituted samples at the selected strain rate according to the PPR criterion. The permeability values were calculated based on the ASTM codes and plotted for all loading conditions. The graph shows that as the duration of loading increases, the permeability values decrease for all four soils. The trend observed in the graph is similar to the graphs obtained in Figure 5.5, indicating a good agreement between the permeability values obtained from both the IL tests. The curves obtained based on the values of 'k' show that the permeability trend for all soils is Red mud > Varanasi local soil > Black cotton soil > Marine soil. This implies that Red mud has the highest permeability while marine soil has the lowest permeability. The variation of permeability to effective stress is an important factor that needs to be considered while analyzing the consolidation behaviour of soil.

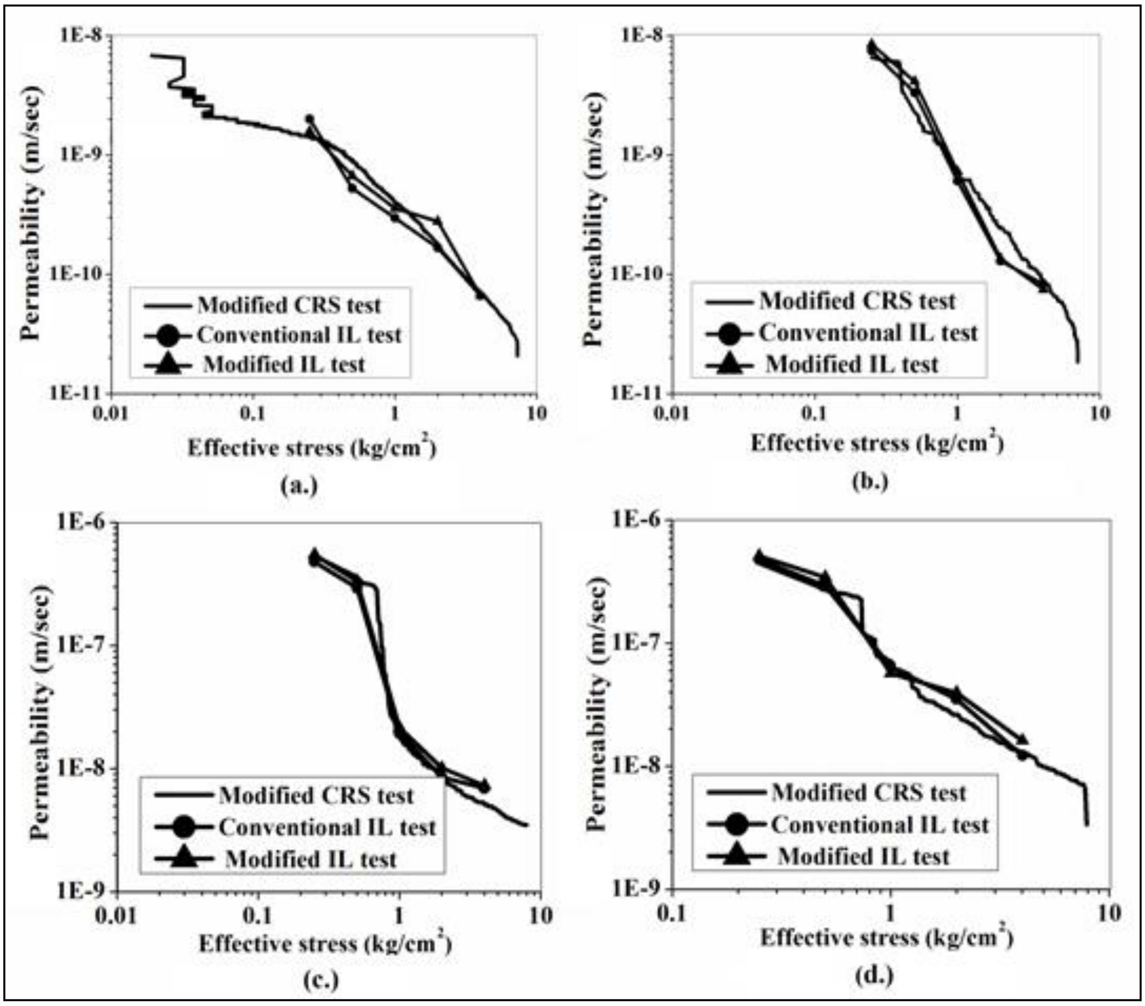


Figure 5.5 Permeability (k) versus Effective Stress (log σ'_v) for geomaterials: (a) Marine Soil, (b) Black Cotton Soil, (c) Red Mud, (d) Varanasi Local Soil (Conversion 1kg/cm^2= 100kPa)

5.4.6. Pre and Post- experimental test data

The parameters based on the test data discussed in the previous paragraphs have been listed in following Table 5.2.

Table 5.2 Summary of Post- experimental test data

Sample Type	Final deformation, (ΔH) (mm)	Initial moisture content (%)	Initial void ratio (e_o)	Final void ratio (e_f)	Maximum pore pressure, (u_b)(kg/cm^2)
CRS consolidation with modified oedometer					
CRSMS1	13.594	61.51	1.738	0.817	1.23
CRSMS2	13.313	61.54	1.737	0.836	1.38
CRSMS3	12.947	60.72	1.733	0.890	1.63
CRSMS4	11.838	61.35	1.758	0.94	1.91
CRSBCS1	11.618	40.52	1.116	0.588	1.71
CRSBCS2	11.018	40.05	1.118	0.531	2.21
CRSBCS3	10.994	40.41	1.115	0.538	3.46
CRSBCS4	10.335	40.13	1.104	0.573	4.05
CRSRM1	8.623	29.72	0.788	0.473	0.63
CRSRM2	8.575	29.65	0.787	0.476	0.71
CRSRM3	7.927	30.28	0.776	0.468	0.78
CRSRM4	7.233	29.41	0.776	0.463	0.89
CRSLS1	7.358	14.28	0.767	0.471	0.28
CRSLS2	7.259	14.16	0.773	0.451	0.35
CRSLS3	7.063	14.76	0.786	0.458	0.46
CRSLS4	6.707	14.54	0.778	0.464	0.51
IL consolidation with conventional oedometer					
ILMS-C	6.012	59.95	1.738	0.99	No Provision
ILBCS-C	5.264	40.16	1.103	0.660	
ILRM-C	3.625	29.15	0.747	0.528	
ILLS-C	3.524	14.56	0.789	0.51	
IL consolidation with conventional oedometer					
ILMS-M	12.324	60.18	1.7	0.992	No Provision
ILBCS-M	10.614	40.43	1.104	0.662	
ILRM-M	7.625	30.08	0.751	0.527	
ILLS-M	7.524	14.28	0.778	0.512	

5.4.7. Compression index (c_c)/ Recompression index (c_r)/ Coefficient of consolidation (c_v) versus Consistency limits

Figures 5.6-5.8 display the correlation between the compression index, coefficient of consolidation, liquid limit, and plasticity index. The compression index values were compared between experimental (IL and CRS) and empirical relations from previous studies (Skempton and Jones, 1944; Terzaghi and Peck, 1967; Sridharan and Nagaraj, 2000). The results indicate that the consolidation parameters obtained from CRS tests are in good agreement with IL test results obtained using conventional and modified cells. Regression analysis was performed to develop correlations between consolidation parameters (c_c, c_r, and c_v) listed in Table 5.3, and the liquid limit (w_l) and plasticity index (I_p) of geomaterials. Figures 5.8 and 5.9 present the results of the regression analysis, and newly developed equations of best fit between consolidation parameters and index properties are presented in equation form in the figures. The conventional regression analysis was performed using Origin Pro 8.5 software. The results of the analysis are promising, with R^2 values greater than 0.9 in all cases.

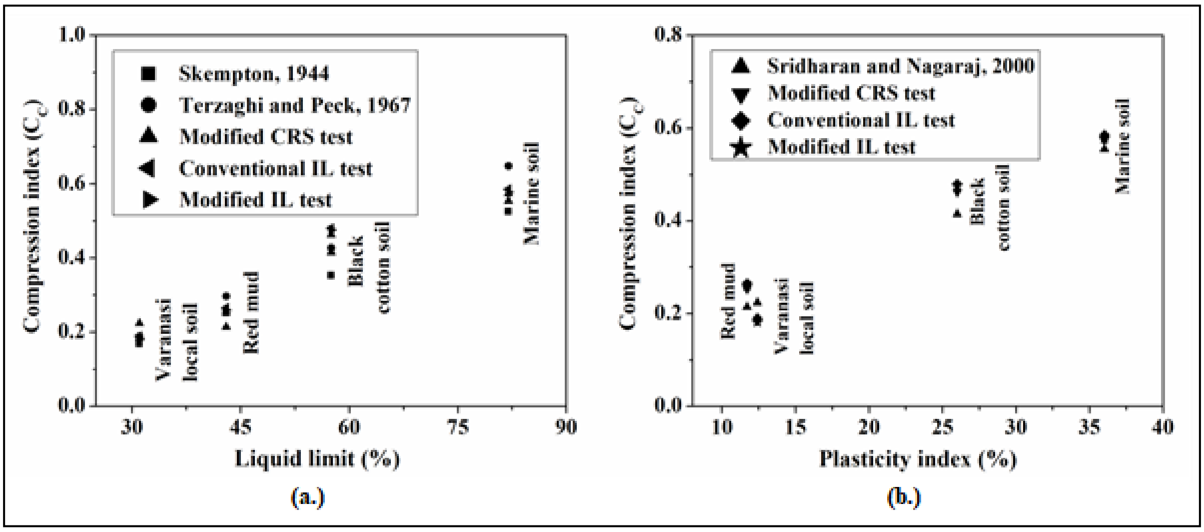


Figure 5.6 Relationship between: (a) Compression Index vs Liquid Limit, (b) Compression Index vs Plasticity Index

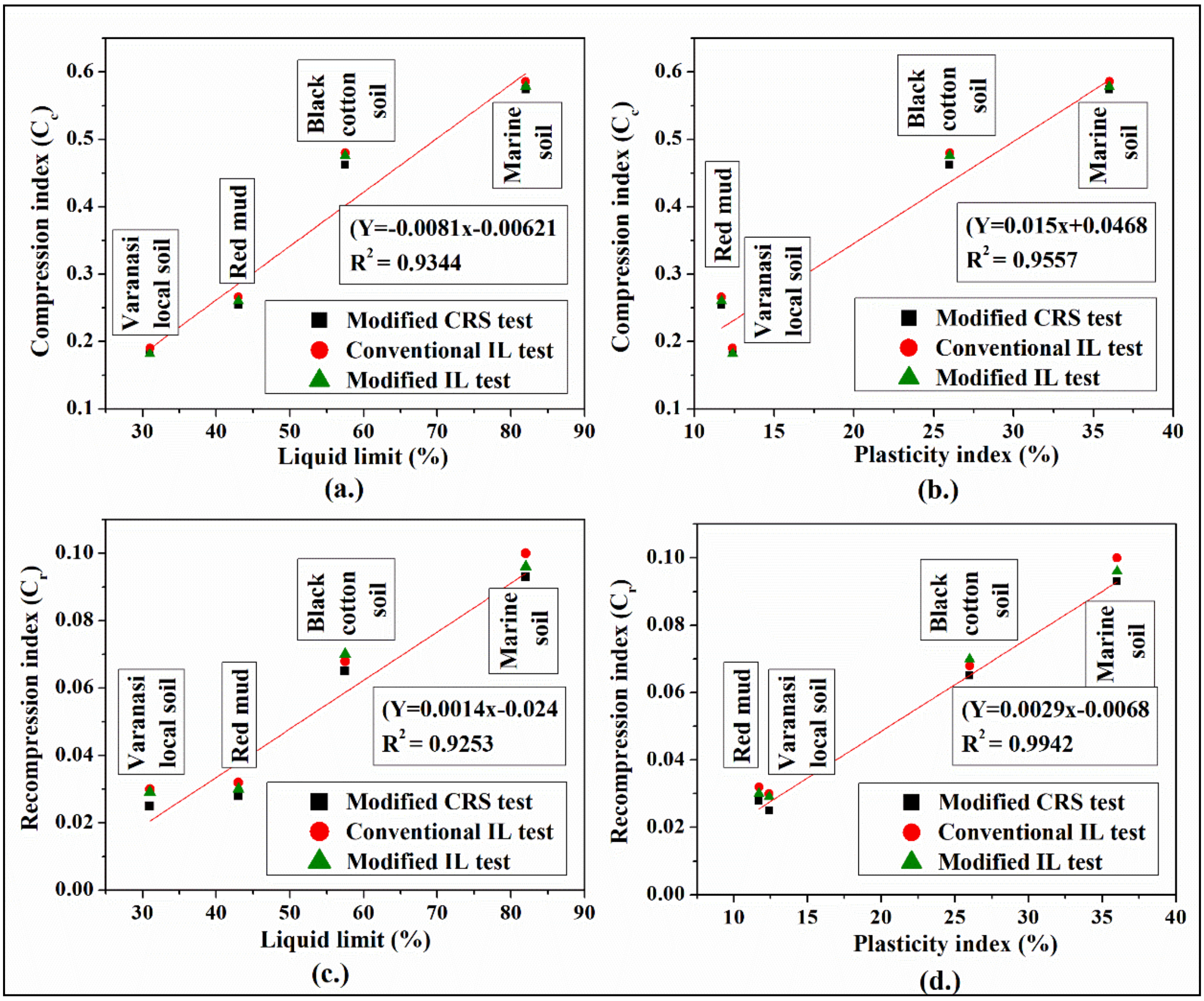


Figure 5.7 Best-fit curves of geomaterials for (a) Compression index vs liquid limit and (b) Compression index vs plasticity index (c) Recompression index vs liquid limit and (d) Recompression index vs plasticity index

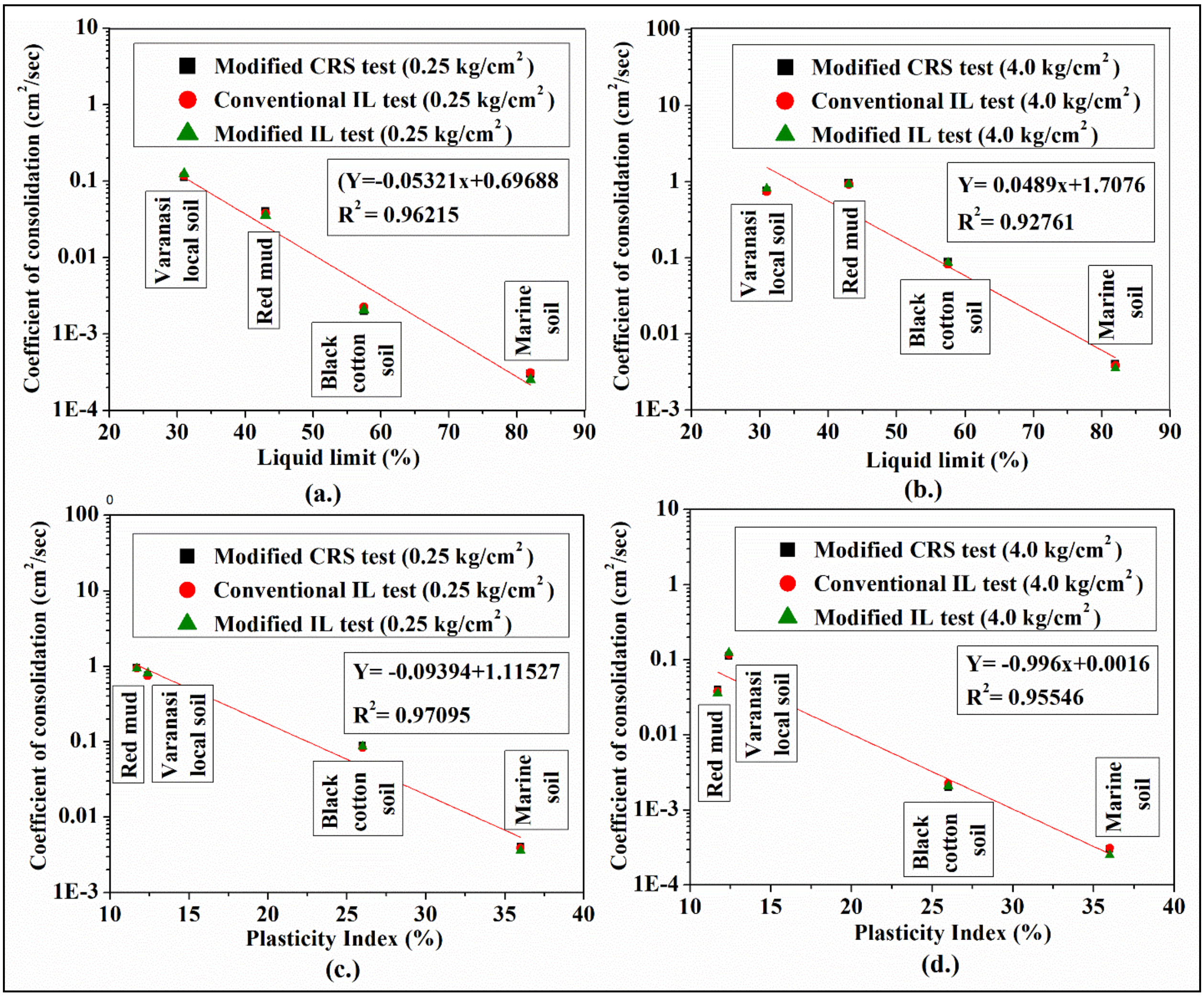


Figure 5.8 Best-fit curves of geomaterials for: (a) Coefficient of Consolidation versus Liquid Limit at 0.25 kg/cm^2 loading intensity, (b) Coefficient of Consolidation versus Liquid Limit at 4.0 kg/cm^2 loading intensity, (c) Coefficient of Consolidation versus Plasticity Index at 0.25 kg/cm^2 loading intensity, (d) Coefficient of Consolidation versus Plasticity Index at 4.0 kg/cm^2 loading intensity (Conversion 1cm^2/sec= 1x10^{-4} m^2/sec)

Table 5.3 Consolidation parameters obtained from consolidation tests

Parameter	Marine soil	Black cotton soil	Red mud	Varanasi local soil
CRS consolidation tests with modified consolidation ring				
Suitable strain rate (mm/min)	0.002	0.0025	0.1	0.05
Suitable strain rate (%/min)	0.005	0.00625	0.25	0.125
Coefficient of axial compressibility (a_v)	0.093	0.071	0.03	0.029
Compression index (c_c)	0.574	0.462	0.254	0.185
Recompression index (c_r)	0.093	0.065	0.028	0.025
Coefficient of consolidation (c_v), in cm^2/s	1.726×10^{-2} – 8.85×10^{-5}	1.727×10^{-1}– 3.697×10^{-4}	8.089×10^{-1} – 1.862×10^{-2}	4.983×10^{-1} – 8.014×10^{-2}
Permeability (k), in m/s	6.82×10^{-9}– 3.49×10^{-12}	2.542×10^{-8}– 1.403×10^{-11}	4.905×10^{-7}– 1.258×10^{-9}	2.753×10^{-7} – 3.395×10^{-9}
Loading time duration (mins)	7120	4310	90	97
IL consolidation tests with conventional consolidation ring				
Coefficient of axial compressibility (a_v)	0.1	0.08	0.034	0.035
Compression index (c_c)	0.586	0.48	0.266	0.19
Recompression index (c_r)	0.1	0.068	0.032	0.03
Coefficient of consolidation (cv), in cm^2/s	3.87×10^{-3} – 3.107×10^{-4}	8.243×10^{-2} 2.26×10^{-3}	0.982 –0.038	0.741–0.118
Permeability (k), in m/s	2.005×10^{-9}– 6.644×10^{-11}	7.478×10^{-9}– 8.273×10^{-11}	4.802×10^{-7} – 6.963×10^{-9}	3.592×10^{-7} – 1.232×10^{-8}
IL consolidation tests with modified consolidation ring				
Coefficient of axial compressibility (a_v)	0.098	0.076	0.033	0.031
Compression index (c_c)	0.578	0.476	0.26	0.182
Recompression index (c_r)	0.096	0.07	0.03	0.029
Coefficient of consolidation (c_v), in cm^2/s	3.56×10^{-3} – 2.508×10^{-4}	9.656×10^{-2} 4.061×10^{-3}	0.916 –0.029	0.812 – 0.134
Permeability (k), in m/s	1.581×10^{-9}– 6.643×10^{-11}	8.478×10^{-9}– 7.473×10^{-11}	5.481×10^{-7} – 7.323×10^{-9}	5.136×10^{-7} – $1.625 \quad 10^{-9}$

5.5. Summary

The study evaluated the effectiveness of the constant rate of strain (CRS) loading method in consolidating high to low plastic geomaterials, such as Marine soil, black cotton soil, red mud, and Varanasi local soil. The CRS test results were validated and compared to those of the incremental loading (IL) technique, using both conventional and modified consolidation rings. The study found that the CRS technique is faster and more accurate than the conventional IL technique. However, the test duration was slightly longer for high-plastic soils. The CRS technique generates a variable pore water pressure, dependent on the soil type and strain rate. The proposed CRS technique showed good agreement with IL tests in determining parameters such as c_c, c_r, c_v, a_v and k. Simple regression analyses were conducted, and prediction formulae were proposed, which showed promising results in predicting consolidation parameters.