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Three-Dimensional Consolidation Settlement of Normally Consolidated Silt-Clay Matrices

Dr. Krishna Nandan Prasad
Associate Professor (Civil)
Katihar Engineering College, KATIHAR,
Bihar -854109

SYNOPSIS - The experimental results of artificially NC silt-clay matrices are used to determine immediate as well as consolidation settlements by conventional and Stress-Path method which is presented in paper. The necessity for a three-dimensional approach is discussed and a method of predicting settlement is proposed based on the use of elastic theory. The results of settlements evaluated by the Stress-Path method are compared with the proposed method and found in agreement with the sam. In SCM under study, attempt has been made by author to correlate One-dimensional consolidation settlement with Three-dimensional consolidation settlement by introducing a correction coefficient (β)

I. INTRODUCTION

Terzaghi – Taylor and Skempton - Peck – Mc Donald proposed methods to evaluate consolidation settlement based on the assumption that soil is laterally confined during consolidation. In this condition, the deformation of soil is one-dimensional only. In actual particle, there is a lateral deformation of in-situ soil take place when a structure is founded on a thick bed of soil. The lateral strain on settlement depends on the Young's modulus, Poisson's ratio and anisotropics which also vary with the stress level and the manner of application of incremental stresses .Conventional methods of settlement analysis is still used in most of the design practice because most of the reliable methods like "stress-path method" which generally yields better results is sophisticated and time consuming experimental techniques.

The experimental results of artificial normally consolidated silt-clay matrices are used to evaluate immediate as well as consolidation settlement which is presented herein. The necessity for a three-dimensional approach is discussed and a method of predicting settlement is proposed which is based on the use of elastic displacement theory in combination with the soil parameters experimentally determined over a representative range of stresses. The results of the settlements computed by the Stress-Path method are compared with the proposed method and found in agreement with the same.

II. FABRICATION OF INSTRUMENT FOR PREPARING REMOULDED SAMPLES FOR SILT-CLAY MATRICES

2.1. Modified Oedometer Instrument

Soil samples have been prepared in the laboratory similar to the field. In field soil slurry is deposited every year on each other by transportation with water and they get consolidated by its overburden pressure. To simulate the same condition in the laboratory, it is required to consolidate the soil slurry of silt-clay matrix by dead load placed by any means. The soil slurry is highly compressible and instantaneous load causing excess pore water pressure in the soil. The standard oedometer instrument cannot be used for preparing the consolidation soil samples because its range is limited only upto 10mm. For preparing samples from the soil slurry a deformation between 50mm to 100mm is required which depends upon the type of soils. Hence to achieve a large displacement, it was required to modify the oedometer instrument which fulfilled the above requirements. To achieve the above requirements, an instrument which could consolidate the soil upto 100mm was fabricated and shown in Fig. 9.1

2.2 Experiment set-up of modified oedometer instrument

Let W_1 = Reaction produced on the mould

W_2 = Weight applied at the free end

In standard oedometer instrument

l_1 = distance from the reaction W_1 to the free end = 46mm

l_2 = distance from the reaction W_2 to the mould end = 550mm

Hence $W_1 l_1 = W_2 l_2$

$$\frac{W_1}{W_2} = \frac{l_2}{l_1}$$

If δ_1 = deflection produced under the load W_1

δ_2 = deflection produced under the load W_2

Since W_1 and W_2 are constant, then δ is proportional to length, so

$$\frac{\delta_2}{\delta_1} = \frac{l_2}{l_1}$$

Hence $\frac{W_1}{W_2} = \frac{l_2}{l_1} = \frac{\delta_2}{\delta_1}$

Hence in order to increase the deflection under the applied load on CBR mould, the lever arm reaction was required to be increased.

Therefore, increased lever arms as shown in Fig. 1 are kept $l_1 = 270mm, l_2 = 1270mm$.

In this modified oedometer instrument, load is placed at the free end which transfer 4.70 times more load on perforated CBR mould through lever arm.

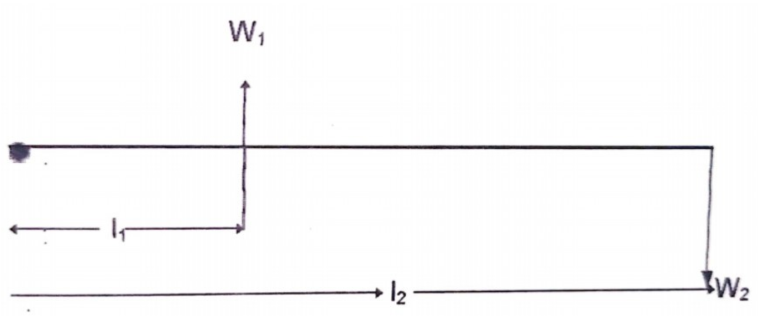

FIG. 01: MODIFIED OEDOMETER INSTRUMENT

2.3 Modified CBR Mould

In field consolidation, the drainage is in the vertical and radical directions. Hence to increase the rate of consolidation in the laboratory, small holes of diameter 1mm and 10mm apart were made on the cylindral surface, top circular plate and base plate of the CBR mould.

3. TEST PROCEDURE

Bhagalpur (Bihar) local Ganga's silt and naturally deposited kaolinite of kahalgaon- Rajmal region of eastern Bihar and Jharkhand were used to prepare soil samples of silt-clay matrices. Properties of soils used for preparing silt- clay matrix are given in Table 1.

A known dry weight of clay (Kaolinite) and dry mix of the known amount of silt were added to prepare silt-clay matrix and these matrices are shown in the Table 2. The numerical prefix attached to the SCM designation represents the % of clay by weight in the silt-clay matrix.

Remoulded samples were prepared by consolidating the soil slurry. For preparing remoulded samples, water was added in the amount twice that of the liquid limit for each silt – clay matrix to get the slurry of soil sample. All care was taken to remove the entrapped air from the sample as possible and the slurry was allowed to stand for each 24 hours so as to get saturated. wattman filter paper was cut in sizes and placed on the perforated base and around the radial drainage cylindrical surface of the modified CBR mould. The slurry was then transferred to the modified CBR mould and then left for 24 hours form self consolidation. Then the soil surface was properly levelled and covered with filter paper, and placed perforated circular plate on the top of the soil and left again for 24 hours. The CBR mould was then placed on the modified oedometer instrument for consolidation. The axial pressure on the samples were increased gradually upto 150 KN/m^2 which was similar to that followed in the standard consolidation test. The loading pattern which was followed in preparation of silt-clay matrices as shown in Fig. 1 which confirms to the IS specification for consolidation test. The soil was allowed to consolidate under each load for 24 hours. Three – dimensional drainage was allowed during consolidation. This process yielded a block sample. The standard consolidation test soil samples of 60mm diameter for one series of tests were extracted from this block sample in consolidation rings.

The above procedure was repeated for samples containing different percentage of kaolinite (clay) i.e 0%, 10%, 20%, 30%, 40%, 50% ,60%, 70%, 80%,90% and 100% of kaolinite by weight . 60SCM means 60% clay and 40% silt.

3.2 Determination of index properties and consolidation characteridtics of silt-clay Matrices

The liquid limit, plastic limit, void ratio, coefficient of consolidation, compression index, coefficient of volume compressibility etc were determined from laboratory test results for each SCM as per the recommendations of the Bureau of Indian Standards for different tests. The test results of the soil classification of different silt- clay matrices used herein are given in Table 2.

IV. TEST RESULTS AND DISCUSSION

4.1 Test results

The test results have been presented in figure from Fig. 1 to Fig.8. Fig.2represents the variation of void ratio with consolidation pressure for different SCM soil samples. The curve reflects that the void ratio is decreasing with increase the consolidation pressure. Upto 100 KN/m^2 consolidation pressure, the void ratio decrease considerably. For consolidation pressure more than 100 KN/m^2, decrease in the void ratio is too small and in 60 SCM, this variation is practically negligible. Relatively in large size particles, the pore water would be more easily dissipated and the particles would find its easier to readjust them so that on the application of consolidation pressure, there would initially be a large reduction in void ratio and consequently a continuing small decrease as the pressure is increased. 60 SCM gives the minimum void ratio as compared to other samples for all the consolidation pressure.

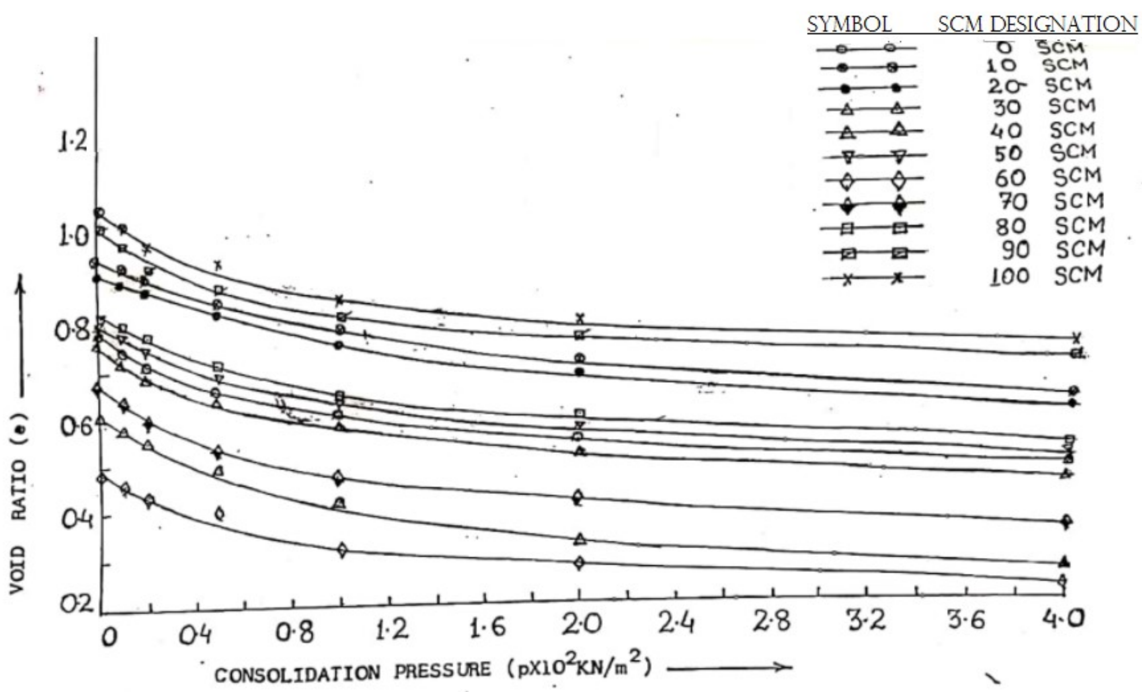

FIG. 02: VARIATION OF VOID RATIO WITH CONSOLIDATION PRESSURE.

Fig. 3 shows the variation of m_v with % clay content in SCM soil samples. For pressure range of 0 to 10 KN/m^2, 10 KN/m^2 to 20 KN/m^2 and 20 to 50 KN/m^2 , it was observed that there is a decrease in m_v upto 20 SCM is small.

Almost all pressure ranges, there is a least decrease in m_v upto 60 SCM. More than 60 SCM, m_v increase with increase in % of clay content. This shows that with increase the pressure, rate of volume change decrease and at 60 SCM has lowest value of m_v

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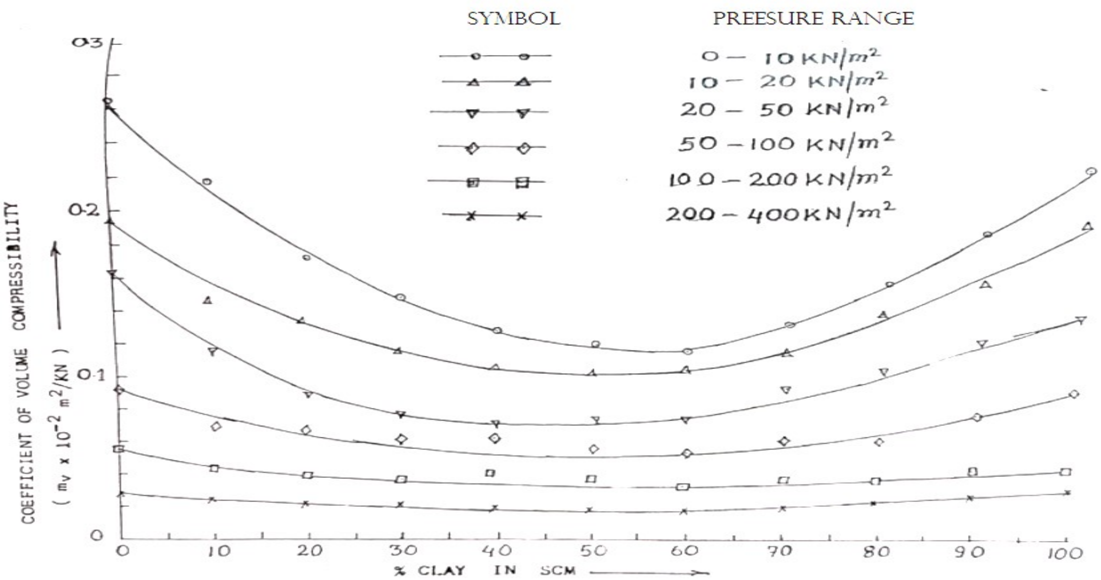

Fig. 03: VARIATION OF COEFFICIENT OF VOLUME COMPRESSIBILITY WITH CLAY CONTENT IN SCM

Fig. 4 shows the variation of m_v with increase in consolidation pressure (p). tt shows that m_v increases with consolidation pressure upto 20 KN/m^2 for all SCM. More than 20 KN/m^2 and upto 200 KN/m^2, there is a large variation of m_v (decrease) in almost all SCM. But for more than 200 KN/m^2, the trend of decrease of m_v is least at 60 SCM at all pressure range. This shown that settlement of SCM would be smaller at 60 SCM. Hence minimum quantity of silt (local Bhagalpur soil) to be added in kaolinite deposits for minimum settlement of structure is 40% of local soil by weight.

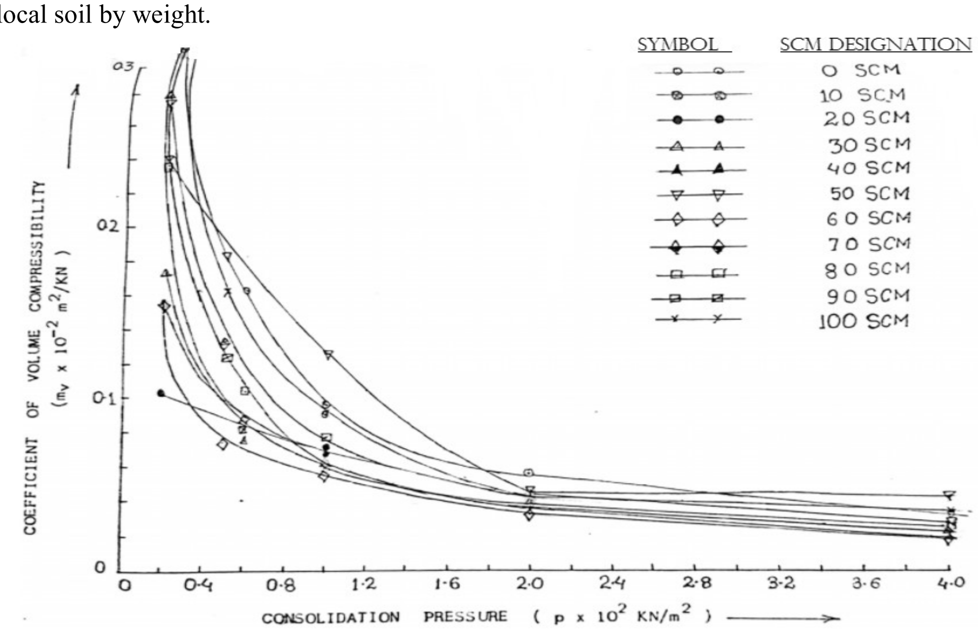

Fig. 04: VARIATION OF COEFFICIENT OF VOLUME COMPRESSIBILITY WITH CONSOLIDATION PRESSURE

Fig. 5 represents the variation of c_c with SCM .Terzaghi and Peck curve is above the experimental curve and almost parallel between 0 to 60 SCM. Both curve coincide above 90 SCM. Hence for clay content 60% and more, Terzaghi and Peck empirical relation may be used to determine the value of c_c for settlement calculation .

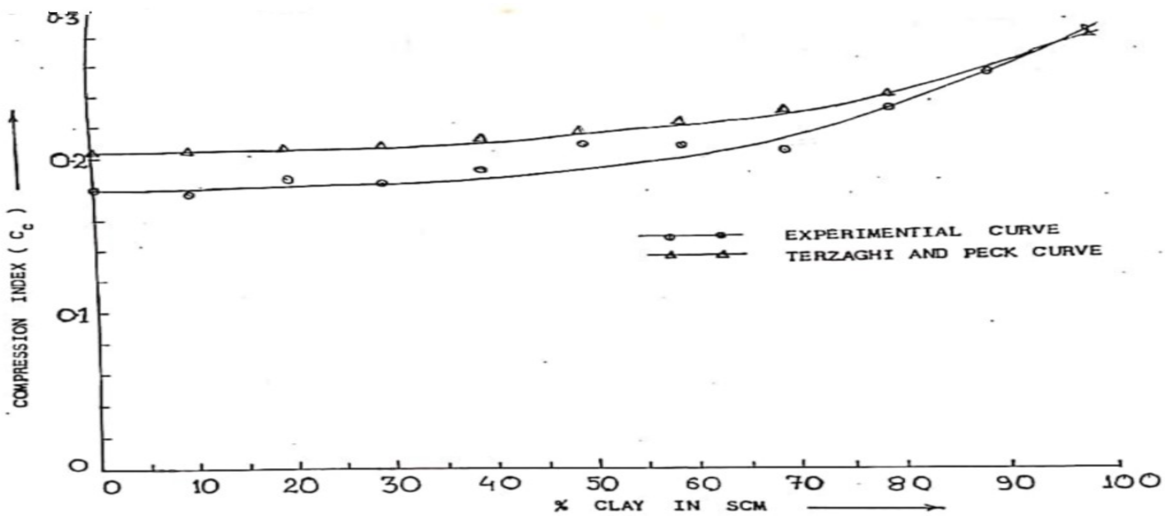

FIG. 05 VARIATION OF COMPRESSION INDEX WITH CLAY CONTENT IN SCM

Fig. 6 represents the variation of c_v with SCM. The value c_v is increasing with% of clay content in SCM. This variation is more at low consolidation pressure ranges and least variation of high pressure ranges. For pressure 0 to 50 KN/m^2, the variation in large but for pressure range 50 to 400 KN/m^2, the variation is small and least at 400 KN/m^2.

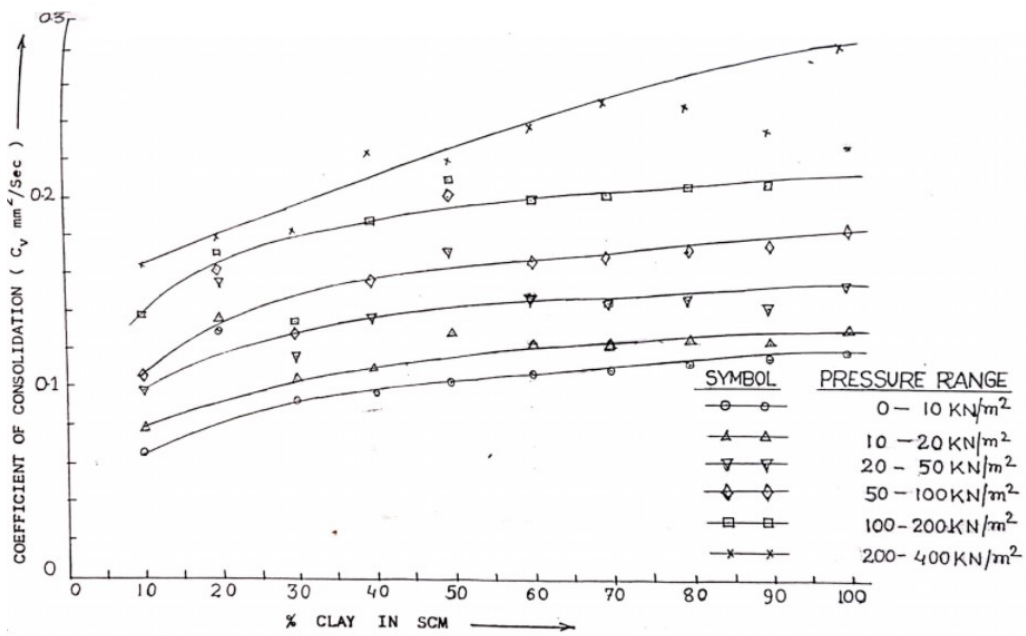

Fig. 06: VARIATION OF COEFFICIENT OF CONSOLIDATION WITH CLAY CONTENT IN SCM

Fig. 7 represents the variation of c_v with the consolidation pressure. The value of c_v increase with an increase in pressure. This variation is more in silt and least in 60 SCM soil sample. For different SCM, increasing pressure increases the rate of consolidation throughout the time of its application and consequently the c_v increases as the amount of load increases. These characteristics would be expected to show substantial variation depending on the particle size and perfection of clay crystals as indicated by the Cornell University Investigation. The value of c_v was also calculated from t_{50}. For all pressure ranges, it has been founded to increase with increase in pressure.

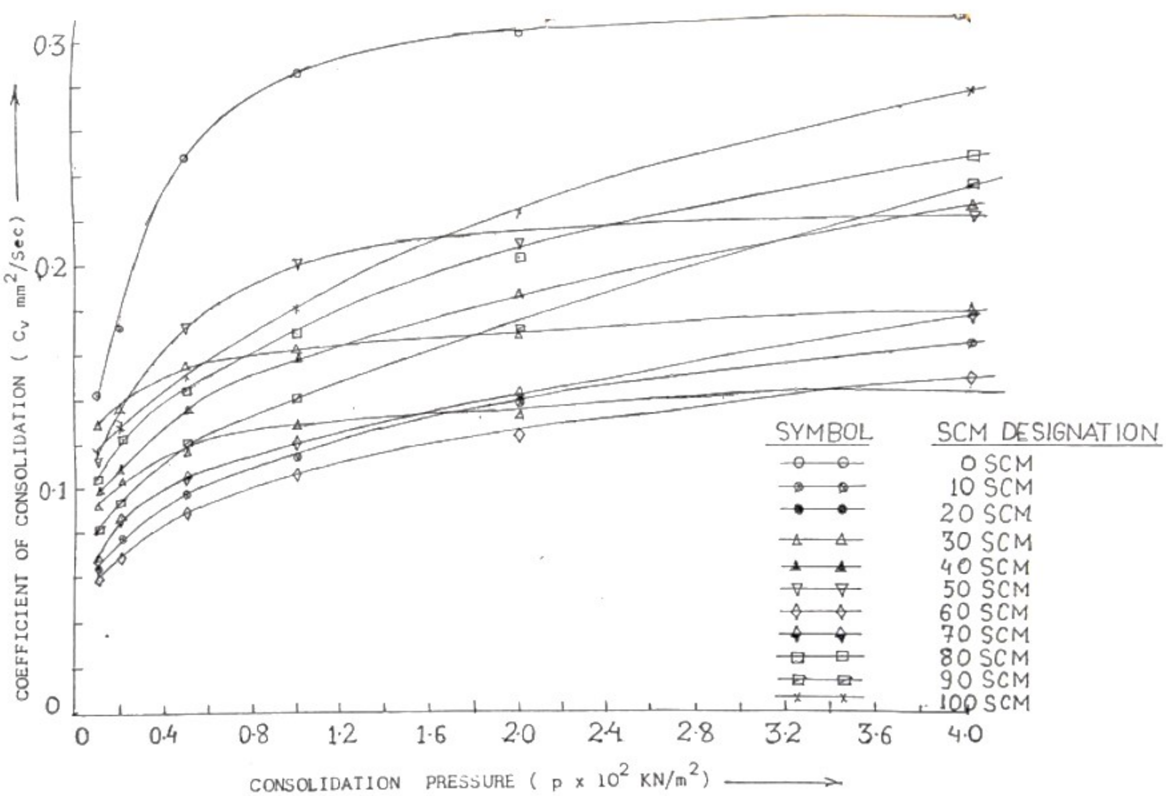

FIG. 07: VARIATION OF COFFICIENT OF CONSOLIDATION WITH CONSOLIDATION PRESSURE

Fig. 8 represents the variation of one -dimensional consolidation settlement with three- dimensional consolidation settlement.

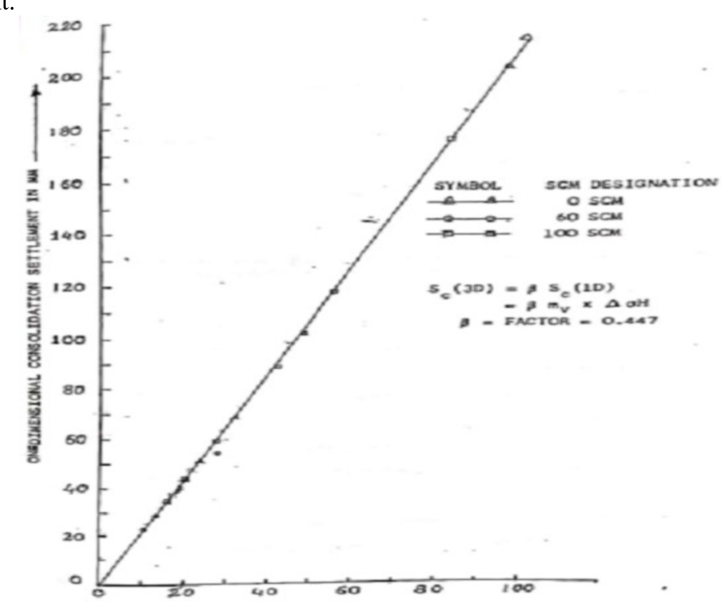

FIG. 08: VARIATION OF ONE-DIMENSIONAL CONSOLIDATION SETTLEMENT WITH THREE –DEMENSIONAL CONSOLIDATION SETTLEMENT

Initial, consolidation and total settlement of square footings of size 1 m to 5 m, thickness of compressible layers of 2 m to 10 m for applied pressure of 450 KN/m^2 have been calculated by Stress-Path and conventional methods for 0SCM, 60 SCM and 100SCM which is given in Table 3.

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A comparison between One-dimensional consolidation with Three-dimensional consolidation settlement calculated by Stress-Path and conventional for 0SCM, 60 SCM and 100 SCM. The value of consolidation settlement and correction factor β proposed by author based on the above computation are given in Table 4 .

4.2 Discussion on Test Results

Silt and kaolinite (clay) are fine-grained soil. D_{10} of silt is about 20 times D_{10} of kaolinite. Kaolinite fills voids between the silt particles and consequently coats the silt particle surface, hence increase the cohesion of silty soil. With the increase the clay content in SCM, liquid limit, plastic limit and plasticity index increases. Value of I_p remains constant upto 60 SCM and beyond that I_p increases sharply. A decrease in particle size would be accompanied by an increase in total surface and increase in I_p would expected (platters and winkler 1958). Also repeated wetting and moderate drying frequently lead to increase the plasticity characteristics of clay minerals.

Curve 2 indicates that void ratio is decreasing with increase in consolidation pressure upto 100 KN/m^2 consolidation pressure, void ratio decreases considerably and therefore decrease in void ratio is too small. At 60 SCM, void ratio is minimum as compared to other SCM for all consolidation pressures.

Curve 3 indicates that there is a small variation in m_V at 60 SCM for all pressure range from 0 to 400 KN/m^2 . More than 60 SCM, m_V increase with increasing % of clay content. This shows that with increase in consolidation pressure, m_v decreases. Hence the value of m_V is least at 60 SCM.

Curve 4 indicates that m_V is large with consolidation pressure upto 20 KN/m^2 for all SCM. For more than 20 KN/m^2 and upto 200 KN/m^2 , these is a large decrease in m_v in all most all SCM. But more than 200 KN/m^2, the decrease in m_v is slight at 60 SCM and has least value for all pressure ranges. It concludes that settlement of soil corresponding to 60 SCM would be too small. Hence 40% sill by weight may be kept minimum quantity of silt to be added in kaolinite deposits for minimum settlement of structure.

Curve 5 indicates that Terzaghi and peck curve is above the experimental curve and almost paralled between 0 to 60 SCM. Both curves coincide above 90 SCM. Hence for more than 60 SCM soil, Terzaghi and Peck empirical relation may be used to determine the value of C_c for calculating settlement.

Curve 6 indicates that C_v is increasing with % of clay content in SCM. The variation is large at low pressure and least variation at high pressure ranges. For pressure from 0 to 50 KN/m^2, the variation is large, but for pressure range of 50 to 400 KN/m^2, the variation is small and least at 400 KN/m^2.

Curve 7 indicates that the variation of C_v is more in silty soil and least in 60 SCM. For different SCM with increasing consolidation pressure, the rate of consolidation increases throughout the time of application and consequently the value of C_v also increases with the increase in consolidation pressure. There characteristics would be expected to show substantial variation depending upon particle size and clay crystals as indicated by Cornell University Investigation. Confirming to this phenomenon, decrease in m_v with increase in pressure, when γ_ω and K remains almost constant.

The value of consolidation settlement calculated from conventional methods (based on one – dimensional consolidation theory) gives higher value of settlement as compared to the “stress-path method”. To determine reliable value of consolidation settlement ,a correction coefficient (β) has been suggested by author.

In the case of SCM under study, attempt has been made by author to correlate one- dimensional consolidation settlement (ID) with three – dimensional consolidation settlement (3D) as shown in Fig. 8. Almost a linear relationship has been found to exit between 3D-consolidation settlement and ID- consolidation settlement and as being proposed.

Three – dimensional consolidation settlement

$$=\beta \times \text{one-dimensional consolidation settlement}$$

$$S_c(3D) = \beta S_c(1D)$$

Where β = settlement coefficient = 0.447 (From curve 8)

S_c = consolidation settlement

Settlement coefficient (β) proposed by the author to determine the 3D – consolidation settlement have the following features.

- The three dimensional stress changes that occur in the field during consolidation are primarily depends on the drained Poisson’s ratio and Skempton’s pore- pressure parameter A of the soil.
- As the excess pore water pressure dissipates ,Poisson’s ratio (0.5) decreases and finally drops to its fully drained value at the end of consolidation.
- Stress incremental ratio K at the end of consolidation is not equal to 1 but $1-\frac{\delta}{\Delta\mu}$ K depends on Poisons ratio, geometry of foundation, a pore – pressure parameter A and strain ratio λ of the soil.

- At the end of consolidation, $\Delta\sigma_v$ will remain unchanged and the horizontal stress will here decreased by an amount δ .
- Condition of no lateral strain may be approximately true in cases like that of a loaded area which is very large compared to the thickness of the clay layer. But in the majority of field problems, the above condition may be far from true.
- One – dimensional consolidation settlement with three – dimensional consolidation settlement as shown in Fig. 8 are plotted for $\frac{H}{b} = 0.25\ and\ 0.75(where\ B = 2b)$.
- It is noted that one – dimensional consolidation implies for $\lambda = 1$ and that will occur at a particular stress incremental ratio K. For any other value of K, the strain ratio λ would also be different and assumption of one dimensional strain ($\lambda = 1$) would lead to significant error in settlement analysis.

Correction factor (μ) developed by Scott and used by skempton – Bjerrum for computing the three-dimensional consolidation settlement have following features:

- Scott considered Poisson's ratio 0.5.
- Horizontal stress is adjusted during consolidation in such a manner that the condition of no lateral strain is enforced.
- Stress incremental ratio $(\propto)$ is only dependent on the proportions of the loaded area and the thickness of the clay layer in which consolidation takes place. According to Scott, the value of $\propto$ is constant during consolidation.
- Skempton also pointed out that the sum of the initial settlement plus the corrected consolidation settlement is not equal to the oedometeric settlement since the lateral strains are absent in that equation.
- Scott suggested that to a certain extent, soil behave in an approximately elastic manner and that therefore solutions to soil stressing situations of interest in the field of soil mechanics which are obtained by the methods of the classical theory of elasticity deserve examination. These are problems of elastic equilibrium. Ats large stresses, plastic deformations occur in soils.

V. CONCLUSIONS

The deformation behaviour of an artificially normally consolidated silt-clay matrices have been studied. From experiment results, it has been observed that 60 SCM is best proportion for any construction work. 60 SCM is suitable soil mix from all consideration like bearing capacity and settlement criteria. From the experimental results, it is concluded that ground improvement be done by mixing of 40% local silt in the natural kaolinite deposits at khalgaon –Rajmahal region.

For SCM a relation between one-dimensional consolidation settlement and three – dimensional consolidation settlement has been developed.

3D consolidation settlement
$= \beta \times 1D\ consolidation\ settlement$

Table 1: properties of Materials

properties	Local silt (0SCM)	Kaolinite (100SCM)
Grain Size Analysis	-----	-----
Clay	01.50%	32.00%
Silt	63.50%	42.00%
Sand (fine)	35.00%	26.00%
D_{10}	0.0092mm	0.00043mm
D_{30}	0.0200mm	0.00160mm
D_{60}	0.0400mm	0.01500mm
C_u	4.350	34.88
C_c	1.090	0.40
Consistency limit		
Liquid limit	32.65	39.37
Plastic limit	23.00	27.80

Plasticity index	9.65	12.60
Soil classification	ML	MI
% Free swell % DFS	- -	30.76% 13.33%

Table 2 : Test Results of Soil Classification

SI No	SCM Designation	Grain Size Distribution in %			Specific gravity	Atterberg's limit			Soil type classifications
		Fine sand	**Silt**	**Clay**		Liquid limit W_L	Plastic limit W_p	Plastic index I_p	
1	100 SCM	26.00	42.00	32.00	2.58	39.37	27.8	12.6	MI
2	90 SCM	26.90	44.15	28.95	2.59	37.0	25.8	11.2	MI
3	80 SCM	27.80	46.30	25.90	2.59	35.7	25.1	10.6	MI
4	70 SCM	28.70	48.54	22.85	2.60	34.8	24.6	10.2	ML
5	60 SCM	29.60	50.60	19.80	2.61	34.2	24.2	10.0	ML
6	50 SCM	30.50	52.75	16.75	2.62	33.7	23.9	9.8	ML
7	40 SCM	31.40	54.90	13.70	2.64	33.3	23.6	9.7	ML
8	30 SCM	32.30	57.05	10.65	2.65	33.0	23.4	9.6	ML
9	20 SCM	33.20	59.20	07.60	2.68	32.8	23.2	9.6	ML
10	10 SCM	34.10	61.25	04.15	2.70	32.7	23.1	9.6	ML
11	0 SCM	35.00	63.50	01.50	2.71	32.65	23.0	9.65	ML

Table 3: Comparison of Settlement Computed by Different Method.

SCM Designation	Widt of square footing m	Equivalent dia of square footing B=2b	Applied footing pressure KN/m^2	Layer	Thickness of compressible layer H	$\Delta\sigma KN/$	Mid depth of compressible layer Zm	Stress Path Method							Convectional method			
								K	λ	Si mm	Total Si mm	Consolidation Settlement $Sc = \lambda\, m_v \Delta\sigma H$		Total settlement S in mm	Immediate Settlement Si in mm	Consolidation Settlement $Sc = \lambda\, m_v \Delta\sigma H$		Total Settlement S in mm
												Sc	Total Sc			Sc in mm	Total Sc in mm	
60 SCM	1	1.128	450	1st	1	316.0	0.05	0.687	0.455	2.83	3.34	43.13	53.15	56.49	10.53	94.80	118.95	129.48
				2nd	1	80.5	1.5	0.777	0.415	0.51		10.02				24.15		
	2	2.257	112.5	1st	2	78.8	1.0	0.672	0.464	1.48	1.75	21.94	27.01	28.76	5.27	42.30	54.36	59.63
				2nd	2	20.1	3.0	0.766	0.420	0.27		5.07				12.06		
	3	3.385	50.0	1st	3	35.2	1.5	0.687	0.455	0.94	1.11	14.41	17.74	18.85	3.51	31.70	39.73	43.24
				2nd	3	8.90	4.5	0.777	0.415	0.17		3.33				8.03		
	4	4.514	28.1	1st	4	19.6	2.0	0.686	0.456	0.70	0.83	10.73	13.24	14.07	2.63	23.52	29.57	32.20

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				2nd	4	5.04	6.0	0.777	0.415	0.13		2.51				6.05		
	5	5.65	18.0	1st	5	12.6	2.5	0.686	0.456	0.57	0.67	8.62	10.62	11.29	2.11	18.90	23.70	25.81
				2nd	5	3.2	7.5	0.777	0.415	0.10		2.00				4.80		
100 SCM	1	1.128	450	1st	1	316.0	0.5	0.687	0.455	2.83	3.34	74.77	92.14	95.48	10.53	164.32	206.18	216.71
				2nd	1	80.5	1.5	0.777	0.415	0.51		17.37				41.86		
	2	2.257	112.5	1st	2	78.8	1.0	0.672	0.464	1.48	1.75	38.03	46.81	48.56	5.27	81.95	102.85	108.12
				2nd	2	20.1	3.0	0.766	0.420	0.27		8.78				20.90		
	3	3.385	50.0	1st	3	35.2	1.5	0.687	0.455	0.94	1.11	24.98	30.74	31.85	3.51	54.91	68.79	72.30
				2nd	3	8.90	4.5	0.777	0.415	0.17		5.76				13.88		
	4	4.514	28.1	1st	4	19.6	2.0	0.686	0.456	0.70	0.83	18.59	22.94	23.77	2.63	40.77	51.25	53.88
				2nd	4	5.04	6.0	0.777	0.415	0.13		4.35				10.48		
	5	5.65	18.0	1st	5	12.6	2.5	0.686	0.456	0.57	0.67	14.94	18.48	19.15	2.11	32.76	41.08	43.19
				2nd	5	3.2	7.5	0.777	0.415	0.10		3.54				8.32		
	1	1.128	450	1st	1	316.0	0.5	0.687	0.455	2.83	3.34	64.47	79.50	82.84	10.53	142.20	178.43	188.96
				2nd	1	80.5	1.5	0.777	0.415	0.51		15.03				36.3		
	2	2.257	112.5	1st	2	78.8	1.0	0.672	0.464	1.48	1.75	32.91	40.51	42.26	5.27	70.92	89.01	98.28
				2nd	2	20.1	3.0	0.766	0.420	0.27		7.60				18.09		
	3	3.385	50.0	1st	3	35.2	1.5	0.687	0.455	0.94	1.11	21.62	26.61	27.72	3.51	47.52	59.54	63.05
				2nd	3	8.9	4.5	0.777	0.415	0.17		4.99				12.02		
	4	4.514	28.1	1st	4	19.6	2.0	0.686	0.456	0.70	0.83	16.09	20.05	20.88	2.63	35.28	44.35	49.96
				2nd	4	5.04	6.0	0.777	0.415	0.13		3.96				9.07		
	5	5.65	18.0	1st	5	12.6	2.5	0.686	0.456	0.57	0.67	12.93	15.92	16.59	2.11	28.35	35.55	37.66
				2nd	5	3.2	7.5	0.777	0.415	0.10		2.99				7.20		

TABLE-4

Comparison of one -dimensional consolidation settlement with three -dimensional consolidation settlement computed by different methods.

Width of footing in m	Computed consolidation settlement in mm						Correction factor		Three- dimensional consolidation settlement Sc(3D) in mm					
	60 SCM		100SCM		0 SCM				60 SCM		100SCM		0 SCM	
	Stress path Method Sc(3D)	Conventional method Sc (ID)	Stress path Method Sc(3D)	Conventional method Sc (ID)	Stress path Method Sc(3D)	Conventional method Sc (ID)	Scott(1963) & Recommended by IS Code IS 80009, Part-I 1976	Proposed by Author for different SCM	Skempton -Bjerrum (1957)	Proposed by Author	Skempton -Bjerrum (1957)	Proposed by Author	Skempton- Bjerrum (1957)	Proposed by Author
							μ	β	$\mu m, \Delta\sigma$ $, H$	$\beta m,, \Delta\sigma$ $, H$	$\mu m, \Delta\sigma$ $, H$	$\beta m,, \Delta\sigma$ $, H$	$\mu m, \Delta\sigma$ H	$\beta m,, \Delta\sigma$ $, H$
1	53.15	118.95	92.14	206.18	79.50	178.43	0.82	0.447	97.54	53.17	169.07	92.16	146.31	79.76
2	27.01	54.36	46.81	102.85	40.51	89.01	0.82	0.447	44.36	24.30	84.34	45.97	72.99	39.79
3	17.74	39.73	30.74	68.79	26.61	59.54	0.82	0.447	32.58	17.76	56.41	30.75	48.82	26.61
4	13.24	29.27	22.94	51.25	20.05	44.35	0.82	0.447	24.00	13.08	42.03	22.91	36.37	19.82
5	10.62	23.70	18.48	41.08	15.92	35.55	0.82	0.447	19.43	10.59	33.69	18.36	29.15	15.89

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