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Hardening Behavior of Stabilized Marl with Cement

F.Kalantary
Asst. Prof., Dept. Of Civil Eng., Guilan University
M.Aghajani
MSc. Geotech. Student, Guilan University

Abstract

Hardening of stabilized soils with cement and lime has been the subject of much investigation [1]. In this study it has been endeavored to experimentally investigate the mechanical behavior of stabilized red-marl with ordinary Portland cement. Various tests were carried out on cylindrical and prismatic, compacted specimens to evaluate tensile and compressive yield loads and modulus of elasticity of the material .The effect of cement content and curing time was studied on mechanical characteristics as well as parameters of the failure criterion.

Sample with cement contents of 4, 6, 8 and 10% and curing times of 2 and 4 weeks were prepared at constant densities and moisture content.

Introduction

In engineering practice, what one looks for in soil is firstly strength, then durability and certain permeability. At all time the mass is required to retain its original compacted volume. However, in actual fact hardly any soil in its original state possesses the above-mentioned qualities, and thus the practice of soil stabilization has come about.

This practice is concerned with any process by which these properties may be improved. There are a number of methods to achieve this.

Stabilization by mixing in a stabilizer/binder is probably one of the earliest practices. Usually when the soil properties are such that by pure compaction, it can not reach the necessary standards, then use is made of additives which when properly mixed with soil and appropriately processed, stabilizes the mass.

There are several different stabilizers; the most celebrated of all are lime and cement.

Cement stabilization has been very much associated with coarse grained, non-plastic soils, where as lime stabilization has been associated with fine-grained clayey soils.

However, marl has already 30-50% lime content, and in cases where no other more suitable borrow areas are with in the economical vicinity of the project site, such as south Khuzestan flood plains, and when high durability sub-base or earth fill is required, addition of cement has proven the most viable of the options.

Thus, an experimental program was envisaged, not only to determine the improved mechanical characteristics of the stabilized marl, but also to make use of the data to develop a hardening parameter in terms of variable factors such as cement content and curing time.

These characteristics may be employed to numerically or otherwise model the behavior of sub-base or earth fills under traffic or foundation loading as a semi-rigid medium between foundation and soft earth underneath.

Experimental Program

In order to investigate the hardening effect of cement on marl, a low-plasticity red-marl was chosen with the following basic properties:

$$G_S = 2.7 \;,\; \gamma_{\text{dmax}} = 1.85\,\text{Mg/m}^3 \;,\; w_{\text{opt}} = 16\%$$

The marl was initially dried and ground and kept in an airtight container.

Although it was known that addition of cement would change the basic properties, all of the samples were prepared with the optimum moisture content and maximum dry density of the pure marl.

The process of sample preparation was to calculate the required mass of soil-cement and moisture content according to the volume of cylindrical and prismatic moulds. The inside dimensions of the moulds are as follows:

Cylindrical mould

- Diameter = 50 mm
- Height = 100 mm

- Volume = 196349.5 mm3

Prismatic mould

- Cross-section = 50*50 mm2
- Length = 150 mm
- Volume = 375000 mm3

Four different cement contents of 4, 6, 8 and 10% were considered for each set of samples and two samples were prepared in order to reduce the probability of errors.

Table 1 provides the calculated masses of each constituent of the various samples. The calculated mass was increased by 10% in order to provide for measurement of moisture content. An electronic weighing device with an accuracy of two decimals was used to make up the material.

Table 1: Calculated masses of samples

	Cylindrical			
	4%	6%	8%	10%
M_s	384.21	379.96	369.97	363.24
M_c	15.37	22.62	29.6	36.32
M_w	63.93	63.93	63.93	63.93
M_{tot}	463.5	466.51	460.5	463.49
	Prismatic			
	4%	6%	8%	10%
M_s	733.78	719.93	706.6	693.75
M_c	29.35	43.2	56.53	69.375
M_w	122.1	122.1	122.1	122.1
M_{tot}	885.23	885.23	885.23	885.23

The dimensions of the samples were all measured accurately with a micrometer before testing and since the moulds were of very high quality, the final dimensions of the produced specimens were found to be with in a tolerance of 0.7 mm and 2.0 mm in cross-section and length wise respectively, which must be associated with elastic rebound of specimens on extrusion from moulds.

The achieved dry densities were all grater than 95% of the maximum dry density obtained by modified proctor test.

The cylindrical specimens were used in unconfined compression test as well as direct and indirect tensile tests, whereas prismatic bars were used in two-point bending tests only.

In the unconfined compression test and direct tensile tests two dial gauges with an accuracy of measuring 0.002 mm displacement were positioned on either sides of the specimens at equal distances so that any un-even deformation could be recorded accurately, since the loading cap must have been pinned.

The strain controlled compression tests were performed with a fixed rate of imposed displacement of 1.27 mm/min. However, since the range of load in the tensile and bending tests were very small, manual incremental loading was thought to be more suitable.

Interestingly, the only consistent and reliable test results were obtained firstly from compression tests and then direct tensile tests. The Brazilian and two point bending tests yielded erratic results.

The curing times that were considered in this part of the research program were 14 and 28 days. Specimens were rapped up in cling film upon extrusion from moulds, and kept at room temperature (i.e. 19°c)

Tests Results

Two main features of the samples were sought at this stage of the research program, one was Young's modulus and the other was tensile and compressive strength. Due to the brittleness of the material under consideration, yield and failure were almost one and quite abrupt. Hence, the ultimate sustained load was assumed as Unconfined Compressive Strength (UCS) or Tensile Strength (TS).

However, the deformation modulus was more tedious to work out due to various effects. The shapes of stress-strain curves were non-linear. In the initial part of loading the material responded softly until the loading cap was fully set in. After this initial effect, the material behaved in almost a hyperelastic fashion. However, since it was decided not to enter such complication as non-liner elasticity, the secant modulus in the working range of load (i.e. 50% of yield) was determined, omitting all other effects.

A summery of the results from unconfined compression tests and uniaxial-tensile tests are provided in table 2 and 3 respectively. Table 4 provides the averaged values for both tests.

It appears that the measured tensile strength of the specimens are of order of 11~16%, of the unconfined compressive strength, with an average of 12.5% (excluding 6%-14 day sample).

This results ties in with the well-known rule of thumb for calculation of tensile strength in term of compressive strength.

The variation of compressive and tensile strength with cement content and curing time are shown on figures 1 to 4.

Table 2: E (GPa) & UCS (KPa) values of specimens in compression test

C.C. (%)	14 Days			
	1		2	
	E	UCS	E	UCS
4	0.9903	2013.1	0.9060	1878.8
6	1.0960	2138.8	1.0038	2212.3
8	1.3797	3178.3	1.4245	3178.3
10	1.7788	3899.4	1.2264	3843.5
	28 Days			
4	0.9826	2265.7	0.7132	2073.9
6	1.1225	2716.6	0.8065	2786.9
8	1.5151	3435.0	2.2973	3628.0
10	1.7929	4968.7	1.3064	4217.7

Table 3: E (GPa) & T.S (KPa) values of specimens in tension test

C.C. (%)	14 Days			
	1		2	
	E	T.S	E	T.S
4	1.3555	227.4	2.2006	303.1
6	1.7242	457.4	1.2524	482.4
8	1.7743	378.6	-	75.6?
10	-	-	-	-
	28 Days			
4	1.5893	252.5	1.2907	228.3
6	1.6603	432.3	1.1956	457.1
8	1.3408	489.6	1.1840	455.1
10	1.6137	506.8	1.7644	456.0

Table 4: Average values of E (GPa), UCS (KPa) and TS (KPa) in tension and compression tests.

C.C. (%)	14 Days			
	Compression		Tension	
	E	UCS	E	T.S
4	0.9384	1930.5	1.5668	240.0
6	1.0591	2175.6	1.3706	458.5
8	1.4021	3178.3	1.7743	378.6
10	1.6866	3871.5	_	_
	28 Days			
4	0.9845	2244.4	1.3654	384.
6	1.083	2751.8	1.4279	452.2
8	1.602	3531.5	1.3147	473.9
10	1.8956	4443.0	1.6891	487.8

As is evident from figure 1, there is almost a 20% gain in compressive strength by doubling the curing time.

However, due to erratic results obtained from 14-day curing time, tensile tests, no definite conclusions may be drawn from figure 2.

There is also on average a 20% gain in compressive strength by every 2% increase in cement content (fig.3). Again, the tensile test results provide no firm conclusion (fig.4).

Unfortunately, the samples tested in two point bending and Brazilian and tensile tests at 14 days did not yield favorable results; the 10% cement content samples were all unacceptable, as was one of the 8% cement content samples. Although the 6% cement content samples provided a consistent set of results, but they did not conform to the general pattern of behavior. It was therefore decided to concentrate on the results obtained from 28 days samples.

There appear from figure 5 that no significant difference exists in Young's modulus at different curing time in compression tests. Again, the values obtained from tensile tests are erratic as well as being consistently of higher magnitude than the corresponding values in compression tests. The variation of deformation modulus with cement content appears to be linear.

Hardening behavior

Although it is a known fact that the failure envelope of frictional materials is non-linear in the $\tau_{oct} - \sigma_{oct}$ space [2], and the rate of increase of shear strength with confining pressure diminishes at high confining pressures, the envelope is assumed to be linear in this study. In other words, a two-parameter failure criterion such as Drucker-Prager or Mohr-Coulomb suffices, since the mean pressures encountered in practice are nominal.

The loading path of the uniaxial compressive and tensile tests in the $\tau_{oct} - \sigma_{oct}$ space follows a line inclined at $\pm 1:\sqrt{2}$ (assuming compression to be positive), passing through the origin and ending at failure.

Therefore, two points on the failure envelope may be determined and thus a linear equation may be set up for each set of samples. (Fig.6)

$$\tau_{oct} = S + m.\sigma_{oct}$$

Where in uniaxial tests:

$$\sigma_{oct} = \frac{\sigma_1}{3}$$

and

$$\tau_{oct} = \frac{\sqrt{2}}{3}\sigma_1$$

and using Mohr-Coulomb concept:

$$\sigma_c = \frac{2C\cos\varphi}{1-\sin\varphi}$$

and

$$\sigma_t = \frac{2C\cos\varphi}{1+\sin\varphi}$$

Therefore:

$$m = \sqrt{2}\frac{(\sigma_c - \sigma_t)}{(\sigma_c + \sigma_t)}$$

or $\quad m = \sqrt{2}\sin\varphi$

$$S = \frac{\sqrt{2}}{3}\frac{\sigma_t.\sigma_c}{\sigma_t + \sigma_c}$$

or $\quad S = \frac{\sqrt{2}}{3}\cos\varphi$

Based on 28-day samples the values of S and m or c and φ may be determined (Table 5) and their variation with cement content defined (Fig. 7 and Fig. 8)

Table 5: Shear strength parameters

C.C.. %	σ_t MPa	σ_c MPa	σ_t / σ_t	$\varphi°$	C Mpa
4	0.384	2.244	~17%	45.05	0.464
6	0.452	2.751	16%	45.87	0.558
8	0.473	3.531	13%	49.79	0.646
10	0.487	4.443	11%	53.36	0.735

On the other hand, it appears that the ratio of $\frac{\sigma_t}{\sigma_c}\left(\equiv\frac{T.S}{UCS}\right)$ decreases with increase in cement content, causing the increase in the angle of internal friction (Fig. 9)

Thus knowing the cement content and uniaxial compressive strength, all parameters may be defined.

Conclusion

It has been verified that the ratio of $\frac{\sigma_t}{\sigma_c}$ decreases with increase in stiffness of stabilized soils.

The approximate range of this ratio was found to be 10~17%. Thereby, through this method, shear strength parameters (i.e. c and φ or $\breve{S}$ and m) may be arrived at through uniaxial compression test.

References

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Fig1-Variation of UCS versus cement content

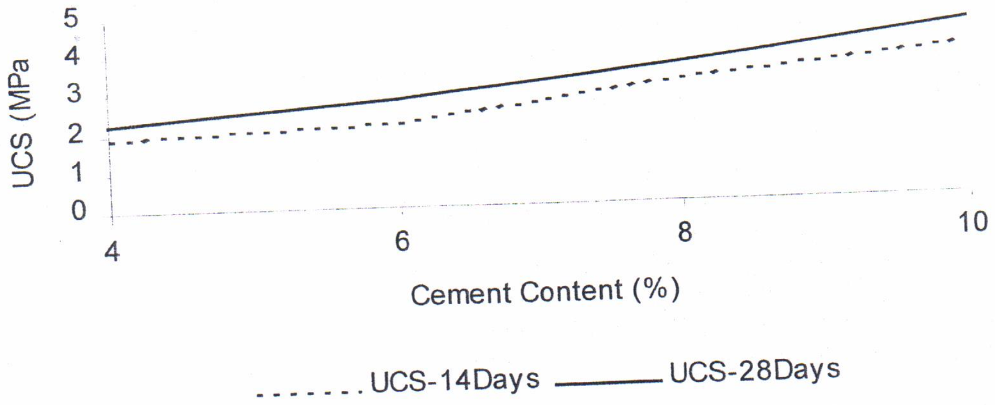

Fig2 : Variation of T.S versus cement content

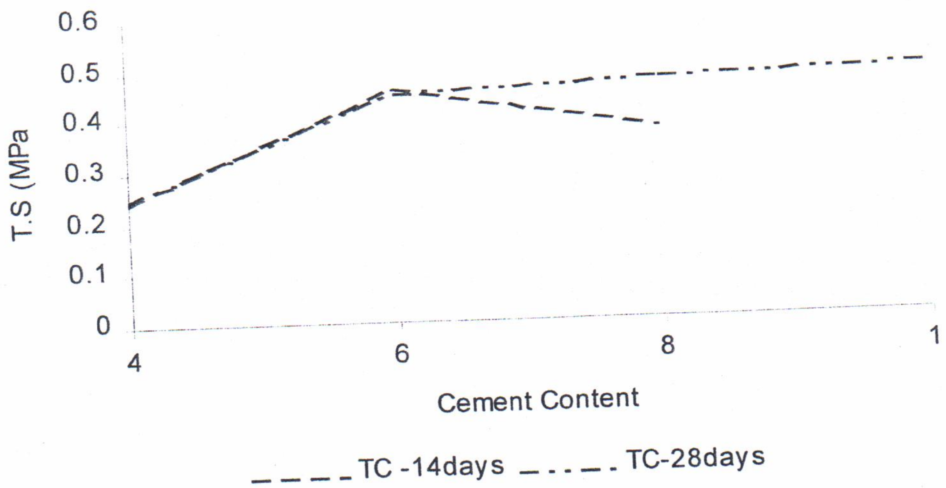

Fig3-Variation of UCS versus Curing time

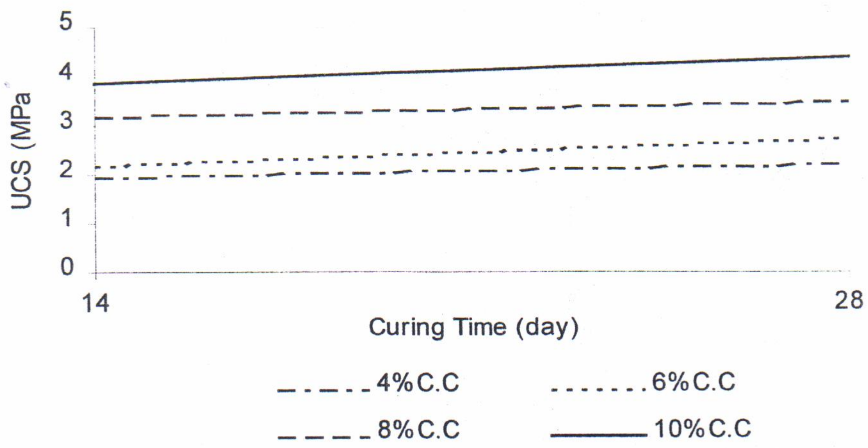

Fig4-Variation of T.S versus Curing time

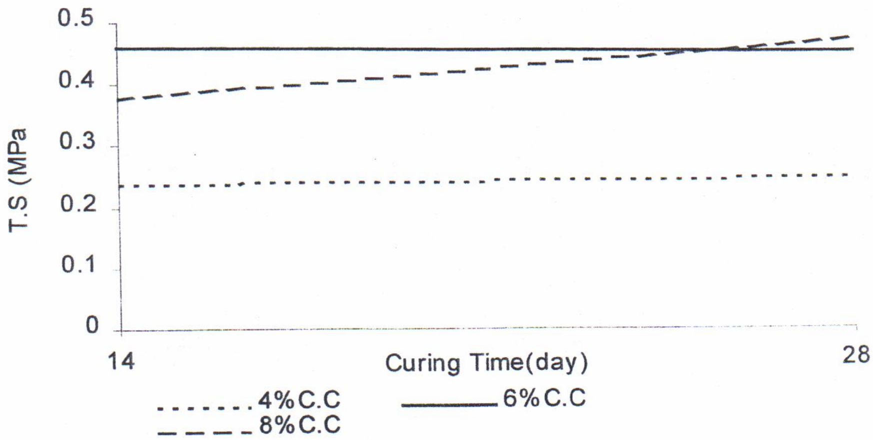

Figure 5 : Variation of E versus c.c

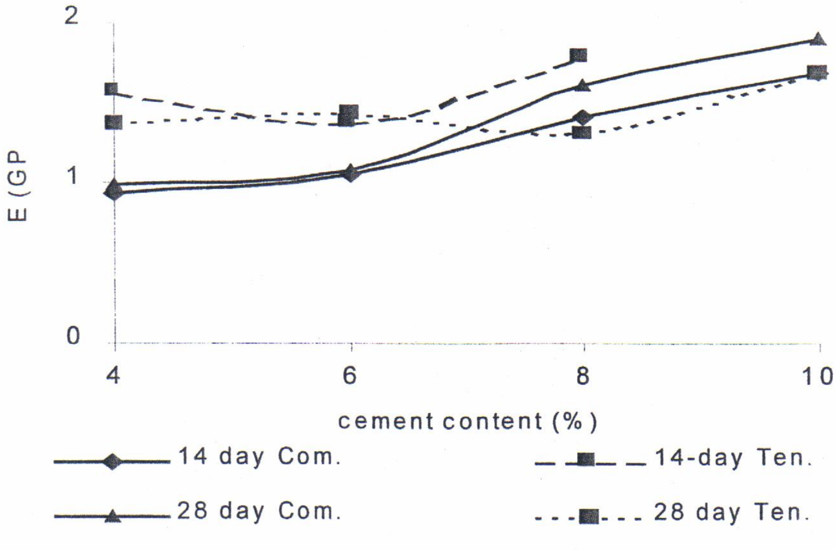

Fig 6 : Hardening Criterion

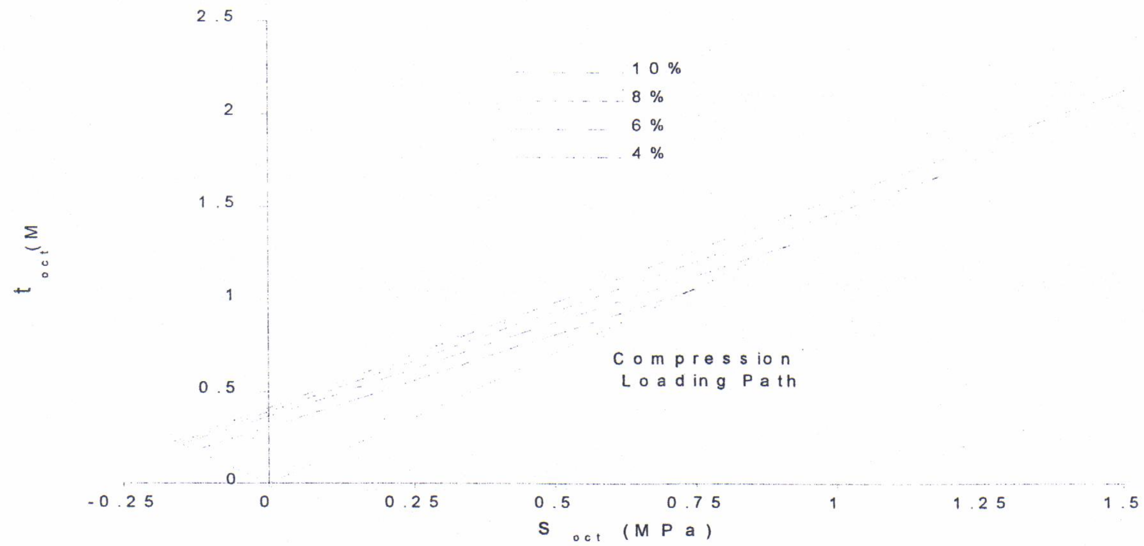

Fig 7 : Variation of C versus c.c

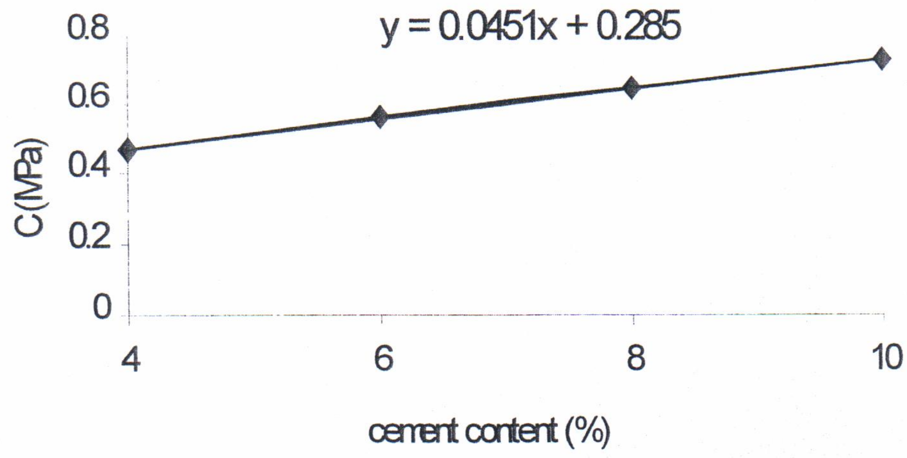

Fig 8 : Variation of f versus cement content