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Density, load, and fly ash effect on stabilization of high plasticity soil with lime

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ABSTRACT

The reuse of recycling materials or industrial waste materials with aims to reduce environmental pollution strongly supports the concept of green building. Fly Ash is the result of the combustion of pulverized system coal at the PLTU Tenayan and is no longer included in the B3 waste category. The use of fly ash as a building material, mine restoration, and roads in this decade, is to replace cement or lime. The fly ash composition is mixed with lime for the sub-base and will be applied on high-plasticity soils. A fix-mixture of soil and lime 5%, mixed with fly ash up to 30% of the mixture. The samples test was made at optimum moisture content, with density values around the maximum dry density (MDD) i.e. under or above MDD. Consolidated testing was performed with and without curing. Changes in load are represented by the load increment ratio (LIR). The selected LIR values were 1.0; 1,5; and 2.0. The results showed that the higher of density, the volume of the void is lower. The soil compression index value is the same for all density values if the soil structure has not been destroyed. or fatigued yet. In samples with crushed/broken soil structures, the value of the compressibility index decreased sharply. Curing successfully decreased the void ratio and compressibility of the soil. The strength of fly ash will decrease when reacting with water, so if the soil is burdened, the void ratio decreases drastically. The formation of strong molecular bonds between fly ash and lime takes time. So, the compressibility value of the sample by curing for 28 days is better than without curing. The composition levels between fly ash and lime also affect the compressibility index of the mixture. The optimum combination occurs in samples with a fly ash content of 25%.

1. Introduction

The reuse of used materials or debris from buildings and the use of industrial waste to reduce environmental pollution strongly supports the concept of green building. Most of the "toxic and hazardous materials" (B3 waste) come from specific sources, specifically from waste produced by factories or industries. Actions to control and manage B3 waste must go through measurable and structured procedures with a sequence of storage, accumulation, lifting, utilization, management, and stockpiling. Coal ash (fly ash and bottom ash, FABA), the result of burning coal for power plants, was previously included in the category of B3 waste, but based on Government Regulation

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Number 22 in 2021 concerning Implementation of Environmental Protection and Management, FABA material produced from the combustion process Pulverized Coal (PC) combustion systems or chain grate stokers, such as steam power plants, are no longer categorized as B3 waste. FABA (as well as CCP/Coal Combustion Products) can be used as building materials (brick, concrete brick), cement substitution, road pavement, mine access roads, underground mining, and mine restoration. Fly ash is the result of coal combustion with a pulverized system with a grain size of <0.075 mm (passing filter# No.200) and bottom ash with a grain size of >0.075 mm. Fly ash can lose its strength when it comes to contact with water. While bottom ash, even though it has grain size like sand , but whenever it is loaded, the granules can crack due to its brittleness. Additives, which have pozzolanic properties, such as cement, lime, rice husk ash, fly ash, and base ash can be used to improve poor subgrade characteristics (Howayek et al., 2011), (Kikumoto et al., 2010) and (Zukri, 2013), but the use of added materials must be following the type of soil to be repaired (Hicks, 2002). Utilization of waste, which is pozzolanic, is highly recommended in the current decade (Wardani, 2008), (Wang and Wu, 2006) and (Deepak et al., 2020).

The use of fly ash and bottom ash (FABA) has been widely carried out to improve the characteristics of poor expansive soils (Ikeagwuani & Nwonu, 2019) , (Anggara et al., 2021) , (Dissanayake et al., 2017) and (Nugroho et al., 2022).). Fly ash can also be used as a substitute for cement (geo polymer) by adding an alkali activator (Kosnatha & Utomo, 2007).

Marine sand is relatively abundant and can be used as a substitute for river sand. (Feng et al., 2021) made a mortar from a mixture of marine sand (marine sand cement mortar) which increases the corrosion resistance of chemical ions. After adding fly ash, the setting time is shorter and the working properties and durability are significantly improved. (Jang & Lee, 2016), investigated the effect of fly ash characteristics on the strength development of fly ash geopolymers. The results provided insight into the development of geo-polymer strength that occurs due to the transformation of Aluminium-rich gels into Silicon-rich gels. The delay in the high strength development, due to the increment in the particle size of the fly ash, was more evident with the increment of vitreous phase content and the SiO_2/Al_2O_3 ratio of the fly ash.

Kuo et al., (2014) used steel manufacturing waste to produce cement-free concrete with the addition of superplasticizers to increase water absorption and increase the compressive strength of cement-free concrete. The feasibility of fabrication was evaluated by the slump, setting time, compressive strength, length alteration, and sulfate resistance. Clay stabilization with fly-ash-based geo polymer that combines ground granulated blast-furnace slag (GGBFS), to increase soil strength (Abdullah et al., 2017). Laboratory experiments were conducted on clay samples stabilized with fly ash geo polymer and cement. The results show that adding fly ash (class F) based geo polymer / GGBFS helps, when synthesized in certain concentrations, in achieving setting time and compressive strength. The lower water/binder ratio and higher activator content can help accelerate the setting and strength development of fly ash geopolymers that are cured at room temperature.

The addition of fly ash, as an additive, has been shown to increase the CBR value of clay (Apriyanti & Hambali, 2014) , (Nugroho et al., 2021) , (Kusuma et al., 2013) , (Darmawan et al., 2018) and (Nugroho, Fatnanta, et al., 2021) and reduce the swelling potential of the soil (Cheshomi et al., 2017) and (Lembasi et al., 2021)

The deformation behavior of silt stabilized with fly ash cement was studied experimentally with various stress paths. The surface of the soil failure is significantly more curved than the original soil. Bond failure began at a very low strain, and finished at about 1% axial strain. One-way cyclic loading did not cause sample degradation, considerable degradation occurred when the sample was subjected to two-way cyclic loading (Lo & Wardani, 1999). Soil strength and bearing capacity of the soil depend on the shear strength of the soil and are influenced by the physical and mechanical properties of the soil such as unit weight, plasticity, and grain type (Fernando et al., 2021; Nugroho et al., 2022; Nugroho and Fernando, et al. 2021)

One-dimensional consolidation test (1-D consolidation), can be used to study various soil characteristics (Nakai et al., 2012). The increase in the bearing capacity of the fly ash stabilized soil (Ozdemir, 2016) and the improvement in the expansion and shrinkage properties of the expansive soil (Mohanty, 2015) can also be studied from the 1-D consolidation test.

The current study aimed to observe the changes in the compressibility index, which is a characteristic of soil compression, in high-plasticity soils, stabilized by 5% lime, due to the addition of fly ash. The present research includes laboratory testing using an Oedometer, with variations in fly ash content, the ratio of increased loads, and sample treatment.

Calculation of the weight of dry soil samples, original soil mixture, fly ash, and lime, which must be prepared for testing uses Eq.(1).

$$W_{dry} = V \times \gamma_{dry} \quad (1)$$

With planned water content at an optimum water content of original soil, w = 25,2%, therefore :

$$W_{wet} = W_{dry} \times (1 + 0{,}252) \quad (2)$$

Unit weight of the sample can be written down as :

$$\gamma_{bulk} = \frac{W_{wet}}{V} \quad (3)$$

Where

V: volume of mold

γ_{bulk}: planned dry density

W_{dry}: weight of mixed soil (dry condition)

W_{wet}: weight of mixed soil

Changes in the value of the soil compressibility index, namely void ratio, coefficient of compression (Cc), and coefficient of expansion (Cs) will be reviewed from the influence of fly ash content, additional load, and soil density. The compression index and expansion index can be calculated using Eq. (4).

$$C_s = C_c = \frac{\Delta e}{\log(\sigma_2/\sigma_1)} \quad (4)$$

2. Methodology

The original soil in the form of high plasticity soil was taken from the border of Pekanbaru city. The original soil was mixed with fly ash with several dry weight ratios into a sample (Mix Sample, code MS) to be later stabilized using 5% lime. For each mix-sample and original soil, the maximum density value was determined by standard proctor testing in the laboratory. Maximum dry density for each sample variation is used as the median value of dry density or the weight of the volume for making samples for the consolidation test. The consolidation test sample is a mix-sample stabilized with 5% lime. Consolidation test samples were made by pressing (static) with a hydraulic jack according to the desired volume weight target, and by adding water according to the optimum moisture content (OMC) of the original soil by mixing it evenly with the dry sample mixture. The variation of the consolidation test sample is shown in Table 1.

Table 1. Samples variation and target of density

Description of Samples	Original soil (%)	fly ash (%)	sample ID	dry_density (gr/cm^3)	Water content (%)	unit weight (gr/cm^3)	MS (%)	Lime (%)
Var I MDD=1,37	80	20		1,45	25,2	1,815	95	5
	80	20		1,41	25,2	1,765	95	5
	80	20	MS_1	1,37	25,2	1,715	95	5
	80	20		1,33	25,2	1,665	95	5
	80	20		1,29	25,2	1,615	95	5
Var II MDD=1,29	75	25		1,37	25,2	1,715	95	5
	75	25		1,33	25,2	1,665	95	5
	75	25	MS_2	1,29	25,2	1,615	95	5
	75	25		1,25	25,2	1,565	95	5
	75	25		1,21	25,2	1,515	95	5
Var III MDD=1,27	80	30		1,35	25,2	1,690	95	5
	80	30		1,31	25,2	1,640	95	5
	80	30	MS_3	1,27	25,2	1,590	95	5
	80	30		1,23	25,2	1,540	95	5
	80	30		1,19	25,2	1,490	95	5

Figure 1. Experimental setup and the tools used to create consolidation test samples

Table 2. Load and load increment ratio of consolidation samples

Samples Description	LIR	unit	Loading Stage (day)						
			-1	-2	-3	-4	-5	-6	-7
Variation I [MS]=80 MH:20 FA MS 95%+L 5%	1,0	kg	1,00	2,00	4,00	8,00	16,00	8,00	4,00
	1,5	kg	1,00	2,50	6,25	15,60	39,60	19,50	9,77
	2,0	kg	1,00	3,00	9,00	27,00	81,00	40,50	20,30
Variation II [MS]=75 CH:25 FA MS: L=95:5	1,0	kg	1,00	2,00	4,00	8,00	16,00	8,00	4,00
	1,5	kg	1,00	2,50	6,50	15,60	39,60	19,50	9,77
	2,0	kg	1,00	3,00	9,00	27,00	81,00	40,50	20,30
Variation III [MS]=70 CH:30 FA MS: L=95:5	1,0	kg	1,00	2,00	4,00	8,00	16,00	8,00	4,00
	1,5	kg	1,00	2,50	6,25	15,60	39,60	19,50	9,77
	2,0	kg	1,00	3,00	9,00	27,00	81,00	40,50	20,30

3. Results and discussions

Atterberg Limits test on the original soil, obtained liquid limit (LL) = 54%; plastic limit (PL)=38%; so the plasticity index (PI) = 16%. The shrinkage limit (SL) test resulted in a water content value of 30%.

Table 3. Compressibility coefficient of samples without curing

Samples	MDD (kN/m^3)	Weight (kN/m^3)	Compression coefficient, LIR, (Cc)			Swelling coefficient, LIR, (Cs)		
			1,0	1,5	2,0	1,0	1,5	2,0
Var-I, [80]:20	13,44	15,50	0,685	0,171	0,315	0,016	0,017	0,019
		16,09	0,531	0,454	0,202	0,010	0,021	0,030
		16,58	0,130	0,274	0,334	0,014	0,017	0,018
		16,97	0,090	0,272	0,543	0,009	0,018	0,029
		17,47	0,133	0,266	0,371	0,012	0,018	0,014
Var-II, [75]:25	12,65	14,52	0,159	0,234	0,513	0,007	0,016	0,031
		15,11	0,157	0,238	0,317	0,010	0,016	0019
		15,50	0,080	0,189	0,565	0,006	0,015	0,028
		15,99	0,093	0,180	0,339	0,014	0,015	0,020
		16,58	0,064	0,097	0,223	0,008	0,013	0,032
Var-III, [70]:30	12,46	14,42	0,152	0,236	0,422	0,014	0,016	0,020
		14,81	0,060	0,128	0,350	0,006	0,014	0,026
		15,30	0,095	0,241	0,373	0,018	0,019	0,021
		15,70	0,213	0,118	0,340	0,011	0,005	0,022
		16,28	0,085	0,126	0,242	0,009	0,017	0,010

Tools used to create consolidation test samples are given in figure 1. The consolidation test (Oedometer test) was carried out for 7 days with varying gradual loading and conditions during testing with and without curing for 28 days. The gradual addition of loads was distinguished as an anticipation of the load on the soil in the field due to the addition or construction of different constructions. The additional load is determined by 3 types, namely: LIR = 1.0; LIR=1,50; and LIR=2.0. LIR stands for Load Increment Ratio, which is the ratio of the load to be added to the load that is working on the ground. Table 2 is the loading setting on the consolidated test sample.

Table 4. Compressibility coefficient of samples with curing 28 days

Samples	OMC, (kN/m^3)	Weight, (kN/m^3)	compression coefficient, LIR, (Cc)			swelling coefficient, LIR, (Cs)		
			1,0	1,5	2,0	1,0	1,5	2,0
Var-I, [80]:20	13,44	15,40	0,685	0,171	0,371	0,016	0,017	0,019
		15,89	0,531	0,454	0,543	0,010	0,021	0,030
		16,38	0,130	0,274	0,334	0,014	0,017	0,018
		16,87	0,090	0,272	0,202	0,009	0,018	0,029
		17,56	0,133	0,266	0,315	0,012	0,018	0,014
Var-II, [75]:25	12,65	14,91	0,159	0,234	0,513	0,007	0,016	0,031
		14,91	0,157	0,238	0,317	0,010	0,016	0,019
		15,40	0,080	0,189	0,565	0,006	0,015	0,028
		15,89	0,093	0,180	0,339	0,014	0,015	0,020
		16,28	0,064	0,097	0,223	0,008	0,013	0,032
Var-III, [70]:30	12,46	14,42	0,152	0,236	0,422	0,014	0,016	0,020
		14,81	0,060	0,128	0,350	0,006	0,014	0,026
		15,21	0,095	0,241	0,373	0,018	0,019	0,021
		15,70	0,213	0,118	0,340	0,011	0,005	0,022
		16,28	0,085	0,126	0,242	0,009	0,017	0,010

3.1 Influence of fly ash content

Mixed fly ash in the consolidated sample has 3 (three) variations, namely a fraction of 20%, 25%, and 30% of the original soil before being stabilized with 5% lime. Differences in compressibility index values are presented in figures 3, 4, and 5.

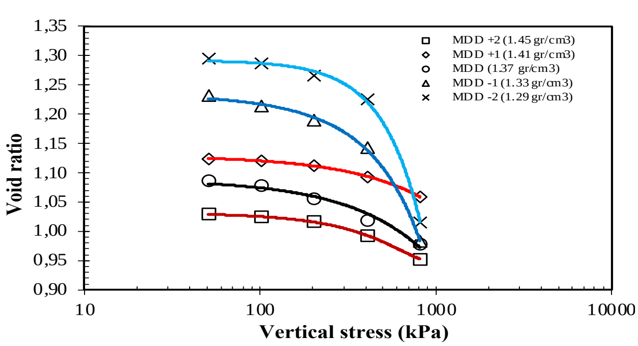


Figure 2. Swelling coefficient, Cs soil variation I, LIR=1,0

Based on figure 3, variation I soil with a density equal to or greater than MDD, the swelling index value has the same value, as seen from the gradient of the parallel line. n soils with a density less than MDD, the value of Cs is greater than in soils with a density greater than MDD.

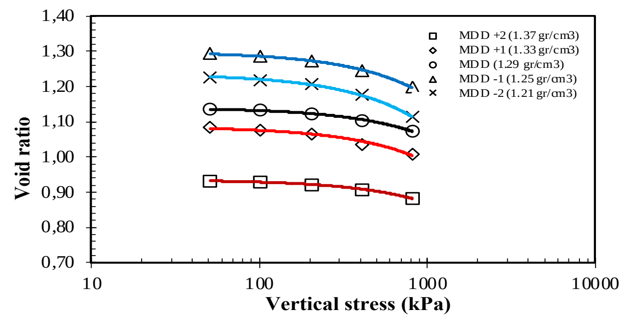


Figure 3. Swelling coefficient, Cs soil variation II, LIR=1,0

Variation II soil with fly ash content of 25% (Figure 4), soil with a density of more than the MDD value has the same gradient and is smaller than the sample at a density below the MDD value. the difference in slope between the soil with a density above and below the MDD is smaller than the soil with a 20% fly ash content (soil variation I).

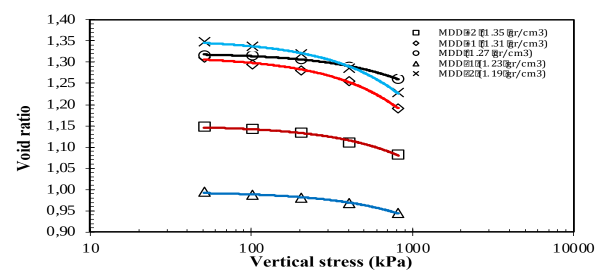


Figure 4. Swelling coefficient, Cs soil variation III, LIR=1,0

Variation III, namely soil with 30% fly ash content, gradient or Cs value in soil with a density below MDD is smaller than soil variation I and variation II. The small swelling coefficient value is shown in Figure 5, where the line is more flat or sloping than the other variations. In Variation II soil with fly ash content of 25% (Figure 4), soil with a density of more than the MDD value has the same gradient and is smaller than the sample at a density below the MDD value. the difference in slope between the soil with a density above and below the MDD is smaller than the soil with a 20% fly ash content (soil variation I).

3.2 Influence of soil density

The denser of soil, the smaller of pore volume, so the change in the pore number is also low. If the soil is getting denser, then the void value before testing (initial void ratio) will be smaller. When several soils with different densities are given the same load, the smallest settlement occurs in the soil with the greatest density. The decrease is a change in the void ratio in the soil due to changes in load, so the decrease in the void ratio due to the addition of the smallest load occurs in the densest soil. This can be explained by observing figure 6.

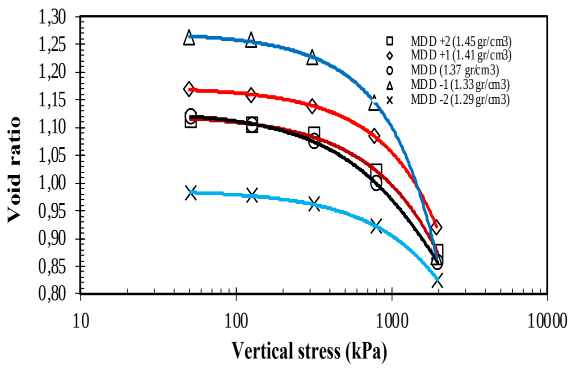
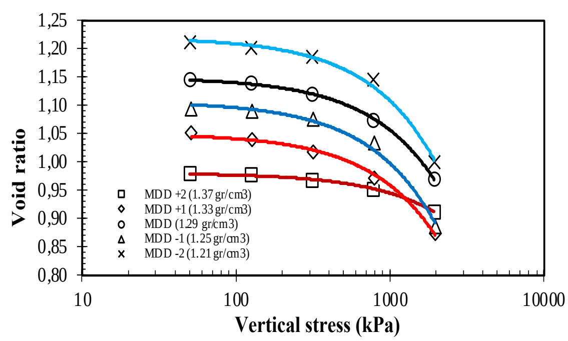


Figure 5. Consolidation Graph for LIR=1,50 on Soil Variation



b) Variasi II, LIR=1,5

Figure 6. Consolidation Graph for LIR=1,5 on Soil Variation

Fig.5 and 6 explain that a lower void ratio ocurred as the soil density increased. If the soil density is higher, the void ratio and curve gradient will also become lower.

3.3 Influence of load addition

A high increment in load can cause the potential for soil to experience consolidation to increase. The settlement is greater when the working load is also greater. A very large load change is possible to create a huge settlement and also cracks.

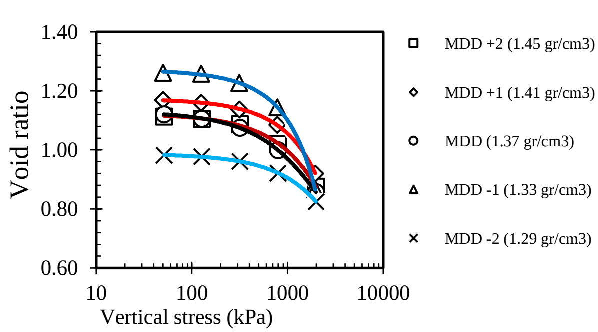


Figure 7. Soil Variation I, LIR=1,5

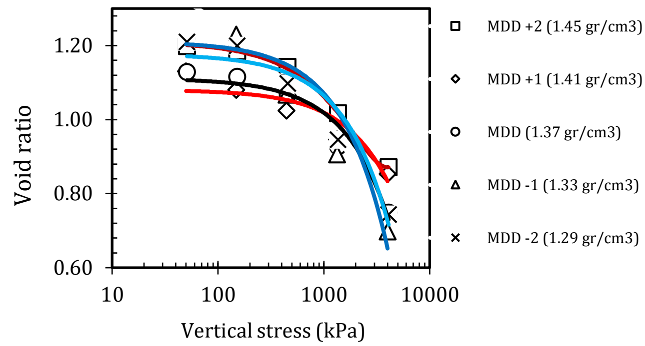


Figure 8. Soil Variation I, LIR=2,0

Based on figures 7 and 8, the addition of a larger load will make the soil more compact. At LIR = 1.50 the smallest void value is around 0.90 while for LIR = 2.0, the void ratio can drop to 0.70. The higher the density of the soil, the more brittle the soil will be so that it is easy to crack when given a large load. At large loads (LIR=2.0), at the highest density, the curve coincides with the soil with the lowest density (Figure 7). This means that there is a possibility that the soil with the highest density has cracked during the loading process.

4. Conclusion

Improvement efforts to achieve a higher density than maximum dry density (MDD) are better than compaction efforts with a density below MDD. The results of the sample consolidation test above the MDD value resulted in the lowest void ratio and compressibility index. On the other hand, a large/excessive compaction effort, which results in a density above MDD, can reduce the bearing capacity of the soil due to over-compacting. It can be seen from the value of the compressibility index of the soil compared to soil with a density below MDD.
The use of fly ash as an additive , the optimum level depends on the content of lime as a stabilizer. Calcium content in lime and fly ash will react with calcium content in the soil to form molecular bonds. A fly ash content of 25% is the optimum mixture so that the soil density and void ratio of the mixture reach the best value.

Authors' contribution

All authors contributed equally to the preparation of this article.

Declaration of competing interest

The authors declare no conflicts of interest.

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