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Improvement of a clayey soil with alkali activated low-calcium fly ash for transport infrastructures applications

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Abstract

The improvement of geotechnical properties is often achieved by the addition of traditional binders, such as cement or lime. However, the use of such binders implies a considerable financial and environmental cost that needs to be mitigated. An unconventional solution, similar to cement in terms of performance but more environmentally friendly, consists in the use of binders made from alkaline activated industrial residues. The technique consists on the activation of raw materials (such as fly ash or blast furnace slag) rich in Si, Al, or even Ca, with high pH alkaline solutions. The present work was developed aiming the possible stabilization, using different fly ash contents, of a clayey soil with sand. The activator solution was composed of sodium hydroxide and sodium silicate. The extended experimental campaign included unconfined compressive strength (UCS), California Bearing Ratio (CBR), pulse velocity tests and triaxial tests to assess the geomechanical improvement induced by the new binder. As a mean of comparison, the experimental campaign included also the stabilization of the same soil with either cement or lime. The obtained data indicates that the use of alkaline activation as a soil stabilization technique provides competitive geomechanical results, when compared with those obtained with traditional binders.

Keywords: soil stabilization, alkaline activation, fly ash, cement binder, lime binder, geomechanical characterization

1. Introduction

During the construction of transport infrastructures, it is common to deal with underperforming soils that do not comply with the mechanical behaviour required for use as pavement foundation. Since the cost and technical difficulties associated with the replacement of these soils by others with better geomechanical quality is high, alternative methods were developed that allow the mechanical improvement of the original, on site soil (Ingles & Metcalf, 1972; Sherwood, 1993; Little, 1995; Little & Nair, 2009). Stabilization with traditional binders, such as cement and/or lime, is one of the most implemented method, when it is desirable to improve geomechanical behaviour, but also to reduce sensitivity to moisture content variation (i.e., shrink and swell) (Petry & Little, 2002; Xing et *al*., 2009). However, cement production is linked to a substantial environmental impact footprint along with the mining of high amounts of raw materials, its subsequent processing, as also to high energy consumptions and detrimental greenhouse gases. Along with the production of 1 ton of cement, approximately 1 ton of carbon dioxide is released (Scrivener & Kirkpatrick, 2008; Provis & van Deventer, 2014). In this context and essentially due to environmental questions, it is mandatory to find new, more sustainable binders, able to replace cement without losses in mechanical effectiveness.

The use of binders obtained from alkaline activation (AA) represents an environmental friendly alternative to Portland cement, since it allows the reutilisation of industrial wastes (Davidovits, 2002; Cristelo et *al*., 2015; Rios et *al*., 2016a; Cristelo et *al*., 2017). For the majority of the industrial residues usually associated with AA, alkaline activation starts with the dissolution of silica (SiO_2) and alumina (Al_2O_3) by a high pH liquid phase. Then, polymerization occurs, where the molecules agglutinate to form larger molecules that precipitate as a gel. Industrial wastes, also named precursors, are solid particles chemically interesting, since they are rich in SiO_2 and Al_2O_3 in an amorphous state (due to the thermal treatment to which they were subjected), making them less stable, and subsequently receptive to chemical reactions capable to produce a new, better organized material. In the case of alkaline activation of fly ash, the rate of dissolution depends strongly on the alkalinity level of the precursor-activator mixture, which is strongly dependent on the activator used (Fernández-Jiménez & Palomo, 2005; Fernández-Jiménez et *al*., 2005). By mixing these binders with the soil, it is intended to improve its mechanical characteristics. The use of the AA technique of fly ash and ground granulated blast furnace slag with sodium silicate solution and/or sodium hydroxide has recently started to be applied in soil stabilisation applications as an alternative to cement (Cristelo et *al*., 2012a; Cristelo et *al*., 2012b; Cristelo et *al*., 2013; Sargent et *al*., 2013; Phummiphan et *al*., 2016; Rios et *al*., 2016a; Rios et *al*., 2016b; Sargent et *al*., 2016; Sharma *et al.,* 2016; Singhi *et al.,* 2016; Rios et *al*., 2017; Rios et *al*., 2018). Other works were also recently published about the performance of differents binders, based mainly on natural pozzolana and lime for ground improvement applications (Harichane et *al*., 2011; Harichane et *al*., 2012; Swaidani et *al*., 2016). However, these studies are still scarce, therefore it is relevant the assessment of the potential of alkali activated low-calcium fly ash in the stabilisation of clayey soils, in the context of transport infraestructures.

An extensive experimental campaign was developed, comprising unconfined compressive strength (UCS) tests, at differents curing times, to mechanically characterize both the original soil and the soil-binder mixtures. Three precursor percentages were defined, and mixed with the same activator content. The influence of the fly ash content was then possible to quantify, both in terms of deformability and strength. Similar tests were performed with traditional binders (cement and lime), each one with 3 distinct binder contents, for a comparative analysis. The most effective mixture of each of the 3 types of binder was selected and subsequently subjected to experimental characterization for improved quantification of deformability and sensitivity to water action, dynamic deformability evolution along time and the evolution of shear strength parameters through California Bearing Ratio (CBR), pulse velocity testing and consolidated undrained triaxial tests (CU), respectively.

2. Experimental program

2.1. Materials characterisation

The soil used throughout the present study was collected in the North of Portugal, near the city of Porto, and was classified as CL - lean clay with sand, according to the Unified Soil Classification (ASTM D 2487, 2006) and as A-4(3), according to the classification of soils for highway construction (ASTM D 3282, 1997). This is a soil with unsatisfactory geomechanical characteristics that need to be improved in a real case scenario, due to its high fine content with a high methylene blue value. Table 1 summarizes all information obtained during this testing phase.

Table 1 - Main geotechnical properties of the soil

Identification test		**Results**		**Units**
Particle size distribution	LNEC E 196 (1966)	Gravel	0.1	%
		Sand	39.2	%
		Clay and silt	60.7	%
Atterberg limits	NP 143 (1969)	Liquidity limit	28	%
		Plasticity limit	19	%
Methylene blue test	NP 933-9 (2002)	*MB*	11.2	g/kg
Specific gravity of soil solids	NP 83 (1965)	*G*	2.54	-
Modified compaction test	LNEC E 197 (1966)	ω_{opt}	14.4	%
		$\rho_{\text{d,máx}}$	1.81	Mg/m^3
California Bearing Ratio	LNEC E 198 (1967)	Unsoaked CBR	48	%
		Soaked CBR	14	%
		ε_{a}†	1.2	%
Oedometric test	ASTM D2435/D2435M, (2011)	λ	$364 \cdot 10^4$	-
		κ	$48 \cdot 10^4$	-
		e_0‡	0.423	-
Unconfined compressive strength	ASTM D 2166 (2000); ASTM D 1633 (2000)	UCS	363	kPa
		ε_{a}	1.2	%
		E_{sec}§	32	MPa

Triaxial (CU)	BS 1377-7 (1990)	c'_{cs}	28	kPa
		φ'_{cs}	17	º
		E_{sec}¶	{15, 20, 32}	MPa

† Expansion after soaking for 96 hours, ‡ void ratio at σ_0' = 1kPa, § secant deformability modulus at $\varepsilon_a = 0.5\%$, ¶ secant deformability modulus at $\varepsilon_a = 0.5\%$ and σ_3 = {50, 100, 200}kPa.

The chosen as the source of alumina and silica resulted from the combustion of coal at the thermo-electric powerplant of Pego (Portugal), and consists of a fly ash type F (ASTM C618, 2012), thoroughly characterized in previous studies (Cristelo et *al*., 2017). The chemical composition of the fly ash was determined by energy-dispersive spectroscopy (EDS) and is given in Table 2. It consists mainly of silica (Si) and alumina (Al), with a combined total of approximately 71%.

Table 2 - Chemical composition of the fly ash

Element	Si	Al	Na	Mg	P	S	K	Ca	Ti	Fe
Fraction (%)	48.81	21.77	1.31	1.56	0.58	1.17	4.42	3.85	1.79	14.74

The activator, in solution form, was one part sodium hydroxide (originally in pellets which were mixed with water to form a 10 molal concentration solution) and two parts sodium silicate (already in solution form with a Na_2O/SiO_2 ratio of 0.5). The traditional binders used comprised Portland cement – CEM II/ B-L 32.5 N – and hydrated lime, containing at least 93% of calcium hydroxide $Ca(OH)_2$.

2.2 Specimen preparation

Table 3 summarises every mixture composition and their respective compaction properties defined from the Proctor tests (LNEC E 197, 1966). The binder percentage was defined in terms of the total solid phase weight. Comparing the results obtained from each of the modified compaction tests with the results obtaind in the soil compaction (Table 1), it was visible that optimum compaction conditions were nearly constant. Thus, the compaction tests was not perfomed over the remaining mixtures, and the dry volumetric mass ρ_d obtained on the compaction test performed on the mixture with the same binder used insted. For the definition of the optimum water content of the liquid phase of the alkaline activated mixtures, several specimens were produced with different water content, but keeping the same dry volumetric mass. By analyzing the consistency of the mixtures and the final aspect of the specimens (existence of surface hollows and surface moisture) it was possible to conclude that the formulation with a water content of 13% in the liquid phase lead to better results from a mechanical point of view.

Table 3 - Identification and characterization of all mixtures tested

Mixt.	**Solid phase**				**Compaction conditions**	
					Liquid phase	
	Soil (%)	Lime (%)	Cem. (%)	Fly ash (%)	ω_A† (%)	ρ_d (Mg/m^3)
Soil	100	0	0	0	14.4	1.81
L5	95	5	0	0	14.2	1.80‡
L7.5	92.5	7.5	0	0	14.2	1.80
L10	90	10	0	0	14.2	1.80‡
C5	95	0	5	0	14.5	1.84‡
C7.5	92.5	0	7.5	0	14.5	1.84
C10	90	0	10	0	14.5	1.84‡
A10	90	0	0	10	13‡	1.80‡
A15	85	0	0	15	13	1.80
A20	80	0	0	20	13‡	1.80‡

† moisture content of the liquid phase of the mixtures

‡ assumed based on performed compaction tests with same binder

To produce each specimen, dry soil was initially mixed with the binder until a homogeneous mixture was obtained. For the mixtures with fly ash, it was necessary to prepare the activator a few hours before the mixing. For that, pellets of sodium hydroxide were mixed with water to form a 10 molal concentration solution. Since the reaction is highly exothermic, it was required to allow for cooling for at least one day in a closed container. The activator was then obtained by adding two parts of silicate and one part of hydroxide. The liquid phase (tap water or activator) was then added to the soil with a moisture content of ω_{opt} if used on traditional binders, or ω_A if used on fly ash, and further mixing was applied. The resulting stabilised soil was statically compacted in three layers, inside a cylindrical stainless steel mold with 70 mm of diameter and 140 mm height, to obtain the desired unit weight, as defined in ASTM D1632 (2007). After 48 hours, the specimens were removed from the mold and wrapped in cling film, after which they were stored again in a humid chamber at 20ºC and 95% of relative humidity.

2.3 Testing procedures

Curing periods of 7, 14, 28 and 90 days were considered, after which UCS tests were performed (ASTM D 2166, 2000; ASTM D 1633, 2000). For reproducibility reasons, each UCS result is the average of three tested specimens. The tests were carried out under monotonic displacement control, at a rate of 0.18 mm/min. Following the UCS tests, and for each binder used, the mixture with the best geomechanical behaviour was selected for CBR (LNEC E 198, 1967) and pulse velocity testing (ASTM C 597, 2002). CBR tests were performed after a curing period of 28 days, using 2 load plates of 2.5 kg, under soaked and unsoaked conditions. P-wave velocity was measured after several curing periods, up until 189 days. Five recordings were obtained on each measurement, and the average value was taken as the final result. Only P-waves were recorded due to limitations of the ultrasound testing equipment. Finally, consolidated undrained triaxial tests (BS 1377-7, 1990) were performed to provide a comparasion between the shear behaviour of soil mixtures with alkali activated fly ash (after 28 curing days) with that of the original soil.

3. Results and discussion

3.1. Unconfined compressive strength

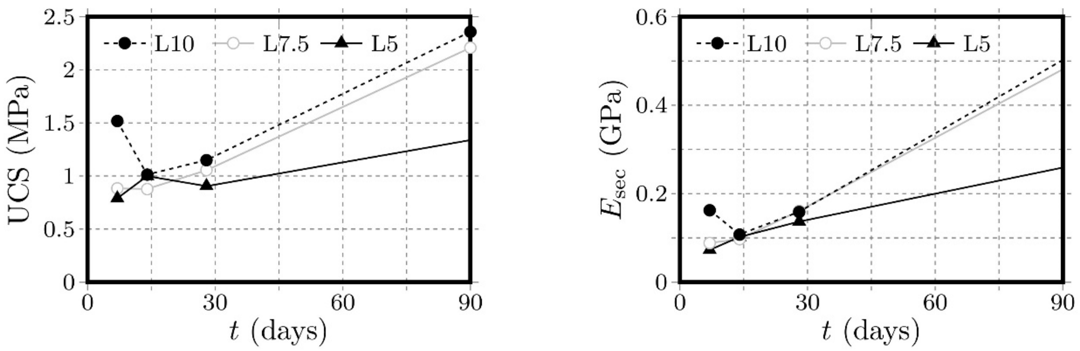
Table 4 summarizes the mean peak UCS, the strain at failure ε_r and the secant deformability modulus E_{sec} (for $\varepsilon_a = 0.5\%$). Most results provided in this table, except for the mixtures with fly ash, produced low coefficients variation values (UCS ≤ 12.6% and $E_{sec} \leq$ 24.6%), which is a good indicator of the quality of the data obtained. The mixtures with fly ash produced the highest coefficients of variation (UCS ≤ 20.0%, $\varepsilon_r \leq 33.6\%$ and $E_{sec} \leq 39.1\%$), which is probably related with the increased difficulty, compared with the cement and lime-based formulations, in achieving homogenous mixtures.

Table 4 - Mean peak unconfined peak strength UCS, strain at failure ε_r and secant deformability modulus E_{sec} at 7, 14, 28, 90 curing days.

Mixt.	7 days			14 days			28 days			90 days		
	UCS	ε_r	E_{sec}	UCS	ε_r	E_{sec}	UCS	ε_r	E_{sec}	UCS	ε_r	E_{sec}
L5	0.78	8.68	73	1.00	9.95	102	0.90	8.20	138	1.35	7.29	259
L7.5	0.88	9.00	88	0.87	7.89	96	1.06	8.36	159	2.22	6.86	481
L10	1.53	8.93	162	1.02	9.48	108	1.15	8.86	160	2.37	6.62	504
C5	2.22	7.59	456	2.52	8.52	443	2.98	7.35	601	3.60	7.89	701
C7.5	2.36	7.63	499	3.77	8.21	733	3.82	7.28	849	5.25	7.03	1171
C10	3.41	8.02	682	4.51	7.88	895	5.98	7.58	1314	7.20	7.45	1317
A10	0.96	13.6	136	1.16	9.58	188	1.51	5.15	447	2.94	4.47	894
A15	1.09	11.0	197	1.36	8.51	301	2.41	5.56	629	4.52	5.59	1166
A20	1.09	11.9	159	1.44	7.09	307	3.21	5.76	767	8.57	5.41	2460

Note: UCS and E_{sec} in megapascal, ε_r in permillage.

Figure 1 shows the evolution, as a function of time, of the mean values of the UCS and E_{sec} for all lime-based mixtures. It is clear that, after curing for 7 days, the mixtures with lime achieve strength gains, compared with the reference value established by the unstabilised soil, higher than 200% (5% lime), and approximately 400% (10% lime). This rapid initial strength increase is due to the well-known flocculation of the soil (i.e. aggregation of the clay particles of the soil in flakes), which enables the initial clay to behave like a sandy material, with an improved friction angle. It is therefore not related with the long-term strength gain, which is due to the pozzolanic reactions between the clay minerals and the calcium (Cristelo et *al*., 2009). From 7 to 14 curing days, the deformability modulus E_{sec} of mixtures L5 and L7.5 increased by 40% and 10%, relatively to the values obtained after 7 days, while the E_{sec} of mixture L19 decreased by 34%. With respect to strength, mixture L5 achieved a gain of 28%, while mixture L7.5 strength practically did not change and mixture L10 showed a strength decrease of 34%. This fickleness of the evolution of E_{sec} and UCS was expected, as reported in other case studies found in literature (Duvigneaud, 2008; Neves, 2009; Amaral et *al*., 2011). Evaluating the development from 14 to 28 curing days, all mixtures achieved significant strength gains, of 35%, 65% and 48%, for mixtures L5, L7.5 and L10, respectively. After 28 days, all mixtures showed a UCS value around 1 MPa. At 28 curing days, L5 reached E_{sec} = 138 MPa, while L7.5 and L10 achieved $E_{sec} \approx$ 160 MPa. Resuming, up to 28 curing days, concerning the evolution of strength and stiffness, it was noticeable that even if it was function of the mixture, they all ended showing similar results. From 7 to 28 curing days (i.e., after the initial strength gain due to flocculation) no relevant increase in strength and stiffness was recorded, as a consequence of an ‘induction’ period in which the dissolution of the Si and AL from the soil was occurring. During this period, cores of hydrated calcium silicate began to appear on the clay particles contact points, but since the quantity of completed reactions is still low, they are unable to induce a noteworthy improvement in the mixture behaviour (Cristelo et *al*., 2009). The most important increase in strength and stiffness was observed between 28 to 90 curing days, with mixtures L5, L7.5 and L10, offering strength increments of 49%, 110% and 106%, respectively. At 90 days, L5 provided UCS = 1.3 MPa and E_{sec} = 259 MPa, while L7.5 and L10 achieved E_{sec} = 480 MPa and UCS = 2.2 MPa and E_{sec} = 504 MPa and UCS = 2.37 MPa, respectively. Comparing the mixtures’ behaviour (strength and stiffness), after 90 curing days, with that of the original soil, it is noticeable a gain in L5 between 4x to 8x, while in L7.5 and L10 this gain rised to 6x to 15x. These results are in accordance with published studies showing that higher lime contents induce higher gains (in strength and stiffness) (Cristelo et *al*., 2009). However, a limit lime content value around 8% was found, since no gain was obtained between 8% and 10%. This is a well-known behaviour, and results from the fact that no more pozzolan (clayey soil) is available to react with the excessive lime, which in turn becomes more prone to react with the carbon dioxide and form Ca carbonates. This should be avoided, as these $CaCO_3$ compounds constiture week points in the mixture.

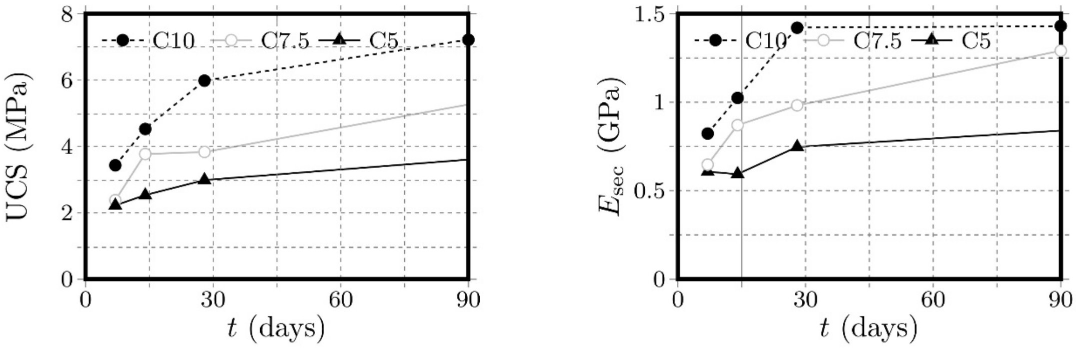


(a) UCS (b) E_{sec}

Figure 1 – UCS (MPa) and E_{sec} (GPa) evolution with the curing time in mixtures with lime

Figure 2 shows the evolution, as function of time, of the mean values of the UCS and E_{sec} for all cement-based mixtures. Between 7 and 14 curing days, all mixtures deliver strength increments, 11% for C5, 59% for C7.5 and 32% for C10. Regarding the deformability modulus E_{sec}, it did not change on C5, but increases 47% and 31% in C7.5 and C10, respectively. From 14 to 28 curing days, mixtures C5, C7.5 and C10 achieved gains in strength of 18%, 1% and 32%, and in stiffness of 36%, 16% e 47%. The highest values at 28 curing days were reached by C10, with UCS ≈ 6 MPa and E_{sec} = 1.3 GPa. Between 28 and 90 curing days the development of strength and stiffness was quite weak, with the highest values achieved by C7.5 with an increase of 37% in strength and 38% in stiffness. Summing up, a different trend was observed when compared with lime mixtures. Now more than 80% of the total strength gains appear during the first 28 curing days (as expected), and the remaining increase up to 90 curing days. Also, at 28 curing days, the higher cement content clearly produced the higher strength (100%

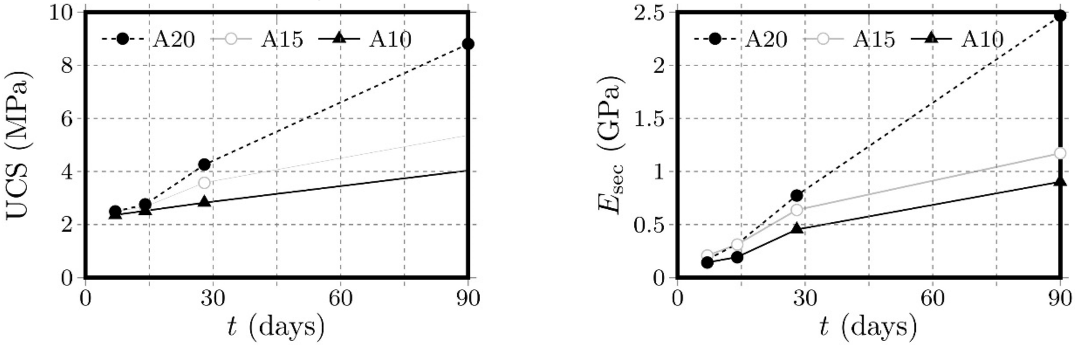
and 37% increase between 5% and 10% and between 7.5% and 10%, respectively), contrary to what was observed with the lime, in which case the strength difference between the 5% and 10% contents and between 7.5% and 10% was 76% and 7%, respectively. Comparing the mixtures behaviour (strength and stiffness) at 90 curing days with that of the original soil, it is noticeable a gain in C5 between 10x to 22x for strength and stiffness, while in C7.5 and C10 this gain is among 15x to 19x for strength and 36x to 41x in stiffness.



(a) UCS (b) E_{sec}

Figure 2 - UCS (MPa) and E$_{\text{sec}}$ (GPa) evolution with the curing time in mixtures with cement

Figure 3 shows the evolution, as a function of time, of the mean values of the UCS and E_{sec} for all mixtures that use alkaline activated fly ash as binder. After 7 curing days, all mixtures achieved UCS ≈ 1 MPa and E_{sec} ≈ 150 MPa. From 7 to 14 curing days, strength gains of 21%, 25% and 32%, and stiffness gains of 38%, 53% and 93%, were registered for mixtures A10, A15 and A20, respectively. At 14 days, it became possible to identify some differences between the mechanical behaviour of the mixtures (especially in terms of stiffness), since A10 reaches UCS = 1.2 MPa and E_{sec} = 189 MPa, and the remaining mixtures UCS ≈ 1.4 MPa and E_{sec} ≈ 300 MPa underlining the influence of the ash content in the mixtures. Between 14 and 28 curing days, gains of 30%, 77% and 123% (strength) and 137%, 109% and 150% (stiffness), in mixtures A10, A15 and A20, respectively are observed. At 28 curing days, the most competent mixture was A20, with UCS = 3.2 MPa and E_{sec} = 67 MPa. Performing a comparative analysis between the mechanical behaviour of all mixtures with 28 curing days, it is noticeable that fly ash provided better results than those obtained with lime, but lower than those obtained with cement. However, the importance of the mechanical behaviour at this stage should not be overestimated, since the binder in soil-lime and soil-fly ash mixtures induces substantial mechanical improvement far behond 28 curing days. In opposition, with cement, and after 28 curing days, the strength is around 85% to 90% of its maximum value. After 90 curing days, A10 and A15 achieved gains in strength and stiffness around 90% in relation to the 28 days, while A20 achieved a strength gain of 167% and a stiffness gain of 221%. Like soil-lime mixtures, soil-fly ash mixtures presented a substantial strength increase between the 28th and the 90th curing day. In soil-lime mixtures the slow strength development is due to the later (after 28 days) surge of pozzolanic reactions, while in soil-fly ash mixtures the low Ca content results in the formation of N-A-S-H type gel, with a slower development than the C-S-H gel obtained in soil-cement mixtures. Comparing the mixtures behaviour (strength and stiffness) after 90 curing days with that of the original soil, it is noticeable a gain in A10 and A15 between 8x to 12x for strength and 28x to 36x in stiffness, while in A20 this gain is equal to 23x for strength and 76x for stiffness. The quantity of used fly ash has a more significant influence on the mechanical behaviour than with the remaining binders tested, namely because lime as an optimum content value, after which the carbonation of lime consumed in the reactions starts to decrease the compressive strength. To conclude, mixture A20 presented the best mechanical performance at 90 curing days, with a stiffness of 2.5 GPa (nearly the double of the stiffness of mixture A10 at the same time), with the lime-based mixtures showing the lower results.



(a) UCS

(b) E_{sec}

Figure 3 - UCS (MPa) and E_{sec} (GPa) evolution with the curing time in mixtures with fly ash

3.2 Second stage of the experimental program

Based on the mechanical behaviour derived from the UCS tests, the original soil and mixtures L10, C10, A15 and A20 were selected for additional characterization, namely seismic wave velocity evolution along time, CBR testing (except for mixture A15) and triaxial tests (only for the original soil and mixture A15).

3.2.1 CBR *testing*

Since CBR tests are used as standard on pavement design, they were used to quantify the improvement achieved over the original soil, stabilized with distinct binders. Tests were performed according to a conventional standard (LNEC E 198, 1967) that requires the pre-soaking of the sample for a period of 96 hours. However, unlike the original soil, none of the soil-binder samples presented swelling during this stage. Consequently, it is assumed that the binder is able to mitigate this undesirable effect. Table 5 summarizes the CBR results obtained with the mixtures selected for the second stage of the experimental campaign. From these results, it is visible that the use of binders induces higher values of CBR, but the increment is highly variable. The most efficient mixture was the C10, with a strength gain of approximately 6x, followed by the A20, with a gain of approximately 5x, and by the L10, which doubled the

CBR value of the original unsoaked soil. Regarding the soaked results, it is perceptible a high reduction (around 70%) of the CBR value of the original soil, compared with the unsoaked value. This reduction was smaller on the stabilized samples, with a maximum of only 14% (mixture L10).

Table 5 - CBR results after 28 curing days

CBR test	Mixture			
	Soil	L10	C10	A20
Unsoaked	48%	92%	318%	269%
Soaked	14%	79%	283%	248%

3.2.2 Pulse velocity testing

Pulse wave tests were performed according to ASTM C597 (2002), on different specimens, in order to study the evolution, with time, of the dynamic deformability modulus, E_0, that was obtained from:

$$v_p = \frac{h}{t} = \sqrt{\frac{E0(1-\mu)}{\rho\,(1+\mu)(1-2\mu)}} \qquad (1)$$

where v_p is the p-wave velocity, ρ is the volumetric weight of the sample, μ is the dynamic Poisson's coefficient, *h* is the sample height and *t* is the time the p-wave takes to travel throughout the sample. Wave propagation time *t* was measured after every 12 hours, until 7 curing days; after every 24 hours between 7 and 24 curing days; twice a week between 28 and 100 curing days; and monthly until 189 days of curing. These *t* values allowed the quantification of the evolution of the dynamic deformability modulus given in Figure 4, with μ being assumed equal to 0.25 (Amaral et *al*., 2011). This value of μ was assumed equal in all samples since it is valid range is small and its exact values are unknown. The value of E_0 the original soil is 0.6 GPa.

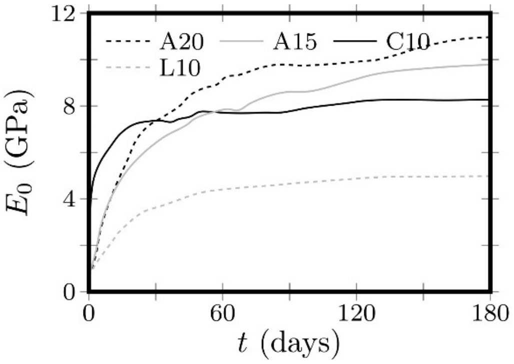


Figure 4 - Evolution of E_0 with curing time

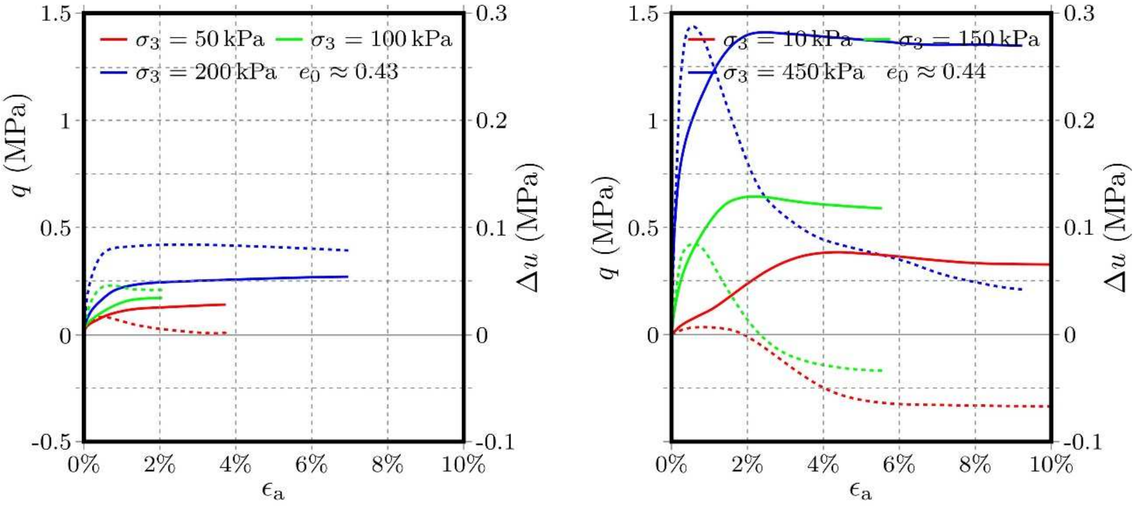
Initially (i.e. $t = 0$), mixture C10 had a dynamic deformability modulus of 4.2 GPa, nearly 5x higher than the 0.9 GPa found in the remaining mixtures (L10, A15 and A20). Up to 7 curing days, the mixtures with fly ash achieved similar values of E_0, but above this period the higher ash content of mixture A20 becomes noticeable, particularly after 13 curing days. At 28 curing days mixtures L10, C10, A15 and A20 reached E_0 values of 3.6 GPa, 7.3 GPa, 6.5 GPa and 7.4 GPa, respectively. After this curing period, it occurs a stabilization of the dynamic deformability modulus in mixture C10, however not present in the remaining mixtures. Mixture A20 achieved the same stiffness of C10 mixture at 28 curing days, while mixture A15 required 60 days. Both these fly ash mixtures show increasing E_0 up to 189 curing days. At the final day of P-wave measurements, the following E_0 values were obtained: 5.0 GPa for L10, 8.2 GPa for C10, 9.8 GPa for A15, and 10.9 GPa for A20.

Comparing the stiffness obtained in the pulse velocity testing with that obtained in the UCS tests, it is possible to conclude that they are coherent and follow the same trend for all the soil-binder mixtures under analysis. Both tests shown that the smallest stiffness increase occurred in the soil improved with lime. After 28 days of curing there was an irrelevant increase in stiffness in the soil-cement mixture. In fact, for the C10 mixture and after 28 days, the UCS tests showed a practically constant stiffness ($E_{\mathrm{sec}} = 1.3$ GPa) up to 90 days, and the pulse velocity tests, at 28 and 90 days, $E_0 = 7.3$ GPa and $E_0 = 8.2$ GPa respectively, which translates into a slight increase of 12% of the E_0 value. However, contrary to the cement mixtures, it is evident a fairly significant increase of stiffness in all alkaline activated mixtures in the same period. At 28 and 90 days of cure, the UCS tests performed over A20 mixture showed, respectively, $E_{\mathrm{sec}} = 0.8$ GPa and $E_{\mathrm{sec}} = 2.5$ GPa, which represents an increase of 225%, while the pulse velocity testing showed $E_0 = 7.4$ GPa and $E_0 = 10.9$ GPa, which represents an increase of 47%.

Summing-up, the lime mixture is clearly the least effective one, the evolution rate of the cement is quite high until 28 curing days and practically stabilizes after that, and fly ash mixtures reveal a significant E_0 value increase up until day 189. These general behaviours are due to the effect of a C-H-S type gel in soil-cement mixtures (known for its rapid development), and the effect of secondary chemical reactions in the remaining mixtures (i.e., the crystallization of the initial zeolitic gel in ash-based mixtures and the pozzolanic reactions in lime-based mixtures), requiring longer curing processes to develop their full potential.

3.2.3 Triaxial testing

Consolidated undrained triaxial tests were performed to provide a comparison between the shear behavior of mixture A15 (after 28 curing days) with that of the original soil. Undrained tests were selected since they simultaneously allow for effective and total stress analysis, while consuming less time than drained tests. Three tests were performed, in the original and improved soil, in order to obtain effective (c' and φ') and total (c and φ) strength parameters. For the unstabilised soil test, the confinement stresses were 50 kPa, 100 kPa and 200 kPa, as proposed by BS 1377-7 (1990), while values of 10 kPa, 150 kPa and 450 kPa were used for mixture A15, to account for a possible higher dispersion of the results induced by the stabilization process and the higher strengths achieved. Figures 5 summarises the obtained results, which allowed the definition of the strength parameters given in Table 6.



(a) Original soil

(b) Mixture A15

Figure 5 - Consolidated undrained triaxial tests performed after 28 curing days (q evolution indicated with solid lines and Δu with dash lines)

Table 6 - Shear strength parameters from consolidated undrained triaxial tests

Mixt.	Total stress				Effective stress			
	Peak		Critical state		Peak		Critical state	
	c_p	φ_p	c_{cs}	φ_{cs}	c_p	φ_p	c_{cs}	φ_{cs}
Soil	Not present		25 kPa	19º	Not present		17 kPa	28º
A15	88 kPa	33º	75 kPa	33º	27 kPa	47º	7 kPa	42º

Stabilisation of the original soil produced, as shown in Figure 5, and in accordance with other authors (Rios & Viana da Fonseca, 2013; Rios et *al*., 2016b), a peak resistance, which was not present in the original soil. Nevertheless, the major influence of the fly ash binder was the substantial increment of the critical state friction angle, rising 50% in effective stress analysis and 73% in total stress analysis. Also, cohesion parameters of mixture A15 raised 200% in terms of total peak strength (due to the bounds created between the soil particles). In effective stress, as expected, both values are very low. Regarding pore pressure evolution, the behaviour shown by the A15 tests is quite distinct than that of the original soil, with the former showing a significant decline shortly before the peak strength. This is due to the more fragile response of the stabilised specimens, which suffered a sudden volume increase as soon as the failure surface started to form. At $\varepsilon_a = 0.5\%$, it is quite visible the impact on stiffness of the fly ash binder, which increased with the confinement values, reaching 13 MPa, 72 MPa and 192 MPa for 10, 150, 450 kPa, respectively, comparatively to the 15 MPa, 20 MPa and 32 MPa obtained on the original soil for confinement values of 50, 100, 200 kPa. Figure 6 illustrates the failure surface of the original soil (with a barrel shape) and mixture A15 (with a failure surface).

(a) Original soil

(b) Mixture A15

Figure 6 - Triaxial failure surface of the original soil and mixture A15

4. Conclusions

This paper compares the behaviour of a clayey soil improved with alkaline activated low calcium fly ash with the same soil improved with traditional binders, namely lime and cement. The basic liquid phase (i.e. the activator) was maintained constant on all ash mixtures so that the only variable would be the binder content. This allowed the interpretation of the binder content influence with respect to the soil-binder strength and stiffness. A thorough campaign of unconfined compressive strength tests was performed at 7, 14, 28 and 90 curing days, showing that:

- the addition of a binder to the soil (independently of the binder) induced higher stiffness and strength and a fragile post-peak response;
- similarly to what is normally verified with the addition of traditional binders, the use of higher contents of fly ash also induces higher strength and stiffness;
- the cement induced a faster increase of the mechanical characteristics up to 28 curing days, but after this initial phase the evolution became very slow;
- in the lime mixtures, the increments of strength and stiffness were lower than in the soil-cement and soil- ash mixtures during the 90 curing days. However, significant increases in strength and stiffness were observed between 28 and 90 days of cure;
- ash mixtures show a slower increase in the mechanical characteristics from 0 to 28 curing days than that of cement mixtures. However, at 90 curing days, the ash mixtures provided similar or even higher strength and stiffness than cement mixtures.

Besides the UCS tests, CBR, pulse velocity and triaxial tests were also performed on selected mixtures. From these tests the following conclusions were also drawn:

- all binders induced a significant increase in the CBR values and a strong reduction of sensitivity to water action (i.e., swelling). Soaking the samples induced a CBR reduction in the original soil of around 70%, but quite smaller on the stabilized samples in which the maximum reduction (recorded on a lime mixture) was only 14%;

- pulse velocity tests confirmed that, at 28 curing days, cement mixtures provide the highest stiffness among all mixtures, but before 90 curing days all ash mixtures overcome the stiffness of the cement mixture;
- triaxial tests performed at 28 curing days confirmed the high mechanical improvement of all shear strength parameters and also of the stiffness.

Summing up, it is possible to conclude that the application of the alkaline activation technique of fly ash in the improvement of a soil revealed very interesting results from a mechanical point of view, superior to the results observed in the soil-cement mixtures at 90 days of age and in the soil-lime mixture at all curing time under analysis.

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