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Effect of Zeolite Utilization on Volume Change and Strength Properties of Expansive Soil as Landfill Barrier

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Abstract

Sand-Na-bentonite mixture is widely used as engineering barrier material which usually possesses hydraulic conductivity below the regulatory limit (10^{-7} cm/s). However, in some areas natural Na-bentonite is not easily available, instead an abundantly prevailing local expansive soil can be an alternative. This study assesses the suitability of a local expansive soil mixed with zeolite, readily obtained from natural reserves in Turkey, to be proposed as a landfill liner in a semi-arid climate. The choice of zeolite is due to its already well understood high adsorption capacity for heavy metals as well as its pozzolanicity. The volume change, strength, and hydraulic conductivity characteristics were studied with the effect of durability through ageing. When expansive soil to zeolite ratio of 0.5 was used, the results indicated improved properties with curing. Swell potential was observed to decrease by 85% within 90-day curing period, while 30-34% reduction was noted in shrinkage and compressibility characteristics. The hydraulic conductivity was observed to remain below the regulatory limit under all confining pressure ranges studied, provided the curing time was at least 90 days. Moreover, the mixture attained improved strength characteristics with time, and proved to be sustainable over the period studied. Therefore, it was concluded that expansive soil mixed with zeolite could be a good alternative to sand-Na-bentonite, mainly in developing areas of growing population and environmental degradation.

Keywords: Expansive soil, zeolite, volume change, strength

Introduction

Wastes which include municipal industrial, mining, nuclear and packaging by-products accumulating in landfill areas generate contamination, imposing detrimental health and environmental problems. Landfill barriers, liners and covers are widely used in the waste containment facilities to prevent ingress of precipitation as well as seepage of leachate to the environment and underground water. Compacted clays which have been conventionally used as liners, are subjected to desiccation crack formation which increases the hydraulic conductivity. Cracks occurring in clayey landfill liner which cannot be closed disrupt the long-term performance of landfills, hence only suitable in areas where temperature and moisture variations are insignificant. In developing countries, cost-effective liners can be produced using locally available natural materials. Performance of landfill liners is influenced by desiccation cracks, which form preferential flow paths upon rewetting, hence increasing hydraulic conductivity. Therefore, clayey soils have disadvantage when moisture fluctuations are erratic. Daniel and Wu (1993) stated that compaction of clay liners on the wet side of the optimum yields lower hydraulic conductivity values, however this might cause a high potential for shrinkage during drying. Therefore, especially under low overburden stresses rewetting of the compacted soil might not cause self-healing, particularly important in the final cover systems, whereas higher overburden effect will constrain crack formation and propagation in the liners.

In recent studies, sand-bentonite mixtures are commonly used as barrier materials in different percentages of sand and bentonite. Bentonite, which is montmorillonitic clay, has low permeability and high plasticity, however experiences tremendous cracking upon desiccation which increases during drying-wetting cycles. The sand component of a compacted bentonite/sand mixture contributes to the strength whereas the bentonite component fills the

pore space between the sand grains in order to reduce the hydraulic conductivity (Akgün et al. 2006). The most important parameter required in the design of barriers is the saturated hydraulic conductivity, and according to regulations the compacted barriers should possess a value lower than 10^{-7} cm/s (Landfill Technical Guidance Manual 1997).

Recently zeolite is considered as an innovative landfill material due to its filtering properties as well as low volumetric shrinkage capacity. It was observed that bentonite-zeolite mixtures are not influenced by moisture content variations and possess very low hydraulic conductivity as well as low volumetric shrinkage potential. Bentonite-embedded zeolite (BEZ) was proposed as an alternative to bentonite-embedded sand (BES), which satisfies lower hydraulic conductivity values when compared to BES and geosynthetic clay liners. In addition, tests showed that the shrinkage behaviour of BEZ is satisfactory for landfill liner applications (Kaya et al. 2006). BEZ liners are superior to BES liners because of the high adsorption capacity of zeolite, therefore; it would appear to be more advantageous to use BEZ instead of BES in certain hydraulic barrier system applications, such as hydraulic landfill liners and covers (Kaya and Durukan 2004).

The objective of this article is to examine the volume change and strength characteristics of expansive soil and zeolite (clinoptilolite) mixture within curing periods of up to 90 days, in order to assess the pozzolanic strength of zeolite and its suitability as a landfill material in a semi-arid climate. Since the requirement of a good landfill liner material is its durability, low hydraulic conductivity, low volume change and high strength under various environmental conditions, testing program was aimed to study the effect of time on swell-shrinkage,

hydraulic conductivity, compressibility, unconfined compressive strength and flexural strength.

Background on Zeolite

Natural zeolites are derived from acidic and volcanic tuffs, which occur in the form of rock forming minerals in many parts of the world. They are commonly found in saline alkaline lake deposits, vertical sedimentary sequences, hydrothermally altered volcanic or sedimentary rocks and deep-sea sediments (Bish and Guthrie 1994). There are five types of naturally occurring zeolites found in abundance in the form of brittle, solid rocks, and are of commercial interest due to their favorable ion exchange properties. These are clinoptilolite, erionite, chabazite, mordenite and phillipsite. Zeolites are composed of tecto-silicates with open framework, consisting of cages and tunnels which adsorb smaller molecules, like a “molecular sieve”. Therefore, they possess high adsorption capacity and can act as a filter in the event of leachate through liner. Mainly the clinoptilolite type has a selectivity to heavy metals. Pollutant adsorption capacity and heavy metal removal properties of zeolites are investigated by many researchers indicating the ability to absorb and trap heavy metals present in the leachate, such as Pb^{2+} , Zn^{2+} , Cd^{2+} , Ni^{2+} , Fe^{2+} and Mn^{2+} (Kayabalı, 1997; Kayabalı and Kezer 1998; Jacobs and Förstner 1999; Langella et al. 2000; Cincotti et al. 2001; Mier et al. 2001; Tuncan et al. 2003; Erdem et al. 2004; Ören and Kaya 2006; Motsi et al. 2009; Misaelides 2011).

Yükselen-Aksoy (2010) focused on the swelling potential, compressibility, hydraulic conductivity and shear strength properties of two different natural zeolites in order to use in geotechnical and geoenvironmental applications. Test results showed that compressibility and

swelling potential of zeolites were not high and hydraulic conductivity was suitable for the limitation of landfill liners and can be used for these applications. Ören and Özdamar (2013) analysed the effect of zeolite particle size and compaction water content on the hydraulic conductivity behaviour. They concluded that when the compaction water content increased the hydraulic conductivity of compacted zeolite reduced and compacted granular zeolites have higher hydraulic conductivities than compacted fine zeolites.

Zeolites are suitable to be used in combination with bentonite as hydraulic properties of natural zeolite can be adjusted to meet particular requirements by adjusting the grain size through crushing and sieving process (Kaya and Durukan 2004). Kaya et al. (2006) studied the suitability of using BEZ as alternative to BES and concluded that when zeolite was used volumetric shrinkage reduced, indicating that the mixture was not susceptible to moisture fluctuations, and the hydraulic conductivity was in the order of 10^{-10} cm/s. Galvão et al. (2008) compared compaction characteristics, volumetric shrinkage strain and hydraulic conductivity of bentonite embedded sand and bentonite embedded zeolite and they suggested that BEZ is a good material for a landfill liner. Kayabalı (1997) investigated the use of a bentonite-zeolite layer beneath clay liner and concluded that this would reduce the thickness of clay liner by 25%, and would also act as a chemical filter.

Pozzolanic activity of zeolites are investigated by various researchers who studied the chemical, mechanical and microstructural properties of clinoptilolite blended cements and concluded that zeolitic tuffs are excellent pozzolans, and that silica rich zeolites react faster than aluminum rich ones (Yılmaz et al. 2007; Caputo et al. 2008; Mertens et al. 2009; Demirbaş 2009; Uzal et al. 2010; Özen 2013). Pozzolans are siliceous and aluminous

materials, which in themselves possess little or no cementitious property, but they are able to combine with calcium hydroxide ($\text{Ca}(\text{OH})_2$) at ordinary temperatures and in the presence of water to form new reaction compounds possessing binding character.

Thus natural zeolites, which are hydrated aluminosilicates containing high amounts of SiO_2 and Al_2O_3 , exhibit pozzolanic behaviour. This characteristic is attributed to the large surface area available for interaction with lime, due to the porous structure of zeolites, and their ability for ion exchange. Pozzolanic process in zeolites is rather complex and is influenced by the surface area, particle size, Si/Al ratio, cation exchange capacity (CEC), and mineralogy. In the presence of water and lime hydration process results in rapid release of alkali-earth cations (AlOH_4^-) followed by formation of an unstable structure further dissolution of $\text{Ca}(\text{OH})_2$ into Ca^{2+} and OH^- ions resulting in a high pH (Zhang et al. 1995). The AlOH_4^- and Ca^{2+} react to form hydrated calcium aluminate (CAH), and the process continues with replacement of O^- by OH^- which yields the break down of Si-OH-Si bonds in the structure, resulting in the formation of hydrated calcium silicates (CSH) which form on the surface of zeolites, if the rate of formation of CSH is higher than the rate of dissolution of SiOH. Zeolitic tuff-CaO reactions initially produce a layer of unstable CSH gel on the surface of zeolite, after a series of diffusion controlled topochemical reactions, which transforms into tobermorite as a result of heterogeneous nucleation on the solid-liquid interface. Mehta and Monteiro (2006), Martinez-Ramirez et al. (2006) also stated that the main hydration product of the pozzolanic activity is calcium silicate hydrate (CSH).

Materials and Methods

Materials

Expansive soil was collected from Nicosia, Cyprus, and zeolite was obtained from Ege Zeolit Ltd., Bigadiç, Turkey. Physical properties of the two materials, including Atterberg limits (ASTM D4318-10e1), specific gravity (ASTM D854-14), linear shrinkage (BS 1377-2:90) and particle size distribution (ASTM D422-63(2007)e2) are indicated in Table 1. The expansive soil is constituted of mostly silt with appreciable amount of clay, and is classified as silt with high plasticity (MH) according to the Unified Soil Classification System (ASTM D2487-11). Zeolite contains almost equal amounts of sand and silt sized particles, with a small amount of clay sized fraction. The chemical composition detected by X-ray fluorescence (XRF) spectrometer is presented in Table 2. Hence the zeolite used in this study consists of a total percentage of SiO_2 , Fe_2O_3 , and Al_2O_3 of more than 70%, which is the requirement for a material to be considered as a pozzolan (ASTM C618-15).

Table 1. Physical properties of materials.

Table 2. Chemical properties of materials.

X-ray diffraction (XRD) analysis showing the mineralogical composition of soils used in this study are as represented in Figure 1 (a) and (b). Expansive soil includes kaolinite, illite and chlorite which are clay minerals, as well as non-clay minerals of feldspar and calcite. Zeolite contains clinoptilolite, quartz, montmorillonite minerals and minor amounts of illite and feldspar.

Figure 1. X-ray diffraction pattern of (a) expansive soil, (b) zeolite.

The samples were tested in five different categories of expansive soil and zeolite. First group is expansive soil (N) and the second group is 50% expansive soil and 50% zeolite (NZ). Different curing time periods were applied to NZ specimens (0, 7, 28 and 90-days).

Therefore, laboratory tests were performed on five groups which are N, NZ (0-d), NZ (7-d), NZ (28-d) and NZ (90-d).

Methods

Initially, expansive soil and zeolite were dried at 50°C in the oven and pulverized to particle size finer than 425 µm. Then, optimum water content and maximum dry density values were determined by standard Proctor energy (ASTM D698-12e2) for N and NZ groups. Soil specimens used in swell, consolidation, shrinkage and unconfined compression tests, were mixed with the optimum water content, mellowed for 24 h and compacted to the maximum dry density under standard Proctor energy (ASTM D698-12e2). Compacted soil specimens were sealed with layers of polyethylene wrap and preserved in desiccators for the required curing period.

Consistency limits (ASTM D4318-10e1) and specific gravity (ASTM D854-14) tests were done for expansive soil, zeolite and uncured expansive soil-zeolite mixture.

Samples of 75 mm diameter and 15 mm height were used in one-dimensional swell (ASTM D4546-14) and consolidation tests (ASTM D2435-11). Swelling was conducted under 7 kPa surcharge pressure, and in fully inundated condition, followed by consolidation test with incremental loading.

For the shrinkage test, a set of compacted specimens of the same size as used in one-dimensional swell were fully inundated and full swell was achieved. Upon completion of one-

dimensional swell, soil specimens were drained and were stored in a temperature controlled room. Diameter and height of the soil samples were measured by a digital vernier caliper as well as recording wet mass at different time intervals, along the desiccation path until the volume change ceased.

Unconfined compression test (ASTM D2166-06) was conducted on compacted soil samples with the dimensions of 38 mm diameter and 76 mm height, using triaxial equipment in the unconfined condition with a speed of 0.76 mm/min.

Flexural strength test (ASTM C348-14) was applied on prismatic soil specimens of 40 mm x 40 mm x 160 mm. Soil samples were compacted with their optimum water contents and maximum dry densities by static compaction. Flexural load was applied on soil beams with a speed of 0.5 mm/min. Three identical compacted samples were tested for each soil group and the average flexural strength was determined.

XRF and XRD readings were done by preparing samples in pellets and using Rigaku ZSX Primus II type equipment, and in powder form with Rigaku Ultima IV type equipment respectively. SEM specimens were prepared by coating with a thin film of gold for better image processing. The investigation was done using a Jeol JSM 6610-LV type SEM analyser.

Experimental Results and Discussions

Physical Properties

The physical properties of liquid limit, plasticity index (ASTM D4318-10e1), linear shrinkage (BS 1377-2:90) and specific gravity (ASTM D854-14) when zeolite was added yielded reduced values of 50, 17, 13 and 2.51 respectively. Therefore, expansive soil and zeolite

mixtures (NZ) can be classified as a borderline between low and high plasticity silt, since zeolite possesses low plasticity (Table 1).

Compaction Characteristics

The compaction curves obtained by standard Proctor test are depicted in Figure 2. Studying the dry density-water content relationships, it can be observed that there is a minimal change in the optimum water content, from 26% to 24.5%, and in the maximum dry density, from 1380 kg/m³ to 1390 kg/m³. Therefore, it can be deduced that expansive soil (N) and expansive soil-zeolite mixtures (NZ) have exhibited almost similar dry density and optimum water content values, the degree of saturation at the optimum value being closer to the 100% saturation line for sample N.

Figure 2. Compaction curves of N and NZ.

Volume Change

One-dimensional Swell

Swell percentage ($\Delta H/H_0$) versus time curves obtained from one-dimensional swell test results for five different soil groups are given in Figure 3. The expansive soil possesses a high swell potential of 4.95% which is reduced by 53% in NZ (0-d) mixture to 2.35%. A further reduction of swell potential is observed with the increase of curing period. Swell potential of NZ (7-d), (28-d) and (90-d) are 1.57%, 0.38%, 0.35% respectively. Therefore, there is a marked decrease in percent swell value from 0-7 days curing to 28-days after which depicts an insignificant reduction until 90 days curing. These findings are in good agreement with Demirbaş (2009), and Yükselen-Aksoy (2010) who also reported the pozzolanic behaviour of zeolite reducing swell potential. As can be observed in Figure 3, the tendency of secondary

swell which progresses after the completion of primary swell in untreated soil diminished completely after the addition of zeolite. Therefore, it can be inferred that zeolite is very beneficial in improving swelling characteristics, especially in preventing the creep behaviour of secondary swell, which thrives over a prolonged period of time following the primary swell stage. This behaviour is attributed to the reaction product of calcium silicate hydrate which caused binding and cementation of soil particles enhancing the integrity of specimens against volume change.

Figure 3. Percent swell versus time curves.

Compressibility

Figure 4 depicts the void ratio versus effective consolidation pressure relationships obtained from one-dimensional consolidation test results. The parameters of compression index (C_c) and swell pressure (p_s') are summarized in Table 3. Generally a decreasing trend was observed in compression indices after treatment of expansive soil. NZ (0-d) has a compression index of 30% less than the original soil, which reduced further with curing, reaching to 30% additional reduction from 0-day to 90-day curing period. This substantial reduction in compression index is due to 50% zeolite addition, unlike Hong et al. (2012) who investigated on the effect of zeolite contents $\leq 10\%$ added to enhance the sorption capacity of sand-bentonite mixtures, and observed an insignificant effect on consolidation behaviour. This improvement is explained again with the pozzolanic process causing structural changes due to rearrangement of particles with curing time, being coated by CSH gel, developing into formation of tobermorite in time.

Figure 4. Consolidation curves.

Table 3. Compressibility characteristics of all soil groups.

Coefficient of consolidation (c_v) and coefficient of volume compressibility (m_v) values determined from consolidation test results, under different ranges of consolidation pressures were used to evaluate the saturated hydraulic conductivity (k_{sat}) using Equation 1.

$$k_{sat} = c_v m_v \gamma_w \quad (1)$$

where, k_{sat} is the saturated hydraulic conductivity, c_v is the coefficient of consolidation, m_v is the coefficient of volume compressibility, γ_w is the unit weight of water.

Saturated hydraulic conductivity values corresponding to different curing times are illustrated as a set of curves in Figure 5. The hydraulic conductivity values at different curing times, under varying pressure ranges depict that expansive soil has the lowest values at all pressure ranges which increased after zeolite inclusion. Hydraulic conductivity values are highly dependent on the fines content which is significantly higher in expansive soil than the fines fraction of zeolite. The cured samples of NZ have provided hydraulic conductivity values lower than the typical regulatory limit of 1×10^{-7} cm/s under all pressure ranges for 90-day cured samples only, verifying similar research findings by Ören et al. (2011), Ören and Özdamar (2013) and Du et al. (2015). The test results revealed a considerable reduction of 75% in saturated hydraulic conductivity values at all pressure ranges within 90-day curing period. The hydraulic conductivity values in 7-day and 28-day curing periods displayed an inconsistency which might be attributed to the instability of the initial phase of the pozzolanic process which reached equilibrium state after 90 days. The results indicate that the hydraulic conductivity values of 28-day cured specimens are also below the regulatory limit for pressure ranges above 100 kPa. The reductions in the hydraulic conductivity values can be attributed to

the calcium silicate hydrate forming during pozzolanic process, binding the particles and causing the reduction of pore spaces.

Figure 5. k_{sat} values under different consolidation pressures and curing times.

Shrinkage

Shrinkage tests are evaluated in terms of volumetric ($\Delta V/V_0$), axial ($\Delta H/H_0$) and diametral ($\Delta D/D_0$) shrinkage strains with respect to time and the results are presented in Figure 6. Volumetric shrinkage strain is observed to reduce by 58% from 0.19 to 0.08 with the addition of zeolite, followed by a further reduction of 38% from 0.08 to 0.05 within 90-day curing period.

Figure 6. Volumetric, axial and diametral shrinkage strains.

An average of 5% volumetric strain observed in this study at the end of 90-day curing period can be considered agreeable based on Kleppe and Olson (1985) who reported that volumetric shrinkage strain should be less than 5% for minor cracks not to occur.

In addition, volumetric shrinkage test results were fitted by the hyperbolic model (Fredlund et al. 2002) given in Equation (2).

$$e(w) = a_{sh} \left[\frac{w^{c_{sh}}}{b_{sh}^{c_{sh}}} + 1 \right]^{\frac{1}{c_{sh}}} \quad (2)$$

where, a_{sh} : fitting parameter (minimum void ratio), b_{sh} : fitting parameter (shrinkage limit), c_{sh} : fitting parameter (curvature of the hyperbola), w : gravimetric water content. The shrinkage property of a soil is characterized by its shrinkage curve which is normally presented as a

water content-void ratio plot. It is one of the tools in understanding the constitutive behaviour of soils. It represents the relation between the evolution of the shrinkage process and the subsequent structural changes, hence a practical tool to follow these hydro-structural property variations along the desiccation path (Fredlund et al. 2002).

Shrinkage curves and fitting parameters for all soil groups are shown in Figure 7 and Table 4 respectively. The final void ratio (a_{sh} parameter) and shrinkage limit (b_{sh} parameter) increased by 28% and 39% respectively after zeolite addition, which can be attributed to the increase in fines content, followed by corresponding reductions of 12.5% and 25% of 90-day cured specimens.

Figure 7. Shrinkage curves (a) NZ (0-d), (b) NZ (7-d), (c) NZ (28-d) and (d) NZ (90-d).

Table 4. Hyperbolic fitting parameters of the shrinkage curves.

Strength Properties

Unconfined Compressive Strength

Unconfined compressive strength of N was 411 kPa and decreased by 31% after zeolite inclusion (NZ) whereas failure strain increased by 1.8 fold. However, an increasing trend of unconfined compressive strength (UCS) of NZ mixtures was observed with ageing as presented in Figure 8. The biggest increment of UCS was obtained within 28-day curing period, which has increased from 290 kPa at 0-day to 390 kPa, followed by a lower rate of increment reaching to 394 kPa in 90-day curing period. Hence, the strength increment is almost negligible after 28 days. This value is considered to be within the range recommended by Daniel and Wu (1993) who have suggested the minimum unconfined compressive strength

of 200 kPa for compacted clay liners in arid areas, which is the lower limit of the “stiff” range based on Peck et al. (1974).

Figure 8. Unconfined compressive strength test results on NZ.

Flexural Strength

Tensile strength is an important parameter in the projects which are exposed to heavy loads such as, airfield pavements, highways, landfills and embankments (Vaniček 2013), therefore behaviour of stabilized soils under tension should be well understood. In this study, tensile strength is indirectly studied by flexural strength test, yielding the load-deflection curves for all soil groups shown in Figure 9. Flexural strength results calculated by using Equation 3 are listed in Table 5.

$$f = \frac{1.5PL}{bd^2} \quad (3)$$

where P is the maximum flexural load, b is the width, d is the depth and L is the span length of the flexure beam.

Figure 9. Load-deflection relationships.

Table 5. Flexural strength test results.

Initially, flexural strength of N is reduced after zeolite addition however, it varied in an increasing trend with curing time. Flexural strength of NZ (0-d) showed an increment of 57% from 47 kPa to 74 kPa after 90 days curing period which is a noteworthy improvement of tensile behaviour. Flexural strength of expansive soil is improved after stabilization and this complies with Muntohar (2011). This improvement of flexural strength is the end product of the pozzolanic reactions, strong cementation bonds developing in time yielded higher toughness yet reduced ductility.

Relationships between flexural strength with respect to swell and shrinkage strains, as well as compression index are displayed in Figures 10-12. In Figure 10, the reduction in swell potential can be observed to be 85% from 0-day to 90-day cured specimen, which corresponds to an increment of 70% in flexural strength. In Figure 11, a reduction of 34% volumetric shrinkage caused 30% increment in flexural strength. The reduction in compression index is observed in Figure 12 to be about 75% at the end of the 90-day curing period, which resulted in approximately 30% increment in flexural strength. Therefore, swell potential, volumetric shrinkage strain and compressibility are reduced significantly with the utilization of zeolite, the rate of change of each varying in different amounts with respect to flexural strength.

Figure 10. Relationship between flexural strength and swell potential.

Figure 11. Relationship between flexural strength and volumetric shrinkage strain.

Figure 12. Relationship between flexural strength and compression index.

SEM Analysis

Figure 13 (a) shows the scanning electron microscopy (SEM) image of expansive soil-zeolite (NZ) in the as compacted state, which shows flocculated soil fabric of NZ mixture. Figure 13 (b) reveals the micro-structural change due to hydration process within the expansive soil-zeolite mixtures resulting in the formation of elongated tobermorite-like gels (CSH) around the soil particle edges within curing period of more than 90 days. Figure (c) depicts the more homogeneous structure formed with further curing over 120 days, which enhanced the CSH fiber formation combining the zeolite and soil particles together. It is this change in structure and fabric which caused the improvement in mechanical properties of mixtures in terms of

reducing volume change, compressibility and increasing strength and durability in time. This hydration process proceeds continuously over long time periods, yet with a decreasing rate (Mitchell and El Jack 1966).

Figure 13. SEM micrographs of cured NZ mixtures after (a) 0 day, (b) 90 days, (b) 120 days.

Conclusions

An experimental study on the effect of curing on volume change and strength characteristics of expansive soil and zeolite mixture was perused and its suitability for geotechnical and geo-environmental applications was discussed, deriving the following conclusions:

1. Swell potential of expansive soil was reduced by 53% from 4.95% to 2.35% after addition of zeolite (NZ). This value was further reduced by 85% to a swell potential of 0.35% after 90-day curing period.
2. Desiccation behaviour of zeolite added expansive soil revealed substantial reductions in volumetric, axial and diametral strains within 90 days. Average reduction in volumetric strains was observed to be 58% from 0.19 to 0.08 when zeolite added to expansive soil, which through pozzolanic effect yielded 38% more reduction from 0.08 to 0.05 within 0-day to 90-day curing periods. Compressibility characteristics were improved by 30% reduction from 0.275 to 0.195 in the compression index with the addition of zeolite. A further reduction of 30% from 0.195 to 0.135 was observed throughout 90-day curing period.
3. Hydraulic conductivity values at different pressure ranges indicate that expansive soil-zeolite mixtures have approximately 83% higher values than untreated expansive soil due

to increased grain size with the use of zeolite, exceeding the regulatory limit of 1×10^{-7} cm/s in samples under pressures lower than 196 kPa and curing periods less than 90 days. However, these values were reduced by 80% within 90-day curing period to values well below the regulatory limit of 1×10^{-7} cm/s under all confining pressure ranges studied.

4. The unconfined compressive strength of expansive soil-zeolite mixture was observed to improve by 36% from 290 kPa to 394 kPa, whereas an improvement of 57% was recorded in flexural strength, increasing from 47 kPa to 74 kPa within 90-day curing period.
5. Linear relationships of flexural strength versus swell percent, compression index, and volumetric shrinkage were observed with increasing curing time, the rate of decrease of volumetric shrinkage with increasing flexural strength being the most significant.

The aforementioned improvements were based on the structural changes at micro level due to pozzolanic reactions, displayed by the scanning electron micrographs, which indicate the improved structure of the expansive soil-zeolite mixtures by cementation product of tobermorite.

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Figure Captions

Figure 1. X-ray diffraction pattern of (a) expansive soil, (b) zeolite.

Figure 2. Compaction curves of N and NZ.

Figure 3. Percent swell versus time curves.

Figure 4. Consolidation curves.

Figure 5. k_{sat} values under different consolidation pressures and curing times.

Figure 6. Volumetric, axial and diametral shrinkage strains.

Figure 7. Shrinkage curves (a) NZ (0-d), (b) NZ (7-d), (c) NZ (28-d) and (d) NZ (90-d).

Figure 8. Unconfined compressive strength test results on NZ.

Figure 9. Load-deflection relationships.

Figure 10. Relationship between flexural strength and swell potential.

Figure 11. Relationship between flexural strength and volumetric shrinkage strain.

Figure 12. Relationship between flexural strength and compression index.

Figure 13. SEM micrographs of cured NZ mixtures after (a) 0 day, (b) 90 days, (b) 120 days.

Table 1. Physical properties of materials.

Physical properties	Expansive soil (N)	Zeolite (Z)
Liquid limit (%)	65	41
Plastic limit (%)	36	29
Plasticity index (%)	29	12
Linear shrinkage (%)	18	8
Specific Gravity	2.69	2.35
Clay size (%)	43	15
Silt size (%)	50	41
Sand size (%)	7	44

Draft

Table 2. Chemical properties of materials.

Chemical composition	Expansive soil (N)	Zeolite (Z)
SiO ₂ (%)	36.5	61.7
CO ₂ (%)	17.9	7.05
CaO (%)	16.2	4.69
Al ₂ O ₃ (%)	11.8	12.7
Fe ₂ O ₃ (%)	6.87	1.58
MgO (%)	6.26	2.18
Other elements	4.47	10.1

Draft

Table 3. Compressibility characteristics of all soil groups.

Material	C_c	p_s' (kPa)
N	0.275	83
NZ (0-d)	0.195	46
NZ (7-d)	0.192	25
NZ (28-d)	0.145	-
NZ (90-d)	0.135	15

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Table 4. Hyperbolic fitting parameters of the shrinkage curves.

Material	N	NZ (0-d)	NZ (7-d)	NZ (28-d)	NZ (90-d)
a_{sh}	0.566	0.727	0.645	0.666	0.638
b_{sh}	0.23	0.32	0.25	0.25	0.24
c_{sh}	3.64	5.34	9.55	17.71	14.22

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Table 5. Flexural strength test results.

Material	Flexural strength (kPa)	Deflection at failure (mm)
N	61	0.41
NZ (0-d)	47	0.35
NZ (7-d)	50	0.21
NZ (28-d)	67	0.20
NZ (90-d)	74	0.25

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(a)

(b)

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Draft

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Draft

(a)

(b)

(c)

Figure 13. SEM micrographs of cured NZ mixtures after (a) 0 day, (b) 90 days, (b) 120 days.
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