STABILIZATION SWELLING OF EXPANCIVE SOIL USING SCRAP TYRE RUBBER

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Abstract

One of the most practical solutions to mitigate the impacts of expansive soil is to mix the soil with non-expansive materials, such as scrap tyre rubber, to decrease the clay fraction and increase the permeability of the soil, which reduces soil clay sheet molecules' water adsorbing activity and decreases the amount of swelling pressure and soil shrinkage. The main aim of the stabilization of the soil beneath the foundations of building structures is to decrease the effect of potential swelling pressure and swelling percentage in the soil. There are some methods available to minimize the effects of expansive soil on structures, which are classified according to the type of structure, climate condition and ability to perform the method.

Scrap tyre rubber can be a potential solution for the swelling potential of an expansive soil, such as kaolin clay, due to its non-sensitivity to moisture change (non-expandability) also environmental benefits by re-using waste tyres. This study investigates the possibilities of using tyres as stabilizers for an expansive soil such as kaolin clay. Several tests were done on a mixture with different proportions of tyre and kaolin clay to test properties such as swelling pressure, deformability, and the moisture content of the mixture.

The results indicate that rubber can be used to decrease the swelling potential of expansive soil, with some increase in deformation related to its proportion.

Keywords: Expansive soil, Scrap tyre rubber, Swelling, Deformability.

1. INTRODUCTION

Expansive soils undergo a volume change in soil mass when water content changes (Gueddouda et al., 2011). The variation in the amount of water content causes swelling or shrinkage in the soil, therefore expansive soils is sometimes called swell/shrink soils

(Gueddouda et al., 2011; Jones and Jefferson, 2012). Generally, volume changes in expansive soils are related to hydration and dehydration of the soil clay minerals, which is due to the change in amount of water content due to either climate variations (e.g. evaporation and rainfall), or localised site changes such as surface drainage, pipe leakage, drain pipes and planting. The process of volume change (swelling or shrinkage) in the soil is due to the attraction of the positive water molecules to the negatively charged clay (soil) particles (Budhu 2011; Mitchell and Soga, 2005). In reality, human engineering solutions cannot fundamentally change the level of water in the soil, and the best model is the natural evapotranspiration by which trees convey water from the soil to the atmosphere, from root to leaf (Cheney, 1988; Jones and Jefferson 2012; Terzaghi et al., 1996).

The process of shrinkage causes cracks, and thereby the soil opens slightly; the ensuing crack area allows the direct entry of water into the soil, subsequently increases swelling and the presence of water. Additionally, the cracks may become filled with different types of sedimentary minerals which further affect swelling pressures. Indeed, expected deformations of expansive soil are significantly greater than elastic and plastic theory prediction (Jones and Jefferson, 2012). Both swelling and shrinkage amounts depend on the following factors (Bell and Culshaw 2001; Houston et al., 2011):

- Soil characteristics, such as mineralogy
- Water statement in the soil (temporary and spatial)
- Geometry and structural stiffness, particularly foundation stiffness

The main aim of the stabilization of the soil beneath the foundations of building structures is to decrease the effect of potential swelling pressure and swelling percentage in the soil. It comprises the following methods (Jones and Jefferson, 2012):

- Isolate the structure from soil movements (alternative design); the footing should be as narrow as possible and continuous (Chen, 2010).
- Resist movements of the foundation using a stiff foundation design (e.g. mat foundation).
- Use ground modification techniques.
- Combine these methods together.

The aim of this study is first to improve the property of the native soil (kaolin clay), such as in terms of decreasing swelling pressure that causes heaving of structure (and consequently crack or failure). The effectiveness of adding tyre to the soil will depend on the soil condition and the amount of rubber used. Second, the study evaluates some geotechnical properties of the expansive soil rubber mixture, such as deformability, and Atterberg limits.

The Atterberg limit test will be done to look the plasticity index of the mixture compare to the native. Finally, the soil deformability will be tested through the one-dimensional oedometer test on compacted samples. Swelling percentage and pressure will also be determined. All the tests will follow the British Standard (Methods of test for soils for civil engineering purposes).

2. MATERIALS AND EXPERIMENTAL METHODS

2.1 Materials

2.1.1 Soil (kaolin clay)

Kaolin clay, also known as China clay, was used in this study to look at the different properties of the clay itself as a reference specimen, then it was mixed with different percentages of rubber size (2mm) to compare the properties of the mixture to the native soil through different conditions of loading and moisture content. It is available in the lab as a single unit bag weighing about 25 kg, and it is white in colour (Figure 1). In recent years it has been used in the production of clay brick (Akwilapo and Wiik, 2003). Its name is derived from Gaoling (Kao-Ling), which is a high hill in the Jingdezhen, Jiangxi province of China. Although it was mined as Chinese clay, the mineral was first described in Brazil in the year 1867, and it is available in some parts of the world. It is formed by the chemical weathering of aluminum silicate minerals like feldspar. Kaolin clay is from the group of hydrous aluminum silicates. Also, it included other common clay minerals such as dickite, halloysite, nacrite, kaolinite and allophone. Their chemical composition is 0.8% alumina, 46.3% silica, and 13.9% water (Prasad, 1991). The mineral composition of kaolin clay is usually found in soils, sediments, sedimentary rocks and hydrothermal deposits. The main ingredients which are in the formation of kaolin clay minerals are formed through the cycles of rock formation may be differ in its physical properties; however, it may have the same chemical composition with other clay minerals in its group.

Figure 1: Kaolin or china clay

2.1.2 Scrap tyre rubber

Scrap tyre rubber was used in this study as a stabilizer material mixed with kaolin clay to analyse the behaviour of the mixture with different percentages of rubber. It was cut into small pieces and then sieved to separate the different sizes of the rubber, which was 2mm, to study the effect of particle size of the rubber on the geotechnical properties of the mixture. The size (2mm) (Figure 2) was used in all of experiments of this study for two reasons: first, nobody used this size before; and second, there are some limitations for each test according to the British Standard 1377-1 (see Table 1).

Figure 2: Tyre chips

Table 1: Particle size limitation

Type of test	Maximum size of particle
Direct shear (shearbox)	H/10
Consolidation	H/5
Compressive strength (cylinder, H/D = about 2)	D/5
Permeability	D/12

where
H = height of specimen
D = diameter of specimen

(British Standard, 1377-1 1990)

It can be noted from the Table 1 that for the consolidation test the maximum particle size of the mixture should be less than 1/5 height of the sample. The height of the consolidation ring is 19mm. This means that the maximum size of the soil particle should not be greater than 3.8mm.

The rubber was used in different percentages, such as 5%, 15% and 30% by mass. 5% and 15% of rubber content was used based on the study of Edil and Bosscher (1994). They showed that the mixture of soil and rubber with a percentage of less than 20% by mass (or approximately 30% by volume) has a unique deformation (compressibility) very similar to untreated soil. The last percentage, 30%, was used as a way of consuming large quantity of scrap tyre rubber in civil engineering projects from the environmental pollution concern. In case of increasing deformation in the mixture, suggestions for future work can be made using other stabilizing materials with the tyre-soil mixture to compensate this loss.

2.2 Experimental methods

The following are the tests conducted on the kaolin clay as a control sample and different percentages of Expansive soil rubber mixture as a stabilized sample.

2.2.1 Index properties

Atterberg limit test was determined in accordance with British Standard, 1377: PART 2: 1990 to find the liquid and plastic limit of the soil, also calculating plasticity index using a multipoint method. These limits represent the behaviour of the soil state at different moisture contents. Also, they are used in the classification of soil (casagrande chart). The liquid limit of soil can be defined as the amount of moisture content that changes the state of the soil from plastic to liquid. The plastic limit corresponds to change in the soil from semi solid to plastic state.

Liquid limit can be determined using Casagrade's apparatus through calculating the amount of moisture content at which the two halves of a soil cake in the cup (Casagrande's device) will flow together for a distance of 12mm along the bottom of the groove separating the two halves, when the cup is dropped at the rate of two drops per second for about 25 times for a distance of 1cm between grooves (Verwaal and Mulder, 2004).

The test was run three times with different amounts of water in the sample in order to get the required number of blows at each stage. At the end of each test the sample was taken at the contact point of the soil cake in the cup and put it in the oven at about 110°C for about 24 hours to calculate the moisture content within the soil. A graph was plotted between the percentage of water content within the soil at each stage and the number of blows at the same stage. By using trend line, the liquid limit was calculated at the number of blows 25.

Both liquid limit and plastic limit are used together to calculate the plasticity index used in the classification of cohesive soils against liquid limit on the plasticity chart.

After finding liquid limit value, plastic limit was run according to British Standard, 1377: PART 2:1990. About 20g of the sample that passed sieve No.40 (425-μm) was taken. The soil was kneaded and then mixed with some distilled water to form a plastic ball. About six balls were created between the fingers then they were rolled between the palms of the hands until their diameters were 3.2mm. Those trials were accepted when the samples at that diameter cracked. If the samples did not crack at that diameter the test was repeated, either by adding water (if the sample cracked before 3.2mm diameter) or by using a new sample (if the sample was wet). The test was repeated three times. At the end of each test, about 6g of the soil through the cracked area was taken to calculate the moisture content of the sample. By taking the average moisture content of the samples the plastic limit was calculated.

2.2.2 The specific gravity

Gas jar method was used to determine the value of specific gravity of the soil and expansive soil rubber mixtures according to the British Standard, 1377: PART 2:1990. Specific gravity can be defined as the ratio of the density of soil to the density of gas-free distilled water at the same temperature. The test was undertaken using the following procedures relating to equation 1.

The gas jar and ground glass plate were cleaned and dried then weighed together (m1). About 200 gram of the specimen (dry) was added to the gas jar and weighed them with ground glass plate (m2). About 500 mL of de-aired water was added to the samples through the gas jar, and the gas jar was closed by a stopper and shaken by a shaker for about 20 minutes. After this period the stopper were removed carefully by washing it above the gas jar. More water was added to fill the gas jar and the plate was put on the gas jar carefully to fill the cylinder completely, ensuring that there was no trapped air under the plate, then they were weighed (m3). The sample was removed, and the gas jar was cleaned, dried and filled completely with water and then weighed with the base plate (m4). The stages were repeated for all the mixtures of kaolin clay alone and mixtures of 5%, 15% and 30% of tyre size 2mm with kaolin clay. The specific gravity was calculated by the following formula:

$$\mathrm{Gs} = \frac{m2 - m1}{(m4 - m1) - (m3 - m2)} \quad \text{Equation 1}$$

2.2.3 Oedometer test

Oedometer (swell-consolidation) tests were conducted conforming to British Standard, 1377: Part 5: 1990 on compacted (remoulded) specimens. The samples were prepared in the lab at optimum water content and maximum dry density. This method was selected during the sample preparation to calculate the maximum swelling potential of the samples in spite of better controlling of the specimen density. There are two methods of preparing remoulded samples in the absence of undisturbed samples. First, compact the specimen (soil) into a proctor mould at optimum moisture content and maximum dry density using 2.5kg hammer as a compaction effort then driving a consolidation ring into a compacted specimen. The second method is preparing samples in the mould tested by calculating the amount of material and volume of water for the required density. Although both methods produce disturbed samples, each of them has pros and cons in terms of disturbances and uniformity of the density through the layers.

In this study the second method of compacted sample was used for preparing the specimens. Knowing the volume of the oedometer ring and required (target) sample density, the required amount of water, dry soil and rubber was calculated. The appropriate amount of water was then added to the soil or expansive soil rubber samples and mixed thoroughly and then compacted in uniform layers and compaction effort in the swell consolidation ring. The specimen ends were then smoothed by using spatula and filling in surface voids with the same moisture material.

Before preparing the sample the inside diameter, height and weight of the swell-consolidation ring was measured, and also after compacting and levelling both ends of the specimens again the samples were weighed with the ring. The ring with the specimen was put in the consolidation cell. A pore stone and filter paper were placed at the bottom and top of the specimen. The ring was fixed with in consolidation cell by tightening the screws, then the loading cap was placed carefully at the centre of the ring. The consolidation cell was put on the frame platform and the loading beam was adjusted so that the lever arm was slightly above horizontal position, and the loading yoke just made contact with the loading cap.

2.2.3.1 Swelling

Swelling is an increase in volume of a soil gradually under negative excess pore water pressure, which is in reverse to consolidation (Craig, 2004). It happened due to change in moisture content with in the soil or due to unloading. For instance, heaving may occur at the bottom of an excavated pit of saturated clay due to swelling. In this study, swelling pressure and swelling percent were determined in conformity with British Standard, 1377: Part 5-4.1: 1990 for the kaolin clay and all the mixture of kaolin with different percentage of rubber content within the mixture. Swelling pressure is the amount of vertical pressure that should be put on a laterally confined soil specimen in an oedometer ring to prevent its swelling when allowing water to access the sample. Swelling percent is the total swelling height of the specimen (ΔH) from the dial gauge reading divided by the initial height of the sample (H_o) equation 2 :

$$\text{Swell \%} = \frac{\Delta H}{Ho} \quad \text{Equation 2}$$

The test was begun by placing a seating load on the lever arm and the compression reading from the dial gauge was recorded. At the same time water was added to fill the consolidation cell. After adding water to the sample, the dial gauge reading (positive value) indicated that there is a swelling in the specimen under seating load. The maximum swelling value from the dial gauge reading was recorded after it reached approximate equilibrium. This procedure took several hours. This value was used in calculating the swelling percent. Loads were added to the lever arm gradually to counterbalance the swelling potential until the dial gauge reading reached almost zero and equilibrium. The total amount of load was recorded and converted to stress which represents as a swelling pressure by the following formula equation 3:

$$\text{Stress} = \frac{9810 * m * a}{A} \quad \text{Equation 3}$$

Where:

m: is the mass, put on the lever arm (kg)

a: is the lever arm ratio;

A: is the area of the specimen in mm^2

2.2.3.2 Consolidation

Consolidation can be defined as the plastic deformation of a soil mass, which is a function of time due to expulsion of excess pore water pressure in the soil mass. This happens due to change in the applied load during construction. Deformations start at the time of load application and continue for a long period of time, which may take several years. During these times the water in the voids of the soil mass under excess pressure slowly spread or squeeze out. After the primary consolidation has been completed due to the dissipation of excess pore pressure, soil re-arrangement will start due to the viscous nature of the soil mass.

After the swelling test has been completed, the consolidation test was started by putting the load on the beam arm and the deformation (compression) of the specimen was measured by a dial gauge. The load was applied to the beam, and is usually doubled every 24 hours. Readings were taken for each loading from the micrometre dial gauge after that period. The specimen was kept saturated (under water) throughout the test. Knowing the initial dimensions of the specimens and the deformation of the specimens at the end of each load application, and the specific gravity, the void ratio was calculated for each loading stage.

After the loading stages reached up to 32kg on the lever arm, the samples were unloaded by removing load from the lever arm as the same pattern for loading stages. Unloading caused the specimens to rebound elastically and to swell. Data obtained from unloading increments were treated in the same manner as loading increments. Using a semi-logarithmic graph paper, a plot of void ratio vs. pressure relationship was drawn. The void ratio-logarithmic stress plot was used to determine the soil compressibility defined by compression index (Cc) and recompression index (Cr). After completion of the test, the ring with the specimen was taken out from the consolidation cell. The porous discs were removed carefully, and the sample was wiped and weighed with the ring. A sample was taken from each specimen for calculating final moisture content within the soil by placing in the oven for 24 hours at 110°C.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Index properties

Atterberg limit tests were conducted for kaolin clay to find liquid limit and plastic limit. The test is used for finding the consistency of the soil by looking at the liquid and plastic limit values used in calculating plasticity index. Both liquid limit and plasticity index were used in the classification of the soil according to Casagrande's chart.

Using Casagrande apparatus, liquid limit was determined for the kaolin clay after passing sieve no. 40 and taking three tests for plotting a chart between moisture content and number of blows from the Casagrande device. As shown in Figure 3, the liquid limit was determined as 70.2 for 25 blows after making a trend line for the test points (test procedure).

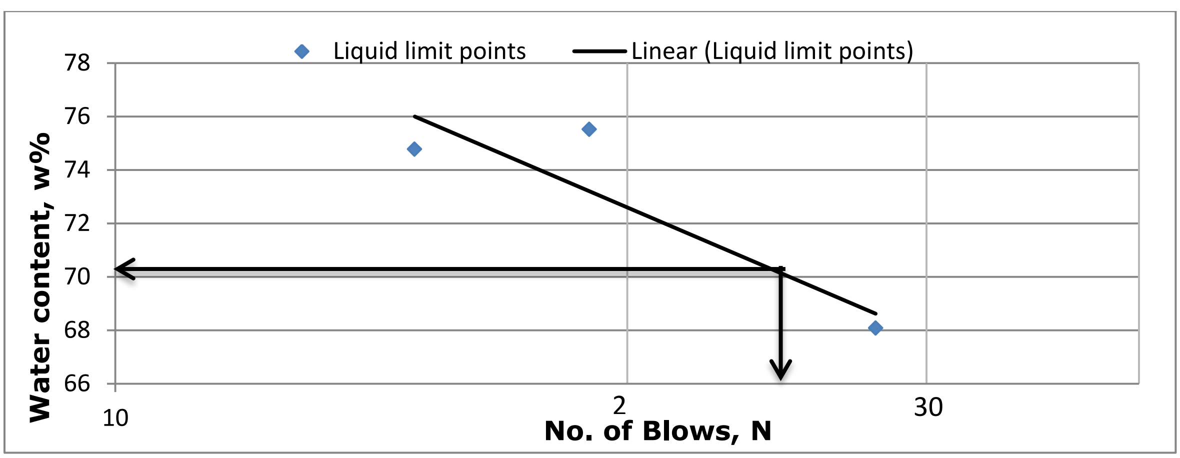

Figure 1: Liquid limit chart

After finding liquid limit, the plastic limit was determined by taking the average of moisture content of the three samples taken, and the result was 31. Finally, the plasticity index (the difference between liquid limit and plastic limit) was calculated as 39. From these results, and according to Figure 4, the soil would be classified as inorganic clay with high plasticity. Also, from Table 2 the soil could be classified as very high expansive soil, which has the ability to swell and shrink when its moisture content changes.

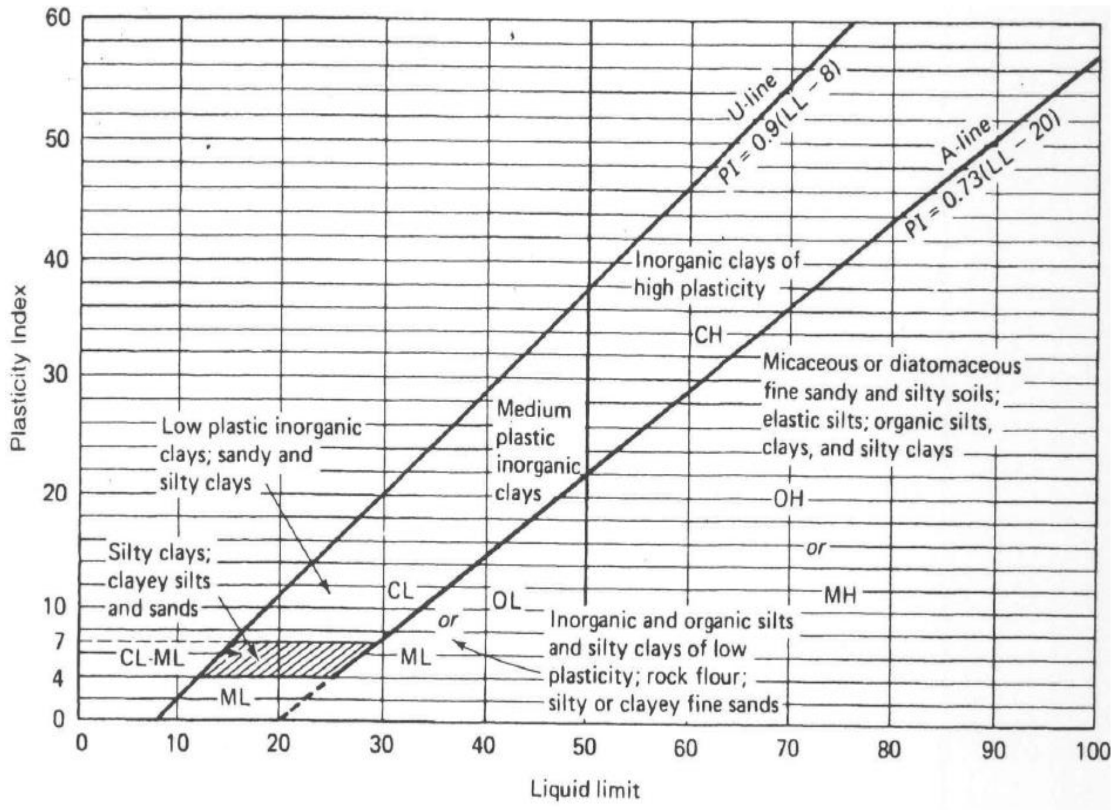

Figure 2: Casagrande's plasticity chart

Table 2: Expansive soil classification based on the soil plasticity

Liquid limit	Plastic limit	Potential for volume change
20 -35	< 18	Low
35 - 50	15 - 28	Medium
50 - 70	25 - 41	High
> 70	> 35	Very high

(Holtz and Gibbs, 1956)

Adding rubber to the soil (kaolin clay) has no effect on the liquid limit and plastic limit values, because the rubber would be retained on sieve No. 40 (0.425 mm) during liquid limit test, and for the plastic limit because the particle size of the rubber used was 2 mm, so it cannot create a thread and roll it until the diameter of the thread becomes 3.2mm. Thus, the mixture remains the same in terms of the value of liquid limit and plastic limit, with decreasing amount of clay fraction in a unit mass of soil.

3.2 *The Specific* gravity

The specific gravity was determined for all the specimens by weighing them in the air and then in de-aired water. From the results in Table 3, it can be seen that the specific gravity decreases with increasing percentage of rubber content. This is because the rubber is a lightweight material compared to clay particles. It invades a greater volume while in the water and the mass will be less, so the unit weight is decreased.

Table 3: The Specific gravity of materials tested

Material	**Specific gravity**
kaolin clay	2.57
5% tyre 2mm + kaolin clay	2.44
15% tyre 2mm + kaolin clay	2.33
30% tyre 2mm + kaolin clay	2.14
5% tyre 5mm + kaolin clay	2.43
15% tyre 5mm + kaolin clay	2.34

3.3 Oedometer 1d *consolidation* test

Swelling consolidation tests were carried out for four specimens; kaolin clay, 5%, 15% and 30% expansive soil rubber mixture with tyre size 2mm. All of the specimens were compacted at optimum moisture content and maximum dry density. Samples with high dry densities (low void ratios) usually contain greater proportions of clay minerals, so their swelling potentials will be high.

From this test swelling percentage, swelling pressure and consolidation settlement were calculated for all of the specimens under same condition of loading and saturation. Swelling can be defined as an upward movement of the sample during the test, while consolidation is downward movement of the sample, which is used in the calculation of the total settlement of the specimens. Settlement can be classified as immediate (occurs after application of load), consolidation (squeezing of water from the voids) and creep (re arrangement of particles). In this test, only consolidation settlement was calculated to make a comparison of deformation between the different samples. The swell consolidation test results are summarised in Table 4.

Table 4: Swell-Consolidation test results

Materials	Moisture content %	Density (gm/cm³)	Cc	Cr	Swell percent %	Swell pressure (Kpa)
Kaolin clay	29%	1.46	0.1661	0.042	4.65	55.3
5% tyre + kaolin clay	29%	1.39	0.1495	0.050	3.45	42.9
15% tyre + kaolin clay	27.5%	1.34	0.1329	0.058	2.22	22.1
30% tyre + kaolin clay	25%	1.29	0.1163	0.0747	1.33	12.0

3.3.1 Swelling

Swelling tests were carried out on the specimens to calculate swelling percent and swelling pressure by applying a seating load at the beginning of the test not more than 6kpa, at the same time adding water to the oedometer cell to make the sample saturate according to the British Standard. The samples were compacted to maximum dry density and they were partially saturated. After adding water to the samples, they gradually become saturated due to capillary suction, and their volume increased one dimensionally (upward), which caused the dial gauge reading to change to a positive value. Swelling percentage was calculated as the amount of increase vertically in the height of the specimen divided by its initial height. Swelling pressure was determined for each sample by calculating the amount of loads, which were put on the lever arm of consolidation apparatus in order to decrease the reading of the dial gauge to zero and equilibrium.

From Table 4, it can be noted that increasing the percentage of rubber to the native soil (i) reduced the swell percent, and (ii) reduced the swell pressure. This is because swelling of an expansive soil is influenced by many factors, such as clay mineral composition, amount of non-clay material present, density, void ratio, and cementation (Mokhtari and Dehghani, 2012). In other words, the presence of rubber (non-clay materials) reduces the clay-mineral content per unit mass of the mixture, which means that the total surface area of expansive clay particles decreases which causes swelling. Another reason is that, with increasing rubber content optimum moisture content decreases, which has a great effect on swelling of expansive soil.

Both swell percent and swell pressure for the specimens are presented in Table 4. It can be seen that (form the table 4) increasing rubber content to the native soil (kaolin clay) decreases the swell percent and swelling pressure by more than 20% for each 5% increase in tyre size 2mm to the mixture.

In practice, the size of the clay body, overburden stress and moisture content (climate condition) have a great impact on the volume change of a clayey soil. The distinction between them is that a thick clay body with highly expandable property of clay particles will have no swelling potential if there is a sufficient weight as the overburden stress to balance swelling pressures. Furthermore, the location of the clay layer is another factor that affects the swelling potential either beneath or above the water table, and near the surface of the ground or at depth (environmental effect). This may change moisture content within the soil mass. Thus, soil in the warm area has a high swelling potential because the soil is unsaturated and will have abilities for suction of a large amount of water, and its swelling will change according to its degree of saturation. In cool, humid regions the swelling potential of the soil will be lowest because moisture content within the soil is highest, and the amount of precipitation/ evaporation ratios is lowest. When using a mixture of soil and rubber, the above factors should be considered.

3.3.2 Consolidation

After swelling test was completed, consolidation test was begun for all specimens: kaolin clay alone; 5%, 15% and 30% tyre size 2mm with kaolin clay mixture. The tests were conducted simultaneously, beginning with loading all the specimens by 1kg and doubling them in the next 24 hours until it reached 32kg. After this value the test was reversed to unloading stage by removing loads from the lever arm in the same way as the loading stage until it returned to 1 kg. The result of the tests was plotted to show the displacement and void ratio per each loading and unloading cycles, as shown in Figures 5 and 6 qualitatively, and in the Table 4 and 5 quantitatively.

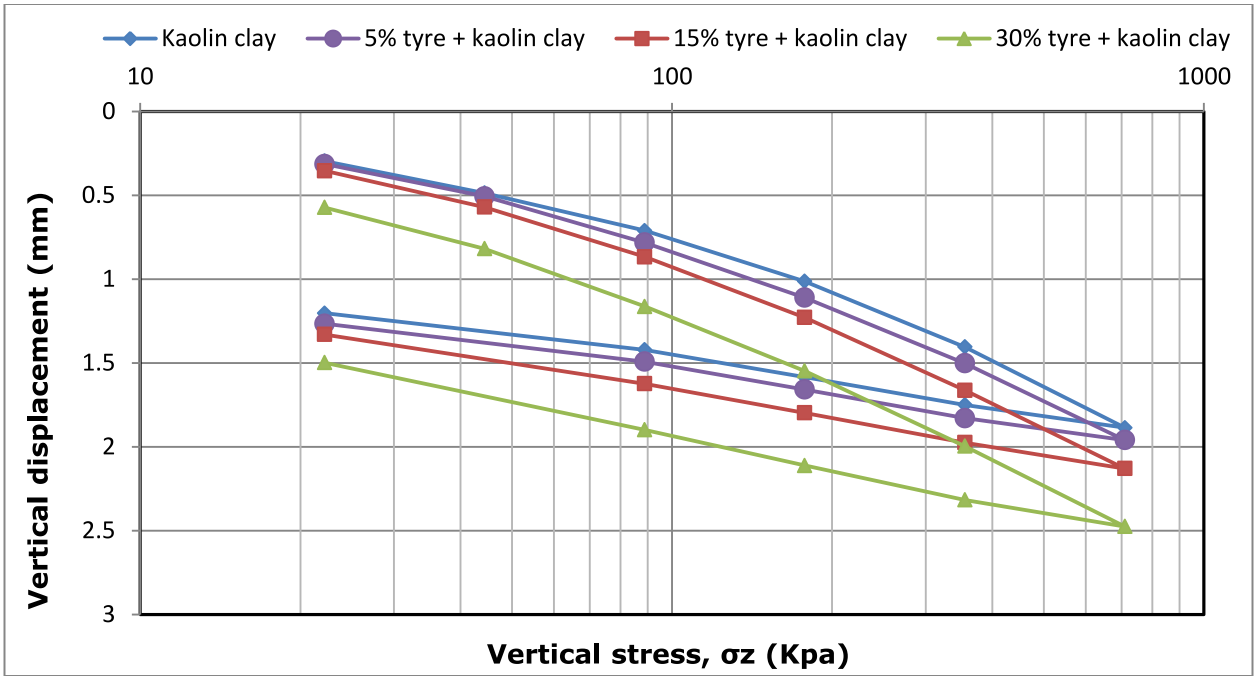

Figure 5: Vertical stress VS vertical displacement response of the kaolin clay and expansive soil rubber mixture with scrap tyre rubber 2mm

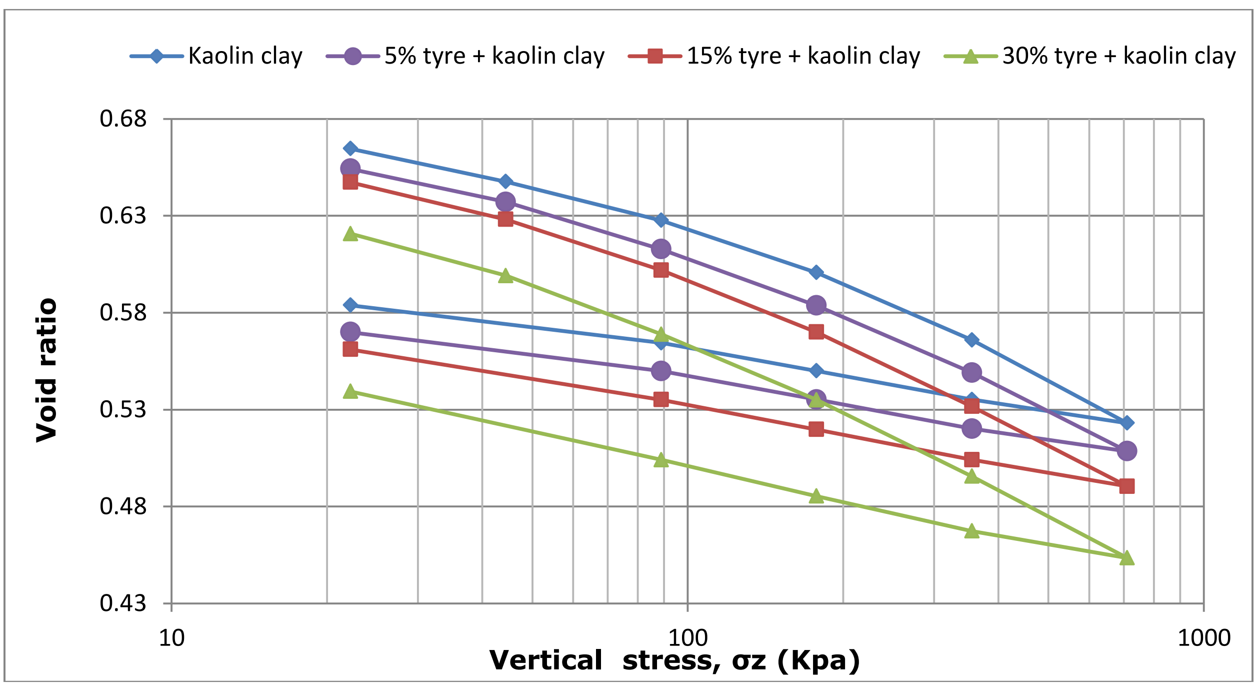

Figure 6: Void ratio VS logarithmic vertical stress of the kaolin clay and expansive soil rubber mixture with scrap tyre rubber 2mm

From Figure 5 which is a distinction between vertical stress and displacement, it can be seen that the displacement of kaolin clay under stress of 22.2 kpa is 0.296 mm while for the 5%, 15% and 30% of expansive soil rubber mixtures are 0.314, 0.354 and 0.573mm, respectively. Also, the total strain for the kaolin clay sample is 1.591mm; about 43% of this strain was recovering to the initial state as an elastic strain, and 57% was irrecoverable (known as plastic strain). Table 5 explains the total, elastic and plastic strain for all specimens.

The results of the void ratio vs. logarithmic stress shown in the Figure 6 and the Table 4 are essentially identical for all of the specimens with some variations in the slope of the normal consolidation line (Cc) and the slope of unload reload line (Cr). This is because the void ratio - logarithmic pressure curve is not affected by the size of the sample, load increment ratio, load duration and drainage path. The major factors affecting the shapes of the curves are (a) the soil, represented chiefly by its mineralogy and grain shapes; (b) the initial structures of the specimens; and (c) the initial state (void ratio and normal stress) (Poulos, 1971). From Table 4 it can be seen that the slope of the normal consolidation line represented by Cc decreases with increasing the percentage of rubber. The reason may due to the deformability of the rubber material when it is loaded. In contrast the slope of the unload reload line (URL), which is represented by (Cr), increases with increasing rubber content, which may refer to the properties of the rubber (recoverable property) when unloaded.

Table 5: Total, elastic and plastic strain of the kaolin clay and expansive soil rubber mixtures

Material	**Total strain mm**	**Elastic strain %**	**Plastic strain %**
Kaolin clay	1.591	43	57
5% tyre 2mm + kaolin clay	1.646	46	54
15% tyre 2mm + kaolin clay	1.776	49	51
30% tyre 2mm + kaolin clay	1.902	51	49

Application of these mixtures in practice will depend on the type of the design (application of stresses to soils) which can be categorized in two parts: (a) deformation of foundation soil controlling the design; and (b) failure of the foundation soil is controlling the design. Deformation controlled and failure controlled problems can be clarified by considering a building on a soil foundation (Poulos, 1971). For example, for rigid or very flexible building the deformation of the foundation soil does not affect the building; in this case it is only necessary to check the safety of the foundation against failure. Conversely, for intermediate flexibility building (the building fails by small deformation of the foundation soil or it has a limited deformation for some reasons), the deformation of the foundation soil must not be excessive because it causes failure of the building. In summary, excessive deformations of the foundation soils must be prevented in the case of building failure first; and failure of the foundation soil must be prevented if the foundation soil fails first.

4. CONCLUSION AND RECOMMENDATIONS

4.1 Conclusion

The following conclusions were reached based on the interpretation of the results of this study.

1. The soil which was used in this study was high plasticity soil, and the size of the tyre particle (2mm) which was used as a stabiliser had no effect on the plasticity of the mixture, because the particle sizes were greater than 0.42 mm, which cannot pass sieve no. 40. Furthermore, the percentage of the tyre that was used was less than 35%, which the mixture could be classified as fine-grained soil. Beyond this percentage it can be expected to have an effect on the plasticity of the mixture, because this time the mixture might be classified as coarse-grained soil. Thus, it is suggested that for future study the tyre is ground into powder to change the behaviour of the mixture through its plasticity.

2. Specific gravity which is a unit weight of the mixture for the unit weight of water decreases with increasing rubber content in the mixture.

3. The swell values of the mixture were lowered dramatically with increasing the percentage of tyre content for two reasons. First, the rubber will replace the clay particles that cause the swelling potential of the native soil by adsorbing moisture. Second, the rubber is impermeable, thus increasing its percentage will affect decreased optimum moisture content within the mixture to reach the maximum dry density.

4. From the consolidation test the native soil has a displacement under certain loading and the displacement increases dramatically with increasing rubber content. Adversely, the mixture of rubber and kaolin clay has a higher ability to recover after

unloading compare to the native soil and the recoverability increases with increasing percentage of rubber content.

Recommendations

From the test results of this study (using scrap tyre rubber as stabilizer) it can be concluded that rubber content within expansive soil (kaolin clay) reduces the swelling potential of the mixture. However, it decreases settlement (deformation). From these conclusions and from environmentally concerns about firing tyres, air pollution, and from the economic perspective, it can be recommended that for future study two steps:

1. try powdering tyres in order to:
   i) Get a uniform distribution of the tyre throughout the mixture from the laboratory and also from the site in order to get a good strength through a mass of soil and decreasing differential settlement. Furthermore, this will be more protected from firing and degradation of the tyre.
   ii) Change the liquid and plastic limits of the mixture, because tyre in the form of powder can pass through sieve no. 40, which will affect the behavior of the mixture when subjected to variation of moisture content.

2) From the environmental perspective, the increasing number of tyres in the world every year represents a harmful waste of a very valuable engineering material. It can be suggested that other additive materials can be added to the mixture of tyre and kaolin clay to compensate the strength loss of the mixture due to the deformable property of the rubber under applied load. For safety reasons, proper covered soil is needed on the top and side of the embankment. In this way a large quantity of scrap tyre can be used to reduce pollution while providing a cheap and cost-effective engineering material.

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