Improving expansive clay subgrades using recycled glass: Resilient modulus characteristics and pavement performance

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Abstract

The scarcity of sound soils, especially in urban areas, often forces engineers to construct the pavement on problematic subgrade soils such as expansive clays. The associated cost involved in replacing the existing problematic soil is avoided by adopting treatment techniques. In this study, a type of high plasticity expansive clay was mixed with 10, 20, and 30% sand-size recycled glass (RG) as a non-chemical soil treatment approach. An extensive investigation comprising ~~of~~ experimental works, numerical modeling, and pavement performance analysis was undertaken. After determination of the physical properties of clay and RG, resilient modulus characteristics of clay and the three clay-RG mixtures were carried out through an experimental program. Subsequently, the obtained resilient modulus data sets were incorporated into a finite element analysis program in order to analyze the stress-strain response of pavement models founded on clay and RG-treated subgrades. The compressive and tensile strains achieved through the analysis of the pavement models under traffic loads were next used to compare each pavement model with respect to fatigue and rutting performances. The experimental results showed up to a 113% increase in resilient modulus of clay by the addition of 30% RG. The outcomes of the analysis on pavement systems modeled using the experimental input showed a considerable reduction in compressive and tensile strains by treating the clay subgrade with RG. Consequently, the strain reduction exhibited a significant increase in fatigue life and rutting life of pavements founded on RG treated clay subgrades. The outcomes of this research aim to encourage the construction industry to consider the utilization of environmentally clean recycled aggregates, such as RG, for improving subgrades with problematic soils and hence, promote sustainable construction materials and approaches.

Keywords: Recycled glass, expansive clay, subgrade treatment, pavement response analysis, rutting and fatigue life

1 Introduction

The urbanization, industrialization, and the consequent dramatic population increase, especially in metropolitan areas, have led to considerable growth in construction activities. Transportation infrastructure projects, especially those related to pavements of roads, are a continuous construction activity in urban areas to meet the transport needs of the growing population. The typical structure of flexible pavements comprises an asphalt concrete surface course, unbound granular base (and an optional subbase) course with the subgrade soil as the foundation. In urban areas, due to space limitations and hence, dictated road alignments, pavements may need to be constructed on problematic subgrades, such as expansive clay soils. Expansive soils are typically rich in hydrophilic minerals such as illite and montmorillonite, making them significantly sensitive to moisture changes. Expansive clays swell (increase in volume) as a result of increased moisture content and shrink (decrease in volume) due to drying. This behavior of expansive soils as a road subgrade results in heave, subsidence, and uneven road surfaces, which lead to several types of pavement distresses, such as the emergence of cracks on the surface of the road and premature deterioration [1, 2].

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1 Introduction

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Traditionally, to mitigate the significant potential for volume change in expansive clay, chemical binders such as lime and cement have been used [2]. Several researchers have made attempts to promote solid waste as alternative construction materials to the traditional lime and cement treatment methods to mitigate the environmental and economic drawbacks. The soil improvement by the utilization of solid wastes can be achieved by chemical and/or non-chemical methods. Chemical stabilizers improve the properties of expansive clay through chemical reactions, whereas with non-chemical stabilizers, the soil improvement is achieved by reinforcing the soil structure. In recent years, various types of solid wastes have been introduced and evaluated as chemical stabilizers, such as fly ash [3], calcium carbide residue [4], lime kiln dust [5]. Several scholars have investigated the improvement of subgrade soils using a combination of chemical and non-chemical stabilizers, such as spent coffee and geopolymers [6], short polypropylene fibers and polyvinyl alcohol polymer [7], rice husk ash and cement [8], polyethylene terephthalate fiber and fly ash [9]. Under non-chemical stabilizing, researchers have mainly used various types of fibers, such as carpet waste fibers [10], rubber fibers [11] polyester fibers [12]. However, the non-chemical stabilization approach using sand-like particles, such as recycled glass, has been scarcely investigated in the literature.

Recycled glass (RG) is a product of recycling industries and is broadly used in construction projects as an individual sand-size construction material [13, 14] or in combination with other natural or recycled aggregates [15, 16]. RG consists of various colored crushed glass particles and often debris such as paper, plastic, and food waste, if not washed. Containing glass particles with different colors is the main obstacle for RG to be re-used in bottle production industries. This drawback, combined with the desirable engineering properties of recycled glass, makes the construction industry a suitable destination for this material [14]. In Australia, the application of RG as a construction material is encouraged in practicing construction guidelines, such as Austroad's Guide to Pavement Technology, Part 4 [17], and Sewerage Standards for Embedment [18]. The environmental footprint of RG has been investigated by several researchers [19, 20] who concluded that RG is an environmentally clean material that complies with regulatory requirements such as EPA-Victoria [21] and U.S.EPA [22].

Despite the early promise, comprehensive investigations on RG's true capacity to stabilize subgrade soils have been very limited to date. Eberemu, et al. [23] mixed up to 20% of RG with the maximum particle size of 4.75 mm with a type of lateritic clay and carried out geotechnical tests on the mixtures. Their results showed up to a 15.5% reduction in plasticity index, up to 44% increase in friction angle, and up to 70% increase in California Bearing Ratio (CBR). Wartman, et al. [24] added up to 90% of two types of well-graded sand-size RG to fine soils, which resulted in increased unit weight and improved shear strength characteristics of the soils. Strength characteristics of cement stabilized expansive clay mixed with up to 20% of fine recycled glass (< 300 μm) were studied by Ikara, et al. [25]. Their results showed further increase in bearing capacity (CBR), and unconfined compressive strength (UCS) of the cement stabilized expansive clay.

The few studies mentioned above that investigated the behavior of clay-RG mixtures mainly focused on their basic geotechnical properties such as plasticity, CBR and UCS. The evaluation of the response and performance of the subgrade through nearly static experimental methods, such as CBR, do not satisfactorily simulate the behavior of a pavement system that undergoes repeated loadings of vehicular traffic [26]. For a more realistic evaluation of the performance and stress-strain response of pavement materials under repeated loading, the resilient modulus (Mr) concept was introduced by Seed, et al. [27], which accounts for the stiffness characteristics of pavement materials. Ever since, pavement experts have repeatedly evaluated the performance of treated and untreated subgrade soils using the resilient modulus characteristics commonly obtained by Repeated Load Triaxial (RLT) testing. However, to the best of the authors' knowledge, no experimental research has been carried out on the resilient properties of subgrade clays improved by the addition of sand-size recycled glass.

While several studies have focused on the experimental evaluation of treated subgrades, analysis of their resilient response in pavement systems using the experimental results is still lacking. In this research, an experimental program was used to evaluate the resilient modulus characteristics of clay-RG samples, followed by a numerical stress-strain response analysis. The Mechanistic-Empirical Pavement Design Guide (MEPDG) [28] strongly encourages the utilization of the resilient modulus obtained through RLT testing for design purposes. Experimentally obtained resilient moduli can be used to determine the model coefficients of the predictive constitutive resilient modulus models. These coefficients remain the same for a certain type of soil and are typically used as a direct or indirect input in the pavement design and analysis software packages as the Level 1 MEPDG input.

The stress-strain response analyses in this research were carried out through a three-dimensional viscoelastic finite element analysis program, FlexPAVE™. This computer program is capable of evaluating the pavement behavior under repeated loads of various moving vehicles, at various pavement temperatures under different subgrade conditions [29]. Several researchers have validated the response analyses of FlexPAVE™ through comparison with field observations [30, 31]. In this research, the RLT test results on clay-RG mixtures were incorporated in FlexPAVE™ analysis as input parameters. The compressive and tensile strains achieved through the analyses were next used for a fatigue and rutting performance comparison analysis of the pavements modeled over untreated clay subgrades and those over RG-treated subgrades. The review of the literature showed that the study on the stress distribution and associated strains due to traffic loads on treated or untreated subgrades has not been focused on as much as experimental studies.

The experimental results, stress-strain response analyses, and performance analyses of the RG-treated subgrade soils used in this research aim to promote the application of recycled materials to support the circular economy while achieving the improved performance of subgrade materials. The sand-size recycled glass can be an alternative to relatively costly and less environmentally-friendly traditional methods of soil stabilization in road construction projects.

2 Materials and Methods

2.1 Materials and basic properties

Materials used in this research included a natural expansive clay found in the majority of the western metropolitan area of Melbourne, Australia, and sand-size recycled glass, supplied by a commercial recycling facility in Melbourne, Australia. In addition, for numerical modeling purposes, a typical asphalt concrete (AC) and aggregate base course (ABC) commonly used in North Carolina, USA being "S9.5B", and "Belgrade", respectively, were used. While physical and mechanical properties of the clay and RG were determined through an experimental program, properties of S9.5B and Belgrade ABC were obtained from previous studies [32, 33].

For carrying out experiments on clay, first, lumps of clay collected from a depth of 0.2-0.8 m were left in the oven, set to 50 ̊C for four days to dry. Dry clay lumps were next crushed to 5 mm pieces using a laboratory-scale crusher and subsequently ground using a soil grinder. Figure 1 shows the dried lumps of clay, together with the crusher, and the grinder used for preparing clay samples.

For the determination of the particle size distribution of Clay, a wet sieving procedure was followed. Figure 2 shows the plasticity and physical properties of Clay and RG. Coefficient of Uniformity (C_u) of 6 ($C_u \geq 6$) and Coefficient of Curvature (C_c) of 1.1 ($1 \leq C_c \leq 3$) classifies RG as a well-graded sand (SW) according to the UCSC classification scheme. Based on the liquid limit (LL) and plastic limit (PL) values of clay, presented in Figure 2, and following the USCS classification scheme, the Clay is classified as high plasticity clay (CH), containing 11.8% sand-size particles.

In order to quantitatively assess the expansiveness of the Clay, shrink-swell tests were carried out following the AS-1289.7.1.1 [34] procedure. This test includes two companion tests -shrinkage test and swelling test. In this research, using a 50 mm diameter thin-walled tube, three undisturbed core samples were obtained from a depth of 0.5 - 1 m for each shrinkage and swelling test.

Shrinkage samples were trimmed and observed to be free of defects and/or voids. Small pins were weighted and pushed into the core from the two ends to facilitate a consistent length measurement throughout the shrinkage process. The mass and dimensions of samples at the initial state, during the air-drying process, and after oven-drying at 105-110°C were taken. The maximum shrinkage strains (ε_{sh}) were then calculated based on the measured lengths. The swelling test is a simplified oedometer test in which the sample is mounted in a rigid stainless steel ring of approximately 20 mm height and a diameter of 45 mm. In this test, the ring containing the sample was placed between porous stones and mounted in the cell of the test apparatus. The apparatus was equipped with dial gages to measure the swelling (or settlement). Initially, an overburden pressure of 5 kPa was applied for 5 minutes followed by a pressure of 25 kPa for 30 minutes and the initial settlement was recorded. Next, the sample was submerged with distilled water and readings were undertaken until less than 5% variation in the swelling was observed for a period of at least 3 hours. The measured sample heights were used to determine the maximum swell strains (ε_{sw}). Shrink-Swell index (I_{ss}) was next determined using Equation 1.

$$I_{ss} = \frac{\varepsilon_{sh} + \frac{\varepsilon_{sw}}{2}}{1.8} \qquad (1)$$

Shrink-Swell index (I_{ss}) can be used for estimation of instability index which is the product of the lateral restrained factor, α, and I_{ss} [35]. Utilizing the instability index, the characteristic surface movement (y_s) can be determined for one type of soil layer using Equation 2 adopted from AS-2870 [36].

$$y_s = \frac{I_{pt} \times \Delta\psi \times h}{100} \qquad (2)$$

Where y_s is characteristic surface movement (mm), $\Delta\psi$ is the change in suction (taken 1.2 pF as recommended by AS-2870 [36]), and h is the thickness of the soil layer (taken 1 m for subgrade). For the Clay used in this research, the average ε_{sh}, ε_{sw} and I_{ss} were 11.26, 2.18 and 6.86, with standard deviations of 0.59, 0.20 and 0.38, respectively. Based on the I_{ss} value of 6.86 and considering the α factor of 1 (considering cracked zone near the surface of the subgrade) the value of y_s is determined to be 82.3 mm and hence, the construction site covered by such clay is classified as "extremely reactive" based on AS-2870 [36].

In this research, in order to investigate the improvement of resilient modulus characteristics of Clay using recycled glass, three mixtures of clay-glass (CG) with gravimetric RG contents of 10% (CG10), 20% (CG20) and 30% (CG30) were prepared to be compared with Clay samples as the benchmark.

2.2 Compaction and repeated load triaxial testing

The characterization of pavement subgrades in terms of resilient modulus (Mr) is strongly encouraged in the design of new pavements [28]. Values of Mr are typically obtained through repeated load triaxial (RLT) testing on undisturbed or reconstituted cylindrical specimens. In this research, RLT testing was carried out following the AASHTO-T307-99 [37] procedure to simulate repeated traffic loadings on pavements. A vehicle wheel traveling on a pavement structure is known to generate a stress pulse comprising of deviator and confining stress components [38]. Following the AASHTO-T307-99 [37] procedure, a haversine-shaped loading pulse with a loading period of 0.1s and a resting period of 0.9 s was applied. The confining stress and deviator stress for subgrade materials in AASHTO-T307-99 [37] range from 13.8 to 41.4 kPa and from13.8 to 68.9 kPa, respectively. Table 1 presents the confining stress-bulk stress combinations applied in each loading sequence. Sequence No. 0 was the conditioning sequence and included 1000 load repetitions. The test continued with 15 sequences of 100 load repetitions each. The Mr value in each sequence (load combination) is the average of the Mr values achieved in the last 5 load repetitions.

For specimen preparation, clay and RG were initially mixed at the dry state to obtain homogenous 2 kg mixtures. The proportion of the two materials in the mixture was determined by calculating the amount of clay and RG using gravimetric contents as explained in the previous section. For instance, for preparing CG20 samples, 400 g of RG was mixed with 1600 g of clay to form a 2 kg dry sample. Next, the required amount of water, calculated based on the dray mass of the dry sample and the OMC, as presented in Figure 4, was measured and added. The mixtures were blended in a mechanical mixer for 3 minutes to ensure water was thoroughly mixed. The blends were then stored in containers sealed with plastic films for at least 48 hours to cure. Cylindrical RLT specimens were prepared at the optimum moisture content (OMC) and targeting Maximum Dry Density (MDD) achieved under a standard effort of 600 kN-m/m^3 [39]. Based on AASHTO-T307-99 [37], the diameter of the RLT sample should be greater than 5 times the maximum particle size of the specimen. Given the maximum particle size of 4.75 mm in samples, a split mold with a diameter of 50 mm and a height of 100 mm was used for sample preparation. The materials were compacted in the steel cylindrical mold using an electric vibratory hammer, capable of 3000 blows per minute as specified by AASHTO-T307-99 [37], in four layers to achieve the required density. After compaction, specimens were carefully removed from the split mold, and to avoid moisture loss; they were quickly placed on the triaxial cell pedestal. Next, the rubber membrane was placed over the sample and was sealed to the pedestal and the top loading cap with a set of O-rings as recommended by AASHTO-T307-99 [37].

A total of 15 datasets of Mr-σ_c-Θ were obtained through RLT testing on each of the four samples. These data sets were used to determine the model coefficients (k) of two commonly used constitutive Mr predictive models through regression analysis. The k parameters remain the same for a certain type of soil. The models used in this research included the two-parameter (also known as bulk stress) model proposed by Hicks and Monismith [40] and the modified universal model recommended by AASHTO [41] as presented in Equation 3 and Equation 4, respectively. Both models are used for fine and granular soils and the universal model is used in the MEPDG design procedure [28].

$$M_r = k_1.\theta^{k_2} \quad (3)$$

$$M_r = k_1.p_a.\left(\frac{\theta}{p_a}\right)^{k_2}.\left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3} \quad (4)$$

In these equations, *k_1 to k_3* are regression parameters, p_a is the normalizing pressure which is equal to atmospheric pressure, θ is the bulk stress, which is the sum of vertical and horizontal stresses, and τ_{oct} is the octahedral shear stress ($\frac{\sqrt{2}}{3}\sigma_d$, where σ_d is the deviator stress).

2.3 Numerical modeling for stress-strain response analysis

In this research, modeling of the pavement structure and pavement response analysis was carried out using the FlexPAVE™ program. Figure 3 shows the pavement profile that was assigned to the model for stress-strain response analysis. The modeled profile is a typical North Carolina pavement profile that is used for roads with less than 3 million equivalent single axle loads (ESALs) [32]. The profile comprised of a 10.16 cm of asphalt concrete (AC) layer, over a 15.24 cm layer of aggregate base course (ABC) which was over subgrade layers. The pavement profile was analyzed under a 40 KN wheel load that is one of the pair of wheels in the 80 kN single axle load moving at a speed of 80 km/h with a pavement temperature of 23°C.

In the modeled profile, a typical AC type, being S9.5B was used for the surface course. The S9.5B AC is a hot mix asphalt made of PG 58-28 asphalt with a nominal maximum aggregate size of 9.5 mm, and contains 40% of reclaimed asphalt pavement. The stiffness properties of S9.5B are presented in Table 2 in terms of Prony coefficients (ρ_i) extracted from Cho [32]. Prony coefficients are input parameters in FlexPAVE^{TM} and represent the viscoelastic properties of AC. Prony coefficients are obtained by fitting results of dynamic modulus testing [42] at various load frequencies and temperatures to the generalized Maxwell model [32]. A typical quarry material, named "Belgrade" was defined as ABC with the average measured Mr value of 101 kPa following AASHTO-T307-99 [37] procedure and Poisson's Ratio of 0.4 adopted from the report by Chow, et al. [33]. Using RLT test results of Chow, et al. [33] fitted to the Universal Model (Equation 4), k_1, k_2, and k_3 parameters for Belgrade ABC were obtained as 0.863, 0.640, and 0.202, respectively. These parameters can be used for the estimation of Mr values under the loading conditions presented in Figure 3.

Typically, although a minimum depth of 25 cm has been specified for stabilizing the subgrade, the recommended depth is 30 cm [43]. As such, in the models developed in FlexPAVE™, the top 30 cm of the subgrade (Subgrade $_{(top)}$) was defined as an independent layer to assign the untreated subgrade, being Clay, and the three RG treated subgrades, being CG10, CG20, and CG30. The remaining thickness of the subgrade (Subgrade $_{(bottom)}$) was defined as an infinite layer made of Clay.

The Mr value at a specific depth and location of the pavement or subgrade layers depends on the stress state at that point. The stress state is governed by the wheel load, surcharge from the above layers, and at-rest conditions. Therefore, to develop a realistic numerical model, determining the Mr value under the loading conditions that apply to the specific pavement profile using the predictive models is required. The determination of the Mr values can be done through constitutive resilient modulus models, such as those presented in Equations 3 and 4. In this regard, an iterative method was followed as recommended in the Pavement Mechanistic-Empirical software manual [44]. In this iterative approach, the following steps were followed:

1. An initial Mr value was assigned to the layer in question, and the analysis was run. For this, the average of the Mr values obtained through the 15 loading sequences of the RLT test was used as the initial Mr.

2. The vertical and horizontal components of the stress at the mid-depth (as recommended by Huang [45]) of the ABC/subgrade layer of interest were extracted from FlexPAVE^{TM} and added to the surcharge generated by the above layers.

3. The predictive resilient modulus models were used to estimate the Mr corresponding to the specific estimated stress state.

4. The estimated Mr was compared with the Mr initially assigned to the subgrade layer.

5. If the assumed and obtained Mr through steps 1 and 4 were more than 2%, the procedure was repeated as the second iteration. The iterations continued until the assigned and estimated Mr values converged.

3 Results and discussions

This section presents the experimental results, stress-strain response analysis by incorporating the experimental results into the numerical model, and pavement performance comparison analysis. First, RLT test data were obtained and were analyzed by fitting into the two predictive resilient modulus models. Next, resilient modulus properties of Clay and the three mixtures were assigned in the numerical model and the stress-strain response analysis was carried out. Vertical and horizontal strains obtained through the response analysis were next used for comparing the potential rutting and fatigue life of pavement systems constructed on the four types of subgrades.

3.1 Resilient modulus test results

Compaction tests were carried out to determine the OMC and MDD of each sample for the preparation of RLT specimens. Figure 4 shows the relationship between moisture content and dry density of the four mixtures with MDD (t/m^3) and OMC (%) corresponding to each compaction curve presented. Results show that increasing the RG content from 0% to 30% results in a 22% decrease and a 9% increase in the OMC and MDD, respectively. Reduction of the OMC could be attributed to the significantly lower water absorption potential of glass particles compared to Clay [14], as well as greater MDD achieved in samples with higher RG content which led to fewer available voids to be filled with water. Figure 4 also demonstrates that the decreasing and increasing trends of OMD and MDD, respectively, for RG contents up to 30% are approximately linear. This trend could potentially continue, although in a non-linear form, until reaching the RG contents of 40% and 50%. After this, the mixture may transition from a fine-graded blend, in which clay governs the behavior of the mixture, into a coarse-graded blend according to the USCS classification scheme. It is expected that at RG contents > 50%, with an increase in the RG content, the OMC and MDD slightly increase and decrease, respectively. This trend was also observed and reported by Wartman, et al. [24] who carried out compaction tests on clay-aggregate mixtures with coarse contents between 0 and 100%. Investigating the compaction properties of clay-RG mixtures at RG contents > 30% is recommended to be undertaken in future studies.

Figure 5 presents the Mr values obtained in each loading sequence presented in Table 1. Solid lines in Figure 5 represent the average Mr value obtained through the 15 loading sequences. The RLT test results showed that, in general, Mr values increased by increasing the RG content. This can partially be attributed to the higher density (and hence lower void ratio) of specimens with higher RG content. Higher density in one type of material has been repeatedly reported to result in a greater resilient modulus [46, 47]. Another reason for the increase in Mr of mixtures by the addition of RG is the increased percentage of particles with rough surfaces. The greater roughness of particle surfaces in a specimen is known to yield in the higher resilient modulus [48, 49]. By adding 10, 20, and 30% RG to the 11.8% existing natural sand particles with relatively rough surfaces, the percentage of particles with a rough surface in the mixture increased, leading to a greater resilient modulus. In general, the resilient modulus is known to increase when the proportion of fines in a mixture decreases [49].

The solid lines in Figure 5 show greater growth in Mr values by increasing the RG content from 10% to 20% compared to other increments of RG. This is more clearly presented in the last column of Table 3, which presents the average Mr values for the mixes and percentage of increased Mr by adding RG. While the addition of 10% RG to Clay and CG20 results in a 26% and 15% increase in Mr, respectively, the increase of Mr by addition of 10% RG to CG10 is 46%. This can be attributed to the fact that adding 20% RG to the Clay that naturally contains close to 12% sand-size particles results in changing the classification of the soil from "Clay" to "Sandy Clay" [50], and hence, a more significant increase in the Mr values. The percentage of sand-size particles (natural sand + recycled glass) is presented in the second column of Table 3, with CG20 containing more than 30% sand-size particles and hence, classified as "sandy clay" [50].

Figure 6 schematically demonstrates the distribution of sand-size particles in the clay matrix for CG10 and CG20. The increase in RG content together with the presence of natural sand particles result in greater potential for coarse particle-on-particle interactions in CG20 compared to CG10. In CG10 specimens, the coarse particles are too distant to provide particle-on-particle interactions, and hence the behavior of the mixture is dominantly governed by the clay. With the increase in RG content, in addition to the greater possibility of coarse particle interactions, there may be another factor that contributes to a more stable force chain within the clay-aggregate mixture. The clay trapped between the sand-size particles becomes stiffer during compaction and further loadings during RLT testing, and forms bridge-like microstructures between the sand-size particles. This contributes to a greater distribution of external loads within the mixture structure, and hence a more stable force chain is offered as also discussed by Fei [51]. This results in a less compressible specimen and consequently a greater resilient modulus.

Figure 7 illustrates plots of resilient modulus versus maximum axial stress (i.e. deviator stress) for the mixtures studied in this research. Two points can be concluded from the resilient modulus response of the mixtures under various combinations of axial and confining pressures. Firstly, higher confining pressure results in a greater resilient modulus in each specimen. Higher confinement can increase the inter-particle interlocking and internal friction of the particles, and hence, less potential for strains, as explained by Nguyen and Mohajerani [26] and Bhuvaneshwari, et al. [52]. With resilient modulus defined as the ratio of cyclic axial stress to the recoverable strain, reduction of strain can result in greater Mr. Secondly, the increases in axial stress, under the same confining pressure, result in greater Mr. This can be due to the greater stress hardening of the specimens that occurred under 100 repetitions of a greater axial stress as explained by Puppala, et al. [53]. It should be noted that several researchers such as Bhuvaneshwari, et al. [52] and Liu, et al. [47], among others, have reported that the resilient modulus of untreated expansive soils was reduced with an increase in axial stress due to stress softening; however, this contrasts the experimental results on the Clay used in this research. This could be due to the presence of more than 10% sand size particles in the natural untreated soil used in this research.

3.2 Data analysis using predictive models

The validity of the resilient modulus of the two-parameter and modified universal predictive models was studied using the obtained RLT test datasets for all four mixtures. Plots of Figure 8 compare the 60 measured and predicted Mr values. The higher visually evident concentration of data points in the vicinity of the 1:1 line for the modified universal model (Figure 8 (b)) shows that this model can provide a more accurate prediction.

Table 4 presents "k" coefficients of the two-parameter and modified universal models achieved through regression analysis of the test results. The "k" coefficients depend on the material type and physical properties of the material that is tested. Table 4 also shows the coefficient of determination (R^2) and the result of the "goodness of fit" of each model for each mixture following the Witczak, et al. [54] criteria. Witczak, et al. [54] have proposed a subjective criteria for the determination of the "goodness of fit", in which $R^2 \geq 90$, $0.70 \leq R^2 \leq 0.89$, $0.40 \leq R^2 \leq 0.69$, and $0.20 \leq R^2 \leq 0.39$, respectively, represent "Excellent", "Good", "Fair", and "Poor" fit. While the two-parameter model shows "Good" to "Excellent" fit with the experimentally obtained Mr values the modified universal model shows "Excellent" fit with results obtained for all four mixtures. Therefore, for the Mr analysis of the subgrade layers and further response analyses of this study, "k" coefficients obtained for the modified universal model were adopted. In this model, k_1 is proportional to the modulus of elasticity, hence, always a positive value; k_2 should be positive as an increase in bulk stress results in stress hardening of the specimen and accordingly, greater resilient modulus, and since the increase in octahedral shear stress results in stress softening and hence, lower resilient modulus, k_3 should be negative [46]. The k_1, k_2, and k_3 coefficients obtained using the modified universal model are positive, positive, and negative, respectively.

3.3 Stress-strain response of pavement systems

In order to determine the resilient moduli of the mixtures under loading conditions defined in the FlexPAVE™ model, the iterative approach explained in Section 2.3 was followed. In the first iteration, the average Mr values achieved through RLT testing, presented in Table 3, were assigned to the subgrade, and the model was run. In each iteration, the stresses at the mid-point of the 300 mm thick $\text{subgrade}_{(top)}$ layer and those of another point located 150 mm below the $\text{subgrade}_{(top)}$-$\text{subgrade}_{(bottom)}$ interface were extracted. Figures 9 (a) to 9 (c) show the distribution of the vertical stress (σ_z), transverse stress (σ_x), and longitudinal stress (σ_y), respectively, in the depth of pavement models with CG20 as $\text{subgrade}_{(top)}$, as an example. The negative sign of σ_z indicates compressive stress, whereas the positive sign of σ_x and σ_y is an indication of tensile stresses.

Table 5 presents the final estimated values of Mr for each sample as well as the number of iterations carried out until less than 1% difference between assumed and estimated Mr was achieved. The resilient moduli presented in Table 5 were assigned to the subgrade layers of the pavement model, and the stress-strain response analysis was carried out.

Figure 10 compares the strain bulbs formed in two of the models, being the model with $\text{subgrade}_{(top)}$ of Clay and that with $\text{subgrade}_{(top)}$ of CG30. The distribution of the strains indicates overall lower strains in the depth of the pavement profile when the natural subgrade is mixed with RG. In particular, the reduction of horizontal strains (ε_x and ε_y) at the AC-ABC interface, and the reduction of vertical strain (ε_z) are observed in the plots.

The plots shown in Figure 11 demonstrate horizontal strains (transverse and longitudinal) at the surface course-base course interface (101.6 mm below the surface) and vertical strains at the base course-subgrade interface (254 mm below the surface) for all four mixtures. The tensile (horizontal) strain at the interface of the surface layer and the base layer generated due to the traffic loading is a major cause of the fatigue cracking and governs the fatigue life of pavements. The compressive (vertical) strain at the interface of the subgrade and aggregate base or subbase layer is known as rutting strain and controls the rutting life of pavements. Based on plots presented in Figure 11, greater RG content resulted in lower transverse (ε_x) and longitudinal (ε_y) strains that are of tensile nature and lower vertical strain (ε_z).

It is well known that the unbound granular layer (base or subbase) with a higher resilient modulus that overlays the softer subgrade layer controls the overall deformation at the subgrade level by spreading the stress [38]. Similarly, placing a RG-treated layer that is a stiffer layer, over a softer layer of untreated subgrade results in less vertical deformation at the subgrade level, and hence, less potential for rutting at the surface, as can be observed in the plots of Figure 10. The decreased magnitude of tensile strains at the AC-ABC interface as a result of increased RG content can be attributed to the lower displacement of the pavement at this depth. Figure 12 (a) demonstrates the typical displacement behavior of flexible pavements under traffic loads, as well as the location of critical compressive vertical (ε_z) and tensile horizontal (ε_t) strains that govern the fatigue and rutting distresses, respectively. Figure 12 (b) compares the vertical displacement at the surface course-base course interface for models with clay subgrade and CG30 over clay subgrade. Considering the greater magnitude of the vertical deformation (D_z) in the pavement model founded on Clay compared to that founded on CG30, and accordingly, a greater sag induced in the surface layer, a higher tensile strain was expected at the bottom of asphalt concrete, as is the case presented in Figures 10 and 11.

3.4 Discussion on the pavement performance analysis

The main objective in pavement design is to provide sufficient thickness of structural layers for the service loads and ground conditions to resist structural distresses. The two major pavement distress types are fatigue cracking and rutting. While a major cause for rutting is the accumulation of vertical strains at the subgrade level, fatigue cracks occur due to the horizontal tensile strains at the bottom of the surface course. Equations 5 and 6 are widely used for the determination of fatigue (N_f) and rutting life (N_r) of pavements in terms of standard axle load repetitions, respectively [45].

$$N_f = f_1 \times \varepsilon_t^{-f_2} \times E^{-f_3} \quad (5)$$

$$N_r = f_4 \times \varepsilon_z^{-f_5} \quad (6)$$

In these equations, f_1 to f_5 are regression coefficients, ε_t and ε_z are the tensile (horizontal) and compressive (vertical) strains, respectively, and E is the average elastic modulus of the surface layer. The average elastic modulus of S9.5B surface layer for the vehicular speed of 80 km/h and the pavement temperature of 23°C is 4,224 MPa. Table 6 presents the f_1 to f_5 coefficients [45] proposed by two well-known organizations, being the Asphalt Institute, USA and the Transport and Road Research Laboratory, UK (Currently TRL).

It should be noted that values of N_r and N_f are not the real-life allowable number of repetitions, as coefficients f_1 to f_5 are proposed based on laboratory tests under conditions that may be different from the field [45]. The realistic allowable number of repetitions requires correction factors by comparing the field and laboratory conditions, which is out of the scope of this research. However, the obtained N_r and N_f values are sufficient for comparison purposes. The current research investigates whether the rutting and fatigue performance of pavements improves by stabilizing natural clay subgrades using RG. For the comparison analysis, the pavement model with clay subgrade was taken as the reference. The percentages of difference between N_f and N_r of other pavement models with those of the pavement system with clay subgrade were calculated and presented in Table 7.

Results presented in Table 7 show that the addition of even 10% of RG to Clay leads to 40 to 57% improved fatigue life and 22 to 25% improved rutting life. In general, the greater fine content in the mixture is known to result in a lower resilient modulus [49] and permanent deformation [55]. Therefore, introducing coarse particles (RG) in the clay matrix contributes to lower vertical and horizontal strains at the subgrade level and hence, an improved fatigue and rutting life of the pavement system. Another point obtained from Table 7 is the dramatic increase of fatigue life and rutting life by increasing the RG content from 10% to 20%. This is attributed to the fact that the addition of 20% sand-size particles to Clay samples results in the blend to transition from "Clay" classification to "Sandy Clay" classification and thus, a significant improvement of the subgrade mechanical characteristics. As illustrated in Figure 6, greater potential for coarse particle interaction, as well as the formation of clay micro-bridge structures in CG20 compared to CG10 result in a more stable force chain within the subgrade. This results in lower compressibility and hence, lower deformations, which results in lower potential for rutting and fatigue distresses.

4 Conclusions

In this research, the improvement of the expansive clay subgrade properties using recycled glass (RG) was investigated. First, density-moisture relationships and resilient modulus responses of untreated Clay and Clay-RG mixtures were investigated. Next, using numerical modeling techniques, the stress-strain response analysis of the pavement systems constructed on the untreated subgrade and those founded on the RG-treated subgrade was carried out. Using the outcomes of the response analysis, the rutting and fatigue performance of the pavement systems on untreated and treated subgrades were compared. The following conclusions were made based on the outcomes and analyses of this study.

- Increasing the RG content in the Clay-RG mixtures resulted in a greater maximum dry density. The addition of 30% RG resulted in a 9% increase in maximum dry density and a 22% reduction of the optimum moisture content required for field compaction and preparation of the natural subgrade.
- The experimentally obtained resilient moduli of samples increased by increasing the RG content in the mixtures.
- The most significant increment in the resilient modulus occurred when the RG content was increased from 10% to 20%. This could be attributed to the transition of "clay" soil, which naturally contained about 12% sand particles into a "sandy clay" soil by adding 20% sand-size RG to the Clay.
- Increasing the RG content in the RG-treated subgrade layer led to a reduction of up to 27% in compressive strains at the base course-subgrade interface and a reduction of up to 75% in tensile strains at the bottom of the pavement surface layer.
- Decreased compressive and tensile strains by improving the clay subgrade using recycled glass as mentioned above, resulted in increased fatigue life and rutting life of the pavement structure.

In this research, a combination of experimental results and numerical analysis of the pavement behavior revealed that the addition of recycled glass as a construction material can significantly improve the behavior and performance of pavement subgrades. The outcomes of this research aim to promote the application of sustainable construction materials and methods as alternatives to the relatively costly and environmentally harmful traditional approaches for the subgrade treatment. Highway designers and contractors normally prefer the traditional construction approaches due to decades of experience with traditional soil stabilizers. Resolving ambiguities and uncertainties of the performance of non-traditional methods through rigorous experimental and analytical research works can improve the construction industry's confidence in such sustainable approaches.

5 Acknowledgments

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Tables

Table 1. Stress combinations in RLT test for subgrade soils as recommended by AASHTO-T307-99 [37]

Sequence Number	0	1	2	3	4	5	6	7
Confining stress, σ_c (kPa)	41.4	41.4	41.4	41.4	41.4	41.4	27.6	27.6
Bulk stress, ϴ (kPa)	151.9	138.0	151.9	165.5	179.5	193.2	96.5	110.4
Sequence Number	8	9	10	11	12	13	14	15
Confining stress, σ_c (kPa)	27.6	27.6	27.6	13.8	13.8	13.8	13.8	13.8
Bulk stress, ϴ (kPa)	123.8	137.9	151.7	55.3	69.0	82.9	96.7	110

Table 2. Prony coefficients for asphalt concrete

ρ_i (s)	E_i (kPa)	ρ_i (s)	E_i (kPa)
2.00E+11	2,380	2.00E-01	3,173,760
2.00E+10	4,130	2.00E-02	3,761,120
2.00E+09	7,330	2.00E-03	3,104,720
2.00E+08	13,420	2.00E-04	2,497,320
2.00E+07	25,710	2.00E-05	1,851,650
2.00E+06	52,450	2.00E-06	1,323,710
2.00E+05	115,270	2.00E-07	917,490
2.00E+04	270,180	2.00E-08	624,310
2.00E+03	641,430	2.00E-09	419,320
2.00E+02	1,401,350	2.00E-10	279,300
2.00E+01	2,533,100	2.00E-11	185,000
2.00E+00	3,595,620	E_∞	60,490

Table 3. Average values of Mr for each mix and percentage of increased Mr by adding RG

Blend	Sand size particles (%)	Average Mr (MPa)	Mr increase compared to Clay (%)	Mr increase by adding 10% RG (%)
Clay	11.8	52.0	0	0
CG10	21.8	65.5	26	26
CG20	31.8	95.9	84	46
CG30	41.8	110.7	113	15

Table 4. The model k coefficients and evaluation of the "Goodness of Fit"

Model	Two-parameter model				Modified universal model			
Parameter	Clay	CG10	CG20	CG30	Clay	CG10	CG20	CG30
k_1	5.67	1.61	15.24	30.27	0.514	0.631	0.929	1.121
k_2	0.46	0.77	0.38	0.27	0.522	0.868	0.418	0.320
k_3	-	-	-	-	-0.529	-0.807	-0.298	-0.448
R^2	0.939	0.936	0.946	0.821	0.989	0.982	0.969	0.911
Goodness of fit	Excellent	Excellent	Excellent	Good	Excellent	Excellent	Excellent	Excellent

Table 5. Estimated Mr values used for modelling

Sample	Number of iterations	Estimated Mr for:	
		Subgrade (top)	Subgrade (bottom)
Clay	3	47.2	30.3
CG10	3	54.7	32.0
CG20	3	85.4	31.5
CG30	3	95.4	31.6

Table 6. Regression coefficients used in this research (adopted from Huang [45])

Organization	Proposed Regression Coefficients				
	f_1	f_2	f_3	f_4	f_5
Asphalt Institute	0.0795	3.291	0.854	1.36E-09	4.48E+00
Transport and Road Research Laboratory	1.7E-10	4.32	0	6.18E-08	3.95E+00

Table 7. The percentage of difference in Nf and Nr of models with CG subgrades compared to the benchmark model with Clay subgrade

Criteria	Agency/Organization	Difference from Clay Subgrade (%)		
		CG10	CG20	CG30
Fatigue life (X direction)	Asphalt Institute	40	337	499
	Transport and Road Research Laboratory	55	593	949
	Average	47	465	724
Fatigue life (Y direction)	Asphalt Institute	41	357	534
	Transport and Road Research Laboratory	57	636	1028
	Average	49	497	781
Rutting life	Asphalt Institute	25	147	190
	Transport and Road Research Laboratory	22	122	156
	Average	23	135	173

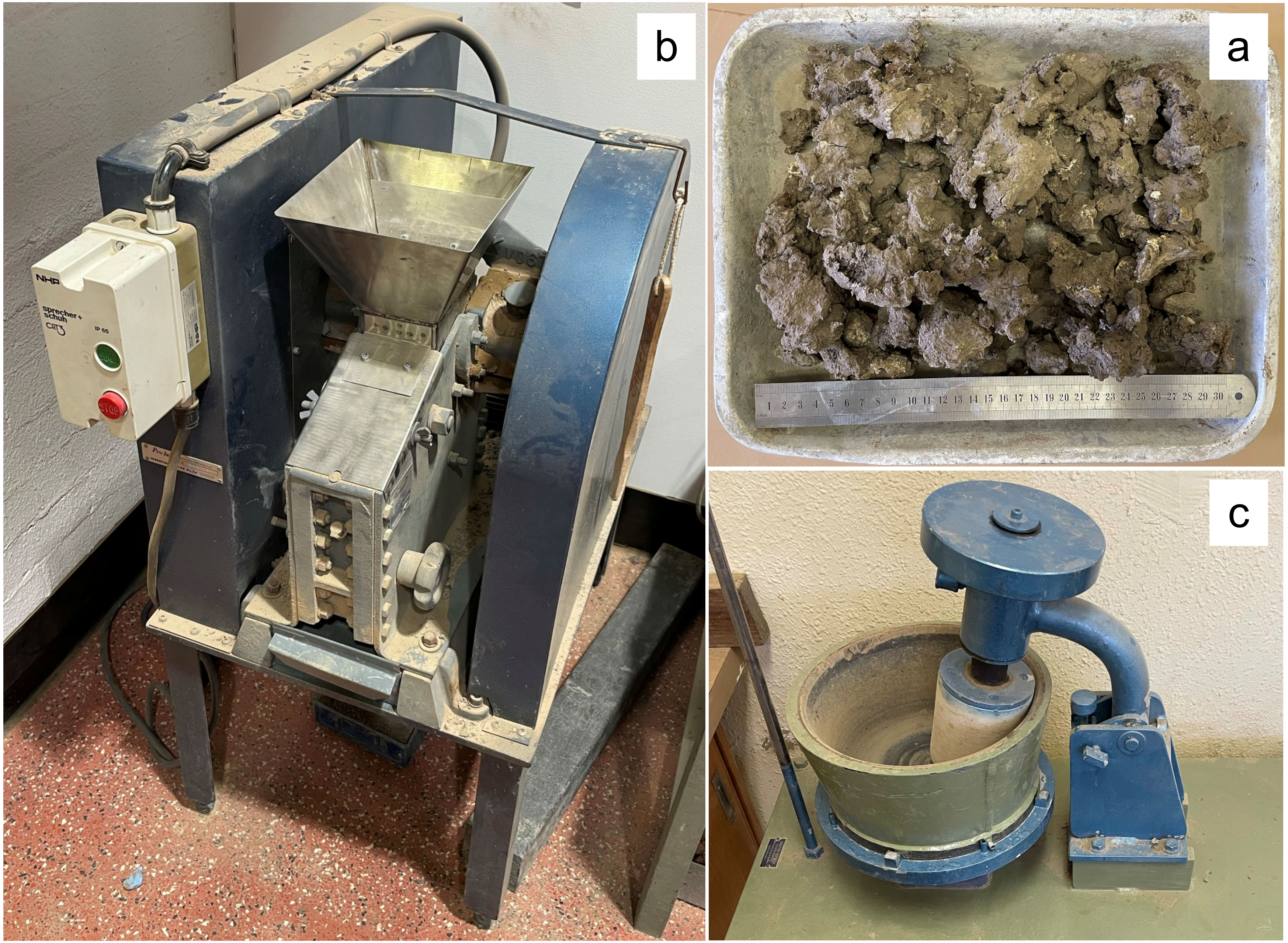
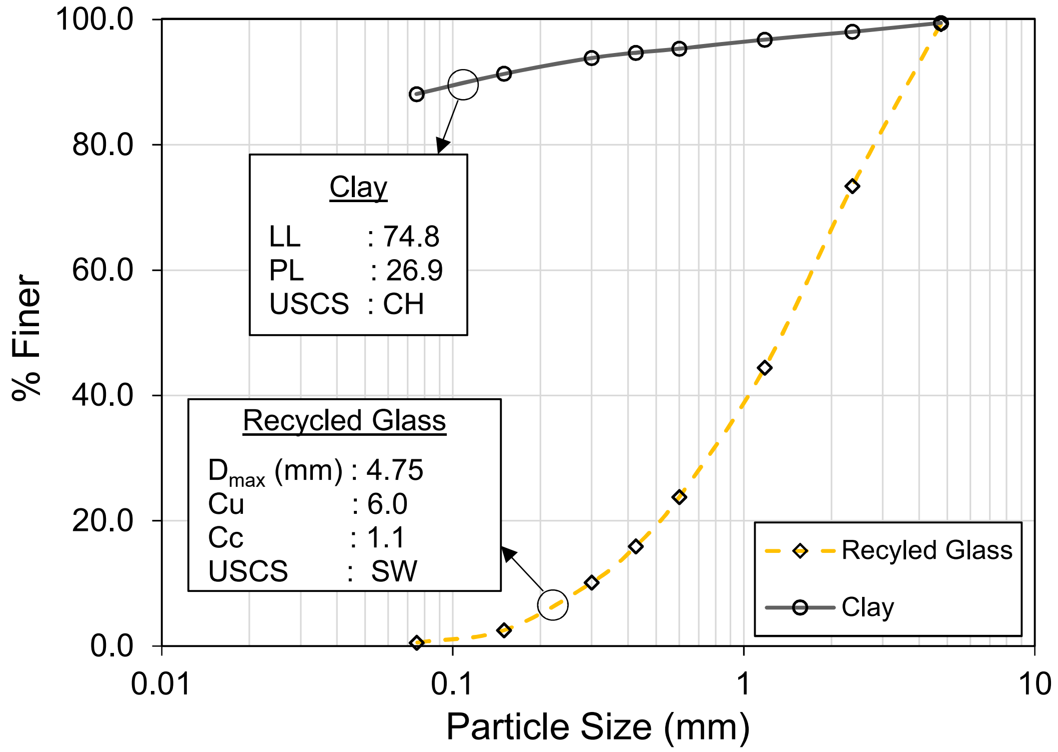

b
a
c
NHP
sprecher+ schuh
IP 65
STOP

Figure 2

Click here to access/download;Figure;Fig. 2-PSD_Classification.pdf

Click here to view linked References



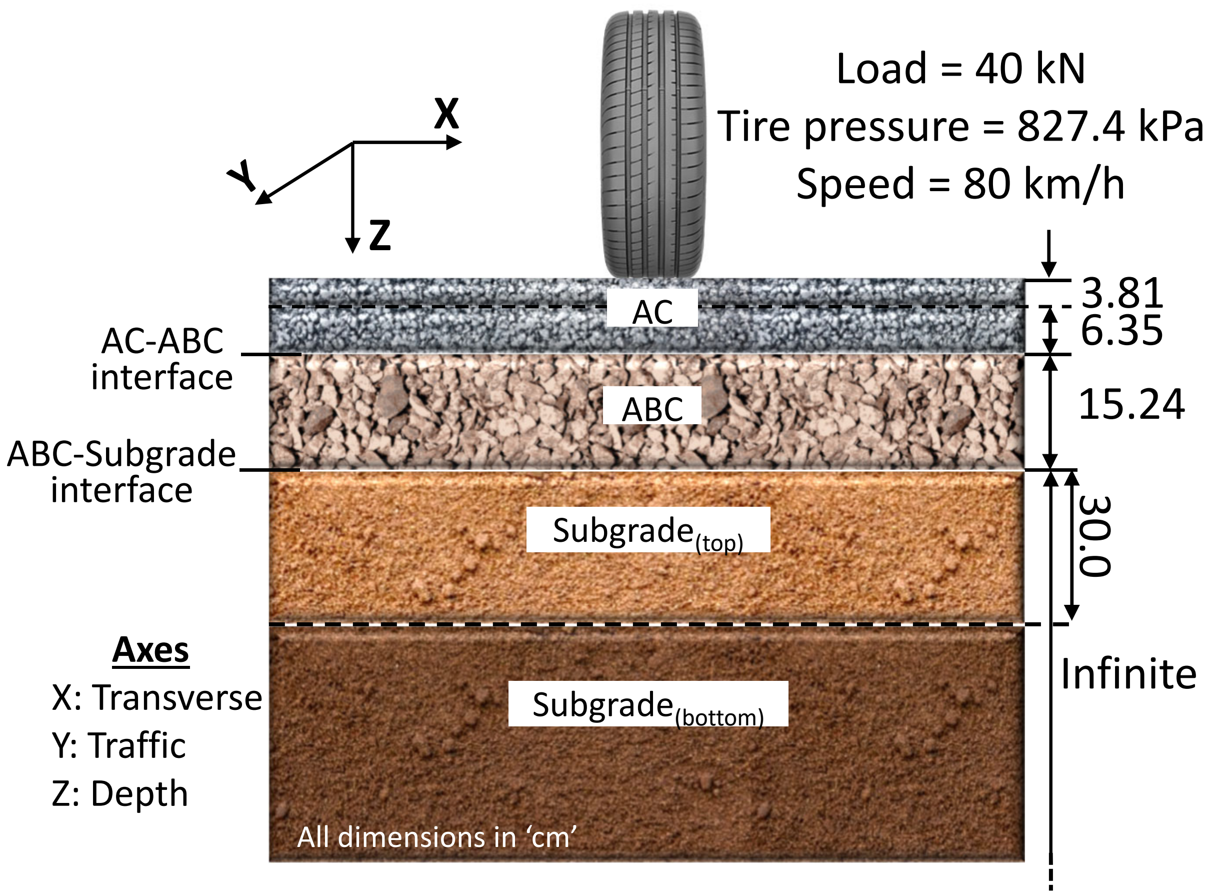

Load = 40 kN
Tire pressure = 827.4 kPa
Speed = 80 km/h
X
Y
Z
AC
ABC
Subgrade(top)
Subgrade(bottom)
AC-ABC interface
ABC-Subgrade interface
3.81
6.35
15.24
30.0
Infinite
Axes
X: Transverse
Y: Traffic
Z: Depth
All dimensions in 'cm'

Figure 4

Click here to view linked References

Click here to access/download;Figure;Fig. 4-Compaction_Results.pdf

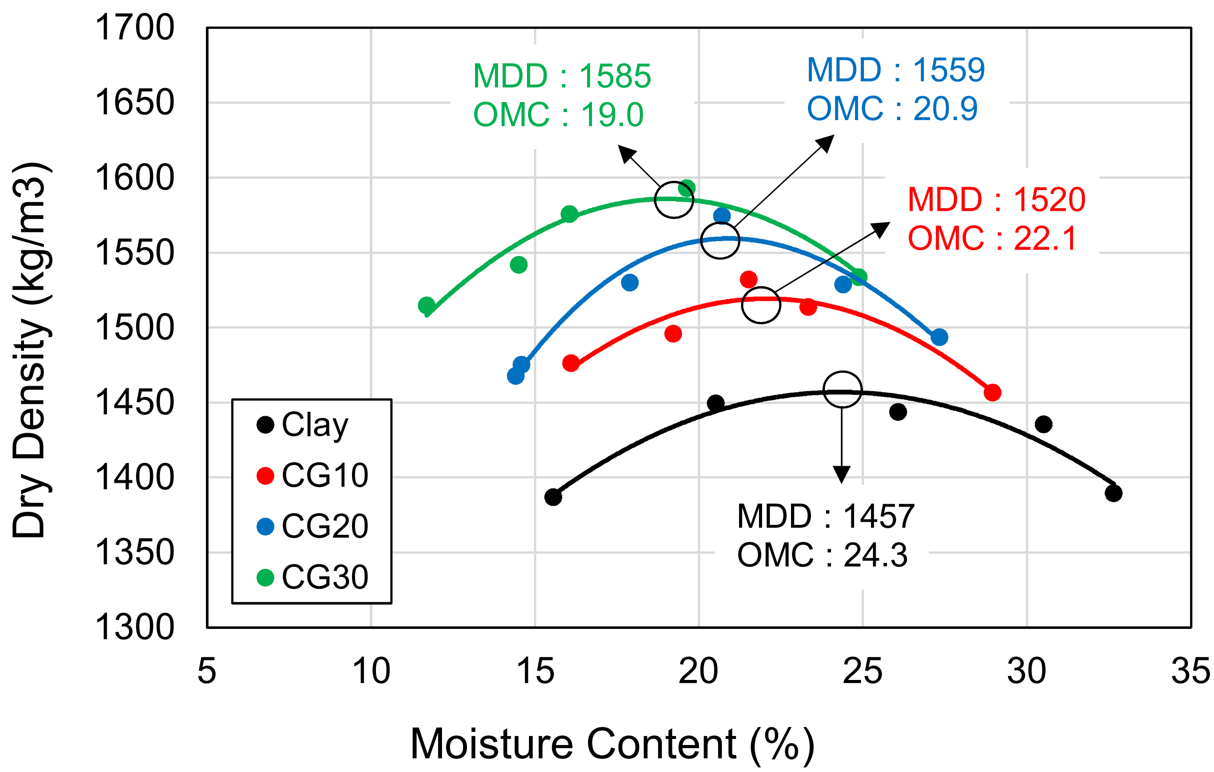
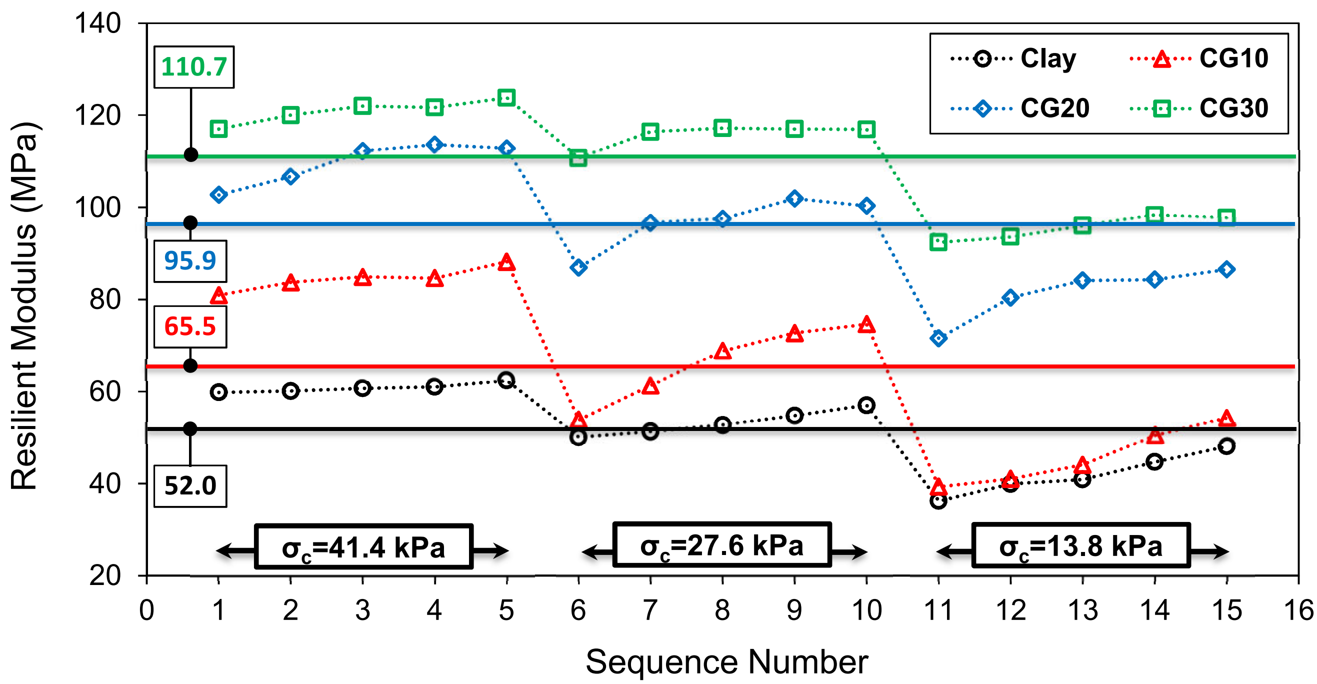
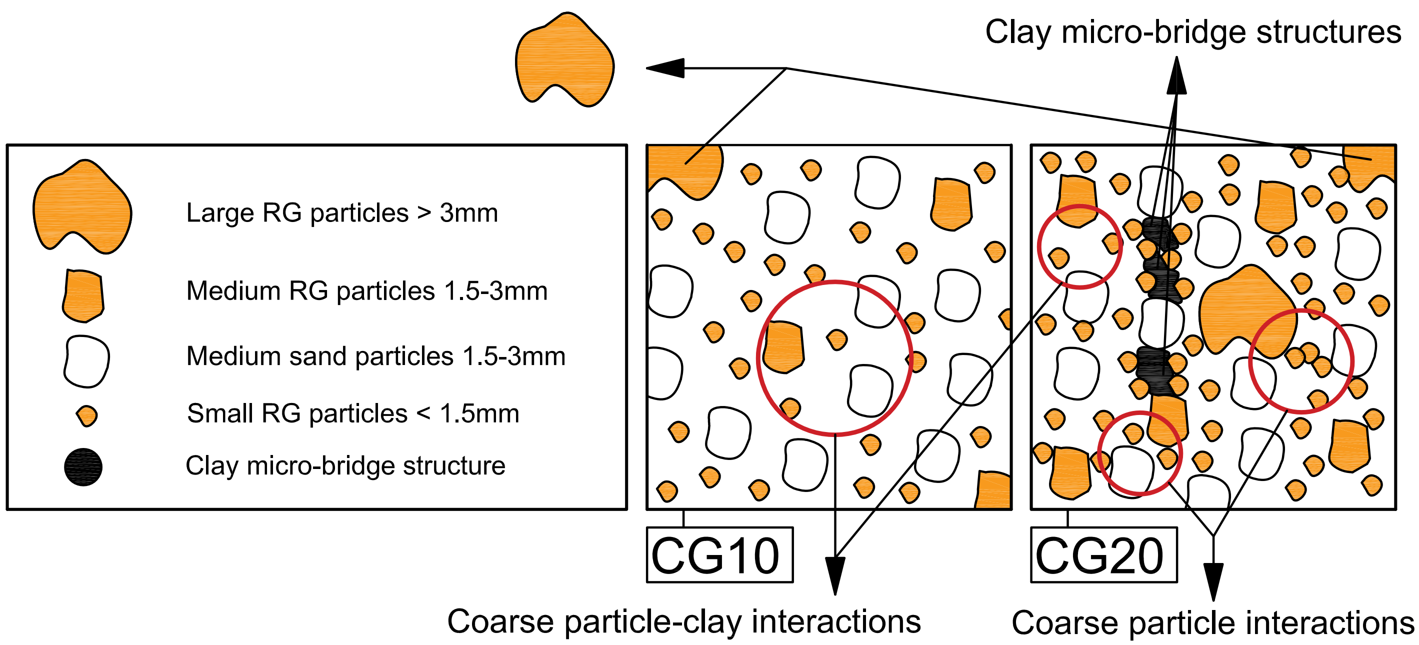


Figure 5

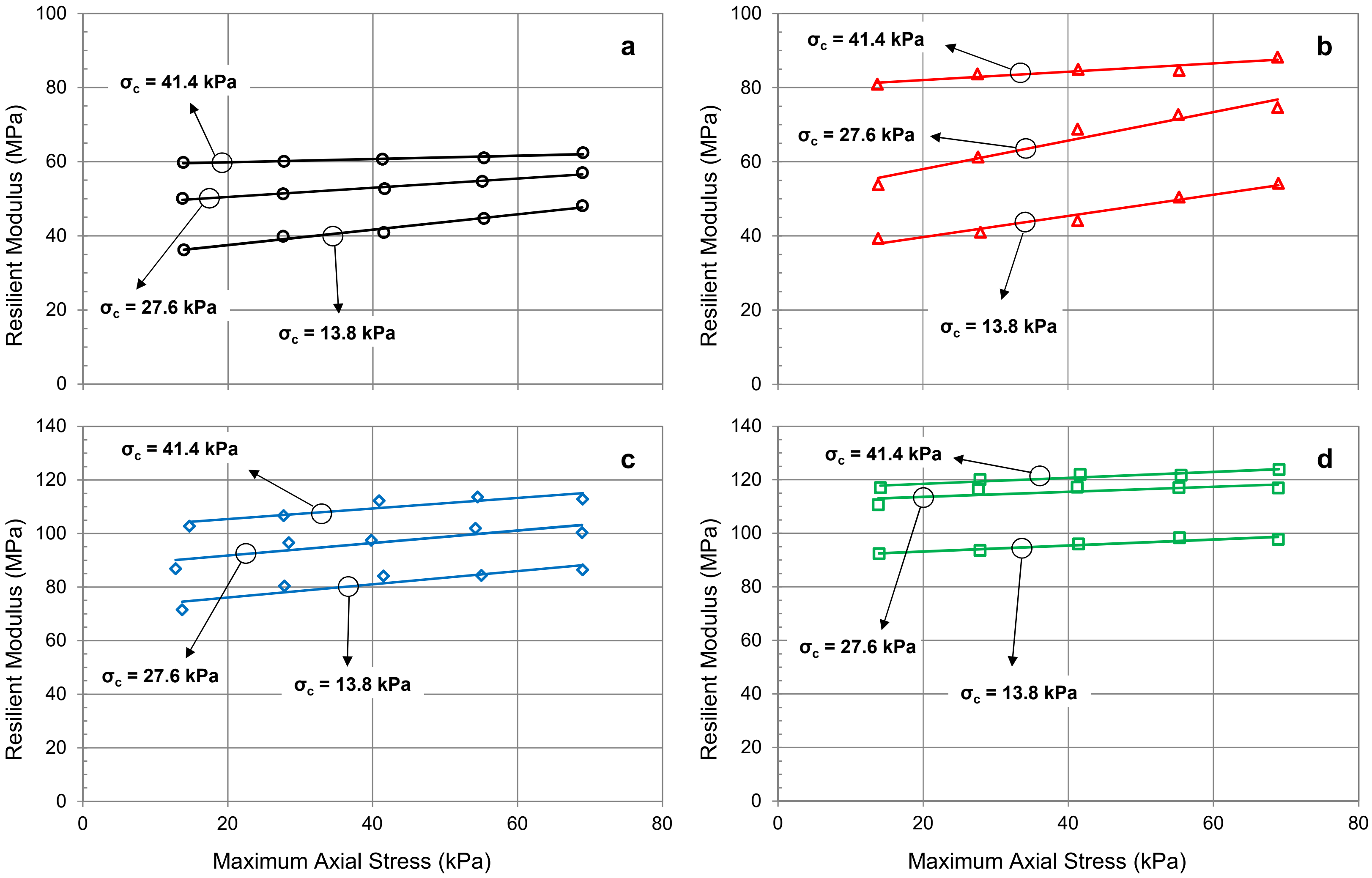


Resilient Modulus (MPa)
Sequence Number
Clay
CG10
CG20
CG30
110.7
95.9
65.5
52.0
σc=41.4 kPa
σc=27.6 kPa
σc=13.8 kPa
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

Figure 6



Clay micro-bridge structures
Large RG particles > 3mm
Medium RG particles 1.5-3mm
Medium sand particles 1.5-3mm
Small RG particles < 1.5mm
Clay micro-bridge structure
CG10
CG20
Coarse particle-clay interactions
Coarse particle interactions


a
b
c
d
σc = 41.4 kPa
σc = 27.6 kPa
σc = 13.8 kPa
Resilient Modulus (MPa)
Maximum Axial Stress (kPa)

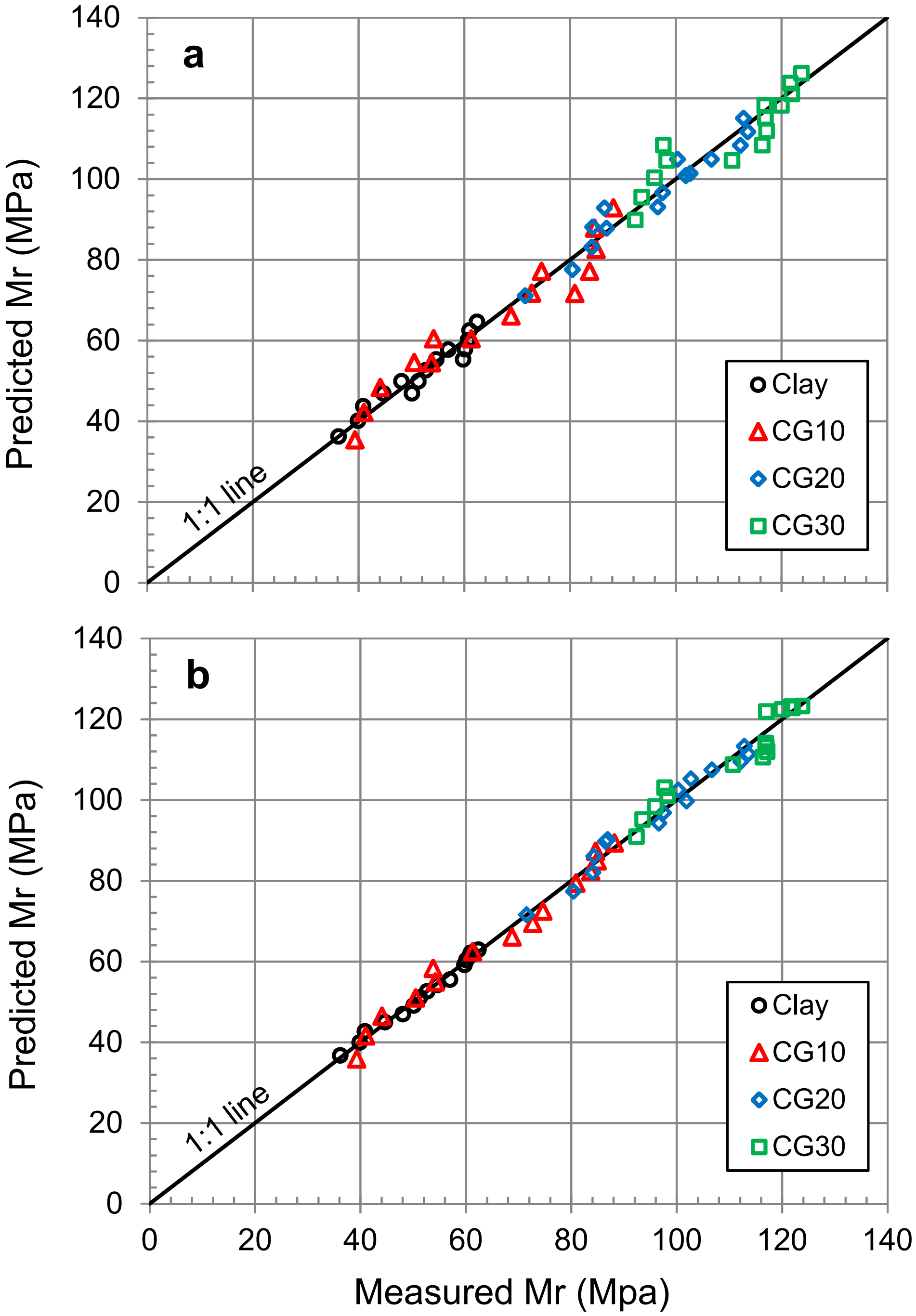
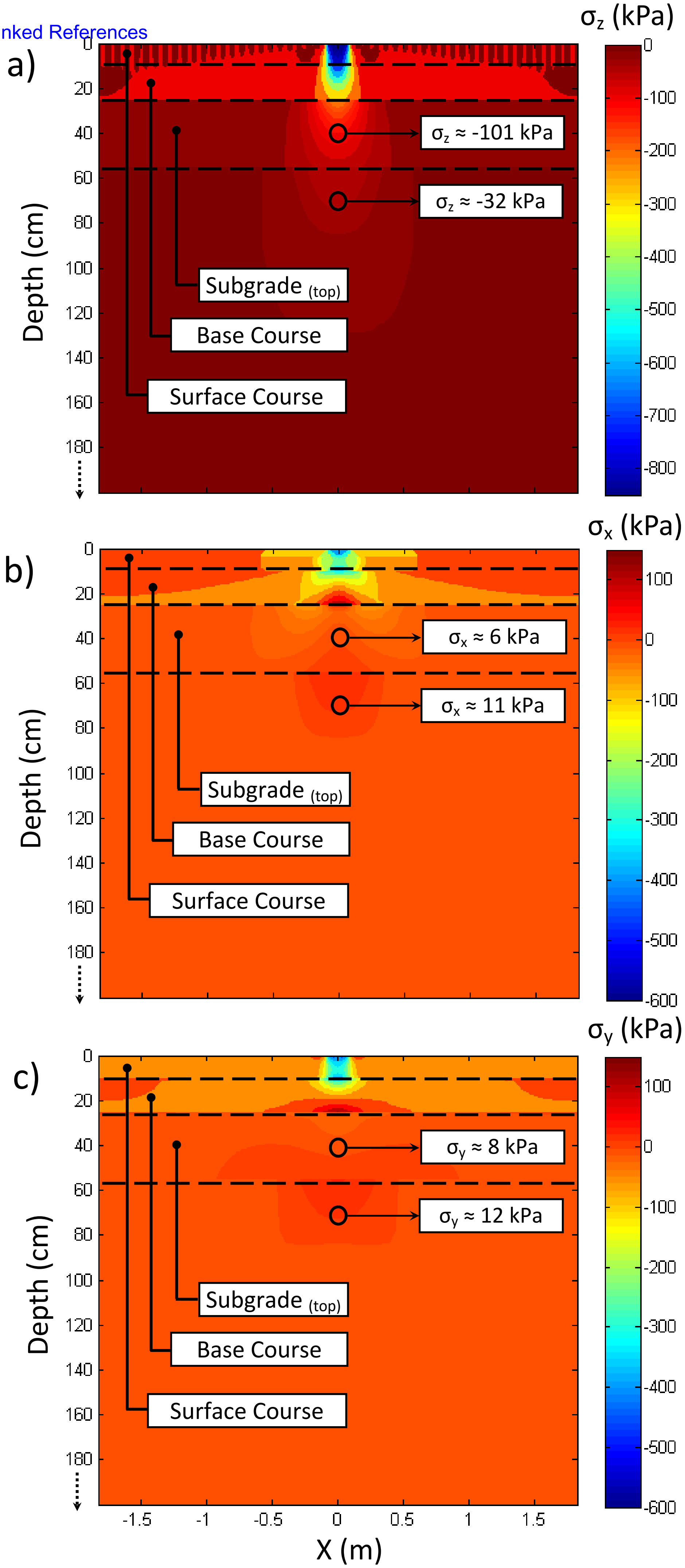
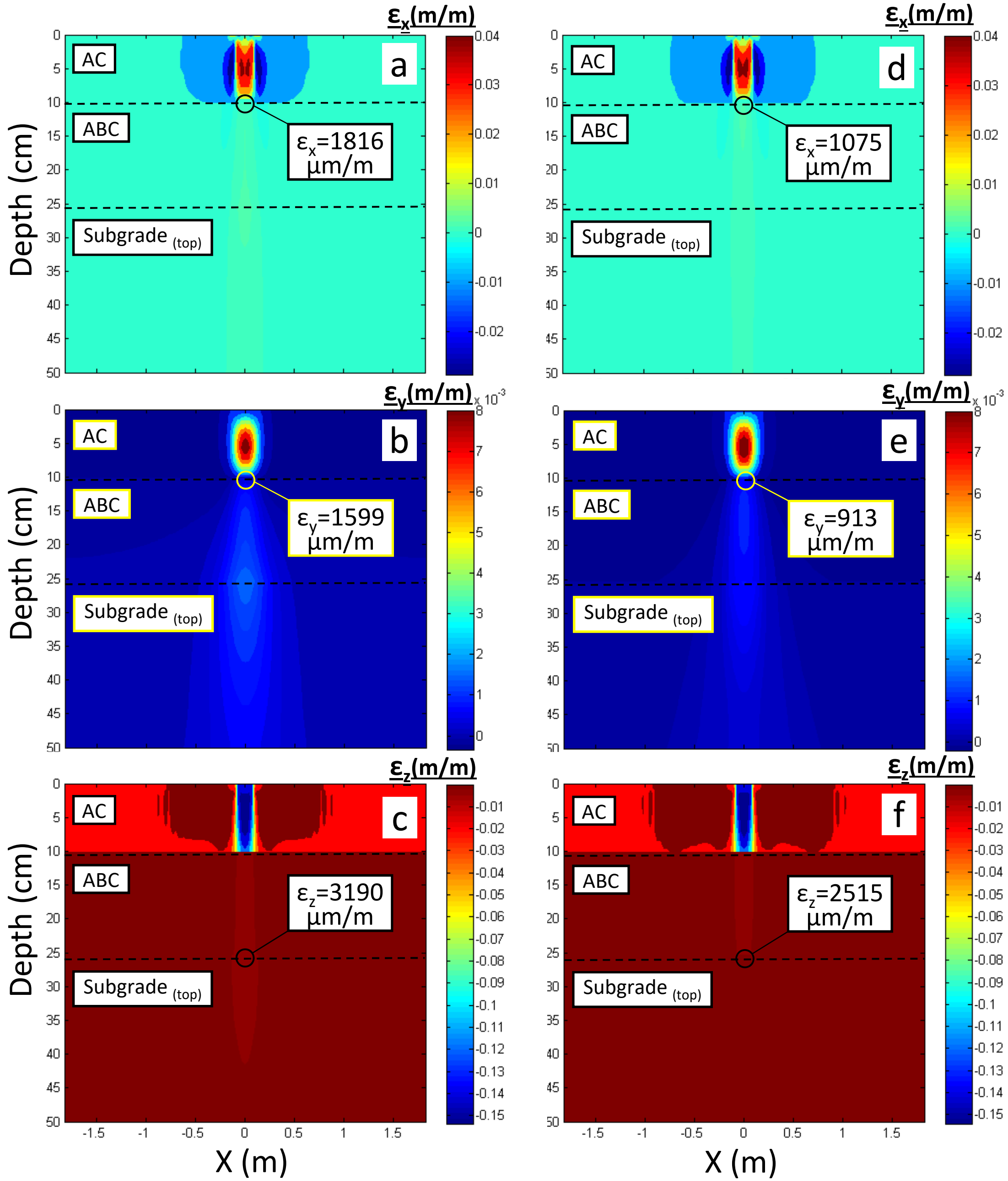

a
b
Predicted Mr (MPa)
Measured Mr (Mpa)
1:1 line
Clay
CG10
CG20
CG30
0
20
40
60
80
100
120
140

Figure 9


a)
σz (kPa)
0
-100
-200
-300
-400
-500
-600
-700
-800
σz ≈ -101 kPa
σz ≈ -32 kPa
Subgrade (top)
Base Course
Surface Course
Depth (cm)
0
20
40
60
80
100
120
140
160
180
b)
σx (kPa)
100
0
-100
-200
-300
-400
-500
-600
σx ≈ 6 kPa
σx ≈ 11 kPa
Subgrade (top)
Base Course
Surface Course
Depth (cm)
c)
σy (kPa)
100
0
-100
-200
-300
-400
-500
-600
σy ≈ 8 kPa
σy ≈ 12 kPa
Subgrade (top)
Base Course
Surface Course
Depth (cm)
-1.5
-1
-0.5
0
0.5
1
1.5
X (m)



a
b
c
d
e
f
AC
ABC
Subgrade (top)
εx=1816 µm/m
εy=1599 µm/m
εz=3190 µm/m
εx=1075 µm/m
εy=913 µm/m
εz=2515 µm/m
εx(m/m)
εy(m/m)
εz(m/m)
x 10-3
Depth (cm)
X (m)

Figure 11

Click here to view linked References

Click here to access/download;Figure;Fig.11-H_V_Strain_vs_time.pdf

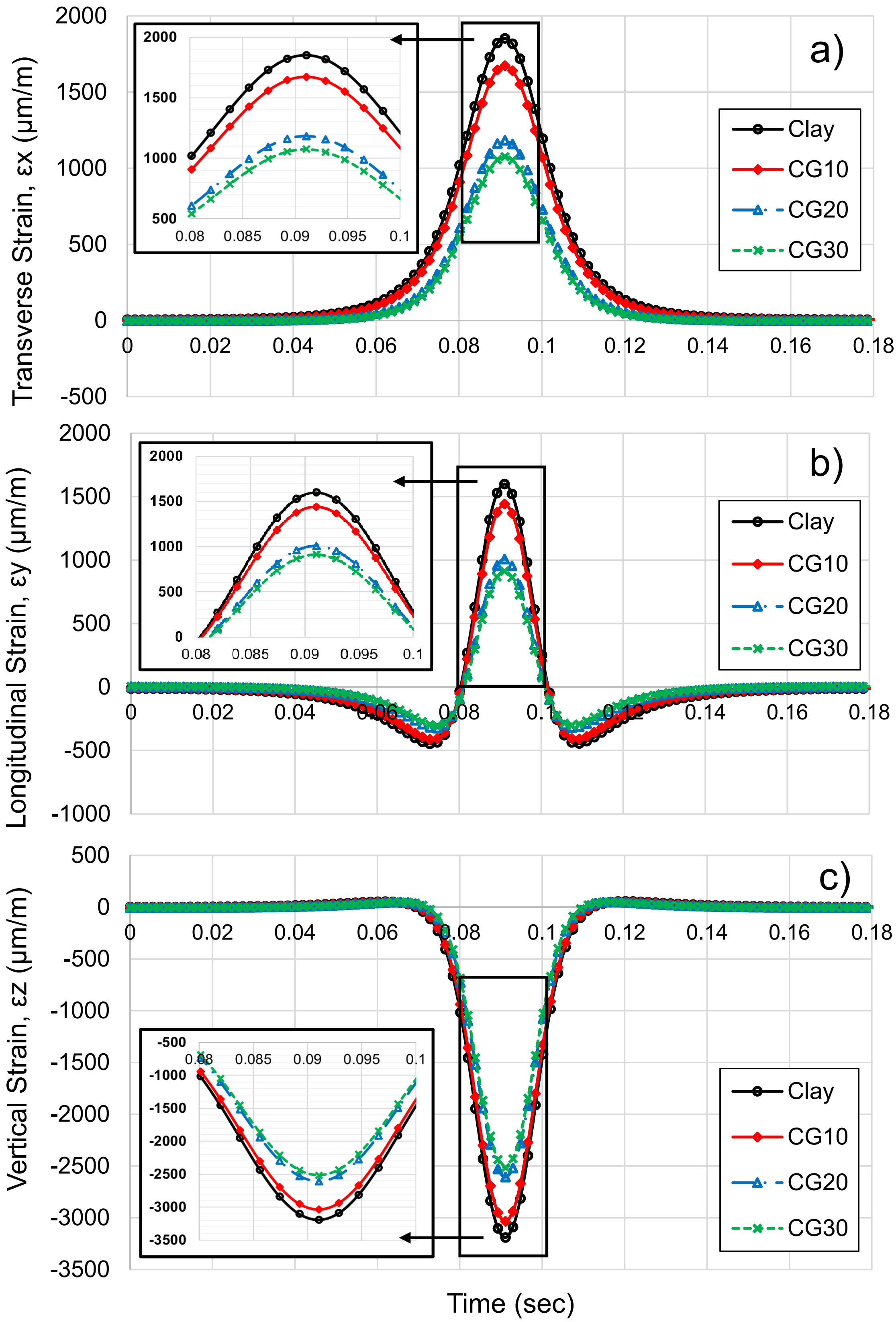
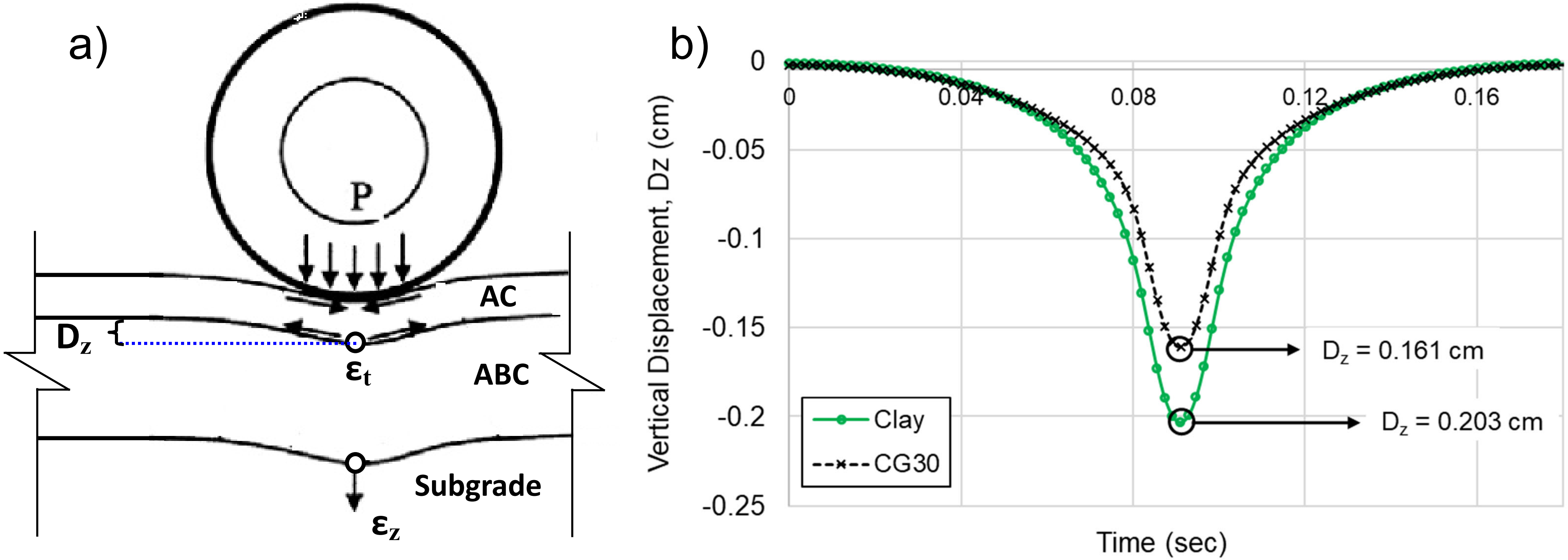


Figure 12


a)
P
AC
Dz
εt
ABC
Subgrade
εz
b)
0
-0.05
-0.1
-0.15
-0.2
-0.25
0
0.04
0.08
0.12
0.16
Vertical Displacement, Dz (cm)
Time (sec)
Dz = 0.161 cm
Dz = 0.203 cm
Clay
CG30