Effects of quantity and plasticity of fine particles on the workability and resilient behaviour of aggregate-soil mixtures for granular pavement layers

Effects of Quantity and Plasticity of Fine Particles on the Workability and Resilient Behaviour of Aggregate-Soil Mixtures for Granular Pavement Layers

Journal:	*Road Materials and Pavement Design*
Manuscript ID	Draft
Manuscript Type:	Original Scientific Paper
Keywords:	aggregate-soil mixture, fine content, plasticity index, compaction, resilient modulus

Effects of Quantity and Plasticity of Fine Particles on the Workability and Resilient Behaviour of Aggregate-Soil Mixtures for Granular Pavement Layers

The paper presents the results of a laboratory investigation seeking to quantify the effects of fine quantity and type on the workability and resilient response of aggregate-fine soil mixtures for subbase formation. Laboratory devices were used to simulate in-field conditions and compact samples and evaluate their resilient response under pulsing loads. Four different types of fines passing through a 63 mm sieve (one non-plastic silt and three silty-clays exhibiting plastic behaviour) in three different quantities (5.4, 10.8 and 16.2%) were combined with coarser aggregates while maintaining the same grading distribution. A variant of the generalized model proposed in the Mechanistic Empirical Pavement Design Guide was calibrated to distinguish the contribution of fine type to resilient modulus from that of moisture content and suction. The results evidenced an improvement in mixture workability during the compaction process when using plastic fines instead of non-plastic ones. However, the quantity of plastic fines should be limited (lower than 10.8% within the aggregate skeleton) in order to achieve a high compaction level and to attain a hardening resilient behaviour rather than the softening one observed in the case of granular materials containing non-plastic fines.

Keywords: aggregate-soil mixture; fine content; plasticity index; compaction; resilient modulus

Word count: 7300

Introduction

The construction and maintenance of road and airport pavements, and railway tracks require large volumes of selected granular materials (GMs) for the formation of granular subbases, as well as granular base and formation layers for railways. GMs have to satisfy grading requirements, which can be met by milling rocks and sediments, and/or blending aggregates and soils of different size distributions and origins. Furthermore, GMs must be sufficiently workable to facilitate the action of rollers which improve the layer structural properties, and stiff enough to distribute and dissipate stresses in the subgrade, enabling the whole multilayer pavement system to work properly (Lekarp, Isacsson, & Dawson, 2000; Uthus, Hoff, & Horvli, 2005; Ekblad & Isacsson, 2008; Pratibha, Sivakumar Babu, & Madhavi Latha, 2015).

The inclusion of fine particles in a coarser aggregate skeleton may also occur incidentally through mechanisms like (a) *interpenetration* between adjacent layers with different grading distributions, (b) the *pumping* of water mixed with fine particles through open joints or cracks in the pavement, and (c) the *diffusion* of fine-water due to capillarity and water table movement (Hajek, Kazmierowski, Sturm, Bathurst, & Raymond, 1991; Huang, 2003; Giroud, 2009; Duong et al., 2014). In all these mechanisms, a slight modification of the finest part of the grading curve occurs when fines invade the pores between larger grains.

The properties of GMs also depend on the characteristics of fine particles. When they are of limited proportions and combined with moisture, fines (smaller than 500 μm) provide cohesion to GMs, acting as stabilizing agents. Yoder & Witczak (1975) coined the term “binder-soil” to indicate this specific role assumed by fines in a GM. They identified the positive contribution of fines in terms of stability as per the California Bearing Ratio test, specifically when the quantity of fines smaller than 75 mm was between 6 and 8% of the solid mass. In such proportions, fines fill the voids between larger particles conferring stability and stiffness to the GMs.

ASTM D1241 (American Society for Testing Materials, 2015b) suggests that the ratio of the percentage of material passing at 75 μm to that at 0.425 mm (called the “dust ratio” - *DR*) should not exceed 0.67. It also provides maximum plasticity index (PI) and liquid limit (LL) values of 6 and 25 respectively. Osouli, Salam, & Tutumluer (2016) and Osouli, Salam, Tutumluer, & Shoup (2017) showed that the effect of the *DR* on strength is highly dependent on the proportion of fines lower than 75 μm.

Soliman & Shalaby (2015) investigated the effects of the percentage of fine particles in six different GM gradations obtained from two sources of limestone and gravel, prepared with two moisture contents, and with a *DR* ranging between 0.30 and 0.76, on the permanent deformation accumulated in triaxial conditions. They found that limestone with a fine content of 4.5%, and gravel with a fine content of 9% performed better in terms of permanent deformation accumulation than materials with other fine content levels.

Figure 1A reports the effects of fine plasticity and water content (*w*), on the dry density and resilient modulus of GMs, which constitute fundamental properties for construction and in-service performances (Monahan, 1994).

Notwithstanding that Mishra, Tutumluer, & Butt (2010) shows no significant differences in maximum dry density ($\gamma_{d\,max}$) values for materials with non-plastic or plastic fines, Cerni & Camilli (2011) and Mokwa, Cuelho, & Browne (2008) observed that a GM with plastic fines exhibits a lower $\gamma_{d\,max}$, and requires a larger amount of water to be workable. Furthermore, plastic fines reduce the sensitivity of dry density to fluctuations in the amount of water.

Resilient modulus values (*RM*), according to the Enhanced Integrated Climatic Model (EICM) included in the Mechanistic-Empirical Pavement Design Guide (American Association of State Highway and Transportation Officials, 2015), are strongly related to the moisture condition and to the plasticity of the GM.

This finding is illustrated in Figure 1B, with a representation of the RM/RM_{opt} trend between the *RM* at the generic saturation degree (*S*) and the modulus at saturation conditions (RM_{opt}) corresponding to the optimal water content (S_{opt}).

[FIGURE 1 near here]

When naturally present (i.e. silty and clayey gravels) or added in small amounts to coarse aggregates (i.e., aggregate-soil mixtures) (Rollings & Rollings, 1995), fines influence the matric suction (Fredlund & Rahardjo, 1993) leading to an apparent cohesion, which is partially responsible for the high increase in *RM* with respect to the optimal moisture conditions.

The investigation of Duong et al. (2016) confirms that the *RM* of ballast contaminated by subgrade soils forming the "interlayer" in a railway track increased due to very fine particles in unsaturated conditions, while close to saturation a sensible decrease, especially for high quantities of fines, was observed.

Lekarp et al. (2000) commented on the difference of opinion vis-à-vis the effects of fine quantity and type on resilient modulus. Early contributions indicated that an increase in the quantity of fines had a limited effect on or led to a slight reduction in *RM* (Thom & Brown, 1987; Kamal, Dawson, Farouki, Hughes, & Sha'at, 1993). In contrast, Barskale & Itani (1989) observed a significant increase in *RM* passing from a quantity of 0 to 10%. These inconsistencies may be attributed to differences in fine quantity and type combined with differences in water content. Moreover, many other experimental results involved soil samples with different gradations and *DRs*, in which constitutive and compositional variables were not always strictly controlled and might have influenced the resilient modulus.

Mishra, Tutumluer, & Butt (2010) and Mishra, Tutumluer, & Xiao (2010) investigated the effects of fines on the RM of engineered unbound aggregates, with fine quantity levels ranging between 0 and 16%, two different fines with a *PI* of 0 and 12%, in combination with three moisture contents around the optimum. Measured *RM* data were then modelled according to the generalized model proposed in the Mechanistic Empirical Pavement Design Guide (American Association of State Highway and Transportation Officials, 2015):

$$\mathrm{RM} = k_1 \cdot p_a \cdot \left(\frac{I_1}{p_a}\right)^{k_2} \cdot \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{1}$$

where I_1 is the first stress invariant (i.e., the sum of the three principal stresses, $I_1 = s_1 + s_2 + s_3$), t_{oct} is the shear octahedral stress that in triaxial conditions is equal to $\sqrt{2}/3\ s_d$ (i.e., the difference s_1 - s_3), and p_a is the reference atmospheric pressure (equal to 0.10133 MPa).

Moreover, they found that the plasticity of fines and moisture conditions did not significantly affect the k_1 model parameter, while plastic fines were found to influence k_3. Conversely, the k_1 model parameter was significantly affected by the quantity of fines. In their work, they did not provide models to quantify changes in k_i as a function of changes in plasticity behaviour and the quantity of small particles (Mishra, Tutumluer, & Butt, 2010; Mishra, Tutumluer, & Xiao, 2010).

Caicedo, Coronado, Fleureau, & Gomes Correia (2009) as well as Coronado, Caicedo, Taibi, Correia, & Fleureau (2011), investigated the effects of different fine content and *PI* values on non-standard aggregates (materials made up of grains including those with a higher plastic fine content which were more crushable than ordinary unbound soil-aggregate mixtures). They observed that materials with a high percentage of fines and a high PI exhibit high RM values and small permanent strain values, although these materials are highly sensitive to any variation in water content.

Other authors (Yau & Von Quintus, 2002; Archilla, Ooi, & Sandefur, 2007; Nazzal & Mohammad, 2010) have provided equations to correlate the parameter of the MEPDG model with the physical and water sensitivity parameters of a number of GMs and soils.

In order to quantify the influence of fine particles on the engineering properties of aggregate-soil mixtures, the authors investigated the effects on workability and resilient modulus determined by the quantity and type of fines, according to the definition of the EN ISO 14688-1 (European Committee for Standardization, 2018) in which a "fine" particle is identified as a particle smaller than 63 mm. For this purpose, four fines of different origins and *PI* values were mixed with a single source of aggregate to preclude any effects related to shape, angularity, and texture of grains, which had already been the focus of investigation in Gu, Sahin, Luo, Luo, & Lytton (2015).

The aggregate-soil (AS) engineered mixtures were blended to obtain three reference grading curves normally adopted for road constructions (Centro Interuniversitario Sperimentale di Ricerca Stradale, 2001), in which three different silty clays with $11 \leq PI \leq 25$ and non-plastic fines ($PI = 2$) were employed. The plasticity index range of fines used in the investigation is typical for fines in soils employed as road materials. The quantity of fines (f) used were fixed at 5.4, 10.8 and 16.2% of the mixture weight, the corresponding dust ratio (DR) was from 0.27 to 0.69, while the moisture contents (w) were 4.5, 5.5, and 6.5%. Figure 2 summarizes the information relating to the grading curves considered in this investigation.

[FIGURE 2 near here]

Materials and methods

Base materials

Three Reference Grading Distributions (RGDs) are employed as per Italian technical specifications (Centro Interuniversitario Sperimentale di Ricerca Stradale, 2001). The differences between the RGDs depend on the quantity of materials passing through and retained at the 63 mm sieve as reported in Table 1. Their reconstruction was carried out by subjecting the aggregate to washed sieve separation and then combining all single size fractions finer than 20 mm and retained at the 16, 12.5, 8, 4, 2, 0.5 mm and 63 mm sieves in the needed quantities.

The apparent particle density (g_g) of each single size fraction was measured according to EN 1097-6 (European Committee for Standardization, 2013), with a progressive reduction in the particle density when the size fraction decreased being observed.

Four different fine types were employed in the investigation: the fine type contained in the natural aggregate used to build the coarser granular skeleton (designated as “nat”), and three silty-clays with *PI* values ranging from 11 to 25 and designated as Cisterna (“cis”), Belvedere (“bel”), and Scarperia (“sca”). Fines were sourced from different geographical areas in Central and Northern Italy (Table 2), and an assessment of their water sensitivity was carried out by means of the liquid limit (*LL*) and plastic limit (*PL*) procedures according to CEN ISO/TS 17892-12 (European Committee for Standardization, 2004b).

To avoid any secondary effects that could have altered the reference grading curves, the authors removed all the fractions retained at the 63 mm sieve from the three silty-clays. Table 2 also reports the plasticity index (*PI*), the silt content (*SC*), and the clay content (*CC*), the g_g of fines, and their classification according to ASTM D2487 (American Society for Testing Materials, 2011). The four fines exhibited a variation in *PI* ranging from 2 (non-plastic) to 25 (medium-high plasticity). Figure 3 reports the grading curves for the fines obtained by means of a sedimentation test performed with a hydrometer according to CEN ISO/TS 17892-4 (European Committee for Standardization, 2004a).

[TABLE 1 near here]

[TABLE 2 near here]

[FIGURE 3 near here]

Aggregate-soil mixtures

Table 3 reports the properties of the eleven AS mixtures investigated, including the results of Atterberg limits and PI obtained by combining the fines passing through 63 mm (Table 2) with the fraction 0.063-0.4 mm of the reference aggregate. Following the combination of aggregate and soils, the PI values were significantly lower than those recorded for the fine materials alone reported in Table 2, since they were measured on the fraction passing through the 0.4 mm sieve also including the 0.063-0.4 mm fraction of the reference aggregate.

Due to the small amount of it available, the Belvedere clay (bel) material was used in modest quantities of 5.4 and 10.8% in the preparation of AS mixtures.

Table 3 also contains the classification of AS mixtures according to the two main reference systems: the method of the ASTM D3282 (American Society for Testing Materials, 2015a), which is similar to the current Italian standard UNI 11531-1 (UNI Ente Nazionale Italiano di Unificazione, 2014), and the Unified Soil Classification System (USCS) reported in the ASTM D2487 (American Society for Testing Materials, 2011). In both cases, the mixtures were classified as coarse-grained soils, and the same AS mixture can be attributed to different classes for both classification systems, except mixture AS-sca_16.2f which results in a clayey gravel (GC) belonging to the A2-4 class. Using the AASHTO system, the classification of mixtures ranged from A1-a to A2-4; while using the USCS system, they ranged from well-graded (GW) to clayey gravel (GC), and from well graded sand (SW) to silty sand (SM). On the strength of their classification, all the designated mixtures are deemed to represent very good or excellent material for the formation of both subgrade and subbase layers.

[TABLE 3 near here]

Compaction of samples

A preliminary investigation into the optimum moisture content (w_{opt}) for RGD2 (Table 1) was carried out according to the Proctor method reported in the EN 13286-2 (European Committee for Standardization, 2010) at the modified reference energy (2.68 MJ/m^3, corresponding to 56 blows of the compacting hammer on each of the five layers with a mass of 4540 g falling from a height of 0.457 m). Values of w_{opt} equal to 6.5% and $g_{d\ max}$ of 2281 kg/m^3 were obtained.

To simulate field compaction by rollers, AS samples were prepared at the gyratory shear compactor (GSC) (Cerni & Camilli, 2011; Riviera, Bellopede, Marini, & Bassani, 2014). Cylindrical samples 100 mm in diameter and 200 mm in height were prepared by introducing the AS mixtures into the mould in four layers of equal thickness, each one compacted to reach the desired height and the predefined density.

Even though some materials can reveal a variation in their particle size distribution under gyratory compaction (Caicedo et al., 2009; Caicedo, Ocampo, & Vallejo, 2016), for natural aggregates with a high resistance to fragmentation (i.e. LA < 25), like those employed in this investigation, negligible crushing phenomena can be expected. This has been demonstrated by the authors in a recent publication in which the same unbound granular material used here was compared with construction and demolition waste aggregate (Bassani & Tefa, 2018); after 100 gyrations at the GSC, negligible differences were recorded in particle size distribution with respect to the initial values. As a result, variation in fine content due to compaction was not considered in this investigation.

The compaction procedure at the GSC was performed on some AS mixtures with different fine type and content values, and with the water content ranging between ± 2% around w_{opt} of 6.5% determined on the RGD2. Unfortunately, after their demoulding, most of the samples with w_{opt} + 1% and w_{opt} + 2% of moisture failed due to the excessive water content that visibly saturated the matrix, thus leading to samples which were too weak and which could not be handled in the lab for mechanical tests. Hence, the investigation field for partially saturated conditions was limited to the dry part of the compaction curve for water content values equal to 4.5, 5.5, and 6.5%.

During the compaction process, the height of each sub-layer of specimens was recorded to estimate the compaction parameter C_x. The value C_x is associated with each mould revolution on the basis of the following equation (2) (Riviera et al., 2014):

$$C_x = 100 \cdot \frac{\gamma_d \cdot h_f}{\gamma_g \cdot h_x} \quad (2)$$

and represents the complement to 100 of the void content of the sample, which can be occupied by air and water. In eq.2, γ_g is the apparent particle density of grains (European Committee for Standardization, 2013), γ_d is the density of the sample at the end of the compaction process according to AASHTO T312 (American Association of State and Highway Transportation Officials, 2015), h_x and h_f represent the height of the sample measured by the GSC at the generic gyration (x) and at the end of the compaction process respectively. Evidence from literature (Bassani et al., 2009; Riviera et al., 2014) has demonstrated that the relationship between C_x and the logarithm for the number of gyrations (x) is linear for several different materials including GMs. Thus, the following equation (3) was used to interpret the AS mixture compaction of each sub-layer:

$$C_x = C_{x=1} + k_g \cdot Log(x) \tag{3}$$

where the regression parameter k_g indicates the workability (i.e., the propensity of GMs to become denser during the compaction effort), while $C_{x=1}$ is the initial degree of compaction (also called self-compaction) at the first gyration (x = 1). Hence, a higher k_g indicates GMs that can be compacted readily and, therefore, reach high field density values with a lower number of roller passes in full scale construction. This propensity is recognized in the field of road and structural construction materials as “workability”. In this investigation, the average value of the four different k_g obtained from each sub-layer was assumed to be representative of the workability of the entire sample.

Resilient modulus test

Resilient modulus (*RM*) tests were carried out immediately after the preparation of cylindrical samples, in order to avoid any loss of moisture from the samples. The *RM* test, which is generally regarded as the leading stress-strain response parameter for granular materials under traffic load conditions, was performed in the triaxial apparatus where a deviatoric haversine-shaped form stress (σ_d) was applied to the upper surface of the cylindrical specimen, while the cell pressure was maintained constant for such conditions. Regarding the testing equipment employed: the load was measured inside the triaxial chamber, while the deformation was measured by means of two LVDTs mounted outside the chamber.

RM is the ratio between the maximum deviatoric stress ($\sigma_{d,max}$) recorded at each load application (i.e. the maximum cyclic stress as per AASHTO T307, (American Association of State and Highway Transportation Officials, 2013)), and the maximum recovered vertical strain ($\varepsilon_{z,max}$). The AASHTO T307 (American Association of State and Highway Transportation Officials, 2013) testing protocols for base/subbase materials envisages fifteen loading sequences of 100 cycles each with an increasing deviatoric and confining stress level and the application of a preconditioning of 500 cycles before the start of the first loading sequence.

Results, data modelling and discussion

Effects of fine quantity and type on workability

The combination of the eleven AS mixtures and the three water contents resulted in thirty-three compaction curves. Figure 4 contains two graphs representing the evolution of the degree of compaction (C_x) as a function of the logarithm of the number of gyrations (Log x) for samples with a water content of 6.5% and containing the Natural fine (Figure 4A) and Scarperia clay (Figure 4B). The data refer to the three reference fine contents adopted in the investigation (5.4, 10.8, 16.2%). According to eq. 3, C_x evolves linearly in the semi log plot.

Self-compaction ($C_{x=1}$) is higher in mixtures prepared with the lower fine content (5.4%), and lower when the fine content increases. This can be explained by the fact that due to a lower quantity of fines, larger particles are closer to each other and this can lead to a reduction in the mixture volume without the application of any external force.

This difference at the initial phase of the compaction process is maintained during compaction, with only minor variation due to a difference in the propensity of materials to reach denser configurations. The two examples in Figure 4 indicate that although the $C_{x=1}$ values for the two AS mixtures were initially comparable, during compaction the AS mixtures with plastic fines exhibited higher workability and reached denser packing structures (the differences may be appreciated by looking at the $C_{x=100}$ values reported at the end of the data point for Log(x) = 2).

After a first data analysis, $C_{x=1}$ was not found to be influenced by water content but rather by fine quantity and type, albeit values were relatively constant ranging from 69.4 % to 73.6%. Conversely, the quantity and type of fines evidently affect k_g as depicted in Figure 5.

The curves in Figure 5 reveal a general increase in workability when the quantity of fines increases from 5.4% to 10.8%, while, in most of the mixtures, it decreases when passing from 10.8 to 16.2%.

This phenomenon can be explained by noting that when the volume of fines increases, coarser particles are spaced further apart with a resulting drop in the number of points of contact between particles. During compaction, these larger grains tend to float in the mastic formed by water and fine particles; during the process, their position changes gradually to reach the final spatial configuration in the matrix. As can be seen in Figure 5, only samples containing Cisterna clay and a moisture content of 5.5% exhibited a different behaviour in terms of workability.

Figure 6 highlights the potential effects of water content and fine plasticity on k_g. When present in limited quantities (below the optimum moisture content), water acts as a lubricant for granular materials facilitating the reciprocal movement of particles, and, therefore, the densification process.

[FIGURE 4 near here]

[FIGURE 5 near here]

[FIGURE 6 near here]

Considering the fine quantities of 5.4% and 10.8%, mixtures with Belvedere clay showed the highest values of k_g, with the exception of the samples containing 4.5% moisture and 5.4% fines. Results evidence that, for the same fine type and water content, the plasticity of fine particles improves the workability of AS mixtures. This result confirms and quantifies the fundamental role of fines in the densification process of granular materials.

Plastic fines may be added to improve workability and, at the same compaction energy levels, may lead to denser granular matter than that obtainable with the same quantity of non-plastic fines. However, AS mixtures with an excessive proportion of fines, be they plastic or non-plastic, are more difficult to compact than those with a fine content of around 10%.

Resilient modulus data modelling

The graphs from Figure 7 to Figure 10 summarize all the *RM* test results. Each point represents the average *RM* value of the last five pulses of each loading sequence. Data are plotted on a log-log scale with the *RM* as a function of the first stress invariant (I_1) which was found to be the stress parameter which had the greatest effect on the variation in RM of the investigated AS mixtures.

Figure 7 contains the data of the AS-nat mixture with fine quantities of 5.4% (Figure 7A), 10.8% (Figure 7B), and 16.2% (Figure 7C). In each graph, the curves correspond to the three moisture contents. In the case of 5.4% natural non-plastic fines, the highest *RM* was obtained for the higher water content (6.5%). Conversely, with an increase in the quantity of fines the *RM* with a moisture content of 6.5% decreased when compared to the lower moisture content.

Figure 8 reports the results for AS-cis mixtures. The sample containing 4.5% moisture was always stiffer than those with a higher moisture content, although the degree of stiffness varied with fine quantity.

Figure 9 shows the results of AS-bel mixtures at the three different moisture contents. When the fine quantity value changes from 5.4 to 10.8%, there are no relevant changes in mixture behaviour, with drier mixtures always stiffer than wetter ones. Finally, Figure 10 reports the results for AS-sca. In this case, of particular interest is the resilient behaviour of the three different AS combinations obtained with the moisture content of 5.5%. Although the mixture exhibited the lowest *RM* values at a clay content of 5.4%, these values (of RM) increased progressively passing from 10.8 to 16.2%.

RM data have been used to calibrate both the MEPDG model (eq. 1) and a variant proposed here with the aim of separating the contribution of matric suction from that of stress state conditions (Salour & Erlingsson, 2015; Han & Vanapalli, 2015). Matric suction (ψ) is assumed here as an independent stress variable in the MEPDG model according to the following equation:

$$\mathrm{RM} = k_1 \cdot p_a \cdot \left(\frac{I_1}{p_a}\right)^{k_2} \cdot \left(1+\frac{\tau_{oct}}{p_a}\right)^{k_3} \cdot \left(1+\frac{\psi}{p_a}\right)^{k_4} \tag{4}$$

in which k_4 is the regression parameter associated with matric suction. As in the MPEDG model (eq. 1), the term containing ψ includes the reference atmospheric pressure (p_a), and the additive term 1 to avoid RM resulting as null in saturated conditions ($\psi = 0$).

[FIGURE 7 near here]

[FIGURE 8 near here]

[FIGURE 9 near here]

[FIGURE 10 near here]

The unit of measurement for RM in eq. 4 is controlled by the unit selected for p_a, while the four parameters are independent of the unit of measurement. Three of these four regression parameters reflect the variations in RM due to stress: k_1 is the model intercept, k_2 is the exponential regression coefficient associated with the first stress invariant (values of $k_2 > 0$ should always be expected since all GMs exhibit a stiffening behaviour with respect to these stress parameters), and k_3 is the regression coefficient associated with the octahedral shear stress (when positive the material exhibits a shear hardening behaviour, when negative it exhibits a shear softening behaviour). According to eq. 4, the RM increases as the suction (Y) increases for $k_4 > 0$, while it decreases if Y increases for $k_4 < 0$.

Table 4 includes AS mixture characteristics such as the reference grading curves (RGD), fine quantity (f), water content (w), the apparent density of grains forming the AS mixture (g_g), the dry density measured at the end of the compaction process (g_d), the void index (e), the degree of saturation (S), and the estimated matric suction (Y). The values of Y were derived from the Soil-Water Characteristic Curve (SWCC) as a function of the degree of saturation (S) according to Fredlund & Xing (1994), calibrating the fitting parameters on the basis of the grain size distribution and plasticity index of each AS mixture as per Perera, Zapata, Houston, & Houston (2005).

The quality of modelling was assessed by means of the standard error ratio (S_e/S_y), in which S_e is the standard error of the estimate and S_y the standard deviation of measures. The smaller S_e/S_y, the greater the accuracy of the prediction (Pellinen & Witczak, 2002), and it is generally considered good in the range 0.55 to 0.36, and excellent when it falls below 0.35 (Witczak, 2002; Tran & Hall, 2005).

Referring to the AASHTO approach (American Association of State Highway and Transportation Officials, 2015), eq. 1 is normally calibrated with data measured on the same sample for specific moisture conditions, so the effects of moisture are taken directly into account in the three k_i model constants. In this case, the effects of fine and moisture amounts may be captured through a statistical analysis of the measured data, e.g., through the analysis of variance (Mishra, Tutumluer, & Butt, 2010; Mishra, Tutumluer, & Xiao, 2010).

[TABLE 4 near here]

Figure 11 shows a comparison between the measured and modelled RM data using eq. 4. The data points of the two graphs indicate the measured and estimated *RM* at each loading sequence for all samples. The data reported in Figure 11 confirms the good prediction capability (R^2 = 0.933, S_e/S_y = 0.260) of eq. 4.

[FIGURE 11 near here]

Effects of fine quantity and type on resilient modulus

The regression parameters in eq. 4 were obtained by fitting the measured data as per the least squares method. The results are reported in the four graphs in Figure 12 with the standard error ratio S_e/S_y. Figure 12A shows a clear increase in the k_1 parameter when the fine content (*f*) passes from 5.4% to 16.2%, with a very marked increase in the case of AS-sca. In the case of AS-nat, for mixtures with equal fine content values, superior k_1 values were observed for the mixtures with non-plastic fines compared to those containing plastic fines.

k_2 values are depicted in Figure 12B. The positive values confirm the typical stiffening behaviour of GMs relative to the first invariant. k_2 generally decreases with an increase in fine content (with the sole exception of AS-bel) and is affected by the plasticity of fines: the most plastic fine (Scarperia clay) shows the lowest k_2 values.

The k_3 values displayed in Figure 12C are both positive and negative. Generally, an increase in fine content corresponds to a decrease in k_3, highlighting the different behaviour of the materials when it comes to the octahedral shear stress (i.e., the deviatoric stress). In particular, it seems that plastic fines in small quantities may impart a shear hardening behaviour (AS-cis and AS-sca with 5.4% and 10.8% of fine), while an excess can overfill the matrix reducing the number of grain-to-grain contact points and, thereby, compromise the shear behaviour of the whole GM, as evident in the case of AS-bel with 10.8% fine content and, especially, in the samples of AS-sca with 16.2% fine content. The non-plastic natural fines show a shear softening behaviour (negligible in the case of AS-nat with 10.8% of fine).

The influence of suction on RM is taken into account by the k_4 parameter (Figure 12D). Positive k_4 values are observed in the case of plastic fines, since plastic fines magnify the effects of matric suction in the AS mixture, but its values tend to decrease with an increase in fine content. The AS-mixtures containing non-plastic fines exhibit negative k_4 values that tend to increase with an increment in fine content. No evident trend can be discerned in the variation of the k_4 parameter with fine plasticity.

[FIGURE 12 near here]

Conclusions

Aggregate-soil (AS) mixtures may be obtained either incidentally as per mechanisms like interpenetration between layers, pumping or diffusion of silt and clay grains mixed with water, or by the deliberate blending of coarse aggregates and fine soils. Their attitude to densification and to traffic loading depends on several factors, such as the typology and quantity of fines. At present, there is a lack of information regarding the effects of fines on fundamental properties affecting the laying operations (workability) and service life (resilient modulus) of pavement subbase materials.

This paper investigated the behaviour of engineered blends made up of aggregate and fine plastic soils of different size and origins (plasticity). On the basis of the experimental investigation described above, the main contributions of this paper are:

1. in dry to optimum moisture conditions, fine content values in excess of 16.2% reduce the workability of the AS-mixtures irrespective of the type of fine used;
2. a fine quantity value of 10.8% was found to optimize workability for all the fine types considered in this investigation;
3. under the same moisture conditions, plastic fines improve the mixture workability more than non-plastic ones; this investigation confirms that workability tends to increase when the plasticity index (PI) of the fraction passing through the 0.4 mm sieve increases;
4. according to the suction integrated resilient modulus model proposed in this paper (eq. 4), fine quantity and type have a significant effect on the k_i model parameters;
5. the model intercept (k_1) increases with an increase in fine content and, non-plastic fines provide a greater k_1 value than those showed by the AS mixtures containing plastic fines;
6. the model parameter associated with the first stress invariant (k_2) generally decreases as fine quantity and plasticity increases; higher k_2 values are achieved with samples containing non-plastic fines;
7. the model parameter associated with the octahedral shear stress (k_3) generally tends to decrease when the fine content increases; in this case, small amounts of

plastic fine (f < 10.8%) provide a shear hardening behaviour, which is not evident in the case of non-plastic fines;

8. the model exponent associated with the matric suction (k_4), has a different trend owing to the nature of plastic fines; it reflects the positive contribution of suction to RM, being significantly greater than zero in the case of AS mixtures containing plastic fines and negative for AS mixtures containing non-plastic fines.

As mentioned, *RM* test results have been interpreted with a modified version of the MEPDG model (eq. 1) which proved robust when predicting the behaviour of AS mixtures for values of the matric suction lower than 230 kPa (Table 4). The proposed model (eq. 4) enabled an evaluation of the effects of fine quantity and type on the k_i regression parameter associated with stress-state conditions, excluding the contribution of the variation in matric suction to the same parameters.

These results integrate the existing knowledge of the effects of plastic fines in dry to optimum moisture conditions such as subbase layers operating in well-drained pavement structures. The results from this investigation suggest a different approach to the limitation of plastic fine content and type reported in ASTM D1241 (American Society for Testing Materials, 2015b). Furthermore, according to the indications from the compaction study, to benefit from the higher workability of a granular matrix, fine content values should not exceed 10-11%.

It is worth remembering that GMs in pavements operate under partially-saturated conditions which must be preserved by the impermeability of the bounded top layers, and by the anti-capillarity attitude of layers like subgrades and embankment foundations.

The practice of using fine soils as a minor component of aggregates employed in unbound layers can also be seen as an alternative efficient use (or re-use) of plastic fine soils that are currently rejected in pavement and track applications.

Acknowledgements. The experimental activities in the laboratory were carried out with the involvement of Ms. Camila Andrea Rodríguez Pimentel and Mr. Davide Salinardi, former students at the Politecnico di Torino, who are gratefully acknowledged for their contributions.

References

American Association of State and Highway Transportation Officials. Standard method of test for determining the resilient modulus of soils and aggregate materials, Pub. L. No. AASHTO T 307-99, AASHTO T 307-99 (2013).

American Association of State and Highway Transportation Officials. Standard method of test for preparing and determining the density of asphalt mixture specimens by means of the superpave gyratory compactor, Pub. L. No. AASHTO T 312-15, AASHTO T 312-15 (2015).

American Association of State Highway and Transportation Officials. (2015). *Mechanistic-empirical pavement design guide: a manual of practice.*

American Society for Testing Materials. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), Pub. L. No. ASTM D2487-11, ASTM D2487-11 (2011).

American Society for Testing Materials. Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, Pub. L. No. ASTM D3282-15, ASTM D3282-15 (2015).

American Society for Testing Materials. Standard Specification for Materials for Soil-Aggregate Subbase, Base, and Surface Courses, Pub. L. No. ASTM D1241-15, ASTM D1241-15 (2015).

Archilla, A. R., Ooi, P. S., & Sandefur, K. G. (2007). Estimation of a Resilient Modulus Model for Cohesive Soils Using Joint Estimation and Mixed Effects. *Journal of Geotechnical and Geoenvironmental Engineering*, *133*(8), 984–994. https://doi.org/10.1061/(ASCE)1090-0241(2007)133:8(984)

Barskale, R. D., & Itani, S. Y. (1989). Influence of aggregate shape on base behavior. *Transportation Research Record*, (1227).

Bassani, M., Santagata, E., Baglieri, O., Ferraris, M., Salvo, M., & Ventrella, A. (2009). Use of vitrified bottom ashes of municipal solid waste incinerators in bituminous mixtures in substitution of natural sands. *Advances in Applied Ceramics*, *108*(1), 33–43. https://doi.org/10.1179/174367608X364285

Bassani, Marco, & Tefa, L. (2018). Compaction and freeze-thaw degradation assessment of recycled aggregates from unseparated construction and demolition waste. *Construction and Building Materials*, *160*, 180–195. https://doi.org/10.1016/j.conbuildmat.2017.11.052

Caicedo, B., Coronado, O., Fleureau, J. M., & Gomes Correia, A. (2009). Resilient Behaviour of non Standard Unbound Granular Materials. *Road Materials and Pavement Design*, *10*(2), 287–312. https://doi.org/10.3166/rmpd.10.287-312

Caicedo, B., Ocampo, M., & Vallejo, L. (2016). Modelling comminution of granular materials using a linear packing model and Markovian processes. *Computers and Geotechnics*, *80*, 383–396. https://doi.org/10.1016/j.compgeo.2016.01.022

Centro Interuniversitario Sperimentale di Ricerca Stradale. Norme Tecniche Prestazionali per Capitolati Speciali d'Appalto (2001).

Cerni, G., & Camilli, S. (2011). Comparative Analysis of Gyratory and Proctor Compaction Processes of Unbound Granular Materials. *Road Materials and Pavement Design*, *12*(2), 397–421. https://doi.org/10.3166/rmpd.12.397-421

Coronado, O., Caicedo, B., Taibi, S., Correia, A. G., & Fleureau, J.-M. (2011). A macro geomechanical approach to rank non-standard unbound granular materials for pavements. *Engineering Geology*, *119*(1–2), 64–73. https://doi.org/10.1016/j.enggeo.2011.02.003

Duong, T. V., Cui, Y. J., Tang, A. M., Dupla, J. C., Canou, J., Calon, N., … De Laure, E. (2014). Physical Model for Studying the Migration of Fine Particles in the Railway Substructure. *Geotechnical Testing Journal*, *37*(5), 20130145. https://doi.org/10.1520/GTJ20130145

Duong, T. V., Cui, Y. J., Tang, A. M., Dupla, J. C., Canou, J., Calon, N., & Robinet, A. (2016). Effects of water and fines contents on the resilient modulus of the interlayer soil of railway substructure. *Acta Geotechnica*, *11*(1), 51–59. https://doi.org/10.1007/s11440-014-0341-0

Ekblad, J., & Isacsson, U. (2008). Influence of water and mica content on resilient properties of coarse granular materials. *International Journal of Pavement Engineering*, *9*(3), 215–227. https://doi.org/10.1080/10298430701551193

European Committee for Standardization. Geotechnical investigation and testing - Laboratory testing of soil - Part 4: Determination of particle size distribution, Pub. L. No. CEN ISO/TS 17892-4:2004, CEN ISO/TS 17892-4:2004 (2004).

European Committee for Standardization. Geotechnical investigation and testing - Laboratory testing of soil - Part 12: Determination of Atterberg limits, Pub. L. No. CEN ISO/TS 17892-12:2004, CEN ISO/TS 17892-12:2004 (2004).

European Committee for Standardization. Unbound and hydraulically bound mixtures - Part 2: Test methods for laboratory reference density and water content - Proctor compaction, Pub. L. No. EN 13286-2:2010, EN 13286-2:2010 (2010).

European Committee for Standardization. Tests for mechanical and physical properties of aggregates - Part 6: Determination of particle density and water absorption, Pub. L. No. EN 1097-6:2013, EN 1097-6:2013 (2013).

European Committee for Standardization. Geotechnical investigation and testing - Identification and classification of soil - Part 1: Identification and description, Pub. L. No. EN ISO 14688-1:2018, EN ISO 14688-1:2018 (2018).

Fredlund, Delwyn G., & Rahardjo, H. (1993). *Soil mechanics for unsaturated soils*. John Wiley & Sons.

Fredlund, D.G., & Xing, A. (1994). Equations for the soil-water characteristic curve. *Canadian Geotechnical Journal*, *31*(4), 521–532. https://doi.org/10.1139/t94-061

Giroud, J. P. (2009). An assessment of the use of geogrids in unpaved roads and unpaved areas. In *Jubilee Symposium on Polymer Geogrid Reinforcement. Identifying the Direction of Future Research, ICE, London, 8th September*.

Gu, F., Sahin, H., Luo, X., Luo, R., & Lytton, R. L. (2015). Estimation of Resilient Modulus of Unbound Aggregates Using Performance-Related Base Course Properties.

Journal of Materials in Civil Engineering, *27*(6), 04014188. https://doi.org/10.1061/(ASCE)MT.1943-5533.0001147

Hajek, J. J., Kazmierowski, T. J., Sturm, H., Bathurst, R. J., & Raymond, G. (1991). *Field Performance of Open Graded Drainage Layers*. Downsview, Ont.: Research and Development Branch, Ontario Ministry of Transportation.

Han, Z., & Vanapalli, S. K. (2015). Model for predicting resilient modulus of unsaturated subgrade soil using soil-water characteristic curve. *Canadian Geotechnical Journal*, 1–15. https://doi.org/10.1139/cgj-2014-0339

Huang, Y. H. (2003). *Pavement Analysis and Design* (2 edition). Upper Saddle River, NJ: Pearson.

Kamal, M. A., Dawson, A. R., Farouki, O. T., Hughes, D. A. B., & Sha'at, A. A. (1993). Field and laboratory evaluation of the mechanical behavior of unbound granular materials in pavements. *Transportation Research Record*, 88–88.

Lekarp, F., Isacsson, U., & Dawson, A. (2000). State of the Art. I: Resilient Response of Unbound Aggregates. *Journal of Transportation Engineering*, *126*(1), 66–75. https://doi.org/10.1061/(ASCE)0733-947X(2000)126:1(66)

Mishra, D., Tutumluer, E., & Butt, A. (2010). Quantifying Effects of Particle Shape and Type and Amount of Fines on Unbound Aggregate Performance Through Controlled Gradation. *Transportation Research Record: Journal of the Transportation Research Board*, *2167*, 61–71. https://doi.org/10.3141/2167-07

Mishra, D., Tutumluer, E., & Xiao, Y. (2010). Particle Shape, Type, and Amount of Fines and Moisture Affecting Resilient Modulus Behavior of Unbound Aggregates (pp. 279–287). American Society of Civil Engineers. https://doi.org/10.1061/41104(377)34

Mokwa, R., Cuelho, E., & Browne, M. (2008). Laboratory Testing of Soil Using Superpave Gyratory Compactor. *Transportation Research Board*.

Monahan, E. J. (1994). *Construction of Fills*. John Wiley & Sons.

Nazzal, M. D., & Mohammad, L. N. (2010). Estimation of resilient modulus of subgrade soils for design of pavement structures. *Journal of Materials in Civil Engineering*, *22*(7), 726–734.

Osouli, A., Salam, S., & Tutumluer, E. (2016). Effect of plasticity index and dust ratio on moisture-density and strength characteristics of aggregates. *Transportation Geotechnics*, *9*, 69–79. https://doi.org/10.1016/j.trgeo.2016.07.005

Osouli, A., Salam, S., Tutumluer, E., & Shoup, H. (2017). Fines inclusion in a crushed limestone unbound aggregate base course material with 25.4-mm maximum particle size. *Transportation Geotechnics*, *10*, 96–108. https://doi.org/10.1016/j.trgeo.2017.02.001

Pellinen, T., & Witczak, M. (2002). Use of stiffness of hot-mix asphalt as a simple performance test. *Transportation Research Record: Journal of the Transportation Research Board*, (1789), 80–90.

Perera, Y. Y., Zapata, C. E., Houston, W. N., & Houston, S. L. (2005). Prediction of the soil–water characteristic curve based on grain-size-distribution and index properties. *Advances in Pavement Engineering, Geotechnical Special Publication*, *130*, 49–60.

Pratibha, R., Sivakumar Babu, G. L., & Madhavi Latha, G. (2015). Stress–Strain Response of Unbound Granular Materials Under Static and Cyclic Loading. *Indian Geotechnical Journal*, *45*(4), 449–457. https://doi.org/10.1007/s40098-015-0155-5

Riviera, P. P., Bellopede, R., Marini, P., & Bassani, M. (2014). Performance-based re-use of tunnel muck as granular material for subgrade and sub-base formation in road construction. *Tunnelling and Underground Space Technology*, *40*, 160–173. https://doi.org/10.1016/j.tust.2013.10.002

Rollings, M. P., & Rollings, R. S. (1995). *Geotechnical Materials in Construction* (1 edition). New York: McGraw-Hill Professional.

Salour, F., & Erlingsson, S. (2015). Resilient modulus modelling of unsaturated subgrade soils: laboratory investigation of silty sand subgrade. *Road Materials and Pavement Design*, *16*(3), 553–568. https://doi.org/10.1080/14680629.2015.1021107

Service d'Etudes Techniques des Routes et Autoroutes, & Laboratoire Central des Ponts et Chaussées. (2000). *Réalisation de remblais et des couches de forms. Guides des Terrassements Routiers* (No. GTR SETRA-LCPC 2ème Ed). Paris, France: Ministère de l'Equipement, du Logement et des Transports.

Soliman, H., & Shalaby, A. (2015). Permanent deformation behavior of unbound granular base materials with varying moisture and fines content. *Transportation Geotechnics*, *4*, 1–12. https://doi.org/10.1016/j.trgeo.2015.06.001

Thom, N. H., & Brown, S. F. (1987). Effect of Moisture on the Structural Performance of a Crushed-Limestone Road Base. In *Transportation Research Record*.

Tran, N., & Hall, K. (2005). Evaluating the predictive equation in determining dynamic moduli of typical asphalt mixtures used in Arkansas, (74E).

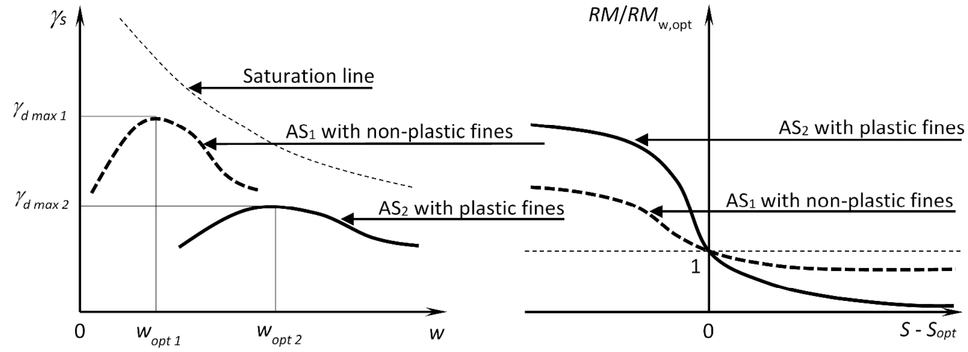
UNI Ente Nazionale Italiano di Unificazione. Costruzione e manutenzione delle opere civili delle infrastrutture - Criteri per l'impiego dei materiali - Parte 1: Terre e miscele di aggregati non legati, Pub. L. No. UNI 11531-1:2014, UNI 11531-1:2014 (2014).

Uthus, L., Hoff, I., & Horvli, I. (2005). A study on the influence of water and fines on the deformation properties of unbound aggregates. In *Proceeding of the 7th International Conference on the Bearing Capacity of Roads, Railways and Airfields. Trondheim, Norway* (pp. 1–13).

Witzcak, M. W. (Ed.). (2002). *Simple performance test for Superpave mix design*. Washington, D.C: National Academy Press.

Yau, A., & Von Quintus, H. L. (2002). *Study of LTPP Laboratory Resilient Modulus Test Data and Response Characteristics* (No. FHWA-RD-02-051). Georgetown Pike McLean, VA: Federal Highway Administration Office of Infrastructure Research and Development Federal Highway Administration.

Yoder, E. J., & Witczak, M. W. (1975). *Principles of Pavement Design*. John Wiley & Sons.



(A) (B)

Figure 1. Conceptual illustration of the effects of moisture and plasticity of fines on (A) maximum dry density (g_d) and water content (w), and (B) on the resilient modulus (RM) measured at current moisture conditions ($RM_{\mathrm{w,opt}}$ is RM at optimal water content) for two different aggregate-soil (AS) mixtures (AS_1 with non-plastic fines, AS_2 with plastic fines)

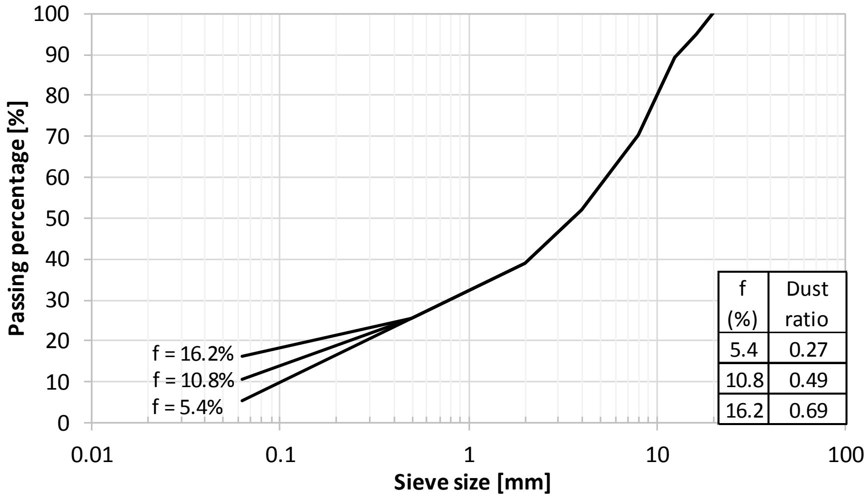


Figure 2. Grading curves and corresponding fine content (f) and dust ratios considered.

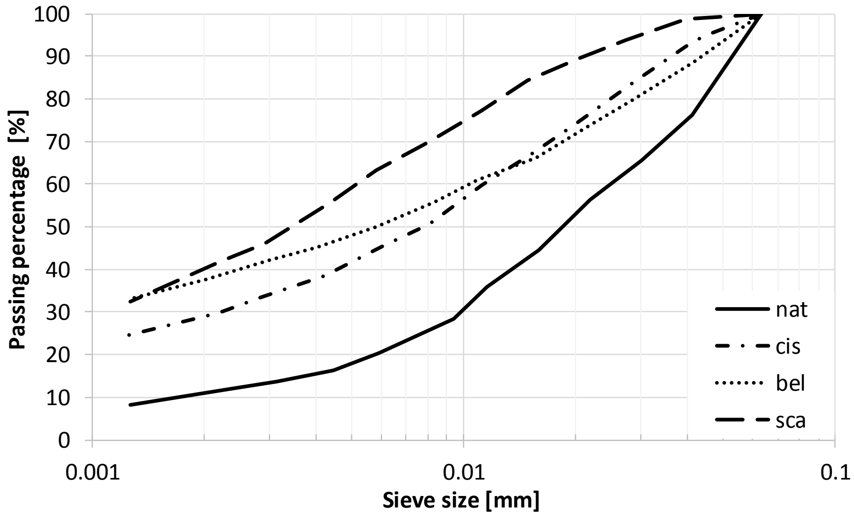
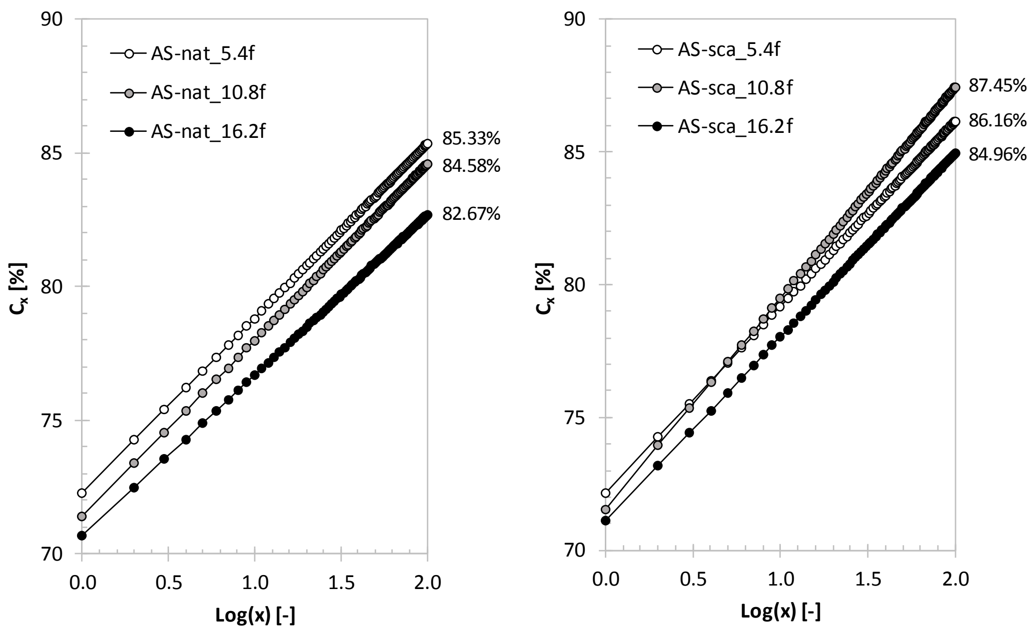
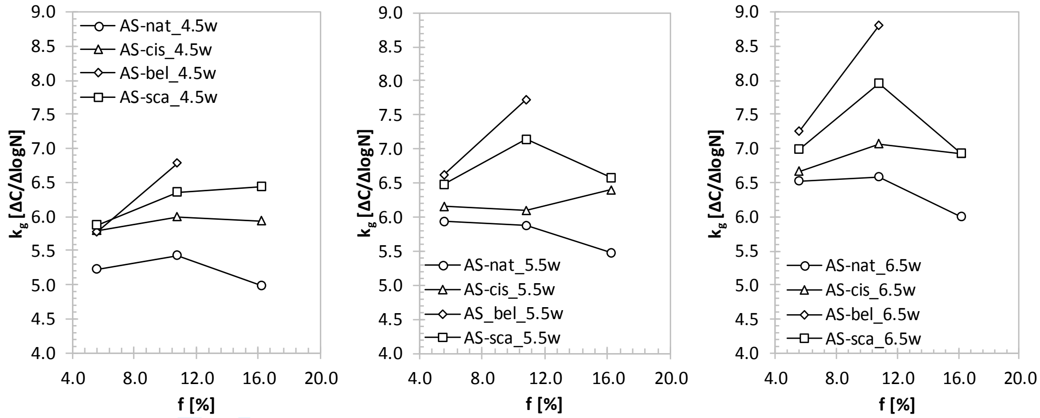


Figure 3. Gradation of the four fine types



(A) (B)

Figure 4. Compaction curves of samples compacted with Natural fine (A) and Scarperia clay (B) with 6.5% moisture content (x is the number of gyrations, while C_x indicates the compaction)



(A) (B) (C)

Figure 5. Workability (k_g) trend as a function of fine quantity (A w=4.5%, B w=5.5%, and C w=6.5% of water content)

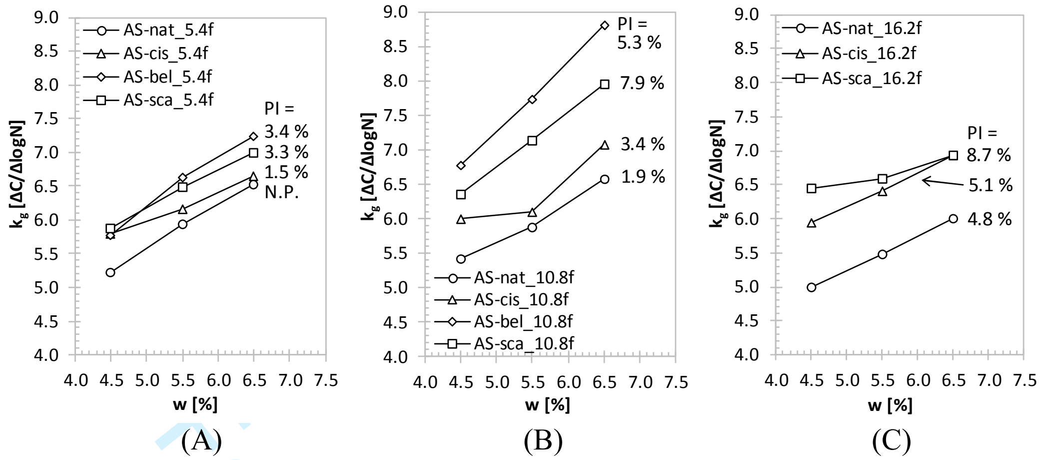
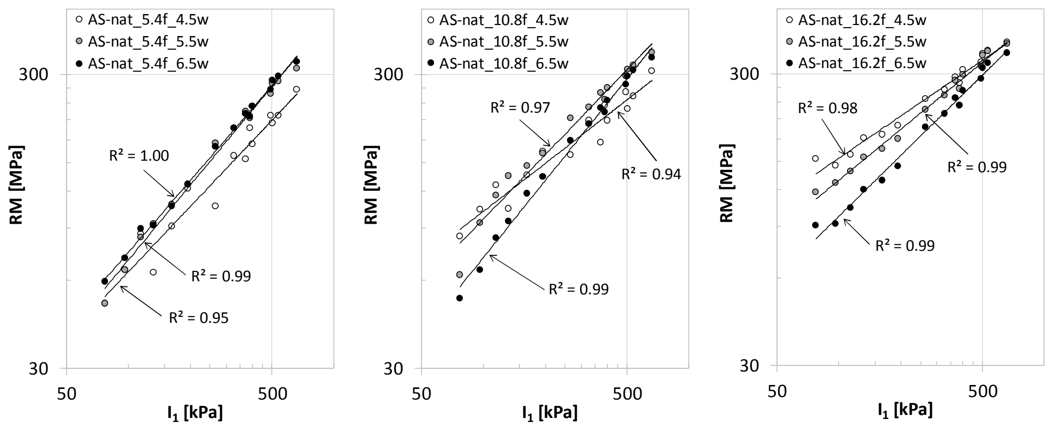


Figure 6. Workability (k_g) trend as a function of moisture content (A f=5.4%, B f=10.8%, and C f=16.2% of fine content)



(A) (B) (C)

Figure 7. Resilient modulus as a function of fine quantity (A f=5.4%, B f=10.8%, and C f=16.2%) and moisture content for AS mixtures obtained with natural fines (nat)

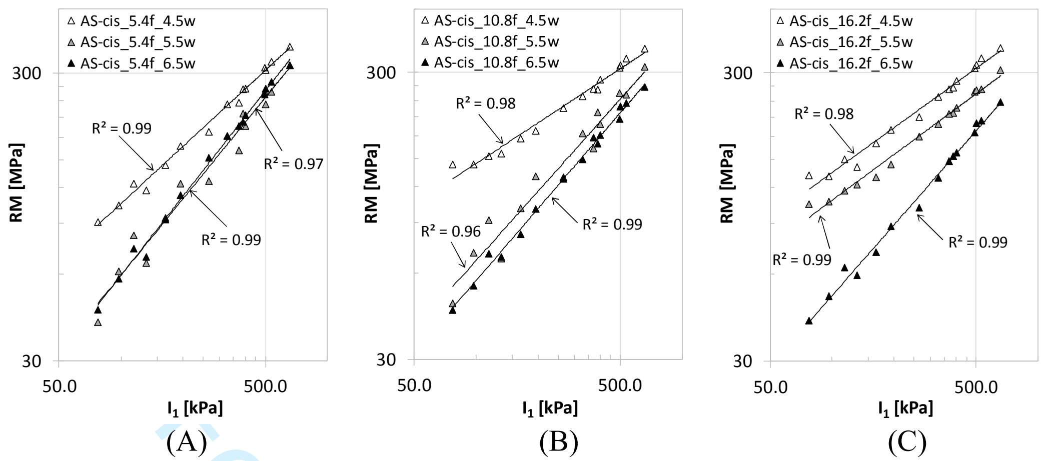
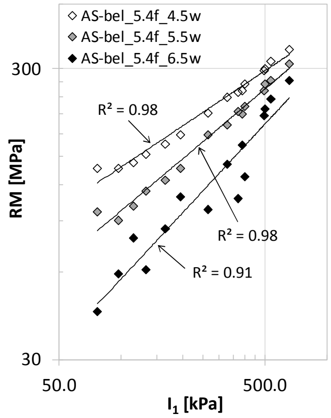
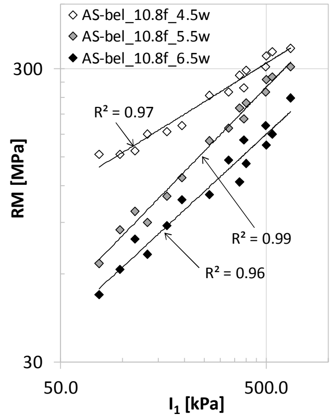


Figure 8. Resilient modulus as a function of fine quantity (A f=5.4%, B f=10.8%, and C f=16.2%) and moisture content for AS mixtures obtained with Cisterna clay (cis)

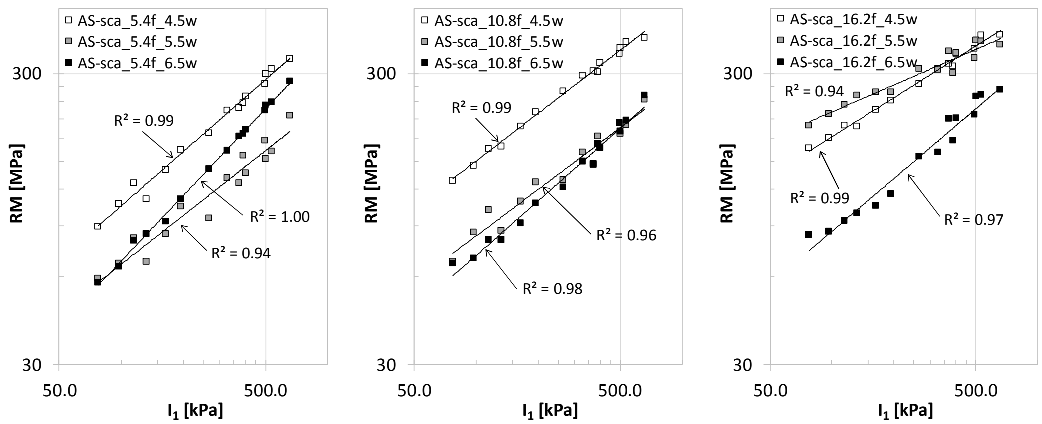


(A)



(B)

Figure 9. Resilient modulus as a function of fine quantity (A f=5.4%, and B f=10.8%) and moisture content for AS mixtures obtained with Belvedere clay (bel)



(A) (B) (C)

Figure 10. Resilient modulus as a function of fine quantity (A f=5.4%, B f=10.8%, and C f=16.2%) and moisture content for AS mixtures obtained with Scarperia clay (sca)

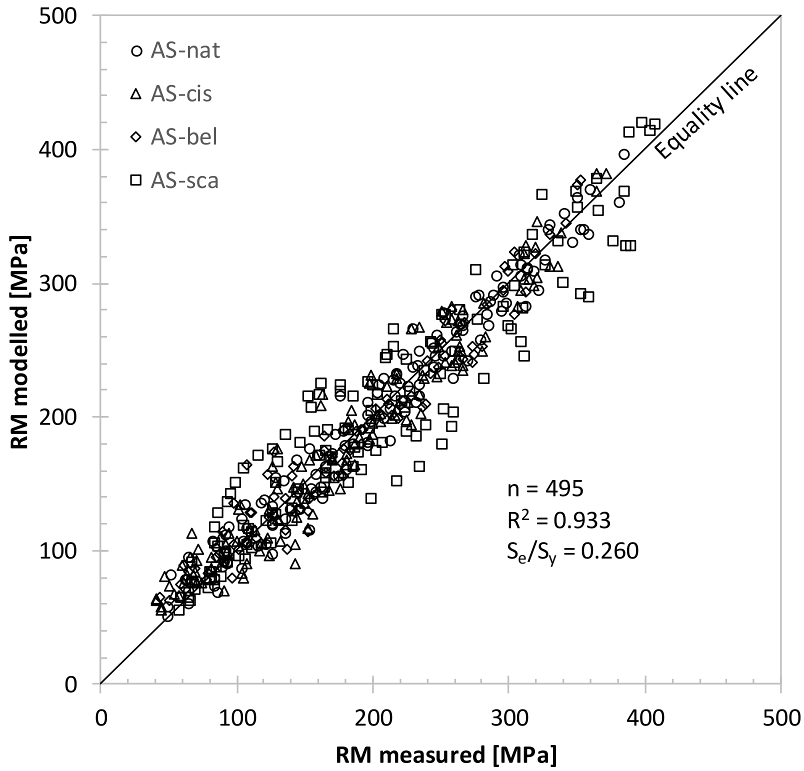
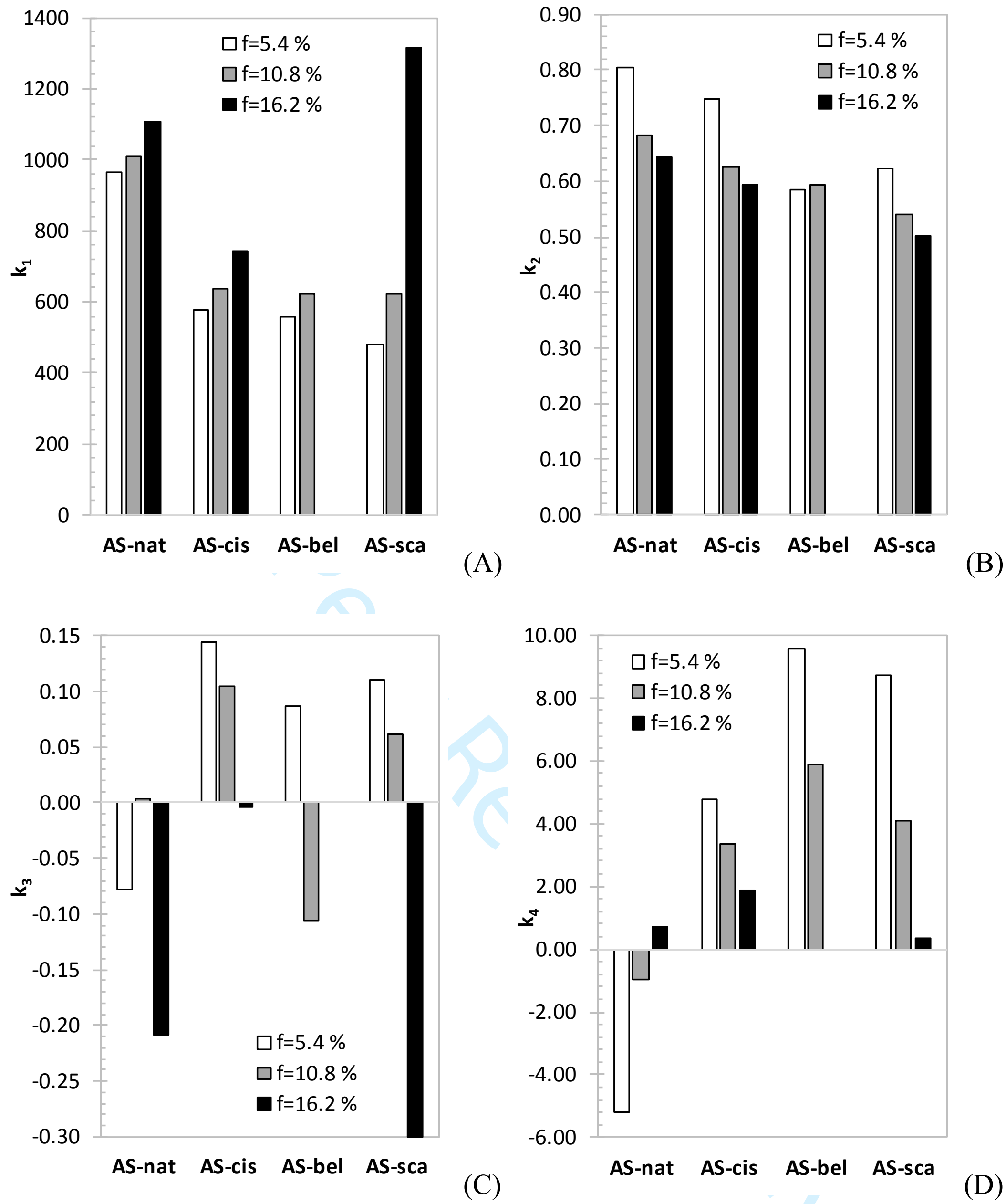


Figure 11. Comparison between measured and modelled *RM* data using the modified MEPDG (eq. 4). “n” indicates the number of observations, R^2 the coefficient of determination, S_e/S_y the standard error ratio.



S_e/S_y	AS-nat	AS-cis	AS-bel	AS-sca
f = 5.4%	0.16	0.23	0.26	0.41
f = 10.8%	0.25	0.32	0.22	0.18
f = 16.2%	0.17	0.21	-	0.50

Figure 12. Effects of fines on the regression parameters k_1 (A), k_2 (B), k_3 (C), k_4 (D). S_e/S_y ratios indicate the modelling quality for the eleven combinations of fine type and content.

Table 1. Reference mixture gradation and apparent particle density (γ_g) of fractions retained at specific sieve openings

Sieve	γ_g	Retained mass
[mm]	[kg/m^3]	(%)
16	2.799	5.1
12.5	2.744	5.5
8	2.747	19.2
4	2.744	18.2
2	2.702	13.0
0.5	2.674	13.4
0.063	2.663	20.2 (RGD1), 14.8 (RGD2), 9.4 (RGD3)
< 0.063	(see Table 2)	5.4 (RGD1), 10.8 (RGD2), 16.2 (RGD3)

Notes: (*) RGD = Reference Grading Distribution

Table 2. Atterberg limits and particle density of fines (all passing through 63 μm sieve)

Origin	Fine designation	*LL* ([1])	*PL* ([1])	*PI*	*SC*	*CC*	Plasticity ([2])	γ_g ([3])	USCS ([4])
	-	(%)	(%)	(%)	(%)	(%)	-	[kg/m^3]	-
Natural fine, Torino, IT	nat	17	15	2	47.4	52.6	non-plastic	2.743	ML (gravelly silt)
Cisterna, Asti, IT	cis	34	23	11	25.7	74.3	low	2.483	CL (lean clay)
Belvedere, Alessandria, IT	bel	35	17	18	28.4	71.6	medium	2.543	CL (lean clay)
Scarperia, Firenze, IT	sca	46	21	25	11.0	89.0	medium-high	2.573	CL (lean clay)

Symbols: *LL* = liquid limit; *PL* = plastic limit; *PI* = plasticity index; *SC* = silt content; *CC* = clay content; γ_g = apparent particle density, USCS = Unified Soil Classification System.

Notes: ([1]) CEN ISO/TS 17892-12 (European Committee for Standardization, 2004b), ([2]) SETRA (Service d'Etudes Techniques des Routes et Autoroutes & Laboratoire Central des Ponts et Chaussées, 2000), ([3]) EN 1097-6 (European Committee for Standardization, 2013), ([4]) ASTM D2487 (American Society for Testing Materials, 2011)

Table 3. Classification of AS combinations finer than 0.4 mm

Aggregate-soil (AS) mixture designation	*f* (%)	*LL* ([1]) (%)	*PL* ([1]) (%)	*PI* (%)	AASHTO /UNI classification ([2])	USCS classification ([3])
AS-nat_5.4f	5.4	16.3	0.0	0	A1-a, A1-b, A2-4	SW-SM
AS-nat_10.8f	10.8	16.9	15.0	1.9	A1-a, A1-b, A2-4	GW-GM
AS-nat_16.2f	16.2	22.0	17.2	4.8	A1-b, A2-4	GC-GM
AS-cis_5.4f	5.4	17.7	16.2	1.5	A1-a, A1-b, A2-4	SW-SM
AS-cis_10.8f	10.8	21.3	17.9	3.4	A1-a, A1-b, A2-4	GW-GM
AS-cis_16.2f	16.2	22.9	17.8	5.1	A1-b, A2-4	GC-GM
AS-bel_5.4f	5.4	19.2	15.8	3.4	A1-a, A1-b, A2-4	SW-SM
AS-bel_10.8f	10.8	22.5	17.2	5.3	A1-a, A1-b, A2-4	GW-GC
AS-sca_5.4f	5.4	20.2	16.9	3.3	A1-a, A1-b, A2-4	SW-SM
AS-sca_10.8f	10.8	25.4	17.5	7.9	A2-4	GW-GC
AS-sca_16.2f	16.2	26.5	17.9	8.7	A2-4	GC

Symbols: *f* = fine content; *LL* = liquid limit; *PL* = plastic limit; *PI* = plasticity index.
Notes: ([1]) CEN ISO/TS 17892-12:2004 (European Committee for Standardization, 2004b), ([2]) ASTM D3282 (American Society for Testing Materials, 2015a) and UNI 11531-1:2014 (UNI Ente Nazionale Italiano di Unificazione, 2014), ([3]) ASTM D2487 (American Society for Testing Materials, 2011)

Table 4. Regression parameters of resilient modulus data modelling according to eq. 4

AS	RGD	f	w	g_g	g_d	e	S	ψ
		(%)	(%)	[kg/m^3]	[kg/m^3]	-	(%)	[kPa]
AS-nat	1	5.4	4.5	2717	2249	0.208	58.7	8.8
			5.5		2289	0.187	80.0	5.8
			6.5		2290	0.186	94.8	3.7
	2	10.8	4.5	2722	2258	0.206	59.5	16.0
			5.5		2284	0.192	78.0	9.1
			6.5		2290	0.188	93.9	5.0
	3	16.2	4.5	2726	2229	0.223	55.0	37.0
			5.5		2255	0.209	71.8	19.0
			6.5		2273	0.199	89.0	9.3
AS-cis	1	5.4	4.5	2702	2276	0.187	64.9	8.1
			5.5		2286	0.182	81.6	5.8
			6.5		2290	0.180	97.7	3.0
	2	10.8	4.5	2692	2273	0.184	65.8	16.0
			5.5		2243	0.200	74.0	12.0
			6.5		2289	0.176	99.5	1.7
	3	16.2	4.5	2681	2259	0.187	64.5	32.0
			5.5		2284	0.174	84.7	12.0
			6.5		2289	0.171	100	0.0
AS-bel	1	5.4	4.5	2706	2258	0.199	61.3	8.0
			5.5		2288	0.183	81.5	5.3
			6.5		2289	0.182	96.6	3.1
	2	10.8	4.5	2699	2276	0.186	65.3	14.0
			5.5		2289	0.179	82.9	8.1
			6.5		2290	0.178	98.3	3.0
AS-sca	1	5.4	4.5	2708	2258	0.200	61.1	8.0
			5.5		2289	0.183	81.4	5.3
			6.5		2285	0.185	95.0	3.5
	2	10.8	4.5	2703	2235	0.209	58.1	23.0
			5.5		2290	0.180	82.5	8.8
			6.5		2290	0.180	97.5	3.3
	3	16.2	4.5	2697	2205	0.223	54.4	230.0
			5.5		2248	0.200	74.2	75.0
			6.5		2282	0.182	96.5	9.8