INNOVATIVE STABILIZATION TECHNIQUES FOR WEAK CRUSHED ROCKS USED IN ROAD UNBOUND LAYERS: A LABORATORY INVESTIGATION

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ABSTRACT

The “Ferry-free coastal highway route E39” project includes building several long tunnels along the southwestern Norwegian coast. The tunnelling operations will generate a large quantity of blasted rocks; these could be used in the road unbound layers close to the place of production to provide a sustainable cost-benefit application. The existing design guidelines define strength requirements for road unbound layers in terms of Los Angeles and micro-Deval tests. Even if the major part of the rocks has igneous origin and could potentially fulfil the standard tests, the damage induced by the confined heavy blasting makes the materials fail the check procedures. The research investigates how to enable the use of the “weak” rocks by investigating three possible techniques. The first approach is the mixture between the different types of rocks available in situ. The second approach is additive application; two different non-traditional additive types are examined: one is polymer-based, the other one is lignin-based. The third approach is the attempt to modify the rocks mineralogical structure by overheating. The research test campaign uses both the aforementioned standard tests and repeated triaxial load tests. Rocks mixture and additives application are viable and sustainable methods to improve the mechanical properties of the “weak” crushed metamorphic rocks. Overheating does not turn out to be an effective and convenient procedure.

Keywords: Stabilization, Crushed rock, Pavement unbound, Los Angeles test, Micro-Deval test, Repeated triaxial load test.

1 INTRODUCTION AND BACKGROUND

Norwegian Public Roads Administration (NPRA) is currently running the "Ferry-free coastal highway route E39" project, which improves the viability along the southwestern Norwegian coast for a total length of about 1100 km from Trondheim to Kristiansand (NPRA, 2017). The project includes the building of several bridges and tunnels, while aiming for creating a sustainable infrastructure. The project is crucial to regional and national development as the industries located along the route generate about half of Norway's traditional export (Dunham, 2016).

The extended tunnelling systems will generate a very large quantity of blasted rocks. They could potentially be used as viable substitutes for natural aggregates in the road unbound layers close to the place of production. Previous experience regarding the recycling strategies of tunnel excavation materials highlighted the importance of this challenge for construction management and economics (Burdin and Monin, 2009; Haritonovs et al., 2016; Lieb, 2009; Resch et al., 2009).

Using the excavated geomaterials is beneficial from economic, environmental and social points of views (Chittoori et al., 2012; Petkovic, 2005; Riviera et al., 2014); energy consumption reduction and limited greenhouse gas emissions are the most beneficial advantages (Aatheesan et al., 2008; Arulrajah et al., 2013; Gomes Correia et al., 2016; Núñez et al., 2008). The usage of blasted materials in pavement applications is a sustainable solution to minimise the waste while reducing the demand for scarce quarried materials, activity which is resource intensive and consumes large amounts of energy (Fladvad et al., 2017). The transport distance of the blasted and crushed rocks should be within 20 - 30 km to represent a competitive solution compared to the purchase of quarry virgin aggregates (Berger, 1978; Neeb, 1992). Furthermore, the concern about environmentally-friendly and sustainable constructions is becoming more and more relevant in Norway, as it pledges to become climate neutral by 2030 (Teknologirådet, 2012).

The existing strength requirements for road unbound layers are connected to relatively simple tests: the Norwegian pavement design manual N200 (NPRA, 2014a) sets limits in terms of grain shape (CEN, 2012a), flakiness index value (CEN, 2012b), Los Angeles (LA) value (CEN, 2010) and micro-Deval (MDE) value (CEN, 2011). By respecting the specified thresholds, the road is expected to perform adequately without encountering premature damage (Barbieri et al., 2017).

The goal of the research is to explore techniques to improve the mechanical properties of the crushed rocks not complying with the standard requirements; the aim is to enable their use in the road unbound layers close to the place of production. This would entail savings on the consumption of natural resources and transport reduction; thus promoting a beneficial impact on sustainability.

The research examines mechanical and chemical treatments to make the weak materials suitable for the application. Three types of crushed rocks are chosen and investigated. One type meets the code requirements ("strong" rocks), while the other two do not ("weak" rocks); XRD diffractometer (XRD) analyses describe the mineralogy.

The first studied technique is to mix the weak rocks with the strong ones available in situ. The second considered approach is additive application. Currently there are several stabilization methods, i.e. cement, bitumen, lime, fly ash, gypsum (Arulrajah et al., 2016; Behnood, 2018; Jiang and Fan, 2013; Mohammadinia et al., 2017; Myre, 2014; NPRA, 2014b; Siripun et al., 2010). Two non-traditional stabilization techniques are examined: one is polymer-based and the other one is lignin-based. The third investigated method is to overheat the rocks to check for any possible induced changes in the mineralogical composition, which may strengthen the rocks. Both the standard tests (LA and MDE) and Repeated Triaxial Load Tests (RTLTs) are used to assess the materials performance.

2 METHODOLOGY

2.1 MATERIALS INVESTIGATED

2.1.1 GEOLOGICAL CHARACTERIZATION

The E39 highway alignment comes across different types of bedrocks (NGU, 2017; Ramberg et al., 2013). The major part of the rocks is igneous and supracrustal of Precambrian ages (1700 - $900 \cdot 10^6$ years) variably influenced by metamorphism and deformation related to the Caledonian orogeny. They mainly comprise granite, granodiorite and granitic to dioritic gneiss. There are also areas with Caledonian rocks; these locations are anyway at maximum 20 - 30 km far from the most widespread aforementioned geology. Metamorphic rocks occur close to Bergen (gabbro and augen gneiss). Zones of foliated Caledonian metamorphic rocks (e.g. mica-schist and phyllite) are locally present, in particular around Boknafjord area close to Stavanger. Three types of crushed rocks produced by the current tunnel excavations close to Bergen are investigated: they properly represent the variety in the geology spread along the entire highway alignment (Barbieri et al., 2019) as shown in Figure 1.

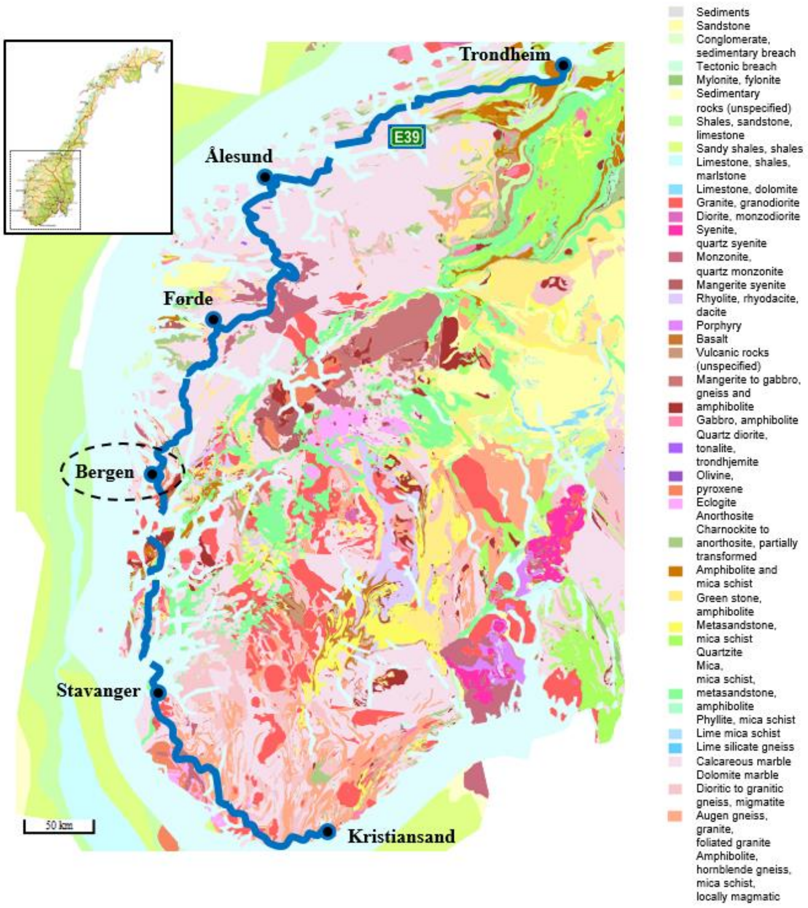

Fig. 1. E39 highway alignment and bedrock geology (Barbieri et al., 2019).

The three materials are denominated M1, M2 and M3, all being mixtures of different local rocks:

- Material M1. Mafic igneous origin, partly modified by metamorphism (amphibolite), minor amounts of felsic gneisses and mica-schist.
- Material M2. Metamorphic origin, fine-grained felsic and micaceous rocks.
- Material M3. Metamorphic origin, very fine-grained felsic and micaceous rocks.

Batches of each material type are prepared for XRD diffractometer analyses to identify the main mineralogical compositions according to Rietveld mineral quantification. Samples are crushed, split, milled to 10µm and analysed as powder preparate in the XRD diffractometer. Figure 2 displays semi-quantitative weight proportions of the most abundant minerals.

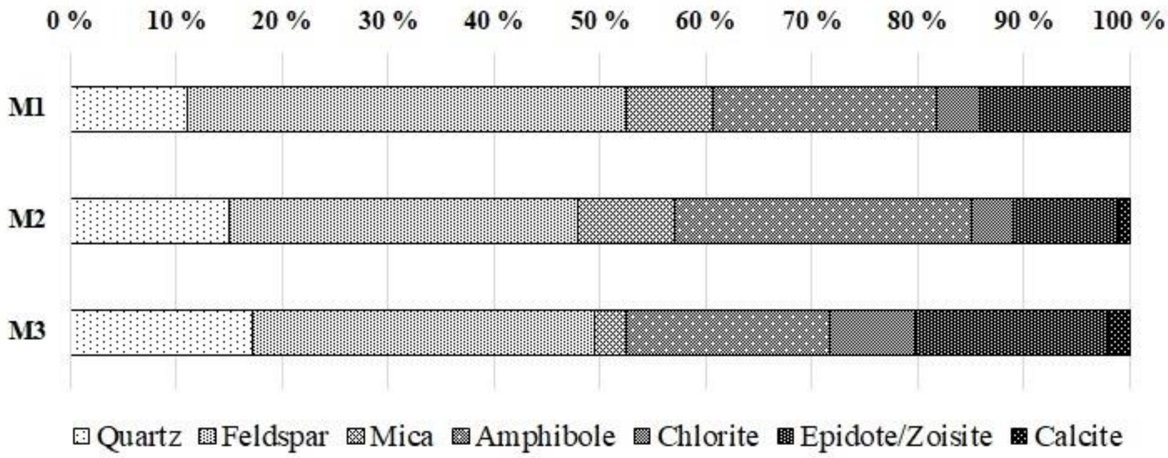

Fig. 2. Bulk mineralogy of the investigated crushed rocks.

Quartz, feldspar and amphibole are the predominant minerals in all M1, M2 and M3 mixtures (and main constituents of amphibolites and gneisses). M3 is richer in chlorite, epidote-zoisite and calcite compared to M1 and M2. Moreover, M3 has a higher content of foliated felsic rocks: networks of fine epidote-zoisite particles partly replace feldspars. Igneous rocks M1 are modified by metamorphism, e.g. amphibolization and replacement of coarse igneous feldspar by aggregates of fine epidote and feldspar. Finer-grained felsic and micaceous rocks appear more dominant in M2 and especially in M3.

2.1.2 STANDARD TESTS CHARACTERIZATION

The pavement design manual N200 (NPRA, 2014a) sets requirements for the use of crushed rocks. It is possible to use this resource in the road base layer as paved crushed rocks and in the road subbase layer as unsorted crushed rocks if Los-Angeles (LA) standard test (CEN, 2010) and micro-Deval (MDE) standard test (CEN, 2011) are fulfilled. The LA limit values are respectively 30 and 35 for base layer and subbase layer, the MDE limit value is 15 for both of them. Further requirements in terms of upper and lower grain size distribution curve are demanded for the base layer (Figure 3). The grain size distribution curve of the subbase layer must be within 20/120 mm.

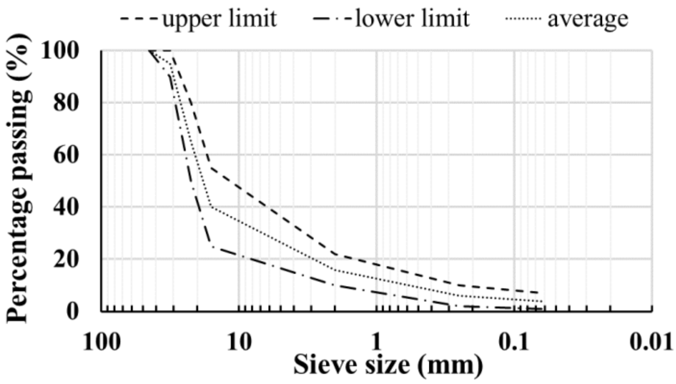

Fig. 3. Grain size distribution limit curves for base layer.

Figure 4 displays the materials values related to LA and MDE standard check procedures. Material M1 fulfils the code requirements. Both materials M2 and M3 have LA values lying close to the limit, and exceed the threshold regarding MDE values. Material M1 is designated as "strong" and materials M2 and M3 are designated as "weak" in the research.

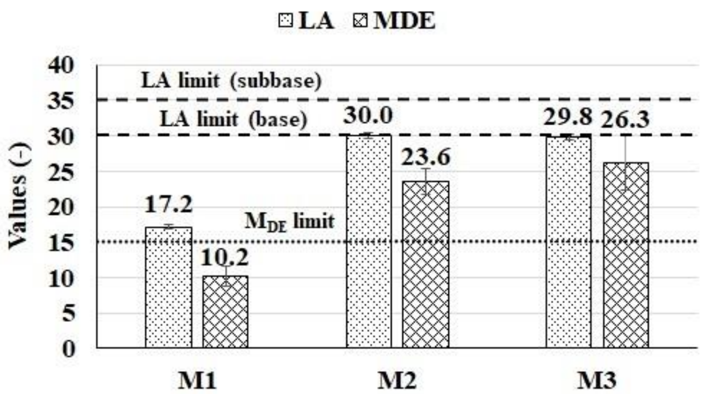

Fig. 4. Los Angeles and micro-Deval values of investigated materials (Barbieri et al., 2019).

2.1.3 REPEATED TRIAXIAL LOAD TEST

The Repeated Triaxial Load Test (RTLT) gives a comprehensive insight into material properties by assessing the stiffness and the resistance to permanent deformation (CEN, 2004). RTLT is one of the best methods available for laboratory simulation of traffic loading on unbound granular materials (UGMs); it reproduces the stress conditions in flexible pavements more adequately than other available methods like the CBR test (Barksdale, 1971). UGMs behaviour is connected to the following parameters: stress level, moisture content, dry density, grading and mineralogy, etc. (Lekarp et al., 2000a, 2000b; Uthus et al., 2007).

The preparation of the specimen follows a defined procedure. Firstly, 7300 g of material is prepared according to the grading curve displayed in Figure 3. The amount of fines content, namely the material passing the 0.0063 mm sieve, is equal to 292 g. Consequently, the desired amount of water, and additive if needed by the test, is added. The mixture is divided into four parts and rests in as many impermeable bags for 24 h. The operator then compacts the four layers inside a steel mould; the bulk density and dry density are assessed (CEN, 2003).

The optimum moisture content (OMC) evaluated for all the materials M1, M2 and M3 is w=5%. A Kango 950X vibratory hammer (total weight 35 kg, frequency 25 - 60 Hz, amplitude 5 mm) is used to compact the layers inside the mould, the compaction time is 30 s per layer. All the samples have a diameter of 150 mm and the final height varies between 170 and 190 mm. The sample height differs from the indication given by the code, where the height is recommended to be twice the diameter of the sample (CEN, 2004). Research regarding the influence of the height to diameter ratio with respect both to resilient modulus and permanent deformations demonstrates that samples with a ratio ranging from 1:1 to 1.5:1 show little differences (Dongmo-Engeland, 2005).

RTLT apparatus exerts a uniform confining pressure in all the directions (σ_t, triaxial or confining stress) and an additional vertical dynamic stress (σ_d, deviatoric stress), which stepwise increases at different levels of σ_t. The RTLT apparatus performs the multi-stage low stress level (MS LSL) loading procedure: five sequences are associated with five different σ_t values (σ_t = 20, 45, 70, 100, 150 kPa). In addition, six steps associated to six given σ_d values form each sequence (CEN, 2004). Figure 5 displays the five loading sequences and the respective loading increments according to bulk stress θ ($\theta = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_d + 3\sigma_t$; σ_1, σ_2, σ_3 are principal stresses) and σ_d. Each load step consists of 10,000 load pulses at 10 Hz frequency. A loading sequence is interrupted if the axial permanent deformation reaches 0.5%. Pressurised water is the confining medium; a hydraulic jack exerts σ_d according to a sinusoidal pattern, a minimum value of 5 kPa assures contact between the specimen end plate and the jack.

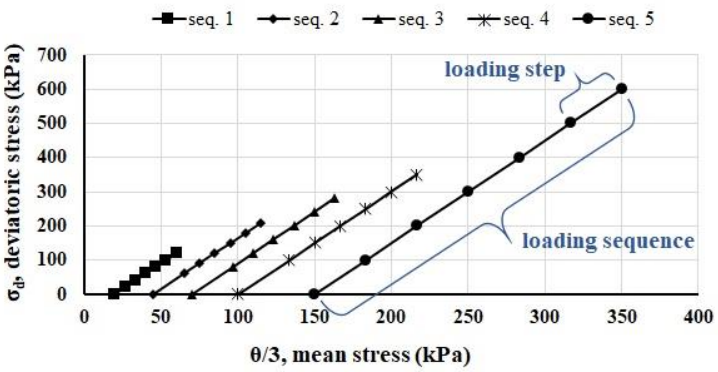

Fig. 5. Loading sequences for the MS LSL procedure.

The resilient modulus M_R associated with a change in the dynamic deviatoric stress σ_d^{dyn} and constant σ_t is defined as follows

$$M_R = \frac{\Delta\sigma_d^{dyn}}{\varepsilon_a^{el}}\,, \tag{1}$$

where ε_a^{el} is the axial resilient strain. Several non-linear relationships have been proposed to describe M_R with reference to bulk stress θ (Lekarp et al., 2000a). The following k-θ relationship is adopted (Hicks and Monismith, 1971)

$$M_R = k_1\sigma_a\left(\frac{\theta}{\sigma_a}\right)^{k_2}\,, \tag{2}$$

where σ_a is a reference pressure (100 kPa) and k_1, k_2 are regression parameters. The relationship enables a clear comparison in two-dimensional plots between the materials performances.

The permanent deformation is investigated through the Coulomb approach (Hoff et al., 2003). The Coulomb criterion relates the mobilized shear strength to the development of permanent deformations and the maximum shear strength to incremental failure. The mobilized angle of friction ρ and the angle of friction at incremental failure φ respectively express the degree of mobilized shear strength and the maximum shear strength. The angle of friction and the angle of friction at incremental failure identify three different ranges of material behaviour: elastic, elasto-plastic and failure. The strain rate $\dot{\varepsilon}$ is a measure of the speed of the permanent deformation; this parameter refers to the development of permanent deformation per cycle. Table 1 defines the two boundary lines between the three aforementioned ranges: each load step is categorised considering the average strain rate for the cycles from 5000 to 10,000 (Hoff at al. 2003).

Permanent strain rate	**Range**
$\dot{\varepsilon} < 2.5 \cdot 10^{-8}$	elastic zone
$2.5 \cdot 10^{-8} < \dot{\varepsilon} < 1.0 \cdot 10^{-7}$	elasto-plastic zone
$\dot{\varepsilon} > 1.0 \cdot 10^{-7}$	plastic (incremental failure) zone

Table 1. Permanent strain rate values defining the material range boundary lines.

The equations defining the elastic limit line and incremental failure line are respectively

$$\sigma_d = \frac{2 \sin \rho \; (\sigma_3 + a)}{1 - \sin \rho}, \tag{3}$$

$$\sigma_d = \frac{2 \sin \varphi \; (\sigma_3 + a)}{1 - \sin \varphi}; \tag{4}$$

a regression analysis is used to assess the boundary lines. As a simplification, the apparent attraction *a* is assumed to be 20 kPa for all the samples (Uthus et al., 2007).

2.2 TECHNIQUES TO IMPROVE THE MECHANICAL PROPERTIES

The research investigates different approaches to improve the mechanical properties of the "weak" metamorphic rocks, namely materials M2 and M3. Both standard tests and RTLTs evaluate the performance related to the assessed techniques. Replicate specimens are used and average results are estimated: two samples for a MDE test (each MDE test requires a double sample in turn), three samples for a LA test and two samples for a RTLT are considered for each testing condition described below.

2.2.1 MIXTURE OF THE ROCKS AVAILABLE IN SITU

The major part of the rocks spread along the highway alignment have mafic igneous origin and are suitable for road construction; therefore, a convenient solution could be to mix appropriately the different rock types that are available in situ. Material M1 is mixed with materials M2 or M3 according to three proportions in mass (25%, 50%, 75%). LA and MDE tests respectively express resistance against fragmentation and wearing (Erichsen et al., 2011): these standard procedures validate the appropriateness of the mixing approach.

2.2.2 POLYMER-BASED ADDITIVE APPLICATION

The tested polymer-based additive is water-soluble, non-leachable and UV, heat stable. The additive is a nanoscale technology and is made of two components C1 and C2; the additive modifies the rocks' surfaces and mechanical improvements can be measured at a macroscale level (Huang and Wang, 2016; Paul and Robeson, 2008; Roco, 2003; Sobolev and Shah, 2015).

Component C1 is an acrylic co-polymer emulsion based on acetic acid and methanol. The particle size is lower than 90 nm and has almost the same number of polymer particles as soil particles. Component C2 is a polymeric dispersion based on propylene glycol and alkoxy-alkyl silyl. After hydrolysis, the formed silanol (Si-OH) group can condense with another silanol group belonging to the silicate-containing surface of the rocks and form a siloxane linkage (= Si-O-Si=), namely a strong chemical covalent polar bond. Therefore, component C2 converts the water absorbing silanol groups presented on the rocks surface to a 4-6 nm layer of hydrophobic alkyl siloxane. Components C1 and C2 impart water resistance, better lubrication for compaction and bonding action bonds at ambient temperature. The existing positive experience refers to silty and clayey soils (Daniels and Hourani, 2009; Ugwu et al., 2013), therefore the research experiments with a new application context. The additive loses its effect in conditions that are seldom achieved in road construction: prolonged exposure to base (Wasserman et al., 1989) or air temperature above 200°C (Kim et al., 2003).

The product is mixed at OMC and added to M2 or M3. Two different additive proportions are tested: 1 kg C1 + 1 kg C2 for 200 l water (proportion P1) and 10 kg C1 + 10 kg C2 for 200 l water (proportion P2), no special curing procedures are necessary. Proportion P2 has been studied after the initial proportion P1 suggested by the product supplier, since the results regarding P1-treated materials have not been too different from the untreated materials (especially for M3).The materials M2 and M3 enhanced performances are assessed by RTLTs. Furthermore, the beneficial coating effect promoted by the additive is evaluated by the standard procedures in terms of resistance against fragmentation (LA test) and wearing (MDE test). In this case, materials M2 and M3 are soaked with the additive (50% C1, 50% C2) and tested after 24 hours to let the crushed rocks dry.

2.2.3 LIGNIN-BASED ADDITIVE APPLICATION

The lignin-based additive (also referred to as lignosulfonate) is a renewable product of pulp and paper industry. It comes from lignin, which is generated by extracting fiber and wood pulp from plant biomass; lignin global annual production is approximately equal to 50 million tons (Angenent et al., 2004). Lignosulfonate is an organic polymer that consists of both hydrophilic and hydrophobic groups; it is a non-corrosive and non-toxic chemical (Alazigha et al., 2018). Previous experiments investigating the strength and density modification of unpaved road using lignosulfonate showed promising outcomes for silty and clayey soils (Alazigha et al., 2018; Chen et al., 2014; Santoni et al., 2002; Ta'negonbadi and Noorzad, 2018; Zhang et al., 2018). As in the case for the polymer-based additive, the product application to crushed rocks could bring to a wider acceptance of this admixture.

The product is mixed at OMC and added to M2 or M3; the mass percentage of lignosulfonate added to the crushed rocks is 1.5%. RTLTs assess the materials M2 and M3 enhanced performances. Lignosulfonate needs a curing time to dry in order to become effective and attach properly to the material particles (Santoni et al., 2002). To simulate a long field curing process, each RTLT sample is firstly conditioned at 50°C for 24 hours and then at 22°C (room temperature) for 24 hours before testing. LA and MDE tests assess the beneficial coating effect provided by the lignin-based additive. Materials M2 and M3 are soaked with the product and undergo the same curing procedure.

2.2.4 OVERHEATING

The temperature sensitivity of rocks is subject to the geological formation process and mineralogical composition; they can achieve increased mechanical strength by heating (Zhang et al., 2009). Research done on diabase shows an enhanced compressive strength after heating at 190°C and 345°C compared to the investigation results referring to 27°C and 110°C (Simpson and Fergus, 1968). Another study is related to drying six different rocks (marble, limestone, granite, slate and two sandstones) with a temperature higher than 100 °C: an irreversible average 6% increase in the compressive strength properties is attained (Obert et al., 1946). These experiences prove that exposing the materials to high temperatures may change the original mechanical strength. Materials M2 and M3 are conditioned at 175°C and 250°C for 24 hours and 48 hours. After cooling down, micro-Deval standard tests are accomplished.

3 TEST RESULTS AND DISCUSSION

3.1 MIXTURE OF THE ROCKS AVAILABLE IN SITU

Three proportions in mass (25%, 50%, 75%) of “weak” material M2 or M3 are tested in combination with “strong” material M1. Figure 6a and Figure 6b display the results for M2 and M3 respectively.

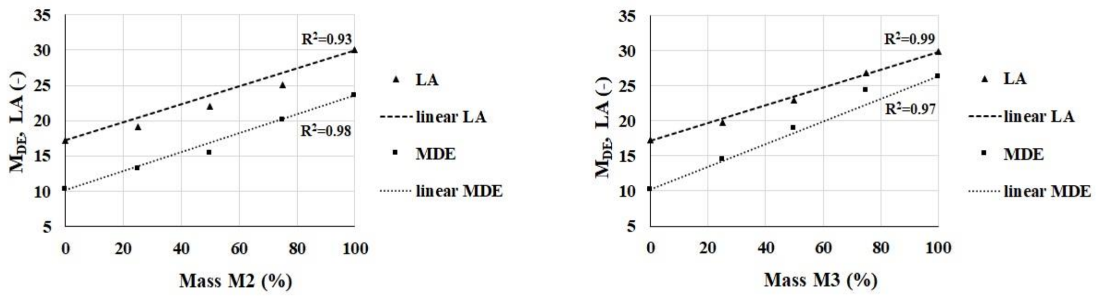

Fig. 6. LA and MDE results and linear trend distributions
for mixtures made of M2 and M1 (a), M3 and M1 (b).

The results distribution highlights a linear trend regarding both LA and MDE tests. The following equations describe the observed data in a mixture made of two materials i and j

$$LA_{i+j} = LA_i \frac{m_i}{m_i+m_j} + LA_j \frac{m_j}{m_i+m_j}, \quad (5)$$

$$M_{DE,i+j} = M_{DE,i} \frac{m_i}{m_i+m_j} + M_{DE,j} \frac{m_j}{m_i+m_j}; \quad (6)$$

where m_i, m_j are the masses and LA_i, LA_j, $M_{DE,i}$, $M_{DE,j}$ are the standard test values assessed for materials i, j. A proper mixture of different crushed rocks types is an effective method to meet the code requirements and, possibly, to maximize the use of the "weak" material.

This requires an extra processing step and space to store the two (or more) qualities of rock; on the other hand, the clear linear results for different combinations should bolster a stable production: the economic feasibility has to be evaluated depending on the local specific conditions. The mixed materials are not tested in the RTLT device, but it is reasonable to believe that the mechanical properties show similar linear trends.

3.2 POLYMER-BASED ADDITIVE APPLICATION

RTLTs assess the stiffness and the deformation properties of the investigated materials at OMC w=5%; the behaviour of untreated M1, M2 and M3 serves as a comparison basis. Figure 7a displays the bulk density and dry density at OMC, Figure 7b illustrates the bulk density after the addition of the polymer-based product according to proportions P1, P2.

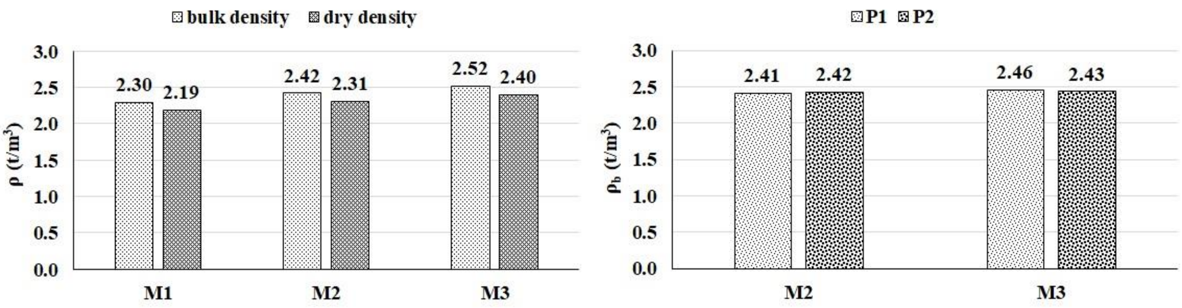

Fig. 7. M1, M2 and M3 bulk density and dry density at OMC w=5% (a),
M2 and M3 bulk density after additive application with P1, P2 proportions (b).

Resilient modulus k-θ relationships are evaluated (equation 2) through data regression, Figure 8 displays the results. Figure 8a shows the performance of M1, M2 and M3 at OMC without additive; material M1 is stiffer than materials M2 and M3. Figure 8b illustrates the enhanced behaviour of M2 and M3: they get stiffer when changing from proportion P1 to proportion P2; the "weak" rocks performance becomes comparable with the "strong" rocks behaviour. The results show the product is more effective on M2 than M3 probably because of the geological composition. The additive performance is dependent on the quantity of silicate minerals on the rocks surface. M2 and M3 have similar content of quartz and feldspar, but M2 is richer in amphibole and with a more distinct content of mica. M3 has a higher content of epidote-zoisite, which has lower Si-contents, as well as calcite ($CaCO_3$) which does not contain silicon.

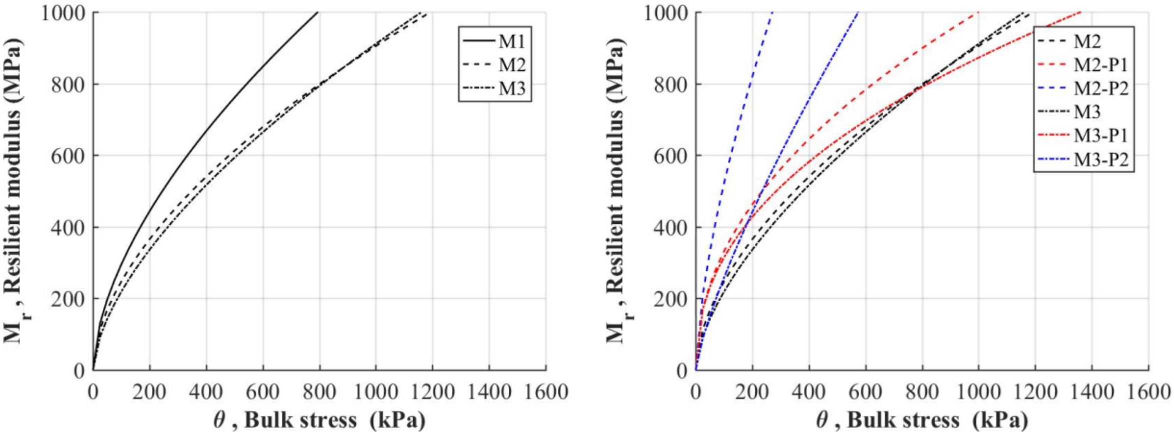

Fig. 8. M1, M2 and M3 resilient modulus at OMC w=5% (a),
M2 and M3 resilient modulus after additive application with P1, P2 proportions (b).

The additive effects also pertain to the deformation properties of the materials. Figure 9 depicts the mobilized angle of friction ρ and the angle of friction at incremental failure φ for the crushed rocks without and with the polymer-based stabilization; the additive application enhances both the angles.

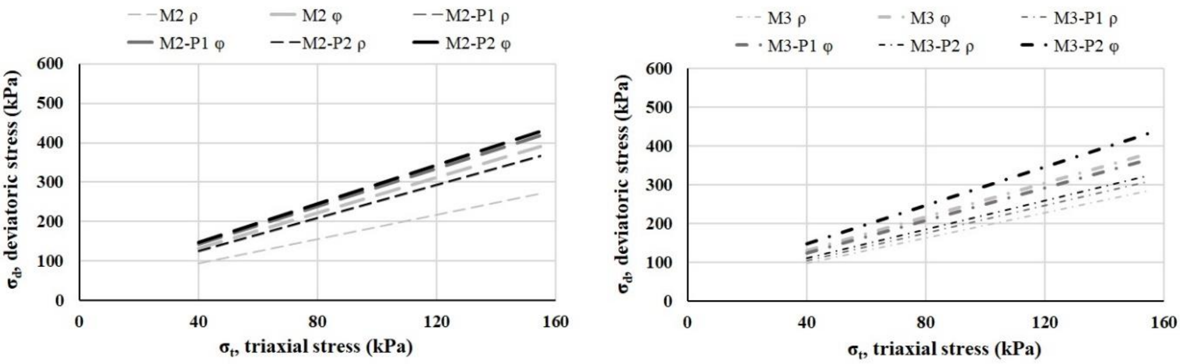

Fig. 9. Mobilized angle of friction ρ and angle of friction at incremental failure φ of M2 (a) and M3 (b) for untreated and additive-treated conditions.

Table 2 reports the k-θ model regression parameters and the values of the boundary angles.

Material	Model parameters		R^2	Limit angles	
	k_1	k_2		ρ	φ
M1	2994	0.59	0.99	58.4	64.9
M2	2467	0.56	0.99	57.2	65.8
M2-P1	3314	0.48	0.99	64.4	67.3
M2-P2	5206	0.65	0.99	64.6	67.8
M3	2184	0.62	0.99	58.5	65.3
M3-P1	3142	0.44	0.99	60.4	64.4
M3-P2	2576	0.78	0.99	61.6	68.0

Table 2. Regression parameters of M_R model and ρ, φ angles for M1, M2 and M3.

LA and MDE standard tests are used to assess the coating effect provided by the polymer-based product. Figure 10 shows that the additive supplies a significant beneficial effect when it comes to MDE wearing, it also provides a lesser benefit regarding the LA fragmentation. The steel balls used in the LA test imply higher impact loads compared to MDE test; therefore, the thin coating protection is more effective in the latter case.

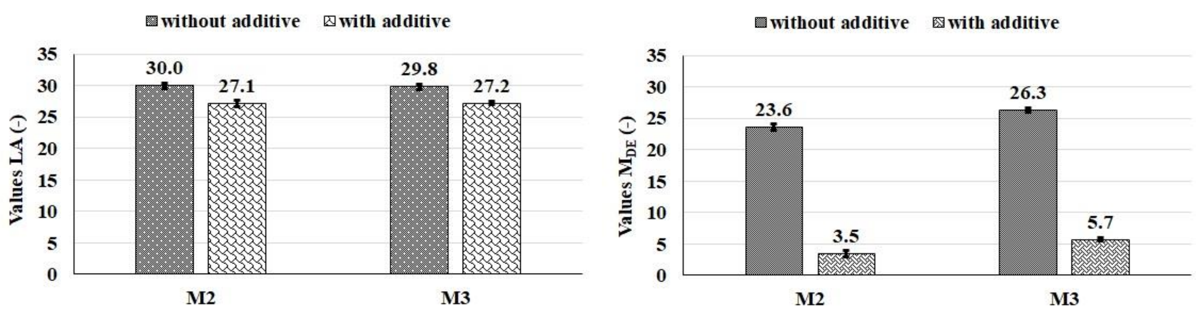

Figure 10. Coating effect provided by the polymer-based additive assessed by standard procedures: LA test (a) and MDE test (b).

3.3 LIGNIN-BASED ADDITIVE APPLICATION

RTLTs assess the stiffness and the deformation properties of the investigated materials, which are mixed with 1.5% lignosulfonate in mass at OMC w=5%; the measured water contents after the curing process are between 2% and 2.5%. The behaviour of the untreated materials at w=1% serves as a cautious comparison basis, this is a cautious comparison since M_R gradually reduces as w increases (Erlingsson et al., 2017). Figure 11a displays the bulk density and dry density at w=1%, Figure 11b illustrates the bulk density relative to the main stages of the curing process after additive application.

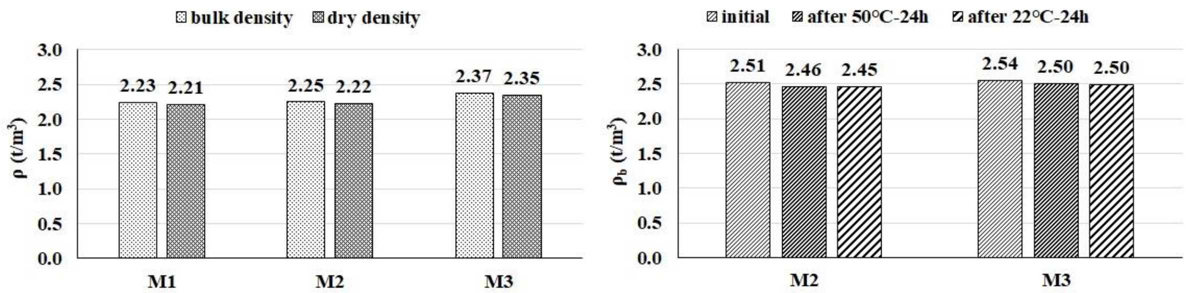

Fig. 11. M1, M2 and M3 bulk density and dry density at w=1% (a),
M2 and M3 bulk density after additive application (b).

Resilient modulus k-θ relationships are evaluated (equation 2) through data regression, Figure 12 displays the results. Figure 12a shows the performance for M1, M2 and M3 at w=1%; material M1 is stiffer than M2 and M3. Figure 12b illustrates the enhanced stiffer resilient curves of materials M2 and M3.

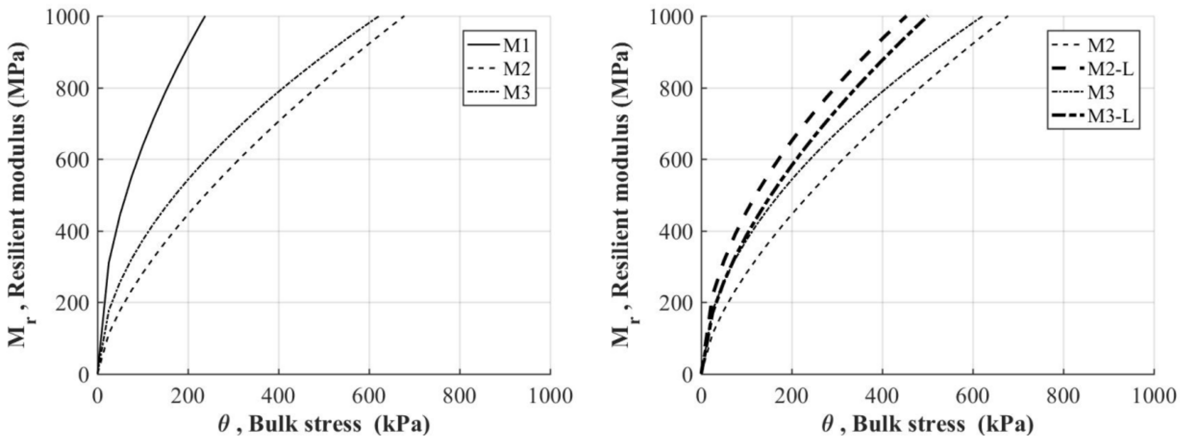

Fig. 12. M1, M2 and M3 resilient modulus at w=1% (a),
M2 and M3 resilient modulus after additive application (b).

The additive effects also pertain to the deformation properties of the materials. Figure 13 depicts the mobilized angle of friction ρ and the angle of friction at incremental failure φ for the crushed rocks without and with the lignin-based stabilization; the additive application enhances both the angles.

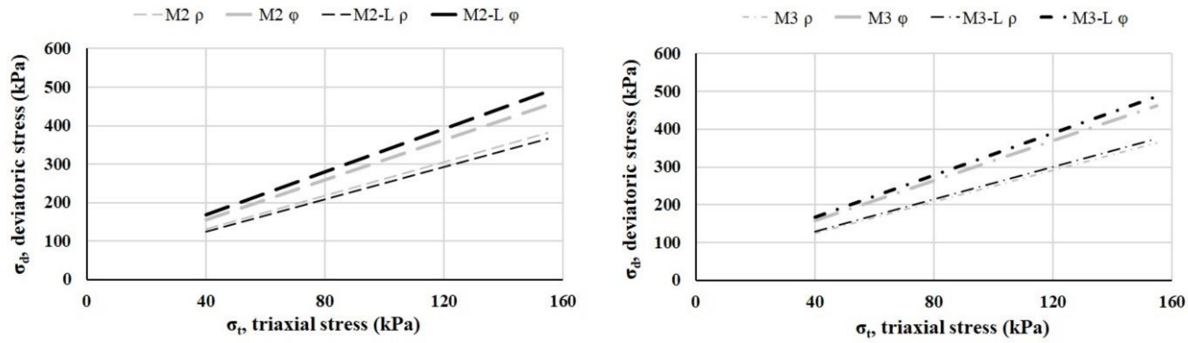

Fig. 13. Mobilized angle of friction ρ and angle of friction at incremental failure φ of M2 (a) and M3 (b) for untreated and additive-treated conditions.

Table 3 reports the k-θ model regression parameters and the values of the boundary angles.

Material	**Model parameters**		**R^2**	**Limit angles**	
	k_1	**k_2**		**ρ**	**φ**
M1	6378	0.52	0.99	62.1	67.3
M2	2816	0.66	0.99	65.4	68.9
M2-L	4530	0.52	0.99	64.4	70.3
M3	3737	0.54	0.99	64.3	69.2
M3-L	3869	0.59	0.99	65.0	70.2

Table 3. Regression parameters of M_R model and ρ, φ angles for M1, M2 and M3.

LA and MDE standard tests are used to assess the coating effect when soaked with the lignin-based additive. Figure 14 shows that the product provides a beneficial effect in terms of LA fragmentation; there is also a small enhancement in terms of MDE wearing. The improvement in MDE values is not as pronounced as one would expect because lignosulfonate is highly moisture susceptible (Santoni et al., 2002); this property is especially relevant when referring to running water, as in the case of MDE test.

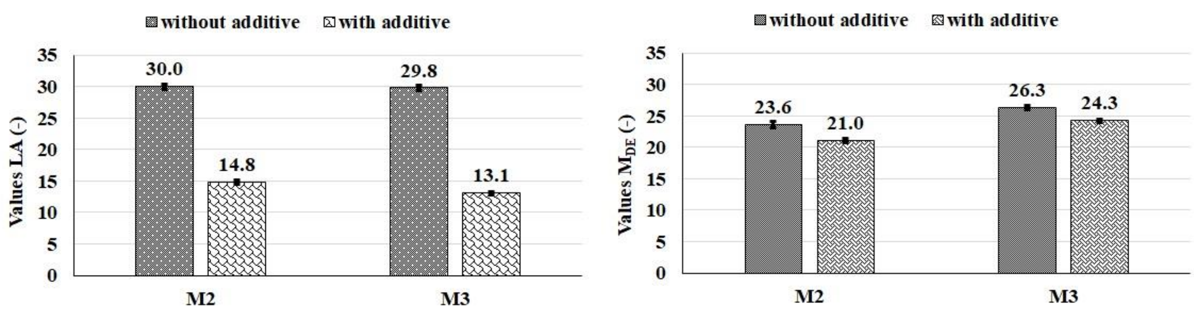

Figure 14. Coating effect provided by the lignin-based additive assessed by standard procedures: LA test (a) and MDE test (b).

3.4 OVERHEATING

Materials M2 and M3 are tested with the MDE procedure to assess if the overheating can induce modifications in the mineralogical structure, which may strengthen the rocks and improve the MDE results. The values of the three tested conditioning temperatures (105°C, 175°C, 250°C) do not seem to exert a major influence. On the other hand, the duration of the overheating seems to be a more relevant factor, as the results connected to the 48-hour conditioning are better than those connected to the 24-hour conditioning (Figure 15). Even so, the induced improvement is considerably limited as the highest observed decrease among the original MDE values is equal to approximately two units. The tested conditioning temperatures do not significantly improve the weak materials to make them meet the code requirements. Anyway, overheating the crushed rocks would demand an intensive energy use even if a small beneficial effect was found.

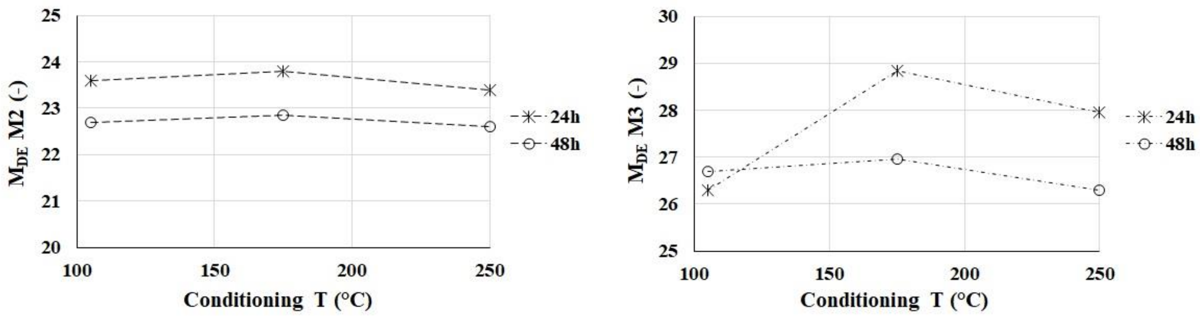

Fig. 15. MDE values after overheating M2 (b) and M3 (b).

4 CONCLUSIONS

The research investigated different approaches to improve the mechanical properties of crushed rocks to serve as construction materials in the road unbound layers. Three types of crushed rocks M1, M2 and M3 were tested; M1 satisfies the code requirements, while M2 and M3 do not meet them. The following conclusions can be drawn; they could be generalized by further testing of more materials:

(1) Both RTLTs and LA, MDE tests identify that material M1 has a better performance than materials M2 and M3. RTLTs can also assess the beneficial effect of the additive; on the other hand, there are no standard LA and MDE test procedures concerning an additive application.

(2) An appropriate mixture of the crushed rocks available in situ may represent a convenient solution to fulfil LA and MDE requirements. The LA and MDE values of the mixture can be evaluated by a linear relationship based on the mass quantity of each employed material.

(3) The pavement design code does not prohibit mixing different materials and certifying the obtained admixture as the chosen construction material. However, it is possible that weaker grains might be exposed to extra wear and/or crushing when they are mixed with the strong particles, especially if the difference in LA/MDE is large. In addition, there may be practical problems when it comes to securing a good mix and avoid segregation.

(4) Both polymer-based additive and lignin-based additive are non-traditional stabilization approaches to improve the mechanical properties of crushed rocks and enable their use in the pavement unbound layers. The investigated additives coat and bond the material particles closely together; the polymer-based product has a rapid effect, the lignin-based product needs a curing time.

(5) The effect of the polymer-based additive is dependent on the silicate content of the rocks surface. The lignin-based additive application may need to consider the environmental conditions, as it is water susceptible. A suitable solution could be to apply this admixture during days without rainfall and then cover the treated layer with a top (e.g. bituminous) layer; another option could be to make sure that the water can properly drain (e.g. good profile and well-maintained ditches).

(6) The promising results obtained by laboratory tests and discussed in this study should be further investigated, for instance by means of a field-test campaign. Furthermore, repeating the measurements after a long time interval may be useful to assess the long-term response of the stabilizing products and assessing the possible formation of micro plastics on the surface.

(7) The attained overheating procedure does not produce a substantial improvement in the materials performance.

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