Modelling the compressive mechanical behaviour of granite and sandstone historical building stones

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ABSTRACT

Building stones, particularly sandstone and granite, are very important in the building elements of Portugal’s historical and cultural heritage. Experimental research, based on uniaxial compressive tests, was carried out on selected representative samples of lithotypes of rocks used in historic built heritage, with a view to evaluating the compressive mechanical behaviour of different building stones. The results showed that porosity plays a central role in the compressive behaviour of granites and sandstones. As porosity can be evaluated in field conditions with non-destructive tests it was decided to derive an analytical model to predict compressive behaviour based on the knowledge of porosity of the building stones. A cubic polynomial function was adopted to describe the pre-peak regime under compression to implement the model. Furthermore, a statistical correlation between mechanical and porosity data had to be defined. Good agreement between experimental and analytical compressive stress–strain diagrams, from which the mechanical properties like compressive strength and modulus of elasticity can be derived, was achieved.

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1. Introduction

Built heritage such as castles, churches and palaces play an important role in the cultural life of Portugal. In general, massive masonry walls characterize the construction of these ancient constructions and natural stone is the most widely applied material. The use of dimension stones in traditional constructions is closely related to the distribution of rock outcrops. Granitic rocks are predominant in the northern and central regions of Portugal, but it also possible to find them in some important monuments in the south. Sandstone is less widely distributed in Portugal, but its use is common in traditional buildings at regional level, particularly in the Western regions close to the sea (Peniche, Lourinhã and Silves). Fig. 1 shows some traditional buildings with granite and sandstone loadbearing masonry.

Conservation, rehabilitation and strengthening of the built heritage are clearly required by modern societies, meaning that appropriate intervention techniques on materials and structures should be available. The proper rehabilitation of ancient buildings should be based on appropriate diagnosis and understanding of the existing materials [1]. In addition, the principles of safeguarding archi-

tectural heritage according to the international charters of Athens cited by Venice [2] and Krakow [3] recommend that studies should be carried out on the building stone with the lowest degree of intrusion and fullest respect for their physical integrity. In fact, one of the main problems of diagnosis with respect to an ancient building is the difficulty of removing material for mechanical and physical characterization. The principle of minimum intrusion has been broadly taken into account by the scientific community, which has been proposing alternative non-destructive techniques to evaluate the mechanical and physical properties of construction stone [4,5]. Ultrasonic pulse velocity (UPV) and the Schmidt hammer (rebound hammer) are two examples of simple and inexpensive solutions that can predict the elastic mechanical properties and the weathering state of building stones [6]. Porosity is a property that can be estimated also by a Schmidt hammer and by ultrasonic pulse velocity [6–8].

The dependence of the compressive mechanical properties on the physical properties of rocks has been reported by several authors [9–14]. This relation was also assessed for granite using a set of statistical correlations between mechanical and physical properties [15]. In general, increasing porosity is associated with decreasing compressive and tensile strength and a lower modulus of elasticity. This behaviour is to great extent related to the higher heterogeneity and presence of weak bonds such as pores, voids and microcracks in very porous rocks.
The dependence of the basic mechanical properties on the physical properties (porosity) can make the mechanical evaluation of existing building stones in old masonry walls much easier.

Following this idea, a model is proposed that describes the compressive mechanical behaviour of distinct building stones based on their physical properties. The analytical model proposed simulates the mechanical compression behaviour of granite and sandstone in terms of the stress–strain relation as a function of physical (porosity) and mechanical parameters (compressive strength and modulus of elasticity).

The implementation of this method involves a first phase of experimental investigation of the physical and mechanical properties of the building stones under compressive loading (modulus of elasticity and compressive strength). Once the model has been defined, it is intended to use it to predict the basic engineering properties, based on porosity, which can be given by non-destructive tests.

The major significance of the proposed method is the possibility of gathering enhanced information on the basic engineering properties of ancient building stones without using destructive testing. It should be stressed that compressive strength and the modulus of elasticity are the most important mechanical properties needed to estimate masonry's compressive strength. In addition, these properties have a major role in the numerical simulation of old buildings.

2. Selection of rock lithotypes

The analytical model was developed for sandstones and granites based on the results of experimental work on the mechanical and physical properties of granitic rocks and sandstones.

The granitic stones studied were mostly collected from the northern region of Portugal, i.e. from Affife (AF), Ponte de Lima (PTA), Mondim de Basto (MDB) and Gonça (GA). Mineralogical, textural and structural characteristics were used to select granite types. In this paper only the results obtained for fine to medium and medium granites are reported. The mean length of sections intercepted by a single circle was measured in order to evaluate the grain size of the granitic types, in accordance with the principles of the Hilliard single-circle procedure described in ASTM E112-88 (1995) [16]. Four circles were studied for each granitic facies and sections in the less weathered granitic types were considered. Mean length of sections measured was about 0.5–0.6 mm in GA and AF and about 0.7–0.9 mm in MDB and PTA lithotypes. The smallest grain sizes were about 0.3 mm in GA, MDB and PTA lithotypes, while the smallest grain size of 0.1 mm was recorded in AF granite.

The sandstones were collected in Atouguia da Baleia, in Peniche, a region in the centre of Portugal [17]. Four varieties, which are representative of the two lithotypes in existing monuments, were identified. It should be noted that neither coeval quarries nor outcrops of similar materials to those used in the monuments could be found in areas near to Peniche. Thus, stone masonry walls were selected in the vicinity of the built heritage and some samples were extracted from them, taking into account their similarity in terms of appearance, mineralogical composition, texture and structure, to the stone in the monuments. Physical tests were also carried out to determine porosity. The four varieties have similar porosity to the stone found in the monuments. Both lithotypes have the same classification according to Folk [18], i.e. they are classified as lithic arkose [17].

The lithotype designated A + B, which includes the varieties A and B, has around 34–40% carbonates and 30–32% quartz, whereas the lithotype C + M encompasses typology M which has about 20–21% carbonates and 45–51% quartz. The carbonate content in both lithotypes is so significant such that they were designated as lithic arkose with carbonate cement. In this paper only the results of varieties A, B and M are shown.

Lithotype A + B exhibits macroscopically well defined lineations and variety A has clearly visible laminations. Lineations were not detected in variety M. However, in thin sections under a polarizing microscope, variety A exhibits one preferred orientation of mica minerals and variety B shows no preferred orientations, with lineations being randomly distributed. Thin sections of variety M show two preferred orientations of mica minerals. All these varieties have about 4–6% mica minerals.
The average size of grains of quartz and feldspar in the sandstone varieties A and B ranges from 0.1 to 0.13 mm, and in variety M, the average size is about 0.24 mm. Sandstones A and B are generally fine-grained, whereas variety M sandstones are medium to fine to grained [17].

The smallest grain sizes in granite lithotypes GA, MDB and PTA are similar to the average size of grains in sandstone variety M. Granite lithotype AF has the smallest grain sizes of the four granitic lithotypes studied, which correspond to the average size of sandstone lithotype A + B grains.

3. Experimental programme

3.1. Introduction

The experimental programme was carried out in the laboratory and involved uniaxial compression tests to obtain the stress–strain diagrams and the mechanical engineering properties (compressive strength and modulus of elasticity), and porosity tests to obtain physical properties (porosity and density). In this section, the details of experimental testing are provided and experimental results are discussed.

3.2. Preparation of samples

The granite lithotypes selected in this study are part of a group that was subject to extensive experimental research for the mechanical characterization of different types of granite which are typical of most historical and vernacular buildings in the north of Portugal [8]. For the mechanical characterization of granites, it was decided to use cylindrical specimens with a diameter of 75 mm and a height to diameter ratio of approximately two. These measurements followed the recommendations of ISRM [19] so that representative samples of the studied granites could be obtained. The granite selected exhibited no significant planar anisotropy. The direction of loading was always parallel to the rift plane. As no macroscopic lineations were detected in variety M, the prismatic specimens were randomly cut [17].

The authors carried out tests to determine other physical properties, such as bulk density. Additional tests were carried out on the sandstones to determine the absorption of water at low pressure and by capillarity, as well as to determine the mercury intrusion porosimetry [17].

The porosity tests were carried out on all specimens used in the mechanical characterization to enable a direct correlation between porosity and mechanical properties. The porosity tests were carried out on all specimens used in the mechanical characterization to enable a direct correlation between porosity and mechanical properties. The average porosity obtained for granites and sandstones are presented in Table 1. The values range from 0.42% (granite GA) to 5.23% (granite MDB). The MDB porosity is rather high, indicating that this granite is considerably more weathered than the other granites studied, and this is denoted macroscopically by the change of colour and the rough surface. According to Goodman [9] the expected porosity in fresh granites is lower than 1% but the porosity of igneous rocks tends to rise to 20% or more as weathering degree increases.

The porosity of weathered granites can reach values near the lower porosity of sound sandstones. The M sandstone samples exhibit the highest porosity of the varieties studied. In relation to the sandstones, there is a clear difference in the porosity of varieties A, B and M, with values ranging from 3.6% to 18.6%.

The permeability of weathered granites can be a simple way to assess their quality and can assist with the interpretation of the results achieved by mechanical characterization [9]. Previous studies have shown that mechanical properties such as compressive strength and elastic modulus are dependent on porosity and density [12, 20, 21].

The porosity and density of the granites were determined according to the method suggested by ISRM [19], while the porosity and density of the sandstones were obtained following the Recommendations of RILEM [22] and EN1936 [23]. Both standards suggest using a vacuum to saturate the specimens. Fig. 2 shows the experimental apparatus. The hydrostatic weighing was carried out after air voids were filled with trapped water. The grain mass, , is defined as the equilibrium mass of the sample after oven drying at a temperature of 105°C. The pore volumes accessible to water were then determined by using the Archimedes principle allowing to calculate porosity and real densities.

The authors carried out tests to determine other physical properties, such as bulk density. Additional tests were carried out on the sandstones to determine the absorption of water at low pressure and by capillarity, as well as to determine the mercury intrusion porosimetry [17].

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Weathering of sandstone is responsible for a greater increase in their porosity than in that of granite, with figures of up to 40% and nearly 50% being achieved in sandstone [10]. Those higher figures are very close to those reported by Turgul and Zarf [24] for weathered sandstones.

Ludovico-Marques [17] presented the pore size distribution of sandstone varieties B and M obtained by mercury intrusion porosimetry. Microporosity settled as the percentage of pores radii lower than 7.5 μm [25], is 80–85% in variety B and about 75% in variety M.

Several authors studied granites in the North of Portugal which generally show microporosity values higher than macroporosity values [26–29]. Microporosity of MDB granite (Lamares type) is around 80% and it is very similar to microporosity values of B sandstones. Microporosity values of fresh granites cracks develop in the subparallel direction to loading, at an angle almost 10° below the axial longitudinal axis of the specimen. In weathered granites cracks develop in the subparallel direction to loading, at an angle almost 10° below the axial longitudinal axis of the specimen. In weathered granites the unstable microcracking occurs for the crack damage stress level, and it is associated with the point of reversal in the total volumetric strain diagram. This stage is connected to the maximum compaction of the specimen and to the onset of dilation since the increase in volume generated by the cracking process is larger than the standard volumetric decrease due to the axial load. For this reason a rapid and significant increase of the lateral strains is observed, as a result of the volume increase. The microcracking spreading is no longer independent, the local stress fields begin to interact and the previously formed microcracks tend to coalesce. After the peak load is reached, the material becomes weaker and the strain is concentrated in the weaker elements (strain localization), which constitute the damaged zone [8].

After the peak load is reached, the compressive behaviour is characterized by microcracking growth as strain localization occurs. Macrocracks result from coalescence of the microfractures developed until the peak load is reached. At this stage, the unstable microcracking occurs for the crack damage stress level, and it is associated with the point of reversal in the total volumetric strain diagram. This stage is connected to the maximum compaction of the specimen and to the onset of dilation since the increase in volume generated by the cracking process is larger than the standard volumetric decrease due to the axial load. For this reason a rapid and significant increase of the lateral strains is observed, as a result of the volume increase. The microcracking spreading is no longer independent, the local stress fields begin to interact and the previously formed microcracks tend to coalesce. After the peak load is reached, the material becomes weaker and the strain is concentrated in the weaker elements (strain localization), which constitute the damaged zone [8].

3.4. Characterization of mechanical behaviour

3.4.1. Experimental procedures for monotonic uniaxial compression tests

The uniaxial compression tests on the sandstones and granites were carried out in two Portuguese Universities. The sandstones were tested at the Laboratory of Structures of Universidade Nova de Lisboa, and the granites were tested at Laboratory of Structures of University of Minho.

The uniaxial compression tests on the sandstones used a Seidner servo-controlled press, model 3000D, with load capacity up to 3000 kN and a piston stroke of 50 mm [17]. The tests were carried out under axial displacement control at a rate of 10 μm/s. One displacement transducer (LVDT) was attached at each side of the specimen between plates of the testing machine. The average displacement was calculated from the displacements measured in the four LVDTs. These displacement transducers have 100 mm of stroke and 100 m/s. The expansion of the ring is made possible by the lateral spring. The lateral increment measured in the specimen is approximately 0.3 mm/m when the axial load is 2.5 times the axial compression strength. The stress–strain diagrams can only be recorded up to peak stress and the softening branch presents distinct negative slopes, with some of them even being positive (snap-back). The softening branch is much smoother in low-strength rocks. This trend can be seen in both sandstones and granites. Figs. 5 and 6 show the stress–strain diagrams for both rocks under analysis. It is observed that the post-peak response of low strength sandstones can be captured, whereas the stress–strain diagrams can only be recorded up to peak stress in the case of hard sandstones. Vasconcelos et al. [15] have shown that the post-peak behaviour of high strength granites can be recorded more easily if the circumferential displacement control is adopted in the compression tests. It is clear that, as with sandstone, the post-peak behaviour of granite depends on its strength. For high strength granites (GA, PTA) the post-peak shows a sharp decrease in stress for increasing displacements, whereas for soft granites (AF and MDB) the post-peak is smoother. In GA granites there are even few abrupt failures after peak load is reached. This behaviour can also be related to the structure of rocks described by the porosity, as it arises from the internal distribution and arrangement of the grains and internal microfractures, pores and voids.

The ‘post mortem’ evaluation of the failures of the tested specimens reveals that they appear to be related to weathering and consequently to the porosity level. In fresh granites cracks develop in the subparallel direction to loading, at an angle approximately 10° below the axial longitudinal axis of the specimen. In weathered and high porosity granites, however, the macrocracks occur within a shear band (Fig. 7), but in sandstone the double shear crack bands appear to be better defined in high porosity sandstones (Fig. 8). Clear double shear develops in the lithotype M sandstone specimens, whereas a more distributed subvertical cracking is more usua-
ally observed in the lithotype A + B specimens (Fig. 8). This can be associated with the existence of microcracks aligned according a preferential plane, as pointed out by Gupta and Rao [37].

The pre-peak behaviour is also dependent on the lithotype. Lower strength rocks also generally have lower values of initial stiffness than high strength rocks (Figs. 5 and 6). Comparison of the compressive strength and porosity values (Table 1) makes it clear that the porosity regulates the behaviour of rocks in uniaxial compression as it is the result of the distribution and arrangement of grains. It is shown that high porosity granites, which are associated essentially with more weathered levels, have considerably lower stiffness and lower compressive strength. The dependence of the compressive strength and stiffness of sandstones on porosity follows the same trend as the granites.

The strain corresponding to peak stress increases in both the sandstones and the granites as the compressive strength decreases, which is related to the lower stiffness of low strength granites and sandstones. This means that this parameter is also dependent on porosity. Increasing values of porosity are associated with increasing deformation at peak stress, as shown in Table 1, confirming that porosity plays a major role on the behaviour of rocks under compression.

4. Analytical modelling of compressive behaviour


Given the important role of porosity in the compressive behaviour of rocks, it was decided to find an analytical model that makes it possible to describe compressive behaviour from the knowledge of porosity.

The analytical model was developed for sandstone by Ludovico-Marques [17] and has also been used to predict the compressive behaviour of granites. As mentioned in 1, according to Vasconcelos et al. [15], the mechanical properties of homogeneous granite (without significant planar anisotropy) are also reasonably correlated with porosity.

The compressive stress, $\sigma$, developed in granites and sandstones can be determined by Eq. (1):

$$\sigma = f(e/e_R) \times \sigma_c$$

where $\sigma_c$ is the compressive strength of rocks and $f(e/e_R)$ is the shape function characterizing the pre-peak behaviour of the rocks under study.

The shape function $(f)$ is obtained by normalizing the compressive stress by the compressive strength, $\sigma_c$, and it is dependent on the strain, $e$, normalized by the strain at peak strength $(e_R)$:

$$f(e/e_R) = \frac{\sigma}{\sigma_c}$$

The shape function is calibrated based on the results of uniaxial compression tests for both the types of rocks considered in this work. Thus, from the analytical expressions defined by Ludovico-Marques [17] for sandstone and taking into account the experimental stress–strain diagrams for granite, it can be seen that the
pre-peak behaviour is nicely described by a cubic polynomial function. Thus the shape function valid both for the sandstones and granites is given by Eq. (3):

$$f(\varepsilon/\varepsilon_R) = -\frac{(\varepsilon/\varepsilon_R)^3}{C_0} + 1.47(\varepsilon/\varepsilon_R)^2 + 0.5(\varepsilon/\varepsilon_R)$$

(3)

The coefficient 1.47 that multiplies the square term in Eq. (3) tends to 1.5, so that the shape function tends to value 1 when the strain tends to $\varepsilon_R$.

Through direct substitution of Eq. (3) in Eq. (1), the compressive stress, $\sigma$, is given by:

$$\sigma = -\frac{(\varepsilon/\varepsilon_R)^3}{C_0} + 1.47(\varepsilon/\varepsilon_R)^2 + 0.5(\varepsilon/\varepsilon_R) \times \sigma_c$$

(4)

It should be stressed that the model can be easily applied to other types of rocks because their behaviour in the regime of pre-peak is easily adjusted to a cubic polynomial function.

4.2. Correlation between porosity and compressive strength and strain at peak stress

To implement the analytical model intended to describe the compressive behaviour of distinct types of rocks through porosity, statistical correlations need to be found between porosity and both experimental compressive strength and the strain at peak stress. Fig. 9 illustrates the variation of uniaxial compressive strength with porosity in the samples of sandstone and granite lithotypes. The regressions that best fit the experimental results listed in Table 1 are set forth as Eq. (5) for the sandstones and Eq. (6) for the granites:

Fig. 9. Failure modes of granite specimens tested in monotonic uniaxial compression: (a) fresh granites; (b) weathered granites.
\[ \sigma_c = 206.7e^{-0.129n} \quad (5) \]

\[ \sigma_c = 148.8e^{-0.263n} \quad (6) \]

In fact, there is a very significant correlation between the compressive strength of sandstones and granites with the porosity, which confirms that this parameter determines the behaviour of rocks under compression as discussed previously. This result is consistent with the correlations found between compressive strength and porosity of stones by other authors [20,21]. It should be noticed that porosity in granite may be hindered by high anisotropy, due to the fact that the compressive strength varies with the direction of loading and porosity is not obviously a directional property.

Another important parameter for the complete definition of the analytical model is the strain at compressive strength, which can be similarly correlated with the physical properties of materials. Fig. 10 shows the variation of strain at failure \( (e_R) \) with porosity in the samples of sandstone and granite lithotypes. As previously mentioned, there is a clear trend for the compressive strain at peak stress to increase as the porosity decreases, indicating that rocks with high porosity are more deformable.

As the porosity increases, so does the dispersion of strain values at failure \( (e_R) \), as Fig. 10 illustrates. Nevertheless, the coefficient of variation corresponding to strain at failure \( (e_R) \) obtained from a statistical analysis of sandstone variety M data (Table 1) is smaller than 10% and for the sandstone variety A is around 12%.

The regressions that best fit the experimental results listed in Table 1 are set forth as Eq. (7) for the sandstones and Eq. (8) for the granites:

\[ e_R = 0.0043n^{0.215} \quad (7) \]

\[ e_R = 0.0041n^{0.214} \quad (8) \]

Taking all the sandstone and granite specimens, the relation between strain at peak stress and porosity is given by the granites Eq. (9):

\[ e_R = 0.0042n^{0.222} \quad (9) \]

The better correlation \( (R^2 = 0.942) \) found between porosity and strain at peak stress is given for the granites by Eq. (8). The correlation between porosity and strain at peak strain found for the sandstones is considerably worse. Thus, it was decided to consider for the analytical model the correlation found, taking into consid-

Fig. 8. Failure modes of sandstone specimens tested in monotonic uniaxial compression. (a) A variety. (b) B variety. (c) and (d) M variety, front and rear features of specimens.
The results of both rocks. Most values of strain at compressive strength \( \varepsilon_R \) calculated through Eq. (9) vary by less than 10\% of the experimental values obtained for the sandstone and granite. In the granites only specimen GA5 differs by 16\%. In the sandstone samples AP13, BP13 and MP2 differ by 16\%, 18\% and 19\%, respectively.

It is clear that porosity, particularly the porosimetry distribution, influences the compressive mechanical behaviour of the material. On one hand, the higher amount of pre-existing microcracks, pores and voids contributes to higher initial deformations caused by their closure. The more porous microstructure has more voids, which reduces the stiffness of the stone skeleton and contributes to the increase in the strain at peak stress. Additionally, rocks with more porous microstructure also show a more remarkable nonlinear behaviour in the pre-peak regime, which also helps to increase the strain at peak stress [38].

The proposed model makes it possible to simulate the compressive mechanical behaviour of rocks in terms of stress–strain and so to find the compressive strength and the modulus of elasticity based on the knowledge of porosity. The proposed method is of major significance in terms of practical applications to the study of stone masonry buildings for the estimation of elastic properties, when the extraction of rock core samples is forbidden. Notice that the estimation of the stone’s mechanical properties is very important when it is needed to assess stability based on numerical simulation or simplified methods.

### 4.3. Comparison of experimental and analytical results

The analytical model for each stone is fully defined by substituting the compressive strength and strain at peak stress by the expressions that correlate them with porosity. So, in Eq (4), the compressive strength \( \sigma_c \) must be replaced by Eqs. (5) or (6) for

\[
\sigma_c = 206.7e^{-0.129n}
\]

\( R^2 = 0.987 \)

\[
\sigma_c = 148.8 e^{-0.263n}
\]

\( R^2 = 0.989 \)

\[
\varepsilon_R = 0.0043 n^{0.125}
\]

\( R^2 = 0.942 \)

\[
\varepsilon_R = 0.0042 n^{0.222}
\]

\( R^2 = 0.898 \)

\[
\varepsilon_R = 0.0041 n^{0.214}
\]

\( R^2 = 0.942 \)

\[
\varepsilon_R = 0.0043 n^{0.215}
\]

\( R^2 = 0.597 \)

\[
\varepsilon_R = 0.0042 n^{0.222}
\]

\( R^2 = 0.898 \)

\[
\varepsilon_R = 0.0041 n^{0.214}
\]

\( R^2 = 0.942 \)

\[
\varepsilon_R = 0.0043 n^{0.215}
\]

\( R^2 = 0.597 \)

\[
\varepsilon_R = 0.0042 n^{0.222}
\]

\( R^2 = 0.898 \)

\[
\varepsilon_R = 0.0041 n^{0.214}
\]

\( R^2 = 0.942 \)

\[
\varepsilon_R = 0.0043 n^{0.215}
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\( R^2 = 0.597 \)

\[
\varepsilon_R = 0.0042 n^{0.222}
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\( R^2 = 0.898 \)

\[
\varepsilon_R = 0.0041 n^{0.214}
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\( R^2 = 0.942 \)

\[
\varepsilon_R = 0.0043 n^{0.215}
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\( R^2 = 0.597 \)

\[
\varepsilon_R = 0.0042 n^{0.222}
\]

\( R^2 = 0.898 \)

\[
\varepsilon_R = 0.0041 n^{0.214}
\]

\( R^2 = 0.942 \)
the sandstones and granites respectively, and the strain at compressive strength \( (\varepsilon_p) \) by Eq. (9):

\[
\varepsilon = 206.7e^{-0.123n} - \left( \frac{E}{0.0042n^{0.222}} \right)^3 + 1.47 \left( \frac{E}{0.0042n^{0.222}} \right)^2 + 0.5 \left( \frac{E}{0.0042n^{0.222}} \right)
\]

(10)

In case of granites the stress–strain diagram can be obtained by the following equation:

\[
\varepsilon = 148.8e^{-0.263n} - \left( \frac{E}{0.0042n^{0.222}} \right)^3 + 1.47 \left( \frac{E}{0.0042n^{0.222}} \right)^2 + 0.5 \left( \frac{E}{0.0042n^{0.222}} \right)
\]

(11)

The performance of the analytical expression is assessed by comparing the experimental stress–strain diagrams with those obtained by Eqs. (10) and (11). From Figs. 11 and 12, where experimental and analytical stress–strain diagrams for the sandstones and granites are compared, it can be seen that in general good agreement is achieved between the experimental and analytical results. It is possible to observe that better agreement with respect to stiffness is achieved for the sandstones. But the analytical model allows the proper estimation of the compressive strength of the granites. This estimation is slightly better than in case of sandstones.

5. Conclusions

This paper has given a general overview of the compressive behaviour of two distinct types of masonry stones that are commonly used in old masonry buildings of historical value or in vernacular architecture. Besides the experimental details on the uniaxial compressive tests, a discussion of the main results has been provided, viz. the stress–strain diagrams and the values of the key parameters characterizing the pre-peak behaviour such as compressive strength and strain at peak stress. It has been shown that the compressive behaviour is influenced largely by porosity, which is a property connected to the arrangement of the internal skeleton of stones. More porous rocks have clearly lower compressive strength and higher strain at peak stress. The higher porosity was also found to influence the rocks’ stiffness (modulus of elasticity). Stiffer rocks are associated with low porosity.

Given this dependency, it was decided to derive an analytical model to describe the compressive mechanical behaviour of sandstones, and the model was then extended to granites. This model was defined by taking into account the general shape of the pre-peak stress–strain diagrams obtained in the experimental tests and by considering that it is well defined by a cubic polynomial function. The polynomial function is dependent on the compressive strength and on the strain normalized by the strain at peak stress. The final model for sandstones and granites was stated by considering the statistical correlations found between the compressive strength and strain at peak stress with porosity.

The performance of the analytical model was evaluated by comparing the analytical stress–strain diagrams with the stress–strain diagrams obtained in uniaxial compressive tests. A good agreement between the analytical and experimental results was found, meaning that the compressive behaviour can be predicted well when porosity is known. The major advantage of this procedure is that the mechanical properties can be estimated under compression without requiring the destructive testing of samples.

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