DILATANCY MODEL FROM GEOTECHNICAL TO STRUCTURAL ENGINEERING: A RESEARCH CASE FOR "EXPERT-GENERALIST"?

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Abstract

From past to present days, the evolution of science has driven towards deep fragmentation of research fields. With the continuous growth of knowledge, the increasing specialization seems a natural process that is essential for innovation and the search for new discoveries. The increasing specialization has given birth to new disciplines with the consequent tendency to "*divide*" experts of different fields. In this context, the role of the "*expert-generalist*", which is a figure or a learning processes that enhance the connections between disciplines, seems a further opportunity. This article presents a research case for the "*expert-generalist*" in the field of earthquake engineering where one of the most influential theory of geotechnical engineering has been successfully transferred to structural engineering with confirmation by experimental data. The phenomenon of dilatancy, which is traditionally found in dictionaries of geotechnical engineering, has proved to have practical implications in the definition of the shear strength of masonry samples. A recently proposed formulation that modifies the current approaches for the definition of the shear strength of mortar joints has been applied to a larger scale, for the modification of the shear-sliding failure mechanism of masonry walls.

Keywords: Dilatancy; Masonry; Expert-generalist; Shear-sliding, Failure mechanisms.

1. Introduction

Unlike modern researchers, scientists of the past were expert in different disciplines. This interdisciplinary vocation has led to the development of models still used today in different fields of research. Isaac Newton (mathematician, physicist, astronomer, etc.), Robert Hooke (biologist, geologist, physicist, etc.) and Charles Coulomb (engineer and physicist) are just few examples of great scientists of the past whose knowledge spans through different disciplines and their formulations are the basis of the modern approaches that are used still today in different fields. Over time, from past to present days, the evolution of science has driven towards deep fragmentation of research fields. The increasing specialization has given birth to new disciplines with the consequent growth of publications. A recent research [1] showed that the number of academic publications is doubling every 9 years.

Alike to biological speciation, which is the evolutionary process depicted by Charles Darwin (1859) in which populations evolve to become distinct species due to natural selection, scientific specialization may differentiate similar domains to such a degree to develop very different subfields and disciplines. With the continuous growth of knowledge, the increasing specialization seems a natural process that is essential for innovation and the search for new discoveries. In this contest, the specialist or "*expert*" is very important because this figure is characterized by a very specific knowledge and has the capabilities to follow the evolution of a certain discipline. One disadvantage of this increasing specialization is the tendency to reduce the common interests of different experts, bringing to a limited interaction up to the isolation with the potential loss of opportunities for new and high-impact discoveries.

In 1990 Prof. Sholz, discussing about earthquakes, wrote: " *It is a consequence of the way in which science is organized that the scientist is trained by discipline, not by topic, and so interdisciplinary subjects such as this one tend to be attacked in a piecemeal fashion from the vantage of the different specialties that find application in studying it. This is disadvantageous because progress is hindered by lack of communication* *between the different disciplines, misunderstandings can abound, and different, sometimes conflicting, schools of thought can flourish in the relative isolation of separate fields*."

The role of figures or processes that enhance the connections between different disciplines, improving the synergies between experts in different fields seems therefore essential. The term "*expert-generalist*", which has been borrowed from the field of economy, it is used in this article to describe figures, processes or learning approaches that promote the study across different fields of research but avoiding the superficial knowledge. This approach allows to study problems from a more general perspective, simplifying connections and relationships between experts of different fields and the buildup of different viewpoints that may evolve to develop new shared approaches or to transfer of models across different disciplines.

Although this general principle may seem attractive, the implementation of this approach in the society can encounter several obstacles. For example, in Italy each academic discipline falls within a specific scientific disciplinary sector (SSD), each one characterized by different standards. This system, which seems to be a byproduct of the overspecialization of research fields, was essentially setup for practical reasons but has the disadvantage to discourage the interaction between experts with different SSD due to a marked separation of interests.

Nowadays, earthquake engineering is a field of research that requires knowledge in different disciplines such as seismology, structural engineering and geotechnical engineering, each one characterized by its "*endemic*" language, typical features and distinctive models. Although certain subfields of earthquake engineering such as soil-structure interaction (e.g. see [2, 3]) or definition of seismic input (e.g. see [4, 5]) may operate as a bridge, driving to lower the barriers and laying the foundations for common models and languages, the distinction between the disciplines still seems very marked.

This article presents a research case for the "*expert-generalist*" that was the result of the learning process established at the *UME Graduate School* (Understanding and Managing Extremes) that has been set up by *IUSS Pavia* (University School for Advanced Studies) and the synergy between the *University of Pavia* and *EUCENTRE* (European Centre for Training and Research in Earthquake Engineering). The interdisciplinary environment and the connections between experts of different fields had set up the basis for transferring one of the most prominent models of geotechnical engineering to structural engineering. In particular, the model of dilatancy developed within the theory of critical state soil mechanics, has been used as a conceptual model to interpret the experimental results of masonry specimens suggesting the modification of the laws for the definition of the shear strength of masonry. This paper summarizes the research carried out so far, trying to extend the theory from the small scale of the specimens to that of the walls.

2. History of dilatancy, from geotechnical to structural engineering

The mechanism of dilatancy is the volume change observed in granular materials when subjected to shear displacement. The term "*dilatancy*" was originally introduced in 1885 by Osborne Reynolds (physicist, engineer, scientist, pioneer) to denote a particular type of behaviour exhibited by a collection of particles in contact. Reynolds conducted original researches which led to the publication of many papers of outstanding interest and fundamental importance, covering a wide range of physical and engineering problems. He laid the foundations for subsequent work on the theories of turbulence, convective heat transfer, lubrication and hydraulic scale models. His classical experiments in the fields of fluid mechanics and heat transfer are acknowledged by the widespread use of expressions which bear his name (e.g. Reynolds number, Reynolds equation, etc). In his later years, while studying the sub-mechanics of the universe, Reynolds tried to replace the ether theory with the idea that the universe is filled with tiny granular particles; hence the interest in sand and dilatancy. Reynolds was concerned mainly with dilatancy as a phenomenon, whereas friction and strength did not attract his direct attention. In contrast to the results in the field of fluid mechanics, his fundamental and original experiments on the dilatancy of granular media did not form the basis of any subsequent work in soil mechanics.

A further advancement on dilatancy, where this phenomenon was put in relation to friction and strength, can be found in the textbook Fundamentals of Soil Mechanics (1948) by Taylor [6] that influenced a generation of soil engineers. Donald Wood Taylor was an early contributor to the emerging field of soil mechanics, long before the field evolved to its current name "*geotechnical engineering*" [7]. Taylor used the term

"*interlocking*" to describe the effect of dilatancy. Curiously, it is worth noting that a similar term "*aggregate interlock*" is used today in the field of structural engineering to identify the same mechanism in the definition of the shear strength of reinforced concrete [8, 9]. Taylor was instrumental in recognizing the strength contributed by the interlocking of soil particles as distinct from friction between soil particles or true cohesion among particles. This aspect was absent in Terzaghi's Theoretical Soil Mechanics (1943) where the shear strength formulation relied entirely on the Mohr-Coulomb model. Taylor's concept was not inconsistent with Mohr-Coulomb theory but observed that the peak envelope was higher than the residual envelope because the "*angle of internal friction, in spite of its name, does not depend solely on internal friction since a portion of the shearing stress on the plane of failure is utilized in overcoming interlocking*".
Taylor's work anticipated the later developments of the Critical State Soil Mechanics (1968) by Schofield and Wroth [10] that is an effective stress framework describing mechanical soil response, currently used in different subfields of geotechnical engineering (e.g. interpretation of experimental tests, constitutive modelling, liquefaction).
This brief history of dilatancy is instrumental to introduce the Terzaghi's Mohr-Coulomb error, which Schofield discussed in a paper entitled "*Mohr-Coulomb error correction*" [11]. As showed in Fig. 1, without enters too much in the technicisms, the Terzaghi's error consists to interpret that the peak strength found in the shear tests of newly remoulded dense soils is due to friction and "*true*" cohesion. However, in the newly worked soil, cohesive bonds are negligible therefore there is no "*true*" cohesion. The peak strength is due to the interlocking (i.e. dilatancy) and friction among the soil particles.

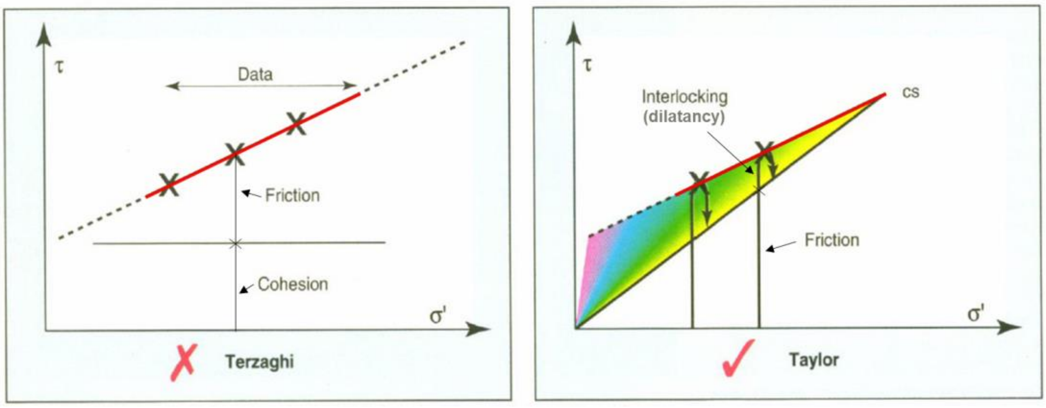


Fig. 1 – Peak strengths in newly remoulded dense soils are caused by particle interlocking, not chemistry of bonds (i.e. "*true*" cohesion), after [11].

As will be discussed in the following sections of the article, the Terzaghi's error affects not only geotechnical engineering but also structural engineering. When the mechanism of dilatancy is neglected during the interpretation of direct shear test of masonry specimens, the Terzaghi's error may overestimate the initial shear strength under zero compressive stress (f_{vko}).
Within the field of geotechnical engineering, the understanding of the mechanism of dilatancy and the Terzaghi's Mohr-Coulomb error initially had difficulty to permeate into practice, partly because of the theoretical structure of the historic argument. Since, the gap between research and practice revealed many significant consequences in geotechnical engineering, this article aims to highlight the role of the "*expert-generalist*" in transferring the knowledge already developed within the field of geotechnical engineering to structural engineering.
The article first summarizes the results of an ongoing research conducted in synergy between IUSS Pavia, University of Pavia and EUCENTRE. Then, it proposes a modification of the formula for the definition of the shear strength of masonry walls defined by Eurocode 6 in order to include the influence of dilatancy. Finally, the failure domains obtained using the Eurocode formulation and the new model are compared.

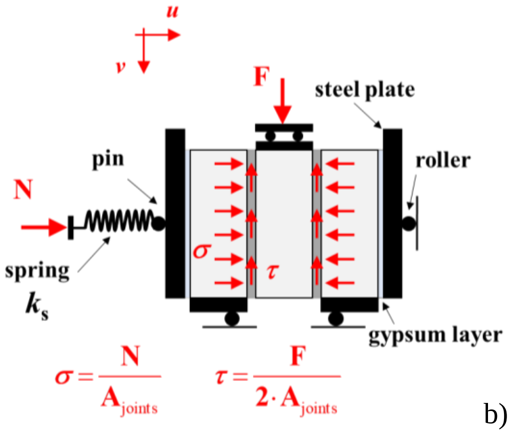
3. Experimental and numerical results

The role and the implications of dilatancy in the definition of the shear strength of masonry samples have been studied with experimental and numerical investigations that are briefly presented in this article. More detailed information can be found in [12, 13].

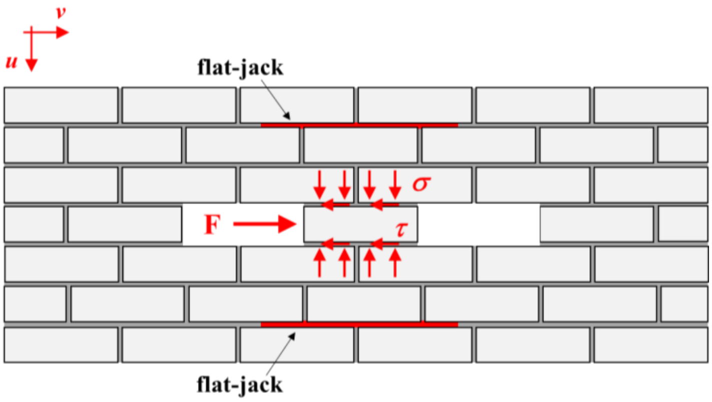
All specimens are characterized by calcium silicate (CS) bricks (212 x 102 x 72 mm) and cement mortar joints with thickness equal to 10 mm. The density of masonry and bricks are 1835 kg/m3 and 1900 kg/m3, respectively. The experimental program includes also characterization tests on units, mortar and masonry assemblies. Tests have been carried out at the DICAr Laboratory of University of Pavia. The compressive strength of mortar is 7.24 MPa and the flexural strength is 2.87 MPa. The compressive strength of bricks is 18.67 MPa and that of the masonry is 6.20 MPa. The Young's modulus of masonry in compression is 4182 MPa and the flexural bond strength is 0.24 MPa.

The experimental campaign consists of several triplet tests (EN 1052-3) and laboratory shove tests (ASTM C1531). The shear force is applied with a static hydraulic jack according to the prescriptions. The oil pressure is slowly increased. If the sampling frequency is adequate (e.g. 60 Hz), the peak and the immediate post-peak phase can be recorded with reasonable accuracy. The relative displacements have been recorded Linear Variable Differential Transformers (LVDTs). The setup of the tests are showed in Fig. 2; v is the average shear displacement and u is the displacements perpendicular to the shear force (i.e. measure of expansion and contraction).

The boundary conditions of the triplet test are reported in Fig. 2a. The steel plates rest on rollers that allow rotations. A soft spring with stiffness k_s=1070 N/mm is interposed between the specimen and the horizontal jack that applies the compression in bed joints (σ) to allow the expansion (dilatancy) of the joints with minimal variation of normal stress, which should be kept constant during the execution of the test. The configuration of the in-situ test is reported in Fig. 2b. With this test, the variation of the compression (σ) is controlled by flat-jacks.



a)



b)

Fig. 2 – Configuration of direct shear tests on masonry samples a) triplet test and in-situ (shove) test.

Shear tests were executed on several specimens under different levels of compression (σ) that were defined before the application of the shear force. The shear test on one specimen started with the application of the shear force on the intact specimen (i.e. with uncracked joints). This first phase of the test ended with the unloading after the cracking of the joints. Then, the shear test has been repeated on the same specimen with cracked joints for the definition of the residual shear strength. The multi-step approach was instrumental for the characterization of different resisting mechanisms that in the peak shear strength are simultaneously active. The numerical research has been conducted in order to verify the consistency of the proposed analytical formulation [12].

The tendency of mortar joints to dilate during direct shear tests (i.e. dilatancy) has already been described for masonry by various researchers [12, 13, 14, 15, 16, 17]. During the shear failure, when the cracking surface is not perfectly flat (e.g. cracks trough the mortar joints), the shear displacements tend to increase the volume of the sample. When normal compression is present, the mechanism of dilatancy increases the shear resistance because the work generated by the expansion opposes the work done by the compression force. Dilatancy can be measured experimentally with the following equation:

$$\tan(\psi) = -\frac{du}{dv} \tag{1}$$

where ψ is the dilatancy angle, dv and du are respectively, the plastic displacement in the shear direction and in the direction perpendicular to shear displacement, expressed in incremental terms (see also Fig. 3a).

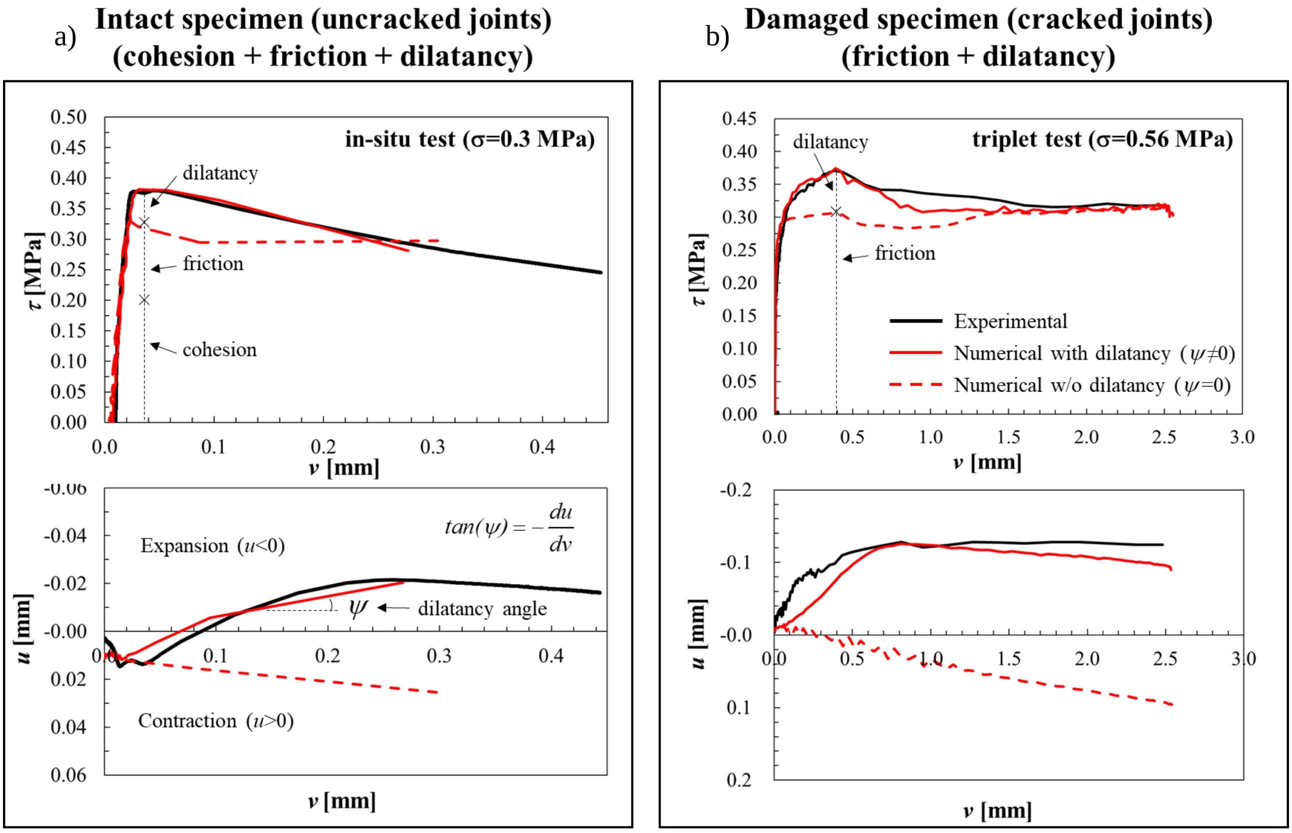


Fig. 3 – Experimental and numerical results of direct shear tests carried out on masonry specimens in different mechanical conditions: a) with intact joints before the test ($f_{vko}\neq0$) and b) with cracked joints already before the execution of the test ($f_{vko}=0$).

As showed in Fig. 3a, three mechanisms contribute to the definition of the peak shear strength of intact specimens: cohesion, friction and dilatancy. Differently from soils, when the joints of masonry specimens are uncracked, the "*true*" cohesion (f_{vko}) is not negligible. When joints are cracked (Fig. 3b) $f_{vko}=0$ therefore, the active mechanisms are friction and dilatancy. It is worth noting that the expansion of masonry specimens tends to vanish at large shear displacements (v) and/or with large value of compression (σ). This phase is called "*constant volume*" because it is characterized by a dilatancy angle (ψ) equal to zero. Friction, which is the only active resisting mechanism in this phase, is named "*friction at constant volume*" By repeating the direct shear test on the same masonry specimen with cracked mortar joints is possible to isolate the three resisting mechanisms.
Fig. 3 shows that a peak phase with softening is typical not only of the direct shear test executed on the masonry specimen with initially undamaged joints (i.e. initial cohesion before the test $f_{vko}\neq0$) but it is also evident in the specimen with already cracked joints (i.e. $f_{vko}=0$ before the test). The softening of the intact specimen is governed by the evolution of cohesion and dilatancy angle whereas, in the damaged specimen the softening is due exclusively to dilatancy.
To better clarify the role of dilatancy angle and the contribution of this parameter in the definition of the shear strength, several numerical analyses have been repeated by using the same model with different value

of dilatancy angle: (*i*) characterized experimentally (i.e. dilatancy active because $\psi \neq 0$) and (*ii*) with $\psi = 0$ (i.e. dilatancy not active). The numerical results show that the amount of shear strength due to the dilatancy angle is not negligible (Fig. 3). It is worth noting that when dilatancy angle approximates zero (see Fig. 3b), the effect of dilatancy vanishes and the shear strength is only controlled by friction angle at constant volume (ϕ_{cv}).

4. Friction model for mortar joints including the mechanism of dilatancy

A simple friction model for mortar joints that include the mechanism of dilatancy has been developed based on experimental results [12, 13]. This formulation extends the friction model currently used in masonry standards that neglect dilatancy (e.g. Eurocode 6 and ASTM C1531). The proposed model has been implemented in Abaqus and it has been used to perform the numerical analysis previously introduced [12]. The friction model used for masonry is based on the Coulomb's law, which is characterized by two parameters: the initial shear strength in absence of compression (i.e. f_{vko} or cohesion) and friction (μ). It has been proved that also dilatancy plays a role in the definition of the peak shear strength [12, 13]. The proposed model introduces the dilatancy angle (ψ) in the standard formulation in order to consider the effects of dilatancy.

The initial shear strength in absence of compression (f_{vko}) characterizes the complex bonding in the mortar and between the unit-mortar interfaces. According to the standard friction model based on the Coulomb's law, the contribution of f_{vko} to the overall shear strength is valid up to the complete crack formation. Beyond this point f_{vko}=0. The softening of this parameter starts at the onset of cracking when friction and dilatancy begin to mobilize [13]. The experimental data indicate that the peak shear strength of masonry samples strongly depends on σ (see for example Fig. 5a). Since, by definition, f_{vko} is independent of the compression (σ), according to the friction model, this experimental evidence can be explained with the mobilization of the friction during the crack propagation. As showed in Fig. 4, dilatancy is strictly connected to friction and the shear strength is also influenced by dilatancy angle [12, 13].

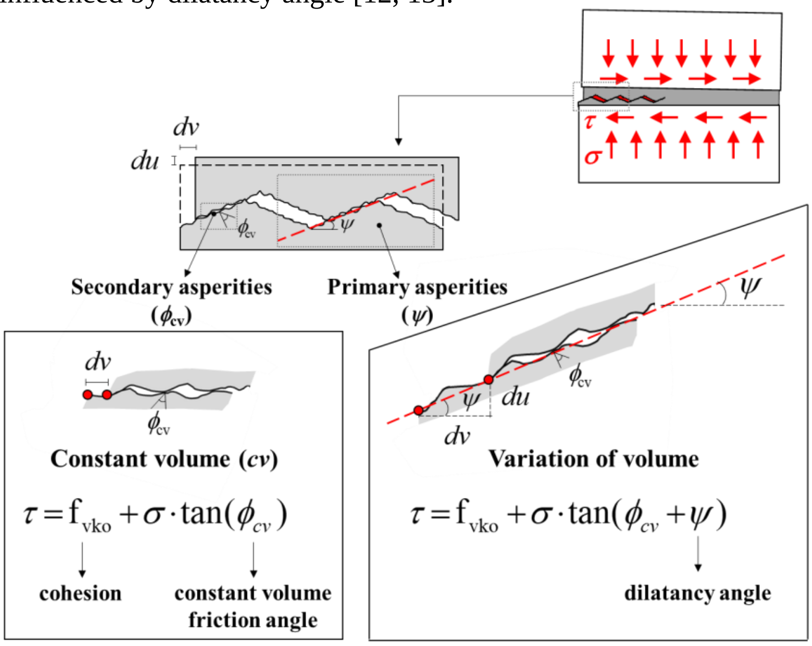


Fig. 4 – Friction model with dilatancy that is proposed for the shear failure of mortar joints.

In the proposed model (see Fig. 4), the cracking surface forming in a mortar joint is defined by a composition of asperities with different size [12, 13]. Primary asperities are the largest ones and define the dilatancy angle (ψ) while secondary asperities, which act at a smaller scale, characterize the friction angle at constant volume (ϕ_{cv}). When the cracking surface is relatively flat (e.g. debonding at the unit-mortar interface), the shear displacement is not accompanied by significant expansion (ψ=0). On the other hand, when the

cracking surface is not flat (e.g. crack passing through the mortar joint), the shear sliding generates expansion. At the microscopic level, the dilatancy angle ($\psi \neq 0$) provides a quantitative indication on the complex geometry of the primary asperities in the cracking surface (Fig. 4).
The friction angle (ϕ) of mortar joints defining the friction coefficient $\mu=\tan(\phi)$, is defined by two components (see Fig. 4):

$$\phi = \phi_{cv} + \psi \tag{2}$$

where ϕ_{cv} is the friction angle at constant volume, depending exclusively on the material and without generating volume changes of the sample. ψ is the dilatancy angle that can be computed with Eq. (1). It governs the expansion of the mortar joints because it is a measure of the primary asperities in the cracking surface of mortar joints (see Fig. 4). ψ varies depending on the level of compression and shear displacement whereas ϕ_{cv} is relatively constant because it depends exclusively on the roughness of the sliding surface. The introduction of Eq. (2) in Coulomb's law couples the shear strength and the dilatancy angle in the same equation:

$$\tau = f_{vko} + \sigma \cdot tan(\phi_{cv} + \psi) \tag{3}$$

The Coulomb's law is used for the characterization of the shear strength of mortar joints (EN 1052-3) and for the shear strength of unreinforced masonry (Eurocode 6). The characteristic shear strength of unreinforced masonry wall (f_{vk}) is defined by Eurocode 6 with the following relation:

$$f_{vk} = f_{vko} + \sigma \cdot 0.4 \tag{4}$$

where f_{vko} is the parameter already discussed that can be defined from experimental data (e.g. EN 1052-3) or by using the suggested values of Eurocode 6 whereas, 0.4 is the fixed value assumed for the friction coefficient (μ). Since Eq. (2) can be rewritten in terms of friction coefficient as follows:

$$\mu = tan(\phi_{cv} + \psi) = \frac{\mu_{cv} + \mu_{\psi}}{1 - \mu_{cv} \cdot \mu_{\psi}} \tag{5}$$

where $\mu_{cv}=tan(\phi_{cv})$ is the friction coefficient at constant volume and $\mu_{\psi}=tan(\psi)$ is the variable amount of friction coefficient due to the dilatancy, the substitution of friction coefficient in Eq. (4) with Eq. (5) led to the following relation:

$$f_{vk} = f_{vko} + \sigma \cdot \left(\frac{\mu_{cv} + \mu_{\psi}}{1 - \mu_{cv} \cdot \mu_{\psi}} \right) \tag{6}$$

Eq. (6) modifies the relations for the definition of the characteristic shear strength of mortar joints (EN 1052-3) and unreinforced masonry wall provided by Eurocode 6, allowing to account for the mechanism of dilatancy. If the friction coefficient at constant volume is set equal to 0.4 and dilatancy angle is equal to zero, $\mu_{\psi}=0$ and Eq. (6) becomes equal to Eq. (4).

5. Experimental characterization of mechanical parameters

Fig. 5a shows the experimental characterization of f_{vko} and friction coefficient (μ) with the triplet test on intact masonry specimens, according to EN 1052-3 as recommended by Eurocode 6. As already discussed, this procedure does not explicitly consider dilatancy. The European standard EN 1052-3 gives the possibility to interpret data from triplet tests by using at least nine different samples and then fitting the values of peak shear stress (τ_{peak} in Fig. 6) in the τ-σ plane with Coulomb's law.
For each datapoint present in Fig. 5a, Fig. 5b reports the experimental measure of the dilatancy angle. The dilatancy angle has a high value at low compression approaching zero and then becoming negative at high values of σ. A relation between $\tan(\psi)$ and σ has been found by fitting the datapoint using a natural logarithmic function:

$$\tan(\psi) = \mu_{\psi} = a \cdot \ln(\sigma) + d \tag{7}$$

where the a=-0.12 and d=-0.017 are parameters found in the regression that depend on the type of masonry. Since the dilatancy angle increases the shear resistance [12, 13], at low compression the shear strength is significantly influenced by the dilatancy angle. Fig. 5c shows that the dilatancy angle decreases with the increase of σ and shear displacements (v), becoming negative for large values of σ and v. ψ<0 can be physically interpreted by the damaging (i.e. reduction) of the primary asperities in the cracking surface.

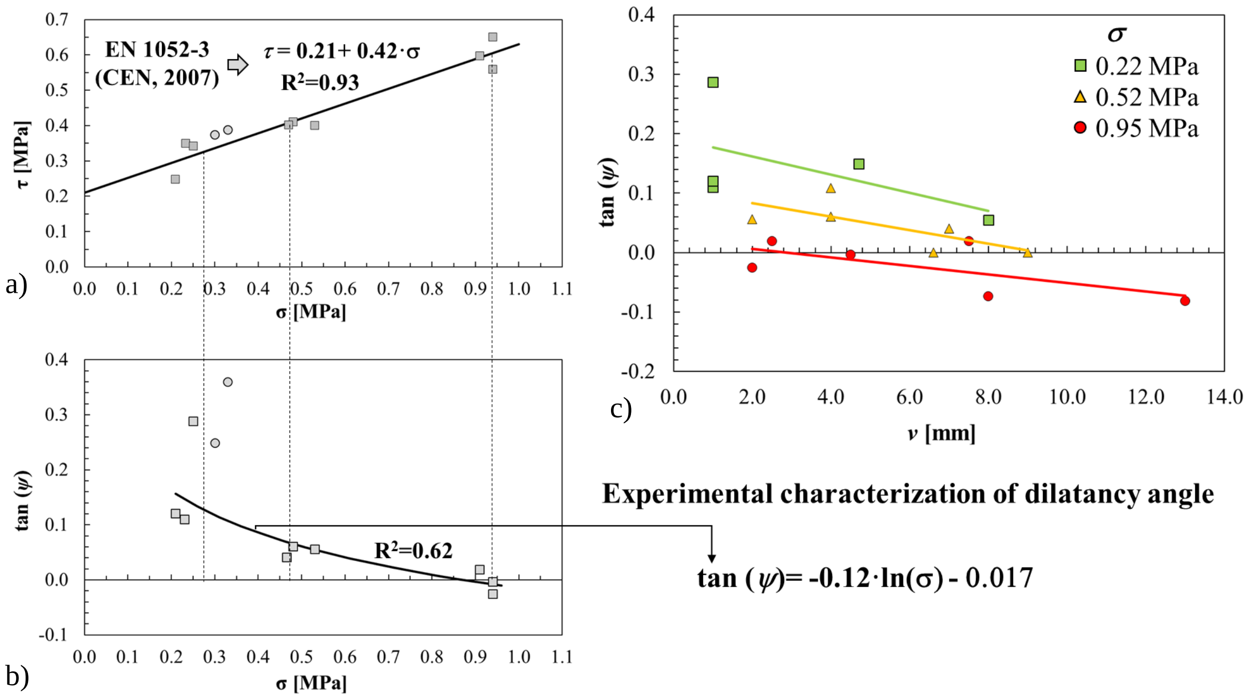


Fig. 5 – (a) Experimental characterization of parameters from triplet test, according to European standards EN 1052-3. Round points refer to in-situ tests. (b) Characterization of dilatancy angle from experimental data. (c) Variation of dilatancy angle with shear displacement (v) and compression (σ).

Eq. (7) and Fig. 5b show that the influence of the dilatancy angle on the peak shear strength depends on the level of compression. When volumetric expansion of the specimens has been recorded during the direct shear tests, the definition of the shear strength by using the peak values is affected by dilatancy. Since the peak values at low normal stress are generally higher than those at high values of σ due to the dilatancy angle ψ>0, the definition of f_{vko} and friction by fitting peak values as suggested by EN 1052-3 may be biased.
For the same reasons previously discussed about the Terzaghi's error, when dilatancy is significative, there is the tendency to overestimate f_{vko} and to underestimate the friction coefficient (see Fig. 7). To overcome this problem, a new procedure that considers the effects of dilatancy has been formulated [13]. The new methodology is summarized in Fig. 6.
In the first step of the procedure (see in Fig. 6a) the peak shear strength (τ_{peak}) and the constant volume shear strength (τ_{cv}) values are defined for each level of compression (σ). As showed in Fig. 6a, τ_{cv} values can be identified by a dilatancy angle approximately equal to zero. Then, instead of finding parameters by fitting directly the peak values as prescribed by EN 1052-3, the friction angle at constant volume (ϕ_{cv}) is defined by fitting the constant volume values (τ_{cv}) with a linear regression (Fig. 6b). Since only the friction mechanism is active at the constant volume phase, the value of friction can be defined with improved accuracy. In the next step (Fig. 6c), f_{vko} is found by fitting the peak values. In the last step (Fig. 6e), a nonlinear Coulomb's law is defined by including the dilatancy angle (ψ) in Eq. (3). The analytical expression of ψ is given by Eq. (7). It is worth noting that when the dilatancy angle is approximately zero, the peak strength is equal to the strength at constant volume and the proposed procedure converges to the standard method of EN 1052-3.
Finally, Fig. 7 shows the comparison between the characterization of parameters from the results of triplet

test found with the proposed methodology and according to the EN 1052-3. The results of the shove tests (i.e. square points in Fig. 7) have been reported in the same plot for comparison. The initial shear strength defined with the EN 1052-3 is higher (43%) than the value found with the proposed approach and friction is lower (38%) than the constant volume friction coefficient (μ_{cv}). The overestimation of f_{vko} and the underestimation of friction coefficient happen because the dilatancy is not explicitly considered in the EN 1052-3, but it is embedded in the other parameters. However, the overestimation of of f_{vko} and the underestimation of friction coefficient due to the inclusion of dilatancy is not consistent with the physical interpretation because, when σ=0 the dilatancy angle has no effects on the shear strength. Moreover, a peak friction coefficient (μ=0.42) lower than the residual friction (i.e. friction coefficient at constant volume μ_{cv}=0.58) has no physical meaning. It should be noted that the overestimation of f_{vko} in the direct shear tests of masonry is generated by the misinterpretation of the phenomenon of dilatancy as occurred for the Terzaghi's error in geotechnical engineering in the study of newly remoulded dense soils, previously discussed.

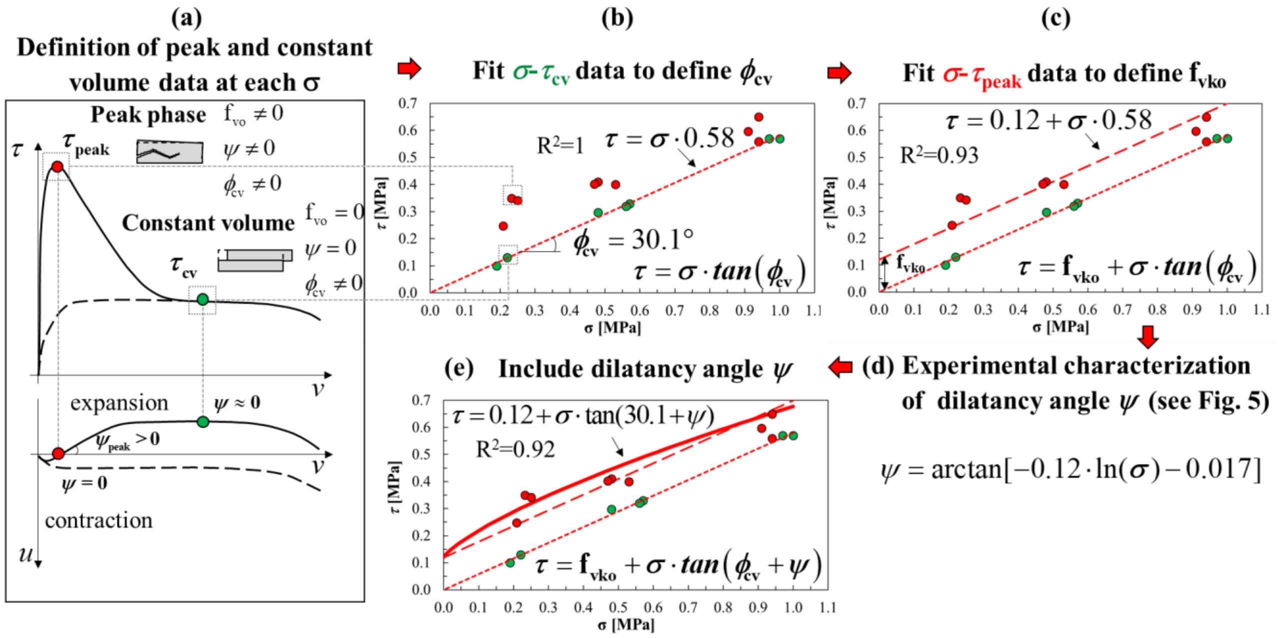


Fig. 6 – Proposed procedure for the experimental characterization of mechanical parameters considering the effects of dilatancy.

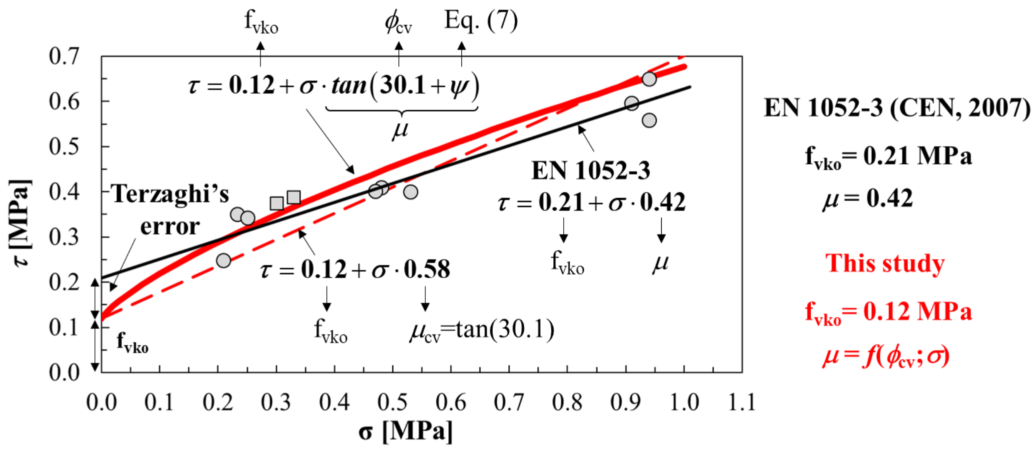


Fig. 7 – Comparison between the characterization of parameters carried out with the proposed methodology and according to the EN 1052-3 approach.

6. From small to large scale: from mortar joints to failure domains of masonry walls

Up to now, the proposed formulation mainly refers to mortar joints of small masonry specimens. To a larger scale, the Eurocode 6 defines the shear strength of unreinforced masonry walls with Eq. (4). Since both the approaches are based on the Coulomb's law, the proposed model for the shear failure of mortar joints has been extended to masonry walls with Eq. (6).

In a broader perspective, the principal failure mechanisms of masonry walls subjected to seismic actions are [18]: (*i*) rocking failure, (*ii*) shear-cracking and (*iii*) shear-sliding. The ultimate shear capacity of a unreinforced masonry pier (V_u) due to the shear-sliding failure mechanism can be defined as follows [18]:

$$\frac{V_u}{b \cdot t} = \frac{1.5 \cdot f_{vko} + \sigma \cdot \mu}{1 + 3\frac{f_{vko} \cdot \alpha_V}{\sigma}} \tag{8}$$

where b and t are the length and the thickness of the pier, respectively; f_{vko} and μ are the parameters previously discussed (i.e. cohesion and friction). α_v is the shear ratio that depends on the boundary conditions of the wall. Considering typical test layouts, it is equal to the ratio h/b, where h is the height of the pier, when the pier is fixed on one end and free to rotate on the other (i.e. cantilever), and it is equal to $0.5h/b$ when the pier is fixed at both ends. As already discussed, Eurocode 6 a constant friction coefficient μ =0.4, neglecting the mechanism of dilatancy. Fig. 8 show the failure domains for a masonry pier with h/l=0.6, h/t=15 and the mechanical parameters previously introduced describing the characterization tests on calcium silicate bricks (i.e. compressive strength of bricks f_b=19 MPa; compressive strength of masonry: f_m=6 MPa; conventional tensile strength of masonry f_t=0.45 MPa).

Two shear-sliding failure domains have been defined according to Eurocode 6, using Eq. (8): (*i*) by setting 0.8 time the characteristic value of f_{vko} defined according to EN 1052-3 (f_{vko}=0.8·0.21 MPa), which is the black line in Fig. 8 and (*ii*) by using the value of f_{vko} found with the proposed procedure (f_{vko}=0.8·0.12 MPa), which is the red line in Fig. 8. Consistently to the prescriptions of Eurocode 6, the friction coefficient (μ) has been set equal to 0.4 in both domains. The failure domains computed with f_{vko} defined with the proposed methodology are more conservative than EN 1052-3 [19]. The difference is more pronounced when the pier is fixed at both ends (Fig. 8 b). The mobilization of the shear-sliding failure mechanism to higher values of compression is also observed.

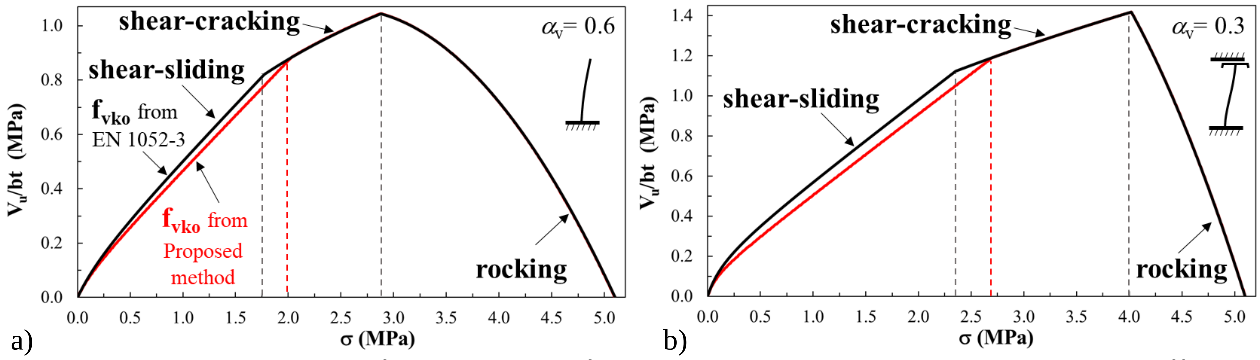


Fig. 8 – Comparison between failure domains of a masonry pier according to Eurocode 6, with different boundary conditions: a) cantilever and b) both ends fixed. The shear-sliding domains have been defined with μ =0.4 and f_{vko} from EN 1052-3 (black line) and the proposed method (red line).

A modification of Eq. (8) is now proposed in order to include the phenomenon of dilatancy. Since the proposed formulation for the failure mechanism of mortar joints introduces the dilatancy angle in the friction coefficient (μ), if μ in Eq. (8) is substituted by Eq. (5), the effect of the dilatancy angle (μ_ψ) is introduced in Eq. (8), which modifies as follows:

$$\frac{V_u}{b \cdot t} = \frac{1.5 \cdot f_{vko} + \sigma \cdot \left(\frac{\mu_{cv} + \mu_{\psi}}{1 - \mu_{cv} \cdot \mu_{\psi}} \right)}{1 + 3 \frac{f_{vko} \cdot \alpha_V}{\sigma}} \tag{9}$$

With Eq. (9) it is possible to define the shear-sliding failure mechanism of masonry piers considering the phenomenon of dilatancy. It is worth noting that when dilatancy is not active (μ_{ψ} =0), Eq. (9) becomes equal to the original formulation. Eq. (9) has been defined from a theoretical viewpoint, the consistency with experimental data should be verified by means of experimental investigations.

Differently from the previous case, Fig. 9 compares the failure domains defined according to Eurocode 6 and EN 1052-3, which are already reported in Fig. 8 (i.e. black lines), and the shear-sliding domains defined with the proposed formula (red line): Eq. (9). The new domains have been computed using 0.8 times the characteristic parameters defined experimentally (see Fig. 6): f_{vko}=0.12 MPa, μ_{cv}=0.58 and μ_{ψ} from Eq. (7). Although the proposed failure domains are computed with a lower value of f_{vko}, at low compressions the difference with Eurocode 6 is negligible because the friction coefficient at constant volume is higher and the dilatancy angle is positive. For higher compressions, the difference between the proposed method and Eurocode 6 is significantly because the dilatancy angle reduces (i.e. assuming values lower than zero due to damaging of asperities). A significant mobilization of the shear-sliding failure mechanism to higher values of compression is also observed. The newly proposed shear-sliding failure domains are generally more conservative for σ>1 MPa whereas, at lower compression levels, the dilatancy angle tends to increase the strength.

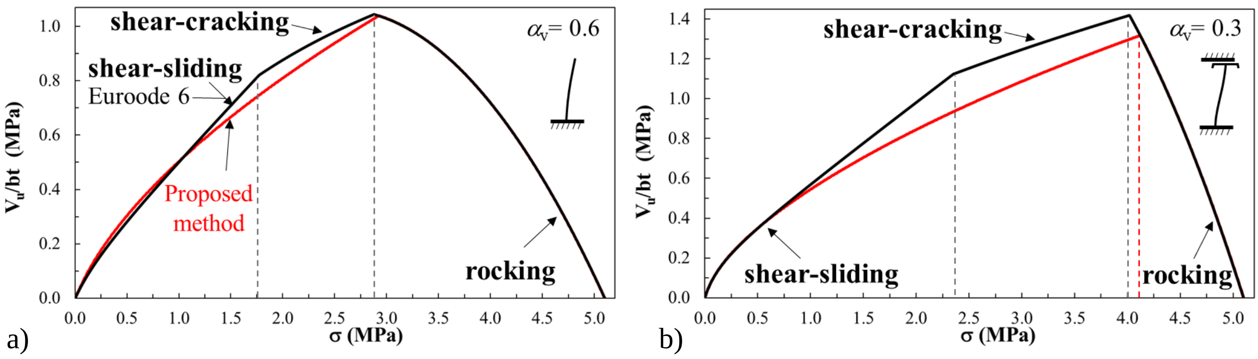


Fig. 9 – Comparison between failure domains of a masonry pier according to Eq. (8) of Eurocode 6 (black line) and with Eq. (9) of the proposed procedure (red line). a) cantilever and b) both ends fixed.

6. Conclusions

The increasing specialization seems a natural process that is essential for innovation and the search for new discoveries. One disadvantage of this process is that it tends to reduce the common interests among experts of different disciplines. The term "*expert-generalist*", which has been borrowed from the field of economy, it is used in this article to describe figures, processes or learning approaches that promote the study across different fields of research aiming to develop new shared approaches or transfer models across different disciplinesThis article presents a research case for the "expert-generalist" that was the result of the learning process established at the UME (Understanding and Managing Extremes) Graduate School that has been set up by IUSS Pavia (University School for Advanced Studies) and the synergy between the University of Pavia and EUCENTRE (European Centre for Training and Research in Earthquake Engineering). The interdisciplinary environment and the connections between different experts had set up the basis for the transferring one of the most prominent models of geotechnical engineering to structural engineering. In particular, the model of dilatancy developed within the theory of critical state soil mechanics, has been successfully applied to structural engineering for the definition of the shear strength of masonry. With the support of experimental data, the proposed approach shows that current standards (e.g. Eurocode 6 and EN

1052-3) have the tendency to overestimate the initial shear strength of mortar joints (f_{vko}) and underestimate the friction coefficient. Finally, the new formulation has been applied to a larger scale for the modification of the equations commonly used for the definition shear-sliding failure domain of masonry walls. The comparison with Eurocode 6 shows that the proposed approach is generally more conservative for walls with compressions higher than 1 MPa whereas, at lower compression levels the dilatancy angle tends to generate higher strengths.

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