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The influence of the two-component grout on the behaviour of a segmental lining in tunnelling

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Abstract

Filling material is present around the segment lining when a shielded Tunnel Boring Machine is used to excavate a tunnel. The two-component grout is becoming lately one of the most used filling materials. Its mechanical properties evolve over time. Unfortunately, there are not many studies in the literature on the specific mechanical characteristics of these materials. This work presents the results obtained from an extensive laboratory test campaign that allowed to fully characterize the two-component filling material during the setting period. In particular, the values of the stiffness and resistance parameters were obtained over time, where uniaxial compression tests and oedometer tests were carried out. A detailed study of the effect of the presence of the filling material on the behavior of the support system (segmental lining + filling material) was developed for two of the most widespread analytical methods for the analysis of the behavior of tunnels and structures of

24 support: the convergence-confinement method and the Einstein and Schwartz method.
25 Subsequent parametric analyses made it possible to consider the variability of the
26 influencing parameters within the typical variability ranges obtained from the laboratory
27 test campaign or known from the available scientific literature. From the study carried out,
28 it was possible to note that it is necessary to consider the presence of the filling material in
29 the evaluation of the stiffness of the support system, when using the convergence-
30 confinement method to estimate the loads acting on segmental lining. In this regard, it is
31 necessary to have a reliable estimate of the elastic modulus of the filling material in the
32 period of loading of the segmental lining. On the other hand, the presence of the ring of
33 filling material is negligible when evaluating the state of stress of the segmental lining with
34 specific methods capable of considering the rock-support interaction. In particular,
35 adopting the Einstein and Schwartz method, it is possible to define the bending moments
36 and normal forces acting in the support structure, referring to the stiffness parameters of
37 the segmental lining alone.

38 **Key words:** two-component grout; curing time; oedometer; unconfined compressive
39 strength; TBM; convergence-confinement method; Einstein and Schwartz method.

41 **Abbreviations and nomenclature**

42 A_s Area of the cross section of the support, through a plane passing through the axis of
43 the tunnel;

44 b_s Width of the support section in the direction of the tunnel axis, considered equal to 1
45 m;

46 c Cohesion of the rock mass;

47 C^* Compressibility ratio of the support;

48 E Elastic modulus of the rock mass

49 E_{fm} Elastic modulus of the filling material;

50 $E_{s,eq}$ Equivalent elastic modulus of the support;

51 E_{sl} Elastic modulus of the segmental lining concrete;

52 F^* Flexibility ratio of the support;

53 I_s Moment of inertia of the cross section of the support, through a plane passing
54 through the axis of the tunnel;

55 K_0 Lateral earth coefficient at rest;

56 k_{sys} Reaction line of the support the stiffness;

57 M_{max} Maximum bending moment present in the lining;

58 N Normal force present in the lining;

59 p Pressure inside the tunnel acting on the walls;

60 p_{eq} Final entity of the loads acting on the support structure;

61	p_0	Hydrostatic initial stress state (undisturbed);
62	R	Tunnel radius;
63	R_{pl}	Plastic radius of the tunnel;
64	$t_{s,eq}$	Thickness of equivalent support;
65	t_{fm}	Thickness of the filling material;
66	t_{sl}	Thickness of the segmental lining;
67	u_{eq}	Final displacement of the tunnel wall in the radial direction;
68	u_{max}	Maximum displacement of the tunnel wall in the absence of supports;
69	u_0	Displacement of the tunnel wall when the support structure is installed;
70	u_R	Radial displacement of the tunnel wall;
71	ν	Poisson ratio of the rock mass;
72	ν_{fm}	Poisson's ratio of the filling material;
73	ν_{sl}	Poisson's ratio of the concrete constituting the segmental lining;
74	α_f	Angle of the failure plane;
75	φ	Friction angle of the rock mass;
76	Ψ	Dilatancy of the rock mass;
77	σ_r	Radial stress in the point where the stress state is evaluated during the design of
78		the support structure;
79	σ_{vert} or σ_v	Vertical load during the oedometer tests;
80	$\sigma_{\vartheta,sl,in}$	Circumferential stress at the intrados of the segmental lining due to moment;

81 $\sigma_{\vartheta, fm, ex}$ Maximum circumferential load on the extrados of the filling material due to
82 the moment;

83 $\sigma_{\vartheta, sl}$ Constant load in the segmental lining section due to the nominal force;

84 $\sigma_{\vartheta, fm}$ Constant load in the section of the filling material due to the nominal force;

85 ξ Incremental coefficient taking into into account the transfer of stresses from one ring
86 to the adjacent one, in correspondence with the longitudinal joints of the segmental lining;

87 η Reduction coefficient taking into account the presence of longitudinal joints in
88 segmental lining;

89 ϕ' Slope the Mohr Coulomb envelope.

90

91

1. Introduction

During the excavation with shielded tunnel boring machines (S-TBM) pre-cast segments are commonly installed as tunnel lining and for support purpose. Due to overcutting of the circular tunnel profile due to the TBM cutterhead excavation, a circular gap between the lining and the surrounding ground is formed (thickness about 13 to 20 cm, e.g. Talmon and Bezuijen, 2005; Beghoul and Demagh, 2019). This space must be filled by pumping or injecting some materials, such as the so-called “annulus grout” (Fig. 1). The practical aims for the use of the filler are linked to control the ground deformation around the lining gap and the reduction of the ground loss; there is also a role to contribute in a regular and homogeneous distribution of the contact pressure at the interfaces ground/grout and grout/segmental lining.

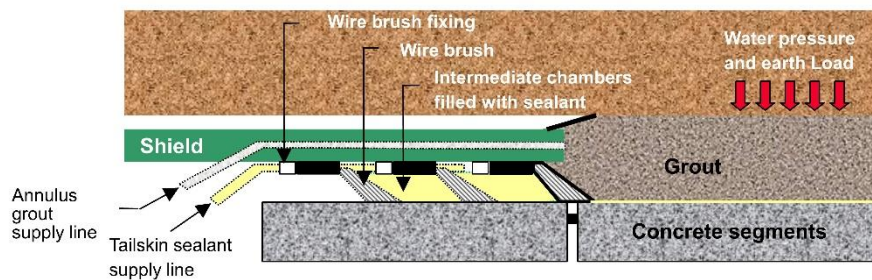


Fig. 1. Sketch of the annulus grout.

There are two basic types of annular grouts currently in use: thick mortar type grouts and highly mobile two-component grouts. The typical mix-design in a m^3 system for a two-component grout is very variable and depends strongly on the project specification but in general it consists of cement (280-450 kg), bentonite (30-60 kg), water (730-860 kg), retarder 3-5 kg and accelerator, normally sodium silicate (60-80 kg).

The accelerator (“B” component) is generally added just before pumping phase of the mix of water, bentonite, retarder and cement (“A” component).

113 The excavation of the tunnel produces a relaxation of the initial tensional state, leading to
114 deformations in the ground from certain distance ahead of the excavation face. The
115 convergence in the tunnel is the inward displacements of the soil/rock as consequence of
116 the relief of the initial stress. In case of no support, the ground is free to deform into the
117 cavity until arriving to the equilibrium state.

118 The role of the two-component grout is very important for the correct mechanized
119 tunneling procedure in order to minimize surface settlements due to any over-excavation
120 generated by the passage of the TBM (e.g. Maidl et al., 1995) or to minimize the loads
121 acting on the segmental lining, although grouting can also be used for the same aims (e.g.
122 Komiya et al., 2001). The two-component grout should be water-tight, be pumpable, be
123 workable, able to fill the void, not able to shrink, to stiff quickly and to be wash-out resistant
124 (e.g. Thewes and Budach, 2009). Its mechanical strength, though, should be only slightly
125 higher than that of the surrounding soil to prevent substantial normal force components
126 being removed from the annular gap mortar in the final state (DAUB, 2013). Simultaneous
127 backfilling with two-component grouts, in comparison with the mortar type grouts, keeps in
128 general lower settlements during TBM excavation (Hirata, 1989) and normally the lining
129 pressures, few rings behind the TBM, do not change significantly in the long-term
130 (Hashimoto et al., 2004).

131 However, not many works dealt with the behavior of the two-component grouts both
132 experimentally and numerically (e.g. Pelizza et al., 2011; Shah et al., 2018; Ochmański et
133 al., 2018; Todaro et al., 2019). For instance, it is well-known that the mechanical properties
134 of the two-component grout change based on the mix-design type (Flores, 2015; Todaro et
135 al., 2019). Besides, tail void grouting, as it cannot be directly observed after the tunnel
136 construction, is difficult to simulate (Dai et al., 2010). Oh and Ziegler (2014), Shah et al.
137 (2018), Ochmański et al. (2018) more recently Ochmański et al. (2020) performed a

138 numerical analysis regarding the effects of the two-component grout on the tunnel
139 settlement. Bezuijen and Talmon (2003) and Dias and Bezuijen (2015) investigated the
140 consolidation of the grout, which seems to be dependent on the permeability of the
141 surrounding soil as the latter determines the rate of outflow. Therefore, a very
142 impermeable soil like a clay could stop the grout from consolidating (Vu et al. 2016),
143 however in permeable soils, fluid loss occurs during grout consolidation (Talmon and
144 Bezuijen, 2005). Furthermore, Bezuijen and Talmon (2003) and Dias and Bezuijen (2015)
145 illustrate that during this consolidation, the grout mixture becomes thicker (the cement-
146 water ratio increases, decreasing the viscosity of the mixture). Talmon and Bezuijen
147 (2005) investigated the stress-strain modelling of the grout.

148 However, the role of the material considering the deformability and resistance values that
149 characterize it during the loading phase of the segmental lining tunnel are not fully
150 investigated. The two-component grout when hardens should transmit the tunnel
151 deformation to the ground. In literature, ground settlement in tunneling caused by ground
152 loss (i.e. the difference between actual and theoretical excavation volume) considers also
153 tail loss which occurs along the annular void between ground and concrete segmental
154 lining (as a result of shrinkage or compression of backfill grout material). The gap model
155 proposed by Lee et al. (1992) is based on simple elastic equations for the squeezing of
156 tunnel face and the contraction of excavated cavity and it has some limitations and
157 uncertainties for practical use (Park et al., 2018). Finally, features of specific products and
158 fulfillment of requirements should be addressed when adopting design criteria and
159 accepting construction procedures: materials, environmental requirements and use of
160 chemicals products should be approved by Client and Boards for a reliable practical quality
161 assurance (Oggeri and Ova, 2004). Furthermore, the use of numerical modeling in
162 tunneling, both with two-dimensional and three-dimensional methods, requires the
163 definition of the characteristics of all the materials that are used as support structures or

164 rock reinforcement interventions (Do et al., 2014a; 2014b; 2015; 2016; Pelizza et al.,
165 2000).

166 In this paper, a mix-design of a two-component grout was analyzed in the detail.
167 Unconfined compressive strength (UCS) and oedometer tests at varying curing ages were
168 investigated. From the analysis of the laboratory results it was possible to detect the
169 behavior of this material and in particular the deformability and strength values that
170 characterize it during the loading phase of the segmental lining tunnel.

171 Two very interesting simplified approaches are available in the literature to study the
172 behavior of the supporting structures in tunnelling: the convergence-confinement method
173 (e.g. Oreste, 2007; Oreste et al., 2016; 2018a; 2018b; Spagnoli et al., 2016; 2017) and the
174 method of Einstein and Schwartz (1979). The convergence-confinement method considers
175 the ground response to the advancing tunnel face and the interaction with installed support
176 (e.g. Oke et al. 2018), whereas the latter method assumes an annular support
177 continuously connected with the wall of a deep and circular tunnel.

178 This article describes the techniques for correctly considering the presence of the filling
179 material in the gap between the tunnel segmental lining and the tunnel wall using the two
180 widespread simplified analysis methods listed above. Some calculation examples will allow
181 to evaluate the effect of the filling material on the static behavior of the tunnel segmental
182 lining, starting from the mechanical parameters evaluated by the extensive laboratory
183 testing campaign. From the developed analyses it will be possible to obtain useful
184 information on how to consider the presence of the filling material ring in the design phase
185 of the segmental lining of a tunnel.

186 **2. The mechanical behavior of the two-component grout in the laboratory**

187 Testing procedures

188 The following mix-design was tested during this experimental campaign: water 800 g,
189 bentonite 35 g, cement (CEM I 52.5) 350 g, retarder 17.5 g (solution contains 20% solid
190 therefore retarder dosage by weight of cement is 1%) and water glass (sodium silicate) 85
191 g (water glass is the “B” component so it is about 7% weight of the mix represented by the
192 component “A”). Based on the grout component quantities, a preliminary laboratory testing
193 set has been carried out. The purpose of such a trial was to identify the basic mechanical
194 properties of the grout at different duration of the curing and to observe the failure modes
195 as well. Following the experimental raw data on compressive strength, deformability and
196 unit weight have been processed in order to be adopted in the mathematical modelling for
197 interaction and also for general description of the mechanical behaviour.

198 Preparation of fluid grout has been carried out with bentonite slurry hydration (duration at
199 least 24 h) and subsequent mixing with retarder, cement and waterglass catalyst, by
200 respecting the mass percentages provided for the standard mix. Both a manual dispersion
201 and a high-speed rotating mixer (up to 8000 rpm) have been adopted during this phase.
202 Weight has been determined by means of 0.01 g precision scale.

203 Sample preparation has been carried out by using specimen casing where the catalyst has
204 been added to fluid grout and fast rotation of mixer has allowed to disperse and
205 homogenize the grout. Then the casing has been recovered in a box for curing in water.
206 Curing procedure has been selected the following timeline for testing: 1 hour; 24 hours; 7
207 days; 28 days.

208 Preparation of specimen requires great care and repeated attempts are necessary to
209 obtain suitable material. This is due to the great sensitivity of final features of the specimen
210 to addition and mixing of water glass. This step has been made with the mentioned high
211 speed mixer in order to be sure that dispersion of water glass is homogeneous and quick
212 inside the grout when still fluid.

213 Grout (without component “B”) viscosity testing with Marsh cone carried out by using a
214 funnel as described in the API Recommended Practice 13B-1 (2014) produced a result
215 between 30 and 45 seconds. Bleeding of the grout (without component “B”) according to
216 the DIN EN 480-4 (2006) was less than 3% at 3 hours. Final mixing of components “A”
217 and “B” produced a total gel time of less than 9 seconds.

218 Uniaxial compression testing has been carried out in a Belladonna mechanical press for
219 soils, equipped with bidirectional displacement rate control device (Fig. 2). Transducers
220 used to measure load and displacements have been respectively a full bridge load cell
221 (CCT model, full scale 5 kN and precision of 1 N) and LVDT devices (HBM models,
222 precision 0.001 mm). Vertical displacement has been measured following the relative
223 movement of the base of the specimen and radial displacements have been measured by
224 using two transducers mounted on opposite and diametral alignment across the specimen.
225 An alternative solution for displacement measurement has not been possible. As the
226 external surface of specimen was not showing adhesive properties and due to the short
227 timeline available between preparation and testing for the majority of specimen, strain
228 gages have not been glued onto the specimen. Radial LVDT transducers have been
229 mounted to be in contact within the medium third of the specimen height.

230 Advancing rate has been adapted in the range of 0.15÷0.45 mm/min and the suitable
231 results have been obtained for the range 0.30÷0.45 mm/min; this selection is a good
232 compromise to avoid creep behavior (excess of lateral swelling) or sudden failure (vertical
233 cracks). Specimen diameter has been selected at 48 mm; for one additional sample the
234 diameter was 52 mm and for two large diameter specimens the value was 75 mm.

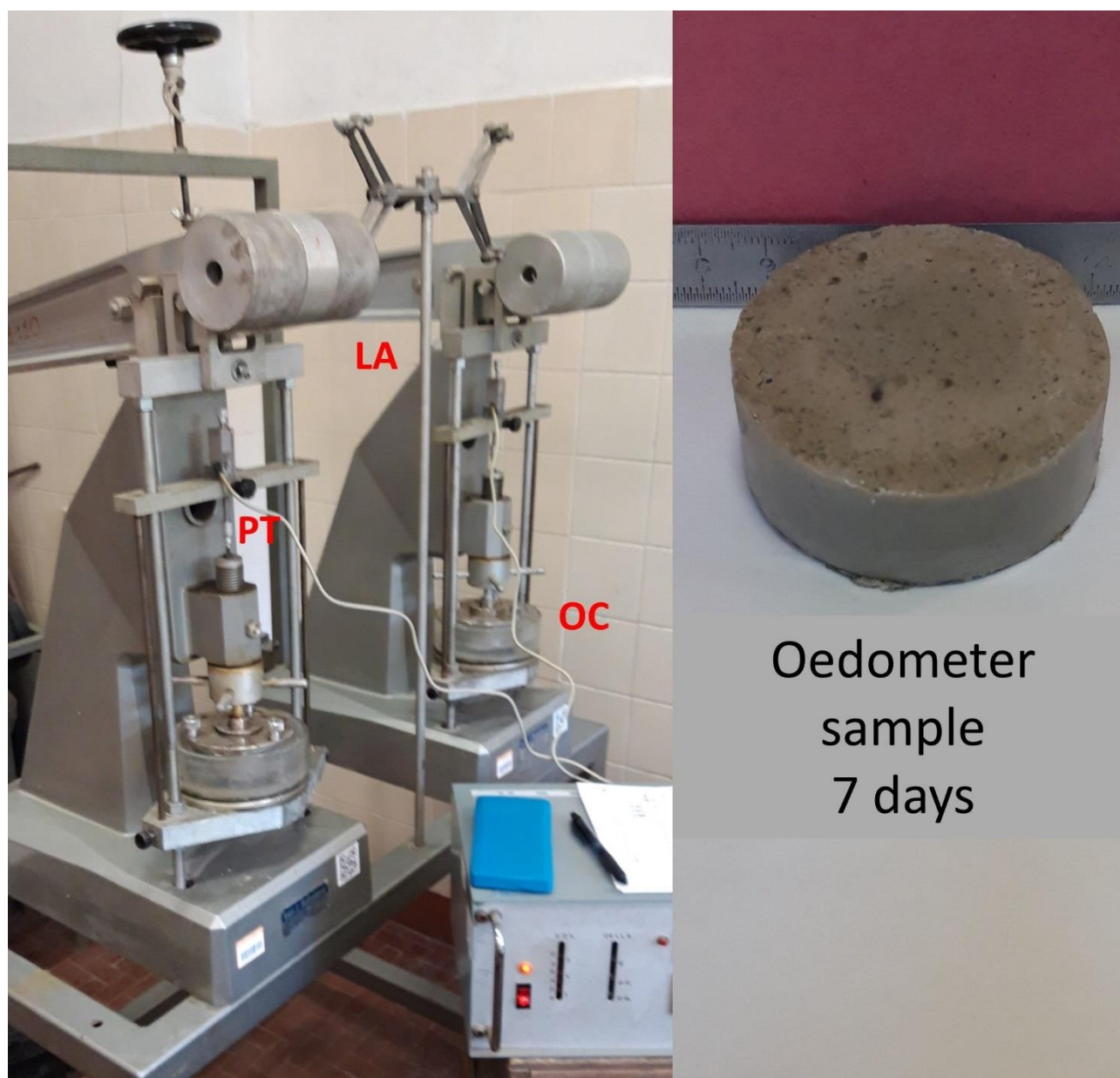


235

236 **Fig. 2 Testing equipment for compression (left) and enlargement of the UCS sample**
 237 **(right). LC: load cell; GS: grout sample; LVDT: radial coupled LVDT transducers;**
 238 **vLVDT: vertical LVDT transducer; PB: base of press plate; RCD: device for advance**
 239 **rate control.**

240 Oedometer testing has been performed by using a standard mechanical oedometer
 241 Belladonna equipment (Fig. 3). Ring type has been selected with net diameter of 50 mm
 242 and height of 20 mm; this size is sufficient to respect grain size distribution of formed
 243 grout. The standard test methods for one-dimensional consolidation properties of soils
 244 using incremental loading have been adapted to respect the fact that grout is curing during
 245 testing, and thus a compromise was necessary to avoid long term duration typical of
 246 oedometer tests in soils (from days to weeks for and an entire loading-unloading cycle).

247 Specimens have been maintained saturated during cycles, and displacement have been
248 measured by means of potentiometric transducers with precision of 0.01 mm.



249

250 **Fig. 3. Twin Bishop oedometer equipment used for grout testing (left) and**
251 **oedometer two-component grout sample (right). LA: loading arm; OC: oedometer**
252 **cell with specimen; PT: potentiometric transducer for vertical settlement.**

253 Additional testing has been devoted to obtaining some auxiliary information with the
254 determination of apparent unit weight (geometrical determination, so unit weight has been
255 determined with the geometrical measurement of the volume of specimen casing and of

256 the weight of ingredients used to fill that volume) and surface strength (by means of soil
257 pocket penetrometer).

258

259 Laboratory test results

260 *Compression test results*

261 The main results after uniaxial compression testing are reported in Tab. 1. Strength is
262 considered as the maximum value of stress obtained, for the great majority of cases, at
263 yield at the end of the elastic domain. Deformability values are indexed as secant moduli
264 and Poisson coefficient at 25%, 50% and 75% of the elastic domain and as tangential
265 values at 50% of the elastic domain.

266

[illegible]

n.1	1.240	0.92	44.4	52.0	55.2	76.7	0.03	0.03	0.04	0.06
n.2	1.233	0.40	10.6	16.9	19.0	34.5	0.15	0.14	0.11	0.12
n.3	1.239	0.84	29.2	37.8	47.7	77.5	0.14	0.10	0.08	0.04
n.4	1.143	0.50	27.6	38.1	39.8	68.6	0.03	0.03	0.05	0.03

268 **Table 1. Results obtained for the two-component grout at different curing timelines in uniaxial condition of compression.**

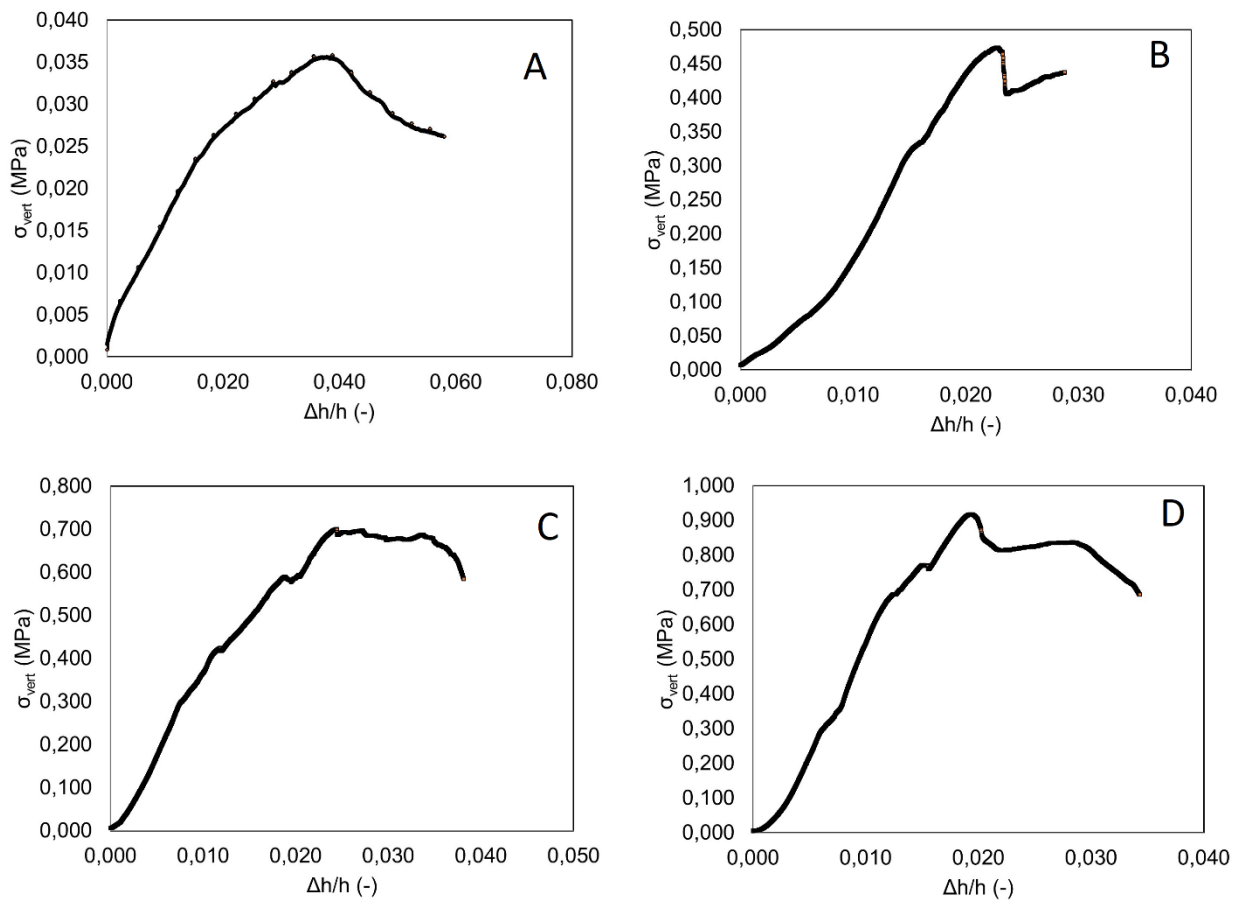
269 **Diameter of specimen is usually 48 mm if not indicated. The pedix “s” means secant value, the pedix “t” means tangent**

270 **value, the pedix “fm” means filling material, UCS: uniaxial compressive strength. Mix components are in the same proportion**

271 **for the specimen as referred in chapter 2.**

272

273 In Fig. 4 there is a representative sequence of vertical stress – vertical strain curves at
 274 different curing timelines.



275

276 **Fig. 4. Vertical stress - strain curves at different curing timelines for the grout. (A)**
 277 **curing time 1 hour, (B) curing time 24 hours, (C) curing time 7 days and (D) curing**
 278 **time 28 days. Each x and y-axis have different scale, adapted for each graph in**
 279 **order to properly show the curve shape. Legend: σ_{vert} : vertical stress; $\Delta h/h$: vertical**
 280 **strain, ratio of the vertical displacement on the sample height.**

281 The observed UCS values are rated slightly lower than expected due to the higher amount
 282 of retarder introduced. This was intentional, in order to asses also the particular effect of
 283 retarder, even if this is not linked to a specific mix design necessarily adopted in practice.

284 Testing in compression has generally been regular and vertical stress vs. vertical strain is
285 reliable both in the elastic and in the post peak field. A clear yielding and softening
286 behaviour has been observed, with some subvertical and inclined cracks prevailing. In
287 some cases, a pseudoconical shape at failure has been found at the extremities of the
288 specimen, thus respecting the ideal Mohr-Coulomb criterion failure geometry (Fig. 5). The
289 grain size of the cured grout specimens considered as valid appears regular and
290 homogeneous, without veins or lenses at different consistency.

291 The adoption of a linear Mohr – Coulomb criterion allows one to establish a relationship
292 between the angle of the failure plane α_f and the slope ϕ' of the Mohr Coulomb envelope.

293 The failure angle measured relative to the plane of the major principal stress is:

$$294 \quad \alpha_f = 45^\circ + \frac{\phi'}{2} \quad (1)$$

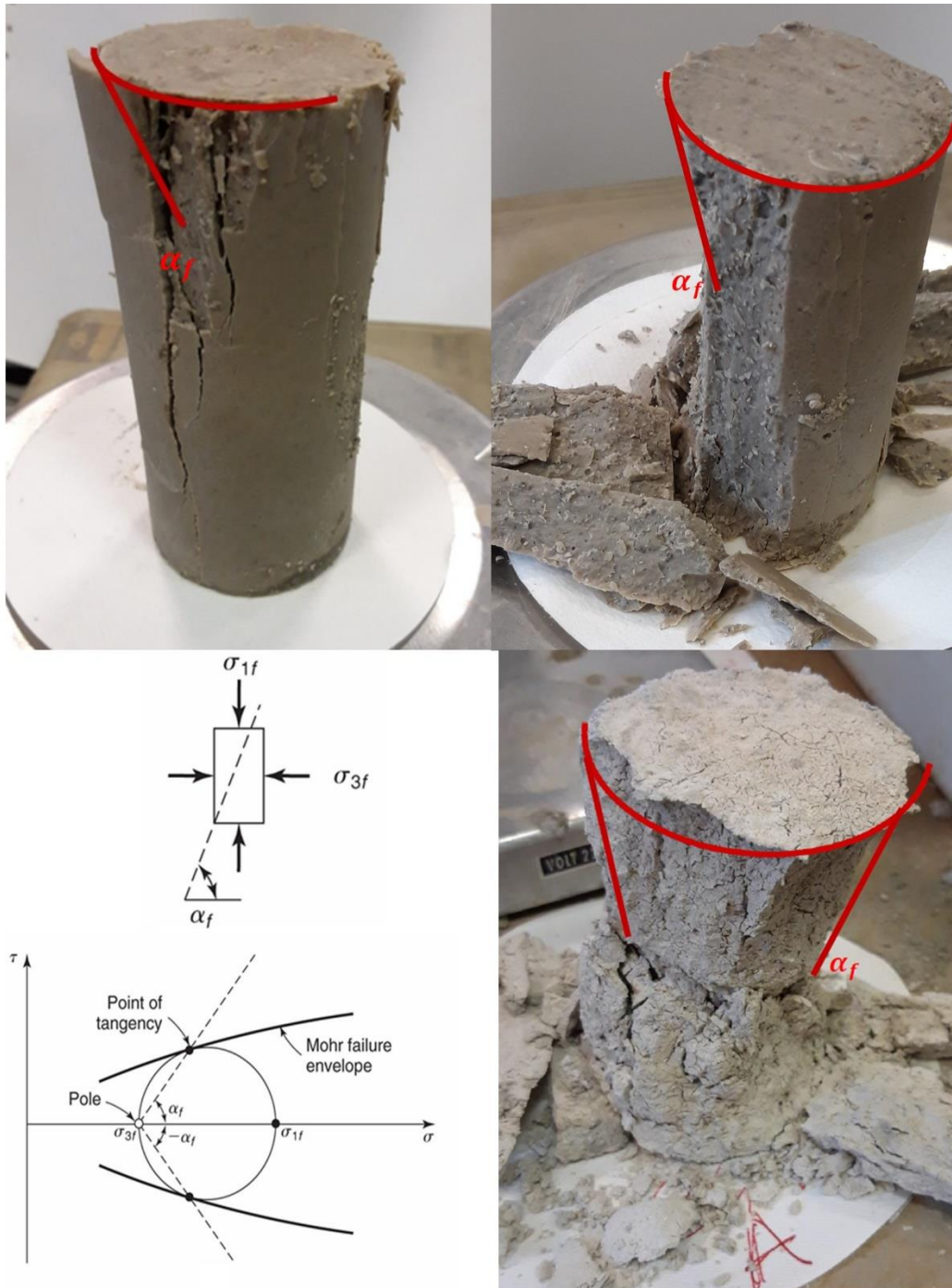
295 In case of UCS of a cohesive material, it is possible also to set:

$$296 \quad UCS = \frac{2 \cdot c' \cdot \cos \phi'}{1 - \sin \phi'} \quad (2)$$

297 The evidence that some of the failed specimens have exhibited a quite defined angle of
298 the failure planes can lead to an estimation of the friction angle and also of the cohesion of
299 the grout.

300 In Fig. 5 some examples of measured angles at failure are shown; it is necessary to
301 outline that this behaviour has not been clearly observed at the shorter curing time (1 h),
302 probably due to the wide peak and softening of the stress – strain curve in compression
303 that is linked to the distribution of growing cracks of the fresh grout. Easier to be observed
304 at longer curing timelines, where the linear part of the stress–strain curve allows to
305 maintain more defined and extended cracks. Common measured values of α_f are inside
306 the range of $65^\circ \div 68^\circ$, corresponding to values of ϕ' of about $40^\circ \div 46^\circ$. Due to the relatively
307 small number of tested specimens, it is not still possible to establish a trend of the friction

308 angle depending on the curing time. It is necessary to outline that shear strength
309 parameters are also depending on the water/cement ratio, on the amount of water glass
310 catalyst and on the percentage of bentonite, clearly variable for each grout mixing type and
311 thus affecting the evolution of such parameters with curing. The raw correspondence
312 among the involved parameters α_f , ϕ' , UCS, and c' , according to the available data,
313 provides values of c' in the following ranges: after 24hours: 85÷95 kPa; after 7 days:
314 90÷100 kPa; after 28 days: 135÷155 kPa.



315

316 **Fig.5. Some specimens after failure: on the upper ends the typical conical shape is**
 317 **developed and lateral slabbing as well, due to fine and homogenous grain size of**
 318 **the grout. The “A” specimen (bottom-right picture) has a diameter of 75 mm and it is**
 319 **desiccated. The graphical scheme of the specimen is showing the position of the**
 320 **failure plane and the corresponding link with the Mohr failure envelope.**

321 Figure 6 shows a representative specimen during the compression test and at failure at 28
322 days curing time. Failure appears as progressive, with slabbing and inclined cracking
323 propagation.



324

325 **Fig. 6. Sequence of loading and failure after 28 days curing of the grout.**

326 Less easy to be interpreted is the radial strain, at least for two reasons: the first is the
327 possibility to locate the LVDT in the zone of growing microcracks, that can both push out
328 the transducers or to leave them to move inside the crack opening; the second is that, in
329 any case, the greater and compulsory behaviour is clearly following the closure of
330 micropores and damage of cemented structure of the grout, and this happens along the

331 vertical direction. Concerning the possible evolution of strength parameters, observation
332 are still provisional due to the limited number of tested specimen that cannot allow one to
333 be so confident with characteristic values of strength parameters; preliminary results
334 seems that the greater effect will be on cohesion rather than in friction angle.

335

336 Oedometer test results

337 Confined compression tests can provide essential data in order to understand the behavior
338 of the grout mix, at different curing timelines and at different levels of vertical stress. The
339 fine-grained grout has allowed to use the 50 mm ring diameter as considered to be
340 representative for the geometrical scale of the material. Curing and testing have been
341 carried out in saturated conditions.

342 During this testing session four curing timelines have been adopted and loading –
343 unloading cycles have been carried out, namely following the following scheme:

- 344 • curing 1 hour: 6 new specimens, each of them loaded and unloaded at the
345 respective vertical stress of 50, 100, 200, 400, 800 and 1600 kPa; loading phase
346 has been extended for about 20 minutes, in order to respect the corresponding
347 duration of curing; unloading phase has been driven directly by removing the total
348 applied load for another 20 minutes;
- 349 • curing 24 hours: 6 new specimens with the same procedure just described, apart for
350 the duration of loading and unloading phases, which have been extended to 2
351 hours;
- 352 • curing 7 days: 2 new specimens, both loaded at 400 kPa for 6 hours for
353 comparison; then the load on the first specimen has been raised to 1200 kPa for
354 another 4 hours and finally unloaded to zero by measuring displacements for a time

lapse of 2 hours; the second specimen after loading at 400 kPa has been unloaded to zero in a time lapse of 2 hours;

- curing 28 days: 2 new specimens, for comparison, each of them step loaded at the respective vertical stress of 50, 100, 200, 400, 800 and 1600 kPa; loading phase has been extended for about 30 minutes for low loads (that is 50, 100 and 200 kPa, as settlements were stabilized), and for 18 h for higher loads (that is 400, 800 and 1600 kPa); the unloading phase has been stepped by reducing the total applied load to 800 kPa, then 200 kPa and finally to zero, carrying out settlement measurements for 24 hours at each step.

Figure 7 shows some examples of total settlement for the four different adopted curing timelines. The results can be interpreted following the main direction of one dimensional consolidation approach for soils, even a fundamental difference has to be outlined: grout presents a structure which is still chemically reactive and water contained in pores can be pushed aside (classical effect for soil grains) but can also migrate during reaction and therefore the expected properties at rest cannot be fulfilled. The classical approach by Casagrande (1936) can therefore be applied, even if with care.

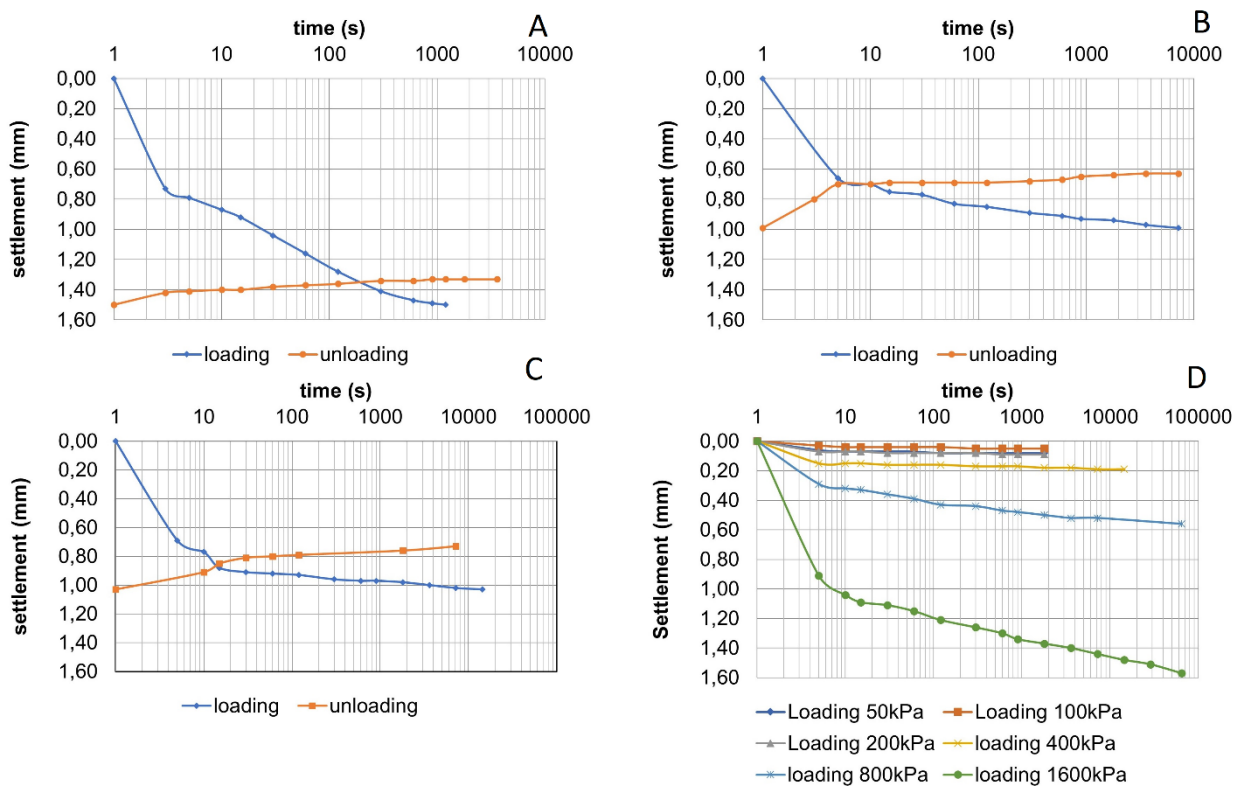
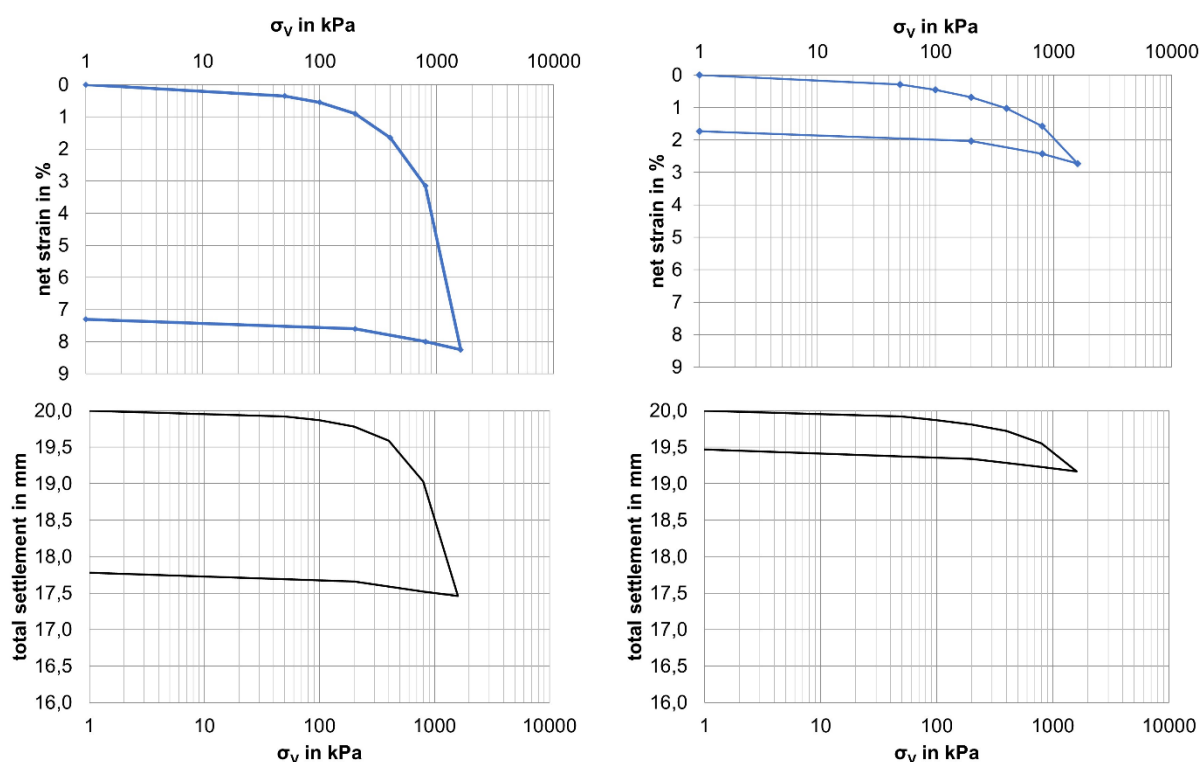


Fig. 7. Example of raw data in log t vs settlement at different grout curing timelines, obtained from oedometer testing (diameter 50 mm and height 20 mm). A) curing time 1 hour under 100kPa load, B) curing time 24 hours under 800kPa load, C) curing time 7 days under 1200kPa load, D) curing time 28 days under all loads.

Figure 8 presents the comparison among the specimens respectively of net strain, due to both mechanical and drainage process of deformation (consolidation), and the total settlement (viscous effects) for two selected curing ages, 1 and 24 hours. These results are interesting because they put in evidence that there is a relevant plastic deformation at the various stress levels, and that is not recoverable, for the various loading levels; the amount of recovered settlement after the unloading phase is low, sometimes negligible. Besides, specimens before and after oedometer test exhibit a clear geometrical change (see Fig. 9).

393 unloading at 100 kPa and 400 kPa: the difference in the residual thickness due to
 394 plastic settlements is remarkable.

395 Figure 10 summarize a typical stress-strain behaviour for the soil-like materials at the end
 396 of the curing period (28 days). A critical range of stress for the stability of the grout
 397 structure seems to confirm that grout has a meta-stable structure, due to diffusion of
 398 bentonite inside the material and to the initial high water/cement ratio.



399

400 **Fig. 10.** The diagrams are showing for the two specimens tested after 28 days of
 401 curing (OED1, left and OED 2, right) for loading and unloading cycle. Net strain
 402 values refer to “consolidation” phase of loading-unloading, while total settlement
 403 refers to the raw values at the beginning and at the end of each cycle.

404 Some interesting features arise from the diagrams showing the values of the constrained
 405 moduli, obtained in confined conditions (Fig. 11). It is shown that the curing time is
 406 affecting properties of the grout and its workability as well. Strength and stiffness are both

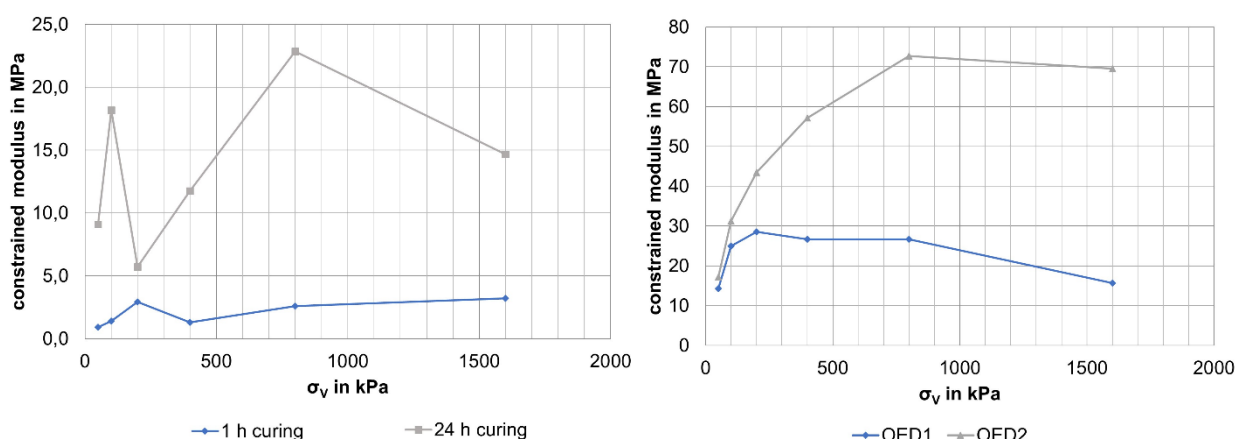
407 “mixing-sensitive”, and this fact should be taken into account for engineering application; in
408 fact, on one side, changes in mixing energy, temperature, moisture and time for mixing
409 can modify the structure, the behaviour and the properties of the grout. On the other side,
410 at the scale of the machine, it is necessary that a suitable compromise is reached for the
411 mixing timing: in fact water glass addition should be operated just to allow the quick setting
412 of the gel and of the cementitious structure, but not so anticipated for the risk of clogging of
413 the grout pipes and nozzles. The final result of a viscous grout to properly fill and support
414 the gap between the lining extrados and the ground excavated profile is the goal.

415 Curing is affecting the values of constrained modulus, as it happened already for the
416 uniaxial compressive test. Moreover, it is interesting that for short curing timelines, a peak
417 value of the modulus appears (namely at 1h and 24 h). This fact seems to be in agreement
418 with the external aspect of the grout, which is similar to a medium consistent clay at the
419 beginning of the curing period, and that reaches the characteristics of a hard soil for long
420 curing time. The presence of these peaks in the modulus graph was well described by
421 Janbu (1969) in the case of fine-grained soils. Its importance is related to the local range
422 of applied stress that is linked to pre-consolidation pressure. It is clear that such
423 phenomenon cannot occur during the preparation of the grout, but this evidence can be
424 interpreted as a “meta-stable pressure” for the grout structure “in formation”. This proposal
425 is quite interesting as it can justify the clear change of behaviour for clay-like conditions of
426 the grout. The similarity to real soil behaviour is arising from the possibility to observe a
427 good adherence to settlement vs time and vertical pressure vs strain of the loading steps
428 of tested grout mix. Moreover the results in Fig.11 concerning the modulus vs the stress
429 level are typical of consolidated clays. That is a convenient reason to make confident to
430 approach with one-dimensional consolidation theory. For sure grout is an artificial material,
431 with cemented bonds that are different from cohesion arising from water suction and

432 surface membrane effect due to polarity of clay particles: the discussion on such difference
 433 should be considered for specific laboratory comparison.

434

435



436

437 **Fig. 11. Scaled graphs showing the distribution of constrained moduli at different**
 438 **curing ages (1 hour and 24 hours at left, 28 days at right) and depending on the**
 439 **vertical applied stress. Note that the y-axis is different to better show the shape of**
 440 **the graphs.**

441 As referred to the original thickness of the specimen (20 mm) and to the specific step in
 442 the vertical effective stress ($\Delta\sigma_v$), in the previous graphs the values of the constrained
 443 moduli can be considered as tangent values. As far as the constrained modulus is
 444 concerned (referring to the full value of applied vertical stress σ_v), after 28 days of curing
 445 time the following values have been measured (Tab. 2):

Secant constr. modulus at 28 d in MPa	σ_v 50 kPa	σ_v 100 kPa	σ_v 200 kPa	σ_v 400 kPa	σ_v 800 kPa	σ_v 1600 kPa
OED 1	14.3	18.2	22.2	24.2	25.4	19.4

OED 2	17.2	22.2	29.4	38.4	50.6	58.6
-------	------	------	------	------	------	------

446 **Table 2. The secant values of the constrained moduli at 28 days of curing are listed.**

447 Another relevant issue is concerning the lateral expansion of the grout under loading.
448 Some data arise from the compression testing, showing that the Poisson coefficient “ ν ”
449 exhibits generally low values. In order to find a confirmation of this behaviour, a correlation
450 between oedometer results and compression results has been arranged, taking into
451 account the basic correlation valid for linear elasticity and for homogenous and isotropic
452 materials in constrained conditions:

$$453 \quad M_{constr} = \frac{(1-\nu)}{(1+\nu)(1-2\nu)} \cdot E \quad (3)$$

454 Where M_{constr} is the oedometer modulus, ν is the Poisson ratio and E is the elastic
455 modulus.

456 The cross checking of the raw data in the uniaxial compression tests and in the oedometer
457 tests allows to obtain values of Poisson coefficient in the range of 0.03÷0.15, confirming
458 the results obtained by direct measures during uniaxial compression tests. The selection of
459 a proper value should follow some criteria such as:

- 460 • linearity of the stress – strain envelope;
- 461 • type of expected conditions of confinement in the real case;
- 462 • level of stress expected in site.

463 Nevertheless, this appear as one of the most sensitive features of the grout behaviour,
464 thus requesting a larger data base of raw experimental data.

Some additional information can be obtained by means of a further processing of the available data. In order to measure the permeability, it was decided to indirectly obtain through an oedometer test, as for very low permeability values (10^{-8} m/s) it is preferable to use indirect tests (e.g. Colombo and Colleselli, 1996). The indirect coefficient of permeability k can be obtained by as combination of the consolidation coefficient C_V and of the coefficient of volume change m_v by adopting the formula $k = C_V * m_v * \gamma_w$. The intermediate terms can be obtained if some assumptions are taken into account: a) the behaviour of the grout during confined loading is similar to that of fine natural granular materials; b) the interpretation of raw data should follow the one-dimensional consolidation theory; c) the Casagrande method to interpret the rheology of the grout is valid to identify the compressibility features and characteristics of the grout; d) the general limitations on the validity of the indirect coefficient of permeability, known for soils, are maintained also for the artificial grout while carrying out the interpretation of the results. Following these statements, the values of k are presented in the Tab. 3.

Vertical stress in kPa	1 h curing time	24 h curing time	7 d curing time	28 d curing time OED1	28 d curing time OED2
50	$1.35 \cdot 10^{-7}$	$2.15 \cdot 10^{-7}$	-	$2.75 \cdot 10^{-7}$	$2.85 \cdot 10^{-7}$
100	$4.32 \cdot 10^{-8}$	$7.16 \cdot 10^{-8}$	-	$1.56 \cdot 10^{-7}$	$1.78 \cdot 10^{-7}$
200	$8.87 \cdot 10^{-7}$	$9.41 \cdot 10^{-7}$	-	$2.26 \cdot 10^{-7}$	$1.11 \cdot 10^{-7}$
400	$2.25 \cdot 10^{-8}$	$3.21 \cdot 10^{-7}$	$1.25 \cdot 10^{-7}$ $2.73 \cdot 10^{-9}$	$2.39 \cdot 10^{-7}$	$8.42 \cdot 10^{-8}$
800	$3.35 \cdot 10^{-8}$	$1.63 \cdot 10^{-7}$	-	$2.30 \cdot 10^{-7}$	$6.53 \cdot 10^{-8}$
1200	-	-	$1.29 \cdot 10^{-7}$	-	-
1600	$1.09 \cdot 10^{-8}$	$9.07 \cdot 10^{-8}$		$2.61 \cdot 10^{-7}$	$6.63 \cdot 10^{-8}$

Table 3. Values of indirect theoretical permeability coefficient k in cm/s.

480 It is important to underline again the fact that the above listed values are obtained through
481 an indirect procedure and not by means of a drainage test. The consequence of this fact is
482 that an interpretation and an engineering judgement is necessary to properly adopt reliable
483 values in practical design. The following issues should be taken into account: a) the order
484 of magnitude of $1.1 \cdot 10^{-7}$ cm/s to $9.4 \cdot 10^{-7}$ cm/s seems reasonable for the majority of tested
485 cases; b) the values at $1.0 \cdot 10^{-8}$ to $9.0 \cdot 10^{-8}$ cm/s are quite optimistic outside the laboratory
486 scale; moreover, settlements and time-dependent movements for both ground and
487 segmental lining can affect the global permeability; c) values in the order of 10^{-9} cm/s
488 seems to be unrealistic. As final consideration, it can be observed that the tested grout has
489 low to very low permeability parameters for a wide range of operational stresses.

490 *Auxiliary testing*

491 Together with data provided in the sheet for cylindrical specimens, some additional
492 information arises from the specimens prepared for oedometer testing. In these cases,
493 apparent unit weight is varying in the range 1.20 to 1.40 g/cm³ for fresh and cured
494 specimen respectively. Interesting is the reduction of apparent unit weight after loading-
495 unloading cycle for fresh specimen at high level of consolidation pressure: the reduction
496 moves to 0.75 to 1.08 g/cm³. After drying at natural environmental conditions, apparent
497 unit weight for both cylindrical and disc specimen reduces to less than 0.80 g/cm³,
498 reaching also 0.71 g/cm³; desiccated grout is friable and crispy. Soil pocket penetrometer
499 (Soil Test model) has provided, for the first disc specimen, a penetration strength at 1 hour
500 of 90 - 80 - 85 - 80 - 80 - 80 kPa; after 2 hours, the strength was increased to about 120 -
501 120 - 125 kPa. The second disc specimen presented a penetration strength after 1 hour of
502 about 70 - 60 - 85 kPa, and after 2 hours of about 125 - 150 - 90 - 140 - 90 - 110 kPa. Both
503 specimens after 24 hours reached the full range of the soil pocket penetrometer at more

504 than 400 kPa; this same result was obtained on the lateral surface of the large diameter
505 cylindrical specimen (75 mm).

506 **3. Simplified methods of tunnel segmental lining analysis**

507 Two methods for the behavior analysis of retaining structures are commonly used in the
508 tunnel field: the convergence-confinement method and the Einstein and Schwartz method
509 (1979). These methods have the advantage of being able to effectively evaluate the
510 complex mechanism of interaction between the support and the walls of the tunnel, using a
511 simplified approach that does not require the use of numerical calculation methods.

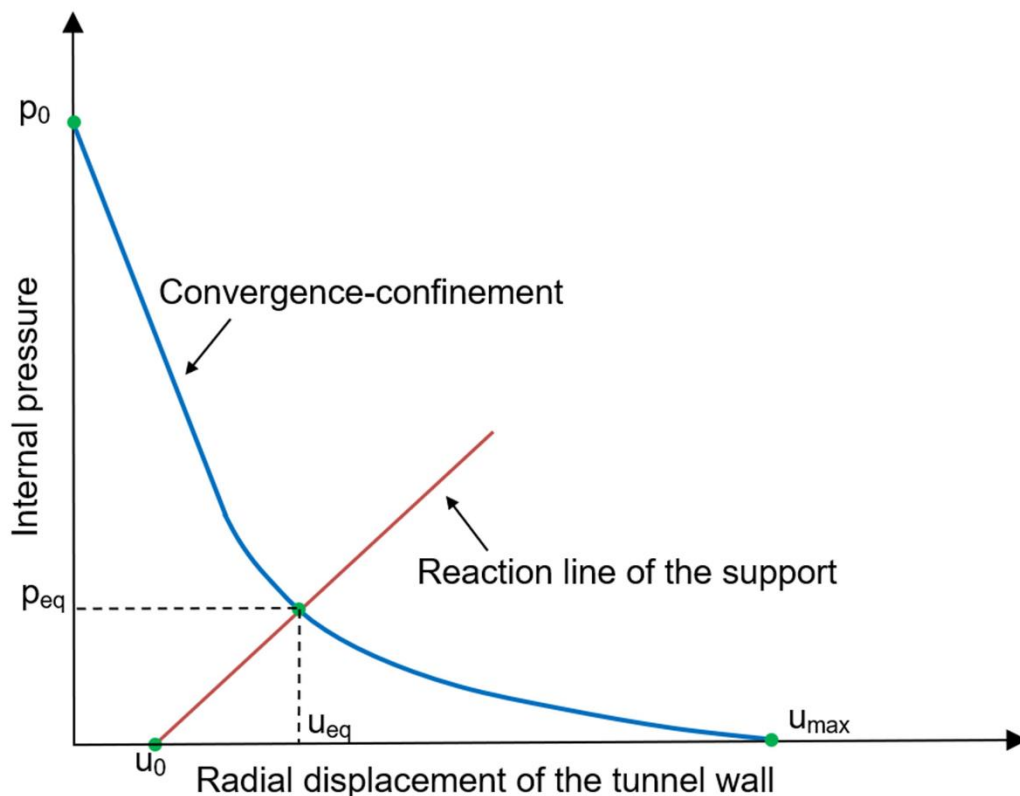
512 More specifically, the convergence-confinement method is able to evaluate the final entity
513 of the loads acting on the support structure, p_{eq} through the intersection of the
514 convergence-confinement curve and the reaction line representing the support structure,
515 determining the displacement of the tunnel wall in the radial direction (u_{eq}) (Fig. 12). The
516 convergence-confinement method is based on the following fundamental assumptions:

- 517 • Circular geometry of the tunnel and radial symmetry of the problem analyzed in the
518 two dimensions (vertical section)
- 519 • Deep tunnel hypothesis (depth of the tunnel relatively high compared to its radius)
- 520 • Homogeneous mechanical characteristics of the rock around the tunnel;
- 521 • Hydrostatic initial stress state (undisturbed) p_0 with lateral earth coefficient at rest
522 $K_0 = 1$.

523 The main problem in using the convergence-confinement method lies in being able to
524 correctly simulate the three-dimensional nature of the support loading mechanism in a two-
525 dimensional model. The fundamental parameter for this purpose is the displacement u_0 of
526 the tunnel wall when the support structure is installed. Various calculation techniques are

527 available in the literature capable of producing an estimate of this parameter and therefore
 528 having reliable results from the convergence-confinement method.

529 Fig. 12 shows the results of the convergence-confinement method through the analysis of
 530 two curves: the convergence-confinement curve and the reaction line of the supporting
 531 structure. p is the pressure inside the tunnel, acting on the walls, u_R is the radial
 532 displacement of the tunnel wall, p_0 is the lithostatic stress present in the rock, u_0 is the
 533 radial displacement of the tunnel wall upon installation of the support; p_{eq} and u_{eq} are
 534 respectively the final load acting on the support structure and the final displacement of the
 535 tunnel wall in the presence of the support structure and u_{max} is the maximum
 536 displacement of the tunnel wall in the absence of supports.



537

538 **Fig. 12 Results of the convergence-confinement method through the analysis of two**
539 **curves: the convergence-confinement curve and the reaction line of the supporting**
540 **system.**

541 In the simplest case of ideal elastic-plastic behavior of the rock and Mohr-Coulomb failure
542 criterion, the convergence-confinement curve is expressed by the following relationship
543 (Oreste, 2009):

544 For $p < [p_0 \cdot (1 - \sin(\varphi)) - c \cdot \cos(\varphi)]$:

545 $u_R =$

$$546 \frac{1+\nu}{E} \cdot \left\{ \left[\frac{R_{pl}^{N_\Psi+1}}{R^{N_\Psi}} \cdot \sin(\varphi) + (1 - 2 \cdot \nu) \cdot \left(\frac{R_{pl}^{N_\Psi+1}}{R^{N_\Psi}} - R \right) \right] \cdot \left(p_0 + \frac{c}{\tan(\varphi)} \right) - \frac{1+N_\Phi \cdot N_\Psi - \nu \cdot (N_\Psi+1) \cdot (N_\Phi+1)}{(N_\Phi+N_\Psi) \cdot R^{(N_\Phi-1)}} \cdot \right. \\ 547 \left. \left(\frac{R_{pl}^{(N_\Phi+N_\Psi)}}{R^{N_\Psi}} - R^{N_\Phi} \right) \cdot \left(p + \frac{c}{\tan(\varphi)} \right) \right\} \quad (4)$$

548 where R_{pl} is the plastic radius of the tunnel:

$$549 R_{pl} = R \cdot \left[\frac{\left(p_0 + \frac{c}{\tan(\varphi)} \right) \cdot (1 - \sin(\varphi))}{p + \frac{c}{\tan(\varphi)}} \right]^{\frac{1}{(N_\Phi-1)}} \quad (5)$$

$$550 N_\Phi = \frac{1+\sin(\varphi)}{1-\sin(\varphi)} \quad (6)$$

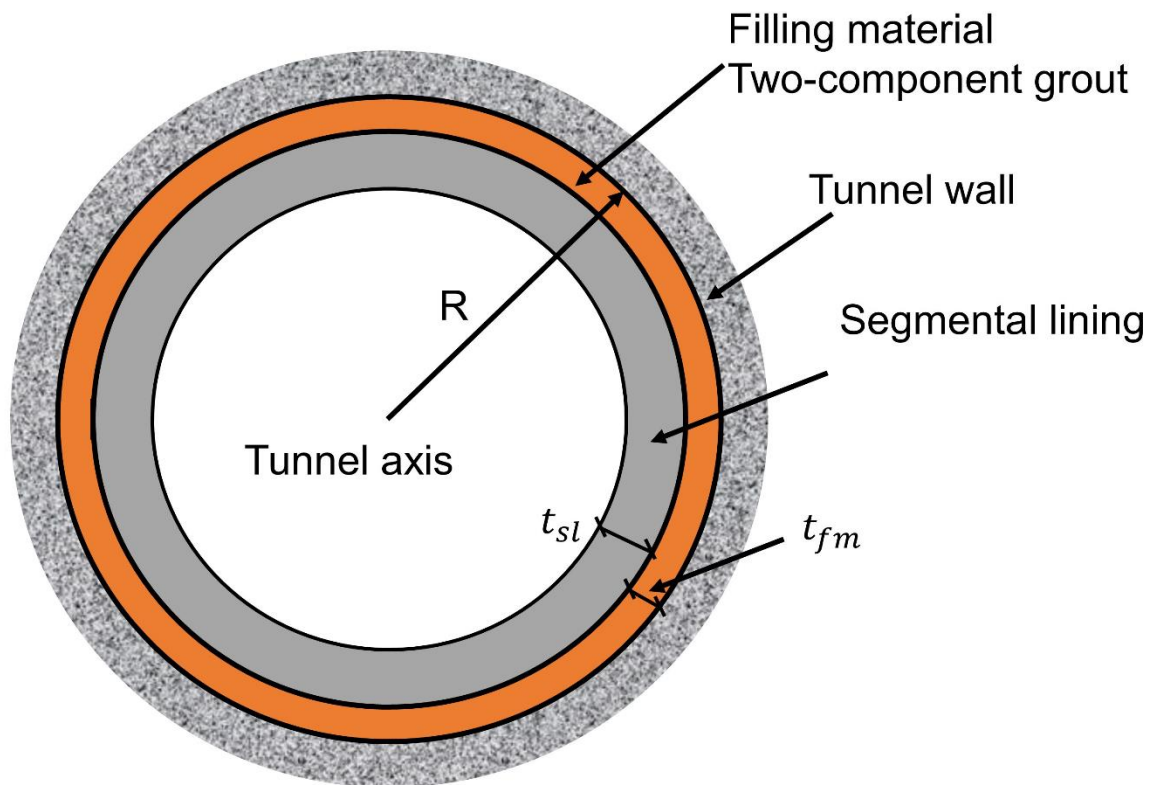
$$551 N_\Psi = \frac{1+\sin(\Psi)}{1-\sin(\Psi)} \quad (7)$$

552 R is the tunnel radius, c , φ and Ψ are respectively the cohesion, friction angle and
553 dilatancy of the rock mass, E and ν are respectively the elastic modulus and the Poisson
554 ratio of the rock mass.

555 For $p > [p_0 \cdot (1 - \sin(\varphi)) - c \cdot \cos(\varphi)]$:

$$u_R = \frac{1+\nu}{E} \cdot (p_0 - p) \cdot R \quad (8)$$

The reaction line of the support in the case of a segmental lining must take into account the presence of the filling material in the gap between the segmental lining and the tunnel wall (Fig. 13).



560

Fig. 13. Geometric sketch of the support system consisting of segmental lining and filling material within a circular tunnel. Legend: R : radius of the tunnel; t_{sl} : thickness of the segmental lining; t_{fm} : thickness of the filling material (not to scale).

Similarly to what was developed for the shotcrete lining plus inner steel set support system (Oreste, 2003; Oreste et al., 2018a, 2018b), it is possible now to define for the reaction line of the support the stiffness, k_{sys} , of the system consisting of the segmental lining and the annulus of the filling material with the following relationship:

$$k_{sys} = \frac{2 \cdot E_{fm} \cdot (1 - \nu_{fm}) \cdot R \cdot \left[\frac{E_{fm}}{(1 + \nu_{fm})} + (R - t_{fm}) \cdot k_{sl} \right]}{E_{fm} \cdot (1 - 2 \cdot \nu_{fm}) \cdot R^2 + (R - t_{fm})^2 \cdot \left[E_{fm} + (1 - 2 \cdot \nu_{fm}) \cdot (1 + \nu_{fm}) \cdot k_{sl} \cdot t_{fm} \cdot \left(1 + \frac{R}{(R - t_{fm})} \right) \right]} - \frac{E_{fm}}{(1 + \nu_{fm}) \cdot R} \quad (9)$$

where:

$$k_{sl} = \frac{E_{sl}}{(1 + \nu_{sl})} \cdot \frac{(R - t_{fm})^2 - (R - t_{fm} - t_{sl})^2}{(1 - 2 \cdot \nu_{sl}) \cdot (R - t_{fm})^2 + (R - t_{fm} - t_{sl})^2} \cdot \frac{1}{(R - t_{fm})} \quad (10)$$

E_{fm} and ν_{fm} are respectively the elastic modulus and the Poisson's ratio of the filling material; E_{sl} and ν_{sl} are respectively the elastic modulus and the Poisson's ratio of the segmental lining; t_{fm} and t_{sl} are respectively the thickness of the filling material and segmental lining; k_{sl} is the stiffness of the segmental lining. As can be seen from the previous equations, knowing the characteristics of the filling material (thickness, elastic modulus and Poisson's ratio) it is possible to identify the stiffness of the support system which allows to draw the reaction line of the support. In fact, the stiffness of the system represents the inclination of the reaction line with respect to the horizontal axis in the diagram of Fig. 12.

Since the elastic modulus of the filling material E_{fm} varies significantly during the setting period and, therefore, during the loading of the segmental lining, it is necessary to define a representative average value for the considered period. To evaluate it, it is necessary to know not only the trend of the elastic modulus of the filling material over time during the setting period (curing time), but also the advancement speed of the TBM inside the tunnel and the duration of the various excavation and installation of supports. It is a question of addressing this problem with the same approach used for the evaluation of the average elastic modulus of shotcrete during the loading phase of the lining in the tunnel, with the advancement of the excavation phase (e.g. Oreste et al., 2019).

Another very widespread calculation method in the analysis of the behavior of the supporting structures of tunnels is the method of Einstein and Schwartz (1979). This method assumes an annular support continuously connected with the wall of a deep and circular tunnel. Two different cases were examined by the authors: the full slip case and the no-slip case. The solution obtained allows to consider the interaction between the supporting structure and the rock mass around the tunnel, assuming a material with an elastic behavior both for the rock and for the support. For the analysis of the segmental lining with the filling material present in contact with the rock wall, it is more appropriate to refer to the full-slip case, which provides the following equations for the evaluation of the maximum bending moment, M_{max} , and the normal force, N , present in the lining (in particular in the center of the crown and in the middle of the side-wall) (Einstein and Schwartz, 1979; Guan et al., 2015):

$$M_{max} = (1 + \xi) \cdot \frac{p_{eq} \cdot R^2 \cdot (1 - K_0)}{(1 + K_0) \cdot (1 - a_0^*) + (1 - K_0) \cdot (3 - 6 \cdot a_2^*)} \cdot (1 - 2 \cdot a_2^*) \quad (11)$$

$$N_{crown} = \frac{p_{eq} \cdot R \cdot (1 + K_0)}{(1 + K_0) \cdot (1 - a_0^*) + (1 - K_0) \cdot (3 - 6 \cdot a_2^*)} \cdot (2 \cdot a_2^* - a_0^*) \quad (12)$$

$$N_{sidewall} = \frac{p_{eq} \cdot R \cdot (1 + K_0)}{(1 + K_0) \cdot (1 - a_0^*) + (1 - K_0) \cdot (3 - 6 \cdot a_2^*)} \cdot (2 - a_0^* - 2 \cdot a_2^*) \quad (13)$$

where:

$$a_0^* = \frac{C^* \cdot F^* \cdot (1 - \nu)}{C^* + F^* + C^* \cdot F^* \cdot (1 - \nu)} \quad (14)$$

$$a_2^* = \frac{(F^* + 6) \cdot (1 - \nu)}{2 \cdot F^* \cdot (1 - \nu) + 6 \cdot (5 - 6 \cdot \nu)} \quad (15)$$

$$C^* = \frac{E \cdot R \cdot (1 - \nu_s^2)}{E_s \cdot A_s \cdot (1 - \nu^2)} \quad (16)$$

$$F^* = \eta \cdot \frac{E \cdot R^3 \cdot (1 - \nu_s^2)}{E_s \cdot I_s \cdot (1 - \nu^2)} \quad (17)$$

p_{eq} is vertical load acting on the support structure in the vertical direction, in correspondence with the crown tunnel (evaluated, for example, through the convergence-confinement method);

K_0 is the lateral earth pressure at rest in the rock, in the initial undisturbed conditions;

R is the tunnel radius;

E and ν are respectively the elastic modulus and the Poisson ratio of the rock;

E_s and ν_s are respectively the elastic modulus and the Poisson ratio of the support structure;

A_s and I_s are respectively the area and moment of inertia of the cross section of the support, through a plane passing through the axis of the tunnel. The cross section therefore has a rectangular section, whose width is equal to 1 m in the direction of the tunnel axis and the height is equal to the thickness of the support

C^* and F^* are compressibility ratio and flexibility ratio of the support, respectively.

ξ is the incremental coefficient that takes into account the transfer of stresses from one ring to the adjacent one, in correspondence with the longitudinal joints of the segmental lining. Guan et al. (2015) were able to note how this parameter varies from 0.44 to 0.46, with an intermediate value equal to 0.45 and is not influenced by the characteristics of the ground and the depth of the tunnel, but only by the geometric and mechanical characteristics of the longitudinal and transverse joints of the segmental lining;

627 η is reduction coefficient taking into account the presence of longitudinal joints in
628 segmental lining. In this regard Guan et al. (2015) suggest reducing the bending stiffness
629 by a coefficient η , which was found to vary between 0.4 and 0.7, with an intermediate
630 value of 0.55. This coefficient was found to be higher in more compact soils and for
631 tunnels at greater depths.

632 The maximum moment M_{max} assumes the same value in the center of the crown and on
633 the sidewall, but a different sign: the one in the crown will be considered positive and the
634 one on the sidewall negative, for the typical case in which K_0 is less than 1; if K_0 is greater
635 than 1, the moment on the sidewall will be positive and the moment at the crown tunnel is
636 negative.

637 The normal forces are different in the tunnel crown and on the sidewall: by associating the
638 values of the normal forces with the bending moments, it is possible to check the
639 hypothesized lining in the two critical points highlighted above: the center of the crown
640 tunnel and the center of the. In the verification process, the thickness of the segmental
641 lining is changed to obtain a value considered compatible with the safety and stability
642 conditions of the support structure and the tunnel.

643 When the use of segmental lining as a support structure is planned, it is important to
644 consider composite support is obtained which also includes a ring of filling material
645 between the segmental lining and the wall of the tunnel. The evaluation of the equivalent
646 elastic modulus of the support must consider the presence of the two materials (Fig. 13).
647 Assuming the preservation of the flat sections of the lining (segmental lining and filling
648 material), it is possible to identify the equivalent elastic modulus $E_{s,eq}$ of the support. This
649 equivalent elastic modulus turns out to be different in relation to the bending and

650 compression of the lining and, therefore, must be evaluated through different equations
 651 with reference to the compressible ratio C^* and the flexibility ratio F^* .

652 Equivalent elastic modulus of the support with reference to the compressibility ratio C^* :

$$653 \quad E_{s,eq(C^*)} = E_{sl} + E_{fm} \cdot \frac{t_{fm}}{t_{sl}} \quad (18)$$

654 Equivalent elastic modulus of the support with reference to the flexibility ratio F^* :

$$655 \quad E_{s,eq(F^*)} = \frac{4}{t_{sl}^3} \cdot \left\{ E_{sl} \cdot [y_0^3 + (t_{sl} - y_0)^3] + E_{fm} \cdot \left[(t_{sl} + t_{fm} - y_0)^3 - (t_{sl} - y_0)^3 \right] \right\} \quad (19)$$

656 Where:

$$657 \quad y_0 = \frac{E_{sl} \cdot t_{sl}^2 + E_{fm} \cdot (t_{fm}^2 + 2 \cdot t_{sl} \cdot t_{fm})}{2 \cdot (E_{sl} \cdot t_{sl} + E_{fm} \cdot t_{fm})} \quad (20)$$

658 Being y_0 the distance of the neutral axis of the section from the intrados of the segmental
 659 lining.

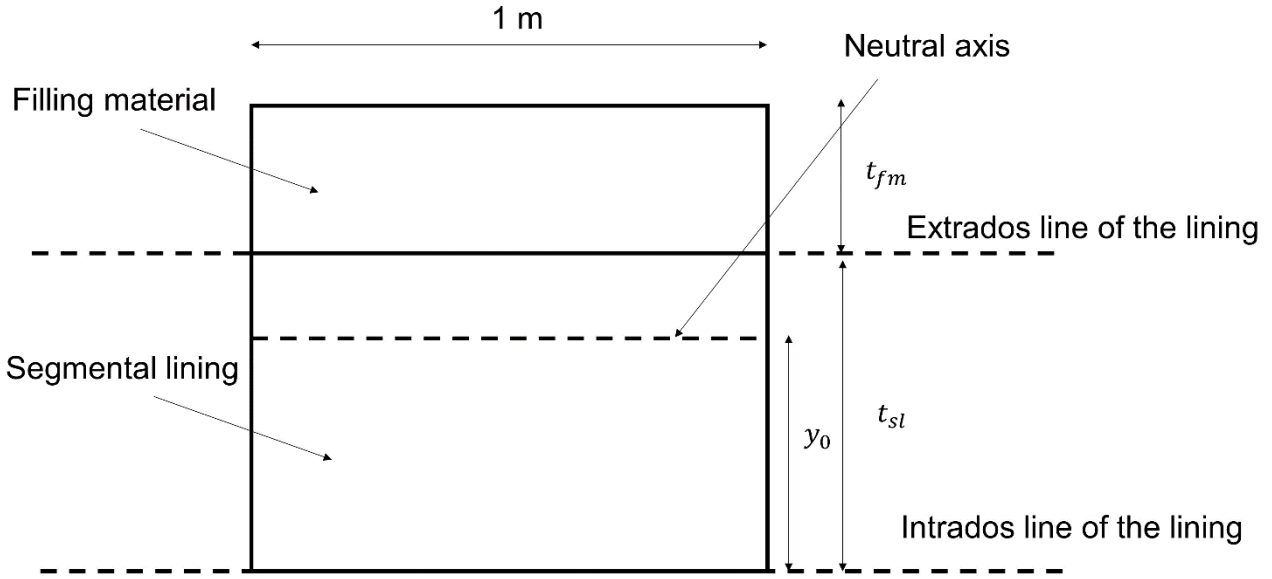
660 The values of the equivalent elastic modules reported above were obtained assuming an
 661 equivalent support of thickness equal to the thickness of the segmental lining: $t_{s,eq} = t_{sl}$.

662 For this reason, the values of the area of the support section A_s and of the moment of
 663 inertia I_s must be evaluated with the following two equations:

$$664 \quad A_s = b_s \cdot t_{sl} \quad (21)$$

$$665 \quad I_s = \frac{b_s \cdot t_{sl}^3}{12} \quad (22)$$

666 Where b_s is the width of the support section in the direction of the tunnel axis, considered
 667 equal to 1 m.



668

669 **Fig. 14. Cross section of the segmental lining with the presence of the filling**
 670 **material. Legend: t_{sl} is thickness of the segmental lining; t_{fm} is thickness of the**
 671 **filling material; y_0 is distance of the neutral axis of the section from the intrados of**
 672 **the segmental lining.**

673 Based on the calculations that can be developed considering the equivalent support
 674 described above, it will be possible to identify the maximum moments along the
 675 development of the support and the normal forces at the center of the crown and the
 676 sidewall. Starting from these results, it will be possible to determine the circumferential
 677 stresses σ_θ in the segmental lining and in the filling material using the following equations.

678 Circumferential stress at the extrados of the segmental lining $\sigma_{\theta,sl,ex}$ due to moment M :

$$679 \quad \sigma_{\theta,sl,ex} = \frac{12 \cdot M}{E_{s,eq(F^*)} \cdot b_s \cdot t_{sl}^3} \cdot (t_{sl} - y_0) \cdot E_{sl} \quad (23)$$

680 Circumferential stress at the intrados of the segmental lining $\sigma_{\theta,sl,in}$ due to moment M :

$$681 \quad \sigma_{\theta,sl,in} = - \frac{12 \cdot M}{E_{s,eq(F^*)} \cdot b_s \cdot t_{sl}^3} \cdot y_0 \cdot E_{sl} \quad (24)$$

682 Maximum circumferential stress on the extrados of the filling material $\sigma_{\vartheta, fm, ex}$ due to the
 683 moment M :

$$684 \quad \sigma_{\vartheta, fm, ex} = \frac{12 \cdot M}{E_{s, eq(F^*)} \cdot b_s \cdot t_{sl}^3} \cdot (t_{fm} + t_{sl} - y_0) \cdot E_{fm} \quad (25)$$

685 Constant stress in the segmental lining section due to the normal force N :

$$686 \quad \sigma_{\vartheta, sl} = \frac{E_{sl}}{E_{s, eq(C^*)}} \cdot N \quad (26)$$

687 Constant stress in the section of the filling material due to the normal force N :

$$688 \quad \sigma_{\vartheta, fm} = \frac{E_{fm}}{E_{s, eq(C^*)}} \cdot \frac{t_{fm}}{t_{sl}} \cdot N \quad (27)$$

689 The stress effects of the bending moment M and the normal force N must be summed
 690 algebraically to obtain the overall stress state in the filling material and in the segmental
 691 lining; in addition to the circumferential stresses, the radial stresses present in the two
 692 materials must be considered: p_{eq} at the extrados of the filling material and segmental
 693 lining and 0 at the intrados of the segmental lining. The overall stress state has to be
 694 compared to the strength of the material (the limit stress state), according to the Mohr-
 695 Coulomb strength criterium:

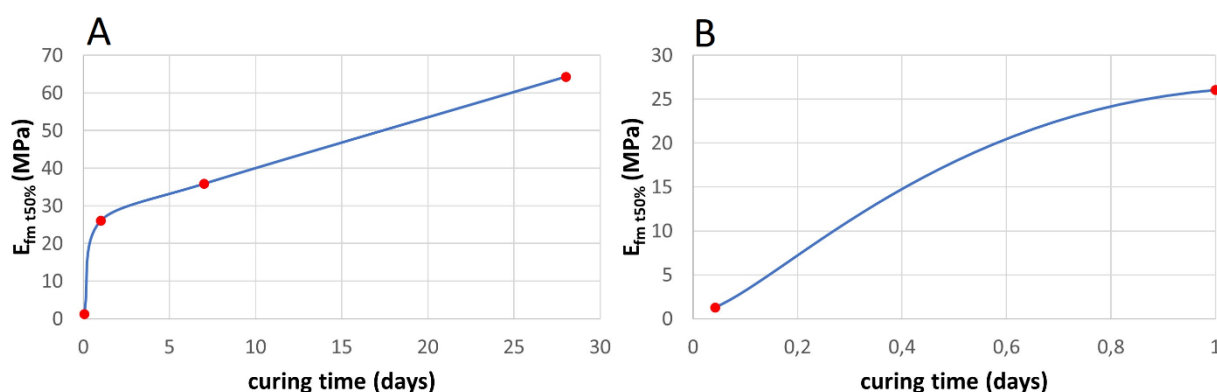
$$696 \quad \sigma_{lim} = UCS + \frac{1 + \sin(\varphi)}{1 - \sin(\varphi)} \cdot \sigma_r \quad (28)$$

697 Where UCS and φ are respectively the uniaxial compression strength and the friction angle
 698 of the material (concrete or filling material) and σ_r is the radial stress in the point where the
 699 stress state is evaluated during the design of the support structure (extrados of the filling
 700 material annulus, extrados or intrados of the segmental lining).

701 The knowledge of the overall stress state in the critical points of the support system allows
 702 to proceed to the verifications regarding the strength of the materials and, therefore, to the
 703 design of the segmental lining through the definition of its thickness and the internal
 704 reinforcement necessary to absorb the developing bending moments.

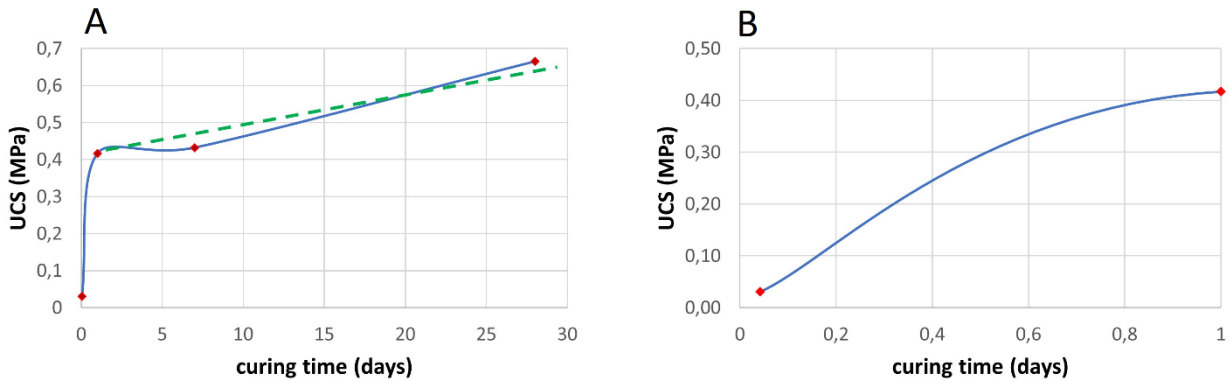
705 4. The effect of the two-component grout on the behavior of the tunnel 706 segmental lining

707 From the laboratory tests on the two-component grout used as filling material, it is possible
 708 to detect how the value of the Poisson's ratio remains relatively low and uncertain (variable
 709 in the range 0.03-0.15) during the curing time. Besides, both the elastic modulus of the
 710 material and the UCS show a complex trend over time, described by the trends shown in
 711 Figs. 15 and 16. The values plotted in the figures are the average values between those
 712 measured in the laboratory tests of uniaxial compression for the different maturation times
 713 analyzed: 1 h (i.e. 0.04167 d), 1 day, 7 days, 28 days.



714

715 **Fig. 15. Trend of the tangent elastic modulus for a stress state equal to 50% of the**
 716 **uniaxial compressive strength ($E_{fm,t 50\%}$) as the curing time varies. A): full time**
 717 **interval (0-28 days); B): detail on the first day of curing time (0-1 days).**



718

719 **Fig. 16. Trend of uniaxial compressive strength (UCS) as the curing time varies. A):**
 720 **full time interval (0-28 days); B): detail on the first day of curing time (0-1 days).**
 721 **Legend: dotted line: hypothetical linear trend for the period following the first day of**
 722 **curing.**

723 More specifically, it is possible to hypothesize for the elastic modulus and for the UCS a
 724 strong increase in the first day of curing and a linear trend for the subsequent period, up to
 725 the 28 days of curing. On the first day of curing, it is plausible to hypothesize a parabolic
 726 trend of the curves shown in the previous figures (Figs. 15 and 16). Assuming that the
 727 parabolic curves connect with the straight lines for a curing time equal to 1 day and that
 728 the additional condition that set UCS and $E_{fm,t \ 50\%}$ zero for the time equal to $t = 0$ is valid,
 729 we obtain the following equations that allow us to describe the trends of the elastic
 730 modulus and of the UCS over time, according to the parabola-trapezium scheme.

731 $E_{fm,t \ 50\%}$ in the period 0-1 day:

$$732 \quad E_{fm \ t50\%} = -5,7713 \cdot t^2 + 31,839 \cdot t \quad (29)$$

733 $E_{fm,t \ 50\%}$ in the period 1-28 days:

$$734 \quad E_{fm \ t50\%} = 1,4274 \cdot (t - 1) + 26,0667 \quad (30)$$

735 UCS in the period 0-1 day:

$$736 \quad UCS = -0,3329 \cdot t^2 + 0,74989 \cdot t \quad (31)$$

737 UCS in the period 1-28 days:

$$738 \quad UCS = 0,0089 \cdot (t - 1) + 0,417 \quad (32)$$

739 The identification of the average elastic modulus of the E_{fm} of the filling material during the
740 loading of the segmental lining (with the advancement of the excavation face) is of great
741 importance, as this parameter has a great influence on the stiffness of the k_{sys} system
742 and, therefore, on the load induced on the segmental lining (Fig. 12). Another fundamental
743 parameter is the thickness of the filling material t_{fm} . In order to investigate the influence of
744 these parameters and the Poisson's ratio of the filling material ν_{fm} a parametric analysis
745 was developed, proceeding with the calculation of the stiffness of the k_{sys} system (eq. 9)
746 considering all the following values of the influencing parameters, within the ranges of
747 variability obtained from the laboratory tests described in this paper or from the scientific
748 literature:

- 749 • Tunnel radius R : 2, 3.5 and 5 m;
- 750 • Elastic modulus of the filling material E_{fm} : 15, 30 and 45 MPa;
- 751 • Poisson's ratio of the filling material ν_{fm} : 0.03, 0.09 and 0.15;
- 752 • Thickness of the filling material t_{fm} : 0.12, 0.18 and 0.24 m;
- 753 • Thickness of the segmental lining t_{sl} : 0.3 and 0.4 m.

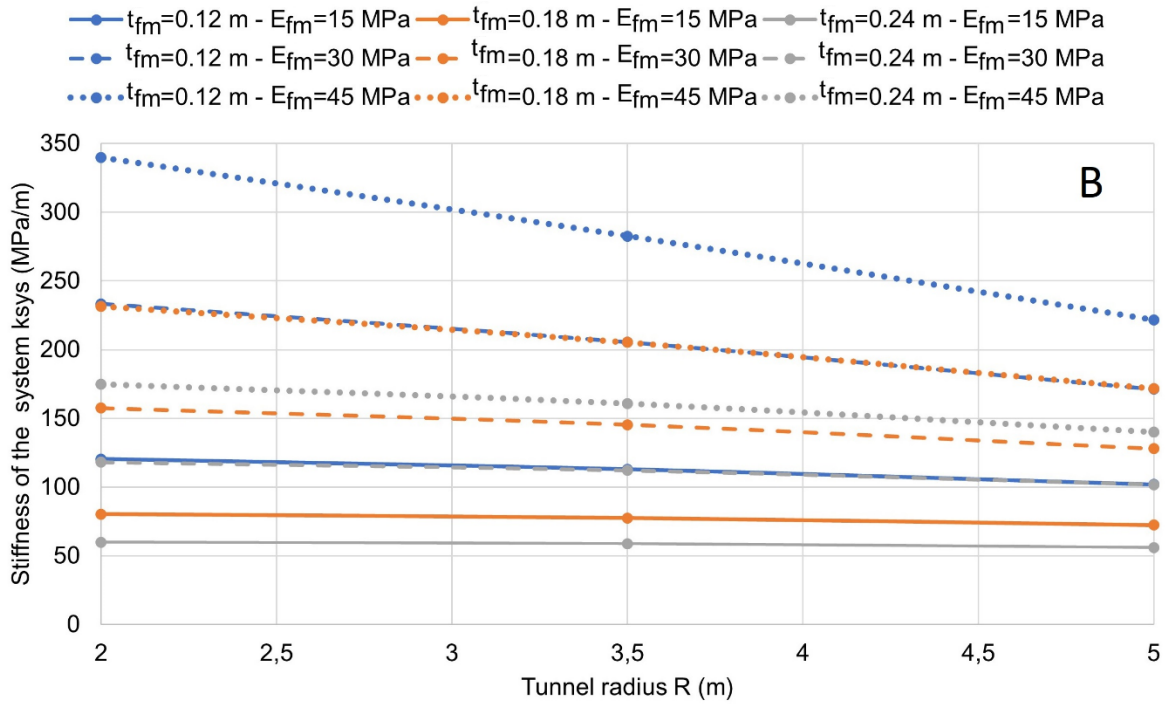
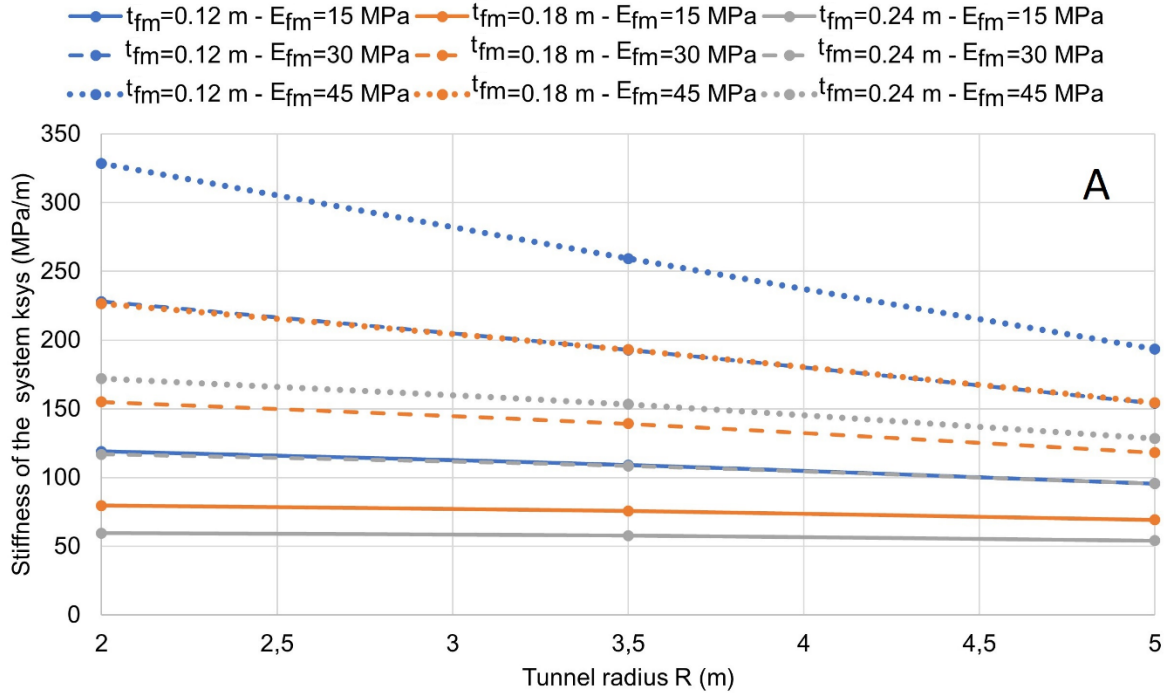
754 For the concrete forming the segmental lining, an elastic modulus E_{sl} of 30,000 MPa and a
755 Poisson's ratio ν_{sl} of 0.15 were considered. On the basis of the 162 analyzes carried out, it
756 was possible to detect how the Poisson's ratio of the filling material has little influence on

757 the stiffness k_{sys} of the system in all the examined cases. For this reason, it is possible to
758 plot the results obtained by neglecting this parameter, assuming it equal to 0.09, the
759 intermediate value of the variability interval detected by laboratory tests (0.03-0.15).

760 Figure 17 (A and B) shows the values of the stiffness of the support system k_{sys} as the
761 radius of the tunnel R varies, for the different values of E_{fm} and t_{fm} considered,
762 respectively for a thickness of the segmental lining of 0.3 and 0.4 m.

763

764



765

766 **Fig. 17. Trend of the stiffness of the support system (segmental lining + filling**
 767 **material annulus) k_{sys} as the tunnel radius R varies, for different values of the elastic**
 768 **modulus E_{fm} and the thickness t_{fm} of the filling material, for the case of segmental**
 769 **lining with a thickness of 0.3 m (A) and 0.4 m (B).**

770 From Fig. 17, it can also be seen that the radius of the tunnel R does not have a
771 detectable influence on the stiffness of the system when the values of E_{fm} are low and
772 those of t_{fm} high. Furthermore, for small tunnel radii, the thickness of the segmental lining
773 does not affect the rigidity of the system. Once the correct value of the stiffness of the k_{sys}
774 system has been identified from the graphs of Fig. 17, it is possible through the
775 convergence-confinement method to obtain an estimate of the load p_{eq} acting on the
776 support system, in order to proceed with the design of the segmental lining and the
777 definition of its thickness. In some cases, it is necessary to limit the load acting on the
778 support system, to avoid an excessive stress state induced in the concrete constituting the
779 segmental lining. This is the case of tunnels at great depths, for which it is possible to
780 intervene appropriately on the thickness of the filling ring (increasing it) and on the
781 mechanical characteristics of the filling material (limiting the elastic modulus) in order to
782 obtain a low k_{sys} stiffness and, therefore, reduced load values on the support system.

783 In order to obtain a low elastic modulus value of the filling material, appropriate additives
784 (e.g. higher retarder dosage) of the two-component material can be used or the excavation
785 machine's advancement speed can be increased (if the technical conditions allow it), in
786 order to load the segmental lining in relatively short times, when still the filling material
787 shows limited mechanical parameters.

788 The design of the segmental lining proceeds with calculation methods that allow to study
789 the development of the bending moments and of the axial forces along the development of
790 the support, starting from the load p_{eq} that can be evaluated with the convergence-
791 confinement method. Among the most common is the method of Einstein and Schwartz
792 (1979), whose stiffness parameters C^* and F^* (the compressibility ratio and flexibility ratio
793 of the support, respectively) were illustrated previously.

794 Also in this case, an extensive parametric analysis has been developed on the parameters
795 that influence the coefficients C^* and F^* in order to verify the effect of the presence of the
796 filling material on the behavior of the support structure.

797 The same values already used in the analysis relating to the stiffness k_{sys} of the system
798 have now been adopted, with the addition of 3 different values of the elastic modulus of
799 the rock mass: 3,162 MPa (relative to a rock mass having GSI = 30), 10,000 MPa (GSI =
800 50) and 31,622 MPa (GSI = 70), where GSI is the Geological Strength Index of the rock
801 mass (Hoek, 1994; Hoek et al., 1995; Hoek and Brown, 1997; Serafim and Pereira, 1983) .

802 The total of the performed analyzes was therefore equal to 486. It was possible to verify
803 that in all cases the effect of the presence of the filling material ring is always less than 1.5
804 ‰ on parameter C^* and always less than 1.5% on parameter F^* . In practical terms,
805 therefore, in the analysis of the behavior of segmental lining through the Einstein and
806 Schwartz method, it is possible to neglect the presence of the filling material ring in the
807 evaluation of bending moments and normal forces acting on segmental lining. The design
808 of the segmental lining can therefore proceed quickly, evaluating the stress state induced
809 in the segmental lining, for different values of its thickness, after having calculated the
810 acting bending moments and the normal forces.

811 **5. Conclusions**

812 Where a two-component grout is used during TBM excavation, the mechanical properties
813 evolve over the time immediately after the injection, just during the loading of the linings. It
814 is, therefore, important to be able to analyze the mechanical behavior of this filling
815 material, in order to evaluate the effects of its presence on the stress state induced in
816 segmental lining. An extensive laboratory program has been performed considering a
817 particular mix-design of a two-component grout. Uniaxial compression tests and

818 oedometer tests, were carried out and allowed to characterize this type of material from a
819 mechanical point of view over time. In particular, the stiffness (elastic modulus, Poisson's
820 ratio, oedometer modulus) and the strength parameters (uniaxial compressive strength,
821 friction angle and cohesion) were evaluated during the curing period of the material. The
822 test campaign showed a certain sensitivity of the material to the preparation methods of
823 the mixture and in particular in the gel formation phase with the mixing of the two
824 components. The material presents a relatively modest density of the material obtained,
825 due to a relatively high water/cement ratio, an elastic-plastic behavior of the material of the
826 "softening" type, and specific compression threshold levels observed in the oedometer
827 test, beyond which water seems to be expelled from the pores and a failure of the solid
828 skeleton previously formed appears, with the consequent appearance of irreversible
829 deformations.

830 Constrained compression has shown the role of a metastable structure in the formed
831 grout, that determines a change of the settlement rate when a transition pressure is
832 reached (meta stable pressure).

833 A detailed analysis of the influence of the filling material on the behavior of segmental
834 lining was carried out using two widespread methods of calculating the tunnel support
835 structures: the convergence-confinement method and the Einstein and Schwartz (1979)
836 method. In particular, for the convergence-confinement method, the stiffness of the
837 support system (segmental lining + filling ring) was evaluated. It has been noted how this
838 stiffness of the support system can significantly affect the value of the load acting on the
839 segmental lining. For the method of Einstein and Schwartz the stiffness coefficients of the
840 lining were obtained, taking into account the presence of the filling material ring. An
841 extensive parametric analysis allowed to identify the effects of the presence of the filling
842 material in the two calculation methods considered. From the study carried out it was

possible to detect how the filling material has a significant effect on the stiffness of the support system in the convergence-confinement method and, therefore, on the load acting on the segmental lining. Instead, it has reduced and even negligible effects on the overall stiffness of the support system in the Einstein and Schwartz method used to carry out a first design of the segmental lining. With this latter calculation method, the analysis can proceed neglecting the presence of the filling material ring and directly obtaining the stresses affecting the segmental lining by adopting the stiffness parameters relating solely to segmental lining.

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Conflict of interests

Authors declare they have no conflict of interest.

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