Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1274

The application of an unconventional construction method to install jet grouting columns for the refurbishment of a wharf in the Genoa harbour.

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ABSTRACT: The east quai of the Eritrea Wharf is made of superposed concrete blocks and over the years has shown significant deformation phenomena (displacements, cracks, settlements on the working platform). The refurbishment and securing intervention was based, among other things, on the underpinning of the structure by jet grouting, drilling through the quay-wall concrete blocks. Jet grouting activities started in December 2022. During the construction of the first 25 columns, adverse effects leading to significant displacements of the structure were observed: horizontally, up to 15 cm at the top. Activities were stopped and a supplement of investigation was carried out and monitoring was activated. An exceptional presence of voids was found just below the foundation, incompatible with the execution of jet grouting, in particular if the double-fluid method was adopted, due to the use of compressed air as auxiliary jetting fluid. It was necessary to conceive a new construction method to minimize the adverse effects of jet grouting on the wharf structure: this was the single-fluid method with simultaneous water pre-cutting. Field trials were carried out to validate this construction method and the jetting parameters. The as built path of drillings was assessed by an inclinometric chain and the achieved diameter measured within the fresh column by using the CheckGrout equipment. Eventually, the foreseen design columns were installed, after filling the voids, using the tested construction method and the jetting parameters selected by the field trials, without significant further displacement on the wharf structure.

KEYWORDS: quay, soil improvement, jet grouting, grouting, field trial, monitoring.

1 INTRODUCTION

The Port of Genoa covers a total area of approximately 7 million square meters, extending continuously for 20 km along a coastal strip protected by breakwaters. It is divided into various territorial areas due to the discontinuities of the coastal land and the presence of the San Benigno promontory, which separates the Sampierdarena basin from the Historical Port area. These areas serve different functions depending on the zone, including commercial, industrial, passenger, oil-related and yachting purposes.

The eastern quay of Eritrea wharf (Figure 1) is located in the western part of the Sampierdarena Basin and is primarily used for commercial activities. It is part of the port infrastructure built in the 1930s during the westward expansion of the Port of Genoa. The quay is 440 meters long, constructed from superposed concrete blocks, with an average foundation level 11 meters below sea level (Figure 2). The rear side is filled with quarry stone and heterogeneous anthropogenic fill to reduce pressure. Over the years, the quay has been subject to deformation phenomena, resulting in the outward displacement of the quay-wall edge and in the corresponding settlements of the yard area behind the quay-wall structure.

Various investigations and interventions have been carried out over time to restore and secure the structure. These included resurfacing the quay yard, underpinning works, restoring the downhill slope profile, and implementing topographic monitoring integrated with continuous inclinometer surveys. These activities revealed a trend of the quay rotating outward, causing the edge to shift. It was determined that the structure was in a precarious static condition, evidenced by signs of failure such as decimetric depressions behind the capping beam, bulging in the central area, significant cracking, and undermining phenomena with the displacement of the quay's cyclopean foundation blocks.

The Technical and Financial Feasibility Project (TFFP) proposed a series of quay consolidation interventions to stop the ongoing degradation and increase the static capacity of the mooring quay, enabling future seabed deepening and increasing mooring capacity through the upgrade of the bollard system:

Figure 1. Aerial view of the Eritrea wharf (eastern quai on the right).

- underpinning of the quay using reinforced jet-grouting columns arranged in three rows;
- installation of an anchoring system consisting of strand or bar tie rods;
- demolition and reconstruction of the quay superstructure, including replacement of quay fixtures and existing 100-ton bollards with new 150-ton bollards.

Based on the TFFP, in 2020 the Western Ligurian Sea Port Authority launched a Design & Build tender for the refurbishment and the securing of the quay.

2 CONCISE GEOTECHNICAL BACKGROUND

The soil profile on site can be summarized as follows.
Beneath the quay-wall made of superposed concrete blocks:

- From -11.50/-12.50 to -13.50/-14.50 m a.s.l.: *foundation layer*, composed of stone blocks, cobbles, and gravel, sometimes in a silty-sandy matrix; during the construction phase, widespread voids were found immediately beneath the quay foundation.

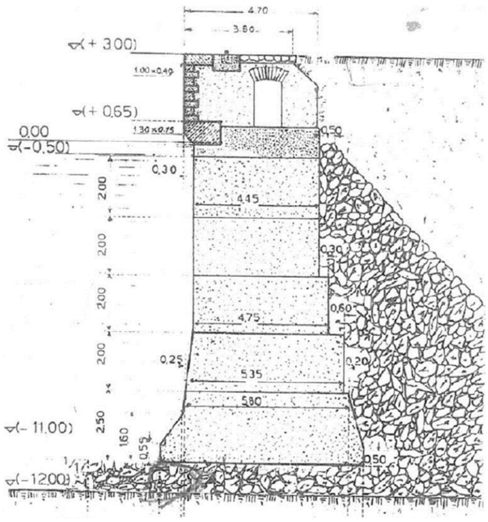

Figure 2. Typical historical cross section of the quay.

- From -13.50/-14.50 to -30.00 m a.s.l. (maximum investigation depth): *seabed deposit*, consisting of a generally uniform soil composed of silty sand with thin gravel lenses, always in an abundant silty sand matrix.

Behind the quay-wall:

- From +2.60/+2.85 m a.s.l. (quay surface) to -11.50/-13.50 m a.s.l.: *fill material*, predominantly granular and heterogeneous (locally including brick fragments), composed of gravel and cobbles in a silty-sandy matrix, ranging from loose to moderately compact, with variable thickness.
- From -11.50/-13.50 to -30.00 m a.s.l.: *seabed deposit*, as previously described.

The average geotechnical characteristics of these three soil layers, derived from in situ tests (SPT) and laboratory tests (CIU triaxial and direct shear tests), are provided in Table 1.

Table 1. Average geotechnical parameters of the in-situ soils.

Soil Layer	γ_{sat} (kN/m^3)	ϕ' (°)	c' (kPa)	E (MPa)
Fill	19	34	2	30
Foundation Layer	22	40	-	-
Seabed Deposit	19	34	2	80

3 THE REFURBISHMENT AND SECURING PROJECT

The executive-level design was developed, as part of the Design & Build contract, by the Banchina Eritrea Joint Venture, composed of Dott. Carlo Agnese S.p.A., I.CO.P. S.p.A., and Injectosond S.r.l., which was awarded the contract. The design was carried out in accordance with the requirements of the Western Ligurian Sea Port Authority, which also included the future deepening of the port seabed in the water area between Ethiopia wharf and Eritrea wharf.

The design was based on the results of an extensive supplementary investigation campaign, which included:

- topo-bathymetric surveys, carried out on land using topographic and 3D laser scanning technologies, and at sea using multibeam surveys and underwater video inspections carried out by means of Remotely Operated Vehicles (ROVs);
- geotechnical investigation by means of new additional cored boreholes carried out in multiple phases.

This campaign aimed to provide the most up-to-date and comprehensive picture of the current state and to enable specialized technical studies focused on analyzing geotechnical models and the structural equilibrium of the quay. The objective was to identify the main causes of instability and determine the appropriate technical design solutions to implement.

Executive-level design included the following aspects (Figure 3):

- Construction of an underpinning system beneath the existing quay by the installation of jet-grouting columns, following drilling through the existing quay-wall blocks. This was intended to create a continuous structural element aligned with the current facing, including reinforcement with steel bar to counteract bending moments.
- Construction of a continuous micropile wall along the most critical section at the foot of the quay, to improve the containment of jet grouting during its construction and to protect the injected mass from erosion caused by propellers and bow thrusters.
- Demolition and reconstruction of the quay superstructure, over a width of 17.00 meters.
- Construction of the new reinforced concrete superstructure supported by three staggered rows of piles, each 800 mm in diameter and 22.00 meters long, replacing the inclined tie rod system originally foreseen in the TFFP.
- Replacement of the existing 100-ton bollards with new 150-ton nodular cast iron bollards.

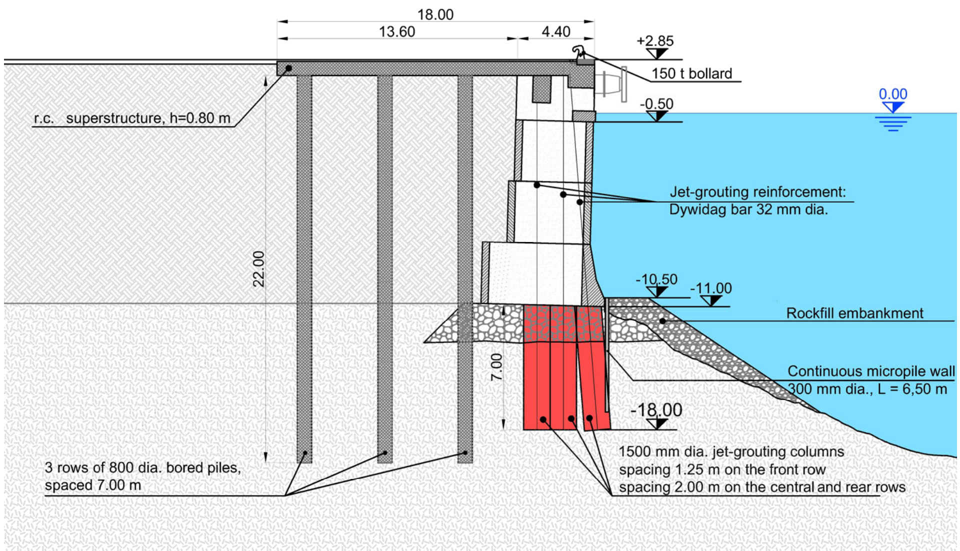

Figure 3. Executive-level design: typical cross section of the quay.

The approved and validated solution assigns the load-bearing function originally attributed to the tie rods in the TFFP to the structural elements consolidating the yard behind the quay. This provides significant advantages in terms of synergy between the quay and yard consolidation works, improved usability and durability of the infrastructure, and optimization of site safety levels.

4 START OF THE REFURBISHMENT AND SECURING WORKS AND ADVERSE EFFECTS ON THE QUAY

The refurbishment and securing works began in December 2022 with the execution of the first jet-grouting columns for the quay-wall's underpinning. According to the executive-level design, the jet-grouting columns were planned with a diameter of 1.50 m and a length of 7 meters, from -11.00 to -18.00 meters a.s.l. (i.e., at a depth between 13.60/13.85 and 20.60/20.85 m below the working platform). They were installed by using the double-fluid construction method,

Following the execution of the first 25 columns, significant adverse effects were observed on the quay structure, including substantial horizontal displacement of the quay head seawards, up to 15 cm. At that point, works were suspended, and an in-depth supplementary investigation and monitoring campaign was launched, consisting of:

- direct inspection of the quay front by underwater ROV surveys;
- indirect investigation using down-hole geoelectric instrumentation to create a mapping of voids in the quay-wall's foundation;
- topographic surveys of the quay-wall top edge;
- continuous inclinometer monitoring of the quay-wall using a series of single inclinometers installed 1.5 m below the quai platform, with the relevant dataloggers;
- borehole video inspections;
- an additional core sampling campaign.

All investigations consistently highlighted the widespread presence of voids, in plan view, within the foundation layer and within the soil immediately beneath the quay-wall foundation.

The presence of such voids was considered incompatible with the execution of jet-grouting treatment, particularly due to the high pressure and flow rate of the fluids used in the double-fluid injection technique. It is likely that the adverse effects observed were also influenced by the combined effect of the use of compressed air – as auxiliary fluid in the double-fluid jet-grouting – and the pre-existing voids, which together represent the most probable reason for the detected deformations.

Following various studies and evaluations carried out jointly with the Site Supervision Team and the Design Team, an operational procedure was proposed. Its primary aim was to ensure the safety of the quay and, at the same time, to restore the soil beneath the quay toe to a condition suitable for the successful execution of the underpinning treatment originally planned in the project.

5 INJECTIONS FOR VOID FILLING UNDER THE QUAY-WALL FOUNDATION

As a propaedeutic activity for the resumption of the soil improvement works for the quay-wall underpinning, the filling of voids present within the layer below the quay-wall footing was designed and carried out. The void-filling injections were performed through vertical boreholes drilled through the concrete blocks of the quay-wall along their entire height, reaching into the foundation base and underlying soil to a depth of 1.50 m.

The injection borehole grid matched the layout of the planned jet-grouting columns, allowing the same predrilled holes in the concrete blocks to be used for both activities. The injection holes were arranged in three parallel rows, with each row consisting of alternating primary and secondary holes. In each row, the primary holes were injected first, followed by the secondary ones. Moreover, the two outer rows were injected first, and the central row – confined by the previously injected adjacent rows – was completed last.

The predrilling through the quay blocks was carried out using rotary-percussion drilling with a 180 mm diameter bit, stopping immediately upon reaching the base of the deepest quay-wall block. Drilling then continued by rotary drilling for an additional 1.50 m, using a temporary casing, which also served as the conduit for the subsequent injection.

The injection process involved pumping through the temporary casing a cement-based mortar, prepared on-site using a dry premix based on selected aggregates in a suitably recomposed particle size curve from 0 to 3 mm, cement binder, plasticizer admixture to improve workability and to promote pumping. The main characteristics of the mortar were:

- fresh density = 2200 kg/m^3
- no-shock spreading (EN 1015-3 mod.) = 190-230 mm
- UCS at 28 days ≥ 20 MPa.

The mortar was injected bottom-up in steps: 50 cm height increments in the soil beneath the foundation, and 1.50 m increments within the quay-wall structure. After each step was completed, the casing was raised accordingly.

Among the different mortar formulations developed during the mix design phase – each one with a different level of fluidity – the injection began with the most fluid mixture, gradually transitioning to higher-viscosity mixtures if void saturation was not achieved with the previous mix.

6 SELECTION OF THE JET GROUTING CONSTRUCTION METHOD

In light of the adverse effects observed on the quay-wall during the execution of the first 25 jet grouting columns – effects likely influenced, at least in part, by the use of compressed air as auxiliary fluid in the double-fluid jet construction method – it was considered inappropriate to continue using the double-fluid method.

On the other hand, based on prior experience and available literature (e.g., AGI, 2012), the simpler single-fluid method was considered incapable of producing the 1.50 m diameter columns required by the project. It was therefore necessary to adopt a different construction methodology that could minimize or avoid negative impacts on the structure – partly thanks to the absence of compressed air – while still achieving columns that met the design specifications.

A relatively unconventional method, though already successfully used in similar situations, was selected for trial in a test field: this is the single-fluid method with simultaneous water pre-cutting.

This approach, derived from the triple-fluid method (but without using compressed air to amplify the effect of high-pressure water), has the advantage of eliminating compressed air entirely. Additionally, it allows column execution in a single pass (upward withdrawal), unlike the traditional water pre-cutting in advance of jetting, which requires two sequential passes. It is also more efficient than the more conventional single-fluid with pre-cutting method, as the jet grouting is performed in soil that has just been disrupted and is still in a fluid state, thus avoiding the sedimentation that typically occurs between the two passes of the more conventional single-fluid

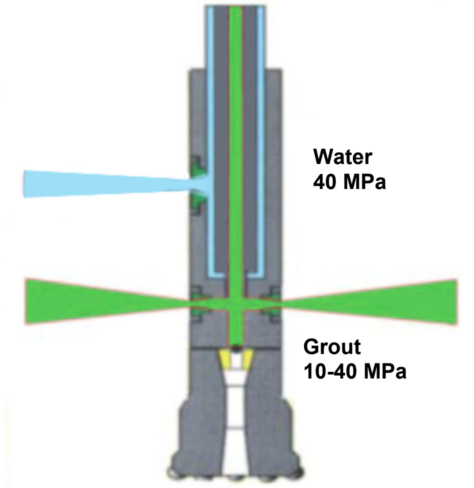

Figure 4. Schematic of the jet grouting monitor for single-fluid with simultaneous water pre-cutting.

with pre-cutting. In fact, in those cases, the second high-pressure injection must first re-fluidify the previously pre-cut material (already sedimented) before it can develop its additional effect of soil disruption and mixing with the binder.

The single-fluid method with simultaneous water pre-cutting involves the simultaneous ejection of two high-speed fluids (water and cement grout), requiring two high-pressure pumps. A dual-channel drill string is used – similar to that in the double-fluid method – but capable of withstanding high pressures in both the central and annular conduits. At the base of the string, a special monitor (Figure 4) is installed, which positions the water nozzle(s) and the grout nozzle(s) 1 meter apart.

The effect of this jet grouting construction method is to initially disrupt the soil with the high-velocity water jet, followed by a second disruption phase with the high-velocity cement grout, expanding the influence radius to the desired extent. Simultaneously, this second phase achieves mixing of the disaggregated soil with the binder.

7 FIELD TRIALS FOR SINGLE-FLUID JET GROUTING WITH SIMULTANEOUS PRE-CUTTING

In parallel with the execution of the void-filling injections beneath the toe of the quay-wall, experimental field trials were designed and implemented with the aim of validating the new operating methodology in the on-site soil conditions. The objective was to verify whether the selected method could produce columns with the required diameter, physical-mechanical properties, continuity, and homogeneity as specified in the project.

To this end, four trial groups of columns were planned, each one with a different set of jet-grouting construction parameters. The geometry of the trial field is shown in Figure 5.

During the execution of all the trial columns, drilling and jet-grouting parameters were automatically measured, displayed and recorded using LUTZ LT3 instrumentation installed on the drilling rig: during drilling, penetration and rotation speed, thrust torque and drilling fluid pressure; during jetting: withdrawal and rotation speed, water and grout pressure and flow rate.

For each borehole used to construct the test columns, deviation from the theoretical axis was measured to determine their actual path, and thus the true spatial positions of the test columns at various depths. This measurement was essential for the correct interpretation of both in-progress and final quality controls. Deviation measurements were taken at the end of each borehole using an inclinometer chain (SAASCAN, Measurand Company – Canada), which was inserted into the drill string with appropriate orientation. The actual borehole paths were then incorporated into a 3D model of the trial field, creating a snapshot of the true geometry of the jet-grouted columns at any given depth.

The construction parameters used to jet-grout the test columns are summarized in Table 2.

Table 2. Construction parameters to install the test columns

	Group A	Group B	Group C	Group D
W/C ratio of the cement grout	0.8	0.8	0.8	0.67
No. and diameter (mm) of the grout nozzles	1 ∅ 5.5	1 ∅ 5.0	2 ∅ 4.5	2 ∅ 4.0
No. and diameter (mm) of the water nozzles	1 ∅ 5.0	1 ∅ 5.0	2 ∅ 5.0	2 ∅ 5.0
Grout pressure (bar)	400	400	100	100
Grout flow-rate (l/min)	305	250	205	155
Water pressure (bar)	400	400	400	400
Water flow-rate (l/min)	320	320	320	320
Withdrawal speed (cm/min)	36.5	28.0	25.5	19.0
Rotation speed (giri/min)	6÷12	4÷8	4÷8	3÷6
Specific energy (MJ/m)	68.5	81.5	58	75
Specific grout consumption (l/m)	840	900	800	825
Specific cement consumption (kg/m)	750	805	715	735

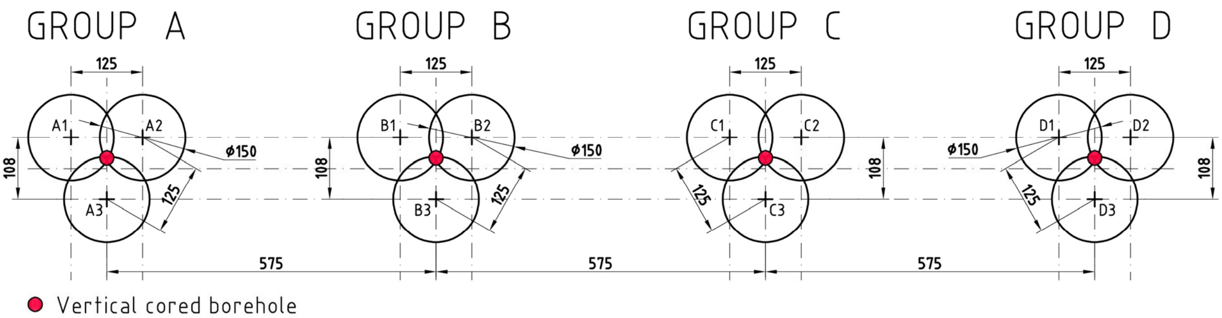

Figure 5. Layout of the trial field.

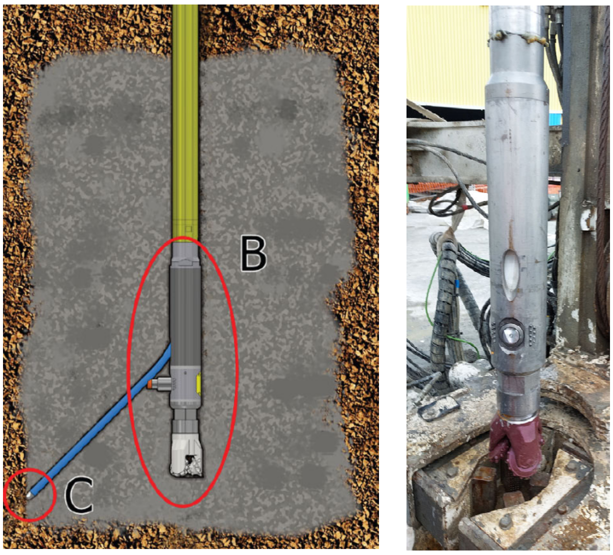

Figure 6. Left: operating scheme of the CHECKGROUT device. Right: special measuring monitor.

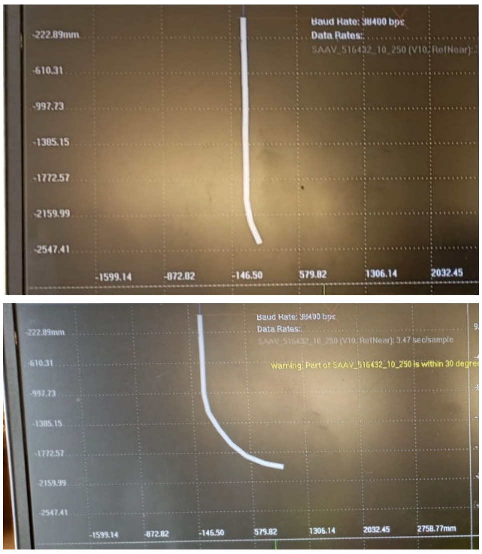

Figure 7. Example of screenshots of the inclinometer chain path during the test: above just before exiting the monitor, and below once reached the untreated soil at the column's boundary.

The achieved diameter of each test column was measured immediately after execution within the fresh column, before cement setting began. A specific measuring device called CHECKGROUT (CRM Company) was used for this purpose. The operating scheme of the test is illustrated in Figure 6, left.

After completing a column, the jet-grouting monitor was replaced with a special measuring monitor (Figure 6, right). A flexible tube called Diaflex was inserted into the central conduit of the drill string. Then, the inclinometer chain was inserted into the Diaflex tube. This assembly was pushed through the special measuring monitor until it entered the still-fluid material of the column and reached the untreated soil at the outer boundary. The path of the inclinometer chain was displayed in real-time on a PC connected via WiFi to the data acquisition unit. Figure 7 shows example screenshots of the inclinometer chain path: above, just before exiting the monitor; and below, after reaching the untreated soil at the column's boundary.

Radius measurements were taken at two depths (15.00 and 18.00m below the working platform) and repeated in four directions. The column diameter in each direction was calculated as the sum of two opposing radii. When four directional measurements (spaced at 90°) were available, the reference diameter was taken as the average of the two orthogonal diameters. The most reliable diameter measurements were obtained from column no. 1 in each trial set, since these were installed first, in undisturbed soil and remained isolated. Measurements from columns 2 and 3 were affected by the presence of the previously constructed hardened columns – evidenced by cases where the inclinometer encountered these pre-existing elements – resulting in underestimated diameters for columns 2 and 3. A summary of the results achieved at the depth of 15m is presented in Table 3.

For Group A, the diameter values appear to be underestimated, as column 1 (the only isolated one) was not measured due to an issue with the measuring equipment.

As a cross-check on column diameter, a vertical cored borehole was drilled at the triple intersection point of each column group (see Figure 5). Additionally, for Group A – where the achieved diameter appeared smaller – a slanted cored borehole was performed across columns A1 (not measured with CHECKGROUT) and A2. In all cored boreholes, the actual path was measured using the SAASCAN inclinometer chain to determine the true location of the borehole and its alignment with the columns.

All the control cored boreholes confirmed the continuity and homogeneity of the columns. Each borehole passed through consolidated material along the entire design height of the columns. The coring recovery ratio was 100%, with RQD values ranging from 90% to 100% in vertical cores and 84% to 94% in the slanted core.

The vertical cored boreholes through the triple intersections also confirmed the interpenetration between adjacent columns in all four groups and verified that column radii met or exceeded the design values.

Table 3. Summary of the results achieved with the diameter measurements by CHECKGROUT at the depth of 15 m from the working platform.

Column	Measure 1		Measure 3		Diameter 1	Measure 2		Measure 4		Diameter 2	Av. Diameter
	(°)	(m)	(°)	(m)	(m)	(°)	(m)	(°)	(m)	(m)	(m)
A1	test not performed										
A2	45°	0.542	225°	0.625	1.167	135°	0536	315°	0.802	1.388	1.388
A3	60°	0.882	240°	0.365	1.247	150°	0.537	330°	0.693	1.230	-
B1	0°	0.790	180°	0.836	1.626	90°	0.815	270°	0.996	1.811	1.718
B2	0°	0.389	180°	0.819	1.208	90°	0.574	270°	0.790	1.364	-
B3	0°	0.823	180°	0.549	1.372	90°	0.393	270°	0.806	1.199	-
C1	0°	0.750	180°	0.855	1.605	90°	0.889	270°	0.650	1.539	1.572
C2	45°	0.870	225°	0.785	1.655	135°	0.769	315°	0.824	1.593	-
C3	90°	0.526	270°	0.862	1.388	test not performed					-
D1	0°	0.860	180°	0.856	1.716	90°	0.939	270°	0.742	1.681	1.698
D2	45°	0.485	225°	0.753	1.238	135°	0.481	315°	0.747	1.228	-
D3	60°	0.800	240°	0.715	1.515	150°	0.216	330°	0.787	1.003	-

Selected cylindrical core samples were tested in the laboratory. First, density was measured, followed by unconfined compressive strength and elastic modulus tests. Additionally, indirect tensile strength tests were performed at the request of the Engineer. A summary of the laboratory test results in terms of physical and mechanical characteristics is presented in Table 4.

Table 4. Summary of the physical and mechanical characteristics of the improved soil: average values.

Group	γ_{sat} (g/cm^3)	UCS (MPa)	E (MPa)	ITS (MPa)
A	1.559	5.8	978	0.683
B	1.626	6.7	1050	0.255
C	1.633	4.2	968	0.620
D	1.546	2.9	738	0.330

γ_{sat} = Density of the saturated sample
UCS = Unconfined Compressive Strength
E = Elastic Modulus
ITS = Indirect Tensile Strength (Brasilian Test)

Based on both in-situ and laboratory test results, no test group showed a decisive advantage over the others. The selected jet-grouting parameter set for execution of the design columns was that of Set C, which involved:

- a cement grout with a water/cement ratio (w/c) = 0.8;
- high-pressure water injection at 40 MPa;
- medium-pressure cement grout injection at 10 MPa;
- specific disruption energy of 58 MJ/m;
- specific cement content of 715 kg/m.

8 RESUMPTION OF THE SITE ACTIVITIES

The design columns were installed – following the filling and saturation of voids in the foundation layer – using the tested methodology and the set of construction parameters selected during the trial field phase.

The work proceeded at a sustained pace, with no or minimal additional displacements induced on the quay-wall. Once all planned operations were completed, including ancillary works such as the installation of new 150-ton bollards and paving work, the quay was subjected to static testing, which yielded positive results, and commissioning. The quay was then returned to the terminal operator GMT, which resumed its activities at the usual production pace.

9 CONCLUSIONS

The installation of jet-grouting columns beneath or in proximity to existing structures or buried utilities may sometimes induce adverse effects in terms of displacements and/or overstresses on them. Evidence from multiple case studies indicates that such effects are more likely when compressed air is used as an auxiliary fluid to enhance soil disaggregation by means of high-velocity cement grout or water jets (i.e., in double-fluid and triple-fluid systems). Conversely, the single-fluid system – being the only one that does not rely on compressed air – generally cannot achieve column diameters comparable to those obtained with double-fluid or triple-fluid methods, even when preliminary water pre-cutting is employed, which requires two separate stroke/withdrawal phases.

To increase the achievable column diameter using the single-fluid system, simultaneous water pre-cutting represents a viable and effective solution. This technique derives from the triple-fluid method but omits the use of compressed air, thus removing the air-induced enhancement of the water jet. As a result, besides minimizing adverse effects on adjacent structures through the elimination of compressed air, this method enables the construction of significantly larger-diameter columns with a single stroke. Although these diameters remain in the lower range of those typical of a double-fluid system, they are significantly greater than those obtainable with conventional single-fluid jet grouting, with or without preliminary water pre-cutting, depending on soil conditions.

This operational approach is more efficient than the single-fluid system with conventional preliminary water pre-cutting. In the simultaneous configuration, the grout jet is applied to soil that has just been disaggregated and remains in a fluid or semi-fluid state, preventing sedimentation. In contrast, when pre-cutting and grouting occur in separate phases, sedimentation inevitably occurs between the two strokes. As a result, the grout jet must first overcome the inertia of re-fluidizing the settled material before it can exert its intended effect.

The case study presented in this paper – related to the refurbishment and securing works along the east quay of the Eritrea wharf in the Port of Genoa – provides a significant example of successful application of the single-fluid jet-grouting technique with simultaneous water pre-cutting. This method enabled the construction of all the 1.50-m-diameter columns required for the quay-wall underpinning, while minimizing adverse effects on the existing structure. Such effects had instead become clearly apparent during the initial phase of the project, following the execution of only 25 jet-grouted columns using the double-fluid system.

10 ACKNOWLEDGEMENTS

The authors would like to thank the Port System Authority of the Western Ligurian Sea, in particular Ing. Francesca Arena (Project Manager) and Ing. Fabrizio Mansueto (Engineer), for their valuable technical contribution and for authorizing the publication of this article.

Thanks are also extended to the technical teams of Dott. Carlo Agnese S.p.A., I.CO.P. S.p.A., and Injectosond S.r.l., for their continuous support and dedicated efforts. Special thanks go to CRM, particularly Dr. Valerio Salvi and Dr. Marco Angelici, for their valuable collaboration during on-site measurements using the CHECKGROUT and SAASCAN instruments.

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