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Optimization of GFRP reinforcement in precast segments for metro tunnel lining

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ABSTRACT

The possibility of replacing the traditional steel reinforcement with glass fiber reinforced polymer (GFRP) cages in precast concrete tunnel segmental lining has been shown by the authors in previous papers. The use of GFRP rebars as structural reinforcement in precast tunnel segments, allows several advantages in terms of structural durability or in cases of temporary lining that will have to be demolished later. Furthermore, this reinforcement type can be a suitable solution to create dielectric joints, ensuring the interruption of possible stray currents, which often lead to corrosion problems. Nevertheless, this peculiar application requires curvilinear shape of the reinforcement, and then different production process and rebar geometries. In the present work, a suggestion for the optimization of the GFRP reinforcement for tunnel segment is given. Four different GFRP cage typologies are analysed and applied as a reinforcement in full-scale tunnel segments. Both bending and point load tests are developed and the structural performances of the specimens are compared and discussed. Finally, the best solution, in terms of cost-benefit analysis is proposed.

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1. Introduction

The application of glass fiber reinforced polymer (GFRP) reinforcement in concrete structures, has encountering an increasing interest worldwide, for several applications in civil engineering. The main advantages of the GFRP material are the lightweight, the high tensile strength and non-corrosive properties [21,9]. Furthermore, it is worth highlighting that GFRP material is nonconductive for electricity and non-magnetic. Nevertheless, the GFRP reinforcement may suffer static fatigue when subjected to high-level long-term tensile stresses [4] and the structural performances can be affected by the low value of the Young's modulus and by the poor bond behaviour [12,22]. Finally, the material durability can be improved and controlled through a suitable choice of the composite constituents [16,9]. The application of GFRP reinforcement as a substitute of the traditional steel rebars in the precast segments of tunnel lining, could represent a suitable solution to the challenges of underground construction in terms of maintenance cost and durability. In underground structures mechanically excavated by means of tunnel boring machine (TBM), the lining is composed of precast elements, placed by the TBM during the

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excavation process and used as reaction elements during the advancing phase [14]. This peculiar application appears suitable for the adoption of GFRP reinforcement for different reasons. First of all, it allows to overcome the problem of the structural durability, often jeopardized by the steel reinforcement corrosion. Furthermore, the use of this non-corrosive reinforcement in tunnel segments allows reducing the concrete cover that is usually a weak point for this kind of structure, since it can crash during handling or due to TBM thrusts. Due to the further property of being chemical inert, the GFRP reinforcement provides a low conductivity against the stray currents and is adequate for creating dielectric joints in tunnels [8,20]. Indeed, a tunnel ring made with precast GFRP reinforced segments breaks the electrostatic continuity of the ordinary cages (reinforced with steel) and guarantees an effective remedy to the problem of stray currents and corrosion resulting therefrom.

Finally, the GFRP reinforcement solution appears very interesting for the parts of the tunnel that have to be eventually removed (niches, vent channels, openings and stations), due to the easiness of demolition and disposal.

The mechanical properties of straight GFRP bars, are nowadays well known [17,5,18,6,3,15,2,11], and codified [13,1,10]). Nevertheless, for application in tunneling lining, the GFRP reinforcement should present a curvilinear configuration. At this aim, both the







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geometry and the manufacturing process are different from the ones of the straight rebars.

The results of experimental full-scale tests on precast concrete segments reinforced with GFRP bars [19,20,8] showed the effectiveness of this solution, able to satisfy the requested capacity. Main aim of this paper is the optimization of the GFRP reinforcement geometry in concrete precast segments, for improving the crack control, in the perspective of a costbenefit analysis.

Four GFRP reinforcement cage prototypes are studied and produced. Both flexural and point-load tests, simulating the bending behavior and the TBM thrust phase, are developed on full-scale segments, with typical geometries of metro linings. The obtained results are discussed and compared with the ones obtained on traditionally steel reinforced elements. Finally, the best solution in terms of cost-benefit analysis is proposed.

2. Experimental program

The experimental program is developed with reference to structural elements belonging to a typical metro tunnel.

Full scale segments were made with different typologies of reinforcement (four different types of GFRP reinforcement plus a reference segment with traditional steel reinforcement). For every reinforcement typology, two segments were cast at the Laboratory of Material and Structures of the University of Rome Tor Vergata.



Fig. 1. Metro tunnel geometry.



Fig. 2. Segment geometry.

One segment was subjected to bending whereas the other was tested simulating the TBM thrust. As a consequence, one segment was tested for every reinforcement typology and every testing setup, for a total of ten full-scale tests. 2.1. Segment geometry

The analyzed metro tunnel has an internal diameter of 5800 mm and a lining thickness equal to 300 mm (Fig. 1). The



Fig. 3. a) Traditional GFRP longitudinal reinforcement; b) New GFRP "closed-rings" reinforcement.



Fig. 4. Reinforcement details: a) SR; b) GFRP-RR; c) GFRP-LR; and d) GFRP-WR.

Table 1

Design and experimental values of the material properties for the interaction diagrams definition.

Material	Property	Unit	
Concrete Steel	Compressive strength Tensile strength (yielding) Tensile modulus	MPa MPa GPa	50 510 210
GFRP	E-CR fiber glass Density Conductivity Pristine fiber tensile strength Young's modulus Elongation at break CTE, 23–300 °C	g/cm ³ W/m·K MPa GPa % ×10 ⁻⁶ °C ⁻¹	2.62 1.22 3750 81 4.9 6
	Impregnated strand Tensile strength Tensile modulus	MPa GPa	2200–2600 81
	Unidirectional composite Tensile strength Tensile modulus Poisson's ratio Fiber volume fraction	MPa GPa - %	1200 48 0.33 60
	Vinyl ester matrix Density Tensile strength Young's modulus Poisson's ratio CTE	g/cm ³ MPa GPa - ×10 ⁻⁶ °C ⁻¹	1.15–1.35 73–81 3.0–3.5 0.36–0.39 50–75

lining precast concrete segment has a width of 1420 mm, as shown in Fig. 2.

2.2. Curvilinear GFRP reinforcement

For typical applications, the GFRP reinforcement consists of straight bars manufactured by means of well-established pultrusion industrial technology. In underground tunnels, reinforcements with curvilinear configuration is required and the pultrusion process cannot be adopted. At the aim, a modified pultrusion process named "pull-forming", has been developed, able to produce curvilinear bars with a constant and large curvature radius. The behavior of precast concrete segments with GFRP reinforcement made with the pull-forming technology is reported in Caratelli et al. [8] and Spagnuolo et al. [20]. The reinforcement cage consisted of longitudinal coupled curvilinear bars (intrados/extrados) closed to the edges by means of "C" shape pieces (Fig. 3a). In this case, the assembling of the cage can become difficult and expensive. In fact, since a thermosetting resin is used as matrix for the GFRP composite, from technological point of view, the curvilinear bars cannot be bended in plastic way and, furthermore, cannot be welded. Additionally, many ligatures by hands would be necessary, taking much time for their manufacturing. In this research, a new manufacturing technology has been developed, able to realize cages composed of GFRP "closed-rings", as shown in Fig. 3b.



a)



b)

Fig. 5. Bending test: a) front view; b) lateral view.



Fig. 6. Position of the precast concrete segment on the supports.

In the light of this new technological process, different GFRP cages prototypes have been designed, considering technical, commercial and technological feasibility.

These aspects are the basis of engineering industrialization, leading to create a technological system that can be standardized, in order to maximize the compatibility, interoperability, safety, repeatability, and quality.

2.3. Reinforcement cages: steel reference and GFRP prototypes

Starting from a traditional steel reinforcement (SR) cage, used as reference, four different GFRP reinforcement cages were designed and realized.

The reference steel reinforcement (SR) consists of 12 Ø12 longitudinal rebars both intrados and extrados surface, closed to the ends by means of two "C" shaped stirrup. The transverse reinforcement is made with straight rebars closed by means of different "C" shaped stirrups. Minimum concrete cover is equal to 50 mm (Fig. 4a).

The first solution for GFRP type concerns the closed "Ring Reinforcement" (GFRP-RR), as mentioned above. The main longitudinal GFRP reinforcement consists of 12 closed-rings equals to 12 mm equivalent diameter. As well as for the longitudinal reinforcement, transverse reinforcement (stirrups) is made of "closed-rings" with cross-sectional area equal to an equivalent round bar of 8 mm in diameter (Fig. 4b).



Fig. 7. Bending test set up: a) Frame system for distributing the load along the midspan width; and b) Instrumentation - wire transducers (WT) and LVDTs.



Fig. 8. Point load test set up: a) intrados view; b) extrados view; c) loading steel pad view; d) LVDTs between the steel pads (intrados surface); e) lateral view; and f) potentiometer detail.

The final cage is made by means of interlinked closed-rings (longitudinal/transversal): it is an innovative solution that would facilitate the assembly operation.

The second GFRP cage solution, is a "Lattice Reinforcement" (GFRP-LR). It is a combination of reinforcement curvilinear bars, which are interlinked by means of lattice structures (Fig. 4c). In the main direction, the closed-rings are replaced with bended rebars (staggered between them) and lattice reinforcements. The GFRP reinforcement cage consists of 9 Ø16 and 8 Ø16 curvilinear bars in intrados and extrados surface respectively, with minimum concrete cover of 50 mm. Furthermore, 12Ø8 lattice reinforcements are interspersed with the curvilinear bars (Fig. 4c – Lattice #1 to # 12).

In the transverse direction, besides the stirrups (14 bars with diameter of 8 mm), the lattice structures (characterized by the same stirrups diameter) are placed (Fig. 4c – #19). Due to the above described elements, a complex and rigid system has been achieved.

The third GFRP solution is a "Wirenet Reinforcement" (GFRP-WR). In this case a wirenet is placed in the extrados side with a mesh of 150×140 mm and diameter of 13 and 8 mm respectively. The wirenet in intrados has a mesh of 140×140 mm and diameter of 13 and 8 mm respectively. "C" shaped stirrups close the edges



Fig. 9. Point load test: loading-unloading cycles.

with equivalent bars of diameter from 8 to 14 mm and 110 Ø8 "pins" have the task of confining and spacing the two wirenets. Fig. 4d shows GFRP-WR reinforcement details.

Finally, a further solution was considered, named GFRP-RR(+B) obtained from the GFRP-RR prototype, through a sand coating of the closed ring reinforced reinforcement (Fig. 12), with the aim of improving the bond properties.

2.4. Materials

All GFRP reinforcement cages were made with E-CR glass and vinyl ester resin.

The properties of the GFRP reinforcement constituents are illustrated in Table 1. In the same table the concrete average strength, measured on six $150 \times 150 \times 150$ mm cubes, and the yielding and ultimate stress of the steel rebars for the SR solution, are summarized.

3. Testing set up

Two different testing set-up were carried out:

- a bending test aiming to represent the transient load conditions;
- a point load test aiming to simulate the TBM thrust.
- 3.1. Bending test set up

The test aims verifying the segment performance under prevalent bending moment action, and then it is also representative of the provisional loading stages as demoulding, storage and handling.

The typical testing set-up [7,8] is shown in Fig. 5. The segment was placed on roller supports with a span (L) equal to 2.0 m (Fig. 6). A frame system was used to distribute the load along the transverse direction (Fig. 7a).

Three wire transducers (WT) were used for measuring the vertical displacement, while two linear variable differential transformer (LVDTs) were applied in order to measure the crack opening. Both wires and LVDTs were placed on intrados surface (Fig. 7b).



Fig. 10. Bending test results: load versus displacement comparison.

Table 2

Failure modes of the segments.

Reinforcement	Failure mode	P _{crack} (kN)	w _{crack} (mm)	P _{max} (kN)	$\delta_{max}^{(a)}$ (mm)	$rac{\delta_y}{(mm)}$	δ_1 (mm)	μ (-)
SR	Rebars rupture ^(b)	145.0	0.10	471.7	56.7	7.7	-	7.4
GFRP-RR	Rebars rupture ^(b)	88.0	0.50	625.9	72.9	-	52.8	1.4
GFRP-LR	Rebars rupture ^(b)	107.5	1.30	550.7	71.8	_	50.9	1.4
GFRP-WR	Rebars rupture ^(b)	71.0	0.05	471.1	69.5		40.9	1.7

^(a) δ_{max} calculated at 0.85 P_{max} . In this case, no collapse was seen at that point. ^(b) The failure occurred for the achievement of the tensile strength by the intrados rebars.



Fig. 11. Bending test results: crack pattern. a) SR; b) GFRP-RR; c) GFRP-LR; and d) GFRP-WR.

3.2. Point load test set up

The point load test set-up is shown in Fig. 8. The point loads, simulating the TBM thrust, are applied on the segment, adopting the actual pad configuration and geometry used by the TBM (Fig. 8a – c). Two 2000 kN hydraulic jacks push on each pad (maximum 4000 kN per pad), for a total maximum force of 12.000 kN.

A uniform support is considered, as the segment is placed on a stiff beam suitably designed (Fig. 8a, b).

For each loading steel pad, a couple of potentiometers (placed on intrados and extrados segment surface respectively) with a length equal to 1420 mm were adopted to measure the vertical displacements (Fig. 8f). Furthermore, two LVDTs, used to measure possible cracks opening, were placed each between a pair of steel pads (Fig. 8d).

Scheme of the crack pattern at different loading steps were recorded, as well as the opening of the main cracks by means of a crack width gauge card.

Two complete loading-unloading cycles, with steps of about 250 kN were carried out, and, in particular, as summarized in Fig. 9:

• First cycle: 0–1580 kN (loading cycle up to service load); and

• Second cycle: 0–2670 kN (loading cycle up to the maximum TBM thrust load).

4. Test results

4.1. Bending test results

The results, expressed in terms of load versus midspan displacement, obtained from the full-scale tests carried out on the reference steel reinforced element (SR) and the GFRP reinforced segments without sand coating (GFRP-RR, GFRP-LR and GFRP-WR) are compared in Fig. 10. The main outcomes are summarized in Table 2, including the load at first crack (P_{crack}), the maximum crack width (w_{crack}), the maximum load (P_{max}), the maximum midspan deflection (δ_{max}), the midspan deflection at steel yielding (δ_y) for SR segment, and deflection when the stiffness drastically changes (δ_1) for GFRP reinforced elements. Furthermore, an indication of the ductility factor (μ), defined as the ratio (δ_{max}/δ_y) for tra-

Table 3

GFRP-RR solution: comparison to the service load between reinforcements with and without bond treatment.

Reinforcement	Coating treatment	P _{crack} (kN)	w _{crack} (mm)	R _w (%)
GFRP-RR	None	88.0	0.50	-
GFRP-RR(+B)	Sand	106.7	0.15	70



Fig. 12. GFRP-RR(+B).



Fig. 13. Comparison between GFRP-RR and GRRP-RR(+B).

ditional steel reinforced segment and as the ratio (δ_{max}/δ_1) for the GFRP reinforced one, is given. The observed failure mode is also pointed out.

After first cracks, several cracks developed in all the segments as shown in Fig. 11, where the complete crack patterns and widths are plotted for all the examined cases.



Fig. 14. Bending test: crack pattern comparison. a) GFRP-RR; and b) GFRP-RR(+B) segment.



Fig. 15. Point load test: crack pattern. a) SR; b) GFRP-RR; c) GFRP-LR; d) GFRP-WR.

Table 4				
Point load	test:	maximum	crack	width.

	Loading cycle	SR	GFRP-RR	GFRP-LR	GFRP-WR	GFRP-RR(+B)	$w_{max}^{(e)}$ according to provisions
w _{crack} (mm)	SL ^(a)	0.20	1.00	1.20	1.30	0.60	
	UT ^(b,c)	0.35	1.25	1.50	2.00	1.20	
	UL ^(d)	0.15	0.40	0.70	0.70	0.25	0.50

(a) SL = Service load (1580 kN).

(b) UT = Unblocking thrust (2670 kN).

^(c) For metro tunnel, TBM pushing capacity coincides with unblocking thrust.

^(d) UL = Unloading (0 kN).

(e) w_{max} for RC segments with traditional steel reinforcement, according to the Codes, must be smaller than or equal to 0.30 mm.



Fig. 16. Point load test results: crack pattern of GFRP-RR(+B) segment.

Because of higher Young's modulus of steel rebars with respect to GFRP ones, the SR segment showed a behavior stiffer than the GFRP ones. Fig. 11 points out how the crack patterns are comparable for each segment tested, regardless of the reinforcement adopted. In all cases analysed, the failure occurred for the attainment of the tensile strength in the intrados reinforcement.

As already found in Ref. [20], all precast concrete segments shown a comparable structural behavior in terms of maximum displacements, despite of the brittleness of the GFRP reinforcement.

Looking at the results, whilst the GFRP-WR segment showed a failure load equal to the reference SR one, the other two prototypes, GFRP-RR and GFRP-LR, exhibited significantly higher failure loads, with increases of about 32.7% and 16.7% respectively, with respect to the reference element.

In the light of the results obtained and taking into account the three manufacturing process aspects, mentioned above (technical feasibility, commercial feasibility and technological feasibility), the GFRP-RR represents the best solution among the prototypes tested.

Nevertheless, as expected, higher cracks width were measured in the GFRP reinforced element, with respect to the traditional reinforced one (SR), as shown in Table 2 and Fig. 11.

In order to enhance this behavior by improving the bond performances, a sand coating of the GFPR ring reinforcement was carried out, and a new segment named GFRP-RR(+B) was cast and tested. The measured load-displacement diagram is shown in Fig. 13 and compared with the GFRP-RR typology. The main outcomes are further shown and compared in Table 3, including: load at first crack (P_{crack}), maximum crack width at first crack (w_{crack}). Furthermore, crack width reduction (R_w) is included.

The coating treatment, besides increasing the load at first crack, led to a reduction of about 70% the width crack, passing from 0.50 mm to 0.15 mm.

Fig. 14 shows, from a quality point of view, the crack patterns comparison between the solutions. In both cases, several cracks

have been developed. The sand coating of the GFRP-RR(+B) segment led to more widespread cracks (Fig. 14b) with lesser width compared the same solution without surface treatment (Fig. 14a).

4.2. Point load test results

The point load test results, in terms of crack pattern for SR and GFRP reinforcement typology without sand coating, and for each load step, is shown in Fig. 15.

In all the analysed cases, the first cracks opened between the load pads, and extended their length, at the load increase, in the whole element height. For high load values, close to the maximum one, bursting cracks opened under the steel pads.

In Table 4 the maximum crack width (w_{crack}) measured in each cycle considered, is summarized. Furthermore, the maximum crack width (w_{max}) suggested by Ref. [10] for concrete structures with GFRP reinforcement, is shown.

The maximum value of the crack width, under the serviceability and maximum loads, are higher in GFRP specimens, if compared to steel reinforced element ones.

Once again, as shown in Table 4, the GFRP-RR prototype represents the best solution among those under evaluation. This is confirmed, not only during the loading steps in which the crack width are smaller than those observed on GFRP-LR and GFRP-WR but, looking at the crack width at the end of the test, when the load was removed, the cracks are within the limitation imposed by CNR [10].

Finally, the segment reinforced with sand coating rings reinforcement, GFRP-RR(+B), was cast and tested (Fig. 16). Looking at the results (Table 4), in term of maximum crack width, the sand coating increased the segment performance leading to cracks width decreasing from 0.40 mm (without bond) to 0.25 mm (with bond) with a reduction of 37.5 percentage points.

Three different typologies of GFRP reinforcement were designed and realized. The obtained results were discussed and compared with the one measured on a reference segment with traditional steel reinforcement.

The first solution, named GFRP-RR consists of closed "Ring Reinforcement" for both longitudinal and transverse reinforcement; the second one, GFRP-LR, is a "Lattice Reinforcement" and it is a combination of curvilinear bars, which are interlinked by means of lattice structures. The third prototype is a "Wirenet Reinforcement" (GFRP-WR), in which the reinforcement cage consists of a wire net in extrados and intrados with "C" shaped stirrups. Finally, in order to better control the crack pattern and width, a solution obtained through a sand coating of the closed ring reinforcement, named GFRP-RR (+B) was produced and tested.

If compared to the behavior of a traditional steel reinforced segments, the obtained results show the effectiveness of the proposed reinforcement with GFRP bars in tunnel linings. In particular, based on the experimental structural behaviors, it can be concluded that, with reference to both the performances under flexure and TBM thrust, the best solution appears the Closed Ring Reinforcement (RR). A further enhancement, in terms of crack width, can be obtained if a sand coating is carried out. Nevertheless, if no severe requirement, in term of crack openings for high load levels are imposed in the design process, the Closed Ring Reinforcement appears to be the most suitable solution.

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