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Variable, full-scale tester for tunnel linings

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Abstract
This contribution presents a new tester for tunnel lining segments in the final serviceability load stage. It aims to concentrate the resources of testing laboratories on a single segment capturing most of the real conditions present in real tunnels. Its variability is presented, that is, the geometrical and mechanical configurations it can adapt to. Its performance is validated by means of a prototype test on a conventionally reinforced tunnel lining segment. Horizontal and vertical loads are applied simultaneously to simulate similar loading conditions as in real tunnels. To this end, three hydraulic cylinders are coupled, so that forces up to 5 MN in both directions can be applied. The external loads are introduced radially in a semi-distributed manner at eight different points of the segment extrados. The kinematics recorded show a proper load distribution on the specimen. Moreover, it brings into light its weak spot, that is, the longitudinal joint.

KEYWORDS
full-scale test, longitudinal joint, partial area loading, precast tunnel segments, splitting, tunnel lining

1 | INTRODUCTION

To cover their needs for transportation infrastructure, cities have resorted to various tunneling techniques as a practical option. Regardless of the chosen construction technique, the structure results in a—mostly reinforced concrete—lining that, in the final loading stage, is exposed to external distributed loads that lead to a combination of internal bending moments, normal and shear forces, transmitted through a continuous or a segmented structure (Figure 1). Among tunnel excavation techniques developed to optimize construction times while reducing disturbances to the surface traffic, tunnel boring machine (TBM) driven tunnels stand out. Therein, the lining is discretized with segments that are exposed to bending. Additionally, they are subject to concentrated forces at the ring joint from the TBM thrust cylinders and at the longitudinal joint for the transmission of internal—eccentric—normal forces that generate corresponding splitting stresses in the segment body. Analytical and numerical models are used to predict internal forces and deformations that are corroborated with experimental data. Many of them were compared, for instance, by Zhao et al. Given that the joints are regularly exposed to higher utilization ratios, their design should be in focus. Beneath the load introduction, a multiaxial stress state...
appears, leading to compressive stresses in transversal direction that are in equilibrium with tensile stresses at a deeper location. The analysis of this phenomenon was mainly divided in problems with spatial (e.g., Spieth$^3$) and plane load expansion (e.g., Ibell & Burgoyne$^4$; Wurm & Daschner$^5$), and several approaches were proposed with the goal of predicting the admissible load introduction. Although some of them consider the amount of reinforcement (e.g., Niyogi$^6$), the current state of the art usually relies on pure geometrical considerations, as done with the cubic and the square root approach, described by Equations (1) and (2), respectively, where $f_{cd}$ is the allowable compressive strength of concrete, $A_{c0}$ is the contact area, and $A_{c1}$ is the load distribution area.

$$R_{cr} = f_{cd} \cdot A_{c0} \cdot \sqrt{A_{c1}/A_{c0}} \tag{1}$$

$$R_{sr} = f_{cd} \cdot A_{c0} \cdot \sqrt{A_{c1}/A_{c0}} \tag{2}$$

The current normative allows an increase of up to three times the contact pressures in spatial cases, whereas only 1.1 times in plane cases,$^7$ although experimental campaigns demonstrated this criterion to be too conservative (e.g., Schmidt-Thró et al.$^8$). In any case, the additional compressive strength can only be reached if the arising transversal tension forces are covered by reinforcement, materialized in form of rebars (closed stirrups or welded rebars), steel fibers (e.g., Gong et al.$^9$ and Plückelmann et al.$^{10}$), or a combination of both (e.g., Smarslik & Mark$^{11}$). For the first case, reinforcement should be placed such that its barycenter coincides with the position of the resultant tensile force, and given that its location changes with the eccentricity of the load applied, the design must find a compromise to cover all significant load cases.

To guarantee the prescribed safety and minimize costs, it is decisive to replicate the real behavior of the structural elements that compose the tunnel in full-scale experiments so that size effects are discarded. Various efforts have been made to this end, concentrated on improving tunnel construction technology in general or on evaluating the security and structural behavior of the lining system of particular construction sites, for instance, the experimental setup constructed for the Elbe river tunnel in Germany.$^{12}$ Blom and van Oosterhout$^{13}$ reported a full-scale test setup with three coupled rings and alternated position of the longitudinal joints. Radial forces, simulating uniform and ovalization loading, and axial forces were applied to get a deeper understanding about the lining behavior at the assembly and the serviceability stage. Experimental results were used to validate an analytical model that implements the rotational stiffness at the longitudinal joints and the interaction between adjacent rings.$^{14}$ Further experiments were performed on full rectangular shaped rings to analyze the safety and reliability in the design method for the Kyoto Municipal Subway in Japan.$^{15}$ Liu et al.$^{16}$ studied the mechanical behavior of tunnels in serviceability stage with focus on nearby constructions. To this end, a full ring was subjected to the force of 24 hydraulic jacks divided into three groups to simulate active loads and soil reaction. The system failure was governed by the longitudinal joints. Liu et al.$^{17}$ extended the investigation to a more complex multiple ring setup to analyze the effect of staggered joints on the structural bearing capacity. Different from laboratory experiments, Molins and Arnau$^{18}$ executed a loading test in situ at the Line 9 of the metro of Barcelona, where the external loads were introduced with hydraulic jacks previously installed at the outer surface of the segments close to the tunnel crown. The goal was to determine the performance of steel fiber reinforced concrete (SFRC) segments in hard ground. The approaches so far mentioned have in common that at least one full ring was tested. While this mostly captures the real tunnel mechanics, the equipment and loads required can be unattainable. Moreover, these test setups are generally difficult to adapt to variable tunnel geometries.
Alternatively, other approaches relied on testing a single element. Mid-span loaded segments were especially employed to study the capabilities of hybrid-reinforced linings, exposing the post-cracking enhancement of steel\textsuperscript{19–21} and polypropylene\textsuperscript{22} fibers inclusion under pure bending. Other experimental setups counted the addition of normal forces. Hemmy and Hestermann\textsuperscript{23} studied the use of steel fiber reinforcement focusing on the final load stage with conditions found in real tunnels, that is, large normal forces with small eccentricities. For this purpose, they built a testing device capable of actively changing the M/N configuration in isolated lining segments. Two vertical loads were applied on the segment extrados and were controlled independent of the horizontal ones introduced at the longitudinal joints. Gipperich et al.\textsuperscript{24} used a similar concept, instead with a mid-span vertical load, to explore the bearing capacity of steel fiber high and ultra-high-strength concrete linings. Gehwolf et al.\textsuperscript{25} built a modular test setup that counted with a sliding support to test segments of variable length and was employed to compare conventional and SFRC segments.\textsuperscript{26} Yet, these approaches bear the disadvantage that the boundary conditions simulated do not fully reflect those existing in real tunnels. Hence, loads at the longitudinal joint are introduced through steel hinges and the effects of partial area loading, present in real tunnels, remain excluded. Furthermore, load introduction should be evenly distributed along the outer lining surface to better represent the active soil and water pressures.

Some authors satisfactorily conducted studies on simplified, isolated geometries to assess the mechanics of discontinuity regions in lining segments.\textsuperscript{8,11,27–29} With such test setups, however, further global effects on the segment body cannot be included.

The goal of the present study is, therefore, to investigate the lining segment behavior in final state by means of a newly developed testing device that encompasses the advantages of the previously cited methods (i.e., a test on a single specimen to optimally concentrate the capacity of the testing labs) and provides boundary conditions capable of reproducing partial area loading effects and semi-distributed loads at the extrados, such that most of the relevant effects are present in one single experiment. The concept presented provides variability, so that it can be applied to different geometrical and statical configurations, as displayed in the following section. Although the method is presented for the case of TBM driven tunnels, it can be applied without loss of generality to any isolated region of a continuous tunnel constructed with another excavation method (e.g., the new Austrian tunneling method) or even to other arched structures.

### 2 DEVELOPMENT OF THE VARIABLE TESTING DEVICE

#### 2.1 Variable parameters

The characteristics of the projected lining depend on the purpose of the tunnel, the structural demand, and boundaries related to the construction procedure.\textsuperscript{1} The variables that define the problem, in the scope of this paper, are divided into geometrical and mechanical. The former comprehend the internal radius \( r \), the lining thickness \( t \), and the ring configuration, that is, the number of segments that compose the ring (Figure 2). For the case of TBM driven tunnels, the ratio \( r/t \) lies between 9 and 12.5 for internal diameters greater than 5.5 m and between 7.5 and 12.5 for internal diameters between 4 and 5.5 m. For tunnels with internal radius under 2 m, \( t \) varies between 15 and 28 cm.\textsuperscript{30,31} The number of elements \( n \) that make up a ring, and therefore the length \( L \) of each one, should be chosen such that the ratio of the curved length along the centroid to the segment thickness is equal or greater than 10.\textsuperscript{30} Additionally, the chosen configuration must also be compatible with the position of the thrust cylinders of the TBM.\textsuperscript{31} A further variable in tunnel projects is the segment width, which can be disregarded if the problem is modeled as planar, as pursued in this paper. The second group of variables includes those related to internal and external forces. For given external loads in the final stage, the lining is internally mainly subjected to normal forces and bending moments, besides local tensile stresses at the longitudinal joints\textsuperscript{30,31} due to partial area loading effects. At the longitudinal joint, it traduces to a force \( R \) transmitted along \( \delta \) (a fraction of its width \( a \)) with a certain eccentricity \( e \) (Figure 2). At the same time, \( R \) can be decomposed in longitudinal (\( h \)) and transversal (\( v \)) direction, resulting \( H \) and \( V \), respectively. Table 1 presents a summary of the geometrical parameters and their most common range in tunnels.

![Figure 2](image-url)
2.2 Design

As a solution to the problem stated, a new device that provides more realistic boundary conditions and tests one of the segments that compose a tunnel ring is presented (Figure 3). It pursues to simulate analogous conditions to the ones presented in Figure 2, that is, a single-span curved member supported at the longitudinal joints with bending stiffness provided by the adjacent segments. It is subject to radial loads introduced at the segment extrados (representing the action of the surrounding ground) and to constant normal force and bending moment (response of the structure), the latter in case that the normal force is applied with a certain eccentricity at the longitudinal joint. Indeed, it is focused on the perpendicular component of the external acting loads, analogously to the full-ring setups described above and to common practice in bedded models, which are recommended for an economical design approach of shallow tunnels. This load configuration is achieved with two loading frames constructed with steel quality S355, horizontal and vertical, working together to simulate the final loading stage. The vertical load \( V \) is introduced with a steel frame activated by two hydraulic cylinders (V-East-3.0 MN and V-West-2.5 MN) attached by a HEM 1000 cross beam. \( V \) is evenly distributed to two HEM 700 profiles and by means of single-span beams (HEM 300), to eight points on the extrados of the segment, which simulates a distributed pressure and seeks to minimize concentrated bending moments. To guarantee the application of radial loads and for them to be equally distributed independent of the segment local stiffness, \( 310 \times 100 \times 33 \) [mm] reinforced elastomeric pads are placed between the steel frame and the specimen. Providing large tangential deformations at low load ratios, shear stresses on the extrados of the segment are reduced. Bolted hinges allow to change the radial loads position and angle, so that they adapt to different segments and their deformation during the experiment. Finally, \( V \) is transmitted to reinforced concrete fundaments (and from there to the ground) through a steel block that simulates the continuity of an adjacent segment in a real ring. The transmission to the steel block follows with a bolted wedge that can be replaced depending on the segment geometry. Given its variability, it can be adapted to study the influence of connectors at the longitudinal joint. This goal, however, was not pursued in this experiment.

The introduction of the horizontal load \( H \) is done through a self-compensating frame with external

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**TABLE 1** Geometrical parameters in tunnel boring machine (TBM) driven tunnels and their common range of variation

| Parameter | \( r \) [m] | \( n \) [-] | \( t \) [m] |
|-----------|-------------|-------------|-------------|
| Range     | 1.1–7.0     | 6–10\(^a\)  | 0.15–0.60   |

*Key segment included.*
Dimensions $5.33 \times 1.75 \times 1.5$ [m] insulated from the ground at one end with ultra-high-molecular-weight polyethylene. In this way, the laboratory facilities are not affected by $H$. The frame consists of four HEB 300 longitudinal profiles, identified as A, B, C, and D in Figure 3, bolted to the corners of two rigidized 100 mm thick steel plates. The load, driven by a hydraulic cylinder ($H=5.0$ MN), is transmitted through a variable amount of steel plates (which allows to test segments of different lengths) to a steel block, mentioned above. This block is responsible for the efficient encounter of the loads $R$ from the segment, $H$ from the cylinder, and the reactive load $V/2$ from the ground support. Indeed, $R$ is the resultant force generated from $V/2$ and $H$. Midway two reinforced elastomeric pads with dimensions $380 \times 360 \times 33$ [mm] arranged horizontally protect the cylinder against shear forces. The steel-concrete encounter is solved with two 3 mm hardboard cutouts of width $d$ (load introduction width) that smooth over concrete surface irregularities and thus avoid stress peaks. They are placed between the specimen and the steel wedge and hold in position through the pressure of the segment to be tested. There, shear stresses are almost ruled out by coupling the three cylinders involved. Being V-East and V-West displacement-controlled, the resulting load $V$ is used as input for the horizontal cylinder. The software that controls the system multiplies the input value by a constant, to be chosen freely, and uses the result ($H$) to drive the horizontal cylinder (driven by load control). Thus, a constant proportion between $H$ and $V$ throughout the experiment can be achieved.

At the north—reactive—side, $H$ is directly transmitted to the steel block through two horizontally arranged reinforced elastomeric pads with dimensions $570 \times 420 \times 10$ [mm]. This guarantees an even stress distribution and rules out the transmission of vertical forces and bending moments to the horizontal frame. Hence, they are completely absorbed by the reinforced concrete fundament. The use of elastomeric bearings introduces larger deformations in horizontal direction whose influence is covered with the previously described system employed to control the cylinders. Given that $H$ is load controlled, the cylinder automatically compensates the deformations of the elastomeric bearings (and of the horizontal steel frame). However, this produces relative displacements between the steel and the concrete blocks that must be allowed to avoid a loss of the effective load $H$ introduced in the test specimen. This is accomplished with the use of a 0.5 mm PTFE sliding layer on both sides. This material has already proven to be useful even for bearing high contact pressures$^{35}$ and has been used in complex test setups.$^{36-38}$

| Parameter | $r$ [m] | $L$ [m] | $t$ [m] | $e$ [mm] | $d$ [mm] | $H$ [MN] | $V$ [MN] |
|-----------|---------|---------|---------|---------|---------|---------|---------|
| Range     | 1.50–inf. | 1.00–2.25 | 0.15–0.50 | 0–70 | 33–250 | up to 5 | up to 5 |

The variable components offer a modular system able to test specimens with geometry and loads described in Table 2. There, the upper bound of the contact area at the longitudinal joint ($a$) was assumed equal to $t/2$, and the rest of the mechanical parameters were computed using the linear approach for transmitted bending moments at longitudinal joints.$^{33,39}$ The segment width can take values up to 75 cm, being the only magnitude to be adjusted for larger specimens. This, however, does not modify the results since the studied problem is planar and therefore, independent of the width.

The universal character of the testing device makes it suitable for experiments on arc-shaped structures and, without loss of generality, for elements without curvature.

### 2.3 Verification of the testing device

The device was employed to test a real segment until failure and monitor its performance (Figure 4). This section concentrates on the aspects relevant to the tester.
global behavior and security, whereas the next one deals in more detail with the tested prototype and the test setup. Here, the aspects analyzed are cylinder coupling, symmetry in the load application, and stresses and deformations of the components.

The maximum loads tested were $V = 1350.4$ kN and $H = 2519.8$ kN. Up to 1000 kN ($V = 535.7$ kN), loads increased with a speed of 0.3 mm/min. At this point, the loading process was paused for visual control of the components, and next, loading continued with a speed of 0.5 mm/min until specimen failure.

Figure 5 depicts the relation between the total vertical and the horizontal force applied. The offset from the origin is due to the own weight of the vertical load frame on the specimen prior to the test begin. The slope was always kept constant with value 0.54, coinciding with the one previously imposed to the software for the load control of the horizontal cylinder. Furthermore, the signal fluctuation was less than 1 kN. The result indicates that the cylinder coupling was successfully achieved.

Figure 6 displays the strain values recorded by means of strain gauges glued at the web of the profiles A–D shown in Figure 3 (on their barycenter). The largest maximum value registered was $0.221 \%$ (profile D), whereas the lowest one was $0.197 \%$ (profile B).

The difference, transformed into stresses, represents 5 MPa, which is within the tolerance margins. Turned into deformation values, they lead to a longitudinal deformation of 0.9 mm. Given that the lining segment registered a low deformation value in horizontal direction (around 0.3 mm), it suggests that most of the necessary cylinder hub (6.9 mm) was concentrated at the elastomeric pads, namely 3 mm and 2.7 mm at the south and north side, respectively. Whereas the former was completely taken by the cylinder, the latter led to an eccentricity in the introduction of $V$ on the segment. However, it only represented a variation of 0.1% with respect to the segment length. To evaluate the behavior of the sliding planes, the force introduced $H$ was compared to the reaction of the longitudinal profiles. Indeed, the transformation of the four strain values into forces and their addition conduces to the value 2628.4 kN that, except for a difference of 4.3% due to measurement precision, coincides with $H$.

The results presented so far show not only a symmetrical behavior, but also that the PTFE foil on the concrete blocks and the sliding layer under one of the steel plates satisfactorily minimized transfer of shear stresses, so that the horizontal load was fully introduced in the test specimen.

In the design stage, the behavior of the steel plates was studied with a linear finite element model. The stresses for the maximum load configuration reached in the experiment are displayed in Figure 7.

The stresses obtained by transforming the values recorded by strain gauges glued at the steel plate stiffeners agreed with those of the simulation. This indicates that the mechanical behavior of the tester was the expected one. Furthermore, the stresses were low enough to guarantee low material utilization ratios. The deformations registered at the center of the plate in the simulation was about 0.5 mm, which represents a ratio of 1/3000 with respect to its minimum length, small enough to guarantee...
uniform distribution of the cylinder force. Given that the material of the testing device behaves linearly, the effects can be simply scaled up by a factor 2 for the full capacity (5 MN). Even in that case, deformations and stresses still remain widely between expected security margins.

The elastomeric pads placed on the extrados of the segment isolated it from the vertical frame. Indeed, given that the system is statically determined and that the elastomeric pads have low shear stiffness, the steel components were free to move without changing the load concept in the specimen. Therefore, the vertical frame is not further analyzed.

The security of the tester was therefore guaranteed and the correctness of the load introduction was validated.

3 | PROTOTYPE TESTING OF SEGMENTAL LINING

3.1 | Segment and test setup

This section focuses on the tested prototype and the test setup. The geometrical parameters were automatically defined by the vertical formwork used to manufacture the segment, conducing to the geometry depicted in Figure 8.

![Figure 7: Von Mises stresses for one of the steel plates conforming the horizontal frame](image)

![Figure 8: Geometry and reinforcement of the test specimen: (a) three-dimensional visualization with notation of cross-sections, (b) transversal cross-section, and (c) longitudinal cross-section](image)
The main material properties are listed in Tables 3 and 4, where those related to concrete were computed as the mean value out of three cylinders with 15 cm diameter and 30 cm length. The mean value of the tensile strength $f_{ctm}$ was considered coincident to $f_{ctm,sp}$. Splitting forces were covered with conventional reinforcement, that is, stirrups placed horizontally in the plane of load introduction.

The mechanical parameters were defined, firstly, by choosing the eccentricity $e$ for the load introduction at the longitudinal joint. Thus, computing the maximum eccentricity delivered by the simplified bilinear model$^{33,39}$ of the Leonhardt/Reimann approach$^{41}$ led to 32 mm. In accordance with the considerations set out in the Issue 631 from the German Committee for Structural Concrete$^{42}$ for the calculation of bearing capacity against concentrated loads, a load introduction width $d = 51$ mm was deduced.

Figure 9 shows the test setup where, additionally, the load concept described in Section 2.2 is displayed. The horizontal and vertical displacements at the corners NE, SE, and SW, and the vertical displacements in the middle section of the segment were recorded with linear variable displacement transducers (LVDT) with a frequency of 10 Hz. The same frequency was used for the strains reported at the three selected planes displayed in Figure 9. The kinematics at the remaining corner (NW) were recorded with Digital Image Correlation (DIC), which not only measures displacements at discrete points, but also surface displacement fields that, through internal software computation, lead to fields of localized deformations. Thus, this provided a qualitative approximation of the crack pattern. The measured surface was sprayed with a random speckle pattern so that each zone could be uniquely identified by the photos utilized by the software. Due to the long duration of the test (approximately 1 h) and the large volume of data generated, the frequency used for DIC was 1 Hz. Figure 10 depicts the measurement devices at the southwest corner of the test specimen.

### 3.2 Results and discussion

For the further analysis, the global behavior of the test specimen extracted from the experimental data is compared to the one obtained from a Bernoulli beam model, whereas at the longitudinal joints, the bearing capacity is compared to selected equations that apply to partial area loaded structures.

#### 3.2.1 Load-deflection behavior

The load-deflection curves obtained from the experiment are depicted in Figure 11, together with the theoretical parameters.

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**TABLE 3** Basic properties of concrete

| $f_{cm}$ | $f_{cm,cube}$ | $f_{ctm,sp}$ | $f_{ctm}$ | $E_{cm}$ |
|----------|---------------|-------------|-----------|---------|
| [MPa]    | [MPa]         |            | [MPa]     | [MPa]   |
| 42.8     | 48.7          | 3.7         | 3.7       | 29658.7 |

**TABLE 4** Basic rebar properties

| Designation | $\phi$ [mm] | $f_y$ [MPa] | $E_s$ [MPa] |
|-------------|-------------|-------------|-------------|
| B500B       | 8           | 540         | 201,000     |

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**FIGURE 9** Test setup for the prototype testing. Load concept and measurement sensors are depicted in blue and red color, respectively. Cardinal points in text boxes. Vertical load introduction frame is omitted for clarity.
beam model. The vertical deformation in the middle of the segment body was selected as referential quantity and was contrasted to both the vertical and horizontal force. Here, it must be considered that the load application led the specimen to progressively slide in tangential direction over the plane of the steel wedges, which agrees with the radial displacements experienced in full rings. The hardboard cutouts, however, accompanied this movement, so that the load introduction remained almost unchanged during the experiment. The resulting rigid body motion originated at the longitudinal joints was subtracted from the central displacement measurements recorded by the sensors. The values were calculated as average of those recorded on the east and west side of the segment. The maximum load obtained was $V = 1350.4 \text{ kN}$, which corresponded to $H = 2519.8 \text{ kN}$ and to a deformation $u = 2.2 \text{ mm}$. From the beam model, it was obtained $u = 2.0 \text{ mm}$.

After minor system adaptions took place (reflected in an initial lower stiffness), the load increased almost linearly with a similar stiffness as in the theoretical model. Without a remarkable decaying curve after the maximum load was reached, the specimen exhibited a brittle failure. For the beam model, the full cross-section was used to model the stiffness. Thus, no stiffness loss took place due to bending and the failure occurred solely due to partial area loading effects in the discontinuity region of the longitudinal joint. Due to imperfections in the concrete matrix, the cracking process developed asymmetrically, and therefore, the segment stiffness was uneven, leading to unequal deformations. This is reflected in the slightly divergent behaviors of the west and east side that led to torsion in the lining segment with respect to the longitudinal axis. The effect became larger with load increment. Nevertheless, even at the failure stage, the difference between the eastern and western values was only 0.5 mm. The average curve shows a good agreement with the theoretical model, meaning again that the global behavior of the experimental setup was close to the idealized statical model used for computational purposes.

3.2.2 Crack patterns

At the west side, two predominant cracks developed at the longitudinal joint (Figure 12a). The first one appeared almost longitudinally and next, a second one emerged with an inclination of approximately $11^\circ$, with larger width towards the end of the experiment. This pattern was confirmed by the DIC data depicted in Figure 12b, which shows the major principal strains recorded just before the failure took place. Spalling occurred right after at the red colored area, leaving in evidence small internal cracks in parallel direction to the load introduction. At the east side, one predominant crack with an inclination of approximately $13^\circ$ was observed (Figure 14b). Both inclined cracks extended towards the extrados of the segment. A system of further secondary cracks with predominant longitudinal direction was observed on both sides.

The first crack was the response to splitting stresses generated by the load introduction, whose development was limited by the splitting reinforcement. While the loads increased, the inclined ones could not be properly restrained and advanced more intensively toward failure. In addition, Figure 12a shows that the crack was developed through the upper part of the splitting reinforcements, where its anchorage was not fully developed.
Then, they advanced toward the extrados and acquired a considerable width, leading to the formation of an upper concrete wedge towards collapse. This fact also exposes the weakness of conventionally reinforced longitudinal joints against eccentric normal forces. Indeed, the concrete cover leaves unreinforced regions that are unable to bear tensile stresses and, consequently, considerable damage may occur. This is particularly relevant for both the construction and serviceability stage, given that concrete spalling can lead to water leaks. The results of experimental campaigns on steel fibers\(^9\) and hybrid\(^11\) reinforced joints reveal a proper behavior when using steel fibers. Thus, its addition may enhance the performance of segmental joints.

The wedge next to the load introduction corresponds to the region subject to a biaxial compression stress state, and the small cracks left in evidence through the concrete spalling induce concrete crushing.

### 3.2.3 Behavior and bearing capacity at the longitudinal joints

Since the failure occurred at the northern longitudinal joint, it is necessary to analyze that region in particular. Figure 13 shows the recorded strain gauge data, where it is evidenced that the first layer was initially subject to compression and then changed to tension. Moreover, the second one was always subject to tension and surpassed the strain limit \(2.7\frac{\varepsilon}{\%}\), that is, it yielded. Although the third layer was always subject to tension, the material did not reach its yield stress. Figure 14 depicts the strains recorded at the load levels \(H = 1000\) kN and \(H = 2519.8\) kN, together with the theoretical stress distribution applied to the geometry studied.\(^43,44\) While \(H = 2519.8\) kN corresponds to the failure stage, \(H = 1000\) kN is a reference for uncracked state. Indeed, it is approximately 40% of the maximum value.

The evaluation of the DIC data at the longitudinal joint conducted, subtracting the rigid body motion, to a rotation at the maximum load of 3.53 mrad. Here, it must be considered that the real strain distribution beneath the load introduction is nonlinear because of the concrete plastification. The points employed for the measurement were near to the corners of the specimen and therefore
outside the plasticized area. Thus, the maintenance of plane sections was assumed. The rotation was attributed exclusively to the loads, whereas effects caused by geometrical imperfections, as studied by Caratelli et al., were not considered.

The first reinforcement layer remained under compression for lower loads, correlating with the biaxial compression zone obtained for linear analysis. It is then relevant to analyze what caused the sign change afterwards making use of the material parameters (Tables 3 and 4). The geometry of the case studied leads, for the case of linear analysis and with the notation of Figure 14, to a maximum splitting stress of approximately \( \sigma_0 = \frac{R}{(b \cdot d')} \). Herein, \( b \) denotes the segment width and \( d' \) is the width of the load distribution area. To reach the concrete mean tensile strength \( f_{ctm} \), the necessary stress is \( \sigma_0 = 12.6 \, \text{MPa} \), which multiplied by \( d' \) and \( b \) conduces to \( R = 1523.9 \, \text{kN} \). Disposed in horizontal and vertical direction, the forces take the values \( H = 1472.0 \, \text{kN} \) and \( V = 788.9 \, \text{kN} \), respectively. It coincides with the behavior observed in Figure 13. At that load level, the concrete reached its tensile strength, and the consequent cracks induced a nonlinear behavior in the strain development. Consequently, the lateral deformation caused by the splitting crack opening produced the first layer to change its sign and then, all the three reinforcement layers were subjected to tension. The consequent large transversal deformations decreased the concrete confinement and, finally, its compressive strength was reached. The resultant force value \( R \) at the failure load level can be analogously obtained as the composition of \( H \) and \( V \) with Equation (3).

\[
R = \sqrt{(V/2)^2 + (H)^2}
\]  

With \( R = 2608.7 \, \text{kN} \), it lies between the results obtained with the cubic (Equation (1)) and the square-root (Equation (2)) approach, namely, \( R_{cr} = 2401.6 \, \text{kN} \) and \( R_{sr} = 2908.7 \, \text{kN} \). In the equations, \( A_{c0} = b \cdot d \) and \( A_{c1} = b \cdot d' \). It should be noted that \( R_{cr} \) and \( R_{sr} \) represent average values of expectations. Thus, \( f_{cm} \) is introduced as their basis. Divided by the load introduction area, a contact pressure of 68.2 MPa is obtained, which represents an increment of 59% with respect to \( f_{cm} \).

This value was compared with campaigns II–VI from Ibell and Burgoyne, Wurm and Daschner, Schmidt-Thrö et al., and selected experiments of series S3 from Schmidt-Thrö, as displayed in Figure 15, which partially constitutes the database employed by Wichers and Schmidt-Thrö. In the experiments, specimens were reinforced with stirrups (II–VI from Ibell & Burgoyne; A and B from Schmidt-Thrö et al.; 1 to 4 from Wurm & Daschner) or with welded rebars (C from Schmidt-Thrö et al.; S3 from Schmidt-Thrö). The values were normalized with respect to the theoretical bearing capacity obtained with Equation (1), using \( f_{ctm} \) instead of \( f_{cd} \). This comparison should be interpreted carefully. The conditions prevailing in the present experimental campaign are different from those of the database. Furthermore, the experimental results could be influenced by additional factors that were not part of the analysis. Nevertheless, they offer a qualitative comparison for the understanding of the behavior at the longitudinal joint.

The result is in line with the findings of Schmidt-Thrö et al., who recommended to conservatively use the cubic root approach to compute the capacity of longitudinal joints reinforced with conventional reinforcement. In addition, the bearing capacity registered is considerably
larger than the 1.1-fold increase of compressive stress currently allowed by the Eurocode 2.7

Experimental values of the rotation at the longitudinal joint are next analyzed. The analytical approach of Leonhardt/Reimann41 leads to a rotation of 4.7 mrad for the maximum load reached, that is, a softer behavior than in the experiment. Also, Schmidt-Thrö46 had a similar observation based on full-scale experiments, which conducd to an alternative approximation that applied to the case studied leads to a rotation of 2.32 mrad, considerably lower. Interestingly, the bilinear approach reveals a rotation of 3.57 mrad that agrees very well with the measured value. Here, it must be recalled that the contact area of the longitudinal joint, in this case, was materialized by means of hardboard cutouts using a simplified plastic criterion, as commonly assumed in static analyses of partial area loaded structures. Contrarily, previous experimental campaigns and derived analytical approximations studied the bending moment-rotation behavior measuring the real joint opening for given bending moment and normal forces. Although this makes the results not directly comparable, such a comparison provides a qualitative assessment of the experiment performance.

3.2.4 | Strain state in reference cross-sections

The behavior of the lining segment, extracted from the experimental data, was compared to the beam model previously introduced. Figure 16 depicts the values recorded by the strain gauges at the middle part of the segment and contrasts them with the computed theoretical values at the reference load levels mentioned above. The experimental values shown were computed as average of the measurements obtained from the three reference planes. The results show that the average strain value reached in the upper reinforcement was 0.89‰, whereas in the lower reinforcement, 0.26‰. For the further analysis, a rigid bond between concrete and rebars was assumed. Using the nonlinear stress–strain relation given in Eurocode 27 and the material properties (Tables 3 and 4), the normal forces and their corresponding eccentricity during the experiment were calculated (Table 5). They represent an average difference of 6% with respect to the values obtained from the beam model (normal force of 1036 kN for H = 1000 kN and 2610 kN at the failure stage).

Figure 16 shows a good agreement between the theoretical and the experimental data. Most of the deviations with respect to the expected values were due to the precision of the sensors for low strain levels and the scatter of the material properties of the specimen. It is evidenced that, as expected, the resultant force was transmitted within the central core of the cross-section and therefore the segment was fully compressed. Moreover, the average strain value reached at the top reinforcement led to a
compressive stress of 26.4 MPa, and at the barycenter, to 17.1 MPa, representing 61.7% and 39.8% of $f_{cm}$, respectively. These facts validate the simplifying assumptions made when comparing the deformations at the mid-span with the beam model, that is, no global stiffness loss because of bending and almost linear behavior. For both load levels, the eccentricity of the normal force was lower than the one planed at the load introduction, which was the only source of bending besides the effects of semi-distributed loads at the extrados of the segment. This can be attributed to the rotation of the longitudinal joint mentioned above that produced a trapezoidal and not a uniform distribution of the load introduction. Consequently, the effective eccentricity decreased. Nevertheless, even in the most unfavorable case, the eccentricity variation with respect to the segment thickness only represented 5.4%. Thus, the results showed a good convergence between experimental and theoretical values.

4 | CONCLUSIONS

A new tester is presented for the experimental verification of tunnel linings under final serviceability stage, with possible application to other arched structures constructed with other materials (e.g., masonry). It allows to concentrate the study on a portion of the tunnel cross-section that, in the case of TBM driven tunnels, results in one segment, reducing the load demands of testing laboratories. It also provides the boundary conditions necessary to include the effects of partial area loading at the longitudinal joint. Additionally, the load distribution on the extrados of the segment is semi-distributed, which more closely reflects real conditions of tunnels.

The following conclusions can be summarized:

- Despite the complex experimental setup involved, a symmetric behavior in the load introduction was evidenced, which is necessary to guarantee an even distribution of forces and regard the problem as planar.
- For small eccentricities, failure is governed by the longitudinal joint. There, the pressure reached a value 59% larger than the concrete compressive stress due to partial area loading effects, whereas at the segment body, the maximum stress resulted only 61.7% of the same referential value. Therefore, optimized discontinuity regions should be on focus in the construction of lining segments to achieve a more even material utilization.

The proportion between vertical and horizontal load was chosen such that the load introduction at the longitudinal joint was perpendicular. Further constructive solutions could be applied to efficiently resist shear forces at the contact zone between tester and specimen and thus, variable bending moments along the segment could be applicable.

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DATA AVAILABILITY STATEMENT

Data available on request from the authors

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| Cross-section 1 | Cross-section 2 | Cross-section 3 |
|----------------|----------------|----------------|
| $N$ [kN]      | $e$ [mm]      | $N$ [kN]      | $e$ [mm]      | $N$ [kN]      | $e$ [mm]      |
| $R = 0.4 \times R_{\text{max}}$ | $-977.4$ | $27.8$ | $-1053.9$ | $27.6$ | $-1166.8$ | $29.7$ |
| $R = R_{\text{max}}$         | $-2556.7$ | $27.5$ | $-2778.2$ | $25.8$ | $-3174$ | $28.0$ |
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