AUTHOR QUERY FORM

ELSEVIER	Journal: TUST Article Number: 1321	Please e-mail or fax your responses and any corrections to: E-mail: corrections.esch@elsevier.sps.co.in Fax: +31 2048 52799
--	---	---

Dear Author,

Please check your proof carefully and mark all corrections at the appropriate place in the proof (e.g., by using on-screen annotation in the PDF file) or compile them in a separate list. To ensure fast publication of your paper please return your corrections within 48 hours.

For correction or revision of any artwork, please consult <http://www.elsevier.com/artworkinstructions>.

Any queries or remarks that have arisen during the processing of your manuscript are listed below and highlighted by flags in the proof. Click on the 'Q' link to go to the location in the proof.

Location in article	Query / Remark: click on the Q link to go Please insert your reply or correction at the corresponding line in the proof
Q1 Q2 Q3	<p>Please update the reference 'Arnau and Molins (submitted for publication)'.</p> <p>Please check whether the term 'Variance ()2' is okay as given, and correct if necessary.</p> <p>The artwork of Fig. 9 and Appendix figure A4 seem to be the same. Kindly check.</p>

Thank you for your assistance.

Highlights

► Description of the test performed on the segmental tunnel lining of L9 in Barcelona. ► A maximum load of 3000 kN applied on the tunnel crown by hydraulic flat jacks. ► Decisive influence of tangential ground-structure interaction on lining stresses. ► Isolated ring response in rock: no significant stress transmissions to adjacent rings.

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

Experimental and analytical study of the structural response of segmental tunnel linings based on an *in situ* loading test

Part 1: Test configuration and execution

Climent Molins, Oriol Arnau*

Department of Construction Engineering, Universitat Politècnica de Catalunya (UPC), Jordi Girona 1-3, C1, 08034 Barcelona, Spain

ARTICLE INFO

Article history:
Available online xxxx

Keywords:
Segmental tunnel linings
SFRC
Real scale structural testing

ABSTRACT

During the last years the use of segmental linings in tunnels bored with tunnel boring machines (TBMs) have increased considerably. Despite this, uncertainties remain on the structural behavior of segmental tunnel linings. To overcome them, a new methodology for an *in situ* real scale test on this kind of tunnels was developed and applied on an experimental section placed at the new Line 9 (L9) of the metro of Barcelona. The main particularity of this test falls on its performance in the definitive placement of the rings and, therefore, in the real work conditions of the tunnel. The main advantage to previous real scale test experiences was the consideration of the real **ground–structure** interaction. The experimental section was composed by 15 rings built using steel fibers as a unique reinforcement. Significant loads were applied to the tunnel crown (max 3000 kN) by means of three hydraulic flat jacks embedded at the extrados of the loaded ring. A complete set of internal and external instrumentation was placed on the experimental section to obtain the necessary measurements to characterize the lining behavior. Such measurements were intensively treated and its main results are presented and analyzed in detail in this paper. This *in situ* test provides significant evidences of the structural response of SFRC segmental tunnel linings in hard ground conditions.

© 2011 Published by Elsevier Ltd.

1. Introduction

The evolution of the construction techniques and the development of the tunnel boring machines (TBMs) have allowed the construction of tunnels in almost all kinds of soils in difficult conditions such as low coverage or high ground and water pressures. These drilling advantages imply higher structural loads, both in construction stage to resist higher or more eccentric TBM thrust forces, and in the serviceability stage to resist higher ground and water pressures accomplishing the serviceability requirements.

The use of segmental tunnel linings allows an intensive industrialization of the construction process and, therefore, an accurate control of the segments quality. Moreover it also provides an important reduction of the time required to erect a complete structural member (ring) inside the TBM. This is one of the reasons for the construction speed provided by the TBMs that is turning them the most competitive drilling solution for the majority of modern tunnels.

Furthermore, the particularities of this construction method generate serious questions about its structural response that have to be progressively answered in order to optimize its design and to

obtain the maximum safety at minimum cost. The influence of joints configuration, the **ground–structure** interaction or the real load redistribution capacity of the lining between adjacent rings have to be clarified in this particular structure.

Two different experiences in segmental tunnel linings real scale tests had been carried out at TUDelft (Blom, 2002) and in Germany by STUVAtec. Their focus was on the damages occurred during the construction process and the limit of the radial **ovalization** experimented by rings under earth pressures in soft soils conditions. Additionally, these experiences also tried to study the load transferences between rings depending on the longitudinal axial stress remaining in the lining due to TBM jacks action. The load redistribution to adjacent rings should modify the response of the lining subjected to concentrated loads and, therefore, it is necessary to determine its influence in the actual structural capacity of the lining. These tests only considered the load transfer on a set of three rings due to the extreme difficulties to test more rings. Both tests were performed in labs, not having the possibility to take into account the actual **soil–structure** interaction and the behavior of more than three rings. Soil reaction was simulated by means of a complex system of radial hydraulic jacks.

A ring is composed by multiple segments defining a multiple-hinged structure and, consequently, the reaction of the ground is critical to keep the stability of the system. However, the bond existing between the lining and the ground constraints both the ra-

* Corresponding author. Tel.: +34 934017349; fax: +34 934054135.

E-mail addresses: climent.molins@upc.edu (C. Molins), oriol.arnau@upc.edu (O. Arnau).

dial and the tangential deformations, which results in reaction stresses in the ground (Blom et al., 1999). The load transference from the lining to the ground influences the stress state of the structure and, in consequence, its response.

To overcome the previous limitations, an experimental section of 15 rings was constructed inside the new L9 metro project in Barcelona using steel fibers as a unique reinforcement for concrete. The main objectives of this section were (1) to develop an *in situ* loading test that allows the characterization of the segmental tunnel linings behavior in hard ground conditions and (2) to determine the feasibility of using steel fibers as a unique reinforcement for concrete segments.

The use of steel fibers in segmental tunnel linings has significantly increased during last years with the aim of limiting the local spalling damage that often occurs in the corners of the segments. Nowadays, few authors have intended to determine the contribution of these fibers to the structural capacity of the segmental tunnel linings in order to reduce the amount of necessary conventional reinforcement (de Waal, 2000; Plizzari and Tiberti, 2006; Kasper et al., 2007; Tiberti et al., 2008). The *in situ* test of SFRC lining was an excellent opportunity to obtain significant information about it.

This paper describes the configuration and performance of the *in situ* test and analyzes the results obtained on it, involving a direct contribution to the knowledge of the structural response of segmental tunnel linings. The characteristics of the experimental section, the test set-up and its development are intensively explained. Finally, a selection of the most significant results is presented and analyzed in detail to highlight main conclusions.

2. Experimental

2.1. General aspects of the Line 9 project

Line 9 (L9) is an ambitious project for the construction of a new subway line in Barcelona. Designed as a bypass along the north part of the city and to link the most characteristic points of the neighboring towns, L9 comprises a total length of 47.8 km and 52 stations entailing an investment of more than 6.000 M€.

The line runs under a high density populated area that is often irregularly urbanized, making more difficult the surface works. Out of the deltas of the rivers that bound Barcelona, the uneven topography of the city provides high local tunnel overburdens, achieving a maximum close to 90 m. In consequence, the majority of the tunnel linings were planned to be constructed by means of different TBM's, three of diameter ($\emptyset$) equal to 12 m and two of $\emptyset$ 9.4 m. The construction of one large diameter tunnel was selected in deep sections instead of the twin tube option due to its best economical commitment tunnel-shaft to allow the allocation of the station inside the tunnel.

An experimental section of 15 rings was constructed inside the section 4a of Line 9. Section 4a is located between the city of Santa Coloma de Gramanet and the Besòs River and is characterized for its record in slenderness – ratio between the internal diameter ($\emptyset_{int}$) of the lining and its thickness (t), $\emptyset_{int}/t$ – of 31.1. The rings of this section are composed by seven segments (A1–A5, B and C) plus one key segment (K) defining a universal ring with a external diameter of 11.6 m, 0.35 m thickness, 1.80 m width and 0.57° conicity (Fig. 1).

This record slenderness (Table 1) increase the interest to study its structural response in order to improve the design and the construction techniques employed for segmental tunnel linings.

The longitudinal joints (between segments of a same ring) present a packing material (packer) consisting of a bituminous sheet of 1750 × 204 mm with 2 mm thickness. The circumferential joints (between adjacent rings) present a set of 30 plastic packers of 785 × 204 mm with 2 mm thickness.

Fig. 1. Ring geometry. Dimensions in mm.

Table 1
Most slender tunnels already constructed in the world.

Tunnel project	$\emptyset_{int}$ (mm)	Thickness (mm)	Slenderness ($\emptyset_{int}/e$)
Section 4a. L9 of Barcelona	10,900	350	31.1
Dublin Port Tunnel	10,840	350	31.0
Heathrow Baggage Handling	4500	150	30.0
Section 4b. L9 of Barcelona	10,900	400	27.3
L8 and L9 of Madrid Subway	6700	250	26.8

Table 2
Concrete mix.

Component	Weight (kg/m ³)
Cement CEM I 52.5R	410
Washed sand 0–5 mm	580
Granitic sand 0–5 mm	333
Gravel 5–12 mm	410
Gravel 12–20 mm	595
Water	115
Super plasticizer Glenium TC1332	3
Steel fibers	60

Table 3
Steel fibers characteristics.

Content	60 kg/m ³
Diameter ($\emptyset_f$)	1 mm
Aspect ratio (fiber length/ $\emptyset_f$)	50
Tensile strength	1100 N/mm ²
Geometry	Hooked end

2.2. Material properties

The rings of the experimental section were constructed using 60 kg/m³ of steel fibers as unique reinforcement instead of the 60 kg/m³ of conventional rebars plus the 30 kg/m³ of steel fibers that was the normal reinforcement of that lining. Only a slight amount of reinforcement bars were placed in some segments to hold the internal instruments necessary to monitor the test. Con-

Fig. 2. Two hydraulic flat jacks on a segment (a) and its conceptual work scheme (b).

Fig. 3. Selected configuration of the concentrated loads for the test.

crete mix to achieve the design strength ($f_{ck} = 50 \text{ N/mm}^2$) and the main characteristics of the fibers are shown in Tables 2 and 3.

2.3. Ground properties

Section 4a runs inside a granodiorite rock formation that presents different alteration levels depending on the local tunnel depth. The experimental section is located 33.5 m below the surface inside a low alteration granodiorite which presents a high modulus of elasticity ($11,225 \text{ N/mm}^2$). The groundwater table is located 18.6 m over the tunnel axis, producing significant hydraulic pressures over the structure.

3. Test set-up

3.1. Test configuration

As aforementioned, the main limitation presented by previous full scale test experiences was the realistic consideration of the ground response against the ring deformation. After the analysis

Fig. 4. Extensometers (a) and load cells (b) employed in the *in situ* test.

Table 4

Description of the internal instruments placed in the experimental section.

Ring	Position of K segment	Segment	Extensometers	Load cells	Flat jacks	Casting date	Placing date
1836	7	A2	12	1		14/04/2004	23/02/2005
1837	2	A4	12	1		30/03/2004	23/02/2005
1838	11	A1	14	2		01/04/2004	24/02/2005
		A2	14	2	2	07/04/2004	24/02/2005
		A3	14	2	1	01/04/2004	24/02/2005
		A4	14	2		02/04/2004	24/02/2005
		A5	14	2		05/04/2004	24/02/2005
		B	14	2		02/04/2004	24/02/2005
		C	14	2		02/04/2004	24/02/2005
		K	4	0		02/04/2004	24/02/2005
1839	1	A3	12	1		14/04/2004	24/02/2005
1840	8	A4	12	1		30/03/2004	24/02/2005

Fig. 5. Vertical transducers in the tunnel cross section (a) and 2D transducers in joints (b).

Table 5
SFRC compression test results.

Description	Compression test
Number of specimens	36
Average value	54.13
Typical deviation	2.90
Variance (σ^2)	8.47
Minimum value	47.57
Maximum value	60.20
Coefficient of variation	5.37%

Table 6
Average results obtained in NBN SFRC test set.

F_{f300} (28 days)	N/mm ²
f_r	6.67 N/mm ²

surrounding hardened grout and ground (Fig. 2b). The employed jacks present a maximum nominal force of 1700 kN and a maximum travel of 25 mm. Their external diameter was $\varnothing_{ext} = 460$ mm and the pressure plate diameter was $\varnothing_{pres} = 365$ mm. All jacks were placed over $\varnothing 700$ mm and 30 mm thickness steel plate (Fig. 2b) in order to achieve a better distribution of the applied force into the segment. The selected configuration was based on three concentrated loads located in the tunnel crown and distributed symmetrically, encompassing a total sector of 48° (Fig. 3).

3.2. Prediction of maximum loads

Two different numerical analyses were previously carried out to predict the behavior of the segmental lining during the test and to determine the maximum load per jack that should provide an advanced crack status without any risk: (1) individual ring model

of different laboratory options, it was concluded that the unique way to obtain the real segmental tunnel rings behavior was to perform an *in situ* test in the actual work conditions of the tunnel.

The conceptual definition of the test was based on applying radial loads close to the tunnel crown and the acquisition of the rings deformations and displacements by means of a complete set of instruments. The first difficulty of the *in situ* test was to decide how to generate external loads over a ring definitively placed inside the tunnel. Different options were analyzed selecting the use of hydraulic flat jacks that were previously embedded at the extrados of the tested ring (Fig. 2a) which apply the force against the

Fig. 6. NBN flexural test scheme (a) and the average behavior obtained in L9 SFRC (b).

Fig. 7. Designed and real positions of the hydraulic flat jacks.

Fig. 8. Load diagram performed in the *in situ* test.

Table 7
Load phases applied during the *in situ* test.

Phase	Stage	Active jacks	Load/jack (kN)
0	1	Jack 1 + Jack 2 + Jack 3	100
1	2	Jack 1 + Jack 2	500
	3	Jack 1	500
	4	Jack 2	500
2	5	Jack 1 + Jack 2	1500
	6	Jack 1	1500
	7	Jack 2	1500

considering the non-linear behavior of concrete in both tension and compression and (2) a 3D shell elements model that considers the longitudinal load redistribution through the ring joints (Molins et al., 2005). The maximum load per jack was fixed in 1500 kN.

3.3. Monitoring

Five of fifteen rings that compose the experimental section of SFRC were intensively instrumented internally and externally. The internal instruments of every segment of the loaded ring (numbered as 1838) was composed by fourteen extensometers (Fig. 4a), twelve measuring the circumferential strain (placed by pairs to allow the curvature measurement of different cross sections) and two measuring the longitudinal strain. Two load cells (Fig. 4b) were placed at $\frac{1}{4}$ and $\frac{3}{4}$ of the length of each segment.

Every one of the four rings adjacent to the loaded one (numbered as 1836, 1837, 1839 and 1840) contains one segment with twelve extensometers circumferentially placed (using the same configuration presented for the loaded ring segments) and one load cell centered at the segment extrados side. Table 4 summarizes the internal instrumentation placed in the experimental section and the dates of casting and placing.

The external instruments consist of two different configurations of displacement transducers. In order to obtain the movements of

the rings in the tunnel crown zone, displacement transducers were vertically placed across the tunnel section (Fig. 5a).

Eight sets of two transducers were placed in every longitudinal joint of the loaded ring to measure the circumferential joint movement at both sides. Four transducers were also placed in the crown zone of the loaded ring to obtain, with the combination of the longitudinal joints transducers, 2D measurements of the rings movements (Fig. 5b).

The relative longitudinal displacements between adjacent rings were registered by means of 24 transducers placed in the circumferential joints of the instrumented rings.

3.4. Control and characterization of segments material

To control and characterize the mechanical properties of SFRC employed on the segments of the experimental section, four different tests were carried out:

- (1) Compression test on cylindrical 300×150 mm specimens to control the compressive strength (results are presented in Table 5).
- (2) Elasticity modulus test, with an average value of $38,700 \text{ N/mm}^2$.
- (3) The characterization of the SFRC flexural response was carried out by means of a set of twelve 4-point beam tests that were performed according to NBN regulations (IBN, 1992) (Fig. 6a). Tests were performed over prismatic specimens $600 \times 150 \times 150$. The main average results are presented in Table 6 and the mean load-deflection diagram obtained is shown in Fig. 6b.
- (4) Additionally, the Barcelona test (Molins et al., 2009) was employed to control the SFRC post cracking properties in real segments. For that purpose, several cores were directly drilled on segments that were never installed. This work allows the evaluation of the resistance differences presented by SFRC depending on the segments zones and bending plane caused by fiber distribution and orientation (Mora, 2008).

Fig. 9. Schematic representation of the movements and external pressures at stage 5.

Fig. 10. Local arch mechanism originated by the force applied by jack 2.

4. Development

Previously to the test performance, it was necessary to establish the actual state of the experimental section. For that purpose, a complete check-out was carried out covering: (1) a topographic

survey of the instrumented rings, (2) the measurement of the joints lags, (3) the determination of the final position of the hydraulic jacks and the instrumentation devices and (4) a complete description of the cracks on the segments caused by the manufacturing, transport and installation processes besides the initial groundwater and backfill grout pressures. Despite the accurate

Fig. 11. Lining-ground tangential force transmission mechanism.

Fig. 12. Experimental increments of axial stresses reached in loading stage 5 (in N/mm^2) and analytical distributions varying tangential stiffness.

definition of the test carried out prior to segments manufacturing, some significant differences were observed in the hydraulic jacks

position (Fig. 7), forcing new analyses to contrast the suitability of the maximum selected load per jack. Furthermore, the eccentric-

Table 8

Values of the parameters employed to calculate the rotational stiffness of joints.

Parameter	Value
Width of segments (b)	1800 mm
Height of joint (l_j)	204 mm
Concrete modulus of elasticity (E_c)	38,700 N/mm ²

Table 9Vertical deflections at the tunnel crown during the *in situ* test.

Phase	Stage	Deflection (mm)	Remaining deflection (mm)
1	2	0.537	0.231 (43%)
	4	0.543	0.267 (49%)
2	5	2.627	0.840 (32%)
	7	3.076	1.220 (39%)

Fig. 14. L9 ring joint packer tangential behavior subjected to a compressive stress (σ_c) of 15 N/mm² (Cavalero, 2009).

ity of the jacks in reference to the middle axis in the width of the ring (located at 90 cm of the ring edges) should produce an asymmetry in the ring behavior that had to be taken in consideration.

4.1. Loading process

The test started on Tuesday 13th of December 2005 and finished on Friday 16th. In the initial phase (numbered as 0) a load of 100 kN per jack was applied to all jacks and maintained along 18 h to compensate the relaxation experimented by the hydraulic pressure. The first stage of phase 1 consisted in the achievement of 500 kN per jack in all jacks, but when the pressure was close to 26 bar (260 kN), the jack number 3 suddenly failed, leaving two active jacks for the rest of the test. Therefore, phase 1 was reduced to 500 kN per jack acting simultaneously in jacks 1 and 2 and maintaining it during 14 h (Fig. 8). At that moment, it was necessary to quickly determine the suitability of continuing with the designed test plan. In consequence, a set of non-linear analyses were carried out according to the new test configuration, concluding that the initially planned load procedure could go ahead without significant additional risks. Finally, the same load of 500 kN was applied to every jack individually to conclude phase 1. The same procedure was employed for phase 2 but achieving a load of 1500 kN/jack and reducing the maintenance time of the simultaneous load. The load appliance was finally divided in seven differ-

ent stages that are detailed in Table 7. The failure of jack 3 produced reductions on the maximum applied force and in the planned loading scenarios. Despite that, the remaining load capacity was enough to produce in the lining significant movements, pressures, strains and damages (cracking) to adequately analyze the structural response of the lining.

5. Results

The instrumentation placed in the experimental section provides a large amount of measurements that require an enormous work of selection and treatment to be presented in a clear and intelligible form. Fig. 9 shows the schematic representation adopted for the ring movements and pressures for the loaded ring at stage 5, corresponding to the maximum load applied on the ring (1500 kN/jack). Load cells provide important information about the zone where the structure increase/decrease its pressure against the surrounding ground, thus supplying evidences of the ring resistance mechanisms and the ground-structure interaction. The joints movement measurements provide information about the joints behavior that can help to understand its involvement on the ring structural response. Schematic results of all stages are presented in Appendix A.

Fig. 13. Temporal evolution of jack force and radial displacements between rings at the crown during stages 6 and 7.

Fig. 15. Crack patterns obtained at the end of stage 5. Numbers define the load per jack when the crack or a certain part of it appraised.

5.1. Pressure on the extrados

Pressure distribution in load cells after loading (Fig. 9) shows a reduction of pressure on the upper part of the loaded ring. This reduction results of the loose of contact between the lining and the

ground due to jacks action. It is also possible to notice the increase of pressure where the ground provides reaction against the lining deformation. The extrados pressure did not experience any variation in the lower part of the ring showing that, for the particular conditions of this test, the load only affects the upper part of the structure.

Fig. A3. External instruments measures of the loaded ring at stage 4.

These results show an arch behavior under a concentrated load (F_L) discharging the structure stresses to the ground. It is expected that the reaction zone (space where the ground presents forces F_{GR}) depends on the ground stiffness and the ring geometry due to the displacement concentration in joints (Fig. 10). A stiff ground should provide short arch lengths and small reactions zones whilst soft grounds should provide larger arch lengths and ground reaction zones.

Additionally, for stiff grounds it is also discussed the significance of the lining-ground interaction through tangential mechanisms that transmit the forces directly to the ground (Fig. 11). This mechanism is based on the tangential adherence between the interface formed by the lining, the grout and the ground and affects directly the ring stress-distribution. Consequently, the structural response of the lining is also affected. The increase of radial pressure between the lining and the ground in the ground reaction zone should increase the tangential stress transmission capacity of the interface. This topic was analyzed using results provided by the extensometers embedded in the segments to monitor the circumferential strains.

5.2. Circumferential extensometry

Fig. 12 shows the average values of the axial stress measured in the loaded ring during stage 5 (1500 kN in jacks 1 and 2), obtained after applying the value of the deformation modulus of concrete to the measured increments of strain produced by loading. The maximum increment of compression stress was reached in the tunnel crown and it was gradually lost in a short space out of the jacks zone (segments A1 and B), denoting the axial force transmission from the ring to the ground through tangential mechanisms. Extensometers data process was very demanding. Results had to be carefully analyzed and some of them were disregarded due to the low reliability of their signals. Some interference was proved to be caused by cracks across the instruments.

To assess the results provided by the extensometers, a bar elements structural model of the loaded ring was developed employing the software Robot office. The joints were reproduced by means of lineal hinges, which rotational stiffness (c_r) was estimated by means of the relation (1) (Blom, 2002), where b is the width of the segments, l_t is the height of the joint contact and E_c is the concrete modulus of elasticity. The parameters adopted for its determination are shown in Table 8.

$$c_r = \frac{bl_t^2 E_c}{12} \quad (1)$$

Ground-structure interaction was modeled according to Winkler hypothesis by means of radial and tangential springs. In order to consider the loose of contact in the loading zone, a non-linear stiffness was assigned to the radial springs (K_r) according to relation (2), which corresponds to the analytical solution of a circular tunnel in elastic ground. A ground modulus of elasticity (E_s) of 11,225 N/mm² and a Poisson ratio (ν) of 0.24 were employed whilst R corresponds to the lining external radius. Initially, the tangential spring stiffness (K_t) was assumed according to the relation proposed by Plizzari and Tiberti (2006), fixing the tangential spring stiffness as 1/3 of the radial stiffness (Eq. (3)). Departing on this assumption, different analyses were carried out reducing the tangential stiffness at 50%, 20% and also disregarding it at all. Fig. 12 shows the axial stress distribution assuming the different values of tangential stiffness.

$$K_r = \frac{E_s}{R \cdot (1 + \nu)} \quad (2)$$

$$K_t = \frac{K_r}{3} \quad (3)$$

The full consideration of the tangential stiffness provided the best fitting of the experimental results in both, the maximum stresses reached in the tunnel crown and the stress transference

Fig. A4. External instruments measures of the loaded ring at stage 5.

length out of the jacks' zone. Lower values of tangential stiffness produced a reduction of peak axial stresses and a smoothing of the distribution, obtaining significant stresses at the bottom of the ring. Disregarding the tangential stiffness at all produces a uniform distribution of the axial stress. Therefore, in the experimental section conditions, the tangential force transmission plays a decisive role on the lining stress distribution. The adequate consideration of the tangential mechanisms presents a great importance in the determination of the segmental tunnel linings structural behavior in hard ground conditions.

Despite the significance of the applied loads, the high stiffness of the ground implies low deformations to provide the necessary reaction to equilibrate the loads. In consequence, low deflections were measured on the tunnel crown. Table 9 summarizes the most significant vertical deflections obtained at the tunnel crown during the *in situ* test. A maximum deflection of 3.1 mm was obtained during stage 7, presenting a post-load remaining value of 1.2 mm (39%). As can be observed, the remaining deflections measured in the initial phases are higher than in the last ones. This fact is caused by the non-linear behavior with high remaining deformations presented by the bituminous packer placed at longitudinal joints, which is described on the part 2 of this paper (Arnau and Molins, submitted for publication).

As was previously explained, a ring is composed by various segments defining a multiple-hinged structure. Joints present a lower rotational stiffness than concrete segments caused by the softness of the packing material, the smaller height of the packer sheet (204 mm) and the incapability of the joint to transfer tensile stresses. Additionally, significant cracks appear under jacks positions, diminishing the rotational stiffness of the segment at the load appliance zone and consequently reducing the length of the concrete segment without structural hinges. Therefore, it was expected that the ring displacements were based more on concentrated rotations at joints and cracked sections than in the elastic curvature of the concrete segments.

5.3. Longitudinal redistribution

The structural response of the segmental tunnel linings also depends on the load redistribution capacity from a loaded ring to its adjacent ones. A high redistribution means a response similar to a continuous pipe whilst a low redistribution capacity means a unique ring resistance and the consequent weakness in front of concentrated loads. The study carried out by Liao et al. (2008) concluded that the bending moment in tunnel cross section decreases with the shear stiffness and, therefore, structural measures should be taken to strengthen the shear resistance ability between rings.

The longitudinal redistribution is governed by the shear force transmission capacity between rings that mainly depends on the joint configuration, the deformability of the rings (it also implies the surrounding ground influence) and the construction process. The rings of the experimental section present a flat ring joint configuration using plastic packers to regularize the contact zone and to transmit forces. In consequence, the tangential transmission capacity will be determined by the maximum shear force resisted by the joint before its slipping, which depends of the remaining axial stress in the lining, originally caused by the TBM thrust force during the tunnel construction.

The external transducers placed at the ring joints of the tunnel crown provided significant data about the global structural behavior of the lining. Fig. 13 shows the relative displacements between adjacent rings measured in the tunnel crown during the individual application of the maximum load in jacks 1 and 2 (stages 6 and 7). The action of jack 1 (stage 6) does not produce significant movements in the tunnel crown joints showing the localization of the experimented displacements. Moreover, the action of jack 2, which is much closer to the tunnel crown (Fig. 9), is much more noticeable by the instruments placed at the crown. The measurements show that movements only occurred in the limiting joints of the loaded ring.

Fig. A5. External instruments measures of the loaded ring at stage 6.

The magnitude of the obtained relative displacements (max 3.1 mm) clearly denotes the slipping of the 2 mm packer placed at ring joints. An experimental campaign to determine the maximum tangential stress and displacement for different packers employed in Spanish tunnels was carried out by Cavalaro (2009) at the laboratory of the Universitat Politècnica de Catalunya (UPC). Results for Line 9 ring joint packer showed that for a high normal compressive stress (15 N/mm^2), the maximum displacement before slipping was close to 0.3 mm (Fig. 14).

The joint slipping means that their force transmission capacity is exhausted and therefore the load redistribution to the adjacent segments is clearly limited. Furthermore, the instruments placed in the adjacent rings did not present a significant response against the different loading stages, showing a quasi-individual response of the loaded ring. For the particular conditions of the *in situ* test, it can be considered that no significant stress redistributions occurred from the loaded ring to the rest of the structure caused by the ring interaction.

5.4. Crack patterns

Additionally to the all measurements obtained by the instrumentation, the crack patterns of the segments were meticulously registered as the load increased. Fig. 15, shows the crack patterns of segments A2 and A3 of the loaded ring obtained along the test. The numbers close to cracks are indicators of the stage and load value when the crack (or a significant part of it) was recorded (view table of Fig. 15).

First significant cracks appeared in stage 5 for a load around 900 kN per jack (60% of the total loading). No cracks produced by loading were detected out of segments A2 and A3 where the jacks were located. Fig. 15 neatly shows that cracks in the intrados of the ring were concentrated under jacks' positions. Most cracks are orientated in the longitudinal direction according to the ring arch mechanism, but radial cracks caused by the spreading of the concentrated loads across the section of the ring were also appraised. An advanced crack status was achieved during the test registering a maximum crack opening of 1.31 mm in segment A2 during stage 7.

The obtained crack patterns provide significant information of great interest for the study and modeling of segmental linings constructed with steel fibers as a unique reinforcement for concrete.

6. Conclusions

An innovative technique to perform real scale tests on segmental tunnel linings has been successfully developed and carried out. The *in situ* test provides evidences of the lining behavior on its real work conditions and, therefore, considering all parameters involved in its structural response. This is, as far as known by the authors, the unique approach available up to the present to assess a realistic consideration of the ground-structure interaction. Additionally, the test performance over a lining constructed using steel fibers as unique concrete reinforcement supplies relevant information about the structural contribution of such material to this structural typology.

The study, performance and analysis of the *in situ* test carried out at L9 experimental section provided several conclusions about the structural response of SFRC segmental tunnel linings in hard ground conditions:

- The loaded ring presents a localized arch behavior under concentrated loads. It is assumed that the ground stiffness and the ring geometry mostly determine the length of the arch and the ground reaction zones.
- The tangential ground-structure interaction mechanism presents a decisive influence on the ring stress distribution and, in consequence, on the structural response of the lining. In this case, a tangential stiffness equal to 1/3 of the radial stiffness provides the best fitting approach.
- The deformation of the loaded ring should be mainly caused by concentrated rotations in the ring joints and in the cracked sections under the jacks.
- For the particular conditions of the section 4a of L9 it can be considered that no significant stress transmissions from the loaded ring to the adjacent ones exist.

Fig. A6. External instruments measures of the loaded ring at stage 7.

– The crack patterns are fully consistent with an arch behavior of the loaded ring. Radial cracks were result of the spreading of the load across the section of the ring.

The *in situ* test carried out at the experimental section of L9 provides significant evidences about the structural behavior of this segmental tunnel lining in its particular hard ground conditions. The collected data allows the verification and calibration of the numerical models employed to understand and design segmental tunnel linings. Part 2 of the present paper (Arnau and Molins, submitted for publication) describes the numerical simulation of the *in situ* test. The main phenomena involved in the experimental section structural response and its modeling techniques are described. Analytical results are then compared with the experimental evidences in terms of general response, displacements, joints behavior and crack patterns.

Acknowledgements

The authors thank *Gestió d'Infraestructures, S.A.* (GISA), the public company responsible of the design and construction of the L9 of the Barcelona's subway, for the effort made to provide all the necessary staff and equipment to perform the test and also the construction company FCC Construcción S.A., for its promotion and collaboration in the investigation of the behavior of packers.

Appendix A. Resume of external instruments measures

Figs. A1–A6 show the measurements registered at the external instrumentation of the loaded ring during the different stages of the *in situ* test.

References

- Arnau, O., Molins, C., submitted for publication. Experimental and analytical study of the structural response of segmental tunnel linings based on an *in situ* loading test. Part 2: Numerical simulation. Tunnelling and Underground Space Technology.
- Blom, C.B.M., 2002. Design Philosophy of Concrete Linings for Tunnels in Soft Soils. PhD thesis. Technische Universiteit Delft.
- Blom, C.B.M., van der Horst, E.J., Jovanovic, P.S., 1999. Three-dimensional structural analyses of the shield-driven "Green Heart" tunnel of the high-speed line south. Tunnelling and Underground Space Technology 1 (2), 217–224.
- Cavalero, S.H.P., 2009. Aspectos tecnológicos de túneles construidos con tuneladora y dovelas prefabricadas de hormigón. PhD thesis. Universitat Politècnica de Catalunya (in Spanish).
- de Waal, R.G.A., 2000. Steel Fibre Reinforced Tunnel Segments for the Application in Shield Driven Tunnel Linings. PhD thesis. Technische Universiteit Delft.
- Institut Belge de Normalisation (IBN), 1992. Essais des Bétons Renforcés de Fibres – Essai de flexion sur éprouvettes prismatiques, NBN B 15-238 (in French).
- Kasper, T., Edvardsen, C., Wittneben, G., Neumann, D., 2007. Lining design for the district heating tunnel in Copenhagen with steel fibre reinforced concrete segments. Tunnelling and Underground Space Technology 23, 574–587.
- Liao, S.M., Peng, F.L., Shen, S.L., 2008. Analysis of shearing effect on a tunnel induced by load transfer along longitudinal direction. Tunnelling and Underground Space Technology 23, 421–430.
- Molins, C., Mari, A.R., Aguado, A., 2005. Design of the loading test of the segmental tunnel lining of L9 subway of Barcelona. In: Proceedings of the III ACHE – Congreso de Puentes y Estructuras de Edificación de la Asociación, November 14–17, 2005, Zaragoza, Spain, pp. 893–905 (in Spanish).
- Molins, C., Aguado, A., Saludes, S., 2009. Double Punch test to control the energy dissipation in tension of FRC (Barcelona test). Materials and Structures 42, 415–425.
- Mora, F., 2008. Distribution and Orientation of Fibers in Tunnel Segments, using the Barcelona Test. PhD thesis. Universitat Politècnica de Catalunya (in Spanish).
- Plizzari, G.A., Tiberti, G., 2006. Steel fibers as reinforcement for precast tunnel segments. In: Proceedings of the ITA-AITES 2006 World Tunnel Congress and 32nd ITA General Assembly, 6pp.
- Tiberti, G., Plizzari, G.A., Walraven, J.C., Blom, C.B.M., 2008. Concrete tunnel segments with combined traditional and fiber reinforcement. In: Proceedings of the fib 2008 Symposium, Tailor Made Concrete Structures, Amsterdam, pp. 199–205.