Full-round Numerical Modelling of a Macro Synthetic Fibre Reinforced Tunnel Boring Machine Tunnel in the Shanghai Metro Extension

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ABSTRACT

The authors were engaged to optimise a traditional rebar cage reinforced concrete segmental lining by means of finite element analysis (FEA) using macro synthetic fibres for the Shanghai metro extension. The aim of this study was to reduce or entirely replace the original steel reinforcement of the segmental lining, in order to make the precast production process faster and more economic. The outer diameter of the tunnel was 6200 mm, the thickness of the segments was 350 mm.

The numerical calculations were performed by Atena Finite Element software, which is based on the combination of Menétrey-William and Rankine failure surfaces on the compression and tension sides respectively. The effect of the macro synthetic fibres was modelled as tension stiffening on the tension surface.

A one-to-one full scale model of an assembled tunnel ring using the original reinforcement cages was tested at Tongji University in Shanghai, July 2013. The ring was tested to differing load levels where load and displacement were measured until failure. The author's first step was to calculate the real displacement characteristics of the original reinforced concrete (RC) ring, then calculate the numerically modelled version, so as to compare them. The material parameters of the concrete and rebar used had been determined through basic material tests conducted at the Tongji University. (Bi, Liu and Wang, in press).

The numerical modelling undertaken has shown good correlation with the full-scale physical test results. Optimisation of the lining reinforcement was achieved through the addition of fibres, resulting in a substantial reduction of the steel rebar cages. Several results have been computed with the numerical model using different dosages of macro synthetic fibres, showing that the mass of steel reinforcement could be reduced by 50 and 75 per cent respectively.

INTRODUCTION

Macro synthetic fibre reinforcement is gaining popularity among engineers and designers, not only in pavements and shotcrete, but as primary reinforcing, replacing mesh and/or shear links, thus mitigating the complex rebar detailing.

Macro synthetic fibres were first used in shotcrete in mines and civil tunnels (Bernard *et al*, 2014), then slowly introduced to the precast concrete industry (eg water tanks), infrastructure (eg tramline, road) and the tunnelling industry as primary reinforcing for tunnel segments (Ridout, 2008; Martin, 2012). Today, macro synthetic fibre has a wide range of references in every field of the construction industry.

To reduce or entirely replace steel rebars in a concrete elements with steel fibre already has significant history but with synthetic fibre this is still rather revolutionary. In 2013 the author designed the first synthetic fibre only reinforced concrete grandstand in a sporting stadium in Debrecen, Hungary. This was done without shear links, using only prestressed strands (Kovacs and Juhasz, 2013). The system was verified in a real scale laboratory test and compared with the original steel stirrups design. The results exceeded all expectations. The structure had the same capacity as the one with steel shear stirrups and exhibited the same ductile behaviour.

In framing the development of this technology, a further experiment was conducted with a 28 m long beam using the same solution, without steel stirrups. Within these investigations macro synthetic fibres were compared to steel fibres. The results showed that the two materials had the same capacity and same ductile behaviour at given fibre dosages (Kovacs and Juhasz, in press). As well as improving ductility, macro synthetic fibres have been shown to have a number of other advantages compared to all types of steel reinforcement

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including resistance to corrosion, safe and light to handle and improved economy (Bernard *et al*, 2014).

Today we can design and verify all material properties and any scenario where the beneficial influence of macro synthetic fibres might be significant, by means of finite element analysis (FEA) solutions. The author has focused on a new kind of FEA with an advanced fibre reinforced concrete (FRC) material model, utilising the software suite Atena (Cervenka Consulting). The new FRC material model, called modified fracture energy (Juhasz, 2013), provides a relatively simple way to model FRC.

The work referred to in this paper relates to a typical tunnel boring machine (TBM) tunnel in Shanghai's Metro extension, the object being to optimise the reinforcing solutions using macro synthetic fibres. In this paper, the authors are presenting their solutions in comparison to the physical real scale tests, which were carried out at Tongji University in Shanghai by Professor Lui (Bi, Liu and Wang, in press).

ORIGINAL DESIGN AND LABORATORY TEST

Geometry and reinforcement

This typical Shanghai metro tunnel example has an inner diameter of 5500 mm, an outer diameter of 6200 mm and a wall thickness of 350 mm. One ring is made from six segments. The key and the invert segment have a different geometry and reinforcement whilst the lateral ones are identical. The invert segment has an angle of 84 degrees, the key 16 degrees and the four sides 65 degrees.

The longitudinal length of one segment is 1200 mm. The segments were connected with 400 mm long and 30 mm diameter straight bolts at two points, so the six segments were connected at 12 points. The longitudinal bolts are similar to the circumferential. Only one ring was checked in the laboratory test so the longitudinal connections were not included in the test. The segments were hoisted at two points whereas the key segment was hoisted at a single point only. Geometry and loading configuration can be seen in Figures 1 and 2.

The aspect ratio of the segments, which is the developed length over the lining thickness, computes to 12.2 for the invert segment and 9.5 for the regular lateral ones. Given that a segment aspect ratio not exceeding ten generally provides a safe opportunity for a fibre only solution, the lateral segments present no problem (Winterberg, 2012). However, the invert segment's aspect ratio is over this threshold, so it was decided to use combined steel and synthetic fibre reinforcement for the numerical studies.

The steel reinforcement, as per the original design, altogether amounts to a total of 559 kg per ring. This yields an average reinforcement ratio of 72 kg per cubic metre of concrete for the invert and side segments. This is not a very high degree of reinforcement, however, the other driver in using FRC for segmental linings is the enormous gain in productivity, giving this technology more and more momentum in countries with already high or still soaring labour costs, such as China. Replacing the complex rebar cages of a segment cuts out the time required for cutting and bending, fixing or welding, placing and checking of the position of the cage. Entire replacement by fibres can reduce the segment production cycle time down to 50 per cent, such as in the case of the Hobson Bay project in Auckland, New Zealand, which in return leads to substantial cost savings (Winterberg and Vollmann, 2009).

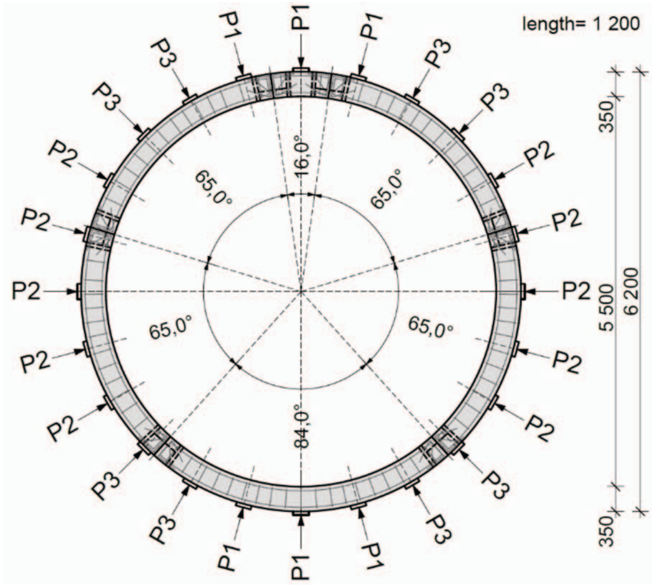

FIG 1 – Geometry, reinforcement and loading configuration of the tunnel.

FIG 2 – One-to-one laboratory test of the tunnel.

Materials

The general tests to characterise the materials were carried out after the full-scale laboratory test. The mean concrete compressive strength was determined to be 50 MPa cubic, the strain at this stress was 1.8‰ (0.0018). A concrete class C40/50 according to the European design standard was used, Eurocode 2, for the finite element analysis. The grade of the steel bar reinforcement was HRB335 with a yield strength of 335 MPa, using ribbed bars.

Full scale laboratory test

Tongji University in Shanghai has carried out a full scale test in their laboratory loading a full segmental ring and measuring the load and the referring displacements. The ring was loaded at 24 points, with hydraulic jacks located every 15 degrees. The load was distributed on the ring by means of transverse beams onto the segments as a line load. This closely spaced, distributed load modelled the loading from the soil under permanent condition. The load configuration and the one to one laboratory test set-up can be seen in Figures 1 and 2.

During loading the 24 loading jacks applied varying levels of load. At the invert and at the crown a load P_1 was acting at three points respectively. At the benches the load P_2 was applied as a function of P_1 and at the walls a load P_3

as a function of P_1 and P_2. Because of the different loads the structure not only experienced different central thrust force, but has undergone eccentricity as well, modelling the real conditions of the tunnel.

The loading phase had two stages. In the first stage the functions of the loads were the following:

$$P_2 = 0.65P_1 \text{ and } P_3 = \frac{P_1 + P_2}{2} = 0.825P_1$$

Load P_1 was increased until P_2 reached its design value, which was 292.5 kN.

In the second stage the load P_2 was unchanged, where load P_3 had the following function:

$$P_3 = \frac{P_1 + P_2}{2}$$

and P_1 was increased until its design value of 455 kN. With these functions it was able to model the loading changes as a function of the tunnel depth below surface.

The displacement was measured at 14 points of the ring at the following angles: 0, 9, 40, 74, 90, 105, 139, 180, 223, 255, 270, 288, 320 and 353 degrees. The most important positions herein were the 0, 180, and the 90, 270 degrees positions, which are measuring the horizontal and vertical displacements. From these results a load-displacement diagram was generated, showing load P_1 over the displacements at the different angles, which can be seen in Figure 3.

FINITE ELEMENT ANALYSIS OF THE TUNNEL

Material tests for input of the finite element analysis

To produce the most realistic calculation, material tests were carried out with different dosages of macro synthetic fibres, using the original concrete mix design at Tongji University. The testing was undertaken with the help of Dr Stefan Bernard (TSE) and Andrew Ridout (EPC). Four point beam tests and round determinate panel (RDP) (ACI C1550) tests were conducted with plain and fibre reinforced concrete.

Four point beam tests were carried out using 150 mm × 150 mm section beams, 550 mm long. The beam was supported on 450 mm span and the load was applied in the third points. The beam was loaded until 4 mm central displacement and the load-displacement diagram was recorded. This generated information about the behaviour of the fibre reinforced concrete, such as flexural strength, post crack strength and ductility. According to the Japanese standard (Japan Society of Civil Engineers, SF-4, 1985) the R_{e3} value could be calculated to provide the measure of ductility, which is the ratio of average post-crack residual load and peak (first crack) load in percentage terms. The material tests for FRC were done with 6 and 10 kg/m³ dosage of macro synthetic fibre. The resultant R_{e3} numbers were 54.6 and 81.6 respectively. Figure 4 shows the test results and the numerical modelling of the beam test.

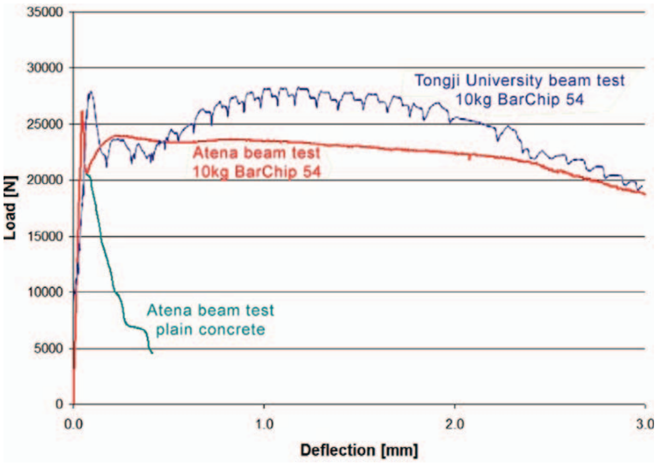

FIG 4 – Typical panel test and beam test configuration.

Material model of concrete and fibre reinforced concrete

After the physical material tests, inverse analysis was carried out and the described fracture energy, as obtained from the fibre performance, was determined for the different fibre dosages. The inverse analysis was carried out by means of a virtual beam test, which is modelling the real beam in the finite element software, yielding the same load and deflection that occurred in physical tests. The inverse analysis iterations were made until the R_{e3} numbers matched those in the physical material tests. The R_{e3} values were 56.7 at

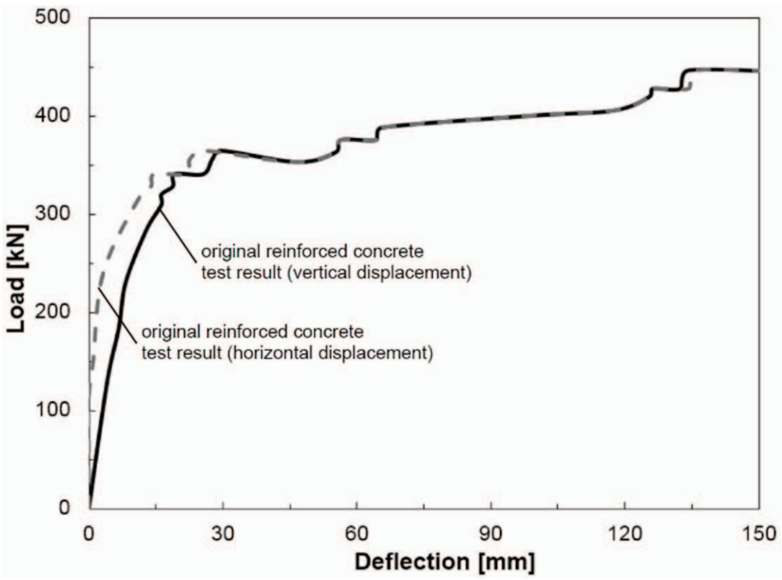

FIG 3 – Load-deflection diagram result of the one-to-one laboratory test.

6 kg/m^3 and 85.1 at 10 kg/m^3 of dosage, respectively. The inverse analysis showed good correlations with the real beam test results, see Figure 4.

Modified fracture energy is a new and simple way to model the behaviour of FRC in tension and bending (Juhasz, 2013). The main idea is to use the concrete fracture energy (G_f) as an initial value and then increase it with an additional fracture energy (G_{ff}) from the post-crack FRC performance (Figure 5). We use a tension stiffening effect on the tensile surfaces of the concrete elements, where the post-crack performance contributed by the fibres defines the added fracture energy (Figure 6). The stress-crack width diagram was limited to 3 mm crack width opening, as a limit from an engineering point of view. However, it never reached this value even in the ultimate limit state (ULS) condition.

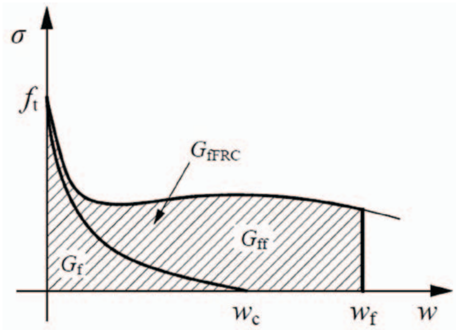

FIG 5 – Fracture energy diagram.

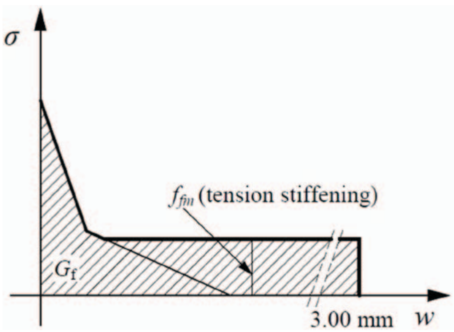

FIG 6 – Tension stiffening diagram.

The concrete was modelled as a three dimensional (3D) brick element with a material model consisting of a combined fracture-plastic failure surface (Cervenka and Papanikolaou, 2008). Tension is handled herein by a fracture model, based on the classical orthotropic smeared crack formulation and the crack band approach. It employs the Rankine cube failure criterion, and it can be used as a rotated or a fixed crack model. The plasticity model for concrete in compression uses the William-Menétrey failure surface (Menétrey and William, 1995). Changing aggregate interlock is taken into account by a reduction of the shear modulus with growing strain, along the crack plane, according to the law derived by Kolmar (Kolmar, 1986).

The concrete has a stress-strain diagram according to Eurocode 2. The crack width was calculated from the stress-crack width diagram, determined by means of inverse analysis, with the help of the characteristic length, which is a function of the size of the element and the angle of the crack within the element. This method is the only one that could realistically represent the cracks in the quasi-brittle material. This is the main advantage of this advanced material model. The failure surface can be seen in Figure 7.

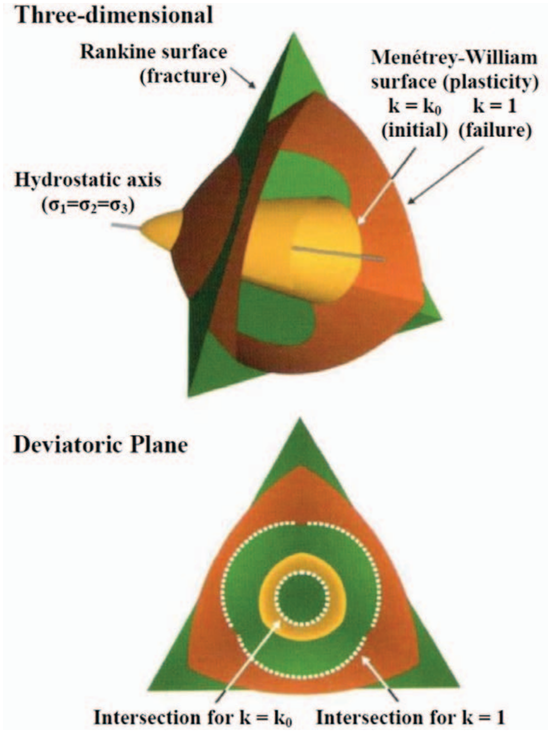

FIG 7 – Failure surfaces of the concrete.

Other materials for finite element analysis

Steel rebars and bolts were modelled as discrete link elements with a uniaxial ideal elastic-plastic stress-strain material model. The rebars link element was connected to every single concrete brick element which was crossed. The bolts had no connection with the concrete brick elements, however, at both ends they were held by the nuts on the concrete surface, which were only able to undergo tension.

The connection surface of two adjacent segments was connected with an interface material, which could hold compression only through friction on the surface. With this special interface material the connections of the segments were modelled, which could be open or closed for bending, where tension would be held by the connection bolts. The connection surface and the material model of the interface material can be seen in Figure 8.

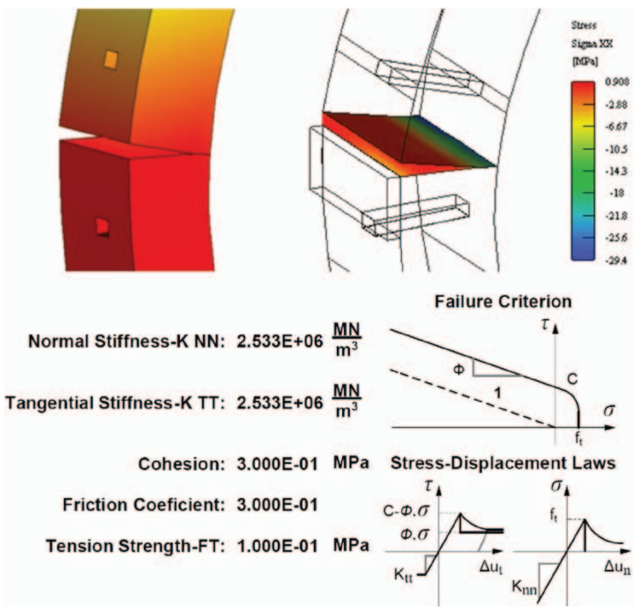

FIG 8 – Connection surface and interface material model.

Numerical model of the tunnel

After defining the accurate material model the geometry was defined. Firstly, the concrete and reinforcement and then the loads and supports must be defined. The tunnel is symmetric at the horizontal and the vertical axis, so only a quarter of the full ring geometry is sufficient to model the structure with symmetrical support conditions on the symmetrical plane. This also helps to define the boundary conditions and makes the calculation faster. Finally, the monitoring points need to be defined, where the load and resulting displacements were to be measured. The loads were positioned at exactly the same locations and with the same values as in the full scale laboratory test.

Calculation of the finite element analysis

The FEA was solved with the Newton-Raphson method. The lack of this method, being unable to handle load drops after crack localisation, was overcome by the fact that explicit load drops did not occur during the trials (Figure 3). The calculation took 4.88 GB memory and solved millions of equations in approximately 1000 steps, taking 12 hours until the results were available. The full model comprised 6300 elements, although only a quarter of the ring was taken into account.

RESULTS OF THE FINITE ELEMENT ANALYSIS

Result of the original reinforced concrete solution

After running the FEA the results were checked. Most important was the load-displacement diagram, which was compared to the full scale laboratory result. It can be seen in Figure 9 that the result of the laboratory test and the result of the FEA are similar in both characteristic and values and show the same maximum load capacity.

The maximum compressive stress of the concrete was 35.9 MPa, while the maximum crack width just before complete failure was 5.0 mm. The steel bars were grouped according to the stress-levels experienced and selection was based on the ones that could be said to be not providing any input and which could be reduced or completely omitted as a first step. According to the computed stress levels, the remaining steel bars could be reduced in diameter in accordance with the computed stress value. Full animation could be done and the entire loading process could be observed from the formation of the crack until total failure. In this way the weak points of the tunnel and the type of failure could be visualised easily. For this type of tunnel and its loading, the mechanism of failure was at the segment joints where the connecting bolts burst out from the concrete, see Figure 10.

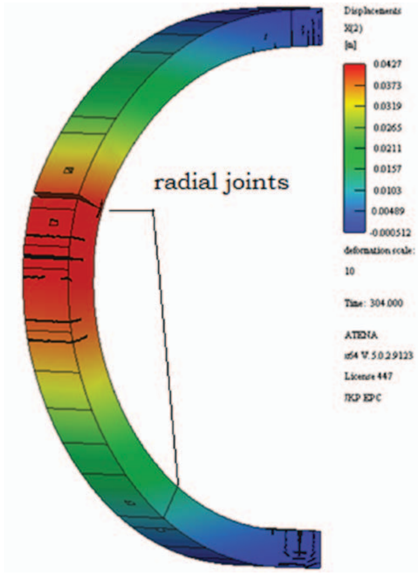

FIG 10 – Horizontal displacement and crack configuration at failure (ultimate limit state).

Optimising the segmental lining with synthetic fibre reinforcement

After the successful modelling of the original RC ring, the optimisation could be started, re-calculating with reduced steel bars and added fibre. Firstly, the lowest stress-level steel bars were omitted and replaced with a moderate dosage of fibre. Then, with increasing fibre dosage, more and more steel bars were omitted. These calculation processes were laborious as adding fibre also changes the occurring deformations, thereby changing the arising eccentricity, and eventually, can change the failure mode, too. However, the ruling failure mode always was radial joint bursting at the bolt pockets.

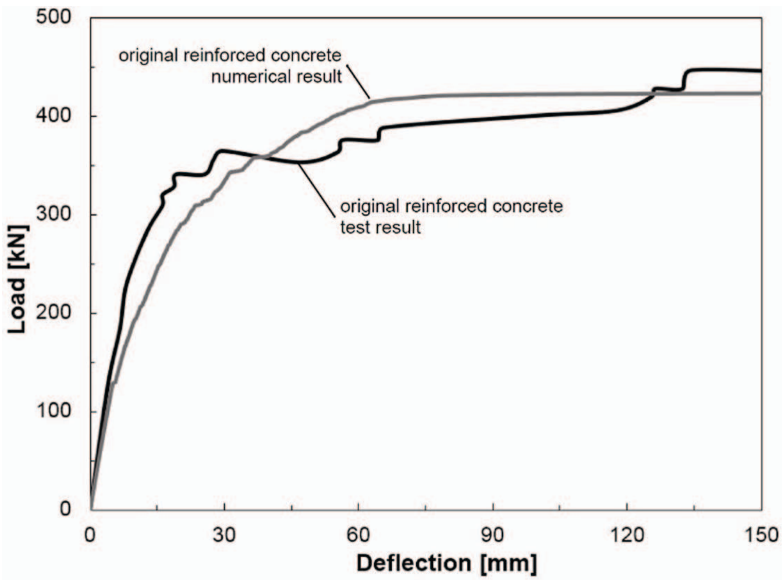

FIG 9 – Load-deflection diagram result of the original RC solution.

Optimised results

After these calculations were made three viable solutions were proposed, which can be taken from Table 1.

Adding fibre in conjunction with steel bar reduction improved control of both crack width and crack propagation. The crack width of the original RC solution was 5.0 mm before total failure, where in solution one this was reduced to only 2.3 mm with less visible cracks. This is a reduction of crack widths of more than 50 per cent, which provides a substantial gain of durability. The results of the calculations with optimised reinforcement can be seen in Figure 10 and 11.

From the calculated solutions, number two seemed to be the most viable. However, the FEA is valid only for this given situation, where for other conditions more parameters would need to be checked. The characteristic failure mode occurred at the radial joints (connection bolts and their surrounding area) so review and redesigning of this part could lead to a more optimised solution. The final recommendation is 7 kg/m^3 macro synthetic fibre in conjunction with 50 per cent steel rebar reduction. This solution is planned to be verified by physical laboratory testing in the near future.

CONCLUSIONS

A full-scale TBM segmental tunnel ring was modelled with an advanced material model in FEA. By using the modified fracture energy method an accurate material model can be used for FRC. A typical Shanghai Metro extension TBM tunnel segmental ring was tested to full scale at the Tongji University, Shanghai, and the load-displacements results were compared with the FEA. The similarity was deemed appropriate, so the model has been justified. Low-stressed steel bars within the section were reduced and macro synthetic fibre was added to the structure. By using macro synthetic fibre reinforcement the volume of steel bars could be reduced. This, in return, leads to significant cost savings in both material and labour, as well as a reduction of the production cycle times.

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TABLE 1
Optimised solutions of the tunnel.

	Original solution	Solution 1	Solution 2	Solution 3
Added fibre (kg/m^3)	-	6	7	10
Maximum crack width (mm)	5.0	2.3	2.1	1.9
Bottom segment (kg)	134.7	88.1	61.3	22
Side segment (kg)	95.9	56.1	49.5	28.6
Key segment (kg)	40.7	31.8	23.1	17.1
Total tunnel	559 kg steel	344.3 kg steel (-39.8%) +46.3 kg fibre	282.4 kg steel (-49.5%) +54.0 kg fibre	140.4 kg steel (-75.0%) +77.2 kg fibre

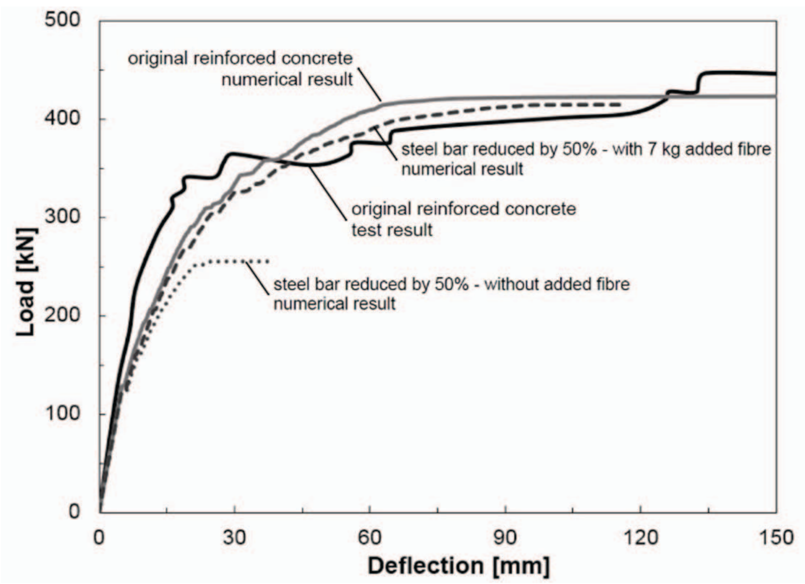

FIG 11 – Load-deflection diagram result of optimised solutions.

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