Use of Steel Fibres in Precast Segment Linings – The Hobson Bay Sewer Tunnel Experience

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ABSTRACT

Steel fibre reinforced shotcrete for the lining of traditionally driven tunnels has developed to a state-of-the-art. However, steel fibres as primary reinforcement also gain more and more significance in mechanised tunnelling with tunnel boring machines (TBM). Replacing conventional reinforcement by steel fibres offers a considerable potential on cost and time savings in the production of precast concrete segments. This paper reports on the successful use of steel fibres in the Hobson Bay sewer tunnel project in Auckland. It was the first earth pressure balance TBM excavated tunnel in New Zealand. The most important aspects on the application and the design of steel fibre reinforced concrete for segmental linings are presented.

INTRODUCTION

Until today, precast concrete segments are usually made with conventional reinforcement. Those typically welded cages require a serious amount of manual labour. Further, a sufficient concrete cover must be provided with regards to function and durability of the segments. Accordingly, the unprotected edges and corners are often subject to spalling during stocking, transport and installation. Steel fibre reinforced concrete (SFRC) can have the same load bearing capacity as ordinary reinforcement, especially in typical tunnel situations with relatively low bending moments due to the high compressive stresses. SFRC also shows a higher degree of tightness, because the cracking behaviour is markedly improved, leading to finer crack widths [1, 2]. The addition of steel fibres and the resulting higher ductility and quality, however, is gaining increasing attention among designers and manufacturers since in this way the curved reinforcement in the segments can be partly or even entirely substituted by steel fibres [3, 4]. This allows for considerable savings in the production by cutting down the manual labour and hence, the production time. Furthermore, the increased robustness that is provided by the steel fibres leads to reduced rejects.

Steel fibres are in particular used for increasing the load bearing capacity of concrete and reinforced concrete structures. Apart from this, special polypropylene multifilament fibres are used for passive fire protection, since they can markedly reduce or entirely prevent concrete spalling [5, 6].

MECHANISED TUNNELING WITH SFRC

Segmental tunnel linings usually are final linings, i.e. they represent the final structure. The technology and the process engineering of the tunnel drive are dependent on the geological conditions. Soft soils and lose rock require a different methodology than TBM drives in hard rock. The principle of securing cavities markedly differs from the methods used in hard rock. Based on the fact that loose rock soils are unstable, tunnel driving in these areas must be conducted with a tunnelling shield that quasi-temporarily protects men, machine and materials from collapsing soil. The process technology applied for excavating the soil will depend on the geological characteristics of the soil and, accordingly, the complexity of the work required for supporting the tunnel face. The final permanent support – which is generally provided by segmental rings – is likewise constructed under the protection of the shield. Such a ring usually consists of 4 to 8 segments, depending on the diameter of the tunnel, and the additional keystone required by the prevailing geometrical and technological situation. In the final state this will be generally a single-shell construction, since the tunnel boring machine will carry along the shield, leaving behind the segmental ring that has been installed in the soil. Due to the typical load combinations in hard rock, where additionally superimpositions of soil and water pressure must be considered, predominantly combinations from bending and normal forces are the ruling loads for the design. Here, the loads from the construction state are set equal to those from permanent loading, i.e. temporary and final lining are designed basically for the same forces. Potential influences from the process engineering are not respected. The usually dominant normal forces keep the cross section under compression or the bending moments low respectively. This results in relatively little stresses that can be easily taken by the steel fibres. The tensile capacity of SFRC correlates strongly with the tensile strength of the binder matrix [7]. Hence, there are natural load limits for the application of SFRC.

A significant distinction must be made with regards to the required load bearing capacity of the precast segments in lose rock. From the structural design aspect there is a difference between the
construction state and the final state of the structure. In the chain from the manufacture of the segments to the installation and the final state after grouting, every loading situation must be precisely recorded and checked. Figure 1 shows the given loading states and the corresponding checks.

In the construction state, the segments are initially not subjected to external loads, being mechanically brought into position by an erector under the protection of the TBM shield. The only loadings that the precast segments experience prior to installation of the rings are those from self-weight, transport and stacking. Stacking is the rule on the typically highly restricted spatial situation on tunnel construction sites and/or precast plants.

Shortly after installation, the segments are exposed to the full jacking forces applied by the hydraulic jacks of the shield machine. These jacks push the machine forward, shoring against the last completed ring in the subsoil and can generate massive problems for the precast segments, particularly in case of imperfections. Such can derive from inaccurate installation or imprecise manufacture of the segments for example. This results in unpredictable load effects on the segments and can often lead to splitting cracks in or near joints.

The segments are exposed to an additional load component when driven out of the shield as here the grouting of the cavity in the tail sets in. The gap behind the shield between soil and lining, which results from process-related reasons, is filled by injection and/or grouting. This process creates an additional load that is imposed on the segments as external load. Once a segment has left the shield and the grouting zone, only external loads from soil, traffic and, possibly, groundwater continue to act on the tunnel as permanent loading. It is process-specific that loads acting in the axis of the tunnel are imposed within the scope of the jacking situation; in contrast to the final loads that act on the structure nearly exclusively transversely to the direction of jacking.

Steel fibres can be used in segmental lining, on the one hand to reduce or even substitute the ordinary reinforcement and to manufacture a concrete of water proof quality, i.e. for increased tightness requirements. On the other hand, fibre reinforced concretes help to reduce risks deriving from the indicated unexpected load combinations. Spalling resulting from impacts during handling of the segments or during transportation can be considerably reduced by the use of steel fibres. The resulting load configuration and/or combination produced by high temporary and permanent loads, which are partly vertically superimposed, as briefly described here, must however be accommodated by the structural design. When the use of fibres in this sense is possible without any restriction, the reduced reinforcement – compared to the reinforcement required for conventionally reinforced concrete segments – may in some cases enable a clearly more cost-efficient series production.

HOBSON BAY SEWER TUNNEL, AUCKLAND
Since more than 90 years a reinforced concrete sewer pipe has been gracing the Hobson Bay near Auckland in New Zealand (Figure 3). Not only the location and the appearance of this structure, but also its deteriorating state led to the decision to replace this viaduct with a tunnel underneath the bay. This would also open up the bay for recreational purposes and other uses.

For this, a tunnel of three kilometres length with an internal diameter of 3.70m was driven from the...
pumping station in Orakei to Logan Terrace in Parnell (Figure 4). The tunnel lining consists of precast segments made of pure steel fibre reinforced concrete. The tunnel runs up to a depth of 40m underneath the Hobson Bay and to a 95m depth below the Orakai Ridge.

Fig. 4: Course of the new sewer tunnel (blue) and the existing viaduct (grey)

MAIN PARTIES OF THE PROJECT
The major companies involved in the project with a cost volume of some 119 million New Zealand Dollars are the following:
- Client: Watercare Services Ltd. (New Zealand’s largest company for potable water and sewage)
- Contractor: McConnell Dowell Constructors Ltd. (design-and-build for the tunnel), Fletcher Construction Company Ltd. (for the shafts)
- Tunnel segment concept and lead segment design: Babendererde Ingenieure GmbH, Bad Schwartau, Germany
- Segment design and precast segment supplier: Wilson Tunnelling Ltd., Auckland
- Steel fibre manufacturer: Officine Maccaferri S.p.A., Bologna, Italy

GEOMETRY OF THE TUNNEL AND THE SEGMENTS
- Tunnel length: 3.0km
- Internal diameter: 3.70m
- Wall thickness: 250mm
- Ring setup: 4 + 2 segments
- Segment slenderness: 8.3 (rhombic) 9.6 (trapezoidal)
- Ring width: 1.00...1.20m
- Erector points: 1 (central)

The ring consists of four rhombic and two trapezoidal segments (Figure 5). Each two rhombic segments follows a trapezoidal segment in the ring setup. The socket for the centrally located erector point of the segments also served as inlet port for grouting.

The slenderness of the segments was designed to be in a range so that the tensile splitting forces, which occur during ram thrusting, are not critical with regards to concrete spalling between the jacks. Such spalling effects are more likely to occur at higher slenderness from approx. 10 in large bore tunnels [8].

Fig. 5: 3D view on the segmental ring setup

STEEL FIBRES REPLACE ORDINARY REINFORCEMENT
The initial design proposal provided a conventional reinforcement with welded rebars for the precast segments (120kg per segment). To improve the process engineering and coevally to reduce the costs in production, Wilson Tunnelling Ltd. had developed the concept to reinforce the segments with steel fibres instead of welded ordinary reinforcement. This concept was adopted in the segment lead design by the German engineers Babendererde Ingenieure GmbH, which provided for manufacturing the segments merely of steel fibre reinforced concrete. The required calculations and structural analyses were prepared and checked in cooperation with Arup Australia [9]. The moment-normal force capacity envelopes for the steel fibre reinforced concrete were based on the design parameters given in DBV [10] and RILEM [11]. An example for the capacity limit curves and relevant loading cases is given in Figure 6:

Fig. 6: Capacity limit curves for SFRC and ruling loading cases [9]
These load bearing curves of the steel fibre reinforced concrete were based on the design values as shown in Table 1 to be determined and verified in tests on bending beams.

### RILEM recommendation

- Characteristic value of the limit of proportionality (LOP), $f_{\text{LOP}} = 7.0 \text{ N/mm}^2$
- Residual tensile strength at CMOD$_{1}$ = 0.5mm (midspan $\delta_{\text{LOP,1}} = 0.46\text{mm}$), $f_{\text{R,1}} = 3.5 \text{ N/mm}^2$
- Residual tensile strength at CMOD$_{4}$ = 3.5mm (midspan $\delta_{\text{LOP,4}} = 3.00\text{mm}$), $f_{\text{R,4}} = 3.0 \text{ N/mm}^2$

### DBV recommendation

- Characteristic flexural strength, $\beta_{\text{BZ}} = 7.0 \text{ N/mm}^2$
- Mean equivalent flexural strength at $(\delta_2 = \delta_2 + 0.65\text{mm})$, $\beta_{\text{BZ,2,m,g}} = 3.5 \text{ N/mm}^2$
- Mean equivalent flexural strength at $(\delta_3 = \delta_3 + 3.15\text{mm})$, $\beta_{\text{BZ,3,m,g}} = 3.0 \text{ N/mm}^2$
- $\delta_2$: deflection at the peak flexural strength

### Table 1: Design values according to RILEM and DBV as adopted for the design [9]

Both of these specifications distinguish between plain concrete performance (flexural strength or cracking resistance) and post-crack performance that is given by the steel fibres. Moreover, they differentiate the post-crack performance of SFRC for SLS and ULS (serviceability limit state and ultimate limit state, respectively) by reading the test results at different stages of deflection or crack opening respectively.

The choice of steel fibres to be used was determined by comparative suitability tests that have been carried out at the institute of construction materials technology at Ruhr-University of Bochum in Germany. Two steel fibres from different manufacturers have been tested [12]. These are cold drawn steel wire fibres with hooked ends of different geometry:

- **Type 1:** L/D = 60/0.75mm
- **Type 2:** L/D = 50/0.75mm

### Table 2: Steel fibres examined for the project

The concrete used in the tests was composed as exactly as possible with construction materials available in Germany to match the intended mix design as given by the precast plant, which was appointed for this project. This was in order to obtain practice-related results for the project. The designers had determined a concrete strength class of C50/60 for the segments to be installed, with an average target value of 72 MPa compressive strength on cylinders (100/200 mm) after 28 days. In the tests carried out at Ruhr-University an average compressive strength of 86.5 MPa (cubes 150 mm) at a variation coefficient of 4.5% was measured after 28 days, which well correlated with the local target value.

The flexural tensile strength or the limit of proportionality (LOP) according to EN 14651 [13] respectively, has been determined to 8.3 MPa. The variation coefficient ranged here at 4.4%; accordingly, the results are hence very uniform. Within the scope of the bending tests, three fibre contents each were investigated: 30, 40 and 50 kg/m$^3$, equivalent to 0.38, 0.51 and 0.64% vol. The results of the bending tests are presented in Figure 7. The average residual strengths are here shown related to defined crack width openings (CMOD = crack mouth opening displacement). It can be clearly seen that the working capacity of the steel fibre concretes does not exhibit the uniform performance behaviour over the entire duration of the test that is typical for these longer steel fibres. Instead, a more or less marked decline in the sustainable stress is noticeable that sets in as the crack opening widens. The reason for this is the brittleness, i.e. the high concrete strength that leads to the release of very high energies at the time of cracking that must be redistributed on the fibres. Here the fact comes to bear that steel fibre type 2, due to its shorter length at the same wire diameter, has 20% more fibres per unit weight that become effective in the crack. This prevents the individual fibre from beginning to yield or even to snap, possibly leading to a zipper effect in the crack.
The different performance of the fibre types further lies in the design of their anchorage elements. The steel fibre type 1 has a very short and sharp-edged end hook, while type 2 has a longer hook length and at the same time milder angles (see Table 2). The pull-out resistance, which determines the working capacity of a steel fibre in the concrete matrix, is for this reason formed over a larger area and through much less concentrated deviation forces in the curvatures. This leads to a generally more ductile behaviour of the composite, since local concrete failure resulting from concentrated stresses in the area of the end hooks can be prevented in this way.

Markedly, the difference in the performance of the two steel fibres becomes more visible with increasing fibre content so that almost three times the residual strength at a crack mouth opening of 3.5mm can be achieved at a dosage of 50kg/m³, while the other fibre does not even reach the required values (see Table 1). Based on these results, the designers and the contractor selected the steel fibre type 2 (Maccaferri Wirand® FF3) for their project.

**DESIGN AND QUALITY ASPECTS**

Based on the results of the tests and in accordance with the specified design values, dosages of 40kg/m³ for normal and 45kg/m³ for more difficult geological conditions respectively were determined for the steel fibres. The concrete of strength class C50/60 for the sewer tunnel was designed for exposure class XA3 “highly aggressive chemical environment.” In the calculation, consideration was given to the fact that, according to the DBV-Merkblatt [10], a sacrificial layer of 35mm thickness was provided for the steel fibre reinforced concrete that is not considered in the structural analysis. Accordingly, a cross-sectional thickness of only 215mm of steel fibre reinforced concrete is available for the ULS based on which the structural analysis is performed.

The addition and dispersion of the steel fibres during the batching process was optimised by means of an automatic fibre dosage system. The chosen steel fiber supplier have developed a variety of mechanical batching systems for the rapid and unproblematic addition and intermixing of their steel fibres. A robust drum dispenser with a specially designed outlet for highest dosage precision was chosen for this project. The unit was adapted and integrated in the on-going process in the precast plant (Figure 8). The machine is electronically connected with the central batching panel and runs under fully automatic control of the batching plant. Only refilling from big bags must be conducted manually by means of a fork lift.

The mean compressive strength of 50 MPa required for handling of the segments was attained after ten days. After casting, the segments were steam-cured for four hours and transported to the construction site only after three additional days of storage. Apart from the cylinders for controlling the compressive strength, beams were cast during the on-going production of the segments within the scope of quality assurance in order to check the specified post-cracking strengths of the steel fibre concrete. The tests were performed in a lab in Penrith, Australia, since this is the only accredited institute for such complex tests in Oceania [14]. The results for the fibre content of 40kg/m³ are presented in Figure 7b (grey dots at CMOD 0.5 and 3.5mm). The good agreement with the values from the preliminary tests at the Ruhr-University of Bochum validates the results obtained there.

The examination of the beam crack sides after the test showed that the fibres were well distributed. Most had straightened during pull-out where approx. 10% yielded. The evaluation of the failure rate attests to the high quality and the robustness and integrity of the steel fibre reinforced segments: from a total of around 15,000 segments manufactured, only seven were sorted out as defective in the course of the production! Six further segments experienced only light damage during installation. These numbers represent a milestone in segmental lining with precast concrete elements.
TUNNELING THE HOBSON BAY

The Hobson tunnel was driven with a Lovat EPB (earth pressure balance) shield tunnel boring machine. The TBM was lowered segment by segment in May 2008 through the Orakei shaft of 35m depth and assembled in a temporary shunt tunnel. The machine was blessed and named ‘Te Kaha’, literally ‘the strength’ in Maori, before it was lowered down the shaft (Figure 9). Tunnel boring commenced on June 11, 2008 as stipulated in the contract.

After installation of two rings at any one time, the gap between the rings and the soil was grouted while the segments were leaving the shield and the connection with the soil was established. After the start-up phase boring proceeded at a rate of approximately 120 meters per week. For this, up to 150 segments daily were transported to the construction site and lowered through the shaft where they were transported by train to the TBM through the tunnel. After a construction time of approximately eight months, the breakthrough could be celebrated on 05 February 2009. Following completion and commissioning of the pumping station, the old viaduct will be demolished in July 2010 and the Hobson Bay will be released for a new infrastructure.

REFERENCES


[7] Ortu, M., 2000, Rissverhalten und Rotationsvermögen von Stahlfaserbeton für Standsicherheitsuntersuchungen im Tunnelbau, VDI-Verlag, Düsseldorf (Germany)


[12] RUB – Test report, 2007, Comparative bending tests on steel fibre reinforced concretes according to RILEM TC 162-TDF and DIN EN 14651, Lehrstuhl für Baustofftechnik, Ruhr-Universität Bochum (Germany)
