Raw materials
Retaining structures
Gabions
Segment joints

TBM
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A Word on...

Germany’s Raw Materials
Thorsten Diercks

Even after the cessation of hard coal mining in late 2018, Germany remains a raw materials producer, where lignite, potash and salt, ceramic resources, industrial minerals, mineral oil and natural gas are mined. The knowledge acquired from German mining can also be exploited to overcome global raw materials issues.

Raw materials • Germany • Securing resources • Knowledge

Application of Gabion Structures in the Transport Infrastructure
Frank Heimbecher and Lukas Tophoff

Gabion structures are increasingly being used in the transport infrastructure. Especially internal stability and connectors have to be verified by means of system tests simply because no generally valid models are yet available. This article is dealing with the relevant codes of practice currently available and findings derived from tests on internal stability and connectors relating to back-anchoring with geogrids.

Geotechnics • Retaining structures • Gabions • Geogrids • Codes of Practice • Tests

Bridge “Brug van den Azijn” in Belgium – 75° inclined Mechanically Stabilized Earth Walls
Francesco Masola, Giulia Lugli and Michael Arndt

The architects of Bridge “Brug van den Azijn” (Vinegar Bridge) in Antwerp, Belgium, designed a challenging facade. Maccaferri had to adapt the Mechanically Stabilized Earth (MSE) wall system “MacRes System” to meet the challenging requirements.

Geotechnics • Reinforced earth • Bridge • Concrete • Geosynthetics • Belgium

Innovative Resealing of Segment Joints by Means of Injection Needles
Götz Tintelnot

Leaky segment joints can be resealed efficiently directly through the joint by means of an injection drill needle. This article explains the innovative, cost-saving “keyhole technology”.

Tunnelling • Injection technology • Sealing • Resealing • Efficiency • Innovation

Open Spaces: Overcoming Caverns and Voids at France’s Galerie des Janots
Desiree Willis

An open-type, Main Beam TBM manufactured by The Robbins Company is excavating the challenging 2.8 km long drive of the Galerie des Janots in France to improve water supply. The contractor Eiffage Civile engineering and the TBM have to overcome caverns and other challenges.

Tunnelling • TBM • Geology • Cavern • Water supply • France

Composite Pile Roofs as Alternative to conventional Pipe Umbrellas in Tunnelling
Frank von Havranek

Pipe umbrellas have proved their worth in tunnelling for temporary supporting during driving – especially in the critical area involving the advancing wall and working face in the case of rocks with low cohesion, shallow overburdens, tunnel entrances and fault zones. Composite pile roofs made of steel supporting members with small cross-sections and dynamic grouping offer an alternative to conventional pipe umbrellas. The bearing behaviour and the execution of composite pile roofs as well as pros and cons vis-à-vis conventional pipe umbrellas are examined and unclarities discussed.

Tunnelling • Pipe umbrella • Composite pile roof • Modelling • Shotcrete

Innovative Load Distribution Plate for Segments
Arno Korte

When excavations are held up, segments can be subjected to the elements for excessively long periods prior to being installed in the tunnel. A new weather-resistant load distribution plate is intended to prevent damage quite apart from making assembly more efficient and secure catering for more straightforward logistics.

Tunnelling • Segment • Product development • Efficiency • HSE • Building operation

Large Scale Monitoring using BRILLOUIN Optical Fibre Sensor Systems in the Fields of Geotechnics and Mining
Stephan Großwig, Jürgen Glötzl, Maria-Barbara Schaller and Ulrich Weber

The potential of advanced fibre optical Brillouin Distributed Strain and Temperature Sensor (DSTS) based systems for both safety monitoring of infrastructure as well as early visualization of man-made geological hazards such as occur at all historical mining areas has been demonstrated in various projects.

Mining • Tunnelling • Geotechnics • Infrastructure • Monitoring • Measuring technology • Ground movement • Research

EU “Blue Mining” Project – Building a large-scale Test System and Flow Tests for vertical Transport Systems in Deep Sea Mining
Toni Müller, Jort van Wijk and Helmut Mischo

An important goal of the European “Blue Mining” research project was to further develop vertical transport systems for deep sea mining. Findings from case studies, modelling and investigations on lab scale had to be validated under realistic condi-
Towards this end, the TU Bergakademie in conjunction with the IHC MTI B.V., Delft, Netherlands developed a large-scale test system in an abandoned mine shaft and conducted an initial series of flow tests for the vertical transportation of solids/water mixtures. This article deals with the planning, building and trials with the large-scale system, the execution of the tests as well as initial results.

**Mining • Deep sea mining • Research • Flow tests • Vertical transport systems • Measuring technology**

**Mining**

46 Automated Lubrication: Retrofit Pilot Project in a German Quarry

Nikolaus Fecht

The Stone Age has a Future – with this slogan, the quarry company SVA on the Swabian Alb is promoting an industry that is currently booming – not simply because there are so many ongoing road construction projects in Germany. For the project to run smoothly, it relies on its production equipment always being operational, and in turn on its automated lubrication. Bieolomatik’s retrofit pilot project demonstrates how all this works.

Geotechnics • Mining • Quarry • Construction material • Stone and earth • Equipment • Maintenance

**Mining, Tunnelling and Geotechnics**

49 Shafts for Woodsmith Mine in North Yorkshire in the United Kingdom

**GeoResources Team**

In North Yorkshire a deep underground mine is being constructed by Sirius Minerals Plc. The mine will extract polyhalite, a high-quality organic fertiliser. All mining and transportation operations will take place underground. The main focus of this article are the service, production and tunnelling shafts.

**Mining • Geotechnics • Tunnelling • Shaft sinking • Diaphragm wall**

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Cover:

The architects of Bridge “Brug van den Azijn” (Vinegar Bridge) in Antwerp, Belgium, designed a challenging 75 degree facade. Maccaferrì had to adapt the Mechanically Stabilized Earth (MSE) wall system “MacRes System” to meet the challenging requirements.

Read more on page 12.

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Even after the cessation of hard coal mining in late 2018, Germany remains a raw materials producer, where lignite, potash and salt, ceramic resources, industrial minerals, mineral oil and natural gas are mined. The knowledge acquired from German mining can also be exploited to overcome global raw materials issues.

Raw materials • Germany • Securing resources • Knowledge

For many decades, hard coal mining was easily the most important branch of mining in Germany, both with regard to the value of resources and investments as well as measured against the size of the workforce. Germany was ranked among the top 10 internationally. In keeping with political decisions relating to coal dating back to 2007, hard coal mining as such will cease at the end of 2018 when the Prosper-Haniel (late 2018) and Ibbenbüren mines (as from August 2018) close. Notwithstanding, Germany will remain a country with resources both with regard to utilising raw materials as well as mining them at the current rate of 760 million tonnes per annum.

Lignite

Germany is the world’s largest lignite producer with an output of around 170 million tonnes per year. Lignite-fired power stations accounted for around 23% of the electricity generated in 2017. Lignite is economic, the most important domestic source of energy and ensures Germany depends less on imported energy.

Potash and Salt

The German potash and rock salt branch is also one of the most significant in the world. A major portion of the potash produced is exported. The K+S Group is one of the world’s biggest potash and in turn, fertiliser producers enjoying a 10% share of the global potash market. In addition, salt mining in Germany is assured a long-term future thanks to the many existing resources.

Ceramic Raw Materials and Industrial Minerals

It is a far less well-known fact that domestic ceramic raw materials and industrial minerals, such as special clays, kaolin, quartz sand, quartzite, feldspar or bentonite are basic materials for many different branches of industry. In Germany, they are mined at a large number of smaller operations distributed throughout the German federal lands.

Mineral Oil and Natural Gas

Domestic mineral oil and natural gas production is largely confined to the federal Land of Lower Saxony. It can be assured of a long-term future providing that unconventional extraction methods can be improved.

Natural Stone, Sands and Gravels

Reserves of mineral resources such as natural stones, sands and gravels are well distributed. Demand for domestic primary stone and earth resources exceed 500 million tonnes annually accounting for an annual turnover of approx. 33 billion €.

Securing Raw Materials with domestic Production

As a result, the present German government emphasised in its coalition agreement from February 2018 that it is relying on domestic mineral resources. It stated explicitly that a secure and sufficient supply of raw materials and access to resources represent decisive factors for Germany as an economic hub. This also applies even if certain raw materials such as batteries and permanent magnets must be acquired and imported by German industry on world markets.

The advantages of domestic raw materials production include value augmentation and the retention of capital-intensive investments for mining in Germany, resulting in substantially better controllability of environmental influences of mining projects quite apart from the high level of training.

Legal Framework

German Mining Law provides an appropriate legal framework, as it enables special characteristics akin to mining to be appropriately taken into consideration when permits are issued. The law is often regarded as exemplarily in other countries among other things owing to the strict requirements called for in the interests of protection and provision on behalf of the environment and those affected. Following transparent involvement of the public, the requirements faced by securing resources are weighed up against ecological needs and with the concerns of neighbours on an equal basis. German mining’s achievements in the environmental sphere – for example, with regard to conserving water and species or in waste issues – enjoy world renown in similar vein to reutilising former mining areas.

Consequently, environmental aspects speak for German mining when lent closer observation.
Training and Research
Aachen, Bochum, Clausthal-Zellerfeld and Freiberg are tradition-steeped German seats of higher learning for studying mining and associated research and development. These universities maintain intensive international contacts and tackle new research topics, such as digitalisation or Mining 4.0, increasing efficiency, industrial safety, environmental protection and sustainability.

Supply Industry
Germany possesses an efficient and innovative supply industry for mining. The professional association for mining in the VDMA (German Engineering Industry) for instance, expects a growth in turnover of approx. 8% this year with regard to German mining machinery manufacturers as opposed to turnover of 2.7 billion € for 2017. The positive development in Asia, Russia and America is the main reason for this upswing. The bauma, one of the world’s largest construction and mining fairs held every three years in Munich with the next one scheduled for spring 2019, will no doubt confirm this trend.

Global Challenges
Current global political developments involving unrest, new trade barriers and volatile situations affecting many raw materials represent major challenges for politics, suppliers, service providers and manufacturing companies.

Although the contrary may often be erroneously maintained, Germany does not lack raw materials. Even although this country cannot extract all the resources it needs domestically, reserves of many raw materials are available – for many decades if not centuries to come. This secures in part, the necessary resources in addition to the existing mining knowledge, required by our universities as well as the supply and service industry.

To consolidate a secure supply of raw materials Germany must safeguard access to domestic reserves as well as procure further resources from international markets. Furthermore, it has to propagate its raw materials knowledge beyond its own borders to overcome global challenges facing man and the environment.

Yours,
Thorsten Diercks

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Application of Gabion Structures in the Transport Infrastructure

Prof. Dr.-Ing. Frank Heimbecher and Lukas Tophoff M.Sc., both Faculty of Civil Engineering, Münster University of Applied Sciences, Germany

1 Introduction

Gabion structures are becoming increasingly more significant in the field of transport infrastructure facilities as retaining structures, facing shells and noise abatement walls (Fig. 1). They fulfil many demands for a sustainable and efficient infrastructure as economic, durable and ecological structures. Towards this end, planners, clients and structural engineers all apply the same standards, codes of practice and leaflets. Not every requirement can be confirmed according to relevant standards thus, tests on a 1:1 scale are often essential. At present, it is still not possible to provide verification of internal stability and verification of connectors for back-anchored structures using explicit models [1]. As a consequence, verification based on tests is both prescribed [2] as well as targeted. This article concentrates on the relevant new codes of practice and verification based on tests.

2 Gabions as Supporting Structures in Developing Infrastructure

In accordance with ZTV-ING Part 2 Section 4, gabion walls acting as retaining structures are classified as engineering structures as from a height of 1.50 m and restricted to a visible height of 6.00 m [5]. These restrictions reflect the high splitting tensile forces in the gabions [1], which result from the currently accepted model assumptions [1]. According to [3] only gabion baskets are approved for federal highways. In 2003, a comprehensive leaflet was introduced via the ZTV-ING in 2003 in the form of the M Gab [4]. After the leaflet had been revised in 2014, in 2017 the TL Gab-Stb [2] was published as a supplement and since then introduced in several federal states in the federal highway sector. The M Gab contains regulations pertaining to the choice and verification management of suitable gabion structures whereas the TL Gab-Stb determines the demands posed on the components (wire, filling material and connecting elements). Table 1 contains a list of contents of the two codes. The TL Gab-Stb and M Gab are applied in tandem.

As far as verifying stability is concerned, through the introduction of the TL Gab-Stb it is necessary to verify the internal stability by means of load tests on the reference unit. With regard to verifying the stability of connecting the gabion wall with the anchoring elements, the M Gab requires this to be proved independent of the selected system and backed up by test certificates [3]. Verification in this case too, is only possible after undertaking tests. Appropriate possibilities are cited in the following.

---

Table 1: Overview of the contents of codes of practice

<table>
<thead>
<tr>
<th>Aspect</th>
<th>TL Gab-Stb (2)</th>
<th>M Gab (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground and foundation</td>
<td>x</td>
<td>+</td>
</tr>
<tr>
<td>Gabion baskets</td>
<td>+</td>
<td>o</td>
</tr>
<tr>
<td>Back-anchoring</td>
<td>x</td>
<td>o</td>
</tr>
<tr>
<td>Filling material</td>
<td>+</td>
<td>o</td>
</tr>
<tr>
<td>Backfill material</td>
<td>x</td>
<td>+</td>
</tr>
<tr>
<td>Verifications of stability</td>
<td>o</td>
<td>+/o*</td>
</tr>
<tr>
<td>Production</td>
<td>o</td>
<td>+</td>
</tr>
<tr>
<td>Documentation</td>
<td>+</td>
<td>o</td>
</tr>
<tr>
<td>Quality assurance</td>
<td>+</td>
<td>o</td>
</tr>
</tbody>
</table>

Provisions: + detailed, o general, x none
* internal stability

---

Fig. 1: Gabion structure used in the federal highway sector
3 Investigations on internal Stability

According to DIN EN 1997-1 [5] verification is required to ensure that no internal failure affects a component (Fig. 2). The interaction of the gabion’s individual component parts has so far not been clearly defined by means of a model or equivalent verification concepts. The Münster University of Applied Sciences and the Brandenburg University of Technology Cottbus-Senftenberg in Germany have as a consequence undertaken successful load tests for a number of years on reference units to verify internal stability – within the scope of research projects, degree theses by students, the provision of expert reports and official approvals. The tests can also be used for the verification management of internal stability in order to establish the required value for the equivalent concrete compressive strength $f_{ck}$ and applied according to $M_{Gab}$ [4] for dimensioning purposes.

The FH Münster’s test unit can tackle samples on a 1:1 scale given a total height of 2.00 m with a testing force of up to 1,000 kN. However, a maximum testing force of 250 kN has sufficed for the load tests carried out so far, as they simulate the equivalent dimensioning stress for a 6.00 m high gabion as a gravity wall. Towards this end, a load of 160 kN was determined as the limit for the SLS (Serviceability Limit State) and 240 kN as the limit in the ULS (Ultimate Limit State) for 6.00 m high gabion structures (Fig. 3). From the many tests (Fig. 4) performed in keeping with current test conditions the following boundary conditions were laid down for carrying out the tests, with further recommendations provided in [1]:

- Determining the abort criteria when executing the tests:
  - Wire breakages
  - Failure of welds, struts and connecting materials
  - Pronounced grid size increases
  - Filling material falling out
  - Exceeding the stone compressive strength

- 2 or 3-sided storage of the gabions to simulate the lateral integration of the basket in the wall

- Determining and considering predeformation during installation

- Structural separation of measuring technology from the test frame

- Setting up measuring points in and around the maximum deformations (upper area, if necessary determining by preliminary tests)

- Application of rigid load plates and fine gravel cushions to introduce even load

Fig. 5 displays a typical load test. It was undertaken with centric and eccentric load. In addition, the grid mesh widths, the struts, the filling material and the method (by hand, poured) were also different. The load tests reveal that serviceability generally results under slight absolute and relative deformations below the 2% limit value at 160 kN (SLS) as well as the internal sta-
bility (ULS) of the individual elements under the test conditions.

4 Verifying Connecting Possibilities of back-anchored Supporting Structures

Retaining structures built by the gravity construction method quickly reach their limits both statically and economically. Back-anchoring is frequently the sole possibility of producing a competitive structure. The combination featuring back-anchored geogrids represents a particularly economic alternative. The traction of the reinforcing elements can be very different depending on the system. During investigations at the Münster University of Applied Sciences and the Institute of the Kiwa GmbH TBU, Greven, various connecting structures were tested. As a result, it was revealed that the maximum acceptable forces and the relevant deformations of the various structures differ.

4.1 Connecting the Geogrids with Pegs and Shackles

The connecting force depends on various factors given shackle and peg connectors at the rear of the gabion. These include the distance between shackles, the grid width and the materials used for the gabion grid, the peg diameter and the chosen geogrid. The deformations in the peg result in stress peaks in the geogrid strands. As a result, product-specific verification is required for each alternative means of connection. Within the scope of the investigation pronounced deformations affecting the gabion grid were detected in the case of high connecting forces, which led to the filling material being redistributed. Admittedly, high connecting forces in excess of 100 kN were verified for this structure, however, the deformations affecting the peg lead to high load concentrations on the back-anchored geogrids (Fig. 6).

4.2 Connecting by Friction and Connecting Grids

An additional, shortened gabion grid attached to the front of the gabion can sustain frictional forces as well as the connecting forces from the peg connector (Fig. 7). The tests reveal that the structure can admittedly sustain the connecting forces of structures up to 6 m in height. However, major deformations occur at the connecting grids in the case of high connecting forces, which for their part lead to load concentrations on the back-anchored geogrid. The test set-up is presented in Fig. 8.

4.3 Connecting by Friction through back-anchored Steel Grid

Apart from directly coupling the geogrid to the gabion basket, the possibility exists of decoupling the two supporting systems – the gabion wall and the geogrid-reinforced earth (Fig. 9). In this case, the gabion wall must sustain the earth pressure forces, which are produced at the rear side of the gabion wall on account of its limited deformability. The forces are taken by a gabion grid installed horizontally in the backfill soil, which is attached by a peg to the gabion wall.
a load of 50 kN/m² deformations of less than 10 mm were measured. Further tests also revealed slight deformations. Connecting the grids by a peg also turned out to be restrictive during these tests. As far as the grid mesh width and peg design are concerned, there is still improvement necessary. This type of connector offers the advantage of separating the subsections gabion and reinforced earth both in terms of planning and construction technology.

5 Conclusion and Outlook

Thanks to the introduction of the TL Gab-StB a code of practice has been created, which regulates in particular the filling materials and the quality demands pertaining to the gabion meshing. In this way, the technical principles are codified so that gabion structures can be set up that are stable and serviceable in the long term.

The internal stability of the gabion structure must currently be verified by means of large-scale tests. Similarly, the connecting strengths of back-anchored structures must be verified in the form of tests. Both verification processes differ depending on the gabion system, the filling material as well as the type of connector. Thanks to large-scale investigations it was possible to provide proof of this nature. As a result, it is also possible to simulate or verify structural heights, which exceed the height limit of 6 m according to ZTV-ING Part 2 [3]. Modifications to the gabion basket or the connector can consequently be checked quickly and reliably.

It is possible to verify that the connector’s supporting capacity is sufficient up to a height of 6 m for gabion structures, which are back-anchored with a gabion grid in the backfill soil. The frictional force increases with the structural height. The decoupling arrived at between the geogrid-reinforced body of earth and the gabion wall in the case of a frictional connection enables tasks to be clearly defined.

In order to secure high quality during the life cycle of gabion structures, a Quality Association for Gabions (GfG) has been established [7]. In future, it will promote quality criteria for both planning as well as the execution process, which in some cases, are even more stringent than the requirements set by the current codes of practice.

6 References


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Introduction

Mechanically Stabilized Earth (MSE) walls with concrete facing panels are well-known systems used in Europe and in the rest of the world to support or enable the construction of infrastructures in tight urban corridors, forming retaining walls, abutments and wing walls [1].

Polymeric geosynthetics have been introduced in the wall market representing an evolution and a significant advantage for both cost-effectiveness and performances. Geostrip-reinforced MSE walls are recommended for non-conventional retained soils such as poor subsoil conditions, warm climates or chemically aggressive environments, where steel reinforcements could represent an issue.

The general behaviour of the MSE structure depends on the interaction between the soil reinforcing elements and the surrounding soil. Soil-reinforcement interaction is extremely important because it is the mechanism by which forces are transferred between the reinforcements and the surrounding soil. The design of the MSE structure and its general behaviour depend on this interaction between soil and the reinforcing elements based on the properties of the materials used and the construction methods. As an alternative to traditional reinforcing elements, high adherence polymeric soil reinforcing strips have been introduced to increase the design life of the wall even in highly aggressive environments, reducing the overall project costs and providing design flexibility [2, 3, 4]. Several walls have been equipped with measuring devices in recent years to study and demonstrate the behaviour and the performances of such structures.

The Maccaferri “MacRes System” is a MSE wall with concrete facing panels (Fig. 1) in which the soil is reinforced with linear high adherence polymeric reinforcements placed in the backfill in successive layers [5]. The retaining wall is designed to withstand the forces exerted by the retained ground or “backfill” and other externally applied loads and to transmit these forces safely to the foundation. Due to the polymeric nature, both strips (Fig. 2) and connections in the MacRes System are resistant against deterioration in fact the wall’s design life is 120 years [6].

Facade of the Bridge “Brug van den Azijn” in Antwerp, Belgium

Flemish Waterways increasing Capacity

The Flemish Waterways Authority (“De Vlaamse Waterweg”) plans to enlarge the section of canals to increase the transport capacity of Belgian waterways. The modernization of the “Albert Canal” foresees increasing the bridge structure and creating a wider waterway. This project involves eliminating of all bottlenecks that might limit the Albert Canal’s capacity. The “Brug van
den Azijn” (in English: “Vinegar Bridge”) is a three span higher arch bridge designed to replace the older Deurnebrug beam bridge, that connected the Antwerp districts of Merksem and Deurne. The Vinegar Bridge’s free vertical clearance is now 9.10 m with a passage width of 63 m (Fig. 3).

Facade Design

For this project, the architects decided to design an articulated facade to provide the people living in the area with a real eyecatcher of a bridge. This prompted several non-standard details that had to be considered during the stability check calculations. First of all, the panels were not vertical but inclined by 75°; this represented the first project in which the MacRes System has not been used as a vertical structure (Fig. 4). This non-standard structure required several stability checks due to the unusual shape of the two bridge ramps.

The second interesting challenge was that the architects did not want coping beams neither on top of the wall nor in the corners. Therefore, the design of over 500 different type of panels was necessary to fit the inclination and geometry of the top of the wall, the non-standard corners, stairs and cycle paths (Fig. 5). The initial architectural design was made using 1,5 m x 0,75 m MacRes concrete panels. The problem was solved by manufacturing a horizontal false joint during the casting phase: the real joints (between the concrete panels) and the false joints are perfectly aligned to create the facing pattern as described in the tender specifications.

Moreover, the final architectural finishing of the wall had to be uniform. The retaining structures were realized with the special panels mentioned above but the true bridge abutments were concrete structures cast on-site. For this reason, it was decided to use the same MacRes concrete panels as “cladding” all over the surfaces. There are no visible differences between the reinforced soil structure walls and the cladding areas under the bridge. The false and real joints alignment was possible thanks to precise topographic measurements on-site and AutoCAD modelling. An anchorage system was designed to connect the 1-tonne heavy panels to the concrete facade, using only INOX nails and steel plates (Fig. 6).

The project includes 3,850 m² of MacRes System and 1,820 m² of cladding surfaces: two ramps 300 m long each side, structures up to 10.5 m high and 80,000 linear metres of ParaWeb reinforcing strips.

Facade Installation

The project was completed in a few months, the construction of the MacRes retaining walls started in October 2017 and ended in February 2018 (Figs. 7 and 8). The cladding surfaces were installed in March 2018.

The panels were treated with a special anti-graffiti agent, which was sprayed once the jobsite was completed (Fig. 9). Maccaferri used a special product that protects the concrete from UV-rays without closing the
Conclusion

On April the 2^{nd}, 2018, the new steel arch bridge was officially inaugurated over the Albert Canal in Deurne (Fig. 10). Several authorities participated in the opening ceremony. A total of 15 million € has been invested in the new bridge modernization: larger ships can now sail underneath the structure and public transport passengers and cyclists can travel smoothly and safely.

The peculiar black stone facing and the impressive variegated shape of the two bridge ramps represented an important challenge for all the people who worked for this project. Maccaferri, together with its local partner Texion and its concrete supplier De Jong Beton are proud to have been part of this team.

References

[1] Lugli et al.: Lime stabilized backfill in MSE retaining structures application and advantages
ParaWeb™ straps for reinforced soil retaining walls and bridge abutments.


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Innovative Resealing of Segment Joints by Means of Injection Needles
Götz Tintelnot, TPH Bausysteme GmbH, Norderstedt, Germany

Introduction
Leaky structural joints in engineering structures located in groundwater are a well-known problem. Segment joints in tunnel tubes, which have been produced by tunnel boring machines, are also affected (Fig. 1). When grouting is needed to seal such segment joints, resealing injections targeted through the segment joint represent an effective and cost-saving alternative to those customarily applied so far with elaborately produced concrete drill holes. The TPH Bausysteme GmbH was able to obtain initial positive findings in applying such targeted resealing measures during follow-up grouting operations for the tunnel tubes of the Finne Tunnel in Thuringia as part of the German Unity Transport Project 8.2 “New Erfurt-Halle/Leipzig Rail Route” [1, 2]. At the time, this method represented an innovation; in the interim it has been further advanced based on the experiences made with the Finne Tunnel. This advanced method is dealt with in this article.

Comparison between conventional and innovative Resealing

Conventional Resealing with inclined Drilling of Segment
So far leaky joints have frequently been tackled with drilling through the segment at an inclined angle and resealed by introducing a grout. The disadvantages were [1, 2]:
- The great effort expended on drilling
- The damage affecting the segments and their reinforcement
- Poor accessibility of leaks in spite of high grout consumption

Targeted Joint Resealing through the Segment Joint
The innovative method applied in the Finne Tunnel for resealing segment joints and a scientific paper by Kirschke, Schälicke and Fraas from 2013 [1, 2] on the background and motivation as well as the findings from the initial application did away with the taboo that segment seals may not be drilled through. Practical application has shown that even in the event of major energy expenditure when drilling the tunnel joint until the seal is reached using conventional reinforced concrete drills, there is no threat of the segment seal being accidently penetrated (Fig. 2). In the case of the targeted resealing of segment joints the direct path via the segment joint is used to seal the segment. This is carried out by penetrating the seal and placing the grouting agent at the waterside through it. The advantages of this innovative method are:
- The injection agent and in turn, the sealing effect can be substantially more accurately positioned in the joint channel between the segment sections than in the case of a slant drill hole conducted through the segment joint.
through the segment concrete behind the segment seal.

- In contrast to the conventional procedure the processor only has to enlarge the unreinforced segment joint with an 18 mm diameter concrete drill hole in order to be able to seal the segment (Fig. 2). There is no need for considerably more complex and protracted slant drilling through reinforced concrete, thus the amount of drilling is substantially reduced and the disturbing effect of the segment reinforcement impacting drilling is completely eliminated.

- The need for concrete repair so far required after injection work – the segment concrete after all represents an essential part of the sealing system – is obviated. As experience shows that the concrete covering on the face surfaces of the segments is sufficiently large, 18 mm diameter concrete drill holes in the segment joint require no subsequent reworking and the concrete drill hole can remain untreated after the injection.

Further developed Joint Resealing with Injection Drill Needle

Of late, a tool known as the injection drill needle has come into use for the innovative method of resealing joints. It unites all functions in one, which are necessary for penetrating the segment seal, the subsequent injecting and permanent and pressure water tight closure of the drill hole. Fig. 3 shows the injection tool known as the injection drill needle as well as the relevant injection pipe, onto which the needle is screwed in a clockwise direction. When the segment seal is penetrated by this 3 to 5 mm thick stainless-steel needle the sealing material is merely displaced and material is not removed as is the case when cutting tools are applied. As a result, the process-related compression of the segment sealing frame is further increased so that it is not possible for the needle to be pressed out by the water pressure acting on it externally [3].

The following working steps are required for using injection drill needles, which are described as follows in greater detail [3]:

1. Screwing in the injection drill needles (Figs. 4 + 5)
2. Selection of suitable grouting agents and injecting the structural joints (Figs. 6 + 7)
3. Removing the grouting pipe

Working Step 1: Inserting the Injection Drill Needle

The injection drill needle screwed on to the grouting pipe is conducted into the segment joint expanded by a drill until the segment seal is reached (Fig. 2). Subsequently, it is gently rotated through the segment seal with the aid of a cordless screwdriver and a drilling bit until the waterside is reached (Figs. 4 + 5).

Working Step 2: Selecting suitable Grouting Agents and Injecting the structural Joint

On account of the anticipated redistributions in the rock and the resultant associated structural move-
ments or settlements, essentially elastic or expandable grouting agents are preferable. In this case there are two-component acrylate gels and expandable polyurethanes, which are particularly suitable, that for example are described in the ABI leaflet [4]. The grouting agents should possess proof of suitability for being injected into the reinforced concrete, e.g. in accordance with EN 1504-5 [5]. For assuring the quality of factory made maintenance products, planners, contracting authorities and responsible contractors can no longer rely on systems, which had been examined by independent third parties to establish their suitability – in Germany known as the "BASf List" for tunnelling. According to the new German Construction Law, products intended for safety and maintenance systems as well as recognised test centres will no longer be contained in the "BASf list" as from Dec. 31, 2018. "Verifiable certificates“ according to Art. 30 of the Construction Products Regulation (BauPVO) can be provided as an alternative to project-specific proof of suitability by the responsible contractor in individual cases [6, 7, 8]. Such certificates, so-called DIBt assessments are supplied in Germany by the German approval body Deutsches Institut für Bautechnik (DIBt) at the request of the manufacturer. As contact with the groundwater during grouting is possible, proof relating to the hygienic compatibility of the groundwater e.g. in keeping with DIN 19631 [9] is required. The acrylate gels Rubbertite/Polinit or Variotite/Polinit as well as the expandable two-component polyurethane resin Pur-o-Crack Plus have proved their worth as grouting agents for resealing segment joints [10, 11, 12, 13].

Compared to customary concrete injections, the grouting process is executed applying very slight grouting pressure, as it involves targeted filling a joint space (Figs. 6 + 7). The compressive force of the installed segment gasket represents the maximum possible injection pressure in the joint, and the segment gasket acts to seal the joint towards the inside and must not be overcompressed. The progress and distribution of the selected grouting agents can be controlled via the material outlets on the neighbouring packers.

Working Step 3: Removing the Grouting Pipe

Once the grouting process is concluded the injection pipe can be removed. There is no need to wait until the grouting agent has hardened, as the injection drill needle fitted with a non-return valve remains in the segment gasket’s drill hole as an abandoned tool after completion of the grouting work, closing it so it remains watertight on a permanent basis.

A non-return valve in the injection drill needle prevents the water located on the outside of the segment from penetrating when it is being screwed into position, and prevents the grouting agent from backflowing during injecting. This enables the worker responsible to move on to the next injection point after completing the grouting operation – without waiting for the grouting agent to harden – comparable with a one-day
packer. Towards this end, the grouting pipe is turned anti-clockwise from the injection drill needle, which remains in the segment gasket as a seal.

**Findings on resealing Joints with Injection Drill Needles and Prospects**

The findings that TPH Bausysteme GmbH has made with targeted resealing of joints and more lately with injection drill needles can be described as thoroughly positive (Fig. 8).

Concrete drilling operations in an un-reinforced segment joint can be accomplished far more easily than alternatively drilling concrete through the reinforced segment concrete – regardless of whether in the form of a slant drill hole or drilling parallel to the joint. Grouting operations in the Rastatt Tunnel revealed that given a 50 cm thick segment the necessary roughly 35 cm deep joint drill holes can be executed in less than 2 minutes.

The injection drill needles can easily be drilled through the segment gasket using a cordless screwdriver requiring very little force. The needles remain stable and watertight in the gasket owing to the compression of the segment sealing frame, thus it is not possible to extract the needles by accident.

The most important consideration, however, is that the injection drill needle at no time represented the restrictive element with regard to the rate of flow. The lower grouting rates in comparison to classical “masking” could be achieved at any time directly in the joint area with extremely low grouting pressures.

When using the grouting pipe with valve opener, which temporarily cuts off the non-return valve in the needle, even seepage water can be dealt with from one needle to another. Such deliberately created infiltrating needle, even seepage water can be dealt with from one needle to another. Such deliberately created infiltrating

The responsible contractor needs not accept any restrictions compared to grouting work using a steel drill packer. Seen from the viewpoint of the TPH Bausysteme GmbH it can therefore only be a question of time until targeted resealing using the injection drill needle establishes itself as state-of-the-art.

**References**


[8] Bundesanstalt für Wasserbau (BAW): BAWEmpfehlung „Instandsetzungsprodukte“.


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A Tunnel Boring Machine (TBM) is operating deep below the community of Cassis, France (Fig. 1). The area’s limestone is known for groundwater, karstic cavities, and voids. Suddenly the TBM grinds through the rock face to reveal a void of blackness beyond. The crew has hit the top of a massive, uncharted cavern that extends below the tunnel. After lighting up the cavern to verify its features, a Speleologist ropes off and descends into the void. Devoid of any concrete, the cavern is estimated at about 22 m long, 15 m wide, and 14 m deep, or about 4,500 m³ of open space (Figs. 2 and 3).

This is just one of the challenges – the most recent – encountered during excavation of the Galerie des Janots tunnel. The open-type, Main Beam TBM manufactured by The Robbins Company is excavating the challenging 2.8 km long drive for contractor Eiffage Civil Engineering.

Upgrading the Water Supply

Galerie des Janots is one of the fourteen operations designed to save water and protect resources, which are being carried out by the Aix-Marseille-Provence metropolis, water agency Rhône Mediterranean Corsica, and the French State Government. The Janots gallery will improve access to water in the communities east of the Aix-Marseille-Provence metropolis (Cassis, Roquefort-la-Bédoule, La Ciotat and Ceyreste). The future pipeline will replace existing pipelines – currently located in a railway tunnel – that have significant safety and vulnerability deficiencies with estimated water losses of 500,000 m³ per year. The current pipes have a

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Fig. 1: Jobsite for the Galerie des Janots
The small jobsite for the Galerie des Janots, a 2.8 km long tunnel mean to improve water supply in the French communities of Cassis, Roquefort-la-Bédoule, La Ciotat and Ceyreste.
Source of the photos: The Robbins Company

Fig. 2: Unknown cavity
At the 2,157 m mark, the machine grazed the top of an unknown cavity. A Speleologist roped off and descended to map the extent of its size.

Fig. 3: Size of the cavern
Devoid of any concrete, the new cavern is estimated at about 22 m long, 15 m wide, and 14 m deep, or about 4,500 m³ of open space.

Fig. 4: Assembly of the Robbins TBM Augustine in limited space
At 250 metric tons, and 135 m long, the TBM and back-up system could not fit at the small jobsite abutting residences and other buildings.
capacity limited to 330 l/s, which is largely insufficient in the summer months. The objective of the operation is to increase capacity to 440 l/s. According to Danielle Milon, Mayor of Cassis, “This is an investment of 55 million EUR with 11 million in aid from the water agency. This project required 10 years of reflection and work to improve water supply. And water is essential for the development of each municipality, and for citizens’ well-being.”

The tunnel passes under Le Parc National des Calanques, with cover between 15 and 180 m. Geotechnical studies of the area showed limestone, with the possibility of both filled and empty karst caverns.

A Launch in limited Space

The 3.5 m diameter Robbins TBM, christened “Augustine”, was commissioned on March 3, 2017. The TBM was extensively modernized and upgraded during the rebuild for the Galerie des Janots project in La Ciotat, France (Fig. 1). At 250 metric tons, and 135 m long, the TBM and back-up system could not fit at the small jobsite abutting residences and other buildings (Fig. 4).

“Very little on-site storage was allotted in the launch area, with just 25 m outside the portal for assembly – the exact length of the TBM from the cutterhead to the rear legs (Fig. 5). The logistics of the machine arriving in sequence and being assembled on time was vital,” explained Robbins Field Service Supervisor Andy Birch, who assisted Eiffage during the TBM assembly. “We did a two-stage assembly. First we assembled the TBM and five decks of the back-up system, then we did the remaining decks.” After assembly, the crew launched the machine, working 24 hours a day to excavate the tunnel and maintain equipment during the bore.

Excavating in tough Ground

At the machine’s launch, the crew was optimistic about getting through the obstacles presented by karstic limestone (Fig. 6). “Limestone is an easy rock to dig, but one can be confronted with the phenomenon of karst,” said Loïc Thévenot, Director of Underground Works for Eiffage during the TBM launch. “For this purpose, the tunnel boring machine is equipped with a probe drill. If the karst is small, we will fill it with concrete. If it is large, we plan to erect a small parallel gallery.”

To further identify cavities ahead of the TBM, the crew installed a geotechnical BEAM system, standing for Bore-tunneling Electrical Ahead Monitoring. BEAM is a ground prediction technique using focused electricity-induced polarization to detect anomalies ahead of the TBM.

Despite all the preparation, ground conditions were particularly tough in the first 1,000 m of boring. The crew encountered limestone with powdery clays, but this became an obstacle when groundwater was added to the mix. “In some areas we encountered water-bearing rock that turned the material into a sticky clay. This caused the cutterhead to block, though thankfully not for a great distance. We were able to unblock the cutterhead manually using a clay spade and shovel,” said Birch. The crew could tell when blocking occurred due to torque spikes on the cutterhead and reduced muck on the conveyor belt (Fig. 7). “We reduced water sprays
through the cutterhead to avoid creating stick clay,” continued Birch.

While at the site, Robbins Field Service advised and gave instruction regarding technical issues and operational functions of the TBM, as well as instructed on maintenance, and ways to sustain good advance rates. The weak rock and clay conditions necessitated ground support including resin-anchored bolts and rings in bad ground, topped with wire mesh and a 10 to 15 cm thick layer of shotcrete (Fig. 8). Some small filled and empty karst cavities were encountered (Fig. 9), and these were systematically drained if needed and filled with grout or foam (Fig. 10).

**Uncovering a Cavern**

At the 1,035 m mark, the crew hit a cavern on the TBM’s left side. The cavern, studded with stalactites and

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**Fig. 8:** Ground support
The weak rock and clay conditions necessitated ground support including resin-anchored bolts and rings in bad ground.

**Fig. 9:** Drainage and filling of karst cavities
Filled karst cavities with water and mud were also encountered in the first 1,000 m of boring, and these often needed to be drained and then filled with foam.

**Fig. 10:** Small karst cavities
Small karst cavities were encountered – empty ones were filled with grout or foam.

**Fig. 11:** Cavern with stalactites and stalagmites
At the 1,035 m mark, the crew hit a cavern on the TBM’s left side measuring 8,000 m³ in size.
stalagmites and measuring 8,000 m$^3$ in size, was grazed by the TBM shield (Fig. 11). The crew named the cavern “grotte Marie Lesimple” after their site geologist.

“We hit the corner of it. To cross it, we had to erect a 4 m high wall of concrete so the TBM would have something to grip against,” explained Marc Dhiersat, Project Director of Galerie des Janots for Eiffage (Fig. 12). A small door allowed access inside the cavity, which formed naturally at a point 60 m below the surface. The TBM was started up and was able to successfully navigate out of the cavern in eight strokes without significant downtime to the operation – the process took about two weeks.

“This is certainly unusual, to come across a cavern of this size and significance. It is somewhat related to the geology, with karstic and volcanic formations having the most potential for underground cavities,” said Detlef Jordan, Robbins Sales Manager Europe. Karst cavities were a known risk during the bore, but the cavern was not shown in vertical borehole reports or in the geophysical survey conducted from the surface along the alignment.

**A New Challenge**

After clearing the cavern the ground stabilized. The machine averaged excavation rates of 20 to 22 m/d in two shifts, with a dedicated night shift for maintenance. Crews ran the excavation five days per week, achieving over 400 m in one month. This performance continued until the 2,157 m mark, when the machine grazed the top of an unknown cavity that extended deep below the tunnel path. This was the structure measured at 22 m long, 15 m wide, and 14 m deep. Crews probed in front of the cutterhead and as of August 2018 are working to stabilize and secure the cavity with foam and concrete.

“We should be able to excavate a by-pass gallery near the shield in few weeks, and we may need to do that manually through the cavern using traditional excavation techniques,” explained Dhiersat.

**Conclusion and Outlook**

While Galerie des Janots has been a tunnel fraught with obstacles, the lessons learned have been invaluable. The crew was able to pick up on anomalies around the TBM in time to deal with them safely, and it has successfully put to the test methods to extract a TBM after encountering caverns and karsts, getting the operation moving again. Eiffage is optimistic that the new cavity and the TBM can be stabilized and secured quickly, but a large amount of work remains, as ground treatment in front of the cutterhead will be necessary and will take some time. Currently 700 m of boring remains.
**Composite Pile Roofs as Alternative to conventional Pipe Umbrellas in Tunnelling**

Dipl.-Ing. Frank von Havranek, Friedr. Ischebeck GmbH, Ennepetal, Germany

**Introduction**

Tunnel drives cause additional stresses in the surrounding rock over and beyond the actual rock pressure. The interface between working face and the advancing wall is especially sensitive regarding stress concentrations (Fig. 1). Substantial stress peaks can result above and behind the upper part of the working face, which can exceed the permissible rock stresses. As long as the tunnel supports, for example the temporary shotcrete shell and the subsequent inner shell, are not effective, local additional temporary supporting measures are necessary. Supporting or reinforcing elements are applied as safeguards. The installation of conventional pipe umbrellas is standard practice and state of the art.

Umbrellas are normally applied in areas with shallow rock overburden – in other words, wherever flat trajectory slopes are cut or where existing buildings with shallow overburden must be undercut so that no self-supporting arching effect can be created. However, relatively unstable layers of rock or sliding slopes are mostly to be found in steepest, practically vertical slopes. Furthermore, typical conditions for application are fault zones which suddenly occur during the drive, low rock cohesion, instable funnels or silos. The task of the umbrellas is to temporarily secure the length of advance by supporting the advancing wall and forming a rock reinforcement. A central aim of the load-bearing and supporting effect of umbrellas is to restrict rock settlements as tunnelling progresses. So-called composite pile roofs were developed as a possible alternative to conventional pipe umbrellas. Composite pile roofs achieve a good bonding effect using officially approved steel supporting members of the Titan type [1] and are intended to activate the surrounding rock (Fig. 2).

Pipe umbrellas have proved their worth in tunnelling for temporary supporting during driving – especially in the critical area involving the advancing wall and working face in the case of rocks with low cohesion, shallow overburden, tunnel entrances and fault zones. Composite pile roofs made of steel supporting members with small cross-sections and dynamic grouting offer an alternative to conventional pipe umbrellas. The bearing behaviour and the execution of composite pile roofs as well as pros and cons vis-à-vis conventional pipe umbrellas are examined and unclarities discussed.

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**Fig. 1:** Sensitive area for stress peaks

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This article deals with the load-bearing behaviour and the design of composite pile roofs – type Titan – compared to classical pipe umbrellas. It examines the systems, their verifications and the design. In this connection, pros and cons and further requirements relating to development or clarification are dealt with.

**Static Models for the load-bearing Behaviour of Pipe Umbrellas and Composite Pipe Roofs**

The aim of supporting is to relieve the rock by reducing the horizontal stresses in the rock, lowering the rock exploitation factor λ resulting from the reinforcing effect and in turn, increasing the relative safety. The load-bearing behaviour of the umbrellas is essentially governed by the bond between the steel supporting member and the rock. It is known in the case of smooth ground anchor bars that only a steel stress of 105 N/mm² can be ap-
plied for the bond. Ribbed Gewi® bars [2] on the other hand, can transmit 230 N/mm² and ribbed Ischebeck® Titan bars [1] approx. 500 N/mm² for long-term applications.

The pipes of conventional pipe umbrellas are only injected on a one-off basis from grouting valves – mainly to diminish slippage, and they are capable of being relocated longitudinally in practice. Conventional pipe umbrella pipes thus develop their load-bearing and supporting effect more simply as pure flexural beams on two supports, with the pipe located on two floating bearings, as the simplified static model shown above in Fig. 3 illustrates. As supports A and B are accepted to be sliding bearings that are axially displaceable, the flexural rigidity EI solely governs the deflection. Large flexural moments require a large inertia moment, which explains why thick-walled steel cross-sections that are as large as possible are applied.

In the case of the Titan composite pile roof the steel supporting bar is firmly embedded in the crown's shotcrete as well as in the rock behind the working face via a verifiable bond. This is why a different load-bearing and supporting system is created than in the case of the pipe umbrella. Supports A and B are fixed clamped. A conceptual “structural system” is created in the greatly simplified static model (Fig. 3 below). The load-bearing effect depends on the shearing bond between the reinforced steel supporting bar and the surrounding, dynamic radially grouted rock. The slender bar or “tensioned cable” permanently fixed at both sides between the axial non-relocatable abutments bears the rock in the driving zone. By and large, only axial forces are activated. The elongation and the settlement, which normally are confined to a few millimetres, determine the deflection.

**Conventional Pipe Umbrellas**

As described, conventional pipe umbrellas develop their effect as flexural beams. That is why they are produced as rigidly as possible. This is accomplished by ensuring that the deflection of the pipes itself is largely confined. The determining factors are the flexural moments, however, compression forces from the working face must also be sustained. Inevitably, large pipe cross-sections are the outcome. Thus, the state of the art foresees steel pipes with diameters ranging from 76 to 168 mm being applied [3]. The one-way drill bit driven by the drill rods runs in front of the pipes to be installed. These pipes possess injection valves and are extended shot-by-shot by means of pipe couplings of varying types. As far as the couplings are concerned, a distinction is drawn between three different types, with their diversity indicative of the special challenge that is faced:

- Simple standard threaded connection with cut thread and strong static weakening of the pipe at the coupling point given constant pipe internal diameter
- Expensive nipple connection with roughly the same static values as the pipe, however, with smaller internal diameter than the pipe
- Compression connector, which can be relatively quickly executed thanks to the compression device and which slightly weakens the static values and the pipe internal diameter

In practice the pipes are drilled from relatively large pockets in the crown (sawtooth profile). The pipe is supported in the crown at its exposed end by the steel arch or located in shotcrete, the other end is inserted in the rock behind the active zone at the working face. Depending on the rock the relative rock exploitation λ equals roughly 0.78 given a pipe gap of 0.44 m and a pipe diameter of 160 mm.

**The Titan Composite Pile Roof**

Titan composite pile roofs were applied for the first time in building the Metro in Santiago de Chile [4] shortly after the turn-of-the-century (Fig. 2). The pat-
Titan Steel supporting Bar

The core of the composite pile roof is formed by the Titan steel supporting bar, which is especially popular in foundation engineering for micropiles for sustainable foundations, back-anchors and securing slopes. As an officially approved system it is state of the art and in practical use in Germany and beyond [1].

Types of Steel

The steel supporting bar acts as a drilling rod and a reinforcing bar at one and the same time. It is subjected to extremely high dynamic load especially in solid rock in the case of percussive installation using hydraulic hammers and transference of the impact energy to the sacrificial bit so that a tough, ductile steel with high notch impact strength is necessary.

Subsequently, the same rod serves to sustain loads amounting to several hundred kilonewtons. For safety reasons it should behave in such a manner that it provides prior notice of failure should it be overloaded in order to avoid sudden malfunction, in other words before evident deformations become visible. Only a few steels combine these properties, including the fine-grain structural steel S460NH. As a result, this steel is explicitly listed in the DIN EN 10210 [6], which is referred to both by the DIN EN 1537 [7] for ground anchors and the DIN EN 14199 [8] for micropiles. The S460NH attains practical values of up to 100 joules at −20°C for notch impact work W. The cited norms pose further demands. Thus, the maximum yield stress $f_y$ is limited to 500 or 600 N/mm², a yield-to-tensile ratio of $R_m/R_e > 1.08$ is called for and elongation at maximum load of $A_p > 5.0$ % demanded.

Dimensions

Steel supporting bars of sizes 40/16 and 52/26 with 40 or 52 mm external diameter and 16 or 26 mm internal diameter are normally used. The steel supporting bars are delivered in 3 m section lengths and connected with couplers with central stop and centred in the drill hole with spacers set every 3.00 m.

Ribs

Fig. 4 shows different thread forms, on the right for temporary round threads (R-threads) and on the left the reinforcing steel thread for the Titan steel supporting bar conventionally used in special foundation engineering for sustainable structures. R-threads are standardised as drill rod threads in accordance with DIN 10208 and possess a very flat flank angle of around 17° with high spreading force on the grout body. It was devised for easy release of the drilling tools. In contrast, the Titan thread corresponds to the standards for reinforcing steel DIN 488 and DIN EN 10080. The taper amounts to 45°, which caters for an optimal bonding effect, the effect confirmed in the approval. The spreading forces are marked in Fig. 4 by a red arrow and the differences are easily discernible. The round thread results in a threefold greater splitting effect and in turn, cracks in the grout body, which can extend to its surface, which would represent a high safety risk by forming dangerous longitudinal cracks in the composite pile roof.

Thus, as a consequence, DIN 488 [9] and DIN EN 10080 [10] as well as DIN EN 1992-1-1 [11] call for certain criteria relating to the rib geometry and steel quality, namely a flank angle $\alpha$ of $> 40°/45°$, a maximum carbon content of $C < 0.20$ % and a relative rib area of $f_p = 0.056$. The Titan steel supporting bars with $f_p > 0.14$ exceed the demanded required rib area more than twofold. This assures verification of the crack width limitation and illustrates the supporting bar’s high bonding capacity. Crack width verification applies for maximum crack widths of $< 0.1$ mm. That is one reason why the steel supporting bars were officially approved for long-term applications. The standard corrosion protection according to approval is thus equivalent to “double corrosion protection” for plastic rib pipes of other known reinforcing steels.

Self-drilling anchors with R-threads are applied in tunnelling for temporary use as rock anchors, which for instance, are inserted at right angles to the crown or working face. In this way, in such cases, grout bodies created by static grouting are restrained at all sides and the rock sustains a part of the ring stresses, as a result of which spalling on the grout bodies can be avoided. The situation with self-drilling spiles with R-thread is different. These are generally 3 to 4 m long, are sometimes drilled at a relatively slight incline to the crown, however, only individual blocks are nailed, subjected to shearing and frequently used as “angst spiles”.

Conventional anchors and spiles in tunnelling thus possess an essentially different load-bearing behaviour from steel supporting bars used for the Titan composite pile roof. The applied R-thread form and most steel qualities fail to conform with the norms consequently these anchors are not unproblematic seen from the technical viewpoint and involve greater risk during application.

Fig. 4: Comparing thread forms – round steel according to DIN 102018 (otr) and Titan steel supporting bar according to DIN 488 (otr)
Grout Bodies

Steel supporting bars with cement flushing are installed by rotary percussive drilling and dynamically grouted. The grout body that is created determines the load-bearing effect. The bonding effect with the rock is based on two essential points:

- The useful steel stress of approx. 500 N/mm² for the grout in keeping with the approval
- Dynamic grouting over the entire length of the steel supporting bar, which produces a grout body with a diameter of roughly 140 to 200 mm and good interlocking between the grout body and the rock by means of a rotary injection with a build-up of pressure ranging from 5 to 15 bar (Fig. 5).

Dynamic grouting has already been successfully applied in special foundation engineering. However, it has not been generally accepted everywhere in tunnelling. It calls for special skills on the part of the drilling crew as well as knowledge of mechanical engineering and equipment technology.

How the Composite Pile Roof System is formed

Titan composite pile roofs are geometrically similar to the 3 to 4 m long spile umbrellas, however, they can be substantially longer – up to 18 m – and enable driving to be accomplished undisturbed for a lengthier period. The essential differences to pipe and spile umbrellas are the verifiable bonding effect and different load-bearing behaviour.

The number of layers of steel supporting bars in the composite pile roof can be varied. The structural design and the load model of a two-layer Titan composite pile roof are presented schematically in a 3D-view in Fig. 6. The steel supporting bars are inserted up to 18 m long, forming a shell one above each other in a number of layers at roughly 0.40 m gaps almost parallel to the crown, rising slightly with a gradient of a few degrees.

Each steel supporting bar drilled from the crown overlaps the one located above by several metres so that several layers can act at the same time depending on the length of the steel supporting bars. The steel supporting bars are continuously grouted dynamically over their entire length so that they are interconnected force-locking and form-fit with the rock. Thanks to this reinforcement the rock develops its inherent load-bearing behaviour on two levels:

- In the longitudinal direction of the tunnel under the bell curve q_l(x) of the funnel forming above the working face
- In the tunnel cross-section through the arching effect between the individual steel supporting bars

The rock stresses from the funnel moving in conjunction with the drive are not simply critical in the unsecured area in front of the working face, but behind it as well. A slip plane forms here, which can lead to failure of the working face due to the load imposed by the funnel. The composite pile roof is also active in this critical area. The composite pile roof as load-bearing and supporting system has to be adapted to the construction process. The lengths of the individual supporting bars and the gaps between umbrellas must be determined accordingly. The length of the supporting bars (length of umbrella) is decisively governed by the length of the section to be driven and secured in several rounds trouble-free including the still non-self-supporting “fresh” shot-
In tunnelling involving pressures of 5 to 15 bar leads to interlocking with the soil or rock and in turn, reinforcing and improving the rock. The effectiveness of the composite pile roof largely depends on the diameter of the bond or grout body. Conventional bit diameters amount to between 90 and 130 mm. From a large number of retrieved grout bodies, soil or rock-related enlargements can be applied in constructional terms. These for instance amount to 50 mm in sand and gravelly sand and 75 mm in medium and coarse gravel. These values can also be applied for heavily weathered or disintegrated rock. In this way, composite/grout body diameters of around 140 to 200 mm can be assumed. Reference values of 25 mm are known for the enlargement in cohesive soil (Fig. 7), which correlate with the value of 20 mm for other soils set well on the safe side for external flushing cited in DIN SPEC 18539 [12]. Here the limits of application become discernible.

Cement grouting is particularly effective in soft grounds or heavily weathered rock with grain sizes exceeding 2 mm. The application of the composite pile roof is in accordance with the basic concept of the NATM (New Austrian Tunnelling Method), which permits settlements thanks to its elastic behaviour, enabling rock pressure to be relieved thus increasing the rock’s self-bearing behaviour. By and large, only axial forces are activated in the steel supporting bars, comparable with the cable bearing model [13, 14].

Furthermore, the composite pile roof is characterised in the longitudinal direction of the tunnel by the bond between the grout bodies and the parcels of soil of the funnel and in a lateral direction through the arching effect formed between the grout bodies, which prevent the soil located above from passing through. According to Eckl [13, 14] theoretical cohesion of the rock with 31 to 100 kN/m² can be applied.

The rock stresses above the tunnel roof and working face in particular are reduced. Depending on the rock for example, a relative rock exploitation λ of 0.85 is attained for a gap α of 0.44 m for the Titan steel supporting bars 40/16. Through optimisation this value can if necessary be reduced still further, in other words improved. During tunnelling, axial load changes in the steel supporting bars can also occur, which are covered by the approval [1].

In practice, single-layer composite pile roofs are also applied. Single-layer because in the “upper reinforcement layer” the bonding length in the rock is still located in the funnel, in other words, in the event of failure of the funnel, it would be moved along with it. The two-layer version, in other words, the arrangement of a twin-layer bond with the rock behind the funnel, is more practical because then the two layers of the assumed flexural beam can be differentiated in the compression and tensile zone with the second layer affording additional security against failure. Fig. 8 shows the assessment approach for a single-layer composite pile roof as flexural beam, whose steel supporting bars act in the tensile zone as (tensile) reinforcement, while only...
the compressive strength of the soil/rock is applied in the compression zone. In the case of pressure load from the working face, as can occur in certain situations during driving, a pronounced downward deflection occurs on account of the additional load imposed by the tunnel. The steel supporting bars are also subjected to tension. In the case of two reinforcement layers, the upper one then acts as a compression strut. Both approaches must be discussed against the background of the permissible deformations, necessary cross-sections and the soil/rock characteristic values.

Three failure mechanisms can restrict the bonding effect:

- If the parcels of soil suspended in front of the working face within the driving area detach from the steel supporting bar resulting in the equilibrium being disturbed
- Soil flowing through between the steel supporting bars resulting in a loss of the arching effect between the steel supporting bars
- Abutment failure, through either the steel supporting bar being torn from the subsequent shotcrete shell or its bond with the rock fails

Consequently, the reliability of the bond is decisive for the composite pile roof functioning and thus not just a straightforward “extended” spile umbrella with drilling rod thread (R-thread) can be used. What is new is that in the case of this merely temporary application, verifying the bond is equally as important as in the case of permanent application in special foundation engineering. The installation of composite pile roofs is preferably executed in rock with low cohesion and practically parallel to the crown with very shallow overburden. In this way, the grout bodies remain practically unrestrained in the rock, the resultant ring stresses must be accepted by the grout bodies themselves without lateral support, otherwise there is a danger of the grout body spalling. This would represent a considerable safety risk and greatly restrict serviceability.

It is imperative that detailed geometrical planning and static dimensioning as well as their reliable on-site implementation are accomplished. Towards this end, the composite pile roof for the actual project must be presented diagrammatically in such a way that the individual steel supporting bars can be installed on the construction site in an unambiguous and precise manner.

**Comparison between conventional Pipe Umbrella and Titan Composite Pile Roof**

A major advantage of the composite pile roof compared with conventional pipe umbrellas is that the recesses and in turn, the sawtooth profile of the excavation are substantially smaller or simply do not exist so that the need to fill these recesses with shotcrete is reduced or avoided altogether. Support arches are then only required for a single cross-section. Optimally, drilling takes place directly through the support arches, which, however, requires coordination of the support arch type, gap between arches, drilling angle and maximum diameters of the steel supporting bars, their sleeves and bits. Further practical advantages are that the same drilling technology is applied as for anchors and spiles, augmented by a mud flushing head for dynamic grouting and a twin piston pump with storage tanks. The small cross-section facilitates easy handling: heavy casing pipes, drill rods and grouting valves are unnecessary. The flexural sensitivity of the pipe joints of the pipe umbrella is also dispensed with. The sleeves are simply loaded axially and possess a central stop with gasket. Grouting can be successfully accomplished with grout pressure and a grout quantity which can be separately adjusted at the injection point (GIN value) and documented with pressure data loggers.

Furthermore, measures are needed to prevent the cement paste from flowing back from the mouth of the drill hole in such a manner so that the grout pressure can be built up for dynamic grouting. This issue can be resolved by using a preventer system, which is part of the engineering equipment. Such preventer systems have already been successfully applied in special foundation engineering (Fig. 9). The challenge facing tunnelling is in adapting this technology to the existing very constricted spatial conditions and for drilling on the rise quite apart from not releasing the counter-pressure built up in the rock after decoupling the drilling machine. Developments in this direction are forging ahead.
Umbrellas are installed so that surface settlements are minimised. Towards this end, measurements have been executed which demonstrate that the composite pile roof’s settlement pattern is similar to that of the umbrella shield [4]. In this way, it has been shown that the system functions in practice.

According to Brandl the axial forces, which normally largely stem from the working face, do not run congruently towards the umbrella shield. This presumably can be explained by the fact that the steel supporting bars above the bond with the rock sustain and remove forces over their entire length thus creating a divergent balance of forces.

The comparison of conventional pipe umbrellas with smooth pipes with 160 mm diameter resulted in a relative exploitation factor λ for the rock of 0.78, and an umbrella with R32 steel supporting bars an utilisation factor λ of 0.85. Thus, this latter umbrella was assessed at merely 9% below the conventional umbrella shield [13, 14]. By applying Titan steel supporting bars with the characteristics of reinforcement steel and slightly larger diameters optimisation is conceivable.

**Closing Assessment and Outlook**

The conventional pipe umbrella is often overvalued owing to its rigidity. Spile shields consisting of steel supporting bars with R-thread constitute a safety risk and thus have scarcely made an impact on the market so far. Titan composite pile roofs can be applied as a worthwhile alternative to pipe umbrellas providing that the rock conditions are suitable, the assessment approach clearly defines reality and that they are designed consistently in keeping with valid norms and reliably executed on-site.

A suitable rock could for example exist with a degree of weathering V3 (weathered), an average force transmission of approx. 110 kN/m and shear transference resulting from cohesion of 250 kN/m². According to Maidl, given such boundary conditions, evidently sufficient shear resistances and the equivalent if not in fact, improved load-bearing behaviour thanks to the clearly better bonding behaviour are attained than with conventional pipe umbrellas [15].

A series of very positive references were acquired with Titan composite pile roofs during the construction of Metro Line 6 in Santiago de Chile (Fig. 2) and in Scandinavia. Various planning and technical aspects were resolved on the spot on-site. As a result, statics completely worked out by hand are available.

Successful solutions applied in practice in a number of countries have yet to be transferred to all the others. The applicable boundary conditions and regulations must be observed. Thus, issues relating to modelling, e.g. using FE methods, must be resolved.

Dynamic grouting has still to become generally established in tunnelling. Work is still progressing to improve preventers and drill hole seals as well as the penetration of support arches. The equipment and qualifications of responsible contractors are still in need of improvement. Once unresolved questions are tackled jointly by planners, the construction industry and suppliers, the Titan composite pile roof represents an interesting alternative (Figs. 10, 11 and 12) or addition to conventional pipe umbrellas.

**References**

Composite Pile Roofs as Alternative to conventional Pipe Umbrellas in Tunnelling

Fig. 12: Installing a Titan composite pile roof when opening up a cross-passage


[5] Pat.-Nr. 102 34 255 and Report


**Fig. 12: Installing a Titan composite pile roof when opening up a cross-passage**

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Innovative Load Distribution Plate for Segments
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Background

Hardwood fibre boards are applied in tunnelling as load distribution plates between segments. Currently, standard, untreated hardwood fibre boards are generally used (Fig. 1). These load distribution plates are usually glued on at the production plant immediately after the segments are manufactured to comply with the sequence of operations. Subsequently, the segments are frequently stored exposed to the elements in large numbers as well as with various dimensions, concrete qualities and reinforcements (Fig. 2) until installed in the tunnel (Fig. 3) – quite possibly for a period of several weeks or months. As a result, it can happen that the load distribution plates, which are 93% wooden material, absorb moisture, swell and exceed the required thickness by 50 to 80%. In such cases, the sealing function of the segmental lining is no longer 100% assured, and the danger exists that water can penetrate the tunnel. Eradicating damage is associated with high costs.

Cause of long Segment Storage Periods

The problem of unscheduled protracted storage of segments inevitably prevails if delays relating either to the project or ground affect a shield drive or the tunnel boring machine (TBM) is out of action. Some time ago for example, several thousand segments for a large tunnel project were exposed to the elements after the TBM malfunctioned over a period of several months. The load distribution plates had to be removed laboriously.
by hand to be replaced with new plates so that the generally highly-reinforced and expensive segments could be installed once the TBM started up again.

Special Further Development for Tunnelling

In order to avoid damage when storing segments out in the open over lengthy periods, the Georg Michael Beteiligungs GmbH, Bremen, devised a special plate to meet the requirements of tunnelling. The so-called “mago-tunnelling-specialboard-plus” for tunnelling possesses the following advantages:

- High strength
- Weather and UV-resistance
- Improved assembly relating to efficiency, industrial safety and environmental protection

Strength

The plate is mainly produced from coniferous wood from Eastern Europe or Scandinavia. High strength results from a slow growth process. The high proportion of resin also contributes towards the plate’s stability. Sustainability is confirmed by the FSC Controlled Wood test with the number FSAC-Std-40 to 004 V2-1-Standard.

Weather Resistance

The side of the fibre plate that is visible until the segments are installed in the tunnel is glued to a PE foil – a blue grid foil if desired (Fig. 4 above). Thanks to this foil coating the load distribution plate is largely weather-resistant regarding moisture and UV rays. These special plates do not absorb any moisture even given lengthy storage periods of the segments exposed to the elements. Consequently, no deformations or swelling occur. Thus, the plate retains its desired thickness, and the special plates can be assembled in the production plant as needed once the segments have been cured.

Assembly

At their rear, the special plates are provided with 50 mm wide, double-sided self-adhesive strips at their longitudinal sides (Fig. 4 below). These adhesive strips facilitate assembly onto the segments. The protective foil can easily be stripped off even using work gloves. Black glue, which presents environmental problems, is no longer necessary. As a result, there is no need any longer for the measures designed to protect staff, the expensive disposal of residual glue in the form of hazardous waste and the time-consuming cleaning of working tools as was the case with untreated hardwood fibre boards. The self-adhesive foil does not contain any substances that harm the environment and is thus classified as non-hazardous according to EU guidelines. The segments stay clean and are not contaminated with black glue residues.

Conclusion

The new “mago-tunnelling-specialboard-plus” for tunnelling rationalises segment production, enhances industrial safety for the workforce and cuts down on ecological impacts. Furthermore, the logistics for storing the segments and availability for installation are improved.
Large Scale Monitoring using BRILLOUIN Optical Fibre Sensor Systems in the Fields of Geotechnics and Mining

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The technique of distributed fibre optic sensor analysis reached a stage of development allowing long-term – large-scale monitoring systems to be designed. It features distributed high-resolution strain measurement based on BRILLOUIN scattering allowing it to observe potential candidates for expected ground movements such as ground falls, sinkholes, or land slide movements at the actual onset of instability. By expanding this technology to old mining areas characterized by scant knowledge of mine geometry, it is possible to establish a monitoring system for both risk management and alarm functionality thus increasing safety and reducing costs for restoring damaged infrastructure.

**Distributed BRILLOUIN Scattering-based Sensing Technology**

Best results for large scale area monitoring can be expected from BRILLOUIN-based sensors using BRILLOUIN loss technique [1, 2, 3], whereby two counter-propagating laser beams, a pulse and a CW (continuous wave), exchange energy through an induced acoustic field. When the beat frequency of the laser beams equals the acoustic (BRILLOUIN) frequency, νB, the pulsed beam experiences maximum amplification from the CW beam. By measuring the depleted CW beam and scanning the beat frequency of the two lasers, a BRILLOUIN loss spectrum centered on the BRILLOUIN frequency is obtained.

The sensing capability of BRILLOUIN scattering arises from the dependence of the BRILLOUIN frequency νB on the local acoustic velocity and refractive index in glass, which possesses linear strain dependence through

\[ \nu_B(T_0, \varepsilon) = \nu_B(0, \varepsilon_0) + \nu_{B0}(T_0, \varepsilon) \]

whereby:

- \( \nu_B \): BRILLOUIN frequency
- \( \nu_{B0} \): reference BRILLOUIN frequency
- \( \varepsilon_0 \), \( T_0 \): the strain and temperature corresponding to a reference BRILLOUIN frequency \( \nu_{B0} \)

By varying the spatial resolution, it can provide the scale of material strain measurement and structural strain monitoring.

Spatial information along the length of the fibre can be obtained through optical time domain analysis (OTDA) by measuring propagation times for light pulses travelling in the fibre. This allows continuous distributions of the strain to be monitored. The spatial resolution (gauge length) can be varied according to the application required, even after the fibres have been installed in the structure, by simply altering the length of the applied light pulse. These systems offer unmatched flexibility of measurement locations and the ability to monitor a virtually unlimited number of locations simultaneously.

**Examples of Setups already used for large Scale Monitoring Projects**

**Ground Motion Detection by Monitoring Railroad Embankments**

In similar fashion to old mining areas – karst areas are characterized by the sudden appearance of local sinkholes and major soil subsidence. Such phenomena represent a potential risk for infrastructure and life located in these areas. Installation of a two-stage fibre optic measurement and warning system can be designed and installed within these areas just beneath the surface close to existing infrastructure as well as below or within new constructions. The measurement system is separated into two parts. The alarm functionality is based on the integral optical fiber length measurement (Pulsar 1400) and the high resolution distributed strain measurement uses the distributed strain sensing technology (BRILLOUIN scattering).

To measure the deformations within the area of the railway trace construction, expansion-sensitive fi-
bre optic cables were installed as a sensor array in the underground (Figs. 1+2) – a grid-styled arrangement in six parallel cable trenches. In the event of a ground movement the strain sensitive array is stretched or compressed in these areas. For alarm fibre length variations are monitored automatically by one Pulsar 1400 instrument.

The alarm boundaries of length variations were determined by geotechnical experts. If a length change is measured which exceeds this limit, localization of the strain incident occurs using the high resolution distributed fibre optic strain measurement based on BRILLOUIN scattering.

**Ground Motion Detection by Monitoring Highways**

Similar ground motion detection such as monitoring of railroad embankments, highways or motorways connecting cities can be used for installing fibre optical systems. Ground motion induced due to historical mining activity such as sudden sinkholes can be recognized at an early stage to reduce the potential risk for infrastructure and life located in these areas. Installation of a two-stage fibre optic measurement and warning system can be designed and installed just below the surface of the road just within a diamond saw notch. The measurement system with alarm and high resolution distributed strain measurement functionality is optimized for long term monitoring.

**Ground Motion Detection by Monitoring Tunnels**

During the recent years several R&D projects have been initiated to develop a method for advance convergence measurements during the construction of new tunnels or during the renovation of existing ones. The method is suitable for analysing measurements of tunnel convergences to identify induced movements of the rock due to ground motion. The same setup can be used to derive information about ground movements for example in urban areas by instrumentation of metro constructions and waste water tunnels and foundation instrumentation (Fig. 3). The BRILLOUIN technology enables areas of imminent or accelerated ground strain to be identified.

**Ground Motion Detection by Monitoring of vertical Boreholes**

Another method suitable for detecting depth profiles of ground movement by BRILLOUIN technology is to equip a separate pipe with optical fibres. Once the pipe is installed in a borehole it is sensitive to ground motion with respect to depth – a very useful and accurate setup for ground movements as it detects compression elongation (Figs. 4+5). A monitoring system like this was installed in Spain for landslide monitoring. The first data readings indicate a high sensitivity and fast responding system. This method can be applied to all inclined boreholes – from vertical to horizontal. In combination with a Pulsar 1400 device it is a sensitive alarm system, which has the capacity of a high-resolution ground motion detector to activate alarm at the onset of ground motion and monitor it very precisely (Fig. 6).
Conclusions

Optical fibre-based monitoring systems are developing increasingly to become high-resolution autonomous geotechnical, ground, and infrastructure health-monitoring systems. Systems like BRILLOUIN DSTS from OZ Optics Ltd., Omnisens SA or Fibristerre Systems GmbH provide a higher resolution than conventional monitoring temperature and strain measuring systems. The challenge now is to find new monitoring setups using the full range of options of this technology. Sensor and cable – there is no difference – provide measurements in high local resolution. Monitoring of old mining areas takes advantage of this enabling areas of ground motion to be identified at an early stage. The data can be used to predict and prevent hazards.

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EU “Blue Mining” Project – Building a large-scale Test System and Flow Tests for vertical Transport Systems in Deep Sea Mining

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Introduction
Since the 1970s and 80s, the possibilities of mining raw materials from the ocean floor have been examined in various research projects. In this connection, extraction methods at great depths ranging down to 5,000 m were successfully tried out in test projects. For example, up to 70 t/h of manganese nodules (MnN) were mined by means of Airlift methods in test operations [1]. Further projects were designed to demonstrate the economic feasibility in addition to the technical viability. However, subsequent projects were held up owing to changes on the world market after the end of the Cold War. Deep sea (water depth in excess of 200 m) deposits have once again become the focus for securing raw materials on account of the increasing need for rare elements for the electro industry and changes on the raw materials market.

Within the scope of European raw materials policy aimed at securing critical resources, various deep sea mining research projects were embarked on. Projects that are currently planned such as the Nautilus Project to mine the Solwara 1 [2] deposit in the Bismarck Sea or the Japanese project for mining off the coast of Okinawa [3] are geared to water depths of 2,500 to 3,000 m. In such cases, vertical transport systems (VTS) represent a challenge as well as for the mining machinery that is required. The European Blue Mining research and development project has concentrated on further advancing transport systems for deep sea mining [4]. This article deals with the planning, building and trials with the large-scale system, the execution of the tests as well as initial results.

Overview of the Investigations on VTS in the EU Blue Mining Project
Within the scope of the EU Blue Mining Project for advancing vertical transport systems for deep sea mining, continuous transport systems were evaluated – first of all for carrying seafloor massive sulphides (SMS) from a depth of as much as 1,600 m (Fig. 2) and secondly for transporting manganese nodules from a depth of around 5,000 m (Fig. 3). The basis for these calculations were formed by known deposits and their characteristic values, e.g. the German license area of the Clarion Clipperton Zone (CCZ) in the Central Pacific
for the MnN study. In addition, economic factors also had a significant role to play such as the resultant targeted production rate of 400 t/h regarding the design of the transport system [4]. Two continuous VTSs were examined more closely, the Airlift method and pump system consisting of centrifugal pumps, as are currently in use in the offshore sector (Fig. 4).

The VTSs were analysed by several project partners in the case studies with the aim of identifying and discussing critical components and operational states. Then concepts for test systems were worked out based on the results of the case studies and prior research studies, geared to examining vertical pipe flows. In this connection, particular importance was laid on investigating settling velocity and wall frictions in the loop media with different particle fractions and associated flow effects such as the clogging effect. Recognitions and computational models from previous investigations on lab test stands (test stand height up to 12 m, transport height up to 9 m, pipe diameter DN 100 mm [7]) were to be validated in the process under real conditions and proportions as well as new knowledge gained for implementing VTS for deep sea mining.

Parameters for Setting up and Planning the Test System

The test system was set up in a shaft in the form of a closed flow loop, in which sample material (gravel and sand) was injected into a water flow, which could again be separated from the water phase once the tests were concluded. The loop medium was transported through a centrifugal pump within a large U-shape comprising a downpipe and riser. The vertical transportation process within the riser was to be monitored by appropriate measurement technology. Furthermore, the riser had to run perpendicularly, without bends or tapering and be equipped with pressure sensors at regular intervals. The sensors had to be installed in an accessible manner so that they could be checked and replaced if necessary.

Technical and technological Approach

Thus, various technical and technological factors prevailed for setting up the test system. For example, the aspired size for the height of the vertical riser was determined to be at least 50 m so that useful data could be obtained in comparison with a smaller test facility. Preferably the vertical riser should be more than 100 m high. Other parameters such as the pipe diameter were determined by the existing technology. The existing centrifugal pump with a capacity of P\text{mech} = 55 kW substantially influenced the set-up. Further determining design parameters were:

- Pipe internal diameter DN = 150 mm
- Optimal length of the riser (riser length) L ≥ 100 m
- Ratio particle-pipe size d/DN < 1/5 and thus d < 30 mm
- Maximum flow velocity v\text{max} = 5 m/s
- Maximum solid volume concentration in the circuit c_{v, max} = 20 %
Based on these parameters, the sample material was chosen in accordance with Table 1. Material properties and defined magnitudes such as solid volume concentration and height of the test system are for their part, determining factors for designing the pipeline system. The following measurement systems were applied to compile the necessary data:

- Establishing the pressure profile along the riser and parts of the downpipe through measuring with piezoelectric pressure sensors at regular intervals
- Temperature measurements by means of thermoelectric sensors
- Measuring the solid volume content in the flow through measuring the electric conductivity of the flowing medium by means of a Conductivity Concentration Meter (CCM) in a 1 m long pipe section comprising transparent PVC

Table 1: Sample material

<table>
<thead>
<tr>
<th>Material</th>
<th>Particle size [mm]</th>
<th>Bulk density [kg/m^3]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0/2</td>
<td>2,610</td>
</tr>
<tr>
<td>Gravel</td>
<td>8/16</td>
<td>2,630</td>
</tr>
</tbody>
</table>

Applying a high-speed camera to record the flow
Volume flow measurement above the CCMs by means of electromagnetic means (Electro Magnetic Flowmeter, EMF)

On-Site Conditions

The local conditions at the available shaft were just as decisive for the final design and the technical parameters. The chosen site was the 8th lightwell of the Rothschenberg Adit (Rothschenberg Stolln (RSS)). This is located in the municipality of Halsbrücke in the vicinity of the German town of Freiberg and is linked to the RSS just like the shafts of the “Reiche Zeche” research and education mine (Forschungs- und Lehrbergwerk (FLB)) belonging to the TU Bergakademie Freiberg. The 8th lightwell was worked between 1872 and 1877 as part of the RSS development. The shaft is 136 m deep. At a depth of roughly 42 m the Anna Adit connects with the shaft. The shaft down to the depth of the Anna Adit comprises natural stone masonry. Below the Anna Adit where the rock is stable, the shaft has not been lined.

On account of flooding in 2002, the RSS bypass in the proximity of the Halsbrücke spare vein collapsed. With the aim of redeveloping the original course of the Rothschenberg Adit in the interests of flood protection, the 8th lightwell was upgraded in 2003. Within the scope of the renovation work the shaft served as the main transport shaft and for man-riding. Towards this end, old installations in the transport and pipe branches made of timber were removed and replaced by platforms, ladders and partitions made of galvanised steel. The floor is lined by protective netting for mining and tunnelling. Since work concluded, the Saxon Mining Office has made use of the shaft for accessing the RSS for inspections. No mechanised means of transport were available on the spot and had to be included in the planning process for the construction measures and tests. Furthermore, an exact assessment of the gaps between platforms and intermediate beams and a measurement campaign to register the natural airflow in summer and winter as well as radon measurements were required. The ventilation measurements revealed a continuous upcast airflow. Artificial ventilation was not required for the construction and operation of the system. However, the resultant climate in the mine posed special challenges for the technology that had to be applied. As a result of condensation of the rising and saturated climate and ingressing groundwater via fissures, the shaft is subject to permanent precipitation.

Set-Up of the Test System

In the following the individual components of the test system are presented and briefly described. In this connection, they will be divided into surface and underground installations. Towards this end, Fig. 5 is intended to provide a rough overview of the complete facility and the shaft. The set-up and design of the facility...
subsequently comply with the previously established parameters.

**Surface Installations**

**Pump Control**

The controlling and monitoring of the transport process was undertaken from an office container. All sensor data from the shaft and the pump were visualised in the container and monitored by a single person.

**Preparing Samples plus the Pump**

As can be gleaned from Figs. 6+7, the facility constituted two main components, first of all the 55 kW centrifugal pump in a 10-foot container and the installations for preparing the samples in a 20-foot container. This compact and mobile system for injecting the sample material into the flow loop and subsequently to separate it was devised and built especially for the tests. Two persons were commissioned to check the system during the tests and to operate the shutoff valve of the sample hopper and the opening flaps for separation, monitor the separation process and if need be, discontinue the process.

The system comprises the following components:

▶ Two sedimentation tanks each with 7.5 m³ capacity, with the pipeline system for the complete facility and the pump holding roughly 5.6 m³
▶ Two hoppers as storage tanks for the sample material, which were augmented with the appropriate amount of sediments prior to the tests
▶ A separation tank, in which three Big Bags could be hung as filters

**Technical Transportation System**

For transporting material during the development and dismantling phases a windlass – type MPW 25 was installed at the mine. It was built back in 1982 by the SDAG Wismut Cainsdorf and upgraded in 2011. It possesses a payload of 25 kN given a mechanical performance of 5.5 kW, with the transport speed reduced to 0.16 m/s as the transport process is uncontrolled. The technical transportation system was made available for the utilisation period – first of all, for the possible carrying of material and secondly to rescue persons in case of emergency. For this purpose, the special rescue device shown in Fig. 8 was mounted.

**Underground Installations**

**Pipeline System and Sensors**

Owing to the low conductivity, polyethylene (PE) was chosen for both the riser as well as the downpipe to ensure that the measurement results were minimally affected. The pipes were dimensioned in keeping with DIN EN 8074 and 8075. In contrast to steel pipes, the choice of pipes was made according to the necessary wall thickness and the external diameter. Calculating the required wall thickness thus took place in keeping
with the technical specifications based on the following values:
▶ Desired pipe internal diameter DN = 150 mm
▶ Maximum hydraulic pressure through transport process $p_{max} = 21$ bar
▶ Assumed service life $X = 1,700$ h
▶ Average temperature in the shaft $t = 11.5$ °C
▶ Safety factor $C = 2$

This resulted in pipe Type DIN 8074 – 200 – SDR 7.4 – PE 100 with the following values:
▶ External diameter $D = 200$ mm
▶ Wall thickness $en = 27.4$ mm
▶ Pipe mass $m = 14.9$ kg/m
▶ Resultant internal diameter $DN = 145.2$ mm

The pipes were produced with an average length of 11.5 m in keeping with the gaps between the intermediate beams for the floor and attached by means of heavy-duty clamps to flexible brackets on the intermediate beams. The pipes were suspended in the shaft by a mobile crane for assembly purposes (Fig. 9). Starting at the very bottom of the shaft, the pipes were initially placed on a pre-installed U-segment made of steel and then attached.

### Steel U-Form and Dump Valve at the Foot of the Test System

Above the shaft’s bottom landing, a massive U-segment made of steel was positioned. The U-form served primarily to divert the flow from the downpipe to the riser, however, it also possessed an appropriately designed ball valve with drive. This valve allows emptying the test system either during standard operation or in a case of emergency to avoid becoming clogged with sediments. A collecting and drainage basin was set up in front of the dump valve in order to trap the sediments before reaching the RSS.

The arrangement and design of the dump valve at the lowest point of the U-form, however, turned out to be problematic. During separation of a sand-gravel test the system had to be opened up as only a part of the sediment could be transported owing to low flow rates during separation. As the valve needed some 45 seconds until it was completely open and the downpipe and riser were drained via this valve, a roughly 10 m long section of the riser became clogged. This was resolved by means of a high-pressure flushing application. Apart from this particular case, no further clogging occurred.

### Placing the Sensors

In order to be in a position to replace defective sensors if necessary, the pressure sensors were placed in keeping with the platform gaps. The necessary drill holes for the sensors were executed prior to the installation of the pipes in the shaft. Consequently, it was imperative that the pipes were installed in the proper sequence and alignment. After completion, the exact distances between the pressure sensors were remeasured. Table 2

---

**Table 2:** Diagram of how the sensors are positioned as well as gaps between pressure sensors. ($P =$ pressure sensors, $k =$ conductivity measurement and $T =$ temperature sensors)

<table>
<thead>
<tr>
<th>Diagram of sensor positions</th>
<th>Pressure sensor</th>
<th>Gap to sensor located below [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P01</td>
<td>11.20</td>
</tr>
<tr>
<td></td>
<td>P02</td>
<td>9.94</td>
</tr>
<tr>
<td></td>
<td>P03</td>
<td>11.99</td>
</tr>
<tr>
<td></td>
<td>P04</td>
<td>11.22</td>
</tr>
<tr>
<td></td>
<td>P05</td>
<td>11.24</td>
</tr>
<tr>
<td></td>
<td>P06</td>
<td>11.23</td>
</tr>
<tr>
<td></td>
<td>P07</td>
<td>11.21</td>
</tr>
<tr>
<td></td>
<td>P08</td>
<td>11.15</td>
</tr>
<tr>
<td></td>
<td>P09</td>
<td>10.89</td>
</tr>
<tr>
<td></td>
<td>P10</td>
<td>5.94</td>
</tr>
<tr>
<td></td>
<td>P11</td>
<td>7.57</td>
</tr>
<tr>
<td></td>
<td>P12</td>
<td>4.23</td>
</tr>
<tr>
<td></td>
<td>dump valve</td>
<td>–</td>
</tr>
</tbody>
</table>

**Table 3:** Parameter of the executed tests

<table>
<thead>
<tr>
<th>Designation</th>
<th>Flow rate $v$ [m/s]</th>
<th>Solid volume concentration $c_v$ [%]</th>
<th>Test material</th>
</tr>
</thead>
<tbody>
<tr>
<td>E01</td>
<td>2, 3, 4</td>
<td>0</td>
<td>water</td>
</tr>
<tr>
<td>E02</td>
<td>2, 3, 4</td>
<td>5, 10, 15</td>
<td>sand</td>
</tr>
<tr>
<td>E03</td>
<td>2, 3, 4</td>
<td>5, 10, 15</td>
<td>gravel</td>
</tr>
<tr>
<td>E04</td>
<td>2, 3, 4</td>
<td>5, 10</td>
<td>sand, gravel, 1:1</td>
</tr>
<tr>
<td>E05</td>
<td>2, 3, 4</td>
<td>5, 10</td>
<td>sand, gravel, 1:2</td>
</tr>
<tr>
<td>E06</td>
<td>2, 3, 4</td>
<td>5, 10</td>
<td>sand, gravel, 2:1</td>
</tr>
<tr>
<td>E07</td>
<td>2, 3, 4</td>
<td>5</td>
<td>manganese nodule</td>
</tr>
</tbody>
</table>
displays the measured distances. These gaps were subsequently used for checking the sensors (measuring hydrostatic pressure and comparison with calculated value). In addition to the position of the pressure sensors, the diagram in Table 2 contains the positions for the temperature measurements (T01 riser and T02 downpipe) and the measuring points for registering the conductivity of the CCM (k01 and k02).

### Executing the Tests

The tests were carried out in the summer months of 2017. Table 3 provides an overview of the projected test series. In this connection, the designation such as E02 relates to the test material (in this case sand), with individual tests being undertaken for each volumetric solid concentration. After the tests, samples of the used materials were taken for screen analyses and correspondingly marked. The designation E02 c 1 = 10% thus signifies that the individual test was executed with 10% sand in the transport loop (roughly 0.5 to 0.6 m³ of sand).

Firstly, water was solely used to try out the system, with all test cycles being conducted and tested to see they were functioning. Towards this end, separation was also simulated. Initial tests were undertaken involving only a small quantity of sand. The aim was to continue testing the system and check the calculated frictional values. Furthermore, it was essential to obtain basic experience in operating the fully assembled system, towards which end all individual components as well as the separation plant had to be tried out prior to being installed on-site. Monitoring and control of the flow process was undertaken by the pump control. The measured pressures as well as the flow rates and solid volumetric concentrations were displayed via a monitor, with the pump being activated by a frequency converter. Fig. 10 shows such a data output. On account of the increasing instability of the flow process with decreasing flow rate, actuation required plenty of experience and knowledge in dealing with the system. As the system was devised as a closed loop the solid volumetric concentration (measured via k01 and k02) increased successively as can be discerned from the wave form depicted in Fig. 10. In order to avoid the system clogging and in turn, associated pressure surges, the system operator must interrupt the transport process in time and introduce separation.

In addition to the originally planned test set-up, experiments with manganese nodules were executed at the end of the test period with the support of the Federal Institute for Geosciences and Natural Resources (BGR). Towards this end, up to 900 kg of original manganese nodules from the German CCZ license area was made available. However, first of all, they had to be crushed and screened to attain the necessary particle size (similar to gravel 8/16). This took place at the Institute of Mineral Processing Machines at the TU Bergakademie Freiberg. The manganese nodules that were used were then subjected to further treatment processing tests. The individual tests were carried out as follows:

1. **Preparation**
   Prior to each test being undertaken the sedimentation tanks were filled with fresh water. At the start of the tests they served as water reservoirs for filling the system. Furthermore, new Big Bags were suspended in special frames above each separation tank and the hoppers filled with the appropriate sediments.

2. **Filling the System**
   First of all, the test system was filled with water from one of the sedimentation tanks and the system ventilated. Subsequently, it was possible to check if the pressure sensors were functioning by means of hydrostatic pressure and calculated comparative values.
3. Water Test
Prior to each solids-laden test, a run was carried out only with water (E01) to test all measuring devices. In the process, the flow rate was gradually reduced from 4 to 1 m/s. Then the system was deactivated, the measurements evaluated and the hydrostatic pressure checked once more (testing for leaks). The frictional losses could be determined by means of the measured pressure values and compared with the theoretically calculated values and if need be, sensors replaced or recalibrated.

4. Injecting Sediments
Each test run involving a solids content commenced with the flow rate once again being increased to 3.5 m/s. The flow was now conducted through a bypass system beneath the hopper and the valves of the tanks opened. The flow gradually caused the prepared sediments to pass into the loop. The water displaced from the system escaped from the test facility via the ventilation.

5. Test Run
After filling was completed, the flow rate was increased to 4 m/s and, once the system had been stabilised, retained for about 5 minutes. Subsequently, the rate was gradually reduced and retained in each case for some 5 to 10 minutes, to enable the measurement data to be recorded.

6. Concluding the Experiment and Separation
Work on separating the sediments started once the solid volumetric content measured at the CCM attained a value in excess of c > 30% and at the same time the flow rate dropped below 2 m/s. This minimum rate is geared to the minimal rate, which is required to maintain the flow in the horizontal pipe sections. For this purpose, the rate was first of all increased again and the second full sedimentation tank connected up with the loop. The transported volumetric flow was carried via the Big Bags into the separation tank, with the sediments trapped in the Big Bags. The solid-water mixture in the test system was replaced by fresh water during this process. Subsequently, the test facility was emptied via the dump valve at the bottom of the U-form. The water from the separation tanks was pumped into one of the sedimentation tanks, in which the fine sediments were deposited and removed. The clear water phase was then made use of for tests. The used sediments were removed from the separation plant and stored in a container. Only unused gravels and sands were utilised for each test. This test procedure was designed to last for a period of 75 minutes per test, providing that the transportation process did not end prematurely. Owing to the necessary preparations and follow-up activities for each test, it was only possible to execute a single test involving the transport of solid material per day.

Assessment and Outlook
The presented flow tests were undertaken successfully during the summer and late summer of 2017. Subsequently, the system was dismantled. The closing report was completed to coincide with the end of the project in spring 2018 [8].

Owing to the large amount of data, the tests are still being evaluated so that currently only a short insight can be provided. In addition to the data collected directly from the ongoing trial such as images taken by high-speed cameras, pressure curves, material concentrations and volumetric flows, subsequently sample materials were further tested and screen analyses carried out so that conclusions relating to particle fracturing can be drawn. The measurement data gained relating to pressure curves and material concentration first of all had to be further processed on account of the noise produced by the system and brought into context with the applied material composition and the flow rates. Frictional values and mass slip have to be determined from these data, which can provide an indication of the sinking speeds of the individual particle fractions.

The identified characteristic values are included in the calculation model [7] mentioned in the introduction for validation purposes and are in this way, intended to help improve calculations for major systems and particle fractions. Furthermore, samples were taken from the pipes so that conclusions could be drawn on the wear of the pipe material during transportation. In addition to the standard pipes made of PE, sample pipes with a wear-resistant inner coating were installed and tested on the surface. The planning and construction as well as execution of the tests supplied recognitions, which can be utilised for further tests, as e.g. for the arrangement and dimensioning of dump valves.

Acknowledgements
This project and the ensuing research activities received funding from the 7th EU Research Framework Programme Grant Agreement Number 604500. The tests were carried out by the project partners Royal IHC (Netherlands) and the TU Bergakademie Freiberg. The project partners involved would like to express their special thanks to Messrs. Gerodur MPM Neustadt in Saxony, who provided the PE pipes for the tests.

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Large-scale Test System and Flow Tests for vertical Transport Systems in Deep Sea Mining

MinMG 45
Müller, Wijk and Mischo:
GeoResources Journal 3 | 2018


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Automated Lubrication: Retrofit Pilot Project in a German Quarry

Nikolaus Fecht, Specialised Journalist, Gelsenkirchen, Germany

The Stone Age has a Future – with this slogan, the quarry company SVA on the Swabian Alb is promoting an industry that is currently booming – not simply because there are so many ongoing road construction projects in Germany. For the project to run smoothly, it relies on its production equipment always being operational, and in turn on its automated lubrication. Bielomatik’s retrofit pilot project demonstrates how all this works.

Introduction

It concerns gravel, that is to say about aggregates, mixtures and additives for construction projects of all kinds. Located on the Swabian Jura, the Schottervertrieb Vorder Alb GmbH & Co. KG (SVA) produces several thousand tonnes per day from White Jura rock for customers from the Greater Stuttgart area (Figs. 1 + 2). But the end products – finely ground – are also found in toothpaste or animal fodder (calcium). In the complex manufacturing processes, the lubrication systems in the two ballast works play an important role; they were installed by the company Bielomatik Leuze GmbH + Co. KG (Bielomatik) from nearby Neuffen. The Schotterwerk Bauer Söhne GmbH + Co. KG (Bauer), Erkenbrechtsweiler received a new plant for the BHS mixing facility, the quarry of the Alfred Moeck KG, Lenningen received a lubrication system for its BHS mixing plant (Fig. 3) and binder + co received screening plants. Toni Pranghofer, operations manager at Alfred Moeck KG in Lenningen explains: “What spoke for Bielomatik was both the proximity to the location and the good experience Bauer had with a retrofitted Bielomatik lubrication system several years ago. In addition, we want to have the same technology in both plants. So it makes it easier for us to store spare parts, to ensure everything’s more uniform.”

Breaking new Ground in the Stone and Earth Industry

Bielomatik is breaking new ground with this contract because so far, the company has not fulfilled any retrofit projects in the stone and earth industry sector. “Similar to the printing industry, where we are well established, a particularly harsh, dusty environment
exists here,” explains Frank Müller, Service Manager of the Lubrication Systems division at Bielomatik. “However, the amount of dust is much higher here. The main sticking point is the high temperature fluctuations in the unheated buildings, which is why heated pipes are used.” Pranghofer continues: “Therefore, we do not need special lubricants for wintry conditions. As a result, we can as far as possible use the same grease in all systems.” In progressive lubrication systems from Bielomatik, SVA uses mainly smooth long-life lubricating greases.

**Planning and Execution of Retrofitting**

The older manual lubrication systems from other manufacturers were converted to the automatic Bielomatik systems without having to stop production during operation. “Only a few measures took place in the evening after operations ended or at the weekend,” says the manager. In a total of 380 working hours, Bielomatik service technician Oliver Oswald equipped the systems with the most diverse products from the Bielomatik central lubrication portfolio, while the operations manager coordinated cooperation with his staff; among other things, Oswald installed control systems, pneumatic drum pumps, pipes, electric impeller pumps, distributors and fittings. In the pilot project, 300 metres of steel pipe was laid in order to supply various bearing points with grease at the 80 lubrication points (Figs. 4 + 5). Pranghofer also contributed towards the project. At the neighboring ballast works Bauer, he devised a solution that he also had installed: The electronic control cabinets were equipped with dust-proof glass control windows, so that employees can check the status of the displays at a glance – without letting the omnipresent dust get in the cabinet (Fig. 6).

These are not standard solutions because the degree of lubrication required depends on the location. Some bearing points receive 80 grammes of grease per day, some just a few grammes per week. “Oswald has teamed up with plant manager Pranghofer to then determine the grease requirements based on the data from the individual bearing points and the manufacturer’s lubrication plans,” Müller explains. “The whole thing is like a tree: The pump is the root from which our system branches off to the individual lubrication points.” Using this information and that provided by the plant manager, the service technician has completely planned the plant’s lubricant systems.

The close interaction of the service technician with the customer has proved to be particularly successful for customised solutions: In one case, the operations manager only wanted to connect one sieve system to the lubrication system, but Oswald recommended that there should be a connection for all adjacent storage locations, e.g. of conveyor systems. Pranghofer says: “Now, the new plant is fully supplied with lubricant from two Bielomatik systems automatically.” The longer service life of the machine, the reduced time required for personnel deployment and the lower number of
failures and malfunctions all speak for this form of continuous lubrication.

Moeck provided the electrical control for the pump and integration into the entire control system. “In our plant control system, there are now two windows that indicate whether the lubrication is running or not,” adds Pranghofer. “I only get a simple error message on the screen, but it is sufficient. Because then a technician goes to the plant and fixes the malfunction.”

**Conclusions and Outlook**

The company cannot provide parameters as to whether there are any positive economic consequences, but the customer’s satisfaction is reflected in a pending new installation: Service technician Oswald will soon be in action connecting the jaw crushe (a machine for shredding very large pieces of rock) to the Bielomatik automatic grease supply (Fig. 7).

**Alfred Moeck KG, Lenningen**

Founded in 1947, the company has developed into a large, well functioning quarry and ballast plant supplying the Greater Stuttgart area with products made of “White Jura” material. Together with the neighbouring gravel plant Jakob Bauer Söhne GmbH & Co. KG, Moeck founded the joint gravel distribution company Schottervertrieb Vordere Alb GmbH & Co. KG (SVA) in the municipality of Erkenbrechtsweiler in 2006, which has since united both companies under one roof.

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**Nikolaus Fecht**

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**Bielomatik Leuze GmbH + Co. KG**

With a workforce of 650 throughout the world including 350 at its headquarters in Neuffen/Baden-Wuerttemberg in Germany, the bielomatik Leuze GmbH+Co.KG, established in 1946, carries out progressive production technology in lubrication technology and plastic welding systems. Configuration and production of minimum quantity lubrication systems for machining metalworking as well as systems and components for central lubrication of machines and plants. The plastic welding systems sector embraces machinery and complete production lines for welding and processing plastic parts with heating element, vibration, infrared, gas convection or laser technology.

**Contact:**

www.bielomatik.com
Shafts for Woodsmith Mine in North Yorkshire in the United Kingdom

GeoResources Team

Introduction

In Withby, North Yorkshire, one of the UK’s deepest underground mines is being constructed by Sirius Minerals PLC (Fig. 1). The mine will extract polyhalite, a high-quality organic fertiliser consisting of the minerals calcium, magnesium, sulphur and potassium [1]. The project is unique because all mining and transportation operations will take place completely underground, once the Woodsmith Mine is completed (Fig. 2). This means that polyhalite can be extracted with minimal impact on the region’s tourism and the North York Moors National Park.

Shafts

Two deep shafts, the 1,594 m deep production shaft and the 1,565 m service shaft, will access the polyhalite shelf seam. In addition to the deep shafts, a third shaft for a tunnel boring machine (TBM) is sunk to build the Mineral Transport System (MTS) tunnel.

The deep shafts have been designed by Sirius Minerals Plc and external consultants taking advantage of the knowledge and experience of their shaft sinking partners. The design and construction methodology corresponds with the geological conditions.

The three shafts are sunk in the following stages:

- **Diaphragm walling**
- **Main shaft sinking**

Diaphragm Walling

The upper sections of Woodsmith Mine’s service and production shafts will all be built using diaphragm wall techniques to create the headgear chambers and foreshafts, to a depth of 60 and 120 m [2]. Diaphragm walling was chosen to provide maximum strength and impermeability. As part of the 2.5-billion-euro project, Bauer Technologies Ltd., a subsidiary of Bauer Spezialtiefbau GmbH, has been contracted to build circular deep diaphragm wall shafts [3]. Construction of the diaphragm wall for the service shaft began in December 2017. There are three repeated phases to the diaphragm walling construction process (Fig. 3):

- **Cut:** Each panel is 2.8 m long, 1.2 m wide and cut to the required depth of 60 m for the outer diaphragm wall and 120 m for the inner one. The cutting wheels rotate towards each other, breaking up the soil and rock, which is carried to the surface. As the excavated material is removed, bentonite is pumped into the excavated panel to provide support and to prevent it from collapsing.

- **Cage:** Once a panel has been cut, reinforced steel cages are lowered into the excavation. The cages are fabricated off-site and are delivered in 14 m sections. The cages are aligned and spliced together as they are lowered.

- **Concrete:** Once the cage is in place, concrete is pumped into the excavation from the base of the panel to the surface, displacing the bentonite as it goes. The displaced bentonite is treated and recycled.
Among other equipment, three Bauer BC 40 cutters on MC 96 and MC 128 duty-cycle cranes, and three complex BE 500 and BE 550 de-sanding plants from Bauer MAT Slurry Handling Systems are being used in the project (Figs. 4+5) [3]. The bentonite suspension is enriched with a specially developed polymer-based additive to minimize losses into the fractured and porous strata.

Different measurement methods are being combined to produce a clear 3D model of the diaphragm walling works using BIM (Building Information Modelling). This will ensure that the required maximum vertical deviation of 200 mm for 120 m is not exceeded.

**Main Shaft Sinking**

Once Bauer Technologies has completed the diaphragm shafts, in a follow on operation the Production and Service Shafts will be sunk to a final depth of 1,500 m by Sirius’s shaft sinking contractor DMC Mining Services Ltd., Vaughan, Ontario, Canada. The contractor will use the Herrenknecht Shaft Boring Roadheader (SBR) to construct the main shafts (Fig. 6) [4]. The SBR has recently completed excavation of two deep shafts in Saskatchewan, Canada. The SBR is equipped with a roadheader boom and a rotating cutting drum. It is telescopic and allows for the excavation of the entire shaft cross-section with a depth of one metre in a single cycle [5].

**Tunnelling**

Sirius Minerals PLC undertook an early contractor involvement (ECI) process for Drive 1. Strabag SE was the winning tenderer of a design-and-build contract for approx. 13 km tunnel section with portal and ramp [6]. The section is part of a 37 km tunnel with a diameter of 4.7 m.

**References**


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