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Energy • Power • Storage • Cavern • Research • Development • Think tank

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Cover:
Onsets of Shaft #2 of the new potash mine Ust Jaiwa, OAO Uralkali
Source of photo: Deilmann-Haniel GmbH (S. Rossow)
www.deilmann-haniel.com
Competition for land is on the increase in cities, ports and seaside resorts. Climatic changes have resulted in higher water level rises being measured on coasts and incoming waves tend to possess more energy. Nature protection and economic aspects deserve more attention. Thus our society is faced with the relevant question whether alternatives exist to the current practice of dike construction. Alternatives which are more space-saving, economic and closer to nature without diminishing the high safety standard that dikes have attained.

Tried-and-tested Practice in Germany

Let us consider dike construction as practiced in Germany as well as in neighbouring countries. Dikes on the North and Baltic Sea coasts are produced in a classical manner with a sandy dike core and a clay or marl seal on the outer slope of the dike, the dike top as well as on the inner dike slope. This construction method has been applied for centuries and it has developed and proved itself fortified by findings and research efforts from the last fifty years following the major storm floods in 1962 and 1976. The grass-covered clay slope fulfils a sealing function as well as the task of protecting against erosion. As far as dike geometries are concerned, a tendency towards increasingly higher dikes with ever flatter outer and inner slopes has prevailed in recent decades. The dike height is the outcome of an increase in design water levels and in an increase of safety requirements. The flat dike slopes are intended to distribute the energy of the incoming wave over an extensive area thus preventing concentrated wave pressure impacts.

Need for Change

Although or perhaps owing to the fact that current dimensions and construction methods have stood the test of time for so long, it makes sense and is advisable to question current practices and consider alternatives. There are manifold reasons for changes and innovations – not simply in Germany – including:

- the reduction in the availability of soils for dike constructions,
- restricted areas for extending dikes both offshore and onshore,
- aspects pertaining to nature conservation,
- aspects pertaining to tourism,
- as well as economic aspects in particular and
- desired future capacity to adapt to changing hydrologic general conditions resulting from climate change.
Potential for Change

We wish to stimulate a thought process by dint of a number of ideas and approaches:

Natural soils capable for sustainable dikes must in particular fulfil a scaling function while at the same time be able to cope with the elements and be installable. In this connection, an interesting example is the DredgeDike project with the aim of using dredged spoil as dike material.

New challenges are also faced by the dike core based on environmental protection reasons. For instance, according to the Wattenmeer Strategy 2100, dike reinforcements are no longer permitted to lead to increasing the climate-related sand deficit in the Wattenmeer (tidal flats) in Schleswig-Holstein in Germany.

Should climatic changes necessitate higher dikes in future, they will be considerably wider if current construction methods are adhered to. Sea dikes in Germany at present show slope gradients of between 1:6 and 1:10 for the outer slope and between 1:3 and 1:5 for the inner one. Given such inclines increasing the dike height by one metre for example requires the dike foundation to be widened by ten to fifteen metres. As frequently such an amount of space is not available, the question arises as to whether construction methods that display greater stability against erosion and are less susceptible to wave impacts permit more compact dike dimensions.

Long dike sections run through intensively exploited areas such as cities, ports or bathing resorts. Generally speaking, at present for safety reasons dikes are not available for other purposes. There are examples of multifunctional dikes or multiple utilisation of dikes both internationally as well as occasionally in Germany. In view of ever growing pressure on our coasts it is worthwhile considering whether and under which construction technical prerequisites such multifunctional solutions can be increasingly applied without any drop in safety standards.

It is customary in practice to identify limits during safety tests and the dimensioning of dikes. In concrete terms this applies for instance to the application of berms and perpendicular structures on the slope or top (wave walls) and slope gradients of less than 1:8. So far no generally valid approach for establishing the influence of perpendicular structures on the slope or top or very flat or wide berms exists.

Compensation areas are required for the dike as a protective structure. In many cases such areas are lacking either in the proximity of the dike or on the dike itself. As a result, it is necessary to seek expensive compensation areas for dike construction measures and design them to harmonise with nature. The dike itself has so far not been available for this purpose. Modified “ecological” construction methods could in future, facilitate direct compensation at dikes. “Building with Nature” instead of “Building in Nature” should be the slogan for future developments.

With costs ranging from 3 to 5 million euros per km, dike construction is very expensive in Germany – and given that the total dike length is more than 1,200 km alone along the German coasts. Developing and trying out new construction methods and materials in order to reduce costs are essentially imperative.

Efficient dike maintenance and innovative methods for dike monitoring afford a considerable potential for saving costs and enhancing safety. Are inspection patrols to identify the breaching of a dike still sufficient and contemporary nowadays? Would it not be better if dikes like other protective structures were permanently monitored by sensors over their entire length during storm surges?

Let us jointly tackle the social and technical challenge by:

- Carrying out sustained PR work in order to make the population aware of the importance of coastal and flood protection – not simply during critical phases brought on by storm floods, hurricanes and tsunamis
- Contributing our expert knowledge to political commissions and decisions
- Developing innovative technical solutions – in other words new methods for construction and dimensioning, building products and materials and monitoring methods for dikes taking nature conservation and economic aspects into consideration
- Exchanging experiences across borders, for example in January 2018 at the International Symposium on Hydraulic Engineering in Aachen (IWASA) at the RWTH Aachen (https://iwasa.de/en), and learning from each other
- Finding holistic solutions by seeing the bigger picture beyond one’s own ivory tower

Yours,
Holger Schüttrumpf + Peter Fröhle

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Rapid urban development in Chile since the early 2000s motivates real estate developers to devise higher buildings and deeper excavation pits, in order to use the available spaces more efficiently. As an additional complication, Chile is among the countries with the highest seismicity in the world. Therefore both the structural and the geotechnical design are confronted with complex and challenging conditions, and have to fulfil the safety requirements anchored in national construction regulations. The best technical and economical solutions have to be found.

The implementation of worldwide well-known technologies and construction methods, such as micropiling, has enabled designers to come up with optimal solutions in order to satisfy urban development requirements.

The following article presents the case study of the “Piedra Real” project, located in Concepción, Chile’s second largest city. The entire construction project as well as its specific geological and geotechnical parameters is briefly described.

This paper focuses mainly on the geotechnical design of the permanent uplift reinforcement of the building, provided by self-drilling grouted Ischebeck Titan micropiles. Design considerations regarding the structural capacity and durability of the reinforcement system are presented.

Rapid urban development in Chile has resulted in higher buildings and deeper excavation pits. This article describes the challenges to achieve uplift and seismic safety of the eighteen-storey Edificio Piedra Real in Concepción in Chile. The main focus is on the geotechnical design and the durability of the permanent uplift reinforcement using micropiles. Furthermore, the interaction with other relevant aspects is discussed.

Furthermore, the direct interaction of the uplift reinforcement system with other relevant aspects of the project, such as the temporary shoring of the excavation pit and temporary groundwater lowering, are discussed.

Finally, an overview of other potential application fields for the case study solution, such as road and railway infrastructure, is presented.

2 The Project Piedra Real in General and the Challenges of Uplift Retention

The project Piedra Real (Las Heras) is located in the downtown area of Concepción (Bio-Bio Region), the second largest city in Chile (Fig. 1). The condo consists of four 18-storey buildings, with two underground parking levels. The foundation of the complex was ac-
complished by a reinforced concrete slab with a surface area of about 5,680 m². The projection of the towers accounts for about 40 % of the total building area (Fig. 2).

The architectural project required a free height of 7.0 m for the underground parking levels (measured from the top of the foundation slab). Due to the local groundwater conditions, the foundation slab is subjected to hydrostatic uplift, which – together with seismic actions – requires permanent reinforcement of the foundations to absorb the tension forces. The reinforcement was accomplished by self-drilling grouted Ischebeck Titan micropiles.

3 Geological Description

The city of Concepción is built on tertiary sediments in a valley created by a graben, with metamorphic and granitic formations to the north, east, south and beneath these Tertiary sediments. The sedimentary valley is depicted along with the 3 major faults that run through the area (Fig. 3, [1]).

4 Chilean Seismicity – The Maule Earthquake

Chile is one of the countries with the highest seismic activity in the world. According to the USGS [2], 3 out of the 20 largest earthquakes, recorded worldwide since 1900, have occurred in Chile.

On Saturday 27 February, 2010 (03:34 local time) an earthquake with a magnitude (Mw) of 8.8 struck the central-south region of Chile, with a following tsunami that hit the coastal areas (Fig. 4). This event had a deep impact on the public perception of the high vulnerability of the infrastructure to seismicity, since about 75 % of the Chilean population was affected, and an estimated loss of about US$ 30 billion resulted [6, 7].

The earthquake-induced ground tremors had a total duration of about 140 s, with the strongest phase lasting 40 to 50 s [3]. In the region most affected by the tremors, ground accelerations exceeded 0.05 g for over 60 s in most records, and more than 120 s in the records for the Concepción area [5].
Influence of Seismicity on the Chilean Design and Building Regulations

Chile has experienced a high number of large magnitude earthquakes, and as a result, the relevant Chilean Design Codes NCh430.Of 2008 [10] and NCh433 Of 96 [11] have been subsequently updated, based on the lessons learned from past incidences of damage. The Maule Earthquake was no exception, and as a result, up to two amendments – in the form of decree laws – were implemented to update and improve building practice:

▶ DS N° 60 (December 13, 2011) [12] sets out the new requirements for design of reinforced concrete structures, replacing the NCh430-Standard
▶ DS N° 61 (December 13, 2011) [13] sets out the new requirements for seismic building design, modifying and/or complementing the NCh433-Standard, regarding the soil classification and the design spectra.

Also other national standards have been implemented, to establish the requirements of special geotechnical works, such as excavations NCh3206.Of 2010 [14].

Fig. 4: The Maule earthquake in 2010:
Left: Main shocks and aftershocks of Mw ≥ 4 between 2/27/10 and 3/26/10 [3]
Middle: Strong motion record of Downtown Concepción [4]
Right: Preliminary processed records of maximum accelerations [5]

Fig. 5: Examples of damaged buildings: Fully collapsed Alto Rio Tower (left) and partially collapsed O’Higgins Tower (right) [9]
6 Uplift Reinforcement – Design and Implementation

6.1 Geotechnical Parameters of the Subsoil

The geotechnical parameters for the construction site were defined by geotechnical prospection as presented in Table 1. The groundwater table is located near the surface at depths between 1.5 and 3.5 m (average groundwater table –2.0 m). Due to seasonal fluctuation, it was recommended to consider a design groundwater table at a depth of 0.5 m below ground level [15]. The competent soil for the foundation of the structures corresponds to the layer H3, which was classified under category C according to the DS N° 61.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness [m]</th>
<th>$\gamma / \gamma'$ [kN/m$^3$]</th>
<th>$\phi'$ [°]</th>
<th>$c'$ [kN/m$^2$]</th>
<th>$N_{opt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1: sandy artificial fillings</td>
<td>1.9 - 3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>H2: SM / SP - SM loosen to medium dense</td>
<td>2.7 - 4.5</td>
<td>17/95</td>
<td>28</td>
<td>0.0</td>
<td>10 - 30</td>
</tr>
<tr>
<td>H3: SP - SM / SM dense</td>
<td>undefined (below 6.0 m under ground level)</td>
<td>18/10.5</td>
<td>40</td>
<td>0.0</td>
<td>40 - 60</td>
</tr>
</tbody>
</table>

Table 1: Geotechnical Parameters

6.2 Structural Design

The structures were designed in accordance with the above mentioned Chilean Standards NCh433 and decree laws DS N°60 and DS N°61. The towers were conceived with seismic dissipaters installed at the sides (Fig. 6), in order to reduce the seismic effects both in the structural and non-structural elements [16].

Focusing on the foundation design [17], the structural analysis resulted in the reinforcement requiring to absorb the following:

- **Tension loads**, mainly caused by uplift in the areas outside the projection zones of the towers
- **Compression loads** at certain locations under the towers, where the admissible load-bearing capacity of the subsoil was exceeded. The reinforcement was produced by 108 CFA-bored piles (diameter 0.6 m), however their design is not discussed further in this article.

The distribution of the required reinforcement is presented in Fig. 7 (for half of the structure), and summarized in Table 2 (for tension loads).

6.3 Geotechnical Design of the Uplift Reinforcement

There is no official Chilean norm for the design of special geotechnical works, thus the geotechnical design of the reinforcement elements was carried out in compliance with the German Standard DIN 1054:2010-12 [19] and the EA-Pfähle 2007 [20], considering the safety concept based on the Partial Safety Factor Approach, where the relationship \( Ed \leq Rd \) (1)

\[
E_d \leq R_d \]

with:

- \( E_d \) – design effect of actions
- \( R_d \) – design resistance

Fig. 8 displays the geotechnical verifications required for the design of the uplift reinforcement and the results are summarized in Table 3. The reinforcement was produced with self-drilling Ischebeck Titan grouted micropiles. Micropiles transfer the loads (tension and/or compression) coming from the structures to the foundation ground via skin friction.

The Titan micropiles consist of continuously threaded hollow bars, made out of seamless fine-grained steel pipes (S460 NH), installed via rotary percussive drill-

<table>
<thead>
<tr>
<th>Total quantity [-]</th>
<th>Design (ultimate) load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>516</td>
<td>400</td>
</tr>
<tr>
<td>104</td>
<td>530</td>
</tr>
<tr>
<td>32</td>
<td>710</td>
</tr>
</tbody>
</table>

Table 2: Summary of the required reinforcement for tension loads [18]
ing. During the drilling process, the micropiles are continuously grouted (dynamic injection), building a rough interlocking at the interface grout-soil, thus increasing the skin friction. According to [20], the characteristic skin friction value of $q_{s,k} = 215 \text{kN/m}^2$ was adopted for the layer H3. The components, the installation process and a typical cross-section of the grouted body are presented in Fig. 9.

### 6.4 Durability

For permanent reinforcement systems, the design load-bearing capacity needs to be guaranteed during the serviceability of the planned structures. In the case of micropiles, it must be ensured that the steel load-bearing elements are effectively protected against corrosion.

The permanent corrosion protection of 100 years of the Titan micropiles is provided only by dint of sufficient grout cover, as highlighted in the National Technical Approval Z.34.14-209, granted by the German Institute of Building Technology (DIBt) [22]. The steel quality and thread geometry of the Titan hollow bars induce a regular cracking pattern in the grouted body, with crack widths smaller than 0.1 mm, considered to be self-healing (Fig. 10).

### 6.5 Installation of the Micropiles and Load Tests

The micropiles were installed from the bottom of the excavation pit (Fig. 11). Up to three drilling machines were used for the installation: two Tamrock rigs and one Morath drifter (HB70), attached to a telescopic jib (Manitou). A drilling rate of approximately 100 m/day/equipment was achieved.

After installation, load tests were carried out in order to verify the adopted design considerations, especially regarding the skin friction of layer H3. Three test mi-
cropiles with different outside and inside diameters were executed: one 73/53, one 52/26 and one 40/16. The micropiles were subjected to maximum test loads, equal to 90% of the yield force (at 0.2% elongation) for the correspondent micropile type, without reaching the ultimate limit state of the pull-out resistance (Fig. 12). The required safety level and the adopted design considerations were validated. The registered displacements for the design loads were between 9 and 13 mm.

According to [22] the required grout cover for the micropiles considered in the design was 40 mm, in order to guarantee permanent corrosion protection. Measurements at the micropiles’ necks showed grout covers of at least 45 mm, thus fulfilling the durability requirements.

7 Interaction with other relevant Project Items

During the preliminary engineering approach, a 2.5 m thick bottom slab was contemplated to resist the uplift forces, taking into account the average groundwater level. For the planned two underground parking levels, the project required an excavation pit with a free height of 9.5 m. Temporary excavation shoring consisting of an anchored soldier pile wall and a network of well-points to lower the groundwater had to be implemented. The
The corresponding requirement is schematically presented in Fig. 13.

The implementation of the presented uplift reinforcement solution had also a positive effect on the temporary shoring and the groundwater lowering. The excavation depth was considerably reduced to 7.6 m, thereby making an optimised solution of the shoring and the groundwater lowering possible (Fig. 14).

The lateral support for the soldier pile wall was also accomplished with Titan tension piles (passive anchors). This solution was also proven to be more convenient than the originally intended use of strand anchors, since the higher installation speed of self-drilling anchors (> 100 m/day) accelerated the installation of the excavation shoring. Furthermore, installation of anchors and micropiles was carried out using the same equipment, thus simplifying the logistics at the construction site and reducing its costs.

8 Summary and Conclusions

The implementation of an uplift reinforcement system for an 18-storey building located in the city of Concepción in Chile, consisting of self-drilling grouted micropiles is described. The difficult conditions of the project,

---

**Fig. 11:** Installation of the micropiles (left) and load tests (right)

*Source: Pilotes Terrates SA*

---

**Fig. 12:** Results of the executed load tests
mainly related to the seismic activity of the region as well as the local geology, imply complex design challenges in order to provide optimal solutions that fulfil the safety requirements established by the national construction regulations.

The presented solution highlights the technical benefits of micropiling – in this case, the significant reduction of the amounts of reinforced concrete and the associated logistics along with time-consuming preparation and installation. Other relevant project factors, such as the temporary shoring and groundwater lowering, necessary to accomplish underground levels could also be optimised. This had a favourable impact on the project as a whole, in terms of structural requirements, execution time and cost reduction. Successful interaction between structural and geotechnical designers is required from the early stages of the planning process on in order to facilitate and optimise the design processes.

It is evident that the use of micropiles as uplift reinforcement systems can be applied to other types of infrastructure, such as road and rail underpasses, caissons and tunnels.

9 References

A presentation and paper about the project was first published at the 13th International Workshop on Micropiles, Vancouver, BC, Canada. ISM, ADSC and DFI partnered to present this workshop from March 29 to April 1 on the theme of “Micropiles: Resisting and Mediating the Effects of Mother Nature”:

López, F.; Fernandez, J. M.: Edificio Piedra Real – Concepción, Chile – Case study of an uplift reinforcement project.

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Technical Protection against Rockfall – Design, Monitoring and Maintenance according to the Austrian Guideline ONR 24810

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Dr. Gernot Stelzer, Trumer Schutzbauten GmbH, Kuchl, Austria
Dr. Maik Hamberger, Trumer Schutzbauten GmbH, Fuerth, Germany

1 Introduction

In 2008, the European Organisation for Technical Assessment (EOTA) published a new guideline for the testing and certification of rockfall catchment fences using flexible nets entitled “ETAG 027: Guideline for European Technical Approval of Falling Rock Protection Kits” [1]. The document presented a harmonized approach to product testing that replaced national guidelines/standards and focuses on the testing methodology and material documentation but stops short of performance evaluation. The acceptance ETAG 027 has been far reaching since its publication and is the standard used throughout the world with the exception of Switzerland where traditional testing guidelines are sustained.

Soon after the acceptance of ETAG 027, it was recognized in Austria that an additional guideline that focused on the verification, application, and performance evaluation of rockfall catchment fences was required. As such, these topics were incorporated into a comprehensive guideline published by the Austrian Standards Institute entitled "ONR 24810, Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance" in 2013. It covers not only rockfall catchment fences but also many other forms of mitigation including, stabilisation with anchoring and mesh/nets, embankments, and galleries. Though it was published in German, the document received interest from throughout the world and in early 2017 an English version was published and is now available from Austrian Standards Plus [2, 3, 4]. The guideline addresses how mitigation structures are to be implemented, in particular the standardization of site investigation, design, construction and maintenance.

Only those sections of the ONR 24810 that pertain to rockfall catchment fences are discussed herein.

2 Consequence Classes

Integrated throughout the ONR 24810 is the concept of consequence classes which were adopted from the European Standard EN 1990:2003 “Eurocode: Basis for structural design” [5]. The classes are a qualitative rating of the potential outcome should a structure fail, in particular, with regards to the degree of loss of human life, and economic, social or environmental impacts while considering both the effects on the area of protection as well as the effects on the mitigation system’s integrity. Table 1 details the three different levels of consequence classes: high, medium and low.

Once a consequence class is assigned to a project, it is then used to determine the required level of safety for various aspects, such as the design and resistance forces, trajectory analysis and geometry of mitigation structures, or performance criteria. In general, as the consequence class rises, so does the required safety level applied.

3 Site Investigation

The ONR 24810 begins with a description of the necessary elements of a site investigation where the goal is to verify the hazard and to collect the necessary information to carry out technically correct mitigation. The site investigation includes both a desk and field component.

The desk investigation is carried out prior to the field investigation and is focused on collecting baseline information of both the hazard, areas of interest to protect, and the elements at risk. It includes review of historical data, databases, maps (e.g., topographical, geological, infrastructure, etc.) and other sources that help focus field investigations.

Table 1: Description and examples of consequence classes CC1 to CC3

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>Description</th>
<th>Examples of buildings and civil engineering works</th>
</tr>
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<tbody>
<tr>
<td>CC 1</td>
<td>Low consequence for loss of human life, and economic, social or environmental consequences small or negligible</td>
<td>Agricultural buildings where people do not normally enter (e.g., storage buildings), greenhouses</td>
</tr>
<tr>
<td>CC 2</td>
<td>Medium consequence for loss of human life, economic, social or environmental consequences considerable</td>
<td>Residential and office buildings, public buildings where consequences of failure are medium (e.g., an office building)</td>
</tr>
<tr>
<td>CC 3</td>
<td>High consequence for loss of human life, or economic, social or environmental consequences very great</td>
<td>Grandstands, public buildings where consequences of failure are high (e.g., a concert hall)</td>
</tr>
</tbody>
</table>
The field investigation verifies and expands on information collected during the desk investigation. It can be subdivided into three zones: initiation, transition and deposition. Some examples of information collected for each zone are:

- **Initiation zone**
  - Rock mass characterization, joint and discontinuity patterns, and analysis, failure mechanisms, etc.

- **Transition zone**
  - Morphology, dampening buffers, evidence of frequency, bounce height indicators, etc.

- **Deposition zone**
  - Site morphology, relief (relative to initiation zone), identification of debris from previous events, evidence of frequency, bounce height indicators, accessibility (in particular for construction and maintenance), location of elements at risk, etc.

The results of the field investigation are then used for determining inputs that will be used for the verification of the mitigation structure, i.e., block size distribution, event frequency distribution, and bounce height distribution. In addition, homogeneous areas are identified and a pre-selection of locations for mitigation measures is defined.

### 4 Design Parameters

#### 4.1 Design Block Selection

There are two methods of defining the design block: A simplified approach or standard approach. The simplified approach allows for an expert to define the design block based on his experience and information obtained during the site investigation, but it can only be employed if at least one of the following are true:

- Less than 100 blocks present in the deposition zone
- Less than 100 jointed rock bodies present in initiation zone

- Consequence class defined as CC1
- Event frequency falls under EF1 or EF2 (Table 2)

Where the requirements of the simplified approach cannot be met, a standard approach must be used. Thereby the design block is defined based on the event frequency recorded during the site investigation as (Table 2):

- 98\textsuperscript{th} fractile of the block size distribution when the frequency class is rated as very high frequency
- 97\textsuperscript{th} fractile of the block size distribution when the frequency class is rated as high frequency

<table>
<thead>
<tr>
<th>Event frequency class</th>
<th>Event frequency n</th>
<th>Fractile for design block size</th>
</tr>
</thead>
<tbody>
<tr>
<td>EF 4 (very high)</td>
<td>n \geq 10 (\geq 10 events per year)</td>
<td>V\textsubscript{98}</td>
</tr>
<tr>
<td>EF 3 (high)</td>
<td>1 \leq n &lt; 10 (1 bis 10 events per year)</td>
<td>V\textsubscript{97}</td>
</tr>
<tr>
<td>EF 2 (low)</td>
<td>0,03 \leq n &lt; 1 (1 event per year to 1 event per 30 years)</td>
<td>V\textsubscript{96}</td>
</tr>
<tr>
<td>EF 1 (rare)</td>
<td>n &lt; 0,03 (&lt; 1 event per 30 years)</td>
<td>V\textsubscript{95}</td>
</tr>
</tbody>
</table>

#### 4.2 Energy and Bounce Height

The method for determining the design energy and bounce height are left to the user’s discretion, though they should be carried out using state-of-the-art modelling techniques for trajectory analysis that is based on the data obtained from the site investigation and the design block. The results are checked against data collected during the site investigation to ensure they are realistic. The design energy and bounce height is then taken from the model at the location of the desired mitigation structure and used for verifying potential systems.
5 Verification of a Rockfall Catchment Fence

Once the design parameters are defined, a verification of a specific rockfall catchment fence can be conducted based on the principle that the design values of the event are less than or equal to the resistance values of a structure, i.e., $E_d \leq R_d$. In this effort, the energy verification of the structure capacity is carried out independently from the verification of the structure height. In addition, special performance criteria can also be implemented.

5.1 Energy

The energy capacity of a structure is verified by comparing the design impact energy $T_{E,d}$ to the resistance capacity of the structure $T_{R,d}$ and is valid if the impact energy is less than or equal to the capacity of the structure. If this is not true, then a higher capacity system must be selected. Hereby the following equations apply:

$$T_{E,d} \leq T_{R,d}$$

with $T_{E,d} = T_{E,k} \cdot \gamma_{E,kin}$ .......................... (2)

and $T_{R,d} = T_{k,MEL} / \gamma_{T,R}$ .......................... (3)

Whereby:

$T_{E,k}$ $99^{th}$ fractile of the energy distribution obtained for the location of interest

$\gamma_{E,kin}$ partial factor of safety defined according to the consequence class in Table 3

$T_{k,MEL}$ Maximum Energy Level (MEL) class reported by manufacturer in ETAG 027 documentation

$\gamma_{T,R}$ partial factor of safety defined according to the consequence class in Table 4

5.2 Bounce Height

The required height of a structure is verified by comparing the design bounce height $h_{E,d}$ with the resistance height of the structure $h_{R,d}$ and is valid if the bounce height is less than or equal to the resistance height of the structure. If this is not true, then a system with a larger height must be selected. Where a sufficiently high enough system is not available within an energy class, then a higher capacity system that meets the height requirements must be used. Hereby the following equations apply:

$$h_{E,d} \leq h_{R,d}$$

with $h_{E,d} = h_{E,k} \cdot a_1$ .......................... (5)

and $h_{R,d} = h_{R,k} / a_2$ .......................... (6)

Whereby:

$h_{E,k}$ 95th fractile of the bounce height distribution, taken at the upper surface of the block

$a_1$ geometric coefficient defined according to the consequence class in Table 5

$h_{R,k}$ allowable nominal height of the system according to ETAG 027

$a_2$ reduction coefficient defined according to the consequence class in Table 6

5.3 Performance Criteria

The evaluation of system performance is carried out by comparing the allowable effects of an impact on a system according to the consequence class and as defined in Table 7 with actual damages recorded during the ETAG 027 testing. The recording of the necessary information is required under the testing guidelines and must be made available by the manufacturers. Important details are shown in Fig. 1.

6 Verification of Anchorage

The anchors of the structure are verified by comparing the design force $E_d$ to the resistance capacity of the anchor $R_d$, which is covered by two cases:

- the failure of the reinforcement element regarding tensile strength $R_{t,d}$
- pull-out capacity $R_{a,d1}$

It is valid if the design force is less than or equal to the resistance capacity of the anchor. If this is not true, then an anchor with a higher capacity must be selected. Hereby the following equations apply:

$$E_d \leq R_d$$

with $E_d = E_k \cdot \gamma_{E,i}$ .......................... (8)

and $R_d = R_{t,0,2k} / (\gamma_i \cdot \eta)$ .......................... (9)

and $R_{a,d1} = R_{a,k1} / \gamma_{a}$, with $R_{a,k1} = (R_{a,m})_{min} / \gamma_2$ .......................... (10)

Whereby:

$E_k$ characteristic value equal to the maximum force monitored during ETAG 027 MEL test

$\gamma_{E,i}$ partial factor of safety equal to 1.5
If literature values are used in place of anchor pull-out tests then an additional model factor $\eta_p$ is applied to the characteristic value of resistance to pull-out based on the consequence class as per Table 10.

### 7 Constructive Rules

The ONR 24810 also lays out some general rules regarding the appropriate layout and construction of rockfall catchment fences that are not addressed in ETAG 027. They are based on expert opinion and field experience.

- **Distance between catchment fence and object of protection**
  The minimum spacing between the objects of protection and the rockfall catchment fence is equal to the maximum elongation as reported for the MEL test in the ETAG 027 documentation with a factor of safety of 1.2, whereby the minimum distance is the maximum elongation plus 1 m.

- **Post spacing**
  Maximum and minimum post spacing shall be ± 2 m from the as-tested layout.

- **Row length without internal anchor**
  A rockfall catchment fence shall not have sections longer than 60 m where the forces in the bearing and middle ropes are led into the subsurface.

- **End field placement**
  An effort should be made to place end fields outside of the primary hazard area. If the rockfall catchment fence shows openings between the end post and the net of ≥ 10% of the residual height, then it is absolutely necessary.

- **Direct rock wall connection**
  There are two accepted scenarios for connecting a fence into a rock wall that differ in how the fence reacts to impacts in the end field, specifically the degree to which the net is pulled away from the wall. The accepted configurations are shown in Figs. 2a and 2b.

- **Gully nets**
  Where gaps are present below the lower bearing rope due to undulating topography, the same net

---

**Table 7:** Optional requirements for rockfall catchment fences depending on consequence classes

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>Unacceptable damages during an MEL test</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC1</td>
<td>No additional requirements: ETAG 27 certification sufficient.</td>
</tr>
<tr>
<td>CC2</td>
<td>No opening of nets greater than or equal to 0.4 m below the residual height, between the lower bearing rope and net. No openings between the end posts and the net greater than or equal to 10% * of the nominal height if the end fields are located within the hazardous area. No rupture of the main nets, bearing ropes or retaining ropes. A rupture of the sewing rope or component used to attach the primary net to the bearing ropes is allowed if a new load bearing net border develops as non-positive connection to the bearing rope.</td>
</tr>
<tr>
<td>CC3</td>
<td>No opening of nets greater than or equal to 0.2 m below the residual height, between the lower bearing rope and net. No openings between the end posts and the net greater than or equal to 10% of the nominal height if the end fields are located within the hazardous area. No rupture of the main nets, bearing ropes or retaining ropes or the strands. Single wires are allowed to break (as long as it is not through the entire strand). A rupture of the sewing rope or component used to attach the primary net to the bearing ropes is not allowed.</td>
</tr>
</tbody>
</table>

* If the lateral openings are greater than or equal to 10% of the nominal height, the length of the line must be extended by a half module length. If the end module lies outside of the hazardous area, this condition can be neglected.

---

**Table 8:** Model factors $\eta$ for resistance of micropiles against failure

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>CC1</th>
<th>CC2</th>
<th>CC3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\eta$</td>
<td>1.30</td>
<td>1.30</td>
<td>1.50</td>
</tr>
</tbody>
</table>

**Table 9:** Partial factor of safety $\gamma_{s,t}$ for resistance of micropiles against pull-out

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>CC1</th>
<th>CC2</th>
<th>CC3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{s,t}$</td>
<td>1.20</td>
<td>1.20</td>
<td>1.40</td>
</tr>
</tbody>
</table>

**Table 10:** Distribution coefficient $\xi$, depending on number n of pretests

<table>
<thead>
<tr>
<th>n</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>≥ 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi$</td>
<td>1.40</td>
<td>1.20</td>
<td>1.05</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Technical Protection against Rockfall according to the Austrian Guideline ONR 24810

8 Maintenance and Inspection

Immediately following the installation of a mitigation structure, the system is at its peak performance. Following this, the system’s lifespan and usefulness will continuously degrade over time. As depicted in Fig. 4, the lifespan is dependent on the maintenance and inspection program implemented. There is a critical point where the damages to a structure become irreversible and the mitigation must be considered non-functional and no longer provides protection. The point in time at which this occurs can be prolonged by maintenance as determined during routine inspections. The ONR 24810 describes a standardized approach to implementing inspection protocols with a time schedule as per Table 12. Inspections are divided into five types.

- **First recording (FR)**
  The first recording is carried out after new constructions or for structures first entering inventory. The inspection is conducted in combination with a control inspection under the supervision of an expert and is focused on a detailed inventory and logging of any shortcomings.

- **Ongoing monitoring (OM)**
  Regular interval inspection aimed at determining impairment due to events, which is performed by trained personnel from the caretakers or stakeholders of the protection measure. The functionality of the entire structure is reviewed during a visual inspection, including checking brake functionality, elongation and residual capacity, net deformation.

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>CC1</th>
<th>CC2</th>
<th>CC3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inspection interval in years FR</td>
<td>initial</td>
<td>initial</td>
<td>initial</td>
</tr>
<tr>
<td>OM</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>C</td>
<td>10</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>SI and A</td>
<td>as required</td>
<td>as required</td>
<td>as required</td>
</tr>
</tbody>
</table>

Table 11: Modell factor $\eta_P, t$ for resistance of anchorage of rockfall catchment fences

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>CC1</th>
<th>CC2</th>
<th>CC3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\eta_P, t$</td>
<td>1.25</td>
<td>1.75</td>
<td>2.50</td>
</tr>
</tbody>
</table>

Figs. 3a and 3b show schematics of two potential solutions for gullies.

![Fig. 3: Solutions for gully nets where a) additional anchoring and an additional bearing rope are used or b) additional anchoring without an additional bearing rope is used](image)

![Fig. 2: Accepted solutions for connecting a system into a rockwall where a) extra internal anchoring is used or b) direct connection is used](image)

![Fig. 4: Demonstration of preservation status of protection measures with regular and neglected maintenance tasks over the service life](image)
and damage, damages to ropes, verification of nominal height, evaluation of debris in the system, etc. The inspection can be carried out by line-of-sight.

- **Control (C)**
  The control inspection is conducted by trained personnel or qualified experts on a schedule determined by the consequence class (see Table 12). It serves to assess individual components in a more detailed manner and must be done on-site. Deficiencies are to be identified and subsequently repaired or, alternatively, a more frequent inspection interval implemented.

- **Special inspection (SI)**
  Special inspections (Fig. 5) are independent of routine inspections and are performed at the recommendation of ongoing monitoring or following exceptional events that have acted upon the structure (e.g. impact event, storm, avalanche, mass movement, accident, etc.).

- **Audit (A)**
  The Audit is conducted when another inspection, e.g. control, cannot determine the functionality of the mitigation structure and is conducted by a knowledgeable expert or possibly an inter-disciplinary expert team. The nature of the inspection depends on the component(s) being inspected and may include more intrusive/involved test procedures to help determine the overall safety or state of the system (e.g. anchor pull tests).

9 **Summary**

The ONR 24810 provides a comprehensive framework for the planning, implementation, construction and subsequent maintenance of rockfall mitigation measures. Regarding rockfall catchment fences, it includes methodology for verifying the energy, height and anchorage requirements of a particular rockfall catchment fence based on a design event and ET AG 027 documentation provided by catch fence manufacturers. Constructive rules and maintenance routines are presented that help ensure a proper installation and the continued safe upkeep of the system.

10 **References**


[1] EOTA: Guideline for European technical approval of falling rock protection kits (ETAG 027), February 2013


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Neckar Bridge in Stuttgart – Gabions for Working Platforms installed underwater

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Dipl.-Ing. Thomas Groß, Hülskens Wasserbau GmbH & Co. KG, Wesel, Germany

The Neckar Bridge as part of the Stuttgart 21 project, required the installation of bored piles for the foundations. Off-site prefilled gabions have been used to build temporary work platforms due to limited space.

**Geotechnics • Hydraulic engineering • Gabion • Construction management • Logistics**

The planning and implementation of new construction projects within urban centres poses major challenges for the project team. The infrastructure of city centres is often at its maximum capacity due to constantly increasing traffic volumes before improvement measures can even start to be taken. Limited space in the immediate vicinity of the construction site, as well as the impetus to reduce the duration of closures and diversions, require the use of off-site prefabricated components, whose delivery and installation can largely be integrated into a “just-in-time” construction process.

**Off-site prefilled Gabions**

Gabions are stone filled, welded or woven wire mesh baskets, which have been used for more than 130 years as river bank protection, mass gravity retaining walls, within reinforced soil structures, and increasingly in recent years as acoustic barriers. In contrast to conventional gabions which are assembled and filled on site, prefilled and transported gabions include larger wire diameters to the mesh, integrated lifting elements, as well as limited settlement of the rock fill through the use of particular vibrating tables during assembly. The productivity per vibrating table is about 20 to 25 units per day – each unit 2 m x 1 m x 1 m. Seven units can typically be transported per truck depending on load capacity; this corresponds to the weight of stone required to fill seven gabions in conventional on-site filling.

Double twisted wire mesh gabions and mesh river mattresses have been used successfully since 1894 as a scour protection intervention for the reinforcement of channel banks and beds, since they are able to adapt to various underground geometries and post-installation differential settlement. The high deformation capacity of Cubirock gabions under load is an essential aspect in the selection of suitable characteristics for hydraulic engineering.

**Application Neckar Bridge in Stuttgart**

The construction of the Neckar Bridge as part of the Stuttgart 21 project, required the installation of bored piles for the foundations of the bridge piers, using heavy construction equipment in the area of the embankment (Axis 400) and the area midstream (Axis 500). In order to set up temporary work platforms for the construction equipment, gabion support walls were required and were built mostly under water, and were backfilled with suitable material (Fig. 1).

Maccaferri was commissioned in spring of 2016 with the production, filling and delivery of 1,720 m³ Cubirock units. The client was the Bavarian construction company Max Bögl. The pre-measurements carried out by Maccaferri with own software, were coordinated with the specifications of the engineering companies involved on the project CDM Smith Consult GmbH and...
sbp gmbh, both from Stuttgart. The geological survey was carried out by the branch in Esslingen of Dr. Spang Ingenieurgesellschaft für Bauwesen, Geologie und Umwelttechnik mbH. The filling of the gabion units took place in the Baresel quarry in Ehningen, about 20 km from the building site, by the gabion specialist company Rock-Build from Bratislava, Slovakia. The deliveries to the construction site took place in October to December 2016 in two steps: by truck from the quarry (Fig. 2) to the port of Bad Canstatt and from there by ship to the construction site. Ship transport and the underwater installation of the gabions were carried out by Hülskens Wasserbau GmbH & Co. KG, Wesel (Fig. 3).

After completion of the installation the work platforms will be completely dismantled. This will provide the final channel cross-section required within the project plan, after having been a reduced navigable channel during the construction phase. For this purpose, it is necessary to be able to lift and remove the gabions, stored under water over a long period of time, without the risk of damage and also to avoid the divers being exposed to danger (Fig. 4). It is conceivable, that the gabions could be reused after completing the works.

Further Applications

Similar temporary work platforms have already been constructed in Copenhagen in 2012 to 2013 with more than 6,000 m³ of Cubirock in an urban lake. During the works, the entire material supply and installation as part of the City Metro Tunnel project, was successfully deployed from the platforms.

Not only hydraulic projects can benefit from the use of this concept. In January 2017, a further 400 m³ Cubirock pre-filled and transported gabions were delivered to Switzerland. They were distributed in small quantities of 20 units each to the local authorities of smaller villages along alpine national roads, in order to be able to immediately clear road sections after landslides, and to set up mass gravity retaining walls to secure unstable slope sections “just in time”.

Outlook

The use of pre-filled and transported gabions could be interesting for projects where it is difficult to create a traditional and cost-effective on-site built structure due to time or space constraints or where the access is difficult due to the load-bearing capacity of the ground.

The rapid reaction time to natural hazards is also of interest to railway companies, which could drastically reduce the duration of track closures, due to the use of pre-filled and transported gabions.

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The Fehmarn Belt Tunnel: A Megaproject for connecting Europe
Ajs Dam, Deputy Project Director, Femern A/S, Copenhagen, Denmark

Living in a country that consists of a peninsula and a vast number of islands both large and small, few appreciate the value of fixed links quite as much as the Danes do. Over the course of just one generation, Denmark has completed two major infrastructure projects. These are the Great Belt Link, which connects the Danish islands of Funen and Zealand and the Oresund Fixed Link between Malmö and Copenhagen. Both projects have increased mobility and prosperity in Denmark and its neighbouring countries.

The Danish government has assigned the state-owned company Femern A/S with planning, constructing and operating the Fehmarn Belt Tunnel. Femern A/S is part of Sund & Bælt Holding A/S, which is 100 per cent owned by the Danish Ministry of Transport. Sund & Bælt Holding A/S has extensive experience in executing huge infrastructure projects such as the bridge across the Great Belt or the Oresund Fixed Link. In 1998, the opening of the Great Belt Bridge reduced travel time between the two biggest Danish islands from one and a half hours to just 15 minutes. Later, in 2000 the Oresund Fixed Link brought Copenhagen and southern Sweden closer together, merging the housing and job market of the two countries.

The megaproject Fehmarn Belt Tunnel will connect mainland Europe and Scandinavia and surely be another continuation of success for Femern A/S (Fig. 1).

The project company expects the Fehmarn Belt Fixed Link construction to begin in 2020 and to be opened in 2028.

The State Treaty between Germany and Denmark
The idea of a fixed link between northern Germany and eastern Denmark is more than 150 years old. Back in 1863 the engineer Gustav Kröhnke had the vision of connecting the German island of Fehmarn with the Danish island of Lolland. The first step towards realising the Fehmarn Belt tunnel was the signing of the state treaty between Germany and Denmark in 2008. Both parliaments ratified the treaty in 2009 thus paving the way to build the Fehmarn Belt Tunnel. According to the treaty’s legal regulations, Denmark is obliged to bear the project’s costs and responsibilities for construction, service and financing. After considering several solutions for a fixed link, in 2011, the Danish government decided in favour of an immersed tunnel as the best solution to connect both islands. In 2016, Femern A/S signed four major construction contracts worth almost DKK 30 billion (EUR 4 billion) with the international contractor consortia that will be responsible for the construction of the Fehmarn Belt Tunnel between Rødbyhavn and Puttgarden (Fig. 2). The contracts account for over 75 percent of the project’s total construction costs thus creating a stable and reliable basis for the construction budget.

The final and long-desired milestone for the start of construction is German plan approval by the authority...
responsible for plan approval in Schleswig-Holstein. Femern A/S hopes to receive a plan approval decision in 2018. However, opponents of the project have already announced their intention to appeal the plan approval decision in a German court. In this case construction could start in 2020.

The Construction of the Fehmarn Belt Tunnel

The Fehmarn Belt Tunnel will be a showcase project: at 18 km it will be the world’s longest underwater tunnel combining road and rail tubes. It will feature some of the most advanced and innovative technology in terms of its design and construction. The Fehmarn Belt Tunnel will be five times longer than the Oresund Tunnel and three times longer than the Transbay Tube Tunnel in San Francisco, which is currently the world’s longest immersed tunnel.

The tunnel will consist of 89 individual elements, which will be constructed onshore and placed in a previously dug trench on the seabed (Fig. 3). The tunnel elements will be produced at dry-docks and subsequently floated into the already mentioned trench. Here a distinction must be drawn between two different types of elements: 79 standard elements each 217 m long and ten special elements with a length of 39 m, which will be installed every 2 km between the standard elements. Since the special elements contain two levels, they are higher and wider than the standard elements. They include a lay-by for service vehicles as well as a lower deck for the installation of electric and mechanic facilities. Portal buildings will be set up at both ends of the tunnel with the intention of storing the electric and mechanic facilities and equipment.

The tunnel itself will consist of two motorway and two rail track tubes. (Fig. 4) Each 11 m wide motorway tube will have two traffic lanes, one hard shoulder, one edge stripe and one guide wall. The rail track tubes, with a width of 6 m, will each contain one electrified track and will be able to cope with passenger trains travelling at speeds of up to 200 km/h and freight trains at speeds up to 140 km/h.

Every 100 m an escape route and an emergency door will be accessible individually in all tubes to ensure safety no matter what the emergency might be.
Benefits for Travellers and Transportation

Transportation through the Fehmarn Belt Tunnel will be more efficient. Currently, it takes 45 minutes for cars, lorries and passengers to cross the Baltic Sea by ferry. This does not include the waiting times for the scheduled departure. Freight trains in fact have to make a 160 km detour through the Danish mainland to reach Germany and vice versa. Meanwhile, it will take only ten minutes to cross the Fehmarn Belt via tunnel by car and just seven minutes by train. Travelling by train from Hamburg to Copenhagen in particular will benefit crucially, since today it takes four and a half hours to reach one’s destination and in future, travel time will be less than three hours. Travelling by train will become more attractive to commuters and tourists throughout entire northern Germany and Scandinavia. Europe and Scandinavia will be connected 365 days a year, regardless of wind or weather conditions, waiting times or bookings. The shorter route for freight trains will make transportation by train more competitive within Europe while the overall project will help to shift freight traffic from the road to the more environmentally friendly mode of rail transport.

According to forecasts, the level of traffic will substantially increase after launching the tunnel. Today, about 5,300 vehicles use the ferry between Puttgarden and Rødby. This number should increase to 12,441 vehicles, 73 freight trains and 40 passenger trains per day only five years after the opening of the Fehmarn Belt Tunnel.

Economic Benefits

One of the major benefits of the Fehmarn Belt Tunnel will be the increased flexibility of the “internal market”. This means, the movement of goods, people, services and capital will be facilitated by the improvement of infrastructure. Opportunities for growth extend from Ostholstein and Lolland down to Hamburg in the south and up to Copenhagen in the north. The direct link to the Scandinavian market and to the Oresund region will reinforce the export of various commodities and services for Schleswig-Holstein and Hamburg. Furthermore, the Oresund region, connected through the Fehmarn Belt Tunnel, will be an important hub for Scandinavian logistics, creating the chance to come together as one major region with over nine million inhabitants – from Hamburg, by way of Lübeck and Fehmarn right up to Copenhagen and Malmö.

The Fehmarn Belt Tunnel also creates new jobs, fosters knowledge development in the region and will thus contribute to increase prosperity and employment in the EU while creating a more integrated, open and competitive Europe. Net savings related to time as well as the lower transport costs due to shorter travelling times thanks to the Fehmarn Belt Tunnel will represent huge benefits for companies all over Europe. These savings are the equivalent of a productivity increase for businesses, since the costs per unit of output fall. Competitiveness will increase. The shorter travelling times will enable companies to reach a larger market than before. This offers the opportunity to reach new customers and suppliers. Businesses will have the opportunity to op-

Fig. 4: Tunnel entrance
timise their production and logistics. Many businesses and business sectors will witness differences in efficient management. This is of significance for global enterprises but also for medium-sized companies in the fields of logistics, tourism, consumer goods etc. Irrespective of where people and businesses locate along the axis between the existing centres of Hamburg and Copenhagen, some ten million consumers can be reached within only a few hours’ transport time. The commuting catchment area will also increase with the improvement of the infrastructure. The ability to recruit from a wider catchment area additionally increases the probability of finding employees with exactly the right skills to match business needs. More than 600,000 companies offer excellent job opportunities to the nine million highly qualified people living in the Fehmarn Belt region.

**Outlook**

In every sense of the word, this is a megaproject in terms of planning construction and scale. The Fehmarn Belt Fixed Link will be – by far – the world’s longest immersed tunnel and the longest tunnel for combined rail and road traffic. The cross-border nature of the project adds significantly to the complexity of the plan approval phase and is a showcase for cross-border cooperation. Interoperability and safety regulations extending to complex approval procedures are some of the reasons why Femern A/S is preparing thoroughly to ensure a quick start to construction, when planning permission arrives. Femern A/S is glad to contribute to a more efficient Europe and its infrastructure once again.

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**Ajs Dam**

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Damage to segmental Lining of Tunnels during the Construction Phase – Causes and Avoidance Strategies

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1 Motivation
In the past, numerous tunnels have been produced worldwide including here in Germany by means of mechanised tunnelling with segmental lining (Fig. 1). Tricky geological, static and local conditions had to be overcome in the case of many tunnels so that the tunnel tubes could ultimately be approved for rail, metro, urban rail or road traffic.

During the construction phase damage is frequently caused to the support, i.e. the tunnel lining, resulting in cost and time consuming repairs. The type of damage ranges from straightforward and non-critical hairline cracks by way of massive concrete spalling right up to the reinforcement layer and ingressing water extending to major misalignments.

The objective of this article is the targeted presentation and evaluation of damage mechanisms seen from the point of view of the client and those responsible for project monitoring/supervision. The focus will concentrate especially on the wide range of causes and the substantial influence of backfilling the annular gap on the support quality. The report is intended to serve as the basis for discussing the potential for improvement in construction management.

First of all, the load and deformation situation is dealt with, then the tasks and significance of backfilling the annular gap examined and pointers on quality requirements provided. After listing established types of damage and indications of damage frequencies, the causes of damage are discussed, avoidance strategies extrapolated as well as conclusions drawn.

2 Load and Deformation Situation for Installing Segments
In the case of mechanised tunnelling with shield machines the individual segmental elements are assembled to form a staggered ring pattern with a central position as possible in relation to the shield tail, protected by the shield tail of the tunnel boring machine (TBM). By means of various coupling systems – such as the frequently applied pot-and-cam system or the groove-
and-tongue system, temporary bolting as well as the frictional forces in the ring joint – the individual rings are coupled with one another. In this way, the different deformations of the individual rings are compensated, which result on account of varying local and chronological stresses occurring during the excavation as well as the different longitudinal joint positioning of neighbouring rings (Fig. 2).

In the recent past, so-called centering aids with a defined shearing limit have been installed to facilitate the assembly process and to support the coupling of annular joints with a smooth joint albeit not applied in theory. In this case the theoretical interconnection is achieved solely by the grip produced by friction in the annular joint (smooth joint system) rather than a combination of friction/form fit (e.g. pot-and-cam system).

The following load situation results for the structural rings n to n-2 (Fig. 2): whilst the drive for ring n is taking place, the ring n-2 completely and ring n-1 partially vacate the shield tail step-by-step. In this case, ring n-2 experiences the full earth and water pressure at the end of the drive, whereas ring n-1 is still mainly affected by annular gap grouting. In this connection, ring n-1 generally experiences a degree of ovalisation varying from a so-called “standing” or “lying” ring or individual segments experience a misalignment with the neighbouring segments of the same ring. As the ring is still partially within the shield tail and only a part section is influenced by the annular gap grouting pressures and coupling with the more pronouncedly ovalised ring n-2 has already occurred, a so-called “tulip effect” additionally takes place. After the excavation is completed the ring coupling with the then installed ring n results in a restorative effect and a coercive ovalisation for the installed ring n as such [1].

In addition, the stresses resulting from assembly and production tolerances, the loads from the thrusting forces and the control processes as well as in particular the buoyancy tendency of the segments and the tunnel tubes must be taken into consideration.

3 Tasks and Value of Backfilling the annular Gap

Backfilling of the annular gap during a shield drive is intended to retain the primary stress state of the surrounding subsoil by immediately supporting the annular gap with annular gap mortar as far as possible thus minimising surface settlements. For this purpose, the annular gap mortar must create a stiff and solid radial bedding for the segmental lining as early as possible so that damage caused by the introduction of loads from the TBM and the back-up trailer as well as the buoyancy tendency of the tubes can be avoided effectively during the mortar’s liquid phase. Generally speaking during this phase, it is attempted to arrive at increased drainage of filtrate water into the surrounding subsoil so that the flowability of the annular gap mortar is reduced as a result of the decreasing water content given a simultaneous rise in the shearing strength leading to...
increased strength development through the hydration of the cement. As the mortar grouting pressure is normally higher than the supporting pressure at the face and any existing water pressure is phased out, the mortar is drained owing to the hydraulic gradient vis-à-vis the existing pore water pressure $z$ in the direction of the neighbouring subsoil. The relations of the individual stiffnesses are relevant for the static-structural aspects of the interaction between subsoil, annular gap backfilling and segment and have to be taken into account at the planning stage [2].

In the case of geologies, which fail to enable the filtrate water to be transferred to the soil in spite of suitable grouting pressures, essentially longitudinal drainage via the control gap leading towards the extraction chamber is possible. However, this is really only of practical significance in the event of the protracted flow run of the drainage water in a longitudinal direction when given a very slow rate of advance by the TBM. In the event of hydraulically compact or quasi compact conditions in the soil, the annular gap mortar is scarcely or not at all be consolidated by releasing filtrate water. In such cases concrete-technological measures are needed to increase the stiffness of the annular gap material, e.g.:
- Changing the grading line (Fuller Curve) and the mix composition
- Increased cement contents
- Application of concrete additives to accelerate hydration etc. in combination with the retaining forces (press forces)

The recipes for annular gap mortars are in concept to be classified between soft and solid rock with regard to their composition. Thus when applying conventional mortars in solid rock basically low to very low water/binding agent values must be observed in contrast to soft ground. As this does not suffice in the case of very compact soils, in such cases it is necessary to add binding accelerators.

4 Quality Requirements

The quality requirements posed on a tunnel lined with segments are defined in various codes of practice depending on its type of use for metro and S-Bahn tunnels, road tunnels as well as rail tunnels. In this connection, priority is accorded tightness, surface integrity and longevity. For example, according to Ril 853 [3] for rail tunnels, among other things, joint geometry is necessary, which successfully copes with the inevitable assembly inaccuracies as well as deformations and constraints caused by tunnelling and external influences. Furthermore, production tolerances have to be selected in such a manner that the various segments in all intended combinations and positions can be assembled to form rings and the rings to produce a segmental lining and it is assured that no unacceptable constraints and stress concentrations occur. A further central demand relates to the production of a pressurised water-tight tunnel structure, which has in general, to fulfil the requirements of tightness class 2 in keeping with Ril 853.4201.
5 Types of Damage and Frequency

Studies relating to already completed tunnels indicate that the demands posed by the codes of practice on the segmental lining are frequently not entirely fulfilled or not fulfilled and accomplished over the entire length of the structures directly after excavating the tube. Far rather numerous, different types of damage and damage mechanisms can be identified. The main groups of damage are compiled in Table 1 for orientation purposes.

Often different views prevail relating to the number of types of damage that occur. Whereas the individual ring is interpreted as the smallest unit on the part of the manufacturer, as far as the client is concerned damage relates to the individual segments. As multiple cases of damage frequently prevail per ring, often damage statistics exist for a tunnel, which vary by 10 to 20 % of the damaged units.

Regardless of the means of presentation all cracks > 0.2 mm, concrete spalling undershooting the concrete covering and water ingresses must however be professionally repaired (Fig. 6) e.g. based on ZTV-ING [4]. Towards this end, essentially repair concepts are provided by the manufacturer, which define the use of suitable materials and the corresponding expertise of the personnel.

6 Discussion of segmental Damage caused by individual Factors of influence

In the event of damage to the segment, the cause of damage often cannot be clearly attributed to an incident or circumstances. The interrelationships between ring assembly and the further methodical stages right up to the complete bedding of the ring in the subsoil and the resultant large number of parameters and possibilities of influence are so complex that generally speaking several factors of influence have a role to play. Regardless of this the following factors, which can exert an influence on segmental damage, are discussed individually:

- Annular gap mortar
- Annular joint coupling
- Buoyancy effect
- Annular space grouting pressure
- Ring level quality
- Press forces and hydraulic control
- Assembly situation during ring construction
- Rate of advance
- Standstill and restart
- Kinematic interaction, TBM control and ring assembly sequence
- Concrete quality and segment production

In this connection, damage analyses were evaluated and a number of observations and studies for various shield excavations in soft and solid ground during the last ten years referred to. Furthermore, pointers for avoiding damage through the individual factors of influence were provided.

<table>
<thead>
<tr>
<th>No.</th>
<th>Damage pattern</th>
<th>Moisture state</th>
<th>Other Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>cracks perpendicular to the annular joint, starting from the side facing the drive</td>
<td>dry, temporarily or permanently water-bearing</td>
<td>partial expansion over entire ring width</td>
</tr>
<tr>
<td>2</td>
<td>cracks or hairline cracks perpendicular to the annular joint, starting from the bolt pockets</td>
<td>dry, temporarily or permanently water-bearing</td>
<td>–</td>
</tr>
<tr>
<td>3</td>
<td>cracks or hairline cracks parallel to the annular joint</td>
<td>temporarily or permanently water-bearing</td>
<td>–</td>
</tr>
<tr>
<td>4</td>
<td>cracks or hairline cracks with diffuse orientation (Fig. 3)</td>
<td>temporarily or permanently water-bearing</td>
<td>–</td>
</tr>
<tr>
<td>5</td>
<td>air side spalling in direction of the circumference at annular joint at the side facing the drive (Fig. 4)</td>
<td>–</td>
<td>especially given pot-and-cam systems, partially to reinforcement layer with concrete slabs amounting to a few decimetres</td>
</tr>
<tr>
<td>6</td>
<td>mountain side spalling in direction of the circumference at annular joint at side facing the drive</td>
<td>ingressing water through leaky joint strip possible</td>
<td>in the case of pot-and-cam systems in the vicinity of the pot</td>
</tr>
<tr>
<td>7</td>
<td>displacement of neighbouring rings or ring segments (Fig. 5)</td>
<td>with and without ingressing water via the joint</td>
<td>joint leakage due to: spalling or cracks or exceeding the maximum joint strip misalignment</td>
</tr>
<tr>
<td>8</td>
<td>surface concrete spalling in corner area and the erectors</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Fig. 6: Repaired concrete spalling
Source: IMM
6.1 Annular Gap Mortar

The annular gap mortar represents the agent for embedding the ring in the subsoil and has thus a substantial influence on possible damage to the segment. Cases of damage can occur in particular should deformations result, which were not or only insufficiently observed, thus causing deformations in the segment. Such deformations should be avoided by dint of a rapid and secure bedding process including the targeted complete filling of the annular space and speedily increasing the stiffness of the annular gap mortar in the annular space. Thus damage then occurs to the segment should these demands on the annular gap mortars either not be or only inadequately be observed.

In the case of excavations in soft grounds, e. g. gravels or sands, there are certain advantages:

- First of all, it is easier to fill the annular space completely in the case of most of the fluid-supported shields as no or only slight overbreak occurs.
- Secondly, generally speaking the annular gap mortar is well drained thanks to the mostly adequate available porosity of the prevailing subsoil with corresponding large kₐ-values, providing sufficient grouting pressures are available.

On the other hand, in the case of excavations in solid rock, the following effects in particular can cause damage to segments:

- Infiltrations of the annular gap mortar into the extraction chamber can occur and, as project experiences have revealed, enhance the buoyancy effect, even if these infiltrations are compensated by added amounts of mortar.
- The mortar’s lack of drainage potential already described also causes buoyancy effects.

6.2 Annular Joint Coupling

By and large the pot-and-cam system and more recently to an increasing extent the smooth joint system have been applied to interconnect the annular joints of neighbouring segmental rings. Thanks to the geometrical interlocking of the pot-and-cam system or friction within the smooth joint, partly combined with centering cones, as well as the application of temporary bolting in the annular and longitudinal joints, the aim is to compensate for deformations between the individual rings.

Frequently during tunnelling projects with pot-and-cam systems, misalignments (Fig. 5) and – depending on the direction of the misalignment – spalling at the air or mountain side or cracks in the annular joint and as damage scenario overloading of the pot-and-cam system (approx. 8 mm scheduled slip) are established. For cases of damage at the air side this generally signifies dry spalling in the form of concrete slabs; for cases of damage at the mountain side, cracks with flowing or leaking water affecting the sealing gasket and joint leakages can occur.

Frequently damaged levels with large-scale spalling, misalignments exceeding 15 mm in size and even enlarged gap dimensions were also determined. However, at the same time, similar cases of damage were also documented with misalignments measuring only 3 mm. On the other hand, misalignments of up to 20 mm could be established, which caused no damage at all. Overlapping, non-evident or unidentified influences can be responsible for these different results. Through twisting the annular joints towards one another the slip in the pot-and-cam system can become enlarged so that no damage results even in the case of larger misalignments. Conversely in the case of rings with an initial misalignment liable to cause damage, this misalignment can diminish during the subsequent assembly and excavation processes.

Static dimensioning with a tension force of 1.5 times the restoring force of the gasket frame in keeping with Ril 853.4005 [3] is executed for the segment bolts. Bolting is carried out immediately after the segmental elements have been assembled with the aid of a compressed air impact wrench with defined torque. Grommets are in some cases used as centering aids for the segment bolts so that two different states can result depending on the pretensioning forces. On the one hand, the washer can have contact with the concrete if the grommet is located in the bolt channel, and on the other hand, no contact at all if the grommet protrudes well outside the bolt channel. Essentially, the pretensioning force of the segment bolts initiates effects on the ring stiffness and in turn, on the capacity for the individual segments as well as neighbouring rings to move. The applied grommets increase the scope for movement on account of their plastic deformability as well as indirectly owing to the possibly low pretensioning forces if there is no contact between the washer and the concrete.

The theoretically established pressing and tightening torques for bolting segments should be checked and retained in the transition area of the liquid to solid phase of the annular gap mortar – some 10 to 15 rings depending on the rate of advance. This is to ensure that traction prevails in the longitudinal and cross joints and the segment link chain is stiffened.

In the case of the longitudinal crack damage scenario starting from the bolt pockets (No. 2 in Table 1) a local overstressing of the contact surfaces e. g. as a result of unscheduled high pretensioning forces can be presupposed, which leads to the concrete structure rupturing. In the process, crack lengths extending over the entire segment width can occur. The creation of cracks is favoured by misalignments, by means of which the pretensioning force is additionally increased. This type of damage can be reduced by the scheduled application of the pretensioning forces and assembling the segments without misalignments occurring.

Hard fibreboards are set in the annular joint to transfer load where high loads are introduced. Attention must be paid to suitable stress-strain behaviour of the boards, suitable adhesive as well as professional adhe-
sive bonding and skilled stage of the segments. During one project excessive misalignments of the segments were attributed to the fact that hard fibreboards were bonded with unsuitable adhesive, which detached after assembly and favoured displacements (Fig. 7).

6.3 Buoyancy Effect

In the segmental ring and grouting mortar system the structural rings float as scheduled immediately after leaving the shield tail in the fresh liquid mortar matrix. In this connection, the segmental ring attempts to move upward during the liquid phase on account of the hydrostatic pressure until compulsory bedding takes place in the soil prevailing above the roof.

The shear strength of the mortar mass and its chronological development after it has been grouted into the annular gap exert a substantial influence on the buoyancy effect [5]. An adjustment must take place here related to the annular gap width, the unit weight of the liquid mortar, the subsoil conditions as well as the procedural parameters [6]. Buoyancy can be effectively reduced through prompt drainage of the liquid mortar matrix and/or concrete-technological adjustments such as grain-to-grain structure, Fuller Curve and diminished water/binding agent contents. An equilibrium of forces prevails via the minimum press forces for introducing the determining pretensioning force of the tubes providing the pretensioning forces are higher in each construction phase than the forces deriving from the buoyancy tendency and the transferability of the pretensioning forces into the annular joint is assured.

The necessary consolidation of the annular gap mortar and the bedding of the rings must be accomplished at the latest when load is imposed by the back-up trailer. In this respect, considerable differences result regarding the mortar technology and excavation parameters for excavations in soft grounds with regard to the necessary support pressure and the possible rapid drainage and in solid rock structures with hydraulically denser formations.

Owing to the movements or hampered movements during the application of ring couplings stresses are initiated in the annular joint, which can damage the reinforced concrete precast parts. Providing neighbouring rings are uniform and float upwards by less than 20 mm according to observations, the influence on the ring joints as far as cases of damage are concerned is usually non-critical. On the other hand, the lever arm of the link chain is enlarged as a result of each additional ring exposed to the buoyancy forces in the liquid mortar mass. In this way, excessive stresses and cases of damage can result – frequently at ring n - 3 or higher.

Furthermore, buoyancy effects or misalignments are critical, which occur over a short distance (a few segment rings). They can cause spalling and contacts between slanting pot-and-cam flanks by exceeding the maximum ring clearance. In the event of excessive strains cracks in the pot wall can be created and shear off cams or pot walls (Fig. 8). The ring coupling additionally activated in the case of a misalignment can lead to contact between the segment bolts and the bolt channel wall. This can on the one hand, result in shear stresses affecting the bolts and on the other, an increase in the contact force of the bolt head and possibly cracks in the bolt pockets extending crescent-shaped around the pocket to reach the annular joint.

In the event of misalignments of about 20 mm in size frequently the maximum permissible misalignment of the installed sealing gasket is exceeded in addition to mechanical damage being caused to the concrete so that sustained tightness is no longer evident or simply cannot be verified.

When “smooth” joints are applied the problem complex surrounding spalling and crack formation is admittedly minimised, nonetheless, it is essential that the degree of misalignment is confined to assure tightness.

6.4 Annular Gap Grouting Pressure

Grouting pressure peaks occurring locally during the filling of the annular gap can exert an influence on the segment quality. Essentially the quantitatively largest...
loads affecting the segmental ring prevail during the grouting process. The partial load at the end of the ring upon vacating the shield tail is unfavourable in terms of the link chain as the entire ring is first fully loaded once it has been further advanced. The joints at the end of the ring are further compressed as a result of the grouting pressure, whereas the joints at the beginning of the ring gape simultaneously so that induced stresses can occur in the longitudinal and annular joints and favour damage given interaction with other effects such as buoyancy.

On the other hand, if minimum grouting pressures are not maintained this can also lead to the segment quality being damaged as the annular gap has possibly not been properly backfilled. Subsequently this can lead to the segment not being adequately bedded and to increasing buoyancy effects especially in the case of excavations in solid rock. Minimum grouting pressures relate to the occurring support pressure plus an offset so that backfilling is completely accomplished. The offset is the grouting pressure to be added to the support pressure in order to assure the effect of the friction of the mortar (depending on the viscosity) in the pilaster as from the pressure measured at the pilaster entry as well as "pumping up" the mortar from the pilaster outlet of the upper pilaster to the roof. Generally speaking, 0.5 to 1.0 bar must be applied.

6.5 Ring Level Quality

Longitudinal cracks perpendicular to the annular joint, starting at the side facing the excavation, represent the typical case of damage if a bending load occurs around the strong segmental axis, in other words, the rotary axis perpendicular to the outer/inner side. A load is known in this case as a deep beam (Fig. 9). Such a load can only occur if there are bearing deficits in the form of a so-called trough or saddle position owing to unevenness in the ring level (Fig. 10). In such cases, press forces must be diverted through bending around the strong axis. Cracks thus mainly occur in the roof and floor area, i.e. in the TBM’s main control area. The loads stemming from the driving cylinders are no longer conducted through the segments in the form of a column model. Longitudinal cracks can occur given a support gap of roughly 2 mm in size commensurate with the above cited model.

If longitudinal cracks occur practically over the entire ring width starting from the side facing the excavation, it can be assumed essentially that a saddle position is responsible. The damage pattern is reversed for the trough position. Classification as a tensile strength crack appears implausible. This would tend to be confirmed if the crack width were to extend over the entire thickness of the structural part.

Basically details on the point in time when the damage occurred are necessary to determine or verify the causes for the crack with greater precision. On the one hand, prior damage can result in the form of hairline

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*Fig. 8: Misalignment in the annular joint, damage pattern – spalling of pot wall [2]*

*Fig. 9: Longitudinal cracks in the segment owing to bending around the strong axis resulting from withdrawal of support (diagram)*

*Source: IMM*
cracks, which are scarcely identifiable at the point in time they occur, or the load exceeds the stress limit of the segmental elements to such a degree that visible cracks occur within the confines of the shield jacket. It is imperative that qualified personnel maintain ongoing visual contact in order to establish the point in time when the damage occurred.

Furthermore, an examination of the position of the ring level vis-a-vis the TBM and the intended alignment provide information on a possible inclination of the annular joint in the direction of the excavation or downwards. In as much as indications for this state are determined, the formation of a saw-tooth effect appears plausible as a further damage mechanism. In this case, constraints result due to the spatial curve prescribed by the TBM and the subsequent tunnel lining.

### 6.6 Press Forces and Hydraulic Control

In conjunction with the required flat ring level, uniform cylinder control and driving with synchronous cylinder pressure/path during the excavation and ring assembly should be mentioned. The segmental elements can be subject to asynchronous load particularly given uneven introduction of force. Deviations within the hydraulic circuits for the single and double cylinders cannot always be precluded. In addition, damaged or wrongly installed constant pressurisers and the cylinder refeeding system can lead to increased loads.

During the excavation, high differential differences of the opposing groups of cylinders result during control movements and especially in the event of open excavations involving top-heaviness of the TBM. These differences lead to substantial load concentrations, which also tend to bring about displacement of the rings. A reduction in the press differences can be brought about by targeted application of an eccentric overcut in the desired steering direction. Furthermore, the control movements should be introduced at an early stage so that the TBM does not “veer off” course.

Excessive stresses affecting the segments can equally well occur by exceeding the statically feasible press forces e.g. as a result of the TBM jamming or as a result of an overcut that is simply too small. Consequently it is advisable to design the segments for the maximum press forces that can possibly be installed.

The press cylinders also exert a considerable influence on the positional stability of the segments, which form a link chain. The equilibrium ratio between the buoyancy forces and annular friction of the annular joints is by and large determined by the press cylinders. Thus in the case of excavations in solid rock, which do not require stabilisation of the face by supporting forces, minimum press force pressures are determined and it is assured they are adhered to in terms of construction management. This can be assured e.g. by reducing the overcut or – as far as possible – by activating statically unnecessary support pressure.

Furthermore, the issue of whether the theoretically applied frictional coupling in the annular joint is actually applicable in the assumed quantity should also not be neglected. It has been shown during projects that unfavourable properties of the adhesive for attaching the fibreboards can have a negative effect on the coefficient of friction. Thus it is advisable to carry out suitability tests prior to application.

### 6.7 Installation Situations during Ring Assembly

A qualitatively high-grade ring assembly also contributes to cutting down on constraining forces affecting the ring when it vacates the shield tail quite apart from reducing cases of damage. The ring itself is assembled manually and thus depends substantially on the experience of the crew involved as far as the quality is concerned. Further factors of influence include speed and accuracy, which actually contradict one another and usually find themselves hinging on the economic considerations of the responsible contractor during ring assembly. Owing to increasing speed of assembly in addition to direct cases of damage such as spalling at the erector cones or defects such as misaligned sealing gaskets in the corner areas, inaccuracies occur, which can lead to further cases of damage in the segmental ring in the fresh mortar once the segmental ring has surrendered the shield tail. These can include:

- Assembly misalignments in the annular joint as well as longitudinal joint
- Excessively “large” or “narrow” installed segmental rings
- Rolling of the rings
- Tilted installation of segments, usually towards the outside, accompanied by V-shaped formation of the longitudinal joints (staggered ring assembly).

Before the segmental ring vacates the shield tail, the ring is affected by pressure from the shield tail gasket’s...
lubrication chambers. The described installation processes can actually lead to premature activation of ring coupling and subsequently in undrained mortar to excessive stress and in turn, to individual segments being damaged.

6.8 Rate of Advance

The rate of advance basically exerts an influence on the opportunities for drainage of the annular gap mortar in a longitudinal direction. Apart from improved drainage, reducing the rate of advance essentially stiffens the mortar matrix so that buoyancy effects can be diminished. The excavation cycle comprising driving and ring assembly would be extended depending on the established filtrate water drainage from the annular gap mortar per structural ring, providing that the general conditions governing deadlines and operations permit it.

6.9 Standstill and Restarting

Given lengthier breaks in driving, e.g. for maintenance purposes to attend to the extraction tools or mechanical engineering as well as standstills in tunnelling the entire mortar grouting system is cleaned and the mortar in the annular space adjoining the shield jacket sets completely according to schedule. When the shield machine restarts, the freshly assembled ring \( n+1 \) with unset annular gap mortar links up with the already backfilled segmental ring \( n \) with set mortar. As the number of segmental rings increase after the restart, a softly bedded link chain is created with a sudden transition of the bedding to the segmental rings installed prior to the break in tunnelling. Great differences in stiffness thus prevail in the transition area. These differences in stiffness can lead towards major misalignments and spalling occurring at coupling points in the annular joint between the “stiff” segmental ring \( n \) prior to the restart and the adjoining subsequent ring \( n+1 \). The misalignments can also continue at the next rings \( n+2 \) and \( n+3 \) should stress not be completely reduced on account of restricted deformations in the joint from ring \( n \) to \( n+1 \).

The effect usually occurs in the case of open shield drives in compactly to very compactly bedded subsoil and can be attributed to the fact that the rings \( n+x \) are exposed to the buoyancy effect owing to an insufficient amount of filtrate water being drained off, whereas the ring \( n \) is no longer affected and cannot float upwards. It is also revealed here that the requirements stemming from the installation situations especially given compact to very compact subsoil must be adequately observed when producing the mortar to preclude such effects.

6.10 Kinematic Interaction, TBM Control and Ring Assembly Sequence

Insufficient amounts of air can be produced between the shield jacket with the constriction at the buffer strip in the shield tail and the outer side of the segment given the kinematic interaction between the tunnelling machine and the tunnel tube. As a result, damage can occur to the ring owing to contact with the steel jacket or crumpling – on the one hand, on account of the scheduled curved run and on the other, owing to the interaction of different tolerances for ring assembly. These so-called shield tail air masses are normally checked manually before and after ring assembly even although automatic or semi-automatic systems are available. Thus a tendential change in the ring assembly quality or ovalisation of the ring or the shield tail can be established and corresponding counter-measures adopted.

Providing the geometrical aspects are properly taken into consideration, active control movements can also lead to the lining sustaining damage, which can extend beyond the maximum curved radius (< correction curve radius). Especially during correction runs involving narrow radii or counter control movements in quick succession annular joint misalignments can occur as the thrusting cylinders of the ring just assembled press against the direction of steering [7]. In this case the ring coupling is normally activated, which can lead to spalling and cracks given excessive load.

“Hectic” control movements resulting from faulty operation, e.g. caused by a lack of experience on the part of the shield operator or excessive reaction and sagging of the TBM, must be avoided. If the shield jacket possesses a rigid design without a shield tail joint, the TBM is susceptible to rapid and jerky control movements, which can benefit the described contacts between shield and segmental ring.

It is essential to apply the calculated ring assembly sequence, i.e. the juxtaposition of the segmental rings with varying keystone positions to avoid T- and cross-joints as well as to accomplish the planned spatial curve to enable the excavation to be free of constraints and avoid damage. Deviations can lead to leakages and constraints owing to inadequate TBM control.

6.11 Concrete Quality and Segment Production

The concrete quality and segment production [8] are also of significance for producing defect-free segment tunnels. Apart from adhering to the properties foreseen at the planning stage and defined in the pertinent concrete standards such as concrete quality, reinforcement content, concrete covering and production quality, the concrete age of the segments being installed and the manufacturing tolerances that must be adhered to are of relevance. In some cases, on account of logistical and scheduling bottlenecks, the question arises as to what extent segmental elements can be installed prior to the defined 28-d ultimate strength. In this case, compressive strength tests are frequently undertaken. Providing the required strength is established prior to 28 days, the green light is often given to the installation of the fresh segments. The production of the segments should be regularly supervised.

7 Strategy to avoid Damage

Recommendations of action to arrive at a holistic avoidance strategy for cases of damage to segmental linings.
result from project findings and analyses involving the following components, explained as follows:

- Lucid verification for earth and water pressures and courses of action
- Kinematic verification
- Early geotechnical and geological-hydraulically characterised analysis of the general conditions pertaining to the subsoil to determine the mortar mix with sufficient filtrate water drainage
- Segment design with smooth joint and centering cone

7.1 Verification for Earth and Water Pressures and Courses of Action

It is obligatory to take into consideration the predicted loads stemming from earth and water pressures as well as the process states foreseen at the planning stage namely storage, transport and installation as well as the operating states especially the press forces. These have to conform to valid rules and standards.

7.2 Kinematic Verification

During the planning of the execution it has to be proved theoretically within the scope of a kinematic study that the predetermined route parameters for the horizontal and vertical curved run with the planned tunnelling machine with segmental lining can essentially be fulfilled. In this connection, the feasibility of the unconstrained installation of segmental rings in the shield tail has to be presented. In addition, it has to be proved that the support can follow the curved run prescribed by the TBM and pass the shield tail without being hampered.

Towards this end, the curved run taking into account concinity of the segments, the forward stroke of the press cylinders, assembly tolerances during ring construction, deformation of the shield jacket and the manufacturing tolerances during segment production have to be included as investigation criteria. Furthermore, the possible interactions must be analysed.

Basically in conjunction with curved radii, a distinction must be drawn between short-distance (correction run) and long-distance (route-bound) curved runs. A short-distance curve as in the case of a correction run, can already be accomplished by installing a single conic ring after exploiting the entire ring concinity. In this connection it is implied that a ring position results from the segment design, which coincides with the position of the maximum ring concinity. In the case of a long-distance curve a suitable ring sequence combination is essential on account of the cross and T-joint problem complex. In this connection, the entire ring concinity cannot be exploited. Thus the maximum overall concinity initially results from the setup of the various ring positions. In addition, corrections in the given rectangular direction to the curved run must be possible in order to master long curved runs. The need for this is derived from the error proneness of the combination man/machine. The overall concinity must be expanded and designed to that effect.

7.3 Mix for annular Gap Mortar derived from Analysing Subsoil and Filtrate Water Drainage

As explained considerable differences exist for the general and project-specific requirements posed on the annular gap mortar as a result of the general conditions of the subsoil. This must be analysed by referring to mechanical engineering, as for example a hydro-shield has to be evaluated differently from a TBM-S or open/half-open EPB shield drive. Thus it is imperative that a distinction is drawn regarding the mechanical engineering to be applied depending on the prevailing subsoil conditions.

The tendering documents as well as the corresponding expert reports contain sufficient details on the geotechnical and geological-hydraulically parameters, which among other things exert an influence on the drainage capacity of the applied annular gap mortar. The nature of the shield excavation results from the tendering documents or the choice of the bidder. It is advisable to undertake an early, qualified geotechnical and geological-hydraulically analysis of the subsoil conditions, involving the inclusion of the operational requirements, which each bidder determines independently, to provide a mortar mix. In this case the objective should be to secure the early (n - 2) ring bedding in order to avoid damage. In keeping with the analysis, series of trials under lab conditions with suitable annular gap mortar mixes have to be carried out. The drainage possibilities for releasing the filtrate water are to be correspondingly depicted in keeping with the general conditions of the subsoil. In this connection, the facts contained in the analysis have to be verified and a suitable annular gap mortar mix selected by the contractor.

As avoiding cases of damage is always in the interest of the client, it is recommended that the call for an analysis with subsequent series of trials becomes a part of the technical service description and that the analysis is tabled together with the offer.
7.4 Annular Joint Formation

Taking the project findings and the analysis of various damage studies into consideration it is ascertained that the scheduled annular gap coupling by means of the pot-and-cam system or similar systems combined with inadequately suited annular gap mortar does not favour the production of a tunnel free of defects. This results in particular from the scenario of a pot flank shearing off without warning bringing with it large-scale concrete spalling and/or leaky joint strips. Clear advantages pertaining to the system with the smooth joint are discernible in this case.

8 Summary and Outlook

As could be shown, the system involving a tunnelling machine with segmental lining represents an established method for driving tunnels in a safe manner. At the same time, it was underlined that the tunnel lining following up on the excavation is affected by a wide range of influences. Planning measures, merely geared to the segments as such, are insufficient. Far rather the interaction between TBM, segment, annular gap and subsoil must be analysed during the construction phase and planned accordingly. Annular gap grouting is accorded particular significance as important damage scenarios occur during the liquid mortar phase. The main differences between drives in soft ground and in more compact solid rocks are elementary as far as designing mortar mixes is concerned. The mortar mix and applied technology with the rates of advance and the grouting pressures have to be geared to each other and configured in a suitable manner. The described recognitions can contribute towards avoiding time and cost-intensive maintenance work.

9 Literature


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Underwater Marieholm Tunnel connects the Banks of the Göta älv River in Sweden
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Introduction
One of the most interesting tunnels in Scandinavia, the Marieholm Tunnel, is currently being realized in the city of Gothenburg. In future, the 306 m long underground connection will ensure that the river Göta älv can be quickly crossed using the available six vehicle lanes. The structure is being constructed in the form of an immersed tunnel: three tunnel sections, each measuring more than 100 m long, are being produced in a dry dock with the help of a Peri Variokit Tunnel Formwork Carriage and will subsequently be submerged in a trench excavated on the riverbed (Fig. 1).

Important and challenging Infrastructure Project
The Göta älv connects Lake Vänern in the interior of the country with the open sea, with the 93 km long river being an important connection for inland transportation operations. Gothenburg is situated at the mouth of the Göta älv in the so-called Kattegat – the relatively shallow 80 m deep sea area between Jutland in Denmark and the Swedish west coast. Due to the increase in traffic volume in the second largest city in Sweden, the new tunnel is currently being constructed and will later connect the city districts of Marieholm and Tingstad.

After completion, the Marieholm Tunnel will feature three carriageways in each direction along with a central service tunnel. Difficult ground conditions and the sensitive ecosystem of the Göta älv river have presented very challenging conditions during building operations. But the construction method has also brought its own challenges as the tunnel is being realized as an immersed tunnel.

Construction Time of 6 Years and Service Life of 120 Years
The scheduled construction time for the tunnel is 6 years with the opening planned for 2020. For producing the three tunnel segments, each with a length of more than 100 m, an 18 m deep dry dock was created on the banks of the Göta älv as a direct extension of the finished tunnel itself. For this, a 120 m long and 40 to 60 m wide working area was enclosed using steel sheet pilings and then pumped dry.

After production of the individual tunnel segments has been completed, the dry dock is flooded with water and the respective segment is pulled out on the surface of the river and then lowered to its designated position on the riverbed. The individual segments are subsequently joined together and the joints tightly sealed.

Due to the increase in traffic volume in the second largest city in Sweden the Marieholm Tunnel is being realized and will ensure that the river Göta älv can be quickly crossed. Peri Sweden and Züblin Scandinavia AB created a cost-effective formwork solution to produce the three tunnel segments in a dry dock.

Tunnelling • Immersed tunnel • Scaffolding • Efficiency • Sweden

After being used to manufacture the three tunnel segments, the dry dock area will be converted to form the transition section from the tunnel opening to the access road using the cut-and-cover construction method.

Due to the length of the tunnel segments reaching more than 100 m, the lowering procedure is a major task for the construction team. The tunnel has been designed for a service life of 120 years.

Individual Exterior – standard Interior
For forming and concreting of the individual segments as well as the tunnel exits, Peri developed a comprehensive formwork solution in close collaboration with Züblin Scandinavia AB, Sweden. In addition to ensuring the simplest possible formworking operations, the focus was on maximizing the proportion of system components.

The cross-section of the three tunnel segments measures around 10 m in height along with a total width of more than 30 m with a middle wall separating each segment into two individual tubes. In a first step, the construction team formed and concreted the bottom slab and external walls of the twin-tube tunnel segments in
one pour with casting segments of about 20 m. For the lateral stopends of the 1.50 m thick bottom slab as well as the 1.30 m thick external walls with heights of almost 6 m, Peri developed a project-specific solution based on the Vario GT 24 Girder Wall Formwork. The internal wall was formed using the Trio Panel Formwork system.

For the subsequent forming of the arches and slabs, two tunnel formwork carriages based on the Variokit Engineering Construction Kit were designed and delivered – each complete with a hydraulic lifting/lowering unit (Figs. 2 and 3). Heavy-duty wheels and corresponding crane rails were used to move the almost 25 m long carriages. For the rounded arches in the upper area of the tunnel, special elements were installed. SCS Climbing Brackets support the wall formwork on the outer side of this top tunnel section (Fig. 4).

Peri Up Stair Towers and reinforcement scaffolding guaranteed safe and easy access to the dry dock and various working areas (Fig. 5). After completion of the three segments, adjacent tunnel sections were realized in deep excavation pits using the cut-and-cover construction method.

Optimized Planning and Supply of Materials

This unusual tunnel project required intensive and close cooperation between all parties involved – from the initial planning idea through to the final return delivery. Peri engineers were involved very early in the planning so that the execution idea could be developed into a cost-effective overall concept in collaboration with Züblin Scandinavia AB. The solution created was an optimized mix consisting of standard systems and project-specific formwork ensured efficient on-site work operations. The advantage of the solution was its high degree of flexibility combined with a low weight which meant that the slab formwork carriage was relatively light weighing around 75 t. In addition, the carriage could easily be divided in a longitudinal direction thanks to its modular design: for the final removal of the formwork, the carriage was decoupled thus creating a total of six formwork carriage units which could then be easily removed from the dry dock in spite of the very limited work space.

Lastly, the option of renting materials was a very important decision-making criterion for the implementation of the Peri solution. On the one hand, this made it possible to fulfill all specific material requirements for the project; on the other hand, the optimum supply of materials to the construction site could be guaranteed. Both aspects provided additional cost optimization.
Steel and Iron Ore

World steel production is still growing at a rate of about 1 %/a and has reached above 1.6 Gt/a in 2016 at a capacity of more than 2 billion tonnes per year. The iron ore and coking coal producers are looking spellbound at the developments in the Chinese steel industry (Fig. 1). Until recently it seemed that its steel production had peaked in 2014 at about 700 Mt/a [3] but the country has not implemented the officially announced reduction in steel output as domestic demand surprisingly recovered and also exports increased. In 2015 and 2016 the 27 state-owned and 400 small mills still produced about 740 Mt (quality adjusted figure), which represents about 50 % of the global steel production. China officially closed 65 Mt of outdated steel capacity in 2016 and announced further cuts of 50 Mt/a in 2017, but simultaneously improved the utilisation of the other plants with the result that the proportion of loss making steel production fell to less than 40 %. WoodMackenzie expects stagnating production rates until 2030 and while Chinese Baosteel Corp anticipates a drop of up to 200 Mt/a over the next 10 years the “Belt and Road Initiative” may further drive steel consumption. The Chinese excess steel production is currently estimated at 360 Mt/a and the yearly exports of more than 100 Mt add to the already over-supplied world market, dragging down world steel prices and causing import tariffs with the hardest consequences in US. While China’s future steel production is not clear, India has plans to triple its steel output until 2030 to 300 Mt/a. It would then surpass Japan as the world’s second biggest producer.

The Chinese domestic iron ore production (Fig. 2) is positioned in the high-cost segment of global supply. Which makes it worse is the comparatively poor quality of the ores. Now China is slowly on its way to close inefficient production, just like in coal. After closing about 200 Mt/a capacity, China now produces about 100 Mt/a of (quality adjusted) iron ore at cash costs of slightly less than USD60/t but imported more than 1 Gt of ore in 2016 of which 650 Mt were delivered from Australia. Due to environmental issues, steel production is reduced in the winter months, with lower iron ore demand. For the same reason, high quality ores are more in demand, which will further increase imports. If China is serious with its domestic clean up and this trend is sustainable this will have a significant influence on the exporters especially in Australia. High-grading of ore production will be inevitable.
Global demand for iron ore currently stands at 2.1 Gt, from which about 1.5 Gt (plus 90 Mt in 2016) applies to the seaborne iron ore trade. The world market, which is expected to further grow by 2.9 % in 2018, is dominated by Australian (800 Mt) and Brazilian (400 Mt) suppliers which could increase their export volumes in 2017 by 5 and 12 % respectively. These producers will further continue to expand their unbeatable share of “low-cost” ores in the market, which need mine site processing by dry-crushing and screening only. It is no wonder that the world’s four most valuable mines are Australian and Brazilian operations with a combined 2016 production value of AUD30 billion. Trying to escape the oligopoly of Vale, Rio Tinto and BHP Billiton, China has undertaken huge investments projects in Australia and some African countries.

Direct reduced iron (DRI) products are also globally traded at an increasing volume (70 Mt in 2016) with the Middle-East producing roughly half of the global output. In the western world Cliffs Natural Resources is an important producer with its new 1.6 Mio t/a HBI plant in Ohio, USA.

While the overall production cost of seaborne supplies in 2016 were USD34/t (lowest supplier USD23/t) the market price in 2016 averaged at USD58/t. Iron ore is now traded at new high prices of around USD70/t but long-term prices are expected in the range of USD40/t to USD50/t when new capacities of 60 to 70 Mt/a will enter the market during the next two years. The Samarco JV production in Brazil will not re-start before 2018. While Australia and Brazil continue to add supply, the high prices also attract marginal producers in Canada, Sierra Leone and Iran to start exporting again. The Sino Iron project of CITIC is a big question. It has to be kept in mind that those prices are benchmark prices for ores containing 62 % Fe. Those premium ores as well as lump and pellets continue to be preferred in the market where China’s steel mills become fussy about the quality of their feedstock to meet stricter pollution laws. Lower quality ores are sold at discounts of up to 40 % but higher grade ores, such as Vale’s 67%-Fe products are enjoying a premium of 15 %. When the Chinese port benchmark price was USD63/t in November 2017 low quality suppliers (e.g. 56.7%-Fe) received about USD38/t only, while premium pellet producers were getting as much as USD88/t. FMG is the lowest cost producer at cash costs of around USD12/t but its 58%-Fe product just realizes 73 % of the benchmark price while BHP and Rio Tinto having costs of USD15/t and respectively USD13/t for their high quality products. The market is currently in balance but it may become a problem for FMG and its peers when global supply is significant rising.

A special problem with iron ore prices is its high volatility which makes it very difficult for the steel industry to forecast its raw materials costs. It also affects the strategy of the producers and government budgets of the producer countries. After the traditional benchmark pricing system was abandoned in 2010, trading of iron ore on various spot and futures markets has dramatically accelerated. Now the industry tries to find solutions to fight speculation without returning to fixed-price mechanism, because it became too difficult to settle.

**Base Metals**

The demand for classic base metals, such as aluminium, copper, lead, nickel, tin and zinc is highly volatile and it is uncertain whether the current development of rising prices is sustainable. The current developments in the renewable energy sector, including energy storage and the increasing use of electrical cars will drive the need for base metals, such as copper, nickel, tin and vanadium besides the technical minerals (see next chapter).

China accounts for 56 % of the world’s primary Aluminium production of roughly 60 Mt. It has recently reopened abandoned smelters to take part in the rising global demand. China is self-sufficient and floods the world market with a surplus production of about 10 Mt. Although there is new production capacity announced for 2017 and 2018 the government restricted output by 30 % because of environmental issues in the winter (heating) season. This could be a reason that aluminium was the best performing metal this year. Economically minable bauxite contains about 30 to 54 % Aluminium oxide with the rest being a mixture of silica, various iron oxides, titanium oxide and other impurities. According to the US Geological Survey (USGS) the world’s leading producers in 2016 were Australia (82 Mt), China (65 Mt), Brazil (34.5 Mt), India (25 Mt) and Guinea (20 Mt) [4]. The Chinese aluminium producers are dependent to 40 % on imports in a 300 Mt/a bauxite market. Chinese imports will grow further from 56 Mt in 2015 to 85 Mt in 2025 because the quality of its domestic resources is significantly deteriorating. China traditionally imported bauxite from Malaysia and Indonesia but uncertainties of supply in Asia forces to look for new trading partners. Bauxite is exported in large quantities mostly from Guinea, Brazil, Dominican Republic, Jamaica and increasingly Australia. Alcoa and Rio Tinto are the largest bauxite miners at about 50 Mt/a each. They export bauxite at a price of about USD50/t.

Following recent industrial actions in Chile and political issues in Indonesia the Copper market is hot. Global copper demand has been rising at an average of 8 % per year in the 2006-16 period but surged 9 % in the third quarter of 2017 raising the price to about USD7,000/t, but this level may not be sustainable. China consumes about 48 % of global copper and its national initiative to improve the manufacturing industry could create an extra demand of more than 230,000 t/a by 2025. Although new production capacities are in the pipeline, there is a supply deficit seen in the near
future (Fig. 3). Those emerging producers will be in Mongolia, Argentina, Peru, Panama, Bolivia, USA and Ecuador. There are also plans to reopen the once largest Copper mine “Bougainville” in PNG.

Lead is running well for the producers because of ongoing supply issues. The battery market accounts for 86 % of global consumption. According to USGS data indicates that China (2.4 Mt/a) is by far the global leader in production, followed by Australia (500,000 t) in a 4.8 Mt/a world market. The global production will approach 5 Mt/a when new projects in Russia (Ozernoe), Mexico (San Rafael) and Australia (Dugald River) and others will commence production. The price for lead has already reached nearly USD2,500/t.

Global mine production of Nickel fell by 30,000 t to 2.25 Mt/a in 2016. The main producers were the Philippines (500,000 t) followed by Russia (256,000 t), Canada (255,000 t) and Australia (206,000 t). Uncertainties in the Philippines (minus 54,000 t) and Indonesia (plus 39,000 t) through mining policies are creating turmoil which lifted the price above USD13,000/t but recently eased back to about USD11,500/t. Nickel used to be a key ingredient in steel production but it is also used in the battery industry at about 100,000 t/a, a number that soon could grow to more than 400,000 t in the late 2020s. This battery supply chain also includes cobalt which is predominantly a by-product of copper-nickel sulphide production. Korean manufacturers are favouring nickel-manganese-cobalt batteries. On voice at 2017 Australian Nickel Conference said it was “almost impossible to forecast what is going on” in terms of potential future demand from this extremely complex and rapidly evolving battery sector.

The Tin market shows a growing supply/demand imbalance, pushing the price to about USD20,000/t so far in 2017. Nearly 50 % of the consumption is attributed to the high-tech electronic industry and possibly also for use in lithium-ion batteries and new generation solar panels. China is the leading producer with 100,000 t/a followed by Indonesia (55,000 t) and Myanmar (33,000 t) as part of 280,000 t world production which fell by 9,000 t in 2016.

Zinc was the winner in 2016 but concerns of supply shortages have eased. The 2016 production was 11.9 Mt, down from 12.8 Mt when Glencore removed 500,000 t from the market to revive the zinc price which then reached about USD3,300/t. New production capacities come on stream this year in China (controls about 38 % of the market), Peru and India as well as in Australia from 2018.

Technology Minerals

A new boom in the world of commodities is developing in the area of technology minerals. The demand for those, also called “critical resources”, like cobalt, graphite, lithium, rare-earth and vanadium is rapidly rising while some base metals, such as copper, lead, nickel and tin being part of the story. The price increases are the result of fast emerging regenerative electricity generation, including the associated battery energy storage technologies and electric vehicle (EV) development. Future demand and prices of those commodities are hard to predict as they change from some astronomical figures to words of warning by other analysts. The development of consumption could be miscalculated as it happened in the rare-earth market, where the almost hysterical situation in previous years has now calmed down. Another demand aspect will be the influence of recycling volumes from outdated electric plants. China is dominating the global battery production and hosts the world’s largest market for EVs. There are also uncertainties about the question what composition of minerals will form the best (highest intensity) energy storage (battery) technology in the future. The technology minerals are now preferred target for smart companies and small-scale mining in many areas of Africa, Australia, Asia and South America. Those operations in the developing world are often associated with mismanagement and corruption. There is also concern that endangered fauna and flora is being destroyed. New and specific processing technologies are competing and changes in the cost structurers could be subject to some risk for the early investor.

The Cobalt market is dominated by the so-called Democratic Republic of the Congo (DRC) where 59 % of global production is concentrated and where Glencore plays a major role. In the DRC cobalt (0.2 to 0.9 % Co) is typically a by-product in copper (2.5 to 5 % Cu) resources. Other producers are China (7 %) and Canada (5 %), followed by Australia and Madagascar. By 2025, demand for battery quality cobalt could grow to 123,000 t up from 48,000 t in 2016. Cobalt is traded at the London Metal Exchange (LME) but there are increasing direct supplies from junior miners in the future. Some investors are hoarding cobalt, anticipating hefty profits as demand increases. The LME Cobalt price which peaked in 2008 at more than USD100,000/t currently stands at around USD62,000/t at cost between USD12,000 to 18,000/t to the large producers. Mine supply was 120,000 t in 2016, but some analysts expect the market to reach 1 Mt/a after 2030.

Natural flake Graphite is in high demand. While premium prices are realized in the refractories, foundries and crucibles sectors for large flake products (50 %
of global demand) the lower value market is in small to fine flake concentrates used as battery anodes. Global natural graphite production was about 1.2 Mt in 2016. It was concentrated in China (70%), India (11%) and Brazil (8%), followed by Turkey, Mexico and Canada but there are new larger projects in African countries such as Tanzania, Mozambique, Namibia and Madagascar.

The expected high demand for electric car batteries results in a predicted demand growth for Lithium of 16%/a between 2016 and 2022 (Fig. 4). This forecast is highly speculative and there is always the fear that early oversupply may drive prices down, demolishing the market value of producers. A growth in supply of 20% from 2016 to 2019 is already projected for producers in Western Australia (WA) which produced 12,000 t in 2016. Strong investment is reported from South America. Chile hosts the largest reserves and is the second largest producer (35.7%) behind Australia (41.5%) and Argentina (11.8%) (Fig. 4). The world production of Lithium in 2016 was 186,000 t Lithium carbonate equivalent (LCE) which is 35,000 t contained lithium. Future demand is expected to reach 400,000 to 500,000 t LCE by 2025. About 51% of the current production is sourced from brines with the balance coming from hard rock. Brine projects have long been considered low-cost but the projects have longer lead times. Hard rock spodumene can be brought into production faster but generally at higher operation costs. But this cost advantage of the brine producers is eroded when product quality is taken into account. Hard rock production reaches 99.5% (battery grade Lithium) while the production from brines only reaches 98.5% (technical grade Lithium). The upgrade of those products would cost about USD1,500/t. Fig. 5 shows the operating costs of the advanced “SiLeach process” (Lithium Australia) to produce battery-grade lithium carbonate from hard-rock (pegmatite) sources. This process is on the bottom end of the cost curve at less than USD3,000/t compared to Chinese production costs of more than USD7,000/t and similar to production costs from brines. The prices for battery-grade lithium carbonate from South America are now around USD14,000/t (Lithium hydroxide USD18,000/t) constantly closing the price gap with hydroxide and ahead of UBS’s estimate for 2018 of USD12,560/t and long-term projections of USD9,000/t. China is currently investing heavily in hydroxide capacity, and 75% of the world’s hydroxide is currently produced using WA spodumene. But even with lowest cost production in the so-called lithium triangle of Argentina, Bolivia and Chile from brines of the huge salt lakes, Australia is further attracting investment into its hard-rock projects containing spodumene-type ores because there are some irritations concerning mining legislations in the South American countries. Other production comes from China and Zimbabwe.

Rare-earth elements (REE) are a group of 17 minerals. Once only produced in China they have now found some competition from Australia, but is still a 90% Chinese business (Fig. 6). In 2016 about 175,000 t were globally produced from which 155,750 t in China and 15,750 t in Australia. Russia was third with 3,000 t. China (67%) is by far also the largest consumer followed by Japan and the USA. The price for Neodymium-Praseodymium (NdPr) products has increased during 2017 significantly from USD30/kg to USD45/kg.

Vanadium production (76,000 t in 2016) is dominated by China (42,000 t), followed by Russia (16,000 t) and South Africa (12,000 t). It was mainly used in the steel industry (91%) but future demand for batteries may reach a share of 30 to 40% by 2020. The price for vanadium pentoxide soared in July 2017 to more than USD24,800/t as environmental restrictions in China were expected to impact supply.

Salt, Potash and Phosphate

The Asia Pacific region consumes about three quarters of global Salt supply. Global demand is going to grow by 20% until 2020. This could lift the price of salt for use in the chemical industry during this period from USD38/t to USD42/t. High-purity salt for medical use reaches much higher prices. The largest salt pro-
Producers in 2016 were China (58 Mt) followed by USA (42 Mt) and India (19 Mt). Australia produces 12 Mt/a from its seawater evaporation fields (Rio Tinto, Mitsui) in the Pilbara of WA, where the German K+S AG is now entering the scene with its Ashburton project to get access to the attractive Asian market.

Two key fertilizer raw materials produced by mining are potash and phosphate rock. The main producers of Potash (potassium chloride or potassium sulphate) in 2016 were Canada (10 Mt), Russia (6.5 Mt), Belarus (6.4 Mt) and China (6.2 Mt) as part of a 60 Mt (K₂O) total global supply. With the merger of the Canadian supplier Potash Corp. and Agrium a new giant in the potash industry, named Nutrien with a stock value of USD36 billion has emerged, becoming the world’s largest potash producer of the Russian Uralkali. A further concentration took place when Canadian Mosaic purchased the Brazilian potash division of Vale. In a market world continuously characterised by excess capacity these mergers should lead to significant cost reductions from synergies which may have a critical impact on the position of the K+S Group (3 Mt/a) with its new “Legacy” project in Canada and on other smaller producers. A market balance is expected between 2025 and 2030 when possibly about 7 Mt/a of capacity may disappear from the market, but there is also substantial latent additional capacity available (Fig. 7). BHP plans potash to become its fifth strategic pillar. After unsuccessfully trying to acquire the Canadian Potash Corp. for USD40 billion in 2010, BHP had started to develop its own Jansen project in Saskatchewan where already more than USD4 billion were spent. But the project needs a further USD5 billion in short-term for the total USD13 billion investment to make the mining complex ready for exports by 2023. To share costs and risks, BHP is obvious now seeking for a 25% stake.

The Phosphate market is in over-supply too. Prices peaked in 2008 at USD430/t and were relative stable from 2014 to 2016 at USD115/t to USD123/t, but have fallen in 2017 to about USD100/t. Morocco hosts three-thirds of the world’s phosphate reserves with about 50 Gt at an average grade of 31.5 % P₂O₅. The main phosphate rock producers are China (138 Mt/a), Morocco (30 Mt/a) and USA (28 Mt/a) within a 260 Mt global production.

Uranium

The spot price for natural uranium (yellow cake) has fallen for years reaching a low of USD19/lb U₃O₈ in December 2016. It now stands at about USD23/lb and even decisions in Kazakhstan, Niger and recently in Canada (Cameco’s McArthur River and Key Lake
Langer Heinrich mine in Namibia. The world’s largest producer Cameco (17% of world production) is closing the loss-making Rabbit Lake mine in Saskatchewan which was once showpiece of a German involvement in uranium mining. They cannot compete against cheap uranium production from Australia and Kazakhstan.

**Costs and Productivity**

In general, the international mining industry managed to survive the most recent price crisis with impressive results. A prime example is the Australian iron ore producer FMG that could reduce its costs to less than one third within four years (Fig. 8). Improvements in mining technology, in particular by automation from blast hole drilling over haul trucks to rail transportation, have significantly reduced the operating costs in surface mining. Other mines are improving productivity through “continuous mining” with in-pit crusher and belt conveying (IPCC) systems by replacing traditional “truck and shovel” operations. New energy supply systems, including solar hybrid systems, are reducing electric power costs at remote mine sites. As a result of the overall continuously decreasing ore grades, there is also a new trend towards underground mining in the metal mining sector. Autonomous operating equipment will also be introduced there and improve the economics of those mines.

**Exotic Mining**

In search of high-grade deposits of special metals some eccentric investors are now seriously targeting deep sea mining, an idea dating back to the 1960s. After 10 years of exploration and test drilling Nautilus Mining is now about to commence mining the extensive copper-cobalt, manganese, nickel and gold resources in the Bismarck Sea, north of PNG. Remote controlled machines are already constructed in England and now available for use. Operations are scheduled to start in 2019 but there are coalitions building up against this kind of mining which definitely will destroy marine life.

More futuristic is the US based company “Planetary Resources” which considers itself as a potential “asteroid miner” (Fig. 9). For its Ceres project, it has just secured around USD21 million in funding, aiming to explore and evaluate mineral resources on asteroids close to earth, containing nickel, iron, cobalt and metals of the platinum group. About USD10 billion was already invested in space service companies which would be available for such jobs soon. It has to be mentioned that those plans do not target to bring commodities to earth but to use them for manufacturing in space, including producing water in space which could be separated into hydrogen and oxygen and used as rocket fuel.

**Conclusions**

The commodity super cycle is continuing. Population growth in Asia and Africa with further impetus to industrialization, infrastructure investments and improvements of quality of life will stimulate sustained
commodity demand. The global energy industry is changing. Power generation from regenerative energies is replacing coal fired power stations especially in western Europe. While the consumption of iron ore and base metals may continue to rise at smaller growth rates the demand for technology minerals will increase significantly in line with the development of regenerative energy systems including battery technologies and electric vehicles. The international minerals industry is increasing productivity and output with the use of new technologies in mining and processing, thus further reducing specific costs. The concentration process in the mining industry will continue as small companies generally cannot bear the rising risks and project costs by themselves. However, this is not to say that comfortable niches will continue to be available for smart companies, in particular in the field of critical minerals. Since the minerals consuming industry in Europe (with some exceptions) is still not willing to invest in raw materials projects, the supply will depend on the intuition of mining investors for long-term market trends. The dynamics in the markets will remain volatile with significant price fluctuations allowing certain producers in the right segment and at the right time to realize extremely high profits.

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Milestone in the Development of a large Battery in underground Salt Caverns – Successful Test for one of the Key Components

Dietmar Bücker, Press Officer, EWE Gasspeicher GmbH, Oldenburg, Germany

EWE Gasspeicher GmbH, Oldenburg in Germany, currently operates 38 caverns in Germany and uses these for storing natural gas. In cooperation with the University of Jena EWE Gasspeicher plans to build a large battery in underground salt caverns. The so-called brine4power project has recently achieved a significant milestone.

**Energy • Power • Storage • Cavern • Research • Development • Think tank**

A key component of the new redox flow application is organic polymers (plastics), explained Prof. Ulrich S. Schubert of the Center for Energy and Environmental Chemistry Jena (CEEC Jena) at the University of Jena (Fig. 2). They are dissolved in saturated salt water, also known as brine. This creates an electrolyte, i.e. a liquid that can attract or release electrons. “The polymers to be developed had to meet very specific chemical requirements. Among other things, they must be readily soluble in saturated salt water, ensure a certain flow property of the brine-polymer mixture and be chemically and electrochemically stable in their dissolved state in order to be able to attract and release electrons in the long term. These special requirements have been met by the polymers being further developed by the Friedrich Schiller University in the basic preliminary tests that have now been carried out using the original brine formula from EWE,” Schubert explained.

“This means that we have taken a big step closer to achieving our goal of building the largest battery in the world,” said Ralf Riekenberg, head of the brine4power project at EWE Gasspeicher GmbH. Despite this success, many issues still need to be clarified before the storage principle demonstrated by the University of Jena in underground caverns can be used. “However, I continue to expect that we will have an operating cavern battery by about the end of 2023,” said the EWE expert.

**Policies are needed**

Technology that works is, however, only one of the prerequisites for the project to succeed. “New policies are also needed here,” said Riekenberg, adding, “Under
energy laws, only natural gas storage facilities have been defined as storage facilities. A definition and thus a legal framework for energy/electricity storage facilities commensurate with their role are currently missing. This lack of categorisation of energy storage facilities within the energy industry means that they are currently classified as energy end-users, and therefore operators are generally required to pay all end-user fees such as the network fee, the German Renewable Energy Act (Erneuerbare-Energien-Gesetz – EEG) levy and electricity tax.”

According to Riekenberg, this means that an electricity storage facility, such as the one planned by EWE Gasspeicher GmbH, stores excess electricity that would not have even been produced without the storage facility because the wind turbines that could produce it would otherwise be switched off due to a network overload. Nevertheless, the storage facility operator is generally required by law to pay the aforementioned end-user fees, in addition to the actual electricity price.

“Although this problem has already been recognised by politicians to some extent, the exceptions for electricity storage facilities introduced in this regard do not yet provide a stable and legally secure framework for storage operators. It is up to the policymakers to change this,” said Ralf Riekenberg. This could result in electricity storage facilities becoming an important option for flexibility and contributing to balancing generation. “brine4power could then become the missing piece of the puzzle in a German energy turnaround.”

“In addition to these legal conditions, it also directly affects the technology itself. At the same time, the further research and development of polymer redox flow battery technology in Germany must be intensively supported. Only then can end-to-end battery production in Germany be established again, including the associated closed value chain and the creation of many jobs,” Schubert added. “Particularly if coal-fired power plants are soon to be decommissioned, there is a need to create ‘green’ alternatives capable of satisfying the base load. The polymer redox flow battery would be perfect for this.”

The Idea

The development of the CEEC Jena at the Friedrich Schiller University gave experts at EWE Gasspeicher GmbH the idea – currently being assessed by the patent office – to use underground salt caverns as containers. These are cavities situated in a salt dome normally used for storing natural gas and some are so large that Cologne Cathedral could fit inside them.

“Since salt water in caverns is also known as brine and we intend to store power according to the redox flow principle, we have named the project brine4power, or b4p for short,” says project manager Ralf Riekenberg.

Dietmar Bücker

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Project brine4power

EWE is developing the world’s largest battery as a green storage facility for green electricity using redox flow technology: b4p storage.

Redox flow batteries are liquid batteries. The Friedrich Schiller University in Jena has developed a new and pioneering metal-free redox flow battery based on a salt solution (brine). EWE will use this new development to operate salt caverns as electricity storage facilities. The maximum storage capacity of this redox flow battery is limited only by the size of the storage container for the electrolyte liquids. Both charged electrolytes can then be stored for several months.

Two caverns, each with a volume of 100,000 m³, will be used for the world’s largest battery. They are flushed with water in salt domes, generating the required brine on site. To realize this ambitious project, an expert-team composed by several partners coming from different scientific and industry areas performs the required research and development to create this largest battery in the world.

Center for Energy and Environmental Chemistry (CEEC Jena)

Sun and wind are important sources of renewable energy, but they suffer from natural fluctuations: In stormy weather or bright sunshine electricity produced exceeds demand, whereas clouds or a lull in the wind inevitably cause a power shortage. For continuity in electricity supply and stable power grids, energy storage devices will become essential. So-called redox-flow batteries are the most promising technology to solve this problem. The power supply from outside (for example, from electricity generated by wind turbines or photovoltaic power plants) effectively snatches away the electrons from the catholyte (oxidation) and feeds them to the anolyte, which binds them to itself (reduction), charging the battery. On discharging, the “stronger electron bonder”, the catholyte, snatches back the electrons from the weaker one, the anolyte, creating a useful flow of electricity. That is the general principle. However, they still have one crucial disadvantage: They require expensive materials and aggressive acids.

A team of researchers at the Friedrich Schiller University Jena (FSU Jena), in the Center for Energy and Environmental Chemistry (CEEC Jena) and the JenaBatteries GmbH (a spin-off of the University Jena), made a decisive step towards a redox-flow battery which is simple to handle, safe and economical at the same time: They developed a system on the basis of organic polymers and a harmless saline solution. In first tests the redox-flow battery from Jena could withstand up to 10,000 charging cycles without losing a crucial amount of capacity.

The scientists present their battery technology in the scientific journal ‘Nature’ (DOI:10.1038/nature15746).

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Time and Subject Schedule 2018*

Four English and four German issues of GeoResources are planned for the year 2018. For the bauma 2019 the first English and German issues in 2019 will be supplemented by a Market Place including company portraits. You can find the publication dates and the special topics in the tables. For more detailed information please download our Media Information or please feel free to contact us.

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