NOVEMBER 2008

# tunnels & tunnelling INTERNATIONAL

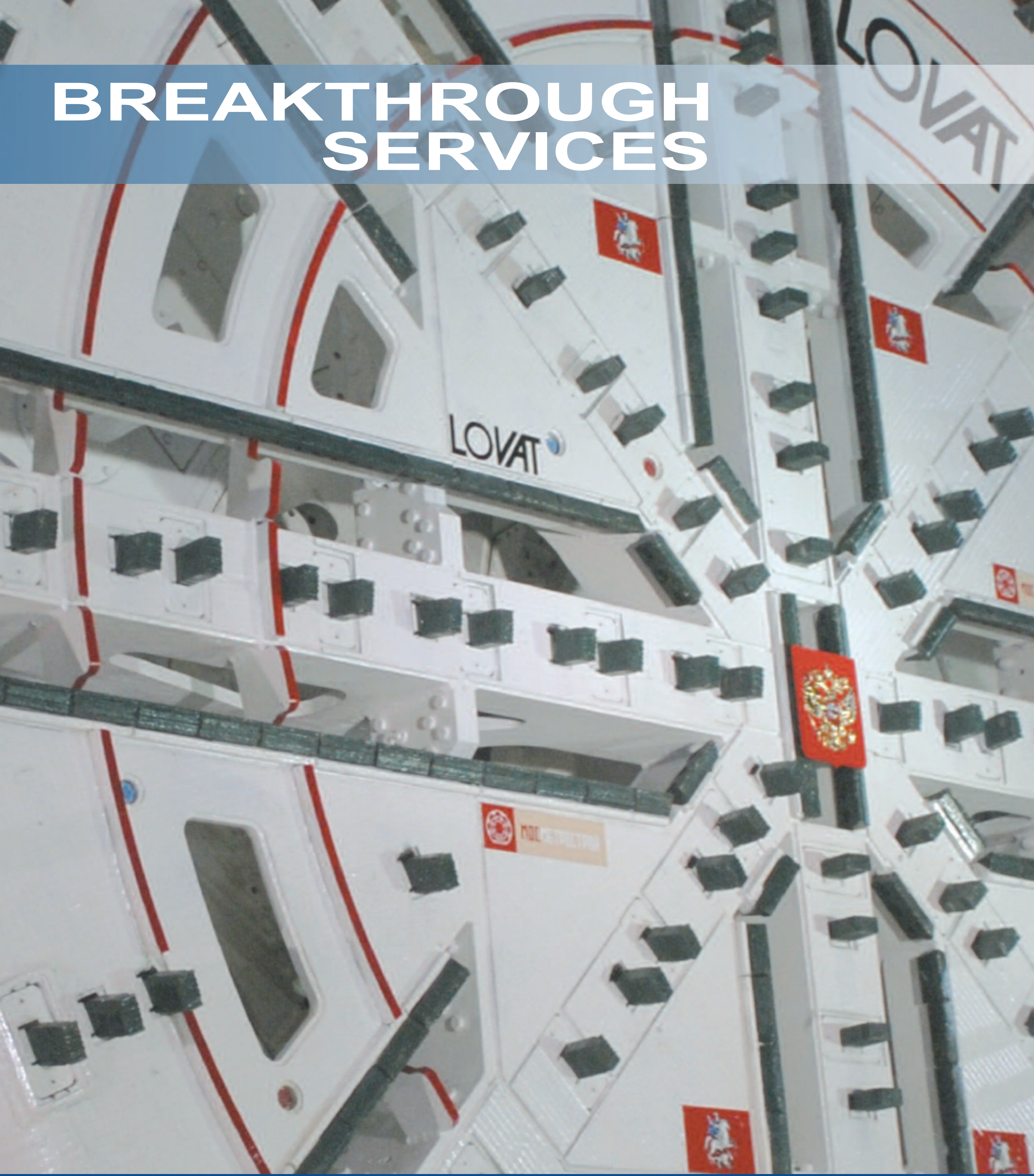
## SÃO PAULO COLLAPSE

Results of the official investigation of the Pinhiros Station collapse in 2007

## CONSUMABLES

Research on TBM disc cutter performance in boulder laden soils

# BREAKTHROUGH SERVICES




**Providing our clients with a complete support system, from conception to completion**

WWW.LOVAT.COM

**PROJECT MANAGEMENT**
**OPERATOR TRAINING**
**TUNNELLING TECHNICAL SERVICES**
**SITE MANAGEMENT SERVICES**

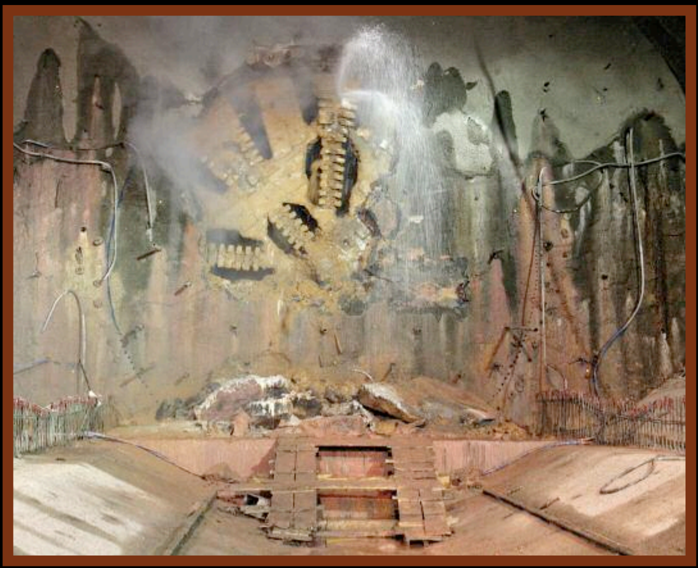
LOVAT
BREAKTHROUGH TECHNOLOGY
For the Tunnelling Industry
Registered **ISO9000:2000** Certificate:008106

# Contents

FRONT COVER:
More than two thirds of the TBM bored section on Line 4 of the São Paulo Metro, in Brazil, has now been completed by the Consorcio Via Amarela JV (p11).

WEB ADDRESS
www.tunnelsonline.info

CONVERSIONS
US$1.00
€0.79
£0.65



11
TBM breakthrough on Line 4 of the São Paulo Metro in Brazil



29
Shotcrete application in the Lenihan Dam outlet tunnel

32.08 E

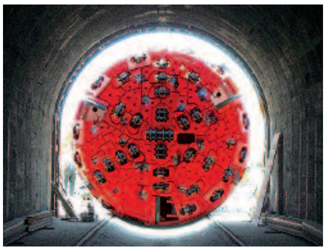
# CALIFORNIA: PIONEERING TUNNELLING PROJECT SUCCESSFULLY COMPLETED.

One of the most complex tunnel construction projects has been successfully completed in the San Bernardino mountains in Southern California. The Herrenknecht S-234 Single Shield TBM achieved successful breakthrough on August 20, 2008, just a few hundred meters from the San Andreas Fault. Its identical sister machine, the S-233, reached its target as early as the beginning of May. These two tunnel tubes were excavated to secure the drinking water supply for the Los Angeles metropolitan area for the future.

The excavation of the tunnels, measuring 6,840 and 6,059 meters in length was extremely difficult due to the strongly fractured, water-saturated rock. For this reason, the two machines (Ø 5.76m each) were equipped with active shield articulation cylinders, efficient drainage systems and a special probe drilling and injection technology to improve the construction ground ahead of the machine. The TBM sealing systems were also specially designed to resist the static water pressures of up to 10 bar. A difficult job worth the effort – showing that even the most challenging tunnelling projects can be realized safely with the necessary know-how and outstanding teamwork.

## LAKE ARROWHEAD | USA

| PROJECT DATA | CONTRACTOR |
|---|---|
| **S-233, S-234**<br>**2x Single Shield TBMs**<br>Diameter: 5,760mm<br>Driving Power: 2,000kW<br>Tunnel lengths:<br>1x 6,840, 1x 6,059m<br>Geology: granite, gneiss | Shea-Kenny JV |



**Herrenknecht AG**
D-77963 Schwanau
Phone +49 7824 302-0
Fax +49 7824 3403
marketing@herrenknecht.com

**Herrenknecht Tunnelling Systems, Inc.**
USA-Auburn (WA) 98001
Phone +1 253 833 7366
Fax +1 253 833 7108

**www.herrenknecht.com**

HERRENKNECHT Tunnelling Systems

# Risky business?

Prompted by a ‘near-miss’ while driving to work this morning, I was reminded that we are all required to manage risk in our every-day lives. If you drive around a blind bend on a quiet narrow country road, there may be something just around the corner you don’t expect. That risk can be managed if you are concious of it and drive cautiously.

However, if you are driving a little too quickly, you add to that risk. If the road also happens to be covered in wet leaves, that level of risk increases again. If you keep adding other factors into the mix - such as an old car without traction control, or an extra distraction – the level of risk increases to the point where the chance of an accident occurring becomes not just possible, but probable. Not unforeseeable, but likely.

In July, *T&TI* editor, Tris Thomas wrote an editorial in which he questioned the use of the phrase ‘unforeseen ground conditions’ as an reason for some tunnelling incidents. He pointed out that, more often than not, such events are far from ‘unforeseeable’ and as an industry we must be held accountable if we want to retain our credibility.

It is encouraging to see this same sentiment echoed by the authors of this month’s article on the official report into the Pinheiros Station collapse, during construction for São Paulo Metro’s Line 4, last January (p16). Despite the complex geological conditions of the Pinheiros Station area, the ground conditions were actually found to be more or less exactly as predicted at the time of bidding. The accident is instead attributed to a number of deficiencies and omissions in engineering processes relating to the design, construction and management of the project; and a lack of quality control and risk management.

The collapse of the station had increased consequences, resulting from an inadequate emergency plan that failed to save the lives of seven people - by not ensuring the swift evacuation of all workers and members of the public from the area surrounding the shaft.

All who work in the tunnelling industry will have heard much on the subject of risk assessment and management in recent years. But the message from Brazil is that there are still lessons that need to be learnt - and put into action.

Unfortunately, while both myself and my car did indeed manage to make it into work in one piece this morning, I arrived to be greeted with the tragic news of the Hangzhou Metro collapse, in eastern China, this weekend. Early reports state that a 75m-long section of tunnel collapsed in the Xiaoshan District of the city, trapping 50 workers, swallowing a part of a highway, and breaking the banks of a local river, in the process.

It is too early to make any real comment on this horrific tragedy, but with five workers lives now confirmed lost, another 16 feared dead, and numerous injured, it has to be the most catastrophic tunnelling accident I have yet seen during my tenure on the magazine.

With this in mind, I urge every single member of the tunnelling community - from apprentices to project managers - to re-consider the contribution they can make towards improved safety and management of risk on their project.

**Amanda Foley**

## COMPANIES IN THIS ISSUE

# Seli new DSU debut

The debut drive of Seli's new compact DSU TBM system (shield, back-up and transport), which is intended to be an alternative to general use of open shields and where concrete segmental lining is not used, is getting underway at the Crevola Toce III hydropower project in Italy.

Seli aims for the system to make TBM use more competitive with heading and bench excavation in shorter bores. At Crevola Toce III, though, the 4.2m diameter shield is to drive 8,589m, or 94%, of the tunnel for the project being developed by energy utility Enel.

The manufacturer is supplying another, 4.5m diameter compact DSU to the Dragados-Besalco JV for the 8,125m long mine bore on the Los Bronces project, in Chile.

Seli said the compact DSU system is designed to give double shield advantages but be shorter, simpler and more easy to operate, and it is bolted for dismantling. The integrated package should also give production and safety advantages and cope with all ground conditions, it added.

The TBM has three operational modes – gripper in either stable or unstable rock, and single shield in fault zones, which should be helped by the short length of the machine. It has a probe drill mounting and also roof drills for rock bolts.

In terms of lining, the system works with typical NATM systems but where segmental lining is warranted it has ring erector for steel but does not handle precast concrete. Back-up is typically 50m-65m and include a double capacity dust scrubber for machines of that size.

At Crevola Toce III, geology along the alignment consists of gneiss, mica schists, marbles, lime schists and dolomitic sediments. The shield (DS-0420-121) has cutter discs of 19" diameter and rotational speed of up to 11rev/min. Tunnel lining will comprise steel ribs, shotcrete, rock bolts and lining plates.

**Right: Seli has developed a new compact DSU system, the first use of which is on the Crevola Toce III project, Italy**

Construction of the headrace tunnel will also include an intermediate stretch approximately 540m long about 6,150m into the drive from the outlet portal. The TBM is to be dismantled and reassembled at the inlet portal to drive in the opposite direction while the weaker zone is opened by drill and blast.

The contractor for tunnelling is a Seli-led joint venture with Monti and Giacomini. The contract was awarded last year with a value of Euro35.5M (US$50.1M in October 2007, now US$45.8M) for the excavation and other works, and is due for completion in late 2010.



# Shaft, station changes for Copenhagen

Design development for next phase of Copenhagen metro has cut the number of shafts and resulted in shallower construction at some stations.

The client, Metroselskabet, said that it has been decided to drop 15 shafts – three-quarters of the initial total – from the 15km long, twin bore Cityringen project. It said that safety for the metro system would be improved by technology, such as installing sprinklers in trains, and that fewer shafts meant less disruption from construction traffic.

Three of the project's 17 stations are to constructed at shallower levels than usual in the Danish capital's metro system, and the possibility of raising a fourth is being investigated. In earlier planning, most stations had platforms at approximately 18m-19m below the surface, and the deepest sites at 24m and 28.5m, respectively.

Metroselskabet declined to give details about the design changes, or whether the project will have two or three civils packages. The call for tenders is to be issued early next month. Bid deadline is mid-2009 for awards in 2010.

Geology along the alignment comprises limestone for much of the route and relatively thicker sand strata for a sizeable section. The project is expected to see both EPB and slurry shields, and earlier this year it was expected that four or five TBMs would be needed.

Cityringen is the fourth phase of the metro's expansion and is to be operational by 2018. The client has been looking to take on relatively more geological risk than in earlier stages, completed over 2002-7. Consultant for the civil works is a joint venture of Cowi, Arup and Systra (*T&TI*, July, p7, and p20-23).

# BTS move for Young Members

The British Tunnelling Society (BTS) last month established a Young Members Committee with an inaugural meeting at the headquarters of the Institution of Civil Engineers, in London.

To raise the profile of the tunnelling sector among young engineers, not least at a time of a pronounced skills gap, the Committee was conceived of earlier this year.

The Young Members Committee will also support the activities of the BTS main committee.

The BTS Young Members Committee has four sub-groups that will meet most frequently, at monthly intervals – the promotion, education, management and support units. Quarterly meetings of the entire Committee are anticipated.

**Above:** **Breakthrough celebrations for Bombela Civils JV, the Bouygues-led team building the Gautrain rail link, in South Africa**

# Gautrain milestone

First hole through has been achieved on the 16km of tunnel being excavated for the Gautrain rail link in South Africa, which is being developed as a public-private partnership (PPP) project.

The breakthrough on the single bore project was on the drill and blast section between Marlboro Portal and the shaft at Mushroom Farm Park, which is near Sandton station. Excavation at the ends of the 4,215m long tube started in February and June last year, respectively. The longest drive on the double-track (74m$^2$) bore was 2,728m from Marlboro.

Tunnelling activity on the project peaked this year with a total of eight faces, including a TBM drive. Just over 13km, or approximately 80%, of the tunnel excavation will be done by drill and blast. About two-thirds of the tunnels are single track with bores of 45m$^2$. A total of 15 jumbos are being used on the project: 2 x 3-boom, 9 x 2-boom, and 4 x single boom rigs.

Geology along the alignment consists manly of weathered granite and there is also silt, clay and boulders. There are several faults. Groundwater is at least 25m down and cover varies 15m-80m.

The contractor is the Bouygues-led joint venture Bombela Civils, and the tunnel was designed by Atkins. The entire, 77km long rail project is being developed by Bombela Concession Co and will have three underground stations (Sandton, Rosebank and Park) and seven surface stations, all of which will be 180m long.

In the other underground sections of the project, to the south – Sandton to Rosebank to Park stations – there has been good progress made. Between Sandton and Rosebank there are three drill and blast faces, and total distance excavated was 1,661m by the end of September – the latest data available, or 38% of the section. The section is single track.

On the 5,554m long section between Rosebank and Park station, in Johannesburg, there are both TBM and drill and blast construction methods employed. Just over half of the section, which is also single track, has been bored by the end of September.

While the drill and blast excavation has still some way to go, the 6.68m diameter Herrenknecht TBM (S-386) had completed 2,233m, or almost 80%, of its run from Rosebank by the end of the September. Coming in the opposite direction, the drill and blast face had progressed 693m.

Progress with the shield was interrupted in July when a large hole opened in the road above the shield, which at that point was not operating in EPB mode. At the location below Oxford Road, investigations blamed a leaking sewer that altered the soil characteristics. Concerns over the stability of other utilities led to road closures as the TBM proceeded with its drive (*T&TI*, August, p4).

# Luhri bores change profile

Plans for the headrace tunnel on the Luhri hydropower project, in India, have shifted from having horseshoe-shaped cross-sections to circular, paving the way for TBM excavation.

Plans have also seen an increase in the size of the proposed powerhouse cavern as the project's installed power capacity is increased.

Satluj Jal Vidyut Nigam (SJVN), a joint venture electricity company owned by the state of Himachal Pradesh, where the Luhri site is located, and the Government of India, has been undertaking early development work on the scheme. Over the last it intensified site investigation work under the early design development requested by the client, the state Government of Himachal Pradesh. The utility was awarded the development work in 2004 and last month signed a Memorandum of Understanding (MoU) to build the project by 2015.

Under the design development, SJVN has changed the headrace from a 10.5m wide horseshoe-shaped tube to a circular, 9m diameter bore. The utility has also significantly extended its length – from 29km to 38.1km.

While the tailrace tunnel is short, it has also been changed from the to a circular, 9m diameter tunnel.

The size of the underground powerhouse has changed, becoming longer but the height is slightly less and the width remains almost the same. The cavern is to be 156.4m x 23.5m x 44m compared to the earlier concept of 131m x 24m x 47m.

# Pfander second road tube bore underway

Excavation of the second tube of the Pfander road tunnel in Austria is underway with the contractor, a joint venture of Beton- und Monierbau (Bemo) and Alpine Bau, using the modified and refurbished TBM from the A41 Mt Sion project in France.

The use of a TBM to drive a motorway tunnel is a first for Austria which has traditionally used the drill and blast method, said the national roads authority (Asfinag). It awarded the Euro123M (US$157M) contract to the JV in the third quarter on 2007.

The JV is to drive a 6,586m long tunnel through molasses along side the existing, east tunnel that skirts the towns of Bregenz and Lochau at Lake Bodensee in the Vorarlberg region, at the far west of Austria where it meets Germany and Switzerland. The entire length of the new, west tube link for the A14 motorway is 7.5km, and it is due to be completed in mid-2012.

The contractor bought the machine and equipment from Herrenknecht – before the manufacturer rebought the shield from the Bouygues Construction-led JV that was working last year on the twin tube, approximately 3km long Mt Sion tunnel section of A41 in the French Alps, near the Swiss border (*T&TI*, September 2007, p20-22).

Herrenknecht refurbished and modified the single shield TBM (previously S-333, now S-474) and the diameter remains 11.835m. The shield – named "Angelika" – is expected to advance on average 20m per day and the drive to last less than a year, then another year for completion of concrete lining work.

A key modification was to changeover from a wet grout system used at Mt Sion to the dry grout and follow-on mix for use at Pfander. The back-up system was also modified with approximately 28m added, taking the total length to approximately 200m, to provide additional facilities to work on the tunnel invert.

The existing Pfander tunnel was opened in 1980 and traffic usage is approximately 26,000 vehicles daily (24 hours). While traffic hold-ups are common on the route, which is in the main north-south route in the Rhine valley, the demand on the route is forecast to increase even more – to approximately 46,000 vehicles per day by 2020.

The total budget for the new tunnel, construction of cross passages and ventilation shafts, and refurbishment of the existing tube is Euro218M (US$278M).


**Set to bore 2nd road tube at Pfander tunnel, Austria**

# Robbins EPBM for Mexico City

The joint venture contractor building Line 12 of Mexico City metro is to use a 10.2m diameter Robbins EPBM for the excavation of the western section of the line.

Geology along the 6.2km long alignment, between Mexicaltzingo and Mixcoac in the south west of the Mexican capital, comprises clay and sand with large boulders, predicted up to 800m wide.

Robbins is building the shield in the US and China, and the TBM will have a 1.2m diameter, two-stage ribbon type screw conveyor to deal with the large boulders.

The EPBM will be the largest ever TBM to be used in Mexico. It will also have active articulation for building tight curves (250m radius) without deforming segments.

The JV contractor is led by Ingenieros Civiles Asociados SA (ICA) with Carso Infraestructura y Construccion (Cicsa) and Alstom Mexicana as partners. It has contracted Robbins to deliver the TBM, back-up and cutting tools, and plans to launch the shield in the third quarter of 2009, and the new metro line is to be commissioned before 2012.

Line 12 was approved for construction last year and will be approximately 24.5km long in total. The line will run from Mixcoac in the west, which will interchange with Line 7, to Tiahuac in the east. There will be about 20 stations.

In the stretch to be tunnelled by the EPBM the intermediate stations are Insurgentes Sur, 20 de Noviembre, Zapata (interchange with Line 3), Parque de los Venados, Eje Central and Ermita (interchange with Line 2).

The eastern section of Line 12, from just beyond Mexicaltzingo, is to be commissioned in advance of the tunnelled section – by 2011, said the client, Metro de la Ciudad de Mexico. It awarded the Pesos 15.3bn (US$1.17bn) fixed price, fixed term contract, excluding VAT, to the JV in July. The bid was submitted in late April.

# Ring bores for Prague

**Above: Skanska pushes drive for outer ring road bore in Prague**

Contractors Skanska and Subterra are well advanced with excavation work on the 1.7km long twin bores in project 513 for the outer ring road at Prague.

Skanska has excavated more than two-thirds of the larger (three lane), right tube for uphill traffic. Subterra will complete the balance of the tunnelling on the tube. For the other, smaller (two lane) tunnel for downhill traffic the contractors are equally splitting the work. Tunnelling is round-the-clock, six days per week.

Both contractors are using Atlas Copco boomers for the heading excavations – Skanksa is employing Rocket Boomer E2Cs and non-electronic 352s, and Subterra has also Rocket Boomer E2Cs as well as L2C drill rigs. For the bench works there are Atlas Copco hydraulic breaker – MB 700 and MB 2,500.

The changing, difficult geology includes some schist and much sedimentary strata. Tunnel lining comprises lattice girders set at 2m intervals, 12 Swellex rock bolts (3.5m and 4m lengths) per set plus mesh and shotcrete.

The geology has presented a number of challenges. Earlier this year there was a roof collapse and the E2C's basket helped save a life, said the manufacturer. In the weakest sedimentary strata with foliations the contractors use 3m log grouted rebar umbrella of 20 spiles at 400mm centres outside the profile.

The tubes have 3% gradient and will have a mid-point 37m deep ventilation shaft and eight cross passages.

The remaining sections of the tubes, which will be 2km long in total, are designed as cut-and-cover. Skanska undertook the portal works at each end.

Other road tunnelling work underway in the city include the Spelc twin tube excavation, part of the Blanka section of the inner ring road which includes the Zlichov, Mrazovka and Strahov tunnels. Contractor Metrostav is also using Atlas Copco boomers for the 2km long, bored stretch of the 3km tunnel which is to be in use from 2011 (*T&TI*, September, p8).

Tenders are also out for additional drainage and improvement works needed at Strahov tunnel. The submission deadline is 3 November.

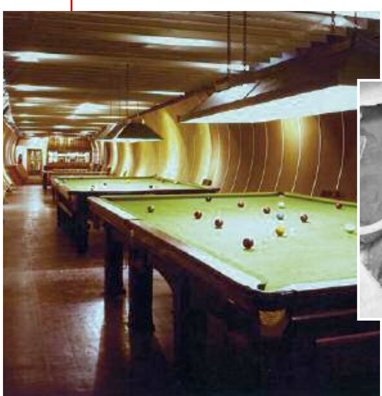
Skanska Czech Republic is also part of a JV that will build a further, adjacent, 9km long part of the new ring road to the south east of the city. The works, which are mainly surface, are to be finished in 2010.

# London tunnel sale

A tunnel complex in London that at various times was a deep-level air raid shelter then home to spies, sensitive government records and a strategic telephone exchange, but is now disused, has been put up for sale by its owner, telecoms giant BT.

The 'Kingsway' tunnel system was built in 1940 and could hold up to 8000 people. Running off two main tubes, each 400m long, the complex is, in total, approximately 1.6km long. The tunnels were mined more than 35m below the heart of London.

BT is taking prospective buyers on visits to the lair. While the facility has had the full range of utility services during its heyday, and past use of some space in less critical times for recreation, such as snooker (see photo), may suggest future possibilities, BT said restrictions will prevent them becoming a hotel, offices or homes.



**Above, Left: The 1940s built tunnels in London has varied uses, and are now for sale**

# Istanbul sees metro progress

Turkish joint venture contractor Gulermak-Dogus has achieved another breakthrough on the new north-south link of Istanbul's metro, the Basak Konutlari- Bagcilar extension, holing through at Guney Sanayi station.

Best progress achieved by the 6.5m Lovat EPBM was 22m in a day, and the average rate was 16m on the drive south from Basak Konutlari station. Geology along the alignment of the twin bore project consists mainly of clays, silts and sands with a groundwater pressure up to 5 Bar.

The TBM, a RME257SE Series 23100 – one of two on the project – has completed 3,640m of tunnel, or more than half of the total run to Mahmutbey station. From Guney Sanayi, the shield will be relaunched on the 1,900m drive to Istoc station and then the 910m drive to Mahmutbey station.

In total, the JV contractor has taken three Lovat TBMs for its work on various parts of Istanbul's expanding metro network. Apart from the 23100 models, there was previously delivered a RME275SE 17900 EPBM in late 2005 to be refurbished for work on the earlier, east-west twin bores on the Otogar-Bagcilar extension of the network.

Just beyond Bagcilar the east-west line ends, at Kirazli 1, which is also marks the south end of the Basak Konutlari- Bagcilar extension. Early this year, Lovat reported that its other RME275SE – "S3" – was driving north on the latest extension, first holing through at the next station, Kirazli, before proceeding to Mahmutbey and onward north (*T&TI*, March, p10).

# TBM breakthroughs for São Paulo metro

More than two-thirds of the TBM bored section on Line 4 of São Paulo Metro has been completed, with shield excavation due to be completed on schedule by next June.

The 9.46m diameter EPBM was launched in early 2007 near Faria Lima station and has driven east more than 4,300m, passing through Fradique Coutinho, Oscar Freire, Paulista and Higienopolis-Mackenzie stations.

Seli's crew driving the Herrenknecht shield for the Odebrecht-led joint venture contractor has been pushing progress since March when it was relaunched after a lengthy stop at Oscar Freire. It reached the station a year ago.

In May, the EPBM holed through at Paulista and then proceeded to Higienopolis-Mackenzie, which it reached in late August. Cover to some sections of road reduced to 4m, said JV contractor Consorcio Via Amarela (CVA). The shield was relaunched recently and is en route to Republica with approximately 1,900m left to bore.

Geology along the alignment typically comprises Tertiary soils of silt and sand along with gneiss of the São Paulo and Resende basins. CVA said the next sections have critical, softer geology and will see the alignment pass very close to old building foundations in the downtown area.

Tunnel lining is segmental (7+1) concrete of 8.43 i.d., each ring being 350mm thick and 1.5m long.

When it reaches Republica the shield will be pulled through, and then it will be again at Luz station, before then being relaunched for the last time to then terminate just beyond at the ventilation shaft at Joao Teodoro.

CVA is building the metro extension, which is a single bore, twin-track line. In total, Line 4 will have a total length of 12.8km and, eventually, 11 stations. The new line will boost the capacity of the city's metro network by approximately a fifth (*T&TI*, April, p8).

Seli was awarded the tunnelling contract just over three years ago for US$410M (2005 prices).

Separately, further investigation into the collapse at the Pinheiros station tunnel, the Line 4 project attracted loans from the World Bank (US$95M) and the Inter-American Development Bank (IADB) of US$95M and US$129M, respectively. The loan has a term of 25 years following a five-year grace period.

**Above & right: São Paulo metro has seen steady TBM progress**

# Metro North challenges for Dublin

Geological challenges have been outlined for tunnel construction of the Metro North tram link in Dublin, in a report by the independent engineering expert panel as part of the public consultation process.

Challenges outlined include stretches of weak ground as well as the proposal for relatively shallow stations and, therefore, tunnels. The tunnel design though might be changed by the concessionaire of the public-private partnership (PPP) scheme, which is currently out to tender.

Metro North will be 18km long and run north-south in the city. The route will run from Lessenhall in the north of the city on to the airport, and also south to the heart of Dublin, to link with existing lines at St Stephen's Green.

Approximately two-thirds of the link will be in tunnel, mostly bored with some cut and cover. The link will have twin tunnels of 6.75m excavated diameter, a crossover cavern and cross passages at nominal spacing of up to 250m.

Four prequalified joint ventures have been issued with tender documents: Cathro; Celtic Metro Group; Dublin Express Link; and, MetroExpress (*T&TI*, May, p12). The winning bidder will have a 30-year concession to design, finance, build and operate the link.

Early last month, the draft expert report was issued. Among the five authors are David Donaldson of Donaldson Associates to focus on tunnelling, and Dr Michael DeFreitas of First Steps Ltd, the lead expert who also focused on hydrogeology and ground response to excavation.

The running tunnels will mostly be in the city centre but also at the airport. Spaced approx 7m apart, the bored tubes will be excavated by TBM with 15m cover.

Geology along the alignment consists mostly of limestone bedrock overlain by glacial till gravel, sands, silts and clays.

On two sections, however – near the launch site at Hampstead Park and a stretch of approximately 1km long near Mater/Parnell Square, where the groundwater level of 12m below the surface – the strata will be mixed or entirely sands and gravels. Extra site investigation is needed near the TBM launch site, and it is expected that consolidation grouting will be needed at the site.

The construction challenge in the sections of weaker ground is made more complicated by the architectural concept of shallow stations. Further site investigation will help fix the limestone bedrock profile. The report notes that the contractor's design may alter the vertical alignment but that EPBMs or slurry shields will be needed.

The crossover tunnel will be located below the sports ground next to Ferguson Road, about 35m below the surface with approximately 20m of cover. To be built in similar geology to the tubes, the excavation method could be mechanised or drill and blast. The report notes the precedent of blasting in the city's limestone and boulder clay in the mid-1970s for the 5km long, 4m wide, Grand Canal Drainage Tunnel.

# Higher costs payment at Hallandsås

Higher costs due to unforeseen poor ground has led to the allowable costs to increase by US$78M for the Skanska-Vinci JV building the Hallandsås twin tube rail tunnel project, in south west Sweden,

More than half of the excavation on the first tube has been finished in the complicated rock and groundwater conditions of the sensitive wetland region. The single TBM on the project is driving between sections built in the earlier, abandoned drill and blast effort to build the 8.6km long twin tube tunnels.

On the second attempt to build the link, the JV contractor and the client – the Swedish railway administration (Banverket) – have had their own challenges, with excavation proceeding first on the east tunnel. The geology along the aligment comprises gneiss, amphibolite and dolorite with UCS of more than 250MPa but the abrasive rock is heavily fractured, blocky and there have been both falls and heavy inflows.

Skanska said that the boring equipment had been wearing out more than expected and progress was slower than anticipated. In April, when the drive met tunnels built by the earlier construction attempt, the opportunity was taken to replace the 10.53m diameter cutterhead of the Mixshield (S-246). The shield was also refurbished and the disc cutters were increased in size from 17" to 19". Progress rates on the northbound drive have improved after the relaunch.

The increase in contract cost follows negotiations between the JV and Banverket, which said agreement had been reached to share responsibility for the delay. Banverket added that it planned to adjust the contract concerning the completion of the works.

The JV's project director, Ander Rehnstrom, said: "We are not pleased that the project is delayed. However, we now have agreed on a solution with which both are satisified."

He added: "We also consider that we have a functioning technical solution and that the project can be executed in an environmentally safe manner."

The TBM was launched in October 2005 and has completed just over 3.4km, or 40% of the 9.4m i.d. first tube, excluding the earlier works. The TBM has just over 1km left to excavate on the first bore. The JV now expects to complete its work in 2014, said Skanska, which is leading the JV.

Bankverket said last month the project is now expected to be entirely operational in 2015 – three years later than originally planned. The delay and setbacks will have increased the total cost of the rail link by US$104M, it added.

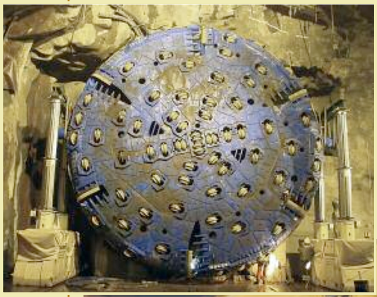
A major problem during the project was the environmental damage to wetland when groundwater levels were drawn down by up to 60m by the excavation.

The TBM is designed to work in 13 Bar. To deal with the significant overbreak and inflows, the tunnelling work involves a minimum annular gap of 220mm outside the 540mm thick segments. Each ring then requires approximately 18m$^3$ of backfill and then secondary grouting. By May this year the depressed groundwater levels had recovered to support the wetlands (*T&TI*, June, p11).

Banverket said progress has been slower on the works than expected but has noted recent, gradual improvement, and that more than 800m of tunnel has been built since mid-year. However, inflows continue to present problems and recent progress rates have been down to just less than 30m in a week as pre-treatment is carried out ahead of the face and the lining is sealed. Preparations for ground freezing are underway at the north end of the works.

Construction on the project started in 2004 – or, rather, was re-started by the client after a seven-year interval. The earlier works, which ended in 1997, saw 1.7km excavated for each tube at the south end of the project and 1.2km for each tube in the north end. Counting the earlier works and the TBM progress, the client said more than half of the main bores has been built.

Banverket added that the total cost of the project, including works from the 1990s, would be approximately US$1.37bn adjusted to current values.



# Awards for tunnel leaders Oakervee, Myers

Leading UK tunnelling engineers and project leaders Douglas Oakervee and Alan Myers have received awards for the contributions and services to the construction industry.

Douglas Oakervee has received the 2008 Gold Medal of the Institution of Civil Engineers (ICE) – the highest honour the professional body can bestow. A former ICE President (2003-4), he is currently heading up the development of the Crossrail project, which will involve significant major tunnelling in central London. In a statement, he commented: 'Civil engineering and construction have been my life and passion for over fifty years. To be recognised in such a way by my peers is one of the proudest moments in my career.'

Alan Myers has been inducted into the Hall of Fame of the British Construction Industry Awards (BCIA) as one of the engineers to have made a substantial contribution to the UK construction industry over the last 21 years. He was one of the two engineers given the Outstanding Contribution Award.

Presently with Halcrow, in Dubai, he held key roles in the Channel Tunnel, Heathrow Express and Channel Tunnel Rail Link (CTRL – now High Speed-1, HS-1).

# HS-2 support grows

Support for the second high-speed rail line (HS-2) in the UK is growing with the Arup plan gaining the backing of the official Opposition, the Conservative Party.

The consultant confirmed funding talks were ongoing despite the credit crunch.

HS-2 would be a step to turn Heathrow Airport into more a multi-mode transport hub for non-London through travel. The Conservatives would support the 'innovative proposal' if it is in government.

Proposals for HS-2 envisage up to 48km of tunnel excavation in twin tubes across north London. The link could be 7.5m i.d., It is hoped the £4.5bn (US$8bn) link could be built by 2019 (*T&TI*, September, p10).

Arup said early political support is vital to build the project. Such support was key to the first high-speed link (HS-1, which opened fully a year ago.

# Lovat ups disc offering

Lovat is planning to introduce 19" diameter discs following its recent introduction of 17" cutters to the market, and is gaining further active support from its new owner, Caterpillar.

Caterpillar is also making its proprietary steel available for the manufacturers cutters to improve wear performance. In addition, it has made use of its muscle by negotiating a price reduction and secured deliveries of Timken bearings for Lovat's TBMs.

Lovat said the 19" discs are under development, and its R&D efforts are able to draw upon the resources of Caterpillar's Technology Centre. The expanded disc offering follows the introduction almost two years ago of 15" and 15.5" cutters.

The 17" discs will be used on machines in Russia and Spain. They will be fitted to the 4.24m diameter hard rock TBM bought by local contractor Bamtonnelstroy for the Krolsky Water Drainage Tunnel project. In Spain, the Terrassa rail project is to be excavated by Lovat EPBMs will also see use of the larger discs.

The parent group's leverage was first shown recently when Caterpillar began to offer financing for Lovat's TBMs. The City of Edmonton, in Canada, paved the way with take-up of the new offer (*T&TI*, October, p7).

## Robbins opens HK office

Robbins has opened an office in Hong Kong to help push business development in the Asia Pacific region.

The company said the new operation – Robbins Asia Pacific Pty Ltd – will focus on expanding the regional EPB market and also support hard rock TBMs and small diameter boring.

Headed by David Salisbury, the new branch is in Kowloon and will cover East Asia and Australia but not China or India, as the company already has three offices elsewhere in Asia. The new branch will also cover the Middle East. Salisbury was formerly with Arup in Hong Kong.

The branch is to focus on sales, technical support and procurement. Going forward, it will be built up to also provide engineering, field service and other support. It is also to help the development of the company's manufacturing facility in Guangzhou, which is due to be fully operational by the middle of next year.

Salisbury said: 'Fabrication and procurement of components from local sources has the potential to make machine assembly quicker and more efficient.' He also said that the location of other offices meant it was difficult to have rapid, face-to-face meetings with East Asian customers.

## Sweco on Helsinki metro expansion

Consultant Sweco has been awarded a project management contract for the Western Metro subway to link the Finnish cities of Helsinki and Espoo.

The project budget is approximately Euro700M (US$906M) and the scheme involves construction of a 13km long subway with eight stations. The project is due for completion by late 2013.

Sweco's project management contract on the extension to Helsinki's metro is worth approximately Euro10M (US$12.9M), the company said.

The expansion project is being built in joint venture by the two cities, which have established a development company Lansimetro Oy (Western Metro Ltd).

The underground link will extend west from the present terminus at Ruoholahti to the new one at Matinkyla with seven intermediate stations - Lauttasaari, Koivusaari, Keilaniemi, Otaniemi, Tapoila, Niittymaa ad Niittykumpu.

Separately, Helsinki will see its rail infrastructure extend in the north of the city with the 18km long Ring Line. Poyry and WSP Finland are working for the Finnish Rail Administration (RHK) to develop the design for twin 8km long rail tunnels in the route from the Martinlaakso area to Vantaa airport and ties-in to the Paarata Stambanan rail link.

Construction of the urban rail loop is expected to begin next year. In the early planning stages of the scheme it was anticipated that the scheme could be completed around 2011 (*T&TI*, August 2007, p7).

## Prado Sud financing closed

Financing for the Prado Sud toll tunnel contract in Marseilles, France, has been completed by the concessionaire, a venture of Vinci and Eiffage.

The 1.5km long Prado Sud tunnel project is an extension to the Prado Carenage tunnel and is scheduled for completion in early 2013. It will be a singe tube with a double-deck arrangements for the two lanes of traffic in each direction.

The concession was awarded in March, has a 46-year term and calls for the financing, design, construction and operation of the new connection.

A total of Euro189M (US$241M) has been raised for the project of which 80% is debt with a 10-year term and the balance is a subsidy from the local authority – Marseilles provence Metropole Urban Community.

Earlier this year, when the concession was awarded, the cost was cited as Euro193M (US$246M) and it was envisaged that 75% would be invested in studies and works stages.

The new tunnel will link the A50 motorway with principal roads in the city – Avenue du Prado II and the Boulevard Michelet. The concessionaire is majority-owned by Vinci Concessions, and the construction work is to be undertaken by Vinci Construction France and Eiffage Travaux Publics in a JV split on the same basis.

The companies are also the majority shareholders in the toll tunnel operator Societe Marseillaise du Tunnel Prado Carenage.

## Widening at Wellington

Plans to widen the rail tunnels north of Wellington, New Zealand, could be accelerated as part of an economic stimulus package if the ruling Labour Party is returned to office in the General Election this month.

Works estimated to cost NZ$150M (US$89M) are required to widen the rail tunnels that are too narrow to take new container sizes, which results in longer transport and more road journeys. Prime Minister Helen Clark said the tunnel widening works could begin next year and would have a construction period of two and a half years.

The tunnels are between Paekakariki and Pukerua Bay to the north of Wellington. The works would be part of the stimulus package to be unveiled in December. The Prime Minister said that the rail bottleneck 'has to be fixed', and added that it would have immediate benefits to journey times, reliability and freight capacity on the North Island Main Trunk Line.

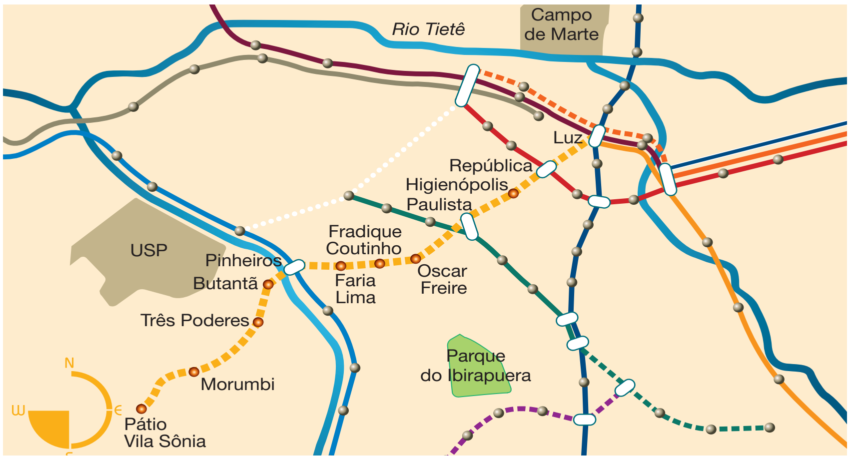
A briefing on the proposed widening project was given to the Prime Minister last month by the local authority, Greater Wellington regional Council, and the state rail infrastructure company, OnTrack. Last year the Government committed NZ$500M (US$296M) to improve the rail infrastructure in the Wellington region.

With a focus on developing the stimulus package for the country, the Prime Minister added that capital works were important and can be a boost to economic activity in the wake of the international financial crisis.

# GROUND CONTROL SOLUTIONS

# Lessons from Brazil: Pinheiros examined

JM Barros, W Iyomasa and AA Azevedo, of the Institute for Technological Research (IPT), in São Paulo, Brazil, Z Eisenstein, of the University of Alberta (UofA), Edmonton, Canada, and AP Assis, of the University of Brasilia (UnB), Brasilia, Brazil, present the findings of the tragic Pinheiros Station collapse last year in São Paulo




Shortly after the tragic incident that occurred during construction of Pinheiros Station on São Paulo Metro's new Line 4, on 12 January 2007, the State Government and the Public Prosecutor of São Paulo commissioned the Institute for Technological Research (IPT) to investigate the causes and provide recommendations based on lessons learnt.

After 17 months of intense work IPT issued its report, which pointed to shortcomings in engineering processes, as well as a number of risk factors, as the leading causes of the incident. The report also discussed inherent contract weaknesses, followed by recommendations of more suitable contract arrangements, which would improve risk management and quality control during future projects.

## Metro Line 4 and Pinheiros Station

Currently under construction, Line 4 (Yellow Line) of the São Paulo Metro is 12.5km long, linking the city centre (Luz Station) to the western neighbourhoods, up to Vila Sonia (Figure 1), with four interchange stations (Luz with Line 1 and CPTM suburban trains; Republica with Line 3; Paulista with Line 2; and Pinheiros with CPTM Line C).

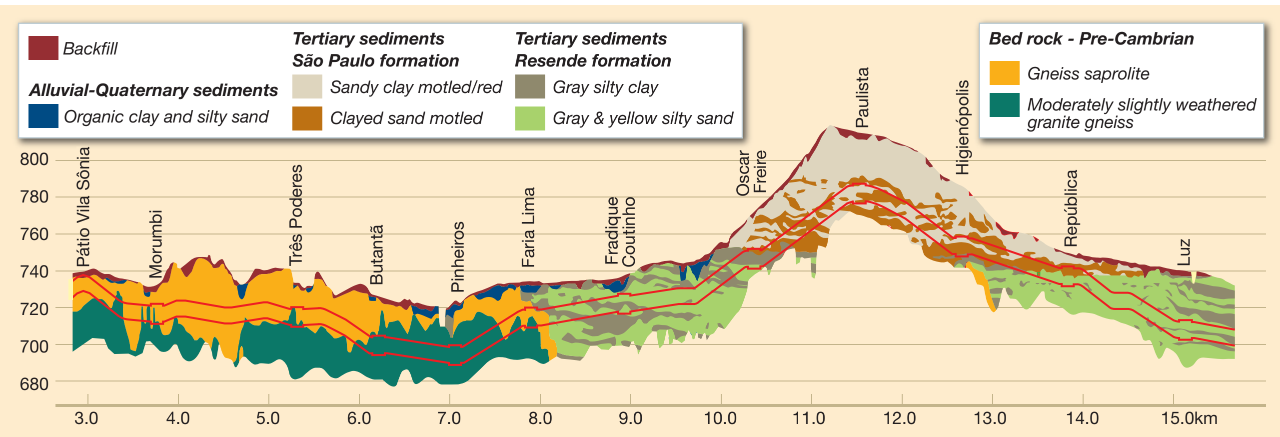
Construction of Line 4 is divided into three lots. Lot 1 is being excavated by EPBM, Lot 2 by conventional tunnelling (NATM) and Lot 3 by surface methods. The stations are being built by cut and cover and by NATM. Figure 2 shows the local geological conditions of Line 4, Lots 1 and 2.

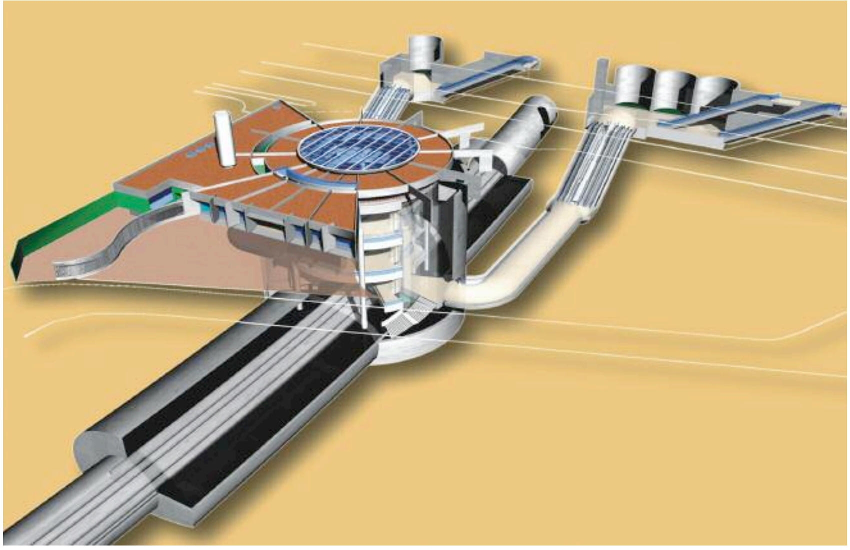
Pinheiros Station was being built by NATM and included a large-diameter shaft (40m diameter x 36m in depth), two platform tunnels (18.6m wide x 14.2m high x 46m long) and two access tunnels to the CPTM Station (Line C). The station has side-platforms, with a central double-track tunnel (9.6m diameter).

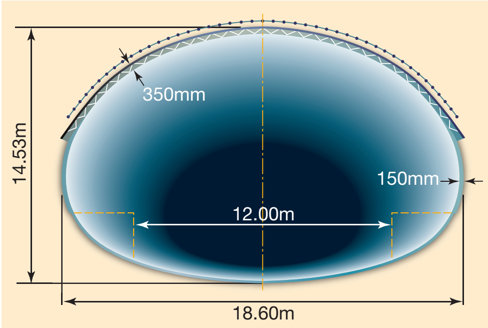
## The accident

The design of the Pinheiros Station specified the following construction sequence: i) Excavation of the shaft (named Capri shaft) down to the first working level (invert of the platform tunnel headings – at an elevation of 696.8m); ii) simultaneous excavation of the headings of the two platform tunnels, one in the Butanta direction (BT) and the other in the Faria Lima direction (FL); iii) excavation of

Above: **Fig 1 - Map of São Paulo's Metro**
Below: **Fig 2 - Geology of Lots 1 and 2**








Above: Artist's impression of Pinheiros Station; Right: Fig 3 - Platform tunnel section

the bench (second working level – at an elevation of 692.8m) and then, the invert of the platform tunnels.

In terms of support, the typical cross-section of the Platform Tunnel FL (figure 3) encompassed a sprayed concrete arch for the heading (350mm thick, enlarged to 580mm), reinforced with lattice girders spaced at 830mm, and lateral walls of sprayed concrete for the bench (150mm thick) reinforced with steel fibres. The invert had only a thin layer (70mm) of sprayed concrete, with no structural function. If necessary, steel bolts could be applied during bench excavation.

The Pinheiros Station accident occurred on the 12 January 2007, at the Platform Tunnel FL, during bench excavation - executed from the running tunnel end of the platform tunnel towards the shaft - when the excavation was almost complete (close to Capri shaft). The first failure signs occurred inside the tunnel at around 14.30hrs. At 14.54hrs the collapse day-lighted in the form of a large crater at Capri Street. In addition to enormous material damage to the construction site and facilities, and also the neighbouring population and public infrastructure, seven people died as direct consequence of the incident.

The Pinheiros Station incident was the most serious in the history of the São Paulo Metro, as well as in urban underground construction in Brazil, and due to the number of fatalities was reported internationally. For all these reasons, it was of paramount importance that an independent investigation be carried out to clarify the causes of the incident and to identify lessons to be learnt, through recommendations for future works.

A few days after the accident, following an agreement between the State Government, the Public Prosecutor of the São Paulo State, and all the parties involved (São Paulo Metro Company and the contracting consortium CVA), including other authorities (Investigative and Criminal Police), IPT was commissioned to carry out the technical investigation and issue a final report.

## IPT investigation

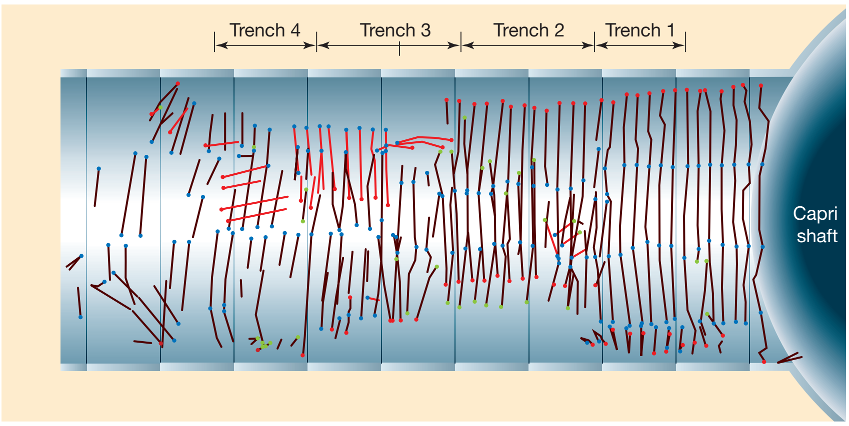
Having been commissioned with carrying out one of the most in-depth technical investigations of a Brazilian civil-works accident, IPT assembled a team of specialists (geological, geotechnical, structural and construction engineering, construction and risk management, etc), named the IPT Commission for the purposes of this paper. An outside board of consultants was also appointed to support and guide the IPT Commission, including six professionals (four Brazilians and two foreign nationals). In addition, an independent auditing firm (Rina International) was charged with following, checking and certifying the work conducted.

The IPT Commission collected and analysed all documentation that could potentially be related to the accident, from the bidding process to final design and construction reports and drawings, including data and follow-up reports of the works. This task generated a large amount of documents (approximately 6000), which were carefully processed. From these analyses numerous questions arose, leading to an extensive process of clarification between IPT and the parties involved.

Another important part of the investigation was the excavation of the collapse debris. A full-time team of professionals (geologists, engineers and topography crew) was set to work recording the excavations 24hrs/day, 7 days/week. During the height of the works around 30 people were involved in this work, their scope including: Geological mapping of the collapse area and residual structures; mapping and photography of the 'archaeological' excavation of collapse debris, determining its geographical position, as well as material testing (Figure 4).

It worth noting that, despite receiving positive and professional collaboration from

Below: Fig 4 - Mapping of the site, showing excavation changes in the bench height and final position of the lattice girders

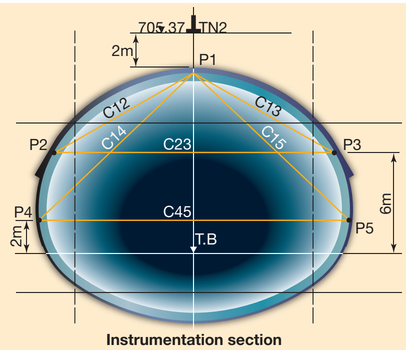


all parties during the collapse excavation - particularly from the construction consortium CVA - there was an inherent conflict of interest between the investigation team (IPT), looking for collapse mechanism indicators and causation clues, and the interests of the construction contractors, who were in charge of safety, scheduling and costs. It seems advisable, in similar situations, that the collapse site be recovered by an independent contractor or at least under the guidance of a supervision company in charge of the recovery process.

In addition to the above tasks, the IPT Commission also ran a series of interviews with tunnellers who witnessed the collapse and other professionals involved in the various stages of design and construction of the Station (client, designer and contractor).

## IPT Report

On 06 of June 2008, IPT concluded, issued and delivered its report to all the interested parties (São Paulo State Government, Public Prosecutor, Investigative and Criminal Police, Metro Company and CVA Consortium). The IPT Report consists of a main report and 46 appendices, totalling approximately 3000 pages. Also a video animation was prepared to facilitate communication with the general public and media.

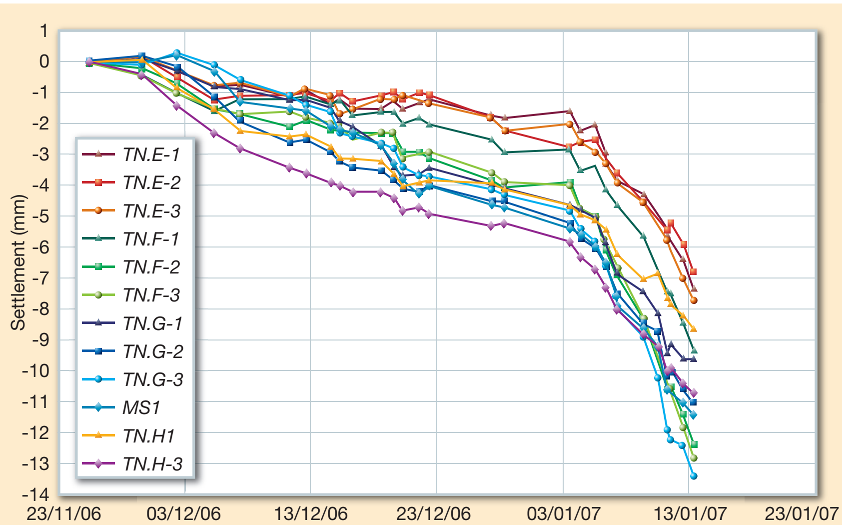


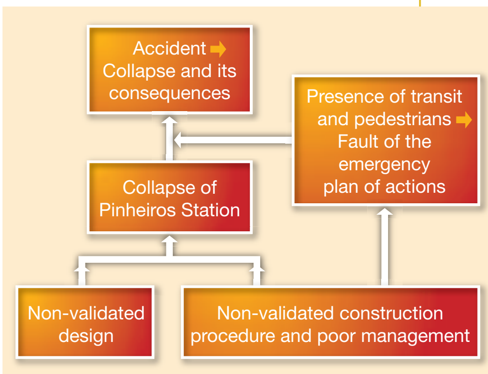

The main report is divided into eleven chapters. The first three of which are related to the nomination of the IPT Commission, the objectives of the technical investigation and the report, the organisation and scope of the works and the report. The fourth chapter is devoted to urban tunnelling, focusing on growing demand, constraints and difficulties in an urban environment, construction methods with an emphasis on NATM principles, and finally on the most recent accidents and lessons. It concludes by stressing the high-level engineering required in all phases of urban tunnelling, taking into account the risks associated with this type of underground works.

Chapter 5 presents contemporary trends in contractual arrangements for underground works, considering the need to incorporate risk assessment and management into projects from an early stage. Design-Bid-Build and Design-Build contract types are reviewed, stressing advantages and disadvantages. Types of project management and quality control are also discussed. The main conclusions of this chapter are that risks associated with underground works require special contractual arrangements and, despite the form of contract, the owner should keep a certain level of control throughout design and construction.

The remaining chapters deal directly with the Pinheiros Station design and construction and the incident. Chapter 6 presents an analysis of the bid documents, especially information related to the

Left & below: **Fig 6 - Typical monitoring cross-section and extensometer data (displacements) during bench excavation of the platform tunnel FL**



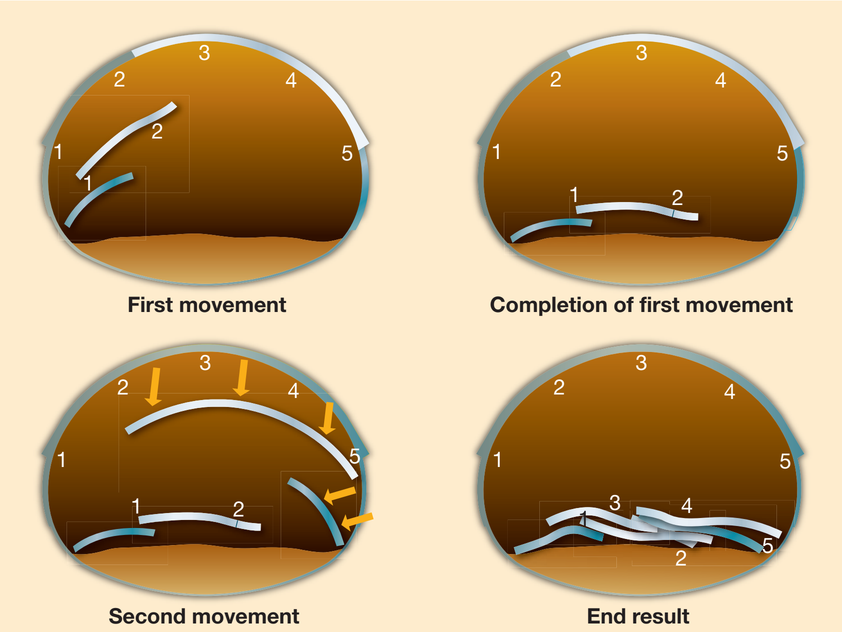




Above: **Fig 5 - Risk factors and causes**

geological and geotechnical data and full disclosure of knowledge at the time of bidding. As the Line 4 bid had been postponed for 10 years, the amount of geological and geotechnical investigation, and the level of engineering design, had been continuously updated and can be deemed reasonable and adequate. The main objective of this chapter was to clearly establish the geological model used at the time of bidding, in order to compare this with the geological conditions encountered during construction and after the incident, as this is often seen as a common cause of tunnel accidents.

It is worth mentioning at this point, that some parties involved claimed that IPT participated in the geological studies prior to bidding and therefore could not be considered independent in the investigation process. Therefore, to clarify: i) It is true that IPT ran complementary geological studies of Line 4 prior to the bidding process, and that its reports were available in the bidding documents; ii) it is not true that IPT established the geological model for the Pinheiros Station area. Neither did IPT consolidate the geotechnical design parameters. The basis of the geological model was established during initial studies undertaken between 1992 and 1994, and was only completed, improved and ratified by complementary studies, among them, the IPT study; consolidation of all geological and geotechnical studies for bid purposes was charged to another engineering company (hired specifically for this objective).

The geological model called for a banded gneiss (biotitic and granitic), with vertical foliation and other discontinuities families, one of them perpendicular to foliation. These two perpendicular discontinuity families favour deep and irregular weathering profiles, in both directions. As the tunnel's longitudinal axis in the Pinheiros Station trench aligns with the gneiss foliation, these vertical discontinuity families could play an




Left & below: **Fig 7 - Sequence of the failure mechanism and construction debris during the archeological excavation**

important role in tunnel behaviour.

Chapter 7 studies and analyses the contract between São Paulo Metro Company and the consortium CVA, as well some subcontracts (such as the contract between CVA and its design consortium). The main objective of this chapter was to identify responsibilities and relations among the parties involved. The main contract is Design-Build, with very little control from the owner, which was quite usual for this type of underground project 10-15 years ago.

Since then, a number of claims and accidents have prompted improvements to this type of contract, leading to different contractual arrangements incorporating risk assessment and management, where the owner plays a more active role during all design and construction. The conclusion of this chapter takes the form of a number of recommendations for future contracts of this type for underground works in Brazil, in compliance with the international trend.

Chapter 8 reviews and analyses the complementary geological studies after bidding and contract award, design documents (reports, drawings, specifications etc), the construction means and methods, including follow-up reports, quality control, construction management, monitoring data and so on. It concludes that several engineering processes, related to the design and construction, presented severe shortcomings or errors prior to the accident.

Chapter 9 chronologically describes the accident itself, in terms of events occurring in the last month, last days, last moments before the accident, and then, the actions just after the accident. Emphasis is placed on monitoring data as well as the deficiency of contingency and emergency actions.

Chapter 10 presents the investigation work, failure mechanism indicators (as indicated by geology and collapse debris mapping), the resulting failure mechanisms including possible triggers and, finally, a causal analysis of the accident.

Chapter 11 summarises conclusions, lessons and recommendations derived from the Pinheiros Station accident. All detailed information, calculations, photographs, mapping, test results and so on are presented in the 46 appendices, as well a glossary of common tunnelling terms.

## IPT report main findings

The analyses of all engineering processes related to the design and construction of Pinheiros Station revealed a series of omissions and errors, which are called contributors or risk factors. A combination of these factors constitutes a cause or causes of the accident. For the sake of simplicity, the Pinheiros accident is divided into two events: i) the structural collapse of the Platform Tunnel FL; ii) the Pinheiros Station accident, comprising the structural collapse and its consequences.

A series of shortcomings in engineering processes led to the collapse of the Platform Tunnel FL. This collapse, in association with deficiencies in the emergency plan, namely no proper evacuation plan for workers, neighbouring inhabitants and lack of transit closure, led to the fatalities (Figure 5).

Regarding the causes of the structural collapse of Platform Tunnel FL, two causal lines were identified, one relating to design and the other to construction procedure. The main contributing risk factors related to design were:

i) Over-simplified geomechanical model, which disregarded geological structures (discontinuities) that could play an important role in tunnel behaviour and stability.

ii) Based on the over-simplified geomechanical model, a structural concept for the tunnel was proposed using an open support system (heading arch and footings), which could be inappropriate for this type of rockmass.

iii) The type of modelling and assumptions made for the calculations were far from the reality. Even so, they suggested the structural concept was not appropriate for the tunnel, indicating failure zones under the arch footings (bench side walls); in addition, one of the most relevant design weaknesses was the lack of definition of threshold values (warning and emergency) for monitoring, required for evaluation of tunnel behaviour.

iv) The above factors resulted in the construction of a fragile design with severe deficiencies (no indications of internal checking or external verification).

v) During construction, the design should be validated by observation and monitoring, as prescribed by NATM principles; follow-up reports (observation) and geomechanical mapping carried out by the design team (ATO) on site, were poor in detail. Despite this, monitoring data indicated large unstable values as well as a highly uncommon pattern for tunnelling during bench excavation (Figure 6). There is no evidence of back-analysis or contingency actions until the day before the collapse, when a meeting was set to discuss the problem.

vi) Another sign of poor participation by the designer during construction was that design changes, with agreement from the designer (ICE), were undertaken without any reports or calculations to support them.

vii) The combination of a fragile design with severe deficiencies, poorly followed-up during construction and not validated by monitoring data (back-analysis, correction, improvement) constitutes the causal line to the structural collapse of the Pinheiros Station, in terms of design.

During construction, several contributors and risk factors were added, some of them so severe they constitute the causal line in terms of construction procedure. They are:

i) During bench excavation, notable changes in design prescriptions were carried out, which, due to their importance, could constitute design violations (inversion of the direction of excavation, from a safer scheme specified in the design to a more unfavourable one; increase of the bench height; changing of the blasting sequence to a scheme that does not preserve the quality of the remaining rockmass at the bench walls, which are the foundations of the heading arch footings).

ii) Quality control performed during construction by the contracting consortium (self-certification) was far from that expected for this type of urban underground works project; there was no clear policy of number, location and procedure of tests or corresponding remedial actions in case of negative results. One example is the quality control of the sprayed concrete; considering the importance of this support element and its properties, in the short and long term, the control of early-age strength and steel fibre content were very deficient.

iii) In such a scenario - fragile design not validated, design violations, poor quality control and discrepant monitoring data - the rate of excavation in January 2007 was much higher than the previous month. This is not a question of whether the excavation rate was lower or higher than that specified in design, but rather a question of why the excavation rate was increased when the monitoring data showed accelerating and discrepant tunnel displacements (Figure 6).

iv) In addition to all the above factors, a meeting was called to analyse the problem and a decision was reached to install rock bolts on the bench sidewalls; a combination of poor communication among parties and deficient construction management led to a lack of rock bolt installation and, at the same time, continuation of the works (one blast during the meeting, two the next morning, and one at noon, just a couple of hours before the collapse).

Construction procedure - violating design, poor quality control, higher excavation rates and disregard of monitoring data - constitutes the causal line, in terms of construction. Both design and construction lines of causation merged, constituting the structural collapse of the Platform Tunnel FL and Pinheiros Station.

## Failure mechanism

During excavation of the collapse debris, all types of mechanism indicators were considered in order to establish the most plausible mechanism and triggers. Mechanism indicators were found by scene photography, mapping, monitoring data and the position of tunnel support debris.

Afterwards, summarising all the above information and calculations, the following mechanism was established: i) The driving force of the collapse was the almost full-overburden loading, due to decompression of the ground by the excavation of a large shaft, presence of two sub-vertical families of discontinuities and shallow overburden, which inhibited the arching effect.

ii) The presence of one sub-vertical family longitudinal to the axis and slightly dipping to the right wall, caused sliding of this wall and compression of the left wall footing.

iii) During excavation of the first bench, when the face reached a certain cross-section, where a set of discontinuities were located just behind the bench right wall, the discrepant tunnel behaviour started.

iv) The excavation continued requiring a process of stress redistribution, attempting to balance the excessive loads, which resulted in the discrepant tunnel behaviour.

v) When the excavation face reached a second cross-section, where another set of discontinuities was located just behind the bench left wall, the unbalanced loads could not be redistributed along the tunnel support (lack of foundation) nor to the excavation face (3D effect), due to the proximity of the shaft where the sections are already highly stressed and there is no front wall. Figure 7 depicts the sequence of failure, where one of the bench side walls failed, triggering the final failure mechanism.

Due to the presence of four families of discontinuities, all known from geological investigation, possible triggers could have happened in several locations, but most likely it was in the bench left wall, guided by a dipping discontinuity. It is important to note that all calculations indicated the rockmass behind the bench side walls was overloaded due to the structural tunnel model and construction procedure and sequence.

## Conclusions

The Pinheiros Station accident was caused by a series of deficiencies, omissions or plain errors in engineering processes relating to design, construction and management. The accident had increased consequences, including the seven fatalities, due to deficiencies in the emergency plan, which saw the failed evacuation of workers, neighbouring inhabitants, and closure of nearby roads.

The geological model available at bid was ratified by the CVA studies following bidding and by geological mapping after the collapse. Despite the complex geological conditions of the Pinheiros Station area no new relevant information was disclosed during construction or after collapse, which could be considered different from that already foreseen by the geological models. For this reason, by no means should unforseen geological conditions be considered as the cause of this accident.

Several recommendations for lessons to be learnt from this incident have been stressed in the IPT report, most related to the deficiencies verified in the engineering processes. However, recommendations related to contractual arrangements for future underground works in an urban environment are also made.

The main recommendations for contractual arrangements are: i) The owner should play an active role in all stages of the project in terms of design and construction; ii) The contract should clearly specify a set of technical specifications, in addition to those for performance, in order to reach a fair balance between quality, scheduling and cost; iii) A set of means and processes should be specified in order to ensure proper quality control, full disclosure of its results, and independent auditing; iv) Risk assessment and management, as well a clear policy of risk sharing, should be incorporated. T&T

## ACKNOWLEDGMENTS

The authors would like to express their gratitude to all members of the IPT Commission, for the dedication, professionalism, responsibility and ethics demonstrated during this investigation. Thanks are also devoted to all involved parties, in particular the São Paulo Metro Company and the consortium CVA for their professional collaboration despite the sensitive relations.

## REFERENCES

1. IPT 2008. Investigação e Análise do Colapso da Estação Pinheiros da Linha 4. Relatório Técnico Final No 99 642-205, Instituto de Pesquisas Tecnológicas do Estado de São Paulo, São Paulo, p384

# Circle of challenges

Stages 4 & 5 of Singapore's Circle Line are due to commence operation from 2010. *T&TI* reports on progress achieved to date and some of the challenges engineers have faced during construction

At US$4.4bn, the Singapore Circle Line extension is one of the most ambitious tunnelling projects currently being undertaken in South East Asia. It involves extending the line 33km, over a five-year period, at a rate of up to 24m/day. When complete, the line will cut travel times and allow commuters to bypass busy interchanges like City Hall and Raffles Place and will link with the North-South, East-West and North-East Lines.

The last stages of the work, Four (CCL4) and Five (CCL5), are currently well underway and on schedule to open from 2010.

## CCL5 Contract

The CCL5 Contract has proved to be one of the most testing parts of the whole Circle Line construction and main contractor Sembawang Engineers & Constructors Pte Ltd (SEC) have had to contend with a range of engineering challenges. Nearly all of these have been down to the complex geological profile of the areas tunnelled. This phase of the line has encountered highly variable ground conditions and has involved excavating highly alternating sedimentary and alluvial deposits.

Simon Hoblyn, who has been with the Singapore Land Transit Authority (LTA) for three years and a total of 20 years in the tunnelling industry, is the project manager working alongside LTA director Rama Venkta on CCL5. Rama Venkta says that the work has been challenging: "The ground conditions are predominately sedimentary rock, but have been extremely variable in weathering. A significant number of interfaces and valleys of alluvial soft clays and fluvial sands were also encountered during the tunnelling works. One section may involve boring through hard rock material while the next soft clay. This has meant we have had a lot of factors to consider while tunnelling in such mixed face conditions along this part of the Circle Line."

The much publicised collapse of Singapore's Nicoll Highway in 2004 as a result of earlier Circle line extension work (for which prosecutions followed, see box on p24), was found to be the result of misinterpretation of local geology, making the current correct reading of the rock formations that much more crucial.

Above and below right: **Two of the CCL5 Contract Herrenknecht EPBMs**

Another key issue has been groundwater levels, since tunnelling activity has the potential to partially drain water-laden clay, causing unwanted ground movement and settlement. To minimise disruption, SEC has employed three 6.6m diameter Herrenknecht EPBMs equipped with rock cutterheads dressed with 41 x 17" cutterdisks.

"The clay acts a bit like a sponge - the moment you drain away the water it has the capacity to shrink. You therefore need to use the right equipment or make adequate

## PROJECT FACTFILE

**Cost**
Total cost of Circle Line Extn - S$6.7bn (US$4.4bn)
Value of CCL5 Contract - S$335M (US$222M)

**Timescale of CCL5**
Originally scheduled to take 16 months. Due to re-sequencing of works, following the Nicoll collapse, tunnelling will now last 22 months

**Workforce**
The project team for CCL5 consists of 20 LTA management, engineering and inspection staff; 25 contractor's management, engineering and supervisory staff; and 230 tunnel workers

**Contractors**
Sembawang Engineers & Constructors is main contractor and Tri Tech Engineering for instrumentation

**Tunnelling techniques**
EPBM; Rock TBM; Soft ground SCL; Rock SCL; Cut & Cover

**Machinery used**
3 x 6.6m Herrenknecht EPBMs, TBM supply via diesel locos and rolling stock, plus gantry crane


Above: **Map showing the various phases of Singapore's Circle Line**

allowances. We used pressure-balanced TBMs and intensive monitoring to ensure that the watertable was kept within allowable limits," says Venkta.

Delay during design, as a result of the Nicoll Highway collapse, and challenging ground conditions has caused station construction along the course of the Circle Line to be either re-scheduled or re-sequenced. As a consequence, some 7.5km of critical path tunnelling, which was originally slated to take 16 months with three TBMs, will take 22 months instead.

Hoblyn says CCL5 has benefited from Singaporean buildings' generally high quality foundations but has encountered difficult ground formations. This has meant careful equipment planning and execution at each stage of the work.

"You can have very hard rock in one section of the alignment and soft clay in the next, so you need to know exactly what the geology is so that you can plan ahead the sort of equipment that you are going to need and when to change it," said Venkta.

### Urban considerations

Aside from the geology, CCL5's contractors have had to allow for the fact that Singapore is a crowded island and they consequently have had to navigate their way underneath a range of structures, from public roads to residential housing and commercial blocks.

CCL5's four underground stations, two cut and cover crossover boxes and an interchange station, linked by 3.5km of twin bore tunnels with 500m of single bore and overrun tunnel, are substantially located underneath public roads and within a built-up corridor. Along one section, the Circle Line follows an elevated 3-lane dual carriage highway, passing close to and between its piled foundations with at times less than 2m clearance from the TBM's extrados to the bored piles of the viaduct pier.

Advanced planning allowed for the selection of a suitable tunnelling corridor and mitigation against the effects of tunnelling, such as negative skin friction on the piles due to tunnel-induced settlement. State-of-the-art monitoring is supplemented with traditional instrumentation to keep track of progress and ensure the safety of overlying structures and the public in general. Without such work, settlement of the soft clays could have compromised the piles. "We would only be talking minor movement of perhaps 50mm surface settlement, but even this small amount of movement could generate additional loading on the piles resulting in settlement to the pier."

To date, strict procedures and vigilance have led to the successful excavation of 80% of the total alignment, with minimal surface disruption. T&T

## NICOLL COLLAPSE

On 20 April 2004 a tunnel support structure within one of the open cut stations on the Circle Line Extension gave way. The resulting 30m (100ft) collapse spread across six lanes of Singapore's Nicoll Highway, killing four people and injuring three (*T&TI*, February 2007, p31).

An investigation showed that a key reason was the misinterpretation of local geology and overestimated soil shear strengths. The structure was consequently under designed to resist lateral earth pressure. Errors in detailing the structural connections for the bracing system were made and the collapse occurred when the lower level of bracing became overloaded and there was inadequate capacity to redistribute the loads among the remaining supports.

Although large wall deflections occurred during the excavation, the measured strut loads were smaller than expected. As a result, project engineers were apparently unaware of the potential for a catastrophic failure.

A committee of inquiry found main contractor Nishimatsu Construction Company as well as Singapore's Land Transport Authority responsible for the collapse. Subcontractors were reprimanded and issued warnings in connection with the accident. The incident was a major contributory factor to delays in the line extension and the affected station has been shifted about 100m (330ft) away from the accident site.

Four men faced charges in the wake of the Nicoll Highway Collapse. The Nicoll Highway incident came as a result of cut-and cover work, so the implications for TBM tunnelling work concerned the evaluation of risk assessment.

# Supporting Tauern's 2nd tube

The Tauern Highway ranks among the most important transport links in Austria, being an essential north-south passenger and goods vehicle route across the Alps. Of the 12 tunnels on this highway, which equate to a total length of approximately 24km, the 6.5km long Tauern Tunnel is by far the longest and best known.

The tunnel is one of two remaining single-tube tunnels on the highway and is currently a traffic major bottleneck. Therefore, to provide a continuous two-lane highway connection, excavation works for a second tube started in September 2006. According to the work schedule, tunnel excavation is due to be completed by the end of this year, with traffic operation planned for mid-2012.

Construction of the first Tauern tunnel, in the early 1970s, marked a milestone – and not just in Austrian tunnelling history. For the first time since World War II, a traffic tunnel was excavated in squeezing rock mass. Far beyond the borders of Austria, the experiences gained during the construction of this project contributed to the worldwide development of modern tunnelling techniques.

These difficult ground conditions resulted from the need to excavate through a thrust plane as well as an overburden height of over 1000m. Another technical challenge during the course of the construction works was an initial heading through a debris (landslide) slope, for an approximate length of 330m. This material primarily consists of cohesionless fine to middle grained gravel that is partially interspersed with larger boulders.

## Supporting the 2nd tube

All rock reinforcement and support systems, such as lattice girders, rebar rockbolts with special rib geometry for squeezing rock mass conditions, forepoling boards, and injection spiles, were supplied by Alwag. Especially during excavation through the debris (landslide) zone the use of 2.5-3m long forepoling boards, which were rammed into the ground above the lattice girders, proved their capability as an effective pre-support system for the extremely difficult ground conditions.

Due to the squeezing rock conditions, long-term deformations of up to 1.2m were measured during construction of the first tunnel tube. These large deformations led to overstressing and complete rupture of the shotcrete lining. As a consequence, longitudinal deformation gaps were introduced to divide the tunnel lining into sections to protect the shotcrete lining against overstressing. By doing so, the lining could accommodate larger radial displacements, up to the point where the deformation gaps closed, without damage.

However, a disadvantage of this method was that the load-bearing capacity of the shotcrete lining was affected, due to its division into segments. As a consequence, additional uncontrolled displacements took place that had a negative effect on the excavation.

To mitigate these problems during construction of the second tube, deformable ductile elements are installed in the deformation gaps. These AT–LSC Elements (Lining Stress Controllers) were developed at Graz University of Technology in cooperation with Alwag, who is now responsible for the worldwide distribution of this patented support system for squeezing rock mass conditions.

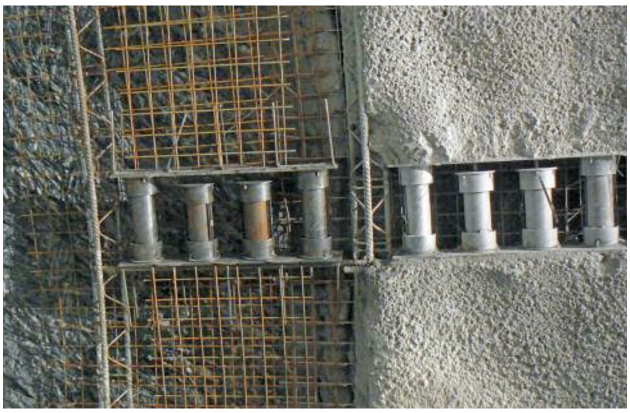
By using AT–LSC Elements, large deformations that occur during excavation of the second tube of the Tauern Tunnel can be controlled and the load bearing capacity of the support and ductility of the tunnel lining is ensured.

In combination with AT–LSC Elements, rebar rock bolts with an ultimate load of 350kN and special rib geometry for squeezing rock masses (related rib area between 0.02 and 0.04) ensured successful excavation through difficult ground.

**Alwag**
**Web:** www.alwag.at

## PROJECT FACTFILE

| | |
|---|---|
| **Owner:** | ASFINAG |
| **Contractor:** | PORR Tunnelbau |
| **Design:** | IGT – Geotechnik und Tunnelbau |
| **Supervision:** | ARGE Spirk & Partner<br>Müller + Hereth |
| **Alwag's scope:** | Supply of PANTEX lattice girders; IBO-Self-drilling anchors and spiles; rebar rock bolts with standard and special Alwag rib geometry for increased bond capacity; expandable friction bolts; rebar spiles; tube spiles; forepoling boards; AT–LSC-Elements (Lining Stress Controllers); pull testing equipment for anchors and rockbolts; technical assistance |



# Glass fibre reinforced shotcrete

AR Spritzfil-cem chopped strands were designed as a competitive alternative to traditional steel and polypropylene fibre reinforcement. To develop the AR reinforcement, Owens Corning worked together with industry customer Salerno-based ReCC – Rebar and Concrete Composites, the technical departments of several universities and a laboratory specialising in concrete technology.

The new AR Spritzfil-cem chopped strands are based on the same Owens Corning alkali resistant glass fibre technology that has delivered the successful Cem-FIL and Anti-Crak alkali resistant glass fibres. These products are claimed to offer an advance on traditional steel and polypropylene fibre reinforcement by bringing longer life, greater application flexibility, durability and higher performance, and in many cases lighter weight.

"Our particular focus for development was fibre reinforcement for shotcrete," says Andrea Brucato, sales leader, OCVTM Reinforcements Italy. "With a property profile that includes a higher tensile strength than that of steel and a much greater modulus than polypropylene, AR Spritzfil-cem chopped strands can be highly competitive in this application."

Benefits offered by AR Spritzfil-cem reinforcement include:

- Good mechanical performance characteristics such as crack and impact resistance
- Good workability with productivity and safety advantages
- Non-corroding material
- Up to 20% less rebound than alternatives, resulting in lower fibre and concrete usage
- Less spray pump maintenance required and reduced hose wear

Industry standards for specifying sprayed concrete are well developed and therefore to ensure compliance, concrete reinforced with AR Spritzfil-cem chopped strands has been tested by Italy's Geoconsult Srl and certificated as meeting the requirements set by engineering and construction companies. An added confirmation comes from the excellent results being reported by the first

customers to use the new product.

With the acquired Saint-Gobain reinforcement and composite fabrics businesses, Owens Corning Composite Solutions now has more than 40 production facilities in 16 countries and more than 9,000 employees.

**Owens Corning**
**Web:** www.owenscorning.com

# Tunnelling for the new Lenihan outlet

NAT 2008 Student Prize finalist, Shawna Von Stockhausen, of Hatch Mott MacDonald, describes the construction of a tunnel through Franciscan Mélange in California, for the Lenihan Dam Outlet Project

The Lenihan Dam Outlet Modification Project includes the construction of a new tunnel outlet at the Lexington Reservoir, in Los Gatos, California. Owned and operated by the Santa Clara Valley Water District, the Lenihan Dam's original steel lined outlet has buckled over the years, requiring numerous repairs. This led the California Department of Water Resources, Division of Safety of Dams (DSOD) to restrict peak discharge to 2m$^3$/sec, compared to the previous 11.5m$^3$/sec. The new conduit will consist of a 1.37m (54") concrete lined steel pipe, sited inside a 610m long maintenance tunnel running through the eastern abutment of Lenihan Dam.

The project also includes a multi-port intake structure, a shaft located in Lexington Reservoir and an outlet structure adjacent to Los Gatos Creek. The existing outlet conduit beneath the dam will be abandoned in place after completion of the new outlet.


## Contract award

The Lenihan Dam Outlet Modification Project was awarded to FCI Constructors, with Drill Tech Drilling & Shoring as its tunnelling subcontractor, with a lowest bid of US$39,173,160. Notice to Proceed was issued on 10 September 2007 with a completion date of 03 September 2009.

Below: **Portal construction underway**

The District normally oversees its own construction projects, but in this case (due to the tunnelling works) it appointed Hatch Mott MacDonald (HMM) the Construction Manager. The project was designed by Jacobs Associates (JA) in association with Montgomery Watson Harza (MWH).

## Project Schedule

The two-year construction contract includes three project milestones:

- Extend existing outlet at the downstream end of the dam to allow flows through the existing pipeline by 15 November 2007
- Complete intake shaft excavation and all other work to allow the reservoir water elevation to be increased above El.576 no later than 15 November 2008
- Complete all work on the project within 725 days from Notice to Proceed

In order for FCI to complete the intake shaft excavation, the water elevation in the reservoir must be lowered. FCI is allowed, as per the contract, a period of five months from 15 June 2008 until 15 November 2008 to work within the banks of the reservoir. During this time the reservoir level will be kept between El.576 and El.557. If the District had been unable to guarantee drawdown of the reservoir the contract allowed for postponing construction one year until summer 2009; fortunately, this has not been necessary.

## Communication structure

The project offices are located in a lot adjacent to the dam. FCI and Drill Tech each have an office and the project owner, construction management team and design support representative share an office. This arrangement provides the opportunity for daily face-to-face interaction between those involved in the project. General progress meetings are held each week with other specific issue oriented meetings (i.e. technical or environmental) held as needed. Major correspondence including submittals and requests for information are exchanged electronically and in hard copy when requested, streamlining communications.

The partnering process is in use on this project. Quarterly general partnering

Top: **Installation of spiling;** Middle: **Standing a steel set with the roadheader;** Above: **Application of shotcrete to blocked steel set**

meetings are held off-site and facilitated by a third party partnering consultant as a means of openly discussing the successes and challenges of the project and ways that challenges can be addressed. Online partnering surveys and management level partnering meetings are also held monthly.

## Local geology

The project is located in the foothills of the Santa Cruz Mountains, approximately 10 miles from the epicenter of the 1989 Loma Prieta Earthquake. The geology along the tunnel alignment is mostly Franciscan Mélange, characterised by relatively competent blocks of varied rock types embedded in sheared matrices of weaker soil-like or rock material. The matrix material is often shale, sandstone, serpentinite, or clay. Blocks can range in size from sands to boulders or larger. Typical rock types for the blocks in the mélange are sandstone, greenstone, serpentinite, chert, shale, and argillite[1].

The ground encountered in the tunnel so far has been classified using the Terzaghi system as moderately jointed to blocky and seamy when the heading is within large rock blocks, to crushed when the weaker sheared matrices are dominant.

The potential challenges for excavation of a tunnel in Franciscan Mélange include the heterogeneity of the face, the potential for squeezing ground, and the durability of the weaker matrix as an invert material. The presence of a large hard block in the face that is otherwise comprised of weaker material can cause crews to switch excavation methods in the middle of a round. When Mélange is located at an area with high cover there is a higher potential for tunnel convergence, or squeezing ground.

The weaker matrix material is subject to degradation and is easily ground up by repeated vehicle traffic. Due to this, the invert had to be protected during excavation and could not be left as exposed ground. An initial invert, or mudslab, of cast-in-place reinforced concrete was required in the contract. The mudslab was placed approximately once every two weeks prior to weekend closure. In areas where squeezing ground was experienced the mudslab was placed more frequently. This served not only to protect against degradation but also as a means to control convergence by acting as an invert strut.

## Mobilisation

The first activity in the field was extending the existing outlet pipe approximately 30m downstream in the Los Gatos Creek. This was necessary in order to prepare the site for tunnel excavation. Portal development was the next activity. The portal is located to the east of Los Gatos Creek in St. Joseph's Hill and is the future location of the outlet structure and the point from which tunnelling originated.

Excavation of the colluvium and weathered greenstone in the portal cut was performed with a Caterpillar 330CL Excavator and took about two weeks to complete. The portal consists of an approximately 6m high shotcrete and soil nail wall set back into St Joseph's Hill and was constructed in four lifts.

## Initial tunnel ground support

The primary method of ground support in the tunnel is 150mm (6") steel sets (W6x25) at a typical spacing of 1.2m with varying amounts of fibre-reinforced shotcrete. Timber is used for cribbing and blocking, with a typical blocking point spacing of 900mm. When necessary, in areas of low stand-up time to limit overbreak, No. 9 rebar spiles are used for pre-excavation ground support in the crown of the tunnel. When used, spiles are spaced approximately 300-400mm apart and generally 3m-3.5m deep, typically from quarter arch to quarter arch. Spiles are installed with a lookout angle of approximately 12° to 15°.

Three ground support types are used in the tunnel. The difference in support types is the thickness of shotcrete applied between the steel sets. A description of the ground support types used follows below:

- Type 1: A minimum of 75mm of fibre reinforced shotcrete, where required
- Type 2: A minimum of 150mm of fibre reinforced shotcrete
- Type 3: A minimum of 230mm of fibre reinforced shotcrete

After excavation and mucking has occurred the steel set is installed. First, the leg pieces of the horseshoe shaped steel set are installed at the proper spacing with the use of collar pipes and the proper location with the use of a laser guide. After the legs are in place the arch pieces of the steel set are installed. The arch is lifted and held in place with the roadheader until it has been bolted to the leg pieces and additional collar pipes in the arch are installed. Once the steel set is in the proper location timber cribbing and blocking takes place. Several rounds are often completed and steel sets installed before shotcrete is applied. The shotcrete thickness is completed generally within two tunnel diameters (8.5m) of the face.

Determination of the required initial support is made by the design representative and documented on a Required Excavation Support Sheet (RESS). When specified on the RESS, fibre reinforced shotcrete is applied for Type 1 ground support. If fibre reinforced shotcrete is not specified for Type 1 ground support Drill Tech has the option of applying non-fibre reinforced shotcrete. In those situations Drill Tech has typically elected to do so. In the first 386m of the tunnel Type 1 ground support has been used for approximately 80% of the excavation, Type 2 ground support has been used for approximately 17% of the excavation, and Type 3 ground support has been used for approximately 3% of the excavation.

## Tunnelling

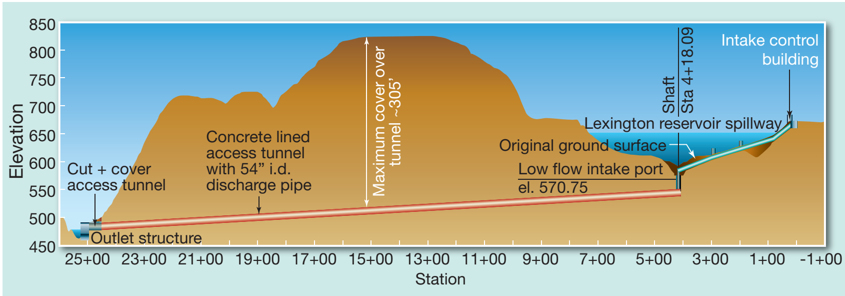
The tunnel is horseshoe shaped with an excavated width of 4.3m and an excavated height of 3.8m (figure 1). The tunnel is approximately 610m long with two curves and slopes at approximately 3% upstream (Figure 2). Excavation was completed using two methods: Mechanical excavation with a roadheader and drill and blast excavation. The primary equipment used for excavation included a roadheader, a scooptram, a drill jumbo, and jackleg rock drills. The roadheader selected was an Alpine AM50 reconditioned by Antraquip Corporation. The drill jumbo was used for probe hole drilling and drill and blast excavation. The scooptram used to remove muck from the tunnel was a Wagner ST3 scooptram with a $2.3m^3$ bucket.

During roadheader excavation the scooptram bucket is filled with muck while the roadheader operates. The excavation typically pauses while the scooptram removes muck from the tunnel. The picks on the cutterhead of the roadheader are replaced approximately every 30m.

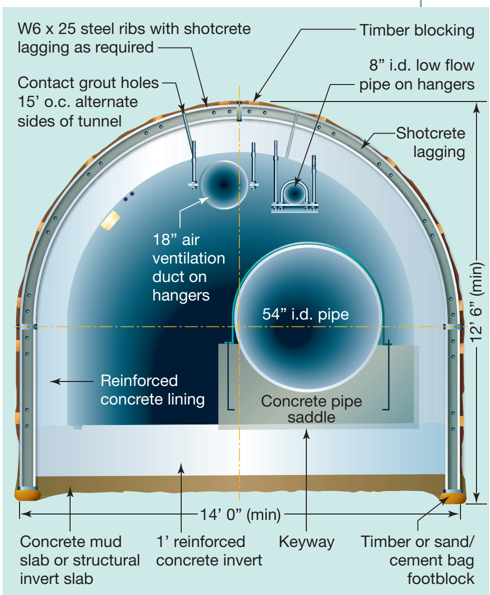
During drill and blast excavation the holes are drilled approximately 600mm longer than the round length (1.8m holes for a 1.2m round, 2.1m holes for a 1.5m round). Perimeter holes are drilled approximately 600mm apart and all other holes are drilled approximately 750mm apart. A full face of drill and blast typically requires between 60 and 70 holes. Drill Tech is using both nitroglycerine and non-nitroglycerine based explosives. Standard non-electric tunnel delays 0-19 are used. Each shot is initiated with non-electric shock tube or safety fuse.

The ground conditions present at the face determine the excavation method used. The roadheader is used in softer ground and drill and blast excavation is used in harder ground and to remove hard blocks in a softer face. Ground conditions beyond the excavated face are investigated via probe holes drilled into the face. During probing, two holes are drilled, each 15m long. Subsequent probing sessions are required to overlap by 6m. Locations of the probe holes in the face are in the left quarter arch and in the lower right of the face. Lookout on probe holes is typically 5° to 7°.

Average cycle time statistics have been generated from data collected by inspectors from HMM for each excavation method. Cycle time for each round includes the time required to install spiling (if used), perform excavation by either method, remove tunnel muck, and erect the steel set. Shotcrete application is not included in cycle time as it is typically applied after several rounds are completed. Figure 3 shows the breakdown of time required for each activity in the different cycles based on the average cycle length of a round. The average time to date to excavate and place a steel set with the roadheader is 3.4 hours compared to 6.4 hours for drill and blast with a 1.2m steel set spacing and 7.3 hours for 1.5m spacing.




Above: **Fig 2 – Longitudinal section**
Right: **Fig 1 – Tunnel cross section**




The largest factor in cycle time is the time required to remove the muck from the tunnel. One benefit of roadheader excavation is that it allows for concurrent excavation and mucking, whereas mucking must take place after detonation of explosives and an appropriate ventilation time when drill and blast is used. Time required to remove muck from the tunnel was seen to increase with tunnel advancement because of the travel time required for the scooptram to discard muck with each trip.

Steel sets were placed at 1.5m spacing for 30m of the tunnel, as per the RESS, because ground conditions were favourable. The increase in spacing allowed for an increased rate of tunnel advancement. When ground conditions changed excavation returned to the 1.2m spacing typically required by the contract.

Tunnel excavation began on 07 November 2007 and was expected to be complete by the time of this publication. Excavation was performed 24 hours a day with two 12 hour shifts Monday through Friday each week with mudslab placement approximately every other Saturday. At the time of writing this article, the average production rate was 3.4m/day with the highest production rate for a single shift of 6m, the highest production rate for a single day of 7.3m and the highest production rate for a single week of 28m.

## Convergence monitoring

In the first 385m of the tunnel, Drill Tech installed convergence monitoring points at seven locations determined by the designer. Monitoring points are typically installed within 1.8m of the face. Measurements are taken with a tape extensometer. The first measurement was generally taken within 48 hours of installation of the monitoring point. Subsequent measurements are generally taken every other day for the first two weeks and weekly thereafter. The rate of convergence is seen to slow over time. To date, maximum convergence measured in the tunnel is 0.80%.

## Pre-excavation grouting

The first time that groundwater inflows reached the pre-excavation grouting criteria of 38l/m from any single feature or 0.38l/m per 300mm of probe hole was on 11 March 2008 at tunnel station T9+80. Flows in excess of 410l/m were measured emanating from 4 probe holes in the face. The face at the time was covered with shotcrete and consisted of crushed shale. After 12 shifts of grouting that pumped 100 bags of Type III cement, tunneling resumed.

Right: **Drill and blast excavation**

## ACKNOWLEDGMENTS

Special thanks are due to the Santa Clara Valley Water District and its staff for permission to publish this paper as well as colleagues from Hatch Mott MacDonald including Dave Young, Mike Murray, and Ronnie Strasser for their mentoring and encouragement

### Site restrictions

The footprint of the site at the portal that is available for staging and laydown is limited. The portal site is located at the base of Lenihan Dam. This requires vehicles and equipment to travel down a 20% grade and around sharp turns in order to transport material into and out of the site. The site for the future intake structure lies within the banks of Lexington Reservoir. Work within the banks of the reservoir is only allowed to take place between 15 June 2008 and 30 November, due to environmental concerns and permit conditions. There is also limited laydown and staging area near the location of the future intake structure. The limitations imposed by the overall project site require special considerations for material handling and scheduling work.

### Safety

The project currently has an excellent safety record with over 30,000 man hours worked and no reportable incidents. All personnel who work on-site at the portal or in the tunnel go through a safety orientation before they are permitted access to the site. Personnel who go through the safety orientation are given a sticker to place on their hardhat to show that they are eligible for site access. Safety measures include a mine rescue team and half face respirators during roadheader excavation.

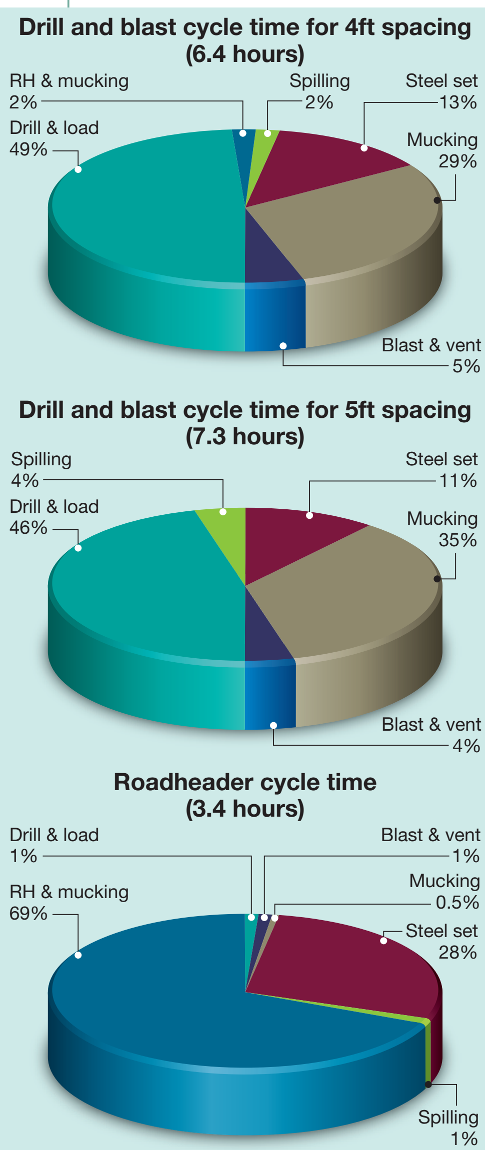
A five person mine rescue team was trained. Because of the dangers associated with using heavy equipment in the confined tunnel size, the number of people permitted in the tunnel at any one time is kept to a maximum of nine and foot traffic is kept to a minimum while the scooptram is operating.

Half face respirators are required to be worn during roadheader excavation due to the amount of dust generated even with the ventilation system set to exhaust and the use of water from a hose and misters on the roadheader to wet the face.

### Environmental

The location of the project site led to a number of environmental challenges. Water quality was an issue of particular concern because of the proximity of the construction site to Los Gatos Creek. Strict water quality standards are enforced for any discharge into the creek and require the dissolved oxygen, temperature, and the change in pH and turbidity to be recorded. The project site is also a potential habitat for a number of protected species in California. Special care had to be taken when dusky footed woodrat nests were relocated out of the limits of disturbance for the project.

The contract requires that noise pollution be monitored during construction activities. Noise levels from construction are not permitted to exceed county limits of 60dBA daily, except Sundays, between 7.00am and 7.00pm, and 50dBA daily from 7.00pm to 7.00am. Noise monitoring is required when a new construction activity begins and measurements are taken from a nearby residence, half a mile from the construction site on the other side of State Route 17. To date, noise from construction activities has not been seen to exceed the county limits with noise from Route 17 registering higher than construction noise.




Left: Fig 3 – Average cycle times for each excavation method

Use of face respirators

When drill and blast excavation is used FCI is required to perform vibration monitoring with maximum allowable peak particle velocities of 12.5mm/sec at frequencies of 10 hertz or less and progresses linearly to 50.5mm/sec at a frequency of 40 hertz or above. Vibration monitoring occurs approximately 75m from the portal area at the abutment of an existing pedestrian bridge and at the closest abutment of a bridge across the dam approximately 290m from the portal.

To date, vibration monitoring during drill and blast excavation has not caused peak particle velocities above the maximum allowable levels. The trigger level of the seismograph is set at 60mm per second and in most cases was not met.

### Current progress

The tunnel will hole-through at the base of the intake shaft. The excavation and support of the intake shaft will be conducted between 15 June 2008 and 15 November 2008 during reservoir drawdown. The shaft will be a minimum of 4.5m in diameter and approximately 10.6m deep. The ground support for the shaft is designed to consist of steel liner plate and steel ring beams.

Final lining of the tunnel will be cast-in-place reinforced concrete. The walls and arch will be 300mm thick and the final invert will be 450mm thick. FCI plans to use two 9m traveling forms. The estimated rate of concrete lining placement is 18m per day. T&T

## REFERENCES

1. Lenihan Dam Outlet Modification Project Geotechnical Baseline Report. April 2007, Jacobs Associates

# Immersed in a Korean challenge

W Janssen, managing director of Tunnel Engineering Consultants (TEC), which is technical advisor to Daewoo E&C, and BH Yang, managing director of the GK Immersed Tunnel Site for Daewoo E&C, describe the Busan-Geoje immersed tube tunnel, in South Korea

Busan is the second largest city in South Korea, located in the south-east of the country and bordered by the Korean Strait to the south and mountains to the North. The new Busan-Geoje Fixed Link is part of a dual-carriage highway that will improve connections to Geoje Island, crossing navigation channels and connecting small archipelago islands to reduce the current 2.5 hour drive from Busan to just 45 minutes.

The principle components of the Link are two cable stayed bridges and a 3240m long immersed tube tunnel with two-lane traffic tubes in each direction. The tunnel has a number of distinctive features, including its long length, its depth of over 50m, its offshore location, aggressive marine conditions, soft subsoil and geographical alignment constraints. These features, combined with the overall scale of the project, make the design and construction of this tunnel a major challenge. It is expected the project will extend the use of immersed tube technology for major crossings.

## The project

The project has been developed as a Public Private Partnership (PPP) project. GK Fixed Link Corporation, a cooperative formed by seven Korean contractors led by Daewoo Engineering & Construction, was awarded the concession to design, construct and operate the Link for a period of 40 years. A JV of Tunnel Engineering Consultants (TEC) and Halcrow are providing technical advice.

Design of the permanent works is now complete and construction of the permanent works has begun, with the last of the first batch of four elements having been immersed in May 2008.

The Link crosses three navigation channels: The main channel between Gaduk and Jungjuk islands, with a width of 1800m, a depth of 18m and no height restrictions; and two secondary channels located between Jungjuk-Jeo islands and Joe-Geoje islands, which have a minimum width of 435m and two times 202m, with clearance heights of 52m and 36m respectively. The water depth for both secondary channels is 16m. Given the requirements, a tunnel is the obvious way to cross the main channel.

The relatively steep shores of Jungjuk and Gaduk islands (figures 1 and 2), and the need for a deep bored tunnel alignment approximately 25m to 30m below the seabed, made it impossible specify a bored tunnel between the two islands. The gradient would be too great and the slopes too long for driver comfort and safety. For this reason an immersed tube tunnel, just under the seabed, was a logical choice.

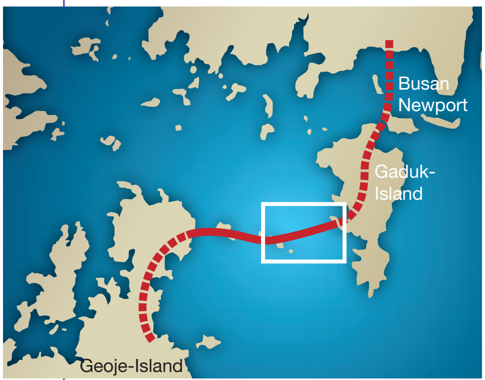
The geological strata varies along the tunnel alignment but top-down typically consists of marine clay, followed by marine sand, and gravel on top of the bedrock.

Marine clay forms the seabed along the tunnel alignment except the shore areas (figure 3) where outcrops of bedrock and shallow sand and gravel layers are found. The thickness of the marine clay exceeds 20m along most of the alignment and locally reaches a thickness of 30m. Most of the tunnel will be founded on this layer.

The clay typically comprises consolidated to slightly over-consolidated soft structured clays. The clay is characterised by internal chalk compounds, which means it acts relatively stiff under low stresses but very soft when the chalk compounds are broken. Plasticity varies from very to extremely high, with an index ranging from 56% to 85% with an average of 68%. The saturated unit weight of marine clay is 13.9kN/m$^3$-15.4kN/m$^3$, with a mean value of 14.6kN/m$^3$.

The channel is exposed to the Pacific Ocean via the Korean Strait and the East China Sea to the South. This affects the marine conditions on site. The maximum design wave height, Hs, is 9.20m and the corresponding mean wave period, Tm, is 15 sec. The principle wave direction, due to typhoons, is south.

Below: Fig 1 – Site location
Right: Aerial view of the project




The current is mainly influenced by the tide, which is typically semi-diurnal with a spring tide range of 1.60m and a maximum current of 0.80m/sec at the tunnel alignment. The waves in the channel comprise three main components:

- Locally generated wind waves, mainly from the north-west and north-east during the winter season
- Deep water generated wind waves, mainly from the south and south-east, during the summer season
- Deep water swell waves, mainly from the south and south-east

Marine works have to account for swell waves with a Hs of more then 0.30m and a period of >6 seconds. In the summer season waves exceed these values most of the time.

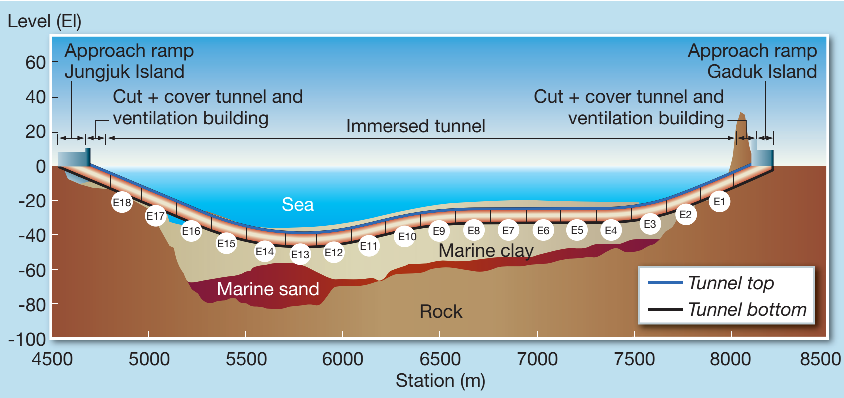
Seismicity in South Korea is mainly governed by the Tsushima offshore and the Yangsan onshore fault systems, located in the depression between the Pohang Bay and Busan. However, only a few major events have been recorded. This explains why, on a large-scale basis, seismic hazard analyses lead to a 'low hazard' rating for Korea. The closest fault to the project is the Yangsan onshore fault and the decisive (characteristic) earthquake will be an event on the Yangsan Fault at a distance of 5km-10km to the east of the project and with a moment magnitude of 5.7-6.

## Features of the tunnel

The Busan-Geoje immersed tube is unique in a number of ways, posing several challenges. From its deepest point, the highway's alignment climbs 95m to its highest elevation, on the single span cable stayed bridge over the middle navigation channel. The maximum gradient is 5.2%, exceeding the maximum design gradient of 4% set by the Authorities under standard conditions. Due to a depression in the seabed at about 350m east of the western portal, a gradient of 5.2% can only be achieved by positioning the tunnel structure about 8m above the original seabed, this requires an underwater embankment to cover and protect the tunnel.

Preliminary soil investigations available at tender indicated a relatively thin thickness of the soft marine clay at the location of the depression, which would have allowed for the use of soil improvement in combination with the raised embankment to bury the tunnel. More detailed soil investigation during design however, showed an extension of the marine clay under the depressed seabed. This would require the vertical alignment to be modified to a deeper position, resulting in a gradient of 6%, which was unacceptable to the Authorities.

An extensive study was therefore carried out to explore other technical options. From a number of alternatives, varying from Sand Compaction Piles (SCP), soil replacement, preloading and deep cement mixing, Sand Compaction Piles for the western side and soil replacement for the eastern side were selected as the most technically appropriate and cost effective methods. These areas extend over a considerable distance at both sides of the tunnel in order to support the sub-sea embankment, which rises about 16m above the original seabed and has to protect the tunnel against grounding ships and erosion.




Right: Fig 3 - Geological profile
Above: Cross section of the elements

The SCP area was pre-loaded with rock to accelerate settlement of the subsoil and minimise settlement after placing the tunnel elements. In addition to the SCP piles at this location, it became clear later in the design process that the marine clay had to be improved over almost the total length of the alignment, for which the use of Deep Cement Mixed (DCM) piles was selected.

The tunnel comprises 18 pre-cast elements, each about 180m in length and approximately 48,000t in weight. The concrete cross section is about 100m$^2$ with outer dimensions of 26.5m x 9.75m. Two elements on the Daejuk island side are tapered and vary in width from 26.5m to 28.5m to create space for a climbing lane.

The elements are constructed in a traditional casting basin, located about 36km from site, which is closed off by a concrete gate structure. The elements are constructed in parallel using a single production line for each element. The casting facility moves along the element length allowing full section casting at various locations. The first batch of four elements took about 11 months and the second four were produced in eight months. The target for each of the remaining batches is seven months.

At the Jungjuk island side of the alignment the seabed is about 35m below the average water level, resulting in 47.5m water to the underside of structure, increasing to about 55m due to wave action. The use of immersed tunnels in Western Europe is limited to a water depth of about 15m. The deepest currently is the Caland tunnel, in Rotterdam, with 26m water to the underside of structure. The Bosphorus immersed tube, in Istanbul, will locally have almost 60m.

The deep alignment has an impact on the immersion process and provisions to prevent water ingress in the tunnel. However, despite the fact that past experience is limited to a water depth of 26m, the concept remains technically feasible.

The elements have been designed so that a defined part of the cross section is under compression ensuring a sufficient barrier

Below: Precast yard and model for fabrication of tunnel elements

Immersion operation underway

against water ingress through the concrete. The full cross section is cast in one process avoiding horizontal construction joints and hydration cracking due to thermal tensile stresses in the second cast. Segment joints are provided with a double seal: A modified injectable waterstop and an omega (W) to ensure reliability during an earthquake.

The trench for the immersed tube was dredged by Van Oord in 2006, in advance of soil improvement of the marine clay, using a HAM 316 trailing hopper suction dredger. To achieve the design dredging depth of about 55m, the dredge pipe of the HAM 316 (with a dredging depth of 40m) had to be modified.

Hydraulic model tests were carried out at the DHI laboratory in Denmark to investigate the effect of the large waves on the permanent tunnel structure. For an extreme event, a typhoon wave of Hs = 9.2m and a return period of 10,000 years was defined. From the model tests the tunnel was found to be subject to vertical uplift, caused by wave troughs passing the tunnel and horizontal loads caused by the varying water head over the tunnel cross section. Both horizontal and vertical forces have been investigated. They increase when the grain size of the backfill material decreases. These forces are however dynamic: Change in direction and intensity cause small movements of the tunnel element allowing water pressure to balance around the tunnel.

Where the tunnel protrudes above the original seabed large waves will have an impact on the stability of the rock-protection. Hydraulic model tests have shown that pre-cast artificial rock elements of more then 30t are needed. In order to reduce the weight and thickness of this layer Accropode- and Core-loc blocks have been chosen for the most affected part of the tunnel.

At both ends of the tunnel the islands have been extended artificially to allow the construction of transition zones between the immersed tunnel and the approaches. Hydraulic model tests performed at the Korean hydraulic institute Kordi showed that Tetrapods with weights of 50t, 60t and 70t are best suited to protect the reclaimed extensions at these locations.

As discussed, the marine clay is the dominant type of soil along the alignment. Its thickness varies but usually exceeds 30m and is located directly below the foundation level of the tunnel. This very soft marine clay, with a very high plasticity combined with low saturated unit weight, low rate of over-consolidation and the structural behaviour of the soil, have been crucial to the foundation method chosen.

Normally, the aggregate equaling the weight of the immersed tunnel, combined with the backfill and rock protection, is less than the weight of the excavated trench material. Due to this, assuming that the original soil does not settle, minimum construction induced settlement will occur. On this basis, immersed tunnels are not usually provided with piled foundations.

The Busan situation is special in this respect. Due to the offshore conditions, the backfill material needs to have a higher unit weight than the marine clay in order to: (i) Resist the uplift forces caused by wave action; and (ii) lock in the tunnel horizontally. This results in an increase of effective stress under the backfill and causes settlement of the backfill and the tunnel. The increase in effective stress could well be in the range of the over consolidation level, implying the risk of increased settlement because of the reduced stiffness in soil behaviour (the ratio between re-compression and compression index is almost 14). In addition, the magnitude of settlement will vary along the alignment due to variations in soil characteristics and amount of backfill. The latter depends on the accuracy of trench dredging, which was expected to be low due of the extreme depths of the tunnel trench and the severe marine conditions.

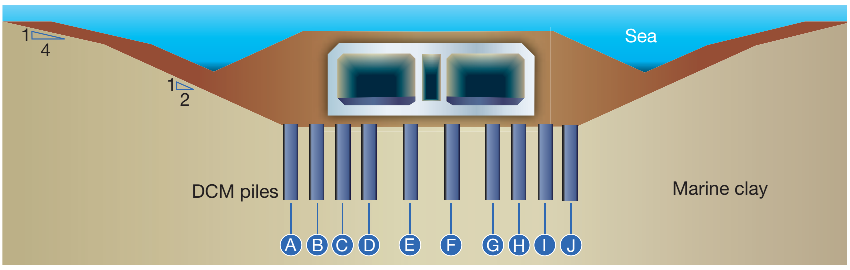
The segmental concrete tunnel has the ability to adjust to limited differential settlements but large joint openings should be avoided. For this reason it was decided to improve the marine clay. At the near-shore ends of the tunnel the marine clay is replaced by gravel. But over the majority of the alignment the clay has been improved by installing Deep Cement Mixing piles.

This method involves injecting cement directly into the clay forming 900mm diameter in-situ columns of clay/cement. Equipment has been used that allows the installation of four columns at a time, forming a square of 1.80m x 1.80m. Where the tunnel alignment is above the original seabed 1.2m-1.8m diameter Sand Compaction Piles have been used. On top of the DCM piles a gravel bed is formed, on top of which the elements are immersed (figure 4).

The use of DCM piles also allows for a gradual change of subsoil stiffness at the locations where the subsoil in the tunnel alignment changes from marine clay to bedrock at the both ends of the tunnel alignment. As such it reduces the differential settlement over these areas.

A two-level earthquake hazard design approach has been adopted. The two earthquake hazard levels are the Operating Design Earthquake (ODE - return period of 100 years) and the Maximum Design Earthquake (MDE, return period of 1000 years). In respect of strength, the MDE is regarded as Ultimate Limit State, but in order to survive seismic loads (prevention of major failure and maintaining safety) the MDE is regarded as Service Limit State, with the

**Right: Offshore equipment for making SCP piles up to 70m depth**




Above: **Fig 4 – Cross section DCM piles**

requirement that all joints shall remain watertight and rebar stress does not exceed yield strength - fyk.

Peak Ground Acceleration (PGA) of 0.154g for the MDE event is specified in the Korea codes. Design Peak Ground Velocity (PGV), and Peak Ground Displacement (PGD) of rock outcrop motions, based on the specified PGA and the characteristics of the controlling earthquakes, is in the range of PGV: 8-10cm/sec and PGD of 2-3cm. Free-field soil motions at discrete elevations of the tunnel elements are determined using SHAKE. Bedrock ground motions can be accelerated to 0.35g and the maximum free-field displacements (PGD) at rock and tube level are 2.9cm and 5.5cm, respectively.

Ground movement along the tunnel alignment has been determined for each direction for various displacement time histories. The soil structure interaction has been divided into three types of tunnel responses: Worming (axial compression and extension along the alignment of the tunnel); snacking (longitudinal vertical and horizontal curvature); and racking (distortion of the cross section).

The first and second responses were analysed as longitudinal models, in which the tunnel segments have been modelled as frame elements supported in three directions by soil springs. Free field displacements at tunnel level were applied to the soil springs.

The racking was analysed using a 2D finite element model. The models were used to estimate the openings of the joints and forces in the structure and shear keys during seismic event.

## Conclusion

The Busan-Geoje immersed tube tunnel is unique for many reasons. Its features go well beyond common concrete immersed tunnel technology. Not all of the special design aspects were identified in depth at the beginning of the project, but awareness developed during the design process.

At the time of writing this paper, the design stage had been completed and construction was well on its way. The first four elements have been successfully immersed in the tunnel trench and the next batch is waiting for immersion, expected in October 2008. The completion of the Link is expected in 2010. T&T

# Dealing with inflows

Construction of access tunnels A and B for the Jinping hydro power project in China was delayed for almost six months due to high pressure water inflows. Ziping Huang, of Norconsult AS, and Shiyong Wu, of the Ertan Hydropower Development Co (EHDC), detail the inflow problems

The 17.5km long Jinping Access Tunnels serve as exploratory, test and construction adits for the Jinping II headrace tunnels. They are also vital for construction and materials transport access to both Jinping I and Jinping II hydro projects. Bypassing 138km of roads located in steep valleys, the early completion of the twin tunnels is important not only to the construction of Jinping, but also continued development of other projects along the Yalong River valley[1].

The Access Tunnels are characterised by their long length, high overburden, high groundwater pressure and complex geology. Therefore, effective methods have been needed to solve key technical issues such as large high-pressure water inflows, high rock stresses leading to rock bursts and squeezing, as well as possible adverse ground conditions in large fault zones.

The Access Tunnels consist of two parallel tunnels spaced 35m apart. Tunnel A has a cross section of 30$m^2$, with tunnel B measuring 35$m^2$. Work on the tunnels started in October 2003, with breakthrough planned for the end of 2007. Actual breakthrough of Tunnel B occurred on 16 May 2008, with breakthrough of Tunnel A on 08 August 2008. Large water inflows were the main reason for the delays.

## Ground conditions

The rocks along the twin access tunnels' alignment belong to the Triassic period (Figure 1). They mostly comprise carbonate rock and slate, metamorphosed sandstone and a small amount of green schist. The carbonates occupy 16.23km of the 17.5km. Intact rock has a saturated (wet) uniaxial compressive strength (UCS) $\sigma c$ = 60-95MPa. All rock, except for the schist (and mudstone) is brittle. According to the Chinese Rock Mass Classification System[2] 96% of rock masses belong to Class II and Class III.

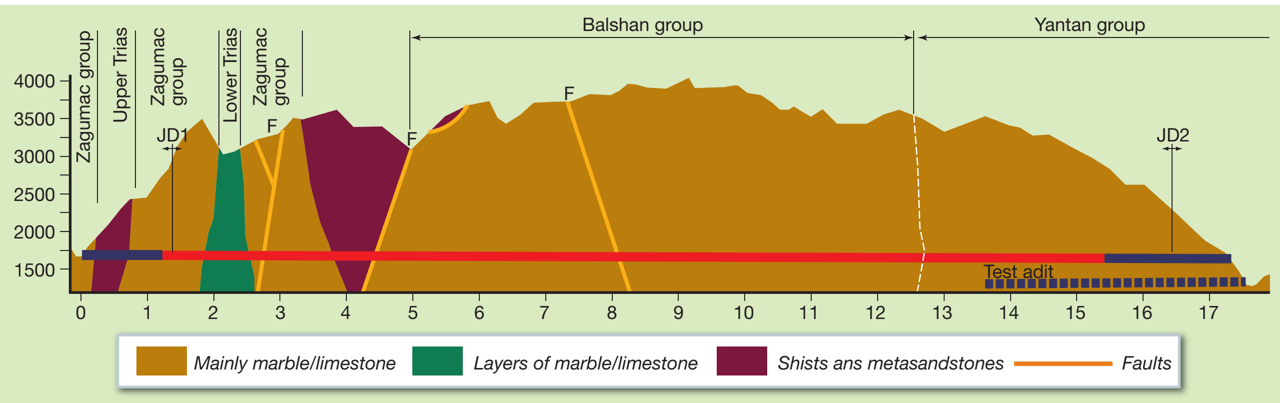
The rock mainly dips eastwards in the eastern half of the alignment and towards the west in the western half, with strike generally N-S to NNE-SSW. They therefore intersect the Access Tunnels mostly at a right angle.

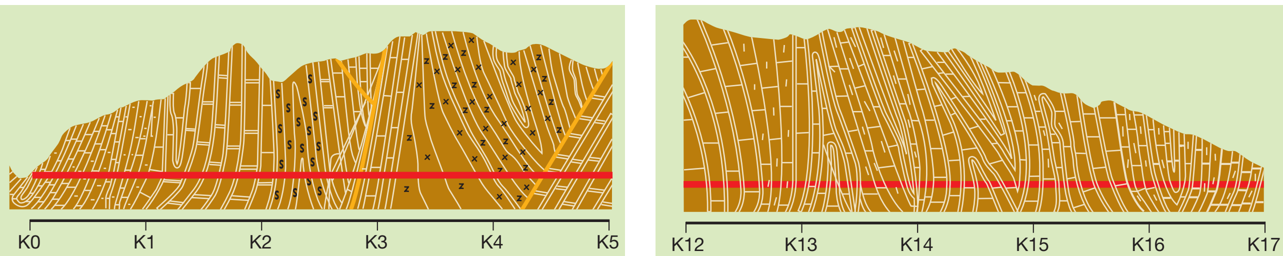
During excavation 187 faults were encountered in the twin adits with maximum thicknesses ranging from 1.7m to 5m. Tensile faults with a strike of NWW-NEE dominate. There were approximately six joint sets observed, mainly tight. Joints along bedding/foliation planes in limestone/ marble are often open in nature and most faults and joint sets generally have the same orientation; joint sets 1 and 2 plus fault F8 cross the Access Tunnels at an oblique angle, the rest mainly lie across the tunnels. The folds developed tightly and mainly strike SN (Figure 2). The NNE oriented structures are featured as a type of extrusion while the NWW oriented are tension and torsion[3].

## Water-bearing structures

The steep topography results in very high water pressures at tunnel level, with more than 1500m of overburden for 73% of the access tunnel length. As mentioned, a major portion of the Jinping tunnels are located in marble/limestone, ranging from pure marble/ limestone via brecciated marble to

Below: Fig 1 – Section of the Access Tunnels; Bottom: Fig 2 - Folding structures in the west side (l) and east side (r) of the access tunnels



Above: **Inflow at AK13+520 on 23 September 2006**
Right: **Moderate inflows from fractured zones in the west side of Tunnel A**

argillaceous limestone. Therefore karst features, resulting from water travelling along discontinuities (joints, fissures, shears, faults, etc), were a genuine concern. During pre-investigation, it was found that the most developed karst structures were in the Baishan ($T_2b$) and Zagunao ($T_2z$) groups, which occur along more than 10km of the tunnel. With high groundwater pressures, potential water inflows could be large. Such occurrences could lead to problems, time delays and high extra costs.

However, based on thorough pre-investigation and study, it is believed unlikely that a large continuous karst cave system exists at the level of the access tunnels. It is more likely that groundwater occurs under the following circumstances:

- Intersection between different rock types, (e.g. where the tunnel moves from a soft rock mass to a hard soluble rock mass)
- Development of folding, where the tunnel crosses larger folds, especially a syncline
- EW and NNE striking discontinuities (faults or large joints)
- Intersection between discontinuities striking EW and some rock layers

## Planned water treatment strategy

Based on the geological conditions and concerns about karst development, the water treatment strategy was to detect possible large water inflows ahead of the tunnel advance to ensure safe working and to try and avoid delays and extra costs. Pre-grouting was to be carried out throughout the water-bearing zone. Methods to detect water-bearing structures included geological forecasts, geophysical methods and drill holes. The 'geological method' makes use of geological information gained from field investigations and construction, so it was important the geologists closely followed the excavation and the results of other forecasting methods used. The geophysical methods made use of propagation, reflection and/or refraction of waves in the rock masses, including Tunnel Seismic Prediction (TSP) for long distances from 50m to 100m and 1-2 hours testing period, Ground Penetrating Radar (GPR) for distances of 0m to 20m ahead of the tunnel face, and Ground Sonar (GS). Probe holes 5m-50m long (typically 30m) were drilled ahead of the tunnel face, using the jumbos employed for blasting. Modern jumbos can record features such as drill speed, thrust, penetration, and amount of water supply. These results were combined with geology in a forecast of water-bearing structures and their positions.

Norwegian experience of sealing water inflows in different types of tunnels has shown a combination of medium length probe holes and pre-grouting are a good solution considering time consumption and cost. To carry out this approach, grouting equipment, experienced advisers and a trained crew are required to select the right grout mix[4]. Emergency treatment of large water inflows had also been planned.

## Large water inflows

The largest water inflows experienced were a 7m³/sec flow into the twin tunnels on the east side, with a judged maximum pressure of 4-5MPa (maybe even be higher); and 3.4m³/sec with maximum water pressure of 6-7MPa in the west side tunnels. After grouting, the inflows reduced to 1.4m³/sec in the west side tunnels. In the east side, total inflows of about 7m³/sec did not alter during excavation. Part of Tunnel A is therefore now used as a drainage tunnel.

*Inflows into the west side of the tunnels*

Numerous inflows were experienced in the west side of the access tunnels[7]. A summary of only the most significant of these is given in Table 1 and Table 2 for Tunnel A and Tunnel B, respectively (p40). As shown, there were large inflows in two sections in Tunnel A and four sections in Tunnel B. Descriptions of the inflows are given briefly in the notes. The initial inflow represents the maximum amount as the inflow reduced with time in all cases - the total steady inflow is much less than the sum of the initial inflow. Water inflows after post grouting are listed as well.

*Inflows into the east side of the tunnels*

**Inflow at BK14+888:** The first significant high-pressure water inflow encountered in the eastern section was in Tunnel B, at Station BK14+888 on 08 January 2005. It was near the roof, at 3.2m depth, from a blast hole. The inrush was estimated at 200l/sec with an assumed pressure of 4.7MPa. The water jetted out 18m and negatively influenced work conditions about 50m from the face. The contractor had used GPR to forecast groundwater. This showed that in the section BK14+460 to BK14+445 the rock was not uniform, suggesting groundwater existed. TSP had also been used, but gave no evidence of water. Probe drilling ahead of the tunnel with six 5m long holes in the contour along the walls (with no overlap) had also been performed in this section. According to the Contractor, it did not detect any water under (high) pressure. Some water had occurred in the blast holes, but it was not until drilling blast holes in the upper part of the face that the inrush took place. Inflow measured 164l/sec in the funnel pipe. This remained the case until 30 March 2005, when there was another inflow in Tunnel A at AK14+756. It was almost a year before further excavation could be carried out[5].

According to the geological observations, there is a fractured tensile zone oriented N70°W, NE∠65°, opening up to 10cm, in $T_2y^5$ rock, i.e. dark/grey limestone and marble. The water seemed to be supplied from this structure. In several locations, in the left and right side walls, there are tensile torque joints close to EW, which are near vertical. There are karstified fissures and cavities along the fault or tensile joint planes. The karst development most likely formed local channels where the calcite rock dissolved. The channels may form a network along the fault or joint plane.

**Inflow at AK14+762:** On 30 March 2005, 10 minutes after blasting, water burst into Tunnel A at Station AK14+762 in the lower part of the left side wall, near the tunnel face. The water came from a karst channel. The high-pressure water inrush came from a cavity 1.2m wide, 2.8m high and 4.2m long. Maximum inflow was measured at 6.7m³/sec. The water bearing structure is

**Table 1: Inflows greater than 20l/sec in the west side of Access Tunnel A**

| Note | Date | | Station | | Section Length | Initial/Max inflow | Inflow after post grouting |
|---|---|---|---|---|---|---|---|
| | From | To | From | To | m | l/s | l/s |
| 1. | 24.07.2004 | 03.09 | Ak1+088 | +117 | 29 | 300 | |
| 2. | 23.05.2006 | 11.06 | AK5+150 | +236 | 86 | 1600 | 500 |
| **Total inflows (steady inflow)** | | | | | 825 | 2765 | 825 |

1) Tunnel A at AK1+117 encountered a large water inflow of about 150l/sec and pressure 1.2MPa, jetting 16m from the wall. More than 20 locations experienced various inflows and dropping water - these inflows varied from 20 to 150l/sec. Inflow vanished after about 40 days. Sealing work took two months. Inflows are from N55W/70NE fault f1 influenced fracture zones that consist of karst denudation fissures, dissolution small cavities connected by fissures. Water related to a small fault has a width of 0.2-0.5m and an influenced zone of 1-2m. The gushing water came from a 1-2cm wide opening. The rock is marble in $T_2z$ group.

2) Inflow from blasting or rockbolt holes ranged from 50-100l/sec, pressure 1-2MPa. Water jetted a maximum distance of 45m to the tunnel. Up to 7 October 2006, there was still about 1050l/sec total inflow, with seepage all along the section with more than 30 inflows. There are faults f10 and f11 crushed zones oriented NEE and SEE, along which exist well developed karst denudation fissures, channels and cavities, and some karst caverns in sizes of 0.5mx0.5m, 0.8mx1.2m, 1.2mx1.4m in the hanging wall of Fault f11, in marble of $T_2b$ group.

controlled by two sets of tensile fractured zones oriented N65°E, NW∠82° and N70°W, NE∠64°, in rock $T_2y^5$. The water inflow is constant at over $2m^3$/sec[6].

**Inflow at AK13+878:** On 15 March 2006, water burst into Tunnel A at Station AK13+878, jetting out 10m, and measuring about 2.2-2.7$m^3$/sec. Water came from the bedding plane of N5°W, NE∠70°. Three days earlier, there was water inflow of about 15l/sec at Station AK13+892 from a bedding plane of N10°W-NE∠65° in rock $T_2y^6$. The GPR test indicated a water bearing fractured zone strike N70-80°W. The fractured zone crosses the bedding plane, forming a water channel network. The inflow has remained constant to date.

**Inflows at AK13+520 and AK13+494:** On 18 July 2006, gushing water of about 1.26$m^3$/sec was encountered at AK13+520. The water inflow remained constant, maybe even increasing a little. The water bearing structure of a fractured zone has an orientation of about NWW.

When preparing blast holes at AK13+494 on 26 January 2007, a large amount of water (about 2.8$m^3$/sec) gushed into Tunnel A. The water bearing structure is oriented N80°-85°W, NE∠82° in rock $T_2y^5$. Inflow reduced but remains constant to today.

## Characteristics of large inflows

There is a discrepancy on the change of inflow with time when comparing inflows in $T_2z$ rock group with those in $T_2b$ and $T_2y$ rock groups. In $T_2z$ rock of Zagulao group, the large amount of water that burst into the tunnel at AK1+177, BK1+130 and AK2+637 reduced rapidly and vanished in a short time. This means that the water bearing zones are large but are not connected to continuous water resources. The formations of three rock groups are also different. In the Zagulao group, there are other types of rocks than carbonates, such as slates, metamorphosed sandstone and a small amount of green schist that are not soluble and thus probably isolate the water bearing zones.

$T_2b$ rock of Baishan group and $T_2y$ rock of Yantang group are pure carbonate rock. The water bearing structures consist of networks of fractured zones and karst channels that have good connections to water supply resources towards the southern region, meaning the inflows were continuous once encountered.

There are approximately two sets of faults or fractured zones with strikes close to SN and NEE-NWW, dip angles mostly from 70° to 88°. The NEE-NWW oriented faults or fractured zones with majority of tensile faults contributed the major percentage of the large water inflows. The faults are classified as Class II and Class III according to the Chinese Classification System on Geological Structural Planes. These faults normally extend from tens of meters to hundreds of meters, with widths of <1m for Class III and >1m for Class II.

## Discussion

The characteristics of the water bearing structures determined the method chosen to deal with the inflow. In forecasting inflows, identification of faults or fracture zones is extremely important. The probability is higher for a planar structure to be detected than for a karst channel or network. Therefore, it is possible to implement forecasting methods more efficiently than was generally done, although there were some inflows from karst cavities or channels that were extremely difficult to detect. Regardless, it is not certain that pre-grouting of the high-pressure water would be successful at all locations even after the water was detected. Drainage should have been an essential consideration when tunneling in such conditions.

The preliminary approach of forecasting and pre-grouting first, followed by drainage and excavation, was therefore changed. Instead, forecasting (using TSP and GPR, sometimes probe drilling) was carried out first to a certain extent to detect any large karst caverns, to allow excavation through water bearing zone without any pre-grouting, leaving water encountered to be drained. This was then sealed by post grouting.

However, since there was no drainage channel in the initial design, part of Tunnel A had to be used as a drainage tunnel together with an excavated drainage shaft and other adits. However, this increased excavation time and meant greater efforts were required to facilitate Tunnel A's function as a traffic route – its intended use. This change of approach ultimately led to the TBM construction of a drainage tunnel, which was due to reach the three largest inflows in the east side in September 2008. If this decision

**Table 2: Inflows greater than 20l/sec in the west side of Access Tunnel B**

| Note | Date | | Station | | Section Length | Initial/Max inflow | Inflow after post grouting |
|---|---|---|---|---|---|---|---|
| | From | To | From | To | m | l/s | l/s |
| 1. | 02.09 2004 | - | BK1+078 | +164 | 86 | 7000 | 0 |
| 2. | 20.04.2005 | 09.05 | BK2+637 | +690 | 55 | 15600 | |
| 3. | 02.05.2006 | 09.05 | BK5+071 | +098 | 27 | 400 | 130 |
| 4. | 08.06.2006 | 12.06 | BK5+307 | +346 | 39 | 300 | 115 |
| **Total inflows (steady inflow)** | | | | | 1057 | 2815 | 682 |

1) Water inflow of 200l/sec from a probe hole at BK1+130 burst into the tunnel 35m away. Within the 30m long tunnel, water came from everywhere and there were more than 10 holes with inflows. The maximum inflow at BK1+137 on 23 September 2004, reduced to 1000l/sec after 30mins, 260l/sec after 24 hours, and vanished after the Tunnel A at AK1+270 was excavated. Inflows are related to N55W/70NE fault f1 and the fracture zone that consists of karst denudation, dissolution small cavities connected in a fissure in marble of $T_2z$ group.

2) At BK2+637, inflows started on 20 April 2005. Maximum inflow of about 15.6m3/ecs reduced to 240l/sec after 19days, and vanished after two months. The flow has a large amount of sediment and stopped work for a week. Located in Class I structure surface, Fault f6 influenced the fractured zone that consists of karst denudation, dissolution small cavities connected in a fissure, and a karst cavern of 15m high, 5m wide found in marble of $T_2z$ group.

3) Related to fault f9 and its influence zone consisting of six fractured zones of NE orientation. Water burst into the tunnel 45m away, at a pressure of 4MPa, dropping water all along the section with inflows on the south side wall. The fractured zone consists of well developed fissures, karst denudation fissures, openings and karst cavities in marble of $T_2b$ group.

4) Water bearing structures influence the zone in the hanging wall of fault f11, consisting of tensile fissures and karst cavities in marble rock of $T_2b$ group. The fractured zone is connected with the water bearing zone in section AK5+150~+240 in Tunnel A.

had been made sooner, it would have allowed more efficient drainage. Depending on TBM progress, the plan to reopen Tunnel A to traffic within the scheduled time is uncertain. To seal the three largest water inflows, a possible alternative may be the construction of weirs to keep out the water and reconstruct the drainage channel in Tunnel A to pass water out of the tunnel.

It is not only the features of the water bearing structures that can affect the treatment of inflows. Implementation of the planned approach by forecasting and pre-grouting requires the best in today's drilling and grouting equipment, materials and expertise, combined with on site decision making by experienced personnel. At the end of 2004, drilling machines with air-leg feeders were abandoned for modern jumbos, however old-fashioned grouting equipment remained in use throughout the excavation. It was not capable of effectively treating the large high-pressure inflows[3].

It is important that experienced geologists and grouting experts make timely decisions at site; when to stop excavation and start grouting, how to improve the probe drilling and geophysical forecasting methods, identify geological conditions and establish criteria based on measurements from probe holes or blasting or rockbolt holes and geophysical prediction data[5].

Breakthrough of the Jinping A and B Access Tunnels was achieved in May and August 2008. Even though excavation was delayed for almost half a year as a result of the large high-pressure water inflows encountered, it is still a achievement in rock tunnelling.

Given the high pressure and large amount of underground water at this project, and difficulties in both forecasting and pre-grouting water inflow, it is unclear whether successful forecasting and pre-grouting would be feasible all the way along the tunnels within a reasonable time limit.

Therefore it is also important to consider the design of a water drainage channel within such tunnels, or a separate drainage tunnel, for successful excavation under such challenging hydrogeological conditions. T&T

## REFERENCES

1. Norconsult Report No1, 2004. Rock stability analysis, classification and support. September, 2004.
2. X Ren, T Li & X Chen, 2001. Structural analysis for the liner and surrounding rocks of diversion tunnel in Jinping waterpower station[J]. Chinese Journ. of Rock Mech & Eng, 20(1), p16–19
3. X Ren, M Wang & S Wang, 2004. Study on the external water pressure of deep lying tunnel in Jinping hydropower station[J]. Hydrogeology & Eng. Geology, 31(3) 85–88
4. Norconsult Report No3, 2004. Forecasting of groundwater, pre-grouting methods & emergency treatment of unexpected flowing water. September, 2004.
5. Norconsult Report No5, 2005. Treatment of the water in-burst in East-end Tunnel B and future risk-reducing measures. February, 2005.
6. S Wu, J Wang & G Wang, 2007. Underground water and its treatment strategy in auxiliary tunnels of Jinping Hydropower Project. Chinese Journ. of Rock Mech & Eng, 26(10)
7. Z Huang & S Wu, 2008. Dealing with water inflows at Jingping [unabridged version]. International Waterpower & Dam Construction, October, p24-30.

# Discs and boulders

DS Kieffer, Professor at Graz University of Technology, Austria, C Leelasukseree, a Lecturer at Chaing Mai University, Thailand, and GGW Mustoe, Professor at Colorado School of Mines, USA, describe recent research into the behaviour of TBM disc cutters in boulder rich ground

Ground characterised by hard boulders embedded in a soft matrix represents one of the most challenging conditions for TBMs, as the cutterhead must cope with the non-optimal conditions of cutting hard blocks while simultaneously excavating through soil-like material. Dowden and Robinson[1] and Hunt & Mazhar[5] summarise several case histories relating to this problem and provide guidance with respect to site investigations and baselining strategies. Some rock types constituting boulder laden ground include:

- *Sedimentary rocks* – cemented alluvial channel deposits, colluvium/talus, landslide debris, and glacial till
- *Igneous rocks* – agglomerates, lahars, and pyroclastic deposits
- *Metamorphic rocks* – tectonic mélange and cataclastic fault zones
- *Decomposed rocks* – corestone bearing saprolites

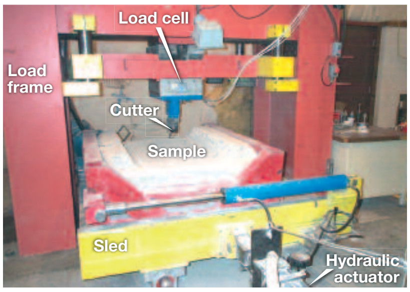
Disc cutters are commonly utilised to cut hard blocks; however, compared to the understanding of cutter behaviour in hard continuous rocks, little is known regarding mechanisms in highly heterogeneous bouldery ground. To advance this understanding a range of full-scale tests have been performed using a linear cutting machine at the Colorado School of Mines (CSM). This paper summarises the observed mechanisms and their implications.

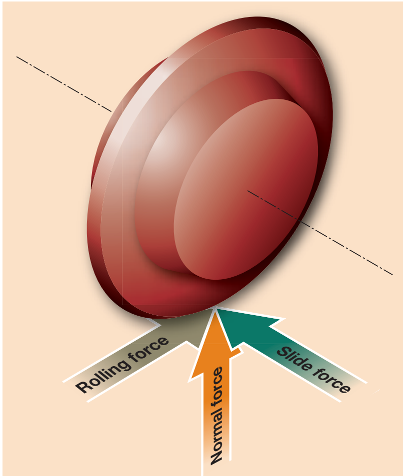
## Linear cutting machine tests

The Linear Cutting Machine (LCM) at the Earth Mechanics Institute of CSM is used for simulating full-scale rock cutting conditions (figure 1). Through nearly 30 years of comparisons between LCM test results and actual field performance, this apparatus has proven effective in developing accurate performance estimates for different cutting tools[1]. For the tests described herein, a

**Table 1: Physical properties of grantic boulders**

| Physical property | Maximum value | Minimum value | Average value |
|---|---|---|---|
| Density *(g/cm³)* | 2.78 | 2.61 | 2.68 |
| UCS *(MPa)* | 336 | 97 | 183 |
| Static elastic Modulus *(GPa)* | 39 | 26 | 33 |
| Static Poisson's ratio | 0.19 | 0.17 | 0.18 |
| BTS *(MPa)* | 5 | 14 | 19 |
| Cohesion *(MPa)* | - | - | 17.5 |
| Friction angle *(Degrees)* | - | - | 57.6 |




Above: **Fig 1 – Linear Cutting Machine**
Right: **Fig 2 – Disc cutter force components measured during LCM tests**

load cell and 150mm disc cutter were mounted on a stiff frame, with the load cell recording normal, rolling, and side forces acting on the cutter (figure 2). The normal force is perpendicular to the rock surface and provides a measure of the force required to indent the sample, from which TBM thrust force requirements can be estimated. From the rolling force, TBM torque and power requirements can be estimated; and from the side force bending moments on the cutter can be estimated.

Rock specimens are cast in steel boxes that ride on a sled, which travels on a pair of rails beneath the disc cutter. The sled motion is specified with a servo-controlled hydraulic actuator. The LCM data acquisition system records each cutting force component at a rate of 1000 data points per second. These data are analysed statistically to estimate the average, minimum and maximum cutting force components.

Due to the difficulty of obtaining undisturbed full scale samples of hard boulders embedded in a weak matrix, the LCM samples consisted of granitic blocks cast into simulated matrices of sand grout. The blocks utilised in the LCM tests were natural alluvial channel deposits (with maximum dimensions of about 140mm-360mm) collected along Clear Creek in Golden, Colorado. The blocks were selected based on similarity of rock type, grain size, texture, weathering, and shape. Laboratory tests performed on nine boulders included Unconfined Compressive Strength (UCS), Brazilian Tensile Strength (BTS), static elastic constants tests, and triaxial shear tests, and the estimated physical properties derived there from are summarised in Table 1. In total, three LCM samples having nominal dimensions of 1050mm (L) x 1050mm (W) x 440mm (D) were constructed, with matrix materials fabricated to represent: (1) weakly cemented sand; (2) very weak sandstone; and (3) strong sandstone.

Strength and deformability parameters for the matrix materials, based on triaxial shear and UCS testing, are summarised in Table 2 and ratios of boulder to sand grout strength and deformability parameters are shown in Table 3. Leelasukseree[3] presents a full discussion of the laboratory test programme.

Each LCM sample included three granitic boulders cast within the sand grout material. The samples were tested after minimum 28 days of curing, and in each test disc cutter penetration and spacing were held constant at 3.8mm and 51mm, respectively. A schematic of the cutting process, achieved by sequentially offsetting the disc laterally, then progressively cutting down through the sample, is shown in Figure 3.

## Test Results

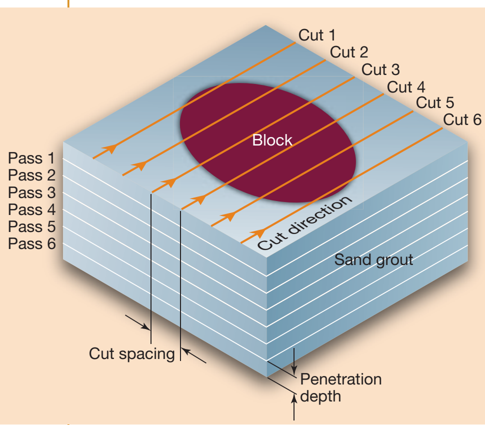
To emphasise basic differences with respect to the behaviour of blocks embedded in

**Table 2: Physical properties of simulated sand grout matrices**

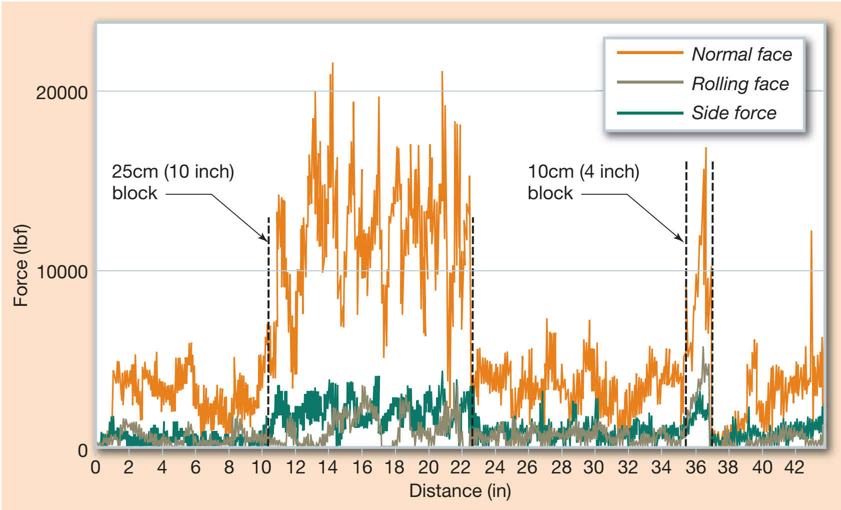
| Physical property \ Sand grout stiffness | | Stiff mix | Medium mix | |
|---|---|---|---|---|
| Cement content (%) | | 20 | 7 | 3 |
| UCS -28 day strength *(MPa)* | | 34 | 1.30 | 0.32 |
| Estimated physical properties | Elastic modulas *(GPa)* | 19.1 | 0.12 | 0.04 |
| | Cohesion *(MPa)* | 5.5 | 0.28 | 0.08 |
| | Friction angle (°) | 37.5 | 43.6 | 37.5 |

**Table 3: Ratios of bolder to sand grout strength and deformability parameters**

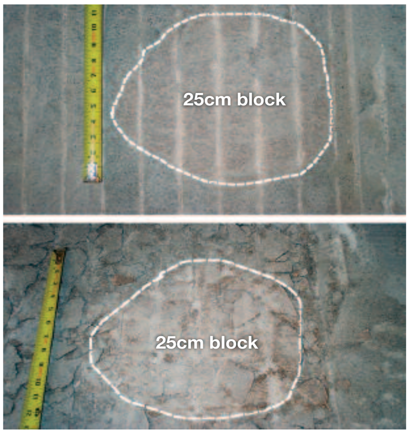
| Boulder to sand grout ratio \ Sand grout mix | Stiff sand grout | Medium sand grout | Soft sand grout |
|---|---|---|---|
| UCS | 4.7:1 | 125:1 | 503:1 |
| Elastic modulas | 1.7:1 | 278:1 | 906:1 |
| Cohesion | 3.2:1 | 63:1 | 221:1 |
| Friction angle | 1.5:1 | 1.3:1 | 1.5:1 |



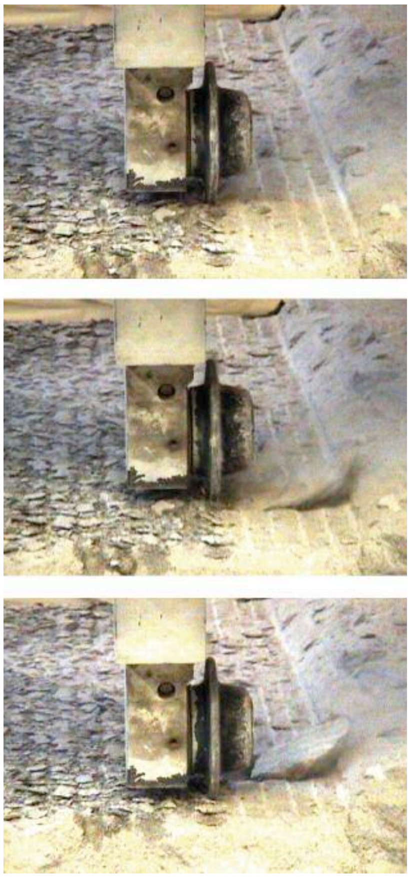

Above: **Fig 3 – Sample cutting process**




Above: **Fig 4 - Example of data collected during a single disc cut across LCM sample box (highest normal forces correspond to cutter passing over granitic blocks)**




Above: **Fig 5 - Classical chip formation (boulders embedded in stiff sand grout)**



Above: **Fig 6 - Adhesive bond failure between block and stiff sand grout**

matrix materials of significantly different strength and stiffness, the following summary of test results concentrates on behavioural modes exhibited by boulders embedded in the stiffest and softest sand grout matrices (Table 2). Figure 4 shows a typical data plot of normal force, rolling force, and side force collected during a single pass of the disc across the sand grout matrix and two granitic blocks. The physical boundaries of the boulders are indicated by dashed lines, and the boulder domains are represented by high values of cutting force.

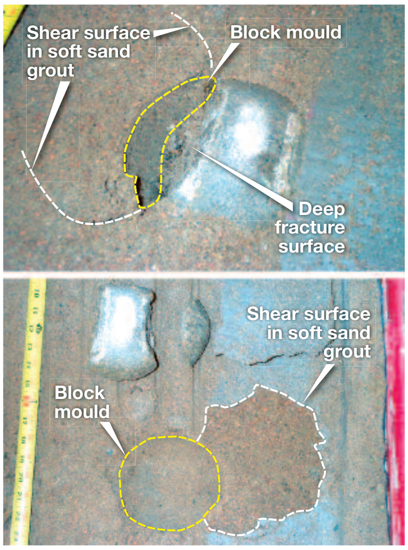
The principal difference of the data collected for the stiff and soft sand grouts is that: (1) peak boulder normal forces are systematically on the order of 20% to 30% less when embedded in soft sand grout; and (2) peak boulder rolling forces are systematically on the order of 20% to 35% less when embedded in soft sand grout. With regard to boulder side forces, no clear distinction between the stiff and soft sand grouts was evident.

For the boulders embedded in stiff sand grout, the ratios of boulder to grout UCS and elastic modulus are approximately 5:1 and 10:1, respectively. For this case, cutting of both the sand grout and boulders occur in the classical chip formation mode associated with disc behaviour in uniform hard rocks (Figure 5). This cutting mode occurs as the disc crushes the block and matrix, causing radial fractures to propagate and coalesce during adjacent cuts.

Figure 6 also depicts chip formation in the boulders and stiff sand grout; however, as the cutter penetrates through the boulder, a condition is reached wherein the boulder dislodges laterally by failure of the adhesive bond between the boulder and sand grout.

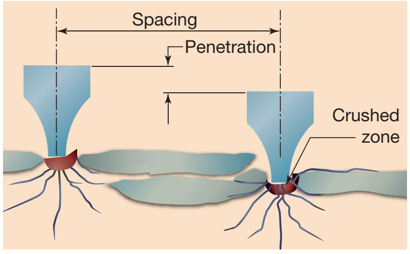
For the boulders embedded in soft sand grout, the ratios of boulder to grout UCS and elastic modulus are approximately 500:1 and 900:1, respectively. For this case, cutting occurred in a markedly different mode than for boulders embedded in stiff sand grout. After the disc cutters penetrated through


Above: Fig 7 - Characteristic fracturing of blocks embedded in soft sand grout
Below: Fig 8 – Shear (bearing capacity) failure through soft sand grout




approximately 10% of the boulder, deep through going fractures developed in the boulders as depicted in Figure 7.

Once the deep fractures formed, subsequent passes of the disc cutter resulted in lateral prying action against the crack surfaces, but instead of an adhesive bond failure along the boulder-sand grout interface, shear failure in a bearing capacity mode occurred (Figure 8).




Above: Fig 9 - Chip formation of blocks embedded in a stiff matrix[7]
Above right: Fig 10 - Conceptual models for behaviour of embedded hard blocks: (a) stiff matrix: shear failure along block-matrix adhesive bond; (b) soft matrix: deep block fracturing leading to matrix bearing capacity failure under eccentric and inclined loading; and (c) effect of eccentric and inclined loading on bearing capacity failure surface

## Discussion and conclusions

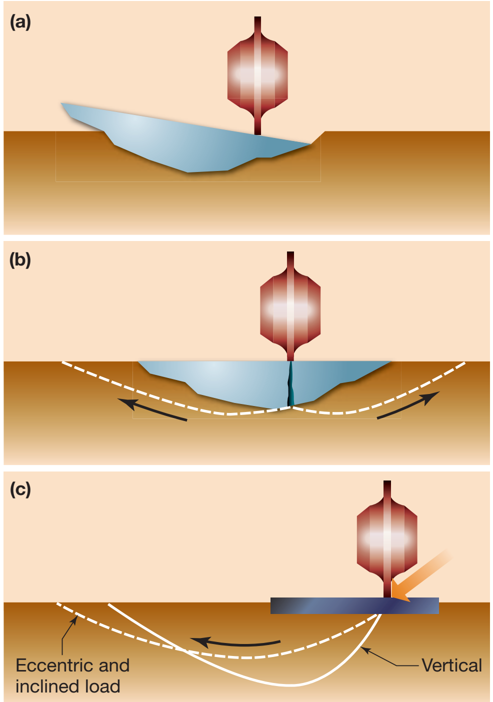
As enumerated below, disc cutter action for boulders embedded in a fine grained matrix include: (1) classical chip formation associated with cutting homogenous hard rocks; (2) boulder plucking by adhesive bond failure; (3) through going boulder fracturing; and (4) boulder plucking by matrix bearing capacity failure. Summarised below are conditions favouring each of these cutting mechanisms as discerned from LCM tests and subsequent numerical simulations[3, 4].

Provided that embedded boulders are stiff and strong and that the ratios of boulder to matrix UCS and stiffness are not exceedingly disparate, overall cutting behaviour will generally correspond to that of hard continuous rock (Figure 9). The main exception to classical hard rock cutting is that boulder plucking can occur via adhesive bond failure as depicted in Figure 10a.

This plucking mechanism is likened to bearing capacity failure under eccentric loading, with the presence of a preferred failure surface along the weak boulder-matrix interface. Load eccentricity is derived from application of the disc cutter force outside of the boulder centroid.

With significant differences of boulder to matrix UCS and stiffness ratios, the tendency is for boulders not to chip, but to experience through-going fractures oriented parallel to the direction of load application (Figure 10b). The reason for through going fractures is that the boulder behaves as a non-uniform beam supported by a compliant foundation (matrix). Compliance of the soft matrix also attenuates the normal and rolling cutter force components by approximately 20-35% (compared to the stiff matrix).

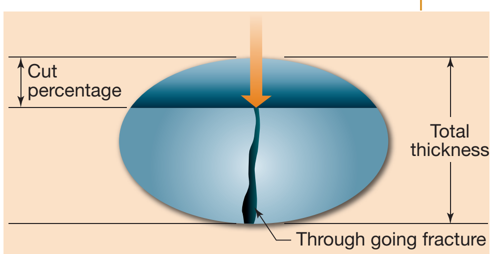
Deformation of the matrix under the

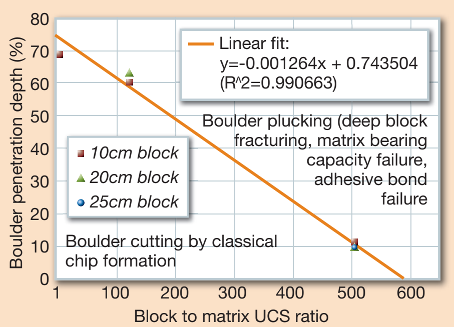


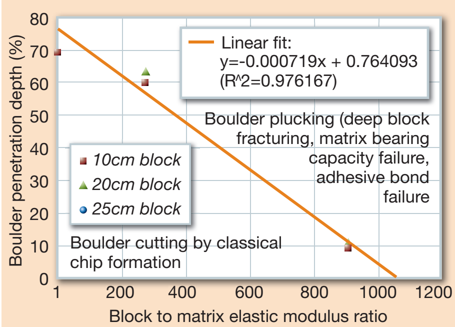

applied disc loading results in high tensile stresses along the lower outer fibre of the beam, leading to brittle tensile fracturing parallel to the maximum principal stress. This behaviour is not observed when the matrix stiffness is sufficiently high. Once the through going boulder fracture develops, plucking of the boulder occurs via: (1) adhesive bond failure as described for the stiff/strong matrix condition; or (2) bearing capacity failure through the matrix material.

As the matrix material becomes sufficiently soft/weak, bearing capacity failure through the matrix is preferred, rather than adhesive failure along the non-optimum geometry of the matrix/boulder interface. Navin et al[6] also identify matrix bearing capacity failure as a mechanism of cutting in boulder laden ground. The main difference here relates to through going boulder fracturing as a precipitator of matrix shear failure. Disc load eccentricity and inclination, resulting from the disc's position on the boulder cross section, and the lateral prying

Below: Fig 11 - Development of through going boulder fracture





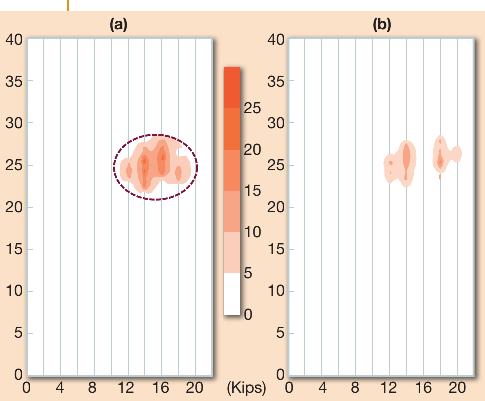

**Top left: Fig 12 - Uniaxial compressive strength basis; Right: Fig 13 - Elastic modulus basis**

action of the disc along the through going fracture surfaces, respectively, can result in reduced bearing capacity (Figure 10c).

The assessment of whether classical disc cutting or boulder plucking will occur is aided by key relationships revealed by the LCM test programme. As a function of boulder to matrix UCS and elastic modulus ratios, there is an increasing tendency for plucking as the disc cutters progressively penetrate the boulder (Figure 11).

As shown in Figure 12, there exists a linear relationship between block to sand grout UCS ratio and the percentage of a boulder cut before through going fractures develop. When the UCS ratio exceeds approximately 600:1, classical disc cutting behaviour and chipping of the boulder are not expected to occur[2]. However, the LCM testing importantly reveals that classical disc cutting behaviour and chipping may not occur even as the UCS ratio approaches unity. As examples: (1) for an UCS ratio of 200, cutting by chipping is expected to occur as the cutters penetrate the first half of a boulder, at which point a through going fracture occurs (causing plucking by matrix bearing capacity or adhesive bond failure);

**Below: Fig 14 – Force contours for block embedded in soft matrix: (a) cutting through a 250mm block; and (b) deep fracturing of same block during subsequent pass (measurements in inches)**




and (2) for an UCS ratio near unity, cutting by chipping is expected to occur as the cutters penetrate the first 70% of a boulder, at which point boulder plucking occurs. Figure 13 shows that a similar linear relationship exists between block to sand grout elastic modulus ratio and the percentage of a boulder cut before plucking develops.

The right-side termini of the lines are not expected to change appreciably. Further research into boulder scale effects is also warranted, although appreciable scale effects were not revealed over the size range considered in the LCM testing programme. Finally, the fourth condition provides an estimate for conditions leading to boulder plucking, as the effect of proximal boulders will be to increase the effective matrix shear strength and therefore decrease the potential for matrix bearing capacity failures.

## Load cell instrumentation

There has been significant recent interest in developing reliable on-board TBM boulder detection systems as a means to mitigate some of the difficulties associated with boulder laden ground. While geophysical imaging techniques such as seismic reflection or radar tomography have been pursued, there are significant complications associated with executing these surveys during TBM drives. Resolution of boulder sized anomalies is extremely difficult, and the interpretation of imaging results is subject to significant uncertainty.

As an alternative, the LCM programme indicates that significant practical utility can be gained by instrumenting each TBM disc cutter with a load cell recording normal forces in real time during cutterhead rotation. For example, Figure 14 shows contours of disc cutter normal forces obtained for the case of a granitic boulder embedded in soft sand grout. In 14a the force contours correspond to the intact boulder dimensions. During a subsequent cutter pass (14b), a gap in force contours evidences deep fracturing of the boulder and that plucking is imminent. The approximate dimensions of the fractured boulder pieces depicted in 14b can also be estimated from force contours.

Disc cutter load cell instrumentation can provide invaluable data related to possible differing site conditions, cutting performance, and face intervention planning. With respect to differing site conditions, real time imaging results can be used to quantify the occurrence and size distributions of boulders actually encountered, which can then be compared to baseline specifications.

The normal force contour plots also provide an unambiguous account of whether boulder cutting behaviour is characterised by classical chip formation, or deep fracturing and plucking. If TBM progress is hindered and face interventions are needed, the normal force contour plots can provide advance notice of where problematic boulder conditions exist. To this extent, it may prove feasible to avoid manned face interventions; breaking down the boulder by drilling through the face and injecting expansive grout in the drill holes; or by grout injection within the matrix surrounding the boulder to increase strength and stiffness, thereby promote disc cutting. T&T

*Please address any questions or queries to Professor Scott Kieffer - Email: kieffer@tugraz.at*

## REFERENCES

1. PB Dowden & RA Robinson, 2001. “Coping with Boulders in Soft Ground Tunneling” Proc. RETC, p961-977.
2. CM Goss, 2000. “FE Modeling of Boulder Excavation in Soft Ground Tunneling by EPB and Slurry Machines using Disc Cutters” PhD Dissertation, Dept of Mining, Colorado School of Mines, Golden, CO.
3. C Leelasukseree, 2006. “Further Understanding of Block-in-Matrix Ground in TBM Tunneling” PhD Dissertation, Dept of Mining, Colorado School of Mines.
4. C Leelasukseree, DS Kieffer & GGW Mustoe, 2006. “Numerical Simulation of Disc Cutter Behavior in Bimrocks” Proc. Amer. Rock Mech Assoc, Golden, CO.
5. SW Hunt & FM Mazhar, 2004. “MTBM and Small TBM experience with Boulders” Proc. NAT, p47-64.
6. SJ Navin, JY Kaneshiro, LJ Stout & GE Korbin, 1995. “The South Bay Tunnel Outfall Project, San Diego, California” RETC Proc. p629-644.
7. J Rostami, 1997. “Development of a force estimation model for rock fragmentation with disc cutters through theoretical modeling and physical measurement of crushed zone pressure” PhD Dissertation, Dept of Mining, Colorado School of Mines, Golden, CO.

# WEBSITE DIRECTORY

# Classified

## RECRUITMENT

# Classified

## RECRUITMENT

International Tunnelling Services
Professionals in tunnelling

**With teamwork to the breakthrough.**

Are you looking for experienced personnel to work on your tunnelling projects?
We are specialised in tunnelling recruitment solutions. We are able to supply qualified personnel of all levels, from back-up labour to project manager.

**Experienced personnel of different nationalities.**

Do you require an efficient, prompt and accurate solution to your recruitment need?
Please contact us to discuss your requirements.
E-mail: info@tunnelcrew.nl

**www.tunnelcrew.nl**

International Tunnelling Services BV
Rivium 2e straat 41b, 2909 LG Capelle aan den IJssel,
The Netherlands Phone: +31 10 266 94 44 Fax: +31 10 266 94 45

**Construction Managers** - Required for Main Contractor for TBM drives and shaft sinking projects in **London & Sussex**. The right candidate must have a good tunnelling background.
**Senior Engineers** - Required for £40m 3m TBM tunnels both in **London & Sussex**.
**Sift Engineers** - Required for large bore tunnels and shaft sinking projects based in and around **London**.
**Environmental Engineer** - Required for Main Contractor to work on Tunnelling projects around **London**.
**Mechanical Engineer** - To work along side Senior Engineers on large TBM drives and shafting sinking projects in central **London**.
**TBM Foreman** - Based in **London & Sussex**. Experience in shaft sinking and large bore tunnelling projects is essential.
**TBM Surveyors** - Based in **London & Sussex**. Experience in monitoring and guiding TBMs is essential.

**Contact: Axis Recruitment (UK) Ltd**
**Tim Laite Tel: 01303 260048 Mob: 07766432531**
**tim@axisjobs.com**

**Do you have a vacancy to fill or some equipment to sell?**
Call: Gary on +44 (0) 208 269 7849
Email: gtween@tunnelsonline.info Fax: +44 (0) 208 269 7840

### UK & OVERSEAS OPPORTUNITIES

Engineering Recruitment

Current opportunities:

- **Principal Tunnelling Engineer** - Birmingham
- **Tunnel M&E Engineer** - Abu Dhabi
- **Geotechnical Engineers - Metro & Piling Projects** - Dubai, Singapore, Hong Kong
- **Senior Geotechnical Engineer** - Delhi India
- **Principal Tunnel Engineers** - Czech Republic, Hungary, Singapore & India
- **ITS Engineer - Tunnels** - Surrey
- **Construction Manager & Senior Tunnel Engineers - Water Project** - London
- **Senior & Section Engineers (Excavated & SCL Tunnel Exp.)** - Surrey
- **TBM Operators (Wirth machine Exp.)** - India
- **Immersed Tunnel Design Engineers** - Abu Dhabi, UAE or UK Based
- **Tunnels Project Directors** - Hong Kong & Surrey
- **Tunnel Engineers, Site & Design** - USA
- **Tunnel/Underground Structure Design Engineers** - UK, Australia, USA & UAE

Tel: **+ 44 (0) 207 754 5999** Email: **careers@v-engineeringrecruitment.com**
Web: **www.v-engineeringrecruitment.com**

## EQUIPMENT FOR SALE

HOCHTIEF
Glendoe Joint Venture

### SITE DEMOBILISATION



- Korfmann GAL 12-550-550 Tunnel Ventilation Fans
- Shotcrete Agitators
- 20, 30 and 50 Minute Self Rescue Units
- Dieci Mini Mixer Trucks L4700
- Formwork
- Streif Rescue Container
- Putzmeister Wet Shotcrete Unit WKM 133
- Grout Pumps
- Tamrock Commando 300
- Boart Longyear Drilling Equipment
- Tamrock Mini 206

and various other equipment.

**Please visit our website**
**www.htglendoe2.com**

**Contact**
**Joanna.Laniewska@htglendoe.com**
**Telephone 01456 459 575**
**Fax 01456 459 501**
Glendoe Road, Fort Augustus, Highlands, UK, PH32 4BZ

# Contacts


HEAD OFFICE
**Progressive Media Markets**
Progressive House
2 Maidstone Road
Sidcup, Kent
DA14 5HZ
United Kingdom

WEB
www.tunnelsonline.info

TEL
+44 20 8269 7789

FAX
+44 20 8269 7840


Tris Thomas

Amanda Foley

Gary Tween

David Wildman

## HOW TO CONTACT US

### EDITORIAL


EDITOR
**Tristan Thomas BSc (Hons)**
Tel: +44 20 8269 7893 Fax: +44 20 8269 7840
Email: tthomas@tunnelsonline.info

DEPUTY EDITOR
**Amanda Foley**
Tel: +44 20 8269 7789 Fax: +44 20 8269 7840
Email: afoley@tunnelsonline.info

CONTRIBUTING EDITOR
**Patrick Reynolds MEng**
Tel: +44 20 8269 7748 Fax: +44 20 8269 7840
Email: patrickjreynolds@hotmail.com

DESIGN
**Natalie Kyne**

TECHNICAL ILLUSTRATOR
**Nick Stenning**

PRODUCTION CONTROLLER
**Loraine Lee**
Tel: +44 20 8269 7799 Fax: +44 20 8269 7840
Email: llee@progressivemediagroup.com


### ADVERTISING


GROUP ADVERTISEMENT MANAGER
**Gary Tween**
Tel: +44 20 8269 7849 Fax: +44 20 8269 7840
Email: gtween@tunnelsonline.info

NORTH AMERICAN MANAGER
**Steven Caming - Granite Media Services**
Tel: +1 603 447 1187 Fax: +1 603 447 6166
Email: granitemediaservices@yahoo.com

EUROPEAN MANAGER
**Randolf Krings**
Tel: +49 611 5324 416 Fax: +49 611 5324 519
Email: t&t@emcmedia.de

ITALIAN OFFICE
**Ediconsult**
Tel: +39 02 477 10036 Fax: +39 02 477 11360
Email: milano@ediconsult.com


### PUBLISHING


PUBLISHING DIRECTOR
**David Wildman**
Tel: +44 20 8269 7778 Fax: +44 20 8269 7840
Email: dwildman@progressivemediagroup.com


## HOW TO SUBSCRIBE

Tel: +44 845 155 1845 7511 Fax: +44 20 8606 7301
Email: progressive@optimabiz.co.uk

**Subscription price** (12 months) mailed anywhere in the world: US$239 / £147 / 220 (UK only £99.00). Claims for missing subscribed copies can only be entertained within three months of publication.

## HOW TO GET BACK ISSUES

Back issues are available from: Optima WDIS Tel: +44 845 155 1845 7511 Fax: +44 20 8606 7301
Email: progressive@optimabiz.co.uk

## HOW TO GET REPRINTS

The contents of T&T is subject to copyright. However, if you would like to obtain copies of an article for marketing purposes high-quality reprints can be supplied to your specification. Please contact the advertising team for full details of this service.

### REPRO/PRINT

Tunnels & Tunnelling International is printed at Broglia Press, Poole, Dorset, United Kingdom




Tunnels & Tunnelling International ISSN number 0041-414X is published monthly for US$226 a year by Progressive Media Markets Ltd, Progressive House, 2 Maidstone Road, Sidcup DA14 5HZ, UK. Periodicals postage paid at Rahway, NJ.

POSTMASTER: send address corrections to Tunnels & Tunnelling International c/o BTB Mailflight Ltd, 365 Blair Rd, Avenel, NJ 07001. US agent: BTB Mailflight Ltd, 365 Blair Rd, Avenel, NJ 07001.


TOUGH ROCK.
TOUGHER TBM.
When a tunnel project appears too challenging, it's a job for Robbins. The industry's most productive equipment, Robbins TBMs have completed for more rock tunnels than all the competition combined. That means you have all the power and ingenuity you need to accomplish any hard rock project.
Move mountains with Robbins.