Durban Harbour tunnel—first use of a slurry tunnel boring machine in South Africa

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Synopsis

Durban Harbour in South Africa is one of the continent's busiest ports. However, the harbour entrance is currently too small for the new generation of container ships. Therefore it is to be deepened and widened by the National Ports Authority in the near future. An existing immersed tube tunnel carrying major sewer, water and other utilities under the harbour first needs to be removed and replaced by a deeper 4.4 m ID services tunnel. Construction of the deeper tunnel saw the first use of a slurry machine in South Africa and was required to negotiate grades up to 20%.

Project description

The city of Durban lies on the eastern coast of South Africa in the KwaZulu-Natal province on the Indian Ocean, and has a population of about 3.3 million. The city is a thriving business and tourist destination. The harbour is in the natural location of Durban Bay with the area of highland called the Bluff on the south side and the city business district to the north (Figure 1). It is the busiest port in Southern Africa, operating 24 hours a day, 365 days a year. The harbour entrance has a navigable channel of 120 m and can accommodate vessels with a maximum draught of about 12 m.

The current harbour entrance configuration, established in the late 1800s, is quite difficult for larger ships to navigate during heavy weather, with a risk that a ship could breach and thereby block the entrance. The harbour is also not able to accommodate the new generation larger ships. Consequently, the National Ports Authority intends to widen the harbour entrance by relocating the breakwater (North Pier) and deepen the channels within the port to 18 m at the ocean side and from about 13 m to 16 m at the port side.

Figure 1—The city of Durban and harbour

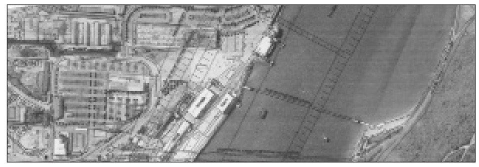

Figure 2—Durban Harbour showing proposed widening of the entrance

At present a number of water, sewer, power and communication utility services are carried across the harbour entrance in an 350 m long, 3.6 m diameter immersed tube tunnel. The major services are twin 1 000 mm diameter wastewater pipelines and a 450 mm diameter potable water pipeline. This tunnel was built in the 1950s and is located at a shallow depth beneath the harbour. Before the widening and deepening of the harbour entrance channel can take place, it is necessary to divert these existing services into a deeper bored tunnel, which requires the first use of a full face slurry tunnel boring machine (TBM) in South Africa.

Once the services have been relocated to the new tunnel, the services in the existing immersed tube tunnel will be stripped out and then the tunnel itself will be removed.

Though initially it was planned to construct a second tunnel using the same TBM and linings to facilitate the installation of a pedestrian crossing or light rail system across

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 This paper was first presented at 2007 Rapid Excavation and Tunneling Conference in Toronto 2007, 10–13 June 2007 and published in the conference proceedings.

the new harbour entrance, it was decided that this tunnel would not be built at this stage.

The lead consultant for the project is Goba (Pty) Ltd of South Africa. The work for the replacement services tunnel has been commissioned by eThekwini Municipality Water and Sanitation Department. Mott MacDonald in the United Kingdom provided specialist advice on the tunnelling for the project.

After a five-month tender period the tunnel contract was awarded to Durban Harbour Tunnel Contractors (a joint venture between Hochtief Construction AG of Germany and the South African construction company Concor).

Though the project originally called for a 4.5 m ID tunnel, it was recognized that to meet the tight construction schedule, a secondhand TBM would be required. Thus to maximize the availability of existing plant, the tender document permitted contractors to offer variants wherein the diameter could be between 4.4 m and 5.0 m ID.

Alignment

The proposed profile of the new harbour entrance largely governs the vertical alignment of the tunnel. However, commercial and residential development constraints on the north side and topographic constraints (the Bluff) meant that deep shafts would be needed for TBM launch and reception. To minimize the depth of the shafts, the position of the tunnel under the shipping channel was as shallow as considered prudent. Studies were carried out to determine the maximum grade that could practically be negotiated by a TBM (and any associated increased tunnelling cost), which could then be compared with the cost of tunnelling at lesser gradients but with increased shaft depths and associated work necessary to bring the services to surface. These studies included research on precedent experience on other projects worldwide and discussions with TBM manufacturers.

This led to the adoption of the unusually steep gradients of 20% on the incline and decline of the tunnel for about 100 m at each side, with a 300 m section at 0.5% grade across the main part of the harbour entrance. The vertical curves between the inclined and subhorizontal sections are on a 300 m radius curve.

There was little opportunity to achieve a shallower gradient on the south side of the harbour, other than deeper shafts, as there was very little flat ground before the terrain rises steeply up the Bluff. Additionally, the position of the TBM reception shaft on the south side of the harbour required sufficient space for a working construction site. Though there was scope for reducing the gradient of the tunnel at the north side of the harbour entrance by increasing the length of the tunnel, this would have disrupted development at this side along Point Road. A shallower gradient was possible on the south side of the harbour but this would have significantly increased the shaft depth required.

Figure 3 shows a section through the harbour channel showing the existing and proposed entrance channels. The cover for the replacement bored tunnel below the existing seabed was chosen as a minimum of 9 m (two tunnel diameters). After the dredging of the deepened harbour channel (after tunnel construction) the minimum cover chosen was 5 m, which occurs at a pinchpoint on the north side of the harbour between the new harbour channel and 20% gradient of the new tunnel. These depths also allow for any over dredging when the harbour widening works take place. Though the removal of the extra cover over the new tunnel would increase the risk of flotation, there was still a sufficient resistance to flotation with the reduced cover.

For the horizontal alignment of the tunnel, the diversion of the existing services across the harbour was from a location clear of the proposed widened harbour at Point Road. Hence, the alignment of the new tunnel is to the west of the existing immersed tube tunnel to pick up the wastewater pipelines where they currently run to the existing tunnel on the north side of the harbour (see Figure 4). On the south side, the pipelines will exit out of the tunnel to tie into the line of the existing pipelines running round the 'Bluff'.

The launching of the TBM was from within a slurry walled shaft on the north side of the harbour and reception on the south side was also into a slurry walled shaft located near an old whaling station slipway.

Geology and setting

Eleven boreholes were drilled as part of the geological and geotechnical investigations, two of which were within the existing shipping channel. Figure 3 shows the idealized geology of the harbour, which is largely Holocene and Pleistocene marine and lagoonal sediments. This is mostly classified as dense fine cohesionless sand with a clay/silt content in the order of 10%. However, lagoonal sediments comprising mostly silt and clay were identified, which could occupy up to 50% of the tunnel face. Approximately 420 m out of the 515 m total tunnel drive would be through these sediments. The strata changes to aeolianite reef sandstone on the south side of the harbour, forming the harder geology of the 'Bluff' and the tunnel drive would pass through this geology for the remainder of the drive. The sandstone on the south side was uniformly grained but heavily laminated with a strength varying between 0.5 and 50 Mpa.

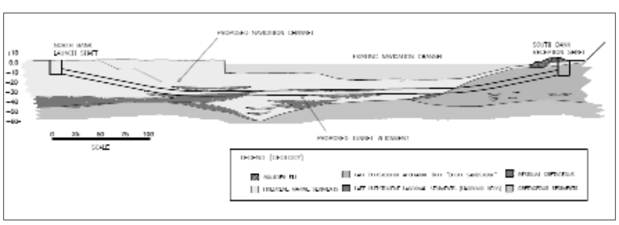

Figure 3—Tunnel long section and geology

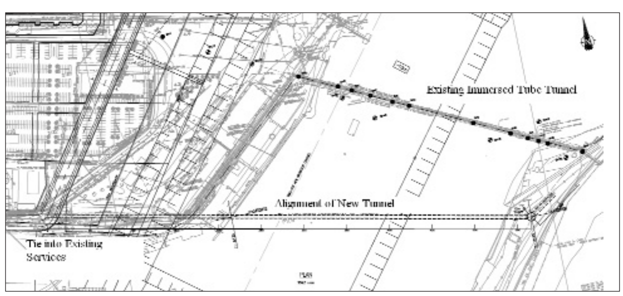

Figure 4—Connection of existing services through new tunnel

A full head of water pressure of around 3.5 bar was also expected.

Design

A feasibility study for the project was undertaken in 2003/2004 and several options for carrying the services across the harbour entrance were evaluated. These included shallow and deep bored tunnels, immersed tube tunnels, microtunnelling and directional drilling. A shallow bored tunnel was selected as the preferred option for the new crossing on a balance of utility and cost.

Detailed design of the permanent works was carried out by Goba (Pty) Ltd of South Africa, in association with Mott MacDonald in London (design of the tunnel lining and advice on tunnelling methods and specifications), Wilson and Pass Inc. in Durban (initial design of the shafts and review of the contractor's shaft designs during implementation), and Drennan Maud and Partners in Durban (geotechnical investigation along the tunnel alignment).

The TBM launch and reception shafts would not form part of the permanent works and so responsibility for the detailed design of the shafts during implementation was that of the contractor. This allowed the contractor to choose a configuration to suit his methods of construction and anticipated loading by the launch and reception of the tunnel boring machine. Jones and Wagener of Johannesburg, South Africa carried out this detailed design on behalf of the Contractor.

The tunnel lining was designed for its ultimate service loading condition by Mott MacDonald. However, the Contractor was responsible for ensuring that the lining design was sufficient to withstand the loads that would be exerted during manufacturing, transport, installation and advance of the TBM. This required considerable liaison between Mott MacDonald in London and Hochtief in Germany.

Tunnel lining design and manufacture

Though a 4.5 m ID tunnel was initially specified, it was known early on in the project that a suitable Herrenknecht mix-shield slurry TBM with a 4.4 m ID lining had been used on the recently completed Kai Tak Transfer Scheme for the West Kowloon Drainage Improvement in Hong Kong. The 4.5 m ID lining design was therefore based on this 4.4 m ID lining, thus allowing for minimal redesign if the Kai Tak lining molds were obtained. Fortunately, all four tenderers for the Durban Harbour tunnel included the Kai Tak TBM in their bids and, as it turned out, both the TBM and lining molds from the Kai Tak project were procured for the Durban Harbour tunnel.

The 4.4 m ID lining was a universal tapered ring, nominally 1200 mm long. There were three ordinary plates, two top plates and a key that could be placed in any one of 16 positions at 22.5° round the tunnel circumference to match the TBM ram spacing and with a taper of 28 mm for a minimum 250 m radius curve.

Shaft design

The North Shaft was designed as a twin cell structure, with a central diaphragm wall to transfer loads exerted by the surrounding ground and external water table. This wall had an opening allowing installation of the TBM shield and first

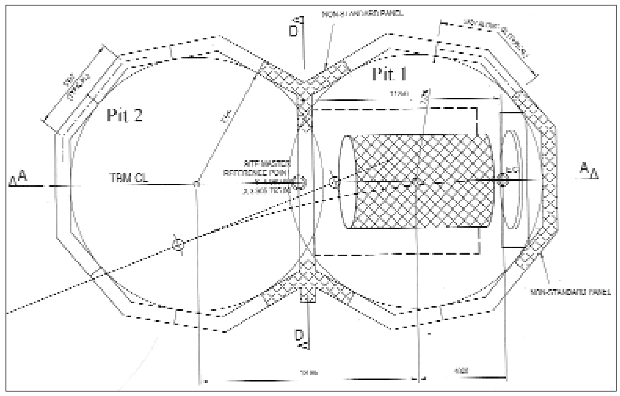

Figure 5—North shaft plan

gantry, to enable launch of the TBM. Pit One was designed to accommodate the TBM support works during tunnelling, while Pit Two allowed the contractor to commence early with construction of the cut and cover civil works, once the TBM and its backup had moved into the tunnel. (see Figure 5).

The jet grouted base plug initially offered by the contractor at the time of tender was changed back to a 3.5 m thick unreinforced mass concrete plug.

The south shaft required a different configuration, incorporating a single cell reception shaft and rectangular inclined shafts to accommodate the cut and cover civil work constructed at an angle to avoid the railway tracks running immediately adjacent to the site as well as the steep rise up the Bluff (see Figure 6).

The launch and reception eyes in the north and south

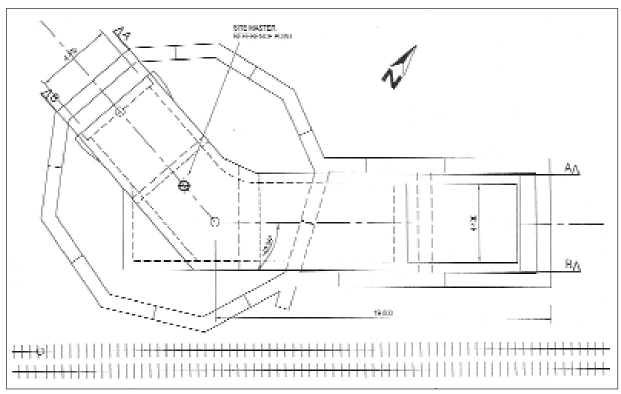

Figure 6—South Shaft plan

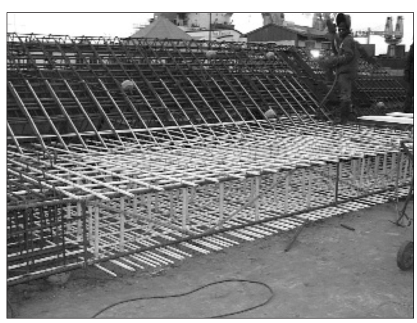

Figure 7—Soft eye formed using glass fibre bars

shafts were constructed using glass fibre reinforcing bars, to enable the TBM to pass through the eye walls without damage to the cutterhead (see Figure 7).

Shaft construction

The shafts were constructed using under-slurry diaphragm wall techniques, using the following procedure:

- Guide walls were constructed at ground level, on the required configuration of the shaft, to ensure the correct alignment of the excavation equipment.
- Alternate primary panels were excavated using a combination of continuous flight auger (CFA) and mechanical grab. Panels were overexcavated by about 0.5 m to ensure fit of the reinforcing cage. All excavations were carried out under bentonite slurry to maintain stability of the sides of the slot beneath the water table.
- Steel stop-ends were installed at each end of the primary panel.
- Reinforcing cages were then lowered into the slot, incorporating 75 mm roller blocks to ensure the correct cover and to allow the cage to move freely down the slot (see Figure 8).
- Concrete was placed from the bottom of the excavation using a tremmie pipe, with an insertion depth of about 500 mm being carefully controlled through constant sounding of the concrete surface.
- Secondary panels were then excavated and cast as infill sections following a similar procedure.

On completion of the diaphragm walls, the shafts were then excavated using a combination of:

- Dry excavation to a point where either the groundwater inflow exceeded pump capacity or the upward pressure of the groundwater resulted in instability of the base of the shaft, whichever occurred sooner
- Underwater dredge pump excavation in softer material
- Underwater mechanical grab excavation
- Suction airlift operated by divers for the final trimming of the base of the excavation prior to casting the base plug.

Figure 8—Shaft diaphragm wall reinforcing panel being lifted into position

Figure 9—TBM established in the north shaft prior to launch

The base plugs were then cast underwater using a high slump pump mix, incorporating an anti-waterlogging admixture to prevent washout and dispersion of the fresh concrete underwater. The casting operation was carried out from a single point using truck mounted concrete pumps, and divers to ensure embedment of the pump line by about 300 mm at all times. Dips taken during the casting process showed good flow of the concrete horizontally, and cores taken after dewatering showed little or no segregation.

After sufficient strength gain on the plugs, the shafts were dewatered. A problem was encountered on the North Shaft shortly after dewatering, with a leak coming from under the base plug of Pit Two through a joint on the intermediate cross panel. The leak caused sand washout and a sinkhole to form outside the shaft adjacent to the tower crane. The shaft was immediately flooded for stability. Extensive grouting underwater from both Pits One and Two was carried out using a combination of cementitious and polyurethane grout before the shaft was again dewatered.

Following completion of the North Shaft and construction of the TBM support works, the TBM was established at the base of the shaft (see Figure 9).

Tunnel construction

The limited space available for the construction site on the south side of the harbour meant that the TBM was launched from the north side with its easier access and sufficient land for the TBM ancillary equipment and segment casting and storage yard.

Tunnel boring machine

It was a requirement of the project that only a slurry TBM was considered for the tunnel excavation. This was due to the ground conditions expected along the tunnel drive and the high water pressure of up to 3.5 bar. Additionally, the removal of excavated material on the 20% slopes by either muck wagon or conveyor would have been difficult with slippage of excavated ground likely using a conveyor method of muck removal. Rail methods of muck removal will have required winches to haul trains up the steep slopes.

Figure 10—TBM cutterhead

The project used the 5.17 m diameter Herrenknecht mix-shield TBM previously used on the Kai Tak Transfer Scheme in Hong Kong. This was the fourth project the TBM had constructed. The TBM was renamed 'Nomfundo' after the Durban mayor's wife.

The cutterhead consisted of 50 scrapers for the sand and clays and 24 disc cutters capable of excavating through the sandstone on the south side of the harbour (See Figure 10).

The TBM operated as a mix-shield slurry TBM with the excavated material being mixed with a bentonite slurry pumped through a pipeline to the surface where it was separated using a screening and sand/mud separation centrifuge plant.

Due to the steep 20% decline and inclines in the tunnel, a trackless system of transport was developed for the operation of the TBM that was specially designed to carry the tunnel segments and grout mix and fit under the backup sledges. This multi-service vehicle was specially designed and fabricated by Techni Métal Systèmes (TMS) in France for the project, and tested using simulated conditions for full load operations on a 20% slope (see Figure 11).

Figure 11—Multi-service vehicle

Performance of TBM and challenges

Limited advance thrust in soft material on the down drive. In the initial stages of the down-grade drive, a combination of the soft material being excavated and the gradient of the drive necessitated low thrust pressures to prevent the TBM 'running away'. Dipping or diving of the cutterhead was also experienced. The consequence was some difficulty in steering the TBM since it was hard to develop the differential thrust needed to keep the front of the shield up. Also the lower thrust on the installed segments gave some problems with ensuring good seating of the rings during the advance.

Thrust ram alignment in the vertical curve. A steep down grade inevitably results in a vertical curve to bring the tunnel back to subhorizontal. One problem encountered here was the angle of the line of thrust from the TBM rams on the leading edge of the built ring. Some damage was picked up on the internal and external peripheries of the leading edge, thought to be due to eccentric loading of the thrust rams with the changed angle as the TBM advanced through the curve. This was solved through the repositioning of the thrust rams and shoes at the midpoint of the drive. The sideways movement of the ram was quite noticeable as soon as the thrust load was released.

Advance rate. An average advance rate of about 12 m per day (or ten rings per day) was originally planned for the tunnelling operation, working on a six-day week, 24 hours per day.

However, problems such as the leak in the North Shaft resulted in a delayed start to tunnelling. This led to the decision by the contractor to change the tunnelling operation to a seven-day week in order to recover. This was achieved using three teams working two 12-hour shifts, on a 10/5 day rotational basis.

With a tunnel of only 515 m length, the challenge would always be to get past the learning curve as quickly as possible. In the case of the Durban Harbour tunnel, this learning curve took place on a 20% down-grade in soft ground conditions—a tall order. The initial rate of advance was slower than originally planned, which persisted through most of the down-grade drive. The rate of advance started to recover when the TBM moved into the subhorizontal drive, but was then affected by encounters with significant quantities of clay, in two main areas, largely as a result of interventions required to clear blockages and problems

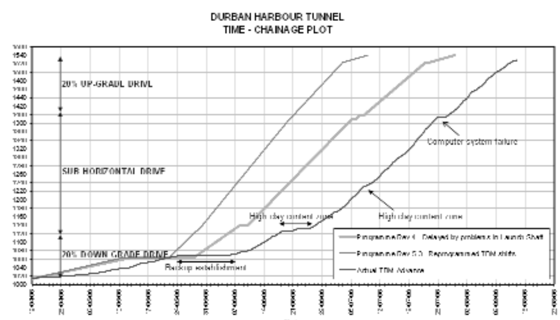

Figure 12—Time-chainage plot

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had been negotiated, the rate of advance through the subhorizontal section and on the 20% up-grade drives were essentially as originally planned, albeit on an extended shift basis (see Figure 12).

Analysis of the total downtime experienced on the TBM drive is summarized in Table I. A significant amount of time was lost because of problems experienced with TBM faults and with the annulus grout system, the latter being mainly a large number of blockages in the grout lines. Segment damage, repair and removal was a significant cause of downtime in the early stages, but this reduced as the teams became familiar with the plant and conditions under which they were operating.

Water inflows occurred on a few occasions, as pressure was exerted on the lining when the ring moved past the rear of the shield, causing the key segment to 'squeeze' out of position, thus compromising the gasket seal. This required additional restraint of the key block to prevent this movement.

Table I

TBM performance analysis

Downtime Description	Total Downtime (hours)
TBM faults	253.75
Ring erector	41
Segment feeder	4.25
Slurry pump	33.25
Slurry pipeline	63
Waste water pump	25.25
Grouting system	246.75
Water supply	1
HP air supply	0
HV electrics	10.5
LV electrics	12
Ventilation	1.5
Multi-service vehicles	6.25
Survey	4.5
Guidance system	0
Segment repair/removal	121
Surface operations	0
Accidents	0
Extend pipelines	13.5
Extend cables	2.5
Control cabin	0
Water iInflow	78.75
Planned maintenance	269.25
Separation plant	41.75
Miscellaneous	140.75
Total downtime hours	**1,370.50**
Total working hours	**2,519.50**
Total production time	**1,149.00**
TBM availability	**46%**

Clay conditions on the tunnel alignment. The geotechnical report based on the investigation carried out along the tunnel alignment indicated the presence of clay lenses intersecting the face of the tunnel over a length of approximately 150 m. While not extensive enough to warrant concern about the selection of the TBM, the clay did cause a number of difficulties during the excavation process.

The excavated material, in this zone, being a very stiff to hard clay, showed a tendency to clog the access chamber behind the cutter head. This blocked the slurry level sensors, and also interfered with the slurry-air interface in this chamber, both of which are crucial to the control of slurry pressure during the drive. This necessitated a number of interventions to clear clay away from the sensors and to unblock flushing lines.

The amount of clay encountered also posed a problem for the separation plant. This was exacerbated by difficulties in determining which combination of flocculent and coagulant would be most suitable for the combination of bentonite and natural clay being encountered.

Annulus grout. On the 20% decline, control of the annulus grout system proved to be absolutely critical. The TBM, being an older machine, was not fitted with a skirt at the rear of the shield to prevent backflow of annulus grout, along the shield. On the down-grade drive, the grout showed an increased tendency to flow forward along the shield.

Problems were experienced with very high thrust pressures being required on the middle part of the down-grade drive, which resulted in a loss of directional control with the inability to develop a differential thrust. This was thought to be due to the presence of annulus grout which had accumulated and hardened on the outside of the shield. Various attempts were made to dislodge this grout, including the fitting of form vibrators to the inside of the shield, which generally proved unsuccessful.

It was also thought that the accumulation of annulus grout on the shield may have blocked the articulation gap, again hampering the ability to articulate the shield and steer effectively. This was partly alleviated by continuous manoeuvring of the shield to remove grout from the articulation joint.

Eventually, an intervention was carried out and the gauge cut was increased by 20 mm to reduce the friction on the shield. This allowed the advance to proceed using normal thrust pressures and with improved directional control.

The main effect of this problem with directional control was a deviation from the designed alignment of some 450 mm horizontally and vertically. While this was out of specification, the decision was made not to force the TBM immediately back onto the designed alignment, as this could have resulted in damage to the segmental lining with high thrust pressures and possible contact of the extrados of the lining with the trailing edge of the shield. Instead, a gradual approach back to the design alignment was adopted, and a correction was made to the VMT guidance system. The principal concern at this stage was with the vertical curve at the bottom of the down-grade, required to bring the tunnel onto the subhorizontal section for the crossing of the harbour channel. However, the TBM was brought onto the correct alignment at the commencement of this vertical curve.

Circumferential joint dowels. The circumferential and radial joints of the precast concrete segmental lining were fitted with dowels and bolts respectively. The circumferential joint dowels were installed using pneumatic spanners immediately prior to erection of each segment, as part of the ring build process.

Problems were initially noticed during the installation of the first rings in the tunnel, with severe spalling around the positions of the dowel sockets. On close inspection of the inserts, once the dowels had been fitted immediately prior to segment erection, hairline cracks were noted in the segment trailing edge face, running through the socket and extending to the inner face of the segment.

On investigation, it was found that the dowels had a nominal diameter greater than that of the inserts. This resulted in compression of the plastic insert on insertion of the dowel, which caused stressing of the concrete around the socket, effectively setting up lines of pretensioning through the segment concrete. During installation of the segments, the slightest misalignment of the segment caused additional stresses on the dowels, which resulted in failure of the concrete around the dowel.

Consideration was given to reaming the inserts to allow the dowels to be installed with less force and to reduce any pretensioning of the concrete. However, the fit of the dowel in the socket is critical to the resistance by the segments to the thrust exerted by the sealing gasket, and to ensuring that the segments remain locked in during installation. Site trials were conducted to test the effect of reaming the sockets on the pullout force of the installed dowel. It was determined that limited reaming would ensure adequate lock-in force and this solution was implemented successfully on site.

Interventions. A total of 12 no interventions were carried out during the tunnel drive. At the start and end of the drive, interventions were carried out to check the condition of the cutterhead after and prior to driving through the shaft walls at the launch and reception pits respectively.

On the down-grade drive, the problems with the accumulation of annulus grout on the tailskin necessitated interventions to investigate possible causes and ultimately to increase the gauge cut.

The remaining interventions were necessitated by the build up of clay around the slurry level sensor and on the slurry/air interface, which disrupted the control of the slurry level and pressure. No interventions were required during excavation of the subhorizontal section or during the upward inclined drive in the sandstone, apart from that required to check the cutterhead condition prior to cutting into the reception shaft.

All interventions were supervised by qualified divers in control of the airlock operation and included in the entry team to work under compressed air conditions.

Slurry treatment plant

The slurry treatment plant was provided by Piggot Shaft Drilling (PSD) from the UK. The principal components of the package separation plant consisted of:

- One SD600DP declined deck primary shaker (removal of oversized solids and clayballs)
- Two SM450PSDP desander/desilting units (separation of coarse particles including sands)
- Two S3 centrifuges (fine particle separation).

The main plant was capable of operating at a capacity of approximately 600 m^3/h, with the twin S3 centrifuges able to operate at around 15 m^3/h each.

The process also incorporated a combination of flocculants and coagulants to enhance the separation of fine particles. Some problems were experienced with the reaction of the flocculant with the natural clays and various chemicals were tested before achieving the most favorable combination.

The clay encountered on the tunnel alignment caused some problems with the separation plant, with the capacity of the hydrocyclones on the desanding units being exceeded, resulting in overtopping. The amount of clay being dispersed into the slurry also resulted in high mud weights, which required extensive cleaning through the separation process.

Tunnel lining production

The manufacture of the lining segments took place on site close to the North Shaft (see Figure 13). With the limited production requirement for the short length of tunnel, and the generally favourable weather in Durban, the use of an indoor production facility was not felt to be warranted. However, portable screens were provided to protect the molds from sun, wind and rain. The molds were retrofitted with form vibrators to assist with compaction of the concrete.

Prior to the commencement of production of segments, a check was carried out on the molds using a three-dimensional photogrammetric digital survey. From this survey, digital models of each mold were produced to 0.1 mm accuracy to confirm compliance with the required accuracy specified.

Extensive trials were carried out on site to determine the optimum consistency/slump for the specified 50 MPa mix, and to obtain the best sequence of casting, opening the molds and floating. Some problems were experienced with cracking of the extrados surface, thought to be due to the additional working of the concrete to achieve a steel float finish. The decision was taken to allow a wood float finish to the extrados, which reduced the incidents of cracking. A possible effect of this change was increased wear on the wire tailskin seals, but the contractor felt that this would not pose a problem over the short length of tunnel.

Steel reinforcing cages amounting to 465 kg per ring were preassembled in the segment manufacture area. Cover to the steel was 50 mm.

Segments were demolded at approximately 18 hours using a vacuum lifting device, and removed to a finishing yard where they were stored on specially fabricated steel frames (see Figure 14). At this stage minor repairs and crack sealing were carried out, and the EPDM Phoenix gasket was fitted. The segments were then moved to a storage area, and stacked in complete rings. Each segment was also given a

Figure 13—View along tunnel alignment showing segment production area

Figure 14—Typical segment in finishing yard

unique code, identifying the date of casting, the segment type and the ring number. This code was logged through the life of the segment, including all pre- and post-installation repairs.

During the tunnelling operation, a day-yard was established, where complete approved rings were stacked ready for installation on the multi-service vehicle for transport to the TBM. At this stage the secondary hydrophilic gasket was fitted to the segments, and all dowel inserts were drilled out as required following the site trials previously described.

Removal of existing immersed tube tunnel

The existing/immersed tube tunnel is approximately 310 m long, and comprises a 3.6 m ID reinforced concrete pipe with a 610 mm wall thickness. Removal has yet to be carried out, but the following possible methods of removal have been identified:

- A reversal of the original immersed tube installation method, involving installation of internal watertight bulkheads, exposure and cutting at original joint positions, adding buoyancy via airbags, and lifting and towing to the approved disposal area out at sea.
- Flooding and cutting the tunnel into lengths that can be lifted and handled by the equipment available in Durban, followed by marine transportation to the approved disposal area.
- Breaking up the tunnel *in situ*, and removal by grab dredger or similar to barges for marine disposal.

At present it is considered that the second option is the more practical as follows:

- Sand that has accumulated either side of the immersed tube tunnel pipe is removed by dredger to the level of the new channel. The tunnel was originally positioned on concrete pedestals spaced approximately 66 m apart prior to backfilling. A suction pipe will be used to remove the sand bedding under the tunnel such that it is left, as far as possible, standing only on the cradles.
- Buoyancy bags each capable of lifting 5 tons are available in Durban. A 20 m length of tunnel, which has a submerged weight of about 40 tons, would thus require eight or so bags for flotation without the need to position bulkheads for additional internal buoyancy.
- Each section of tunnel between pedestals will be cut into 20 m sections leaving a short section remaining on the pedestal. A diamond wire cut or a disc saw could be used and both are expected to cut through the reinforcing steel without problem. Wire saw cutting would be from the outside while disc saw cutting would be from the inside out.
- Holes will be punched or drilled through the tunnel crown, and chains would then be attached to a strong-back inside the tunnel and to the buoyancy bags above the section. The bags would then be inflated one by one until the section lifts clear of the channel bed.
- The tunnel sections will then be towed out to sea for disposal. Alternatively, they can be beached in the harbour and broken up into smaller sections, which could be dumped in landfill or used for new sea wall protection.
- The short section remaining on the concrete pedestals and the pedestals themselves can be drilled through and lifted with bags in a similar fashion or they can be cut into smaller pieces with a top down wire cut if required.

Notwithstanding the above, selection of the preferred method of removal will be left to the successful tenderer on the Durban Harbour entrance widening and deepening contract. Whichever method is selected, a critical aspect will be to ensure that the operation does not result in any disruption to the navigable depth and width of the demarcated navigation channel.

Summary and conclusions

The Durban Harbour Tunnel Project was the first use of a slurry TBM in South Africa and demonstrated how the difficulties of tunnelling on a steep gradient of 20% were able to be overcome. The successful completion of the tunnel means that the proposed widening of the harbour entrance can now proceed as planned.

The paper discusses tunnel design and construction issues of harbour crossings and tunnelling with closed face tunnelling machines on steep gradients. It is rare for tunnels with such steep gradients to be constructed by TBMs with segmental lining and therefore will be of interest to tunnel designers, owners and precast segment manufacturers. With the arrival of larger container ships, the issue of harbour widening to accommodate these ships and diversion of services will be of relevance internationally. It may also become increasingly necessary to negotiate steep gradients with closed face tunnelling machines as underground space in urban environments becomes more constrained.

Acknowledgements

The authors would like to thank Ethekwini Municipality Water and Sanitation Department for their permission to publish this paper. Thanks also to Yogini Vimalanathan of Mott MacDonald for her helpful review comments

This paper was presented at the Society for Mining and Metallurgy and Exploration, 2007 Rapid Excavation and Tunneling Conference in Toronto 2007, 10–13 June 2007, and published in the Conference Proceedings. ◆