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# Discussion: Hull wastewater flow transfer tunnel: tunnel collapse and causation investigation

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A. D. M. Penman, *Geotechnical Engineering Consultant*
A crucial sentence is, 'If a pathway was established through the laminated clay, water under pressure from the lower aquifer could find its way into the tunnel.' It seems likely that such a pathway would have been caused by the blowing of the bottom of the shaft, disrupting the strata around the shaft in the region where the collapse occured. Slight movements of the tunnel lining could open the unbolted joints, allowing the water and sand to flow into the tunnel. Such conditions could not have been predicted from the initial site investigation during design of the tunnel and shaft.

A similar problem occurred during construction of the Hull barrage in the cofferdam for one of the two towers. The sand floor had been prepared by suction, ensuring that the water level in the cofferdam was always higher than that of the river outside. The contractor generously invited the engineers to visit the cleaned floor in a small diving bell. They found a thin layer of soft sand covering the surface, and said it must be taken out to avoid subsequent settlement. Ready-mix concrete was already on its way, so the man holding the suction pipe was asked to quickly suck off this thin layer. He had previously done his best and knew that whatever he did he could not prevent some sand falling back to produce this thin layer. He dashed at the task and quite overlooked the need for balance of water levels. The result was a major blow of the bottom sand. In the panic the contractor decided the best thing was to get the concrete in to hold down any further inflow. Pipes were left in the upper concrete so that the loose sand under the concrete could be grouted to give a strong foundation. This clearly has worked, because the barrage has remained stable. This case shows clearly how easy it is to allow damaging inflow to occur when excavating a shaft in water-bearing sand.

The accident during construction of shaft T3 might have alerted the designers to the risk of disturbed ground around the shaft, knowing that it was to be passed through by the tunnel, but the details of the event may not have been readily available, and certainly from the point of view of the original design this tunnel failure could not have been predicted.

Authors' reply
The authors would like to thank Dr Penman for his interest in the paper and his description of the Hull Barrage cofferdam incident, which is itself a noteworthy and salutary tale. Disturbance of the ground during the excavation of shaft T3 is certainly one of the possible reasons for a pathway for sand to flow through the laminated clay layer.

D. J. Hartwell, *Groundwater Management Consultant*
The paper highlights the unusual and perhaps unique soil conditions prevailing in the area of the tunnel collapse, but largely ignores the impact of the annulus grouting around the outside of the segmental lining. This would appear to be significant, as we are told that the grouting was delayed by 7–10 days (section 9·2) and that tunnelling in the area occurred 'about 2 weeks earlier' than the collapse (section 2). These timings, presumably related to the tunnel boring machine (TBM) maintenance stoppage at the shaft, could suggest that annulus grouting might have been a causation worthy of more detailed consideration. Could the authors confirm that the annulus grouting took place systematically at the tailskin, in the conventional manner, as the TBM advanced?

The case for annular grouting causing tunnel displacement is not unknown, and the natural buoyancy of all but very small-diameter tunnels is obvious. Some 15 years ago a short section of tunnel lining on the Mersey estuary pollution alleviation scheme (MEPAS) in Liverpool was rebuilt to correct the vertical alignment, which was too high: the subsequent annular grouting resulted in the lining floating upwards. The case for a dense fluid under pressure exerting higher uplift forces is a matter of simple calculation.

This phenomenon has been studied in some detail by Bezuijen in The Netherlands.[4] Bezuijen studied grouting pressures and measured the movements immediately behind a TBM for the Sophia tunnel in The Netherlands. Rapid vertical upward movements of 15–20 mm were measured.

For the Hull tunnel, with one end fixed at the shaft and the possible grouting delays, a model where there is displacement some distance from the shaft and curvature induced in the tunnel seems feasible. The resulting tension in the bottom of the tunnel lining, leading to opening of joints in the lining, is consistent with the initial leakage flows observed. This situation may well have been aggravated by the removal of bolts at about the same time ('removed four days prior to the collapse', section 2), allowing the circle joints to open near the invert.

The comment that the initial inflow was 'black and foul smelling' (section 3) is mysterious, although the subsequent collapse mechanism is obviously more clearly understood.

It would be interesting to know more about the grouting, and in particular constituents, density, timing, pressure and volumes, to understand whether this is possibly a mechanism that needs to be considered more widely by the tunnelling industry.

Similar problems might have been avoided elsewhere, in the past, by the use of rapid-gelling thixotropic grouts. The placing of annular grout at the tailskin immediately behind the brush seals is clearly of paramount importance, so that no voids or low pressures are created to allow movement of potentially unstable ground.

### Authors' reply

There was a 7 – 10 day delay between annulus excavation and grouting of the rings in the location of the collapse. This delay was due to maintenance of the TBM as it passed through the shaft. The grouting did take place systematically, but the grout was not placed until the ring had emerged from the tailcan (R2404/R2403 in Fig. 2).

The three-dimensional numerical modelling and the centrifuge testing both considered the possible influence of the softening effect of the tunnel excavation on the soil surrounding the tunnel.

During tunnelling the grout was placed until a pressure limit of 2·5 bar or a maximum volume of 1600 l was reached (the amount contained within the grout car). The theoretical grout volume in the annular void was 1070 l. Grout was injected in the shoulders, relying on the pump pressure and fluidity to penetrate around the lining.

During the recovery works for reconstruction of the tunnel there was an opportunity to exhume the collapsed section of the tunnel and investigate the ground conditions, albeit very disturbed, and recover the collapsed segments. The segments were examined to assess the amount of grout on the back of them, and this confirmed that the rings adjacent to the shaft appeared to be fully grouted, all round the ring.

The initial flow of black and organic-smelling water probably came from the crown of the tunnel, and was from water that was in contact with the organic clay-peat at the crown of the tunnel (Fig. 6). This water was noticed by the locomotive driver as he passed over the leaking ring. The very first small leaks were observed as clear water at the knee joints.

### J. N. Shirlaw, *Golder Associates (Singapore) Pte Ltd*

The authors suggest that the sand that washed into the tunnel appeared to be derived from the aeolian sand, which they identify as having been about 1·4 m below the tunnel invert. So the issues are:

(*a*) How did the aeolian sand get from 1·4 m below the invert to the tunnel?
(*b*) How did the sand enter the tunnel?

Based on the earlier *Geotechnical Engineering* paper by Machon and Stevens,[5] the TBM had a diameter of 4·226 m, although it is not clear whether this was the diameter of the skin or the excavated diameter. The diameter of the extrados of the lining was 4·100 m, so there was an average annular space of at least 63 mm around the lining. The authors identify a 7 – 10 day delay in the grouting of this annulus, and state that this could have resulted in 'softening of any clay materials'.

The authors summarise the results of a pumping test. They state that the glacial sands and gravels below the aeolian sand showed a tidal range of about 2 m. Presumably the aeolian sand was directly connected to this aquifer, recharged by the River Humber. The water pressure in the sand would have been in excess of 2 bar.

The authors give little information on the geotechnical properties of the various strata. Did they check whether the high differential hydraulic head between the aeolian sand and the void around the lining could have resulted in hydraulic fracturing of the laminated clay, thus bringing water and sand into the ungrouted tail void?

Once the sand had entered the tail void, the next question is how it got into the tunnel. Machon and Stevens[5] state that a single ethylene propylene diene monomer (EPDM) gasket was provided for each of the segments. No details of the gasket dimensions and gasket groove are provided in either paper. Typical EPDM gaskets can be compressed by about 5 mm per gasket: that is, up to 10 mm of gasket compression considering the compression of two meeting gaskets.

For sand to enter the tunnel there must have been a physical gap between the gaskets. With a $D_{85}$ for the sand estimated as about 0·3 mm, the gap would need to be over 1 mm to allow quantities of sand to enter the tunnel.

The EPDM gaskets that have been used on projects that I have worked on typically required a force to achieve full compression of between 40 and 80 kN/m of gasket, depending on the gasket. The circle joint at Hull would have been about 12·6 m in length, so the force required would have been between 500 and 1000 kN. Perhaps the authors could provide the relevant information for the gaskets used at Hull.

The force in the gaskets on the circle joint is normally maintained by the shove rams. With a typical face pressure of 2 bar, over 2500 kN of thrust would be required just to balance the face pressure, so under normal circumstances the gaskets should have been fully compressed.

From Fig. 2 of the paper, it was at the joint between rings 2403 and 2404 that the leak was first noticed. When shoving off ring 2404, the head of the TBM would have been within the shaft, there would have been no face pressure, and relatively little propulsion thrust would have been required. In between shoves there would have been little or no force on the lining from the TBM. Although the authors state that the 'erection and bolting procedure essentially ensured that the joints were tightly closed', I would question whether, in the absence of significant thrust from the TBM, there would have been sufficient force to compress the gasket fully, and maintain it compressed. Was the erector arm capable of providing sufficient force to compress the gasket fully on the circle joint, and was there sufficient reaction to maintain that force at this location?

Based on the 7–10 day delay in the grouting quoted by the authors, and the subsequent inflow, it is likely that the void around the rings had filled with water and loose sand before the void was grouted. It would be useful to know more about the grouting, and in particular whether each segment was grouted, the sequence of grouting, and whether any water and/or sand was encountered in preparing for the grouting. If the grouting was done into a void partially filled with sand and water, the grouting would have compressed, but not expelled, the sand around the ring.

The authors identify relative movement between the rings at the point of inflow and those in the 'unmoving' shaft, and talk about 'fixity' of the tunnel at the shaft. The difference between the cut diameter and the lining diameter that produces the tail void would still be present at the shaft, so can it be taken that the rings in the immediate area of the shaft were grouted into the shaft wall much earlier than the other rings were grouted? If no form of seal was provided at the shaft wall, there would have been a continuous void around the rings that extended to the shaft, and ground and groundwater would have had an open path into the shaft; nor would there have been fixity of the tunnel at the shaft.

If the rings in the shaft wall were grouted or otherwise fixed in position, then the potential for relative movement between these rings and the adjacent, ungrouted, rings is evident. The authors have concentrated on possible geotechnical causes for relative movement. In particular they mention the potential for softening and/or consolidation of the organic clay layer at the crown of the tunnel in providing the potential for the 12 mm of differential upward movement they estimate necessary to have caused the problem. With a tail void that was left ungrouted for up to 10 days after construction, surely there was plenty of scope for upward movement of the lining under buoyant conditions, without needing to consider the compression of the organic clay layer?

### Authors' reply

Our responses are set out generally in the order in which they are presented by Mr Shirlaw.

The TBM cut diameter was approximately 4·255 m. This diameter was produced by the gauge cutters located at the head of the machine. The 4·226 m diameter quoted in the paper was determined by the beading on the leading edge off the TBM. Therefore the theoretical annulus would have been approximately 78 mm rather than the 63 mm as described by Mr Shirlaw.

The rings close to the shaft that were not immediately grouted would have been located within the tailcan, with one ring left ungrouted for between 7 and 10 days inside the can. This delay was due to maintenance of the TBM as it passed through the shaft, but the ring was then grouted once the TBM recommenced its tunnelling work. There could have been local softening of the clay materials around this ring.

The aeolian sand does appear to be hydraulically connected to the aquifer, which itself is connected to the River Humber. The water pressure in the sands was at approximately 200 $kN/m^2$ (2 bar).

To meet the restrictions on the length of the paper we were unable to provide information on the geotechnical properties of the various strata. However, historical geotechnical disturbances of the laminated clay were considered in our causation investigation. The laminated clay was at the base of a deep valley that was subsequently infilled. Extruded U102 samples in this material show that laminations were distorted or orientated at up to 45° to the core axis. Various scenarios were postulated: one of these was disturbance due to underlying artesian water pressure during periods of historical erosion. Tests were also carried out to assess the dispersive nature of the laminated clay, but they were non-conclusive. Nearby boreholes indicated that towards the base of the laminated clay there were thin beds (up to 70 mm thick) of fine to medium sand.

Hydraulic fracturing of the laminated clay was not directly considered, but other aspects of the clay were considered.

It is not known when the sand first entered the annulus, if in fact it did. Sand washed into the tunnel at the time of the collapse, which was about 2 weeks after the TBM had passed through the seat of the subsequent collapse. The rings that were not immediately grouted were within the tailcan of the TBM; consequently the size of the annulus would have been far less than 78 mm as presented above.

The grouting mechanism comprised a Keller pumping system interlocked with the main rams of the TBM. The pumping system (pumping a 15:1 pulverised fuel ash/cement grout mix) was fitted with both volume and pressure monitors, and these were interlocked with the TBM so that the pilot was told to stop the TBM advance if the grout pump was not filling the annulus sufficiently quickly (volume control) or if the pressure of the grout in the annulus dropped below or exceeded preset levels (pressure control). The pump operating capacity was far in excess of the rate at which the TBM could produce an annulus volume during an advance phase. The exhumation records of the collapsed area for the recovery phase of the tunnel construction show that all the rings recovered were grouted.

The gasket was a Heinke CS003 EPDM type, placed centrally within the thickness of the ring, and required a thrust of 35 kN/m to close the joint fully (to achieve concrete-to-concrete contact). The CS003 gasket gives a factor of safety of 3 against groundwater pressure ingress at the maximum allowed horizontal misalignment between the gaskets. Ring building used an erector arm with a ball and claw handling fixing to close the joint fully. The gasket preload was subsequently held by spear bolts connecting the ring under construction to the previously built rings. For a six-plate ring this would have required a 70 kN load per segment to close the circle joint fully. The rings did not rely on the face pressure at the TBM head to provide gasket closure. The closure force calculation suggested by Mr Shirlaw is not therefore relevant to the failure mechanism.

The tunnel rings were fully grouted through the advances into the shaft. A Bullflex seal arrangement at the shaft interface closed off the annulus from the shaft and ensured that full grouting pressure in the annulus could be achieved right into the shaft portal structure, thus providing the fixity referred to by the authors.

It was considered possible that the buoyancy effect and tidal variations caused repeated cyclic movement of the tunnel within the ground, particularly with reference to the compressible organic layer located above the crown of the tunnel, even with a fully grouted annulus. This repetitive movement may have created both a path for the sand to travel towards the tunnel and also caused damage to the lining itself and potentially compromised the gasket.

## REFERENCE


4. BEZUIJEN A. and TALMON A. M. Grout properties and their influence on backfill grouting. *Proceedings of the 5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam*, 2005.
5. MACHON A. and STEVENS S. Wastewater flow transfer tunnel: design and construction. *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, 2004, **157**, No. 3, 101–106.