Tunnelling and Underground Space Technology 25 (2010) 811–818

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

Technical note

# Excavation failure during micro-tunneling in fine sands: A case study

J. Jebelli [a,1], M.A. Meguid [b,*], M.K. Sedghinejad [c]

[a] 12th Floor, 455 Rene-Lévesque West Blvd. SNC-Lavalin International, Montreal, QC, Canada
[b] Civil Engineering and Applied Mechanics, McGill University, 817 Sherbrooke Street West, Montreal, Quebec, Canada H3A 2K6
[c] Loshan Manufacturing, Suite 10, 1496 Valiasr Avenue, Tehran, Iran

ARTICLE INFO

*Article history:*
Received 8 November 2009
Received in revised form 25 May 2010
Accepted 26 May 2010
Available online 17 June 2010

*Keywords:*
Pipejacking
Micro-tunneling
Sandy soil
Face failure
Shafts

ABSTRACT

Partly funded by the World Bank, a new gravity sewer line is currently being constructed in Anzali, Iran using micro-tunneling methods. The project includes the installation of 2551 m reinforced concrete pipes with diameter ranging from 600 to 1000 mm at an average depth of 5 m below surface. Micro-tunnel Boring Machine (MTBM) and hydraulic pipejacking have been used to install the sewer line. Pipejacking in saturated highly porous sandy soil poses various challenges during construction including the risk of face failure, possibility of shaft collapse, massive rush of groundwater (in this case from the Caspian Sea) and surface subsidence. This paper provides an overview of the project and summarizes the challenges faced and the techniques used to handle the difficulties encountered during construction.


## 1. Introduction

A sewer line project partly funded by the World Bank is currently being constructed under the supervision of SNC-Lavalin International in Anzali, Iran. The project started in May 2008 and included the installation of 2551 m reinforced concrete pipes with diameter ranging from 500 to 1000 mm at an average depth of 5 m using Micro-tunnel Boring Machine (MTBM) and hydraulic pipejacking. The jacked pipes are part of the sewer network that collects the sewage flow of Anzali city to be transported to the pumping station to be lifted to the existing Anzali city sewage treatment plant.

Micro-tunneling in saturated cohesionless soil has led to partial soil loss due to short stand-up time and high groundwater pressure. Therefore, temporary front support such as bentonite slurry is used to maintain the stability of the working face. The use of bentonite slurry develops a layer of filter-cake on the micro-tunnel front acting as a membrane and inhibiting diffusion of the suspension into ground. The supporting action results from the difference in hydrostatic pressure, $\Delta\rho$, between the slurry and the groundwater (Anagnostou and Kovari, 1996).

To prevent seepage flow towards the excavation face, the slurry pressure must be higher than the groundwater pressure. Muller-Kirchenbauer (1972) and Xanthakos (1979) highlighted the fact that the stabilizing force of slurry depends on the extent of the slurry diffusion into the soil. The less the slurry diffuses, the greater the extent of supporting forces. Obviously, raising the excess slurry pressure in order to stabilize the excavation face could cause an increase in safety, but only to the extent that it keeps the pressure in equilibrium at the excavation front (Anagnostou and Kovari, 1996). The objective of this article is to present an overview of this case study and discuss the challenges faced during construction including excavation face failure, surface subsidence and shaft failure. In addition, various other technical challenges encountered in the process of pipejacking such as dealing with intensive water disposal and the adverse effects on groundwater will also be discussed.

## 2. Soil characteristics

Initial site investigation classifies the soil as poorly graded sand (SP) using the Unified Soil Classification system. Table 1 presents the borehole record of a typical soil profile up to a depth of 6 m below surface. The soil consists of loose to very loose poorly graded sand with water table located at approximately 2 m below surface. Natural and submerged specific gravity of the soil are measured as $\gamma_{wt} = 20.6\ \text{kN/m}^3$ and $\gamma_b = 10.6\ \text{kN/m}^3$, respectively. Fig. 1 shows the grain size distribution for two sand samples located at 1.5 m and 4.5 m below surface. The uniformity coefficient of the samples ranged from 3.2 to 4.4 with no fines. The direct shear tests conducted on selected samples indicated an angle of friction $\phi$ of 27° with no cohesion ($c = 0.0\ \text{kN/m}^2$).

* Corresponding author. Tel.: +1 514 398 1537; fax: +1 514 398 7361.
*E-mail addresses:* jjebellie@yahoo.com (J. Jebelli), mohamed.meguid@mcgill.ca (M.A. Meguid), sedghi@loshan.com (M.K. Sedghinejad).
[1] Tel.: +1 514 393 3900; fax: +1 514 393 3901.

0886-7798/$ - see front matter © 2010 Elsevier Ltd. All rights reserved.
doi:10.1016/j.tust.2010.05.006

**Table 1**
Soil characteristics.

| Identification | | | | | Classification | | | | | | | | Density | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample | | Depth | Description | Symbol log | Particle size distribution % passing | | | | Atterberg, limits Per % | | | % Nat moisture | Bulk g/cm$^3$ | Dry g/cm$^3$ | SP-GR g/cm$^3$ | S.P.T blows per 30 cm |
| No. | Type | m. | | | 76 mm | 4.8 mm | 75 $\Gamma$ | 2 $\Gamma$ | LL | PL | PI | $W$ | $\gamma_{wt}$ | $\gamma_d$ | $\gamma_s$ | $N$ |
| 0201 | ● | 1.5<br>2.0 | Gr. very loose poorly graded SAND SP | | 100 | 96 | 3 | | | | | 25.6 | | | | 3 |
| 0202 | ● | 4.5<br>5.0 | Gr. loose poorly, graded SAND with shell SP | | 100 | 100 | 4 | | | | | | | | | 8 |

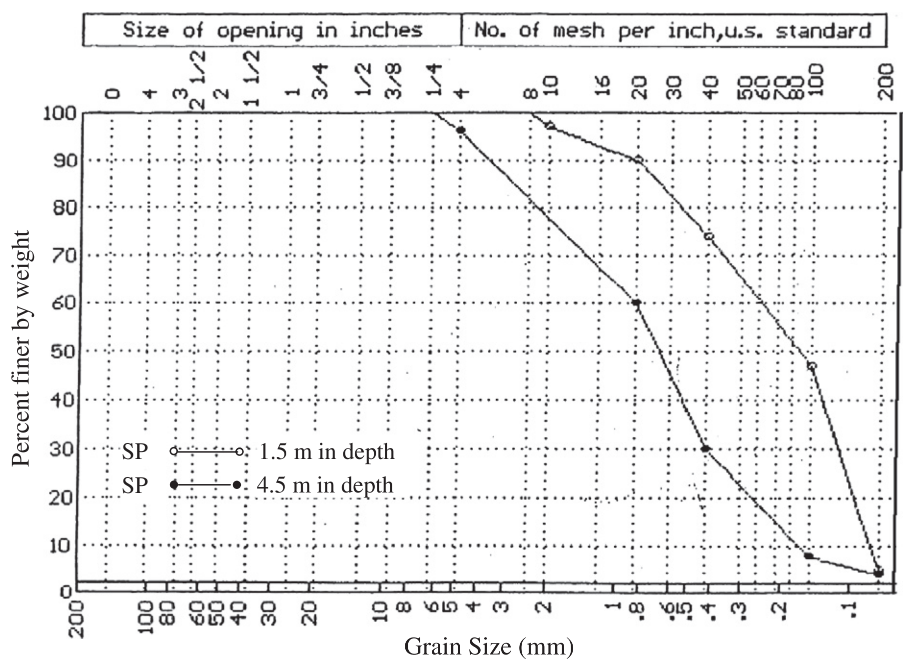



**Fig. 1.** Grain size distribution of the sand.

## 3. Construction of shafts

Temporary shafts are required to launch and retrieve the MTBM before and after the jacking process. Micro-tunneling proceeds from a jacking shaft to a reception shaft. About 42 shafts are to be constructed at the rim of streets each converting to a functional manhole after the completion of pertinent pipejacking drive. Each shaft is constructed by sinking a concrete cylinder as the soil inside is removed by an excavation machine. For the first few shaft rings, the surrounding soils were cement grouted using 0.91 m diameter by 6 m deep boreholes distributed around the shaft area. The main purpose of the pre-grouting was to stabilize the walls and foundation of the excavation during construction. To construct each shaft the seeping groundwater had to be continually pumped out without interruption. After construction of each manhole, the gap between manhole and the shaft is filled using a suitable backfill material. A reinforced concrete thrust block is then built inside the shaft to provide support for the jacking force. The block is oriented such that it is perpendicular to the direction of the pipejacking to prevent any possible deviation from the proposed alignment.

## 4. Construction of pipes

Micro-tunneling pipes are generally subject to large transient axial jacking forces applied during the installation process to advance the pipe. Therefore, selection of the pipe material plays an important role in the constructability of the pipejacking project. The proposed C25 reinforced concrete pipe accommodates both helical and longitudinal reinforcements. Helical reinforcements resist live and dead loads whereas the longitudinal reinforcement provides resistance against the forces imposed by hydraulic jacking. Reinforcements are made of $A_{III}$ rods (yield tensile strength

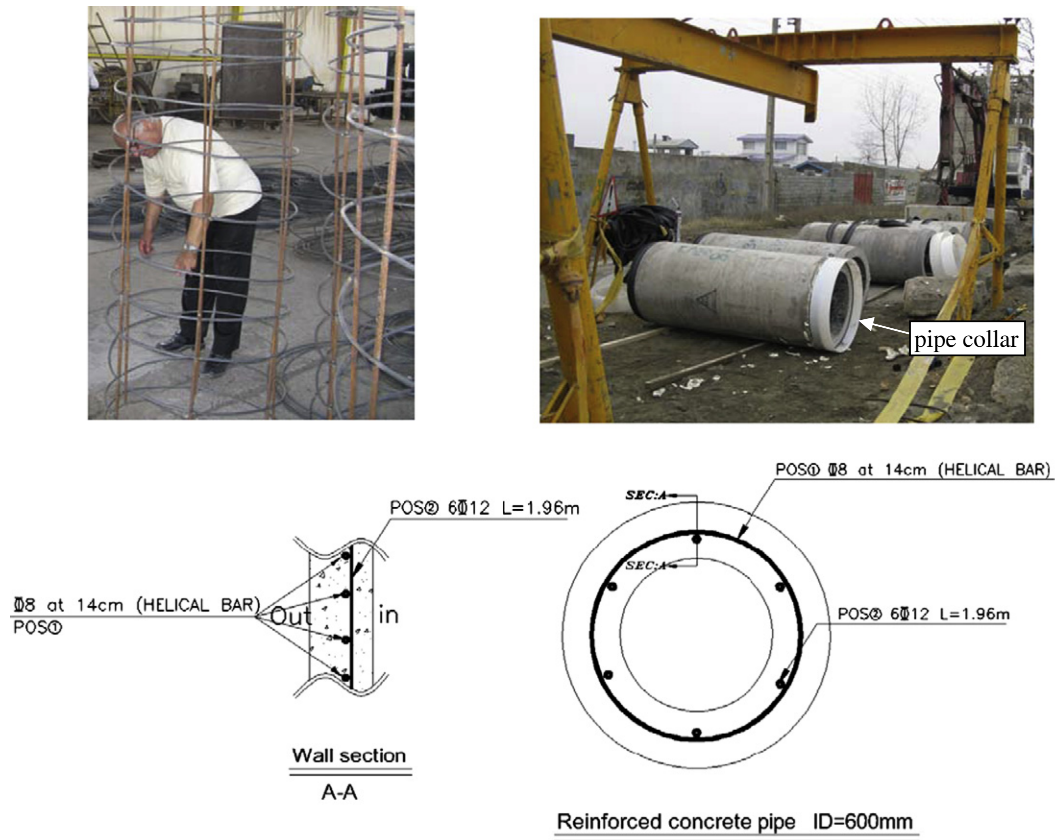



**Fig. 2.** Details of the reinforced concrete pipe.

of 4000 kg/cm$^2$) with diameters and arrangements as shown in Fig. 2. The pipes are constructed from self-compacted concrete that has been prepared in the factory and transported to the site. Utilizing self-compacted wet concrete would allow less water consumption with fewer pores leading to a dense and smooth concrete surface. The maximum thrust imposed by the hydraulic jack on the 600 mm pipe is about 250 tons. The recommended thickness for the 600 mm pipe is 106 mm according to the International Pipe Jacking Association (PJA, 1995).

As shown in Fig. 2, the pipe collars are made of polyethylene (PE) material and the interior is coated using 1.5–2 m thick HDPE layer. The space between two consecutive shafts varies between 60 and 80 m to accommodate primary design perspectives and the necessary site adjustments. After the first few drives the proposed shaft distances have been reduced to about 60 m to facilitate the control of any undesirable soil conditions. The longitudinal profile of the sewer line and the pipejacking shafts are shown in Fig. 3.

The 2-m length reinforced concrete pipes are designed with a safety factor of 4 to bear the maximum axial jacking force. The pipes are also inspected in the factory by performing standard three-edge crush tests on selected pipes. The joints are designed with outer smooth edges to minimize friction forces and at the same time remain water tight under loading with allowable deflection angle of 1°. Pipes are also hydrostatically tested in the factory for 2 bars under the maximum allowable deflection angle.

### 4.1. Joint sealer

The pipe joints are water-sealed using a special rubber gasket made of natural substance (Caoutchoue) that is resistant to Ozone ray. This water proofing elastic rubber can bear up to 5° deflection according to the British Standards (BS 2494, 1990). The maximum pressure imposed on the joints is less than 23.5 N/mm$^2$. Under a maximum bearing pressure of 23.5 N/mm$^2$ the elastic return approaches 56% of its original length. The hourly compression of the rubber sealer amounts to 50% of the primary 8 mm thickness. As Fig. 4 illustrates, the joint sealer covers the edges of two adjacent pipes at their connection point. The gap between two pipes varies from 6 to 15 mm (proportional to the pipejacking forces). The grooves between the two adjacent pipes are filled with Polyorton sealer. A standard hydrostatic leaking test verifies the water tightness of joints after completion of each pipejacking drive. Each completed drive is expected to prove leaks less than 0.15 L per unit wetted area under 5 m head of water for a period of half an hour

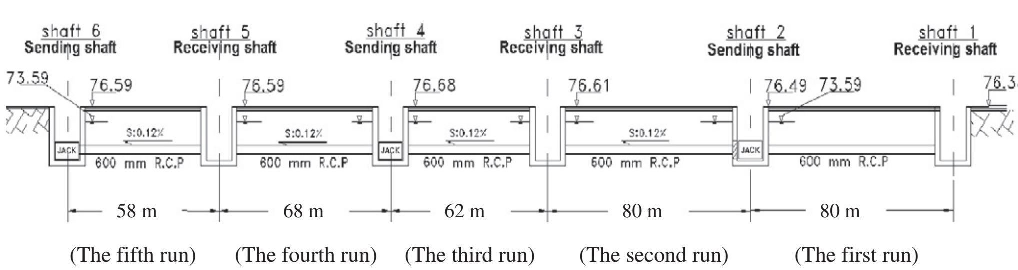



**Fig. 3.** Longitudinal profile of the sewer line.

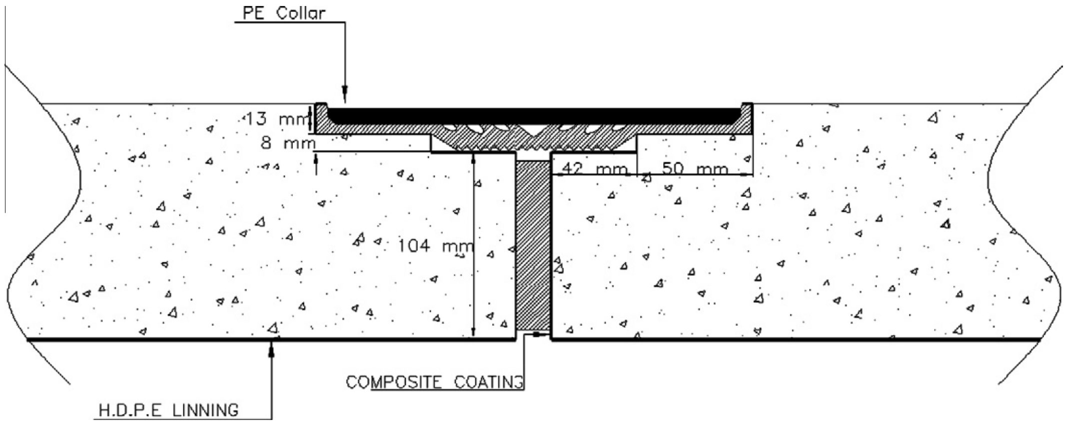



**Fig. 4.** Details of the pipe connection.

(0.15 L/m$^2$/0.5 h). The leak test is carried out after complete dewatering of the two adjacent shafts.

## 4.2. Pipejacking

Jacking pipes require prior construction of at least three consecutive shafts to accommodate the MTBM cutter head in the middle one (sending shaft) and to jack pipes to either side towards the receiving shaft. A full face MTBM with slurry pressure has been employed in this project to perform the excavation. A laser beam apparatus targeted at a moving sensitive plate attached to the back of the shield, dispatches the alignment signals to the control room. The collected data provides information to the operator in order to maneuver the steering cylinder. Each concrete pipe is fed successively into the hydraulic jack carrier and is pushed into the ground using the transfer carrier jack. The micro-tunneling technique utilizes bentonite drilling mud to maintain the equilibrium pressure between the excavation face and groundwater in addition to facilitating the collection of the excavated material. The excavated material including the bentonite mud flows to a sedimentation tank and the deposited sand is then removed from the tank by an excavator. The speed of micro-tunneling, volume of the excavated materials, water feed rates, bentonite feed rates, and cutter head pressure are continuously measured and controlled by the operator.

**Table 2**
Slurry characteristics.

| Tests | Results at 20° | Test method |
|---|---|---|
| Density | <1.1 g/cm$^3$ | Mud density balance |
| Viscosity | 0.02 Pa/s | March cone method (Fann viscometer) |
| Shear test (the gel resistance in 10 min) | 4–40 Pa or 1.4–10 Pa | Shearometer |
| pH | 9.5–12 | The pH strips |

To ease jacking loads on pipes, a lubricant is used throughout the pipejacking process. The lubrication system consists of a mixing tank, sedimentation tank and a pump to convey the pipejacking fluid from the holding tank to the application points at the rear of the cutter head. The fluid thickness has been measured on daily

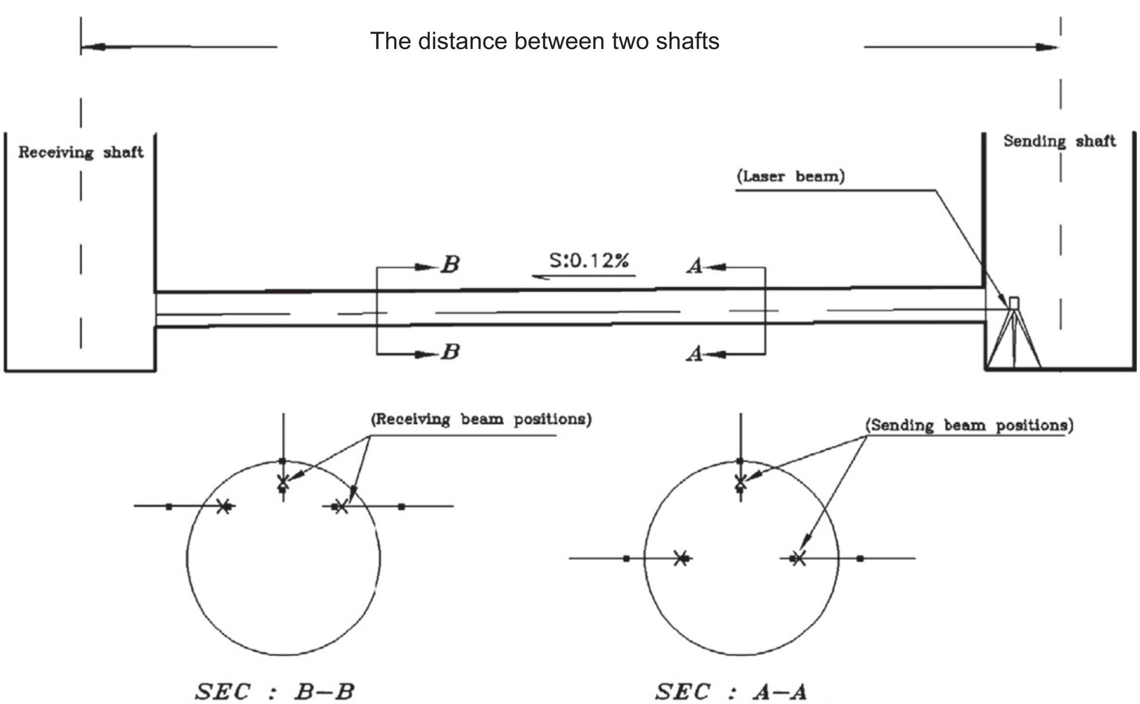



**Fig. 5.** Pipejacking alignment using a laser beam apparatus.

basis. The results of the density, viscosity, shear and pH tests are reported in Table 2.

Ground surface elevation and the pipe behavior during and after pipejacking have been continuously inspected. Unexpected horizontal or vertical deflection is examined using a laser beam apparatus as shown in Fig. 5. A marked plate installed in the receiving shaft allows four laser beams dispatched from the adjacent sending shaft. Laser beam deviation from the marked locations on the receiving plate is measured in order to detect and fix possible alignment problems.

## 5. Observations and discussions

### 5.1. Excavation face failure

Face stability of excavations made in saturated sand with an internal angle of friction as low as 27° and zero cohesion relies mainly on the slurry pressure, $P_s$. Slurry pressure is a temporal variable that is a function of ground reaction pressure, $P_r$, and resultant excess slurry pressure $\Delta p$. Accordingly, it can be expressed as:

$$P_s f(P_r, \Delta p, t) \tag{1}$$

$$\frac{\partial P_s}{\partial t} = \frac{\partial P_r}{\partial t} + \frac{\partial \Delta p}{\partial t} \tag{2}$$

Integrating Eq. (2) for a specific time interval $t_2$–$t_1$:

$$\int_{t1}^{t2} \frac{\partial P_s}{\partial t} = \int_{t1}^{t2} \frac{\partial P_r}{\partial t} + \int_{t1}^{t2} \frac{\partial \Delta p}{\partial t} \tag{3}$$

Given that the boundary conditions in Eq. (3) denote the pipejacking starting time ($t_1$) and ending time ($t_2$) at 8:00 AM and 18:00 PM, respectively it yields:

$$\int_{t2}^{t1} P_s = \int_{t1}^{t2} P_r + \int_{t1}^{t2} \Delta p \tag{4}$$

$$(P_{s(t2)} - P_{s(t1)}) = (P_{r(t2)} - P_{r(t1)}) + (\Delta p_{(t2)} - \Delta p_{(t1)}) \tag{5}$$

Considering that, the average pressure values are the only available data from the pipejacking daily logs (Table 3) therefore:

$$P_s = P_r + \Delta p \tag{6}$$

The allowable excess slurry pressure in Eq. (6), $\Delta p$, is estimated using the following expression (Anagnostou and Kovari, 1996):

$$\Delta p = \frac{2\,\tau_f\, e_{\max}}{d_{10}} \tag{7}$$

where $\tau_f$ is the slurry yield strength, $e_{\max}$ is the allowable slurry diffusion distance in soil and $d_{10}$ is the characteristic grain diameter. Substituting for $\Delta p$ in Eq. (6) yields:

$$P_s = P_r + \frac{2\,\tau_f\, e_{\max}}{d_{10}} \tag{8}$$

**Table 3**
The measured slurry and ground pressure in an average drive.

| Date | Starting time | Finishing time | Average slurry pressure (Bar) | Average ground reaction pressure (Bar) |
|---|---|---|---|---|
| 27/1/2009 | 8:00 AM | 18:30 | 0.15 | 0.35 |
| 28/1/2009 | 8:00 AM | 18:30 | 0.25 | 0.30 |
| 29/1/2009 | 8:00 AM | 19:00 | 0.25 | 0.35 |
| 30/1/2009 | 8:00 AM | 17:00 | 0.25 | 0.30 |
| 31/1/2009 | 8:00 AM | 18:30 | 0.25 | 0.30 |
| 1/2/2009 | 8:00 AM | 18:30 | 0.20 | 0.35 |
| Average | | | 0.225 | 0.325 |

Based on the collected data for an average drive $\tau_f$ = 0.01 kPa, $e_{\max}$ = 100 mm and $d_{10}$ = 0.2 mm thus the excess slurry pressure is calculated as $\Delta p$ = 10 kPa. It is worth mentioning that the yield strength $\tau_f$ of the suspension depends essentially on the bentonite concentration. The extent of slurry diffusion is also governed by fine soil particle fraction. The calculated excess pressure, $\Delta p$, defines the theoretical bearable soil pressure before failure. Under negligible diffusion distance ($e_{\max}$ < 100 mm), slurry acts essentially as a membrane.

According to the simplified model of pressure equilibrium shown in Fig. 6, slurry exerts an average pressure of 0.225 Bar ($P_s$ = 22.5 kPa) at the excavation face whereas soil reacts with average pressure of about 0.325 Bar ($P_r$ = 32.5 kPa). Therefore, based on Eq. (8):

$$\text{If } P_s > P_r + \frac{2\,\tau_f\, e_{\max}}{d_{10}} \text{ slurry may diffuse into the ground} \tag{9}$$

$$\text{and if } P_s < P_r + \frac{2\,\tau_f\, e_{\max}}{d_{10}} \text{ the soil will be unstable with a risk of face failure} \tag{10}$$

Fig. 7 shows the measured slurry pressures versus ground reaction pressures (based on Table 3). Assuming a negligible slurry diffusion ($e_{\max}$ < 100 mm), the bisect line ($P_s = P_r$) divides the graph into two zones. Above the bisect lies the area where the ground reaction pressure, $P_r$, is greater than the slurry pressure, $P_s$, thus the soil may become unstable and the collapse mechanism demonstrated in Fig. 8 may develop. Points that fall within the marked strip located immediately above the bisect line (Fig. 7) are considered to be in equilibrium and the excavation face remains stable. Situations may arise where points are located below the bisect line with slurry pressure that is more than the ground pressure. These cases correspond to "Higher Potential for Diffusion of Slurry" zone presenting another possibility for soil failure.

In all cases where the face pressure crosses the safe "Stable" boundary due to either lack of sufficient thrust or deficiency in slurry pressure some occasional collapse may develop creating a sinkhole that extends to the ground surface, as shown in Fig. 8. Although such degenerations remained local and manageable in this project, in the first few drives it caused ground subsidence at the street level. The measured slurry pressure, cutter head torque and extracted soil volume of an average micro-tunneling drive are summarized in Table 4. The normalized pressure ($P_s/P_r$), normalized torque ($T/J$) and soil volume ($V/V_0$) have been also calculated using the jacking force, $J$, and unit length of the micro-tunnel volume, $V_0$.

Fig. 9 shows the normalized slurry pressure and MTBM torque versus normalized extracted soil volume. This figure demonstrates the inverse trend of normalized slurry pressure versus normalized

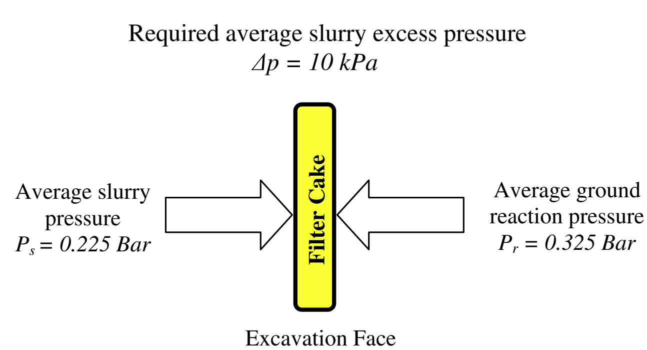



**Fig. 6.** Simplified pressure equilibrium at the excavation face.

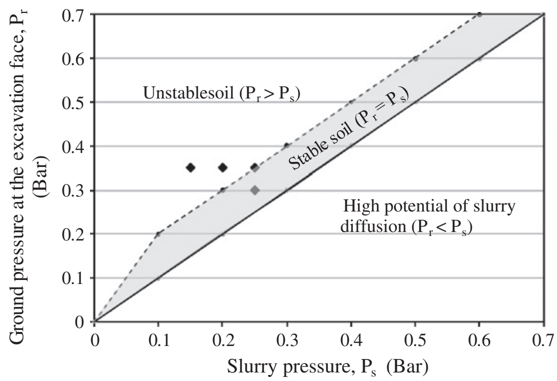
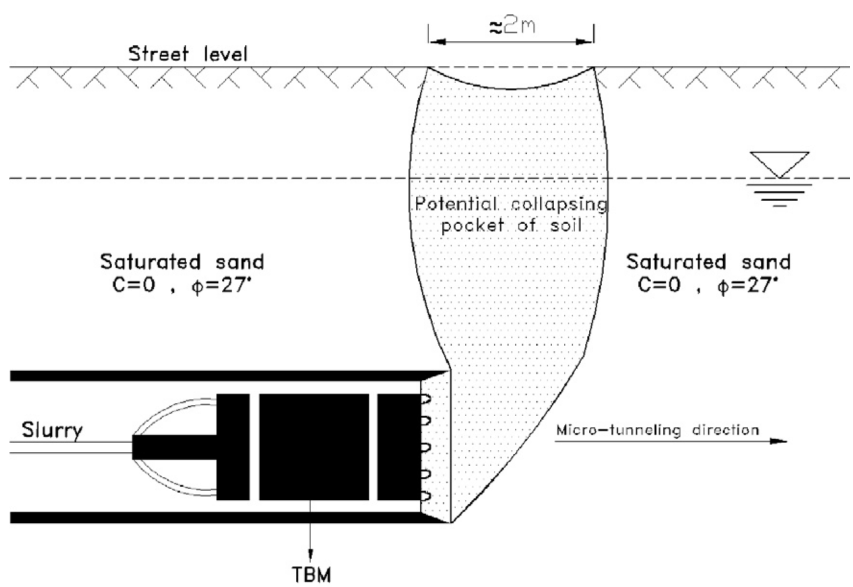



**Fig. 7.** Stability of the excavation face.

extracted soil volume. The trend implies that as the slurry pressure at excavation face decreases the normalized extracted soil volume increases. To avoid face failure the $V/V_0$ ratio should constantly approach unity during the course of pipejacking ($V/V_0 = 1$). The normalized extracted soil volume is also positively proportional to the increase in values of normalized MTBM torque. These trends explicitly indicate that the optimized slurry pressure and MTBM torque play an important role in maintaining stable excavation face at ideal $V/V_0 = 1$ zone, where there will not be any soil over-extraction.

To prevent face failure and saturated sand from escaping through the slurry circulation, the following corrective measures were implemented in the work procedure:

- The cutter head rotational torque and speed was proportionally increased to achieve pressure equilibrium at the excavation face and hence reduce the risk of soil collapse.
- The bentonite feed to the system was adjusted such that it is less than 50 $m^3$/h to control the volume of the transported material.
- The slurry concentration at the recovery tank was continuously kept at about 40 kg bentonite per cubic meter of water to safeguard the continuity of operation.
- Two out of 5 cutter head openings were closed to minimize the risk of soil over-extraction by cutter head openings.
- The advanced detection of possible cavities below pavement was adopted by passing over a 40 tons loaded truck after the completion of each drive.
- The real time data acquisition of sand concentration in the sedimentation tank became quite evident to comply with a very short stand-up time at the excavation front.




(a) Failure mechanism

(b) Street subsidence

**Fig. 8.** Failure mechanism of the excavation face.

**Table 4**
The normalized pressures and soil volumes.

| Measured values | | | | | Normalized values | | |
|---|---|---|---|---|---|---|---|
| Average slurry pressure $P_s$ (Bar) | Average ground reaction pressure $P_r$ (Bar) | Average MTBM torque $T$ (Bar) | Average jacking force $J$ (Bar) | Soil extracted volume $V$ ($m^3$) | (Average slurry pressure)/(ground reaction pressure) ($P_s/P_r$) | (Average MTBM torque)/(Jack force) ($T/J$) | (Average extracted soil volume)/(micro-tunnel volume) ($V/V_0$) |
| 0.15 | 0.35 | 100 | 55 | 5.8 | 0.43 | 1.8 | 1.9 |
| 0.25 | 0.30 | 110 | 77.5 | 9.0 | 0.83 | 1.4 | 1.1 |
| 0.25 | 0.35 | 100 | 100 | 1.0 | 0.71 | 1.0 | 1.0 |
| 0.25 | 0.30 | 110 | 110 | 1.5 | 0.83 | 1.0 | 1.5 |
| 0.25 | 0.30 | 100 | 155 | 10.5 | 0.83 | 0.6 | 1.3 |
| 0.20 | 0.35 | 100 | 220 | 4.5 | 0.57 | 0.5 | 0.7 |

### 5.2. Shaft failure

Primary assessment of soil data showed that due to short stand-up time the cement grouting of sub-soil at 6 m depth can lead to the creation of solid pockets and stabilize the surrounding soil around the shafts during the course of excavation. This was intended to prevent the collapse of walls into the excavated shafts. In the first few shafts the cement grout was injected in a series of 75 mm diameter by 6 m deep boreholes. However, observations showed no indications of the anticipated cement pockets in the soil and yet the walls were unstable. This was explained by the possible diffusion of cement mortar into groundwater. Therefore, the injection of cement grouting in other shafts was abandoned.

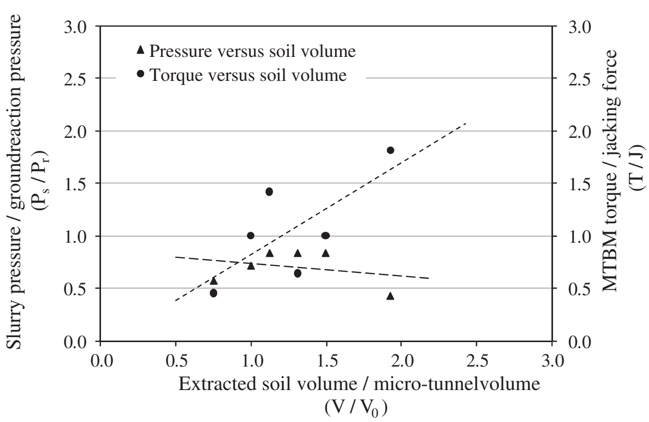



**Fig. 9.** Relationship between normalized slurry pressure, applied torque and normalized soil volume.

The average local groundwater depth along the sewer line is about 3 m below surface. At any time interval at least three consecutive shafts must be continuously dewatered so that the pipejacking and/or construction of manholes could be carried out uninterrupted. Closeness to the Caspian Sea caused massive volumes of water to infiltrate through the permeable soil layers towards the pumping shafts causing real trouble for the effluent disposal. To dispose of the pumping water, a series of sequential backward shafts (manholes in the case of constructed ones) had to be utilized in order to approach the Caspian Sea for possible disposal.

### 5.3. Groundwater effects

Groundwater condition has significant effects on the soil behavior during excavation and the success of the pipejacking process. Dewatering the accumulated water in shafts causes a depression cone around the discharging points. Consequently, as shown in Fig. 10 the pipejacking route crosses both wet and dry zones created by the depression cones as the jacking pipes move towards the receiving shaft. The fluctuating soil moisture content along the pipejacking route necessitates real time control of the MTBM operation. In some instances, the untimely response to groundwater fluctuations has caused horizontal or vertical alignment deviation in the last few jacked pipes. To control the groundwater effect

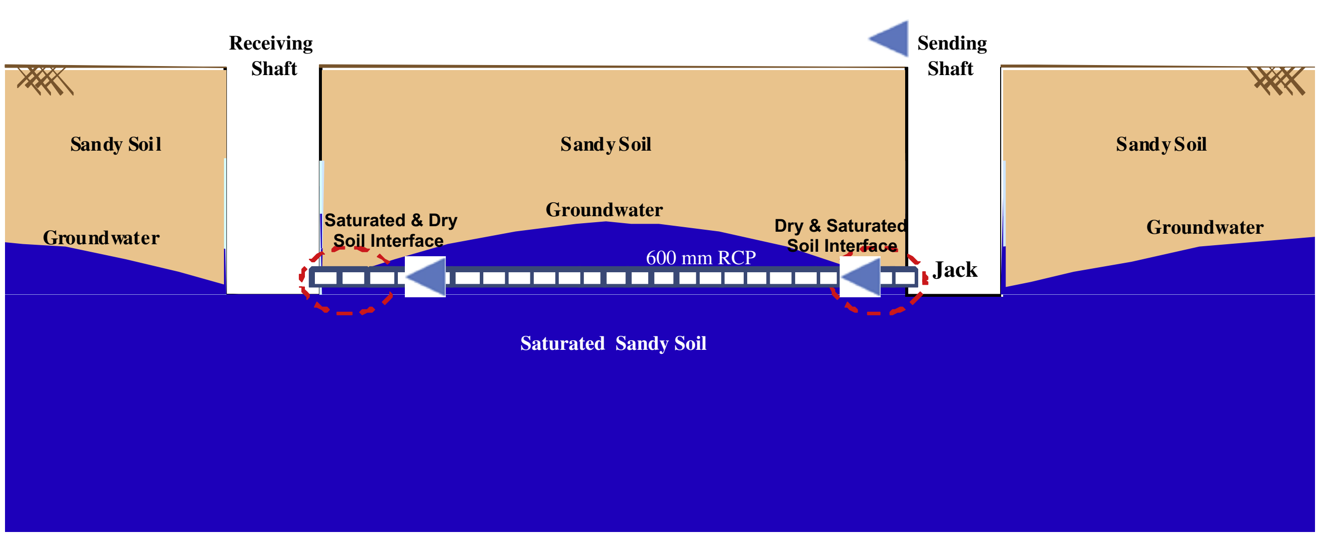



**Fig. 10.** Schematic view of the groundwater effects during pipejacking.

along the pipejacking route, closely spaced well points had to be implemented along the pipejacking alignment in order to allow earlier and continuous monitoring of groundwater prior to pipejacking (ASCE, 1998).

## 6. Summary and conclusions

The case study of a new sewer installation using micro-tunneling in saturated sandy soil is presented in this paper. The project involved the installation of about 2.5 km of reinforced concrete pipes with diameters ranging from 600 to 1000 mm at an average depth of 5 m below surface. Micro-tunnel Boring Machine (MTBM) and hydraulic pipejacking have been used to install the sewer line. Several problems have been encountered during the construction process including face instability, shaft failure and groundwater related issues. Description of the subsurface conditions and construction technique, and the criterion used for the face stability of the excavation are summarized. In addition the measures taken to control the above problems are also described. Great care has to be taken in similar micro-tunnelling projects to minimize the adverse effects of short stand-up time of the soil and seepage pressure on the stability of the required shaft and tunnel excavations.

## Acknowledgements

This is to acknowledge the valuable contributions that were provided by Mr. Pierre Beaudoin the project director and Mr. Raymond Bousquet the SNC-lavalin's senior geotechnical expert.

## References

Anagnostou, G., Kovari, K., 1996. In: Mair, Taylor (Eds.), Face Stability in Slurry and EPB Shield Tunneling. Geotechnical Aspects of Underground Construction in Soft Ground. Belkema, Rotterdam, pp. 453–458.

ASCE, 1998. Standard Construction Guidelines for Micro-Tunneling. Committee Ballot, Revision 7, Reston, Virginia 20191-4400, p. 45.

BS 2494, 1990. Specification for Elastomeric Seals for Joints in Pipework and Pipelines. British Standards Institution, 20 p.

Muller-Kirchenbauer, H., 1972. Stability of slurry trenches. In: Proc. 5th Europe Conf. SMFE, Madrid, vol. I. pp. 543–553.

Pipe Jacking Association (PJA), 1995. An Introduction to Pipe Jacking and Microtunnelling Design. PJA, London, UK.

Xanthakos, P., 1979. Slurry Walls. McGraw-Hill, New York. p. 662.