Low Input Techniques for the Installation of Sewer Lines in Saturated Sandy Soil: A Case Study

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ABSTRACT

This paper discusses the low input methods used in the construction of large diameter sewer lines in cohesionless soil near the Caspian Sea in Iran. Partially funded by the World Bank, the main features of the project include the installation of 77 km of 200 to 1200 mm diameter polyethylene and glass reinforced pipes as well as two conventional treatment plants covering 252,000 inhabitants in year 2029. Installation of the sewer along the Caspian Sea coast lines, characterized by the presence of saturated sandy soil and water flow, posed various challenges during the course of construction. Challenges include the risk of sudden collapse of excavated trenches, possibility of collapse of machineries, risk of street subsidence and the threat of damage to the neighboring properties. Given the required extended length of pipe lying and the high cost of utilizing high technique facilities, the use of low input simple techniques to stabilize the soil and proceed with the secure pipe installation was vital. This paper provides an overview of the challenges and the nature of the adopted low input techniques used to handle the encountered problems.

KEYWORDS: Low input techniques, sewer pipes, support excavation, earth pressure, seepage, grouting

INTRODUCTION

Lack of wastewater collection and treatment along the coast line of Caspian Sea and the Anzali Lagoon in northern Iran is a major environmental concern. These two vital hydrological entities are presently under immense environmental stress from the discharge of untreated sewage from its neighboring cities and village dwellers. Sewage from the two cities of Rasht and Anzali as well as many other rural villages is conventionally directed to the Caspian Sea and Anzali Lagoon intensifying the ever increasing environmental pollution threat in the area. Moreover, due to the high groundwater level, the untreated sewage has lead to groundwater contamination causing several infectious diseases (The World Bank Report, 2005). In view of these conditions and the associated health hazard implementation of a sewage collection system and treatment plant has become a high priority. Soon after the inception of the framework of the project, construction of a new sewer network partly funded by the World Bank was initiated in May 2008. The construction work continued under the supervision of SNCLAVALIN International that included the installation of 77 km of 200 to 1200 mm diameter polyethylene (PE) and glass reinforced (GR) pipes as well as two modern treatment plants. The project would cover 20,440 household connections and 252,000 inhabitants in year 2029.

Installation of sewer lines along the coast of the Caspian Sea which is dominated by cohesionless sandy soil and high groundwater level posed various challenges during the course of the project. Construction challenges include the risk of sudden collapse of excavated trenches, street subsidence and damage to the neighboring properties. Given the extended length of the pipeline and the high cost associated with utilizing high end facilities, the use of low input techniques that can achieve the project objectives and at the same time keep the project cost manageable was very vital (Hansen, 2006). This paper provides an overview of the challenges and the nature of the locally adopted techniques used to handle the encountered problems.

SOIL CHARACTERISTICS

Initial site investigation classifies the soil as sand (SP) using the Unified Soil Classification System. Table 1 presents a typical soil profile from the ground surface up to a depth of 10 m. The soil consists of loose to very loose poorly graded sand with groundwater table located at a depth of approximately 1.5 m below ground surface. Fig. 1 shows the grain size distribution obtained for three soil samples located at 1.5, 4.5 and 7.5 m below surface. The direct shear tests conducted on the soil samples indicated an angle of friction of $\phi = 28°$ and a cohesion of $c = 0.0\ kN/m^2$. The in-situ SPT test results showed an increasing N-value with depth ranging from 10 to 28 blows per 30 cm. The presence of cohesionless sand coupled with a relatively high water table indicates that the soil in the area is likely to be unstable during trench excavation.

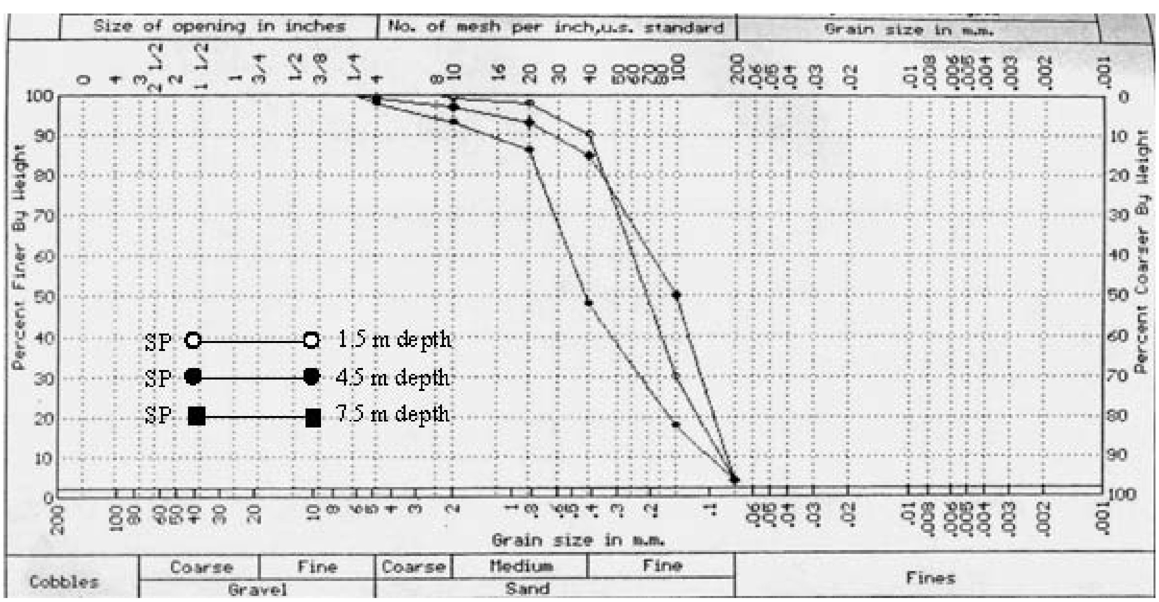


Figure 1: Grain size distribution

Table 1: Borehole record showing typical soil profile

Identification					Classification								Density			Strength				
Sample		Depth	Description	Symbol Log	Particle Size Distribution (% Passing)				Aterberg Limits (%)			%Nat Moist	Bulk Dens g/cm^3	Dry Dens g/cm^3	SP Grav g/cm^3	SPT Blows per 30 cm	Triaxial Test (Deg& kg/cm^2)		Effective Stress Test (Deg& kg/cm^2)	
No.	Type	m			75 mm	4.8 mm	75 mic	2 mic	LL	PL	PI	W	γ_{wt}	γ_d	γ_s	N	φ	c	φ'	c'
		-0.75																		
1601	●	-1.5	Gr. loose poorly graded sand SP		100	98	2									10				
1602	●	-4.5	Gr. medium poorly graded sand SP		100	100	3									17				
1603	●	-7.5	Gr. medium poorly graded sand SP		100	99	4					17.9	2.09	1.77	2.66	19			28	0.0
1604	●	-9.0	As above													28				

Figure 2: In-trench dewatering of excavated strips a) Installation of the initial trench box

PRE-DEWATERING OF THE SITE

Excavation initially commenced with no dewatering provision in place hoping to find away to install the pipes before water gets into the excavated trench. However, closeness to the Caspian Sea caused massive volumes of water to infiltrate through the permeable sand towards the excavated trenches causing significant construction difficulties. Several suction pumps had to be continuously used to dewater the trench so that the construction of manholes or installation of sewer pipes could be carried. It was realized during construction that the saturated sandy soil is unstable and excavation cannot be performed without the application of prior stabilization technique, such as soil freezing, grouting or dewatering. Soil freezing could not be employed in this project due to the high cost and lack of qualified expertise. A pilot test on the application of a viable grouting technique also failed because of inefficiency of ordinary grout injection in the existing cohesionless soil which required special treatment. Therefore, the utilization of pre-dewatering system along the excavated strips was unavoidable (Ohio EPA DSWIM, 2005).

Due to lack of space in the congested streets, open pumping technique using drains and ditches could not be utilized. Instead, a concise pre-drainage technique which involved the installation of an efficient dewatering system (typically located outside of the excavated area) implemented prior to excavation was recommended. The three considered methods of pre-drainage included deep wells, well-points and ejectors. Given the fact that deep well systems would be usually suitable for construction sites of limited areas, well-point system was deemed appropriate for dewatering in these long sandy strips with high permeability soil. However, contractors argued that local experience with well-point systems was not successful due to overwhelming flow of loose sand towards the wells even with protecting well screens in place. Therefore the chosen dewatering practice turned into using in-trench sump pump approach (illustrated in Fig. 2), which proved to be fairly successful.

EXCAVATION SUPPORT SYSTEM

The excavated trenches in saturated sandy soil had to be supported to secure pipe installation and speedup the construction process. However, due to the added expense of the proposed stabilizing techniques (i.e. dewatering and grouting) and the insufficient fortification in the contract documents, contractors were reluctant to use temporary shoring systems to support the excavation walls. In addition, safety of the workers inside the excavated trenches was not among the contractors' priorities knowing that workers are insured and all medical expenses would be covered by the insurance company. In fact, in some occasions unsupported excavation walls created an unbalanced pressure causing soil movement and resulting in surface subsidence at the street level and bulging of the vertical face of the trench. When left uncorrected, this condition further deteriorated causing face failure and entrapment of workers in the trench.

Use of trench boxes

For the loose sandy soil with zero cohesion existed at the site, the use of trench boxes within the excavated area deemed effective as they do not rely on the passive resistance of the soil. To comply with the minimum safety requirement and ease of working conditions, portable steel trench boxes were first used to support the sides of the excavated trenches. A series of trench boxes (also called a trench shield) were progressively placed in the excavated strips to prevent wall movement from disrupting the excavation process and injuring workers. As illustrated in Fig. 3a, the trench box consisted mainly of two large steel plates parallel to the walls of the trench, and horizontal cross-members holding the two plates apart. The lower edge of the box rests on the bottom of the trench, and the top edge extends above the ground surface. The workers stayed between the plates of the trench box, so that if the excavation wall collapses, the soil would be stopped by the presence of the box. As work progresses, the trench boxes are removed and prepared for reuse in the next sections.

Although the initial excavation size was slightly larger than the width of the box, as the excavation progresses, the loose sand that had no stand-up time moved rapidly filling up the gap behind the walls and the supported soil leading to the development of active and passive pressures. The active lateral pressure was assumed to develop on the sides of the box as the soil mass is allowed to stretch sufficiently to mobilize its shear strength. However, the initial trench boxes failed to cope with the soil movement causing damage to the boxes and interrupting the installation process as illustrated in Fig. 3b.

(*a*) Installation of the initial trench box (*b*) Installed trench boxes

Figure 3: Installation of trench boxes showing the in-trench dewatering

Designing stable trench boxes

The above conditions has lead to redesigning of the trench boxes to improve their performance, as illustrated in Fig. 4, by using additional hinges at the top and middle of the wall height.

Figure 4: Improved trench boxes

The new design considered the pressure acting on the wall to be at-rest. This condition assumes zero wall rotation which corresponds to the condition of no lateral strain. The at-rest condition was chosen for the new box design to increase the safety factor and ensure that the boxes withstand the ultimate lateral pressure imposed by the supported soil (Weber, 2010).

The earth pressure coefficient for the box design was calculated using the well-known Jaky formula for at-rest earth pressure coefficient,

$$K_o = 1- \sin \phi \tag{1}$$

where K_o = the earth pressure coefficient at rest and ϕ = the angle of internal friction. The at-rest earth pressure was then calculated by multiplying K_o by the effective overburden pressure.

Two-stage excavation procedure

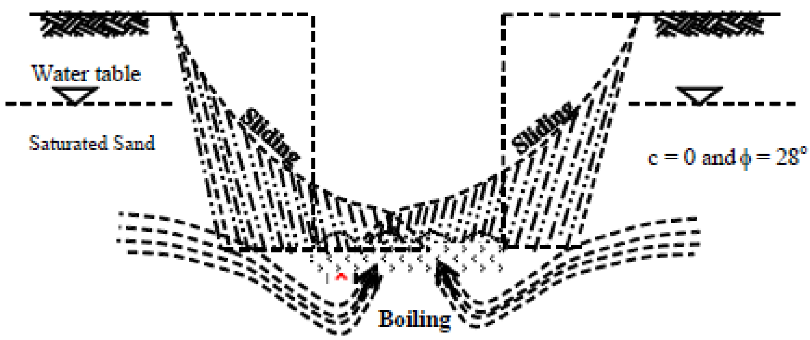
Another interesting approach that has been used in this project is the construction of two-stage deep cuts proved to be a very successful practice in places where contractors could afford to have a wider access to the exaction strips. In a first run a wider shallow trench would be excavated to be used as platform for the second box protected deep cut to the required grade as illustrated in Fig. 5. This simple approach was very welcomed by the contractor’s since no extra equipment was involved. However, this technique highly relies on the skills of the well experienced machine operator. In fact, a continuous in-trench dewatering along with two-stage deep cut proved as the most successful sewer installation approach in comparison to the other observed experiences in this sandy soil.

Figure 5: Two-stage excavation method

DEALING WITH UPLIFT PRESSURE

Upward water flow through the excavation base was observed at several locations along the pipe alignment during the excavation process. This was evidenced by the sand boiling at the excavation base as illustrated in Fig. 6a. Sand boiling was also associated with sliding of the excavation even when trench boxes were used (Fig. 6b).

(*a*) Unstable box wall



(*b*) Sand boiling at the base of excavation

Figure 6: Simultaneous sliding and boiling in the excavated trenches

Unstable condition caused by hydrostatic uplift developed during pipe installations since the unit weight of PE and GRP pipes could not balance off the uplift force. In order to avoid floating of pipes during installation, either water infiltrating into the excavation face must have be prevented by reducing the piezometric head (e.g., using prior dewatering techniques) or anchoring and supporting the pipe to the ground. In fact, both approaches were put in to practice at several locations. In this particular project, the intrench sump pump dewatering technique proved to be the applicable approach. However, in some occasions the high water head forced the contractor to adopt a concrete around the pipe. As Fig. 7 illustrates, concrete casings tend to surround the pipes during the installation preventing them from floating. Each concrete casing comes in two similar segments. The first half (the saddle) sits at the bottom of a gravel trench and the second half embraces the pipe. The spacing between concrete casing segments are two meters apart and the saddles are tailored to fit the diameter of the sewer pipes. Obviously, as the diameter of pipe grows so does the dimension of the casings as well as their weights which had a significant effect on uplift resistance (Choobbasti et al., 2009).

Figure 7: Concrete casings used to protect the pipes against uplift

MANHOLE EXCAVATION

Primary assessment of soil data (low numbers of SPT blows and zero cohesion) showed that due to zero standup time the cement grouting of sub-soil around the manhole and deep cuts may provide solid pockets and stabilize the surrounding soil during the course of excavation. This was intended to stabilize the soil particles against the dragging force motivated by the hydraulic gradient during the excavation. The stabilized soil would then tend to prevent boiling of particles into the trench yielding in failing excavated face. Any soil particle in saturated circumstance at rest is subject to three forces. The first one is gravitational force *W*, which acts downwards to the centre of particle (Fig. 8).

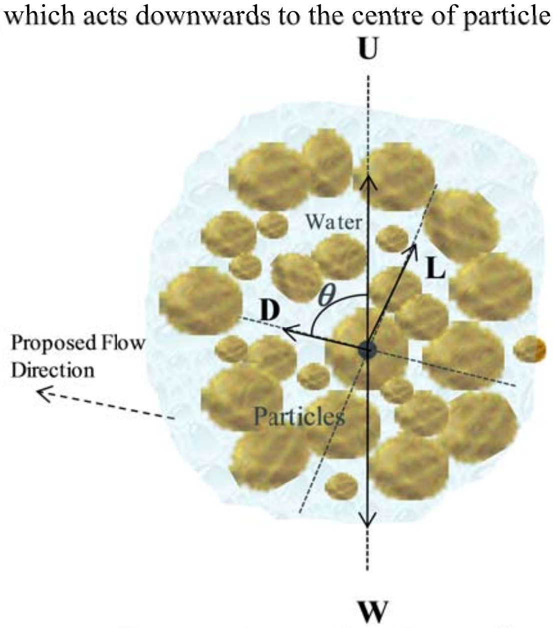


Figure 8: Forces acting on the submerged sand particles

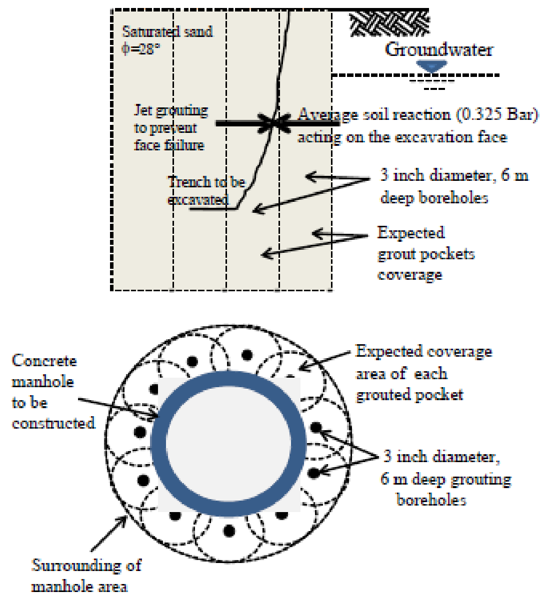


Figure 9: The expected diffusion pattern of cement grout

For spherical sand particle with uniform density the gravitational force has the value:

$$W = \pi r^3 g \rho_s \tag{2}$$

where r = Particle radius

g = Acceleration of gravity

ρ_s = Particle density

The second force is the buoyant force B, which acts vertically upwards at the centre of the particle and for a spherical sand particle, is equal to:

$$B = \pi r^3 g \rho_w \tag{3}$$

where ρ_w = water density.

The resultant of the gravitational force and the buoyant force balances the vertical upward hydrodynamic force, U (the third force, uplift) so that $U = W - B$, therefore,

$$U = \pi r^3 g (\rho_s - \rho_w) \tag{4}$$

The uplift force U in an undisturbed soil remains in balance with gravitational force unless the particle is subjected to motion. This motion could come as result of soil excavation and the consequent induced hydraulic gradient towards the trench. In this case, the uplift force may be decomposed into two different components in the direction of flow, as it is shown in Fig. 8. These two forces include the drag D, and lift L, parallel and perpendicular to the flow direction, respectively (Happel and Brennet, 1983). Assuming that the drag component, D, makes an angle θ with the uplift force, U, therefore

$$D = U \cos\theta \tag{5}$$

Hence: $D = \pi r^3 g(\rho_s - \rho_w) \cos\theta$ Also the lift component L is:

$$L = U \sin\theta \tag{6}$$

According to Eq. 13, D is function of θ. As this angle grows, so does the value of D.

Also:

as $\theta \rightarrow \pi/2$ then $D \rightarrow 0$

and as $\theta \rightarrow \pi$ then $D \rightarrow -U$ acting in the direction of acceleration of gravity To arrive to the quantity of total drag, D_t, Eq. 5 has to be integrated for all particles present in a unit volume of the soil (independent variable n):

$$D_t = \pi r^3 g(\rho_s - \rho_w)\cos\theta \tag{7}$$

$$\frac{\partial D_t}{dn} = \pi r^3 g(\rho_s - \rho_w)\cos\theta \tag{8}$$

To obtain specific solution for Eq. 8, both sides of equation may be integrated while boundary conditions for independent variable n are used:

$$D_t = \int_{n1}^{n2} [\pi r^3 g(\rho_s - \rho_w)\cos\theta] dn \quad \text{as} \quad n_1 = 1 \ \text{and} \ n_2 \rightarrow n \tag{9}$$

$$D_t = \left|^{n2} \{[\pi r^3 g(\rho_S - \rho_W)\cos\theta] * n_2 + c_1\} - \left|^{n1} \{[\pi r^3 g(\rho_S - \rho_W)\cos\theta] * n_1 + c_1\} \right.\right. \tag{10}$$

Using analytical boundary value approach for the above equation may be unfeasible, as the upper boundary of n is practically unknown. However, the soil reaction (pressure) at excavation face was measured using a calibrated soil pressure measuring device and the average value was found to be about 0.325 bar (equivalent to 4.7 psi). In the absence of inertia effect due to low Reynolds number, the measured pressure could roughly be considered equivalent to the total drag force, D_t.

As indicated in the previous section, boiling has developed as a result of unbalanced forces at the excavation base. It was speculated that grouting may tend to balance the forces if it has the strength to neutralize the dragging force D_t. Therefore, if the applied grouting pressure, G, exceeds 0.325 bar, the base stability may improve and excavation face may withstand the seepage forces. Hence, the applied jet grout pressure should be as follows:

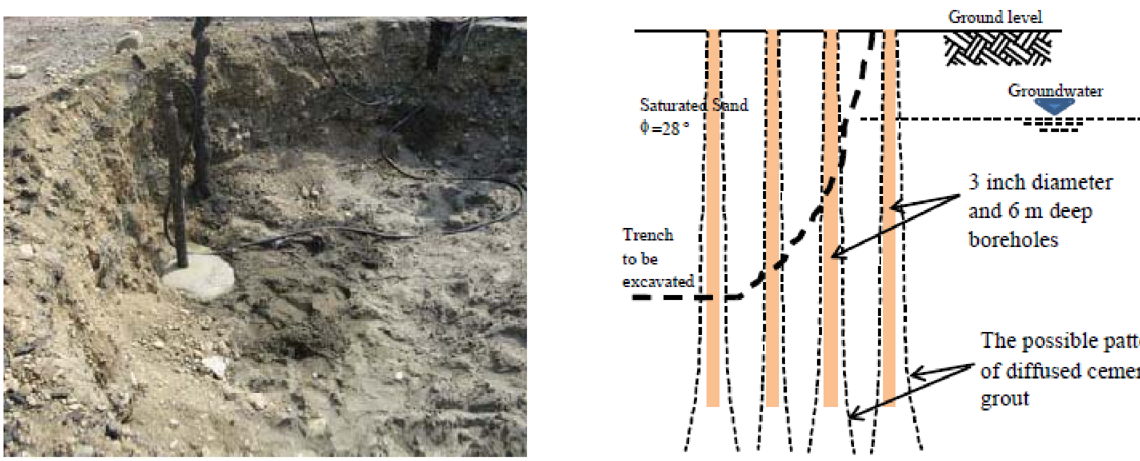
$G \geq D_t$

$G \geq$ 0.325 bar $\geq$ 4.7 psi

To investigate the efficiency of grouting, 50 psi cement grout was injected in a series of 3 inch diameter by 6 m deep boreholes around the selected manhole areas as shown in Figure 9. It was anticipated that, as it is illustrated in Fig 10a, each jet grouted borehole would create a pocket of solidified pillar overlapped with other adjacent pockets providing a stabilized ground for a secured excavation face.

However, later observations in the grouted soil segments did not show enough strength and yet the walls were unstable. This was explained by the possible vertical and horizontal diffusion of cement mortar into the adjacent soil as illustrated in Fig. 10b. It was concluded that due to high soil permeability and no rigid vertical and horizontal boundaries, the grout palm diffused and diluted into the sandy soil.

Since grouting could not be employed, it was decided to rely on the fortified trench boxes for the stability of the excavated face.



(*a*) Jet grouting boreholes (*b*) the pattern of diffused grout

Figure 10: The actual diffused grout around the manhole

HYDRAULIC TESTING OF THE COMPLETED PIPES

The difficulty of supplying and disposing of huge amount of water for hydraulic test of large diameter sewer pipes stimulated the idea of utilization of infiltration test method vs. the conventional ex-filtration approach (ASTM, 2009). The other fascinating advantage of infiltration was the enormous time saving since each run of infiltration test could cover several hundred meters of sewer lines vs. only 60 m length in conventional test. This approach was initially adapted for GRP sewer pipes where the groundwater level is predominantly above the sewer pipes. The adapted approach allowed replacing the conventional exfiltration method which tends to fill up the pipes with water and then to observe the loss of pressure in due time. The in-filtration approach requires at least 60 cm hydrostatic head above the top of sewer pipes. To ensure the presence of required hydrostatic head, a 2 inch diameter piezometer (perforated PE pipe) is installed one meter from the side of the sewer line 24 h before the test begins. As it is shown in Fig. 11, any possible infiltration of ground water into sewer pipes is measured with a free weir installed at the end each selected reach.

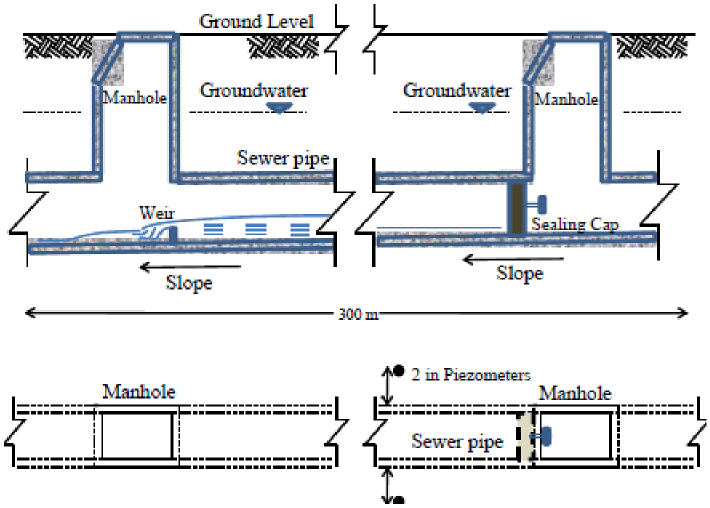


Figure 11: Infiltration test for the sewer pipes

Figure 12: Free weir flow used to measure water leakage

The infiltration method was successfully put into practice for 600 mm pipe every 300 m reach (Fig. 12). The leaked water was measured using free weir flow. The expected discharge was estimated using the following equation (Brater et al., 1996):

$$Q = (8/15)\ C_d\ (\sqrt{2g})\ (\tan\alpha)\,(H^{5/2}) \tag{11}$$

where: Q = Discharge (m^3/s)

C_d= Weir discharge coefficient

g = Acceleration of gravity (9.8 m/s^2)

α = Half of weir notch angle (degrees)

H = Depth of water on the weir (m)

From the literature for α = 45°, the weir discharge coefficient was defined as C_d = 0.58 (Subramanya, 2008). Therefore, Eq. 11 is simplified to:

$$Q = 1.37(H^{5/2}) \quad (12)$$

The measured leakage was compared to the standard expected leakage for GRP pipes using the following equation:

$$Q_a = (\sqrt{a} / \sqrt{1.8}) * (18.5) \quad (13)$$

where Q_a = The acceptable discharge through the sewer line (L/1 mm dia/1000 m/day

a = average ground level observed along the sewer line

Test results showing a flow rate more than Qa were then rejected.

CONCLUSIONS

A case study involving sewer pipe installation using low input techniques in saturated sandy soil is presented in this paper. The project involved the installation of about 77 km of PE and GRP sewer pipes with diameter ranging from 200 to 1,200 mm. Several problems have been encountered during the construction process including face instability, sand boiling, hydrostatic uplift pressure and massive rush of groundwater to the excavation trenches. Description of the construction techniques and the criteria used for the face stability of the excavation is summarized. In addition, some of the measures taken to control the above problems are also described. A great deal of effort has to be taken in similar sewer installation projects in developing countries to minimize the cost and the adverse effects of low stand-up time of the soil and groundwater pressure on the stability of the required deep excavations.

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